CHAPTER 6

HYDROLOGICAL INVESTIGATION AND ANALYSIS

6.1 PROCEDURE

The Mindanao section of the Pan Philippine Highway begins at Surigao City in the north and ends at Davao City in the south. The total length is about 403kms. Within this length, the road crosses various rivers and catchment areas except the case of the road passing on the ridge of hill or upland.

When rainfall falls on each catchment area, runoff water from the upper catchment area passes across the highway to downstream. If the runoff water can not go under the highway, floods occurs at the cross point of water current and the highway.

This section presents basic hydrological studies performed to decide suitable and safe countermeasures against flood. The hydrological analysis process is shown in Figure 6-1.1.

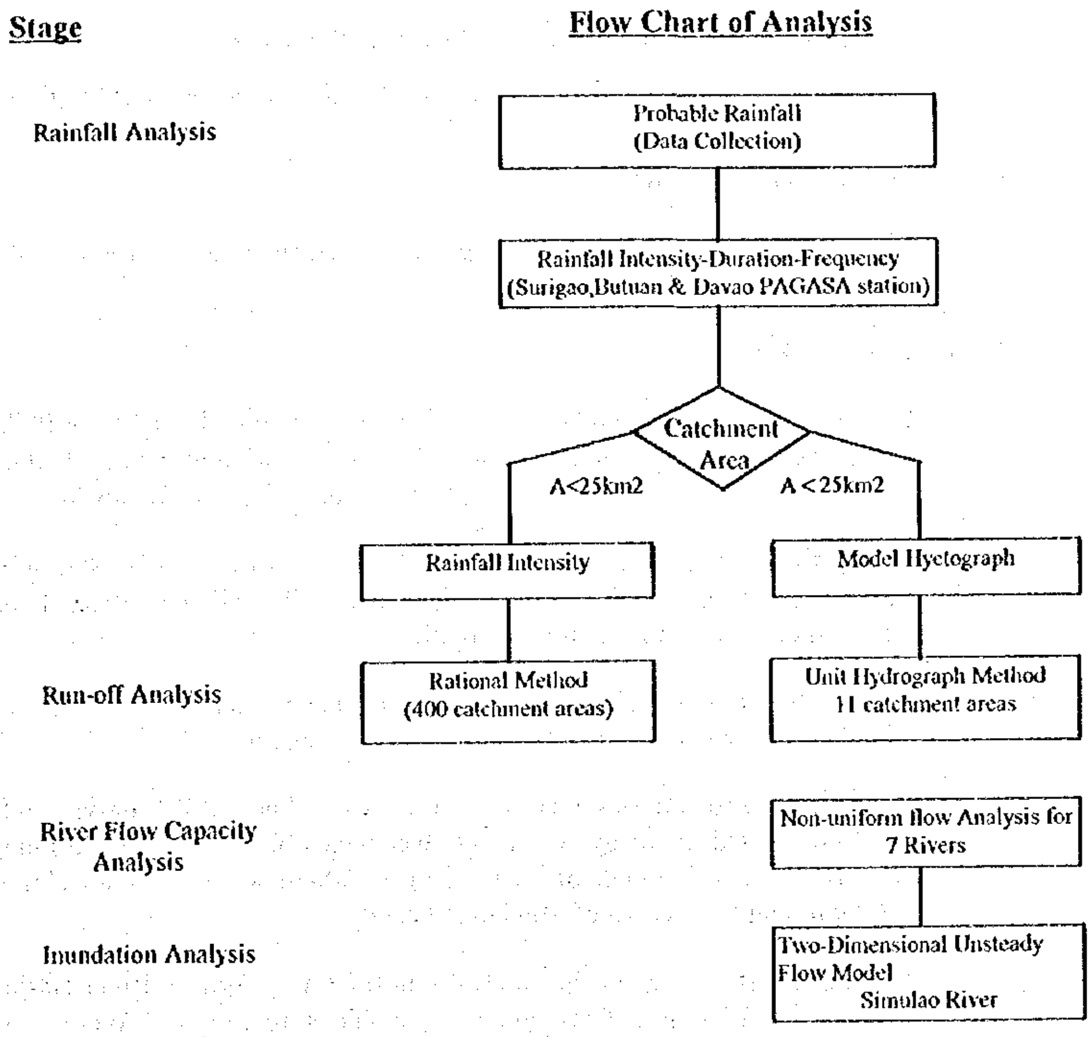


FIGURE 6.1-1 FLOW CHART OF HYDROLOGICAL ANALYSIS

6.2 PREPARATION WORKS AND FIELD SURVEY

For the preparation works of hydrological study, boundary of catchment areas along the Study Road were determined based on the topographic maps at scale of 1:50,000 obtained from the National Mapping and Resource Information Authority (NAMRIA).

Approximately 400 catchment areas were identified. The area, river length, higher and lower elevation of each catchment area were measured as the required basic data. These data were used for run-off analysis.

The field survey on hydrology was carried out from October 10 to November 1st. 1995. It was performed principally in 18 flood sections selected during the Feasibility Study. The purpose of field survey were as follows:

- To determine catchment area condition including sediment production in the upper stream of flood sections for run-off analysis;
- To determine the boundary of catchment areas which were indistinct in the preparation works;
- To determine the point of overflow from the rivers;
- To confirm the inundation area as well as the water depth of big floods.

6.3 DATA COLLECTION

Data collection was performed to get the physical data and reports of other projects related to this project.

6.3.1 Rainfall Data

Rainfall data of the three climatological stations were collected from Philippine Atmospheric Geophysical Astronomical Services Administration (PAGASA). These stations are located in Surigao City, Butuan City and Davao city.

During the field survey, effort was made to obtain other rainfall station's data and 13 years rainfall data were obtained from NIA Simulao River Irrigation Field Office at the municipality of Trento.

An inventory of data collected and the annual maximum daily rainfall/the annual rainfall are shown in Table 6.3-1 and 6.3-2, respectively.

In the northeast monsoon season from November to February, rainfall is generally caused about by northwest wet winds blowing from the Philippine Sea. Thus, heavy rainfall of more than 3,500mm a year occurs along east coastal mountainous area of Mindanao Island.

The northwest wet winds becomes dry in the lower Agusan River Basin after crossing the east mountains and less rainfall is recorded. Average annual rainfall is 3,620 mm at Surigao, 1,605 mm at Butuan, 3,003 mm at Trento and 1,853 mm at Davao.

TABLE 6.3-1 INVENTORY OF RAINFALL DATA

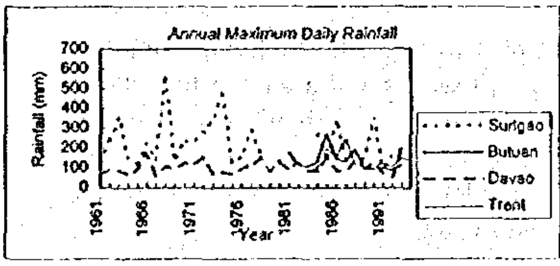
No	Station Name	Elev (m)	60	61	62	63	64	65	66	67	68	69	70	71	72	73	74	75	76	77	78	79	80	81	82	83	84	85	86	87	88	89	90	91	92	93	94
1	Suligao	39	A	A	A	A	A	A	A	A	A	A	A	A	A	A	A	A	A	A	A	B	B	B	B	B	A	A	A	A	A	A	A	A	A		
2	Butuan	18																						A	A	A	A	A	A	A	A	A	A	A	A	A	
3	Trent	30																							B	A	A	A	A	A	A	A	A	A	A	A	A
4	Davao	18	A	A	A	A	A	A	A	A	A	A	A	A	A	A	A	A	A	A	A	A	A	A	A	A	A	A	A	A	A	A	A	A	A	A	

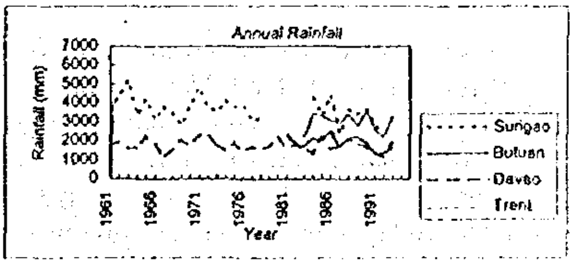
A: Complete Data B: Partial Data

TABLE 6.3-2 ANNUAL MAXIMUM DAILY RAINFALL AND ANNUAL RAINFALL

YEAR	RAINFALL STATION Surigao	Butuan	Trent	Davao
1961	122.4			67.3
1962	236.7			86.6
1963	351.8			84.8
1964	151.7			68.6
1965	117.0			91.4
1966	218.9			174.3
1967	139.3			62.7
1968	564.7			106.7
1969	131.6			96.8
1970	235.3			124.3
1971	231.7			114.6
1972	278.7			150.3
1973	348.5			73.2
1974	472.9			78.7
1975	130.7			71.1
1976	154.4			97.1
1977	287.2			108.2
1978	181.1			150.0
1979				83.6
1980				130.1
1981		175.8		90.0
1982		110.0		129.4
1983		108.8	78.5	86.0
1984	269.1	127.8	101.6	83.9
1985	156.0	271.6	198.1	149.6
1986	339.0	147.0	157.5	87.7
1987	147.0	130.3	245.1	85.9
1988	123.0	191.4	118.1	132.2
1989	149.7	96.3	101.6	96.9
1990	344.6	93.9	97.0	120.0
1991	99.6	98.2	124.9	74.2
1992	153.0	86.4	104.1	51.6
1993		111.3	148.1	193.0
1994			140.2	

YEAR	RAINFALL STATION Surigao	Butuan	Trent	Davao
1961	3547.1			1762.4
1962	4461.4			1933.5
1963	5126.2			1576.0
1964	3376.2			1628.3
1965	4171.1			2200.8
1966	3128.2			1808.0
1967	3773.9			1128.2
1968	3378.9			1551.3
1969	2831.6			2004.2
1970	3925.0			1791.7
1971	4769.2			2251.4
1972	3729.9			2225.0
1973	3543.6			1769.4
1974	4099.3			1503.3
1975	3767.7			1884.3
1976	3765.6			1502.5
1977	3088.5			1675.4
1978	3179.3			1584.8
1979				1704.2
1980				2165.8
1981		2341.6		1683.1
1982		1867.0		1752.1
1983		1696.5	2183.7	1627.0
1984	4253.2	2113.4	3429.9	1336.6
1985	3326.9	1979.1	3298.8	2192.5
1986	4339.5	2482.7	3048.1	1549.6
1987	2407.7	1691.5	2947.9	1710.9
1988	3696.6	2155.9	3393.6	2095.7
1989	3395.6	2227.5	2796.4	1850.3
1990	3656.6	1915.4	3591.9	1685.0
1991	2782.0	1353.4	2499.2	1339.4
1992	2253.8	1351.2	2277.2	1176.3
1993		1604.9	3340.6	1853.1
1994			3230.5	
Average	3621.3	1906.2	3003.2	1742.5





6.3.2 Other Reports Related to This Project

There are two big development plans related to the project namely; 1) the Upper Agusan River, which is located on the upper stream of Monkayo Town, under the Upper Agusan Development Project by DPWH and, 2) the Liboganon River Irrigation Project by National Irrigation Administration (NIA).

Summary of these projects are described as follows;

1) The Upper Agusan River

The Agusan River is the third largest river in the country and has a basin area of 11,700km2. The river length is about 350km from river mouth at Butuan Bay. This river was included as one of the projects of Nationwide Flood Control Plan and River Dredging Program(1982).

The Agusan River is divided topographically into the lower, middle and upper Agusan. The upper Agusan is upper stream from the intersecting point with the river and the highway where Kalaw Bridge is located at Monkayo Town. The Study of Lower Agusan Development Project was completed in 1982 and the Upper Agusan Development Project in 1984.

Peak discharge at Kalaw Bridge is estimated at 2900m3/s for 100-years probable flood in present condition. If the development plan is realized in the future, the peak discharge is estimated to increase to 5800m3/s acoording to the basic flood control plan.

Return Period	Peak Discharge
50-year without project	2,500 m^3/s
100-year without project	2,900 m^3/s
50-year with project	4,910 m^3/s
100-year with project	5,780 m^3/s

SOURCE: Master Planning and Detailed Engineering Flood Control and Drainage of the Upper Agusan Development Project

(2) The Liboganon River

Development of the Liboganon River under the Second Davao Irrigation Project by NIA started in 1975 as one of the four projects of the Mindanao Irrigation Study.

This project was a composite development plan to upgrade the living condition of rural areas. A flood dike was one of the civil works and a 21 km long flood protection dike was constructed to protect the Liboganon irrigation service area of 10,500ha. located entirely on the right bank side of Liboganon River. To confirm the flood prediction, a hydraulic model test was performed by the National Hydraulic Reserch Center in 1980 and 1982.

On the other hand, the left bank was planned by the Liboganon Flood Control Study (1988) and its constructon is being implemented by DPWH.

The Liboganon Flood Control Study was planned on the basis of the present condition. At the location of the existing highway, the flood was assumed to pass over the highway, thus no new bridge was planned at this location.

In this report, the peak discharges for different return period were calculated at point of the highway or the Gov. Miranda Bridge as listed below;

Return Period	Peak Discharge
10-year	1,600 m^3/s
25-year	2,100 m^3/s
50-year	2,489 m^3/s

SOURCE: The Liboganon Flood Control Study (1980)

The general layout plan and profile of the flood protection dikes was designed for a 25-year return period and allowance was 1m from H.W.L. to top of dike elevation. The cross sections of Liboganon River were obtained from NIA. It is to be noted that the topographic elevation of NIA is 2.9 m higher than our reference point.

6.4 RAINFALL ANALYSIS

6.4.1 *Probable Rainfall*

The rainfall data of three stations of PAGASA, namely Surigao City, Butuan City and Davao City have been analyzed by PAGASA and probable rainfall has been developed from 5 minutes up to 24 hours duration as shown in Table 6.4-1.

Probable rainfall of three PAGASA's and Trento NIA's stations was also checked by the Study Team by use of Gambel Method using the annual maximum daily data (24 hours data). Figure 6.4-1 shows the result of calculation. The results of Surigao and Davao stations were almost the same values as PAGASA. In the case of Butuan data, our result was smaller than PAGASA's. Probable rainfall of Trento showed similar values to the Study Team's prediction of Butuan.

The Study Team used 13 years rainfall data (1980-1993), for analysis where as PAGASA used 43 years data. In this project, PAGASA's probable rainfall was adopted for analysis.

Topograpy influences climate and also rainfall pattern changes at a boundary like a ridge of mountain. This project consists of 19 packages. From the climatologic and topographic viewpoint, the applicable rainfall condition for each package are recommended to use the following station's value:

Package 1	: Surigao station	(North Coast Area)
Packages 2-15	: Butuan station	(Agusan River Baisin)
Packages 16-19	: Davao station	(Liboganon River Baisin)

TABLE 6.4-1 RAINFALL INTENSITY OF THREE PAGASA CLIMATOLOGICAL STATION

SURIGAO City

Return Period (year)	Rainfall Intensity duration Frequency (mm)														
	5 mins	10 mins	15 mins	20 mins	30 mins	45 mins	60 mins	80 mins	100 mins	120 mins	150 mins	3 hrs	6 hrs	12 hrs	24 hrs
2	15.9	23.8	30.3	35.9	45.7	56.3	63.5	72.9	81.6	88.8	98.6	107.4	142.9	177.9	206.3
5	25.7	35.2	44.2	51.6	65.3	79.2	88.8	102.2	115.5	126.8	142.8	157.8	214.6	273.1	314.1
10	32.3	42.8	53.4	61.9	78.3	94.4	105.5	121.5	137.9	152.0	172.0	191.2	262.0	336.2	385.4
15	36.0	47.1	58.6	67.8	85.6	102.9	114.9	132.5	150.5	166.2	188.5	210.0	288.8	371.8	425.7
20	38.5	50.1	62.2	71.9	90.7	108.9	121.5	140.1	159.4	176.1	200.1	223.2	307.6	396.7	453.9
25	40.5	52.4	65.0	75.1	94.7	113.6	126.6	146.0	166.2	183.8	209.0	233.4	322.0	415.9	475.6
50	46.6	59.5	73.7	84.8	106.9	127.8	142.2	164.2	187.2	207.4	236.4	264.7	366.5	475.0	542.4
100	52.7	66.5	82.2	94.5	118.9	141.9	157.8	182.2	208.0	230.8	263.6	295.8	410.7	533.6	608.8

Return Period (year)	Rainfall Intensity duration Frequency (mm/hr)														
	5 mins	10 mins	15 mins	20 mins	30 mins	45 mins	60 mins	80 mins	100 mins	120 mins	150 mins	3 hrs	6 hrs	12 hrs	24 hrs
2	190.8	142.8	121.2	107.7	91.4	75.1	63.5	54.7	49.0	44.4	39.4	35.8	23.8	14.8	8.6
5	308.4	211.2	176.8	154.8	130.6	105.5	88.8	76.7	69.3	63.4	57.1	52.6	35.8	22.8	13.1
10	387.6	256.8	213.6	185.7	156.6	125.9	105.5	91.1	82.7	76.0	68.8	63.7	43.7	28.0	16.1
15	432.0	282.6	234.4	203.4	171.2	137.2	114.9	99.4	90.3	83.1	75.4	70.0	48.1	31.0	17.7
20	462.0	300.6	248.8	215.7	181.4	145.2	121.5	105.1	95.6	88.1	80.0	74.4	51.3	33.1	18.9
25	486.0	314.4	260.0	225.3	189.4	151.5	126.6	109.5	99.7	91.9	83.6	77.8	53.7	34.7	19.8
50	559.2	357.0	294.8	254.4	213.8	170.4	142.2	123.2	112.3	103.7	94.6	88.2	61.1	39.6	22.6
100	632.4	399.0	328.8	283.5	237.8	189.2	157.8	136.6	124.8	115.4	105.4	98.6	68.5	44.5	25.4

BUTUAN CITY

Return Period (year)	Rainfall Intensity duration Frequency (mm)														
	5 mins	10 mins	15 mins	20 mins	30 mins	45 mins	60 mins	80 mins	100 mins	120 mins	150 mins	3 hrs	6 hrs	12 hrs	24 hrs
2	11.9	18.0	23.9	26.8	30.7	36.7	39.2	46.1	51.0	54.6	59.2	63.1	80.3	102.0	125.8
5	17.2	24.2	32.1	35.5	39.7	47.6	49.3	56.1	62.1	67.5	73.3	78.8	104.1	156.7	187.1
10	20.8	28.2	37.6	41.2	45.6	54.9	55.9	62.7	69.4	76.0	82.6	89.2	119.9	193.0	227.7
15	22.8	30.5	40.6	44.5	48.9	58.9	59.7	66.4	73.5	80.8	87.8	95.0	128.7	213.4	250.6
20	24.2	32.2	42.8	46.8	51.2	61.8	62.3	69.0	76.4	84.2	91.5	99.1	135.0	227.7	266.6
25	25.3	33.4	44.4	48.5	53.0	64.0	64.4	71.0	78.6	86.8	94.3	102.3	139.7	238.8	279.0
50	28.6	37.2	49.5	53.9	58.6	70.8	70.6	77.2	85.5	94.8	103.1	112.0	154.5	272.8	317.0
100	31.9	41.0	54.6	59.3	64.1	77.5	76.8	83.4	92.3	102.8	111.7	121.7	169.1	306.5	354.8

Return Period (year)	Rainfall Intensity duration Frequency (mm/hr)														
	5 mins	10 mins	15 mins	20 mins	30 mins	45 mins	60 mins	80 mins	100 mins	120 mins	150 mins	3 hrs	6 hrs	12 hrs	24 hrs
2	142.0	108.0	95.6	80.4	61.4	48.9	39.2	34.6	30.6	27.3	23.7	21.0	13.4	8.5	5.2
5	206.4	145.2	128.4	106.5	79.4	63.5	49.3	42.1	37.3	33.8	29.3	26.3	17.4	13.1	7.8
10	249.9	169.2	150.4	123.6	91.2	73.2	55.9	47.0	41.6	38.0	33.0	29.7	20.0	16.1	9.5
15	273.6	183.0	162.4	133.5	97.8	78.5	59.7	49.8	44.1	40.4	35.1	31.7	21.4	17.8	10.4
20	290.4	193.2	171.2	140.4	102.4	82.4	62.3	51.8	45.8	42.1	36.6	33.0	22.5	19.0	11.1
25	303.6	200.4	177.6	145.5	106.0	85.3	64.4	53.3	47.2	43.4	37.7	34.1	23.3	19.9	11.6
50	343.2	223.2	198.0	161.7	117.2	94.4	70.6	57.9	51.3	47.4	41.2	37.3	25.8	22.7	13.2
100	382.8	246.0	218.4	177.9	177.9	103.3	76.6	62.6	55.4	51.4	44.7	40.6	28.2	25.5	14.8

DAVAO CITY

Return Period (year)	Rainfall Intensity duration Frequency (mm)														
	5 mins	10 mins	15 mins	20 mins	30 mins	45 mins	60 mins	80 mins	100 mins	120 mins	150 mins	3 hrs	6 hrs	12 hrs	24 hrs
2	11.1	18.6	24.0	28.5	36.3	44.2	50.0	55.4	59.4	62.6	67.1	71.0	82.1	89.3	93.7
5	14.2	24.3	31.8	38.2	49.8	61.2	70.4	76.9	80.4	82.5	85.9	89.2	104.2	112.7	120.5
10	16.3	28.6	36.9	44.6	58.8	72.4	84.0	91.1	94.3	95.7	98.3	101.3	118.9	128.1	138.3
15	17.4	30.3	39.8	48.3	63.9	78.7	91.6	99.2	102.1	103.1	105.3	108.0	127.1	136.8	148.3
20	18.2	31.9	41.8	50.8	67.4	83.2	96.9	104.8	107.6	108.3	110.2	112.8	132.9	142.9	155.3
25	18.8	33.0	43.4	52.8	70.1	86.6	101.1	109.1	111.8	112.3	113.9	116.5	137.4	147.6	160.7
50	20.7	36.6	48.2	58.8	78.5	97.1	113.7	122.5	124.8	124.7	125.6	127.7	151.1	162.1	177.3
100	22.6	40.2	53.0	64.8	86.9	107.5	126.3	135.7	137.8	137.0	137.1	138.9	164.8	176.5	193.8

Return Period (year)	Rainfall Intensity duration Frequency (mm/hr)														
	5 mins	10 mins	15 mins	20 mins	30 mins	45 mins	60 mins	80 mins	100 mins	120 mins	150 mins	3 hrs	6 hrs	12 hrs	24 hrs
2	133.2	111.6	96.0	85.5	72.6	58.9	50.0	41.5	35.6	31.3	26.8	23.7	13.7	7.4	3.9
5	170.4	145.8	127.2	114.6	99.6	81.6	70.4	57.7	48.2	41.3	34.4	29.7	17.4	9.4	5.0
10	195.6	169.2	147.6	133.8	117.6	96.5	84.0	68.3	56.6	47.9	39.3	33.8	10.7	10.7	5.8
15	208.8	181.8	159.2	144.9	127.8	104.9	91.6	74.4	61.3	51.5	42.1	36.0	11.4	11.4	6.2
20	218.8	191.4	167.2	152.4	134.8	110.9	96.9	78.6	64.6	54.2	44.1	37.6	11.9	11.9	6.5
25	225.6	198.0	173.6	158.4	140.2	115.5	101.1	81.8	67.1	56.2	45.6	38.8	12.3	12.3	6.7
50	248.4	219.6	192.8	176.4	157.0	129.5	113.7	91.9	74.9	62.4	50.2	42.6	13.5	13.5	7.4
100	271.2	241.2	212.0	194.4	173.8	143.3	126.3	101.8	82.7	68.5	54.8	46.3	14.7	14.7	8.1

PROBABLE RAINFALL

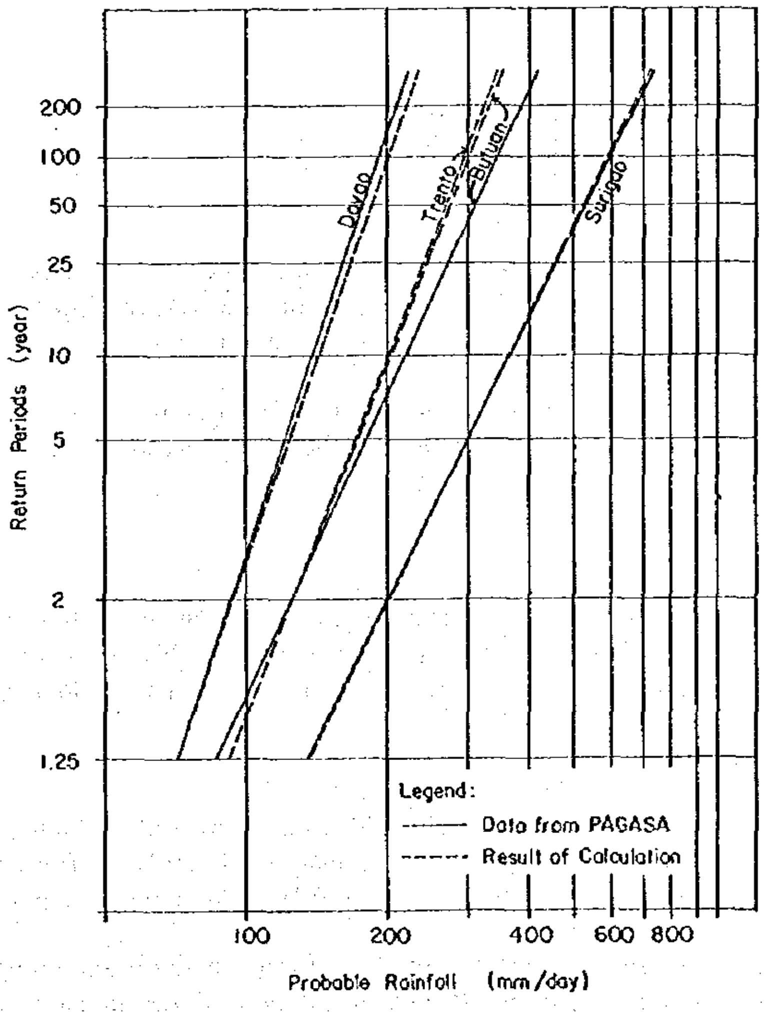


FIGURE 6.4-1 COMPARISON OF CALCULATED RESULT WITH PAGASA'S DATA

6.4.2 Equations and Curves to Express Rainfall Intensity-Duration-Frequency

The following equations are generally applied to express the relationship between rainfall intensity and duration:

(a) *Talbot Type Equation* : $I_P = a / (T_P + b)$

(b) Sherman Type Equation : $I_P = a / T_P^n$

(c) Kuno Type Equation : $I_P = a / (T_P^{0.5} + b)$

(d) Horner Type Equation : $I_P = a / (T_P + b)^n$

where,

I_P : Rainfall Intensity

T_P : Rainfall Duration

a,b,n: Constants

Constants of the rainfall intensity-duration-frequency equations were estimated by least-square regression analysis giving the relationship between probable rainfall intensities and corresponding rainfall duration.

In this study, two (2) types of rainfall intensity duration curves were developed to consider ranges of rainfall for the small catchment and large catchment areas.

Short duration curves

The curves were developed on the basis of probable rainfall intensities of which the duration is from 5 to 100 minutes and preferably used for small catchments where the run-off concentration time is less than 100 minutes.

Long duration curves

The curves of Butuan station were developed for time ranges of 100 minutes to a day which is the preferable value for run-off analysis of hourly hydrographies. There are several catchment area for which runoff lag time is over one hour.

Probable rainfall intensities were estimated from these equations and compared with the values estimated from the PAGASA's data as shown in Table 6.4-2. The equations for each station and each type are shown in Table 6.4-3.

The Sherman Type Equation which shows the smallest difference between input data and estimated data is the most applicable equation to express the rainfall intensity-duration.

Those curves were used to estimate the rainfall intensity and the probable model hyetgraphies for run-off analysis.

TABLE 6.4-2 (1/4) RAINFALL DEPTH - DURATION CURVES

(2) SURIGAO CLIMATOLOGICAL STATION

Return Period (Year)	Rainfall Duration (min)	Rainfall Intensities Estimated					Difference of Rainfall Intensities			
		(1) Rainfall Depth (mm/hr)	(2) Eq. of Talbot (mm/hr)	(3) Eq. of Sherman (mm/hr)	(4) Eq. of Kuno (mm/hr)	(5) Eq. of Horner (mm/hr)	(1)-(2) (mm/hr)	(1)-(3) (mm/hr)	(1)-(4) (mm/hr)	(1)-(5) (mm/hr)
2	15	30.3	29.2	30.0	28.8	30.9	1.1	0.3	1.5	0.6
	20	35.9	30.6	35.1	31.1	36.1	5.3	0.8	4.8	0.2
	30	45.7	34.1	43.8	36.0	44.7	11.6	1.9	9.7	1.0
	45	56.3	41.2	54.6	44.7	55.0	15.1	1.7	11.6	1.3
	60	63.5	51.9	63.9	56.1	63.6	11.6	0.4	7.4	0.1
	80	72.9	79.3	74.7	79.5	73.4	6.4	1.8	6.6	0.5
	100	81.6	168.6	84.4	125.7	82.0	87.0	2.8	44.1	0.4
	AVERAGE						17.0	1.2	10.7	0.6
5	15	44.2	43.0	44.5	42.5	44.8	1.2	0.3	1.7	0.6
	20	51.6	45.1	51.4	45.9	51.8	6.5	0.2	5.7	0.2
	30	65.3	50.0	63.2	52.8	63.5	15.3	2.1	12.5	1.8
	45	79.2	59.7	77.5	64.7	77.7	19.5	1.7	14.5	1.5
	60	88.8	74.1	89.7	79.9	89.6	14.7	0.9	8.9	0.8
	80	102.2	109.3	103.8	109.7	103.3	7.1	1.6	7.5	1.1
	100	115.5	207.8	116.2	163.3	115.3	92.3	0.7	47.8	0.2
	AVERAGE						19.6	1.0	12.5	0.9
10	15	53.4	52.2	54.1	51.7	54.0	1.2	0.7	1.7	0.6
	20	61.9	54.7	62.3	55.6	62.2	7.2	0.4	6.3	0.3
	30	78.3	60.6	76.0	63.8	75.9	17.7	2.3	14.5	2.4
	45	94.4	72.0	92.7	77.9	92.7	22.4	1.7	16.5	1.7
	60	105.5	88.9	106.8	95.6	106.8	16.6	1.3	9.9	1.3
	80	121.5	129.2	123.0	129.7	123.1	7.7	1.5	8.2	1.6
	100	137.9	236.3	137.2	189.1	137.4	98.4	0.7	51.2	0.5
	AVERAGE						21.5	1.2	13.8	1.2
15	15	58.6	57.4	59.3	56.8	59.2	1.2	0.9	1.8	0.6
	20	67.8	60.2	68.4	61.1	68.1	7.6	0.6	6.7	0.3
	30	85.6	66.5	83.2	70.1	82.9	19.1	2.4	15.5	2.7
	45	102.9	79.0	101.3	85.3	101.1	23.9	1.6	17.6	1.8
	60	114.9	97.2	116.4	104.4	116.5	17.7	1.5	10.5	1.6
	80	132.5	140.4	133.8	141.0	134.2	7.9	1.3	8.5	1.7
	100	150.5	252.9	149.1	203.8	149.8	102.4	1.4	53.3	0.7
	AVERAGE						22.7	1.4	14.5	1.3
20	15	62.2	61.0	63.2	60.4	62.8	1.2	1.0	1.8	0.6
	20	71.9	63.9	72.6	65.0	72.1	8.0	0.7	6.9	0.2
	30	90.7	70.6	88.2	74.4	87.8	20.1	2.5	16.3	2.9
	45	108.9	83.8	107.3	90.5	107.0	25.1	1.6	18.4	1.9
	60	121.5	103.0	123.2	110.6	123.3	18.5	1.7	10.9	1.8
	80	140.1	148.3	141.5	148.9	142.0	8.2	1.4	8.8	1.9
	100	159.4	265.1	157.5	214.4	158.5	105.7	1.9	55.0	0.9
	AVERAGE						23.6	1.5	15.1	1.4
25	15	65.0	63.8	66.1	63.2	65.7	1.2	1.1	1.8	0.7
	20	75.1	66.8	75.9	67.9	75.3	8.3	0.8	7.2	0.2
	30	94.7	73.8	92.2	77.8	91.6	20.9	2.5	16.9	3.1
	45	113.6	87.5	111.9	94.5	111.6	26.1	1.7	19.1	2.0
	60	126.6	107.5	128.4	115.4	128.5	19.1	1.8	11.2	1.9
	80	146.0	154.5	147.4	155.1	148.0	8.5	1.4	9.1	2.0
	100	166.2	274.5	164.0	222.5	165.2	108.3	2.2	56.3	1.0
	AVERAGE						24.3	1.7	15.6	1.5
50	15	73.7	72.4	75.1	71.7	74.3	1.3	1.4	2.0	0.6
	20	84.8	75.8	86.0	77.1	85.1	9.0	1.2	7.7	0.3
	30	106.9	83.7	104.2	88.1	103.3	23.2	2.7	18.8	3.6
	45	127.8	99.0	126.1	106.8	125.7	28.8	1.7	21.0	2.1
	60	142.2	121.3	144.5	130.2	144.6	20.9	2.3	12.0	2.4
	80	164.2	173.3	165.5	174.0	166.6	9.1	1.3	9.8	2.4
	100	187.2	303.1	183.8	247.5	185.9	115.9	3.4	60.3	1.3
	AVERAGE						26.4	2.0	16.9	1.8
100	15	82.2	81.0	84.0	80.2	82.9	1.2	1.8	2.0	0.7
	20	94.5	84.7	96.0	86.1	94.7	9.8	1.5	8.4	0.2
	30	118.9	93.4	116.0	98.4	114.8	25.5	2.9	20.5	4.1
	45	141.9	110.4	140.2	119.1	139.6	31.5	1.7	22.8	2.3
	60	157.8	135.0	160.4	144.7	160.6	22.8	2.6	13.1	2.8
	80	182.2	191.9	183.4	192.8	184.9	9.7	1.2	10.6	2.7
	100	208.0	331.9	203.5	272.3	206.4	123.9	4.5	64.3	1.6
	AVERAGE						28.6	2.4	18.2	2.0

TABLE 6.4-2 (2/4) RAINFALL DEPTH - DURATION CURVES

(2) BUTUAN CLIMATOLOGICAL STATION

Return Period (Year)	Rainfall Duration (min)	Rainfall Intensities Estimated (1) Rainfall Depth (mm/hr)	(2) Eq. of Talbot (mm/hr)	(3) Eq. of Sherman (mm/hr)	(4) Eq. of Kuno (mm/hr)	(5) Eq. of Horner (mm/hr)	Difference of Rainfall Intensities (1)-(2) (mm/hr)	(1)-(3) (mm/hr)	(1)-(4) (mm/hr)	(1)-(5) (mm/hr)
2	15	23.9	26.6	23.7	25.6	24.9	2.7	0.2	1.7	1.0
	20	26.8	27.5	26.5	27.0	26.6	0.7	0.3	0.2	0.2
	30	30.7	29.4	31.1	29.8	30.0	1.3	0.4	0.9	0.7
	45	36.7	32.8	36.5	34.0	34.9	3.9	0.2	2.7	1.8
	60	39.2	37.2	40.8	38.5	39.8	2.0	1.6	0.7	0.6
	80	46.1	45.1	45.7	45.6	46.0	1.0	0.4	0.5	0.1
	100	51.0	57.5	49.9	54.5	52.1	6.5	1.1	3.5	1.1
	AVERAGE						2.6	0.6	1.5	0.8
5	15	32.1	35.1	32.0	34.0	33.5	3.0	0.1	1.9	1.4
	20	35.5	36.2	35.2	35.7	35.4	0.7	0.3	0.2	0.1
	30	39.7	38.4	40.4	38.9	39.1	1.3	0.7	0.8	0.6
	45	47.6	42.4	46.4	43.7	44.6	5.2	1.2	3.9	3.0
	60	49.3	47.3	51.1	48.8	49.8	2.0	1.8	0.5	0.5
	80	56.1	56.0	56.4	56.4	56.6	0.1	0.3	0.3	0.5
	100	62.1	68.6	60.8	65.4	63.2	6.5	1.3	3.3	1.1
	AVERAGE						2.7	0.8	1.6	1.0
10	15	37.6	40.7	37.4	39.6	39.2	3.1	0.2	2.0	1.6
	20	41.2	41.9	41.0	41.4	41.2	0.7	0.2	0.2	0.0
	30	45.6	44.4	46.6	44.9	45.2	1.2	1.0	0.7	0.4
	45	54.9	48.7	52.9	50.2	50.9	6.2	2.0	4.7	4.0
	60	55.9	54.0	57.9	55.6	56.4	1.9	2.0	0.3	0.5
	80	62.7	63.2	63.4	63.6	63.6	0.5	0.7	0.9	0.9
	100	69.4	76.1	68.0	72.8	70.6	6.7	1.4	3.4	1.2
	AVERAGE						2.9	1.1	1.7	1.3
15	15	40.6	43.9	40.5	42.7	42.4	3.3	0.1	2.1	1.8
	20	44.5	45.1	44.2	44.6	44.5	0.6	0.3	0.1	0.0
	30	48.9	47.7	50.0	48.3	48.6	1.2	1.1	0.6	0.3
	45	58.9	52.3	56.5	53.7	54.5	6.6	2.4	5.2	4.4
	60	59.7	57.8	61.7	59.4	60.2	1.9	2.0	0.3	0.5
	80	66.4	67.2	67.3	67.6	67.6	0.8	0.9	1.2	1.2
	100	73.5	80.4	72.0	77.0	74.8	6.9	1.5	3.5	1.3
	AVERAGE						1.9	2.3	5.4	2.1
20	15	42.8	46.1	42.7	44.9	44.7	3.3	0.1	2.1	1.9
	20	46.8	47.4	46.5	46.9	46.8	0.6	0.3	0.1	0.0
	30	51.2	50.1	52.4	50.7	50.9	1.1	1.2	0.5	0.3
	45	61.8	54.8	59.1	56.3	56.9	7.0	2.7	5.5	4.9
	60	62.3	60.4	64.4	62.1	62.8	1.9	2.1	0.2	0.5
	80	69.0	70.1	70.1	70.4	70.4	1.1	1.1	1.4	1.4
	100	76.4	83.4	74.9	79.9	77.7	7.0	1.5	3.5	1.3
	AVERAGE						3.1	1.3	1.9	1.5
25	15	44.4	47.8	44.3	46.5	46.4	3.4	0.1	2.1	2.0
	20	48.5	49.1	48.2	48.6	48.5	0.6	0.3	0.1	0.0
	30	53.0	51.8	54.3	52.5	52.8	1.2	1.3	0.5	0.2
	45	64.0	56.6	61.1	58.2	58.9	7.4	2.9	5.8	5.1
	60	64.4	62.4	66.4	64.1	64.9	2.0	2.0	0.3	0.5
	80	71.0	72.3	72.3	72.6	72.5	1.3	1.3	1.6	1.5
	100	78.6	85.8	77.1	82.2	79.9	7.2	1.5	3.6	1.3
	AVERAGE						3.3	1.3	2.0	1.5
50	15	49.5	53.1	49.5	51.7	51.8	3.6	0.0	2.2	2.3
	20	53.9	54.4	53.6	53.9	54.0	0.5	0.3	0.0	0.1
	30	58.6	57.4	60.0	58.1	58.4	1.2	1.4	0.5	0.2
	45	70.8	62.5	67.2	64.2	64.8	8.3	3.6	6.6	6.0
	60	70.6	68.7	72.8	70.5	71.1	1.9	2.2	0.1	0.5
	80	77.2	79.0	78.9	79.4	79.1	1.8	1.7	2.2	1.9
	100	85.5	93.0	83.9	89.3	86.9	7.5	1.6	3.8	1.4
	AVERAGE						3.5	1.5	2.2	1.8
100	15	54.6	58.3	54.7	56.9	57.1	3.7	0.1	2.3	2.5
	20	59.3	59.8	59.0	59.2	59.5	0.5	0.3	0.1	0.2
	30	64.1	62.9	65.7	63.7	64.1	1.2	1.6	0.4	0.0
	45	77.5	68.4	73.3	70.2	70.7	9.1	4.2	7.3	6.8
	60	76.8	74.9	79.1	76.7	77.2	1.9	2.3	0.1	0.4
	80	83.4	85.7	85.4	86.1	85.6	2.3	2.0	2.7	2.2
	100	92.3	100.2	90.6	96.4	93.8	7.9	1.7	4.1	1.5
	AVERAGE						3.8	1.7	2.4	1.9

TABLE 6.4-2 (3/4) RAINFALL DEPTH - DURATION CURVES

(2) BUTUAN CLIMATOLOGICAL STATION

Return Period (Year)	Rainfall Duration (min)	Rainfall Intensities Estimated (1) Rainfall Depth (mm/hr)	(2) Eq. of Talbot (mm/hr)	(3) Eq. of Sherman (mm/hr)	(4) Eq. of Kuno (mm/hr)	(5) Eq. of Horner (mm/hr)	Difference of Rainfall Intensities (1)-(2) (mm/hr)	(1)-(3) (mm/hr)	(1)-(4) (mm/hr)	(1)-(5) (mm/hr)
2	100	51.0	55.7	51.2	54.3	51.1	4.7	0.2	3.3	0.1
	120	54.6	56.7	54.6	55.9	54.6	2.1	0.0	1.3	0.0
	150	59.2	58.1	59.0	58.2	59.1	1.1	0.2	1.0	0.1
	180	63.1	59.6	62.9	60.5	63.1	3.5	0.2	2.6	0.0
	360	80.3	70.7	80.2	74.0	80.4	9.6	0.1	6.3	0.1
	720	102.0	112.4	102.2	108.5	102.0	10.4	0.2	6.5	0.0
	AVERAGE						5.2	0.1	3.5	0.0
5	100	62.1	68.9	61.1	66.6	63.4	6.8	1.0	4.5	1.3
	120	67.5	70.2	66.4	68.9	67.1	2.7	1.1	1.4	0.4
	150	73.3	72.3	73.5	72.3	72.4	1.0	0.2	1.0	0.9
	180	78.8	74.6	79.9	75.7	77.6	4.2	1.1	3.1	1.2
	360	104.1	91.6	109.8	97.2	106.2	12.5	5.7	6.9	2.1
	720	156.7	168.5	150.8	163.1	155.8	11.8	5.9	6.4	0.9
	AVERAGE						6.5	2.5	3.9	1.1
10	100	69.4	77.5	67.7	74.7	71.1	8.1	1.7	5.3	1.7
	120	76.0	79.1	74.3	77.4	75.3	3.1	1.7	1.4	0.7
	150	82.6	81.6	83.1	81.5	81.5	1.0	0.5	1.1	1.1
	180	89.2	84.3	91.1	85.6	87.7	4.9	1.9	3.6	1.5
	360	119.9	105.1	129.1	112.2	123.5	14.8	9.2	7.7	3.6
	720	193.0	206.7	183.0	200.1	191.2	13.7	10.0	7.1	1.8
	AVERAGE						7.6	4.2	4.4	1.7
15	100	73.5	82.4	71.5	79.3	75.4	8.9	2.0	5.8	1.9
	120	80.8	84.1	78.7	82.2	79.9	3.3	2.1	1.4	0.9
	150	87.8	86.9	88.4	86.7	86.6	0.9	0.6	1.1	1.2
	180	95.0	89.8	97.3	91.1	93.3	5.2	2.3	3.9	1.7
	360	128.7	112.5	139.8	120.4	133.1	16.2	11.1	8.3	4.4
	720	213.4	228.1	201.0	220.9	211.0	14.7	12.4	7.5	2.4
	AVERAGE						8.2	5.1	4.7	2.1
20	100	76.4	85.9	74.2	82.5	78.4	9.5	2.2	6.1	2.0
	120	84.2	87.7	81.8	85.7	83.1	3.5	2.4	1.5	1.1
	150	91.5	90.6	92.2	90.3	90.2	0.9	0.7	1.2	1.3
	180	99.1	93.7	101.7	95.0	97.3	5.4	2.6	4.1	1.8
	360	135.0	117.8	147.4	126.2	139.9	17.2	12.4	8.8	4.9
	720	227.7	243.3	213.7	235.6	224.9	15.6	14.0	7.9	2.8
	AVERAGE						8.7	5.7	4.9	2.3
25	100	78.6	88.5	76.3	85.0	80.7	9.9	2.3	6.4	2.1
	120	86.8	90.4	84.2	88.3	85.6	3.6	2.6	1.5	1.2
	150	94.3	93.4	95.1	93.1	93.0	0.9	0.8	1.2	1.3
	180	102.3	96.6	105.0	98.0	100.4	5.7	2.7	4.3	1.9
	360	139.7	121.8	153.2	130.7	145.1	17.9	13.5	9.0	5.4
	720	238.8	255.0	223.4	247.0	235.6	16.2	15.4	8.2	3.2
	AVERAGE						9.0	6.2	5.1	2.5
50	100	85.5	96.7	82.7	92.7	87.8	11.2	2.8	7.2	2.3
	120	94.8	98.8	91.7	96.4	93.3	4.0	3.1	1.6	1.5
	150	103.1	102.2	104.1	101.8	101.6	0.9	1.0	1.3	1.5
	180	112.0	105.8	115.4	107.3	110.0	6.2	3.4	4.7	2.0
	360	154.5	134.3	171.0	144.4	161.3	20.2	16.5	10.1	6.8
	720	272.8	291.0	253.4	281.9	268.6	18.2	19.4	9.1	4.2
	AVERAGE						10.1	7.7	5.7	3.1
100	100	92.3	104.8	89.0	100.4	94.9	12.5	3.3	8.1	2.6
	120	102.8	107.1	99.1	104.4	101.0	4.3	3.7	1.6	1.8
	150	111.7	110.8	112.9	110.4	110.1	0.9	1.2	1.3	1.6
	180	121.7	114.8	125.7	116.5	119.4	6.9	4.0	5.2	2.3
	360	169.1	146.5	188.6	157.8	177.2	22.6	19.5	11.3	8.1
	720	306.5	326.8	283.1	316.6	301.2	20.3	23.4	10.1	5.3
	AVERAGE						11.2	9.2	6.3	3.6

TABLE 6.4-2 (4/4) RAINFALL DEPTH - DURATION CURVES

(3) DAVAO CLIMATOLOGICAL STATION

Return Period (Year)	Rainfall Duration (min)	Rainfall Intensities Estimated: (1) Rainfall Depth (mm/hr)	(2) Eq. of Talbot (mm/hr)	(3) Eq. of Sherman (mm/hr)	(4) Eq. of Kuno (mm/hr)	(5) Eq. of Horner (mm/hr)	Difference of Rainfall Intensities: (1)-(2) (mm/hr)	(1)-(3) (mm/hr)	(1)-(4) (mm/hr)	(1)-(5) (mm/hr)
2	15	24.0	28.7	25.0	27.6	23.6	4.7	1.0	3.6	0.4
	20	28.5	29.8	28.7	29.4	29.2	1.3	0.2	0.9	0.7
	30	36.3	32.3	34.9	32.9	36.5	4.0	1.4	3.4	0.2
	45	44.2	37.0	42.4	38.6	43.9	7.2	1.8	5.6	0.3
	60	50.0	43.3	48.7	45.2	49.4	6.7	1.3	4.8	0.6
	80	55.4	56.0	55.9	56.4	55.2	0.6	0.5	1.0	0.2
	100	59.4	79.2	62.3	72.1	60.0	19.8	2.9	12.7	0.6
	AVERAGE						6.3	1.3	4.6	0.4
5	15	31.8	38.4	33.5	37.0	-	6.6	1.7	5.2	-
	20	38.2	40.0	38.7	39.4	-	1.8	0.5	1.2	-
	30	49.8	43.6	47.4	44.4	-	6.2	2.4	5.4	-
	45	61.2	50.3	58.1	52.6	-	10.9	3.1	8.6	-
	60	70.4	59.5	67.1	62.2	-	10.9	3.3	8.2	-
	80	76.9	78.5	77.5	78.9	-	1.6	0.6	2.0	-
	100	80.4	115.7	86.6	103.3	-	35.3	6.2	22.9	-
	AVERAGE						10.5	2.5	7.6	
10	15	36.9	44.8	39.1	43.1	-	7.9	2.2	6.2	-
	20	44.6	46.7	45.3	46.0	-	2.1	0.7	1.4	-
	30	58.8	51.0	55.7	52.0	-	7.8	3.1	6.8	-
	45	72.4	59.0	68.5	61.8	-	13.4	3.9	10.6	-
	60	84.0	70.1	79.3	73.4	-	13.9	4.7	10.6	-
	80	91.1	93.6	91.7	93.9	-	2.5	0.6	2.8	-
	100	94.3	140.7	102.8	124.4	-	46.4	8.5	30.1	-
	AVERAGE						13.4	3.4	9.8	
15	15	39.8	48.3	42.4	46.6	-	8.7	2.6	6.8	-
	20	48.3	50.5	49.1	49.8	-	2.2	0.8	1.5	-
	30	63.9	55.2	60.4	56.3	-	8.7	3.5	7.6	-
	45	78.7	64.0	74.3	67.0	-	14.7	4.4	11.7	-
	60	91.6	76.2	86.1	79.8	-	15.4	5.5	11.8	-
	80	99.2	102.1	99.8	102.3	-	2.9	0.6	3.1	-
	100	102.1	154.8	111.9	136.3	-	52.7	9.8	34.2	-
	AVERAGE						15.1	3.9	11.0	
20	15	41.8	51.0	44.6	49.0	-	9.2	2.8	7.2	-
	20	50.8	53.1	51.7	52.4	-	2.3	0.9	1.6	-
	30	67.4	58.0	63.6	59.3	-	9.4	3.8	8.1	-
	45	83.2	67.4	78.4	70.6	-	15.8	4.8	12.6	-
	60	96.9	80.4	90.9	84.2	-	16.5	6.0	12.7	-
	80	104.8	108.1	105.4	108.3	-	3.3	0.6	3.5	-
	100	107.6	164.9	118.2	144.8	-	57.3	10.6	37.2	-
	AVERAGE						16.3	4.2	11.8	
25	15	43.4	52.9	46.3	50.9	-	9.5	2.9	7.5	-
	20	52.8	55.2	53.7	54.4	-	2.4	0.9	1.6	-
	30	70.1	60.3	66.2	61.6	-	9.8	3.9	8.5	-
	45	86.6	70.1	81.6	73.4	-	16.5	5.0	13.2	-
	60	101.1	83.6	94.6	87.6	-	17.5	6.5	13.5	-
	80	109.1	112.7	109.7	112.9	-	3.6	0.6	3.8	-
	100	111.8	172.6	123.1	151.2	-	60.8	11.3	39.4	-
	AVERAGE						17.2	4.5	12.5	
50	15	48.2	58.9	51.6	56.6	-	10.7	3.4	8.4	-
	20	58.8	61.4	59.9	60.6	-	2.6	1.1	1.8	-
	30	78.5	67.2	73.9	68.7	-	11.3	4.6	9.8	-
	45	97.1	78.2	91.3	82.0	-	18.9	5.8	15.1	-
	60	113.7	93.6	106.0	98.2	-	20.1	7.7	15.5	-
	80	122.5	126.8	123.1	127.0	-	4.3	0.6	4.5	-
	100	124.8	196.5	138.2	171.2	-	71.7	13.4	46.4	-
	AVERAGE						19.9	5.2	14.5	
100	15	53.0	64.9	56.8	62.4	-	11.9	3.8	9.4	-
	20	64.8	67.7	66.0	66.8	-	2.9	1.2	2.0	-
	30	86.9	74.1	81.6	75.7	-	12.8	5.3	11.2	-
	45	107.5	86.4	100.9	90.6	-	21.1	6.6	16.9	-
	60	126.3	103.6	117.3	108.6	-	22.7	9.0	17.7	-
	80	135.7	140.9	136.4	141.0	-	5.2	0.7	5.3	-
	100	137.8	220.2	153.2	191.0	-	82.4	15.4	53.2	-
	AVERAGE						22.7	6.0	16.5	

TABLE 6.4-3 EQUATION OF RAINFALL DURATION CURVES

Simurao short duration

R.P.	TYPE 1	TYPE 2	TYPE 3	TYPE 4
2	$I = -2996.0/(T - 117.8)$	$I = 6.84/T^{-0.55}$	$I = -228.6/(\sqrt{T} - 11.82)$	$I = 8.76/(T - 1.76)^{0.49}$
5	$I = -4611.8/(T - 122.2)$	$I = 11.29/T^{-0.51}$	$I = -352.4/(\sqrt{T} - 12.16)$	$I = 2.08/(T - 0.54)^{0.43}$
10	$I = -5698.6/(T - 124.1)$	$I = 14.31/T^{-0.49}$	$I = -435.7/(\sqrt{T} - 12.30)$	$I = 4.14/(T + 0.10)^{0.49}$
15	$I = -6314.8/(T - 125.0)$	$I = 16.02/T^{-0.48}$	$I = -482.9/(\sqrt{T} - 12.37)$	$I = 5.30/(T + 0.39)^{0.49}$
20	$I = -6738.2/(T - 125.4)$	$I = 17.18/T^{-0.48}$	$I = -515.3/(\sqrt{T} - 12.40)$	$I = 6.13/(T + 0.55)^{0.50}$
25	$I = -7069.2/(T - 125.8)$	$I = 18.10/T^{-0.48}$	$I = -540.7/(\sqrt{T} - 12.43)$	$I = 6.78/(T + 0.66)^{0.50}$
50	$I = -8090.6/(T - 126.7)$	$I = 20.93/T^{-0.47}$	$I = -618.9/(\sqrt{T} - 12.50)$	$I = 8.69/(T + 1.01)^{0.50}$
100	$I = -9101.8/(T - 127.4)$	$I = 23.74/T^{-0.47}$	$I = -696.4/(\sqrt{T} - 12.56)$	$I = 0.52/(T + 1.32)^{0.50}$

Butuan short duration

R.P.	TYPE 1	TYPE 2	TYPE 3	TYPE 4
2	$I = -4210.0/(T - 173.3)$	$I = 8.20/T^{-0.39}$	$I = -296.8/(\sqrt{T} - 15.45)$	$I = 0.78/(T + 45.81)^{0.84}$
5	$I = -6116.9/(T - 189.2)$	$I = 12.76/T^{-0.34}$	$I = -434.0/(\sqrt{T} - 16.63)$	$I = 1.13/(T + 54.81)^{0.80}$
10	$I = -7453.5/(T - 198.0)$	$I = 15.98/T^{-0.31}$	$I = -530.6/(\sqrt{T} - 17.29)$	$I = 1.26/(T + 62.22)^{0.79}$
15	$I = -8219.7/(T - 202.3)$	$I = 17.82/T^{-0.30}$	$I = -586.2/(\sqrt{T} - 17.61)$	$I = 1.46/(T + 63.55)^{0.77}$
20	$I = -8777.7/(T - 205.3)$	$I = 19.17/T^{-0.30}$	$I = -626.6/(\sqrt{T} - 17.84)$	$I = 1.37/(T + 68.66)^{0.79}$
25	$I = -9176.7/(T - 207.0)$	$I = 20.13/T^{-0.29}$	$I = -655.8/(\sqrt{T} - 17.97)$	$I = 1.77/(T + 64.23)^{0.75}$
50	$I = 10497.6/(T - 212.9)$	$I = 23.32/T^{-0.28}$	$I = -751.8/(\sqrt{T} - 18.42)$	$I = 1.72/(T + 72.38)^{0.76}$
100	$I = 11845.0/(T - 218.2)$	$I = 26.55/T^{-0.27}$	$I = -850.0/(\sqrt{T} - 18.82)$	$I = 1.84/(T + 77.27)^{0.76}$

Butuan long duration

R.P.	TYPE 1	TYPE 2	TYPE 3	TYPE 4
2	$I = 68528.4/(T - 1329.6)$	$I = 10.22/T^{-0.35}$	$I = -1832.2/(\sqrt{T} - 43.72)$	$I = 1.09/(T - 8.16)^{0.34}$
5	$I = 72215.2/(T - 1148.5)$	$I = 7.41/T^{-0.46}$	$I = -1893.2/(\sqrt{T} - 38.44)$	$I = 1.28/(T + 143.42)^{0.71}$
10	$I = 76938.9/(T - 1092.3)$	$I = 6.67/T^{-0.50}$	$I = -2006.0/(\sqrt{T} - 36.86)$	$I = 0.48/(T + 196.69)^{0.88}$
15	$I = 79970.4/(T - 1070.6)$	$I = 6.42/T^{-0.52}$	$I = -2080.5/(\sqrt{T} - 36.25)$	$I = 0.31/(T + 218.00)^{0.95}$
20	$I = 82249.5/(T - 1058.1)$	$I = 6.30/T^{-0.54}$	$I = -2137.2/(\sqrt{T} - 35.90)$	$I = 0.25/(T + 228.72)^{0.99}$
25	$I = 84013.1/(T - 1049.5)$	$I = 6.21/T^{-0.54}$	$I = -2181.1/(\sqrt{T} - 35.66)$	$I = 0.20/(T + 237.35)^{1.03}$
50	$I = 89736.8/(T - 1028.3)$	$I = 6.06/T^{-0.57}$	$I = -2324.8/(\sqrt{T} - 35.08)$	$I = 0.13/(T + 256.39)^{1.11}$
100	$I = 95609.7/(T - 1012.5)$	$I = 5.99/T^{-0.59}$	$I = -2473.0/(\sqrt{T} - 34.64)$	$I = 0.09/(T + 270.06)^{1.17}$

Davao short duration

R.P.	TYPE 1	TYPE 2	TYPE 3	TYPE 4
2	$I = -3825.4/(T - 148.3)$	$I = 6.80/T^{-0.43}$	$I = -273.7/(\sqrt{T} - 13.80)$	$I = 3.18/(T - 9.34)^{0.34}$
5	$I = -4893.5/(T - 142.3)$	$I = 8.66/T^{-0.50}$	$I = -352.5/(\sqrt{T} - 13.41)$	-
10	$I = -5591.1/(T - 139.7)$	$I = 9.87/T^{-0.51}$	$I = -403.9/(\sqrt{T} - 13.25)$	-
15	$I = -5997.0/(T - 138.7)$	$I = 10.59/T^{-0.51}$	$I = -433.8/(\sqrt{T} - 13.18)$	-
20	$I = -6269.3/(T - 138.0)$	$I = 11.06/T^{-0.51}$	$I = -453.8/(\sqrt{T} - 13.14)$	-
25	$I = -6489.2/(T - 137.6)$	$I = 11.46/T^{-0.52}$	$I = -470.1/(\sqrt{T} - 13.11)$	-
50	$I = -7150.6/(T - 136.4)$	$I = 12.61/T^{-0.52}$	$I = -518.7/(\sqrt{T} - 13.03)$	-
100	$I = -7818.0/(T - 135.5)$	$I = 13.78/T^{-0.52}$	$I = -567.6/(\sqrt{T} - 12.97)$	-

6.5 RUN-OFF ANALYSIS

Flood run-off analysis is attained to provide the standard design discharge for objective catchment area.

The design storm frequencies considered desirable for use in the Philippines are:

Item	Design storm frequencies
Bridges	50-year return period (1 in 50 years)
Box Culverts	25-year return period (1 in 25 years)
Pipe Culverts	10-year return period (1 in 10 years)

6.5.1 Procedure for Run-off Analysis

The run-off analysis was made in accordance with the following procedure:

(a) The area, river length, maximam and lower ground height of each catchment area was obtained from a topographic map with a scale of 1/50,000.

(b) Run-off analysis was performed using the rational method and the unit hydrograph. For the rational method, the normally accepted limitation of a catchment area is 25 km^2 . A catchment area larger than 25 km^2 was analyzed with the unit hydrograph. Number of catchment area smaller than 25km^2 was 401. 25 catchment areas were analyzed by the unit hydrograph.

6.5.2 *Rational Method*

The Rational Method is based on the direct relationship between rainfall and run-off. The formula in metric units is:

$$Q = 0.278\ CIA \qquad (1)$$

where,

Q : Discharge (m^3/s)
C : Runoff Coefficient
I : Rainfall Intensity (mm/hr)
A : Drainage Area (km^2)

Time of concentration is lag time:

$$Tc = L^{1.5} / 51\ H^{0.385} \qquad (2)$$

where,

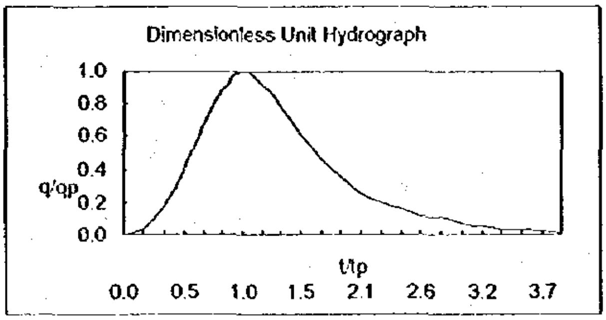
Tc: Time of concentration (min)
L : Length of catchment area along the mainstream
H : Difference in elevation betwwn the most distant ridge in the catchment area and point under review (m)

Discharge is calculated for the two cases of a 10-year and 25-year return period. Result of the calculation is shown in Appendix 6.5-1.

6.5.3 Unit Hydrograph Method

1) Unit hydrograph

A dimensionless unit hydrograph shown in the following figure shows the effect of basin size and essentially eliminates the effect of shape, expect where they are reflected in the estimate of basin lag t_p and runoff volume.



A general expression for basin lag might be expected to take the form.

$$t_p = C_t\,(LLc / S^{0.5})^{0.38} \qquad (2)$$

where

t_p:	Lag time (sec)
C_t:	Coefficient (varied from 1.4 to1.7)
L:	Main Stream distance from outlet to divide (km)
Lc:	Stream distance from outlet to a point opposit the basin centroid
S:	Average Channel Slope

2) Model Hyetograph

Model hyetograph of each catchment area was created by the method of Soil Conservation Service as shown in Table 6.5-1.

$$Q = \frac{(P - Ia)^2}{P - Ia + S}$$

$$F = P - Ia - Q$$

there

Q:	Cumulative Runoff(mm)
P:	Cumulative Rainfall(mm)
F:	Cumulative Infiltration(mm)
Ia:	Initial Abstruction Ia =0.2 S
S:	Potential Maximum Abstruction S = 25,400/CN - 2,540
CN:	Curve Number

TABLE 6.5-1 MODEL HYETOGRAPH BY METHOD OF SOIL CONSERVATION SERVICE

SCS RAINFALL EXCESS　　Location : Simulao River

Unit Duration (D)	4.315	Regional Factor (RF)	1.1	Potential Max. Abstruction (S)	48.38
		Curve Number (CN)	84	Inital Abstruction (Ia)	9.68
		$I = 6.21/T^{-0.59}$		Butuan 100-year R.P. (Long Term)	

Ct	L (km)	Lc(km)	S	Lg (hr)	D (hr)	A (km2)	Qp
0.83	52	24	0.032	23.96962	4.314531	456	3.973284

No.	Rainfall Duration (min.)	Rainfall Intensity (mm/hr)	Rainfall Increment (mm/hr)	Rainfall "RF "D (mm)	Model Hyet R (mm)	Cumulative Rainfall P (mm)	Cumulative Discharge Q (mm)	Cumulative Infiltration F (mm)	Infiltration F'(mm)	Rainfall Excess R-F' (mm)
1	259	36.83	36.83	174.80	0.89	0.89	0.00	0.00	0.00	0.00
2	518	27.72	9.11	43.24	1.01	1.90	0.00	0.00	0.00	0.00
3	777	23.47	4.25	20.15	1.17	3.07	0.00	0.00	0.00	0.00
4	1035	20.86	2.61	12.40	1.38	4.45	0.00	0.00	0.00	0.00
5	1294	19.04	1.82	8.66	1.65	6.10	0.00	0.00	0.00	0.00
6	1553	17.67	1.37	6.51	2.04	8.14	0.00	0.00	0.00	0.00
7	1812	16.59	1.08	5.14	2.61	10.75	0.02	1.05	1.05	1.56
8	2071	15.70	0.88	4.19	3.51	14.26	0.40	4.19	3.14	0.37
9	2330	14.96	0.74	3.51	5.14	19.40	1.63	8.10	3.91	1.23
10	2589	14.33	0.63	3.00	8.66	28.06	5.06	13.32	5.23	3.43
11	2848	13.78	0.55	2.61	20.15	48.21	17.08	21.45	8.13	12.02
12	3106	13.30	0.48	2.29	174.80	223.01	173.90	39.44	17.99	156.81
13	3365	12.87	0.43	2.04	43.24	266.25	215.87	40.71	1.27	41.97
14	3624	12.48	0.39	1.83	12.40	278.65	227.97	41.01	0.30	12.10
15	3883	12.13	0.35	1.65	6.51	285.16	234.33	41.15	0.15	6.36
16	4142	11.82	0.32	1.50	4.19	289.35	238.43	41.25	0.09	4.10
17	4401	11.53	0.29	1.38	3.00	292.35	241.36	41.31	0.06	2.94
18	4660	11.26	0.27	1.27	2.29	294.64	243.60	41.36	0.05	2.24
19	4919	11.01	0.25	1.17	1.83	296.47	245.40	41.40	0.04	1.79
20	5177	10.78	0.23	1.09	1.50	297.97	246.87	41.43	0.03	1.47
21	5436	10.57	0.21	1.01	1.27	299.24	248.11	41.45	0.03	1.24
22	5695	10.37	0.20	0.95	1.09	300.33	249.18	41.48	0.02	1.07
23	5954	10.18	0.19	0.89	0.95	301.28	250.11	41.50	0.02	0.93
24	6213	10.01	0.18	0.84	0.84	302.12	250.93	41.51	0.02	0.82

3) Base Flow

Base flow was estimated $0.07m^3/s/km^2$ from the maximum monthly average discharge data of Simulao River. Base flow was added to the peak discharge for each river.

4) Approvement of model

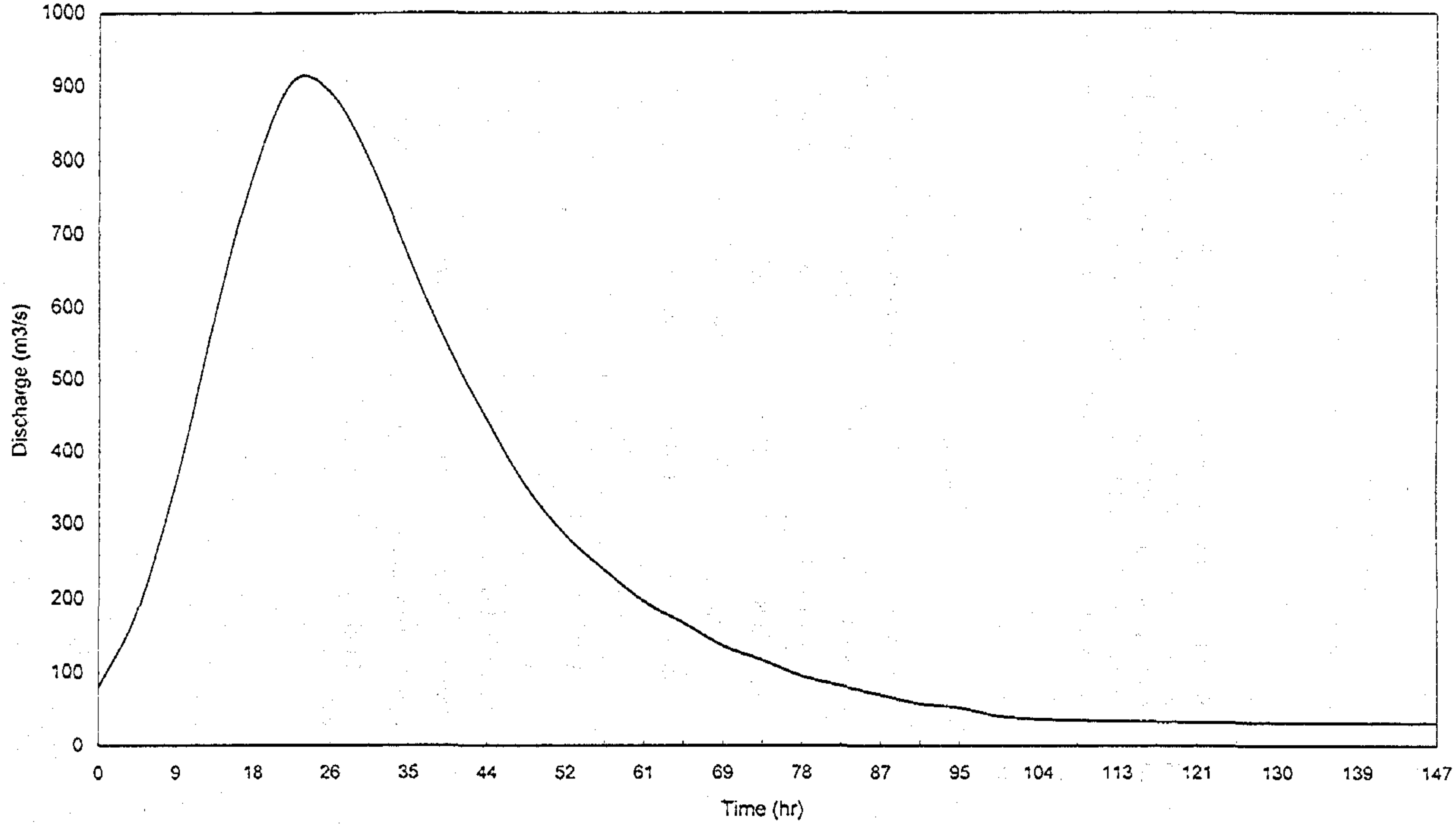
The result of the calculation was further verified on the peak discharge of the NIA weir point in Simulao River. The design storm frequency of weir is for a 100-year return period and $890m^3/s$ is the design storm of Simulao Weir. The calculated hydrograph at NIA weir is shown in Fig 6.5-1 and the peak discharge was $907m^3/s$ which is a similar number to the design storm of the NIA weir.

5) Result

Peak Discharge of each point was calculated as follows:

Location	C/A	Return Period	Peak Discharge
Legaspi River I	$12.7km^2$	50-year	$182m^3/s$
Legaspi River II	$22.5km^2$	50-year	$253m^3/s$
Baliguian Bridge	$14.1km^2$	50-year	$99m^3/s$
Jagupit Bridge	$2.5km^2$	50-year	$28m^3/s$
Guinoyoran	$4.7km^2$	50-year	$51m^3/s$
Sta. Ana	$49.6km^2$	50-year	$388m^3/s$
1200km	$4.7km^2$	25-year	$40m^3/s$
1220km	$3.6km^2$	25-year	$25m^3/s$
Andanan Bridge	$225km^2$	50-year	$417m^3/s$
Simulao River	$696km^2$	25-year	$1034m^3/s$
Simulao (NIA weir)	$456km^2$	100-year	$908m^3/s$
1435km	$31.0km^2$	25-year	$24m^3/s$

FIGURE 6.5-1 HYDROGRAPH AT NIA WEIR IN SIMULAO RIVER



6.6 NON-UNIFORM FLOW ANALYSIS

6.6.1 High Water Level of Rivers

To determine the High Water Level (H.W.L.) at notable points such as the reconstruction bridge sites and flooded portion of river, calculation of non-uniform flow analysis was carried out for seven rivers. The cross section drawings of six rivers, except Libuganon River, was made by this Study Team. The cross sections drawing of Libuganon River was obtained from NIA.

The rivers subjected to this analysis and calculated distance were as follows (the location maps of cross-sections are presented in Appendix 6.6-1):

River Name	Calculated Distance and No. of Cross Section
Legaspi River	3.9 km (14 sections)
Baliguian Creek	1.0 km (6 sections)
Guinoyoran Creek	1.0 km (6 sections)
Andanan River	1.5 km (6 sections)
Simulao River	33.0 km (46 sections)
Agusan River	11.0 km (23 sections)
Liboganon River	13.0 km (11 sections)

The result of this calculation was used to decide the elevation of bridge and to get the condition of overflow water level for inundation analysis.

6.6.2 Non-uniform Flow Analysis

1) Formula of non-uniform flow

The basic equations applied to non-uniform flow are derived from the following equations:

$$\left\{H_2 + \frac{D_2}{2g}\left(\frac{Q_2}{A_2}\right)^2\right\} - \left\{H_1 + \frac{D_1}{2g}\left(\frac{Q_1}{A_1}\right)^2\right\} = hi$$

$$hi = \frac{1}{2}\left\{\frac{N_1^2 Q_1^2}{A_1^2 R_1^{4/3}} + \frac{N_2^2 Q_2^2}{A_2^2 R_2^{4/3}}\right\}\Delta X$$

where

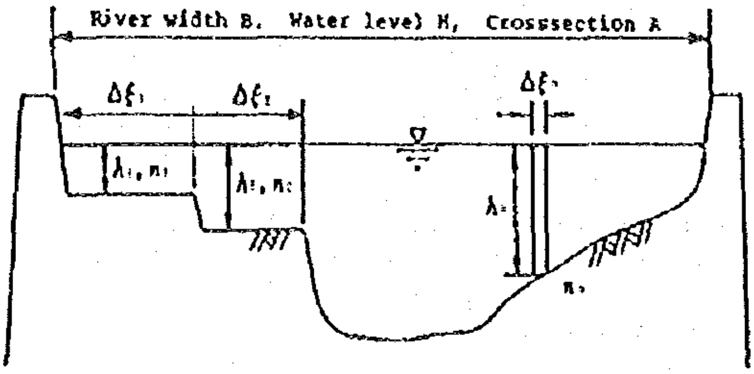
$$D = \alpha \frac{A^2 \int_0^B \frac{h^3}{n^3} d\xi}{\left(\int_0^B \frac{h^{5/3}}{n} d\xi\right)}$$

$$N=\frac{\int_0^B h^{5/3}d\xi}{\int_0^B \frac{h^{5/3}}{n}d\xi}$$

$$R=\left(\frac{1}{A}\int_0^B h^{5/3}d\xi\right)^{3/2}$$

α :	Coefficient of Energy Compensation(0.95-1.0)
B :	Breadth of River
h :	Water Depth
A :	Cross-sectional Area of Flow

In actual calculation, as for the columnar element shown in figure below, the above integration is converted to Σ (sigma symbol).



2) Establishment of Fundamental Condition

A fundamental condition was established for non-uniform flow calculation as follows:

<u>Calculation Case</u>

River Name	Case
Legaspi River I	170m^3/s
Legaspi River II	100m^3/s, 181m^3/s
Baliguian River	99m^3/s
Guinoyoran River	51m^3/s
Andanan River	402m^3/s
Simulao River	1430m^3/s ~ 1020m^3/s
Agusan River	5000m^3/s, 2900m^3/s
Liboganon River	2489m^3/s, 2100m^3/s

Initial Water Level

For a realistic condition at flood time, the initial water level of the first section is decided from several trial calculations that result in a smooth profile of water surface slope from the first section to the second section.

Coefficient of Roughness

Coefficient of roughness is n = 0.03.

3) Result of non-uniform flow calculation

Profile of high water level of each river is shown in Appendix 6.6-1.

High water level of 50-year return period flood at bridge point of each river is summarized below;

River Name	Bridge Name	H.W.L
Guinoyoran Creek	Guinoyoran Bridge	32.0m
Andanan River	Andanan Bridge	21.0m
Agusan River	Monkayo Bypass Bridges	51.0m
Agusan River	Tina Br., Banlag Br.	52.5m
Liboganon River	New Gov. Miranda Br.	5.0m

The location of overflow point of Legaspi, Baliguian and Simulao River is indicated in Appendix 6.6-1. Overflow volume was determined from several trial calculations with a computer flow out program.

River Name	Overflow Point
Legaspi River	3
Baliguian River	1
Simulao River	3 ~4

6.7 INUNDATION ANALYSIS

6.7.1 Inundation in Study Area

The south area of Simulao Bridge suffered from big flood in 1981 and the inundation area spread along the Highway for 9 kms. The flood submerged several parts of the Highway and all types of vehicles had to stop at point about 1356+450. Inundation analysis for Simulao River was carried out to review the reasons for previous flooding condition. Results of the inundation analysis were used to determine countermeasures.

For the inundation analysis, the computer simulation model was established. Based on our field survey of the area affected in 1981, flood marks confirmed the inundation area and depth of water.

From the field survey, the features of flooding in the inundation area were identified as follows:

(a) Overbank flow spreads from poor flow capacity points;

(b) The flood area is the alluvium plain area with a slight slope towards the downstream and overbanking water flows along the river course;

(c) Irrigation canal of NIA and Highway paralleled the river functions like a dike; and

(d) From interview, flood occurs more often after the NIA irrigation channel is constructed.

Considering the above features, flooding in the area shows the storage type and the flow/diffusion type. Therefore, the Two-Dimensional Unsteady Flow Model is employed.

6.7.2 Two-Dimensional Unsteady Flow Model

The basic equations applied to the model are derived from the following equations:

1) Euler's Equation of Motion

$$\frac{\partial u}{\partial t} + u\frac{\partial u}{\partial x} + v\frac{\partial u}{\partial y} + w\frac{\partial u}{\partial z} = X - \frac{1}{\rho}\frac{\partial P}{\partial X}$$

$$\frac{\partial v}{\partial t} + u\frac{\partial v}{\partial x} + v\frac{\partial v}{\partial y} + w\frac{\partial v}{\partial z} = Y - \frac{1}{\rho}\frac{\partial P}{\partial Y}$$

$$\frac{\partial w}{\partial t} + u\frac{\partial w}{\partial x} + v\frac{\partial w}{\partial y} + w\frac{\partial w}{\partial z} = Z - \frac{1}{\rho}\frac{\partial P}{\partial Z}$$

where,

u, v, w	:	velocity of x, y and z directions
X, Y, Z	:	gravity of x, y and z directions
ρ	:	water density (= 1.0)
P	:	pressure

2) Equation of Continuity

$$\frac{\partial u}{\partial x}+\frac{\partial v}{\partial y}+\frac{\partial w}{\partial z}=0$$

For actual application to the two-dimensional model, the equation above are expressed as follows:

Equation of Motion

$$\frac{1}{gA_x}\frac{\partial Q_x}{\partial t}-\frac{Q_xB_x}{gA_x^2}\frac{\partial H}{\partial t}+\frac{\partial H}{\partial x}+\frac{|Q_x|Q_x}{F_x^2}=0$$

$$\frac{1}{gA_y}\frac{\partial Q_y}{\partial t}-\frac{Q_yB_y}{gA_y^2}\frac{\partial H}{\partial t}+\frac{\partial H}{\partial y}+\frac{|Q_y|Q_y}{F_y^2}=0$$

$$F_x=\frac{1}{n}R_x^{2/3}A_x$$

$$F_y=\frac{1}{n}R_y^{2/3}A_y$$

Equation of Continuity

$$\frac{\partial(Bh)}{\partial t}+\frac{\partial Q_x}{\partial x}+\frac{\partial Q_y}{\partial y}=0$$

where,

Q_x, Q_y	: discharge of x and y directions
A_x, A_y	: current area of x and y directions
B_x, B_y	: width of x and y directions
R_x, R_y	: hydraulic depth of x and y directions
g	: gravity acceleration (9.8 m/s^2)
n	: Manning's roughness coefficient
H	: water level
h	: water depth

The above equations are finally transformed into finite difference equations for numerical computation, as follows:

Finite Difference Form of Equation of Motion

$$\frac{1}{gA_{I,J}^{n-\frac{1}{2}}}\frac{Q_{I,J}^{n}-Q_{I,J}^{n-1}}{\Delta t}-\frac{\left(\frac{Q_{I,J}^{n}-Q_{I,J}^{n-1}}{2}\right)\cdot\Delta y}{g\cdot\left(A_{I,J}^{n-\frac{1}{2}}\right)^{2}}\frac{H_{I,J}^{n-\frac{1}{2}}-H_{I,J}^{n-\frac{3}{2}}}{\Delta t}+$$

$$\frac{H_{I+\frac{1}{2},J}^{n-\frac{1}{2}}-H_{I-\frac{1}{2},J}^{n-\frac{1}{2}}}{\Delta x}+\frac{\left|Q_{I,J}^{n-1}\right|Q_{I,J}^{n}}{\left\{\frac{1}{n}\cdot\left(\frac{A_{I,J}^{n-\frac{1}{2}}}{\Delta y}\right)\cdot A_{I,J}^{n-\frac{1}{2}}\right\}^{2}}=0$$

Finite Difference Form of Equation of Continuity

$$\frac{(Bh)_{I,J}^{n}-(Bh)_{I,J}^{n-1}}{\Delta t}+\frac{Q_{I+\frac{1}{2},J}^{n-\frac{1}{2}}-Q_{I-\frac{1}{2},J}^{n-\frac{1}{2}}}{\Delta x}+\frac{Q_{I,J+\frac{1}{2}}^{n-\frac{1}{2}}-Q_{I,J-\frac{1}{2}}^{n-\frac{1}{2}}}{\Delta y}=0$$

where,

suffix I, J: mesh number of x and y directions
suffix n :computation time step number

6.7.3 Inundation Analysis

1) Establishment of Inundation Model

The flood inundation model was established under the following conditions:

- The whole inundation area was divided into mesh blocks of 200 m by 200 m.
- The average ground height of each mesh was obtained using the topographic map with a scale of 1/5,000.
- The overflow discharge level was taken from the result of non-uniform flow analysis.
- Structures such as highway and irrigation channel which may hamper smooth flow of inundation water were taken into consideration assuming them as barriers between the mesh blocks.
- Three cases were simulated; two cases simulate the flood of 1981 and consider both the NIA canal constructed and the NIA canal not constructed. Another case is considered 25-year return period flood with NIA canal.

Figure 6.7-1 shows a three-dimensional model of the inundation area. From this figure, if there are no barrier between the mesh blocks, overflow water occurs two main currents; one is along the river and the other flow is in the tip of the alluvial fan.

2) Results

The inundation area for a 25-year return period flood is shown in Fig. 6.7-2. The inundation area spreads between the NIA dike and the highway and the flood water crosses over the highway at the 1356 km point.

From the comparison between the actual inundated area of 1981 flood and the simulation model, the peak discharge of the 1981 flood is estimated to be 1250~1400m^3/s. This flood scale is equivalent to more than a 50-year return period. It is well supported that the rainfall of this month recorded 1,500 mm at Trento Town which is three times as much deep as the average rainfall of January.

On the other hand, comparing cases between without and with the NIA irrigation channel, the flood area before construction of the NIA channel is greater than that after its construction. Flood depth before construction of the NIA channel is thus shallower than after construction. Therefore, people felt that heavy flood occurs more often after the NIA irrigation channel was constructed.

6.8 SCOURING AROUND PIER

The amount of scouring due to the obstruction of hydraulic structures depends on the shape of structure, the actual river bed materials and the flow condition. The existing Andanan Bridge is scoured heavily. The foundation and piles are exposed. The reason for scouring, is that the direction of current has been changed from northeast to east during a big flood. Therefore, the angle between the foundation and current has increased by 45° - 60°. The angle of pier direction to the flow influences the scouring depth and the scouring depth increases in proportion to the angle of pier direction to flow.

In this section, the scouring around pier is calculated for new Andanan Bridge by the Andru's Relation and Laursen's Relation.

1) Andru's Relation

The maximum scoured depth appears at the foot of the upstream surface of pier. The relation between the maximum scoured depth and the water depth is shown as follows:

$$hs(Q^2/B^2h^3)^{1/3} = 1.8(Q/B)^{2/3} \text{ or } hs = 1.8\ h$$

Where;

Q^2/B^2h^3 : Bed factor by Lacey
Q (m^3/sec) : Flow volume

Three Dimensional Chart in Simulao Inundation Area

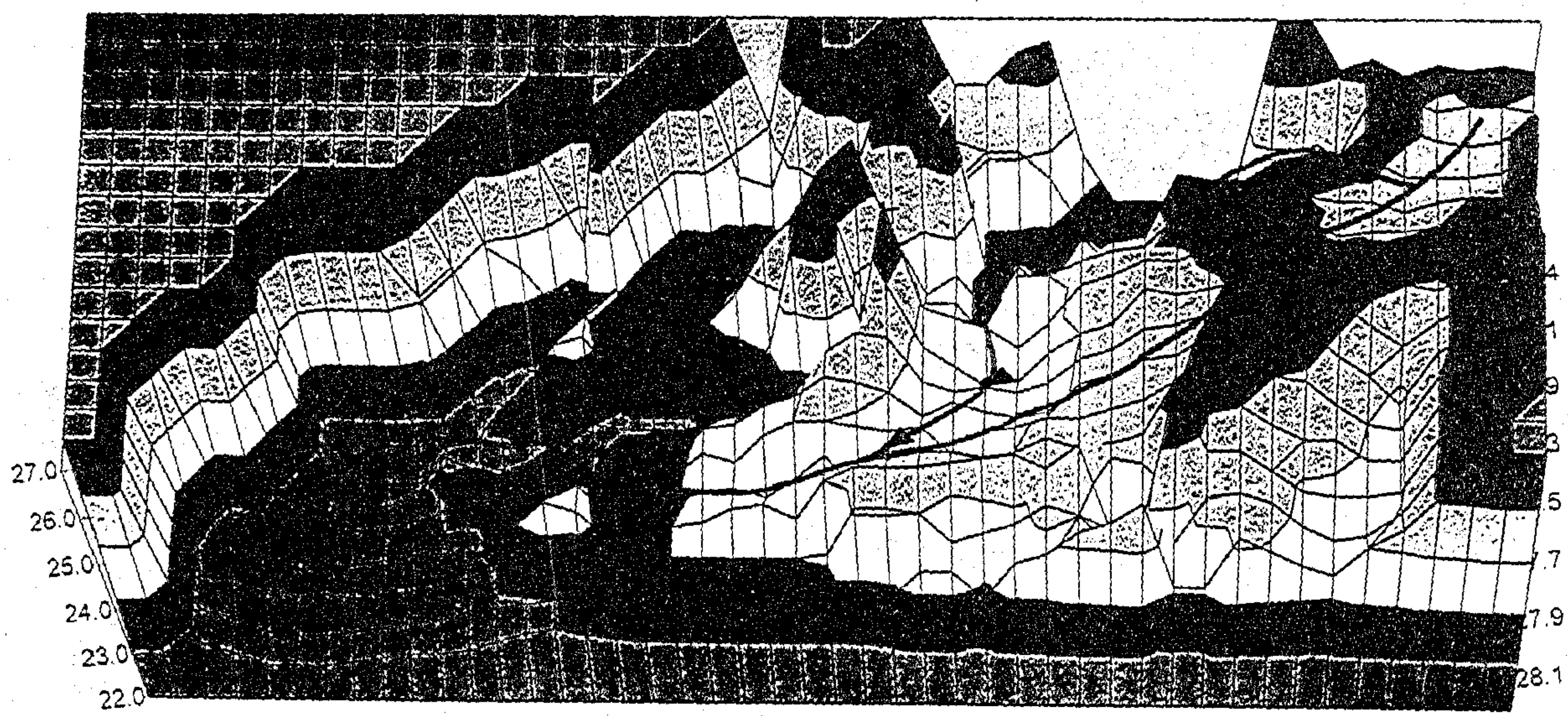
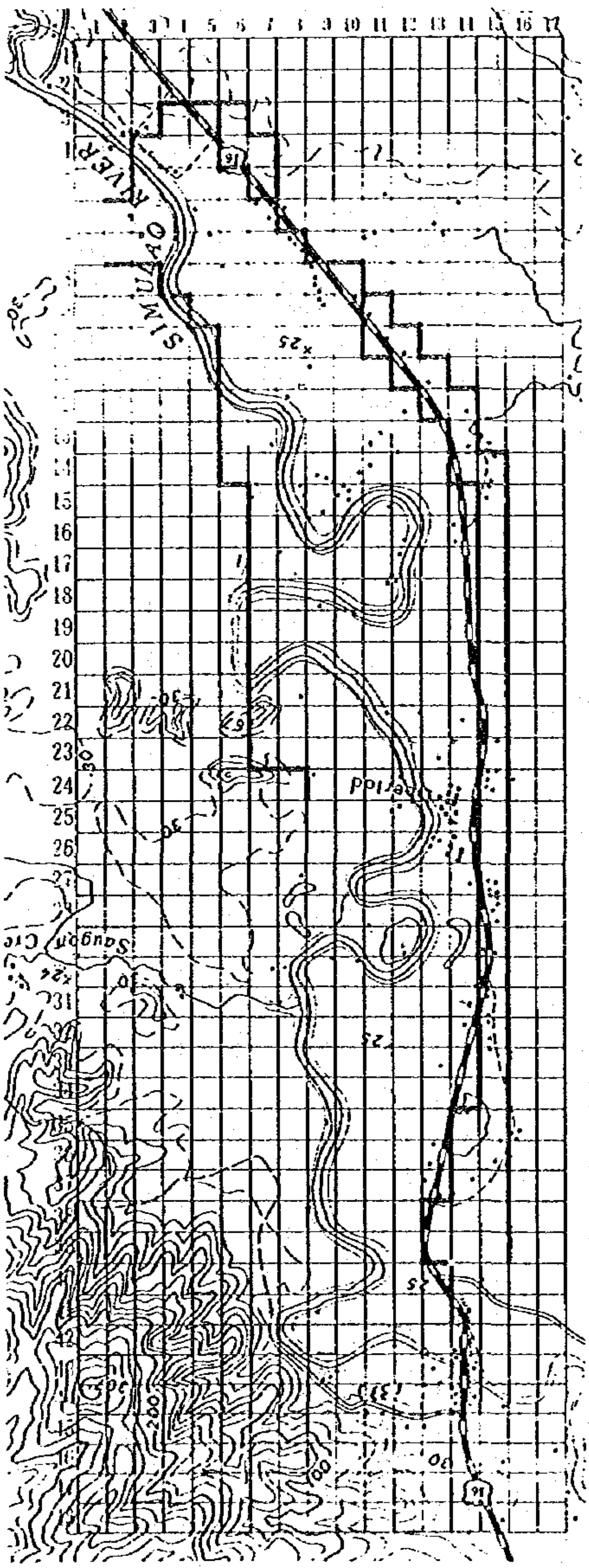


FIGURE 6.7-1 THREE DIMENSIONAL CHART IN SIMULAO INUNDATION AREA

7.7

Model of Inundation Analysis at Simulao River



Model of Inundation Analysis at Simulao River
25-year Return Period Flood Qmax=1020m3/s

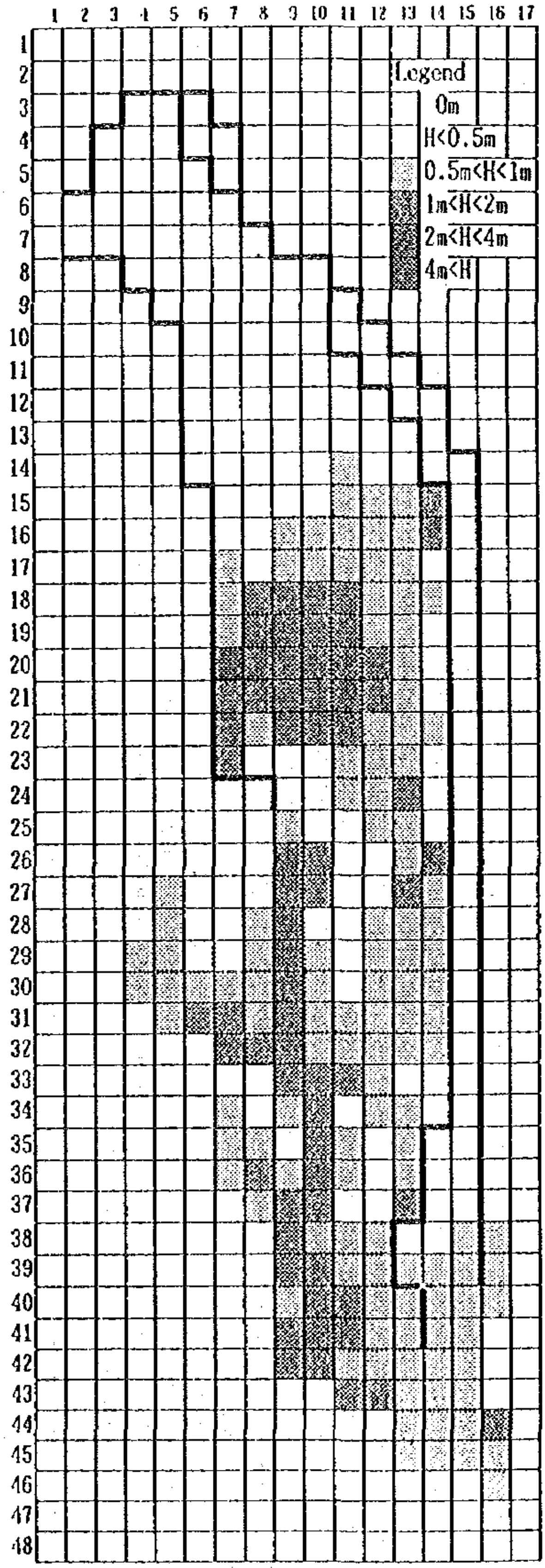
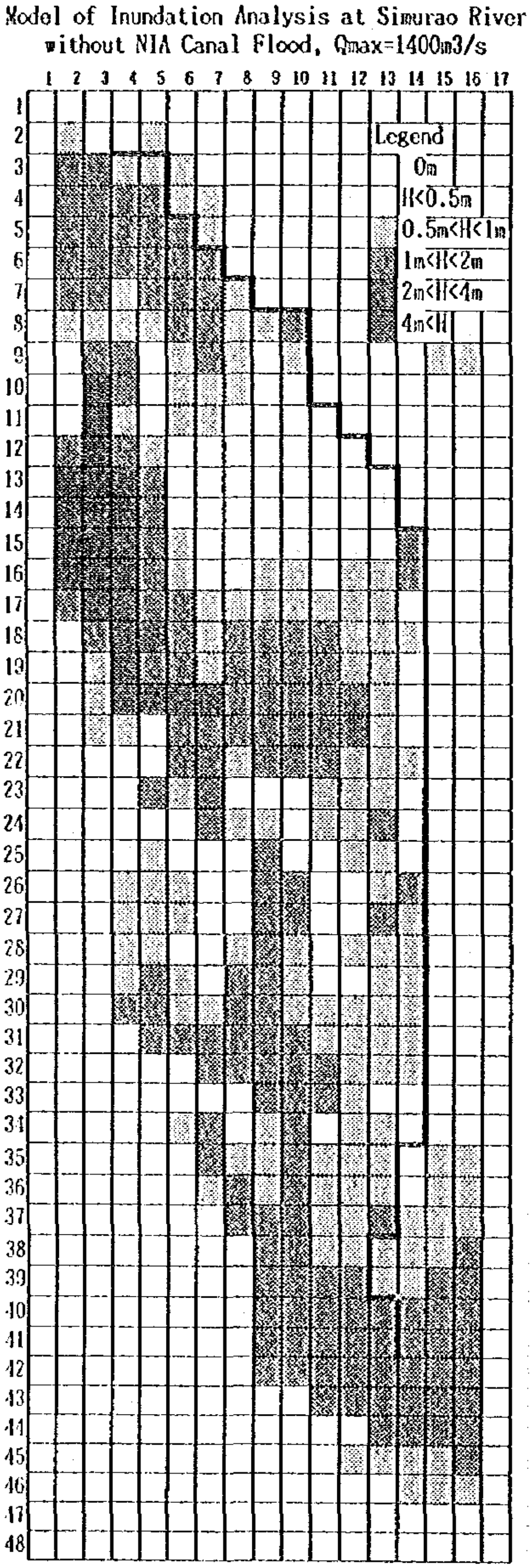


FIGURE 6.7-2 (1/2) INUNDATION AREA OF SIMULAO RIVER



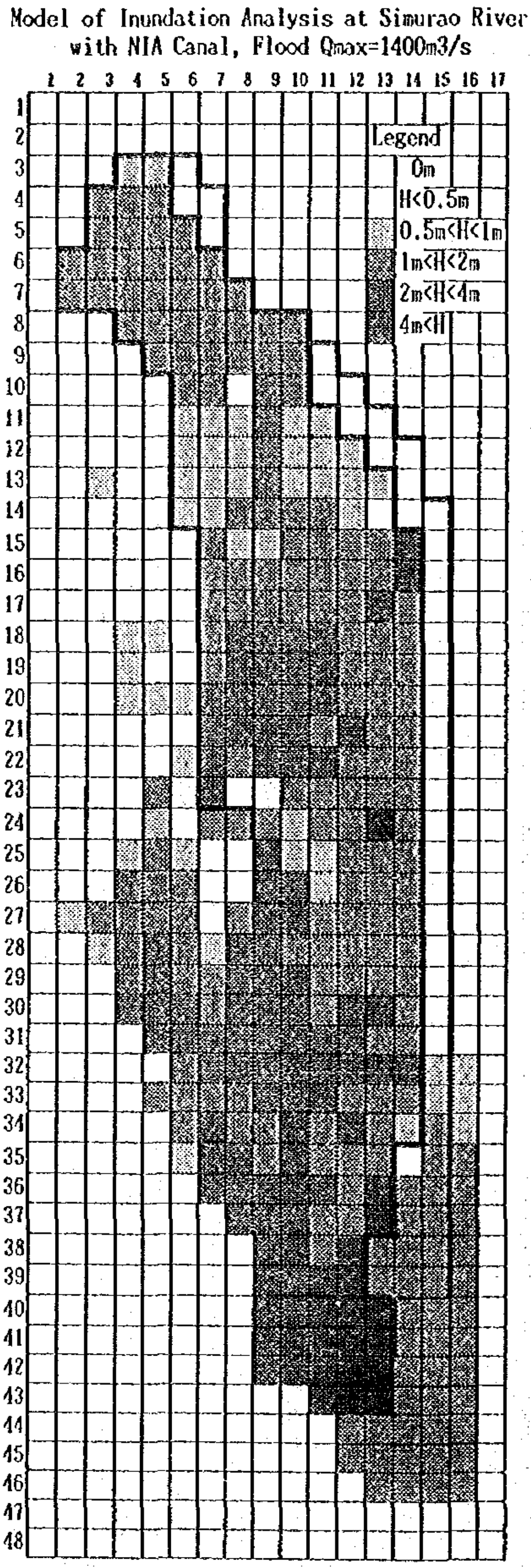


FIGURE 6.7-2 (2/2) INUNDATION AREA OF SIMULAO RIVER

B (m)	: Width of channel
h (m)	: Depth of water
hs (m)	: Depth of scouring from water surface

The values of model test results and observation on actual case are distributed along the curve of $hs = 1.8\ h$ as shown in Figure 6.8-1, and the maximum scoured depth is about 0.8 times of water depth.

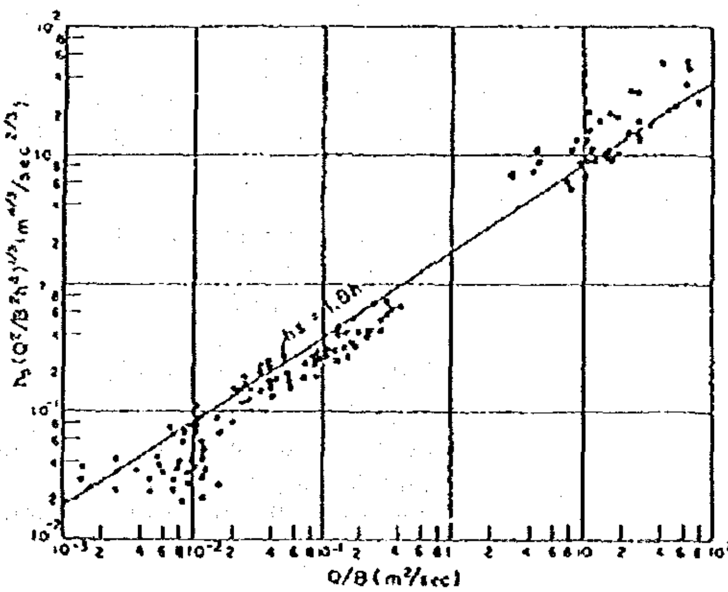


FIGURE 6.8-1 RELATION BETWEEN MAXIMUM SCOURED DEPTH

AND FLOW WATER DEPTH

2) Laursen's Relation

The maximum scoured depth is dominated in general case by the flow quantity, namely the water depth, and partly affected by the velocity and the characteristics of riverbed material. Therefore, the relations were found by model tests considering the shape and size of pier and the direction of flow.

The maximum scour depth at the upstream edge of pier (Zs) is expressed as a relation between the ratio of scoured depth on the pier width and the mean water depth on the pier width. The standard case of rectangular pier with parameters follows;

Ks : Coefficient due to the shape of pier head against the flow is below

Shape and L/b			Ks
Rectangular		□	1.00
Round Head		∪	0.90
Round Edge	2 :1	∧	0.80
	3 :1	∧	0.75
Elliptical Head	2 :1	⊂	0.81
	3 :1	⊂	0.70

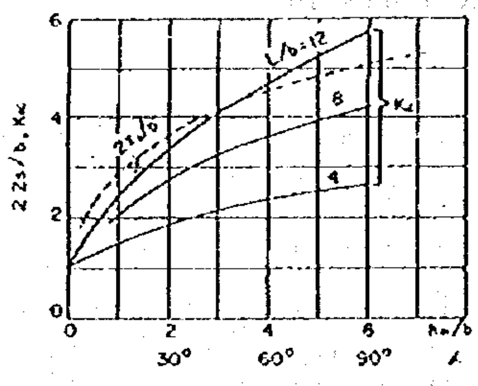
K_α : Parameter relating to L/b

K_τ : Parameter considering riverbed material together with flow quantity, river width and energy gradient.

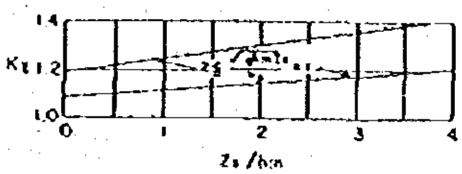
Where;

Zs	: Balanced maximum scoured depth (m)
b	: Width of pier (m)
L	: Length of pier (m)
hm	: Mean water depth (m)
g	: Gravity acceleration (cm/s^2)
I_e	: Energy gradient
U^*m	: Friction velocity of transport $\sqrt{ghmI_e}$ (cm/s)
W_0	: Grain fall velocity (cm/s)
α	: Angle of pier direction to the flow (degree)

When $U^*m / \omega_0 < 0.5$ the sediment transport moves as the bed load, and the relationship of maximum scoured depth and water depth can be shown directly by the following figure.



When $U^*m / \omega_0 > 0.5$ $K\tau$ is found by the following figure. In this case Zs is the maximum scoured depth for the case where $U^*m / \omega_0 < 0.5$.



3) Scour of the Andanan Bridge

The shape of pier for the new bridge are circular columns of 1.9m in diameter. This shape anticipate a change in river current direction in the future. The river bed scouring is calculated based on the hydraulic conditions calculated by non-uniform flow analysis as discussed in Section 5.6.

$Q = 402m3/s$, $\phi = 1.9m$ $(L/b = 1)$, $hm = 2.5m$, $\alpha = 0$ degree,
$U^*m = \sqrt{ghmIc} = \sqrt{9.8 \times 2.5 \times 0.0015} = 0.19 < 0.5$
Therefore

Shape of pier head is round, then $Ks = 0.9$
$L/b = 1$ and $\alpha = 0$ degree, then $Ka = 1.0$
$hm/b = 2.5/1.9 = 1.3$ and $2Zs'/b = 3.0$, then $Zs' = 2.85$
$Zs = Ks \times Ka \times Zs' = 2.85 \times 0.9 \times 1.0 = 2.56$ m

The scouring depth is calculated as $hs = 2.5 \times 0.8 = 2.0m$ from Andru's Relation and $Zs = 2.56m$ from Laursen's Relation. Therefore, in consideration of the results from two relations, the scouring depth is estimated to be 2.5m at new Andanan Bridge.

6.9 SUMMARY OF THE RESULT OF ANALYSIS

1) Flood Section 1 (1160+700~1161+700)

Flood caused by surface runoff from mountain slopes concentrates near 1161 km post. The mountain slope which affects this section is separated by two catchment areas. The discharge volume of each area is $39m^3/s$ and $24m^3/s$. To decentralize flood water, discharge water has to be drained by several pipe culverts and box culverts constructed along the highway.

2) Flood Section 2 (1163+600~1164+100 km)

Flood occurs for the same reason as flood section 1. Draining capacity is insufficient to prevent flood from two mountain slopes near the 1164 km post. Discharge volume is $57m^3/s$ from the first slope and $18m^3/s$ from the second slope. Pipe culverts or box culverts are recommended to decentralize discharge water.

3) Flood Sections 3, 4 and 5 (1165-1168 km) including Legaspi River, Magtiaco Br., and San Pedro Br..

The main current of Legaspi River flows from southeast to northwest. At 2.5 km upstream from Magtiaco Bridge, a tributary connects from the right bank. The total catchment area is 22.5 km^2. Peak discharge volume at main point are shown in Fig. 6.9-1

There are three overflow areas along the river. The first overflow area is at the NIA weir which is located about 10 km upper stream from Magtiaco Bridge. A 10m portion of the left dike, which is just upstream from the weir, has been broken. When 25-year return period flood occurs, volume of $10m^3/s$ overflows occur here. In the portion of the Highway from Km.1166.5 to Km.1168 km, the discharge volume is $48m^3/s$, with $38m^3/s$ from original catchment area and $10m^3/s$ overflow from the river.

The second overflow point occurs at the left bank 2 km downstream from NIA weir. The 50-year return period flood ($180m^3/s$) passes this area, with $80m^3/s$ flowing out to tributary. The flood includes heavy sediments and after flowing out to tributary, the force of flow is dissipated. Thus, sediment is deposited and stored in the river.

Discharge at San Pedro Bridge becomes 110 m^3/s because overflow from the Legaspi river also adds discharge to the original catchment area. The left side of embankment of the Highway, 200m from San Pedro Bridge, has to be protected from flood.

The third overflow point is left dike located 600m upstream from Magtiaco Bridge. This point is proposed to be protected by a concrete dike thus increasing the cross-sectional area of flow.

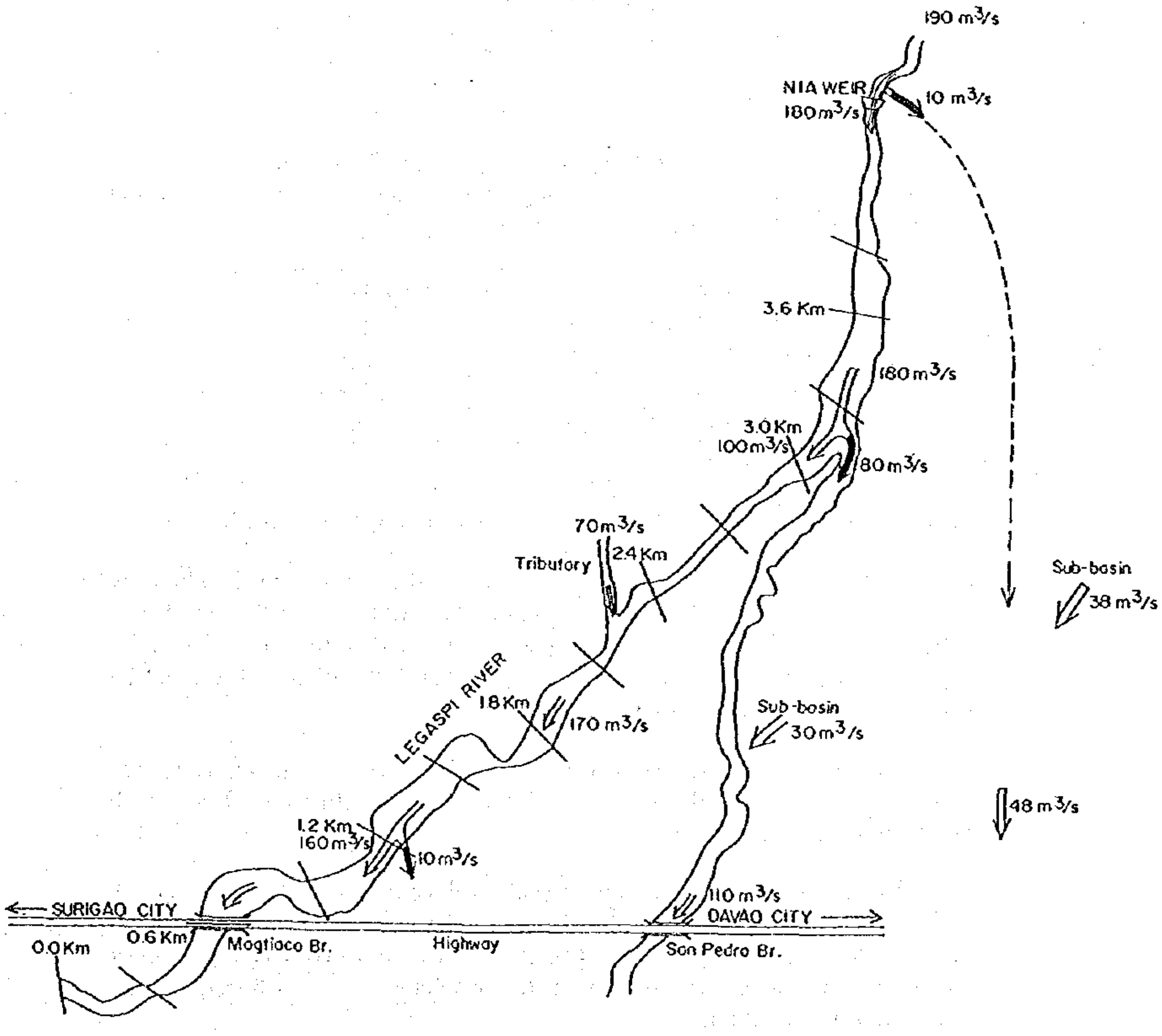
4) Flood Section 6 (1182+100~1182+590) Baliguian River

Baliguian River flows from east to west after passing the mountain areas and turns north at 100m east point from the Highway. After turning, the flow passes parallel to the Highway.

The turning point doesn't have sufficient flow capacity to accommodate a 25-years return period flood and thus a volume of $20m^3/s$ overflows. It is recommended that a box culvert be located at overflow point of the highway..

5) Flood Section 7 (1183+100~1183+260)

This section is a part of the Sayadion River catchment area. During a flood, discharge flows from the original sub-catchment area is increased due to overflow water from Sayadion River. Sediments occur at the exit point of the valley of Sayadion River and thus flood occurs due to insufficient capacity. At this flood section, $19m^3/s$ form the original sub-catchment plus $14m^3/s$ which is 30% of 25-year return period flood (47m3/s) from Sayadion River, or a total of $33m^3/s$ must be accommodated by a box culvert.



LEGASPI RIVER

FIGURE 6.9-1 PEAK DISCHARGE VOLUME OF LEGASPI RIVER

19.1

6) Flood Section 8 (1184+250~1185+200)

Along the Highway, from 1184+450 to 1185+000, the catchment area is 1.05 km^2 and peak discharge volume is $46.9m^3/s$.

7) Flood Section 9 (1187+600~1189+200)

Mountain slope is separated into two parts from a topographic viewpoint and peak discharge volume is shown below;

Flood Section	Peak Discharge
1187+600 to 1188+500	$34m^3/s$
1188+500 to 1189+400	$53m^3/s$

8) Flood Section 10 (1192+00~1193+800)

Flood of this section is similar to the Flood Section 9. This flood section has to be divided into four parts from topographic viewpoint. Subdivided flood section and peak discharge are shown below;

Flood Section	Peak Discharge
1192+100 to 1192+320	$38m^3/s$
1192+320 to 1192+650	$21m^3/s$.
1192+650 to 1193+200	$27m^3/s$
1193+200 to 1193+960	$20m^3/s$

9) Flood Section 11 (1196+800 ~1197+200) including Jagupit Bridge

Flood occurs at two different points. One point is the east side of the Highway from 1196+800 to 1197+000. There is a pond here which has no outlet. When long term rainfall continues, this section is inundated for long time with increased water level. It is recommended the water elevation of the pond be lowered one meter by outlet pipe culvert.

A sedimentation problem occurs at the Jagupit River. The catchment is located in an area of distributed erosive conglomerate. Thus, this area has a high potential for sediment production.

Sabo dam is one of the countermeasures for sediment control. But in this case, it is difficult to find a suitable location to store the large sediment volume, because the valley is very narrow. Therefore, dredging of river bed is the best countermeasure and reforestation in the catchment area will reduce the sediment product volume.

10) Flood Section 12 (1197 +556~1197+571) including Guinoyoran Bridge

The catchment area of this section is similar geologically to the Jagupit River. Then, this point has similar sediment problem. But the problem is not only sedimentation. The flow capacity at the bridge is too small and this point functions like a bottleneck point. Thus, flood water slows at upstream of the bridge, and sediment is deposited at this point.

To keep the smooth flow, the width of river has to be extended and 45m in length is need for new Guinoyoran Bridge.

11) Flood Section 13 (1199+600~1203+870)

This flood section is divided into four parts from a topographic viewpoint. Subdivided flood section and peak discharge are shown below;

Flood section	Peak Discharge
1199+500 to 1200+600	$40m^3/s$
1200+600 to 1201+600	$40m^3/s$.
1201+600 to 1202+120	$16m^3/s$
1202+120 to 1202+620	$21m^3/s$

12) Sta. Ana Bridge (1202+825), Kinahiloan River, Manisuag River

Since an average clearance under the girder to the river bed of Sta. Ana Bridge is presently 2m, the water depth is calculated to check this clearance.

The peak discharge volume at Sta. Ana Bridge is $390m^3/s$ for a 50-year return period flood. The water depth is calculated to be 1.1m by the following relation;

$$Q = v \times A = \frac{1}{n} R^{2/3} I^{1/2} \times R \times B$$

$$R = \left(\frac{Q \times n}{B \times I^{1/2}}\right)^{3/5} = \left(\frac{390 \times 0.03}{100 \times 0.011^{1/2}}\right)^{3/5} \cong 1.07m$$

The allowance under the girder to H.W.L is about 1m. Additional dredging is not required at this time.

13) Flood Section 14 (1219+700~1220+100)

The catchment area is $3.6\ km^2$ and the peak discharge from the flat paddy field is $25m^3/s$.

14) Flood Section 15 (1224+200~1224+640) including Arega River, Agay Br., and Minalum Br.

Arega River separates into two streams at about 2 km upper stream from the Highway as shown in Fig. 6.9.2. The right stream flows toward and passes the Agay Bridge and the left stream passes Minalum Bridge. But presently the left stream is closed by a dike which was built by NIA. Thus, there is insufficient flow capacity for the right stream; peak discharge includes heavy sediment. Sediment deposits, as the result of reduction of flow energy, when flooding occurs.

The peak discharge of Arega River is 228m^3/s for a 50-year return period flood and river width is 40m with two bridges. The water depth is calculated to be 1.6m by following formula;

$$R = \left(\frac{Q \times n}{B \times I^{1/2}}\right)^{3/5} = \left(\frac{228 \times 0.03}{40 \times 0.006^{1/2}}\right)^{3/5} \cong 1.6m$$

Thus, it is recommended that the NIA dike at the separating point be withdrawn and the right stream be dredged to 1.5m in depth.

15) Andanan Bridge (1266+037) and Andanan River

Existing Andanan Bridge is scoured presently and the foundations and piles are exposed. This scouring occurs because of the change in current direction compared to the pier direction; the scouring depth increases in proportion to the angle of pier direction to the flow.

The new bridge uses single columns of 1.9m to anticipate the change of the river current direction in future. The scouring depth is calculated 2.5m at Andanan Bridge.

16) Flood Section 16 (1355+200~1364+200) and Simulao River

From results of inundation analysis, 1981 flood appears to be a 50~100-year return period flood. In the case of 25-year return period, the submerged portion in 1981 at the low point at Km.1356+450 and at Km.1360+500 will not be flooded, although water level comes to near elevation of the road.

A countermeasure in this flood section is to raise the road. It is recommended to raise two portion, from 1355+200 to 1356+900 to E.L.+26m and from 1360+200 to 1360+850 to E.L.+28m.

17) Flood Section 17 (1395 km) including Agusan River, Kalaw Bridge, Tina Bridge, and Banlag Bridge

In 1994, Monkajo Town and its surrounding area suffered from flooding by huge discharge of Agusan River and the Kalaw Bridge was covered with 1m of water. If this phenomenon is simulated by non-uniform flow analysis, the peak discharge of this flood is found to be Q = 2,900m^3/s.

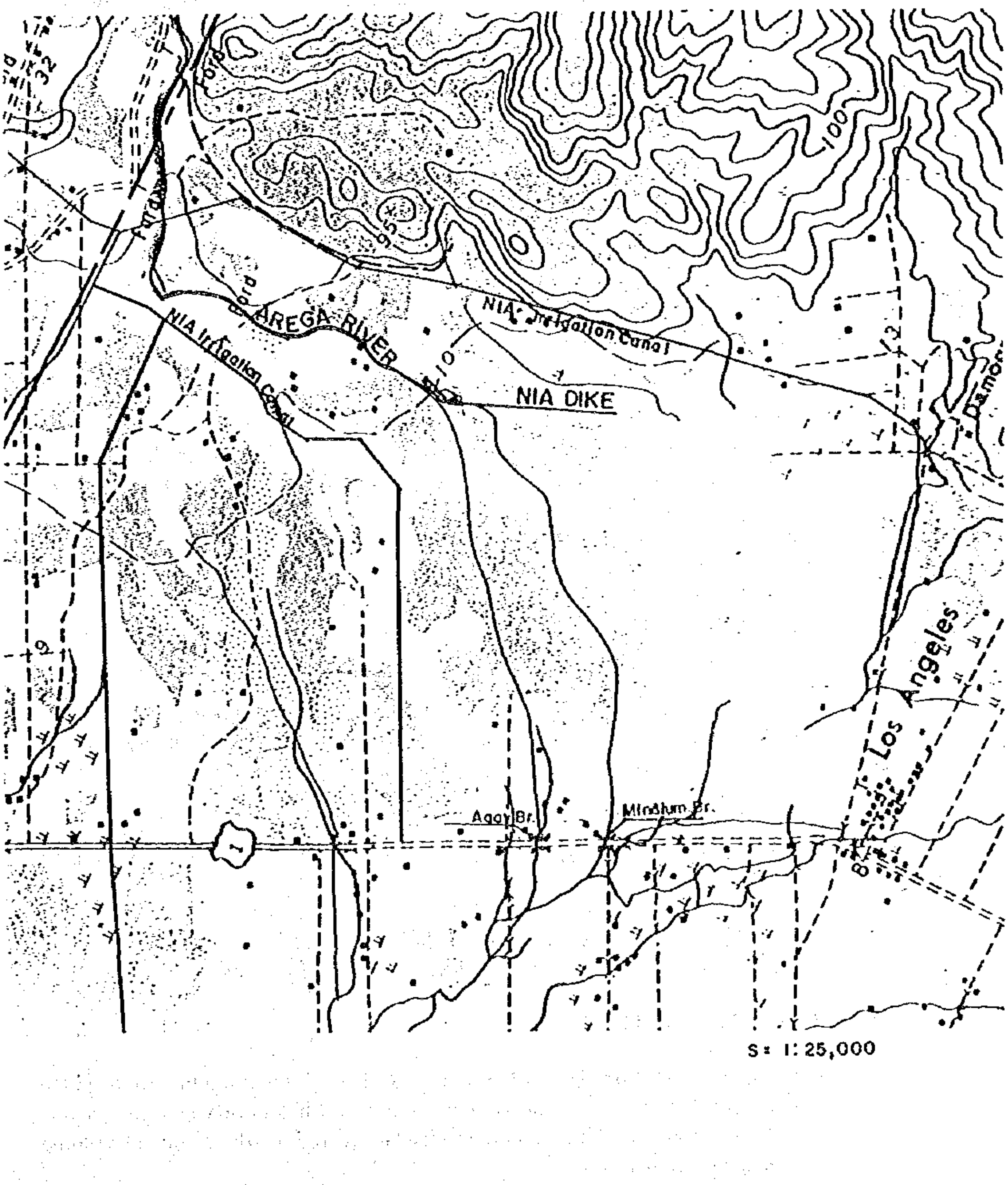


FIGURE 6.9-2 AREGA RIVER AND ITS SURROUNDINGS

fig 9.2

In the F/S, a new bypass route was selected to avoid flooding of the Highway. Thus, the new bridges along the bypass is raised above the flood (Q = 2,900 m^3/s) anticipating a High Water Level of E.L. +51.0m at the new bridge points. The H.W.L of Tina Bridge and Banlag Bridge estimated to be E.L. +52.5m

18) Flood Section 18 (1466.7 km) including Liboganon River and Gov. Miranda Bridge

The left dike of the Liboganon River have already been constructed by NIA. On the right side, a dike is planned by the Liboganon Flood Control Study (1988). It has been intermittently constructed by DPWH.

In this study, flood control was planned considering a 25-year return period by considering the actual existing Gov. Miranda Bridge between dikes. The existence of the bridge is one of the big factors to break the downstream discharge.

For the present study, the bridge reconstruction plan is decided considering the following items;

(a) Bridge length is to be as short as possible. However, the alignment plan of the dike must avoid a sudden transition which affects hydrological condition. Thus, the river width, water depth and velocity should not be changed drastically at the bridge site.

(b) The design storm frequency is a 50-year return period for this bridge. The elevation of NIA dike is considered to be at flood level of a 25-year return period flood. Thus, the elevation of left dike is adjusted to the elevation of the NIA dike.

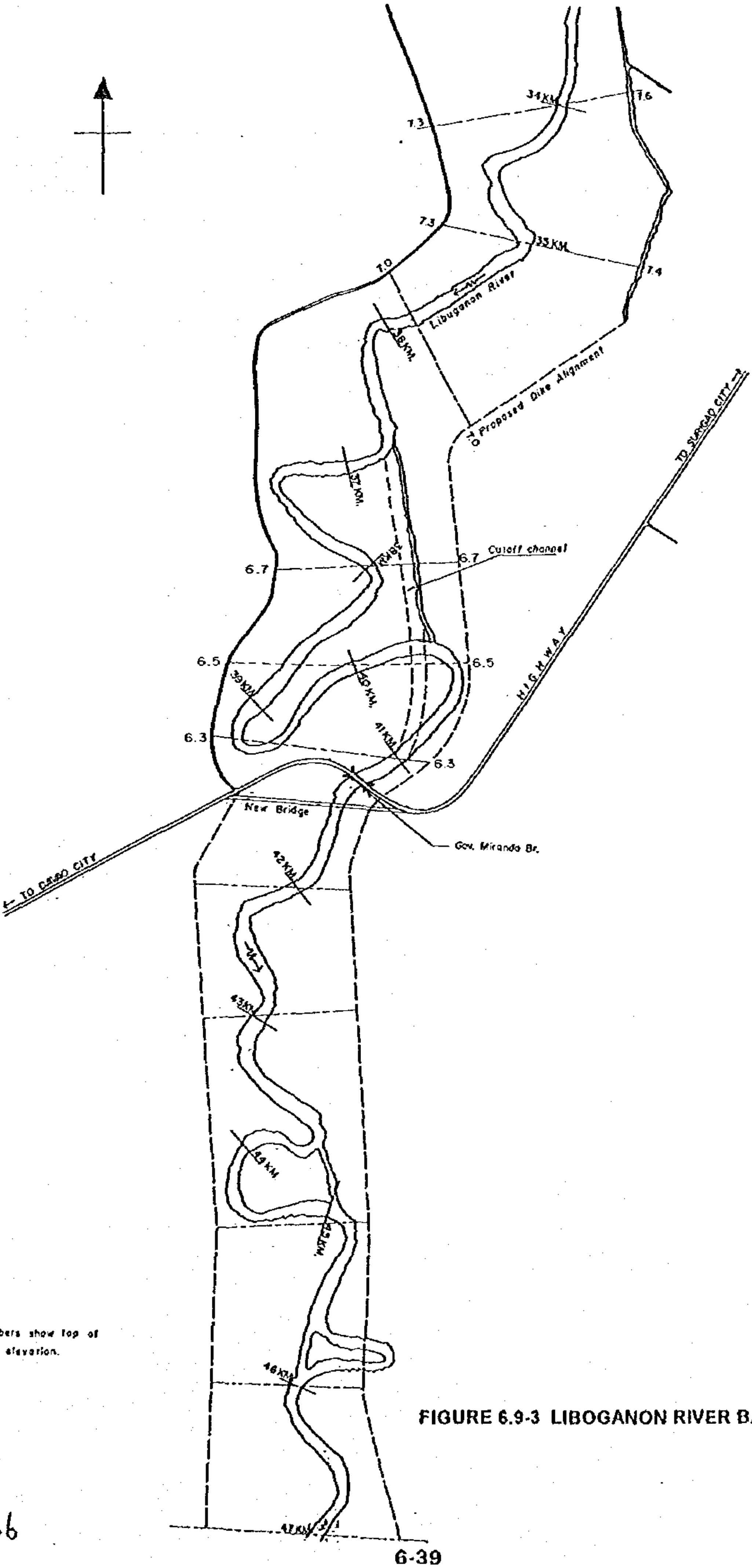
(c) Structures which may hamper smooth flow should not be planned in the vicinity of the new bridge. The existing bridge and Highway will be removed completely for increase of cross-sectional area of river.

(d) To stabilize the current direction, a cutoff channel is recommended to be constructed in the serpentine part in just upper stream of the existing bridge.

The river width at New Gov. Miranda Bridge is recommended to be 650m. The E.L.+5m is H.W.L. and the elevation of dike is shown in Fig. 6.9-3.

6.10 RECOMMENDATION

1) From flood section 7 to 12, these areas have high potential to produce sediment. It is a very fragile geological area and has poor vegetative cover. Reforestation is a effective countermeasure to reduce the sediment volume of small catchment area.



NOTE: Numbers show top of dike elevation.

FIGURE 6.9-3 LIBOGANON RIVER BANK ELEVATION

66

2) Upper Agusan Development Project is not expected to be constructed in the near future. However, if it is constructed, the peak flood discharge increases 4,910 m^3/s for a 50-year return period flood. To move this additional volume, a cutoff channel is recommended to be constructed in the serpentine part of downstream.

3) In the Liboganon River, the dike from Gov. Miranda Bridge to river mouth is not considered in this study. The approach stretch to the river mouth is influenced by many factors of tidal water of Davao Bay. Therefore, this stretch should be carefully studied in the near future.