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COOPERATIVE AGREEMENT ON HUMAN SETTLEMENTS AND NATURAL RESOURCE SYSTEMS ANALYSIS

A PRELIMINARY ASSESSMENT OF PROPOSED FLOOD CONTROL MECHANISMS IN THE BICOL RIVER BASIN

by

Mac McKee

Settlement and Resource Systems Analysis
Clark University/Institute for Development Anthropology
Cooperative Agreement (USAID)

Clark University
International Development Program
950 Main Street
Worcester, MA 01610

Institute for Development Anthropology
Suite 302, P.O. Box 818
99 Collier Street
Binghamton, NY 13902

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for

The Bicol River Basin Development Project

and

The United States Agency for International Development

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PREFACE

In July, 1982, a request was made by the USAID Mission in Manilla for technical assistance on the design of appropriate flood control interventions for the Bicol River Basin. This basin covers much of the southern portion of the island of Luzon, and consists of the major drainage systems of the Sipocot River (draining southward toward San Miguel Bay) and the more spatially extensive basin of the Bicol River, collection system draining northward, also emptying into San Miguel Bay.

This report presents an initial hydrological assessment of the flood control problem and sets out several alternatives and recommendations for consideration and potential implementation.

1.0 INTRODUCTION

1.1 Statement Of Problem

1.1.1. Flooding and flood control in the Bicol River Basin

The Bicol River Basin, located on the southern portion of the island of Luzon has a relatively broad and flat flood plain (see Figure 1.1). The basin is ringed by hills and mountains which produce high runoff rates. This geomorphology and landform configurations, along with natural flood plain vegetation, produces slow conveyance rates of flood waters from the basin. The flood situation is aggravated and enlarged by substantial natural storage occurring in the upper portion of the basin, and by the tidal effects of typhoons on San Miguel Bay, which in turn influence flows as far upstream as Naga City. Damage from floods, then, results largely from flood stages exaggerated by the impact of tides, and from prolonged inundation due to the slow rate of drainage of the large quantities of water stored in the upper basin lakes. The major features of the flood control alternatives that have proposed to address these problems fall into two categories: (1) structures to contain local flooding and prevent flood waters from reaching protected areas, and (2) structures which will increase the rate at which water can be conveyed from the basin, thereby reducing flood peaks and decreasing the duration of inundation.

1.1.2. Previous flood control studies

Several analyses of the Bicol River Basin flood control problem

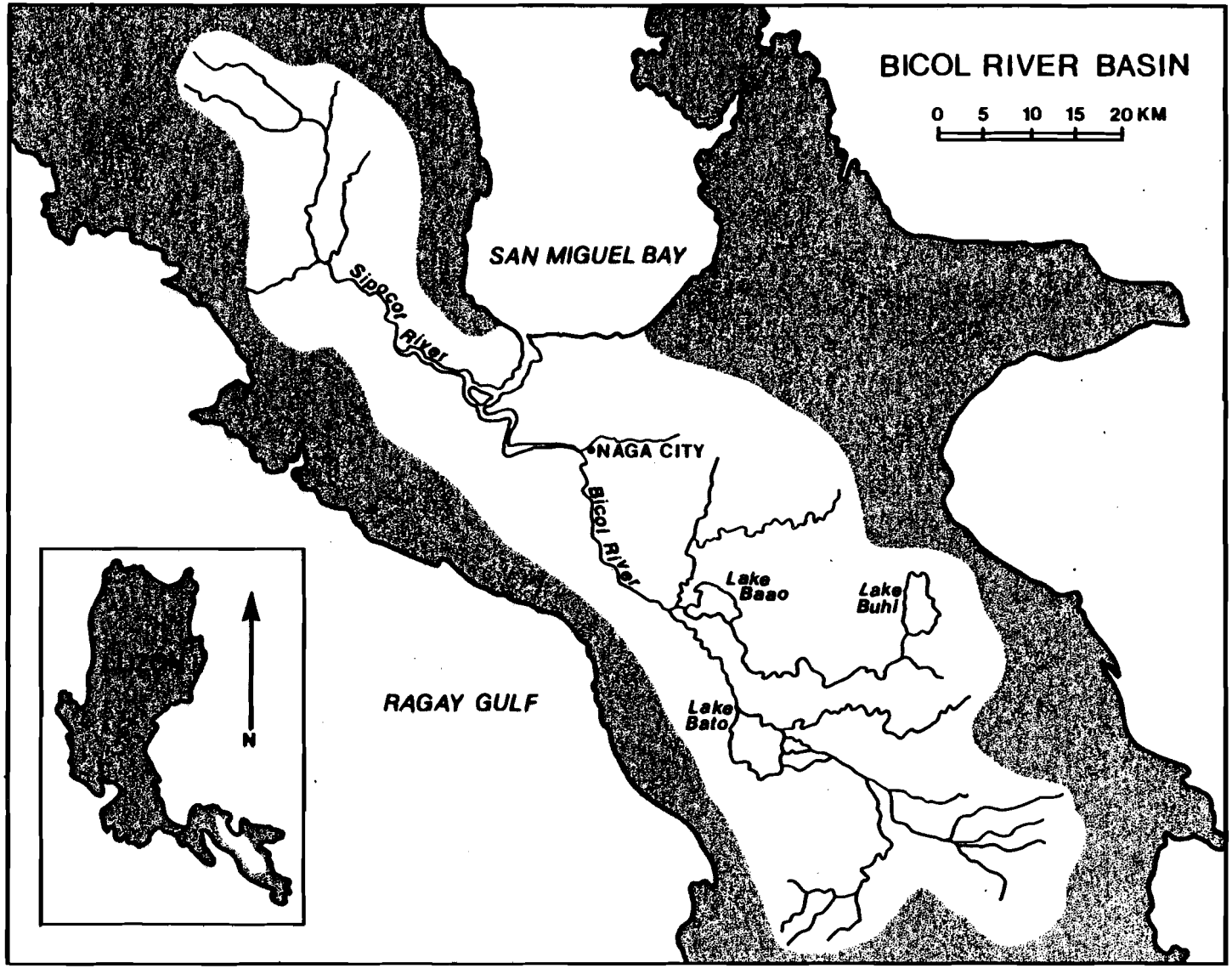

Figure 1.1 Bicol River Basin

have been done previously. In particular, sophisticated computer simulation studies have been conducted by the Asian Institute of Technology (AIT) and also by TAMS Engineering. However, many of these studies focused on individual IDAs in the basin, and none considered the entire range of flood control options presently of interest to the Bicol River Basin Development Project (BRBDP). Many of the studies used different techniques or analysis of flood hydraulics and reported results in ways that make it difficult to assemble a consistent, comprehensive, basin-wide estimate of the optimal flood control design.

Moreover, another problem with previous studies is inconsistency in the use of techniques to estimate flood damages. If flood damages were quantitatively estimated at all, frequently it was done under differing assumptions relative to: return periods, the manner in which primary and secondary damages should be estimated, whether commerical damages should be included, and so on.

This lack of a basin-wide hydraulic analysis of the set of flood control structures currently under consideration as well as the absence of a consistent application of flood damage estimation throughout the basin led to a general underestimation of the value of flood control.

1.2 Types of Information Required

Flood control benefits of an alternative can be computed through construction of a flood damage-frequency curve (see Figure 1.2). This curve simply is a plot of flood damage, measured in monetary terms, against the return frequency of flood events. The expected flood control benefit of a project is the area under the present damage-frequency curve minus the area under the damage-frequency curve

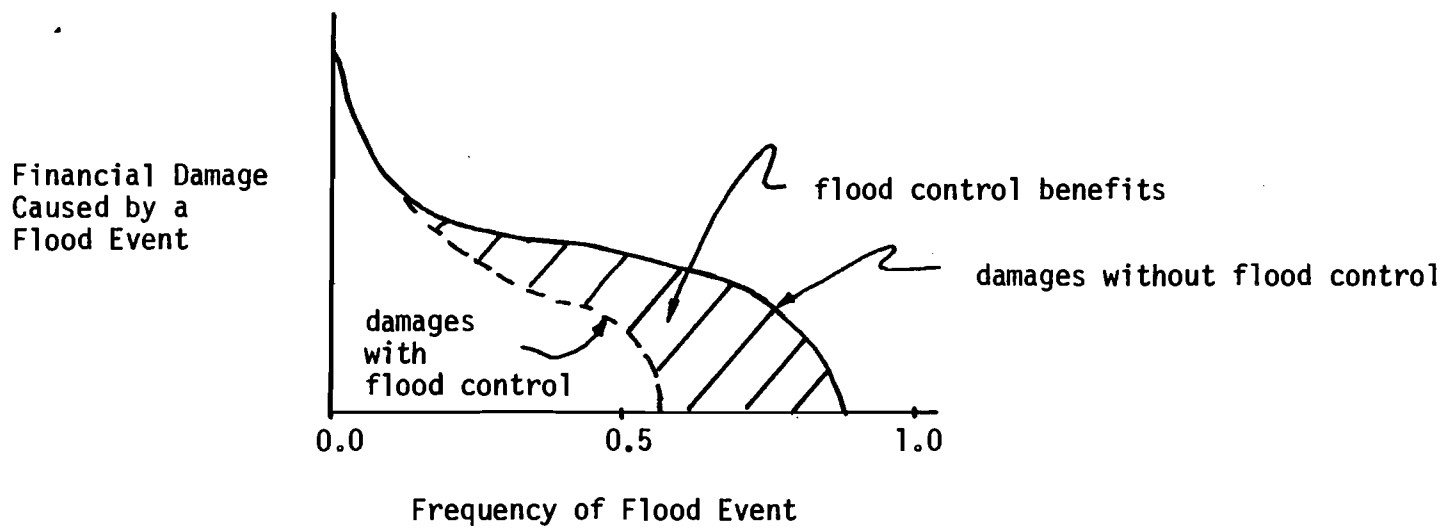

Figure 1.2: Estimation of Flood Control Benefits

that would result if the flood control project were built. The estimation of flood control benefits, then, requires information about the damage that would result from a variety of flood events of different return periods. In particular, data describing the location, areal extent, depth, and duration of inundation of different frequency flood events (both with and without the proposed flood control projects) must be obtained.

1.3 Purpose of the Study

The purpose of this study is to make a first attempt to provide the inundation data necessary for a consistent, basin-wide estimation of flood control benefits. This has been done through the construction and use of a computer model of flood flows in the Bicol River Basin. The study is considered to be a "first attempt" because it uses data that are readily available, though perhaps not of the quality desirable for a finished flood control study. As such, the study serves to identify gaps in the existing database which must be filled before a polished study can be completed.

Discussion to follow covers the development of the computer model of flood flows (Chapter 2), the procedures and data used to calibrate the computer model (Chapter 3), the flood control alternatives presently of interest to BRBDP, and the computer results of the analysis of these alternatives (Chapter 4). Conclusions and recommendations generated from this study regarding areas where further data collection and research are needed are given in the final section (Chapter 5).

2.0 THE FLOOD CONTROL SIMULATION MODEL

2.1 Background

The flood control model developed for this project is based on the procedures outlined in the Asian Institute of Technology flood control study by Ackerman et al. (1975). This model represents the equations of unsteady open channel flow in finite difference form. Because of this, considerable effort is required in the schematization of the river system into a series of nodes and branches. The following sections discuss this schematization, as well as the derivation of the finite difference equations and techniques used in modeling lateral inflows and tides (which represent the set of boundary conditions for the model).

2.2 River Schematization

The unsteady open channel flow equations used by the model are known as the equations of continuity and momentum. In finite difference form, the continuity equation is used to obtain an expression for the elevation of the water surface at selected points along the river as a function of time. These points are called nodes. The finite difference formulation of the momentum equation is used to estimate the discharges along the reaches of the river between consecutive nodes, again as a function of time. These reaches are called branches. For the Bicol and Sipocot river system, the model accommodated a total of 140 nodes and

141 branches.[1]

In using this schematized system of nodes and branches, the computational procedures in the model make the following assumptions:

1. The water surface is horizontal at each node;
2. Only one value of discharge exists along the entire length of a single branch at any instant;
3. Water levels vary linearly along a branch between adjacent nodes; and
4. The storage in each node is considered equal to the volume of the water in all branches from that node to a distance half way to each adjacent node.

Therefore, nodes represent storage in the river system, and branches represent the hydraulic conveyance characteristics of the river system.

2.3 Derivation of the Hydraulic Model

2.3.1 General equations of unsteady open channel flow

Two partial differential equations can be used to describe the flow of water in open channels in flood conditions. The first is the continuity equation:

$$\frac{\partial Q}{\partial x} + T\frac{\partial H}{\partial t} - q = 0 \qquad [1]$$

1 An appendix listing the FORTRAN code for the model is on file with the Clark University Publications under Regional Projects--Natural Resources Management.

where Q is the discharge, x is the distance along the channel, T is the topwidth of the cross section of flow, H is the water surface elevation above some datum, t is time, and q is the lateral inflow per unit length of channel.

The second equation is the momentum equation:

$$2M\frac{Q}{A}\frac{\partial Q}{\partial x} - M\frac{Q^2}{A^2}\frac{\partial A}{\partial x} + \frac{Q^2}{A}\frac{\partial M}{\partial x} + \frac{\partial Q}{\partial t} + gA\frac{\partial H}{\partial x} + \frac{gA|Q|Q}{K^2} = 0 \qquad [2]$$

where M is a dimensionless momentum correction factor, A is the cross sectional area of flow, g is the acceleration of gravity, K is a conveyance correction factor having units of meters to the sixth power per second squared, and Q, x, H, and t are as previously defined.

2.3.2 Finite difference formulation of open channel flow equations

To predict changes in discharge and water elevation, Equations 1 and 2 must be solved simultaneously. Since this is not directly possible in their partial differential form, the standard solution approach is to rewrite them in finite difference form, and then solve algebraically. To do this, the river first is schematized into a series of nodes and branches (as discussed above). The finite difference form of the continuity equation is written for each node in the system, and the finite difference form of the momentum equation is written for each branch. Water elevations at each node and discharge along each branch can then be obtained by solving the resulting set of equations simultaneously.

Equation 1 can be approximated in finite difference form as

$$\frac{\partial H}{\partial t} \approx \frac{\Delta H}{\Delta t} = \frac{\Sigma Q_{in} - \Sigma Q_{out}}{F} \qquad [3]$$

where ΣQ_{in} is the total inflow into a given river node, ΣQ_{out} is the total outflow from the node, H is the water elevation at the node, t is time, and F is the water surface area at the node.

Equation 2 can be written in finite difference form as:

$$\frac{\partial Q}{\partial t} \approx \frac{\Delta Q}{\Delta t} = \frac{MQT}{A}\left[\left.\frac{\Sigma Q_{in} - \Sigma Q_{out}}{F}\right|_u + \left.\frac{\Sigma Q_{in} - \Sigma Q_{out}}{F}\right|_d\right]$$
$$+ \frac{MQ^2}{A^2 L}(A_d - A_u) - \frac{Q^2}{AL}(M_d - M_u)$$
$$- \frac{gA}{L}(H_d - H_u) - \frac{gA|Q|Q}{K^2} \qquad [4]$$

where Q is discharge along the branch, T is the average branch topwidth, A is the average branch cross sectional area, M is the average momentum correction factor for the branch, L is the length of the branch, K is the conveyance correction factor computed at the center of the branch, u and d are subscripts indicating quantities taken at the branch upstream and downstream nodes, respectively, and H, t, ΣQ_{in}, ΣQ_{out}, and F are defined above.

In general, if J_t is the dependent variable of a finite difference equation at time t, and if there exists an expression f_t for the partial time derivative of J_t, or

$$f_t = \frac{\partial J_t}{\partial t} \qquad [5]$$

then $J_{t+\Delta t}$ can be found by solving the following:

$$J_{t+\Delta t} = J_t + \Delta t\left\{(1 - \theta)f_t + \theta f_{t+\Delta t}\right\} \qquad [6]$$

where θ is selected such that $0<\theta<1$. If θ is set equal to 1, the solution approach is said to be wholly implicit; if θ is set equal to zero, the solution approach is called explicit; and if θ is set equal to one-half, the so-called Crank-Nicholsen approach results. The report by Ackerman et al (1975) recommends a value of 0.55 for θ. By substituting Equations 3 and 4 into Equation 6, the following node and branch equations result:

$$\frac{F}{\Delta t}\Delta H - \theta\Sigma\Delta Q_{in} + \theta\Sigma\Delta Q_{out} = \Sigma Q_{in} - \Sigma Q_{out} \qquad [7]$$

$$\begin{aligned}
&\frac{\theta\Delta t\, gA}{L}\Delta H_d - \frac{\theta\Delta t\, gA}{L}\Delta H_u + \left\{1 - \theta\Delta t\left[\frac{MT}{A}\right.\right. \\
&\left(\left.\frac{\Sigma Q_{in} - \Sigma Q_{out}}{F}\right|_u + \left.\frac{\Sigma Q_{in} - \Sigma Q_{out}}{F}\right|_d\right) + \frac{2MQ}{A^2L}(A_d - A_u) - \frac{2Q}{AL}(M_d - M_u) \\
&\left.\left. - \frac{gA\,Q}{K^2}\right]\right\}\Delta Q - \theta\Delta t\,\frac{MQT}{A}\left(\left.\frac{\Sigma\Delta Q_{in} - \Sigma\Delta Q_{out}}{F}\right|_u\right) \\
&- \theta\Delta t\,\frac{MQT}{A}\left(\left.\frac{\Sigma\Delta Q_{in} - \Sigma\Delta Q_{out}}{F}\right|_d\right) = \Delta t\left\{\frac{MQT}{A}\left[\left.\frac{\Sigma Q_{in} - \Sigma Q_{out}}{F}\right|_u\right.\right. \\
&\left. + \left.\frac{\Sigma Q_{in} - \Sigma Q_{out}}{F}\right|_d\right] + \frac{MQ^2}{A^2L}(A_d - A_u) - \frac{Q^2}{AL}(M_d - M_u) - \frac{gA}{L}(H_d - H_u) \\
&\left. - \frac{gA|Q|Q}{K^2}\right\}
\end{aligned} \qquad [8]$$

where ΔH and ΔQ refer to the unknown changes in water surface elevation and discharge in a time period of length Δt, and other terms are as previously defined, with values taken at time t.

The new values of H and Q at time $t+\Delta t$ are found by:

$$H_{t+\Delta t} = H_t + \Delta H \qquad [9]$$

$$Q_{t+\Delta t} = Q_t + \Delta Q \qquad [10]$$

In general, for $\theta>0$, Equations 7 and 8 must theoretically be stable. However, $\theta>0$ leads to a set of implicit equations which must be solved simultaneously. Typically, the resulting matrix of linear coefficients will not be banded, so solution algorithms require considerable amounts of computer memory. Initially, it was intended that an implicit solution approach for the Bicol River model be used, but memory limitations on the computer selected for model implementation made this very difficult. Therefore, an explicit solution was adopted, i.e., θ was selected to be zero. In this case, equations 7 and 8 become explicit and the solution approach requires comparatively little computer memory. However, as for all explicit approaches, the allowable size of the time increment, dt, is greatly reduced due to stability constraints imposed by the Courant condition.

Setting θ equal to zero, Equations 7 and 8 become

$$\frac{\Delta H}{\Delta t} = \frac{\Sigma Q_{in} - \Sigma Q_{out}}{F} \qquad [11]$$

and

$$\frac{\Delta Q}{\Delta t} = \frac{MQT}{A}\left[\left.\frac{\Sigma Q_{in} - \Sigma Q_{out}}{F}\right|_u + \left.\frac{\Sigma Q_{in} - \Sigma Q_{out}}{F}\right|_d\right] + \frac{MQ^2}{A^2L}(A_d - A_u) - \frac{Q^2}{AL}(M_d - M_u) - \frac{gA}{L}(H_d - H_u) - \frac{gA|Q|Q}{K^2} \qquad [12]$$

In order to ensure stability in the solution of these equations, extensive time averaging was used on all quantities. Also, the last term in Equation 12 was written as:

$$\frac{gA|Q|\ (Q + \Delta Q)}{K^2}$$

The resulting equations become:

$$\Delta H = \Delta t\ \frac{\Sigma\dot{Q}_{in} - \Sigma\dot{Q}_{out}}{F} \qquad [13]$$

and

$$\Delta Q = \left\{ \frac{\dot{M}\dot{Q}\dot{T}}{\dot{A}} \left[\left. \frac{\Sigma\dot{Q}_{in} - \Sigma\dot{Q}_{out}}{\dot{F}} \right|_u + \left. \frac{\Sigma\dot{Q}_{in} - \Sigma\dot{Q}_{out}}{\dot{F}} \right|_d \right] + \frac{\dot{M}\dot{Q}^2}{\dot{A}^2 L} (\dot{A}_d - \dot{A}_u) - \frac{\dot{Q}^2}{\dot{A}L} (\dot{M}_d - \dot{M}_u) - \frac{g\dot{A}}{L} (\dot{H}_d - \dot{H}_u) - \frac{g\dot{A}|\dot{Q}|\dot{Q}}{\dot{K}^2} \right\} \left(\frac{1}{\Delta t} + \frac{g\dot{A}|\dot{Q}|}{\dot{K}^2} \right)^{-1} \qquad [14]$$

where any quantity $\dot{z}$ is taken as $(1-a)z_t + az_{t+\Delta t}$, where $0<a<1$.

2.3.3. Model representation of channel geometric and hydraulic properites

As can be seen in the finite difference equations of open channel flow, the simulation of water elevations and discharges requires substantial knowledge of such channel geometric properties as topwidths, cross sectional areas, and surface areas. It also requires the estimation of the momentum and conveyance correction factors. The following sections describe how the model treats these channel characteristics.

2.3.3.1 **Channel geometry.** The channel geometry is specified to the model in the manner prescribed by Ackerman et al (1975). The approach consists of characterizing the main channel and berm geometry at both the upsteam and downstream nodes of each branch. An "average geometry" is also specified for each branch and is assumed by the model to be located at the branch center.

The cross sectional geometry of the main channel is expressed in terms of specific elevations, topwidths, and areas. At the upstream end, downstream end, and center of each branch, the model is given information about the main channel cross sectional geometry in terms of four specific elevations, four specific topwidths, and two specific areas. The topwidth and elevation relationship is assumed by the model to be linear, but at different slopes for different elevation intervals. Figure 2.1 illustrates the topwidth-elevation relationship for main channels. Given the water surface elevation, E, at a node or branch center, the model uses the following equations to estimate the corresponding main channel topwidth and cross sectional area:

$$T = \begin{cases} T_2 - \dfrac{T_2 - T_1}{E_2 - E_1}(E_2 - E) & \text{for } E \leq E_2 \\[2ex] T_2 + \dfrac{T_3 - T_2}{E_3 - E_2}(E - E_2) & \text{for } E_2 < E \leq E_3 \\[2ex] T_4 - \dfrac{T_4 - T_3}{E_4 - E_3}(E_4 - E) & \text{for } E_3 < E \leq E_4 \\[2ex] T_4 & \text{for } E > E_4 \end{cases}$$

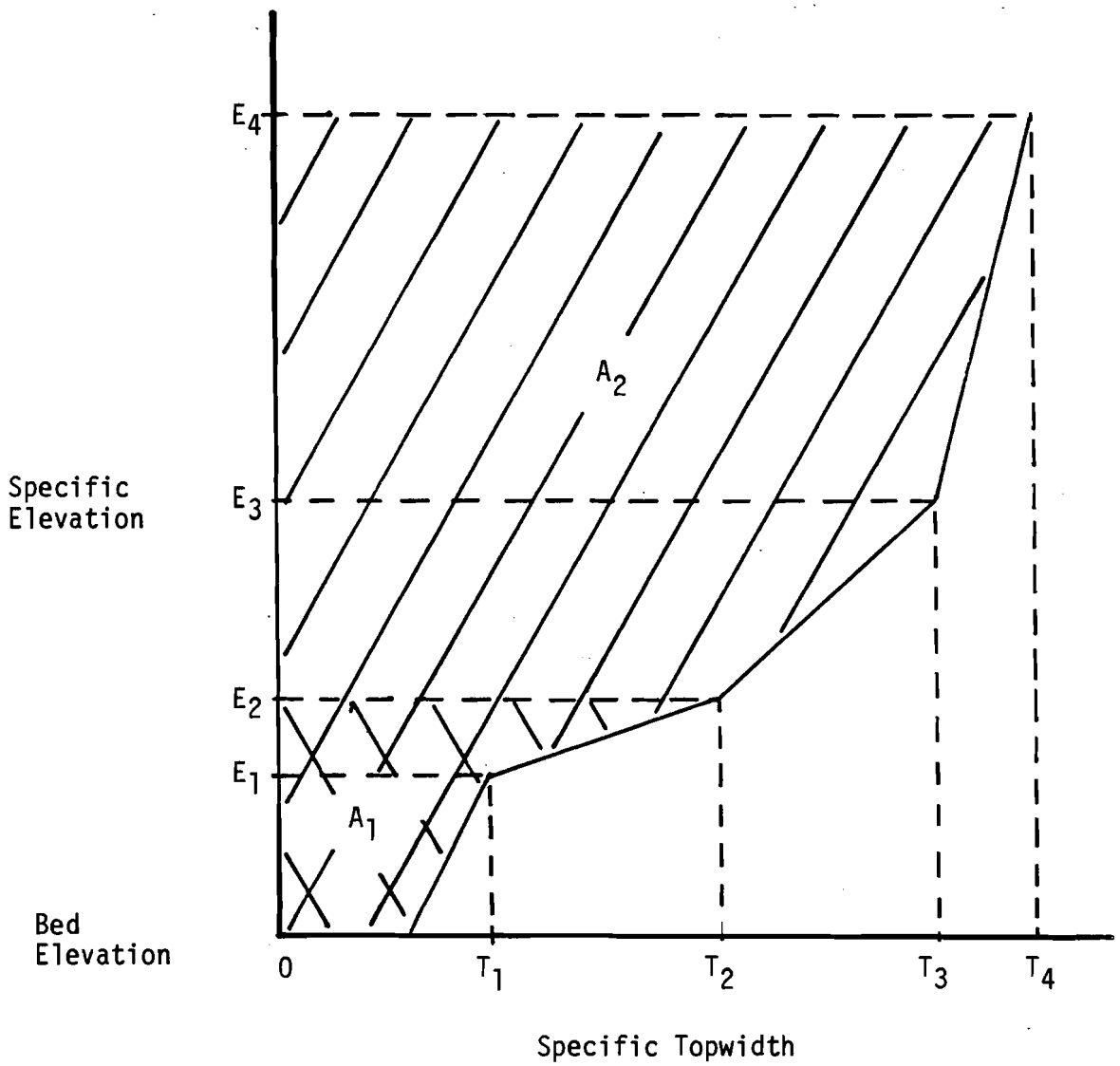

Figure 2.1: Main Channel Topwidth, Elevation, and Cross Sectional Area Relationships

where E is the water elevation, E_i is the ith specific elevation, $i=1, 2, 3, 4$, T_i is the ith specific topwidth, and T is the topwidth at elevation E.

The main channel cross sectional area is estimated from:

$$A = \begin{cases} A_1 + (E - E_2)\ \dfrac{T + T_2}{2} & \text{for } E < E_3 \\ A_2 + (E - E_4)\ \dfrac{T + T_4}{2} & \text{for } E \geq E_3 \end{cases}$$

where E, T, E, and T_i are as defined above, A_j is the specific cross sectional area (at elevations E_2 and E_4), $j=1,2$, and A is the cross sectional area at elevation E.

The cross sectional area and topwidth of the berms are computed in a fashion somewhat similar to that of the main channel. For the berms, three specific elevations and topwidths are specified as input data for the upstream and downstream nodes and the center of each branch, for both left and right sides. This is illustrated in Figure 2.2. The equations used for estimating berm topwidth and cross sectional area are:

$$T = \begin{cases} 0 & \text{for } E \leq E_b \\ T_1\ \dfrac{E - E_b}{E_1 - E_b} & \text{for } E_b < E \leq E_1 \\ T_1 + (T_2 - T_1)\ \dfrac{E - E_1}{E_2 - E_1} & \text{for } E_1 < E \leq E_2 \\ T_2 + (T_3 - T_2)\ \dfrac{E - E_2}{E_3 - E_2} & \text{for } E_2 < E \leq E_3 \\ T_3 & \text{for } E > E_3 \end{cases}$$

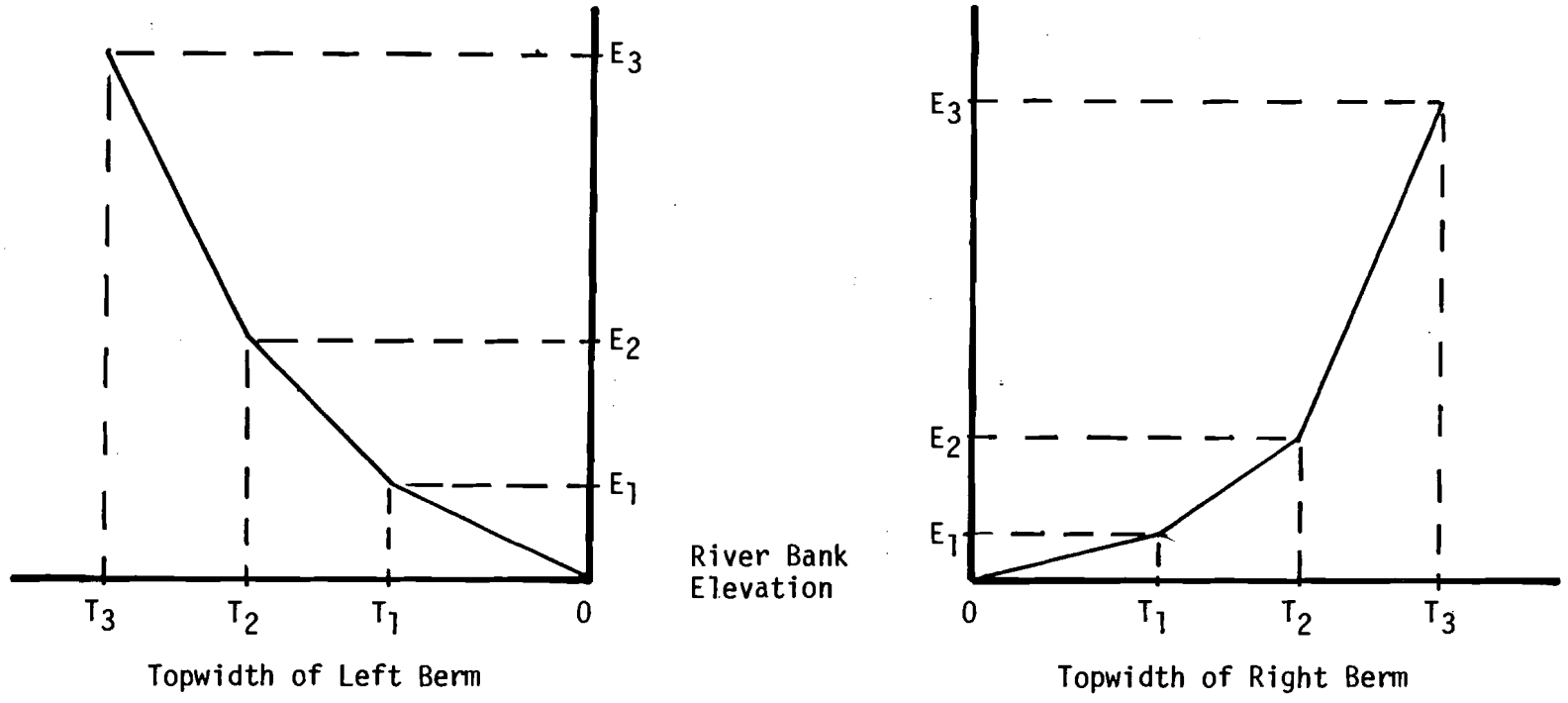

Figure 2.2: Left and Right Berm Topwidth and Elevation Relationships

where E is the water elevation, E_b is the elevation of the bank, E_i is the ith specific elevation, $i=1,2,3$, T_i is the ith specific topwidth, and T is the estimated topwidth.

The berm cross sectional area is computed by:

$$A = \begin{cases} 0 \quad \text{for } E \leq E_b \\ \frac{T}{2}(E - E_b) \quad \text{for } E_b < E \leq E_1 \\ \frac{T_1}{2}(E_1 - E_b) + \frac{T + T_1}{2}(E - E_1) \quad \text{for } E_1 < E \leq E_2 \\ \frac{T_1}{2}(E_1 - E_b) + \frac{T_2 + T_1}{2}(E_2 - E_1) \\ \qquad + \frac{T + T_3}{2}(E - E_2) \quad \text{for } E_2 < E \leq E_3 \\ \frac{T_1}{2}(E_1 - E_b) + \frac{T_2 + T_1}{2}(E_2 - E_1) \\ \qquad + \frac{T_3 + T_2}{2}(E_3 - E_2) \\ \qquad + T_3(E - E_3) \quad \text{for } E > E_3 \end{cases}$$

where E, E_b, E_i, T_i, and T are defined above, and A is the estimated berm cross sectional area.

These equations are due to Ackerman et al. (1975). Note that the specific elevations on the left and right berms need not be the same, nor do specific elevations need to be taken at constant intervals.

2.3.3.2. **Schematization of berm geometry and roughness coefficients.** Equations 3 and 4 and the subsequent derivations are restricted to the condition that the flow is in one direction, or that distances along the main channel and berms are equal. Accordingly, additional schematization of berm geometry is required. The berm characteristics which must be further adjusted are the topwidth, cross sectional area, and roughness coefficient. Derivations of the schematization procedures are given in Ackerman et al (1975). If Δx_s is the main channel flow distance along a branch and Δx_a is the berm distance along the branch, then the expressions for the necessary quantities are:

$$T_s = T_a \frac{\Delta x_a}{\Delta x_s}$$

$$A_s = A_a \frac{\Delta x_a}{\Delta x_s}$$

$$n_s = n_a \left(\frac{\Delta x_a}{\Delta x_s}\right)^{3/2}$$

where T_a, A_a, and n_a are the actual berm topwidth, cross sectional area, and roughness coefficient, respectively, and T_s, A_s, and n_s are their respective schematized counterparts.

2.3.3.3 **Estimation of Channel Hydraulic Properties.** Equation 8 requires the estimation of two hydraulic properties, the momentum correction factor, M, and the conveyance correction factor, K.

The momentum correction factor is computed at the upstream and downstream nodes of each branch and then averted. It is found from:

$$M = \frac{\sum_{i=1}^{3} \frac{Q_i^2}{A_i}}{\frac{Q^2}{A}} \qquad [15]$$

where i is an index on sections of flow, that is, $i=1$ for the main channel, $i=2$ for the left berm, and $i=3$ for the right berm, Q_i is the flow obtained from the Manning equation for the ith section of flow; A_i is the schematized cross sectional area of flow of the ith section; Q is the sum of the Q_i, and A is the sum of the A_i.

The conveyance correction factor is computed at the center of each branch as:

$$K^2 = \left(\sum_{i=1}^{3} \frac{1}{n_i} \; A_i \; R_i^{2/3} \right)^2 \qquad [16]$$

where i and A_i are as defined above, n_i is the schematized Manning friction term, and R_i is the hydraulic radius of the ith flow section. Ackerman et al. (1975) recommend approximating the hydraulic radius as:

$$R_i = \frac{A_i}{T_i} \qquad [17]$$

2.4 Meteorologic And Hydrologic Inputs

The finite difference equations of the model require as boundary conditions information describing lateral inflows into the various nodes of the model as well as tidal elevations at San Miguel Bay and Pasacao. Estimation of lateral inflows is done following the procedures recommended by Ackerman et al. (1975). Tidal data at various times are

specified to the model, and tidal elevations are then found through trigonometric interpolation.

2.4.1 Modeling of lateral inflows

Following the procedures of Ackerman et al. (1975), the estimation of lateral inflows was done by first dividing the total drainage area into 183 watersheds. Some of these watersheds are gaged and their discharges can be directly specified to the model as input data. The remainder of the watersheds are ungaged, and their contribution to lateral inflows must be estimated using synthetic techniques.

2.4.1.1 **Gaged watersheds.** In the earlier work by AIT, Ackerman et al. (1975) report a total of 11 gaged watersheds, whose discharges were input directly to the model as a portion of the boundary conditions during the calibration procedure. However, in a re-examination of the discharge records, it appears that three of these watersheds suffer at high flow conditions from tidal, backwater, or overbank effects. Accordingly, this study used the gage record of only eight of these 11 watersheds.

2.4.1.2 **Ungaged watersheds.** To synthetically estimate the discharges of the ungaged watersheds, the model uses information about the watershed location and area as well as an estimated runoff coefficient and a distribution graph based on the consecutive two-day rainfall. In addition, baseflow is computed as a function of watershed area.

Total direct runoff from an ungaged watershed is computed as:

$$Q_T = R_2 \, R_f \, A \qquad [18]$$

where Q_T is the total watershed discharge in m^3, R_2 is the consecutive two-day rainfall in mm, R_f is the runoff coefficient for the watershed, and A is the watershed area in km^2 .

This runoff volume is then distributed through time by using the distribution graphs computed by Ackerman et al. (1975). Note that one distribution graph applies for watersheds of less than 50 km, and another for larger watersheds. Also it was noted in the work of Ackerman et al. (1975) that the runoff coefficients are specified as a function of watershed type and location. Watersheds located within the flood plain are assigned a relatively high runoff coefficient. Other ungaged watersheds are assigned a runoff coefficient on the basis of their location relative to the particular sub-region of the Bicol in which they are located. Three sub-regions are specified: (1) watersheds in the drainage area of the Sipocot River, (2) watersheds in the drainage of the Bicol River from approximately the Bicol-Sipocot confluence to Lake Bato, and (3) watersheds in the drainage area of the upper Bicol (see Figure 2.3). Any consecutive two-day rainfall for these three regions is based on precipitation records taken at Sabang, Pili, and Guinobatan, respectively.

Baseflow for ungaged watersheds is added to the distributed runoff. It is computed from:

$$Q_B = 0.063 \, A^{0.87} \qquad [19]$$

where Q_B is baseflow in cms, and A is the watershed area in km^2 . This equation was empirically derived by Ackerman et al. (1975).

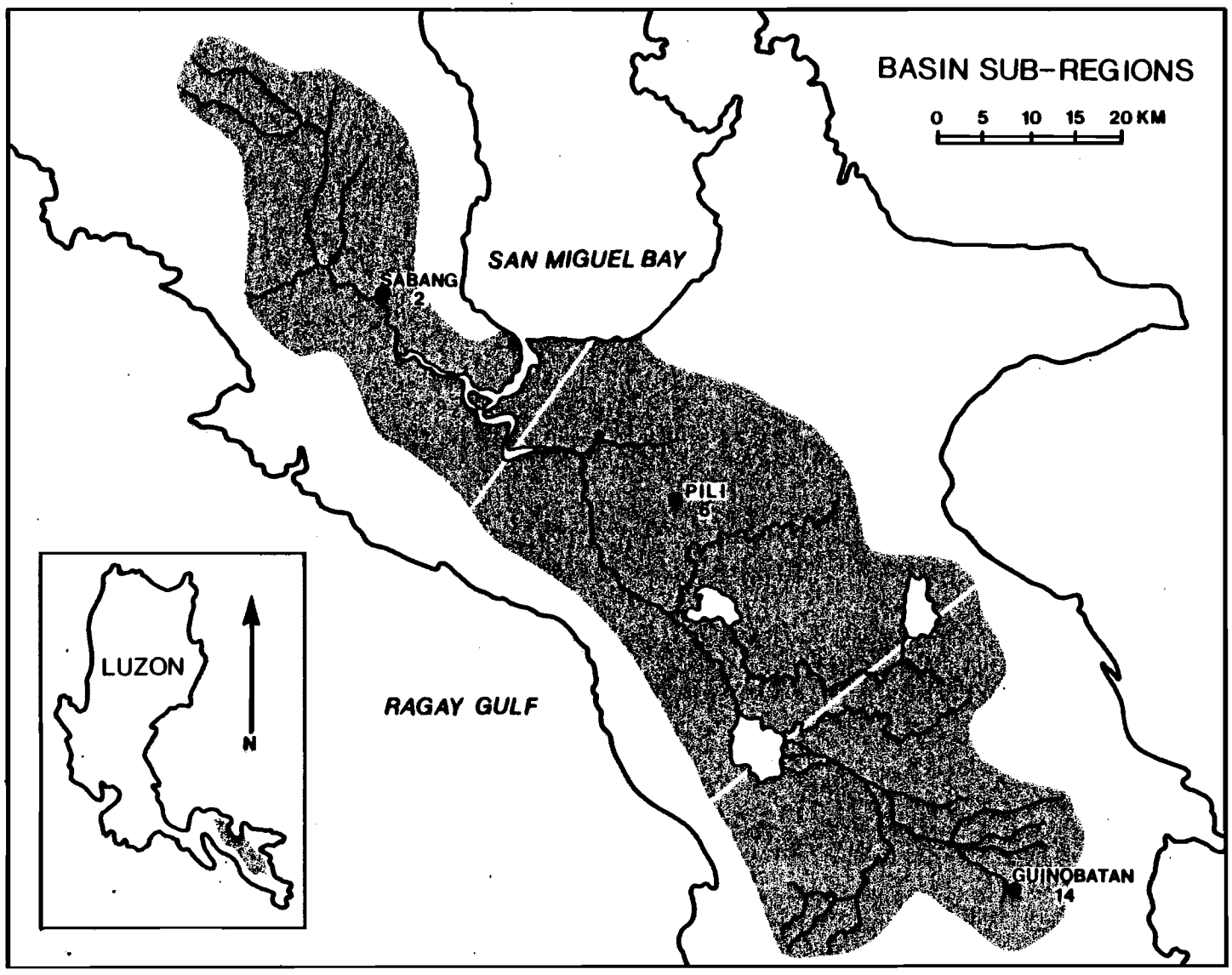

Figure 2.3 Basin Subregions Used in Estimating Runoff from Ungaged Watersheds

2.4.2. Modeling of tidal elevations

Tidal records at San Miguel Bay and Pasacao provide the remaining portion of the necessary boundary conditions. For the calibration process, tidal records at San Miguel Bay for the time period surrounding Typhoon Ruping (September, 1982) were used. Subsequent maximum and minimum tidal elevations and their times of occurence are given as input data, and the model uses a trigonometric interpolation procedure to obtain the tidal elevation for a particular point in time. For production runs, design tidal maximum and minimum elevations at San Miguel Bay and Pasacao were obtained using the procedure given by Ackerman et al. (1975).

3.0 MODEL CALIBRATION

3.1 Purpose of Calibration

Many of the coefficients in the hydrologic and hydraulic sections of the model are not known with any degree of accuracy. Moreover, many of these coefficients vary with time, and their values are likely to change as flood conditions change. The purpose of model calibration is to obtain an estimate of the values of these coefficients that will make the model perform in a similar manner to that observed in the real system's performance. These estimates are then used as constants in the model production runs. The coefficients which are to be estimated in the calibration process are the energy loss coefficients (Manning's n) for the main channel and left and right berms of each branch, and the pre and post-storm runoff coefficients.

3.2 Procedures Used in Calibration

Calibration is basically an iterative process. The procedure involves four steps: (1) picking values for the unknown coefficients, (2) running the model with these values using known boundary conditions, (3) comparing the model results with observed flood conditions, and (4) if observed and computed water elevations do <u>not</u> agree, selecting a new set of trial values and repeating the process. In principle, this searching process can be programmed into the computer and thereby

automated.

3.3 Data Used in Calibration

As stated above, model calibration requires that the model be run under known initial and boundary conditions for a previous flood event. The event selected in this case was Typhoon Ruping which occurred in September, 1982. This event was preceded by several days of near steady-state conditions. It also produced very high peak water elevations throughout much of the Bicol River Basin as well as significant downstream tidal effects, and from this standpoint, it could be considered as a good opportunity for calibrating a model postulated to examine severe flood flows.

3.4 Results of Calibration

There were four gaging stations from which reliable water surface elevations were available for the period of time surrounding Typhoon Ruping (September 5 to 13, 1982). These stations are located at Naga, Baliwag, Ombao, and Sto. Domingo. In addition, an estimated water elevation record for Lake Bato was synthetically constructed from water elevation relationships with other stations that have been observed to occur during other typhoons. This provided a total of five water elevation records to be used as targets in the calibration process.

There were two objectives of calibration. The first was to get the model to match the pre-typhoon conditions in terms of observed water elevations at the above five locations. This was done by adjusting main

channel Manning n's and pre-typhoon runoff coefficients. The second objective was to get the model to match the observed peak water elevations. After the model was made to simulate pre-storm conditions, a six-day period surrounding Typhoon Ruping was simulated, and predicted peak water elevations from the simulation were computed, with alternates sequentially selected to force the output to match as closely as possible the observed water elevation peaks. This was done by adjusting berm roughness coefficients and post-typhoon runoff coefficients. Due to lack of time, no attempt was made to fit the entire water elevation time curve.

Figure 3.1 presents the observed versus predicted water elevations for the final calibration constants. Note that pre-storm water elevations match closely, and peak elevations show reasonable fit. However, for the nodes further downstream, the model produces a much less accurate fit for the entire stage hydrograph than for the upstream nodes. This may be caused by errors in the channel geometry data and in estimates of Manning n's.

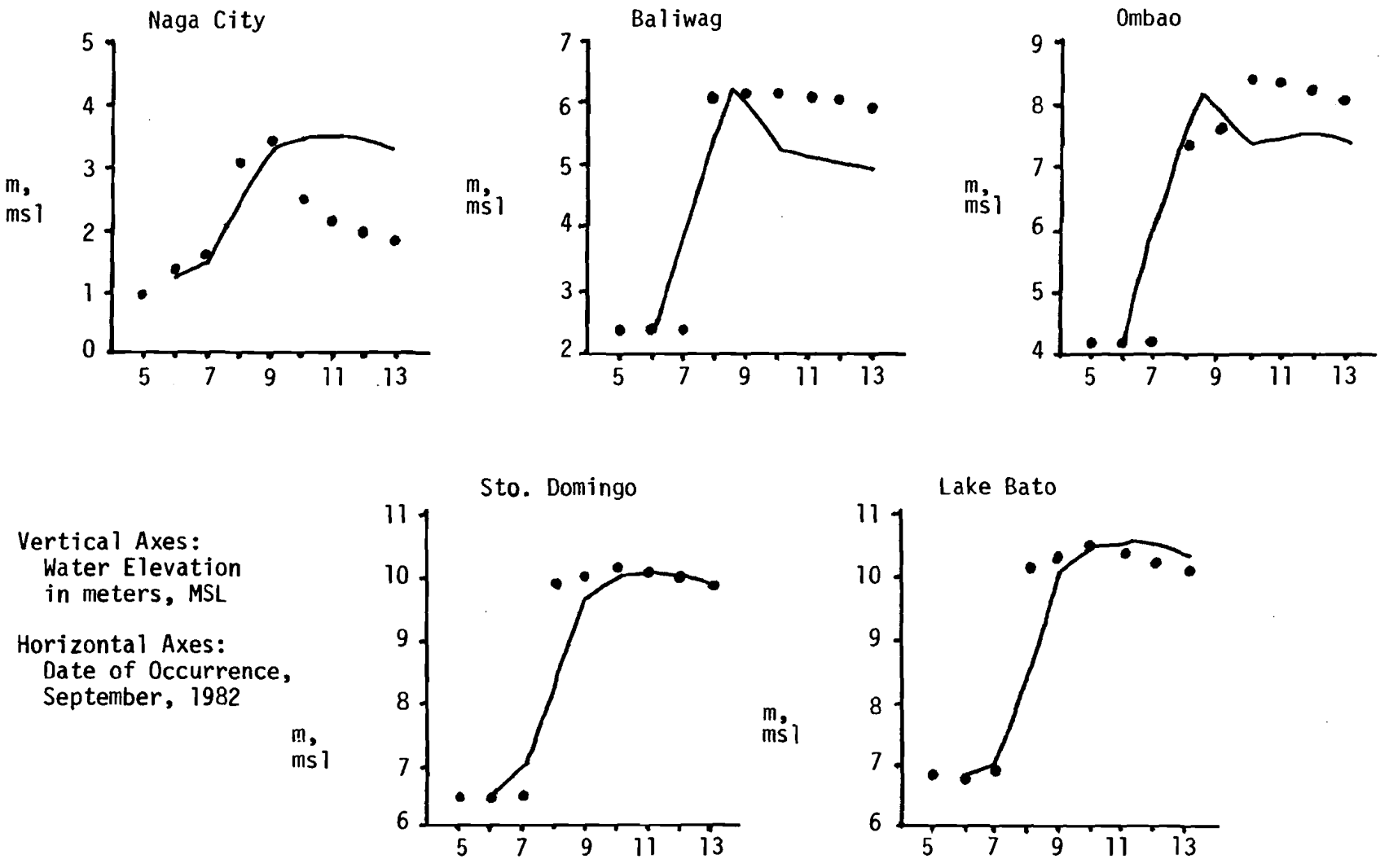

Figure 3.1: Results of Calibration: Estimated Water Elevations (solid line) and Observed Water Elevations (individual points) at various locations for September 5-13, 1982

4.0 PROPOSED ALTERNATIVES AND PRODUCTION RUN RESULTS

4.1 Overview

As stated in Section 1.1, the two most important problems contributing to flood conditions in the Bicol River Basin are the downstream tidal effects caused by high water elevations at San Miguel Bay and the poor conveyance characteristics of the upper basin. Serious flood control proposals must deal with both of these problems, and the four alternatives simulated in this study represent a variety of approaches.

4.2 Four Alternatives Proposed

The major features of the four alternatives now will be summarized. Alternative 1 proposes: (1) some channel improvements and extensive provision for levees from the Libmanan-Sipocot and Naga-Calabunga IDA's near the mouth up to Lakes Baao and Bato to control downstream tidal and highwater effects, (2) use of Lake Bato as a flood reservoir through the construction of a regulation structure at its outlet, (3) use of Lake Baao as a flood reservoir to impound flood waters from both the Pawili and Iriga Rivers through the provision of a ring levee around the lake, and control structures for diverting flows from these rivers into or past the lake, (4) improvement of portions of the Barit, San Francisco, and Waras Rivers, and (5) construction of a diversion channel

from Ombao to Pasacao to divert the entire upper basin flood flow of the Bicol, and thereby increasing water conveyance out of that portion of the basin.

Alternative 2 is similar in all respects to Alternative 1 except that no ring levee or control works are considered for Lake Baao, and up to 200 cms would be allowed to flow into the lower Bicol River past the Ombao-Pasacao diversion point.

Alternative 3 consists of an extensive program of channel improvements on the Bicol River from Lake Bato to San Miguel Bay. No levees would be provided on the river, but considerable efforts would be taken to widen, straighten, and deepen the river channel. Lakes Bato and Baao would be used for flood control with control structures as proposed in Alternative 1.

Alternative 4 is similar to Alternative 3 in that channel improvements but no levees would be considered in the lower basin. In addition, a Ombao-Pasacao diversion channel would be proposed to divert all of the upper basin flood flows.

Table 4.1 summarized the flood control mechanisms proposed for these four alternatives.

4.3 Simulation Results

4.3.1 Procedures

4.3.1.1 **Model input for production runs.** To construct a curve such as that given in Figure 1.2 as a target for a proposed alternative, the condition which produces the alternative under flood conditions must be

Table 4.1: Summary of Flood Control Alternatives

	Alternative 1	Alternative 2	Alternative 3	Alternative 4
1. Levees and channel improvements along lower Bicol	X	X		
2. Extensive channel improvements but no levees along lower Bicol			X	X
3. Control structure on Lake Bato	X	X	X	X
4. Ring levee and control structures on Lake Baao	X		X	X
5. Improvements along sections of Barit, San Francisco, Waras Rivers	X	X	X	X
6. Diversion channel at Ombao to divert entire flow of upper Bicol to Pasacao	X			X
7. Diversion channel at Ombao to divert most of the flow of upper Bicol to Pasacao		X		

simulated for a variety of flood return periods. The return periods selected for the production runs are the 5-, 10-, 25-, and 50-year events, which are represented in the model by the appropriate two-day rainfalls at each of the three rainfall gaging stations discussed in Section 2.4.1. Estimates of these two-day rainfall quantities were obtained from the frequency analyses reported by Ackerman et al. (1975). The model uses these two-day rainfall figures to estimate the lateral inflow boundary conditions.

The tides at San Miguel Bay and Pasacao also represent boundary conditions that must be provided for as input to the model. This is done by specifying consecutive low and high tides and their times of occurrence for the entire duration of the simulation. For the production runs, design tides were computed using the Fourier coefficients reported by Ackerman et al. (1975) and by superimposing on these a hypothetical tidal surge during the second day of the typhoon.

4.3.2.1 Model predictions. After the flood plain and channel geometry of the configuration suggested by an alternative are specified, and after subroutines describing proposed control structures are made available, the model uses the tidal and two-day rainfall data to simulate flood flows through the conditions of the proposed alternative. The model gives a periodic report of predicted water elevations at nodes, and discharges through branches. For the production runs, these reports were generated at six-hour intervals. Upon completion of a simulation, the model identified maximum simulated water elevations at each node, and yields the estimated maximum number of hectares flooded at each node. The model also can be used to predict the duration of inundation

if the calibration process has been done to address the entire stage hydrograph.

4.3.2. Outcome of simulation runs

The model was used to make 20 production runs, one run for each return period for each of the proposed alternatives. An additional set of runs using the present river conditions as a fifth alternative was also done. The results of these 20 runs are summarized in Table 4.2, showing the maximum water elevations predicted for various locations on the Bicol and Sipocot Rivers. A complete listing of maximum water elevations for each alternative was also developed.[1] Note that while water elevations are reported to the nearest centimeter, data and model accuracy is certainly no better than the nearest 0.1 meters.

One cautionary word is in order regarding the simulation results for Alternatives 3 and 4. These alternatives were formulated and specified to the model under the assumption that all flood flows in the lower Bicol could be contained within the banks of the improved channel. As far as the mathematical model is concerned, this in effect turns the river into a very narrow trough (as compared to the present flood plain), with infinitely high walls. In the reaches from Naga to Ombao, the peak lateral inflows occur simultaneously with the maximum tidal elevation at San Miguel Bay, thus causing unrealistically high water elevations. Though these elevations are temporary and subside after a

1 This material is available as an Appendix to this report (though unpublished) on file with the Clark University publications under Regional Projects--Natural Resources Management).

Table 4.2: Summary of Maximum Projected Flood Levels in the Bicol River Basin

Location	Node No.	Flood Levels (meters, MSL) by Return Period																			
		Existing Condition				Alternative 1				Alternative 2				Alternative 3				Alternative 4			
		5-yr	10-yr	25-yr	50-yr	5-yr	10-yr	25-yr	50-yr	5-yr	10-yr	25-yr	50-yr	5-yr	10-yr	25-yr	50-yr	5-yr	10-yr	25-yr	50-yr
1. San Miguel Bay	1	3.61	3.61	3.61	3.61	3.61	3.61	3.61	3.61	3.61	3.61	3.61	3.61	3.61	3.61	3.61	3.61	3.61	3.61	3.61	3.61
2. Downstream C.O.C. 1	127	2.76	2.80	2.84	2.86	2.96	2.96	3.03	2.98	2.96	2.97	2.98	2.99	3.11	3.08	3.09	3.10	3.06	3.07	3.08	3.12
3. Downstream C.O.C. 2	5	2.32	2.52	2.73	2.84	2.42	2.65	2.97	3.12	2.76	3.01	3.31	3.51	3.82	4.26	4.80	5.18	3.23	3.54	3.93	4.20
4. Upstream C.O.C. 2 and Downstream C.O.C. 3	9	2.54	2.73	2.94	3.09	3.04	3.34	3.73	3.98	3.66	4.01	4.44	4.72	4.19	4.71	5.42	5.95	3.49	3.84	4.25	4.57
5. Camaligan	11	2.98	3.17	3.40	3.56	3.89	4.26	4.74	5.06	4.77	5.20	5.72	6.06	5.64	6.44	7.11	7.69	4.61	5.12	5.71	6.07
6. Mabulo Bridge, Naga	12	3.19	3.39	3.64	3.81	4.06	4.44	4.92	5.26	4.98	5.41	5.94	6.29	5.94	6.78	7.50	8.10	4.81	5.35	5.96	6.34
7. Milaor	13	3.72	3.97	4.28	4.49	4.33	4.74	5.25	5.59	5.32	5.77	6.31	6.67	6.91	8.11	8.72	9.57	5.45	6.05	6.72	7.14
8. Upstream C.O.C. 3	16	4.32	4.59	4.92	5.15	4.80	5.23	5.78	6.15	5.88	6.35	6.92	7.29	8.51	10.50	11.59	13.77	5.96	6.79	8.05	8.78
9. Baliwag Nuevo	18	5.65	5.98	6.38	6.66	4.84	5.26	5.79	6.15	5.95	6.40	6.93	7.29	7.85	9.49	10.10	11.28	5.99	6.62	7.41	7.94
10. San Ramon, Bula, and Proposed inlet OPDC	20	6.75	7.09	7.49	7.76	6.93	7.43	7.98	8.38	6.08	6.50	7.00	7.34	7.91	9.44	10.16	11.25	6.00	6.63	7.41	7.94
11. Ombao	21	7.50	7.82	8.21	8.48	7.07	7.58	8.15	8.70	7.21	7.57	8.01	8.32	7.97	9.27	10.22	11.16	6.15	6.79	7.57	8.12
12. Confluence of Bicol and Pawili	22	7.97	8.29	8.67	8.93	7.20	7.17	8.48	9.06	7.83	8.16	8.57	8.85	8.02	9.22	10.26	11.12	6.27	6.91	7.69	8.24
13. Bicol River @ km 80	25	8.09	8.38	8.81	9.11	7.29	7.81	8.53	9.11	7.98	8.30	8.73	8.96	8.05	9.29	10.29	11.14	6.35	6.99	7.77	8.31
14. Sto. Domingo Bridge	27	9.05	9.41	9.85	10.15	8.02	8.52	9.23	9.76	9.02	9.33	9.86	10.18	8.15	9.49	10.36	11.26	6.65	7.26	7.99	8.52
15. Bicol River @ km 92, proposed L. Bato outlt	28	9.22	9.59	10.03	10.34	8.24	8.79	9.58	10.16	9.02	9.39	9.85	10.18	9.96	10.51	11.19	11.66	9.96	10.51	11.19	11.65
16. Lake Bato	29	9.49	9.87	10.33	10.64	9.30	9.77	10.37	10.78	9.30	9.70	10.19	10.53	9.98	10.53	11.20	11.64	9.98	10.53	11.19	11.66
17. Junction, Barit, San Francisco, Waras R.	58	9.59	9.84	10.15	10.34	14.42	16.17	18.43	20.07	9.59	10.30	11.29	11.71	14.42	16.17	18.43	20.07	14.42	16.17	18.43	20.07
18. Inlet of Pawili R. to Lake Bato	59	*	*	*	*	11.92	12.35	12.85	13.18	*	*	*	*	11.92	12.35	12.85	13.18	11.92	12.35	12.85	13.18
19. Lake Baao	60	7.88	8.23	8.67	8.97	11.95	12.39	12.92	13.26	7.77	8.11	8.58	8.76	11.94	12.39	12.92	13.26	11.94	12.39	12.92	13.26
20. Barit River @ Nabua	48	11.93	12.34	12.85	13.20	14.93	16.53	18.68	20.27	16.49	17.86	19.47	22.07	14.93	16.53	18.68	20.26	14.93	16.53	18.68	20.26
21. Outlet of proposed cutoff channel for Libmanan River	101	2.20	2.23	2.29	2.31	2.50	2.54	2.61	2.60	2.50	2.54	2.58	2.60	2.60	2.61	2.92	3.15	2.56	2.60	2.64	2.70
22. Junction of Libmanan and Bicol Rivers	102	2.28	2.42	2.58	2.68	2.38	2.42	2.71	2.85	2.38	2.60	2.88	3.07	2.99	3.34	3.77	4.05	2.68	2.96	3.29	3.52
23. Inlet of proposed cut off channel, Libmanan River	104	2.64	2.83	3.05	3.19	2.64	2.88	3.19	3.34	2.72	2.98	3.29	3.49	3.12	3.48	3.93	4.22	2.91	3.21	3.57	3.60
24. Libmanan River @ town	105	3.37	3.67	4.01	4.20	3.41	3.73	4.11	4.32	3.44	3.77	4.15	4.38	3.60	3.98	4.45	4.75	3.49	3.85	4.26	4.51
25. Sipocot/Libmanan River @ km 25	107	5.49	5.97	6.49	6.80	5.66	6.16	6.71	7.04	5.66	6.16	6.71	7.04	5.55	6.04	6.58	6.92	5.52	6.01	6.55	6.39
26. Sipocot River @ Sipocot Bridge	109	7.91	8.62	9.35	9.91	7.97	8.66	9.45	10.04	7.97	8.66	9.45	10.04	7.95	8.64	9.43	10.00	7.93	8.64	9.42	9.99
27. Sipocot River @ Sabang	110	7.95	9.34	9.56	10.66	8.02	9.83	11.78	12.40	8.02	9.84	11.78	12.35	8.00	9.75	11.71	12.34	7.99	9.71	11.68	12.32
28. Inlet, C.O.C. 3	90	4.09	4.35	4.67	4.89	4.80	5.23	5.80	6.27	5.93	6.46	7.09	7.51	8.12	9.78	9.75	10.21	5.96	6.76	7.72	8.24
29. Proposed OPDC, @ San Fernando	94	*	*	*	*	5.86	6.34	6.88	7.24	5.14	5.56	6.06	6.40	*	*	*	*	5.15	5.74	6.46	6.95
30. @ Danao, Pamplona	96	*	*	*	*	4.73	5.19	5.70	6.03	4.24	4.52	4.98	5.30	*	*	*	*	4.12	4.67	5.33	5.79
31. @ Pasacao Beach	98	*	*	*	*	4.10	4.09	4.09	4.09	4.09	4.09	4.09	4.09	*	*	*	*	4.07	4.09	4.09	4.09
32. Outlet of OPDC @ Ragay Gulf	99	*	*	*	*	4.07	4.07	4.07	4.07	4.07	4.07	4.07	4.07	*	*	*	*	4.07	4.07	4.07	4.07

*not included in the alternative

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day or so, they reflect an inconsistency between the assumptions used to formulate the boundary conditions and the river geometry. These unrealistic maximum water elevations for Alternatives 3 and 4 indicate that, at least for the reaches of the river between Naga and Ombao, insufficient storage is provided in the channel to accommodate the peak lateral inflows. This results because the downstream tidal levels are too high to allow sufficient conveyance from the reaches at a time when inflows are at a maximum.

5.0 CONCLUSIONS AND RECOMMENDATIONS

5.1 Assessment Of The Alternatives

5.1.1 Nature of the Results

In general, the results of the various simulation runs are believed to be conservative in that the predicted water elevations probably are marginally higher than the elevations that actually will be experienced for any given alternative. A variety of reasons explain this. For example, the tidal surges used in the design tides at San Miguel Bay and Pasacao probably represent extreme conditions, both in terms of magnitude and positioning in time relative to that of the normal high tide. Also, it is unlikely that the entire basin simultaneously and uniformly will receive a 25- or 50-year rainfall from any given typhoon. The production runs, then, were made under fairly conservative conditions. However, only very limited sensitivity analyses were conducted, and then only during the calibration process. The degree to which the model might be sensitive to errors in channel geometry and calibration parameters has not been fully examined. Moreover, the extent to which the model results might be sensitive to alternative and less conservative assumptions regarding boundary conditions is unknown.

5.1.2. Rank order of alternatives

In general, from the standpoint of the degree to which flood

inundation is controlled, the best alternative proposed in the analysis is Alternative 1, followed in order by Alternatives 2, 4, and 3. As noted in the previous chapter, the lower basin results for Alternatives 3 and 4 are indicative of an inconsistency between the assumptions under which channel geometry and boundary conditions were defined. Accordingly, these results indicate a need to redefine the proposed alternatives to provide more channel storage volume. Consequently, the actual water elevations produced for these two alternatives should not be used in any economic analyses.

5.1.3. Control structures

All the model production runs indicate that careful examination should be given to the control structures proposed to regulate outflows from Lake Bato, and flood inflows into Lake Baao from the Iriga River. The same can be said for the diversion structure at Ombao, if an alternative is selected involving the Ombao-Pasacao diversion channel, but diverting only a portion of the Bicol flow into the channel.

The structure for the Iriga River-Lake Baao control appears to be underdesigned. The model results indicate that the structure never reduces flood flows into the lake to zero, but at the same time produces very high upstream water levels and fairly extensive localized flooding.

The Lake Bato control structure might also be underdesigned. The weir-type structure was designed to produce a certain discharge at a given lake elevation. However, this design was offered under an assumption of free overfall conditions. The model indicates that free overfall conditions rarely exist at the point where the structure is to be located, and, therefore, the design discharge is not reached at the

prescribed lake elevation. The model also indicates that operation of the structure during flood flows in order to limit the maximum discharge through the structure might be complicated. This again is due to the downstream backwater effects that drown the free overfall. This same problem also will be encountered at the Ombao-Pasacao diversion, if it is desired to permit some maximum flow (up to 200 cms is considered in Alternative 2) to pass from Ombao to the lower Bicol. Gaging and controlling flow through these structures will be considerably complicated by backwater effects.

5.2 Recommendations

5.2.1 Recommendations for additional work

Several recommendations regarding the data used by the model and therefore potentially affecting the model results are made.

5.2.1.1 **Channel geometry data.** Much of the channel geometry data used in the model runs were assembled quickly and as a result were not analyzed adequately. In particular, the analyses used to estimate branch center geometry need to be re-calculated. Also, geometry data for Lakes Baao and Bato also should be checked to determine that they are consistent with present and proposed flood plain conditions.

5.2.1.2 **Frequency analyses, distribution curves, tidal data.** The frequency analyses used to obtain rainfall data now are available. These should be incorporated into a new frequency analysis. Similarly, more data now are available to estimate distribution curves for

computing lateral inflows. This work, also originally performed in 1974, should be updated.

In addition, the analysis used to produce the Fourier coefficients for computing tidal elevations at San Miguel Bay and Pasacao should be updated. Pasacao, in particular, should be re-examined and a better fitting set of coefficients estimated.

5.2.1.3 **Calibration and sensitivity analysis.** After an updated analysis of geometric, hydrologic, and tidal data is peformed, the model should be re-calibrated. This new calibration should include a much more extensive sensitivity analysis and should be designed to track the entire stage hydrograph. The model would then be more reliable for making inundation duration estimates.

5.2.2 Recommendations for model improvements.

5.2.2.1 **Model re-programming.** If portions of the model were rewritten, an estimated 10 to 15 percent increase in efficiency could be achieved. This is not significant when considering the cost of computer time in the U.S.; howevr, computer costs in the Phillippines run 20 to 100 times higher. These facts notwithstanding, increased model efficiency would be worth the cost of reprogramming.

5.2.2.2 **Problem generator.** An even more useful addition to the model is a problem generator. This program would be used to compile and catalogue data descriptive of channel geometry. It could be used to accurately construct a data set describing alternatives in a manner compatible with input formats expected by the model. This would reduce

the manual data handling requirements, and errors, and would increase the generation of data sets for production runs.

5.2.3. Recommendations for BRBDP.

A considerable effort has gone into the construction of the flood simulation model. In many ways, this is a duplication of past efforts by AIT and TAMS. To avoid future replication, BRBDP should upgrade its engineering staff to include personnel trained to operate the model. This will require that one or more of the BRBDP flood control engineers be trained and acquire experience in the subjects of unsteady open channel flow, finite difference solutions of partial differential equations, and computer programming. It would also be helpful if the BRBDP staff had increased access to computational facilities.