US Army Corps
of Engineers
Portland District

Technical Appendices
Appendix A Hydrology and Hydraulics
Appendix B Sedimentation
Appendix C Economic Analysis
Appendix D Cost Estimates
Volume II

Mount Pinatubo Recovery Action Plan Long Term Report

Eight River Basins
Republic of the Philippines

MT. PINATUBO
JUNE 1991 ERUPTION
PHILIPPINES

March 1994

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MOUNT PINATUBO
LONG TERM ACTION REPORT

EIGHT RIVER BASINS
REPUBLIC OF THE PHILIPPINES

VOLUME II - TECHNICAL APPENDICES

APPENDIX A -- HYDROLOGY AND HYDRAULICS
APPENDIX B -- SEDIMENTATION ANALYSIS
APPENDIX C -- ECONOMIC ANALYSIS
APPENDIX D -- COST ESTIMATES

Prepared by
Portland District
U.S. Army Corps of Engineers

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TECHNICAL APPENDIX

A

HYDROLOGY AND HYDRAULICS

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MOUNT PINATUBO
RECOVERY ACTION PLAN
LONG TERM REPORT

TECHNICAL APPENDIX A

HYDROLOGY AND HYDRAULICS

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MOUNT PINATUBO RECOVERY ACTION PLAN
LONG TERM REPORT

TECHNICAL APPENDIX A
HYDROLOGY AND HYDRAULICS

1. INTRODUCTION

1.1 Authorization

Under authority of the Economy Act (31 U.S.C. 1535) and Section 632 of the Foreign Assistance Act (22 U.S.C. 2357), the United States Agency for International Development (USAID) requested the Department of the Army, acting through the U.S. Army Corps of Engineers (USACE), to prepare a comprehensive Recovery Action Plan (RAP) for controlling sedimentation and flooding resulting from the June 1991 volcanic eruption of Mount Pinatubo, and subsequent hydrologic events. The RAP is being prepared in accordance with a Participating Agency Service Agreement (PASA) signed on June 18, 1992 between USAID/Philippines and the Department of the Army.

1.2 Purpose and Scope

This appendix to the Long Term Action Report is to present hydrology and meteorology pertinent to the design of measures to address long-term flooding and sediment control measures for all eight major river basins impacted by Mount Pinatubo.

1.3 System of Units

The hydrologic output from this study is reported in SI units. Volumes are most often reported in cubic decameters (dam^3). One decameter is equal to 10 meters; therefore, 1 dam^3 is equal to 1,000 m^3. Table 1.3.1 provides SI/English conversions for all SI units used in this appendix.

2. REGIONAL ANALYSES

2.1 Study Area

Mount Pinatubo is located approximately 100 km northwest of Manila in the Zambales Mountains on the west coast of Central Luzon. The eruption of Mount Pinatubo in June 1991 deposited enormous volumes of easily erodible fine-grained pyroclastic material on the flanks of the mountain. Debris flows and shallow flooding worsened by blockage of natural drainages have caused significant economic damage and loss of life.

Mount Pinatubo is drained by eight principal river systems. Clockwise from the north, they are:

O'Donnell-Bangat Rivers (tributary to the Tarlac River)
Sacobia-Bamban Rivers
Abacan River
Pasig-Potrero Rivers
Gumain-Porac Rivers
Santo Tomas River
Maloma River
Bucao River

Drainage basins for these river systems are shown on Plate 1.

The principal drainages can be conveniently split into two groups: west-side and east-side drainages. The west-side drainages (the Santo Tomas, Maloma, and Bucao) drain directly to the South China Sea. Of the east-side drainages, the O'Donnell-Bangat Rivers join the Bulsa River to form the Tarlac River, which flows north to the Agno River and thence to Lingayen Gulf. The remaining east-side drainages (Sacobia-Bamban, Abacan, Pasig-Potrero, and Gumain-Porac) are all tributary to the Pampanga River and the Pampanga Delta which flow south into Manila Bay.

Data used in the analyses were largely obtained from the eight principal drainages and from their immediate surrounding areas. The area considered in the regional analyses generally lies between 14°45' N to 15°45' N and 119°45' E to 121°00' E of each drainage. A small amount of additional data was obtained from more distant locations.

2.2 Physiography

2.2.1 Pre-eruption Conditions. Prior to the June 1991 eruption, the peak of Mount Pinatubo was at 1,745 meters elevation. The upper slopes of the mountain had a dense network of steep and deeply incised drainages. Above 1,000 meters elevation, slopes ranged from 20° to 65° (Pierson et al., op. cit.) with headwater channel gradients in excess of 400 meters per kilometer (m/km). Channel gradients on the lower part of

the mountain flatten out to about 10 to 20 m/km at elevations of 200 to 300 meters. Channel gradients in the area of the Pampanga River and the Pampanga Delta are extremely flat, dropping to as low as 0.1 m/km to 0.2 m/km.

The upper slopes of Mount Pinatubo were generally densely covered by shrubs and tall grass before the eruption. The flatter and lower areas on the mountain supported a variety of crops including sugar cane, cassava, and maize.

Much of the low-lying region surrounding the Pampanga River east of Mount Pinatubo is intensively cultivated. This is one of the Philippines' principal rice-growing areas. The more deeply flooded parts of the delta are used for fish-farming and other types of aquaculture.

Little detailed information is available on soils and surficial geology of Mount Pinatubo. However, there is widespread evidence of past eruptions, including large expanses of old pyroclastic deposits in the Sacobia, Abacan, and Pasig-Potrero Rivers on the east side and in the Marella River valley on the southwest side of the mountain (Pierson, et al., op. cit.; JICA, 1978[1]). Analyses of available hydrologic data indicate that soils on Mount Pinatubo are generally highly permeable, though exposures of hard rock have been reported since the eruption in the Gumain and Bangat basins.

2.2.2 Impacts of the June 1991 Eruption. The post-eruption peak of Mount Pinatubo is approximately 1,600 meters, a reduction of more than 130 meters in peak elevation. By necessity the hydrologic analysis is based almost entirely on pre-eruption hydrometeorologic data. The eruption of Mount Pinatubo caused substantial changes in the hydrologic regime of the principal drainages. Some changes (such as reduction in infiltration due to ashfall deposits) are believed to be relatively short-lived and are not reflected in the hydrologic modeling. Other changes, such as in the gross configuration of the drainage systems, are considered to be permanent relative to the life span of possible engineering measures, and are reflected in the modeling effort. The principal hydrologic impacts of the eruption are discussed briefly below.

Changes in Headwater Tributary Areas. The June 1991 eruption filled much of the upper portion of five basins (O'Donnell, Sacobia, Pasig-Potrero, Santo Tomas, and Bucao) with pyroclastic deposits. The drainage patterns that developed in these deposits resulted in numerous changes in sub-basin boundaries within the Bucao Basin and resulted in much of the headwaters of the Sacobia being captured by the Abacan River. However, the Sacobia headcut upstream and recaptured its pre-eruption headwaters plus approximately 4-1/2 km of channel length at the upstream end of the

[1] Japan International Cooperation Agency, 1978, *Planning Report on the Pasig-Potrero River Flood Control and Sabo Project*. Main Report 78-38-1/6.

Abacan headwaters (i.e., Abacan above the Gates of Abacan). It appears that in October 1993 the Pasig-Potrero captured about 21 km^2 of the Sacobia River headwaters.

In addition to the changes in basin and sub-basin boundaries resulting from new drainage patterns through the pyroclastic deposits, the eruption itself left a 5.9 km^2 caldera that captured portions of the headwaters of the O'Donnell, Sacobia-Bamban, Gumain-Porac, Santo Tomas, and Bucao Rivers.

With the exception of the headwater drainage areas of the Pasig-Potrero and Sacobia Rivers, hydrologic modeling for post-eruption conditions is based on drainage patterns and drainage areas inferred from aerial photography dated November 1991 and October 1992 and assumes the October 1992 drainage patterns are relatively stable. The portion of the Sacobia River headwaters that was captured by the Pasig-Potrero in October 1993 was determined by District personnel overflights.

Blockages and Lake Breakouts. In the period immediately after the eruption, pyroclastic deposits caused numerous blockages of the drainage system on the upper slopes of Mount Pinatubo. The formation of lakes behind these blockages and their subsequent failure contributed to debris flows along the principal drainages. Given the large amounts of unstable material remaining in the headwater drainages, temporary blockages and sudden breakouts of debris-dammed lakes may be a continuing hazard in some basins. No attempt was made to account for lake breakouts in the hydrologic modeling.

Following the June 1991 eruption, Lake Mapanuepe formed in the Santo Tomas River basin near the mouth of the Mapanuepe River. At the invert elevation of its outlet, Mapanuepe Lake has a surface area of approximately 8 km^2. The formation of Lake Mapanuepe resulted from a blockage of the Mapanuepe River outlet caused by recurrent lahars and severe aggradation on the Marella River. This lake was the source of repeated lake breakouts in the months immediately following the eruption, but a man-made outlet channel from the lake has now stabilized the situation. Attenuation of the flood wave moving through Mapanuepe Lake due to temporary storage of water in the lake was accounted for in the Santo Tomas hydrologic model.

Channel Degradation and Aggradation. All the principal rivers draining Mount Pinatubo have been affected by extreme channel degradation or aggradation at some point along their course. Channel degradation and aggradation have significantly changed the physical configuration of some of these rivers' drainage systems.

The lower reaches of the Pasig-Potrero, Bamban, and Gumain Rivers have aggraded to the extent that they are now perched between levees approximately three to five meters above the surrounding terrain, and lateral drainage is unable to enter the main channel. These features are reflected in hydrologic modeling.

While beyond the scope of this study, siltation of the Guagua River and other very low-gradient channels leading to the Pampanga Delta has resulted in severe flooding of low-lying land around Bacolor and San Fernando, and is reported to have caused a general increase in the depth and duration of flooding throughout the lower reaches of the Pampanga River and its delta.

Reduction in Infiltration. The June 1991 eruption covered an area of approximately 4,500 km^2 with airfall deposits of fine ash greater than 5 cm in depth, with ash depths of between about 5 and 50 cm on the principal drainages basins covered by this study (Pierson et al., Fig. 2, op. cit.). It is generally believed that deposits of fine ash will reduce pre-eruption infiltration rates, hence causing an increase in the volumes and rates of post-eruption runoff. In the two years since the eruption, much of this widespread deposit of ash has washed off during the monsoon rains, or the low-infiltration crusted surface that forms on these fine grained deposits has been broken up by new plant growth. The reduction in pre-eruption infiltration rates is believed to be a relatively short-lived phenomenon and was not considered in the hydrologic modeling presented in this report.

Loss of Vegetation. The eruption of Mount Pinatubo buried or otherwise destroyed all vegetation over an area of approximately 300 km^2. Loss of vegetation greatly reduces evapotranspiration losses and eliminates the potential for both interception storage and storage in leaf litter and surface soils high in organic material. Loss of vegetation will cause an increase in the volumes and rates of post-eruption runoff. However, the climate of the area around Pinatubo is conducive to rapid plant growth, and it is generally believed that re-vegetation of all but the most unstable parts of the mountain will be relatively rapid. No attempt was made to account for loss of vegetal cover in the hydrologic analyses.

2.3 Climatology

2.3.1 General. The Mount Pinatubo area, on the west coast of Central Luzon, at 15° N latitude, has a tropical climate dominated by the Northeast Monsoon during the winter months (November through May) and by the Southwest Monsoon during the summer months (June through October) which are the rainy-season flood-producing months. The seasonal reversal of airflow results in a pronounced seasonality in prevailing winds and rainfall. Severe weather conditions (high winds and heavy rain) are associated with typhoons, which most commonly occur during the Southwest Monsoon season (June through October).

2.3.2 Climatic Records. Daily rainfall data were available from 15 stations in the vicinity of Mount Pinatubo. Of these, data for 13 stations were obtained from the Philippine Atmospheric, Geophysical, and Astronomical Services Administration (PAGASA). Data for two other stations were obtained from the U.S. Navy (Cubi Point Naval Air Station) and the U.S. Air Force (Clark Air Force Base). The stations, periods

of record obtained, and other relevant information are listed in Table 2.3.1. Data availability is summarized in the time-line on Figure 2.3.1. The stations are located on Plate 1. A total of about 330 station years of daily rainfall data was obtained. The majority of the PAGASA stations have a record length of from 15 to 20 years. Longer term records (30 to 40 years of data) are available from Iba, Zambales; Cubi Point NAS; and Clark AFB. Data on station elevations were not available. However, reference to 1:50,000 scale topographic maps published by the National Mapping and Resource Information Authority (NAMRIA) indicate that all daily rainfall stations have elevations in the range of 0 meters to 150 meters (Clark AFB). No daily data were available from stations at higher elevations. The elevation of Mount Pinatubo prior to the eruption was 1,745 meters.

A network of six automatic tipping bucket rain gages was installed at relatively high elevations on Mount Pinatubo by the Philippine Institute of Volcanology and Seismology (PHIVOLCS) after the June 1991 eruption. PHIVOLCS high altitude rainfall data were obtained for the period August 1, 1991 to January 10, 1993. Data from these gages were transmitted by radio telemetry either to the Pinatubo Volcano Observatory at the former Clark AFB or to PHIVOLCS main facility in Manila in the format of cumulative bucket tips recorded at pre-selected time intervals. One bucket tip corresponds to 0.635 mm (0.025 inches) of rainfall. The station names, locations, elevations, and other relevant information are listed in Table 2.3.2. The stations are located on Plate 1. There have been considerable difficulties in operating the PHIVOLCS gage network and significant periods of data are missing due to, for example, transmission problems and accumulations of ash in the gages. When these gages are operational, data are reported at least once an hour, and more generally at 20- or 30-minute intervals.

Hourly data were sparse. Gages at Iba, Hacienda Luisita, and Porac provided hourly data, but the record at these stations is described as "fragmentary." Hourly rainfall data were obtained for three storm events at Iba, three events at Hacienda Luisita, and five events at Porac. Information on the data obtained is provided in Table 2.3.3. The stations are located on Plate 1. Hourly rainfall data outside the immediate study area were available for one extreme 24-hour period from *Weather Bureau Technical Paper No. 42*[2], which provided hourly data from Baguio City, located approximately 140 km north-northeast of Mount Pinatubo at 1,370 meters elevation (4,500 feet), for a typhoon-related event in September 1911 which generated the then world's record 24-hour rainfall of 1,168 mm (45.99 inches). The general lack of good hourly rainfall data placed severe limitations on hydrologic modeling for the present study.

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[2] Weather Bureau, U.S. Dept. of Commerce, 1961, *Generalized Estimates of Probable Maximum Precipitation and Rainfall-Frequency Data for Puerto Rico and Virgin Islands for Areas to 400 Square Miles, Durations to 24 Hours, and Return Periods from 1 to 100 Years.* Technical Paper No. 42.

Daily pan evaporation data were obtained from three stations in the vicinity of Mount Pinatubo, all operated by PAGASA. The stations, the periods of record obtained, and other relevant information are listed in Table 2.3.1. The stations are located on Plate 1.

Screening of Daily Data. Daily rainfall data were screened to identify clearly anomalous (high or low) values and to summarize periods of missing data. The completeness of rainfall records is highly variable, with no missing data reported at Clark AFB (station USA02) and over 30 percent of the record missing at Camiling (Station R0315). The records from Cubi Point NAS (USA01), Clark AFB (USA02), and Iba, Zambales (D324), the three stations with the longest records, are all relatively complete.

The following points were noted during screening:

- The maximum reported daily rainfall of 748.5 mm for the period of record at Iba (D324) on July 10, 1980 is suspect. No other neighboring stations reported daily rainfall greater than 150 mm on that date. Examination of hourly data for this event showed that the daily amount for this day was in fact 78.5 mm, and the record was corrected accordingly.

- The rainfall record for Magalang, Pampanga (A020) is too short to be of value. A longer record is available from Bai Magalang (R0312), approximately 5 km east of the Magalang gage.

- The record for Bai Magalang (R0312) is essentially complete between February 1977 and January 1992. However, the maximum recorded daily rainfall of 103.1 mm on November 14, 1977 is significantly lower than that at other neighboring stations.

- The record from Camiling (R0315) is too short and too fragmented to be of value.

- The record for Mayantoc, Tarlac (R0316) is quite fragmented with approximately 20 percent of the daily records reported missing. The record is particularly poor for the period 1978 through 1984, when 27 months are missing, including most records for July, August, and September.

- The record for Palawig, Zambales (R0318) is highly suspect, particularly for 1982 when no rainfall was recorded even though the station was reported to be in operation. Also, the Palawig station does not report nearly as many rainfall events during the relatively dry months of January through April as do other coastal stations. Over the 15 years of record, only nine measurements, including trace rainfalls, were reported in these four months. It is possible the station is only operated on a seasonal basis.

- The record for San Felipe, Zambales (R0319) is suspect. The maximum reported daily rainfall of 162 mm for the period of record is far below that for other coastal stations.

As a result of this basic screening, data from Camiling, Palawig, and Magalang were dropped from further consideration.

Double Mass Analysis. Double mass analyses were conducted on daily rainfall data using the following groups of stations:

Long-term stations

Group 1: Clark AFB (USA02); Cubi Point NAS (USA01); Iba (D324).

West coast stations

Group 2: Cubi Point NAS (USA01); Z-NAS, San Marcelino (R0322); San Felipe (R0319); Santa Rita Elementary School (R0320); Iba (D324).

Northeast stations

Group 3: Mayantoc (R0316); CLSU, Munoz (A017); Hacienda Luisita (A016).

Southeast stations

Group 4: Clark AFB (USA02); Bai Magalang (R0312); Julian Subdivision (R0313); Masantol (R0314).

The entire concurrent rainfall record for each group of stations was used for the double mass analysis. In computing cumulative rainfall amounts, days were ignored for which data were missing for any station in the group. The double mass analyses of the long-term stations indicated that, with the exception of an unusually high rainfall amount recorded at Iba, Zambales (D324) in 1964, the records for these stations are fairly consistent for the period 1961-1990.

The double mass analyses of the west coast stations indicated that, with the exception of San Felipe, the records are relatively consistent over the period 1975–1990. Removing San Felipe from consideration should improve the results for other stations.

The double mass analyses of the northeast stations indicated no obvious long-term changes in slope, although the plot for Mayantoc showed an unusual shape with a number of small slope changes and an overall tendency towards diminished rainfall in later years. Considering these results and the preliminary screening noted earlier, the data from Mayantoc (R0316) are highly suspect.

The double mass analyses of the southeast stations indicated that the data for Bai Magalang underwent several major slope changes between 1977 and 1991. Again, considering the results of the screening and double mass analysis, the data for Bai Magalang are suspect. For Masantol and Clark AFB the double mass analyses do not indicate any major shifts in recorded rainfall. At Julian Subdivision there is an apparent long-term shift at about 1987. Comparatively less rainfall is recorded at Julian after this date. It is not known if station relocation or other changes took place which might explain this shift.

As a result of the double mass analysis, data from San Felipe, Mayantoc, and Bai Magalang were dropped from further consideration.

Spatial Variation of Daily Data. Spatial variations in rainfall data were examined by computing cross correlations between daily data at all stations and by examining rainfall amounts in individual major storm events. A matrix of cross-correlation coefficients for all stations is shown in Table 2.3.4. Cross-correlations were computed for individual pairs of stations using all available data for which daily rainfall exceeded 50 mm at either or both stations. The maximum cross-correlation is 0.53 between Iba (Station D324) and Santa Rita Elementary School (Station R320), which are approximately 20 km apart. If stations with suspect records are eliminated from the matrix (e.g. Palawig, San Felipe, Bai Magalang), the cross-correlations generally decrease with distance between stations, as might be expected. The low cross-correlations indicate substantial spatial variations in daily rainfall amounts over relatively short distances.

Relationships between daily rainfall amounts at various stations during major storm events were examined qualitatively. Major storms in western Luzon are associated with large weather systems (the Southwest Monsoon or typhoons) which affect the entire area around Mount Pinatubo. However, daily rainfall amounts within individual events show great spatial variability, especially when rainfall depths from stations on the east and west sides of Mount Pinatubo are compared. Spatial variations of rainfall during Typhoon Didang in May 1976 appear to be typical. This event produced the maximum two-day rainfall depth of 988 mm for the 40-year period of record at Iba. Significant rainfall depths were recorded at all other west coast stations, although storm depths were relatively less severe. The maximum two-day rainfall depth at Cubi Point during this event, for example, was 485 mm, a depth exceeded six times in Cubi Point's 33-year record. Comparison of rainfall depths at Iba for this event with rainfall depths from the east side of Pinatubo (for example, at Clark AFB) show even greater variation.

Analysis of PHIVOLCS Data. Review of the daily rainfall records and monthly totals recorded at PHIVOLCS stations indicates significant problems with many of the stations. PHIVOLCS data do not include a flag to indicate missing data or equipment malfunction. Therefore any gaps in the record were filled with a value of 0.0. This may lead to inconsistencies when comparing PHIVOLCS data to PAGASA or other neighboring daily rainfall gages. Following is a review of the available PHIVOLCS data.

Station RG-1 — Mt. Caudrado: Data for August and September 1991 appear to be fairly complete. Monthly totals are not significantly greater than those recorded at lower elevations but individual storm events show increased daily precipitation. Data beyond October 1991 are fragmented and are not useful in the hydrologic analyses.

Station RG-2 — BUGZ: Data for August and September 1991 are relatively complete. Similar to station RG-1, the monthly totals are not substantially larger than the lower elevation PAGASA stations. Data from mid-August 1992 to mid- September 1992 may also be of some value. As was the case for RG-1, the data for much of 1992 were recorded as no rainfall. As noted earlier, there is no way of distinguishing between missing data due to station malfunction and data that actually register 0.0.

Station RG-3 — PI2: Data for all months are unrealistically low. The maximum monthly precipitation recorded was 535 mm in August 1992. This is significantly lower than the 804 mm recorded at Bai Magalang, for instance.

Station RG-4 — Mt. Culianan: Data for all months are unrealistically low. The maximum single day rainfall in 1991 was 37.4 mm. In comparison, the PAGASA station at Iba, Zambales reported six events greater than 100 mm during August and September 1991.

Station RG-5 — Gumain: Rainfall records for the August 20, 1991 event appear to be complete through midday on August 21, 1991. With the exception of this event, the Gumain data are fragmented, unrealistically low, and probably not of value.

Station RG-6 — Sacobia: Data collection did not begin until mid-September 1991. Rainfall totals for July and August 1992 are substantially less than those reported at low elevation stations. The record is fragmented and is not useful in the hydrologic analyses.

The PHIVOLCS records are too short and fragmented to be used for determining variations of rainfall with elevation on Mount Pinatubo.

Evaporation Data. Daily pan evaporation data were obtained from three stations in the vicinity of Mount Pinatubo, all operated by PAGASA. The stations, the periods of record obtained, and other relevant information are listed in Table 2.3.1. The stations are located on Plate 1.

Most of the missing data in the evaporation records can be attributed to pan overflows during wet weather. Examination of the available data showed the record at

Hacienda Luisita to be fragmented and occasionally reporting unusually large (in excess of 30 mm/day) daily evaporation depths. Data from this station were consequently dropped from further consideration. The data from Magalang were also dropped because the record was too fragmented and too short to be of value.

The data from CLSU, Munoz are believed to be the most reliable pan evaporation data available. Missing periods in this record were filled with mean daily values for each month to obtain estimates of annual pan evaporation. The estimated annual pan evaporation is given in Table 2.3.5.

2.3.3 Precipitation. The rainfall regime of the Mount Pinatubo area is highly seasonal, with a pronounced wet season from approximately June through October, coincident with the Southwest Monsoon, and a dry season from approximately November through May. Rainfall amounts along the west coast of Central Luzon are enhanced by orographic uplift during the Southwest Monsoon. Mountains to the west and southwest of Mount Pinatubo, with peaks at about 1,000 meters, act as partial orographic barriers during the Southwest Monsoon, with the result that the west and southwest flanks of Mount Pinatubo probably receive somewhat less rainfall than would otherwise be expected. Mount Pinatubo itself, and the Cabusilan Mountains to the south, present a more significant orographic barrier during the Southwest Monsoon, such that the east and northeast sides of Mount Pinatubo lie in a rain shadow, with low-lying interior areas east of Mount Pinatubo receiving only about half of the total annual rainfall experienced on the coast. Plots of mean monthly rainfall at Cubi Point NAS and Clark AFB (Figures 2.3.2 and 2.3.3) illustrate the seasonal distribution of rainfall totals on the west side and east side of Mount Pinatubo, respectively. The mean annual rainfalls at Cubi Point NAS and Clark AFB are about 3,600 mm and 2,000 mm, respectively.

Maximum daily rainfall amounts at stations in the Mount Pinatubo area are generally caused directly or indirectly by tropical cyclones, which are most prevalent between May and November. Data available from PAGASA indicate that between 1948 and 1991, an average of 16 tropical cyclones per year (tropical depressions, tropical storms, or typhoons) affected weather conditions at various regions in the Philippines. Of these, typically three or four per year affected weather conditions around Mount Pinatubo.

While large one-day rainfall amounts can be caused by the direct passage of a typhoon over the area (the maximum recorded one-day rainfall at Cubi Point NAS of 442 mm in May 1966 is believed to be one such example), rainfall events of longer duration with greater total event volumes and comparable one-day depths may result from intensification (or surges) in the Southwest Monsoon flow during passage of typhoons to the northeast of Luzon.

The Southwest Monsoon flow can bring long periods of near-continuous heavy rain. Intense localized rainfall is associated with intense convective activity in groups or pockets of storm cells embedded in the general southwesterly flow. Surges in the

southwesterly flow due to the influence of typhoons may increase wind speeds by 10 to 20 knots with a concomitant increase in rainfall.

Normal Annual Precipitation (NAP) Map. Figure 2.3.4 shows isohyets of mean annual rainfall as determined from assessments of available long-term rainfall and streamflow data. Long-term rainfall records are available only for elevations generally less than 100 meters near the coast and in the interior lowland regions. However, the study area includes much higher elevations: the pre-eruption summit elevation of Mount Pinatubo, for example, was 1,745 meters. Streamflow and evaporation records were used to estimate mean annual rainfall at the higher elevations which correspond to the watershed areas of interest, as described in the following paragraphs:

1) The annual rainfall data, summarized by Table 2.3.6, were sufficient to determine an isohyetal line corresponding to 1,900 mm/yr of rainfall in the lowland areas east of the Zambales and Cabusilan mountain areas, and to show that rainfall decreases with eastward distance from the mountains. Coastal rainfall gages to the west of the mountains indicate rainfall of around 3,600 mm/yr to 4,200 mm/yr.

2) Annual yields from watersheds with streamflow gages were determined in terms of annual depth of runoff over the basin area as summarized by Table 2.3.7. Data for the Bagsit and (upper) Camiling stations, key stations for rainfall assessment because they have relatively high-elevation basins, show average annual runoff amounts of approximately 3,400 mm and 3,700 mm, respectively.

3) Average basin rainfalls were estimated by adjusting the runoff data by 1,550 mm to account for the estimated average annual evapotranspiration in the basins. This evapotranspiration amount was computed as being equal to about 80 percent of pan evaporation reported for lower elevations, assuming there would be no significant moisture deficit in the mountains. In transposing the estimated basin rainfall amounts to the maps, it was assumed that the average basin rainfall would occur at the centroid of the basin, and that rainfall would increase with elevation.

4) Knowing the rainfall at the lower basin elevations from the rain gages, and having assumed the rainfall for the centroid of the basin, rainfall at the upper basin was determined as the amount necessary to generate the observed runoff.

The analysis resulted in findings that, for mountains next to the coast, without any intervening topographic barriers, rainfall isohyetal lines of 5,000 mm/yr and 6,000 mm/yr correspond approximately to elevations of 750 meters and 1,200 meters, respectively. At Mount Pinatubo, which is separated from the coast by other mountains and may

experience a rain shadow effect, isohyetal lines of 4,000 mm/yr and 5,000 mm/yr correspond approximately to elevations of 500 meters and 1,000 meters, respectively.

The NAP map in Figure 2.3.4 differs significantly from those published with other studies in the area. Other available NAP maps show no significant increase of rainfall with elevation over Mount Pinatubo. Most of the annual rainfall in this region is associated with the Southwest Monsoon, which brings moist air from the southwest across the South China Sea during the months of June through October, striking the west coast of Luzon. An intensification of monsoon rain on the west side of the Zambales and Cabusilan Mountains due to orographic uplift is expected, and hence an increase of rainfall with elevation. The role of orography in increasing monsoon rainfall has been confirmed by meteorologists formerly stationed at Cubi Point NAS.

Additional support for the role of orography causing increasing rain with elevation is found in *Weather Bureau Technical Paper No. 42*[3]. This paper verified the orographic component of a hurricane model using wind and rain data recorded at Baguio City, Philippines in September 1911. Baguio City is located approximately 140 km NNE of Mount Pinatubo, at elevation 1,370 meters (4,500 feet). The rainfall event was associated with a typhoon and prevailing southwest winds.

2.3.4 Air Temperature. The mean annual air temperature at Cubi Point NAS (at sea level on the southwest of Mount Pinatubo) is approximately 26-28° C[4]. The annual average maximum temperature at Cubi Point NAS is 31.4° C and the annual average minimum is 23.7° C. A plot of mean monthly temperatures at Cubi Point NAS is given in Figure 2.3.5. Temperatures near the summit of Mount Pinatubo (pre-eruption elevation of 1,745 meters) are expected to be about 5 to 10° C cooler than those at Cubi Point and Munoz, by consideration of adiabatic lapse rates.

2.3.5 Winds. Wind speed data are available from Cubi Point NAS and annual monthly wind roses have been published by the Naval Weather Service Environmental Detachment. The annual wind rose is shown in Figure 2.3.6. The numbers shown on the wind rose (e.g. 19.6, 9.9, .9, etc.) represent the percent of time that wind comes from the indicated directions. The various ranges of wind speed and the symbols that represent these ranges are shown in the lower right hand corner of Figure 2.3.6. The percent of time that wind in a given speed range comes from a given direction can be scaled from the appropriate symbol on the wind rose (e.g. 19.6 percent of the time wind comes from the northeast and approximately 4 percent of the time wind comes from the northeast at a speed between 11 and 16 KTS). As indicated in the center of the wind

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[3] See Footnote 2.

[4] Wernstedt, F.L, 1972, *World Climatic Data*, Climatic Data Press.

rose, 25.9 percent of the time wind speed is less than 4 knots (KTS). The direction of winds less than 4 KTS is not indicated by the wind rose. As previously indicated, the winds come from two predominant directions: from the southwest during June through October and from the northeast during November through May.

2.4 Hydrology

2.4.1 Discharge Records. Daily streamflow data were available from 18 stations in the vicinity of Mount Pinatubo. Data were obtained from the Department of Public Works and Highways (DPWH) and the Bureau of Research and Standards (BRS), which currently have responsibility for collection and dissemination of streamflow data. Responsibility for streamflow data collection has changed hands several times in past years and most of the data obtained were originally collected by either the Bureau of Public Works or the National Water Resources Council (NWRC). The stations, drainage areas, periods of record obtained, and other relevant information are listed in Table 2.4.1. No daily streamflow data were obtained prior to 1957, although published records indicate that earlier data are available for some stations, as indicated in Table 2.4.1. The last year for which data were officially published was 1972. A small amount of post-1972 data were obtained. Streamflow stations are located on Plate 1.

Peak annual flow data were obtained at the 18 stations providing daily streamflow data. Station names, periods of record, and other relevant information are given in Table 2.4.1. For the most part, peak annual flows were extracted from two publications:

- *Philippine Water Resources Summary Data, Volume 1 — Streamflow and Lake or River Stage Ending December 31, 1970.* Partial copy. Exact title, publisher, and date of publication unknown.

- *Philippine Water Resources Summary Data, Volume 2 — Streamflow and Lake or River Stage Ending December 31, 1980.* Republic of the Philippines, Department of Public Works and Highways, Bureau of Research and Standards, Quezon City, June 1991.

The first of these publications contains peak annual flow data for the period of record through December 1970. The second publication covers the period 1971 through 1980. Some additional peak annual flow data subsequent to 1980 were obtained from unpublished sources.

Most of the streamflow data in the study area were obtained from staff gage readings that were reportedly read two or three times per day. Data at some stations were collected using an automatic water level recorder. Attempts were made to obtain these data for selected flood events. However, no data could be obtained other than mean daily flows and published annual peak flows.

Review of Rating Curves. Discharge measurement data and/or rating curves were available for review for about one half of the stations considered in the analysis. The available information yielded the following general observations which should apply for all stations.

- The rating curves tend to be unstable, and most stations required a number of different rating curves over time as channel conditions changed. Substantial shifts in rating curves were most apparent in rivers subject to aggradation, where vertical shifts of 1 meter or more were observed over the period of record.

- Measured (gaged) discharge points are available only at flows significantly less than peak annual discharges. Measured data were apparently used as the basis for shifting previously defined rating curves to match the measured points as well as to define new rating curves when the extent of shifting exceeded certain (unspecified) limits. In general, the low-flow records are considered to be quite good for those periods when discharge measurements were made on a regular basis.

- Prior to 1970, discharge measurements were made on a regular basis at most stations, typically at least three measurements per year. After 1970, the frequency of discharge measurements decreased significantly, with most stations showing one or more years with no measurements at all.

- Plotted stage-discharge curves appear to be biased towards matching the lower limit of measured discharges for a given stage. If this observation is accurate, the implication is that available water supplies are conservatively reported but that peak discharges tend to be underestimated.

- The basis for extrapolating the rating curves beyond the measured discharges is generally not known. The following three methodologies were indicated in the literature for some of the stations:

1) "The upper portion of the curve was extended by area-velocity method."

2) "Due to the unavailability of the cross-section of the river, the upper portion of the curve was extended by logarithmic plotting."

3) There are infrequent references to some peak discharges as having been determined by "slope-area" method.

In summary, pre-1970 reported low discharges are considered to be quite accurate but the discharges may not reflect natural hydrologic conditions due to irrigation withdrawals

on many of the rivers. Peak flows are subject to considerable error due to a lack of data for estimation plus shifting of the stage-discharge relationships during flood events.

Screening of Daily Data. Daily streamflow data were screened to identify clearly anomalous (high and low) values and to summarize periods of missing data. Data availability is summarized in the time-line in Figure 2.4.1.

The principal observations from screening the data are as follows:

- Useful streamflow data are sparse after 1972. Data available after 1972 suggest staff gages were read not on a regular daily basis, but at irregular intervals several times a month. For example, the record for Maloma for 1987 contains a period of 21 days in the wet season with a constant flow of 7.2 m^3/s with an abrupt change on August 17 from 7.2 to 372 m^3/s. The records for 1987 and other years contain similar periods of "constant" flow.

- As can be seen from Figures 2.3.1 and 2.4.1, there is very limited overlap of daily rainfall and daily streamflow data. This makes it impossible to reconstruct storm isohyetal maps for which reliable concurrent streamflow data are available.

- Data from the two stations on the Porac River near Del Carmen and Valdez show inconsistencies. It is possible that the data near Valdez reflect irrigation diversions or diversion of flows into the Gumain Floodway. However, no information is available on the nature of diversions or the physical configuration of the floodway. As a result, data from the Porac River near Valdez and the Gumain Floodway were dropped from further consideration.

- Records from many stations show unreasonable abrupt changes in flow rate. These occur at many places throughout the record and are too numerous to document individually.

Double Mass Analysis. Double mass analysis was conducted on daily streamflow data using, in the first instance, the following group of four stations:

Porac River near Del Carmen (084A)
Gumain River (086A)
Bucao River (093A)
Santo Tomas River (094A)

These stations were selected because they are of direct interest to the hydrologic analyses (all are affected by the eruption of Mount Pinatubo) and have a relatively long common record. The entire concurrent record for the group of stations was used for

double mass analysis. The analysis was done in terms of mean monthly cumulative runoff in millimeters using published drainage areas to convert from discharge rate in m^3/s to runoff depth in millimeters. The plot for Porac River near Del Carmen indicates a progressive increase in flows relative to other stations throughout its period of record. The plot for the Santo Tomas indicates a dramatic reduction in flows starting in 1964. The reason for the reduction in flows is not known but could result from gage errors or irrigation diversions. However, no information was available as to whether such diversions occurred. Plots for the Gumain and Bucao Rivers showed a reasonably consistent record.

For completeness, double mass analyses were done for all other available records by plotting cumulative monthly runoff at the station of interest against the cumulative mean monthly runoff from a control group made up of stations on the Porac, Gumain, Bucao, and Santo Tomas Rivers. All plots exhibited inconsistencies in the records. Records for the two gages on the Camiling River appeared to be the most reliable outside the control group.

Review of Peak Annual Flow Data. Table 2.4.2 presents a summary of all available peak flow data expressed in m^3/s. Table 2.4.3 presents the same data on a normalized yield basis computed by dividing each peak flow (m^3/s) by the published basin drainage area (km^2) at the gage. These tables include data found to be unrepresentative of actual annual peak flow amounts.

Some of the published peak annual flow data were determined to be unrepresentative of actual annual peak flow amounts when the peak was published for:

- a year in which there was only a partial record of flow, and for which there were no discharge records for significant portions of the high-flow months of July through December;

- a year or longer period for which there were serious questions as to the adequacy of the rating curve used at the station.

Tables 2.4.4 and 2.4.5 summarize peak flow and normalized peak flow data which have been screened to exclude doubtful records.

Examination of data in Tables 2.4.2 through 2.4.5 show very low peak flow yields for the Pasig-Potrero and O'Donnell Rivers and very high yields for the Caulaman River. The low yields on the Pasig-Potrero and O'Donnell could result from local geologic conditions although no information is available to confirm this. The reason for the unusually high yield on the Caulaman is unknown.

Most streamflow gaging stations used in the hydrologic analyses rely on a staff gage (which is read between once and three times a day, depending on the gage site) and its

associated rating curve. The reported "peak annual flow" at these stations appears to be the largest of the discrete number of available observations. There are no known crest-stage gages at the gage sites, and few, if any, of the reported peak flow measurements appear to be based on observed high water marks. The following points are noted:

- Because continuous stage records are lacking at many sites, many reported "peak annual flows" may understate true instantaneous peak annual flows. However, due to the considerable uncertainty in the magnitude of high flows resulting from extrapolation of rating curves, it cannot be determined if the reported "peak annual flows" are in fact understated.

- Measurement procedures appear to be inconsistent from year to year. In some years, some stations report the peak annual flow as having the same value as the maximum daily annual flow, implying that only one stage measurement was made on that day, even though the gage was reported to be read two or three times a day.

Review of Annual Runoff Data. The annual runoff (mm) for each complete year of streamflow record was computed for all stations and is shown in Table 2.3.7. Some serious inconsistencies are evident:

- The Porac River near Valdez (drainage area 118 km^2) shows a significantly lower yield than the Porac River near Del Carmen (drainage area 111 km^2). The Valdez record is presumably affected by irrigation diversions or diversions to the Gumain Floodway.

- The Santo Tomas River shows an abrupt drop in yield in 1967 (drop was also indicated on the double mass plot).

- The Pasig-Potrero River at Hacienda Dolores (drainage area 28 km^2) shows an abrupt drop in yield in 1969.

Because of significant periods of missing data, the annual runoff data from several stations could only be estimated for two or three years and were not considered to be of value to the hydrologic analyses.

Due to lack of overlapping rainfall and streamflow data, only limited comparison of annual runoff and annual rainfall data was possible. Plots of annual rainfall at Clark AFB against annual runoff on the Gumain, Porac, and O'Donnell Rivers are given in Figures 2.4.2 through 2.4.4, and a plot of annual rainfall at Cubi Point NAS against runoff on the Santo Tomas is given in Figure 2.4.5. With the exception of the Gumain River (Figure 2.4.2), these plots do not show a good relationship between annual rainfall

and annual runoff. The plot for the Santo Tomas River (Figure 2.4.5) would improve somewhat if suspect data after 1967 were dropped from the analysis.

Review of Gaging Stations. The records available for analyses are listed in Table 2.4.6. The principal findings of the review of stations from which these records were obtained are provided below for each gage site:

Bulsa River (Station W010A): The headwaters for the Bulsa River are on the east slopes of the Zambales Mountains, 15 to 45 km north of Mount Pinatubo. The record shows unusually high yield (i.e. normalized peak flow in $m^3/s/km^2$) relative to other stations whose headwaters originate on Mount Pinatubo, with some periods reporting extreme (and likely erroneous) monthly runoff (e.g. runoff for July 1972 was reported as 3.8 meters, with monthly rainfall at Iba of 1.7 meters and at Hacienda Luisita 1.6 meters). The high yield relative to basins on Mount Pinatubo may result in part from different geologic conditions. However, the Bulsa's double mass curve is concave upward, indicating a progressive increase in flows with time relative to other stations, and its rating curves show a progressive upward shift due to aggradation.

O'Donnell and Bangat Rivers (Stations W011A, W011B, and W012A): The records from the O'Donnell and Bangat Rivers show very significant and irreconcilable inconsistencies. Peak flows on the Bangat (Station W012A, drainage area 90 km^2) are invariably much higher (by as much as a factor of 6) than those recorded at the downstream gage on the O'Donnell (drainage area 240 km^2). A possible high flow diversion exists from the O'Donnell above gage W011B into the Bangat above gage W012A. However, no diversion has been identified out of the system between the Bangat River gage W012A and the O'Donnell River gage W011A. Records for these stations are less than 10 years in length.

Camiling River (Stations W023A and W023B): The headwaters of the Camiling River are on the east slopes of the Zambales Mountains from 40 to 60 km north of Mount Pinatubo. The records show an unusually high yield (i.e. normalized peak flow in $m^3/s/km^2$) relative to other stations originating on Mount Pinatubo. The record for both these stations is less than 10 years in length.

Pasig-Potrero River (Stations W081A and W082A): The record from station W081A (drainage area 242 km^2) may have been tidally affected and stage records only are available for much of the record. Discharge records are available for only five years from Station W081A and six years from Station W082A.

Porac River (Stations W083A and W084A): The records from the two Porac River gages (Station W083A with drainage area 118 km^2 and W084A with drainage area 111 km^2) show significant and irreconcilable inconsistencies, with the downstream gage frequently showing significantly higher peak flows than the upstream gage despite the very small difference in drainage area. It is likely that the records are affected by the operation of both irrigation and flood control projects. However, no information could be obtained on the configuration or operation of these schemes.

Gumain Floodway (Station W085A): This station is at the downstream end of the Gumain Floodway and the record is affected by upstream flood control and irrigation projects. No information could be obtained on the configuration or operation of these projects and how they affect the flow record at gage W085A.

Gumain River (Station W086A): The headwaters for the Gumain River are on the southeast slopes of Mount Pinatubo. The available daily record is 15 years in length. There are no known upstream diversions into or out of the system. The record appears to be of reasonable quality. A water level recorder was in operation for most of the period of record.

Caulaman River (Station W087A): The Caulaman River originates on the east slopes of Mount Bitnung 15 km south of Mount Pinatubo. It has not been possible to determine the exact location of this gage. Assuming the reported drainage area is correct, the records show an extremely high yield relative to other stations. Reported monthly and event runoff is occasionally (and likely erroneously) extremely high. For example, the reported runoff for September 1963 was 1.85 meters with reported rainfall at Cubi Point NAS of 0.93 meters and at Clark AFB of 0.51 meters.

Colo River (Station W088A): The Colo River basin lies approximately 30 km south of Mount Pinatubo. The station is downstream from an irrigation dam. The effect of this dam on the record is not known.

Bagsit River (Station W092A): The Bagsit river originates on the west slopes of the Zambales Mountains approximately 40 km north of Mount Pinatubo. The basin was judged to be too far from Pinatubo to be representative of hydrologic conditions of interest.

Bucao River (Station W093A): The Bucao River originates on the northwest slopes of Mount Pinatubo. The available record of daily flows is 15 years in length. Although the record quality is uncertain because of extreme extrapolation of the available rating curves, the record prior to 1972 appears to be relatively consistent. There are occasional periods of suspiciously high reported daily runoff volumes. A water level recorder was apparently in operation for part of the record.

Santo Tomas (Station W094A): The Santo Tomas River originates on the southwest slopes of Mount Pinatubo. The available record of daily flows is 15 years in length. The double mass curve for the Santo Tomas showed discharge volumes after 1967 to be significantly reduced by irrigation diversions upstream of the gage. However, the 11 years of data prior to 1967 appears to be relatively consistent.

Maloma River (Station W099B): The Maloma River originates on the west slope of Mount Pinatubo. Seven years of flow data are available; however, the record is judged to be exceedingly poor and too unreliable to be of use.

2.4.2 Streamflow Characteristics. Table 2.4.7 lists the 15 streamflow gages considered in the hydrologic analyses, together with rainfall and evaporation gages. The list excludes those gages determined to be unreliable or not useful to the analyses. Gage locations are shown on Plate 1. The data available for each streamflow gage consist of reported mean daily discharges and annual peak instantaneous discharges. No continuous or short-duration flow hydrograph data are available.

The convention adopted to identify streamflow gage data consists of a five-character code:

- the first character is "W" to signify a streamflow gage;
- the second through fourth characters identify the stream gage number, based where possible on gage numbers published by Philippine agencies;
- the fifth (and final) "A" or "B" character signifies whether the gage actually had a published gage number: "A" signifies that the number was published, and "B" signifies that no gage number had been published and that one was arbitrarily assigned for purposes of this analysis.

Periods of record of available daily streamflow data are summarized by Table 2.4.7 and Figure 2.4.6 together with rainfall and evaporation data. Record lengths vary from five to 16 years and are mostly within the period 1957 to 1972. Peak instantaneous streamflow data considered in this analysis are summarized by Table 2.4.8. Record lengths vary from five to 33 years, with very little data available after 1972.

Summary hydrographs at each of the gages are presented on Figures 2.4.7 through 2.4.21. The high points shown on the figures reflect discrete major flood events; differences in the presence or absence of specific peaks from gage to gage are due in part to differences in the periods of record.

Plots of mean monthly minimum, average, and maximum daily discharges are presented on Figures 2.4.22 through 2.4.36. The mean monthly maximum daily discharge for the month of June, for example, was determined by averaging the maximum daily values for June from each year of record.

The plots on Figures 2.4.7 through 2.4.36 indicate a strong seasonal pattern of low flows during the months of January through April and high flows during the months of June through October. High flows indicated for May mostly reflect a major storm in May 1966. The Bucao River (W093A) has no data for this month; hence, the Bucao River data as shown on Figures 2.4.19 and 2.4.34 are misleading by wrongly suggesting that high flows have not occurred on this river in May.

Plots on Figures 2.4.37 through 2.4.51 show the daily flow duration curves for each of the streamflow gages. Plots on Figures 2.4.52 and 2.4.53 show normalized flow duration curves for all gages, in which the curves were normalized by dividing the curve ordinates by the average daily flow for each gage.

The normalized flow duration curves show pronounced differences between the gages at higher discharges. While the reasons for these differences are not known, inaccuracies in the reported peak discharges are probably one factor.

2.4.3 Runoff. Figures 2.4.54 through 2.4.68 summarize the annual runoff for each of the streamflow gages.

2.4.4 Frequency Analyses. The HEC-FFA Flood Frequency Analysis program Version 3.0, which computes flood frequencies using a Log Pierson Type 3 fit in accordance with guidelines described in *Bulletin 17B* of the U.S. Water Resources Council[5], was used to assess frequency characteristics of both flood flow and rainfall data.

Rainfall Data. HEC-FFA frequency analyses of maximum annual 1, 2, 5, 10, and 15-day duration rainfall amounts were conducted for all daily data rain gage stations.

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[5] U.S. Water Resources Council, March 1982, *Guidelines for Determining Flood Flow Frequencies*. Bulletin 17B.

Tables 2.4.9 through 2.4.13 summarize the rainfall frequency data as computed from the raw data and also after multiplication by factors to convert from observational day data to *n*-hour amounts. The following empirical factors suggested by the U.S. Weather Bureau[6,7] were used:

n-days	n-hours	conversion factor
1	24	1.13
2	48	1.04
5	120	1.02
10	240	1.01
15	360	N/A

The plotted analytical frequency curves fit the data well for all *n*-day durations at all gages as typified by Figures 2.4.69 and 2.4.70.

The frequency data support the earlier observation that daily and multi-day rainfall is significantly greater along the coast than in the interior.

Streamflow Data. The criteria for selecting stations for frequency analysis were generally as follows:

- minimum of 10 years record (excluding periods of clearly erroneous data);

- headwaters originating on Mount Pinatubo (records from streams originating in other areas were not selected for frequency-based calibration because of lack of information on surficial geology and concern about whether such records would be representative of hydrologic conditions on Mount Pinatubo);

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[6] Weather Bureau, U.S. Dept. of Commerce, no date, *Rainfall Frequency Atlas of the United States for Durations from 30 Minutes to 24 Hours and Return Periods from 1 to 100 Years.* Technical Paper No. 40.

[7] Weather Bureau, U.S. Dept. of Commerce, 1964, *Two- to Ten-Day Precipitation for Return Periods of 2 to 100 Years in the Contiguous United States.* Technical Paper No. 49.

- clearly defined drainage area with consistent records and no significant upstream diversions into or out of the system (e.g., for irrigation or flood control).

It was determined in the review of gaging stations in Section 2.4.1 that three stations met the above criteria. These were the Gumain River (W086A), the Bucao River (W093A), and the Santo Tomas River (W094A).

For each of these gages, frequency analyses were conducted on the following maximum annual data:

A) Peak instantaneous discharge
B) Mean daily discharge (and corresponding 1-day volume)
C) Mean 3-day discharge (and corresponding 3-day volume)

Also for each of the gages, different data sets were assessed according to data availability and reliability, as follows:

A) Screened data set within the period 1957 through 1972. As previously discussed, these data were screened by double-mass analysis and other methods to eliminate obviously unreliable data.

B) Extended data set including the screened data plus additional pre-1957 and post-1972 annual maximum peak instantaneous and daily discharges. The extended portion of this data set was obtained from Philippine summary sheets listing only annual maximum peak instantaneous and daily discharges. Data outside the 1957 through 1972 period are considered "not screened" and therefore of unknown quality. Table 2.4.7 shows periods of record available at each gage using the extended data set.

Relative to the screened data set, the extended data set added 16 data points for frequency analyses of peak and daily flows on the Gumain River and nine data points for frequency analysis of peak and daily flows on the Santo Tomas River. No additional data points were gained for the Bucao River or for analysis of multi-day (i.e., 3-day) flows and volumes at any of the gages.

C) Subsets of the above data sets (A and/or B), created by excluding years in which data anomalies were noticed on or near the day of maximum annual flow. In practice, data subsets were analyzed for the Santo Tomas and Bucao Rivers only, and in each case excluded data for year 1962.

Gumain River Frequency Analysis.

Two data sets were analyzed for the Gumain River (W086A): 1) screened data for 1957 through 1971 and 2) extended data for 1947 through 1979. The screened data set is believed to be of high quality because the data were collected using an automatic water level recorder. Post-1971 data, and possibly much of the pre-1957 data, were based on staff gage readings only. Furthermore, all post-1972 data are subject to greater uncertainty due to a significant deterioration in the gaging program after that date. The most significant difference between the two data sets is that the maximum peak instantaneous discharge reported under the extended data set, 740 m^3/s in 1976, is nearly double the highest value reported under the screened data set, 375 m^3/s in 1964.

Figures 2.4.71 through 2.4.76 show the results of the frequency analyses on data for the Gumain River (W086A), plotting the maximum event data together with expected probability frequency curves as computed by the HEC-FFA program. These figures are discussed briefly in the paragraphs which follow.

1) Figure 2.4.71 shows the frequency analyses results arranged to show the peak instantaneous, 1-day, and 3-day flows together for the screened data set. Extended data set results (for which 3-day data were not available) are shown by Figure 2.4.72. Figure 2.4.71 shows that the computed frequency curves fit the peak, 1-day, and 3-day values from the screened data set reasonably well, although the lines are not parallel as might be expected. For the extended data set, Figure 2.4.72 shows a good fit for the 1-day data but only a fair fit for the peak instantaneous data, due to the extraordinarily high peak instantaneous data point for year 1976, which has a considerable influence on the shape of the frequency curve and on estimates of flood flows.

2) Figures 2.4.73 and 2.4.74 show the frequency analyses results arranged to compare the effects of the alternative data sets for the peak instantaneous and 1-day flows respectively. The figures show that flows estimated for a given return period are sensitive to the data set used. The frequency curves computed for the extended data set are steeper than those for the screened data set, and differences between the curves are greatest at higher return periods. For example, expected probability estimates of the 100-year peak instantaneous flow vary from about 500 m^3/s based on the screened data set to 660 m^3/s based on the extended data set. Estimates of the 100-year 1-day flow vary from about 340 m^3/s based on the screened data set to 420 m^3/s based on the extended data set.

3) Figures 2.4.75 and 2.4.76 show the 1- and 3-day volume frequency analyses. These data and curves are the same as for 1- and 3-day flows, after conversion to volume units.

Bucao River Frequency Analysis.

Two data sets were analyzed for the Bucao River (W093A): 1) screened data for 1957-1965 and 1967-1971 and 2) a data subset consisting of the same screened data in (1) but excluding data from year 1962. Data from year 1966 were excluded from both of the data sets because there were no data for that year's peak flow month (based on other gages) of May.

The year 1962 was excluded from the second Bucao River data set because a single value, 2,220 m^3/s, was reported to be both the peak instantaneous discharge and the maximum average daily discharge for the year. As a peak instantaneous discharge, 2,220 m^3/s ranks second in the station's history. However, as a maximum average daily discharge, this value would be more than double the second highest daily value of 992 m^3/s reported in 1961. Furthermore, total runoff from the basin for the five-day period in 1962 of July 20-24 was 942 mm, compared to a total rainfall at Iba for the same period of only 530 mm. Even allowing for spatial variations in rainfall and increases in rainfall with elevation, the total runoff volume appears unreasonably large in comparison to Iba rainfall.

The Bucao River data are of mixed quality. An automatic water level recorder was in place for 1957 through 1964; post-1964 data are based on staff gage readings made twice daily. The reported maximum annual discharge for year 1962 was measured/computed by the "slope-area" method.

Figures 2.4.77 through 2.4.83 show the results of the frequency analyses on data for the Bucao River (W093A), plotting the maximum event data together with expected probability frequency curves as computed by the HEC-FFA program. These figures are discussed briefly in the paragraphs which follow.

1) Figures 2.4.77 and 2.4.78 show the frequency analyses results arranged to show the peak instantaneous, 1-day, and 3-day flows together for each of the data sets, i.e. the screened data set with and without year 1962 data. The curves generally fit the data best for the data set which excludes year 1962 data.

2) Figures 2.4.79 through 2.4.81 show the frequency analyses results arranged to compare the effects of the alternative data sets for the peak instantaneous, 1-day, and 3-day flows respectively. The figures show that all estimated flows, and the 1- and 3-day flows in particular, are sensitive to the data set used, and that curves computed on the data sets including year 1962 data are steeper than those for which that year was excluded. Differences between the curves are greatest at higher return periods. For example, expected probability estimates of the 100-year peak instantaneous flow vary from about 3,740 m^3/s, excluding year 1962 data, to 4,330 m^3/s,

including 1962 data. Estimates of the 100-year 1-day flow vary from about 1,260 m^3/s excluding 1962 data to 2,730 m^3/s when including 1962 data.

3) Figures 2.4.82 and 2.4.83 show the 1- and 3-day volume frequency analyses. These data and curves are the same as for 1- and 3-day flows, after conversion to volume units.

Santo Tomas River Frequency Analysis.

Four data sets were analyzed for the Santo Tomas River (W094A): 1) screened data for 1957 through 1967; 2) extended data for 1948 through 1967; 3) screened data set (1) excluding year 1962 data; and 4) extended data set (2) excluding year 1962 data.

All Santo Tomas River data are based on staff gage readings. Year 1962, which includes the historical maximum peak instantaneous and daily values, was excluded from the final sets of data because of anomalies in the daily data: the minimum daily discharge for 1962 is reported to occur just two days after the maximum flow. As the Santo Tomas River gage was located 400 meters downstream of a structure that is believed to be a major irrigation project, it is possible that high flows during the flood may have been supplemented by releases of reservoir storage, and that the very low flows after the flood reflect the recovery of reservoir storage.

Figures 2.4.84 through 2.4.92 show the results of frequency analyses on data for the Santo Tomas River (W094A), plotting the maximum event data together with expected probability frequency curves as computed by the HEC-FFA program. These figures are discussed briefly in the paragraphs which follow.

1) Figures 2.4.84 and 2.4.85 show the frequency analyses results arranged to show the peak instantaneous, 1-day and 3-day flows together for the screened data set with and without year 1962 data. Extended data set results (for which 3-day data were not available) are shown by Figures 2.4.86 and 2.4.87. The curves are generally more parallel for the extended data sets than for the screened data sets. The best fit of expected probability curve to the data is for 1-day discharges with the extended data set excluding 1962 (Figure 2.4.87).

2) Figures 2.4.88 through 2.4.90 show the frequency analyses results arranged to compare the effects of the alternative data sets for the peak instantaneous, 1-day, and 3-day flows respectively. The figures show that all estimated flows are sensitive to the data set used, and that curves computed on the data sets including year 1962 data are steeper than those in which that year was excluded. Differences between the curves are greatest at higher return periods. For example, expected probability estimates of the 100-year peak instantaneous flow vary from about

800 m^3/s based on the extended data set excluding year 1962 data to 1,560 m^3/s based on the screened data set including year 1962 data. Estimates of the 100-year 1-day flow vary from about 520 m^3/s based on the extended data set excluding year 1962 data to 1160 m^3/s based on the screened data set including year 1962 data.

3) Figures 2.4.91 and 2.4.92 show the 1- and 3-day volume frequency analyses. These data and curves are the same as for 1- and 3-day flows, after conversion to volume units.

Summary.

Frequency analyses of the maximum streamflow data are greatly complicated by the uncertain quality of the available data and related uncertainty over which of the possible data sets is more representative of "true" conditions. The choice of data set significantly influences estimated high return period flows and volumes.

For all three gages assessed, the fit of the computed curves to the data was generally improved by disregarding high peak values over which there was some justifiable uncertainty. However, a review of rainfall data showed that these and other (uncertain) extreme flows were associated with very heavy rainfall expected to result in extreme flows. It is inappropriate to either fully accept or disregard the extreme value data.

Given the available record of uncertain data and the sensitivity of the analyses to single data points, single-value estimates of 100-year or other return period flows are not made. Instead, the families of expected probability frequency curves shown by Figures 2.4.71 through 2.4.92 are used to indicate a plausible range of reasonable values. Ranges for 2- and 100-year return periods are summarized in Table 2.4.14. The results of this analysis were used to calibrate the hydrologic model, HEC-1, which was used to generate design flood hydrographs.

2.4.5 Design Storms. The objective of the design storm analysis was to develop rainfall hyetographs for 2-, 10-, 50-, 100-, and 500-year hypothetical storms of appropriate duration to result in similar return period flooding of streams affected by the Mount Pinatubo eruption. The assessment was complicated by the fact that no rainfall data are available to describe conditions in the mountain watershed areas of interest.

The approach taken to develop the design storms is described in the following report sections. In summary:

1) Isopluvial maps of 2-, 10-, 50-, 100-, and 500-year rainfall over durations of 1-, 2-, and 5-days were developed from the frequency analysis of available rain data and with the assumption that the ratios of coastal-to-

mountain rainfall amounts would be the same as for annual rainfall amounts shown by the NAP map.

2) U.S. Weather Bureau area-reduction factors were reviewed in light of the available rainfall data, and a methodology for applying area-reduction factors jointly with elevation-based variations in rainfall was developed.

3) Short-duration (less than 24-hour) storm characteristics were assessed from available published intensity-duration-frequency data and from the limited available hourly data. A methodology was developed to construct and distribute design storm hyetographs throughout the basins of interest.

Frequency-Duration Isopluvial Maps. Figures 2.4.93 through 2.4.107 show isopluvial maps of 2-, 10-, 50-, 100-, and 500-year rainfall over durations of 1-, 2-, and 5-days.

The isopluvial lines shown for the coast and interior lowlands are based on the rainfall frequency analyses results summarized by Tables 2.4.9 through 2.4.13. Isopluvial lines for the mountain watershed areas are based on the assumption that rainfall near the 1,000-meter contour around the summit of Mount Pinatubo is, for all return periods, equal to the average rainfall at the coastal gages multiplied by a factor of 1.35 for all durations and return periods. The 1,000-meter contour follows the top of a narrow ridge to the south of Mount Pinatubo and an irregular circular path about 5 km in diameter around Pinatubo's 2.5 km diameter (post-eruption) crater rim. The 1.35 factor is based on average annual data for coastal rain gages and on the NAP map.

Tables 2.4.15 through 2.4.17 provide data to examine the assumption that the ratio of same-return-period rainfall at the coast, at high elevations, and in the interior is relatively constant for all durations and return periods. Table 2.4.15 presents data and ratios of interior gages to coastal gages for multi-day duration events. Tables 2.4.16 and 2.4.17 present short-duration data and ratios for stations representing interior, coastal, and high-elevation sites. Figure 2.4.108 plots the ratios for the short-duration data; Figure 2.4.109 plots the ratios between Hacienda Luisita and Iba for both short- and long-duration events.

Several observations are made from the data:

1) For durations of six hours and longer, the ratio of same-frequency-duration rainfall between any two stations is essentially constant, independent of the duration. This ratio is approximately equal to the ratio of average annual rainfall between any two stations.

2) For durations of one hour or less, rainfall depth is essentially independent of station location or elevation. For example, the 100-year 1-hour rainfall depth at coastal sites is essentially the same as the depth at sites in the mountains and the interior.

3) There is no clear indication of the effect of return period on the ratio of same-frequency-duration rainfall between two stations. The short-term data suggest that differences between two stations become slightly greater (ratios further from 1.0) with increasing return period. However, the long-term data suggest the opposite, that differences become slightly less (ratios closer to 1.0) with increasing return period. The discrepancy may result from the fact that the data analyses were conducted using different theoretical distributions. The short-duration data were assessed by PAGASA using a Gumbell distribution, which has a fixed skew. The long-duration data were assessed as part of this study using a Log Pierson III distribution (HEC-FFA) in which skew is calculated from the data.

In summary, for rainfall durations of six hours and greater, the data show that mountain rainfall of a given frequency can reasonably be estimated as a constant multiple of same-frequency average coastal rainfall. A constant multiplier equal to the ratio of average annual rainfall appears reasonable for durations of six hours or longer. No adjustment, i.e. a multiplier of 1.0 applied to coastal rainfall, is appropriate for durations of one hour or less.

Area Reduction Factors. Area-reduction factors are used to estimate average basin-wide or area rainfall amount from point values. In relatively flat topography, the 100-year 1-hour rainfall over a basin area is expected to be less than the 100-year 1-hour rainfall at any point within the basin.

Area-reduction factors for the continental U.S. have been developed by the U.S. Weather Bureau[8] based on data from dense rain-gage networks. These factors have previously been considered "reasonable" to be applied to conditions in Hawaii[9], despite a lack of data to test the relationships for Hawaiian conditions. After an inconclusive check for reasonableness relative to the available study area data, the Weather Bureau factors were adopted for use in the hydrologic analysis.

Figure 2.4.110 shows distance-reduction curves derived from the Weather Bureau area-reduction factors. The latter, which were based on circular areas, were converted to

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[8] See Footnote 7.

[9] Weather Bureau, U.S. Department of Commerce, no date, *Two- to Ten-Day Rainfall for Return Periods of 2 to 100 Years in the Hawaiian Islands*. Technical Paper No. 51.

distance-reduction factors by assuming that peak rainfall occurs at the center of the circle and that rainfall decreases linearly with radial distance from the center. This conversion was done to facilitate comparison with the available data and to provide a methodology which could be applied jointly with factors to account for elevation-based variations in rainfall.

Construction of Storm Hyetographs. Table 2.4.18 summarizes frequency-depth-duration data for a hypothetical rain gage located near the summit of Mount Pinatubo at about 1000 meters elevation. These data provide the basis for construction of design storms hyetographs.

The 24-hour, 2- and 5-day data in Table 2.4.18 are approximately equal to 1.35 times average coastal gage values shown in Table 2.4.15; the data vary slightly from the 1.35 multiplier due to rounding effects when tabulating hourly data for HEC-1 model input. The 1-, 2-, 3-, 6-, and 12-hour data in Table 2.4.18 are based on the short-duration data of Table 2.4.16 with emphasis given to the high-elevation station at Baguio City.

While peak discharges on rivers draining Mount Pinatubo generally result from heavy rains lasting less than 12 hours, a design storm duration of five days was selected to provide additional data on total runoff volumes as might be required for later design work. Design storms of a specified return period were constructed by embedding same-frequency events; i.e., the 100-year 24-hour rain was embedded within the 100-year 2-day rain, which was embedded within the 100-year 5-day rain. This general approach was consistent with the available daily rainfall data.

During early efforts to calibrate the HEC-1 model to the region, it became apparent that the conservative approach of fully embedding all same-frequency durations within a single design storm yielded excessively high peak discharges. Unfortunately, peak discharges on the basins of interest result from short-duration (typically 3-hour to 12-hour) events, while hourly data were available for only two significant storm events.

Figure 2.4.111 plots hourly data and associated frequency-duration characteristics for a severe storm recorded at Iba (RD324) in May 1976. While the maximum 48-hour rain corresponds to a 200-year storm and the maximum 24-hour rain corresponds to a 25-year storm, the maximum hour corresponds to only a 2-year storm.

Figure 2.4.112 plots hourly data and associated frequency-duration characteristics for a severe storm recorded at Baguio City in September 1911. The rainfall data were obtained from Weather Bureau Technical Report No. 42[10] and reflect a (former) world

[10] See Footnote 2.

record 24-hour rainfall amount. While the maximum 24-hour rain corresponds to about a 200-year storm, the maximum hour corresponds to only about a 20-year storm.

The available historic storm data support the proposition that a single design storm should not fully embed all same-frequency depth-durations, i.e., that a 1 in 100 year 24-hour rainfall should not contain a 1 in 100 year 1-hour rainfall. Possible physical explanations for this are that the monsoon or typhoon conditions responsible for large daily rainfalls are not conducive to the formation of intense cloudbursts, or that the monsoon or typhoon winds associated with large daily rainfalls do not allow intense cloudbursts to stay in one place long enough for significant rainfall amounts to accumulate at a point.

Figure 2.4.113 plots the 2-, 10-, 50-, 100-, and 500-year hypothetical storms adopted as "base storms" before adjustments for elevation-based total storm depth and for depth-area reduction. These hyetographs are for a hypothetical station located at approximately 1,000 meters elevation near the summit of Mount Pinatubo, and embed same-frequency events from six hours duration through five days duration. A constant hourly intensity is assumed throughout the maximum 6-hour period.

A constant hourly intensity over the maximum 6-hour period produces 1-hour to 24-hour characteristics which are similar to those observed in the two historic storms for which hourly data were available. The maximum hourly rain in a 25-year 24-hour storm has a return period of about two years, and the maximum hourly rain in a 200-year 24-hour storm has a return period of about 20 years.

The procedure described below was developed to construct multi-basin rainfall hyetographs for a single storm event. In this description, "summit rain" refers to rainfall characteristics at the hypothetical station at approximately 1,000 meters elevation near the summit of Mount Pinatubo. "Base storms" refer to design rainfall hyetographs for this hypothetical station as shown by Figure 2.4.113.

1) The basin of interest was subdivided into smaller sub-basins, considering isohyetal gradients through the basin as one factor.

2) The average annual rainfall for each sub-basin was determined from the NAP map (Figure 2.3.4), and the ratio of sub-basin rain to summit rain was determined. For any sub-basin, this ratio is a constant for all return periods and for all durations greater than six hours.

3) For each sub-basin, the base storm was adjusted to correct for the ratio of sub-basin rain to summit rain. The ratio was applied directly for all durations of six hours and longer, and factored within the peak 6-hour period so that a single peak hour was not adjusted. The justification for maintaining the peak hourly intensity at the average 6-hour intensity for

the base storm is the previously stated observation that peak hourly rainfall intensities appear to be independent of location and elevation. Figure 2.4.114 illustrates this step in sub-basin hyetograph development by showing a 50-year base storm before and after the adjustments for a sub-basin normally receiving 80 percent of the summit rain.

4) A storm center was assumed to be located in the middle of the sub-basin with the highest rainfall. The distances from the assumed storm center to the mid-points of all other sub-basins were determined. For the sub-basin containing the assumed storm center, the average distance to the storm center was estimated as 2/3 of the radius of a circle having an area equal to the sub-basin area.

5) Finally, depth-area corrections were applied to each sub-basin storm (i.e., the base storm after correction for the ratio of sub-basin rain to summit rain). These corrections followed the depth-duration-distance curves shown by Figure 2.4.110 for durations of six hours and greater. The 6-hour adjustment curve of Figure 2.4.110 was applied to all durations within the peak 6-hour period because of the earlier assumption of a constant hourly intensity within the peak 6-hour period for the base storm. Figure 2.4.114 illustrates this final step in sub-basin hyetograph development by showing a 50-year sub-basin storm before and after depth-area corrections for a sub-basin located 10 km from an assumed storm center.

Sub-basin storm hyetographs derived in this manner were used as input to HEC-1 models used in the basin analyses.

3. BASIN ANALYSES

3.1 Introduction

Hydrologic basin analyses were performed on all eight major river basins impacted by the eruption of Mount Pinatubo. Hydrologic analyses products were required at specific locations within the basins for use in the design of mitigation measures. The primary products required from the hydrologic analyses are, for the purposes of the following discussion, grouped as follows: (1) 2-, 10-, 50-, 100-, and 500-year instantaneous flood peaks, 1-day volumes, and 3-day volumes and the hydrographs corresponding to these peaks and volumes, (2) flow duration curves, and (3) flow velocities and depths. The modeling methodologies used to obtain products in groups (1), (2), and (3) are discussed in Sections 3.2, 3.3, and 3.4 respectively. The results of the modeling are presented in Sections 3.5 and 3.6.

3.2 Rainfall/Runoff Modeling Methodology

3.2.1 Introduction. Rainfall/runoff processes were simulated with the use of an HEC-1 computer model. Model runs were made with the 2-, 10-, 50-, 100-, and 500-year hypothetical storm events obtained from the regional analyses in order to obtain estimates of the 2-, 10-, 50-, 100-, and 500-year instantaneous flood peaks, 1-day volumes, and 3-day volumes and the corresponding hydrographs.

Methods used to obtain HEC-1 model parameters are discussed in Section 3.2.2. The methodology used in the calibration of the HEC-1 model is discussed in Section 3.2.3. The methodologies used in the construction of the basin models are described in Section 3.2.4.

3.2.2 HEC-1 Rainfall/Runoff Model Parameters. Hydrologic modeling for this study was done using an extended memory version of HEC-1. The extended memory version of HEC-1 allows event simulation for up to 2,000 computational time steps, as compared to only 300 time steps in earlier releases of the model. The extended memory version, however, assumes the time base of the unit hydrograph is less than 300 time steps. All modeling work was done using metric units. Minor modifications to the extended memory program were made by the Hydrologic Engineering Center (HEC) to allow output of unit hydrograph ordinates to three significant digits when working in metric units.

Computational Time Step. Hydrologic modeling for rivers draining Mount Pinatubo is complicated both by the lack of good hydrometeorologic data and by the flat response of event hydrographs. Storm event hydrographs are often quite flat, with recorded ratios of instantaneous peak to maximum daily average flows of as little as 1.1 or 1.2. Furthermore, major storms affecting this area are frequently several days in duration, resulting in a need to model hydrologic response from a storm event for periods of up to about 10 days.

The flat response of the drainage basins under study naturally produces a flat unit hydrograph with an extended time base. HEC-1 assumes that the length of the unit graph time base is less than 300 time steps. Any volume in a unit hydrograph beyond this point is lost, such that the program fails to conserve mass. In order to model the flat response of these rivers while avoiding unacceptable loss of volume in the tails of the unit hydrographs, modeling had to be done at a 1-hour time step.

While a 1-hour time step is appropriate for modeling most of the larger study basins, modeling was also required to simulate flows from a number of small headwater catchments with drainage areas as small as 4.4 km^2, and with times of concentration as low as 0.4 hour. While this situation would normally require modeling at a time step of perhaps as small as 10 minutes, tests conducted during the analysis found the model response to be relatively insensitive to time step due to the flat response of hydrographs and the relatively large amounts of storage implied. Tests conducted on small sub-basins with modeling time steps from five minutes to one hour showed a difference in simulated peak flows of about 2 percent -- substantially less than the degree of uncertainty in the basic data used for model development.

Unit Hydrograph. Three synthetic unit hydrographs are available in HEC-1:

- Clark Unit Hydrograph
- Snyder Unit Hydrograph
- SCS Dimensionless Unit Hydrograph

The shape of the SCS unit hydrograph is controlled by a single parameter, namely, basin time lag. Preliminary simulations showed it to be incapable of reproducing the peak-to-volume characteristics of observed hydrographs in the Pinatubo region. The SCS unit graph produced a much sharper response than observed, such that it was not possible to maintain both observed peaks *and* volumes. Most of the HEC-1 modeling was therefore done using the Clark Unit hydrograph. This method uses three parameters to compute a unit graph: time of concentration, a storage coefficient, and a time-area curve. The greater flexibility inherent in the Clark method was found to greatly improve modeling results. The Clark storage coefficient provides an avenue for modeling significant attenuation of peak flows in the highly permeable volcanic deposits on Mount Pinatubo. The SCS unit graph was retained and used to represent runoff from the relatively large impervious paved areas of two sub-basins in the Abacan River catchment that incorporate parts of Clark AFB and nearby urban areas.

Parameters for the Clark unit hydrograph were determined as follows:

- Time of Concentration

In the absence of observed data, time of concentration was determined by an empirical equation provided by the U.S. Bureau of Reclamation[11] as:

$$T_c = \left(\frac{11.9 \cdot L^3}{\Delta H} \right)^{0.385}$$

where
T_c = time of concentration (hours)
L = length of longest watercourse from the point of interest to the watershed divide (miles)
ΔH = elevation change along the longest water course (feet)

- Storage Coefficient

The Clark storage coefficient was determined by calibration to available data, as will be discussed in detail in Section 3.2.3, and expressed as a function of T_c to facilitate application to ungaged sub-basins.

- Time-Area Curve

The time-area curve describes the cumulative area of a sub-basin contributing runoff to the sub-basin outlet as a function of time, expressed as a fraction of the time of concentration. The default time-area curve provided by HEC-1 was used for all sub-basins. HEC-1 uses a dimensionless time-area curve given by the following:

$$AI = 1.414T^{1.5} \qquad 0 \leq T < 0.5$$

$$1 - AI = 1.414(1 - T)^{1.5} \qquad 0.5 \leq T < 1$$

where
AI = cumulative area as a fraction of the total sub-basin area
T = fraction of the time of concentration

Parameters for the SCS unit hydrograph were determined as follows:

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[11] United States Department of the Interior, 1977, *Design of Small Dams*, Bureau of Reclamation, Washington D.C.

- Time Lag

The time lag is the lag (in hours) between the center of mass of rainfall excess and the peak of the unit hydrograph. The lag-time was determined using an empirical expression recommended by the U.S. Soil Conservation Service:

$$T_L = 0.6 \cdot T_c$$

where
T_L = lag time, hours
T_c = time of concentration in hours, estimated using the same expression as described above for the Clark unit hydrograph.

Base Flows. Base flows in the context of the basin analyses are representative flood season low flows expected to occur at simulation sites prior to the onset of the flood hydrograph. They are significantly higher than the low base flows which occur during the dry season.

Flood-season base flows at stream gages were estimated by inspection of time series hydrograph plots. Average annual rainfall for each basin was estimated (as before) to be the average annual runoff plus 1,550 mm to account for evapotranspiration. Comparison of these data yielded the following approximate relationship for estimation of flood-season base flows on streams originating on Mount Pinatubo as a function of average annual basin rainfall and basin area:

$$Qbase = A\,(R/17{,}500 - 0.12)$$

where
Qbase = Typical flood season base flow, m^3/s
A = Basin area, km^2
R = Average annual rainfall over basin, mm/yr

Base flows determined by this relationship for each sub-basin were assumed to be constant (without any recession) throughout each event simulated in the HEC-1 modeling.

Loss Rates. The HEC-1 model allows the computation of loss rates by a number of different methods. These methods include:

- initial and uniform loss rate
- exponential loss rate
- SCS Curve Number method
- Holton loss rate

All modeling for this study assumed a uniform loss rate with no initial loss. Major floods in this area occur primarily during the Southwest Monsoon. Significant amounts of rainfall can be expected prior to flood-producing events which are therefore assumed to occur under essentially saturated conditions.

The uniform loss rate was estimated by calibration to available data as will be described in Section 3.2.3.

Routing Parameters. Routing of flood hydrographs through the stream channels in the basins of interest was done in HEC-1 using the Muskingum method. There is a lack of basic data from which to determine channel routing parameters. Only limited information exists on channel cross-sections or floodplain geometry, no information exists on flood wave travel times, and, as discussed earlier, there is considerable uncertainty in discharge rates. In the absence of basic data, all flow routing was done assuming a flood wave velocity of 2.5 m/s and a Muskingum "X" coefficient of 0.2. A detailed description of the routing scheme implemented in HEC-1 is provided in the HEC-1 User's Manual.

3.2.3 Calibration of HEC-1 Model. HEC-1 model calibration for basins draining Mount Pinatubo was limited by the lack of reliable streamflow data. From the review of gaging stations in Section 2.4.1, it was determined that only three stations had data reliable enough for model calibration. These stations were the Gumain River (W086A), the Bucao River (W093A), and the Santo Tomas River (W094A). Model calibration was further limited by the lack of short-interval rainfall data that were concurrent with data from the above three stations.

Figures 3.2.1 through 3.2.3 show the calibration basins as they existed prior to the eruption of Mount Pinatubo. Sub-basins upstream of the stream gages were delineated to reflect major tributaries and to reflect the elevation-related rainfall gradients up the mountain slopes.

Sub-basins and simulation output sites defined for pre-eruption modeling of each of the three calibration basins were assigned a two-letter prefix beginning with "P" (for pre-eruption) for identification purposes. The two-letter prefix for each of the three calibration basins is:

PG — Gumain River (W086A)
PB — Bucao River (W093A)
PT — Santo Tomas River (W094A)

Sub-Basin Identifiers. Identifiers PG1 through PG9 refer to sub-basin areas within the pre-eruption Gumain basin as shown by Figure 3.2.1. The prefix letters "PG" denote the pre-eruption Gumain basin, and the following numbers identify sequentially-numbered sub-basins. These identifiers are used in tables and figures. The

convention generally followed for sequential numbering of sub-basins was to start at the most upstream sub-basin and to end at the most downstream sub-basin.

Output Site Identifiers. Identifiers PG5US through PG9DS refer to the simulation output sites located at specific points within the pre-eruption Gumain basin as shown by Figure 3.2.1. The first two characters of the identifier indicate the sub-basin within which the site is located. The last two characters, either "US" or "DS", indicate that the site is located at either the upstream or downstream end of the sub-basin. Calibration output results are provided in tables and figures only for the site corresponding to the stream gage location, this being the most downstream site in each calibration basin.

Tables 3.2.1 through 3.2.3 summarize physical and computed parameters for each of the calibration basins. Some of these computed parameters (Unit hydrograph method, storage coefficient, loss rate) were determined through the calibration process as described in the following sections.

Calibration to Historic Event. Calibration of the HEC-1 runoff model comprises two basic steps:

1) Identifying historic events for which concurrent rainfall and runoff data are available and sufficient to describe rainfall over the basin(s) as well as the shape and volume of the resultant runoff hydrograph from the basin.

2) Running the HEC-1 model with known (historic) basin rainfall events for input, and adjusting model parameters until the model suitably reproduces the known (historic) runoff hydrograph for each event.

Because most of the basins draining Mount Pinatubo have maximum times of concentration of the order of three to 12 hours, hourly data were desirable for calibration.

HEC-1 model calibration for basins draining Mount Pinatubo was complicated by a lack of suitable concurrent rainfall and streamflow data. Available streamflow data are often of poor quality and are limited to average daily discharges and annual peak discharges. Only very limited hourly rainfall data are available from any of the rain gage stations.

There is a general mismatch between available streamflow data and available rainfall data. Streamflow data are very sparse after 1972, and only limited rainfall data are available prior to 1972. Hourly rainfall data could be obtained for only a small number of storm events from three stations in the general vicinity of Mount Pinatubo: Iba, Hacienda Luisita, and Porac.

Only one historic event, on September 1, 1970, was found with (marginally) sufficient concurrent rainfall and streamflow data for model calibration. The data consisted of hourly rainfall at Porac and streamflow data for the Gumain River at Pabanlag (W086A), which included daily flows and a peak instantaneous discharge (**NOTE:** In the publication *Philippine Water Resources Summary Data, Vol. II Streamflow and Lake or River Stage*, December 31, 1970, the peak instantaneous discharge used for the historic event calibration is reported to have occurred on September 11 at 1:30 p.m. However, no flows of significance were reported on September 11 at any other nearby gages as were reported on September 1. The form on which the date and time of the peak were recorded did not have enough spaces to record a 2-digit date and a 4-digit time. It is believed that the peak reported on September 11 at 1:30 p.m. actually occurred on September 1 at 11:30 p.m.) The stream gage on the Gumain River at Pabanlag is reported to have had a water level recorder in operation during this event, and the estimates of the daily average flows and the peak instantaneous discharge are probably more reliable than those from any other nearby gage. Although a water level recorder was apparently in operation on the Gumain, the recorder strip charts cannot be located and hence no short-interval (e.g., hourly) flow data are available.

The calibration event was the Gumain River (W086A) peak annual flow of 267.5 m^3/s on September 1, 1970, which had a return period of about one in four years. Concurrent hourly rainfall data were available from a gage at Porac, located about 20 km east of headwater areas of the Gumain River basin. Figure 3.2.1 shows the relative locations of the basin and the Porac rain gage.

Ideally, hourly rainfall data for model calibration would be available from rain gages within the watershed, or at least from several gages closely surrounding the watershed. This methodology would allow for an accurate estimation of the spatial and temporal distribution of rainfall over the basin. Unfortunately, hourly data were available for only the Porac rain gage, and there were no daily data rain gages located sufficiently close to the basin to improve estimation of actual basin rainfall.

In the absence of additional information, hourly rainfall at each of the sub-basins was estimated as being hourly rainfall at Porac multiplied by an adjustment factor to account for increasing rainfall with elevation as shown by the 2-year 24-hour isopluvial map of the study area. For each sub-basin, the adjustment factor was computed to be the ratio of the 2-year 24-hour rainfall at the middle of the sub-basin to that at Porac. By this approach, rainfall amounts on the sub-basin varied from 1.3 to 1.9 times the rainfall at Porac, averaging about 1.6 times the Porac rainfall.

Figure 3.2.4 summarizes key results from three simulations for the historic event. Variables considered in these simulations were loss rate and the Clark storage coefficient. In each case these parameters were adjusted iteratively until the historic peak discharge was reproduced.

A Clark unit hydrograph storage coefficient of 15 to 25 times the time of concentration, with appropriate loss rates, provides a fairly good approximation of the historic hydrograph. When considered together with the frequency-event calibration work, a storage coefficient equal to 25 times the time of concentration and a constant loss rate of 3 mm/hour were adopted for subsequent hydrologic analyses of basins throughout the study area.

Calibration to Frequency Characteristics. In the absence of other suitable historical data, additional calibration efforts were limited to frequency events. The frequency-event-based calibration consisted, for example, of adjusting HEC-1 model parameters so that a synthesized 100-year design storm applied over a gaged basin would generate 100-year flow volumes and peak instantaneous discharge at the gage site.

The stations chosen for the frequency-event-based calibration were the same three chosen for the frequency analyses: the Gumain River (W086A), the Bucao River (W093A), and the Santo Tomas River (W094A). The pre-eruption basins above these gages are shown by Figures 3.2.1 through 3.2.3. Due to irrigation diversions after 1967, post-1967 data from the Santo Tomas were excluded from the frequency-event-based calibration.

Frequency analyses of streamflow data from the calibration basins are described in Section 2.4.4. The results of those analyses, in light of the available record of uncertain data and the sensitivity of the analyses to single data points, were presented as families of flow frequency curves to indicate plausible ranges of reasonable discharges and volumes at various return periods. Ranges of reasonable values for 2- and 100- year return period floods were adopted as the targets to be attained in the frequency event calibration.

The procedure for calibrating the two available parameters, loss rate and storage coefficient, was first to assign a storage coefficient, and then to vary the loss rate until the target value was matched. Tables 3.2.1 through 3.2.3 summarize parameters for the three calibration basins. Table 3.2.4 and Figures 3.2.5 through 3.2.13 summarize the results of the frequency event calibrations, showing the target ranges of reasonable values together with the results obtained from HEC-1 simulations of 2- and 100-year events. The simulations summarized are based on a one-hour time step simulation of design storms of five days in duration.

The 2-year peak instantaneous discharge for the Gumain River (W086A) was taken as the primary target value for calibration, as it was considered to be the most reliable of possible targets. Parameters determined for the Gumain River were then used in models for all three calibration basins for both 2- and 100-year hypothetical storms.

In reviewing the final calibration run outputs, the greatest weight was given to matching peak instantaneous discharge, then to three-day volume. The frequency data and curves

for 1-day volumes were given the least weight because the source data were based on arbitrary calendar-day periods (and often on staff gage readings) which would tend to underestimate true 24-hour maximum values.

Based on the calibration results, it was decided to make subsequent runs of the HEC-1 model using a constant loss rate of 3 mm/hr and the Clark unit hydrograph method with storage coefficient computed as 25 times the time of concentration for each sub-basin. These parameters were considered to yield generally good results for both peak flows and total flow volumes for all three calibration basins. The simulated 100-year peak flow for the Bucao River appears low (Figure 3.2.8), but may be within reason when considering that the two extreme high recorded flows may have actual return periods in excess of about 10 to 20 years as suggested by the plotting positions based on the available period of data record.

Sensitivity Analysis. Most of the sensitivity analysis was conducted concurrently with model calibration discussed in the previous sections and as summarized by Table 3.2.4 and Figures 3.2.5 through 3.2.13. That analysis consisted of varying paired combinations of constant loss rate and Clark unit hydrograph storage coefficient.

Increasing (or reducing) the loss rate within reasonable bounds from the adopted 3 mm/hr value has a relatively large impact on total runoff volume and a relatively small impact on peak discharge. A typical 2-year 5-day storm near the summit of Mount Pinatubo, for example, would have an average rainfall intensity of about 6.5 mm/hr considering the full 5-day period, and a peak hour rainfall intensity of about 30 mm/hr. Increasing the loss rate from 3 to 6 mm/hr for determining the rainfall excess would obviously have a far greater impact on the total runoff volume than on the peak hourly flow.

Increasing (or reducing) the Clark unit hydrograph storage coefficient has a significant impact on the peak discharge, but no impact on the total runoff volume. Increasing the storage coefficient dampens the peak flow and flattens the shape of the hydrograph. The impact of the storage coefficient on the shape of the hydrograph is illustrated by Figure 3.2.4.

With the adopted unit hydrograph (Clark), storage coefficient (25 times time of concentration), and loss rate (3 mm/hour), model results were found to be generally insensitive to variations in model time step and routing parameters. Tests using the Gumain Basin of varying the model time step from five to 60 minutes, the Muskingum "X" coefficient from 0.2 to 0.4, and the Muskingum "K" coefficient within a range reflecting flood wave velocities of 1 to 5 m/s all yielded peak discharge results which were within about 2 percent of each other. Again, this lack of sensitivity results from the large storage in the system implied by the large storage coefficient used with the Clark unit hydrograph.

3.2.4 HEC-1 Basin Models

Sub-basin and Output Site Numbering Convention. Sub-basins are designated by a 2- or 3-character alpha-numeric identifier. The first character, which designates one of the eight major basins, is one of the following:

Sacobia-Bamban	- "S"	Pasig-Potrero	- "P"
Abacan	- "A"	Santo Tomas	- "T"
O'Donnell	- "O"	Bucao	- "B"
Gumain/Porac	- "G"	Maloma	- "M"

The second character, which is numeric, identifies the sub-basin within the major basin. The convention generally followed for sequential numbering of the sub-basins within a major basin was to start at the most upstream sub-basin and to end at the most downstream sub-basin.

Output sites are designated by a 4- or 5-character alpha-numeric identifier. The first two or three characters identify the sub-basin in which the site is located. The last two characters, either "US" or "DS", indicate that the site is located at either the upstream or downstream end of the sub-basin.

Structure of Basin Models. In constructing the models, sub-basins were delineated so that output would be provided for at required locations. Some of these sub-basins were further divided to define major tributary streams and to provide better definition of rainfall gradients through the basins. Additionally, (1) sub-basins S7 in the Sacobia-Bamban Basin, P7 in the Pasig-Potrero Basin, and G19 in the Gumain-Porac Basin were delineated to isolate narrow dike-confined sections of the channels, and (2) sub-basin T7 in the Santo Tomas Basin was delineated in order to define Lake Mapanuepe.

Definition of sub-basin boundaries to provide output at desired sites sometimes resulted in the creation of some very small basins. These areas were not given sub-basin numbers or explicitly identified in the HEC-1 models. Instead, for modeling purposes, these small areas were re-allocated to nearby sub-basins such that schematically there would be no intervening areas or routing reaches (e.g., the small areas just upstream of sub-basin S7 in the Sacobia-Bamban Basin (see Figure 3.5.4) were re-allocated to upstream sub-basins S3, S4, and S6 such that, schematically, simulation sites S3DS, S4DS, S6DS, and S7US all exist near site S7US without any intervening areas or routing reaches). Routing reaches through these small areas were ignored because flow travel times are significantly shorter than the one-hour model time step.

Parameter Estimation. Physical and computed parameters at simulation output sites were obtained as described below.

- Stream elevations were determined from 1:50,000 topographic maps dated 1986.

- Basin areas upstream of each site were computed as the sum of all contributing sub-basin areas.

- Basin times of concentration were computed using the formula presented in Section 3.2.2 based on the slope and length characteristics of the entire basin.

- Average annual basin rainfall amounts were estimated based on isohyets of mean annual rainfall over the study area as shown by Figure 2.3.4.

- Average annual streamflows at each site were estimated on the basis of average annual basin rainfall, basin area, and a formula that will be presented in Section 3.3.

- In some basins where simulation output sites do not share a common headwater area, more than one storm pattern is required in order to maximize flows at all sites within a given basin. Maximum flows at each site were assumed to result from storms centered over the headwater sub-basin with the highest rainfall as determined from the isohyetal map.

Physical and computed parameters for sub-basins were obtained as described below.

- Physical parameters of area, internal flow path, channel length, and elevation changes were measured from 1:50,000 scale maps dated 1986. Internal flow path lengths and elevation changes are provided for all sub-basins, as required for determining time of concentration for local sub-basin runoff. Channel lengths through sub-basins, and corresponding elevation changes, are applicable only in cases where there is an upstream basin generating incoming channel flow to be routed through the sub-basin.

- As noted in Section 2.4.5, the ratio of sub-basin event rainfall to summit rainfall was determined to be approximately constant for all return periods and for all durations greater than six hours. For convenience, the design storm parameter of sub-basin rainfall as a percentage of summit rainfall was determined from 50-year 24-hour rainfall isopluvials for the study area as shown by Figure 2.4.95. This parameter is used to correct for elevation-related rainfall variations when constructing sub-basin storm hyetographs.

- The distance of each sub-basin from the center of the design storm(s) is used to determine the depth-area (depth-distance) correction when constructing sub-basin storm hyetographs. These distances were measured

from the middle of the sub-basin with the storm center to the middle of each other sub-basin. Distances to other storm locations are given only if the sub-basin contributed to the flows at the sites requiring alternative storm locations.

• Runoff parameters of time of concentration, storage coefficient, infiltration loss, and base flow are all required for input to the HEC-1 model to define runoff from each sub-basin. Times of concentration were computed using the formula presented in Section 3.2.2 together with the slope and length characteristics of each sub-basin. The storage coefficient for the Clark unit hydrograph for each sub-basin was computed as 25 times the time of concentration. Infiltration rates were assumed to be constant at 3 mm/hour for all sub-basins except for (1) the perched S7, P7, and G19 sub-basins for which an artificially high 1,000 mm/hour loss rate was applied to ensure no rainfall excess from the basin; (2) T7 (Lake Mapanuepe) for which no infiltration loss was assumed; and (3) 0.8 km^2 of A2 and 2.6 km^2 of A4 which were assumed impervious due to the effects of urbanization. The high loss rate was applied to the perched sub-basins to reflect high but unquantified losses expected through highly permeable perched channel reaches. Base flows from each sub-basin were estimated on the basis of average annual sub-basin rainfall, area, and the formula presented in Section 3.2.2.

Design Storms. The 2-, 10-, 50-, 100-, and 500-year storms, developed as described in Section 2.4.5, were used for the HEC-1 modeling.

Effects of Eruption. Due to lack of justification for changes to any other parameters such as loss rate or storage coefficient, eruption impacts reflected in the HEC-1 models of post-eruption basins were limited to (1) physical changes in basin areas, (2) the formation of Lake Mapanuepe, and (3) perched channel reaches on the Bamban, Potrero, and Gumain Rivers.

3.2.5 Confidence Limits on Computed Frequency Events. The 5 and 95 percent confidence limits on the HEC-1 computed instantaneous peaks, maximum 1-day volumes, and maximum 3-day volumes were estimated as follows:

1) The 5 and 95 percent confidence limits were obtained on all streamflow gage data sets on which frequency analyses were conducted (see Section 2.4.4). These confidence limits were obtained in accordance with guidelines described in *Bulletin 17B* of the U.S. Water Resources Council.

2) For each streamflow gage data set, the change in percent from the expected value to the 5 and 95 percent confidence limits were calculated and averaged. The average percentage changes obtained from the streamflow gage data sets were applied to the HEC-1 generated instantaneous peaks, maximum 1-day volumes, and maximum 3-day volumes to obtain the 5 and 95 percent limits.

3.3 Flow Duration Curve Modeling Methodology

3.3.1 Flow Duration Curve. Flow duration curves at gaged locations were presented in Section 2.4.2 of the regional analyses portion of this appendix. Computed flow duration curves at hydrologic simulation output sites were computed on the basis of the following flow data and plotting positions:

1) Average annual flow above each simulation output site was estimated from the normal annual precipitation (NAP) map of the study area. Rainfall shown by that map for the mountain watershed areas had been estimated using a water balance approach to match observed average flows from the watersheds. Thus, for the mountain watersheds, the NAP map is functionally equivalent, after correction for evapotranspiration, to a map of mean annual runoff (and flows) over the study area.

Conversion from average annual basin rainfall shown by the NAP map to average annual flows for specific basins in the vicinity of Mount Pinatubo was accomplished with the following approximate formula derived from data at gaged basins:

$$Q_{avg} = A\,(R/31{,}500 - 0.05)$$

where:

Q_{avg} = Average annual discharge, m^3/s
A = Basin area, km^2
R = Average annual rainfall over basin, mm/yr. In deriving the formula (and also the NAP map), average annual basin rainfall was assumed to be equal to average annual runoff plus 1,550 mm for evapotranspiration losses.

Normalized daily flow duration curves from gaged basins indicate that the average annual flows are exceeded from 15 to 40 percent of the time, averaging about 25 percent (see Figure 2.4.52). For the computed flow duration curves, the average annual flow was plotted to be exceeded about 25 percent of the time.

2) The 2-year 24-hour (average) flow was computed by HEC-1 simulations. The 2-year flow is expected to occur once every two years, on average, based on frequency characteristics of an annual series which considers only the single highest data point for each year of record. However, flow duration curves assume a partial duration series which consider all data points within each year. According to Kite[12], the 2-year return period from an annual series is equivalent to a 1.44-year return period in a partial duration series. For the computed flow duration curves, the 2-year 24-hour (average) flow was plotted to be exceeded about 0.19 percent of the time, corresponding to one day in 1.44 years.

3) The 10-year 24-hour (average) flow was computed by HEC-1 simulations. The 10-year flow is expected to occur once every 10 years, on average. No adjustment was made for the minor difference in partial duration vs. annual series annual return period, 9.5 years vs. 10 years. For the computed flow duration curves, the 10-year 24-hour (average) flow was plotted to be exceeded about 0.027 percent of the time, corresponding to one day in 10 years.

Flows and plotting positions were interpolated between and extrapolated beyond the three points described above considering the shape of flow duration curves for gaged streams in the study area, as shown by Figures 2.4.52 and 2.4.53.

3.3.2 Confidence Limits on Flow Duration Curves. The 5 and 95 percent confidence limits on the flow duration curves were estimated as follows:

1) The 5 and 95 percent confidence limits of the 2- and 10-year 24-hour (average) flow and the mean annual flow were obtained on all streamflow gage data sets on which frequency analyses were conducted (see Section 2.4.4). These confidence limits were obtained in accordance with guidelines described in *Bulletin 17B* of the U.S. Water Resources Council.

2) For each streamflow gage data set, the change in percent from the expected value to the 5 and 95 percent confidence limits were calculated and averaged. The average percentage changes obtained from the streamflow gage data sets were applied to the HEC-1 generated 2- and 10-year 24-hour (average) flow and to the mean annual flow that was obtained as described in Section 3.3.1. This resulted in the 5 and 95 percent confidence limit on the 2-year 24-hour flow, 10-year 24-hour flow, and average annual flow. The 5 and 95 percent confidence limit flows thus

[12] Kite, G.W., 1977, *Frequency and Risk Analysis in Hydrology*, Water Resources Publications.

obtained were used to develop the 5 and 95 percent confidence limit flow duration curves using the same methodology described in Section 3.3.1.

3.4 River Hydraulic Modeling Methodology

Flow depths and velocities were required for the hydraulic design of mitigation measures in seven of the eight major basins. These depths and velocities were obtained through use of the HEC-2, Water Surface Profile Model, normal depth calculations, and critical depth calculations. Channel and cross-section variations, transport of sediment, bed and bank roughness, and spill resistance, all of which create turbulence and energy losses, tend to increase *with increasing discharge*. These energy losses result in flow conditions on steep natural streams that may approach but do not exceed critical flow except in very localized areas of the channel.[13] On reaches that modeling results indicated were of supercritical slope, velocities were obtained from a supercritical flow analysis and depths from a subcritical flow analysis. The velocities and depths thus obtained were, in most cases, approximately equal to the critical depth and velocity because most of these reaches were near a critical slope. On reaches that modeling results indicated were of subcritical slope, velocities and depths were obtained from a subcritical flow analysis. A Mannings' "n" value of 0.25 was used for all calculations.

Cross-sectional data for both HEC-2 modeling and normal/critical depth calculations were obtained from a number of sources. In a few instances, detailed surveyed cross-sectional data were available. In many instances, cross-sections were obtained from a digital terrain model (DTM). At some locations that lacked adequate surveyed or DTM data, the data required for input to the HEC-2 model were obtained from aerial photographs, channel centerline profiles, and professional judgement from team members that observed the site of interest. At some locations no depths and velocities were obtained for existing conditions because adequate data could not be obtained.

3.5 Hydrologic Results

3.5.1 Unit Hydrographs. Table 3.5.1 presents the peak discharge of the unit hydrograph for each sub-basin. The unit hydrograph peaks correspond to a rainstorm of one hour duration with a total rain depth of 1 mm. Figure 3.5.1 shows a typical unit hydrograph computed by the HEC-1 program. Most unit hydrographs are very similar to the shape typified in Figure 3.5.1. The exceptions are presented in Figures 3.5.2 and 3.5.3. These plots for sub-basins A2 and A4 show composite unit hydrographs representing the combination of the SCS unit hydrograph (with no storage coefficient) applied for effectively impervious areas and the Clark unit hydrograph applied for non-impervious areas. These two figures dramatically illustrate both the peak flow increases

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[13] Jarrett, R.D., November 1984, *Hydraulics of High-Gradient Streams*, Journal of Hydraulic Engineering, Vol 110, No. 11, ASCE.

expected from urban development in the Pinatubo region, and the effect of a large storage coefficient on the shape of a unit hydrograph.

3.5.2 Sacobia-Bamban Basin. The Sacobia-Bamban basin is 146 km^2 in area, extending northeasterly from the base of Mount Pinatubo to the interior lowlands of the island of Luzon. Plate 1 shows the location of the basin relative to the study area and hydrometeorological data stations. Figure 3.5.4 shows the basin at a larger scale.

The basin headwater area consists of steep and narrow parallel valleys drained by the Sacobia, Sapang-Cauayan, Marimla, and Malago Rivers. Of these, only the Sacobia and Malago extend to near the base of Mount Pinatubo; the other rivers originate at lower elevations down the mountain's northeastern slope. The Bamban River begins at the confluence of the Sacobia and Marimla Rivers about 25 km northeast of the crater rim, just upstream of the Highway 3 road crossing near the village of Bamban.

Elevations for the Sacobia River and other headwater tributaries range from about 1,100 meters in the headwaters of the Sacobia and Malago Rivers to 55 meters at the confluence defining the start of the Bamban River above the Highway 3 crossing. The Bamban River component of the basin is relatively flat, dropping only about 23 meters over its 12 km long reach. Most of the Bamban River is contained within a diked channel section which is now perched above the surrounding topography. Perching of the Bamban River is a consequence of significant aggradation resulting from the June 1991 eruption. This condition presumably leads to a net water loss resulting from percolation through the channel bed and under the levee system. For modeling purposes, all event rainfall for sub-basin S7 was assumed to be lost to infiltration (i.e., the infiltration rate was set to an arbitrarily high value greater than the maximum rainfall rate).

The 1:50,000 scale maps dated 1986 indicate no significant population centers or urban development within the basin. Clark AFB is located immediately to the south of the basin, but does not affect basin hydrology.

The Sacobia-Bamban basin, located on the northeast slopes of Mount Pinatubo, is in Pinatubo's rain shadow during the Southwest Monsoon or rainy season. Annual rainfall amounts over the basin vary from a maximum of about 4,000 mm/yr in the upper headwater areas near the summit of Mount Pinatubo to a minimum of about 1,800 mm/yr at the downstream end of the basin in the interior lowlands of the island of Luzon. Similar variations in rainfall over the basin are expected during single storm events. Figure 2.3.4 shows isohyets of average annual precipitation over the study area.

Sub-Basin and Output Site Parameters. Physical and computed parameters of sub-basins within the Sacobia-Bamban River Basin are summarized in Table 3.5.2. Physical and computed parameters at simulation output sites in the Sacobia-Bamban River Basin are summarized in Table 3.5.3. As discussed in Section 2.4.5, the flow at a

given site was obtained by centering the storm over the contributing sub-basin that has the highest rainfall (usually the highest elevation headwater sub-basin). Because the sites listed in Table 3.5.3 do not all share common headwaters, the storms had to be centered over three sub-basins in order to obtain flows at all sites. The column in Table 3.5.3 labeled "Critical Storm Location" identifies the Sacobia-Bamban sub-basin over which the storm was centered to obtain flows at the indicated site.

Design Flood Hydrographs. Design flood hydrographs computed by the HEC-1 model at Sacobia-Bamban simulation output sites for each of the 2-, 10-, 50-, 100-, and 500-year return period hypothetical storms are shown by Figures 3.5.5 through 3.5.11. The hydrographs presented correspond to model(s) with the storm(s) centered over the sub-basin location identified by Table 3.5.3 for maximum flows at each site. Two flood hydrographs computed prior to the October 1993 change in basin configuration are shown in Exhibit A (see Section 2.2.2).

Design Discharge/Volume Frequency Curves. Peak discharge and maximum 24-hour and 3-day flow volume data from each of the 2-, 10-, 50-, 100-, and 500-year design flood hydrographs are summarized by Table 3.5.4.

Flow Duration Curves. Daily flow duration curve data for the Sacobia-Bamban simulation output sites, computed following the method described in Section 3.3, are summarized by Table 3.5.5. The data shown for site S7DS do not account for seepage losses during low-flow periods from the perched upstream channel, and probably over-estimate the low-flow characteristics. Exhibit A presents data at two sites that were computed prior to the October 1993 change in basin configuration (see Section 2.2.2).

3.5.3 Abacan River Basin. The Abacan basin is 51 km^2 in area, originating about 4 km east of the crater rim of Mount Pinatubo and extending easterly to the interior lowlands of the island of Luzon. Plate 1 shows the location of the basin relative to the study area and hydrometeorological data stations. Figure 3.5.12 shows the basin at a larger scale. The basin headwater area consists of two steep and narrow parallel valleys drained by the Abacan River and one major tributary, Sapang-Bayo Creek. The basin headwaters originate on Mount Pinatubo's eastern slope at elevations about 1,000 meters below the crater rim. Sapang-Bayo Creek joins the Abacan River about 4 km upstream of the Highway 3 crossing and about 2 km south of Clark Air Base. The lower portion of the basin below Highway 3 is mostly confined within dikes.

Elevations for Abacan River/Sapang-Bayo Creek range from about 500 meters in the upper headwater areas to 130 meters at Sapang-Bayo/Abacan confluence to 10 meters at the end of the dike-confined channel section. Unlike the Bamban River, the dike-confined channel section is not perched above the surrounding landscape.

The 1:50,000 scale maps dated 1986 indicate significant urban development and "densely built up" areas in the Abacan basin in the vicinity of Clark AFB and Angeles City.

Portions of Clark AFB's runways and hangers extend into the basin and also presumably drain to the Abacan River. The extent of urban development relative to the basin size is believed to be sufficient to have a noticeable impact on basin hydrology.

The Abacan basin, located on the eastern slopes of Mount Pinatubo, is in Pinatubo's rain shadow during the Southwest Monsoon or rainy season. Annual rainfall amounts over the basin vary from a maximum of about 3,000 mm/yr in the upper headwater areas on the slopes of Mount Pinatubo to a minimum of about 1,800 mm/yr at the downstream end of the basin in the interior lowlands of the island of Luzon. Similar variations in rainfall over the basin are expected during single storm events. Figure 2.3.4 shows isohyets of average annual precipitation over the study area.

Sub-Basin and Output Site Parameters. Physical and computed parameters of sub-basins within the Abacan River Basin are summarized in Table 3.5.6. Physical and computed parameters at simulation output sites in the Abacan River Basin are summarized in Table 3.5.7. As discussed in Section 2.4.5, the flow at a given site was obtained by centering the storm over the contributing sub-basin that has the highest rainfall (usually the highest elevation headwater sub-basin). Because the sites listed in Table 3.5.7 all share common headwaters, the storms had to be centered over only one sub-basin in order to obtain flows at all sites. The column in Table 3.5.7 labeled "Critical Storm Location" identifies the Abacan sub-basin (A1) over which the storm was centered.

Design Flood Hydrographs. Design flood hydrographs computed by the HEC-1 model at Abacan basin simulation output sites for each of the 2-, 10-, 50-, 100-, and 500-year return period hypothetical storms are shown by Figures 3.5.13 through 3.5.15.

Design Discharge/Volume Frequency Curves. Peak discharge and maximum 24-hour and 3-day flow volume data from each of the 2-, 10-, 50-, 100-, and 500-year design flood hydrographs are summarized by Table 3.5.8.

Flow Duration Curves. Daily flow duration curve data for each of the Abacan basin simulation output sites, computed following the method described in Section 3.3, are summarized by Table 3.5.9.

3.5.4 O'Donnell Basin. The study area considered herein for the O'Donnell basin includes two major rivers, the O'Donnell and the Bulsa. The O'Donnell River drains the northern slopes of Mount Pinatubo, and has a basin area upstream of the confluence with the Bulsa of about 266 km^2. The Bulsa River primarily drains the eastern slopes of the Zambales mountains, and has a basin area upstream of the confluence with the O'Donnell of about 510 km^2. The entire basin extends about 2 km below the O'Donnell-Bulsa confluence and has a total area of about 817 km^2. It is the largest of all basins being assessed under the present work.

Plate 1 shows the location of the basin relative to the study area and hydrometeorological data stations. Figure 3.5.16 shows the basin at a larger scale.

Basin headwater areas on Mount Pinatubo consist of steep and narrow parallel valleys drained primarily by the O'Donnell, Apalong, and Bangat Rivers. Of these three tributaries, only the O'Donnell sub-basin extends fully to Pinatubo's crater rim where the post-eruption elevation is about 1,200 meters. The Apalong and Bangat Rivers originate from a secondary peak on Mount Pinatubo which, with a pre-eruption summit elevation of about 1,500 meters, may now be the highest point on the mountain.

Basin headwater areas for the Bulsa River on the east slopes of the Zambales Mountains reach a maximum elevation of about 1,600 meters. These headwater areas include numerous steep and narrow stream-cut valleys which seem generally less entrenched into the mountain slopes than those on Pinatubo.

The stream elevation at the confluence of the Bulsa and O'Donnell Rivers, located near the downstream end of the basin study area, is about 40 meters.

The 1:50,000 scale maps dated 1986 indicate several small population centers (O'Donnell, Santa Lucia, Moriones), but none of sufficient size or scale to appreciably affect the hydrology at sites of interest within the basin.

The O'Donnell basin, which generally drains in a northeasterly direction, is in the rain shadow of the Zambales Mountains and, to a lesser extent, of Mount Pinatubo, during the Southwest Monsoon or rainy season. Annual rainfall amounts over the basin vary from a maximum of about 6,000 mm/yr in the upper headwater areas of the Bulsa River in the Zambales Mountains, and 5,000 mm/yr in the upper headwater areas of O'Donnell River tributaries draining Mount Pinatubo, to a minimum of about 1,800 mm/yr at the downstream end of the basin in the interior lowlands of the island of Luzon. Similar variations in rainfall over the basin are expected during single storm events. Figure 2.3.4 shows isohyets of average annual precipitation over the study area.

Sub-Basin and Output Site Parameters. Physical and computed parameters of sub-basins within the O'Donnell River Basin are summarized in Table 3.5.10. Physical and computed parameters at simulation output sites in the O'Donnell River Basin are summarized in Table 3.5.11. As discussed in Section 2.4.5, the flow at a given site was obtained by centering the storm over the contributing sub-basin that has the highest rainfall (usually the highest elevation headwater sub-basin). Because the sites listed in Table 3.5.11 do not all share common headwaters, the storms had to be centered over three sub-basins in order to obtain flows at all sites. The column in Table 3.5.11 labeled "Critical Storm Location" identifies the O'Donnell sub-basin over which the storm was centered to obtain flows at the indicated site.

Design Flood Hydrographs. Design flood hydrographs computed by the HEC-1 model at basin simulation output sites for each of the 2-, 10-, 50-, 100-, and 500-year return period hypothetical storms are shown by Figures 3.5.17 through 3.5.27.

Design Discharge/Volume Frequency Curves. Peak discharge and maximum 24-hour and 3-day flow volume data from each of the 2-, 10-, 50-, 100-, and 500-year design flood hydrographs are summarized by Table 3.5.12.

Flow Duration Curves. Daily flow duration curve data for each of the O'Donnell basin simulation output sites, computed following the method described in Section 3.3, are summarized by Table 3.5.13.

3.5.5 Gumain/Porac Basin. The Gumain/Porac basin is 302 km^2 in area, extending in a generally southeasterly direction from Mount Pinatubo to the Pampanga Delta. Plate 1 shows the location of the basin relative to the study area and hydrometeorological data stations. Figure 3.5.28 shows the basin at a larger scale.

The Gumain/Porac basin includes two major rivers, the Gumain and the Porac. The headwaters of the Gumain River consist of steep, well-incised tributaries originating near the crater rim of Mount Pinatubo and along the ridge which extends south from Mount Pinatubo, separating the Gumain/Porac basin from the westerly-flowing Santo Tomas tributaries. The Gumain River flows approximately 32 km southeast from the crater rim of Mount Pinatubo to its confluence with the Porac River at the head of the Gumain Floodway. Elevations within the basin range from about 1,600 meters on the ridge line approximately 4 km south of Mount Pinatubo and about 1,200 meters at the crater rim to about 10 meters at the head of the Gumain Floodway.

The headwaters of the Porac River originate on the southeast slopes of Mount Pinatubo, approximately 5 km southeast of the crater rim. The river flows east and then south some 39 km to its confluence with the Gumain River at the head of the Gumain Floodway. Elevations within the Porac basin range from 1,150 meters at the high point to 10 meters at the head of the Gumain Floodway.

The lower reaches of the Gumain and Porac Rivers contain a number of major irrigation and flood control projects, including the Gumain Floodway. One major aspect of these projects was the diversion of the Porac River into the Gumain Floodway system; the Porac's natural course appears to be in a channel which flows about 4 km north of the floodway. However, it has not been possible to obtain information on the exact configuration and operating policies for these projects or their current (post-eruption) condition. For purposes of hydrologic modeling of post-eruption conditions, it was assumed that flood flows from both the Gumain and Porac Rivers will be directed to and confined within the Gumain Floodway.

The Gumain Floodway begins at the confluence of the Gumain and Porac Rivers and continues downstream approximately 8 km to its outlet in the Pampanga Delta at an approximate elevation of 5 meters. The floodway, represented as sub-basin G19, has aggraded significantly since the eruption and is now perched above the surrounding landscape. This condition presumably leads to a net water loss resulting from percolation through the channel bed and under the levee system. For modeling purposes, all event rainfall for sub-basin G19 was assumed to be lost to infiltration (i.e, the infiltration rate was set to an arbitrarily high value greater than the maximum rainfall rate).

The drainage area delineated during this study for the Gumain/Porac basin at the downstream end of the Gumain Floodway is 302 km^2. This is less than the 370 km^2 basin area previously published for a stream gage located along the floodway. The previously published value is believed to have included areas which can no longer drain to the lower Gumain because of extreme channel aggradation.

The 1:50,000 scale maps dated 1986 indicate several small population centers (i.e. Pabanlag, Del Carmen, and Santa Rita), but none are of sufficient size to appreciably affect the hydrology at sites of interest within the basin.

Annual rainfall amounts over the basin vary from a maximum of about 5,000 mm/yr in the Gumain River headwater region on Mount Pinatubo to 2,000 mm/yr at the downstream end of the basin at the western edge of the Pampanga Delta. Similar variations in rainfall over the basin are expected during single storm events. Figure 2.3.4 shows isohyets of average annual precipitation over the study area.

Sub-Basin and Output Site Parameters. Physical and computed parameters of sub-basins within the Gumain/Porac River Basin are summarized in Table 3.5.14. Physical and computed parameters at simulation output sites in the Gumain/Porac River Basin are summarized in Table 3.5.15. As discussed in Section 2.4.5, the flow at a given site was obtained by centering the storm over the contributing sub-basin that has the highest rainfall (usually the highest elevation headwater sub-basin). Because the sites listed in Table 3.5.15 do not all share common headwaters, the storms had to be centered over two sub-basins in order to obtain flows at all sites. The column in Table 3.5.15 labeled "Critical Storm Location" identifies the Gumain/Porac sub-basin over which the storm was centered to obtain flows at the indicated site.

Design Flood Hydrographs. Design flood hydrographs computed by the HEC-1 model at basin simulation output sites for each of the 2-, 10-, 50-, 100-, and 500-year return period hypothetical storms are shown in Figures 3.5.29 through 3.5.35.

Design Discharge/Volume Frequency Curves. Peak discharge and maximum 24-hour and 3-day flow volume data from each of the 2-, 10-, 50-, 100-, and 500-year design flood hydrographs are summarized in Table 3.5.16.

Flow Duration Curves. Daily flow duration curve data for each of the Gumain/Porac basin simulation output sites, computed following the method described in Section 3.3, are summarized in Table 3.5.17. The data shown for site G19US do not account for seepage losses from the perched upstream channel during low-flow periods and probably over-estimate the low-flow characteristics.

3.5.6 Pasig-Potrero Basin. The Pasig-Potrero basin is 77 km^2 in area, originating at the Mount Pinatubo crater rim and extending first in an easterly direction and then, further downstream, in a southeasterly direction to the Pampanga Delta. Plate 1 shows the location of the basin relative to the study area and hydrometeorological stations. Figure 3.5.36 shows the basin at a larger scale.

The basin headwater area is drained by five streams: the Bucbuc, Yangca, Timbu, and Papatac Rivers, and a stream that prior to the eruption was the uppermost headwater stream of the Sacobia River. The Papatac River is formed at the confluence of the Bucbuc and Yangca. The Pasig River is formed at the confluence of the Papatac and Timbu Rivers. Below the former site of the Mancatian Bridge, the Pasig's name changes to Potrero.

Elevations in the basin range from about 1,200 meters near the crater rim to near zero at the confluence of the Potrero River with the Guagua River. The Potrero River component, which comprises almost half of the basin length, is relatively flat, dropping about 100 meters over its 18 km length.

The 1:50,000 scale maps dated 1986 do not indicate any urban development that would significantly affect basin hydrology. The town of Bacolor near the downstream end of the basin has been affected by shallow flooding from the Potrero River, but is not considered to lie in the basin because the Potrero River is isolated from the surrounding topography by levees and because the channel from about 4-1/2 km below the former site of the Mancatian Bridge to the confluence with the Guagua River is perched above the surrounding topography.

The Pasig-Potrero basin, located on the eastern slopes of Mount Pinatubo, is in Pinatubo's rain shadow during the Southwest Monsoon or rainy season. Annual rainfall amounts vary from a maximum of about 5,000 mm/yr in the upper headwater areas on the slopes of Mount Pinatubo to a minimum of about 1,800 mm/yr at the downstream end of the basin in the Pampanga Delta. Similar variations in rainfall over the basin are expected during single storm events. Figure 2.3.4 shows isohyets of average annual precipitation over the study area.

Sub-Basin and Output Site Parameters. Physical and computed parameters of sub-basins and at simulation output sites in the Pasig-Potrero basin are summarized in Tables 3.5.18 and 3.5.19, respectively. As discussed in Section 2.4.5, the flow at a given site was obtained by centering the storm over the contributing sub-basin that has the highest rainfall (usually the highest elevation headwater sub-basin). Because the sites listed in Table 3.5.19 do not all share common headwaters, the storms had to be centered over two sub-basins in order to obtain flows at all sites. The column in Table 3.5.19 labeled "Critical Storm Location" identifies the Pasig-Potrero sub-basin over which the storm was centered to obtain flows at the indicated site.

Design Flood Hydrographs. Design flood hydrographs computed by the HEC-1 model at Pasig-Potrero simulation output sites for each of the 2-, 10-, 50-, 100-, and 500-year return period hypothetical storms are shown by Figures 3.5.37 through 3.5.45. Two flood hydrographs computed prior to the October 1993 change in basin configuration are shown in Exhibit A (see Section 2.2.2).

Design Discharge/Volume Frequency Curves. Peak discharge and maximum 24-hour and 3-day flow volume data from each of the 2-, 10-, 50-, 100-, and 500-year design flood hydrographs are summarized in Table 3.5.20.

Flow Duration Curves. Daily flow duration curve data for the Pasig-Potrero simulation sites, computed following the method described in Section 3.3, are summarized in Table 3.5.21. The data shown for site P7DS do not account for seepage losses from the perched upstream channel during low-flow periods and probably over-estimate the low-flow characteristics. Exhibit A presents data at two sites that were computed prior to the October 1993 change in basin configuration (see Section 2.2.2).

3.5.7 Santo Tomas Basin. The Santo Tomas basin is approximately 262 km^2 in area, extending in a southwesterly direction from Mount Pinatubo to the South China Sea. Plate 1 shows the location of the basin relative to the study area and hydrometeorological data stations. Figure 3.5.46 shows the basin at a larger scale.

The Santo Tomas River system incorporates two major tributaries, the Marella River and the Mapanuepe River, which join to form the Santo Tomas. The headwaters of the Marella River originate near the crater rim of Mount Pinatubo at an elevation of about 1,500 meters and along the ridge extending south from Mount Pinatubo which separates the Santo Tomas basin from the easterly flowing Gumain River tributaries. The Marella River drains the southwest slopes of Mount Pinatubo and combines with the Mapanuepe River at an elevation of about 90 meters. The reach length from the confluence of the Marella and Mapanuepe Rivers to the crater rim is about 28 km. The headwaters of the Mapanuepe River originate near the divide between the Santo Tomas and Gumain basins at an elevation of around 1,000 meters. The Mapanuepe River sub-basin includes a large mine site, a mine tailings dam, and Lake Mapanuepe. Approximately 4.2 km^2 of the mine site does not contribute surface runoff to the

watershed and hence was not included in the hydrologic model. The impoundment behind the tailings dam is small in comparison to Lake Mapanuepe and no flow routing was done through this facility.

Lake Mapanuepe was formed following the June 1991 eruption of Mount Pinatubo as a result of blockage of the Mapanuepe River outlet by recurrent lahars and severe aggradation on the Marella. Under current conditions, the Mapanuepe River joins the Marella River approximately 1.5 km downstream from the outlet of Lake Mapanuepe. The surface area of Lake Mapanuepe at the invert elevation of its current outlet, 121.6 meters, is about 8.0 km^2; topographic contours show that the lake surface area would be about 17 km^2 at a water level elevation of 140 meters. Lake storage volumes between these two elevations were computed using the conic method for reservoir volumes as described in the HEC-1 manual. The following stage/discharge characteristics were used for modeling purposes:

Lake Elevation (m)	Outlet Discharge (m^3/s)
< 121.6	0
121.6 to 129.6	$23.57d[10d/(10+2d)]^{0.667}$
< 129.6	$2.36a\{a/[26+(2.08)(d-8)]\}^{0.667}$

where: d = overflow depth = lake elevation - 121.6
a = flow area = 80 + (7.78+0.28d)(d-8)

The Santo Tomas River begins at the Marella-Mapanuepe confluence. The Santo Tomas is joined by the Santa Fe River approximately 10 km downstream from gage W094A. The Santo Tomas then flows a further 12 km through coastal lowlands to Highway 7, and an additional 1 km to the South China Sea.

The 1:50,000 scale maps dated 1986 indicate several small population centers (San Rafael, Dalanaon and Aglao), but none are of sufficient size or scale to appreciably affect the hydrology at sites of interest within the basin.

Annual rainfall amounts over the basin vary from a maximum of about 5,000 mm/yr in the Marella River headwater region on Mount Pinatubo to a minimum of about 3,600 mm/yr at the downstream end of the basin near the coastline. Similar variations in rainfall over the basin are expected during single storm events. Figure 2.3.4 shows isohyets of average annual precipitation over the study area.

Sub-Basin and Output Site Parameters. Physical and computed parameters of sub-basins within the Santo Tomas River Basin are summarized in Table 3.5.22. Physical and computed parameters at simulation output sites in the Santo Tomas River Basin are summarized in Table 3.5.23. As discussed in Section 2.4.5, the flow at a given site was obtained by centering the storm over the contributing sub-basin that has the highest rainfall (usually the highest elevation headwater sub-basin). Because the sites listed in Table 3.5.23 do not all share common headwaters, the storms had to be centered over two sub-basins in order to obtain flows at all sites. The column in Table 3.5.23 labeled "Critical Storm Location" identifies the Santo Tomas sub-basin over which the storm was centered to obtain flows at the indicated site.

Design Flood Hydrographs. Design flood hydrographs computed by the HEC-1 model at basin simulation output sites for each of the 2-, 10-, 50-, 100-, and 500-year return period hypothetical storms are shown by Figures 3.5.47 through 3.5.51.

Design Discharge/Volume Frequency Curves. Peak discharge and maximum 24-hour and 3-day flow volume data from each of the 2-, 10-, 50-, 100-, and 500-year design flood hydrographs are summarized by Table 3.5.24.

Flow Duration Curves. Daily flow duration curve data for each of the Santo Tomas basin simulation output sites, computed following the method described in Section 3.3, are summarized by Table 3.5.25.

3.5.8 Bucao Basin. The Bucao basin is 656 km^2 in area, extending in a generally northwesterly direction from Mount Pinatubo and southwesterly from the Zambales Mountains to the South China Sea. Plate 1 shows the location of the basin relative to the study area and hydrometeorological data stations. Figure 3.5.52 shows the basin at a larger scale.

The Bucao basin incorporates the Bucao River and its two major tributaries, the Balin-Buquero River and the Balintawak River. The central portion of the basin includes a large area of relatively flat and low-lying terrain nestled between the mountains which define the basin perimeter: Mount Pinatubo, the Zambales Mountains, and the coastal mountains located between Mount Pinatubo and the South China Sea.

The headwaters of the Bucao River originate on the northwest slopes of Mount Pinatubo 2 to 5 km north of the crater rim at an elevation of about 900 meters. The river flows in a generally westerly direction through rugged terrain for approximately 28 km to its confluence with the Balintawak River at an elevation of about 50 meters. The Bucao then enters a broad flat valley and continues to flow west approximately 4 km to its confluence with the Balin-Buquero and a further 12 km to the Highway 7 crossing. The Bucao enters the South China Sea approximately 2 km below Highway 7.

The headwaters of the Balin-Buquero River originate to the south of the Bucao River headwater areas and extend to the crater rim of Mount Pinatubo at an elevation of about 1,500 meters. The Balin-Buquero and its principal tributaries (such as the Maronut) drain the western slopes of Mount Pinatubo and the northeastern slopes of the coastal mountain range which lies between Mount Pinatubo and the South China Sea. The Balin-Buquero flows in a generally northwesterly direction for approximately 20 km from the crater rim of Mount Pinatubo to its confluence with the Maronut River at an elevation of about 90 meters. Below the confluence with the Maronut, the Balin-Buquero enters a broad flat valley and continues to flow northwest for a further 12 km to its confluence with the Bucao at an elevation of about 40 meters. The drainage area of the Balin-Buquero above its confluence with the Bucao is approximately 217 km^2.

The headwaters of the Balintawak River originate to the north of the Bucao River headwater areas and drain the southern slopes of the Zambales Mountains at elevations of up to 1,670 meters. The Balintawak River flows in a generally southwesterly direction through rugged terrain for approximately 20 km to its confluence with the Bucao River at an elevation of 90 meters. The drainage area of the Balintawak upstream of its confluence with the Bucao is approximately 166 km^2.

The headwater areas of the Bucao and Balin-Buquero Rivers were severely disturbed by the June 1991 eruption of Mount Pinatubo, with massive deposits of pyroclastic material filling in entire river channels and destroying much of the pre-eruption drainage system. Post-eruption changes to drainage boundaries occurred to most of the Bucao and Balin-Buquero headwater drainages. The Maronut River was the most affected by the eruption, with a reduction in catchment area from 31 km^2 to 11 km^2 as a result of gross changes in catchment topography.

The 1:50,000 scale maps dated 1986 indicate several small population centers (San Juan, Poonbato and Maguiguis), but none are of sufficient size or scale to appreciably affect the hydrology at sites of interest within the basin.

Annual rainfall amounts over the basin vary from a maximum of about 6,000 mm/yr in the Zambales Mountains and 5,000 mm/yr in upper headwaters on Mount Pinatubo to 3,800 mm/yr in the coastal lowlands. Similar variations in rainfall over the basin are expected during single storm events. Figure 2.3.4 shows isohyets of average annual precipitation over the study area.

Sub-Basin and Output Site Parameters. Physical and computed parameters of sub-basins within the Bucao River Basin are summarized in Table 3.5.26. Physical and computed parameters at simulation output sites in the Bucao River Basin are summarized in Table 3.5.27. As discussed in Section 2.4.5, the flow at a given site was obtained by centering the storm over the contributing sub-basin that has the highest rainfall (usually the highest elevation headwater sub-basin). Because the sites listed in Table 3.5.27 do not all share common headwaters, the storms had to be centered over six

sub-basins in order to obtain flows at all sites. The column in Table 3.5.27 labeled "Critical Storm Location" identifies the Bucao sub-basin over which the storm was centered to obtain flows at the indicated site.

Design Flood Hydrographs. Design flood hydrographs computed by the HEC-1 model at simulation output sites for each of the 2-, 10-, 50-, 100-, and 500-year return period hypothetical storms are shown by Figures 3.5.53 through 3.5.63.

Design Discharge/Volume Frequency Curves. Peak discharge and maximum 24-hour and 3-day flow volume data from each of the 2-, 10-, 50-, 100-, and 500-year design flood hydrographs are summarized by Table 3.5.28.

Flow Duration Curves. Daily flow duration curve data for each of the Bucao basin simulation output sites, computed following the method described in Section 3.3, are summarized by Table 3.5.29.

3.5.9 Maloma Basin. The Maloma basin is 150 km^2 in area, originating about 7 km southwest of Mount Pinatubo and extending in a westerly direction to the South China Sea. Plate 1 shows the location of the basin relative to the study area and hydrometeorological data stations. Figure 3.5.63 shows the basin at a larger scale.

The Maloma basin includes two major rivers, the Maloma River and the Gorongoro/Kakilingar River, which join before discharging into the South China Sea. The basin primarily drains the coastal mountains to the west of Mount Pinatubo; drainage of Mount Pinatubo itself is limited to the extreme eastern headwaters of the Maloma River which extend to the lower southwest slopes of Mount Pinatubo at an elevation of only about 600 meters.

The Maloma River flows west from Mount Pinatubo in a narrow canyon through the coastal mountain range which lies between Mount Pinatubo and the South China Sea. It is then joined by the Gorongoro/Kakilingar River about 6 km upstream of the Highway 7 bridge at an elevation of less than 10 meters. Elevations within the Maloma basin range from sea level to about 1,000 meters, with the highest elevations occurring within the coastal mountains.

The Gorongoro/Kakilingar River originates entirely from the coastal mountains to the west of Mount Pinatubo, and flows westward in a deep narrow valley through the coastal mountains. Elevations within the Gorongoro/Kakilingar catchment range from about 800 meters in the upper headwater areas to less than 10 meters at the confluence with the Maloma.

The 1:50,000 scale maps dated 1986 and 1991 indicate several small population centers (Payodpod, Maquineng, and Maloma), but none are of sufficient size to appreciably affect the hydrology at sites of interest within the basin.

Annual rainfall amounts over the basin vary from a maximum of about 5,000 mm/yr in the coastal mountains to a minimum of about 4,000 mm/yr in the low lying area between Mount Pinatubo and the coastal mountains, and along the coast near the South China Sea. Similar variations in rainfall over the basin are expected during single storm events. Figure 2.3.4 shows isohyets of average annual precipitation over the study area.

Sub-Basin and Output Site Parameters. Physical and computed parameters of sub-basins within the Maloma River Basin are summarized in Table 3.5.30. Physical and computed parameters at simulation output sites in the Maloma River Basin are summarized in Table 3.5.31. As discussed in Section 2.4.5, the flow at a given site was obtained by centering the storm over the contributing sub-basin that has the highest rainfall (usually the highest elevation headwater sub-basin). Because the sites listed in Table 3.5.31 do not all share common headwaters, the storms had to be centered over three sub-basins in order to obtain flows at all sites. The column in Table 3.5.31 labeled "Critical Storm Location" identifies the Maloma sub-basin over which the storm was centered to obtain flows at the indicated site.

Design Flood Hydrographs. Design flood hydrographs computed by the HEC-1 model at basin simulation output sites for each of the 2-, 10-, 50-, 100-, and 500-year return period hypothetical storms are shown by Figures 3.5.65 through 3.5.68.

Design Discharge/Volume Frequency Curves. Peak discharge and maximum 24-hour and 3-day flow volume data from each of the 2-, 10-, 50-, 100-, and 500-year design flood hydrographs are summarized by Table 3.5.32.

Flow Duration Curves. Daily flow duration curve data for each of the Maloma basin simulation output sites, computed following the method described in Section 3.3, are summarized by Table 3.5.33.

3.5.10 Confidence Limits.

Frequency Events. Table 3.5.34 presents the 2-, 10-, 50-, 100-, and 500-year peak discharges estimated with the use of the HEC-2 model and the 5 and 95 percent confidence limits about these peak discharges at each hydrologic site considered in this study. On Figure 3.5.69 are plotted the HEC-1 estimated peak discharges for the 2- through 500-year events and the 5 and 95 percent confidence limits about these peaks at a typical hydrologic site (Site O9US in the O'Donnell River basin).

Table 3.5.35 presents the 2-, 10-, 50-, 100-, and 500-year maximum 24-hour volumes estimated with the use of the HEC-2 model and the 5 and 95 percent confidence limits about these volumes at each hydrologic site considered in this study. On Figure 3.5.70 are plotted the HEC-1 estimated peak discharges for the 2- through 500-year events and the 5 and 95 percent confidence limits about these peaks at a typical hydrologic site (Site O9US in the O'Donnell River basin).

Table 3.5.36 presents the 2-, 10-, 50-, 100-, and 500-year maximum 3-day volumes estimated with the use of the HEC-2 model and the 5 and 95 percent confidence limits about these volumes at each hydrologic site considered in this study. On Figure 3.5.71 are plotted the HEC-1 estimated peak discharges for the 2- through 500-year events and the 5 and 95 percent confidence limits about these peaks at a typical hydrologic site (Site O9US in the O'Donnell River basin).

Flow Duration Curves. Table 3.5.37 presents the data for the 5 percent confidence limit about the flow duration curve at each hydrologic site considered in this study. Table 3.5.38 presents the data for the 95 percent confidence limit about these same flow duration curves. On Figure 3.5.72 are plotted the computed flow duration curve and the 5 and 95 percent confidence limit about the computed curve at a typical hydrologic site (Site S7DS in the Sacobia-Bamban River basin).

3.6 River Hydraulic Modeling Results

Table 3.6.1 presents clearwater flow depths. Table 3.6.2 presents bulked flow (i.e., sediment + water flow) depths.[14] In Table 3.6.3 are clearwater flow velocities. Table 3.6.4 presents bulked flow velocities. Notes contained in Table 3.6.5 provide information on each reach that is useful for the correct interpretation of the data in Tables 3.6.1 through 3.6.4.

Figures 3.6.1 through 3.6.11 present the results of the hydraulic modeling of clearwater flows at bridges located in the reaches indicated on Tables 3.6.1 through 3.6.5. Figures 3.6.12 through 3.6.22 present the results at these same bridges but for flows that have been increased for suspended sediment.

3.7 HEC-1 Input

Enclosure 1 provides a listing of the HEC-1 input used for the modeling of the 100-year event in the Sacobia-Bamban River basin. Storms were centered over three different sub-basins of the Sacobia-Bamban Basin in order to obtain flow estimates at all required sites (see Section 3.5.2, Sub-Basin and Output Site Parameters). Therefore, Enclosure 1 includes three separate HEC-1 input files, one for each of the three assumed storm centers. HEC-1 input for the 2-, 10-, 50-, and 500-year events on the Sacobia-Bamban Basin are identical to Enclosure 1 except the values on the PI cards, which represent incremental storm precipitation, are different.

The HEC-1 input for other basins was similar to the input provided in Enclosure 1.

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[14] Technical Appendix B, Sedimentation, contains information on flow bulking for suspended sediment.

ENCLOSURE, PLATE, AND EXHIBIT A FOR

TECHNICAL APPENDIX A

HYDROLOGY AND HYDRAULICS

ENCLOSURE 1

HEC-1 INPUT FOR THE 100-YEAR EVENT ON THE PASIG-POTRERO BASIN:

```
ID PASIG-POTRERO RIVER: 100-YEAR EVENT
ID POST-ERUPTION CONDITIONS
ID US ARMY CORPS OF ENGINEERS
*FREE
*DIAGRAM
IM
IT 60 01JAN00 0100 241
IO 1
IN 60
*
*************************************************************
******************
*  100-Year Storm, 5-Day Event:
*  Ratio of Storm Volume from Isopluvial Maps (Sub-Basin to Summit) : .55
*  Distance from Sub-Basin to Assumed Storm Center : .0 km
*  Distance Assumed for Depth-Area Correction: 0.92 km
*
*************************************************************
*******************
PG PT
PI    4.20   4.20   4.20   4.20   4.20   4.20   4.20   4.20   4.20
PI    4.20   4.20   4.20   4.20   4.20   4.20   4.20   4.20   4.20
PI    4.20   4.20   4.20   4.20   4.20   4.20   4.20   4.20   4.20
PI    4.20   4.20   4.20   4.20   4.20   4.20   4.20   4.20   4.20
PI    4.20   4.20   4.20   4.20   4.20   4.20   4.20   6.48   6.48
PI    6.48   6.48   6.48   6.48   6.48   6.48   6.48   6.48   6.48
PI    6.48   6.48   6.48   6.48   6.48   6.45   6.45   6.45  24.93
PI   24.93  24.93  41.55  41.55  41.55  76.39  51.52  43.93  24.93
PI   24.93  24.93   6.45   6.45   6.45   6.45   6.45   6.45   6.45
PI    6.45   6.45   6.48   6.48   6.48   6.48   6.48   6.48   6.48
PI    6.48   4.20   4.20   4.20   4.20   4.20   4.20   4.20   4.20
PI    4.20   4.20   4.20   4.20   4.20   4.20   4.20   4.20   4.20
PI    4.20   4.20   4.20   4.20   4.20   4.20   4.20   4.20   4.20
PI    4.20   4.20   4.20
*
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*************************************************************
*******************
*  100-Year Storm, 5-Day Event:
*  Ratio of Storm Volume from Isopluvial Maps (Sub-Basin to Summit) : .93
*  Distance from Sub-Basin to Assumed Storm Center : .0 km
*  Distance Assumed for Depth-Area Correction: 1.7 km
*
*************************************************************
********************
PG P0
PI     7.37    7.37    7.37    7.37    7.37    7.37    7.37    7.37    7.37
PI     7.37    7.37    7.37    7.37    7.37    7.37    7.37    7.37    7.37
PI     7.37    7.37    7.37    7.37    7.37    7.37    7.37    7.37    7.37
PI     7.37    7.37    7.37    7.37    7.37    7.37    7.37    7.37    7.37
PI     7.37    7.37    7.37    7.37    7.37    7.37    7.37   11.34   11.34
PI    11.34   11.34   11.34   11.34   11.34   11.34   11.34   11.34   11.34
PI    11.34   11.34   11.34   11.34   11.34   11.26   11.26   11.26   43.48
PI    43.48   43.48   74.75   74.75   74.75   82.17   77.19   76.12   43.48
PI    43.48   43.48   11.26   11.26   11.26   11.26   11.26   11.26   11.26
PI    11.26   11.26   11.34   11.34   11.34   11.34   11.34   11.34   11.34
PI    11.34    7.37    7.37    7.37    7.37    7.37    7.37    7.37    7.37
PI     7.37    7.37    7.37    7.37    7.37    7.37    7.37    7.37    7.37
PI     7.37    7.37    7.37    7.37    7.37    7.37    7.37    7.37    7.37
PI     7.37    7.37    7.37
*
*************************************************************
*******************
*  100-Year Storm, 5-Day Event:
*  Ratio of Storm Volume from Isopluvial Maps (Sub-Basin to Summit) : .70
*  Distance from Sub-Basin to Assumed Storm Center : 4.2 km
*
*************************************************************
*******************
PG P1
PI     5.40    5.40    5.40    5.40    5.40    5.40    5.40    5.40    5.40
PI     5.40    5.40    5.40    5.40    5.40    5.40    5.40    5.40    5.40
PI     5.40    5.40    5.40    5.40    5.40    5.40    5.40    5.40    5.40
PI     5.40    5.40    5.40    5.40    5.40    5.40    5.40    5.40    5.40
PI     5.40    5.40    5.40    5.40    5.40    5.40    5.40    8.34    8.34
PI     8.34    8.34    8.34    8.34    8.34    8.34    8.34    8.34    8.34
```

```
PI     8.34   8.34   8.34   8.34   8.34   8.14   8.14   8.14  31.44
PI    31.44  31.44  52.60  52.60  52.60  78.53  59.60  53.54  31.44
PI    31.44  31.44   8.14   8.14   8.14   8.14   8.14   8.14   8.14
PI     8.14   8.14   8.34   8.34   8.34   8.34   8.34   8.34   8.34
PI     8.34   5.40   5.40   5.40   5.40   5.40   5.40   5.40   5.40
PI     5.40   5.40   5.40   5.40   5.40   5.40   5.40   5.40   5.40
PI     5.40   5.40   5.40   5.40   5.40   5.40   5.40   5.40   5.40
PI     5.40   5.40   5.40
*
*************************************************************
*******************
*  100-Year Storm, 5-Day Event:
*  Ratio of Storm Volume from Isopluvial Maps (Sub-Basin to Summit) : .65
*  Distance from Sub-Basin to Assumed Storm Center : 6.70 km
*
*************************************************************
*******************
PG P2
PI     4.93   4.93   4.93   4.93   4.93   4.93   4.93   4.93   4.93
PI     4.93   4.93   4.93   4.93   4.93   4.93   4.93   4.93   4.93
PI     4.93   4.93   4.93   4.93   4.93   4.93   4.93   4.93   4.93
PI     4.93   4.93   4.93   4.93   4.93   4.93   4.93   4.93   4.93
PI     4.93   4.93   4.93   4.93   4.93   4.93   4.93   7.74   7.74
PI     7.74   7.74   7.74   7.74   7.74   7.74   7.74   7.74   7.74
PI     7.74   7.74   7.74   7.74   7.74   7.50   7.50   7.50  28.95
PI    28.95  28.95  43.69  43.69  43.69  74.17  53.38  46.75  28.95
PI    28.95  28.95   7.50   7.50   7.50   7.50   7.50   7.50   7.50
PI     7.50   7.50   7.74   7.74   7.74   7.74   7.74   7.74   7.74
PI     7.74   4.93   4.93   4.93   4.93   4.93   4.93   4.93   4.93
PI     4.93   4.93   4.93   4.93   4.93   4.93   4.93   4.93   4.93
PI     4.93   4.93   4.93   4.93   4.93   4.93   4.93   4.93   4.93
PI     4.93   4.93   4.93
*
*************************************************************
*******************
*  100-Year Storm, 5-Day Event:
*  Ratio of Storm Volume from Isopluvial Maps (Sub-Basin to Summit) : .55
*  Distance from Sub-Basin to Assumed Storm Center : 8.7 km
*
*************************************************************
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*******************
PG P3
PI     4.11    4.11    4.11    4.11    4.11    4.11    4.11    4.11    4.11
PI     4.11    4.11    4.11    4.11    4.11    4.11    4.11    4.11    4.11
PI     4.11    4.11    4.11    4.11    4.11    4.11    4.11    4.11    4.11
PI     4.11    4.11    4.11    4.11    4.11    4.11    4.11    4.11    4.11
PI     4.11    4.11    4.11    4.11    4.11    4.11    4.11    6.50    6.50
PI     6.50    6.50    6.50    6.50    6.50    6.50    6.50    6.50    6.50
PI     6.50    6.50    6.50    6.50    6.50    6.35    6.35    6.35   24.52
PI    24.52   24.52   35.41   35.41   35.41   65.10   43.91   37.44   24.52
PI    24.52   24.52    6.35    6.35    6.35    6.35    6.35    6.35    6.35
PI     6.35    6.35    6.50    6.50    6.50    6.50    6.50    6.50    6.50
PI     6.50    4.11    4.11    4.11    4.11    4.11    4.11    4.11    4.11
PI     4.11    4.11    4.11    4.11    4.11    4.11    4.11    4.11    4.11
PI     4.11    4.11    4.11    4.11    4.11    4.11    4.11    4.11    4.11
PI     4.11    4.11    4.11
*
*****************************************************************
*******************
*  100-Year Storm, 5-Day Event:
*  Ratio of Storm Volume from Isopluvial Maps (Sub-Basin to Summit) : .50
*  Distance from Sub-Basin to Assumed Storm Center : 9.7 km
*
*****************************************************************
*******************
PG P4
PI     3.74    3.74    3.74    3.74    3.74    3.74    3.74    3.74    3.74
PI     3.74    3.74    3.74    3.74    3.74    3.74    3.74    3.74    3.74
PI     3.74    3.74    3.74    3.74    3.74    3.74    3.74    3.74    3.74
PI     3.74    3.74    3.74    3.74    3.74    3.74    3.74    3.74    3.74
PI     3.74    3.74    3.74    3.74    3.74    3.74    3.74    5.92    5.92
PI     5.92    5.92    5.92    5.92    5.92    5.92    5.92    5.92    5.92
PI     5.92    5.92    5.92    5.92    5.92    5.79    5.79    5.79   22.37
PI    22.37   22.37   31.99   31.99   31.99   57.64   39.50   33.35   22.37
PI    22.37   22.37    5.79    5.79    5.79    5.79    5.79    5.79    5.79
PI     5.79    5.79    5.92    5.92    5.92    5.92    5.92    5.92    5.92
PI     5.92    3.74    3.74    3.74    3.74    3.74    3.74    3.74    3.74
PI     3.74    3.74    3.74    3.74    3.74    3.74    3.74    3.74    3.74
PI     3.74    3.74    3.74    3.74    3.74    3.74    3.74    3.74    3.74
PI     3.74    3.74    3.74
```

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*
*************************************************************
*******************
*  100-Year Storm, 5-Day Event:
*  Ratio of Storm Volume from Isopluvial Maps (Sub-Basin to Summit) : .42
*  Distance from Sub-Basin to Assumed Storm Center : 15.0 km
*
*************************************************************
*******************
PG P5
PI     3.12   3.12   3.12   3.12   3.12   3.12   3.12   3.12   3.12
PI     3.12   3.12   3.12   3.12   3.12   3.12   3.12   3.12   3.12
PI     3.12   3.12   3.12   3.12   3.12   3.12   3.12   3.12   3.12
PI     3.12   3.12   3.12   3.12   3.12   3.12   3.12   3.12   3.12
PI     3.12   3.12   3.12   3.12   3.12   3.12   3.12   5.01   5.01
PI     5.01   5.01   5.01   5.01   5.01   5.01   5.01   5.01   5.01
PI     5.01   5.01   5.01   5.01   5.01   5.01   5.01   5.01  19.34
PI    19.34  19.34  24.37  24.37  24.37  44.86  30.77  25.88  19.34
PI    19.34  19.34   5.01   5.01   5.01   5.01   5.01   5.01   5.01
PI     5.01   5.01   5.01   5.01   5.01   5.01   5.01   5.01   5.01
PI     5.01   3.12   3.12   3.12   3.12   3.12   3.12   3.12   3.12
PI     3.12   3.12   3.12   3.12   3.12   3.12   3.12   3.12   3.12
PI     3.12   3.12   3.12   3.12   3.12   3.12   3.12   3.12   3.12
PI     3.12   3.12   3.12
*
*************************************************************
*******************
*  100-Year Storm, 5-Day Event:
*  Ratio of Storm Volume from Isopluvial Maps (Sub-Basin to Summit) : .40
*  Distance from Sub-Basin to Assumed Storm Center : 20.8 km
*
*************************************************************
*******************
PG P6
PI     2.97   2.97   2.97   2.97   2.97   2.97   2.97   2.97   2.97
PI     2.97   2.97   2.97   2.97   2.97   2.97   2.97   2.97   2.97
PI     2.97   2.97   2.97   2.97   2.97   2.97   2.97   2.97   2.97
PI     2.97   2.97   2.97   2.97   2.97   2.97   2.97   2.97   2.97
PI     2.97   2.97   2.97   2.97   2.97   2.97   2.97   4.79   4.79
PI     4.79   4.79   4.79   4.79   4.79   4.79   4.79   4.79   4.79
```

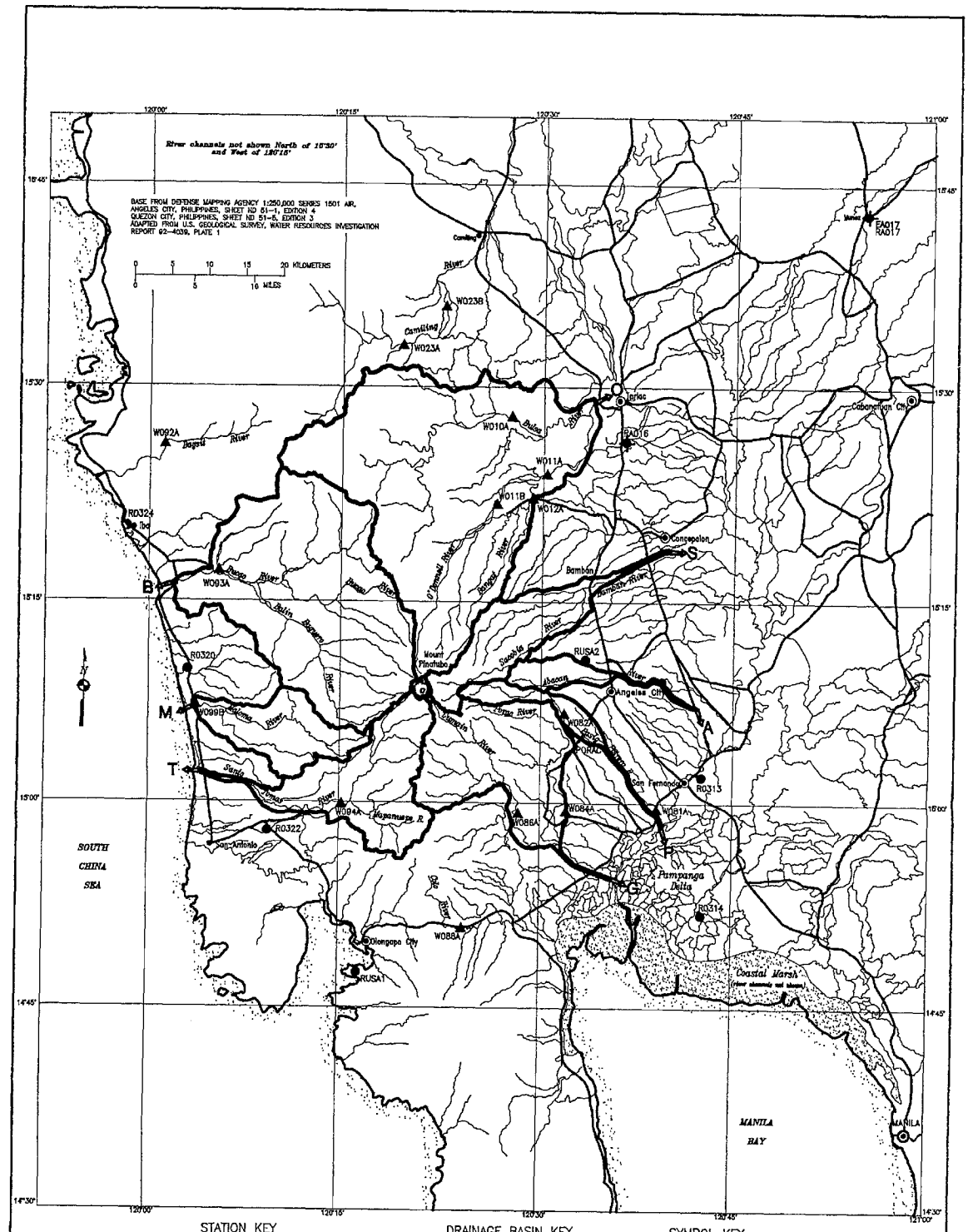
```
PI     4.79   4.79   4.79   4.79   4.79   4.81   4.81   4.81  18.57
PI    18.57  18.57  21.85  21.85  21.85  41.92  28.17  24.11  18.57
PI    18.57  18.57   4.81   4.81   4.81   4.81   4.81   4.81   4.81
PI     4.81   4.81   4.79   4.79   4.79   4.79   4.79   4.79   4.79
PI     4.79   2.97   2.97   2.97   2.97   2.97   2.97   2.97   2.97
PI     2.97   2.97   2.97   2.97   2.97   2.97   2.97   2.97   2.97
PI     2.97   2.97   2.97   2.97   2.97   2.97   2.97   2.97   2.97
PI     2.97   2.97   2.97
*    *********
*
*************************************************************
*******************
*  100-Year Storm, 5-Day Event:
*  Ratio of Storm Volume from Isopluvial Maps (Sub-Basin to Summit) : .39
*  Distance from Sub-Basin to Assumed Storm Center : 28 km
*
*************************************************************
*******************
PG P7
PI     2.88   2.88   2.88   2.88   2.88   2.88   2.88   2.88   2.88
PI     2.88   2.88   2.88   2.88   2.88   2.88   2.88   2.88   2.88
PI     2.88   2.88   2.88   2.88   2.88   2.88   2.88   2.88   2.88
PI     2.88   2.88   2.88   2.88   2.88   2.88   2.88   2.88   2.88
PI     2.88   2.88   2.88   2.88   2.88   2.88   2.88   4.67   4.67
PI     4.67   4.67   4.67   4.67   4.67   4.67   4.67   4.67   4.67
PI     4.67   4.67   4.67   4.67   4.67   4.71   4.71   4.71  18.18
PI    18.18  18.18  22.95  22.95  22.95  40.36  27.96  23.21  18.18
PI    18.18  18.18   4.71   4.71   4.71   4.71   4.71   4.71   4.71
PI     4.71   4.71   4.67   4.67   4.67   4.67   4.67   4.67   4.67
PI     4.67   2.88   2.88   2.88   2.88   2.88   2.88   2.88   2.88
PI     2.88   2.88   2.88   2.88   2.88   2.88   2.88   2.88   2.88
PI     2.88   2.88   2.88   2.88   2.88   2.88   2.88   2.88   2.88
PI     2.88   2.88   2.88
*  *********
*  KM For the following hydrographs: Stations that begin with a "P"
*  KM (e.g., P1DS, P5US, etc.) are the total hydrograph at the
*  KM indicated location but may (on the upper sub-basins) also be
*  KM a sub-basin runoff hydrograph.  Stations that begin with an "RF"
*  KM are sub-basin runoff hydrographs for the indicated area and do not
*  KM represent a hydrograph at any location.  Stations that begin with
```

```
*  KM an "RT" are the resulting hydrograph after being routed from the
*  KM indicated location to the next downstream location and do not
*  KM represent a hydrograph at any location.
* KM
*  KM Two storms were applied over P3 (Timbu). One to obtain the
*  KM hydrograph at P3DS, the other for use in obtaining hydrographs
*  KM at other locations since a storm centered over P0 gives greater
* KM flow at these other locations than a storm centered over P3
*      **********
KK P3DS
KM Hydrograph for P3DS
BA 6.0
BF 0.3
PR PT
LU 0,3
ZW A=PINATUBO_DESIGN B=PASIG P3DS C=FLOW F=100-YR COMPUTED
UC 1.0,25.5
*      ************
KK P0DS
KM Hydrograph calculation for P0DS.
BA 21.3
BF 3.0
PR P0
LU 0,3
ZW A=PINATUBO_DESIGN B=PASIG P0DS C=FLOW F=100-YR COMPUTED
UC 0.7,17.5
*      *************
KK RTP0DS
KM Muskingum routing of P0DS to P1DS.
RM 1,.7,.2
*      ********
KK RFP1
KM Sub-basin runoff calculation for P1DS.
BA 9.3
BF 0.7
PR P1
LU 0,3
UC .8,20.8
*      **********
KK P1DS
```

```
KM Hydrograph calculation for P1DS.
ZW A=PINATUBO_DESIGN B=PASIG P1DS C=FLOW F=100-YR COMPUTED
HC 2
*      **********
KK P2DS
KM Hydrograph calculation for P2DS.
BA 4.4
BF 0.3
PR P2
LU 0,3
ZW A=PINATUBO_DESIGN B=PASIG P2DS C=FLOW F=100-YR COMPUTED
UC .4,11.0
*      *********
KK P4US
KM Hydrograph calculation for P4US.
ZW A=PINATUBO_DESIGN B=PASIG P4US C=FLOW F=100-YR COMPUTED
HC 2
*      *********
KK RTP4US
KM Muskingum routing from P4US to P4DS
RM 1,.4,.2
*      *********
KK RFP4
KM Sub-basin runoff calculation for P4.
BA 3.1
BF 0.1
PR P4
LU 0,3
UC .9,23.5
*      *********
KK P4DS
KM Hydrograph calculation for P4DS.
ZW A=PINATUBO_DESIGN B=PASIG P4DS C=FLOW F=100-YR COMPUTED
HC 2
*      **********
KK P3DS
KM Hyd. calc. for P3DS for use at pts. other than P3DS
BA 6.0
BF 0.3
PR P3
```

```
LU 0,3
UC 1.0,25.5
*       **********
KK P5US
KM Hydrograph calculation for P5US.
ZW A=PINATUBO_DESIGN B=PASIG P5US C=FLOW F=100-YR COMPUTED
HC 2
*       **********
KK RTP5US
KM Muskingum routing from P5US to P5DS
RM 1,1.1,.2
*       **********
KK RFP5
KM Sub-basin runoff calculation for P5.
BA 12.6
BF 0.0
PR P5
LU 0,3
UC 1.9,48.3
*       **********
KK P5DS
KM Hydrograph calculation for P5DS.
ZW A=PINATUBO_DESIGN B=PASIG P5DS C=FLOW F=100-YR COMPUTED
HC 2
*       **********
KK RTP5DS
KM Muskingum routing from P5DS to P6DS.
RM 1,.5,.2
*       **********
KK RFP6
KM Sub-basin runoff calculation for P6.
BA 17.7
BF 0.0
PR P6
LU 0,3
UC 2.2,55.
*       **********
KK P6DS
KM Hydrograph calculation for P6DS
HC 2
```

```
*      **********
KK RTP6DS
KM Muskingum routing from P6DS to P7DS.
RM 2,1.56,.2
*      *********
KK RFP7
KM Sub-basin runoff calculation for P7.
BA 2.4
BF 0.0
PR P7
LU 0,1000
UC 4.82,121
*      ********
KK P7DS
KM Hydrograph calculation for P7DS
ZW A=PINATUBO_DESIGN B=PASIG P7DS C=FLOW F=100-YR COMPUTED
HC 2
*      ********
ZZ
```



STATION KEY

Daily Rainfall Data

RA016 Hacienda Luisita, Tarlac
RA017 CLSU, Munoz, Nueva Ecija
RD324 Iba, Zambales
RO313 Julian Subd, San Fernando, Pampanga
RO314 Masantol, Pampanga
RO320 Sta Rita Elem Sch, Cabangan, Zambales
RO322 Zhoa, San Marcelino, Zambales
RUSA1 Cubi Point Naval Air Station
RUSA2 Clark Air Force Base

Hourly Rainfall Data

RA016 Hacienda Luisita, Tarlac
RD324 Iba, Zambales
PORAC Santa Cruz, Porac, Pampanga
(Limited hourly data available)

Daily Evaporation Data

EA017 CLSU, Munoz, Nueva Ecija

Streamflow Data

W010A Bulog R.
W011A O'Donnell R., Petablub
W011B O'Donnell R., Potting
W012A Bangat R.
W023A Camiling R., Namsalon
W023B Camiling R., Poblacion
W081A Pasig–Potrero R., Cabetiaan
W082A Pasig–Potrero R., Hda Dalorna
W084A Porac R., Del Carmen
W086A Gumain R.
W088A Colo R.
W092A Bagsit R.
W093A Bucao R.
W094A Sto. Tomas R.
W099B Maloma R.

DRAINAGE BASIN KEY

(listed clockwise from north)

O O'Donnell
S Sacobia – Bamban
A Abacan
P Pasig – Potrero
G Gumain / Porac
T Santa Tomas
M Maloma
B Bucao

SYMBOL KEY

- ▲ Streamflow data
- ● Daily rainfall data
- ◑ Daily and hourly rainfall data
- ○ Hourly rainfall only
- ✦ Daily rainfall and evaporation data
- ⇨ Basin outlet

northwest hydraulic consultants Kent, Washington	Portland District U.S. Army Corps of Engineers Portland, Oregon

Locations of Drainage Basins and Hydrometeorological Data Stations

SUBMITTED	DESIGNED	DRAWN	CHECKED
CADD FILE: 20682.3.1.1	DATE: June 1993		PLATE 1

TECHNICAL APPENDIX A

EXHIBIT A
DATA PRIOR TO THE OCTOBER 1993 CHANGE
IN PASIG/SACOBIA BASIN CONFIGURATION

A headwater basin of the Sacobia-Bamban River (formerly identified as sub-basin S1) was captured by the Pasig-Potrero River in October of 1993 and is now assigned a sub-basin identifier of P0. In the Sacobia-Bamban Basin, this change in basin configuration reduces the runoff through sub-basins S2, S3, and S4 thereby reducing the runoff at simulation output sites S2DS, S3DS, S3DS+S4DS, S7US, and S7DS. In the Pasig-Potrero Basin, this change in basin configuration increases the runoff through sub-basins P1, P4, P5, P6, and P7 thereby increasing the runoff at simulation output sites P1DS, P4US, P4DS, P5US, P5DS, and P7DS. Runoff was not affected through sub-basins or at simulation output sites not listed above.

Figures A-1 through A-4 are flood hydrographs at two simulation output sites on the Sacobia-Bamban River and two simulation output sites on the Pasig-Potrero River that were affected by the change in basin configuration. These hydrographs were computed for conditions that existed prior to the October 1993 capture of a Sacobia-Bamban headwater basin by the Pasig-Potrero River. Relative changes in the hydrology of these sub-basins and other affected sub-basins can be estimated by comparing Figures A-1 through A-4 to the appropriate post-October 1993 figures described in the text of Technical Appendix A (Figures 3.5.6, 3.5.11, 3.5.43, and 3.5.45).

Table A-1 shows flow duration curves at the same simulation output sites indicated on Figures A-1 through A-4 computed for conditions that existed prior to the October 1993 capture of a Sacobia-Bamban headwater basin by the Pasig-Potrero River. Relative changes in the hydrology of these sub-basins and other affected sub-basins can be estimated by comparing the data in Table A-1 to the appropriate post-October 1993 data in tables described in the text of Technical Appendix A (Tables 3.5.5 and 3.5.21).

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Prior to October 1993 Loss of
Headwater Basin to Pasig - Potrero River
Computed Flood Hydrographs
Sacobia-Bamban Basin at Site S3DS

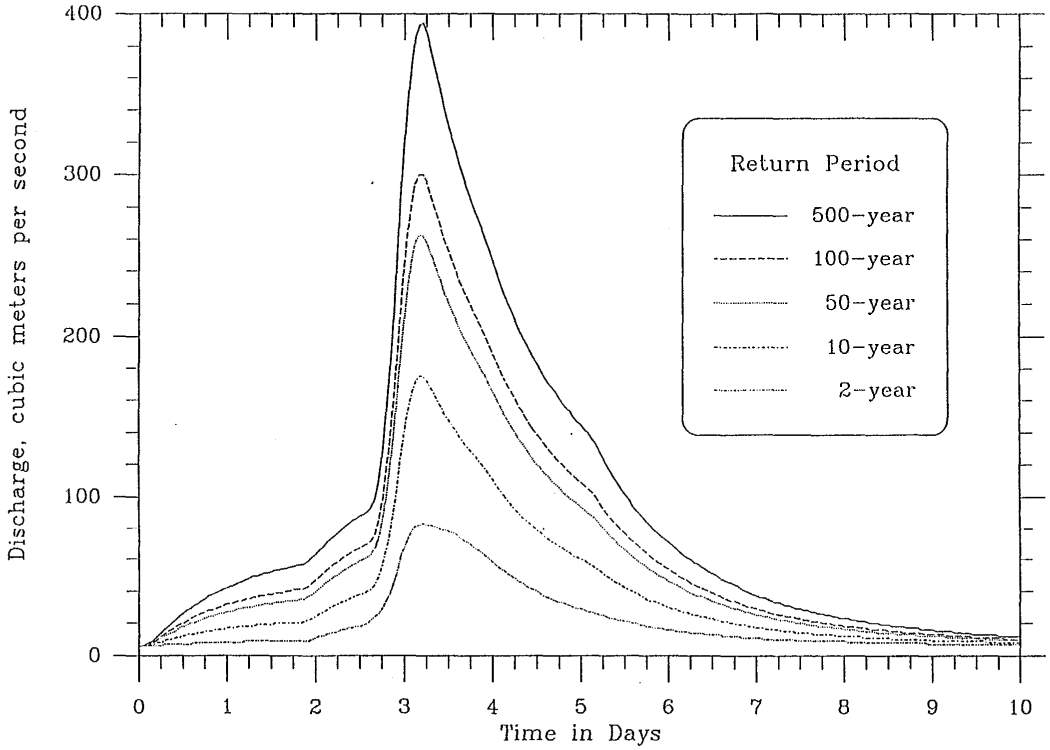

Figure A-1

Prior to October 1993 Loss of
Headwater Basin to Pasig - Potrero River
Computed Flood Hydrographs
Sacobia-Bamban Basin at Site S7DS

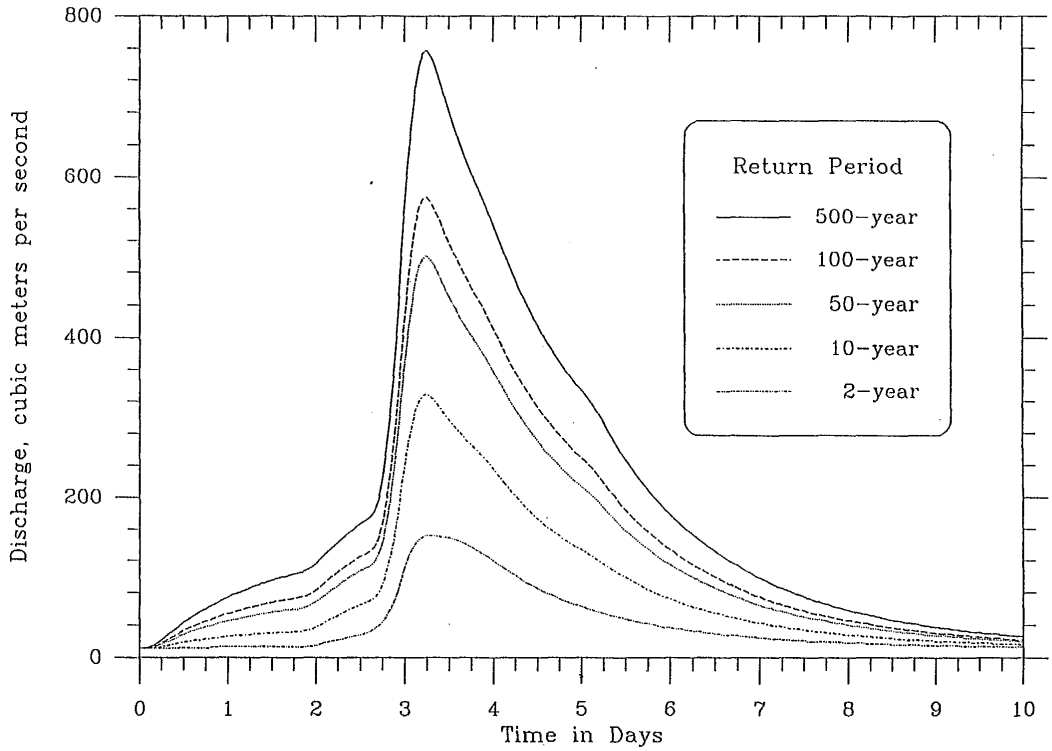

Figure A-2

Prior to October 1993 Capture of Sacobia Headwater Basin

Computed Flood Hydrographs

Pasig-Potrero Basin at Site P5US

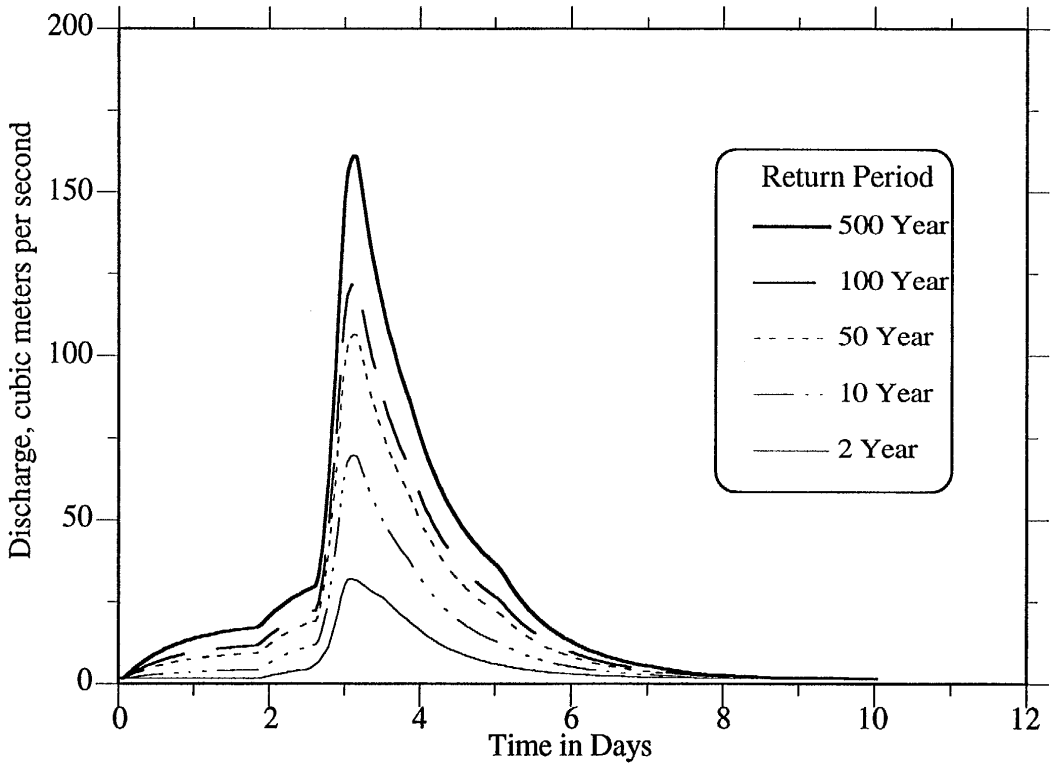

Figure A-3

Prior to October 1993 Capture of Sacobia Headwater Basin

Computed Flood Hydrographs
Pasig-Potrero Basin at Site P7DS

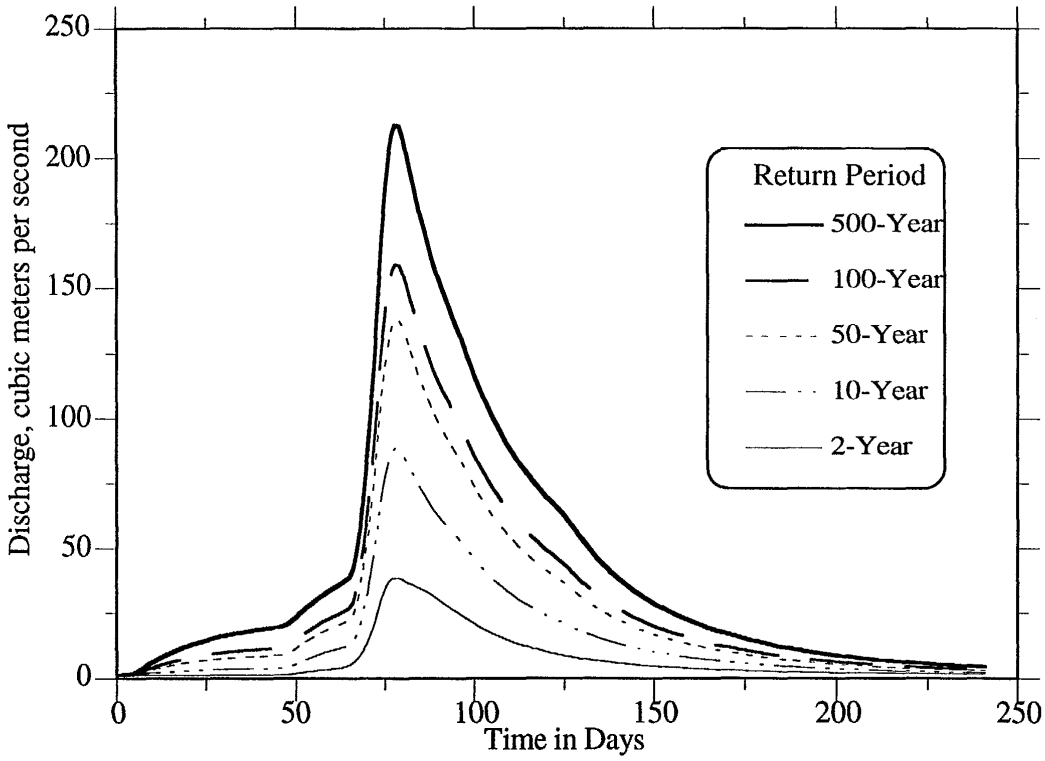

Figure A-4

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Table A-1
Prior to October 1993 Change in Basin Configuration

Sample Computed Flow Duration Curves

Sacobia-Bamban and Pasig-Potrero Basins

Data in cubic meters per second

% OF TIME EXCEEDED	S3DS	S7DS	P5US	P7DS
100	0	0	0	0
50	1.5	3.4	0.5	0.7
25	*	*	1.2	1.7
20	4.8	10.7	1.6	2.2
10	7.9	17.8	2.8	3.7
5	11.8	25.9	5	5.5
2	20.1	43.4	8.8	9.3
1	29.8	63.1	12.6	13.7
0.5	42.4	88.3	17.3	19.4
0.2	68.1	142	26.5	32.1
0.1	91.7	195.4	36.5	45
0.05	115.4	249.3	46.6	58.2
0.02	140.6	305.9	57.2	71.8

* Not calculated.

TABLES FOR

TECHNICAL APPENDIX A

HYDROLOGY AND HYDRAULICS

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Table 1.3.1
Conversion, SI to English Units

Units of Length	mm	m	dam	km	inches (in)	feet (ft)	yard (yd)	miles (mi)
1 millimeter (mm) =	1	0.001	0.0001	(10)^-6	0.0394	0.0033	-	-
1 meter (m) =	1,000	1	0.1	0.001	39.37	3.281	1.094	-
1 decameter (dam) =	10,000	10	1	0.01	393.7	32.81	10.94	-
1 kilometer (km) =	(10)^6	1,000	100	1	39,370	3,281	1,094	0.6214

Units of Area	m^2	km^2	in^2	ft^2	acres	mi^2
1 square meter (m^2) =	1	(10)^-6	1550	10.76	-	-
1 square kilometer (km^2) =	(10)^6	1	-	-	247	0.3861

Units of Volume	m^3	dam^3	ft^3	yd^3	acre-ft
1 cubic meter (m^3) =	1	0.001	35.32	1.307	-
1 cubic decameter (dam^3) =	1,000	1	35,320	1,307	0.8108

Note: Dash mark (-) indicates very small or very large conversion that is not commonly used.

TABLE 2.3.1
Daily Rainfall and Evaporation Data

Station ID	Station Name	Station Location		Available Record		Agency	Parameters Available[1]
		Latitude	Longitude	Start Date	End Date		
D324	Iba, Zambales	15°20' N	119°58' E	1951-01-01	1992-10-31	Pagasa	R
A016	Hacienda Luisita, Tarlac	15°26' N	120°36' E	1968-01-01	1992-09-30	Pagasa	R,E
A017	CLSU, Munoz, Neuva Ecija	15°43' N	120°54' E	1974-01-01	1992-09-30	Pagasa	R,E
A020	Magalang, Pampanga	15°13' N	120°39' E	1988-01-01	1991-09-30	Pagasa	R,E
R0312	Bai Magalang, Pampanga	15°13' N	120°42' E	1977-01-01	1992-09-30	Pagasa	R
R0313	Julian Subd, San Fernando, Pampanga	15°02' N	120°42' E	1970-01-01	1991-12-31	Pagasa	R
R0314	Masantol, Pampanga	14°52' N	120°42' E	1970-01-01	1992-09-30	Pagasa	R
R0315	Camiling, Tarlac	15°42' N	120°24' E	1976-01-01	1990-05-01	Pagasa	R
R0316	Mayantoc, Tarlac	15°36' N	120°21' E	1972-01-01	1990-11-30	Pagasa	R
R0318	Palawig, Zambales	15°26' N	119°54' E	1975-01-01	1991-09-30	Pagasa	R
R0319	San Felipe, Zambales	15°04' N	120°04' E	1975-10-01	1990-06-30	Pagasa	R
R0320	Sta Rita Elem Sch, Cabangan, Zambales	15°10' N	120°03' E	1975-10-01	1992-11-30	Pagasa	R
R0322	Znas, San Marcelino, Zambales	14°58' N	120°09' E	1975-10-01	1992-11-30	Pagasa	R
USA01[2]	Cubi Point Naval Air Station	14°48' N	120°17' E	1958-04-01	1990-12-31	U.S.N.	R
USA02[2]	Clark Air Force Base	15°11' N	120°35' E	1950-01-01	1991-09-30	U.S.A.F.	R

[1] R — daily rainfall; E — daily pan evaporation

[2] For purposes of this study, USA01 and USA02 are station identifiers given to the daily raingauges operated by the U.S. Navy and U.S. Air Force.

TABLE 2.3.2
PHIVOLCS Rainfall Stations

STATION I.D.	STATION NAME	LOCATION FROM MT. PINATUBO	ELEVATION (m)	DATE INSTALLED
RG-1 (201)	Mt. Cuadrado	South	1070	July 1991
RG-2 (202)	BUGZ	Southwest	600	July 1991
RG-3 (203)	PI2	Northeast	640	July 1991
RG-4 (204)	Mt. Culianan	Northwest	550	Aug. 1991
RG-5 (205)	Gumain	Southeast	820	Aug. 1991
RG-6 (206)	Sacobia	East/Northeast	510	Sept. 1991

TABLE 2.3.3
Hourly Rainfall Data

Station ID	Station Name	Station Location		Periods of Data Obtained
		Latitude	Longitude	
D324	Iba, Zambales	15°20' N	119°58' E	1972-07-14 to 1972-08-10 1976-05-17 to 1976-05-31 1980-07-08 to 1980-07-11
A016	Hacienda Luisita, Tarlac	15°26' N	120°36' E	1970-08-28 to 1970-09-06 1972-07-14 to 1972-08-10 1974-08-12 to 1974-08-19
PORAC[1]	Santa Cruz, Porac Pampanga	15°05' N	120°33' E	1970-08-28 to 1970-09-03 1972-07-14 to 1970-08-05 1974-08-12 to 1974-08-19 1976-05-19 to 1976-05-29 1977-11-11 to 1977-11-16

[1] Official Station ID unknown.

TABLE 2.3.4
Cross Correlations of Daily Rainfall Data

	Station ID														
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
1	1.00	0.08	0.08	-0.17	0.10	0.18	0.12	0.06	0.07	0.29	-0.07	0.53	0.31	0.25	0.13
2	0.08	1.00	0.06	-0.20	-0.16	0.37	0.14	0.23	0.08	-0.15	-0.40	0.16	0.04	0.02	0.32
3	0.08	0.06	1.00	-0.30	-0.06	-0.14	-0.15	-0.07	-0.20	-0.24	-0.46	0.05	-0.06	-0.08	-0.13
4	-0.17	-0.20	-0.30	1.00	-0.02	-0.20	0.02	-0.09	-0.27	-0.54	-0.42	-0.16	-0.34	-0.22	-0.17
5	0.10	-0.16	-0.06	-0.02	1.00	-0.22	-0.12	-0.11	-0.24	-0.13	-0.24	0.11	0.06	0.08	-0.16
6	0.18	0.37	-0.14	-0.20	-0.22	1.00	0.38	0.04	0.11	-0.21	-0.30	0.14	0.11	0.20	0.40
7	0.12	0.14	-0.15	0.02	-0.12	0.38	1.00	0.01	-0.04	-0.24	-0.22	0.18	0.11	0.27	0.26
8	0.06	0.23	-0.07	-0.09	-0.11	0.04	0.01	1.00	-0.33	-0.16	-0.32	0.12	0.17	-0.03	0.21
9	0.07	0.08	-0.20	-0.27	-0.24	0.11	-0.04	-0.33	1.00	-0.18	-0.38	0.02	-0.11	0.04	0.07
10	0.29	-0.15	-0.24	-0.54	-0.13	-0.21	-0.24	-0.16	-0.18	1.00	-0.37	0.32	0.21	0.11	-0.06
11	-0.07	-0.40	-0.46	-0.42	-0.24	-0.30	-0.22	-0.32	-0.38	-0.37	1.00	-0.01	-0.06	-0.05	-0.28
12	0.53	0.16	0.05	-0.16	0.11	0.14	0.18	0.12	0.02	0.32	-0.01	1.00	0.42	0.37	0.19
13	0.31	0.04	-0.06	-0.34	0.06	0.11	0.11	0.17	-0.11	0.21	-0.06	0.42	1.00	0.38	0.20
14	0.25	0.02	-0.08	-0.22	0.08	0.20	0.27	-0.03	0.04	0.11	-0.05	0.37	0.38	1.00	0.28
15	0.13	0.32	-0.13	-0.17	-0.16	0.40	0.26	0.21	0.07	-0.06	-0.28	0.19	0.20	0.28	1.00

Station Key

ID	File Name	Station Name
1	D3245192.WAT	Iba, Zambales
2	A0166892.WAT	Hacienda Luisita, Tarlac
3	A0177492.WAT	CLSU, Munoz, Nueva Ecija
4	A0208892.WAT	Magalang, Pampanga
5	R3127792.WAT	Bai Magalang, Pampanga
6	R3137091.WAT	Julian Subd, San Fernando, Pampanga
7	R3147092.WAT	Masantol, Pampanga
8	R3157690.WAT	Camiling, Tarlac
9	R3167290.WAT	Mayantoc, Tarlac
10	R3187591.WAT	Palawig, Zambales
11	R3197590.WAT	San Felipe, Zambales
12	R3207592.WAT	Sta Rita Elem Sch, Cabangan, Zambales
13	R3227592.WAT	Znas, San Marcelino, Zambales
14	USNRPMB.WAT	Cubi Point Naval Air Station
15	CLARK.WAT	Clark Air Force Base

TABLE 2.3.5
Estimated Annual Pan Evaporation (mm)
CLSU, Munoz

Year	Pan Evaporation (mm)
1974	1855
1975	2084
1976	1972
1977	2109
1978	1731
1979	1842
1980	1968
1981	1796
1982	1921
1983	2153
1984	1895
1985	2010
1986	1852
1987	2006
1988	2008
1989	1943
1990	1914
1991	2049
1992	1919

Table 2.3.6
Annual Rainfall, mm

RAIN GAGE	Iba RD324	Luisita RA016	CLSU RA017	Julian R0313	Masantol R0314	Sta Rita R0320	Znas R0322	Cubi RUSA1	Clark RUSA2
YEAR									
1950									2207
1951	3039								1782
1952	4073								1756
1953	2916								2167
1954	2716								1315
1955	1927								1439
1956	3531								1761
1957									1441
1958									1732
1959								2500	1097
1960	5088							3829	2331
1961	5481							3794	2079
1962								3535	1970
1963	3784							3477	2205
1964	8594							3075	1966
1965	3739							3539	2054
1966	3747							3302	2773
1967	5072							4396	2001
1968	1972	1754						2146	1694
1969	3585	1465						2688	1727
1970	4272				1796				2346
1971	2745	1887		1669	1625			2964	2191
1972	4659	3526		3678	3128			4308	4120
1973	3324	1300		1459	1368			2572	1604
1974	4124	2384	2526	2629	2172			4138	2619
1975	2528		2045	1438	1638			2868	1516
1976	4374	2475	2650	2654	2232	4888	4154	4226	2705
1977	3901	1713	1505	1410	1472	4112	4250	3768	1765
1978	5227	2011	1998	2119	2212		5099	5402	2347
1979	3551	1518	1522	1483	1558	4131	3293	4058	1817
1980	3960	1742	1675	1885	2072	3946	2564	2585	1742
1981		1692	1788	1393	1580			3233	1775
1982		1562	1789				3446	3857	1389
1983	2120	1305	1347	985	1685	2467	2260	2566	1034
1984	4137	2107		2126	2428	4277	3025	4758	1920
1985	4119		2211	2264		5154	3561	4944	2391
1986	4024	1983	2292	2312		4930	3983	4612	2313
1987	2562	1171	1337	875	1864	2849		2590	1446
1988	3874		2042	1060	2222	4025	2984	3303	1807
1989		1524	2056	1381	1989	4133	3424	3556	1971
1990	3509		2294	1884	2575	4833	3688	4245	2298
1991	4021		1760	1334	2038	4355	3928		
# OBS	34	18	17	20	19	13	14	31	41
MEAN	3832	1840	1932	1802	1982	4162	3547	3575	1966
STD	1211	536	378	663	426	748	703	801	526

TABLE 2.3.7
Annual Runoff from Basin, mm

RIVER	BULSA	O'DONNEL	O'DONNEL	BANGAT	CAMILING	CAMILING	PASIG-P	PASIG-P	PORAC	PORAC	GUMAIN FW	GUMAIN	CAULAMAN	COLO	BAGSIT	BUCAO	STO TOMAS	MALOMA
DA (km^2)	405	240	112	90	142	280	242	28	118	111	370	128	72	76	68	615	177	151
YEAR																		
1955																		
1956																		
1957						1294				475		2052	3109	1688		3080	1331	
1958						1520				407		2591	2997	2387		3762	3647	
1959			1012			576			205	646	476	912	820	691		1681	1331	
1960			2767						1262	1773	3968	3362		2225		3340	4242	
1961	1918		1906						1138	935	1313	2796		1782		3321	3069	
1962	1896		781			1981				1586	2485	2241	2781		2956	3267	3870	
1963						2217			1891	1084	3192	2131	3627			3605	3628	
1964	2284		1197		3928	1387			1104	1401	2242	2451	2161	1493	4876	1764	1802	
1965	1521		1150		3467	1050			635	1317	678	2163	1956	1937		1583	1269	
1966	2866		1460		3687					2660	3384	3643	2433	2820	2018	3233	2212	
1967	2538			1957	4710			1423		2046	3186	3065		2920	3755	4003	677	
1968	2767	1146		1509	3867		179	1092	380	2434	3640	2065		2307	3050	2670		
1969	1675	1017		1714	3324		130	232	203	2271	1925	1399			3566	1532	201	
1970	2625	882		3008	3988			359	341	1730	3132	2314	1624	3237	3849	2386	258	
1971	1694		1824	2721	3199			885	296	874	3379	2698	645	3910		2349		
1972	5140				2870						2146							
1973											4605							
1974											528							
1975																		
1976																		
1977																		
1978											461							
1979																		
1980																		
1981																		
1982																		
1983																		
1984			2722															5205
1985																		
1986											3939							
1987																		
1988																		
1989											2657							
1990											1047							3823
1991																		
# OBS.	11	3	9	5	9	7	2	5	10	15	20	15	10	12	7	15	13	2
MEAN	2448	1015	1647	2182	3671	1432	154	798	746	1443	2419	2392	2215	2283	3439	2772	2118	4514
STD	963	108	679	582	507	511	25	447	544	688	1270	674	928	824	824	811	1369	691

TABLE 2.4.1
Streamflow Data

STATION ID[3]	STATION NAME	DRAINAGE AREA (km²)	DATE RECORDS REPORTED TO START	AVAILABLE RECORDS	
				DAILY DATA	MAXIMUM ANNUAL FLOW DATA
10A	Bulsa	405	Aug 60	Aug 60 - Dec 72	1960 - 1972
11A	O'Donnell, Palublub	240	Jan 65	Jan 65 - Dec 72[1]	1964 - 1972[2]
11B	O'Donnell, Capas	112	Nov 58	Nov 58 - Dec 67	1962 - 1967
12A	Bangat	90	Nov 66	Nov 66 - Dec 72	1967 - 1972
23A	Camiling, Nambalan	142	Mar 64	Mar 64 - Dec 72	1964 - 1972[2]
23B	Camiling, Poblacion	280	Dec 54	Jan 57 - Nov 66	1957 - 1966
81A	Pasig-Potrero, Cabetican	242	Apr 65	Apr 65 - Dec 69	1965 - 1969
82A	Pasig-Potrero, Hda Dolores	28	Sep 66	Sep 66 - Jul 72	1966 - 1972
83A	Porac, Valdez	118	Oct 58	Oct 58 - Dec 71	1959 - 1975[2]
84A	Porac, Del Carmen	111	Oct 45	Jan 57 - May 72	1946 - 1971[2]
85A	Gumain Floodway	370	Sep 58	Sep 58 - Dec 72[1]	1959 - 1990[2]
86A	Gumain, Pabanlag	128	Oct 45	Jan 57 - Dec 71	1946 - 1979[2]
87A	Caulaman	72	Aug 54	Jan 57 - Dec 72	1955 - 1973[2]
88A	Colo	76	Mar 55	Jan 57 - Dec 71	1955 - 1979[2]
92A	Bagsit	68	Jul 60	Jul 60 - Jun 72	1960 - 1972
93A	Bucao	615	Oct 55	Jan 57 - Dec 70	1956 - 1971
94A	Santo Tomas	177	Apr 47	Jan 57 - Dec 72[1]	1947 - 1971
99B	Maloma	151	not known	Jan 84 - Apr 91	1984 - 1990

[1] Limited data exists after 1972
[2] Some years missing within period of record.
[3] Station ID's '*nn*B' are unofficial ID's

TABLE 2.4.2

Peak Discharges in m^3/s, Including Records of Doubtful Quality

RIVER	BULSA	O'DONNEL	O'DONNEL	BANGAT	CAMILING	CAMILING	PASIG-P	PASIG-P	PORAC	PORAC	GUMAIN FW	GUMAIN	CAULAMAN	COLO	BAGSIT	BUCAO	STO TOMAS	MALOMA
DA (km^2)	405	240	112	90	142	280	242	28	118	111	370	128	72	76	68	615	177	151
YEAR																		
1946										509		187						
1947										199		116					287	
1948										484		198					432	
1949										232		127					279	
1950										400		155					373	
1951										232		75					242	
1952										283		139					312	
1953										481		188					279	
1954										202		94					250	
1955										268		218	669	148		43	327	
1956												218	97	95		1092	172	
1957						376				5		247	571	102		766	274	
1958						316				7		192	339	119		1088	315	
1959									24	21	366	151	629	73		701	328	
1960	752		210						168	129	491	255	240	112	94	2525	662	
1961	1958								155	68	455	206	284	112	55	1140	576	
1962	2259		45						30	400	860	310	850	145	186	2220	855	
1963	1757		18			928			333	55	759	310	669	94	225	1380	346	
1964	1087	26	29		644	1280			330	124	762	375	708	118	107	1078	288	
1965	704	18	18		353	213	48		255	134	612	165	755	76	84	607	289	
1966	1087	36	73	65	443	610	73	4.3	321	227	742	283	960	102	111	494	270	
1967	692	103	18	353	1262		71	7.2	153	225	814	253	181	68	60	529	225	
1968	1958	115		423	474		61	8.9	38	36	399	127	40	102	94	744	10	
1969	1221	64		365	675		49	6.0	45	72	879	48	158	111	86	705	5	
1970	2082	176		414	666			2.2	83	146	857	268	677	108	91	885	14	
1971	452	70		339	58			8.8	56	43	720	216	103	108	42	190	5	
1972	1542	205		540	166			8.9		12	641	222	189		154		5	
1973	219	34			40				22		641	194	6	57		90		
1974									132		255	292		88				
1975					2				470		30	68		59		78		
1976					17							740		81		94		
1977											162		3	103		95		
1978					17						100	268	2	81		46		
1979					12						162	64	3	66		212		
1980																		
1981																		
1982																		
1983		421															65	
1984		97																145
1985		3									318							46
1986											8014							1375
1987											344							373
1988											237							188
1989											237							149
1990											286							615
1991											144							5
# OBS.	14	13	7	7	14	6	5	7	16	26	27	33	22	24	13	23	27	8
MEAN	1269	105	59	357	345	620	60	6.6	163	192	751	211	370	97	107	730	277	362
STD. DEV.	636	108	64	134	361	376	11	2.4	135	154	1448	122	313	24	50	649	196	424

TABLE 2.4.3

Normalized Peak Flows in m^3/s / km^2, Including Records of Doubtful Quality

RIVER	BULSA	O'DONNEL	O'DONNEL	BANGAT	CAMILING	CAMILING	PASIG-P	PASIG-P	PORAC	PORAC	GUMAIN FW	GUMAIN	CAULAMAN	COLO	BAGSIT	BUCAO	STO TOMAS	MALOMA
DA (km^2)	405	240	112	90	142	280	242	28	118	111	370	128	72	76	68	615	177	151
YEAR																		
1946										4.58		1.46						
1947										1.79		0.90					1.62	
1948										4.36		1.54					2.44	
1949										2.09		0.99					1.57	
1950										3.60		1.21					2.10	
1951										2.09		0.58					1.37	
1952										2.55		1.08					1.76	
1953										4.34		1.47					1.58	
1954										1.82		0.73					1.41	
1955										2.41		1.70	9.29	1.94		0.07	1.85	
1956												1.70	1.35	1.25		1.78	0.97	
1957						1.34				0.05		1.93	7.92	1.34		1.25	1.55	
1958						1.13				0.06		1.50	4.70	1.56		1.77	1.78	
1959									0.21	0.19	0.99	1.18	8.74	0.97		1.14	1.85	
1960	1.86		1.88						1.42	1.16	1.33	1.99	3.34	1.47	1.39	4.11	3.74	
1961	4.83								1.31	0.61	1.23	1.61	3.94	1.47	0.81	1.85	3.25	
1962	5.58		0.40						0.26	3.60	2.32	2.43	11.80	1.91	2.73	3.61	4.83	
1963	4.34		0.16			3.31			2.82	0.49	2.05	2.43	9.29	1.23	3.31	2.24	1.95	
1964	2.68	0.11	0.26		4.54	4.57			2.80	1.12	2.06	2.93	9.83	1.56	1.57	1.75	1.63	
1965	1.74	0.07	0.16		2.49	0.76	0.20		2.16	1.21	1.65	1.29	10.49	1.00	1.23	0.99	1.63	
1966	2.68	0.15	0.65	0.73	3.12	2.18	0.30	0.15	2.72	2.05	2.00	2.21	13.33	1.34	1.64	0.80	1.53	
1967	1.71	0.43	0.16	3.92	8.89		0.29	0.26	1.29	2.02	2.20	1.98	2.52	0.90	0.88	0.86	1.27	
1968	4.83	0.48		4.70	3.34		0.25	0.32	0.32	0.32	1.08	0.99	0.55	1.34	1.39	1.21	0.06	
1969	3.01	0.27		4.05	4.75		0.20	0.21	0.38	0.65	2.37	0.37	2.19	1.47	1.26	1.15	0.03	
1970	5.14	0.74		4.60	4.69			0.08	0.70	1.32	2.32	2.09	9.40	1.42	1.34	1.44	0.08	
1971	1.12	0.29		3.77	0.41			0.31	0.47	0.39	1.94	1.68	1.43	1.42	0.62	0.31	0.03	
1972	3.81	0.86		5.99	1.17			0.32		0.11	1.73	1.73	2.63		2.27		0.03	
1973	0.54	0.14			0.28				0.18		1.73	1.52	0.08	0.75		0.15		
1974									1.12		0.69	2.28		1.16				
1975					0.02				3.98		0.08	0.53		0.77		0.13		
1976					0.12							5.78		1.06		0.15		
1977											0.44		0.04	1.36		0.15		
1978					0.12						0.27	2.09	0.03	1.06		0.07		
1979					0.08						0.44	0.50	0.04	0.86		0.34		
1980																		
1981																		
1982																		
1983		1.75															0.37	
1984		0.40																0.96
1985		0.01									0.86							0.30
1986											21.66							9.11
1987											0.93							2.47
1988											0.64							1.25
1989											0.64							0.99
1990											0.77							4.07
1991											0.39							0.03
# OBS.	14	13	7	7	14	6	5	7	16	26	27	33	22	24	13	23	27	8
MEAN	3.13	0.44	0.53	3.96	2.43	2.22	0.25	0.24	1.38	1.73	2.03	1.65	5.13	1.28	1.57	1.19	1.57	2.40
STD. DEV.	1.57	0.45	0.57	1.49	2.54	1.34	0.04	0.09	1.14	1.39	3.91	0.95	4.34	0.31	0.74	1.06	1.11	2.81

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TABLE 2.4.4

Peak Discharges in m^3/s, Excluding Records of Doubtful Quality

RIVER	BULSA	O'DONNEL	O'DONNEL	BANGAT	CAMILING	CAMILING	PASIG-P	PASIG-P	PORAC	PORAC	GUMAIN FW	GUMAIN	CAULAMAN	COLO	BAGSIT	BUCAO	STO TOMAS	MALOMA
DA (km^2)	405	240	112	90	142	280	242	28	118	111	370	128	72	76	68	615	177	151
YEAR																		
1946										509		187						
1947										199		116					287	
1948										484		198					432	
1949										232		127					279	
1950										400		155					373	
1951										232		75					242	
1952										283		139					312	
1953										481		188					279	
1954										202		94					250	
1955										268		218	669	148			327	
1956												218	97	95		1092	172	
1957						376				5		247	571	102		766	274	
1958						316				7		192	339	119		1088	315	
1959									24	21	366	151	629	73		701	328	
1960	752		210						168	129	491	255	240	112	94	2525	662	
1961	1958								155	68	455	206	284	112	55	1140	576	
1962	2259		45							400	860	310	850	145	186	2220	855	
1963	1757		18			928			333	55	759	310	669	94	225	1380	346	
1964	1087	26	29		644	1280			330	124	762	375	708	118	107	1078	288	
1965	704		18		353	213	48		255	134	612	165	755	76	84	607	289	
1966	1087	36	73		443	610	73		321	227	742	283	960	102	111	494	270	
1967	692	103	18	353	1262		71	7.2	153	225	814	253	181	68	60	529	225	
1968	1958	115		423	474		61	8.9	38	36	399	127	40	102	94	744		
1969	1221	64		365	675		49	6.0	45	72	879	48	158	111	86	705		
1970	2082	176		414	666			2.2	83	146	857	268	677	108	91	885		
1971	452	70		339	58			8.8	56	43	720	216	103	108	42	190		
1972	1542	205		540	166			8.9			641	222	189		154			
1973		34							22		641	194		57				
1974									132		255	292		88				
1975									470			68		59				
1976												740		81				
1977											162			103				
1978											100	268		81				
1979											162	64		66				
1980																		
1981																		
1982																		
1983		421																
1984		97																145
1985											318							46
1986																		1375
1987											344							373
1988											237							188
1989											237							149
1990											286							615
1991																		
# OBS.	13	11	7	6	9	6	5	6	15	25	24	33	18	24	13	16	21	7
MEAN	1350	122	59	405	527	620	60	7.0	172	199	504	211	451	97	107	1009	351	413
STD. DEV.	587	109	64	67	331	376	11	2.4	134	153	251	122	288	24	50	592	157	430

TABLE 2.4.5
Normalized Peak Flows in m^3/s / km^2, Excluding Records of Doubtful Quality

RIVER	BULSA	O'DONNEL	O'DONNEL	BANGAT	CAMILING	CAMILING	PASIG-P	PASIG-P	PORAC	PORAC	GUMAIN FW	GUMAIN	CAULAMAN	COLO	BAGSIT	BUCAO	STO TOMAS	MALOMA
DA (km^2)	405	240	112	90	142	280	242	28	118	111	370	128	72	76	68	615	177	151
YEAR																		
1946										4.58		1.46						
1947										1.79		0.90					1.62	
1948										4.36		1.54					2.44	
1949										2.09		0.99					1.57	
1950										3.60		1.21					2.10	
1951										2.09		0.58					1.37	
1952										2.55		1.08					1.76	
1953										4.34		1.47					1.58	
1954										1.82		0.73					1.41	
1955										2.41		1.70	9.29	1.94			1.85	
1956												1.70	1.35	1.25		1.78	0.97	
1957						1.34				0.05		1.93	7.92	1.34		1.25	1.55	
1958						1.13				0.06		1.50	4.70	1.56		1.77	1.78	
1959									0.21	0.19	0.99	1.18	8.74	0.97		1.14	1.85	
1960	1.86		1.88						1.42	1.16	1.33	1.99	3.34	1.47	1.39	4.11	3.74	
1961	4.83								1.31	0.61	1.23	1.61	3.94	1.47	0.81	1.85	3.25	
1962	5.58		0.40							3.60	2.32	2.43	11.80	1.91	2.73	3.61	4.83	
1963	4.34		0.16			3.31			2.82	0.49	2.05	2.43	9.29	1.23	3.31	2.24	1.95	
1964	2.68	0.11	0.26		4.54	4.57			2.80	1.12	2.06	2.93	9.83	1.56	1.57	1.75	1.63	
1965	1.74		0.16		2.49	0.76	0.20		2.16	1.21	1.65	1.29	10.49	1.00	1.23	0.99	1.63	
1966	2.68	0.15	0.65		3.12	2.18	0.30		2.72	2.05	2.00	2.21	13.33	1.34	1.64	0.80	1.53	
1967	1.71	0.43	0.16	3.92	8.89		0.29	0.26	1.29	2.02	2.20	1.98	2.52	0.90	0.88	0.86	1.27	
1968	4.83	0.48		4.70	3.34		0.25	0.32	0.32	0.32	1.08	0.99	0.55	1.34	1.39	1.21		
1969	3.01	0.27		4.05	4.75		0.20	0.21	0.38	0.65	2.37	0.37	2.19	1.47	1.26	1.15		
1970	5.14	0.74		4.60	4.69			0.08	0.70	1.32	2.32	2.09	9.40	1.42	1.34	1.44		
1971	1.12	0.29		3.77	0.41			0.31	0.47	0.39	1.94	1.68	1.43	1.42	0.62	0.31		
1972	3.81	0.86		5.99	1.17			0.32			1.73	1.73	2.63		2.27			
1973		0.14							0.18		1.73	1.52		0.75				
1974									1.12		0.69	2.28		1.16				
1975									3.98			0.53		0.77				
1976												5.78		1.06				
1977											0.44			1.36				
1978											0.27	2.09		1.06				
1979											0.44	0.50		0.86				
1980																		
1981																		
1982																		
1983		1.75																
1984		0.40																0.96
1985											0.86							0.30
1986																		9.11
1987											0.93							2.47
1988											0.64							1.25
1989											0.64							0.99
1990											0.77							4.07
1991																		
# OBS.	13	11	7	6	9	6	5	6	15	25	24	33	18	24	13	16	21	7
MEAN	3.33	0.51	0.53	4.50	3.71	2.22	0.25	0.25	1.46	1.80	1.36	1.65	6.26	1.28	1.57	1.64	1.98	2.73
STD. DEV.	1.45	0.45	0.57	0.75	2.33	1.34	0.04	0.09	1.14	1.37	0.68	0.95	4.00	0.31	0.74	0.96	0.89	2.85

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TABLE 2.4.6

Streamflow Data

River	Station ID	Drainage Area (km^2)	Approximate Period of Daily Record[1]	Period with Stage Recorder[2]	Originates on Mount Pinatubo	Comments
Bulsa	W10A	405	1960-1972	none	no	
O'Donnell at Palublub	W11A	240	1967-1972, 1984	none	yes	Significant inconsistencies between this record and upstream gages W11B and W12A
O'Donnell at Patling	W11B	112	1959-1967	None	yes	Possible diversion above this gage into the Bangat above gage W12A
Bangat	W12A	90	1967-1972	1967-1970	yes	See comments for stations W11A and W11B
Camiling at Nambalan	W23A	142	1964-1972	1969	no	
Camiling at Poblacion	W23B	280	1957-1966	1957-1963 1965-1966	no	
Pasig-Potrero at Cabetican	W81A	242	1965-1969	none	yes	Gage tidally affected
Pasig-Potrero at Hda. Dolores	W82A	28	1967-1972	none	yes	
Porac at Valdez	W83A	118	1959-1971	none	yes	Significant inconsistencies between this record and upstream gage W84A
Porac at Del Carmen	W84A	111	1957-1971	none	yes	See comments for Station W83A
Gumain Floodway	W85A	370	1959-1973 1977-1979 1986-1990	none	yes	Record affected by operation of flood control projects
Gumain	W86A	128	1957-1971	1957-1971	yes	
Caulaman	W87A	72	1957-1972	1957-1971	no	
Colo	W88A	76	1957-1971	none	no	
Bagsit	W92A	68	1960-1971	none	no	
Bucao	W93A	615	1957-1971	1957-1964	yes	
Santo Tomas	W94A	177	1957-1971	none	yes	Post-1967 record affected by upstream irrigation diversions
Maloma	W99B	151	1984-1990	none	yes	

Notes:

1. The period of available daily record is generally shorter than the period of record for peak instantaneous flows.
2. In absence of stage recorder, water level determined by staff gauge read 2 or 3 times per day.

Table 2.4.7
Daily Data Rainfall, Evaporation, and Streamflow Stations

Type	ID #	Station Name	Record Considered
RAIN	RD324	Iba, Zambales	1951 - 1992
"	RA016	Hacienda Luisita, Tarlac	1968 - 1992
"	RA017	CLSU, Munoz, Neuva Ecija	1974 - 1992
"	R0313	Julian Subd., San Fernando, Pampanga	1970 - 1991
"	R0314	Masantol, Pampanga	1970 - 1992
"	R0320	Sta Rita Elem Sch, Cabangan, Zambales	1975 - 1992
"	R0322	Znas, San Marcelino, Zambales	1975 - 1992
"	RUSA1	Cubi Point Naval Air Station	1958 - 1990
"	RUSA2	Clark Air Force Base	1950 - 1991
EVAP	EA017	CLSU, Munoz, Neuva Ecija	1974 - 1992
FLOW	W010A	Bulsa River, Villa Aglipay; (405 km^2)	1960 - 1972
"	W011A	O'Donnell River, Palublub; (240 km^2)	1965 - 1972
"	W011B	O'Donnell River, Patling; (112 km^2)	1958 - 1967
"	W012A	Bangat River, Sta Lucia; (90 km^2)	1966 - 1972
"	W023A	Camiling River, Nambalan; (142 km^2)	1964 - 1972
"	W023B	Camiling River, Poblacion; (280 km^2)	1957 - 1966
"	W081A	Pasig-Potrero River, Cabetican; (242 km^2)	1965 - 1969
"	W082A	Pasig Potrero River, Hda Dolores; (28 km^2)	1966 - 1972
"	W084A	Porac River, Del Carmen; (111 km^2)	1957 - 1972
"	W086A	Gumain River, Pabanlag; (128 km^2)	1957 - 1971
"	W088A	Colo River, San Benito; (76 km^2)	1957 - 1971
"	W092A	Bagsit River, Dampai; (68 km^2)	1960 - 1972
"	W093A	Bucao River, San Juan; (615 km^2)	1957 - 1971
"	W094A	Santo Tomas River, Dalanawan; (177 km^2)	1957 - 1967
"	W099B	Maloma River, Maloma; (151 km^2)	1984 - 1991

Table 2.4.8

Peak Instantaneous Discharges, m³/s

RIVER	BULSA	O'DONNELL	O'DONNELL	BANGAT	CAMILING	CAMILING	PASIG-P	PASIG-P	PORAC	GUMAIN	COLO	BAGSIT	BUCAO	STO TOMAS	MALOMA
DA (km2)	405	240	112	90	142	280	242	28	111	128	76	68	615	177	151
YEAR															
1946									509	187					
1947									199	116				287	-
1948									484	198				432	
1949									232	127				279	
1950									400	155				373	
1951									232	75				242	
1952									283	139				312	
1953									481	188				279	
1954									202	94				250	
1955									268	218	148			327	
1956										218	95		1092	172	
1957						376			5	247	102		766	274	
1958						316			7	192	119		1088	315	
1959									21	151	73		701	328	
1960	752		210						129	255	112	94	2525	662	
1961	1958								68	206	112	55	1140	576	
1962	2259		45						400	310	145	186	2220	855	
1963	1757		18			928			55	310	94	225	1380	346	
1964	1087	26	29		644	1280			124	375	118	107	1078	288	
1965	704		18		353	213	48		134	165	76	84	607	289	
1966	1087	36	73		443	610	73		227	283	102	111	494	270	
1967	692	103	18	353	1262		71	7.2	225	253	68	60	529	225	
1968	1958	115		423	474		61	8.9	36	127	102	94	744		
1969	1221	64		365	675		49	6.0	72	48	111	86	705		
1970	2082	176		414	666			2.2	146	268	108	91	885		
1971	452	70		339	58			8.8	43	216	108	42			
1972	1542	205		540	166			8.9		222		154			
1973		34								194	57				
1974										292	88				
1975										68	59				
1976										740	81				
1977											103				
1978										268	81				
1979										64	66				
1980															
1981															
1982															
1983		421													
1984		97													
1985															145
1986															46
1987															1375
1988															373
1989															188
1990															149
															615
# OBS.	13	11	7	6	9	6	5	6	25	33	24	13	15	21	7
MEAN	1350	122	59	405	527	620	60	7.0	199	211	97	107	1064	351	413
STD. DEV.	587	109	64	67	331	376	11	2.4	153	122	24	50	571	157	430

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Table 2.4.9
Frequency Analysis of 1-Day Maximum Annual Rainfall

Return Period; (Chance of Exceedance in any one year)

Rain Gages (Rainfall, mm)	Years of Record	2-years (50%)	10-years (10%)	50-years (2%)	100-years (1%)	500-years (0.2%)
RD324 Iba	38	247	399	545	610	772
RA016 Hda Luisita	20	123	210	292	329	420
RA017 CLSU	18	127	204	286	324	425
R0313 Julian Subd.	22	144	273	417	488	678
R0314 Masantol	22	151	240	331	374	482
R0320 Sta Rita Sch	17	231	419	608	696	915
R0322 Znas	18	200	346	504	580	783
RUSA1 Cubi Point NAS	33	246	375	489	537	653
RUSA2 Clark AFB	41	140	255	376	433	580

1-day data (above) multiplied by 1.13 to obtain 24-hour duration amounts

RD324 Iba	38	279	451	616	689	872
RA016 Hda Luisita	20	139	237	330	372	475
RA017 CLSU	18	144	231	323	366	480
R0313 Julian Subd.	22	163	308	471	551	766
R0314 Masantol	22	171	271	374	423	545
R0320 Sta Rita Sch	17	261	473	687	786	1034
R0322 Znas	18	226	391	570	655	885
RUSA1 CubiPoint NAS	33	278	424	553	607	738
RUSA2 Clark AFB	41	158	288	425	489	655

Table 2.4.10
Frequency Analysis of 2-Day Maximum Annual Rainfall

Return Period; (Chance of Exceedance in any one year)

Rain Gages (Rainfall, mm)	Years of Record	2-years (50%)	10-years (10%)	50-years (2%)	100-years (1%)	500-years (0.2%)
RD324 Iba	38	375	600	820	920	1173
RA016 Hda Luisita	20	170	297	417	470	599
RA017 CLSU	18	168	298	447	521	724
R0313 Julian Subd.	22	203	402	625	735	1026
R0314 Masantol	22	219	359	494	554	705
R0320 Sta Rita Sch	17	369	617	851	955	1209
R0322 Znas	18	323	524	723	814	1044
RUSA1 Cubi Point NAS	33	359	543	709	782	957
RUSA2 Clark AFB	41	196	364	543	628	850

2-day data (above) multiplied by 1.04 to obtain 48-hour duration amounts

RD324 Iba	38	390	624	853	957	1220
RA016 Hda Luisita	20	177	309	434	489	623
RA017 CLSU	18	175	310	465	542	753
R0313 Julian Subd.	22	211	418	650	764	1067
R0314 Masantol	22	228	373	514	576	733
R0320 Sta Rita Sch	17	384	642	885	993	1257
R0322 Znas	18	336	545	752	847	1086
RUSA1 Cubi Point NAS	33	373	565	737	813	995
RUSA2 Clark AFB	41	204	379	565	653	884

Table 2.4.11
Frequency Analysis of 5-Day Maximum Annual Rainfall

Return Period; (Chance of Exceedance in any one year)

Rain Gages (Rainfall, mm)	Years of Record	2-years (50%)	10-years (10%)	50-years (2%)	100-years (1%)	500-years (0.2%)
RD324 Iba	38	590	892	1164	1282	1567
RA016 Hda Luisita	20	239	428	624	715	950
RA017 CLSU	18	241	435	660	773	1084
R0313 Julian Subd.	22	287	567	892	1055	1498
R0314 Masantol	22	317	560	808	924	1219
R0320 Sta Rita Sch	17	597	939	1256	1396	1740
R0322 Znas	18	502	811	1126	1274	1652
RUSA1 Cubi Point NAS	33	570	872	1137	1250	1520
RUSA2 Clark AFB	41	271	502	765	895	1251

5-day data (above) multiplied by 1.02 to obtain 120-hour duration amounts

RD324 Iba	38	602	910	1187	1308	1598
RA016 Hda Luisita	20	244	437	636	729	969
RA017 CLSU	18	246	444	673	788	1106
R0313 Julian Subd.	22	293	578	910	1076	1528
R0314 Masantol	22	323	571	824	942	1243
R0320 Sta Rita Sch	17	609	958	1281	1424	1775
R0322 Znas	18	512	827	1149	1299	1685
RUSA1 Cubi Point NAS	33	581	889	1160	1275	1550
RUSA2 Clark AFB	41	276	512	780	913	1276

Table 2.4.12
Frequency Analysis of 10-Day Maximum Annual Rainfall

Return Period; (Chance of Exceedance in any one year)

Rain Gages (Rainfall, mm)	Years of Record	2-years (50%)	10-years (10%)	50-years (2%)	100-years (1%)	500-years (0.2%)
RD324 Iba	38	834	1291	1710	1895	2344
RA016 Hda Luisita	20	342	585	818	922	1180
RA017 CLSU	18	338	547	766	871	1145
R0313 Julian Subd.	22	392	801	1270	1755	2127
R0314 Masantol	22	450	760	1076	1224	1603
R0320 Sta Rita Sch	17	909	1422	1914	2137	2691
R0322 Znas	18	729	1108	1463	1622	2012
RUSA1 Cubi Point NAS	33	860	1289	1646	1794	2136
RUSA2 Clark AFB	41	365	662	983	1137	1546

10-day data (above) multiplied by 1.01 to obtain 240-hour duration amounts

RD324 Iba	38	842	1304	1727	1914	2367
RA016 Hda Luisita	20	345	591	826	931	1192
RA017 CLSU	18	341	552	774	880	1156
R0313 Julian Subd.	22	396	809	1283	1773	2148
R0314 Masantol	22	455	768	1087	1236	1619
R0320 Sta Rita Sch	17	918	1436	1933	2158	2718
R0322 Znas	18	736	1119	1478	1638	2032
RUSA1 Cubi Point NAS	33	869	1302	1662	1812	2157
RUSA2 Clark AFB	41	369	669	993	1148	1561

Table 2.4.13
Frequency Analysis of 15-Day Maximum Annual Rainfall

Return Period; (Chance of Exceedance in any one year)

Rain Gages (Rainfall, mm)	Years of Record	2-years (50%)	10-years (10%)	50-years (2%)	100-years (1%)	500-years (0.2%)
RD324 Iba	38	1020	1561	2014	2204	2645
RA016 Hda Luisita	20	400	674	953	1083	1417
RA017 CLSU	18	404	620	839	940	1201
R0313 Julian Subd.	22	442	897	1414	1670	2357
R0314 Masantol	22	528	883	1230	1388	1784
R0320 Sta Rita Sch	17	1100	1737	2332	2597	3246
R0322 Znas	18	905	1373	1750	1903	2251
RUSA1 Cubi Point NAS	33	1030	1528	1941	2113	2507
RUSA2 Clark AFB	41	427	779	1188	1395	1967

Table 2.4.14
Streamflow Data Frequency Analyses Summary
Expected Probability Discharges in m^3/s

(ranges correspond to alternative data sets)

Gumain River (W086A)

Return Period	2-year	100-year
Peak Instantaneous Q	186 to 226	495 to 663
1-Day Average Q	128 to 152	337 to 417
3-Day Average Q	(n/a) to 109	312 to (n/a)

Bucao River (W093A)

Return Period	2-year	100-year
Peak Instantaneous Q	895 to 962	3740 to 4330
1-Day Average Q	656 to 697	1260 to 2730
3-Day Average Q	521 to 548	891 to 1870

Santo Tomas River (W094A)

Return Period	2-year	100-year
Peak Instantaneous Q	307 to 356	798 to 1560
1-Day Average Q	251 to 299	524 to 1160
3-Day Average Q	207 to 221	562 to 1080

Table 2.4.15
Composite Analysis of Rainfall Frequency Data

Return Period; (Chance of Exceedance in any one year)

AVERAGE RAINFALL (mm) FOR	DURATION	2-years (50%)	10-years (10%)	50-years (2%)	100-years (1%)	500-years (0.2%)
COASTAL GAGES	24-HR	261	435	606	684	882
COASTAL GAGES	2-DAY	371	594	807	902	1140
COASTAL GAGES	5-DAY	576	896	1194	1327	1652
COASTAL GAGES	10-DAY	841	1290	1700	1881	2319
COASTAL GAGES	15-DAY	1014	1550	2009	2204	2662
INTERIOR GAGES	24-HR	155	267	385	440	584
INTERIOR GAGES	2-DAY	199	358	525	605	812
INTERIOR GAGES	5-DAY	276	508	765	890	1224
INTERIOR GAGES	10-DAY	381	678	992	1194	1535
INTERIOR GAGES	15-DAY	440	771	1125	1295	1745

Ratio of Interior to Coastal Rainfall for same Frequency-Duration Events

INTERIOR/COASTAL	24-HR	0.59	0.61	0.63	0.64	0.66
INTERIOR/COASTAL	2-DAY	0.53	0.60	0.65	0.67	0.71
INTERIOR/COASTAL	5-DAY	0.47	0.56	0.64	0.67	0.74
INTERIOR/COASTAL	10-DAY	0.45	0.52	0.58	0.63	0.66
INTERIOR/COASTAL	15-DAY	0.43	0.49	0.55	0.58	0.65
INTERIOR/COASTAL	Average	0.49	0.56	0.61	0.64	0.68

NOTES:

1) Ratio of Interior/Coastal Mean Annual Rain: 1904 mm / 3779 mm = 0.50

2) Peak high-elevation rainfall on Mount Pinatubo estimated to be approximately 1.4 times the average coastal rainfall for all frequency-duration amounts. Higher rainfalls, to approximately 1.7 times the average coastal rainfall, are estimated to occur at higher elevations coastal mountains located approximately 40 km north of Mount Pinatubo.

Coastal Gages:
- RD324 Iba
- R0320 Sta. Rita Elem. School
- R0322 Znas
- RUSA1 Cubi Point NAS

Interior Gages:
- RA016 Hda Luisita
- RA017 CLSU
- R0313 Julian Subdivision
- R0314 Masantol
- RUSA2 Clark AFB

Table 2.4.16
Short-Duration Rainfall Frequency Data
For Representative Interior, Coastal, and High-Elevation Stations
Data in mm

Return Period; (Chance of Exceedance in any one year)

Station	DURATION	2-years (50%)	10-years (10%)	50-years (2%)	100-years (1%)	500-years (0.2%)
LOWLAND INTERIOR						
Hda. Luisita	1 HOUR	52	85	114	126	n/a
Hda. Luisita	2 HOURS	60	102	139	154	n/a
Hda. Luisita	3 HOURS	69	115	155	172	n/a
Hda. Luisita	6 HOURS	84	130	171	188	n/a
Hda. Luisita	12 HOURS	101	179	248	277	n/a
Hda. Luisita	24 HOURS	127	246	350	394	n/a
WINDWARD COAST						
Iba	1 HOUR	61	87	109	119	n/a
Iba	2 HOURS	83	126	164	180	n/a
Iba	3 HOURS	98	152	200	220	n/a
Iba	6 HOURS	138	224	300	332	n/a
Iba	12 HOURS	187	372	533	601	n/a
Iba	24 HOURS	238	482	697	788	n/a
HIGH ELEVATIONS						
Baguio City	1 HOUR	56	80	102	111	n/a
Baguio City	2 HOURS	80	145	203	227	n/a
Baguio City	3 HOURS	100	197	281	317	n/a
Baguio City	6 HOURS	157	340	500	568	n/a
Baguio City	12 HOURS	231	528	789	900	n/a
Baguio City	24 HOURS	319	674	985	1117	n/a

Data extracted from Hydrology and Flood Forecast Center, PAGASA, 1981: "Rainfall Intensity-Duration-Frequency Data of the Philippines" Volume 1, First Edition.

Baguio City is located approximately 140 km NNE of Mt. Pinatubo, at elevation 1370 m (4500 feet).

Table 2.4.17
Analysis of Short-Duration Rainfall Frequency Data From Representative Interior, Coastal, and High-Elev. Stations For Same-Frequency-Duration Events

Return Period; (Chance of Exceedance in any one year)

Ratio of Stations	DURATION	2-years (50%)	10-years (10%)	50-years (2%)	100-years (1%)	AVERAGE
INTERIOR TO COAST						
Luisita/Iba	1 HOUR	0.86	0.98	1.04	1.06	0.98
Luisita/Iba	2 HOURS	0.72	0.81	0.85	0.86	0.80
Luisita/Iba	3 HOURS	0.70	0.75	0.77	0.78	0.75
Luisita/Iba	6 HOURS	0.61	0.58	0.57	0.57	0.58
Luisita/Iba	12 HOURS	0.54	0.48	0.46	0.46	0.48
Luisita/Iba	24 HOURS	0.53	0.51	0.50	0.50	0.51
INTERIOR TO HIGH ELEVATIONS						
Luisita/Baguio	1 HOUR	0.94	1.06	1.12	1.13	1.06
Luisita/Baguio	2 HOURS	0.75	0.70	0.68	0.68	0.70
Luisita/Baguio	3 HOURS	0.68	0.58	0.55	0.54	0.58
Luisita/Baguio	6 HOURS	0.53	0.38	0.34	0.33	0.39
Luisita/Baguio	12 HOURS	0.44	0.34	0.31	0.31	0.35
Luisita/Baguio	24 HOURS	0.40	0.36	0.36	0.35	0.36
COAST TO HIGH ELEVATIONS						
Iba/Baguio	1 HOUR	1.10	1.08	1.07	1.07	1.07
Iba/Baguio	2 HOURS	1.04	0.87	0.81	0.79	0.87
Iba/Baguio	3 HOURS	0.98	0.77	0.71	0.69	0.78
Iba/Baguio	6 HOURS	0.88	0.66	0.60	0.58	0.68
Iba/Baguio	12 HOURS	0.81	0.70	0.68	0.67	0.71
Iba/Baguio	24 HOURS	0.75	0.72	0.71	0.71	0.71

Table 2.4.18

Depth-Duration Data for Design Storms
Hypothetical Station Located near Summit of Mount Pinatubo
Data in mm

Return Period; (Chance of Exceedance in any one year)

DURATION	2-years (50%)	10-yrs (10%)	50-yrs (2%)	100-yrs (1%)	500-yrs (.2%)
1-Hour	57	81	103	113	146
2-Hours	82	140	193	215	280
3-Hours	102	186	261	293	381
6-Hours	159	311	446	505	655
12-Hours	240	478	694	789	1022
24-Hours	352	587	830	936	1196
2-Days	501	802	1101	1230	1544
5-Days	777	1209	1624	1804	2236

NOTES

1. The depth-duration-frequency data summarized above are for a hypothetical station located at approximately 1000 meters elevation near the summit of Mount Pinatubo. This station has a mean annual rainfall of about 5000 mm.

2. The 1-hour duration depth-frequency data are effectively independent of station location. The 1-hour data presented above are directly applicable to any site (coastal, mountain, or interior) throughout the study area.

3. Depth-frequency data for durations of 6 hours and longer may be transposed to other sites throughout the study area after multiplication by a site-specific factor. The factor is equal to the ratio of the desired site's mean annual rainfall in millimeters to 5000 mm, the mean annual rainfall for the hypothetical station.

4. Same-frequency durations are not fully embedded in a single storm; i.e., a 100-year 24-hour storm is not expected to include a 100-year 1-hour duration. The available data showed a maximum 20-year 1-hour rain within a 200-year 24-hour storm, and a maximum 2-year 1-hour rain within a 25-year 24-hour storm. This relationship of 1-hour to 24-hour return periods can be approximated by assuming a constant rainfall intensity over the peak six-hour duration.

Table 3.2.1

Sub-Basin Parameters

Gumain Basin, Pre-Eruption

Sub-Basin ID	PG1	PG2	PG3	PG4	PG5	PG6	PG7	PG8	PG9
Physical Parameters									
Area (km2)	9.2	16.5	6.7	22.8	11.0	13.1	12.3	19.0	6.1
Longest Flow Path within Sub-Basin (km)	4.3	8.3	3.9	9.0	7.0	7.8	7.9	11.9	6.4
Elevation Change along Flow Path (m)	665	865	925	575	700	1,323	1,060	1,050	190
Channel Length to Upstream Basin (km)	N/A	7.9	N/A	7.4	4.7	N/A	N/A	9.2	5.3
Elevation Change along Channel (m)	N/A	290	N/A	290	60	N/A	N/A	210	30
Design Storm Parameters									
Estimated Annual Rainfall (mm)	4,280	3,920	4,370	4,070	3,610	4,220	4,160	3,670	3,310
Percent of Rainfall near Pinatubo Summit	86	78	87	81	72	84	83	73	66
Distance to Assumed Storm Center (km)	10.4	10.0	6.2	6.7	8.7	2.5	1.3	4.4	14.2
Runoff Parameters									
Time of Concentration (hrs)	0.4	0.8	0.3	1.0	0.7	0.6	0.7	1.1	1.1
Clark Storage Coefficient (hrs)	10.0	20.0	7.5	25.0	17.5	15.0	17.5	27.5	27.5
Uniform Infiltration Loss (mm/hr)	3	3	3	3	3	3	3	3	3
Baseflow (cms)	1.2	2.1	0.9	2.9	1.4	1.7	1.6	2.4	0.8
Flow Routing Parameters									
Travel Time (hrs)	N/A	0.9	N/A	0.8	0.5	N/A	N/A	1.0	0.6
Number of Sub-Reaches	N/A	1	N/A	1	1	N/A	N/A	1	1

Table 3.2.2

Sub-Basin Parameters

Bucao Basin, Pre-Eruption

Sub-Basin ID	PB1	PB2	PB3	PB4	PB5	PB6	PB7
Physical Parameters							
Area (km2)	72.3	62.0	23.7	67.0	39.1	12.5	43.3
Longest Flow Path within Sub-Basin (km)	16.8	17.4	8.3	13.9	14.3	10.3	15.6
Elevation Change along Flow Path (m)	940	1,600	875	980	620	440	700
Channel Length to Upstream Basin (km)	N/A	N/A	2.9	12.5	N/A	N/A	5.8
Elevation Change along Channel (m)	N/A	N/A	35	105	N/A	N/A	80
Design Storm Parameters							
Estimated Annual Rainfall (mm)	4,100	4,200	4,200	4,200	3,900	4,100	4,050
Percent of Rainfall near Pinatubo Summit	82	84	84	84	78	82	81
Distance to Assumed Storm Center (km)	6.0	3.0	10.5	13.6	9.2	5.4	7.6
Runoff Parameters							
Time of Concentration (hrs)	1.8	1.5	.8	1.4	1.7	1.3	1.8
Clark Storage Coefficient (hrs)	45.0	37.5	20.0	35.0	42.5	32.5	45.0
Uniform Infiltration Loss (mm/hr)	3	3	3	3	3	3	3
Baseflow ($m^{3/s}$)	9	8	3	9	5	2	6
Flow Routing Parameters							
Travel Time (hrs)	N/A	N/A	0.3	1.4	N/A	N/A	0.6
Number of Sub-Reaches	N/A	N/A	0	1	N/A	N/A	1

Table 3.2.3

Sub-Basin Parameters

Santo Tomas Basin, Pre-Eruption

Sub-Basin ID	PT1	PT2	PT3	PT4	PT5
Physical Parameters					
Area (km2)	45.6	34.6	9.4	20.0	75.7
Longest Flow Path within Sub-Basin (km)	10.5	9.2	8.3	5.5	11.0
Elevation Change along Flow Path (m)	1,471	1,027	250	770	965
Channel Length to Upstream Basin (km)	N/A	5.0	6.1	N/A	8.1
Elevation Change along Channel (m)	N/A	116	68	N/A	20
Design Storm Parameters					
Estimated Annual Rainfall (mm)	4,820	4,370	4,070	4,280	4,100
Percent of Rainfall near Pinatubo Summit	96	87	81	86	82
Distance to Assumed Storm Center (km)	0.0	6.0	10.1	14.2	13.1
Runoff Parameters					
Time of Concentration (hrs)	0.9	0.9	1.3	0.5	1.1
Clark Storage Coefficient (hrs)	22.5	22.5	32.5	12.5	27.5
Uniform Infiltration Loss (mm/hr)	3	3	3	3	3
Baseflow ($cm^{3/s}$)	4	3	1	2	6
Flow Routing Parameters					
Travel Time (hrs)	N/A	0.6	0.7	N/A	0.9
Number of Sub-Reaches	N/A	1	1	N/A	1

Table 3.2.4

HEC-1 Calibration and Sensitivity Analysis
Simulations with Clark Unit Hydrograph

Stream gauge	Parameter Set	Peak Inst. Q m³/s		24-hr Vol. dam³		3-day Vol. dam³	
		2-yr	100-yr	2-yr (000 s)	100-yr (000 s)	2-yr (000 s)	100-yr (000 s)
Gumain (W086A)	Target	180-230	490-670	11-13	29-36	(n/a)-28	81-(n/a)
	A	220	940	12	54	18	78
	B	220	790	16	53	28	96
	C	220	680	17	49	36	101
Bucao (W093A)	Target	890-960	3750-4350	56-60	108-235	135-142	230-484
	A	731	3370	52	232	93	406
	B	786	2772	64	214	140	471
	C	811	2358	67	191	168	467
Santo Tomas (W094A)	Target	310-360	800-1560	22-26	45-100	54-57	145-280
	A	320	1580	21	102	31	174
	B	330	1190	26	84	49	164
	C	320	970	27	73	60	154

* Target values (range) from HEC-FFA analyses of recorded data; ranges correspond to alternative data sets.

Parameter Set	Constant Loss Rate mm/hr	Unit Hydrograph Storage Coefficient
A	5.75	15 x T_c
B	3.0	25 x T_c
C	1.8	35 x T_c

(T_c = Basin Time of Concentration)

Table 3.5.1

Sub-Basin Unit Hydrograph Peaks

Basin, Sub-Basin	Unit Hydrograph Peak (cms)	Basin, Sub-basin	Unit Hydrograph Peak (cms)	Basin, Sub-basin	Unit Hydrograph Peak (cms)
Abacan, A1	0.093	Gumain-Porac, G6	0.232	O'Donnell, O14	0.427
Abacan,A2	0.169	Gumain-Porac, G7	0.174	O'Donnell, O15	0.518
Abacan, A3	0.081	Gumain-Porac, G8	0.184	O'Donnell, O16	0.354
Abacan, A4	0.323	Gumain-Porac, G9	0.060	O'Donnell, O17	0.175
Abacan, A5	0.038	Gumain-Porac, G10	0.061	Pasig/Potrero, P0	0.320
Bucao, B1	0.431	Gumain-Porac, G11	0.155	Pasig/Potrero, P1	0.118
Bucao, B2	0.330	Gumain-Porac, G12	0.196	Pasig/Potrero, P2	0.102
Bucao, B3	0.086	Gumain-Porac, G13	0.155	Pasig/Potrero, P3	0.063
Bucao, B4	0.313	Gumain-Porac, G14	0.120	Pasig/Potrero, P4	0.035
Bucao, B5	0.505	Gumain-Porac, G15	0.189	Pasig/Potrero, P5	0.070
Bucao, B6	0.246	Gumain-Porac, G16	0.196	Pasig/Potrero, P6	0.086
Bucao, B7	0.140	Gumain-Porac, G17	0.081	Sacobia, S2	0.180
Bucao, B8	0.262	Gumain-Porac, G18	0.141	Sacobia, S3	0.084
Bucao, B9	0.143	O'Donnell, O1	0.162	Sacobia, S4	0.110
Bucao, B10	0.544	O'Donnell, O2	0.286	Sacobia, S5	0.334
Bucao, B11	0.429	O'Donnell, O3	0.201	Sacobia, S6	0.176
Bucao, B12	0.317	O'Donnell, O4	0.156	Santo Tomas, T1	0.452
Bucao, B13	0.107	O'Donnell, O5	0.307	Santo Tomas, T2	0.398
Bucao, B14	0.256	O'Donnell, O6	0.185	Santo Tomas, T3	0.083
Bucao, B15	0.551	O'Donnell, O7	0.166	Santo Tomas, T4	0.896
Bucao, B16	0.342	O'Donnell, O8	0.134	Santo Tomas, T5	0.317
Gumain-Porac, G1	0.237	O'Donnell, O9	0.075	Santo Tomas, T6	0.534
Gumain-Porac, G2	0.221	O'Donnell, O10	0.075	Santo Tomas, T7	1.111
Gumain-Porac, G3	0.221	O'Donnell, O11	0.383	Santo Tomas, T8	0.181
Gumain-Porac, G4	0.246	O'Donnell, O12	0.224	Santo Tomas, T9	0.160
Gumain-Porac, G5	0.170	O'Donnell, O13	0.447	Santo Tomas, T10	0.029

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Table 3.5.2
Sub-Basin Parameters
Sacobia-Bamban Basin

Sub-Basin ID	S2	S3	S4	S5	S6	S7
Physical Parameters						
Area (km2)	30.2	13.4	22.7	41.3	26.4	11.6
Longest Flow Path within Sub-Basin (km)	15.6	11.8	18.9	13.3	11.6	15.6
Elevation Change along Flow Path (m)	720	336	785	945	425	186
Channel Length to Upstream Basin (km)	N/A	9.8	N/A	N/A	10.9	12.5
Elevation Change along Channel (m)	N/A	125	N/A	N/A	100	23
Design Storm Parameters						
Estimated Annual Rainfall (mm)	3,250	2,650	2,860	3,490	2,710	2,110
Percent of Rainfall near Pinatubo Summit	65	53	57	70	54	42
Distance to Assumed Storm Center (km)						
Storm 1 at Sub-Basin S2	0	9	5.5	N/A	N/A	N/A
Storm 2 at Sub-Basin S4	N/A	N/A	0	N/A	N/A	N/A
Storm 3 at Sub-Basin S5	4.4	12.2	7.5	0	10.9	20
Runoff Parameters						
Time of Concentration (hrs)	1.8	1.7	2.2	1.3	1.6	3
Clark Storage Coefficient (hrs)	45	42.5	55	32.5	40	75
Uniform Infiltration Loss (mm/hr)	3	3	3	3	3	1000
Baseflow (cms)	2	1	1	3.3	0.9	0
Flow Routing Parameters						
Travel Time of Flood Wave (hrs)	N/A	1.1	N/A	N/A	1.2	1.4
Number of Sub-Reaches (1-hr time step)	N/A	1	N/A	N/A	1	1

Table 3.5.3
Simulation Output Sites
Sacobia-Bamban Basin

Site ID	Corps-Specified Site	Site Description	Stream Elevation (m)	Upstream Basin area (km2)	Time of Concen-tration (hours)	Average Annual Basin Rain (mm)	Average Annual Flow (cms)	Critical Storm Location	Critical Storm ID
S2DS	Yes	Sacobia River 10 km above confluence with Sapang Cauyan River	180	30.2	1.8	3,250	1.6	S2	1
S3DS	Yes	Sacobia River above confluence with Sapang Cauyan River	55	43.6	3	3,065	2.1	S2	1
S4DS	Yes	Sapang Cauyan River above confluence with Sacobia River	55	22.7	2.2	2,860	0.9	S4	2
S3DS+S4DS	Yes	Sacobia River below confluence with Sapang Cauyan River	55	66.3	3	2,995	3	S2	1
S6DS	Yes	Marimla River above confluence with Sacobia River	55	67.7	2.6	3,190	3.5	S5	3
S7US	Yes	Bamban River below confluence with Sacobia and Marimla Rivers	55	133.9	3	3,095	6.5	S5	3
S7DS	Yes	Bamban River near Concepcion City	32	145.5	4.7	3,016	6.7	S5	3

Table 3.5.4

Computed Maximum Annual Peak Discharge and Volume Frequency Data
Sacobia-Bamban Basin

Site I D	Maximum Annual Peak Discharge (cms)				
	2-year	10-year	50-year	100-year	500-year
S2DS	25	54	82	95	125
S3DS	32	71	109	125	166
S4DS	13	29	45	52	70
S3DS+S4DS	44	98	152	175	232
S6DS	62	138	211	243	321
S7US	102	233	358	413	547
S7DS	102	230	354	409	541

Site I D	Maximum Annual 24-hr Volume (cubic decameters)				
	2-year	10-year	50-year	100-year	500-year
S2DS	2,100	4,200	6,500	7,400	9,800
S3DS	2,700	5,500	8,500	9,800	13,000
S4DS	1,100	2,300	3,600	4,200	5,500
S3DS+S4DS	3,700	7,700	11,900	13,800	18,300
S6DS	5,000	10,500	16,000	18,400	24,300
S7US	8,300	17,900	27,600	31,800	42,000
S7DS	8,300	17,900	27,500	31,700	41,900

Site I D	Maximum Annual 3-Day Volume (cubic decameters)				
	2-year	10-year	50-year	100-year	500-year
S2DS	4,500	9,400	14,500	16,800	22,200
S3DS	5,900	12,200	19,100	22,100	29,400
S4DS	2,500	5,200	8,400	9,800	13,100
S3DS+S4DS	8,200	17,100	27,000	31,400	41,900
S6DS	10,200	21,400	33,400	38,600	51,200
S7US	17,600	38,000	59,600	69,100	92,000
S7DS	17,600	37,900	59,600	69,100	92,000

Table 3.5.5
Computed Flow Duration Curves
Sacobia-Bamban Basin
Data in cubic meters per second

% OF TIME EXCEEDED	S2DS	S3DS	S3DS+S4DS	S4DS	S6DS	S7US	S7DS
100.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
50.0	0.6	0.8	1.2	0.4	1.4	2.6	2.7
25.0	1.6	2.1	3.0	0.9	3.5	6.5	6.7
20.0	2.0	2.6	3.7	1.1	4.4	8.0	8.2
10.0	3.3	4.3	6.1	1.8	7.3	13.3	13.6
5.0	4.7	6.2	8.7	2.6	10.7	19.0	19.4
2.0	7.8	10.1	14.2	4.2	17.9	31.3	31.7
1.0	11.2	14.5	20.1	6.0	26.0	44.7	45.1
0.5	15.5	20.0	27.6	8.2	36.5	61.6	61.8
0.2	23.6	30.5	42.1	12.5	59.1	98.9	95.4
0.1	31.7	41.3	57.4	17.1	82.1	138.7	131.9
0.05	40.9	53.5	74.7	22.3	105.3	179.0	173.1
0.02	52.0	68.1	95.3	28.5	129.5	220.8	221.6

Table 3.5.6

Sub-Basin Parameters

Abacan Basin

Sub-Basin ID	A1	A2	A3	A4	A5
Physical Paramters					
Area (km2)	7.9	12.7	15.8	9.9	5.1
Longest Flow Path within Sub-Basin (km)	6.4	7.9	14.7	9.2	12.7
Elevation Change along Flow Path (m)	305	190	415	85	75
Channel Length to Upstream Basin (km)	N/A	6.7	2.4	2.6	12.5
Elevation Change along Channel (m)	N/A	80	30	20	70
Design Storm Parameters					
Estimated Annual Rainfall (mm)	3,160	2,860	3,010	2,530	1,990
Percent of Rainfall near Pinatubo Summit	63	57	60	51	40
Distance to Assumed Storm Center (km)					
Strom 1 at Sub-Basin A1	0.0	3.6	2.5	9.5	17.9
Runoff Paramters					
Time of Concentration (hrs)	0.9	1.4	2.1	2.2	1.4
Clark Storage Coefficient (hrs)	22.5	35.0	52.5	55.0	35.0
Uniform Infiltration Loss (mm/hr)	3	3	3	3	3
Baseflow (cms)	0.5	0.6	0.8	0.2	0.0
Flow Routing Parmaters					
Travel Time of Flood Wave (hrs)	N/A	0.7	0.3	0.3	1.4
Number of Sub-Reaches (1-hr time step)	N/A	1	1	1	1

Table 3.5.7
Simulation Output Sites
Abacan Basin

Site ID	Corps-Specified Site	Site Description	Stream Elevation (m)	Upstream Basin Area (km2)	Time of Concen-tration (hours)	Average Annual Basin Rain (mm)	Average Annual Flow (cms)	Critical Storm Location
A1DS	Yes	Abacan River approximately 7 km above Sapang-Bayo creek confluence	210	7.9	0.9	3,160	0.4	A1
A3DS	Yes	Abacan River approximately 1.3 km above Highway 3 bridge	100	36.4	2.1	2,990	1.6	A1
A5DS	Yes	Abacan River at Highway 329	10	51.4	4.5	2,800	2.0	A1

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Table 3.5.8

Computed Maximum Annual Peak Discharge and Volume Frequency Data
Abacan Basin

Maximum Annual Peak Discharge (cms)

Site ID	2-year	10-year	50-year	100-year	500-year
A1DS	10	23	35	40	52
A3DS	31	69	106	122	161
A5DS	41	90	138	159	211

Maximum Annual 24-hr Volume (dam3)

Site ID	2-year	10-year	50-year	100-year	500-year
A1DS	800	1,600	2,400	2,800	3,600
A3DS	2,500	5,100	7,800	9,000	11,900
A5DS	3,100	6,500	10,000	11,500	15,300

Maximum Annual 3-Day Volume (dam3)

Site ID	2-year	10-year	50-year	100-year	500-year
A1DS	1,300	2,800	4,300	5,000	6,700
A3DS	4,900	10,300	16,200	18,800	25,000
A5DS	6,300	13,100	21,000	23,900	21,900

Table 3.5.9
Computed Flow Duration Curves
Abacan Basin
Data in cubic meters per second

% OF TIME EXCEEDED	A1DS	A3DS	A5DS
100	0.0	0.0	0.0
50	0.2	0.6	0.8
20	0.5	2.1	2.6
10	0.9	3.4	4.3
5	1.7	5.1	6.3
2	3.0	8.6	10.7
1	4.3	12.7	15.8
0.5	6.0	18.0	22.4
0.2	9.3	29.3	36.6
0.1	12.7	40.3	50.8
0.05	16.1	51.3	65.2
0.02	19.7	62.9	80.2

Table 3.5.10 (sheet 1 of 2)

Sub-Basin Parameters

O'Donnell Basin

Sub-Basin ID	O1	O2	O3	O4	O5	O6	O7	O8	O9
Physical Parameters									
Area (km2)	22.9	47.9	15.2	26.2	57.0	12.3	38.8	27.8	18.1
Longest Flow Path within Sub-Basin (km)	13.1	15.0	8.5	15.0	15.5	7.3	21.6	16.4	16.3
Elevation Change along Flow Path (m)	700	633	1,070	628	510	1,040	817	520	328
Channel Length to Upstream Basin (km)	N/A	11.2	N/A	14.5	13.3	N/A	20.6	N/A	9.9
Elevation Change along Channel (m)	N/A	160	N/A	250	60	N/A	280	N/A	30
Design Storm Parameters									
Estimated Annual Rainfall (mm)	4,160	3,460	4,160	3,460	2,860	3,920	3,310	3,160	2,710
Percent of Rainfall near Pinatubo Summit	83	69	83	69	57	78	66	63	54
Distance to Assumed Storm Center (km)									
Storm 1 at Sub-Basin O1	0.0	9.4	2.7	8.3	20.0	5.2	9.8	12.9	19.6
Storm 2 at Sub-Basin O6	N/A	N/A	N/A	N/A	N/A	0.0	9.4	13.8	N/A
Storm 3 at Sub-Basin O13	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Runoff Parameters									
Time of Concentration (hrs)	1.5	1.8	0.8	1.8	2.0	0.7	2.5	2.2	2.6
Clark Storage Coefficient (hrs)	37.5	45.0	20.0	45.0	50.0	17.5	62.5	55.0	65.0
Uniform Infiltration Loss (mm/hr)	3	3	3	3	3	3	3	3	3
Baseflow (cms)	2.7	3.7	1.8	2.0	2.5	1.3	2.7	1.7	0.6
Flow Routing Parameters									
Travel Time of Flood Wave (hrs)	N/A	1.2	N/A	1.6	1.5	N/A	2.3	N/A	1.1
Number of Sub-Reaches (1-hr time step)	N/A	1	N/A	2	1	N/A	2	N/A	1

Table 3.5.10 (sheet 2 of 2)

Sub-Basin Parameters

O'Donnell Basin

Sub-Basin ID	O10	O11	O12	O13	O14	O15	O16	O17
Physical Parameters								
Area (km2)	40.9	74.3	43.8	33.8	79.1	148.7	56.3	74.3
Longest Flow Path within Sub-Basin (km)	18.9	16.0	16.7	9.1	19.1	27.1	12.3	35.2
Elevation Change along Flow Path (m)	60	530	626	1248	974	925	420	714
Channel Length to Upstream Basin (km)	15.8	N/A	10.9	N/A	14.2	13.7	5.6	18.9
Elevation Change along Channel (m)	40	N/A	79	N/A	206	79	15	30
Design Storm Parameters								
Estimated Annual Rainfall (mm)	2,410	3,770	3,460	5,420	4,220	3,770	2,950	2,710
Percent of Rainfall near Pinatubo Summit	48	75	69	108	84	75	59	54
Distance to Assumed Storm Center (km)								
Storm 1 at Sub-Basin O1	27.9	10.4	16.7	28.0	22.9	27.1	28.3	28.1
Storm 2 at Sub-Basin O6	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Storm 3 at Sub-Basin O13	N/A	16.9	15.6	0.0	7.8	14.7	23.5	29.7
Runoff Parameters								
Time of Concentration (hrs)	5.9	2.1	2.1	0.8	2.0	3.1	1.7	4.6
Clark Storage Coefficient (hrs)	147.5	52.5	52.5	20.0	50.0	77.5	42.5	115.0
Uniform Infiltration Loss (mm/hr)	3	3	3	3	3	3	3	3
Baseflow (cms)	0.7	7.1	3.4	6.4	9.6	14.2	2.7	2.6
Flow Routing Parameters								
Travel Time of Flood Wave (hrs)	1.8	N/A	1.2	N/A	1.6	1.5	0.6	2.1
Number of Sub-Reaches (1-hr time step)	2	N/A	1	N/A	2	2	1	2

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Table 3.5.11
Simulation Output Sites
O'Donnell Basin

Site ID	Corps-Specified Site	Site Description	Stream Elevation (m)	Upstream Basin Area (km2)	Time of Concen-tration (hours)	Average Annual Basin Rain (mm)	Average Annual Flow (cms)	Critical Storm Location
O1DS	Yes	O'Donnell River approximately 1 km below pyroclastic flow deposit	300	22.9	1.5	4,160	1.9	O1
O5US	No	O'Donnell River below confluence with Apalong River	150	112.2	2.8	3,700	7.6	O1
O5DS	Yes	O'Donnell River above confluence with Bangat River	90	169.1	4.5	3,420	9.9	O1
O7DS	Yes	Bangat River above road crossing approximately 1km southeast of O'Donnell village	120	51.1	2.8	3,460	3.1	O6
O9US	No	Bangat River below road crossing (and unnamed tributary) approximately 1 km southeast of O'Donnell village	120	78.8	2.8	3,350	4.4	O6
O10US	Yes	O'Donnell River below confluence with Bangat River	90	266.0	4.5	3,350	15.0	O1
O10DS	No	O'Donnell River above confluence with Bulsa River	50	305.1	6.5	3,230	16.0	O1
O15US	No	Bulsa River approximately 20 km above streamflow gage W010A	174	231.0	3.9	4,110	18.6	O13
O16DS	No	Bulsa River at streamflow gage W010A	80	436.0	6.8	3,840	31.4	O13
O17DS	No	Bulsa River above confluence with O'Donnell River	50	508.5	9.6	3,680	34.0	O13
O18DS	Yes	O'Donnell River above Highway 13	40	817.2	9.9	3,500	49.9	O1

Table 3.5.12

Computed Maximum Annual Peak Discharge and Volume Frequency Data
O' Donnell Basin

Maximum Annual Peak Discharge (cms)

Site ID	2-year	10-year	50-year	100-year	500-year
O1DS	31	65	97	110	144
O5US	121	258	389	446	585
O5DS	150	321	490	563	741
O7DS	47	102	154	177	233
O9US	63	137	210	241	318
O10US	218	468	714	822	1,084
O10DS	223	477	730	841	1,110
O15US	264	546	815	932	1,216
O16DS	392	799	1,201	1,377	1,802
O17DS	409	830	1,251	1,435	1,881
O18DS	616	1,273	1,933	2,221	2,922

Maximum Annual 24-hr Volume (dam3)

Site ID	2-year	10-year	50-year	100-year	500-year
O1DS	2,600	5,000	7,400	8,500	11,000
O5US	9,900	19,800	29,700	34,000	44,500
O5DS	12,200	25,000	38,000	43,700	57,400
O7DS	3,800	7,600	11,500	13,200	17,400
O9US	5,100	10,500	16,000	18,400	24,200
O10US	17,800	36,500	55,700	64,000	84,300
O10DS	18,200	37,600	57,400	66,100	87,200
O15US	21,900	42,600	63,100	72,000	93,700
O16DS	32,400	64,200	96,100	110,000	143,600
O17DS	33,700	67,300	101,100	115,800	151,500
O18DS	50,600	102,600	155,300	178,400	234,200

Maximum Annual 3-Day Volume (dam3)

Site ID	2-year	10-year	50-year	100-year	500-year
O1DS	5,600	11,000	16,400	18,700	24,300
O5US	20,900	42,400	64,300	73,800	96,800
O5DS	26,300	54,100	83,600	96,400	127,200
O7DS	7,800	16,000	24,500	28,300	37,200
O9US	10,900	22,700	35,200	40,600	53,700
O10US	38,300	79,400	123,100	142,200	188,200
O10DS	39,600	82,500	128,300	148,400	196,900
O15US	48,200	95,200	141,900	162,000	211,100
O16DS	74,400	149,000	224,900	258,000	337,600
O17DS	78,300	157,600	239,100	274,800	360,400
O18DS	115,500	236,100	362,100	417,200	549,800

Table 3.5.13
Computed Flow Duration Curves
O'Donnell Basin
Data in cubic meters per second

% OF TIME EXCEEDED	O1DS	O5US	O5DS	O7DS	O9US	O10US
100	0.0	0.0	0.0	0.0	0.0	0.0
50	0.8	3.0	4.0	1.2	1.8	6.0
20	2.4	9.4	12.1	3.8	5.3	18.2
10	3.9	15.6	20.1	6.3	8.8	30.3
5	5.7	22.4	28.6	8.9	12.4	42.8
2	9.5	36.9	46.8	14.6	19.9	69.4
1	13.7	53.0	66.5	20.7	28.0	98.0
0.5	19.0	73.2	91.2	28.3	37.9	133.4
0.2	28.9	111.3	138.5	42.6	56.8	201.5
0.1	38.5	149.5	187.6	57.5	77.1	273.4
0.05	49.3	193.0	243.2	74.3	100.1	354.9
0.02	62.5	245.3	309.8	94.5	127.5	452.2

% OF TIME EXCEEDED	O10DS	O15US	O16DS	O17DS	O18DS
100	0.0	0.0	0.0	0.0	0.0
50	6.4	7.4	12.6	13.6	20.0
20	19.2	22.5	37.1	39.8	58.7
10	32.1	37.5	61.9	66.7	98.2
5	45.0	52.9	85.5	91.5	135.2
2	72.5	85.7	135.5	144.1	213.6
1	101.7	120.8	186.9	197.5	294.0
0.5	137.5	164.3	248.5	260.9	389.8
0.2	206.5	244.0	363.0	379.0	571.1
0.1	280.8	324.5	486.2	508.5	771.1
0.05	364.8	416.3	626.3	655.7	998.0
0.02	465.1	527.5	795.2	832.9	1270.0

Table 3.5.14 (sheet 1 of 3)

Sub-Basin Parameters
Gumain/Porac Basin

Sub-Basin ID	G1	G2	G3	G4	G5	G6	G7
Physical Parameters							
Area (km2)	9.4	16.7	6.8	23.0	11.3	13.4	13.2
Longest Flow Path within Sub-Basin (km)	4.3	8.3	3.9	9.0	7.0	7.8	8.5
Elevation Change along Flow Path (m)	665	865	925	575	700	1,323	900
Channel Length to Upstream Basin (km)	N/A	7.9	N/A	7.4	4.7	N/A	N/A
Elevation Change along Channel (m)	N/A	290	N/A	290	60	N/A	N/A
Design Storm Parameters							
Estimated Annual Rainfall (mm)	4,300	3,900	4,350	4,050	3,600	4,200	4,150
Percent of Rainfall near Pinatubo Summit	86	78	87	81	72	84	83
Distance to Assumed Storm Center (km)							
Storm 1 at Sub-Basin G7	10.4	10.0	6.2	6.7	8.7	2.5	0.0
Storm 2 at Sub-Basin G11	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Runoff Parameters							
Time of Concentration (hrs)	0.4	0.8	0.3	1.0	0.7	0.6	0.8
Clark Storage Coefficient (hrs)	10.0	20.0	7.5	25.0	17.5	15.0	20.0
Uniform Infiltration Loss (mm/hr)	3	3	3	3	3	3	3
Baseflow (cms)	1.2	1.7	0.9	2.6	1.0	1.6	1.5
Flow Routing Parameters							
Travel Time of Flood Wave (hrs)	N/A	0.9	N/A	0.8	0.5	N/A	N/A
Number of Sub-Reaches (1-hr time step)	N/A	1	N/A	1	1	N/A	N/A

Table 3.5.14 (sheet 2 of 3)

Gumain/Porac Basin

Sub-Basin ID	G8	G9	G10	G11	G12	G13	G14
Physical Parameters							
Area (km2)	19.3	6.3	21.6	11.7	29.4	10.3	18.0
Longest Flow Path within Sub-Basin (km)	11.9	6.4	12.8	7.8	13.2	6.4	10.3
Elevation Change along Flow Path (m)	1,050	190	55	860	565	485	295
Channel Length to Upstream Basin (km)	9.2	5.3	9.5	N/A	11.5	N/A	5.8
Elevation Change along Channel (m)	210	30	20	N/A	165	N/A	30
Design Storm Parameters							
Estimated Annual Rainfall (mm)	3,650	3,300	3,300	3,750	3,150	3,300	3,150
Percent of Rainfall near Pinatubo Summit	73	66	66	75	63	66	63
Distance to Assumed Storm Center (km)							
Storm 1 at Sub-Basin G7	4.4	14.2	18.4	4.4	11.9	9.4	13.8
Storm 2 at Sub-Basin G11	N/A	N/A	N/A	0.0	8.1	6.2	10.9
Runoff Parameters							
Time of Concentration (hrs)	1.1	1.1	3.8	0.8	1.6	0.7	1.6
Clark Storage Coefficient (hrs)	27.5	27.5	95.0	20.0	40.0	17.5	40.0
Uniform Infiltration Loss (mm/hr)	3	3	3	3	3	3	3
Baseflow (cms)	1.7	0.4	1.5	1.1	1.8	0.7	1.1
Flow Routing Parameters							
Travel Time of Flood Wave (hrs)	1.0	0.6	1.1	N/A	1.3	N/A	0.6
Number of Sub-Reaches (1-hr time step)	1	1	1	N/A	1	N/A	1

Table 3.5.14 (sheet 3 of 3)

Sub-Basin Parameters
Gumain/Porac Basin

Sub-Basin ID	G15	G16	G17	G18	G19
Physical Parameters					
Area (km2)	14.3	20.5	18.2	36.6	2.3
Longest Flow Path within Sub-Basin (km)	8.3	9.3	12.3	14.4	7.9
Elevation Change along Flow Path (m)	940	535	175	180	5
Channel Length to Upstream Basin (km)	N/A	6.6	7.7	5.8	7.9
Elevation Change along Channel (m)	N/A	115	35	5	5
Design Storm Parameters					
Estimated Annual Rainfall (mm)	3,600	3,450	3,150	3,150	3,150
Percent of Rainfall near Pinatubo Summit	72	69	63	63	63
Distance to Assumed Storm Center (km)					
Storm 1 at Sub-Basin G7	5.3	9.7	15.6	18.1	27.8
Storm 2 at Sub-Basin G11	2.8	7.5	13.8	17.2	N/A
Runoff Parameters					
Time of Concentration (hrs)	0.8	1.1	2.4	2.8	0.9
Clark Storage Coefficient (hrs)	20.0	27.5	60.0	70.0	22.5
Uniform Infiltration Loss (mm/hr)	3	3	3	3	1000
Baseflow (cms)	1.2	1.6	1.1	2.2	0.1
Flow Routing Parameters					
Travel Time of Flood Wave (hrs)	N/A	0.7	0.9	0.6	0.9
Number of Sub-Reaches (1-hr time step)	N/A	1	1	1	1

Table 3.5.15
Simulation Output Sites
Gumain/Porac Basin

Site ID	Corps-Specified Site	Site Description	Stream Elevation (m)	Upstream Basin Area (km2)	Time of Concen-tration (hours)	Average Annual Basin Rai (mm)	Average Annual Flow (cms)	Critical Storm Location
G9DS	Yes	Gumain River at streamflow gage W086A	30	119.5	2.4	3,950	9.0	G7
G10DS	Yes	Gumain River above confluence with Porac River	10	141.1	3.5	3,850	10.2	G7
G12DS	Yes	Porac River approximately 13.5 km above streamflow gage W084A	75	41.1	2.0	3,320	2.3	G11
G17DS	Yes	Porac River at streamflow gage W084A	15	122.4	3.6	3,320	6.8	G11
G18DS	Yes	Porac River above confluence with Gumain River	10	159.0	4.3	3,280	8.6	G11
G19US	Yes	Gumain/Porac River above levee	10	300.1	4.3	3,550	18.8	G7
G19DS	Yes	Gumain/Porac River below levee	5	302.4	5.4	3,550	18.8	G7

Table 3.5.16

Computed Maximum Annual Peak Discharge and Volume Frequency Data
Gumain/Porac Basin

Maximum Annual Peak Discharge (cms)

Site ID	2-year	10-year	50-year	100-year	500-year
G9DS	220	470	700	801	1,048
G10DS	228	485	723	828	1,083
G12DS	43	95	145	166	219
G17DS	128	283	430	494	651
G18DS	146	320	488	561	740
G19US	367	794	1,196	1,371	1,801
G19DS	366	790	1,191	1,366	1,793

Maximum Annual 24-hr Volume (dam3)

Site ID	2-year	10-year	50-year	100-year	500-year
G9DS	16,200	31,800	47,200	53,900	70,300
G10DS	16,900	33,300	49,500	56,600	73,800
G12DS	3,400	7,000	10,600	12,200	16,000
G17DS	10,000	20,700	31,400	36,000	47,400
G18DS	11,500	23,400	36,300	41,700	54,900
G19US	28,000	56,400	84,800	97,200	127,300
G19DS	28,000	56,300	84,700	97,000	127,100

Maximum Annual 3-Day Volume (dam3)

Site ID	2-year	10-year	50-year	100-year	500-year
G9DS	28,600	57,300	85,700	98,000	127,700
G10DS	30,500	61,100	91,800	105,100	137,200
G12DS	6,600	13,800	21,400	24,600	32,600
G17DS	19,400	40,300	62,500	72,200	95,600
G18DS	24,000	48,000	74,900	86,500	114,600
G19US	52,800	107,700	164,500	189,300	248,900
G19DS	52,700	107,000	164,500	189,300	248,900

Table 3.5.17

Computed Flow Duration Curves
Gumain/Porac Basin
Data in cubic meters per second

% OF TIME EXCEEDED	G9DS	G10DS	G12DS	G17DS
100	0.0	0.0	0.0	0.0
50	3.6	4.1	0.9	2.7
20	11.6	13.0	2.9	8.7
10	21.0	23.6	4.8	14.3
5	36.6	40.7	7.1	21.0
2	64.6	71.0	12.0	35.3
1	92.0	99.8	17.5	51.7
0.5	125.3	133.5	24.7	72.7
0.2	187.9	196.2	40.0	117.7
0.1	254.3	266.0	55.1	162.6
0.05	321.3	336.3	70.3	207.9
0.02	392.2	410.7	86.3	255.3

% OF TIME EXCEEDED	G18DS	G19US	G19DS
100	0.0	0.0	0.0
50	3.4	7.5	7.5
20	10.7	24.0	24.0
10	17.7	39.6	39.6
5	25.6	58.4	58.4
2	42.4	98.4	98.4
1	61.1	144.2	144.2
0.5	84.7	203.2	203.2
0.2	129.9	327.1	326.9
0.1	175.7	447.1	446.5
0.05	227.6	568.1	567.2
0.02	289.7	695.6	694.3

Table 3.5.18
Sub-Basin Parameters
Pasig-Potrero Basin

Sub-Basin ID	P0	P1	P2	P3	P4	P5	P6	P7
Physical Parameters								
Area (km2)	21.3	9.3	4.4	6	3.1	12.6	17.7	2.4
Longest Flow Path within Sub-Basin (km)	5.9	8.2	4	8	4	9.5	11.5	14
Elevation Change along Flow Path (m)	460	772	480	420	65	135	180	40
Channel Length to Upstream Basin (km)	N/A	6.5	N/A	N/A	4	9.5	4.5	14
Elevation Change along Channel (m)	N/A	140	N/A	N/A	65	135	65	35
Design Storm Parameters								
Estimated Annual Rainfall (mm)	4,670	3,500	3,500	3,000	2,500	2,300	1,950	1,950
Percent of Rainfall near Pinatubo Summit	93	70	65	55	50	42	40	39
Distance to Assumed Storm Center (km)								
Storm 1 at Sub-Basin P0	0	4.2	6.7	8.7	9.7	15	20.8	28
Storm 2 at Sub-Basin P3	N/A	N/A	N/A	0	N/A	N/A	N/A	N/A
Runoff Parameters								
Time of Concentration (hrs)	0.7	0.8	0.4	1	0.9	1.9	2.2	4.8
Clark Storage Coefficient (hrs)	17.5	20.8	11	25.5	23.5	48.3	55	120
Uniform Infiltration Loss (mm/hr)	3	3	3	3	3	3	3	1000
Baseflow (cms)	3.1	0.7	0.3	0.3	0.1	0	0	0
Flow Routing Parameters								
Travel Time of Flood Wave (hrs)	N/A	0.7	N/A	N/A	0.4	1.1	0.5	1.6
Number of Sub-Reaches (1-hr time step)	N/A	1	N/A	N/A	1	1	1	2

Table 3.5.19
Simulation Output Sites
Pasig-Potrero Basin

Site ID	Corps-Specified Site	Site Description	Stream Elevation (m)	Upstream Basin area (km2)	Time of Concen-tration (hours)	Average Annual Basin Rain (mm)	Average Annual Flow (cms)	Critical Storm Location	Critical Storm ID
P0DS	Yes	Upper sub-basin of the Sacobia captured by Pasig	640	21.3	0.7	4,670	2.1	P0	1
P1DS	Yes	Bucbuc River above confluence with Yangca River	300	30.6	1.5	4,310	2.7	P0	1
P2DS	Yes	Yangca River above confluence with Bucbuc River	300	4.4	0.4	3,500	0.27	P0	1
P3DS	Yes	Timbu River above confluence with Papatac River	240	6	1	3,030	0.28	P3	2
P4US	Yes	Papatac River below confluence of Bucbuc and Yangca Rivers	300	35	1.5	4,210	2.9	P0	1
P4DS	Yes	Papatac River above confluence with Timbu River	240	38.1	2	4,080	3	P0	1
P5US	Yes	Pasig River below confluence of Papatac and Timbu Rivers	240	44.1	2	3,930	3.3	P0	1
P5DS	Yes	Pasig River at Mancatian Bridge	100	56.6	3.1	3,780	4	P0	1
P7DS	Yes	Potrero River above confluence with Guagua River	0	76.7	5.5	3,150	4.9	P0	1

Table 3.5.20

Computed Maximum Annual Peak Discharge and Volume Frequency Data
Pasig-Potrero Basin

Site I D	Maximum Annual Peak Discharge (cms)				
	2-year	10-year	50-year	100-year	500-yr
P0DS	55	112	166	189	246
P1DS	69	143	211	241	314
P2DS	8.5	19	28	32	42
P3DS	5.9	13	21	24	32
P4US	77	161	238	272	355
P4DS	79	166	247	282	369
P5US	84	178	265	304	397
P5DS	87	186	279	319	419
P7DS	90	195	294	337	444

Site I D	Maximum Annual 24-hr Volume (cubic decameters)				
	2-year	10-year	50-year	100-year	500-yr
P0DS	4,100	7,700	11,200	12,700	16,500
P1DS	5,100	9,800	14,400	16,400	21,300
P2DS	500	1,100	1,700	1,900	2,500
P3DS	400	900	1,500	1,700	2,200
P4US	5,700	10,900	16,000	18,300	23,700
P4DS	5,800	11,300	16,600	19,000	24,700
P5US	6,200	12,100	18,000	20,500	26,700
P5DS	6,400	12,800	19,100	21,900	28,600
P7DS	6,700	13,600	20,400	23,400	30,800

Site I D	Maximum Annual 3-Day Volume (cubic decameters)				
	2-year	10-year	50-year	100-year	500-yr
P0DS	7,400	13,900	20,200	23,000	29,600
P1DS	9,200	17,700	26,000	29,600	38,400
P2DS	800	1,600	2,500	2,900	3,900
P3DS	800	1,700	2,700	3,100	4,200
P4US	10,000	19,300	28,500	32,500	42,200
P4DS	10,300	19,900	29,600	33,800	44,000
P5US	10,900	21,400	32,000	36,600	47,800
P5DS	11,500	22,800	34,400	39,500	51,900
P7DS	12,100	24,500	37,400	43,000	57,000

Table 3.5.21
Computed Flow Duration Curves
Pasig-Potrero Basin
Data in cubic meters per second

% OF TIME EXCEEDED	P0DS	P1DS	P2DS	P3DS	P4US	P4DS	P5US	P5DS	P7DS
100.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
50.0	0.8	1.1	0.1	0.1	1.2	1.2	1.3	1.6	2.0
25.0	2.1	2.7	0.3	0.3	2.9	3.0	3.3	4.0	4.9
20.0	2.7	3.5	0.3	0.4	3.8	3.9	4.3	5.2	6.1
10.0	4.9	6.3	0.6	0.6	6.8	7.0	7.7	8.6	10.2
5.0	8.7	11.1	1.1	0.9	12.0	12.4	13.6	12.8	14.7
2.0	15.6	19.7	2.0	1.4	21.5	22.1	24.1	21.8	24.4
1.0	22.5	28.3	2.8	2.1	31.1	31.9	34.5	32.3	35.3
0.5	31.1	39.0	3.8	2.9	43.1	44.1	47.4	46.0	49.2
0.2	47.0	58.8	6.0	4.8	65.6	67.1	71.8	74.6	78.4
0.1	62.5	79.0	8.5	6.9	88.0	90.6	97.0	101.7	107.5
0.05	78.2	99.2	11.0	9.0	110.4	114.3	122.3	129.0	136.9
0.02	95.0	120.9	13.6	11.1	134.4	139.4	149.2	157.9	167.7

Table 3.5.22 (sheet 1 of 2)

Sub-Basin Parameters

Santo Tomas Basin

Sub-Basin ID	T1	T2	T3	T4	T5	T6
Physical Parameters						
Area (km2)	42.3	33.7	11.0	43.6	18.3	26.0
Longest Flow Path within Sub-Basin (km)	9.8	9.2	8.3	5.4	6.0	4.8
Elevation Change along Flow Path (m)	886	1027	220	714	690	721
Channel Length to Upstream Basin (km)	N/A	5.0	6.1	N/A	N/A	N/A
Elevation Change along Channel (m)	N/A	118	68	N/A	N/A	N/A
Design Storm Parameters						
Estimated Annual Rainfall (mm)	4,800	4,350	4,050	4,200	4,250	3,950
Percent of Rainfall near Pinatubo Summit	96	87	81	84	85	79
Distance to Assumed Storm Center (km)						
Storm 1 at Sub-Basin T1	0.0	5.7	9.4	10.4	14.6	16.8
Storm 2 at Sub-Basin T5	N/A	N/A	N/A	5.7	0.0	5.7
Runoff Parameters						
Time of Concentration (hrs)	1.0	0.9	1.4	0.5	0.6	0.5
Clark Storage Coefficient (hrs)	25.0	22.5	35.0	12.5	15.0	12.5
Uniform Infiltration Loss (mm/hr)	3	3	3	3	3	3
Baseflow (cms)	6.5	4.3	1.2	5.2	2.2	2.7
Flow Routing Parameters						
Travel Time of Flood Wave (hrs)	N/A	0.6	0.7	N/A	N/A	N/A
Number of Sub-Reaches (1-hr time step)	N/A	1	1	N/A	N/A	N/A

Table 3.5.22 (sheet 2 of 2)

Sub-Basin Parameters

Santo Tomas Basin

Sub-Basin ID	T7 (lake)	T8	T9	T10
Physical Parameters				
Area (km2)	8.0	42.3	31.4	4.9
Longest Flow Path within Sub-Basin (km)	N/A	16.2	15.6	8.4
Elevation Change along Flow Path (m)	N/A	330	481	110
Channel Length to Upstream Basin (km)	N/A	N/A	13.7	8.4
Elevation Change along Channel (m)	N/A	N/A	81	7
Design Storm Parameters				
Estimated Annual Rainfall (mm)	4,050	3,850	3,700	3,550
Percent of Rainfall near Pinatubo Summit	81	77	74	71
Distance to Assumed Storm Center (km)				
Storm 1 at Sub-Basin T1	14.0	13.7	17.1	24.4
Storm 2 at Sub-Basin T5	6.0	N/A	N/A	N/A
Runoff Parameters				
Time of Concentration (hrs)	0.0	2.5	2.1	1.8
Clark Storage Coefficient (hrs)	0.0	62.5	52.5	45.0
Uniform Infiltration Loss (mm/hr)	0	3	3	3
Baseflow (cms)	0.0	4.2	2.9	0.4
Flow Routing Parameters				
Travel Time of Flood Wave (hrs)	N/A	N/A	1.5	0.9
Number of Sub-Reaches (1-hr time step)	N/A	N/A	2	1

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Table 3.5.23
Simulation Output Sites
Santo Tomas Basin

Site ID	Corps-Specified Site	Site Description	Stream Elevation (m)	Upstream Basin Area (km2)	Time of Concen-tration (hours)	Average Annual Basin Rain (mm)	Average Annual Flow (cms)	Critical Storm Location
T2DS	Yes	Marella River headwater basin	158	76.0	1.5	4,600	7.3	T1
T3DS	Yes	Marella River upstream of confluence with Mapanuepe River	90	87.0	2.2	4,550	8.2	T1
T7DS	Yes	Mapanuepe River approximately 1.9 km downstream of Lake Mapanuepe	90	95.9	1.5	4,150	7.8	T5
T9US	Yes	Santo Tomas River at streamflow gage W094A below confluence of Marella and Mapanuepe Rivers	90	182.9	2.2	4,300	15.8	T1
T10DS	Yes	Santo Tomas River at mouth	2	261.5	5.6	4,150	21.4	T1

Table 3.5.24

Computed Maximum Annual Peak Discharge and Volume Frequency Data
Santo Tomas Basin

Maximum Annual Peak Discharge (cms)

Site ID	2-year	10-year	50-year	100-year	500-year
T2DS	156	320	472	538	699
T3DS	169	347	512	584	760
T7DS	84	172	259	296	384
T9US	224	457	684	783	1,023
T10DS	285	578	867	993	1,297

Maximum Annual 24-hr Volume (dam3)

Site ID	2-year	10-year	50-year	100-year	500-year
T2DS	12,300	23,200	34,000	38,600	50,000
T3DS	13,400	25,400	37,200	42,400	54,900
T7DS	7,000	14,500	21,800	25,000	32,500
T9US	18,600	36,800	54,700	62,500	81,300
T10DS	23,600	47,000	70,100	80,200	104,400

Maximum Annual 3-Day Volume (dam3)

Site ID	2-year	10-year	50-year	100-year	500-year
T2DS	24,100	45,700	66,900	76,000	98,200
T3DS	26,400	50,500	74,000	84,200	108,800
T7DS	18,100	37,600	56,600	64,900	84,600
T9US	42,100	84,200	125,300	143,200	186,100
T10DS	54,300	109,300	163,500	187,000	243,500

Table 3.5.25

Computed Flow Duration Curves
Santo Tomas Basin
Data in cubic meters per second

% OF TIME EXCEEDED	T2DS	T3DS	T7DS	T9US	T10DS
100	0.0	0.0	0.0	0.0	0.0
50	2.9	3.3	3.1	6.3	8.6
20	9.3	10.7	9.0	19.1	25.6
10	16.9	17.6	15.1	31.8	42.7
5	29.2	26.4	20.4	44.9	59.6
2	51.1	45.2	31.6	72.7	95.4
1	71.9	67.2	42.7	102.6	133.2
0.5	96.6	95.9	55.4	139.4	179.2
0.2	140.9	153.7	79.8	208.1	265.1
0.1	188.2	205.6	108.4	278.7	355.7
0.05	235.7	257.8	140.8	358.9	458.6
0.02	286.6	313.5	179.5	455.7	582.5

Table 3.5.26 (sheet 1 of 2)

Sub-Basin Parameters

Bucao Basin

Sub-Basin ID	B1	B2	B3	B4	B5	B6	B7	B8
Pysical Parameters								
Area (km2)	72.3	43.8	10.6	23.7	67.0	39.1	17.3	43.9
Longest Flow Path within Sub-Basin (km)	16.8	15.2	9.2	8.3	13.9	14.3	12.9	15.6
Elevation Change along Flow Path (m)	940	1,165	355	875	980	620	860	700
Channel Length to Upstream Basin (km)	N/A	N/A	N/A	2.9	12.5	N/A	N/A	5.8
Elevation Change along Channel (m)	N/A	N/A	N/A	35	105	N/A	N/A	80
Design Storm Parameters								
Estimated Annual Rainfall (mm)	4,100	4,200	4,050	4,200	4,200	3,900	4,100	4,050
Percent of Rainfall near Pinatubo Summit	82	84	81	84	84	78	82	81
Distance to Assumed Storm Center (km)								
Storm 1 at Sub-Basin B1	3.2	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Storm 2 at Sub-Basin B2	6.0	2.5	4.8	10.5	13.6	9.2	5.4	7.6
Storm 3 at Sub-Basin B3	N/A	N/A	1.2	N/A	N/A	N/A	N/A	N/A
Storm 4 at Sub-Basin B6	N/A	N/A	N/A	N/A	N/A	2.4	N/A	N/A
Storm 5 at Sub-Basin B7	N/A	N/A	N/A	N/A	N/A	N/A	1.6	N/A
Storm 6 at Sub-Basin B11	N/A	N/A	N/A	N/A	N/A	14.6	16.2	14.0
Runoff Parameters								
Time of Concentration (hrs)	1.8	1.4	1.3	.8	1.4	1.7	1.3	1.8
Clark Storage Coefficient (hrs)	45.0	35.0	32.5	20.0	35.0	42.5	32.5	45.0
Uniform Infiltration Loss (mm/hr)	3	3	3	3	3	3	3	3
Baseflow (m^3/s)	8.3	5.3	1.2	2.8	8.0	4.0	2.0	4.9
Flow Routing Parameters								
Travel Time (hrs)	N/A	N/A	N/A	0.3	1.4	N/A	N/A	0.6
Number of Sub-Reaches	N/A	N/A	N/A	0	1	N/A	N/A	1

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Table 3.5.26 (sheet 2 of 2)

Sub-Basin Parameters

Bucao Basin

Sub-Basin ID	B9	B10	B11	B12	B13	B14	B15	B16
Pysical Parameters								
Area (km2)	25.2	72.1	56.9	36.5	10.0	29.4	63.4	45.3
Longest Flow Path within Sub-Basin (km)	11.1	14.0	16.4	8.3	5.6	11.7	14.7	12.5
Elevation Change along Flow Path (m)	234	945	1,535	325	160	821	1,655	655
Channel Length to Upstream Basin (km)	7.9	N/A	N/A	5.8	4.0	5.9	N/A	5.7
Elevation Change along Channel (m)	65	N/A	N/A	40	10	30	N/A	10
Design Storm Parameters								
Estimated Annual Rainfall (mm)	4,000	4,050	4,500	4,050	4,050	4,050	4,600	4,100
Percent of Rainfall near Pinatubo Summit	80	81	90	81	81	81	92	82
Distance to Assumed Storm Center (km)								
Storm 1 at Sub-Basin B1	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Storm 2 at Sub-Basin B2	12.1	17.8	21.6	17.8	18.1	21.0	24.4	26.7
Storm 3 at Sub-Basin B3	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Storm 4 at Sub-Basin B6	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Storm 5 at Sub-Basin B7	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Storm 6 at Sub-Basin B11	12.1	6.4	2.8	6.7	10.2	N/A	N/A	N/A
Runoff Parameters								
Time of Concentration (hrs)	1.9	1.4	1.4	1.2	1.0	1.2	1.2	1.4
Clark Storage Coefficient (hrs)	47.5	35.0	35.0	30.0	25.0	30.0	30.0	35.0
Uniform Infiltration Loss (mm/hr)	3	3	3	3	3	3	3	3
Baseflow (m^3/s)	2.7	8.0	7.8	4.1	1.1	3.3	9.1	5.2
Flow Routing Parameters								
Travel Time (hrs)	0.9	N/A	N/A	0.6	0.4	0.7	N/A	0.6
Number of Sub-Reaches	1	N/A	N/A	1	0	1	N/A	1

Table 3.5.27
Simulation Output Sites
Bucao Basin

Site ID	Corps-Specified Site	Site Description	Stream Elevation (m)	Upstream Basin Area (km2)	Time of Concen-tration (hours)	Average Annual Basin Rain (mm)	Average Annual Flow (cms)	Critical Storm Location
B1DS	Yes	Balin-Buquero River headwater basin	180	72.3	1.8	4,100	5.8	B1
B2DS	Yes	Headwater basin originating as the Maronut River near Mt Pinatubo, channel direction shifted above basin B3 as a result of depositions	145	43.8	1.4	4,200	3.7	B2
B3DS	Yes	Maronut River downstream of channel shift noted in sub-basin B2.	145	10.6	1.3	4,200	0.9	B3
B5US	Yes	Balin-Buquero River below confluence with Maronut River	145	150.4	2.1	4,150	12.3	B2
B5DS	Yes	Balin-Buquero River above confluence with Bucao River	40	217.4	3.5	4,150	17.8	B2
B6DS	Yes	Bucao River headwater basin	200	39.1	1.7	3,900	2.9	B6
B7DS	Yes	Kayanga Creek above confluence with Bucao River	120	17.3	1.3	4,100	1.4	B7
B13DS	Yes	Bucao River above confluence with the Balin-Buquero River	40	301.0	4.0	4,100	24.1	B11
B14US	Yes	Bucao River below confluence with the Balin-Buquero River	40	518.4	4.0	4,150	42.4	B2
B16US	No	Bucao River at streamflow gage W093A	15	611.2	4.8	4,200	50.9	B2
B16DS	Yes	Bucao River at Hwy 7 bridge. (approximately 2.5 km upstream of South China Sea)	5	656.5	5.6	4,150	53.7	B2

Table 3.5.28

Computed Maximum Annual Peak Discharge and Volume Frequency Data

Bucao Basin

Maximum Annual Peak Discharge (cms)

Site ID	2-year	10-year	50-year	100-year	500-year
B1DS	86	175	260	297	387
B2DS	63	132	195	223	290
B3DS	15	32	48	55	72
B5US	200	421	626	716	933
B5DS	283	592	881	1,007	1,314
B6DS	45	93	139	160	208
B7DS	26	54	80	91	119
B13DS	376	787	1,173	1,342	1,752
B14US	643	1,345	2,007	2,296	2,998
B16US	780	1,623	2,420	2,768	3,613
B16DS	834	1,737	2,591	2,963	3,869

Maximum Annual 24-hr Volume (dam3)

Site ID	2-year	10-year	50-year	100-year	500-year
B1DS	7,100	13,900	20,600	23,400	30,400
B2DS	5,200	10,100	14,900	17,000	22,000
B3DS	1,300	2,500	3,600	4,100	5,400
B5US	16,400	32,200	47,500	54,200	70,400
B5DS	23,300	45,800	67,800	77,300	100,500
B6DS	3,700	7,300	10,900	12,500	16,200
B7DS	2,100	4,100	6,000	6,900	8,900
B13DS	31,000	61,200	90,700	103,500	134,700
B14US	53,000	104,700	155,200	177,200	230,600
B16US	64,100	126,300	187,100	213,600	277,800
B16DS	68,600	135,200	200,400	228,700	297,600

Maximum Annual 3-Day Volume (dam3)

Site ID	2-year	10-year	50-year	100-year	500-year
B1DS	16,100	32,000	47,500	54,300	70,500
B2DS	11,100	21,800	32,300	36,800	47,700
B3DS	2,600	5,100	7,600	8,700	11,300
B5US	34,800	69,300	103,000	117,600	152,800
B5DS	49,800	99,400	147,800	168,900	219,600
B6DS	8,200	16,500	24,700	28,300	36,800
B7DS	4,300	8,600	12,700	14,500	18,800
B13DS	66,900	134,100	199,800	228,400	297,200
B14US	114,000	229,200	341,700	390,600	508,500
B16US	137,500	275,200	409,800	468,400	609,400
B16DS	147,100	294,700	439,000	501,800	652,900

Table 3.5.29

Computed Flow Duration Curves
Bucao Basin
Data in cubic meters per second

% OF TIME EXCEEDED	B1DS	B2DS	B3DS	B5US	B5DS	B6DS
100	0.0	0.0	0.0	0.0	0.0	0.0
50	2.3	1.5	0.4	4.9	7.1	1.2
20	7.1	4.7	1.1	15.3	22.1	3.6
10	11.8	7.7	1.9	25.4	36.6	5.9
5	16.7	11.2	2.7	36.6	52.6	8.5
2	27.3	18.8	4.5	60.5	86.8	13.9
1	38.7	27.3	6.6	87.2	124.6	19.9
0.5	53.0	38.2	9.2	120.9	172.3	27.4
0.2	79.4	58.2	14.0	183.4	261.0	41.5
0.1	105.8	77.2	18.7	244.4	348.1	55.6
0.05	135.9	98.9	23.9	314.0	447.4	71.6
0.02	172.3	125.3	30.4	398.1	567.4	91.0

% OF TIME EXCEEDED	B7DS	B13DS	B14US	B16US	B16DS
100	0.0	0.0	0.0	0.0	0.0
50	0.6	9.6	17.0	20.4	21.5
20	1.8	29.8	52.0	62.6	66.2
10	3.0	49.4	86.4	103.9	109.9
5	4.4	70.8	123.2	148.3	157.2
2	7.4	116.4	201.5	243.0	258.2
1	10.8	166.6	287.2	346.7	369.3
0.5	15.2	229.8	394.2	476.5	508.7
0.2	24.3	347.5	593.6	717.9	768.0
0.1	32.8	464.3	793.7	958.7	1026.0
0.05	41.3	597.3	1021.4	1232.8	1319.6
0.02	50.4	757.9	1296.2	1564.0	1674.3

Table 3.5.30

Sub-Basin Parameters
Maloma Basin

Sub-Basin ID	M1	M2	M3	M4	M5	M6
Physical Parameters						
Area (km2)	6.0	68.2	23.2	20.8	18.7	13.0
Longest Flow Path within Sub-Basin (km)	6.2	17.8	7.9	7.7	11.5	6.4
Elevation Change along Flow Path (m)	430	325	401	845	620	383
Channel Length to Upstream Basin (km)	N/A	15.0	N/A	7.0	10.7	5.7
Elevation Change along Channel (m)	N/A	195	N/A	20	90	3
Design Storm Parameters						
Estimated Annual Rainfall (mm)	4,100	4,400	4,250	4,500	4,300	4,200
Percent of Rainfall near Pinatubo Summit	82	88	85	90	86	84
Distance to Assumed Storm Center (km)						
Storm 1 at Sub-Basin M1	0.0	N/A	N/A	N/A	N/A	N/A
Storm 2 at Sub-Basin M3	N/A	N/A	6.9	N/A	0.0	N/A
Storm 3 at Sub-Basin M4	15.0	7.2	7.7	0.0	2.5	5.3
Runoff Parameters						
Time of Concentration (hrs)	0.8	2.8	1.0	0.7	1.3	0.8
Clark Storage Coefficient (hrs)	20.0	70.0	25.0	17.5	32.5	20.0
Uniform Infiltration Loss (mm/hr)	3	3	3	3	3	3
Baseflow (cms)	0.7	9.0	2.9	2.9	2.4	1.6
Flow Routing Parameters						
Travel Time of Flood Wave (hrs)	N/A	1.7	N/A	0.8	1.2	0.6
Number of Sub-Reaches (1-hr time step)	N/A	1	N/A	1	1	1

Table 3.5.31
Simulation Output Sites
Maloma Basin

Site ID	Corps-Specified Site	Site Description	Stream Elevation (m)	Upstream Basin Area (km2)	Time of Concen-tration (hours)	Average Annual Basin Rain (mm)	Average Annual Flow (cms)	Critical Storm Location
M1DS	Yes	Headwater tributary of Maloma River approximately 0.25 km upstream of Payodpod	220	6.0	0.8	4,100	0.5	M1
M4DS	No	Maloma River upstream of confluence with Gorongoro River	5	74.2	4.1	4,400	6.7	M4
M5DS	No	Gorongoro tributary upstream of confluence with Maloma River	5	41.9	2.5	4,250	3.6	M5
M6DS	Yes	Maloma River at streamflow gage W099B	2	150.0	5.1	4,350	13.2	M4

Table 3.5.32

Computed Maximum Annual Peak Discharge and Volume Frequency Data
Maloma Basin

Maximum Annual Peak Discharge (cms)

Site ID	2-year	10-year	50-year	100-year	500-year
M1DS	12	26	38	44	57
M4DS	119	244	360	411	534
M5DS	69	143	212	242	315
M6DS	212	437	646	738	960

Maximum Annual 24-hr Volume (dam3)

Site ID	2-year	10-year	50-year	100-year	500-year
M1DS	900	1,800	2,600	3,000	3,900
M4DS	9,700	18,700	27,400	31,200	40,400
M5DS	5,500	10,700	15,700	17,900	23,200
M6DS	17,100	33,000	48,400	55,200	71,600

Maximum Annual 3-Day Volume (dam3)

Site ID	2-year	10-year	50-year	100-year	500-year
M1DS	1,700	3,300	4,800	5,500	7,200
M4DS	21,500	41,700	61,200	69,700	90,200
M5DS	11,100	21,700	32,100	36,600	47,500
M6DS	36,000	70,000	103,200	117,600	152,300

Table 3.5.33
Computed Flow Duration Curves
Maloma Basin
Data in cubic meters per second

% OF TIME EXCEEDED	M1DS	M4DS	M5DS	M6DS
100	0.0	0.0	0.0	0.0
50	0.2	2.7	1.4	5.3
20	0.6	8.5	4.6	16.3
10	1.2	14.1	7.6	27.1
5	2.0	20.6	11.3	38.9
2	3.6	34.6	19.1	64.0
1	5.2	50.5	28.2	91.7
0.5	7.1	71.0	39.9	126.7
0.2	10.7	112.4	63.6	190.4
0.1	14.4	150.8	85.5	252.1
0.05	18.2	189.4	107.5	322.6
0.02	22.2	230.6	131.0	408.2

Table 3.5.34 (sheet 1 of 2)

Estimated Peak Instantaneous Discharges and Confidence Limits at Hydrologic Output Sites

Site ID	HEC-2 Estimated Peak Discharge					Peak Discharge, Estimated 5% Confidence Limit					Peak Discharge, Estimated 95% Confidence Limit				
	2-Year (cms)	10-Year (cms)	50-Year (cms)	100-Year (cms)	500-Year (cms)	2-Year (cms)	10-Year (cms)	50-Year (cms)	100-Year (cms)	500-Year (cms)	2-Year (cms)	10-Year (cms)	50-Year (cms)	100-Year (cms)	500-Year (cms)
A1DS	10	23	35	40	52	12	30	48	54	67	8	18	24	25	26
A3DS	31	69	106	122	161	37	90	145	166	206	26	54	71	76	81
A5DS	41	90	138	159	211	49	117	188	216	270	34	70	93	99	106
B1DS	86	175	260	297	387	103	228	355	404	495	72	137	175	185	194
B2DS	63	132	195	223	290	75	172	266	303	371	53	103	131	139	146
B3DS	15	32	48	55	72	18	42	65	75	92	13	25	32	34	36
B5US	200	421	626	716	933	239	550	854	974	1,194	167	329	421	446	469
B5DS	283	592	881	1,007	1,314	339	773	1,202	1,370	1,681	236	463	593	627	660
B6DS	45	93	139	160	208	54	121	190	218	266	38	73	94	100	105
B7DS	26	54	80	91	119	31	70	109	124	152	22	42	54	57	60
B13DS	376	787	1,173	1,342	1,752	450	1,027	1,601	1,826	2,242	314	616	789	835	880
B14US	643	1,345	2,007	2,296	2,998	770	1,756	2,739	3,124	3,836	536	1,052	1,351	1,429	1,506
B16US	780	1,623	2,420	2,768	3,613	934	2,119	3,302	3,766	4,622	651	1,270	1,628	1,723	1,816
B16DS	834	1,737	2,591	2,963	3,869	999	2,268	3,535	4,031	4,950	696	1,359	1,743	1,844	1,944
G9DS	220	470	700	801	1,048	263	614	955	1,090	1,341	184	368	471	499	527
G10DS	228	485	723	828	1,083	273	633	987	1,127	1,386	190	380	487	515	544
G12DS	43	95	145	166	219	51	124	198	226	280	36	74	98	103	110
G17DS	128	283	430	494	651	153	369	587	672	833	107	221	289	308	327
G18DS	146	320	488	561	740	175	418	666	763	947	122	250	328	349	372
G19US	367	794	1,196	1,371	1,801	439	1,037	1,632	1,865	2,304	306	621	805	853	905
G19DS	366	790	1,191	1,366	1,793	438	1,031	1,625	1,859	2,294	305	618	801	850	901
M1DS	12	26	38	44	57	14	34	52	60	73	10	20	26	27	29
M4DS	119	244	360	411	534	142	319	491	559	683	99	191	242	256	268
M5DS	69	143	212	242	315	83	187	289	329	403	58	112	143	151	158
M6DS	212	437	646	738	960	254	571	881	1,004	1,228	177	342	435	459	482

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Table 3.5.34 (sheet 2 of 2)

Estimated Peak Instantaneous Discharges and Confidence Limits at Hydrologic Output Sites

Site ID	HEC-2 Estimated Peak Discharge					Peak Discharge, Estimated 5% Confidence Limit					Peak Discharge, Estimated 95% Confidence Limit				
	2-Year (cms)	10-Year (cms)	50-Year (cms)	100-Year (cms)	500-Year (cms)	2-Year (cms)	10-Year (cms)	50-Year (cms)	100-Year (cms)	500-Year (cms)	2-Year (cms)	10-Year (cms)	50-Year (cms)	100-Year (cms)	500-Year (cms)
O1DS	31	65	97	110	144	37	85	132	150	184	26	51	65	68	72
O5US	121	258	389	446	585	145	337	531	607	748	101	202	262	278	294
O5DS	150	321	490	563	741	180	419	669	766	948	125	251	330	350	372
O7DS	47	102	154	177	233	56	133	210	241	298	39	80	104	110	117
O9US	63	137	210	241	318	75	179	287	328	407	53	107	141	150	160
O10US	218	468	714	822	1,084	261	611	974	1,118	1,387	182	366	480	512	545
O10DS	223	477	730	841	1,110	267	623	996	1,144	1,420	186	373	491	524	558
O15US	264	546	815	932	1,216	316	713	1,112	1,268	1,556	220	427	548	580	611
O16DS	392	799	1,201	1,377	1,802	469	1,043	1,639	1,874	2,305	327	625	808	857	906
O17DS	409	830	1,251	1,435	1,881	490	1,084	1,707	1,952	2,407	341	649	842	893	945
O18DS	616	1,273	1,933	2,221	2,922	738	1,662	2,638	3,022	3,738	514	996	1,301	1,383	1,468
P1DS	15	32	49	56	73	18	42	67	76	93	13	25	33	35	37
P2DS	9	20	30	34	45	11	26	41	46	58	8	16	20	21	23
P3DS	6	13	21	24	32	7	17	29	33	41	5	10	14	15	16
P4US	24	52	78	90	118	29	68	106	122	151	20	41	52	56	59
P4DS	27	57	87	100	131	32	74	119	136	168	23	45	59	62	66
P5US	32	70	107	122	161	38	91	146	166	206	27	55	72	76	81
P5DS	35	79	121	140	186	42	103	165	190	238	29	62	81	87	93
P7DS	39	89	138	159	213	47	116	188	216	273	33	70	93	99	107
S1DS	55	112	166	189	246	66	146	227	257	315	46	88	112	118	124
S2DS	76	159	238	273	357	91	208	325	371	457	63	124	160	170	179
S3DS	83	175	262	300	394	99	228	357	408	504	69	137	176	187	198
S4DS	13	29	45	52	70	16	38	61	71	90	11	23	30	32	35
S6DS	62	138	211	243	321	74	180	288	331	411	52	108	142	151	161
S7US	153	332	505	580	764	183	433	689	789	977	128	260	340	361	384
S7DS	152	329	500	575	757	182	430	682	782	969	127	257	336	358	380
T2DS	156	320	472	538	699	187	418	644	732	894	130	250	318	335	351
T3DS	169	347	512	584	760	202	453	699	795	972	141	272	345	364	382
T7DS	84	172	259	296	384	101	225	353	403	491	70	135	174	184	193
T9US	224	457	684	783	1,023	268	597	933	1,065	1,309	187	358	460	487	514
T10DS	285	578	867	993	1,297	341	755	1,183	1,351	1,659	238	452	583	618	652

Table 3.5.35 (sheet 1 of 2)

Estimated Maximum Annual 24-Hour Volume and Confidence Limits at Hydrologic Output Sites

Site ID	HEC-2 Estimated 24-Hour Volume					24-Hour Volume, Estimated 5% Confidence Limit					24-Hour Volume, Estimated 95% Confidence Limit				
	2-Year (dam^3)	10-Year (dam^3)	50-Year (dam^3)	100-Year (dam^3)	500-Year (dam^3)	2-Year (dam^3)	10-Year (dam^3)	50-Year (dam^3)	100-Year (dam^3)	500-Year (dam^3)	2-Year (dam^3)	10-Year (dam^3)	50-Year (dam^3)	100-Year (dam^3)	500-Year (dam^3)
A1DS	800	1,600	2,400	2,800	3,600	958	2,089	3,275	3,810	4,606	667	1,252	1,615	1,743	1,809
A3DS	2,500	5,100	7,800	9,000	11,900	2,994	6,658	10,643	12,245	15,225	2,085	3,991	5,249	5,603	5,980
A5DS	3,100	6,500	10,000	11,500	15,300	3,712	8,486	13,645	15,647	19,575	2,586	5,086	6,729	7,159	7,688
B1DS	7,099	13,919	20,553	23,430	30,438	8,500	18,171	28,045	31,879	38,942	5,921	10,892	13,830	14,585	15,295
B2DS	5,221	10,121	14,899	16,983	22,032	6,252	13,213	20,330	23,107	28,188	4,355	7,920	10,026	10,572	11,071
B3DS	1,254	2,452	3,621	4,130	5,367	1,502	3,201	4,941	5,619	6,867	1,046	1,919	2,437	2,571	2,697
B5US	16,403	32,152	47,507	54,189	70,424	19,641	41,974	64,823	73,730	90,100	13,682	25,159	31,967	33,733	35,388
B5DS	23,330	45,828	67,779	77,346	100,544	27,935	59,828	92,484	105,237	128,636	19,460	35,860	45,608	48,148	50,523
B6DS	3,700	7,346	10,908	12,458	16,223	4,430	9,590	14,884	16,950	20,756	3,086	5,748	7,340	7,755	8,152
B7DS	2,097	4,084	6,022	6,863	8,917	2,511	5,332	8,217	9,338	11,408	1,749	3,196	4,052	4,272	4,481
B13DS	31,032	61,210	90,701	103,534	134,687	37,158	79,910	123,762	140,868	172,319	25,884	47,897	61,033	64,450	67,680
B14US	52,987	104,689	155,204	177,211	230,593	63,447	136,671	211,776	241,113	295,021	44,196	81,919	104,437	110,314	115,873
B16US	64,131	126,315	187,114	213,596	277,846	76,790	164,904	255,317	290,619	355,476	53,492	98,841	125,909	132,964	139,618
B16DS	68,594	135,229	200,375	228,743	297,584	82,134	176,541	273,412	311,228	380,729	57,214	105,817	134,832	142,393	149,536
G9DS	16,200	31,800	47,200	53,900	70,300	19,398	41,515	64,404	73,336	89,942	13,512	24,884	31,761	33,553	35,326
G10DS	16,900	33,300	49,500	56,600	73,800	20,236	43,473	67,543	77,010	94,420	14,096	26,057	33,309	35,234	37,085
G12DS	3,400	7,000	10,600	12,200	16,000	4,071	9,139	14,464	16,599	20,470	2,836	5,478	7,133	7,595	8,040
G17DS	10,000	20,700	31,400	36,000	47,400	11,974	27,024	42,845	48,982	60,644	8,341	16,198	21,129	22,410	23,819
G18DS	11,500	23,400	36,300	41,700	54,900	13,770	30,549	49,531	56,737	70,239	9,592	18,311	24,426	25,958	27,587
G19US	28,000	56,400	84,800	97,200	127,300	33,527	73,630	115,710	132,250	162,868	23,355	44,133	57,062	60,507	63,968
G19DS	28,000	56,300	84,700	97,000	127,100	33,527	73,500	115,573	131,978	162,612	23,355	44,055	56,995	60,383	63,868
M1DS	930	1,796	2,643	3,012	3,913	1,114	2,345	3,606	4,098	5,006	776	1,405	1,778	1,875	1,966
M4DS	9,751	18,695	27,403	31,189	40,393	11,676	24,406	37,391	42,436	51,679	8,133	14,629	18,439	19,415	20,297
M5DS	5,511	10,662	15,692	17,886	23,214	6,599	13,919	21,412	24,336	29,700	4,597	8,343	10,559	11,134	11,665
M6DS	17,120	32,971	48,428	55,156	71,511	20,499	43,044	66,080	75,045	91,491	14,280	25,800	32,587	34,335	35,934

Table 3.5.35 (sheet 2 of 2)

Estimated Maximum Annual 24-Hour Volume and Confidence Limits at Hydrologic Output Sites

Site ID	HEC-2 Estimated 24-Hour Volume					24-Hour Volume, Estimated 5% Confidence Limit					24-Hour Volume, Estimated 95% Confidence Limit				
	2-Year (dam^3)	10-Year (dam^3)	50-Year (dam^3)	100-Year (dam^3)	500-Year (dam^3)	2-Year (dam^3)	10-Year (dam^3)	50-Year (dam^3)	100-Year (dam^3)	500-Year (dam^3)	2-Year (dam^3)	10-Year (dam^3)	50-Year (dam^3)	100-Year (dam^3)	500-Year (dam^3)
O1DS	2,593	5,049	7,442	8,480	11,010	3,105	6,591	10,155	11,538	14,086	2,163	3,951	5,008	5,279	5,533
O5US	9,894	19,809	29,707	34,010	44,471	11,847	25,861	40,535	46,274	56,896	8,253	15,501	19,990	21,171	22,347
O5DS	12,237	25,020	38,025	43,683	57,420	14,653	32,664	51,885	59,435	73,463	10,207	19,578	25,587	27,193	28,854
O7DS	3,785	7,629	11,525	13,222	17,344	4,532	9,960	15,726	17,990	22,190	3,157	5,970	7,755	8,231	8,715
O9US	5,132	10,498	15,990	18,381	24,187	6,145	13,705	21,818	25,009	30,945	4,281	8,215	10,760	11,442	12,154
O10US	17,780	36,523	55,685	64,033	84,300	21,290	47,681	75,982	87,123	107,853	14,830	28,579	37,470	39,861	42,361
O10DS	18,205	37,564	57,406	66,088	87,156	21,799	49,040	78,330	89,919	111,507	15,185	29,394	38,628	41,140	43,796
O15US	21,869	42,603	63,094	72,030	93,664	26,186	55,618	86,092	98,004	119,834	18,241	33,337	42,456	44,839	47,066
O16DS	32,357	64,222	96,099	109,985	143,602	38,744	83,842	131,127	149,646	183,724	26,989	50,254	64,665	68,466	72,160
O17DS	33,736	67,270	101,071	115,826	151,526	40,395	87,821	137,911	157,593	193,862	28,139	52,639	68,011	72,102	76,142
O18DS	50,606	102,570	155,345	178,386	234,218	60,596	133,905	211,968	242,712	299,659	42,210	80,261	104,532	111,045	117,695
P1DS	1,100	2,200	3,300	3,800	5,000	1,317	2,872	4,503	5,170	6,397	918	1,722	2,221	2,366	2,513
P2DS	600	1,200	1,800	2,000	2,700	718	1,567	2,456	2,721	3,454	500	939	1,211	1,245	1,357
P3DS	400	900	1,500	1,700	2,200	479	1,175	2,047	2,313	2,815	334	704	1,009	1,058	1,106
P4US	1,700	3,400	5,100	5,800	7,600	2,036	4,439	6,959	7,891	9,723	1,418	2,661	3,432	3,611	3,819
P4DS	1,900	3,800	5,700	6,600	8,600	2,275	4,961	7,778	8,980	11,003	1,585	2,974	3,836	4,109	4,322
P5US	2,300	4,600	7,100	8,100	10,700	2,754	6,005	9,688	11,021	13,690	1,918	3,600	4,778	5,042	5,377
P5DS	2,500	5,400	8,300	9,500	12,700	2,994	7,050	11,325	12,926	16,248	2,085	4,226	5,585	5,914	6,382
P7DS	2,800	6,200	9,600	11,100	14,900	3,353	8,094	13,099	15,103	19,063	2,335	4,852	6,460	6,910	7,487
S1DS	4,100	7,700	11,200	12,700	16,500	4,909	10,052	15,282	17,280	21,110	3,420	6,025	7,536	7,906	8,291
S2DS	5,900	11,400	17,000	19,400	25,300	7,065	14,883	23,197	26,396	32,369	4,921	8,921	11,439	12,077	12,713
S3DS	6,400	12,700	19,000	21,700	28,400	7,663	16,580	25,926	29,525	36,335	5,338	9,938	12,785	13,508	14,271
S4DS	1,100	2,300	3,600	4,200	5,500	1,317	3,003	4,912	5,715	7,037	918	1,800	2,422	2,615	2,764
S6DS	5,000	10,500	16,000	18,400	24,300	5,987	13,708	21,832	25,035	31,089	4,171	8,216	10,766	11,454	12,211
S7US	12,200	24,900	37,700	43,300	57,000	14,608	32,507	51,442	58,914	72,926	10,176	19,484	25,368	26,954	28,643
S7DS	12,100	24,800	37,600	43,200	56,900	14,489	32,376	51,305	58,778	72,798	10,093	19,406	25,301	26,892	28,592
T2DS	12,270	23,238	33,975	38,642	50,036	14,692	30,337	46,359	52,576	64,016	10,234	18,184	22,862	24,055	25,143
T3DS	13,366	25,423	37,236	42,372	54,893	16,004	33,190	50,809	57,651	70,230	11,149	19,893	25,056	26,377	27,584
T7DS	7,031	14,501	21,828	24,983	32,477	8,419	18,931	29,784	33,992	41,551	5,865	11,347	14,688	15,552	16,320
T9US	18,553	36,807	54,727	62,514	81,298	22,215	48,052	74,675	85,057	104,013	15,475	28,801	36,826	38,915	40,852
T10DS	23,608	47,044	70,146	80,176	104,403	28,268	61,416	95,714	109,087	133,573	19,691	36,812	47,201	49,910	52,463

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Table 3.5.36 (sheet 1 of 2)

Estimated Maximum 3-Day Volume and Confidence Limits at Hydrologic Output Sites

Site ID	HEC-2 Estimated 3-Day Volume					3-Day Volume, Estimated 5% Confidence Limit					3-Day Volume, Estimated 95% Confidence Limit				
	2-Year (dam^3)	10-Year (dam^3)	50-Year (dam^3)	100-Year (dam^3)	500-Year (dam^3)	2-Year (dam^3)	10-Year (dam^3)	50-Year (dam^3)	100-Year (dam^3)	500-Year (dam^3)	2-Year (dam^3)	10-Year (dam^3)	50-Year (dam^3)	100-Year (dam^3)	500-Year (dam^3)
A1DS	1,300	2,800	4,300	5,000	6,700	1,557	3,655	5,867	6,803	8,572	1,084	2,191	2,893	3,113	3,367
A3DS	4,900	10,300	16,200	18,800	25,000	5,867	13,447	22,105	25,579	31,985	4,087	8,060	10,901	11,703	12,563
A5DS	6,300	13,100	21,000	23,900	21,900	7,544	17,102	28,655	32,518	28,019	5,255	10,251	14,131	14,878	11,005
B1DS	16,104	32,032	47,533	54,246	70,505	19,283	41,818	64,859	73,807	90,204	13,432	25,065	31,985	33,768	35,429
B2DS	11,132	21,819	32,257	36,804	47,730	13,329	28,485	44,015	50,076	61,066	9,285	17,073	21,706	22,910	23,984
B3DS	2,589	5,146	7,643	8,729	11,343	3,100	6,718	10,429	11,877	14,512	2,159	4,027	5,143	5,434	5,700
B5US	34,825	69,309	102,962	117,583	152,847	41,699	90,483	140,492	159,983	195,552	29,048	54,234	69,283	73,195	76,806
B5DS	49,802	99,429	147,828	168,892	219,579	59,633	129,805	201,711	229,794	280,929	41,540	77,803	99,473	105,135	110,338
B6DS	8,174	16,526	24,707	28,263	36,846	9,788	21,575	33,713	38,455	47,141	6,818	12,932	16,625	17,594	18,515
B7DS	4,340	8,576	12,710	14,500	18,839	5,197	11,196	17,343	19,729	24,103	3,620	6,711	8,553	9,026	9,467
B13DS	66,895	134,064	199,807	228,368	297,224	80,100	175,021	272,637	310,718	380,268	55,797	104,905	134,450	142,159	149,355
B14US	114,060	229,161	341,669	390,617	508,472	136,575	299,170	466,207	531,473	650,539	95,137	179,318	229,909	243,159	255,507
B16US	137,539	275,206	409,807	468,360	609,360	164,689	359,281	559,182	637,251	779,615	114,721	215,349	275,759	291,554	306,203
B16DS	147,106	294,700	438,995	501,754	652,913	176,145	384,731	599,009	682,686	835,337	122,701	230,603	295,400	312,342	328,089
G9DS	28,600	57,300	85,700	98,000	127,700	34,246	74,805	116,938	133,339	163,379	23,855	44,837	57,668	61,005	64,169
G10DS	30,500	61,100	91,800	105,100	137,200	36,521	79,766	125,261	142,999	175,534	25,440	47,811	61,772	65,425	68,943
G12DS	6,600	13,800	21,400	24,600	32,600	7,903	18,016	29,200	33,471	41,708	5,505	10,799	14,400	15,314	16,382
G17DS	19,400	40,300	62,500	72,200	95,600	23,230	52,612	85,281	98,235	122,311	16,182	31,535	42,056	44,945	48,039
G18DS	24,000	48,000	74,900	86,500	114,600	28,738	62,664	102,201	117,692	146,619	20,018	37,560	50,400	53,846	57,587
G19US	52,800	107,700	164,500	189,300	248,900	63,223	140,602	224,460	257,562	318,443	44,040	84,275	110,692	117,839	125,072
G19DS	52,700	107,000	164,500	189,300	248,900	63,103	139,689	224,460	257,562	318,443	43,957	83,728	110,692	117,839	125,072
M1DS	1,667	3,271	4,840	5,518	7,162	1,996	4,270	6,604	7,508	9,163	1,390	2,560	3,257	3,435	3,599
M4DS	21,533	41,658	61,217	69,711	90,200	25,784	54,385	83,531	94,849	115,402	17,961	32,597	41,193	43,395	45,326
M5DS	11,123	21,749	32,121	36,638	47,507	13,319	28,393	43,829	49,850	60,780	9,278	17,019	21,614	22,807	23,872
M6DS	36,020	70,036	103,176	117,581	152,301	43,130	91,432	140,784	159,981	194,854	30,044	54,803	69,427	73,194	76,531

Table 3.5.36 (sheet 2 of 2)

Estimated Maximum 3-Day Volume and Confidence Limits at Hydrologic Output Sites

Site ID	HEC-2 Estimated 3-Day Volume					3-Day Volume, Estimated 5% Confidence Limit					3-Day Volume, Estimated 95% Confidence Limit				
	2-Year (dam^3)	10-Year (dam^3)	50-Year (dam^3)	100-Year (dam^3)	500-Year (dam^3)	2-Year (dam^3)	10-Year (dam^3)	50-Year (dam^3)	100-Year (dam^3)	500-Year (dam^3)	2-Year (dam^3)	10-Year (dam^3)	50-Year (dam^3)	100-Year (dam^3)	500-Year (dam^3)
O1DS	5,616	11,077	16,398	18,705	24,279	6,725	14,461	22,375	25,450	31,063	4,684	8,668	11,034	11,644	12,200
O5US	20,914	42,390	64,307	73,828	96,762	25,042	55,340	87,747	100,450	123,797	17,444	33,170	43,272	45,958	48,623
O5DS	26,280	54,082	83,563	96,379	127,249	31,468	70,604	114,022	131,133	162,802	21,920	42,319	56,230	59,996	63,943
O7DS	7,793	15,971	24,533	28,254	37,236	9,331	20,850	33,475	38,442	47,640	6,500	12,497	16,508	17,588	18,711
O9US	10,946	22,655	35,152	40,592	53,710	13,107	29,576	47,965	55,229	68,717	9,130	17,728	23,654	25,269	26,989
O10US	38,340	79,363	123,148	142,220	188,181	45,908	103,608	168,035	193,505	240,759	31,979	62,102	82,866	88,532	94,561
O10DS	39,629	82,501	128,337	148,447	196,916	47,452	107,705	175,116	201,977	251,934	33,055	64,557	86,358	92,408	98,950
O15US	48,234	95,225	141,884	162,030	211,084	57,755	124,316	193,601	220,458	270,061	40,232	74,514	95,474	100,864	106,070
O16DS	74,372	148,996	224,919	257,988	337,589	89,053	194,514	306,902	351,018	431,911	62,034	116,589	151,348	160,598	169,638
O17DS	78,266	157,592	239,134	274,762	360,440	93,716	205,736	326,298	373,841	461,147	65,282	123,316	160,913	171,039	181,121
O18DS	115,455	236,108	362,103	417,163	549,828	138,246	308,239	494,090	567,592	703,450	96,301	184,755	243,659	259,684	276,289
P1DS	1,900	4,000	6,000	6,900	9,100	2,275	5,222	8,187	9,388	11,643	1,585	3,130	4,037	4,295	4,573
P2DS	800	1,700	2,700	3,100	4,100	958	2,219	3,684	4,218	5,246	667	1,330	1,817	1,930	2,060
P3DS	800	1,700	2,700	3,100	4,200	958	2,219	3,684	4,218	5,373	667	1,330	1,817	1,930	2,111
P4US	2,800	5,700	8,700	10,000	13,100	3,353	7,441	11,871	13,606	16,760	2,335	4,460	5,854	6,225	6,583
P4DS	3,100	6,300	9,800	11,300	14,900	3,712	8,225	13,372	15,375	19,063	2,586	4,930	6,594	7,034	7,487
P5US	3,800	7,900	12,300	14,200	18,900	4,550	10,313	16,783	19,321	24,181	3,170	6,182	8,277	8,840	9,497
P5DS	4,300	9,300	14,800	17,200	23,100	5,149	12,141	20,195	23,402	29,554	3,587	7,277	9,959	10,707	11,608
P7DS	5,000	11,100	17,800	20,800	28,300	5,987	14,491	24,288	28,300	36,207	4,171	8,686	11,978	12,948	14,221
S1DS	7,400	13,900	20,200	23,000	29,600	8,861	18,146	27,563	31,294	37,870	6,172	10,877	13,593	14,318	14,874
S2DS	11,300	22,100	33,200	38,000	49,600	13,531	28,852	45,301	51,703	63,458	9,425	17,293	22,340	23,655	24,924
S3DS	12,500	24,800	37,500	43,100	56,500	14,968	32,376	51,169	58,642	72,286	10,426	19,406	25,234	26,830	28,391
S4DS	2,500	5,200	8,400	9,800	13,100	2,994	6,789	11,462	13,334	16,760	2,085	4,069	5,652	6,101	6,583
S6DS	10,200	21,400	33,400	38,600	51,200	12,213	27,938	45,574	52,519	65,505	8,508	16,746	22,475	24,029	25,728
S7US	24,600	50,500	78,000	90,000	119,000	29,456	65,928	106,431	122,454	152,249	20,519	39,516	52,486	56,025	59,798
S7DS	24,600	50,400	77,900	90,000	118,900	29,456	65,797	106,295	122,454	152,121	20,519	39,438	52,419	56,025	59,747
T2DS	24,118	45,746	66,888	76,051	98,171	28,879	59,721	91,269	103,475	125,600	20,117	35,796	45,009	47,342	49,331
T3DS	26,442	50,477	73,976	84,169	108,760	31,662	65,898	100,940	114,520	139,148	22,055	39,498	49,778	52,395	54,652
T7DS	18,060	37,557	56,637	64,908	84,614	21,625	49,031	77,281	88,314	108,255	15,064	29,388	38,111	40,405	42,519
T9US	42,136	84,159	125,318	143,202	186,067	50,454	109,870	170,996	194,841	238,054	35,146	65,854	84,326	89,143	93,499
T10DS	54,328	109,255	163,455	186,984	243,521	65,052	142,632	223,034	254,410	311,561	45,315	85,492	109,989	116,398	122,369

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Table 3.5.37 (sheet 1 of 2)

Flow Duration Curve, 5 Percent Confidence Limit Data at Hydrologic Sites (cms)

Site Number	Percent of Time Exceeded												
	100	50	25	20	10	5	2	1	0.50	0.20	0.10	0.05	0.02
A1DS	0	0.2	0.5	0.6	1.1	2.0	3.5	5.1	7.1	11.1	15.6	20.1	24.7
A3DS	0	0.8	1.9	2.5	4.1	6.0	10.2	14.9	21.1	34.9	49.3	63.8	78.9
A5DS	0	1.0	2.4	3.1	5.1	7.5	12.7	18.6	26.2	43.6	62.2	81.1	100.5
B1DS	0	2.8	7.0	8.5	14.1	19.9	32.3	45.6	62.1	94.5	129.5	169.1	216.1
B2DS	0	1.8	4.5	5.6	9.2	13.4	22.2	32.1	44.7	71.8	99.4	127.4	156.5
B3DS	0	0.4	1.1	1.3	2.2	3.2	5.4	7.7	10.7	17.3	24.0	30.8	37.9
B5US	0	5.9	14.8	18.3	30.4	43.5	71.6	102.6	141.6	218.3	299.2	390.7	499.1
B5DS	0	8.6	21.4	26.4	43.8	62.6	102.7	146.7	201.9	310.7	426.2	556.7	711.4
B6DS	0	1.4	3.5	4.3	7.1	10.1	16.5	23.5	32.1	49.4	68.1	89.1	114.0
B7DS	0	0.7	1.7	2.1	3.5	5.2	8.7	12.7	17.8	28.9	40.1	51.4	63.2
B13DS	0	11.6	29.0	35.6	59.1	84.2	137.7	196.2	269.2	413.8	568.5	743.2	950.2
B14US	0	20.4	51.0	62.2	103.4	146.6	238.6	338.2	461.8	706.9	971.8	1271.0	1625.2
B16US	0	24.5	61.3	74.8	124.3	176.5	287.6	408.2	558.3	854.8	1173.7	1534.0	1960.9
B16DS	0	25.9	64.6	79.1	131.5	187.1	305.6	434.7	595.8	914.5	1256.1	1642.1	2099.3
G9DS	0	4.3	10.8	13.9	25.2	43.7	76.8	108.8	147.2	223.5	311.1	399.6	491.7
G10DS	0	4.9	12.3	16.0	26.3	39.3	67.1	99.5	141.6	233.5	325.3	418.3	514.9
G12DS	0	1.1	2.8	3.5	5.8	8.5	14.2	20.6	28.9	47.6	67.4	87.5	108.2
G17DS	0	3.3	8.2	10.4	17.1	25.0	41.8	60.8	85.1	140.2	199.0	258.7	
G18DS	0	4.1	10.4	12.8	21.2	30.5	50.1	71.9	99.2	160.4	226.2	293.0	361.8
G19US	0	9.1	22.6	28.7	47.4	69.4	116.3	169.6	237.9	389.3	547.1	706.8	872.1
G19DS	0	9.1	22.6	28.7	47.4	69.4	116.3	169.6	237.9	389.1	546.4	705.7	870.6
M1DS	0	0.2	0.6	0.8	1.4	2.4	4.3	6.1	8.4	12.8	17.7	22.6	27.8
M4DS	0	3.2	8.1	10.2	16.8	24.5	40.9	59.4	83.1	133.7	184.3	235.5	289.1
M5DS	0	1.7	4.3	5.5	9.1	13.4	22.6	33.1	46.6	75.7	104.8	134.2	164.9
M6DS	0	6.4	15.9	19.5	32.4	46.2	75.7	108.0	148.4	226.7	308.6	401.3	511.8

Table 3.5.37 (sheet 2 of 2)

Flow Duration Curve, 5 Percent Confidence Limit Data at Hydrologic Sites (cms)

Site Number	Percent of Time Exceeded												
	100	50	25	20	10	5	2	1	0.50	0.20	0.10	0.05	0.02
O1DS	0	0.9	2.3	2.8	4.7	6.8	11.2	16.1	22.3	35.7	49.5	63.5	78.1
O5US	0	4.8	11.9	14.5	24.1	34.1	55.4	78.4	106.8	165.0	229.8	302.7	388.4
O5DS	0	4.8	11.9	14.5	24.1	34.1	55.4	78.4	106.8	165.0	229.8	302.7	388.4
O7DS	0	1.5	3.7	4.5	7.5	10.6	17.2	24.3	33.1	50.8	70.4	92.4	118.4
O9US	0	2.1	5.3	6.4	10.6	14.9	23.9	33.5	45.2	69.2	96.4	127.0	163.0
O10US	0	7.2	18.1	21.8	36.3	50.9	82.2	115.4	156.3	240.0	334.9	441.7	567.0
O10DS	0	7.7	19.3	23.0	38.4	53.6	85.8	119.8	161.1	246.1	344.0	454.1	583.1
O15US	0	9.0	22.4	26.9	44.9	63.0	101.4	142.3	192.5	290.5	397.3	518.0	661.4
O16DS	0	15.1	37.8	44.3	74.2	101.9	160.6	220.4	291.4	432.3	595.3	779.3	997.0
O17DS	0	16.4	40.9	47.7	79.9	109.1	170.9	233.0	306.0	451.3	622.6	815.9	1044.3
O18DS	0	24.0	60.1	70.3	117.6	161.1	253.3	346.7	457.1	680.4	944.5	1242.1	1592.3
P1DS	0	0.3	0.7	0.9	1.6	2.8	5.0	7.2	9.8	15.3	21.4	27.6	34.0
P2DS	0	0.1	0.3	0.4	0.8	1.4	2.5	3.7	5.2	8.3	11.7	15.1	18.6
P3DS	0	0.1	0.3	0.4	0.7	1.0	1.7	2.5	3.4	5.7	8.4	11.2	13.9
P4US	0	0.4	1.0	1.3	2.4	4.2	7.5	10.8	15.0	23.6	33.1	42.6	52.6
P4DS	0	0.4	1.1	1.5	2.6	4.7	8.3	12.1	16.7	26.4	36.9	47.7	58.8
P5US	0	0.6	1.5	1.9	3.4	6.0	10.6	15.1	20.6	31.9	44.7	57.7	71.1
P5DS	0	0.7	1.8	2.4	3.9	5.8	9.9	14.7	20.9	35.5	51.2	67.2	83.5
P7DS	0	0.8	2.1	2.7	4.4	6.6	11.2	16.6	23.5	40.1	58.4	77.0	95.9
S1DS	0	1.0	2.5	3.3	5.9	10.4	18.5	26.6	36.5	55.9	76.4	97.2	119.1
S2DS	0	1.8	4.5	5.7	9.5	14.0	23.8	35.1	49.7	81.1	112.1	143.5	176.3
S3DS	0	2.0	4.9	6.3	10.4	15.4	26.1	38.3	54.0	88.6	123.8	159.4	196.4
S4DS	0	0.4	1.1	1.3	2.2	3.1	5.0	7.1	9.6	14.9	21.0	27.8	35.7
S6DS	0	1.7	4.2	5.3	8.7	12.7	21.1	30.6	42.7	70.4	100.5	131.0	162.4
S7US	0	4.1	10.2	12.9	21.3	30.9	51.5	74.7	104.2	170.3	240.5	311.6	385.0
S7DS	0	4.1	10.2	12.8	21.2	30.8	51.3	74.2	103.4	169.1	239.2	310.2	383.5
T2DS	0	3.5	8.8	11.5	18.9	28.3	48.4	71.9	102.7	167.6	230.1	293.1	359.3
T3DS	0	3.9	9.9	12.8	21.1	31.4	53.4	79.0	112.2	182.8	251.3	320.5	393.1
T7DS	0	3.8	9.4	11.0	18.4	25.2	39.5	54.1	71.3	103.0	137.8	177.4	225.1
T9US	0	7.6	19.0	22.9	38.1	53.5	86.1	120.8	163.3	247.8	341.2	446.7	571.4
T10DS	0	10.3	25.8	30.6	51.1	71.0	113.1	157.0	210.0	315.8	435.5	570.7	730.3

TABLE 3.5.38 (sheet 1 of 2)

Flow Duration Curve, 95 Percent Confidence Limit Data at Hydrologic Sites (cms)

Site Number	Percent of Time Exceeded												
	100	50	25	20	10	5	2	1	0.50	0.20	0.10	0.05	0.02
A1DS	0	0.1	0.3	0.4	0.8	1.4	2.5	3.7	5.1	7.9	10.5	13.2	16.0
A3DS	0	0.5	1.3	1.7	2.9	4.3	7.3	10.8	15.4	24.8	33.3	42.0	51.1
A5DS	0	0.7	1.7	2.2	3.6	5.3	9.1	13.5	19.1	31.0	42.1	53.3	65.2
B1DS	0	1.9	4.9	6.0	9.9	14.1	23.1	33.0	45.3	67.1	87.6	111.1	140.1
B2DS	0	1.2	3.1	3.9	6.5	9.5	15.9	23.2	32.6	49.2	64.0	80.9	101.8
B3DS	0	0.3	0.8	0.9	1.6	2.3	3.8	5.6	7.8	11.8	15.4	19.6	24.7
B5US	0	4.1	10.3	12.9	21.3	30.9	51.3	74.2	103.3	155.0	202.4	256.7	323.5
B5DS	0	5.9	14.9	18.6	30.7	44.4	73.6	106.1	147.3	220.6	288.2	365.8	461.1
B6DS	0	1.0	2.4	3.0	5.0	7.2	11.8	17.0	23.5	35.1	46.0	58.6	73.9
B7DS	0	0.5	1.2	1.5	2.5	3.7	6.2	9.2	13.0	20.5	27.1	33.8	40.9
B13DS	0	8.1	20.1	25.0	41.5	59.7	98.7	141.9	196.5	293.7	384.4	488.3	615.9
B14US	0	14.2	35.4	43.7	72.5	103.9	170.8	244.4	337.0	501.8	657.1	835.0	1053.4
B16US	0	17.0	42.5	52.6	87.2	125.1	205.9	295.1	407.3	606.8	793.7	1007.9	1271.0
B16DS	0	17.9	44.9	55.7	92.3	132.7	218.9	314.3	434.8	649.2	849.5	1078.9	1360.7
G9DS	0	3.0	7.5	9.7	17.6	30.8	54.5	78.0	106.8	158.9	210.7	262.7	318.7
G10DS	0	3.4	8.5	10.9	19.7	34.2	59.9	84.5	113.8	166.0	220.3	275.0	333.8
G12DS	0	0.8	1.9	2.5	4.1	6.0	10.2	14.9	21.1	33.8	45.6	57.5	70.2
G17DS	0	2.3	5.7	7.3	12.0	17.7	30.0	44.0	62.2	99.5	134.6	170.0	207.5
G18DS	0	2.9	7.2	9.0	14.9	21.6	35.9	52.0	72.4	109.8	145.4	186.0	235.5
G19US	0	6.3	15.7	20.2	33.3	49.3	83.5	122.8	173.8	276.6	370.3	464.6	565.3
G19DS	0	6.3	15.7	20.2	33.3	49.3	83.5	122.8	173.8	276.4	369.9	463.8	564.3
M1DS	0	0.2	0.4	0.5	1.0	1.7	3.1	4.4	6.1	9.1	12.0	14.9	18.0
M4DS	0	2.2	5.6	7.2	11.8	17.4	29.4	43.1	60.7	91.6	118.6	149.6	188.1
M5DS	0	1.2	3.0	3.9	6.4	9.5	16.2	24.0	34.1	53.9	71.0	88.2	106.9
M6DS	0	4.4	11.0	13.7	22.7	32.8	54.2	78.1	108.3	161.0	208.8	263.8	331.8

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TABLE 3.5.38 (sheet 2 of 2)

Flow Duration Curve, 95 Percent Confidence Limit Data at Hydrologic Sites (cms)

Site Number	Percent of Time Exceeded												
	100	50	25	20	10	5	2	1	0.50	0.20	0.10	0.05	0.02
O1DS	0	0.6	1.6	2.0	3.3	4.8	8.0	11.7	16.3	24.5	31.8	40.3	50.8
O5US	0	2.5	6.4	7.9	13.1	18.9	31.3	45.1	62.5	94.0	123.8	157.8	199.3
O5DS	0	3.3	8.3	10.2	16.9	24.2	39.6	56.6	77.9	117.0	155.2	198.8	251.8
O7DS	0	1.0	2.6	3.2	5.3	7.5	12.3	17.6	24.2	36.0	47.6	60.7	76.8
O9US	0	1.5	3.7	4.5	7.4	10.5	17.1	24.2	33.0	49.1	65.1	83.4	105.6
O10US	0	5.0	12.5	15.3	25.4	36.1	58.8	83.4	114.0	170.2	226.2	290.1	367.5
O10DS	0	5.3	13.4	16.2	26.9	37.9	61.4	86.5	117.5	174.5	232.3	298.2	378.0
O15US	0	6.2	15.5	18.9	31.5	44.6	72.6	102.8	140.4	206.3	268.7	340.4	428.7
O16DS	0	10.5	26.2	31.1	52.0	72.0	114.6	158.9	212.3	306.8	402.5	512.0	646.2
O17DS	0	11.4	28.4	33.5	55.9	77.1	121.9	167.9	222.8	320.3	420.9	536.0	676.9
O18DS	0	16.7	41.7	49.3	82.4	113.9	180.8	249.9	332.9	482.6	638.2	815.8	1032.1
P1DS	0	0.2	0.5	0.6	1.1	2.0	3.6	5.2	7.1	10.8	14.5	18.1	22.1
P2DS	0	0.1	0.2	0.3	0.6	1.0	1.8	2.7	3.8	5.9	7.9	9.9	12.0
P3DS	0	0.1	0.2	0.3	0.5	0.7	1.2	1.8	2.5	4.1	5.7	7.3	9.0
P4US	0	0.3	0.7	0.9	1.7	2.9	5.3	7.8	10.9	16.8	22.4	28.0	34.1
P4DS	0	0.3	0.8	1.0	1.8	3.3	5.9	8.7	12.2	18.7	25.0	31.3	38.1
P5US	0	0.4	1.0	1.3	2.4	4.2	7.5	10.8	15.0	22.7	30.3	37.9	46.1
P5DS	0	0.5	1.3	1.6	2.9	5.0	8.8	12.5	16.8	25.2	34.6	44.1	54.1
P7DS	0	0.6	1.5	1.8	3.4	5.8	10.1	14.2	19.0	28.4	39.4	50.5	62.1
S1DS	0	0.7	1.8	2.3	4.1	7.3	13.1	19.0	26.5	39.8	51.9	64.0	77.2
S2DS	0	1.2	3.1	3.9	7.1	12.3	21.4	30.0	40.1	57.6	76.0	94.4	114.3
S3DS	0	1.4	3.4	4.5	7.3	11.0	18.7	27.7	39.5	63.0	83.8	104.8	127.3
S4DS	0	0.3	0.8	0.9	1.5	2.2	3.6	5.1	7.0	10.6	14.2	18.2	23.1
S6DS	0	1.2	2.9	3.7	6.1	9.0	15.2	22.2	31.2	49.9	67.9	86.1	105.2
S7US	0	2.8	7.1	9.1	14.9	22.0	36.9	54.1	76.1	120.9	162.7	204.8	249.6
S7DS	0	2.8	7.1	9.0	14.9	21.9	36.8	53.7	75.5	120.1	161.8	203.9	248.6
T2DS	0	2.4	6.1	7.8	14.2	24.6	43.1	61.0	82.3	119.3	156.0	192.8	232.9
T3DS	0	2.7	6.9	8.7	15.9	27.4	47.9	67.4	90.4	130.1	170.4	210.8	254.8
T7DS	0	2.6	6.5	7.5	12.7	17.2	26.8	36.3	47.3	67.4	89.7	115.1	145.9
T9US	0	5.3	13.2	16.1	26.7	37.9	61.6	87.3	119.1	175.9	230.7	293.4	370.4
T10DS	0	7.2	17.9	21.5	35.8	50.2	80.8	113.3	153.1	224.1	294.4	374.9	473.4

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Table 3.6.1

Estimated Depths of Clearwater flows

River	Reach of River Modeled (river km)	Maximum and minimum depths in meters estimated to occur within the reach for the 2- through 500-year floods. Following the min and max is the estimated average depth within the reach (in parentheses). Normal depth is indicated for reaches for which only one depth is shown.				
		2-year	10-year	50-year	100-year	500-year
Abacan	1.0	0.4	0.7	0.8	0.9	1.1
Abacan	8.0	0.4	0.7	0.9	0.9	1.1
Abacan						
Downstr. of Expressway Br.	16.7-17.6	0.3-0.6 (0.5)	0.5-0.8 (0.7)	0.6-1.0 (0.8)	0.6-1.1 (0.9)	0.7-1.3 (1.0)
Betw. Expr. and Friendship Br.	17.6-25.4	0.3-0.9 (0.5)	0.5-1.1 (0.7)	0.7-1.3 (0.9)	0.7-1.4 (1.0)	0.8-1.5 (1.1)
Upstream of Friendship Br.	25.4-26.4	0.4-0.6 (0.5)	0.6-0.8 (0.7)	0.7-1.0 (0.9)	0.8-1.1 (0.9)	0.9-1.2 (1.1)
Bamban						
Sacobia and Marimla flow	14-19	0.4	0.7	0.9	0.9	1.1
Sacobia flow only	14-19	0.3	0.4	0.5	0.6	0.7
Bangat						
downstream site	9.0	1.0	1.3	1.5	1.6	1.8
middle site	9.0	1.2	1.7	2.1	2.3	2.7
upstream site	9.0	2.1	2.8	3.3	3.5	3.9
Bucao						
downstream of bridge	0-3	1.1	1.7	2.2	2.3	2.7
upstream of bridge	0-3	0.4-2.6 (1.7)	1.2-3.7 (2.7)	2.1-4.5 (3.5)	2.4-4.8 (3.8)	3.1-5.5 (4.7)
Gumain						
Downstream of river km 16	14.5-16.0	1.2	1.7	2.0	2.1	2.3
Upstream of river km 16	16.0-18.0	0.2-2.3 (1.0)	0.4-2.5 (1.2)	0.5-2.7 (1.4)	0.5-2.7 (1.4)	0.6-2.9 (1.6)
O'Donnell	26.0	0.4	0.6	0.8	1.1	1.1
O'Donnell	29.0	0.4	0.7	0.9	1.0	1.2
Pasig						
Downstream of lower bridge	1-2.3	0.9-1.5 (1.1)	1.4-2.0 (1.6)	1.8-2.3 (1.9)	1.9-2.5 (2.1)	2.2-2.8 (2.4)
Betw. lower and Bypass Br.	2.3-4.1	0.6-1.1 (0.8)	0.8-1.6 (1.3)	1.0-2.0 (1.6)	1.1-2.2 (1.7)	1.2-2.5 (2.0)
Upstream of Bypass Bridge	4.1-4.6	0.6-0.9 (0.7)	0.9-1.2 (1.1)	1.2-1.5 (1.3)	1.3-1.6 (1.4)	1.6-1.9 (1.7)
Pasig	19-21	0.6-2.1 (1.4)	0.9-2.9 (1.9)	1.1-3.3 (2.2)	1.2-3.8 (2.3)	1.4-4.0 (2.5)
Porac						
Near Floridablanca	5-7	1.1-1.9 (1.6)	1.4-2.8 (2.2)	1.7-3.5 (2.7)	1.9-3.7 (2.9)	2.0-4.2 (3.2)
Downstream of bridge	18-20	0.2-1.8 (0.9)	0.3-2.5 (1.2)	0.4-2.9 (1.4)	0.5-3.1 (1.5)	0.5-3.4 (1.6)
Upstream of bridge	20-22.7	0.5-2.3 (1.0)	0.7-3.2 (1.4)	0.9-3.7 (1.7)	1.0-3.9 (1.9)	1.1-4.3 (2.2)
Santo Tomas						
Downstream of Macolcol Br.	0-1.6	0.6-1.0 (0.8)	0.8-1.6 (1.2)	1.0-1.6 (1.3)	1.1-1.7 (1.3)	1.1-2.0 (1.5)
Upstream of Macolcol Bridge	1.6-3.6	0.7-1.5 (1.0)	1.3-1.9 (1.5)	1.5-2.2 (1.6)	1.6-2.3 (1.8)	1.7-2.4 (2.0)
Tarlac						
Downstream of Aquino Br.	town of Tarlac	0.5-1.2 (0.7)	0.7-1.7 (1.1)	0.9-2.1 (1.4)	1.0-2.2 (1.5)	1.2-2.6 (1.8)
Betw. Aquino and Agana Br.	town of Tarlac	0.8-1.9 (1.3)	1.3-2.8 (2.0)	1.8-3.5 (2.5)	2.0-3.7 (2.7)	2.6-4.3 (3.3)
Upstream of Agana Bridge	town of Tarlac	0.5-2.5 (1.3)	1.0-3.7 (2.1)	1.6-4.7 (2.9)	1.9-5.2 (3.4)	2.9-6.6 (4.7)

Table 3.6.2

Estimated Depths for Sediment + Water Flows

River	Reach of River Modeled (kilometers)	Maximum and minimum depths in meters estimated to occur within the reach for the 2- through 500-year floods. Following the min and max is the estimated average depth within the reach (in parentheses). Normal depth is indicated for reaches for which only one depth is shown.				
		2-year	10-year	50-year	100-year	500-year
Abacan	1.0	0.4	0.7	0.9	1.0	1.2
Abacan	8.0	0.4	0.7	0.9	1.0	1.2
Abacan						
Downstr. of Expressway Br.	16.7-17.6	0.3-0.6 (0.5)	0.5-0.9 (0.7)	0.6-1.1 (0.9)	0.7-1.2 (0.9)	0.8-1.3 (1.1)
Betw. Expr. and Friendship Br.	17.6-25.4	0.3-0.9 (0.5)	0.5-1.2 (0.8)	0.7-1.4 (0.9)	0.8-1.4 (1.0)	0.9-1.6 (1.2)
Upstream of Friendship Br.	25.4-26.4	0.4-0.6 (0.5)	0.6-0.9 (0.7)	0.8-1.0 (0.9)	0.8-1.1 (1.0)	0.9-1.2 (1.1)
Bamban						
Sacobia and Marimla	14-19	0.5	0.8	1.0	1.1	1.3
Sacobia flow only	14-19	0.3	0.5	0.7	0.7	0.9
Bangat						
downstream site	9.0	1.0	1.3	1.5	1.6	1.8
middle site	9.0	1.2	1.8	2.2	2.4	2.8
upstream site	9.0	2.2	2.9	3.4	3.6	4.0
Bucao						
downstream of bridge	0-3	1.3	2.1	2.6	2.9	3.4
upstream of bridge	0-3	0.7-3.0 (2.1)	1.9-4.3 (3.3)	2.9-5.3 (4.3)	3.3-5.7 (4.7)	4.3-6.6 (5.6)
Gumain						
Downstream of river km 16	14.5-16.0	0.3	0.5	0.6	0.6	0.7
Upstream of river km 16	16.0-18.0	0.3-2.3 (1.0)	0.4-2.6 (1.3)	0.5-2.7 (1.4)	0.6-2.8 (1.5)	0.7-3.1 (1.6)
	26.0	0.5	1.0	1.1	1.2	1.2
O'Donnell	29.0	0.4	0.7	1.0	1.1	1.3
Pasig						
Downstream of lower bridge	1-2.3	1.2-1.8 (1.4)	1.9-2.4 (2.0)	2.3-2.9 (2.5)	2.5-3.1 (2.7)	2.9-3.5 (3.1)
Betw. lower and Bypass Br.	2.3-4.1	0.7-1.4 (1.1)	1.0-2.1 (1.7)	1.3-2.6 (2.1)	1.4-2.8 (2.3)	1.7-3.3 (2.6)
Upstream of Bypass Bridge	4.1-4.6	0.8-1.0 (0.9)	1.3-1.6 (1.4)	1.6-2.0 (1.7)	1.7-2.1 (1.9)	2.0-2.4 (2.2)
Pasig	19-21	This reach was not modeled for bulked flow as the estimated concentration of the bulked flow was such that the resulting mixture would be a non-Newtonian fluid.				
Porac						
Near Floridablanca	5-7	1.1-2.0 (1.6)	1.4-3.0 (2.3)	1.8-3.6 (2.8)	1.9-3.9 (3.0)	2.1-4.4 (3.4)
Porac						
Downstream of bridge	18-20	0.2-1.9 (1.0)	0.3-2.6 (1.3)	0.4-3.1 (1.4)	0.5-3.2 (1.5)	0.6-3.6 (1.7)
Upstream of bridge	20-22.7	0.5-2.4 (1.0)	0.7-3.3 (1.5)	1.0-3.9 (1.8)	1.0-4.1 (2.0)	1.1-4.5 (2.3)
Santo Tomas						
Downstream of Macolcol Br.	0-1.6	0.7-1.2 (0.9)	1.0-1.6 (1.3)	1.1-1.9 (1.5)	1.2-2.0 (1.5)	1.2-2.2 (1.6)
Upstream of Macolcol Bridge	1.6-3.6	0.9-1.6 (1.1)	1.5-2.2 (1.7)	1.7-2.4 (2.0)	1.8-2.5 (2.1)	2.0-2.9 (2.4)
Tarlac						
Downstream of Aquino Br.	town of Tarlac	0.5-1.3 (0.8)	0.8-1.8 (1.2)	1.0-2.2 (1.5)	1.1-2.4 (1.6)	1.3-2.8 (1.9)
Betw. Aquino and Agana Br.	town of Tarlac	0.9-2.0 (1.4)	1.5-3.0 (2.1)	2.0-3.7 (2.7)	2.2-4.0 (2.9)	3.0-4.7 (3.8)
Upstream of Agana Bridge	town of Tarlac	0.7-2.7 (1.4)	1.2-4.0 (2.3)	1.8-5.2 (3.3)	2.3-5.9 (3.9)	3.5-7.3 (5.3)

Table 3.6.3

Estimated Velocities for Clear-Water Flows

River	Reach of River Modeled	Maximum and minimum velocities in meters per second estimated to occur within the reach for the 2- through 500-year floods. Following the min and max is the estimated average velocity within the reach (in parentheses). Velocity at normal depth is indicated for reaches for which only one depth is shown.				
	(river km)	2-year	10-year	50-year	100-year	500-year
Abacan	1.0	0.9	1.2	1.4	1.5	1.7
Abacan	8.0	1.2	1.5	1.8	1.9	2.1
Abacan						
Downstr. of Expressway Br.	16.7-17.6	1.4-1.9 (1.7)	1.7-2.3 (2.1)	1.9-2.6 (2.4)	2.0-2.8 (2.5)	2.1-3.2 (2.7)
Betw. Expr. and Friendship Br.	17.6-25.4	1.4-7.5 (2.0)	0.3-1.1 (0.7)	1.9-6.1 (2.8)	1.9-6.0 (2.9)	2.1-6.1 (3.2)
Upstream of Friendship Br.	25.4-26.4	1.6-2.7 (2.0)	2.0-3.3 (2.6)	2.4-3.4 (2.9)	2.5-3.6 (3.1)	2.7-3.9 (3.4)
Bamban						
Sacobia and Marimla flow	14-19	1.6	2.2	2.5	2.7	3.0
Sacobia flow only	14-19	1.1	1.5	1.9	1.9	2.2
Bangat						
downstream site	9.0	1.8	2.3	2.6	2.7	2.9
middle site	9.0	3.1	4.1	4.7	4.9	5.4
upstream site	9.0	2.5	3.2	3.7	3.9	4.2
Bucao						
downstream of bridge	0-3	2.6	3.5	4.0	4.2	4.7
upstream of bridge	0-3	0.7-2.9 (1.3)	1.0-3.5 (1.4)	0.9-4.0 (1.6)	0.9-4.1 (1.6)	0.9-4.4 (1.8)
Gumain						
Downstream of river km 16	14.5-16.0	0.3	0.4	0.5	0.6	0.7
Upstream of river km 16	16.0-18.0	1.2-2.1 (1.6)	1.6-2.6 (2.2)	1.9-3.0 (2.5)	2.0-3.1 (2.6)	2.2-3.4 (2.8)
O'Donnell	26.0	1.6	2.1	2.4	1.5	1.8
O'Donnell	29.0	2.0	2.8	3.3	3.5	3.9
Pasig						
Downstream of lower bridge	1-2.3	0.9-1.1 (0.9)	1.2-1.4 (1.3)	1.4-1.6 (1.5)	1.5-1.7 (1.6)	1.6-1.9 (1.7)
Betw. lower and Bypass Br.	2.3-4.1	0.7-1.8 (1.0)	1.0-1.8 (1.4)	1.2-2.3 (1.6)	1.2-2.4 (1.7)	1.2-2.5 (2.0)
Upstream of Bypass Bridge	4.1-4.6	0.9-1.9 (1.3)	1.1-2.3 (1.7)	1.3-2.3 (1.9)	1.4-2.4 (2.0)	1.6-2.5 (2.2)
Pasig	19-21	1.9-3.2 (2.5)	2.4-3.5 (2.8)	2.6-3.8 (3.0)	2.6-3.4 (3.0)	2.8-3.5 (3.1)
Porac						
Near Floridablanca	5-7	1.1-2.0 (1.6)	1.3-2.9 (2.1)	1.3-3.2 (2.3)	1.3-3.4 (2.4)	1.3-3.8 (2.6)
Porac						
Downstream of bridge	18-20	1.1-4.9 (2.0)	1.4-5.4 (2.3)	1.5-5.5 (2.5)	2.6-5.2 (3.7)	1.8-5.7 (2.8)
Upstream of bridge	20-22.7	1.7-3.2 (2.4)	2.0-4.3 (3.1)	2.6-5.0 (3.5)	1.5-5.5 (2.5)	2.8-5.7 (4.0)
Santo Tomas						
Downstream of Macolcol Br.	0-1.6	0.9-2.0 (1.5)	0.9-2.4 (1.7)	0.6-2.7 (1.9)	0.7-2.8 (2.0)	0.9-3.3 (1.9)
Upstream of Macolcol Bridge	1.6-3.6	1.3-3.7 (2.1)	1.3-2.2 (1.7)	1.5-2.4 (2.0)	1.6-2.6 (2.1)	1.8-2.8 (2.3)
Tarlac						
Downstream of Aquino Br.	town of Tarlac	0.8-2.6 (1.3)	1.1-3.5 (1.8)	1.2-4.1 (2.1)	1.3-4.3 (2.2)	1.4-4.6 (2.5)
Betw. Aquino and Agana Br.	town of Tarlac	1.2-3.0 (2.0)	1.6-4.2 (2.8)	1.9-4.9 (3.3)	2.0-5.1 (3.4)	2.2-5.6 (3.6)
Upstream of Agana Bridge	town of Tarlac	0.5-2.5 (1.3)	0.8-3.1 (1.8)	1.0-3.7 (1.9)	1.0-3.6 (1.9)	1.1-3.4 (1.7)

Table 3.6.4

Estimated Velocities for Sediment + Water Flows

River	Reach of River Modeled (river km)	Maximum and minimum velocities in meters per second estimated to occur within the reach for the 2- through 500-year floods. Following the min and max is the estimated average velocity within the reach (in parentheses). Velocity at normal depth is indicated for reaches for which only one depth is shown.				
		2-year	10-year	50-year	100-year	500-year
Abacan	1.0	0.9	1.3	1.5	1.6	1.8
Abacan	8.0	1.2	1.6	1.9	2.0	2.2
Abacan						
Downstr. of Expressway Br.	16.7-17.6	0.3-0.6 (0.4)	0.5-0.8 (0.6)	0.6-0.9 (0.8)	0.7-1.0 (0.8)	0.8-1.1 (0.9)
Betw. Expr. and Friendship Br.	17.6-25.4	0.2-0.9 (0.5)	0.4-1.2 (0.7)	0.5-1.3 (0.9)	0.5-1.4 (0.9)	0.6-1.6 (1.1)
Upstream of Friendship Br.	25.4-26.4	0.3-0.6 (0.4)	0.4-0.9 (0.6)	0.5-1.0 (0.8)	0.6-1.1 (0.8)	0.7-1.3 (0.9)
Bamban						
Sacobia and Marimla flow	14-19	1.7	2.4	2.8	3.0	3.3
Sacobia flow only	14-19	1.3	1.8	2.1	2.3	2.6
Bangat						
downstream site	9.0	1.9	2.4	2.7	2.8	3.1
middle site	9.0	3.2	4.2	4.9	5.1	5.6
upstream site	9.0	2.6	3.3	3.8	4.0	4.3
Bucao						
downstream of bridge	0-3	2.9	3.9	4.6	4.8	5.4
upstream of bridge	0-3	0.8-3.2 (1.3)	0.9-3.9 (1.6)	0.9-4.3 (1.7)	0.9-4.5 (1.8)	0.9-4.7 (1.9)
Gumain						
Downstream of river km 16	14.5-16.0	1.3	1.8	2.0	2.1	2.4
Upstream of river km 16	16.0-18.0	1.2-2.2 (1.6)	1.7-2.7 (2.2)	2.0-3.1 (2.6)	2.1-3.2 (2.7)	2.3-3.4 (2.9)
O'Donnell	26.0	1.9	1.7	1.8	1.8	2.0
O'Donnell	29.0	2.1	2.9	3.4	3.6	4.0
Porac						
Near Floridablanca	5-7	1.2-2.3 (1.7)	1.3-3.0 (2.2)	1.3-3.3 (2.4)	1.3-3.6 (2.5)	1.3-4.0 (2.7)
Pasig						
Downstream of lower bridge	1-2.3	1.1-1.3 (1.1)	1.4-1.7 (1.5)	1.6-1.9 (1.8)	1.7-2.0 (1.9)	1.9-2.2 (2.1)
Betw. lower and Bypass Br.	2.3-4.1	0.9-1.8 (1.2)	1.2-2.4 (1.6)	1.4-2.8 (1.9)	1.5-2.9 (2.0)	1.6-3.1 (2.2)
Upstream of Bypass Bridge	4.1-4.6	1.0-2.3 (1.6)	1.4-2.3 (2.0)	1.6-2.6 (2.3)	1.7-2.8 (2.4)	2.0-3.0 (2.6)
Pasig	19-21	This reach was not modeled for bulked flow as the estimated concentration of the bulked flow was such that the resulting mixture would be a non-Newtonian fluid.				
Porac						
Downstream of bridge	18-20	1.8-3.3 (2.5)	2.1-4.5 (3.2)	2.7-5.2 (3.6)	2.8-5.4 (3.8)	2.9-5.9 (4.2)
Upstream of bridge	20-22.7	1.0-5.0 (2.0)	1.4-5.5 (2.3)	1.6-5.6 (2.6)	1.7-5.7 (2.7)	1.8-4.1 (2.6)
Santo Tomas						
Downstream of Macolcol Br.	0-1.6	1.2-2.4 (1.8)	0.6-2.7 (1.8)	0.9-3.2 (1.8)	1.0-3.3 (2.0)	1.3-3.7 (2.4)
Upstream of Macolcol Bridge	1.6-3.6	1.5-3.4 (2.1)	1.5-2.4 (2.0)	1.8-2.8 (2.3)	1.9-3.0 (2.4)	2.0-3.5 (2.6)
Tarlac						
Downstream of Aquino Br.	town of Tarlac	0.9-2.7 (1.4)	1.1-3.7 (1.9)	1.3-4.2 (2.2)	1.3-4.4 (2.4)	1.5-4.8 (2.7)
Betw. Aquino and Agana Br.	town of Tarlac	1.2-3.2 (2.1)	1.7-4.4 (2.9)	2.0-5.1 (3.4)	2.1-5.3 (3.6)	2.2-5.8 (3.7)
Upstream of Agana Bridge	town of Tarlac	0.6-2.5 (1.5)	0.9-3.3 (1.8)	1.0-3.6 (1.9)	1.1-3.5 (1.8)	1.1-3.5 (1.7)

Table 3.6.5 (sheet 1 of 2)
Hydraulic Modeling Notes

River	Reach of River Modeled (kilometers)	Notes
Abacan	1.0	(1) Normal depth calculation. *(2) Sediment + water flows are 10% sediment. (3) Cross-section data from **DTM.
Abacan	8.0	(1) Normal depth calculation. (2) Sediment + water flows are 10% sediment. (3) Cross-section data from DTM.
Abacan	17-26	(1) Slope very close to critical, therefore, depths taken from sub-critical run and velocities from super-critical run. (2) Sediment + water flows are 10% sediment. (3) Cross-section data from ***DPWH 2 March - 21 April 1992 topographic/hydrographic survey.
Bamban	14-19	(1) Depths and velocities were estimated for two possible conditions: (a) Sacobia and Marimla flow occur together. (b) Sacobia flow only (flow from the Marimla is kept separate). (2) Sediment + water flows are 22% sediment for the combined flow and 35% for the Sacobia flow by itself (3) Cross-section data: A constant width between levees (est. by District personnel) was assumed and a constant slope (measured in the field by District personnel) was also assumed, therefore, although an HEC-2 model was run, the results are normal depth and velocity except in the vicinity of the bridge.
Bangat	9.0	(1) Normal depth calculated at 3 sites within river kilometer 9. (2) Sediment + water flows are 10% sediment. (3) Data from DTM.
Bucao	0-3.5	(1) Sediment + water flows are 28% sediment. (2) Cross-section data: Average slope from DTM data used. Levees downstream of bridge assumed 300 m wide. Right levee upstream of bridge assumed to vary linearly such that channel width at bridge is 290 m and channel width 860 m upstream of bridge is 1400 m. Due to these assumptions, results are normal depth and velocity except in the vicinity of the bridge. However, upstream of the bridge results are reported as min, max, and average to reflect the assumed change in channel width.

*All sediment concentrations are by total volume.
** DTM (Digital Terrain Model).
*** DPWH (Department of Public Works and Highways).

Table 3.6.5 (sheet 2 of 2)
Hydraulic Modeling Notes

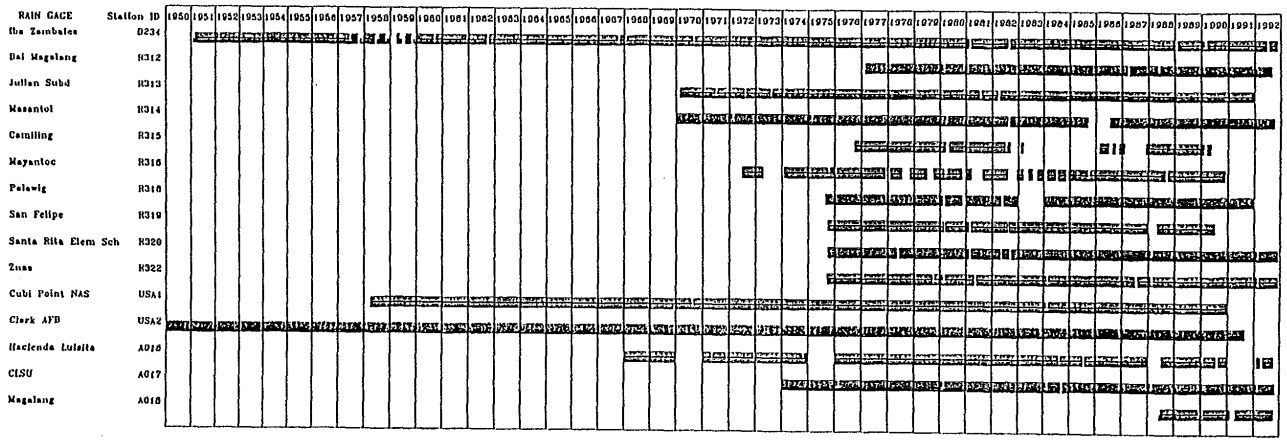
River	Reach of River Modeled (kilometers)	Notes
Gumain	14.5-18.0	(1) Sediment + water flows are 10% sediment. (2) DTM data used from river km 17.0 to 18.0. From river km 14.5 to 17.0 the average slope between river km 17.0 and 18.0 obtained from the DTM data was assumed and the width between levees was assumed to be 700 meters. Therefore, depths and velocities reported for river km 14.5 to 17.0 are normal depths and velocities.
O'Donnell	26.0	(1) Normal depth calculated at constriction using cross-section and slope data from the DTM. (2) Sediment + water flows are 38% sediment.
O'Donnell	29.0	(1) Normal depth calculated at constriction using cross-section and slope data from the DTM model. (2) Model flow is supercritical, therefore estimated depths at critical and velocities at supercritical.
Pasig	0.9-5	(1) Obtained cross-section and slope data from the DTM model. (2) Sediment + water flows are 40% sediment.
Pasig	19-21	(1) Obtained cross-section and slope data from the DTM model. (2) Model results apply from 0.5 to 3.0 km upstream of the former location of the Mancatian bridge. (3) Assumed no backwater effects from Mancatian bridge. (4) Sediment + water flows are 67% sediment, however, this was not modeled as this high of a sediment concentration would cause the flow to be non-Newtonian thus violating a basic assumption of the analyses procedures used.
Porac	5-7	(1) Obtained cross-section and slope data from the DTM model. (2) For clearwater flows, the water surface elevation of the 500-year flood was approximately 0.25 meters above the highest left overbank elevation. (3) For sediment + water flows, the water surface elevation of the 500-year flood was approximately 0.5 meters above the highest left overbank elevation. (4) Sediment + water flows are 10% sediment.
Porac	19.5-20.5	(1) Slope very close to critical, therefore, depths taken from sub-critical run and velocities from super-critical run. (2) Sediment + water flows are 10% sediment. (3) Data from DTM.
Santo Tomas	0-3	(1) Sediment + water flows are 30% sediment. (2) Cross-section data from March 1992 DPWH topographic/hydrographic survey.
Tarlac	town of Tarlac	(1) Sediment + water flows are 12% sediment. (2) Cross-section data: Width between levees and slopes obtained from 1992 DPWH drawings of the Location Plan and river profile for O'Donnell River Dredging.

*All sediment concentrations are by total volume.
** DTM (Digital Terrain Model).
*** DPWH (Department of Public Works and Highways).

FIGURES FOR

TECHNICAL APPENDIX A

HYDROLOGY AND HYDRAULICS



Legend:

- Complete data for period
- Partial data for period
- No data for period

See Plate 1 for station locations

Available Daily Rainfall Data

Figure 2.3.1

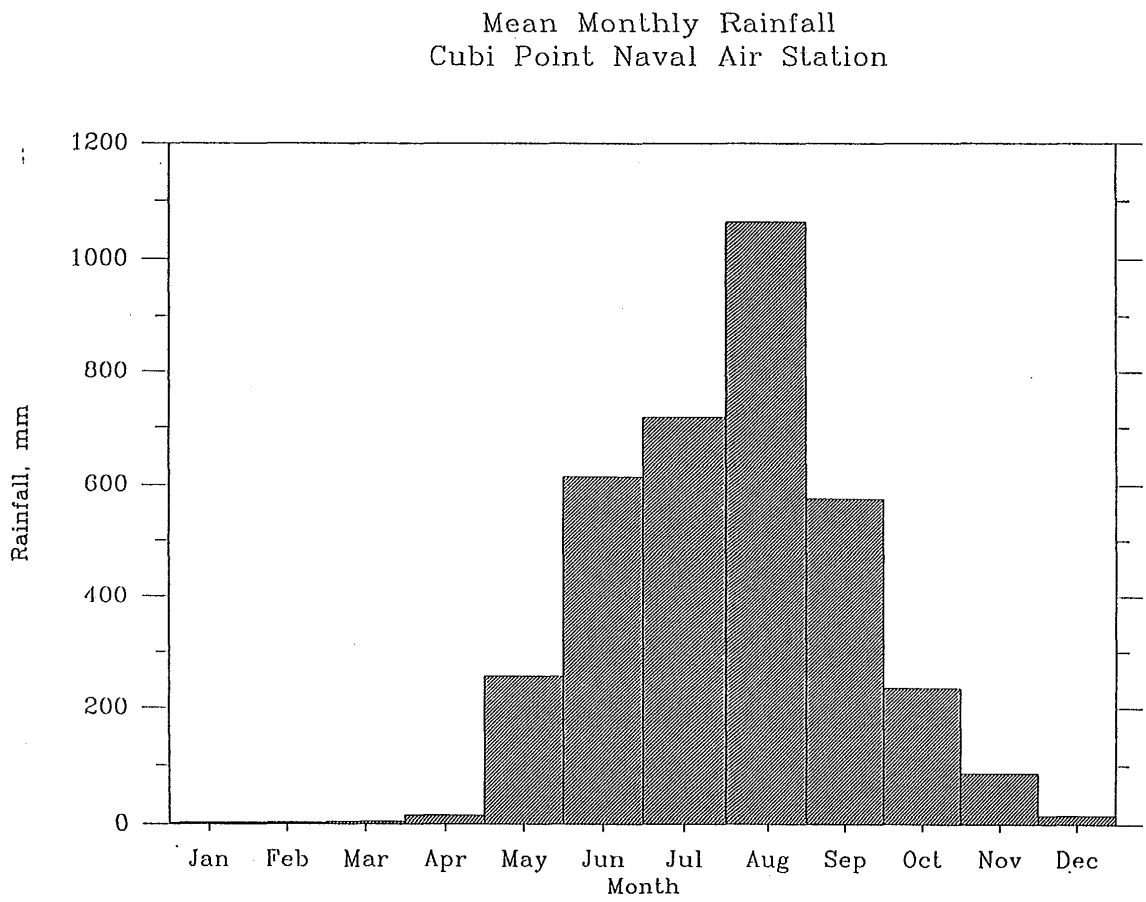

Mean Monthly Rainfall
Cubi Point Naval Air Station
Rainfall, mm
1200
1000
800
600
400
200
0
Jan
Feb
Mar
Apr
May
Jun
Jul
Aug
Sep
Oct
Nov
Dec
Month

Figure 2.3.2

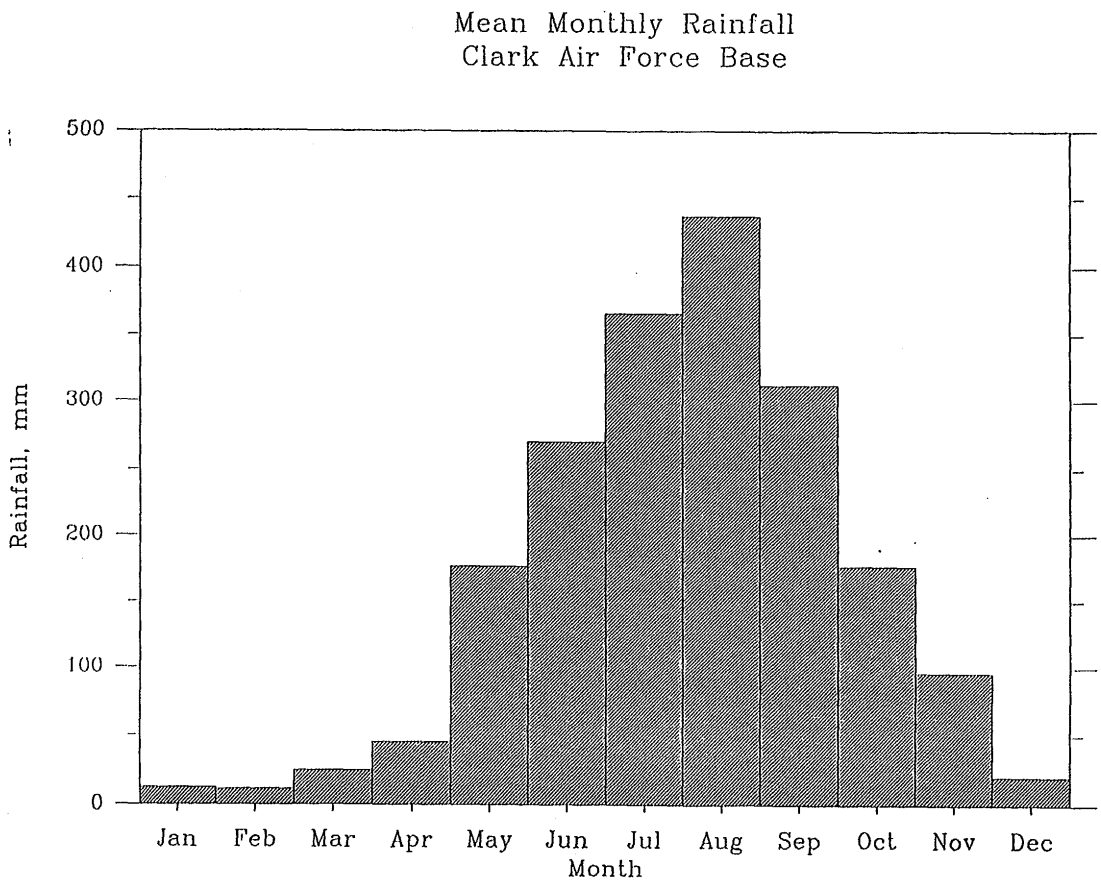


Figure 2.3.3

353

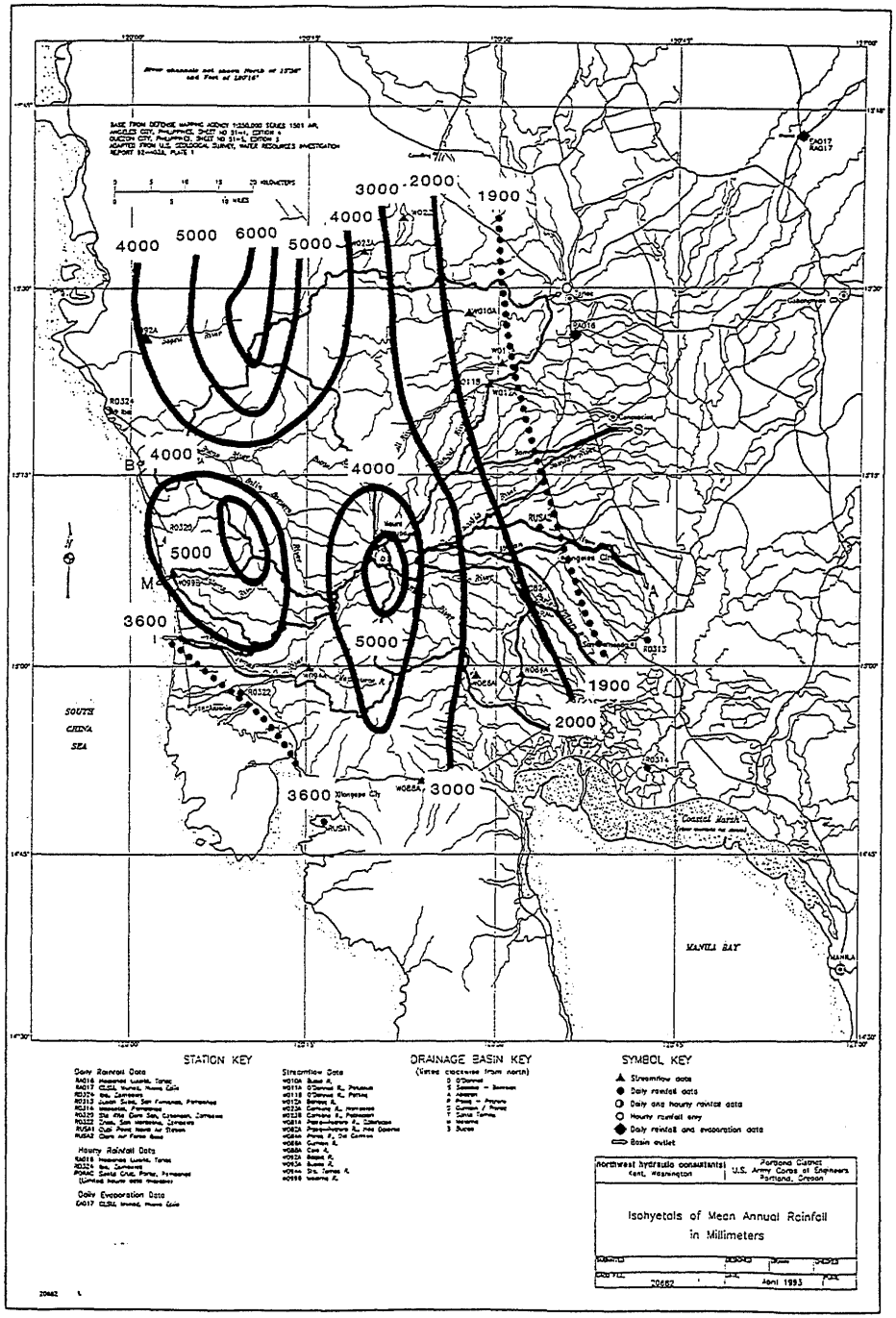


Figure 2.3.4

394

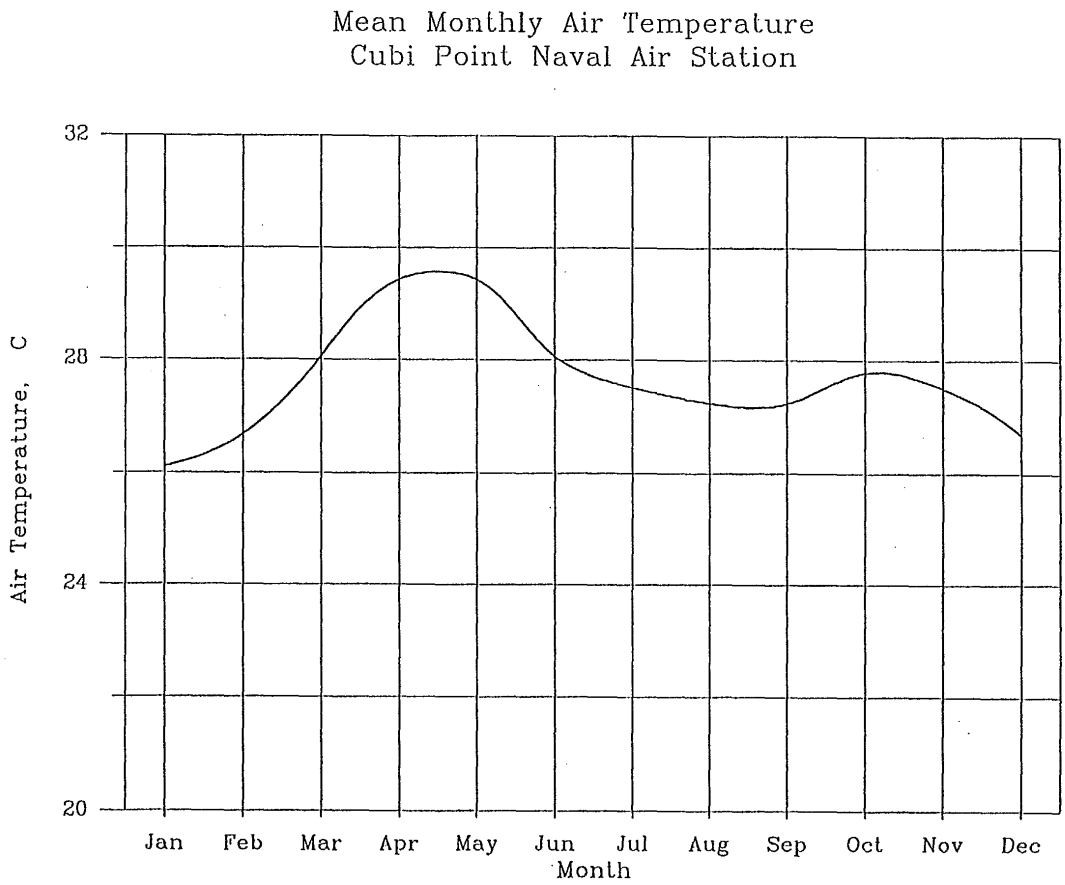


Figure 2.3.5

395

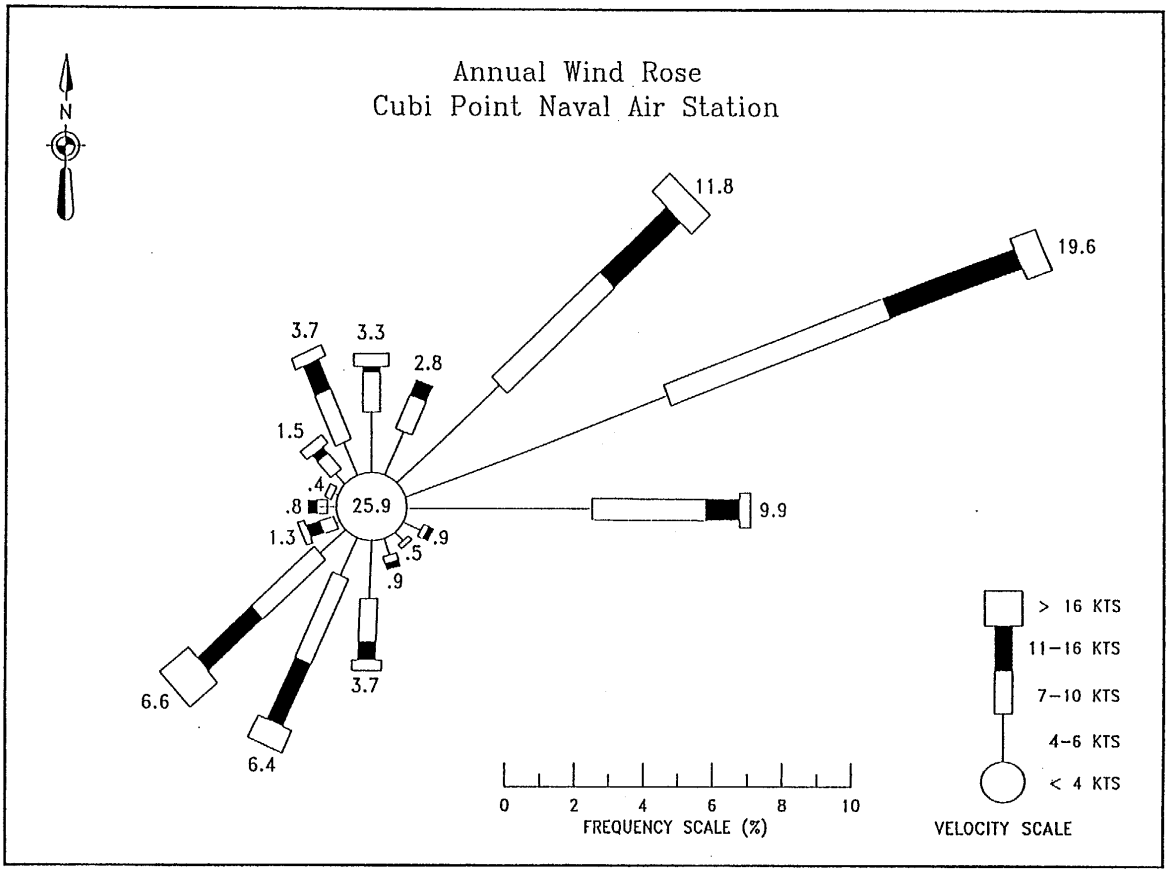
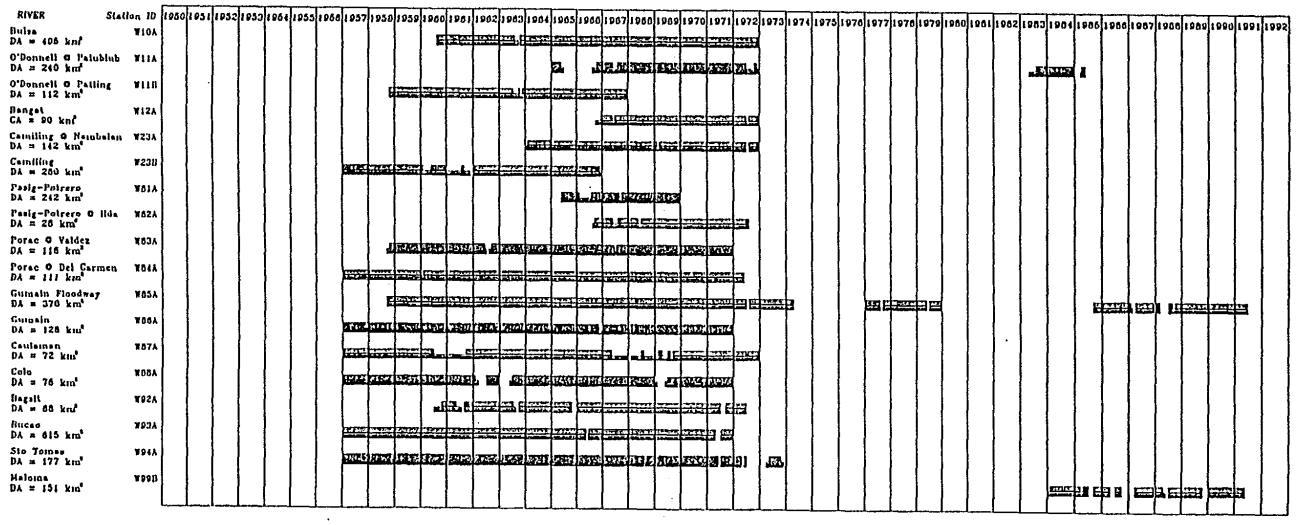


Figure 2.3.6

396



Legend:

- Complete data for period
- Partial data for period
- No data for period

See Plate 1 for station locations

Available Daily Streamflow Data

Figure 2.4.1

397

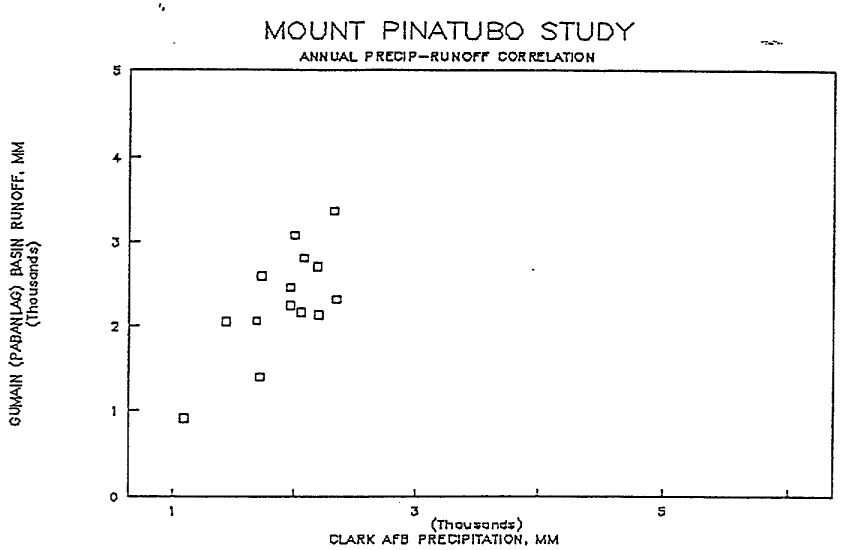

MOUNT PINATUBO STUDY
ANNUAL PRECIP–RUNOFF CORRELATION
GUMAIN (PABANLAG) BASIN RUNOFF, MM
(Thousands)
0
1
2
3
4
5
1
3
5
(Thousands)
CLARK AFB PRECIPITATION, MM

Figure 2.4.2

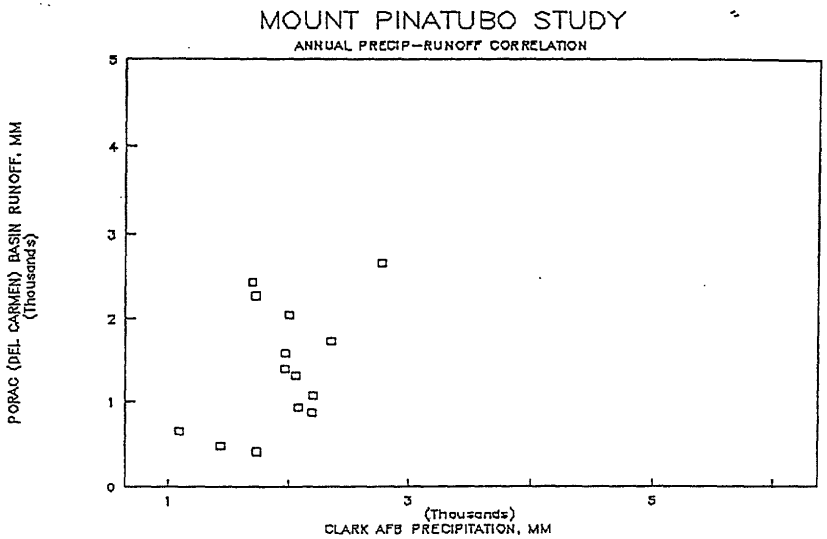

MOUNT PINATUBO STUDY
ANNUAL PRECIP—RUNOFF CORRELATION
PORAC (DEL CARMEN) BASIN RUNOFF, MM
(Thousands)
0
1
2
3
4
5
1
3
5
(Thousands)
CLARK AFB PRECIPITATION, MM

Figure 2.4.3

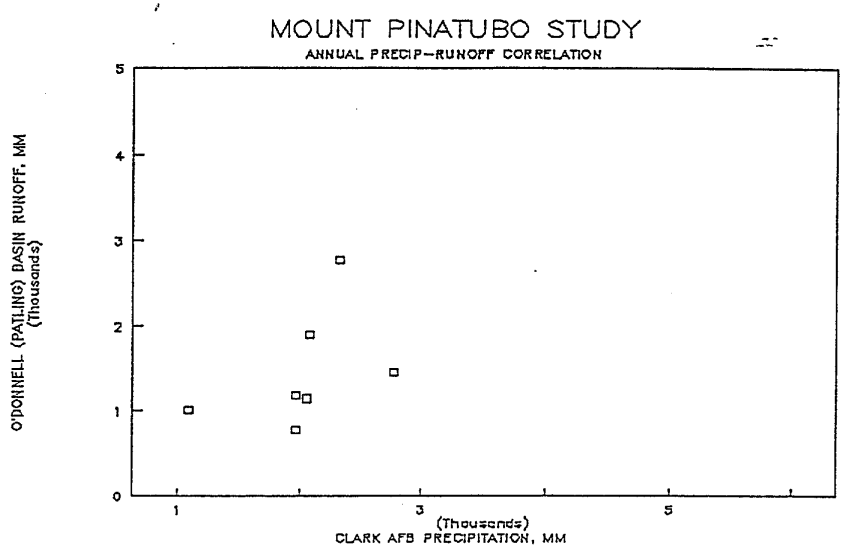

MOUNT PINATUBO STUDY
ANNUAL PRECIP–RUNOFF CORRELATION
O'DONNELL (PATLING) BASIN RUNOFF, MM
(Thousands)
0
1
2
3
4
5
1
3
5
(Thousands)
CLARK AFB PRECIPITATION, MM

Figure 2.4.4

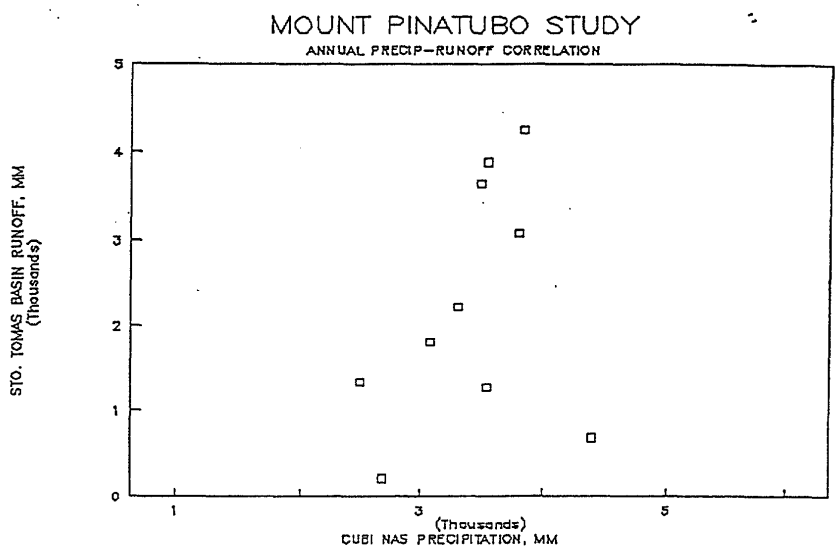
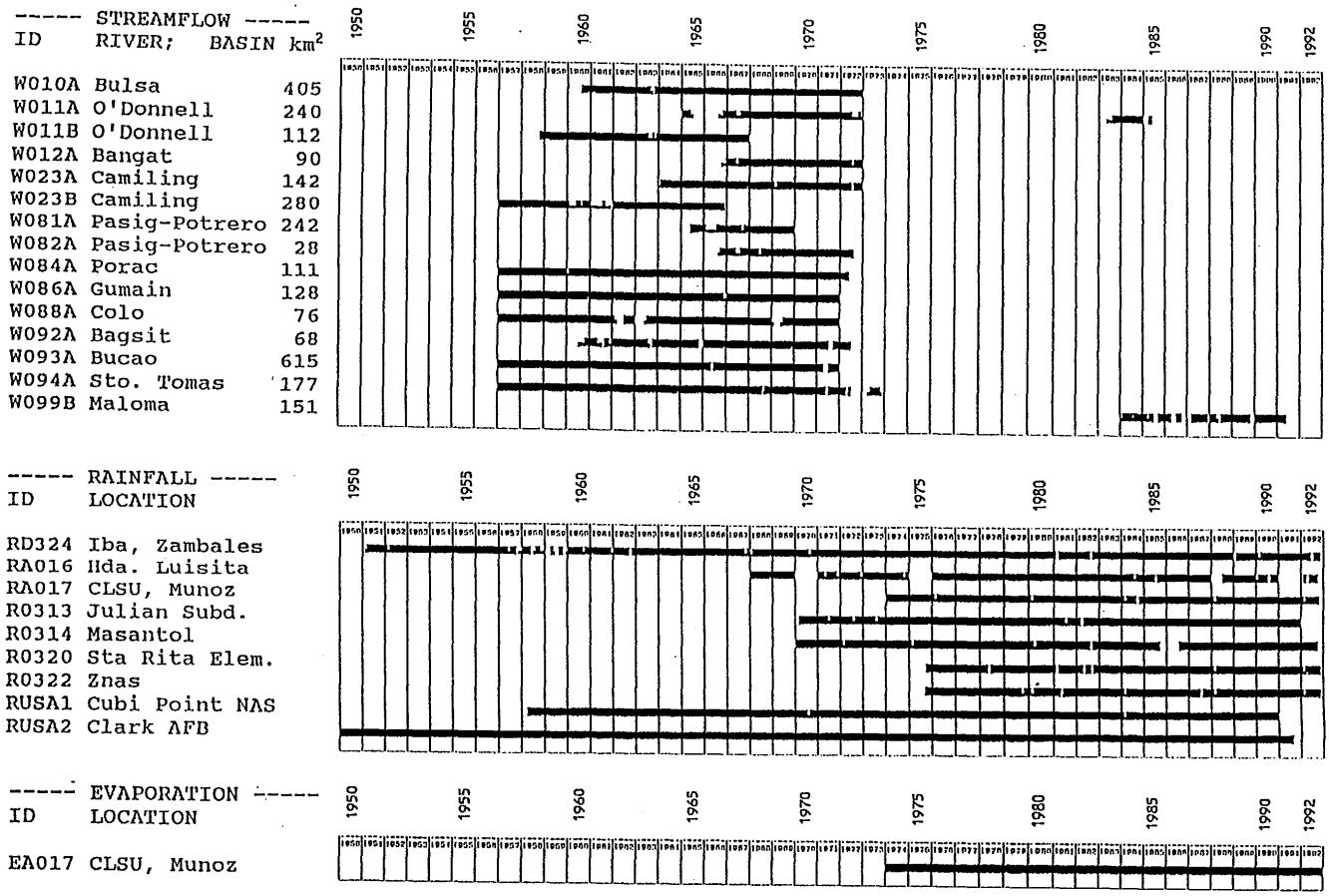

MOUNT PINATUBO STUDY
ANNUAL PRECIP–RUNOFF CORRELATION
STO. TOMAS BASIN RUNOFF, MM
(Thousands)
0
1
2
3
4
5
1
3
5
(Thousands)
CUBI NAS PRECIPITATION, MM

Figure 2.4.5

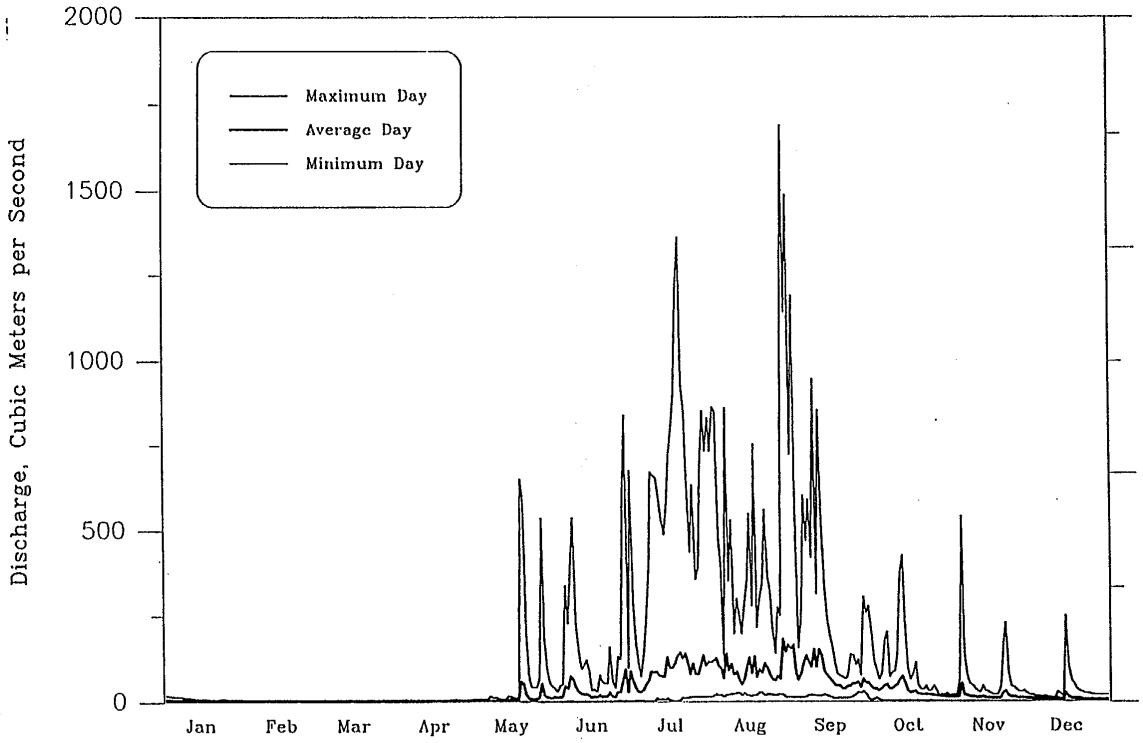


Legend:
- Complete data for period
- Partial data for period
- No data for period

See Plate 1 for station locations

Available Daily Data
Figure 2.4.6

Summary Hydrographs
Station W010A; Bulsa River, Villa Aglipay
Basin Area = 405 sq. km

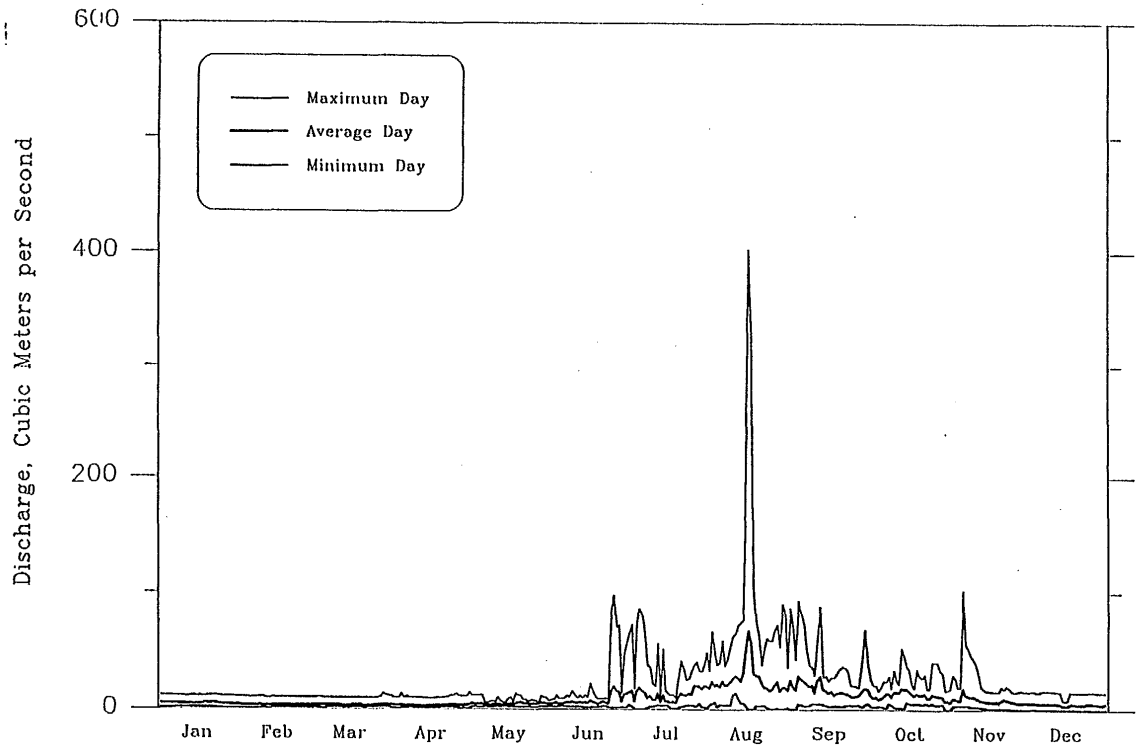


Daily Minimum, Average, and Maximum Discharges

Figure 2.4.7

403

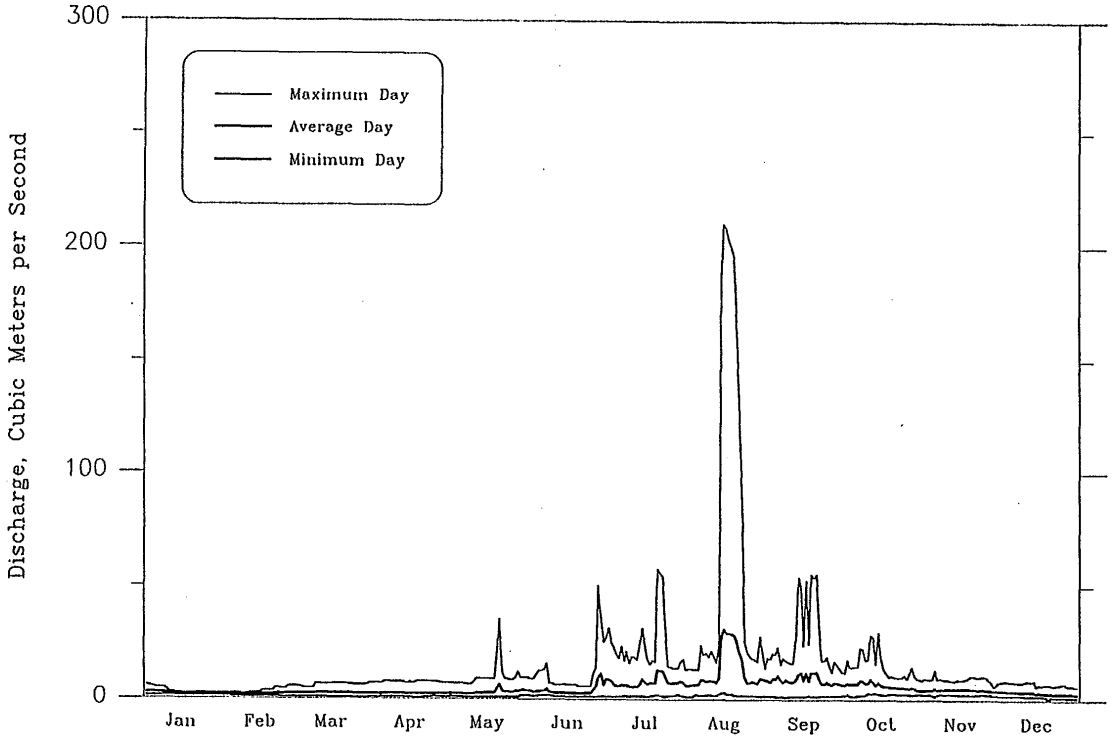
Summary Hydrographs
Station W011A; O'Donnell River, Palublub
Basin Area = 240 sq. km



Daily Minimum, Average, and Maximum Discharges

Figure 2.4.8

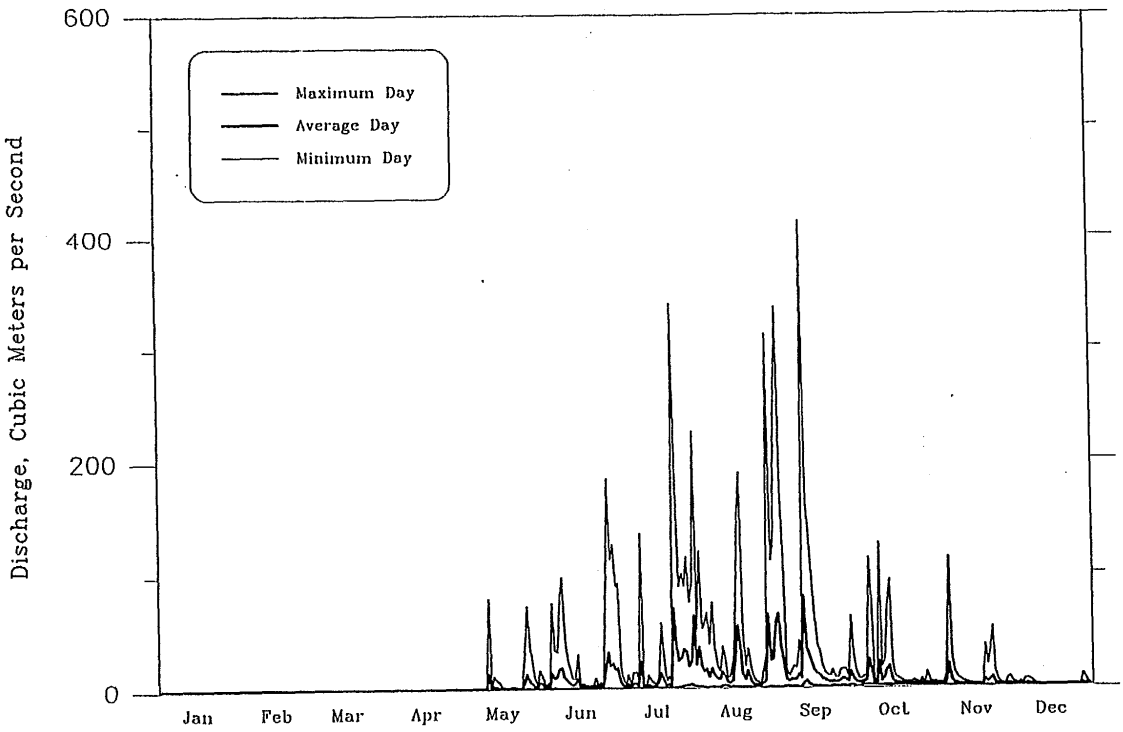
Summary Hydrographs
Station W011B; O'Donnell River, Patling
Basin Area = 112 sq. km



Daily Minimum, Average, and Maximum Discharges

Figure 2.4.9

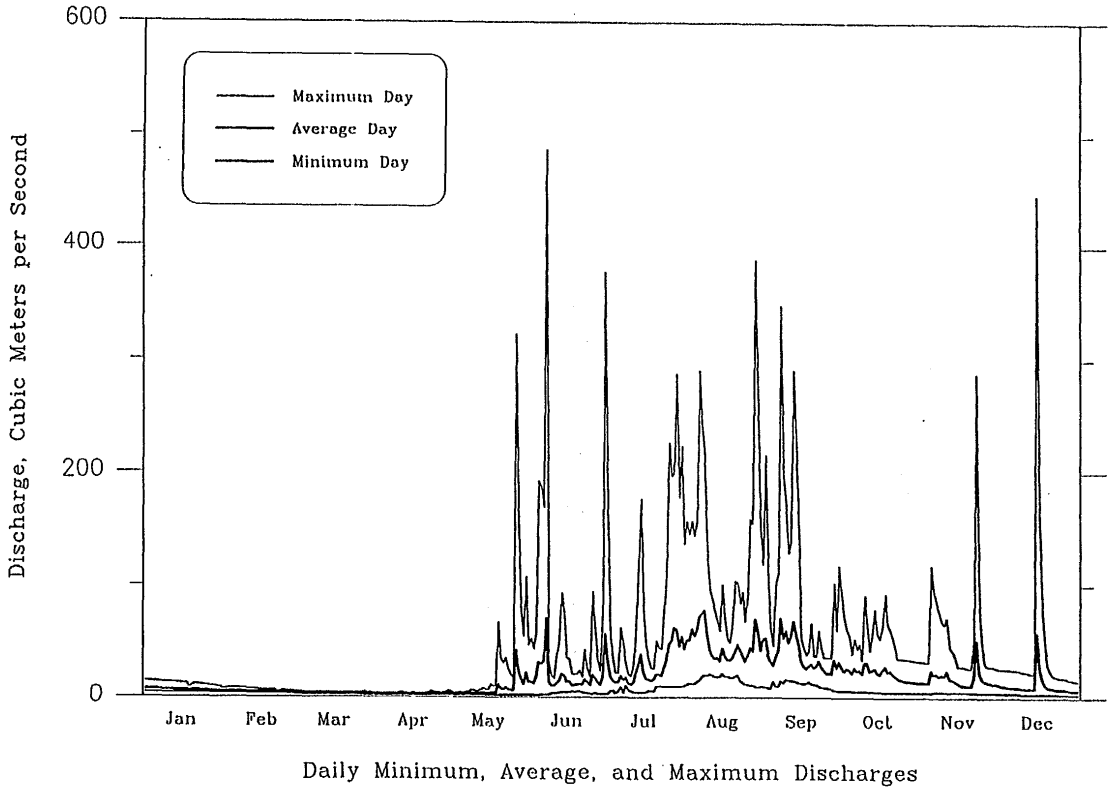
Summary Hydrographs
Station W012A; Bangat River, Sta Lucia
Basin Area = 90 sq. km



Daily Minimum, Average, and Maximum Discharges

Figure 2.4.10

Summary Hydrographs
Station W023A; Camiling River, Nambalan
Basin Area = 142 sq. km



Daily Minimum, Average, and Maximum Discharges

Figure 2.4.11

407

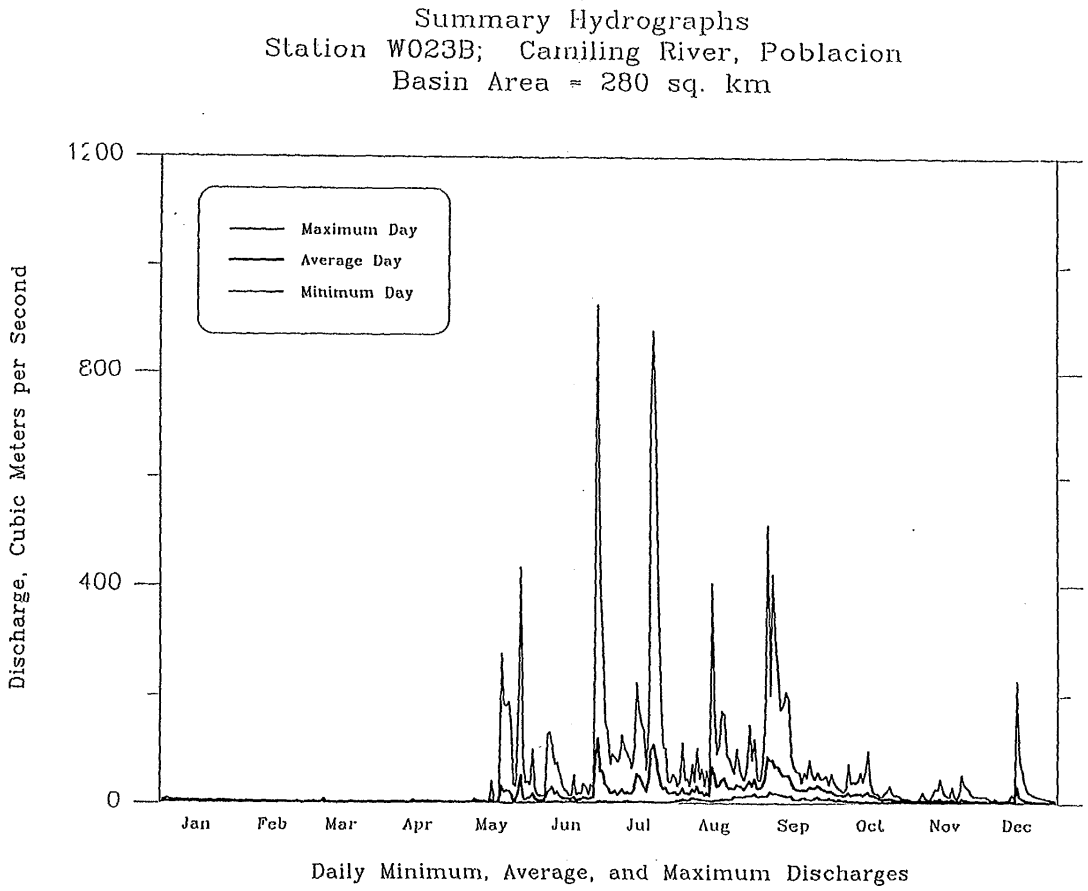

Summary Hydrographs
Station W023B; Camiling River, Poblacion
Basin Area = 280 sq. km
Maximum Day
Average Day
Minimum Day
Discharge, Cubic Meters per Second
1200
800
400
0
Jan
Feb
Mar
Apr
May
Jun
Jul
Aug
Sep
Oct
Nov
Dec
Daily Minimum, Average, and Maximum Discharges

Figure 2.4.12

Summary Hydrographs
Station W081A; Pasig–Potrero River, Cabetican
Basin Area = 242 sq. km

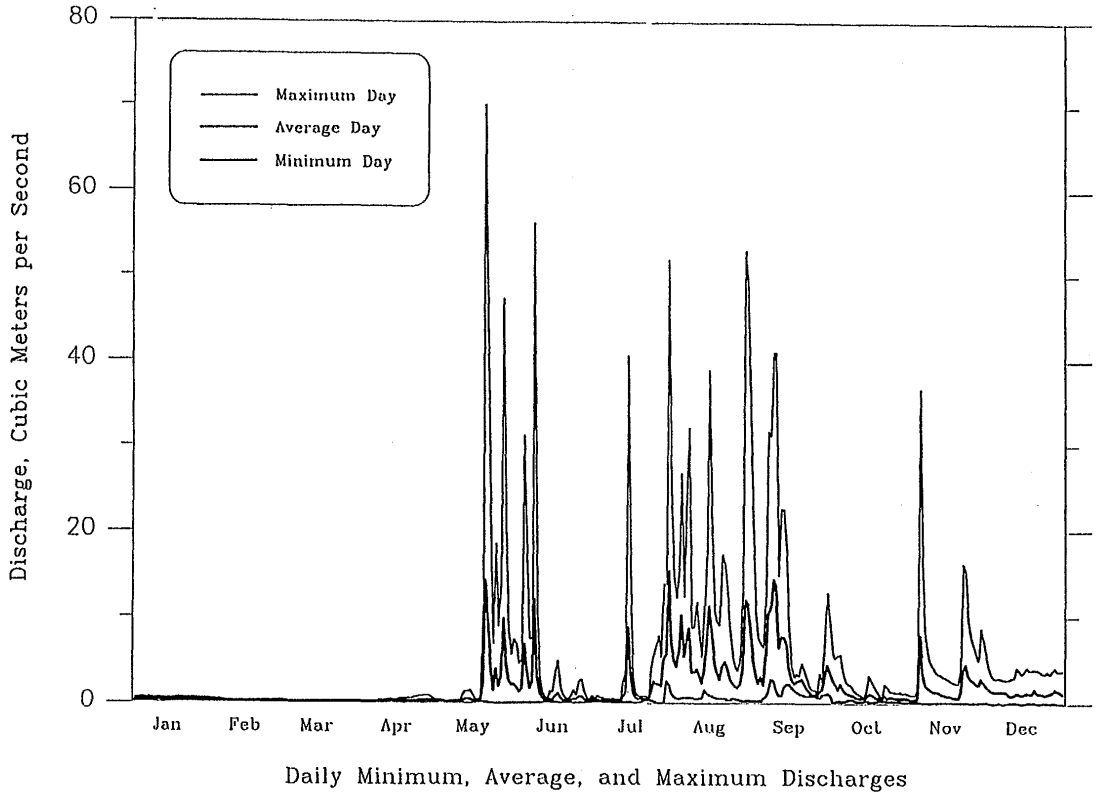


Figure 2.4.13

104

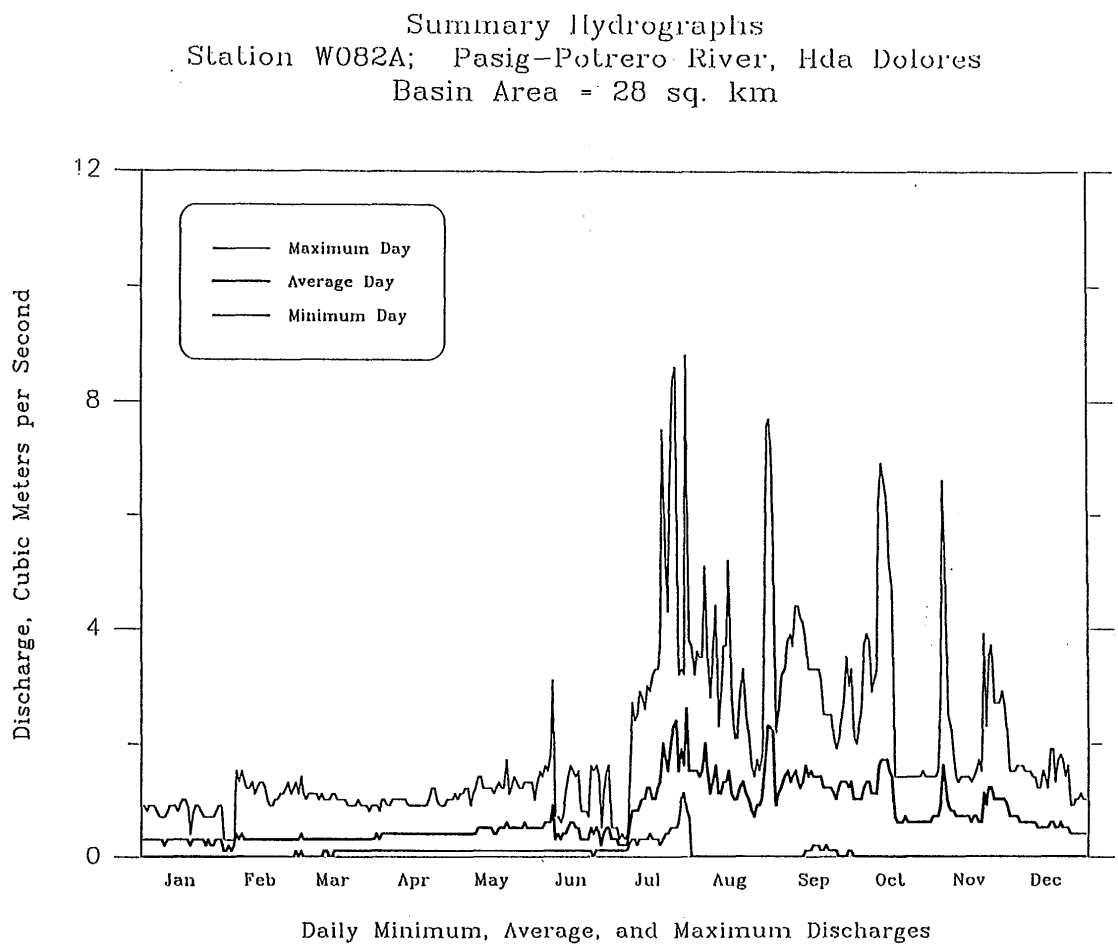
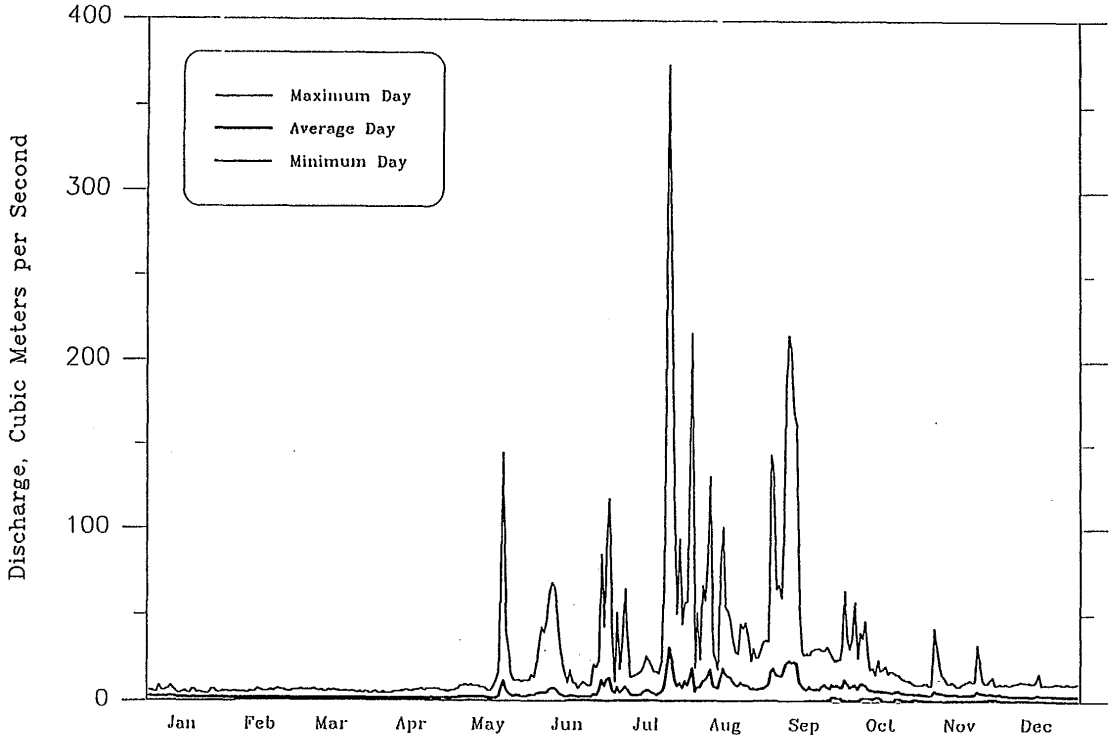

Summary Hydrographs
Station W082A; Pasig-Potrero River, Hda Dolores
Basin Area = 28 sq. km
Maximum Day
Average Day
Minimum Day
Discharge, Cubic Meters per Second
12
8
4
0
Jan
Feb
Mar
Apr
May
Jun
Jul
Aug
Sep
Oct
Nov
Dec
Daily Minimum, Average, and Maximum Discharges

Figure 2.4.14

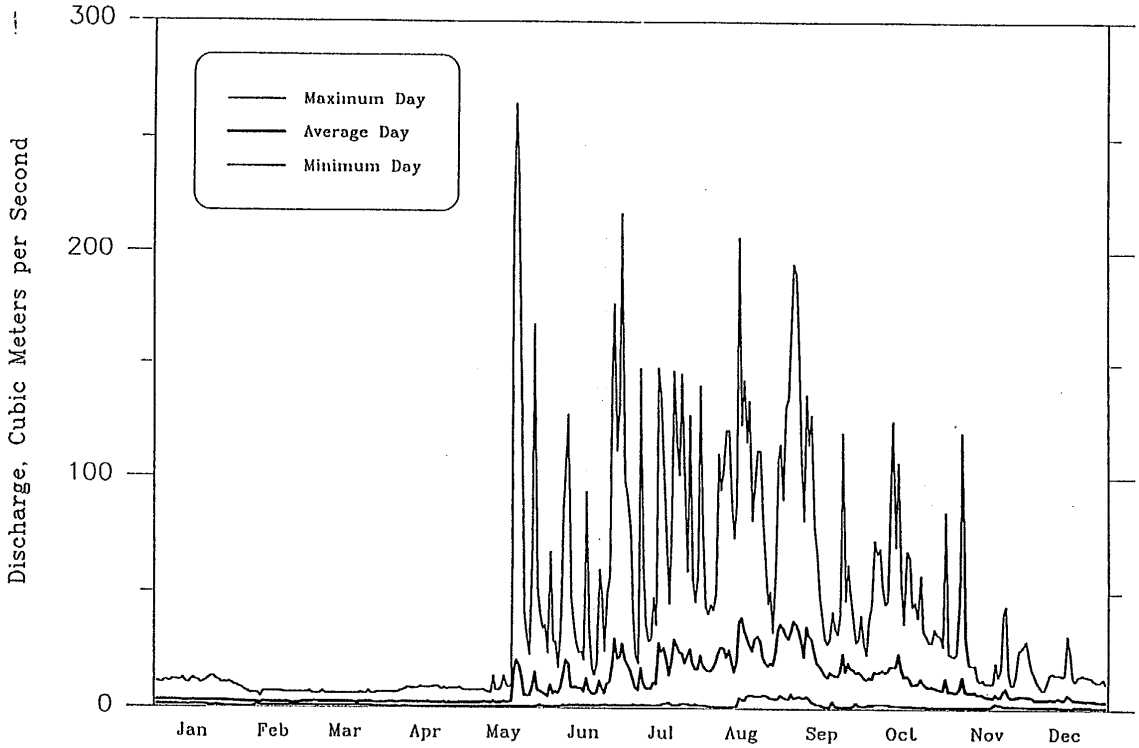
Summary Hydrographs
Station W084A; Porac River, Del Carmen
Basin Area = 111 sq. km



Daily Minimum, Average, and Maximum Discharges

Figure 2.4.15

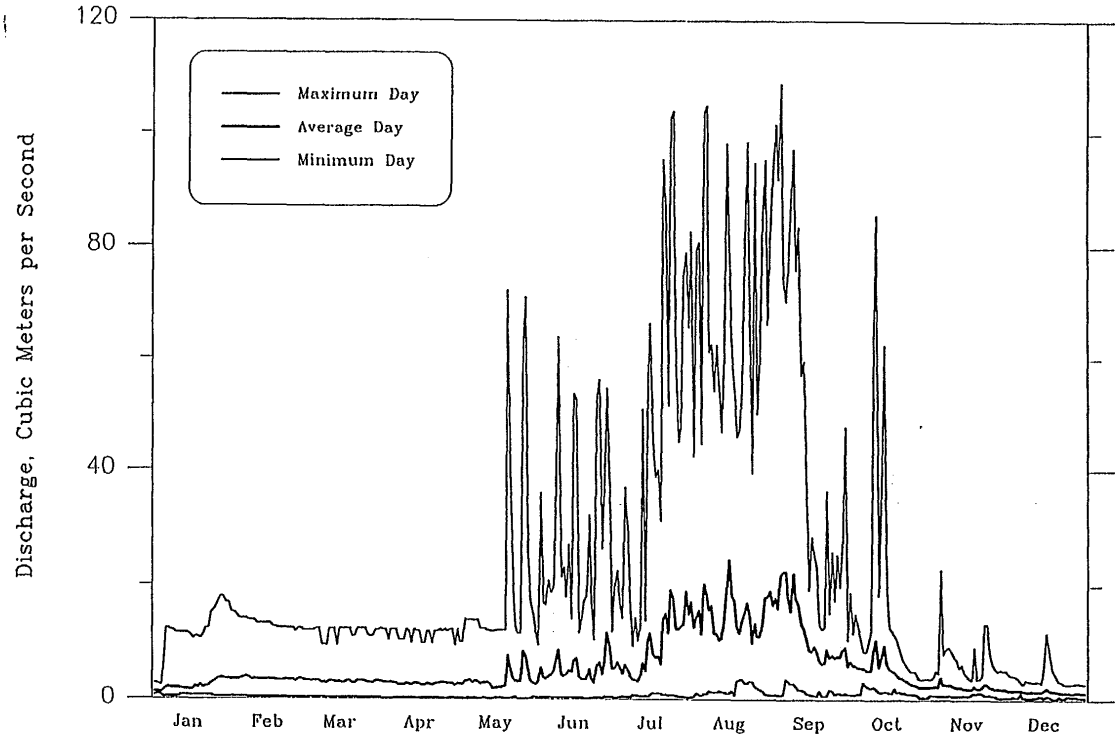
Summary Hydrographs
Station W086A; Gumain River, Pabanlag
Basin Area = 128 sq. km



Daily Minimum, Average, and Maximum Discharges

Figure 2.4.16

Summary Hydrographs
Station W088A; Colo River, San Benito
Basin Area = 76 sq. km

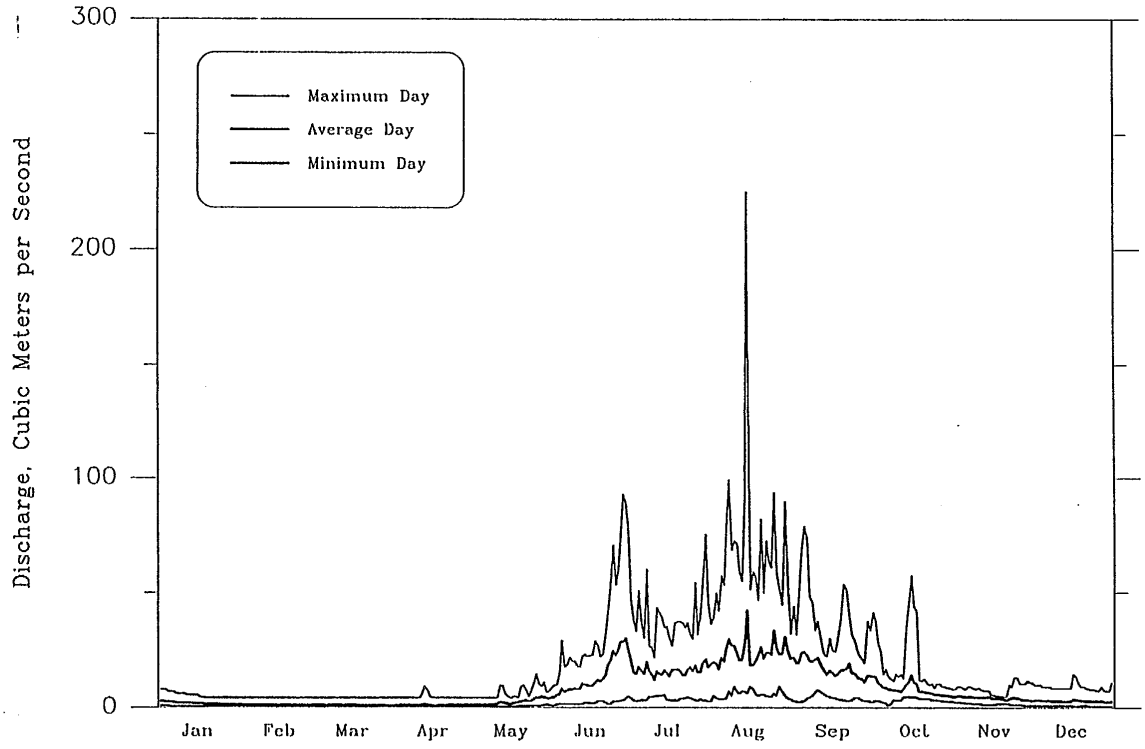


Daily Minimum, Average, and Maximum Discharges

Figure 2.4.17

415

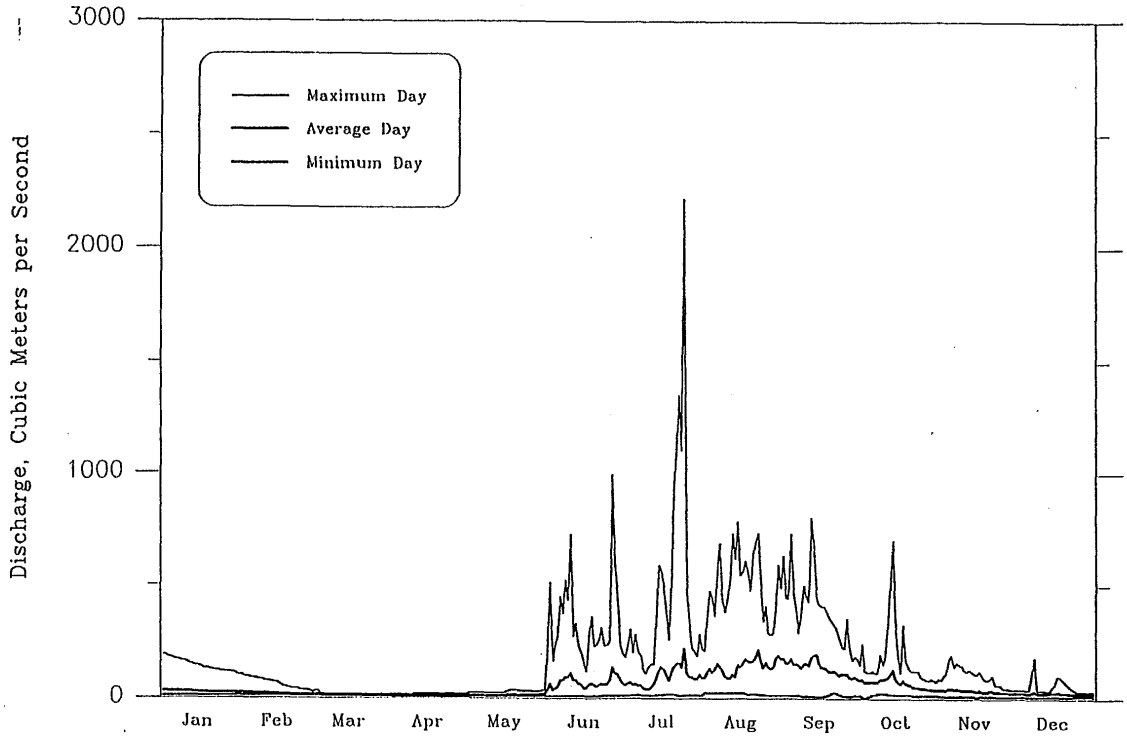
Summary Hydrographs
Station W092A; Bagsit River, Dampai
Basin Area = 68 sq. km



Daily Minimum, Average, and Maximum Discharges

Figure 2.4.18

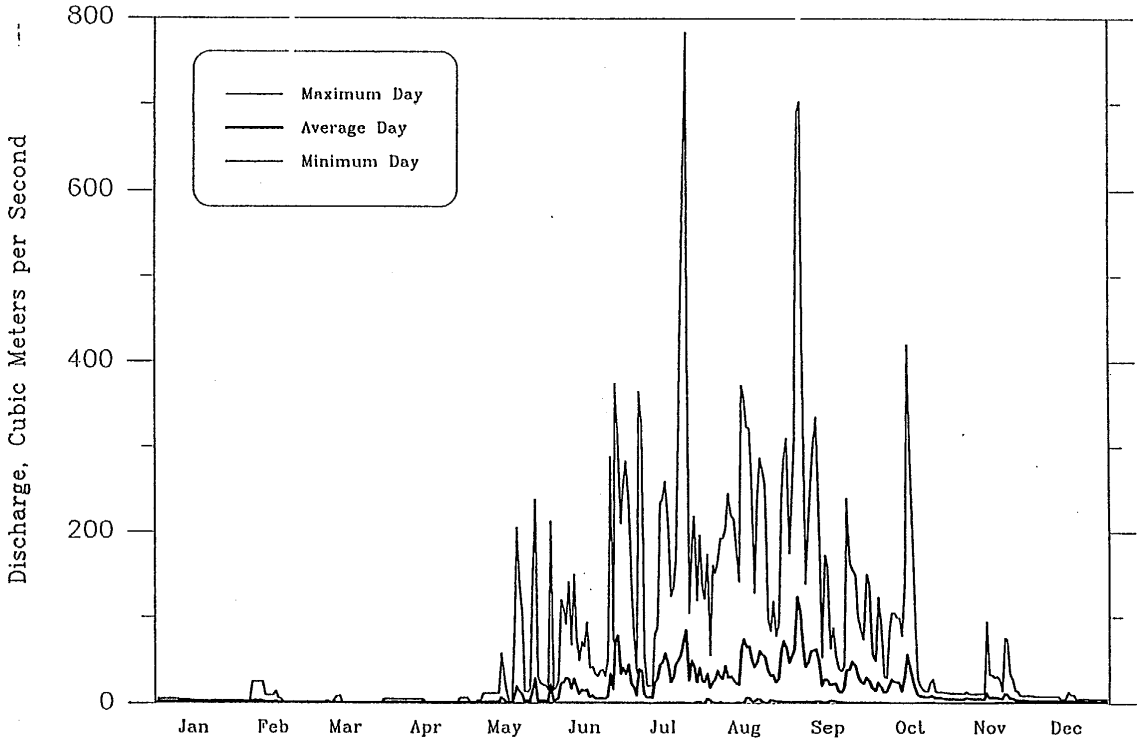
Summary Hydrographs
Station W093A; Bucao River, San Juan
Basin Area = 615 sq. km



Daily Minimum, Average, and Maximum Discharges

Figure 2.4.19

Summary Hydrographs
Station W094A; Santo Tomas River, Dalanawan
Basin Area = 177 sq. km



Daily Minimum, Average, and Maximum Discharges

Figure 2.4.20

Summary Hydrographs
Station W099B; Maloma River, Maloma
Basin Area = 151 sq. km

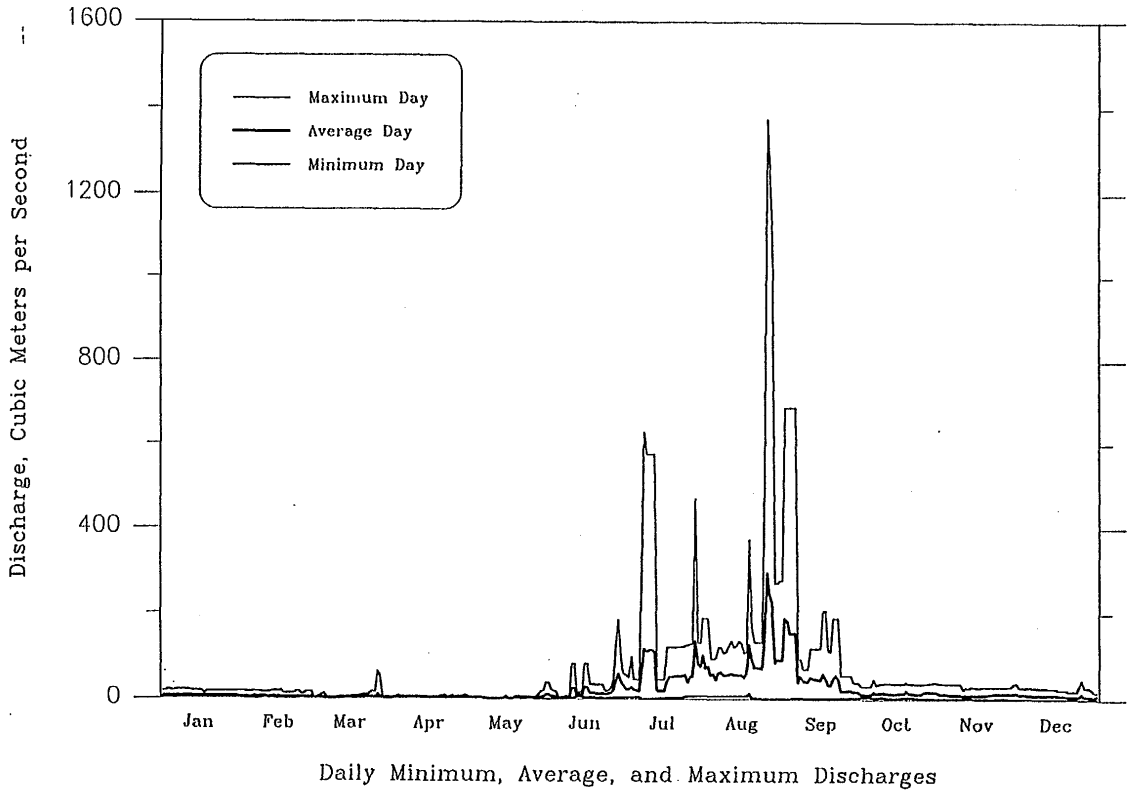


Figure 2.4.21

46

Monthly Flow Analysis
Station W010A; Bulsa River, Villa Aglipay
Basin Area = 405 sq. km

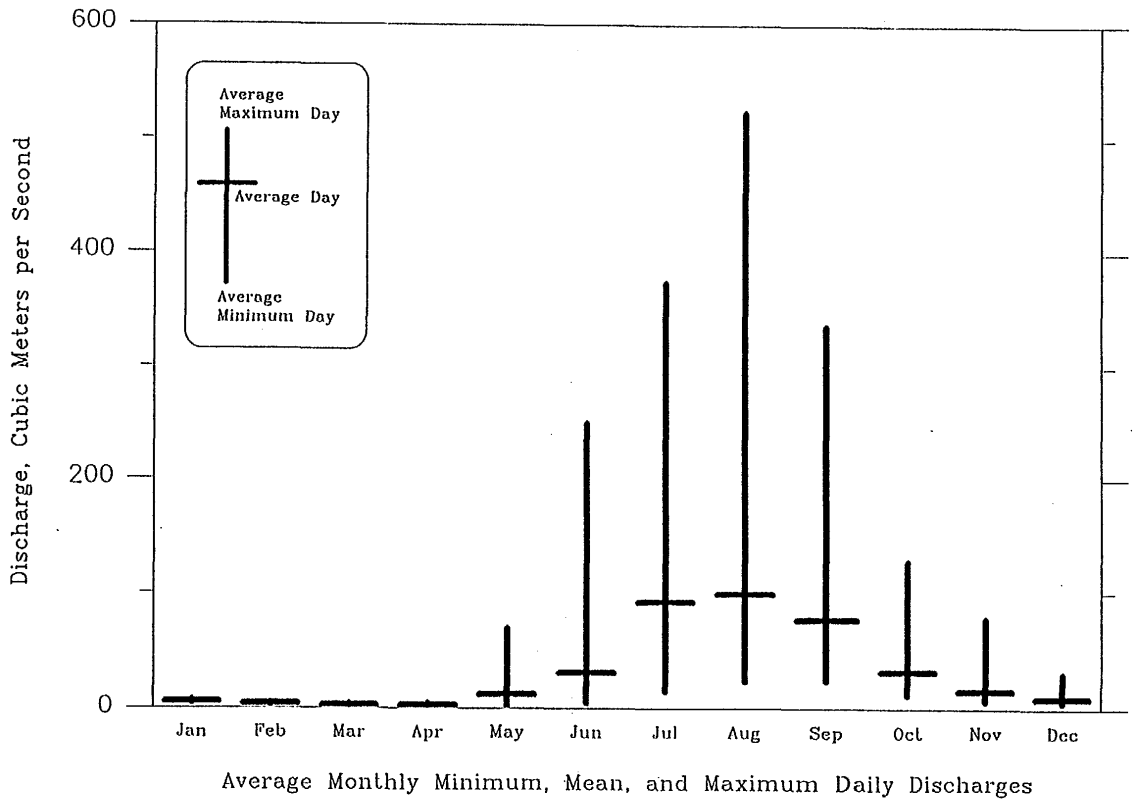


Figure 2.4.22

418

Monthly Flow Analysis
Station W011A; O'Donnell River, Palublub
Basin Area = 240 sq. km

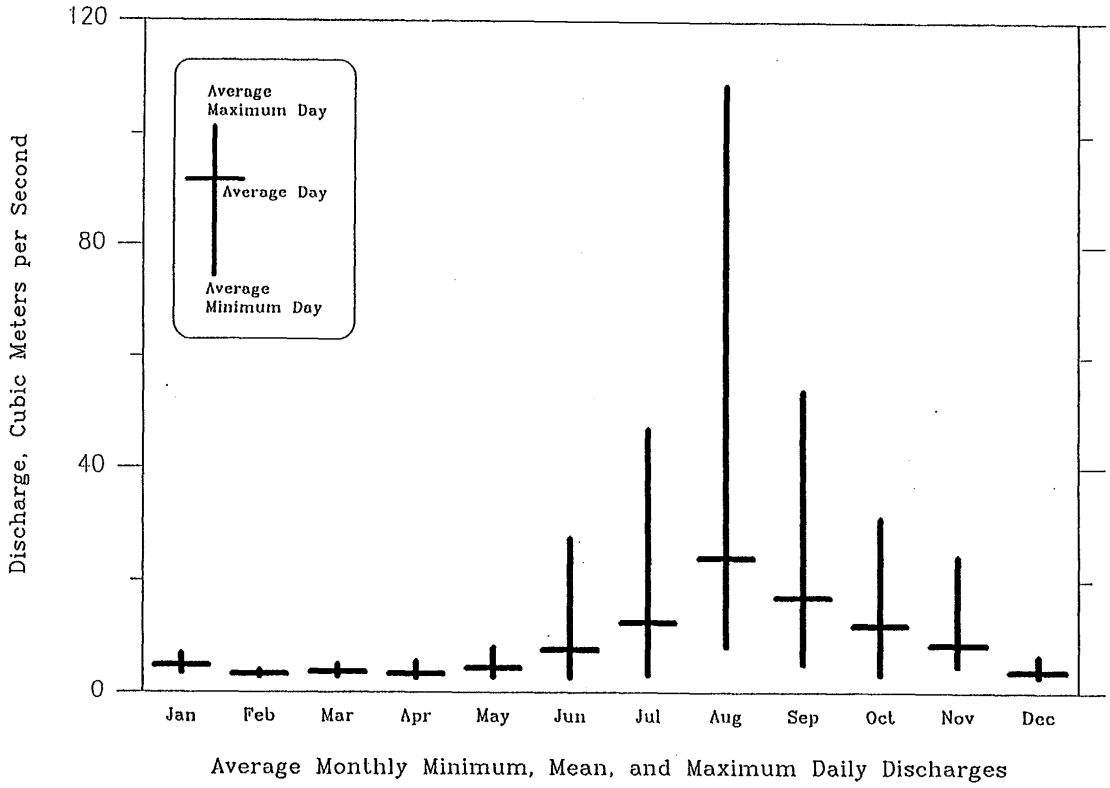


Figure 2.4.23

Monthly Flow Analysis
Station W011B; O'Donnell River, Patling
Basin Area = 112 sq. km

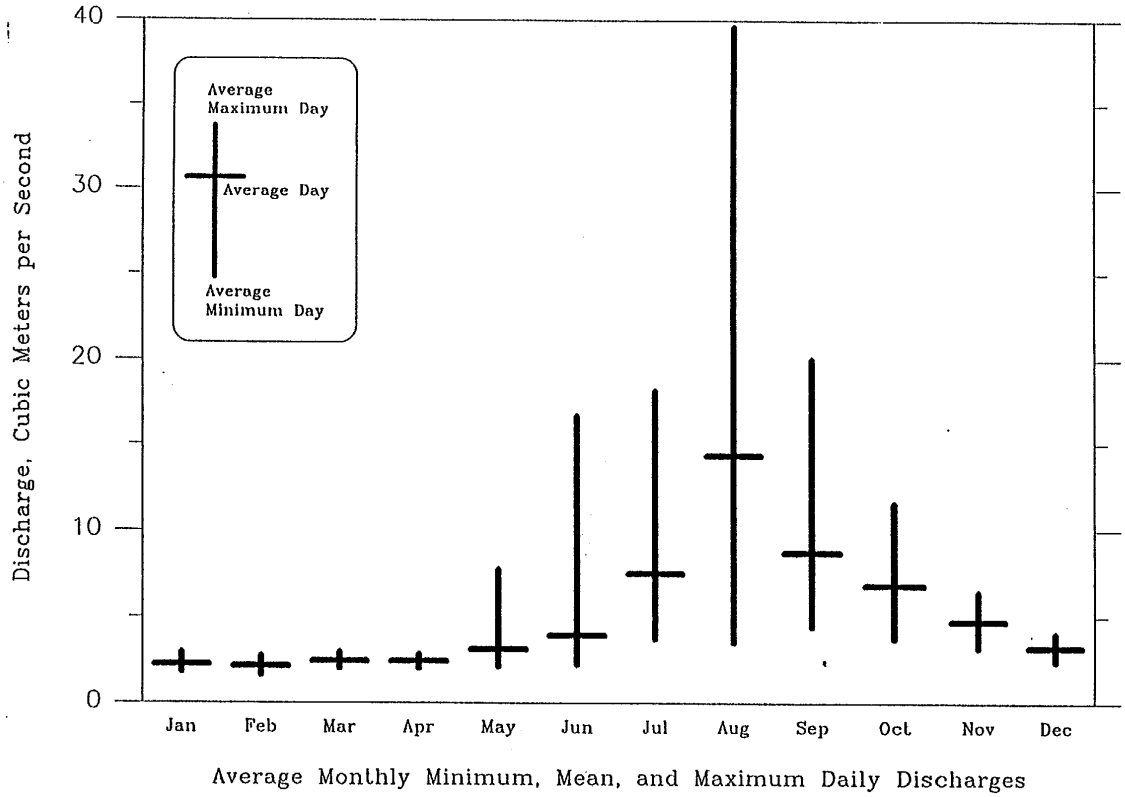
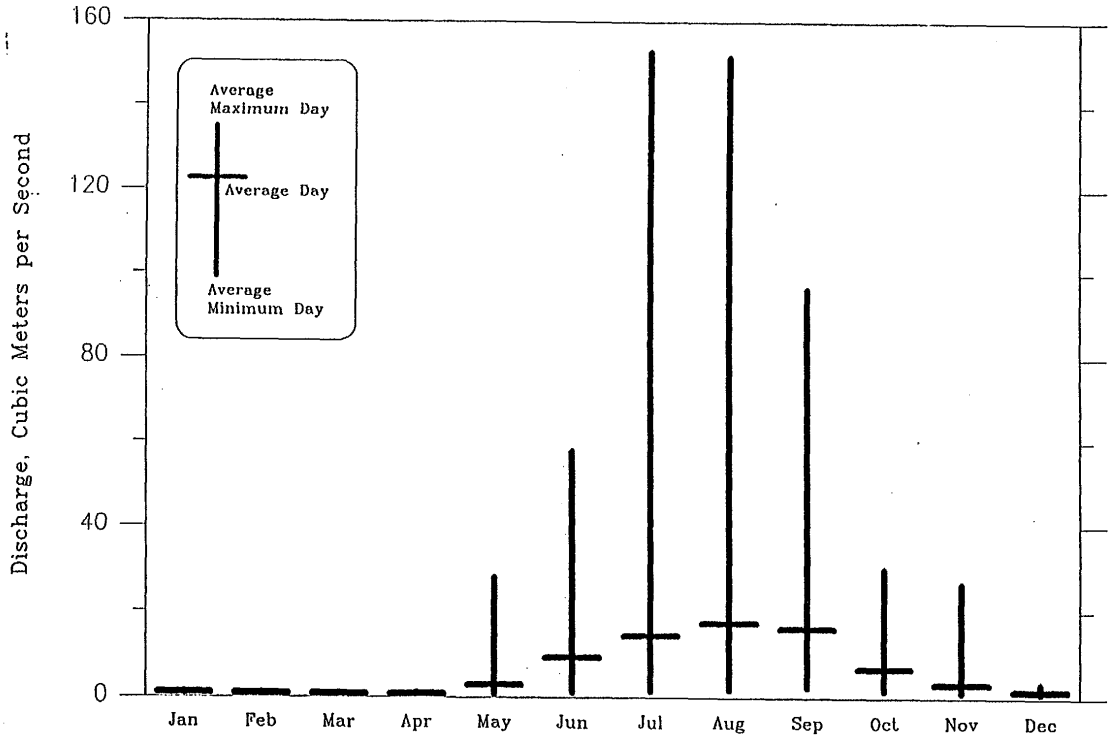


Figure 2.4.24

Monthly Flow Analysis
Station W012A; Bangat River, Sta Lucia
Basin Area = 90 sq. km.



Average Monthly Minimum, Mean, and Maximum Daily Discharges

Figure 2.4.25

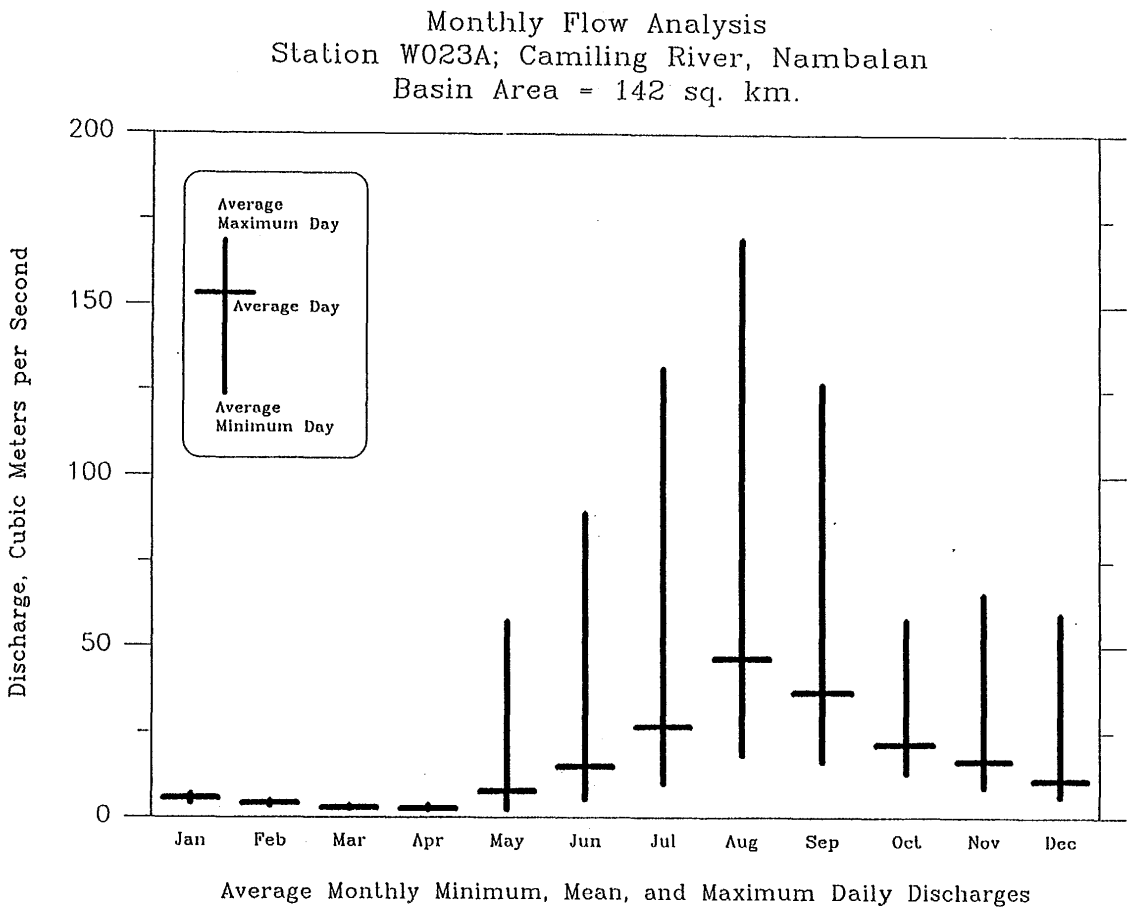

Monthly Flow Analysis
Station W023A; Camiling River, Nambalan
Basin Area = 142 sq. km.
Average Maximum Day
Average Day
Average Minimum Day
Discharge, Cubic Meters per Second
200
150
100
50
0
Jan
Feb
Mar
Apr
May
Jun
Jul
Aug
Sep
Oct
Nov
Dec
Average Monthly Minimum, Mean, and Maximum Daily Discharges

Figure 2.4.26

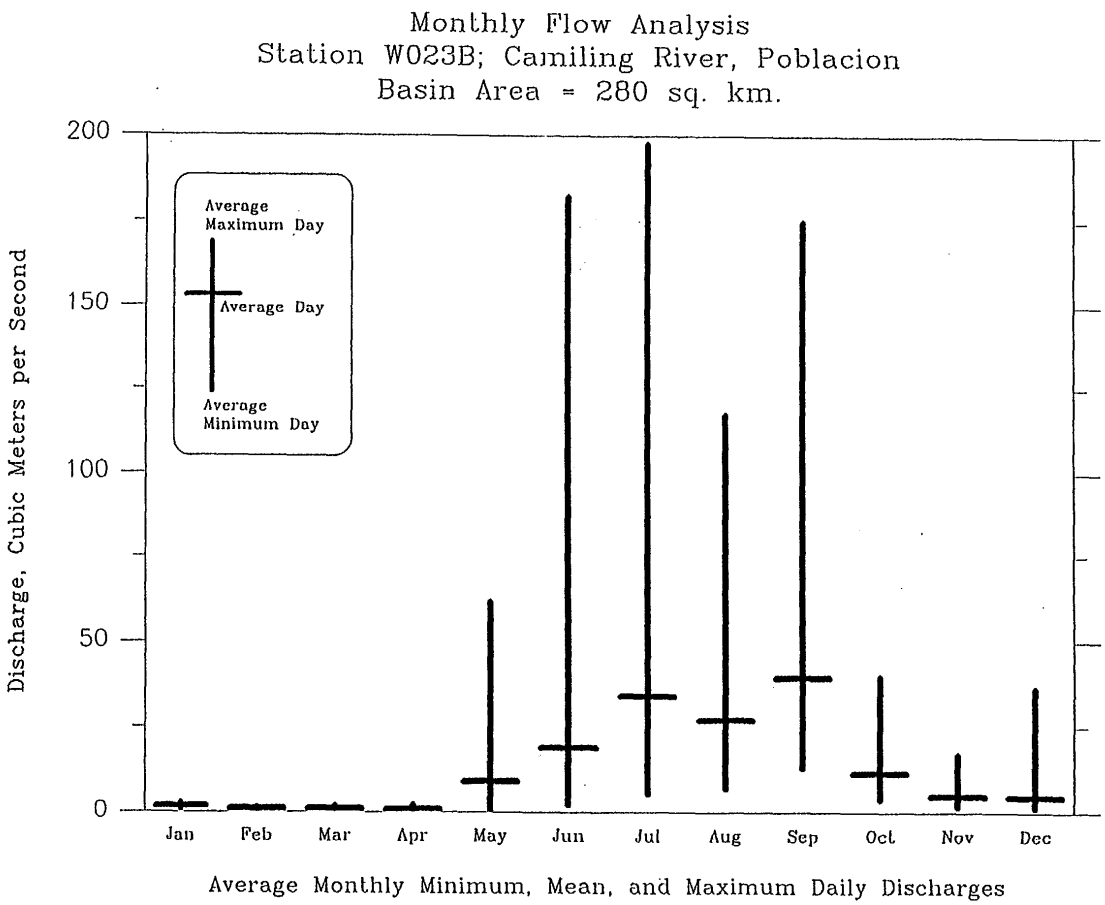

Monthly Flow Analysis
Station W023B; Camiling River, Poblacion
Basin Area = 280 sq. km.
Discharge, Cubic Meters per Second
200
150
100
50
0
Average Maximum Day
Average Day
Average Minimum Day
Jan
Feb
Mar
Apr
May
Jun
Jul
Aug
Sep
Oct
Nov
Dec
Average Monthly Minimum, Mean, and Maximum Daily Discharges

Figure 2.4.27

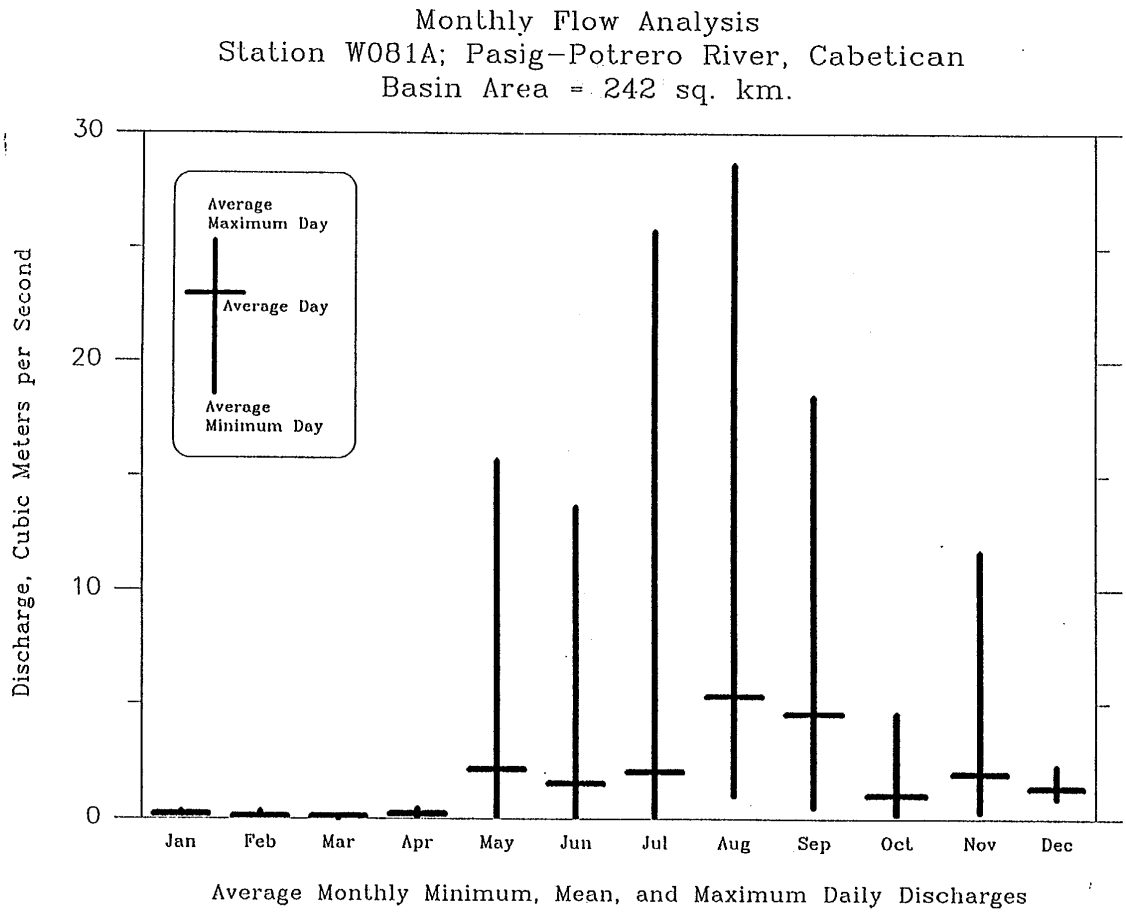
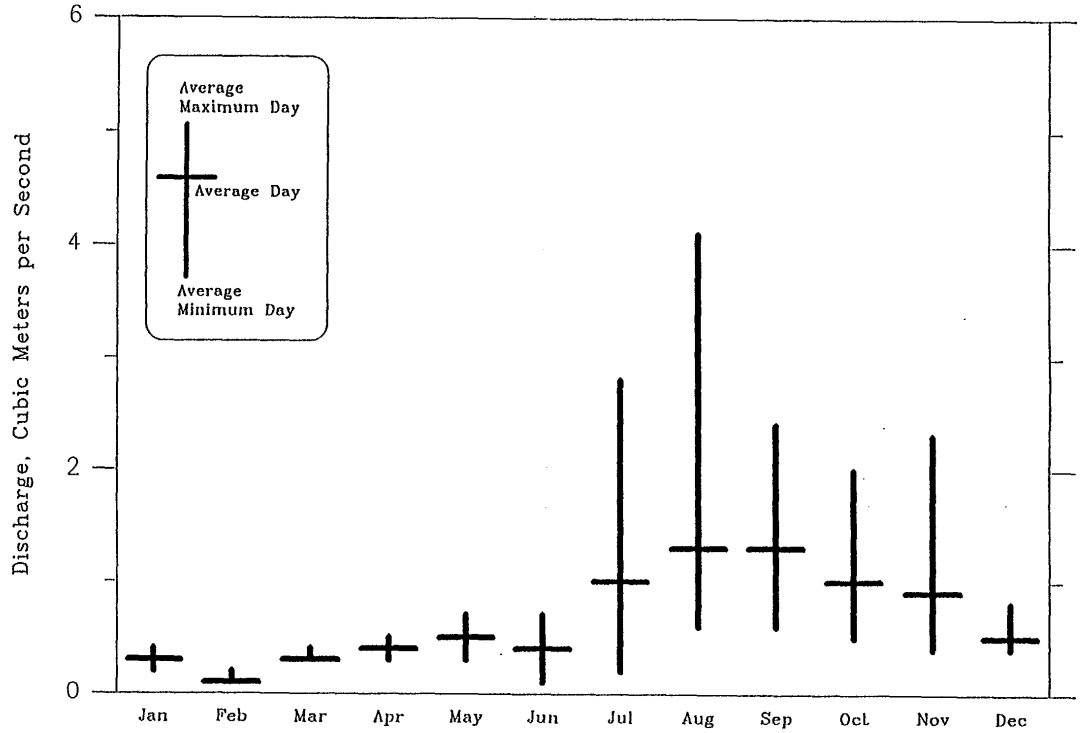

Monthly Flow Analysis
Station W081A; Pasig-Potrero River, Cabetican
Basin Area = 242 sq. km.
Average Maximum Day
Average Day
Average Minimum Day
Discharge, Cubic Meters per Second
30
20
10
0
Jan
Feb
Mar
Apr
May
Jun
Jul
Aug
Sep
Oct
Nov
Dec
Average Monthly Minimum, Mean, and Maximum Daily Discharges

Figure 2.4.28

Monthly Flow Analysis
Station W082A; Pasig-Potrero River, Hda Dolores
Basin Area = 28 sq. km.



Average Monthly Minimum, Mean, and Maximum Daily Discharges

Figure 2.4.29

425

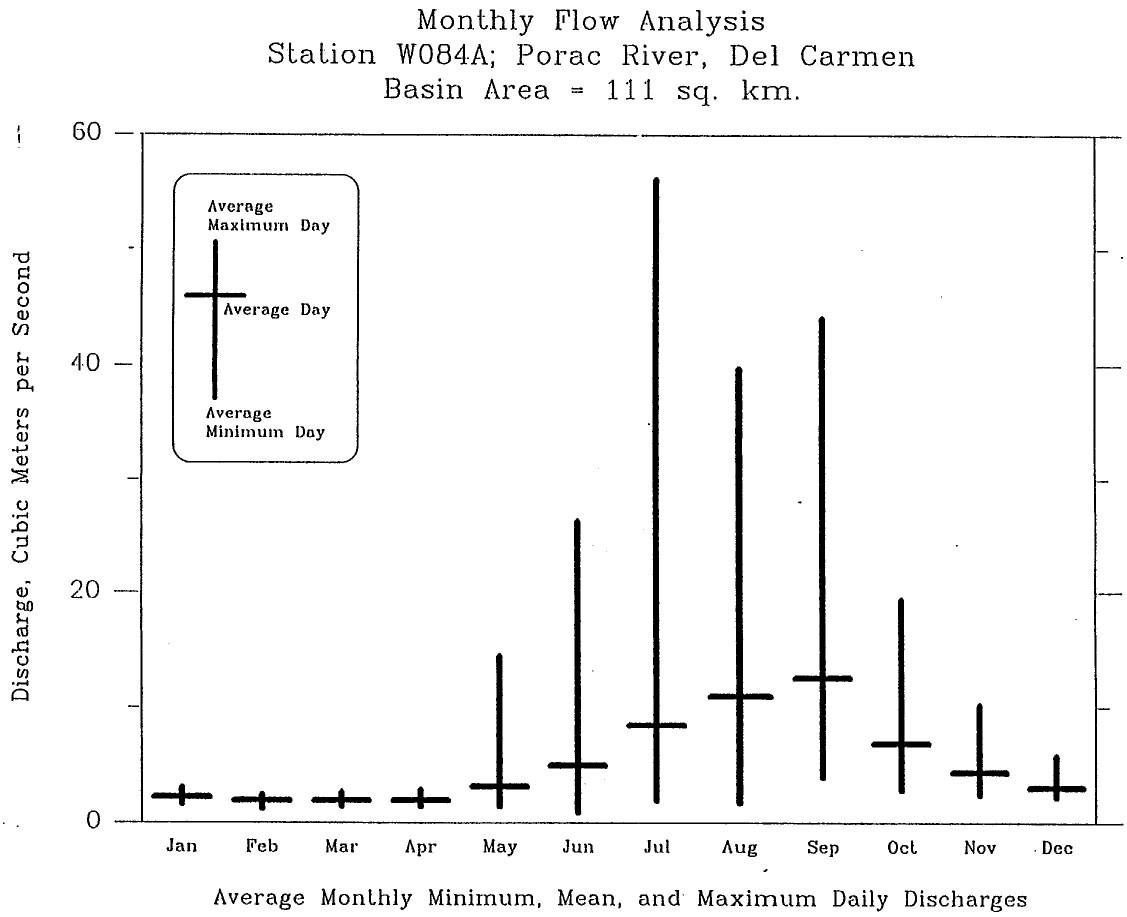


Figure 2.4.30

426

Monthly Flow Analysis
Station W086A; Gumain River, Pabanlag
Basin Area = 128 sq. km

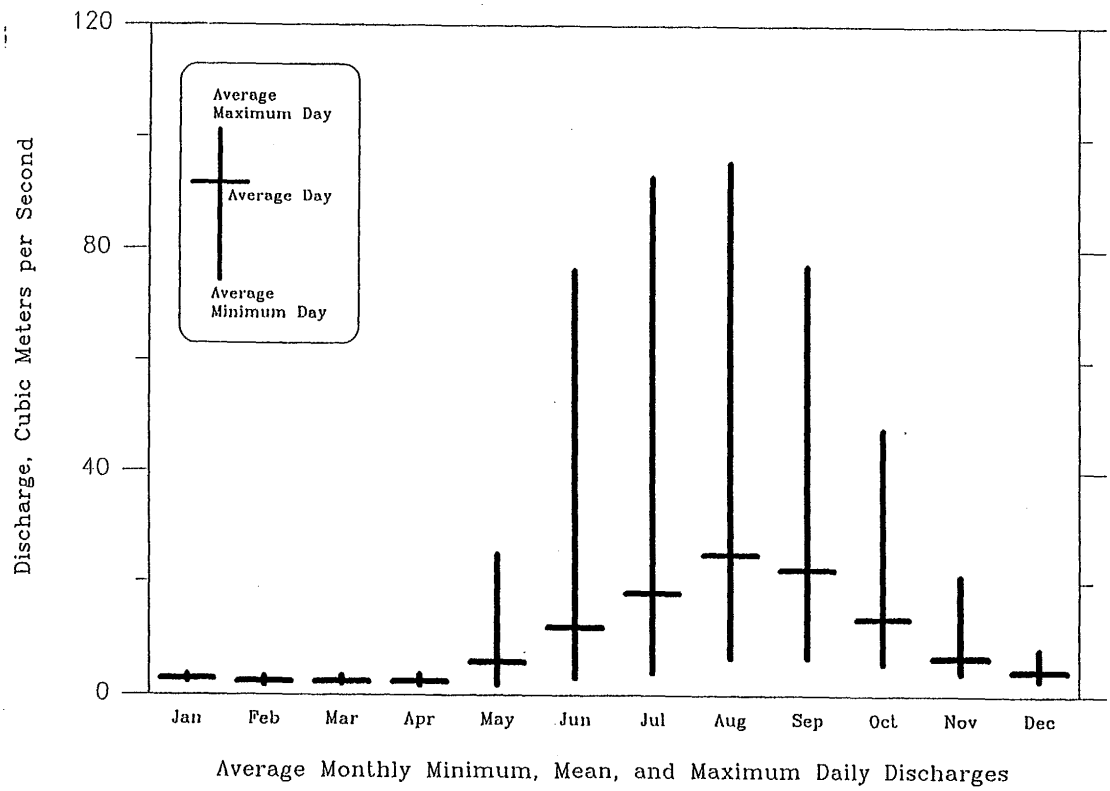


Figure 2.4.31

Monthly Flow Analysis
Station W088A; Colo River, San Benito
Basin Area = 76 sq. km

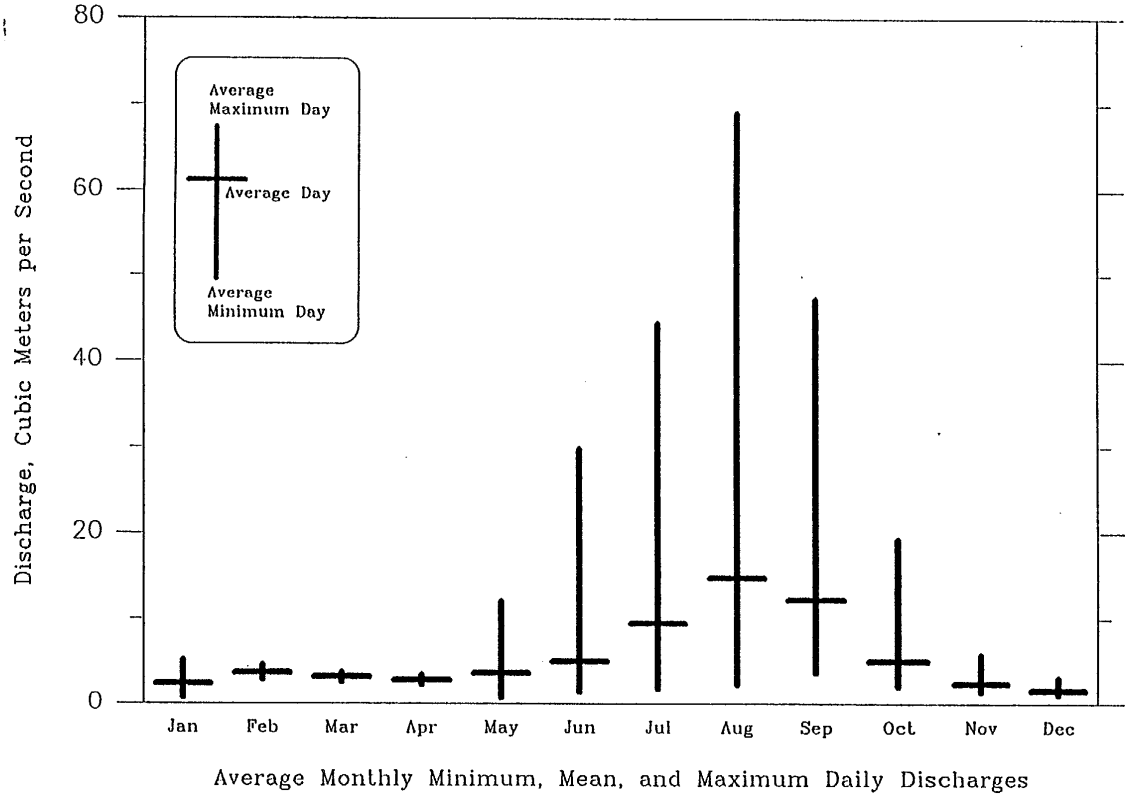
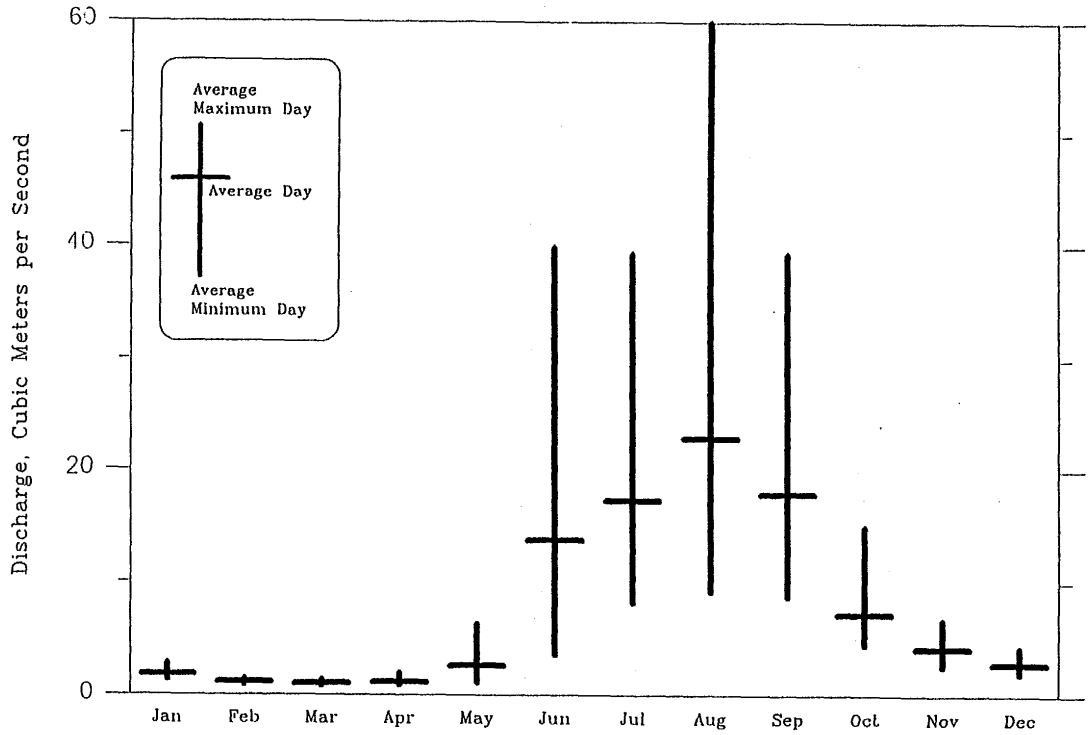


Figure 2.4.32

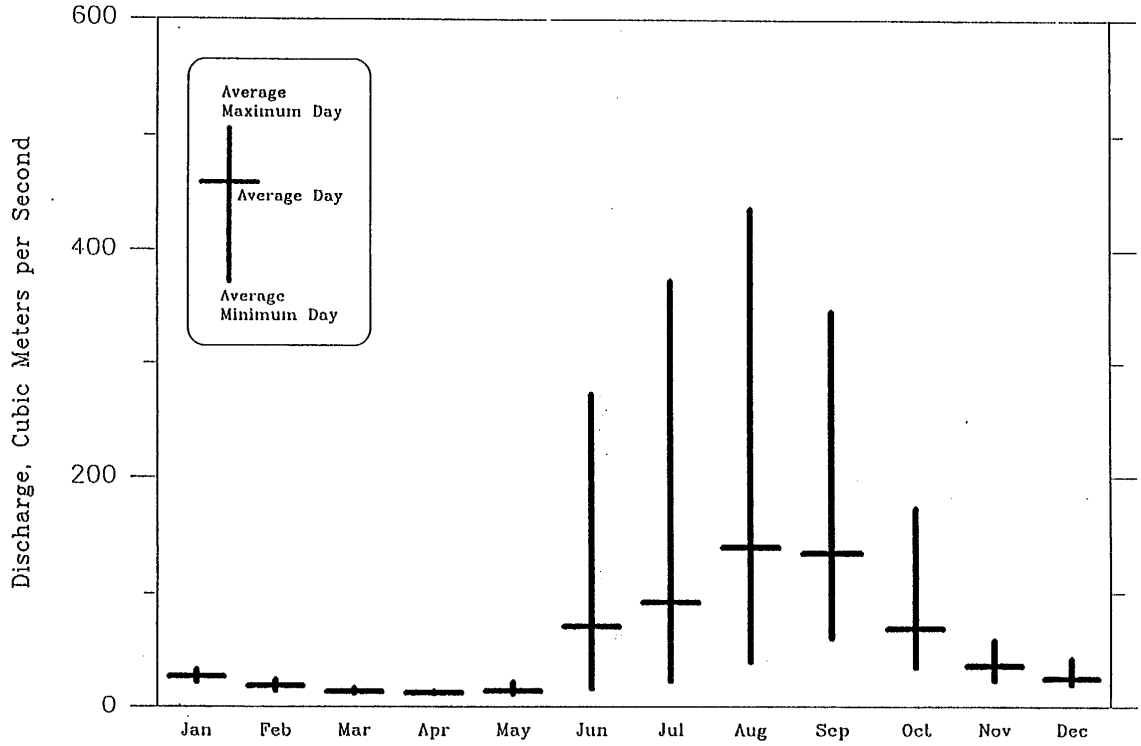
Monthly Flow Analysis
Station W092A; Bagsit River, Dampai
Basin Area = 68 sq. km



Average Monthly Minimum, Mean, and Maximum Daily Discharges

Figure 2.4.33

Monthly Flow Analysis
Station W093A; Bucao River, San Juan
Basin Area = 615 sq. km



Average Monthly Minimum, Mean, and Maximum Daily Discharges

Figure 2.4.34

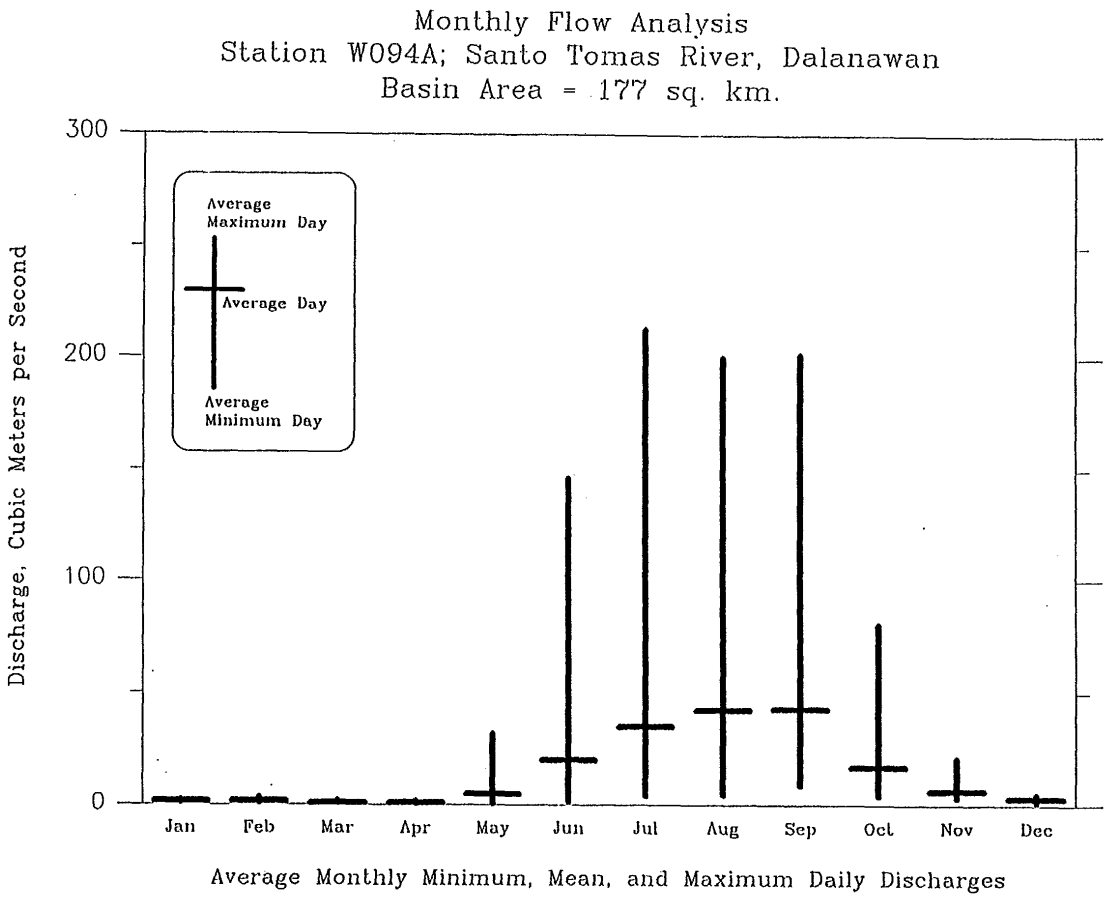

Monthly Flow Analysis
Station W094A; Santo Tomas River, Dalanawan
Basin Area = 177 sq. km.
Average Maximum Day
Average Day
Average Minimum Day
Discharge, Cubic Meters per Second
300
200
100
0
Jan
Feb
Mar
Apr
May
Jun
Jul
Aug
Sep
Oct
Nov
Dec
Average Monthly Minimum, Mean, and Maximum Daily Discharges

Figure 2.4.35

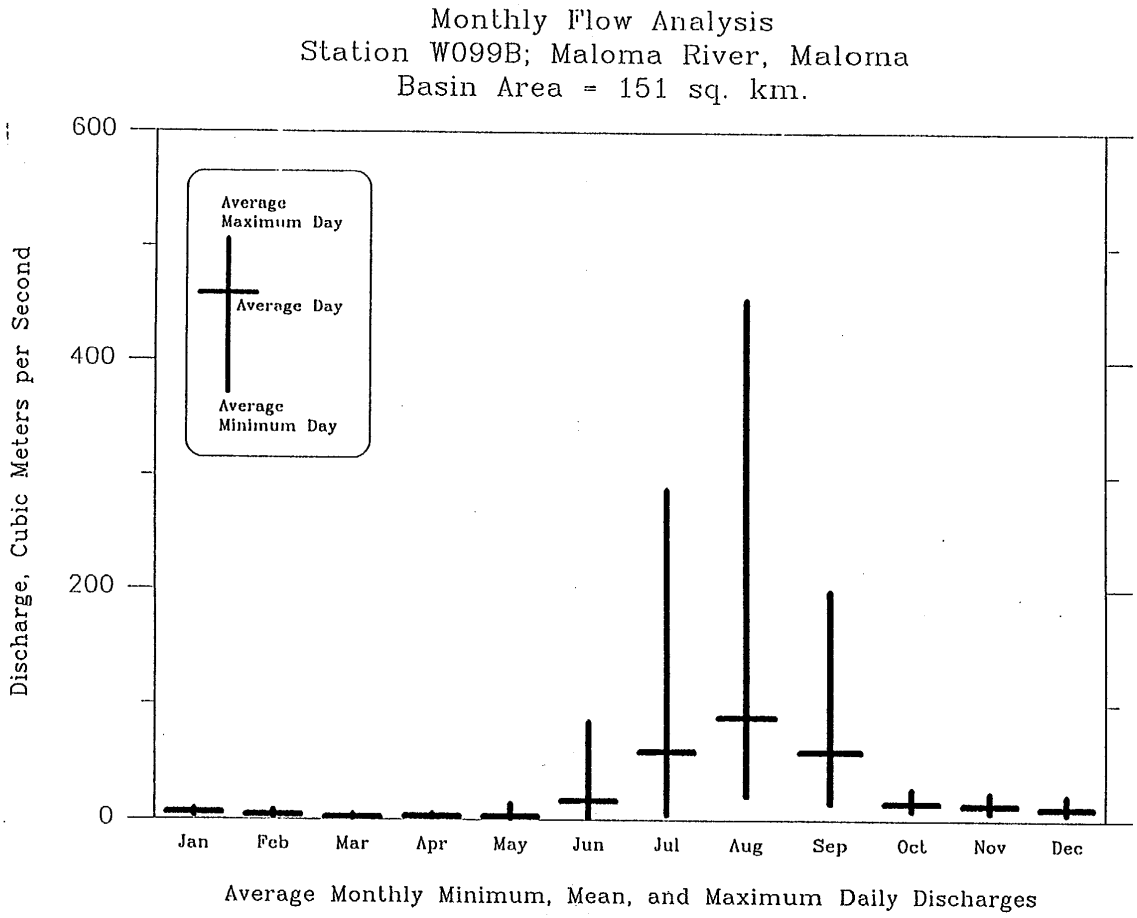

Monthly Flow Analysis
Station W099B; Maloma River, Maloma
Basin Area = 151 sq. km.
Average Maximum Day
Average Day
Average Minimum Day
Discharge, Cubic Meters per Second
600
400
200
0
Jan
Feb
Mar
Apr
May
Jun
Jul
Aug
Sep
Oct
Nov
Dec
Average Monthly Minimum, Mean, and Maximum Daily Discharges

Figure 2.4.36

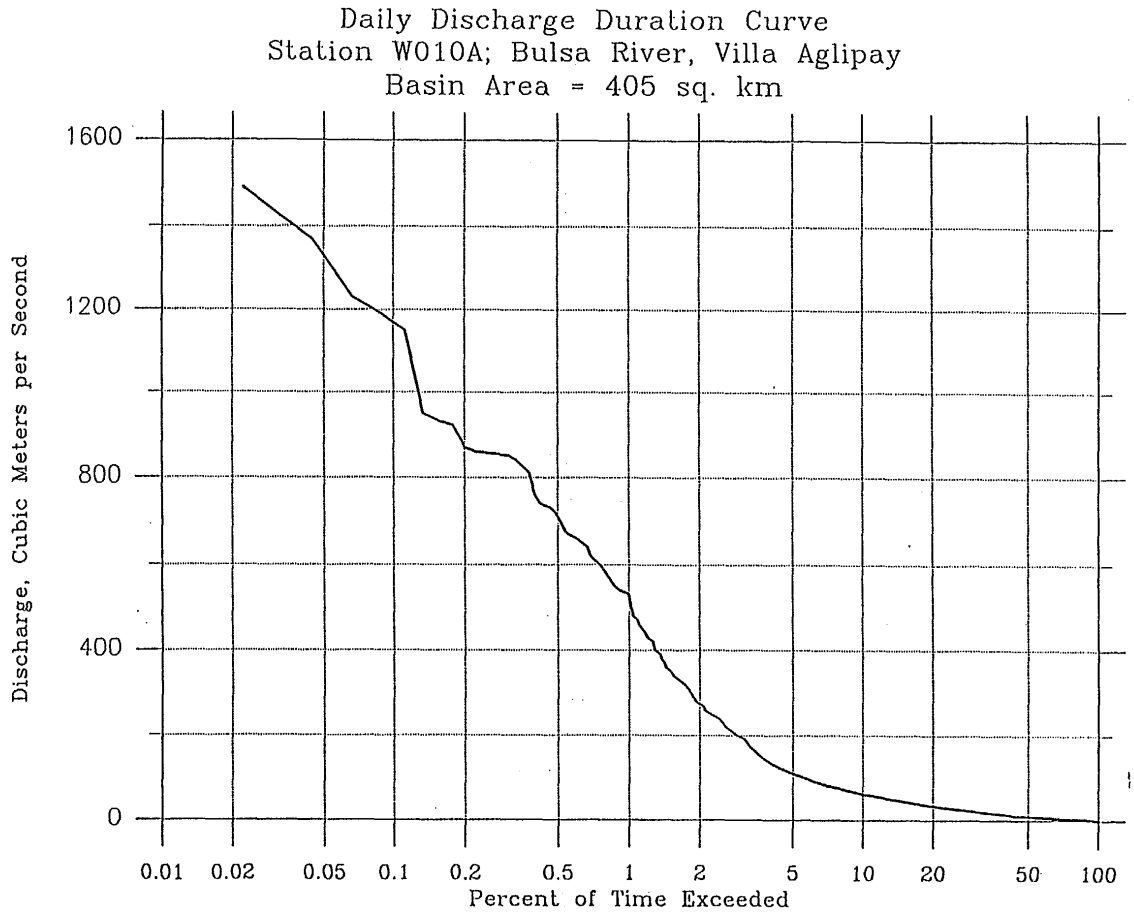


Figure 2.4.37

435

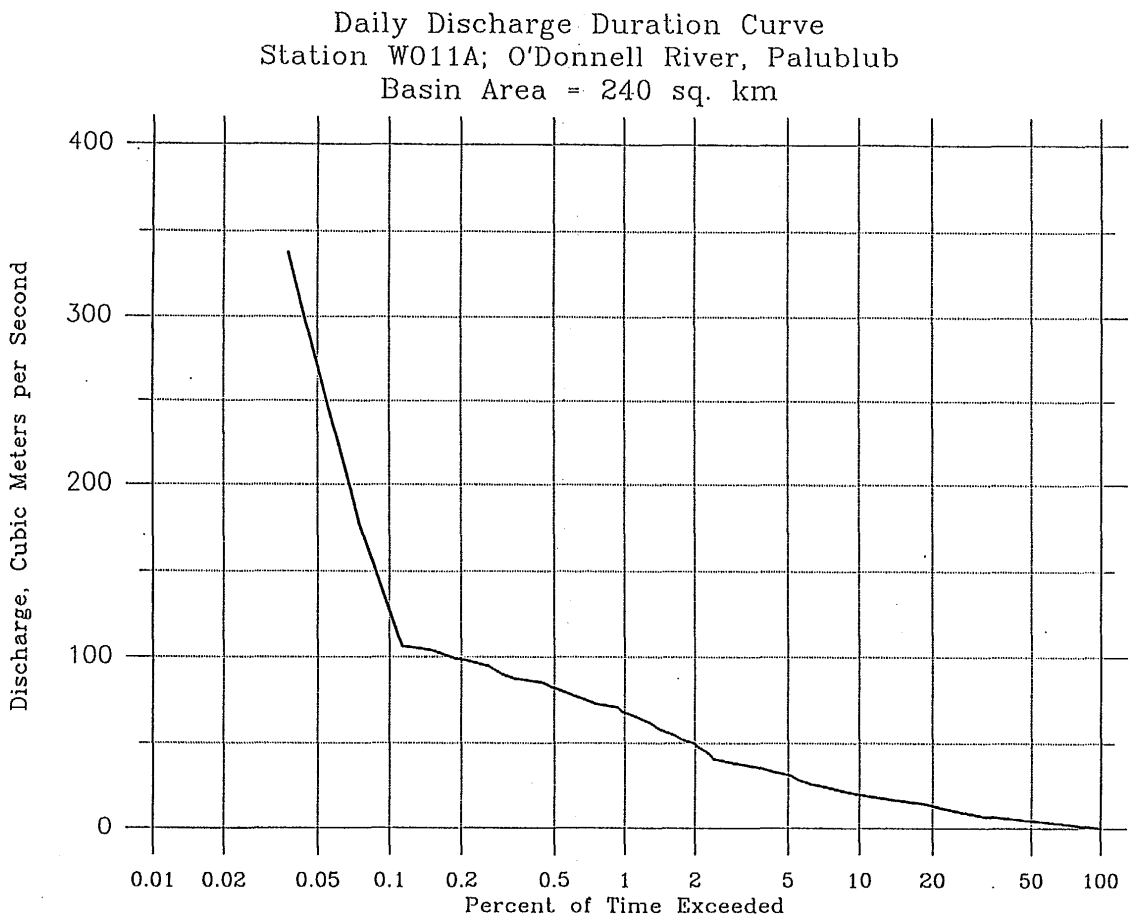

Daily Discharge Duration Curve
Station W011A; O'Donnell River, Palublub
Basin Area = 240 sq. km
Discharge, Cubic Meters per Second
400
300
200
100
0
0.01
0.02
0.05
0.1
0.2
0.5
1
2
5
10
20
50
100
Percent of Time Exceeded

Figure 2.4.38

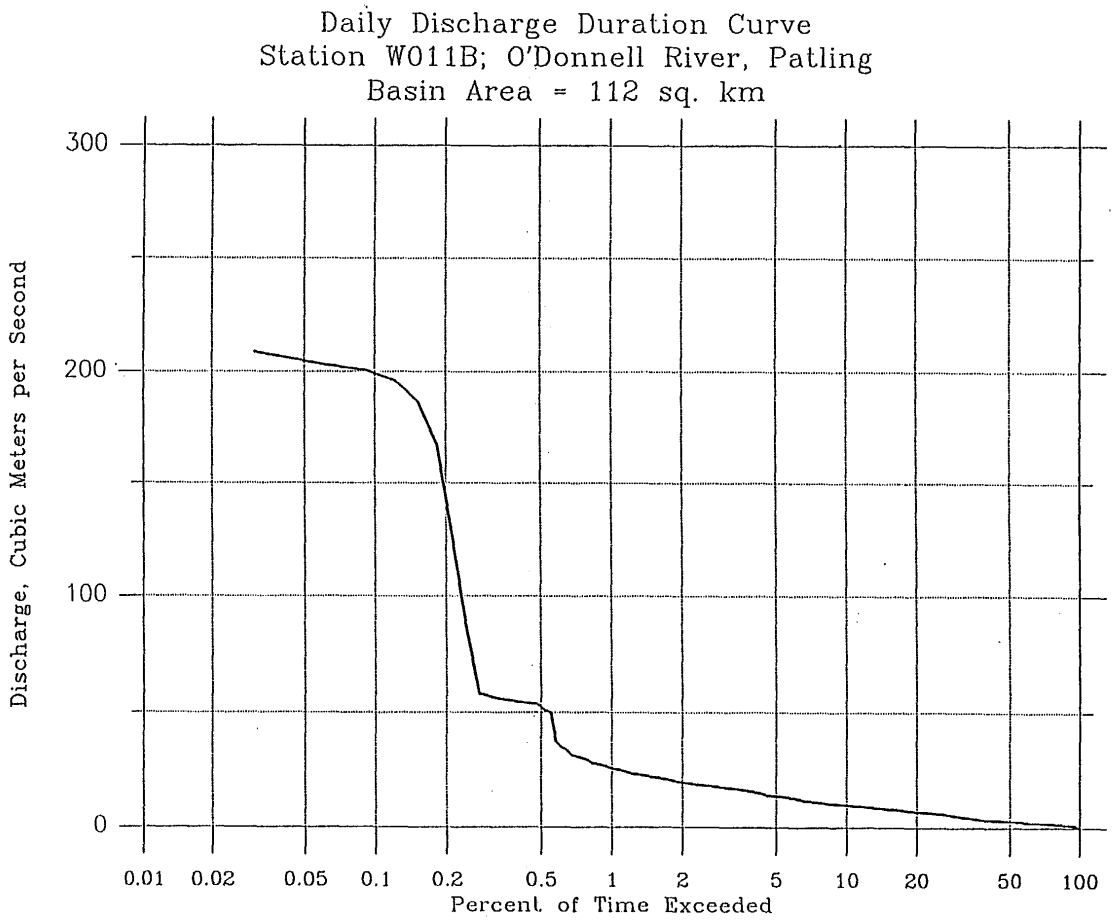

Daily Discharge Duration Curve
Station W011B; O'Donnell River, Patling
Basin Area = 112 sq. km
Discharge, Cubic Meters per Second
300
200
100
0
0.01
0.02
0.05
0.1
0.2
0.5
1
2
5
10
20
50
100
Percent of Time Exceeded

Figure 2.4.39

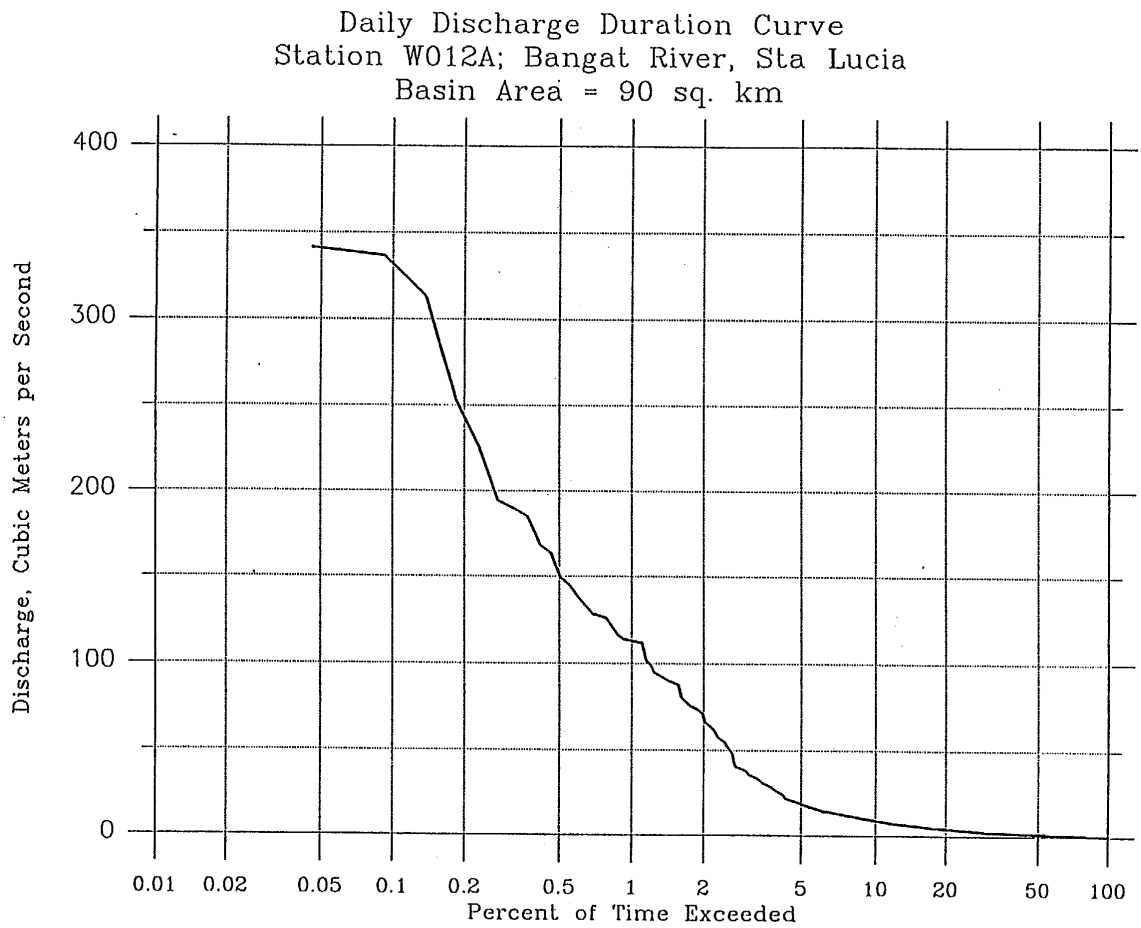

Daily Discharge Duration Curve
Station W012A; Bangat River, Sta Lucia
Basin Area = 90 sq. km
Discharge, Cubic Meters per Second
400
300
200
100
0
0.01
0.02
0.05
0.1
0.2
0.5
1
2
5
10
20
50
100
Percent of Time Exceeded

Figure 2.4.40

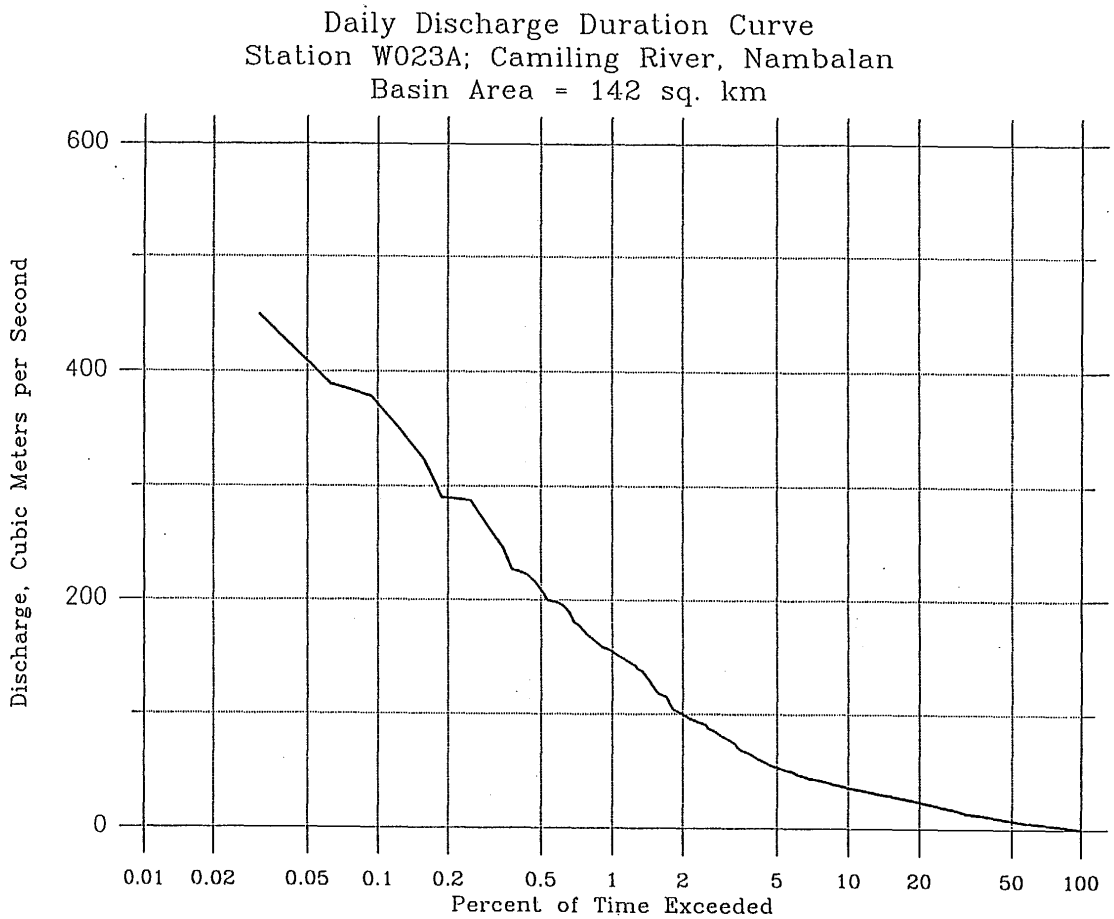


Figure 2.4.41

457

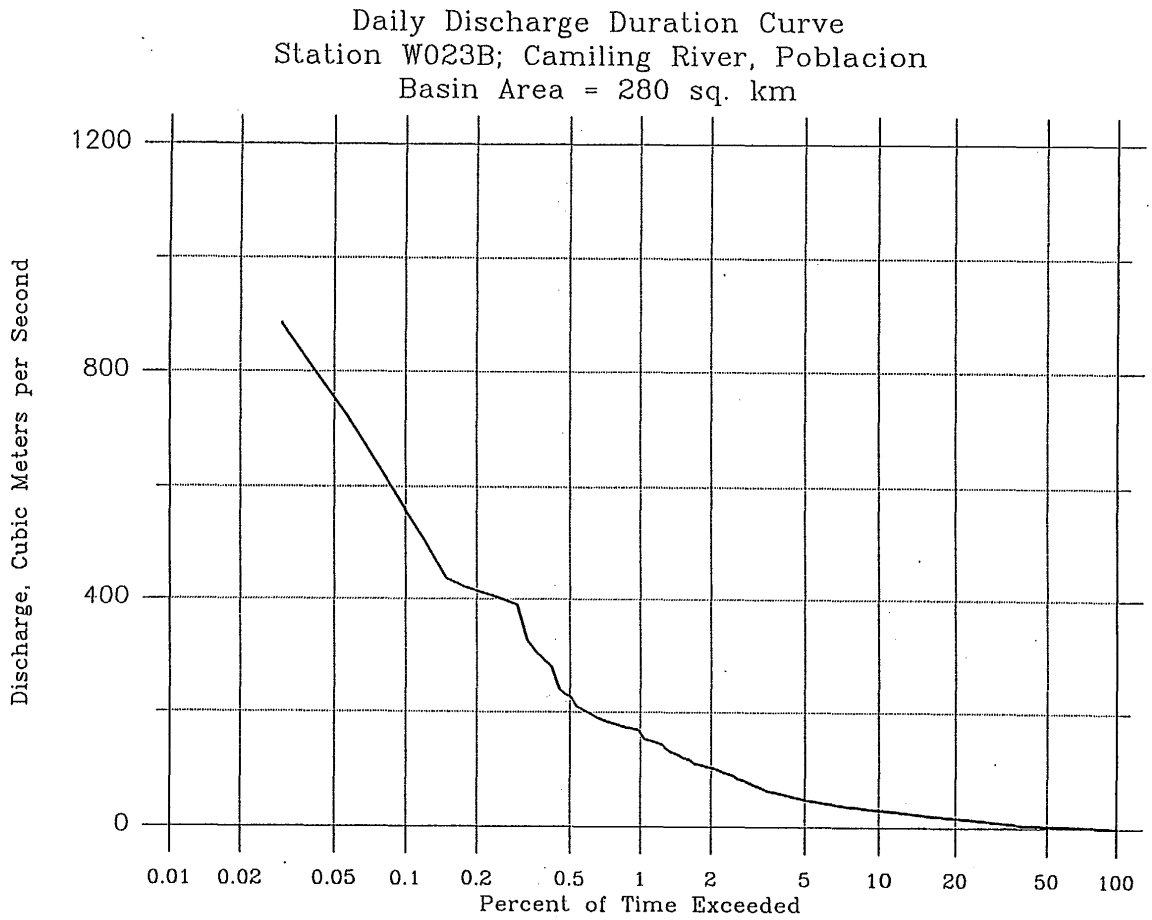

Daily Discharge Duration Curve
Station W023B; Camiling River, Poblacion
Basin Area = 280 sq. km
Discharge, Cubic Meters per Second
1200
800
400
0
0.01
0.02
0.05
0.1
0.2
0.5
1
2
5
10
20
50
100
Percent of Time Exceeded

Figure 2.4.42

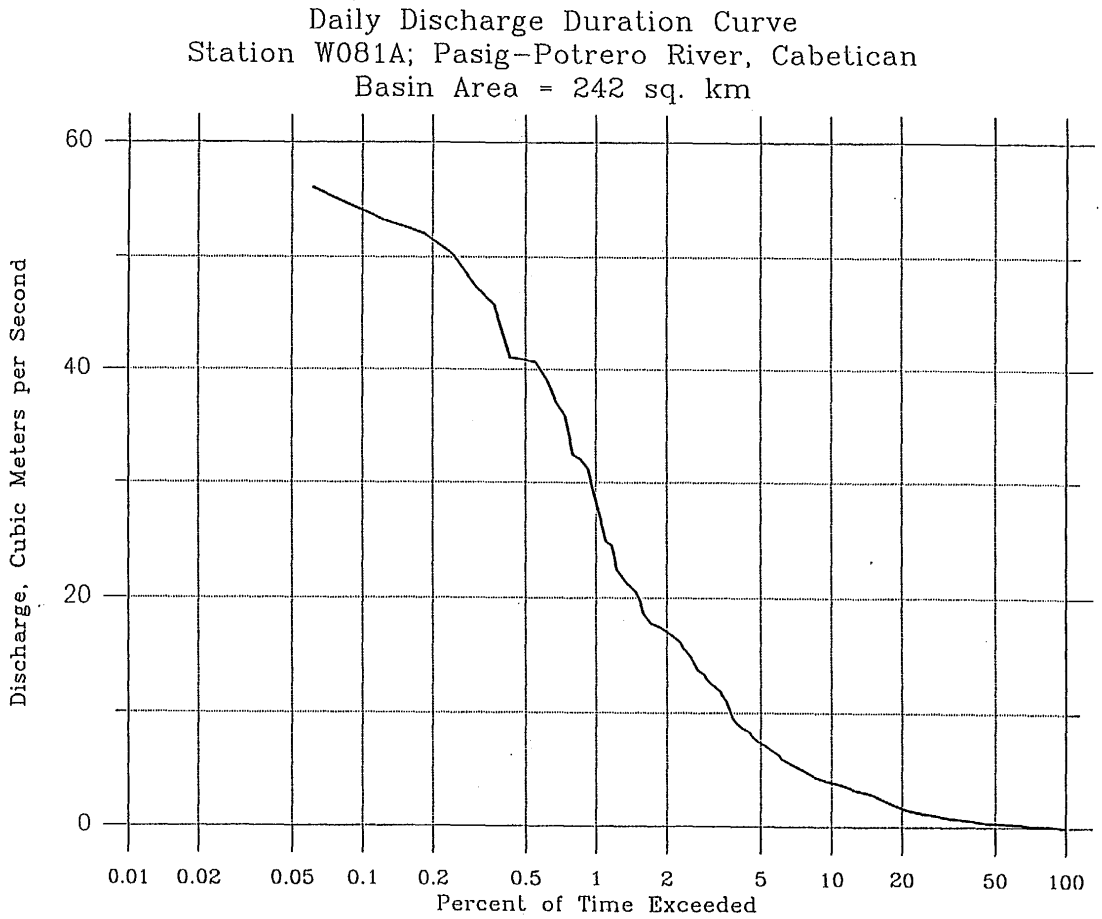


Figure 2.4.43

439

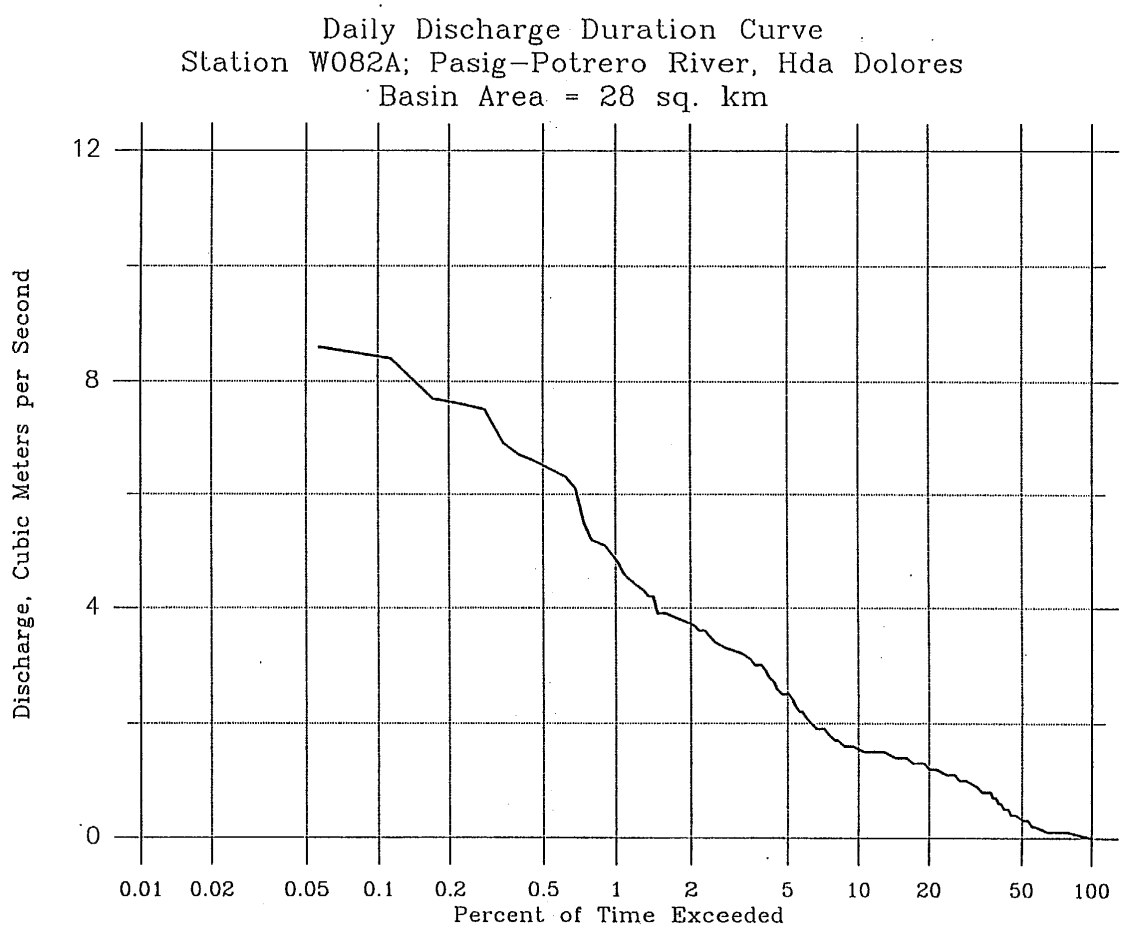


Figure 2.4.44

440

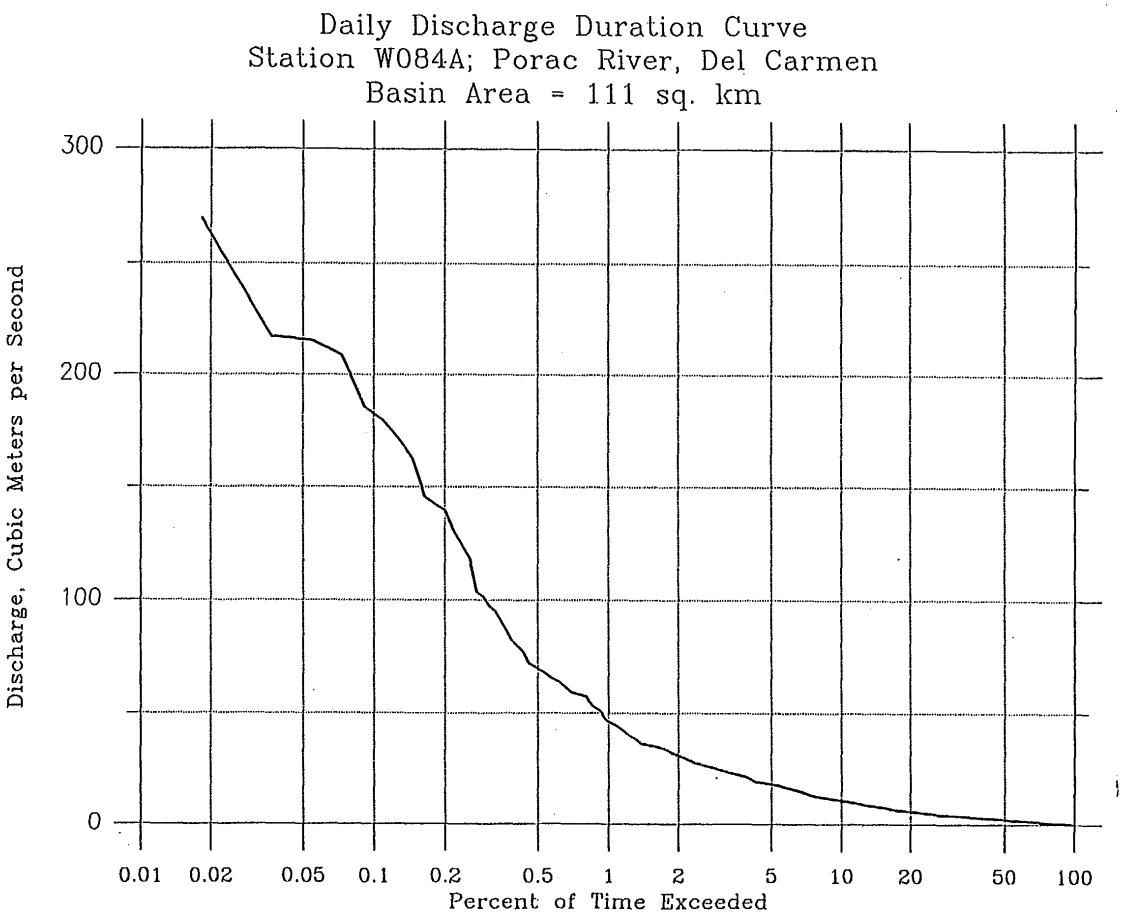

Daily Discharge Duration Curve
Station W084A; Porac River, Del Carmen
Basin Area = 111 sq. km
Discharge, Cubic Meters per Second
300
200
100
0
0.01
0.02
0.05
0.1
0.2
0.5
1
2
5
10
20
50
100
Percent of Time Exceeded

Figure 2.4.45

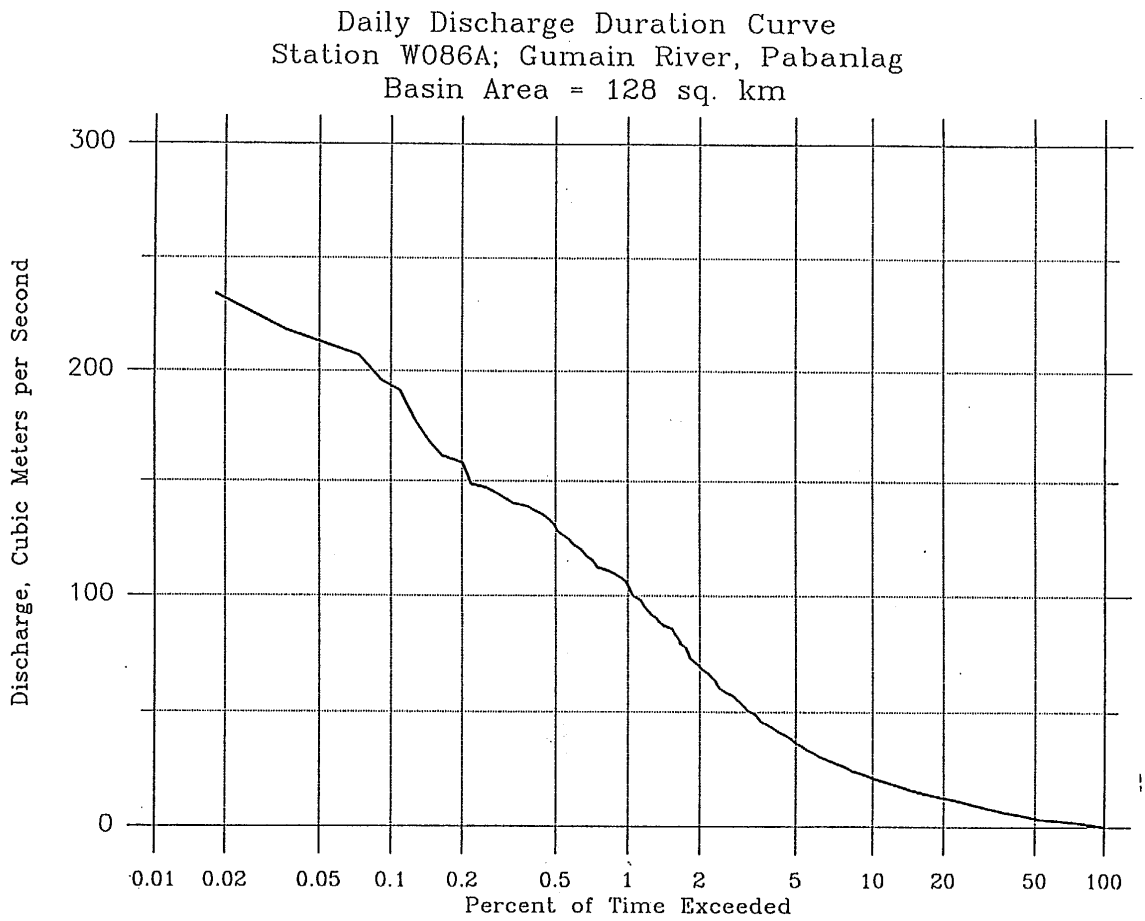

Daily Discharge Duration Curve
Station W086A; Gumain River, Pabanlag
Basin Area = 128 sq. km
Discharge, Cubic Meters per Second
300
200
100
0
0.01
0.02
0.05
0.1
0.2
0.5
1
2
5
10
20
50
100
Percent of Time Exceeded

Figure 2.4.46

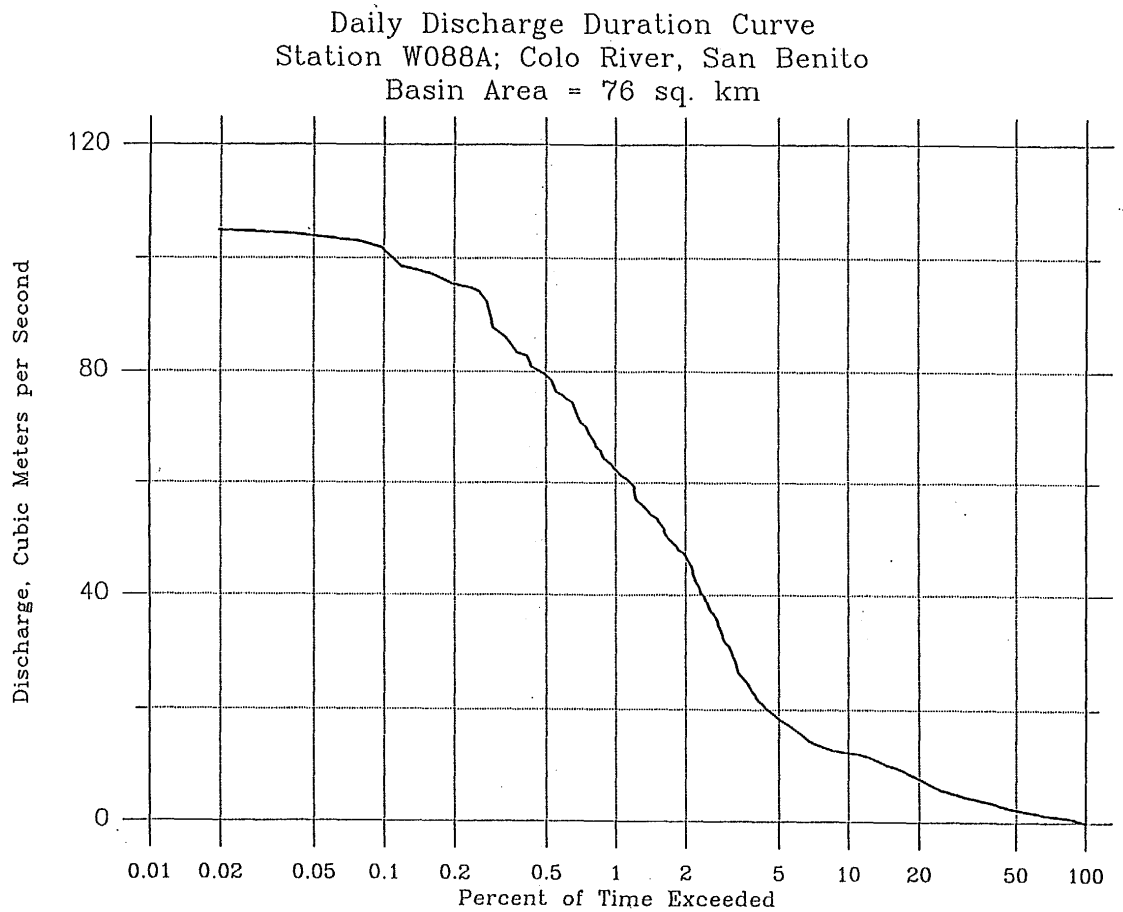


Figure 2.4.47

443

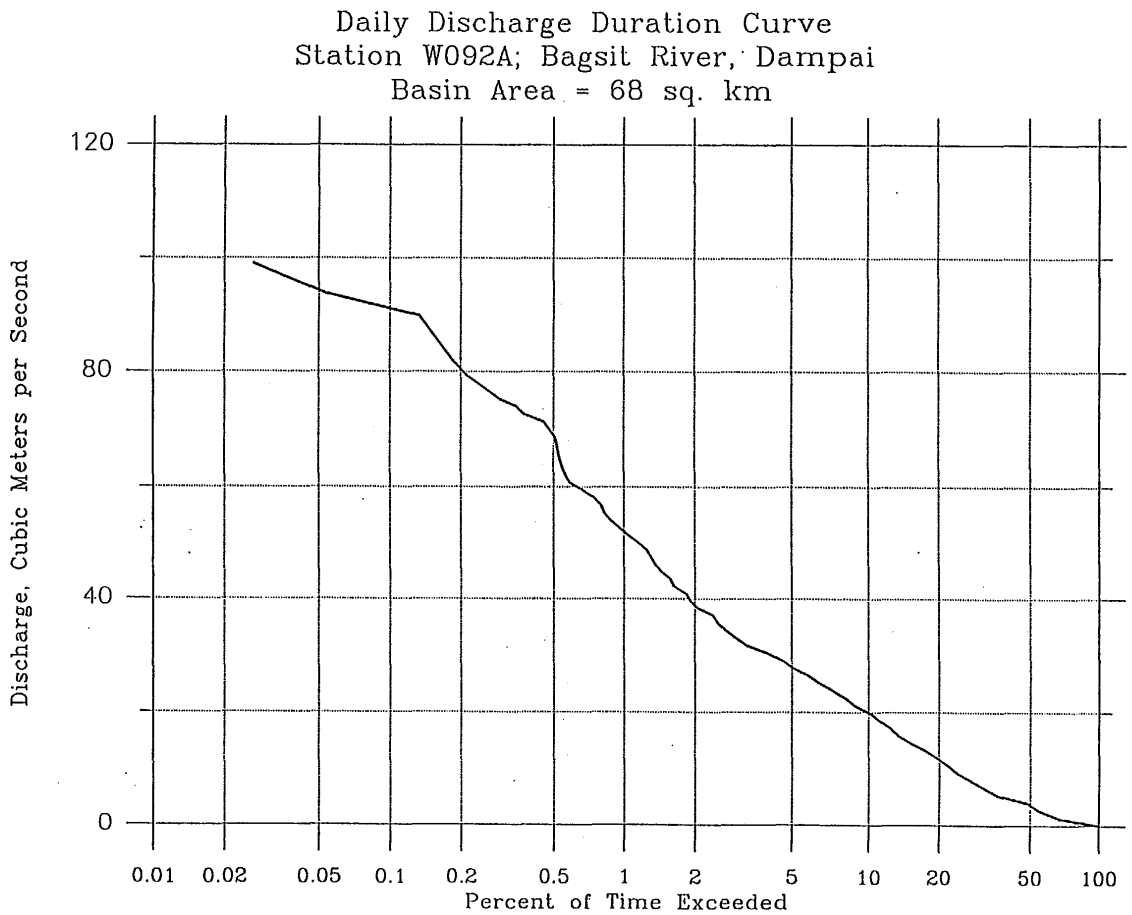

Daily Discharge Duration Curve
Station W092A; Bagsit River, Dampai
Basin Area = 68 sq. km
Discharge, Cubic Meters per Second
120
80
40
0
0.01
0.02
0.05
0.1
0.2
0.5
1
2
5
10
20
50
100
Percent of Time Exceeded

Figure 2.4.48

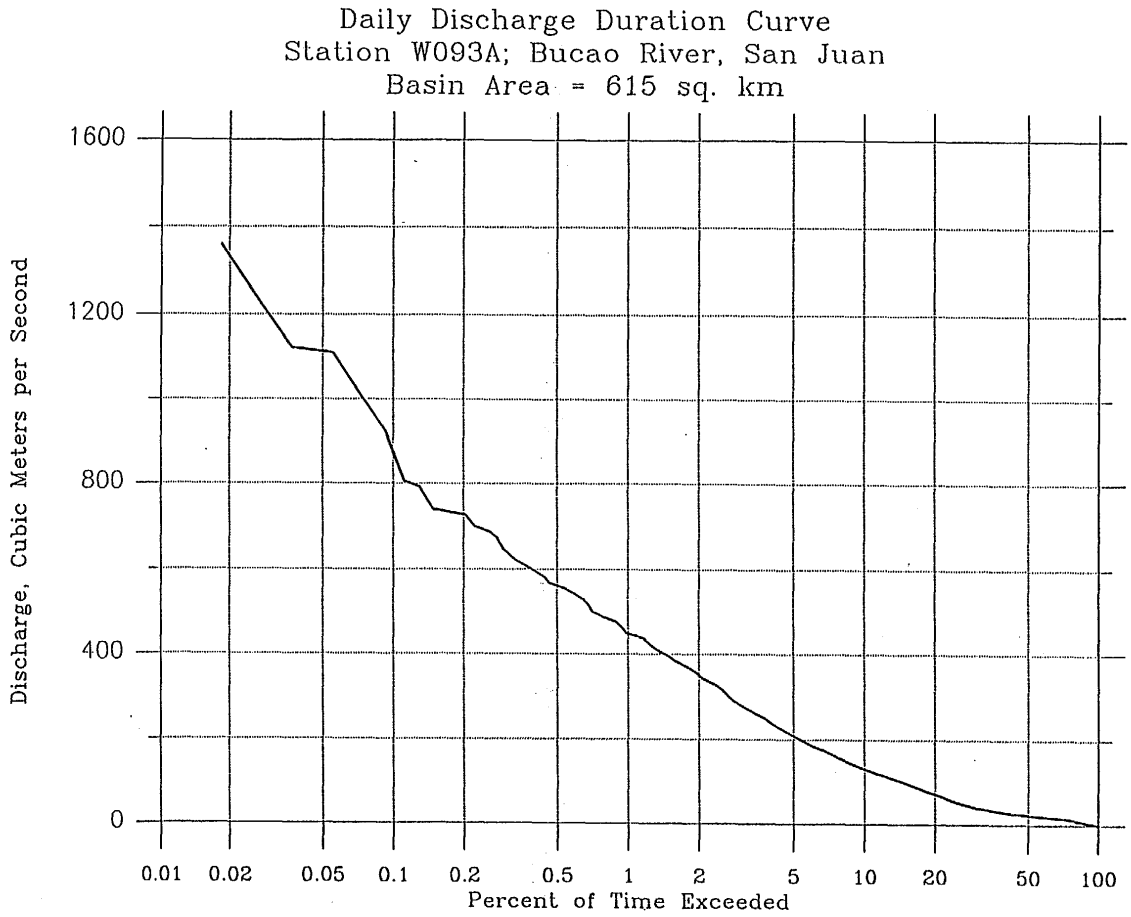

Daily Discharge Duration Curve
Station W093A; Bucao River, San Juan
Basin Area = 615 sq. km
Discharge, Cubic Meters per Second
1600
1200
800
400
0
0.01
0.02
0.05
0.1
0.2
0.5
1
2
5
10
20
50
100
Percent of Time Exceeded

Figure 2.4.49

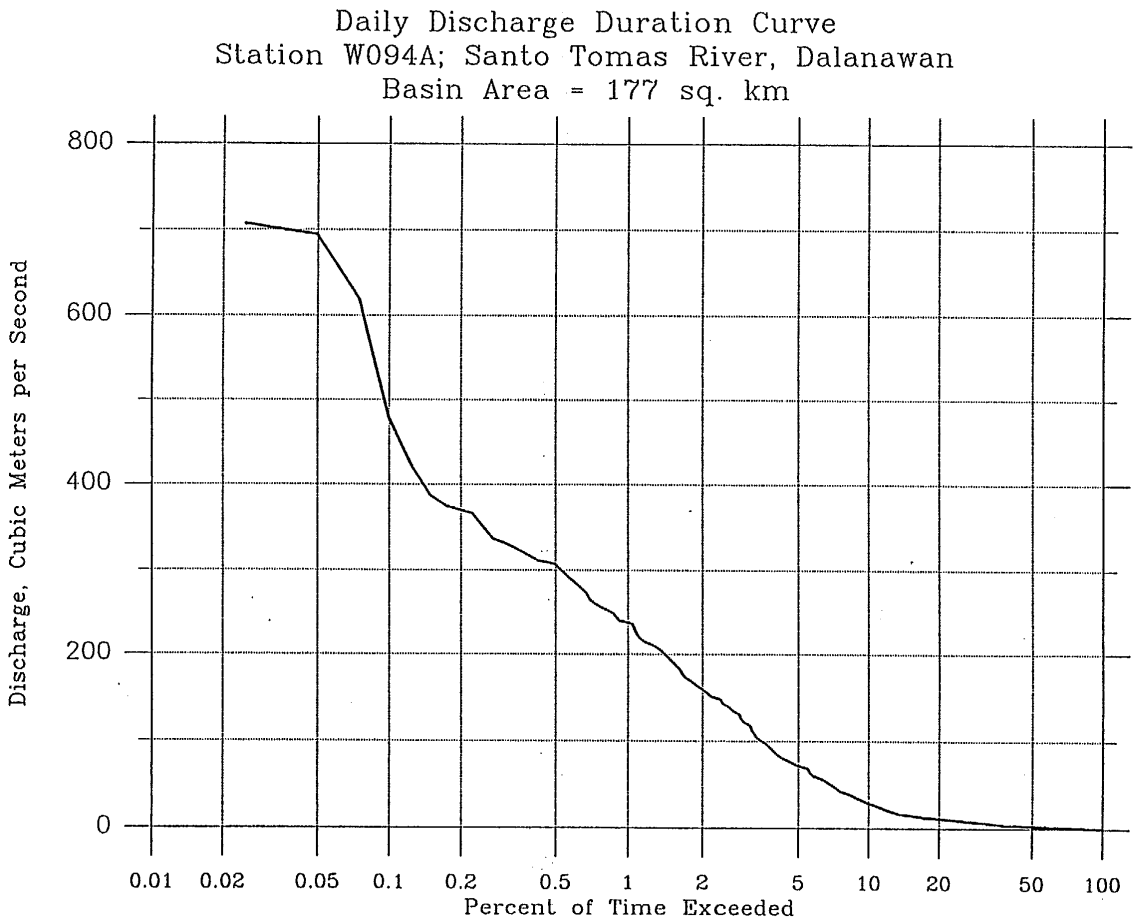

Daily Discharge Duration Curve
Station W094A; Santo Tomas River, Dalanawan
Basin Area = 177 sq. km
Discharge, Cubic Meters per Second
800
600
400
200
0
0.01
0.02
0.05
0.1
0.2
0.5
1
2
5
10
20
50
100
Percent of Time Exceeded

Figure 2.4.50

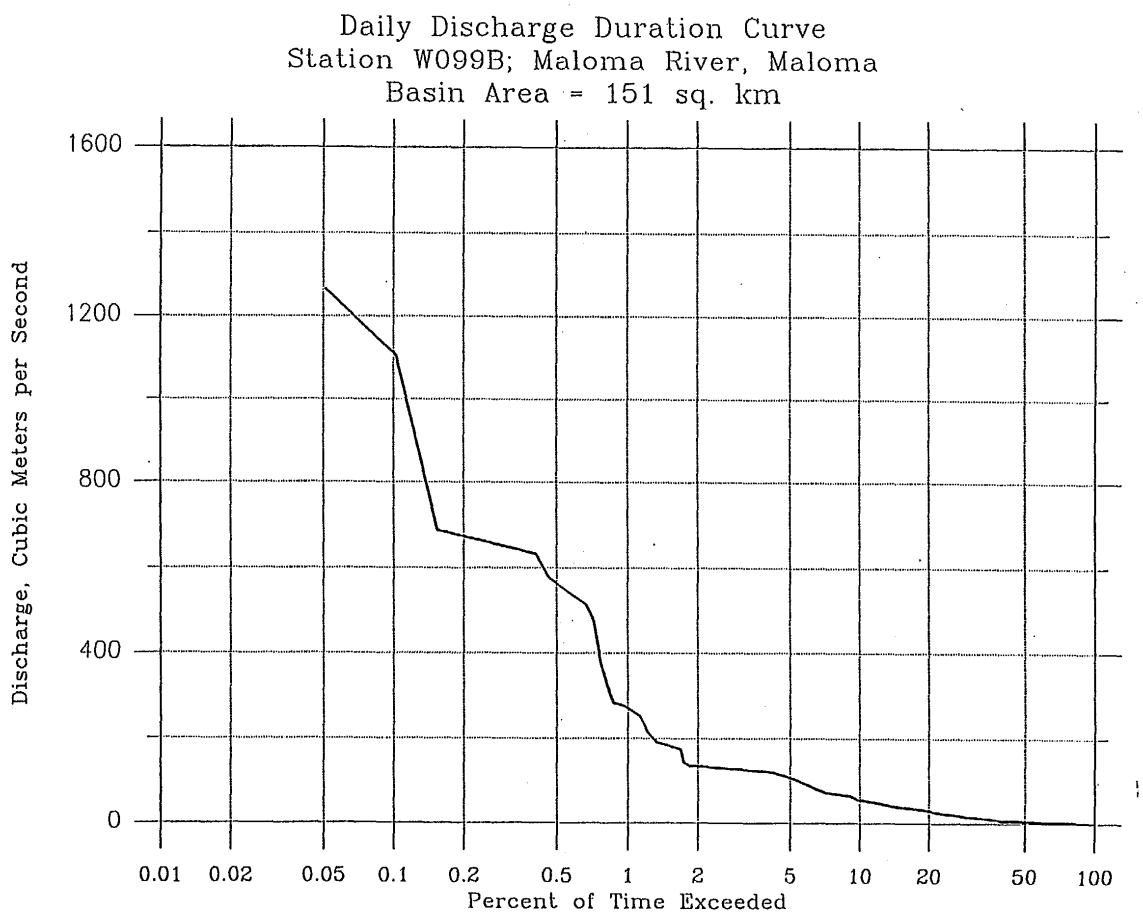


Figure 2.4.51

447

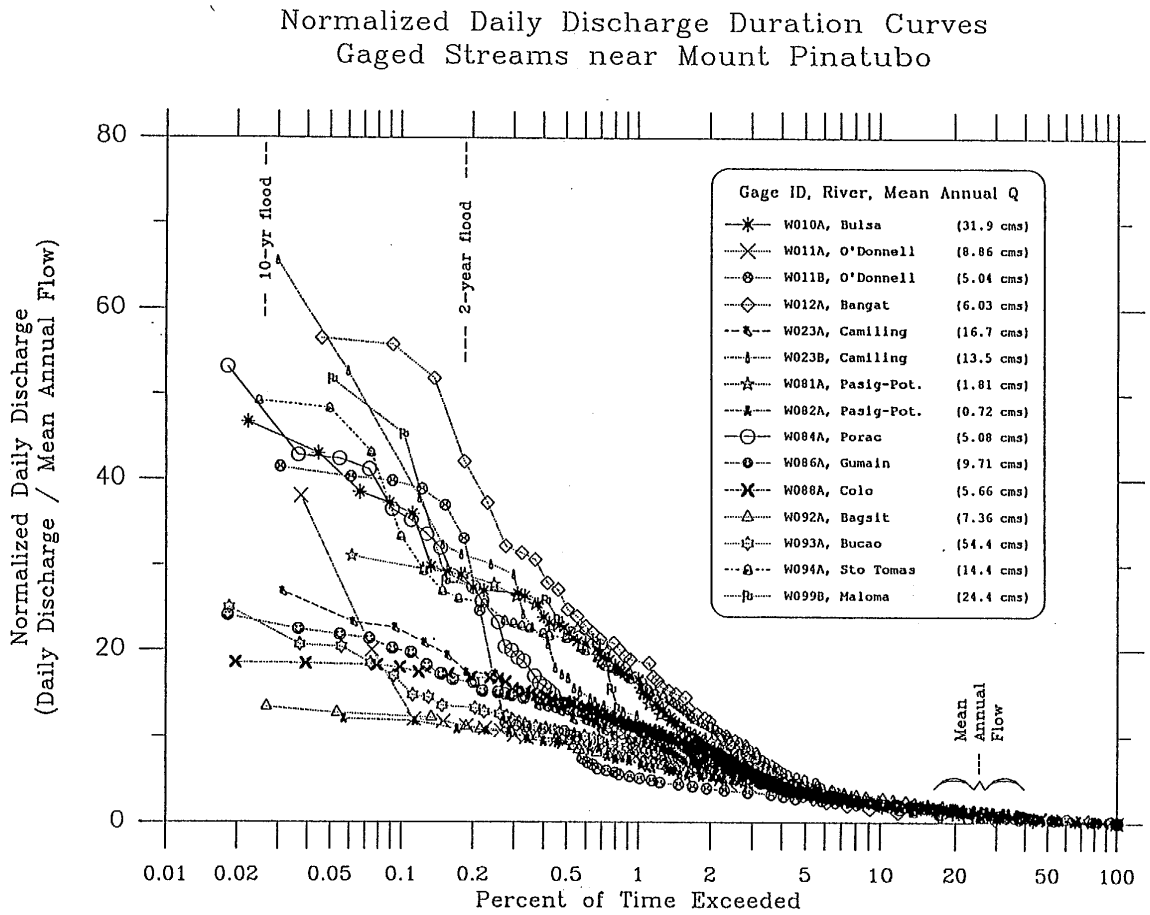

Normalized Daily Discharge Duration Curves
Gaged Streams near Mount Pinatubo
Normalized Daily Discharge
(Daily Discharge / Mean Annual Flow)
Percent of Time Exceeded
10-yr flood
2-year flood
Mean Annual Flow
Gage ID, River, Mean Annual Q
W010A, Bulsa (31.9 cms)
W011A, O'Donnell (8.86 cms)
W011B, O'Donnell (5.04 cms)
W012A, Bangat (6.03 cms)
W023A, Camiling (16.7 cms)
W023B, Camiling (13.5 cms)
W081A, Pasig-Pot. (1.81 cms)
W082A, Pasig-Pot. (0.72 cms)
W084A, Porac (5.08 cms)
W086A, Gumain (9.71 cms)
W088A, Colo (5.66 cms)
W092A, Bagsit (7.36 cms)
W093A, Bucao (54.4 cms)
W094A, Sto Tomas (14.4 cms)
W099B, Maloma (24.4 cms)

Figure 2.4.52

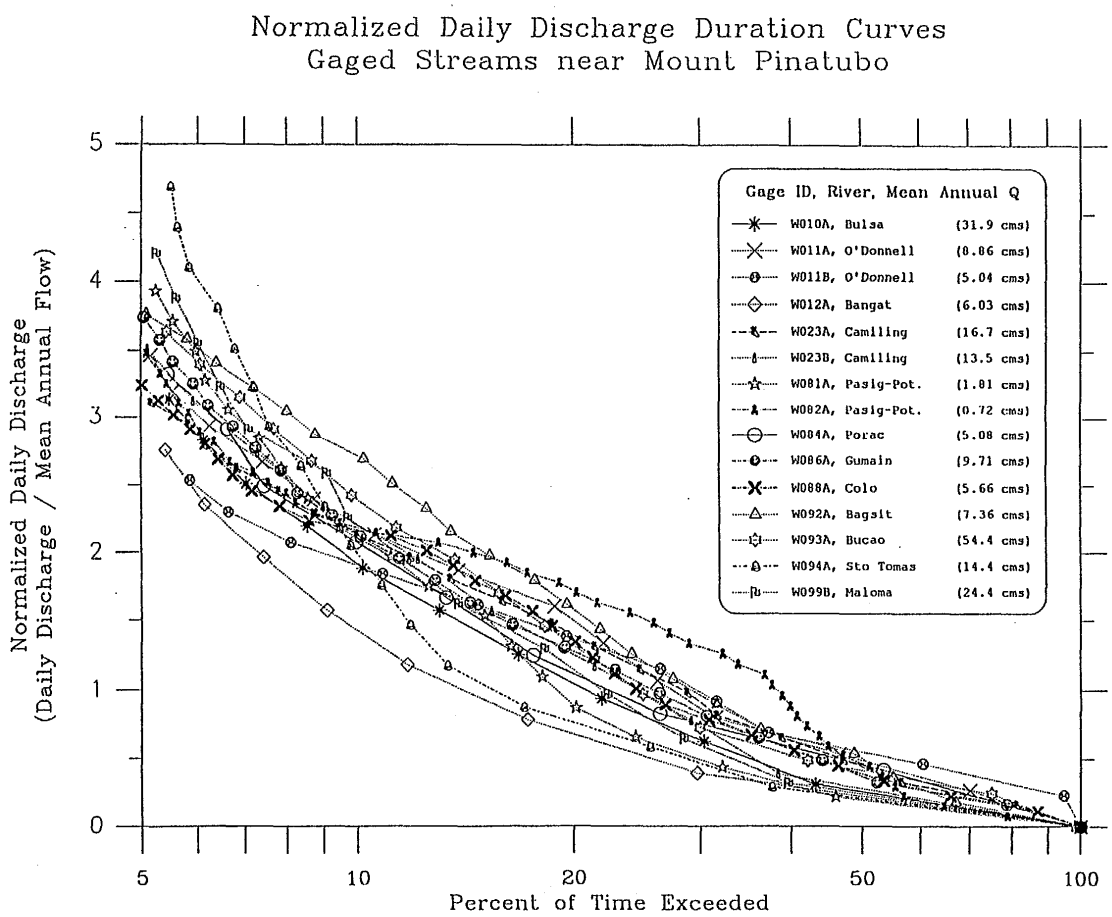


Figure 2.4.53

444

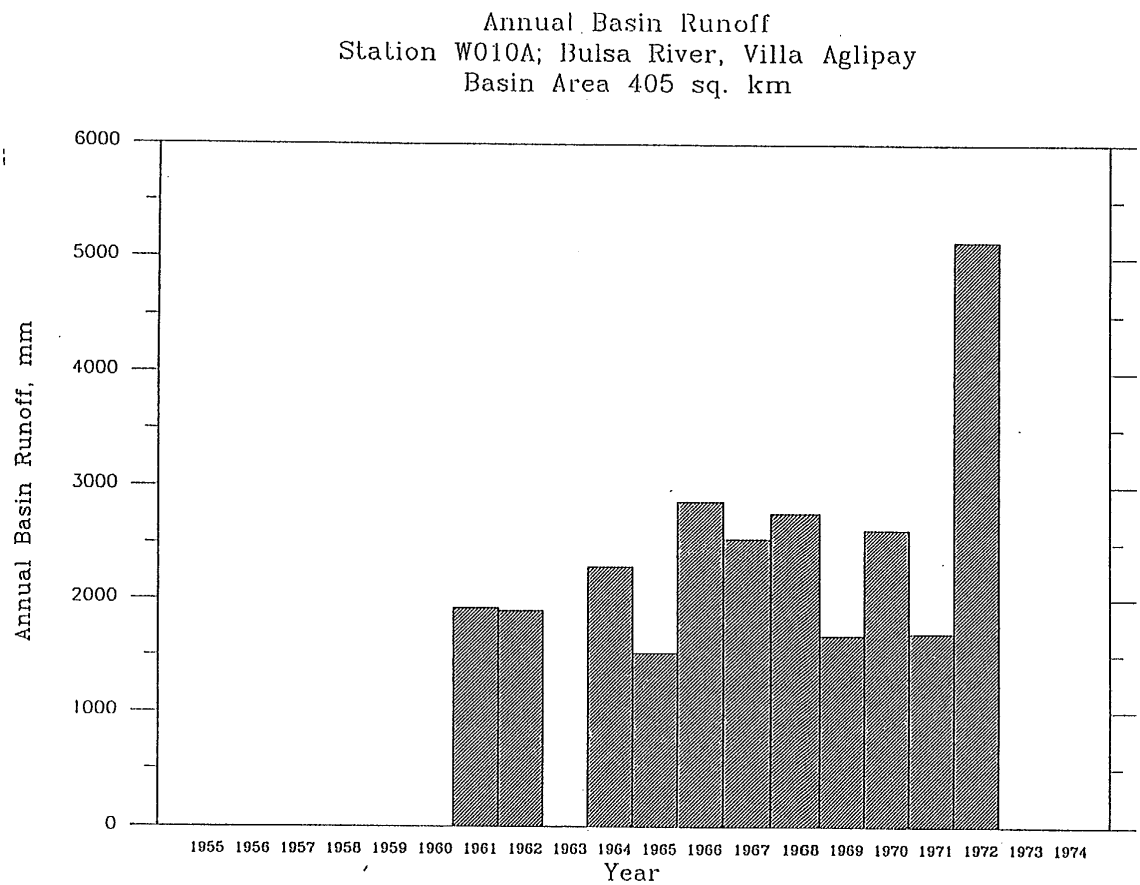

Annual Basin Runoff
Station W010A; Bulsa River, Villa Aglipay
Basin Area 405 sq. km
Annual Basin Runoff, mm
6000
5000
4000
3000
2000
1000
0
1955 1956 1957 1958 1959 1960 1961 1962 1963 1964 1965 1966 1967 1968 1969 1970 1971 1972 1973 1974
Year

Figure 2.4.54

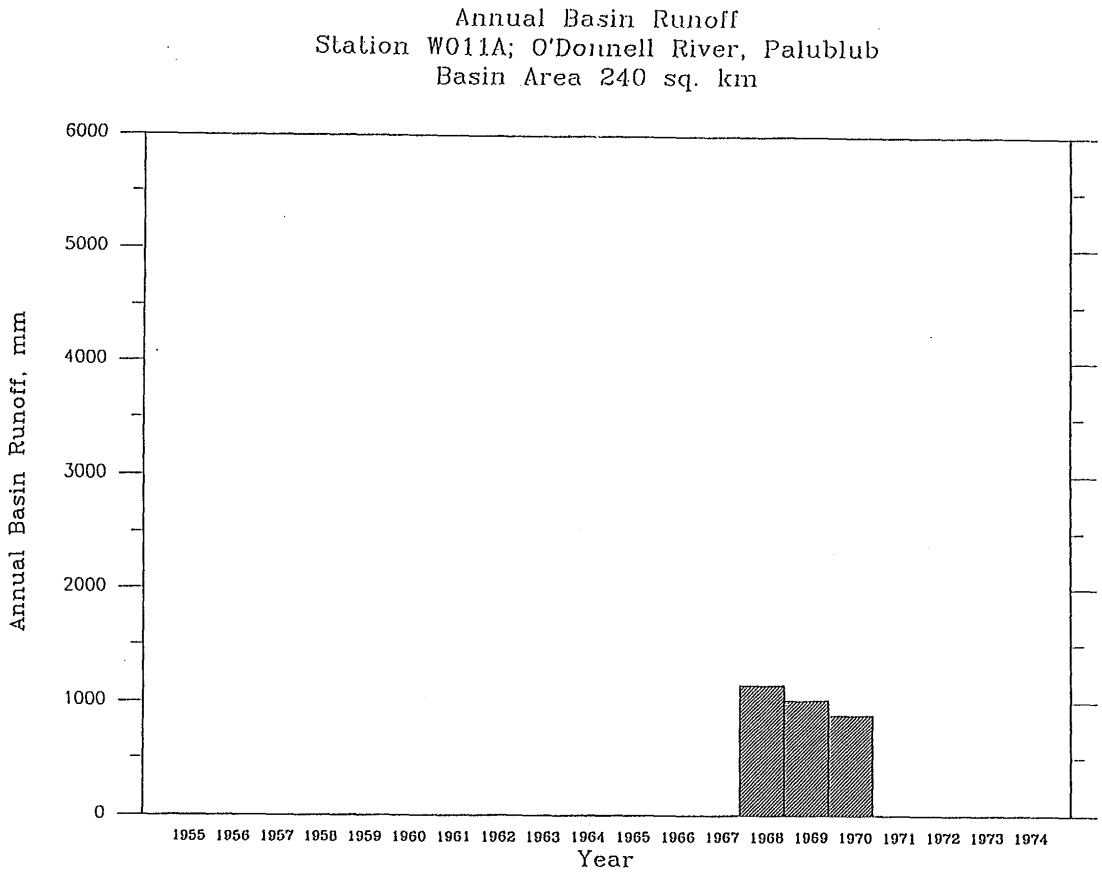

Annual Basin Runoff
Station W011A; O'Donnell River, Palublub
Basin Area 240 sq. km
Annual Basin Runoff, mm
6000
5000
4000
3000
2000
1000
0
1955 1956 1957 1958 1959 1960 1961 1962 1963 1964 1965 1966 1967 1968 1969 1970 1971 1972 1973 1974
Year

Figure 2.4.55

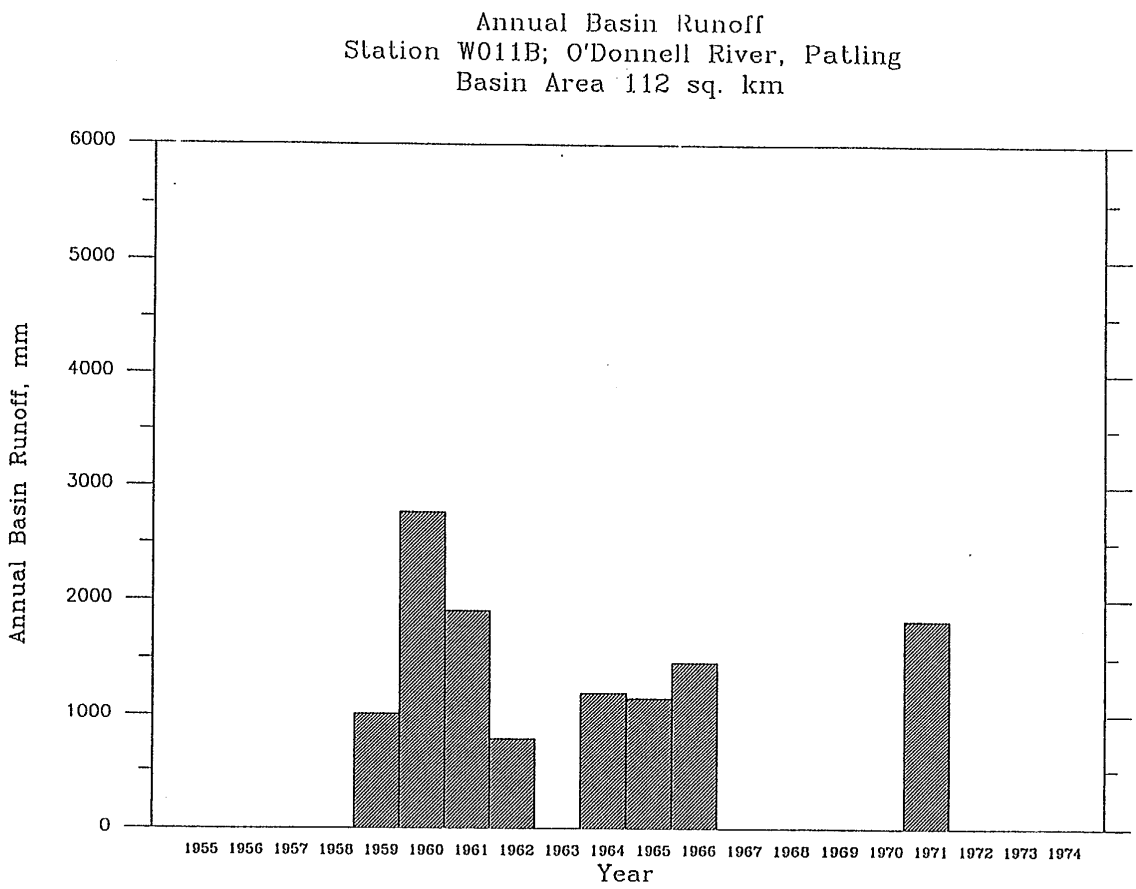

Annual Basin Runoff
Station W011B; O'Donnell River, Patling
Basin Area 112 sq. km
Annual Basin Runoff, mm
6000
5000
4000
3000
2000
1000
0
1955 1956 1957 1958 1959 1960 1961 1962 1963 1964 1965 1966 1967 1968 1969 1970 1971 1972 1973 1974
Year

Figure 2.4.56

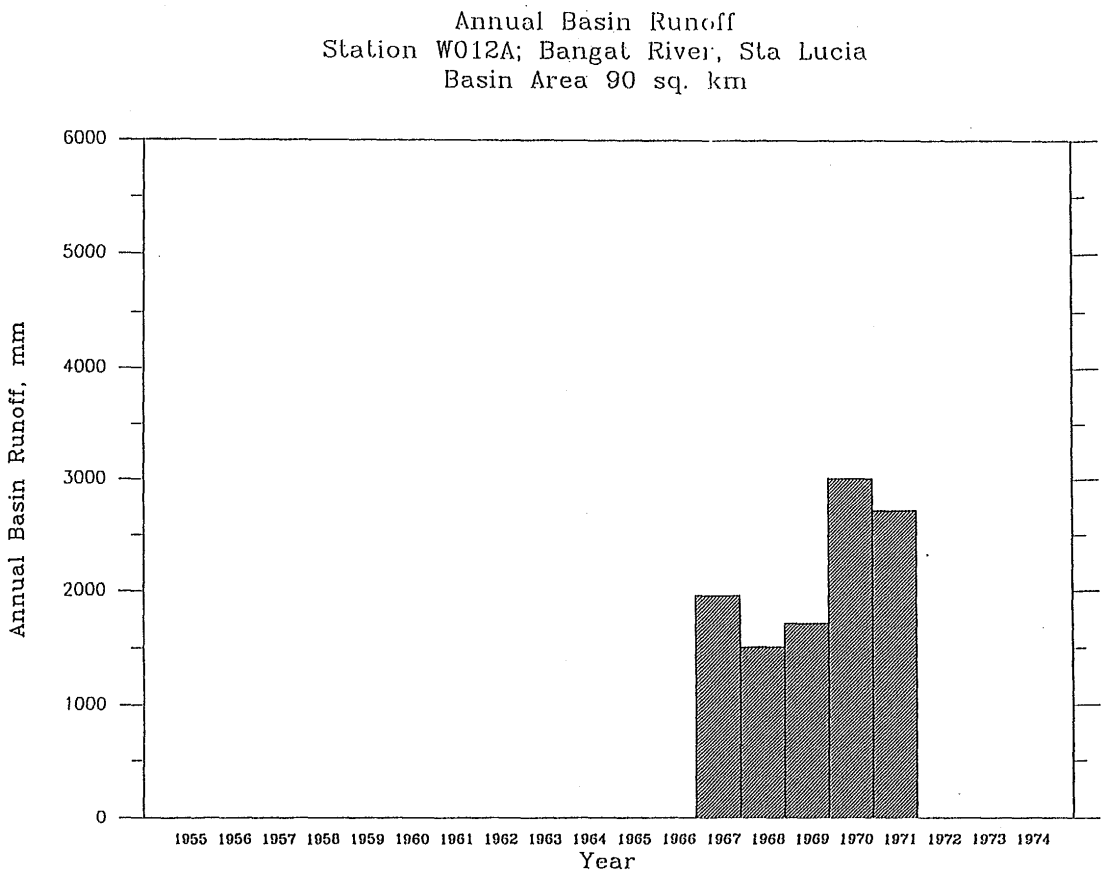

Annual Basin Runoff
Station W012A; Bangat River, Sta Lucia
Basin Area 90 sq. km
Annual Basin Runoff, mm
6000
5000
4000
3000
2000
1000
0
1955 1956 1957 1958 1959 1960 1961 1962 1963 1964 1965 1966 1967 1968 1969 1970 1971 1972 1973 1974
Year

Figure 2.4.57

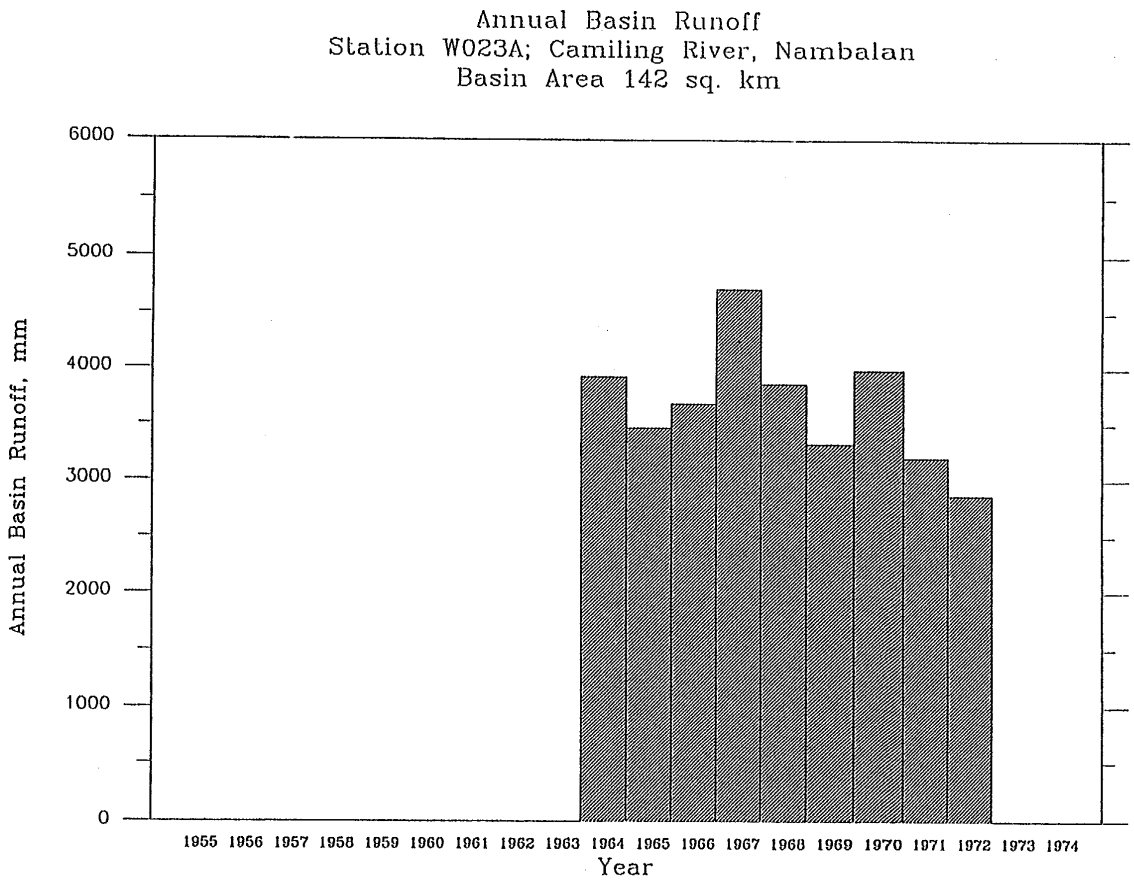

Annual Basin Runoff
Station W023A; Camiling River, Nambalan
Basin Area 142 sq. km
Annual Basin Runoff, mm
6000
5000
4000
3000
2000
1000
0
1955 1956 1957 1958 1959 1960 1961 1962 1963 1964 1965 1966 1967 1968 1969 1970 1971 1972 1973 1974
Year

Figure 2.4.58

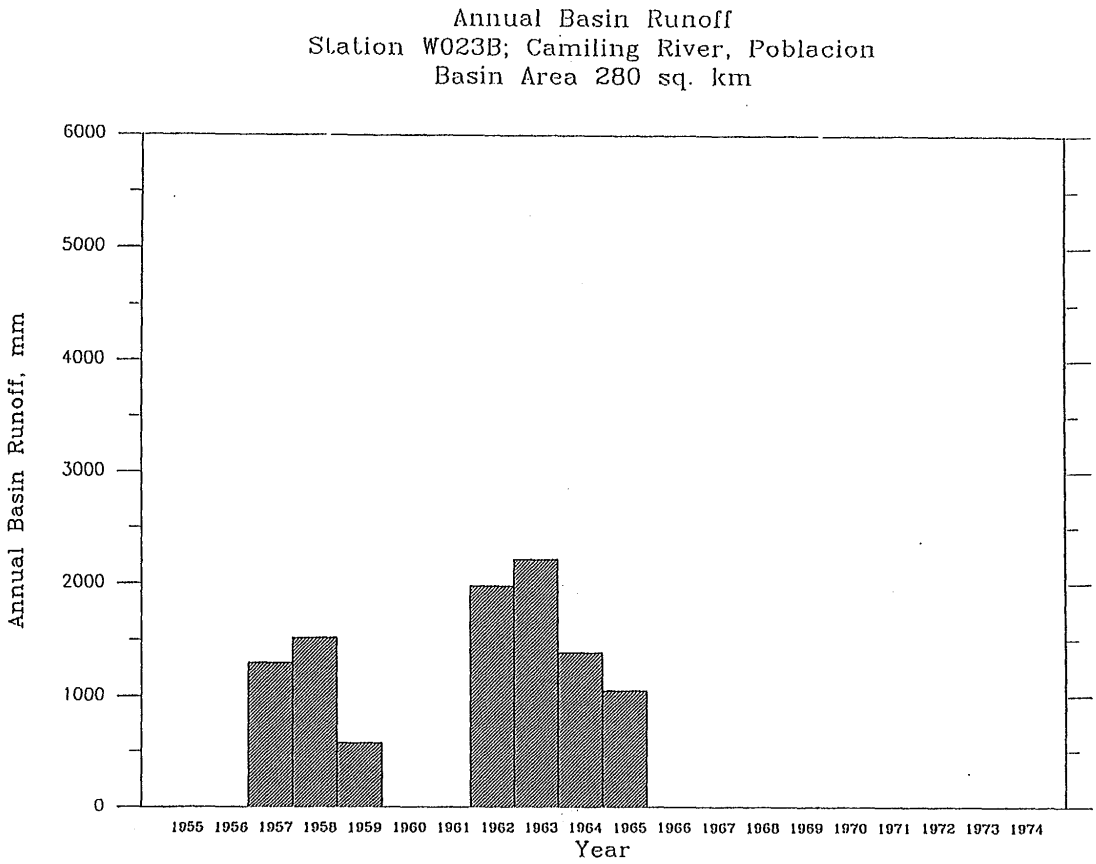

Annual Basin Runoff
Station W023B; Camiling River, Poblacion
Basin Area 280 sq. km
Annual Basin Runoff, mm
6000
5000
4000
3000
2000
1000
0
1955 1956 1957 1958 1959 1960 1961 1962 1963 1964 1965 1966 1967 1968 1969 1970 1971 1972 1973 1974
Year

Figure 2.4.59

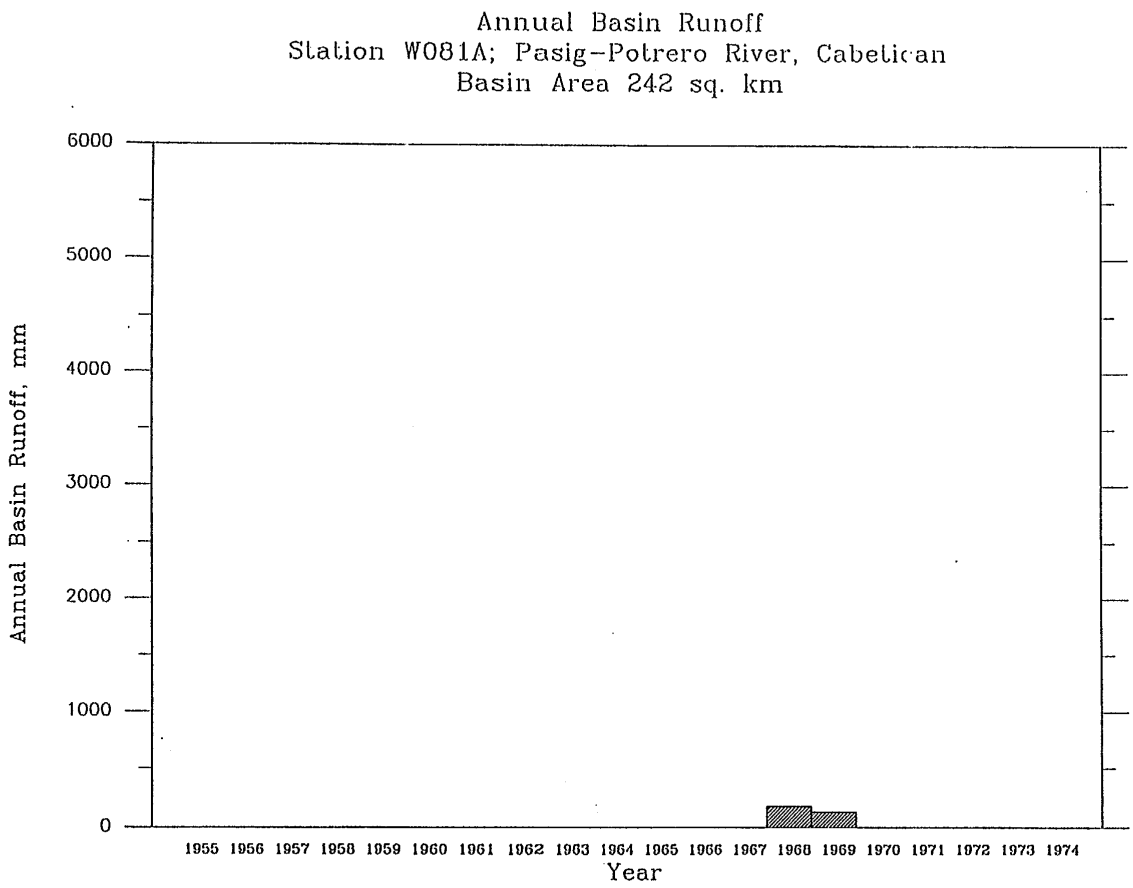

Annual Basin Runoff
Station W081A; Pasig-Potrero River, Cabetican
Basin Area 242 sq. km
Annual Basin Runoff, mm
6000
5000
4000
3000
2000
1000
0
1955 1956 1957 1958 1959 1960 1961 1962 1963 1964 1965 1966 1967 1968 1969 1970 1971 1972 1973 1974
Year

Figure 2.4.60

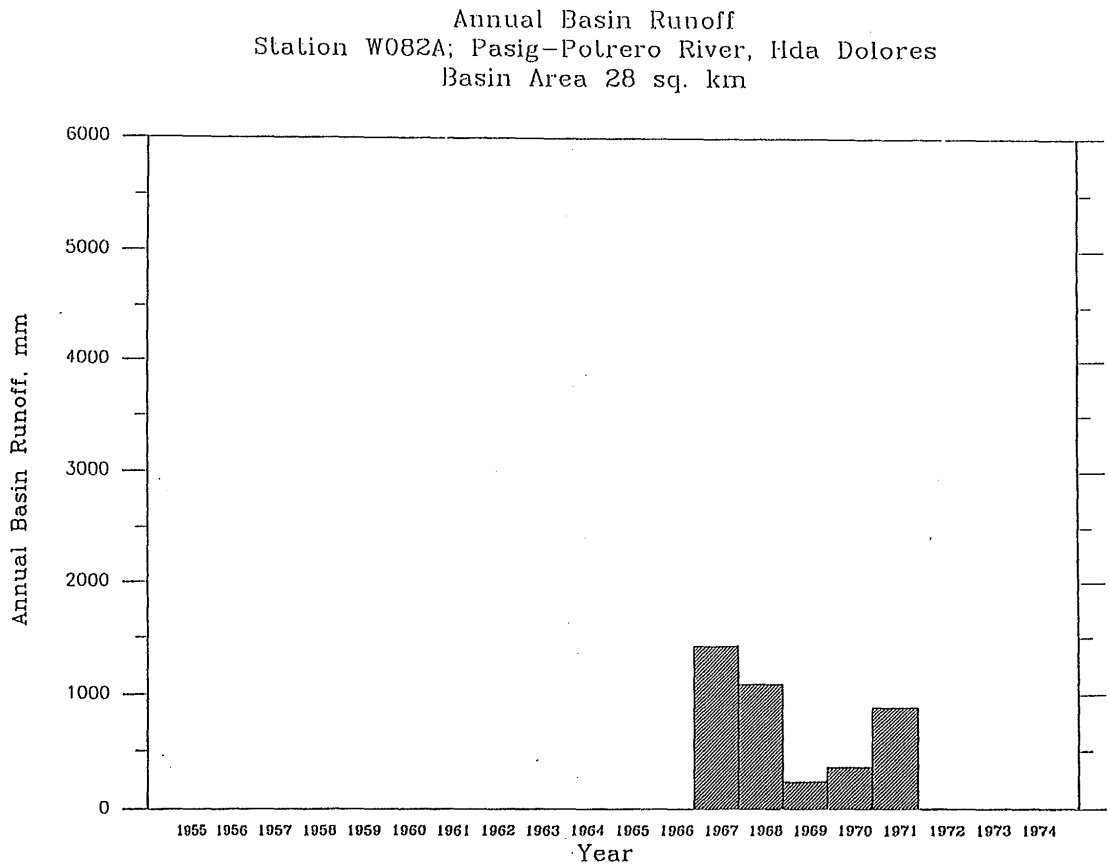

Annual Basin Runoff
Station W082A; Pasig-Potrero River, Hda Dolores
Basin Area 28 sq. km
Annual Basin Runoff, mm
6000
5000
4000
3000
2000
1000
0
1955 1956 1957 1958 1959 1960 1961 1962 1963 1964 1965 1966 1967 1968 1969 1970 1971 1972 1973 1974
Year

Figure 2.4.61

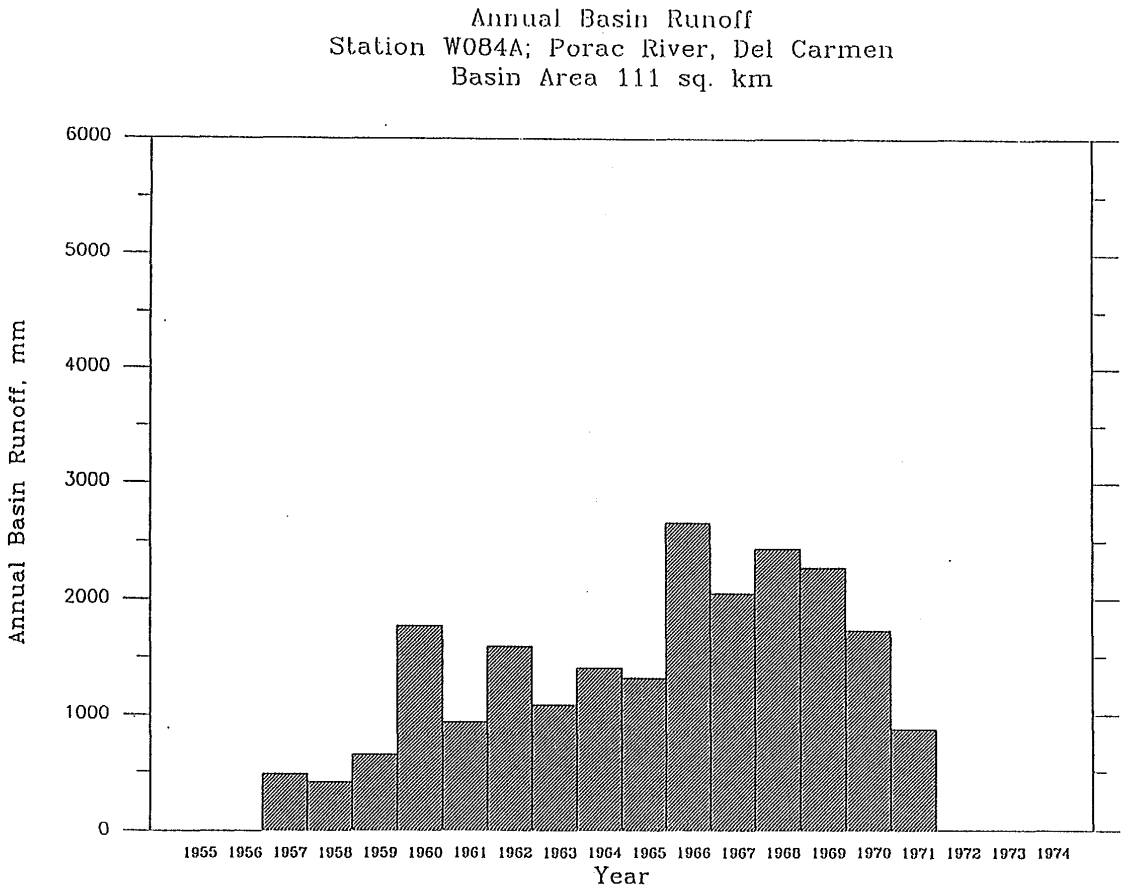

Annual Basin Runoff
Station W084A; Porac River, Del Carmen
Basin Area 111 sq. km
Annual Basin Runoff, mm
6000
5000
4000
3000
2000
1000
0
1955 1956 1957 1958 1959 1960 1961 1962 1963 1964 1965 1966 1967 1968 1969 1970 1971 1972 1973 1974
Year

Figure 2.4.62

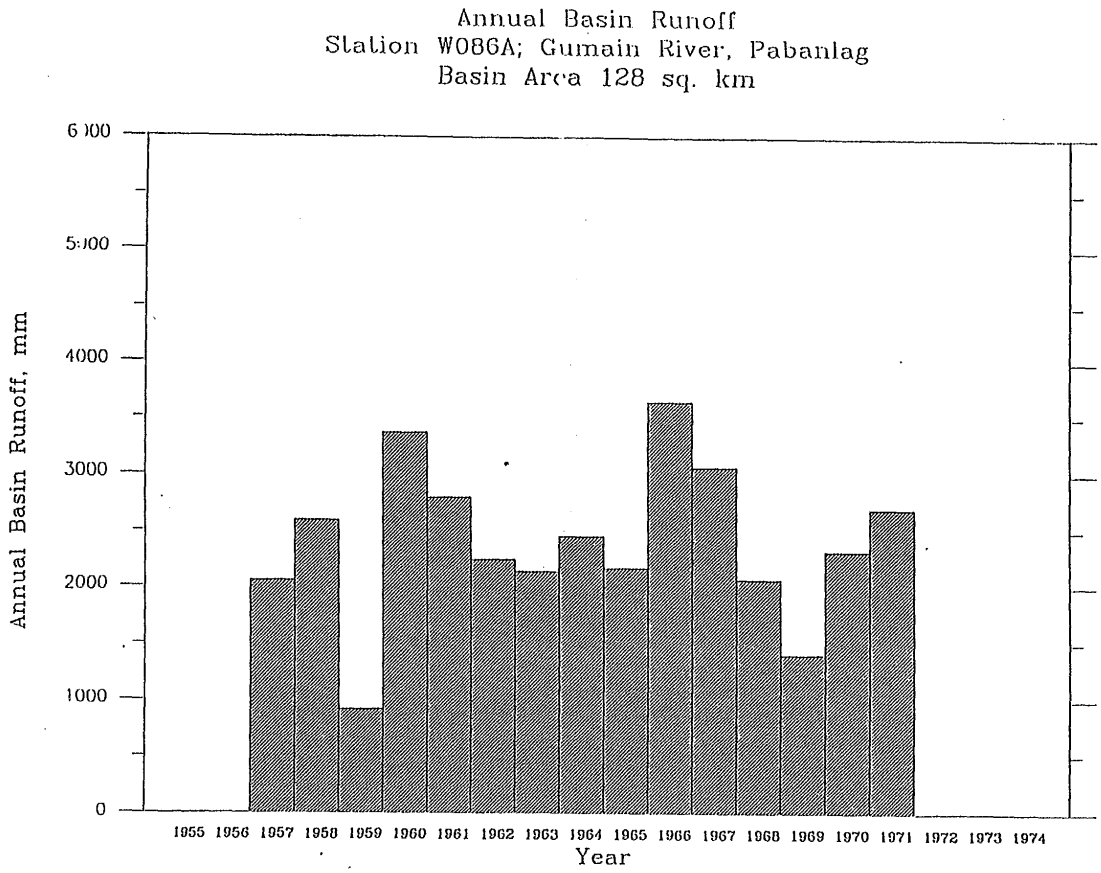

Annual Basin Runoff
Station W086A; Gumain River, Pabanlag
Basin Area 128 sq. km
Annual Basin Runoff, mm
6000
5000
4000
3000
2000
1000
0
1955 1956 1957 1958 1959 1960 1961 1962 1963 1964 1965 1966 1967 1968 1969 1970 1971 1972 1973 1974
Year

Figure 2.4.63

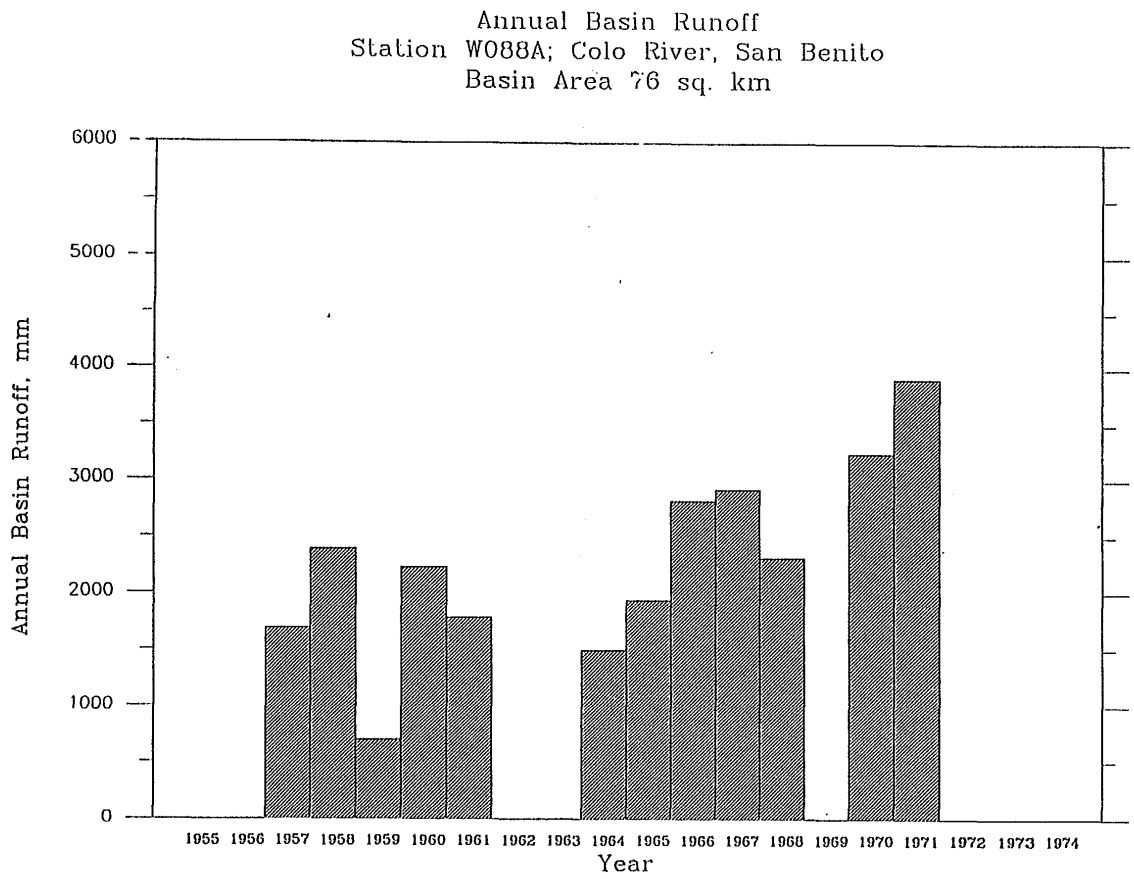

Annual Basin Runoff
Station W088A; Colo River, San Benito
Basin Area 76 sq. km
Annual Basin Runoff, mm
6000
5000
4000
3000
2000
1000
0
1955 1956 1957 1958 1959 1960 1961 1962 1963 1964 1965 1966 1967 1968 1969 1970 1971 1972 1973 1974
Year

Figure 2.4.64

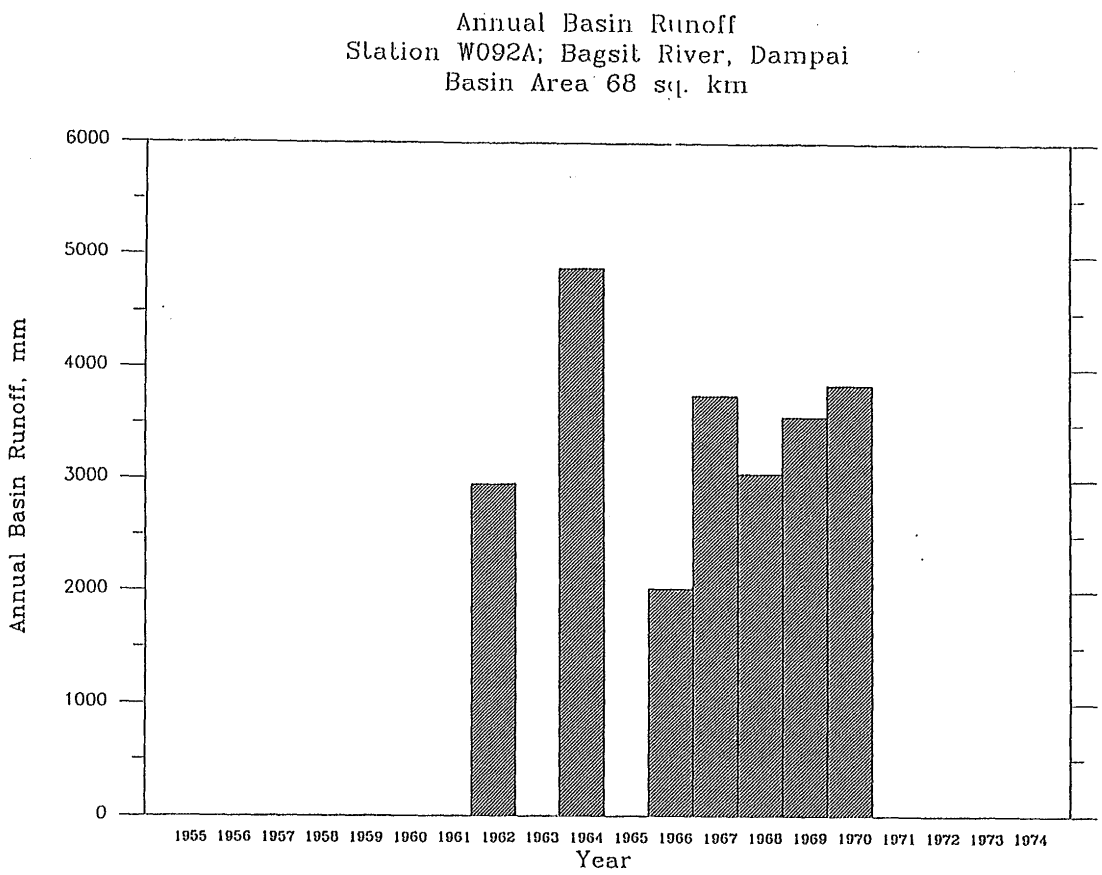

Annual Basin Runoff
Station W092A; Bagsit River, Dampai
Basin Area 68 sq. km
Annual Basin Runoff, mm
6000
5000
4000
3000
2000
1000
0
1955 1956 1957 1958 1959 1960 1961 1962 1963 1964 1965 1966 1967 1968 1969 1970 1971 1972 1973 1974
Year

Figure 2.4.65

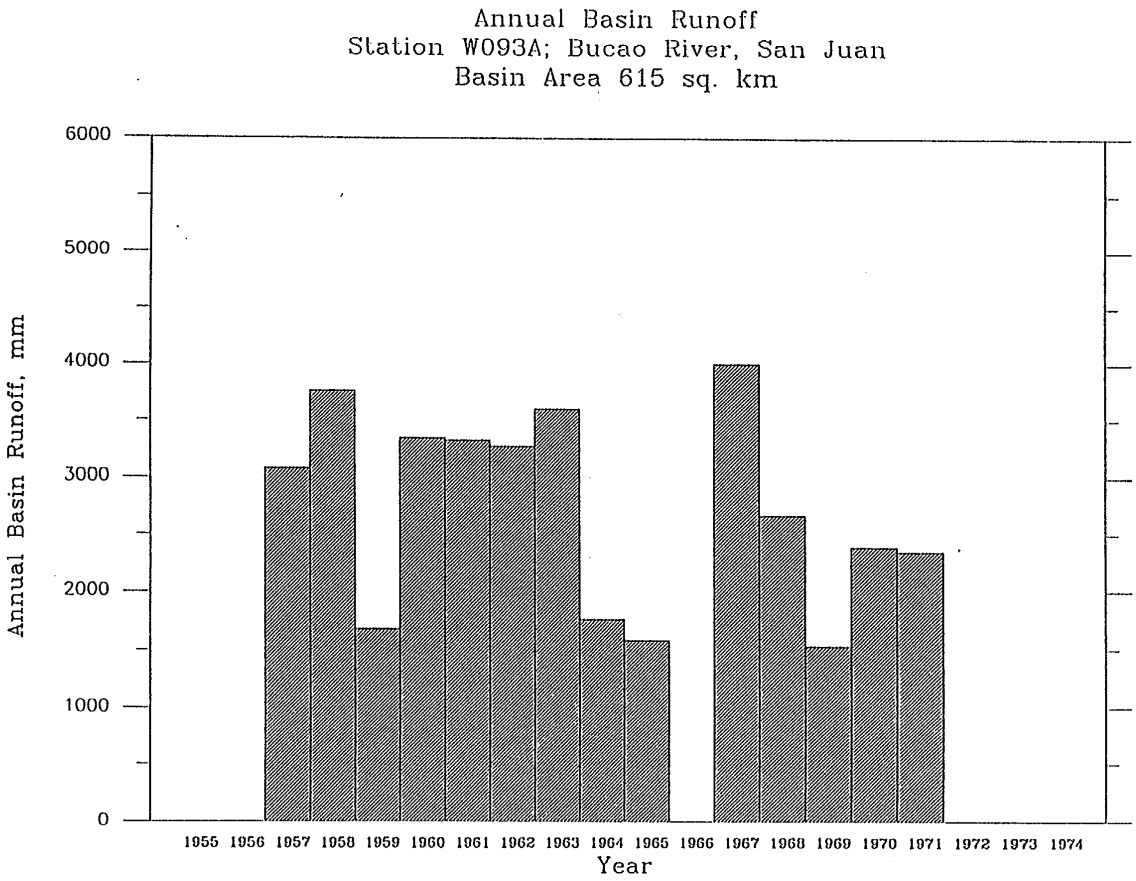

Annual Basin Runoff
Station W093A; Bucao River, San Juan
Basin Area 615 sq. km
Annual Basin Runoff, mm
6000
5000
4000
3000
2000
1000
0
1955 1956 1957 1958 1959 1960 1961 1962 1963 1964 1965 1966 1967 1968 1969 1970 1971 1972 1973 1974
Year

Figure 2.4.66

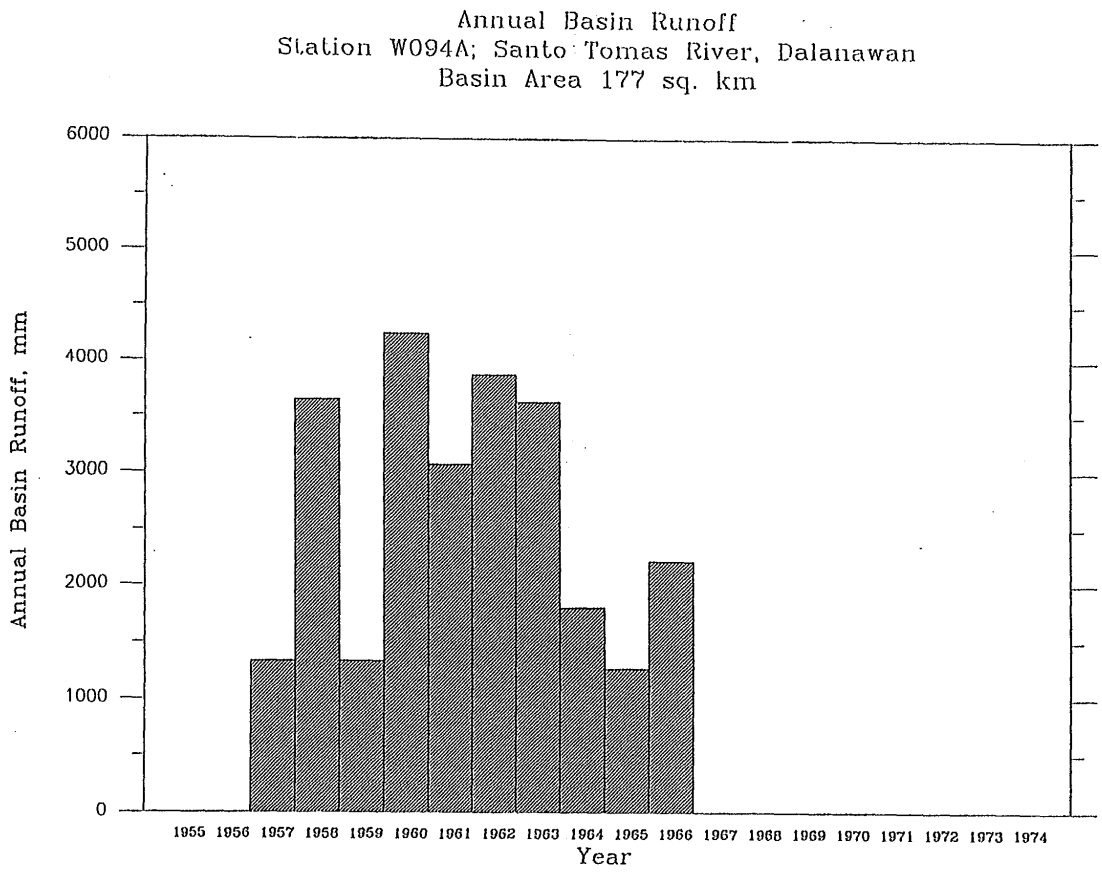

Annual Basin Runoff
Station W094A; Santo Tomas River, Dalanawan
Basin Area 177 sq. km
Annual Basin Runoff, mm
6000
5000
4000
3000
2000
1000
0
1955 1956 1957 1958 1959 1960 1961 1962 1963 1964 1965 1966 1967 1968 1969 1970 1971 1972 1973 1974
Year

Figure 2.4.67

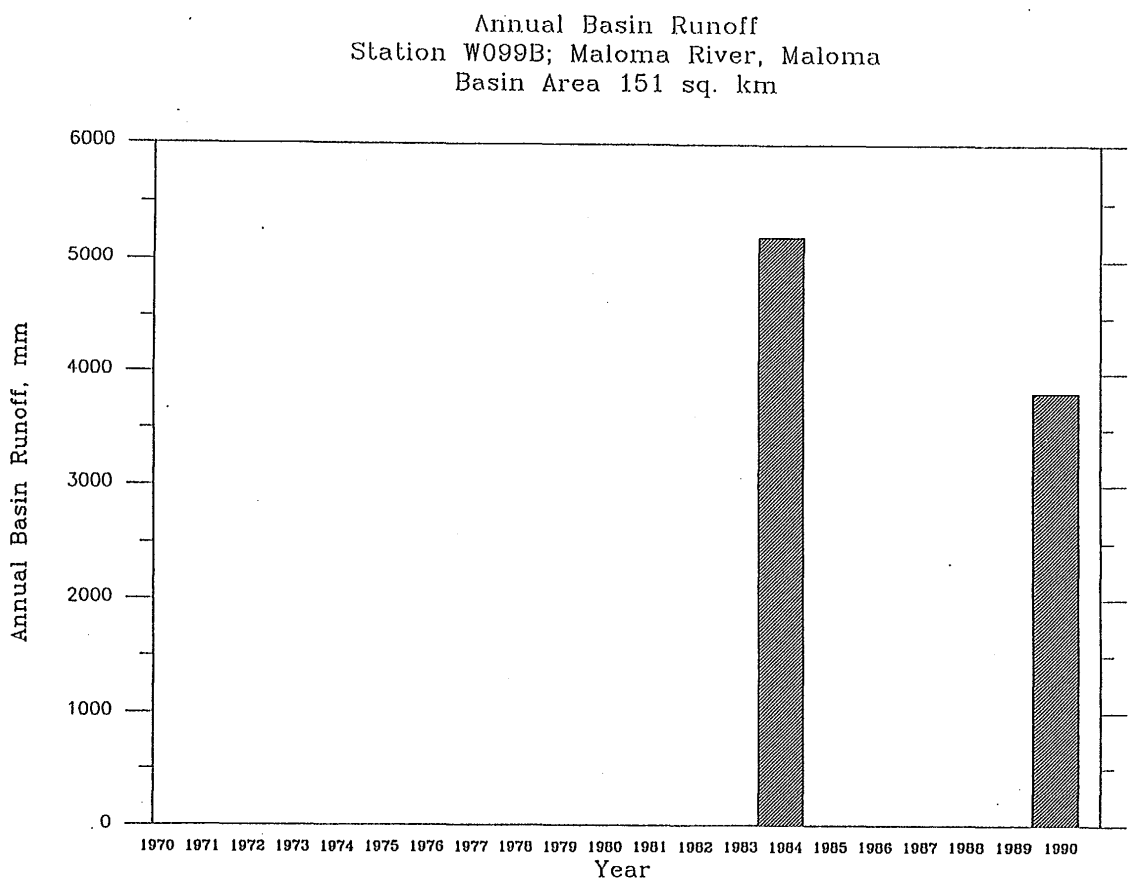
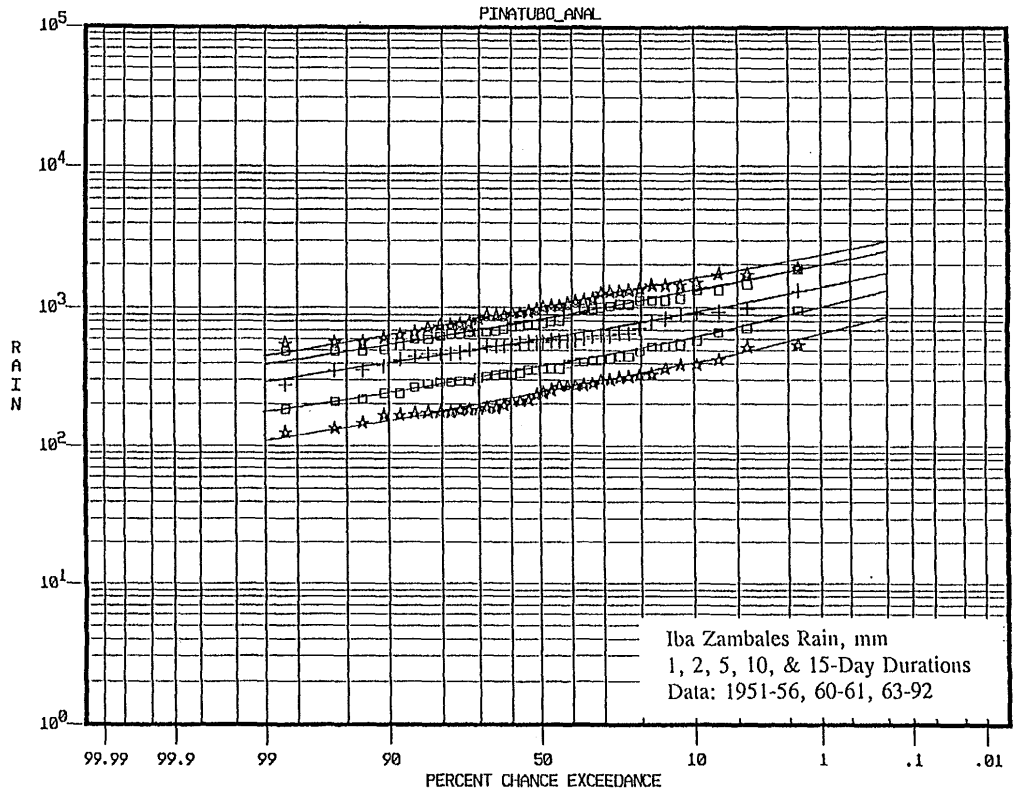
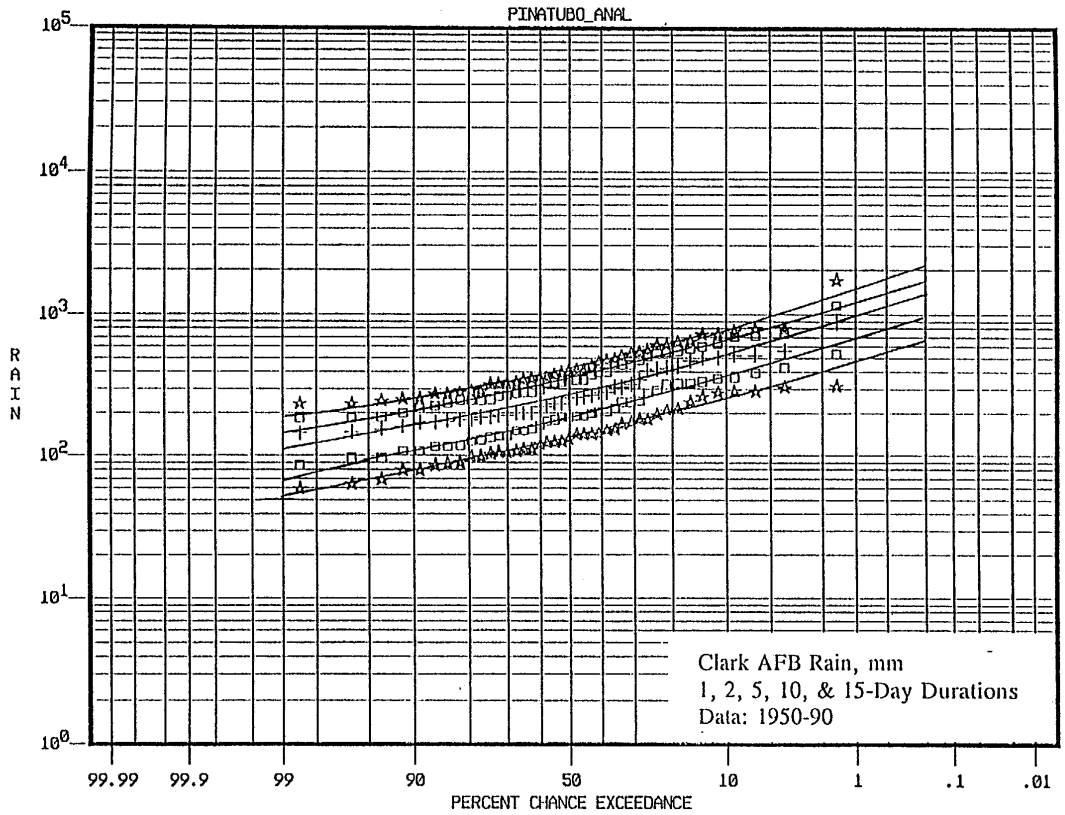

Annual Basin Runoff
Station W099B; Maloma River, Maloma
Basin Area 151 sq. km
Annual Basin Runoff, mm
6000
5000
4000
3000
2000
1000
0
1970 1971 1972 1973 1974 1975 1976 1977 1978 1979 1980 1981 1982 1983 1984 1985 1986 1987 1988 1989 1990
Year

Figure 2.4.68



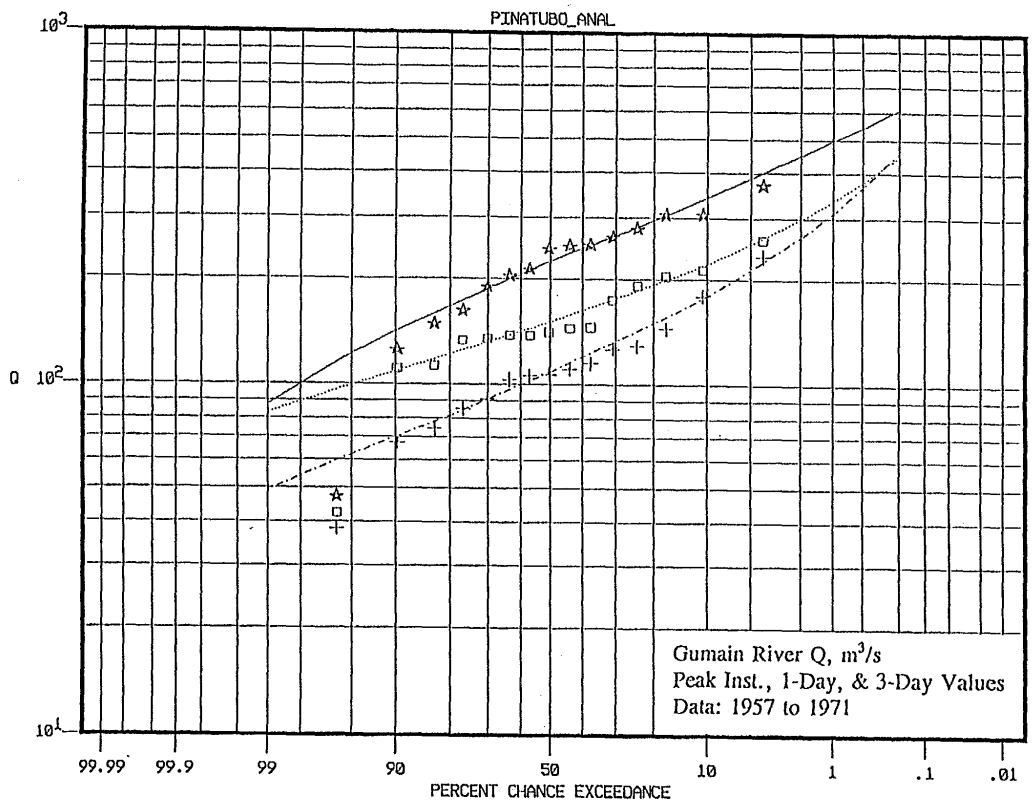
☆ RD324 MAX EVENTS 01-DAY DUR
□ RD324 MAX EVENTS 02-DAY DUR
+ RD324 MAX EVENTS 05-DAY DUR
□ RD324 MAX EVENTS 10-DAY DUR
☆ RD324 MAX EVENTS 15-DAY DUR

Figure 2.4.69



☆ RUSA2 MAX EVENTS 01-DAY DUR
□ RUSA2 MAX EVENTS 02-DAY DUR
-|- RUSA2 MAX EVENTS 05-DAY DUR
□ RUSA2 MAX EVENTS 10-DAY DUR
☆ RUSA2 MAX EVENTS 15-DAY DUR

Figure 2.4.70



☆ W086A MAX EVENTS PEAK INST Q EXCL POST-71

W086A MAX ANALYTICAL EXP PROB PEAK INST Q EXCL POST-71

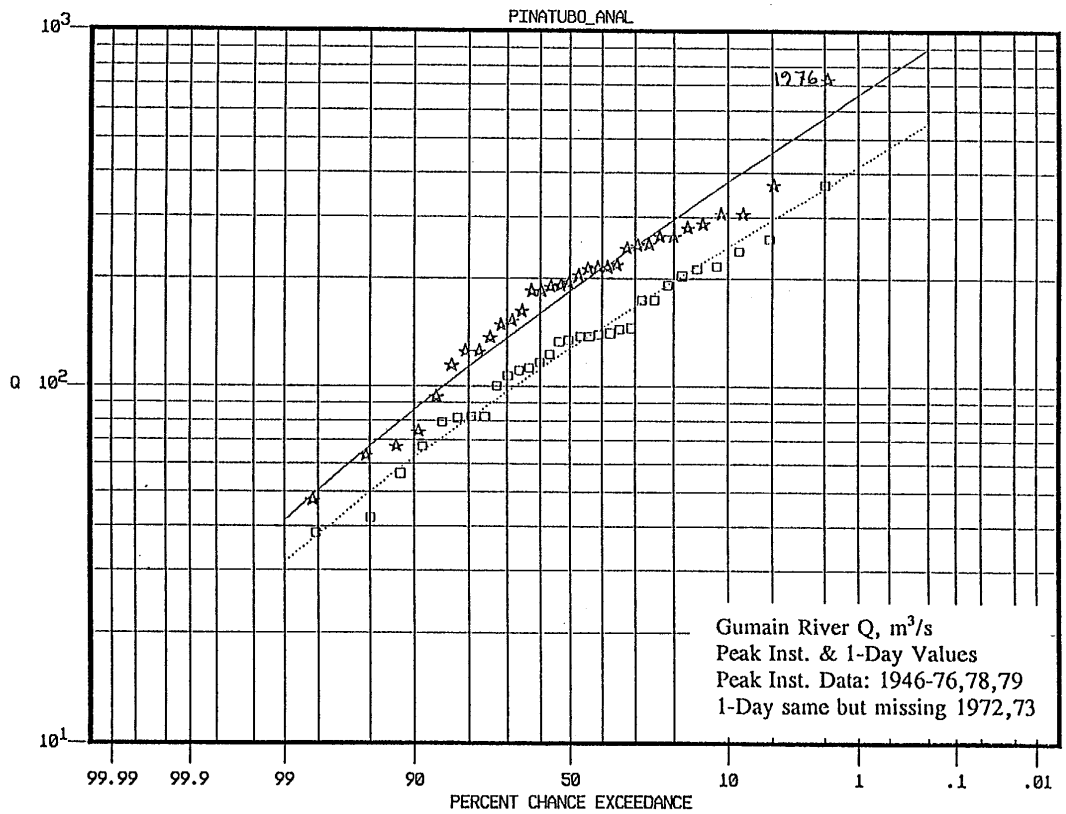
□ W086A MAX EVENTS PEAK 1-DAY EXCL POST-71

W086A MAX ANALYTICAL EXP PROB PEAK 1-DAY EXCL POST-71

+ W086A MAX EVENTS PEAK 3-DAY EXCL POST-71

W086A MAX ANALYTICAL EXP PROB PEAK 3-DAY EXCL POST-71

Figure 2.4.71



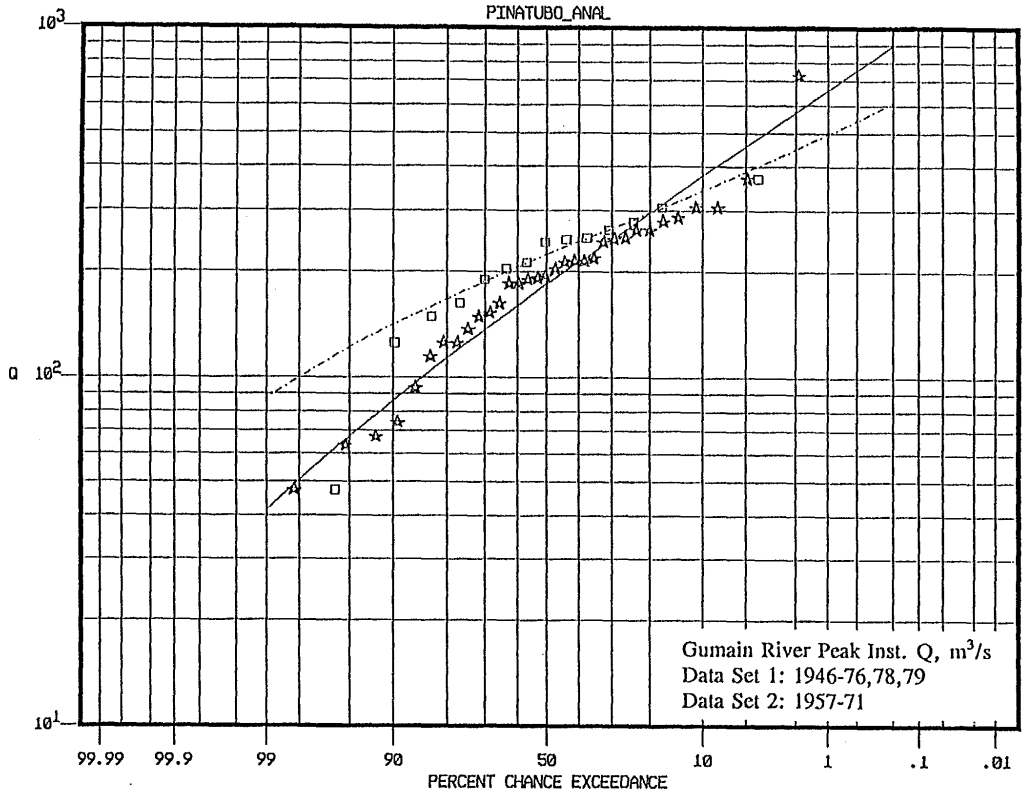
☆ W086A MAX EVENTS PEAK INST W. POST-71

——— W086A MAX ANALYTICAL EXP PROB PEAK INST W. POST-71

□ W086A MAX EVENTS PEAK 1-DAY W. POST-71

····· W086A MAX ANALYTICAL EXP PROB PEAK 1-DAY W. POST-71

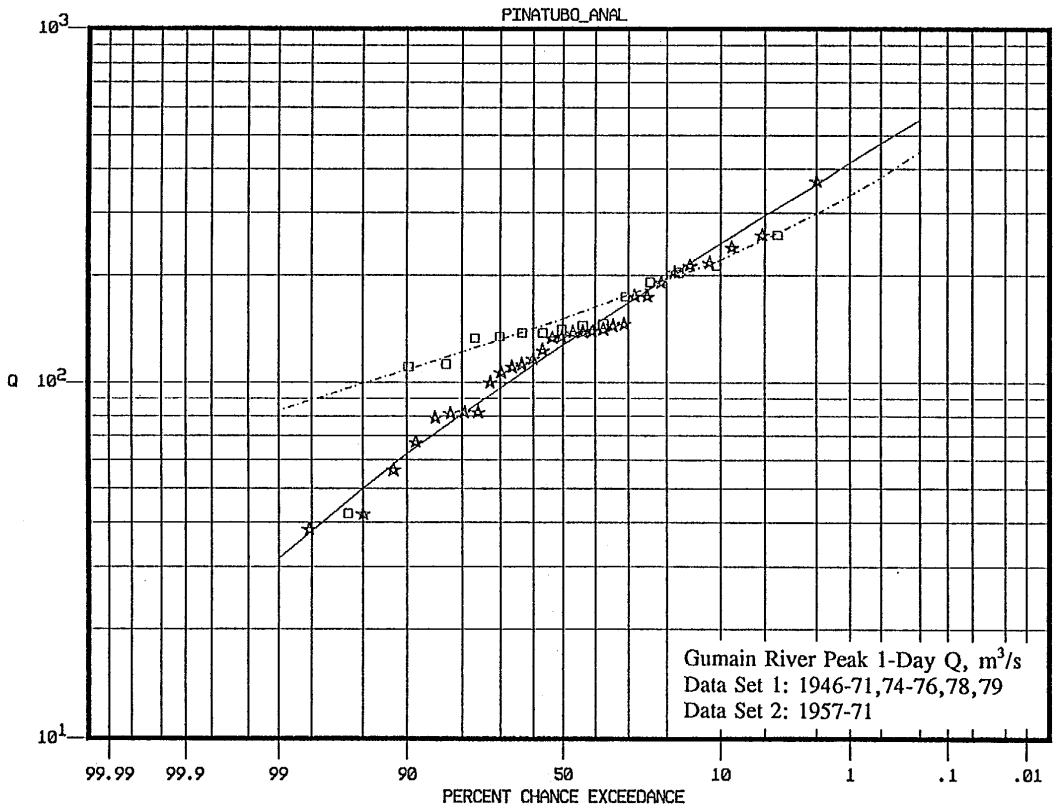
Figure 2.4.72



☆ W086A MAX EVENTS PEAK INST W. POST-71
——— W086A MAX ANALYTICAL EXP PROB PEAK INST W. POST-71
□ W086A MAX EVENTS PEAK INST Q EXCL POST-71
-·-·-·-·- W086A MAX ANALYTICAL EXP PROB PEAK INST Q EXCL POST-71

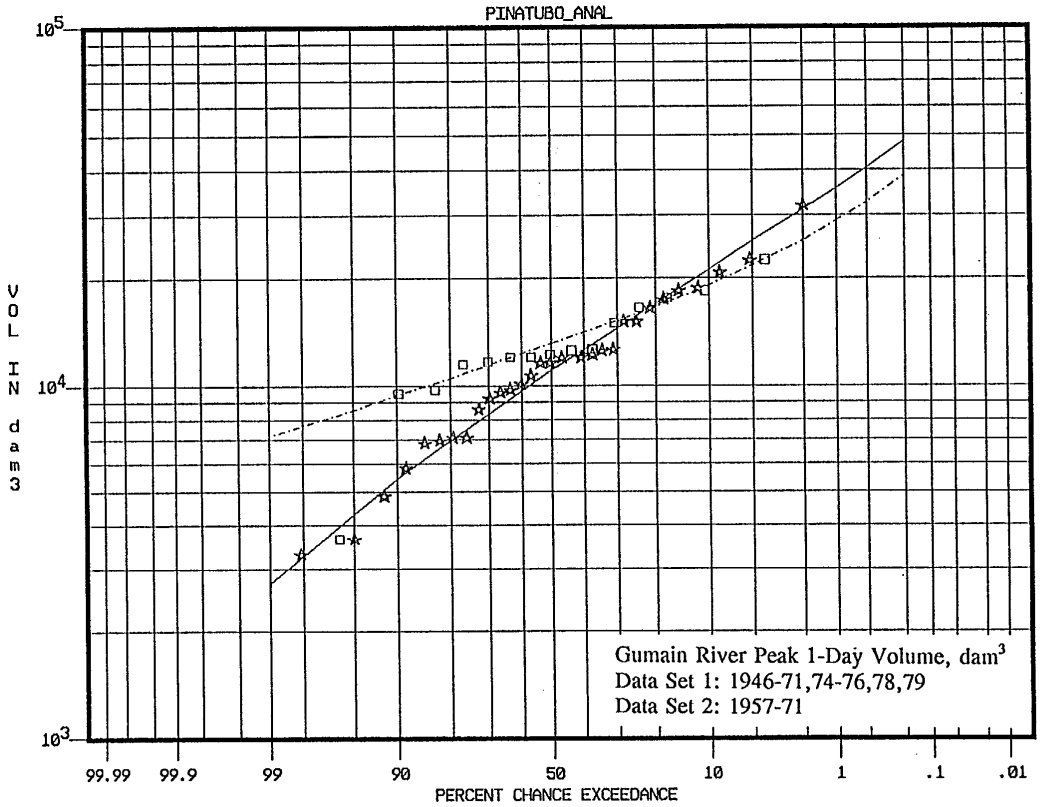
Figure 2.4.73

469



☆ W086A MAX EVENTS PEAK 1-DAY W. POST-71
——— W086A MAX ANALYTICAL EXP PROB PEAK 1-DAY W. POST-71
□ W086A MAX EVENTS PEAK 1-DAY EXCL POST-71
-·-·-·- W086A MAX ANALYTICAL EXP PROB PEAK 1-DAY EXCL POST-71

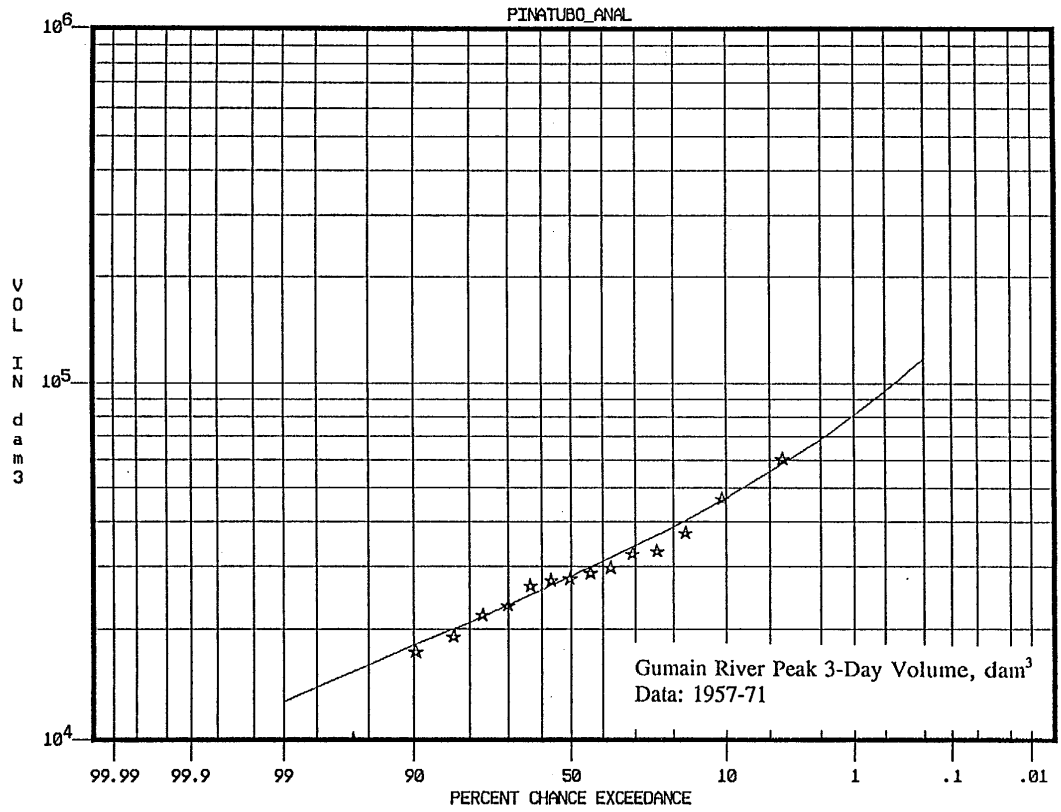
Figure 2.4.74



☆ W086A MAX EVENTS 01-DAY DUR W. POST-71
——— W086A MAX ANALYTICAL EXP PROB 01-DAY DUR W. POST-71
□ W086A MAX EVENTS 01-DAY DUR
-·-·-·- W086A MAX ANALYTICAL EXP PROB 01-DAY DUR

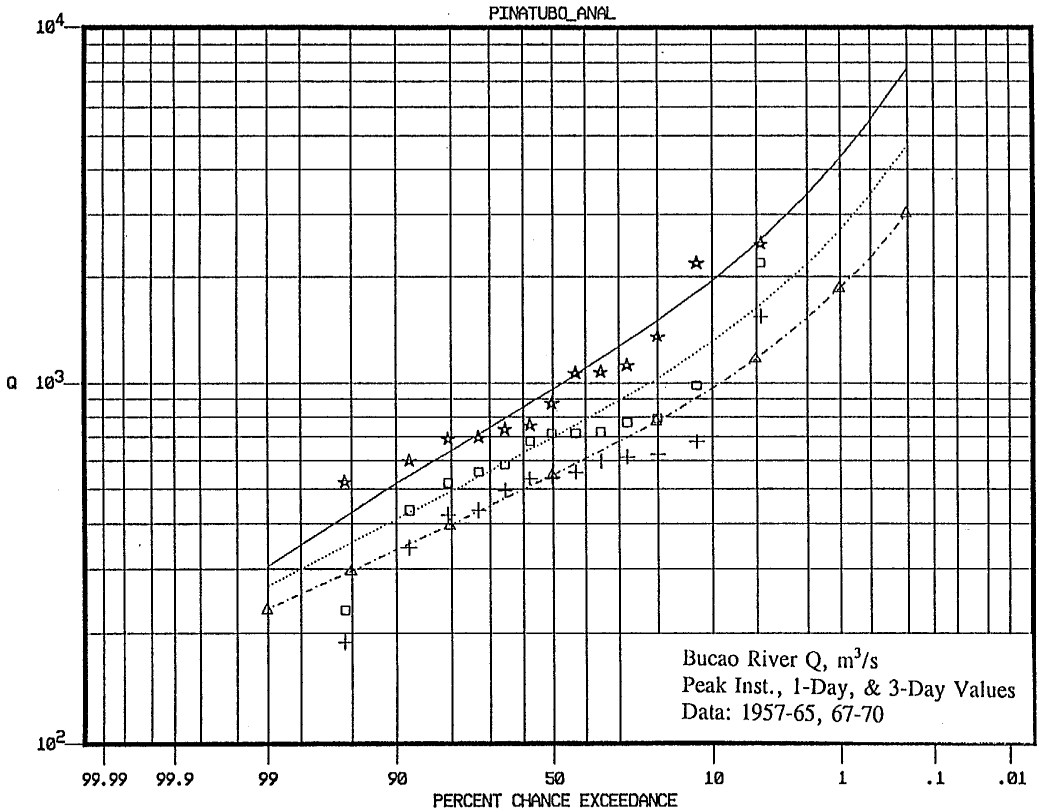
Figure 2.4.75

471



☆ W086A MAX EVENTS 03-DAY DUR
——— W086A MAX ANALYTICAL EXP PROB 03-DAY DUR

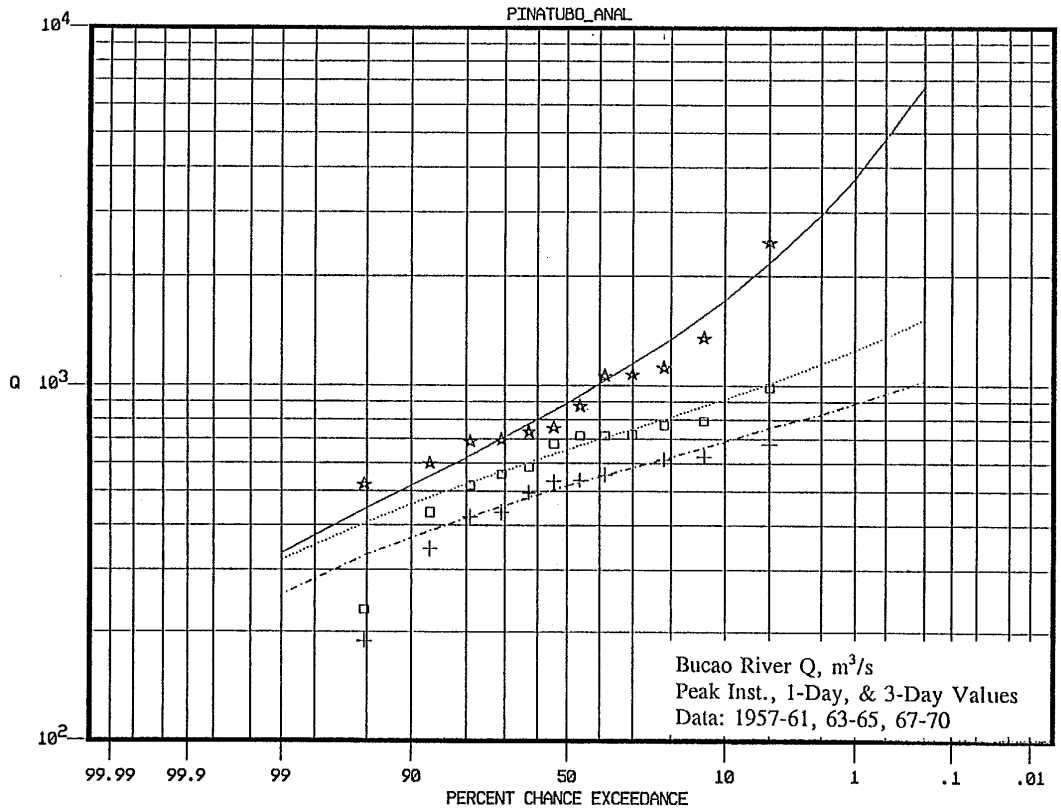
Figure 2.4.76



413

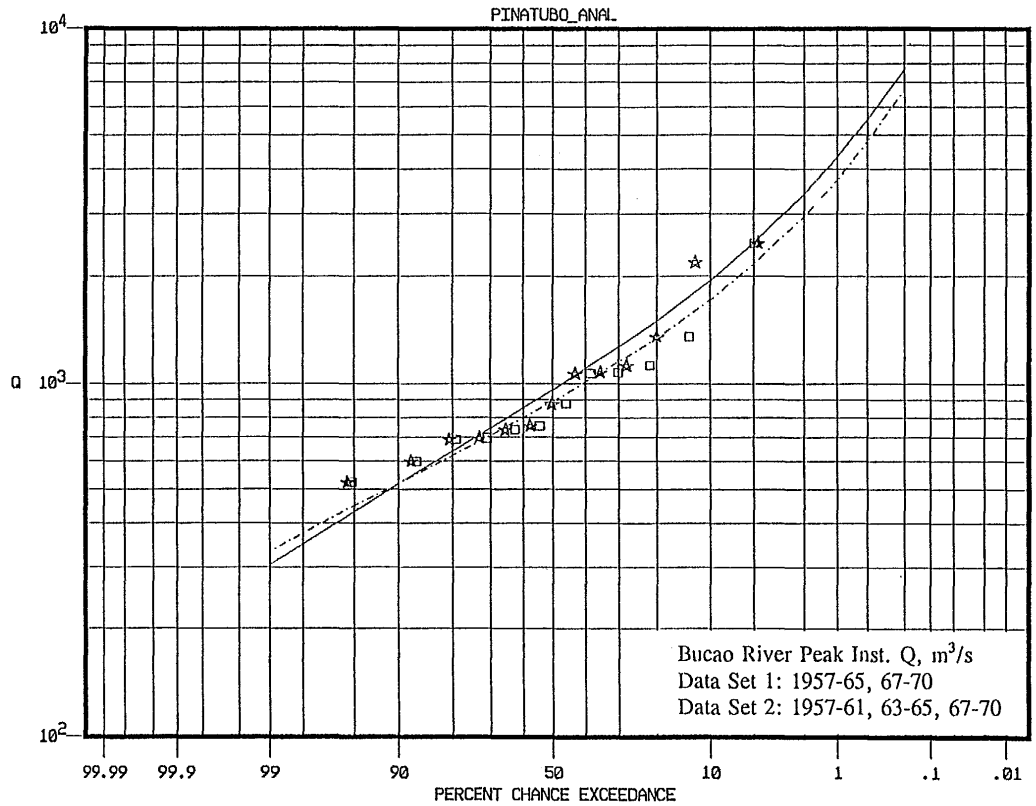
☆	W093A MAX EVENTS PEAK INST Q
———	W093A MAX ANALYTICAL EXP PROB PEAK INST Q
□	W093A MAX EVENTS PEAK 1-DAY Q
.....	W093A MAX ANALYTICAL EXP PROB PEAK 1-DAY Q
+	W093A MAX EVENTS PEAK 3-DAY Q
-·-·-·-△-·-·-·-	W093A MAX ANALYTICAL EXP PROB PEAK 3-DAY Q

Figure 2.4.77



☆ W093A MAX EVENTS PEAK INST Q EXCL 1962
——— W093A MAX ANALYTICAL EXP PROB PEAK INST Q EXCL 1962
□ W093A MAX EVENTS PEAK 1-DAY Q EXCL 1962
····· W093A MAX ANALYTICAL EXP PROB PEAK 1-DAY Q EXCL 1962
+ W093A MAX EVENTS PEAK 3-DAY Q EXCL 1962
-·-·-·- W093A MAX ANALYTICAL EXP PROB PEAK 3-DAY Q EXCL 1962

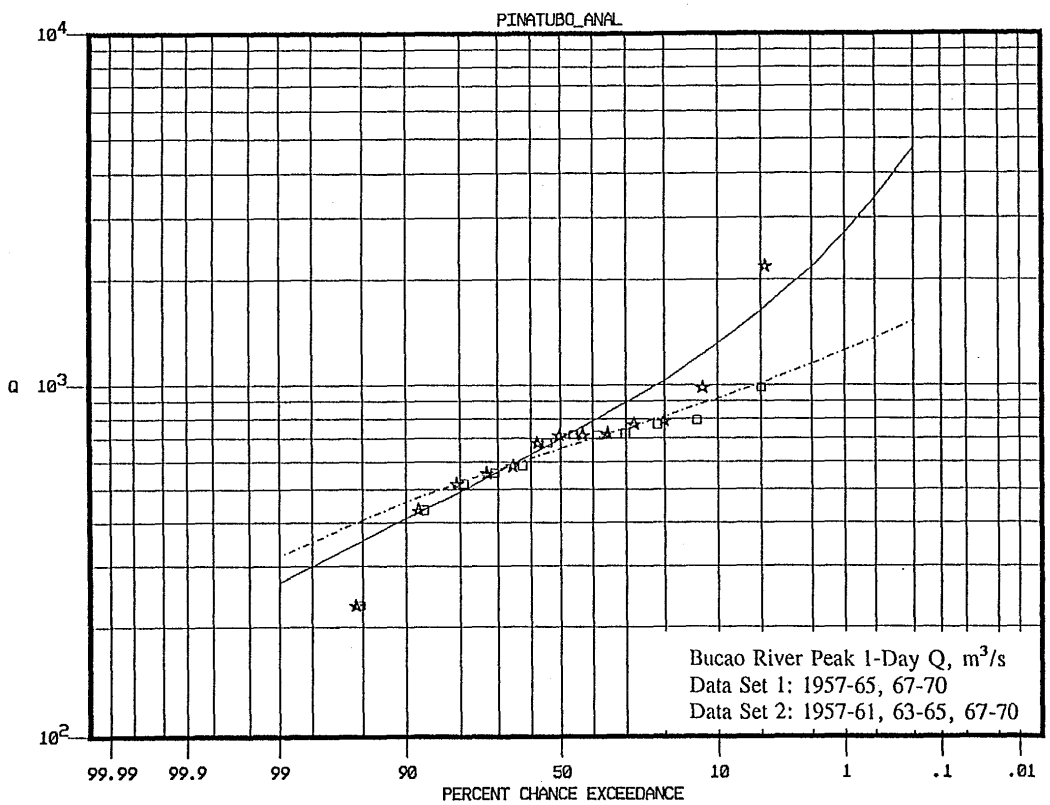
Figure 2.4.78



☆ W093A MAX EVENTS PEAK INST Q
——— W093A MAX ANALYTICAL EXP PROB PEAK INST Q
□ W093A MAX EVENTS PEAK INST Q EXCL 1962
-·-·-·- W093A MAX ANALYTICAL EXP PROB PEAK INST Q EXCL 1962

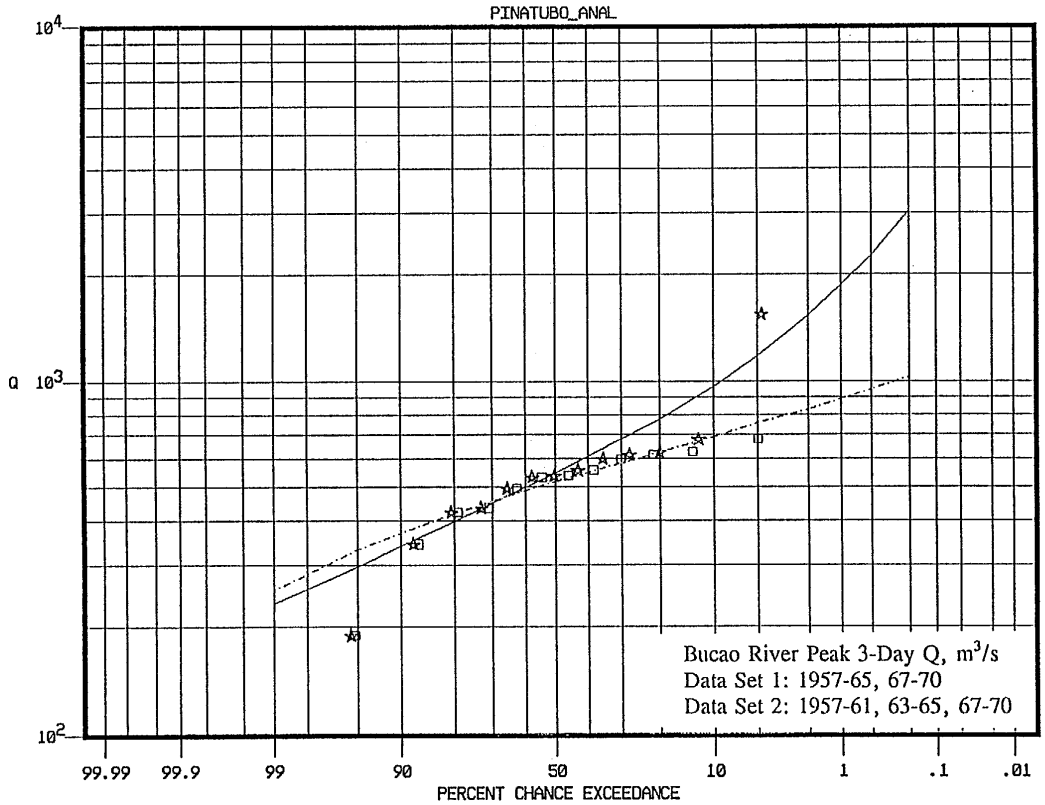
Figure 2.4.79

476



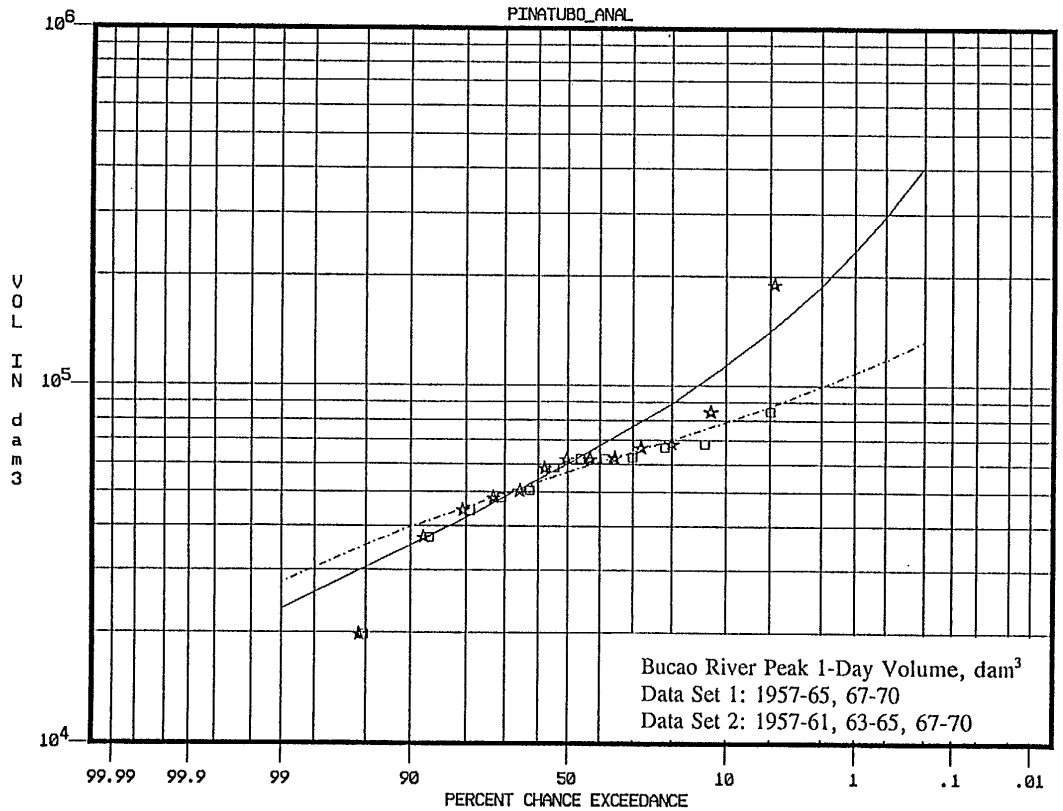
☆ W093A MAX EVENTS PEAK 1-DAY Q
——— W093A MAX ANALYTICAL EXP PROB PEAK 1-DAY Q
□ W093A MAX EVENTS PEAK 1-DAY Q EXCL 1962
-·-·-·- W093A MAX ANALYTICAL EXP PROB PEAK 1-DAY Q EXCL 1962

Figure 2.4.80



☆ W093A MAX EVENTS PEAK 3-DAY Q
——— W093A MAX ANALYTICAL EXP PROB PEAK 3-DAY Q
□ W093A MAX EVENTS PEAK 3-DAY Q EXCL 1962
-·-·-·- W093A MAX ANALYTICAL EXP PROB PEAK 3-DAY Q EXCL 1962

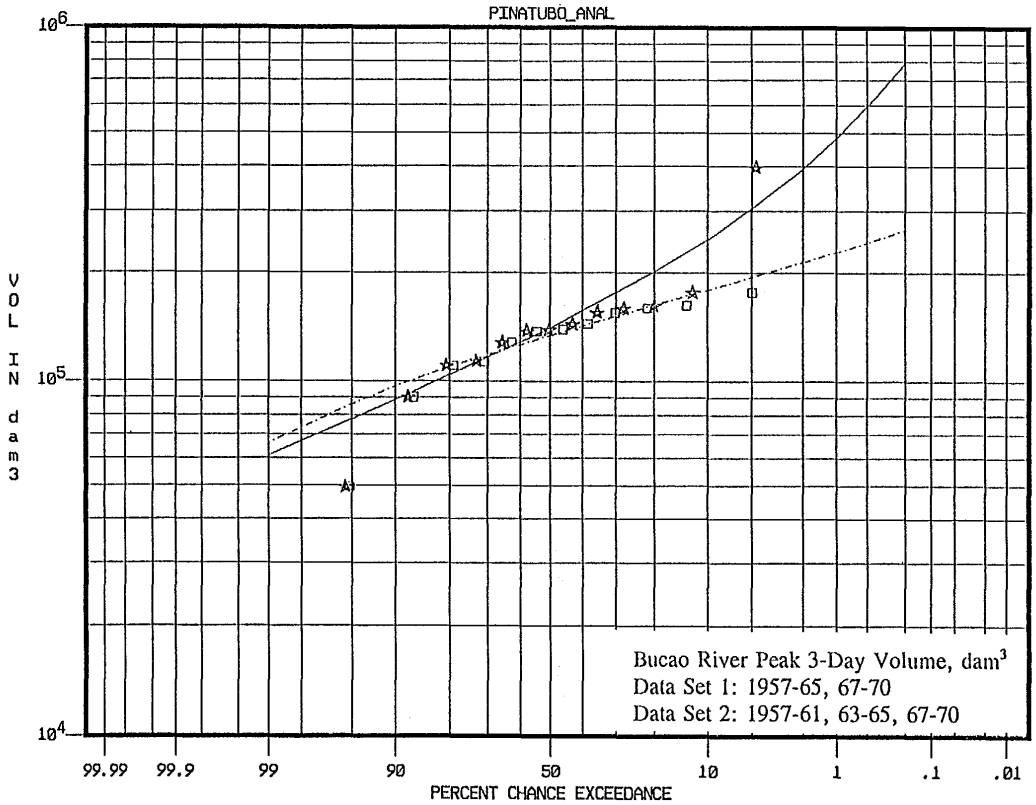
Figure 2.4.81



☆ W093A MAX EVENTS 01-DAY DUR
——— W093A MAX ANALYTICAL EXP PROB 01-DAY DUR
□ W093A MAX EVENTS 01-DAY DUR EXCL 1962
-·-·-·- W093A MAX ANALYTICAL EXP PROB 01-DAY DUR EXCL 1962

Figure 2.4.82

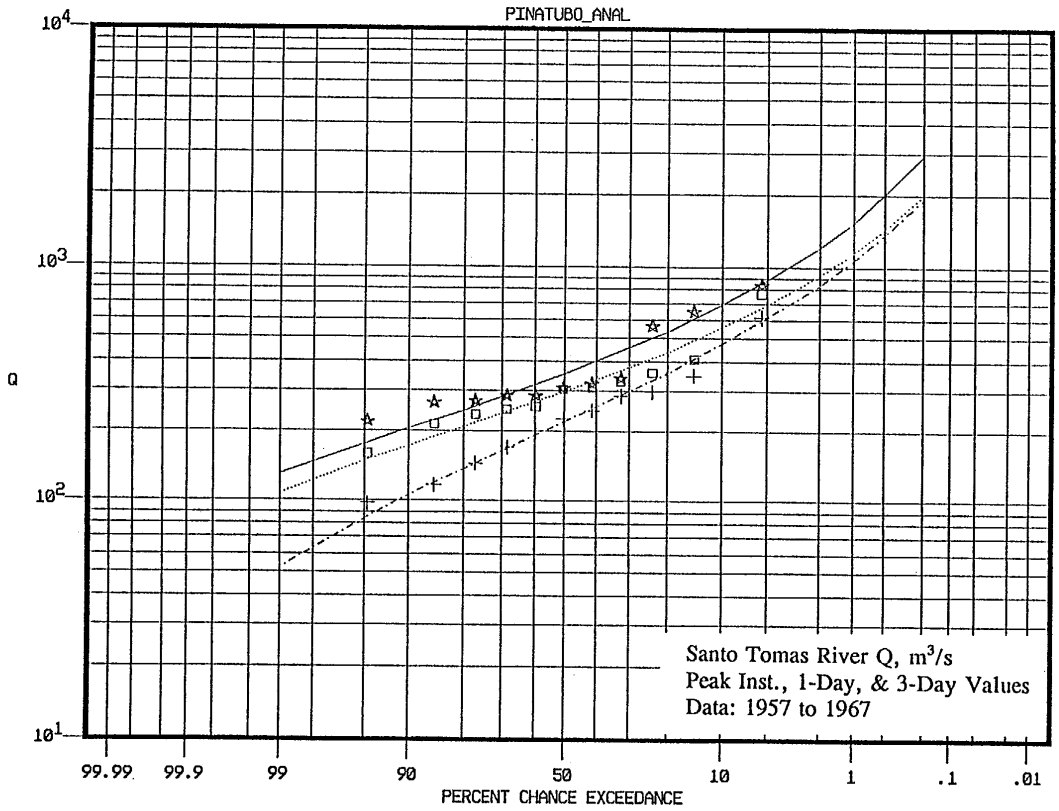
478



☆ W093A MAX EVENTS 03-DAY DUR
——— W093A MAX ANALYTICAL EXP PROB 03-DAY DUR
□ W093A MAX EVENTS 03-DAY DUR EXCL 1962
-·-·-·- W093A MAX ANALYTICAL EXP PROB 03-DAY DUR EXCL 1962

Figure 2.4.83

479



☆ W094A MAX EVENTS PEAK INST Q 57-67

W094A MAX ANALYTICAL EXP PROB PEAK INST Q 57-67

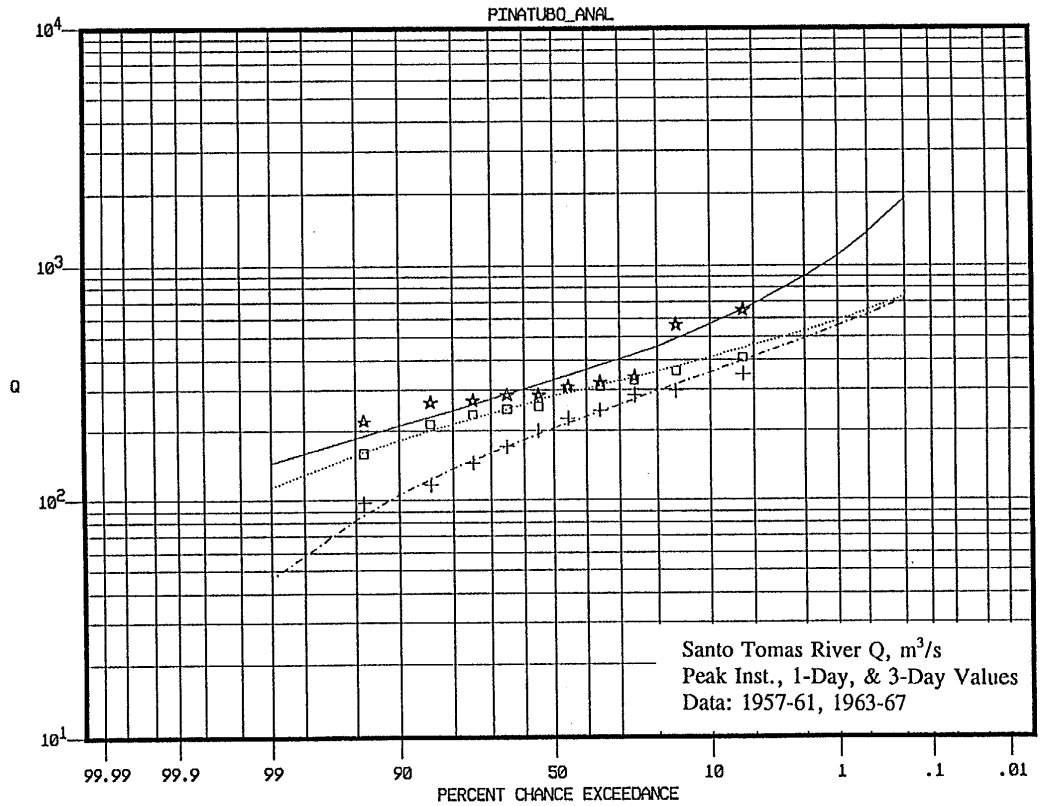
□ W094A MAX EVENTS PEAK 1-DAY 57-67

W094A MAX ANALYTICAL EXP PROB PEAK 1-DAY 57-67

+ W094A MAX EVENTS PEAK 3-DAY 57-67

W094A MAX ANALYTICAL EXP PROB PEAK 3-DAY 57-67

Figure 2.4.84



☆ W094A MAX EVENTS PEAK INST Q 57-67 EXCL 62

W094A MAX ANALYTICAL EXP PROB PEAK INST Q 57-67 EXCL 62

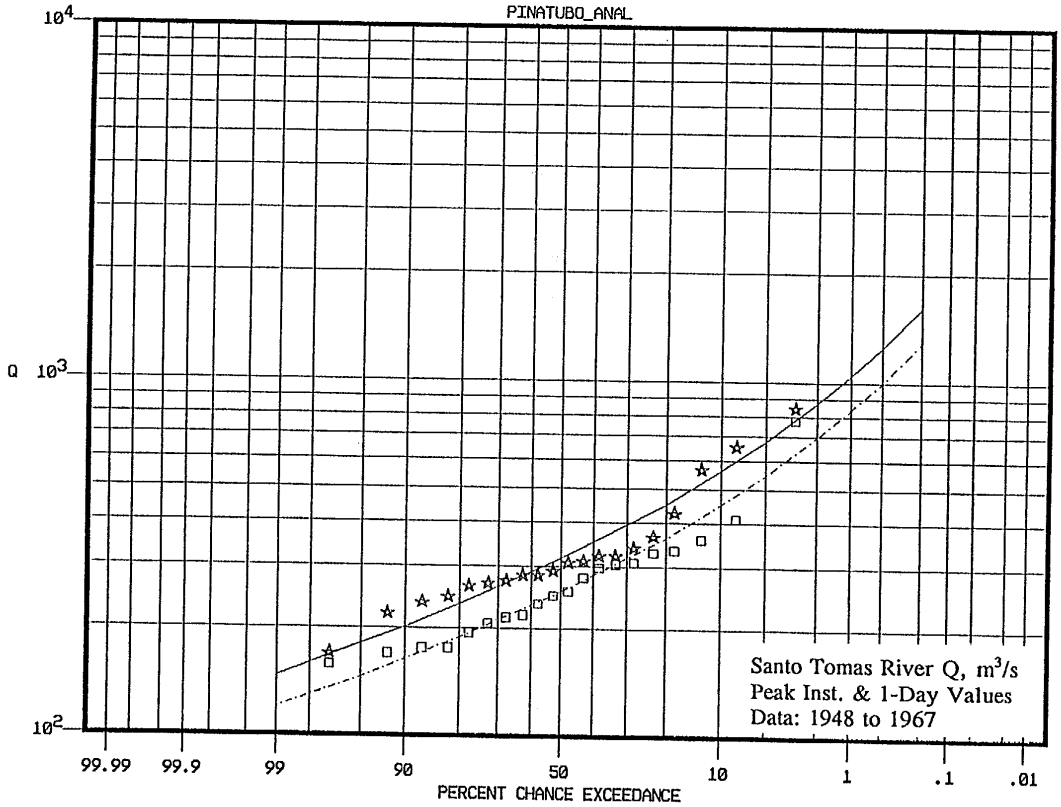
□ W094A MAX EVENTS PEAK 1-DAY 57-67 EXCL 62

W094A MAX ANALYTICAL EXP PROB PEAK 1-DAY 57-67 EXCL 62

+ W094A MAX EVENTS PEAK 3-DAY 57-67 EXCL 62

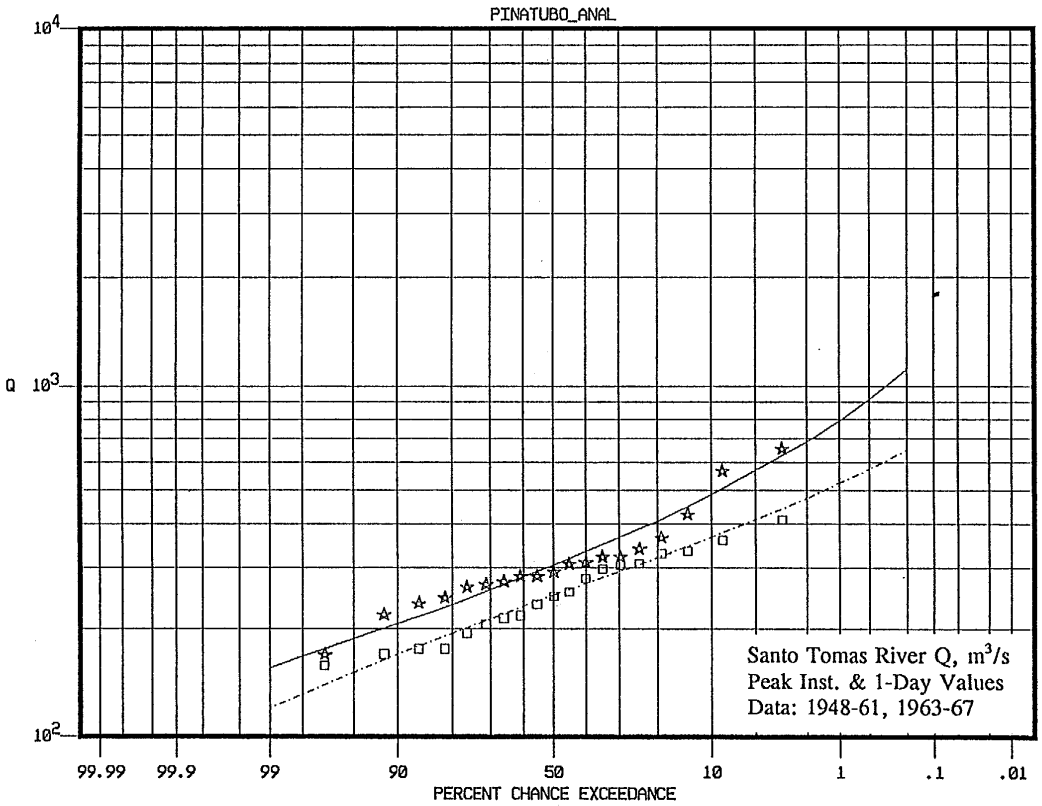
W094A MAX ANALYTICAL EXP PROB PEAK 3-DAY 57-67 EXCL 62

Figure 2.4.85



☆ W094A MAX EVENTS PEAK INST Q 48-67
——— W094A MAX ANALYTICAL EXP PROB PEAK INST Q 48-67
□ W094A MAX EVENTS PEAK 1-DAY 48-67
-·-·-·- W094A MAX ANALYTICAL EXP PROB PEAK 1-DAY 48-67

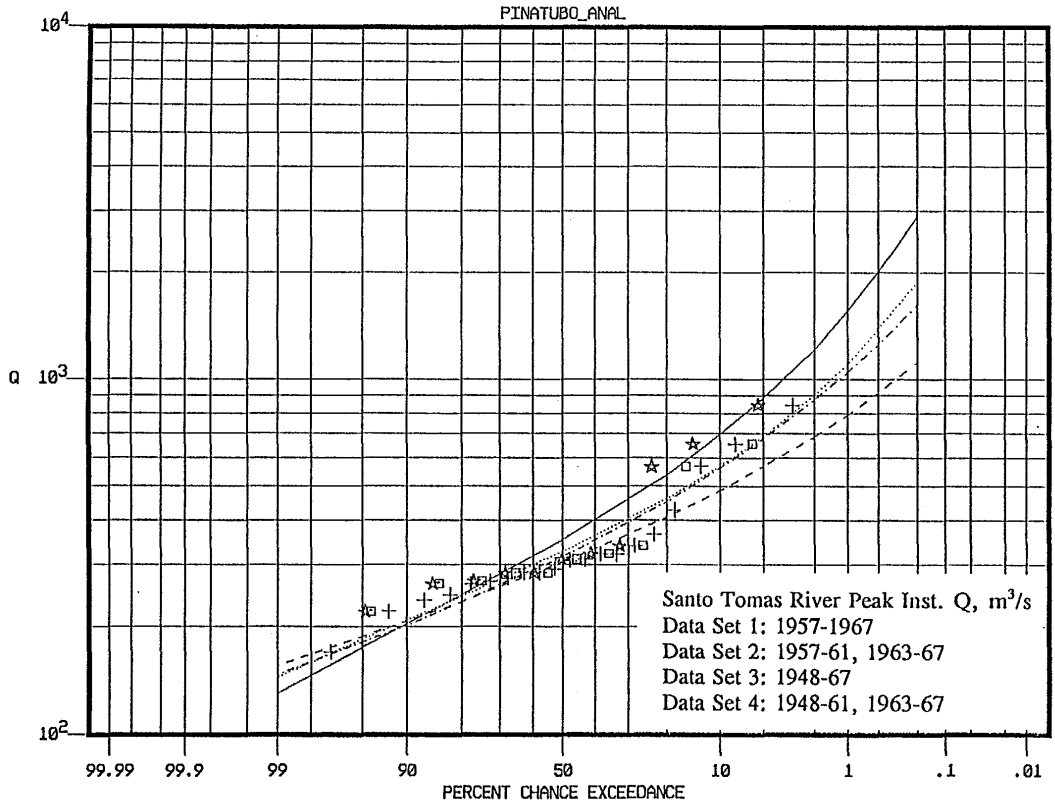
Figure 2.4.86



☆ W094A MAX EVENTS PEAK INST Q 48-67 EXCL 62
——— W094A MAX ANALYTICAL EXP PROB PEAK INST Q 48-67 EXCL 62
□ W094A MAX EVENTS PEAK 1-DAY 48-67 EXCL 62
-·-·-·- W094A MAX ANALYTICAL EXP PROB PEAK 1-DAY 48-67 EXCL 62

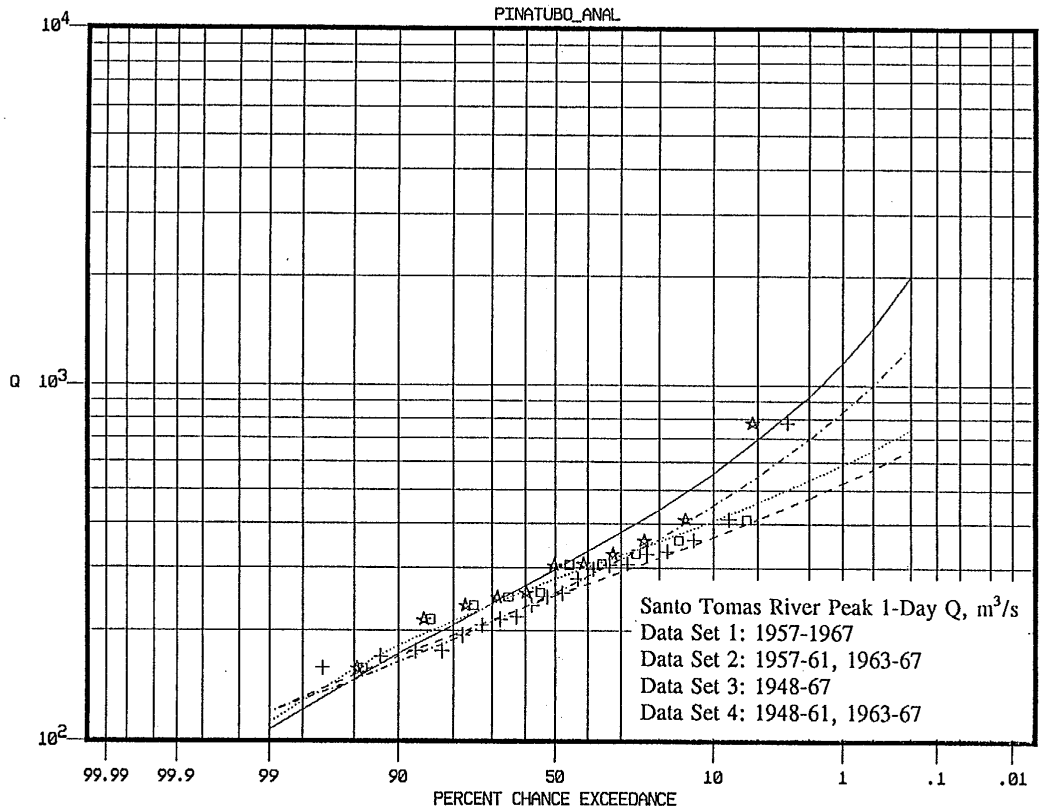
Figure 2.4.87

483



☆	W094A MAX EVENTS PEAK INST Q 57-67	+	W094A MAX EVENTS PEAK INST Q 48-67
———	W094A MAX ANALYTICAL EXP PROB PEAK INST Q 57-67	-·-·-·-	W094A MAX ANALYTICAL EXP PROB PEAK INST Q 48-67
□	W094A MAX EVENTS PEAK INST Q 57-67 EXCL 62	- - - - -	W094A MAX ANALYTICAL EXP PROB PEAK INST Q 48-67 EXCL 62
·····	W094A MAX ANALYTICAL EXP PROB PEAK INST Q 57-67 EXCL 62		

Figure 2.4.88



☆ W094A MAX EVENTS PEAK 1-DAY 57-67

W094A MAX ANALYTICAL EXP PROB PEAK 1-DAY 57-67

□ W094A MAX EVENTS PEAK 1-DAY 57-67 EXCL 62

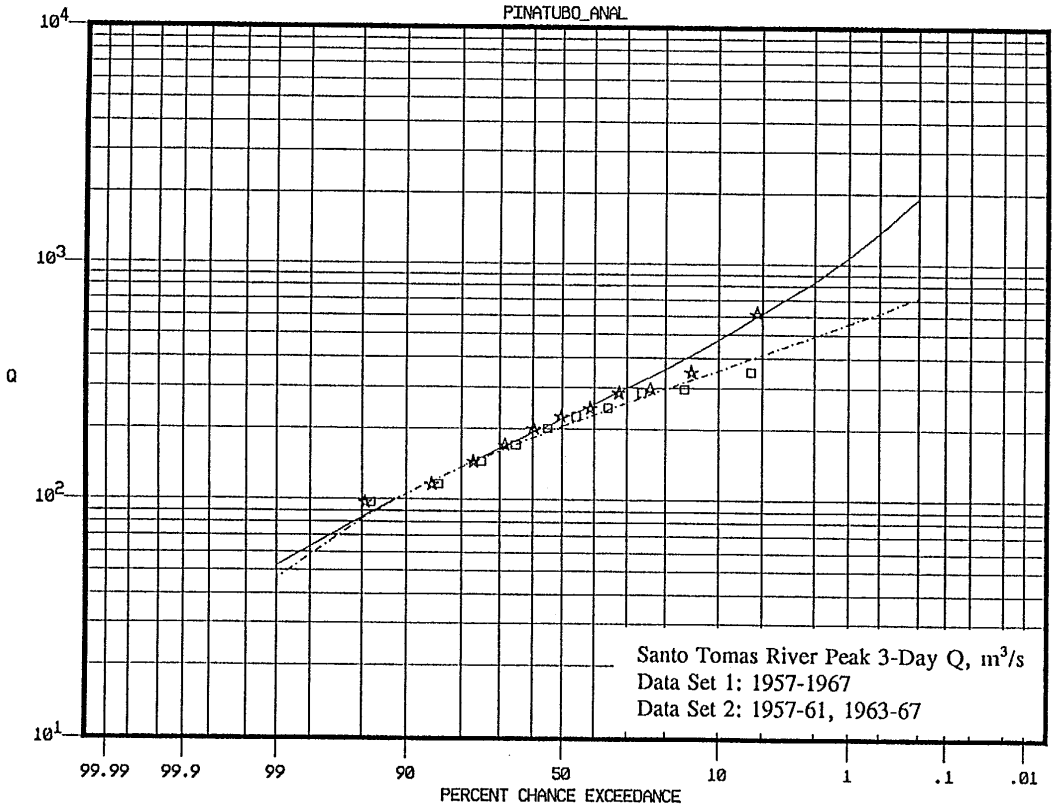
W094A MAX ANALYTICAL EXP PROB PEAK 1-DAY 57-67 EXCL 62

+ W094A MAX EVENTS PEAK 1-DAY 48-67

W094A MAX ANALYTICAL EXP PROB PEAK 1-DAY 48-67

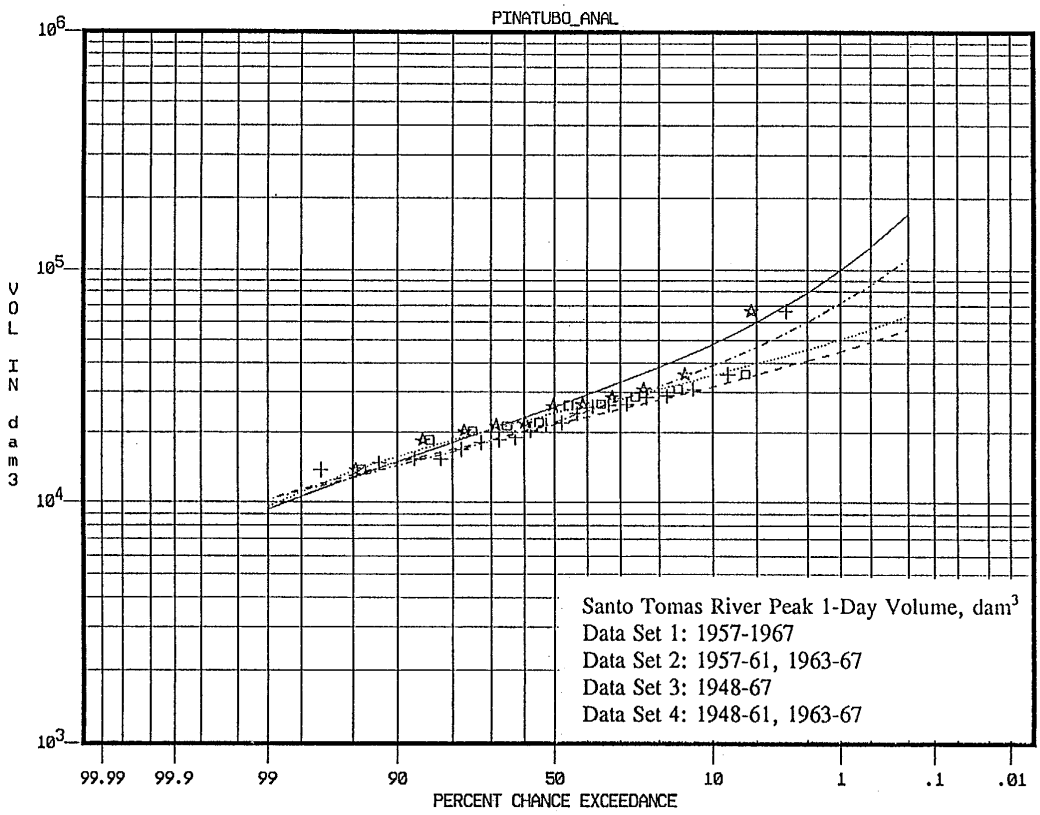
W094A MAX ANALYTICAL EXP PROB PEAK 1-DAY 48-67 EXCL 62

Figure 2.4.89



☆ W094A MAX EVENTS PEAK 3-DAY 57-67
——— W094A MAX ANALYTICAL EXP PROB PEAK 3-DAY 57-67
□ W094A MAX EVENTS PEAK 3-DAY 57-67 EXCL 62
-·-·-·- W094A MAX ANALYTICAL EXP PROB PEAK 3-DAY 57-67 EXCL 62

Figure 2.4.90



☆ W094A MAX EVENTS 01-DAY DUR

——— W094A MAX ANALYTICAL EXP PROB 01-DAY DUR

□ W094A MAX EVENTS 01-DAY DUR EXCL 1962

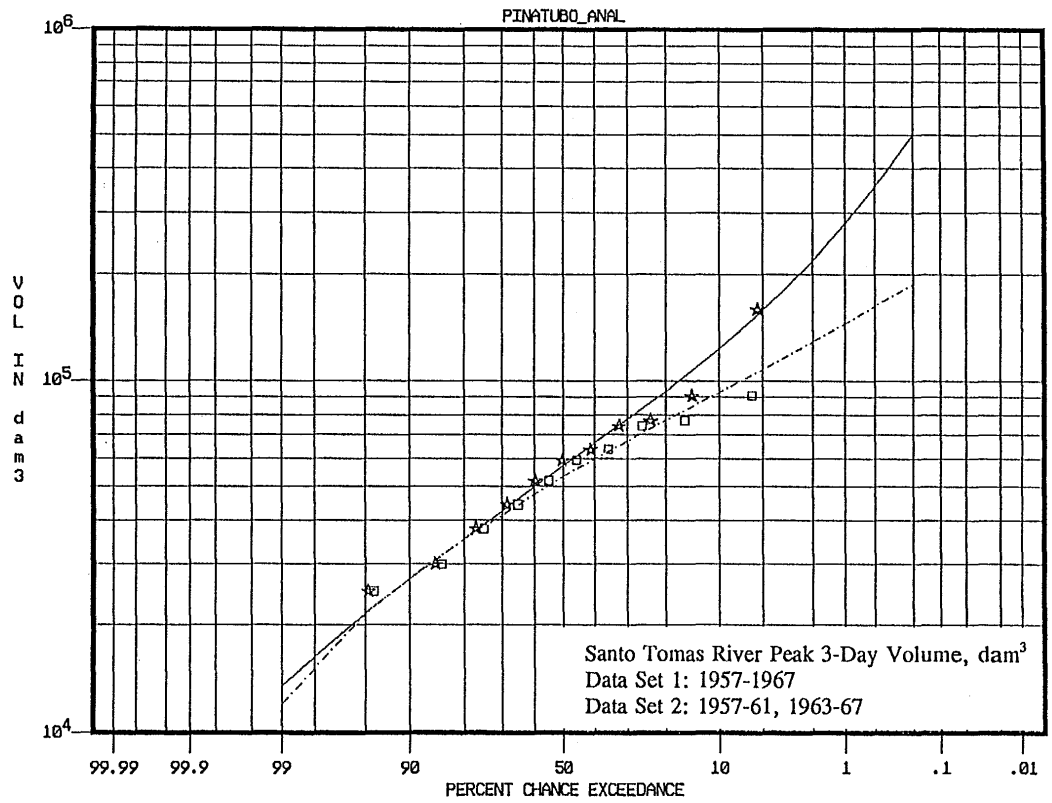
····· W094A MAX ANALYTICAL EXP PROB 01-DAY DUR EXCL 1962

+ W094A MAX EVENTS 01-DAY DUR 48-67

-·-·-·-·- W094A MAX ANALYTICAL EXP PROB 01-DAY DUR 48-67

- - - - - - W094A MAX ANALYTICAL EXP PROB 01-DAY DUR 48-67 EXCL 62

Figure 2.4.91



☆ W094A MAX EVENTS 03-DAY DUR
——— W094A MAX ANALYTICAL EXP PROB 03-DAY DUR
□ W094A MAX EVENTS 03-DAY DUR EXCL 1962
-·-·-·- W094A MAX ANALYTICAL EXP PROB 03-DAY DUR EXCL 1962

Figure 2.4.92

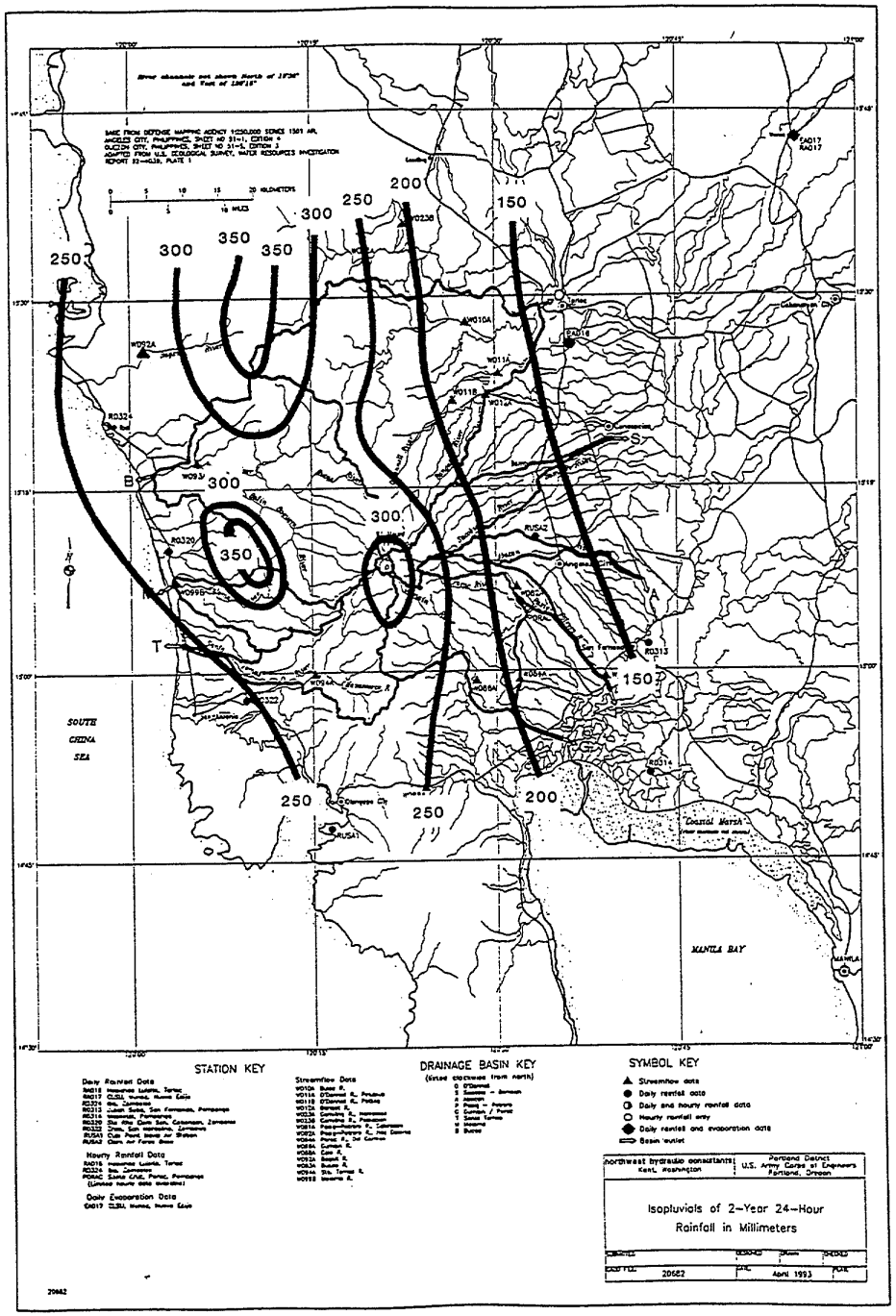


Figure 2.4.93

489

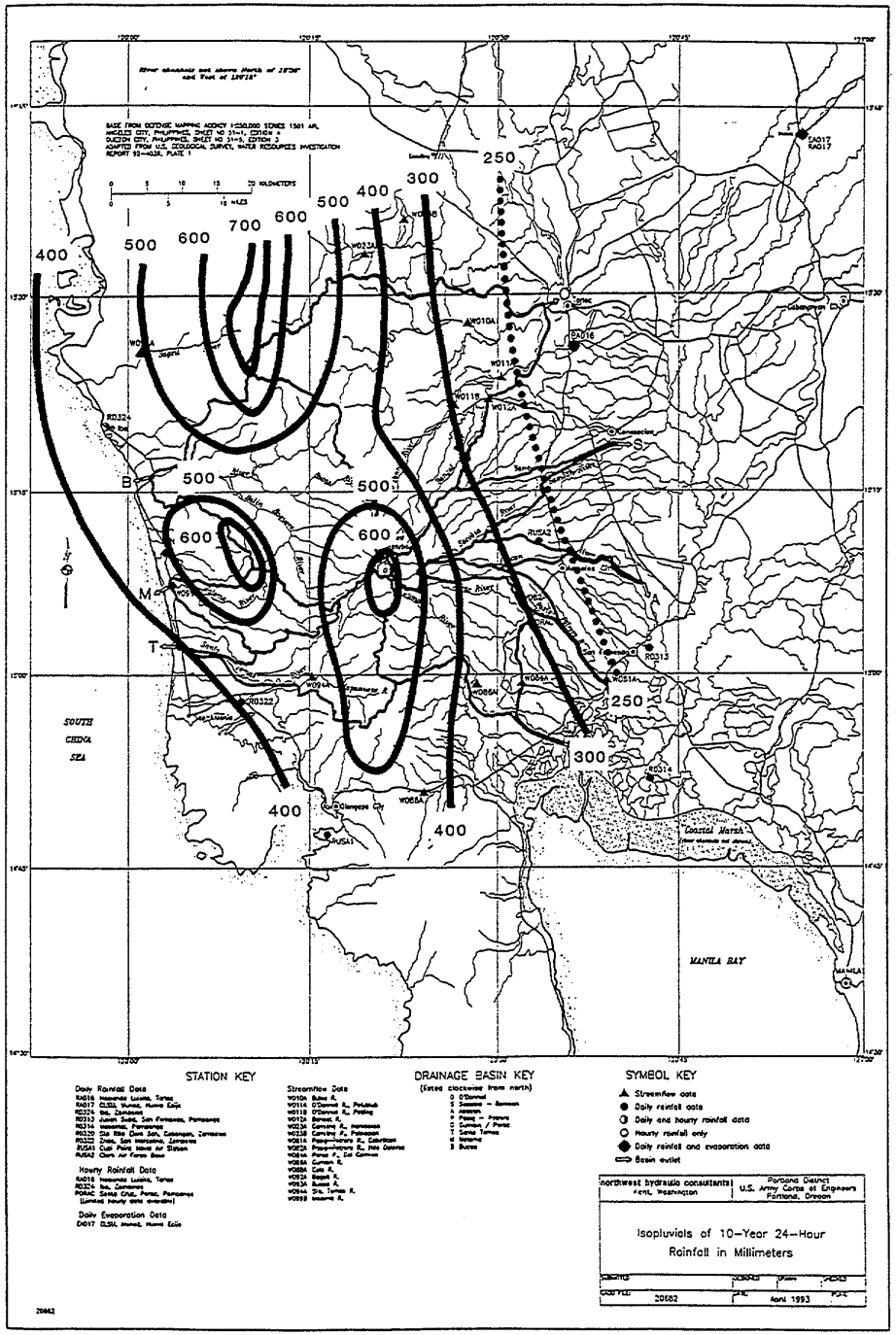


Figure 2.4.94

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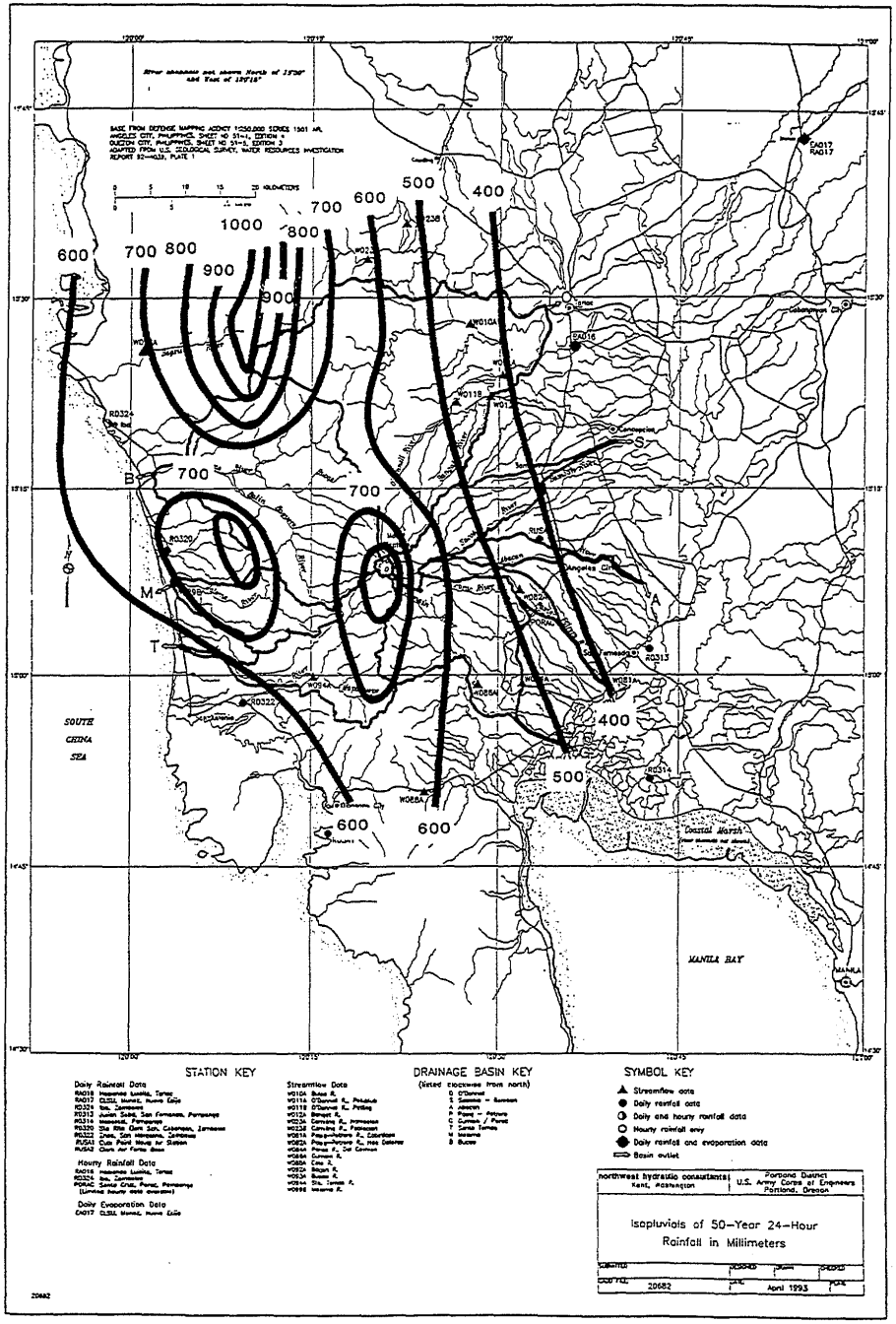

Figure 2.4.95

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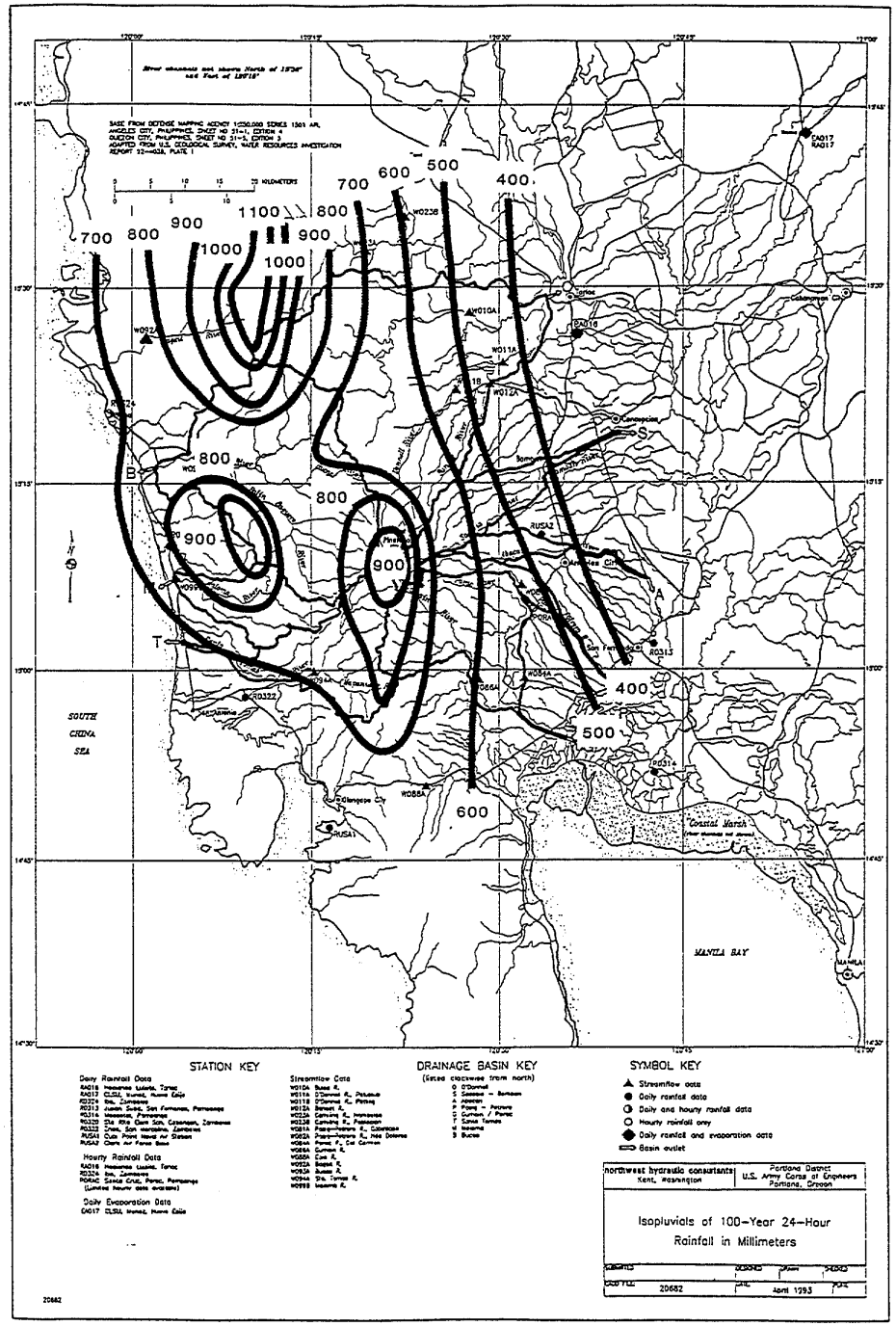
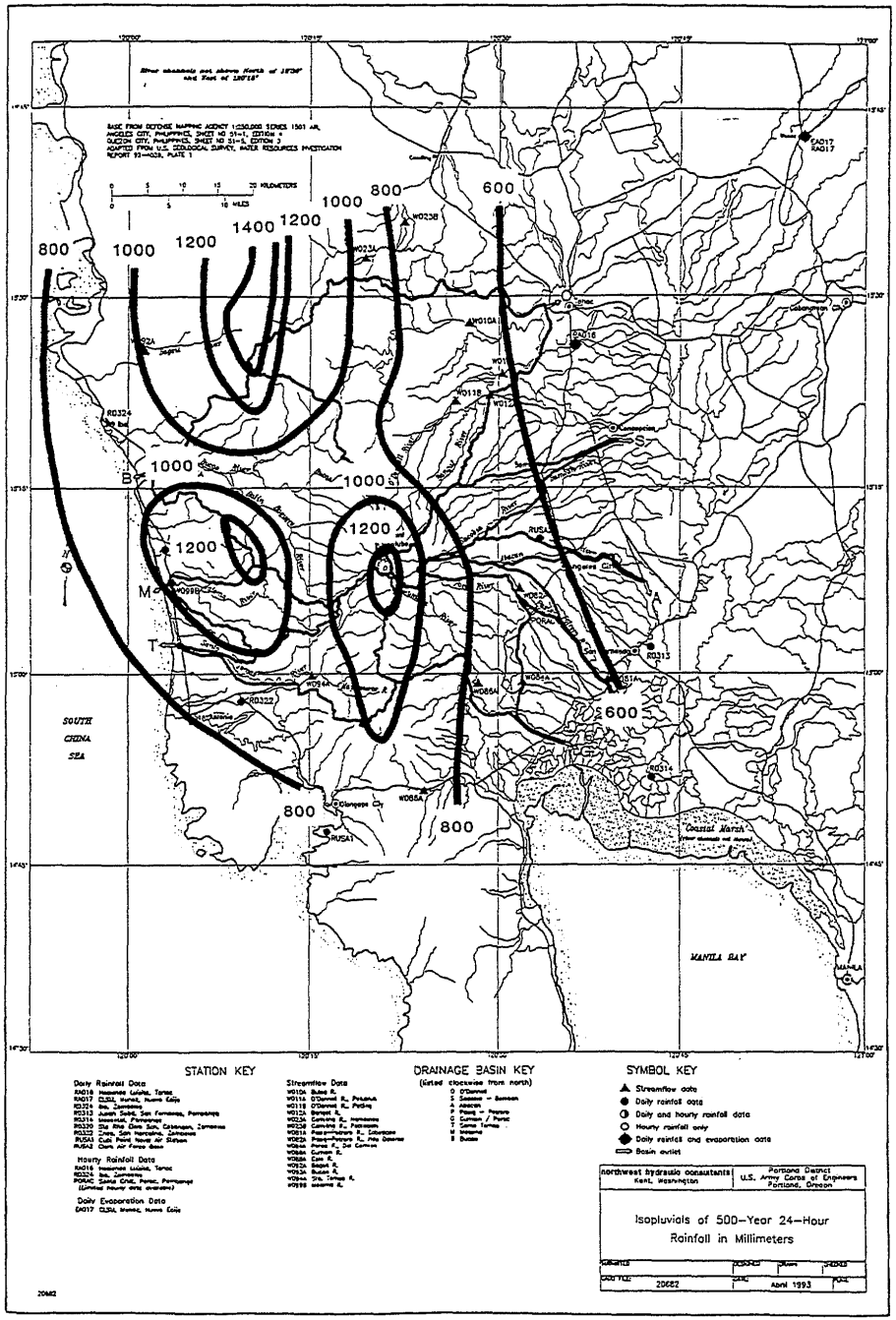


Figure 2.4.96

492


800
1000
1200
1400
1200
1000
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600
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1200
600
800
800
SOUTH CHINA SEA
MANILA BAY
STATION KEY
DRAINAGE BASIN KEY
SYMBOL KEY
Isopluvials of 500-Year 24-Hour Rainfall in Millimeters
April 1993

Figure 2.4.97

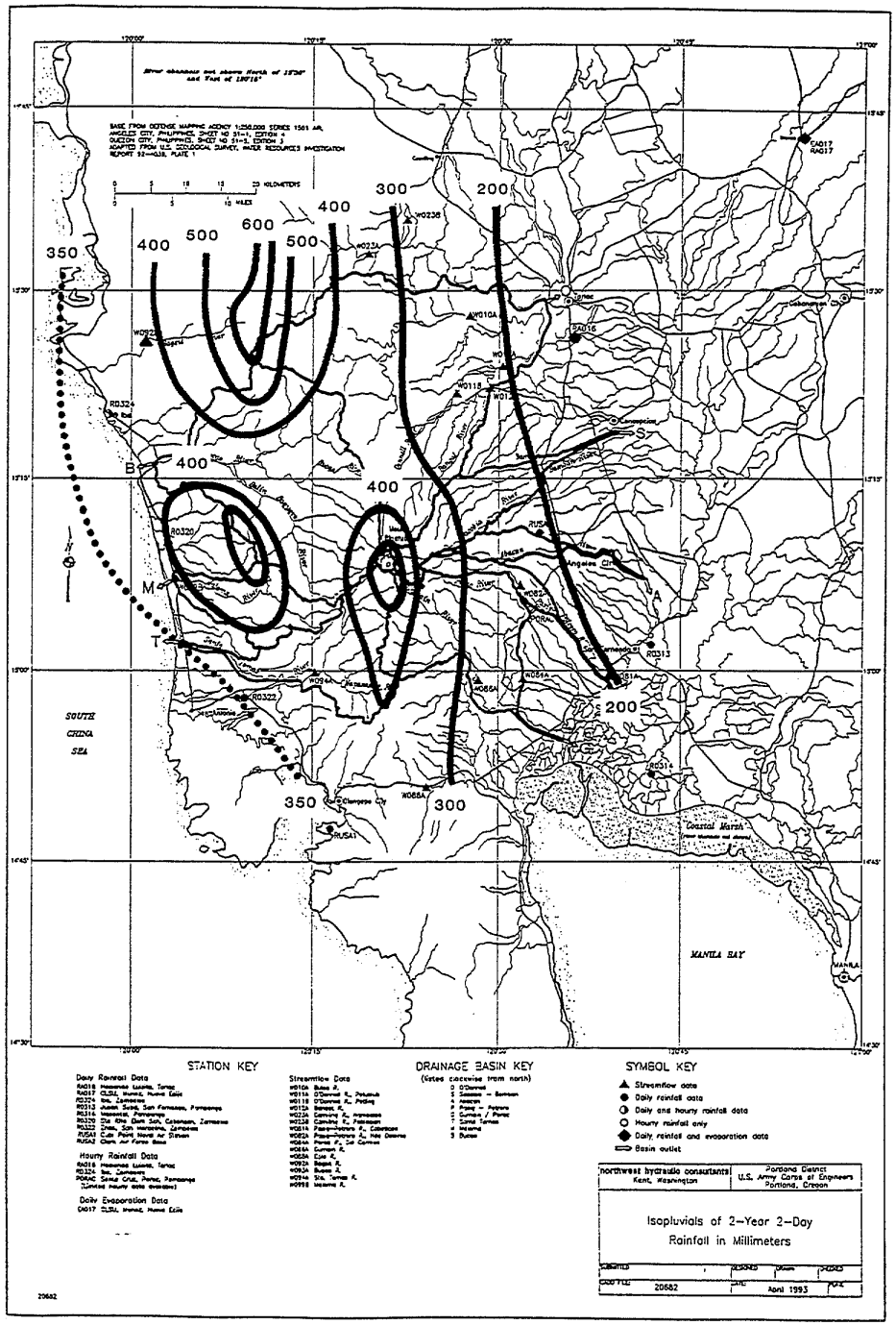

Figure 2.4.98

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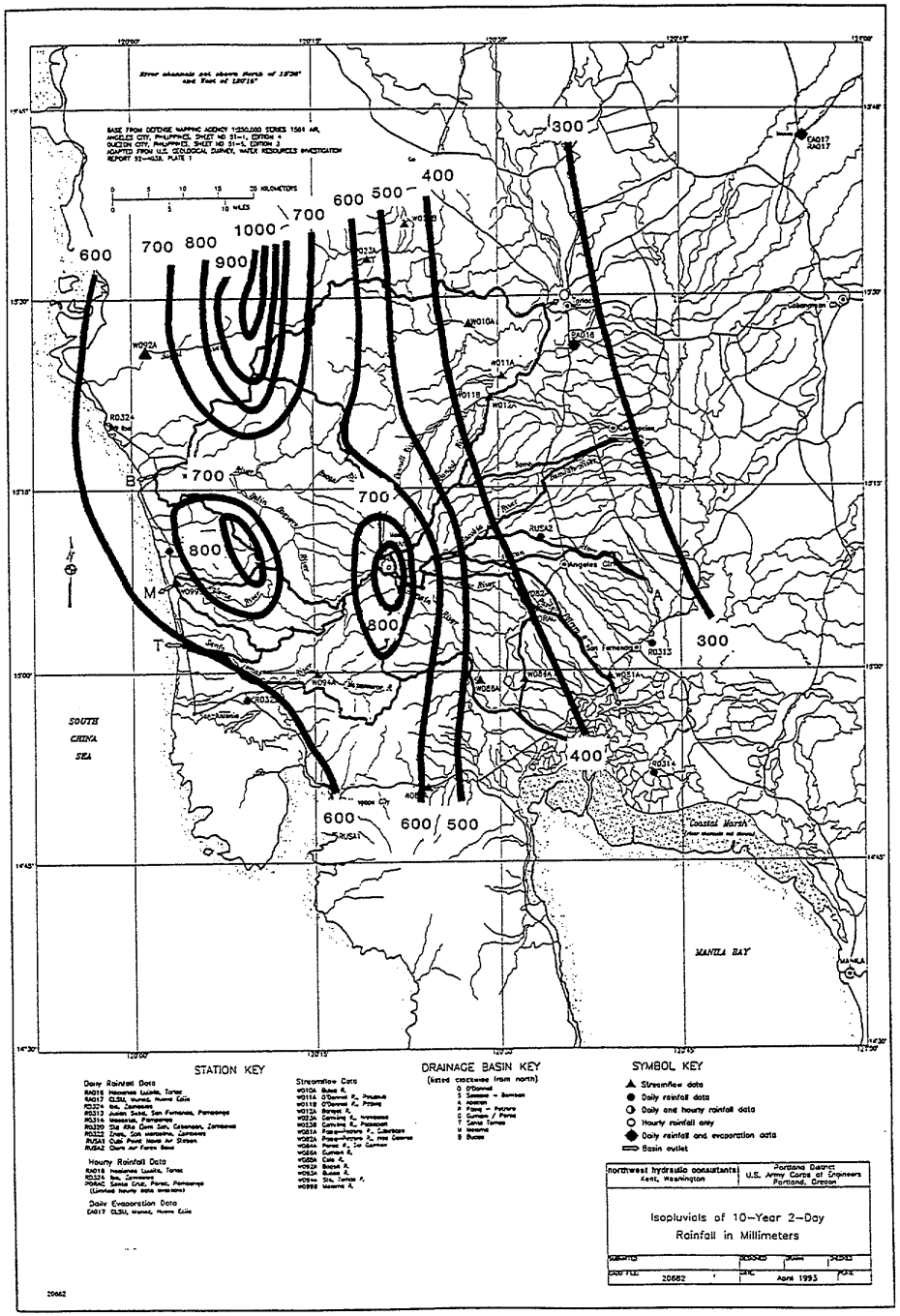

Figure 2.4.99

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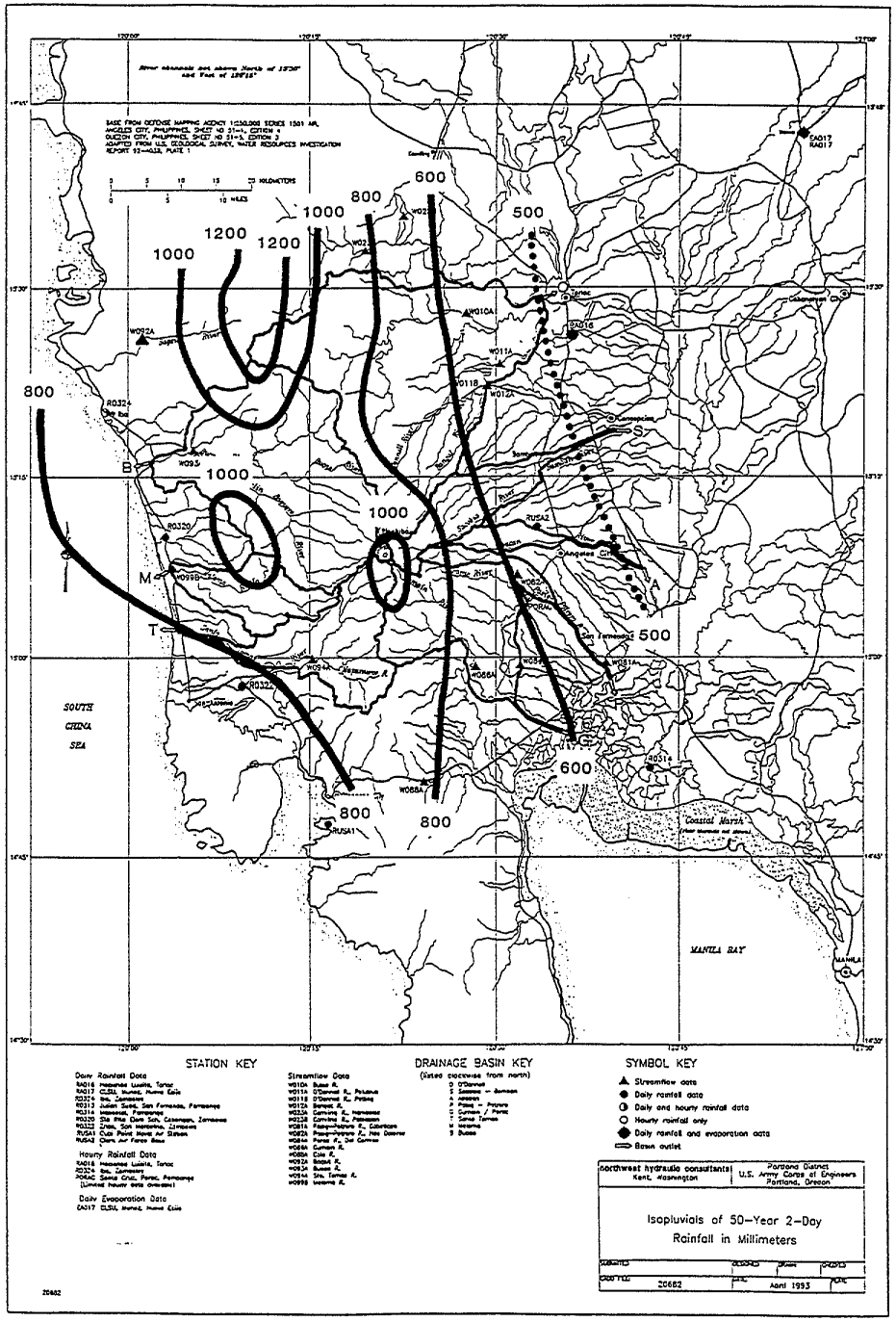

Figure 2.4.100

496

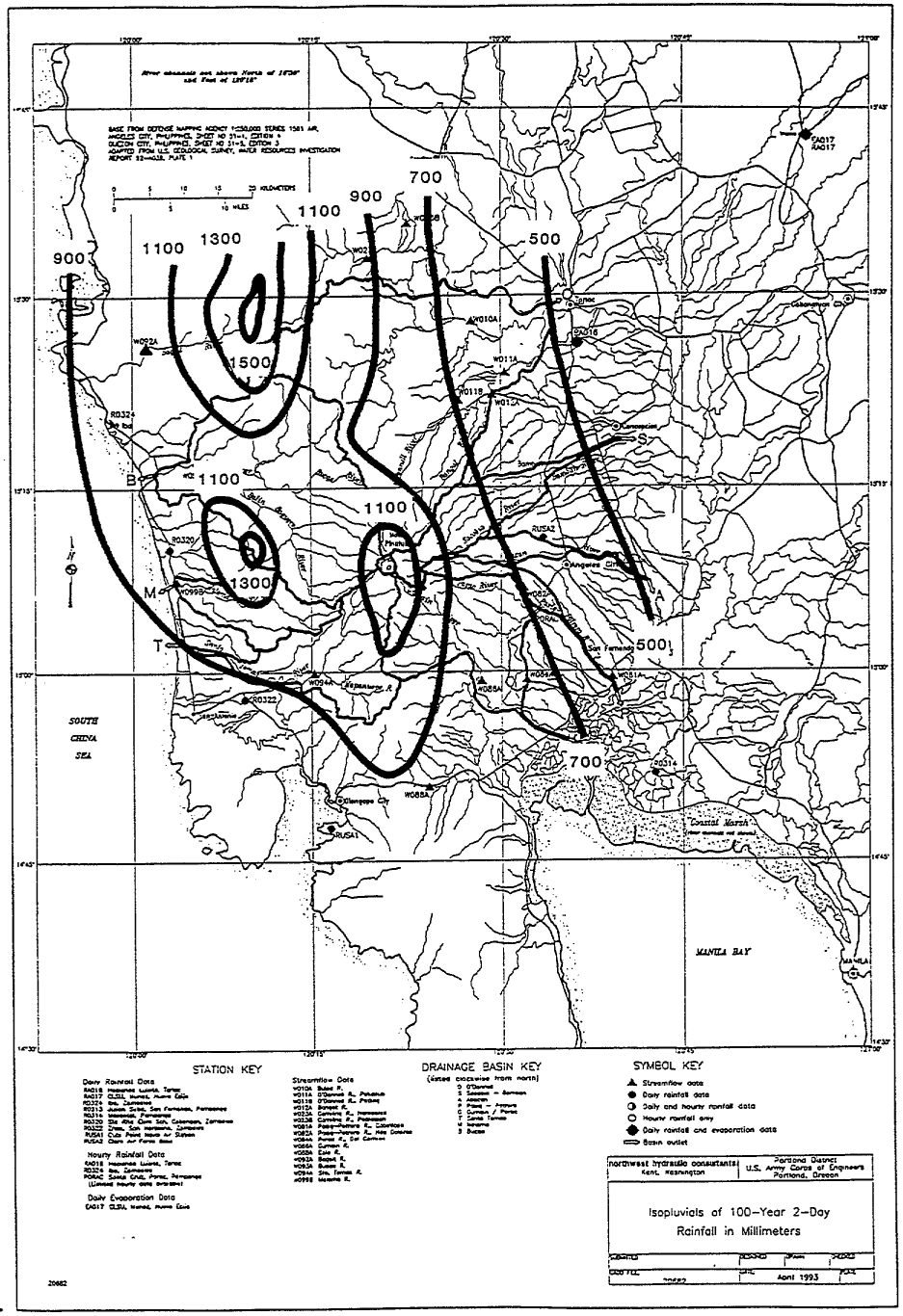

Figure 2.4.101

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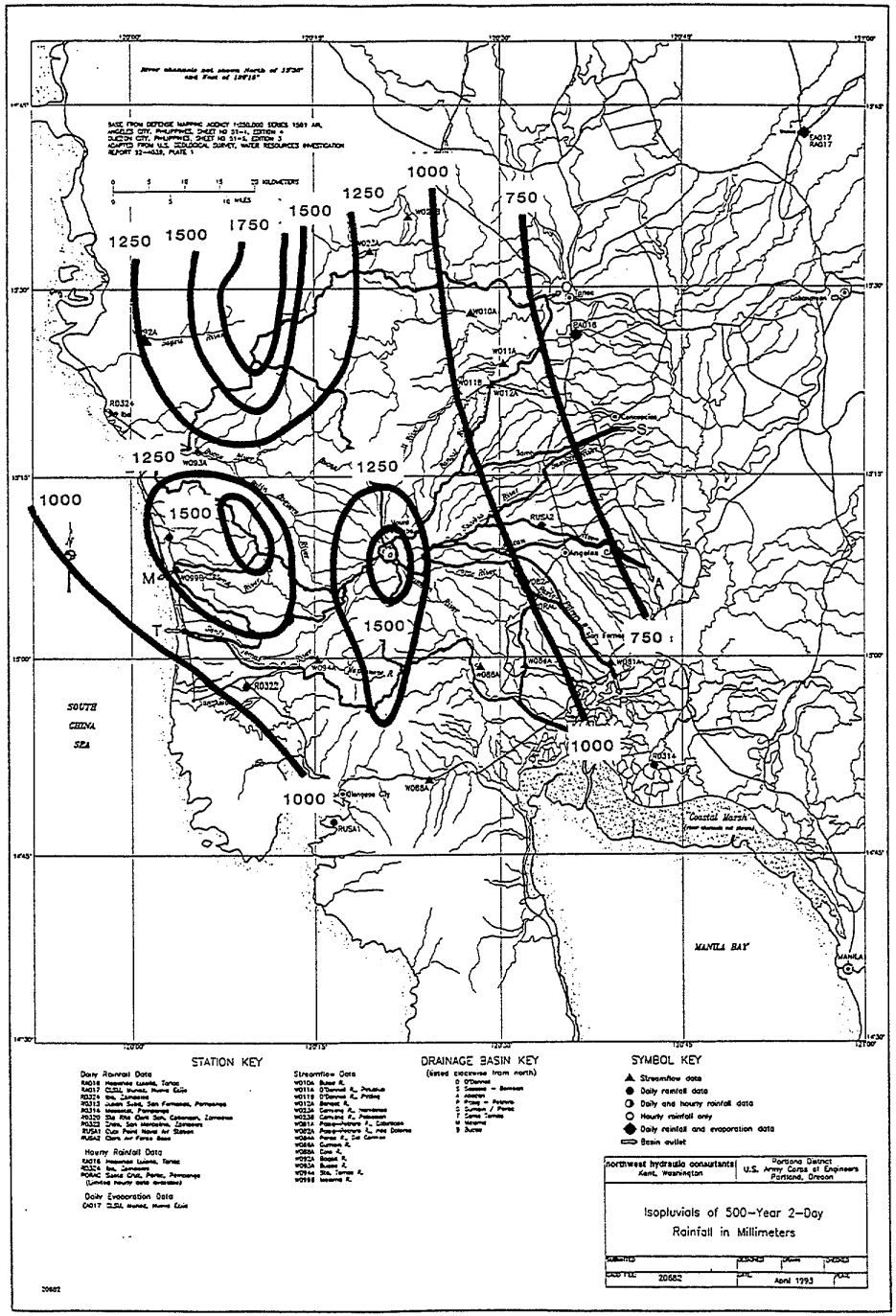

Figure 2.4.102

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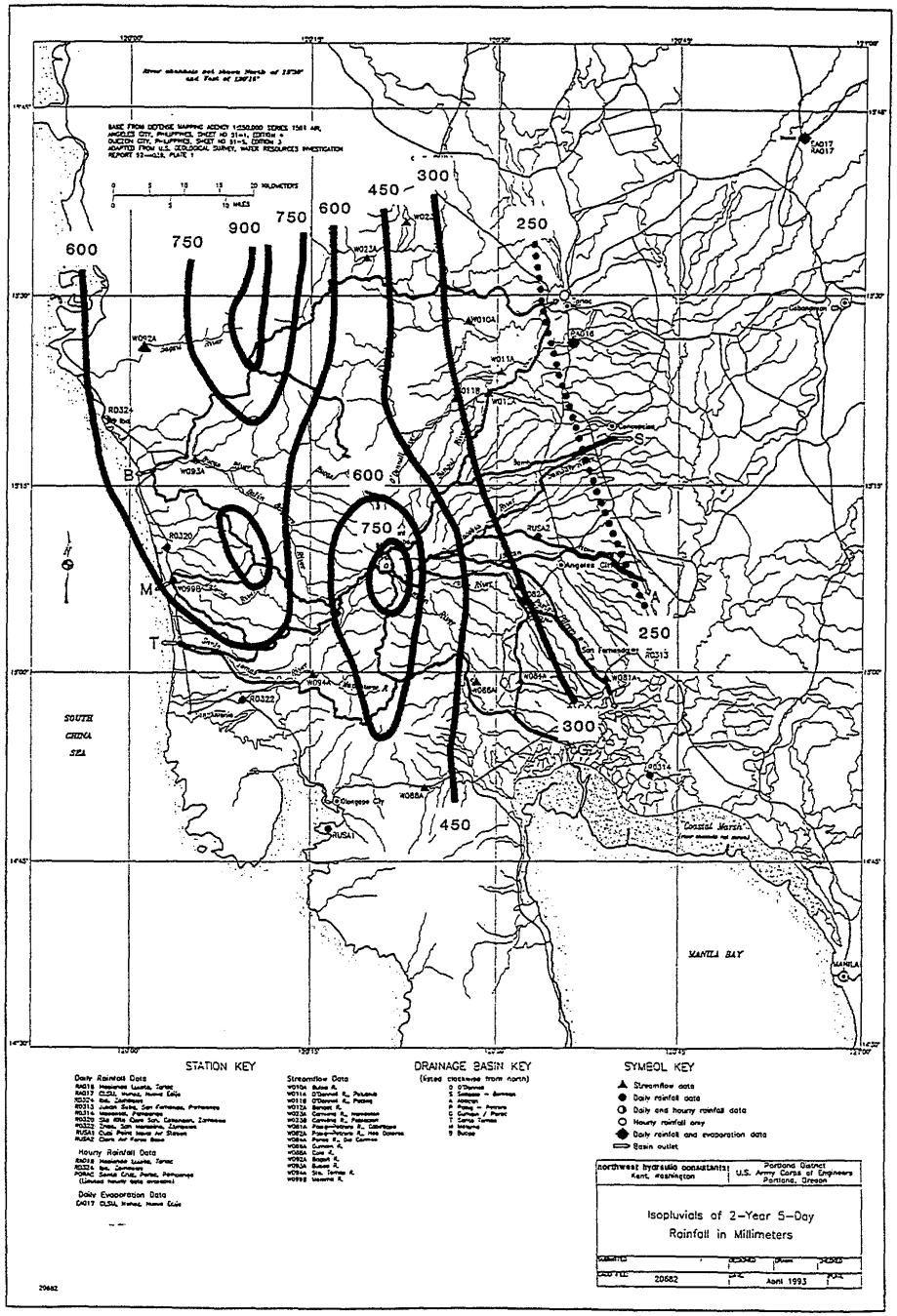

Figure 2.4.103

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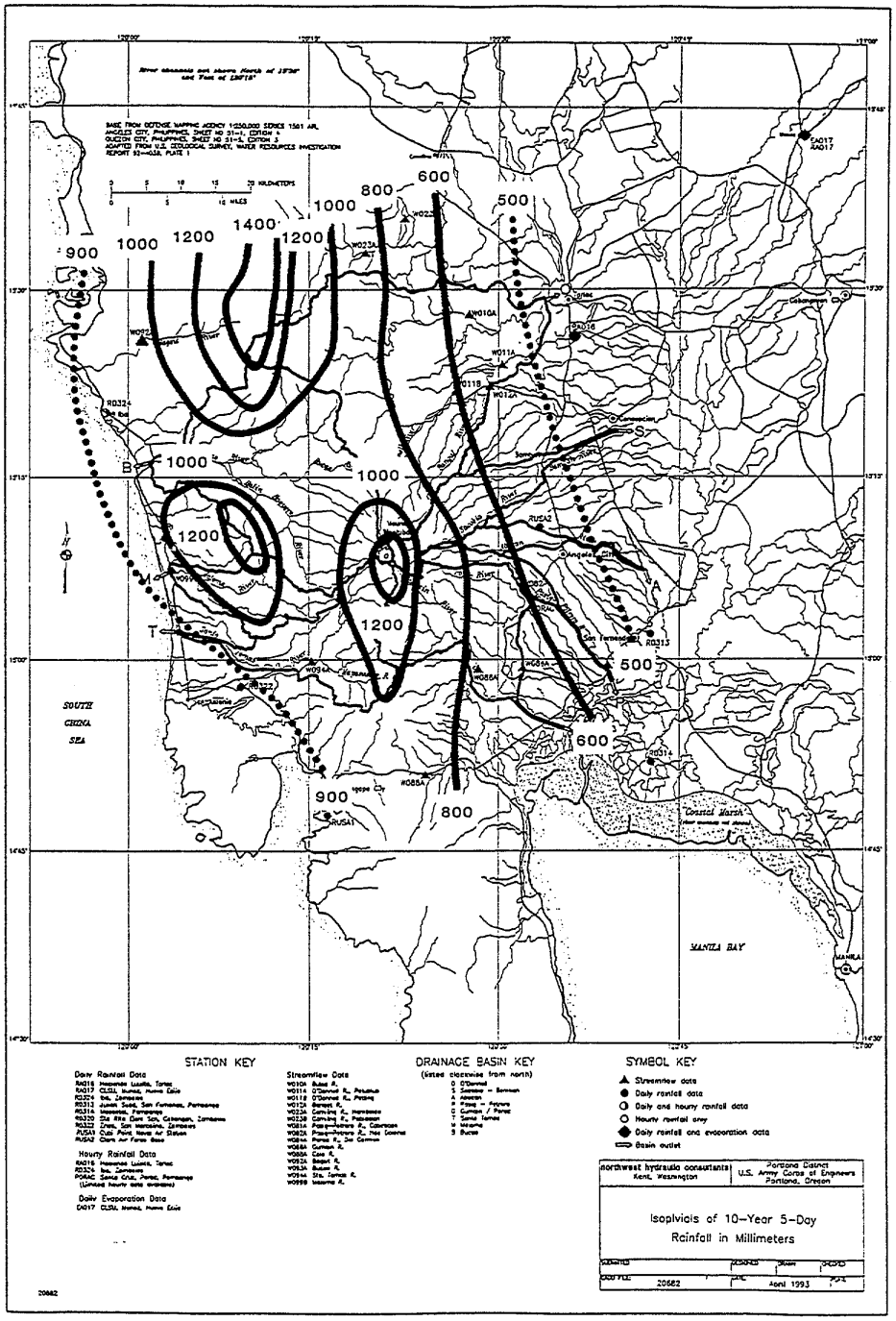

Figure 2.4.104

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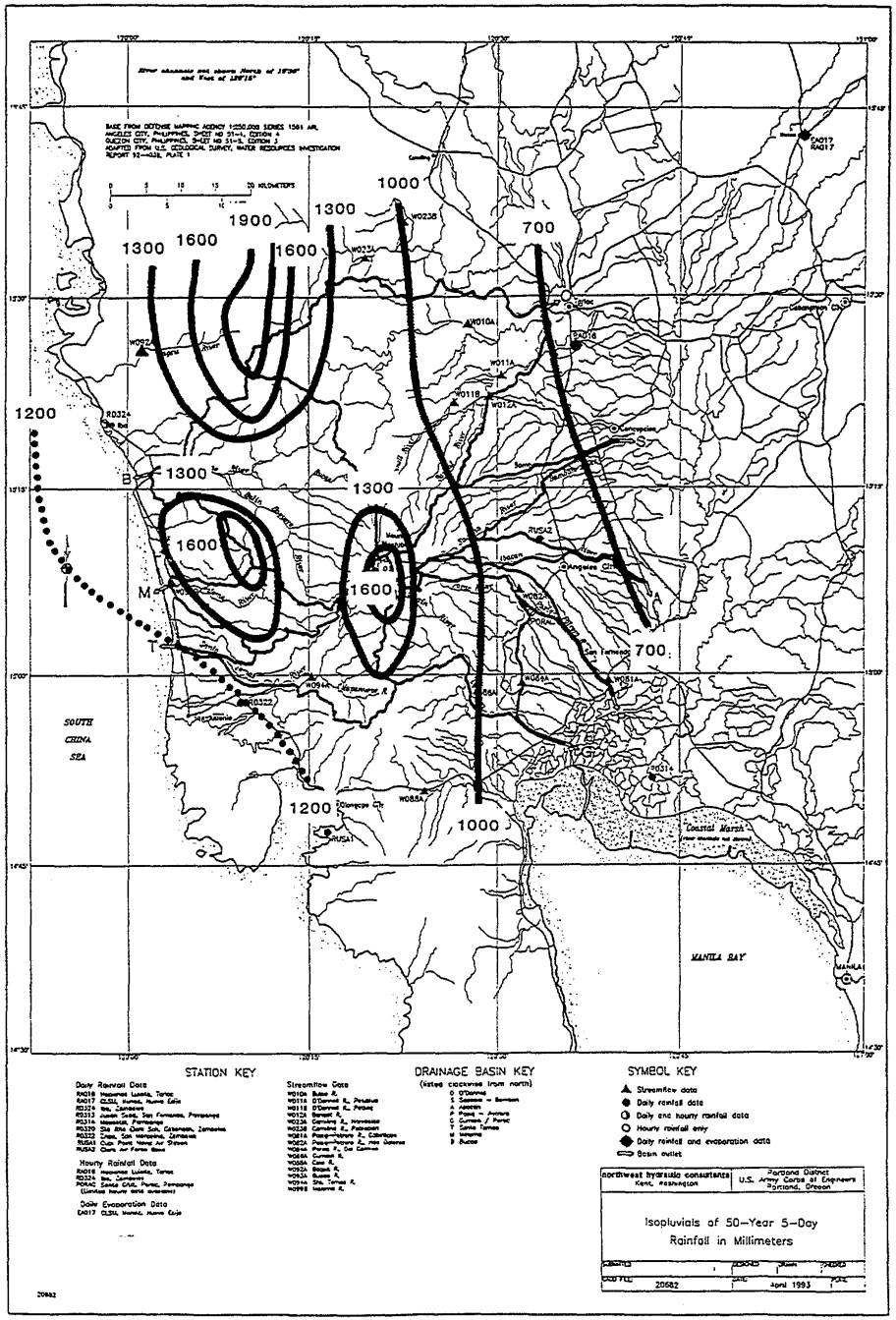


Figure 2.4.105

801

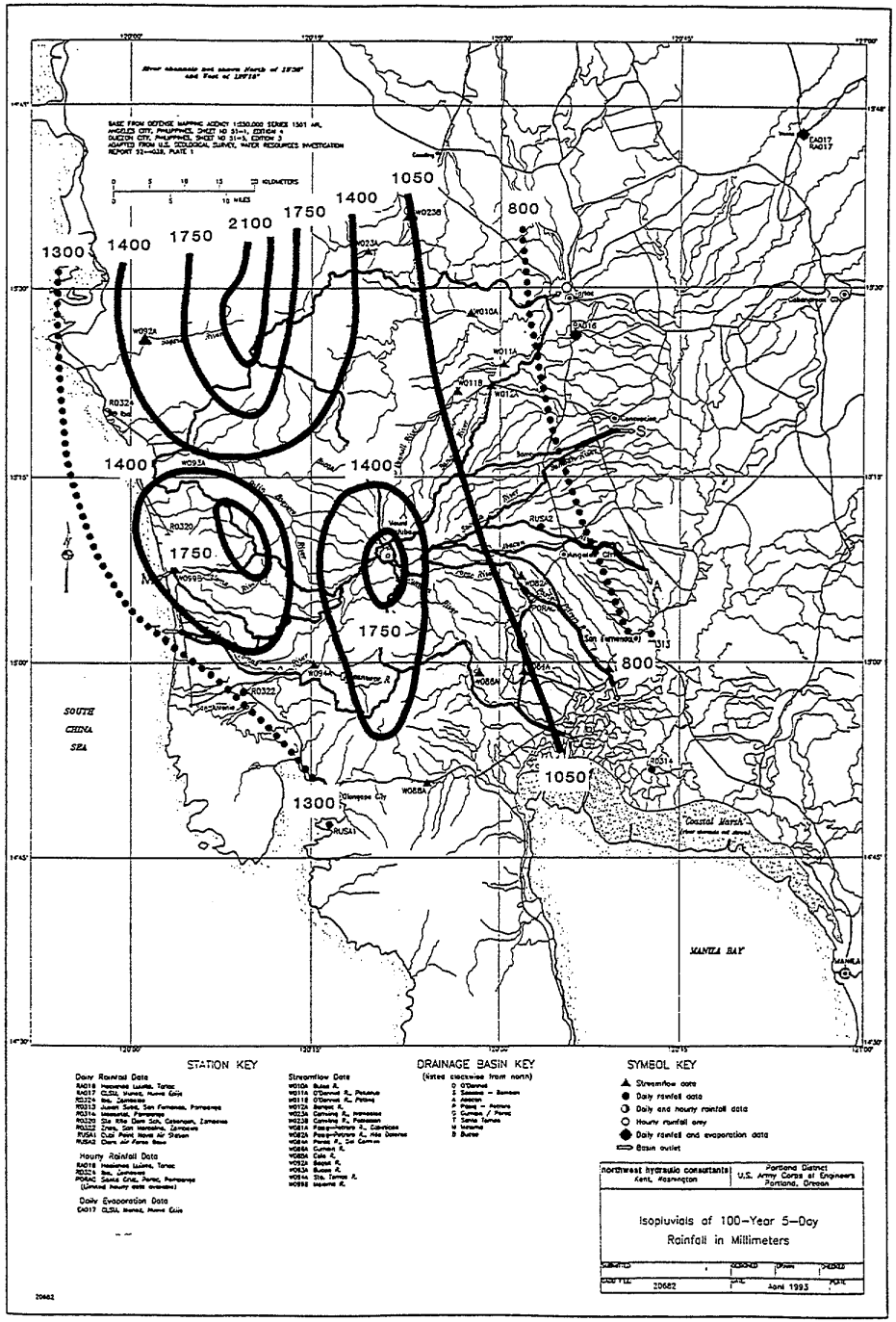


Figure 2.4.106

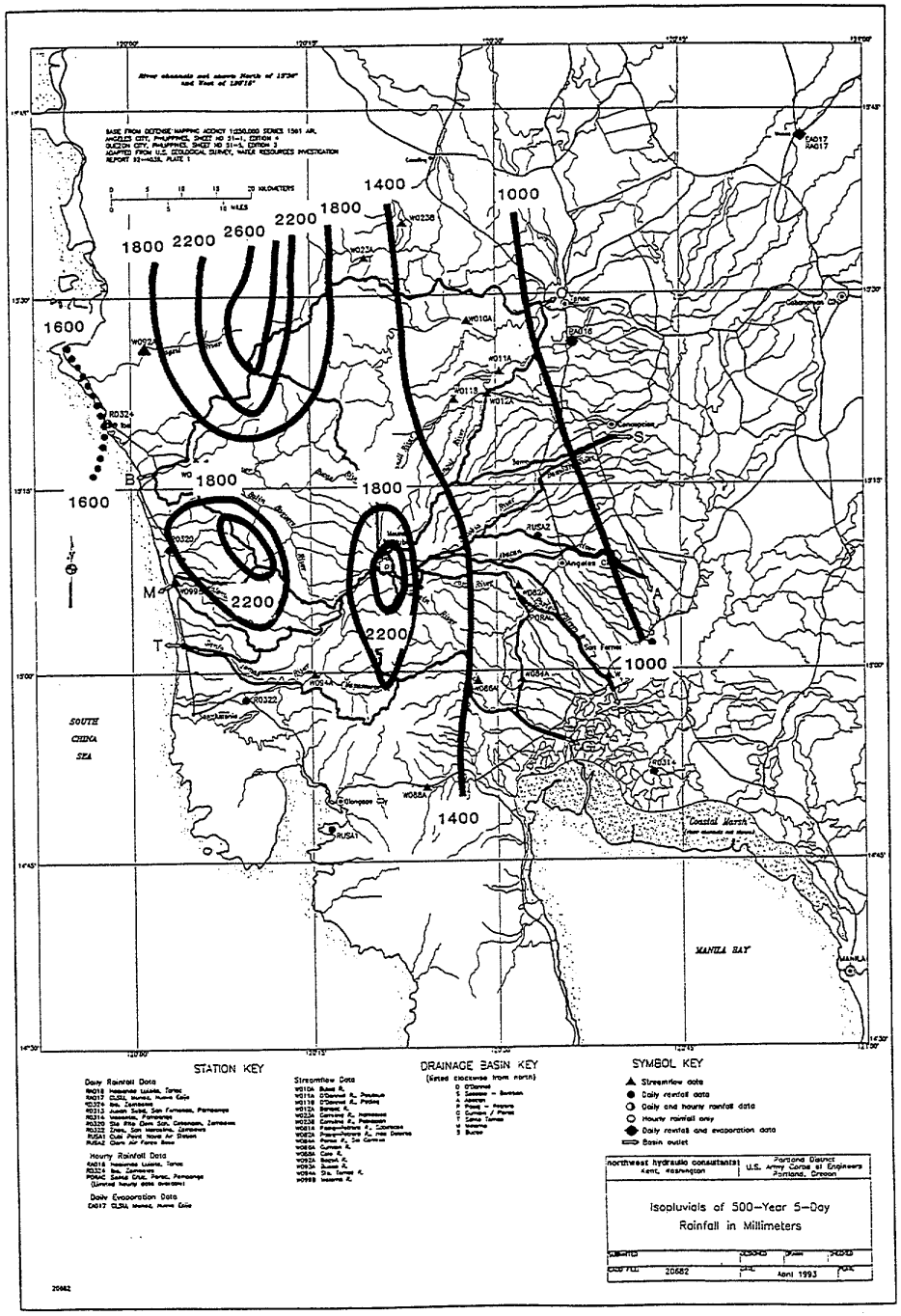
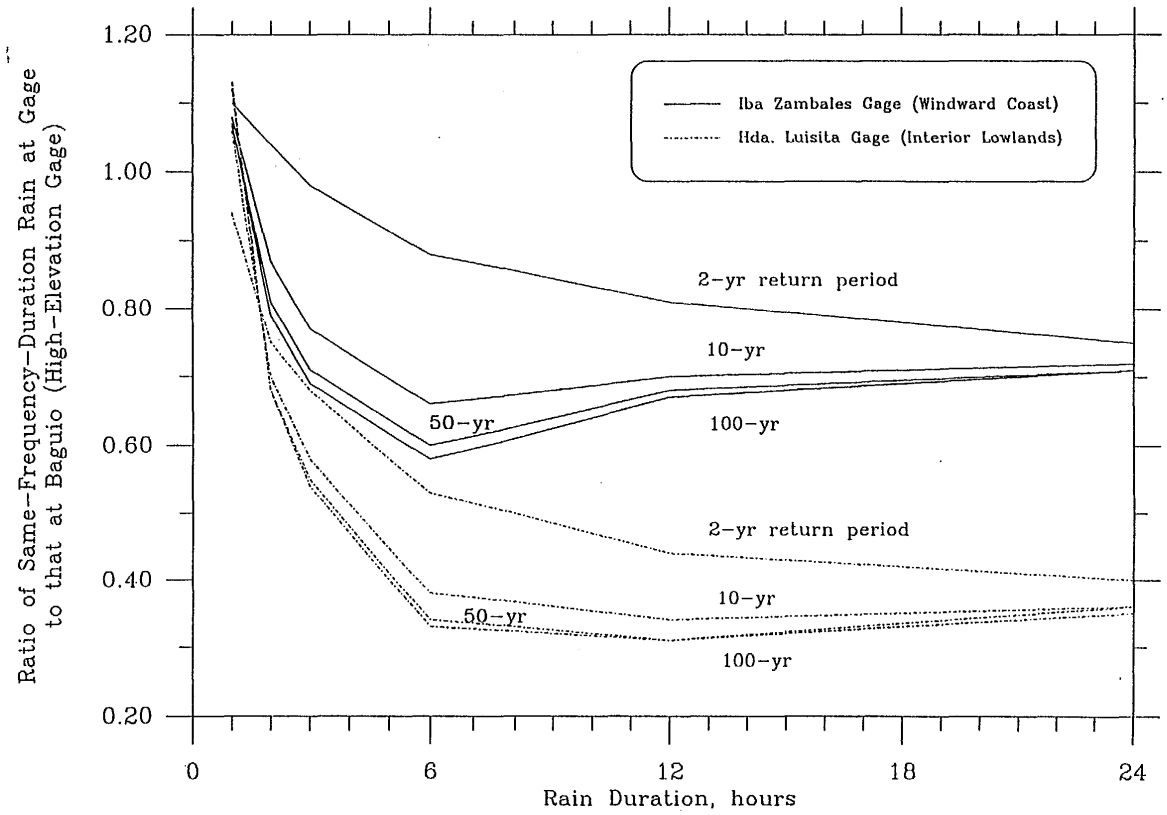


Figure 2.4.107

Effect of Elevation/Location/Return Period on
Point Rain Frequency-Duration Characteristics



NOTE: Ratios derived from Hydrology and Flood Forecast Center, PAGASA, 1981:
'Rainfall Intensity-Duration-Frequency Data of the Philippines' Volume 1, First Edition.

Figure 2.4.108

Effect of Return Period on Station-to-Station
Point Rain Frequency-Duration Characteristics

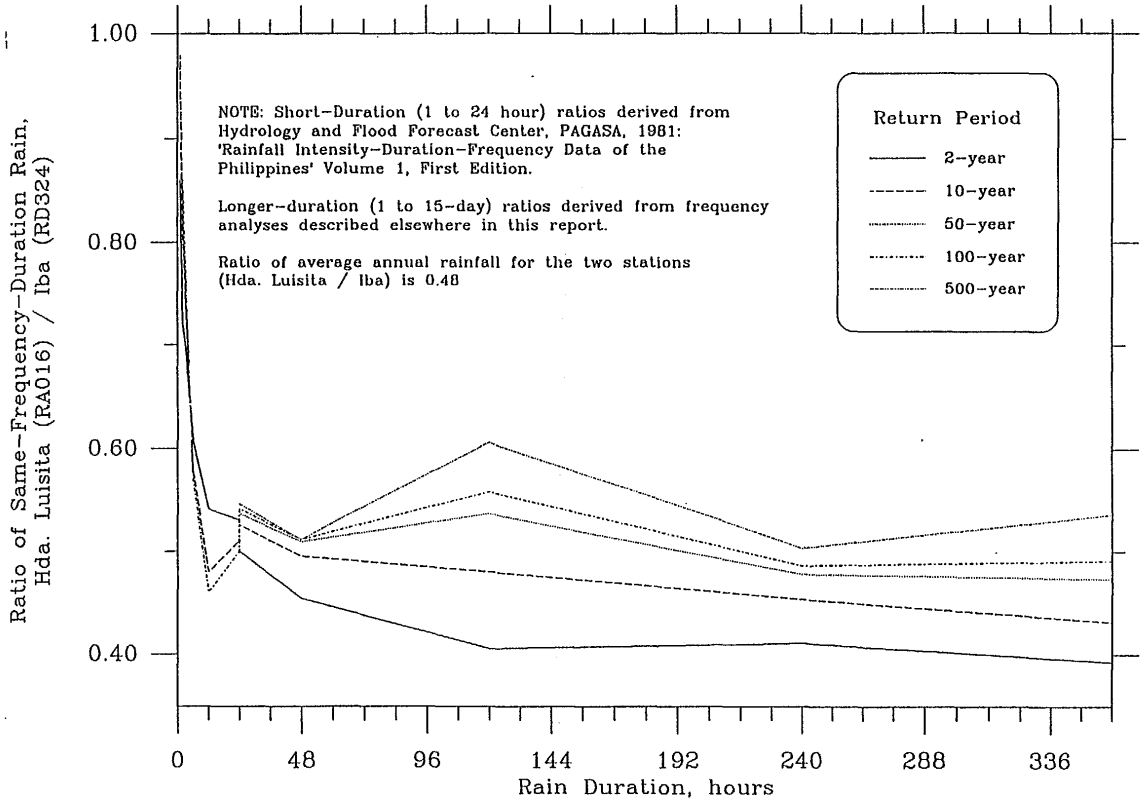


Figure 2.4.109

909

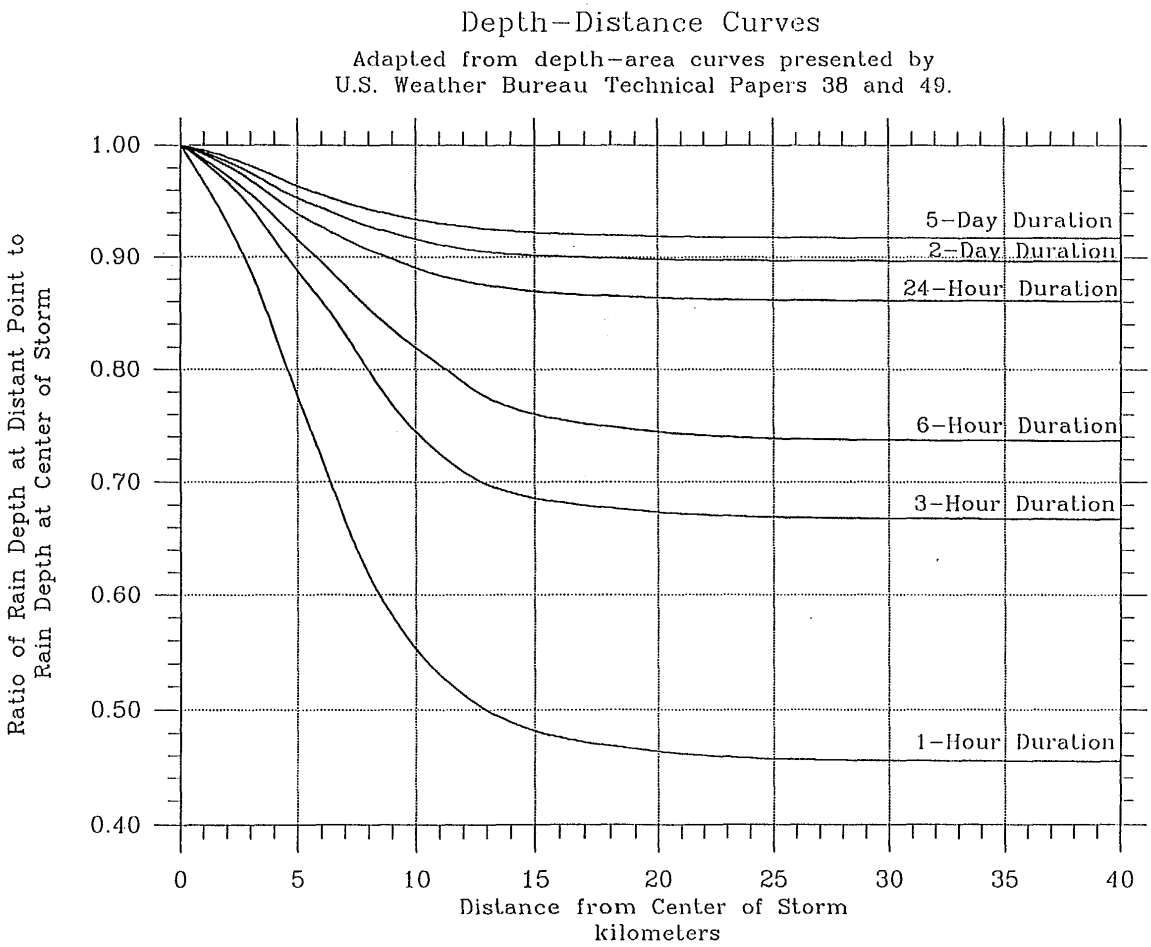


Figure 2.4.110

Historic Rainfall Event @ Iba Zambales, Station RD324
Storm of May 17 – May 29, 1976

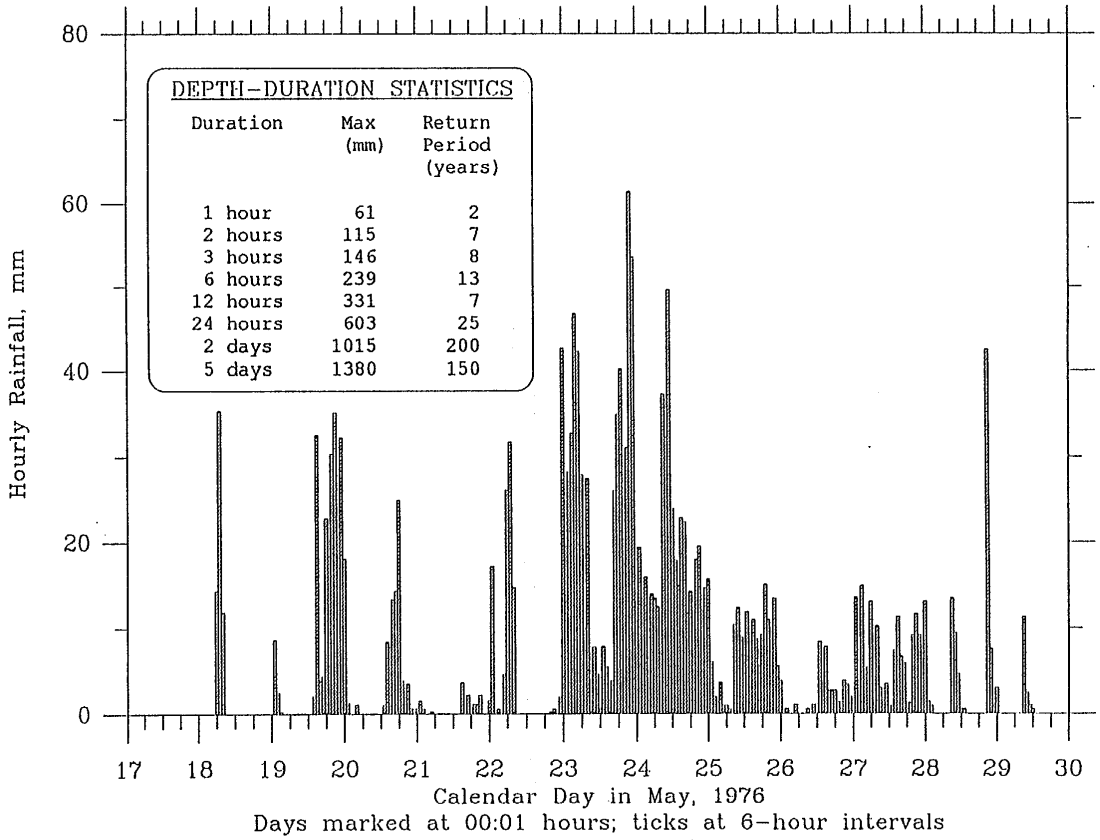


Figure 2.4.111

607

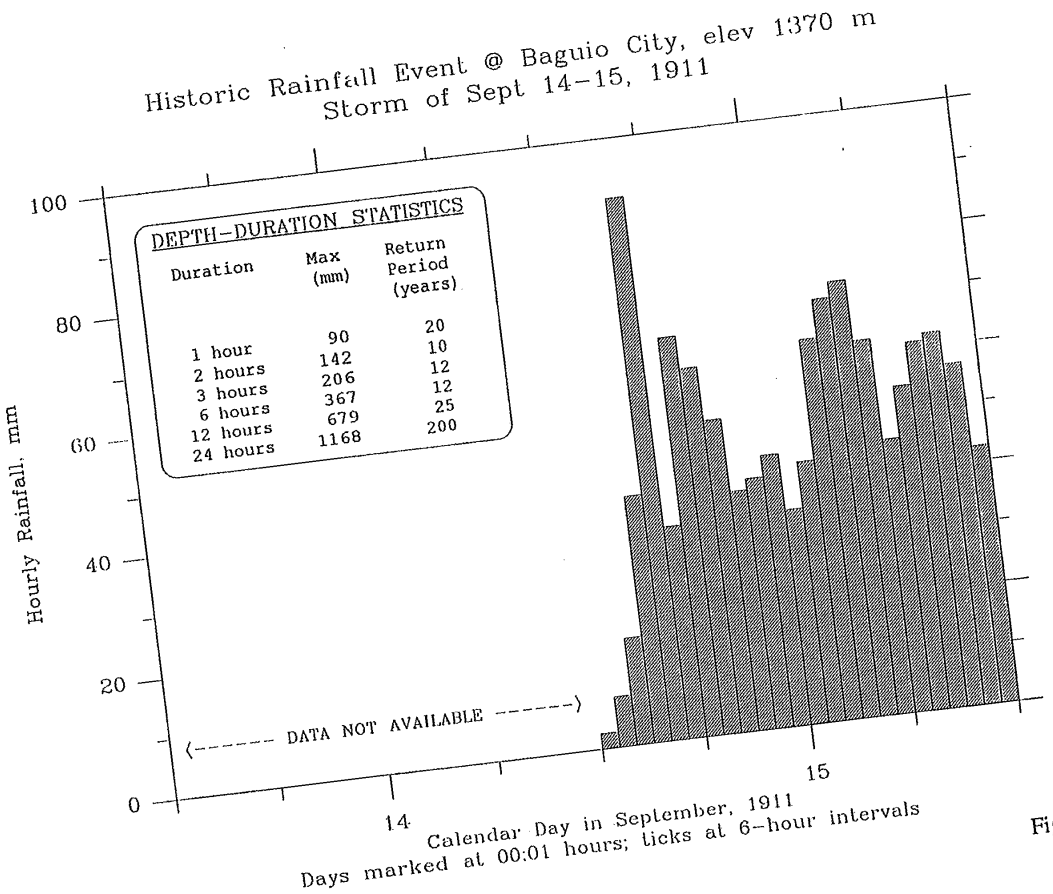


Figure 2.4.112

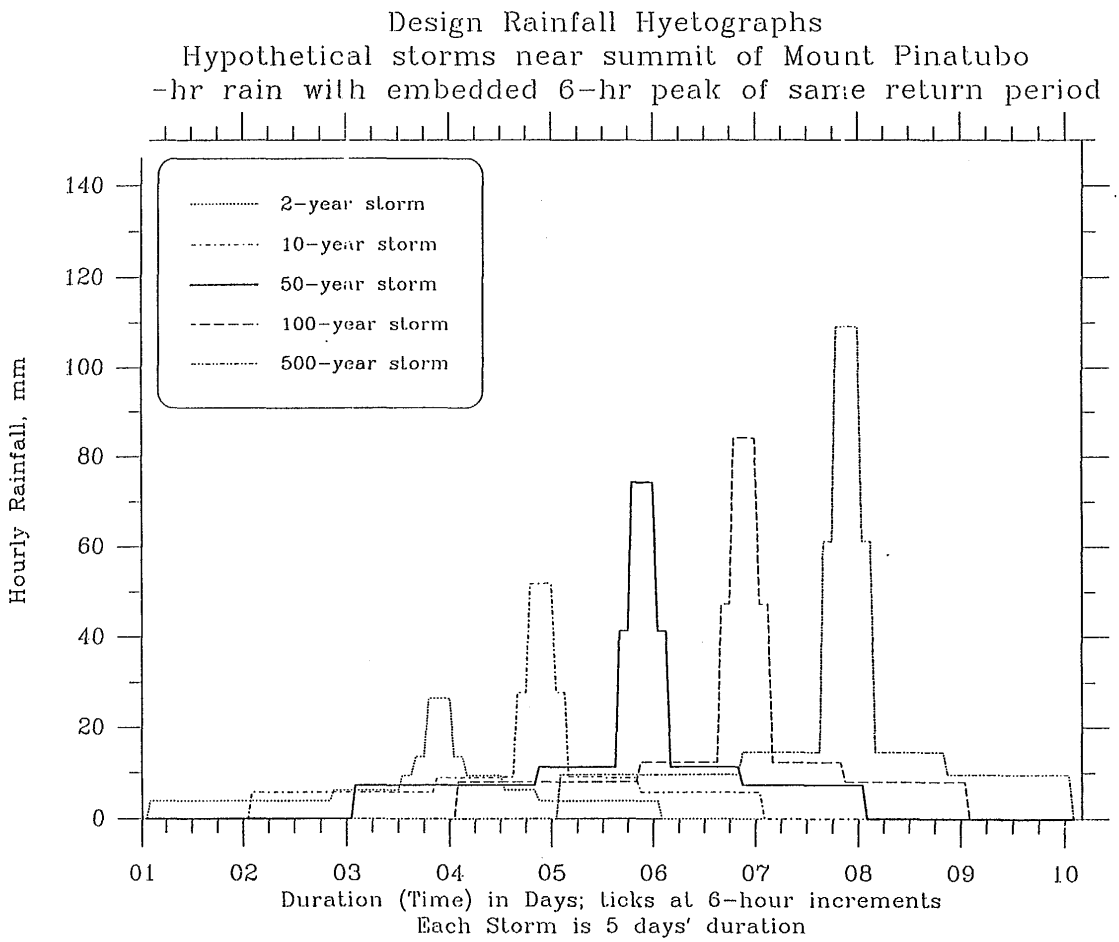


Figure 2.4.113

Design Rainfall Hyetographs
Illustration of Sub-basin Hyetograph Development

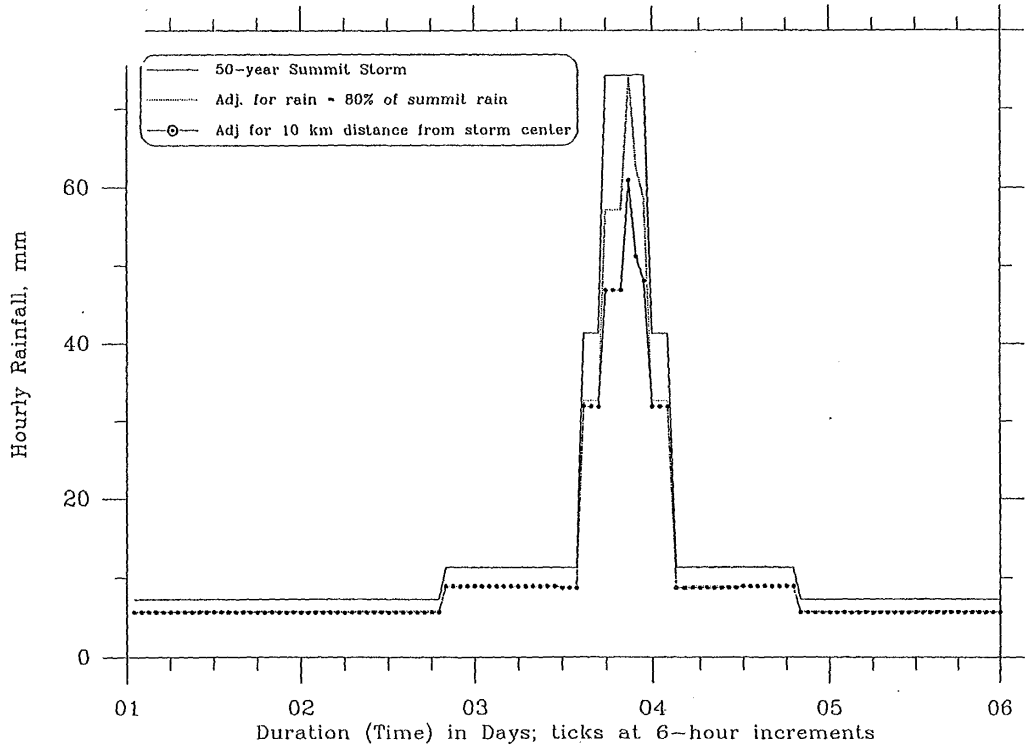


Figure 2.4.114

610

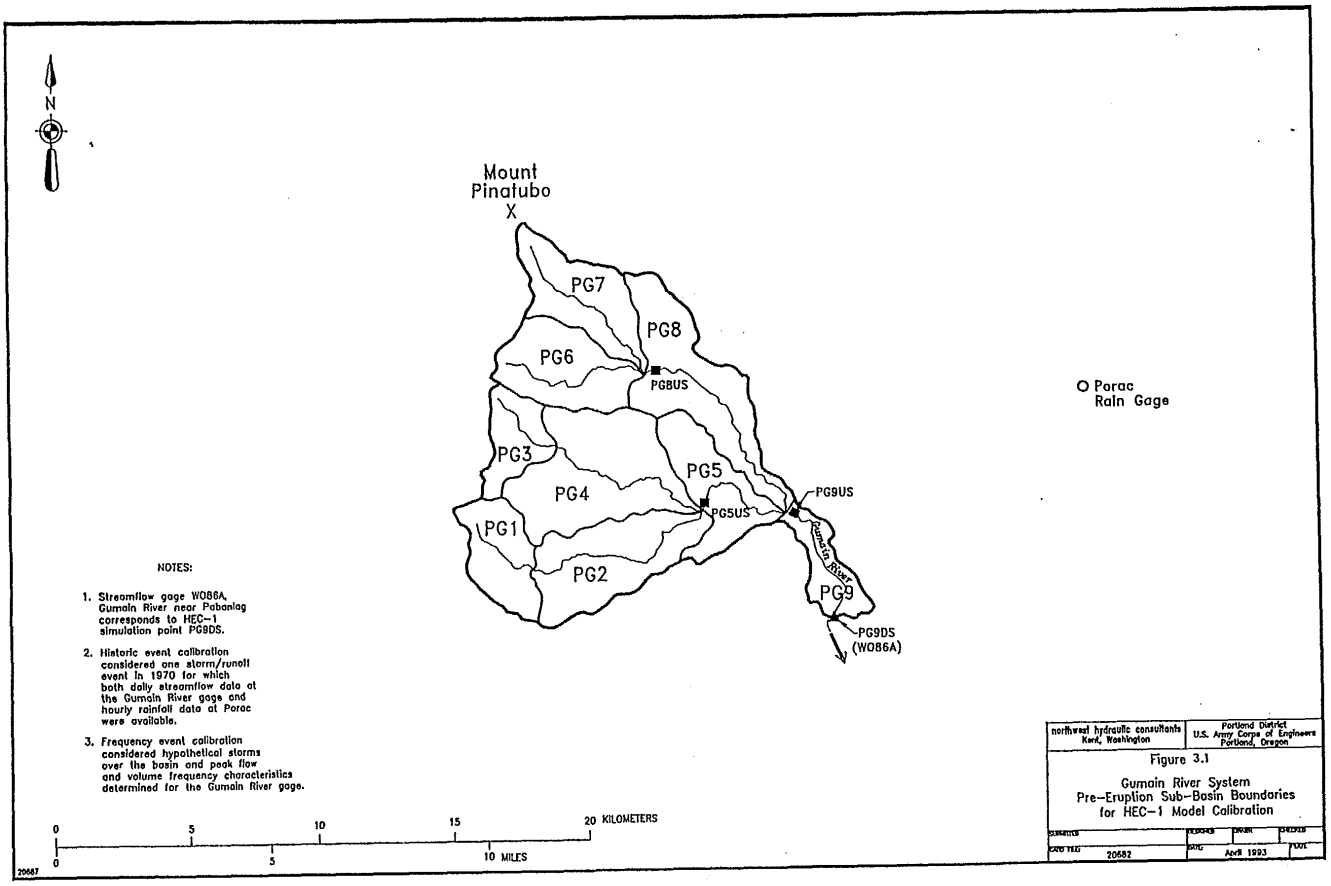

N
Mount
Pinatubo
X
PG7
PG8
PG6
PG8US
PG3
PG5
PG4
PG5US
PG9US
PG1
PG2
Gumain River
PG9
PG9DS
(W086A)
O Porac
Rain Gage
NOTES:
1. Streamflow gage W086A, Gumain River near Pabanlag corresponds to HEC-1 simulation point PG9DS.
2. Historic event calibration considered one storm/runoff event in 1970 for which both daily streamflow data at the Gumain River gage and hourly rainfall data at Porac were available.
3. Frequency event calibration considered hypothetical storms over the basin and peak flow and volume frequency characteristics determined for the Gumain River gage.
0 5 10 15 20 KILOMETERS
0 5 10 MILES
20687
northwest hydraulic consultants
Kent, Washington
Portland District
U.S. Army Corps of Engineers
Portland, Oregon
Figure 3.1
Gumain River System
Pre-Eruption Sub-Basin Boundaries
for HEC-1 Model Calibration
CAD FILE 20682
DATE April 1993

Figure 3.2.1

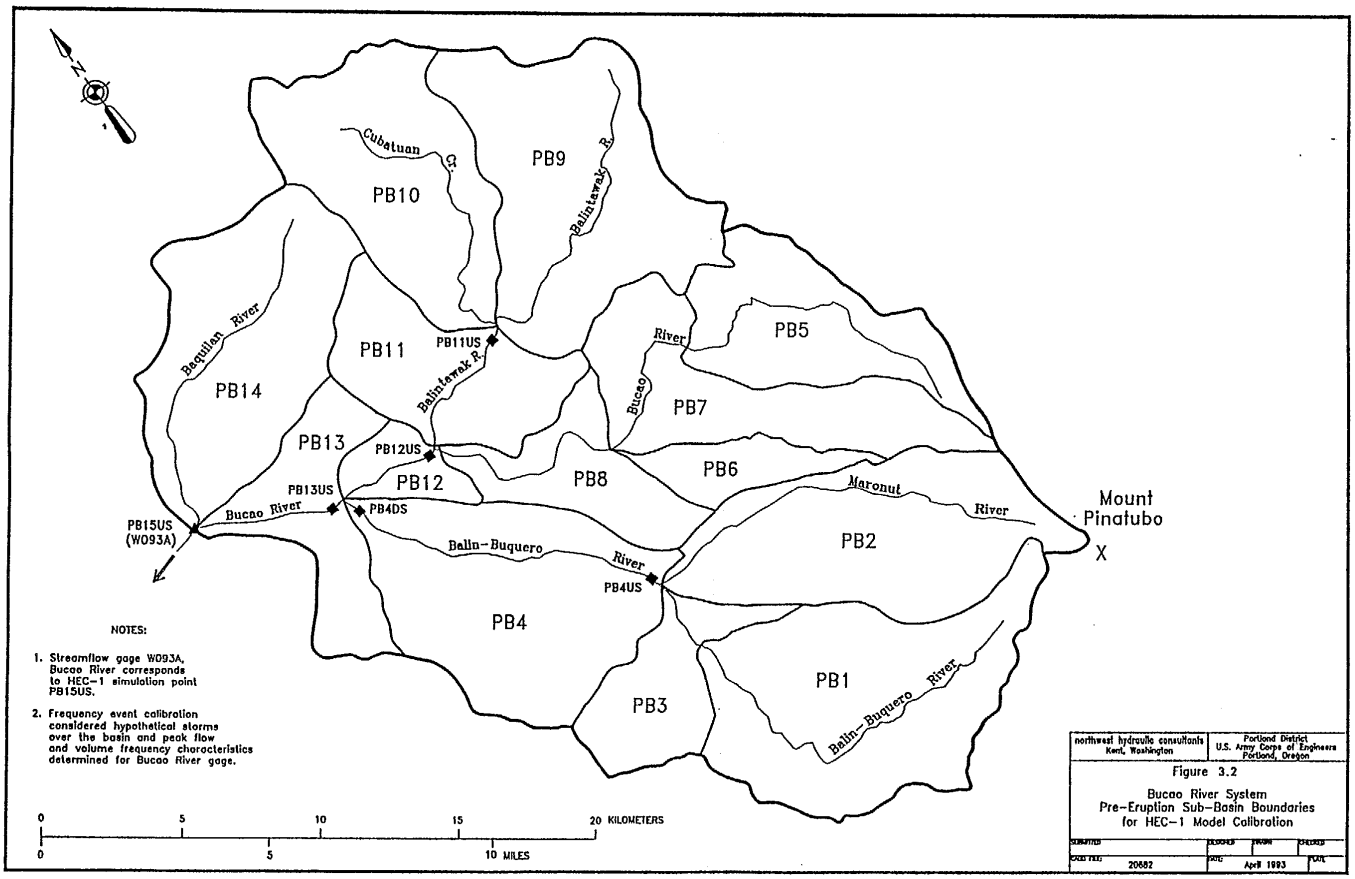

Cubatuan Cr.
PB10
PB9
Balintawak R.
Baquilan River
PB14
PB11
PB11US
Balintawak R.
River
PB5
Bucao
PB7
PB13
PB12US
PB12
PB8
PB6
Maronut
River
PB13US
PB4DS
Bucao River
PB15US
(W093A)
Balin-Buquero
River
PB4US
PB2
Mount
Pinatubo
X
PB4
PB3
PB1
Balin-Buquero River
NOTES:
1. Streamflow gage W093A, Bucao River corresponds to HEC-1 simulation point PB15US.
2. Frequency event calibration considered hypothetical storms over the basin and peak flow and volume frequency characteristics determined for Bucao River gage.
0 5 10 15 20 KILOMETERS
0 5 10 MILES
northwest hydraulic consultants
Kent, Washington
Portland District
U.S. Army Corps of Engineers
Portland, Oregon
Figure 3.2
Bucao River System
Pre-Eruption Sub-Basin Boundaries
for HEC-1 Model Calibration
20682
April 1993

Figure 3.2.2

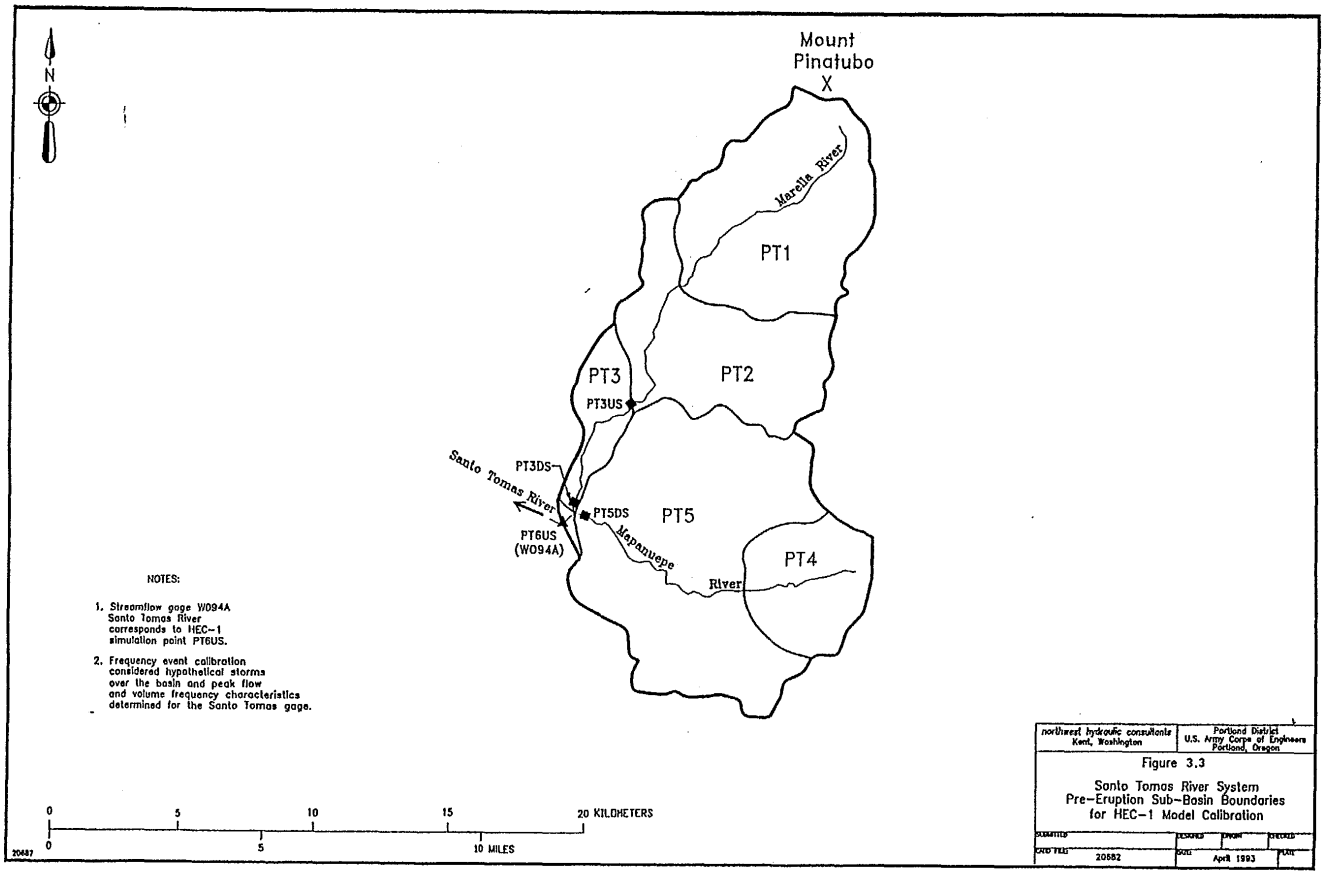

N
Mount
Pinatubo
X
Marella River
PT1
PT2
PT3
PT3US
PT3DS
Santo Tomas River
PT5DS
PT5
PT6US
(W094A)
Mapanuepe
River
PT4
NOTES:
1. Streamflow gage W094A Santo Tomas River corresponds to HEC-1 simulation point PT6US.
2. Frequency event calibration considered hypothetical storms over the basin and peak flow and volume frequency characteristics determined for the Santo Tomas gage.
0
5
10
15
20 KILOMETERS
0
5
10 MILES
northwest hydraulic consultants
Kent, Washington
Portland District
U.S. Army Corps of Engineers
Portland, Oregon
Figure 3.3
Santo Tomas River System
Pre-Eruption Sub-Basin Boundaries
for HEC-1 Model Calibration
20582
April 1993

Figure 3.2.3

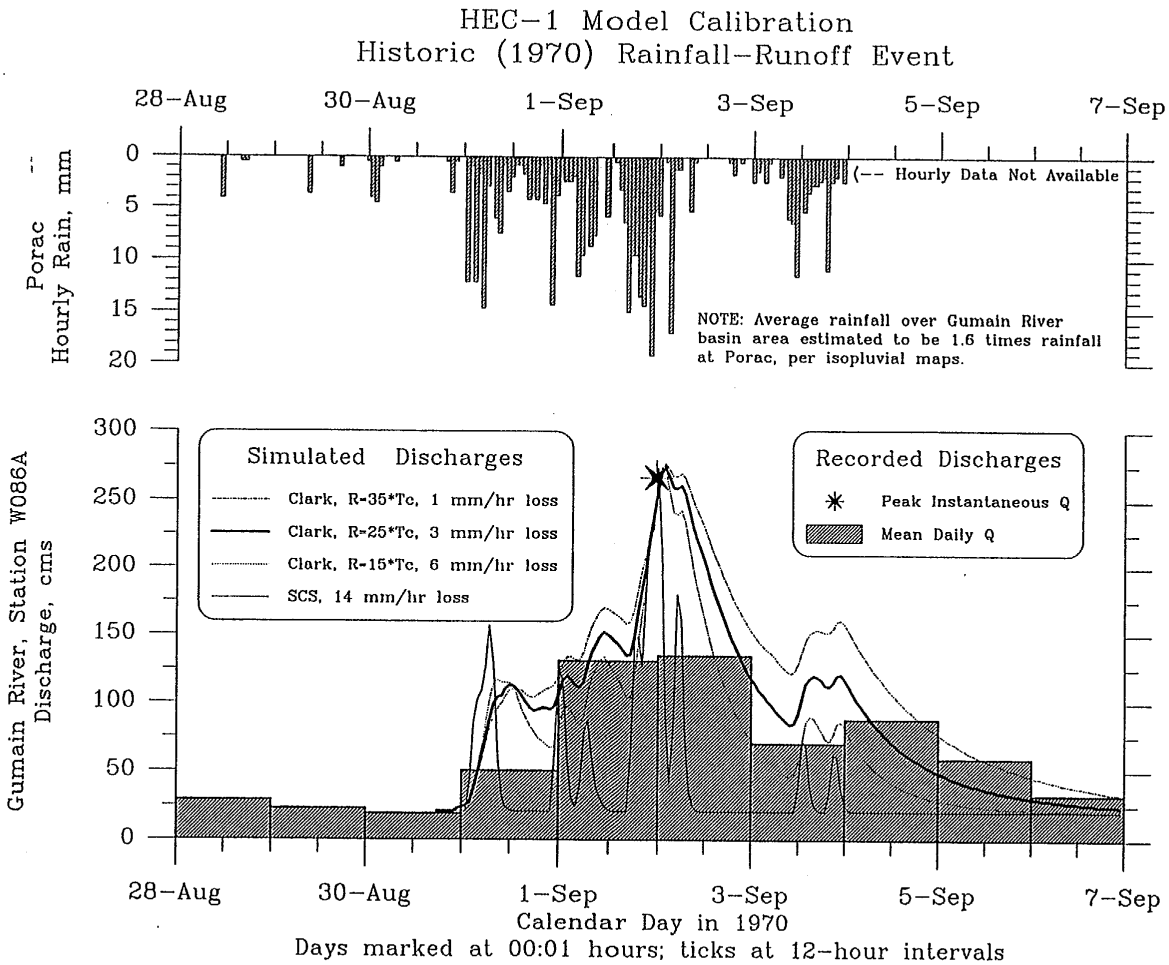
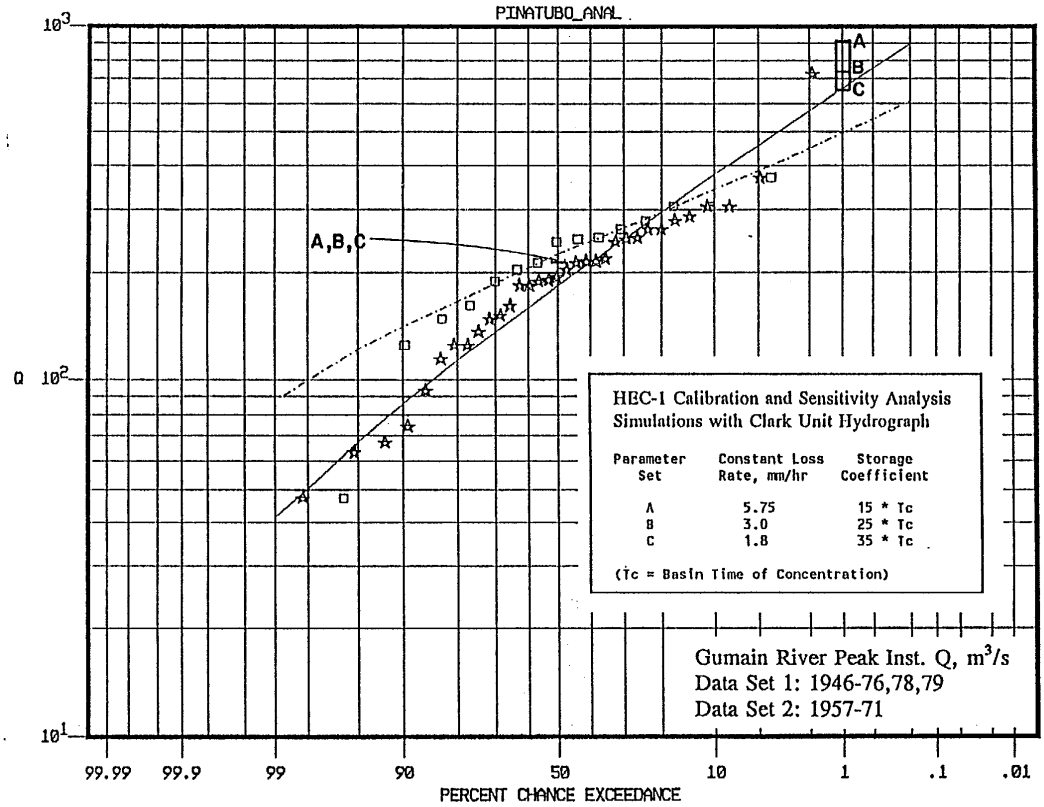


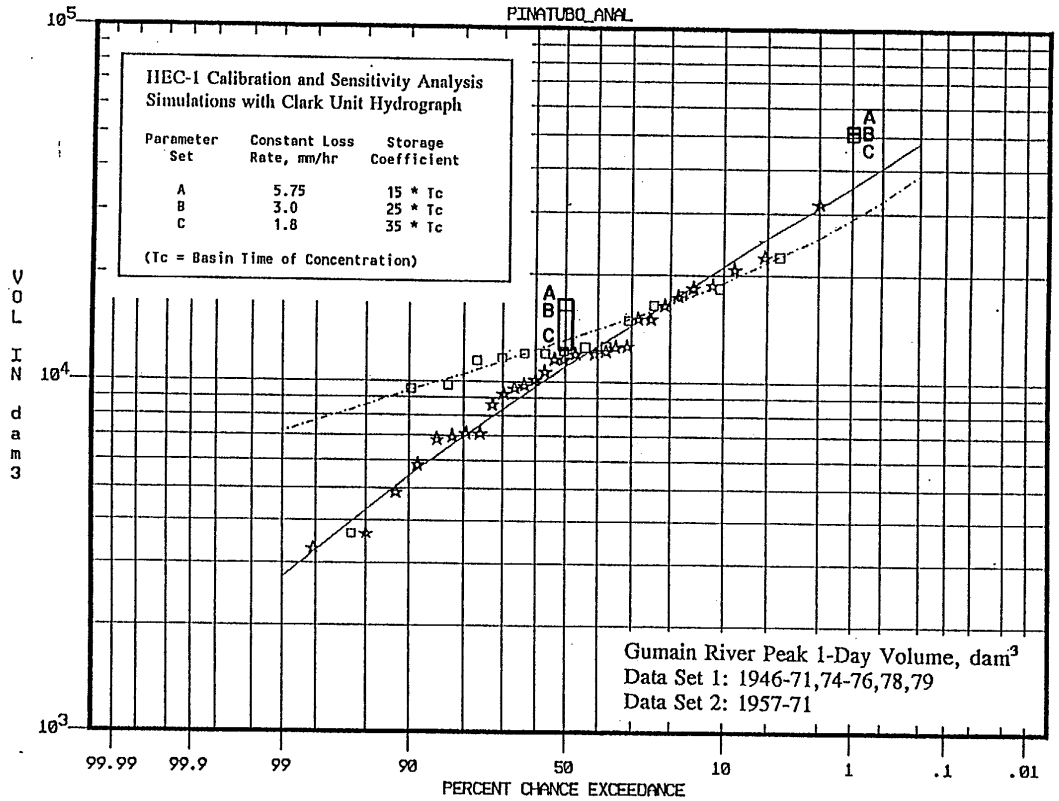
Figure 3.2.4

614



☆ W086A MAX EVENTS PEAK INST W. POST-71
——— W086A MAX ANALYTICAL EXP PROB PEAK INST W. POST-71
□ W086A MAX EVENTS PEAK INST Q EXCL POST-71
-·-·-·- W086A MAX ANALYTICAL EXP PROB PEAK INST Q EXCL POST-71

Figure 3.2.5



☆ W086A MAX EVENTS 01-DAY DUR W. POST-71

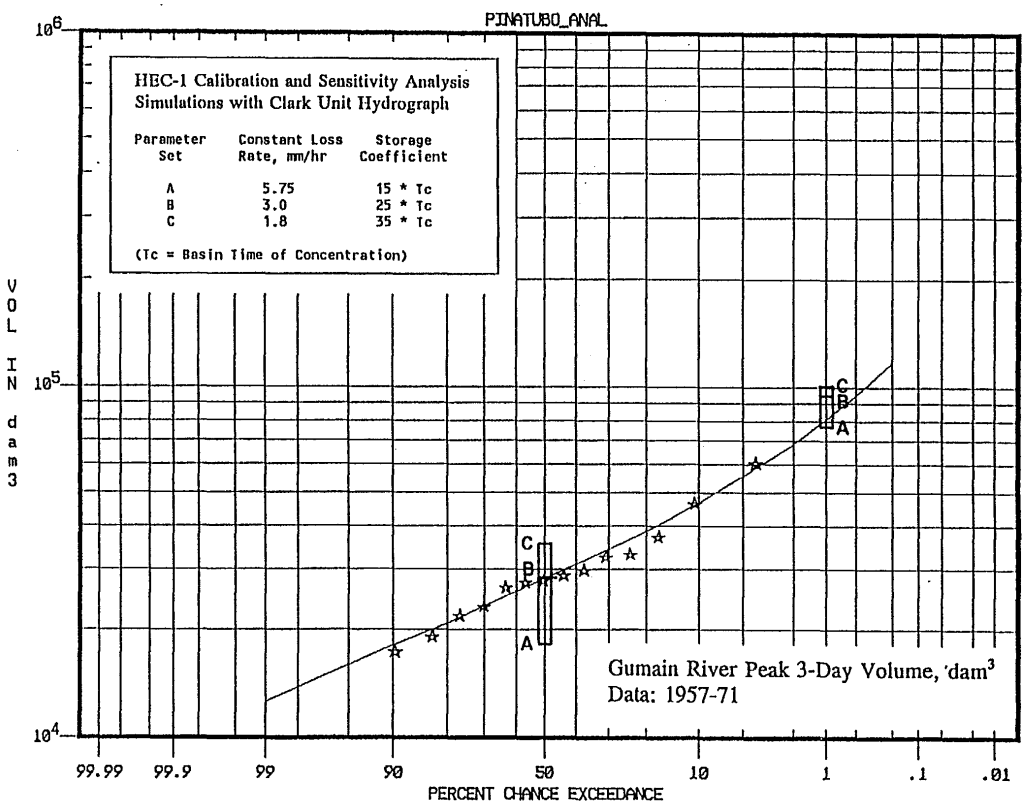
W086A MAX ANALYTICAL EXP PROB 01-DAY DUR W. POST-71

□ W086A MAX EVENTS 01-DAY DUR

W086A MAX ANALYTICAL EXP PROB 01-DAY DUR

Figure 3.2.6

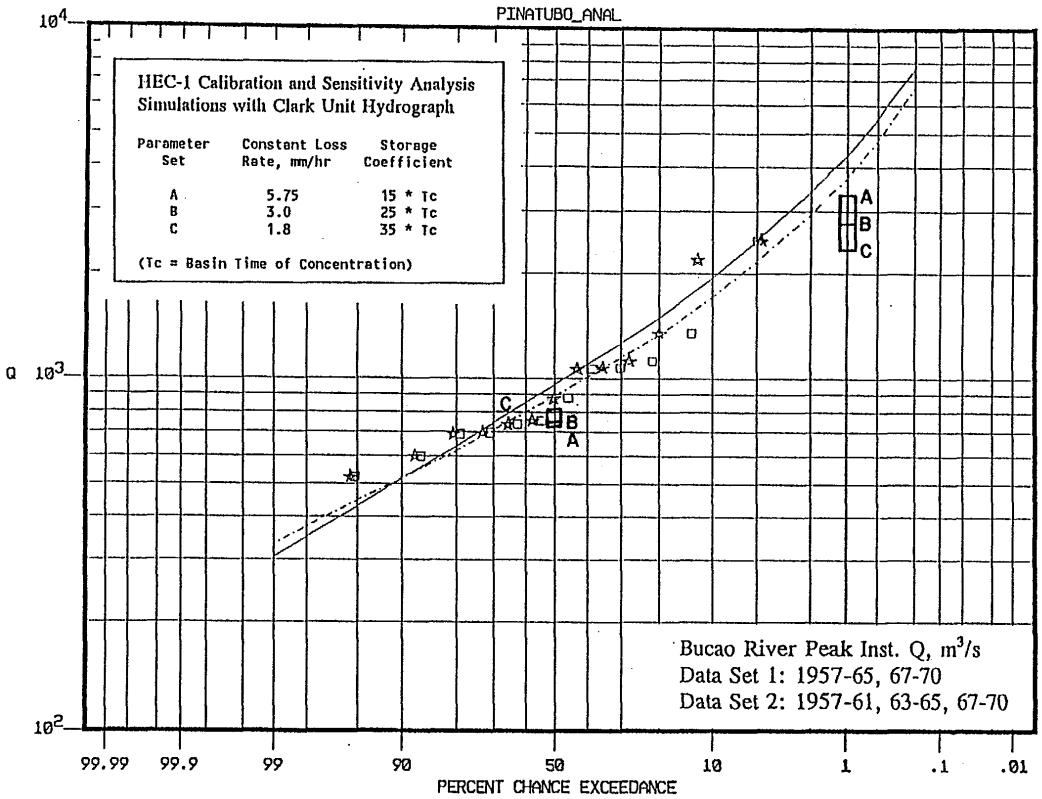
616



☆ W086A MAX EVENTS 03-DAY DUR
——— W086A MAX ANALYTICAL EXP PROB 03-DAY DUR

Figure 3.2.7

617



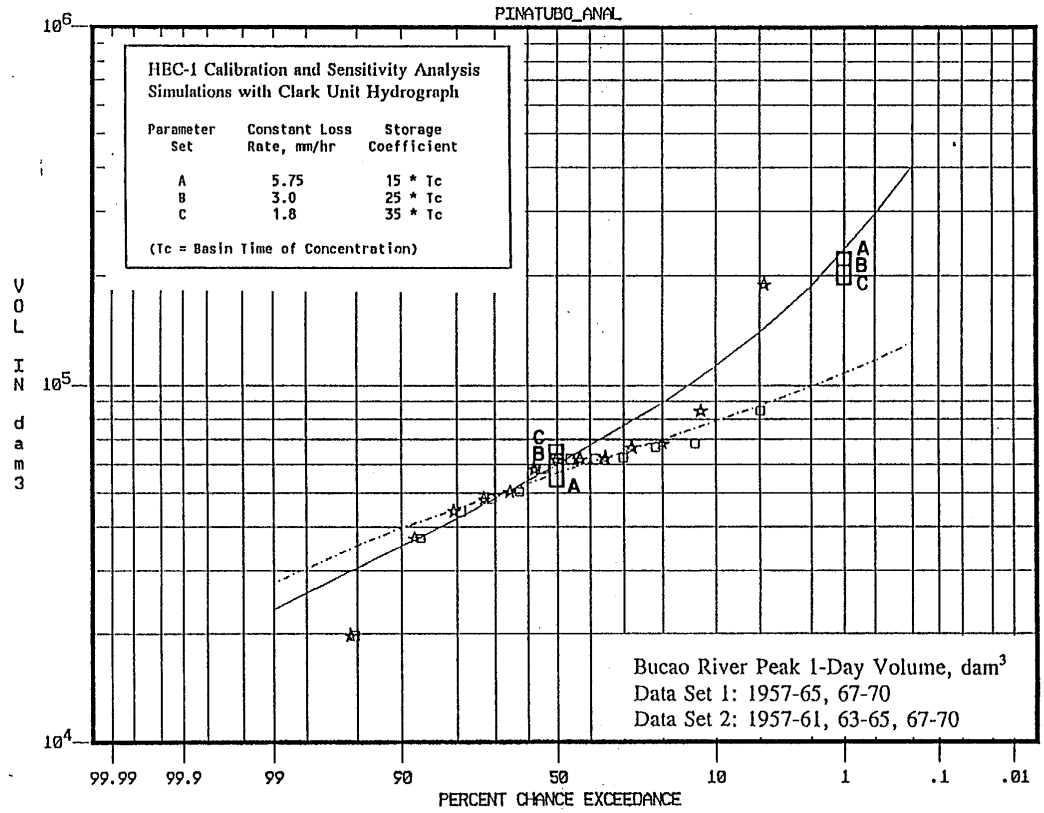
☆ W093A MAX EVENTS PEAK INST Q

——— W093A MAX ANALYTICAL EXP PROB PEAK INST Q

□ W093A MAX EVENTS PEAK INST Q EXCL 1962

-·-·- W093A MAX ANALYTICAL EXP PROB PEAK INST Q EXCL 1962

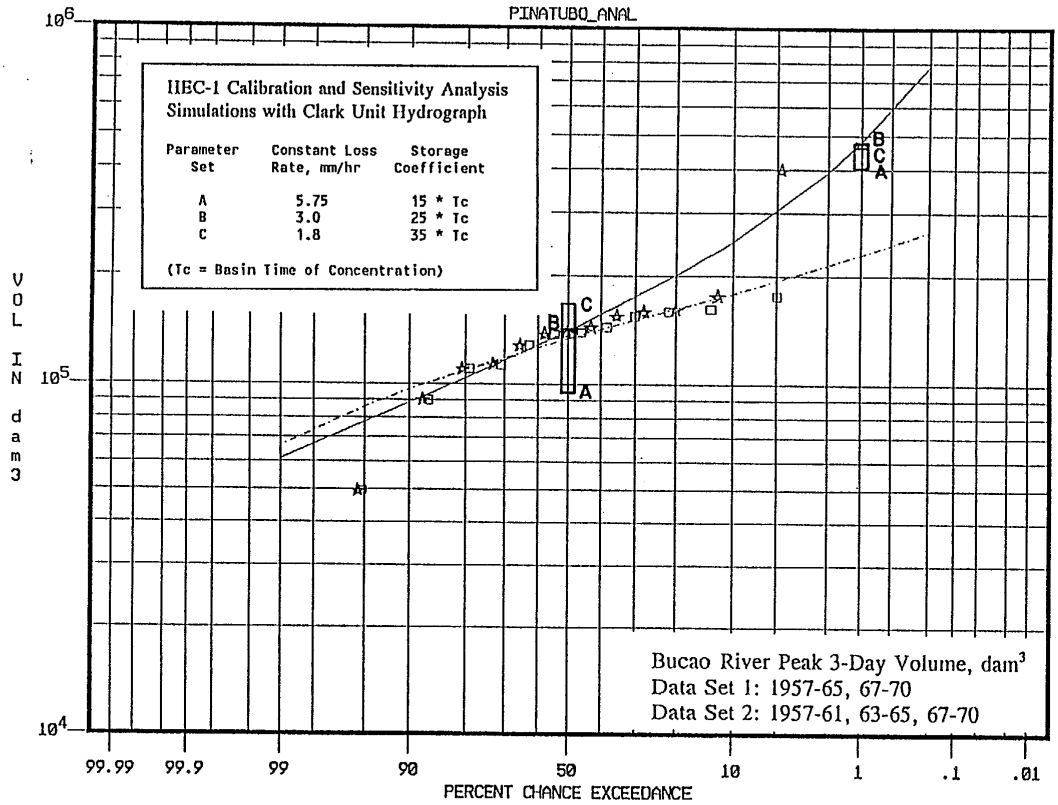
Figure 3.2.8



☆ W093A MAX EVENTS 01-DAY DUR
——— W093A MAX ANALYTICAL EXP PROB 01-DAY DUR
□ W093A MAX EVENTS 01-DAY DUR EXCL 1962
-·-·-·- W093A MAX ANALYTICAL EXP PROB 01-DAY DUR EXCL 1962

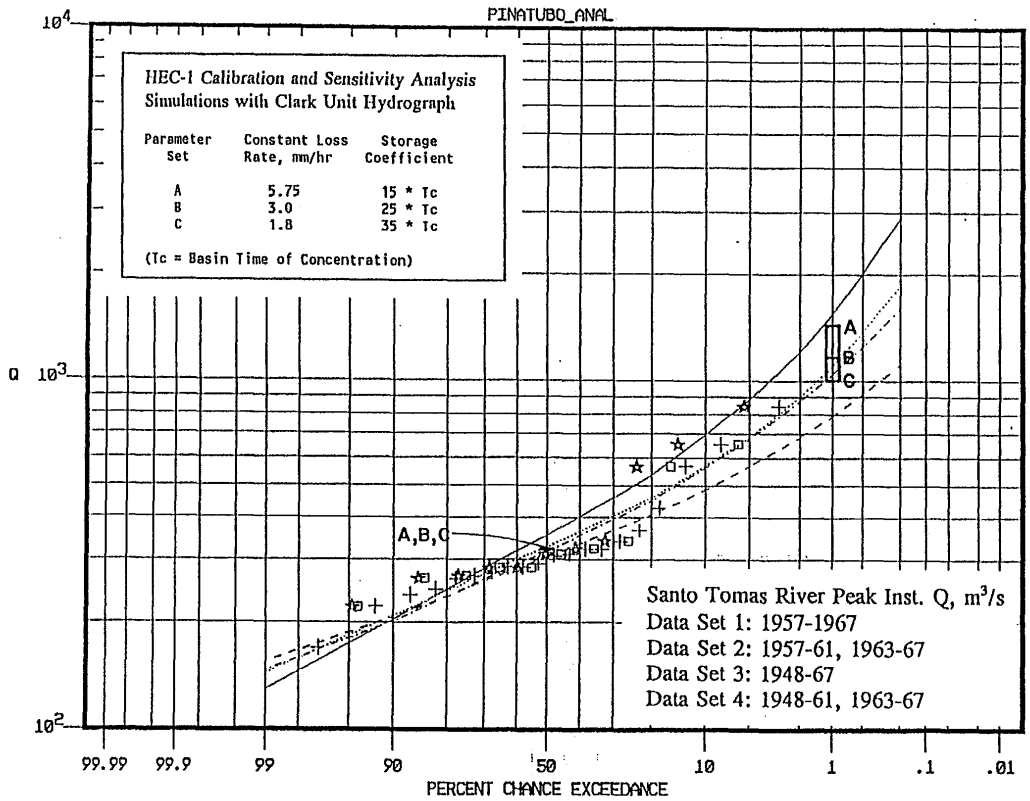
Figure 3.2.9

619



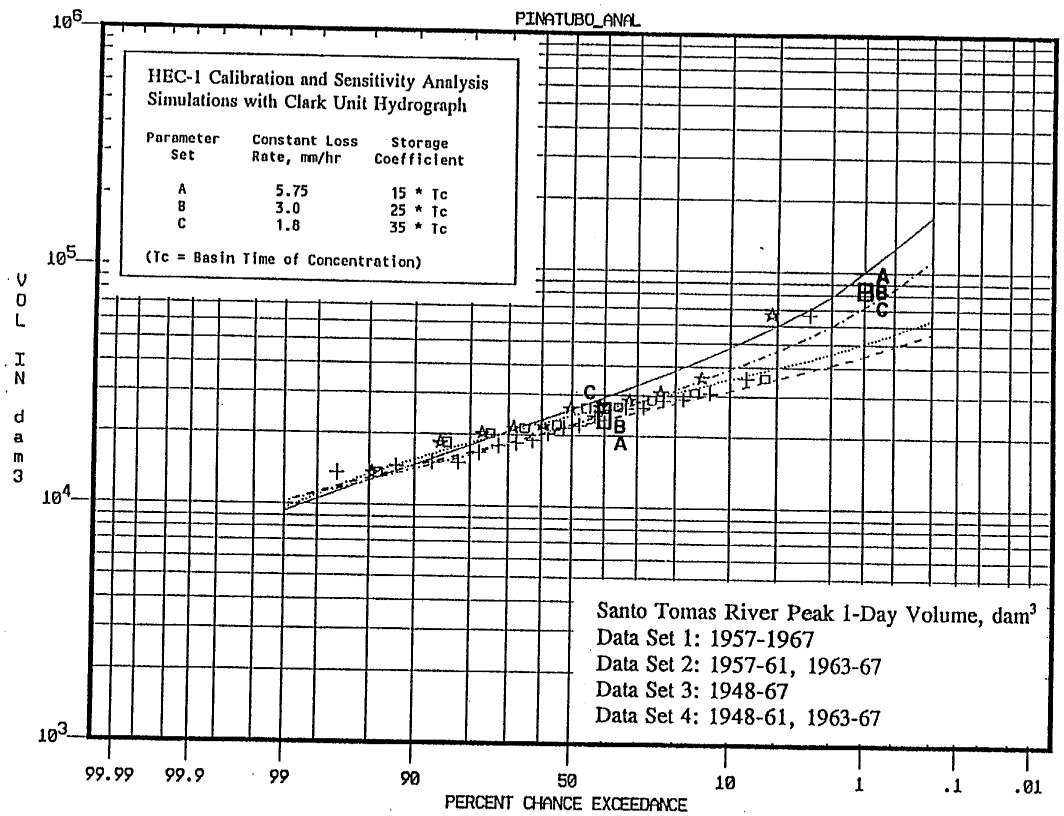
☆ W093A MAX EVENTS 03-DAY DUR
——— W093A MAX ANALYTICAL EXP PROB 03-DAY DUR
□ W093A MAX EVENTS 03-DAY DUR EXCL 1962
-·-·-·- W093A MAX ANALYTICAL EXP PROB 03-DAY DUR EXCL 1962

Figure 3.2.10



☆ W094A MAX EVENTS PEAK INST Q 57-67
——— W094A MAX ANALYTICAL EXP PROB PEAK INST Q 57-67
□ W094A MAX EVENTS PEAK INST Q 57-67 EXCL 62
W094A MAX ANALYTICAL EXP PROB PEAK INST Q 57-67 EXCL 62
+ W094A MAX EVENTS PEAK INST Q 48-67
W094A MAX ANALYTICAL EXP PROB PEAK INST Q 48-67
W094A MAX ANALYTICAL EXP PROB PEAK INST Q 48-67 EXCL 62

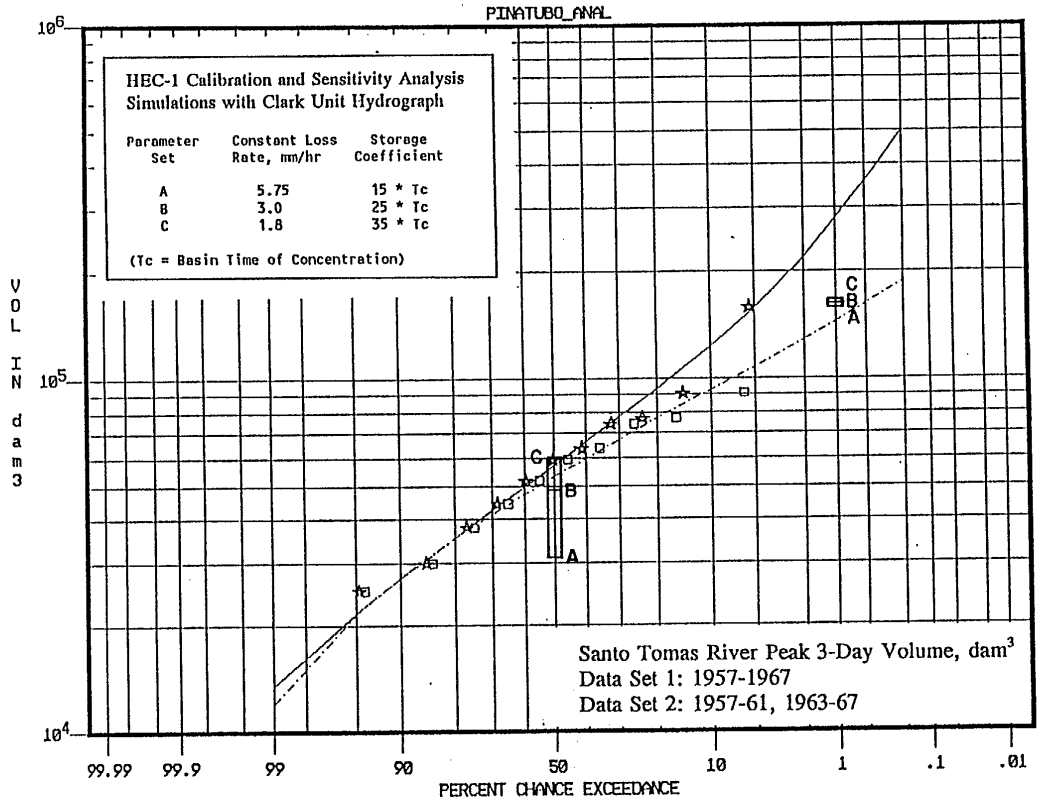
Figure 3.2.11



☆ W094A MAX EVENTS 01-DAY DUR
W094A MAX ANALYTICAL EXP PROB 01-DAY DUR
□ W094A MAX EVENTS 01-DAY DUR EXCL 1962
W094A MAX ANALYTICAL EXP PROB 01-DAY DUR EXCL 1962

+ W094A MAX EVENTS 01-DAY DUR 48-67
W094A MAX ANALYTICAL EXP PROB 01-DAY DUR 48-67
W094A MAX ANALYTICAL EXP PROB 01-DAY DUR 48-67 EXCL 62

Figure 3.2.12



W094A MAX EVENTS 03-DAY DUR
W094A MAX ANALYTICAL EXP PROB 03-DAY DUR
W094A MAX EVENTS 03-DAY DUR EXCL 1962
W094A MAX ANALYTICAL EXP PROB 03-DAY DUR EXCL 1962

Figure 3.2.13

523

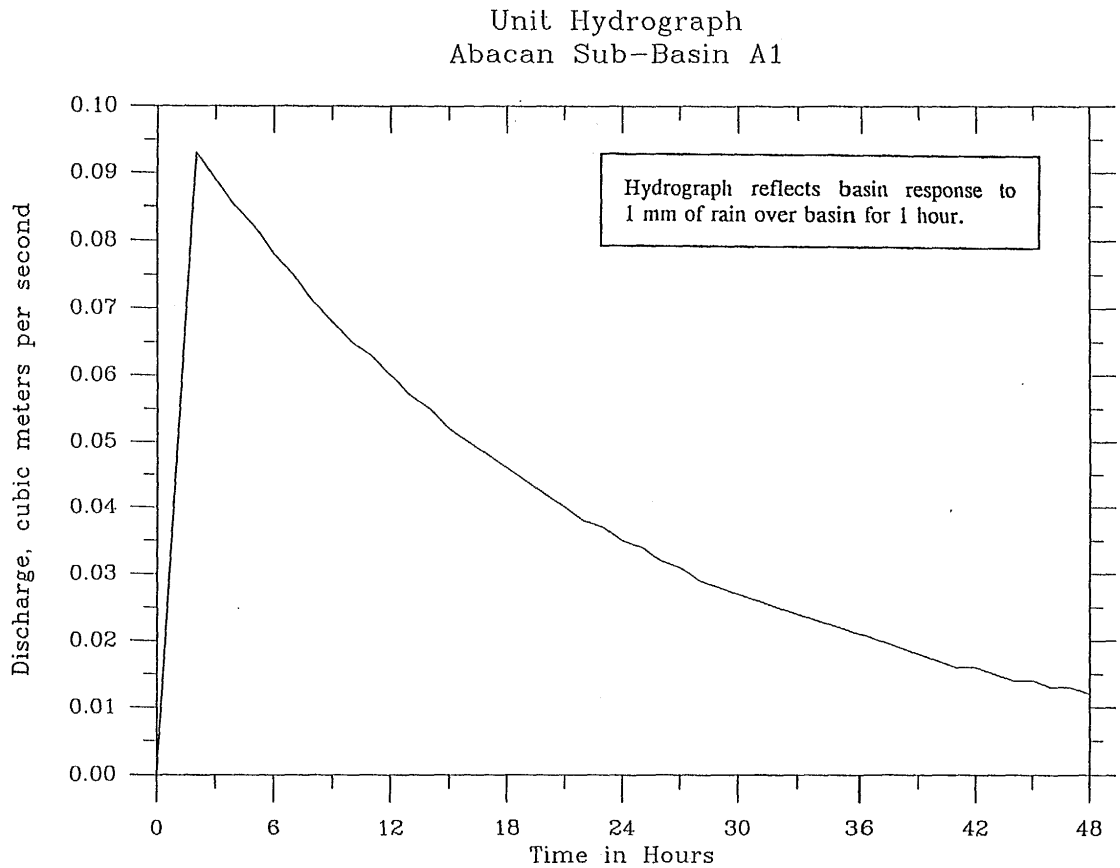

Unit Hydrograph
Abacan Sub-Basin A1
Hydrograph reflects basin response to 1 mm of rain over basin for 1 hour.
Discharge, cubic meters per second
0.10
0.09
0.08
0.07
0.06
0.05
0.04
0.03
0.02
0.01
0.00
0
6
12
18
24
30
36
42
48
Time in Hours

Figure 3.5.1

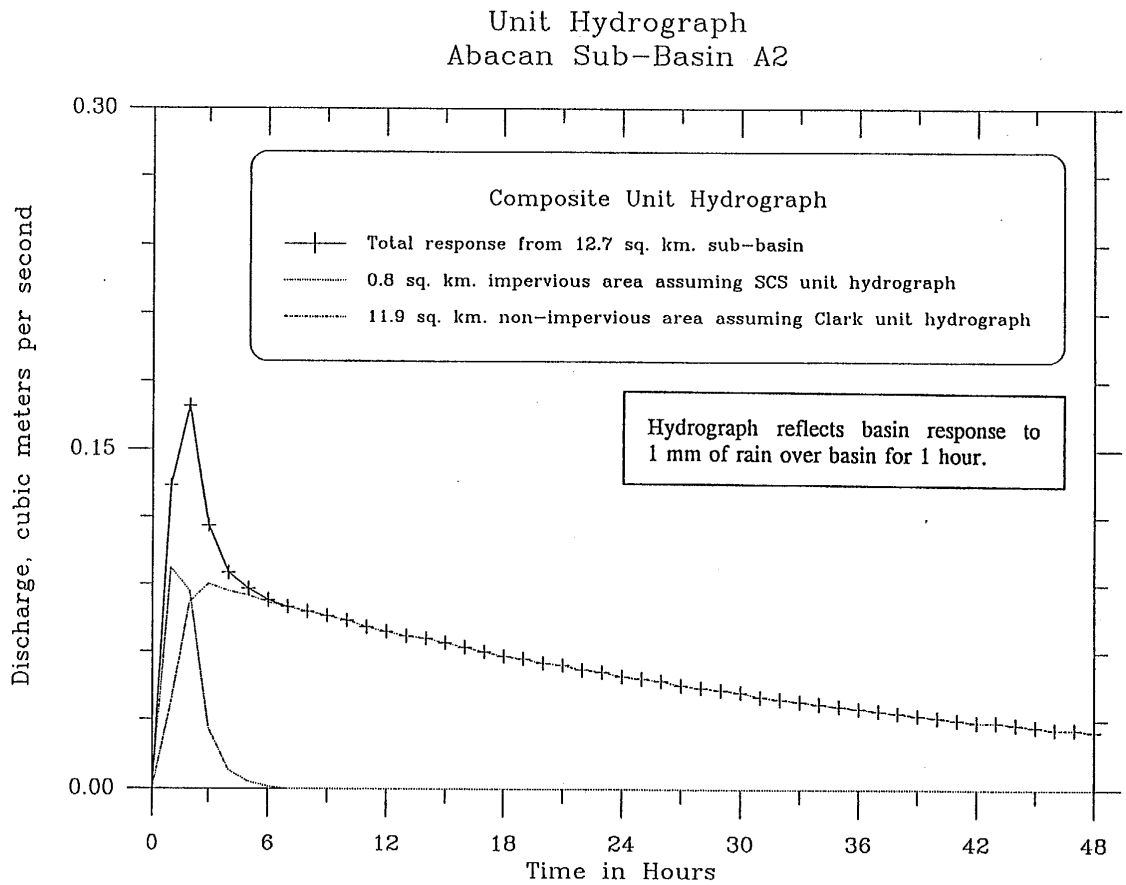

Unit Hydrograph
Abacan Sub-Basin A2
Composite Unit Hydrograph
Total response from 12.7 sq. km. sub-basin
0.8 sq. km. impervious area assuming SCS unit hydrograph
11.9 sq. km. non-impervious area assuming Clark unit hydrograph
Hydrograph reflects basin response to 1 mm of rain over basin for 1 hour.
Discharge, cubic meters per second
0.30
0.15
0.00
0
6
12
18
24
30
36
42
48
Time in Hours

Figure 3.5.2

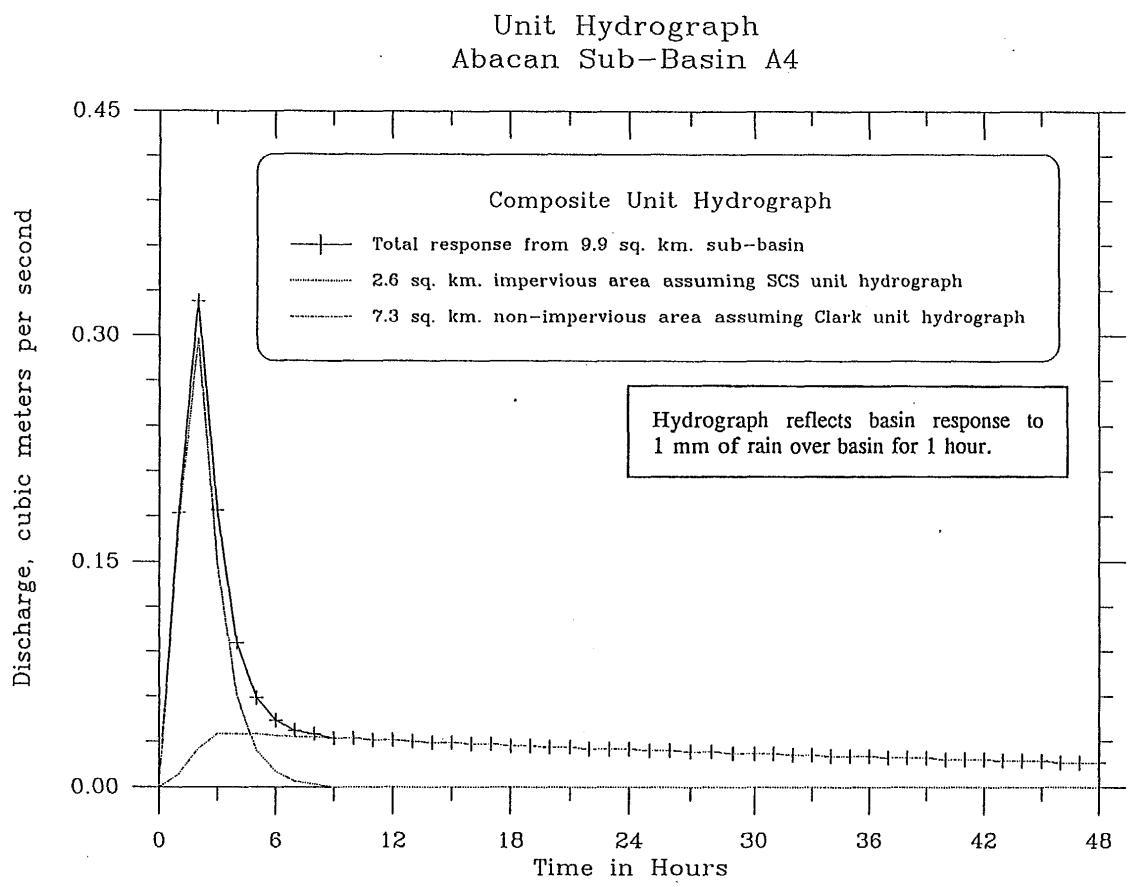

Unit Hydrograph
Abacan Sub-Basin A4
Composite Unit Hydrograph
Total response from 9.9 sq. km. sub-basin
2.6 sq. km. impervious area assuming SCS unit hydrograph
7.3 sq. km. non-impervious area assuming Clark unit hydrograph
Hydrograph reflects basin response to 1 mm of rain over basin for 1 hour.
Discharge, cubic meters per second
0.45
0.30
0.15
0.00
0
6
12
18
24
30
36
42
48
Time in Hours

Figure 3.5.3

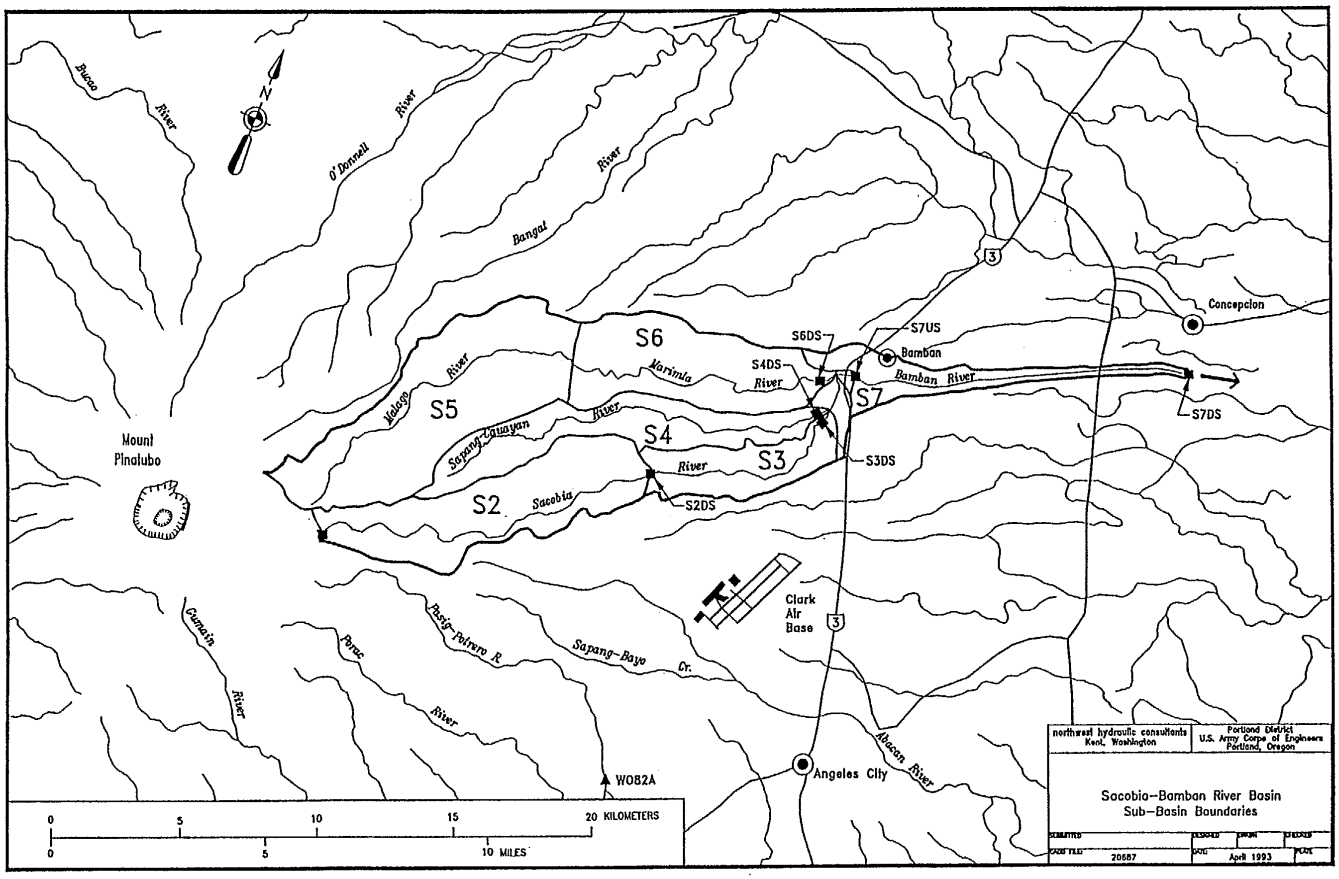


Figure 3.5.4

67

Computed Flood Hydrographs
Sacobia-Bamban Basin at Site S2DS

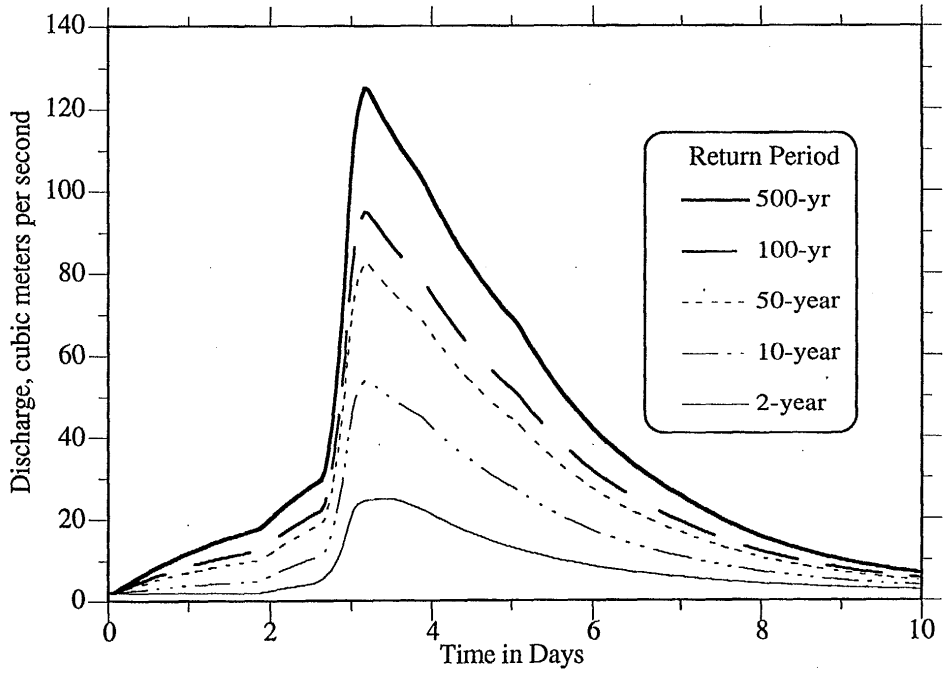


Figure 3.5.5

628

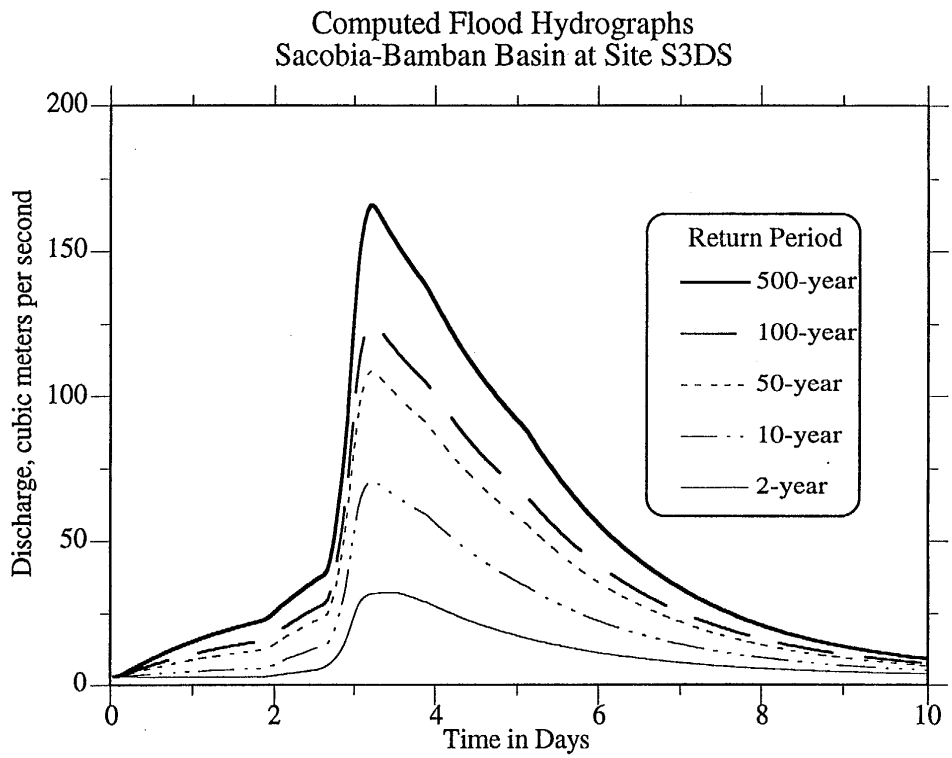

Computed Flood Hydrographs
Sacobia-Bamban Basin at Site S3DS
Discharge, cubic meters per second
200
150
100
50
0
0
2
4
6
8
10
Time in Days
Return Period
500-year
100-year
50-year
10-year
2-year

Figure 3.5.6

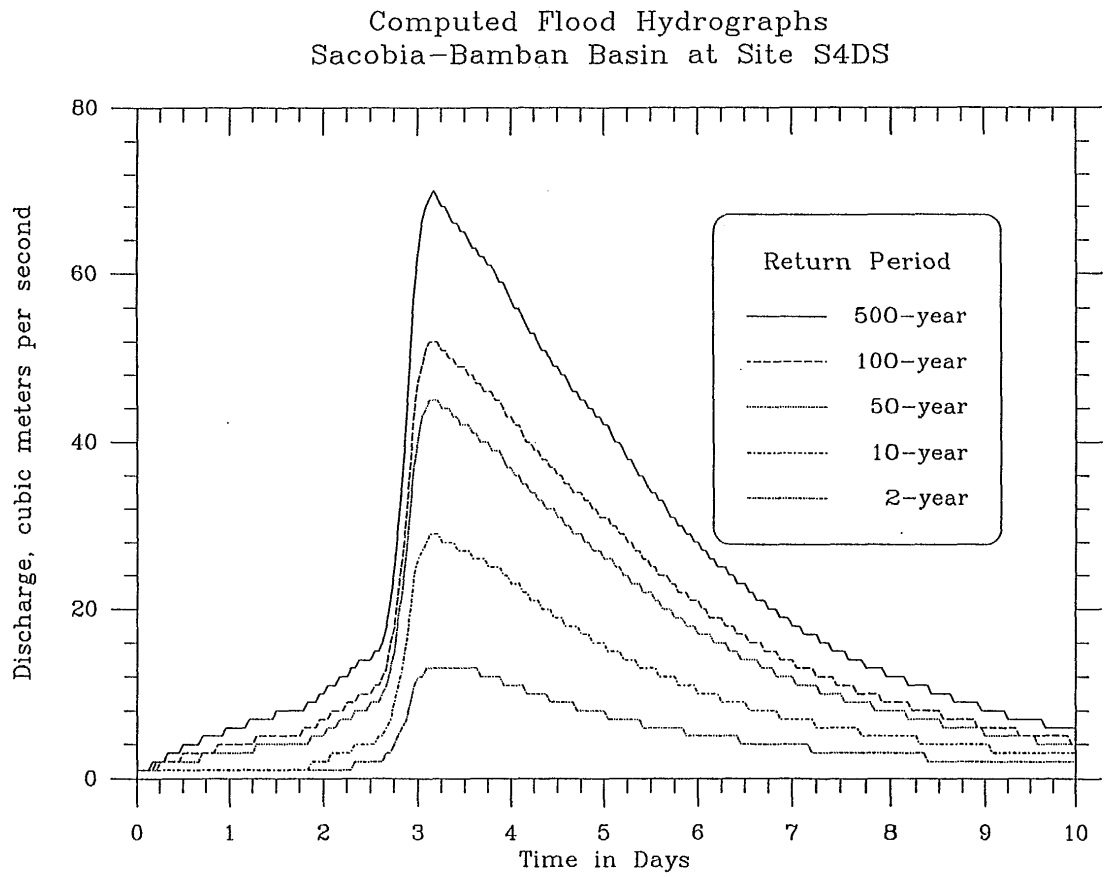


Figure 3.5.7

630

Computed Flood Hydrographs
Site S3DS and S4DS Hydrographs Combined
Sacobia-Bamban Basin

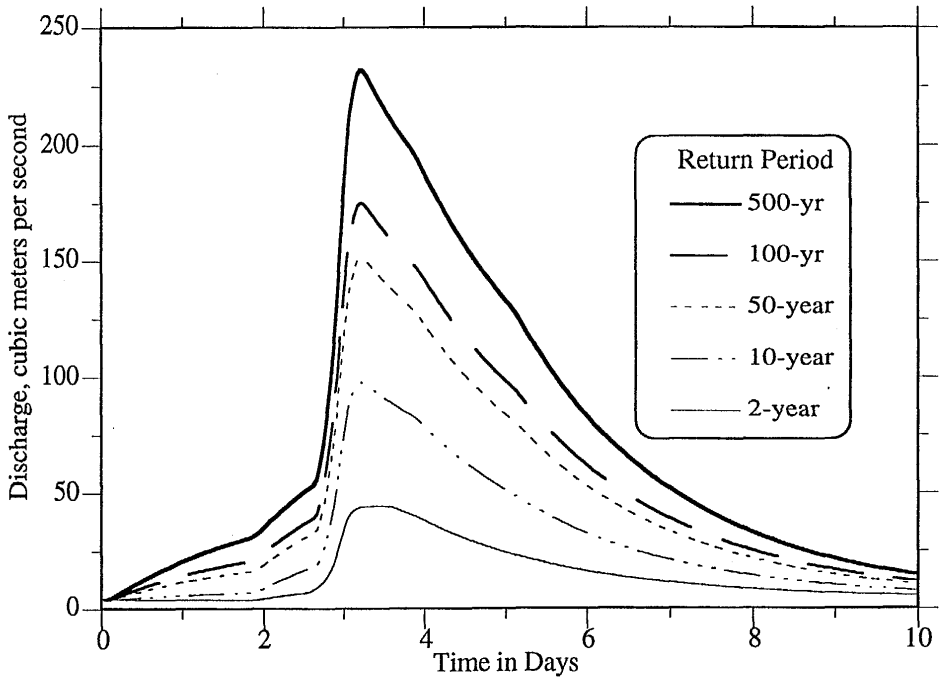


Figure 3.5.8

531

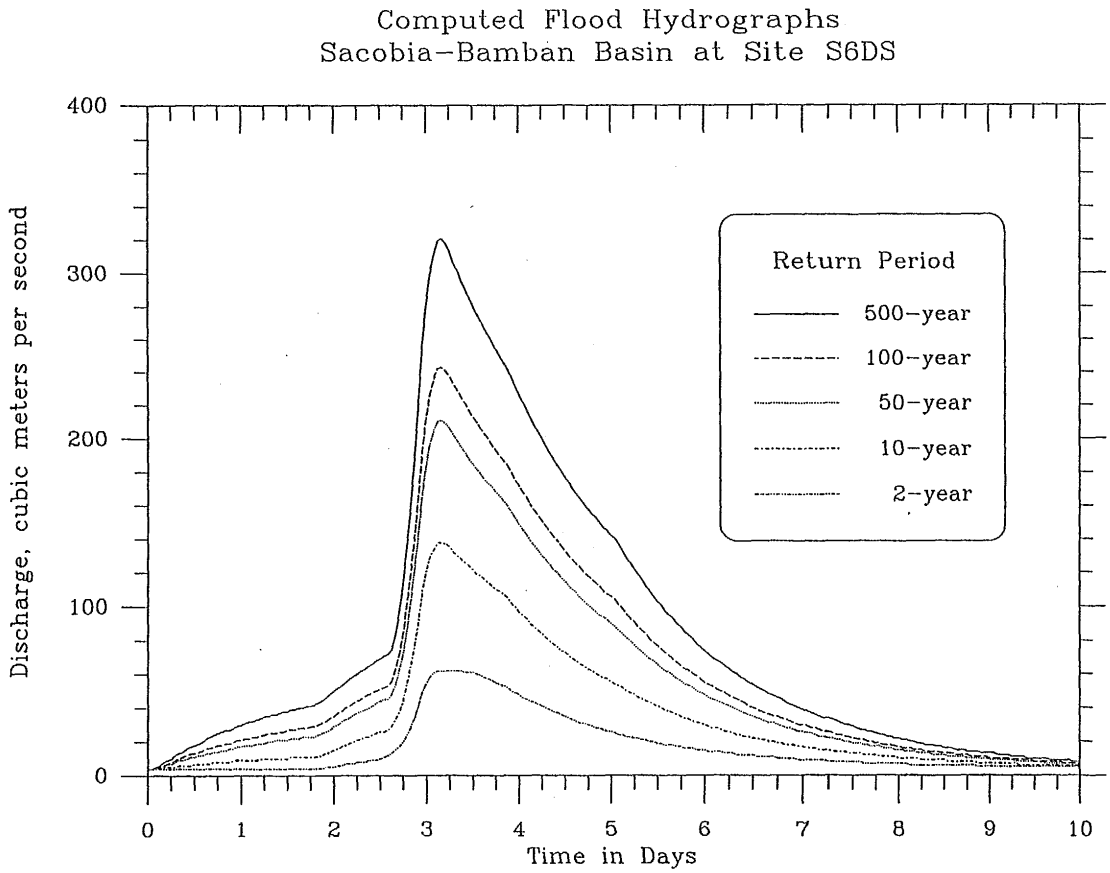

Computed Flood Hydrographs
Sacobia-Bamban Basin at Site S6DS
Discharge, cubic meters per second
400
300
200
100
0
0
1
2
3
4
5
6
7
8
9
10
Time in Days
Return Period
500-year
100-year
50-year
10-year
2-year

Figure 3.5.9

Computed Flood Hydrographs
Sacobia-Bamban Basin at Site S7US

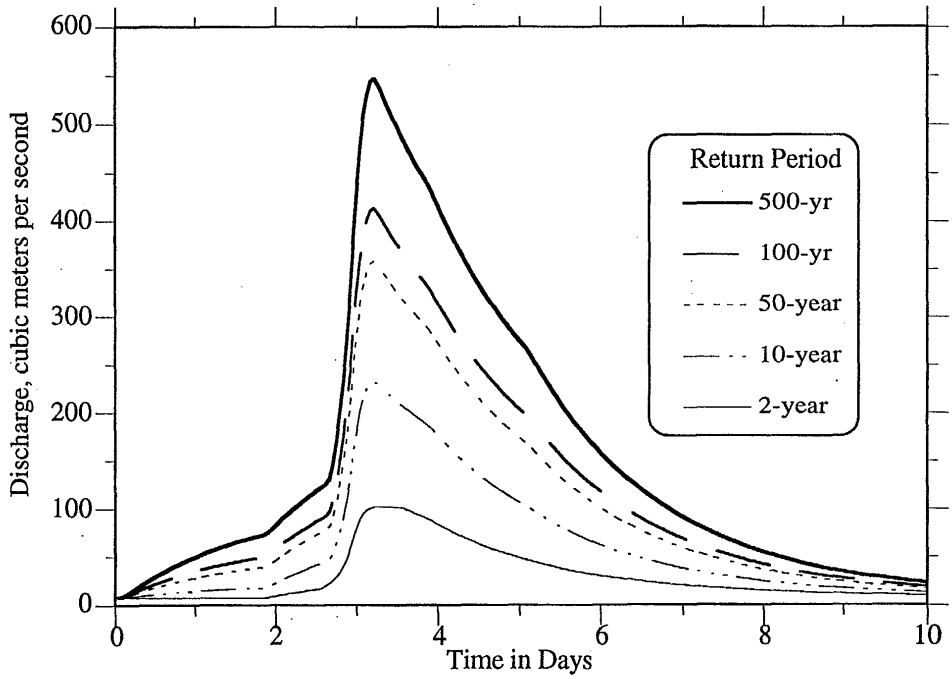


Figure 3.5.10

Computed Flood Hydrographs
Sacobia-Bamban Basin at Site S7DS

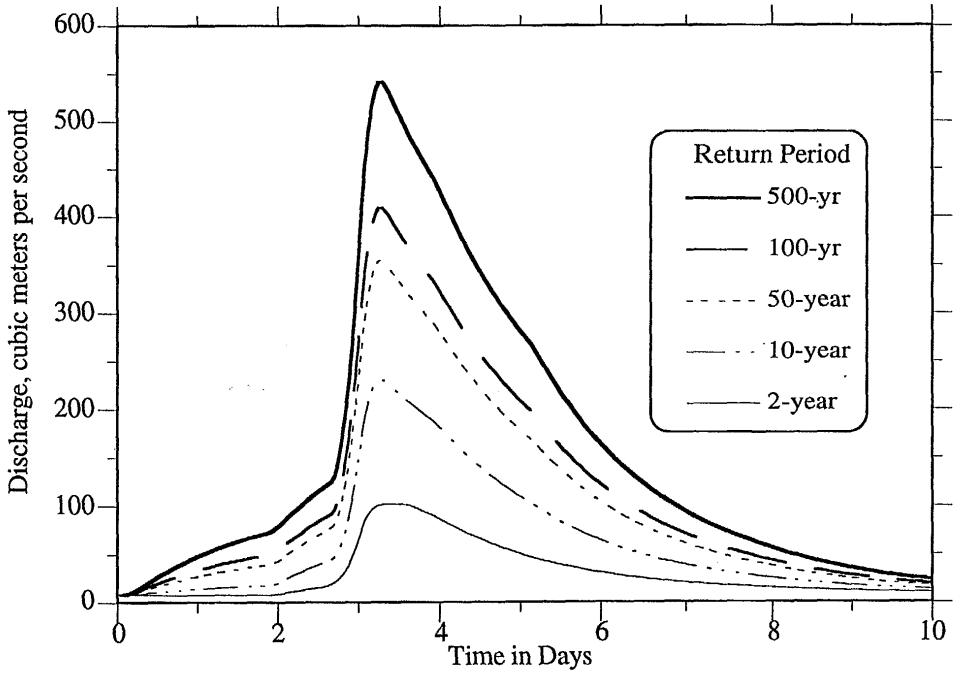


Figure 3.5.11

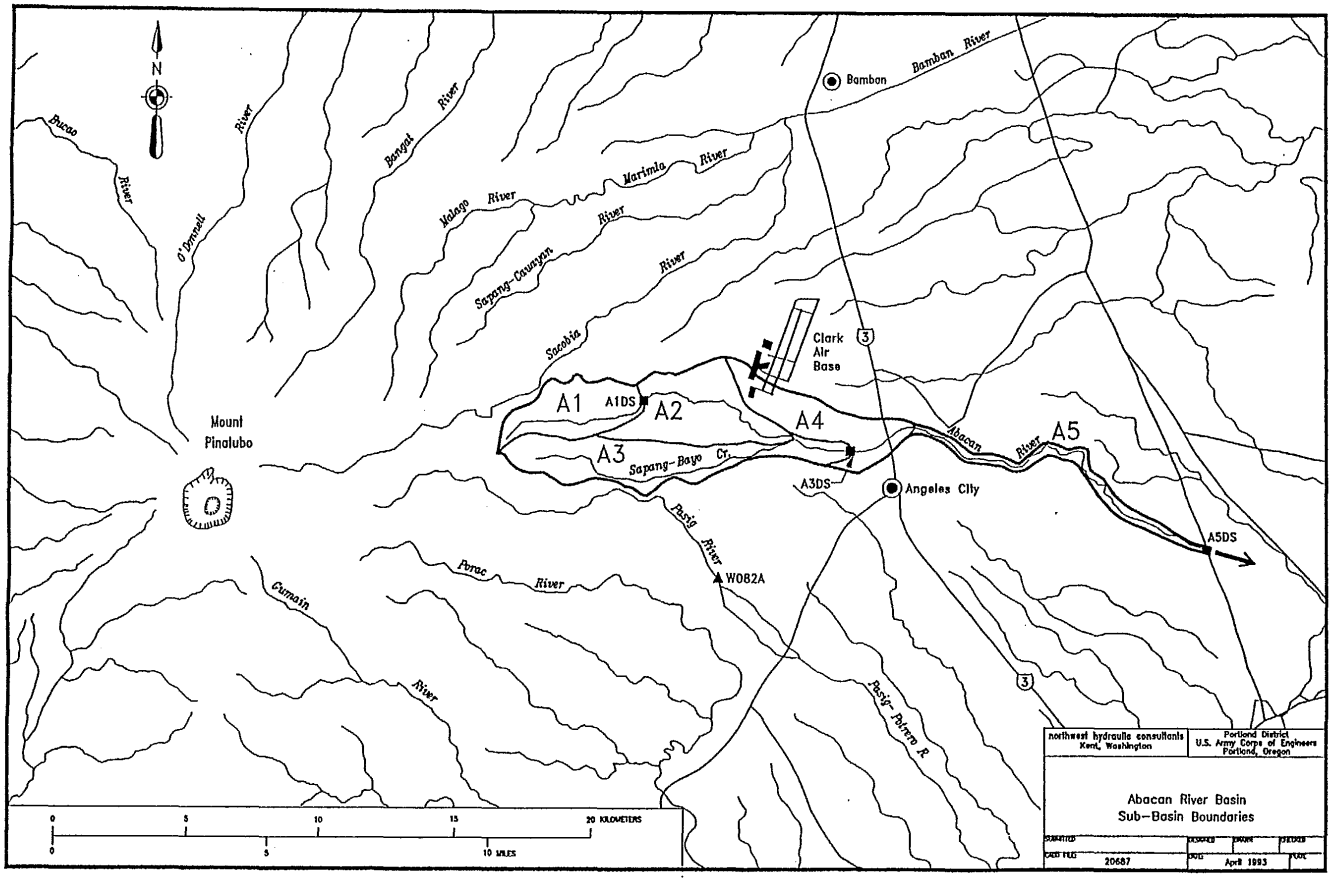


Figure 3.5.12

535

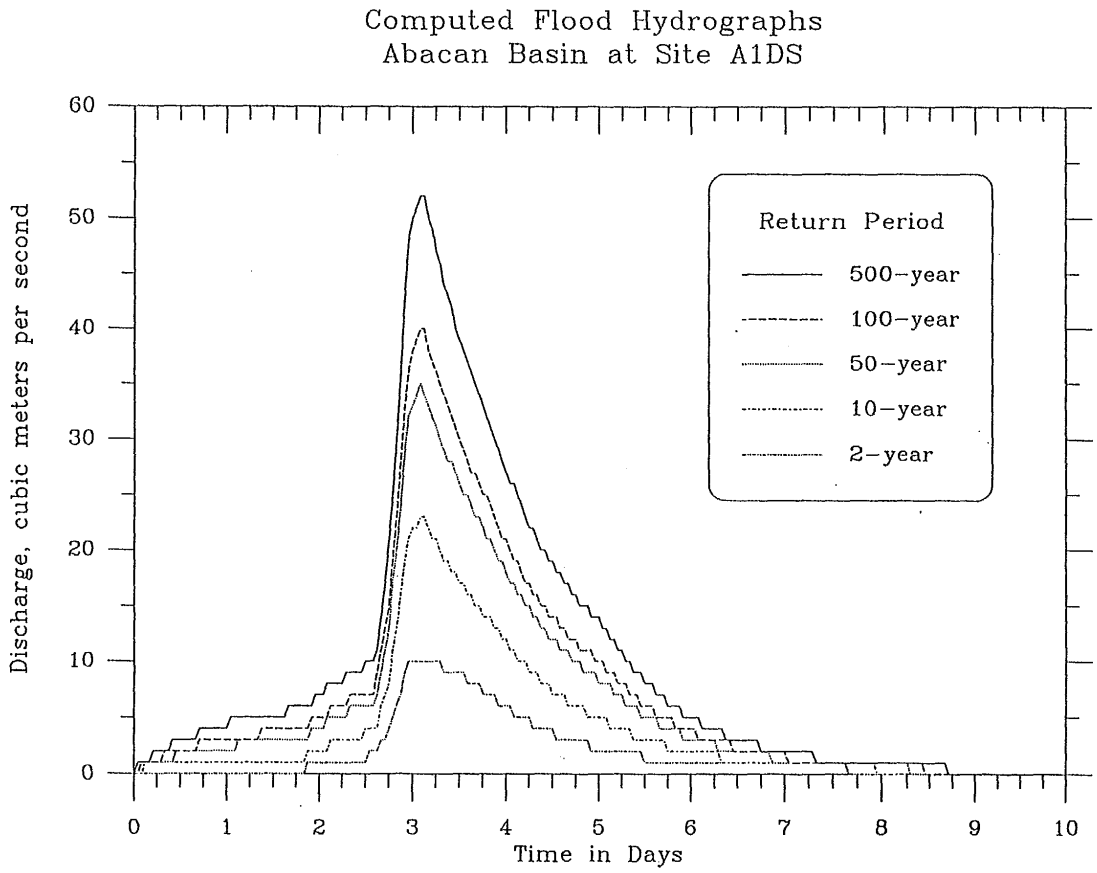

Computed Flood Hydrographs
Abacan Basin at Site A1DS
Discharge, cubic meters per second
Time in Days
Return Period
500-year
100-year
50-year
10-year
2-year
0
10
20
30
40
50
60
0
1
2
3
4
5
6
7
8
9
10

Figure 3.5.13

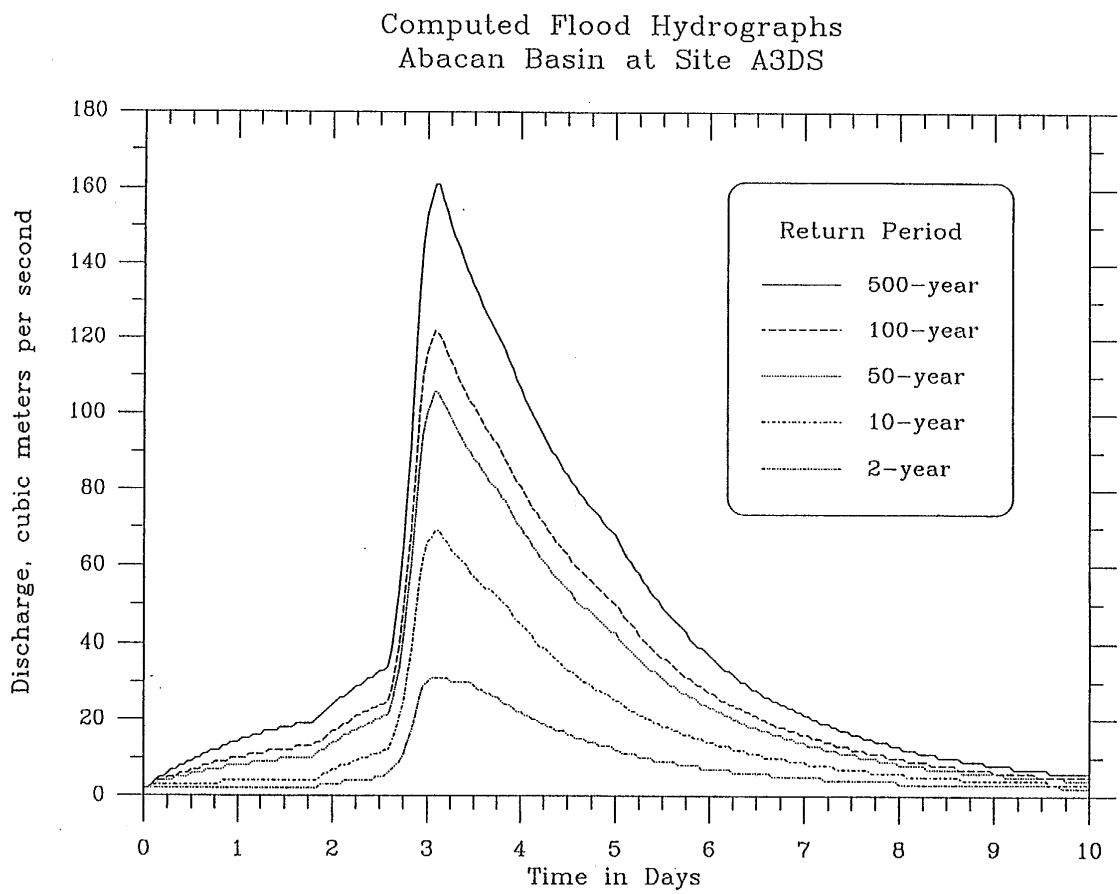

Computed Flood Hydrographs
Abacan Basin at Site A3DS
Discharge, cubic meters per second
Time in Days
Return Period
500-year
100-year
50-year
10-year
2-year
0
20
40
60
80
100
120
140
160
180
0
1
2
3
4
5
6
7
8
9
10

Figure 3.5.14

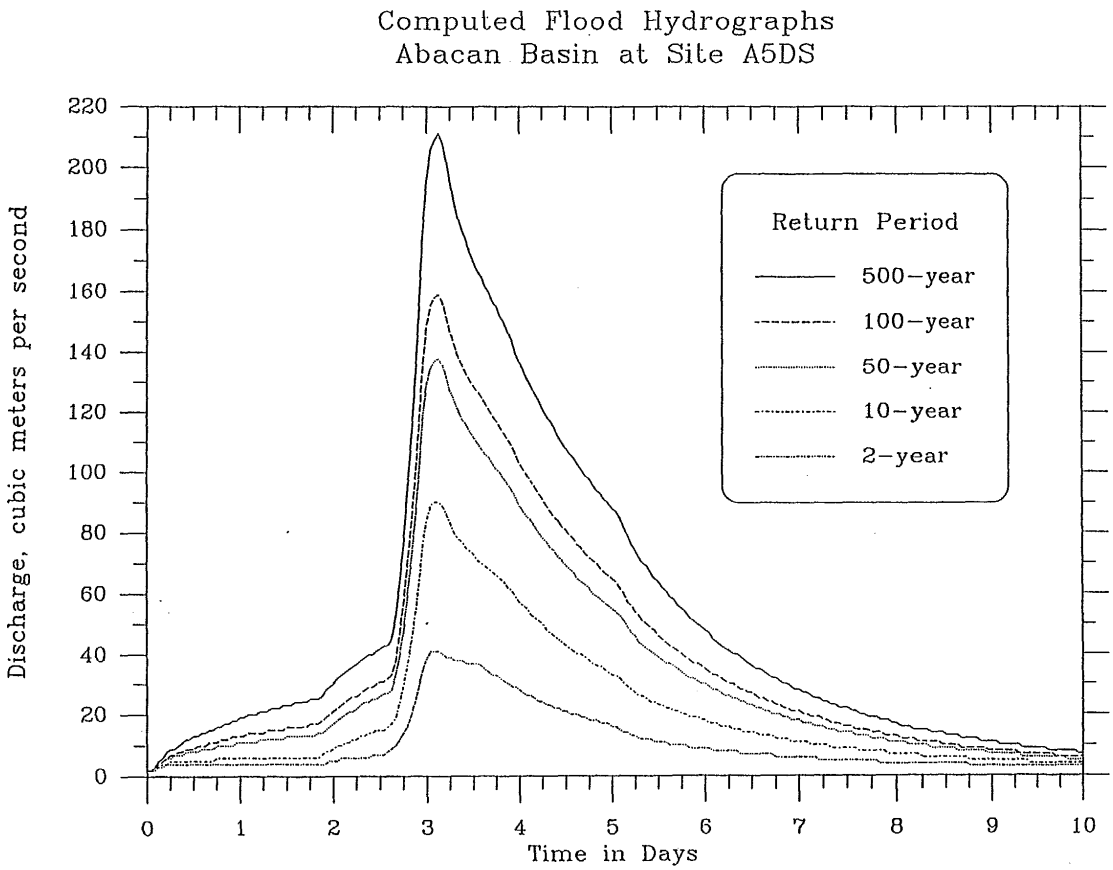

Computed Flood Hydrographs
Abacan Basin at Site A5DS
Discharge, cubic meters per second
Time in Days
Return Period
500-year
100-year
50-year
10-year
2-year
0
20
40
60
80
100
120
140
160
180
200
220
0
1
2
3
4
5
6
7
8
9
10

Figure 3.5.15

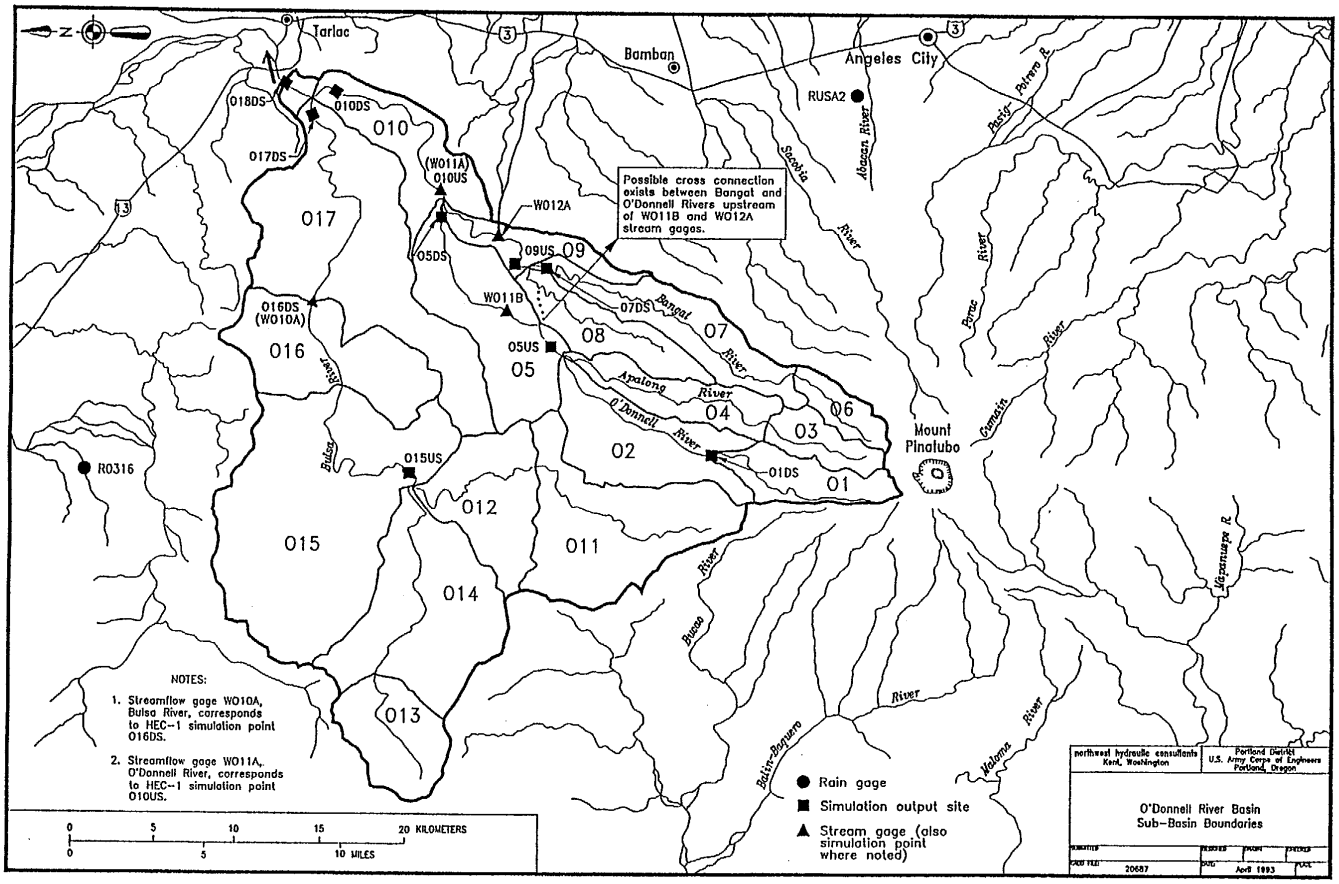

Tarlac
Bamban
Angeles City
RUSA2
018DS
010DS
010
017DS
(W011A)
010US
W012A
Possible cross connection exists between Bangat and O'Donnell Rivers upstream of W011B and W012A stream gages.
017
05DS
09US
09
W011B
07DS
Bangat
07
08
016DS
(W010A)
016
05US
05
Aqalong
River
04
06
03
O'Donnell
River
02
01DS
01
Mount Pinatubo
R0316
015US
012
015
011
014
013
Sacobia
River
Abacan River
Pasig-Potrero R.
Porac
River
Gumain
River
Bulsa
River
Bucao
River
Balin-Boguera
River
Malomo
River
Maganuepe R.
NOTES:
1. Streamflow gage W010A, Bulsa River, corresponds to HEC-1 simulation point 016DS.
2. Streamflow gage W011A, O'Donnell River, corresponds to HEC-1 simulation point 010US.
0 5 10 15 20 KILOMETERS
0 5 10 MILES
Rain gage
Simulation output site
Stream gage (also simulation point where noted)
northwest hydraulic consultants
Kent, Washington
U.S. Army Corps of Engineers
Portland, Oregon
O'Donnell River Basin
Sub-Basin Boundaries
20687
April 1993

Figure 3.5.16

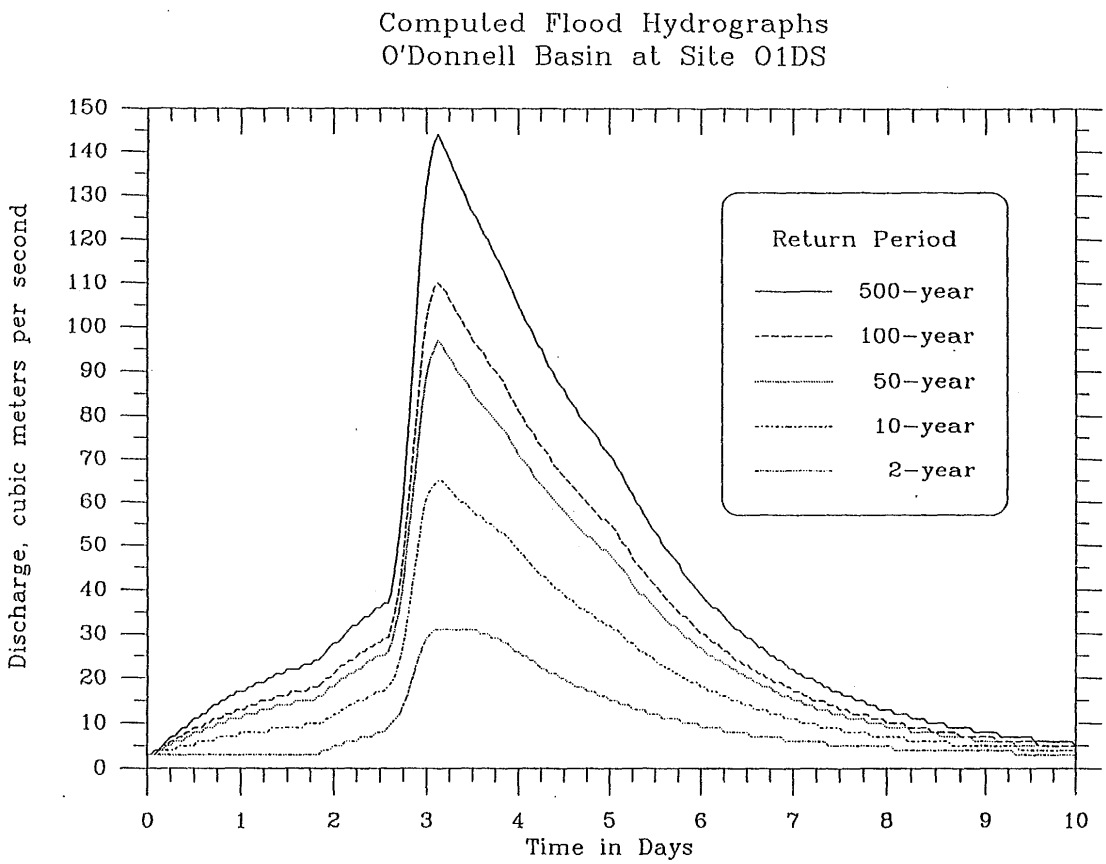

Computed Flood Hydrographs
O'Donnell Basin at Site O1DS
Discharge, cubic meters per second
Time in Days
Return Period
500-year
100-year
50-year
10-year
2-year
0
10
20
30
40
50
60
70
80
90
100
110
120
130
140
150
0
1
2
3
4
5
6
7
8
9
10

Figure 3.5.17

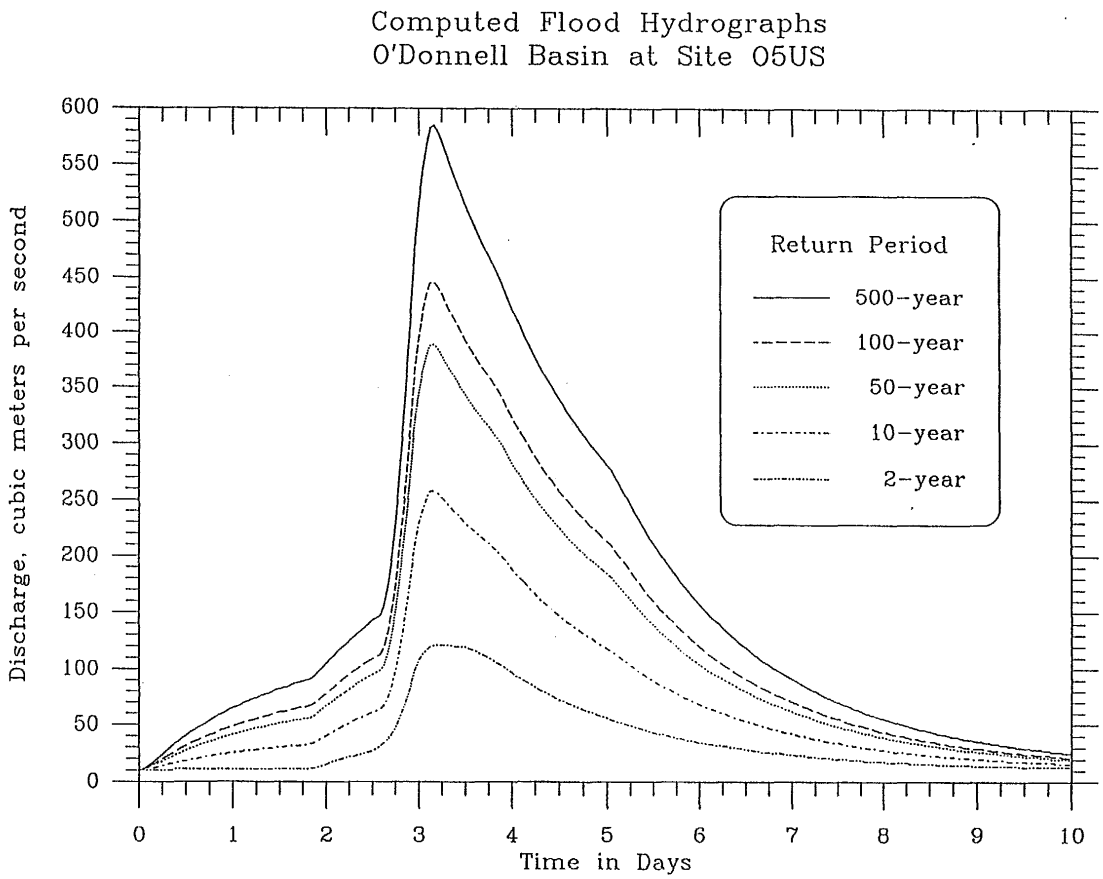


Figure 3.5.18

54

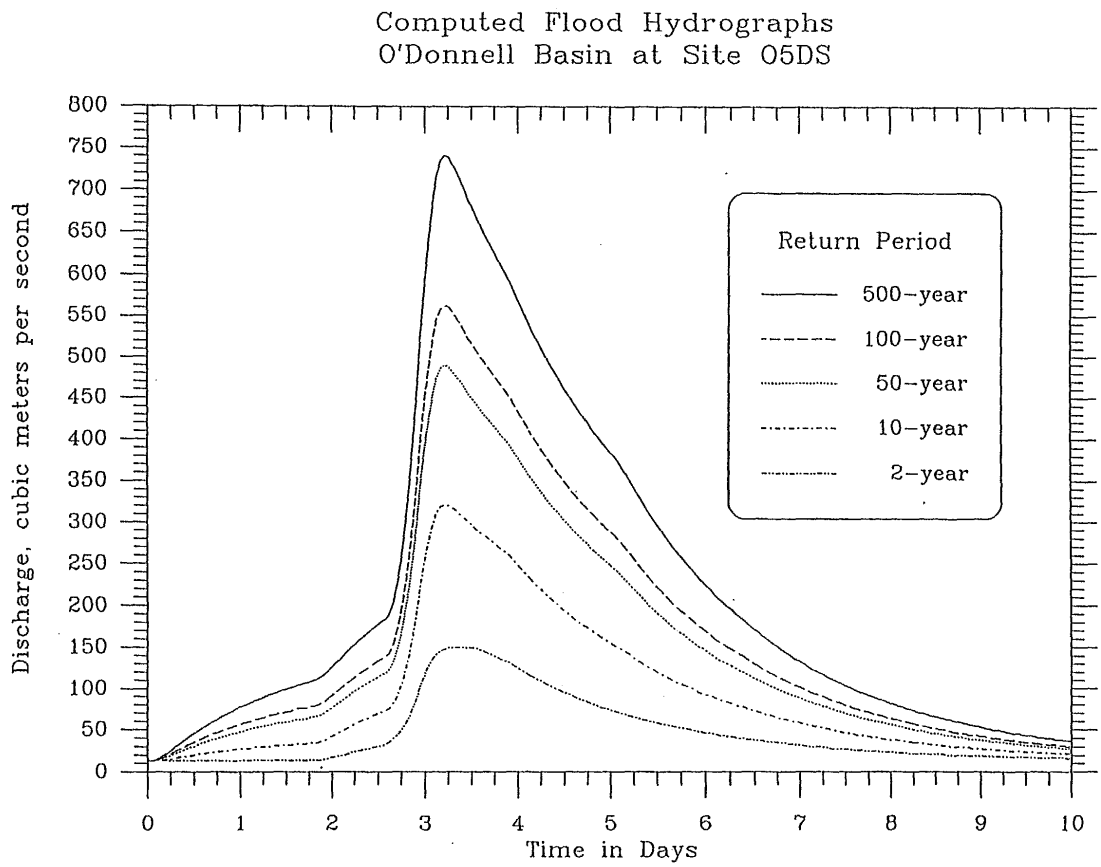

Computed Flood Hydrographs
O'Donnell Basin at Site 05DS
Discharge, cubic meters per second
Time in Days
Return Period
500-year
100-year
50-year
10-year
2-year
0
50
100
150
200
250
300
350
400
450
500
550
600
650
700
750
800
0
1
2
3
4
5
6
7
8
9
10

Figure 3.5.19

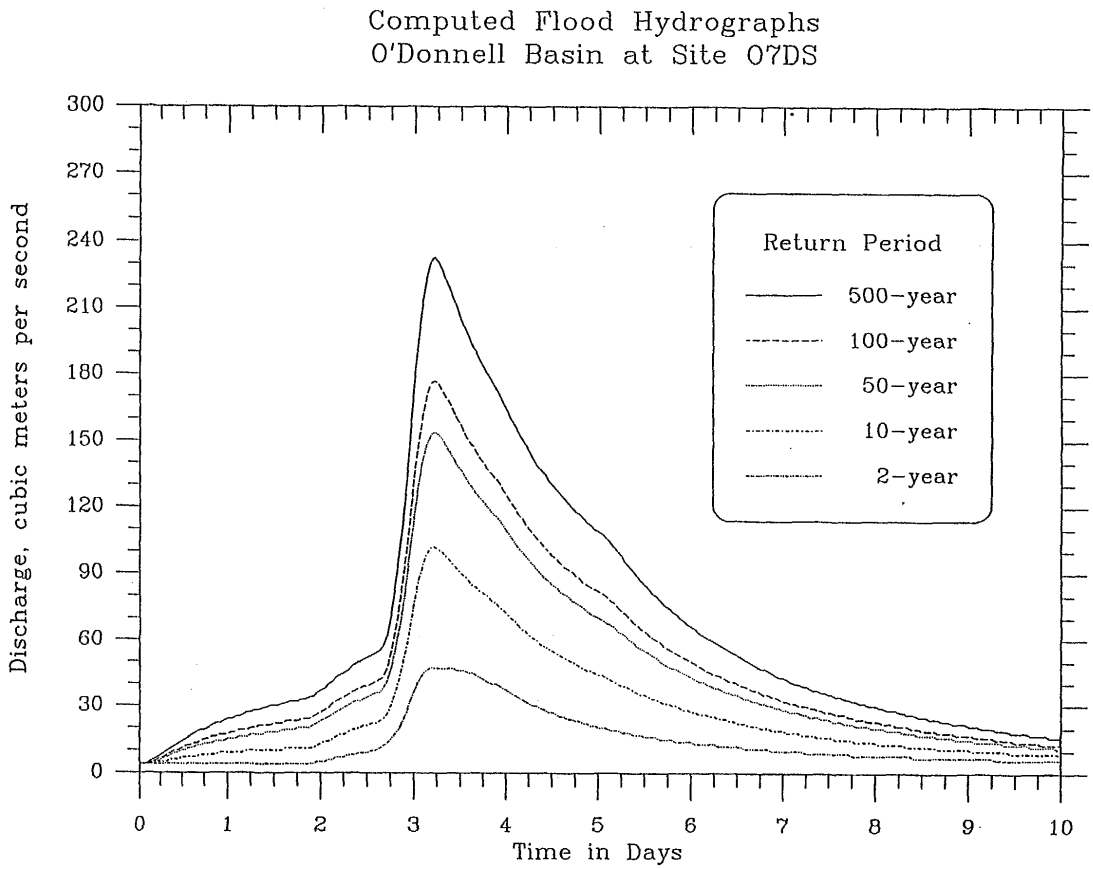

Computed Flood Hydrographs
O'Donnell Basin at Site 07DS
Discharge, cubic meters per second
300
270
240
210
180
150
120
90
60
30
0
0
1
2
3
4
5
6
7
8
9
10
Time in Days
Return Period
500-year
100-year
50-year
10-year
2-year

Figure 3.5.20

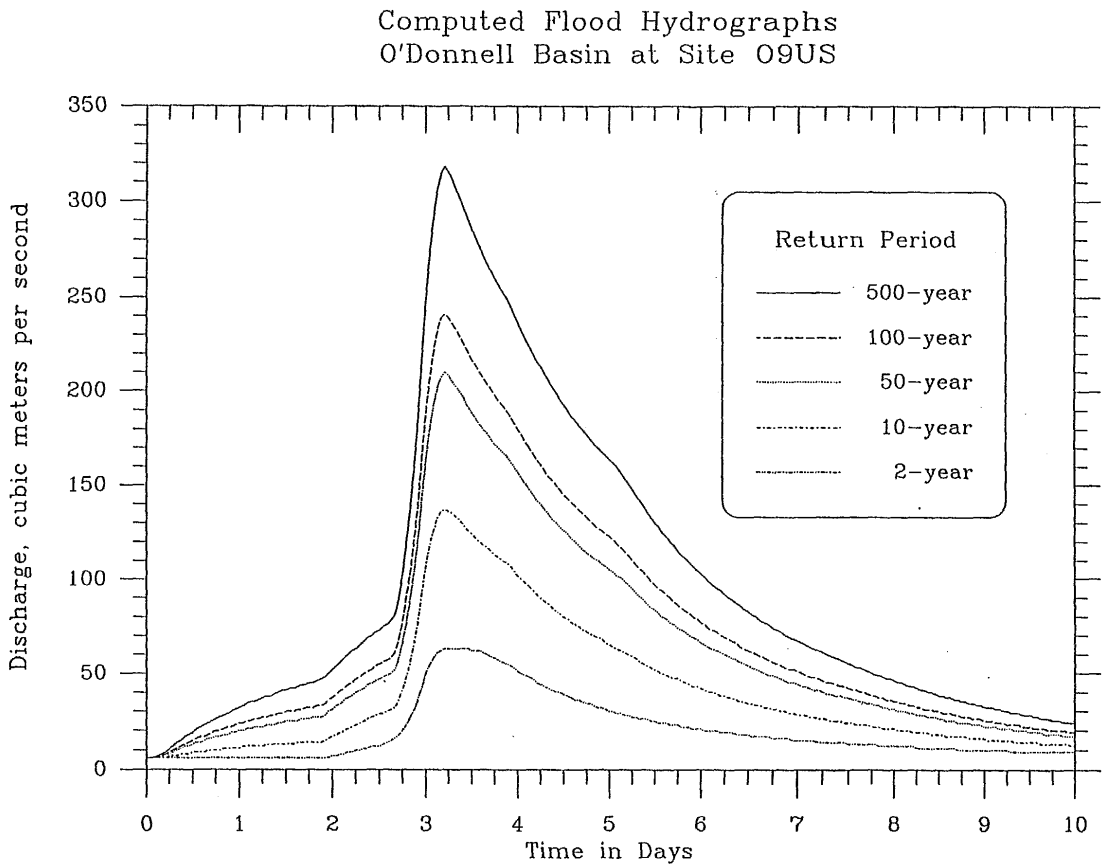

Computed Flood Hydrographs
O'Donnell Basin at Site 09US
Discharge, cubic meters per second
350
300
250
200
150
100
50
0
0
1
2
3
4
5
6
7
8
9
10
Time in Days
Return Period
500-year
100-year
50-year
10-year
2-year

Figure 3.5.21

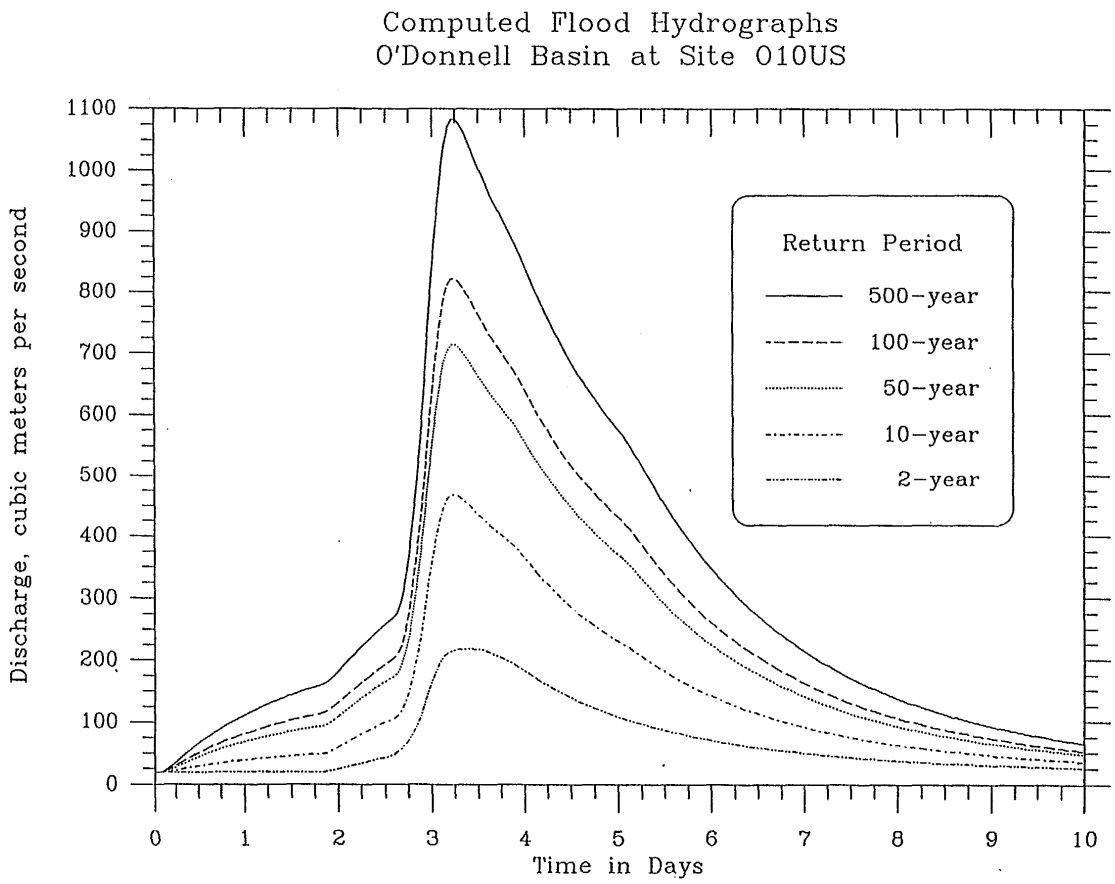

Computed Flood Hydrographs
O'Donnell Basin at Site 010US
Discharge, cubic meters per second
Time in Days
Return Period
500-year
100-year
50-year
10-year
2-year
0
100
200
300
400
500
600
700
800
900
1000
1100
0
1
2
3
4
5
6
7
8
9
10

Figure 3.5.22

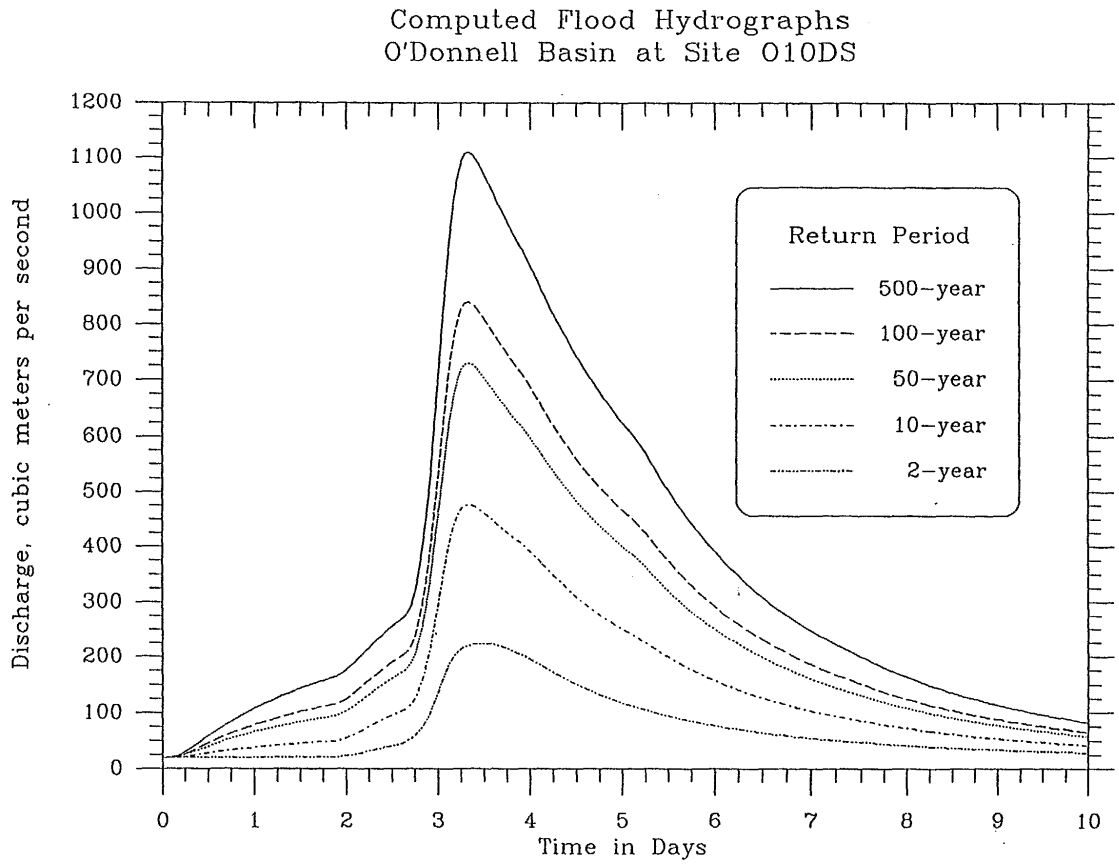


Figure 3.5.23

246

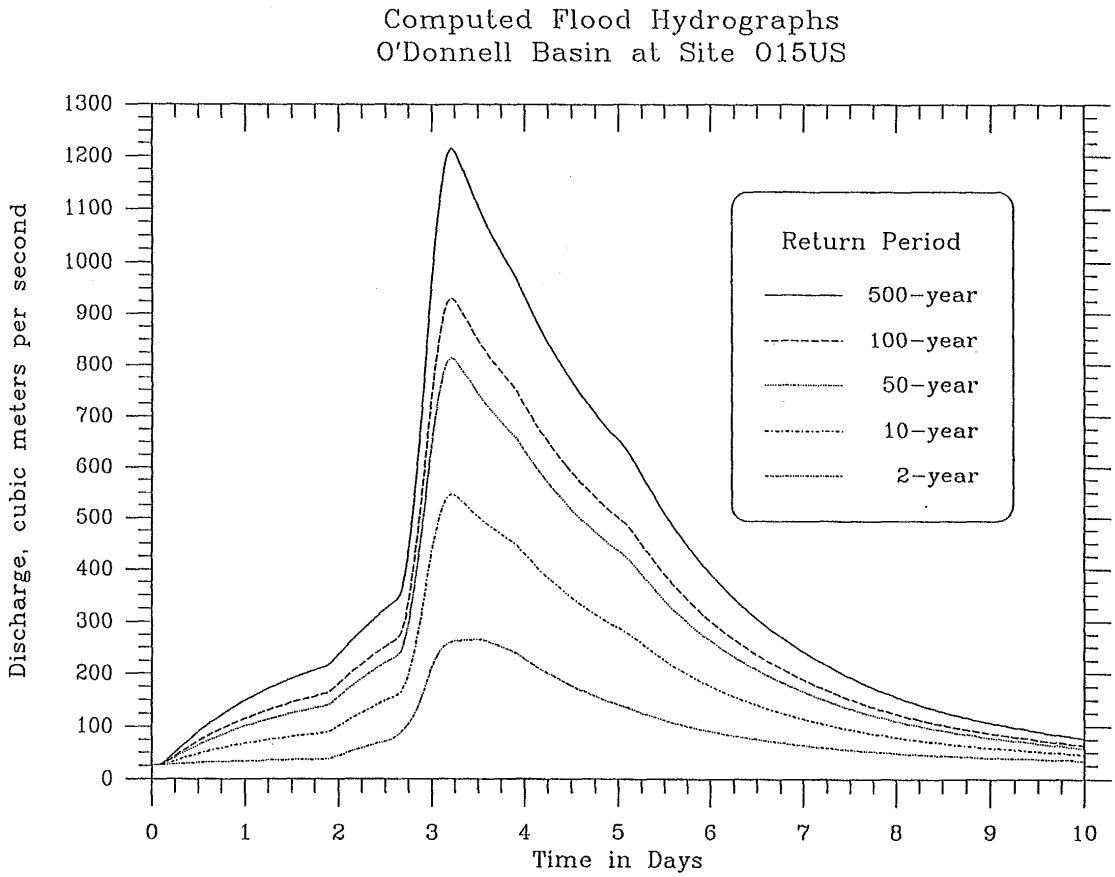

Computed Flood Hydrographs
O'Donnell Basin at Site 015US
Discharge, cubic meters per second
Time in Days
Return Period
500-year
100-year
50-year
10-year
2-year
0
100
200
300
400
500
600
700
800
900
1000
1100
1200
1300
0
1
2
3
4
5
6
7
8
9
10

Figure 3.5.24

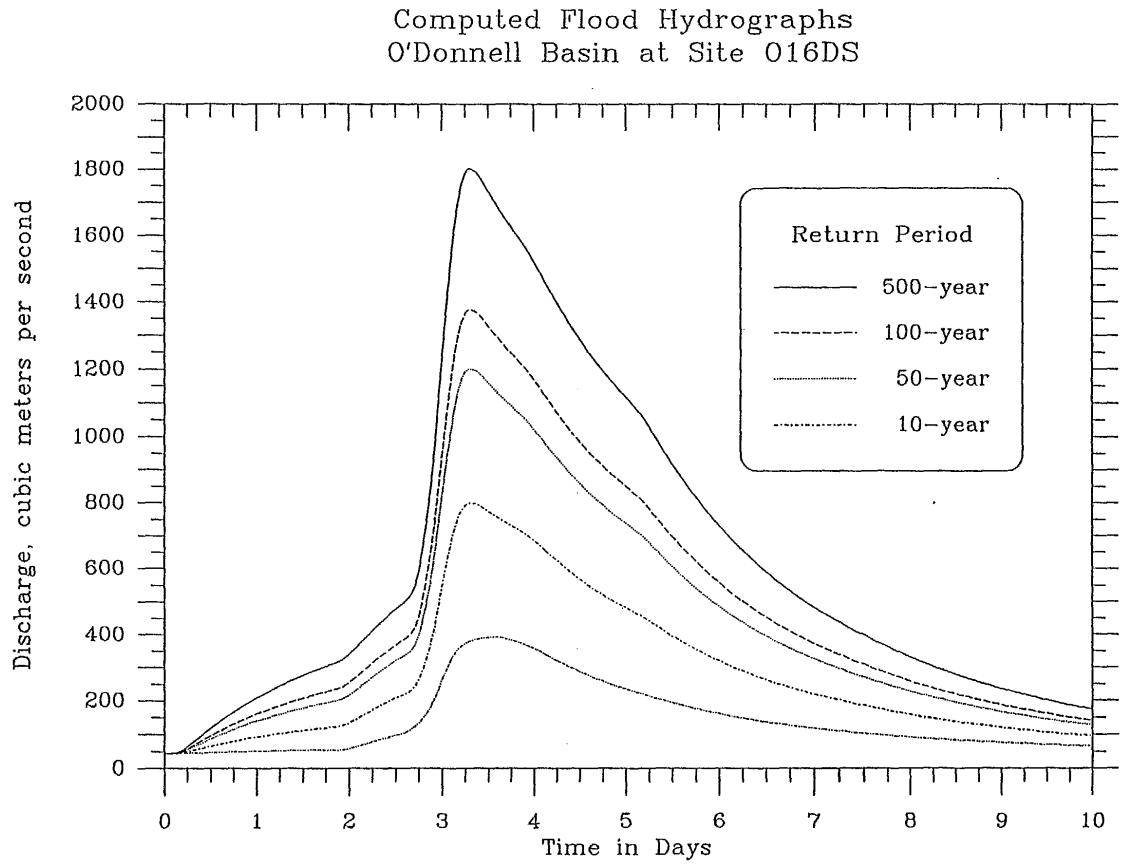

Computed Flood Hydrographs
O'Donnell Basin at Site 016DS
Discharge, cubic meters per second
2000
1800
1600
1400
1200
1000
800
600
400
200
0
0
1
2
3
4
5
6
7
8
9
10
Time in Days
Return Period
500-year
100-year
50-year
10-year

Figure 3.5.25

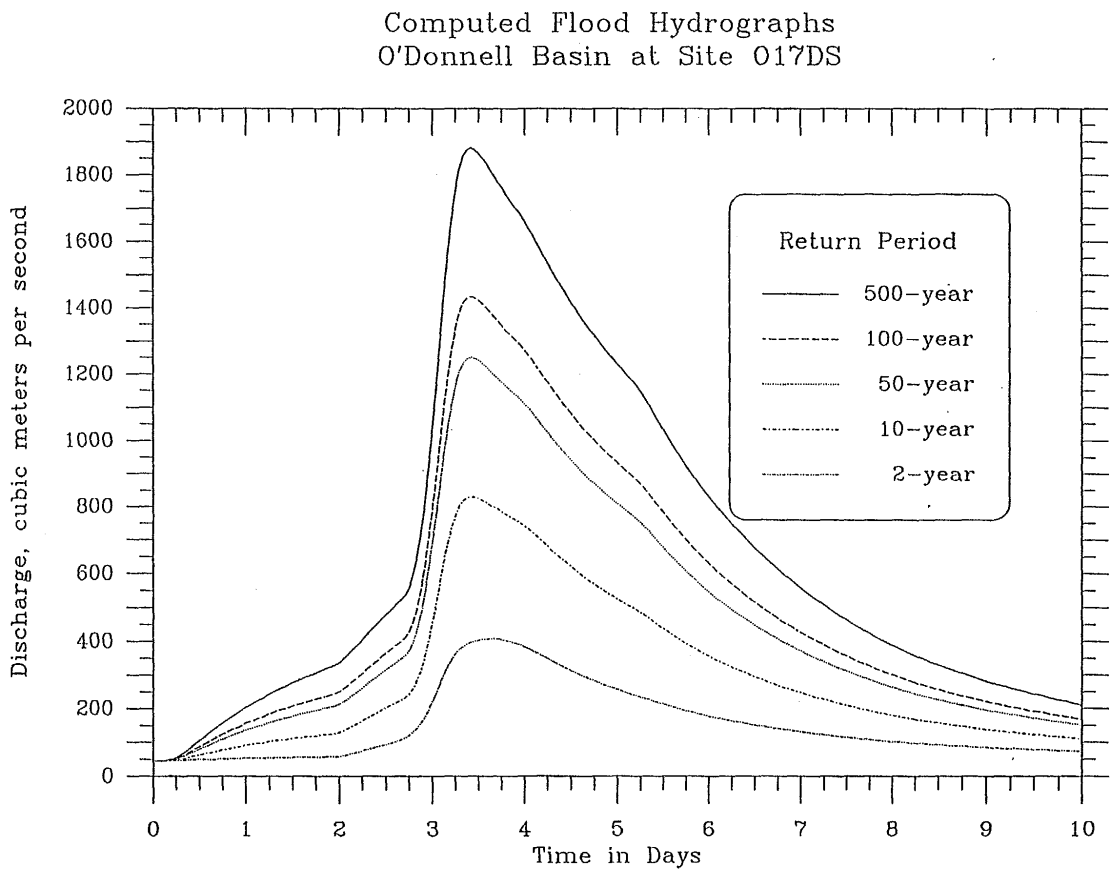

Computed Flood Hydrographs
O'Donnell Basin at Site 017DS
Discharge, cubic meters per second
2000
1800
1600
1400
1200
1000
800
600
400
200
0
0
1
2
3
4
5
6
7
8
9
10
Time in Days
Return Period
500-year
100-year
50-year
10-year
2-year

Figure 3.5.26

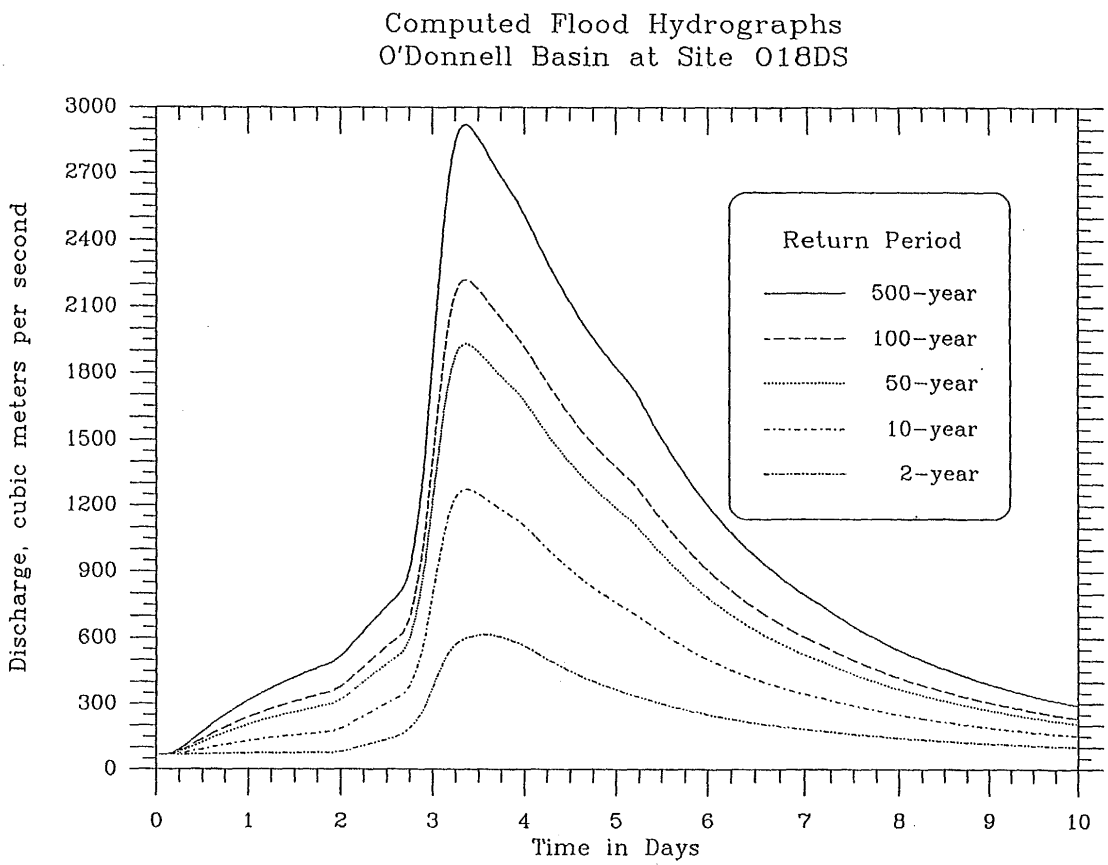

Computed Flood Hydrographs
O'Donnell Basin at Site 018DS
Discharge, cubic meters per second
3000
2700
2400
2100
1800
1500
1200
900
600
300
0
0
1
2
3
4
5
6
7
8
9
10
Time in Days
Return Period
500-year
100-year
50-year
10-year
2-year

Figure 3.5.27

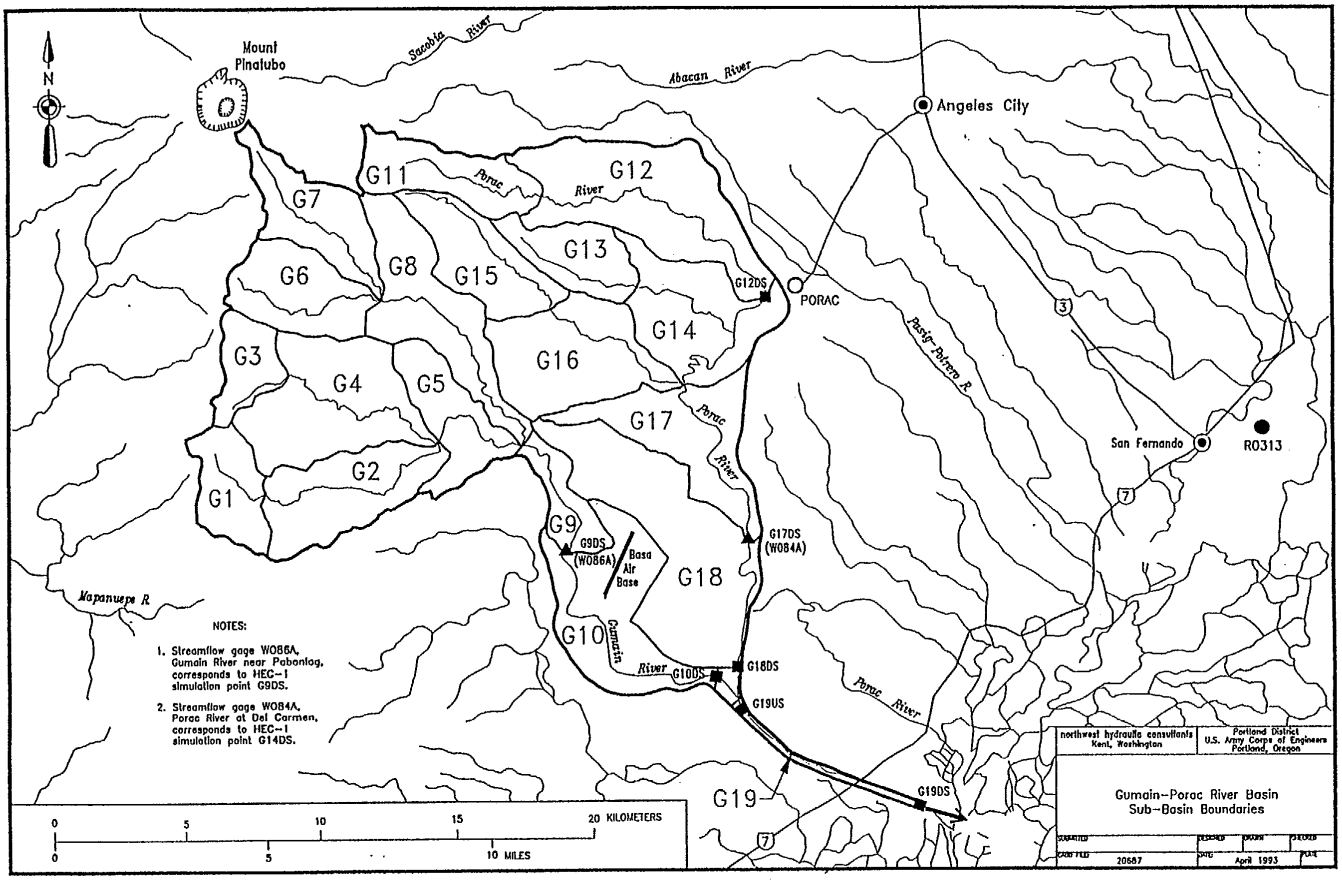

Mount Pinatubo
Sacobia River
Abacan River
Angeles City
N
G1
G2
G3
G4
G5
G6
G7
G8
G9
G10
G11
G12
G13
G14
G15
G16
G17
G18
G19
Porac River
Gumain River
Pasig-Potrero R
Mapanuepe R
G12DS
PORAC
G17DS (W084A)
G9DS (W086A)
Basa Air Base
G10DS
G18DS
G19US
G19DS
San Fernando
RO313
NOTES:
1. Streamflow gage W086A, Gumain River near Pabanlag, corresponds to HEC-1 simulation point G9DS.
2. Streamflow gage W084A, Porac River at Del Carmen, corresponds to HEC-1 simulation point G14DS.
0 5 10 15 20 KILOMETERS
0 5 10 MILES
northwest hydraulic consultants Kent, Washington
Portland District U.S. Army Corps of Engineers Portland, Oregon
Gumain-Porac River Basin Sub-Basin Boundaries
20687
April 1993

Figure 3.5.28

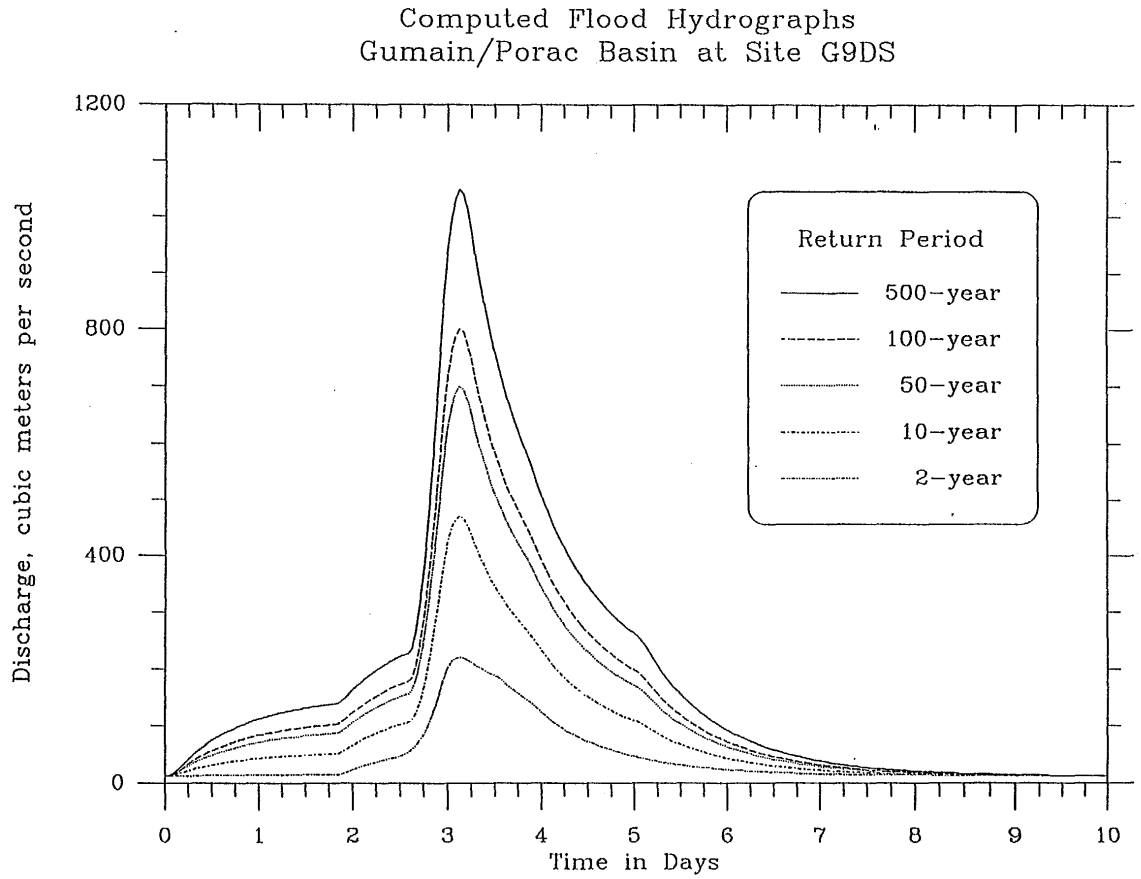

Computed Flood Hydrographs
Gumain/Porac Basin at Site G9DS
Discharge, cubic meters per second
1200
800
400
0
0
1
2
3
4
5
6
7
8
9
10
Time in Days
Return Period
500-year
100-year
50-year
10-year
2-year

Figure 3.5.29

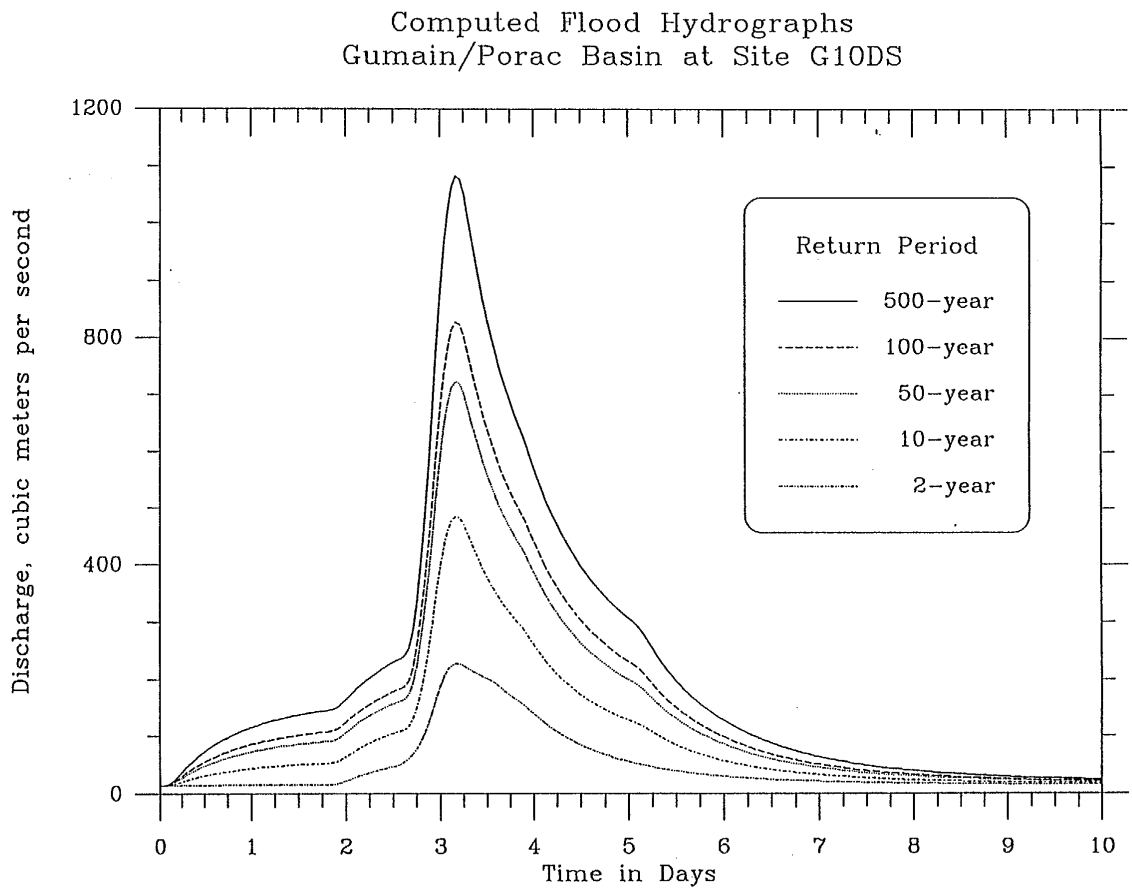


Figure 3.5.30

639

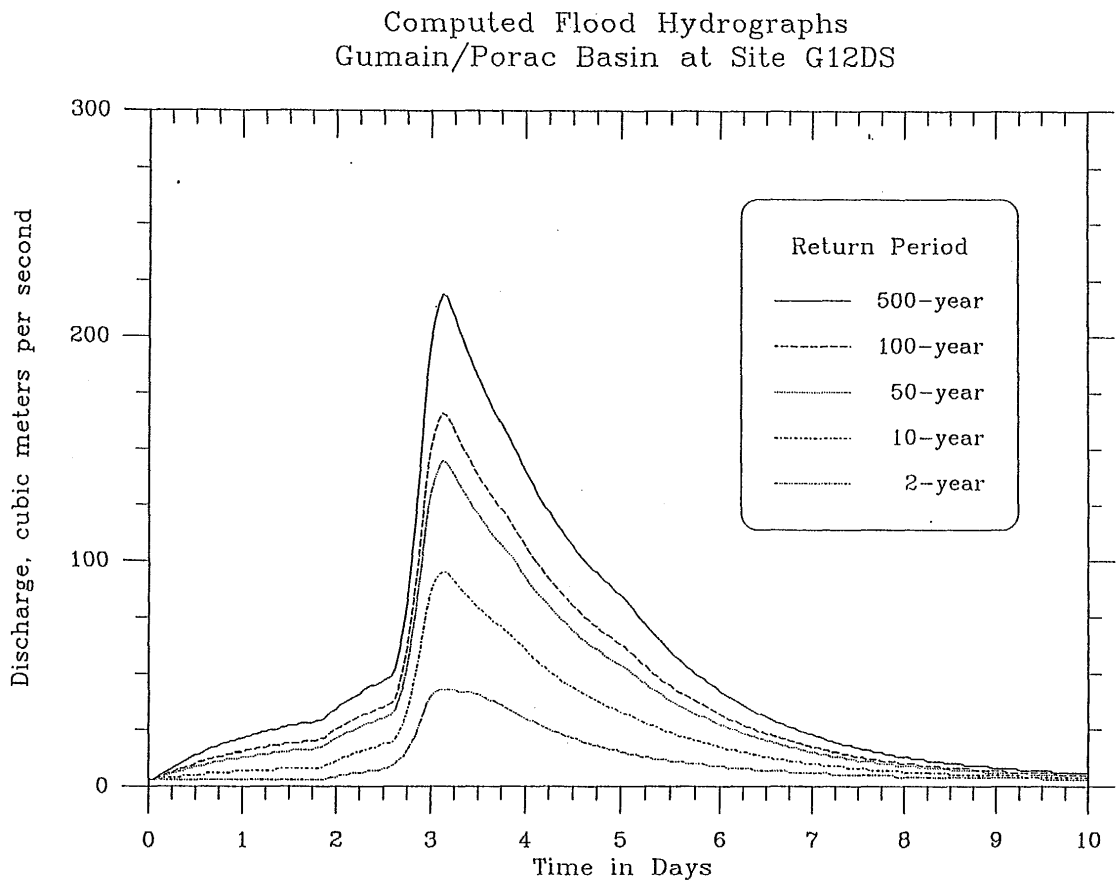

Computed Flood Hydrographs
Gumain/Porac Basin at Site G12DS
Discharge, cubic meters per second
300
200
100
0
0
1
2
3
4
5
6
7
8
9
10
Time in Days
Return Period
500-year
100-year
50-year
10-year
2-year

Figure 3.5.31

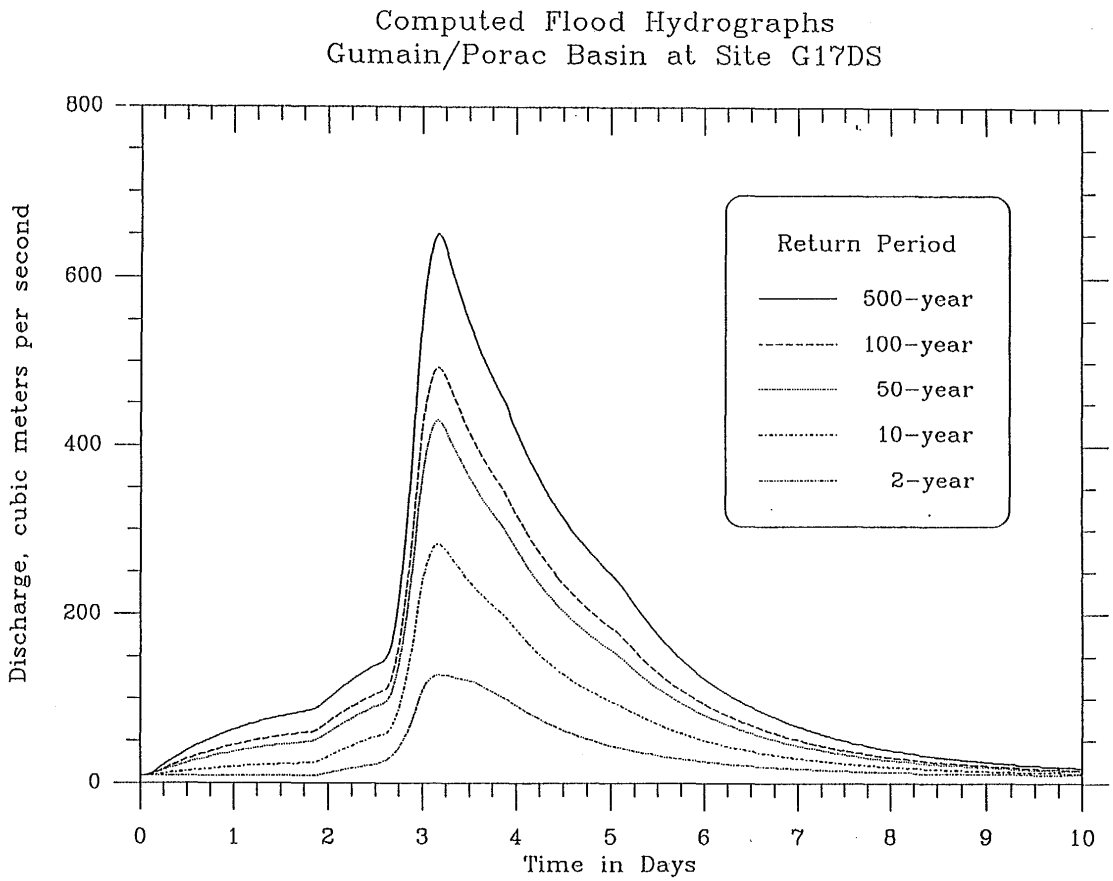


Figure 3.5.32

259

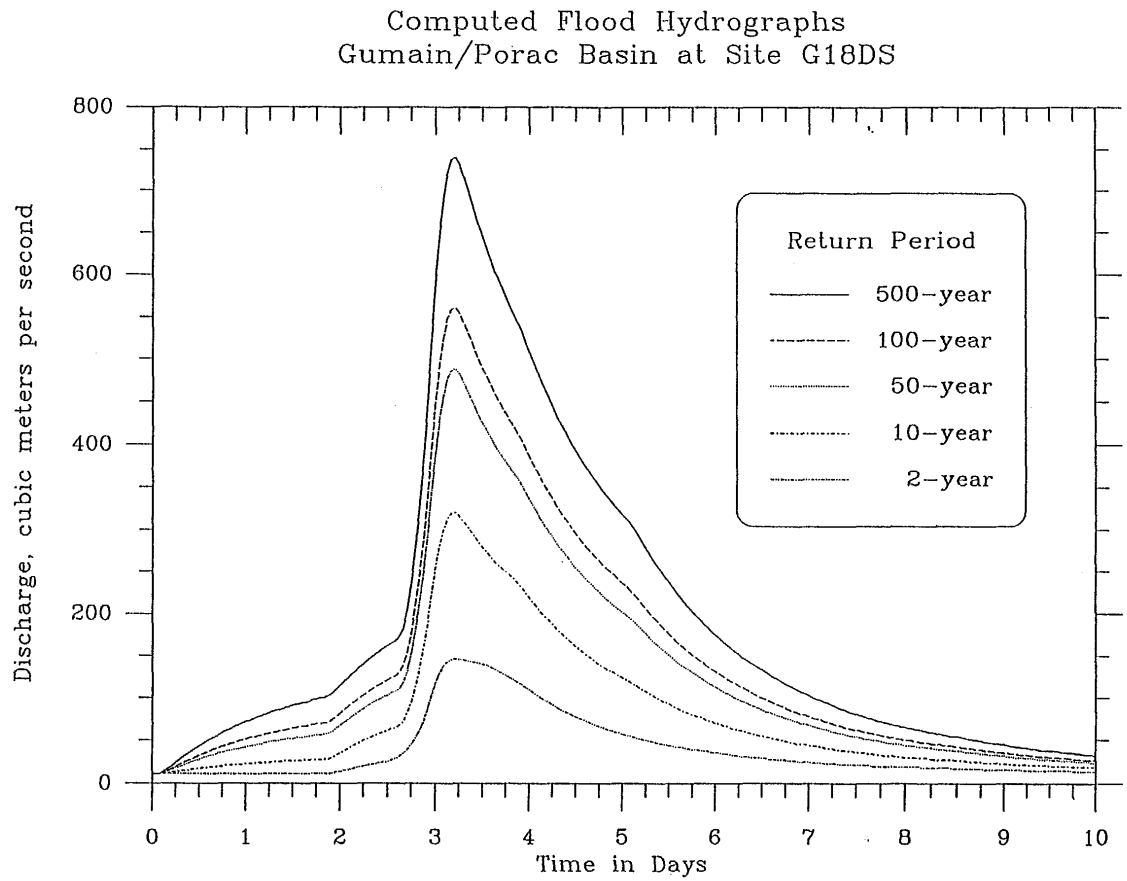

Computed Flood Hydrographs
Gumain/Porac Basin at Site G18DS
Discharge, cubic meters per second
800
600
400
200
0
0
1
2
3
4
5
6
7
8
9
10
Time in Days
Return Period
500-year
100-year
50-year
10-year
2-year

Figure 3.5.33

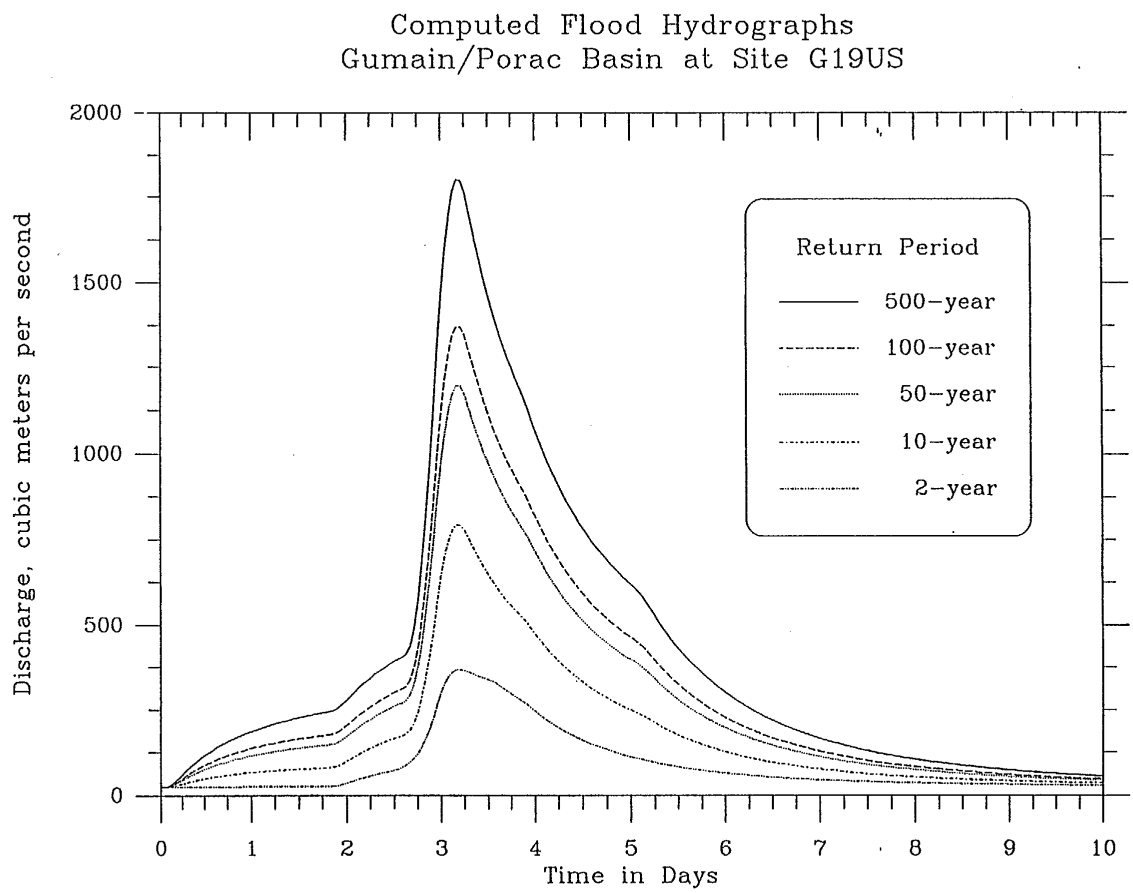

Computed Flood Hydrographs
Gumain/Porac Basin at Site G19US
Discharge, cubic meters per second
2000
1500
1000
500
0
0
1
2
3
4
5
6
7
8
9
10
Time in Days
Return Period
500-year
100-year
50-year
10-year
2-year

Figure 3.5.34

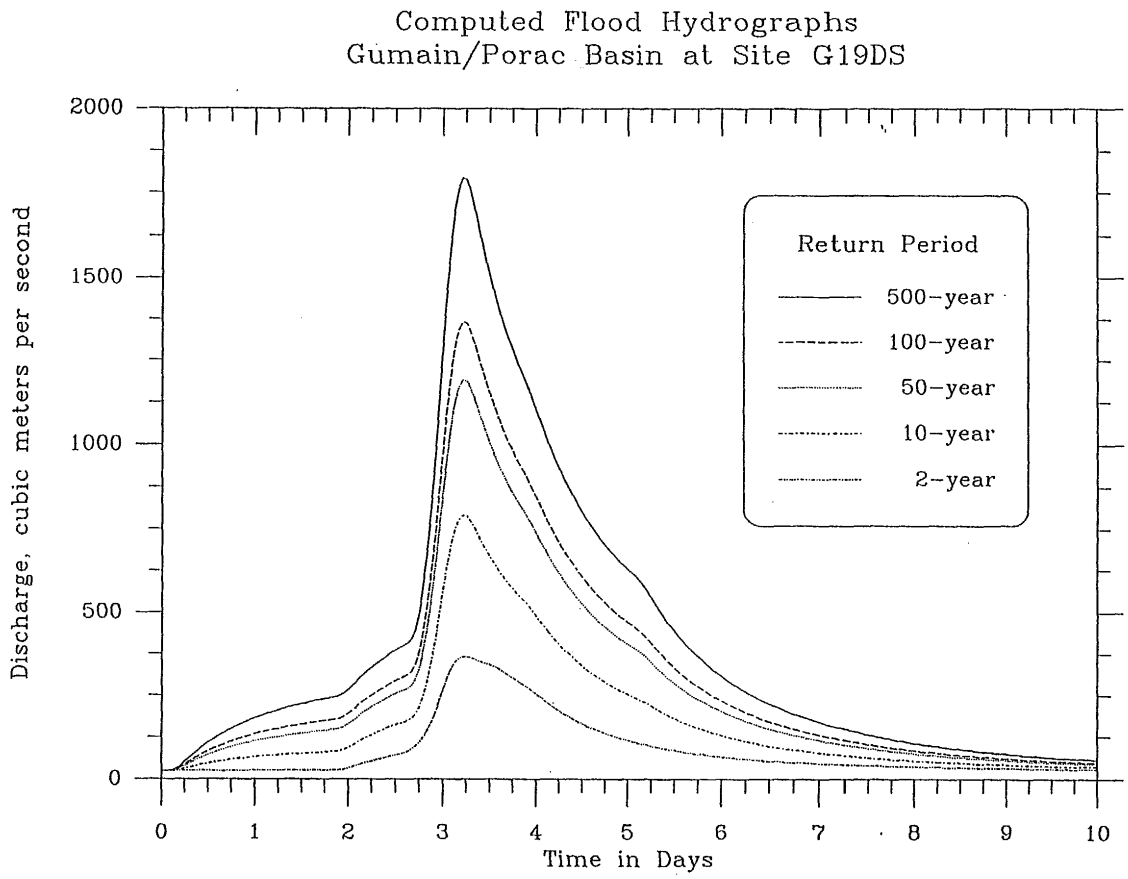

Computed Flood Hydrographs
Gumain/Porac Basin at Site G19DS
Discharge, cubic meters per second
2000
1500
1000
500
0
0
1
2
3
4
5
6
7
8
9
10
Time in Days
Return Period
500-year
100-year
50-year
10-year
2-year

Figure 3.5.35

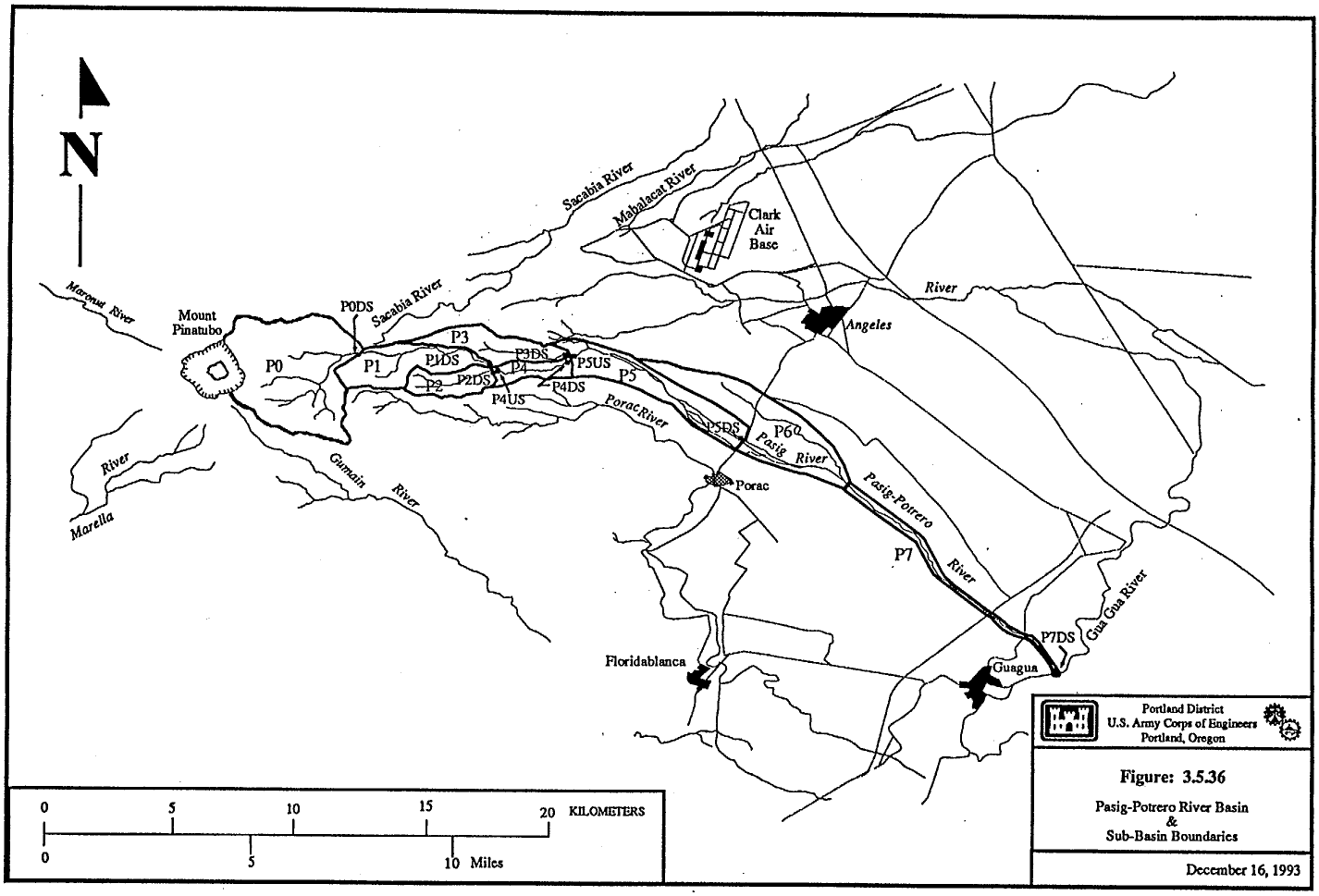


Figure: 3.5.36

Pasig-Potrero River Basin
&
Sub-Basin Boundaries

December 16, 1993

659

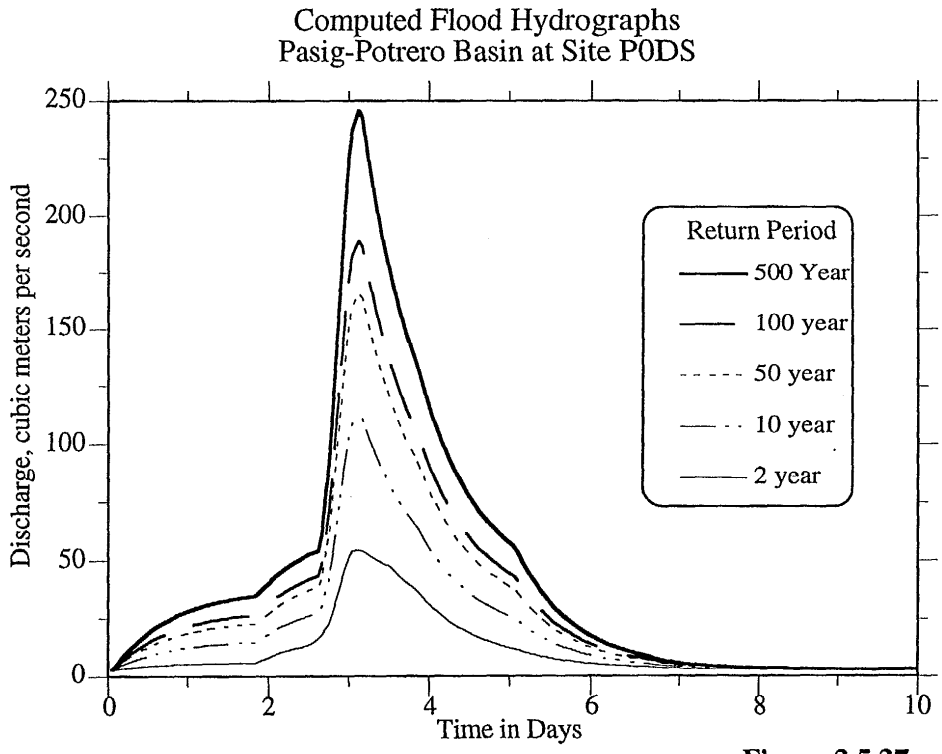

Computed Flood Hydrographs
Pasig-Potrero Basin at Site P0DS
Discharge, cubic meters per second
250
200
150
100
50
0
0
2
4
6
8
10
Time in Days
Return Period
500 Year
100 year
50 year
10 year
2 year

Figure 3.5.37

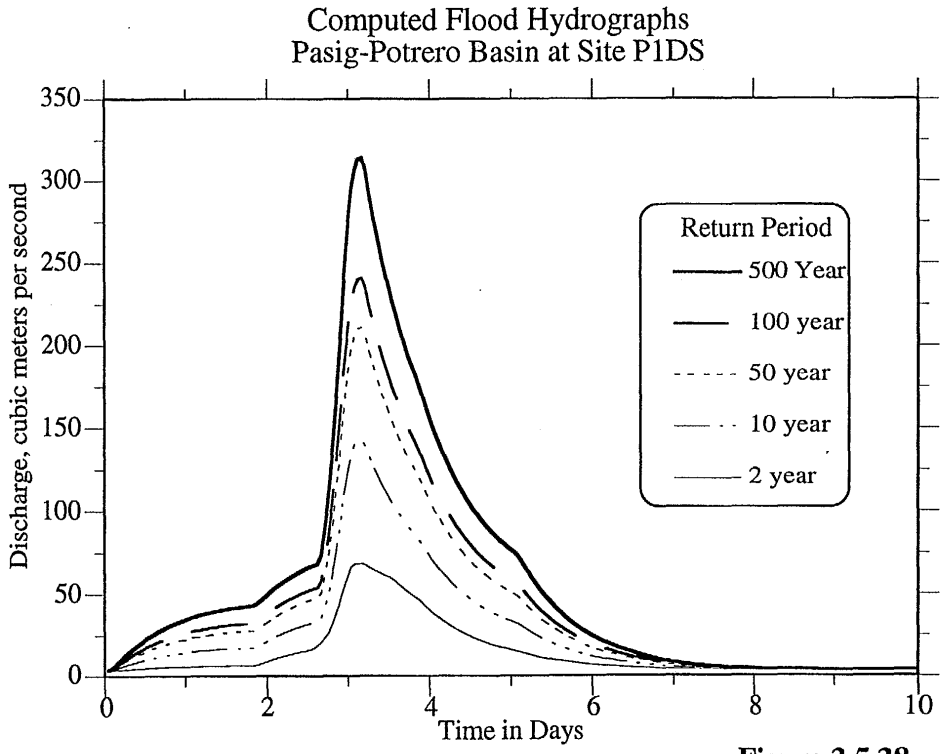

Computed Flood Hydrographs
Pasig-Potrero Basin at Site P1DS
Discharge, cubic meters per second
350
300
250
200
150
100
50
0
0
2
4
6
8
10
Time in Days
Return Period
500 Year
100 year
50 year
10 year
2 year

Figure 3.5.38

Computed Flood Hydrographs
Pasig-Potrero Basin at Site P2DS

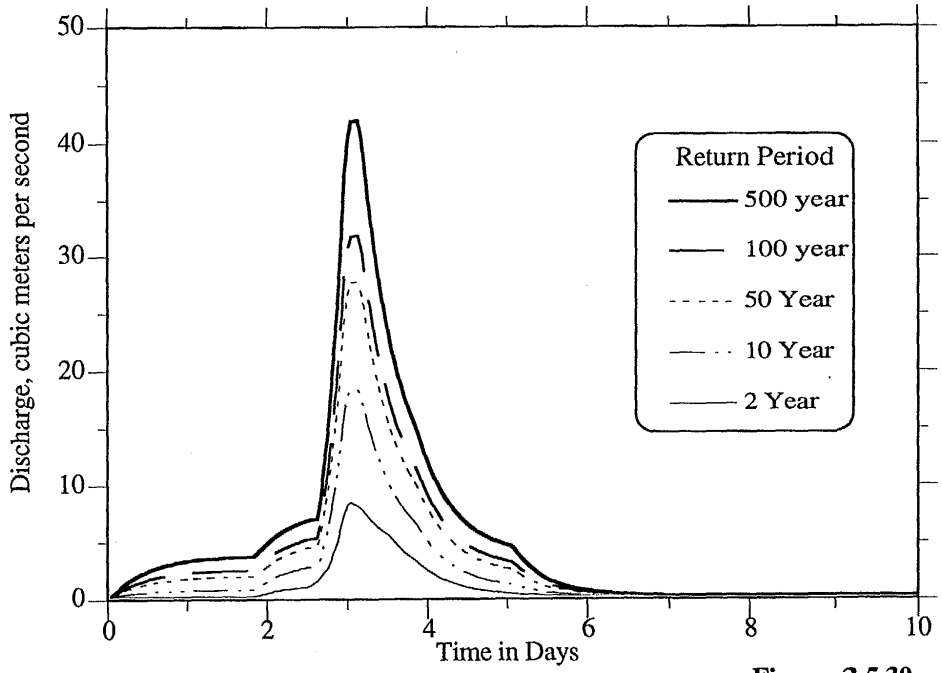


Figure 3.5.39

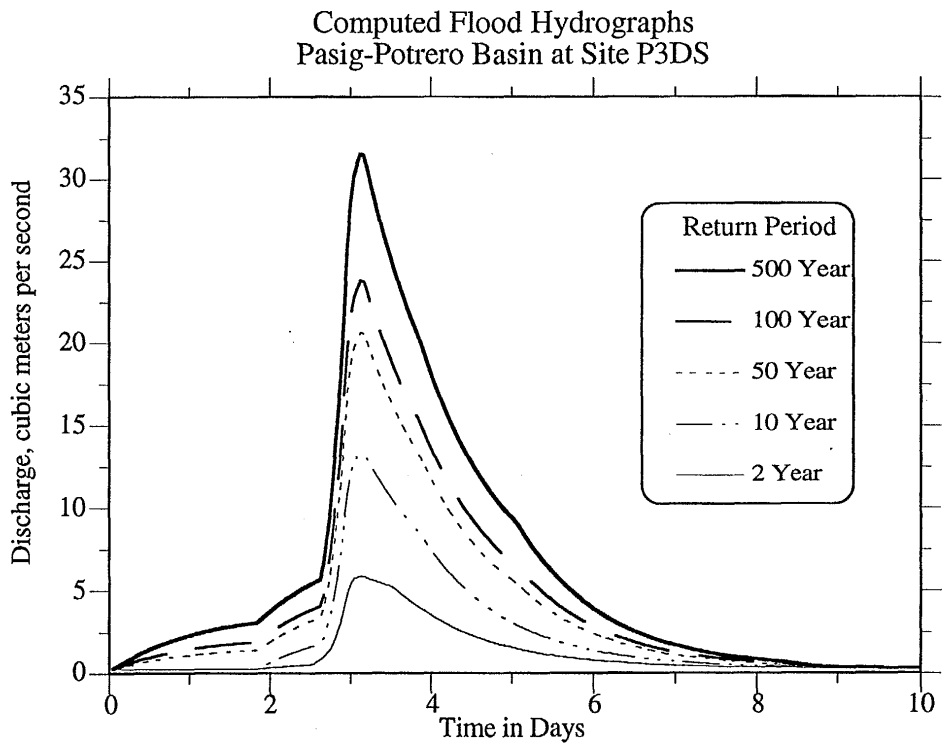

Computed Flood Hydrographs
Pasig-Potrero Basin at Site P3DS
Discharge, cubic meters per second
35
30
25
20
15
10
5
0
0
2
4
6
8
10
Time in Days
Return Period
500 Year
100 Year
50 Year
10 Year
2 Year

Figure 3.5.40

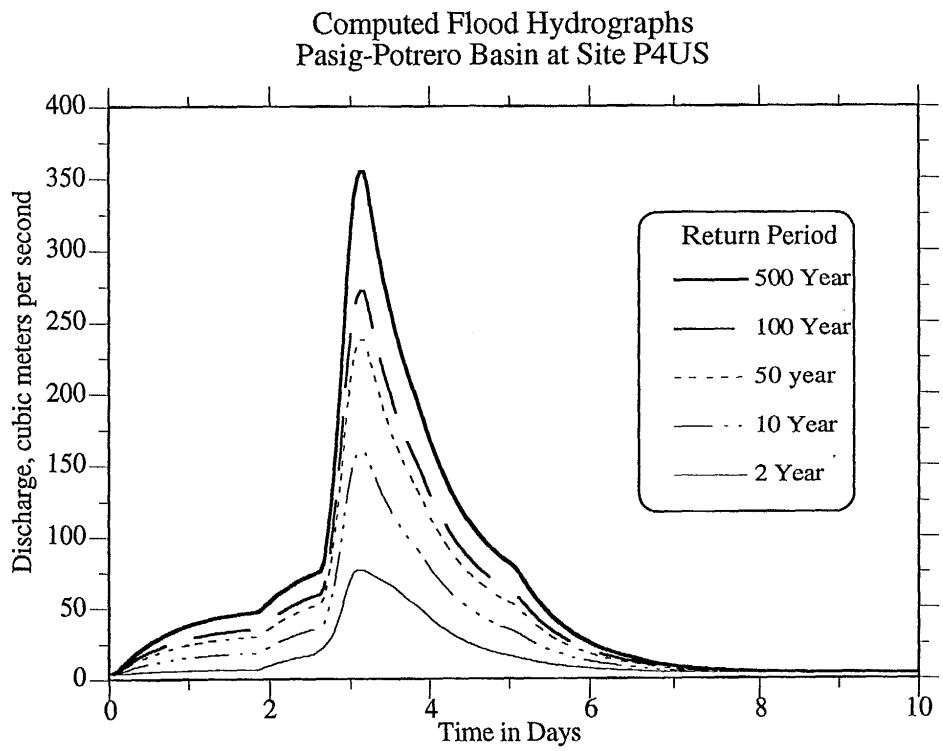

Computed Flood Hydrographs
Pasig-Potrero Basin at Site P4US
Discharge, cubic meters per second
400
350
300
250
200
150
100
50
0
0
2
4
6
8
10
Time in Days
Return Period
500 Year
100 Year
50 year
10 Year
2 Year

Figure 3.5.41

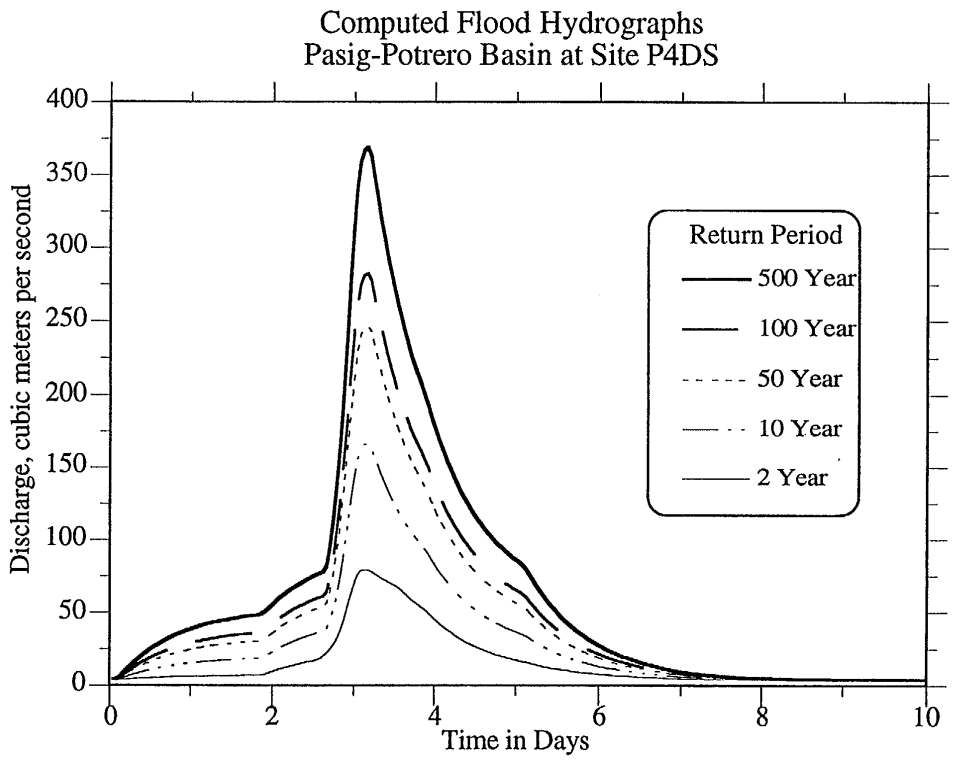

Computed Flood Hydrographs
Pasig-Potrero Basin at Site P4DS
Discharge, cubic meters per second
400
350
300
250
200
150
100
50
0
0
2
4
6
8
10
Time in Days
Return Period
500 Year
100 Year
50 Year
10 Year
2 Year

Figure 3.5.42

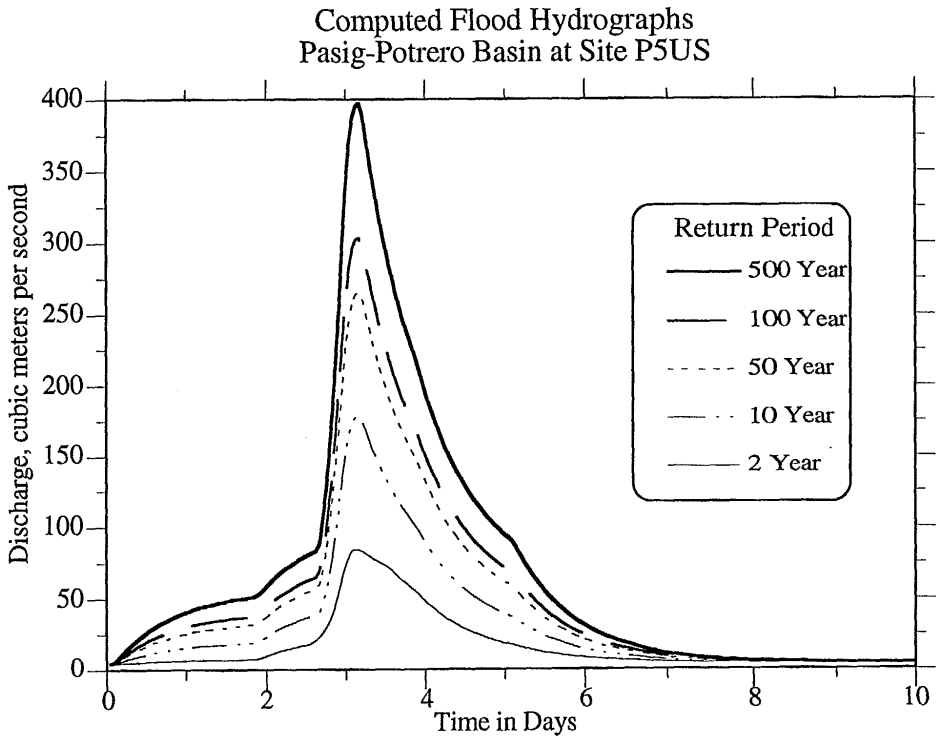

Computed Flood Hydrographs
Pasig-Potrero Basin at Site P5US
Discharge, cubic meters per second
400
350
300
250
200
150
100
50
0
0
2
4
6
8
10
Time in Days
Return Period
500 Year
100 Year
50 Year
10 Year
2 Year

Figure 3.5.43

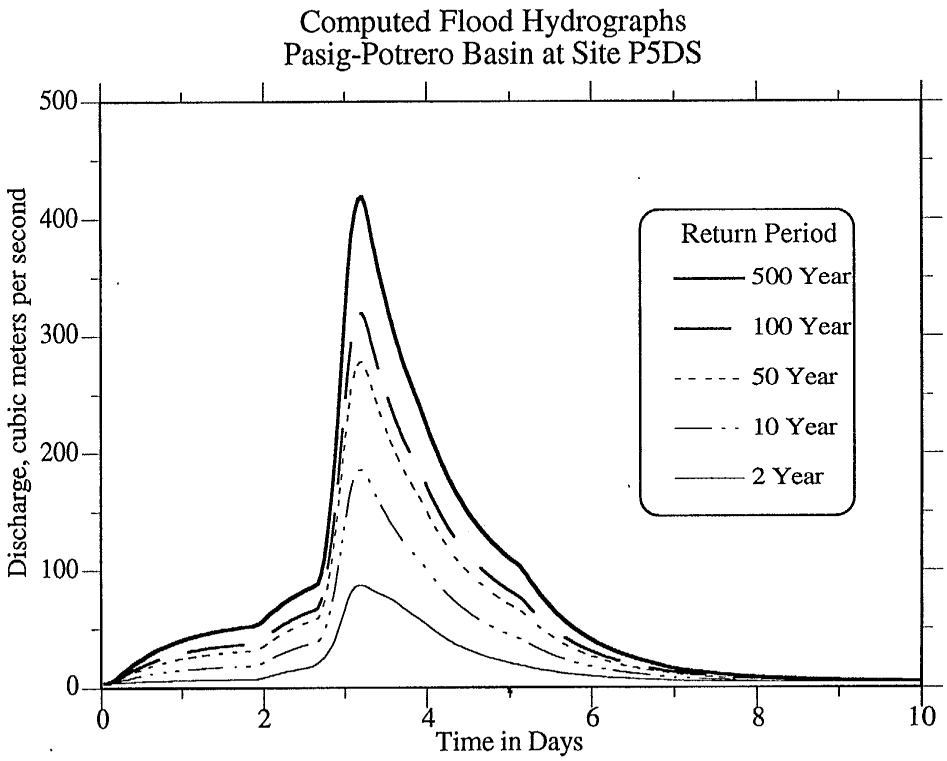


Figure 3.5.44

567

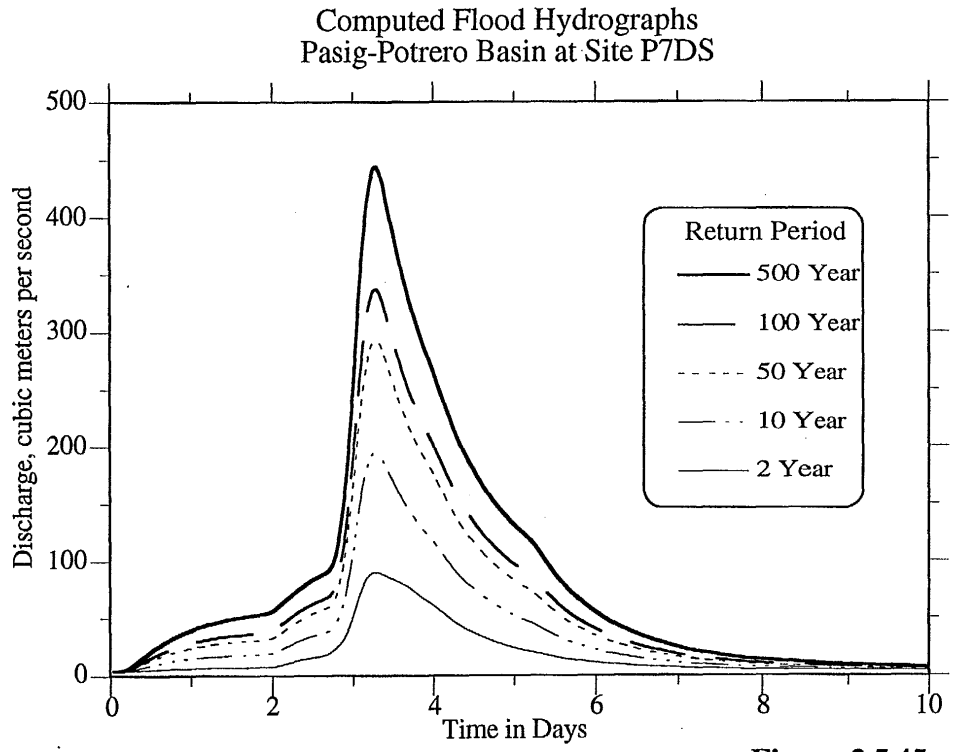

Computed Flood Hydrographs
Pasig-Potrero Basin at Site P7DS
Discharge, cubic meters per second
500
400
300
200
100
0
0
2
4
6
8
10
Time in Days
Return Period
500 Year
100 Year
50 Year
10 Year
2 Year

Figure 3.5.45

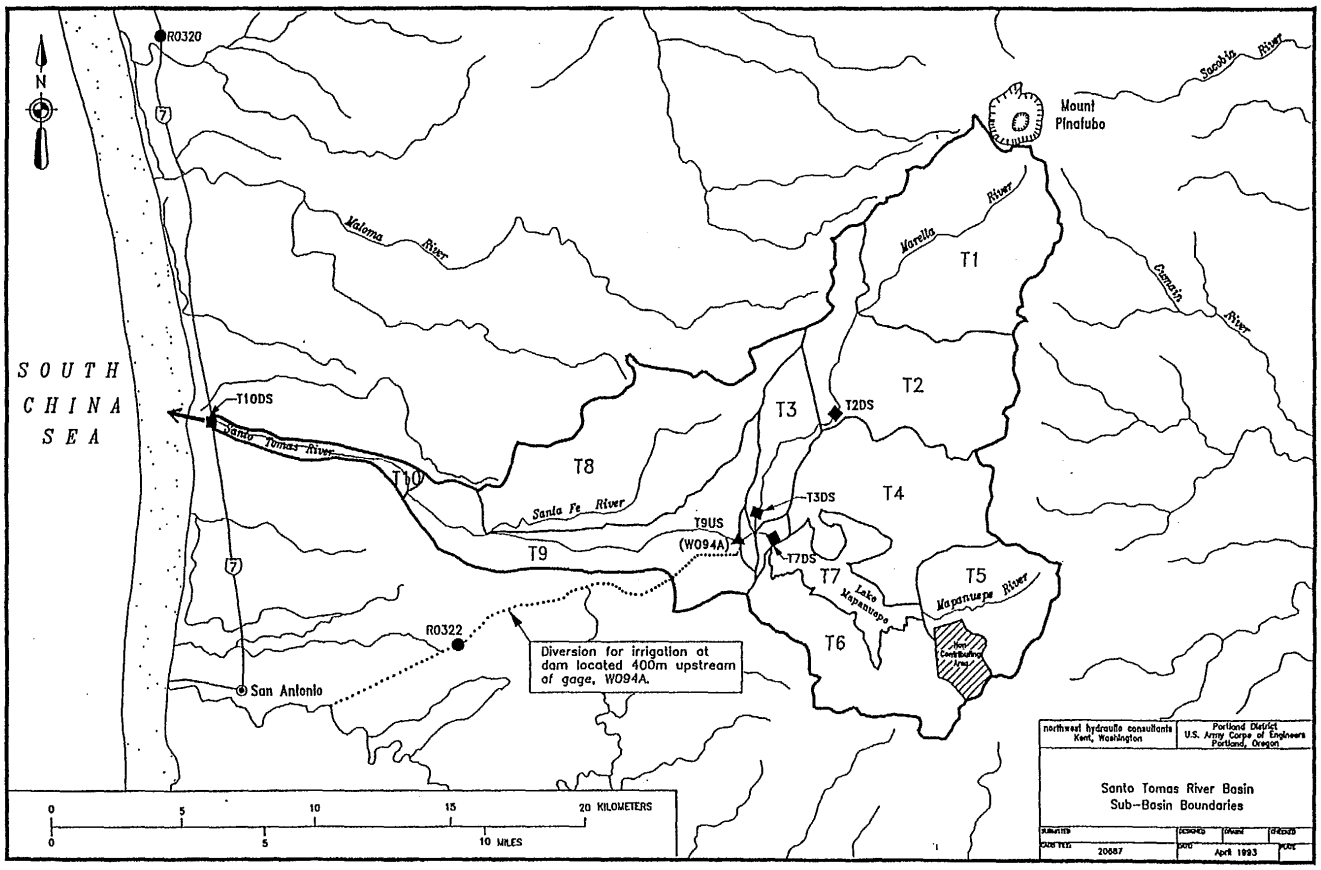

R0320
N
7
Mount Pinatubo
Sacobia River
Maloma River
Marella River
T1
Camain River
SOUTH CHINA SEA
T10DS
Santo Tomas River
T10
T8
T3
T2DS
T2
Santa Fe River
T3DS
T4
T9US
(W094A)
T9
T7DS
T7
Lake Napantuepe
T5
Napantuepe River
T6
R0322
Diversion for irrigation at dam located 400m upstream of gage, W094A.
San Antonio
0 5 10 15 20 KILOMETERS
0 5 10 MILES
northwest hydraulic consultants Kent, Washington
Portland District U.S. Army Corps of Engineers Portland, Oregon
Santo Tomas River Basin Sub-Basin Boundaries
20087
April 1993

Figure 3.5.46

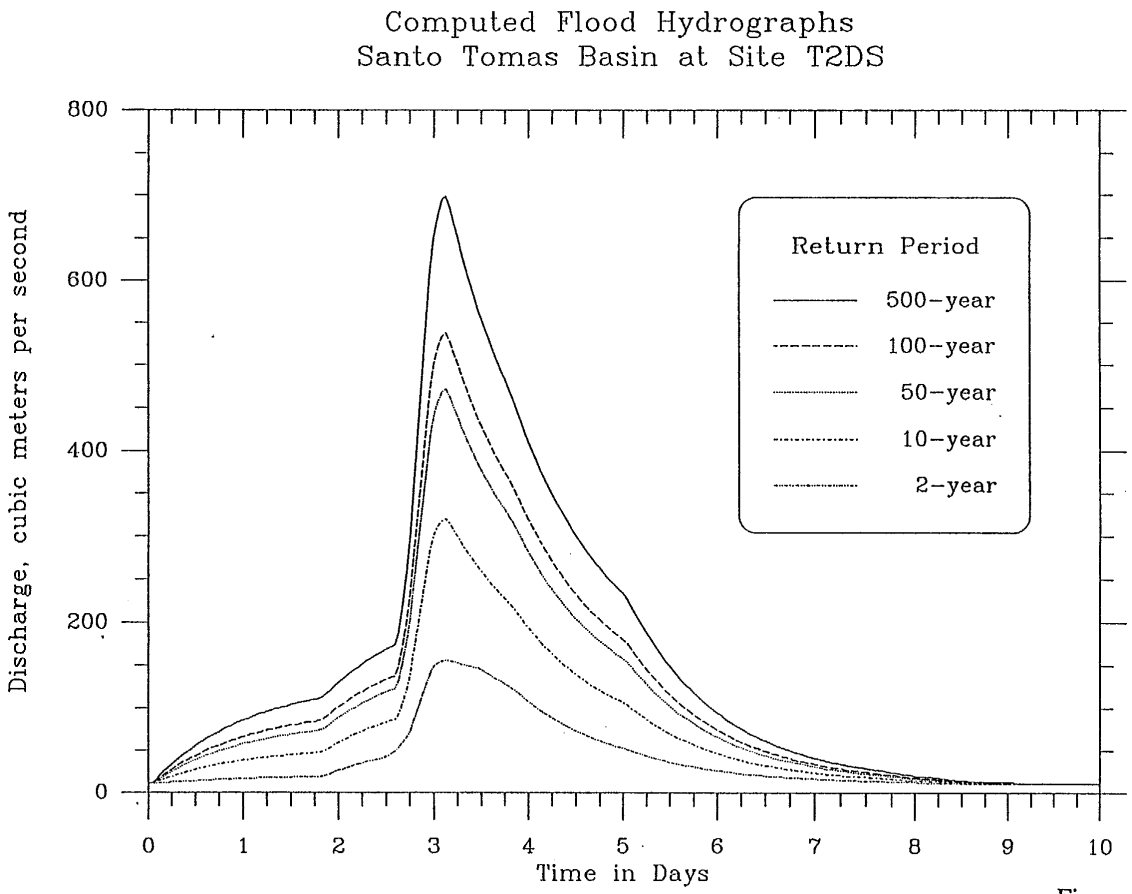

Computed Flood Hydrographs
Santo Tomas Basin at Site T2DS
Discharge, cubic meters per second
800
600
400
200
0
0
1
2
3
4
5
6
7
8
9
10
Time in Days
Return Period
500-year
100-year
50-year
10-year
2-year

Figure 3.5.47

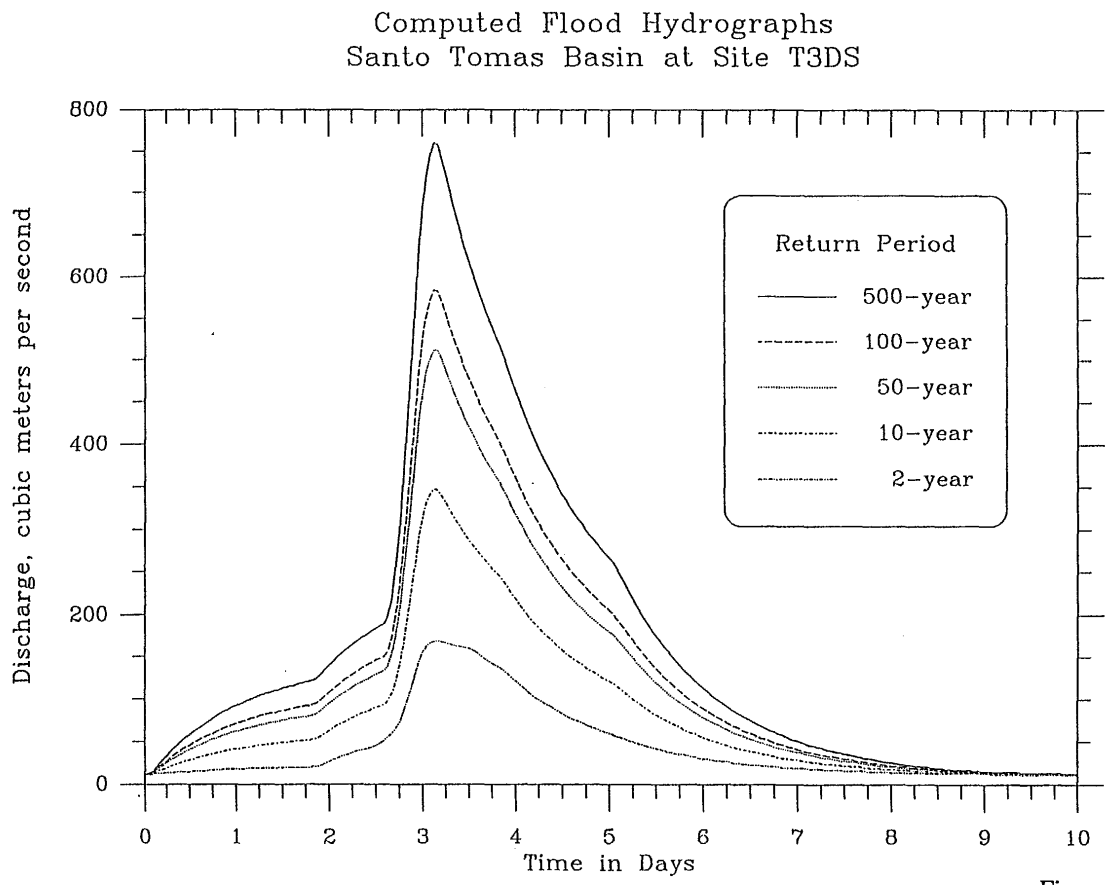

Computed Flood Hydrographs
Santo Tomas Basin at Site T3DS
Discharge, cubic meters per second
800
600
400
200
0
0
1
2
3
4
5
6
7
8
9
10
Time in Days
Return Period
500-year
100-year
50-year
10-year
2-year

Figure 3.5.48

Computed Flood Hydrographs
Santo Tomas Basin at Site T7DS

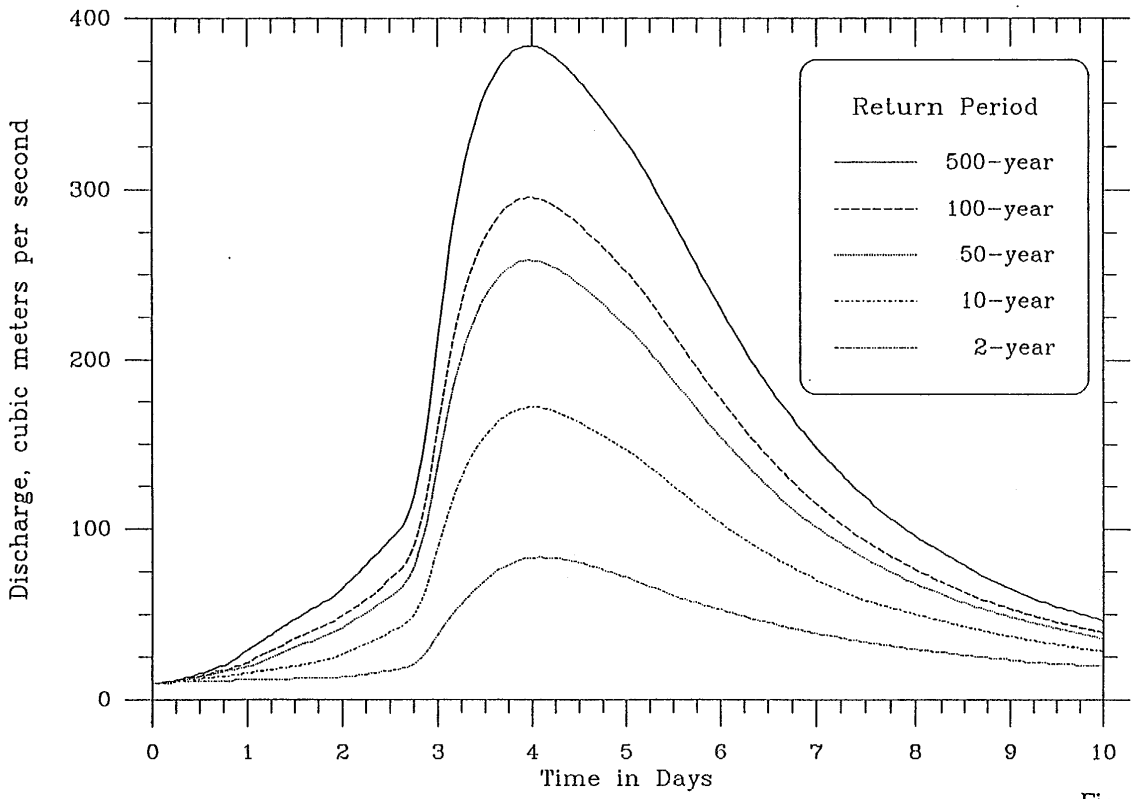


Figure 3.5.49

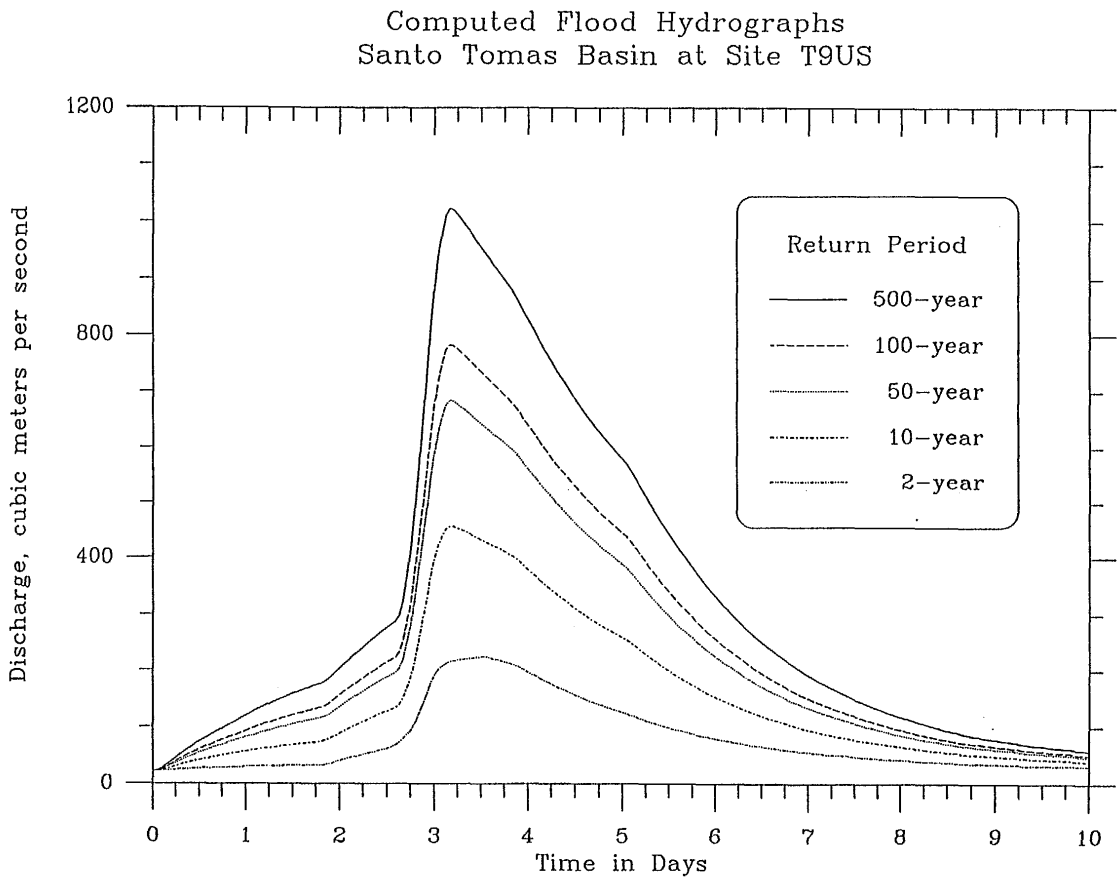

Computed Flood Hydrographs
Santo Tomas Basin at Site T9US
Discharge, cubic meters per second
1200
800
400
0
0
1
2
3
4
5
6
7
8
9
10
Time in Days
Return Period
500-year
100-year
50-year
10-year
2-year

Figure 3.5.50

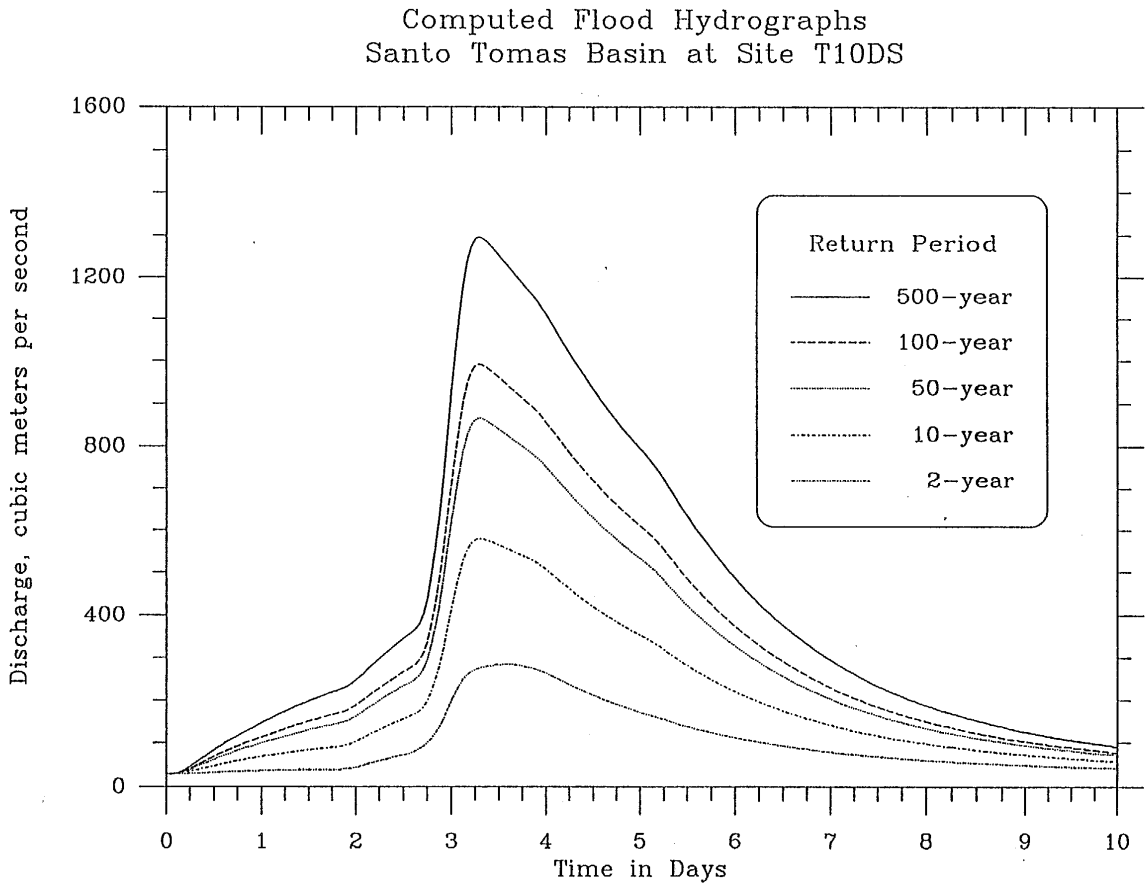

Computed Flood Hydrographs
Santo Tomas Basin at Site T10DS
Discharge, cubic meters per second
1600
1200
800
400
0
0
1
2
3
4
5
6
7
8
9
10
Time in Days
Return Period
500-year
100-year
50-year
10-year
2-year

Figure 3.5.51

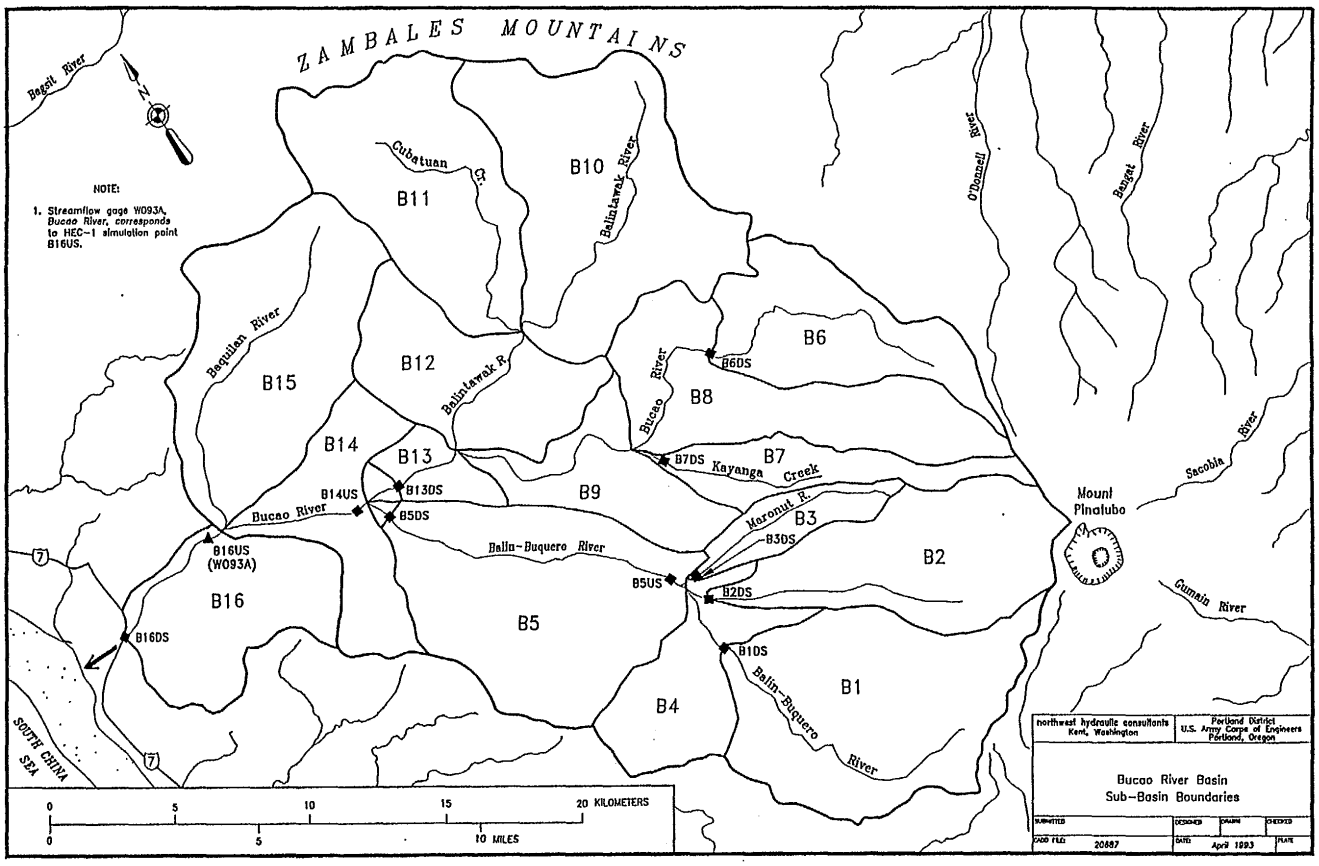


Figure 3.5.52

576

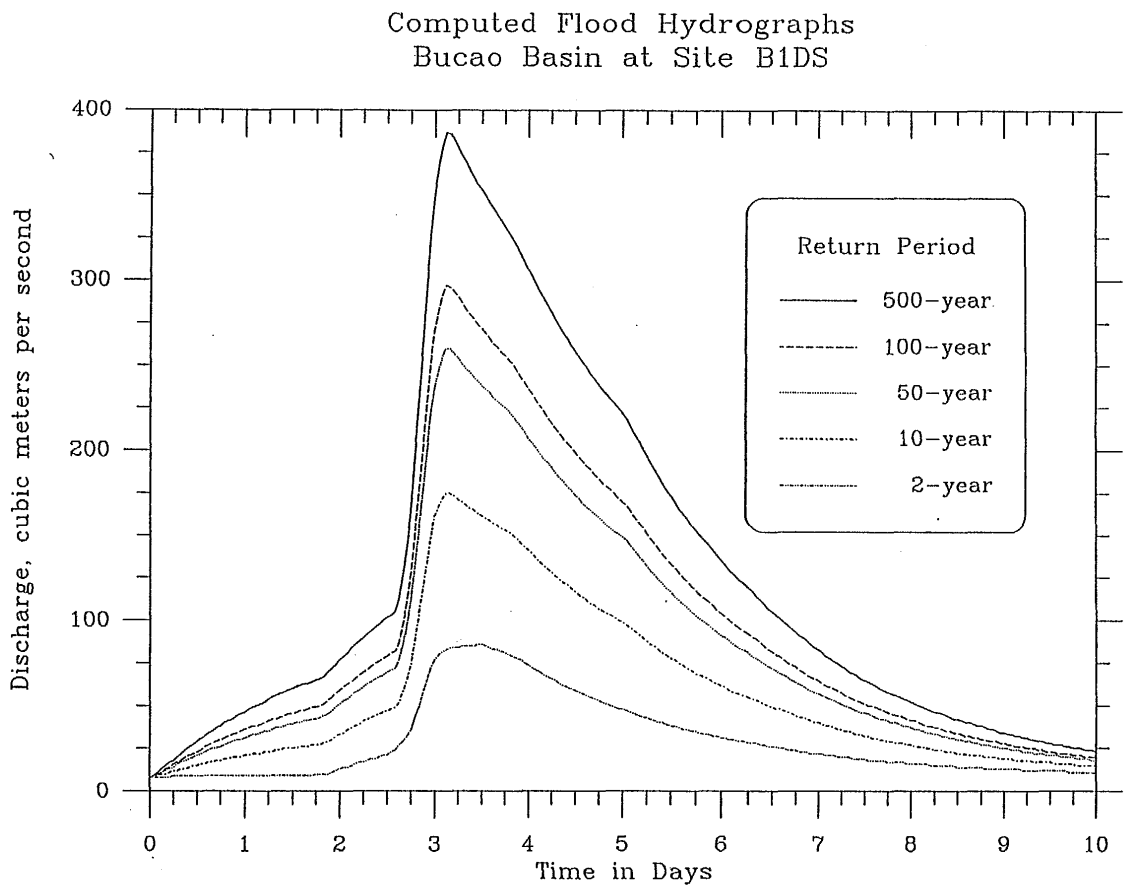

Computed Flood Hydrographs
Bucao Basin at Site B1DS
Discharge, cubic meters per second
400
300
200
100
0
0
1
2
3
4
5
6
7
8
9
10
Time in Days
Return Period
500-year
100-year
50-year
10-year
2-year

Figure 3.5.53

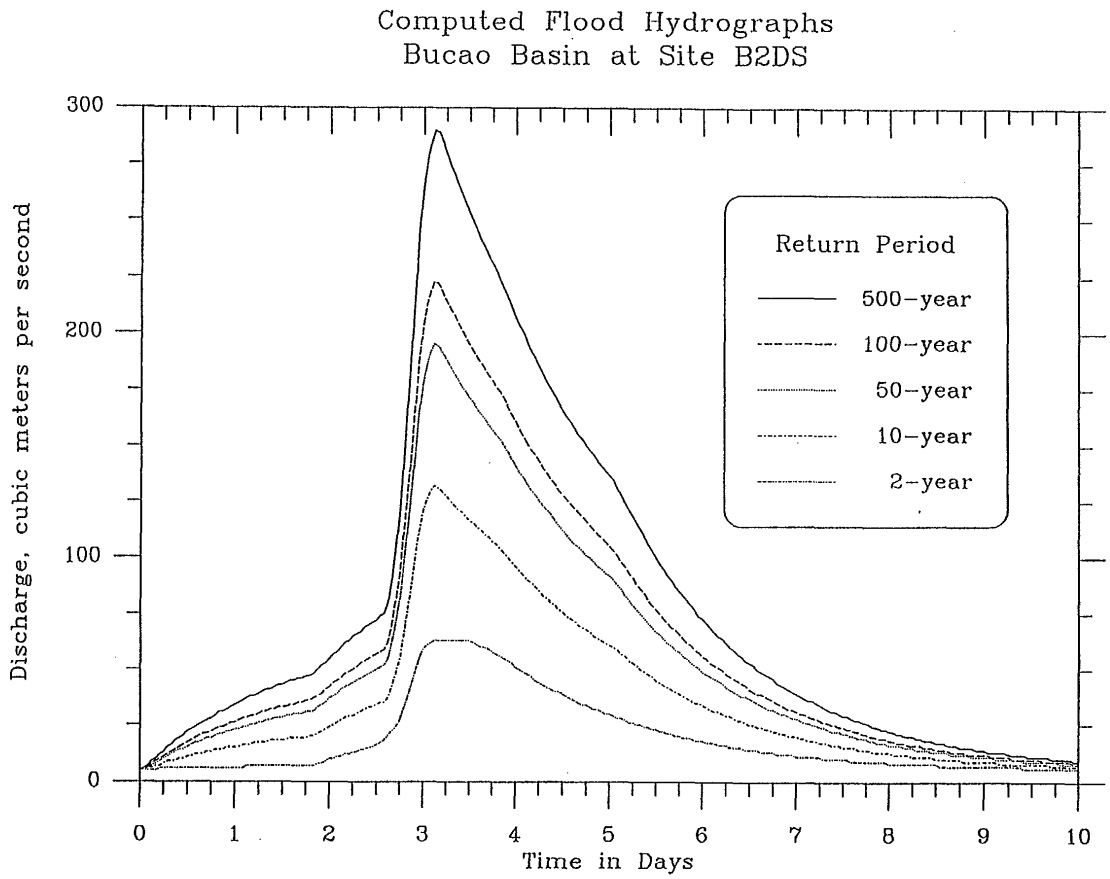

Computed Flood Hydrographs
Bucao Basin at Site B2DS
Discharge, cubic meters per second
300
200
100
0
0
1
2
3
4
5
6
7
8
9
10
Time in Days
Return Period
500-year
100-year
50-year
10-year
2-year

Figure 3.5.54

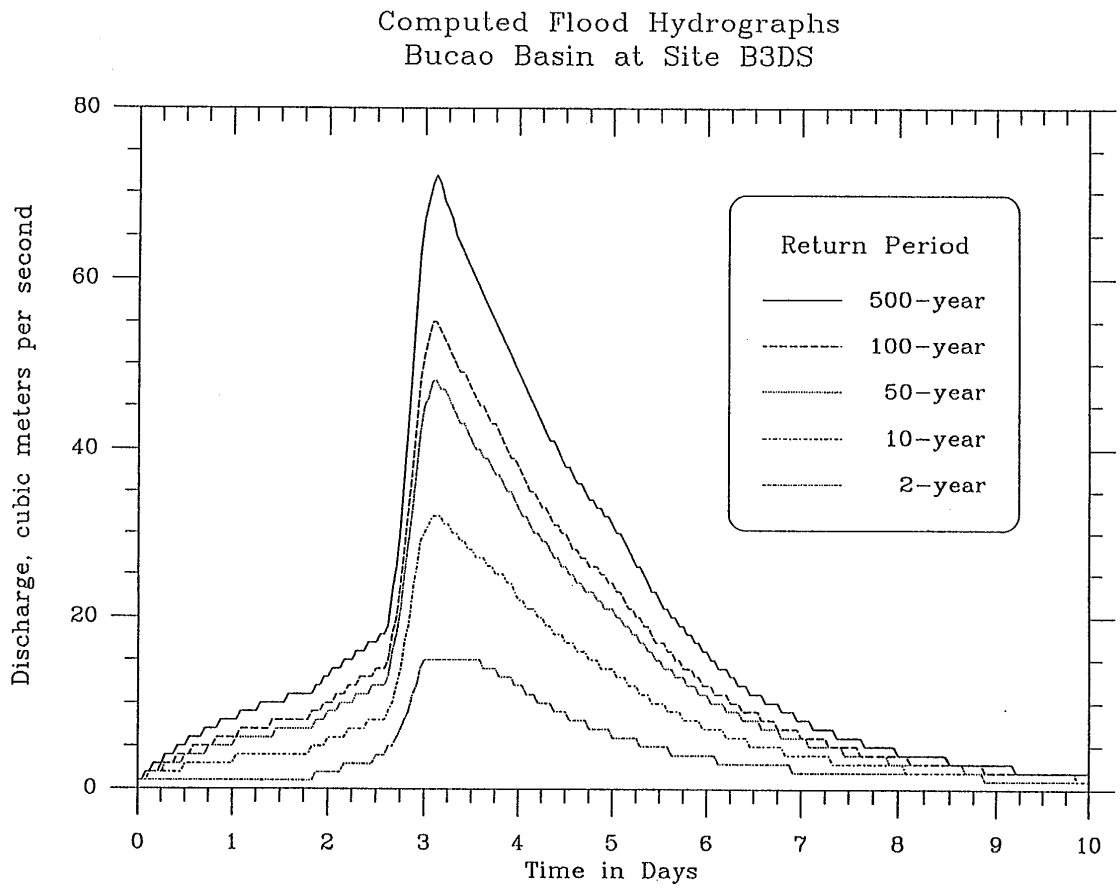


Figure 3.5.55

578

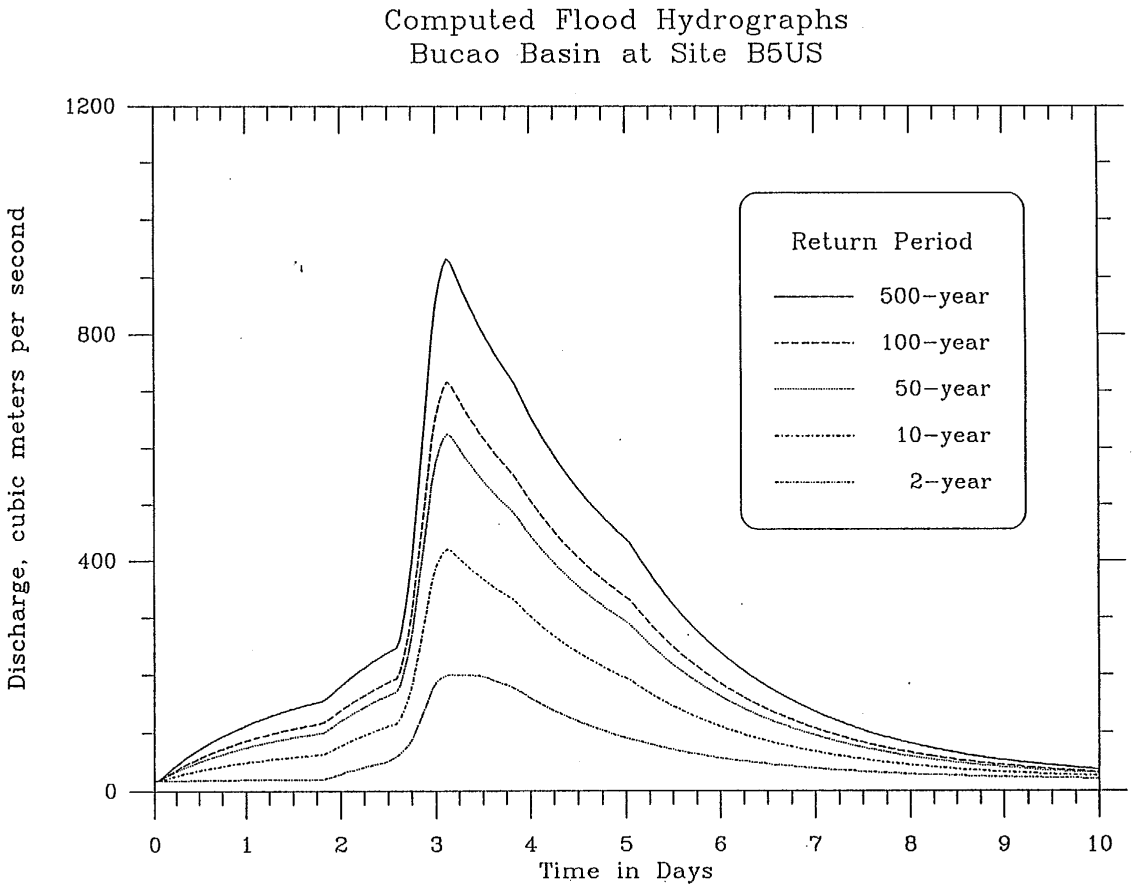


Figure 3.5.56

679

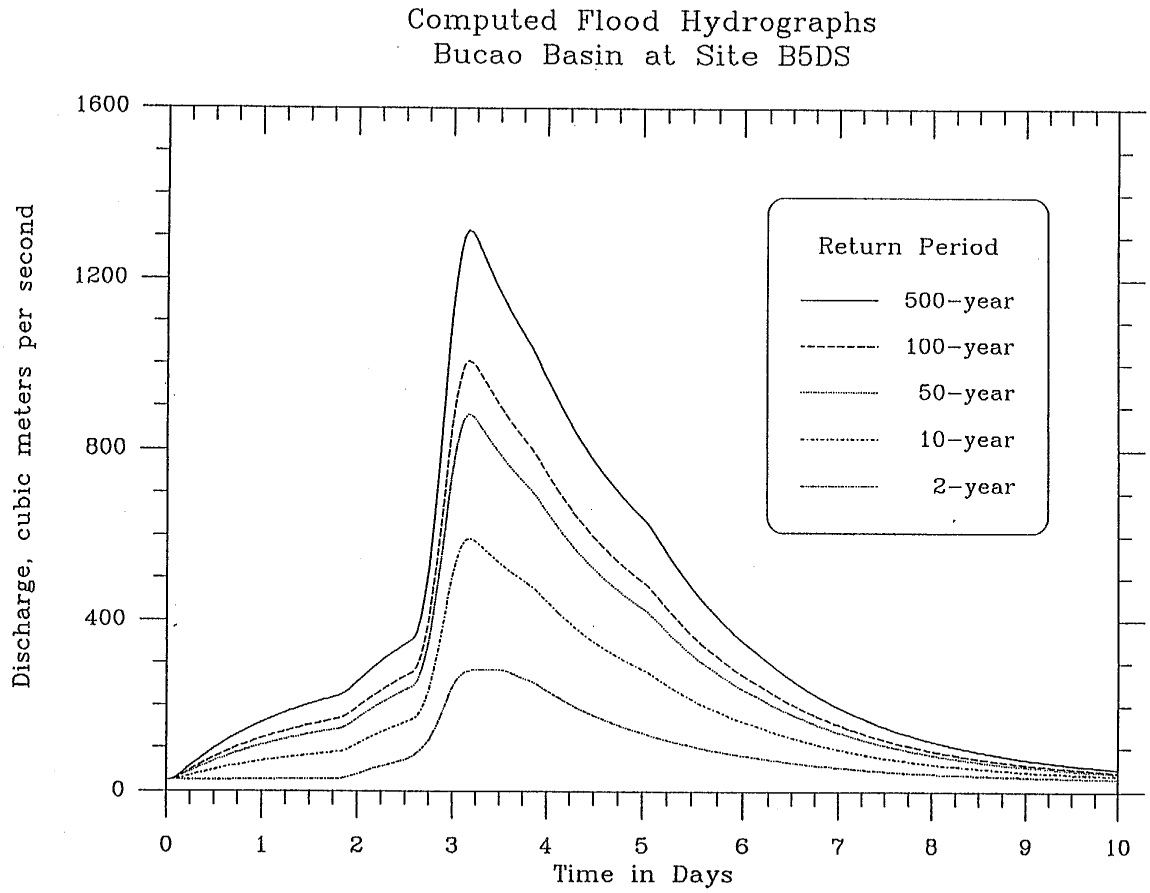

Computed Flood Hydrographs
Bucao Basin at Site B5DS
Discharge, cubic meters per second
1600
1200
800
400
0
0
1
2
3
4
5
6
7
8
9
10
Time in Days
Return Period
500-year
100-year
50-year
10-year
2-year

Figure 3.5.57

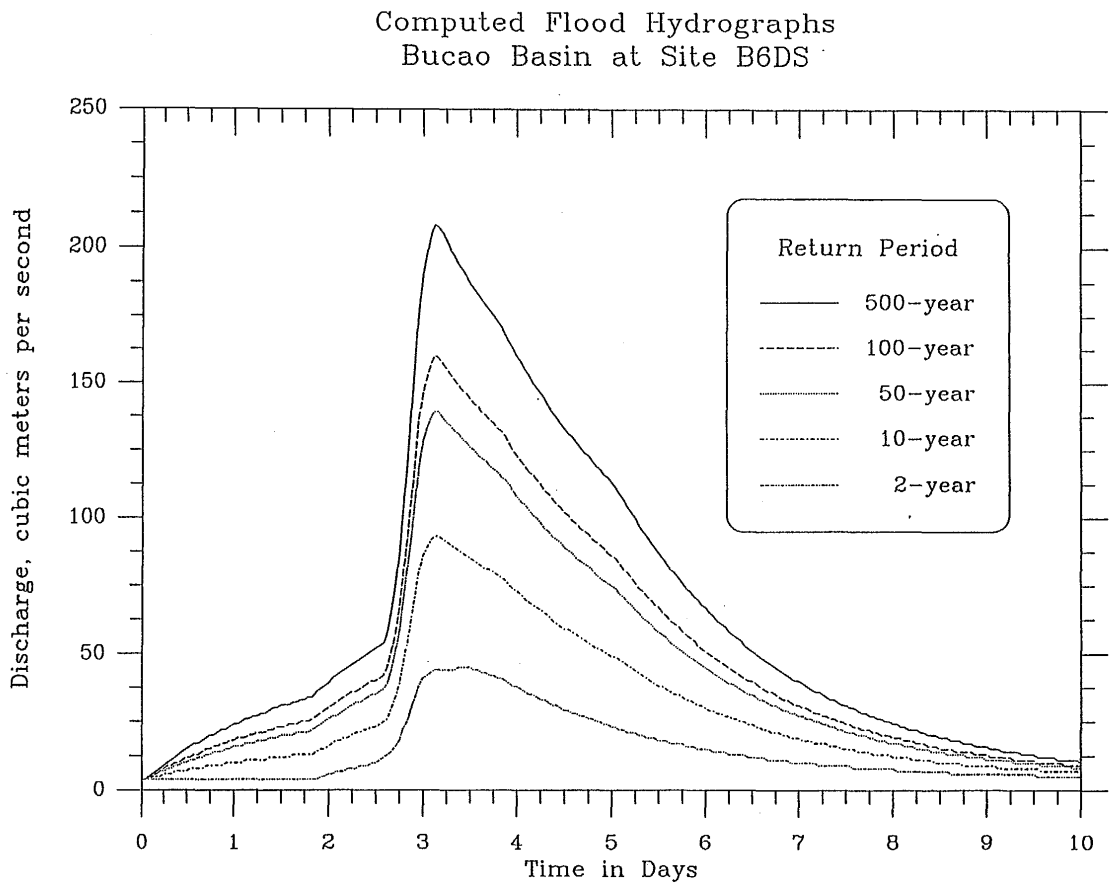

Computed Flood Hydrographs
Bucao Basin at Site B6DS
Discharge, cubic meters per second
250
200
150
100
50
0
0
1
2
3
4
5
6
7
8
9
10
Time in Days
Return Period
500-year
100-year
50-year
10-year
2-year

Figure 3.5.58

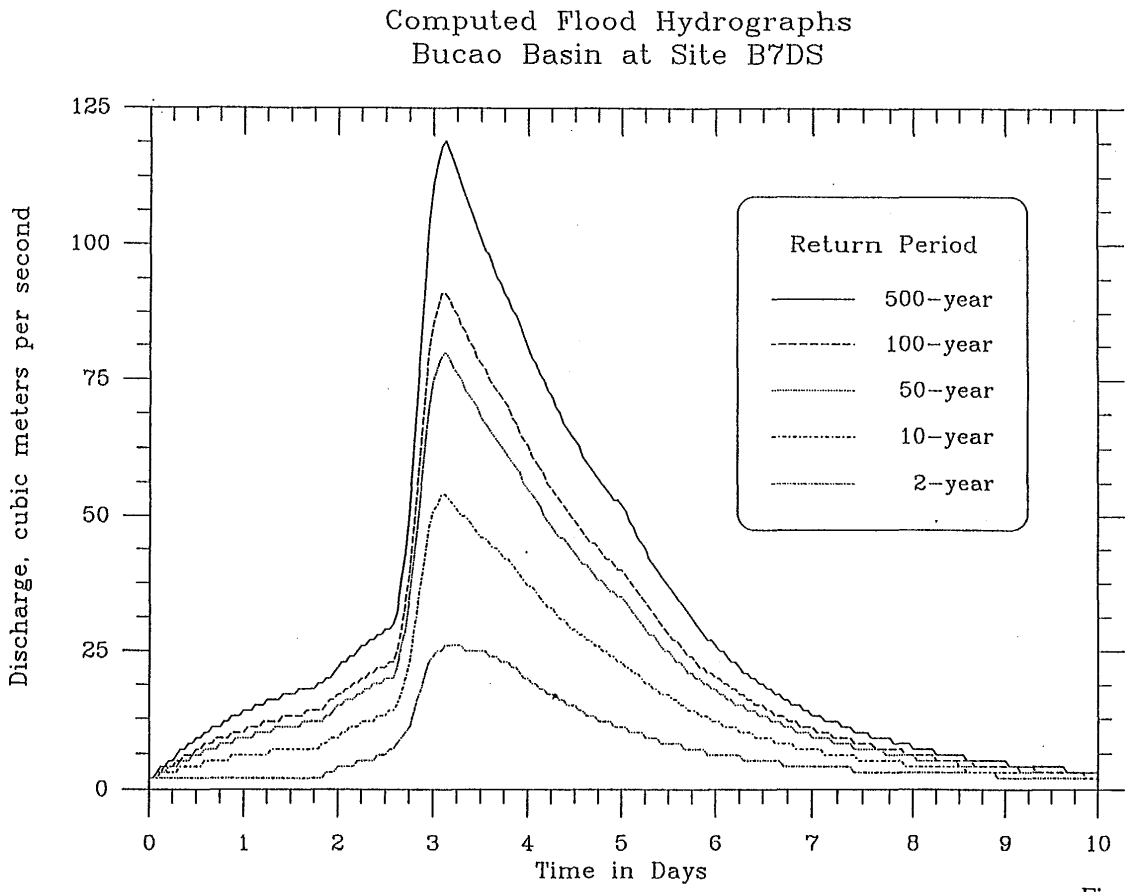

Computed Flood Hydrographs
Bucao Basin at Site B7DS
Discharge, cubic meters per second
125
100
75
50
25
0
0
1
2
3
4
5
6
7
8
9
10
Time in Days
Return Period
500-year
100-year
50-year
10-year
2-year

Figure 3.5.59

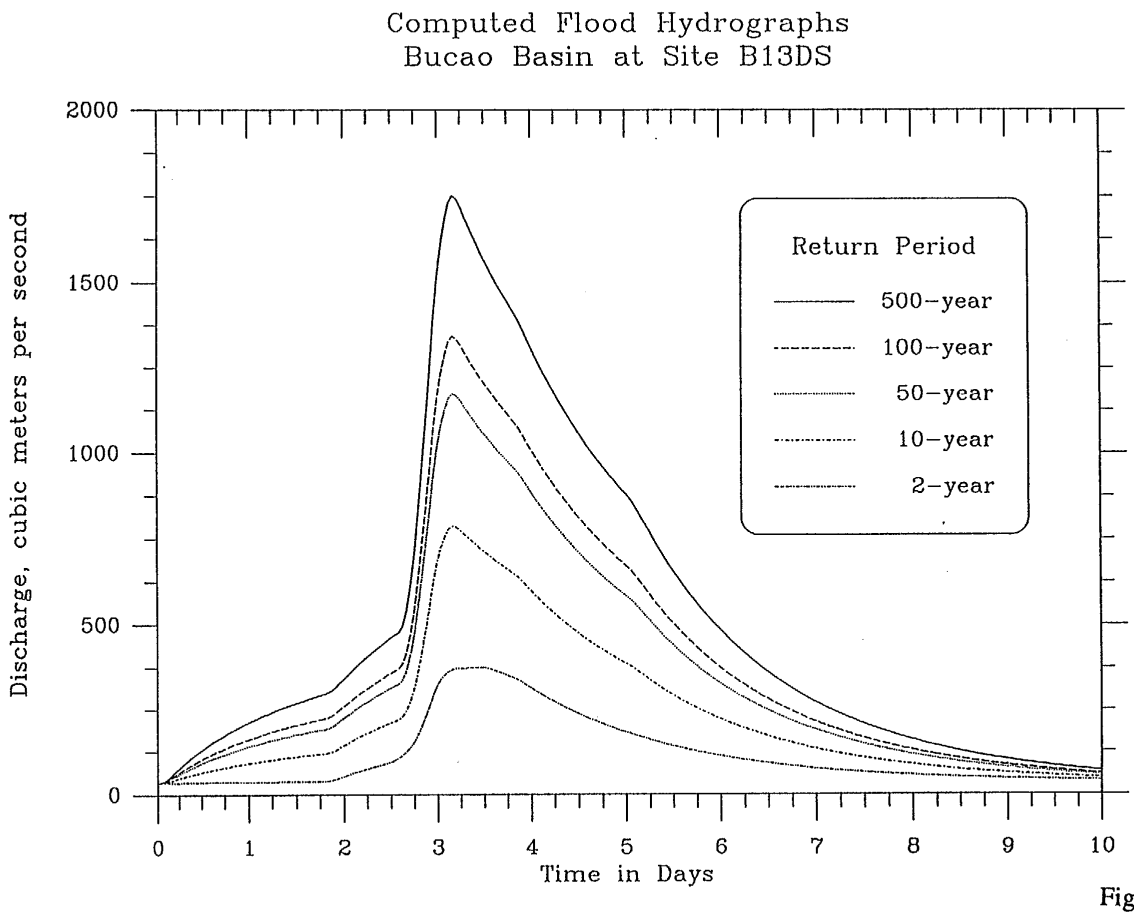

Computed Flood Hydrographs
Bucao Basin at Site B13DS
Discharge, cubic meters per second
2000
1500
1000
500
0
0
1
2
3
4
5
6
7
8
9
10
Time in Days
Return Period
500-year
100-year
50-year
10-year
2-year

Figure 3.5.60

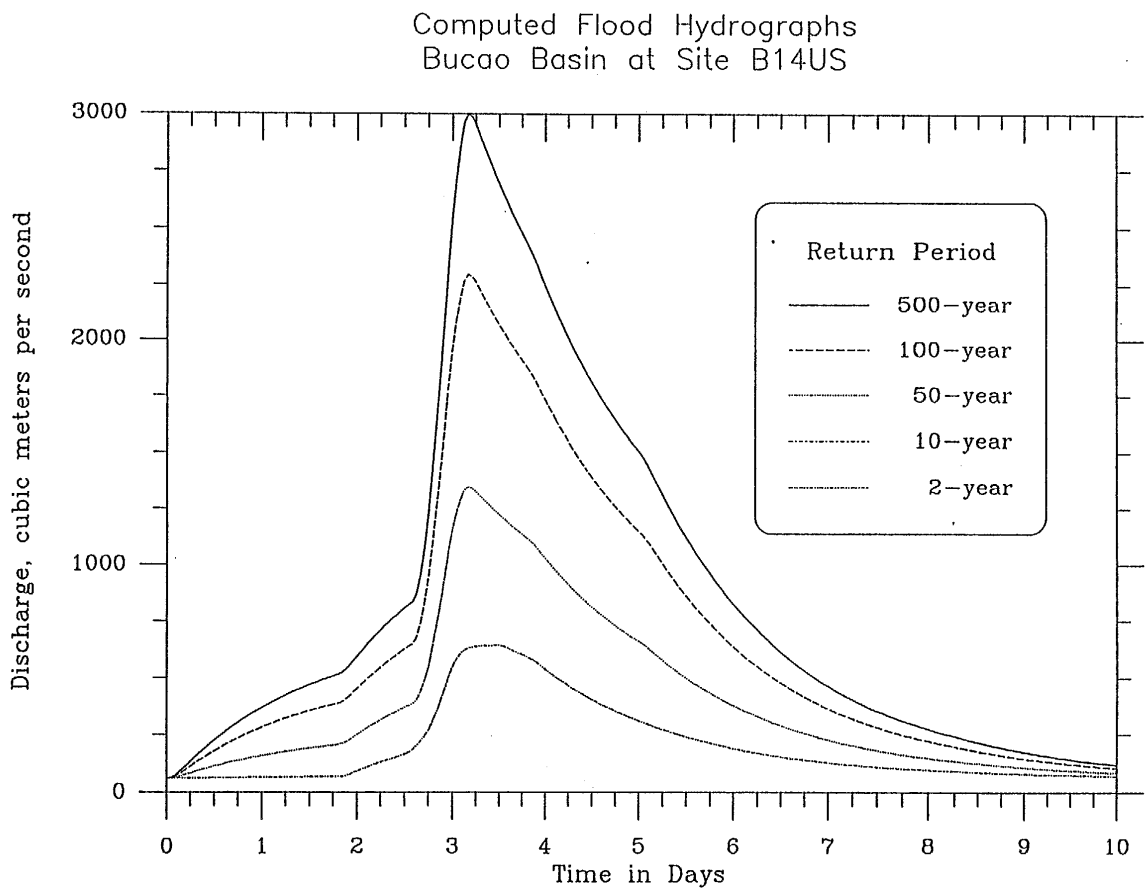

Computed Flood Hydrographs
Bucao Basin at Site B14US
Discharge, cubic meters per second
3000
2000
1000
0
0
1
2
3
4
5
6
7
8
9
10
Time in Days
Return Period
500-year
100-year
50-year
10-year
2-year

Figure 3.5.61

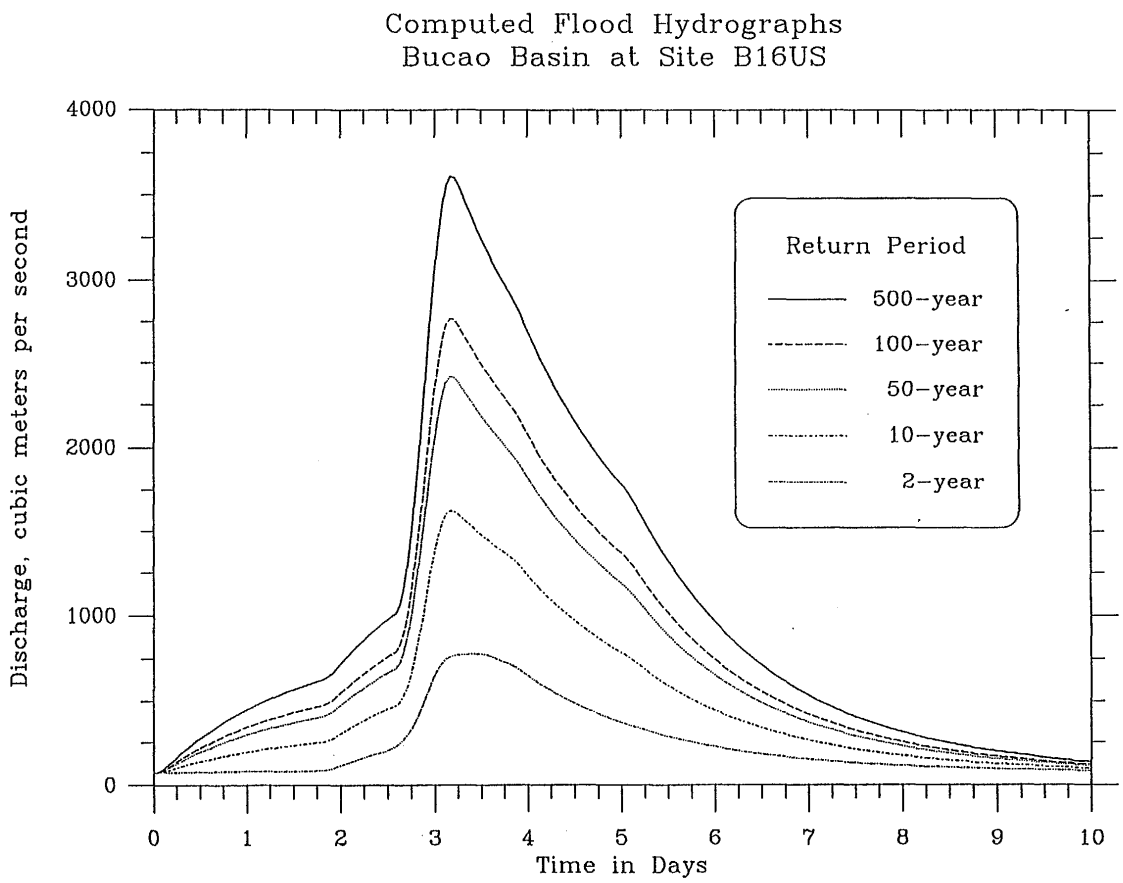

Computed Flood Hydrographs
Bucao Basin at Site B16US
Discharge, cubic meters per second
4000
3000
2000
1000
0
0
1
2
3
4
5
6
7
8
9
10
Time in Days
Return Period
500-year
100-year
50-year
10-year
2-year

Figure 3.5.62

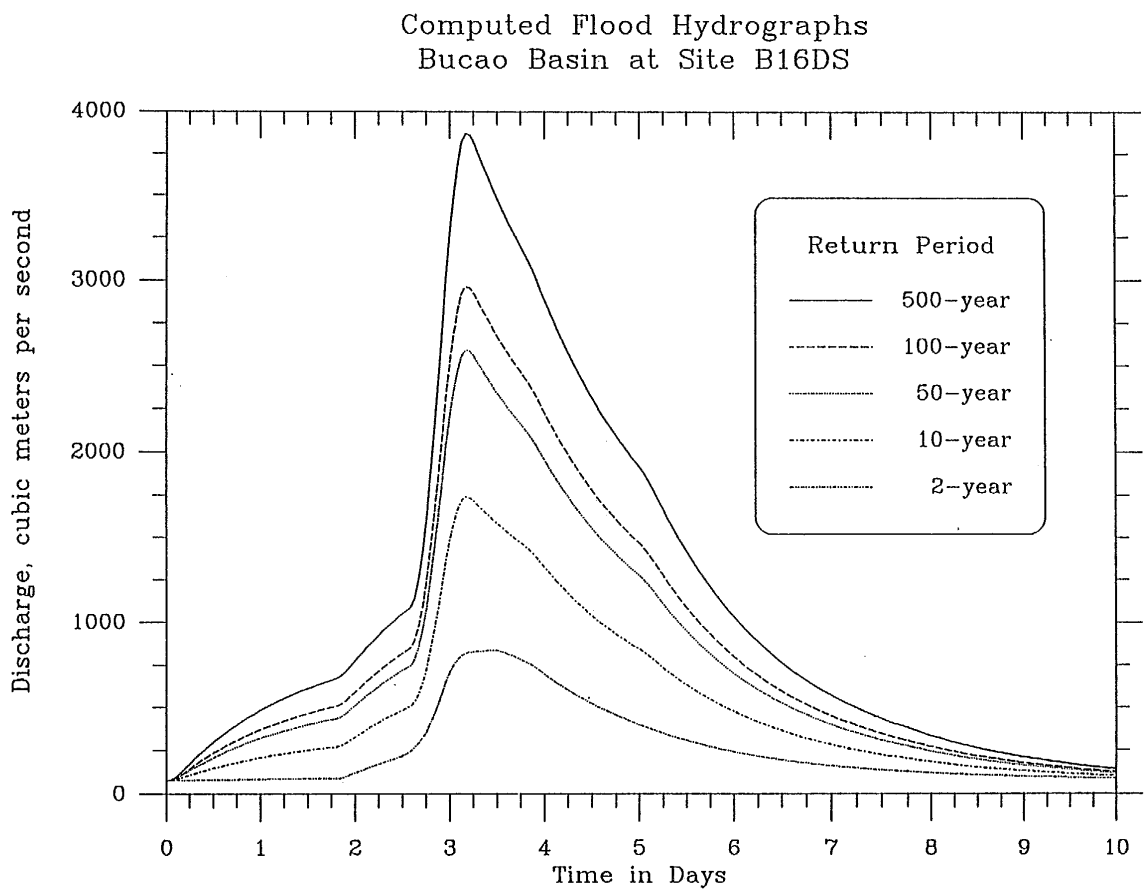

Computed Flood Hydrographs
Bucao Basin at Site B16DS
Discharge, cubic meters per second
4000
3000
2000
1000
0
0
1
2
3
4
5
6
7
8
9
10
Time in Days
Return Period
500-year
100-year
50-year
10-year
2-year

Figure 3.5.63

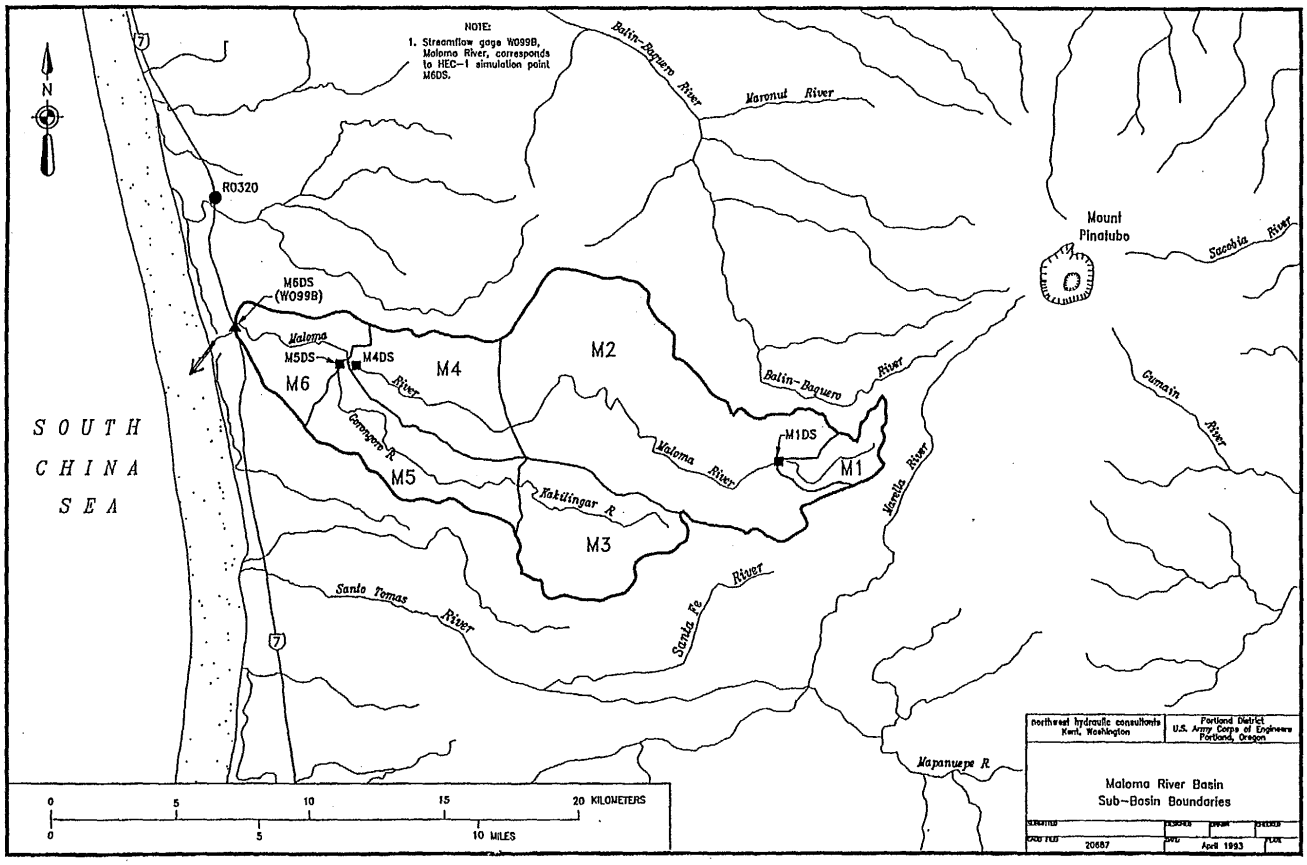

NOTE:
1. Streamflow gage W099B, Maloma River, corresponds to HEC-1 simulation point M6DS.
N
R0320
M6DS
(W099B)
M5DS
M4DS
Maloma
River
M6
M4
M2
M5
M3
M1
M1DS
Balin-Baguero River
Marenui River
Mount
Pinatubo
Sacobia River
Gumain River
Marella River
Maloma River
Tarangero R.
Kakilingar R.
Santa Fe River
Santo Tomas River
Napanuepe R.
SOUTH
CHINA
SEA
0 5 10 15 20 KILOMETERS
0 5 10 MILES
northwest hydraulic consultants
Kent, Washington
Portland District
U.S. Army Corps of Engineers
Portland, Oregon
Maloma River Basin
Sub-Basin Boundaries
20687
April 1993

Figure 3.5.64

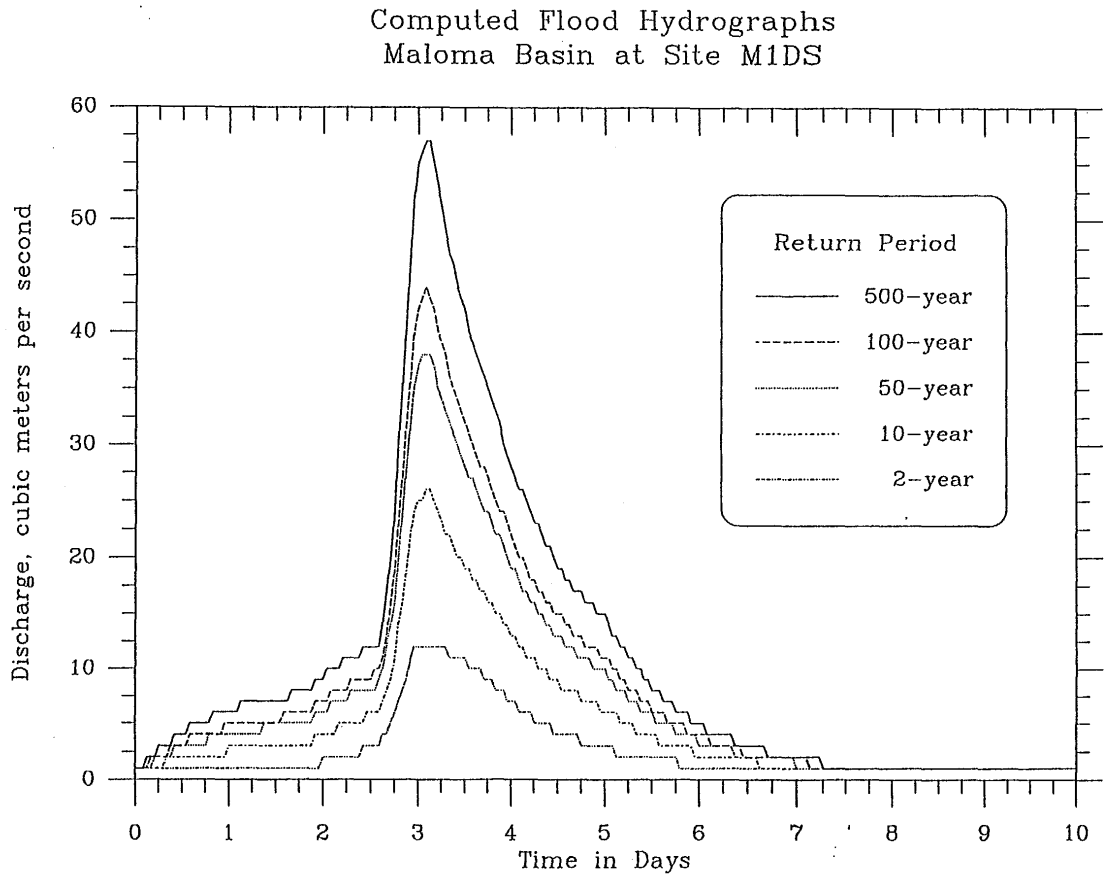

Computed Flood Hydrographs
Maloma Basin at Site M1DS
Discharge, cubic meters per second
Time in Days
Return Period
500-year
100-year
50-year
10-year
2-year
0
10
20
30
40
50
60
0
1
2
3
4
5
6
7
8
9
10

Figure 3.5.65

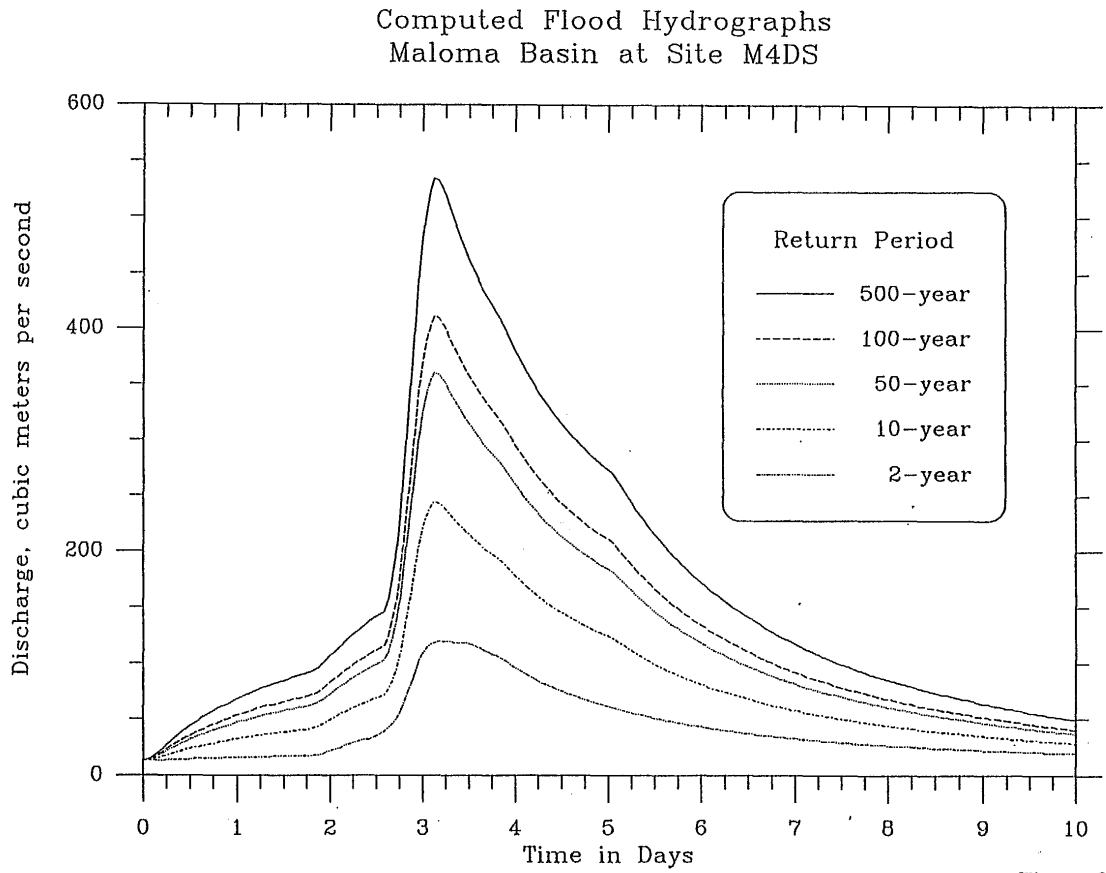

Computed Flood Hydrographs
Maloma Basin at Site M4DS
Discharge, cubic meters per second
600
400
200
0
0
1
2
3
4
5
6
7
8
9
10
Time in Days
Return Period
500-year
100-year
50-year
10-year
2-year

Figure 3.5.66

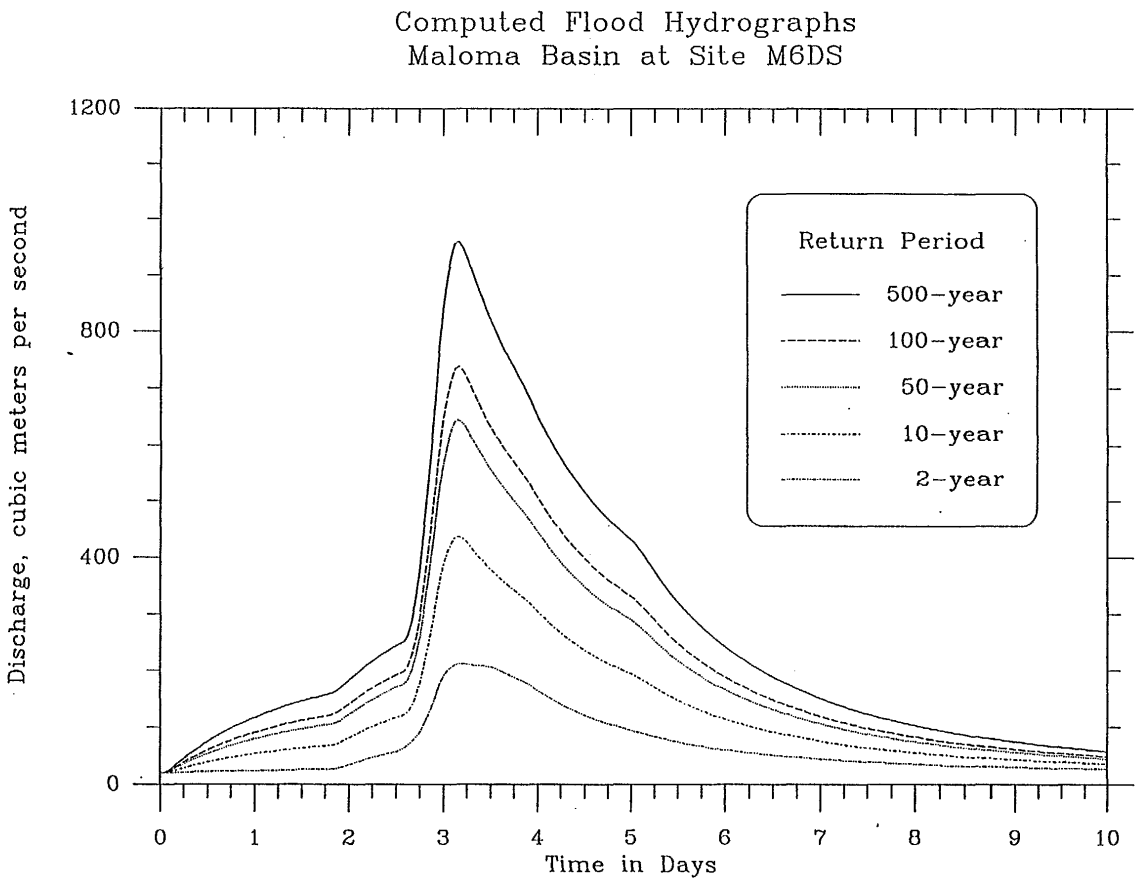

Computed Flood Hydrographs
Maloma Basin at Site M6DS
Discharge, cubic meters per second
1200
800
400
0
0
1
2
3
4
5
6
7
8
9
10
Time in Days
Return Period
500-year
100-year
50-year
10-year
2-year

Figure 3.5.67

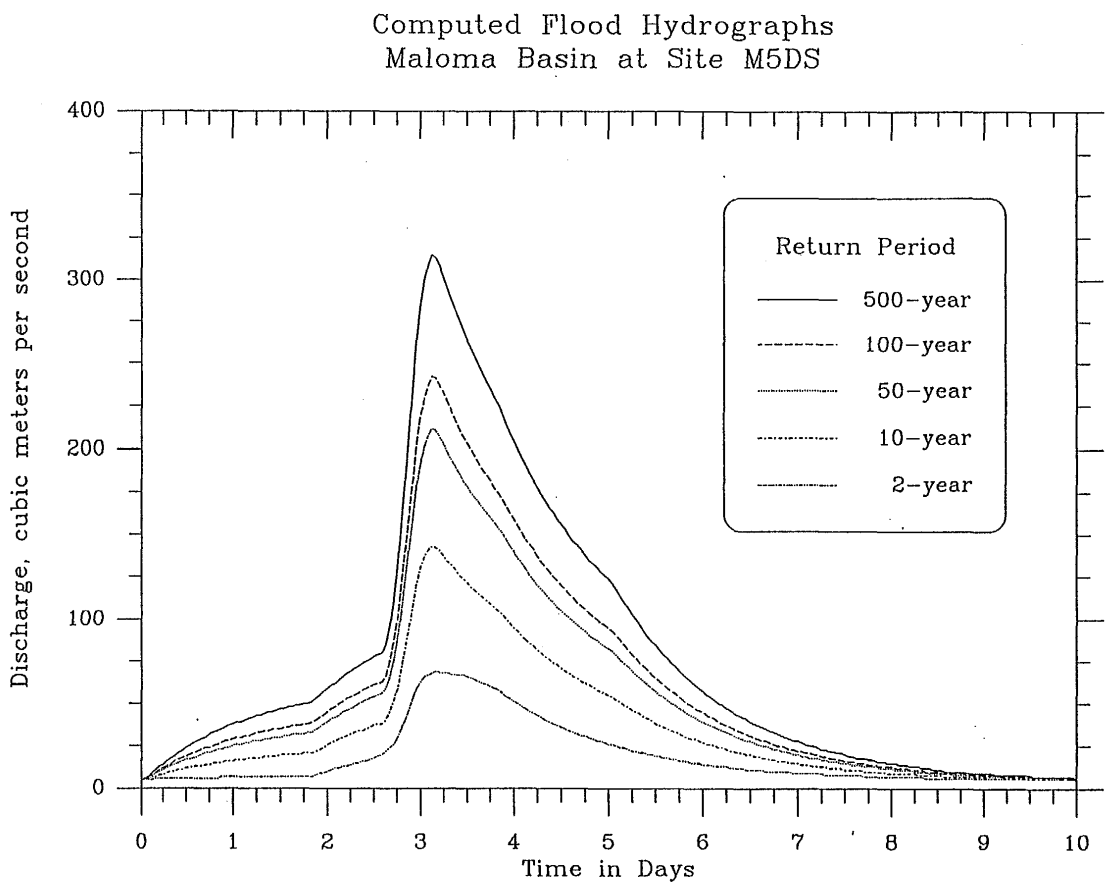


Figure 3.5.68

163

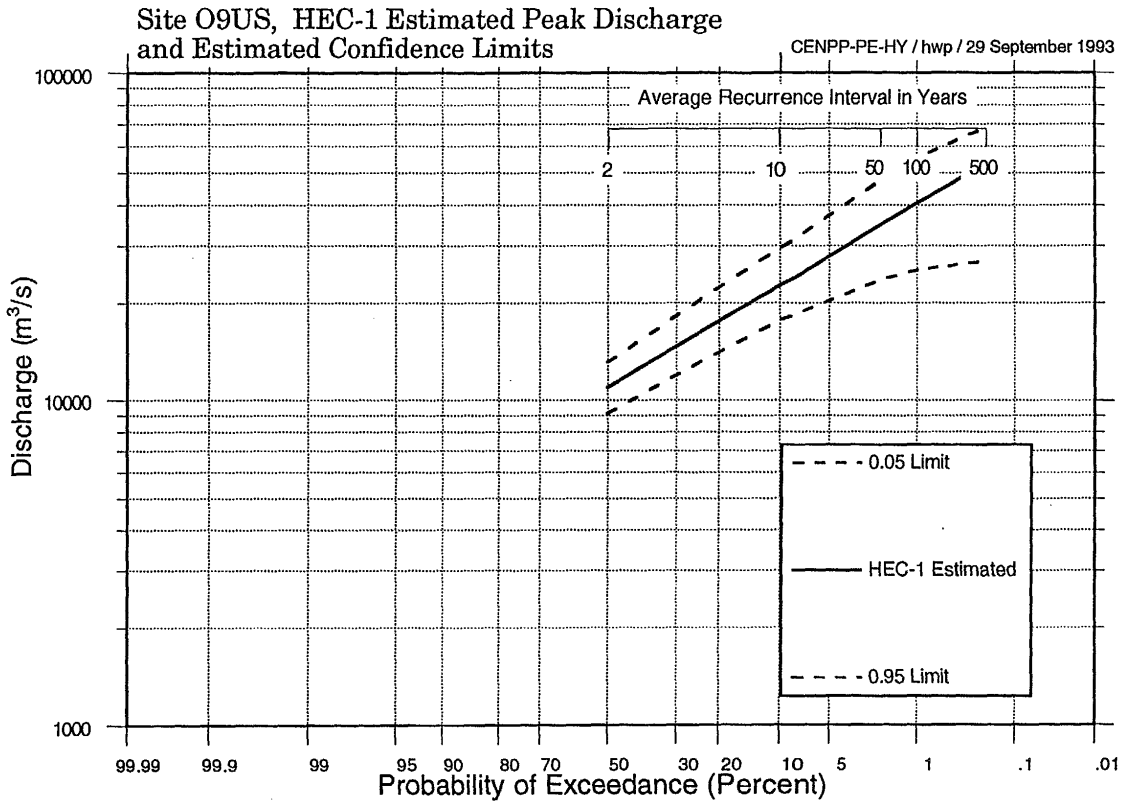

Site O9US, HEC-1 Estimated Peak Discharge
and Estimated Confidence Limits
CENPP-PE-HY / hwp / 29 September 1993
Average Recurrence Interval in Years
2
10
50
100
500
100000
10000
1000
Discharge (m³/s)
0.05 Limit
HEC-1 Estimated
0.95 Limit
99.99
99.9
99
95
90
80
70
50
30
20
10
5
1
.1
.01
Probability of Exceedance (Percent)

Figure 3.5.69

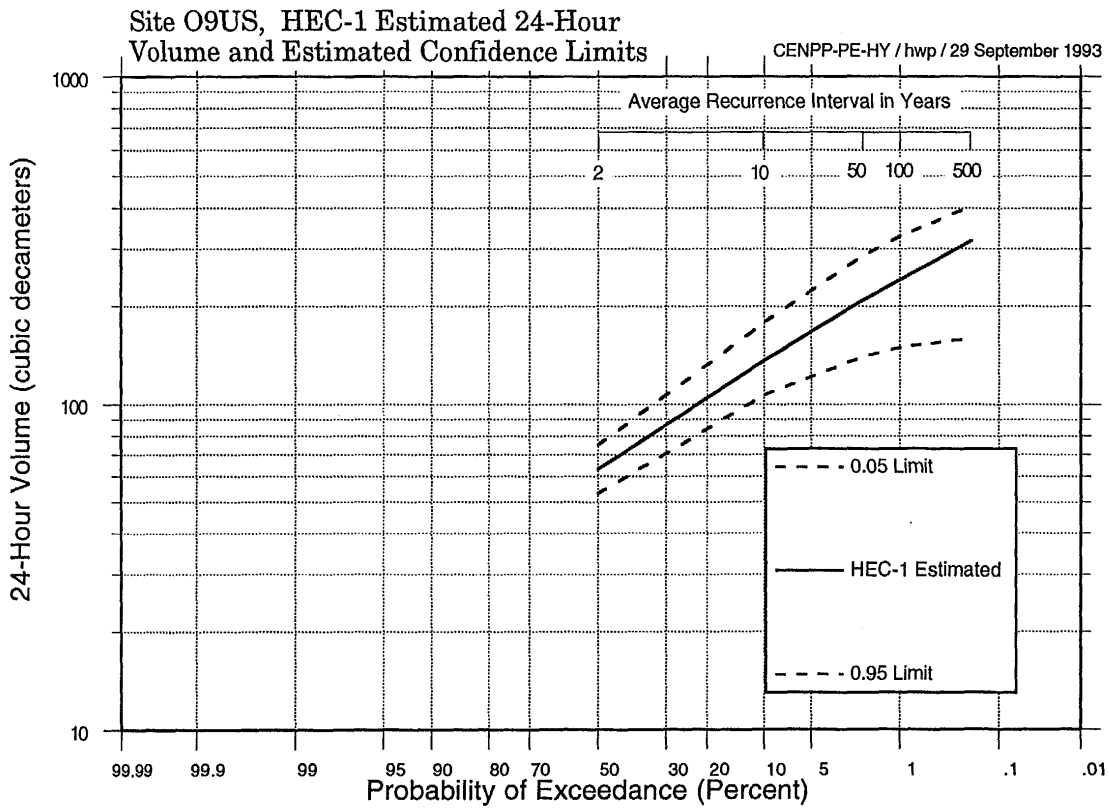


Figure 3.5.70

693

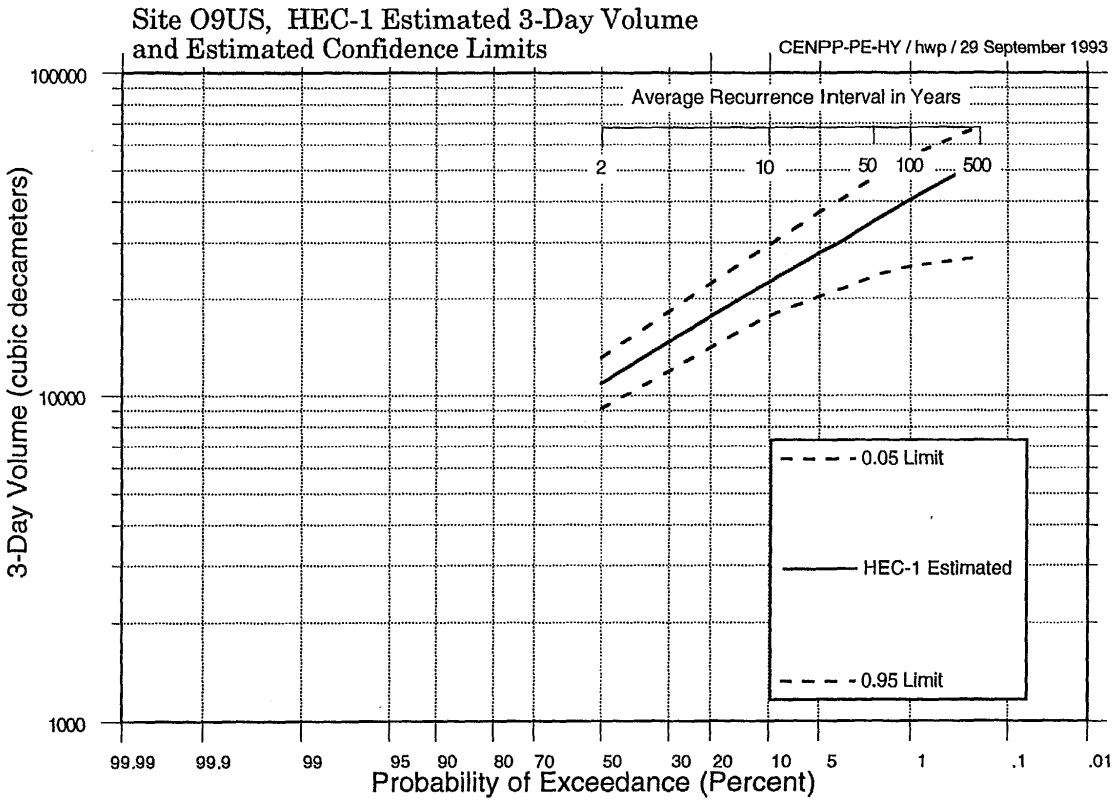


Figure 3.5.71

694

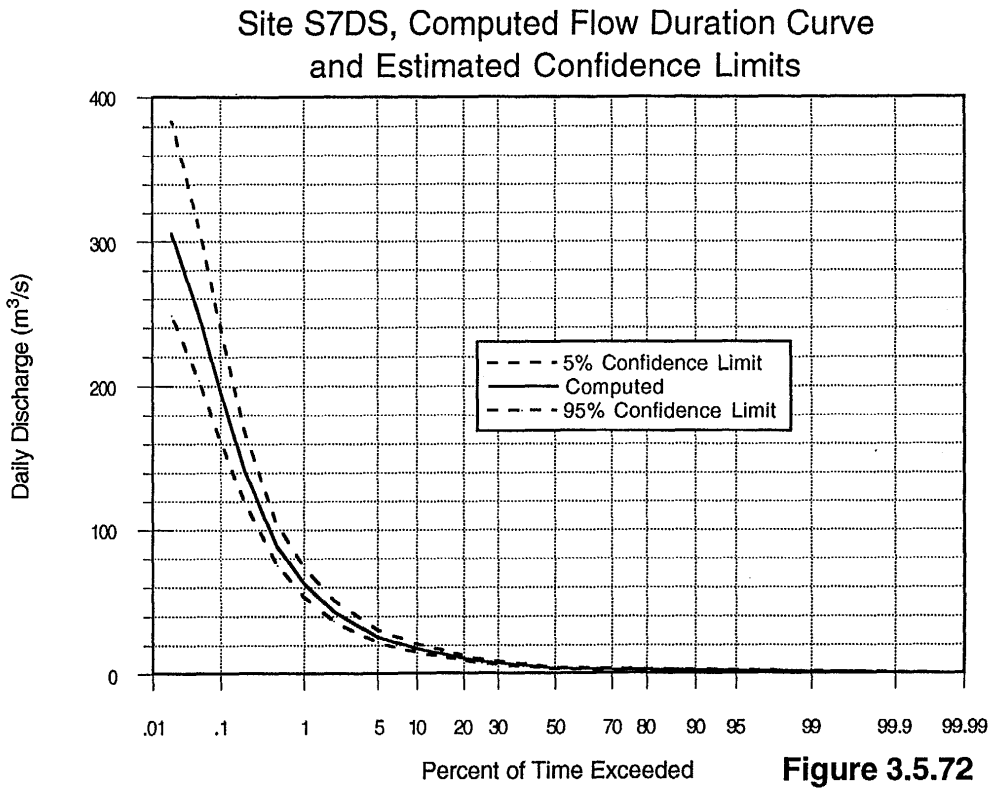

Site S7DS, Computed Flow Duration Curve
and Estimated Confidence Limits
Daily Discharge (m3/s)
400
300
200
100
0
5% Confidence Limit
Computed
95% Confidence Limit
.01
.1
1
5
10
20
30
50
70
80
90
95
99
99.9
99.99
Percent of Time Exceeded

Figure 3.5.72

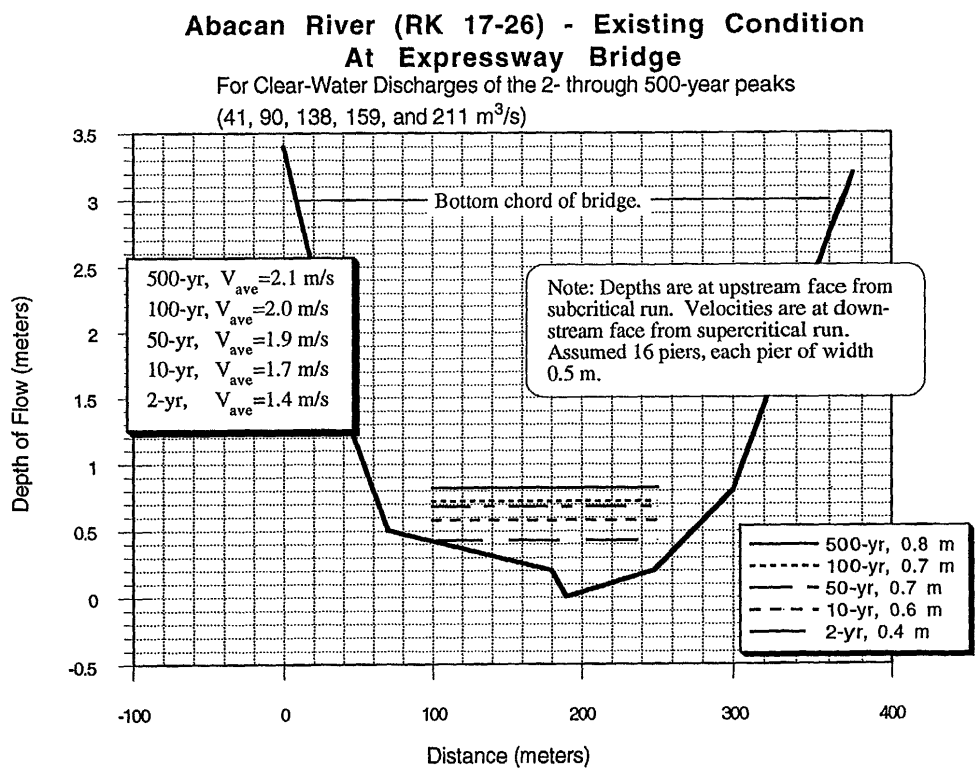

Abacan River (RK 17-26) - Existing Condition
At Expressway Bridge
For Clear-Water Discharges of the 2- through 500-year peaks
(41, 90, 138, 159, and 211 m3/s)
Bottom chord of bridge.
500-yr, Vave=2.1 m/s
100-yr, Vave=2.0 m/s
50-yr, Vave=1.9 m/s
10-yr, Vave=1.7 m/s
2-yr, Vave=1.4 m/s
Note: Depths are at upstream face from subcritical run. Velocities are at downstream face from supercritical run. Assumed 16 piers, each pier of width 0.5 m.
500-yr, 0.8 m
100-yr, 0.7 m
50-yr, 0.7 m
10-yr, 0.6 m
2-yr, 0.4 m
Depth of Flow (meters)
3.5
3
2.5
2
1.5
1
0.5
0
-0.5
-100
0
100
200
300
400
Distance (meters)

Figure 3.6.1

Abacan River (RK 17-26) - Existing Condition At Friendship Bridge

For Clear-Water Discharges of the 2- through 500-year peaks (41, 90, 138, 159, and 211 m^3/s).

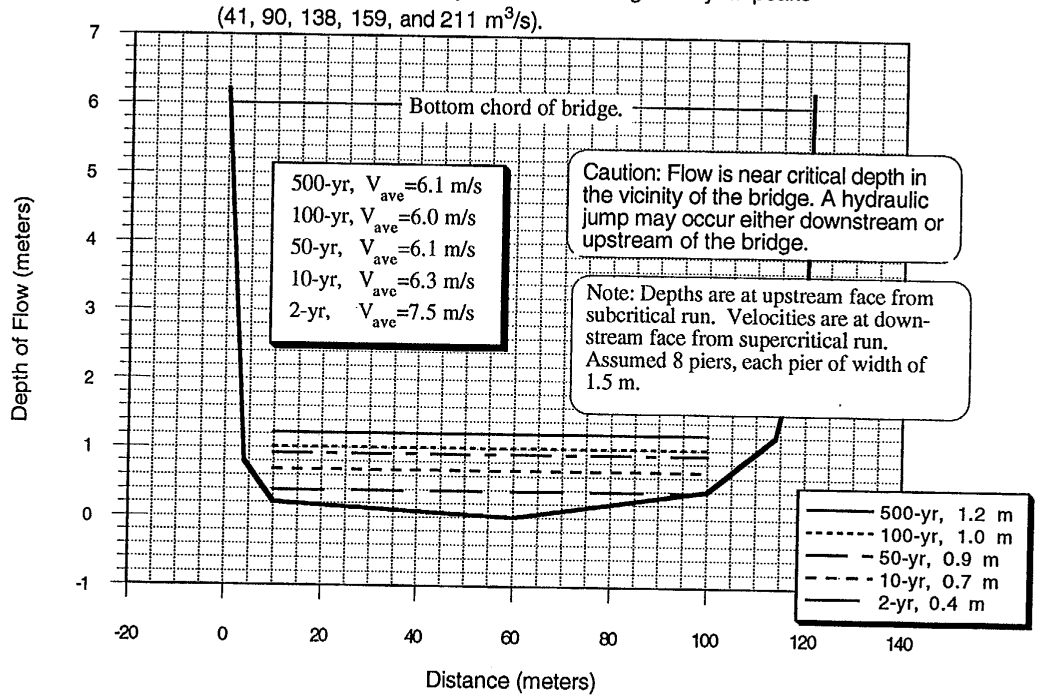


Figure 3.6.2

87

Bamban River (RK 14-19) - Existing Condition
At San Francisco Bridge (Sacobia and Marimla flow)

For Clear-Water Discharges of the 2- through 500-year floods
(102, 230, 354, 409, and 541 m^3/s).

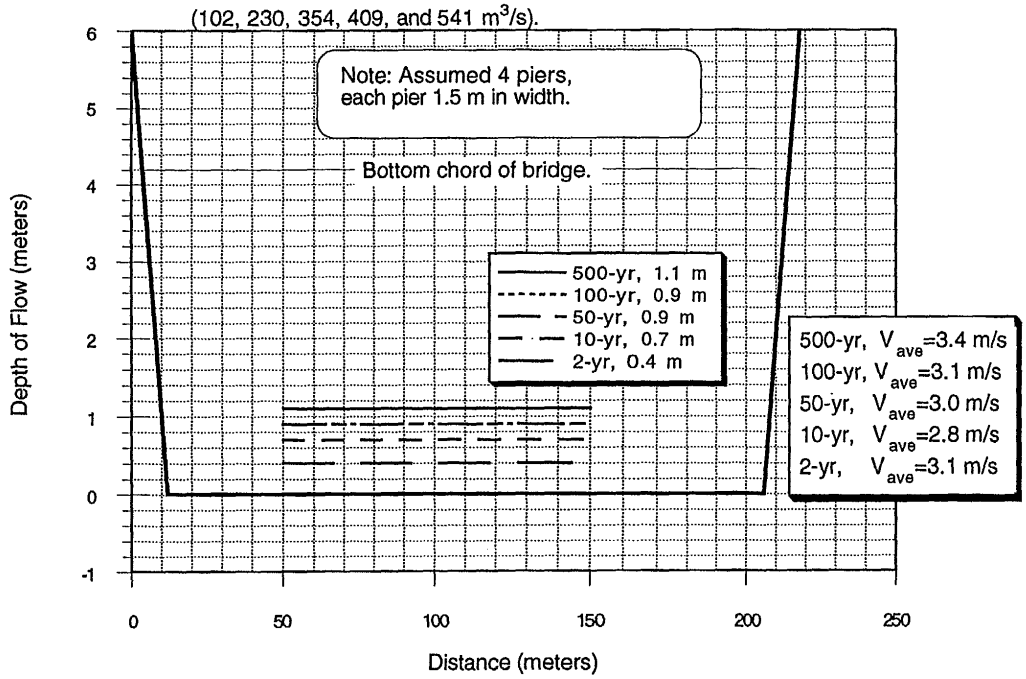


Figure 3.6.3

Bamban River (RK 14-19) - Existing Condition At San Francisco Bridge (Sacobia flow only)

For Clear-Water Discharges of the 2- through 500-year floods (44, 98, 152, 175, and 232m^3/s).

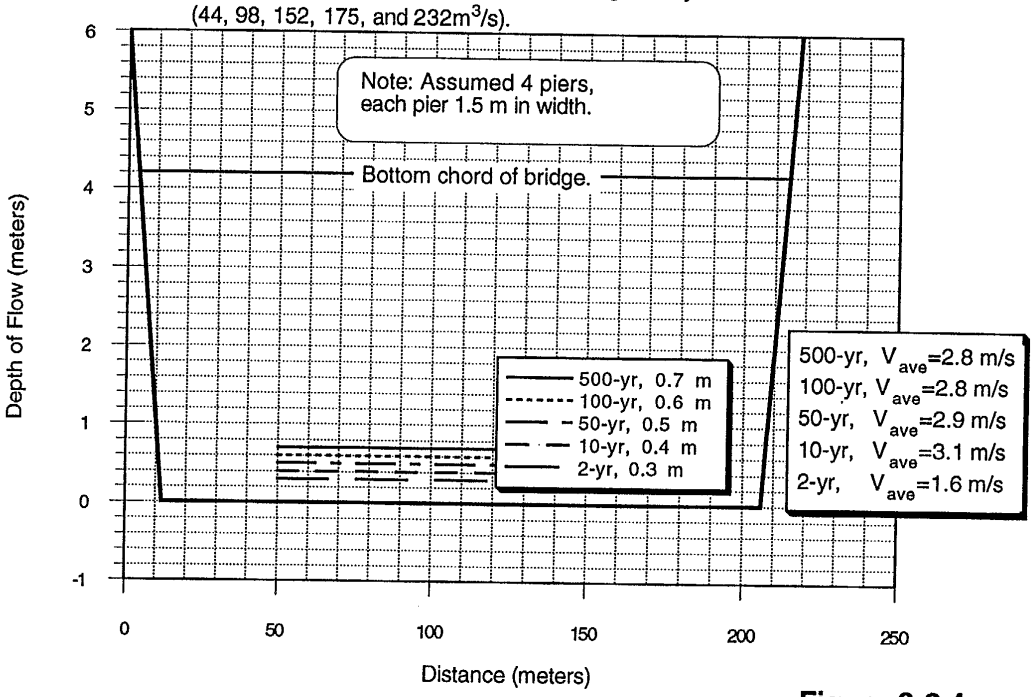


Figure 3.6.4

Bucao River (RK 0-3.5) - Existing Condition At Downstream Bridge

For Clear-Water Discharges of the 2- through 500-year peak flows (834, 1737, 2591, 2963, and 3869 m^3/s).

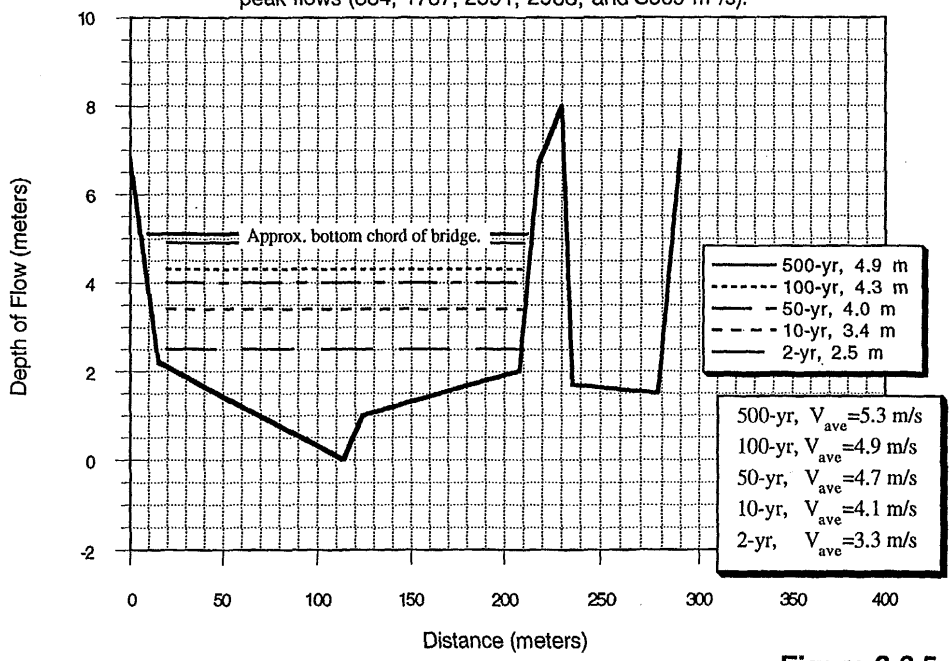


Figure 3.6.5

Pasig-Potrero River (RK 1-5) - Existing Condition At Lower (D/S) Bridge

For Clear-Water Discharges of the 2- Through 500Year Floods (90, 195, 294, 337, and 444 m^3/s).

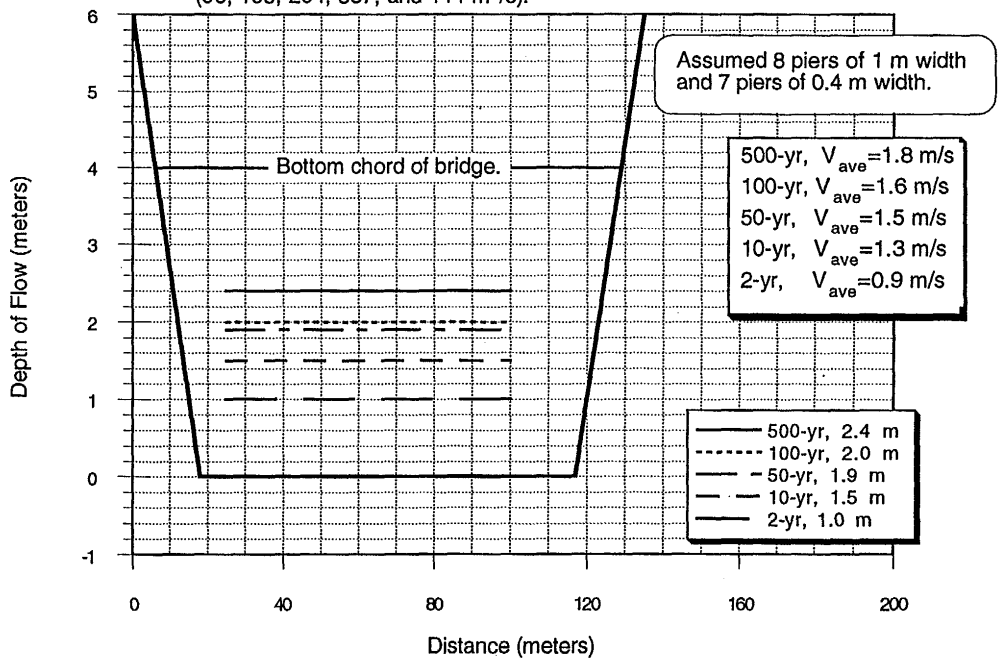


Figure 3.6.6

109

Pasig-Potrero River (RK 1-5) - Existing Condition At Bypass Bridge

For Clear-Water Discharges of the 2- Through 500-Year Floods (90, 195, 294, 337, and 444 m^3/s).

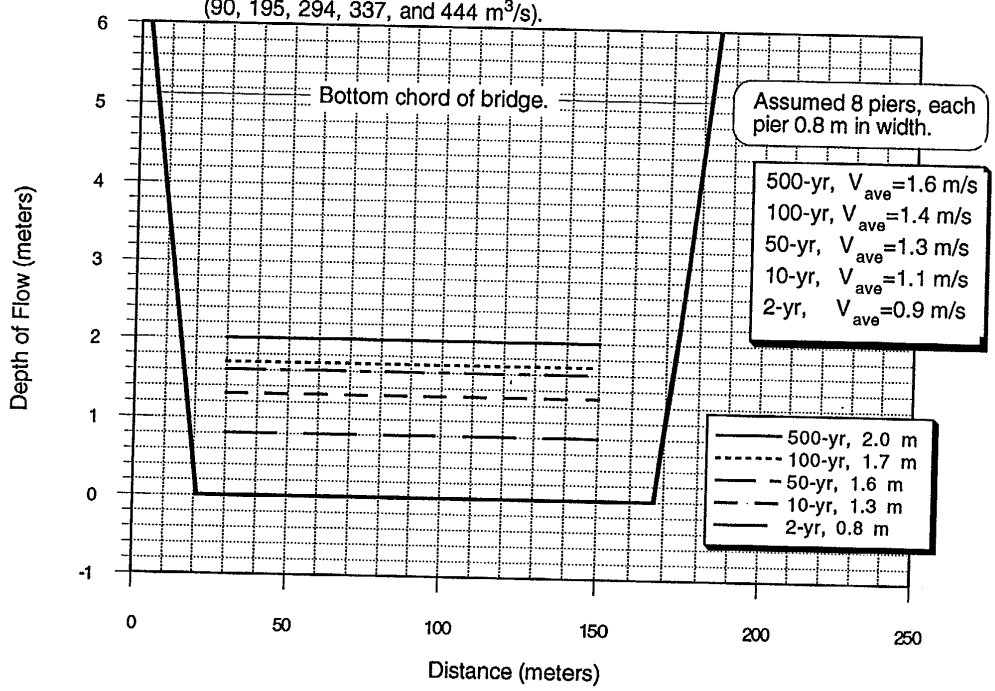


Figure 3.6.7

602

Porac River (RK 19.5 - 20.5) - Existing Condition At Bridge in Town of Porac

For Clear-Water Discharges of the 2- through 500-year peaks (43, 95, 145, 166, and 219 m^3/s)

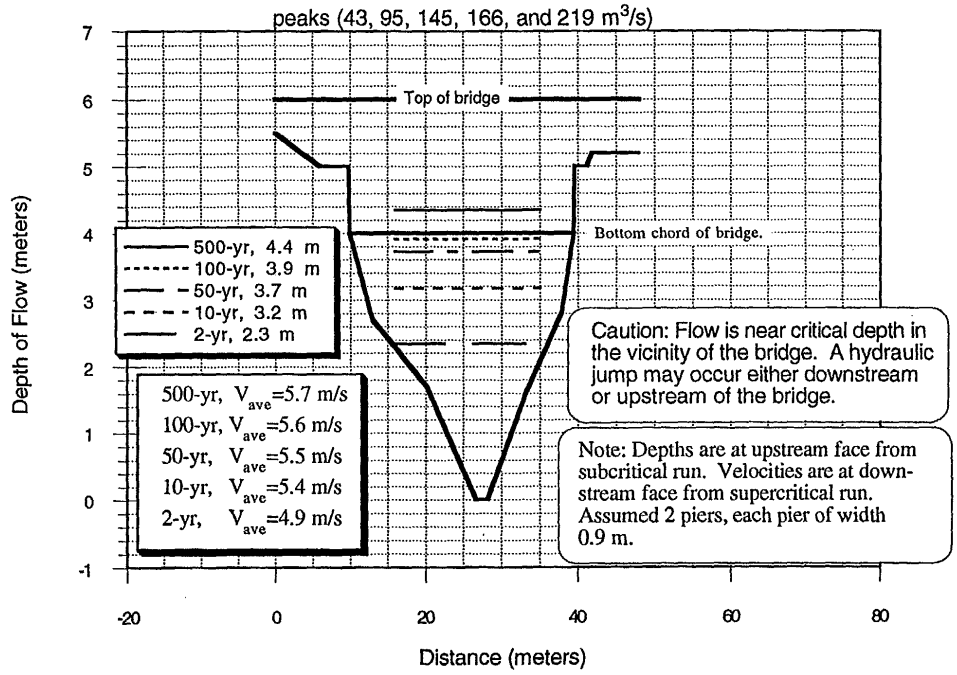


Figure 3.6.8

603

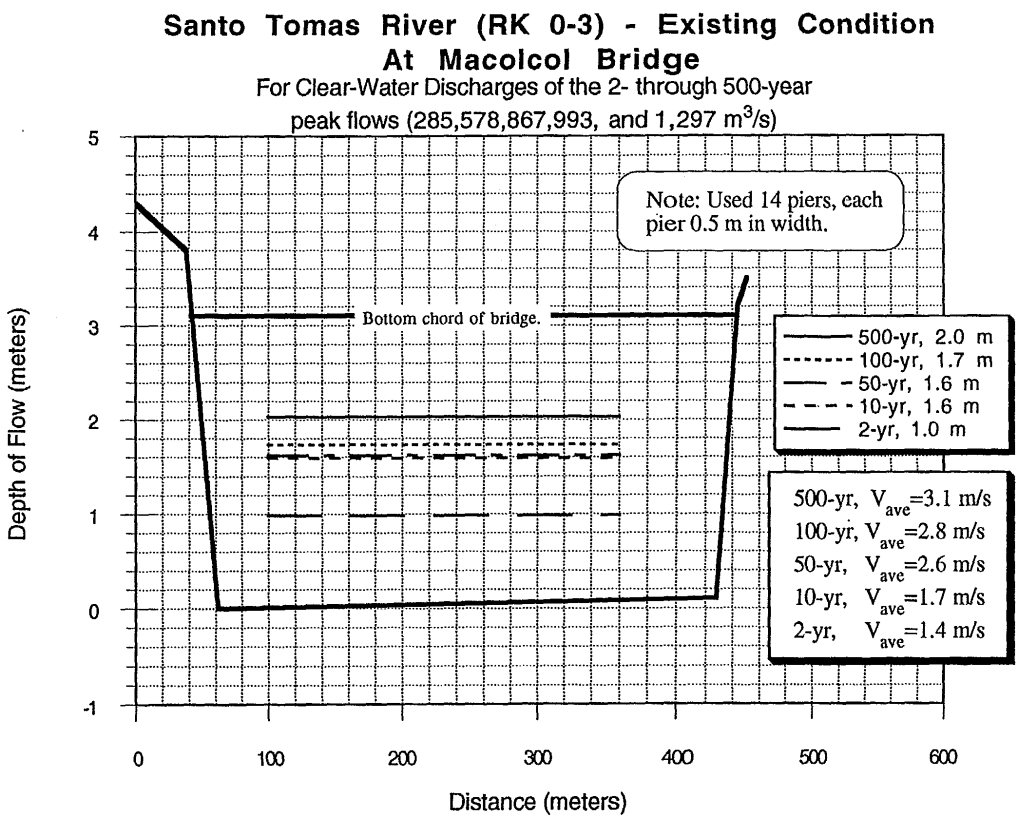

Santo Tomas River (RK 0-3) - Existing Condition
At Macolcol Bridge
For Clear-Water Discharges of the 2- through 500-year
peak flows (285,578,867,993, and 1,297 m³/s)
Note: Used 14 piers, each pier 0.5 m in width.
Bottom chord of bridge.
500-yr, 2.0 m
100-yr, 1.7 m
50-yr, 1.6 m
10-yr, 1.6 m
2-yr, 1.0 m
500-yr, V_ave=3.1 m/s
100-yr, V_ave=2.8 m/s
50-yr, V_ave=2.6 m/s
10-yr, V_ave=1.7 m/s
2-yr, V_ave=1.4 m/s
Depth of Flow (meters)
Distance (meters)
5
4
3
2
1
0
-1
0
100
200
300
400
500
600

Figure 3.6.9

Tarlac River through Tarlac - Existing Condition At Agana Bridge

For Clear-Water Discharges of the 2- through 500-year peaks (616, 1273, 1933, 2221, and 2922 m^3/s)

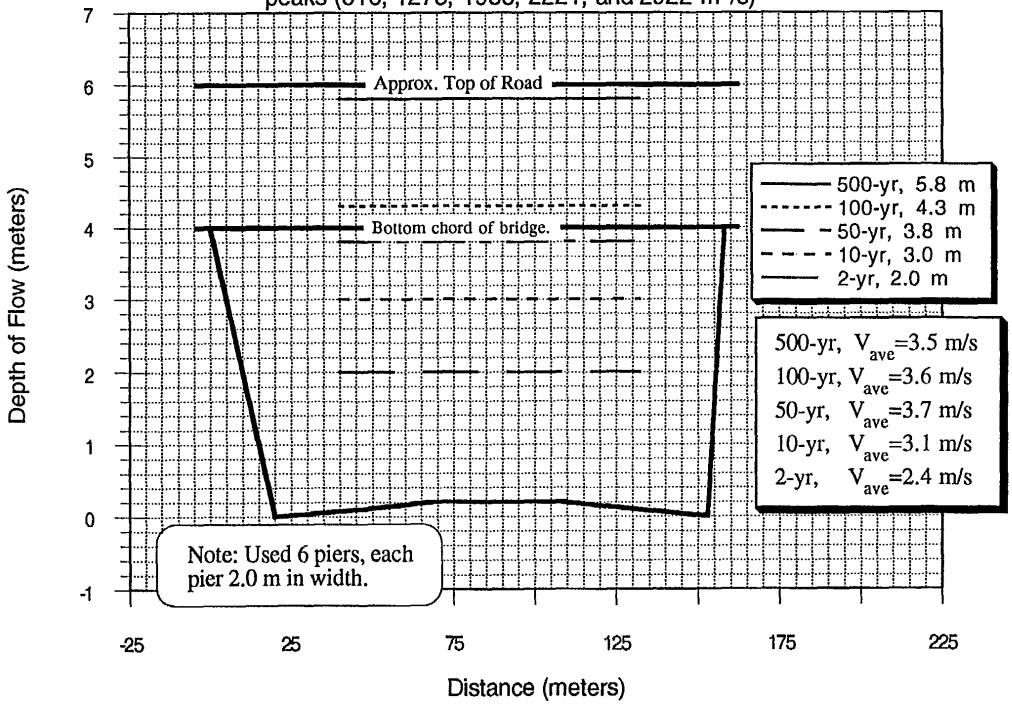


Figure 3.6.10

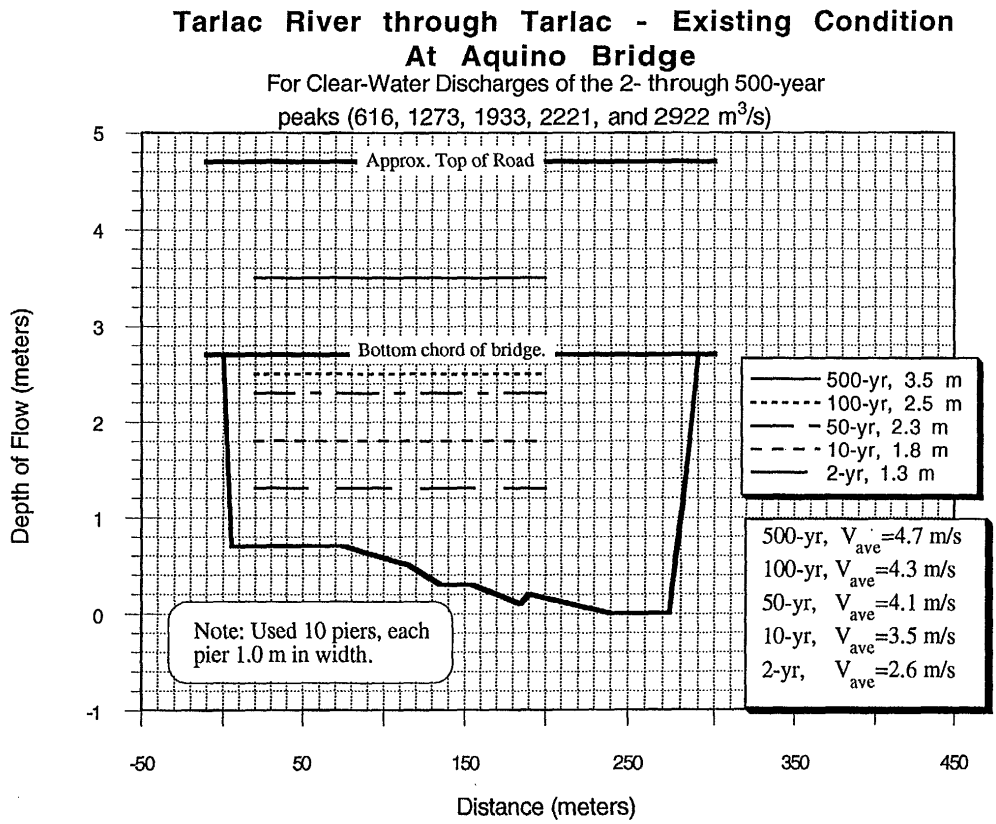

Tarlac River through Tarlac - Existing Condition
At Aquino Bridge
For Clear-Water Discharges of the 2- through 500-year
peaks (616, 1273, 1933, 2221, and 2922 m3/s)
Approx. Top of Road
Bottom chord of bridge.
500-yr, 3.5 m
100-yr, 2.5 m
50-yr, 2.3 m
10-yr, 1.8 m
2-yr, 1.3 m
500-yr, Vave=4.7 m/s
100-yr, Vave=4.3 m/s
50-yr, Vave=4.1 m/s
10-yr, Vave=3.5 m/s
2-yr, Vave=2.6 m/s
Note: Used 10 piers, each pier 1.0 m in width.
Depth of Flow (meters)
Distance (meters)
5
4
3
2
1
0
-1
-50
50
150
250
350
450

Figure 3.6.11

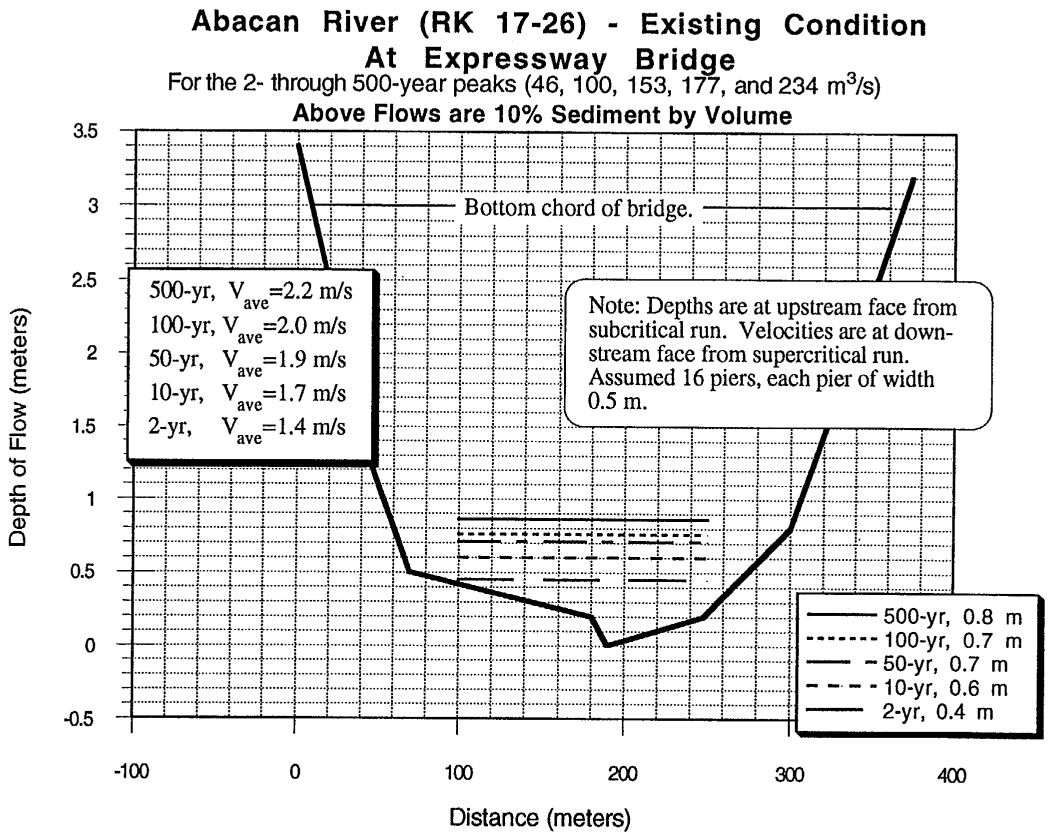


Figure 3.6.12

607

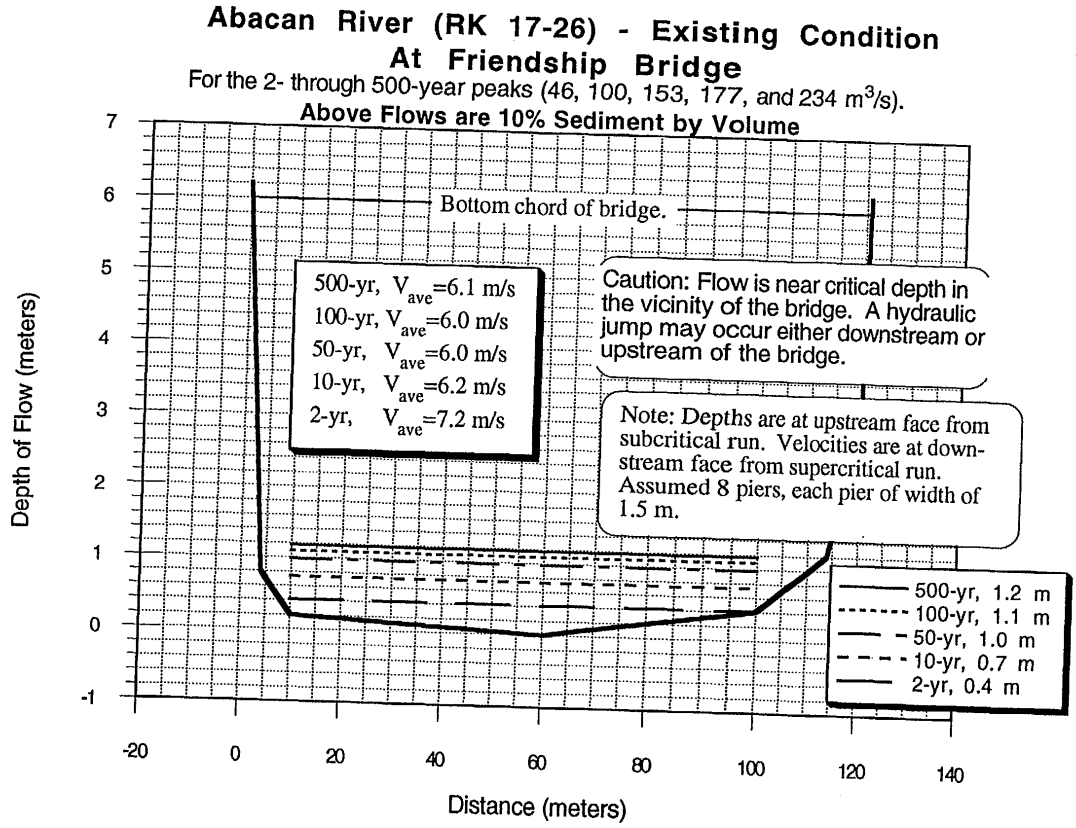


Figure 3.6.13

208

Bamban River (RK 14-19) - Existing Condition
At San Francisco Bridge (Sacobia and Marimla Flow)
For the 2- through 500-year Floods (131, 298, 458, 528, and 700 m^3/s).
Flow is 22% Sediment by Volume

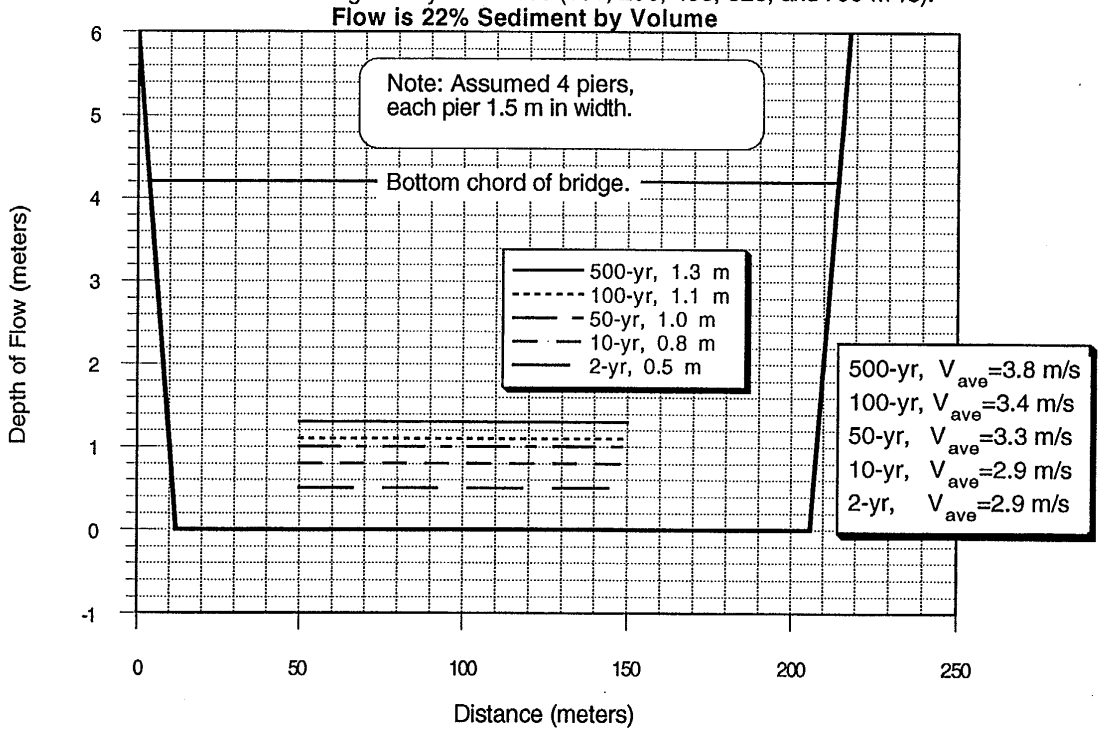


Figure 3.6.14

609

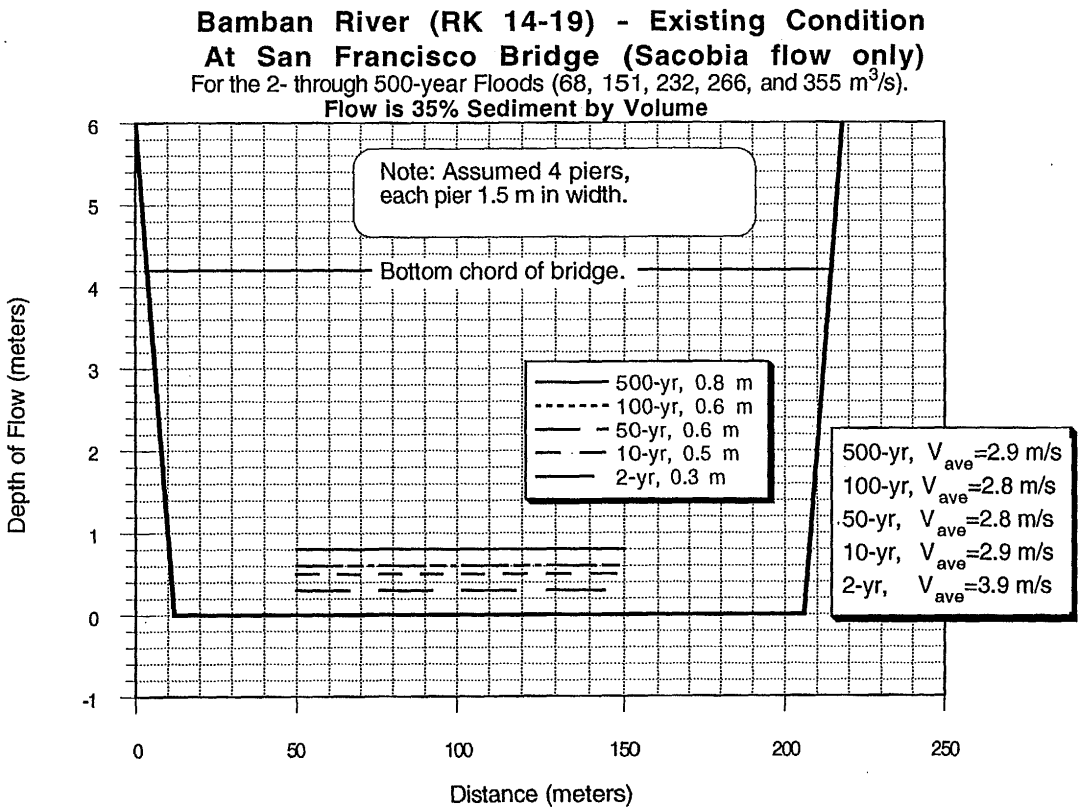


Figure 3.6.15

610

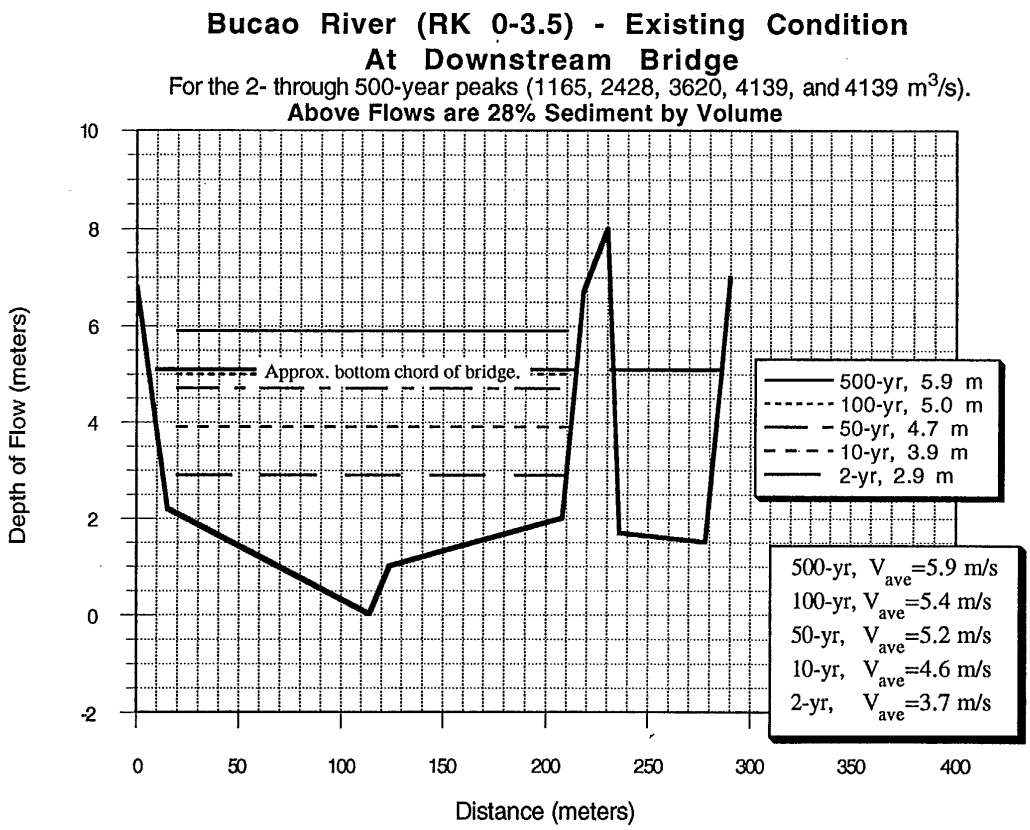

Bucao River (RK 0-3.5) - Existing Condition
At Downstream Bridge
For the 2- through 500-year peaks (1165, 2428, 3620, 4139, and 4139 m3/s).
Above Flows are 28% Sediment by Volume
Depth of Flow (meters)
Distance (meters)
Approx. bottom chord of bridge.
500-yr, 5.9 m
100-yr, 5.0 m
50-yr, 4.7 m
10-yr, 3.9 m
2-yr, 2.9 m
500-yr, Vave=5.9 m/s
100-yr, Vave=5.4 m/s
50-yr, Vave=5.2 m/s
10-yr, Vave=4.6 m/s
2-yr, Vave=3.7 m/s

Figure 3.6.16

Pasig-Potrero River (RK 1-5) - Existing Condition
At Lower (D/S) Bridge
For the 2- Through 500-Year Floods (149, 321, 483, 553, and 727 m^3/s).
Flow is 40% Sediment by Volume

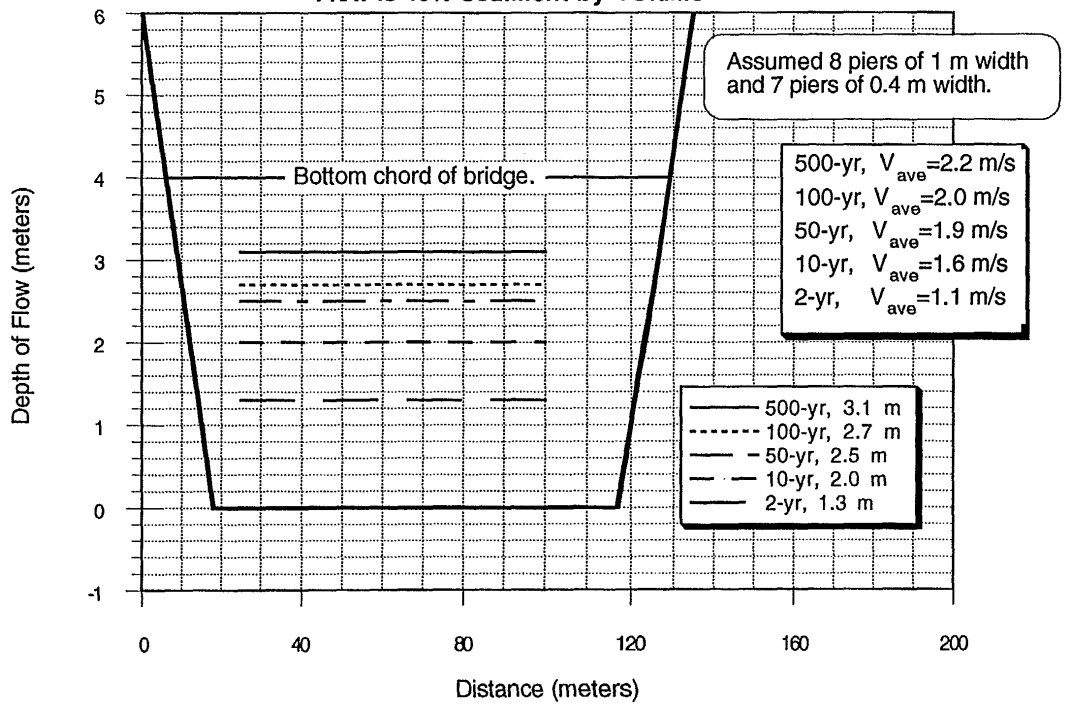


Figure 3.6.17

616

Pasig-Potrero River (RK 1-5) - Existing Condition At Bypass Bridge

For the 2- through 500-year Floods (149, 321, 483, 553, and 727 m^3/s).

Flow is 40% Sediment by Volume

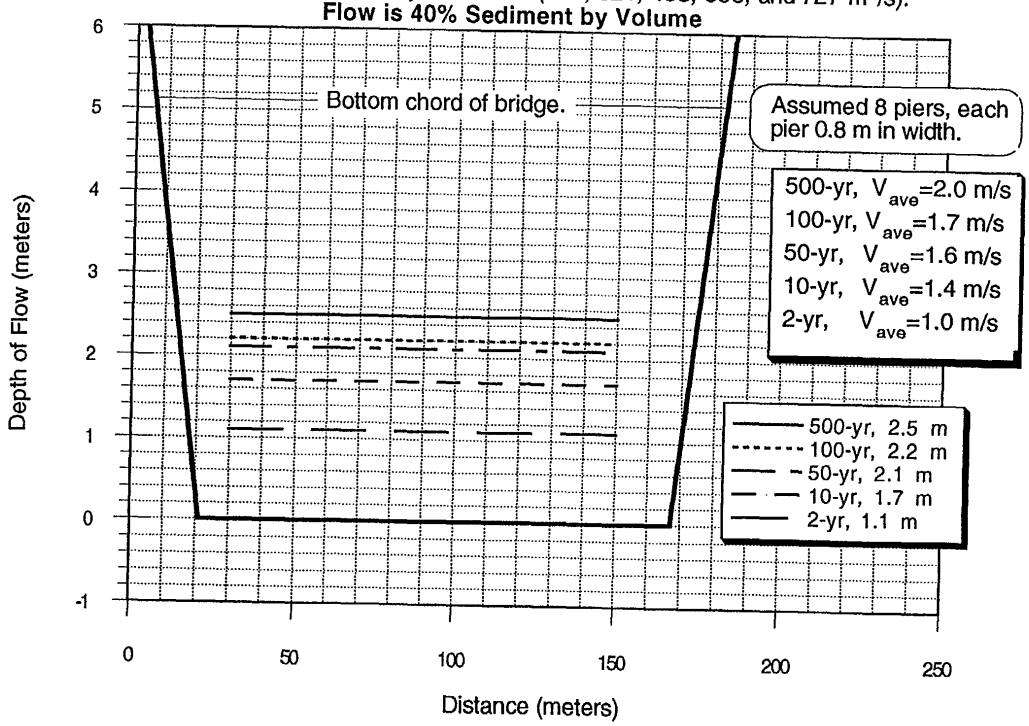


Figure 3.6.18

63

Porac River (RK 19.5 - 20.5) - Existing Condition
At Bridge in Town of Porac
For the 2- through 500-year peaks (48, 106, 161, 184, and 243m^3/s)
Above Flows are 10% Sediment by Volume

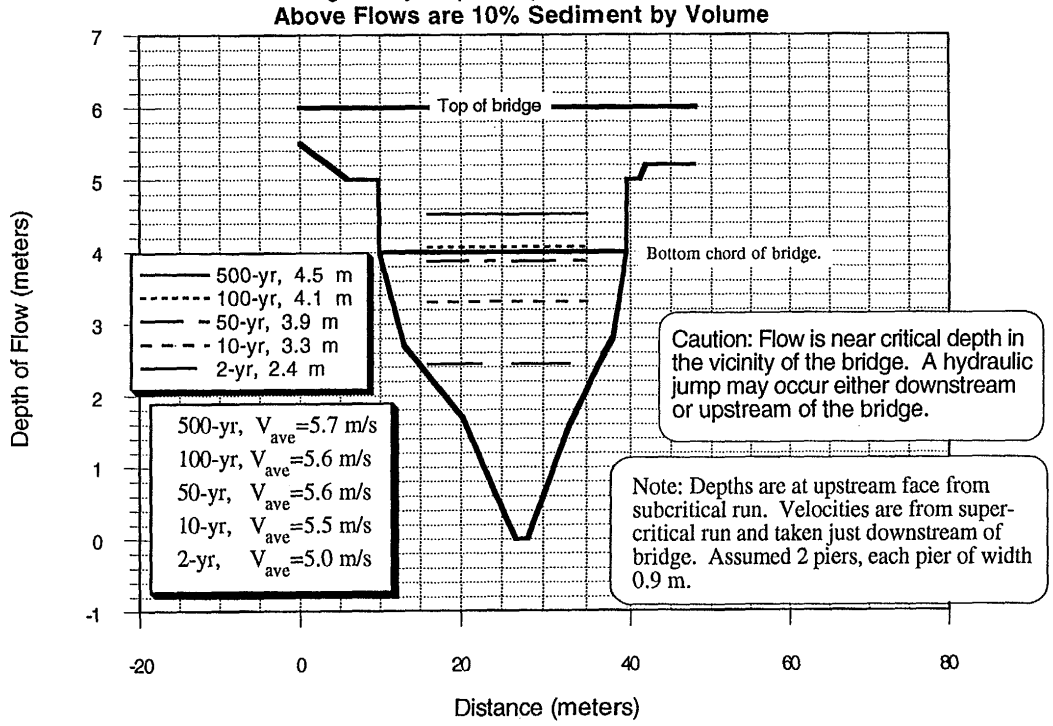


Figure 3.6.19

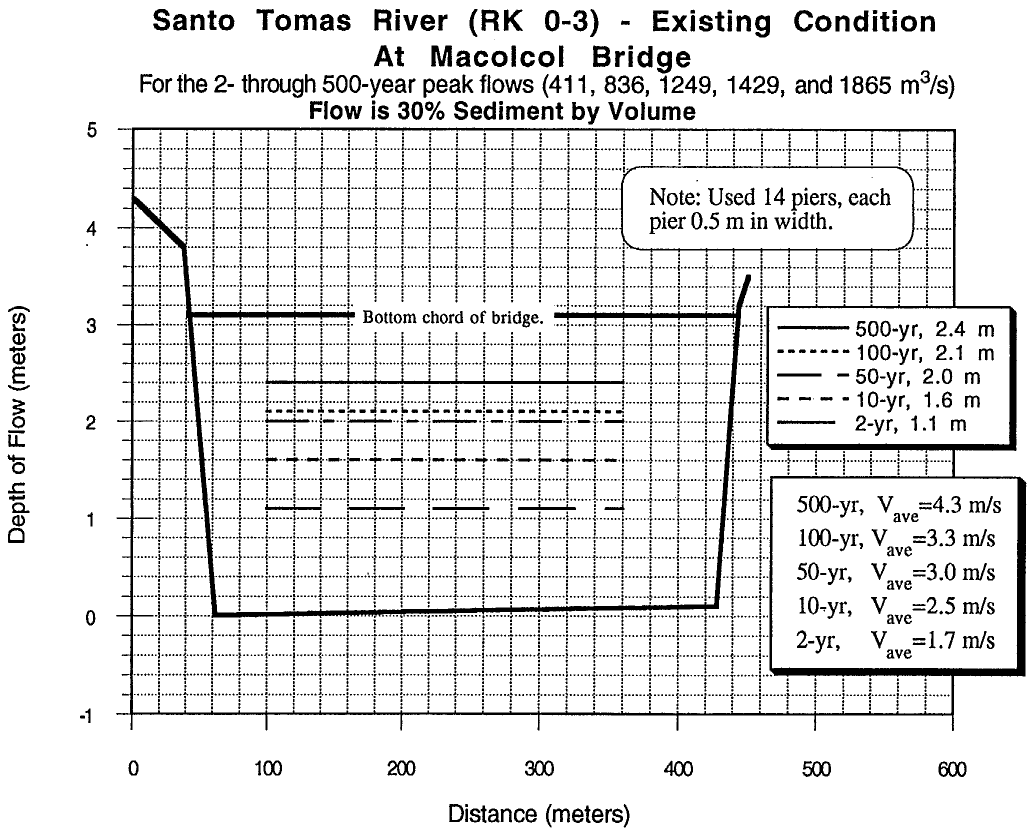

Santo Tomas River (RK 0-3) - Existing Condition
At Macolcol Bridge
For the 2- through 500-year peak flows (411, 836, 1249, 1429, and 1865 m3/s)
Flow is 30% Sediment by Volume
Note: Used 14 piers, each pier 0.5 m in width.
Bottom chord of bridge.
500-yr, 2.4 m
100-yr, 2.1 m
50-yr, 2.0 m
10-yr, 1.6 m
2-yr, 1.1 m
500-yr, Vave=4.3 m/s
100-yr, Vave=3.3 m/s
50-yr, Vave=3.0 m/s
10-yr, Vave=2.5 m/s
2-yr, Vave=1.7 m/s
Depth of Flow (meters)
Distance (meters)
5
4
3
2
1
0
-1
0
100
200
300
400
500
600

Figure 3.6.20

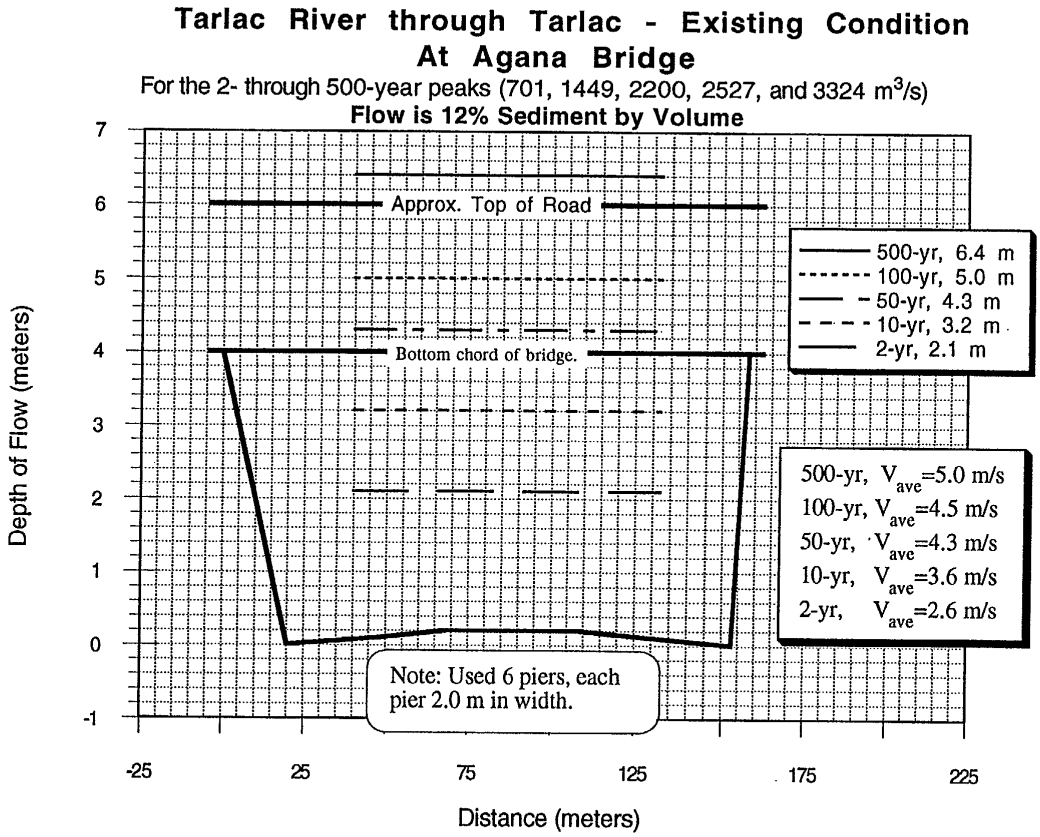

Tarlac River through Tarlac - Existing Condition
At Agana Bridge
For the 2- through 500-year peaks (701, 1449, 2200, 2527, and 3324 m3/s)
Flow is 12% Sediment by Volume
Approx. Top of Road
Bottom chord of bridge.
500-yr, 6.4 m
100-yr, 5.0 m
50-yr, 4.3 m
10-yr, 3.2 m
2-yr, 2.1 m
500-yr, Vave=5.0 m/s
100-yr, Vave=4.5 m/s
50-yr, Vave=4.3 m/s
10-yr, Vave=3.6 m/s
2-yr, Vave=2.6 m/s
Note: Used 6 piers, each pier 2.0 m in width.
Depth of Flow (meters)
Distance (meters)
7
6
5
4
3
2
1
0
-1
-25
25
75
125
175
225

Figure 3.6.21

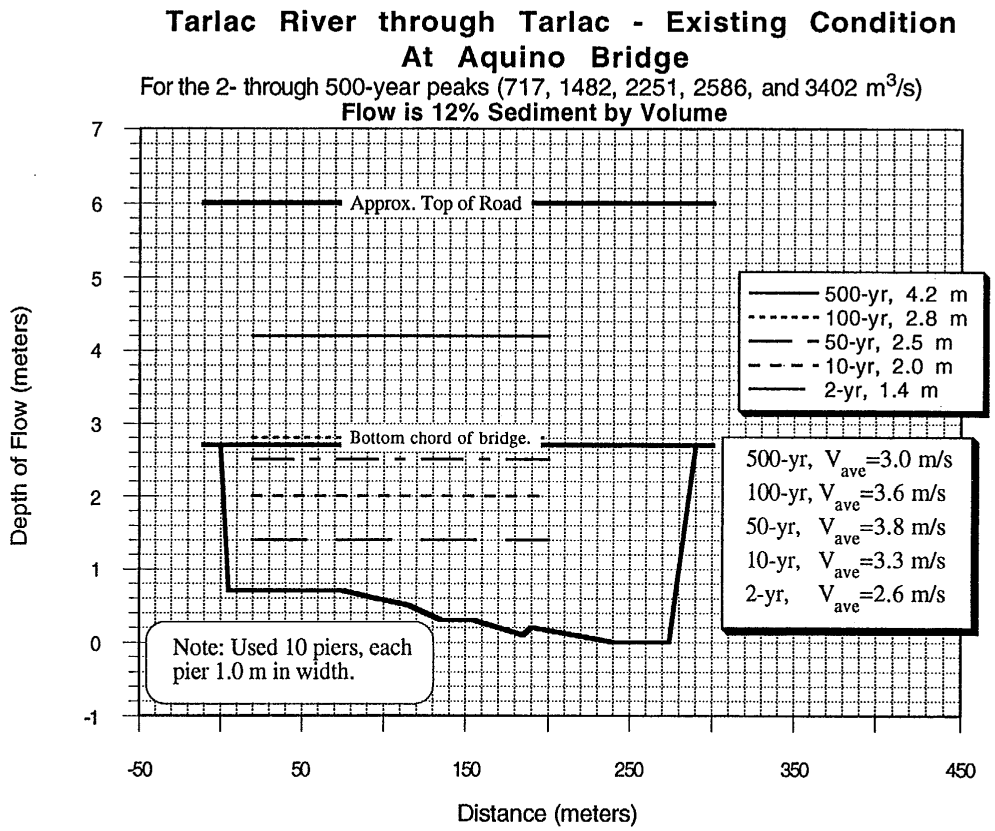


Figure 3.6.22

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TECHNICAL APPENDIX

B

SEDIMENTATION ANALYSIS

MOUNT PINATUBO
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1. INTRODUCTION

A catastrophic eruption of Mount Pinatubo occurred between June 12 and 15, 1991. By the afternoon of June 15, 1991, when the initial eruption phase terminated, about 5.6 billion cubic meters (m^3) of medium- to fine-grained pyroclastic-flow material had been deposited in the upper watershed areas around Mount Pinatubo. Another 1 cubic kilometer (km^3) of ash covered the landscape to a radius of more than 40 km around the mountain (Pierson et al., 1992).

Rainfall-runoff has rapidly eroded eruption material, causing lahars that have flooded low-lying areas. Flooding and sedimentation from Mount Pinatubo lahars have displaced tens of thousands of people from their homes, destroyed bridges and crops, and decreased the amount of land available to agriculture in the lower basin. Several barangays and town centers were flooded and buried by sediment deposits of up to 3 meters deep. Hundreds of people have died as a result of the eruption and its aftermath.

In October 1993, heavy rainfall and rapid erosion caused about 21 km^2 of the Sacobia River basin to be diverted into the Pasig River basin. This large increase in Pasig River drainage area is very likely to cause a tremendous increase in sediment yield in 1994 and beyond. The full impact of this basin change has not been evaluated for this report, but it is judged to present an extreme hazard to communities along the Pasig-Potrero River and also to endanger surrounding areas. Sediment yields and lahars in the Pasig River in 1994, are expected to be similar to those experienced in the Sacobia River in 1991 or 1992. Pasig-Potrero River sediment deposition of 50 to 100 million m^3 is considered possible in 1994.

1.1 Purpose

The purpose of this analysis is to assess the future sedimentation hazards due to continuing erosion of the 1991 pyroclastic deposits around Mount Pinatubo. A sediment yield forecast is presented for each basin containing large amounts of pyroclastic material. The areas most likely to experience sediment deposition were also identified. That information is used throughout this report to determine future damages, plan and design sediment control measures, and to assess the potential benefits (economic and physical) for those control measures.

1.2 Study Area

Mount Pinatubo is located in the Zambales Mountain Range on the west coast of Central Luzon in the Philippines, approximately 100 km northwest of Manila. Eight major river basins -- the Gumain-Porac, Pasig-Potrero, Abacan, Sacobia, O'Donnell, Bucao, Maloma, and Santo Tomas-Marella -- form the drainage basins around the mountain. Before the 1991 eruption, Mount Pinatubo stood 1,745 meters above the sea level. The eruption reduced the peak elevation to some 1,520 meters and caused major perturbations in five of the eight affected basins, filling them with volcanic material (see Figure B-1).

The principal drainages on the east side of Mount Pinatubo include the Gumain-Porac, Pasig-Potrero, Abacan, Sacobia and O'Donnell basins. The O'Donnell River joins the Bulsa River to form the Tarlac River, which flows north to the Agno River and thence to the Lingayen Gulf (see Appendix A). The Gumain-Porac, Pasig-Potrero, Abacan and Sacobia Rivers drain the remaining eastern section, flowing south into Manila Bay through the Pampanga delta along with the Pampanga River.

The principal west-side drainages include the Bucao, Maloma and Santo Tomas basins. The Bucao is the largest of these watershed systems, draining approximately 656 km^2 of the northwestern sector and ultimately flowing to the South China Sea. Headwaters of the Maloma and Santo Tomas-Marella systems originate on the southwestern sector and also drain to the South China Sea (see Figure B-1).

1.3 1991 Eruption of Mount Pinatubo

Starting in April 1991, a series of minor volcanic eruptions occurred on Mount Pinatubo. The culminating phase began on June 12, climaxing sometime after 1400 hours on June 15 with a violent eruption phase (Pierson et al., 1992) -- a classic, catastrophic chamber explosion which ejected pyroclastic-flow material and ash into the atmosphere and deposited it in a radial pattern onto the mountain flanks and watersheds.

The eruption left thick accumulations of pyroclastic-flow material in most of the volcano's upper basins. The Gumain-Porac River system, the southernmost of the east-side river complexes, received no measurable pyroclastic-flow deposits, but its upper basins were thickly blanketed by airfall ash. On the west side, the Maloma River headwaters and the upper reaches of the Santo Tomas and Balin-Buquero Rivers, which originate approximately 7 km southwest of the caldera, received only minimal initial pyroclastic-flow deposits, but both basins were blanketed by airfall ash.

During the eruption, pyroclastic flows, fast-moving mixtures of gas and tephra particles, flowed down the flanks of the volcano, tending to follow existing stream valleys, particularly on the east side. These deposits ranged in thickness from a few meters to as

much as 200 meters in the deeper valley reaches of the upper and middle basins. In addition, airfall ash accumulated in thicknesses ranging from more than 50 cm near the crater to just a trace at distances of 30 km from the mountain. Some rock fragments (lithics) from the former crater and crater walls are also present in deposits, generally within 3 to 5 km of the new crater. Initial pyroclastic flows were extremely hot; by 1992, deposits were maintaining temperatures in the 300° Celsius range, and were expected to hold temperature values for a number of years (R. Hoblitt, U.S. Geological Survey, oral communications, 1991 and 1992). The 1991 tephra (pyroclastic flow) deposits are geologically similar to deposits observed elsewhere from other recent plinian eruptions, resulting from gas-charged, highly silicic magmas moving upward in the cone and venting through the existing crater. The tephra produced from this magma type is lightweight and rich in pumice.

1.4 Regional Geology

The two main physiography provinces within the study area are the Zambales Mountain Range and the Central Luzon basin.

Mount Pinatubo is located in Central Luzon on the eastern edge of the Zambales Mountain Range, an area of orogenic uplift extending from the western coastline to the east central lowlands. Superimposed on this uplift region is a volcanic arc composed of Pliocene to Quaternary composite volcanoes, of which Mount Pinatubo is the highest and youngest. This volcanic arc is trending north-south parallel to and probably associated with the offshore Manila Trench, a subduction zone in the South China Sea that dips eastwardly towards Luzon. The Zambales Range is underlain by dense ultramafic rocks known as the Zambales Ophiolite Complex (Delfin, 1984). These ophiolites consist of predominantly peridotite-basalt rock suites rich in serpentine, chlorite, epidote and other low-grade metamorphic minerals. Minor sedimentary units also exist within the stratigraphic column.

East of Mount Pinatubo lies a sediment-filled depression 80 km wide known as the Central Luzon basin (Delfin, 1984; see Figure B-1), bounded on the west by the Zambales Range and on the east by the Southern Sierra Madre Range, and extending from Manila in the south to the Lingayen Gulf in the north. The oldest sequence of sedimentary units on the western flank of this basin, adjacent to the foothills of the Zambales Range, consists of essentially flat-lying, undeformed late Eocene rocks that directly overlie the Zambales Ophiolite Complex. Most of the sediments filling the Central Luzon basin are volcanically-derived alluvial materials composed mainly of loosely compacted gravels and sands.

1.4.1 Historic Eruption Deposits. Mount Pinatubo is a composite andesitic volcano. The present-day hornblende-dacite dome is constructed upon older sedimentary and ultramafic strata. Underlying older volcanic rocks consist mostly of andesitic agglomerates, tuff breccias and tuffaceous sandstones interspersed with andesitic or basaltic flow rocks. These strata are much older than the pyroclastic-flow deposits that covered the mountain's flanks at the time of the 1991 eruption.

Mount Pinatubo's flanks are draped by massive pyroclastic deposits from a number of past eruptive events, each of which produced significant volumes of unconsolidated volcanic debris. At least two prior eruption episodes have been documented from existing pyroclastic-flow deposits (Delfin, 1984). The youngest is estimated to be 600 years old and Carbon 14 dating on woody material found within the deposits have identified older eruptive phases at 2,600 to 8,000 years before the present (Pierson et al., 1992). Fluvial erosion has highly dissected these older pyroclastic deposits. Fluvial processes transported the volcanic material from the upper basins onto the lower-gradient alluvial-fan complex surrounding the mountain. Pre-eruption channels were broad braided systems transporting high volumes of sediment (JICA, 1978) from the eastern fan apex more than 30 km to the distal reaches coalescing from the Pampanga River delta landward of Manila Bay. West side channels were also transporting large volumes of sediment prior to 1991 (Alejandrino et al, personal communications, 1993).

1.4.2 1991 Eruption Deposits. The 1991 pyroclastic flow deposits are massive poorly sorted units, tens of meters thick and generally very non-cohesive. Stratification is common within these thick units. Initial pyroclastic-flow deposits were emplaced as a series of flow events. Stratification differences resulted from various crater explosions at different locations around the mountain and in each upper basin.

Numerous samples of the 1991 pyroclastic-flow deposits and 1991-1992 lahar deposits were taken in several of the basins during the 1992 rainy season and 1992-1993 dry season as part of this study, and were analyzed for gradation and bulk density. Gradation testing showed the pyroclastic deposits to be typically composed of about 75 percent sand sizes with 10 to 15 percent silty fines and 10 to 15 percent coarse sizes (greater than about 5 mm). Bulk densities ranged from about 1.8 to 2.6 kg/m^3 with an average of about 2.3 kg/m^3. Visual observations in the field show the older pyroclastic and lahar deposits to be similar to the 1991 deposits, but more consolidated with age and burial. Greater consolidation thus increases resistance to erosion. Weathering also forms clay minerals that increase the cohesive strength of the sediment and its resistence to erosion. The 600-year old deposits appear nearly identical to the 1991 pyroclastic-flow deposits, generally discernible only by the well-developed vegetation on the top surface. Still older, more weathered and more dense pyroclastic deposits were also observed, presumably representing the eruptive episode of about 2,000 years ago. These deposits are noticeably more resistant to lateral erosion than the younger deposits due to their greater consolidation. Since the lahar deposits are derived from the pyroclastic deposits,

they have similar characteristics except that the coarser fragments tend to be segregated at the tops of individual flow sheets or separated from the main deposit because of their low density; thus in general the coarse fraction is significantly smaller in the lahar deposits. Gradation tests in samples of the lahar deposits generally showed a composition of about 95 percent sand size or finer.

1.4.3 Current Volcanic Hazards. Mount Pinatubo has settled into a period of reduced eruptive activity following the major eruption of June 1991. This pattern is similar to the historical pattern of similar volcanoes. Current activity at the mountain consists mostly of periods of dome growth accompanied by occasional small-scale explosive events. Earthquake activity fluctuates, with many phases of low-frequency tremors. While the mountain is still active, another very large eruptive event is not probable during the next few years. Meanwhile, significant hazards from the 1991 eruption still exist for those living and working near the volcano. The lahar hazard is still very high, especially during the monsoon season, and will continue to be a significant hazard for many years to come. Occasional ashfalls may yet occur in conjunction with secondary eruptions and/or phreatic explosions, decreasing in frequency and magnitude with time. There has been a marked reduction in the reported number of explosive events from 1991 to 1993.

Secondary pyroclastic-flow events represent a continuing and significant hazard, particularly during the rainy season. The random occurrence and size of secondary pyroclastic flow materials make sediment yield and impact predictions extremely difficult, but these will nevertheless be discussed in the basin analysis. Phreatic explosions appear linked to secondary pyroclastic-flow events (exactly how is unknown at this time) but in themselves are not a major factor in sediment production. They are spectacular, resembling small volcanic eruptions, but in the outyears they will be a non-factor.

1.5 Regional Climate

Mount Pinatubo is located approximately 15° N latitude on the west coastal area of Central Luzon. A tropical climate dominates during the Northeast Monsoon season (November through May). The Southwest Monsoon season dominates during the summer months (June-October). This seasonal airflow reversal results in a two-season condition. The Southwest Monsoon generates typhoons (high winds and heavy rain) and associated severe weather conditions. Eighty percent of the region's rain falls from June through September. This wet season coincides with the time of year when intense tropical storms are spawned in the lower latitudes of the Pacific Ocean and pass close to the Philippine Islands. Data available from the Philippine Atmospheric, Geophysical, and Astronomical Services Administration (PAGASA) indicate that between 1948 and 1991, a yearly average of 16 tropical cyclones (tropical depressions, tropical storms or typhoons) affect weather conditions in the region (Northwest Hydraulic Consultants Inc., 1993). Intense localized rainfall is associated with the major cells, which produce severe and intense storms over a small geographical area.

On average, the east side of the volcano receives less rainfall than the west side. Near Mount Pinatubo, the annual rainfall varies from a low of about 1,700 mm at Clark Air Base, Pampanga, on the east, to more than 3,700 mm at Iba, Zambales, on the west. Additional climatological data are provided in Appendix A.

1.6 Regional Topography

1.6.1 Upper Basins/Headwaters. Prior to the initial phase of volcanic activity in April 1991, Mount Pinatubo stood some 1,745 meters above sea level. The upper basin slopes had a dense drainage network with channels incision from 100 to 150 meters deep. Crater wall slopes ranged from 20^0 to 65^0 (Pierson et al., 1992), with channel gradients of up to 400 m/km. Streams flowed through steep, narrow valleys with channel slopes of 0.02 to 0.10 m/m. This upper basin area was densely covered by shrubs and tall grass before the eruption phase. Eight major watersheds drained these slopes through the transition reach, before exiting onto the alluvial fan and flowing to the delta or sea.

1.6.2 Transition Reach. This reach includes areas within the older pyroclastic deposits and the upper parts of the alluvial fans. Pre-eruption channels incision ranged from 60 to 100 meters upstream end, to only a few meters at the downstream end. Channel gradients through this area flatten to about 0.01 to 0.02 m/m at the fan apex (about 200 meters elevation above sea level) with a corresponding increase in vegetation density and diversity. Crops include sugar cane, cassava, and maize. On the west side, the Bucao and Santo Tomas transition reaches are characterized by a mid- to upper-slope channel complex, controlled by the north-south trending mountain range.

1.6.3 Lower Alluvial-Fan Complex. A broad alluvial-fan complex surrounds Mount Pinatubo (see Figure B-1), with the highest degree of geomorphic development and complexity in the eastern sector. The highest population density and agricultural diversity are also on the east side. The Gumain-Porac Rivers drain an area south of the mountain, flowing across the southeastern portions of the alluvial-fan complex. The east-central and northeastern sectors of this fan complex are dissected by the Pasig-Potrero, Abacan, and Sacobia Rivers. The northernmost portion is affected by the O'Donnell River. Channels are confined by natural banks of not more than a few meters, except where the streams have been channelized. Channel gradients range from near 0.02 m/m in the upper reaches to as little as 0.0001 m/km to 0.0002 m/m in the delta. Channels are broad braided systems covering large geographical areas and transporting fine sediment to the delta.

Geomorphically, the west side alluvial fans can be divided into the similar physiographic sections, but different geologic events have created a different landscape. The north-south trending, westernmost mountains of the Zambales Range separate the upper fan from the smaller lowland area. This lowland alluvial-fan complex is best described as a coastal fan complex.

2. SEDIMENTATION ANALYSES AND METHODOLOGIES

The sedimentation analysis for Mount Pinatubo uses methods developed by the Corps of Engineers during its work on the Mount St. Helens recovery program (Corps, 1984 and 1985). The analysis involves three main components: 1) definition of the geomorphic processes active within the basins, 2) a sediment yield forecast, and 3) a sediment deposition forecast. Each component involves a combination of photogrametric analysis, field investigations, literature reviews, and consultation with other engineers and scientists. This section outlines the methods used. The general results of these analyses are discussed in Chapter 3, and basin specific results are presented in their respective chapters.

2.1 Geomorphic Analysis

The geomorphic analysis involved identifying the physical processes actively reshaping the pyroclastic deposits, and determining the importance of each one to long-term sediment production. Aerial photographs, both vertical and oblique, taken of the pyroclastic deposits in 1991, 1992, and 1993 were studied to identify active geomorphic processes. The processes identified were phreatic explosions, secondary pyroclastic flows, lake formation and failure, and rill, gully and channel erosion.

2.1.1 Photogrametric Analysis. The photographs and field observations indicated that channel erosion, lake formation and failure, and secondary pyroclastic flows were the major factors in sediment production. A topographic analysis was conducted to determine the magnitude of each process and the volume of the pyroclastic deposits. Pre-eruption topography was obtained from 1:50,000 scale, 20-meter contour maps prepared by the Defense Mapping Agency, Washington, D.C. Post-eruption topography was produced from November 1992 aerial photography supplied by USAID-Manila. The volume of 1991 pyroclastic deposits was measured by comparing the pre-eruption topography to the immediate post-eruption surface. On the east side, pre- and post-eruption valley cross sections were produced and the pyroclastic volumes computed using the double end-area method. On the west side, digital terrain models (DTMs) were prepared for the Santo Tomas and Bucao basins. The pre- and post-eruption DTMs were then merged in a Unix-based computer workstation and the pyroclastic deposit volumes were computed. Channel dimensions and erosion volumes were obtained from post-eruption cross sections. Volumes for some of the secondary pyroclastic flows were also computed.

2.1.2 Field Investigations. Field visits were made to each basin to verify the process conclusions and define smaller scale features. Both visual methods and geo-positioning systems were used to locate field sites. Spot measurements were made of active channels, channel slopes, and terrace formations. Soil samples were taken and material types in and adjacent to the channels were noted. Soil temperatures were

observed but not measured. During the field visits, specific investigations included channel descriptions, comparative differences in surface and channel conditions between the initial pyroclastic deposits and the secondary pyroclastic flow areas, flow and deposition characteristics of secondary pyroclastic flows, and the formation and failure of lakes.

2.1.3 Literature Review. The geomorphic processes at Mount Pinatubo were compared to those at other volcanoes and to other research results available in the literature. The rapid channel evolution at Mount Pinatubo was compared to that at two other volcanoes, Mount St. Helens (Meyer and Dodge, 1988) and Mount Mayon (Rodolfo and Arguden, 1991) to help identify limiting factors. The conclusions of the Mount Pinatubo studies were compared to the results of laboratory experiments on channel evolution, presented by Schumm, et al (1987). Although the secondary pyroclastic flows occurring at Mount Pinatubo were not found elsewhere, work on coal spoils in Canada (Sasitharanand et al., 1992) provided some information. The conclusions of the geomorphic analysis were also discussed with representatives of the U.S. Geological Survey (USGS), Philippine Institute of Volcanology and Seismology (PHIVOLCS), and outside consultants.

2.2 Sediment Transport Analysis

2.2.1 Objectives. The overall objective of the sediment transport analysis was to develop an understanding of the processes sufficient to forecast future hazards. Standard methods of analysis, such as streamflow and sediment transport measurements and computer modeling were generally not used in this study. The extreme sediment transport concentrations in the lahars precluded the direct application of sediment transport models and the limited scope of this study did not allow for data collection. However, the magnitude and mechanics of the sedimentation problems allowed for a more generalized approach based on consultation with other experts, limited field investigations and aerial photography.

2.2.2 Transport Processes. The significant transport processes were initially described by PHIVOLCS and USGS personnel, and other observers, who were on site during the 1991 rainy season (Punongbayan et al., 1991; Janda et al., 1991; and Umbal et al., 1991). They described mudflows causing several meters of channel aggradation in a single day. During the initial phase of the study, the sediment transport processes were defined as muddy water, hyperconcentrated flow, and mudflows.

During subsequent field visits, muddy water and hyperconcentrated flow conditions were observed in several rivers, but fate did not allow direct observation of mudflow conditions. Discharge and sediment transport measurements could not be made because of the difficult river conditions and limited study scope. Additional observations of transport processes were provided by PHIVOLCS (Daag and Tungol, personal communication, 1992), Zambales Lahar Scientific Monitoring Group (ZLSMG) (Rodolfo

and Umbal, personal communication, 1993), Dolan (personal communication, 1993), and others throughout the study. A limited amount of sediment transport concentration data for muddy water conditions was obtained from the Philippines' Bureau of Research and Standards (unpublished). Mudflow concentration data for three events in 1992 were provided by ZLSMG (Rodolfo, written communication, 1993).

Sediment transport calculations were made for some basins to estimate long-term potential erosion/deposition conditions downstream of the hyperconcentrated flow and mudflow zones. On the basis of experience at Mount St. Helens (Eriksen, 1989), Yang's sediment transport equation was used for these calculations.

2.2.3 Sediment Deposition. During initial field trips in September 1991 and February 1992, muddy water, hyperconcentrated flow, and mudflow deposits were identified. On subsequent visits, deposits from each of the three sediment transport processes were examined in the field. During field visits, deposition areas were photographed and deposit depths were estimated. Field data and aerial photographs were used to map the extent of the different deposits. In November 1992, a field trip was made for the express purpose of examining August and September 1992 mudflow deposits in the Pasig-Potrero and Sacobia-Bamban rivers.

2.3 Sediment Forecasts

After an understanding of the active processes was developed, the next step was to prepare a sediment yield forecast. The key elements in the forecast are the sediment available for rapid erosion, the initial rate of erosion, and the predicted rate of decline in sediment yields.

2.3.1 Sediment Available. The geomorphic analysis determined that the extremely high sediment yields were the result of rapid channel erosion (many times higher than pre-eruption levels, with transport occurring as hyperconcentrated flow or mudflows) and that these high yields would continue until the main channels reached a "stable" cross-sectional geometry. Judgments were made about the dimensions that each main channel would have when it reached a "stable" condition, based on the geomorphic analysis, pre-eruption channel dimensions, hydrology, and local geologic conditions. The "stable" channel dimensions were plotted on 1992 channel cross sections and the sediment available for rapid erosion was the material remaining within the boundaries of the "stable" main channels.

To forecast the potential sediment yield from secondary pyroclastic flows, sites with topographic and geologic characteristics similar to previous secondary pyroclastic flows sites were identified and potential volumes computed. Where appropriate, the volumes were then added to the sediment available from the main channels to arrive at the total sediment available for rapid erosion.

2.3.2 Initial Erosion Rate. The next step in developing the sediment yield forecast was to determine what the initial average annual sediment yield would be for the first year. This was done by multiplying the average annual storm runoff by an average sediment transport concentration. The storm runoff volume was approximated by the volume of the upper 10 percent time period of the flow-duration curve for each of the pyroclastic drainages (computed during the hydrologic analysis described in Appendix A). An average sediment transport concentration during storm runoff had to be estimated from field observations and discussions with USGS, PHIVOLCS, and ZLSMG personnel, as no suitable data were available. A sediment transport concentration range of 25 to 30 percent by volume was considered representative of the average storm runoff concentration. While concentrations have been higher, this figure near the middle of the hyperconcentrated flow range, seems to reflect an overall average.

2.3.3 Sediment Yield Forecast. Using the initial average annual yield as a starting point, the total sediment available was then distributed over time to generate the sediment forecasts. Annual sediment yields for each succeeding year were generally proportioned downward according to the ratio between the amount of available sediment remaining and the initial sediment available.

2.3.4 Sediment Deposition Forecast. Sediment deposition investigations also utilized field visits, aerial photography, and discussions with PHIVOLCS, ZLSMG, and USGS staff. In addition, there were discussions with DPWH staff and a limited number of river cross sections surveyed by PHIVOLCS.

Sediment deposition was found to be governed by the transport mechanism (muddy water, hyperconcentrated flow or mudflow) and the local topography. The type and location of deposition caused by each transport mechanism was identified in the field. Topographic maps were used to delineate areas threatened by each process. For purposes of economic analysis, the potential deposition reaches were divided into inner and outer zones, and probabilities of being impacted were assigned.

2.3.5 Storm Event Sediment Yields. Sediment yields were computed for large, infrequent storm events for use in designing sediment control measures. Sediment volumes and water-plus-sediment peak discharges were estimated at each sediment control measure site. The upstream basin area and clear water flows at each site were obtained from the hydrologic analysis presented in Appendix A. Calculations were made using the peak discharge and the highest 3-day volume for the 2-, 10-, 50-, 100-, and 500-year storm events in each basin.

Sediment concentration data do not exist for large storm discharges on any of the impacted rivers. Therefore the sediment concentrations were assigned on the basis of watershed conditions and the type of sediment transport mechanism expected at each design site. In the rivers with pyroclastic deposits, at sites where mudflows were expected, a concentration of 67 percent by volume was assigned to the peak discharge

and peak day volume, and a concentration of 40 percent by volume was assigned to the next highest 2-day volumes. At sites downstream of the mudflow zones, a concentration of 40 percent by volume was assigned to discharges for the peak and the three highest days. A concentration of 10 percent by volume was assigned to discharges from basins having no pyroclastic deposits.

This method provides for very large sediment yields during infrequent storm events. As time passes, the same geomorphic processes that reduce annual sediment yields will also work to lower the sediment transport concentrations during storm events. The concentrations used provide appropriately high volumes for design of control measures.

2.4 Sediment Distribution Model

2.4.1 General. A sediment distribution model was developed for each basin to provide input to the economic analysis. The model simulated erosion of the pyroclastic material by storm events and routed the sediment downstream until it was deposited or discharged by the rivers. The model uses Monte Carlo simulation involving numerous iterations to incorporate some of the natural variations and knowledge uncertainties of the sedimentation processes.

2.4.2 Sediment Yields. Sediment yield functions were developed to account for the sediment available and potential erosion rates described above. The rainfall/runoff function provided for a normal distribution of annual storm events, centered around the storm runoff analysis described in section 2.3.2. Secondary pyroclastic flows were assigned probabilities of occurrence and potential volumes that could be added to the sediment available for erosion. Sediment concentrations were a function of runoff and sediment availability, with the first year's sediment transport concentration averaging approximately 25 to 30 percent by volume. The maximum sediment concentration for any storm event was limited to 60 percent by volume.

2.4.3 Sediment Distribution. Sediment yields were distributed throughout the river basins as functions of sediment transport concentration, flow depth, bank height remaining, levee conditions, and topography. Deposition in the channel was controlled directly by the inflowing concentration, but overbank distribution also involved probability functions for levee failure and flow paths. The probability of a hazard zone being impacted was based on the results of the analysis described in section 2.3.4.

2.4.4 Model Limitations and Results. As noted above, this model is a simulation model, the sediment forecasts from the model, approximately imitate the sediment forecasts presented in this report. The Monte Carlo simulation provides a statistical representation of the possible natural variations in sediment yields that are useful in assessing the potential damages or economic benefits. The internal formulation of the model is not physically based, so the model can not be used as an analytical tool. The results from the model provides at least some indication of the variations that can be expected in the sediment yields and distribution. Those variations are reflected in the economic analysis presented in Appendix C.

3. GEOMORPHIC DEVELOPMENT AND SEDIMENT TRANSPORT

3.1 General

The 1991 eruption left 5.6 billion m^3 of pyroclastic flow deposits in the upper watershed around Mount Pinatubo. The deep river valleys near the mountain were filled with up to 200 m of deposits. Since their emplacement, these pyroclastic deposits have experienced rapid geomorphic changes. The formation and subsequent erosion of channels in the deposits have caused many lahars that have done extensive damages in the populated areas lower in the basins. Secondary pyroclastic flows and phreatic explosions have reshaped the deposits, but their impacts have generally been limited to upper basins.

3.2 Pyroclastic-flow Deposits

3.2.1 Emplacement. Pyroclastic flows are formed from the combination of hot volcanic clasts, lithic rock fragments, and gases traveling down the volcano's flanks at gravity-induced velocities of 10 to 300 meters per second (Carey, 1991). Under the influence of gravity pyroclastic flows produce thick and geographically widespread deposits. Most of the pyroclastic-flow material from the initial (June 15) eruption was emplaced between 5 km and 15 km from the new crater (see Figure B-2). Only a thin veneer of new material was retained on the upper 3 to 5 km of the crater's flanks.

Incised valleys in Mount Pinatubo's upper basin areas appear to have had a direct influence on flow and depositional patterns. The high velocity pyroclastic flows appear to have crossed the high, flat plateaus and accumulated in the deep valleys. Only a thin deposit of pyroclastic and airfall ash materials remained on the plateau surfaces. Deposits in the upper valleys surrounding Mount Pinatubo range from 200 meters thick in the proximal area to 50 meters thick at the distal areas. Those deposits tend to be massive poorly sorted units, tens of meters thick and generally very non-cohesive. Stratification is common because the material was emplaced by a series of flow events. Stratification differences resulted from differing material exploded from the crater and varying geographical locations.

Topographical lows in the crater rim sector may have influenced the pyroclastic flow vectors. Prior to the eruption, a large, deep low existed in the northwest side of Mount Pinatubo. Nearly half of the total accumulation of pyroclastic flow deposits are located in this sector. During the eruption, smaller topographical lows formed in the rim at the heads of the other basins containing large pyroclastic deposits.

3.2.2 Pyroclastic Deposit Geotechnical Characteristics. Recent pyroclastic and lahar deposits consist predominantly of sand-sized particles which are angular and composed primarily of quartz, volcanic glass, feldspar, and other high-silica minerals consistent with a dacitic magma source. The pyroclastic deposits are 10 to 15 percent pumice, ranging from fine gravel to boulder sizes, and 10 to 15 percent non-plastic fine material. The deposits are unconsolidated and massive. The older pyroclastic deposits are similar except that they are somewhat weathered and appear to be well consolidated with a higher clay component.

Material deposited on the flanks of the volcano consists of ash, rockfall and pyroclastic material from secondary crater eruptions occurring after June 15, 1991. Deposits of lithic cobbles and boulders can also be found in these areas.

3.2.3 Secondary Pyroclastic Flows. Secondary pyroclastic flows are mass movements of material that have occurred after emplacement of the primary pyroclastic deposits. Figure B-3 shows the general location in the basins of these major mass movements. Two features common to most, but not all, secondary pyroclastic flows are; failure scarps at points of contact between pre-eruption valley walls or channel surfaces and the initial pyroclastic flow deposit, and deeply eroded channels across the toe.

The triggering mechanisms and flow mechanics of secondary pyroclastic flows are still poorly understood. They evidently tend to occur during the rainy season. Perhaps infiltration of rainwater adds enough weight to the deposits that the shear strength of the soil is exceeded. Liquefaction might also be a triggering mechanism, even though the material has a low water content. The investigations of Sasitharanand et al. (1992) into catastrophic flows of well-drained coal spoils in Canada indicate that as little as 10 percent water by volume may be enough to induce flow. Phreatic explosions are another possible triggering mechanism.

Secondary pyroclastic flows can carry large volumes of material many kilometers downstream. Work by Francis and Baker (1977) suggests that large-volume pyroclastic flows are far more mobile than other particulate gravity flows, with deposits abundant in ash or pumice showing the most pronounced mobility. Mount Pinatubo's pyroclastic-flow deposit contain both pumice and fine-grained airfall ash. Once flow is initiated, some type of bulking and increased fluidization seems to occur, the resulting flows appear to range in concentration from debris flows to mudflows to muddy water flows (see Figure B-4). Down-slope travel distances vary, depending on channel configuration and geometry. Down-slope movement of these flows at Mount Pinatubo has been measured at up to 8 km.

Secondary pyroclastic flows can fill downstream channel reaches, increasing the amount of sediment available for future erosion. They may also cause channel blockages, creating in-channel and/or tributary lake formation. Sediment may be temporarily stored, with subsequent lake breaching producing mudflows.

Analysis indicates that once movement has occurred, the scarp and failure surfaces are less susceptible to fluvial erosional processes, perhaps as a result of increased porosity and permeability. Field observations show a lag type of pumice gravel deposit present on these failure surfaces. A few mass movement areas have undergone some fluvial erosion and channelization, however, analysis indicates that erosion was initiated up-slope and migrated across the failure surface.

The potential magnitude and temporal and spatial distribution of secondary pyroclastic-flow events are unknown at present.
The potential for activity remains high in the near future. Such events can produce catastrophic basin modifications that may either increase or decrease the amount of sediment available for erosion. This risk and uncertainty impacts both the short and long term sediment budget.

3.2.4 Phreatic Explosions. While rapid erosion launched the development of new drainage networks, dynamic phreatic explosions also occurred in the deposit, producing ashfall and actively increasing the erodibility of the initial deposit. Such explosions influence channel formation but are not by themselves major sediment producers. Craters from the explosions filled with water to form ponds and lakes. Overtopping of these and other depressions in the pyroclastic-flow deposit also aided the redevelopment of channel networks. Water accumulation in these depressions may have increased phreatic explosion activity.

Phreatic explosions occur mainly during the rainy season when infiltration increases the amount of available moisture coming into contact with hot primary pyroclastic-flow deposits. The process of flash-to-steam upon contact between water and hot volcanic sediment is one of the driving factors. In addition, the action of streamflows undercutting banks in the pyroclastic deposit resulted in bank failure, causing phreatic explosions from contact of channel water with hot pyroclastic sediment.

Spatial and/or temporal predictability of future explosions is currently not possible. These events may continue for several (up to 10) more years, especially in the basins containing the greatest thicknesses of initial pyroclastic-flow deposits, but their frequency should drop off over time as the deposits cool and channel stability increases.

3.3 Drainage Development and Erosion Processes

3.3.1 General. The pyroclastic flow deposits were initially featureless, domed (high-centered) plains following longitudinal axes down the valleys. Initial drainage development occurred very shortly after pyroclastic flow emplacement. The non-cohesive nature of the deposit and other geomorphic processes acting on the material combined rapidly to form a new and highly complex drainage network. A major tropical storm that was battering Central Luzon on June 15, 1991, also helped re-establish drainage networks within a few days. The deposit geometry aided in channel network

redevelopment, with small rills and gullies forming along the center and draining into larger channels along the valley margins.

3.3.2 Rills and Gullies. Rill and gully erosion was the first step in re-establishing the drainage network on the pyroclastic deposits. It was also an important erosion process on the ash covered mountains around Mount Pinatubo.

Rill and gully formation on the pyroclastic surfaces was very rapid in 1991, creating intricate drainage patterns. These small channels were only a few meters wide and deep, but because they were so numerous they produced significant quantities of sediment in 1991. They also provided a highly efficient drainage system for the pyroclastic deposits main channels. Analysis of the 1992 and 1993 rills and gullies found that they had not grown significantly larger than they were in 1991. Rills and gullies are not expected to be important sources of sediment in the future.

Rain events after June 1991 eroded airfall ash from the mountain slopes and deposited it in many river channels. Most of the airfall ash was removed from the mountain slopes during the first rainy season (1991), resulting in large sediment yields to the rivers. The Gumain, Porac, and Maloma rivers, which had no major pyroclastic deposits within their headwaters, were impacted by this erosion and deposition of airfall ash in the lowland channels. These streams are still heavily laden with ash.

3.3.3 Channel Development. Initial drainage development occurred once the pyroclastic flow deposit was in place. The non-cohesive nature of the deposit combined with high rainfall to rapidly form a new and highly complex drainage pattern. The main channels developed predominantly by headcutting and incision of gullies and small channels. As small channels merged, the in-channel discharges increased and the erosion potential also increased. Channel development processes were similar to those identified at Mount St. Helens by Pearson (1986).

The dome-shaped geometry of the initial pyroclastic-flow deposit helped direct channel development toward the margins of the deposits (Tom Pierson, USGS, personal communication, August 1992). The channels along the deposit margins received additional runoff from the adjacent mountains, further increasing their erosion potential. Even during the first year, many of these channels incised 20 to 40 meters into the pyroclastic deposit. Bank collapses temporarily blocked some channels, causing mudflow or hyperconcentrated flow surges when the blockage was overtopped and eroded. Additional mudflows were created by the violent mixing of hot bank material into rapidly flowing discharges. Channel widening due to bank erosion was the dominant sediment source in 1992 and 1993.

The post-eruption stream profiles are elevated above the pre-eruption channels, but show the same general slopes of 4 to 5 percent. The hinge points, the locations at which channel slopes increase, were moved downstream by the pyroclastic deposits. The

resulting slope relations would suggest stream energy may not have increased. The post-eruption channel lengths have increased an average of 2 km. Without an increase in hydrology or stream energy, the river systems have about the same potential transport limits as they did prior to the eruption. Sediment bulking of the flows to hyperconcentrated or mudflow conditions appears to be limited to bank failure mechanisms.

An unusual factor that contributed to channel formation was the overtopping of ponds and lakes formed in craters from phreatic explosions. Analysis of aerial photographs taken in October-November 1991 by the GOP indicates that phreatic explosions occur primarily at points of contact between pyroclastic-flow deposits and pre-eruption valley side slopes. The contact of hot sediment and cold groundwater from the hillsides is one explanation for phreatic explosions along the deposit margins. The potential magnitude and distribution of sediment generated for transport by this geomorphic process is unknown, but its importance is decreasing with time.

3.3.4 Lake Formation and Blockage Failure. Lake formation and failure has been an important sediment producing process in some basins. Its importance has been dependent on a number of geomorphic factors including, pre-eruption basin geology, topography, hydrology, and the initial pyroclastic-flow deposit. Those factors and the resulting sediment impacts have been quite variable from basin to basin. Lake failures have been most serious in the Pasig-Potrero and Santo Tomas basins.

Lake breakouts were a significant mode of large lahar generation in the Pasig-Potrero basin during the first two years following the eruption. A side drainage of the Pasig basin was blocked by secondary pyroclastic flows during both 1991 and 1992, and a lake developed behind the blockage. Both blockages were overtopped and eroded, causing large lahar flows that damaged areas on the lower alluvial fan. The potential for future lake formation has been reduced, because the tributary has filled with sediment and the occurrence of the secondary pyroclastic flows has reduced the potential for more such events.

Mudflow events in the Marella River blocked the Mapanuepe River, also forming a lake. This lake underwent a series of blockages, failures, and mudflows in 1991 and 1992. Construction of an outlet to control lake water level prevented any lake failures in 1993. Nevertheless, there is still some threat of future lake breakouts.

Blockages have formed lakes in other basins without causing serious lahar problems. An unnamed lake formed in the Sacobia basin, just upstream of the Gates of the Abacan, and drained in August 1991 without generating a noticeable lahar (Scott, K. M., USGS, oral communication, 1992). A second lake in the Sacobia basin formed on the Marimla River upstream of Bamban. This lake was formed by mudflows on the Sacobia-Bamban River blocking the valley. This lake has overtopped the blockage, but the blockage has not failed. Several small lakes have also been observed in the Bucao River basin.

3.4 Transport Processes

3.4.1 General. The term "lahar" refers to any rapidly flowing mixture of volcanic material and water. The terms "muddy water," "hyperconcentrated flow," and "mudflow" are used in this report to refer to lahars of increasingly higher sediment transport concentrations. Figure B-5 gives a visual comparison of relative erosion and deposition factors for these various sediment transport mechanisms, showing how sedimentation impacts vary in magnitude and location with the different mechanisms. Each type of lahar presents a different hazard to areas downstream of Mount Pinatubo.

3.4.2 Muddy Water. Muddy water refers to sediment transport concentrations of less than 20 percent by volume, which are typical of storm runoff concentrations observed around the world. However, at Mount Pinatubo such flows occur during minor rainfall events and also under base flow conditions. Suspended sediment samples collected from impacted streams during base flow conditions before and after the 1991 eruption (Bureau of Research and Standards, unpublished) indicate a post-eruption increase of 10 to 100 times the pre-eruption base flow sediment transport levels. Post-eruption concentrations on impacted streams range from near 100 parts per million (ppm) to over 10,000 ppm. The sediment entrainment process for muddy water flows appears to be common bed and bank erosion.

Muddy water flows have caused both erosion and deposition damages, carrying and depositing sediment as far as Manila Bay and the South China Sea. They have also caused infilling of the lower reaches of the Abacan, Gumain, Pasac, Maloma, Olongapo, and Tanguay Rivers, and have caused major damage in the form of toe erosion to levees on the Abacan, Bamban, Gumain, and Santo Tomas Rivers.

3.4.3 Hyperconcentrated Flow. Hyperconcentrated flows generally have sediment transport concentrations between 20 and 50 percent by volume (Julien and Lan, 1989). The action of these flows is similar to that of muddy water flows, but they can transport very large sediment volumes in a short time. The sediment entrainment process seems to require some extraordinary mechanism to raise concentrations above the muddy water range. At Mount Pinatubo, it appears that this mechanism is the collapse of high, hot pyroclastic banks into the rapidly moving storm runoff in confined channels. Once entrained, the concentrations tend to remain high until the flow loses energy down in the valley.

Observations of storm events on the Sacobia River by Daag and Tungol (personal communication, 1992) and Dolan (personal communication, 1993) and on the Marella River by Rodolfo and Umbal (personal communication, 1993) indicate that discharges undergo transition from muddy water to hyperconcentrated flow and then may alternate between hyperconcentrated flow and mudflow before receding back to muddy water base flow conditions. The only hyperconcentrated or mudflow sediment transport concentration data available for analysis (Rodolfo and Umbal, written communication,

1993) support these observations. These data show sediment transport concentrations on the Marella River rising from around 10 percent by volume to nearly 86 percent by volume during a storm on June 27, 1992. They also show a series of measurements taken on July 11, 1992 that fluctuate between 30 and 60 percent by volume. Discharge measurements for those sediment transport concentrations are not available, nor have discharges for any other storm event been measured due to the difficult river conditions. The high sediment load, rapidly shifting channels, and high water temperatures during hyperconcentrated flows or mudflows make measurements very difficult.

Because hyperconcentrated flows behave like muddy water, they can cause flooding and sediment deposition over broad areas of the lower valleys. The most damaging hyperconcentrated flow events have been on the Sacobia and Abacan Rivers in August 1991, and on the Pasig-Potrero River in September 1991 and October 1993. In each of these events, hyperconcentrated flows breached levees and caused widespread damage. Sediment layers deposited in overbank areas by hyperconcentrated flow events were typically less than one meter deep, with nearly flat surfaces. Soil samples from the deposits showed them to be similar in composition to the pyroclastic deposits, with little variation across the floodplain.

3.4.4 Mudflow. Mudflows have sediment transport concentrations of over 50 percent by volume and are commonly said to resemble rapidly moving wet concrete. Like hyperconcentrated flows, mudflows are formed by the collapse of high, hot pyroclastic banks into rapidly moving storm runoff in confined channels. Mudflows (also called "debris flows" in some technical literature) are non-Newtonian fluids that have shear strength (Julien and Lan, 1989). Mudflows continue to move as long as the shear stress exceeds the shear strength. At Mount Pinatubo, mudflows in confined channels have traveled through channel reaches with slopes of only 0.015 m/m.

Mudflows are also capable of building natural levees along their flow margins that serve to keep the flow confined. Mudflow levees have been observed to have surface slopes perpendicular to the main flow path of approximately 2 to 4 percent. Some very large Mount Pinatubo mudflows, confined by their own levees to widths of 200 to 700 meters, have traveled several kilometers across valley reaches with slopes approaching 0.006 m/m.

As noted in the hyperconcentrated flow section above, there is a scarcity of available measured data for Mount Pinatubo mudflows.

Mudflows present a unique hazard, not only because they move very large volumes of sediment very quickly, but also because their depositional characteristics are different from those of muddy water or hyperconcentrated flows. Mudflow deposition can occur on slopes of between 0.006 and 0.02 m/m. Most of the mudflow deposition around Mount Pinatubo has occurred in the transition channel reaches between the base of the

pyroclastic deposits and the gently sloping valley floors. Typical of these reaches are the Marella River upstream of Mapanuepe Lake and the Sacobia River near Clark Air Base.

Mudflow deposition seems to occur in two main ways, either as shallow outwash or as massive units. Shallow outwash deposits occur when the mudflow exceeds channel capacity and fluid spills out onto the overbanks and dewaters. It appears that the transition channels have aggraded through this process and that the mudflow levees are also built this way.

Most mudflow deposition seems to occur in massive units. It appears that as the front of a mudflow is stopped, either by loss of energy or by an obstruction, the entire flow for a kilometer or more upstream may also stop. The patterns of abandon flow paths observed in deposition zones after large mudflows suggest that as the flow stops, a number of reactions can follow: breaches can occur in the natural levees allowing the mudflow to continue on another path, subsequent flows can be diverted near the upstream end of the deposit, or subsequent flows can flow up and over the earlier deposit.

Single mudflow events around Mount Pinatubo have created in-channel deposits up to 7 meters thick. This process has filled several river channels, enabling later flows to spill onto the overbanks. These overbank deposits are in the general range of 1 to 2 meters deep, rarely exceeding 3 meters at the deepest.

The mudflow deposits are similar to the pyroclastic source deposits, but contain a smaller percentage of coarse pumice fragments and may also contain other debris such as wood. The mudflow deposits tend to have a massive (1 to 3 meters) thick bedding structure and some upward sorting by particle size.

3.5 Sediment Forecasts

3.5.1 General. Three types of sediment forecasts are presented in this study. The first is the 10-year forecast of annual sediment yields and deposition patterns. The second is a more generalized long-term forecast of sediment yields and related sediment problems, covering the 25 year project life. The third is a forecast of sediment yields during infrequent storm events that was used to determine storage requirements for sediment control measures.

3.5.2 10-Year Forecasts. Ten-year forecasts were made for basins that contain significant volumes of pyroclastic deposits. These included the Pasig-Potrero, Sacobia-Bamban, O'Donnell, Bucao, and Santo Tomas basins. It was found that a rapid decline in erosion rates could be expected during the next five years in all basins. This expectation is based on the comparison of erosion rates and channel dimensions found at other volcanoes.

At Mount St. Helens (USACE, 1984), sediment yields were very high during the first year as the streams reestablished drainage networks. Stream gradients stabilized greatly after the first year and channel widening became the dominant sediment-producing process. As the channel widths increased, the collapse of high banks directly into flowing water became less frequent and sediment yields declined. At Mount St. Helens, annual sediment yields had declined by 75 percent within five years and have remained around that level (USACE, 1993). Cross sections of mudflow-producing channels at Mount Mayon presented by Rodolfo and Arguden (1991) show a similar trend of channel widening with only minor incision after the first year. These field observations are supported by the experimental results of Schumm et al. (1987). The experimental channels responded very quickly to changes in elevation when a narrow channel was incised to reestablish a more stable gradient. The incision period was then followed by a prolonged period of channel widening and high sediment yields. As the widening slowed, sediment yields declined proportionally.

Since the channel gradients appear to have been reestablished in the first year or two, the width of the "stable" channel then determined the amount of sediment available for future rapid erosion. The maximum amount of channel widening that could occur before the channels became "stable" was set at 300 meters -- approximately the maximum width obtained by channels at Mount St. Helens. In most cases, channel widths at Mount Pinatubo were less than this maximum value, being limited by local features such as pre-eruption geology, runoff volumes, or the widths of pre-eruption channels in older pyroclastic deposits. In some basins an additional volume was included to account for the likely occurrence of secondary pyroclastic flows.

The first-year sediment yield for each basin was determined by multiplying the average annual storm runoff by an assumed sediment concentration (see Section 2.3.2). Subsequent annual yields were roughly proportioned according to the ratio between the initial sediment available and the sediment available at the end of each year. The resulting sediment forecasts tend to decline according to trends predicted by Pierson et al. (1992). Figure B-6 shows the declining trend of an idealized annual sediment yield forecast. Specific basin forecasts are presented in following sections of this Appendix.

These sediment yield forecasts reflect average annual conditions. Variations in sediment yields can be expected due to above- or below- average rainfall, the presence or lack of secondary pyroclastic flows, or major changes in channel alignments. Sediment yields in any one year may therefore be above or below the forecast yield, but the overall trend should follow the forecast.

3.5.3 Long-term Forecast. The long-term forecasts are the conditions expected after channel conditions become significantly more stable. Annual yields during this time are governed by water availability and normal muddy water sediment transport. Before the eruption, all the impacted rivers were broad, braided channels with sandy bed material. The long-term sediment yields and related problems are expected to be very similar to pre-eruption conditions. Specific basin conditions are discussed in following sections of this Appendix.

3.5.4 Storm Event Sediment Yields. The sediment yield during large, infrequent storms, such as typhoons, must also be considered in determining the overall hazard and in designing control measures. An event of the magnitude of a 100-year or larger flood may be capable of producing more sediment than several years of near-average flows. Without specific data, the sediment volumes that are forecast for large storm events can only be considered gross estimates (see Section 2.3.5). Storm event yields are likely to decline for the same reasons as annual yields, but no attempt has been made to account for that likely reduction. Specific volume estimates for storm events in each basin are given in Appendix E. Figure B-7 shows the effect that a large sediment yield from a single large storm occurring in 2010 could have on the declining annual sediment yield trend.

These types of events are not directly incorporated into the annual sediment yield forecast because they are rare occurrences. The sediment volumes for 100-year flood events were included in the designs of the sediment control measures.

4. PASIG-POTRERO RIVER BASIN ANALYSIS

The Pasig-Potrero basin is an area of 77 km^2 originating on the eastern flank of Mount Pinatubo, draining the center section through a deeply incised reach, and exiting the valley onto the alluvial fan-complex above Mancatian. The headwater area is drained by four streams, the Bucbuc, Yangca, Timbu, and Papatac Rivers, which combine to form the Pasig-Potrero River. The upper basin area is about 14 km long with an area of approximately 10 km^2. The uppermost headwaters originate 13 km from the crater.

The following basin analysis is limited to basin conditions prior to October 1993. A natural diversion of the Sacobia River headwaters into the upper reaches of the Pasig-Potrero River basin occurred during October 1993. This increase in drainage area will greatly increase the flow in the pyroclastic deposit main channel and correspondingly increase sediment yields. The increase in sediment yields has not been fully analyzed or included in this report, but is expected to be very large, perhaps as much as 50 to 100 million m^3 in 1994 above the amounts forecast in the report. The new basin conditions are judged to present an extreme threat to communities along the Pasig-Potrero River.

Initial basin analysis between August 1991 to December 1992 indicated that the massive volume of volcanic material deposited in the upper basin presents a number of flooding and sedimentation hazards to the upper and lower alluvial-fan complex. Large-scale secondary pyroclastic flows, lake blockage failures, and rainfall/runoff processes could generate large volumes of sediment, overwhelming the channel system, breaching levees, and impacting large regions of the alluvial-fan complex. The area from Mancatian to Potrero and Bacolor could receive the greatest impact. Sedimentation and shallow flooding would be the dominant processes. Sedimentation within the channel system could also cause levee failures and out-of-channel impacts (USACE, 1993).

Sediment discharged from the Pasig-Potrero upper basin will cause deposition in the Guagua and Cama Chiles Rivers. High sediment yield rates to the lower alluvial-fan complex could cause ponding and increase the potential for severe pondage flooding around Bacolor, San Fernando, Minalin, and Santo Tomas (see Figure B-1).

4.1 Sedimentation History

4.1.1 Pyroclastic Deposits. On June 15, 1991, pyroclastic avalanches flowed down the eastern slope from the exploding crater of Mount Pinatubo, traveling at high speeds under the influence of gravity and following pre-eruption topography, and deposited 302 million m^3 of pyroclastic-flow material in the upper basin. The highly incised pre-eruption channel geometry influenced the pyroclastic-flow depositional pattern. A plateau 1.5 km wide and 4 km long, 11 km from the newly formed caldera, separates the upper Pasig-Potrero and Sacobia River basins. During the initial pyroclastic-flow emplacement, material flowed across this plateau and cascaded into the

deep Pasig-Potrero basin (see Figure B-8). A change in momentum occurred and deposition was initiated, partially filling drainage channels carved into older pyroclastic deposits from previous eruptions, the most recent dating back approximately 600 years. Maximum thickness of the new deposits ranges from about 150 meters in the middle to upper reaches to 30 meters in the upper basin 13 km from the crater.

The pre-eruption stream channel cuts through pyroclastic deposits in the upper reaches of the basin, and through sediment and alluvial deposits in the upper alluvial-fan reaches. The sedimentary deposits in the upper reaches are underlain at variable depths by bedrock consisting of andesite, basalt (dikes as well as flow units), agglomerates, conglomerates, tuff breccias and tuffaceous sandstones. The lower reaches consist of a broad alluvial-fan complex which has been under formation for at least 600 years, as sediments from higher in the basin have been mobilized by erosional processes and redeposited downstream.

The drainage boundaries and areas used for the hydrologic analysis of the Pasig-Potrero River basin are shown in Appendix A of this document. Figure B-8 shows the upstream half of the drainage areas while Figure B-9 shows the downstream drainage area and hazard zones. The lower basin was divided into mudflow and flooding zones for sediment impact analysis. Immediately north of Bucbuc Creek, pyroclastic deposits covered and obliterated the pre-eruption drainage network. Following the eruption, a new drainage network quickly formed. Approximately 1.5 km^2 of the pre-eruption Bucbuc Creek drainage now contributes flow to Timbu Creek.

4.1.2 <u>1991-1993 Erosion and Deposition</u>. Sediment yields in the Pasig-Potrero basin have been dominated by a few large events. These include lake failures in 1991 and 1992, and intense tropical storms in 1992 and 1993. Typhoon Kadiang, in October 1993, not only produced a very high sediment yield, it also triggered the diversion of over 20 km^2 of the Sacobia River drainage area into the Pasig-Potrero basin. The full impacts of this diversion have not yet been determined.

The pre-eruption channel geometry at the confluence of Papatac and Yangca Creeks favors rapid sediment accumulation due to a natural constriction and a reduction in slope. The rapid narrowing of the pre-eruption channel caused the June 1991 pyroclastic flow to deposit approximately 5 million m^3 of material in the confluence area. The blockage measured 100 to over 400 meters in width. During the first rainy season the blockage functioned as a dam and grew to an approximate volume of 10 million m^3 before its failure on September 7, 1991. The failure rapidly eroded the entire 10 million m^3 of sediment, causing it to cascaded onto the alluvial-fan complex (USACE, Portland District, June 1993). An additional 10 to 15 milion m^3 of material was eroded from the Pasig River as this flood moved downstream. The Potrero River levees were breached near Potrero and there was flooding in Potrero, Santa Barbara, Bacolor, and Gaugau. The type and location of the sediment deposits suggest this flood was a hyperconcentrated flow.

A second blockage formed in early August 1992 as the result of a large secondary pyroclastic flow above Yangca Creek on Papatac Creek (Figure B-8). This blockage was larger than the first, with a maximum elevation of 380 meters along the eastern edge. More than 20 million m^3 of material filled the confluence area and Papatac Creek as far downstream as its confluence with Timbu Creek. During a tropical storm on August 29-30, 1992, the blockage was overtopped, but this time it eroded at a slower rate. The August 29-30 storm and another on September 3-4, 1992 combined to deliver 10 million m^3 of sediment to the lower Pasig-Potrero River basin. Both these events were mudflows and they deposited 7 to 10 km upstream of the 1991 event. Deposition filled the channel at Mancatian and for 4 km downstream. Mudflow deposits in the overbank areas were 1 to 2 m deep around Mitla and about 1/2 m near Balas.

Typhoon Rubing on August 17, 1993, caused a mudflow and 3 to 5 million m^3 of deposition around Mancatian. The deposits filled the channel for about 1 km upstream of Mancatian and damaged the northern portion of the barangay. Flooding occurred in Santa Rita, where lahar flowed through an uncompleted portion of the levee. On October 4, 1993, typhoon Kadiang caused hyperconcentrated flows and mudflows that delivered another 20 to 25 million m^3 of material to the lower alluvial fan. Most of this sediment was deposited in the southern overbank from Mancatian to Gaugau. Santa Rita was heavily damaged by the 1993 lahars.

Analysis of channel cross sections taken from aerial photographs indicate that main channel erosion also has been a large source of sediment. From June 1991 to November 1992, main channel erosion was 23 million m^3, with an unmeasured additional amount in 1993. Figures B-10 and B-11, show the v-shaped main channel geometry that existed in November 1992. An unknown volume was also eroded from tributaries and rill and gully systems on the pyroclastic-flow deposit. The main channel developed along the base of Mount Dorst. The new channel appears to be locked against the valley wall. New craters or sinks formed from phreatic explosions are modifying the drainage system, temporarily trapping volumes of sediment within the basin and reducing initial sediment yields.

Downstream from the pyroclastic deposit the channels have undergone a complex response to the increased sediment loads. The beds of Papatac and Timbu creeks, and the Pasig River were eroded 5 to 10 m in 1991, as far downstream as Mancatian. Papatac Creek's bed was raised by the secondary pyroclastic flow in early August 1992, only to be eroded again in late August 1992, when the lake overtopped the blockage. From the Mancatian to the confluence with the Guagua River, sediment deposition raised the channel bottom elevation of the Pasig-Potrero River by 2 to 4 meters. The Manibaug Pasig Creek still enters the Pasig-Potrero River, but downstream of this confluence other pre-eruption tributaries are now lower than the river and consequently are no longer contributing flow. Loss of tributary flow has reduced the drainage area of the Pasig-Potrero River approximately 78 km^2 at the confluence with the Guagua River.

4.2 Sediment Forecast

4.2.1 Sediment Producing Events. Sediment production from the upper basin is expected to remain high. Main channel erosional processes are expected to dominate future sediment yields. Pyroclastic-flow deposits from about 4 km of the main channel above Yangca Creek (see Figure B-8) will provide most of this sediment. As shown on Figure B-10, channel geometry is still V-shaped along the base of Mount Dorst. Inflowing clean water appears to have stabilized the channel location. Bank failures from phreatic explosions will continue to occur. Large secondary pyroclastic flow events in 1991 and 1992 have also increased the stability of this channel reach. Channel reaches below Yangca Creek appear to fluctuate within 5 to 10 meters plus or minus pre-eruption elevations. From this area down to Mancatian, channel geometry is trapezoidal and 50 or more meters wide (Figure B-11).

Lake blockage failures like those of 1991 and 1992 appear unlikely under current basin conditions. The lake area is mostly filled with sediment, reducing the water storage potential of the site. Nevertheless, another large secondary pyroclastic flow event in the upper basin could result in another lake blockage formation and possible failure. The risk of another large secondary pyroclastic flow cannot be confidently determined, but given the basin geometry, additional events seem likely in the next 5 to 10 years. The magnitude and frequency of secondary pyroclastic flows appear to be diminishing over time.

Hyperconcentrated flows and mudflows can be expected to dominate sediment transport from the upper basin to the upper alluvial-fan area and lower. Main channel processes will provide the sediment to generate these flow conditions. Bank collapses along the deep v-shaped channels will be a major source. Secondary pyroclastic flows may deposit material in the channel, increasing the supply of material available for future erosion. Drainage density may be near maximum state and sediment yields from the tributary channels should decline.

4.2.2 10-Year Sediment Forecast. The 10-year sediment forecast for the Pasig-Potrero basin is based upon examination of aerial photographs, maps, photogrametric cross sections, PHIVOLCS channel cross sections, and DPWH channel cross sections as well as interviews and reports of PHIVOLCS and DPWH staff and field investigations. As shown on Figure B-12, sediment yields are forecast to decline from 17 million m^3/year in the first year, to about 3 million m^3/year in five years. The declining sediment yield is linked to channel location within the basin and the "stable" channel configuration. The pyroclastic deposit will be a major sediment source for the next 20 years.

Sediment available for rapid erosion along the channel ranges from 27 to 33 million m^3. For the sediment yield forecast an average of 30 million m^3 was used. This forecast is for a relatively short time (10 years). The potential exists for a number of secondary pyroclastic flows to develop in the upper basin during this period and flow onto the middle and distal sections of the alluvial-fan complex in the lower valley. Sizing of these potential secondary pyroclastic flows is based on analysis of flows occurring in 1991 and early 1992, which suggests that secondary pyroclastic-flow sediment yield will be 17 million m^3 for the first three years. These potential events are not calculated into the 10-year sediment forecast, but due to their random nature are simply added on.

The forecast distribution of sediments over the 10-year period is shown in Table B-1. Hazard zones are outlined in Figure B-9. Nearly half of the deposition is forecast for the first five years. In-channel (including the area inside the 1993 levee between Mancatian and Santa Rita) deposition accounts for 27 million m^3, whereas deposition in the right overbank area is expected to be 7.5 million m^3. Mudflow and hyperconcentrated flows will deliver the bulk of the sediment to the lower alluvial-fan complex.

Analysis indicates that the right overbank inner sections have the highest probability of sediment deposition and flooding. Overbank sedimentation and flooding is expected to be frequent but shallow and widespread within these zones. Because sediment has already filled stream channels, even some leveed reaches, overbank areas can be flooded several times each year. Sediment deposition depths are anticipated to average less than 1 meter per event, but accumulations of up to 2 meters could occur. Mudflows could produce overbank deposits of up to 3 m over a limited area.

Overbank impacts could occur in the Potrero area (see Figure B-9 and Table B-1), but the depth should be less than 0.5 meters. In the Porac and Mancatian zones, out-of-channel deposition could be linked to in-channel sedimentation. Localized deposition could induce major levee breaches and increased overbank sediment and flood impacts. In-channel point deposition is not possible to predict.

The sediment yield curve shown in Figure B-12 represents a forecast of average annual conditions. Variations in sediment yields can be expected due to above- or below-average rainfall, the occurrence or lack of secondary pyroclastic flows, lake failures, or major changes in channel alignments. Therefore, sediment yields in any one year may be higher or lower than the forecast yields, but annual yields are expected to follow the forecast trend.

TABLE B-1 *Annual Pasig-Potrero River sediment deposition in million m^3*

YEAR & REACH IDENTIFICATION	FLOODING AND DEPOSITION ZONES				
	LEFT OVERBANK			RIGHT OVERBANK	
	OUTER	INNER	CHANNEL	INNER	OUTER
1993					
Pasig	-	0.5	4.0	0.5	-
Mancatian	-	0.5	4.5	0.5	-
Potrero	-	-	3.0	2.0	-
Bacolor	-	-	0.5	-	-
1994					
Pasig	-	0.5	2.0	0.5	-
Mancatian	-	0.5	1.5	0.5	-
Potrero	-	-	0.5	1.5	-
Bacolor	-	-	0.5	-	-
1995					
Pasig	-	-	2.0	-	-
Mancatian	-	0.5	1.5	0.5	-
Potrero	-	-	1.5	4.5	-
Bacolor	-	-	0.5	-	-
1996					
Pasig	-	-	0.0	-	-
Mancatian	-	-	1.0	-	-
Potrero	-	-	2.0	0.5	-
Bacolor	-	-	0.5	-	-
1997-1998					
Pasig	-	-	0.0	-	-
Mancatian	-	-	1.0	-	-
Potrero	-	-	1.0	0.5	-
Bacolor	-	-	0.5	-	-
1999-2000					
Pasig	-	-	0.0	-	-
Mancatian	-	-	0.5	-	-
Potrero	-	-	1.0	-	-
Bacolor	-	-	0.5	-	-
2001-2002					
Pasig	-	-	0.0	-	-
Mancatian	-	-	0.0	-	-
Potrero	-	-	0.5	-	-
Bacolor	-	-	0.5	-	-

4.2.3 Long-term Sediment Forecast. The long-term sediment forecast projects 50 years into the future, based on the major assumption that no further major volcanic action occurs. Over the next 50 years, erosion could total 77 million m^3. The analysis indicates that sediment yields initially will be high, but they are expected to return to the pre-eruption range (Figure B-12).

Analysis of pre-eruption aerial photographs (1974) shows a wide and highly braided channel. Sediment transport appears to have been high and the river may have been transport-limited rather than sediment-supply-limited. These active channel data and the JICA (1978) report provide supporting documentation that the Pasig-Potrero River basin had high sediment transport volumes and massive seasonal floods before 1991. Unstable channel conditions and bed aggradation will continue indefinitely.

Future sediment problems in the Pasig-Potrero River system will also depend on the size, location, and timing of secondary pyroclastic flows. These flows could create lakes on Yangca Creek that could be capable of producing sediment yields of 10 million m^3 in a single year. The timing of these events cannot be predicted and they will remain a risk for many years. Secondary pyroclastic flows could create smaller lakes in Bucbuc and Timbu Creeks, but they would probably be too small to produce sediment yields comparable to the Yangca Creek lake.

In the future, the Pasig-Potrero River is likely to undergo periods of erosion and deposition as water and sediment discharge rates change. Sediment transport calculations show the sediment transport potential decreasing in the downstream direction during storm events. Thus, when runoff from heavy rainfall on the pyroclastic deposits moves large amounts of sediment into the Pasig River, deposition is likely. However, when the inflowing sediment load is small, the sandy river bed is likely to erode. The long-term trend will be one of deposition and redevelopment of the historic alluvial fan. The stream may eventually come out-of-bank and migrate to other drainages unless controlled in its present location. Flooding will be a frequent occurrence until the stream channel is stabilized.

The future impact of lake failures (if lakes form) appears to depend on the relative amounts of water and sediment. Evidence suggests that the 1991 lake failure had more water than sediment in the upper reaches of the Papatac, resulting in scour throughout the Pasig River, while the 1992 lake failure may have had more sediment available in Papatac Creek than it could transport, producing deposition in the Pasig River. The potential impact of lake failures will thus change over time as lakes form and fail. If the Pasig River channel completely fills with sediment, the entire alluvial-fan area from Angeles City to Floridablanca could be threatened by potential lake-failure lahars.

Future sediment conditions along the Potrero River will be dominated by the occurrence or lack of lake failures in the headwaters. Rainfall/runoff processes, including monsoons and typhoons, are likely to produce less than 2 million m^3 per year of sediment inflow, resulting in sediment inflow similar to the pre-eruption conditions described by Japan International Cooperation Agency (JICA 1978). The river's ability to transport sediments through the reach was decreased when it perched in 1991 and local inflows were blocked. Lake failures have the potential to produce sediment yields of 10 to 20 million m^3 in a matter of hours. These events will almost certainly be depositional in this reach of the river. The location of deposition can range over the entire reach. As sediment accumulates in the channel, the potential will increase for the river to breach the levees and migrate across the alluvial fan.

Sediment from the Pasig-Potrero River system will slowly but steadily accumulate in delta channels. The severity of Delta flooding will depend on the extent to which deposition restricts the flow to Pampanga Bay. Lake failures may cause small surges of sediment to reach the delta channels, but this does not appear to be a serious problem for the future. Lake failures may cause additional deposition in the Bacolor and Guagua areas. However, raising the San Fernando-Olongapo Highway decreased that probability for Bacolor.

5. SACOBIA-BAMBAN RIVER BASIN ANALYSIS

The Sacobia basin is an area of 146 km^2 extending northeasterly from the upper flanks of Mount Pinatubo to the alluvial-fan complex of the Central Luzon Basin (see Figure B-1). The basin headwater area consists of narrow steep parallel valleys on the mountain's flank. The Bamban drainage includes three other tributaries, the Sapang-Cauayan, Marimla, and Malago Rivers, but the subject of this study is the Sacobia-Bamban River. Only the upper Sacobia River basin was filled with pyroclastic-flow deposit as a result of the 1991 eruption. Approximately 25 km downstream from the crater this stream becomes the Bamban River. Appendix A (Hydrology and Hydraulics) provides a detailed basin discussion.

The following basin analysis is limited to basin conditions prior to October 1993. A natural diversion of the Sacobia headwaters into the upper reaches of the Pasig-Potrero River basin occurred during October 1993. This reduction in drainage area will significantly reduce sediment yields in the Sacobia-Bamban River. However, the full impact of this basin change has not been analyzed and is not accounted for in this report.

5.1 Sedimentation History

5.1.1 Pyroclastic Deposits. When the initial eruption phase terminated on June 15, 1991, 602 million m^3 of pyroclastic-flow material was emplaced in the upper Sacobia River basin. Most of the deposit was in the deep river valley within 5 to 15 km from the caldera, at depths ranging up to 160 meters (see Figures B-13, B-14, B-15, and B-16). These deposits filled the pre-eruption valley with unstable and non-cohesive volcanic material. The upper basin was also covered with 5 to 50 cm of ash. During the initial emplacement, secondary pyroclastic flows were generated which aided sediment redistribution and channel redevelopment. The deposit's non-cohesive nature combined with high rainfall rapidly formed a new drainage network.

5.1.2 1991-1993 Erosion and Deposition. Erosion was very high during 1991, and declined during each of the next two years. Analysis of channel cross sections taken from aerial photographs show erosion of the channels in the pyroclastic deposit, from June 1991 to November 1992 totaling 138 million m^3. No measurement was made for the 1993 erosion volume. During the 1991 rainy season, an unknown volume of airfall ash was eroded from the mountain and valley slopes and deposited along the edges of the pyroclastic flow deposit and in downstream channels on the alluvial fan.

During June and July 1991, much of the Sacobia River discharge was diverted into the Abacan River at the Gates of the Abacan, about 3 km upstream of Clark Air Base. On July 25, 1991 (Pierson and Scott, USGS, personnel communication, July 1992), headward erosion on the main stem of the Sacobia recaptured the upper Sacobia River basin at the Gates of the Abacan blockage and stopped flow to the Abacan River. Subsequent

reduction of drainage area, discharge, and sediment supply in the Abacan River decreased impact in the new Abacan River area of the lower alluvial-fan complex.

Sediment data show that 68 million m^3 and 48 million m^3 of material was deposited in the Sacobia system during 1991 and 1992, accounting for a total of 116 million m^3 out of the 138 million m^3 of erosion in the upper Sacobia River basin during those years. Sedimentation data analysis using a mass balance method suggests that about 22 million m^3 of material flowed down the Abacan River system.

Mudflow and hyperconcentrated flow events delivered sediment to the upper and lower alluvial-fan complex during the 1991 rainy season. The Clark and Lake areas (see Figure B-17) were mainly impacted by thick mudflow events. Five to 10 m of channel aggradation occurred in those two reaches. Hyperconcentrated flow deposits covered a large area of the northern floodplain from Concepcion on downstream (see Figure B-17). The Concepcion area was flooded for several months as a result of levee breaches and sediment deposition in the channel. The volume of deposition in 1991 was calculated at 68 million m^3. Most of the sediment came from main channel erosion in the pyroclastic flow deposits. Headward erosion and bank line failures were the dominant processes. Phreatic explosions were a secondary process.

The 1992 rainy season produced further massive mudflows and hyperconcentrated flows, but geomorphic processes changed. Main channel erosion became secondary to large secondary pyroclastic flow events. During this period, four large secondary flow events occurred in the middle basin (see Figure B-18) which yielded high volumes of sediment to the alluvial-fan complex. Not all of the sediment generated by these events exited the deposit area. Sediment filled long reaches of the channel, priming the system for rapid erosion during storm events. During subsequent storms, especially on August 29-30 and September 3-4, 1992, sediment cascaded out of the middle basin and deposited in the Clark and Lake reach channels and Lake overbank area. Portions of Mabalacat and Bamban were heavily damaged in August and September 1992. Mudflows and hyperconcentrated flows were the transport mechanism in the middle basin, while muddy water flows dominated on the alluvial-fan complex. Only minor sediment and flood impacts occurred in areas downstream of the San Francisco reach (see Figure B-17) in 1992. The Concepcion area experienced some local flooding due to 1991 sediment deposition blocking drainage channels.

The 1992 channel deposits in the Lake reach caused the Sacobia River to be diverted onto the southern floodplain at Mabalacat during all of 1993. Even though sediment yields were much less than the previous years, the diverted flow caused frequent flooding and significant sediment deposition on the floodplain between Mabalacat and the Magalang-San Francisco highway in 1993. The levee along the north bank near Bamban was breached in August 1993, causing prolonged flooding, but little sediment deposition on the northern floodplain.

5.2 Sediment Forecast

5.2.1 Sediment Production Events. Main channel erosion is expected to dominate future sediment yields. Sediment loading from secondary pyroclastic flows and bank line phreatic explosions will continue to disturb channel development. Field observations indicates that secondary pyroclastic flow locations are less susceptible to fluvial erosion and additional movement. These areas appear more stable and are hence likely to show a reduction in sediment yield and/or erosion rates.

Channel development on the pyroclastic deposit is a complex process. Channel geometry and plain form are impacted by secondary pyroclastic flows and bank line phreatic explosions. Pre-eruption basin geology hard points may influence these processes, with links to sediment yield. Trapezoidal and V-shaped channel geometry dominates (Figures B-14, B-15, and B-16). Data indicate that sedimentation processes and yields will continue to fluctuate, with links to the geomorphic processes impacting the upper and middle basin.

Hyperconcentrated flows and mudflows can be expected to occur regularly until channel geometry and alignment are stabilized. The channel bottom will reach a stable width of about 300 meters in a few years. The impact of secondary flow events and phreatic explosions will be reduced over time. Sediment production or erosion will shift down-basin; in about six years, the Clark reach will become a sediment supply zone.

5.2.2 10-Year Forecast. The Sacobia-Bamban River pyroclastic-flow deposit sediment yields (see Figure B-19) are expected to continue the rapid decline observed during the past three years. The sediment yields are forecast to decline from 19 million m^3/year the first year to about 3 million m^3/year in five years. By the year 2001, sediment yields may decline to pre-eruption levels. The development of the "stable" channel condition on the pyroclastic-flow deposits is the controlling factor. Basin analysis shows that under current short-term conditions, large secondary pyroclastic flows are not likely to have a direct impact on sediment yields. Allowing for 10 meters of additional incision along with channel bed widening of up to 300 m, the main channel erosion is expected to be 72 million m^3 in the next 10 years.

The forecast distribution of sediment deposition over the next 10 years is shown on Table B-2. The hazard zones are outlined on Figure B-17. Nearly half of the deposition forecast for the Sacobia-Bamban River is expected to occur in the 12-km channel reach below the initial 1991 deposit. Mudflow deposition will dominate the Clark and Lake reaches, whereas hyperconcentrated flows and muddy water flows will dominate the lower reaches. The first five years will be dominated by mudflow events.

TABLE B-2 *Annual Sacobia-Bamban River sediment deposition in million m^3*

YEAR & REACH IDENTIFICATION	FLOODING AND DEPOSITION ZONES				
	LEFT OVERBANK			RIGHT OVERBANK	
	OUTER	INNER	CHANNEL	INNER	OUTER
1993					
Clark	n/a	n/a	3	0	n/a
Lakes	n/a	n/a	8	4	0
San Francisco	n/a	n/a	0	3	0
Concepcion	n/a	n/a	0	1	0
1994					
Clark	n/a	n/a	3	0	n/a
Lakes	n/a	n/a	2	4	0
San Francisco	n/a	n/a	0	4	1
Concepcion	n/a	n/a	0	1	0
1995					
Clark	n/a	n/a	3	0	n/a
Lakes	n/a	n/a	1	2	0
San Francisco	n/a	n/a	0	3	1
Concepcion	n/a	n/a	0	1	0
1996					
Clark	n/a	n/a	1	0	n/a
Lakes	n/a	n/a	0	2	0
San Francisco	n/a	n/a	0	3	1
Concepcion	n/a	n/a	0	1	0
1997					
Clark	n/a	n/a	0	0	n/a
Lakes	n/a	n/a	0	1	0
San Francisco	n/a	n/a	0	3	0
Concepcion	n/a	n/a	0	1	0
1998-2002					
Clark	n/a	n/a	-1	0	n/a
Lakes	n/a	n/a	0	1	0
San Francisco	n/a	n/a	0	2	0
Concepcion	n/a	n/a	0	1	0

Overbank flooding and sedimentation will be nearly continuous along the south side of the channel because of the 1992 diversion of the Sacobia River at Mabalacat. Overbank sedimentation and flooding are expected to occur mainly in the southern Lake and San Francisco areas, with flood waters depositing some sediment in the Concepcion reach. Flooding will be frequent and flood waters will meander across a large geographical area. Sediment deposition will be a continuous process with a steady, gradual accumulation of material in the overbanks. The depth of deposition will vary from several meters near Mabalacat to a few centimeters in the Concepcion zone.

The sediment yield curve shown on Figure B-19 represents a forecast of average annual conditions. Variations in sediment yields can be expected due to above- or below-average rainfall, the occurrence or lack of secondary pyroclastic flows, or major changes in channel alignments. Therefore, sediment yields in any one year may be higher or lower than the forecast yields, but annual yields are expected to follow the forecast trend.

5.2.3 Long-term Yield Forecast. Erosion over the next 50 years could total 112 million m^3. The 10-year yield forecast is based on no new major volcanic action. Erosion rates are expected to decline (see Figure B-19) as the upper basin channel bed widens; the wider the channel bed, the more the active channel can meander without eroding high banks of pyroclastic deposits. Less frequent contact between channel and bank line reduces sediment loading and sediment yields. Sediment concentrations will drop, and hyperconcentrated flow and mudflow concentration occurrences diminish. Sediment yield to the lower alluvial-fan complex will also diminish. Channel degradation in the 14-km reach below the pyroclastic flow deposit will increase sedimentation and flooding in the lower Sacobia-Bamban system. This long-term sediment hazard has been estimated to be in the 1 to 3 million m^3/year range and will probably continue for years.

Analysis of pre-eruption aerial photographs (1974) shows the Clark and Lake channel reaches to have been highly braided and laden with sediment, indicating that the system was unstable and subject to high sediment yield events before 1991. Future sediment transport in the Sacobia-Bamban River is expected to be similar to those pre-eruption conditions. The type of flooding and sedimentation problems will be different due to the channel infill that has already occurred. Overbank flooding will be a frequent occurrence for the foreseeable future. The southern overbank will be the most frequently flooded because of the 1992 diversion at Mabalacat. However, the unstable channel conditions increase the risk of levee failures and flooding along the north bank. Floods should deposit only small amounts of sediment to the northern floodplain.

6. O'DONNELL RIVER BASIN ANALYSIS

The O'Donnell River basin includes two major rivers, the O'Donnell and the Bulsa. The O'Donnell system drains the northern slopes of Mount Pinatubo, with steep and narrow highly incised parallel channels in the headwaters. The drainage area of the pyroclastic deposit is only 11 km^2. Upstream of the Bulsa confluence O'Donnell River's drainage area is 266 km^2. The Bulsa River drains about 510 km^2 from its headwaters on the upper northeastern slopes of the Zambales Range. The entire basin system covers about 817 km^2 (see Figure B-1).

6.1 Sedimentation History

6.1.1 Pyroclastic Deposits. The 1991 eruption of Mount Pinatubo deposited 241 million m^3 of pyroclastic flow material in the upper O'Donnell River basin. The material was deposited in a deep canyon between 2 and 9 km from the volcano's crater. The toe of the deposit is within an area of pyroclastic plateaus remaining from an ancient eruption. The pyroclastic material is predominantly sand, with lithic gravel and cobble found mainly within 6 km of the crater.

The deposits have maximum depths of 120 to 140 meters. The surface elevation of the deposit appears to have been controlled by the crest of the pre-eruption ridge dividing the O'Donnell and Bucao River basins. For much of the 1991 rainy season, the headwaters of the O'Donnell River within about 6 km from the crater were diverted into the Bucao River basin.

The initial pyroclastic surface was smooth and sloped down toward the east. This surface dip influenced the development of the drainage pattern, causing the main channel to move east, abandoning its pre-eruption alignment about 6.5 km from the crater. The new channel has incised into the pre-eruption ground in this area, just as it has incised into the 1991 material.

Aerial photographs show some phreatic explosion activity in 1991, but this does not seem to have been an important factor in drainage pattern development. Nor have secondary pyroclastic flows had much impact on the system. Only one secondary pyroclastic flow, near the base of the pyroclastic deposit, has been identified in the O'Donnell deposits (see Figure B-20).

6.1.2 1991-1993 Erosion and Deposition. Analysis of channel cross sections taken from aerial photographs show main channel erosion from June 1991 to November 1992 totaling 20 million m^3, while tributary channels and smaller drainages accounted for 6 and 9 million m^3 respectively. A review of the lahar deposits indicates that deposition and therefore erosion did not decline from 1991 to 1992, but were about equal in both years. Increased runoff may have offset the increasing stability of the established

channel, resulting in the nearly equal erosion. Two separate factors may have caused this increased runoff: 1) the recapture of the headwaters sometime around the end of August 1991, and 2) heavy rainfall. Analysis shows that much of the erosion in the upper 3 km of the pyroclastic channel occurred after August 10, 1992, a period corresponding with four weeks of steady rainfall recorded at Iba, Zambales (PAGASA, written communication, 1993).

Deposition in 1991 and 1992 was largely confined to the active channel area in the O'Donnell valley, upstream from the Bangat River (Swiss Disaster Relief, 1993). Downstream from the Bangat River, there may have been some deposition early in 1991, but PHIVOLCS river surveys showed the reach through Tarlac to have scoured in 1992. By the end of 1992, low-lying areas adjacent to the channel upstream of the Bangat River had been covered with less than 1 meter of sediment. Most of the 1992 lahar deposition occurred after August 10. As the channel between Santa Juliana and the Bangat River filled with sediment, flooding became a problem along the south side of the O'Donnell River in that reach.

Like the pre-eruption channel, the new main channel is very straight, with only one large meander. By November 1992, the main channel had incised 20 to 40 meters into both the 1991 pyroclastic material and the pre-eruption ground (see Figures B-21 and B-22). In the upper 4 km of the pyroclastic deposit, the 1992 channel was nearly 100 meters wide, with cobbles covering nearly half the channel. In the lower 4 km of the pyroclastic deposit, the river flows in a V-shaped channel with a narrow, sandy bed.

A review of photographs taken on August 31, 1993 shows that up to that time, 1993 erosion had been much less than the previous two years. In the upper reaches, within about 5 km of the crater rim, erosion was confined to a narrow, nearly straight channel that wandered within the broader 1992 active channel. The 1993 channel in that reach was estimated to be 20 to 30 meters wide and up to 5 meters deep. Farther downstream, the river continued to incise the bottom of the V-shaped channel to a maximum depth of around 5 meters.

The 1993 deposition followed the pattern of the previous two years, occurring mostly in the channels. Mudflow deposits appeared to be located upstream of Santa Juliana, within 10 km of the base of the pyroclastic deposit. Lahars continued to flow across southern overbank areas near the towns of O'Donnell and Santa Lucia. Signs of minor channel degradation were observed in the Bangat River at Santa Lucia in August 1993. In October 1993, flood waters eroded the southern abutment to the Bangat River Bridge at Santa Lucia. Downstream of the Bangat River, sedimentation continued to be minor and no significant flooding was observed.

6.2 Sediment Forecast

6.2.1 Sediment Producing Events. Main channel erosion is expected to dominate future sediment yields. Erosion may be especially high through the lower 4 km of the pyroclastic deposit channel. In this reach the V-shaped channel still has a narrow bed and high steep sides. Other pyroclastic deposit channels do not have sufficient drainage areas to be significant sediment sources.

Hyperconcentrated flows and mudflows can be expected to occur regularly until the deep V-shaped channel achieves a bottom width of over 100 meters. After that occurs, the frequency of these events should be reduced to about one or two per year.

Secondary pyroclastic flows are not expected to contribute significantly to sediment yields. The geometry of the pre-eruption watershed appears to limit the potential for secondary pyroclastic flows to the middle and west channels at the base of the pyroclastic deposit (see Figure B-20). The likelihood of secondary pyroclastic flows at these locations was not well defined, but was considered low. Should such a flow occur, it would have serious local impacts and could increase sediment yields for a short time afterward.

6.2.2 10-Year Forecast. The O'Donnell River pyroclastic deposit sediment yields (see Figure B-23) are forecast to decline from 13 million m^3/year to about 2 million m^3/year over a five year period. After that, sediment yields may continue to decline until yields approach the pre-eruption range. The projected sediment yields are controlled by the limited runoff from the pyroclastic deposit, as well geologic controls that tend to reduce erosion potential. The "stable" channel width ranges from 500 meters to as little as 125 meters. The wider reach lies just upstream of the current V-shaped channel, in an area where the main channel has been migrating to the east. The narrow widths are in a reach where the channel is incised into pre-eruption ground. Allowing for 10 meters of additional incision, main channel erosion is expected to be 27 million m^3 in the next 10 years.

The sediment yields shown in Figure B-23 represent a forecast of average annual conditions. Variations in sediment yields can be expected due to above- or below-average rainfall, the occurrence of unexpected secondary pyroclastic flows, or major changes in channel alignment. Therefore, sediment yields in any one year may be above or below the forecast yields, but on the average, annual yields should follow the forecast trend and total volumes.

The forecast distribution of sediment deposition over the next 10 years is shown in Table B-3. The hazard zones are outlined in Figure B-24. Nearly half of the deposition forecast in the O'Donnell River basin in the next 10 years is expected to occur in the 10-km reach just downstream of the pyroclastic deposit. Most of that deposition will be due to mudflows in the first five years. Overbank areas likely to receive flooding and sediment deposition are all upstream of the Bangat River. Overbank flooding is expected to be frequent but shallow, as floodwaters spread out across the floodplain in the O'Donnell-Santa Lucia area. Sediment deposition from most floods will be less than 0.5 meters, but accumulations could reach 1 to 2 meters in some locations.

Overbank areas in the Maniknik and Tarlac reaches (Figure B-24) may be subject to flooding, but sediment deposition should not significantly exceed pre-eruption rates. Flooding may be more severe or more frequent due to levee failures caused by channel aggradation and/or meandering.

6.2.3 Long-Term Yield Forecast. Erosion over the next 50 years could total 67 million m^3. Except for the first 10 years, the forecast erosion appears to be in the same range as the erosion that might have occurred even if the volcano had not erupted. Pre-eruption aerial photographs (1974) show a very active braided channel in the O'Donnell valley, suggesting high sediment yields from the upper basin. Unstable river conditions are likely to continue indefinitely. Flooding in the O'Donnell area could also remain a frequent problem because the channel infill will allow the river to meander across the overbanks.

How long the threat will exist of secondary pyroclastic flows at the base of the pyroclastic deposit is not currently known. Given the extreme local danger from such events and the limited knowledge about them, they should be considered a long-term hazard.

TABLE B-3 *Annual O'Donnell River sediment deposition in million m^3*

YEAR & REACH IDENTIFICATION	FLOODING AND DEPOSITION ZONES				
	LEFT OVERBANK			RIGHT OVERBANK	
	OUTER	INNER	CHANNEL	INNER	OUTER
1993					
Bombing Range	n/a	1	5	1	n/a
O'Donnell	n/a	0	4	1	0
Maniknik	n/a	0	1	n/a	0
Tarlac	0	0	0	0	0
1994					
Bombing Range	n/a	1	3	1	n/a
O'Donnell	n/a	0	2	1	0
Maniknik	n/a	0	1	n/a	0
Tarlac	0	0	0	0	0
1995					
Bombing Range	n/a	0	2	0	n/a
O'Donnell	n/a	0	2	0	0
Maniknik	n/a	0	1	n/a	0
Tarlac	0	0	0	0	0
1996					
Bombing Range	n/a	0	1	0	n/a
O'Donnell	n/a	0	2	0	0
Maniknik	n/a	0	0	n/a	0
Tarlac	0	0	0	0	0
1997-2002					
Bombing Range	n/a	0	1	0	n/a
O'Donnell	n/a	0	1	0	0
Maniknik	n/a	0	0	n/a	0
Tarlac	0	0	0	0	0

7. BUCAO RIVER BASIN ANALYSIS

The Bucao Basin incorporates the Bucao River and its two major tributaries, the Balin-Buquero River and the Balintawak River. The Balin-Buquero system includes four major tributaries, of which only the Maronut River, is officially named. The watershed follows a generally northwesterly direction from the volcano to the South China Sea.

The upper basins of the Bucao and Balin-Buquero rivers are a large fan complex of relatively flat and low-lying terrain, separated from the coastal alluvial-fan complex by the north-south trending westernmost mountains of the Zambales Mountain Range. This fan complex was heavily impacted by pyroclastic flows during the 1991 and earlier eruptions.

The Balintawak River system, which received no pyroclastic-flow deposits but was only blanketed by airfall ash during the 1991 eruption. The Balintawak River system was not considered a significant sediment source and was not analyzed.

7.1 Sedimentation History

7.1.1 Pyroclastic Flows. At termination of the eruptive phase on June 15, 1991, a total of 3,000 million m^3 of pyroclastic flow material had been deposited in the upper Bucao and Balin-Buquero basins. The bulk of the deposition occurred between 3 km and 10 km down-slope from the new crater. The pre-eruption drainage channels were filled, forming a broad and gentle western slope landscape. The upper basin was also covered with a blanket of airfall ash.

The headwaters of the Bucao River originate on the northwest slopes of Mount Pinatubo (see Figure B-25), at an elevation of about 900 meters. This region was covered with pyroclastic flow deposits ranging from only a few meters on the plateaus, to a maximum thickness of about 150 meters in the river valleys. The upper 6 km of the initial pyroclastic deposit was further modified by a 1992 secondary pyroclastic flow that originated along the divide with the O'Donnell River.

The Balin-Buquero drains the western upper flanks of Mount Pinatubo to the south of the Bucao River (Figure B-25). The Balin-Buquero basin and its tributaries were also covered with pyroclastic flow deposits ranging from only a few meters on the plateaus, to a maximum thickness of 150 meters in the river valleys. Deposition completely filled the Maronut River channel and diverted the river to the south, about 6 km northwest of the crater.

7.1.2 1991-1993 Erosion and Deposition. Eight major and numerous smaller tributaries transport sediment from the pyroclastic deposit to the main stems of the Bucao and Balin-Buquero rivers. A combination of rill and gully erosion, phreatic explosions, and secondary pyroclastic flows aided in the redevelopment of the drainage networks in the tributaries. Secondary pyroclastic flows have been a more important sediment producing process in the Bucao River basin than in the other basins. Cross-section analysis for the period of June 1991 to November 1992 shows that 600 million m^3 of material was eroded from the initial pyroclastic deposit. No estimate is available for the amount eroded during 1993. Fluvial erosion of airfall ash from surrounding mountain slopes appears to have been a main sediment source only during the 1991 rainy season.

A large secondary pyroclastic flow occurred on the Maronut River during the 1991 rainy season. Headscarp movement was initiated about 6 km northwest of the new crater, but the flow crossed the Maronut drainage divide and continued down one of the minor Bucao tributaries and into the Bucao River. A number of phreatic explosions also occurred during the 1991 rainy season.

Secondary pyroclastic flow and phreatic explosion processes continued during the 1992 rainy season. Three major secondary pyroclastic flows appear to have provided the bulk of the 1992 sediment yield. A massive secondary pyroclastic flow occurred along the Bucao/O'Donnell divide. The flow averaged 1,000 meters wide and flowed 6 to 7 km down the channel. A total volume of 50 to 100 million m^3 of sediment was delivered to the main stem of the Bucao River. A second major secondary pyroclastic flow occurred on a tributary of the Balin-Buquero River. Approximately 50 million m^3 of pyroclastic material from the base of the deposit (near mid-basin), cascaded about 6 km into the lower valley. Since the lower slope is nearly flat, most of this flow was deposited in the tributary valley, but some sediments did flow into the main stem of the Balin-Buquero River. The downstream distribution of sediment from this event is not known, but it is likely that some of the sediment reached the Bucao River. The third and smallest major event that occurred during the 1992 rainy season impacted the upper reach of the "new" Maronut River. The failure surface is about 1,000 meters wide by 2,000 meters long. Sediment from this event remained in the upper channel area and thus produced no sedimentation impacts on the coastal alluvial-fan complex.

Field observations during the 1993 rainy season disclosed no additional major modifications on the upper alluvial-fan complex (upper basin area). Fluvial channel erosion appeared to progress the same as in other basins.

Mudflows seem to be limited to the upper reaches of the Bucao and Balin-Buquero rivers. In the middle reaches of the Bucao and Balin-Buquero Rivers, transport processes consisted mainly of hyperconcentrated flow events. The sediment transport concentrations in the lower Bucao River are highly variable, depending on the relative

flow contributions from the sediment laden Bucao and Balin-Buquero rivers, and the cleaner Balintawak River.

Deposition has been very heavy in the upper reaches of the Bucao and Balin-Buquero rivers. Deposition has buried the town of Poonbato, located at the confluence of the Bucao and Balintawak rivers. Channel deposition ranges from around 20 m at Poonbato to about 5 m at the confluence of the Bucao and Balin-Buquero rivers at Malumboy. Deposition continues between this confluence and the South China Sea, with the active channel area or flood plain widening varying markedly from about 200 meters to 1,000 meters. Deposition thicknesses in the channel through the lower coastal fan complex are locally 2 to 3 meters. Distal thickness is less than 1 meter. Streamflow measurement data (Bureau of Research and Standards, unpublished) indicated about 1 m of deposition occurred at the Highway 7 bridge in 1991, with no additional accumulation in 1992. Field observations indicate that the transport mechanisms were muddy water to hyperconcentrated flow events. Flooding has not been a problem in the lower river.

7.2 Sediment Forecast

7.2.1 Sediment Producing Events. Rainfall/runoff is expected to continue to be the primary process delivering sediment to the lower reaches of the Bucao River. However, secondary pyroclastic flows are likely to be an important factor in sediment yields. There is a high risk of secondary pyroclastic flows or landslides in the zone extending from 6 to 12 km downstream from the crater area along the western flanks of the mountain. There is a low probability that the direct runout of these mass movements will reach the main stem of the Bucao River, but they will deposit additional sediment in the main channels that can then be eroded.

Normal geomorphic processes will continue to erode and transport material out of the upper basin. High sediment yields will continue for the next few years. Over time, channel processes and additional drainage development will reduce sediment yield from the upper basin. The V-shaped tributary channels are expected to widen to around 100 meters in the upper reaches and 200 meters near the base of the pyroclastic deposit. Frequent hyperconcentrated flows and mudflows are expected to continue to be generated in the pyroclastic deposit channels for the next 5 to 10 years. After that, unusually heavy rainfall may still generate these flows. Most mudflows will deposit upstream of the confluence of the Bucao and Balin-Buquero rivers.

Clean water entering the Bucao River from the Balintawak River increases transport capability in the lower 20 km, reducing the risk of mudflow hazards in the lower river reach. However, a gradual accumulation of sediment may occur in this reach.

7.2.2 10-Year Forecast. Sediment yields are forecast to decline at a rapid rate (see Figure B-31). Over the next 10 years a total of 101 million m^3 is forecast to be eroded from the pyroclastic deposit, compared to over 600 million m^3 in the first two years following the eruption.

The sediment yield curve shown in Figure B-31 represents a forecast of average annual conditions. Variations in sediment yields can be expected due to above- or below-average rainfall, the occurrence or lack of secondary pyroclastic flows, or major changes in channel alignments. Therefore, sediment yields in any one year may be higher or lower than the forecast yields, but annual yields are expected to follow the forecast trend.

The forecast distribution of sediment deposition over the next 10 years is shown on Table B-4. The hazard zones are outlined in Figure B-32. Nearly half of the deposition is forecast to occur in the headwaters reaches of the Bucao and Balin-Buquero rivers. Most of that deposition will be caused by mudflows during the first three years of the forecast.

An estimated 3 to 4 million m^3/year will continuously be transported into the lower 10 km of the river. Most of this material will be transported to the South China Sea, but about 1 m^3/year is expected to deposit in the San Juan reach. This gradual accumulation of sediment could over time, raise the river bed enough for floods to overtop the levee along the north side of the river. The rate and timing of this deposition is uncertain and should be monitored to improve the estimate.

7.2.3 Long Term Forecast. Erosion over the next 50 years could total 261 million m^3. After the first 10 years, the forecast erosion appears to be similar to what might have occurred even if the volcano had not erupted. The accumulation of sediment in the San Juan reach could continue for many years. However, the river's behavior in this reach is not well understood and additional data should be collected and analyzed to improve this estimate.

How long the threat of secondary pyroclastic flows at the base of the pyroclastic deposit will exist is not currently known. Each occurrence seems to increase the stability of the pyroclastic deposit. However, this process is not well enough understood to justify concluding that if secondary pyroclastic flows do occur, there would be no subsequent risk. Given the extreme local danger from such events, they should be considered a long-term hazard.

TABLE B-4 *Annual Bucao River sediment deposition in million m³*

YEAR & REACH IDENTIFICATION	FLOODING AND DEPOSITION ZONES				
	LEFT OVERBANK			RIGHT OVERBANK	
	OUTER	INNER	CHANNEL	INNER	OUTER
1993					
Headwaters	-	-	25	-	-
Malumboy	-	-	10	-	-
San Juan	-	0	2	-	-
Botolan	-	-	0	0	0
1994					
Headwaters	-	-	10	-	-
Malumboy	-	-	5	-	-
San Juan	-	0	2	-	-
Botolan	-	-	0	0	0
1995					
Headwaters	-	-	5	-	-
Malumboy	-	-	2	-	-
San Juan	-	0	1	-	-
Botolan	-	-	0	0	0
1996					
Headwaters	-	-	0	-	-
Malumboy	-	-	1	-	-
San Juan	-	0	1	-	-
Botolan	-	-	0	0	0
1997-2002					
Headwaters	-	-	0	-	-
Malumboy	-	-	1	-	-
San Juan	-	0	0	-	-
Botolan	-	-	0	0	0

NOTE: Three million m³/year is expected to be discharged to the South China Sea.

8. SANTO TOMAS RIVER BASIN ANALYSIS

The Santo Tomas River basin watershed covers an area of about 262 km^2, draining the southwestern section of the mountain to the South China Sea (see Figure B-33). The Santo Tomas River system incorporates the Marella and Mapanuepe rivers, which join to become the Santo Tomas River. The reach length from the caldera to the confluence of the Marella and Mapanuepe Rivers is about 28 km. The Santo Tomas River begins at this confluence and flows about 23 km to the sea.

8.1 Sedimentation History

8.1.1 Pyroclastic Deposits. The June 1991 eruption left a total of 1,400 million m^3 of pyroclastic material in the Santo Tomas-Marella River basin in two separate deposits. One fills a broad valley between 2 and 7 km south-southwest of the crater. The second deposit lies about 1 to 2 km farther west (see Figure B-33). The two deposits converge about 8 km from the crater rim.

The eastern valley varies in width from about 500 to 2,000 meters and has a slope of nearly 10 percent. The deposits range from 10 to 20 meters deep on the flatter upper terraces of the valley, to over 100 meters deep in the old stream channels (see Figures B-34 and B-35). About 7 km^2 of the upper watershed was diverted into a pre-eruption channel that flows along the southern boundary of the eastern pyroclastic deposit. A moderate-sized secondary pyroclastic flow occurred at the diversion point in 1991 (see Figure B-33). Because of the flow diversion, there are no large stream channels crossing the eastern pyroclastic deposit.

The western deposit ranges from 400 to 800 meters wide and up to 120 meters deep, with a main channel that drains the upper west side of the watershed. Near the base of the pyroclastic deposit the channel has been diverted to the east, around the pre-eruption hill shown on Figure B-33. A large secondary pyroclastic flow occurred in the abandoned river valley in early August 1992, depositing material in the main channels for several kilometers downstream.

8.1.2 1991-1993 Erosion and Deposition. Sediment deposition during 1991 totaled approximately 210 million m^3. Sediment produced by rill, gully, and channel erosion on the pyroclastic surface was aggravated by the secondary pyroclastic flow in the eastern deposit, and by erosion of ash deposits on the steep mountains adjacent to the Marella River.

Most of the deposition occurred in the Marella River, with depths of approximately 30 meters near the base of the pyroclastic deposit and 20 meters at the confluence with the Mapanuepe River. Deposit depths continued to decline downstream in the Santo

Tomas River to about 5 meters near the barangay of San Rafael and 1 meter at the Highway 7 Bridge.

A large lake formed on the Mapanuepe River because of the blockage at its confluence with the Marella River. The lake flooded over 4 km^2 and several communities in the Mapanuepe valley. The Mapanuepe River periodically overtopped the blockage, only to be dammed again by deposits from the Marella River. ZLSMG staff (Rodolfo and Umbal, personal communication, 1993) reported frequent discharges in the hyperconcentrated and mudflow range from both the pyroclastic deposit and Mapanuepe River. Overbank flooding occurred in the lower 7 km of the Santo Tomas River, impacting the towns of San Narciso and San Felipe.

Sediment yields in 1992 again totaled near 200 million m^3. Sediment yields on the Marella River, like those on the O'Donnell, did not follow the declining yield pattern described by Pierson et al. (1992) for other volcanoes. The higher than expected yields were most likely the result of the combination of a secondary pyroclastic flow and heavy rainfall. The drainage channels continued to evolve in a pattern similar to other channels around Mount Pinatubo, with the western channel forming a V-shaped valley incised about 40 meters into the pyroclastic deposit. However, a large secondary pyroclastic flow near the base of the deposit clogged both the east and west channels shortly before the onset of heavy rains in August 1992. Rainfall records from the Dizon Mines (written communication, 1993), which are located 6 km southeast of the Marella/Mapanuepe confluence, show that August 1992 had the third highest monthly rainfall in the last 10 years. It appears that the heavy rains rapidly eroded the secondary pyroclastic flow deposits from the main channels and contributed significantly to the unusually high sediment yield.

An additional 5 to 10 meters of aggradation occurred in the Marella River in 1992. The pre-eruption ridge at the Marella/Mapanuepe confluence was breached on the west side near the base of Mount Bagang. By November 1992, mudflows had traveled as far downstream as San Rafael where they deposited an additional 3 to 5 meters in the channel. However, no significant deposition occurred in the lower 10 km of the Santo Tomas River. The flooding of Dizon Mines by Mapanuepe Lake led to the construction of a new lake outlet on the southern side of the Santo Tomas River. Levees built by the Department of Public Works and Highways before the 1992 rainy season contained river flows and prevented flooding along the lower 20 km of the Santo Tomas River.

The pyroclastic deposit eroded much less in 1993 than in the previous two years. A rough estimate based on photographs places the total 1993 erosion at less than 60 million m^3. Sediment contributions from various source areas on the pyroclastic deposit could not be defined because weather conditions limited access to the area. However, it was clear that re-erosion of earlier deposits in the Marella River had contributed 5 to 10 million m^3 of sediment to the pyroclastic yield.

Sediment erosion and deposition in 1993 were dominated by two large typhoons, Rubing in August and Kadiang in October. Prior to August, deposition had been minor in the Marella River, but it had nearly filled the Santo Tomas River channel in the vicinity of San Rafael. Runoff from typhoon Rubing caused a large mudflow that traveled as far as San Rafael, intersecting and overtopping a recently completed 3-meter-high levee just south of the barangay. The mudflow deposit reached depths of around 4 meters near the levee. Discharges from Mapanuepe Lake, flowing along the south side of the Santo Tomas River channel, were diverted over the levee by the mudflow deposit. The Mapanuepe discharges quickly eroded the levee and overflowed toward San Marcelino and San Antonio. Flooding continued for nearly a week, until the levees were repaired. Most overbank damages were the result of flooding, with sediment deposition generally limited to within 1 km of the levee.

During the late stages of typhoon Rubing, a new channel 2 to 5 meters deep and 100 to 150 meters wide was eroded into the deposits of the Santo Tomas and Marella Rivers from San Rafael upstream to the pyroclastic deposit. The channel was aligned along the west side of the Marella River and to the center and north side of the Santo Tomas. This channel was much larger than any other channel observed in this basin since the eruption.

Typhoon Kadiang produced 326 mm of rain at Dizon Mines on October 5, 1993 (Dizon Mines, written communication, 1993). This was 133 mm more than the next highest daily rainfall since the 1991 eruption, and the second highest daily total in 17 years at Dizon Mines. Sediment yields from the pyroclastic deposit were large during Kadiang, but perhaps not as large as might have been expected from a storm of this magnitude. Channel erosion continued in the Marella River channel downstream of the pyroclastic deposit, with an estimated 5 meters of additional bed erosion. Mudflows, possibly mixed with hyperconcentrated flows, followed the active channel of the Santo Tomas River for about 2 km before crossing the north overbank to Santa Fe, burying the town in 1 to 2 meters of sediment. Deposition in the river bottom near Santa Fe reached depths of up to 8 meters. Along the south side of the Santo Tomas channel, mudflows from the Marella River combined with Mapanuepe Lake discharges to breach the levee at the upstream end, again near San Rafael, and toward the downstream end near San Marcelino (ZLSMG, 1993). Flooding again lasted several days before emergency repairs closed the levee breaches.

8.2 Sediment Forecast

8.2.1 Sediment Producing Events. Rainfall/runoff is expected to continue to be the primary process delivering sediment to the Santo Tomas River. However, secondary pyroclastic flows are likely to be an important sediment source. The currently V-shaped western channel is expected to widen to approximately 100 meters in the upper reaches and 300 meters near the base of the pyroclastic deposit. The eastern branch cuts across

the toe of the pyroclastic deposit at the site of the 1992 secondary pyroclastic flows. It is in this vicinity that additional secondary pyroclastic flows are most probable.

Frequent hyperconcentrated flows and mudflows are expected to continue to be generated in the pyroclastic deposit channels for the next 5 to 10 years. After that, unusually heavy rainfall may still generate these flows. As long as the large channel remains on the Marella River, mudflows will tend to be transported to the Santo Tomas River without as much dispersion or attenuation as before August 1993. Lower concentration flows will likely erode through the Marella River.

8.2.2 10-Year Forecast. Sediment yields are forecast to decline at a rapid rate (see Figure B-36), continuing the trend started in 1993. Over the next 10 years a total of 130 million m^3 is forecast to be eroded from the pyroclastic deposit, compared to 212, 200, and 60 million m^3 in 1991, 1992, and 1993 respectively. Because of the high probability of secondary pyroclastic flows, an additional 15 million m^3 of sediment has been included in each of the first three years of the forecast. Once the sediment yields from the pyroclastic deposit have declined sufficiently, the Marella River channel is expected to begin to erode.

The sediment yield curve shown in Figure B-36 represents a forecast of average annual conditions. Variations in sediment yields can be expected due to above- or below-average rainfall, the occurrence or lack of secondary pyroclastic flows, or major changes in channel alignments. Therefore, sediment yields in any one year may be higher or lower than the forecast yields, but annual yields are expected to follow the forecast trend.

The forecast distribution of sediment deposition over the next 10 years is shown on Table B-5. The hazard zones are outlined in Figure B-37. Nearly half of the deposition is forecast to occur in the Santo Tomas River channel between the Mapanuepe River and a point about 2 km downstream of San Rafael. Most of that deposition will be caused by mudflows during the first three years of the forecast. The southern floodplain will receive most of the non-channel deposition, because the orientation of the Marella River will tend to build a fan toward the southwest. Sediment deposition is most likely near the Santo Tomas River and upstream of San Marcelino, but flooding may spread over most of the southern floodplain. Flooding could become more frequent as sediment deposition causes the river to meander across the broad channel and onto the overbanks.

Some sediment, in quantities smaller than can be analyzed in this study, will continuously be transported into the lower 10 km of the river. Much of this material will be transported to the South China Sea, but an unknown amount will deposit. Monitoring of conditions at the Highway 7 bridge would provide a better understanding of sedimentation in this lower reach.

TABLE B-5 *Annual Santo Tomas River sediment deposition in million m^3*

YEAR & REACH IDENTIFICATION	FLOODING AND DEPOSITION ZONES				
	LEFT OVERBANK			RIGHT OVERBANK	
	OUTER	INNER	CHANNEL	INNER	OUTER
1993					
Upper Mapanuepe	n/a	5	10	n/a	n/a
Mapanuepe	4	n/a	20	n/a	n/a
San Marcelino	1	3	1	n/a	n/a
San Falipe	0	0	1	0	0
1994					
Upper Mapanuepe	n/a	1	5	n/a	n/a
Mapanuepe	4	n/a	15	n/a	n/a
San Marcelino	1	3	1	n/a	n/a
San Falipe	0	0	0	0	0
1995					
Upper Mapanuepe	n/a	1	5	n/a	n/a
Mapanuepe	4	n/a	10	n/a	n/a
San Marcelino	1	3	1	n/a	n/a
San Falipe	0	0	0	0	0
1996					
Upper Mapanuepe	n/a	0	0	n/a	n/a
Mapanuepe	0	n/a	4	n/a	n/a
San Marcelino	0	1	1	n/a	n/a
San Falipe	0	0	0	0	0
1997					
Upper Mapanuepe	n/a	0	0	n/a	n/a
Mapanuepe	0	n/a	2	n/a	n/a
San Marcelino	0	1	1	n/a	n/a
San Falipe	0	0	0	0	0
1998					
Upper Mapanuepe	n/a	0	-2	n/a	n/a
Mapanuepe	1	n/a	2	n/a	n/a
San Marcelino	0	1	1	n/a	n/a
San Falipe	0	0	0	0	0

TABLE B-5 (Continued) *Annual Santo Tomas River sediment deposition in million m^3*

1999					
Upper Mapanuepe	n/a	0	-2	n/a	n/a
Mapanuepe	1	n/a	2	n/a	n/a
San Marcelino	0	1	1	n/a	n/a
San Falipe	0	0	0	0	0
2000					
Upper Mapanuepe	n/a	0	-2	n/a	n/a
Mapanuepe	1	n/a	2	n/a	n/a
San Marcelino	0	1	1	n/a	n/a
San Falipe	0	0	0	0	0
2001					
Upper Mapanuepe	n/a	0	-3	n/a	n/a
Mapanuepe	1	n/a	2	n/a	n/a
San Marcelino	0	1	1	n/a	n/a
San Falipe	0	0	0	0	0
2002					
Upper Mapanuepe	n/a	0	-3	n/a	n/a
Mapanuepe	1	n/a	2	n/a	n/a
San Marcelino	0	1	1	n/a	n/a
San Falipe	0	0	0	0	0

NOTE: An unknown amount of sediment will be discharged to the South China Sea each year.

8.2.3 Long Term Forecast. Erosion over the next 50 years could total 160 million m^3. After the first 10 years, the forecast erosion appears to be similar to what might have occurred even if the volcano had not erupted. Ten years ago, sediment yields from the Marella basin were high enough to annually fill the irrigation reservoir near the Mapanuepe River (Alejandrino et al., personal communication, 1993). Unstable river conditions are likely to continue indefinitely. Flooding and overbank sedimentation is likely to be more severe that prior to the eruption because the river channels have been filled by sediment. In the future, the river will be able to meander outside of the former channel boundaries and routinely cause flooding unless actions are taken to contain the river.

How long the threat of secondary pyroclastic flows at the base of the pyroclastic deposit will exist is not currently known. Each occurrence seems to increase the stability of the pyroclastic deposit. However, this process is not well enough understood to justify concluding that if the forecast three secondary pyroclastic flows do occur, there would be no subsequent risk. Given the extreme local danger from such events, they should be considered a long-term hazard.

9. COMPARISON OF SEDIMENT YIELD FORECASTS

PHIVOLCS and the USGS have collaborated on analyses of the Mount Pinatubo eruption and subsequent lahar problems. As part of that analyses they have calculated the pyroclastic deposit volumes and made total sediment yield forecasts for each basin (Pierson et al., 1992; PHIVOLCS, 1993). Table B-6 presents the pyroclastic deposit and forecast volumes prepared by PHIVOLCS/USGS and those presented in this report.

The PHIVOLCS/USGS sediment forecast is based on fixed percentages of material eroding from each of the pyroclastic deposits. A value of 40 percent was used for the Pasig-Potrero, Sacobia-Bamban, and O'Donnell rivers, and 50 percent was used for the Bucao and Santo Tomas rivers. The higher value was used for the west side rivers to account for the higher annual rainfall on that side of Mount Pinatubo. An additional 10 percent was used in all basins to account for erosion of pre-eruption material. The range of sediment yields resulted from applying the percentages to the range of pyroclastic deposits calculated by PHIVOLCS/USGS.

In this report, a total of 1.9 billion m^3, out of the 5.6 billion m^3 of total pyroclastic flow material deposited on the flanks of Mount Pinatubo during the June 1991 eruption, was forecast to be eroded within 50 years after the eruption (Table B-6). This erosion volume represents 34 percent of the total initial pyroclastic flow deposit volume and includes the erosion amounts which occurred during the period from June 1991 to November 1992. In the individual basins, the forecast erosion to initial deposit ratios are; Pasig-Potrero, 33 percent; Sacobia-Bamban, 41 percent; O'Donnell, 42 percent; Bucao, 29 percent; and Santo Tomas, 41 percent. The variations in yield ratios are due to variations in the complexity of the geomorphic processes occurring within each basin, as described earlier in this report.

TABLE B-6 *USACE and PHIVOLCS/USGS sediment volumes in million m^3*
(Note: This includes 1991-1992 erosion volumes)

	PYROCLASTIC DEPOSIT	TOTAL YIELD
PASIG-POTRERO RIVER		
USACE	302	100
PHIVOLCS/USGS		
Low	300	120
High	500	200
SACOBIA-BAMBAN RIVER		
USACE	602	250
PHIVOLCS/USGS		
Low	700	280
High	1,100	440
O'DONNELL RIVER		
USACE	241	102
PHIVOLCS/USGS		
Low	300	180
High	600	360
BUCAO RIVER		
USACE	3,000	863
PHIVOLCS/USGS		
Low	2,500	1,250
High	3,100	1,550
SANTO TOMAS RIVER		
USACE	1,400	572
PHIVOLCS/USGS		
Low	1,000	500
High	1,300	650
TOTAL OF ALL BASINS		
USACE	5,900	1,887
PHIVOLCS/USGS		
Low	4,800	2,330
High	6,600	3,200

10. CONCLUSIONS

Sediment yields have declined dramatically at Mount Pinatubo, but will remain a dangerous threat for several more years. Mudflows can be expected to occur several times a year on all five rivers for the next 5 to 10 years. After that time, annual sediment yields to the fan areas will drop to near pre-eruption levels. However, mudflows may still be occur on an infrequent bases. The Pasig-Potrero, Sacobia-Bamban, O'Donnell, and Santo Tomas channels on the alluvial fans are already filled with sediment and future lahars can spill onto the populated floodplains.

Sediment yields may be highly variable over both short and long time periods. Secondary pyroclastic flows or other basin disturbances could cause immediate, large surges of sediment. The occurrence or lack of unusually large storms will also cause variations in sediment yields. Monitoring of the pyroclastic deposits and the river channels needs to continue in order to evaluate changing conditions.

In October 1993, heavy rainfall and rapid erosion caused about 21 km^2 of the Sacobia River basin to be diverted into the Pasig River basin. This large increase in Pasig River drainage area is very likely to cause a tremendous increase in sediment yield in 1994 and beyond. The full impact of this basin change has not been evaluated for this report, but it is judged to present an extreme hazard to communities along the Pasig-Potrero River and also to endanger surrounding areas. Sediment yields and lahars in the Pasig River in 1994, are expected to be similar to those experienced in the Sacobia River in 1991 or 1992. Pasig-Potrero River sediment deposition of 50 to 100 million m^3 is considered possible in 1994. A complete analysis of this situation is needed before the 1994 rainy season.

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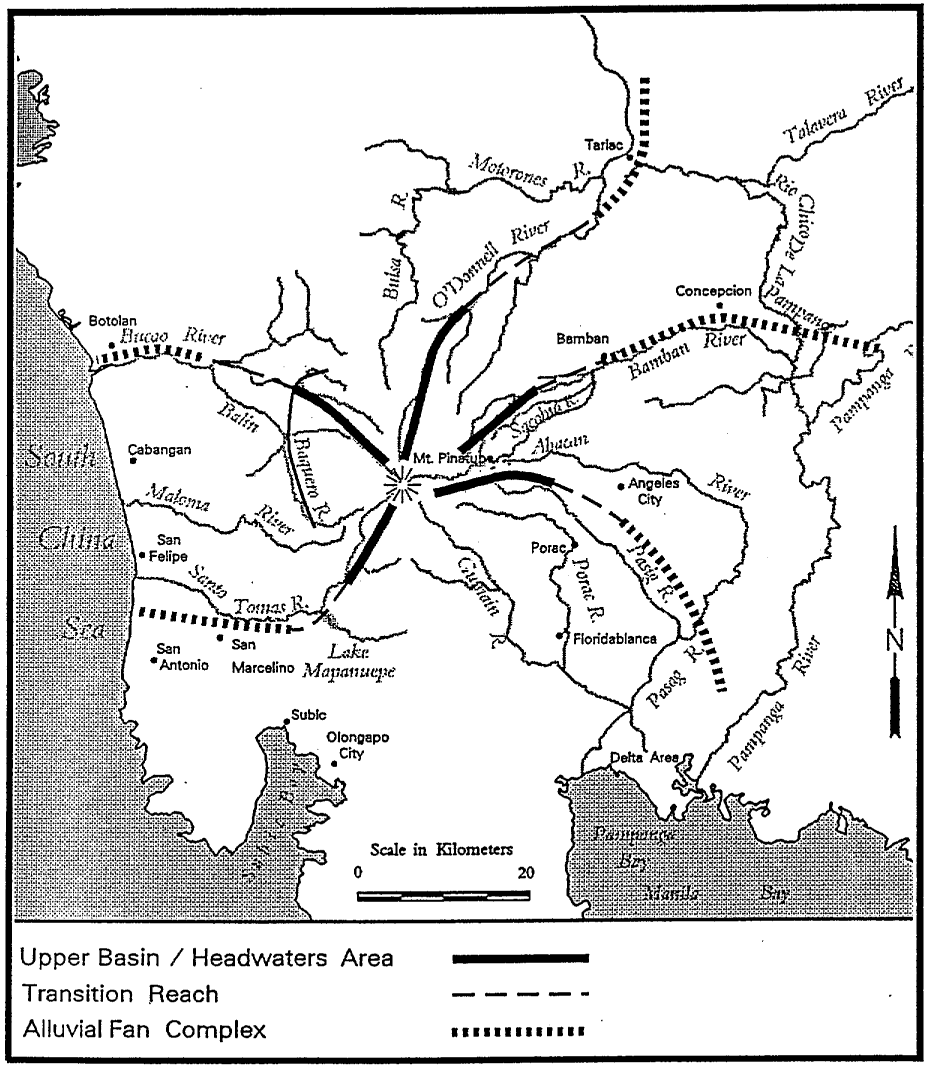

Figure B-1. General Location Map, Mount Pinatubo.
Illustrating the Three Geomorphological Zone in Each of the Major Basins, Upper Basin/Headwaters, Transition Reach, and the Alluvial Fan Complex.

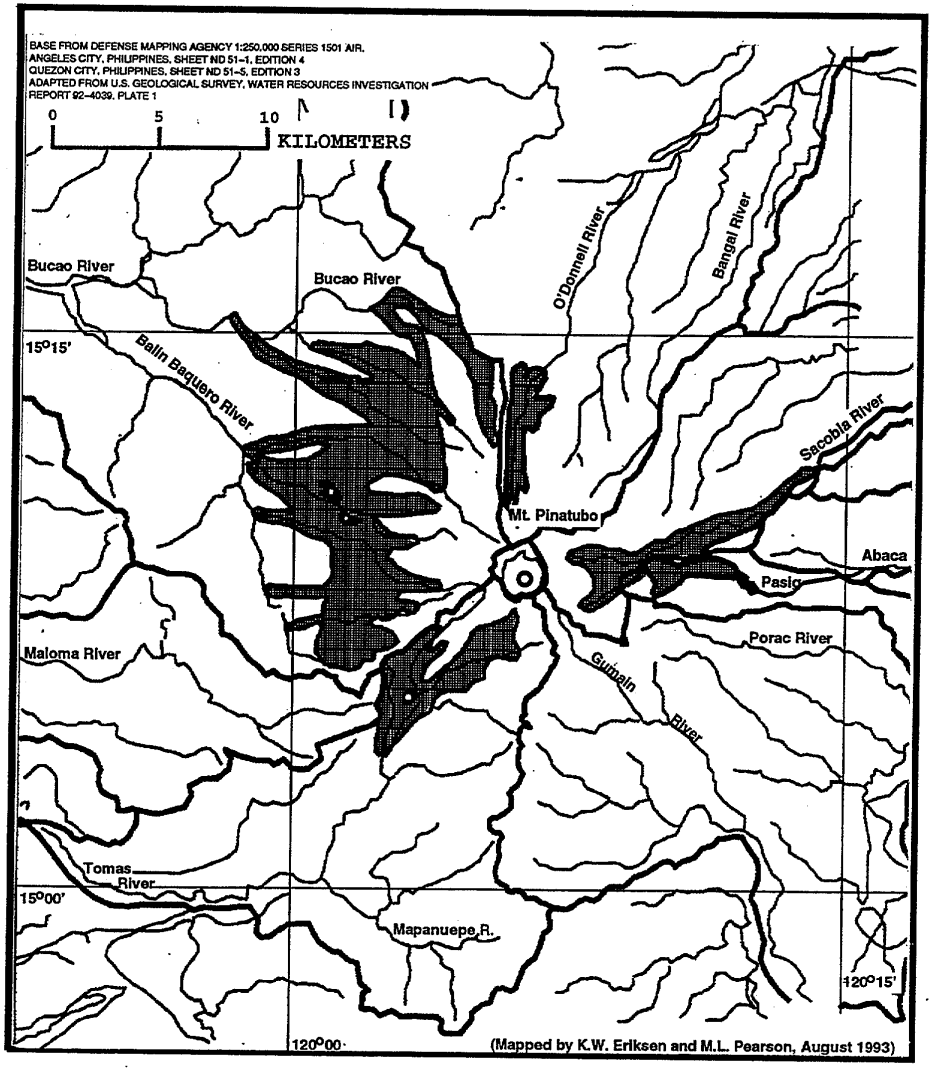

Figure B-2. Pyroclastic Flow Deposit Location Map, Mount Pinatubo. Illustrates Pyroclastic Flow Location in the upper Basin/Headwaters of the five basins.

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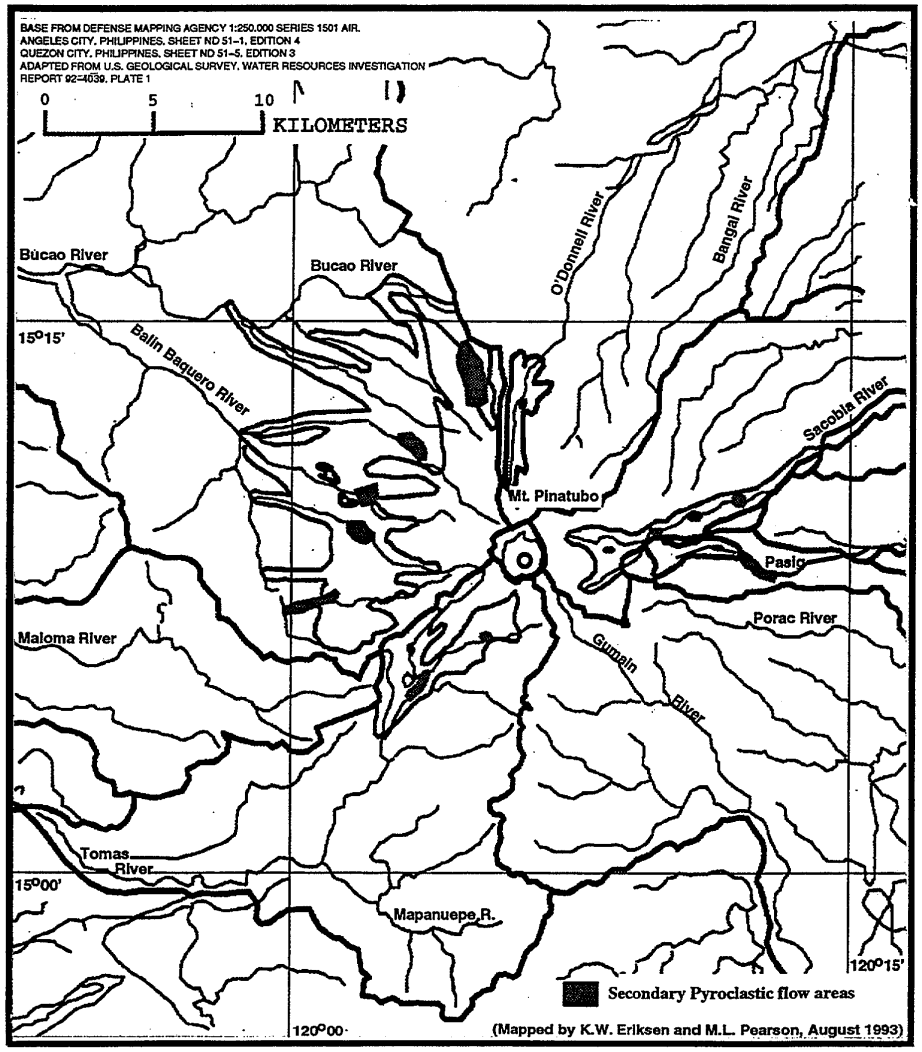

Figure B-3. Secondary Pyroclastic Flow Deposit Location Map, Mount Pinatubo. Illustrates major Secondary Pyroclastic Flow Areas which developed between June 1991 and November 1993.

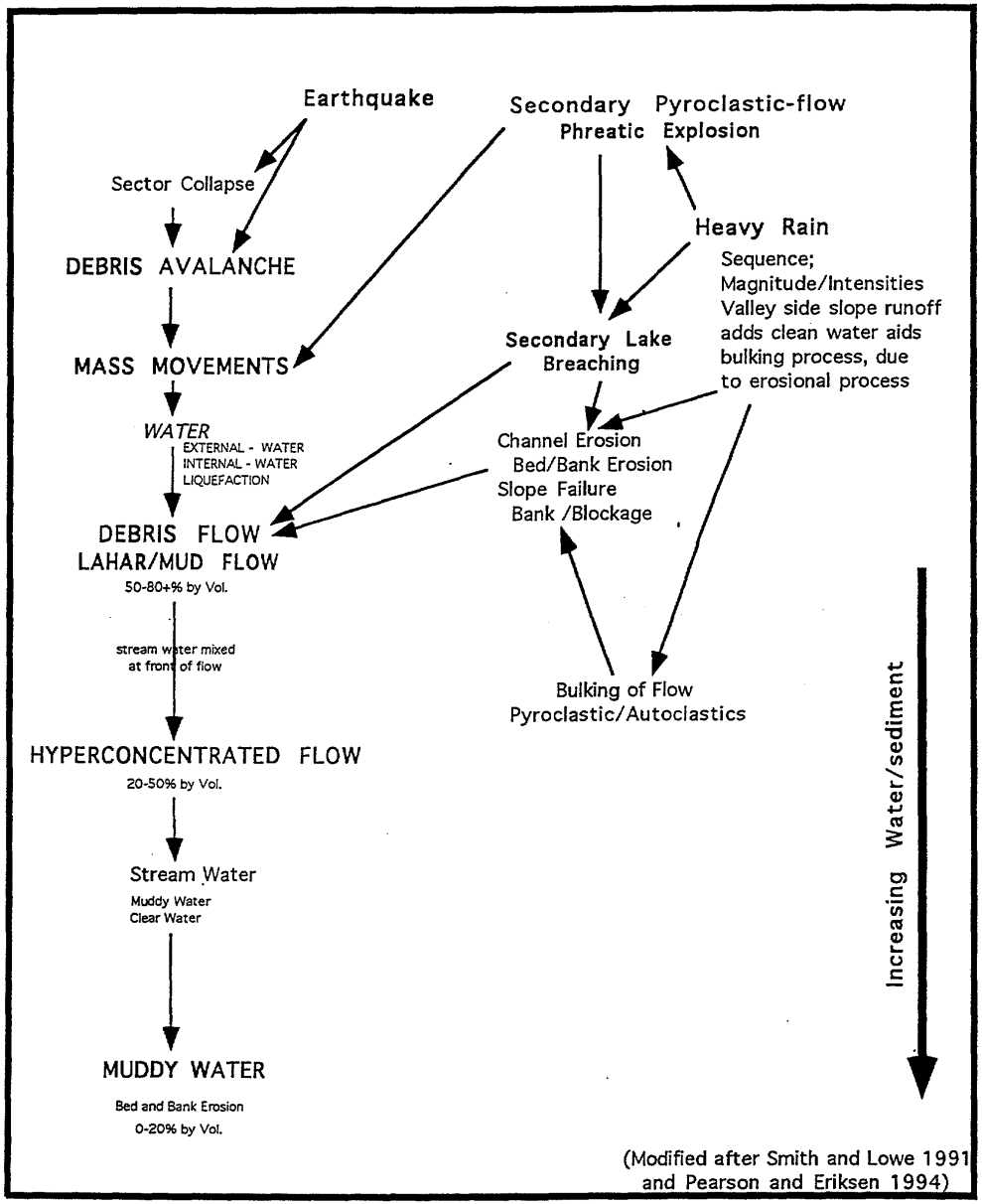

Figure B-4 Schematic of Sediment Production Proceeses.
Illustration of principe processes related to sediment transport and deposition of volcanic sediment. Type of flow phenomena and water to sediment relationships are also indicated.

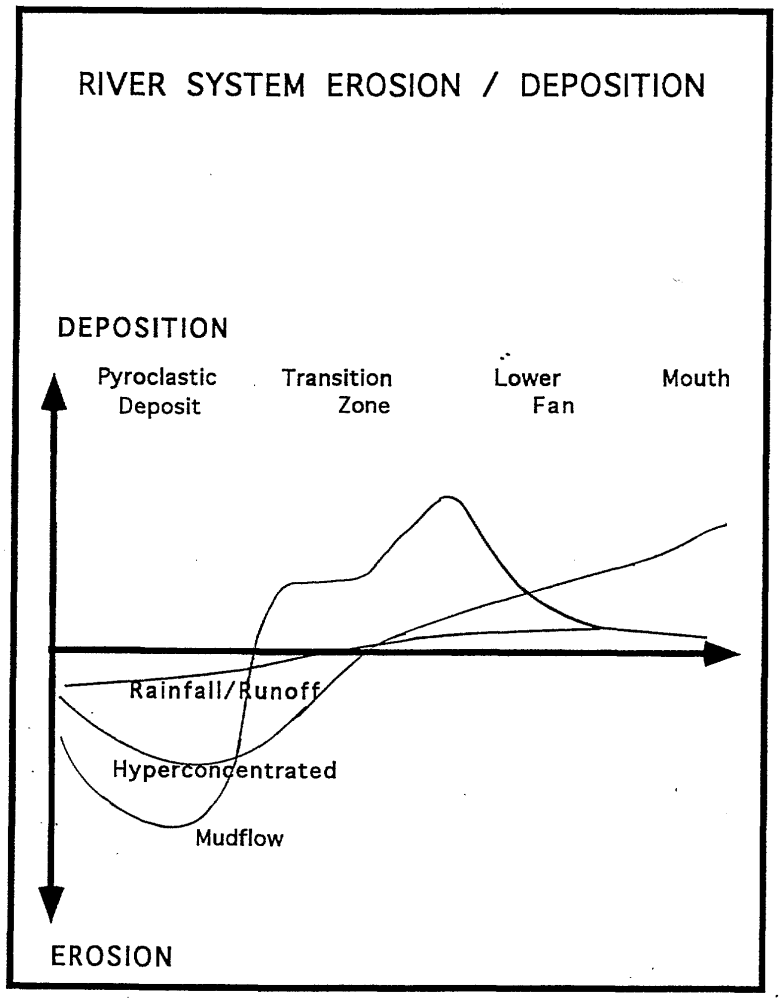

Figure B-5. Schematic Diagram of Relative Magnitude of Erosion and Deposition Through an Impacted Basin.

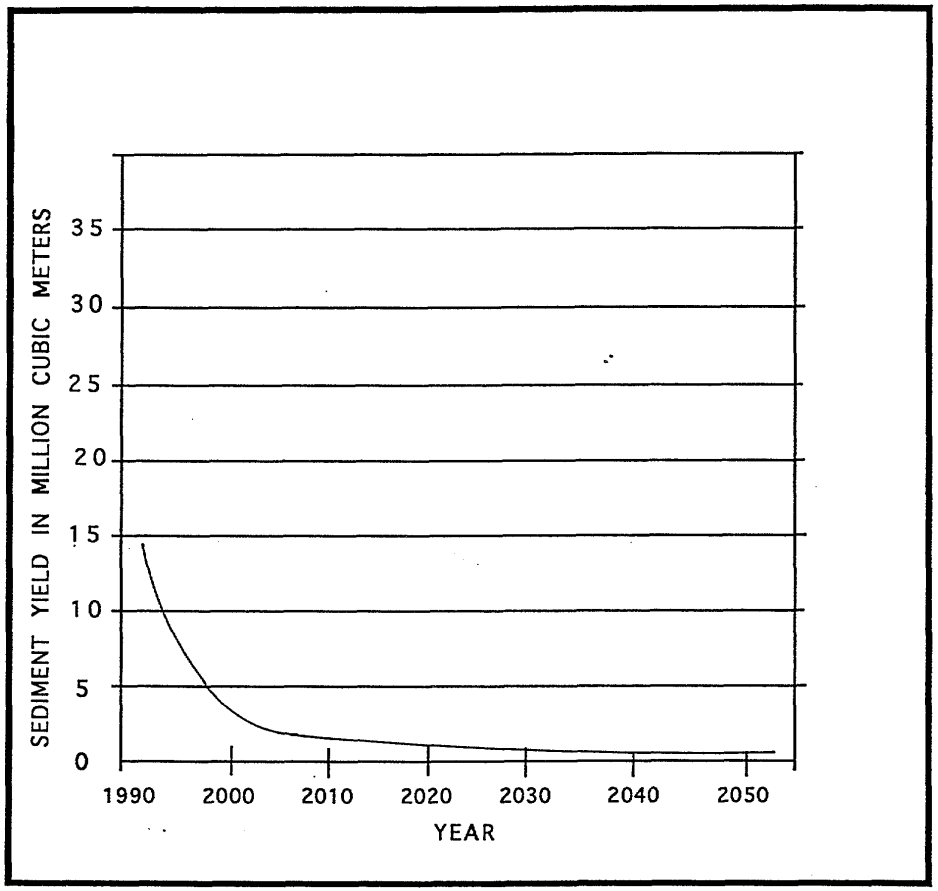

Figure B-6. Idealized Plot of Average Annual Sediment Yield, Mount Pinatubo, The Philippines.

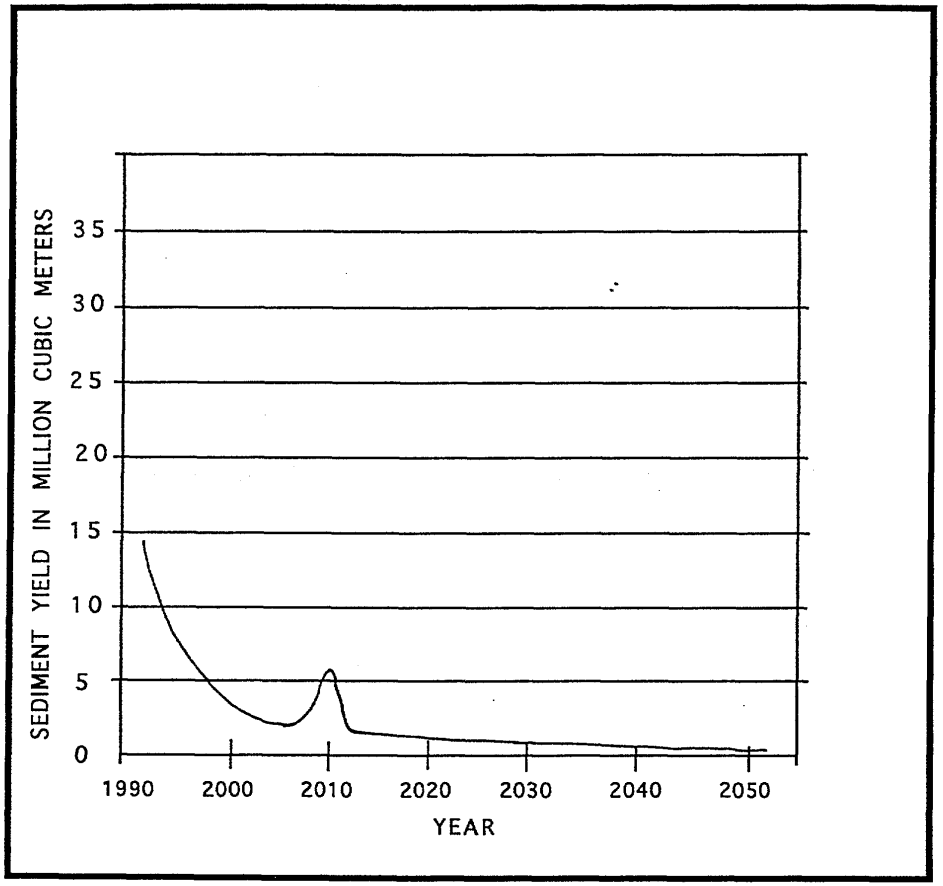

Figure B-7. Potential Impact of Large Storm Events on Average Annual Sediment Yield, Mount Pinatubo, The Philippines.

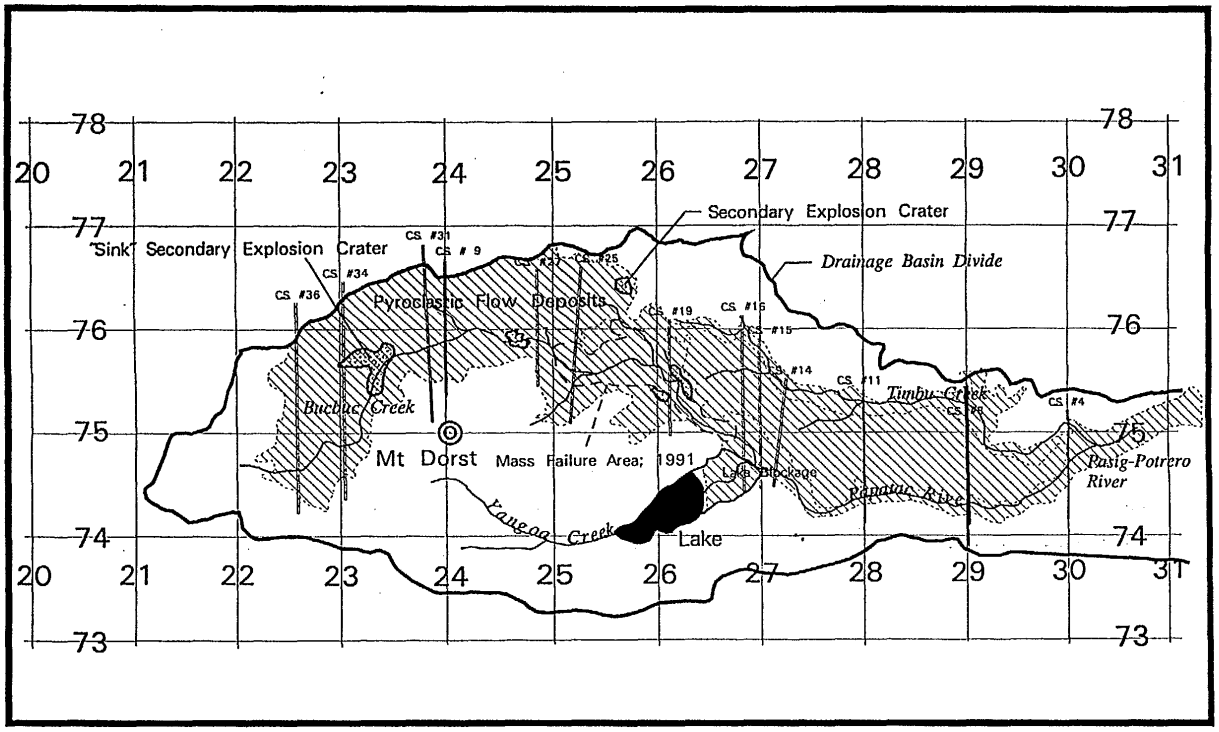

Figure B-8. Upper Pasig/Potrero River Basin Map. Illustrating Pyroclastic flow deposit and major geomorphic features producted between June 1991 - June 1993.

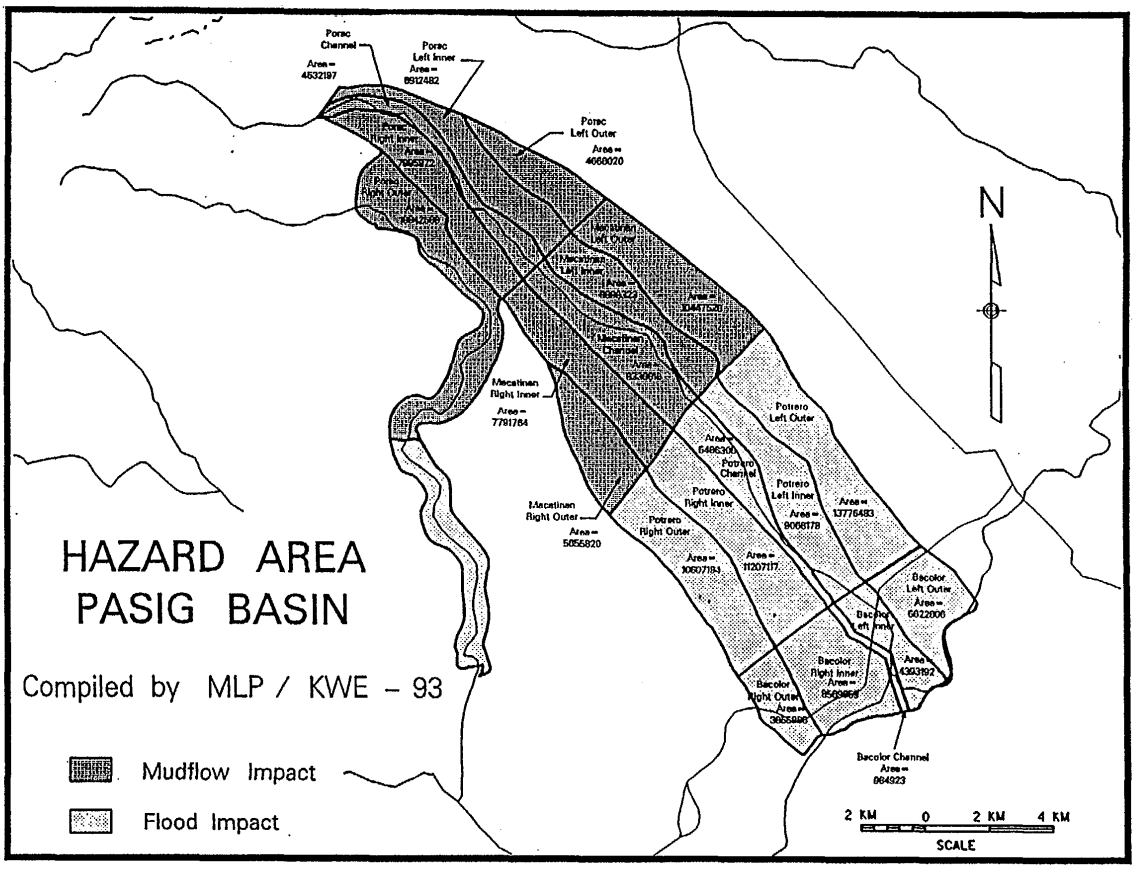

Figure B-9. Lower Pasig/Potrero River Basin Hazard Area.
Diagraming Mudflow and Flood Impact Zones.

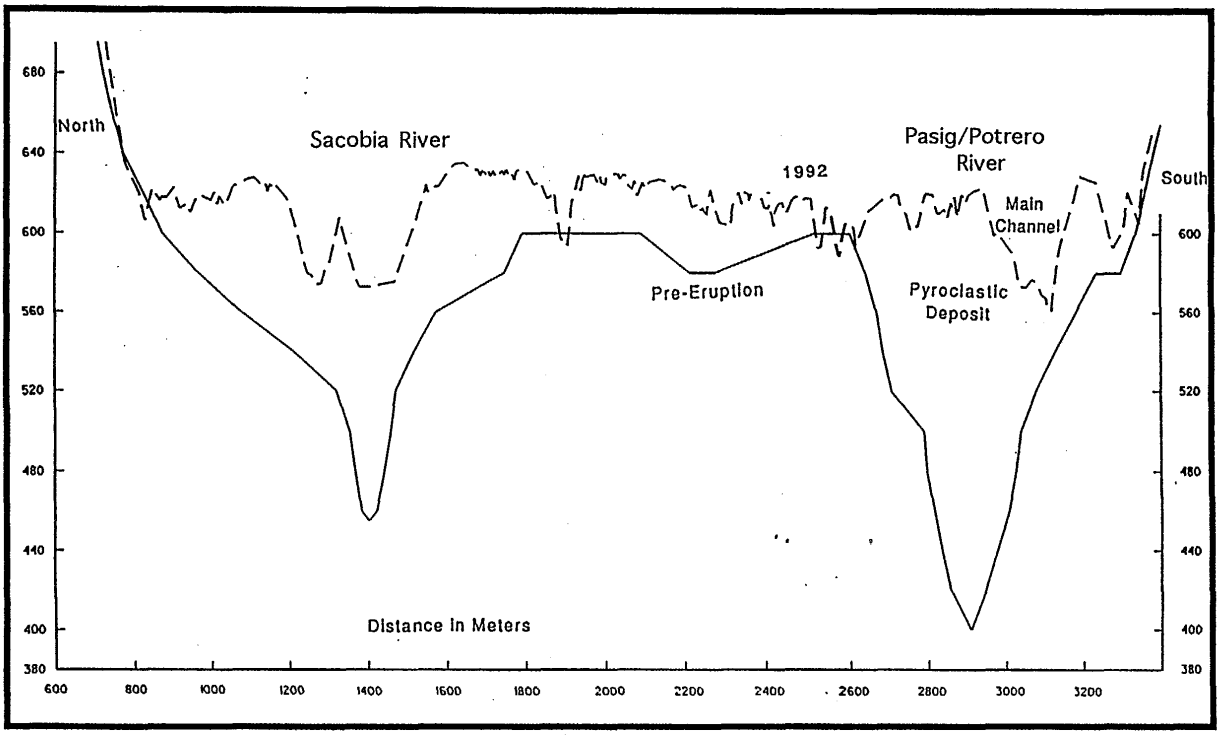

Figure B-10. Upper Pasig/Potrero River Channel Cross-Section #9.
Illustrating Pre- and Post- Eruption channel geometry. Both Sacobia and Pasig/Potrero Rivers are plotted. Data based on DMA topographic map and 1992 aerial photography.

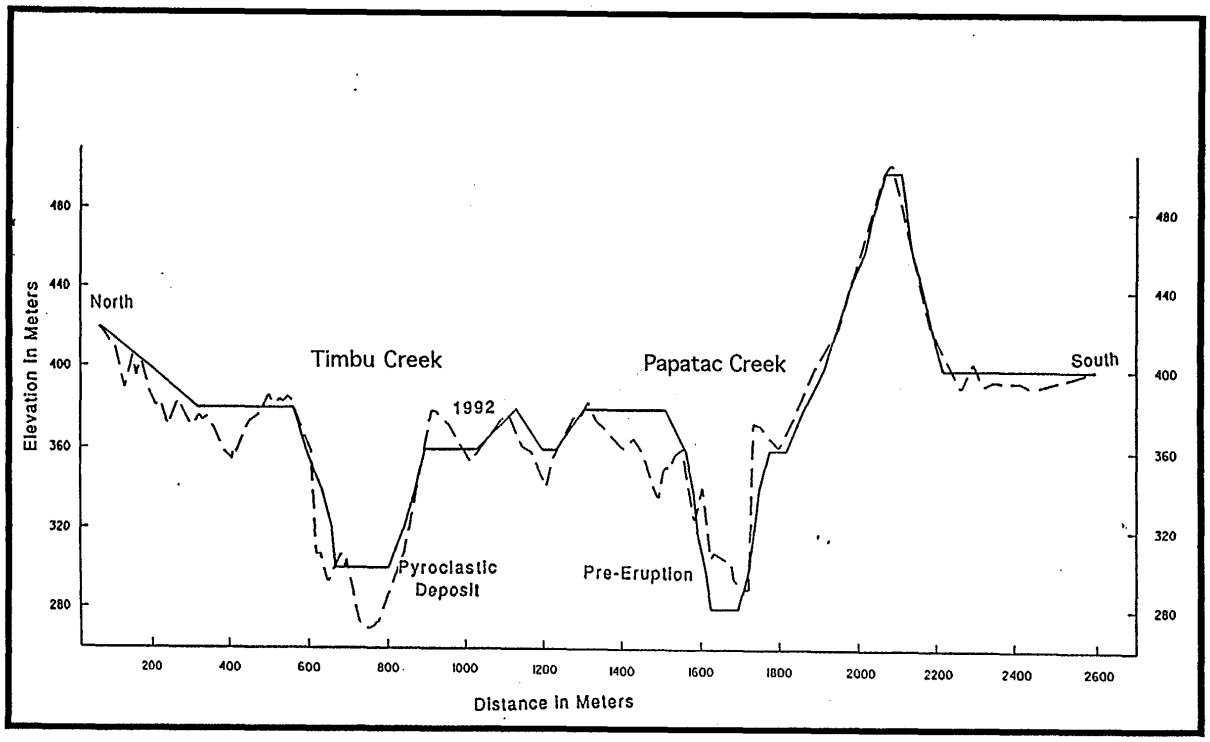

Figure B-11. Upper Pasig/Potrero River Channel Cross-Section #6.
Illustrating Pre- and Post- Eruption channel geometry of the Pasig/Potrero River. Cross-Section illustrates both Timbu and Papatac Creeks which form the Pasig/Potrero River system.
Data based on DMA topographic map and 1992 aerial photography.

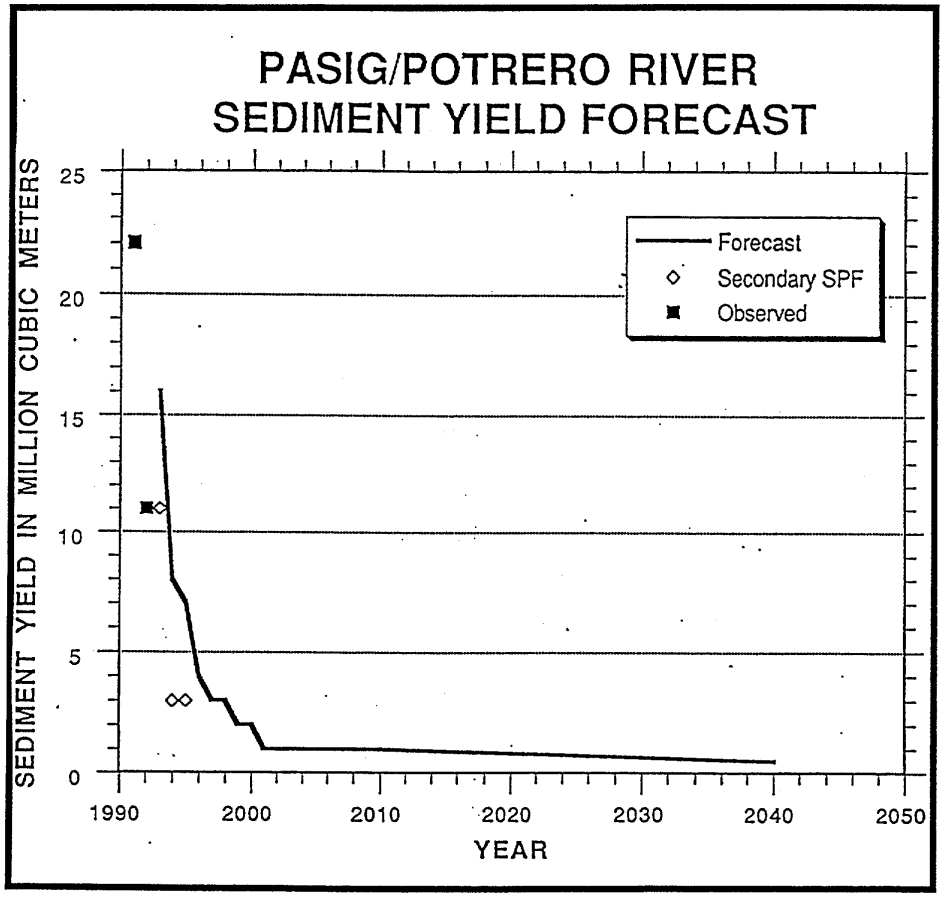

Figure B-12. Sediment Yield Forecast for the Pasig-Potrero River Pyroclastic Deposit.

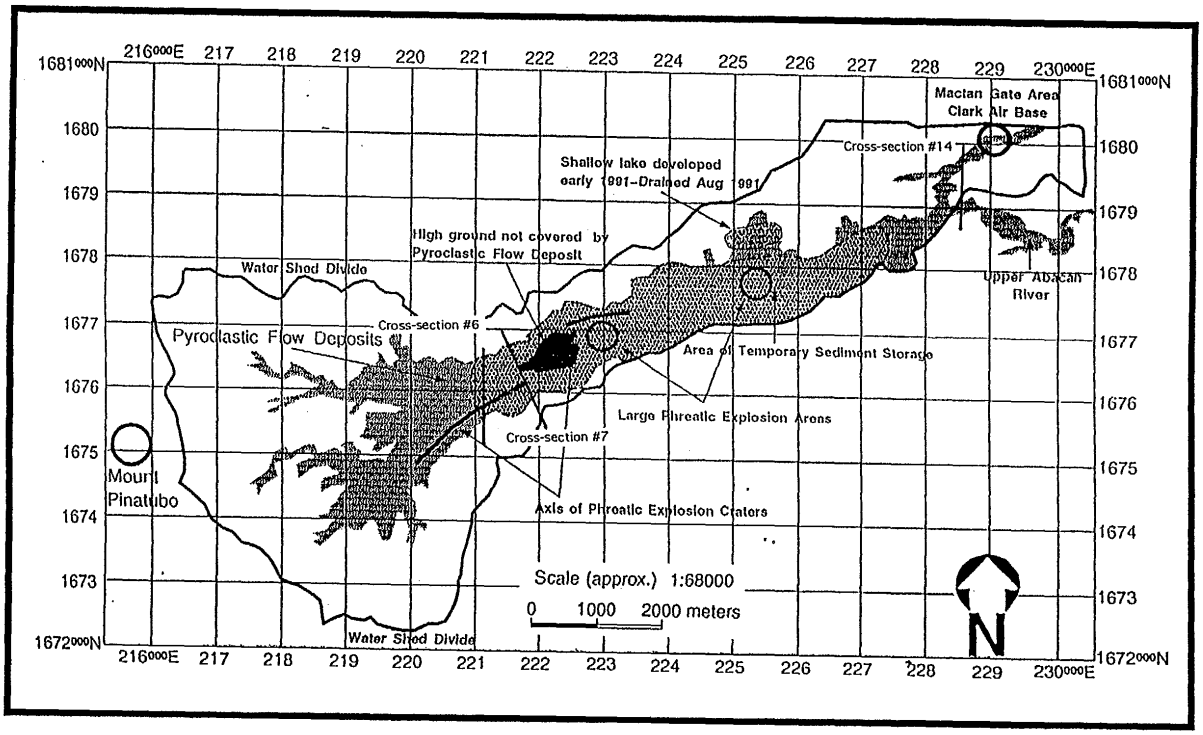

Figure B-13. Upper Sacobia River Basin Map. Illustrating Pyroclastic flow deposit and major geomorphic features producted during 1991.

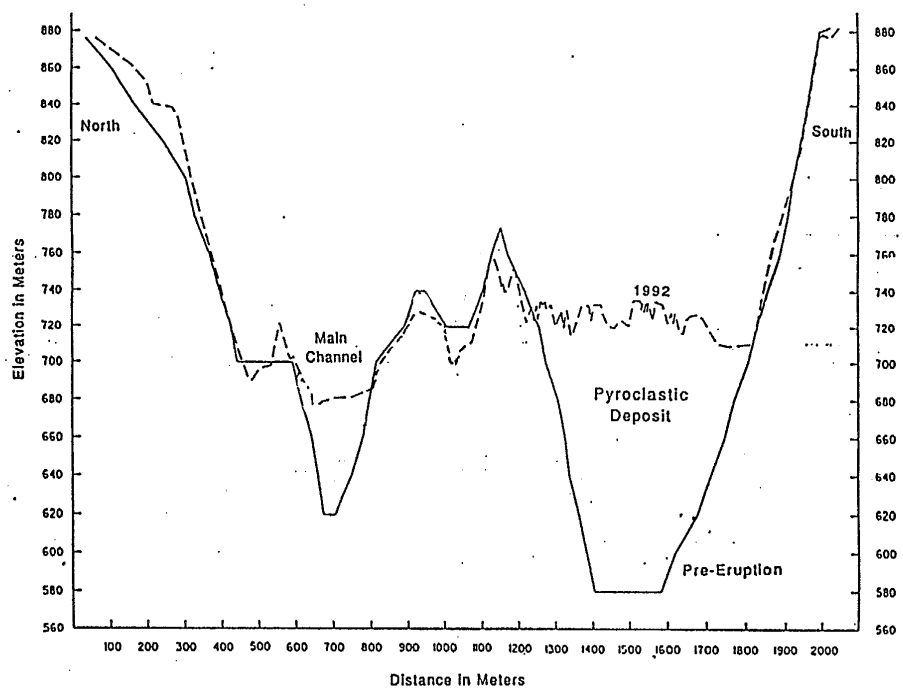

Figure B-14. Upper Sacobia River Chennel Cross-Section #6.
Illustrating Pre- and Post- Eruption channel geometry of the Sacobia River. Cross-Section illustrates the upper two channels of the Sacobia River. Location shown on figure B-13. Data based on DMA topographic map and 1992 aerial photography.

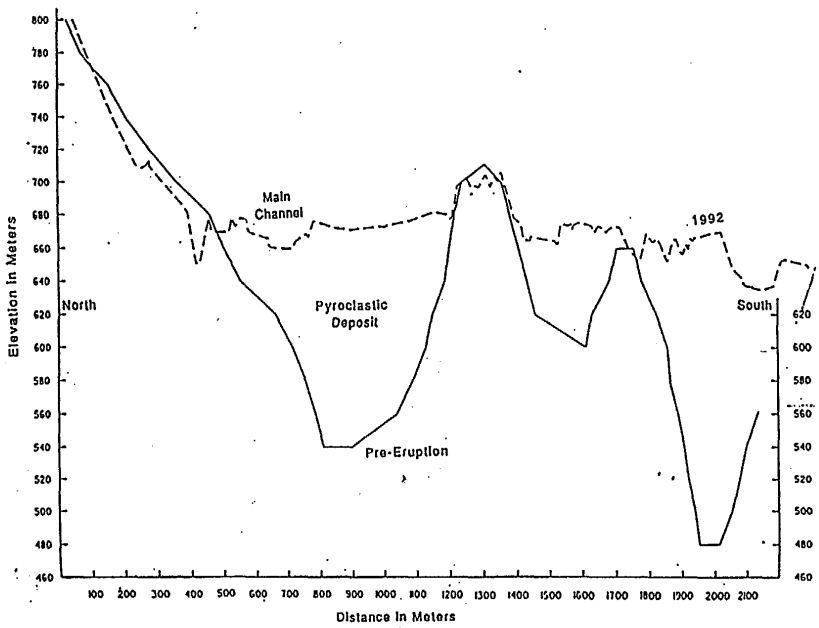

Figure B-15. Upper Sacobia River Channel Cross-Section #7.
Illustrating Pre- and Post- Eruption channel geometry of the Sacobia River. Cross-Section illustrates the Sacobia River and a portion of the Pasig/Potrero River Basin. Location shown on figure B-13. Data based on DMA topographic map and 1992 aerial photography.

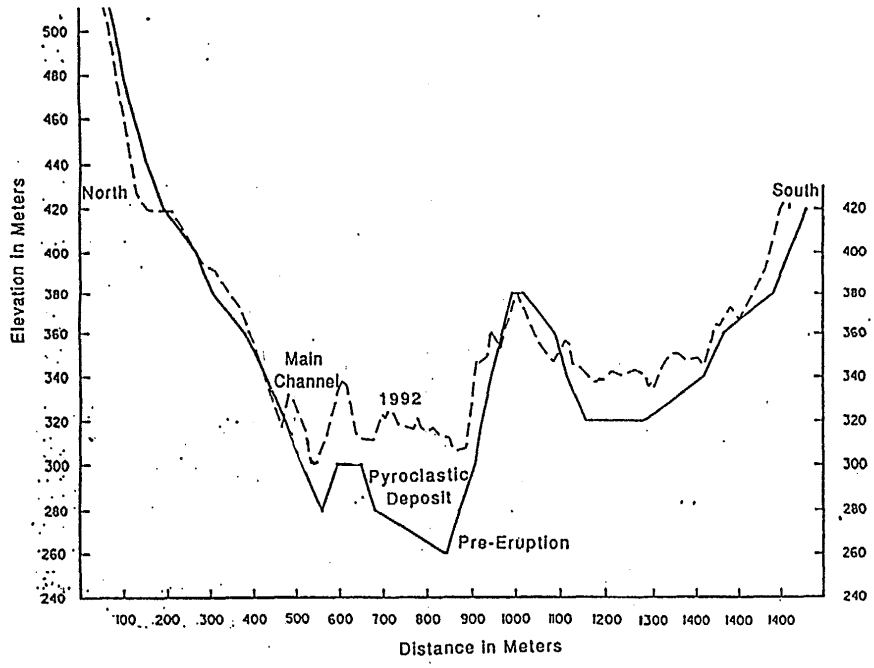

Figure B-16. Upper Sacobia River Channel Cross-Section #14.
Illustrating Pre- and Post- Eruption channel geometry of the Sacobia River. Cross-Section illustrates the Sacobia River and the Abacan River downstream of the "Gates of the Abacan" area. Location shown on figure B-13. Data based on DMA topographic map and 1992 aerial photography.

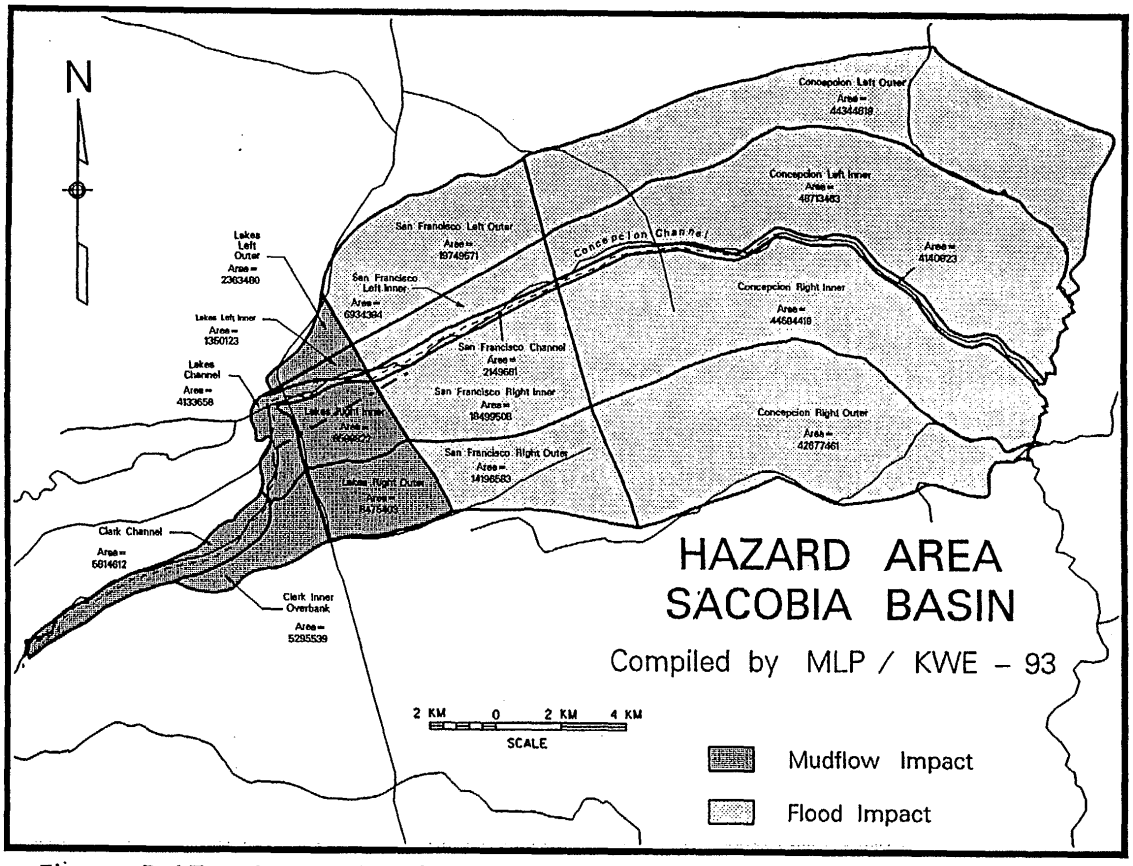

Figure B-17. Lower Sacobia River Basin Hazard Area.
Diagraming Mudflow and Flood Impact Zones.

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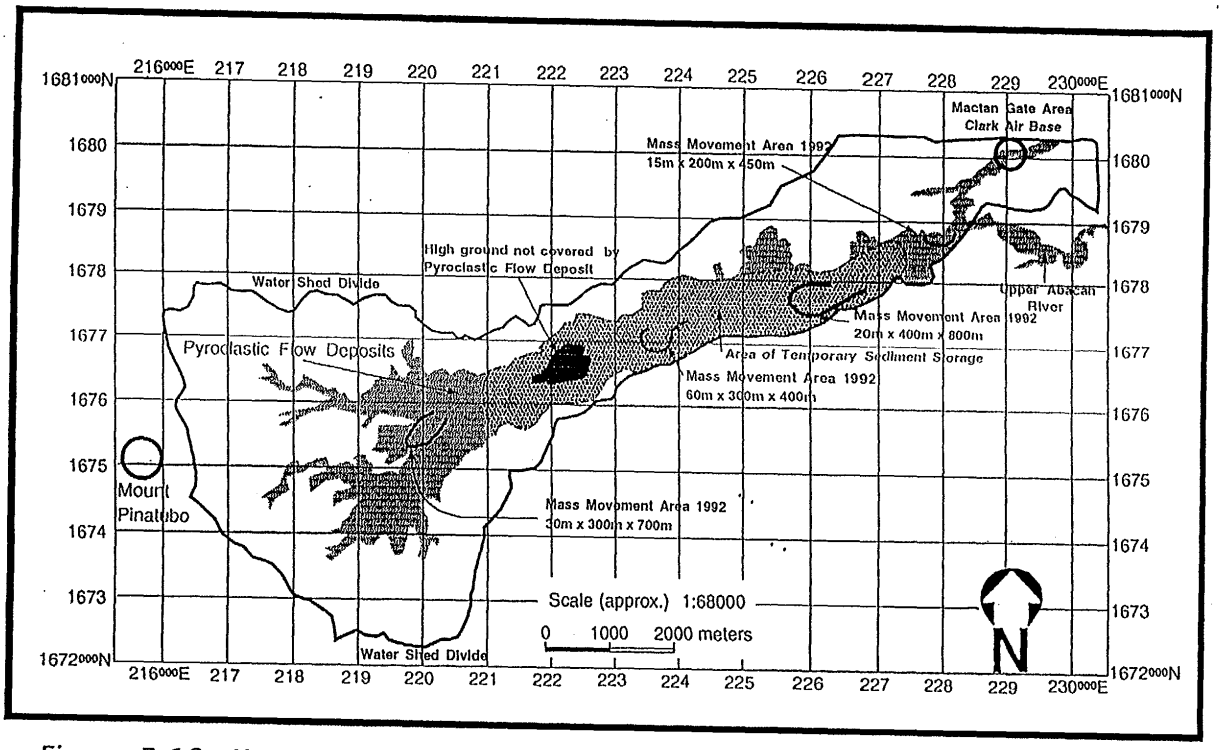

Figure B-18. Upper Sacobia River Basin Map. Illustrating Pyroclastic flow deposit and major geomorphic features producted during 1992.

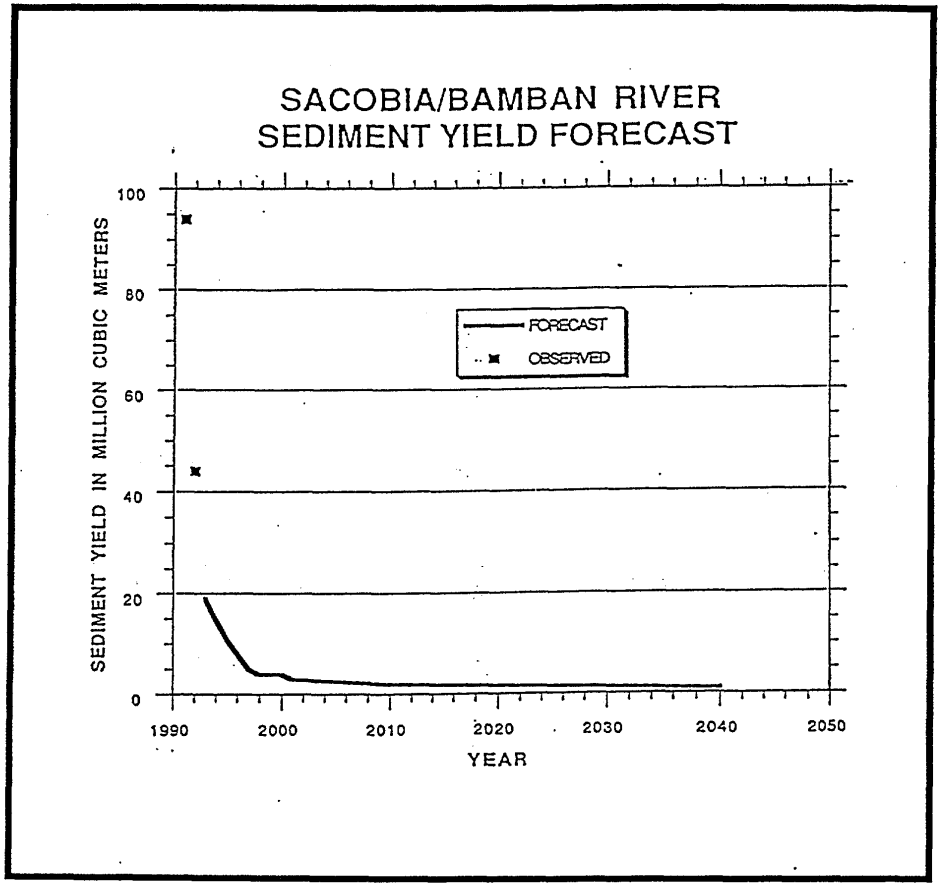

Figure B-19. Sediment Yield Forecast for the Sacobia/Bamban River Pyroclastic Deposit.

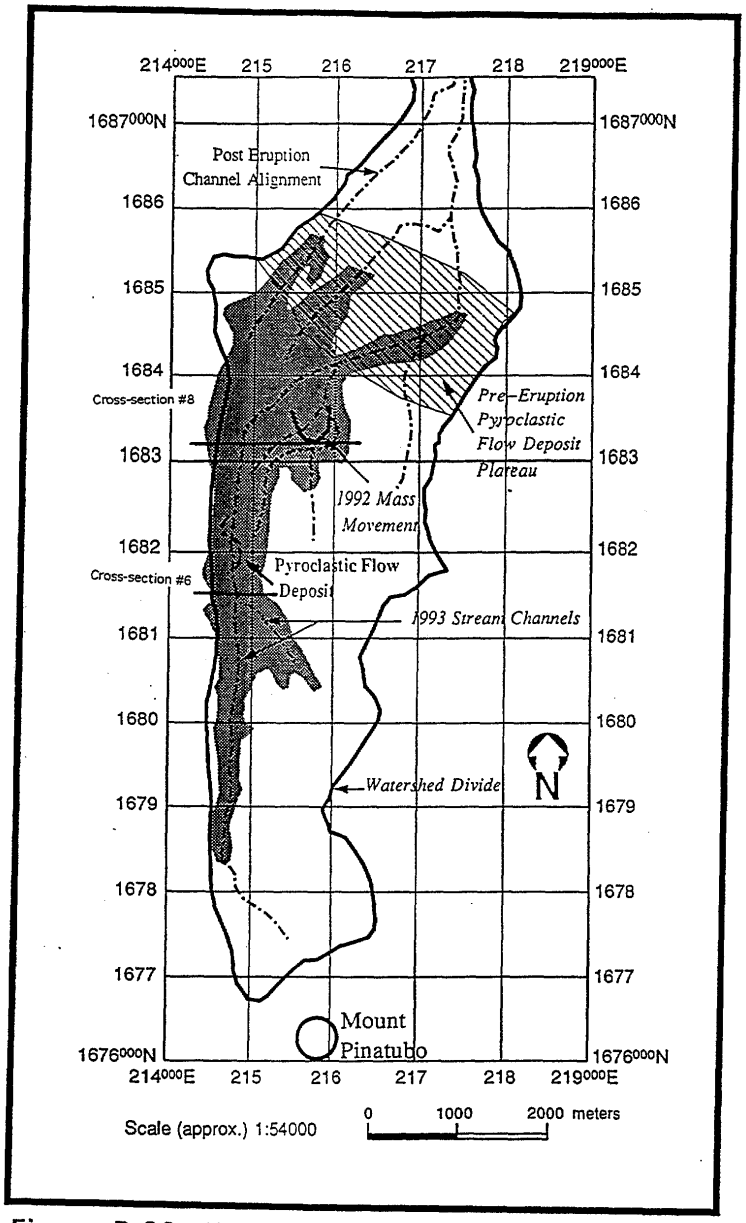

Figure B-20. Upper O,Donnell River Basin Map. Illustrating Pyroclastic flow deposit and major geomorphic features producted between June 1991 - August 1993.

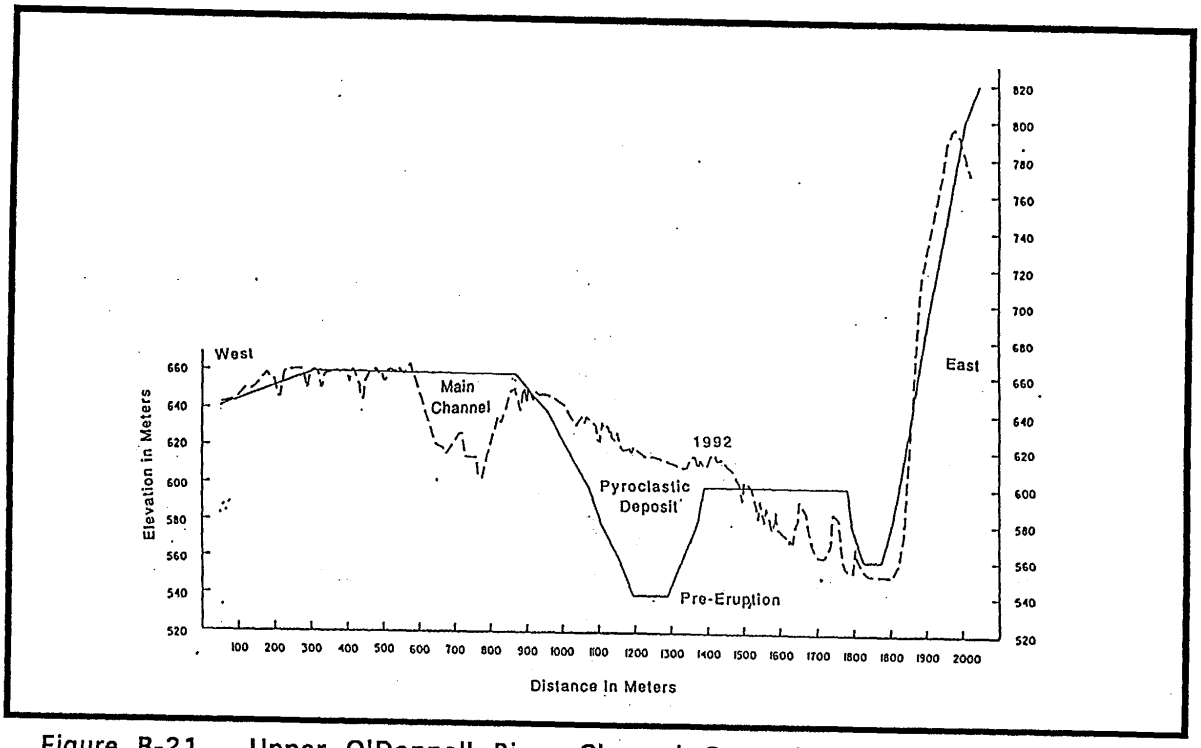

Figure B-21. Upper O'Donnell River Channel Cross-Section #8.
Illustrating Pre- and Post- Eruption channel geometry of the O'Donnell River. Cross-Section Location shown on figure B-20. Data based on DMA topographic map and 1992 aerial photography.

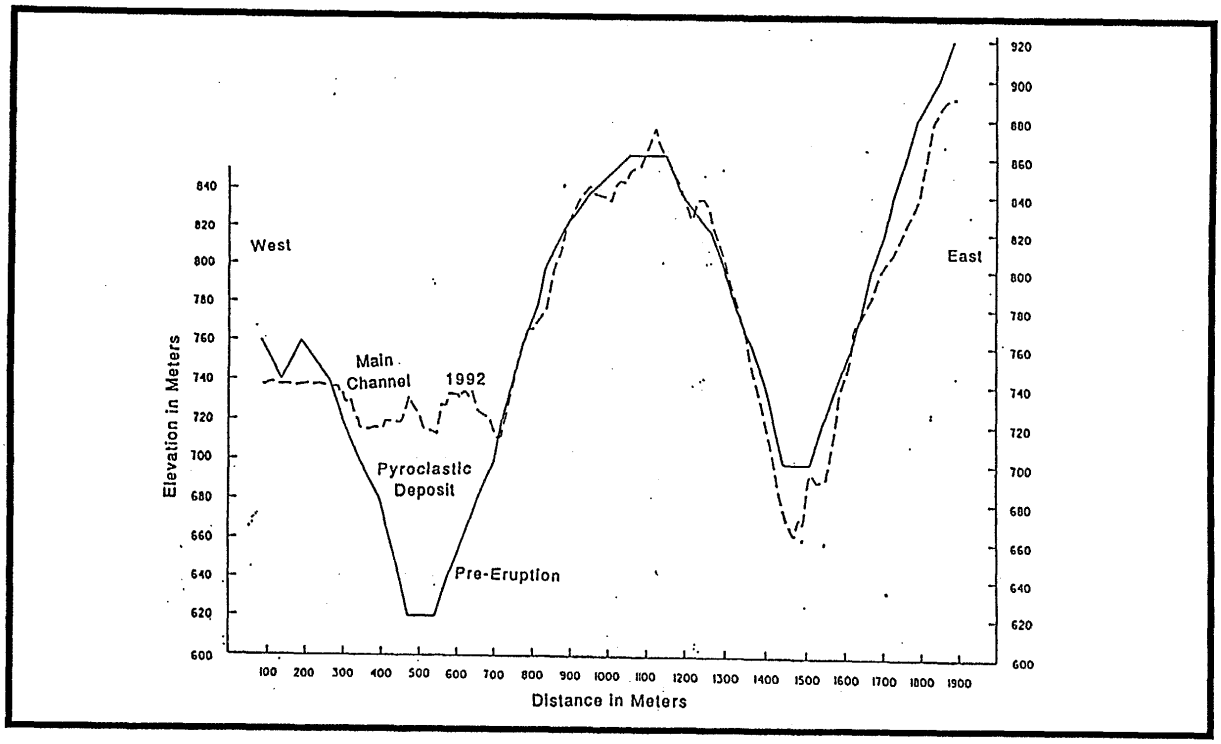

Figure B-22. Upper O'Donnell River Channel Cross-Section #6.
Illustrating Pre- and Post- Eruption channel geometry of the O'Donnell River. Cross-Section Location shown on figure B-20. Data based on DMA topographic map and 1992 aerial photography.

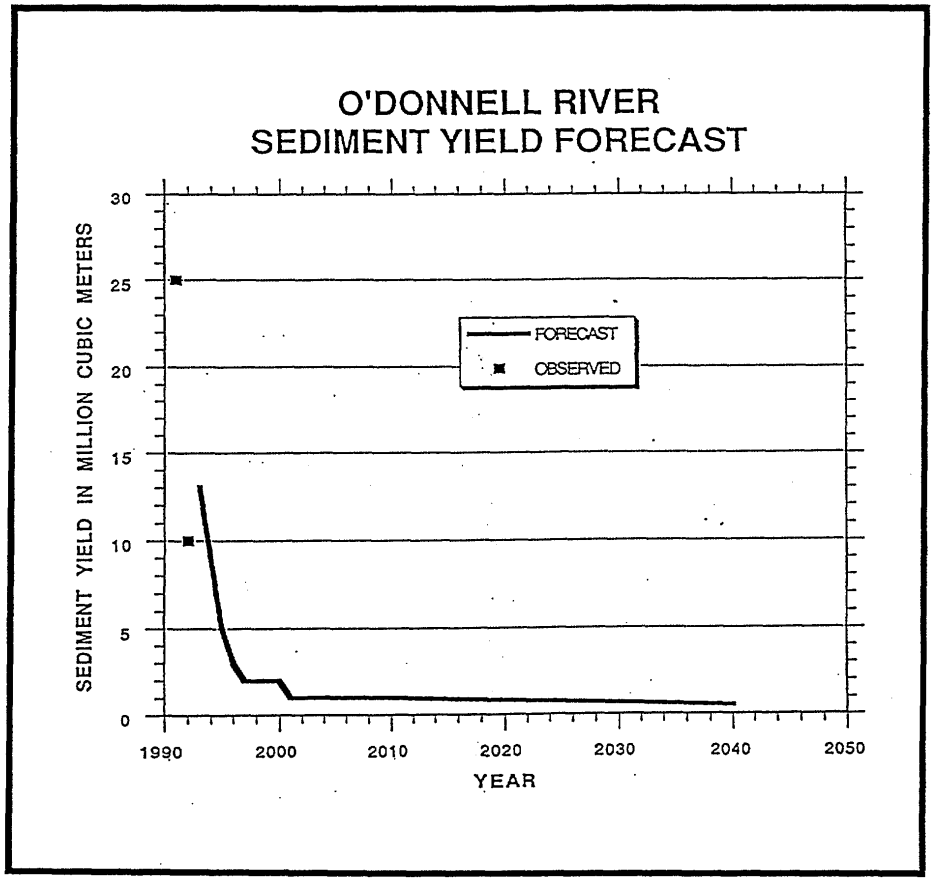

Figure B-23. Sediment Yield Forecast for the O'Donnell River Pyroclastic Deposit.

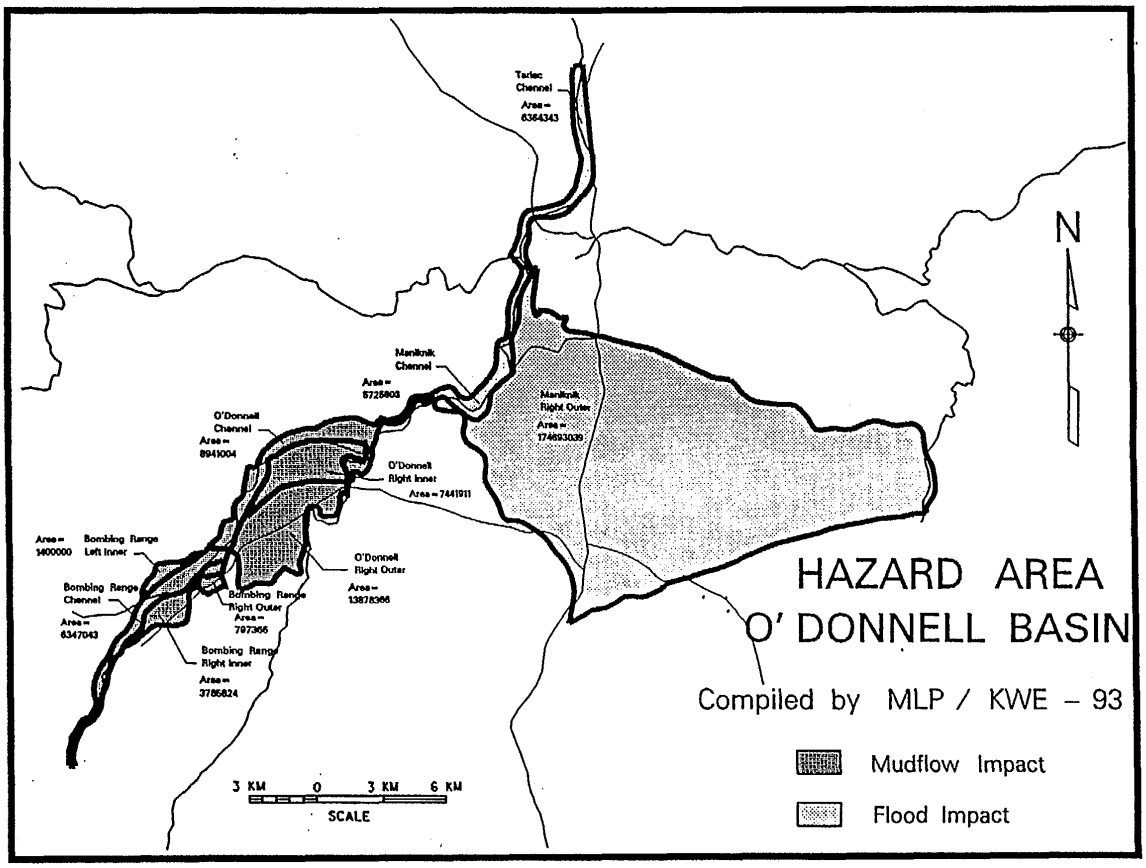

Figure B-24. Lower O'Donnell River Basin Hazard Area.
Diagraming Mudflow and Flood Impact Zones.

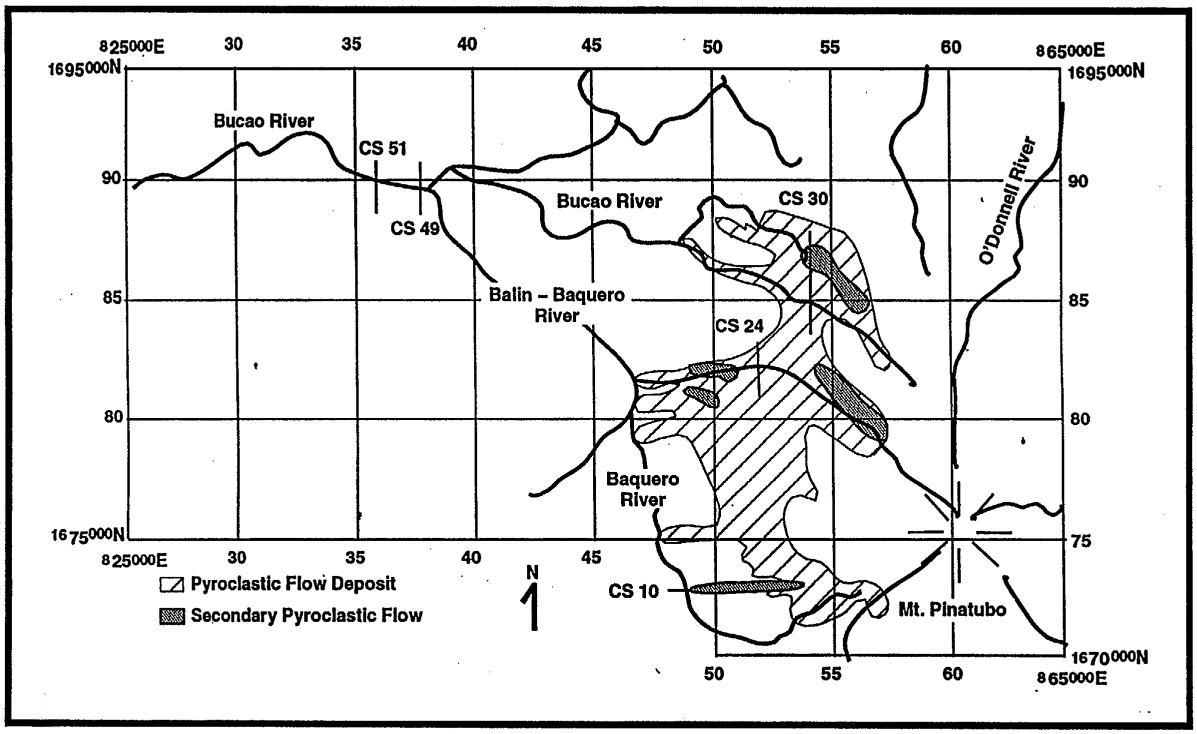

Figure B-25. Upper and Lower Bucao River Basin Map. Illustrating Pyroclastic flow deposit and major geomorphic features producted between June 1991 - June 1993.

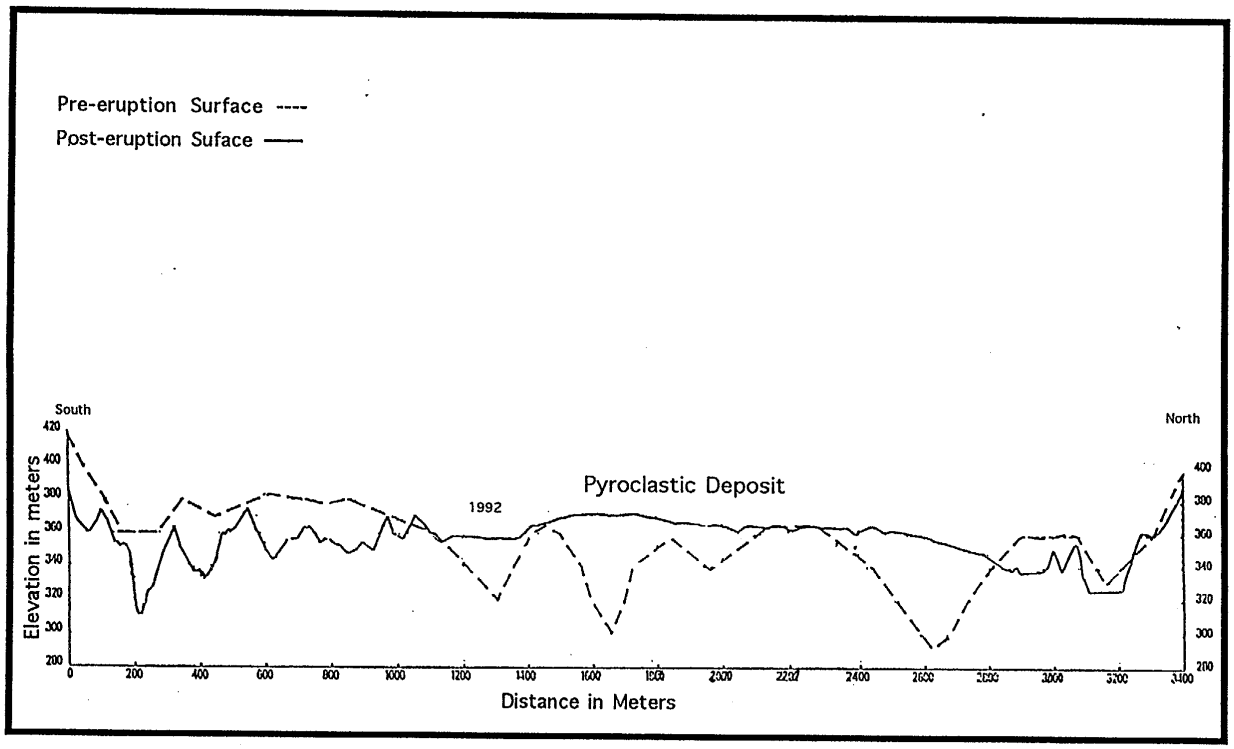

Figure B-26. Upper Bucao River Channel Cross-Section #30.
Illustrating Pre- and Post- Eruption channel geometry of the Bucao River. Cross-Section Location shown on figure B-25. Data based on DMA topographic map and 1992 aerial photography.

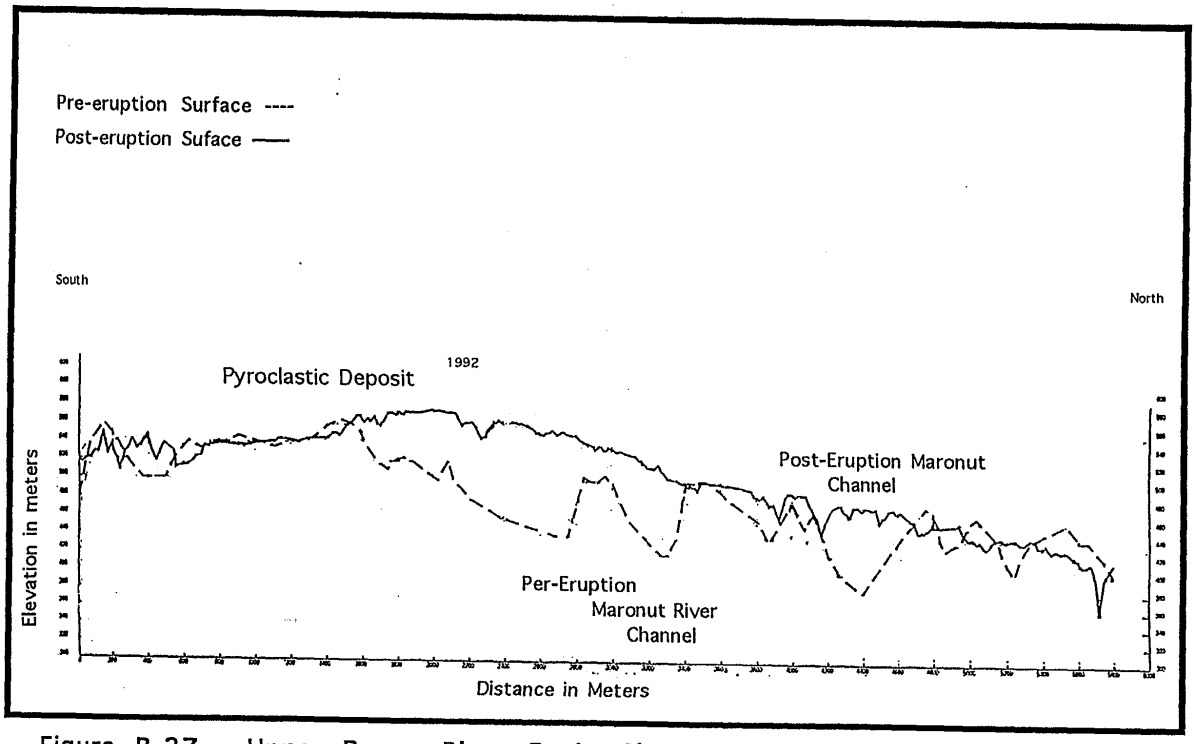

Figure B-27. Upper Bucao River Basin Channel Cross-Section #24.
Illustrating Pre- and Post- Eruption channel geometry of the Maronut River, a tributary to the Balin-Buquero River. Cross-Section Location shown on figure B-25.
Data based on DMA topographic map and 1992 aerial photography.

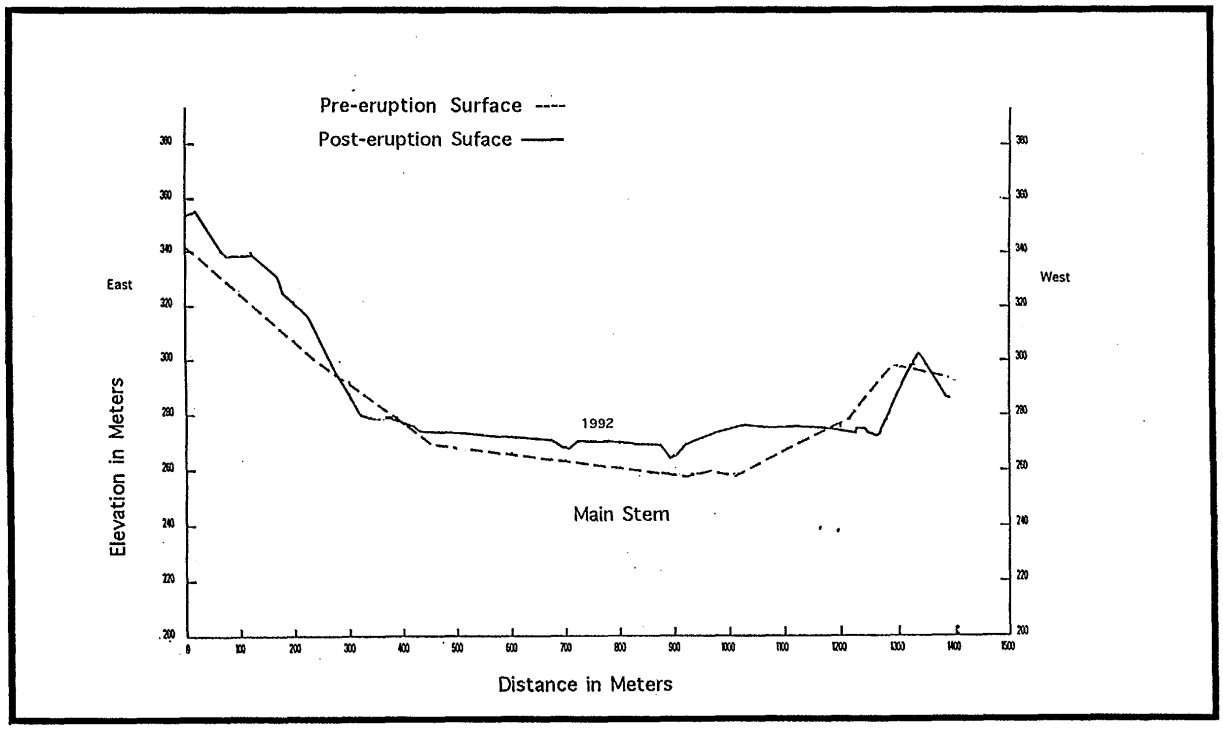

Figure B-28. Upper Bucao River Basin Channel Cross-Section #10.
Illustrating Pre- and Post- Eruption channel geometry of the Balin-Buquero River, a major tributary to the Bucao River. Cross-Section Location shown on figure B-25.
Data based on DMA topographic map and 1992 aerial photography.

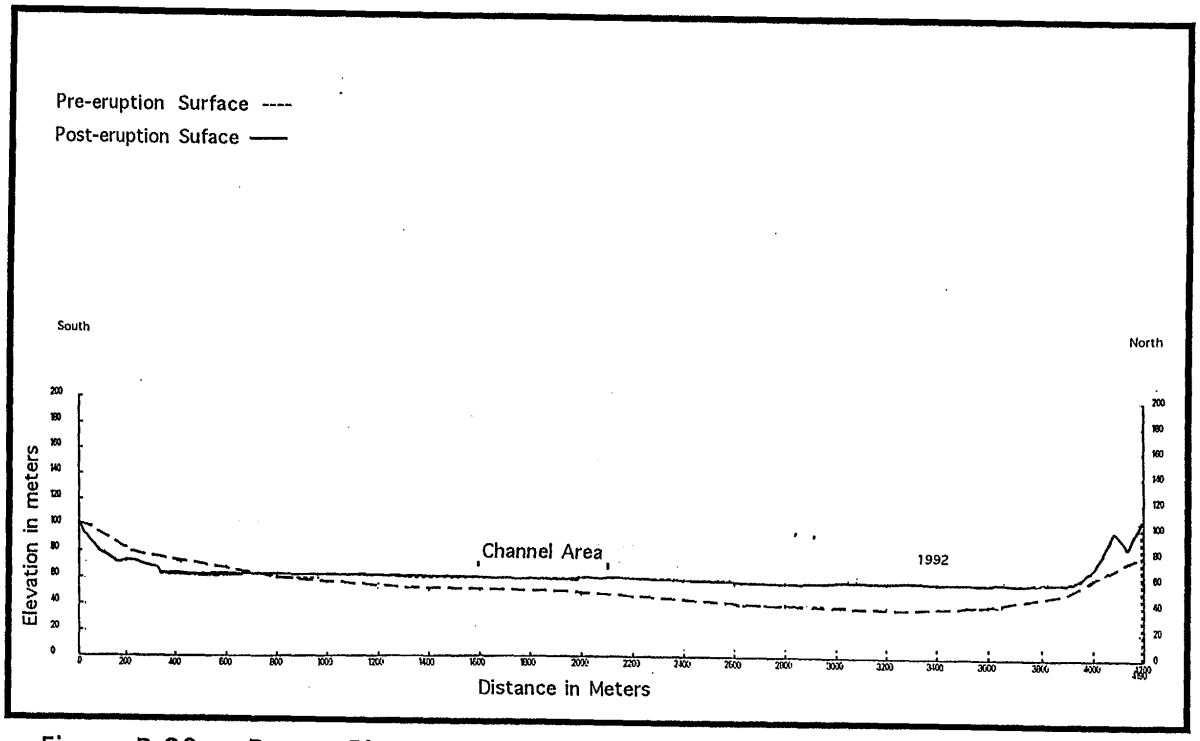

Figure B-29. Bucao River Basin Channel Cross-Section #49.
Illustrating Pre- and Post- Eruption channel geometry of the Bucao River. Cross-Section Location shown on figure B-25. Data based on DMA topographic map and 1992 aerial photography.

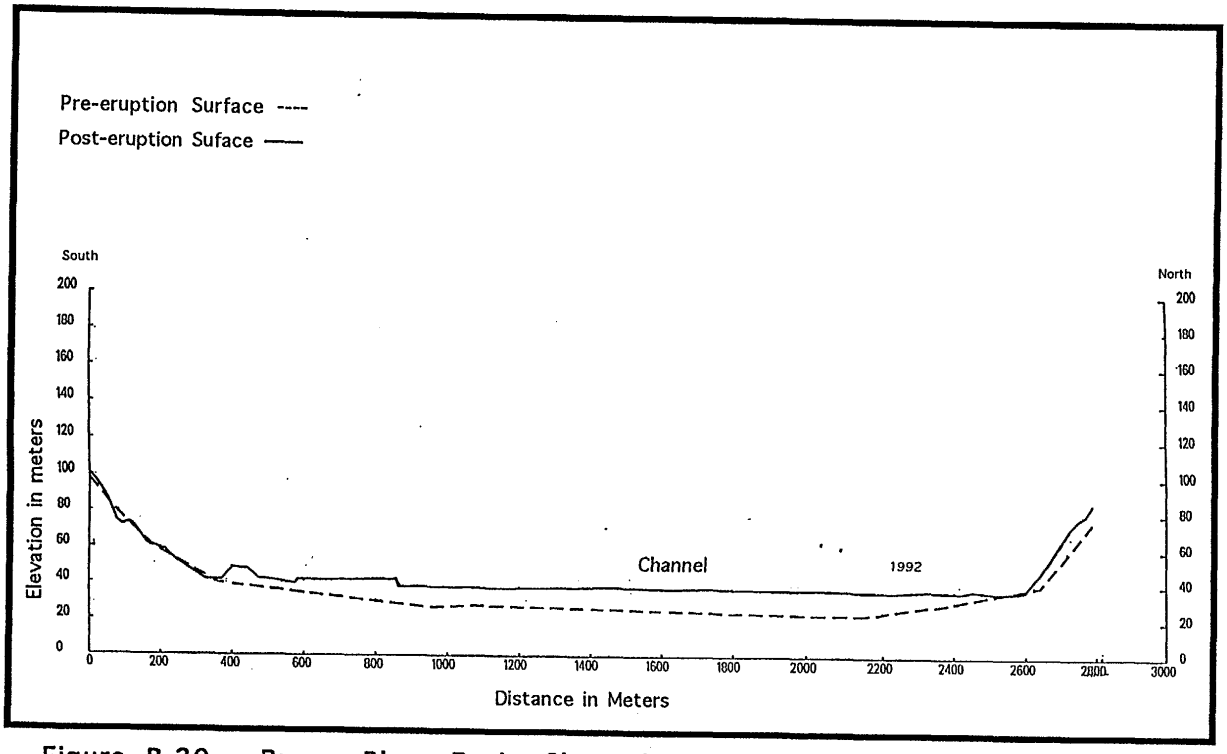

Figure B-30. Bucao River Basin Channel Cross-Section #51.
Illustrating Pre- and Post- Eruption channel geometry of the Bucao River. Cross-Section Location shown on figure B-25. Data based on DMA topographic map and 1992 aerial photography.

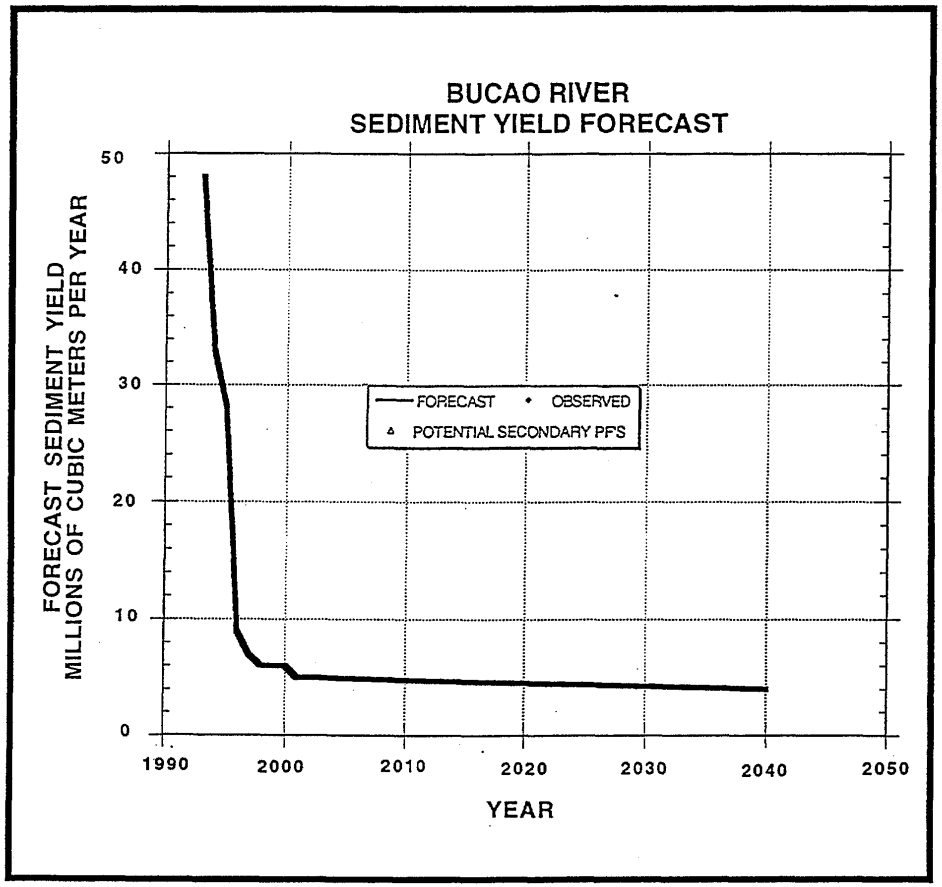

Figure B-31. Sediment Yield Forecast for the Bucao River Pyroclastic Deposit.

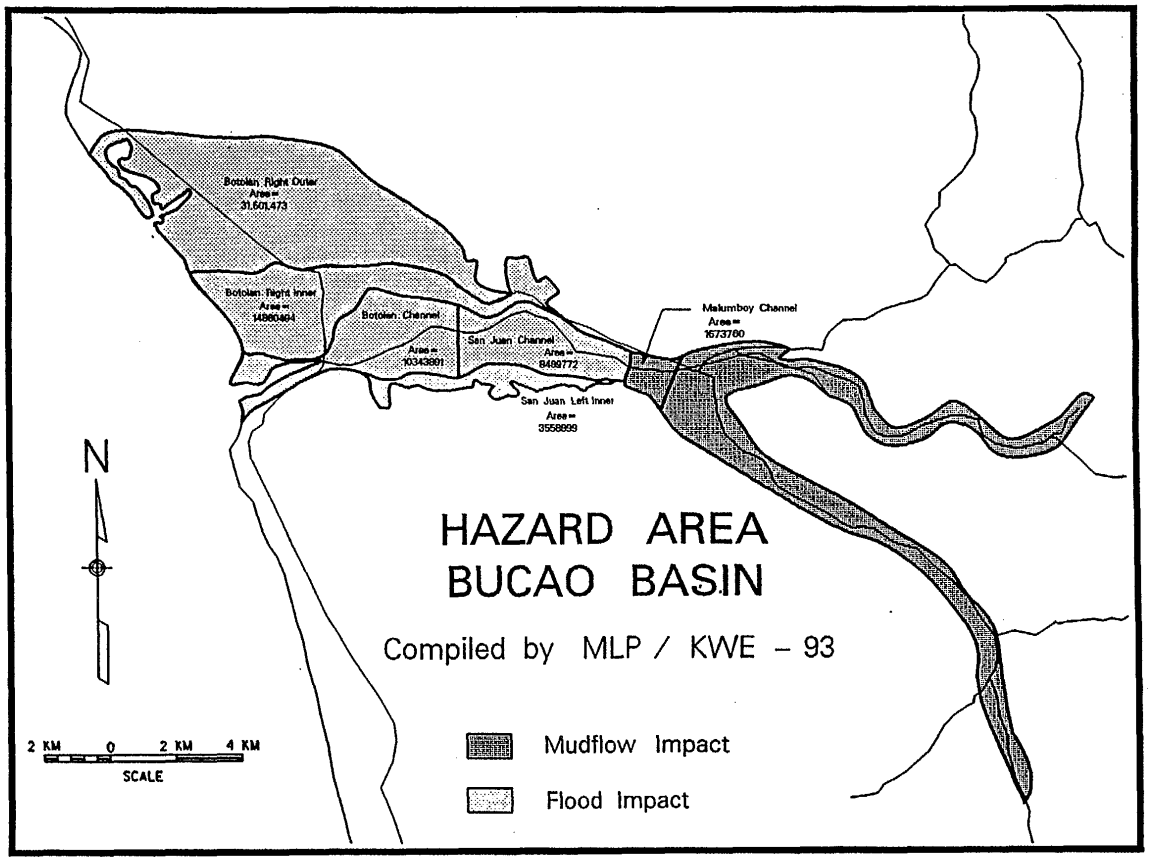

Figure B-32. Lower Bucao River Basin Hazard Area.
Diagraming Mudflow and Flood Impact Zones.

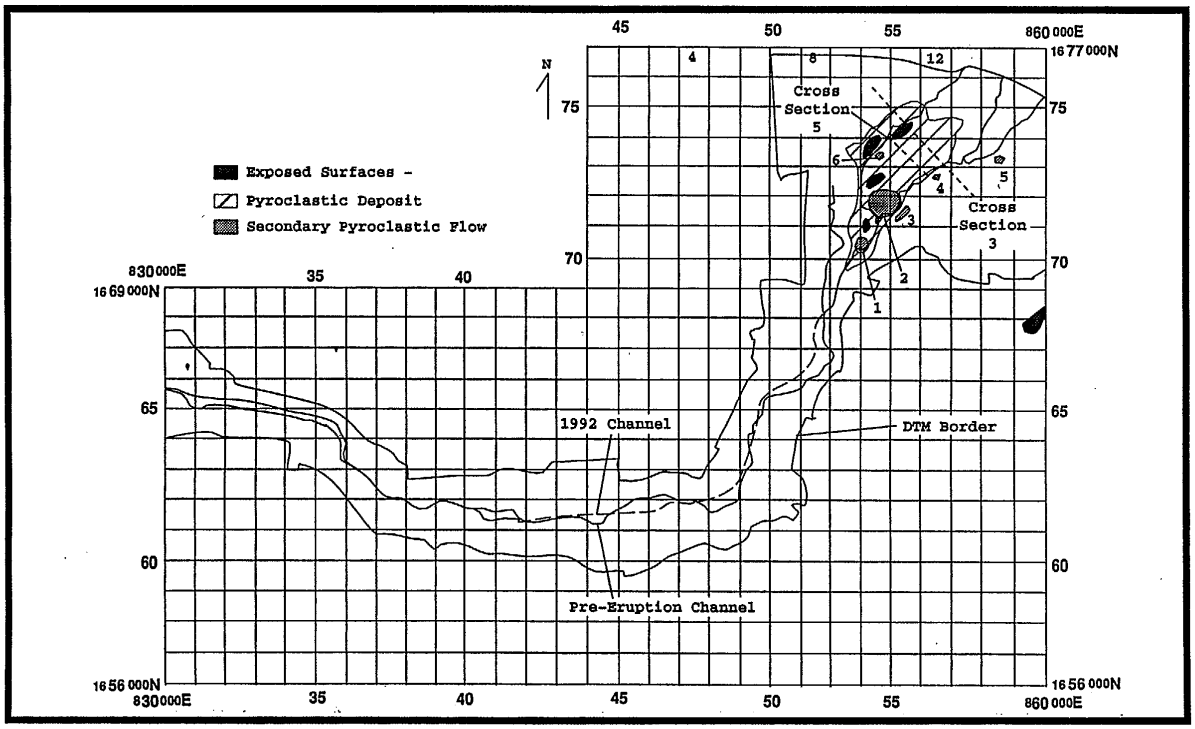

Figure B-33. Upper and Lower Santo Tomas River Basin Map. Illustrating Pyroclastic flow deposit and major geomorphic features producted between June 1991 - June 1993.

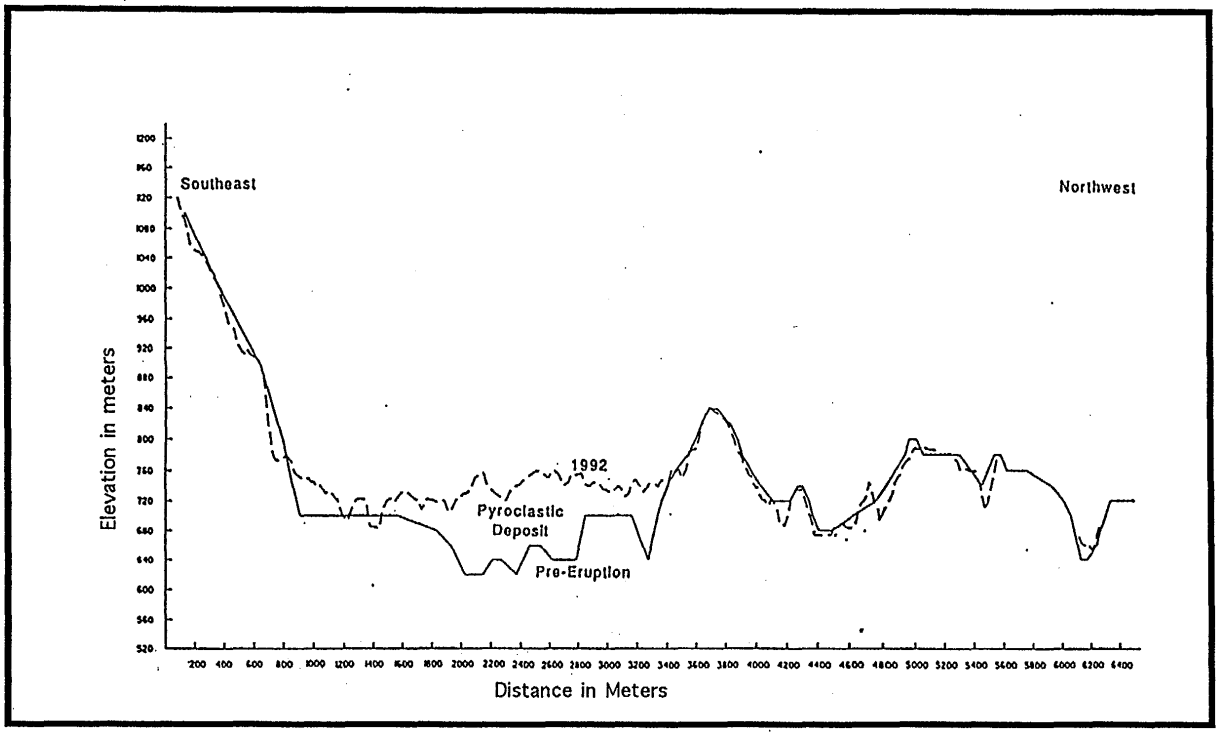

Figure B-34. Santo Tomas River Basin Channel Cross-Section #3.
Illustrating Pre- and Post- Eruption channel geometry of the Santo Tomas River.
Cross-Section Location shown on figure B-33. Data based on DMA topographic map and 1992 aerial photography.

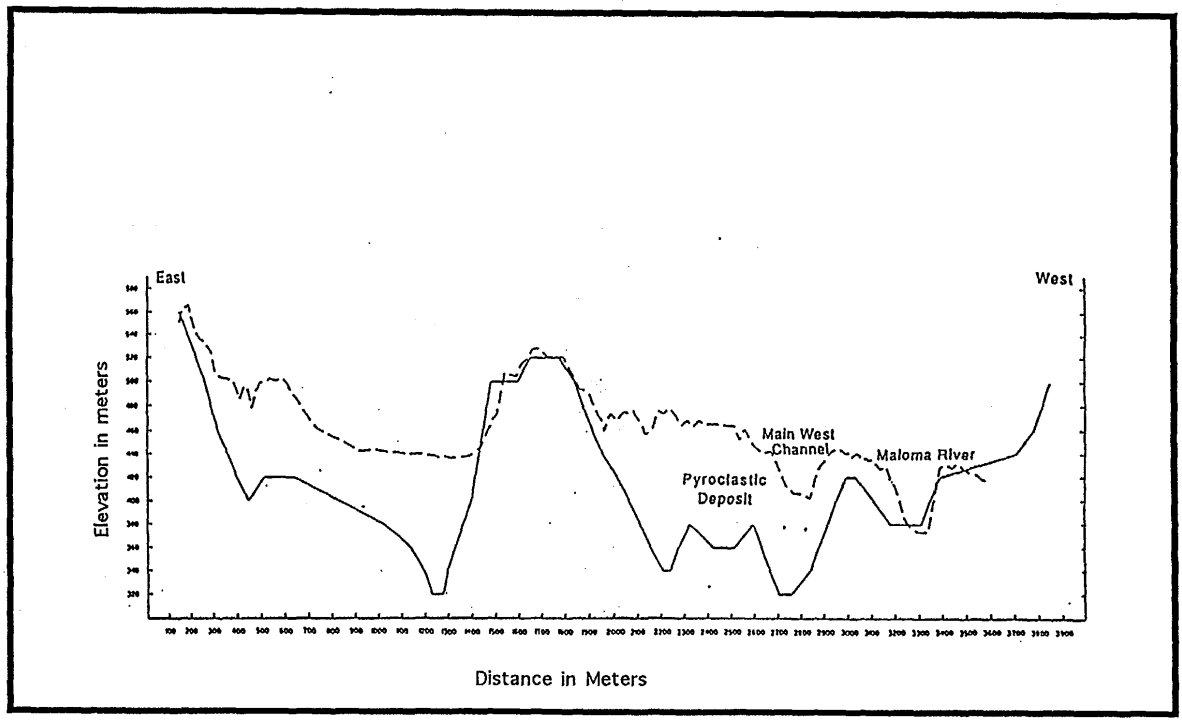

Figure B-35. Santo Tomas River Basin Channel Cross-Section #5.
Illustrating Pre- and Post- Eruption channel geometry of the Bucao River.
Cross-Section Location shown on figure B-33. Data based on DMA topographic map and 1992 aerial photography.

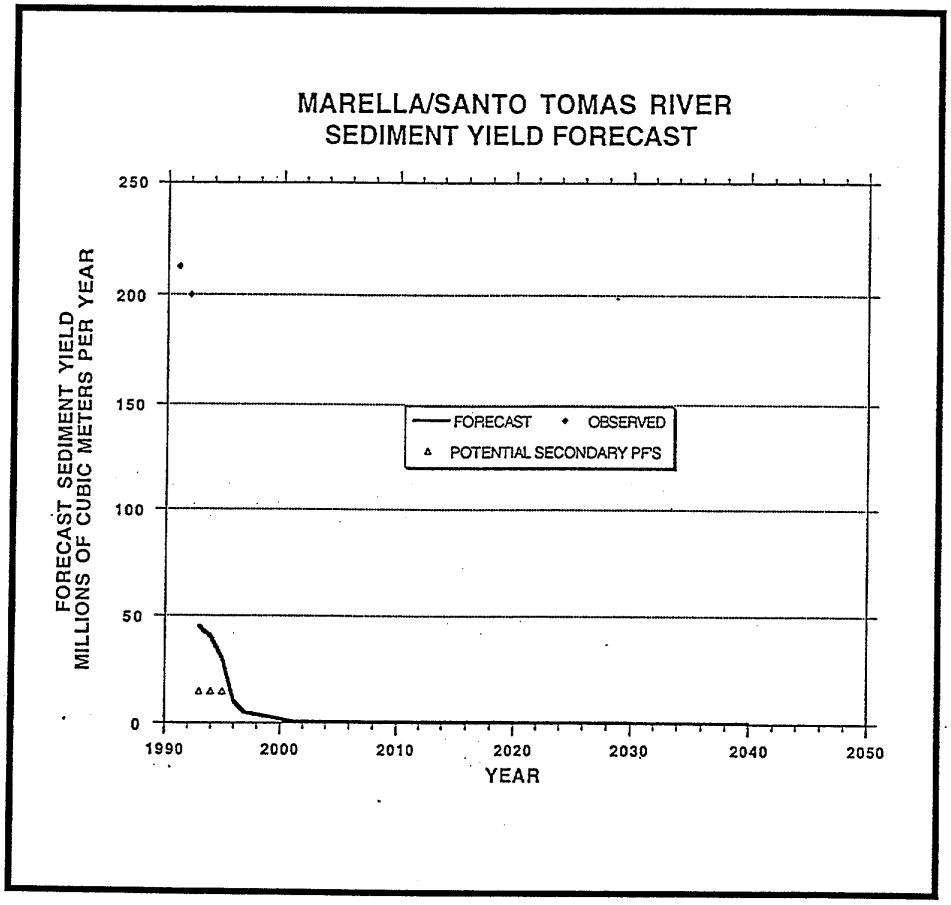

Figure B-36. Sediment Yield Forecast for the Marella/Santo Tomas Pyroclastic Deposit.

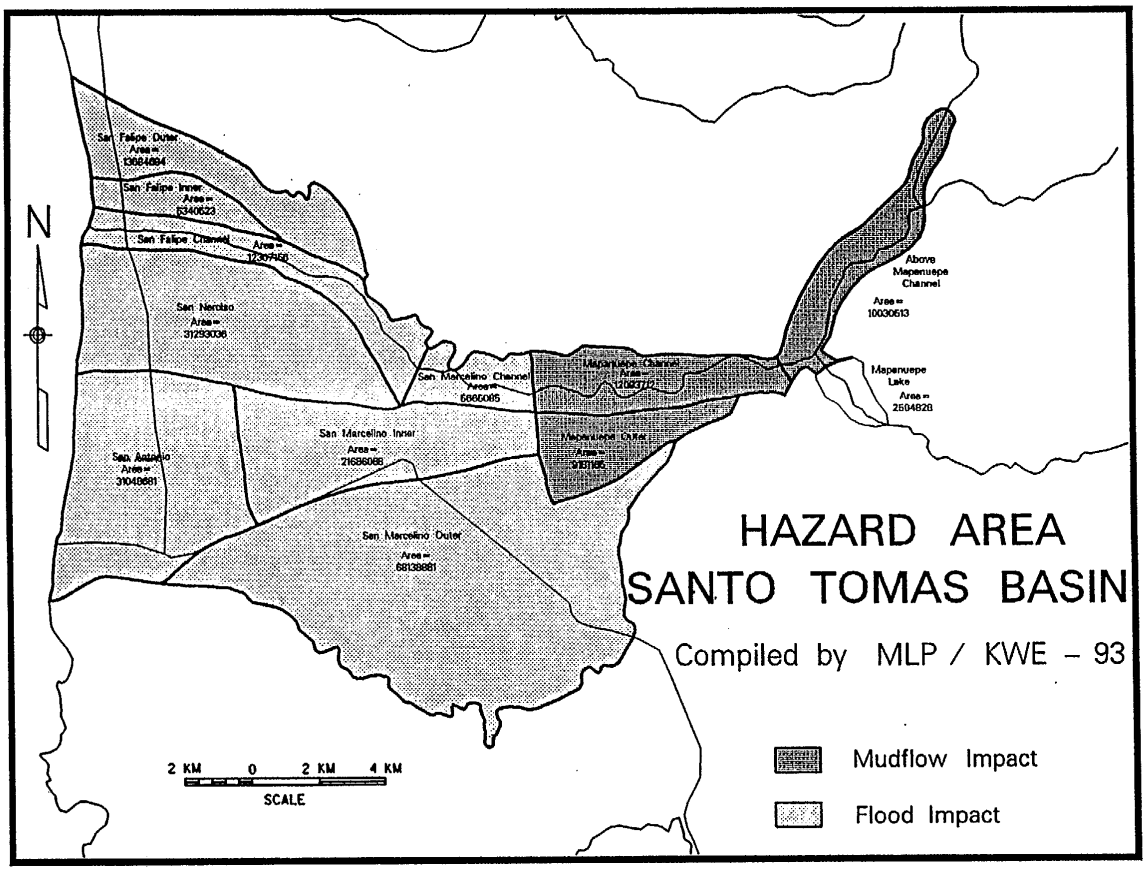

Figure B-37. Lower Santo Tomas River Basin Hazard Area.
Diagraming Mudflow and Flood Impact Zones.

Table 24 -- *Summary of Economic Information for Alternatives*

Basin	Project	Mean Benefits	Economic Costs	Net Benefits	Benefit-Cost Ratio	Internal Rate of Return (%)
Abacan	Bank Protection	191,679,000	67,953,000	123,726,000	2.82	38.7
Bucao	Levee	210,979,000	155,076,000	55,903,000	1.36	17.0
Bucao	SRS	223,778,000	3,317,898,000	-3,094,120,000	0.07	
Delta	Dredging	3,284,870,000	1,079,512,000	2,205,358,000	3.04	130.4
Maloma	Levee	97,635,000	85,218,000	12,417,000	1.15	15.7
Maloma	SRS	97,635,000	184,705,000	-87,070,000	0.53	3.5
Maloma	Channel Excavation	97,635,000	128,977,000	-31,342,000	0.76	7.6
O'Donnell	Levee	187,281,000	188,240,000	-959,000	0.99	12.1
O'Donnell	SRS	249,788,000	2,246,013,000	-1,996,225,000	0.11	
O'Donnell	Channel Excavation	187,281,000	1,244,019,000	-1,056,738,000	0.15	
Pasig-Potrero	Levee	657,849,000	1,548,626,000	-890,777,000	0.42	
Pasig-Potrero	Channel Excavation	657,849,000	1,943,000,000	-1,285,151,000	0.34	
Porac-Gumain	Levee	975,495,000	587,176,000	388,319,000	1.66	23.0
Porac-Gumain	SRS	893,482,000	1,138,975,000	-245,493,000	0.78	8.4
Porac-Gumain	Channel Excavation	975,495,000	561,186,000	414,309,000	1.74	24.3
Sacobia-Bamban	Levee	434,281,000	1,078,450,000	-644,169,000	0.40	
Sacobia-Bamban	SRS	351,663,000	1,410,233,000	-1,058,570,000	0.25	
Sacobia-Bamban	Channel Excavation	434,281,000	1,555,713,000	-1,121,432,000	0.28	4.5
Santo Tomas	Levee	907,490,000	739,658,000	167,832,000	1.23	18.1
Santo Tomas	SRS	723,101,000	3,886,893,000	-3,163,792,000	0.19	
Santo Tomas	Channel Excavation	907,490,000	3,866,500,000	-2,959,010,000	0.23	

Table 24 -- *Summary of Economic Information for Alternatives*

Basin	Project	Mean Benefits	Economic Costs	Net Benefits	Benefit-Cost Ratio	Internal Rate of Return (%)
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Bucao	SRS	223,778,000	3,317,898,000	-3,094,120,000	0.07	
Delta	Dredging	3,284,870,000	1,079,512,000	2,205,358,000	3.04	130.4
Maloma	Levee	97,635,000	85,218,000	12,417,000	1.15	15.7
Maloma	SRS	97,635,000	184,705,000	-87,070,000	0.53	3.5
Maloma	Channel Excavation	97,635,000	128,977,000	-31,342,000	0.76	7.6
O'Donnell	Levee	187,281,000	188,240,000	-959,000	0.99	12.1
O'Donnell	SRS	249,788,000	2,246,013,000	-1,996,225,000	0.11	
O'Donnell	Channel Excavation	187,281,000	1,244,019,000	-1,056,738,000	0.15	
Pasig-Potrero	Levee	657,849,000	1,548,626,000	-890,777,000	0.42	
Pasig-Potrero	Channel Excavation	657,849,000	1,943,000,000	-1,285,151,000	0.34	
Porac-Gumain	Levee	975,495,000	587,176,000	388,319,000	1.66	23.0
Porac-Gumain	SRS	893,482,000	1,138,975,000	-245,493,000	0.78	8.4
Porac-Gumain	Channel Excavation	975,495,000	561,186,000	414,309,000	1.74	24.3
Sacobia-Bamban	Levee	434,281,000	1,078,450,000	-644,169,000	0.40	
Sacobia-Bamban	SRS	351,663,000	1,410,233,000	-1,058,570,000	0.25	
Sacobia-Bamban	Channel Excavation	434,281,000	1,555,713,000	-1,121,432,000	0.28	4.5
Santo Tomas	Levee	907,490,000	739,658,000	167,832,000	1.23	18.1
Santo Tomas	SRS	723,101,000	3,886,893,000	-3,163,792,000	0.19	
Santo Tomas	Channel Excavation	907,490,000	3,866,500,000	-2,959,010,000	0.23	

TECHNICAL APPENDIX

C

ECONOMIC ANALYSIS

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MOUNT PINATUBO
RECOVERY ACTION PLAN
LONG TERM REPORT

TECHNICAL APPENDIX C
ECONOMIC ANALYSIS

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MOUNT PINATUBO
RECOVERY ACTION PLAN
LONG TERM REPORT

TECHNICAL APPENDIX C
ECONOMIC ANALYSIS

1. SOCIO-ECONOMIC PROFILE

1.1 Introduction

This section describes general economic conditions for Region III , or Central Luzon[1] and the provinces of Pampanga, Tarlac and Zambales. In addition to providing background information, this section presents some of the data required to estimate potential damages for the with- and without-project condition for each of the eight river basins being considered in this study. A map showing the location of the river basins and provinces is shown as figure C-1.

1.2. Regional Economy

1.2.1. Area and Location. Region III is composed of six provinces in central Luzon: Bataan, Bulacan, Nueva Ecija, Pampanga, Tarlac, and Zambales. Olongapo in Zambales and Angeles in Pampanga are the principal cities in the Region. Region III has a total land area of 18,231 square kilometers (km ^{2}), or about 6 percent of the land area of the Philippines.

1.2.2. Land Use. Table C-1 presents land use information for Region III. Agriculture areas account for more than one-third of land use, and is mostly rice lands.

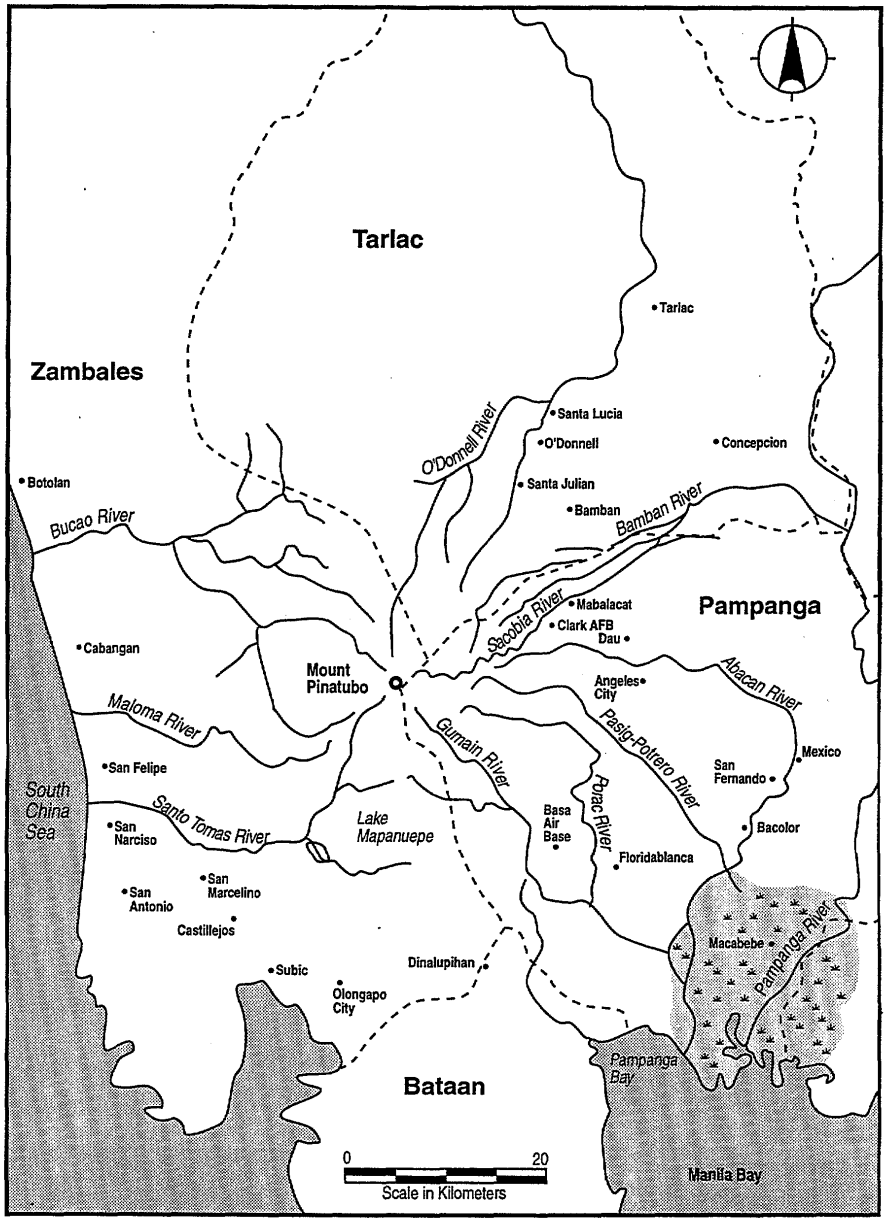
Table C-1 -- *Land Use and Vegetation, Region III, 1991*

	Region III Area (ha)	Percentage of Region
Agricultural Areas	635,345	34.88%
Rice	512,059	28.11%
Sugar Cane	53,125	2.92%
Grassland/Shrubland Areas	605,308	33.23%
Woodlands	361,573	19.85%
Wetlands	73,826	4.05%
Miscellaneous	145,543	7.99%
TOTAL	1,821,595	100.00%

Source: *Present Land Use and Vegetation Map*, Department of Agriculture, Region III, Bureau of Soils and Water Management.

[1]The Philippines is broken into Regions 1 through 12 plus Metropolitan Manila (the National Capital Region.)

Figure C-1 -- *Location of River Basins and Provinces Relative to Mount Pinatubo*



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1.2.3. Population. According to the 1990 Census of Population, Region III had a population of 6.2 million, a 29 percent increase over the 1980 Census. Pampanga is the most populous and densely populated province in Region III. Nueva Ecija is the largest in land area. The region's population is projected to increase to 7.5 million in 2000 and 8.7 million in 2010.[2] Table C-2 summarizes population data for the Philippines and Region III.

Table C-2 -- *Population and Population Density, Philippines and Region III*

	POP 1990	POP 1980	Annual Growth Rate	Land Area (sq. km.)	POP Density persons per km
Philippines	60,685	48,098	2.4%	300,000	202.3
Region III	6,199	4,803	2.6%	18,231	340.0
Bataan	426	323	2.8%	1,373	310.3
Bulacan	1,505	1,096	3.2%	2,625	573.3
Nueva Ecija	1,313	1,069	2.1%	5,284	248.5
Pampanga	1,533	1,182	2.6%	2,181	702.9
Tarlac	860	689	2.2%	3,053	281.7
Zambales	563	444	2.4%	3,714	151.6

Source: *1991 Philippine Statistical Yearbook*, National Statistical Coordination Board, Philippines, Table 1.1.

1.2.4. Production. The dominant industries in Region III are manufacturing, agriculture, and wholesale and retail trade, in that order, as measured by gross domestic product (GDP). The industrial and service sectors each make up about two-fifths of total regional output and agriculture accounts for the remaining one-fifth. Based on output, Region III has a larger industrial sector, a smaller service sector and a slightly smaller agricultural sector than the nation as a whole. Table C-3 summarizes this data.

Between 1980 and 1990 total output of goods and services increased at an average annual rate of 1.7 percent for the nation and 2.6 percent for Region III, inflation adjusted. Between 1990 and 1991 real GDP declined nearly one percent for the nation and nearly 7 percent for Region III. Per capita output declined about 0.7 percent per year for the nation and remained constant for the region from 1980 to 1990.

[2] 1991 Philippine Statistical Yearbook, National Statistical Coordination Board, Philippines, Table 1.8, p. 1-56.

Table C-3 -- *1991 Gross Domestic Product, National and Regional (million pesos, 1991 price level)*

Industry	NATIONAL Gross Domestic Product	NATIONAL Share of Total	REGION III Gross Domestic Product	REGION III Share of Total
Ag, Fishery and Forestry	**262,342**	**21%**	**24,540**	**22%**
Industrial Sector	**424,712**	**34%**	**43,765**	**40%**
Mining and Quarrying	17,504	1%	2,271	2%
Manufacturing	314,500	25%	31,660	29%
Construction	62,083	5%	7,611	7%
Electricity, gas, and water	30,625	2%	2,223	2%
Service Sector	**550,593**	**44%**	**42,231**	**38%**
Transportation	73,909	6%	6,560	6%
Trade	171,699	14%	17,008	15%
Finance & Real Estate	121,771	10%	8,392	8%
Private Services	97,388	8%	6,357	6%
Government Services	85,826	7%	3,914	4%
TOTAL	**1,237,647**		**110,536**	
Per Capita GDP	**20,395**		**17,831**	

Source: *1992 Philippine Statistical Yearbook*, Table 3.9 for national data. Economic and Social Statistics Office, National Statistical Coordination Board, table reprints, for regional data.

1.2.5. Employment. From the perspective of employment, the region is still quite dependent on agriculture but less so than the nation. For the region, the largest employment category is agriculture with 35 percent of total employment, followed by services, trade and manufacturing in that order. Agriculture for both the region and the nation is one of the least productive industries in terms of output per employee, accounting for 21 percent of gross domestic regional product despite the 35 percent share of total employment. Table C-4 provides an employment comparison of the region versus the nation.

Table C-4 -- *Employed Persons by Major Industry, 1991 (thousands)*

Major Industry Group	National	Share of Total	Region III	Share of Total
Agriculture	**10,403**	**46%**	**767**	**35%**
Industrial Sector	**3,686**	**16%**	**456**	**21%**
Mining	150	1%	9	0%
Manufacturing	2,391	11%	277	13%
Electricity, gas and water	99	0%	12	1%
Construction	1,046	5%	158	7%
Service Sector	**8,891**	**39%**	**974**	**44%**
Wholesale and retail trade	3,172	14%	317	14%
Transportation	1,143	5%	157	7%
Finance, insurance, real estate	451	2%	51	2%
Community, social and personal services	4,116	18%	447	20%
Other	9	0%	2	0%
TOTAL	**22,532**		**2,197**	

Source: *1992 Statistical Yearbook*, Table 11.4.

1.2.6. Income and Income Distribution. According to a 1991 income and expenditures survey[3], average family income in 1991 in Central Luzon was P75,960 (at current prices)[4], a real average annual increase of 2.4 percent since 1988.[5] Based on average family income, Region III is more prosperous than the nation as a whole (whose average family income is P65,563) and ranked only behind the National Capital Region. As with the entire nation, income is concentrated in the upper income groups with the top two deciles accounting for 48 percent of income in Region III. The 1991 regional Gini coefficient[6] is 0.4079.

For the nation as a whole, food accounts for 45 percent of expenditures in urban areas and 54 percent in rural areas. The bottom three deciles in the Philippines have expenditures exceeding income. It is assumed these broad patterns apply to Region III as well.

[3]"Highlights of the 1991 Family Inocme and Expenditures Survey," Preliminary Results, National Statistics Office.
[4]This is equivalent to just over $3,000 when evaluated at an exchange rate of 25 pesos to the U.S. dollar.
[5]The year of the previous income survey. See *1988 Family Income and Expenditures Survey*, National Statistic Office.
[6]The Gini coefficient measures income distribution. The smaller the fraction, the more evenly income is distributed.

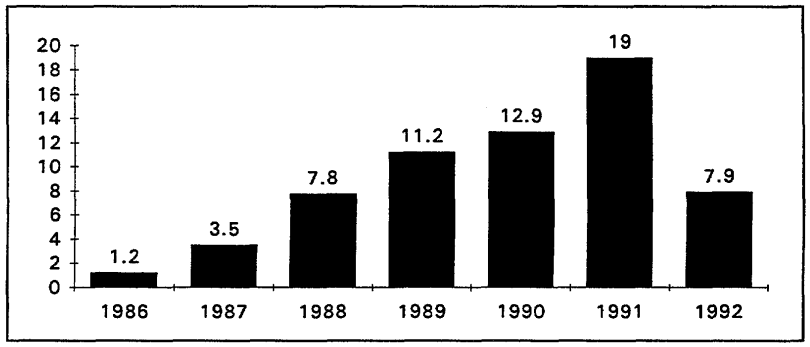
Table C-5 and figure C-2 provide price information for the region and nation. As shown in figure C-2, inflation peaked in 1991 before declining in 1992.

Table C-5 -- *Consumer Price Index, All Items, Region III and the Philippines (1988 = 100)*

	Consumer Price Index	
Year	Region III	Philippines
1986	89.7	88.6
1987	92.8	91.9
1988	100.0	100.0
1989	111.2	112.2
1990	125.5	128.1
1991	149.3	152.0
1992 *	161.1	167.0

* July 1992 value.
Source: *1992 Statistical Yearbook*, Tables 2.12 and 2.18, ibid.

Figure C-2 -- *Percent Change in Region III Consumer Price Index*



1.2.7. Agriculture. The Central Luzon Region in 1991 had 633,000 agricultural operators which was the third highest for any region in the Philippines. Total farm area was 613,000 hectares (ha), with an average farm area of 0.97 ha.[7] Region III is the top palay (unmilled rice) region in the country with 500,000 ha harvested and total production of 1.75 million metric tons.[8] This production represents about one-fifth of total Philippine palay production. Assuming an average price of P4,500 per metric ton, the value of rice production would be nearly P8 billion.

[7]"Number of Agricultural Operators and Area of Farms", Preliminary Results, 1991 Census of Agriculture, Special Release, Number 750, National Statics Office, February 1993. This report does not include agricultural operators of corporate farms and agricultural estates.
[8]National Economic Development Authority (NEDA), Region III, in Provincial Development Reports. NEDA cites Bureau of Agricultural Statistics.

Sugarcane is also an important crop in the region. In 1990 it is estimated that production was about 2,100,000 metric tons, area harvested was 26,500 ha and value of the crop was about P1.25 billion. Region III produces about one-eighth of the nation's sugarcane. About 280,000 ha of land was irrigated in 1990, but irrigation systems serving 54,000 ha were damaged by the eruption and after-effects.[9]

1.3 Provincial Profiles

1.3.1 Introduction. This section of the socioeconomic profile summarizes conditions for three provinces, Pampanga, Tarlac and Zambales, in which the eight major river basins are located and where nearly all the flood and sediment damage is expected to occur.[10]

1.3.2. Location and Land Use. Mt. Pinatubo is at the approximate center of the land area of Pampanga, Tarlac and Zambales. Pampanga is generally located to the southeast of the volcano, Tarlac to the northeast and Zambales to the west. (See figure C-1 for the location of the provinces).

Pampanga has a land area of 2,120 square kilometers. Much of the province's land is flat plain, where land is devoted to rice and sugarcane cultivation. Fishponds are concentrated in the very flat delta area north of Pampanga Bay. Agricultural areas including fishponds account for nearly two-thirds of land use. In the northwestern part of the province leading to Mt. Pinatubo, rolling and mountainous terrain prevails. Table C-6 presents land use information for the provinces.

Tarlac has two distinct geographical areas, one the extensive alluvial plain in the northern part and the other the hills and mountains of the Zambales mountain ranges in west and northwest parts of the province. Rice, corn and sugarcane account for 94 percent of agricultural areas and about 45 percent of the province's 3,050 square kilometers is devoted to agriculture.

Zambales is located at the western coast of Central Luzon and has irregular terrain. A major highway runs through plains and valleys along the western part of the province where most of the population is located. A large part of the province is lightly populated mountainous areas. Only about 12 percent of the 3,714 square kilometers in Zambales is agricultural and about three-fourths of that is paddy rice.

1.3.3 Population. As shown on table C-2, Pampanga is the most populous province with a 1990 total population of 1.53 million, followed by Tarlac with 860,000 and Zambales at 563,000. Between 1980 and 1990, population grew from 2.2 percent to 2.6 percent per year for the provinces. Figure C-3 shows Pampanga and Zambales to be relatively urban and Tarlac to be predominately rural. The population is relatively young with persons 14 years and younger accounting for between 37 percent (for Zambales) and 41 percent (Pampanga) of household population.

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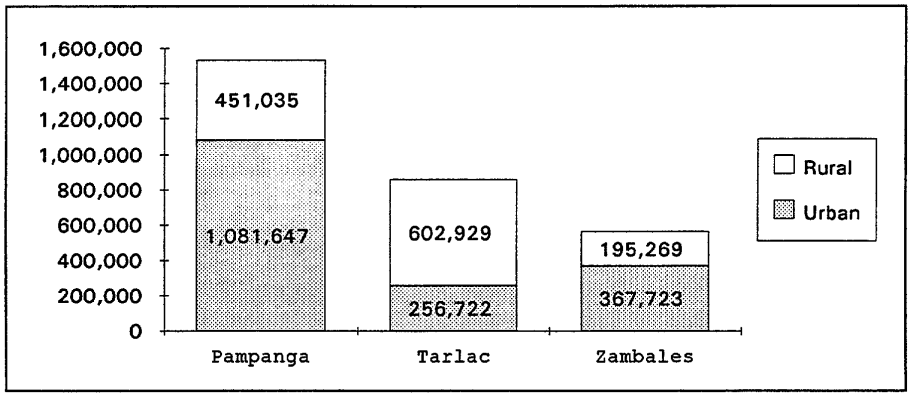
[9]Technical Scientific Committee, 1992.

[10]Small areas of Bataan (in the vicinity of Dinalupihan) and one barangay in Nueva Ecija are in the hazard zones. The Tarlac River also runs through Pangasinan and it is possible flooding and sediment could occur there.

Table C-6 -- *Land Use and Vegetation in Pampanga, Tarlac, and Zambales (area in hectares for 1991)*

	Pampanga Area (ha)	Tarlac Area (ha)	Zambales Area (ha)	Combined Percent
Agricultural Areas	**104,421**	**137,400**	**44,994**	**32.1%**
Rice--Irrigated	42,939	46,150	10 ,049	11.1%
Rice--Non-Irrigated	22,394	52,620	22,050	10.8%
Corn	2,335	9,470	245	1.3%
Sugar Cane	29,796	20,887		5.7%
Pyroclastic Flow Deposition	3,939	5,754		1.5%
Grassland/Shrubland Areas	**19,579**	**84,723**	**209,115**	**35.0%**
Grasses	9,878	64,884	124,507	22.3%
Shrubs & Other	1,729	12,151	54,199	7.6%
Pasture Land			9,022	1.0%
Pyroclastic Flow Deposition	7,972	7,688	21,387	4.1%
Woodlands	**15,884**	**54,252**	**98,934**	**18.9%**
Forest	8,063	46,711	91,952	16.4%
Pyroclastic Flow Deposition	7,821	7,541	6,982	2.5%
Wetlands	**42,341**	**352**	**3,191**	**5.1%**
Swamp/Marsh	1,520	352	292	0.2%
Fishpond	34,984		831	4.0%
Nipa	4,177		1,365	0.6%
Pyroclastic Flow Deposition	1,660			0.2%
Miscellaneous	35,843	28,618	15,206	8.9%
Built-Up Areas	28,999	20,842	3,978	6.0%
Other	6,017	7,195	11,149	2.7%
Pyroclastic Flow Deposition	827	581	79	0.2%
TOTAL	**218,068**	**305,345**	**371,440**	**100.0%**

Figure C-3 -- *Urban and Rural Population, 1990*



1.3.4. Households and Housing. Table C-7 gives total households and persons per household for the three provinces.

Table C-7 -- *Households and Persons per Household, 1990*

Province	Total Households	Persons per Household
Pampanga	268,547	5.7
Tarlac	159,332	5.4
Zambales	115,643	4.9
TOTAL	543,522	

Source: National Statistics Office, 1990 Census of Population and Housing, Table 2.

Construction materials for residential and other buildings in these three provinces vary considerably with 69 percent of roofs constructed with galvanized iron or aluminum and 22 percent of plant material such as cogon or nipa[11]. About one-third of walls are built in concrete/brick/stone, 16 percent in wood, 23 percent a combination of the two materials, and 26 percent in plant material other than wood.[12] House sizes vary widely and table C-8 shows the percent of occupied housing units in each floor size class. The median house size appears to be less than 30 square meters (about 325 square feet).

[11]Thatches made from coarse tall cogon grass or nipa palm leaves.
[12]bamboo/sawali/cogon/nipa. This data is from Published Housing Table 3, 1990 Census of Population and Housing, National Statistics Office.

Table C-8 -- *Occupied Housing Units by Floor Area, Percent Shares*

Floor Area (m)	Pampanga	Tarlac	Zambales	Total
<10	13%	16%	19%	15%
10-19	20%	24%	25%	23%
20-29	18%	19%	20%	19%
30-49	16%	18%	15%	16%
50-69	11%	9%	8%	10%
70-89	7%	5%	4%	6%
90-119	6%	3%	3%	4%
120 and over	9%	5%	5%	7%

Source: Published Housing Table 6B, Census of Population and Housing, 1990, National Statistics Office.

Most houses are owner occupied. For example, in Pampanga 86 percent are owner occupied with only 6 percent rented, the remainder being occupied for free with or without consent of owner. About 91 percent of households in Pampanga are in "single houses", with small fractions for duplexes and multi-unit buildings. Based on sample data for Pampanga, median monthly rent in 1990 was about P500 (current prices).[13] Table C-9 summarizes owner occupancy rates and single family unit shares for the three provinces.

Table C-9 -- *Owner Occupancy Rates and Single Unit Shares*

	Pampanga	Tarlac	Zambales
Percent Owner Occupied	86.6	94.7	70.3
Percent Single Unit	91.6	97.0	75.7

Source: Published Housing Table 7A, Census of Population and Housing, National Statistics Office.

1.3.5. Employment. Table C-10 summarizes 1991 employment and labor force data for the three provinces. Participation rates are fairly consistent across the provinces, with Pampanga's the highest and Zambales' the lowest. Based on percent of employment in non-agriculture, Pampanga is the least agrarian and Tarlac the most agrarian. Pampanga also has the largest industrial sector, measured either in employment or as a share of total employment. At the time of writing this report, 1992 data was not available.

[13]Published table 11, HSG11-54.TBL, Census of Population and Housing, National Statistics Office.

Table C-10 -- *Labor Force and Employment, Pampanga, Tarlac, and Zambales, 1991*

	Pampanga	Tarlac	Zambales
Population > 15 Years	890,000	534,000	382,000
Labor force	575,000	327,000	206,000
Labor force participation rate (percent)	64.6	61.2	53.9
Unemployment rate (percent)	11.3	2.4	15.5
Underemployment rate (percent)	11.5	27.2	14.1
Agriculture employment (percent of total)	102,000 (20.0)	173,000 (54.4)	44,000 (25.3)
Non-agricultural employment (percent of total)	409,000 (80.2)	145,000 (45.6)	130,000 (74.7)
Industry employment (percent of non-ag. emp.)	137,000 (33.5)	33,000 (22.8)	32,000 (24.6)
Service employment (percent of non-ag. emp.)	270,000 (66.0)	114,000 (78.6)	101,000 (77.7)

Source: *Geographic Information System*, Final Report, Volume II, Mount Pinatubo Emergency Project Management Office, Department of Public Works and Highways, August 1992.

1.3.6 Income and Income Distribution. Table C-11 provides 1988 income data for the three provinces. Shown are mean and median family income and per capita income. Pampanga has the highest family income whereas Zambales has the highest per capita income. The difference is explained because Zambales has the smallest family size. Provincial data from the 1991 Income and Expenditures Survey are not yet available.

Table C-11 -- *1988 Family and Per Capita Income (pesos)*

	Pampanga	Tarlac	Zambales	Region III	Philippines
At current prices					
Mean	55,384	34,566	49,884	46,855	40,408
Median	42,302	27,485	29,395	34,867	26,694
Per Capita Income	8,860	6,085	9,396	8,268	6,699
At 1991 prices					
Mean	83,630	52,195	75,325	70,751	61,420
Median	63,876	41,502	44,386	52,649	40,575
Per Capita Income	13,379	9,188	14,188	12,485	10,182

Source: Provincial Development Reports for Pampanga, Tarlac and Zambales, Table 22. *Highlights of 1991 Income survey*, ibid.

Income distribution is more evenly distributed in Pampanga than in Tarlac or Zambales, Region III, or the nation as evidenced by its gini coefficient (see footnote 6) of 0.31 in 1988 compared 0.36 for the region and 0.43 for the nation. Table C-12 summarizes income distribution data for the three provinces.

Table C-12 -- *Family Income by Income Class for Pampanga, Tarlac and Zambales*

	Pampanga		Tarlac		Zambales	
Income (pesos)	% Dist. of Families	% Dist. of Income	% Dist. of Families	% Dist. of Income	% Dist. of Families	% Dist. of Income
Under 10,000	0.62	0.10	5.22	1.30	4.34	0.61
10,000-15,000	2.88	0.71	12.77	4.82	11.44	2.92
15,000-20,000	5.85	1.91	14.01	7.14	12.37	4.47
20,000-30,000	18.58	8.52	24.04	17.34	23.25	11.14
30,000-40,000	16.71	10.46	16.76	17.04	15.68	10.79
40,000-60,000	23.28	20.36	16.48	23.40	12.36	12.06
60,000+	32.08	57.94	10.71	28.96	20.56	58.00
Gini Coef., 1988	0.31		0.35		0.46	
Gini Coef., 1985	0.35		0.35		0.48	

Source: *Provincial Development Reports*, Ibid.

1.3.7 Agriculture. Because agriculture is such an important sector in the region and because flood and sediment damages to agriculture are potentially significant, this section discusses agriculture in more detail at the provincial level. Most of the data presented is extracted from the *Provincial Development Reports*.

Pampanga: Rice and sugarcane are the principal crops, accounting for 60 percent and 23 percent, respectively, of total provincial area devoted to crops. Other important crops include banana, mango and eggplant. Over the period 1986 to 1991, rice production peaked in 1990 and declined in 1991, possibly because of the eruption. On the other hand, sugarcane production, which had been erratic over this period, peaked in 1991. Fish production followed the same pattern as rice.

Tarlac: Rice and sugarcane are the principal crops. Rice yields (2.5 metric tons per ha), however, are considerably below that of Pampanga (3.9 metric tons per ha). Sugarcane production has been steadily growing, nearly doubling over the period of 1986 to 1991.

Zambales: Rice is the major crop in Zambales, despite a significant drop in the area harvested in 1991, possibly attributable to the eruption. Zambales with its vast coastline has large fish production accounting for 14 to 17 percent of regional fish output.

Table C-13 summarizes agricultural production for the three provinces for the year 1991. Similarly, livestock inventories are shown in table C-14. The trend of rice and sugarcane production over 1986 to 1991 in the provinces is shown in Figure C-4. Rice production has been steady and sugarcane output has recently increased.

Table C-13 – *Crop and Fish Production (production in metric tons)*

Crop	Pampanga	Tarlac	Zambales
Rice			
Production	167,449	248,662	45,426
Area Harvested (ha)	42,800	97,990	16,760
Yield (MT/ha)	3.91	2.54	2.71
Sugarcane	927,529	1,320,471	0
Corn	1,579	6,550	241
Banana	6,491	837	997
Mango	4,338	3,504	5,807
Eggplant	2,480	3,781	27
Came	1,555	12,875	637
Fish	44,890	2,365	24,499

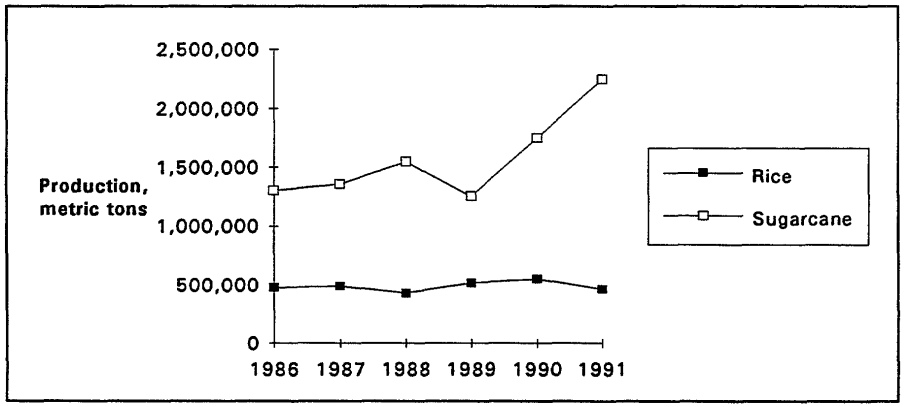
Source: *Provincial Development Reports*, ibid, Tables 8, 9 and 10.

Table C-14 -- *Livestock Inventory (number of head)*

Type	Pampanga	Tarlac	Zambales
Carabao	30,224	51,772	23,810
Cattle	4,505	28,945	12,622
Hog	166,991	154,230	41,006
Chicken	2,597,877	930,790	307,810

Source: *Provincial Development Reports*, ibid, Table 11.

Figure C-4 -- *Rice and Sugarcane Production, 1986 to 1991*



1.3.8. Social. Crude birth rates have shown different patterns across the provinces. Pampanga's rate has increased since 1986 from 18.7 per 1,000 to 28.5 in 1991, whereas Zambales' has fallen from 26.7 to 23.2 per 1,000. All provinces report dramatic declines in the infant mortality rate since 1986.[14] Life expectancy for someone born in 1990 is 64.6 years for the Philippines.[15]

[14] *Provincial Development Reports*, ibid.
[15] *1992 Statistical Yearbook*, Table 9.10, ibid.

Education is nearly comprehensive at the elementary level, with the literacy rate at about 97 percent for each province. About 30 to 40 percent of the adult population (greater than 20 years of age) has at least some high school education.[16]

Table C-15 also provides data about household conveniences in the three provinces.

Table C-15 -- *Social Statistics for Pampanga, Tarlac and Zambales*

	Pampanga	Tarlac	Zambales
Vital Statistics: 1991			
Birth rate (per 1000)	28.5	20.5	23.2
Death rate (per 1000)	2.9	3.2	4.0
Infant mortality rate	7.8	11.1	17.0
Number of Health Centers	40	40	16
Population per center	33,200	29,285	23,058
Education			
Literacy rate (1991)	97.6	96.6	97.0
Percent of Households with:			
Electric lighting	88	64	73
Water sealed toilet	70	69	74
Radio	72	65	64
Television	65	45	46
Refrigerator	29	17	33
Telephone	4	2	2
Motor vehicle	12	10	8

Source: *Provincial Development Reports* for vital statistics, health centers, and education. Other data is from Published Housing Tables 25, 28 and 30, Census of Population and Housing, 1990, National Statistics Office.

1.3.9 Infrastructure. Information presented below is extracted entirely from the *Provincial Development Reports* for the three provinces.[17] The information is summarized in Table C-16.

Roads and Bridges. Pampanga with 2,259 km of roads has the highest road density of the three provinces, 1.07 km of road per km^2 of land area. Twelve percent are national roads, 14 percent provincial, 5 percent municipal, and 69 percent barangay. Overall, less than 20 percent of roads are paved; however, three-fourths of national roads are paved. The North Expressway is the most important transportation facility, linking Manila with the population

[16]Based on Published Population Table 9 for the three provinces, Census of Population, 1990, National Statistics Office.
[17]The source for this information are the *Provincial Development Reports*, ibid.

centers of Pampanga and terminating just south of Tarlac. Lack of funds has prevented extension into Tarlac although rights-of-way have apparently been secured.

Tarlac has an estimated 2,440 km of roads with a density of 0.80 km of road per km^2 of land area. The fractions paved are similar to those for Pampanga. Zambales, the largest province, has the lowest road density (0.32) and the least extensive road network (1,159 km.) One main highway cuts across the province on a north-south axis, paralleling the coastline for the most part.

Postal, telephone and telegraph. Reflecting its higher population density, Pampanga has the highest population-post office ratio, about 45,000:1 whereas as Zambales has the smallest at 27,000:1. On average, about 60 to 70 percent of municipalities are served by telegraph. Telephone density (phone lines per 100 population) varies from 0.22 to 0.63 which indicates there is a tremendous market potential for telephone service.

Water Supply. Potable water service to households ranges from 80 percent in Pampanga to 62 percent in Tarlac. A third of municipalities in Pampanga have formed water districts, whereas over half in Tarlac and Zambales have done so.

Irrigation. Pampanga in 1990 had four National Irrigation Systems (NIS), 93 gravity irrigation systems and 9,763 pump systems, together providing water to 58,908 ha. of cropland. This represents 71 percent of irrigable area. Tarlac in 1990 had three NIS and 885 Communal Irrigation Systems (CIS), serving 39,000 ha or 55 percent of potential irrigable area. The NIS and CIS systems served about 53 percent of potential irrigable area in Zambales, or 16,000 ha.

Power. As of 1991, Pampanga is fully served by four electric cooperatives (PELCO I, II, III) and Pampanga Rural Electric Services Cooperative. Three municipalities are served by MERALCO. Ninety-three percent of households have electric service. In Tarlac, all municipalities are served, 86 percent of barangays have power available, and 75 percent of households are connected. In Zambales, all 13 municipalities are energized, with 97 of the barangays served. Seven out of eight households (88 percent) have electric service.

Flood Control. Pampanga has an extensive network of flood control, river control and drainage systems. The Pampanga *Provincial Development Report* states there are:

> "extensive flood control facilities with 1,800 m of earth dikes, 434 m of spur/rubble dikes, and 3,787 m of revetment structures as of 1990....Drainage facilities ...consist of 18,190 m of lateral structures, 5,611 drainage canals and 35,330 m of esteros and waterways."[18]

An undetermined, but probably large, fraction of these assets was damaged in the aftermath of the eruption and flooding in 1991-1992, but some are currently being rebuilt.

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[18]P. 12, ibid.

In Zambales, there are 33 flood control facilities with an aggregate length of 15,995 meters. Drainage facilities are comprised of lateral structures with a total length of 3,795 meters and two flood gates.

Table C-16 -- *Infrastructure Development*

Subsector/Key Indicatator	Pampanga	Tarlac	Zambales
Transportation			
- Road Length (km)	2259	2440	1159
- Road Density (km/km^2)	1.07	0.80	0.32
- Percent Paved, All Roads	17.5	15.2	24.3
- Percent Paved, National Roads	75.2	85.3	79.9
- Motor Vehicle Registration (number/1000 population)	20.0	31.4	31.5
Communication			
- Post Office to Population Ratio	1:45,453	1:35,073	1:26,925
- Municipalities with Public Telegraph Service (%)			
a. Line Telegraph	76.2	33.3	33.8
b. Radio Telegraph	23.8	61.1	69.2
- Telephone Density (lines/100 pop)	0.63	0.34	0.22
Water Resources			
- Potable Water Supply Coverage (%)	80.4	61.8	63.6
- Municipalities with Water District (%)	33.0	55.6	53.8
- Irrigation Coverage (%)	70.7	55.2	52.9
- Percent actually irrigated out of the total service area			
a. Wet Season	29.7	73.4	52.9
b. Dry Season	17.3	54.5	19.0
Power			
- Electric Coverage			
a. Municipality	100%	100%	100%
b. Barangay	99.5%	86.4%	96.9%
c. Household	93.3%	75.4%	88.1%

1.3.10. Relocation. Development of resettlement sites was initiated for those who have lost their homes or farms because of the eruption. This resettlement program focuses on two groups: members of the Aeta hill tribes, often called highlanders and the displaced population that reside on the plains below Mount Pinatubo, often called lowlanders.

In general, resettlement sites have basic amenities and services, and for lowlanders are relatively close to town centers for higher level services. Resettlement sites may also have productivity centers to offer training and opportunities for employment in light manufacturing industries. New settlements for Aetas are rural, higher in elevation, and emphasize agricultural and natural resource-based livelihoods.

There are 19 resettlement sites located in Tarlac, Zambales, and Pampanga - 10 are upland sites and 9 are lowland sites. Six resettlement sites outside of the affected areas were established by local government units and include three Aeta sites in Palayan City and agricultural sites in Mainang, Tarlac; Sablayan, Mindoro; and Talakag, Bukidnon. The Mount Pinatubo Commission (MPC) also is establishing two additional lowland sites in Pampanga, at Bulaon in Bacolor and Mauwaque in Mabalacat.

The MPC's resettlement activities include land acquisition, housing, civil works, water systems, electrification, school buildings, and community facilities. It provides grants for each family in the form of a core housing loan equivalent to about P20,000 at an interest rate of 6 percent for a repayment period of 25 years. As of December 1993, the MPC has built a total of 12,834 houses, 11,799 of which are occupied, and has completed 81 community facilities, 194 school buildings, installed 270 km of electrical lines, and repaired 22 housing units. About 8,000 and 5,700 families have been resettled in the lowlands and highlands, respectively, by the end of 1993.

2. SEDIMENTATION, FLOOD PROBLEMS AND DAMAGES

2.1 Nature of the Damaging Events

This section provides information on the types of flood and sedimentation problems the residents of Central Luzon are experiencing.[18] Mudflows (lahars), ponding, and erosion are discussed in turn.

2.1.1 Lahar and Sediment Deposition. Lahar refers to a mixture of mud, water and solid particles composed of volcanic debris. The solid component of the mixture consists of pumice and ash from past and recent eruptions and may make up from 20 to 80 percent of the mixture. Rainy periods can activate movement of lahars, sending them cascading down the mountains and river channels.

The thicker lahars, those with as much as 80 percent solids and densities twice that of water, are quite capable of moving boulders, destroying bridges and gabions, and eroding river banks at dozens of meters per day. In this report, thicker lahars are sometimes called mudflows. Mudflow hazard zones are generally limited to the upper end of the alluvial fan in the various river basins.

More dilute lahars can also erode banks and in some cases cause more bank erosion damage than a thick one, by running along the side of a river channel and quickly eroding and undercutting banks. Mudflows, on the other hands, may be more likely to cause vertical erosion in the streambed.

Dilute lahars occur in zones designated as flooding and sedimentation areas in this study. These zones are usually downstream of the mudflow zones in gently sloped land, beyond the point where much of the sediment in a thick lahar has already been deposited.

Thick and dilute lahars can differ in terms of damages to lowland areas. Everything else held constant, dense lahars move at a higher velocity because less energy is spent on turbulence. If it overflows the river banks, 1 to 2 or more meters of sediment can be quickly deposited and existing structures can be bulldozed within a few hundred meters of the channel. Beyond that, as solid materials are deposited and velocity dimishes, the lahar will flow around obstacles such as structure walls and even sugarcane stalks rather than mowing them down.

Diluted lahars travel at a slower rate. When such lahars reach relatively flat land, they begin to deposit sediments. Eventually, so much of the solid particulates are dropped out that the lahar is virtually indistinquishable from a typical flood except for the thin layer of sediment

[18] Some of the following description of lahar is taken from Bautista, "Lahar and Flood: The Bane of Mt. Pinatubo's Lowland Victims," Table 2-5, p. 28, *In the Shadow of the Lingering Mt. Pinatubo Disaster*, CSSP Publications, 1993, Chapter 2.

it leaves behind. Water levels associated with dilute lahars outside of river channels will usually be quite shallow and repeated events or a single event of long duration would be required to leave large sediment deposits.

2.1.2 Ponding. Because volcanic deposits have clogged waterways, some lowland areas not directly affected by lahars are subject to backflooding and siltation. This type of flooding is called ponding. For the most part, ponding is confined to the delta area of Pampanga. Sediment transported from the mountain to the delta by the Abacan, Pasig-Potrero and Porac-Gumain rivers has blocked drainage from the delta. During the rainy season, runoff exceeds the capacity of the drainage network creating flood depths and durations that exceed pre-eruption levels.

2.1.3 Erosion. The lahar problem has exacerbated the problem with erosion. Streambeds and banks are subject to levels of erosion that exceed what would occur with water alone. Bank erosion can lead to local damages to property. Because bank erosion increases the channel area, previous bank erosion can reduce the potential for future erosion assuming a wider channel reduces the probability for flows along the channel bank. This may be the case in Angeles City through which the Abacan River flows.

2.2 Lahar and Flood Hazard Areas

Although the lahar and flood hazard covers much of the entire Central Luzon region, the preponderance of damage centers on Pampanga, Tarlac and Zambales. Only small portions of Bataan, Bulacan and Nueva Ecija will likely be affected.

A number of maps indicating the hazard zones have been prepared, notably by the Philippine Institute of Vocanology and Siesmology (PHIVOLCS) and the USACE. The USACE maps indicate four types of hazard zones: mudflows, flooding and sedimentation, flooding without sedimentation, and ponding. Erosion areas are not indicated on USACE maps.

Table C-17 lists the municipalities affected or threatened by mudflows, flooding and ponding. These municipalities were identified by overlaying Geographic Information System generated maps on the USACE hazard maps. Municipalities are included in this list even if only a small portion may be at risk.[19]

2.3 Flooding and Sedimentation Prior to Eruption: Pasig-Potrero Basin

Available information on flooding and sedimentation in the pre-eruption period is limited. A Japan International Cooperation Agency (JICA) report, however, provides information about several events in the 1960's and 1970's in the Pasig-Potrero basin.[20]

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[19]Similar lists are found in Bautista, "Lahar and Flood: The Bane of Mt. Pinatubo's Lowland Victims," Table 2-5, p. 28, *In the Shadow of the Lingering Mt. Pinatubo Disaster*, CSSP Publications, 1993, Ch 2, pp. 25-26.
[20]*Planning Report on the Pasig-Potrero River Flood Control and Sabo Project*, Sesptember 1978.

Table C-17 -- *Municipalities Threatened by Mudflows, Flooding and Erosion*

Basin	Province	Municipality	Type of Hazard
Bucao	Zambales	Botolan	F&S
		Iba	F&S
Maloma	Zambales	Cambangan	F&S
		San Felipe	F&S
Sto. Tomas	Zambales	Castillejos	F&S
		San Antonio	F&S
		San Felipe	F&S
		San Marcelino	F&S
		San Narciso	F&S
Abacan	Pampanga	Angeles City	Erosion
		Mexico	F&S
		San Fernando	F&S
		San Luis	F&S
		San Simon	F&S
		Santa Ana	F&S
O'Donnell	Tarlac	Capas	F&S
		Conception	F&S
		Gerona	F&S
		Tarlac	F&S
Pasig/ Potrero	Pampanga	Apalit	Ponding
		Bacolor	F&S, Mudflows
		Guagua	F&S
		Minalin	Ponding
		Porac	Mudflows, Erosion
		San Fernando	Ponding
		San Luis	F&S
		San Simon	Ponding
		Sta. Rita	F&S, Mudflows
		Sto. Tomas	Ponding
Porac/ Gumain	Bataan	Dinalupihan	F&S
		Hermosa	F&S
	Pampanga	Floridablanca	F&S, Mudflows, Erosion
		Lubao	F&S, Ponding
		Macabebe	Ponding
		Masantol	Ponding
		Porac	F&S, Mudflows, Erosion
		Sasmuan	Ponding
Sacobia	Pampanga	Mabalacat	F&S, Mudflows
		Magalang	F&S
	Tarlac	Bamban	F&S, Mudflows
		Capas	F&S
		Concepcion	F&S

Note: F&S stands for flooding and sedimentation. Erosion potential was identified by PHIVOLCS.

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The Pasig-Potrero basin has experienced repeated flooding and deposition that resulted in extensive damages to agriculture and structures. Floods in 1966, 1972, 1974, 1976 and 1977 hit areas ranging from Balas down to the Guagua and Bacolor urban areas, terminating at the Guagua River at the downstream end. JICA estimates the submerged areas in these floods to be 5,100 ha, 5,500 ha, 4,500 ha, 3,000 ha and 3,100 ha, respectively.

The 1972 flood was the worst in that period, causing extensive agricultural, structural and transportation damages. In only the 1972 flood did water level exceed the first floor elevation of residential structures and cause structural damage to houses.

Of the 5,550 ha flooded in the 1972 storm, nearly 5,000 ha had inundation lasting 7 days or more with water depths of 45 centimeters (cm) or more. About 1,770 ha were covered by deposits of 50 cm of sand. Generally, the sand deposition areas were in the upstream part of the damage area and the inundation in the downstream area. Most of the damages occurred to agriculture, namely low-lying rice production. It is estimated that about 4,800 houses were damaged by water depths ranging from 50 to 150 cm. Table C-18 summarizes the damages.

Table C-18 -- *1972 Flood Damage, Pasig-Potrero River*
(1991 price level, thousands of pesos)

Damage Category	Damages
Agricultural Production	
Crops	50,431
Agricultural Facilities	5,345
Land Arability and Productivity	29,515
Fishpond Culture	5,758
Subtotal	91,049
Houses	20,387
Transportation facilities	13,363
TOTAL DAMAGES	124,799

Source: *Planning Report on the Pasig-Potrero River Flood Control and Sabo Project*, Japan International Cooperation Agency, September 1978.

2.4 Lahar and Flood Events Since 1991

2.4.1 1991 Rainy Season. Immediately after the eruption, lahar flows began occurring. Rains associated with a typhoon that came on-shore at that time initiated flows down the Sacobia-Bamban system and along the northwest side of the volcano. Some barangays in Conception, Tarlac were hit as early as June 14, 1991. Between June 12 and

September 10, 1991 there were about 200 lahar flows down the major river channels. The Sacobia-Bamban and Abacan experienced the highest rate of lahar flow with an average of one a day, and sometimes 3 to 5 times daily on rainy days.

Estimates presented in Bautista's article[21] indicate that 43 percent of the PHIVOLCS lahar hazard zones were affected by lahar by the end of October 1991. Table C-19 presents lahar affected areas as of 1991 and 1992. The table shows that the area hit in the Sacobia-Bamban basin (81 km^2) exceeded that of any other basin. Large areas of sedimentation also occurred in the Bucao, Santo Tomas and Pasig-Potrero basins.

Other basins, notably the Maloma, had large percentages of the lahar hazard areas impacted. For example, lahar completely covered the Maloma River hazard zone and impacted more than 90 percent of the zone in the Porac-Gumain basin.

Table C-19 -- *1991 and 1992 Lahar Affected Areas* (in km^2)

Basin	Lahar Hazard Zone Area (PHIVOLCS)	1991 Lahar Affected Areas	1992 Lahar Affected Areas
Bucao-Balin Baquero	86	54	na
Maloma	17	18	0
Santo Tomas	126	46	na
Abacan	41	29	0
Pasig-Potrero	100	39	11
Porac-Gumain	34	31	0
O'Donnell	115	33	na
Sacobia-Bamban	251	81	23

Data taken from Bautista, *Lahar and Flood: The Bane of Mt. Pinatubo's Lowland Victims*, Table 2-5, p. 28, *In the Shadow of the Lingering Mt. Pinatubo Disaster*, CSSP Publications, 1993

2.4.2 1992 Rainy Season. Lahar activity moderated in 1992 with no flows reported in the Maloma, Abacan, and Porac-Gumain basins. Damaging flows did, however, course through the Sacobia, Pasig-Potrero and Santo Tomas basins. In late August and early September 1992 mudflows exited normal channels of the Sacobia River and buried parts of Mabalacat, Pampanga and Bamban, Tarlac under 1 to 3 meters of sediment. Low areas in Barangays Dolores, Tabun, Malonzo and Sapang Balen were covered as well as 2 km of the MacArthur Highway. The total area buried was 1,700 ha.[22]

At Bamban, the new lahar deposit blocked the Marimla River, causing floodwaters to flow northward out of the river channel, inundating low-lying areas of Bamban poblacion and Barangays Banaba, San Pedro, Lourdes, Calabasa, Anupul and Pacalcal. About 1,900 ha

[21] Ibid, p. 27.
[22] Department of Public Works & Highways, *Mt. Pinatubo Rehabilitation Plan & Program*, 15 Sep. 1992, p. 4

were flooded. At the Pasig-Potrero River, also in the late August and early September 1992, the lahar overtopped the west bank levee at Barangay Mitla, Porac and flowed in a southeastward direction to Barangay Balas in Bacolor. An area about 5 km in length and 1 to 2 km in width received about 1 to 3 meters of sediment.

The muncipalities of Dinalupihan, Hermosa and Orani in Bataan experienced floods of long duration in 1992. Bautista quotes newpaper accounts on the week of August 20, 1992 in which residents of Hermosa were evacuated from floodwaters of up to 2 meters in depth. The flooding in Hermosa and Orani was probably due siltation in the Pasig-Potrero and Porac-Gumain rivers clogging the delta drainages. The floods in Dinalupihan, however, were caused by lahar deposits in the Gumain system which led to flow diversion in the Caulaman River and the eventual inundation of parts of the municipality.

In Zambales, significant lahar flows occurred along the Santo Tomas in August and September of 1992. The later flows buried sitio Dalanawan in the municipality of San Marcelino, which is about 5 km downstream of the intersecton of the Marella and Mapanuepe rivers. About 15 to 20 meters of the south river bank were eroded away in Barangay San Rafael, San Marcelino. Poonbato, a barangay along the Bucao River which had been severely hit in 1991, received another 4 to 5 meters of debris.

2.4.3 1993 Rainy Season. There have been several instances of flooding and lahars in the 1993 rainy season. Examples of reported events are provided below.

- Early July 1993. On July 2, 1993, lahars hit Barangay Sapang Balen along the Bamban River with a peak flow of 1 meter and leaving deposits of 0.5 meter. The community evacuated as a result of this flow[23] forcing 103 families to leave.[24]

- Early August 1993. In early August 1993, floods swept through "wide areas in western Pampanga and eastern Bataan due to the destruction by lahar of the 2.3 kilometer protective dike in the Porac-Gumain River".[25] The municipalities of Lubao, Pampanga and Dinalupihan and Hermosa in Bataan were reportedly flooded.

On August 6, 1993 lahar penetrated areas in Barangays San Isidro and San Juan in the municipality of Santa Rita in Pampanga. Newpaper reports said a 4 foot-high lahar flow left the Pasig-Potrero River at an unfinished portion of a 7 km dike along the Pasig-Potrero River and covered 10 ha of ricelands in San Basilio. A road linking the barangay to the town proper was also buried.

In Zambales, hundreds of residents of Barangay San Rafael evacuated their homes in early August 1993 after strong flood waters along the Santo Tomas River flowed towards their community. San Rafael is at particular risk, sitting between the river channel and protective dikes.

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[23] *Lahar Update,* 2-3 July 1993, PHIVOLCS Lahar Monitoring Group.
[24] Joe Rondal, July 13, 1993, Louis Berger, memo to USACE.
[25] Manila Bulletin, Monday, Aug. 2, 1993.

• Typhoon Rubing, mid-August 1993. On August 18 to 20, 1993, typhoon Rubing generated mudflows that caused 5 to 7 meters of deposition in the channel upstream of Mancatian on the Pasig, resulting in the overtopping of the levee and 1 to 2 meters of deposition in Mancatian. Estimates are that 150 homes were destroyed, mostly on the left bank. The remaining bridge at Mancatian was left with 1 meter of clearance, and muddy streamflow threatened to erode the right bank levee downstream of Mancatian. Continuous lahar flows reached Balas and San Isidro, Santa Rita. In the Bamban basin, lahar destoyed about 2 km of earth and conrete dike that protects the north part of Masgalang and the southern barangays of Conception.

On the Santo Tomas River, flooding was reported around Castillejos, San Marcelino and San Antonio as a result of levee failure. Lahar deposits blocked clear water flows from Lake Mapanuepe which ran along the levee, causing the flow to overtop the levee. Once overtopped, the levee failed rapidly leading to the flooding. The flooding passed through the Luzon Agricultural College and then destroyed the north approach of Camachile Bridge on Highway 7.

As of early September 1993, the Department of Social Welfare and Development had 19 evacuation centers in Pampanga, Tarlac and Zambales with a total of 2,656 families. The total amount extended for the relief for these families was P2,189,194 or an average of P824 per family. Typhoon damage to infrastructure was estimated at P47,000,000 and to crops at P73,677,090.

• Typhoon Kadiang, 4-7 October 1993. Heavy rains from typhoon Kadiang triggered flooding and lahars on both sides of Mount Pinatubo. The abutments of the Santa Lucia bridge were washed out due to lahar along the O'Donnell-Bangat River. In Pampanga, flooding was experienced in numerous municipalities. Heavy sedimentation reached into the municipality of Santa Rita. In the Santo Tomas basin, several stretches of the existing levee collapsed and sedimentation covered what remained of San Rafael and the community of Santa Fe. Much of Castillejos and San Marcelino were inundated.

The Regional Disaster Coordinating Council for Region III estimated P589 million of damage to agricultural crops and P40 million in damages to national roads and bridges in the Pampanga, Tarlac, and Zambales.[26]

2.5 Lahar and Flood Damages: Post-Eruption

According to Bautista,[27]about 28 barangays had been severely damaged by lahar as of October 1992 with as much as 3 meters of deposition. Eleven of these barangays are in the municipality of Botolan in Zambales which were flooded by the Bucao River. Five are in Tarlac and the other 12 in Pampanga, mostly in the municipality of Bacolor.

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[26]*Update on Tropical Storm Kadiang*, 9 October 1993.
[27]Op cit, p. 29.

Table C-20 lists the impacted barangays and the number of households as of 1990. About 9,585 households (with a population of 54,815) lived in these barangays.

Table C-20 -- *Barangays Hit by Lahar Deposition as of October 31, 1992*

Province	Municipality	Barangay	1990 Households
Pampanga			5,860
	Bacolor	Balas	308
		Duat	300
		Parulog	321
		Potrero	786
		San Antonio	887
		Santa Barbara	507
	Mabalacat	Cacutid	248
		Dolores	1471
		Tabun	528
	Porac	Mitla	287
		San Jose Mitla	217
Tarlac			1812
	Bamban	Malonzo	128
		San Pedro	365
		Bangcu	35
	Conception	Malupa	230
		San Martin	178
		Santa Rita	876
Zambales			1913
	Botolan	Villar	230
		Poonbato	483
		Moraza	112
		Belbel	124
		Burgos	145
		Cabatuan	77
		Malombong	208
		Owaog-Nebloc	34
		Palis	79
		Nacolcol	143
		Maquisquis	273
Total			9585

Source: Bautista, op. cit., Table 2-6, p. 30.

Flooding, or more precisely ponding, became more of a problem in 1992 as the delta channels became silted and clogged. As of October 1, 1992, floods related to the September monsoon rains had not yet subsided in 30 barangays in eight muncipalities in Pampanga and one in Zambales.

Because of the greater incidence of flooding in 1992, the number of affected people increased in 1992 from the number in 1991 (see table C-21). In 1991, a total of 33,400 families experienced the effects of either lahar or flooding. In contrast, the number of affected families in 1992 increased to 164,200 families (August 1992). This number probably underestimates the true figure for 1992 because it misses lahar and flooding in September 1992.

Table C-21 -- *Distribution of Affected Families by Province and Type of Disaster*

	Provincial Distribution of Families Affected by:			
Province	Lahar 1992	Floods 1992	Lahar and Floods 1992	Lahar and Floods 1991
Pampanga	27%	52%	49%	62%
Tarlac	40	5	9	13
Zambales	33	24	25	20
Bataan	-	17	15	5
Bulacan	-	2	2	-
Nueva Ecija	-	<1	-	<1
Total	100%	100%	100%	100%
Total Number	19,932	144,259	164,191	33,400

Source: Bautista, op. cit., p. 32.

2.5.1 Structure Damages. Bautista reports that a total of 3,140 houses were completely destroyed and 3,072 partially damaged in 1992. Forty-four percent of the destroyed structures were the result of lahar (sedimentation) and floods accounted for the rest. Nearly all were located in Pampanga. Flood damage accounted for all the partially damaged houses, 95 percent of which were in Tarlac.

2.5.2 Infrastructure Damages. Lahar inundated 11 km of infrastructure (roads) in 1991 and 4 km in 1992. Over the 2 year period, 13 major bridges were damaged. Immediately after the eruption, lahar flows collapsed six major bridges: the Abacan, Panda, Mancatian, Pabanlag-Pampanga, Santa Fe, and Umaya. According to Bautista, "lahar flows later in 1991 affected two other bridges along the Botolan-Capas road. At about the same time, the Bamban bridge collapsed and parts of the Cuyapa bridge along the North Expressway in Angeles City, which connects Metro Manila with the northern province of Tarlac, Pangasinan and the Mountain Province, were damaged."[28]

2.5.3 Agriculture. Half of the lahar-damaged croplands in 1991 and nearly two-thirds in 1992 were found in the province of Tarlac, a reflection of the agrarian nature of Tarlac's economy (see table C-22). Croplands in Pampanga accounted for a smaller share of

[28]Bautista, footnote 46, p. 244.

the damage area, especially in 1992. Zambales' share of the damage was relatively limited. Lahar and floods have affected as much as 42 percent of total cropland in the three provinces. As a proportion of total cropland area, lahar and floods caused the most damage in Tarlac (52 percent), followed by Pampanga (41 percent) and Zambales (13 percent.).[29]

Rice paddies accounted for the bulk of the cropland affected by lahar, representing about 90 percent of agricultural areas so affected. Moreover, agricultural lands inundated by floods were also mostly rice-based. Although ricelands hit by lahar are severely damaged, floods also cause signficant damage. Since floods generally leave a layer of sediment, rice farmers often must rehabilitate their lands after a flood.

Table C-22 -- *Crop Area Affected by Lahar and Floods , 1991 and 1992*
(km^2)

Hazard	Province	1991 Area	1992 Area
Lahar	Pampanga	65	31
	Tarlac	110	57
	Zambales	19	6
	Total	194	93
Floods	Pampanga	na	157
	Tarlac	na	270
	Zambales	na	4
	Total	432	100

Source: Department of Agriculture, Region III, *Crop Area Damaged by Lahar, as of Selptember 28, 1992.*

A total of 11,540 farmers were affected by lahar in 1991, most located in Pampanga and Tarlac. The number declined significantly in 1992. Table C-23 summarizes this data.

Table C-23 -- *Number of Farmers Affected by Lahar, 1991 and 1992*

Province	Number of Farmers Affected in 1991	1992
Pampanga	4,687	1,306
Tarlac	5,187	4,748
Zambales	1,666	250
Total	11,540	6,304

Source: Department of Agriculture, Region III, *Crop Area Damaged by Lahar, as of September 28, 1992.*

[29]The total cropland area of each province: Tarlac, 841 km^2; Pampanga, 617 km^2; and Zambales, 236 km^2.

3. METHODS OF ANALYSIS AND ECONOMIC MODEL

This section describes the methods used to evaluate the various projects under consideration. General features of the cost-benefit analysis and computer model are discussed, as is a detailed explanation of the damage calculations upon which project benefits are based.

3.1 Basic Elements of the Cost-Benefit Analysis

The economic analysis in this report is a cost-benefit analysis (CBA) approach. The objective is to determine what, if any, projects are worthwhile from an economic standpoint, and if funds are constrained, which subset of measures should be funded. CBA can also be used to determine the optimal scale of projects.

One of the key elements of the analysis is the use of with-project and without-project conditions. For example, project benefits are measured by the difference between flooding damages suffered without-project and damages suffered with-project. Both conditions allow for trends and changing conditions, as opposed to restriction to existing conditions. Benefits and costs are measured over the planning horizon, which in most cases is assumed to equal the life expectancy of the project. When not otherwise specified, a planning horizon of 25 years should be assumed.

Future benefits are discounted because benefits today are worth intrinsically more than the same benefits some years in the future. A real discount rate of 12 percent is used in this study and has been suggested by both the U.S. Agency for International Development and the World Bank.

Project costs are based on opportunity costs, a resource's value in its next best use. When financial costs do not accurately reflect social opportunity costs, a suitable adjustment is made. For instance, unskilled labor, which suffers relatively high levels of involuntary unemployment, is charged at only 80 percent of the going wage rate.

A number of investment statistics are computed for each project including net present value (NPV), the benefit-cost ratio (BCR), and the internal rate of return (IRR).[30] A positive NPV, a BCR greater than one, or an IRR greater than the discount rate all imply that projects are economically efficient.

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[30] Net present value is the difference between discounted benefits and discounted costs. The benefit-cost ratio is the ratio of discounted benefits and costs. The internal rate of return is the discount rate at which discounted benefits equal discounted costs.

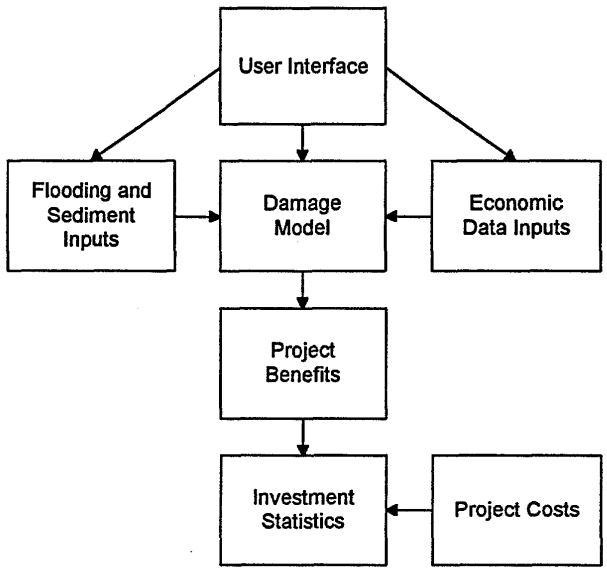
3.2 The Economic Model

The calculation of project benefits and rates of return is subject to numerous kinds of risk and uncertainty. For example, project benefits depend on intrinsically uncertain future rainfall and storm events. Furthermore, economic data such as asset values and quantities, locations and damage schedules are also not precisely known. In this report these uncertainties are represented by probability distributions, and the calculation of project rates of return then becomes a complex probability problem.

The solution of probability problems can be approximated by simulation. The economic model used in this analysis is a simulation model which attempts to represent aspects of the real life situation in Mount Pinatubo and the inherent uncertainty. Both the hydrology and hydraulic data and certain economic data are represented by probability distributions which indicate the likelihood that particular values will occur. The advantage of the simulation framework is that it is an easy way to combine a large number of probability distributions, and it provides a method of quantifying the range of uncertainty of outcomes.

The economic model includes several components including economic, cost and engineering input files, the damage calculation module and the investment analysis module. Figure C-5 is a simplified flowchart of the model components.

Figure C-5 – *Economic Model Components*

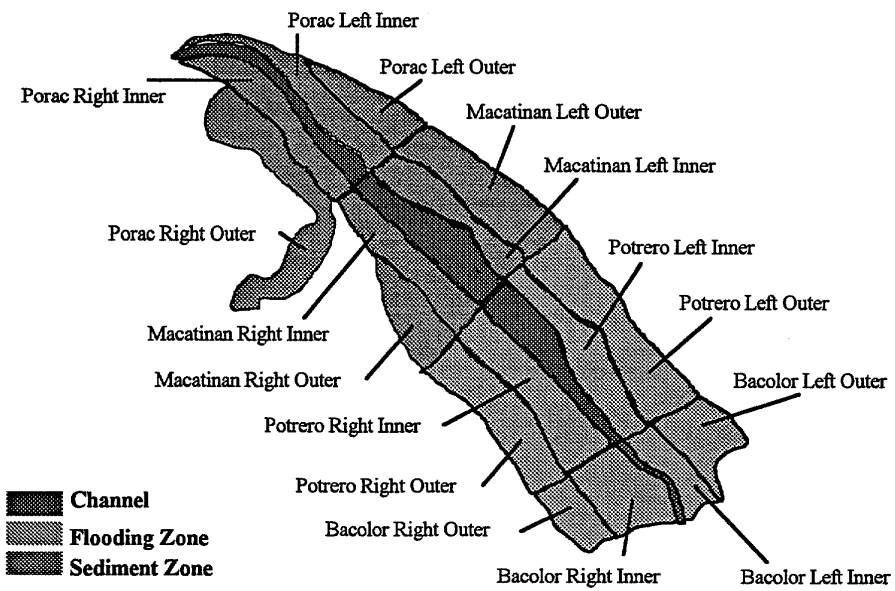


The model is written is Visual Basic which allows for BASIC language programming in a Windows environment. The user interface allows the economist to choose the river basin, project alternative and other parameters such as project life and discount rate. Once these choices are made, the proper engineering, economic and cost data files are accessed.

The engineering files consist of simulated data on sedimentation and flooding for those basins with an active sediment problem.[31] The sedimentation data is annual deposition for each impact zone, and the flooding data is number of flood events entering the impact zone annually. For the three basins with primarily a flooding-type problem, the simulated data consists of percent of hazard zone affected and depth of flooding.[32]

The number of non-channel impact zones identified varies by river basin, with 16 zones for the Pasig-Potrero basin and fewer zones for the other basins. Figure C-6 shows the 16 zones for the Pasig-Potrero basin.

Figure C-6 – *Example of Impact Zone Delineation, Pasig-Potrero Basin*



[31]Those basins with sedimentation problems are the Bucao, O'Donnell, Pasig-Potrero, Sacobia-Bamban and the Santo Tomas.
[32]The Abacan, Maloma and Porac-Gumain are principally affected by flooding, not sediment.

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For each zone, the hydrology and hydraulic sediment model produces simulated sediment accumulations and flood events over a 25-year horizon. This is repeated 500 times in a given simulation and is repeated for both the without- and with-project conditions. This simulated data is accessed by the economic model which also accesses certain economic data files for these zones and which compares damages by zone for both conditions.

The economic files include data on buildings, agriculture and infrastructure for each impact area. In the case of buildings and infrastructure, numbers and values of assets are listed. Separate files contain the damage schedules for buildings and infrastructure which indicate the percent damages associated with flooding and sedimentation events. The agriculture files indicate crop areas, production and gross revenues with and without events, for specific years. This data is discussed in more detail in section 4.

The cost files list the estimated project costs broken down by categories: skilled and unskilled labor, equipment, materials, and contingencies. Future costs are also included.

The heart of the program is the damage module in which damages are calculated for the with- and without-project conditions. Damages are calculated for each year for each impact zone given the specified sediment and flooding events (from the engineering data file) and the economic data. This is repeated for each year until the end of the planning horizon is reached. At this point one iteration is completed and the next can then begin after re-initializing the necessary values. Specifics of the damage model are discussed in section 3.3.

Project benefits are flood and sedimentation damage avoided as the result of the alternative in-place. The model computes these benefits by simply subtracting with-project damages from without-project damages.

Comparison of project benefits and project costs is then made in the Investment Statistics module. As stated above, net present values, benefit-cost ratios, and internal rates of return are calculated.

Multiple iterations comprise one simulation. Each iteration will generate damages, benefits, and investment statistics. One simulation, then, will have a list of outputs: damages, benefits and rates of return for each iteration. The simulation outputs are described with summary measures such as averages and standard deviations computed over all the iterations of a given simulation.

3.3 Damage Models

As previously stated, project benefits are flood and sedimentation damage avoided as the result of the alternative in-place. Flood and sedimentation damage consists of direct damage, indirect damage, and intangible damage which is not quantifiable.

Direct damage includes losses to public and private buildings, agricultural product, and infrastructure assets such as roads and utilities. Indirect damage is the net economic loss suffered as the result of interruption of business activity, transportation disruptions, and costs of activities made necessary by the flood such as evacuation, relocation and emergency flood fighting.

Loss of life, increases in public health costs and damages to the social fabric of the region are examples of intangible losses that are difficult to quantify and are not included in the investment analysis.

This study analyzes damages for the following categories:

- Structures
- Agricultural Production
- Infrastructure
- Transportation Disruptions
- Foregone Income
- Evacuation Costs
- Resettlement Costs
- Clean-Up costs

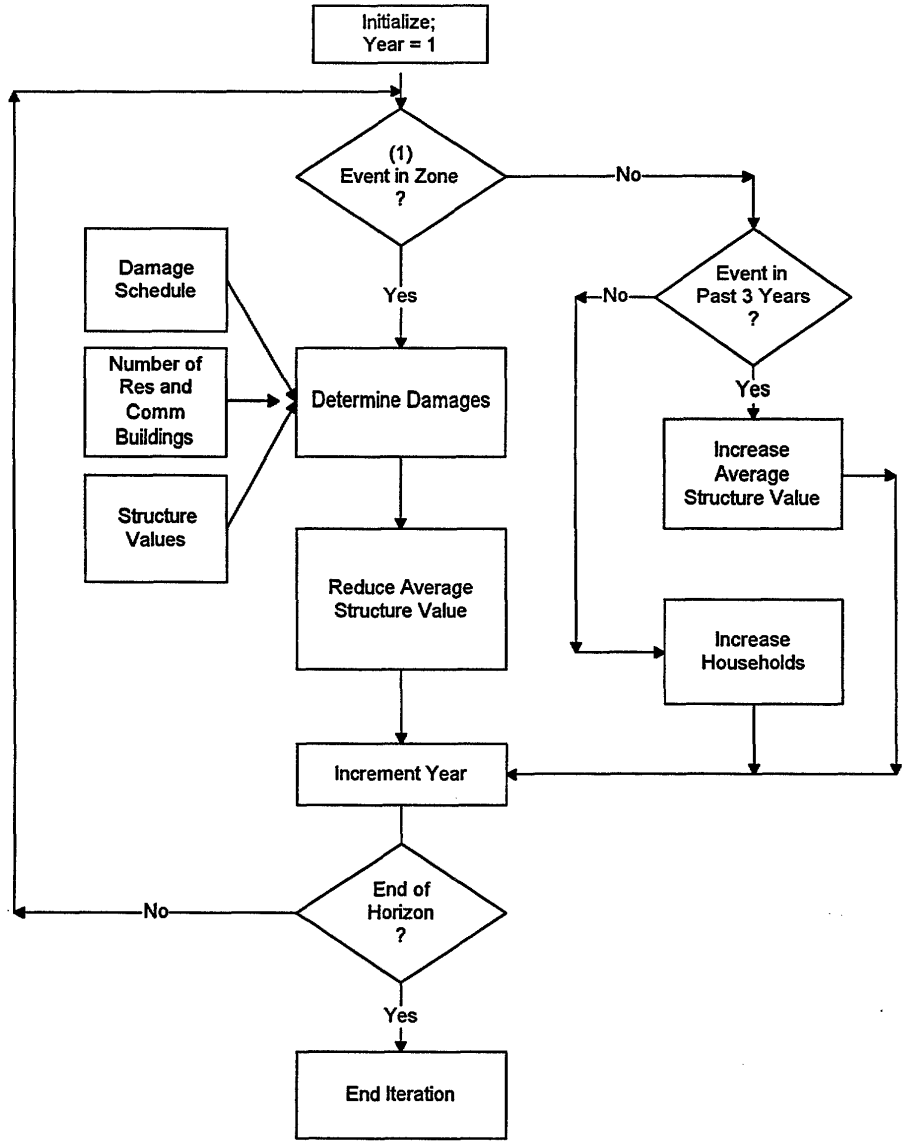
3.3.1 Structure Damages. Damages to structures in a specific hazard area are the product of total structure values in the given zone and percent damages. Total structure values are the product of number of structures and average structure value. Two classes of structures are considered: residential and commercial/other with public buildings classed in the commercial/other category. Percent damages depend only on depth of flooding or sediment deposited in the wake of an event and are given by stage-damage type schedules.

Figure C-7 presents a flowchart of the model. The model begins by inputting the hydrology and hydraulic event data which indicates whether there has been any flooding and/or sedimentation for the given year. If there is no event, no damages are created and the model goes to the next year. If there is an event, the model determines the percent of damages from the damage schedule.

Damages is the product of average house value, percent damages, the number of houses in a given area, and the percent of the area affected.[33] If the event data indicates both sediment deposition and flooding, damages for both events are generated and compared, and the greater of the two is the annual damages.

[33]The percent of area covered is determined by the number of flood events per year relative to the maximum number of flood events which could occur within the simulation model.

Figure C-7 -- *Structure Damages Model*



Average structure values for the next year are reduced by the extent of damage suffered during the event. This step will reduce the monetary damages from future events.

Offsetting this value reduction is a recovery function which assumes that 1 year after the event, one-half of all damage has been repaired or replaced. After 2 years, three-fourths of all damage has been repaired. Finally, 3 years after the event, seven-eighths of the original lost value has been recovered. These percentages are subjective, reflecting observations made in previous damage areas.

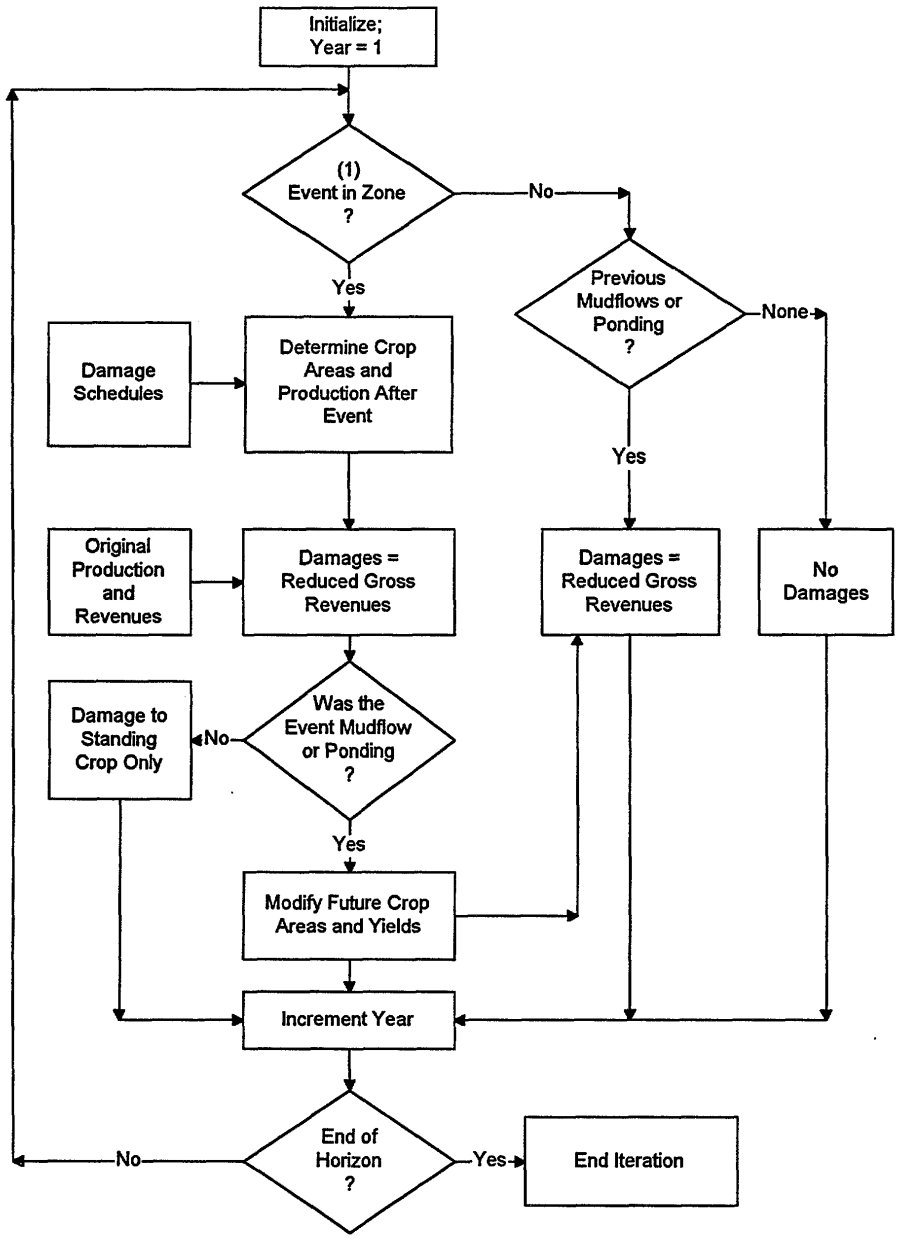
The number of structures is adjusted over time to reflect anticipated population growth. If no flooding and/or sediment event has occurred in the last 3 years, it is assumed that the number of structures will increase by the 1990 to 2000 rural population growth rate for the nation (about 1 percent per year) as projected by the National Statistics Office. Otherwise, there is no change in number of structures from the original count. Section 4 describes the structures, value and damage schedule data in more detail.

3.3.2 Agricultural Damages. Agriculture damages are computed as foregone production in monetary terms attributable to flooding or sedimentation. The estimate of gross revenues is used rather than net revenues (profits) as the measure of losses because it better reflects losses to all factors of production, especially local labor inputs that are not especially mobile.

Damage schedules indicate the extent of lost production and monetary losses over time for each of 14 crop systems given flooding and sedimentation events. The damage schedules are estimates of what crop areas, production and gross revenues would be for the year of the event and selected future years for different types and magnitudes of events. Using these schedules, the model calculates crop areas, productivity and revenue losses over time following a series of events.

Each year in a particular iteration the model inputs the flooding and sedimentation event. If there is an event in the given year, crop areas and production are modified according to sediment, flooding and ponding damage schedules (see figure C-8).

Figure C-8 -- *Agriculture Damages Model*



If a sediment event occurs and deposits more than 50 cm of sediment, damages are calculated using the tables for severe mudflows. If the event deposits more than 10 cm but less than 50 cm, the model calculates damages using moderate sedimentation damage tables. For deposition less than 10 cm, or for flooding without sedimentation, the flood damage tables are used.

Once the post-event crop areas, production, and revenues are determined, the model calculates the difference in total revenues by subtracting post-event revenues from the original level of revenues. That revenue difference is monetary damage of the event for the given year.

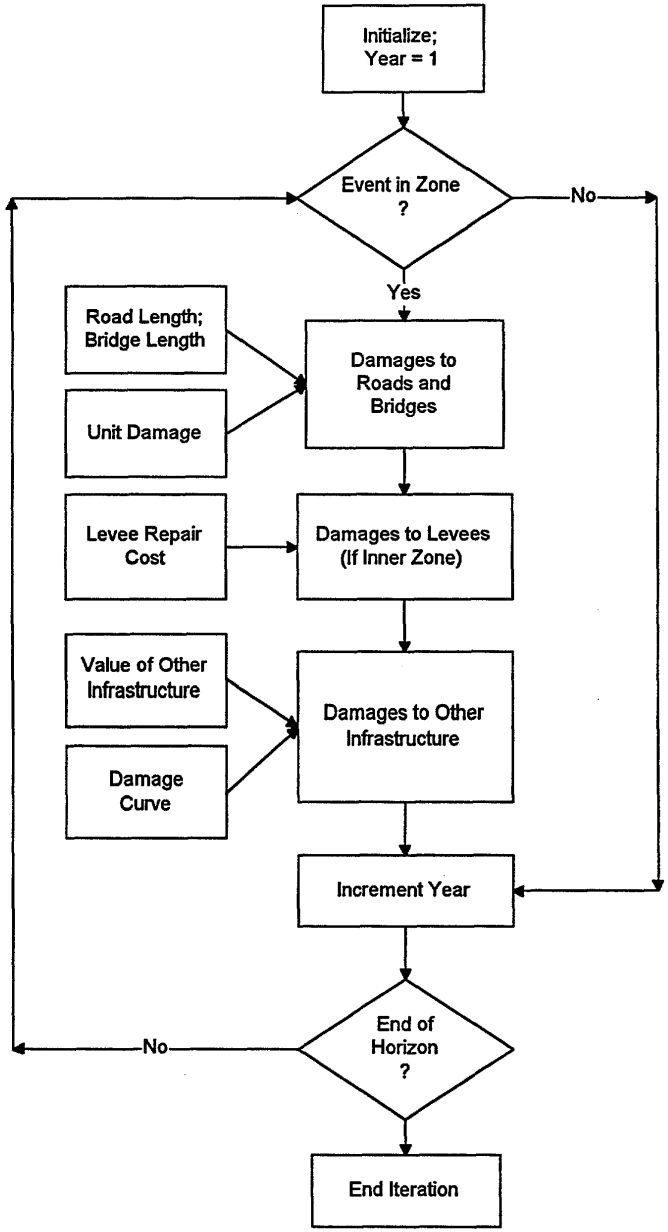
In the case of flooding, production recovers to original levels within 1 year and there is a loss only for the year of the event. In effect, there is damage only to the standing crops. On the other hand, sufficient deposition may lead to long-term reductions in crop yields or crop switching both of which may reduce revenues from original levels for a number of years. In these cases damages are calculated for future years. Again, damages are the difference between original revenues and post-event revenues.

Just how long it takes for production to recover from a damaging event depends on the cropping system and the type of event and severity of the event. According to the damage schedules, even 12 years may not be sufficient for production to recover fully from a major mudflow event for all crop types. On the other hand full recovery from flooding will take place within 1 year.

The model also accounts for cases in which multiple events occur before recovery has been completed from an event in an earlier year. Of course, for shallow flooding with minor deposition this is not a consideration since recovery takes 1 year or less. But for mudflows or heavy sedimentation, the method of handling multiple events is somewhat problematic. In cases where full recovery has not been made following an initial event, the appropriate damage schedule for the second event is determined by the type of events. If both the current and previous sediment are moderate (between 10 and 50 cm), then the moderate sediment damage curves are accessed. Otherwise, damages are based on the severe sediment damage curves. More details on the agriculture damage schedules are presented in section 4 which discusses the data used in this analysis. Variation in simulated damages are entirely attributable to variation in the hydrologic and hydraulic inputs. No probability distributions are present in the agriculture damage schedules.

3.3.3 Infrastructure Damages. Infrastructure includes diverse structures such as roads and bridges, irrigation facilities, water and electric utilities, and flood control structures. (Public buildings are not included in this category but are included as structures). The estimation of infrastructure damages follows the same general logic as for structures and agriculture and are driven by sedimentation and flood events (see figure C-9).

Figure C-9 -- *Infrastructure Damages Model*



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Three classes of infrastructure are considered: roads and major bridges, flood control structures (levees), and all other infrastructure. Lengths of roads by road type for each impact area have been identified using GIS overlay analysis. National bridges have been identified by NEDA Region III.[34] USACE engineers identified existing levee systems. Other infrastructure has been estimated by the urban survey analysis. The values of infrastructure assets at risk in each hazard zone are identified by survey data and other means as discussed in section 4.

Road damages are the product of road stage-damage schedules, depth of flooding or sedimentation, lengths of hard surfaced and loose surfaced roads, road construction costs per kilometer, and percent of area covered by events. The damage schedules for roads were estimated as part of the urban survey, and the depth of event comes from the simulated flooding and sedimentation. As stated above, road lengths came from the GIS data bases and road construction costs are based on Department of Public Works and Highways (DPWH) data. The percent of the impact area affected, in turn, depends on the simulated annual number of flood events relative to the maximum number of possible flood events in a given year.

Damages to bridges are confined to major bridges as identified by NEDA, Region III. Bridge outages are assumed to occur when sediment in the zone(s) in which the bridge is located exceeds 2 meters. Also, if the number of flooding events exceeds a critical number, there is assumed to be a 25 percent probability of bridge failure. If there is a bridge failure, damages are estimated as the product of a unit repair cost (P170,000 per meter[35]) and the length of the bridge.

Damages to levees are incurred whenever a hazard zone adjacent to the river channel experiences a flood event or mudflow. If so, then a "typical" levee repair cost is imposed that is the product of a unit repair cost and the length of damaged levee.[36] The length of damage varies randomly from 0 to 500 meters according to a triangular distribution.[37]

Other infrastructure damages are based on damage schedules as determined by the consultant survey. The survey identified mean, high and low damages to non-road infrastructure for different stages for urban areas. These damages were placed on a per

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[34] *Study on the Projected Extent of Physical and Financial Damage of Lahar Flows in the Coming Years Under Different Scenarios*, October 6, 1992.

[35] This bridge repair cost is the DPWH standard as reported in *Study on the Projected Extent of Physical and Financial Damage of Lahar Flows in the Coming Years Under Different Scenarios*, National Economic Development Authority, Reion III, October 6, 1992.

[36] The cost per repair is based on proposed reconstruction costs of dikes on the Gumain and Ssnto Tomas rivers as reported in *Mt. Pinatubo Rehabilitation & Reconstruction Program, 1992 to 1997*, Presidential Task Force Pinatubo, Sep. 1992. The cost per meter is about P20,000 to P25,000 for the Sacobia, Porac-Gumain, Santo Tomas; P8,000 for the Pasig; and 2,500 to 3,500 for the Abacan and Maloma. No figure is available for the Bucao.

[37] This length is based on field observations of levee breaches on the Santo Tomas and other basins.

urban cell basis[38] and a triangular distribution is used to simulate per cell damages. Damages are the product of the simulated damages per cell, the number of cells and the percent of the zone covered by flooding/sediment.

3.3.4 Transportation Disruptions. The costs of transportation disruptions are limited to traffic disruptions at critical and vulnerable points in the Mount Pinatubo area. There are five major highways providing a large percentage of through traffic services:

Route 7 -- This highway has two distinct segments, one running north and south along the west coast of Luzon, and the other running more generally east and west between Olongapo and San Fernando.

Northern Luzon Expressway -- This modern, divided, and limited-access toll road runs from Metro Manila on the south to just north of Angeles City.

Route 3 -- Also known as the MacArthur Highway, Route 3 generally parallels the Northern Luzon Expressway. This road was the principal link between Manila and Central Luzon before the expressway was built. Route 3 continues due north from Angeles City through Tarlac.

Route 329 -- This road runs north and south but more easterly than Route 3 and the Northern Luzon Expressway. It ties Concepcion, Magalang, and Arayat together and connects with Route 3 at both ends.

Angeles/Dinalupihan Road -- This road does not have a single route number, nor is it of the same quality as the other highways. However, it provides a relatively short route between Angeles and Olongapo in comparison to Route 7.

Only disruptions occurring at major bridge crossings on these routes or approaches to these bridges are considered in this analysis. It is recognized that this understates the true magnitude of possible traffic disruption damages, ignoring, for example, short-term disruptions and delays on local roads and smaller bridges.

The costs of traffic disruption are the additional costs of using an alternative route at critical points. Based on traffic survey data and consultant analysis, the USACE has estimated the magnitude of these costs at seven national bridges per day of outage. In the event of a bridge failure, total damages are the product of the daily outage cost and the number of days of bridge outage (estimated to be 14). In addition, for lesser events not resulting in a bridge outage, damages are the product of the number of days of traffic

[38] As part of the data survey discussed in section 4, the hazard zones were divided into 6.25 hectare grid cells, each cell identified as either urban, agricultural or other.

disruptions (estimated to be 3 days), the daily outage cost, and a factor that reflects the severity of the events.[39]

3.3.5 Other Indirect Damages. In addition to transportation disruptions, indirect damages include foregone production (value added) and evacuation, relocation and clean-up costs. The models for these categories are discussed below, and figure C-10 summarizes the key elements of the estimation process.

Evacuation Costs. For severe flooding and moderate sedimentation, households are assumed to vacate their homes temporarily and to reside in evacuation centers. Evacuation costs depend on the length of evacuation, weekly cost per household for evacuation (P500 per week per household), and the number of evacuated households.

The conditions under which evacuation are assumed to occur are either sedimentation with depths between 0.2 to 1 meter, or at least one flood event per year. The evacuation period is one week in the event of flooding, or one week per 20 cm of deposition in the event of sedimentation, whichever is greater. It is assumed that average sedimentation depths in excess of 1 meter result in permanent relocation. The weekly cost per household for evacuation is based on estimates of administrative and variable costs from existing data.[40]

The number of affected households is the product of the number of households in the affected impact zone, the proportion of the zone that experiences flooding or sedimentation, and an adjustment factor in the case of flooding. The number of households in the affected zone is assumed to equal the number of residential buildings and the proportion of a zone that is affected is assumed to equal the ratio of the number of flood events in the given year to the maximum possible number. An adjustment factor of 50 percent is used to reduce the number of affected households that would evacuate in the event of floods and is based on USACE observation of families continuing to live in flooded houses and on responses to questionnaires at environmental Scoping Sessions. These responses showed that only small percentages of households were evacuated in recent flooding.

Relocation Costs. Relocation costs are limited to cases in which an area is impacted by mudflows/sediment with 1 meter of deposition or greater. In this case all households in the affected part of the impact area are assumed to be relocated at a cost of P60,000 per household.[41] No relocation occurs in the Abacan, Maloma and Porac-Gumain basins since there is no sediment problem there.

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[39]The severity factor is the greater of: (1) the square of the ratio of the number of simulated flood events to the maximum number of flood events or (2) the square of the ratio of the simulated sediment depth to 2 meters.

[40]The cost per family is based on Presidential Task Force Pinatubo estimates of P45 per day per family for basic needs of evacuees. This amounts to P215 per week which is felt to be low. In this analysis P500 per household per week of evacuation is used.

[41]This cost reflects non-structure relocation costs using World Bank and GOP data.

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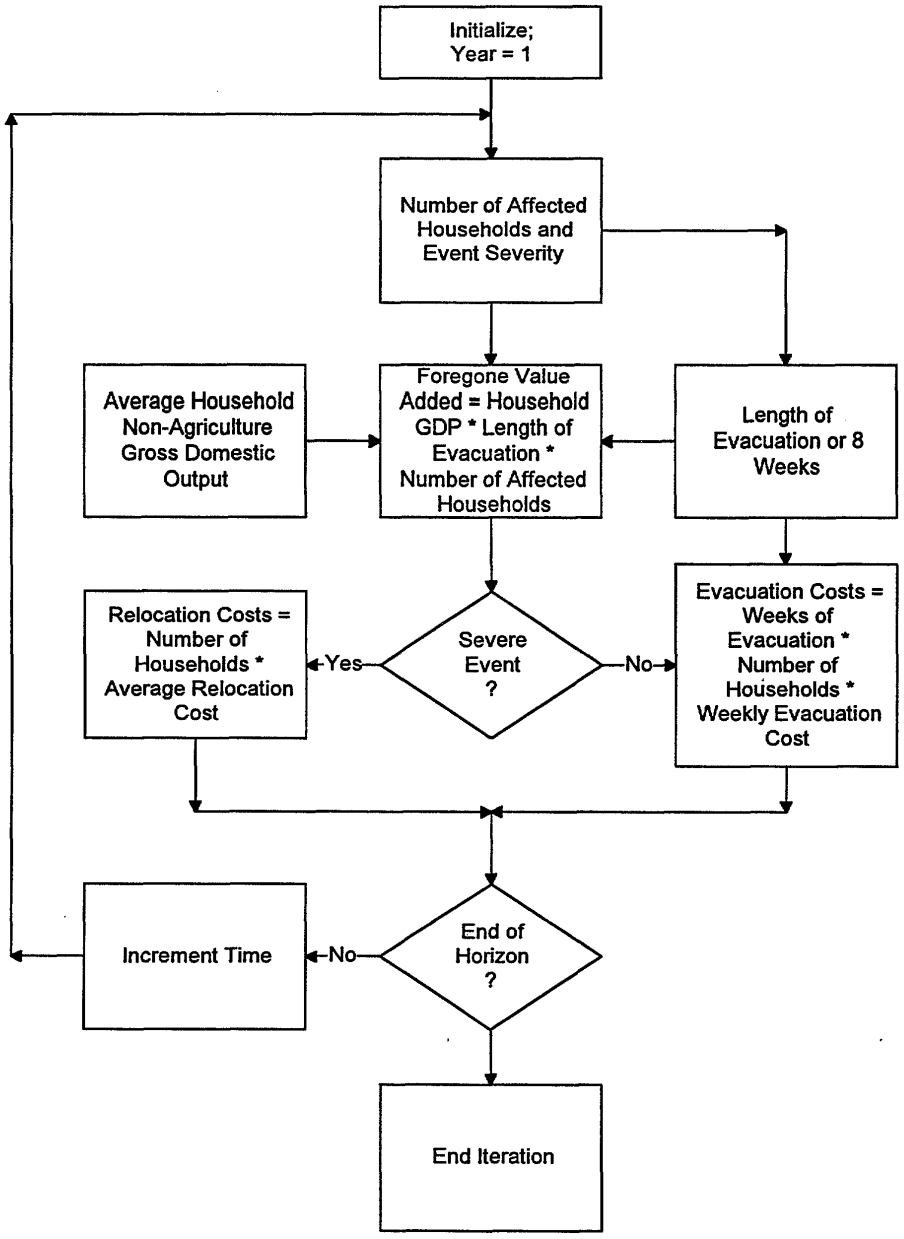
Figure C-10 -- *Indirect Damages Except Clean-Up Costs*



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Damages associated with relocation include the costs of lot site and other land costs as well as administrative costs. The costs of the house and infrastructure at the relocation site are not included because these costs have already been counted in the structures and infrastructure damage categories. Damages are calculated as the simple product of the number of households in the affected part of the impact zone and the unit relocation cost.

Emergency Clean-Up Costs. Based on survey data, an average of 3 man-days of labor were expended to clean flood damage and 6 man-days of labor were required to clean sediment damage. Clean-up costs were estimated as the product of the appropriate number of days of labor, the cost per day[42], the number of buildings in the affected impact zone, and the percent of the zone covered by flooding or sediment.

Foregone Value Added. Estimated reductions to non-agricultural value added are based on the number of affected households and per household gross output. The level of damages is the product of the number of households affected, the duration of the production interruption, and the per household rate of non-farm output.

The number of affected households is determined within the structures damages module and depends on the number of buildings in the affected zone, the percent of an impact zone that is damaged, and an adjustment factor in the case of flooding. As before, the percent of impact zone covered is the ratio of number of flood events to the maximum possible. The adjustment factor equals 50 percent in the case of flooding and is based on the adjustment factor used in the evacuation calculation.

The length of production interruption depends on the type and severity of events. It is assumed that the length of the production interruption equals the length of evacuation periods in the case of evacuation or 8 weeks in the case of relocation. The per household rate of output (6,783 pesos per month at 1992 price levels[43]) is based on 1991 Region III non-agricultural gross domestic product.

Losses to agricultural value added are considered in the agricultural damage model previously described.

[42]It is assumed the cost per man-day is P150 which is slightly above the minimum wage.
[43]Based on data from the Economic and Social Statistics Office, National Statistical Coordination Board.

4. DATA COLLECTION AND DESCRIPTION

The computer model used to evaluate the economics of the different alternatives requires large amounts of data for both engineering and economic variables. This section describes how the data was gathered and developed. Two groups of data are discussed: the hydrology and hydraulic data, and the economic data used in the evaluation of project benefits. Detailed information on the hydraulic, hydrologic, and sedimentation data can be found in Technical Appendicies A and B of the Long Term Report. The economic data was largely obtained by a primary data survey.

4.1 Hydraulic, Hydrologic, and Sedimentation Data

The hydraulic, hydrologic, and sedimentation data that are used in the economic model consist of sedimentation and flooding inputs. A stochastic sedimentation model was developed by the USACE that simulates the physical processes and allocates the sediment and flooding to individual hazard zones. This sedimentation model is applicable to the basins with significant pyroclastic deposits in the upper reaches, namely the O'Donnell, Sacobia-Bamban, Pasig-Potrero, Santo Tomas, and the Bucao. The Abacan, Gumain-Porac, and Maloma basins are primarily threatened by floods and in-channel sediments and stage frequency curves are the important engineering inputs to the economic analysis. For the Pampanga delta, probability distributions for percent of zone covered by flooding, flood depth, and duration were developed for the with- and without- project conditions.

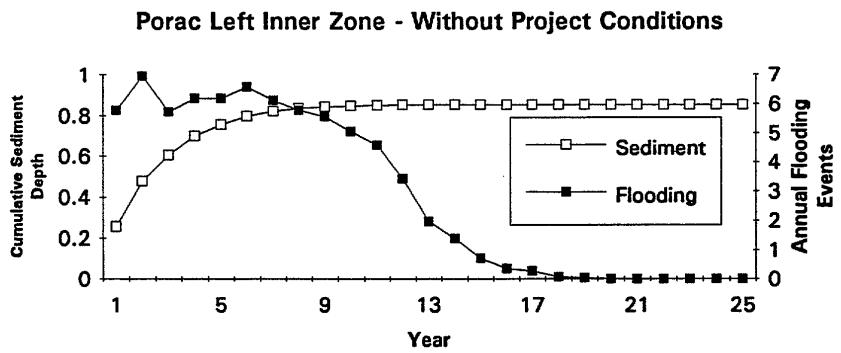
4.1.1 The Sediment Model. For the five basins with a sediment problem, an EXCEL spreadsheet model was developed that produces simulated data on sedimentation and flood events. The sedimentation data is accumulated deposition for each impact zone by year, and the flooding data is the number of flood events entering the impact zone annually. Flooding is a shallow-type with depths between 25 and 45 cm.[44] As discussed in section 3.2, the number of impact zones varies by river basin.

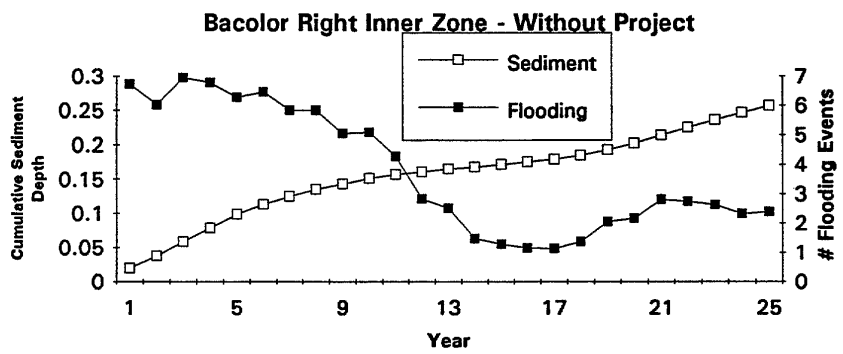
The output (sediment depth and number of flood events) from this sedimentation model is saved as a computer file in a format usable by the economic model. This file includes data for 500 iterations of the sediment model, 25 years of data per iteration (from 1993 to 2017), with coverage for as many as 16 hazard zones per river basin. Because the first year's simulated data in each iteration is for 1993, which has already occurred, that year is dropped before entering the data into the economic model. To compensate for the loss of one year's data, an extra year of data was added at the end of the horizon (year 2018). To simplify matters, the value for events in 2018 was set equal to the average value from 2013 to 2017.

[44] A triangular distribuiton over the given range of depth was used for simulation purposes.

The hydraulic and hydrologic simulated data is provided for both the without- and with-project conditions for all impact zones. The incidence of sedimentation and flooding events is reduced in the with-project conditions for most impact zones and this accounts for the reduction in damages, and hence benefits, associated with each alternative. Figure C-11 shows typical accumulated sedimentation and annual flood event data averaged over all iterations for two zones, the Porac Left Inner zone and the Bacolor Right Inner zone, for the without-project condition. Each of the 500 iterations would have a different pattern, with perhaps no events for some iterations and very major events in others. The averages shown in the figure smooth out the fluctuations and mask the major events.

Figure C-11 – *Sediment Accumulation and Annual Flood Events for Two Pasig Zones (average for without-project condition, depth in meters)*





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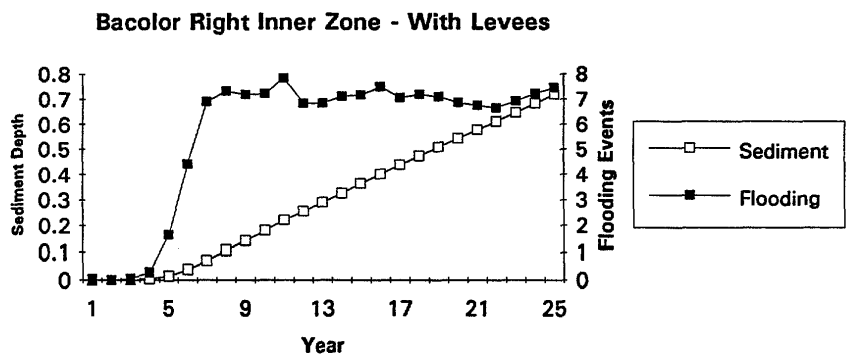
For the with-project condition, the frequency and magnitude of the events are reduced. In the case of the levee alternative for the Pasig basin, complete protection is afforded to the Porac Left Inner zone. Therefore, no damages would be recorded in the with- project condition for this zone. On the other hand, the Bacolor Right Inner zone continues to experience flooding and minor sedimentation, expecially in the later years as shown in figure C-12. Hence, damages would be incurred in this zone for the with- project condition, but at a reduced level.

Figure C-12 -- *Sediment Accumulation and Annual Flood Events for Bacolor Right Inner Zone (average for with-project condition, depth in meters)*



4.1.2 Flood Frequency Curves. For the three basins with minimal sediment problems, stage-frequency data was developed for the without-project condition. The flood frequency data includes estimates for the 2-,100- and 500-year flood events. Associated with each event are percent of hazard zone covered and depth of flooding. These are presented in table C-24. Table C-25 presents the differences in hydraulic inputs according to basin type.

Table C-24 -- *Stage-Frequency Data for the Abacan, Maloma and Porac-Gumain Basins*

Basin	Flood Frequency	Percent of Hazard Area Flooded	Flood Depth (Meters)
Abacan	0.002	75	0.25
	0.010	50	0.25
	0.500	25	0.20
Gumain--Caulaman River	0.002	100	1.30
	0.010	75	1.20
	0.500	50	0.80
Gumain--Downstream of Caulaman	0.002	80	1.00
	0.010	50	1.00
	0.500	25	0.80
Maloma	0.002	80	1.00
	0.010	75	1.00
	0.500	25	0.80

Using the flood frequency data, a flood simulation model was developed that produces percent of hazard zone covered by flood and water depth for the without-project condition. Linear interpolations were made between the 100-year and 2-year events. The output file from the flooding model is analogous to the file from the sedimentation model and serves as a key input for the calculation of without-project damages. To evaluate with-project damages, it is assumed that each alternative provides protection from events with a probability of occurrence of 1 percent or greater.

Table C-25 -- *Differences in Hydraulic Inputs According to Type of Basin*

	Sediment Basins	Flooding Basins
Applicable Basins	O'Donnell Sacobia-Bamban Pasig-Potrero Santo Tomas Bucao	Abacan Porac-Gumain Maloma
Distinquishing Physical Feature	Large pyroclastic deposits	Few pyroclastic deposits
Problem	Sediment deposition and flooding	Flooding; minor sedimentation
Inputs to economic model	Sedimentation model outputs for as many as 16 impact areas per basin. Outputs are average accumulated deposition and annual flood events by impact area.	Stage-Frequency curves; maps indicating areas covered by event probability. Outputs are percent of area covered and flood depth.
Impact on economic analysis	None. Sedimentation model data directly entered into economic model.	Must convert stage-frequency data into simulation file readable by economic model.

4.1.3 Ponding. A number of assumptions for flood area, depth and duration were made for the ponding zone in the delta for the without- and with-project conditions (see table C-26). These assumptions were specified as probability distributions and the economic model drew values in a random fashion from these distributions.[45] Probability distributions account for lack of knowledge of the true extent of the ponding problem and for natural variation in ponding from year to year.

Table C-26 -- *Delta Ponding Assumptions in the Economic Model*

	Minimum	Most Likely	Maximum
Without-Project			
Percent of Zone Covered	10	60	100
Average Flood Depth (cm)	50	100	150
Duration (weeks)	0	4	8
With-Project			
Percent of Zone Covered	10	30	100
Average Flood Depth (cm)	10	50	100
Duration (weeks)	0	1	4

[45] A triangular distribution over the given range was used.

4.2 Primary Economic Data Survey

4.2.1 Introduction. A primary data collection project was conducted to provide information about residential and commercial structures, agricultural activity, infrastructure, and highway traffic activity in the Mount Pinatubo hazard areas. A random survey provided data on structures, agriculture and infrastructure. Traffic counts were made at certain locations to support the analysis of cost of transportation disruptions.

The primary data survey entailed the random selection of certain urban and agricultural blocks of land (grid cells), the inventory of structures, infrastructure and agriculture activity within these cells, and finally extrapolation of the results of these cells to the population. These are discussed in more detail below.

4.2.2 Sample Strata. In order to maximize the usefulness of the data, the population was divided into a number of strata, or subpopulations, and a sample was taken from each stratum. The first differentiating factor for sampling purposes was river basin identity. Eight basins were initially identified for study but the Maloma was excluded due to its small size, leaving seven basin strata.

The next level of differentiation was the type of hazard. At the time the survey was designed, USACE engineers had identified four hazard types: mudflows, shallow flooding and sedimentation, shallow flooding without sedimentation, and ponding. The two shallow flooding hazards were combined for survey purposes, leaving three hazard strata. There was additional differentiation with respect to land use: urban/developed, agriculture, and other/undeveloped. Other areas were not surveyed since by definition there is little of value in these areas.

This translated into six strata for the Pasig basin and four for each of the six other basins. Based on this stratification and on the degree of damage risk for each basin, as assessed in hydrologic investigations, table C-27 shows the number of structures and agricultural areas were targeted for each river basin:

Table C-27 -- *Desired Distribution of Survey Effort*

	River Basin	Number of Structures	Hectares of Agricultural Land
1	Sacobia-Bamban	800	200
2	Pasig-Potrero	800	200
3	Santo Tomas	400	100
4	O'Donnell	200	50
5	Abacan	100	50
6	Gumain	100	50
7	Bucao	100	50
	TOTAL	2500	700

4.2.3 Sampling Method. The basic sampling unit was a square parcel of land, 250 meters per side, called a grid cell. Cells in each sub-population were sampled randomly, with the number of cells chosen based on the number of the structures targeted per river basin and other criteria, including statistical reliability. Each cell chosen was located on the ground and surveyed according to an established scheme.

Defense Mapping Agency (DMA) maps, at a scale of 1:50,000, were used as the base maps. Hazard maps, those indicating the mudflow and other hazard zones, were overlaid on the DMA maps. Finally a grid overlay was used to identify all the possible grid cells, each of 6.25 hectares. Using information on the DMA maps and aerial photographs, each cell was identified by land use: urban, agricultural, or other. Aerial photographs taken in 1992 were used for identification purposes and provided complete coverage of the study area. These photos were at a scale of 1:25,000 and enlarged to approximately 1:10,500 using a digital laser copier.

After assigning code labels to each cell, a random number generator determined which urban and agricultural cells would be surveyed. The chosen cells were plotted on DMA maps and on the accompanying photographs for use in identifying the cells on the ground in the Philippines. The survey was based on early hazard zone maps as prepared by the USACE. Since then, hazard zones have been enlarged in some cases.

4.2.4 Urban Cell Surveys. The objective of the survey was to determine structure numbers, size, type and values and relationships between potential damages and flood/sediment depth for the strata in question. Partly because of budget constraints, a two-tiered approach to surveying was implemented. Under this approach ,50 percent of structures within a zone were subject to internal and external inspection and in-depth interviews with inhabitants. The other 50 percent of structures were only given an external inspection.

The basic plan was to select enough cells in each urban stratum to reach the number of structures per basin as specified in the previous section. The original objective was to survey 2,500 structures which, assuming 70 structures per cell, meant that about 35 cells would be surveyed. This amounted to slightly less than a 1 percent sample. The survey team, once in the field, discovered that the typical cell had somewhat fewer structures than expected. To compensate for this, the number of cells was increased over the planned amount. Table C-28 indicates the total number of urban cells per stratum and the number to be surveyed. The total number of urban cells was based on early editions of hazard zone maps which have subsequently been changed.

Table C-28 – *Number of Urban Cells to Survey*

Basin	Hazard	Total Urban Cells	Cells Selected-- Planned	Cells Surveyed-- Actual
Bamban	Mudflow	194	2	3
	Flood	456	5	13
Pasig	Mudflow	173	2	3
	Flood	312	3	3
	Ponding	595	6	5
O'Donnell	Mudflow	7	0	0
	Flood	460	5	5
Santo Tomas	Mudflow	6	0	0
	Flood	436	4	6
Porac-Gumain	Flood	332	3	2
	Ponding	162	2	2
Abacan	Flood	307	3	4
Bucao	Mudflow	10	0	0
	Flood	199	2	2
TOTAL		3674	37	48

Cells were located on the ground using DMA maps, aerial photos, global positioning system (GPS), and Geographical Information System (GIS). The survey crews took pictures of each structure surveyed for quality control purposes. Three survey instruments were used, one for the detailed residential survey, one for the external residential survey, and one for commercial and municipal structures. The surveys indicated total number of structures within each cell and provide information about each structure surveyed such as construction materials, size, number of floors, occupant data, values, utilities, data on historical events, previous property damages and other losses. The survey forms are found in Exhibit A to this appendix.

4.2.5 Agriculture. The three main components of the agricultural analysis were: (1) estimation of the land area used for major cropping systems in the different strata; (2) analysis of cost and returns of each cropping system; and (3) formulation and estimation of damage function for each cropping system.

Paralleling the urban survey techniques, agricultural grid cells (6.25 ha each) were identified and randomly selected for analysis. About a 1 percent sample was used which covered 700 ha of land. Table C-29 summarizes the number of agriculture cells by stratum and the number surveyed. The total number of agriculture cells by hazard zones reflects early delineation of the zones which have since been modified.

Table C-29 -- *Number of Agricultural Cells to Survey*

Basin	Hazard	Total Ag. Cells	Cells Selected-- Planned	Cells Selected-- Actual
Bamban	Mudflow	462	5	5
	Flood	3,072	31	31
Pasig	Mudflow	885	9	9
	Flood	445	4	4
	Ponding	1,435	14	14
O'Donnell	Mudflow	0	0	0
	Flood	3,383	34	34
Santo Tomas	Mudflow	61	0	0
	Flood	1,884	19	19
Porac-Gumain	Flood	1,241	12	12
	Ponding	885	9	9
Abacan	Flood	1,276	13	13
Bucao	Mudflow	4	0	0
	Flood	475	5	5
TOTAL		15,478	155	155

Information on these cells was obtained by site visits, interviews with farmers and aerial photo interpretation. About 80 percent of cells were visited; the remaining 20 percent of the cells were assessed from photographs alone. The aerial photos used to help identify land uses were taken in 1992, and are described in section 4.2.3.

It had originally been intended that a much larger proportion of the cells would be assessed from photos. It became clear that the land conformations on the photos that indicate the intensity and type of cropping were misleading. This is because the

conformations indicate land use before the eruption and the farmer plot conformations indicate much more intensive cultivation than is presently being followed.[46]

The first tier of the survey was detailed interview with farmers in each of the three major production systems: rainfed lowland, irrigated lowland, and rainfed upland. These surveys provided information of crop sequences, yields, prices, costs, employment and income. In addition, farmers were asked to provide historical information on flood, mudflows, and drought incidence. The Pinatubo Farmer Survey instrument is included in Exhibit A.

Site visits to the individual grid cells provided general information about the crop systems. Land use within each cell and cropping patterns were identified. Another survey form for this purpose was developed, called the Sample Agricultural Cell Attributes, and is included in Exhibit A.

4.2.6 Infrastructure Survey. Infrastructure consists of structures and facilities relating to utilities such as water, sanitation, electric and telecommunications; transportation such as roads and canals; and other items such as irrigation facilities, dikes and levees. Data on these facilities was inventoried for each cell that was surveyed in the urban survey. Surveys estimated the hazard to each facility on an ad hoc basis. Replacement costs were estimated per unit of the structure such as per meter of improved roadway, per meter of irrigation pipe, etc. Each cell was examined separately and an infrastructure value figure "at risk" was arrived at.

4.2.7 Transportation Disruptions Survey. Data was collected on the volume and type of vehicles at various critical locations, the increased distance and time incurred as a result of taking alternate routes, and a cost per kilometer of operating vehicles of different classes. The average daily traffic (ADT) counts were conducted for seven bridges on the major thoroughfares crossing principal rivers. Consideration was given to estimating the ADT at intermediate points, but this option was rejected because of the expense of collecting these data and the difficulty in eliminating double counting.

A single day was randomly selected for each bridge, and the day was divided into peak (0600 to 2200) and off-peak time periods. A 2 hour sample period was chosen randomly from the peak period and a 1 hour sample period from the off-peak period. The day of the survey also was chosen randomly. For comparison purposes only, other ADT count data was obtained from the DPWH for six of the seven locations.

[46]Several factors contribute to this change including the cessation of gravity flow irrigation, low water holding capacity of the extensive cover of ashfall and lahar, problems with rice paddy ponding due to crumbling field dikes, and changed topography.

4.3 Summary of Economic Data

4.3.1 Structures.

Numbers of Buildings. Extrapolated estimates of numbers of residential, commercial and farm houses based on survey data are presented in table C-30. These estimates are for hazard zone delineations as of May 1993 which have since been updated. For each stratum the best estimate is the product of the number of buildings per cell from the survey and the number of cells in that stratum. The low and high estimates are based on two standard deviations below and above the mean value, respectively.

Table C-30 – *Residential, Commercial and Farm Buildings: Survey Estimate*

Basin	Hazard Zone	Residential + Commercial Buildings			Farm Houses
		Best Estimate	Low	High	
O'Donnell	Flooding	19,780	7,016	32,544	644
	Mudflow	*	*	*	*
Sacobia-Bamban	Mudflow	21,987	6,740	37,233	576
	Flooding	19,748	11,633	27,863	3,648
Abacan	Flooding	10,208	1,058	19,357	688
Pasig	Mudflow	10,784	6,782	14,786	444
	Flooding	23,608	8,906	38,310	0
	Ponding	45,014	12,707	77,321	2,872
Porac-Gumain	Flooding	7,470	3,818	11,122	24
	Ponding	12,529	7,021	18,037	0
Santo Tomas	Flooding	31,087	4,733	57,441	3,223
Bucao	Flooding	9,552	3,582	15,522	1,767
Maloma	Flooding	1,178	*	*	200
	TOTAL	212,945	73,996	349,537	14,106

* No survey data.

Because of the small sample size, the confidence intervals for residential and commercial buildings are quite broad which indicates a low level of confidence in the best estimate. No confidence intervals are available for farm buildings.

To verify these estimates, a GIS overlay analysis was conducted to determine the number of households for each of the above hazard zones. Census data indicates the number of households is closely related to the number of occupied housing units.[47] For example, in Pampanga in 1990 there were 245,500 households in 238,700 single family units. Table C-31 provides a comparison of the number of households from the GIS analysis against the survey results for numbers of buildings. The areas with greatest differences are in the ponding areas, the Sacobia-Bamban mudflow zone, the Bucao flooding zone and the Santo Tomas flooding zone.

Table C-31 -- *Comparison of GIS Household Count vs. Survey Estimates (hazard map as of May 1993)*

		GIS	SURVEY ESTIMATES	
Basin	Hazard Zone	Census Number of Households	Residential and Commercial Buildings	Residential, Commercial and Farm Buildings
Abacan	Flooding	11,871	10,208	10,896
Gumain-Porac	Flooding	11,583	7,470	7,494
	Ponding	19,695	12,529	12,529
O'Donnell	Flooding	22,106	19,780	20,444
	Mudflow	227	not surveyed	*
Pasig-Potrero	Flooding	20,005	23,608	23,608
	Mudflow	11,189	10,784	11,228
	Ponding	29,347	45,014	47,886
Sacobia-Bamban	Flooding	22,560	21,987	22,563
	Mudflow	14,283	19,748	23,396
Santo Tomas	Flooding	17,987	31,087	34,310
Bucao	Flooding	5,256	9,552	11,319
Maloma	Flooding	1,099	1,178	1,378
TOTAL		187,224	212,945	223,861

* = Not surveyed.
na = Not applicable.

[47]Table HSG02-54, 1990 Census, shows that the number of households in Pampanga is very close to the number of single family housing units. Another table, however, shows that the number of buildings to be significantly less than the number of households.

Because the survey results are so uncertain, as indicated by the wide confidence intervals, it was decided to incorporate the GIS overlay/census household estimates into the survey estimates. The basic assumptions are that the survey results are relatively accurate in the aggregate, that the census counts are relatively accurate by hazard zone, and that a stable relationship exists across basins between the number of households and the number of buildings. The following method was used:

- The survey total for all urban cells (212,945) was used as the best estimate for total residential and commercial buildings. Sampling statistics indicate a high of 259,861 buildings and a low of 166,028 for a 95 percent confidence interval for the total.

- The ratios of the survey estimates of total buildings to the number of total households in all zones (187,224) were calculated for the best estimate and the high and low cases. This ratio equals 1.137 for the mean case.

- These ratios were multiplied by household counts for each individual hazard zones. The product is an estimate of the number of residential and commercial buildings in each zone for each case, best, high and low. For example, for the Pasig mudflow zone with an estimated 11,189 households, the product of 11,189 and 1.137 is an estimate of urban buildings. This product is 12,726.

- To the above estimates are added the survey's estimate of the number of farm buildings to derive the total number of buildings in each zone.

Based on these calculations and assumptions, table C-32 shows the estimates for building numbers are obtained for the early delineation of the hazard zones.

Table C-32 -- *Best Estimate of Building Numbers by Basin and Hazard Zone (hazard zone as of May 1993)*

Basin	Hazard Zone	Residential and Commercial Buildings	Residential, Commercial and Farm Buildings
O'Donnell	Flooding	25,143	25,807
	Mudflow	276	276
Sacobia-Bamban	Mudflow	16,245	16,821
	Flooding	25,659	29,307
Abacan	Flooding	13,502	14,190
Pasig	Mudflow	12,726	13,170
	Flooding	22,753	22,753
	Ponding	33,379	36,251
Porac-Gumain	Flooding	13,174	13,198
	Ponding	22,401	22,401
Santo Tomas	Flooding	20,458	23,681
Bucao	Flooding	5,978	7,745
Maloma	Flooding	1,250	1,450
	TOTAL	212,945	227,051

One problem with these results is that the estimates are for the hazard zones as of May 1993. Since then, USACE engineers have modified certain river basin hazard areas and further divided these areas into as many as 16 impact zones per basin. In order to deal with these map changes, the number of buildings per urban (and agricultural) cells was determined using the estimated number of buildings from the above table and the number of urban and agricultural cells for the original hazard zones as estimated in the survey effort. For example, in the Pasig mudflow zone, there were an estimated 173 urban grid cells. The quotient of the number of urban buildings in the Pasig mudflow zone (12,726) and the number of urban cells (173) provides an estimate of the number of buildings per cell, or 73.6 buildings per cell. Similarly, it is estimated that there are 72.9 buildings per cell in the Pasig flooding zone.

Subsequently, new counts of the number of cells were made for the revised hazard areas, disaggregated by the individual impact areas within each hazard zone. The number of buildings per cell and the number of cells for each impact area are inputs to the economic model. Table C- 33 presents the number of grid cells for the Pasig basin after adjustment for the hazard zone changes.

Table C-33 – *Grid Cell Count for Pasig-Potrero Impact Zones*

Reach	Zone	Number of Cells Urban Cells	Ag Cells
Porac	Left Outer	20	58
	Left Inner	13	69
	Right Inner	0	45
	Right Outer	52	84
Mancatian	Left Outer	25	142
	Left Inner	8	110
	Right Inner	5	117
	Right Outer	5	43
Potrero	Left Outer	30	152
	Left Inner	33	62
	Right Inner	19	37
	Right Outer	42	110
Bacolor	Left Outer	49	59
	Left Inner	38	30
	Right Inner	55	54
	Right Outer	39	25
	Total	**433**	**1,197**

The product of the number of urban grid cells and the number of buildings per urban grid cell provides an estimate of the number of buildings per zone. For example, in the Bacolor Right Inner zone, the product is 55 times 72.9 = 4,011 urban buildings.

Finally, the building estimates were separated into houses and commercial buildings. Based on the survey results, about 93 percent of total non-farm buildings are residential and 7 percent are commercial or other. Applying these percentages to the Bacolor Right Inner zone, it is estimated that there are 3,730 residential type buildings and 281 commercial/other buildings.

Building and Contents Values. Building and contents values are based solely on survey data and are presented in Table C-34. Values for residential type buildings are disaggregated by basins and hazard zone. Because of the small number of surveyed commercial building values (118), it was decided to use the average of all 118 commercial buildings in all cases.

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For purposes of uncertainty analysis, standard deviations derived from sampling statistics have been computed. It is assumed that there is a normal distribution centered about the average value in the simulations.

Table C-34 -- *Estimated Building Values (in pesos)*

Basin and Hazard Zone	Building Average Value	Building Standard Deviation	Contents Average Value	Contents Standard Deviation
Residential Buildings				
Abacan	169,655	43,746	41,530	10,279
Bucao	87,873	13,281	19,357	2,836
Maloma	87,873	13,281	19,357	2,836
O'Donnell	118,026	35,129	16,473	2,680
Porac-Gumain				
Flooding	115,093	15,218	38,584	18,709
Ponding	39,903	5,141	8,371	1,205
Pasig-Potrero				
Mudflow	149,466	16,736	23,841	2,568
Flooding	71,551	9,062	15,864	2,143
Ponding	121,516	7,678	30,869	2,681
Sacobia-Bamban				
Mudflow	149,327	20,580	37,858	7,030
Flooding	132,626	11,074	25,928	1,711
Sto. Tomas	128,992	10,783	29,721	4,131
Commercial Buildings-- All Basins	1,233,829	450,806	1,205,365	464,085

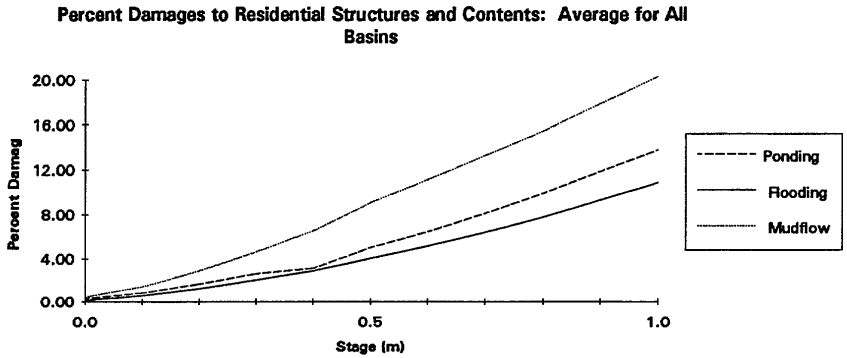
Within the economic model, these values are calculated for each impact zone. Table C-35 shows total building and contents value by zone, again for the Pasig-Potrero basin. This data is for the mean values; within the model values are selected randomly according to normal distributions and so would vary from iteration to iteration.

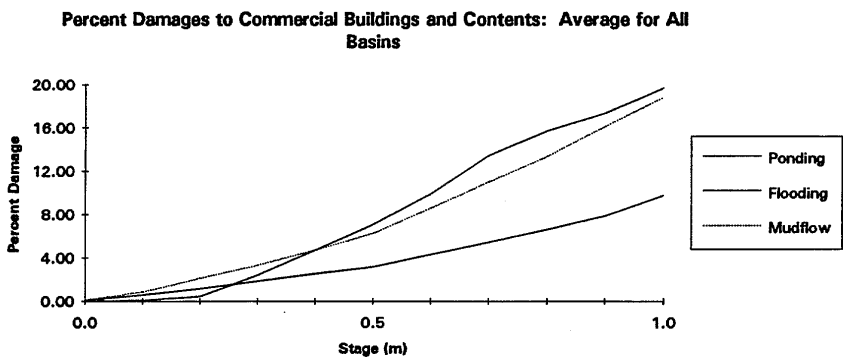
Table C-35 -- *Building and Contents Value by Impact Zone: Pasig-Potrero (pesos)*

		Residential		Commercial		
Reach	Zone	Structure Values	Contents Values	Structure Values	Contents Values	Total Value
Porac	Left Outer	208,836,287	33,311,027	127,064,646	124,133,309	493,345,268
	Left Inner	138,082,729	22,025,279	82,592,020	80,686,651	323,386,679
	Right Inner	3,362,985	536,423	0	0	3,899,408
	Right Outer	537,982,181	85,812,380	330,368,079	322,746,604	1,276,909,244
Mancatian	Left Outer	266,239,302	42,467,258	158,830,807	155,166,636	622,704,004
	Left Inner	90,021,339	14,359,110	50,825,858	49,653,324	204,859,631
	Right Inner	59,869,204	9,549,608	31,766,161	31,033,327	132,218,301
	Right Outer	54,338,962	8,667,491	31,766,161	31,033,327	125,805,942
Potrero	Left Outer	145,588,183	32,279,226	188,964,613	184,605,266	551,437,288
	Left Inner	160,147,001	35,507,149	207,861,074	203,065,792	606,581,016
	Right Inner	92,205,849	20,443,510	119,677,588	116,916,668	349,243,616
	Right Outer	203,823,456	45,190,917	264,550,458	258,447,372	772,012,203
Bacolor	Left Outer	237,794,032	52,722,736	308,642,201	301,521,934	900,680,903
	Left Inner	184,411,698	40,887,020	239,355,176	233,833,337	698,487,231
	Right Inner	266,911,668	59,178,582	346,435,124	338,442,987	1,010,968,361
	Right Outer	189,264,637	41,962,994	245,653,997	239,986,846	716,868,474
	Total	**2,838,879,512**	**544,900,710**	**2,734,353,963**	**2,671,273,381**	**8,789,407,567**

Damage Schedules for Buildings. Percent damage schedules, based on survey responses by building owners or tenants, were developed for residential and commercial buildings. These schedules give percent of building and contents value lost as a function of flood or sediment depth. As with building values, damage schedules are disaggregated by basin and hazard zone for residential buildings. Commercial building damage schedules are disaggregated only by hazard zone. Figure C-13 shows the average damage percent for both residential and commercial buildings for each hazard zone. These curves are applied to the total of structure and contents value.

Figure C-13 -- *Damage Curves for Residential and Commercial Buildings*





4.3.2 Agriculture

Existing Land Use and Production. Table C-36 summarizes crop areas and value of production of agricultural production for the eight river basins within the hazard zones. The Sacobia-Bamban and O'Donnell basins dominate in terms of total agriculture land area, but the Pasig-Potrero and the Porac-Gumain basins have the highest crop values. The latter two basins include the large ponding zone where fish farming, a high value crop, is intensively engaged in. According to survey estimates, fish farming is the highest value crop with annual gross revenues of about $45 million, followed by sugarcane production at $29 million and rice at $23 million.

Both crop areas and production were converted to a "per cell" basis for use within the economic model. With this data and the number of agricultural cells per impact zone, the model can compute value of production for each crop type for each impact zone.

Crop Prices. Table C-37 lists average annual crop prices for the region.

Table C-37 – *Average Prices for Selected Crops in Pinatubo Region*

Crop	Peso per Kg.
Rice	5.63
Sugarcane	7.42
Palopia (Fish)	40.00
Prawns	300.00
Corn	5.97
Sweet Potato	1.88
Gabi (Taro)	7.46
Mango	18.30

Table C-36 -- *Land Area by Crop Type and Total Value of Production*

Total Value of Production by Crop Type and River Basin (Pesos)

Land Use	O'Donnell	Sacobia-Bamban	Abacan	Pasig-Potrero	Porac-Gumain	Sto. Tomas	Maloma	Bucao
Homelot	224,100	1,425,600	232,200	1,119,150	8,100	1,096,200	193,050	600,750
Irrigated 1 Rice + Other	0	23,738,400	0	0	0	0	0	0
Irrigated 2 Rice	32,004,450	44,085,600	0	0	0	0	0	0
Irrigated 2 Rice + Other	38,405,340	0	0	0	0	0	0	0
Rainfed 1 Rice	104,673,627	70,401,312	29,401,704	0	34,157,862	50,982,453	1,030,077	3,204,684
Rainfed 1 Rice + Other	19,074,096	16,369,560	51,848,802	68,342,913	0	34,269,318	3,451,842	10,693,593
Irrigated Upland Crop	0	0	0	0	0	0	0	0
Rainfed Upland Crop	0	0	0	0	0	2,518,560	1,021,680	3,174,336
Sugarcane	304,689,600	304,819,200	18,014,400	115,473,600	39,096,000	3,801,600	0	0
Treecrops	1,701,000	0	0	14,893,200	10,773,000	0	907,200	2,797,200
Natural Range	978,480	852,120	1,013,310	0	157,140	2,956,500	14,580	45,360
Improved Range	0	0	0	0	0	0	0	0
Fishponds	0	25,920,000	23,846,400	731,980,800	430,660,800	4,665,600	0	0
Other Agriculture	103,680	0	0	398,520	660,150	396,090	0	0
TOTAL	501,854,373	487,611,792	124,356,816	932,208,183	515,513,052	100,686,321	6,618,429	20,515,923

Land Area by Crop Type and River Basin (hectares)

Land Use	O'Donnell	Sacobia-Bamban	Abacan	Pasig-Potrero	Porac-Gumain	Sto. Tomas	Maloma	Bucao
Homelot	166	1,056	172	829	6	812	143	445
Irrigated 1 Rice + Other	0	800	0	0	0	0	0	0
Irrigated 2 Rice	755	1,040	0	0	0	0	0	0
Irrigated 2 Rice + Other	755	0	0	0	0	0	0	0
Rainfed 1 Rice	8,231	5,536	2,312	0	2,686	4,009	81	252
Rainfed 1 Rice + Other	1,072	920	2,914	3,841	0	1,926	194	601
Irrigated Upland Crop	0	0	0	0	0	0	0	0
Rainfed Upland Crop	0	0	0	0	0	530	215	668
Sugarcane	7,053	7,056	417	2,673	905	88	0	0
Treecrops	45	0	0	394	285	0	24	74
Natural Range	1,208	1,052	1,251	0	194	3,650	18	56
Improved Range	0	0	0	0	0	0	0	0
Fishponds	0	200	184	5,648	3,323	36	0	0
Other Agriculture	128	0	0	492	815	489	0	0
TOTAL	19,413	17,660	7,250	13,877	8,214	11,540	675	2,096

Gross Yield per Hectare. Table C-38 presents gross yield per hectare for 14 agricultural crop categories.

Table C-38 – *Yield per Hectare in Pesos by Crop Type*

Land Use	Gross Yield Per Hectare (Pesos)
Homelot	1,350
One Rice Crop and Other--Irrigated	29,673
Two Irrigated Rice Crops	42,390
Two Irrigated Rice Crops plus Other	50,868
One Rainfed Rice Crop	12,717
Rainfed Rice Crop plus Other	17,793
Upland Crops	64,800
Field Crops	4,752
Sugarcane	43,200
Treecrops	37,800
Natural Range	810
Improved Range	2,970
Fishponds	129,600
Other Agriculture	810

Agriculture Damage Schedules. The following agriculture damage curves have been used in the economic analysis:

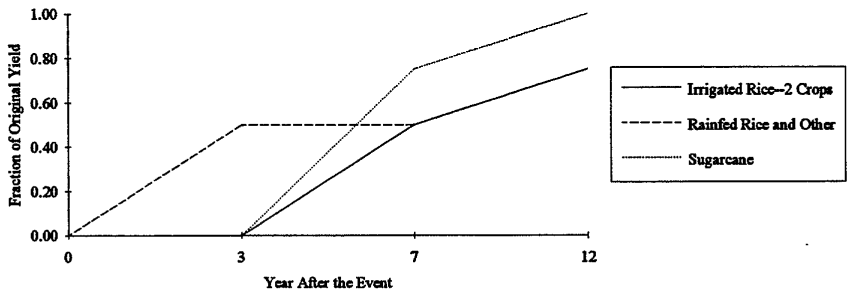
- Sediment, less than 0.5 meter deposition.
- Sediment, greater than 0.5 meter deposition.
- Flooding, 0.5 meter depth.
- Ponding, 0.5 meter, 8 weeks in duration.
- Ponding, 0.25 meter, 4 weeks in duration

The first sediment curve was used for sediment deposition between 10 and 50 cm; the second sediment curve was used for any sediment event in excess of 50 cm. The half-meter flooding curves were used to quantify damages for all flooding type events with the exception of ponding. The two ponding curves were used for damages in the delta.

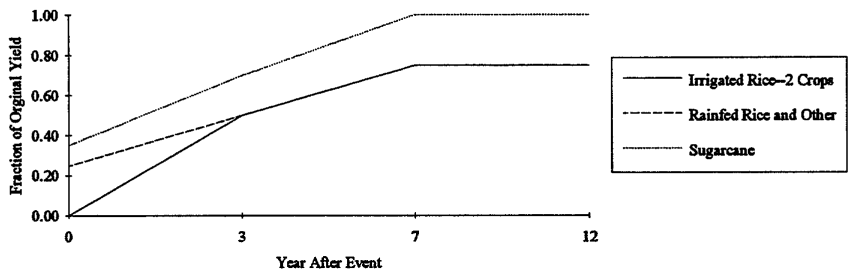
The following two graphs in figure C-14 show yield after sediment deposition for three selected crops for the Sacobia-Bamban mudflow zones. As time passes, crop yields recover towards their original levels, but in some cases never totally regain previous yields. For example, following a severe mudflow (deposition in excess of 50 cm), rainfed rice suffers 100 percent damage in the year of the event, recovers within 3 years to a yield that is 50 percent of the original yield, but only reaches 75 percent of the original yield 12 years after the event. On the other hand sugarcane fully recovers by year 12.

Figure C-14 -- *Sediment Damage Curves*

Yield After Severe Mudflow as Fraction of Original Yield, For Selected Years After Event



Yield After Moderate (< 0.5 meter) Mudflow as Fraction of Original Yield, For Selected Years After Event (Sacobia-Bamban)



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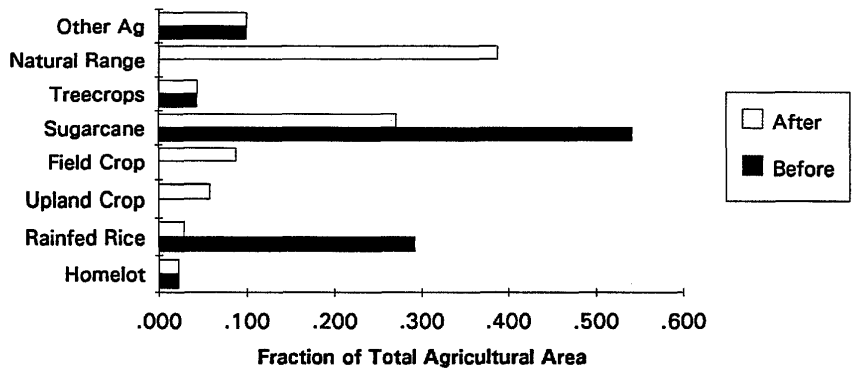
In addition to yield reductions attributable to sediment deposition, cropping patterns may also change. Figure C-15 illustrates such a land use shift for deposition less than 50 cm and greater than 10 cm. The chart shows areas before and after the deposition for only those land uses which would change in terms of land area. As can be seen in the figure, there is a shift out of rice and sugarcane into upland and field crops and natural range. Generally speaking, these other uses have lower over-all value than rice and sugarcane. Over time, however, land uses return to pre-sediment levels although this is not indicated in the figure.

Figure C-15 -- *Crop Areas Before and After 25 cm Deposition As Fraction of Total Area: Pasig Mudflow Zones*



In the case of flooding, full recovery is expected within one year. Table C-39 lists yield after shallow type flooding in terms of fraction of original yield. These fractions apply to all basins for flooding events.

Table C-39 -- *Agricultural Yield by Crop Before and After Flooding*

Land Use	Yield per Hectare Before Flood (pesos)	Fraction of Original Yield After Flood
Homelot	1,300	0.50
One Rice Crop and Other--Irrigated	28,574	0.28
Two Irrigated Rice Crops	40,820	0.50
Two Irrigated Rice Crops plus Other	48,984	0.58
One Rainfed Rice Crop	12,246	0.00
Rainfed Rice Crop plus Other	17,134	0.18
Upland Crops	62,400	0.67
Field Crops	4,576	0.25
Sugarcane	41,600	0.50
Treecrops	36,400	0.75
Natural Range	780	0.75
Improved Range	2,860	0.50
Fishponds	124,800	0.00
Other Agriculture	780	0.50

In the event of long term ponding, damages are limited to the year of the event for all crop types, with the exception of treecrops which would sustain long term root, and yield, damage. Table C-40 summarizes these damages for relevant land uses in the areas susceptible to ponding.

Table C-40 -- *Agricultural Yield Before and After Ponding of 0.5 Meter (8 weeks)*

Land Use	Original Yield per Hectare (pesos)	Yield As A Fraction of Original Yield After Ponding
Homelot	1,300	0.25
One Rainfed Rice Crop	12,246	0.25
Treecrops	36,400	0.00
Fishponds	124,800	0.00

4.3.3 Infrastructure

Infrastructure Assets. As part of the survey effort, data was collected on infrastructure assets within urban grid cells. Based upon this survey data, estimates have been made for values of roads and other infrastructure assets. For purposes of this analysis, however, only the "other infrastucture" data from the survey was used. Road data from the GIS database, expressed in length rather than value, was used in the place of the survey estimates because the GIS data includes roads in all areas whereas the survey

data was restricted to urban areas. Important bridges have been identified by NEDA Region III and are used here. Finally, existing flood control levees have been identified by USACE Engineers.

Table C-41 lists survey-based estimates for non-road infrastructure assets. The zero values for certain hazard zones are a result of no observed assets in the sampled cells. Because of the small sample size, the total value is probably a more accurate estimate of assets than the estimates for individual basins. Also shown is the average asset value per urban cell. No figure for the Maloma basin is shown because no survey work was carried out in this river basin.

Table C-41 – *Estimate of Non-Road Urban Area, Infrastructure Assets by Basin (millions of pesos)*

Basin	Hazard Zone	Mean	High	Low
Abacan	Flooding	49	134	0
Bucao	Flooding	50	198	0
O'Donnell	Flooding	436	9,195	0
Gumain	Flooding	3	8	0
	Ponding	0	0	0
Pasig	Mudflow	11	40	0
	Flooding	85	257	0
	Ponding	37	73	2
Sacobia	Mudflow	0	0	0
	Flooding	3,477	7,160	0
Santo Tomas	Flooding	48	119	0
Total		4,196	17,183	2
Average per Urban Cell		1.19	4.88	0

Road lengths by type of road surface are based on GIS data. These are listed in Table C-42. For purposes of the simulation road, lengths have been identified for each impact area. For example, in the Sacobia-Bamban basin, the entire road length was apportioned to the 16 impact zones outside the channel. This was accomplished using the GIS.

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Table C-42 -- *Road Lengths by Basin (kilometers)*

Basin	Hard Surface	Loose Surface
Abacan	36.7	70.9
Bucao	23.0	30.3
Maloma	3.7	3.6
O'Donnell	41.1	93.3
Pasig	112.1	179.4
Porac-Gumain	21.8	79.2
Sacobia	30.0	130.1
Santo Tomas	42.0	72.3
Total	310.4	659.1

The economic analysis evaluates potential damages to the most important bridges in the Pinatubo region. Table C-43 lists these bridges. This list is based on NEDA Region III data.[48]

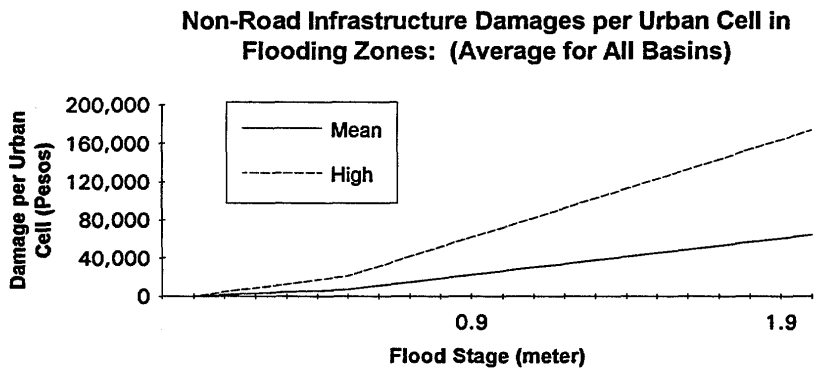
Table C-43 -- *List of National Bridges*

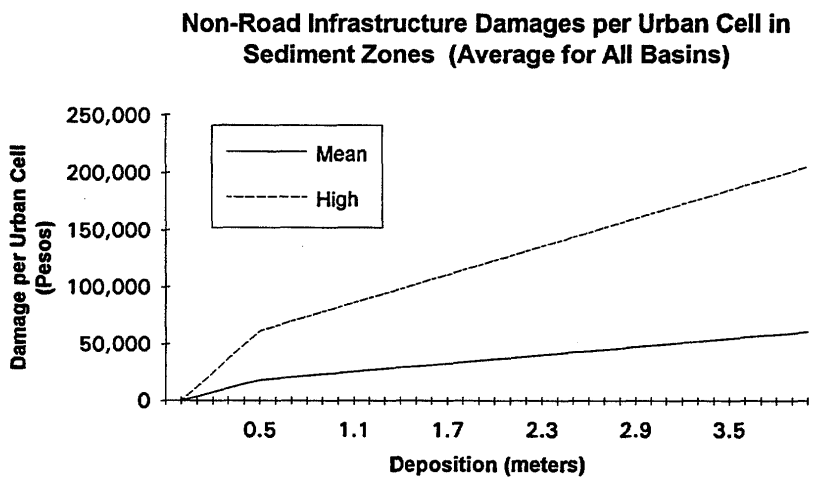
Basin	Bridge Name	Length (m)
O'Donnell	St. Lucia	45
Bamban	San Francisco	400
Abacan	Capaya	400
Pasig-Potrero	Mancatian	150
	Sta. Barbara	150
	Cabetican	na
Porac-Gumain	Sta. Cruz	280
Santo Tomas	Macolcol	380
Maloma	Maloma Bridge	90
Bucao	Bucao Bridge	220

Infrastructure Damage Schedules. On the basis of the urban survey, damages to non-road infrastructure on a per urban cell basis have been estimated. Separate curves have been developed for both flooding and sedimentation. This data is presented in Figure C-16.

[48]NEDA, Region III, *Study on the Projected Extent of Physical and Financial Damage of Lahar Flows in the Coming Years Under Different Scenarios*, October 1992, Table 6. NEDA uses an estimated cost of P170,000 per linear meter for repair costs.

Figure C-16 – *Non-Road Infrastructure Damage Curves*





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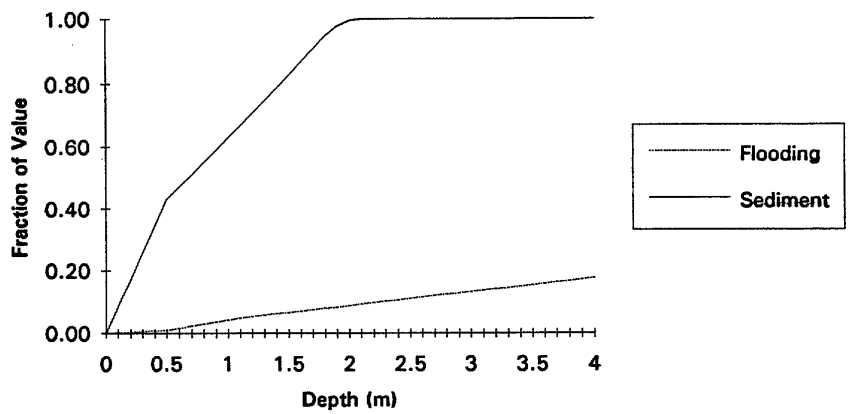
Road damage schedules have also been developed based on the urban survey. Figure C-17 shows percent damages for flooding and sedimentation for roads.

Figure C-17 -- *Percent Damages for Flooding and Sedimentation for Roads*



4.3.4 Transportation Disruptions

Traffic Counts. Traffic counts were made at seven major bridge locations and average daily traffic was estimated. Table C-44 summarizes these estimates as well as results from the 1992 Nationwide Traffic Counting Program.[49] Besides sampling errors and variation, a number of reasons may explain part of the differences between the survey estimates and the Philippine government data:

- The primary data is uncorrected for seasonal variations. (On the other hand, bridge disruptions will almost certainly occur during the rainy season when the survey took place.)
- The primary data was gathered on the bridge itself. Secondary data was gathered on bridge approaches where exits and entrances are possible.
- There might have been significant changes in traffic flows between 1991-92 and 1993 due to effects from the eruption.

[49]Collected from the Department of Public Works and Highways Planning Service, Research and Statistics Division and from the Office of the DPWH Project Coordinator for Mount Pinatubo, DPWH Region III.

Table C-44 -- *Comparison of 1993 Survey with 1992 Nationwide Counting Program*

Bridge	Vehicle Type	1993 Estimate Daily Average	GOP 1992 AADT*
1. Malcolcol	Cars	1,342	533
	Trucks	287	697
2. Cabetican	Cars	6,046	7,075
	Trucks	2,063	568
3. Sta. Barbara	Cars	6,454	7,650
	Trucks	990	9,571
4. Sta. Cruz	Cars	2,063	1,980
	Trucks	424	1,023
5. Mancatian	Cars	560	3,982
	Trucks	344	377
6. Capaya	Cars	6,518	4,460
	Trucks	2,391	553
7. San Francisco	Cars	5,415	NA
	Trucks	2,583	NA

* Annual Average Daily Traffic

Alternate Routes. The additional distance and the additional time required for travel were estimated. Alternate routes are shown on table C-45.

Table C-45 -- *Alternative Route in Event of Bridge Impassability*

Bridge	Alternative Route	Estimated Increase in Distance (km)
1. Malcolcol	Iba-Burgos-Lingayen-San Fernando Expressway Exit	316
2. Cabetican	Bacolor-Olongapo Gapan road via Sta. Barbara Bridge-national Road	6
3. Sta. Barbara	Olongapo Gapan Road-Bacolor via Cabetican Bridge-Olongapo Road	6
4. Sta. Cruz	Lubao-Dinalupihan-Porac-San Fernando Expressway Exit	48
5. Mancatian	Porac-Olangapo Gapan road-MacArthur Highway-Angeles	36
6. Capaya	Capaya-Dau-MacArthur Highway-San Fernando Expressway Exit	23
7. San Francisco	Conception Town-La Paz-Zaragoza-Maharlika Highway-Sta. Rita Exit at Bulacan	112

Vehicle Operating and Labor Costs. Standard vehicle operating costs per kilometer were obtained from DPWH. These include fuel, depreciation , and other operating costs. Personnnel costs for commercial vehicles were estimated using a wage rate of 11 pesos per hour, derived from data available from the National Statistics Office, and considered appropriate for unskilled, casual labor. Operating costs for cars and jeeps are 2.29 pesos per kilometer and 4.30 per kilometer for trucks.

Traffic Disruption Costs. Using the above data, traffic disruption costs were estimated for the loss of the various bridges or of approaches to these bridges. Costs are shown in table C-46. These costs range from nearly P2.9 million per day for the San Francisco bridge to about P108,000 per day for the Mancatian bridge over the Pasig.[50]

Table C-46 -- *Cost of Bridge Outage per Outage Day (pesos)*

Bridge	Vehicle Operating Cost	Labor Cost	Total Cost
Macolcol	1,364,586	125,108	1,489,694
Cabetican	146,046	30,463	176,509
Sta. Barbara	122,152	24,003	146,156
Sta. Cruz	315,686	25,235	340,921
Mancatian	99,353	7,746	107,099
Capaya	582,488	50,717	633,205
San Francisco	2,645,393	247,206	2,892,598

[50]These costs may vary if more than one bridge were to fail simultaneously. For example, if both the Cabetican and Santa Barbara bridge were to fail at the same time, total disruption costs would be higher than the sum of individual costs. Likewise, if bridges were arranged in series such as the western bridges are, multiple outages would be less than the sum of individual costs. This analysis does not account for such dependence.

5. BENEFIT-COST ANALYSIS

This section presents the analysis of benefits and costs for each alternative. The benefits of an alternative are the difference between damages for the without-project condition and the damages with the alternative in place. These benefits are weighed against the project costs which include initial (construction) costs and outyear costs such as operation and maintenance. Net benefits, benefit-cost ratios, and internal rates of return[51] are calculated for each alternative.

This section is organized by river basin, beginning with the Abacan and proceeding alphabetically, except for the delta zone which is treated last. For each basin, simulation results are summarized for without- and with-project damages. Project benefits, costs and the comparison of benefits and costs for each alternative are discussed in turn.

To restate several key parameters, the planning period is 25 years, commencing in 1994 and running through 2018. A 12 percent discount rate is used to present value future streams of benefits and costs. The 25-year planning horizon is specified because the hydraulic and hydrologic data inputs are limited to 25 years which in turn reflects the expectation that the bulk of sedimentation will occur within 10 years. Moreover, the results are generally not sensitive to horizons longer than 25 years because discounting reduces the present worth of future dollar flows so drastically.[52]

With- and without-project damages are calculated for each year of the 25 year horizon for 500 iterations. Mean damages over time are graphically presented here for the with and without project conditions.[53] To provide a picture of the dispersion of results, confidence intervals are placed about these mean values where the upper interval is two standard deviations above the mean and the lower interval is two standard deviations below the mean.[54] In some cases the lower bound may be negative, but this does not imply that negative damages occur in any iterations. This arises in situations where the mean is small relative to the variance and the distribution of results is asymmetric.

To calculate project benefits the model simply subtracts with project damages from the without project damages for each iteration. These benefits are then averaged over all

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[51]The internal rate of return (IRR) is the interest rate at which discounted benefits equals discounted costs. When IRR exceeds the discount rate used to calculate net benefits and benefit-cost ratios, the project is considered an economic investment.

[52]Evaluated at a discount rate of 12 percent, the present value of one dollar 25 years from today is slightly less than 6 cents.

[53] Mean damages are calculated by summing the damages over all iterations and dividing by the number of iterations in a simulation.

[54]The standard deviation is the square root of the sampling variation which is the sum of squared deviations about the mean divided by the number of iterations less one.

iterations and present valued. Confidence intervals are placed about the mean benefits in a manner identical to what was done for damages.

Construction for each alternative is assumed to begin in 1995. Project benefits are always assumed to commence in 1995 despite the fact that construction may last more than one year for certain alternatives. The reason for this assumption is that one or more features of each alternative is expected to be in place in 1995, providing protection against flooding and mudflows in that year.

For each basin costs are categorized by first (construction) costs, annual costs and special outyear costs. Construction costs are further broken down into labor, equipment, materials, tax, overhead and profit. Environmental mitigation costs, which include items such as land costs and relocation costs, are included in first costs. An early warning system is also included for each alternative. For the purposes of the economic analysis tax is ignored since it is a transfer, and unskilled labor is charged at 80 percent of the financial costs to reflect low opportunity costs. Because of these adjustments, economic costs are less than financial costs. Construction costs also include a 30 percent contingency.

For purposes of present valuing, first costs are divided by the construction period and the resulting cost per period is incurred at the beginning of each construction year. In the case of a 2 year construction period beginning in 1995, for example, half the construction cost would be allocated to the beginning of 1995 and half to the beginning of 1996.

Operation and Maintenance costs are generally charged at 0.1 percent of non-excavation first costs. Annual excavation costs are based on expected channel sediment deposition over the project life which varies by basin. Special outyear costs are imposed for major structures at 10 year intervals beginning 10 years after project completion. It is assumed that outyear costs begin upon completion of alternative and all outyear costs are present valued to the beginning of 1994.

Costs are generally stated in pesos but the present value of project costs is also stated in dollars, based on an exchange rate of 27 pesos per dollar.

Based on the present value of benefits and costs, net present value and benefit-cost ratios are calculated. Internal rates of return (IRR) are also computed using the flow of costs and benefits over the planning period. Upon instruction, the model can average the IRR for each iteration or can calculate IRR of the mean benefits and costs.[55]

The display of benefits and costs indicates a greater precision than is warranted by the data or analysis. These numbers replicate output from the computer model which does not round numbers.

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[55]IRR of the average benefits and costs is preferred over the average of IRR for each iteration because for some iterations IRR is not computed. Nevertheless, over a large number of iterations either method gives about the same result.

5.1 The Abacan Basin

5.1.1 Problem. Throughout the Abacan basin the risk of mudflow is low because the upper drainage does not contain significant pyroclastic deposits. The primary problem is the risk of levee breaches and shallow flooding downstream of Angeles City. Also, in channel sediment transported slowly downstream may deposit in channels around Mexico causing an increased risk of shallow flooding.

5.1.2 Alternatives. A bank protection alternative is being considered for the Abacan basin, and is summarized below:

- Bank Protection. Bank and toe protection would be placed on the left and right banks of the Abacan River from the existing sabo structure upstream of Angeles City downstream to the North Expressway. The Sapang Bago Creek from the sabo structure upstream 3 km would also receive bank and toe protection. Downstream of the North Expressway, the existing levee slopes would be seeded to provide erosion protection. A RCC dam 8 meters high would be constructed to replace the existing sabo structure.

5.1.3 Without-Project Condition Damages. On average over all iterations the present value of future damages is P219 million with nearly three-fourths of that in agriculture. Structures damages are relatively small because of the shallow depth of flooding (about 20 cm), where, according to the stage damage curves, buildings suffer only 1 to 2 percent damages. Table C-47 summarizes the present value of damages by type of asset/activity:

Table C-47 – *Mean Without-Project Damages: Abacan (present value, pesos)*

Category	Damages
Agriculture	155,962,879
Structures	53,744,317
Infrastructure	5,980,531
Forgone Production	2,156,153
Evacuation, Relocation and Clean-up	635,752
Transportation Disruption	95,109
Total Damages	**218,574,741**

Figure C-18 shows how mean damages by category vary over the planning horizon.

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Figure C-18 – *Mean Without Project Damages by Category: Abacan*

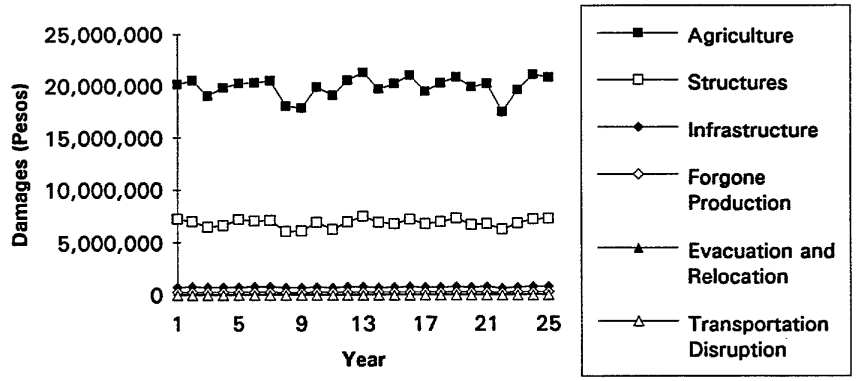
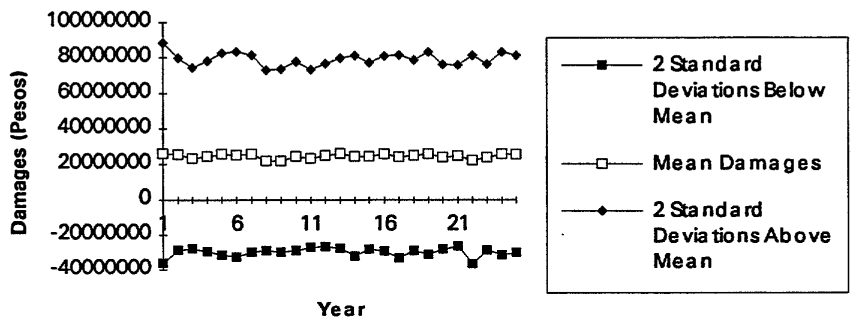


Figure C-19 illustrates the uncertainty of the results, as given by the sampling variance of the simulated damages. In addition to mean damages, a plot of mean damages plus and minus two standard deviations is provided. The standard deviation is a measure of the dispersion of the simulated damages about the mean. The fact that the line for the "low case" is below zero does not indicate that negative damages were simulated, but rather that the dispersion of results was quite large relative to the mean and the distribution of damages is asymmetric.

Figure C-19 – *Range of Without Project Total Damages: Abacan*



5.1.4 With-Project Condition Damages. Bank protection would stabilize the existing river banks through the reaches of significant erosion against flows up to a 100-year event. This alternative would also reduce the amount of siltation in the lower channel. The risk of flooding downstream of Angeles City would be therefore reduced.

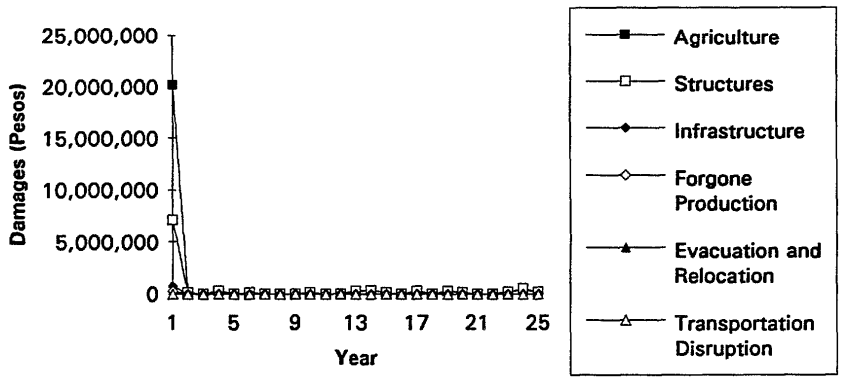
For the purposes of this analysis it is assumed that bank protection will provide protection against 100-year and more frequent flooding events. Under this assumption damages are reduced to P27 million under the with project condition as presented in table C-48.

Table C-1– *Mean With-Project Damages: Abacan (present value, pesos)*

Category	Damages
Agriculture	18,798,577
Structures	7,060,760
Infrastructure	689,034
Forgone Production	259,498
Evacuation, Relocation and Clean-up	76,514
Transportation Disruption	11,140
Total Damages	**26,895,522**

Figure C-20 shows how residual damages are virtually eliminated after the first year, 1994. The reason for continued damages in 1994, is the assumption that the project is implemented in 1995.

Figure C-20 – *Mean Bank Protection Residual Damages: Abacan*



5.1.5 Project Benefits. Project benefits are the difference between without and with project damages and are estimated at P192 million in present value terms. Benefits by category are presented in table C-49.

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Table C-49 -- *Average Project Benefits by Category: Abacan (present value, pesos)*

Category	Sub-category	Benefits
Structures	Residential	30,346,222
	Commercial	14,651,739
	Farm	1,685,597
Agriculture	Rice	93,825,318
	Sugar	11,755,811
	Fish	31,123,053
	Other Agriculture	460,120
Evacuation, Relocation and Clean-up		559,238
Foregone Production		1,896,655
Transportation Disruption		83,970
Other Infrastructure		1,707,303
Roads		539,846
Levees		3,044,349
Bridges		0
Total Benefits		**191,679,219**

5.1.6 Project Costs. Construction costs for the alternative are estimated at P81 million. The present value of economic costs, which deletes taxes and a portion of labor costs and includes future costs, is P68 million. (See Table C-51 for a complete listing of costs.)

5.1.7 Investment Analysis. As shown on table C-50, for the mean case, bank protection has positive net benefits (net present value) of almost P124 million, a benefit-cost ratio (BCR) of 2.8 and an internal rate of return (IRR) of 38 percent. For project benefits two standard deviations (σ) above the mean, the BCR increases to 4.33. Even when project benefits decrease to two standard deviations below the mean, the project remains economically efficient.

Table C-50 -- *Investment Analysis: Abacan Bank Protection Alternative*

	Mean	Two σ Above Mean	Two σ Below Mean
Net Benefits	123,726,000	226,073,000	21,379,000
BCR	2.82	4.33	1.31
IRR	38%	na	na

σ = standard deviation

Table C-51 -- *Alternative Costs for the Abacan*

Financial Construction Costs (pesos)

Line Item	Bank Protection
Profit	5,621,290
Home Overhead	1,102,214
Field Overhead	4,082,273
Vat Tax	4,638,947
Skilled Labor	2,140,795
Unskilled Labor	911,826
Equipment	10,772,496
Material	32,564,350
Environmental Mitigation	150,000
Subtotal	61,984,191
Contingency (30%)	18,595,257
Total First Costs	**80,579,448**

Annual Costs

Line Item	Bank Protection
Dredging	0
Excavation	0
O&M	80,579
Total Annual Costs	**80,579**

Special Future Costs (every 10 years)

Item	Bank Protection
Major Maintenance	2,860,000

Present Value of Financial Costs, 1994 Base

First Costs	71,945,936
Annual Costs	560,050
Future Special Costs	1,086,902
Total costs, pesos	**73,592,888**
Total costs, dollars	**2,725,663**

Present Value of Economic Costs, 1994 Base

First Costs	66,349,770
Annual Costs	516,488
Future Special Costs	1,086,902
Total costs, pesos	**67,953,160**
Total costs, dollars	**2,516,784**

5.2 The Bucao Basin

5.2.1 Problem. There is a high risk of mudflows developing in the upper basin with subsequent transport of sediment into the lower basin. The lower basin, however, can store large quantities of sediment due to its width. Transport efficiency of sediment to the sea is high because of the river slope and clean water from tributaries. Storage and transport efficiency lower the risk of mudflow and flooding in developed areas of this basin. Also, the risk of failures of Highway 7 and the bridge appears to be low. Nonetheless, there is some probability of overtopping or failure of the existing levee system when, for example, the river may confine itself along the levee.

5.2.2 Alternatives. Two engineering alternatives are under consideration for the Bucao basin, as summarized below:

- Levee Alternative. The existing right bank levee above the Highway 7 bridge would be raised to a height of 7 meters between RK 2.5 and RK 5.5, and would transition from 7 meters at RK 5.5 to 9 meters at RK 8. Slope and toe protection would be added to the levee, and channel banks below the Highway 7 bridge..

- Sediment Retention Structure. An earth and rock fill structure 54 meters high would be constructed at RK 13. The existing right bank levee (RK 2 to RK 8) would have slope and toe protection added.

5.2.3 Without-Project Condition Damages. Without project damages total P250 million with 40 percent of damages incurred by structures. Table C-52 summarizes average damages by category.

Table C-52 – *Mean Without-Project Damages: Bucao (present value, pesos)*

Category	Damages
Agriculture	10,649,955
Structures	100,408,492
Infrastructure	64,882,670
Foregone Production	24,463,575
Evacuation, Relocation and Clean-up	35,730,265
Transportation Disruption	13,964,976
Total Damages	**250,099,933**

Figure C-21 presents the time path of average damages by category. As can be seen, damages peak around year 8.

Figure C-21 – *Mean Without-Project Damages by Category: Bucao*

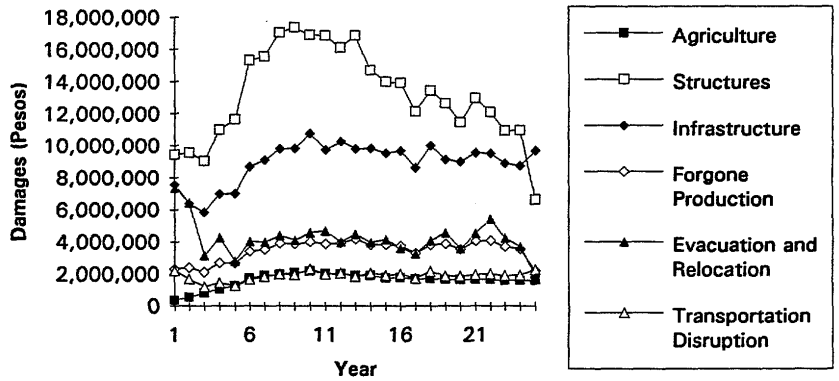
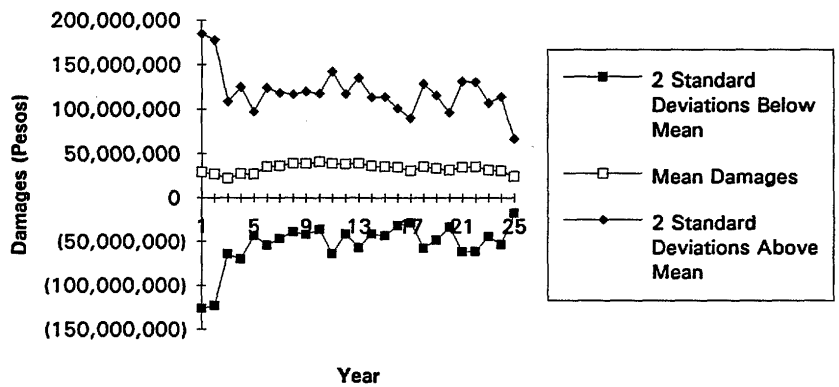


Figure C-22 shows the variation of simulation results over the 500 iterations. Standard deviation of damages about the mean were calculated for the 500 iterations and the graph below shows mean damages and 2 standard deviations above and below the mean.

Figure 22 – *Range of Without-Project Damages: Bucao*



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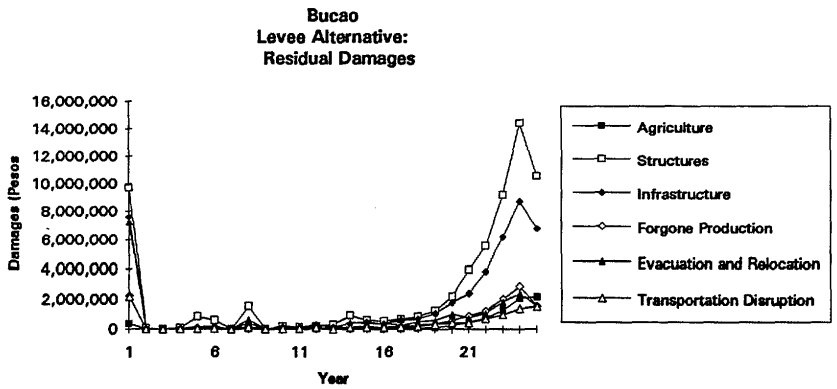
5.2.4 With-Project Condition Damages. As shown on table C-53, residual damages are P39 million for the levee alternative and P26 million for the SRS alternative.

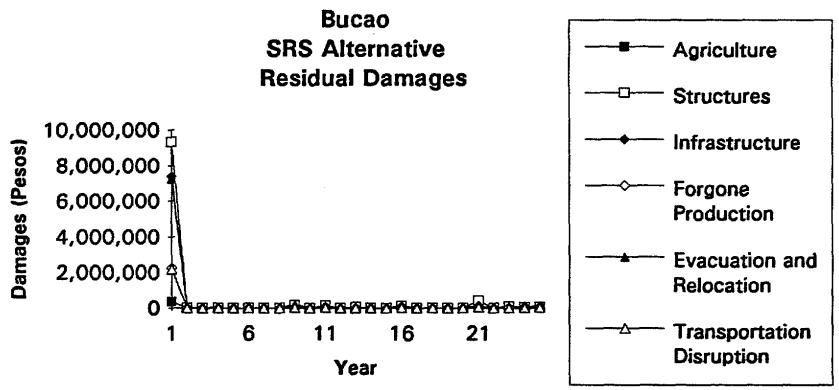
Table C-53 -- Mean With-Project Damages: Bucao
(present value, pesos)

Category	Levee	SRS
Agriculture	891,569	325,971
Structures	14,398,495	8,527,967
Infrastructure	10,006,250	6,730,333
Forgone Production	3,069,408	2,050,286
Evacuation, Relocation and Clean-up	8,090,375	6,679,686
Transportation Disruption	2,664,471	2,007,470
Total Damages	**39,120,567**	**26,321,714**

Figure C-23 shows residual damages for each alternative. Damages are high for the first year, 1994, because project construction and benefits are not assumed to begin until 1995. Damages in the levee alternative pick up again in the later years, but the SRS continues to provide protection, on average, from 1995 on.

Figure C-23 – *Mean With-Project Residual Damages: Bucao*





5.2.5 Project Benefits. As shown in table C-54, avoided damages range from P211 million for the levee alternative to P224 million for the SRS alternative.

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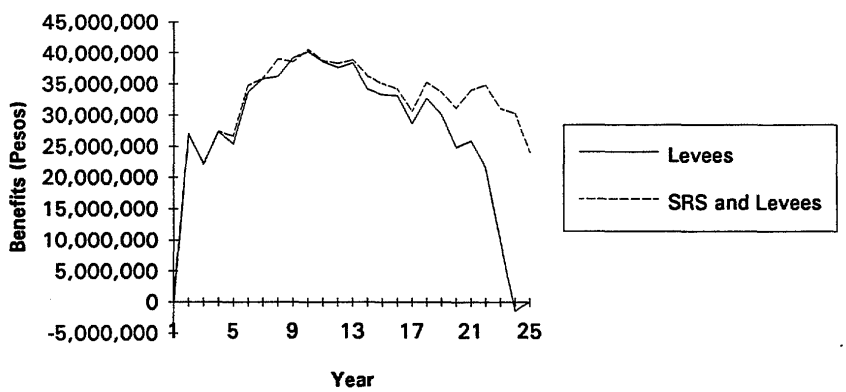
Table C-54 -- *Average Project Benefits by Category: Bucao (present value, pesos)*

Category	Sub-category	Levee	SRS
Structures	Residential	31,094,534	32,947,212
	Commercial	47,752,646	51,455,067
	Farm	7,162,817	7,478,246
Agriculture	Rice	7,158,166	7,576,709
	Sugar	0	0
	Fish	0	0
	Other Ag	2,600,220	2,747,275
Evacuation, Relocation and Clean-up		27,639,890	29,050,579
Foregone Production		21,394,167	22,413,289
Transportation Disruption		11,300,505	11,957,506
Other Infrastructure		26,672,498	27,951,703
Roads		5,011,899	5,278,693
Levees		17,122,154	18,338,297
Bridges		6,069,869	6,583,644
Total Benefits		**210,979,366**	**223,778,220**

Figure C-24 provides a comparison of benefits over time for the two alternatives. Note that the two alternatives have similar benefits until the latter part of the planning horizon when it appears the SRS becomes more effective than the levee alternative.

Figure C-24 -- *Project Benefits by Alternative: Bucao*



5.2.6 Project Costs. Construction costs are about P187 million for the levee alternative and P4,700 million for the SRS alternative. As shown in table C-55, the present value of economic costs is P155 million and P3,318 million for the two alternatives, respectively. More cost detail is in table C-57.

5.2.7 Investment Analysis. As table C-56 shows, the relatively inexpensive levee alternative has benefits exceeding costs for the mean and high benefits case. Costs exceed benefits in all cases for the SRS alternative.

Table C-55 -- *Present Value of Benefits and Costs: Bucao (present value, pesos)*

	Project Benefits			
Alternative	**Mean**	**2 σ Above Mean**	**2 σ Below Mean**	**Project Cost**
Levee	210,979,366	421,122,551	836,181	155,076,196
SRS	223,778,220	427,373,243	20,183,196	3,317,897,579

σ = standard deviation

Based on project costs and simulated benefits, the levee alternative is a viable project for the mean case, as indicated by positive net benefits and the benefit-cost ratio.

The SRS alternative is not economic because of its exceedingly high cost with modest benefits.

Table C-56 -- *Investment Statistics: Bucao*

Levee Alternative

	Mean	**2 σ Above Mean**	**2 σ Below Mean**
Net Benefits	55,903,170	266,046,355	(154,240,015)
BCR	1.36	2.72	0.01
IRR	17.01%	na	na

SRS Alternative

	Mean	**High**	**Low**
Net Benefits	(3,094,119,359)	(2,890,524,336)	(3,297,714,383)
BCR	0.07	0.13	0.01
IRR	na	na	na

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Table C-57 – *Alternative Costs for the Bucao*

Construction Costs, Financial (Pesos)

	Levee	SRS
Profit	13,043,143	328,552,447
Home Overhead	2,557,479	64,422,048
Field Overhead	9,472,144	238,600,179
Vat Tax	10,763,800	271,136,567
Skilled Labor	4,424,008	61,935,607
Unskilled Labor	1,443,220	59,836,504
Equipment	81,101,674	1,742,059,612
Material	20,669,100	847,533,950
Environmental Mitigation	150,000	230,000
Subtotal	143,624,568	3,614,306,914
Contingency (30%)	43,087,370	1,084,292,074
Total First Costs	**186,711,938**	**4,698,598,988**

Annual Outyear Costs

Item	Levee	SRS
Dredging Costs	0	
Excavation Costs	0	
O&M	186,712	4,698,599
Other Annual Costs	0	
Total Annual Costs	186,712	4,698,599

Special Future Costs (every 10 years)

Item	Levee	SRS
SRS Major Maintenance	0	30,000,000

Present Value of Financial Costs, 1994 Base

	Levee	SRS
First Costs	166,707,088	3,567,821,642
Annual Costs	1,297,701	23,244,314
Future Special Costs	0	8,115,057
Total Cost (Pesos)	**168,004,788**	**3,599,181,013**
Total Cost (Dollars)	**6,222,400**	**133,303,000**

Present Value of Economic Costs, 1994 Base

Economic Costs

	Levee	SRS
First Costs	153,878,358	3,288,358,904
Annual Costs	1,197,838	21,423,618
Future Special Costs	0	8,115,057
Total Cost (Pesos)	**155,076,196**	**3,317,897,579**
Total Cost (Dollars)	**5,743,563**	**122,885,096**

5.3 The Maloma Basin

5.3.1 Problem. Due to channel instability, a high risk of flooding may persist for the lower part of the Maloma over the next 5 to10 years. Localized channel filling related to sediment transport may produce overbank flooding and sedimentation. Flooding is the major event that will cause damage.

5.3.2 Alternatives. The engineering alternatives under consideration for the Maloma basin are summarized below:

- Levee Alternative. Levees 3 meters high with slope and toe protection would be constructed on the right and left banks of the Maloma River. The left bank levee would run upstream from the Highway 7 bridge (RK 2.5) to the Gorongoro River (RK 8.5), and the right bank levee would extend upstream from the bridge to RK 5. A new river channel west of the bridge would be cut along a straight alignment through sand dunes which presently force the river southward. Slope and toe protection would be provided to the channel.

- Sediment Retention Structure Alternative. In addition to the levees and river re-alignment described for the levee alternative, a RCC dam 18 meters high would be constructed at RK 19.5.

- Channel Excavation Alternative. A channel 100 meters wide by 2 meters deep would be excavated from the confluence with the Gorongoro River (RK 8) to the Highway 7 bridge. Disposal berms would be setback 100 meters from the channel, and bank protection would be placed on the channel side slopes. A new river channel would be excavated as described for the levee alternative.

5.3.3 Without-Project Condition Damages. The present value of without-project damages is about P112 million with the large majority of damages sustained by structures. Damages by category are in table C-58.

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Table C-58 – *Mean Without-Project Damages: Maloma (present value, pesos)*

Category	Damages
Agriculture	9,798,044
Structures	82,562,993
Infrastructure	15,182,506
Forgone Production	1,191,278
Evacuation, Relocation and Clean-up	351,254
Transportation Disruption	3,580,110
Total Damages	**112,666,186**

Figure C-25 shows mean damages by category over the planning horizon.

Figure C-25 – *Mean Without-Project Damages by Category: Maloma*

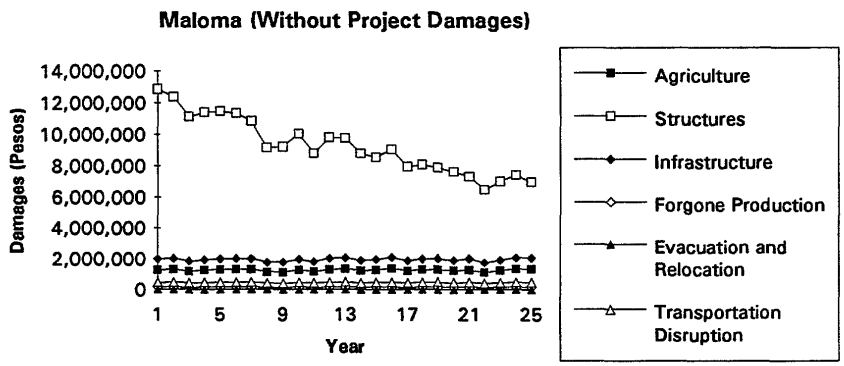

Figure C-26 presents mean damages and two standard deviations above and below the mean over the planning horizon.

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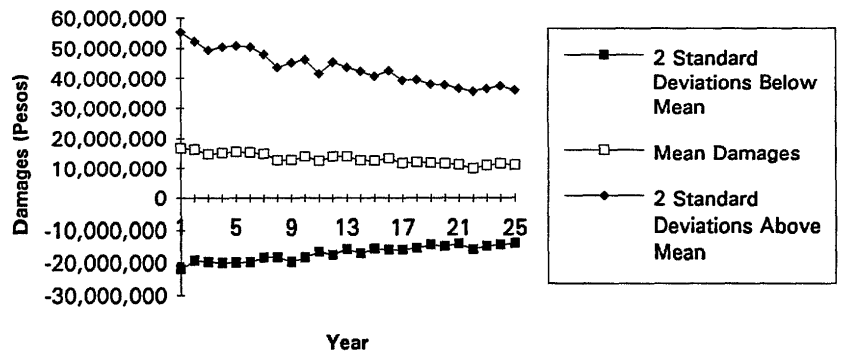
Figure C-26 – *Range of Without-Project Damages: Maloma*



5.3.4 With-Project Condition Damages. All alternatives are assumed to provide equal protection against 100-year and more frequent events and under this assumption, the present value of damages is reduced to about P15 million for each alternative (see table C-59).

Table C-59 – *Mean With-Project Damages: Maloma (present value, pesos)*

Category	Levee	SRS	Channel Excavation
Agriculture	1,173,298	1,173,298	1,173,298
Structures	11,448,254	11,448,254	11,448,254
Infrastructure	1,805,653	1,805,653	1,805,653
Forgone Production	142,328	142,328	142,328
Evacuation, Relocation and Clean-up	41,966	41,966	41,966
Transportation Disruption	419,330	419,330	419,330
Total Damages	**15,030,829**	**15,030,829**	**15,030,829**

5.3.5 Project Benefits. Project benefits, the difference between without- and with-project damages, are about P97 million for each alternative. Benefits are equal for each project because of the assumption that each has identical outputs--protection against all events up to and including 100-year floods. Benefits are presented in table C-60 in present value terms.

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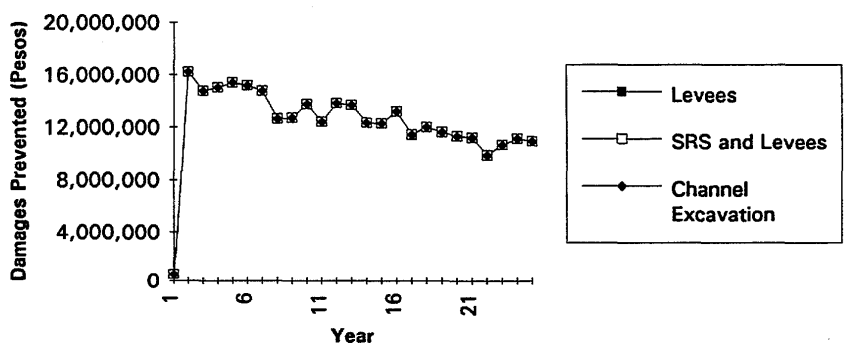
Table C-60 -- *Average Project Benefits by Category: Maloma (present value, pesos)*

Category	Sub-category	Levee	SRS	Channel Excavation
Structures	Residential	15,138,830	15,138,830	15,138,830
	Commercial	53,335,541	53,335,541	53,335,541
	Farm	2,640,368	2,640,368	2,640,368
Agriculture	Rice	6,720,957	6,720,957	6,720,957
	Sugar	0	0	0
	Fish	0	0	0
	Other Agriculture	1,903,789	1,903,789	1,903,789
Evacuation and Relocation		309,288	309,288	309,288
Foregone Production		1,048,951	1,048,951	1,048,951
Transportation Disruption		3,160,780	3,160,780	3,160,780
Other Infrastructure		8,061,248	8,061,248	8,061,248
Roads		2,271,255	2,271,255	2,271,255
Levees		3,044,349	3,044,349	3,044,349
Bridges		0	0	0
Total Benefits		**97,635,357**	**97,635,357**	**97,635,357**

Figure C-27 charts non-discounted benefits over time for each alternative. Because each alternative has the same outputs, more or less, benefits are virtually indistinguishable among the three alternatives.

Figure C-27 -- *Avoided Damages by Project: Maloma*



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5.3.6 Project Costs. Construction costs in financial terms range from P83 million for the levee alternative to P242 million for the SRS alternative. Channel excavation will cost about P136 million initially. Both the channel excavation alternative and the levee alternative will require P2.36 million annually for future maintenance excavation that is not incurred in the SRS alternative. On the other hand, the SRS alternative will require about P16 million in major maintenance expenditures every 10 years. In terms of life cycle economic costs, the present value for the levee alternative is P85 million, P185 million for the SRS, and P129 million for channel excavation (see table C-63 for cost data).

5.3.7 Investment Analysis. Table C-61 presents benefits and costs for each alternative. Note that mean project benefits exceed costs only for the levee alternative.

Table C-61 -- Present Value of Benefits and Costs: Maloma (present value, pesos)

Alternative	Project Benefits			Project Cost
	Mean	2 S.D. Above Mean	2 S.D. Below Mean	
Levee	97,635,357	160,426,311	34,844,403	85,218,000
SRS	97,635,357	160,426,311	34,844,403	184,705,000
Channel Excavation	97,635,357	160,426,311	34,844,403	128,977,000

Table C-62 provides investment statistics for each alternative. Only the levee alternative has positive net benefits and a benefit-cost ratio greater than one for the mean case.

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Table C-62 -- *Investment Statistics: Maloma*

Levee

	Mean	2 σ Above Mean	2 σ Below Mean
Net Benefits	12,417,357	75,208,311	-50,373,597
BCR	1.15	1.88	0.41
IRR	15.67%	na	na

SRS

	Mean	2 σ Above Mean	2 σ Below Mean
Net Benefits	-87,069,643	-24,278,689	-149,860,597
BCR	0.53	0.87	0.19
IRR	3.5%	na	na

Channel Excavation

	Mean	2 σ Above Mean	2 σ Below Mean
Net Benefits	-31,341,643	31,449,311	-94,132,597
BCR	0.76	1.24	0.27
IRR	7.59%	na	na

σ = standard deviation

Table C-63 -- *Alternative Costs for the Maloma Basin*

Construction Costs, Financial (Pesos)

Line Item	Levees	SRS + Levees	Channel Excavation
Profit	5,710,044	16,905,751	9,085,553
Home Overhead	1,119,617	3,314,853	1,781,481
Field Overhead	4,146,728	12,277,234	6,598,078
Vat Tax	4,712,191	13,951,402	7,497,815
Skilled Labor	1,606,802	3,215,637	2,278,294
Unskilled Labor	1,311,557	2,883,327	1,811,551
Equipment	25,119,194	55,313,501	50,591,129
Material	19,084,355	78,101,555	20,297,180
Environmental Mitigation	1,194,000	365,000	4,665,000
Subtotal	64,004,488	186,328,260	104,606,081
Contingency (30%)	19,201,346	55,898,478	31,381,824
Total First Costs	**83,205,834**	**242,226,738**	**135,987,905**

Annual Costs, financial

Item	Levees	SRS + Levees	Channel Excavation
Annual Dredging Costs			
Annual Excavation Costs	2,360,000		2,360,000
O&M	46,595	205,893	43,516
Total Annual Costs	**2,406,595**	**205,893**	**2,403,516**

Special Future Costs (every 10 years)

Item	Levees	SRS + Levees	Channel Excavation
Major Maintenance		15,990,000	

Present Value of Financial Costs, 1994 Base

First Costs	Levees	SRS + Levees	Channel Excavation
First Costs	74,290,924	193,929,251	121,417,773
Annual Costs	16,726,516	1,140,794	16,705,115
Future Special Costs	0	4,844,365	0
Total (Pesos)	**91,017,439**	**199,914,410**	**138,122,887**
Total (Dollars)	**3,371,016**	**7,404,237**	**5,115,662**

Present Value of Economic Costs, 1994 Base

First Costs	Levees	SRS + Levees	Channel Excavation
First Costs	68,516,948	178,808,532	112,294,413
Annual Costs	16,701,346	1,051,846	16,682,389
Future Special Costs	0	4,844,365	0
Total (Pesos)	**85,218,293**	**184,704,743**	**128,976,802**
Total (Dollars)	**3,156,233**	**6,840,916**	**4,776,919**

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5.4 The O'Donnell Basin

5.4.1 Problem. The risk of mudflows in the upper reaches of the O'Donnell remains high for the next 5 to 10 years. There is a moderate risk of flooding near O'Donnell and Santa Lucia because sediment deposition has filled the channel. There is a moderate risk of flow diversion towards the Bamban and Rio Chico de la Pampanga rivers resulting in shallow flooding and sediment deposition over a wide area upstream of Tarlac. The flood risk at Tarlac is low. Downstream of Tarlac, channel aggradation could damage irrigation system intakes, but this is not evaluated here due to lack of clear hydrologic evidence of the magnitude of the problem and lack of data on the irrigation systems and agriculture in this area.

5.4.2 Alternatives. The engineering alternatives under consideration for the O'Donnell are:

- Levee Alternative. Levees 4 meters high with slope and toe protection would be constructed on the right bank of the O'Donnell river from Sta. Juliana (RK 27) to the confluence with the Bangat River (RK 15.5); upstream along the left bank of the Bangat River to a point 1 km upstream of the Sta. Lucia bridge (RK 0 to RK 5.5); and on the right bank of the Bangat from RK0 to RK 4. The existing 3 meter high levee on the O'Donnell between Tarlac and high ground at RK 10 would be provided with slope and toe protection.

- Sediment Retention Structure Alternative. A 41 meter high earth and rock fill embankment dam would be constructed at RK 33 (7 km above Sta. Juliana.) Low-level weirs would be constructed at the sites of the two original sabo structures on the Bangat River. Levees would be constructed or improved as in the levee alternative except their height is 3 meters.

- Channel Excavation Alternative. Channel excavation 500 meters wide and 2 meters deep would occur between RK 14.5 and RK 27. Material would be disposed of in berms set back a distance of 100 meters from the channel. Annual removal of sediments is required to maintain protection. Slope and toe protection would be placed on the existing levee between Tarlac and RK 10.

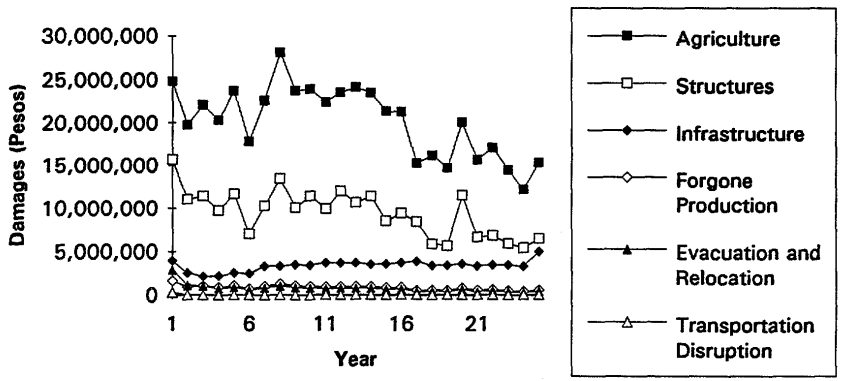
5.4.3 Without-Project Condition Damages. Table C-64 summarizes the without-project damages. Average damages in present value terms are P297 million of which 57 percent are agricultural damages.

Table C-64 -- *Mean Without-Project Damages: O'Donnell (present value, pesos)*

Category	Damages
Agriculture	170,592,632
Structures	84,718,462
Infrastructure	24,595,742
Forgone Production	8,114,424
Evacuation, Relocation and Clean-up	8,566,865
Transportation Disruption	809,992
Total Damages	**297,398,117**

Figure C-28 shows the time path of mean damages by category. Notice the peak in non-discounted damages about mid-way through the planning horizon, probably accounted for by increased flood events in the right outer Makniknik impact zone, a very large area.

Figure C-28 -- *Mean Without-Project Damages: O'Donnell*



There is a large confidence interval around these estimates relative to some of the other basins, as can be seen in figure C-29. The explanation for this large range again probably lies with the hydraulic and hydrologic model outputs which drive the economic model. Although this has not been investigated, it is likely that for any specific year there are many iterations where flooding/deposition is minimal, especially in Makniknik right outer zone, and a few iterations where significant flooding occurs. Consequently, the standard deviation may be large relative to the mean.

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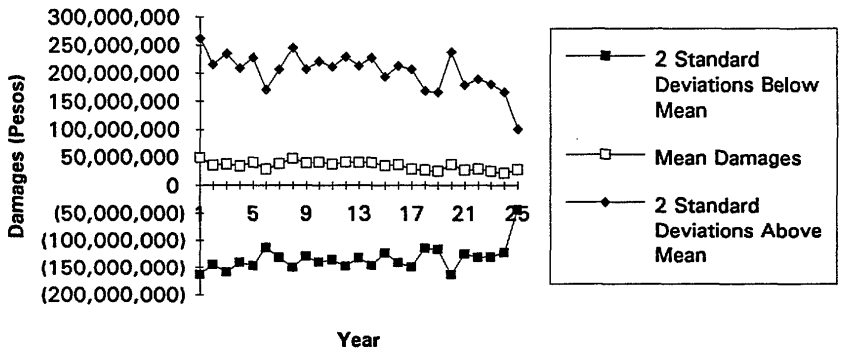
Figure C-29 – *Range of Without-Project Damages: O'Donnell*



5.4.4 With-Project Condition Damages. As table C-65 shows, the with-project (residual) damages range from P110 million for the levee alternative and the channel excavation alternative, to P48 million for the SRS alternative. (The levee and channel excavation alternatives have identical residual damages because the hydraulic and hydrologic inputs for the levee alternative also were used for the channel excavation alternative).

Table C-65 – *Mean With-Project Damages: O'Donnell (present value, pesos)*

Category	Levee	SRS	Channel Excavation
Agriculture	61,303,863	24,042,400	61,303,863
Structures	35,618,110	15,572,508	35,618,110
Infrastructure	4,795,348	3,510,131	4,795,348
Forgone Production	3,386,103	1,539,188	3,386,103
Evacuation, Relocation and Clean-up	4,640,652	2,661,139	4,640,652
Transportation Disruption	373,016	284,588	373,016
Total Damages	**110,117,093**	**47,609,954**	**110,117,093**

Figures C-30 and C-31 show average residual damages by category over time for the levee and SRS alternatives. Most of the damages are concentrated in the first decade of the planning period for the levee alternative. The results for the levee alternative apply to the channel excavation alternative as well. After the first year, the SRS provides nearly complete protection.

Figure C-30 -- *O'Donnell Levee Alternative: Average Residual Damages by Category*

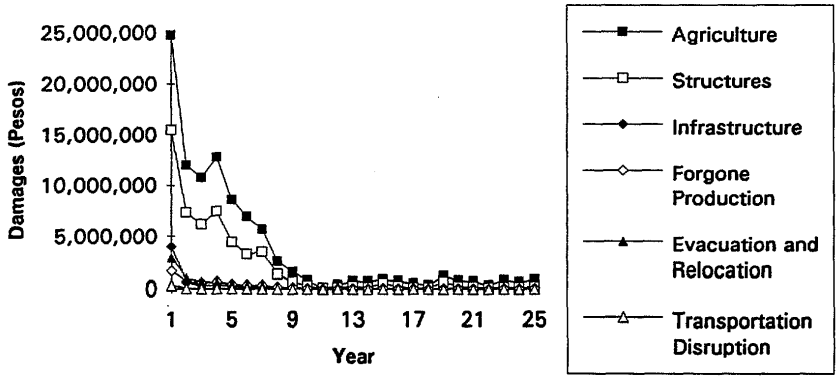
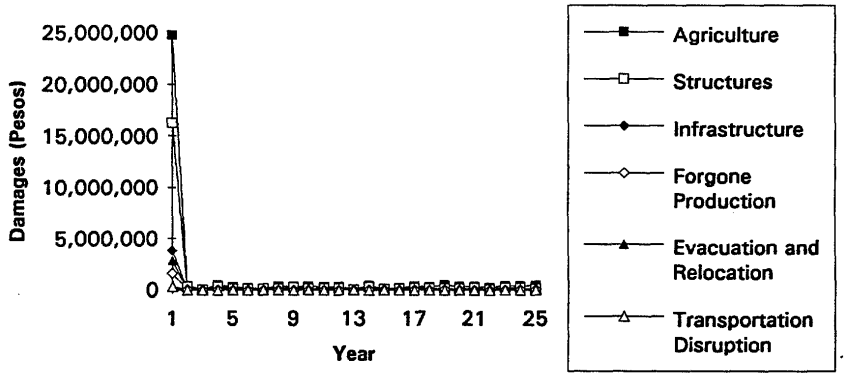


Figure C-31 -- *O'Donnell SRS Alternative: Average Residual Damages by Category*



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5.4.5 Project Benefits. Mean benefits range from P187 million for levee and channel excavation alternatives to P250 million for the SRS alternative. Table C-66 provides avoided damages by category.

Table C-66 – *Average Project Benefits by Category: O'Donnell (present value, pesos)*

Category	Sub-category	Levee	SRS	Channel Excavation
Structures	Residential	16,590,612	22,507,165	16,590,612
	Commercial	31,731,877	45,624,259	31,731,877
	Farm	777,863	1,014,529	777,863
Agriculture	Rice	53,163,819	71,454,780	53,163,819
	Sugar	55,781,258	74,643,302	55,781,258
	Fish	0	0	0
	Other Agriculture	343,693	452,150	343,693
Evacuation and Relocation		3,926,213	5,905,725	3,926,213
Foregone Production		4,728,320	6,575,236	4,728,320
Transportation Disruption		436,976	525,404	436,976
Other Infrastructure		461,322	498,819	461,322
Roads		(18,006)	250,416	(18,006)
Levees		19,357,077	20,336,376	19,357,077
Bridges		0	0	0
Total Benefits		**187,281,024**	**249,788,163**	**187,281,024**

5.4.6 Project Costs. The levee alternative is the least cost option with initial construction costs of about P223 million, compared to P3,171 million for the SRS alternative and P1,008 million for channel excavation. In addition to regular O&M costs, the channel excavation alternative will require the annual excavation of 1 million m^3 at a cost of P59 million per year. Major maintenance on the SRS is required every 10 years after project completion, at an estimated cost of P30 million. The present value of economic costs over the planning period range from P185 million for the levee alternative to P2,243 for the SRS. The channel excavation alternative falls mid-way between the two with present value of costs at P1,241 million, with about a third of that cost accounted for by future excavation costs. (See table C-**Error! Bookmark not defined.**).

5.4.7 Investment Analysis. Table C-67 compares costs and benefits and shows that the levee alternative is the only one where benefits exceed costs for the mean or high benefits case.

Table C-67 -- *Present Value of Benefits and Costs: O'Donnell*
(present value, pesos)

Alternative	Project Benefits: Mean	Project Benefits: 2 σ Above Mean	Project Benefits: 2 σ Below Mean	Project Cost
Levee	187,281,024	683,983,276	(309,421,228)	188,239,943
SRS	249,788,163	670,811,055	(171,234,728)	2,246,013,390
Channel Excavation	187,281,024	683,983,276	(309,421,228)	1,244,018,768

σ = standard deviation

Based on the foregoing costs and benefits, the investment rates of return are shown on table C-68. Based on the mean case, the levee alternative is very close to being economic with a benefit-cost ratio only slightly below one.

Table C-68 -- *Investment Statistics: O'Donnell*

Levee

	Mean	2 σ Above Mean	2 σ Below Mean
Net Benefits	-958,919	495,743,333	-497,661,171
BCR	0.99	3.63	-1.64
IRR	na	na	na

SRS

	Mean	2 σ Above Mean	2 σ Below Mean
Net Benefits	-1,996,225,227	-1,575,202,335	-2,417,248,118
BCR	0.11	0.30	-0.08
IRR	na	na	na

Channel Excavation Alternative

	Mean	2 σ Above Mean	2 σ Below Mean
Net Benefits	-1,056,737,744	-560,035,492	-1,553,439,996
BCR	0.15	0.55	-0.25
IRR	na	na	na

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Table C-69 -- *Alternative Costs for the O'Donnell Basin*

Construction Costs, Financial (Pesos)

Line Item	Levees	SRS + Levees	Channel Excavation*
Profit	15,537,525	221,816,174	70,702,332
Home Overhead	3,046,547	43,493,367	13,863,202
Field Overhead	11,283,606	161,086,546	51,345,194
Vat Tax	12,822,279	183,052,893	58,346,811
Skilled Labor	5,098,649	53,368,632	17,177,228
Unskilled Labor	1,656,378	32,258,196	6,353,223
Equipment	69,822,419	1,130,351,117	537,801,381
Material	51,645,375	614,550,981	22,136,275
Environmental Mitigation	3,164,050	1,908,375	150,000
Subtotal	174,076,828	2,441,886,281	777,875,646
Contingency (30%)	52,223,048	732,565,884	233,362,694
Total First Costs	**226,299,876**	**3,174,452,165**	**1,011,238,340**

Annual Costs, financial

Item	Levees	SRS + Levees	Channel Excavation
Annual Excavation Costs			59,000,000
O&M	226,300	3,174,452	40,450
Total Annual Costs	**226,300**	**3,174,452**	**59,040,450**

Special Future Costs (every 10 years)

Item	Levees	SRS + Levees	Channel Excavation
Major Maintenance	0	30,000,000	0

Present Value of Financial Costs, 1994 Base

First Costs	Levees	SRS + Levees	Channel Excavation
First Costs	202,053,461	2,410,480,053	902,891,375
Annual Costs	1,572,848	15,704,248	410,347,772
Future Special Costs	0	8,115,057	0
Total (Pesos)	**203,626,309**	**2,434,299,358**	**1,313,239,147**
Total (Dollars)	**7,541,715**	**90,159,236**	**48,638,487**

Present Value of Economic Costs, 1994 Base

First Costs	Levees	SRS + Levees	Channel Excavation
First Costs	186,785,942	2,223,412,826	833,692,542
Annual Costs	1,454,001	14,485,507	410,326,225
Future Special Costs	0	8,115,057	0
Total (Pesos)	**188,239,943**	**2,246,013,390**	**1,244,018,768**
Total (Dollars)	**6,971,850**	**83,185,681**	**46,074,769**

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5.5 The Pasig-Potrero Basin

5.5.1 Problem. The risk of mudflows in the Pasig basin is high for at least 10 years. Upstream of Mancatian, there is a high risk that the channel will fill with material causing mudflows and river diversions to areas adjacent to the Pasig basin. The risk of flow diversion to Porac is high, whereas the risk of flow diversion to the Abacan is low. Downstream of Mancatian, there is potential for levee breaches, flooding and sediment deposition throughout the basin. There is also a risk that sediment discharged from the Pasig will deposit in downstream channels causing flooding in the delta in or near Bacolor San Fernando, Minalin and Santo Tomas.

5.5.2 Alternatives. The two engineering alternatives under consideration for the Pasig-Potrero are summarized below.

- Levee Alternative. This alternative includes these features:

  Right bank levees from RK 20 to RK 26: A levee 10 meters high with a hardened face toe protection, and sodded back slope would be constructed on both banks following the existing levee alignment to high ground.

  Right and Left Bank Levees from RK 12 to RK 20: A levee 10 meters high with a hardened face, toe protection, and sodded back slope would be constructed on both banks following a straight alignment.

  Right Bank Levee from RK 7.5 to RK 12: A levee 10 meters high and transitioning to 7 meters at RK 7.5 with slope and toe protection, and a sodded back slope would be constructed on the right bank.

  Slope Protection: Slope and toe protection would be added to the landward side of the existing right bank levee between RK 7.5 to RK 12.

  Control Structure: A RCC control structure 7 meters high would be constructed at RK 7.5 where the right and left bank levees tie into the existing levees. the structure will allow the area between the levees from RK 7.5 to RK 12 to function as a sand pocket.

  Channel Excavation: The existing channel below RK 7.5 would be excavated an additional 2 meters and the slopes protected against erosion. The channel would be excavated to the confluence with the Sapang Labuan.

  Sump: A sump (in-channel basin) with a capacity of about 0.5 million m^3 would be dredged in the Sapang Labuan below the mouth of the Pasig-Potrero.

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- Channel Excavation Alternative. This alternative includes these features:

Channel Excavation: A channel would be excavated 3 meters deep and 700 to 1,800 meters wide extending from about 3 km upstream to about 4 km downstream of Mancatian. Excavated material would be disposed of as a berm paralleling the channel on both sides and set back 100 meters from the channel.

Right and Left Bank Levees from RK23 to RK 26: A levee 6 meters high with a hardened face, toe protection and sodded back slope would be constructed on both banks following the exiting levee alignment beginning upstream of the disposal berm, and extending upstream about 3.5 to high ground.

Right Bank Levee from RK 7.5 to RK 14: A 3 meter high levee with slope and toe protection, and a sodded back slope would be constructed on the right bank following a straight alignment from Mancatian towards the existing levee at Santa Rita.

Left Bank Levee from RK 12 to RK 14: A 3 meter high levee with slope and toe protection, and a sodded back slope would be constructed on the left bank following existing alignment from the downstream side of the left disposal berm and tying into the existing right bank levee.

Slope protection, channel excavation, and a sump as described for the levee alternative also would be included in this alternative.

5.5.3 Without-Project Condition Damages. As shown in table C-70, under the without-project condition the present value of damages over the 25 year horizon are P855 million. Structures account for about 47 percent of this total.

Table C-70 -- *Mean Without-Project Damages: Pasig-Potrero (present value, pesos)*

Category	Damages
Agriculture	206,115,064
Structures	405,589,442
Infrastructure	82,047,226
Forgone Production	54,529,200
Evacuation, Relocation and Clean-up	101,080,682
Transportation Disruption	6,187,035
Total Damages	**855,548,649**

Figure C-32 and C-33 show average non-discounted damages by category and the variance of total damages about the mean, respectively.

Figure C-32 – *Mean Without-Project Damages: Pasig-Potrero*

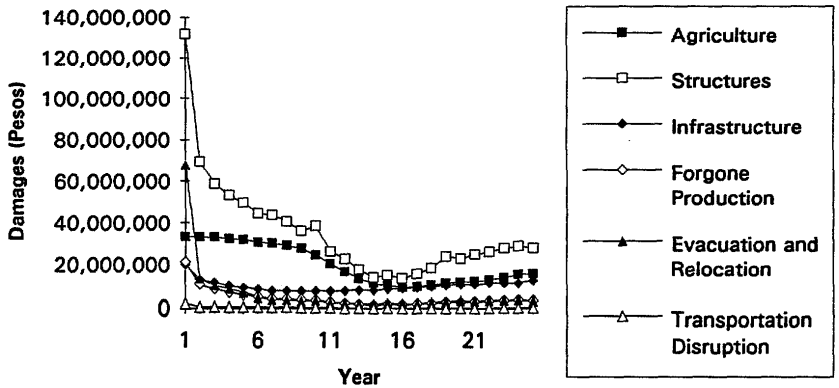
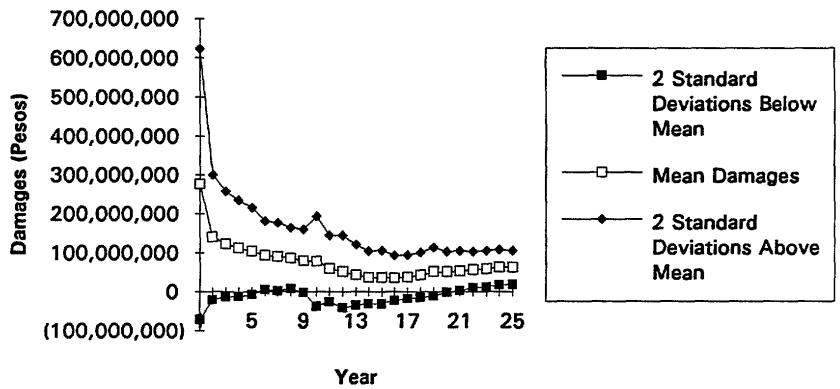


Figure C-33 – *Range of Without-Project Damages: Pasig-Potrero*



5.5.4 With-Project Condition Damages. All alternatives provide at least the following outputs:

- Containment of 100-year flow.
- Storage to retain or control expected sediments during the first 10 years.
- Transport or storage capacity to contain sediment produced during a 100-year storm event.

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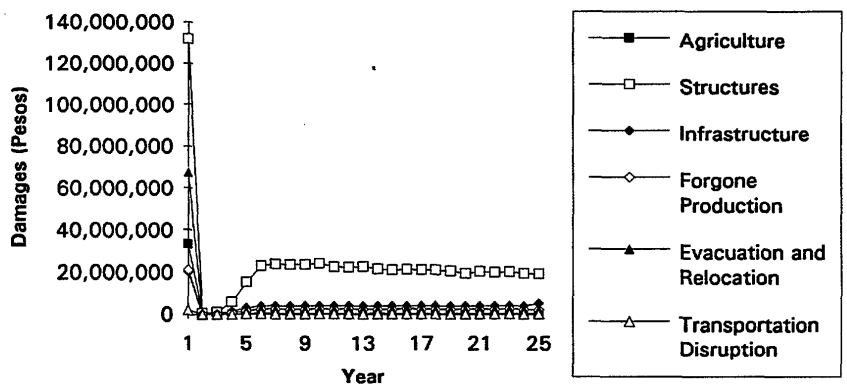
Based on these project capabilities, expected damages are reduced to P402 million over the planning period. Structures continue to dominate damages, now accounting for over 60 percent of damages. Hydraulic and hydrologic simulated data was provided only for the levee alternative and it is assumed that the channel excavation alternative has similar outputs.

Table C-71 – *Mean With-Project Damages: Pasig-Potrero (present value, pesos)*

	Levee	Channel Excavation
Agriculture	32,165,290	32,165,290
Structures	225,701,425	225,701,425
Infrastructure	37,725,687	37,725,687
Forgone Production	30,658,774	30,658,774
Evacuation, Relocation and Clean-up	73,005,451	73,005,451
Transportation Disruption	3,476,688	3,476,688
Total Damages	**402,733,315**	**402,733,315**

Figure C-34 shows the time path of expected damages for the levee alternative.

Figure C-34 – *Pasig-Potrero Levee Alternative: Residual Damages*



5.5.5 Project Benefits. As shown on table C-72, both alternatives provide P658 million in average avoided damages. In addition to these benefits, both alternatives are credited with benefits equal to the value of 500,000 m³ of reduced delta dredging for a present value of P204 million.

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Table C-72 -- *Average Project Benefits by Category: Pasig-Potrero (present value, pesos)*

Category	Sub-category	Levee	Channel Excavation
Structures	Residential	76,128,804	76,128,804
	Commercial	102,750,258	102,750,258
	Farm	1,008,956	1,008,956
Agriculture	Rice	35,125,233	35,125,233
	Sugar	21,439,800	21,439,800
	Fish	117,073,013	117,073,013
	Other Agriculture	311,728	311,728
Evacuation and Relocation		28,075,231	28,075,231
Foregone Production		23,870,426	23,870,426
Transportation Disruption		2,710,347	2,710,347
Other Infrastructure *		206,634,796	206,634,796
Roads		9,127,239	9,127,239
Levees		33,580,288	33,580,288
Bridges		12,533	12,533
Total Benefits		**657,848,652**	**657,848,652**

* Includes avoided dredging in the delta of 500,00 m^3 per year at a unit cost of P59/ m^3.

5.5.6 Project Costs. Construction costs for the two alternatives are very close, P1,735 million for the levee alternative and P1,595 million for the channel excavation alternative. The latter alternative, however, has higher life-cycle costs in present value terms because of annual excavation costs of P83 million which are not incurred in the levee alternative. It is estimated that 1.4 million m^3 per year will be excavated annually over the planning horizon in the excavation alternative. Annual sump costs are estimated at P30 million, with dredging quantity of 500,000 m^3 per year. The present value of economic costs for the levees alternative is P1,548 million and for the channel excavation alternative is P1,942 million. (See table C-75 for cost data).

5.5.7 Investment Analysis. As shown in table C-73, benefits are estimated to range from a low of about P410 million to a high of P906 million, with a mean value of P658 million.[56] Estimated life cycle costs for both alternatives lie above this range.

[56]The low number represents 2 standard deviation below the mean and the high is 2 standard deviations above the mean.

Table C-73 -- *Present Value of Benefits and Costs: Pasig-Potrero*
(present value, pesos)

Alternative	Project Benefits			Project Cost
	Mean	2 σAbove Mean	2 σ Below Mean	
Levee	657,848,652	905,851,212	409,846,091	1,548,625,663
Channel Excavation	657,848,652	905,851,212	409,846,091	1,942,999,507

Table C-74 provides investment decision data which concludes that neither alternative is economically efficient.

Table C-74 -- *Investment Statistics: Pasig-Potrero*

Levee Alternative

	Mean	High	Low
Net Benefits	(890,777,012)	(642,774,451)	(1,138,779,572)
BCR	0.42	0.58	0.26
IRR	na	na	na

Channel Excavation Alternative

	Mean	High	Low
Net Benefits	(1,285,150,855)	(1,037,148,295)	(1,533,153,416)
BCR	0.34	0.47	0.21
IRR	na	na	na

Table C-75 -- *Alternative Costs for the Pasig-Potrero Basin*

Construction Costs, Financial (Pesos)

	Levee	Channel Excavation
Profit	120,831,021	111,010,580
Home Overhead	23,692,357	21,766,780
Field Overhead	87,749,470	80,617,705
Vat Tax	99,715,307	91,611,029
Skilled Labor	36,099,980	30,300,821
Unskilled Labor	12,685,899	7,787,548
Equipment	722,300,953	803,340,667
Material	226,066,240	74,681,250
Environmental Mitigation	5,690,000	5,690,000
Subtotal	1,334,831,227	1,226,806,380
Contingency (30%)	400,449,368	368,041,914
Total First Costs	**1,735,280,595**	**1,594,848,294**

Annual Outyear Costs

Item	Levee	Channel Excavation
Dredging Costs	29,500,000	29,500,000
Excavation Costs	0	82,600,000
O&M	1,735,281	350,867
Total Annual Costs	**31,235,281**	**112,450,867**

Special Future Costs (every 10 years)

Item	Levee	Channel Excavation
Control Structure Maintenance	4,680,000	0

Present Value of Financial Costs, 1994 Base

	Levees	Excavation + Levees
First Costs	1,466,356,370	1,347,687,493
Annual Costs	207,504,787	747,042,853
Future Special Costs	1,588,006	0
TotalCost (Pesos)	**1,675,449,164**	**2,094,730,347**
Total Cost (Dollars)	**62,053,673**	**77,582,605**

Present Value of Economic Costs, 1994 Base

	Levee	Channel Excavation
First Costs	1,354,028,622	1,245,338,764
Annual Costs	193,009,035	697,660,743
Future Special Costs	1,588,006	0
TotalCost (Pesos)	**1,548,625,663**	**1,942,999,507**
Total Cost (Dollars)	**57,356,506**	**71,962,945**

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5.6 The Porac-Gumain Basin

5.6.1 Problem. On the Gumain basin, the risk of mudflows is low because the upper drainage does not contain significant pyroclastic deposits. The threat of flooding is high because much of the channel is filled with sediment. There is a high potential for diversion of the Gumain flows into the Caulaman River because the channel filled with sediment and levees have been damaged. Bank erosion, flooding and channel meandering are local problems in and around Floridablanca. There is a high risk of levee breaches and shallow flooding downstream of Floridablanca because of potential bank erosion.

In the Porac basin the flood risk is considered to be near pre-eruption levels but local bank erosion and channel alignment problems are expected. There is a high risk that sediment may deposit in the fan at the mouth of the Gumain floodway and in downstream delta channels, causing ponding-type flooding in the delta.

5.6.2 Alternatives. Three structural alternatives are being considered for the Gumain-Porac basin, as summarized below.

- Levee Alternative. Levees 3 meters high would be constructed on the right and left bank from Pabanlag to the confluence with the Porac River. The Gumain channel would be excavated between the Porac and the Pasag rivers. Bank protection would be placed along the Porac River from the diversion structure (RK 3.5) and continuing upstream through San Francisco. Annual removal of about 200,000 m^3 of sediments will be required to maintain the level of protection.

- Sediment Retention Structure Alternative. In addition to the levees, excavation and bank protection described for the levee alternative, a 30 meter high RCC dam would be constructed at RK 23.5 on the Gumain, and a weir structure 6.5 meters high constructed at RK 18 west of Basa Air Base .

- Channel Excavation Alternative. A channel 200 meters wide and 2 meters deep would be excavated from the confluence of the Gumain-Dalan Bapor with the Pasag River and would continue upstream to RK 9.5. Above RK 9.5 to RK 16.5, the channel would be excavated to 200 meters wide and 1 meter deep. Excavated materials would be deposited on the river banks and set back 100 meters from the channel. Bank protection will be placed along the Porac River beginning at the Porac diversion structure (RK 3.5) and continuing upstream for 6 km through San Francisco. Annual removal of about 200,000 m^3 of sediments will be required to maintain the level of protection.

5.6.3 Without-Project Condition Damages. As shown on table C-76, the present value of without-project damages is P1,023 million, with damages to structures accounting for more than two-thirds of that total.

Table C-76 -- *Mean Without-Project Damages: Porac-Gumain (present value, pesos)*

Category	Damages
Agriculture	228,101,740
Structures	725,868,332
Infrastructure	55,424,056
Forgone Production	6,987,135
Evacuation, Relocation and Clean-up	5,642,298
Transportation Disruption	515,920
Total Damages	**1,022,539,480**

Figure C-35 shows how average damages (non-discounted) vary over time by category.

Figure C-35 -- *Mean Without-Project Damages by Category: Porac-Gumain*

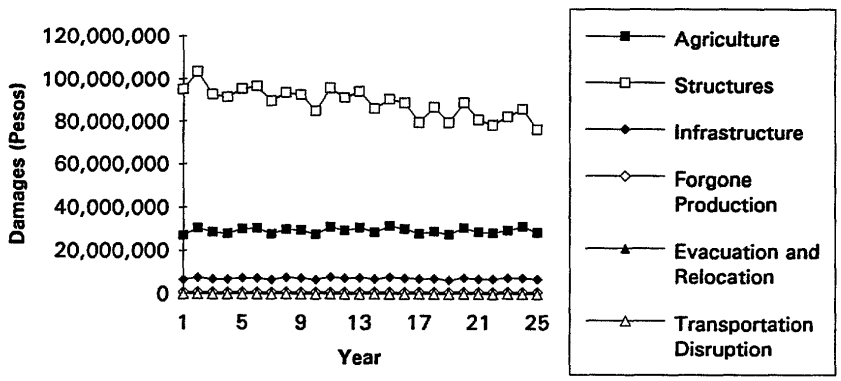

Figure C-36 presents mean damages and two standard deviations above and below the mean damages which demonstrates the volatility of the results in the simulation.

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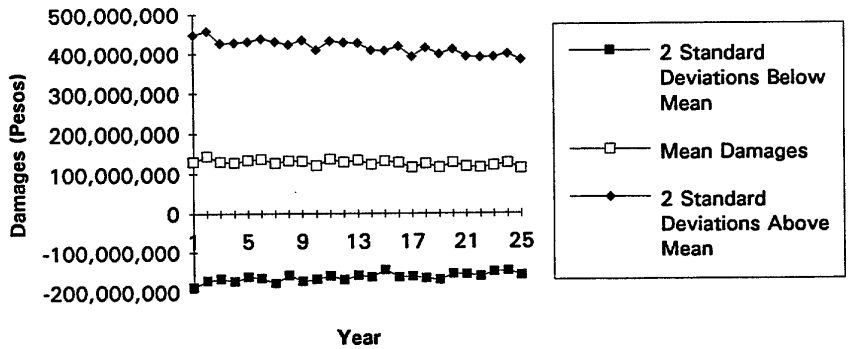
Figure C-36 -- *Range of Without-Project Damages: Porac-Gumain*



5.6.4 With-Project Condition Damages. It is estimated that each alternative will provide protection against a 100-year flood event, and expected flood damages are reduced to a present value of almost P129 million, as shown in table C-77.

Table C-77 -- *Average With-Project Damages: Porac-Gumain (present value, pesos)*

	Levee	SRS	Channel Excavation
Agriculture	26,578,310	26,578,310	26,578,310
Structures	94,389,428	94,389,428	94,389,428
Infrastructure	6,521,105	6,521,105	6,521,105
Forgone Production	823,693	823,693	823,693
Evacuation, Relocation and Clean-up	682,976	682,976	682,976
Transportation Disruption	62,226	62,226	62,226
Total Damages	**129,057,739**	**129,057,739**	**129,057,739**

5.6.5 Project Benefits. Avoided damages, which are project benefits, are P893 million for each alternative. One additional benefit, reduced delta dredging, is conferred to the channel excavation and levee alternatives. The removal of channel material in these two alternatives will prevent the movement of 200,000 m^3 of sand and silts to the delta each year over the planning period, thereby reducing future annual dredging in the delta by that amount, at a savings in present value terms of P82 million.[57] Including that benefit increases channel excavation and levee alternative benefits to P975 million. Table C-78 presents avoided damages by category.

[57]The SRS alternative would eventually reduce delta dredging, reducing the amount of material that would move downstream of the dam, through the channel and into the delta. This benefit, however, would not be enjoyed until some uncertain date in the future and for that reason are excluded from this analysis.

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Table C-78 -- *Average Project Benefits by Category: Porac-Gumain (present value, pesos)*

Category	Sub-category	Levee	SRS	Channel Excavation
Structures	Residential	214,202,513	214,202,513	214,202,513
	Commercial	416,702,484	416,702,484	416,702,484
	Farm	573,907	573,907	573,907
Agriculture	Rice	37,737,820	37,737,820	37,737,820
	Sugar	26,165,486	26,165,486	26,165,486
	Fish	134,614,767	134,614,767	134,614,767
	Other Agriculture	3,005,357	3,005,357	3,005,357
Evacuation and Relocation		4,959,322	4,959,322	4,959,322
Foregone Production		6,163,442	6,163,442	6,163,442
Transportation Disruption		453,694	453,694	453,694
Roads		12,500,299	12,500,299	12,500,299
Other Infrastructure *		101,314,835	19,301,508	101,314,835
Levees		17,101,145	17,101,145	17,101,145
Bridges		0	0	0
Total Benefits		**975,495,068**	**893,481,741**	**975,495,068**

* Includes avoided annual dredging in the Delta of 200,000 m^3 at a unit cost of 59 pesos per m^3 over the horizon for the levee alternative and channel excavation alternative.

5.6.6 Project Costs. Initial construction costs for the three alternatives range from P580 million for channel excavation, P610 million for levees, to P1,439 million for the SRS. Annual excavation of 200,000 m^3 will be required for the channel excavation and levees alternatives, at a cost of P12 million per year. The sediment retention structure will require periodic major maintenance of P30 million every decade. In terms of present value of lifetime economic costs, the channel excavation is the least costly option at P561 million, followed by levees at P587 million, and SRS at P1,139 million.

5.6.7 Investment Analysis. For the mean case, benefits exceed costs for the levee alternative and channel excavation alternative (see table C-79). Both alternatives include avoided damages plus avoided future delta dredging costs of P82 million. Costs exceed mean benefits for the SRS alternative. The addition of avoided dredging cost, benefits to the SRS would not change the direction of this inequality for the mean case.

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Table C-79 -- *Present Value of Benefits and Costs: Porac-Gumain (pesos)*

Alternative	Project Benefits: Mean	Project Benefits: 2 σ Above Mean	Project Benefits: 2 σ Below Mean	Project Cost
Levee	975,495,068	1,631,368,118	319,622,018	587,175,570
SRS	893,481,741	1,549,354,790	237,608,692	1,138,975,493
Channel Excavation	975,495,068	1,631,368,118	319,622,018	561,185,990

σ = Standard Deviation

Table C-80 provides investment statistics for each alternative. Both the levee and the channel excavation alternatives have positive net benefits for the average case. The levee alternative has slightly more impressive rates of return. The SRS alternative is not economic for the mean case but is economic when benefits are two standard deviations above the mean.

Table C-80 -- *Investment Statistics: Porac-Gumain*

Levee

	Mean	2 σ Above Mean	2 σ Below Mean
Net Benefits	414,309,078	1,070,182,128	-241,563,972
BCR	1.74	2.91	0.57
IRR	23.0%	na	na

SRS

	Mean	2 σ Above Mean	2 σ Below Mean
Net Benefits	-245,493,752	410,379,297	-901,366,801
BCR	0.78	1.36	0.21
IRR	8.4%	na	na

Channel Excavation

	Mean	High	Low
Net Benefits	388,319,498	1,044,192,548	-267,553,552
BCR	1.66	2.78	0.54
IRR	24.3%	na	na

Table C-81 -- *Alternative Costs for the Porac-Gumain*

Financial Construction Costs (Pesos)

Item	Channel Excavation	Levees	SRS + Levees
Profit	40,557,912	42,668,042	100,576,305
Home Overhead	7,952,532	8,366,283	19,720,844
Field Overhead	29,453,821	30,986,232	73,040,164
Vat Tax	33,470,251	35,211,628	83,000,186
Skilled Labor	10,678,447	12,946,047	18,985,883
Unskilled Labor	4,924,529	4,963,413	10,707,397
Equipment	252,254,736	239,706,815	394,318,277
Material	66,844,800	94,500,000	405,990,300
Environmental Mitigation	150,000	150,000	200,000
Subtotal	446,287,028	469,498,460	1,106,539,356
Contingency (30%)	133,886,108	140,849,538	331,961,807
Total First Costs	**580,173,136**	**610,347,998**	**1,438,501,163**

Annual Costs

Item	Channel Excavation	Levees	SRS + Levees
Dredging Costs	0	0	0
Excavation Costs	11,800,000	11,800,000	0
O&M	179,854	347,898	1,179,571
Total Annual Costs	11,979,854	12,147,898	1,179,571

Other Outyear Costs

	Channel Excavation	Levees	SRS + Levees
Major Maintenance Every 10 Years			30,000,000

Present Value of Financial Costs, 1994 Base

	Channel Excavation	Levees	SRS + Levees
First Costs	518,011,729	544,953,570	1,215,570,179
Annual Costs	83,263,361	84,431,319	7,319,956
Future Special Costs	0	0	10,179,528
Total (Pesos)	**601,275,090**	**629,384,889**	**1,233,069,663**
Total costs, dollars	**22,269,448**	**23,310,551**	**45,669,247**

Present Value of Economic Costs, 1994 Base

	Channel Excavation	Levees	SRS + Levees
Present Value of Total First Costs	478,019,136	502,930,709	1,122,039,236
Present Value of Annual Costs	83,166,853	84,244,861	6,756,729
Future Special Costs	0	0	10,179,528
Total (Pesos)	**561,185,990**	**587,175,570**	**1,138,975,493**
Total costs, dollars	**20,784,666**	**21,747,243**	**42,184,278**

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5.7 The Sacobia-Bamban Basin

5.7.1 Problem. In the Sacobia-Bamban basin, there is a high risk of mudflows caused by erosion of the pyroclastic deposits in the upper drainage for the next 10 years. Mudflow deposition is expected to occur mainly from the upstream end of Clark AFB to downstream of Bamban/Dolores. There is a high risk of shallow flooding and sediment deposition on the south overbank downstream of Dolores but upstream of the San Francisco bridge. A moderate risk of shallow flooding due to levee breaching exists downstream of the San Francisco bridge.

5.7.2 Alternatives. The alternatives being considered for the Sacobia-Bamban basin are summarized below.

- Levees Alternative. The principal features include:

  A right bank levee from 3.5 km upstream of Highway 3 extending in a curving fashion to the San Francisco bridge. Height would vary from 10 to13 meters.

  A 6 to 13 meter high middle levee on the left bank of the Sacobia, separating the Sacobia from the Sapang Cauayan River and the Marimla River.

  Reconstruction of the existing left bank Bamban levee.

  A control structure.

  Seeding of the left and right bank levees below the San Francisco Bridge.

- Sediment Retention Structure Alternative. The main features are:

  Two retention structures, one located 1 km upstream of the Highway 3 alignment with a height of 17 meters, and the other 6 km upstream having a height of 24 meters.

  Levees as in the levee alternative but at a reduced height, 3 to 5 meters.

  Reconstruction of the existing left bank levee and seeding as in the levee alternative.

- Channel Excavation Alternative. The components of this alternative are:

  Excavation of a channel 500 to 800 meters wide and 4 meters deep from 1.5 km. upstream of the Highway 3 to 7 km downstream of the highway.

  Construction of a 4 meter high right bank levee.

Reconstruction of the existing left bank levee.

Seeding of the left and right bank levees below the San Francisco bridge.

Annual Excavation.

5.7.3 Without-Project Condition Damages. The present value of without-project damages is P790 million and is broken down by category in table C-82.

Table C-82 -- *Mean Without-Project Damages: Sacobia-Bamban (present value, pesos)*

Category	Damages
Agriculture	303,303,635
Structures	282,019,032
Infrastructure	123,462,226
Forgone Production	26,991,873
Evacuation and Relocation	36,285,802
Transportation Disruption	18,241,341
Total Damages	**790,303,910**

Figure C-37 presents non-discounted damages over the planning period.

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Figure C-37 – *Mean Without-Project Damages: Sacobia-Bamban*

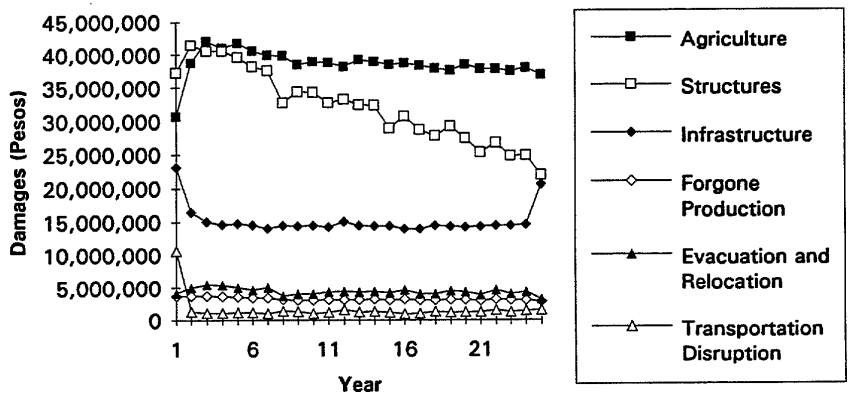
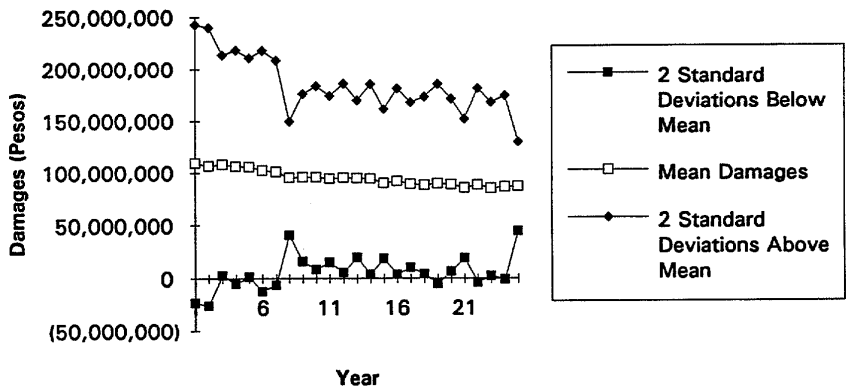


Figure C-38 presents mean damages and two standard deviations above and below the mean damages.

Figure C-38 – *Range of Without-Project Damages: Sacobia Bamban*



5.7.4 With-Project Condition Damages. All alternatives provide at least the following outputs:

- Containment of 100-year flow.
- Storage to retain or control expected sediments during the first 10 years.
- Transport or storage capacity to contain sediment produced during a 100-year storm event.

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Residual damages range from P356 million for the levee and channel excavation alternatives to P439 million for the SRS alternative. Simulations of damages were executed only for the levee alternative and the SRS alternative. No hydraulic and hydrologic input files were available for the channel excavation alternative, so for that case, damages are assumed equal to the levee alternative.

Table C-83 – *Mean With-Project Damages: Sacobia-Bamban (present value, pesos)*

	Levee	SRS	Channel Excavation
Agriculture	145,589,675	224,376,696	145,589,675
Structures	114,229,076	110,827,350	114,229,076
Infrastructure	54,035,089	55,932,091	54,035,089
Forgone Production	13,757,895	18,966,209	13,757,895
Evacuation and Relocation	13,102,946	14,752,887	13,102,946
Transportation Disruption	15,308,517	13,785,619	15,308,517
Total Damages	**356,023,198**	**438,640,852**	**356,023,198**

Figures C-39 and C-40 show how residual damages vary over the planning horizon for the levee and SRS alternatives. (The channel excavation alternative is similar to the levee alternative.) Based on a comparison of residual damages with without-project damages, it appears that each alternative delays the onset of damages to about 9 or 10 years into the future.

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Figure C-39 -- *Sacobia-Bamban Levee Alternative: Residual Damages*

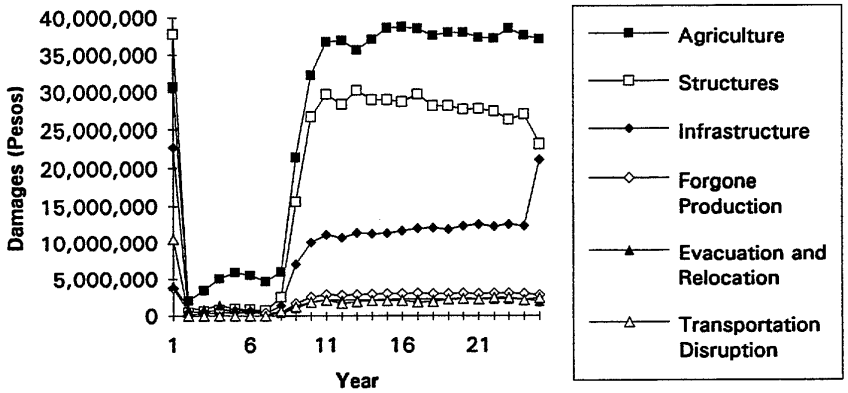
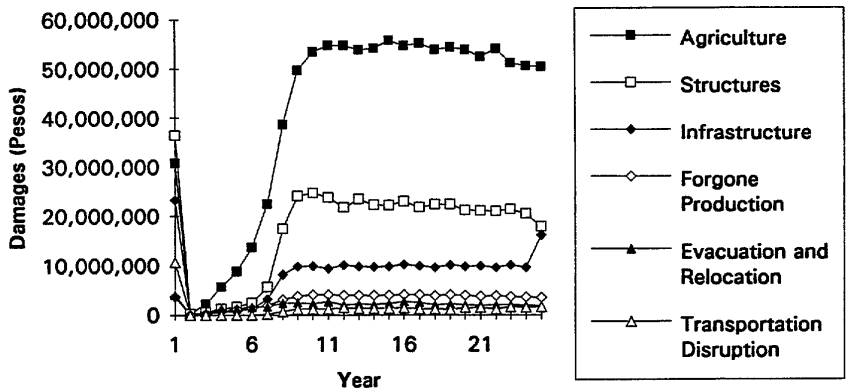


Figure C-40 -- *Sacobia-Bamban SRS: Residual Damages*



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5.7.5 Project Benefits. Table C-84 summarizes project benefits for each alternative. Avoided losses range from about P625 million for the levee and channel excavation alternatives to P508 million for the SRS alternative.

Table C-84 – *Average Project Benefits by Category: Sacobia-Bamban (present value, pesos)*

Category	Sub-category	Levee	SRS	Channel Excavation
Structures	Residential	76,843,630	76,736,221	76,843,630
	Commercial	79,329,410	82,446,322	79,329,410
	Farm	11,616,915	12,009,140	11,616,915
Agriculture	Rice	67,841,593	36,779,229	67,841,593
	Sugar	78,432,976	37,642,029	78,432,976
	Fish	10,794,405	4,081,739	10,794,405
	Other Ag	644,986	423,942	644,986
Evacuation and Relocation		23,182,856	21,532,915	23,182,856
Foregone Production		13,233,978	8,025,664	13,233,978
Transportation Disruption		2,932,824	4,455,722	2,932,824
Other Infrastructure		16,257,617	18,822,906	16,257,617
Roads		2,330,918	1,841,100	2,330,918
Levees		49,323,723	45,666,625	49,323,723
Bridges		1,514,879	1,199,503	1,514,879
Total Benefits		**432,765,833**	**350,463,554**	**432,765,833**

5.7.6 Project Costs. Both in terms of first costs and the present value of lifetime costs, the channel excavation alternative is the least expensive. Initial costs for this alternative is P490 million versus P1,365 million for the levee alternative and P1,988 million for the SRS (including contingency). The channel excavation alternative will require outyear funding of P189 million per year for additional excavation work.[58] The present value of costs for the excavation alternative is P1,555 million compared to P1,078 million for the levee alternative and P1,410 million for the SRS. (See table C-87 for cost data).

5.7.7 Investment Analysis. Table C-85 summarizes benefits and costs for the alternatives in present value terms. For no alternatives do benefits exceed costs, even for two standard deviations above the mean. One reason for this may be because so much damage has already occurred in this basin.

[58]This assumes that 3.2 million cubic meters per year on average will be excavated over the project life.

Table C-85 -- *Present Value of Benefits and Costs: Sacobia-Bamban (present value, pesos)*

Alternative	Project Benefits: Mean	Project Benefits: 2 σ Above Mean	Project Benefits: 2 σ Below Mean	Project Cost
Levee	434,280,711	677,163,168	191,398,255	1,078,698,026
SRS	351,663,057	634,951,685	68,374,430	1,410,232,575
Channel Excavation	434,280,711	677,163,168	191,398,255	1,555,712,949

σ = standard deviation

Based on the foregoing data, investment statistics have been computed and are compiled in table C-86. No alternative is economic for the given range of net benefits.

Table C-86 -- *Investment Statistics: Sacobia-Bamban*

Levee Alternative

	Mean	2 σ Above Mean	2 σ Below Mean
Net Benefits	(644,417,315)	(401,534,859)	(887,299,771)
BCR	0.40	0.63	0.18
IRR	na	na	na

MRS and Levees Alternative

	Mean	2 σ Above Mean	2 σ Below Mean
Net Benefits	(1,058,569,518)	(775,280,890)	(1,341,858,145)
BCR	0.25	0.45	0.05
IRR	na	na	na

Channel Excavation Alternative

	Mean	2 σ Above Mean	2 σ Below Mean
Net Benefits	(1,121,432,238)	(878,549,781)	(1,364,314,694)
BCR	0.28	0.43	0.12
IRR	na	na	na

Table C-87 -- *Alternative Costs for the Sacobia-Bamban Basin*

Construction Costs, Financial (Pesos)

Line Item	Channel Excavation	Levee	SRS
Profit	33,548,496	94,695,212	138,244,012
Home Overhead	6,578,137	18,567,689	27,106,669
Field Overhead	24,363,469	68,769,217	100,395,071
Vat Tax	27,685,760	78,146,837	114,085,308
Skilled Labor	10,588,969	29,059,407	31,064,777
Unskilled Labor	1,711,390	8,591,676	18,034,832
Equipment	242,486,337	543,866,228	539,523,003
Material	22,070,900	199,951,060	552,230,460
Environmental Mitigation	8,150,000	8,180,000	8,200,000
Subtotal	377,183,458	1,049,827,326	1,528,884,132
Contingency (30%)	113,155,037	314,948,198	458,665,240
Total First Costs	**490,338,495**	**1,364,775,524**	**1,987,549,372**

Annual Costs, financial

Item	Excavation	Levee	SRS
Annual Dredging Costs	0	0	0
Annual Excavation Costs	188,800,000	0	0
O&M	93,164	1,364,776	1,987,549
Total Annual Costs	**188,893,164**	**1,364,776**	**1,987,549**

Special Future Costs (every 10 years)

Item	Channel Excavation	Levee	SRS
Control Structure, MRS Maintenance	0	15,730,000	30,000,000

Present Value of Financial Costs, 1994 Base

Item	Channel Excavation	Levee	SRS
First Costs	414,348,537	1,153,270,133	1,509,220,446
Annual Costs	1,172,197,102	8,469,263	9,832,553
Future Special Costs	0	5,337,466	8,115,057
Total (Pesos)	**1,586,545,639**	**1,167,076,862**	**1,527,168,056**
Total (Dollars)	**58,760,950**	**43,225,069**	**56,561,780**

Present Value of Economic Costs, 1994 Base

Item	Channel Excavation	Levee	SRS
First Costs	383,558,808	1,065,535,593	1,393,041,867
Annual Costs	1,172,154,141	7,824,968	9,075,651
Future Special Costs	0	5,089,764	8,115,057
Total (Pesos)	**1,555,712,949**	**1,078,450,325**	**1,410,232,575**
Total (Dollars)	**57,618,998**	**39,942,605**	**52,230,836**

5.8 The Santo Tomas Basin

5.8.1 Problem. About 130 million m^3 of sediment is expected to erode over the next 10 years. Because the river has filled in the San Rafael and Santa Fe areas, a very high risk exists for the river to overtop the levees and exit the channel to the south, causing shallow flooding and sedimentation. Most of the deposits will fall in the San Rafael area, but substantial sedimentation and flooding is expected in the San Marcelino area.

5.8.2 Alternatives. The alternatives being considered are:

- Levee Alternative. On the left bank, a new levee with a hardened face, toe protection and sodded back slope would be constructed between Lawin and RK 12.5 on the river side of the existing levee. The levee would have a height of 13 meters, tapering to 9 meters at the downstream end. The existing 3-meter high levee between Lawin and Vega Hill would also be reconstructed. In addition, the existing left bank levee downstream of Vega Hill would be replaced with a hardened levee following the existing alignment at the upper end but with a new, straight alignment at the lower stretch.

  On the right bank a hardened levee would be built along a straight alignment from the Highway 7 bridge to high ground at RK 9

- Sediment Retention Structure Alternative. This alternative combines a 45 meter high structure at RK 7 on the Marella and levees on the downstream areas to provide protection prior to construction of the SRS. The levees would follow the alignments as outlined in the levee alternative.

- Channel Excavation Alternative. A channel 1 km wide by 10 km long would be excavated 4 meters deep between RK 12 and RK 21. The existing left bank levee between Lawin and Vega Hill would be reconstructed, and a 3-meter high right bank levee would be constructed following the existing alignment.

5.8.3 Without-Project Condition Damages. Average without-project damages total P1,244 million in present value terms. Damages to structures dominate, accounting for about 60 percent of the total. Table C-88 shows the distribution of damages by category, and figures C-41 and C-42 show how without-project damages vary over time and the dispersion of the simulation results.

Table C-88 -- *Mean Without-Project Damages: Santo Tomas (present value, pesos)*

Category	Damages
Agriculture	264,353,906
Structures	734,843,635
Infrastructure	40,949,973
Forgone Production	99,595,198
Evacuation and Relocation	98,030,789
Transportation Disruption	6,718,619
Total Damages	**1,244,492,120**

Figure C-41 -- *Mean Without-Project Damages by Category: Santo Tomas*

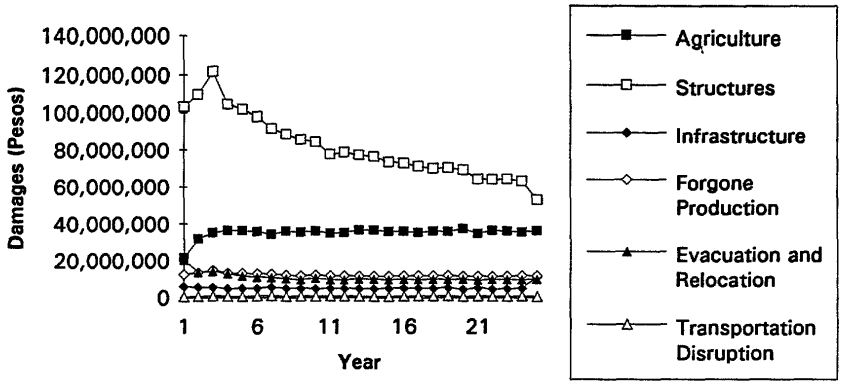

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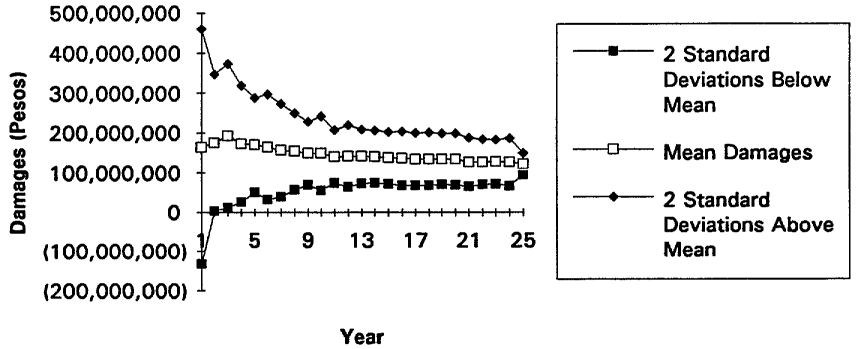
Figure C-42 – *Range of Without-Project Damages: Santo Tomas*



5.8.4 With-Project Condition Damages. All alternatives provide at least the following outputs:

- Containment of 100-year flow.
- Storage to retain or control expected sediments during the first 10 years.
- Transport or storage capacity to contain sediment produced during a 100-year storm event.

Simulations of damages were executed only for the levee and SRS alternatives. No hydraulic and hydrologic input files were available for the channel excavation alternative, so for that case, damages are assumed equal to the levee alternative. Based on these simulations, average residual damages range from about P337 million for the levee alternative to P521 million for the SRS alternative (see table C-89).

Table C-89 – *Mean With-Project Damages: Santo Tomas (present value, pesos)*

	Levee	SRS	Channel Excavation
Agriculture	53,685,477	98,070,063	53,685,477
Structures	198,754,658	295,487,337	198,754,658
Infrastructure	24,131,765	47,888,304	24,131,765
Forgone Production	25,955,028	32,000,000	25,955,028
Evacuation and Relocation	29,552,299	36,100,877	29,552,299
Transportation Disruption	4,922,844	11,844,830	4,922,844
Total Damages	**337,002,071**	**521,391,411**	**337,002,071**

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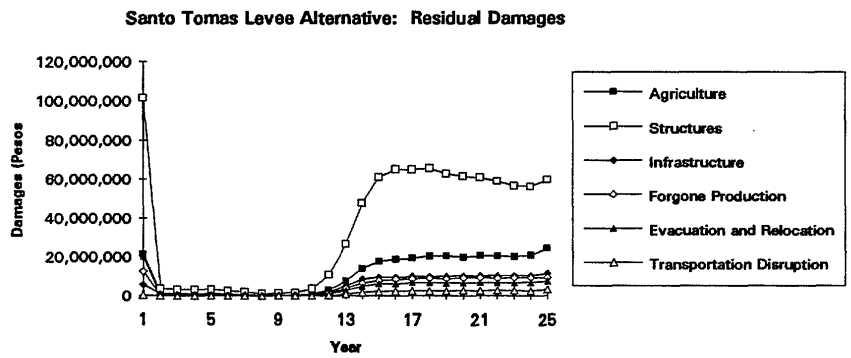
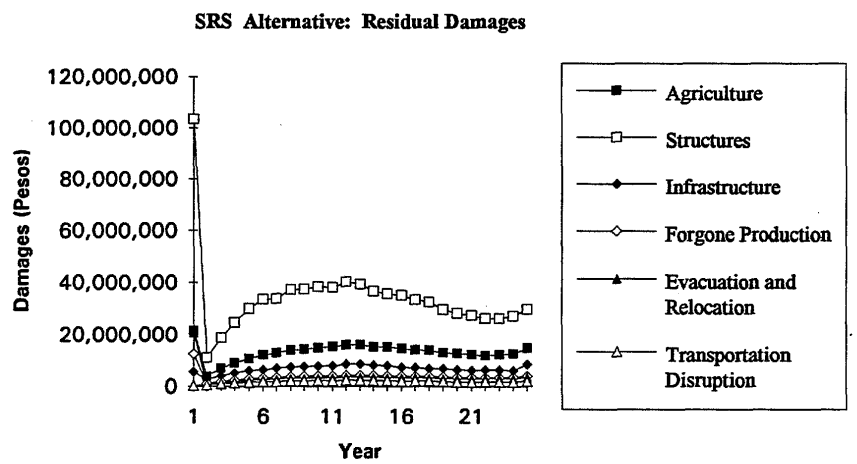
Figure C-43 plots damages over time by category, and shows that damages are reduced at any point in time in comparison to the without-project condition. Also, especially for the levee alternative (and for the channel excavation alternative), damages are delayed.

Figure C-43 – *Mean With-Project Residual Damages*





5.8.5 Project Benefits. The difference between without- and with-project damages are project benefits and are presented in table C-90. The sum of benefits ranges from P810 million for levee and channel excavation alternatives to P642 million for the SRS.

Table C-90 -- *Mean Project Benefits by Category: Santo Tomas (present value, pesos)*

Category	Sub-category	Levee	SRS	Channel Excavation
Structures	Residential	200,041,416	170,750,410	200,041,416
	Commercial	297,788,087	234,040,881	297,788,087
	Farm	38,259,473	34,565,006	38,259,473
Agriculture	Rice	181,718,764	141,932,891	181,718,764
	Sugar	6,311,247	5,473,267	6,311,247
	Fish	11,763,340	9,446,826	11,763,340
	Other Ag	10,875,078	9,430,858	10,875,078
Evacuation and Relocation		68,478,491	61,929,912	68,478,491
Foregone Production		73,640,171	67,595,199	73,640,171
Transportation Disruption		1,795,775	(5,126,210)	1,795,775
Other Infrastructure		7,733,957	9,093,856	7,733,957
Roads		3,996,830	3,818,417	3,996,830
Levees		5,087,421	(19,850,605)	5,087,421
Bridges		0	0	0
Total Benefits		**907,490,050**	**723,100,709**	**907,490,050**

5.8.6 Project Costs. The Santo Tomas basin has two of the most costly projects under consideration in this study--the SRS and channel excavation alternatives. The SRS has financial construction costs approaching P5.5 billion with the present value of lifetime economic costs estimated at P3.9 billion. The next most costly project, channel excavation, has first costs of P3.3 billion and substantial future costs due to the requirement for annual maintenance excavation of 3.6 million m^3 per year on an average annual basis. Levees are the least expensive alternative with construction costs at P939 million and lifetime economic costs of P740 million. (See table C-93 for cost data).

5.8.7 Investment Analysis. Table C-91 lists present value of economic benefits and costs for each alternative. Benefits are provided for the average benefits and two standard deviations above and below the average. For the mean case, benefits exceed costs only for the levee alternative.

Table C-91 – *Present Value of Benefits and Costs: Santo Tomas (present value, pesos)*

Alternative	Project Benefits			Project Cost
	Mean	2 σ Above Mean	2 σ Below Mean	
Levee	907,490,050	1,185,438,083	629,542,016	739,658,125
SRS	723,100,709	1,112,579,889	333,621,529	3,886,893,152
Channel Excavation	907,490,050	1,185,438,083	629,542,016	3,866,500,335

σ = standard deviation

Based on this simulation analysis, table C-92 shows the investments statistics which indicate that only the levee alternative is, on average, economically efficient.

Table C-92 – *Investment Statistics: Santo Tomas*

Levee Alternative

	Mean	2 σ Above Mean	2 σ Below Mean
Net Benefits	167,831,925	445,779,958	(110,116,109)
BCR	1.23	1.60	0.85
IRR	18.14% (a)	na	na

(a) Based on costs as of 1 Feb 1994

SRS

	Mean	2 σ Above Mean	2 σ Below Mean
Net Benefits	(3,163,792,443)	(2,774,3113,263)	(3,553,271,623)
BCR	0.19	0.29	0.09
IRR	na	na	na

Channel Excavation

	Mean	2 σ Above Mean	2 σ Below Mean
Net Benefits	(2,959,010,285)	(2,681,062,252)	(3,236,958,319)
BCR	0.23	0.31	0.16
IRR	na	na	na

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Table C-93 -- *Alternative Costs for the Santo Tomas Basin*

Construction Costs, Financial (Pesos)

Line Item	Levee	SRS	Channel Excavation
Profit	63,751,277	382,567,776	228,264,291
Home Overhead	12,500,250	75,013,289	44,757,704
Field Overhead	46,297,224	277,826,998	165,769,274
Vat Tax	52,610,482	315,712,498	188,374,175
Skilled Labor	20,140,303	97,187,217	56,564,793
Unskilled Labor	5,934,798	48,205,532	19,288,071
Equipment	383,969,714	2,154,841,976	1,762,642,564
Material	116,060,000	856,890,250	45,246,321
Environmental Mitigation	20,939,160	20,989,160	150,000
Subtotal	722,203,208	4,229,234,696	2,511,057,193
Contingency (30%)	216,660,962	1,268,770,409	753,317,158
Total First Costs	**938,864,170**	**5,498,005,105**	**3,264,374,351**

Annual Costs, financial

Item	Levee	SRS	Channel Excavation
Annual Excavation Costs (a)	0	0	212,400,000
O&M	938,864	5,498,005	195,862
Total Annual Costs	**938,864**	**5,498,005**	**212,595,862**

Special Future Costs (every 10 years)

Item	Levee	SRS	Channel Excavation
Major Maintenance	0	30,000,000	0

Present Value of Financial Costs, 1994 Base

First Costs	Levee	SRS	Channel Excavation
First Costs	793,364,175	4,174,840,553	2,758,479,601
Annual Costs	5,826,224	27,199,035	1,319,286,776
Future Special Costs	0	8,115,057	0
Total (Pesos)	**799,190,398**	**4,210,154,646**	**4,077,766,378**
Total (Dollars)	**29,599,644**	**155,931,654**	**151,028,384**

Present Value of Economic Costs, 1994 Base

First Costs	Levee	SRS	Channel Excavation
First Costs	734,265,901	3,853,671,472	2,547,306,606
Annual Costs	5,392,224	25,106,622	1,319,193,729
Future Special Costs	0	8,115,057	0
Total (Pesos)	**739,658,125**	**3,886,893,152**	**3,866,500,335**
Total (Dollars)	**27,394,745**	**143,959,006**	**143,203,716**

5.9 Delta

5.9.1 Problem. Sedimentation in the delta waterways has prevented drainage to Pampanga Bay and increased both the depth and duration of ponding-type flooding.

5.9.2 Alternative. The dredging alternative consists of the following work:

- Dredging Alternative. The main channel of the Pasag River from the mouth of the Pasig-Potrero downstream to the Pampanga Bay would be dredged to 3.5 meters below the normal water surface to its full pre-eruption width. In addition, the Dalan Bapor channel would be re-established to full width and a depth of 3.5 meters. Future maintenance dredging also will be required.

5.9.3 Without-Project Condition Damages. As shown on table C-94, damages are estimated for the Pasig-Potrero ponding zone and the Porac-Gumain ponding zone combined. Without- project damages over the planning period are estimated at P7.3 billion, with agriculture accounting for 60 percent of total damages.

Table C-94 -- *Mean Without-Project Damages: Delta (present value, pesos)*

Category	Damages
Agriculture	4,466,701,406
Structures	2,726,673,731
Infrastructure	25,291,696
Forgone Production	77,207,732
Evacuation and Relocation	22,765,068
Transportation Disruption	0
Total Damages	**7,318,639,632**

Figure C-44 shows damages by category. Damages to structures decline over time as continued flooding erodes value of buildings.

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Figure C-44 -- *Mean Without-Project Damages by Category: Delta*

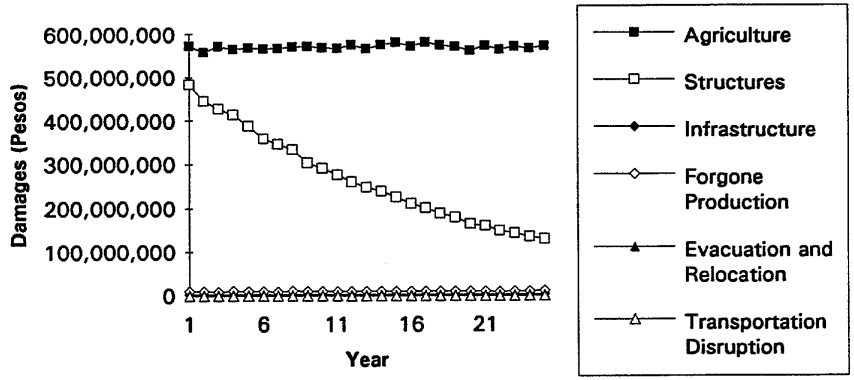

Figure C-45 represents the range in results over all 500 iterations of the simulation as represented by two standard deviations about the mean

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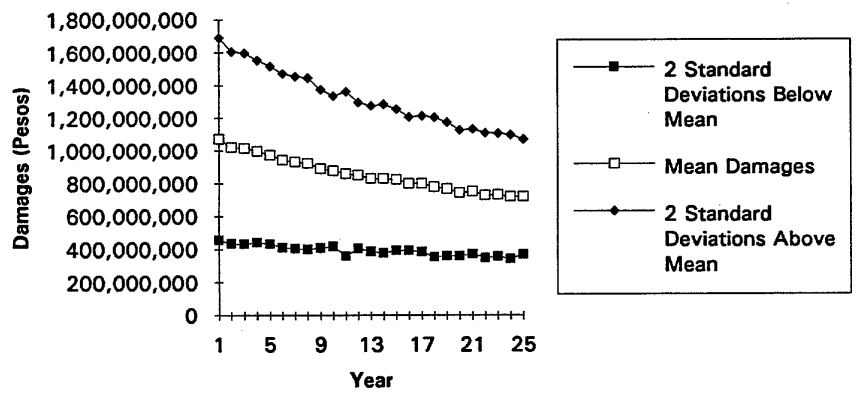
Figure C-45 -- *Range of Without-Project Damages: Delta*



5.9.4 With-Project Condition Damages. As shown in table C-95, mean damages are reduced to P4 billion, about half of the without-project condition damage estimate.

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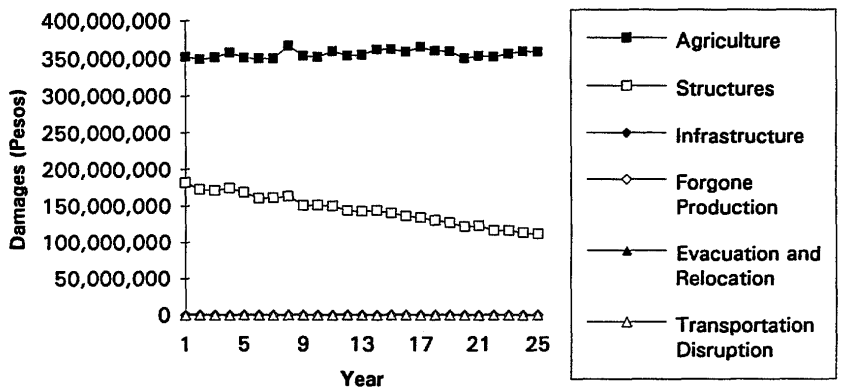
Table C-95 -- *Mean With-Project Damages: Delta (present value, pesos)*

	Dredging
Agriculture	2,783,104,722
Structures	1,246,454,862
Infrastructure	4,210,046
Forgone Production	0
Evacuation and Relocation	0
Transportation Disruption	0
Total Damages	**4,033,769,630**

Figure C-46 shows mean damages by category over the planning period.

Figure C-46 -- *Mean With-Project Residual Damages: Delta*



5.9.5 Project Benefits. Total expected benefits of dredging the delta waterways is P3.3 billion with 70 percent of that coming from reduced agricultural damage (see table C-96).

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Table C-96 -- *Mean Project Benefits by Category: Delta (present value, pesos)*

Category	Sub-category	Benefits
Structures	Residential	822,163,104
	Commercial	591,878,699
	Farm	66,177,066
Agriculture	Rice	35,912,769
	Sugar	0
	Fish	1,627,411,366
	Other Ag	20,272,549
Evacuation and Relocation		22,765,068
Foregone Production		77,207,732
Transportation Disruption		0
Other Infrastructure		0
Roads		21,081,650
Levees		0
Bridges		0
Total Benefits		**3,284,870,003**

5.9.6 Project Costs. Initial construction costs are P952 million with equipment accounting for P481 million of this sum. It is assumed that 700,000 m^3 of maintenance dredging will be required over the life of the project, based on a pass-through of silt and sand of 200,000 m^3 from the Gumain basin and 500,000 m^3 from the Pasig basin. The present value of life cycle costs are estimated at P1,080 million with first costs accounting for 73 percent of this total. (See table C-99 for cost data).

5.9.7 Investment Analysis. Project benefits range from P2.5 billion to P4.1 billion with an expected value of P3.3 billion (see table C-97). The range of benefits lies above the present value of estimated project costs of P1.1 billion over the planning period.

Table C-97 -- *Present Value of Benefits and Costs: Delta (pesos)*

Alternative	Project Benefits			Project Cost
	Mean	2 σ Above Mean	2 σ Below Mean	
Dredging	3,284,870,003	4,102,925,752	2,466,814,254	1,079,511,588

Based on this analysis, delta dredging has an expected benefit-cost ratio of 3.04. Table C-98 summarizes the investment rates of return for this alternative.

Table C-98 -- *Investment Statistics: Delta Dredging*

	Mean	High	Low
Net Benefits	2,205,358,415	3,023,414,164	1,387,302,666
BCR	3.04	3.80	2.29
IRR	130.35%	na	na

Table C-99 – *Delta Dredging Costs*

Financial Construction Costs (pesos)

Line Item	Dredging
Profit	62,698,453
Home Overhead	12,293,814
Field Overhead	45,532,645
Vat Tax	51,741,642
Skilled Labor	18,964,636
Unskilled Labor	17,379,542
Equipment	481,072,241
Material	0
Land	43,000,000
Subtotal	732,682,973
Contingency (30%)	219,804,892
Total First Costs	**952,487,865**

Annual Costs

Line Item	Dredging
Dredging	41,300,000
Excavation	0
O&M	952,488
Other Annual Costs	0
Total Annual Costs	**42,252,488**

Special Future Costs (every 10 years)

Item	Dredging
Major Maintenance	0

Present Value of Financial Costs, 1994 Base

First Costs	850,435,594
Annual Costs	293,666,705
Future Special Costs	0
Total costs, pesos	**1,144,102,298**
Total costs, dollars	**42,374,159**

Present Value of Economic Costs, 1994 Base

First Costs	786,343,794
Annual Costs	293,167,794
Future Special Costs	0
Total costs, pesos	**1,079,511,588**
Total costs, dollars	**39,981,911**

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6. SUMMARY

6.1 Alternative Rankings

6.1.1 Basin Results. According to the cost-benefit analysis for the mean case, the Abacan, Bucao, Maloma, O'Donnell, Porac-Gumain, Santo Tomas and the delta all have at least one economically efficient alternative as indicated by positive net benefits, a benefit-cost ratio greater than one and an internal rate of return greater than 12 percent. Neither the Pasig-Potrero nor the Sacobia-Bamban alternatives appear to be economic for the mean case. Furthermore, when alternative benefits for these two basins are evaluated at two standard deviations above the mean, the alternatives continue to produce negative net benefits. Table C-100 at the end of this chapter lists the alternatives and rates of return by basin in alphabetical order for the mean case.

6.1.2 Alternative Rankings. Table C-101 presents alternatives ranked top to bottom by expected net benefits (net present value.) Eight alternatives have positive net benefits, two of which are in the Porac-Gumain basin.[59] Considering only the levee alternative for the Porac-Gumain, the present value of costs for the remaining 7 economically efficient alternatives is P2.87 billion pesos, or about $106 million.[60]

Table C-102 lists the alternatives sorted by benefit-cost ratio. The same eight alternatives have benefit-cost ratios greater than one, but the ordering is slightly different.

6.1.3 Risk and Uncertainty. The methodology used in this research has emphasized risk and uncertainty, and the results in section 5 have been stated as ranges. One of the main results of the analysis is the wide range in benefits for each alternative, the result of uncertainty in the Hydrologic and Hydraulic inputs to the economic model and of uncertainty in the economic data and relationships in the economic model itself.

Table C-103 summarizes the range of project benefits for each alternative, listing mean benefits and benefits two standard deviations above and below the mean, as well as project costs. The wide range about the average (mean) indicates considerable uncertainty about the results.

For benefits two standard deviations below the mean only two alternatives are economically viable-- the Abacan and Delta projects. Based on this result, one can be quite confident that investments in these projects are worthwhile.

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[59]The two Gumain projects are mutually exclusive in the sense that if one alternative is implemented, the other would no longer be economically viable.
[60]At an exchange rate of P27/$.

For two standard deviations above the mean, two projects become economically efficient that are not economic for the mean case - the Maloma channel excavation alternative and the Porac-Gumain SRS alternative. These may be considered to be marginal alternatives. None of the alternatives proposed for the Pasig-Potrero and Sacobia-Bamban basins are economically efficient for the entire range of benefits.

6.2 Other Considerations and Future Analysis

The cost-benefit analysis conducted as part of this report has taken a comprehensive look at the various alternatives. One of the stengths of this research is that a generally consistent method of study has been applied to all alternatives. However, there are a number of limitations to the analysis and areas where future research might be worthwhile.

6.2.1 Incremental Analysis. No incremental analysis was performed as a part of this report. Incremental analysis can be used to find the optimal height of a levee, for example, or to investigate which features of a project are economically efficient and which are not. For example:

- In the cost-benefit analysis, each alternative was treated as an indivisible unit. The individual features of an alternative were not analyzed separately. For example, in the Pasig-Potrero levee alternative, separate features of the levee system were not subject to incremental analysis. Had this been done, certain levee segments, such as the segment in the Porac reach which prevents Pasig overflow into the Porac River, may have proved economic even though the alternative as a whole did not. Future research might address this question.

- No incremental analysis was used to size features and no evaluation of the optimal size of alternative features was accomplished. The design standard of protecting against 10 years of sediment and 100-year event was a given, and each alternative was designed to this standard. The economic analysis took the alternative size as a given and proceeded to evaluate alternatives without considering various levee heights or dam heights. Thus, there is no way of knowing if net benefits could be increased by either increasing or decreasing the scale of the projects.

6.2.2 Timing Analysis. Related to the issue of incremental analysis is the issue of timing for alternatives. The assumption was made that construction would begin in 1995 at which time benefits would start to accrue. Different start dates were not evaluated. In addition, no evaluation was made of implementing alternatives in stages. The time sensitivity of project economics may be a worthwhile topic for future research.

6.2.3. Economic Data. Much of the analysis depends on rather uncertain economic data, the result of a fairly small sample size. This results in uncertain estimates for numbers of buildings, for instance. Additional analysis could address this problem by

supplementing the ground survey work with photo analysis or more expensive new survey work if deemed necessary.

Another potential source of error lies with the stage-damage functions for buildings which were based on survey responses and observations. One fruitfull area of additional analysis may be to validate these estimates with independent evaluation of damages functions by building engineers or appraisers.

6.2.4 Hydrologic and Hydraulic Data. Three different sets of hydraulic and hydrologis data were used in the analysis raises concerns about the consistency of these inputs into the economic analysis. To give an example, flooding in the sediment model output[61] was assumed to range from 25 to 45 centimeters, but in the stage frequency tables[62] flooding depth ranged up to 120 centimeters. Consequently, building damages were often greater in the latter case and may have contributed to the reason these three flooding basins all have economic alternatives. Moreover, the sediment model probably captures only a part of the uncertainty in sediment and flooding. Refinements to the hydraulic and hydrologic data would confirm or modify economic results.

6.2.5 Basins as Systems. In this analysis each basin was generally considered as separate and independent from other basins.[63] Although this assumption is probably satisfactory for several basins, it likely introduces errors in the Pasig-Potrero, Porac-Gumain and delta basins. Future analysis might investigate the system condition for these basins.

6.2.6 Efficiency of Pasig-Potrero and Sacobia-Bamban Alternatives. One evident result of the analysis is that for the populous Pasig-Potrero and Sacobia-Bamban basins, no alternatives evaluated are economically efficient. Conducting further research could help explain this result. Factors to consider include: hydrologic inputs to the economic model for the without- and with-project conditions; or economic data and relationships in the model. Further investigations along with system consideration and optimization analyses could identify alternatives that show greater economic viability.

[61]These outputs were used for the O'donnell, Sacobia, Pasig, Sto. Tomas and Bucao.
[62]These tables were used for the Abacan, Gumain and Maloma.
[63]The exception being the credit given Gumain and Pasig projects for reduced delta dredging.

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Table C-100 -- List of Alternatives by Basin with Rates of Return

Basin	Project	Mean Benefits	Economic Costs	Net Benefits	Benefit-Cost Ratio	Internal Rate of Return (%)
Abacan	Bank Protection	191,679,000	67,953,000	123,726,000	2.82	38.7
Bucao	Levee	210,979,000	155,076,000	55,903,000	1.36	17.0
Bucao	SRS	223,778,000	3,317,898,000	-3,094,120,000	0.07	
Delta	Dredging	3,284,870;000	1,079,512,000	2,205,358,000	3.04	130.4
Maloma	Levee	97,635,000	85,218,000	12,417,000	1.15	15.7
Maloma	SRS	97,635,000	184,705,000	-87,070,000	0.53	3.5
Maloma	Channel Excavation	97,635,000	128,977,000	-31,342,000	0.76	7.6
O'Donnell	Levee	187,281,000	188,240,000	-959,000	0.99	12.1
O'Donnell	SRS	249,788,000	2,246,013,000	-1,996,225,000	0.11	
O'Donnell	Channel Excavation	187,281,000	1,244,019,000	-1,056,738,000	0.15	
Pasig-Potrero	Levee	657,849,000	1,548,626,000	-890,777,000	0.42	
Pasig-Potrero	Channel Excavation	657,849,000	1,943,000,000	-1,285,151,000	0.34	
Porac-Gumain	Levee	975,495,000	587,176,000	388,319,000	1.66	23.0
Porac-Gumain	SRS	893,482,000	1,138,975,000	-245,493,000	0.78	8.4
Porac-Gumain	Channel Excavation	975,495,000	561,186,000	414,309,000	1.74	24.3
Sacobia-Bamban	Levee	434,281,000	1,078,450,000	-644,169,000	0.40	
Sacobia-Bamban	SRS	351,663,000	1,410,233,000	-1,058,570,000	0.25	
Sacobia-Bamban	Channel Excavation	434,281,000	1,555,713,000	-1,121,432,000	0.28	4.5
Sto. Tomas	Levee	907,490,000	739,658,000	167,832,000	1.23	18.1
Sto. Tomas	SRS	723,101,000	3,886,893,000	-3,163,792,000	0.19	
Sto. Tomas	Channel Excavation	907,490,000	3,866,500,000	-2,959,010,000	0.23	

Table C-101 -- Alternative Rankings by Net Benefits

Basin	Project	Mean Benefits	Costs	Net Benefits	Benefit-Cost Ratio	Internal Rate of Return (%)
Delta	Dredging	3,284,870,000	1,079,512,000	2,205,358,000	3.04	130.4
Porac-Gumain	Channel Excavation	975,495,000	561,186,000	414,309,000	1.74	24.3
Porac-Gumain	Levee	975,495,000	587,176,000	388,319,000	1.66	23.0
Sto. Tomas	Levee	907,490,000	739,658,000	167,832,000	1.23	18.1
Abacan	Bank Protection	191,679,000	67,953,000	123,726,000	2.82	38.7
Bucao	Levee	210,979,000	155,076,000	55,903,000	1.36	17.0
Maloma	Levee	97,635,000	85,218,000	12,417,000	1.15	15.7
O'Donnell	Levee	187,281,000	188,240,000	-959,000	0.99	12.1
Maloma	Channel Excavation	97,635,000	128,977,000	-31,342,000	0.76	7.6
Maloma	SRS	97,635,000	184,705,000	-87,070,000	0.53	3.5
Porac-Gumain	SRS	893,482,000	1,138,975,000	-245,493,000	0.78	8.4
Sacobia-Bamban	Levee	434,281,000	1,078,450,000	-644,169,000	0.40	
Pasig-Potrero	Levee	657,849,000	1,548,626,000	-890,777,000	0.42	
O'Donnell	Channel Excavation	187,281,000	1,244,019,000	-1,056,738,000	0.15	
Sacobia-Bamban	SRS	351,663,000	1,410,233,000	-1,058,570,000	0.25	
Sacobia-Bamban	Channel Excavation	434,281,000	1,555,713,000	-1,121,432,000	0.28	4.5
Pasig-Potrero	Channel Excavation	657,849,000	1,943,000,000	-1,285,151,000	0.34	
O'Donnell	SRS	249,788,000	2,246,013,000	-1,996,225,000	0.11	
Sto. Tomas	Channel Excavation	907,490,000	3,866,500,000	-2,959,010,000	0.23	
Bucao	SRS	223,778,000	3,317,898,000	-3,094,120,000	0.07	
Sto. Tomas	SRS	723,101,000	3,886,893,000	-3,163,792,000	0.19	

Table C-102 -- Alternative Rankings by Benefit-Cost Ratio

Basin	Project	Mean Benefits	Costs	Net Benefits	Benefit-Cost Ratio	Internal Rate of Return (%)
Delta	Dredging	3,284,870,000	1,079,512,000	2,205,358,000	3.04	130.4
Abacan	Bank Protection	191,679,000	67,953,000	123,726,000	2.82	38.7
Porac-Gumain	Channel Excavation	975,495,000	561,186,000	414,309,000	1.74	24.3
Porac-Gumain	Levee	975,495,000	587,176,000	388,319,000	1.66	23.0
Bucao	Levee	210,979,000	155,076,000	55,903,000	1.36	17.0
Sto. Tomas	Levee	907,490,000	739,658,000	167,832,000	1.23	18.1
Maloma	Levee	97,635,000	85,218,000	12,417,000	1.15	15.7
O'Donnell	Levee	187,281,000	188,240,000	-959,000	0.99	12.1
Porac-Gumain	SRS	893,482,000	1,138,975,000	-245,493,000	0.78	8.4
Maloma	Channel Excavation	97,635,000	128,977,000	-31,342,000	0.76	7.6
Maloma	SRS	97,635,000	184,705,000	-87,070,000	0.53	3.5
Pasig-Potrero	Levee	657,849,000	1,548,626,000	-890,777,000	0.42	
Sacobia-Bamban	Levee	434,281,000	1,078,450,000	-644,169,000	0.40	
Pasig-Potrero	Channel Excavation	657,849,000	1,943,000,000	-1,285,151,000	0.34	
Sacobia-Bamban	Channel Excavation	434,281,000	1,555,713,000	-1,121,432,000	0.28	4.5
Sacobia-Bamban	SRS	351,663,000	1,410,233,000	-1,058,570,000	0.25	
Sto. Tomas	Channel Excavation	907,490,000	3,866,500,000	-2,959,010,000	0.23	
Sto. Tomas	SRS	723,101,000	3,886,893,000	-3,163,792,000	0.19	
O'Donnell	Channel Excavation	187,281,000	1,244,019,000	-1,056,738,000	0.15	
O'Donnell	SRS	249,788,000	2,246,013,000	-1,996,225,000	0.11	
Bucao	SRS	223,778,000	3,317,898,000	-3,094,120,000	0.07	

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Table C-103 -- Range of Project Benefits with Project Costs

Basin	Project	Benefits Mean	Benefits High	Benefits Low	Costs
Abacan	Bank Protection	191,679,219	294,026,000	89,333,000	67,953,000
Bucao	Levee	210,979,366	421,122,551	836,181	155,076,000
Bucao	SRS	223,778,220	427,373,243	20,183,196	3,317,898,000
Delta	Dredging	3,284,870,003	4,102,925,752	2,466,814,254	1,079,512,000
Maloma	Levee	97,635,357	160,426,311	34,844,403	85,218,000
Maloma	SRS	97,635,357	160,426,311	34,844,403	184,705,000
Maloma	Channel Excavation	97,635,357	160,426,311	34,844,403	128,977,000
O'Donnell	Levee	187,281,024	683,983,276	(309,421,228)	188,240,000
O'Donnell	SRS	249,788,163	670,811,055	(171,234,728)	2,246,013,000
O'Donnell	Channel Excavation	187,281,024	683,983,276	(309,421,228)	1,244,019,000
Pasig-Potrero	Levee	657,848,652	905,851,212	409,846,091	1,548,626,000
Pasig-Potrero	Channel Excavation	657,848,652	905,851,212	409,846,091	1,943,000,000
Porac-Gumain	Levee	975,495,068	1,631,368,118	319,622,018	587,176,000
Porac-Gumain	SRS	893,481,741	1,549,354,790	237,608,692	1,138,975,000
Porac-Gumain	Channel Excavation	975,495,068	1,631,368,118	319,622,018	561,186,000
Sacobia-Bamban	Levees	434,280,711	677,163,168	191,398,255	1,078,450,000
Sacobia-Bamban	SRS	351,663,057	634,951,685	68,374,430	1,410,233,000
Sacobia-Bamban	Channel Excavation	434,280,711	677,163,168	191,398,255	1,555,713,000
Sto. Tomas	Levee	907,490,050	1,185,438,083	629,542,016	739,658,000
Sto. Tomas	SRS	723,100,709	1,112,579,889	333,621,529	3,886,893,000
Sto. Tomas	Channel Excavation	907,490,050	1,185,438,083	629,542,016	3,866,500,000

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EXHIBIT

A

ECONOMIC SURVEY FORMS

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MT. PINATUBO

FLOOD AND LAHAR DAMAGE SURVEY - INTERVIEW

RESIDENTIAL STRUCTURES

Identifying Information

1. Surveyor: ☐☐☐☐☐☐☐☐ 2. Date: ☐☐/☐☐ 3. Time: ☐☐:☐☐

4. Municipality: ☐☐☐☐☐☐☐☐ 5. Baranguay: ☐☐☐☐☐☐☐☐

6. Basin: ☐☐ 7. Hazard: ☐☐ 8. Cell: ☐☐ 9. Photo: ☐☐ ☐☐

First floor elev. relative to: 10. Ground: ☐☐☐.☐ 11. Road: ☐☐☐.☐

12. Respondent: ☐☐☐☐☐☐☐☐☐☐☐☐☐☐☐☐

13. Primary material: ☐☐ 14. Percent: ☐☐

15. Secondary material: ☐☐ 16. Percent: ☐☐

17. No. floors: ☐ 18. Condition code: ☐☐

19. No. unattached structures: ☐☐ 20. Percent volume loss: ☐☐

21. Own or Rent: ☐

22. Occupants: ☐☐ 23. Renters: ☐☐

24. Total rent: P☐☐☐☐☐

25. Property value: P☐☐☐☐☐☐☐

26. Replacement value: P☐☐☐☐☐☐☐

27. Total square meters: ☐☐☐☐

28. Water D&C: ☐☐ 29. Water B&Cl: ☐☐

30. Energy: a. ☐☐ b. ☐☐ c. ☐☐

Layout

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Historical Events

	No. Events	Date of Most Recent Event	Cost of Damage(P)	Depth
31. Water	☐	☐☐/☐☐	☐☐☐☐☐	☐☐
32. Mud	☐	☐☐/☐☐	☐☐☐☐☐	☐☐
33. Ponding	☐	☐☐/☐☐	☐☐☐☐☐	☐☐
34. Rebuilt	☐	☐☐/☐☐	☐☐☐☐☐	

	No. Events	Date	Distance(km)	Duration(wks)
35. Evac/Reloc.	☐	☐☐/☐☐	☐☐☐☐	☐☐

36. Willingness to relocate: ☐☐

37. Total content value: P☐☐☐☐☐☐☐

Property Damage

	.5m		2m	
	% Lost	Damage (P)	% Lost	Damage (P)
38. Property	☐☐	☐☐☐☐☐	☐☐	☐☐☐☐☐☐
39. Structure	☐☐	☐☐☐☐☐	☐☐	☐☐☐☐☐☐
40. Contents	☐☐	☐☐☐☐☐	☐☐	☐☐☐☐☐☐
41. Other	☐☐	☐☐☐☐☐	☐☐	☐☐☐☐☐☐

Associated Losses

42. Annual income: P☐☐☐☐☐☐☐

43. Home business (Y/N) ☐
If no, stop here.

44. Monthly sales: P☐☐☐☐☐

45. Monthly expenses: P☐☐☐☐☐

46. Business seasonal (Y/N) ☐

47. If yes, which months are sales greatest? ☐☐,☐☐,☐☐

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MT. PINATUBO

FLOOD AND LAHAR DAMAGE SURVEY - INTERVIEW

COMMERCIAL AND OTHER STRUCTURES

Identifying Information

1. Surveyor: ☐☐☐☐☐☐☐☐ 2. Date: ☐☐/☐☐ 3. Time: ☐☐:☐☐

4. Municipality: ☐☐☐☐☐☐☐☐ 5. Baranguay: ☐☐☐☐☐☐☐☐

6. Basin: ☐☐ 7. Hazard: ☐☐ 8. Cell: ☐☐ 9. Photo: ☐☐ ☐☐

First floor elev. relative to: 10. Ground: ☐☐☐.☐ 11. Road: ☐☐☐.☐

12. Respondent: ☐☐☐☐☐☐☐☐☐☐☐☐☐☐☐☐

13. Primary material: ☐☐ 14. Percent: ☐☐

15. Secondary material: ☐☐ 16. Percent: ☐☐

17. No. Floors: ☐ 18. Condition code: ☐☐

19. No. unattached structures: ☐☐ 20. Percent volume loss: ☐☐

21. Own or Rent: ☐

22. Renters: ☐☐

23. Total rent: P☐☐☐☐☐☐☐

24. Property value: P☐☐☐☐☐☐☐

25. Replacement value: P☐☐☐☐☐☐☐

26. Total square meters: ☐☐☐☐

27. Water: ☐☐

28. Energy: a.☐☐ b.☐☐ c.☐☐

Layout

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Historical Events

	#	Date of Most Recent Event	Cost of Damage(P)	Depth	Weeks Closed
29. Water	☐	☐☐/☐☐	☐☐☐☐☐	☐☐	☐☐
30. Mud	☐	☐☐/☐☐	☐☐☐☐☐	☐☐	☐☐
31. Ponding	☐	☐☐/☐☐	☐☐☐☐☐	☐☐	☐☐
32. Rebuilt	☐	☐☐/☐☐	☐☐☐☐☐		

	#	Date	Distance(km)	Duration (wks)
33. Evac/Reloc.	☐	☐☐/☐☐	☐☐☐☐	☐☐

34. Willingness to relocate: ☐☐

35. Total content value: P☐☐☐☐☐☐☐

Property Damage

	.5m % Lost	.5m Damage (P)	2m % Lost	2m Damage (P)
36. Property	☐☐	☐☐☐☐☐	☐☐	☐☐☐☐☐
37. Structure	☐☐	☐☐☐☐☐	☐☐	☐☐☐☐☐
38. Contents	☐☐	☐☐☐☐☐	☐☐	☐☐☐☐☐
39. Other	☐☐	☐☐☐☐☐	☐☐	☐☐☐☐☐

40. Clean-up costs:

	.5m	2m
Labor:	P☐☐☐☐☐	Labor: P☐☐☐☐☐
Equip:	P☐☐☐☐☐	Equip: P☐☐☐☐☐

Business Losses

41. Monthly sales: P☐☐☐☐☐

42. Monthly expenses: P☐☐☐☐☐

43. Business seasonal (Y/N) ☐

44. If yes, which months are sales greatest? ☐☐,☐☐,☐☐

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MT. PINATUBO

FLOOD AND LAHAR DAMAGE SURVEY - VISUAL

Identifying Information

1. Surveyor: ☐☐☐☐☐☐☐☐ 2. Date: ☐☐/☐☐ 3. Time: ☐☐:☐☐

4. Municipality: ☐☐☐☐☐☐☐☐ 5. Baranguay: ☐☐☐☐☐☐☐☐

6. Basin: ☐☐ 7. Hazard: ☐☐ 8. Cell: ☐☐ 9. Photo: ☐☐ ☐☐

First floor elev. relative to: 10. Ground: ☐☐☐.☐ 11. Road: ☐☐☐.☐

12. Respondent: ☐☐☐☐☐☐☐☐☐☐☐☐☐☐☐☐

Structure Evaluation

13. Primary material: ☐☐ 14. Percent: ☐☐

15. Secondary material: ☐☐ 16. Percent: ☐☐

17. No. floors: ☐☐

18. Condition code: ☐

19. No. unattached structures: ☐☐

20. Percent volume loss: ☐☐

Layout of Structures on Property

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MT. PINATUBO

FLOOD AND LAHAR DAMAGE SURVEY - GRID CELL

Identifying Information

1. Surveyor: ☐☐☐☐☐☐☐☐ 2. Date: ☐☐/☐☐ 3. Time: ☐☐:☐☐

4. Municipality: ☐☐☐☐☐☐☐☐ 5. Baranguay: ☐☐☐☐☐☐☐☐

6. Basin: ☐☐ 7. Hazard: ☐☐ 8. Cell: ☐☐ 9. Photo: ☐☐ ☐☐

10. GPS elevation at cell corners: a. ☐☐☐.☐ b. ☐☐☐.☐
c. ☐☐☐.☐ d. ☐☐☐.☐

11. No. of structures: ☐☐

INFRASTRUCTURE INVENTORY

	Dimensions	Percent Loss at .5m	Percent Loss at 2m
12. Highways/roads	☐☐☐☐☐☐☐☐	☐☐	☐☐
13. Rail lines	☐☐☐☐☐☐☐☐	☐☐	☐☐
14. Telephone lines	☐☐☐☐☐☐☐☐	☐☐	☐☐
15. Other telecommunications	☐☐☐☐☐☐☐☐	☐☐	☐☐
16. Water pumps/wells	☐☐☐☐☐☐☐☐	☐☐	☐☐
17. Other sanitation	☐☐☐☐☐☐☐☐	☐☐	☐☐
18. Power lines	☐☐☐☐☐☐☐☐	☐☐	☐☐
19. Pipelines	☐☐☐☐☐☐☐☐	☐☐	☐☐
20. Irrigation	☐☐☐☐☐☐☐☐	☐☐	☐☐
21. Levees/dikes	☐☐☐☐☐☐☐☐	☐☐	☐☐
22. Bridges	☐☐☐☐☐☐☐☐	☐☐	☐☐
23. Water canals	☐☐☐☐☐☐☐☐	☐☐	☐☐

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GRID CELL LAYOUT

a. b.

c. d.

Describe Contents of Grid Cell

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SAMPLE AG CELL ATTRIBUTES

RIVER BASIN_____ PHOTO #_____ CELL #_____

MAP #_____ ANALYST_____ DATE_____

DATA SOURCES: PHOTO ONLY Y/N RECON. Y/N FARMER Y/N OTHER_____

SKETCH OF CELL SHOWING BOUNDARIES OF MAJOR LAND USE CLASSIFICATIONS : REMARKS REGARDING THIS CELL

IN THE LEFT LEADING BLANKS ENTER PERCENTAGE OF TOTAL CELL AREA. EXTREME LEFT BLANKS MUST TOTAL 100; SUBCATEGORIES MUST EQUAL CATEGORY TOTAL. ENTER OPTIONAL DESCRIPTIONS IN LEFT BLANKS.

_____AGRICULTURAL AREA

- _____HOME GARDEN/ORCHARD_____
- _____IRRIG. LOWLAND: # RICE CROPS/YR___ # OTHER CROPS/YR___
- _____RAINFED LOWLAND: # RICE CROPS/YR___ # OTHER CROPS/YR___
- _____UPLAND_____
  - _____ANNUAL CROPS: INTENSIVE CROPS Y/N FIELD CROPS Y/N
  - _____SUGARCANE_____
  - _____TREECROPS_____
  - _____RANGE: NATURAL Y/N_____ IMPROVED Y/N_____
- _____FISHPONDS_____
- _____OTHER AGRICULTURAL AREA_____

_____URBAN/VILLAGE/INFRASTRUCTURE_____

_____RIVER/RIVERBED/BEACH/SEA/LAHAR/OTHER_____

_____MOUNTAINS/OTHER_____

PINATUBO FARMER SURVEY

RIVER BASIN_____ PHOTO #_____ CELL #_____

LAND USE IN THIS CELL TO BE DESCRIBED IN THIS INTERVIEW_____

INTERVIEWEE #_____ ANALYST_____ DATE_____

FARMER NAME_____ BGY_____ MUN._____

Carabao___ # Cattle___ # Pigs___ # Goats___ # Chickens___

Ducks___ # Geese___ # Horses___ Other Livestock_____

Value of One: Carabao_____ Cattle_____ Pig_____

Goat_____ Chicken_____ Duck_____ Other above_____

Value of annual harvest from home garden (fruit, vegetables, etc.)

(Describe)_____

(Choose one of the farmer's plots that is typical of the LAND USE INDICATED ABOVE. Write the interviewee # on the sketch of this cell, at the location of this plot.) Plot Size _____ sq. m.

INPUT, OUTPUT OR OPERATION	1st CROP_____		2nd CROP_____		3rd CROP_____	
	Amount	Value	Amount	Value	Amount	Value
LAND PREP.						
PLANTING						
WEEDING						
SPRAYING						
OTHER CARE						
HARVESTING						
PROCESSING						
SEED/STOCK						
FERTILIZER						
CHEMICALS						
OTHER INPUTS						
YIELD						
PROFIT						

TECHNICAL APPENDIX

D

COST ESTIMATES

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MOUNT PINATUBO
RECOVERY ACTION PLAN
LONG TERM REPORT

TECHNICAL APPENDIX D
COST ESTIMATES

Background Information

This appendix discusses assumptions used to develop cost estimates for alternatives described in the Long Term Report. The costs presented here address all eight river basins and the Pampanga delta.

The construction period for the Recovery Action Plan is expected to cover two construction seasons, running from December through June. Most construction work is estimated using a single shift of ten (10) hours per day and six (6) days per week.

On some of the larger structure construction work, it is assumed that there will be three (3) shifts per day, of eight (8) hours per shift, and six (6) days per week. Dredging work is based on two (2) shifts per day, twelve (12) hours per shift, and six (6) days per week.

Unit prices for direct cost were computed first using the standard crew and material method. The unit prices were input into the Microcomputer Aided Estimating System (MCACES) software, where indirect costs were added. Overhead percentages used were eight (8) percent for field overhead, two (2) percent for home office overhead, and ten (10) percent for profit. Ms. Angelina C. Forcadilla, Chief Estimator, Department of Public Works and Highways, Republic of the Philippines, confirmed that a total of ten (10) percent overhead and ten (10) percent profit should be used.

All pricing developed for the costs of the alternatives are based on January 1994 data. The currency is in pesos. Statistical data from various sources were reviewed, studied, and used as guiding references in the preparation of the unit prices for the alternatives.

Construction materials costs were taken from *Metro Construction Materials Prices and Indexes*, published by the Construction Industry Authority of the Philippines.

Equipment rates, including operating labor, are based on the *Equipment Guide, Edition No. 19*, dated November 1989, of the Associated Construction Equipment Lessors, Inc., (ACEL), plus a rate adjustment for fuel price increase, December 5, 1990.

All labor rates include basic wage, leave, bonus, social security, medicare, and worker's compensation.

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It is expected that all levees would be constructed using dump trucks and loaders. The assumed average haul distance is 3 kilometers for trucks filled by loaders.

Roller compacted concrete (RCC) is assumed to be batched at the job site by 15 cubic meter per hour dry batch plants and mixed in 5 cubic meter transit mixers and hauled to placement.

Unit prices were developed for the following items:

Dredging 16 " Dredge 800 M
Excavation Loader and Trucks
Embankment Random Compaction
Rock Fill Under Slab
Excavation Drainage Ditches
Rock Blasting
Filter Fabric
Cobbles/Boulders Fill
Fine Gravel Fill
Coarse Gravel Fill
Hydro-seed
1 Meter-Diameter Concrete Pipe
Load and Haul Rock
Demolition Gabions
RCC 4 Sack Mix
RCC 7 Sack Mix
Cast-In-Place Concrete Walls including Forms, Rebar, etc
Concrete Slope Protection

A thirty (30) percent contingency was included to account for the uncertainties associated with this work. Costs for lands, relocations, engineering, design, construction management, and inflation are not included in these estimates.

Care and diversion of water is assumed to be two (2) percent of cost.

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Excavation Loader and Trucks

This method of excavation will be used for excavation common, channel excavation, and toe trench excavation. This cost is for excavation, loading, hauling, and turn and dump only. Embankment costs are separate. The following was assumed: use 8 loose cubic meters (lcm) per load on truck, 2.3 cubic meters (cm) loader loading 206 bank cubic meters (bcm) per hour based on cat handbook.

Spot Trucks	0.50	Min.
Load	1.80	Min.
Haul 2 km at 20 km/hr	6.00	Min.
Turn and Dump	1.50	Min.
Haul 2 km	4.00	Min.
	13.80	Min. Cycle

Number of Trucks Required = 13.80/1.8 = 7.67 say 8 trucks.
Dozer will be Pushing to Loader.

A. Equipment:

Dozer	1 Each
Loader	1 Each
Trucks	8 Each
Grader on Haul Road	1 Each
Water Truck on Haul Road	1 Each

B. Labor:

Operators	3 Each
Truck Drivers	9 Each
Laborers	2 Each
Foreman	1 Each

Embankment random compaction cost is for leveling and compacting material after being dumped by pump trucks. The following was assumed:

A. Equipment:

Dozer 140 HP	1 Each
Grader	2 Each
Vib Roller 125 HP	1 Each
Water Tank Truck	1 Each
Water Pump	1 Each

B. Labor:

Foreman	1 Each
Laborer	4 Each
Operator	4 Each
Truck Driver	1 Each

C. Output: 500 bcm per hour

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Roller Compacted Concrete

Roller compacted concrete (RCC) will use two different mixes. One will be a 4 sack mix or about 1,500 to 2,000 psi at 90 days and the second a 7 sack mix of about 2,500 psi at 90 days. The 4 sack mix will be used for slope protection, structures subject to infrequent flows, and for the interior of all mass RCC structures. The 7 sack mix will be used for spillway and overflow sections subject to continuous or intermittent flows. All RCC will use the lahar material as aggregates and sand.

Mixing

RCC will be mixed on-site in portable batch plants with a capability of 50 cubic meters (cm) per hour. Following batching, the RCC will be deposited into a ready mix truck for final mixing and transfer to the placement site. Five ready mix trucks with the capacity of 5 cm will support each batch placement. The RCC should be batched, placed, and compacted within 45 minutes. It is anticipated that the RCC will be hauled directly to the placement site in ready mix trucks. Expected dump time for each ready mix truck is 3 to 4 minutes. The trucks are expected to dump the RCC in from of the bulldozer prior to spreading.

Spreading

After being deposited, the RCC will be worked and spread by a D7 bulldozer into thin uniform lifts usually 23 to 30.5 cm thick.

Compaction

After spreading, the total surface area of each RCC lift will be compacted by a minimum of four passes with a self-propelled double drum vibratory roller.

Mixing and Placing RCC

A. Equipment:

Concrete Batch Plant 50	1 Each
Loader 2.0 cm @ BP	2 Each
Water Truck	1 Each
Water Pump	1 Each
Transit Mixers 5 cm	5 Each
Vib Roller	1 Each
Dozer D7	2 Each
Grader	1 Each

B. Labor:

Foreman	1 Each
Skilled Laborer Batch Plant	5 Each
Laborer	10 Each
Operators	5 Each
Truck Drivers	6 Each

C. Output: 50 C per hour

D. Material Cost per cm 7 Sack Mix:

Portland Cement	7 bags @ P110/bag =	P770
Incidentals (15%)		P115
Total Material Cost per cm		P885

E. Material Cost per cm 4 Sack Mix:

Portland Cement	4 bags @ P110/bag =	P440
Incidentals (15%)		P 66
Total Material Cost per cm		P506

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Mount Pinatubo, Philippines
Direct Cost Marked Up

Division No.	Description	Pesos Direct Cost	Per Unit	Pesos Marked Up Cost	Pesos Cost Rounded	U.S. Dollar Cost	Cost Per Cy or SY	Unit
02100 1001	Dredging 16" Dredge	39.32	CM	52.41	52	$1.94	$1.48	CY
02200 1001	Excavation Loader & Trucks	44.45	CM	59.25	59	$2.19	$1.68	CY
02200 1004	Embankment Random Compaction	8.35	CM	11.13	11	$0.41	$0.32	CY
02200 1008	Rock Blasting	116.99	CM	155.94	156	$5.78	$4.42	CY
02200 1009	Filter Fabric	7.27	SM	9.69	10	$0.36	$0.30	SY
02200 1010	Cobbles /Bolders Fill	303.26	CM	404.23	404	$14.97	$11.45	CY
02200 1011	Fine Gravel Fill	443.26	CM	590.84	591	$21.88	$16.73	CY
02200 1012	Coarse Gravel Fill	343.26	CM	457.54	458	$16.95	$12.96	CY
02200 1013	Hydro–Seed	3.42	SM	4.56	5	$0.17	$0.14	SY
02200 1014	1 Meter Conc. Pipe	1553.62	LM	2,070.88	2,071	$76.70	$23.38	LF
02200 1015	Load & Haul Rock	108.37	CM	144.45	144	$5.35	$4.09	CY
02200 1016	Mountain Soil	65.35	CM	87.11	87	$3.23	$2.47	CY
02400 1101	Demolition Gabions	102.23	CM	136.27	136	$5.05	$3.86	CY
03000 1001	RCC Concrete 4 Sack Mix 50 CM/HR	734.63	CM	979.21	979	$36.27	$27.73	CY
03000 1002	RCC Concrete 7 Sack Mix 50 CM/HR	1114.13	CM	1,485.06	1,485	$55.00	$42.05	CY
03000 2001	Cast In Place Conc Walls	2290.09	CM	3,052.54	3,053	$113.06	$86.44	CY
03000 2002	Concrete Slope Protection	1210.05	CM	1,612.92	1,613	$59.74	$45.67	CY

Direct Cost Used In MCACES

Cost Used In Alternatives Summary

MOUNT PINATUBO, PHILIPPINES
RECOVERY ACTION PLAN
LONG TERM ESTIMATE

Designed By: U.S. ARMY CORPS OF ENGINEERS
Estimated By: H. BIRCHARD

Prepared By: CONSTRUCTIONEERING INC
BELLEVUE, WA (206-643-7273)

Date: 02/08/94
Est Construction Time: 730 Days

M C A C E S G O L D E D I T I O N
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Release 5.20J

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	QUANTY UOM	TOTAL DIRECT	VAT TAX	FIELD OH	HOME OH	PROFIT	TOTAL COST
01 MOUNT PINATUBO RECOVERY							
01_AA ABACAN BASIN							
01_AA.01 ALTERNATIVE BANK PROTECTION							
01_AA.01.AA CHANNELS							
01_AA.01.AA_001 BANK PROTECTION	1.00 EA	29,848,647	2,984,865	2,626,681	709,204	3,616,940	39,786,337
CHANNELS	1.00 EA	29,848,647	2,984,865	2,626,681	709,204	3,616,940	39,786,337
01_AA.01.AB STRUCTURES							
01_AA.01.AB_001 RIVER KILOMETER 25	1.00 EA	16,540,820	1,654,082	1,455,592	393,010	2,004,350	22,047,855
STRUCTURES	1.00 EA	16,540,820	1,654,082	1,455,592	393,010	2,004,350	22,047,855
ALTERNATIVE BANK PROTECTION	1.00 EA	46,389,467	4,638,947	4,082,273	1,102,214	5,621,290	61,834,191
ABACAN BASIN	1.00 EA	46,389,467	4,638,947	4,082,273	1,102,214	5,621,290	61,834,191
01_AB PASIG/POTRERO BASIN							
01_AB.01 ALTERNATIVE LEVEE							
01_AB.01.AA CHANNELS							
01_AB.01.AA_001 LEVEE	1.00 EA	970,354,635	97,035,463	85,391,208	23,055,626	117,583,693	1,293,420,625
CHANNELS	1.00 EA	970,354,635	97,035,463	85,391,208	23,055,626	117,583,693	1,293,420,625
01_AB.01.BB STRUCTURES							
01_AB.01.BB_001 RIVER KILOMETER 7.5	1.00 EA	26,798,439	2,679,844	2,358,263	636,731	3,247,328	35,720,604
STRUCTURES	1.00 EA	26,798,439	2,679,844	2,358,263	636,731	3,247,328	35,720,604
ALTERNATIVE LEVEE	1.00 EA	997,153,073	99,715,307	87,749,470	23,692,357	120,831,021	1,329,141,229
01_AB.02 ALTERNATIVE CHANNEL EXC							
01_AB.02.AA CHANNELS							
01_AB.02.AA_001 CHANNEL EXC	1.00 EA	916,110,288	91,611,029	80,617,705	21,766,780	111,010,580	1,221,116,383

		QUANTY UOM	TOTAL DIRECT	VAT TAX	FIELD OH	HOME OH	PROFIT	TOTAL COST
	CHANNELS	1.00 EA	916,110,288	91,611,029	80,617,705	21,766,780	111,010,580	1,221,116,383
	ALTERNATIVE CHANNEL EXC	1.00 EA	916,110,288	91,611,029	80,617,705	21,766,780	111,010,580	1,221,116,383
	PASIG/POTRERO BASIN	1.00 EA	1,913,263,362	191,326,336	168,367,176	45,459,137	231,841,601	2,550,257,612
01_AC	PORAC/GUMAIN BASIN							
01_AC.01	ALTERNATIVE LEVEES							
01_AC.01.AA	CHANNELS							
01_AC.01.AA_001	LEVEES	1.00 EA	352,116,276	35,211,628	30,986,232	8,366,283	42,668,042	469,348,460
	CHANNELS	1.00 EA	352,116,276	35,211,628	30,986,232	8,366,283	42,668,042	469,348,460
	ALTERNATIVE LEVEES	1.00 EA	352,116,276	35,211,628	30,986,232	8,366,283	42,668,042	469,348,460
01_AC.02	ALTERNATIVE CHANNEL EXCAVATION							
01_AC.02.AA	CHANNELS							
01_AC.02.AA_001	CHANNEL EXCAVATION	1.00 EA	334,702,512	33,470,251	29,453,821	7,952,532	40,557,912	446,137,028
	CHANNELS	1.00 EA	334,702,512	33,470,251	29,453,821	7,952,532	40,557,912	446,137,028
	ALTERNATIVE CHANNEL EXCAVATION	1.00 EA	334,702,512	33,470,251	29,453,821	7,952,532	40,557,912	446,137,028
01_AC.03	ALT SEDIMENT RETENTION STRUCTURE							
01_AC.03.AA	CHANNELS							
01_AC.03.AA_001	LEEVES	1.00 EA	353,084,322	35,308,432	31,071,420	8,389,283	42,785,346	470,638,804
	CHANNELS	1.00 EA	353,084,322	35,308,432	31,071,420	8,389,283	42,785,346	470,638,804
01_AC.03.BB	STRUCTURES							
01_AC.03.BB_001	RIVER KILOMETER 18	1.00 EA	48,224,495	4,822,450	4,243,756	1,145,814	5,843,651	64,280,166
01_AC.03.BB_002	RIVER KILOMETER 23.5	1.00 EA	428,693,041	42,869,304	37,724,988	10,185,747	51,947,308	571,420,387
	STRUCTURES	1.00 EA	476,917,536	47,691,754	41,968,743	11,331,561	57,790,959	635,700,553
	ALT SEDIMENT RETENTION STRUCTURE	1.00 EA	830,001,859	83,000,186	73,040,164	19,720,844	100,576,305	1,106,339,357
	PORAC/GUMAIN BASIN	1.00 EA	1,516,820,647	151,682,065	133,480,217	36,039,659	183,802,259	2,021,824,845

		QUANTY	UOM	TOTAL DIRECT	VAT TAX	FIELD OH	HOME OH	PROFIT	TOTAL COST
01_AD	SACOBIA BASIN								
01_AD.01	ALTERNATIVE LEVEES								
01_AD.01.AA	CHANNELS								
01_AD.01.AA_001	LEVEES	1.00	EA	690,846,963	69,084,696	60,794,533	16,414,524	83,714,072	920,854,787
	CHANNELS	1.00	EA	690,846,963	69,084,696	60,794,533	16,414,524	83,714,072	920,854,787
01_AD.01.AB	STRUCTURE								
01_AD.01.AB_001	RIVER KILMETER 16 BAMBAM	1.00	EA	90,621,412	9,062,141	7,974,684	2,153,165	10,981,140	120,792,542
	STRUCTURE	1.00	EA	90,621,412	9,062,141	7,974,684	2,153,165	10,981,140	120,792,542
	ALTERNATIVE LEVEES	1.00	EA	781,468,374	78,146,837	68,769,217	18,567,689	94,695,212	1,041,647,329
01_AD.02	ALTERNATIVE CHANNEL EXC								
01_AD.02.AA	CHANNELS								
01_AD.02.AA_001	CHANNEL EXC	1.00	EA	276,857,598	27,685,760	24,363,469	6,578,137	33,548,496	369,033,459
	CHANNELS	1.00	EA	276,857,598	27,685,760	24,363,469	6,578,137	33,548,496	369,033,459
	ALTERNATIVE CHANNEL EXC	1.00	EA	276,857,598	27,685,760	24,363,469	6,578,137	33,548,496	369,033,459
01_AD.03	ALT MULTIPLE RETENTION STRUCTURE								
01_AD.03.AA	CHANNELS								
01_AD.03.AA_001	LEVEES	1.00	EA	496,912,865	49,691,287	43,728,332	11,806,650	60,213,913	662,353,047
	CHANNELS	1.00	EA	496,912,865	49,691,287	43,728,332	11,806,650	60,213,913	662,353,047
01_AD.03.BB	STRUCTURES								
01_AD.03.BB_001	RIVER KILOMETER 2	1.00	EA	219,947,170	21,994,717	19,355,351	5,225,945	26,652,318	293,175,500
01_AD.03.BB_002	RIVER KILOMETER 6.5	1.00	EA	423,993,042	42,399,304	37,311,388	10,074,075	51,377,781	565,155,589
	STRUCTURES	1.00	EA	643,940,211	64,394,021	56,666,739	15,300,019	78,030,099	858,331,089
	ALT MULTIPLE RETENTION STRUCTURE	1.00	EA	1,140,853,077	114,085,308	100,395,071	27,106,669	138,244,012	1,520,684,136
	SACOBIA BASIN	1.00	EA	2,199,179,049	219,917,905	193,527,756	52,252,494	266,487,720	2,931,364,924

		QUANTY	UOM	TOTAL DIRECT	VAT TAX	FIELD OH	HOME OH	PROFIT	TOTAL COST
01_AE	O'DONNELL BASIN								
01_AE.01	ALTERNATIVE LEVEES								
01_AE.01.AA	CHANNELS								
01_AE.01.AA_001	LEVEES	1.00	EA	128,222,794	12,822,279	11,283,606	3,046,574	15,537,525	170,912,778
	CHANNELS	1.00	EA	128,222,794	12,822,279	11,283,606	3,046,574	15,537,525	170,912,778
	ALTERNATIVE LEVEES	1.00	EA	128,222,794	12,822,279	11,283,606	3,046,574	15,537,525	170,912,778
01_AE.02	ALT CHANNEL EXC								
01_AE.02.AA	Channels								
01_AE.02.AA_001	Levees	1.00	EA	583,468,109	58,346,811	51,345,194	13,863,202	70,702,332	777,725,648
	Channels	1.00	EA	583,468,109	58,346,811	51,345,194	13,863,202	70,702,332	777,725,648
	ALT CHANNEL EXC	1.00	EA	583,468,109	58,346,811	51,345,194	13,863,202	70,702,332	777,725,648
01_AE.03	ALT SEDIMENT RETENTION STRUCTURE								
01_AE.03.AA	CHANNELS								
01_AE.03.AA_001	LEVEES	1.00	EA	100,475,887	10,047,589	8,841,878	2,387,307	12,175,266	133,927,927
	CHANNELS	1.00	EA	100,475,887	10,047,589	8,841,878	2,387,307	12,175,266	133,927,927
01_AE.03.AB	STRUCTURES								
01_AE.03.AB_001	RIVER KILOMETER 33 O'DONNELL	1.00	EA	1,707,950,265	170,795,027	150,299,623	40,580,898	206,962,581	2,276,588,395
01_AE.03.AB_002	RIVER KILOMETER 11 BANGUT	1.00	EA	13,883,287	1,388,329	1,221,729	329,867	1,682,321	18,505,533
01_AE.03.AB_003	RIVER KILOMETER 13.5 BANGUT	1.00	EA	8,219,492	821,949	723,315	195,295	996,005	10,956,057
	STRUCTURES	1.00	EA	1,730,053,044	173,005,304	152,244,668	41,106,060	209,640,908	2,306,049,984
	ALT SEDIMENT RETENTION STRUCTURE	1.00	EA	1,830,528,931	183,052,893	161,086,546	43,493,367	221,816,174	2,439,977,911
	O'DONNELL BASIN	1.00	EA	2,542,219,833	254,221,983	223,715,345	60,403,143	308,056,031	3,388,616,336
01_AF	SANTO TOMAS BASIN								
01_AF.01	ALTERNATIVE LEVEES								

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	QUANTY UOM	TOTAL DIRECT	VAT TAX	FIELD OH	HOME OH	PROFIT	TOTAL COST
01_AF.01.AA CHANNELS							
01_AF.01.AA_001 LEVEES	1.00 EA	526,104,816	52,610,482	46,297,224	12,500,250	63,751,277	701,264,050
CHANNELS	1.00 EA	526,104,816	52,610,482	46,297,224	12,500,250	63,751,277	701,264,050
ALTERNATIVE LEVEES	1.00 EA	526,104,816	52,610,482	46,297,224	12,500,250	63,751,277	701,264,050
01_AF.02 ALT CHANNEL EXC							
01_AF.02.AA CHANNELS							
01_AF.02.AA_001 LEVEES	1.00 EA	1,883,741,755	188,374,175	165,769,274	44,757,704	228,264,291	2,510,907,200
CHANNELS	1.00 EA	1,883,741,755	188,374,175	165,769,274	44,757,704	228,264,291	2,510,907,200
ALT CHANNEL EXC	1.00 EA	1,883,741,755	188,374,175	165,769,274	44,757,704	228,264,291	2,510,907,200
01_AF.03 ALT SEDIMENT RETENTION STRUCTURE							
01_AF.03.AA CHANNELS							
01_AF.03.AA_001 LEVEES	1.00 EA	483,300,677	48,330,068	42,530,460	11,483,224	58,564,443	644,208,871
CHANNELS	1.00 EA	483,300,677	48,330,068	42,530,460	11,483,224	58,564,443	644,208,871
01_AF.03.AB STRUCTURES							
01_AF.03.AB_001 RIVER KILOMETER 7	1.00 EA	2,673,824,298	267,382,430	235,296,538	63,530,065	324,003,333	3,564,036,665
STRUCTURES	1.00 EA	2,673,824,298	267,382,430	235,296,538	63,530,065	324,003,333	3,564,036,665
ALT SEDIMENT RETENTION STRUCTURE	1.00 EA	3,157,124,975	315,712,498	277,826,998	75,013,289	382,567,776	4,208,245,536
SANTO TOMAS BASIN	1.00 EA	5,566,971,547	556,697,155	489,893,496	132,271,244	674,583,344	7,420,416,785
01_AG MALOMA BASIN							
01_AG.01 ALTERNATIVE LEVEES							
01_AG.01.AA CHANNELS							
01_AG.01.AA_001 LEVEES	1.00 EA	47,121,908	4,712,191	4,146,728	1,119,617	5,710,044	62,810,487
CHANNELS	1.00 EA	47,121,908	4,712,191	4,146,728	1,119,617	5,710,044	62,810,487
ALTERNATIVE LEVEES	1.00 EA	47,121,908	4,712,191	4,146,728	1,119,617	5,710,044	62,810,487

	QUANTY UOM	TOTAL DIRECT	VAT TAX	FIELD OH	HOME OH	PROFIT	TOTAL COST
01_AG.02 ALT CHANNEL EXC							
01_AG.02.AA CHANNELS							
01_AG.02.AA_001 CHANNEL EXCAVATION	1.00 EA	74,978,155	7,497,815	6,598,078	1,781,481	9,085,553	99,941,082
CHANNELS	1.00 EA	74,978,155	7,497,815	6,598,078	1,781,481	9,085,553	99,941,082
ALT CHANNEL EXC	1.00 EA	74,978,155	7,497,815	6,598,078	1,781,481	9,085,553	99,941,082
01_AG.03 ALT SEDIMENT RETENTION STRUCTURE							
01_AG.03.AA CHANNELS							
01_AG.03.AA_001 LEVEES	1.00 EA	46,927,558	4,692,756	4,129,625	1,114,999	5,686,494	62,551,432
CHANNELS	1.00 EA	46,927,558	4,692,756	4,129,625	1,114,999	5,686,494	62,551,432
01_AG.03.AB STRUCTURES							
01_AG.03.AB_001 RIVER KILOMETER 19.5	1.00 EA	92,586,463	9,258,646	8,147,609	2,199,854	11,219,257	123,411,829
STRUCTURES	1.00 EA	92,586,463	9,258,646	8,147,609	2,199,854	11,219,257	123,411,829
ALT SEDIMENT RETENTION STRUCTURE	1.00 EA	139,514,021	13,951,402	12,277,234	3,314,853	16,905,751	185,963,261
MALOMA BASIN	1.00 EA	261,614,083	26,161,408	23,022,039	6,215,951	31,701,348	348,714,830
01_AH BUCAO BASIN							
01_AH.01 ALTERNATIVE LEVEES							
01_AH.01.AA CHANNELS							
01_AH.01.AA_001 LEVEES	1.00 EA	107,638,002	10,763,800	9,472,144	2,557,479	13,043,143	143,474,568
CHANNELS	1.00 EA	107,638,002	10,763,800	9,472,144	2,557,479	13,043,143	143,474,568
ALTERNATIVE LEVEES	1.00 EA	107,638,002	10,763,800	9,472,144	2,557,479	13,043,143	143,474,568
01_AH.02 ALT SEDIMENT RETENTION STRUCTURE							
01_AH.02.AA CHANNELS							
01_AH.02.AA_001 LEVEES	1.00 EA	27,460,626	2,746,063	2,416,535	652,464	3,327,569	36,603,256

		QUANTY	UOM	TOTAL DIRECT	VAT TAX	FIELD OH	HOME OH	PROFIT	TOTAL COST
	CHANNELS	1.00	EA	27,460,626	2,746,063	2,416,535	652,464	3,327,569	36,603,256
01_AH.02.AB STRUCTURES									
01_AH.02.AB_001	RIVER KILOMETER 13	1.00	EA	2,683,905,047	268,390,505	236,183,644	63,769,584	325,224,878	3,577,473,657
	STRUCTURES	1.00	EA	2,683,905,047	268,390,505	236,183,644	63,769,584	325,224,878	3,577,473,657
	ALT SEDIMENT RETENTION STRUCTURE	1.00	EA	2,711,365,672	271,136,567	238,600,179	64,422,048	328,552,447	3,614,076,914
	BUCAO BASIN	1.00	EA	2,819,003,674	281,900,367	248,072,323	66,979,527	341,595,589	3,757,551,482
01_AI DELTA									
01_AI.01 EXCAVATION									
01_AI.01.AA CHANNELS									
01_AI.01.AA_001	PASIG DREDGING	1.00	EA	517,416,424	51,741,642	45,532,645	12,293,814	62,698,453	689,682,978
	CHANNELS	1.00	EA	517,416,424	51,741,642	45,532,645	12,293,814	62,698,453	689,682,978
	EXCAVATION	1.00	EA	517,416,424	51,741,642	45,532,645	12,293,814	62,698,453	689,682,978
	DELTA	1.00	EA	517,416,424	51,741,642	45,532,645	12,293,814	62,698,453	689,682,978
	MOUNT PINATUBO RECOVERY	1.00	EA	17,382,878,086	1,738,287,809	1,529,693,272	413,017,183	2,106,387,635	23,170,263,984
	MOUNT PINATUBO, PHILIPPINES	1.00	EA	17,382,878,086	1,738,287,809	1,529,693,272	413,017,183	2,106,387,635	23,170,263,984

01_AA. ABACAN BASIN	QUANTY	UOM	CREW ID	OUTPUT	LABOR	EQUIPMNT	MATERIAL	OTHER	TOTAL COST
01. MOUNT PINATUBO RECOVERY									
01_AA. ABACAN BASIN									
01_AA.01. ALTERNATIVE BANK PROTECTION									
01_AA.01.AA. CHANNELS									
01_AA.01.AA_001. BANK PROTECTION									
USR AA <02200 1001 > EXCAVATION LOADER & TRUCKS	45100	BCM	XCARA	206.00	77,565	1,927,342	0	0	2,004,907
USR AA <02200 1004 > EMBANKMENT RANDOM COMPACTION	18000	BCM	XCARD	500.00	8,440	141,871	0	0	150,311
USR AA <02200 1013 > HYDRO-SEED	1062400	SM	XCARJ	1000.00	75,037	634,072	2,921,600	0	3,630,710
USR AA <02200 1016 > Mountain Soil	24200	CM	XCARD	500.00	11,347	190,738	1,379,400	0	1,581,485
USR AA <02200 1010 > Cobbles/Boulders Fill	67700	BCM	XCARF	10.00	1184412	390,223	18956000	0	20,530,634
USR AA <02200 1018 > Early Warning System	1.00	LS	N/A	0.00	975,300	975,300	0	0	1,950,600
BANK PROTECTION					2332102	4,259,546	23257000	0	29,848,647
CHANNELS					2332102	4,259,546	23257000	0	29,848,647
01_AA.01.AB. STRUCTURES									
01_AA.01.AB_001. RIVER KILOMETER 25									
USR AA <03000 1001 > RCC CONCRETE 4 SACK MIX	12000	CM	XCBGC	50.00	146,688	2,596,958	6,072,000	0	8,815,646
USR AA <03000 2001 > Walls Including Forms, Rebar & Ect.	2050.00	CM	XCBGB	4.00	225,772	1,803,908	2,665,000	0	4,694,679
USR AA <02200 1004 > EMBANKMENT RANDOM COMPACTION	6700.00	BCM	XCARD	500.00	3,142	52,808	0	0	55,949
USR AA <02200 1010 > Cobbles/Boulders Fill	1000.00	BCM	XCARF	10.00	17,495	5,764	280,000	0	303,259
USR AA <02200 1011 > Fine Gravel Fill	240.00	BCM	XCARF	10.00	4,199	1,383	100,800	0	106,382
USR AA <02200 1008 > ROCK Blasting	4500.00	BCM	XCARI	25.00	13,108	324,360	189,000	0	526,468
USR AA <02200 1015 > Load & Haul Rock	7425.00	CM	XCARL	70.00	32,249	772,414	0	0	804,663
USR AA <02200 1001 > EXCAVATION LOADER & TRUCKS	17700	BCM	XCARA	206.00	30,441	756,407	0	0	786,848
USR AA <02400 1101 > Demolition Gabions	500.00	CM	XCARH	1.00	51,115	0	0	0	51,115
USR AA <02000 9999 > CARE & DIVERSION OF WATER	1.00	LS	N/A	0.00	195,850	195,850	0	0	391,700
USR AA <02200 1013 > HYDRO-SEED	200.00	SM	XCARJ	1000.00	14	119	550	0	683
USR AA <02200 1017 > Rock Fill	600.00	CM	XCARM	125.00	447	2,979	0	0	3,426
RIVER KILOMETER 25					720,519	6,512,951	9,307,350	0	16,540,820
STRUCTURES					720,519	6,512,951	9,307,350	0	16,540,820
ALTERNATIVE BANK PROTECTION					3052621	10772496	32564350	0	46,389,467
ABACAN BASIN					3052621	10772496	32564350	0	46,389,467

01_AB. PASIG/POTRERO BASIN	QUANTY	UOM	CREW ID	OUTPUT	LABOR	EQUIPMNT	MATERIAL	OTHER	TOTAL COST
01_AB. PASIG/POTRERO BASIN									
01_AB.01. ALTERNATIVE LEVEE									
01_AB.01.AA. CHANNELS									
01_AB.01.AA_001. LEVEE									
USR AA <02200 1004 > EMBANKMENT RANDOM COMPACTION	10400000	BCM	XCARD	500.00	4876560	81969888	0	0	86,846,448
USR AA <02200 1001 > EXCAVATION LOADER & TRUCKS	10400000	BCM	XCARA	206.00	17886485	444442485	0	0	462,328,971
USR AA <02200 1013 > HYDRO-SEED	1300000	SM	XCARJ	1000.00	91,819	775,879	3,575,000	0	4,442,698
USR AA <03000 2002 > Concrete Slope Protection	276000	CM	XCBGB	6.00	20264380	161911720	151800000	0	333,976,100
USR AA <02200 1010 > Cobbles/Boulders Fill	92400	BCM	XCARF	10.00	1616538	532,594	25872000	0	28,021,132
USR AA <02200 1016 > Mountain Soil	509000	CM	XCARD	500.00	238,670	4,011,795	29013000	0	33,263,466
USR AA <02100 1001 > Dredging 16" Dredge 800 M	500000	CM	XASPA	204.00	1339730	18185490	0	0	19,525,221
USR AA <02200 1018 > Early Warning System	1.00	LS	N/A	0.00	975,300	975,300	0	0	1,950,600
LEVEE					47289483	712805152	210260000	0	970,354,635
CHANNELS					47289483	712805152	210260000	0	970,354,635
01_AB.01.BB. STRUCTURES									
01_AB.01.BB_001. RIVER KILOMETER 7.5									
USR AA <03000 1001 > RCC CONCRETE 4 SACK MIX	15400	CM	XCBGC	50.00	188,250	3,332,763	7,792,400	0	11,313,413
USR AA <03000 2001 > Walls Including Forms, Rebar & Ect.	4692.00	CM	XCBGB	4.00	516,742	4,128,749	6,099,600	0	10,745,091
USR AA <02200 1010 > Cobbles/Boulders Fill	5200.00	BCM	XCARF	10.00	90,974	29,973	1,456,000	0	1,576,947
USR AA <02200 1012 > Coarse Gravel Fill	1432.00	BCM	XCARF	10.00	25,053	8,254	458,240	0	491,547
USR AA <02200 1001 > EXCAVATION LOADER & TRUCKS	32200	BCM	XCARA	206.00	55,379	1,376,062	0	0	1,431,442
USR AA <02200 9999 > CARE & DIVERSION OF WATER	1.00	LS	N/A	0.00	620,000	620,000	0	0	1,240,000
RIVER KILOMETER 7.5					1496397	9,495,801	15806240	0	26,798,439
STRUCTURES					1496397	9,495,801	15806240	0	26,798,439
ALTERNATIVE LEVEE					48785880	722300953	226066240	0	997,153,073
01_AB.02. ALTERNATIVE CHANNEL EXC									
01_AB.02.AA. CHANNELS									
01_AB.02.AA_001. CHANNEL EXC									
USR AA <02200 1001 > EXCAVATION LOADER & TRUCKS	17802150	BCM	XCARA	206.00	30617105	760772288	0	0	791,389,393
USR AA <02200 1004 > EMBANKMENT RANDOM COMPACTION	1330000	BCM	XCARD	500.00	623,637	10482688	0	0	11,106,325
USR AA <02200 1013 > HYDRO-SEED	347800	SM	XCARJ	1000.00	24,565	207,577	956,450	0	1,188,593
USR AA <03000 2002 > Concrete Slope Protection	48600	CM	XCBGB	6.00	3568293	28510542	26730000	0	58,808,835
USR AA <02200 1010 > Cobbles/Boulders Fill	124600	BCM	XCARF	10.00	2179877	718,194	34888000	0	37,786,071
USR AA <02200 1016 > Mountain Soil	212400	CM	XCARD	500.00	99,594	1,674,077	12106800	0	13,880,472
USR AA <02200 1018 > Early Warning System	1.00	LS	N/A	0.00	975,300	975,300	0	0	1,950,600

01_AB. PASIG/POTRERO BASIN

	QUANTY	UOM	CREW ID	OUTPUT	LABOR	EQUIPMNT	MATERIAL	OTHER	TOTAL COST
CHANNEL EXC					38088372	803340667	74681250	0	916,110,288
CHANNELS					38088372	803340667	74681250	0	916,110,288
ALTERNATIVE CHANNEL EXC					38088372	803340667	74681250	0	916,110,288
PASIG/POTRERO BASIN					868742521	525641620	300747490	0	1,913,263,362

01_AC. PORAC/GUMAIN BASIN	QUANTY	UOM	CREW ID	OUTPUT	LABOR	EQUIPMNT	MATERIAL	OTHER	TOTAL COST
01_AC. PORAC/GUMAIN BASIN									
01_AC.01. ALTERNATIVE LEVEES									
01_AC.01.AA. CHANNELS									
01_AC.01.AA_001. LEVEES									
USR AA <02200 1001 > EXCAVATION LOADER & TRUCKS	3777000	BCM	XCARA	206.00	6495890	161409545	0	0	167,905,435
USR AA <02200 1004 > EMBANKMENT RANDOM COMPACTION	878000	BCM	XCARD	500.00	411,694	6,920,150	0	0	7,331,844
USR AA <03000 2002 > Concrete Slope Protection	117100	CM	XCBGB	6.00	8597677	68695154	64405000	0	141,697,831
USR AA <02200 1013 > HYDRO-SEED	336000	SM	XCARJ	1000.00	23,732	200,535	924,000	0	1,148,267
USR AA <02200 1010 > Cobbles/Boulders Fill	76700	BCM	XCARF	10.00	1341867	442,099	21476000	0	23,259,965
USR AA <02200 1016 > Mountain Soil	135000	CM	XCARD	500.00	63,302	1,064,032	7,695,000	0	8,822,334
USR AA <02200 1018 > Early Warning System	1.00	LS	N/A	0.00	975,300	975,300	0	0	1,950,600
LEVEES					17909461	239706815	94500000	0	352,116,276
CHANNELS					17909461	239706815	94500000	0	352,116,276
ALTERNATIVE LEVEES					17909461	239706815	94500000	0	352,116,276
01_AC.02. ALTERNATIVE CHANNEL EXCAVATION									
01_AC.02.AA. CHANNELS									
01_AC.02.AA_001. CHANNEL EXCAVATION									
USR AA <02200 1001 > EXCAVATION LOADER & TRUCKS	5315000	BCM	XCARA	206.00	9141026	227135751	0	0	236,276,777
USR AA <03000 2002 > Concrete Slope Protection	38800	CM	XCBGB	6.00	2848761	22761503	21340000	0	46,950,263
USR AA <02200 1010 > Cobbles/Boulders Fill	149000	BCM	XCARF	10.00	2606755	858,836	41720000	0	45,185,591
USR AA <02200 1016 > Mountain Soil	66400	CM	XCARD	500.00	31,135	523,346	3,784,800	0	4,339,281
USR AA <02200 1018 > Early Warning System	1.00	LS	N/A	0.00	975,300	975,300	0	0	1,950,600
CHANNEL EXCAVATION					15602977	252254736	66844800	0	334,702,512
CHANNELS					15602977	252254736	66844800	0	334,702,512
ALTERNATIVE CHANNEL EXCAVATION					15602977	252254736	66844800	0	334,702,512
01_AC.03. ALT SEDIMENT RETENTION STRUCTURE									
01_AC.03.AA. CHANNELS									
01_AC.03.AA_001. LEEVES									
USR AA <02200 1001 > EXCAVATION LOADER & TRUCKS	3777000	BCM	XCARA	206.00	6495890	161409545	0	0	167,905,435
USR AA <02200 1004 > EMBANKMENT RANDOM COMPACTION	878000	BCM	XCARD	500.00	411,694	6,920,150	0	0	7,331,844
USR AA <03000 2002 > Concrete Slope Protection	117900	CM	XCBGB	6.00	8656415	69164463	64845000	0	142,665,878
USR AA <02200 1013 > HYDRO-SEED	336000	SM	XCARJ	1000.00	23,732	200,535	924,000	0	1,148,267
USR AA <02200 1010 > Cobbles/Boulders Fill	76700	BCM	XCARF	10.00	1341867	442,099	21476000	0	23,259,965
USR AA <02200 1016 > Mountain Soil	135000	CM	XCARD	500.00	63,302	1,064,032	7,695,000	0	8,822,334

01_AC. PORAC/GUMAIN BASIN		QUANTY	UOM	CREW ID	OUTPUT	LABOR	EQUIPMNT	MATERIAL	OTHER	TOTAL COST
USR AA <02200 1018 >	Early Warning System	1.00	LS	N/A	0.00	975,300	975,300	0	0	1,950,600
	LEEVES					17968198	240176124	94940000	0	353,084,322
	CHANNELS					17968198	240176124	94940000	0	353,084,322
01_AC.03.BB. STRUCTURES										
01_AC.03.BB_001. RIVER KILOMETER 18										
USR AA <02200 1001 >	EXCAVATION LOADER & TRUCKS	113600	BCM	XCARA	206.00	195,375	4,854,679	0	0	5,050,055
USR AA <02200 1004 >	EMBANKMENT RANDOM COMPACTION	45300	BCM	XCARD	500.00	21,241	357,042	0	0	378,283
USR AA <03000 1001 >	RCC CONCRETE 4 SACK MIX	31800	CM	XCBGC	50.00	388,723	6,881,940	16090800	0	23,361,463
USR AA <03000 2001 >	Walls Including Forms, Rebar & Ect.	6155.00	CM	XCBGB	4.00	677,866	5,416,123	8,001,500	0	14,095,489
USR AA <02200 1010 >	Cobbles/Boulders Fill	10300	BCM	XCARF	10.00	180,199	59,369	2,884,000	0	3,123,568
USR AA <02200 1011 >	Fine Gravel Fill	2405.00	BCM	XCARF	10.00	42,075	13,862	1,010,100	0	1,066,038
USR AA <02200 9999 >	Care & Diversion Of Water	1.00	JOB	N/A	0.00	574,800	574,800	0	0	1,149,600
	RIVER KILOMETER 18					2080279	18157816	27986400	0	48,224,495
01_AC.03.BB_002. RIVER KILOMETER 23.5										
USR AA <02200 1001 >	EXCAVATION LOADER & TRUCKS	260700	BCM	XCARA	206.00	448,366	11140977	0	0	11,589,343
USR AA <03000 1001 >	RCC CONCRETE 4 SACK MIX	490300	CM	XCBGC	50.00	5993427	106107392	248091800	0	360,192,619
USR AA <03000 2001 >	Walls Including Forms, Rebar & Ect.	20054	CM	XCBGB	4.00	2208597	17646618	26070200	0	45,925,415
USR AA <02200 1010 >	Cobbles/Boulders Fill	23400	BCM	XCARF	10.00	409,383	134,878	6,552,000	0	7,096,261
USR AA <02200 1011 >	Fine Gravel Fill	2475.00	BCM	XCARF	10.00	43,300	14,266	1,039,500	0	1,097,066
USR AA <02200 1014 >	1 Meter Conc Pipe	1200.00	LM	XCARK	2.00	77,730	476,208	1,310,400	0	1,864,338
USR AA <02200 9999 >	Care & Diversion Of Water	1.00	JOB	N/A	0.00	464,000	464,000	0	0	928,000
	RIVER KILOMETER 23.5					9644804	135984338	283063900	0	428,693,041
	STRUCTURES					11725083	154142153	311050300	0	476,917,536
	ALT SEDIMENT RETENTION STRUCTURE					29693281	394318277	405990300	0	830,001,859
	PORAC/GUMAIN BASIN					63205719	886279828	567335100	0	1,516,820,647

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01_AD. SACOBIA BASIN	QUANTY	UOM	CREW ID	OUTPUT	LABOR	EQUIPMNT	MATERIAL	OTHER	TOTAL COST
01_AD. SACOBIA BASIN									
01_AD.01. ALTERNATIVE LEVEES									
01_AD.01.AA. CHANNELS									
01_AD.01.AA_001. LEVEES									
USR AA <02200 1001 > EXCAVATION LOADER & TRUCKS	7417000	BCM	XCARA	206.00	12756160	316964415	0	0	329,720,575
USR AA <02200 1004 > EMBANKMENT RANDOM COMPACTION	7417000	BCM	XCARD	500.00	3477831	58458717	0	0	61,936,549
USR AA <03000 2002 > Concrete Slope Protection	226600	CM	XCBGB	6.00	16637350	132931869	124630000	0	274,199,218
USR AA <02200 1013 > HYDRO-SEED	1569200	SM	XCARJ	1000.00	110,833	936,546	4,315,300	0	5,362,678
USR AA <02200 1016 > Mountain Soil	270500	CM	XCARD	500.00	126,837	2,132,005	15418500	0	17,677,343
USR AA <02200 1018 > Early Warning System	1.00	LS	N/A	0.00	975,300	975,300	0	0	1,950,600
LEVEES					34084311	512398852	144363800	0	690,846,963
CHANNELS					34084311	512398852	144363800	0	690,846,963
01_AD.01.AB. STRUCTURE									
01_AD.01.AB_001. RIVER KILMETER 16 BAMBAM									
USR AA <02200 1001 > EXCAVATION LOADER & TRUCKS	147000	BCM	XCARA	206.00	252,819	6,282,024	0	0	6,534,842
USR AA <02200 1004 > EMBANKMENT RANDOM COMPACTION	28000	BCM	XCARD	500.00	13,129	220,688	0	0	233,817
USR AA <03000 1001 > RCC CONCRETE 4 SACK MIX	71500	CM	XCBGC	50.00	874,016	15473544	36179000	0	52,526,560
USR AA <03000 2001 > Walls Including Forms, Rebar & Ect.	9533.00	CM	XCBGB	4.00	1049893	8,388,611	12392900	0	21,831,404
USR AA <02200 1010 > Cobbles/Boulders Fill	21900	BCM	XCARF	10.00	383,140	126,232	6,132,000	0	6,641,372
USR AA <02200 1012 > Coarse Gravel Fill	2624.00	BCM	XCARF	10.00	45,907	15,125	839,680	0	900,712
USR AA <02200 1014 > 1 Meter Conc Pipe	40.00	LM	XCARK	2.00	2,591	15,874	43,680	0	62,145
USR AA <02200 9999 > Care & Diversion Of Water	1.00	LS	N/A	0.00	945,280	945,280	0	0	1,890,560
RIVER KILMETER 16 BAMBAM					3566775	31467377	55587260	0	90,621,412
STRUCTURE					3566775	31467377	55587260	0	90,621,412
ALTERNATIVE LEVEES					37651086	543866228	199951060	0	781,468,374
01_AD.02. ALTERNATIVE CHANNEL EXC									
01_AD.02.AA. CHANNELS									
01_AD.02.AA_001. CHANNEL EXC									
USR AA <02200 1001 > EXCAVATION LOADER & TRUCKS	5132100	BCM	XCARA	206.00	8826465	219319546	0	0	228,146,011
USR AA <02200 1004 > EMBANKMENT RANDOM COMPACTION	481000	BCM	XCARD	500.00	225,541	3,791,107	0	0	4,016,648
USR AA <03000 2002 > Concrete Slope Protection	29600	CM	XCBGB	6.00	2173281	17364445	16280000	0	35,817,727
USR AA <02200 1013 > HYDRO-SEED	1086000	SM	XCARJ	1000.00	76,704	648,157	2,986,500	0	3,711,362
USR AA <02200 1016 > Mountain Soil	49200	CM	XCARD	500.00	23,070	387,781	2,804,400	0	3,215,251
USR AA <02200 1018 > Early Warning System	1.00	LS	N/A	0.00	975,300	975,300	0	0	1,950,600

01_AD. SACOBIA BASIN	QUANTY	UOM	CREW ID	OUTPUT	LABOR	EQUIPMNT	MATERIAL	OTHER	TOTAL COST
CHANNEL EXC				12300361	242486337	22070900		0	276,857,598
CHANNELS				12300361	242486337	22070900		0	276,857,598
ALTERNATIVE CHANNEL EXC				12300361	242486337	22070900		0	276,857,598
01_AD.03. ALT MULTIPLE RETENTION STRUCTURE									
01_AD.03.AA. CHANNELS									
01_AD.03.AA_001. LEVEES									
USR AA <02200 1001 > EXCAVATION LOADER & TRUCKS	3430000	BCM	XCARA	206.00	5899100	146580550	0	0	152,479,651
USR AA <02200 1004 > EMBANKMENT RANDOM COMPACTION	3430000	BCM	XCARD	500.00	1608327	27034300	0	0	28,642,627
USR AA <03000 2002 > Concrete Slope Protection	246000	CM	XCBGB	6.00	18061730	144312620	135300000	0	297,674,350
USR AA <02200 1013 > HYDRO-SEED	1250000	SM	XCARJ	1000.00	88,288	746,038	3,437,500	0	4,271,825
USR AA <02200 1016 > Mountain Soil	182000	CM	XCARD	500.00	85,340	1,434,473	10374000	0	11,893,813
USR AA <02200 1018 > Early Warning System	1.00	LS	N/A	0.00	975,300	975,300	0	0	1,950,600
LEVEES				26718085	321083281	149111500		0	496,912,865
CHANNELS				26718085	321083281	149111500		0	496,912,865
01_AD.03.BB. STRUCTURES									
01_AD.03.BB_001. RIVER KILOMETER 2									
USR AA <02200 1001 > EXCAVATION LOADER & TRUCKS	263900	BCM	XCARA	206.00	453,870	11277728	0	0	11,731,598
USR AA <02200 1004 > EMBANKMENT RANDOM COMPACTION	77800	BCM	XCARD	500.00	36,480	613,198	0	0	649,678
USR AA <03000 1001 > RCC CONCRETE 4 SACK MIX	224700	CM	XCBGC	50.00	2746733	48628046	113698200	0	165,072,979
USR AA <03000 2001 > Walls Including Forms, Rebar & Ect.	12959	CM	XCBGB	4.00	1427207	11403337	16846700	0	29,677,244
USR AA <02200 1010 > Cobbles/Boulders Fill	23200	BCM	XCARF	10.00	405,884	133,725	6,496,000	0	7,035,609
USR AA <02200 1011 > Fine Gravel Fill	2572.00	BCM	XCARF	10.00	44,997	14,825	1,080,240	0	1,140,062
USR AA <02200 9999 > Care & Diversion Of Water	1.00	LS	N/A	0.00	2320000	2,320,000	0	0	4,640,000
RIVER KILOMETER 2				7435171	74390859	138121140		0	219,947,170
01_AD.03.BB_002. RIVER KILOMETER 6.5									
USR AA <02200 1001 > EXCAVATION LOADER & TRUCKS	422600	BCM	XCARA	206.00	726,810	18059749	0	0	18,786,560
USR AA <02200 1004 > EMBANKMENT RANDOM COMPACTION	319380	BCM	XCARD	500.00	149,757	2,517,264	0	0	2,667,021
USR AA <03000 1001 > RCC CONCRETE 4 SACK MIX	415600	CM	XCBGC	50.00	5080294	89941326	210293600	0	305,315,220
USR AA <03000 2001 > Walls Including Forms, Rebar & Ect.	24997	CM	XCBGB	4.00	2752982	21996235	32496100	0	57,245,317
USR AA <02200 1010 > Cobbles/Boulders Fill	43000	BCM	XCARF	10.00	752,285	247,852	12040000	0	13,040,137
USR AA <02200 1011 > Fine Gravel Fill	3134.00	BCM	XCARF	10.00	54,829	18,064	1,316,280	0	1,389,174
USR AA <02200 1014 > 1 Meter Conc Pipe	70.00	LM	XCARK	2.00	4;534	27,779	76,440	0	108,753
USR AA <02200 1013 > HYDRO-SEED	25600	SM	XCARJ	1000.00	1,808	15,279	70,400	0	87,487
USR AA <02200 9999 > Care & Diversion Of Water	1.00	LS	N/A	0.00	4430000	4,430,000	0	0	8,860,000

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01_AD. SACOBIA BASIN	QUANTY	UOM	CREW ID	OUTPUT	LABOR	EQUIPMNT	MATERIAL	OTHER	TOTAL COST
USR AA <02200 1010 > Cobbles/Boulders Fill	8500.00	BCM	XCARF	10.00	148,708	48,994	2,380,000	0	2,577,702
USR AA <03000 2002 > Concrete Slope Protection	11500	CM	XCBGB	6.00	844,349	6,746,322	6,325,000	0	13,915,671
RIVER KILOMETER 6.5					14946358	144048864	264997820	0	423,993,042
STRUCTURES					22381529	218439723	403118960	0	643,940,211
ALT MULTIPLE RETENTION STRUCTURE					49099613	539523003	552230460	0	1,140,853,077
SACOBIA BASIN					990510601325875568		774252420	0	2,199,179,049

01_AE. O'DONNELL BASIN	QUANTY	UOM	CREW ID	OUTPUT	LABOR	EQUIPMNT	MATERIAL	OTHER	TOTAL COST
01_AE. O'DONNELL BASIN									
01_AE.01. ALTERNATIVE LEVEES									
01_AE.01.AA. CHANNELS									
01_AE.01.AA_001. LEVEES									
USR AA <02200 1001 > EXCAVATION LOADER & TRUCKS	973000	BCM	XCARA	206.00	1673418	41581013	0	0	43,254,432
USR AA <02200 1004 > EMBANKMENT RANDOM COMPACTION	973000	BCM	XCARD	500.00	456,240	7,668,914	0	0	8,125,153
USR AA <03000 2002 > Concrete Slope Protection	29200	CM	XCBGB	6.00	2143913	17129791	16060000	0	35,333,703
USR AA <02200 1013 > HYDRO-SEED	557700	SM	XCARJ	1000.00	39,390	332,852	1,533,675	0	1,905,917
USR AA <02200 1016 > Mountain Soil	213700	CM	XCARD	500.00	100,204	1,684,324	12180900	0	13,965,427
USR AA <02200 1010 > Cobbles/Boulders Fill	78110	BCM	XCARF	10.00	1366534	450,226	21870800	0	23,687,560
USR AA <02200 1018 > Early Warning System	1.00	LS	N/A	0.00	975,300	975,300	0	0	1,950,600
LEVEES					6754999	69822419	51645375	0	128,222,794
CHANNELS					6754999	69822419	51645375	0	128,222,794
ALTERNATIVE LEVEES					6754999	69822419	51645375	0	128,222,794
01_AE.02. ALT CHANNEL EXC									
01_AE.02.AA. Channels									
01_AE.02.AA_001. Levees									
USR AA <02200 1001 > EXCAVATION LOADER & TRUCKS	12529500	BCM	XCARA	206.00	21548915	535446358	0	0	556,995,273
USR AA <02200 1013 > HYDRO-SEED	332100	SM	XCARJ	1000.00	23,456	198,207	913,275	0	1,134,938
USR AA <02200 1010 > Cobbles/Boulders Fill	53200	BCM	XCARF	10.00	930,734	306,645	14896000	0	16,133,379
USR AA <02200 1016 > Mountain Soil	111000	CM	XCARD	500.00	52,048	874,871	6,327,000	0	7,253,919
USR AA <02200 1018 > Early Warning System	1.00	LS	N/A	0.00	975,300	975,300	0	0	1,950,600
Levees					23530453	537801381	22136275	0	583,468,109
Channels					23530453	537801381	22136275	0	583,468,109
ALT CHANNEL EXC					23530453	537801381	22136275	0	583,468,109
01_AE.03. ALT SEDIMENT RETENTION STRUCTURE									
01_AE.03.AA. CHANNELS									
01_AE.03.AA_001. LEVEES									
USR AA <02200 1001 > EXCAVATION LOADER & TRUCKS	613000	BCM	XCARA	206.00	1054271	26196466	0	0	27,250,736
USR AA <02200 1004 > EMBANKMENT RANDOM COMPACTION	613000	BCM	XCARD	500.00	287,436	4,831,494	0	0	5,118,930
USR AA <03000 2002 > Concrete Slope Protection	23900	CM	XCBGB	6.00	1754778	14020616	13145000	0	28,920,394
USR AA <02200 1013 > HYDRO-SEED	504900	SM	XCARJ	1000.00	35,661	301,339	1,388,475	0	1,725,476
USR AA <02200 1010 > Cobbles/Boulders Fill	74900	BCM	XCARF	10.00	1310376	431,724	20972000	0	22,714,099
USR AA <02200 1016 > Mountain Soil	195800	CM	XCARD	500.00	91,811	1,543,241	11160600	0	12,795,651

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01_AE. O'DONNELL BASIN		QUANTY	UOM	CREW ID	OUTPUT	LABOR	EQUIPMNT	MATERIAL	OTHER	TOTAL COST
USR AA <02200 1018 >	Early Warning System	1.00	LS	N/A	0.00	975,300	975,300	0	0	1,950,600
	LEVEES					5509631	48300180	46666075	0	100,475,887
	CHANNELS					5509631	48300180	46666075	0	100,475,887
01_AE.03.AB. STRUCTURES										
01_AE.03.AB_001. RIVER KILOMETER 33 O'DONNELL										
USR AA <02200 1001 >	EXCAVATION LOADER & TRUCKS	7476000	BCM	XCARA	206.00	12857631	319485771	0	0	332,343,403
USR AA <02200 1004 >	EMBANKMENT RANDOM COMPACTION	7476000	BCM	XCARD	500.00	3505496	58923739	0	0	62,429,235
USR AA <02200 1008 >	ROCK Blasting	2505454	BCM	XCARI	25.00	7297886	180593124	105229068	0	293,120,079
USR AA <02200 1015 >	Load & Haul Rock	4134000	CM	XCARL	70.00	17955143	430055295	0	0	448,010,439
USR AA <03000 1001 >	RCC CONCRETE 4 SACK MIX	72000	CM	XCBGC	50.00	880,128	15581750	36432000	0	52,893,878
USR AA <03000 2001 >	Walls Including Forms, Rebar & Ect.	25600	CM	XCBGB	4.00	2819392	22526848	33280000	0	58,626,240
USR AA <02200 1011 >	Fine Gravel Fill	414000	BCM	XCARF	10.00	7242930	2,386,296	173880000	0	183,509,226
USR AA <02200 1012 >	Coarse Gravel Fill	547000	BCM	XCARF	10.00	9569765	3,152,908	175040000	0	187,762,673
USR AA <02200 1009 >	FILTER FABRIC	445000	SM	XCARH	100.00	454,924	0	2,781,250	0	3,236,174
USR AA <02200 1014 >	1 Meter Conc Pipe	270.00	LM	XCARK	2.00	17,489	107,147	294,840	0	419,476
USR AA <02200 1017 >	Rock Fill	4134000	CM	XCARM	125.00	3081980	20525806	0	0	23,607,786
USR AA <03000 1002 >	RCC CONCRETE 7 SACK MIX	32125	CM	XCBGC	50.00	392,696	6,952,274	28446688	0	35,791,658
USR AA <02200 9999 >	Care & Diversion Of Water	1.00	LS	N/A	0.00	13100000	13100000	0	0	26,200,000
	RIVER KILOMETER 33 O'DONNELL					791754611	073390959	555383846	0	1,707,950,265
01_AE.03.AB_002. RIVER KILOMETER 11 BANGUT										
USR AA <03000 1001 >	RCC CONCRETE 4 SACK MIX	8000.00	CM	XCBGC	50.00	97,792	1,731,306	4,048,000	0	5,877,098
USR AA <03000 2001 >	Walls Including Forms, Rebar & Ect.	2413.00	CM	XCBGB	4.00	265,750	2,123,331	3,136,900	0	5,525,981
USR AA <02200 1010 >	Cobbles/Boulders Fill	1700.00	BCM	XCARF	10.00	29,742	9,799	476,000	0	515,540
USR AA <02200 1011 >	Fine Gravel Fill	358.00	BCM	XCARF	10.00	6,263	2,064	150,360	0	158,687
USR AA <02200 1008 >	ROCK Blasting	3600.00	BCM	XCARI	25.00	10,486	259,488	151,200	0	421,174
USR AA <02200 1015 >	Load & Haul Rock	5940.00	CM	XCARL	70.00	25,799	617,931	0	0	643,731
USR AA <02200 1001 >	EXCAVATION LOADER & TRUCKS	10300	BCM	XCARA	206.00	17,715	440,169	0	0	457,883
USR AA <02200 9999 >	Care & Diversion Of Water	1.00	LS	N/A	0.00	141,596	141,597	0	0	283,193
	RIVER KILOMETER 11 BANGUT					595,142	5,325,685	7,962,460	0	13,883,287
01_AE.03.AB_003. RIVER KILOMETER 13.5 BANGUT										
USR AA <03000 1001 >	RCC CONCRETE 4 SACK MIX	4800.00	CM	XCBGC	50.00	58,675	1,038,783	2,428,800	0	3,526,259
USR AA <03000 2001 >	Walls Including Forms, Rebar & Ect.	1330.00	CM	XCBGB	4.00	146,476	1,170,340	1,729,000	0	3,045,816
USR AA <02200 1010 >	Cobbles/Boulders Fill	700.00	BCM	XCARF	10.00	12,247	4,035	196,000	0	212,281
USR AA <02200 1011 >	Fine Gravel Fill	150.00	BCM	XCARF	10.00	2,624	865	63,000	0	66,489
USR AA <02200 1008 >	ROCK Blasting	2900.00	BCM	XCARI	25.00	8,447	209,032	121,800	0	339,279
USR AA <02200 1015 >	Load & Haul Rock	4785.00	CM	XCARL	70.00	20,783	497,778	0	0	518,561
USR AA <02200 1001 >	EXCAVATION LOADER & TRUCKS	7400.00	BCM	XCARA	206.00	12,727	316,238	0	0	328,965

01_AE. O'DONNELL BASIN

		QUANTY	UOM	CREW ID	OUTPUT	LABOR	EQUIPMNT	MATERIAL	OTHER	TOTAL COST
USR AA <02200 1004 >	EMBANKMENT RANDOM COMPACTION	1700.00	BCM	XCARD	500.00	797	13,399	0	0	14,196
USR AA <02200 9999 >	Care & Diversion Of Water	1.00	LS	N/A	0.00	83,823	83,823	0	0	167,646
	RIVER KILOMETER 13.5 BANGUT					346,599	3,334,293	4,538,600	0	8,219,492
	STRUCTURES					801172021	082050937	567884906	0	1,730,053,044
	ALT SEDIMENT RETENTION STRUCTURE					856268331	130351117	614550981	0	1,830,528,931
	O'DONNELL BASIN					********	1737974917	688332631	0	2,542,219,833

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01_AF. SANTO TOMAS BASIN	QUANTY	UOM	CREW ID	OUTPUT	LABOR	EQUIPMNT	MATERIAL	OTHER	TOTAL COST
01_AF. SANTO TOMAS BASIN									
01_AF.01. ALTERNATIVE LEVEES									
01_AF.01.AA. CHANNELS									
01_AF.01.AA_001. LEVEES									
USR AA <02200 1001 > EXCAVATION LOADER & TRUCKS	5470000	BCM	XCARA	206.00	9407603	233759653	0	0	243,167,257
USR AA <02200 1004 > EMBANKMENT RANDOM COMPACTION	5470000	BCM	XCARD	500.00	2564883	43113008	0	0	45,677,891
USR AA <02200 1013 > HYDRO-SEED	972000	SM	XCARJ	1000.00	68,652	580,119	2,673,000	0	3,321,771
USR AA <03000 2002 > Concrete Slope Protection	176000	CM	XCBGB	6.00	12922213	103248053	96800000	0	212,970,267
USR AA <02200 1016 > Mountain Soil	291000	CM	XCARD	500.00	136,450	2,293,581	16587000	0	19,017,030
USR AA <02200 1018 > Early Warning System	1.00	LS	N/A	0.00	975,300	975,300	0	0	1,950,600
LEVEES					26075102	383969714	116060000	0	526,104,816
CHANNELS					26075102	383969714	116060000	0	526,104,816
ALTERNATIVE LEVEES					26075102	383969714	116060000	0	526,104,816
01_AF.02. ALT CHANNEL EXC									
01_AF.02.AA. CHANNELS									
01_AF.02.AA_001. LEVEES									
USR AA <02200 1001 > EXCAVATION LOADER & TRUCKS	40065700	BCM	XCARA	206.00	68907169171220185 5		0	0	1,781,109,024
USR AA <02200 1004 > EMBANKMENT RANDOM COMPACTION	394000	BCM	XCARD	500.00	184,747	3,105,398	0	0	3,290,144
USR AA <02200 1013 > HYDRO-SEED	104444	SM	XCARJ	1000.00	7,377	62,335	287,221	0	356,933
USR AA <03000 2002 > Concrete Slope Protection	78500	CM	XCBGB	6.00	5763601	46050978	43175000	0	94,989,579
USR AA <02200 1016 > Mountain Soil	31300	CM	XCARD	500.00	14,677	246,698	1,784,100	0	2,045,474
USR AA <02200 1018 > Early Warning System	1.00	LS	N/A	0.00	975,300	975,300	0	0	1,950,600
LEVEES					758528701762642564		45246321	0	1,883,741,755
CHANNELS					758528701762642564		45246321	0	1,883,741,755
ALT CHANNEL EXC					758528701762642564		45246321	0	1,883,741,755
01_AF.03. ALT SEDIMENT RETENTION STRUCTURE									
01_AF.03.AA. CHANNELS									
01_AF.03.AA_001. LEVEES									
USR AA <02200 1001 > EXCAVATION LOADER & TRUCKS	5210000	BCM	XCARA	206.00	8960441	222648591	0	0	231,609,033
USR AA <02200 1004 > EMBANKMENT RANDOM COMPACTION	5210000	BCM	XCARD	500.00	2442969	41063761	0	0	43,506,730
USR AA <02200 1013 > HYDRO-SEED	866000	SM	XCARJ	1000.00	61,166	516,855	2,381,500	0	2,959,520
USR AA <03000 2002 > Concrete Slope Protection	154000	CM	XCBGB	6.00	11306937	90342047	84700000	0	186,348,983
USR AA <02200 1016 > Mountain Soil	259000	CM	XCARD	500.00	121,445	2,041,365	14763000	0	16,925,811
USR AA <02200 1018 > Early Warning System	1.00	LS	N/A	0.00	975,300	975,300	0	0	1,950,600

01_AF. SANTO TOMAS BASIN		QUANTY	UOM	CREW ID	OUTPUT	LABOR	EQUIPMNT	MATERIAL	OTHER	TOTAL COST
	LEVEES					23868258	357587919	101844500	0	483,300,677
	CHANNELS					23868258	357587919	101844500	0	483,300,677
01_AF.03.AB. STRUCTURES										
01_AF.03.AB_001. RIVER KILOMETER 7										
USR AA <02200 1001 >	EXCAVATION LOADER & TRUCKS	6009800	BCM	XCARA	206.0010335981	256827928		0	0	267,163,909
USR AA <02200 1008 >	ROCK Blasting	5698200	BCM	XCARI	25.00165977 17	410726256		239324400	0	666,648,373
USR AA <02200 1015 >	Load & Haul Rock	9402030	CM	XCARL	70.0040835703	978082436		0	0	1,018,918,138
USR AA <02200 1004 >	EMBANKMENT RANDOM COMPACTION	6143000	BCM	XCARD	500.00	2880453	48417406	0	0	51,297,859
USR AA <03000 1001 >	RCC CONCRETE 4 SACK MIX	50000	CM	XCBGC	50.00	611,200	10820660	25300000	0	36,731,860
USR AA <03000 1002 >	RCC CONCRETE 7 SACK MIX	125000	CM	XCBGC	50.00	1528000	27051650	110687500	0	139,267,150
USR AA <03000 2001 >	Walls Including Forms, Rebar & Ect.	21900	CM	XCBGB	4.00	2411902	19271015	28470000	0	50,152,916
USR AA <02200 1010 >	Cobbles/Boulders Fill	260000	BCM	XCARF	10.00	4548700	1,498,640	72800000	0	78,847,340
USR AA <02200 1011 >	Fine Gravel Fill	287000	BCM	XCARF	10.00	5021065	1,654,268	120540000	0	127,215,333
USR AA <02200 1012 >	Coarse Gravel Fill	486000	BCM	XCARF	10.00	8502570	2,801,304	155520000	0	166,823,874
USR AA <02200 1009 >	FILTER FABRIC	297000	SM	XCARH	100.00	303,623	0	1,856,250	0	2,159,873
USR AA <02200 1014 >	1 Meter Conc Pipe	300.00	LM	XCARK	2.00	19,433	119,052	327,600	0	466,085
USR AA <02200 1013 >	HYDRO-SEED	80000	SM	XCARJ	1000.00	5,650	47,746	220,000	0	273,397
USR AA <02200 1017 >	Rock Fill	2847000	CM	XCARM	125.00	2122495	14135697	0	0	16,258,192
USR AA <02200 9999 >	Care & Diversion Of Water	1.00	LS	N/A	0.00	25800000	25800000	0	0	51,600,000
•	RIVER KILOMETER 7				•	*********	1797254057	755045750	0	2,673,824,298
	STRUCTURES					*********	1797254057	755045750	0	2,673,824,298
	ALT SEDIMENT RETENTION STRUCTURE					*********	2154841976	856890250	0	3,157,124,975
	SANTO TOMAS BASIN					*********	43014542551018196571		0	5,566,971,547

01_AG. MALOMA BASIN	QUANTY	UOM	CREW ID	OUTPUT	LABOR	EQUIPMNT	MATERIAL	OTHER	TOTAL COST
01_AG. MALOMA BASIN									
01_AG.01. ALTERNATIVE LEVEES									
01_AG.01.AA. CHANNELS									
01_AG.01.AA_001. LEVEES									
USR AA <02200 1001 > EXCAVATION LOADER & TRUCKS	476300	BCM	XCARA	206.00	819,167	20354611	0	0	21,173,778
USR AA <02200 1004 > EMBANKMENT RANDOM COMPACTION	360000	BCM	XCARD	500.00	168,804	2,837,419	0	0	3,006,223
USR AA <02200 1013 > HYDRO-SEED	137700	SM	XCARJ	1000.00	9,726	82,183	378,675	0	470,584
USR AA <02200 1010 > Cobbles/Boulders Fill	52100	BCM	XCARF	10.00	911,489	300,304	14588000	0	15,799,794
USR AA <02200 1016 > Mountain Soil	72240	CM	XCARD	500.00	33,873	569,375	4,117,680	0	4,720,929
USR AA <02200 1018 > Early Warning System	1.00	LS	N/A	0.00	975,300	975,300	0	0	1,950,600
LEVEES					2918359	25119194	19084355	0	47,121,908
CHANNELS					2918359	25119194	19084355	0	47,121,908
ALTERNATIVE LEVEES					2918359	25119194	19084355	0	47,121,908
01_AG.02. ALT CHANNEL EXC									
01_AG.02.AA. CHANNELS									
01_AG.02.AA_001. CHANNEL EXCAVATION									
USR AA <02200 1001 > EXCAVATION LOADER & TRUCKS	1145000	BCM	XCARA	206.00	1969233	48931408	0	0	50,900,642
USR AA <02200 1010 > Cobbles/Boulders Fill	64400	BCM	XCARF	10.00	1126678	371,202	18032000	0	19,529,880
USR AA <02200 1016 > Mountain Soil	39740	CM	XCARD	500.00	18,634	313,220	2,265,180	0	2,597,034
USR AA <02200 1018 > Early Warning System	1.00	LS	N/A	0.00	975,300	975,300	0	0	1,950,600
CHANNEL EXCAVATION					4089845	50591129	20297180	0	74,978,155
CHANNELS					4089845	50591129	20297180	0	74,978,155
ALT CHANNEL EXC					4089845	50591129	20297180	0	74,978,155
01_AG.03. ALT SEDIMENT RETENTION STRUCTURE									
01_AG.03.AA. CHANNELS									
01_AG.03.AA_001. LEVEES									
USR AA <02200 1001 > EXCAVATION LOADER & TRUCKS	476300	BCM	XCARA	206.00	819,167	20354611	0	0	21,173,778
USR AA <02200 1004 > EMBANKMENT RANDOM COMPACTION	336000	BCM	XCARD	500.00	157,550	2,648,258	0	0	2,805,808
USR AA <02200 1013 > HYDRO-SEED	137700	SM	XCARJ	1000.00	9,726	82,183	378,675	0	470,584
USR AA <02200 1010 > Cobbles/Boulders Fill	52120	BCM	XCARF	10.00	911,839	300,420	14593600	0	15,805,859
USR AA <02200 1016 > Mountain Soil	72240	CM	XCARD	500.00	33,873	569,375	4,117,680	0	4,720,929
USR AA <02200 1018 > Early Warning System	1.00	LS	N/A	0.00	975,300	975,300	0	0	1,950,600

01_AG. MALOMA BASIN

		QUANTY	UOM	CREW ID	OUTPUT	LABOR	EQUIPMNT	MATERIAL	OTHER	TOTAL COST
	LEVEES					2907456	24930148	19089955	0	46,927,558
	CHANNELS					2907456	24930148	19089955	0	46,927,558
01_AG.03.AB. STRUCTURES										
01_AG.03.AB_001. RIVER KILOMETER 19.5										
USR AA <02200 1001 >	EXCAVATION LOADER & TRUCKS	70300	BCM	XCARA	206.00	120,906	3,004,260	0	0	3,125,166
USR AA <03000 1001 >	RCC CONCRETE 4 SACK MIX	85800	CM	XCBGC	50.00	1048819	18568253	43414800	0	63,031,872
USR AA <03000 2001 >	Walls Including Forms, Rebar & Ect.	8994.00	CM	XCBGB	4.00	990,532	7,914,315	11692200	0	20,597,047
USR AA <02200 1010 >	Cobbles/Boulders Fill	11200	BCM	XCARF	10.00	195,944	64,557	3,136,000	0	3,396,501
USR AA <02200 1011 >	Fine Gravel Fill	1700.00	BCM	XCARF	10.00	29,742	9,799	714,000	0	753,540
USR AA <02200 1014 >	1 Meter Conc Pipe	50.00	LM	XCARK	2.00	3,239	19,842	54,600	0	77,681
USR AA <02200 9999 >	Care & Diversion Of Water	1.00	LS	N/A	0.00	802,328	802,328	0	0	1,604,656
	RIVER KILOMETER 19.5					3191509	30383354	59011600	0	92,586,463
	STRUCTURES					3191509	30383354	59011600	0	92,586,463
	ALT SEDIMENT RETENTION STRUCTURE					6098964	55313501	78101555	0	139,514,021
	MALOMA BASIN					13107169	131023824	117483090	0	261,614,083

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01_AH. BUCAO BASIN	QUANTY	UOM	CREW ID	OUTPUT	LABOR	EQUIPMNT	MATERIAL	OTHER	TOTAL COST
01_AH. BUCAO BASIN									
01_AH.01. ALTERNATIVE LEVEES									
01_AH.01.AA. CHANNELS									
01_AH.01.AA_001. LEVEES									
USR AA <02200 1001 > EXCAVATION LOADER & TRUCKS	1230000	BCM	XCARA	206.00	2115421	52563871	0	0	54,679,292
USR AA <02200 1004 > EMBANKMENT RANDOM COMPACTION	1230000	BCM	XCARD	500.00	576,747	9,694,516	0	0	10,271,263
USR AA <02200 1013 > HYDRO-SEED	230000	SM	XCARJ	1000.00	16,245	137,271	632,500	0	786,016
USR AA <03000 2002 > Concrete Slope Protection	29300	CM	XCBGB	6.00	2151255	17188454	16115000	0	35,454,709
USR AA <02200 1016 > Mountain Soil	68800	CM	XCARD	500.00	32,260	542,262	3,921,600	0	4,496,123
USR AA <02200 1018 > Early Warning System	1.00	LS	N/A	0.00	975,300	975,300	0	0	1,950,600
LEVEES					5867228	81101674	20669100	0	107,638,002
CHANNELS					5867228	81101674	20669100	0	107,638,002
ALTERNATIVE LEVEES					5867228	81101674	20669100	0	107,638,002
01_AH.02. ALT SEDIMENT RETENTION STRUCTURE									
01_AH.02.AA. CHANNELS									
01_AH.02.AA_001. LEVEES									
USR AA <02200 1001 > EXCAVATION LOADER & TRUCKS	15800	BCM	XCARA	206.00	27,174	675,211	0	0	702,384
USR AA <02200 1016 > Mountain Soil	31500	CM	XCARD	500.00	14,770	248,274	1,795,500	0	2,058,545
USR AA <03000 2002 > Concrete Slope Protection	18800	CM	XCBGB	6.00	1380327	11028769	10340000	0	22,749,097
USR AA <02200 1018 > Early Warning System	1.00	LS	N/A	0.00	975,300	975,300	0	0	1,950,600
LEVEES					2397571	12927554	12135500	0	27,460,626
CHANNELS					2397571	12927554	12135500	0	27,460,626
01_AH.02.AB. STRUCTURES									
01_AH.02.AB_001. RIVER KILOMETER 13									
USR AA <02200 1001 > EXCAVATION LOADER & TRUCKS	4644000	BCM	XCARA	206.00	7987004	198460664	0	0	206,447,667
USR AA <02200 1004 > EMBANKMENT RANDOM COMPACTION	9375000	BCM	XCARD	500.00	4395938	73891125	0	0	78,287,063
USR AA <02200 1008 > ROCK Blasting	5200000	BCM	XCARI	25.00	15146560	374816000	218400000	0	608,362,560
USR AA <02200 1015 > Load & Haul Rock	8580000	CM	XCARL	70.00	37265391	892567594	0	0	929,832,986
USR AA <03000 1001 > RCC CONCRETE 4 SACK MIX	249000	CM	XCBGC	50.00	3043776	53886887	125994000	0	182,924,663
USR AA <03000 1002 > RCC CONCRETE 7 SACK MIX	150000	CM	XCBGC	50.00	1833600	32461980	132825000	0	167,120,580
USR AA <03000 2001 > Walls Including Forms, Rebar & Ect.	53300	CM	XCBGB	4.00	5870062	46901601	69290000	0	122,061,664
USR AA <02200 1011 > Fine Gravel Fill	443000	BCM	XCARF	10.00	7750285	2,553,452	186060000	0	196,363,737
USR AA <02200 1012 > Coarse Gravel Fill	304000	BCM	XCARF	10.00	5318480	1,752,256	97280000	0	104,350,736
USR AA <02200 1009 > FILTER FABRIC	521000	SM	XCARH	100.00	532,618	0	3,256,250	0	3,788,868
USR AA <02200 1014 > 1 Meter Conc Pipe	2100.00	LM	XCARK	2.00	136,028	833,364	2,293,200	0	3,262,592

01_AH. BUCAO BASIN	QUANTY	UOM	CREW ID	OUTPUT	LABOR	EQUIPMNT	MATERIAL	OTHER	TOTAL COST
USR AA <02200 1017 > Rock Fill	4956000	CM	XCARM	125.00	3694797	24607135	0	0	28,301,932
USR AA <02200 9999 > Care & Diversion Of Water	1.00	LS	N/A	0.0026400000		26400000	0	0	52,800,000
RIVER KILOMETER 13				**********	1729132058	835398450		0	2,683,905,047
STRUCTURES				**********	1729132058	835398450		0	2,683,905,047
ALT SEDIMENT RETENTION STRUCTURE				**********	1742059612	847533950		0	2,711,365,672
BUCAO BASIN				**********	1823161286	868203050		0	2,819,003,674

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01_AI. DELTA	QUANTY	UOM	CREW ID	OUTPUT	LABOR	EQUIPMNT	MATERIAL	OTHER	TOTAL COST
01_AI. DELTA									
01_AI.01. EXCAVATION									
01_AI.01.AA. CHANNELS									
01_AI.01.AA_001. PASIG DREDGING									
USR AA <02100 1001 > Dredging 16" Dredge 800 M	13200000	CM	XASPA	204.0035368882	480096941		0	0	515,465,824
USR AA <02200 1018 > Early Warning System	1.00	LS	N/A	0.00 975,300	975,300		0	0	1,950,600
PASIG DREDGING					36344182	481072241	0	0	517,416,424
CHANNELS					36344182	481072241	0	0	517,416,424
EXCAVATION					36344182	481072241	0	0	517,416,424
DELTA					36344182	481072241	0	0	517,416,424
MOUNT PINATUBO RECOVERY					**********	**********	4367114702	0	17,382,878,086
MOUNT PINATUBO, PHILIPPINES					**********	**********	4367114702	0	17,382,878,086