A Method of Flood Runoff Estimation in an Ungauged Catchment (Ok Mani) in the Highlands of Papua New Guinea

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DEDICATION

This Dissertation is dedicated to my parents, Christine (late) and Thomas. I will always cherish and treasure their love, determination and hard work in getting me this far. Without them, I would achieve nothing.

ABSTRACT

Ok Mani stream is one of the major tributaries of the Ok Tedi River in the Western Province of PNG. The catchment is located south of one of the world's biggest open-cut gold and copper mines, the Ok Tedi Copper Mine. The catchment is in one of the areas in PNG that receives the highest rainfall annually and is located within a region of very unstable geology.

One of the mine's overburden storage dumps is located in the Ok Mani catchment and it subsequently failed due to the increasing weight of the overburden. The failure resulted in major changes to the morphology, sediment loads and the biota of the stream and the rivers downstream. The fieldwork of this dissertation was part of a major investigation undertaken to locate an alternative site in the catchment to store the mine's overburden.

The dissertation presents the results of a study undertaken in the headwaters of Ok Mani stream. There is a discrepancy in the current runoff-rainfall record from the catchment, where runoff appears to be significantly greater than the rainfall. The study attempts to quantify the storm runoff from different sub-catchments and seeks to confirm or not the possibility of extra-catchment sub-surface flows into the catchment.

The results indicate that the measured runoff and rainfall are not reliable and that discrepancies between runoff and rainfall did not support the hypothesis of extra-catchment flows. There is evidence that rainfall increases significantly with increasing catchment elevation. However, the study undertaken for this thesis was very short and thus the results obtained are very limited. Therefore, further research into the sources of the excess runoff to that of the rainfall gauged at MANO4 is required before the runoff-rainfall discrepancy is put into perspective.

ACKNOWLEDGEMENTS

I am grateful and acknowledge the following individuals and groups of people who have contributed in one way or another during the course of writing this dissertation.

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ABBREVIATIONS

A	-	area in square kilometres
asl	-	above sea level in metres
C_2	-	runoff coefficient of flood
cumecs	-	cubic metres per second
D	-	total rainfall depth of storm in millimetres
ELVN	-	mean channel elevation in metres
I	-	rainfall intensity in millimetres per hour
km	-	kilometres
km^2	-	square kilometres
KS	-	swamp adjustment factor of sub-catchment
L	-	main stream length in kilometres
m	-	metres
mm	-	millimetres
m/km	-	metres per kilometre
m/s	-	metre per second
m^3/s	-	cubic metres per second
NERC	-	Natural Environment Research Council
NWASCO	-	National Water and Soil Conservation Organisation
NRC	-	National Research Council
OTML	-	Ok Tedi Mining Limited
P_2	-	point storm rainfall in millimetres
PNG	-	Papua New Guinea
Q_P	-	peak flood-flow
r	-	number of revolution
S	-	slope of main stream channel
SMEC	-	Snowy Mountain Engineering Corporation
t	-	duration of storm in hour
V	-	stream-flow velocity in metres per second

Chapter 1

INTRODUCTION

1.1 Concept

River flow characteristics are closely connected with the problem of suitable design criteria for structures in water resource projects, such as spillway designs, culverts, bridges, waste and water retention dams, etc (Nemec 1972). Flood plain management, the design of hydrologic structures and other water-related investigations need to reflect the probability of extreme flood events. An appropriate estimate of this extreme flood is fundamental in ensuring that engineering designs with adequate standards of safety are achieved (NWASCO 1982). Extreme rainfall events and the resulting floods can take thousands of lives and cause billions of dollars in damage (Stedinger, Vogel and Foufoula-Georgiou 1993).

Flood peak discharge has been an essential item in the planning and construction of water resource projects for many decades and the estimation methods developed have been a major tool for engineers, hydrologists, designers and researchers. It is an important tool in the transfer of hydro-engineering information from gauged to ungauged catchments (Rao and Hsieh 1991, Zrinji and Burn 1994), for the estimation of flood flows (NWASCO 1983, Qinliang and Eagleson 1987, SMEC 1990), project design and planning (Nemec 1972, Ward 1975, NRC 1988, SMEC 1990) and other engineering, planning, hydrological and research applications (Heerdegen 1973).

1.2 Purpose of Research

The Ok Mani catchment rainfall-runoff deficiency investigation is part of a large project currently undertaken by Klohn Crippen Consultants of Canada for the Ok Tedi Mining Limited (OTML), Papua New Guinea (PNG). The objective of the project is to look at alternative mine-waste retention sites in the Ok Mani stream catchment as the current one has failed. The Hydrology Section of the Environment Department - OTML has been asked by Klohn Crippen to monitor the rainfall-runoff in the catchment. Currently, two rainfall and one stream-flow gauging stations have been established in the catchment. The data collected and analyzed so far over a period of two years has indicated that runoff is 150 percent greater than rainfall in the catchment, giving rise to an apparent hydrological paradox.

The flow discrepancy is understood to be caused by one or several of three possible factors, including: (i) errors in the gauging and analysis of current data; (ii) high rainfall in the highland areas of the catchment that is not currently being monitored; and (iii) in-flows from adjoining catchments.

The first possible cause has been looked at and dismissed as all the data gauged and analyzed has been of good quality. The second possible cause is currently being looked at. Three raingauges have been installed in the highland areas of the catchment to monitor rainfall in that region. This investigation seeks to confirm or not the existing rainfall-runoff discrepancy which will help determine the third possible cause - to establish whether there are underground flows from adjoining catchments into Ok Mani and to identify possible sources of these inflows.

The investigation involved mapping the upper-most section of the catchment into sub-catchments and quantifying the flows in these sub-catchments into the Ok Mani stream. By use of flood estimation techniques, flows from the sub-catchments were estimated and compared with those from gauged sub-catchments. The extent of inflows from adjoining catchments into Ok Mani can then be established from the comparisons.

Once rainfall in the highland areas of the catchment is monitored and the extent of inflows into the catchment is established, the rainfall-runoff discrepancy in the Ok Mani catchment can then be put into perspective.

1.3 Literature Review

1.3.1 Flood Estimation Methods

Floods of extreme magnitude with a low probability of recurrence are of continuing interest to the hydrologic and engineering communities for purposes of design and planning (Pilgrim and Cordery 1993). Many estimation methods that have been developed are used to estimate magnitudes of these floods and the methods chosen vary from region to region. Despite their widespread use, the methods involve a number of assumptions (Ayoade 1988, Pilgrim 1975) of both climatic and physical catchment parameters. When these assumptions are kept to a minimum, they provide "best estimates" of floods (SMEC 1990).

Various estimation techniques of flood prediction and forecasting exist in the literature, but they can all be categorised, following Ward (1975) as; (i) empirical, (ii) statistical, (iii) analytical and (iv) modelling techniques. Which of the techniques is chosen depends on a number of factors including the purpose for which it is required, the available data and the area and characteristics of the drainage basin (Ward 1975). According to Pilgrim and Cordery (1993), the three most widely used techniques for estimating flood-flows on small drainage basins are the Rational, the U.S Soil Conservation Service and the Regional Flood Frequency methods.

The Flood Estimation Manual prepared by SMEC (1990) for the PNG Bureau of Water Resources (PNGBWR) outlined ten flood-estimation methods. Some are for rural catchments and others for urban, of any size (up to 50 000 km^2 or more) at any location in PNG depending on whether it is rural or urban and for return periods ranging from 2 years to 100 years. The procedures are principally directed to the estimation of peak flood-flows.

The flood data used to develop these estimation procedures come mostly from BWR. It has a data base containing all flood data in PNG, plus the history files and discharge rating curves for all its gauging stations. Other organisations such as OTML, Porgera Joint Venture and the National Weather Service, in certain areas, record and keep stream gauging and climatic data.

A couple of the estimation methods formulated by SMEC (1990) that more-or-less fit the Ok Mani catchment characteristics and the requirements of this rainfall-runoff anomalies investigation and applied in this research are the Rational and Regional Flood Frequency methods. The runoff coefficients for the Rational method were derived from measured flood data on thirty streams with catchment areas ranging from 5 km^2 to 350 km^2 and were related to physical catchment parameters (SMEC 1990). The Regional Flood Frequency method was based on the regression analysis of 66 flood records and various catchment and rainfall parameters. The catchments were located throughout PNG (most on the mainland) and had catchment areas ranging from 5 km^2 to 40 900 km^2. The Rational method is deemed to be more suitable than the Regional Flood Frequency method for catchments of less than 4 km^2 of area. Both methods are among many other flood estimation methods applied in areas where a record of adequate flood data does not exist (NWASCO 1982).

The Rational method is an empirical one and is one of the earliest and best-known techniques to estimate peak flows (Ward 1975). It was developed primarily for use in the design of drainage systems and is still by far the most widely used method in that application (Body 1975). Body further pointed out that it is a venerable procedure where all the essential aspects of the method were set out by Mulvany in 1851, while Kuichling (1889) and Lloyd-Davis (1906) established the method in the United States and United Kingdom respectively.

Since its inception, the method has been amended over and over to suit the climatic and physical conditions of the area to which the method is to be applied. This improves the estimation results as they fail to perform well when used outside the area or conditions under which they have been derived (Ayoade 1988). The method has been applied widely in ungauged areas in PNG (SMEC 1983 [unpublished] and 1990), Australia (SMEC 1990), New Zealand (NWASCO 1982), China (Jiaqi 1987 and Body 1975) and in many other countries for the estimation of flood-flows for water-related engineering design purposes.

Regional Flood Frequency methods have also been applied widely. For example, in the United States (Pilgrim and Cordery 1993, Kirby and Moss 1987, Jarrett 1987 and Cohn and Stedinger 1987), British Isles (NERC 1975), New Zealand (NWASCO 1982), China (Hua 1987, Zhu 1987, Cong and Xu 1987) and in Australia and PNG (SMEC 1990). For ungauged catchments or where the flood data is inadequate, regional equations have been developed which enable the estimation of flood-flows, given the catchment area and rainfall data (NWASCO 1983). The regression relationship is applied in ungauged catchments of similar areal and climatic parameters to determine an estimated flood-flow. This method is one way of extending the data base from a number of sites to cover a region.

Both Rational and Flood Frequency methods require the catchment to be divided into sub-catchments. Each of the methods require certain catchment and climatic parameters for flood-flow estimations. In this study, the choice of method to use in a sub-catchment depends on the catchment area, i.e., the Rational method is applied in catchment areas of 1 km^2 to 4 km^2 and the Regional Flood Frequency method in catchment areas of over 4 km^2.

The flood-flows in this study were measured by applying the float technique, a technique which does have its disadvantages. Firstly, floats cannot be used to take at-point stream-flow velocity and secondly floats record only surface velocities, which are higher than the average stream-flow velocity (Ayoade 1988). Float measurements made carefully under favourable conditions may be accurate to within $\pm$ 10 percent. If a nonuniform reach is selected or too few floats are used, results may be in error by 25 percent or more (DSIRNZ 1988). However, in most studies under favourable conditions, the technique has been known to give reasonable results (NRC 1988).

1.3.2 **Karst Hydrology**

The scientific discipline of hydrology, although a long-established science, cannot easily be applied to karst regions with their very complex drainage systems. A special approach is therefore necessary to understand and predict water circulation in these areas (Bonacci 1987). The literature on water circulation processes in karst regions comes from the work of numerous researchers including Sweeting (1972), Arikan and Ekmekci (1985), Gunay (1985), Jennings (1985), Bonacci (1987) and Degirmenci and Gunay (1990). However, in general, the hydrology aspect has not been dealt with thoroughly and cannot be applied appropriately in practice. Therefore the circulation of water in karst is considered as a "black box" (Bonacci 1987).

Jennings (1985) pointed out that the number of streams are usually few in karstic regions, with a low drainage density because rivers entering karst or within it lose all or part of their volume, distinct from water infiltrating through soil, underground. The surface water in karst can gradually sink through a system of small joints and fissures, but sometimes part or whole of the surface water sinks underground through a great swallow hole or a system of several smaller swallow holes (Bonacci 1987). Jennings also found that frequently a stream has a series of karst swallow holes along its course into which it loses successive fractions of its volume.

Karst springs represent a natural exit for the underground water to the surface of the lithosphere through the hydrologically active fissures of the karst mass (Bonacci 1987). Sweeting (1972) found that streams could disappear into a swallow in one river basin but reappear through a spring in another. A spring could be either an exsurgence (from seepage through karst rocks) or a resurgence (reappearance of a former surface stream) (Jennings 1985). Bogli (1980) distinguished springs as: (i) perennial; (ii) periodic; (iii) rhythmically flowing, intermittent and flow springs; and (iv) episodic. Jennings (1985) pointed out that levels and so changed with time. In the Dinaric Karst in Yugoslavia, from the studies of two karst springs, Bonacci (1987) found that at very high ground water levels (after heavy rainfall), the springs were active, and their catchment areas significantly increased. At low ground water levels (after long dry periods), the watershed lines moved inwards or shrink as water is lost to springs so that their catchment areas were considerably decreased.

A list of references is given in Section 7 of the thesis so that the reader can pursue any further reading or particular topic pertaining to flood prediction and estimation in karstic catchments.

1.4 Background Information

1.4.1 Location

The Ok Mani stream catchment is located in the Western Province of PNG. It is approximately 9.2 km NNW of the Ok Tedi mining township of Tabubil and 4.1 km SW (approximate central point) of the Ok Tedi Gold and Copper Mine site (see Figure 1.1). The catchment is located at 141^0 13' S 5^0 21' E.

1.4.2 The Province

Western province is by far PNG's biggest province by land area of 99 300 km^2. Parts of the province are amongst the areas in PNG which receive the highest annual rainfall. Western Province has the country's biggest river by volume (Fly) and the biggest lake by surface area (Murray) (Maunsell and Partners 1982) (see Figure 1.1). For many years the province was considered to be the poorest (Boyden 1974) with least changes and developments. However, the Ok Tedi Gold and Copper Mine has helped to change this situation, including changes in the economic, physical and the social environments.

1.4.3 Physical Setting

The southern part of the province is mostly low-lying land and subjected to seasonal flooding. Many of these areas are either permanently or seasonally

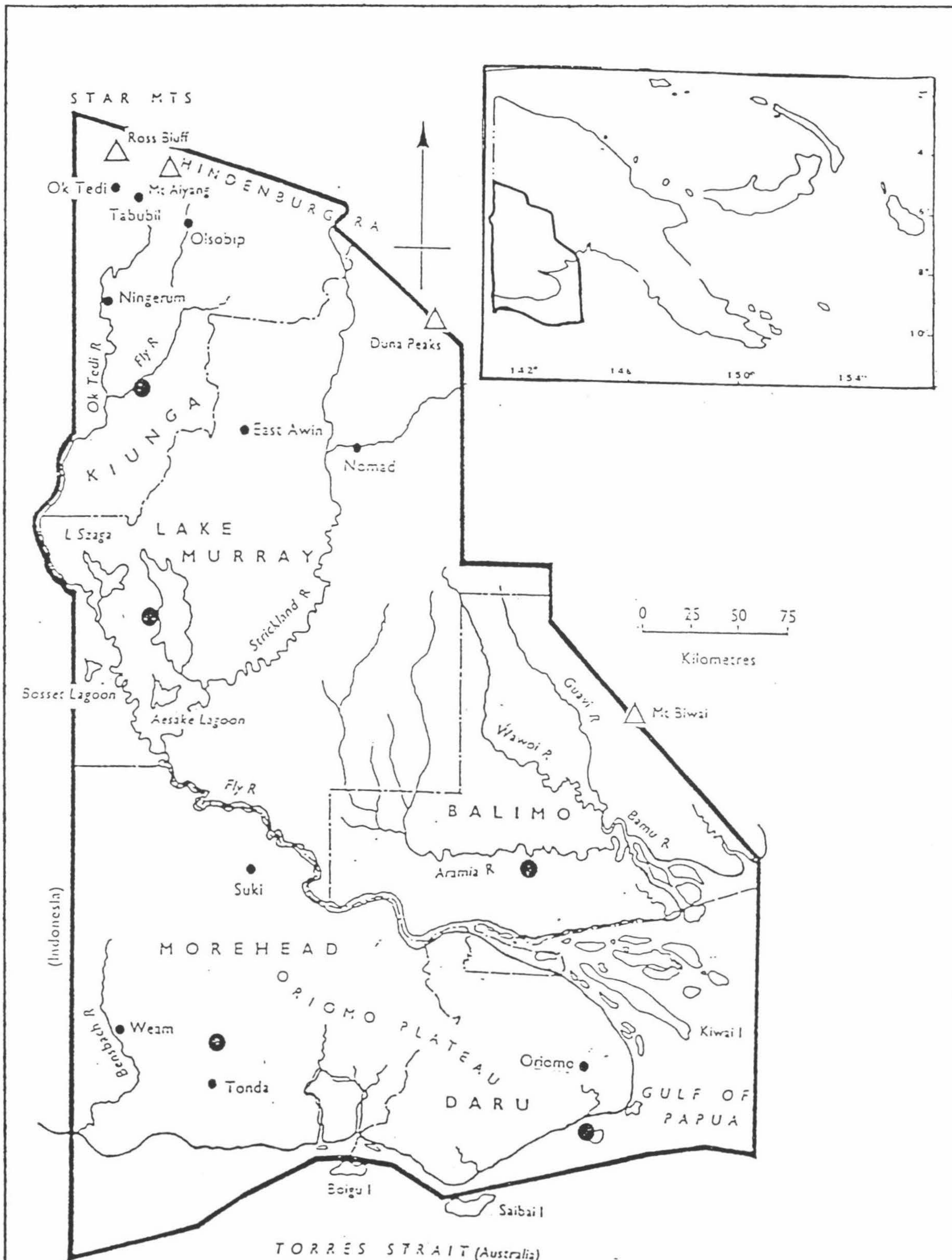


Figure 1.1: Location map: Western Province - Papua New Guinea

flooded. In the north, mountain ranges rise to over 3 500 m. Ok Mani catchment lies within the mountain ranges in the north-west of the province, about 1000 km inland from the Gulf of Papua and approximately 24 km east of the Irian Jaya (Indonesia)-PNG border (see Figure 1.1). It is characterised by rugged topography; that is, steep slopes, dense forest, and a high density network of fast flowing streams in narrow valleys.

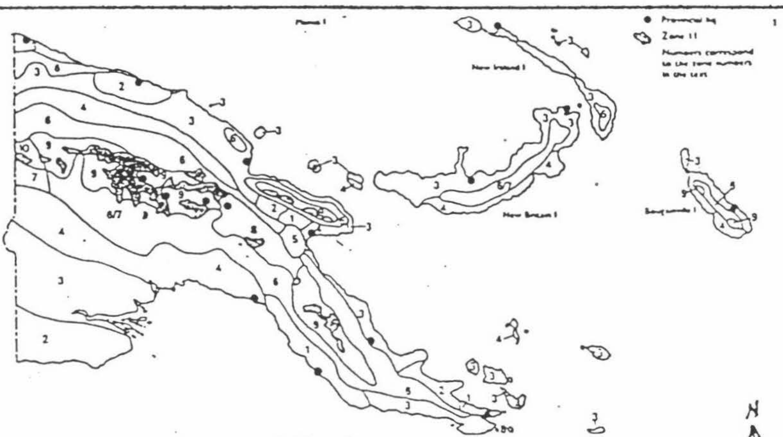
1.4.4 **Climate**

The climate of the Province and Ok Mani catchment is shown in Figure 1.2.1. It is divided into six climatic zones (types 2, 3, 4, 6, 7, and 10; see Rannells 1990).

Annual rainfalls in excess of 10 000 mm are recorded in the upper catchment in the mountain ranges in the north (Eagle and Higgins 1990). Monthly rainfall totals vary little throughout the year and there is very little seasonality (see Figure 1.2.2). Ok Mani catchment is located west of this region and has climate types 7 and 10 (see Figure 1.2.1). That is, the catchment has an annual rainfall of more than 3 500 mm. In contrast, in the southern part of the province, rainfall is influenced by the prevailing monsoons and trade winds which produce distinct wet and dry seasons (see Figure 1.2.2). This southern part has a high seasonality with a January-April maximum. Average annual rainfalls totalling 3 500 mm are recorded in these areas (Eagle and Higgins 1990).

1.4.5 **Vegetation**

The province has a mixture of environments and associated vegetation. Both upper and lower montane forest cover the higher lands in the north, foothills and low mountains. There are lowland fresh water swamps in the middle, and lowland alluvial plains and fans in the lower lands in the south of the province. Towards the coast, the province has saline and brackish swamps with beach ridges and flats along the coast (Paijmans [ed.] 1976). Ok Mani catchment is within the Lower Montane Zone. The catchment has a vegetation of mixed lower montane forest and swamp and palm forest with patches of mid-height lower montane grassland.



Climate types 1-4: up to 500 asl m
1. 1000-1500 mm of rain, May-November dry period
2. 1500-2000 mm of rain, June-October dry period
3. 2000-3500 mm of rain, variety of rain seasons
4. > 3500 mm of rain, May-October heaviest

(Source: Rannells 1990)

Climate types 5-7: 500 to 1400 m asl
5. 1500-2000 mm of rain, November-April slightly heavier
6. 2000-3500 mm of rain, December-March heaviest
7. > 3500 mm of rain

Climate types 8-10: 1400 to 3000 m asl
8. 1500-2000 mm of rain, December-April heaviest
9. 2000-3500 mm of rain, December-April slightly heavier
10. > 3500 mm of rain
11. areas above 3000 m asl, > 3000 mm of rain

Figure 1.2.1: Climate of Papua New Guinea

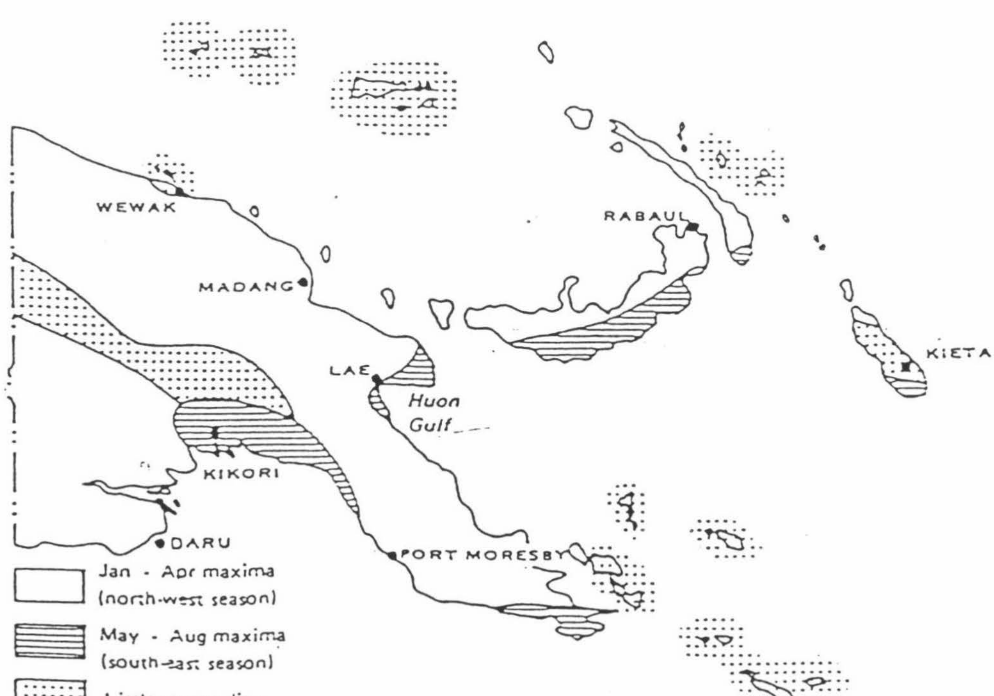


Figure 1.2.2: Seasonality of Rainfall (Source: SMEC 1990)

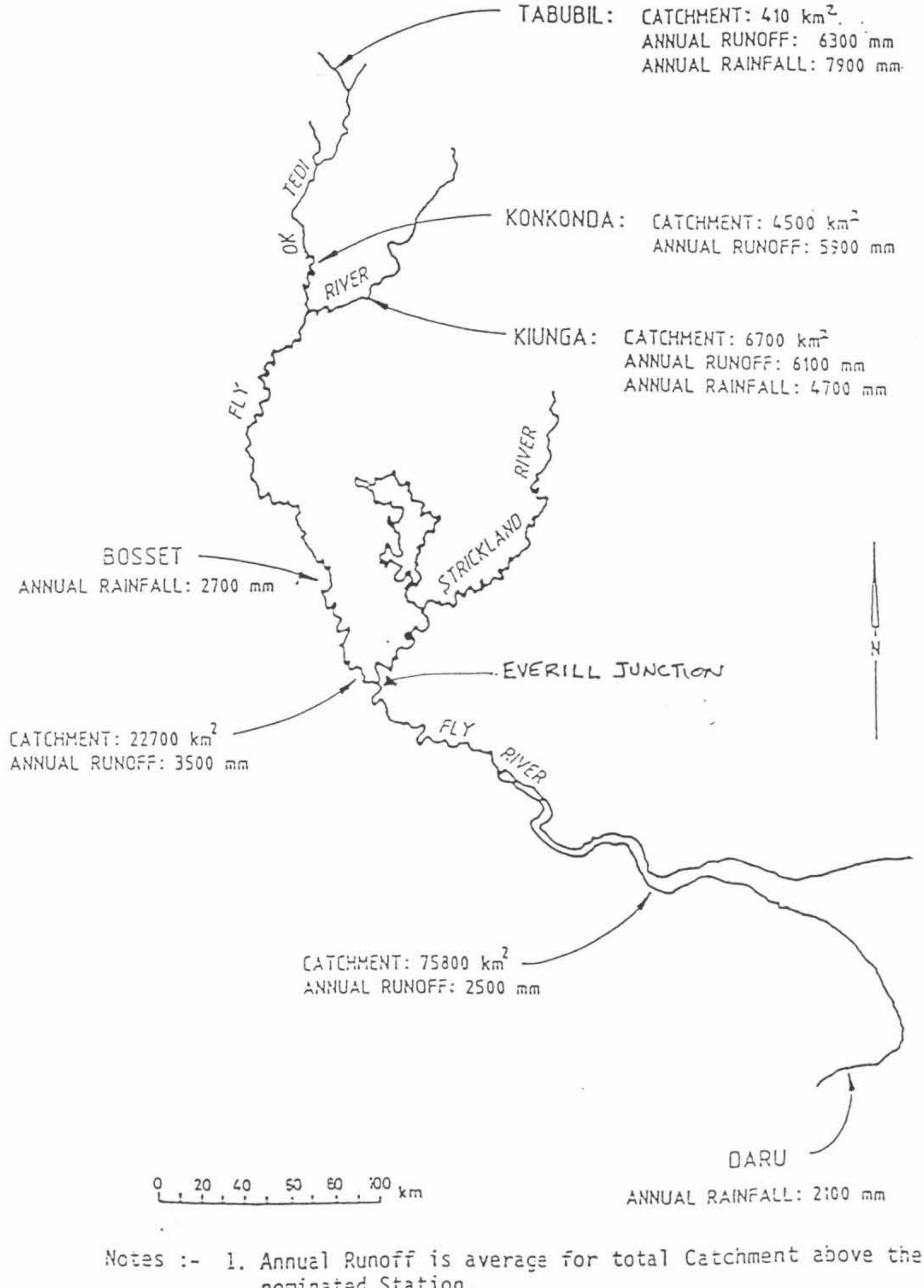
1.4.6 **Hydrology**

Ok Mani stream is one of the major tributaries of the Ok Tedi River which drains into the Fly River (see Figure 1.1). It has a catchment area of 68 km^2 and drains the area to the south and south-west of the mine. It is located within the Ok Tedi region which has a mean annual rainfall of 8 000 mm (maximum of 10 000 mm being recorded in some places in the region) and a very high annual runoff of 6 300 mm at Tabubil from a catchment area of 410 km^2 (see Figure 1.3) (Paltech 1979).

Mean daily flow of Ok Mani stream at MANO4 (OTML gauging station for the upper catchment area of 24.20 km^2) is 15 cumecs with the maximum recorded being 120 cumecs (OTML 1990 [unpublished]). Comparatively, the mean daily flow along the Fly River at Obo is about 2 400 cumecs and downstream of Everill Junction (see Figure 1.3) about 6 000 cumecs (Markham 1991).

Given a mean daily flow of 15 cumecs at MANO4 for the upper catchment of Ok Mani, the mean annual discharge is 1.296 x 10^6 m^3 and 473.04 x 10^6 m^3 per day and year respectively. With a catchment area of 24.20 km^2 x 10^6 m^2, the upper catchment from MANO4 would have an estimated mean annual depth of runoff of 19 550 mm, a figure which is some 2-3 times the annual rainfall.

Mean annual rainfall and runoff is compared with some other locations along the Fly River System in Figure 1.3. These figures are very high by world standards. For example, the most notable comparison is the Amazon River in South America. The average annual runoff is 900 mm with a catchment area of 7.18 x 10^6 km^2 (Maunsell and Partners 1982 [unpublished]). The average annual runoff from the Fly River is 2 500 mm downstream and 6 300 mm upstream at Tabubil (see Figure 1.3), which at these runoffs are respectively over 2.5 and 7 times greater than that for the Amazon River. The estimated depth of annual runoff of 19 550 mm for the upper Ok Mani catchment, is over 22 times more than that of the Amazon River. The extremely high and persistent rainfall in the upper catchment of the Fly River in the north explains these runoff characteristics.



Notes :- 1. Annual Runoff is average for total Catchment above the nominated Station.
2. Annual Rainfall is average at the nominated Station.

Figure 1.3: Catchment Areas, Rainfall, Runoff and the Fly River System

(Source: Paltech 1979)

1.4.7 **Geomorphology**

Ok Mani catchment is characterised by an extensive network of both perennial and intermittent streams between steep slopes with narrow valleys. The upper catchment from Harvey Creek (see Figure 1.4) is relatively steep with the lowest and highest elevation of 640 m and 1 560 m asl respectively within a straight line surface distance of 5 km. This gives an average valley slope of 184 m/km. The main stream flows through a narrow channel varying from 5 m to 10 m in width. Ok Mani channel is mostly boulder-bedded along this reach with no major evidence of material deposition (see Plate 1.1).

The downstream channel to the confluence at Ok Tedi River is a braided stream between 100 m to 350 m wide (see Figure 1.4). The channel in this reach is between an elevation of 400 m and 600 m asl within a straight line distance of 11 km (slope of 18.2 m/km) and is flat and wide compared with the channel upstream. The stream is cobble- to gravel-bedded (see Plate 1.2) and establishes a meandering pattern before it joins the Ok Tedi River. The dominant flow channel within this reach seem to change after storms through sediment transportation and deposition (see Plate 1.2). That is, bed material is eroded during high flows and deposited at low flows, the latter blocking existing channels, thus forcing the water to be distributed to flow in numerous channels and into new areas creating new flow channels and meanders throughout the braided channel.

Most or all the material transported from upstream is deposited in the lower reach as the flow decreases. The decrease in flow and the subsequent deposition of material is due the decrease in flow velocity and volume as a result of the decrease in channel gradient, widening, and meandering from aggradation, thus the stream's capacity or energy to transport decreases. The aggradation in this reach (see Figure 1.4 and Plate 1.2) resulted from the failure of the mine overburden dump in the Harvey Creek catchment, a tributary of Ok Mani stream, after the first year of dumping in 1991.

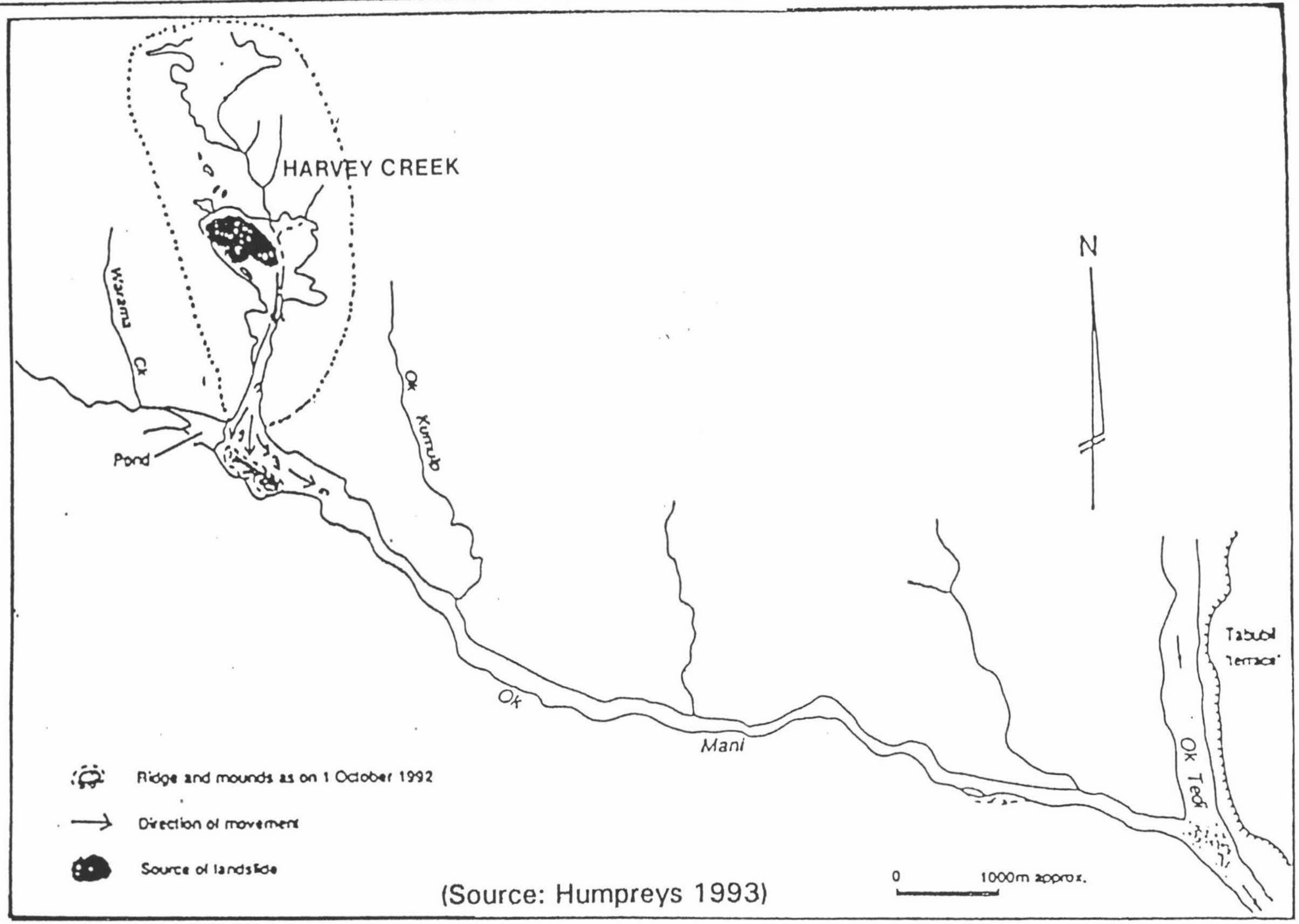


Figure 1.4: Lower Ok Mani braided channel: Harvey Creek catchment and the landslides

Plate 1.1: Upper Ok Mani stream: gravel- to cobble-bedded channel

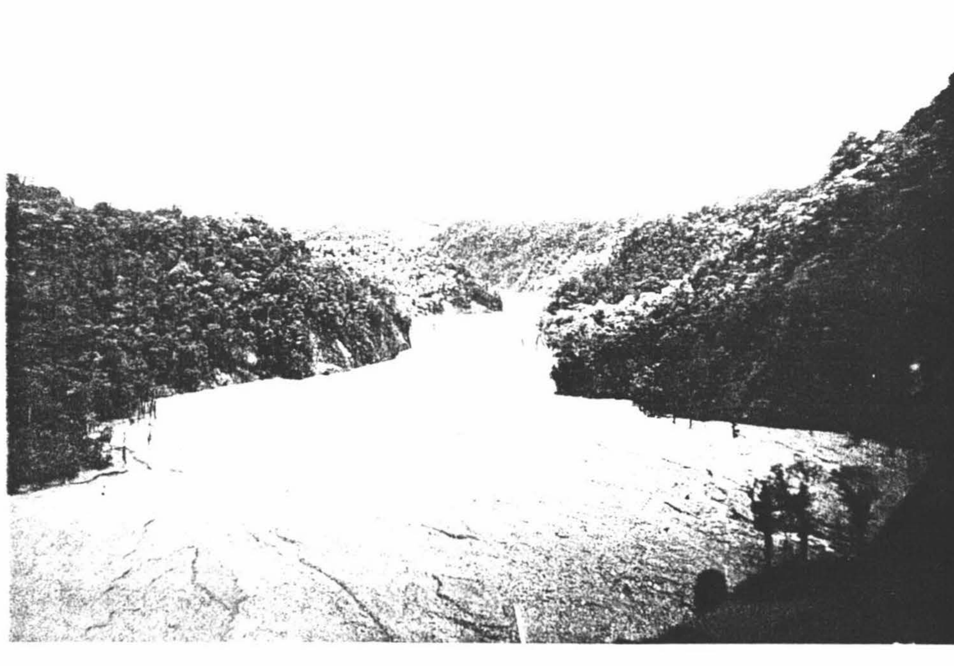


Plate 1.2: Lower Ok Mani stream: gravel-bedded and braided channel

A study carried out by Humphreys in 1993 (unpublished) in the Harvey Creek catchment found that there was a large slope failure within the catchment resulting in a massive landslide (see Figure 1.4 and Plate 1.3). The channel than adjusted to new conditions from the increase in sediment load added to and transported through the catchment. The sediment budget by Humphreys to 1991 (i.e., over the first year of dumping) showed that about 58 percent of the total rock waste added to the catchment was derived from the valley sides and the channel. In other words, 7.6 x 10^6 m^3 of mine-waste induced and accelerated the supply of a further 9.9 x 10^6 m^3 of material from within catchment into Ok Mani. Plate 1.4 shows the fan and pond upstream of Ok Mani and Harvey confluence as a result of the failure and increased sediment supply. The fan blocks the Ok Mani stream resulting in the pond upstream, thus forcing and confining the flow to the south (right of Plate 1.4) of the channel.

1.4.8 **Geology**

The geology of the Province, including Ok Mani and the adjacent catchments is shown in Figure 1.5. The geology of Ok Mani catchment is mainly silts (fine particles larger than those of clay but finer that than those of sand); and marls (a calcareous clay or mudstone with an admixture of calcium carbonate) in the high piedmont. Tertiary limestone extends to the north, north-west and north-east of the catchment (Loffler 1977). To the south are quaternary volcanic pyroclastic materials, both stream and air borne (Maunsell and Partners 1982).

1.4.9 **Ok Tedi Mine**

The Ok Tedi Mining project is a substantial undertaking by world standards by OTML, a subsidiary of Broken Hill Proprietary of Australia. It is one of the largest open-cut copper and gold mines in the world and commenced operation in May 1984. The project involves the mining of Mt. Fubilan's (in the Star Mountains, see Figure 1.1) 500 million tonnes of porphyritic chlorite/monzonite ore containing 880 grams of copper and 0.66 grams of gold per tonne. It is located in one of the wettest areas of the world with an annual rainfall of more than 8 000 mm (Connell and Howitt 1991), about 2000 m asl and in the head-water catchment area of the

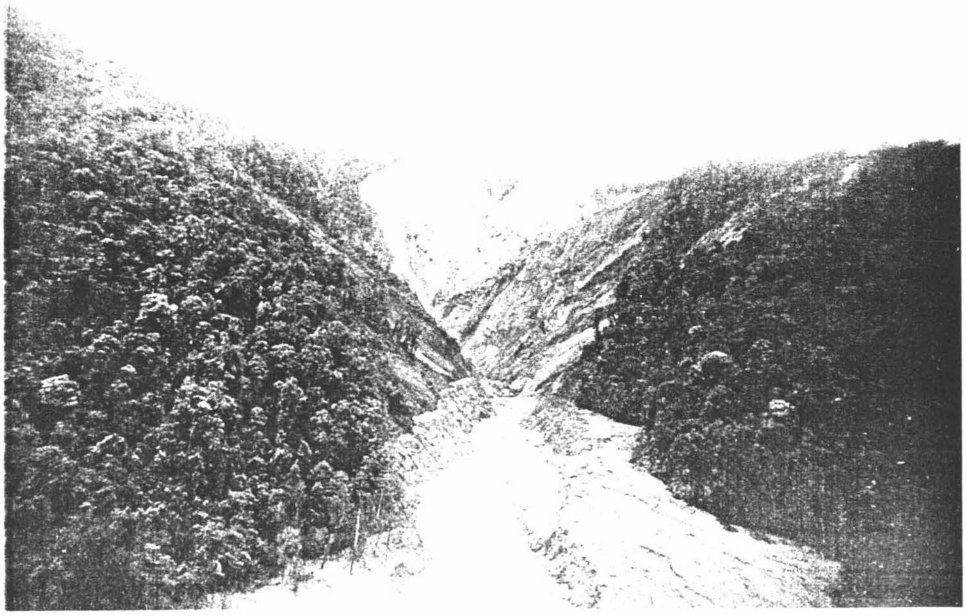


Plate 1.3: Landslides in Harvey Creek catchment due to dumping of mine overburden. Turbid streams, evident of high sediment transport

Plate 1.4: The fan and pond at Ok Mani stream and Harvey creek caused by failure of dump

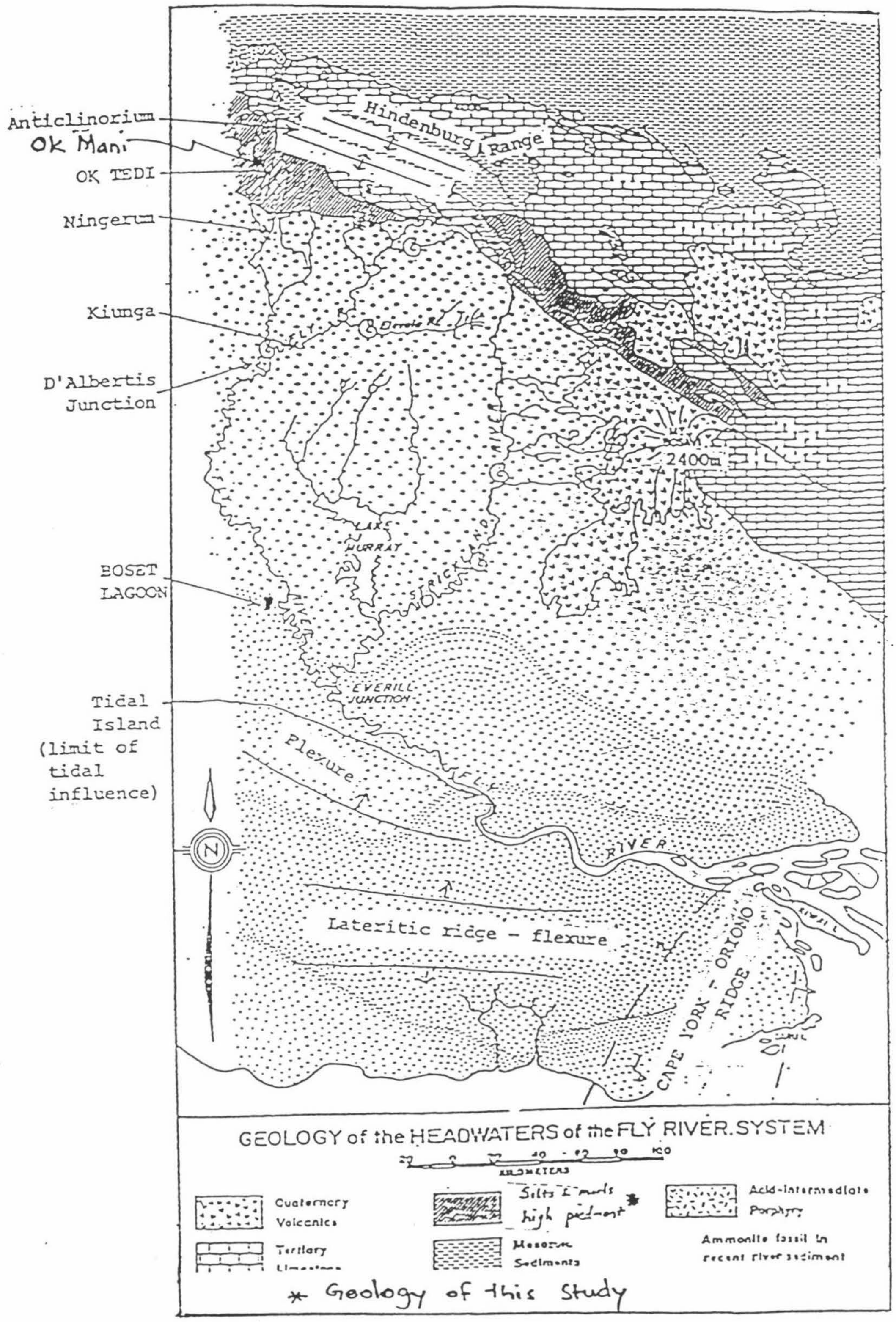


Figure 1.5: Geology of Western Province, PNG

(Source: Paltech 1979)

Fly River system, some 1 000 km inland from the Gulf of Papua (see Figure 1.1).

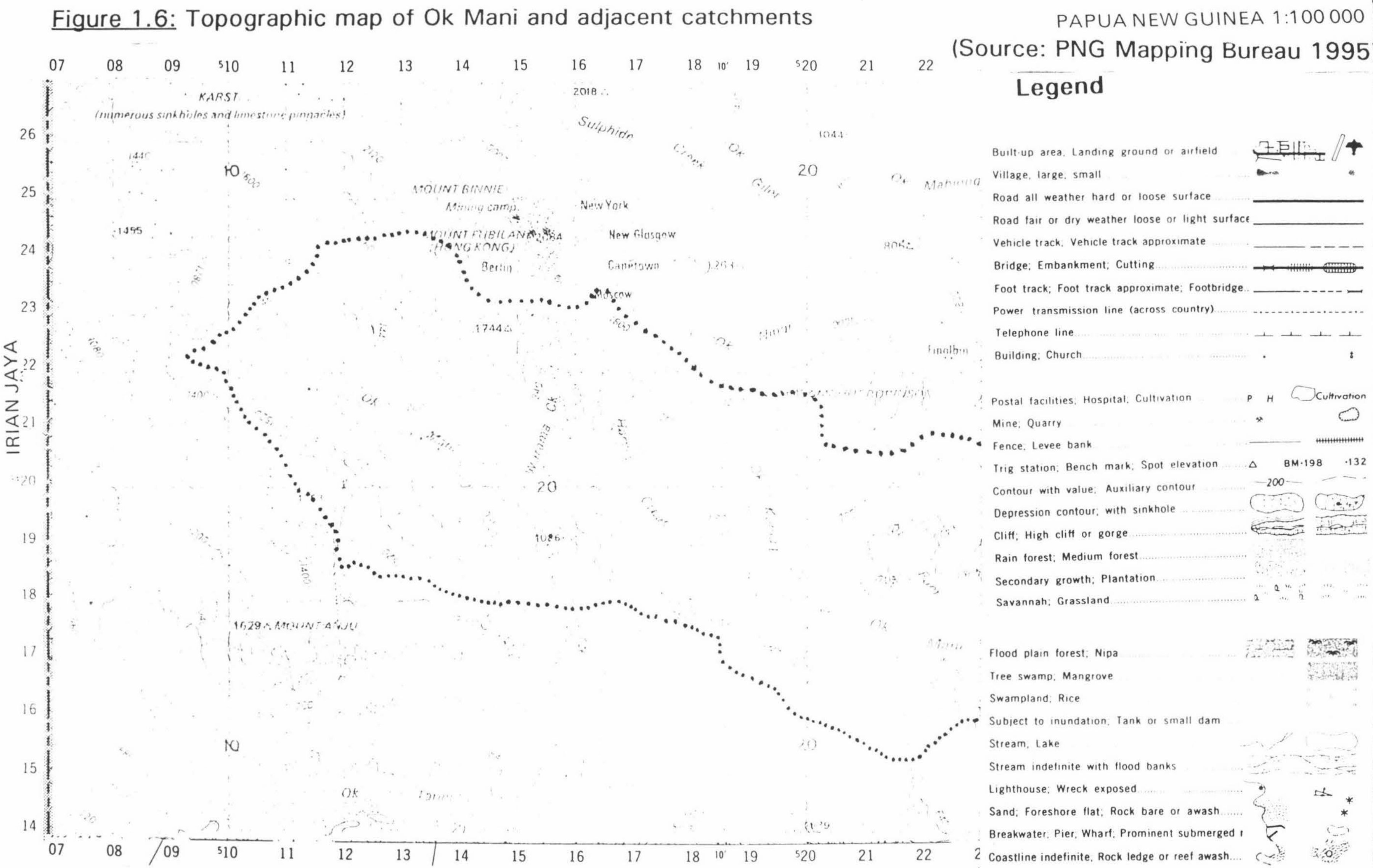
Since the mine's commencement, OTML has progressively removed and hauled the overburden to erodible waste rock dumps north of the mine site in the headwaters of the Ok Tedi River (Northern Dump). In 1991, OTML progressed to storing the overburden in the Harvey Creek catchment south of the mine site (Southern Dump) (see Figure 1.4). This dump failed as discussed in Section 1.4.7. The overburden in eroding dumps is incompetent and readily breaks down to fines. Eagle and Higgins (1990) found that approximately 55 percent of the overburden breaks down to a size of less than 100 microns during transport in the Ok Tedi River, a size which is readily transportable in normal flow conditions as suspended sediment.

1.5 Preliminary Observations

Ok Mani stream is shown in Figure 1.6 and is one of the many highland streams found in the Ok Tedi region. One common characteristic of these streams is that they are fast flowing in narrow channels with high sediment load during floods. OTML spend millions of Kina (PNG's currency) monitoring the main rivers and streams in the region, including Ok Mani, Ok Tedi and the Fly River. It has installed and monitored self-recording flow and rainfall gauges and maintains a computerised data base in the Environment Department.

During the field investigations, it was observed that the streams in the upper catchment from Harvey Creek (see Figure 1.4), during normal flow are clear, with evidence of no or very little (if any) sediment transport. During and after flood events, the streams are turbid, evidence of high sediment transport due to erosion from surface flows in the catchment. Downstream from Harvey Creek to the Ok Tedi confluence, the streams to the north of the channel are always turbid (see Figure 1.4 and Plate 1.3). The high sediment concentrations in these streams are derived from the landslides and other forms of slope instability within the various sub-catchments, a common characteristic of the region.

Figure 1.6: Topographic map of Ok Mani and adjacent catchments



It was also observed that the soil within the catchment is always saturated as it rains every day (mostly heavy and for long duration) and that the dense vegetation cover prevents or reduces evaporation (inspite of the theoretical fact that the denser the vegetation, the greater the evapotranspiration). During rain events, rain was observed to fall first in the higher lands and to gradually spread to the lower land. The steep topography of the catchment promotes immediate surface runoff during a storm as observed during the field investigation.

The Ok Mani catchment is more-or-less surrounded by karst topography, especially to the north and north-west with numerous sinkholes (see Figure 1.6). The boundary of Ok Mani and the adjacent catchment immediately between north and north-west is not definite in that they "overlap" (see Figure 1.6, approximate central location at 05:25:50 S 141:11:50 E).

A number of springs are evident along the Ok Mani channel (see Plates 1.5 and 1.6 for examples). Some of these, from observations, contribute substantial amounts of water to the main stream. The springs, although there is no proof nor have measurements been carried out on them, could be the emergence of sub-surface flows from the adjacent catchment's karstic sinkholes through underground flow channels.

Plates 1.5 & 1.6: Springs: examples of underground springs along Ok Mani channel, possib ly the emergence of the streams sinking in the karst sinkholes in the adjacent catchments

Chapter 2

FLOOD-FLOW ESTIMATION METHODOLOGY

2.1 Methods

2.1.1 Rational Method

The Rational Method applied in this rainfall-runoff anomalies investigation is that adopted and used by Shaw (1991), SMEC (1990), Ayoade (1988) and many others, and is in the form:

$$Q_P = C\ I\ A \tag{2.1}$$

where, Q_P is peak flow (m^3/s), C is a runoff coefficient, I is storm rainfall intensity (mm/hour) and A is catchment area (km^2).

To apply the Rational Method requires measurement and knowledge of the following physical parameters of the catchment under investigation. They include area (km^2), mainstream length (km), slope of main channel (% and m/km) and mean elevation (derived from topographic maps). It also requires the point rainfall depth (mm) (P_2) and duration (t) of the corresponding storm.

The rainfall intensity for the storm is derived as follows:

$$I = D/t \tag{2.2}$$

where, I is rainfall intensity [mm/hour], D is total rainfall depth (mm) of storm and t is duration of storm (hours).

With the catchment physical parameters measured, the runoff coefficient (C_2) is computed from equation 2.3.

$$C_2 = 0.39 - 0.0006\ SLOPE - 0.036\ \ln ELVN + 0.03\ \ln WIDTH \qquad (2.3)$$

where, ln is natural logarithm ($\log_e$), SLOPE in m/km, ELVN (mean elevation) in m asl and WIDTH in km. (Width of the catchment is defined as catchment area (km^2) over the length (km) of main stream channel). In all cases, a minimum value of $C_2 = 0.10$ and a maximum value of $C_2 = 0.40$ is adopted. That is, any calculated runoff coefficient less than the minimum value is assigned a value of 0.10. Likewise, any calculated runoff coefficient exceeding the maximum value is assigned a value of 0.40.

Finally, the estimated peak flood-flow is computed from equation 2.1.

2.1.2 **Regional Flood Frequency Method**

The Regional Flood Frequency Method applied in this rainfall-runoff anomalies investigation is that developed and used by SMEC (1990), and is in the form:

$$Q_P = 0.028\ A^{0.70}\ P_2^{1.12}\ KS \qquad (2.4)$$

where, Q_P is estimated flood-flow (m^3/s), P_2 is adjusted point storm rainfall (mm) with an annual series return period of 2 years, A is catchment area (km^2) and KS is the swamp adjustment factor.

The Regional Flood Frequency Method involves extracting three catchment parameters to estimate the peak flood-flow. These include area (km^2), point rainfall depth (mm) (P_2) of the storm and the swamp adjustment factor, the value 1.0 plus the decimal proportion of swamp or flood prone land along the main river channel.

Finally, the estimated peak flood-flow is computed from equation 2.4.

2.2 Assumptions

The peak flood-flow estimation methods discussed in Section 2.1 and applied in this study involved a number of assumptions. Discussed below are some significant ones involved.

Firstly, the methods use a number of climate and physical catchment parameters (see Sections 2.1.1 and 2.1.2 for parameters involved) to compute the flows. However, runoff from a catchment is determined and influenced by a combination of different climate and physical catchment parameters including those mentioned. Wisler and Brater (1959) listed and considered type of precipitation, distribution of rainfall on basin, direction of storm movement, antecedent precipitation and soil moisture, temperature, humidity, wind, land-use and soil type, orientation and drainage and stream network to be equally important and to have an effect on the magnitude of the flow in a river/stream. The estimation methods were applied in this study assuming on the assumption that the catchment parameters including area, rainfall and flood data, and others that were used to derive the runoff coefficients (Rational and Regional Flood Frequency) are similar to those catchments (i.e., sub-catchment C1, C2 and C3) to which the methods were applied to estimate the peak flood-flows.

Secondly, it was assumed that the storm rainfall, duration and intensity gauged and used to compute the estimated peak flood-flows were uniform throughout the storm and representative of the whole catchment area. That is, the storm rainfall started and ended at the same time and the rain fell at the same rate over the entire area of the catchments.

2.3 Catchment Physical Parameters

2.3.1 Catchment Area

Ok Mani stream has a total catchment area of 68.82 km^2. The upper catchment in which this investigation was undertaken has an area of 24.20 km^2. The catchment was divided into five sub-catchments, C1, C2, C3, C4 and C5 (see Figure 2.7), i.e., each upstream sub-catchment is 'nested' into the next downstream sub-catchment. In other words, sub-catchment C1 is a single catchment, C2 includes C1, C3 includes C1 and C2, C4 includes C1, C2 and C3 and C5 includes C1, C2, C3 and C4, i.e., the whole area of investigation.

However, during the field work, storm-flow gauging in sub-catchments C4 and C5 were not done as there were no storms during the daytime when the sites were visited on 17.03.95 and 19.03.95 respectively. The storms occurred in the night and they were impossible to gauge as it was risky and hazardous. The discussion presented in this thesis only refers to the upper-most catchment area of 9.66 km^2, that is sub-catchments C1, C2 and C3 (see Figure 2.7).

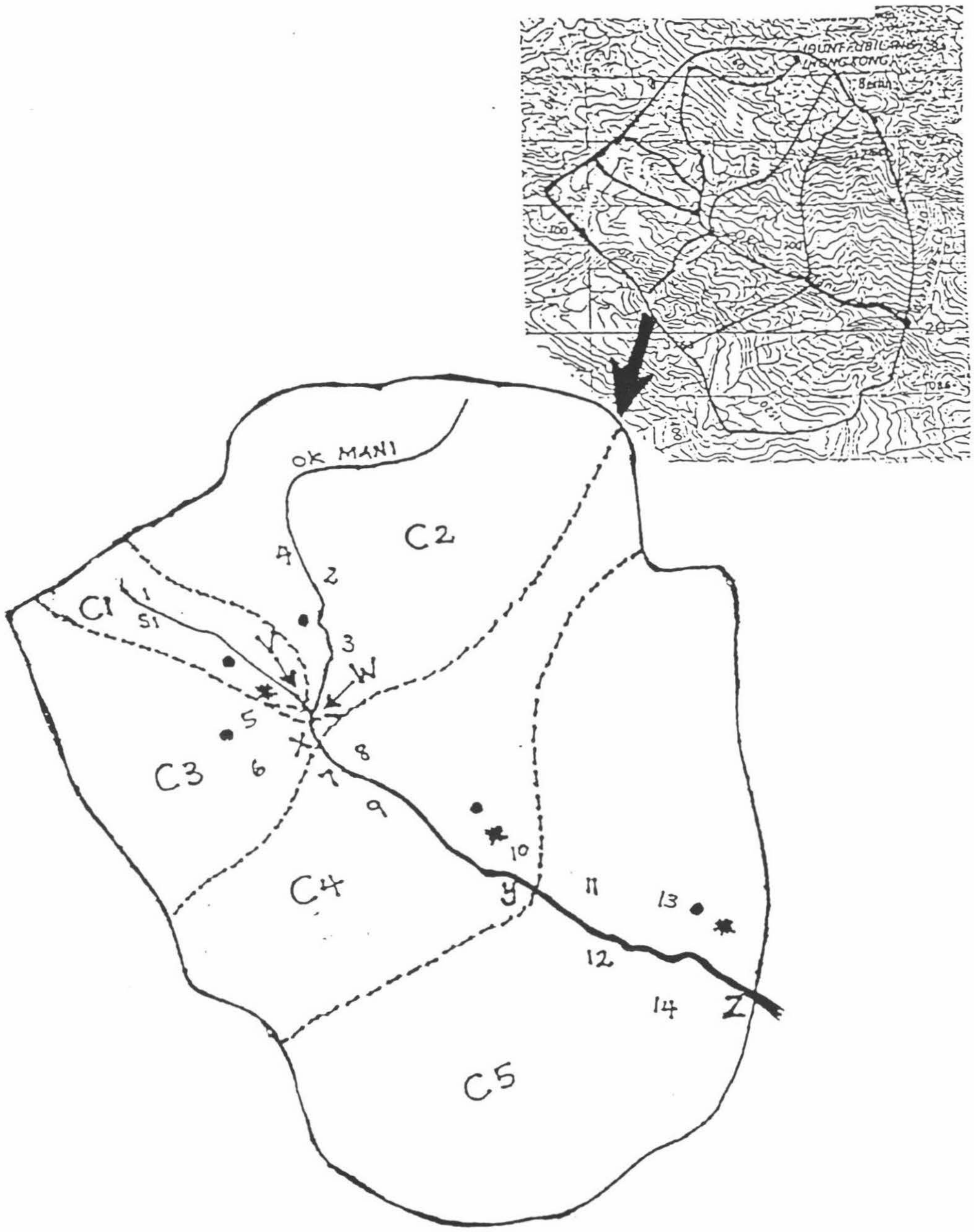
The area of the sub-catchments were measured with a Planimeter and calculated using the Test-square Proportion technique (see Appendix I.1). The measurement method and the calculation procedures are given in Appendix I.2. The area of the sub-catchments calculated are presented in Table 2.1.

Table 2.1: Sub-catchments Physical Parameters

PARAMETERS	C1	C2	C3
Area (km^2)	1.01	7.06	9.66
Stream Length (km)	2.5	4.2	5.0
Mean Elevation (m)	1380	1720	1700
Slope (m/km) [%]	112 [11]	210 [21]	200 [20]
Swamp Factor (KS)	1.00	1.00	1.00
C_2	0.10 *	0.14	0.14

* assigned as a minimum value

2.3.2 **Stream Length**



C1...C5 sub-catchments
1....14 emerging springs
W.....Z gauging points
. raingauge installation site
* camping sites
------ sub-catchment boundaries

Figure 2.7: Ok Mani upper-most catchment of investigation

(figure not to scale)

The length (L) of the main streams in the sub-catchments were measured using a cartometer. The cartometer was set to zero and placed at one end of the main stream on the map and pushed along the streamline to the other end in the sub-catchment. The stream length (cm) was read from the dial of the cartometer.

Using the map scale (1:100 000), the cartometer measurements (cm) were converted into actual stream length distances (km). Because the upstream sub-catchment is nested into the one downstream, the stream length (km) for the downstream sub-catchment is the cumulative of the main stream length (km) measured in that downstream sub-catchment and the stream length measured in the upstream sub-catchment. For instance, main stream length for sub-catchment C3 is the cumulative of mainstream length measured in sub-catchment C3 and C2. (Note that when sub-catchment C1 was nested into sub-catchment C2, the Ok Mani stream in sub-catchment C2 is taken as the main stream and not the stream in sub-catchment C1) (see Figure 2.7). The measured stream lengths are presented in Table 2.1.

2.3.3 **Channel Slope**

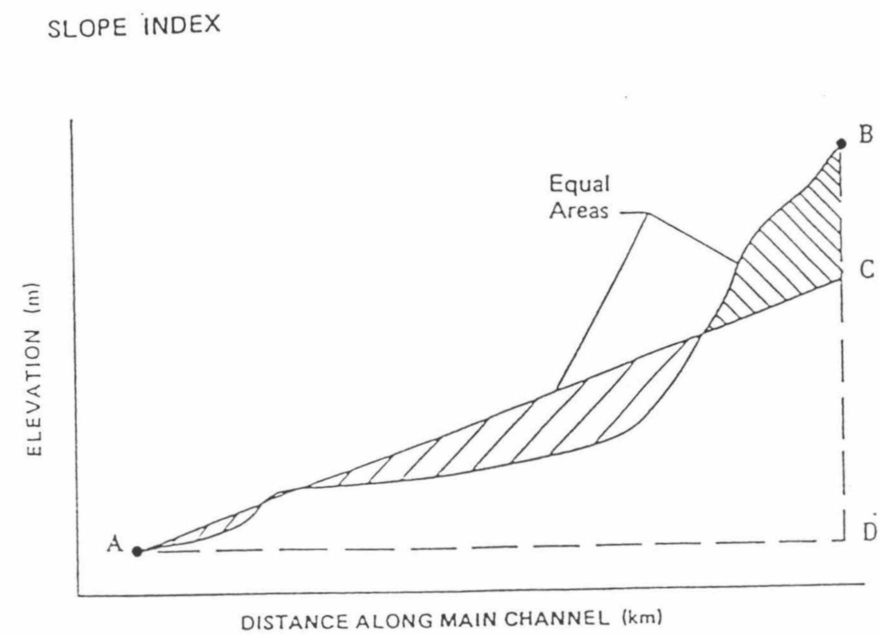
The method for mean channel slope calculation is given in Figure 2.8. The difference in elevation (m) is divided by the stream length (km), that is, the distance along the main channel. The slopes calculated are expressed as metres per kilometre and as percentage and are presented in Table 2.1.

2.3.4 **Swamp Factor**

The method of adjusting a swamp factor (KS) is shown in Figure 2.9. From the 1:100 000 topographic map (see Figure 1.6) there are no swamps or flood-prone land shown along the main stream channels. Therefore, the swamp adjustment factor is 1.00 (see Figure 2.9) for the sub-catchments (see Table 2.1).

2.3.5 **Mean Catchment Elevation**

Within the boundaries of the sub-catchments and from the contour lines given on the topographic map in Figure 1.6, the lowest and the highest elevation (m) were



SLOPE = $\frac{CD}{AD}$ (m/km) = $\frac{CD}{AD} \times 0.1$ (%)

LENGTH = AD (km) along main channel (Source: SMEC 1990)

Figure 2.8: Channel slope calculation method

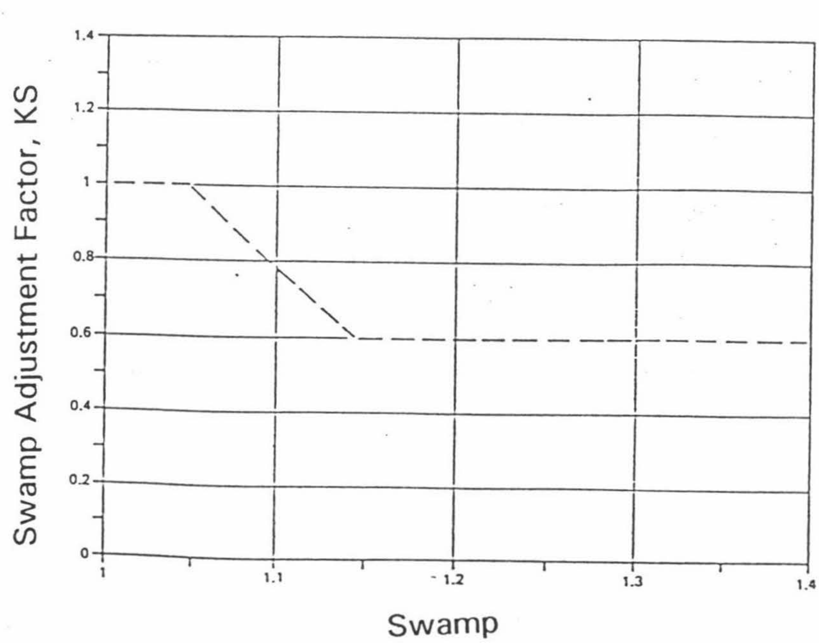


Figure 2.9: Swamp factor adjustment method

(Source: SMEC 1990)

established. In all cases, the two elevations were summed and divided by two to obtain a mean elevation for the sub-catchment. The mean elevation (m asl) calculated for the sub-catchments are presented in Table 2.1.

2.3.6 Point Rainfall and Intensity

Storm rainfall intensities were calculated from equation 2.2. The time the storm began and ended were noted. To calculate the rainfall intensity (I) (mm/hour) of a storm, the amount of rainfall (mm) collected by the raingauge in the sub-catchment over the storm event was divided by the corresponding storm duration (hours). The gauged rainfalls, durations and the computed intensities are presented in Table 2.2.

2.3.7 Hypsometric Adjustment of Rainfall

Depth of catchment rainfall vary spatially and altitudinally, i.e., rainfall decreases from the centre of the storm with increasing catchment area and increases with increasing elevation. Therefore, rainfall gauged at one location of the catchment is not always representative of the catchment rainfall. However, the part of Ok Mani catchment investigated is so small that any spatial constraint would be negligible. Altitudinal variation of rainfall in the catchment seemed significant and therefore the gauged storm rainfall were adjusted relative to elevation as outlined below.

The monthly mean rainfall for June 1994 for 4 stations at different elevations within the Ok Mani catchment are presented in Table 2.3. The areas between the contours of the stations on the topographic map was planimetered and computed as a percentage of the total catchment area. The elevation (m) at each rainfall station was also computed as a percentage. An areal rainfall was calculated for each station by multiplying the monthly mean rainfall and the percentage area and presented in Table 2.3. Finally, an hypsometric curve was constructed by plotting the cumulative percentage of catchment area against increasing elevation in percentage presented in Figure 2.10.

Table 2.3: Ok Mani Catchment Areal Rainfall - 1994

Station Name	Limiting Contours Elevation (m)	% Elevation of 2240 m asl	Catchment Area Between Contours (km2)	% Area of Total	Cumulative Area (%)	Area below Station (%)	Monthly Mean Rainfall Jun-94 (mm)	Areal Rainfall (R/fall x Area)/100 (mm)	Rainfall Factor Relative to that Gauged during Field-work at 1200 m asl
Tabubil R1	540 - 660	3.53	11.97	17.39	17.39	8.70	40.89	7.11	0.05
Kumkit R2	660 - 780	10.59	8.25	11.99	29.38	23.40	49.63	5.95	0.15
Ok Mani R3	780 - 900	17.65	25.66	37.29	66.67	48.00	65.97	24.60	0.25
Falomian R4	900 - 2240	76.47	22.94	33.33	100.00	83.30	75.00 *	25.00	1.08
			Total: 68.82				Mean: 57.87	Total: 62.66	1.87 **

* Rainfall value extrapolated by plotting the catchment rainfall against elevation and using the plot, the mean of rainfall at 2240 m and Ok Mani (R3) was obtained as the rainfall for Falomian (R4)

** Highest elevation of catchment at 2240 m asl

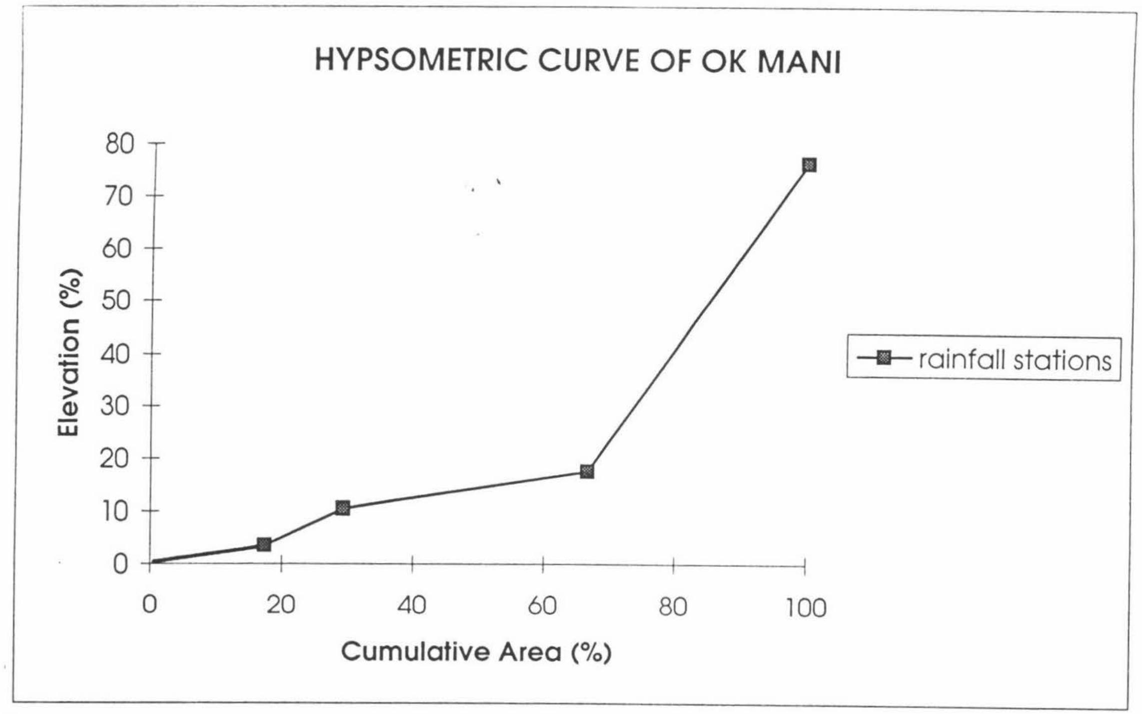


Figure 2.10: Ok Mani catchment Hypsometric Curve

The areal rainfall and the hypsometric curve show that rainfall increases with increasing elevation in the catchment. Therefore, the storm rainfalls gauged at 1200 m asl over the fieldwork period presented in Table 2.3 were adjusted. The rainfall adjustment factors were calculated by taking the elevation 1200 m asl at which the storm rainfall were gauged as a factor equivalent of 1.0 and the elevations of the rainfall stations (R1, R2, R3 and R4, see Table 2.3) were expressed over 1200 m asl. The adjustment factors were then plotted on a graph of catchment elevation (m) against mean monthly rainfall (mm). The rainfall adjustment factors for other elevations were then extrapolated from the plot.

Finally, a mean rainfall adjustment factor was derived from the factors at 1300 m asl of rainfall station R4 and 2240 m asl (see Table 2.3). The gauged rainfalls were then adjusted by this mean factor and are presented in Table 2.2.

2.4 Estimation Method Applicable

The choice of either the flood-flow estimation methods, Rational and Regional Frequency, for a catchment depends on a number of factors and guidelines, including the catchment area, whether the catchment is rural or urban and whether the estimated flow is sufficient or a complete flood-flow hydrograph is required. However, since, SMEC (1990) used area as the main physical catchment factor to derive the methods, the method selected to use in the sub-catchment(s) in this investigation depends on the sub-catchment area. The sub-catchments and the estimation method applicable for the flood-flow estimation are listed in Table 2.4. The Rational method is applied in catchments of area < 4 km^2 while the Regional Flood Frequency method is used in catchments of area ≥ 4 km^2.

2.5 Estimated Flood-flow Calculation

2.5.1 Rational Method

The Rational method is applicable only in sub-catchment C1 in the form of

Table 2.4: Sub-catchments and Estimation Method Applicable

SUB-CATCHMENT	AREA ($km^{2)}$	ESTIMATION METHOD
C1	1.01	Rational
C2	7.06	Regional Flood Frequency
C3	9.66	Regional Flood Frequency

equation 2.1. The estimated peak flood-flows computed (after adjustment of rainfall) are presented in Table 2.5. As an example, the estimated peak flood-flow for sub-catchment C1 on 02.03.95 was computed as:

i. Sub-catchment physical parameters: Area (A) = 1.01 km^2; Mainstream Length (L) = 2.50 km; Slope (S) = 112 m/km = 11.2 % and mean Elevation = 1 380 m asl.

ii. Point rainfall intensity (I) from equation 2.2:
I = 56.24 mm/4 hours = 14.06 mm/hour

iii. Runoff coefficient (C_2) from equation 2.3:

$C_2 = 0.39 - (0.0006 \cdot 112) - (0.036 \cdot \ln 1380) + (0.03 \cdot \ln[1.01/2.5])$

$= 0.035 = 0.10$ (minimum value)

ii. The estimated flood-flow from equation 2.1:

$Q_P = 0.10 \cdot 14.06 \cdot 1.01$

$= 1.42\ m^3/s$ (2 decimal places)

2.5.2 **Regional Flood Frequency Method**

Regional Flood Frequency Method is applicable in sub-catchments C2 and C3 (see Table 2.4). The estimated peak flood-flow in the sub-catchments are computed in the form of equation 2.4. The estimated flood-flows computed (after

Table 2.5: Ok Mani Estimated and Peak Flood-flows

Date	Sub-catchment	Gauging Point	Catchment Area (km2)	Assuming no Evaporation				Assuming Evaporation of 4.0 mm Daily		
				Estimated Peak Flood-flow (m3/s)	Gauged Peak Flood-flow (m3/s)	Difference in Flows (m3/s)	Difference in Flows (%)	Estimated Peak Flood-flows (m3/s)	Difference in Flows (m3/s)	Difference in Flows (%)
02.03.95	C1	V	1.01	1.42	1.98	0.56	28.28	1.40	0.58	29.29
	C2	W	7.06	10.03	10.34	0.31	3.00	9.90	0.44	4.26
	C3	X	9.66	12.49	14.46	1.97	13.62	12.32	2.14	14.80
03.03.95	C1	V	1.01	0.50	0.60	0.10	16.67	0.48	0.12	20.00
	C2	W	7.06	1.03	2.12	1.09	51.42	0.99	1.13	53.30
	C3	X	9.66	1.29	2.98	1.69	56.71	1.24	1.74	58.39
04.03.95	C1	V	1.01	4.19	4.90	0.71	14.49	4.17	0.73	14.90
	C2	W	7.06	15.49	19.20	3.71	18.83	15.42	3.78	19.69
	C3	X	9.66	19.29	21.29	2.00	9.39	19.20	2.09	9.82
15.03.95	C1	V	1.01	2.17	5.21	3.04	58.34	2.15	3.06	58.73
	C2	W	7.06	7.41	21.32	13.91	65.24	7.35	13.97	65.53
	C3	X	9.66	9.23	22.50	13.27	58.97	9.15	13.35	59.33

V gauging point in sub-catchment C1
W gauging point in sub-catchment C2
X gauging point in sub-catchment C3

adjusted of rainfall) are presented in Table 2.5. As an example, the estimated peak flood-flow for sub-catchment C3 on 02.03.95 was computed as:

i. Sub-catchment physical parameters: Area = 9.66 km^2; P_2 = 38.33 mm; Slope = 200 m/km = 20 %; Karst Factor = 0 %; and Swamp Factor = 1.0.

ii. The estimated flood-flow from equation 2.4 is:

$$Q_P = 0.028 \cdot 9.66^{0.70} \cdot 56.24^{1.12} \cdot 1.0$$
$$= 12.49 \ m^3/s$$

Table 2.5 also present the results of the estimated flood-flows assuming evaporation has taken place during the storm events. Since the evaporation rate for the study area was not available, that of Bintulu Climatological Station in Indonesia (see Appendix II) was adopted in this study. Since both places are located in the Tropics, they are assumed to have similar climates and similar evaporation rates.

The hourly evaporation rate of Bintulu of 0.17 mm (from the daily evaporation rate of 4.0 mm, see Table 2.5) was multiplied by the storm duration and a total evaporation (mm) over the storm event was obtained. The total evaporation (mm) was subtracted from the storm rainfall (mm) and the amount of rainwater (mm) available for runoff from the catchment from the storm was established. Finally, the storm rainfall intensity after evaporation and the estimated peak flows were calculated and presented in Table 2.3.

Chapter 3

FIELDWORK METHODS

The fieldwork for this thesis was undertaken in the headwaters of Ok Mani stream. The study involved gauging both base- and flood-flows of the main streams in the sub-catchments and the storm rainfall. The fieldwork and other research took place over a period of six weeks, from 13.02.95 to 25.03.95.

The first two weeks (13.02.95 to 25.02.95) were spent preparing fieldwork instruments and camping equipment. There were two visits to the field site. The first visit was from 27.02.95 to 05.03.95. The entire period was spent in sub-catchments C1, C2 and C3 (see Figure 2.7). The second visit to the site was from 13.03.95 to 20.03.95. During this visit, the first two days (14.03.95 and 15.03.95) were spent in sub-catchments C1, C2 and C3, the next two (16.03.95 and 17.03.95) in sub-catchment C4 and the final two days (18.03.95 and 19.03.95) in sub-catchment C5 (see Figure 2.7).

During the visits, both base- and storm-flows and rainfall corresponding to the storms on 02.03.95, 03.03.95, 04.03.95 and 15.03.95 were gauged (sub-catchments C4 and C5 are omitted in this thesis as the flood-flows were not gauged). The final week (21.03.95 to 26.03.95) of the fieldwork was spent in the OTML - Environment Department office, processing the field data and collecting other relevant information regarding the catchment and the project. The investigation ended on 26.03.95.

Transport to the fieldwork site and back to Tabubil (OTML administrative centre) and from one sub-catchment to the other (except for sub-catchments C1, C2 and C3) was only possible and done by OTML-chartered helicopters. A brief itinerary of the fieldwork is given in Appendix III.

3.1 Stream Gauging

3.1.1 Site Selection

For stream gauging, a straight, regular and reasonably stable reach was identified along and towards the outlet of the main stream in each sub-catchment. Figure 2.7 shows the gauging locations (V, W and X) of both base- and flood-flows.

To gauge the base-flows, the width of the channel cross-section at the gauging point was measured with a tape measure and divided into segments depending on the widths (wide stream channels were divided into more segments than narrow channels). All the cross-sections were divided into 5 segments except 4 for gauging point V. The cross-section and the segments were labelled as shown in Figure 3.11.

To gauge the flood-flows, the length of the reach (stream bank easily accessible from the camp during floods) 5 metres from the stream (to avoid flooding) was measured with a tape measure and recorded. Two-metre long poles were planted half a metre firmly into the ground at each end of the measured reach. The lengths of the reach at the gauging points V, W and X (see Figure 2.7) were 5 m, 20 m and 12 m respectively. Figure 3.12 illustrates the reach.

3.1.2 Duration and Frequency of Gauging

Base-flow gauging in sub-catchments C1, C2 and C3 were done on 02.03.95, 03.03,95 and 04.03.95 during the first field visit and 15.03.95 during the second visit. The gaugings were done between 1000 and 1100 hours each day. A time interval of 10 minutes was allowed between each gauging session. A total of 2 base-flow velocity measurements were done per gauging point per day.

Flood-flow gauging in sub-catchments C1, C2 and C3 at points V, W and X (see Figure 2.7) were done on 02.03.95, 03.03.95 and 04.03.95 during the first field visit and 15.03.95 in the second visit. The stream-flow velocities for the storm on 03.03.95 and 15.03.95 were respectively done at 0.5, 1.0, 1.5 and 2.0 hours after

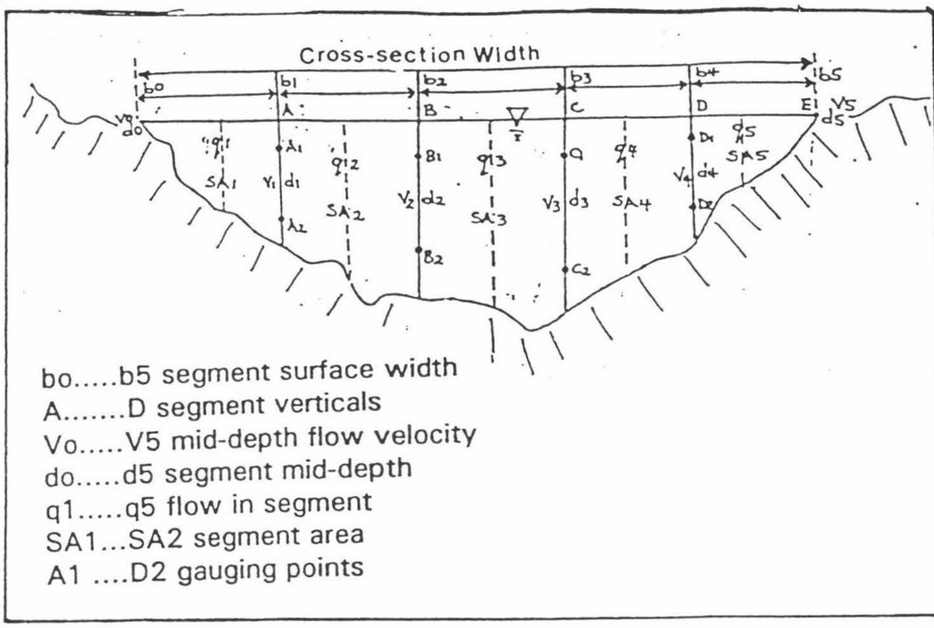


Figure 3.11: Stream channel cross-section for base-flow gauging (figure not to scale)

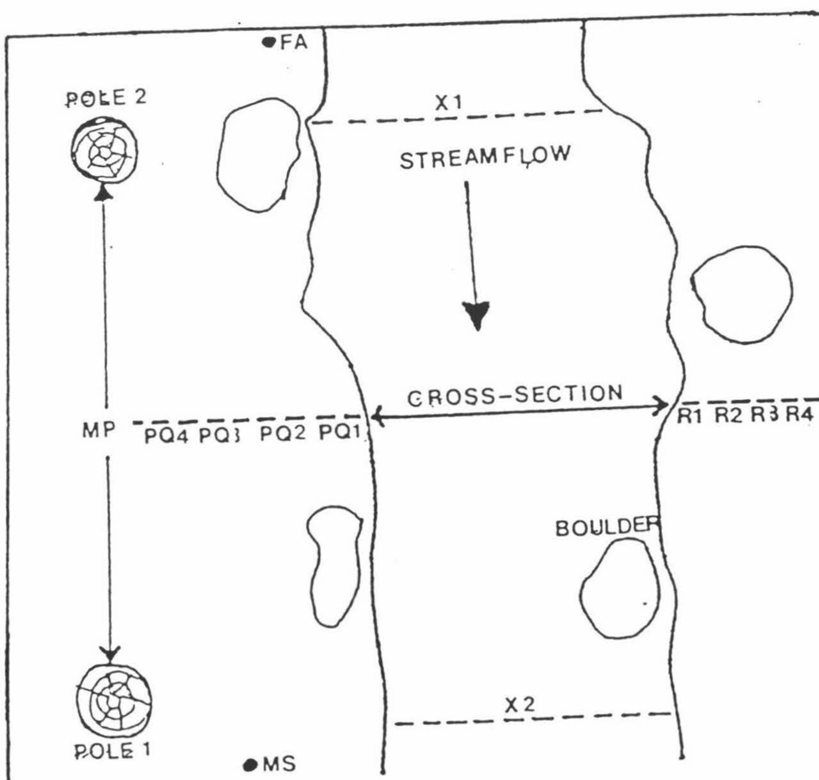


Figure 3.12: Stream reach for storm-flow gauging

the beginning of the storm and on 02.03.95 at 0.5, 1.5, 3.0 and 5.0 hours while the storm on 04.03.95 was gauged at 0.5, 1.0, 2.0 and 3.0 hours after the beginning of the storm. A time interval of 10 minutes was spent between each gauging point in the sub-catchments. A total of 4 mid-stream surface average flow velocities were measured per storm per gauging point over the period.

3.2 Stream-flow Calculations

The indirect Area-Velocity method was applied to calculate the stream-flow. This method requires the measurement of the stream channel cross-section area at mid-point of the reach and the average stream-flow velocity throughout the cross-section.

Base-flow velocities were measured with a conventional current meter, thus applying the Mean-Section method. The Mean-Section method regards the cross-section to be made up of a number of rectangular segments, each bounded by adjacent verticals where V1 and V2 for instance (see Figure 3.11), are the mean flow velocities at the second and third verticals respectively.

Flood-flow velocities were measured applying the float method. The float method involves measuring the mid-stream surface-flow velocity using floats. That is, the time the floats take to travel the reach. The surface-flow velocity was then multiplied by a factor of 0.70 (see Section 3.4.1.2) to approximate average stream-flow velocity.

For both cases, the flow is given as the product of the stream channel cross-section area and the mean stream-flow velocity.

3.2.1 Base-flow Velocity Measurements

Details of the current meter used to measure the stream base-flow velocity and the method of calibration is given in Appendix IV.1. There is a description in Appendix IV.2 of the application of the meter in the stream base-flow velocity measurements.

The base-flow velocity measurements were done 2 times consecutively per gauging point per day. It should be noted that the flow velocity measurements were done at 0.2 and 0.8 of the vertical depth (m) because their mean is equal to the mean velocity of the stream-flow velocity in a logarithmic profile.

3.2.2 **Flood-flow Velocity Measurements**

Because Ok Mani is a mountain stream, flows were high and fast during storms in narrow channels between steep slopes which meant that flow velocity measurements by wading was impractical and hazardous. Under this condition, floats were used to take the surface mid-stream-flow velocity.

The floats were used to take the mid-stream surface-flow velocities within a reach of known distance identified and marked as discussed in Section 3.1.1 and illustrated in Figure 3.12. Flood-flow velocities were gauged 4 times throughout a storm event. The measurement procedures were:

a. The width of the stream channel cross-section was measured and recorded. Firstly, the mid-point (between poles 1 and 2) of the reach was identified and marked (MP) (see Figure 3.12). The distance (m) between the mid-point (MP) and the stream edge (PQ1) was measured with a tape measure and recorded.

b. From the stream edge (PQ1), the opposite stream edge (R1) (see Figure 3.12) was noted by physical objects like rocks, logs, plants and/or other physical markings as the storm-flows were high and fast. This was also done for the successive gauging sessions at the point, i.e., PQ2 and R2, PQ3 and R3, and PQ4 and R4, as the stream width increased with increasing stream-flow.

c. The stop-watch was set to zero. The floats were tossed into the stream two metres upstream of pole 1 (FA), to allow the floats to achieve normal velocity by travelling at the speed of the current. The floats were dry wooden pieces of 30 cm in length and 4 to 5 cm in diameter. When the float reached the point directly opposite pole 1 (X1), the stop-watch was started. As soon as the float reached pole 2 (X2) (see Figure 3.12), the watch was stopped.

d. The time taken (seconds) for the float to travel along the reach between the poles was recorded.

e. Procedures b to d were repeated for the second float for the session.

f. After a gauging session was completed in one gauging location, the procedures a to e were repeated at the other gauging points in the sub-catchments.

g. After the storm was over and the stream-flow had return to normal, the channel width for each gauging session during the flood-flow was measured using a tape measure. That is, the distances (m) between the mid-point (MP) and the corresponding physical object identified and noted on the opposite bank (R1, R2, R3 and R4, see Figure 3.12) before and corresponding to the gauging was measured. The distance (m) was then subtracted from the distance (m) between the mid-point (MP) and the stream edge (PQ1, PQ2, PQ3 and PQ4, see Figure 3.12) over the storm duration measured and recorded before and at each of the surface-flow velocity measurements. The differences obtained were the channel cross-section width (m) for the gauging session.

(Note that as stream-flow increased over the storm duration, R1 increased towards R4 and PQ1 towards MP [i.e., the stream width increasing with stream flow], the distance between PQ1 and MP decreased, therefore, increasing the stream width).

h. The channel width for each storm-flow velocity gauging session obtained in step g was then divided into segments (wider widths were divided into more segments than narrow widths). This was done by tying a nylon rope tightly across the measured stream channel width for the respective gauging session (between PQ1 and R1, PQ2 and R2, PQ3 and R3, and PQ4 and R4; see Figure 3.12) and with a tape measure, the nylon rope was marked into segments. The depth (m) of the segment verticals across the cross-sections were measured with the tape measure and recorded. Note that because the stream channel cross-section is U-shaped, the depth of stream-flow increased as PQ1 increased towards MP and R1 towards R4.

The procedures a to h were repeated during the gauging sessions at all the gauging points (see Figure 2.7) throughout the storm and for all the storms over the period.

3.3 Storm Rainfall Gauging

3.3.1 Site Selection

A relatively flat site in the sub-catchments was selected and cleared of trees and bushes to about 5 metres from the centre to avoid vegetation sheltering of the gauge. The sites of clearing and installation of raingauges are shown in Figure 2.7.

3.3.2 Raingauge Preparation and Installation

The type of raingauge used in the field work is shown in Appendix V.1. They are the 4-inch Standard Daily Marquis 1000 manual gauges. When prepared and installed, they collect rainwater and can be read off from the graduated tube in half millimetres. The procedures of preparation and installation of the raingauges are given in Appendix V.2. After a raingauge was prepared and installed, it was 1.5 metres off the ground level to avoid any rainwater that may splash into the raingauge.

3.3.3 Rainfall Reading and Recording

The rainwater collected at the raingauges was checked and recorded immediately after the storm event as described in Appendix V.2.

3.4 Fieldwork Data Analysis

3.4.1.1 Base-flow Velocity

The current meter was tightly fastened on the wading rod at the predetermined depth (m) and lowered to the bottom of the vertical of the stream cross-section with the propeller facing the stream-flow. The timer on the current meter counter was set manually at 10 seconds and switched on. The reading stopped after 10 seconds and the pulses were read-off from the counter screen and recorded as revolutions (r). The number of revolutions per second (n) were calculated as:

$$n = r/10 \tag{3.3}$$

The n was then inserted into one of the three formulas given below (see Appendix IV.1) depending on the value of n, to compute the stream-flow velocity at the predetermined depth (m) along the vertical of the stream channel cross-section. The formulas are:

i. when $n < 0.85$, $V = 0.1341\ n + 0.0131$ m/s
ii. when $n > 0.85$ but < 7.07, $V = 0.1276\ n + 0.0187$ m/s
iii. when $n > 7.07$, $V = 0.1324\ n - 0.0155$ m/s (3.4)

where, 'n' denotes the number of revolutions of the current meter propeller per second and 'V' denotes the stream-flow velocity in metres per second.

Stream-flow velocity for the predetermined depths (m) (A1, A2, B1, B2, C1, C2, D1 and D2) along the verticals (A, B, C and D, see Figure 3.11) were computed as shown above. A mean flow velocity was calculated for the vertical and finally an average stream-flow velocity for the cross-section was computed. For instance, the mean flow velocity of vertical A was calculated as the sum of the flow velocities at depths A1 and A2 divided by 2 (number of depth readings along vertical A). Finally, an average stream flow velocity for the cross-section for the gauging session was calculated by taking the sum of the mean flow velocities at the verticals (A, B, C, and D, see Figure 3.11) and divided by the number of verticals.

3.4.1.2 **Flood-flow Velocity**

The procedures for flood-flow velocity measurements are discussed in Section 3.2.2. Two floats were tossed into the mid-stream surface per gauging session. The time taken by the floats to travel the reach was averaged and thus an average time (seconds) was obtained for the session. To establish the mid-stream surface-flow velocity, the distance (m) of the reach was divided by the average velocity the floats took to travel the reach.

Floats record mid-surface stream-flow velocities which are usually higher than the average stream-flow velocity, which occurs at 0.6 of the depth. Therefore, the average mid-stream surface-flow velocities obtained were multiplied by 0.70, a factor recommended by Moseley and McKerchar (1992), Shaw (1991) and Ayoada (1988) to bring them close to the average stream-flow velocity.

3.4.2 **Stream-flow**

The stream-flow for both base- and flood-flows were computed applying the Area-Velocity method, and is in the form:

$$\text{Flow } (m^3/s) = \text{Velocity } (m/s) \; . \; \text{Area } (m^2) \tag{3.5}$$

3.4.2.1 **Base-flow**

The procedure for obtaining the stream base-flow velocity and area of the segments in the cross-section is explained in Appendix IV.2. The stream base-flow in each segment was calculated using equation 3.5 given in Section 3.4.2, i.e., area of segment by the flow velocity at mid-point of the segment vertical.

The total flow (Q) at the cross-section was computed in the form:

$$Q\ (m^3/s) = \sum_{i=n^*}^{i=I} qi = q1 + q2 + q3 + q4 + q5 \qquad (3.6)$$

(* number of segment flows)

3.4.2.2 Flood-flow

The channel width of the cross-section during a flood-flow gauging session was measured as explained in Section 3.2.2. The area of a segment in the cross-section was calculated by multiplying the vertical depth (m) and the segment surface distance (m). The area of the cross-section for a gauging session was obtained by summing the area of the segments.

The flood-flows were computed from equation 3.5 given in Section 3.4.2. that is, area of cross-section multiplied by the mid-depth flow velocity obtained from Section 3.4.1.2.

3.4.3 Rainfall Data

The frequency and the method of storm rainfall data collection is discussed in Section 3.3.3. Because the catchment rainfall increases with elevation, the gauged storm rainfall were adjusted as discussed in Section 2.3.7. The estimated flood peak flood-flows computed by Rational method required the rainfall intensity (I) for the storm event. The method of intensity calculation and an example of flow computation is given in Section 2.5.1. The Regional Flood Frequency method required the actual adjusted point rainfall depth (mm) presented in Table 2.2. As an example of the estimated flows computed by the two methods are given in Section 2.5.2.

3.5 Storm Runoff Analysis

3.5.1 Base-flow Separation

The analyzed results of the base- and flood-flows gauged over the field work period are plotted in hydrographs presented in Appendix VI.1. The hydrographs shapes fitted the characteristic of the sub-catchments and to that found by Heerdegen (1973), i.e., high peak discharge and short base-length reflecting smaller area, shorter travel-time, lower sinuosity and somewhat higher drainage density. The volume of surface runoff of a storm, i.e., the area under the hydrograph, was measured with the Planimeter (see Appendix I.1). The storm-flows were computed by applying the Test-square Proportion technique (see Appendix VI.2).

Since the catchment characteristics were not available or known, the base-flow and direct storm runoff were separated by applying one of the methods suggested by Shaw (1991). The method is a satisfactory compromise between; (i) the separation by a horizontal line from the start of the hydrograph rise to the end of the curve's recession, assuming the storm has no effect on or contribution to the ground-water, and (ii) assuming the storm has a marked effect on the base-flow by peaking some time after the storm, which would be quite subjective and various answers would be produced. As Wilson (1974) stated "the dividing line between runoff and base-flow is indeterminate and can vary widely ...... and to analyze its precise position would require a detailed knowledge of the geohydrolgy of the catchment, including the areal extent and transmissibility of the aquifers". This method is straightforward and gives consistent results (Shaw 1991). The base-flow and storm runoff were separated as shown in Appendix VI.1 (indicated by dashed line).

The equivalent depth of runoff (mm) of the storm is evaluated by dividing the flow (base and flood separately) into the catchment area. The difference between the total runoff (i.e., both of base-flow and flood runoff) and the storm rainfall (mm) was computed to establish a gain or loss in the runoff from the storm rainfall. Finally, a runoff:rainfall ratio was calculated for the storm. For example, the depth of runoff for the storm on 02.03.95 in sub-catchment C2 is:

Flood-flow:

Volume of Surface Runoff: (the product of the difference (X) of beginning and end planimeter readings from tracing the flood-flow component of hydrograph and 166473.99 [see Appendix VI.2]).

= Difference of planimeter readings x 166473.99 = 1.27 x 166473.99 = 211422 m^3

Depth of Surface Runoff: surface runoff volume (m^3)/catchment area (km^2) = 211422/[7.06 x 10^6] = 0.0299 m = 29.9 mm

Base-flow:

Base-flow Runoff Volume: (obtained by the same method as the flood-flow above but the planimeter readings come from tracing the base-flow component of the hydrograph).

= Difference of planimeter readings x 166473.99 = 0.22 x 166473.99 = 36624 m^3

Depth of Base-flow Runoff: base-flow runoff volume (m^3)/catchment area (km^2) = 36624 m^3/[7.06 x 10^6] = 0.00519 m = 5.2 mm

Total Volume of Runoff: surface runoff volume + base-flow runoff volume = 211422 + 36624 = 248046 m^3

Total Depth of Runoff: depth of surface runoff + depth of base-flow runoff = 29.9 + 5.2 = 35.1 mm

Storm Rainfall: 56.24 mm (after adjustment, see Table 2.3)

Runoff Gain[+] or Loss[-]: depth of surface runoff (mm) - rainfall (mm) = 29.9 - 56.24 = -26.34 mm

Runoff:Rainfall Ratio: depth of surface runoff (mm)/rainfall (mm) = 29.9/56.24 = 0.53 = 53%

The depth of storm runoff (mm) is then, by definition, the effective rainfall for runoff from the storm. The analysis did not include or consider any part of the rainfall that may be lost in the sub-catchments through infiltration, depression storage, evapotranspiration, etc, as this information was not available.

Chapter 4

RESULTS

4.1 Fieldwork Results

4.1.1 Base-flow

The mean daily base-flow for the streams gauged over the gauging period are presented in Table 4.6. The stream channel cross-section area and flow velocity field measurements are presented in Appendix VII.

The base-flows gauged in each stream over the period (02.03.95 to 04.03.95 and 15.03.95) were generally steady. The variations were all under 0.05 m^3/s except for a high flow of 1.38 m^3/s on 15.03.95 from sub-catchment C3. The flows increased downstream with increasing catchment area. The mean specific base-flow over the gauging period was 0.23 $m^3/s/km^2$ in sub-catchment C1, 0.08 $m^3/s/km^2$ in sub-catchment C2 and 0.07 $m^3/s/km^2$ in sub-catchment C3. The mean specific base-flow decreased downstream with increasing catchment area, i.e., the base-flows showed a decline per unit area.

The almost steady base-flow conditions could result for a number of reasons. Firstly, from observations during the fieldwork, despite the high daily average temperature (30^{0}C), the soil was always saturated. This is due to the dense vegetation cover which keeps the ground at lower temperatures, minimising evaporation and lowering infiltration, as well as the regular and usually high and long duration rainfall which maintains the soil-water content. These two factors keep the soil moisture content high and infiltration capacity low, resulting in high and rapid storm surface runoff during storms. This is further enhanced by the steep terrain of the catchment which speeds up the velocity of storm flow, thereby shortening the period of infiltration and producing a greater concentration of

Table 4.6: Ok Mani Base- and Flood-flows

Date	Gauging Point	Time Storm Started	Time (mins) Flow (m3/s)	Base-flow (m3/s)	Flood-flows (m3/s)			
02.03.95	V	1:00 PM	Time	0	30	60	90	120
			Flow	0.26	0.78	1.79	1.98	0.37
	W	1:00 PM	Time	0	40	70	100	130
			Flow	0.58	4.79	8.51	10.34	6.98
	X	1:00 PM	Time	0	50	80	110	140
			Flow	0.68	6.57	10.08	14.46	5.88
03.03.95	V	4:00 PM	Time	0	30	60	90	120
			Flow	0.21	0.42	0.54	0.60	0.50
	W	4:00 PM	Time	0	40	70	100	130
			Flow	0.52	1.12	2.02	2.12	1.84
	X	4:00 PM	Time	0.00	50	80	110	140
			Flow	0.63	1.46	2.34	2.98	1.96
04.03.95	V	3:00 PM	Time	0	30	60	90	120
			Flow	0.22	1.34	4.23	4.90	2.13
	W	3:00 PM	Time	0	40	70	100	130
			Flow	0.55	7.10	18.99	19.20	10.26
	X	3:00 PM	Time	0	50	80	110	140
			Flow	0.65	9.09	20.59	21.29	10.94
15.03.95	V	5:30 PM	Time	0	30	60	90	120
			Flow	0.24	2.61	5.21	4.30	1.38
	W	5:30 PM	Time	0	40	70	100	130
			Flow	0.56	13.36	21.32	18.81	5.97
	X	5:30 PM	Time	0	50	80	110	140
			Flow	0.70	17.35	22.50	18.54	7.64

V gauging point in sub-catchment C1
W gauging point in sub-catchment C2
X gauging point in sub-catchment C3

surface runoff.

4.1.2 **Estimated Peak Flood-flow**

The estimated peak flood-flows derived from surface runoff and computed for the streams over the gauging period (02.03.95 to 04.03.95 and 15.03.95), both with and without evaporation over the duration of the storms, are presented in Table 2.5. The estimated peak flood-flows over the gauging period showed a strong positive relationship with rainfall depth, rainfall intensity and with catchment area (see Tables 2.3 and 2.5). The estimated peak flow and flow percentage differences for storm events including evaporation or not were small and all under 3.5 percent (see Table 2.5).

4.1.3 **Gauged Flood-flow**

The flood-flows gauged throughout the storm events over the gauging period are presented in Table 4.6 and the peak flows in Table 2.5. The channel cross-section areas and flow velocity field measurements are presented in Appendix VIII. The base- and flood-flows presented in Table 4.6 are plotted in hydrographs given in Appendix V.1. The volume and depth of runoff and the runoff:rainfall ratios analyzed for the individual storms are presented in Table 4.7.

The gauged peak flood-flows in Table 2.5 indicate that some of the flows show a poor relationship with the depth of rainfall, rainfall intensity and with catchment area. For instance, the storm on 04.03.95 with a high rainfall and intensity, discharges a comparatively low flow compared with the 15.03.95 event which had a low rainfall and intensity (see Tables 2.2, 2.5 and 4.6). The low peak flows on 04.03.95 could have resulted from low rainfall in other parts of the catchment. The high peak flow on 15.03.95 from a low rainfall depth and intensity could have resulted from higher rainfall in other parts of the catchment away from the raingauges, i.e., the relationship between the flows and rainfall was not predictable. The flows for the storms on 02.03.95 and 03.03.95 showed a positive relationship and increased with increasing rainfall, rainfall intensity and with catchment area (see Table 2.2, 2.5 and 4.6).

Table 4.7: Ok Mani Base- and Flood-flow Runoff

Date	02.03.95			03.03.95			04.03.95			15.03.95		
Sub-catchment	C1	C2	C3	C1	C2	C3	C1	C2	C3	C1	C2	C3
Flood-flow												
Volume of Surface												
Runoff (m3)	24971	211422	256370	4994	13318	16647	54936	321295	404532	28301	158150	219746
Depth of Surface												
Runoff (mm)	24.70	29.90	26.50	4.90	1.90	1.70	54.40	45.50	41.90	28.00	22.40	22.70
Base-flow												
Volume of Base-flow												
Runoff (m3)	4994	36624	48277	1665	9988	11653	6659	41618	49942	3329	23306	49942
Depth of Base-flow												
Runoff (mm)	4.90	5.20	5.00	1.60	1.40	1.20	6.60	5.90	5.20	3.30	3.30	5.20
Total Volume of												
Runoff (m3)	29965	248046	304647	6659	23306	28300	61595	362913	454474	31630	181456	269688
Total Depth of												
Runoff (mm)	29.60	35.10	31.50	6.50	3.30	2.90	61.00	51.40	47.10	31.30	25.70	27.90
Rainfall (mm)	56.24	56.24	56.24	7.40	7.40	7.40	82.88	82.88	82.88	42.92	42.92	42.92
Depth of Runoff												
mm (Gain+ or Loss-)	31.54 -	26.34 -	29.74 -	2.5 -	5.5 -	5.70	28.48 -	37.38 -	40.98 -	14.92 -	20.52 -	20.22 -
Depth of Surface												
Runoff:Rainfall Ratio	0.44	0.53	0.47	0.66	0.27	0.23	0.66	0.55	0.51	0.68	0.52	0.53
(%)	44	53	47	66	27	23	66	55	51	68	52	53

The peak flood-flow and percentage differences between the estimated and the gauged peak flood-flows were large, the lowest being 0.31 m^3/s (3.0 percent) on 02.03.95 and the highest being 13.91 m^3/s (65.24 percent) on 15.03.95 both in sub-catchment C2 (see Table 2.5). The differences in flow did not show any relationship as its magnitude varies with rainfall, rainfall intensity and with catchment area (see Tables 2.1, 2.3 and 2.5). The large flow differences between the estimated and gauged peak floods-flows and the poor relationship with rainfall, rainfall intensity and catchment area may be caused by two factors: (i) under-estimation of some peak flood-flows caused by a lack of catchment homogeneity; and (ii) Over-estimation of some peak flood-flows caused by certain shortcomings associated with the gauging technique applied, the number of stream channel cross-sections and estimation of stream bed roughness characteristics. These are discussed in Chapter 5.

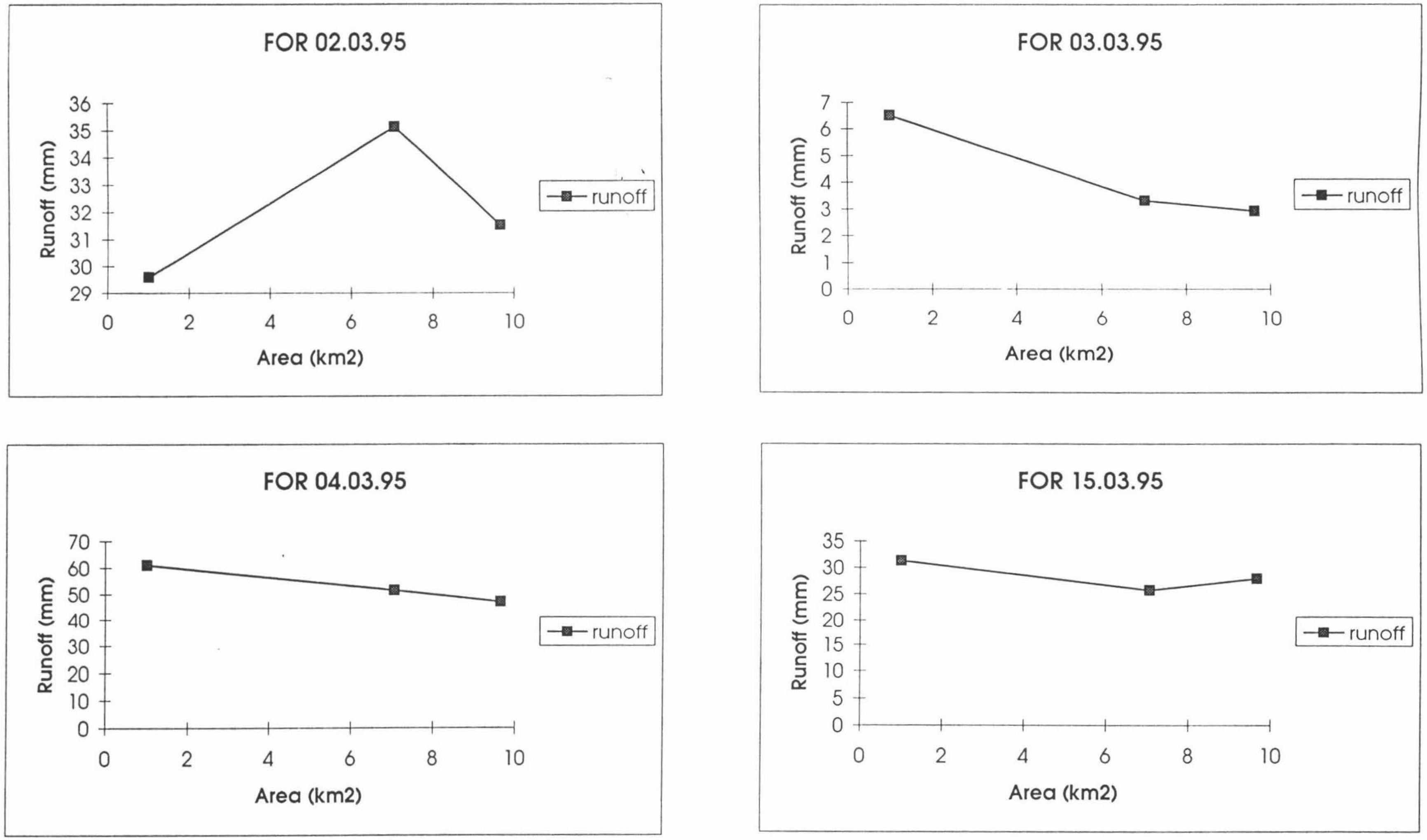
The volume of runoff as surface-flow, base-flow and the total flow showed a positive relationship and increased with increasing depth of rainfall, rainfall intensity and with catchment area (see Tables 2.2 and 4.7). That is, the magnitude of runoff volume is directly determined by the volume of rainfall, the rate at which it falls and the size of the catchment. The two-way analysis of variance to help establish whether there are differences between sub- catchments and storms shows that the differences due to storm data are highly significant, i.e., p(0.020) whereas the differences due to sub-catchments are insignificant, i.e., p(0.835) (see Appendix IX). Further, catchment mean rainfalls are not different (29.3, 34.5 and 34.7 mm) whereas the storm differences are large (14.0, 15.0, 41.7 and 60.7 mm).

The depth of runoff as base-flow, surface-flow and the total flow over the gauging period generally showed a predictable relationship and increased with increasing rainfall (see Tables 2.2 and 4.7). The depths of runoff from the individual sub-catchments showed an inconsistent relationship with catchment areas. The depth of runoff as base-flow, surface-flow and the total flow for the storm on 02.03.95

increased from sub-catchment C1 to C2 and decreased downstream to C3 with increasing catchment area. The depths of runoff for the storms on 03.04.95 and 04.03.95 decreased downstream with increasing catchment area and for the 15.03.95 event, the runoff decreased from sub-catchment C1 to C2 and increased to C3 with increasing catchment area (see Tables 2.2 and 4.7 and Figures 4.13 and 4.14). However, from Figure 4.15, the depth of runoff for the storms over the gauging period showed a predictable relationship and declined per unit catchment area.

For some storms, time to peak of the flood had little relationship with the rainfall, rainfall intensity, catchment area and stream length. For instance, the storm rainfall intensity on 04.03.95 (41.44 mm/hour) is about twice that of the storm on 15.03.95 (21.46 mm/hour). However, the flow of the former in the respective sub-catchments, peaked 1.25 hours into the storm event while the latter peaked 1.00 hour later (see Tables 2.1 and 2.2 and Appendix VI.1). The storms on 02.03.95 and 03.03.95 showed a similar situation, where the intensity of the storm on 03.03.95 was only 3.33 mm/hour and the flow peaked after 1.25 hours into the storm while that on 02.03.95 with an intensity of 9.5 mm/hour took 2.00 hours to peak in sub-catchment C1, 2.50 hours in C2 and 3.00 hours in C3. Further, the time to peak of the flood for 03.03.95, 04.03.95 and 15.03.95 events did not have a predictable relationship with catchment area and stream length. That is, it took the same period of time for the flood to peak during the respective storms, i.e., 1.25 hours on 03.03.95 and 04.03.95 and 1.00 hour on 15.03.95 in all the sub-catchments (see Tables 2.1 and 2.2 and Appendix VI.1).

The calculation of a gain or loss in depth of runoff from the storm rainfall over the period showed that a portion of the storm rainfall was lost, as expected, through evapotranspiration, infiltration, depression storage, etc, (see Table 4.7 and Figure 4.14). In general, increased rainfall led to decreased losses. However, Table 4.7 and Figures 4.13, 4.14 and 4.15 show that the losses did not follow a set relationship with catchment area, i.e., the loss for the storm on 02.03.95 decreased from sub-catchment C1 to C2 and increased downstream to C3 with



<u>Figure 4.13:</u> Plots of sub-catchment area by depth of runoff

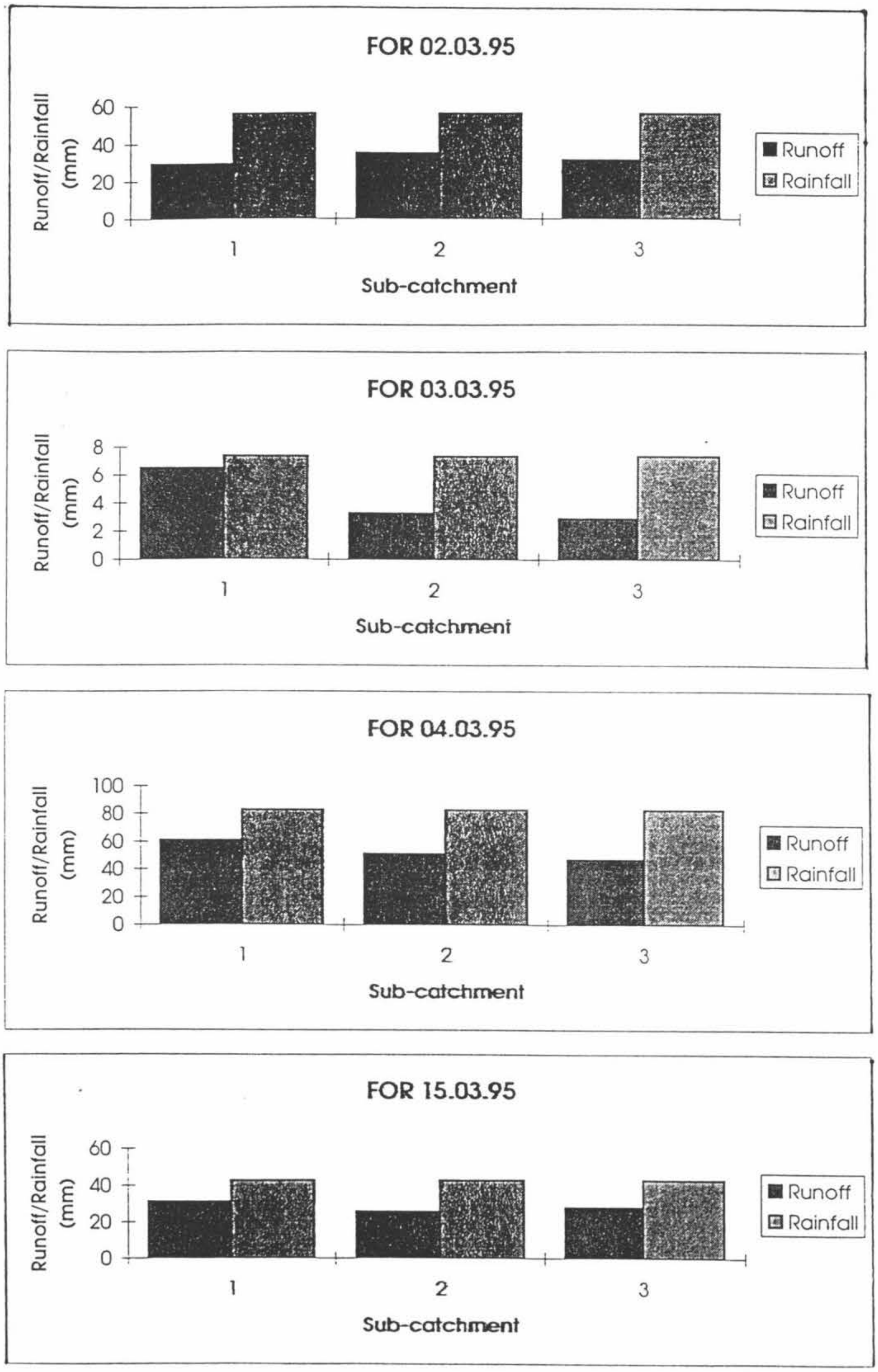
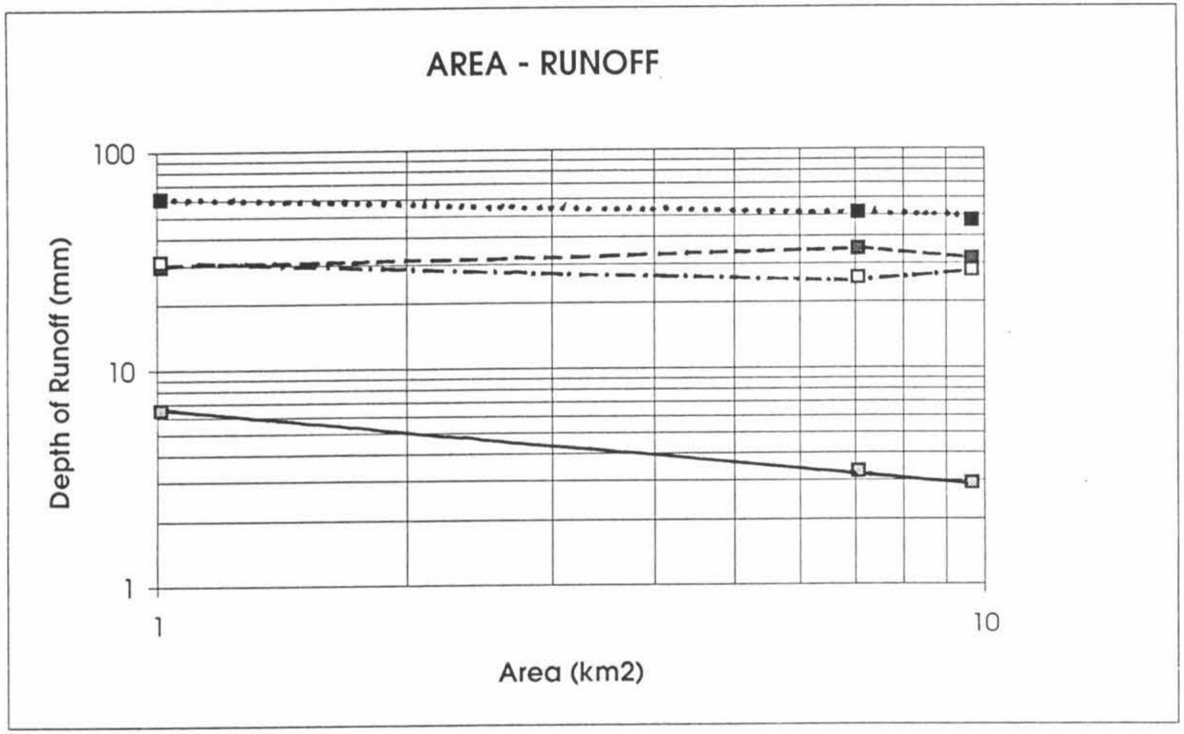


Figure 4.14: Plots of sub-catchment by runoff-rainfall



Mean Slope

02.03.95 = -0.22
03.03.95 = 0.40
04.03.95 = 1.61
15.03.95 = 0.39

---- 02.03.95
____ 03.03.95
.... 04.03.95
_._. 15.03.95

Figure 4.15: Plot of depth of runoff per unit area of catchment

increasing area, for the 15.03.95 event, it increased from sub-catchment C1 to C2 and decreased to C3 and the depth of runoff losses for the 03.03.95 and 04.03.95 events increased downstream with increasing catchment area.

The runoff:rainfall ratios, which indicates losses as some rainfall was lost to the catchment through evapotranspiration, infiltration, depression storage, etc, in the respective sub-catchments were less than 1.0, ranging between 0.23 (23%) in sub-catchment C3 on 03.03.95 to 0.68 (68%) in sub-catchment C1 on 15.03.95 (see Table 4.7). The runoff:rainfall ratios in the sub-catchments did not show a set relationship, i.e., the ratios for the storms on 03.03.95 and 04.03.95 increased downstream with increasing area. The ratios for the 02.03.95 event increased from sub-catchment C1 to C2 and decreased downstream while the runoff:rainfall ratio of the 15.03.95 event decreased from sub-catchment C1 to C2 and increased downstream to C3 (see Table 4.7). However, the ratios generally showed an increase with increasing rainfall and a decrease downstream with increasing catchment area (see Tables 2.2 and 4.7).

Further, the runoff:rainfall ratios for the storms on 02.03.95, 04.03.95 and 15.03.95 in the respective sub-catchments were generally constant. The mean ratio over the period, excluding the 03.03.95 event, in sub-catchment C1 was 59 percent, C2 was 53 percent and in C3 was 50 percent, and showed a decreasing runoff:rainfall ratio with increasing catchment area. The runoff:rainfall ratios for the storm on 03.03.95 in the respective catchments showed a poor relationship because of the very low rainfall. The high runoff ratio of 66 percent in sub-catchment C1 is due to the combined effect of the very low rainfall and small size of catchment area (see Table 4.7).

The evaporation rate of 4.00 mm per day derived from Bintulu Climatological Station, Indonesia, was applied assuming evaporation took place during the storm events and that a portion of the rainfall was lost. The differences, assuming evaporation had taken place or not, were small and all under 0.50 mm/hour (see Table 2.3).

4.1.4 **Storm Rainfall**

The rainfall corresponding to the storms gauged in the sub-catchments over the period (02.03.95 to 04.03.95 and 15.03.95) and the intensities computed are presented in Table 2.2. Areal rainfall for Ok Mani catchment computed from 5 rainfall stations for the month of June 1994 is presented in Table 2.3 and an area-elevation (hypsometric) curve (see Figure 2.10) was constructed from the data in Table 2.3. Because Table 2.3 and Figure 2.10 show that rainfall increases with increasing elevation, the gauged rainfalls were adjusted by an elevation correction factor and presented in Table 2.3. The gauged storm rainfalls increased substantially when adjusted by the mean factor obtained as discussed in Section 2.3.7.

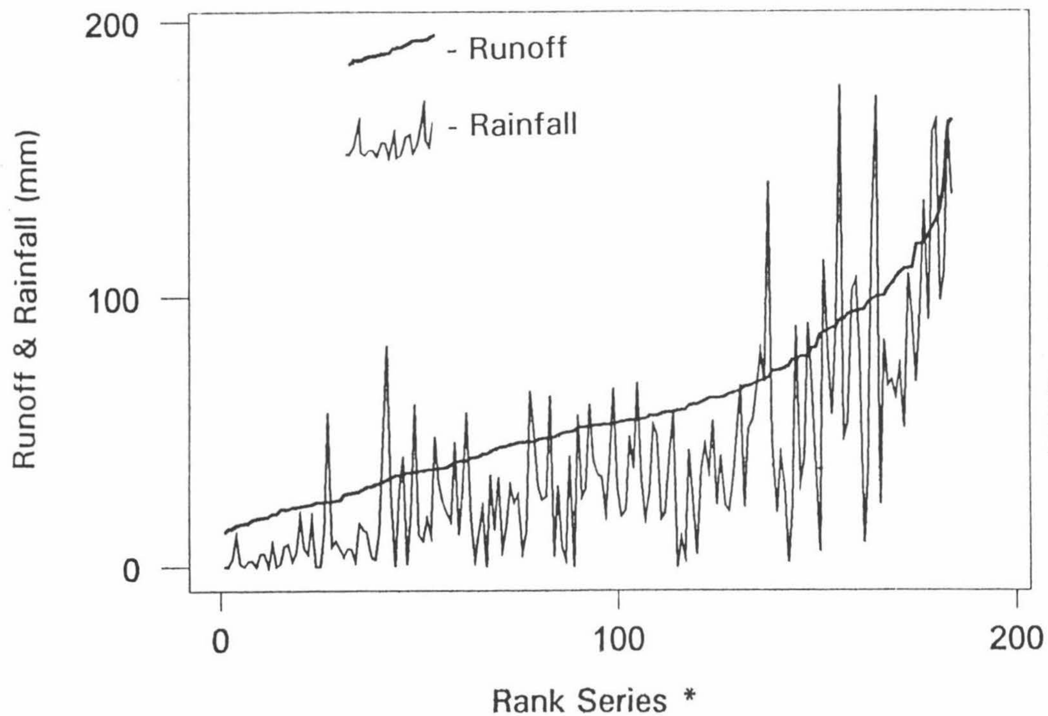
4.2 **OTML Rainfall-Runoff Result**

The rainfall and stream-flow data from the OTML gauging station MANO4 along the Ok Mani stream (see Figure 1.4.1 for location of station in the catchment) for the upper catchment of area 24.20 km^2 analyzed so far, indicates that runoff from the catchment is 50 percent greater than rainfall gauged at the station. As an example, the rainfall-runoff data from the station for part of 1993 is plotted and presented in Figure 4.16. The daily runoff values were ranked from lowest to highest and plotted against the rainfall for that day. It shows that there was a surplus of about 18 mm of runoff per day from the catchment over the rainfall gauged at the station. This surplus runoff could be the result of the relatively high rainfall in the higher elevations of the catchment as indicated from the areal rainfall in Table 2.3 or the surplus runoff could come as extra-catchment flows from the karst sinkholes in the adjacent catchments.

4.3 **Overview of Results**

The base-flows were generally steady over the gauging period and increased downstream with increasing catchment area (see Table 4.6) and showed a decline per unit area.

Figure 4.16: Plot of Runoff/Rainfall relationship at MANO4 Station - Ok Mani catchment for 1993



Mean
Rainfall = 38.5 mm
Runoff = 56.3 mm

Median
Rainfall = 28.0 mm
Runoff = 51.7 mm

* Based on runoff with association of rainfall for that day, which shows how rainfall is variable to runoff

The estimated peak flood-flows showed a positive relationship while the gauged peak flood-flows generally showed an unpredictable relationship with increasing rainfall depth, rainfall intensity and catchment area. Further, a comparison of the estimated and gauged peak flood-flows showed that the gauged peak flood-flows were relatively higher than the estimated flows and the differences were large. The large differences could be due to a lack of catchment homogeneity and the shortcomings associated with float gauging.

The volume of runoff for the storms as base-flow, surface-flow and total flow showed a positive relationship and increased with increasing rainfall, rainfall intensity and catchment area. However, the time to the peak of the flood over the periods showed an unpredictable relationship with rainfall intensity. Further, the depth of runoff generally showed a positive relationship with rainfall and increased with increasing rainfall and intensity. However, the depth of runoff did not show a set relationship with catchment area (see Figure 4.13). There was a loss in depth of runoff for all the storms in the sub-catchments (see Table 4.7 and Figure 4.14). The depth of runoff for the storms generally show a decline per unit catchment area (see Figure 4.15).

The runoff:rainfall ratios for all the storms in the respective sub-catchments were less than 1.0, indicating a loss of a portion of the storm rainfall to the catchment. The ratios, except for the 03.03.95 storm, were generally constant in the respective sub-catchments and showed a positive relationship and increased with increasing rainfall and decreased downstream with increasing catchment area. The ratios for the storm on 03.03.95 were uncharacteristically low as rainfall was very low (see Table 4.7). By contrast, the runoff:rainfall ratio at OTML gauging station MANO4 is greater than 1.0, indicating a runoff surplus compared with the rainfall gauged at the station.

From the discussion the following conclusions can be drawn:

(i) the estimated peak flood-flows in the sub-catchments for all the storms over the period were lower than those gauged;

(ii) the runoff:rainfall ratios over the fieldwork period is less than 1.0, indicating runoff is less than rainfall;

(iii) the result of the rainfall:runoff ratio from MANO4 is greater than 1.0, indicating runoff is greater than rainfall; and,

(iv) the rainfall-runoff results from the fieldwork is not compatible with the two years of data from MANO4.

Although the rainfall-runoff results from the fieldwork showed runoff was less than rainfall and did not seem compatible with that from MANO4, apparently contradicting the hypothesis of extra-catchment flows into Ok Mani, there were certain shortcomings associated with the gauging. These included the very short period (4 days) of gauging and the small amount of data obtained and analyzed, the errors associated with the gauging technique and the assumptions involved as discussed in Section 5.1. These may consequently have had an effect on the data obtained and analyzed, which in turn may have led to the results being far from actual. These results, however, do show a basic consistency. Despite this, the possibility of extra-catchment flows is still not proven wrong and should not be ignored.

Chapter 5

DISCUSSION OF RESULTS

5.1 Estimated and Gauged Peak Flood-flows

The large flow differences between estimated and gauged peak flood-flows and the poor relationship with rainfall depth, rainfall intensity and with catchment area may be caused by such factors as the assumption of catchment homogeneity and certain shortcomings associated with the gauging technique.

For instance, as Cheng (1987) points out, flow estimation methods were traditionally formulated and used assuming the storm has the same frequency as the flood and that the watershed is a lumped-input system, i.e., rainfall is assumed to be uniformly distributed over the watershed and the underlying surface is assumed to be homogeneous. This may not be the case here because the catchments whose climate and physical parameters were used to derive the estimation methods are different from those of the catchments (sub-catchments C1, C2 and C3) to which the methods were applied, subsequently resulting in the under-estimation of peak flood-flows. A number of flood estimation and prediction studies including Tasker and Stedinger (1987), Stallings (1987), NRC (1987) and Zrinji and Burn (1994) have shown that the assumption of catchment homogeneity does not exist and that the estimated peak flood-flows under- or over-estimated the actual flows.

Assuming the estimated peak flood-flows were correct and representative of the actual peak flow in the streams for the respective storms, the high gauged flood-flows may be caused by over-estimation of peak flow due to errors associated with the gauging technique applied such as the number of cross-sections selected along the stream reach and the stream bed roughness characteristics.

The float technique of stream-flow measurements has been found to provide a less accurate result and often results in significant errors compared with the conventional current-meter technique. Still, measurements carefully made under favourable conditions are found to be accurate to within ± 10 percent. If a non-uniform reach is selected or too few floats are used, results may be in error of 25 percent or more (DSIRNZ 1988). The fieldwork results showed that half the measurements fall within this ± 25 percent.

The high peak flood-flows may have resulted from the stream channel cross-section selection and measurement. Selection of a reach is very important as the channel needs to be straight and uniform to locate three to five cross-sections, and there should be minimum scour and deposition (Jarrett 1987). Along these stream reaches, only one cross-section was identified at mid-reach (see Section 3.1.1). Therefore, the cross-section and the flow area calculated may not have been representative of the entire reach, which may have led to over-estimation of the actual flood-flow. Some scouring and deposition which was evident along the reaches, may have had some effects on the flow velocity and the cross-section area and in turn on the flood-flows. That is, scouring and deposition may have caused the stream-flow velocity and cross-section area to be vertically and horizontally non-uniform over the reach, resulting in differences between the calculated and actual flows.

The roughness of the stream bed is also important and could be a source of error as it affects the stream-flow velocity. The stream channel within the reaches is characterised by gravels, cobbles and a few boulders, with an estimated Manning's Roughness Coefficient ('n') between 0.03 - 0.05. (Although Manning's formula was not used to compute the flows, the 'n' values are used to explain the roughness of the stream-bed). This may have an effect by retarding the flow velocity, especially on the sides and bottom of the channel where the boulders were located. The boulders would have a backwater effect, so reducing the flow velocity in those sections of the channel cross-section. The combined effect would result in an under-estimation of the stream-flow. For instance, Jarrett

(1987) from his investigations in the United States established that estimation of flow is affected by Manning's 'n', scour and deposition, expansion and contraction losses, viscosity, unsteady flow, number of cross-sections, the state of flow and stream slope. He further commented that under such conditions, measurement error can be greater than 100 percent and leads to over-estimation of the actual peak flow.

5.2 Runoff - Rainfall

The rainfall-runoff data from the fieldwork, gained from a very limited period, did not seem comparable with that of the OTML gauging station MANO4. The fieldwork runoff:rainfall ratio, as one would expect, was less than 1.0, indicating a proportionate loss of storm rainfall, while the runoff:rainfall ratio from MANO4 was greater than 1.0 (50 percent greater than rainfall), indicating a hydrologically impossible runoff surplus. Since both the estimated and the actual flows and the altitudinally-weighted rainfalls calculated from the fieldwork are all hydrologically acceptable, the data from MANO4 are somewhat questionable.

Therefore, a likely factor causing runoff surplus to rainfall at MANO4 is much higher rainfall in other parts of the catchment compared with that gauged at the station (MANO4). However, the current problem is that there is no long-period record of rainfall distribution data for the catchment to justify this factor. The rainfall data for the year 1994 from four stations in the catchment are presented in Table 2.3. The rainfall data from the four stations are weighted, on the assumption that the data is representative of the catchment rainfall distribution. The gauged rainfall were then adjusted by the mean weighted factor of rainfall, which solves the problem of the lack of record of rainfall distribution data over the catchment.

5.2.1 Rainfall Distribution

The surplus runoff from the catchment recorded at MANO4 could be explained in terms of rainfall variation within the catchment. The runoff-rainfall analyses done so far and discussed in Section 4.1.5 assume that the rainfall gauged at the station was uniform and representative of the catchment. This assumption may be wrong as distribution of rainfall is not uniform as analysis of Ok Mani catchment areal rainfall (see Table 2.3) shows - that rainfall increases with increasing elevation in the catchment. The raingauge and stream-flow gauge at MANO4 are located at 800 m asl and there is a large ungauged catchment area upstream (see Figure 1.6) where the rainfall is assumed to be uniform and representative of the catchment area. From Table 2.3, it can be seen that rainfall increases significantly over small increases of elevation. Therefore, the rainfall gauged at MANO4 probably grossly under-represents the catchment rainfall given and that other parts of the catchment undoubtedly receive much more rainfall which in turn produces the measured runoff at MANO4.

Further, the relationship of storm characteristics to runoff factors (rainfall, intensity, time to peak of the flood, flood-flows, storm duration and centre, stream length and catchment area) for some storms had little association and the relationships were unusual, possibly due to the non-uniformity of rainfall in the catchment. For instance, this could be indicated from the storm on 04.03.95 discharging a comparatively low peak flow from a high rainfall of 56 mm compared with the 15.03.95 event, which discharged a high peak flow from a 29 mm rainfall, inspite of both storms having a 2-hour duration of rainfall (see Tables 2.2, 2.5 and 4.6). This means that the rainfall on 04.03.95 was over-representative and that there was less rain in other parts of the catchment resulting in the lower flows. It also means that the storm's centre may have been in that part of the catchment where the raingauges were installed, resulting in the lower-than-expected flows. Further, it could mean the rainfall gauged on 15.03.95 was under-representative and that there was more rain, higher intensity rain or the storm's centre was in other parts of the catchment, in turn resulting in the higher peak flows.

Cheng (1987) found from his study at the Huangjingtang Station in China, that rainfall was concentrated in the eastern part of the catchment with the rainfall centre near the downstream area. The situation showed that the spatial distribution of rainfall had a great effect on the flow. A small amount of rainfall concentrated near the downstream area may result in the same or higher flow than a large amount of rainfall distributed uniformly over the catchment. Cheng further established that the movement of a storm within the catchment may have significant effects on the flow in the streams.

The non-uniform distribution of rainfall may also be indicated from the rainfall intensity and the time to peak of the flood. Eagleson and Qinliang (1987) pointed out that assuming rainfall depth, intensity and duration of the storm to be the same everywhere in a catchment, the time for the flood to peak depends on the duration, catchment area, surface moisture condition and stream length. Assuming soil moisture and stream length did not have any effect on the time to the peak of the flood and since they are always fixed, the variation in rainfall intensities as expressed in the time of flood to peak, may be explained in terms of rainfall duration, and the position of the storm's centre. That is, since catchment physical parameters are fixed, the low relationship between the storm and stream-flow characteristics could mean that rainfall distribution was nonuniform. The rainfall depth, intensity and duration and the position of the storm centre over the sub-catchments were such that the peak flow of the storms reached the respective gauging points at the same time in the storm event. However, there is also the possibility that the general shape of catchment, length of main stream channel and the fact that they converge at a more-or-less common point could also explain this.

5.2.2 **Extra-catchment Flows**

Ok Mani catchment is more-or-less surrounded by a karst environment with streams disappearing into karst sinkholes (see Figure 1.6 for location of karst sinkholes). Considering the shortcomings associated with gauging including the very short period and the little data obtained and analyzed, errors associated with gauging, stream channel cross-sections and the stream-bed characteristics, the runoff:rainfall result thus obtained may be wrong. The surplus runoff gauged at

MANO4 could come from the surrounding catchments of karst sinkholes in the north, north-west and north-east. Further, the extra runoff could come from the overlapping of the catchment boundary of Ok Mani and the adjacent catchment north-west as catchment area varies according to the ground-water levels and so changes with time. Therefore, any possible sources of extra-catchment flows into Ok Mani catchment should not be over-looked.

For instance, Moreaux, Wilson and Memon (1984) stated that extra-catchment flows from karst sinkholes through underground channels should not be left out in the discussion of river discharges in a karst environment. Further, a number of emerging underground springs are evident along Ok Mani stream (see Plates 1.5 and 1.6). Although there is no evidence of their origin, they could be the emergence of subsurface flows from the karst sinkholes.

Numerous studies, including Arikan and Emekci (1995), Gunay (1995), Degrimenci and Gunay (1990), Benzeden and Tatlioglu (1985) and Harmancioglu and Yevjevich (1985) have shown that discharge of rivers in karst environment are significantly affected by water either sinking, thus resulting in low flows and/or water emerging into the river systems resulting in increased flows. A similar situation could exist in the Ok Mani catchment, where the streams in the adjacent catchments that are seen disappearing (see Figure 1.6) through the karst sinkholes could emerge into the Ok Mani stream. The high runoff:rainfall ratio recorded at MANO4 could indicate such a condition. Further, Bonacci (1987) found that at very high ground-water levels, springs were active, and their catchment areas significantly increased. At low ground-water levels, watershed lines shrink as water is lost to springs so that their catchment areas decreased.

Chapter 6

CONCLUSION AND SUMMARY

Understanding the relationships between rainfall and runoff is fundamental to the design of water-related structures with adequate standards of safety. The derivation of relationships between the two variables over a catchment area and the resulting flow in a river as Shaw (1991) points out is a fundamental problem for the hydrologist. Estimating runoff from rainfall measurements is very much dependent on the time scale being considered. For short durations (hours) the complex interrelationship between rainfall and runoff is not easily defined, but as the time period lengthens, the connection becomes simpler (Nemec 1972). The time interval used in the measurement of the two variables affects the derivation of any relationships, although with continuously recorded rainfall and stream-flow data this constraint can be removed and the determination of water yield from a catchment can be accomplished satisfactorily.

The size of the catchment area being considered also affects the relationships. For a very small area of a homogenous nature, the derivation of the relationship could be fairly simple; for very large catchments and for long time periods, differences in catchment effects may be smoothed out giving relatively simple rainfall - runoff relationships (Nemec 1972). However, in general and for short time periods, great complexities may occur when spasmodic rainfall is unevenly distributed over an area of varied topography and geological composition (Shaw 1992). Further, other physical and hydrological factors, such as evapotranspiration, infiltration and ground-water flow are very significant, and thus any direct relationship between rainfall alone and runoff may not be easily determined.

The nature of the relationships of rainfall to runoff over long periods can also be affected by the climate of the area, where the relationship can be modified by the occurrence of sequences of wet or dry seasons. In taking annual or monthly values for rainfall and runoff comparisons, seasonal effects are removed and consideration of changes in ground-water storage can be neglected (Shaw 1991). Then, the straightforward relationship of Runoff = Rainfall - Loss (mainly evaporation) is established.

Hydrologically, considering the loss of rainfall through evapotranspiration and infiltration to ground-water, runoff from a catchment is expected to be less than the rainfall. In other words, where the rainfall - runoff interrelationship is such that runoff from a catchment is greater than rainfall, the relationship is questionable. However, such a condition may exist in special environments such as karst, where extra-catchment flows from karst sinkholes through underground channels into the catchment may result in runoff being greater than the catchment rainfall.

Ok Mani stream is one of the major tributaries of Ok Tedi which drains into the Fly River. The stream drains the area immediately south of the giant Ok Tedi Gold and Copper Mine, located in one of the wettest areas of PNG and has a very high annual runoff. Ok Mani is a flashy mountain stream and storm flows are immediate and fast and transport high sediment loads. The catchment is covered in forest and is mostly surrounded by karst environment. One of the mine's overburden storage dumps is located in the catchment.

Because of the isolation of the catchment due to steep topography and dense vegetation, very little is known about it. OTML manage and monitor a few rainfall and stream-flow gauging stations within the catchment, including the automatic stream-flow and rainfall gauges at MANO4. Fieldwork for this study was carried out in the headwaters of Ok Mani in an attempt to resolve the apparent hydrological contradiction of runoff exceeding rainfall as recorded over time at MANO4.

The estimated peak flood-flows over the fieldwork period were lower than those gauged for the respective storms and sub-catchments. The runoff for all the storms in the sub-catchments was less than the rainfall gauged as some of the rainfall was lost to the catchment through evapotranspiration, infiltration, depression storage, etc. The rainfall-runoff result, although collected over a very short period of time, did not agree with that from OTML station MANO4, where runoff is surplus to that of the rainfall. These relationships can be explained by a number of factors.

Firstly, the low estimated flows could be caused by the non-existence of the assumptions associated with the estimation methods if the gauged flows were accurate. That is, either the assumption that the catchment parameters including area, rainfall and flood data and other parameters that were used to derive the runoff coefficients of the estimation methods were similar to those in the catchments (i.e., sub-catchments C1, C2 and C3) to which the methods were applied; and/or the assumption that the storm rainfall, duration and intensity recorded and used to compute the estimated peak flows were uniform throughout the storm events and the catchment. This assumption does not appear to be supported as the evidence from the areal rainfall in Table 2.3 and the hypsometric curve in Figure 2.10 shows, where rainfall increases with increasing elevation. Any error in these assumptions could have resulted in the low estimated peak flows. Secondly, if the estimated peak flood-flows were the actual runoff from the storms, the high gauged peak flood-flows could be caused by the errors associated with the gauging technique applied, the number of the stream channel cross-sections selected and the stream-bed roughness characteristics.

The surplus runoff at MANO4 could be caused by a number of factors including under-estimation of runoff, i.e., high rainfall in other parts of the catchment and possible extra-catchment flows. The factors of storm runoff for some storms and in some sub-catchments (rainfall, intensity, duration, catchment area, stream length, peak flows, time to peak of the flood) had little association with one another and some of the relationships were unusual. These relationships could have been indicated and caused by nonuniform distribution of rainfall and the shifting of storm centres in the catchment. The uneven rainfall distribution is also evident from the areal rainfall of Ok Mani catchment in Table 2.3 and the hypsometric curve in Figure 2.10.

The period of fieldwork was very short (4 days) and the investigation covered a very small part of the catchment. Therefore, the data collected and analyzed was very limited. Further, the lack of knowledge of the catchment and the errors associated with the gauging and runoff data analysis means that any definite cause(s) of the surplus runoff recorded at MANO4 cannot be established. Therefore, other possible cause(s) are still open and require further and more detailed investigations. For instance, there is the possibility that the surplus runoff recorded at MANO4 could come from sub-surface flows into the catchment from the karst sinkholes in the adjacent catchments. Further, the overlapping boundaries of Ok Mani catchment and the adjacent catchments during high rainfall and ground-water levels, could be one other source of extra-catchment flows. Still, more than one of these sources could contribute to the excess water in the Ok Mani stream. Obviously, further detailed study and knowledge is required of the catchment. This would include both the climate (rainfall and distribution) and physical catchment parameters.

As a result of the results produced in this research, OTML has installed automatic raingauges in three different locations in the higher elevations of the catchment to monitor rainfall there. I would suggest OTML also investigate the possibility of extra-catchment flows from the adjacent catchments' karst sinkholes and the extra-catchment area from the overlapping of catchments. This could be done by carrying out subsurface flow tracing using dyes, salt, ammonium sulphate, or other methods. With the rainfall data from the higher elevations of the catchment and the findings from the subsurface flow tracing, the Ok Mani stream-flow discrepancy can then be put into perspective.

The results also highlight the extreme difficulties experienced in obtaining field data from such remote and accessibly impossible environments. Were it not for

the provision of a helicopter during the gaugings, this fieldwork could not have been carried out.

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APPENDICES

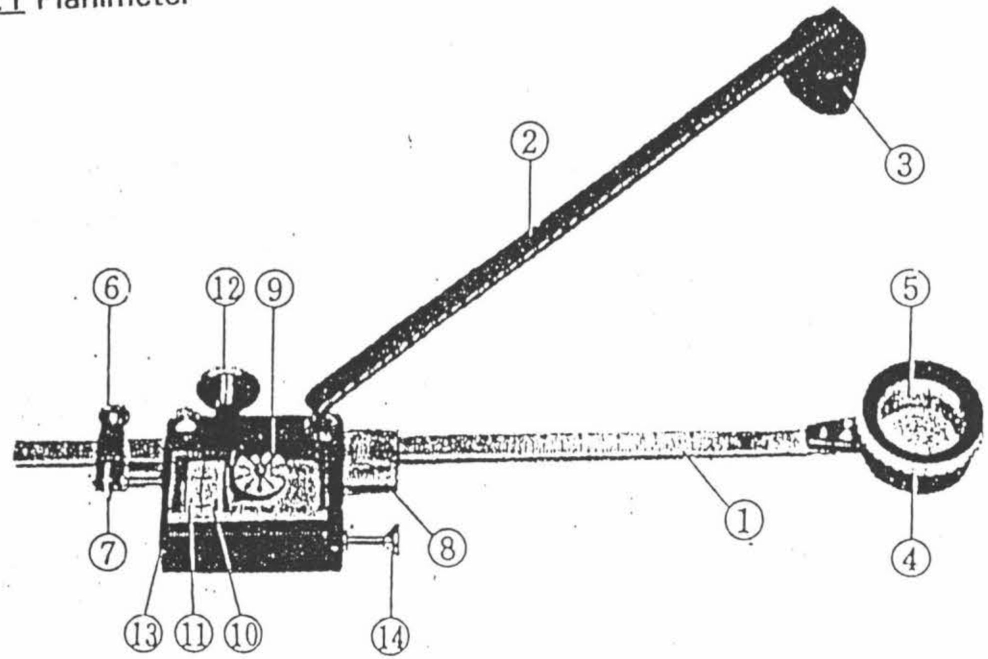
APPENDIX I:

Catchment Area Measurement

I.1 Planimeter

I.2 Procedures

Appendix I.1 Planimeter



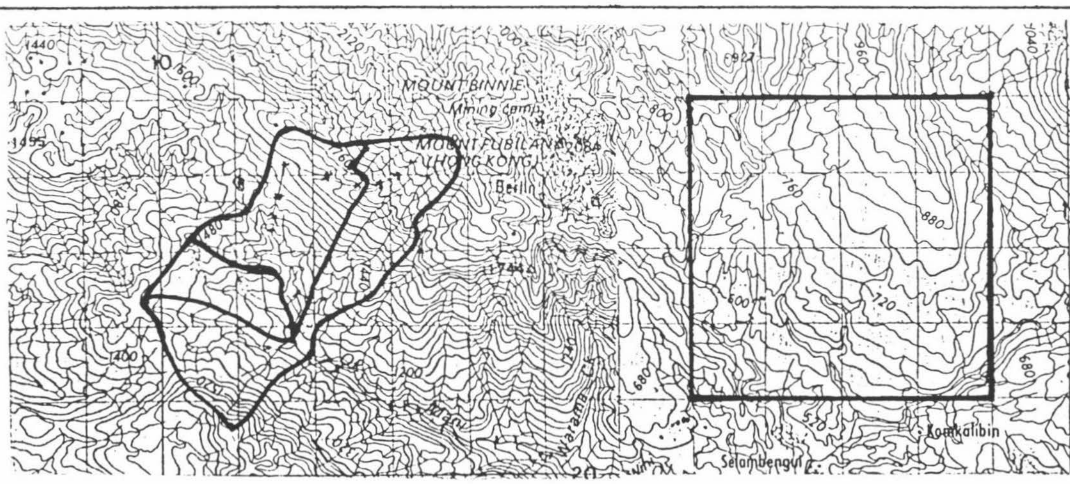
Part	1	Tracer Arm	Part	8	Tracer Arm Vernier
〃	2	Pole Arm	〃	9	Revolution Recording Dial
〃	3	Pole Weight	〃	10	Measuring Wheel
〃	4	Hand Grip	〃	11	Measuring Wheel Vernier
〃	5	Tracing Magnifier	〃	12	Idler Wheel
〃	6	Clamp Screw	〃	13	Carriage
〃	7	Fine Movement Screw	〃	14	Zero Setting Slide Bar

Appendix 1.2

Procedures

The area of the sub-catchments were measured with a Planimeter and were calculated applying the Test-square Proportion technique and not the settings on the adjustable arm on the planimeter (see Appendix I.1). This was because the settings of most planimeters currently in use are not related to metric measure and the settings on the planimeter take no account of the different coefficients of friction which will apply for the different kinds of paper or drawing material on which the instrument may be used. The procedures are:

a. On the photocopied map (1:100 000 topographic map, see Figure 1.6) and with the aide of the grid lines, an accurate test-square of sides 4 cm was drawn as shown in the figure below.



(a) Catchment Area (b) Test square

b. The planimeter was set up as shown on the catchment map as shown in Appendix I.1. The anchor point was placed outside the test-square and the tracing point, starting at one point, was moved carefully (freehand) around the perimeter of the test-square. The readings on the dial, the measuring wheel and the vernier (see Appendix I.1 for parts) were taken at the beginning and end of the operation. The operation was repeated two more times as a check. The difference between the beginning and end readings each time were taken and finally a mean was computed. Below are the results of the exercise;

1.555 to 1.718 = 0.163
1.718 to 1.870 = 0.152
1.870 to 2.032 = 0.162
mean difference = 0.477/3 = 0.159

c. The area of the test-square, according to the scale of the drawing (1:100 000 map, see Figure 1.6) is 16 km^2 and the mean difference derived in Step ii is, as a result of tracing the area of the test-square with the tracing point.

Any other area, drawn to the same scale as the test-square, can be deduced by simple proportion technique with the anchor pointing outside the area to be measured. Suppose the mean difference calculated for the test-square (0.159) was 'X', the area of any sub-catchment, drawn to the same scale as the test-square, is given by:

$$X(16/0.159) = (100.63)X \text{ km}^2$$

The area of the sub-catchments, all of scale 1:100 000, when the beginning and end planimeter readings around their perimeter were known, were deduced by this method and are presented in Table 2.1.

APPENDIX II:

Evaporation Rate of Bintulu Climatological Station, Indonesia

Appendix II: Evaporation Rate of Bintulu Climatological Station - Indonesia

MEAN MONTHLY EVAPORATION CALCULATION SHEET

STATION: Bintulu Airport NO. 3130202 (In INCH)

YEAR	January	February	March	April	May	June	July	August	September	October	November	December	Total
1964	6.11	4.91	6.37	7.01	5.87	5.64	6.54	5.96	6.18	6.16	6.49	5.51	72.75
1965	5.66	4.83	5.46	5.95	5.63	6.19	5.82	5.37	6.23	6.65	5.96	5.40	68.15
1966	6.52	5.70	6.77	6.23	6.46	6.52	6.04	6.43	6.19	6.29	5.40	5.80	74.35
1967	5.62	5.22	5.87	6.78	6.11	5.97	5.40	5.81	5.89	5.68	6.50	5.72	71.57
1968	5.65	5.06	5.35	5.78	6.86	6.07	5.56	6.37	6.16	5.62	5.46	5.56	69.50
1969	5.65	5.09	6.17	5.53	5.90	4.71	5.32	6.28	5.21	6.07	4.69	4.91	65.53
1970	4.49	5.23	7.07	5.76	6.65	5.47	5.16	5.70	5.93	5.82	4.81	4.81	66.90
1971	4.88	4.72	5.81	5.91	6.07	5.59	5.91	5.84	5.49	5.08	4.68	5.15	65.13
1972	4.72	4.33	5.23	5.14	5.52	4.69	5.94	5.08	4.99	5.16	4.65	5.28	60.73
1973	4.75	5.06	5.66	5.16	5.40	5.39	5.62	5.29	4.81	5.15	4.35	4.07	60.71
1974	4.13	3.86	5.41	5.51*	5.19	5.64	5.65	5.52	5.16*	5.00	5.13	4.84	61.04
1975	4.94	4.51	4.80	5.75	5.74	5.11	5.74	5.73	5.18	5.46	5.56	5.24	63.76
1976	4.47	4.93	5.66	5.67	5.47	5.92	5.98	5.21	5.43	4.91	4.84	5.04	63.53
1977	4.52	3.56	4.39	5.42	5.86	5.47	4.85	5.10	5.86	4.76	4.84	4.28	58.91
1978	3.90	3.89	5.11	5.24	4.85	5.21	5.09	5.18	5.02	4.88	4.25	4.95	57.57
TOTAL	76.01	70.90	85.13	86.84	87.58	83.59	84.62	84.87	83.73	82.69	77.61	76.56	980.13
MEAN	5.07	4.73	5.68	5.79	5.84	5.57	5.64	5.66	5.58	5.51	5.17	5.10	65.34

NO. OF YEARS OF RECORD 15 . MEAN MONTHLY AVERAGE : 5.45in or 138.43mm

NOTE: * Record incomplete.

APPENDIX III:

Fieldwork Itinerary

Appendix III

Field Work Itinerary

The field work for this thesis was undertaken in the upper catchment of Ok Mani stream in the Ok Tedi region of Western Province, PNG. The field work commenced on 13.02.95 and was completed on 25.03.95, a period of 6 weeks. The first 2 weeks were spent in the Environment Department - OTML preparing for the field work and collecting existing information on the catchment and the project. The third and fifth week were spent in the field collecting field data and the fourth and sixth week analysing the field data. Below is a brief itinerary of the field work:

Week 1:	13.02.95 - 28.02.95
13.02.95	Left home and flew to Tabubil, the mining township of Tabubil. Met Environment Department personnel and arranged accommodation and boarding.
14.02.95	Project briefing by Senior Hydrographer, Mr. John Wom and the Manager - Environment, Mr. Ian Wood.
15.02.95 - 18.02.95	Study of background and existing information including the catchment physical and climatological parameters in the Environment Department's library.
Week 2:	20.02.95 - 25.02.95
	Preparation of field work equipment and camping gear.
Week 3:	27.02.95 - 05.03.95
27.02.95	Flew out to field work site by helicopter and camped.
28.02.95	Surveyed the field work area (sub-catchments C1, C2 and C3). Identified and prepared sites and equipments of gauging in the main streams in the sub-catchments.
01.03.95	Identified, cleared and installed raingauges in the sub-catchments (C1, C2 and C3).
02.03.95 -	

04.03.95	Gauged storm flows and rainfall in the sub-catchments.
05.03.95	Packed and flew back to Tabubil by helicopter.
<u>Week 4</u>:	06.03.95 - 11.03.95
	Analyzed the field data collected and prepared field equipment and camping gear for the second field visit.
<u>Week 5</u>:	13.03.95 - 20.03.95
13.03.95	Flew out to the field work site (sub-catchment C1, C2 and C3) by helicopter and camped.
14.03.95	Installed raingauges in the sub-catchments (C1, C2 and C3) and prepared gauging sites and equipment.
15.03.95	Gauged both base and storm flows in the main streams and checked and recorded rainfall for the storm.
16.03.95	Packed and flew out by helicopter to sub-catchment C4. Camped and installed raingauge in the area. Identified stream flow gauging site along the main stream channel.
17.03.95	Gauged base flow of main stream and collected storm rainfall data. No storm flow was gauged as the storm occurred in the night and was risky and hazardous.
18.03.95	Packed and flew out by helicopter to sub-catchment C5. Camped and installed raingauge in the area. Identified stream flow gauging site along the main stream channel.
19.03.95	Gauged base flow of main stream and collected storm rainfall data. No storm flow was gauged as the storm occurred in the night and was risky and hazardous.
20.03.95	Field work completed. Packed and flew back to Tabubil.
<u>Week 6</u>:	21.03.95 - 26.03.95
	Field data processing and collection of other relevant information regarding the catchment and the project in the Environment Department's library.
26.03.95	Left Tabubil for home.
15.04.95	Left Papua New Guinea for New Zealand.

APPENDIX IV:

Stream Base-flow Measurement

IV.1 Current Meter and Method of Calibration

IV.2 Description of Preparation and Application of Current Meter

Appendix IV.1 Current Meter and Method of Calibration

(a) Current Meter

(b) CERTIFICATE OF CALIBRATION

Current Meter Model:	OSS-B1	Serial No.	94-10
Fan No.	A	Serial No.	94-10
Diameter:	100 mm	Pitch:	0.125 m
Type of Support:	Composite		
Method of Calibration:	Average Value Equation		

$n < 0.85 \quad V = 0.1341\,n + 0.0131$ m/s

$0.85 < n < 7.07 \quad V = 0.1276\,n + 0.0187$ m/s

$n > 7.07 \quad V = 0.1324\,n - 0.0155$ m/s

Starting Velocity = 0.030 m/s
Maximum Velocity = 5.000 m/s

Note: 'n' denotes the number of revolutions of the propeller per second and 'V' the water velocity in meters per second.

Appendix IV.2

Description of Stream-flow and Velocity Measurements

The stream channel section identified along the reach for base flow gauging were modified by removing boulders, vegetation and other debris to improve the gauging point. Dykes were also constructed using earth and rocks to cut off areas of dead water and to spread the flow evenly across the section.

The current meter was set-up for the base-flow and velocity measurements as shown in Appendix IV.1. The procedures for the measurements were:

a. The channel cross-section at the gauging point was divided into segment areas (SA1, SA2, SA3, SA4 and SA5, see Figure 3.11). The segments' surface widths were read-off from the tape measure and recorded.

b. The graduated wading rod was lowered to the stream bed at each vertical (A, B, C and D, see Figure 3.11) starting at vertical A. Once the bottom end of the rod reached the stream bed, the depth (m) was read-off from the graduated (by 0.1 metre) wading rod and recorded.

c. The depth (m) of the vertical obtained in step 'b' was multiplied by 0.2 to determine the depth (m) from the stream bed to measure the flow velocity for the lower part of the vertical (at A2 for instance, see Figure 3.11). Once the depth (m) was determined at 0.2 of the depth (m), the current meter propeller head was adjusted and tightened along the rod at the predetermined depth (m) (at 0.2 of the depth) from the bottom end.

d. The wading rod with the current meter propeller was then lowered back into the same vertical to the stream bed. After the propeller was at 0.2 of the depth (m) and facing the flow of the stream, the timer on the current meter counter was set at 10 seconds and switched on.

e. The reading stopped after 10 seconds as the timer was set (step 'd'). The spinning of the propeller by the force of flowing water were recorded as the number of revolutions per unit time interval. The reading was expressed as revolutions (r) per second (i.e., r/10 = n, where n is the pro-numeral in the formulas for stream flow velocity calculation) and recorded.

f. When step 'e' was completed, the depth (m) of vertical A was again multiplied by 0.8, this time to measure the flow velocity at the upper depth of the vertical. The product of 0.8 by the depth (m) was subtracted by the product of 0.2 by the depth (m) (see step 'c') so that the distance (m) between 0.2 and 0.8 of the depth (m) is known in order for the propeller to be re-adjusted along the wading rod. The difference was the distance (m) the propeller was shifted along the rod from 0.2 of the depth (m) where the last stream flow velocity was taken.

g. At 0.8 of the vertical depth (m), steps 'e' and 'f' (but at 0.8 of the depth (m)) were repeated.

h. After the vertical stream flow velocity measurements at vertical A were completed, the procedures 'b' to 'g' were repeated at the other verticals (B, C and D, see Figure 3.11) of the cross-section. The whole procedure 'a' to 'h' was repeated during each base flow velocity gauging session at the gauging points (V, W and X, see Figure 2.7).

i. The area of each segment was obtained by multiplying the vertical depth (m) with the segment surface distance (m) of the segment (obtained in steps 'a' and 'b').

j. The flow in each segment area (q1, q2, q3, q4 and q5, see Figure 3.11) was calculated using the Mid-Section method. This method assumes that the flow velocity at each vertical represents the mean flow velocity in a segment area that extends laterally from half the distance from the preceding vertical to half the distance to the next and vertically from the water surface to the bed. That is:

$$q1 = (V0 + V1)/2 \times (d0 + d1)/2 \times (b1 - b0)$$
$$q2 = (V1 + V2)/2 \times (d1 + d2)/2 \times (b2 - b1)$$
$$q3 = (V2 + V3)/2 \times (d2 + d3)/2 \times (b3 - b2)$$
$$q4 = (V3 + V4)/2 \times (d3 + d4)/2 \times (b4 - b3)$$
$$q5 = (V4 + V5)/2 \times (d4 + d5)/2 \times (b5 - b4)$$

where, V0 was the flow velocity at the stream edge (assumed to be zero), V1 for segment area 1, V2 for segment area 2, V3 for area 3, V4 for segment area 4 and V5 was the mean stream flow velocity (m/s) for segment area 5. D0 was the depth (m) of the vertical at the stream edge (assumed to be zero), d1 was the depth (m) of vertical A, d2 for vertical B, d3 for vertical C, d4 for vertical D and d5 was the depth (m) (assumed to be zero) of the stream edge on the opposite bank of the channel cross-section. b0 was the surface distance at the stream edge (assumed to be zero), b1 was the surface distance (m) between the stream edge and vertical A, b2 between verticals A and B, b3 between verticals B and C, b4 between verticals C and D and b5 was the surface distance (m) between vertical D and the stream edge on the opposite bank of the channel cross-section (see Figure 3.11).

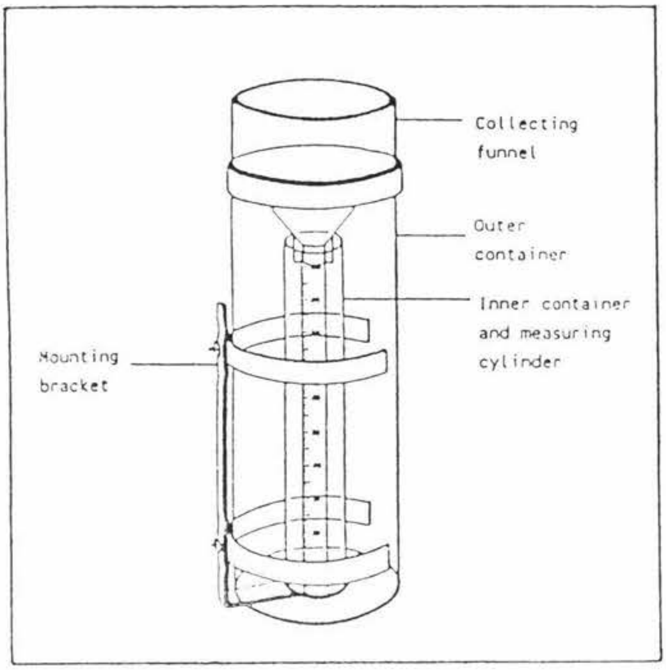
APPENDIX V:

Storm Rainfall Measurement

V.1 Raingauge: 4-Inch Standard Marquis 1000

V.2 Description and Preparation, Installation and Reading and Recording of Rainfall

Appendix V.1 Raingauge



(a) Marquis 1000 Raingauge

(b) Installed Raingauge

APPENDIX V.2

Description of Preparation and Installation of Raingauges and Reading and Recording of Storm Rainfall

The type of raingauges used in the field work is given in Appendix V.1. The procedures of assembling and installations are:

a. To assemble and install a raingauge, a wooden pole of 2 metres was cut and driven half a metre into the ground at the centre of the cleared site in the sub-catchments.

b. With the semicircular bracket at the top and the "scoop" shaped base-support bracket at the bottom, they were fitted to the strap and held by the screws through the holes. The set was then screwed on tightly to the top-end of the pole as shown in Appendix V.1.

c. The measuring tube was fitted into a small circular rim at the bottom centre of the reservoir. The collector with the funnel was fitted-on on top of the reservoir, which the opened-end of the measuring tube fitted directly under the funnel outlet. The raingauge was then fitted into the bracket which sat squarely in the "scoop" shaped base-support and held upright by the semicircular bracket. A spirit level was used to check that the rim was level in all directions.

d. To do the reading and recording, the collector with the funnel was removed from the top of the raingauge and the measuring tube carefully withdrawn from the reservoir (see Appendix V.1). The measuring tube was then held vertically at eye level and the rainwater was read-off to the lower edge of the meniscus against the graduated scale and recorded. The measuring tube holds up to 25 mm of rainwater. For some of the storm events, rainfall exceeded 25 mm. The excess rainwater collected by the collector spilled out over the measuring tube and was collected in the reservoir, which is capable of holding 250 mm of rainwater. In such cases, after recording and pouring out the first full from the measuring tube, the excess rainwater collected in the reservoir was carefully poured into the measuring tube and a second reading was taken. The procedure was repeated until there was no rainwater left in the reservoir.

e. The sum of all the readings was taken as the rainfall depth (mm) for the storm event.

APPENDIX VI:

Flood-flow Analysis

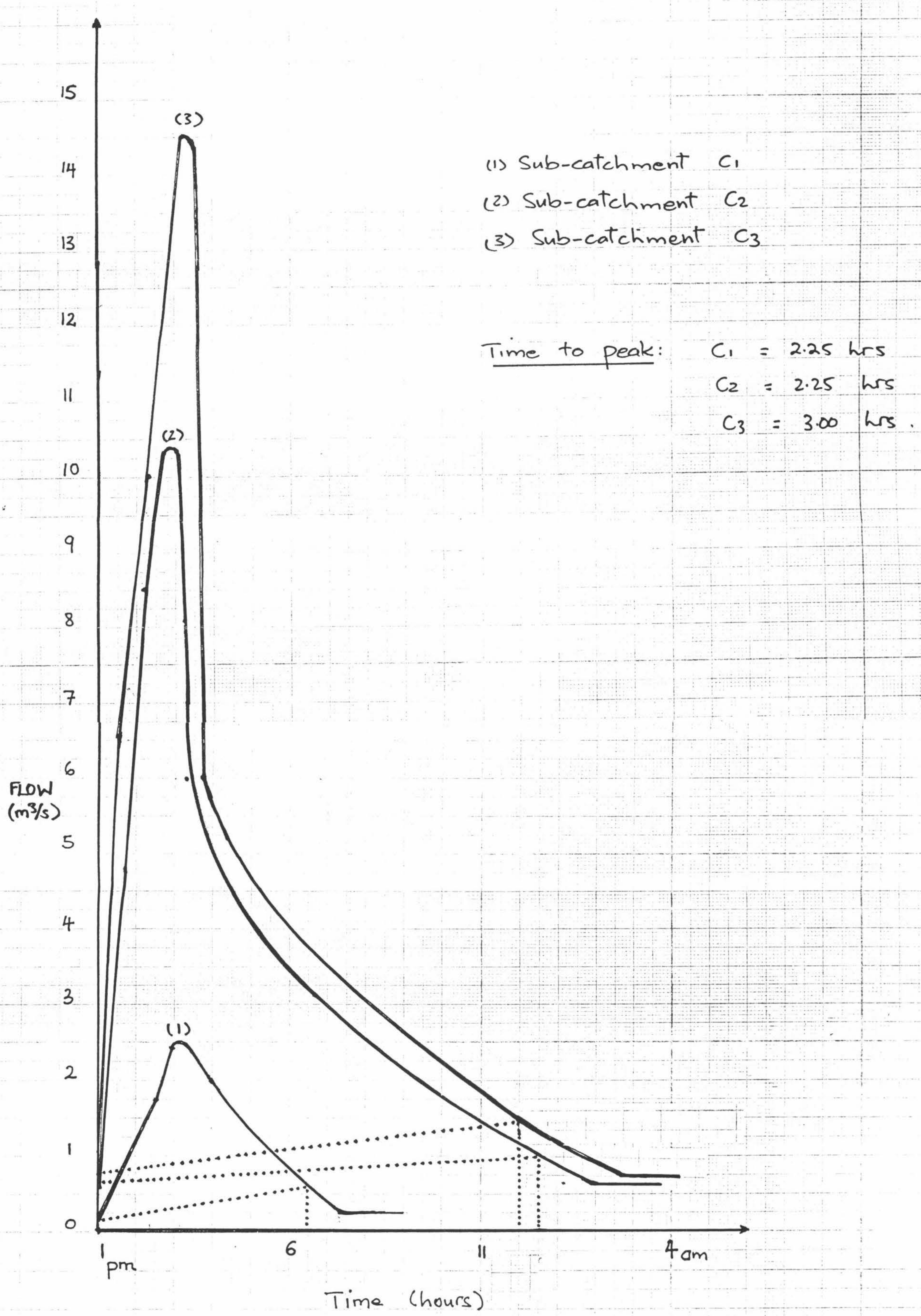
VI.1 Flood Hydrographs

VI.2 Test-square Proportion Technique

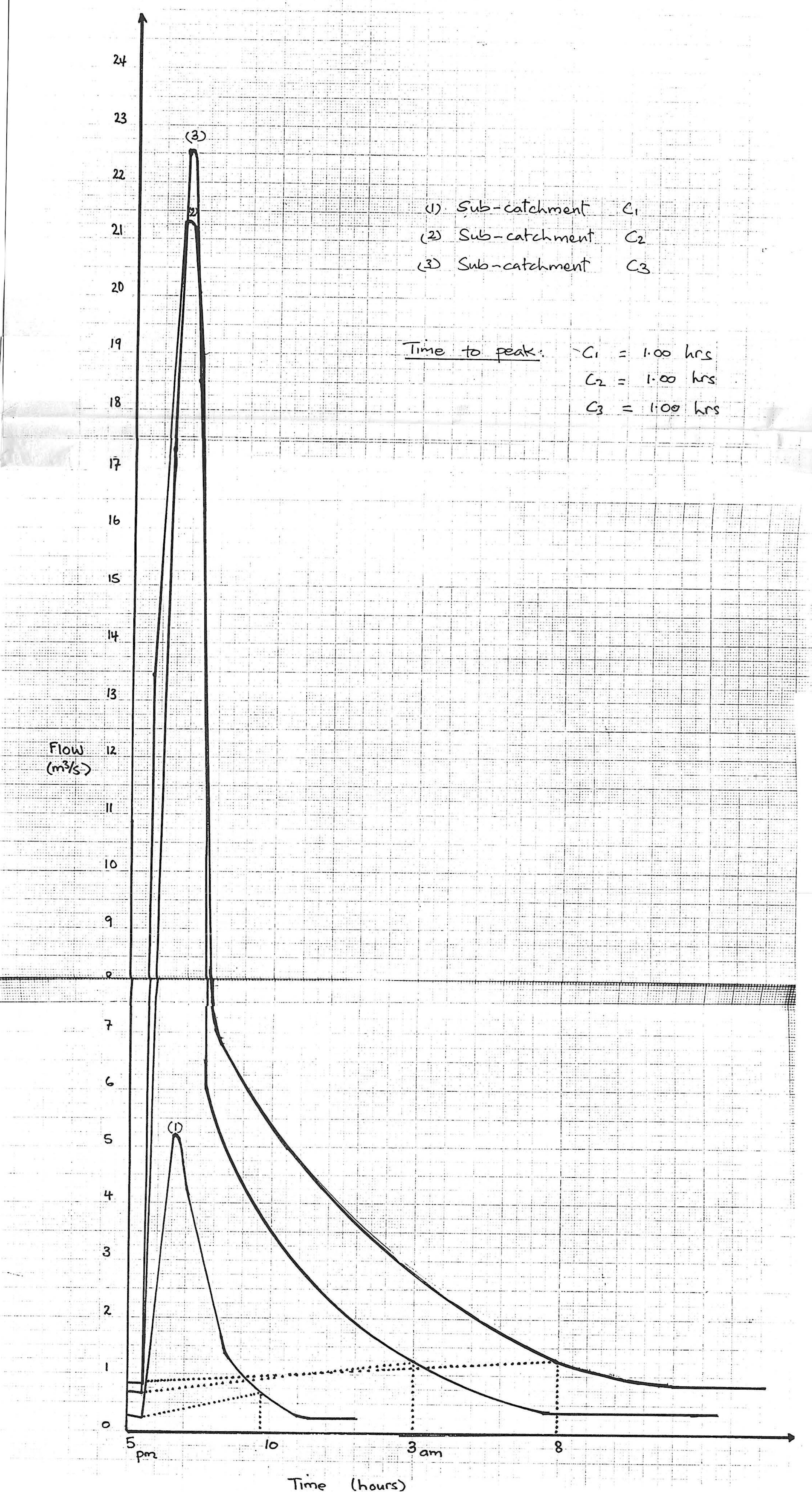
<u>APPENDIX VI.1</u>:

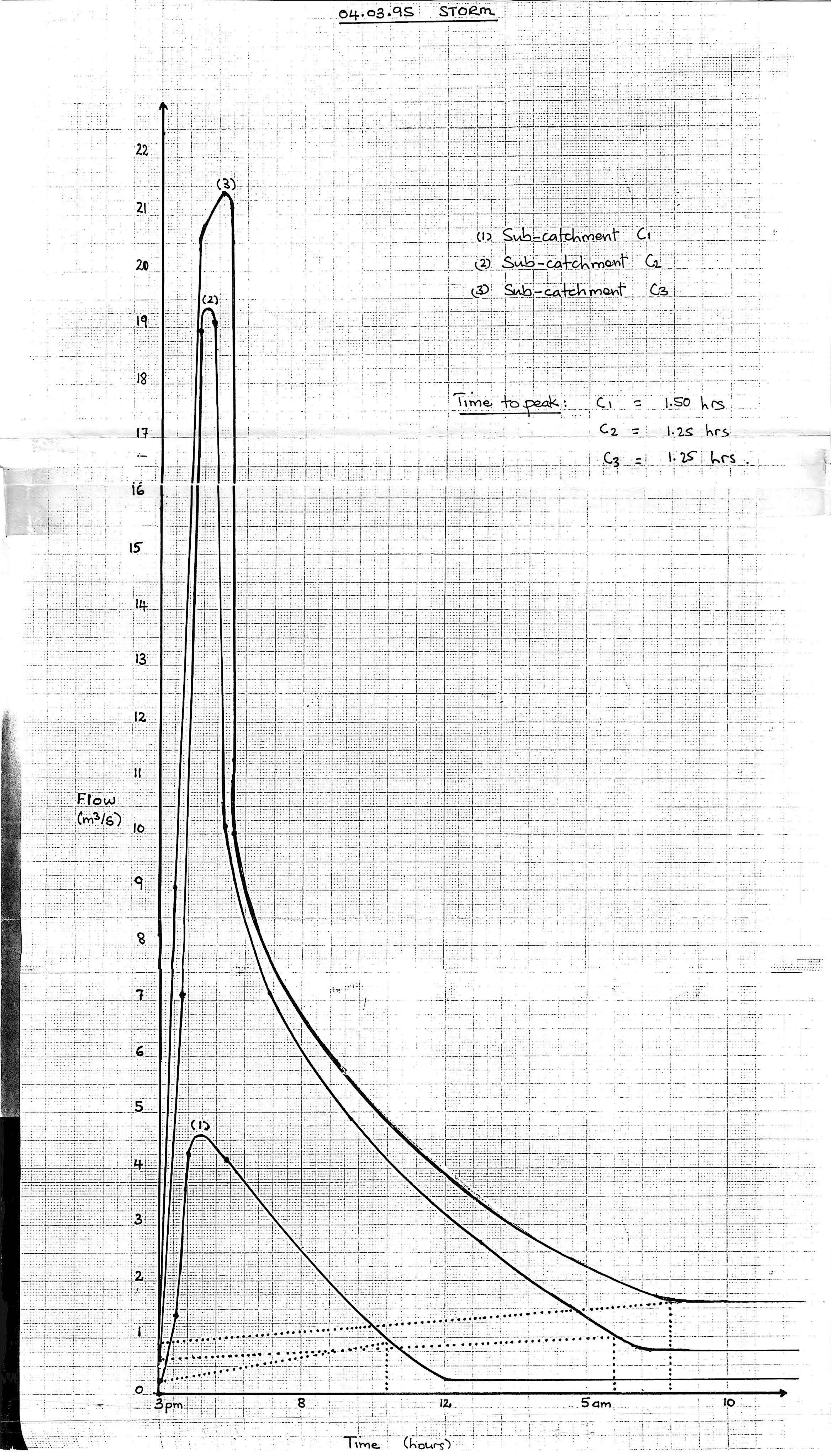
Flood Hydrographs

02.03.95 STORM

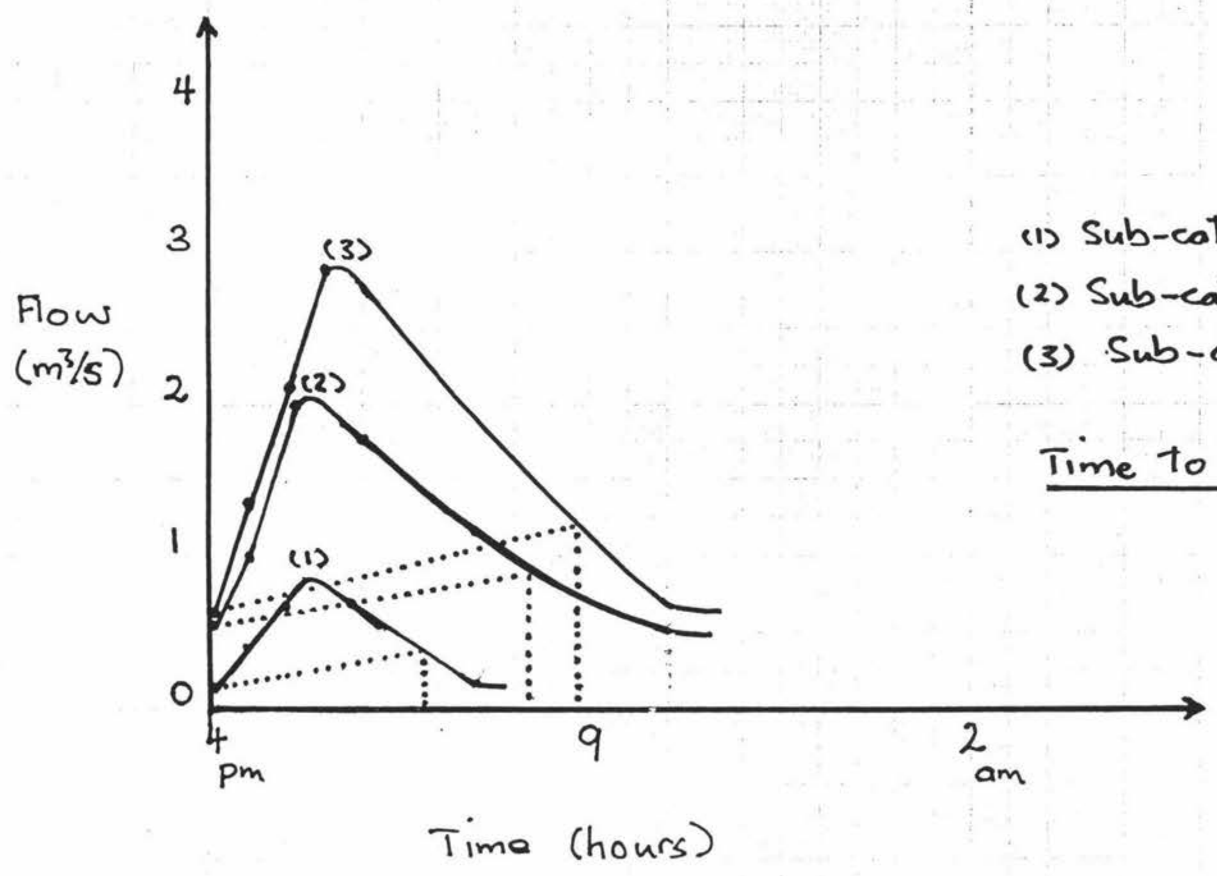


15.03.95 STORM




04.03.95 STORM
(1) Sub-catchment C1
(2) Sub-catchment C2
(3) Sub-catchment C3
Time to peak: C1 = 1.50 hrs
C2 = 1.25 hrs
C3 = 1.25 hrs.
Flow (m3/s)
Time (hours)
3pm
8
12
5am
10
(1)
(2)
(3)

03.03.95 STORM



(1) Sub-catchment C_1

(2) Sub-catchment C_2

(3) Sub-catchment C_3

Time to peak:

C_1 = 1.50 hrs

C_2 = 1.25 hrs

C_3 = 1.50 hrs

APPENDIX VI.2:

Test-square Proportion Technique

APPENDIX VI .2: **Test-square Proportion Technique**

Test Square

4 cm x 4 cm = 16 cm^2 = 2 m^3/s x 4 hours x 3600 seconds
= 28 800 m^3

Planimeter exercise of test-square			
Readings:	0 to 0.183	=	0.183
	0.183 to 0.346	=	0.163
	0.346 to 0.529	=	0.183
	0.529 to 0.691	=	0.162
	Mean difference:		0.691/4 = 0.173

Therefore, the volume of runoff of a flood hydrograph drawn to the same scale as the test-square and planimetered is given by:

$$X \,.\, 28800/0.173 = \underline{166474X}\ m^3$$

where 'X' is the difference between the beginning and end planimeter readings of area under the hydrograph.

APPENDIX VII:

Base-flow Velocity and Cross-section Area Field Measurements

OK MANI CHANNEL CROSS-SECTION AND FLOW VELOCITY FIELD MEASUREMENTS

ate: 02.03.95

ub-catchment: C1

auging Session: 1

Seg-ment	Depth (m)	Dist. to Point (m)	Segment Dist. (m)	Vertical Point	Point Depth (m)	Revs	Time (sec)	Revs/ (sec)	Vel. (m/s)	Av.Vel (m/s)	Mean Vel (m/s)	Area (m2)	Flow (m3/s)
WE1	0.00	0.00		0.00	0.00	0	10	0.00	0.00	0.00			
A	0.34	0.70	0.70	0.20	0.07	59	10	5.90	0.77		0.25	0.12	0.03
				0.80	0.27	18	10	1.80	0.25	0.51			
B	0.36	1.20	0.50	0.20	0.07	16	10	1.60	0.22		0.35	0.18	0.06
				0.80	0.29	10	10	1.00	0.15	0.18			
C	0.34	2.00	0.80	0.20	0.07	42	10	4.20	0.55		0.28	0.28	0.08
				0.80	0.27	13	10	1.30	0.18	0.37			
D	0.00	3.20	1.20	0.20	0.00	0	10	0.00	0.02		0.19	0.20	0.04
				0.80	0.00	0	10	0.00	0.02	0.02			
WE2	0.00	3.20	0.00	0.00	0.00	0	10	0.00	0.00	0.00	0.01	0.00	0.00
											0.27	**0.78**	**0.21**

Gauging Session: 2

Seg-ment	Depth (m)	Dist. to Point (m)	Segment Dist. (m)	Vertical Point	Point Depth (m)	Revs	Time (sec)	Revs/ (sec)	Vel. (m/s)	Av.Vel (m/s)	Mean Vel (m/s)	Area (m2)	Flow (m3/s)
WE1	0.00	0.00		0.00	0.00	0	10	0.00	0.00	0.00			
A	0.34	0.70	0.70	0.20	0.07	59	10	5.90	0.77		0.27	0.12	0.03
				0.80	0.27	22	10	2.20	0.30	0.54			
B	0.34	1.20	0.50	0.20	0.07	23	10	2.30	0.31		0.39	0.17	0.07
				0.80	0.27	13	10	1.30	0.18	0.25			
C	0.32	2.00	0.80	0.20	0.06	47	10	4.70	0.62		0.34	0.26	0.09
				0.80	0.26	19	10	1.90	0.26	0.44			
D	0.00	3.20	1.20	0.20	0.00	0	10	0.00	0.02		0.23	0.19	0.04
				0.80	0.00	0	10	0.00	0.02	0.02			
WE2	0.00	3.20	0.00	0.00	0.00	0	10	0.00	0.00	0.00	0.01	0.00	0.00
											0.31	**0.75**	**0.23**

Sub-catchment: C2

Gauging Session: 1

Seg-ment	Depth (m)	Dist. to Point (m)	Segment Dist. (m)	Vertical Point	Point Depth (m)	Revs	Time (sec)	Revs/ (sec)	Vel. (m/s)	Av.Vel (m/s)	Mean Vel (m/s)	Area (m2)	Flow (m3/s)
WE1	0.00	0.00		0.00	0.00	0	10	0.00	0.00	0.00			
A	0.30	2.10	2.10	0.20	0.06	34	10	3.40	0.45		0.21	0.32	0.07
				0.80	0.24	28	10	2.80	0.38	0.41			
B	0.28	3.10	1.00	0.20	0.06	40	10	4.00	0.53		0.43	0.29	0.13
				0.80	0.22	28	10	2.80	0.38	0.45			
C	0.40	4.10	1.00	0.20	0.08	12	10	1.20	0.17		0.32	0.34	0.11
				0.80	0.32	13	10	1.30	0.18	0.18			
D	0.46	5.60	1.50	0.20	0.09	38	10	3.80	0.50		0.33	0.65	0.22
				0.80	0.37	36	10	3.60	0.48	0.49			
WE2	0.00	6.60	1.00	0.00	0.00	0	10	0.00	0.00	0.00	0.25	0.23	0.06
											0.31	**1.82**	**0.57**

FIELDATA

Gauging Session: 2

Seg-ment	Depth (m)	Dist. to Point (m)	Segment Dist. (m)	Vertical Point	Point Depth (m)	Revs	Time (sec)	Revs/ (sec)	Vel. (m/s)	Av.Vel (m/s)	Mean Vel (m/s)	Area (m2)	Flow (m3/s)
WE1	0.00	0.00		0.00	0.00	0	10	0.00	0.00	0.00			
A	0.30	2.10	2.10	0.20	0.06	34	10	3.40	0.45		0.15	0.32	0.05
				0.80	0.24	10	10	1.00	0.15	0.30			
B	0.28	3.10	1.00	0.20	0.06	36	10	3.60	0.48		0.37	0.29	0.11
				0.80	0.22	31	10	3.10	0.41	0.45			
C	0.40	4.10	1.00	0.20	0.08	9	10	0.90	0.13		0.29	0.34	0.10
				0.80	0.32	10	10	1.00	0.15	0.14			
D	0.46	5.60	1.50	0.20	0.09	36	10	3.60	0.48		0.29	0.65	0.19
				0.80	0.37	31	10	3.10	0.41	0.45			
WE2	0.00	6.60	1.00	0.00	0.00	0	10	0.00	0.00	0.00	0.22	0.23	0.05
											0.27	**1.82**	**0.50**

Sub-catchment: C3

Gauging Session: 1

Seg-ment	Depth (m)	Dist. to Point (m)	Segment Dist. (m)	Vertical Point	Point Depth (m)	Revs	Time (sec)	Revs/ (sec)	Vel. (m/s)	Av.Vel (m/s)	Mean Vel (m/s)	Area (m2)	Flow (m3/s)
WE1	0.00	0.00		0.00	0.00	0	10	0.00	0.00	0.00			
A	0.98	2.00	2.00	0.20	0.20	2	10	0.20	0.04		0.02	0.98	0.02
				0.80	0.78	1	10	0.10	0.03	0.03			
B	1.04	3.50	1.50	0.20	0.21	17	10	1.70	0.24		0.11	1.52	0.16
				0.80	0.83	9	10	0.90	0.13	0.18			
C	0.64	5.00	1.50	0.20	0.13	14	10	1.40	0.20		0.18	1.26	0.22
				0.80	0.51	10	10	1.00	0.15	0.17			
D	0.42	6.50	1.50	0.20	0.08	20	10	2.00	0.27		0.22	0.80	0.18
				0.80	0.34	20	10	2.00	0.27	0.27			
WE2	0.00	8.50	2.00	0.00	0.00	0	10	0.00	0.00	0.00	0.14	0.42	0.06
											0.13	**4.97**	**0.64**

Gauging Session: 2

Seg-ment	Depth (m)	Dist. to Point (m)	Segment Dist. (m)	Vertical Point	Point Depth (m)	Revs	Time (sec)	Revs/ (sec)	Vel. (m/s)	Av.Vel (m/s)	Mean Vel (m/s)	Area (m2)	Flow (m3/s)
WE1	0.00	0.00		0.00	0.00	0	10	0.00	0.00	0.00			
A	0.98	2.00	2.00	0.20	0.20	1	10	0.10	0.03		0.01	0.98	0.01
				0.80	0.78	0	10	0.00	0.01	0.02			
B	1.08	3.50	1.50	0.20	0.22	13	10	1.30	0.18		0.09	1.55	0.14
				0.80	0.86	9	10	0.90	0.13	0.16			
C	0.64	5.00	1.50	0.20	0.13	13	10	1.30	0.18		0.16	1.29	0.21
				0.80	0.51	10	10	1.00	0.15	0.17			
D	0.42	6.50	1.50	0.20	0.08	21	10	2.10	0.29		0.23	0.80	0.18
				0.80	0.34	21	10	2.10	0.29	0.29			
WE2	0.00	8.50	2.00	0.00	0.00	0	10	0.00	0.00	0.00	0.14	0.42	0.06
											0.12	**5.03**	**0.60**

FIELDATA

Date: 03.03.95

Sub-catchment: C1

Gauging Session: 1

Seg-ment	Depth (m)	Dist. to Point (m)	Segment Dist. (m)	Vertical Point	Point Depth (m)	Revs	Time (sec)	Revs/ (sec)	Vel. (m/s)	Av.Vel (m/s)	Mean Vel (m/s)	Area (m2)	Flow (m3/s)
WE1	0.00	0.00		0.00	0.00	0	10	0.00	0.00	0.00			
A	0.32	0.70	0.70	0.20	0.06	59	10	5.90	0.77		0.25	0.11	0.03
				0.80	0.26	18	10	1.80	0.25	0.51			
B	0.34	1.20	0.50	0.20	0.07	24	10	2.40	0.32		0.40	0.17	0.07
				0.80	0.27	17	10	1.70	0.24	0.28			
C	0.34	2.00	0.80	0.20	0.07	49	10	4.90	0.64		0.40	0.27	0.11
				0.80	0.27	28	10	2.80	0.38	0.51			
D	0.00	3.20	1.20	0.20	0.00	0	10	0.00	0.02		0.26	0.20	0.05
				0.80	0.00	0	10	0.00	0.02	0.02			
WE2	0.00	3.20	0.00	0.00	0.00	0	10	0.00	0.00	0.00	0.01	0.00	0.00
											0.34	**0.75**	**0.26**

Gauging Session: 2

Seg-ment	Depth (m)	Dist. to Point (m)	Segment Dist. (m)	Vertical Point	Point Depth (m)	Revs	Time (sec)	Revs/ (sec)	Vel. (m/s)	Av.Vel (m/s)	Mean Vel (m/s)	Area (m2)	Flow (m3/s)
WE1	0.00	0.00		0.00	0.00	0	10	0.00	0.00	0.00			
A	0.34	0.70	0.70	0.20	0.07	58	10	5.80	0.76		0.26	0.12	0.03
				0.80	0.27	22	10	2.20	0.30	0.53			
B	0.36	1.20	0.50	0.20	0.07	17	10	1.70	0.24		0.36	0.18	0.06
				0.80	0.29	10	10	1.00	0.15	0.19			
C	0.34	2.00	0.80	0.20	0.07	49	10	4.90	0.64		0.33	0.28	0.09
				0.80	0.27	23	10	2.30	0.31	0.48			
D	0.00	3.20	1.20	0.20	0.00	0	10	0.00	0.02		0.25	0.20	0.05
				0.80	0.00	0	10	0.00	0.02	0.02			
WE2	0.00	3.20	0.00	0.00	0.00	0	10	0.00	0.00	0.00	0.01	0.00	0.00
											0.31	**0.78**	**0.24**

Sub-catchment: C2

Gauging Session: 1

Seg-ment	Depth (m)	Dist. to Point (m)	Segment Dist. (m)	Vertical Point	Point Depth (m)	Revs	Time (sec)	Revs/ (sec)	Vel. (m/s)	Av.Vel (m/s)	Mean Vel (m/s)	Area (m2)	Flow (m3/s)
WE1	0.00	0.00		0.00	0.00	0	10	0.00	0.00	0.00			
A	0.30	2.10	2.10	0.20	0.06	35	10	3.50	0.47		0.19	0.32	0.06
				0.80	0.24	22	10	2.20	0.30	0.38			
B	0.30	3.10	1.00	0.20	0.06	36	10	3.60	0.48		0.42	0.30	0.13
				0.80	0.24	32	10	3.20	0.43	0.45			
C	0.38	4.10	1.00	0.20	0.08	12	10	1.20	0.17		0.32	0.34	0.11
				0.80	0.30	14	10	1.40	0.20	0.18			
D	0.46	5.60	1.50	0.20	0.09	36	10	3.60	0.48		0.32	0.63	0.20
				0.80	0.37	32	10	3.20	0.43	0.45			
WE2	0.00	6.60	1.00	0.00	0.00	0	10	0.00	0.00	0.00	0.23	0.23	0.05
											0.30	**1.82**	**0.55**

Gauging Session: 2

Seg-ment	Depth (m)	Dist. to Point (m)	Segment Dist. (m)	Vertical Point	Point Depth (m)	Revs	Time (sec)	Revs/ (sec)	Vel. (m/s)	Av.Vel (m/s)	Mean Vel (m/s)	Area (m2)	Flow (m3/s)
WE1	0.00	0.00		0.00	0.00	0	10	0.00	0.00	0.00			
A	0.30	2.10	2.10	0.20	0.06	34	10	3.40	0.45		0.21	0.32	0.07
				0.80	0.24	28	10	2.80	0.38	0.41			
B	0.28	3.10	1.00	0.20	0.06	38	10	3.80	0.50		0.45	0.29	0.13
				0.80	0.22	34	10	3.40	0.45	0.48			
C	0.40	4.10	1.00	0.20	0.08	10	10	1.00	0.15		0.32	0.34	0.11
				0.80	0.32	11	10	1.10	0.16	0.15			
D	0.46	5.60	1.50	0.20	0.09	37	10	3.70	0.49		0.32	0.65	0.21
				0.80	0.37	36	10	3.60	0.48	0.48			
WE2	0.00	6.60	1.00	0.00	0.00	0	10	0.00	0.00	0.00	0.24	0.23	0.06
											0.31	**1.82**	**0.56**

Sub-catchment: C3

Gauging Session: 1

Seg-ment	Depth (m)	Dist. to Point (m)	Segment Dist. (m)	Vertical Point	Point Depth (m)	Revs	Time (sec)	Revs/ (sec)	Vel. (m/s)	Av.Vel (m/s)	Mean Vel (m/s)	Area (m2)	Flow (m3/s)
WE1	0.00	0.00		0.00	0.00	0	10	0.00	0.00	0.00			
A	0.96	2.00	2.00	0.20	0.19	2	10	0.20	0.04		0.02	0.96	0.02
				0.80	0.77	1	10	0.10	0.03	0.03			
B	1.08	3.50	1.50	0.20	0.22	14	10	1.40	0.20		0.10	1.53	0.15
				0.80	0.86	8	10	0.80	0.12	0.16			
C	0.62	5.00	1.50	0.20	0.12	12	10	1.20	0.17		0.15	1.28	0.19
				0.80	0.50	7	10	0.70	0.11	0.14			
D	0.42	6.50	1.50	0.20	0.08	22	10	2.20	0.30		0.22	0.78	0.17
				0.80	0.34	23	10	2.30	0.31	0.31			
WE2	0.00	8.50	2.00	0.00	0.00	0	10	0.00	0.00	0.00	0.15	0.42	0.06
											0.12	**4.97**	**0.59**

Gauging Session: 2

Seg-ment	Depth (m)	Dist. to Point (m)	Segment Dist. (m)	Vertical Point	Point Depth (m)	Revs	Time (sec)	Revs/ (sec)	Vel. (m/s)	Av.Vel (m/s)	Mean Vel (m/s)	Area (m2)	Flow (m3/s)
WE1	0.00	0.00		0.00	0.00	0	10	0.00	0.00	0.00			
A	0.98	2.00	2.00	0.20	0.20	2	10	0.20	0.04		0.02	0.98	0.02
				0.80	0.78	2	10	0.20	0.04	0.04			
B	1.08	3.50	1.50	0.20	0.22	13	10	1.30	0.18		0.11	1.55	0.17
				0.80	0.86	13	10	1.30	0.18	0.18			
C	0.64	5.00	1.50	0.20	0.13	14	10	1.40	0.20		0.18	1.29	0.23
				0.80	0.51	11	10	1.10	0.16	0.18			
D	0.44	6.50	1.50	0.20	0.09	23	10	2.30	0.31		0.24	0.81	0.20
				0.80	0.35	22	10	2.20	0.30	0.31			
WE2	0.00	8.50	2.00	0.00	0.00	0	10	0.00	0.00	0.00	0.15	0.44	0.07
											0.14	**5.07**	**0.69**

FIELDATA

Date: 04.03.95

Sub-catchment: C1

Gauging Session: 1

Seg-ment	Depth (m)	Dist. to Point (m)	Segment Dist. (m)	Vertical Point	Point Depth (m)	Revs	Time (sec)	Revs/ (sec)	Vel. (m/s)	Av.Vel (m/s)	Mean Vel (m/s)	Area (m2)	Flow (m3/s)
WE1	0.00	0.00		0.00	0.00	0	10	0.00	0.00	0.00			
A	0.30	0.70	0.70	0.20	0.06	53	10	5.30	0.69		0.24	0.11	0.03
				0.80	0.24	19	10	1.90	0.26	0.48			
B	0.34	1.20	0.50	0.20	0.07	27	10	2.70	0.36		0.38	0.16	0.06
				0.80	0.27	15	10	1.50	0.21	0.29			
C	0.34	2.00	0.80	0.20	0.07	43	10	4.30	0.57		0.34	0.27	0.09
				0.80	0.27	16	10	1.60	0.22	0.40			
D	0.00	3.20	1.20	0.20	0.00	0	10	0.00	0.02		0.21	0.20	0.04
				0.80	0.00	0	10	0.00	0.02	0.02			
WE2	0.00	3.20	0.00	0.00	0.00	0	10	0.00	0.00	0.00	0.01	0.00	0.00
											0.30	**0.74**	**0.22**

Gauging Session: 2

Seg-ment	Depth (m)	Dist. to Point (m)	Segment Dist. (m)	Vertical Point	Point Depth (m)	Revs	Time (sec)	Revs/ (sec)	Vel. (m/s)	Av.Vel (m/s)	Mean Vel (m/s)	Area (m2)	Flow (m3/s)
WE1	0.00	0.00		0.00	0.00	0	10	0.00	0.00	0.00			
A	0.32	0.70	0.70	0.20	0.06	49	10	4.90	0.64		0.21	0.11	0.02
				0.80	0.26	13	10	1.30	0.18	0.41			
B	0.32	1.20	0.50	0.20	0.06	28	10	2.80	0.38		0.36	0.16	0.06
				0.80	0.26	18	10	1.80	0.25	0.31			
C	0.34	2.00	0.80	0.20	0.07	38	10	3.80	0.50		0.35	0.26	0.09
				0.80	0.27	19	10	1.90	0.26	0.38			
D	0.00	3.20	1.20	0.20	0.00	0	10	0.00	0.02		0.20	0.20	0.04
				0.80	0.00	0	10	0.00	0.02	0.02			
WE2	0.00	3.20	0.00	0.00	0.00	0	10	0.00	0.00	0.00	0.01	0.00	0.00
											0.29	**0.73**	**0.21**

Sub-catchment: C2

Gauging Session: 1

Seg-ment	Depth (m)	Dist. to Point (m)	Segment Dist. (m)	Vertical Point	Point Depth (m)	Revs	Time (sec)	Revs/ (sec)	Vel. (m/s)	Av.Vel (m/s)	Mean Vel (m/s)	Area (m2)	Flow (m3/s)
WE1	0.00	0.00		0.00	0.00	0	10	0.00	0.00	0.00			
A	0.30	2.10	2.10	0.20	0.06	30	10	3.00	0.40		0.18	0.32	0.06
				0.80	0.24	24	10	2.40	0.32	0.36			
B	0.28	3.10	1.00	0.20	0.06	36	10	3.60	0.48		0.40	0.29	0.12
				0.80	0.22	31	10	3.10	0.41	0.45			
C	0.38	4.10	1.00	0.20	0.08	8	10	0.80	0.12		0.35	0.33	0.12
				0.80	0.30	29	10	2.90	0.39	0.25			
D	0.42	5.60	1.50	0.20	0.08	32	10	3.20	0.43		0.33	0.60	0.20
				0.80	0.34	30	10	3.00	0.40	0.41			
WE2	0.00	6.60	1.00	0.00	0.00	0	10	0.00	0.00	0.00	0.21	0.21	0.04
											0.31	**1.75**	**0.53**

Gauging Session: 2

Seg-ment	Depth (m)	Dist. to Point (m)	Segment Dist. (m)	Vertical Point	Point Depth (m)	Revs	Time (sec)	Revs/ (sec)	Vel. (m/s)	Av.Vel (m/s)	Mean Vel (m/s)	Area (m2)	Flow (m3/s)
WE1	0.00	0.00		0.00	0.00	0	10	0.00	0.00	0.00			
A	0.30	2.10	2.10	0.20	0.06	34	10	3.40	0.45		0.16	0.32	0.05
				0.80	0.24	12	10	1.20	0.17	0.31			
B	0.30	3.10	1.00	0.20	0.06	39	10	3.90	0.52		0.41	0.30	0.12
				0.80	0.24	37	10	3.70	0.49	0.50			
C	0.28	4.10	1.00	0.20	0.06	11	10	1.10	0.16		0.34	0.29	0.10
				0.80	0.22	15	10	1.50	0.21	0.18			
D	0.44	5.60	1.50	0.20	0.09	40	10	4.00	0.53		0.33	0.54	0.18
				0.80	0.35	31	10	3.10	0.41	0.47			
WE2	0.00	6.60	1.00	0.00	0.00	0	10	0.00	0.00	0.00	0.24	0.22	0.05
											0.30	**1.67**	**0.50**

Sub-catchment: C3

Gauging Session: 1

Seg-ment	Depth (m)	Dist. to Point (m)	Segment Dist. (m)	Vertical Point	Point Depth (m)	Revs	Time (sec)	Revs/ (sec)	Vel. (m/s)	Av.Vel (m/s)	Mean Vel (m/s)	Area (m2)	Flow (m3/s)
WE1	0.00	0.00		0.00	0.00	0	10	0.00	0.00	0.00			
A	0.96	2.00	2.00	0.20	0.19	2	10	0.20	0.04		0.02	0.96	0.02
				0.80	0.77	2	10	0.20	0.04	0.04			
B	1.08	3.50	1.50	0.20	0.22	14	10	1.40	0.20		0.09	1.53	0.13
				0.80	0.86	4	10	0.40	0.07	0.13			
C	0.62	5.00	1.50	0.20	0.12	14	10	1.40	0.20		0.16	1.28	0.20
				0.80	0.50	11	10	1.10	0.16	0.18			
D	0.42	6.50	1.50	0.20	0.08	20	10	2.00	0.27		0.22	0.78	0.17
				0.80	0.34	18	10	1.80	0.25	0.26			
WE2	0.00	8.50	2.00	0.00	0.00	0	10	0.00	0.00	0.00	0.13	0.42	0.05
											0.12	**4.97**	**0.58**

Gauging Session: 2

Seg-ment	Depth (m)	Dist. to Point (m)	Segment Dist. (m)	Vertical Point	Point Depth (m)	Revs	Time (sec)	Revs/ (sec)	Vel. (m/s)	Av.Vel (m/s)	Mean Vel (m/s)	Area (m2)	Flow (m3/s)
WE1	0.00	0.00		0.00	0.00	0	10	0.00	0.00	0.00			
A	0.98	2.00	2.00	0.20	0.20	1	10	0.10	0.03		0.01	0.98	0.01
				0.80	0.78	1	10	0.10	0.03	0.03			
B	1.08	3.50	1.50	0.20	0.22	15	10	1.50	0.21		0.10	1.55	0.15
				0.80	0.86	8	10	0.80	0.12	0.17			
C	0.62	5.00	1.50	0.20	0.12	14	10	1.40	0.20		0.17	1.28	0.21
				0.80	0.50	9	10	0.90	0.13	0.17			
D	0.42	6.50	1.50	0.20	0.08	23	10	2.30	0.31		0.23	0.78	0.18
				0.80	0.34	21	10	2.10	0.29	0.30			
WE2	0.00	8.50	2.00	0.00	0.00	0	10	0.00	0.00	0.00	0.15	0.42	0.06
											0.12	**5.00**	**0.62**

Date: 15.03.95

Sub-catchment: C1

Gauging Session: 1

Seg-ment	Depth (m)	Dist. to Point (m)	Segment Dist. (m)	Vertical Point	Point Depth (m)	Revs	Time (sec)	Revs/ (sec)	Vel. (m/s)	Av.Vel (m/s)	Mean Vel (m/s)	Area (m2)	Flow (m3/s)
WE1	0.00	0.00		0.00	0.00	0	10	0.00	0.00	0.00			
A	0.30	0.70	0.70	0.20	0.06	60	10	6.00	0.78		0.31	0.11	0.03
				0.80	0.24	33	10	3.30	0.44	0.61			
B	0.34	1.20	0.50	0.20	0.07	28	10	2.80	0.38		0.45	0.16	0.07
				0.80	0.27	13	10	1.30	0.18	0.28			
C	0.34	2.00	0.80	0.20	0.07	48	10	4.80	0.63		0.34	0.27	0.09
				0.80	0.27	13	10	1.30	0.18	0.41			
D	0.00	3.20	1.20	0.20	0.00	0	10	0.00	0.02		0.21	0.20	0.04
				0.80	0.00	0	10	0.00	0.02	0.02			
WE2	0.00	3.20	0.00	0.00	0.00	0	10	0.00	0.00	0.00	0.01	0.00	0.00
											0.32	**0.74**	**0.24**

Gauging Session: 2

Seg-ment	Depth (m)	Dist. to Point (m)	Segment Dist. (m)	Vertical Point	Point Depth (m)	Revs	Time (sec)	Revs/ (sec)	Vel. (m/s)	Av.Vel (m/s)	Mean Vel (m/s)	Area (m2)	Flow (m3/s)
WE1	0.00	0.00		0.00	0.00	0	10	0.00	0.00	0.00			
A	0.30	0.70	0.70	0.20	0.06	60	10	6.00	0.78		0.31	0.11	0.03
				0.80	0.24	33	10	3.30	0.44	0.61			
B	0.34	1.20	0.50	0.20	0.07	28	10	2.80	0.38		0.45	0.16	0.07
				0.80	0.27	13	10	1.30	0.18	0.28			
C	0.34	2.00	0.80	0.20	0.07	48	10	4.80	0.63		0.34	0.27	0.09
				0.80	0.27	13	10	1.30	0.18	0.41			
D	0.00	3.20	1.20	0.20	0.00	0	10	0.00	0.02		0.21	0.20	0.04
				0.80	0.00	0	10	0.00	0.02	0.02			
WE2	0.00	3.20	0.00	0.00	0.00	0	10	0.00	0.00	0.00	0.01	0.00	0.00
											0.32	**0.74**	**0.24**

Sub-catchment: C2

Gauging Session: 1

Seg-ment	Depth (m)	Dist. to Point (m)	Segment Dist. (m)	Vertical Point	Point Depth (m)	Revs	Time (sec)	Revs/ (sec)	Vel. (m/s)	Av.Vel (m/s)	Mean Vel (m/s)	Area (m2)	Flow (m3/s)
WE1	0.00	0.00		0.00	0.00	0	10	0.00	0.00	0.00			
A	0.32	2.10	2.10	0.20	0.06	32	10	3.20	0.43		0.21	0.34	0.07
				0.80	0.26	30	10	3.00	0.40	0.41			
B	0.30	3.10	1.00	0.20	0.06	28	10	2.80	0.38		0.41	0.31	0.13
				0.80	0.24	32	10	3.20	0.43	0.40			
C	0.40	4.10	1.00	0.20	0.08	10	10	1.00	0.15		0.28	0.35	0.10
				0.80	0.32	13	10	1.30	0.18	0.17			
D	0.46	5.60	1.50	0.20	0.09	39	10	3.90	0.52		0.34	0.65	0.22
				0.80	0.37	40	10	4.00	0.53	0.52			
WE2	0.00	6.60	1.00	0.00	0.00	0	10	0.00	0.00	0.00	0.26	0.23	0.06
											0.31	**1.87**	**0.58**

Gauging Session: 2

Seg-ment	Depth (m)	Dist. to Point (m)	Segment Dist. (m)	Vertical Point	Point Depth (m)	Revs	Time (sec)	Revs/ (sec)	Vel. (m/s)	Av.Vel (m/s)	Mean Vel (m/s)	Area (m2)	Flow (m3/s)
WE1	0.00	0.00		0.00	0.00	0	10	0.00	0.00	0.00			
A	0.30	2.10	2.10	0.20	0.06	39	10	3.90	0.52		0.17	0.32	0.05
				0.80	0.24	10	10	1.00	0.15	0.33			
B	0.30	3.10	1.00	0.20	0.06	44	10	4.40	0.58		0.44	0.30	0.13
				0.80	0.24	40	10	4.00	0.53	0.55			
C	0.42	4.10	1.00	0.20	0.08	18	10	1.80	0.25		0.41	0.36	0.15
				0.80	0.34	20	10	2.00	0.27	0.26			
D	0.46	5.60	1.50	0.20	0.09	21	10	2.10	0.29		0.31	0.66	0.20
				0.80	0.37	32	10	3.20	0.43	0.36			
WE2	0.00	6.60	1.00	0.00	0.00	37	10	0.00	0.00	0.00	0.18	0.23	0.04
											0.31	**1.87**	**0.57**

Sub-catchment: C3

Gauginging Session: 1

Seg-ment	Depth (m)	Dist. to Point (m)	Segment Dist. (m)	Vertical Point	Point Depth (m)	Revs	Time (sec)	Revs/ (sec)	Vel. (m/s)	Av.Vel (m/s)	Mean Vel (m/s)	Area (m2)	Flow (m3/s)
WE1	0.00	0.00		0.00	0.00	0	10	0.00	0.00	0.00			
A	0.98	2.00	2.00	0.20	0.20	3	10	0.30	0.05		0.02	0.98	0.02
				0.80	0.78	1	10	0.10	0.03	0.04			
B	1.09	3.50	1.50	0.20	0.22	18	10	1.80	0.25		0.12	1.55	0.18
				0.80	0.87	10	10	1.00	0.15	0.20			
C	0.64	5.00	1.50	0.20	0.13	15	10	1.50	0.21		0.19	1.30	0.25
				0.80	0.51	11	10	1.10	0.16	0.18			
D	0.48	6.50	1.50	0.20	0.10	25	10	2.50	0.34		0.25	0.84	0.21
				0.80	0.38	22	10	2.20	0.30	0.32			
WE2	0.00	8.50	2.00	0.00	0.00	0	10	0.00	0.00	0.00	0.16	0.48	0.08
											0.41	**5.15**	**2.11**

Gauging Session: 2

Seg-ment	Depth (m)	Dist. to Point (m)	Segment Dist. (m)	Vertical Point	Point Depth (m)	Revs	Time (sec)	Revs/ (sec)	Vel. (m/s)	Av.Vel (m/s)	Mean Vel (m/s)	Area (m2)	Flow (m3/s)
WE1	0.00	0.00		0.00	0.00	0	10	0.00	0.00	0.00			
A	0.98	2.00	2.00	0.20	0.20	1	10	0.10	0.03		0.01	0.98	0.01
				0.80	0.78	0	10	0.00	0.01	0.02			
B	1.08	3.50	1.50	0.20	0.22	14	10	1.40	0.20		0.10	1.55	0.15
				0.80	0.86	10	10	1.00	0.15	0.17			
C	0.62	5.00	1.50	0.20	0.12	14	10	1.40	0.20		0.18	1.28	0.22
				0.80	0.50	11	10	1.10	0.16	0.18			
D	0.46	6.50	1.50	0.20	0.09	19	10	1.90	0.26		0.22	0.81	0.18
				0.80	0.37	20	10	2.00	0.27	0.27			
WE2	0.00	8.50	2.00	0.00	0.00	0	10	0.00	0.00	0.00	0.13	0.46	0.06
											0.12	**5.08**	**0.62**

APPENDIX VIII:

Flood-flow Velocity and Channel Cross-section Area Field Measurements

VIII.1 Flow Velocity Measurements

VIII.2 Cross-section Area Measurements

APPENDIX VIII.1:

Flow Velocity Measurements

VALDATA

OK MANI FLOODFLOW

VELOCITY MEASUREMENTS

Date: 02.03.95

Time After Storm: 0.5 hours

Gauging Point	Length of Reach (m)	Time Float Travel Over Reach (sec)	Mid-surface Flow Vel. (m/s)	Mean Surf Vel (m/s)	Multiplication Factor*	Average Flow Vel. (m/s)
V	5.00	6.00	0.83			
	5.00	9.00	0.56			
	5.00	8.00	0.63	0.67	0.70	0.47
W	20.00	10.00	2.00			
	20.00	11.00	1.82			
	20.00	10.00	2.00	1.94	0.70	1.36
X	12.00	6.00	2.00			
	12.00	9.00	1.33			
	12.00	8.00	1.50	1.61	0.70	1.13

Time After Storm: 1.0 hours

Gauging Point	Length of Reach (m)	Time Float Travel Over Reach (sec)	Mid-surface Flow Vel. (m/s)	Mean Surf Vel (m/s)	Multiplication Factor*	Average Flow Vel. (m/s)
V	5.00	5.00	1.00			
	5.00	6.00	0.83			
	5.00	4.00	1.25	1.03	0.70	0.72
W	20.00	8.00	2.50			
	20.00	8.00	2.50			
	20.00	7.00	2.86	2.62	0.70	1.83
X	12.00	7.00	1.71			
	12.00	8.00	1.50			
	12.00	6.00	2.00	1.74	0.70	1.22

Time After Storm: 3 hours

Gauging Point	Length of Reach (m)	Time Float Travel Over Reach (sec)	Mid-surface Flow Vel. (m/s)	Mean Surf Vel (m/s)	Multiplication Factor*	Average Flow Vel. (m/s)
V	5.00	7.00	0.71			
	5.00	8.00	0.63			
	5.00	6.00	0.83	0.72	0.70	0.50
W	20.00	9.00	2.22			
	20.00	9.00	2.22			
	20.00	10.00	2.00	2.15	0.70	1.51
X	12.00	9.00	1.33			
	12.00	8.00	1.50			
	12.00	8.00	1.50	1.44	0.70	1.01

Time After Storm: 5 hours

Gauging Point	Length of Reach (m)	Time Float Travel Over Reach (sec)	Mid-surface Flow Vel. (m/s)	Mean Surf Vel (m/s)	Multiplication Factor*	Average Flow Vel. (m/s)
V	5.00	9.00	0.56			
	5.00	10.00	0.50			
	5.00	9.00	0.56	0.54	0.70	0.38
W	20.00	9.00	2.22			
	20.00	8.00	2.50			
	20.00	8.00	2.50	2.41	0.70	1.69
X	12.00	11.00	1.09			
	12.00	12.00	1.00			
	12.00	12.00	1.00	1.03	0.70	0.72

VALDATA

Date: 03.03.95

Time After Storm: 0.5 hours

Gauging Point	Length of Reach (m)	Time Float Travel Over Reach (sec)	Mid-surface Flow Vel. (m/s)	Mean Surf Vel (m/s)	Multiplication Factor*	Average Flow Vel. (m/s)
V	5.00	8.00	0.63			
	5.00	8.00	0.63			
	5.00	9.00	0.56	0.61	0.70	0.43
W	20.00	12.00	1.67			
	20.00	14.00	1.43			
	20.00	13.00	1.54	1.55	0.70	1.09
X	12.00	18.00	0.67			
	12.00	18.00	0.67			
	12.00	19.00	0.63	0.66	0.70	0.46

Time After Storm: 1.0 hours

Gauging Point	Length of Reach (m)	Time Float Travel Over Reach (sec)	Mid-surface Flow Vel. (m/s)	Mean Surf Vel (m/s)	Multiplication Factor*	Average Flow Vel. (m/s)
V	5.00	7.00	0.71			
	5.00	8.00	0.63			
	5.00	6.00	0.83	0.72	0.70	0.50
W	20.00	11.00	1.82			
	20.00	12.00	1.67			
	20.00	11.00	1.82	1.77	0.70	1.24
X	12.00	16.00	0.75			
	12.00	16.00	0.75			
	12.00	15.00	0.80	0.77	0.70	0.54

Time After Storm: 1.5 hours

Gauging Point	Length of Reach (m)	Time Float Travel Over Reach (sec)	Mid-surface Flow Vel. (m/s)	Mean Surf Vel (m/s)	Multiplication Factor*	Average Flow Vel. (m/s)
V	5.00	9.00	0.56			
	5.00	8.00	0.63			
	5.00	8.00	0.63	0.61	0.70	0.43
W	20.00	11.00	1.82			
	20.00	12.00	1.67			
	20.00	11.00	1.82	1.77	0.70	1.24
X	12.00	17.00	0.71			
	12.00	16.00	0.75			
	12.00	18.00	0.67	0.71	0.70	0.50

Time After Storm: 2.0 hours

Gauging Point	Length of Reach (m)	Time Float Travel Over Reach (sec)	Mid-surface Flow Vel. (m/s)	Mean Surf Vel (m/s)	Multiplication Factor*	Average Flow Vel. (m/s)
V	5.00	9.00	0.56			
	5.00	10.00	0.50			
	5.00	10.00	0.50	0.52	0.70	0.36
W	20.00	13.00	1.54			
	20.00	14.00	1.43			
	20.00	12.00	1.67	1.55	0.70	1.09
X	12.00	19.00	0.63			
	12.00	18.00	0.67			
	12.00	18.00	0.67	0.66	0.70	0.46

VALDATA

Date: 04.03.95

Time After Storm: 0.5 hours

Gauging Point	Length of Reach (m)	Time Float Travel Over Reach (sec)	Mid-surface Flow Vel. (m/s)	Mean Surf Vel (m/s)	Multiplication Factor*	Average Flow Vel. (m/s)
V	5.00	5.00	1.00			
	5.00	6.00	0.83			
	5.00	5.00	1.00	0.94	0.70	0.66
W	20.00	9.00	2.22			
	20.00	9.00	2.22			
	20.00	8.00	2.50	2.31	0.70	1.62
X	12.00	11.00	1.09			
	12.00	10.00	1.20			
	12.00	11.00	1.09	1.13	0.70	0.79

Time After Storm: 1.0 hours

Gauging Point	Length of Reach (m)	Time Float Travel Over Reach (sec)	Mid-surface Flow Vel. (m/s)	Mean Surf Vel (m/s)	Multiplication Factor*	Average Flow Vel. (m/s)
V	5.00	4.00	1.24			
	5.00	4.00	1.24			
	5.00	5.00	1.00	1.16	0.70	0.81
W	20.00	7.00	2.86			
	20.00	8.00	2.50			
	20.00	8.00	2.50	2.62	0.70	1.83
X	12.00	9.00	1.33			
	12.00	8.00	1.50			
	12.00	9.00	1.33	1.39	0.70	0.97

Time After Storm: 2.0 hours

Gauging Point	Length of Reach (m)	Time Float Travel Over Reach (sec)	Mid-surface Flow Vel. (m/s)	Mean Surf Vel (m/s)	Multiplication Factor*	Average Flow Vel. (m/s)
V	5.00	4.00	1.25			
	5.00	5.00	1.00			
	5.00	4.00	1.25	1.17	0.70	0.82
W	20.00	7.00	2.86			
	20.00	9.00	2.22			
	20.00	8.00	2.50	2.53	0.70	1.77
X	12.00	10.00	1.20			
	12.00	8.00	1.50			
	12.00	9.00	1.33	1.34	0.70	0.94

Time After Storm: 3.0 hours

Gauging Point	Length of Reach (m)	Time Float Travel Over Reach (sec)	Mid-surface Flow Vel. (m/s)	Mean Surf Vel (m/s)	Multiplication Factor*	Average Flow Vel. (m/s)
V	5.00	7.00	0.71			
	5.00	6.00	0.71			
	5.00	8.00	0.63	0.68	0.70	0.48
W	20.00	12.00	1.67			
	20.00	12.00	1.67			
	20.00	11.00	1.32	1.72	0.70	1.20
X	12.00	15.00	0.80			
	12.00	16.00	0.75			
	12.00	17.00	0.71	0.75	0.70	0.53

VALDATA

Date: 15.03.95

Time After Storm: 0.5 hours

Gauging Point	Length of Reach (m)	Time Float Travel Over Reach (sec)	Mid-surface Flow Vel. (m/s)	Mean Surf Vel (m/s)	Multiplication Factor*	Average Flow Vel. (m/s)
V	5.00	4.00	1.25			
	5.00	5.00	1.00			
	5.00	5.00	1.00	1.08	0.70	0.76
W	20.00	9.00	2.22			
	20.00	10.00	2.00			
	20.00	9.00	2.22	2.15	0.70	1.51
X	12.00	13.00	0.92			
	12.00	12.00	1.00			
	12.00	14.00	0.80	0.93	0.70	0.65

Time After Storm: 1.0 hours

Gauging Point	Length of Reach (m)	Time Float Travel Over Reach (sec)	Mid-surface Flow Vel. (m/s)	Mean Surf Vel (m/s)	Multiplication Factor*	Average Flow Vel. (m/s)
V	5.00	4.00	1.25			
	5.00	3.00	1.67			
	5.00	4.00	1.25	1.39	0.70	0.97
W	20.00	8.00	2.50			
	20.00	9.00	2.22			
	20.00	9.00	2.22	2.31	0.70	1.62
X	12.00	11.00	1.09			
	12.00	10.00	1.20			
	12.00	11.00	1.09	1.13	0.70	0.79

Time After Storm: 1.5 hours

Gauging Point	Length of Reach (m)	Time Float Travel Over Reach (sec)	Mid-surface Flow Vel. (m/s)	Mean Surf Vel (m/s)	Multiplication Factor*	Average Flow Vel. (m/s)
V	5.00	3.00	1.67			
	5.00	4.00	1.25			
	5.00	3.00	1.67	1.53	0.70	1.07
W	20.00	8.00	2.50			
	20.00	9.00	2.22			
	20.00	8.00	2.50	2.41	0.70	1.69
X	12.00	12.00	1.00			
	12.00	11.00	1.09			
	12.00	13.00	0.92	1.00	0.70	0.70

Time After Storm: 2.0 hours

Gauging Point	Length of Reach (m)	Time Float Travel Over Reach (sec)	Mid-surface Flow Vel. (m/s)	Mean Surf Vel (m/s)	Multiplication Factor*	Average Flow Vel. (m/s)
V	5.00	4.00	1.25			
	5.00	5.00	1.00			
	5.00	5.00	1.00	1.08	0.70	0.76
W	20.00	12.00	1.67			
	20.00	13.00	1.54			
	20.00	13.00	1.54	1.58	0.70	1.11
X	12.00	15.00	0.80			
	12.00	15.00	0.80			
	12.00	16.00	0.75	0.78	0.70	0.55

In most cases, the average stream flow velocity occurs at 0.60 of the depth. Therefore, float velocity is multiplied by 0.70 to bring it close to the average flow velocity

APPENDIX VIII.2:

Cross-section Area Measurements

OK MANI STREAM CROSS-SECTION AREA DURING FLOODFLOWS

Date: 02.03.95

Time After Storm: 0.5 hours

Gauging Point	Channel Width (m)	Segment	Segment Width (m)	Segment Depth (m)	Segment Area (m2)	Cross-Section Area (m2)
V		A	1.50	0.18	0.12	
	5.00	B	1.00	0.55	1.00	
		C	1.00	0.42	0.43	
		D	1.50	0.15	0.11	1.66
W		A	1.50	0.14	0.21	
	6.00	B	1.50	0.75	1.13	
		C	1.50	0.80	1.20	
		D	1.50	0.65	0.98	3.52
X		A	3.00	0.75	1.13	
	12.00	B	3.00	0.76	2.28	
		C	3.00	0.62	1.85	
		D	3.00	0.42	0.63	5.89

Time After Storm: 1.5 hours

Gauging Point	Channel Width (m)	Segment	Segment Width (m)	Segment Depth (m)	Segment Area (m2)	Cross-Section Area (m2)
V		A	2.00	0.20	0.20	
	7.00	B	2.00	0.58	1.16	
		C	2.00	0.52	1.04	
		D	1.00	0.17	0.09	2.49
W		A	2.00	0.24	0.24	
	9.00	B	2.00	0.97	1.93	
		C	2.00	0.87	1.74	
		D	1.00	0.79	0.79	4.68
X		A	4.00	0.82	1.64	
	14.00	B	4.00	0.82	3.29	
		C	4.00	0.68	2.72	
		D	2.00	0.62	0.63	8.26

Time After Storm: 3 hours

Gauging Point	Channel Width (m)	Segment	Segment Width (m)	Segment Depth (m)	Segment Area (m2)	Cross-Section Area (m2)
V		A	2.00	0.26	0.26	
	10.00	B	3.00	0.68	2.04	
		C	3.00	0.50	1.50	
		D	2.00	0.16	0.16	3.96
W		A	3.00	0.15	0.23	
	13.00	B	3.00	0.92	2.76	
		C	3.00	0.82	2.46	
		D	4.00	0.70	1.40	6.85
X		A	4.00	0.92	1.84	
	15.00	B	4.00	1.20	4.80	
		C	4.00	1.60	6.00	
		D	3.00	1.12	1.68	14.32

Time After Storm: 5 hours

Gauging Point	Channel Width (m)	Segment	Segment Width (m)	Segment Depth (m)	Segment Area (m2)	Cross-Section Area (m2)
V		A	1.50	0.15	0.11	
	6.00	B	1.50	0.24	0.36	
		C	1.50	0.28	0.42	
		D	1.50	0.12	0.09	0.98
W		A	2.00	0.16	0.16	
	9.00	B	3.00	0.72	2.16	
		C	3.00	0.58	1.74	
		D	1.00	0.14	0.07	4.13
X		A	3.00	0.46	0.69	
	12.00	B	3.00	0.92	2.76	
		C	3.00	1.20	3.60	
		D	3.00	0.74	1.11	8.16

AREADATA

Date: 03.03.95

Time After Storm: 0.5 hours

Gauging Point	Channel Width (m)	Segment	Segment Width (m)	Segment Depth (m)	Segment Area (m2)	Cross-Section Area (m2)
V		A	1.50	0.15	0.11	
	6.00	B	1.50	0.24	0.36	
		C	1.50	0.28	0.42	
		D	1.50	0.12	0.09	0.98
W		A	2.00	0.16	0.16	
	9.00	B	3.00	0.14	0.43	
		C	3.00	0.10	0.30	
		D	2.00	0.14	0.14	1.03
X		A	3.00	0.32	0.48	
	12.00	B	3.00	0.29	0.88	
		C	3.00	0.33	1.00	
		D	3.00	0.54	0.81	3.17

Time After Storm: 1 hour

Gauging Point	Channel Width (m)	Segment	Segment Width (m)	Segment Depth (m)	Segment Area (m2)	Cross-Section Area (m2)
V		A	2.00	0.16	0.08	
	7.00	B	2.00	0.27	0.54	
		C	2.00	0.29	0.58	
		D	1.00	0.15	0.08	1.08
W		A	3.00	0.17	0.26	
	11.00	B	3.00	0.17	0.51	
		C	3.00	0.23	0.67	
		D	2.00	0.19	0.19	1.63
X		A	4.00	0.36	0.72	
	15.00	B	4.00	0.27	1.10	
		C	4.00	0.44	1.76	
		D	3.00	0.50	0.75	4.33

Time After Storm: 1.5 hours

Gauging Point	Channel Width (m)	Segment	Segment Width (m)	Segment Depth (m)	Segment Area (m2)	Cross-Section Area (m2)
V		A	2.00	0.16	0.16	
	7.00	B	2.00	0.28	0.56	
		C	2.00	0.29	0.58	
		D	1.00	0.17	0.09	1.39
W		A	3.00	0.12	0.18	
	11.00	B	3.00	0.20	0.60	
		C	3.00	0.27	0.81	
		D	2.00	0.12	0.12	1.71
X		A	4.00	0.46	0.92	
	15.00	B	4.00	0.38	1.50	
		C	4.00	0.62	2.46	
		D	3.00	0.72	1.08	5.96

Time After Storm: 2 hours

Gauging Point	Channel Width (m)	Segment	Segment Width (m)	Segment Depth (m)	Segment Area (m2)	Cross-Section Area (m2)
V		A	1.00	0.09	0.05	
	6.00	B	2.00	0.26	0.52	
		C	2.00	0.17	0.35	
		D	1.00	0.90	0.05	0.97
W		A	2.00	0.09	0.18	
	8.00	B	2.00	0.25	0.50	
		C	2.00	0.35	0.70	
		D	2.00	0.32	0.32	1.69
X		A	2.00	0.29	0.29	
	13.00	B	4.00	0.29	1.16	
		C	4.00	0.55	2.20	
		D	3.00	0.42	0.63	4.26

AREADATA

Date: 04.03.95

Time After Storm: 0.5 hours

Gauging Point	Channel Width (m)	Segment	Segment Width (m)	Segment Depth (m)	Segment Area (m2)	Cross-Section Area (m2)
V		A	1.00	0.18	0.09	
	6.00	B	2.00	0.42	0.84	
		C	2.00	0.46	0.92	
		D	1.00	0.36	0.18	2.03
W		A	2.00	0.46	0.46	
	8.00	B	2.00	0.58	1.16	
		C	2.00	1.05	2.10	
		D	2.00	0.66	0.66	4.38
X		A	2.00	0.68	0.68	
	12.00	B	3.00	1.18	3.53	
		C	3.00	1.73	5.20	
		D	3.00	1.40	2.10	11.51

Time After Storm: 1 hour

Gauging Point	Channel Width (m)	Segment	Segment Width (m)	Segment Depth (m)	Segment Area (m2)	Cross-Section Area (m2)
V		A	3.00	0.24	0.36	
	12.00	B	3.00	0.40	1.21	
		C	3.00	0.99	2.98	
		D	3.00	0.38	0.57	5.22
W		A	4.00	0.36	0.72	
	15.00	B	4.00	1.24	4.96	
		C	4.00	1.16	4.64	
		D	3.00	0.64	1.23	10.32
X		A	4.00	0.42	0.83	
	17.00	B	5.00	1.65	8.25	
		C	5.00	1.77	8.85	
		D	3.00	2.20	3.30	21.23

Time After Storm: 2 hours

Gauging Point	Channel Width (m)	Segment	Segment Width (m)	Segment Depth (m)	Segment Area (m2)	Cross-Section Area (m2)
V		A	3.00	0.28	0.42	
	12.00	B	3.00	0.68	2.04	
		C	3.00	0.96	2.88	
		D	3.00	0.42	0.63	5.97
W		A	4.00	0.42	0.84	
	15.00	B	4.00	1.25	5.00	
		C	4.00	1.00	4.00	
		D	3.00	0.67	1.01	10.85
X		A	4.00	0.82	1.64	
	18.00	B	5.00	1.90	9.50	
		C	5.00	1.58	7.91	
		D	4.00	1.80	3.60	22.65

Time After Storm: 3 hours

Gauging Point	Channel Width (m)	Segment	Segment Width (m)	Segment Depth (m)	Segment Area (m2)	Cross-Section Area (m2)
V		A	1.50	0.21	0.16	
	9.00	B	3.00	0.56	1.68	
		C	3.00	0.78	2.34	
		D	1.50	0.34	0.26	4.44
W		A	3.00	0.38	0.57	
	12.00	B	3.00	0.96	2.88	
		C	3.00	1.40	4.20	
		D	3.00	0.60	0.90	8.55
X		A	4.00	1.26	2.52	
	15.00	B	4.00	1.50	5.98	
		C	4.00	2.05	8.20	
		D	3.00	2.59	3.88	20.64

Date: 15.03.95

Time After Storm: 0.5 hours

Gauging Point	Channel Width (m)	Segment	Segment Width (m)	Segment Depth (m)	Segment Area (m2)	Cross-Section Area (m2)
V		A	2.00	0.21	0.21	
	8.00	B	2.00	0.64	1.28	
		C	2.00	0.80	1.60	
		D	2.00	0.34	0.34	3.43
W	10.00	A	1.50	0.36	0.27	
		B	3.00	1.40	4.20	
		C	3.00	1.36	4.08	
		D	1.50	0.40	0.30	8.85
X		A	3.00	0.76	1.14	
	14.00	B	4.00	2.60	10.40	
		C	4.00	2.40	9.60	
		D	3.00	1.70	2.55	26.69

Time After Storm: 1 hour

Gauging Point	Channel Width (m)	Segment	Segment Width (m)	Segment Depth (m)	Segment Area (m2)	Cross-Section Area (m2)
V		A	3.00	0.30	0.45	
	12.00	B	3.00	0.70	2.10	
		C	3.00	0.82	2.46	
		D	3.00	0.24	0.36	5.37
W		A	4.00	0.32	0.64	
	15.00	B	4.00	1.37	5.48	
		C	4.00	1.62	6.48	
		D	3.00	0.36	0.54	13.16
X		A	4.00	0.76	1.52	
	16.00	B	4.00	2.90	11.60	
		C	4.00	3.04	12.16	
		D	4.00	1.60	3.20	28.48

Time After Storm: 1.5 hours

Gauging Point	Channel Width (m)	Segment	Segment Width (m)	Segment Depth (m)	Segment Area (m2)	Cross-Section Area (m2)
V		A	2.00	0.20	0.20	
	8.00	B	2.00	0.64	1.12	
		C	2.00	0.72	1.28	
		D	2.00	0.32	1.42	4.02
W		A	3.00	0.34	0.51	
	14.00	B	4.00	0.98	3.92	
		C	4.00	1.54	6.16	
		D	3.00	0.36	0.54	11.13
X		A	4.00	0.72	1.44	
	16.00	B	4.00	2.79	11.15	
		C	4.00	2.68	10.72	
		D	4.00	1.60	3.20	26.49

Time After Storm: 2 hours

Gauging Point	Channel Width (m)	Segment	Segment Width (m)	Segment Depth (m)	Segment Area (m2)	Cross-Section Area (m2)
V		A	1.50	0.14	0.11	
	6.00	B	1.50	0.56	0.84	
		C	1.50	0.48	0.72	
		D	1.50	0.18	0.14	1.81
W		A	2.00	0.25	0.25	
	10.00	B	3.00	0.70	2.10	
		C	3.00	0.88	2.64	
		D	2.00	0.34	0.34	5.33
X		A	3.00	0.76	0.76	
	12.00	B	3.00	2.07	6.21	
		C	3.00	1.87	5.61	
		D	3.00	1.21	1.21	13.89

Note: Segments A and D are triangles, therefore, Area is 1/2(Depth x Width)
Segments B and C are rectangles, therefore, Area is Depth x Width

APPENDIX IX:

Two-way Analysis of Variance for Storm Data and Catchment

Appendix IX: Two-way Analysis of Variance for storm data and Catchment

Analysis of Variance

Factor	Type	Levels	Values			
DATE	fixed	4	1	2	3	4
CATCHMNT	fixed	3	1	2	3	

Analysis of Variance for RUNOFF

Source	DF	SS	MS	F	P
DATE	3	4576.3	1525.4	7.34	0.020
CATCHMNT	2	77.2	38.6	0.19	0.835
Error	6	1246.2	207.7		
Total	11	5899.7			

Two-way Analysis of Variance

Analysis of Variance for RUNOFF

Source	DF	SS	MS
DATE	3	4576	1525
CATCHMNT	2	77	39
Error	6	1246	208
Total	11	5900	

```
                            Individual 95% CI
    DATE          Mean      ---+---------+---------+---------+--------
       1          15.0        (-------*-------)
       2          41.7                   (-------*-------)
       3          14.0      (--------*-------)
       4          60.7                          (-------*--------)
                            ---+---------+---------+---------+--------
                              0.0      25.0      50.0      75.0

                            Individual 95% CI
CATCHMNT          Mean      ---------+---------+---------+---------+--
       1          29.3        (----------------*----------------)
       2          34.5             (----------------*----------------)
       3          34.7             (----------------*----------------)
                            ---------+---------+---------+---------+--
                                   20.0      30.0      40.0      50.0
```