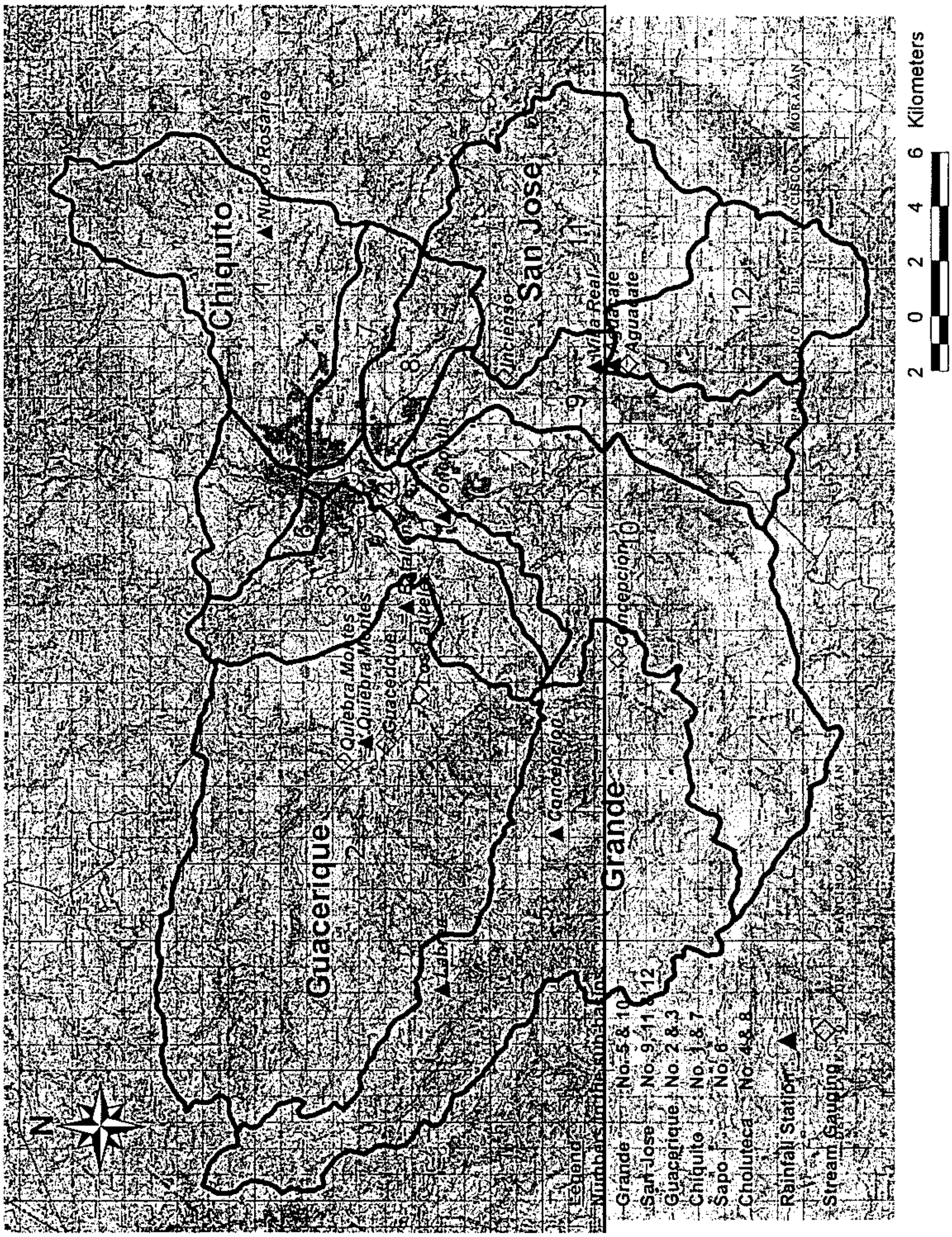
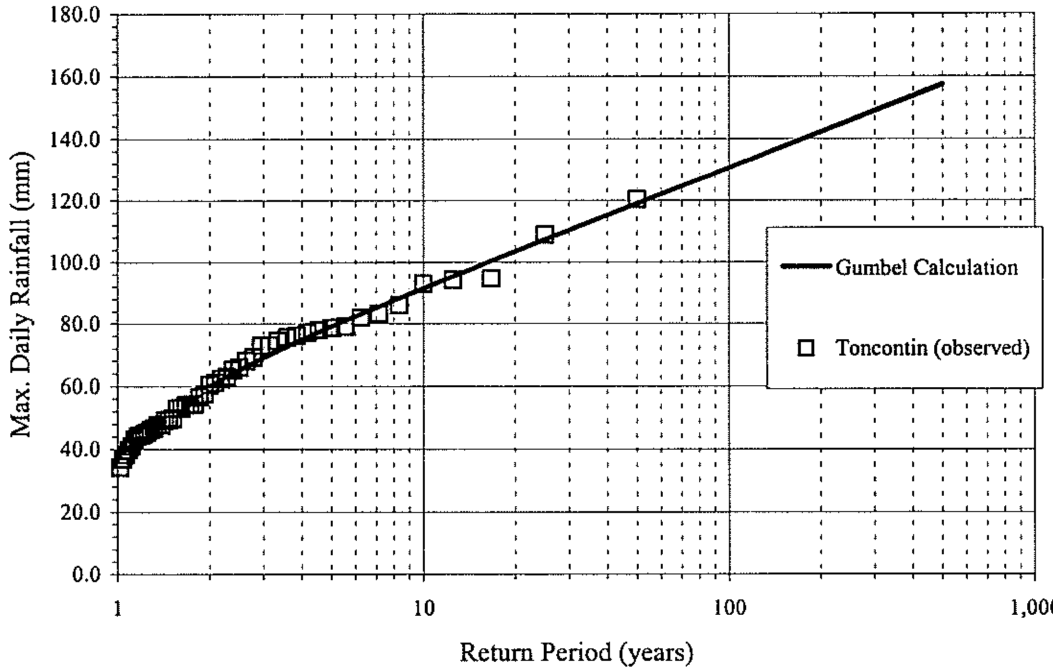


Figure C.2.1	Location of Rainfall and Stream Gauging Stations

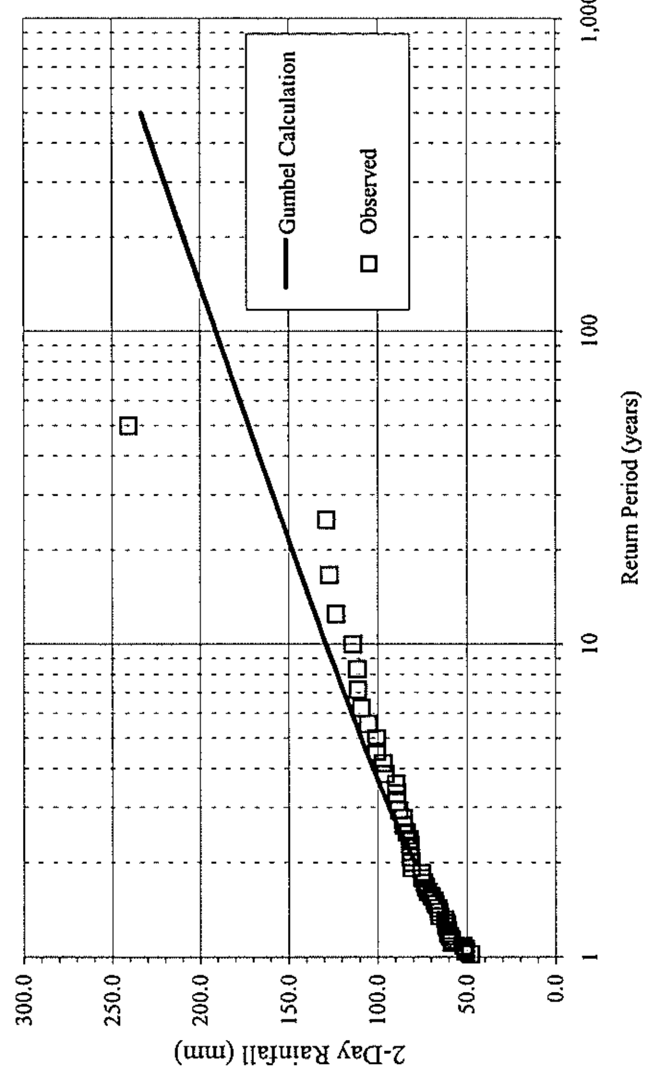
Maximum 1-Day Rainfall and Return Period at Toncontin Station
(Data from 1951 to 1999)



Return Period (yr)	Daily Rainfall (mm)
500	157.5
400	153.7
300	149.0
250	145.9
200	142.2
150	137.4
100	130.6
80	126.9
60	122.1
50	119.0
40	115.2
30	110.4
25	107.3
20	103.5
15	98.6
10	91.5
9	89.7
8	87.6
7	85.2
6	82.4
5	79.0
4	74.8
3	69.1
2	60.2

Figure C.2.2 (1) Maximum 1-Day Rainfall and Return Period at Toncontin Station

Maximum 2-Day Rainfall and Return Period at Toncontin Station
(Data from 1951 to 1999)



Return Period (year)	2-Day Rainfall (mm)
500	233
400	227
300	220
250	215
200	209
150	202
100	191
80	185
60	177
50	172
40	167
30	159
25	154
20	148
15	140
10	129
9	126
8	123
7	119
6	115
5	109
4	103
3	93
2	79

Figure C.2.2 (2)	Maximum 2-Day Rainfall and Return Period at Toncontin Station

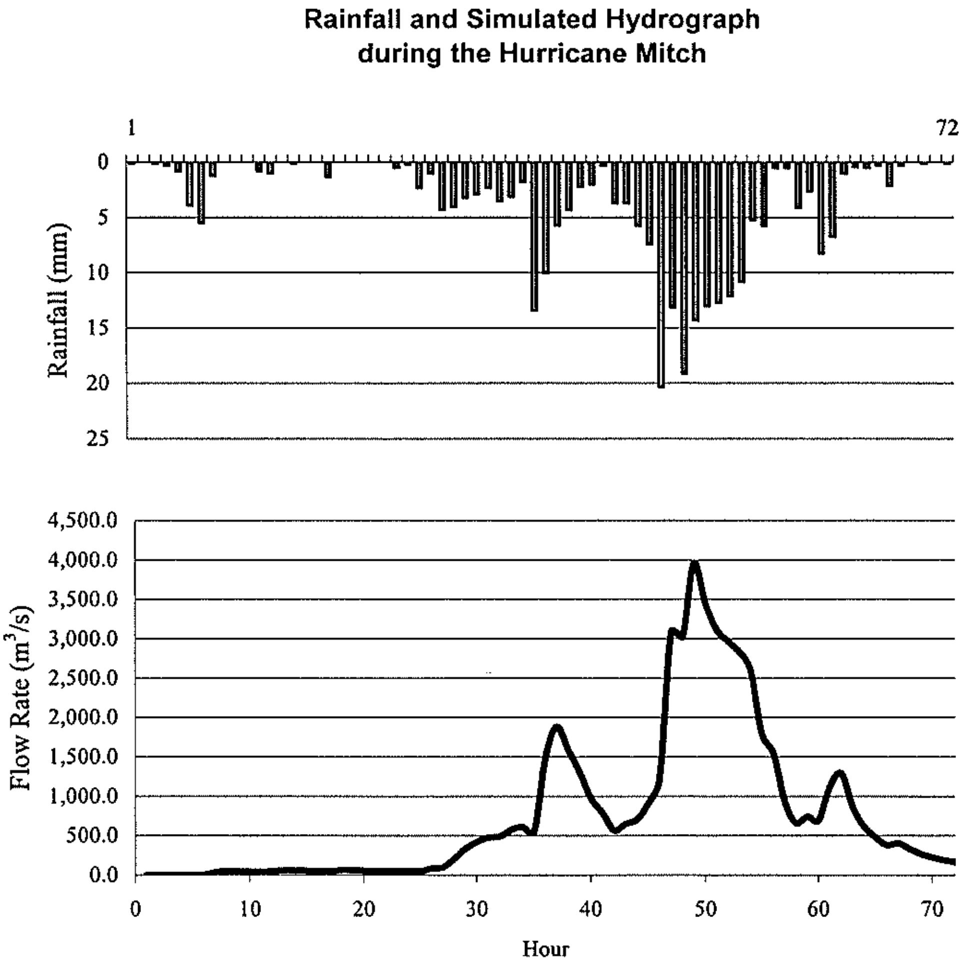


Figure C.2.4	Rainfall and Simulated Hydrograph during the Hurricane Mitch

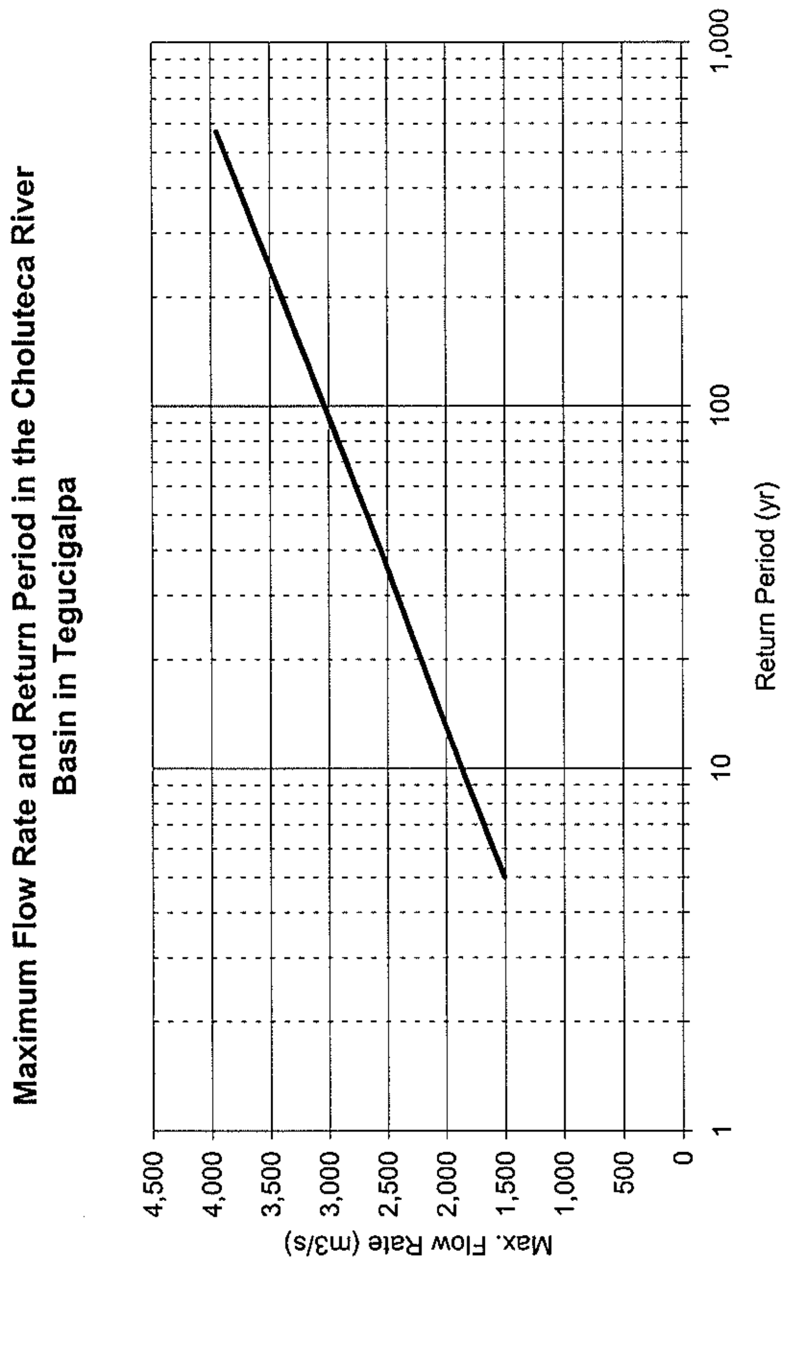


Figure C.2.5	Maximum Flow Rate and Return Period in the Choluteca River Basin in Tegucigalpa

Return Period (year)	1-Day Rainfall (mm)
500	248
400	241
300	232
250	227
200	220
150	211
100	198
80	191
60	182
50	176
40	169
30	160
25	154
20	147
15	138
10	124
9	121
8	117
7	112
6	107
5	101
4	93
3	82
2	65

Maximum 1-Day Rainfall and Return Period at Conception Station (1972-1998)

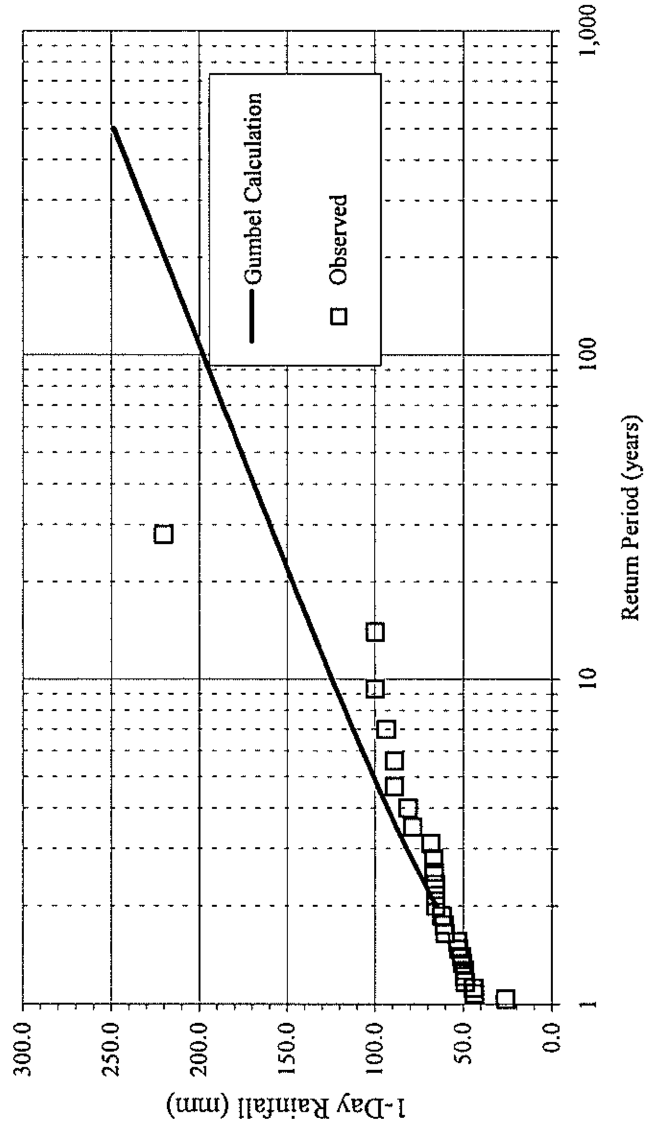
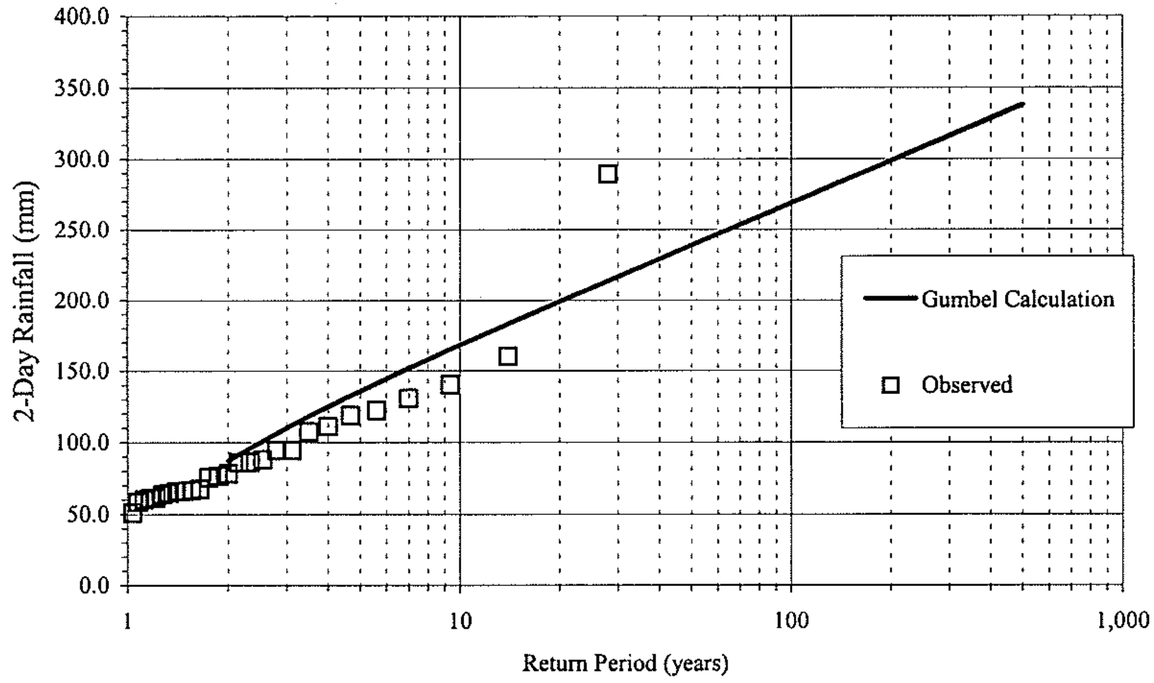


Figure C.3.1 (1)	Maximum 1-Day Rainfall and Return Period at Concepcion Station

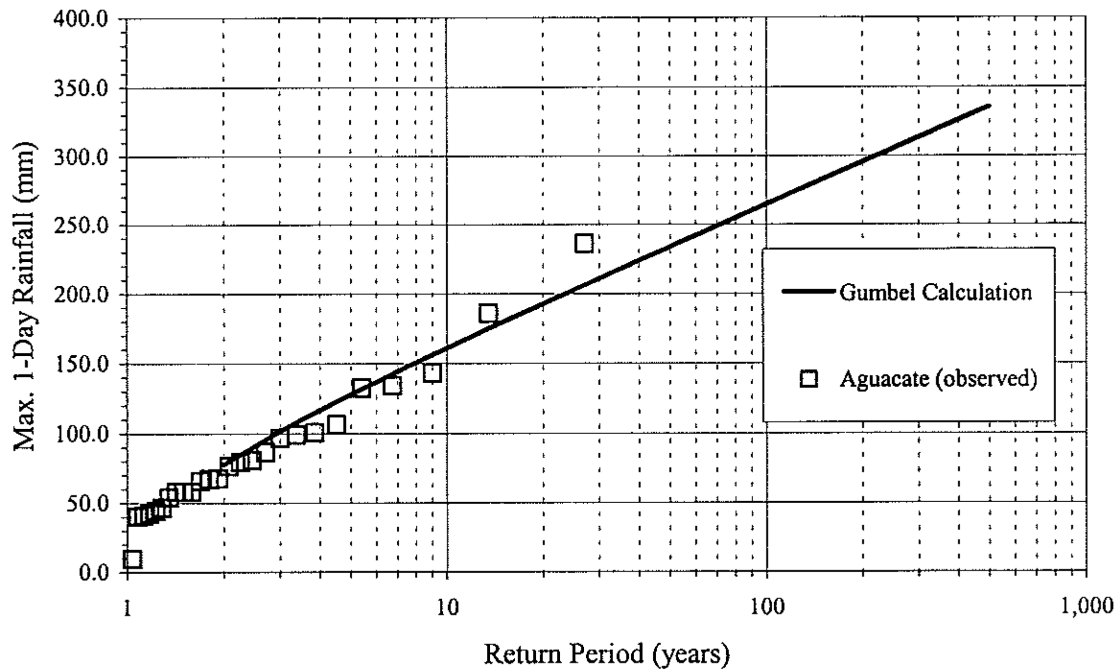
Maximum 2-Day Rainfall and Return Period at Conception Station (1972-1998)



Return Period (year)	2-Day Rainfall (mm)
500	338
400	328
300	316
250	308
200	299
150	286
100	269
80	259
60	247
50	239
40	229
30	217
25	209
20	199
15	186
10	168
9	163
8	158
7	152
6	145
5	136
4	125
3	110
2	87

Figure C.3.1 (2) | Maximum 2-Day Rainfall and Return Period at Concepcion Station

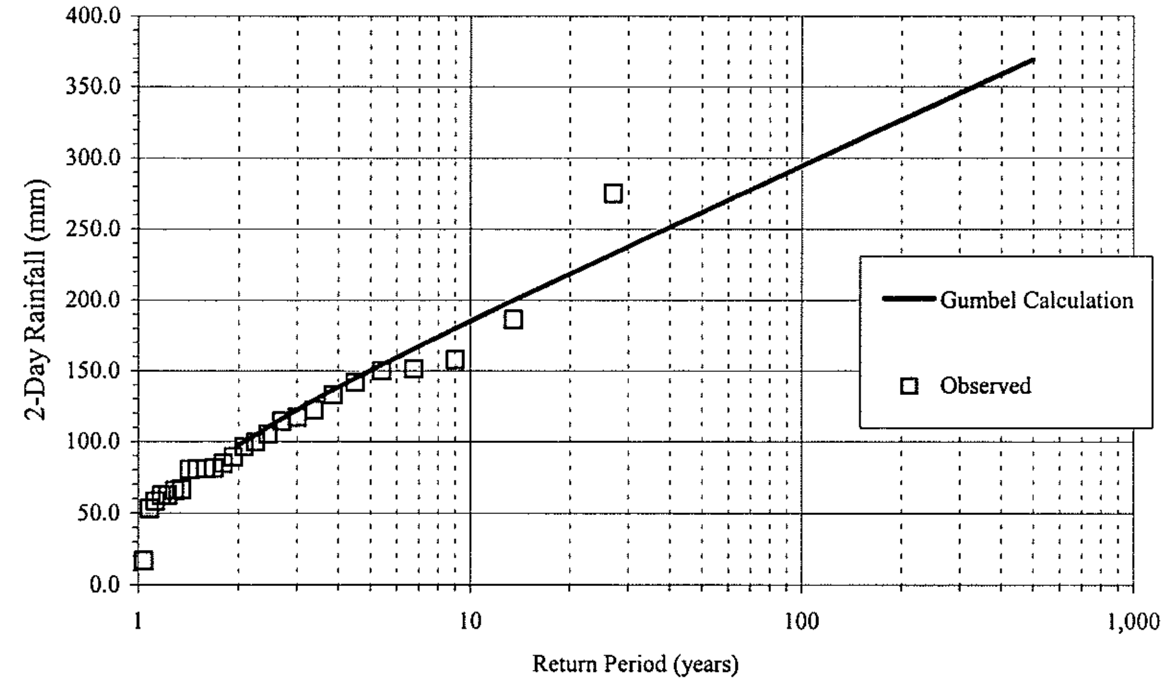
Maximum 1-Day Rainfall and Return Period at Aguacate and Villa Real Station
(Aguacate : 1972-1990, Villa Real : 1991-1998))



Return Period (yr)	Daily Rainfall (mm)
500	335.8
400	326.0
300	313.3
250	305.2
200	295.4
150	282.6
100	264.7
80	254.8
60	242.0
50	233.9
40	223.9
30	211.0
25	202.8
20	192.7
15	179.7
10	161.0
9	156.1
8	150.5
7	144.2
6	136.8
5	127.9
4	116.7
3	101.5
2	77.9

Figure C.4.1 (1) — **Maximum 1-Day Rainfall and Return Period at Aguacate and Villa Real Station**

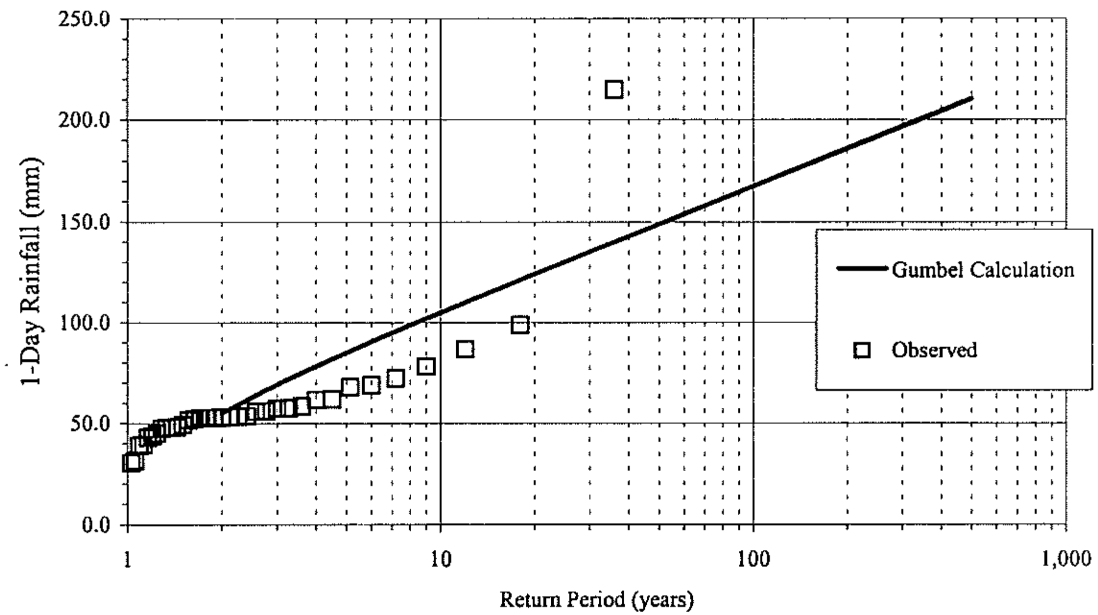
Maximum 2-Day Rainfall and Return Period at Aguacate and Villa Real Station
(Aguacate : 1972-1990, Villa Real : 1991-1998)



Return Period (year)	2-Day Rainfall (mm)
500	369
400	359
300	346
250	337
200	327
150	313
100	294
80	284
60	270
50	262
40	251
30	238
25	229
20	219
15	205
10	185
9	180
8	174
7	167
6	160
5	150
4	138
3	122
2	98

Figure C.4.1 (2) Maximum 2-Day Rainfall and Return Period at Aguacate and Villa Real Station

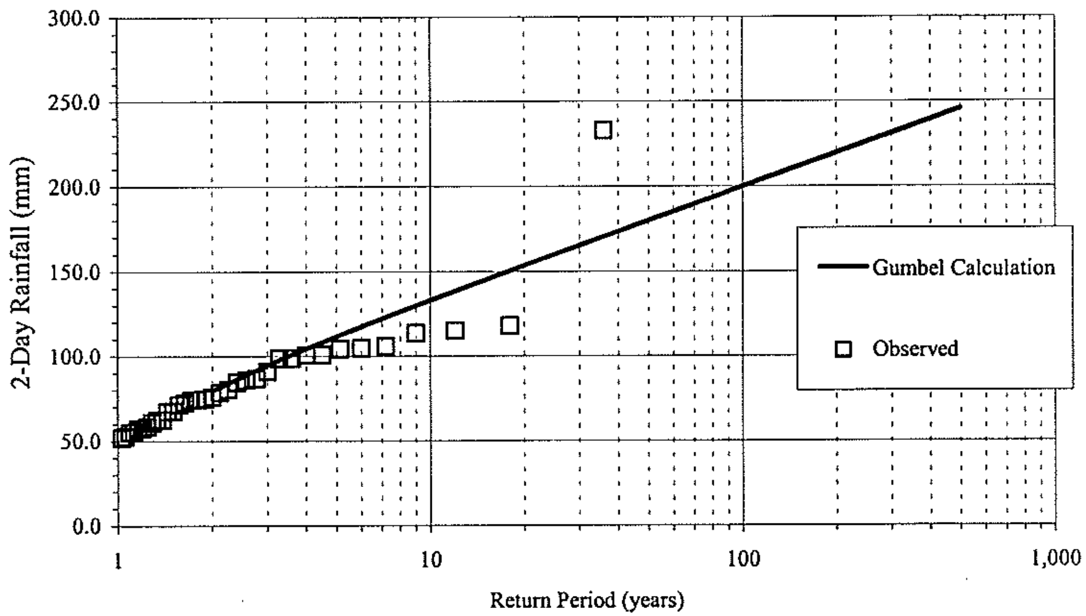
Maximum 1-Day Rainfall and Return Period at Batallon and Quiebra Montes Station
(Batallon : 1963-1990, Quebra Montes : 1991-1998)



Return Period (year)	1-Day Rainfall (mm)
500	210
400	204
300	197
250	192
200	186
150	178
100	167
80	161
60	154
50	149
40	143
30	135
25	130
20	124
15	116
10	105
9	102
8	99
7	95
6	90
5	85
4	78
3	69
2	55

Figure C.5.1 (1)	Maximum 1-Day Rainfall and Return Period at Batallon and Quiebra Montes Station

Maximum 2-Day Rainfall and Return Period at Batallon and Quiebra Montes Station
(Batallon : 1963-1990, Quiebra Montes : 1991-1998)



Return Period (year)	2-Day Rainfall (mm)
500	246
400	239
300	231
250	226
200	219
150	211
100	200
80	193
60	185
50	180
40	173
30	165
25	160
20	153
15	145
10	133
9	130
8	126
7	122
6	117
5	112
4	104
3	95
2	80

Figure C.5.1 (2) Maximum 2-Day Rainfall and Return Period at Batallon and Quiebra Montes Station

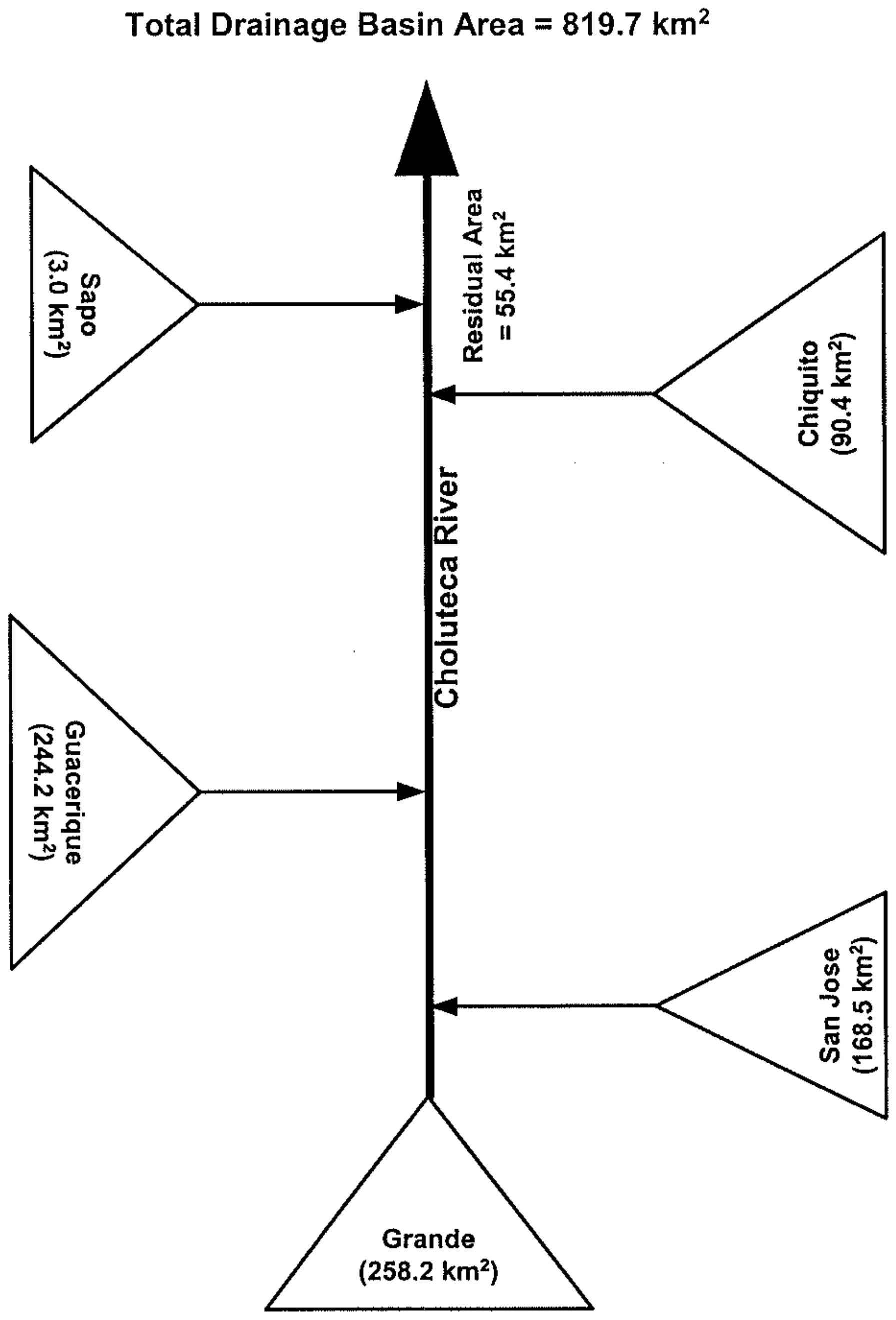


Figure C.7.1	River Model of the Choluteca River Basin in Tegucigalpa

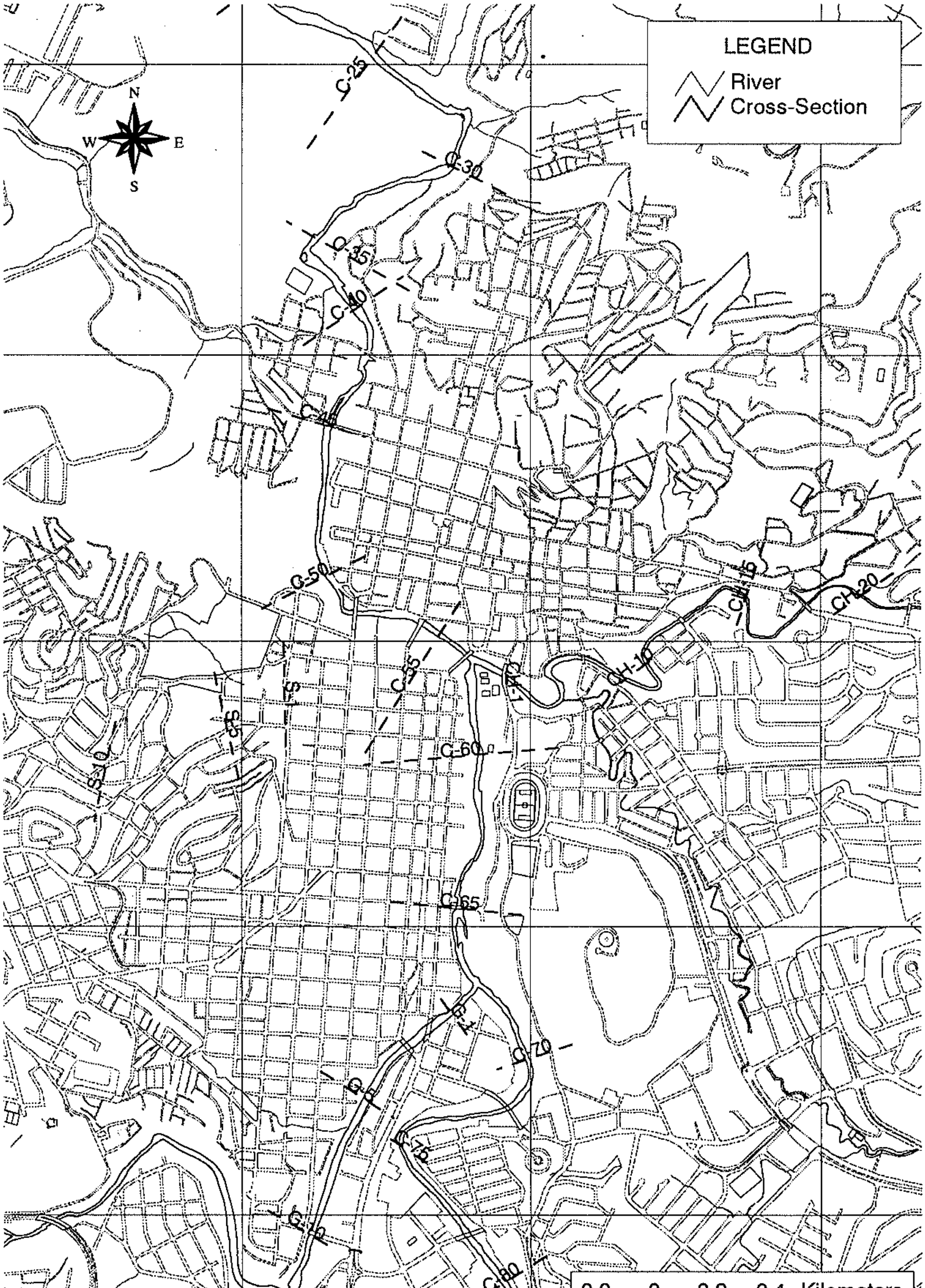


Figure C.7.2	**Locations of River Cross Sections in Central Area**

Comparison of Cross Sections at Berinche in 1996 and 2001

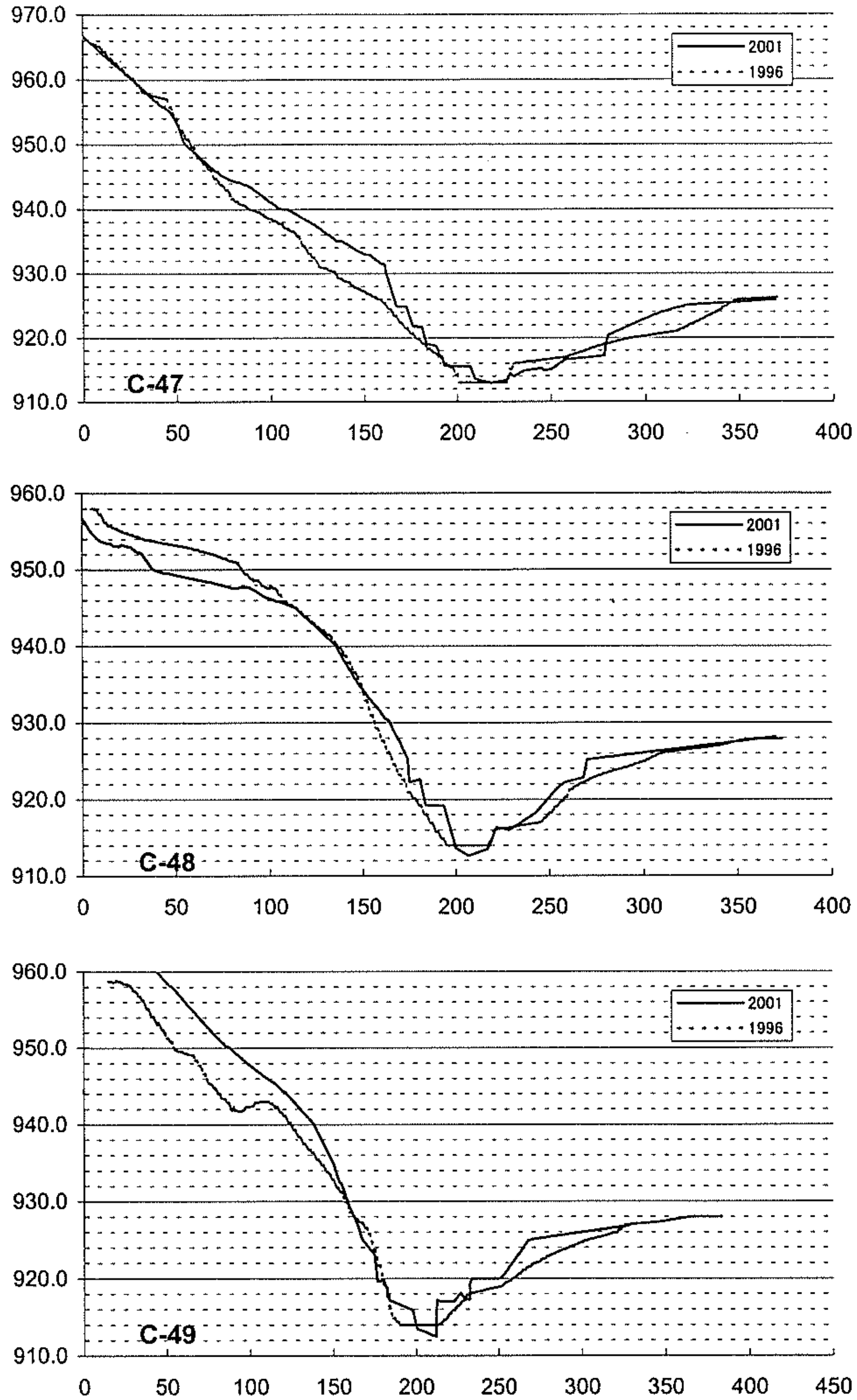


Figure C.7.3 (1)	Comparison of Cross Sections at Berinche in 1996 and 2001

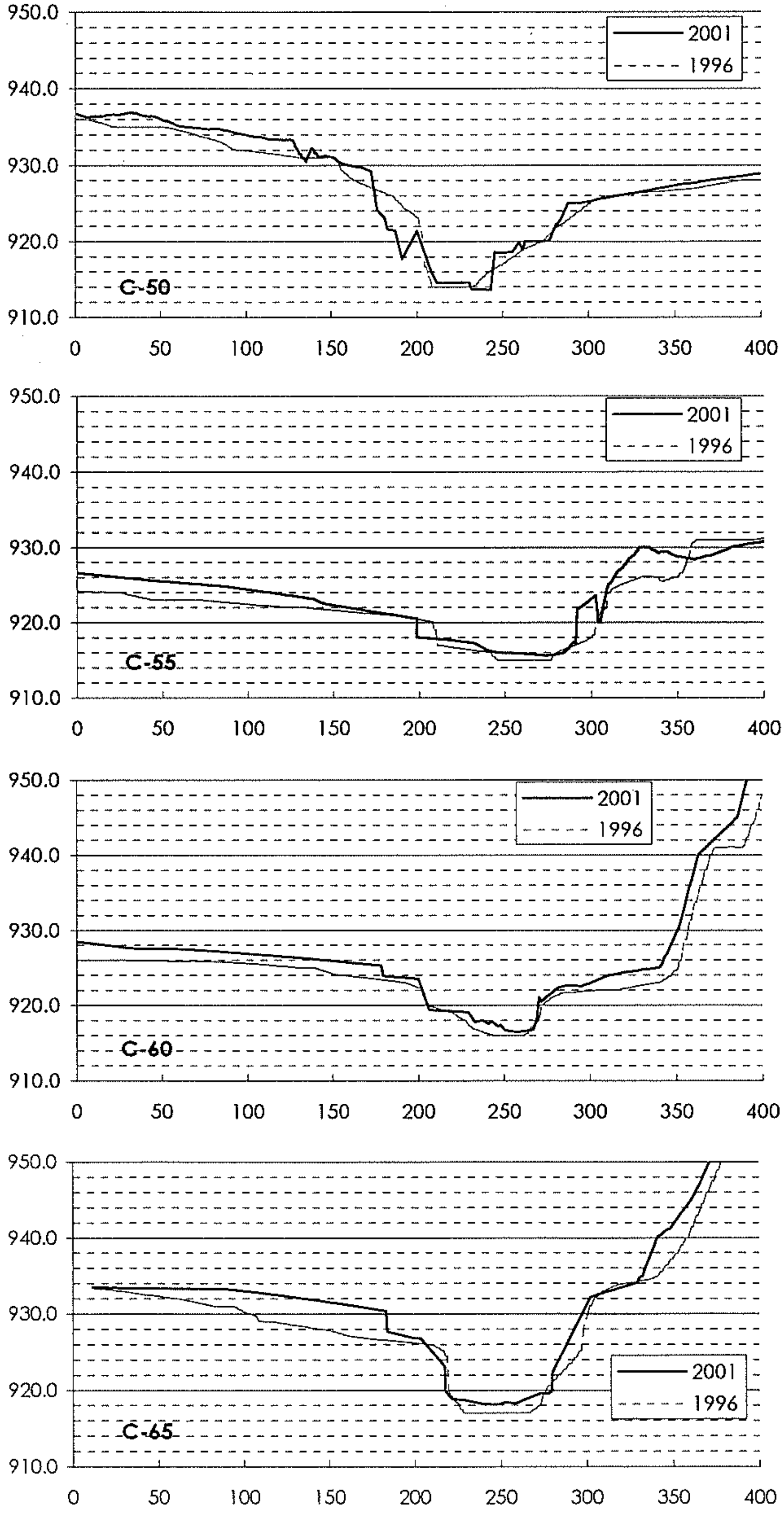


Figure C.7.3 (2)	Comparison of Cross Sections in 1996 and 2001

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1) "Lote 3 : Actualizacion del Plan Maestro de Abastecimiento de Agua Potable de Tegucigalpa" by SANAA in 1989.

2) "Diagnostico de Las Obras de Captacion del Sistema de Abastecimiento Hidrico de Tegucigalpa" by SANAA in 1999.

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6) "Survey Response to Hurricane in Honduras in 1998" by USGS.

7) "Informe de Visita a la Launa del Pescado in 1999" by Ing. Christopher Bonnard, et.al.

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SUPPORTING REPORT C

APPENDIX C

APPENDIX C.1

THEORETICAL APPROACH

APPENDIX C.1 THEORETICAL APPROACH

APPENDIX C.1.1 FREQUENCY ANALYSIS

(1) Theoretical Approach

The standard Gumbel method is used to analyze the relationship of the rainfall or flow rate and its return period. The basic equations are as follows:

$$T = \frac{1}{P(x)} = \frac{1}{1-F(x)} \qquad \text{(C.1.1)}$$

where T = return period, year

$P(x)$ = Probability of Exceedance

$F(x)$ = Probability of Non-exceedance

x = Maximum rainfall or flow rate each year, mm or m^3/s

From a series of data x, $F(x)$ can be calculated by using the Hazen Method or Weibull Method as follows:

$$F(x) = 1-\frac{j}{N+1} \qquad \text{(C.1.2)}$$

Where j = Order of x_j from maximum

N = Total number of the data series

From the above $F(x)$, a new parameter x and y are defied as follows:

$$F(x) = 1-exp\left(-e^{-y}\right) \qquad \text{(C.1.3)}$$

$$y = -ln\{-ln\,F(x)\} = a(x-x_0) \qquad \text{(C.1.4)}$$

where a and x_0 can be calculated from the following equation

$$\frac{1}{a} = \frac{S_x}{S_y} \qquad \text{(C.1.5)}$$

$$x_0 = \bar{x}-\left(\frac{1}{a}\right)\bar{y} \qquad \text{(C.1.6)}$$

$$S_x = \sqrt{\frac{1}{N}\sum_{i=1}^{N}\left(x_i-\bar{x}\right)^2}\ ,\ S_y = \sqrt{\frac{1}{N}\sum_{i=1}^{N}\left(y_i-\bar{y}\right)^2} \qquad \text{(C.1.7)}$$

$$F(x) = 1-exp\left(-e^{-y}\right) = 1-\frac{j}{N+1} \qquad \text{(C.1.8)}$$

Where $\bar{x}$, $\bar{y}$ = Average value of the data series x and y

The relationship between rainfall or flow rate (x) and return period (T) can be converted to the following equation:

$$x = x_0 + \left(\frac{1}{a}\right)y \qquad \text{(C.1.9)}$$

$$y = -ln\{ln\,T - ln(T-1)\} \qquad \text{(C.1.10)}$$

Where x_0 and a are now know parameters

(2) Data Arrangement

The data used as the input for the model are as follows:

- Maximum rainfall (normally hourly) or flow rate each year
- The table of standard parameters for Gumbel method (the relation between the number of samples, average y and the standard deviation of y ($N, \bar{y}$ and S_y)

APPENDIX C.1.2 RAINFALL – RUNOFF ANALYSIS

(1) Theoretical Approach

A storage function method is used to analyze the relationship between the rainfall and runoff. The basic equations are as follows:

$$r_e - q_l = \frac{ds_l}{dt} \qquad \text{(C.2.1)}$$

$$s_l = kq_l^{\,p} \qquad \text{(C.2.2)}$$

where q_l = discharge, mm

r_e = average rainfall in the basin, mm

s = storage, mm

t = time, s

The above equation can be simplified and discretized as follows:

$$q_l \rightarrow q \ , \ s_l \rightarrow s \qquad \text{(C.2.3)}$$

$$r_{e,t} - \frac{q_{t-\Delta t} + q_t}{2} = \frac{s_t - s_{t-\Delta t}}{\Delta t} \qquad \text{(C.2.4)}$$

$$\frac{s_t}{\Delta t} + \frac{q_t}{2} = \left(\frac{s_{t-\Delta t}}{\Delta t} - \frac{q_{t-\Delta t}}{2} \right) + r_{e,t} \qquad \text{(C.2.5)}$$

The Newton – Ralpson method was employed to calculate the above equation by assuming *f(q)* as follows:

$$f(q) = aq^p + bq + C = 0 \qquad \text{(C.2.6)}$$

By using 2nd order Tayler's series, the derivative of *f(q)* is

$$f'(q_1) = paq_1^{\,p-1} + b \qquad \text{(C.2.7)}$$

Therefore, the Newton – Ralpson equation can be expressed as:

$$y - f(q_1) = (paq_1^{\,p-1} + b) \times (q - q_1) \qquad \text{(C.2.8)}$$

$$q_i = q_{i-1} - \frac{aq_{i-1}^{\,p} + bq_{i-1} + c}{paq_{i-1}^{\,p-1} + b} \qquad \text{(C.2.9)}$$

From this equation, q_i can be calculated from q_{i-1}. The program will select the best value of q_i that makes

$$y - f(q_1) = 0 \qquad \text{(C.2.9)}$$

(2) Data Arrangement

The data used as the input for the model are as follows:

- The synthetic rainfall pattern at each return period and
- The necessary parameters in the model (k, p and drainage basin area).

APPENDIX C.1.3 HYDRAULIC SIMULATION

The data on water level and discharge are available from the gauging stations in the basin. Hydrograph is calculated and used as a boundary condition. An unsteady flow program, MIKE11 developed by the Danish Hydraulic Institute (DHI), is used to simulate the flow along the river.

(1) Theoretical Approach

The program can be used to solve the vertically integrated equations of conservation of continuity and momentum (so called "Saint Venant equation") for incompressible and homogeneous fluid. The basic governing equations are:

$$\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} = q \qquad \text{(C.3.1)}$$

$$\frac{\partial Q}{\partial t} + \frac{\partial \left(a \frac{Q^2}{A} \right)}{\partial x} + gA \frac{\partial h}{\partial x} + \frac{gQ|Q|}{C^2 AR} = 0 \qquad \text{(C.3.2)}$$

where Q = discharge, m^3/s

A = flow area, m^2

q = lateral inflow, m^2/s

h = stage above datum, m

C = Chezy resistance coefficient, $m^{1/2}/s$

R = hydraulic radius, m

a = momentum distribution coefficient

g = gravity acceleration, m/s^2

t, x = The axis of time, s, and distance, m, respectively

These equations are transformed into a series of finite difference equations in a computational grid consisting of alternating Q-points (discharge) and h-points (water level). The transformed equations are as follows:

$$\frac{\partial Q}{\partial x} \approx \frac{\frac{\left(Q_{j+1}^{n+1} + Q_{j+1}^{n}\right)}{2} - \frac{\left(Q_{j-1}^{n+1} + Q_{j-1}^{n}\right)}{2}}{\Delta 2x_j} \qquad \text{(C.3.3)}$$

$$\frac{\partial A}{\partial t} = b_s \frac{\partial h}{\partial t} \approx \frac{\left(h_j^{n+1} - h_j^{n}\right)}{\Delta t} \qquad \text{(C.3.4)}$$

$$\frac{\partial Q}{\partial t} \approx \frac{\left(Q_j^{n+1} - Q_j^n\right)}{\Delta t} \tag{C.3.5}$$

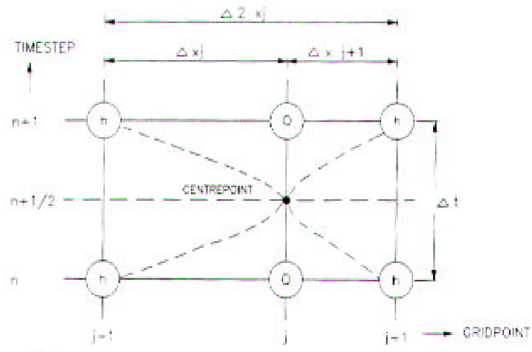
$$\frac{\partial\left(a\,\frac{Q^2}{A}\right)}{\partial x} \approx \frac{\left(\left[a\,\frac{Q^2}{A}\right]_{j+1}^{n+\frac{1}{2}} - \left[a\,\frac{Q^2}{A}\right]_{j-1}^{n+\frac{1}{2}}\right)}{\Delta 2x_j} \tag{C.3.6}$$

$$\frac{\partial h}{\partial x} \approx \frac{\frac{\left(h_{j+1}^{n+1} + h_{j+1}^{n}\right)}{2} - \frac{\left(h_{j-1}^{n+1} + h_{j-1}^{n}\right)}{2}}{\Delta 2x_j} \tag{C.3.7}$$

where b_s = river width, m

n, j = time and distance step

The schematic diagram of time and distance increment is illustrated as follows:



(2) Data Arrangement

The data used as the input for the model are as follows:

- The grid set up from the river survey for river network,
- River cross sections' coordinates,
- River bed and material data, and
- The boundary condition, in this case the hydrograph at the upstream end and the water level at the downstream end.