The Institution of Engineers, Australia
Conference on Hydraulics in Civil Engineering
Hobart 28 –30 November 2001

Modelling of Hydraulic Flood Flows using WBNM2001

R. Van Drie
B.E., Grad.I.E.Aust
Director, BALANCE Research&Development, Australia
M.J. Boyd
B.E., M.Eng.Sc., Ph.D., M.I.Aust
Associate Professor, Faculty of Engineering, University of Wollongong, Australia
E.H. Rigby
B.E., M.Eng.Sc., FIEAust
Director, Forbes Rigby Pty. Ltd., Australia

Abstract: WBNM has traditionally been seen as a hydrologic model, calculating flood hydrographs from storm rainfall hyetographs. However, recent versions of the model have quite complex basin routing procedures, together with detailed culvert hydraulics. These, plus the ability to divert different components of the basin outflows to various points on the catchment, mean that WBNM2001 can model quite complex hydraulic characteristics of flood flows on catchments. This is demonstrated by applying the model to two quite different case studies. The first involves an urban drainage situation where the lack of flow paths contribute to ponds forming on roadways and inundating houses. The second case relates to the simulation of multiple flood plain storage areas that interact due to the presence of a 5km long road embankment and multiple outlet structures. The robust basin routing procedures and culvert and weir hydraulic relations built-in to WBNM2001 allow simulation of flooding problems which have a large hydraulic, rather than hydrologic, component. In these case studies, flow distribution to a variety of flowpaths, and flood water levels were able to be successfully modelled.

Keywords: WBNM2001, hydrology, hydraulics, modelling, simulation

1 INTRODUCTION

WBNM is a hydrologic model originally developed by Boyd et al. in 1979. Since then it has had several significant upgrades. The latest WBNM2001 has included many enhancements that provide the model with a great potential to carry out quite complex hydraulic analysis of hydraulic structures such as culverts and weirs. WBNM2001 utilises culvert hydraulics as determined by the US Federal Highway Administration (1965) for inlet and outlet control. It also utilises the weir equation, and now has the ability to model scourable spillways. Full details of the models capabilities can be found in documentation freely downloaded from http://www.uow.edu/eng/research/.

This paper outlines the results of two complex flood studies carried out by BALANCE Research & Development in which there is an interaction between the hydrology and the hydraulics of the catchments. The first study involved urban residential flooding on a small catchment with a piped drainage system plus overflow paths. The second study involved local runoff into a large river, where the presence of a long road embankment produced large scale flood plain storage of the local runoff.

2 MARK STREET FLOOD INVESTIGATION – WOLLONGONG

Mark Street is a residential development in the Wollongong suburb of Figtree. Figtree is situated within the Allans Creek catchment, on the northern most arm, Byarong Creek. Although Mark Street is within the Allans Creek catchment, its catchment is on a minor tributary and its catchment is a mere 16 hectares as compared to the 41km^2 of Allans Creek and 8.6km^2 of Byarong Creek. Mark St is approximately300 m long and has as a grade of 18%. Mark St intersects with Koloona Avenue, a heavily trafficked local collector road. It similarly has steep grades of around 15%. Both streets were constructed more than 25 years ago, when drainage was not considered in any great detail. As a result a local gully was filled and a residential area created. This area is now subject to flooding resulting in the inundation of several dwellings.

Figure 1 – Extent of the Mark St Catchment also Indicating History of Development

2.1 History of Flood Events

2.1.1 August 1998 Event

On the 17th of August 1998 Wollongong was subjected to a significant rainfall event which caused severe and widespread flooding. The critical burst was enveloped in a longer duration storm lasting several days. The peak rainfall occurred between 6:00pm and 8:00pm on Monday the 17th. Total depth of rain in the burst was 99 mm in 2 hours, and in the longer storm was 290 mm in 24 hours. The Mark St Catchment suffered substantial flooding with several dwellings being inundated.

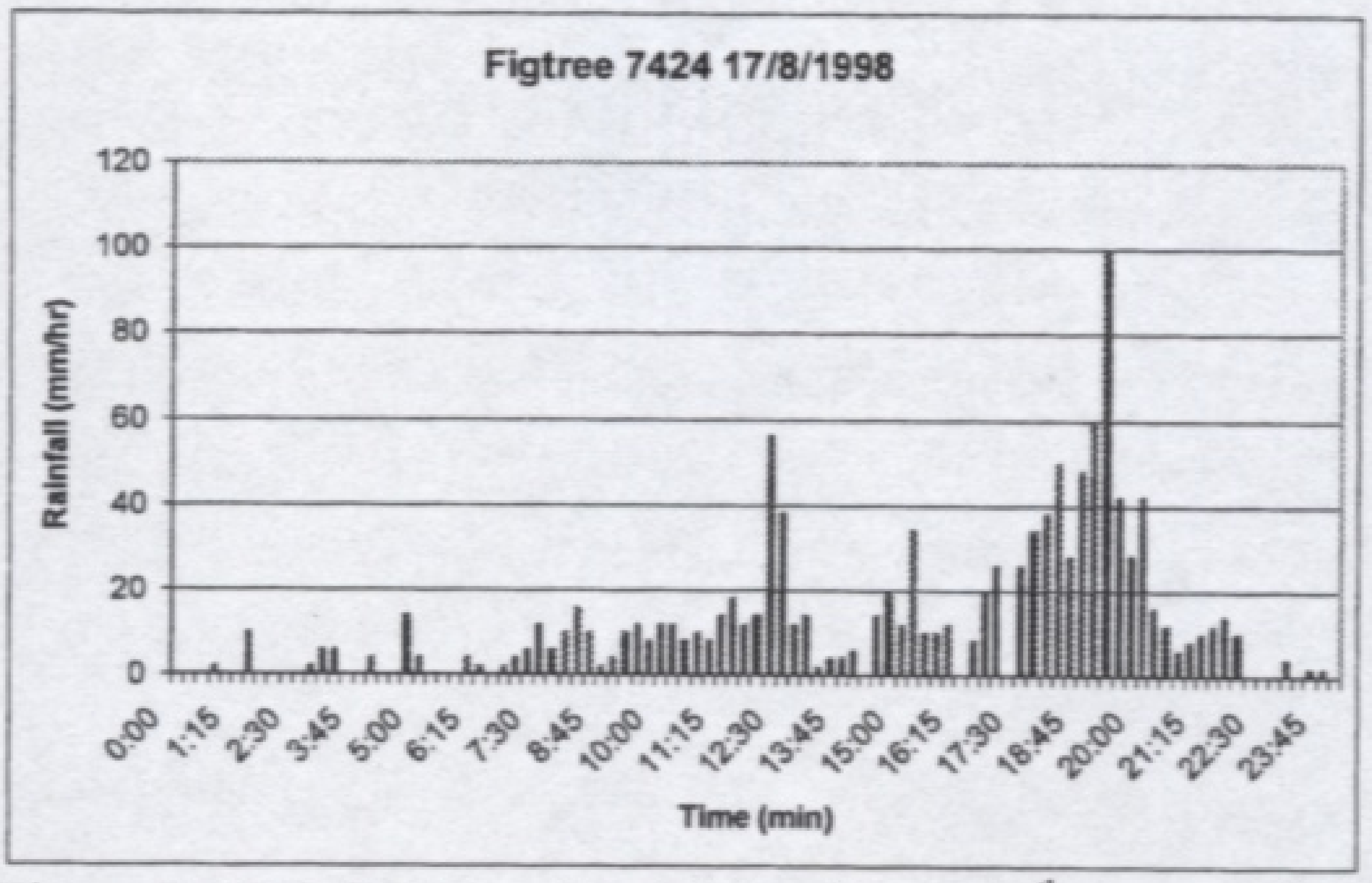

Figure 2 – Rainfall Intensities Figtree Gauge 17th August 1998

2.1.2 October 1999 Event

On the 21st October 1999 Wollongong was again subjected to a significant rainfall event. However, unlike the 1998 storm this was an isolated burst. The thunderstorm cell moved relatively quickly from the south to the north. Allans Creek catchment suffered similar flooding as it did just twelve months before, but from quite different storm conditions.

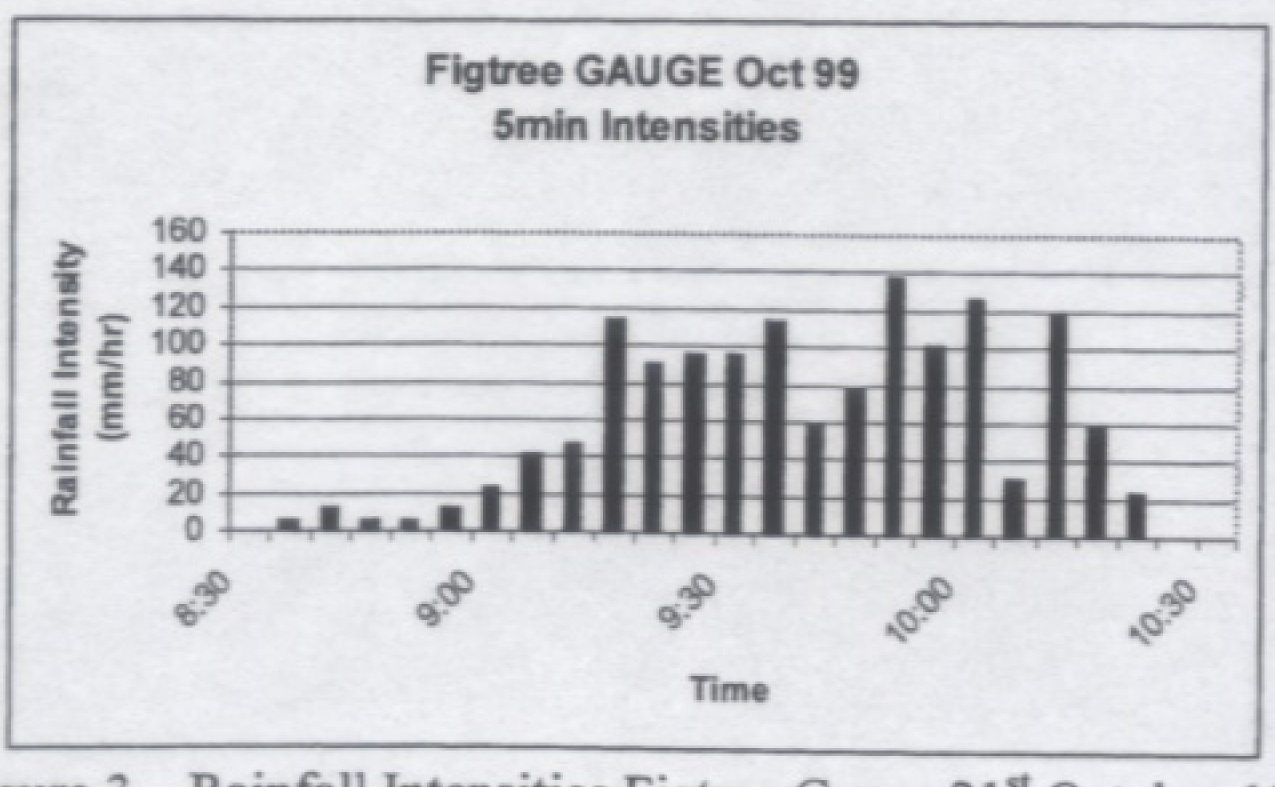

Figure 3 – Rainfall Intensities Figtree Gauge 21st October 1999

Data available for both floods consisted of rainfall hyetographs and daily gauges at 5 locations, as well as observed peak flood levels at various points on the catchment.

2.2 Catchment

2.2.1 Local Mark St Catchment

Mark St catchment is quite small, being only 16 hectares in area. The upper 6.5 ha is quite steep, dropping sharply from RL 130m to RL 50m over a length of 400 m. This section is relatively undeveloped compared to the rest of the catchment but devoid of any substantial vegetation. The next downstream section represents the residential area, primarily Mark St. and Koloona Ave, and falls from RL 50m to RL 30m. The lower most section lies between the last residential lots in Koloona Ave., and Byarong Creek. This section is very flat and forms part of the flood plain of the Byarong Creek System.

2.3 History of Development

2.3.1 Overview of Development History

The original landform of the catchment, prior to European settlement was lush sub tropical vegetation, in steep fertile gullies. Upon European settlement the catchment was utilised as a rural property, therefore the majority of the area was cleared of trees.

The first residential sub division dates back to the early 1960's. Mark St was constructed in 1972. The landform was changed considerably, and drainage was not a primary design consideration.

Since then development has been ongoing with several sub divisions being completed relatively recently. These include developments both upstream and downstream. The upstream development is known as Keira Glenn, while the down stream subdivision is known as Mill Brook School sub division.

2.3.2 Original Development 1960's 1970's

In 1961 R.A. Baxter of RAB Pty Ltd proposed a subdivision off Euroka St. The 1961 flood stalled proposals as Wollongong City Council raised concerns regarding flood issues and the size of the proposed Koloona Ave Bridge. At that time Koloona Ave was not formed. In 1963 & 1964 various plans put forward to cross Byarong Ck on Koloona Ave to gain access to the sub division development site. Koloona Ave was constructed first. Known as Keelogues Estate, this included filling a 1.5m deep Gully that drained the Mark St area. The section of Koloona Ave. immediately downstream of Mark St. exhibits a localised low point in the roadway.

Mark St was constructed in 1972 and included the filling of a gully up to 4m deep to form residential lots and the roadway. Mark St. therefore also has a localised pond that may form when flows exceed the pipe system capacity.

2.3.3 Upstream Development (Kiera Glenn SubDivision)

In 1994 17 new lots were constructed directly upstream of Mark St. In addition, a new road and access way was built to provide access to the new lots. The lots are on quite steep land and the drainage system installed separated flows from further up the catchment from flows derived from the sub division. This was achieved by directing the flows from the sub division to a storage basin and providing a bypass pipe and channel for the upper catchment flows.

2.3.4 Downstream development (Mill Brook School SubDivision)

In 1996 around 40 new lots were constructed downstream of Koloona Avenue. The sub division was constructed on land that had previously been prone to overbank flows from Byarong Creek. In order to develop the site, extensive filling was carried out, effectively removing an overflow path for flows from Koloona Avenue.

2.4 Piped Drainage System

2.4.1 Koloona Ave Drainage

As Koloona Ave was constructed prior to Mark St., the Koloona Ave drainage system was also constructed first. The installed piped drainage system consisted of a single 900mm diameter concrete pipe.

It was placed in a similar alignment to the 1.5m deep gully that existed there prior to development. This drainage system was primarily constructed to convey flows from the western side of Koloona Ave. beneath the roadway and the newly formed lots adjoining the roadway. The outlet was placed just beyond the extent of the new lots discharging into a localised low point that flowed east toward Byarong Creek.

No flow sourced from anywhere upstream continues further down Koloona Ave., all flows are directed east through the residential lots. This includes all surface flows and overflows. Koloona Ave. therefore has a localised pond that may form in the event that flows are in excess of the piped system's capacity. Once pipe capacity or pit inlet capacity is exceeded flows will pond on Koloona Ave. This will continue until the water level reaches a level above that of the overland flow path, at which time overland flow occurs from Koloona Ave. through residential lots.

2.4.2 Mark St Drainage

The construction of Mark St. occurred several years after Koloona Ave. was built. The gully that discharged flows into the Koloona Ave. piped system was extended to beyond the upstream extent of the Mark St. development, beneath the new lots and beneath the roadway of Mark St. A headwall was placed at the upstream end of the pipe. The drainage system consisted of an extension of the 900mm diameter pipe that flowed beneath Koloona Ave. Any flows in excess of the pit inlet capacity or the pipe flow capacity will pond on Mark St. until it reaches a ponded depth exceeding that of the downstream overland flow path. At that point overland flow occurs.

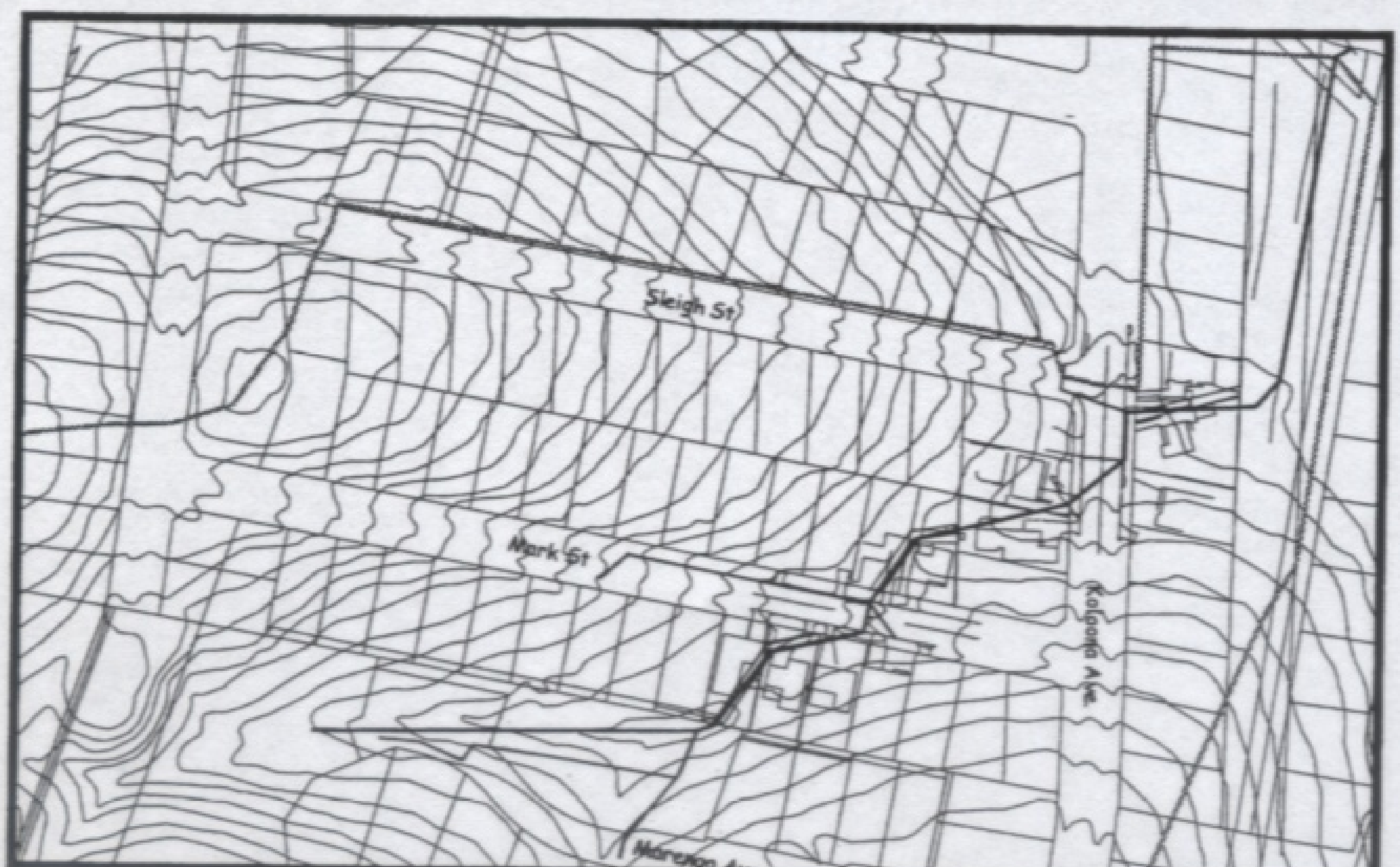

Figure 4 – Mark St. Drainage, 900mm Diam. Pipe Design ~1972

2.4.3 Upstream Development (Kiera Glenn SubDivision)

This development occurred several years after the Mark St subdivision. At that time Wollongong City Council had in place an "On Site Detention" (OSD) Policy. As a result a detention basin was constructed to ensure that the peak flows resulting from the proposed area of development would not be increased above pre-development flows.

In addition, the headwall at the upstream end of the Mark St. development was replaced with a junction pit where the outflow of the storage basin and a new 750mm diameter pipe extension flowed into the original 900mm diameter pipe.

2.4.4 Downstream Development (Mill Brook School SubDivision)

The original flow path from the Koloona Avenue 900mm diameter pipe led from the rear of the properties on Koloona Avenue across a low lying swampy area. In 1996, upon developing the area to the east of Koloona Avenue the original flow path (remnant of original gully) was removed by virtue of the new lots being constructed on fill. Therefore the development included a diversion of the Koloona Ave. pipe by diverting flows through a 90 degree bend to the north via a 900mm diameter pipe laid on a 1% grade.

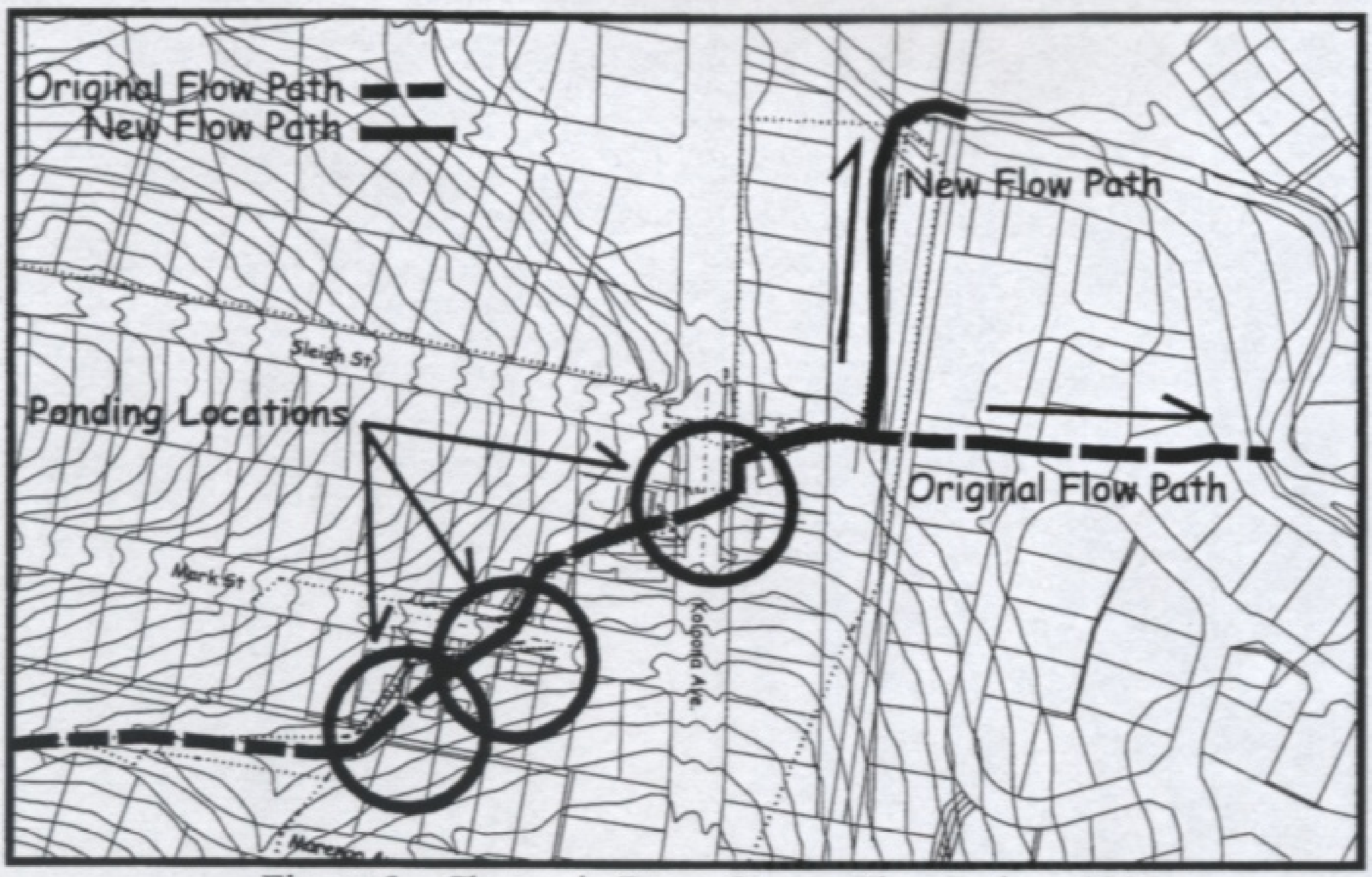

Figure 5 – Change in Down Stream Flow Paths ~ 1996

2.5 Overflow Paths

2.5.1 Flows Through Lots

As a result of these developments, filled gullies became residential lots. Overflow paths were simply forgotten. The only available flow path was between and through dwellings constructed along both Mark St and Koloona Ave. The only formal flow path provided was along the length of the diverted flow of the downstream development. The piped system has a finite capacity which of course is also affected by other factors such as blockage by debris. Flows in excess of the pipe system capacity must flow overland via any flow route available. Again in this instance the only available flow path is through residential lots.

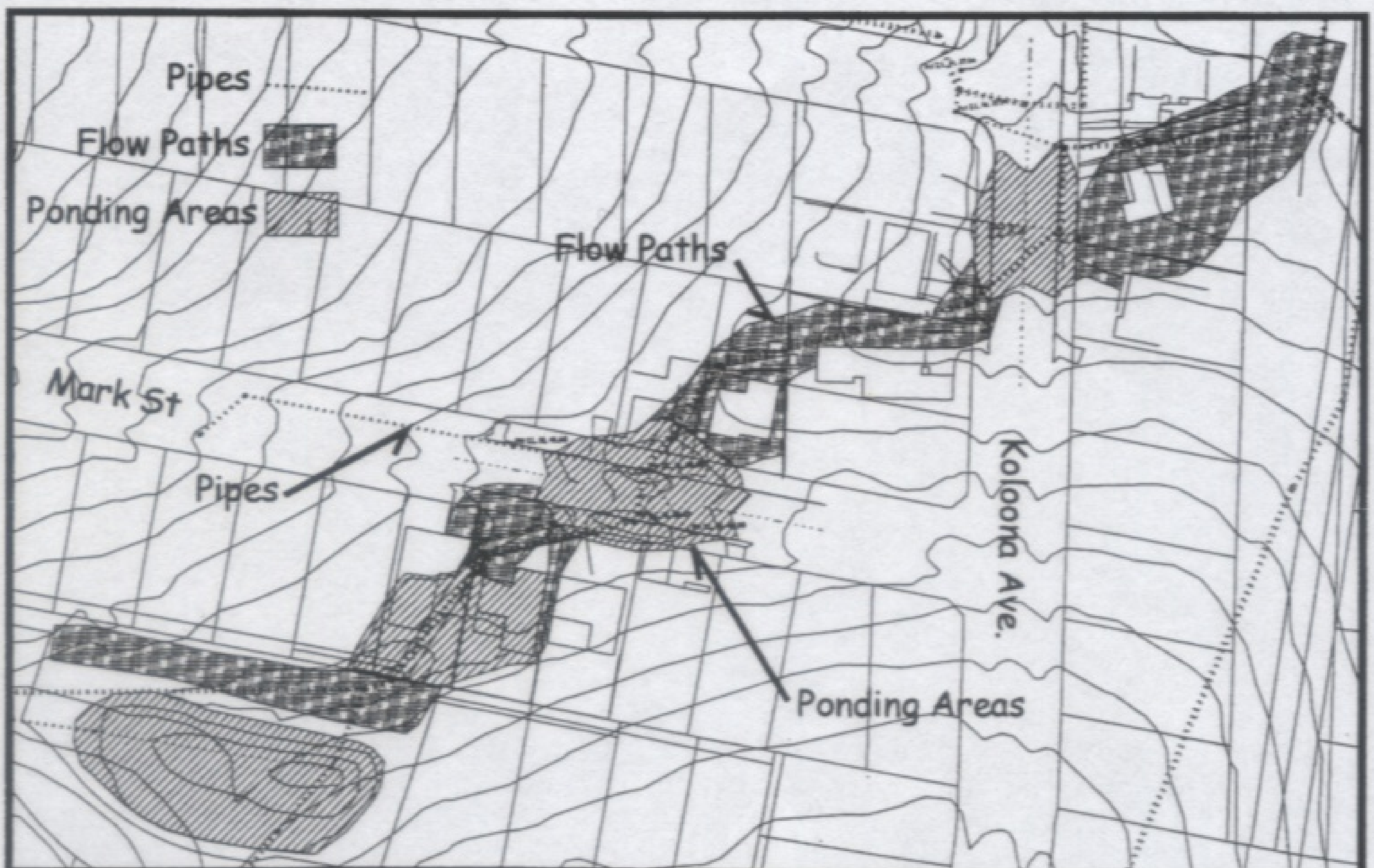

Figure 6 – Overflow Route and Ponding Areas

2.6 WBNM2001 Modelling

2.6.1 Modelling Surface Flows

The catchment hydrology was modelled using WBNM2001(Boyd et al, 2001). WBNM is a robust and regionally validated flood routing model. It has a long history of development and recently has had several additional features added to it. Those of interest to the present study relate to runoff from urban catchments and the handling of hydraulic structures. These include the ability to model multiple outlets of various types, including pipe culverts, box culverts, weirs and scouring embankments. The outlets can be placed at various levels, and tailwater effects can be included. Each of the outlets can be directed to any nominated downstream sub area. This provides great flexibility, enabling the simulation of very complex scenarios, particularly where the hydrology and hydrographs are explicitly linked to the hydraulics. This occurs when the hydraulics of the system have an impact on the shape of the hydrograph, so that storage effects and overtopping effects need to be realistically simulated. For the Mark St catchment, the hydrograph shapes are impacted by each of the ponds as they fill and delay the peak, before a quite drastic increase in peak discharge as the pond crests are overtopped.

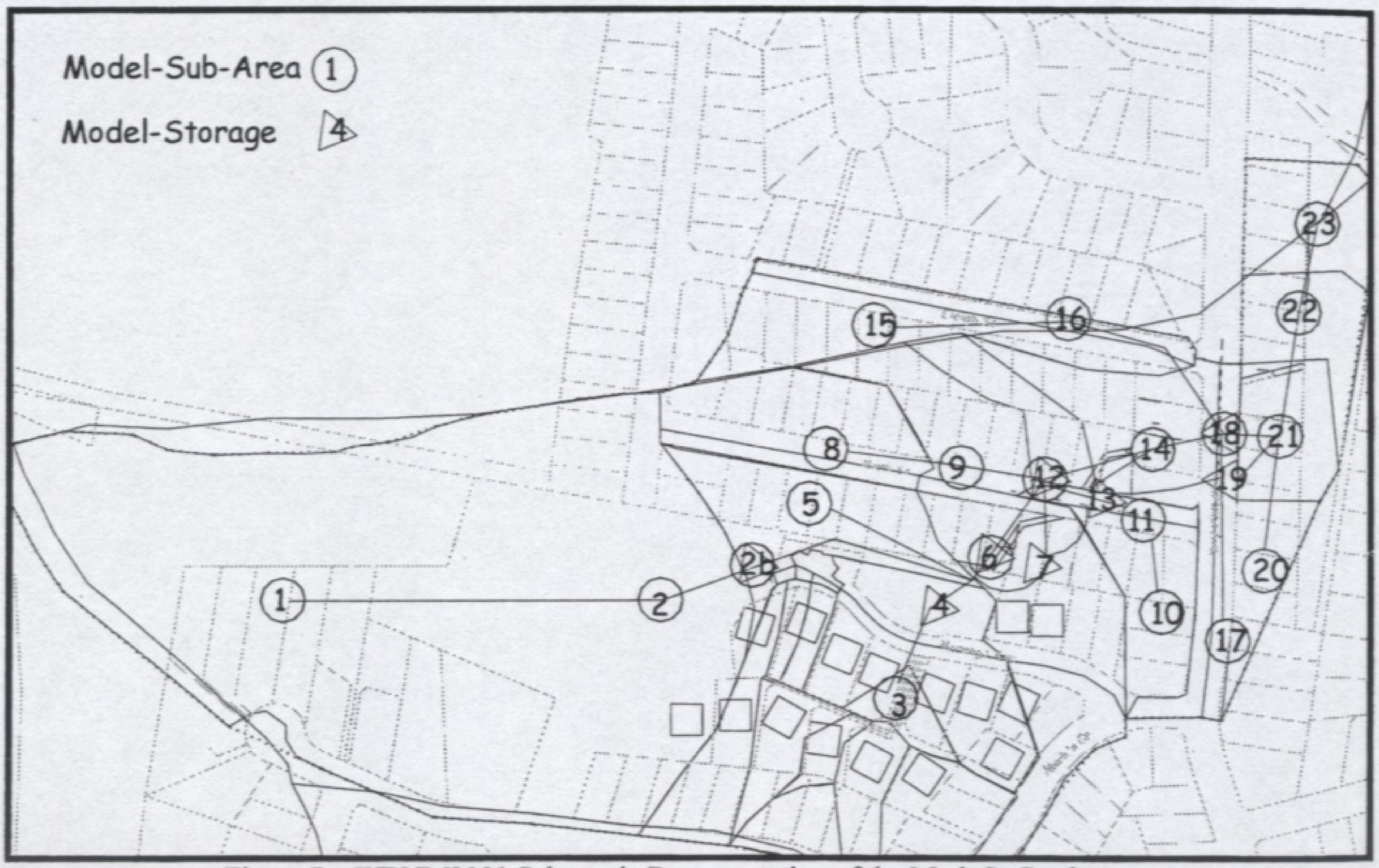

Figure 7 – WBNM2001 Schematic Representation of the Mark St Catchment

2.6.2 Modelling Pipe and Overland Flow

Piped drainage systems are often modelled with software specific to the task, such as DRAINS, MOUSE and SWMM, or using Hydraulic Grade line (HGL) methods. However, WBNM2001 can also be used to model many of the features of these drainage systems in some instances.

The piped drainage system was first analysed using standard HGL methods. However this method had difficulty handling the multiple cascading ponds that operate during significant storm events. In particular, the time variation of water levels and outflows from the ponds, and their interactions, could not be handled.

The analysis was then repeated using WBNM2001 and the observed flood levels were simulated much more accurately. Although no recorded flood levels were measured at the time of the event, photographic evidence is available and these compare very well to modelled results. WBNM2001 allowed modelling of multiple overflow paths, to take into account the splitting of flows at various locations throughout the system. At locations where water did not enter a pipe culvert, either by exceeding its capacity, or by debris blocking, it was diverted to a downstream point, where the water was ponded.

WBNM2001 STRUCTURES SUMMARY TABLE						
SUBAREA	Comment	INFLOW	OUTFLOW	INFLOW	MAX.VOL	MAX.WATER
		PEAK	PEAK	VOLUME	STORED	ELEVATION
		(m3/s)	(m3/s)	(m3 E3)	(m3 E3)	(metres)
SUB_2b	750mm Pipe Headwall	4.622	4.563	9.718	0.108	49.527
SUB_4b	Keira Glenn Basin	2.566	1.446	4.528	1.081	46.1
SUB_6b	900mm Pit Control	6.69	6.616	16.369	0.21	44.529
SUB_7b	Rear Yard Pond Mark St	3.514	3.323	3.234	0.349	44.508
SUB_12b	Mark St Flow Control	7.496	7.184	19.216	0.77	42.653
SUB_13b	Mark St Pond	0.006	0.002	0.031	0.008	42.536
SUB_18b	Koloona St Flow Control	8.099	8.127	21.874	0.196	37.288
SUB_19b	Koloona St Pond	0.002	0.002	0.032	0.001	37.527

Table 1 – WBNM2001 – Storage Structures Summary (Indicating Ponded Water Level)

WBNM was able to successfully route the hydrographs through multiple storages. In total 8 storage structures were modelled. These ranged from ponds at headwall and pits to ponds formed on roadways and rear yards by topography and limited overflow paths. The headwall and pit controls were modelled utilising WBNM2001's pipe hydraulic routines while the ponds on overflow paths were modelled utilising simulated weirs.

WBNM was able to simulate the rise in water level and the subsequent overflow from one pond to the next. The ponding level of each of the three was determined utilising WBNM's robust storage routing procedures with a tailwater controlled pipe outlet (The 900mm diameter pipe) and weirs (overflow path crests).

The WBNM2001 model was made up of two parallel systems, one simulating the pipe hydraulics, and the second simulating the overflow paths. The two systems were linked, allowing flows to pass between them.
The two storm events of 1998 and 1999 were simulated.

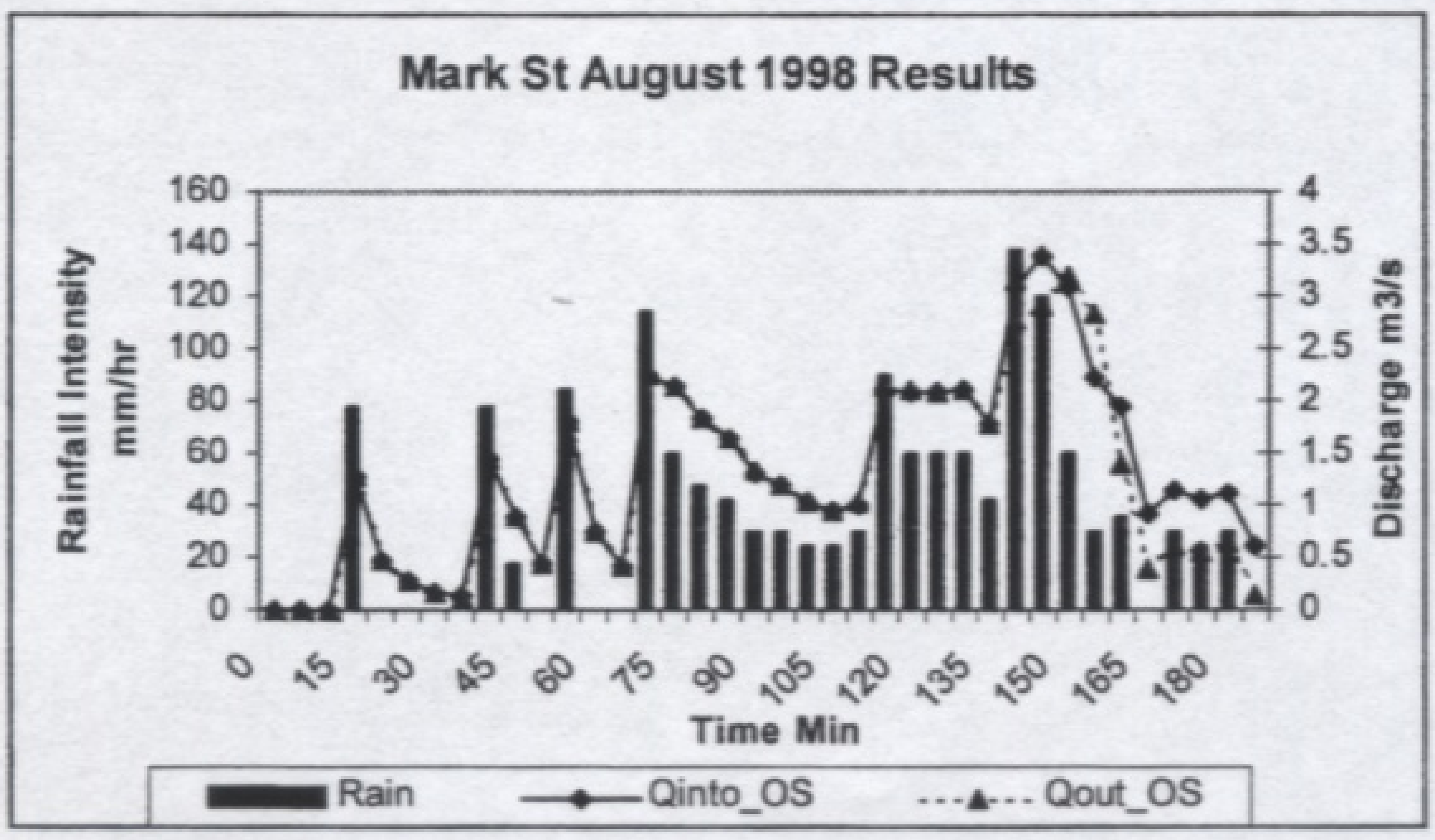

Figure 8 – WBNM2001 – Hyetograph & Hydrographs for 1998 Event

WBNM2001 Multiple Storm Simulation Summary Table			
No.	ARI	RAINFALL	DISCHARGE
	(year)	(mm)	(m3/s)
1	1	45.38	3.47
2	2	59.61	4.01
3	5	81.03	4.85
4	10	94.15	5.78
5	20	110.95	6.92
6	50	133.47	7.94
7	Aug 1998	120	4.01
8	Oct 1999	113.5	4.7
9	100	150.97	9.01

Table 2 – WBNM2001 – Multiple Storm Summary

The results determined by WBNM in terms of levels of ponding accurately matched those observed in both Mark St and Koloona Avenue.

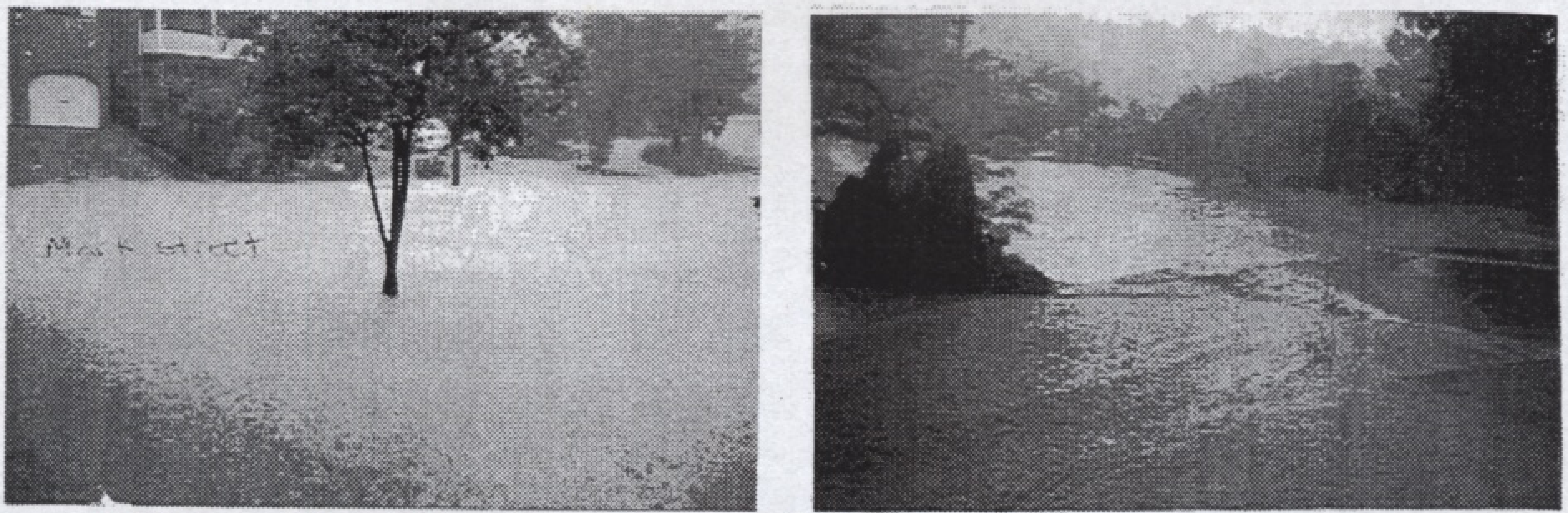

Figure 9 – Photos Of Flood Levels Experience In Mark St & Koloona Ave. In The 1999 Event

3 BOUNDARY ST FLOOD INVESTIGATION PORT MACQUARIE

3.1 Complex Flood Plain Storage Interaction from Multiple Adjoining Catchments

3.1.1 Overview

BALANCE Research&Development were commissioned to undertake a flood investigation of the access road into Port Macquarie Air Port. It is proposed to raise the roadway above the 1:50 year flood level of the nearby Hastings River. However as the Hastings River has a catchment exceeding 3500km^2, it may be prudent to investigate the local catchment in order to determine whether local flooding due to a much shorter duration storm may present an equally devastating situation of cutting off access to the airport.

Boundary St runs off a local arterial road called Hastings River Drive and provides the only means of access to the airport. Hastings River Drive runs parallel to the Hastings River, from the township of Port Macquarie to the Princes Highway. For a distance of almost 5km this roadway is elevated on an embankment with 12 separate structures beneath it, including 2 bridges and 5 segments of roadway that act as overflow weirs during floods. Analysis was undertaken utilising WBNM. It was soon apparent by delineation of the catchment and hydraulic analysis of the Boundary Road culvert that there are some complex catchment interactions occurring. The scope of analysis had to be widened considerably to model the interaction of 3 separate catchments totalling 41km^2 that act as a single catchment due to the presence of the Hastings River Drive embankment.

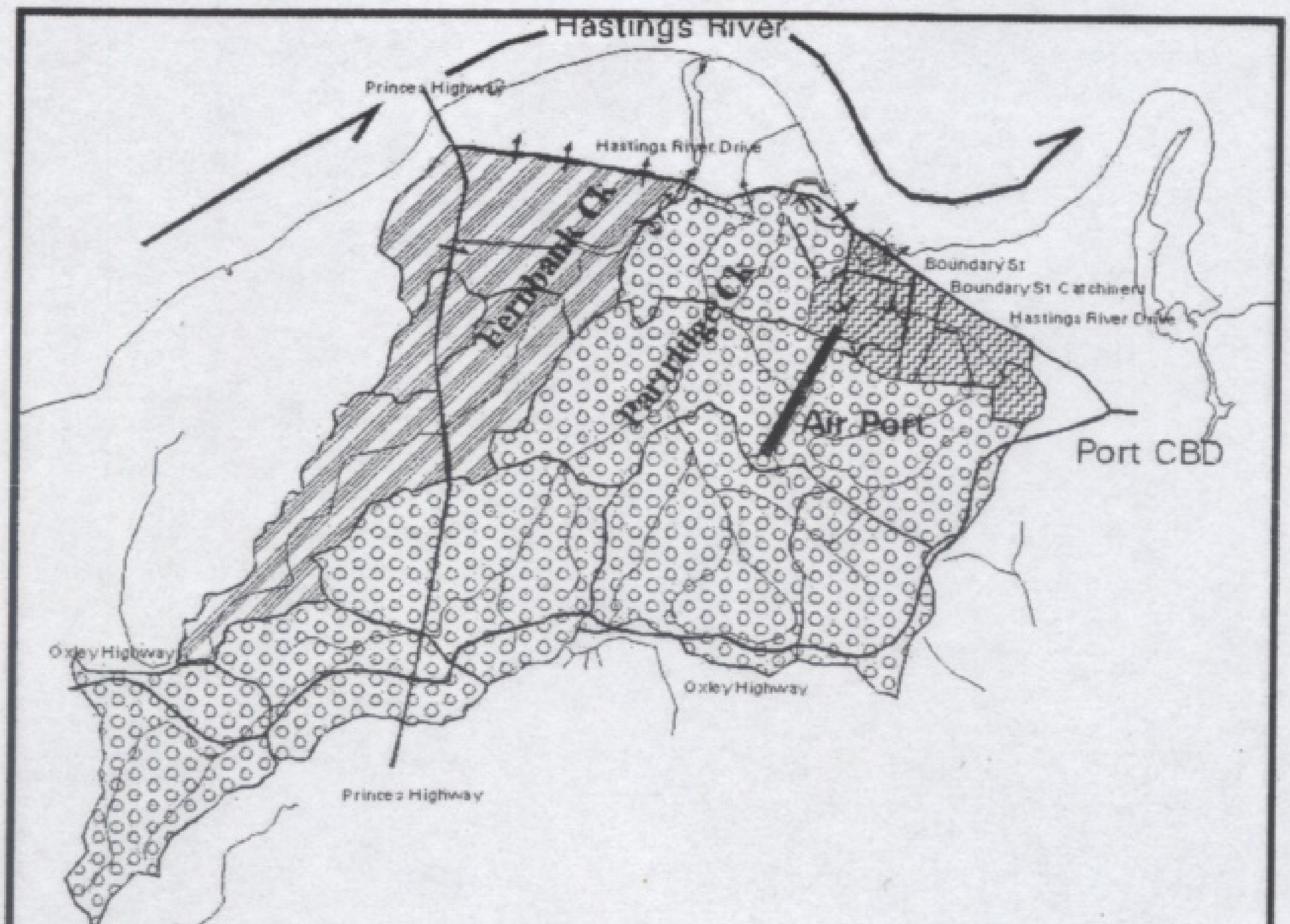

Figure 10 – Local Catchments Upstream of Hastings River Drive Port Macquarie

To complicate matters further, each of the three catchments has a significant amount of floodplain storage due primarily to the Hastings River Drive embankment. As flood flows exceed the in-bank capacity of creeks, these floodplain storages start to fill. The storage significantly delays the time of peak discharge. The time of concentration for the catchment itself is around 2 hours, as is evidenced by the 2 hour storm producing the maximum peak discharges for inflow to the storages. However the time to fill the storages delays the hydrographs, resulting in the critical storm duration increasing to the 12 hour design storm event.

Finally, as each of the catchment's flood plain storage fills, there is also some cross catchment flow occurring. The central catchment of Partridge Creek is the largest, and fills before spilling to the east into the Boundary St tributary. As flows continue to increase it next spills west into the adjoining Fernbank Creek storage. The filling of the 3 storage areas have to be modelled correctly in order to provide a sensible answer. That is, the predicted flood levels in each of the adjoining flood plains need to be accurate, otherwise clearly the assumed direction of cross catchment flows would be incorrect.

3.1.2 Modelling Technique Adopted

WBNM was able to successfully model both the filling of all floodplain storages and the cross catchment flows. Each storage was defined by a stage-storage curve. Each culvert and bridge structure was specified as the primary outflow from its storage. The secondary outflows, cross catchment flows that occurred over stream banks or embankments separating each storage, were modeled as weirs. These weirs connected each flood plain storage. For lower flowrates, all flows were in-bank and contained. As flows increased, they exceeded bank full capacity, and the flood plain storages began to fill. Next, beyond some point the flood plain storages reached a level where flows breached the bank and allowed water to pass from one catchment to the adjoining catchment.

The only complication in the modeling process was in determining which storage was first to direct flows into which adjoining catchment. This meant that several models with slightly differing connectivity were required in order to determine the correct scenario. Once this was determined, calculations were made for large storms to verify that for this case flood levels in each of the adjoining flood plains were equal, and therefore the separate flood plains were acting as one large storage area.

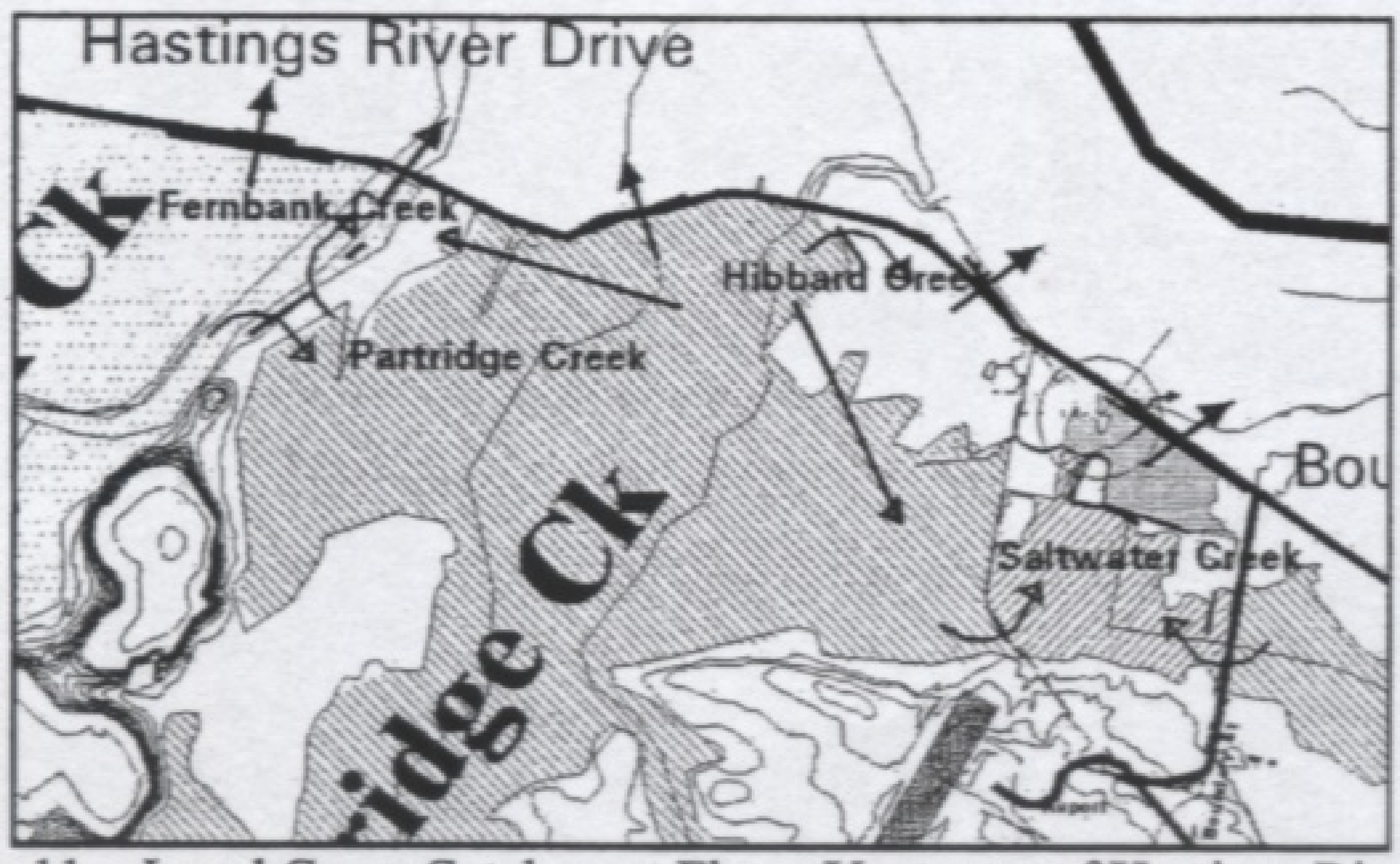

Figure 11 – Local Cross Catchment Flows Upstream of Hastings River Drive

During this study, it became evident that there are advantages in being able to carry out calculations in different ways when using computer models. Currently in WBNM, as in many other models, the sequence of calculations is specified by the connectivity of sub areas. This is primarily because the mode of calculations is to process the full hydrograph for each sub area, moving from the top of the catchment to the bottom. This restricts the model to providing backwater details for only the current sub area. An alternative is for calculations to be made at each time step, for all subareas, all flowpaths and all storages, before moving to the next time step (as is done in some hydraulic models). If this was done, the connectivity between elements of the model would not have any restrictions what so ever, and also downstream water levels would be explicitly known by at each sub area at every time step. This would provide WBNM with an extremely powerful back water computational ability through multiple upstream sub areas, making it a full back water model.

3.1.3 Results of Modelling Procedures Undertaken

The catchment was analysed using a range of design storms, with Average Recurrence Intervals ranging from 1 to 100 years, and durations from 30 minutes to 72 hours.

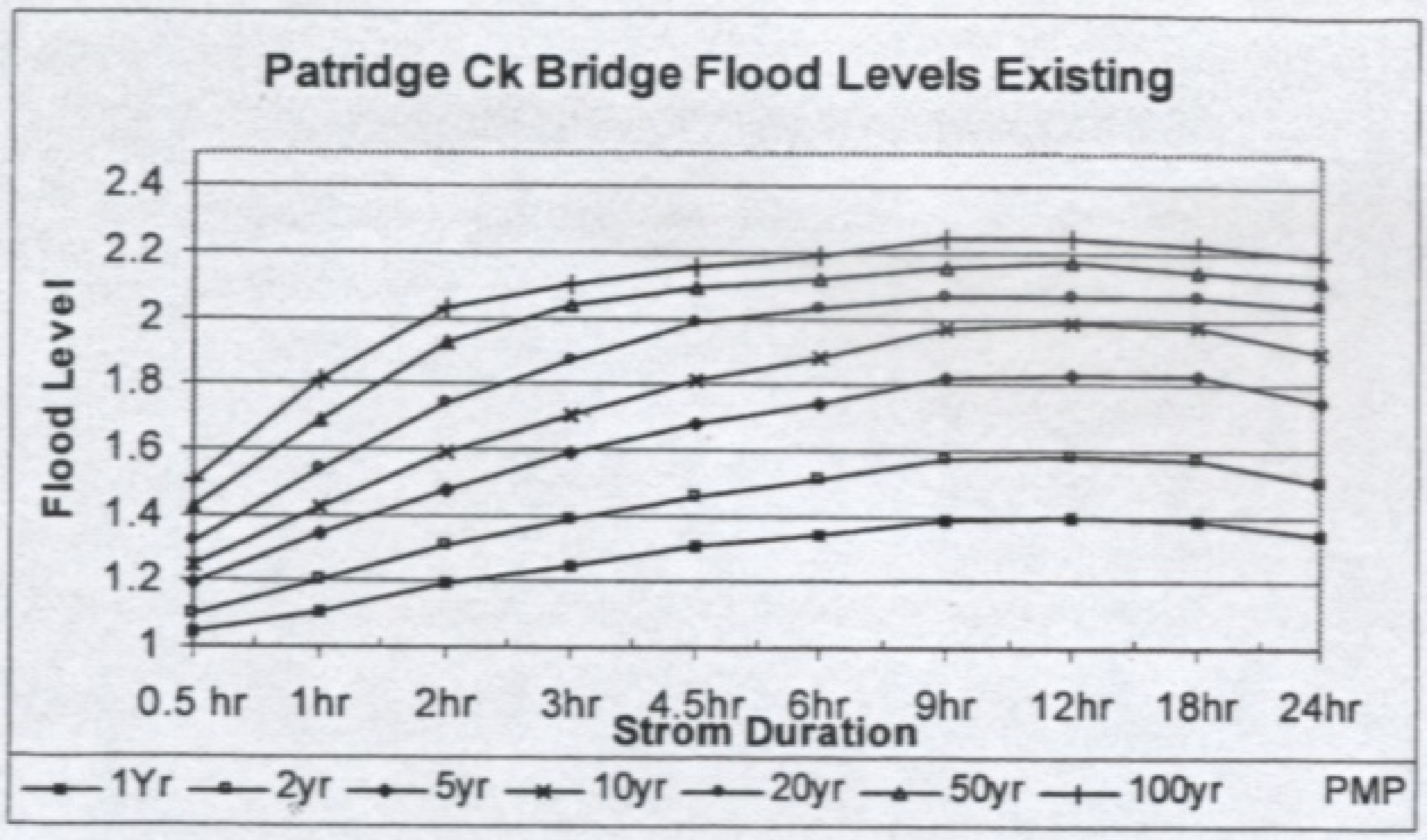

Figure 12 – Resulting Flood Levels For Full Storm Event Spectrum

Results indicated that the Boundary St culvert was indeed affected by cross catchment flows. It was therefore recommended that if the proposal to raise Boundary St was to proceed, a flood gate would need to be installed to ensure that cross catchment flows did not pass beneath the roadway and potentially inundate residential areas to the east of the airport.

4 CONCLUSIONS

The enhancements offered in the latest version of the hydrologic model WBNM (WBNM2001-V101), provide users with a robust model capable of modelling extremely complex situations involving the interaction of hydraulics and hydrology. The ability to model the impact on the peak flows and hydrograph shape from a catchment, as flows are controlled by various structures and flood storages is extremely efficient. Not only does the model realistically derive the peak flows and hydrograph shape it also has the ability to realistically determine extremely complex hydraulic behaviour from multiple outlets at a structure and multiple interacting structures. The results are presented as hydrographs from every outlet in the structure, plus the total outflow, and includes a full stage history of water levels stored in the structure.

The model has been recently used to solve complex urban drainage problems involving well known and long standing flood affected areas in Wollongong. The analysis has shown that the resulting flood levels are a consequence of three separate cascading ponds that form when the capacity of a drainage pipe is exceeded and overland flow occurs. WBNM2001 has accurately simulated the flood levels experienced in both the August 1998 and October 1999 events.

In another case study, WBNM2001 was able to successfully model extremely complex multiple flood plain interaction. In Port Macquarie a long road embankment creates one large flood plain storage, as a combination of three separate flood plain storage areas, which are linked by overflows between them. The correct sequence of filling and overflows between the separate storages, as water levels rose, was able to be simulated.

5 ACKNOWLEDGEMENTS

These studies were carried out for Wollongong City Council and Hastings Council. Significant input by each Council via provision of data is gratefully acknowledged.

6 REFERENCES

Boyd, M.J., Pilgrim, D.H. and Cordery, I. (1979). A Storage routing Model Based on Catchment Geomorphology, *Journal of Hydrology*, Vol.42, pp.209-230.

Boyd, M.J., Bates, B.C., Pilgrim, D.H. and Cordery, I. (1987). WBNM – A General Runoff Routing Model. Institution of Engineers Australia, 4th National Conference on Local Government Engineering, NCP 87/9, pp. 137-141.

Boyd, M.J., Rigby, E.H., VanDrie, R. and Schymitzek, I. (2001). WBNM2001 – Computer Programs and User Guides. http://www.uow.edu/eng/research/

Rigby, E.H., Boyd, M.J. and VanDrie, R. (1999). Experiences in Developing a Flood Hydrology Model: WBNM2000. 8th International Conference on Urban Storm Drainage, Institution of Engineers Australia, pp.1374-1381.