COOPERATIVE AGREEMENT ON HUMAN SETTLEMENTS AND NATURAL RESOURCE SYSTEMS ANALYSIS

FLOOD FLOW MODELLING IN THE BICOL RIVER BASIN

by

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1.0 INTRODUCTION

1.1 STATEMENT OF PROBLEM

The Bicol River Basin located on the southern end of the island of Luzon, Philippines, annually experiences severe floods that result from the typhoons (see Map 1). Development of the region's considerable agricultural resources will necessitate the control of these floods and mitigation of flood damages. Several structural alternatives have been proposed to do this. Evaluation of the economic costs and benefits of these alternatives requires knowledge of how they will behave hydraulically under flood conditions. In particular, it is necessary to have estimates of peak water surface elevations and duration of inundation in order to compute economic benefits and costs of flood control alternatives. In addition, estimates of these quantities must be provided for a range of flood return periods. To obtain this information a digital computer model has been constructed and used to predict the hydraulic effects of flood control alternatives.

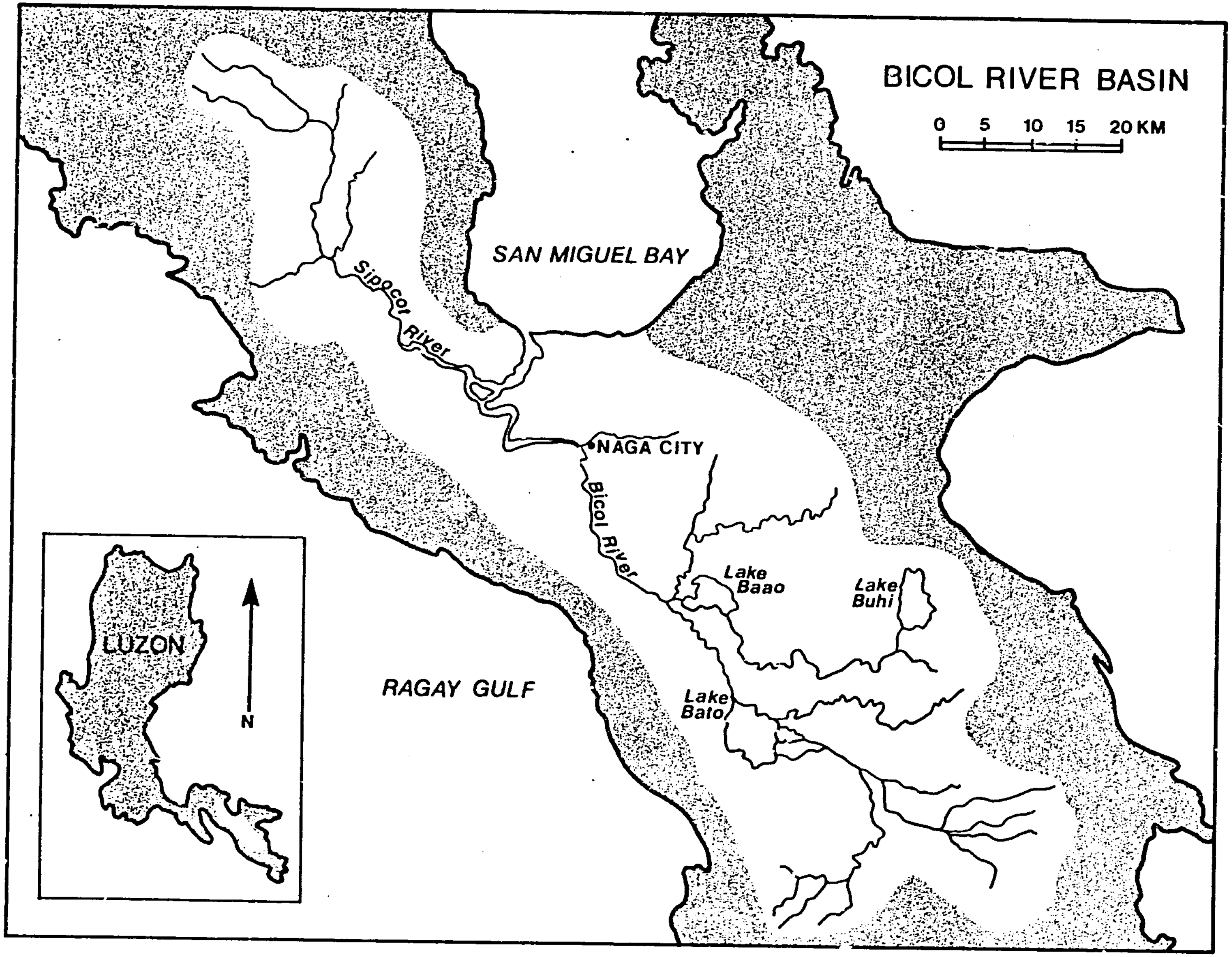
1.2 PURPOSE

The purpose of this paper is to provide an overview of the model, a description of the results of calibration, and a brief summary of the results of the production runs that have been conducted to date.

2.0 OVERVIEW OF THE MODEL

2.1 EQUATIONS OF CONTINUITY AND MOTION

Flood waves and waves due to the slow operation of controlling structures are commonly modeled as gradually varied, unsteady flow phenomena. Two equations are used to describe the hydraulics of such phenomena: continuity and motion. These equations are originally proposed


BICOL RIVER BASIN
0 5 10 15 20 KM
SAN MIGUEL BAY
Sipocot River
NAGA CITY
Bicol River
Lake Baao
Lake Buhi
Lake Bato
RAGAY GULF
LUZON
N

Map 1

by Saint-Venant (1848, 1871), and their validity has since been verified by many observations and experiments.

2.1.1 Continuity

Various mathematically equivalent formulations of the continuity equation can be found in the literature of flood routing (see Chow, 1959; Henderson, 1966; Linsley et al., 1982; Viessman et al., 1977; Wylie and Streeter, 1978). The version of the continuity equation used in the model is

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

where Q is the discharge, x is distance along the river, T is the channel topwidth, H is the water surface elevation above some datum, and t is time. This equation assumes no lateral inflow.

2.1.2 Motion

The equation of motion used in the model can be written as

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

where g is the acceleration of gravity, A is the cross-sectional area of flow, K is the conveyance quantity (using the Manning equation to estimate the friction slope), α is a velocity correction factor (sometimes called the Coriolis coefficient), and Q, t, x, and H are as previously defined. The derivation of this equation (or one of its many equivalent expressions) can be found in Chow (1959), Viessman et al., (1977), and Wylie and Streeter (1978), and many others.

2.1.3 Finite Difference Approximations

The partial differential equations of continuity and motion cannot be solved analytically. Instead, finite difference equations must be used to approximate Equations 1 and 2. These finite difference equations can be solved algebraically and programmed on a digital computer. Once this is done, the computer program can be run on a computer to predict the behavior through time of a flood wave.

2.1.3.1 River Schematization

However, before the finite difference equations can be written, it is first necessary to schematize the river into a system of nodes and branches. In schematizing the river, the flow system is treated mathematically as a number of storage tanks connected by a number of channels. This has been described previously by Ackerman et al., (1975). The channels joining the storage tanks are assumed to have geometric and hydraulic properties of the system, both being time dependent. Storage tanks are called <u>nodes</u> and <u>channels</u> that join adjacent storage tanks, called <u>branches</u>. The function of the node is to accommodate all storage or mass transfer effects, while the branches accommodate the hydraulic frictional effect and inertial effects which exist between two nodes.

2.1.3.2 Finite Difference Expresion of Continuity

In finite difference form, the continuity equation is used to obtain a mathematical expression for computing the elevation of the water surface at selected points along the river as a function of time. These points are the nodes. The finite difference expression of the <u>continuity equation</u> is

$$H_t+1 = H_t + \Delta t \frac{Q_{net}}{F} \qquad [3]$$

where H_t is the water surface elevation at a given node at time t, H_{t+1} is the water surface elevation one time increment later, Δt is the size of the increment, Q_{net} is the net inflow rate from all branches entering and leaving the node, and F is the surface area of the water at the node.

2.1.3.3 Finite Difference Expression of Motion

The finite difference formulation of the equation of motion is used to estimate the discharges along the reaches of the river between consecutive nodes, again as a function of time. These reaches are the branches. The finite difference approximation for the equation of motion is

$$Q_{t+1} = Q_t + \left[\frac{1}{\Delta t} + \frac{gA|Q_{t-1}|}{(cK)^2}\right]^{-1} \left[-gA\,\frac{H_d - H_u}{\Delta x} - gA\,\frac{|Q_{t-1}|Q_t}{(cK)^2} - \frac{Q^2}{2A}\,\frac{\alpha_d - \alpha_u}{\Delta x} + \frac{Q\alpha T}{2A}\left\{\left.\frac{Q_{net}}{F}\right|_d + \left.\frac{Q_{net}}{F}\right|_u\right\} + \frac{Q^2\alpha}{A^2}\,\frac{A_d - A_u}{\Delta x}\right] \qquad [4]$$

where Q_t is the discharge in the branch of time t, Q_{t+1} is the discharge one time period later, g is the acceleration of gravity, A is the average branch cross-sectional area, K is the branch conveyance computed from the Manning equation, c is a calibration constant (normally about one), H_d is the water surface elevation at the downstream node, H_u is that of the upstream node, Δx is the length of the branch, Q is the discharge in the branch (averaged over the previous two time periods), α_d is the velocity correction coefficient of the downstream node, α_u is that of the upstream node, T is the branch topwidth average between the upstream and downstream nodes, $\left.\frac{Q_{net}}{F}\right|_d$ is the net inflow divided by the surface area of the downstream node, $\left.\frac{Q_{net}}{F}\right|_u$ is that of the upstream node, α is the velocity correction coefficient averaged between upstream and downstream nodes, A_d is the cross sectional area at the downstream node, A_u is that of the upstream node, and Δt is as previously defined. The quantities K, H_d, H_u, α_d, α_u, Q, Q_{net_d}, F_u, A, Q_{net_u}, F_d, α, A_d and A_u are all averaged over the previous two time periods.

2.2 MODELING OF BOUNDARY CONDITIONS

Two sets of boundary conditions must be provided as input to the model. These conditions are of the Dirchlet type and consist of (1) tidal elevations through time at San Miguel Bay and Pasacao on the Ragay Gulf, and (2) tributary inflows at nodes along the river, also as a function of time.

2.2.1 Tidal Elevations

For calibration runs or simulation of historic typhoon events, observed tidal elevations are given to the model as downstream water surface elevation boundary conditions. For production runs that do not involve observed historical tides, "typical" tides are estimated using the Fourier series coefficients derived by Ackerman et al.,(1975). In either case, consecutive low and high tidal elevations are specified to the model along with the times at which they take place. The model then uses trigonometric interpolation to estimate intermediate tidal boundary conditions.

2.2.2 Tributary Inflows

Tributary inflows are specified to the model in the form of observed or estimated daily inflow rates for each watershed in the river basin. For calibration runs or simulation of historic flood events, the model allows direct input of observed tributary flows for those watersheds that are gaged and whose gage readings are not affected by backwater or overbank effects. The contribution toward tributary inflows of ungaged watersheds is estimated using the distribution graph procedure proposed by Ackerman et al.,(1975). Use of this procedure requires estimation of runoff coefficients for three regions in the basin, as well as for the flood plain, for both pre- and post-typhoon time periods.

2.3 MODELING OF CONTROL STRUCTURES

Three types of structures presently exist or are proposed in the Bicol River Basin for controlling discharge along selected branches. These are flapgates, free-overfall spillways, and spillways whose discharge will be affected by downstream backwater.

2.3.1 Flapgate Discharges

For branches that are controlled by flapgates, discharge is computed by

$$Q = \begin{cases} cA\sqrt{2g\Delta H} & \text{for } \Delta H > 0 \\ 0 & \text{for } \Delta H \leq 0 \end{cases} \qquad [5]$$

where Q is the discharge through the branch, c is a discharge coefficient (usually approximately 0.6), A is the cross sectional area of the flapgate, g is the acceleration of gravity, and H is given by

$$\Delta H = H_u - H_d - h_e \qquad [6]$$

where H_u is the upstream node water surface elevation, H_d is the downstream node water surface elevation, and h_e is an estimate of the head loss through the flapgate. The quantities H_u and H_d are averaged over the two previous time periods.

2.3.2 Free-Overall Spillway Discharges

The equation for computing discharge over a free overfall is taken from Henderson (1966):

$$Q = \begin{cases} \frac{2}{3} c_d L\sqrt{2g} \; h^{3/2} & \text{for } h > 0 \\ 0 & \text{for } h \leq 0 \end{cases} \qquad [7]$$

where Q is the discharge, L is the effective length of the spillway, g is the acceleration of gravity, h is the height of the water surface above the spillway crest, and c_d is a discharge coefficient computed from

$$c_d = 0.611 + 0.08\frac{h}{W} \qquad [8]$$

where W is the height of the spillway crest above the bottom. Equation 8 is from the experimental work of Rehbock (1929).

2.3.3 Spillway Discharges with Downstream Backwater

In situations where downstream backwater drowns the free overfall, discharges through a spillway are computed from

$$Q = \begin{cases} \frac{2}{3}\sqrt{1 - \frac{h_d}{h}}\; c_d\, L\, 2g\, h^{3/2} & \text{for } h > 0 \\ 0 & \text{for } h \leq 0 \end{cases} \qquad [9]$$

where Q, h, c_d, L, and g are defined in Equation 7, and h_d is the height above the spillway crest of the water surface downstream of the control structure. The term $\sqrt{1 - h_d/h}$ is an approximation of the experimental results reported by Vennard and Street (1976), Vennard and Weston (1943), and Villemonte (1947).

3.0 MODEL CALIBRATION

3.1 PURPOSE OF CALIBRATION

The values of many of the coefficients in the hydrologic and hydraulic sections of the model are not accurately known. 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 so that the model will 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, the effective topwidths of the berms, and the pre- and post-storm runoff coefficients.

3.2 PROCEDURES USED IN CALIBRATION

Calibration is basically an iterative process. The procedure is to (1) pick values for the unknown coefficients, (2) run the model with these values using known boundary conditions, (3) compare the model results with observed flood conditions, and (4) if observed and computed water elevations do not agree, select a new set of trial values and repeat the process. In principle, this searching process can be programmed into a computer and thereby automated. For a model with a great many unknowns, however, computer costs could be prohibitive.

3.3 DATA USED IN CALIBRATION

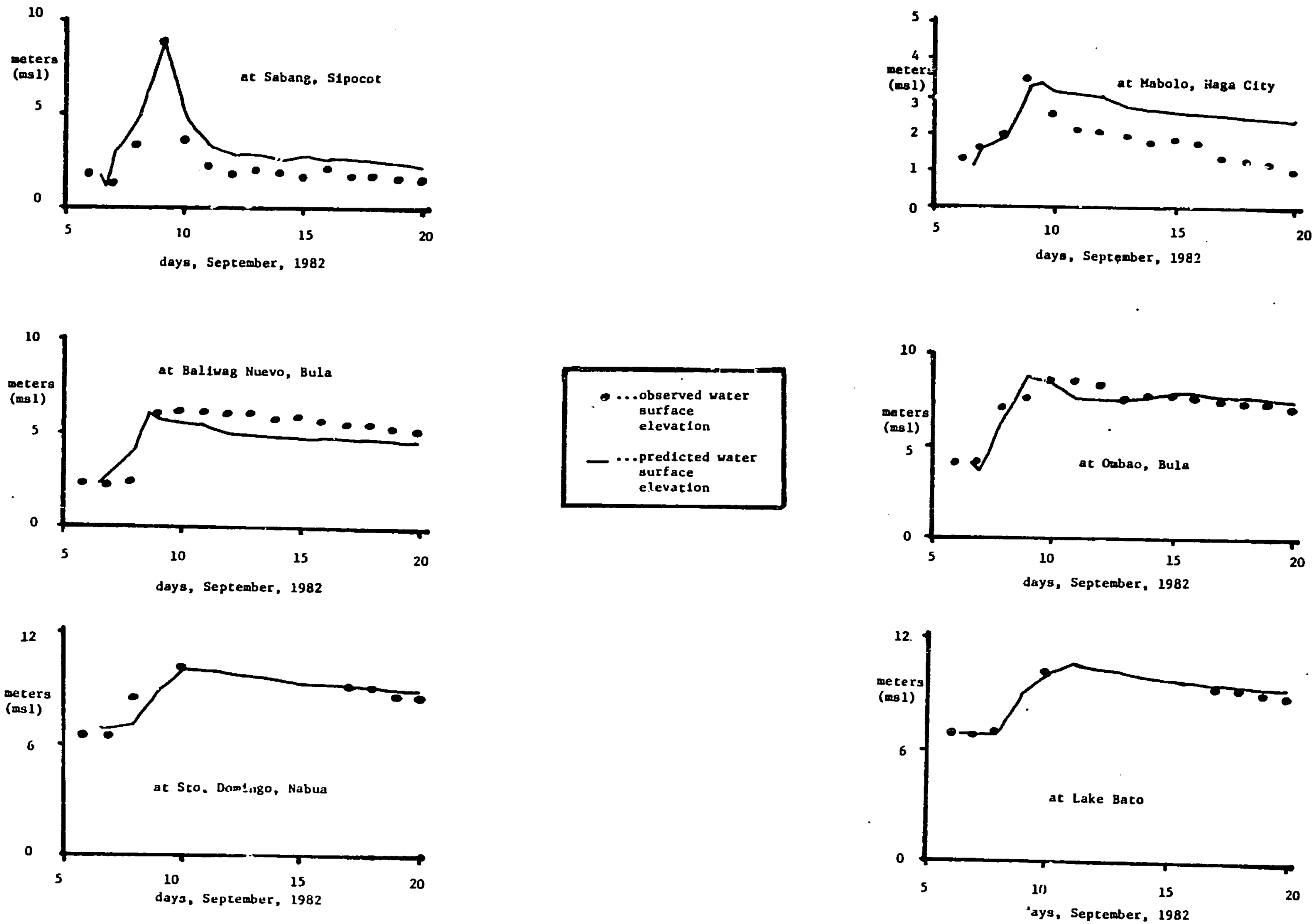
As stated above, model calibration requires that the model be run under known initial and boundary conditions for a previous flood event. The event selected in this case was Typhoon Ruping which occurred in September, 1982. This event was preceded by several days of near steady-state conditions. It also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

3.4 RESULTS OF CALIBRATION

There were five gaging stations from which reliable water surface elevations were available for the period of time surrounding Typhoon Ruping between September 6 to 20, 1982. These stations include: Sabang, 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 six stations from which water elevation records were obtained and 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 six gaging station locations. This was done by adjusting main channel Manning's n and pre-typhoon runoff coefficients. The second objective was to get the model to match the observed peak water elevations and the falling arm of the stage hydrograph. After the model had been made to simulate pre-storm conditions, a 15-day period surrounding Typhoon Ruping was simulated, and predicted stage hydrographs from the simulation were forced to match as closely as possible the observed water stage hydrographs. This was done by adjusting berm roughness coefficients, post-typhoon runoff coefficients, and flood plain effective topwidths. Figure 1 presents the observed versus predicted water elevations for the final calibration constants.

Figure 1: Calibration Run, Predicted versus Observed Water Surface Elevations at Selected Locations, September 6-20, 1982
(Typhoon Ruping, September 7-9, 1982)



4.0 PRODUCTION RUNS

Several flood control alternatives have been proposed over the past 10 years for the Bicol River Basin. One involves extensive levees along much of the Bicol River, a ring levee around Lake Baao, a diversion channel from Ombao to Pasacao, and control structures at Lake Bato, at the point of diversion from Ombao to Pasacao, and on the Pawili River to divert all Pawili River flows into Lake Baao. This recent alternative was examined by the model in order to assess how it might be expected to behave under flood conditions. Simulations were performed for rainfall return periods of 5, 10, 18, 25, and 50 years. This was done both with and without tidal surges at San Miguel Bay and Pasacao. For the runs involving tidal surges, a maximum tide of 3.0 meters was used at San Miguel Bay, and 2.7 meters at Pasacao.

Peak water surface elevations as predicted by the model for these ten production runs are summarized for selected locations on the Bicol and Sipocot Rivers in Table 1. In general, this alternative represents a very significant improvement in the flood hydraulics characteristics of the Bicol River System. The upstream portions of the river from Ombao to Lake Bato have greatly increased conveyance, resulting in much faster rates of drainage from Lake Bato and much shorter periods of inundation. Similar results are observed downstream, with the period of overbank flow at Naga City reduced from approximately 8 days to about 2 days for the 25-year return period.

Table 1: Predicted Maximum Water Surface Elevations for Selected Locations, 5, 10-, 18-, 25-, and 50-year Return Periods, With and Without Tidal Surges

Node No.	Location	5-year		10-year		18-year		25-year		50-year	
		no surge	with surge	no surge	with surge	no surge	with surge	no surge	with surge	no surge	with surge
1	Sam Miguel Bay	1.25	3.00	1.25	3.00	1.25	3.00	1.25	3.00	1.25	3.00
9	Downstream, Curoff Channel 3	2.19	3.05	2.45	3.18	2.66	3.31	2.78	3.38	3.04	3.54
12	Mabulo Bridge, Naga	2.73	3.25	3.05	3.48	3.31	3.67	3.35	3.79	3.76	4.02
19	Downstream from Ombao-Pasacao Diversion point on Bicol River	3.77	3.95	4.07	4.22	4.30	4.43	4.43	4.55	4.65	4.76
20	at Ombao-Pasacao Diversion point	6.09	6.12	6.55	6.57	6.89	6.91	7.11	7.12	7.60	7.61
27	Sto. Domingo Bridge	7.92	7.92	8.40	8.41	8.78	8.78	8.98	8.98	9.38	9.38
29	Lake Bato	8.15	8.15	8.63	8.63	9.00	9.00	9.20	9.21	9.60	9.60
96	at Danao, Pomplona	3.83	4.17	4.20	4.47	4.48	4.71	4.62	4.85	5.00	5.13
99	at outlet of Ombao-Pasacao Diversion Channel	1.29	2.70	1.29	2.70	1.29	2.70	1.29	2.70	1.29	2.70
105	Libmanan River at Libmanan	2.18	3.10	2.41	3.20	2.58	3.28	2.67	3.33	2.86	3.44
110	Sipocot River at Sabang	8.65	8.65	9.34	9.34	9.85	9.85	10.13	10.13	10.60	10.60

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