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Modelling of Hydraulic Flood Flows using WBNM2001

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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.

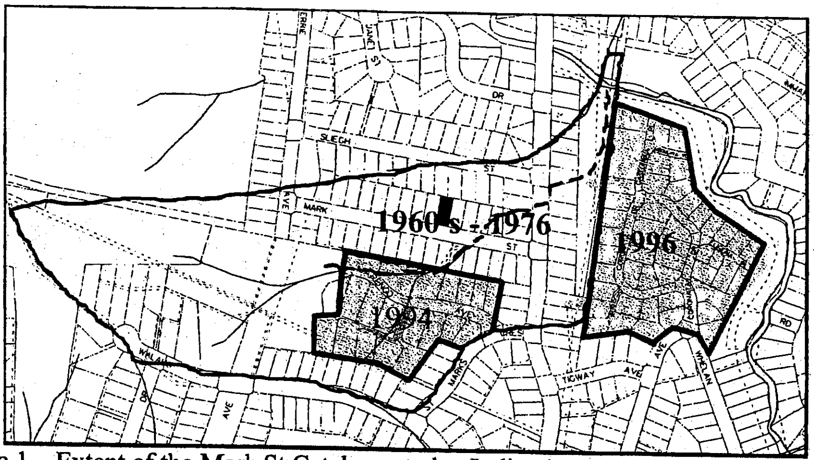

Figure 1 – Extent of the Mark St Catchment also Indicating History of Development

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.

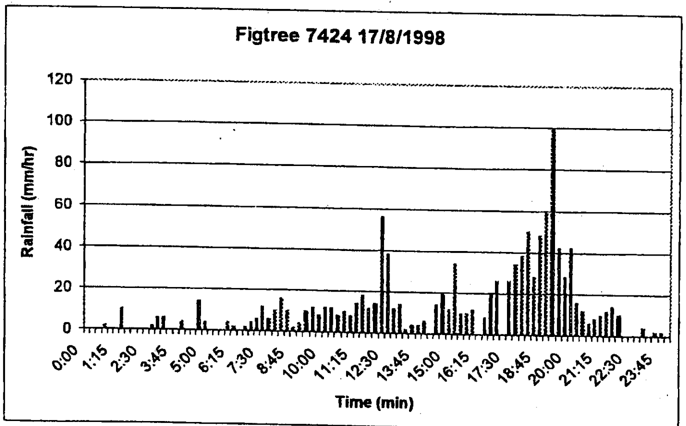

Figure 2 – Rainfall Intensities Figtree Gauge 17th August 1998

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.

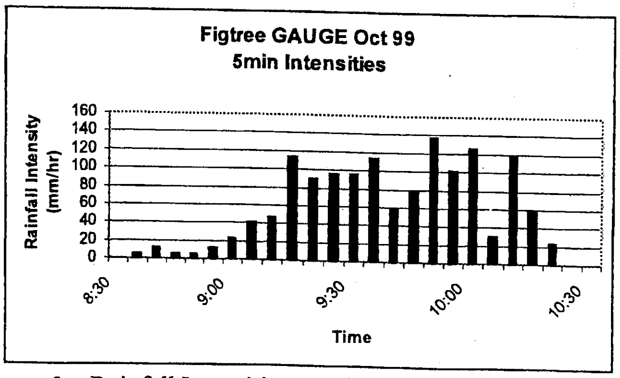

Figure 3 – Rainfall Intensities Figtree Gauge 21st October 1999

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.

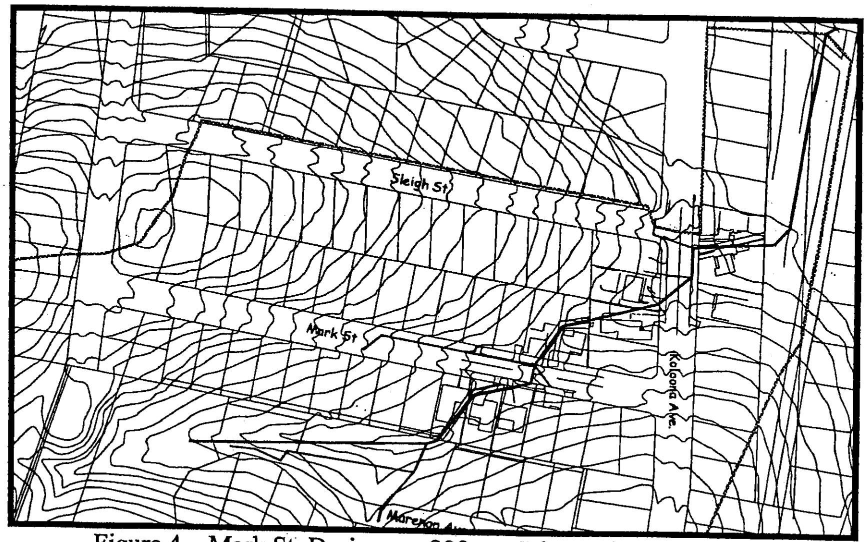

Figure 4 – Mark St. Drainage, 900mm Diam. Pipe Design ~1972

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.

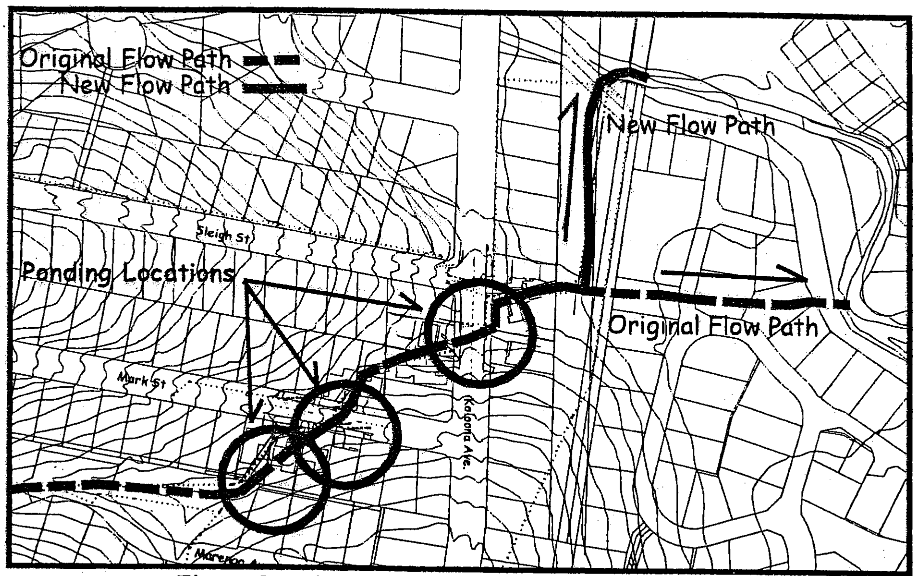

Figure 5 – Change in Down Stream Flow Paths ~ 1996

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.

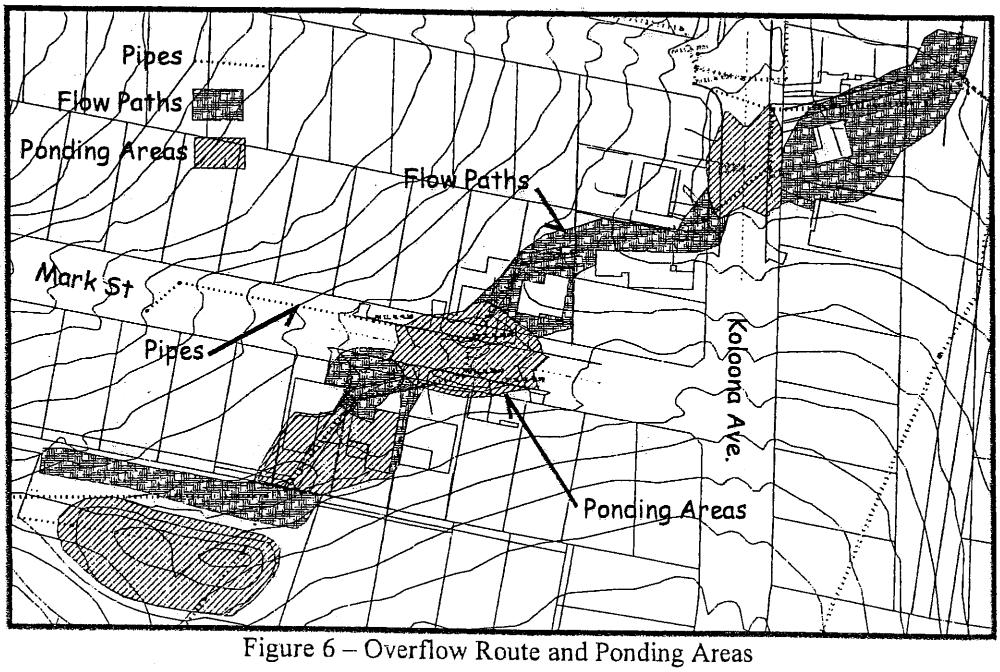

Figure 6 – Overflow Route and Ponding Areas

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.

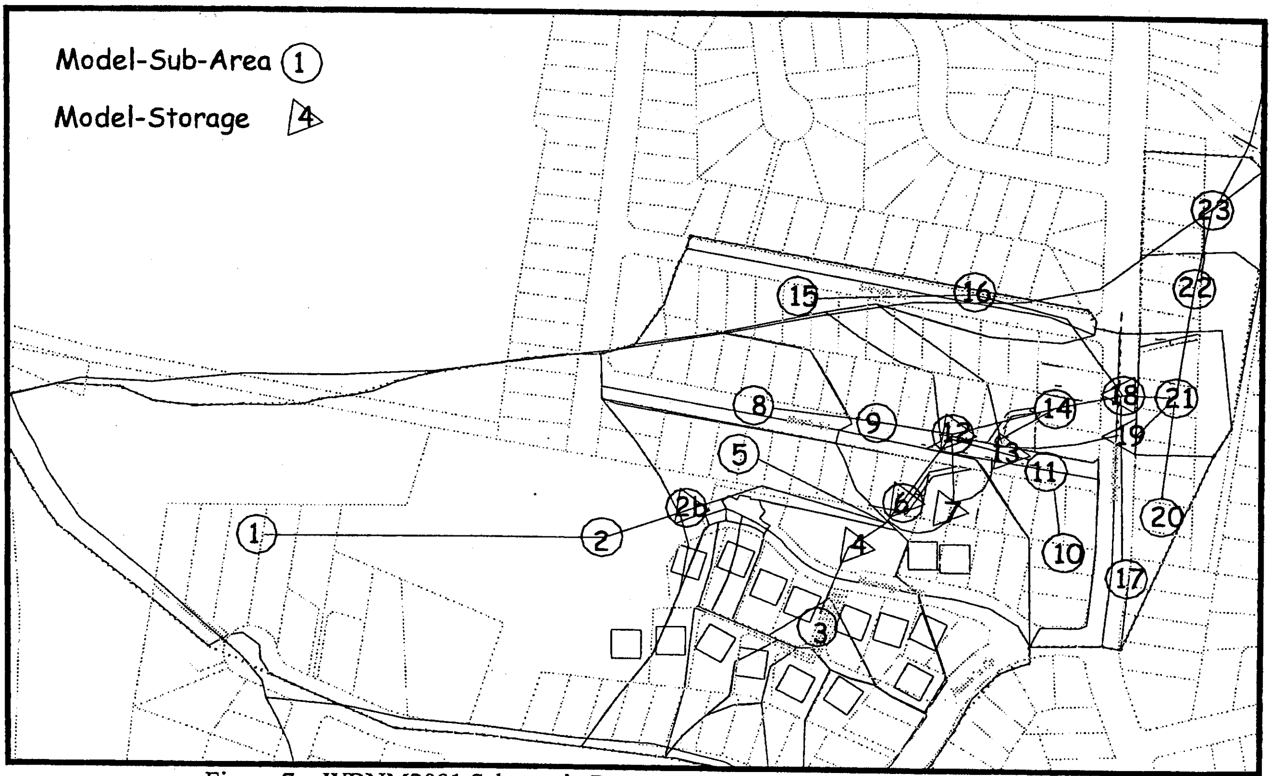

Figure 7 – WBNM2001 Schematic Representation of the Mark St Catchment

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.

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

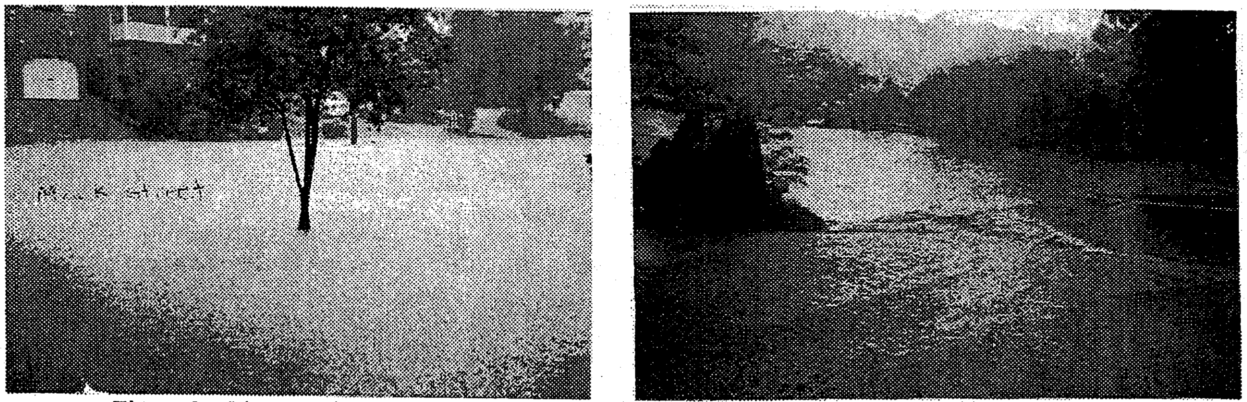

Figure 9 – Photos Of Flood Levels Experience In Mark St & Koloona Ave. In The 1999 Event

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.

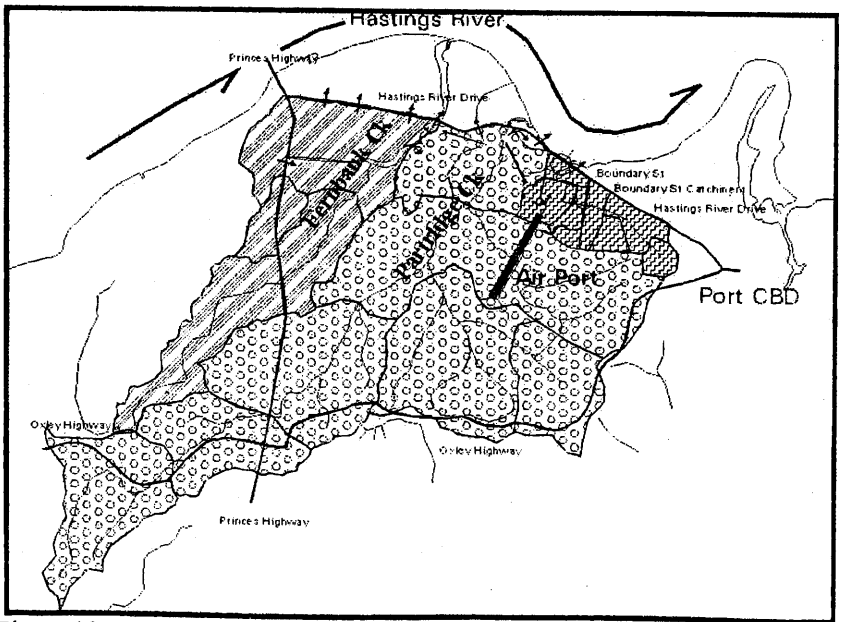

Figure 10 – Local Catchments Upstream of Hastings River Drive Port Macquarie

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.

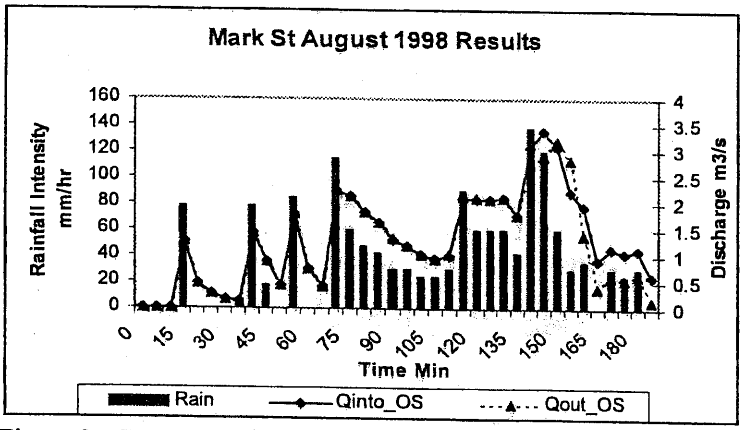

Figure 8 – WBNM2001 – Hyetograph & Hydrographs for 1998 Event

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

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.

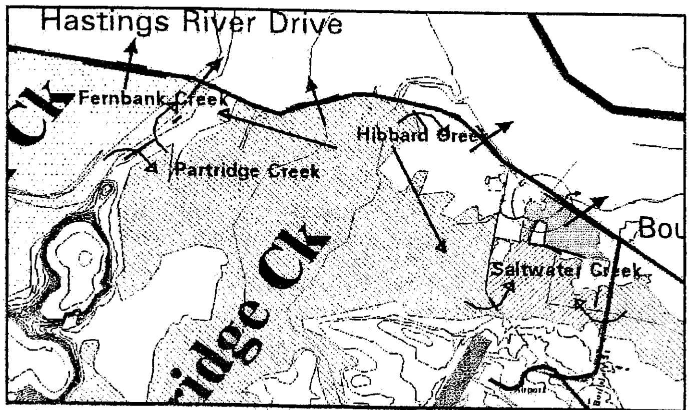

Figure 11 – Local Cross Catchment Flows Upstream of Hastings River Drive

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.

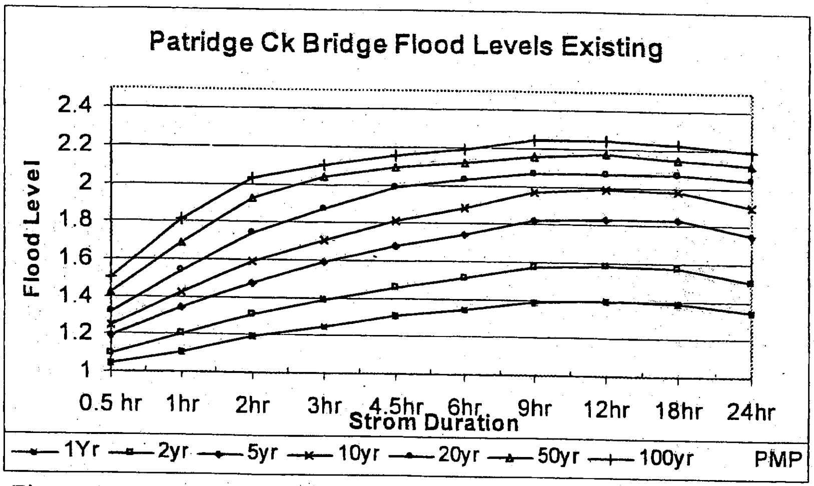

Figure 12 – Resulting Flood Levels For Full Storm Event Spectrum

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.

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Modelling urban catchments with WBNM2000

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Summary

The features of the latest version of WBNM are outlined, with particular reference to urban catchment modelling. Development of lag relations used for modelling overland flow and channel flow are described.

Introduction

The concept of WBNM was first outlined in 1975 [1] and a computer program made available in 1979 [2]. Originally the emphasis was on developing a model which gave a good representation of the physical structure of the catchment, and having a sound relation between the catchment and its lag properties. WBNM had built in relations between the lag time and subcatchment size and fixed nonlinearity, based on studies of lag times on natural catchments [3]. Due to these built in relations, WBNM was particularly easy to apply, requiring the selection or calibration of only one parameter.

Despite its ease of use, the realistic representation of the catchment made WBNM particularly suitable for extension beyond its original intention. Thus division into subcatchments allowed modelling of part urban catchments, with the lag parameter reduced only for the urbanised portion. Additionally, WBNM separated overland flow on the subcatchment surface, from upstream runoff which was confined to the main stream channel. Thus changes to stream channels could be treated separately from changes to land surfaces.

In 1994 [4] the model was substantially upgraded for urban catchments. Each urbanised subcatchment was divided into pervious and impervious surfaces, with separate rainfall losses and lag parameters. Flood detention basins with built in culvert hydraulics, as well as diversion of surcharging flows were added. The computer program also included design storms, using design rainfall intensity-frequency-duration data and design temporal patterns from Australian Rainfall and Runoff [5].

The latest version of the model, WBNM2000, has continued this development for urban catchments. Features introduced include built in onsite stormwater detention, and multiple flow diversions at points of surcharge. To handle the case where typical flood producing storms are considerably longer than the critical duration of the catchment, embedded design storms have been added.

Pervious and impervious runoff from urban catchments

Impervious surfaces on urban catchments reduce infiltration and increase runoff volumes. The total impervious surfaces can be subdivided into those directly connected to the stormwater drainage system, and supplementary impervious surfaces which are not connected, runoff from which flows across pervious surfaces before reaching the drainage system. We would expect the directly connected surfaces to be active for both large and small storms, and the supplementary impervious as well as the pervious surfaces to become active in the larger storms.

For modelling it is desirable to determine the effective directly connected impervious area from plots of rainfall and runoff depths. This value is often found to be less than the value estimated from maps. In a study of 38 urban catchments [6], the effective directly connected area was found to be on average 87% of the directly connected, and 75% of the total impervious areas estimated from maps.

When rainfall and runoff depths are plotted, the larger events often lie above the directly connected impervious line, indicating that runoff is generated on both pervious and impervious surfaces. Because of this, it is more realistic to model urban catchments by considering separately the runoff from pervious and impervious surfaces than to lump the two surfaces together.

Lag times on urban catchments

On urban catchments flow velocities increase, partly due to the hydraulically smoother surfaces, and also because the runoff is concentrated into flow paths which have higher flow depths and therefore higher flow velocities. Removal of meanders in channel "improvements" can increase the slope, thereby increasing the flow velocity. The effect of the increased flow velocities is to reduce lag times, giving hydrographs which follow closely the peakiness of the rainfall hyetograph and which have higher peak discharges. Lag times will be reduced for both overland flow and for flow in channels, and both of these should be considered in modelling.

Many studies of lag times on urban or part urban catchments have been carried out and Table 1 summarises some of these results. Further details are given in [7]. The degree of urbanisation is measured either by the urban fraction URB or by the impervious fraction IMP (range 0 to 1.0). URB is the ratio of urban development area to the total catchment area and does not distinguish between the density of development. This difference can be quite substantial, consider for example urban residential development with single dwellings on separate lots which is typically 45 % impervious, compared to high density commercial development which can exceed 95 % impervious. For this reason IMP is a more physically meaningful measure of urbanisation.

Table 1. Lag Reduction Factors for Urban Catchments.

Study	Number catchments	Area range (hectare)	URB range	IMP range	Lag reduction factor
Rao et al[9]	13	12-5000		0.0-0.38	$(1+IMP)^{-1.66}$
Aitken[10]	6	80-5600	0.25-1.00		$(1+URB)^{-1.97}$
Aitken[10]	11	40-9000	0.0-1.0		$(1+URB)^{-2.74}$
NERC[11]	138	4-61700	0.0-0.84		$(1+URB)^{-1.99}$
Crouch&Mein[12]	3	80-3200		0.25-0.40	$(1+IMP)^{-4.1}$
Desbordes et al[13]	21	0.4-5000		0.15-1.0	$(1+IMP)^{-1.9}$
Schaake et al[14]	19	0.1-62		0.09-1.0	$IMP^{-0.26}$
Espey et al[15]	41	0.04-39		0.02-1.0	$IMP^{-0.18}$

Some studies consider lag reduction on the catchment as a whole. Carter [8] used 24 natural, part urban, and fully urban catchments and found that that lag times for fully urbanised catchments were on average 0.17 times those of natural catchments. The relations in [9], [10], [11], [12], and [13] are for the whole catchment and predict the lag time of fully urbanised catchments to be on average 0.25 times that of natural catchments, although the use of IMP and URB is a little doubtful in some cases.

The studies in [14] and [15] were mainly for small parking lots and may be more applicable to overland flow. These relations predict a lag ratio of about 0.5 for paved surfaces compared to grassed surfaces. The "kinematic wave" equation in Australian Rainfall and Runoff can be used to predict ratios of travel times on paved surfaces compared to other surfaces. Using a roughness coefficient of 0.011 for paved, and a range from 0.15 to 0.48 for grass and lawns gives ratios of travel times on paved compared to these surfaces between 0.15 and 0.06.

Some authors [8], [15], [16] have suggested that the reduction in lag time for the catchment as a whole is greater than the reduction in overland flow times, and that this is because the "improved" drainage system on the urban catchment is more effective in reducing lag times than changes from grassed to paved surfaces. If so, this is a good reason for separate routing of overland flow and channel flow in a model.

Lag relations for urban overland flow in WBNM2000

Lag relations for overland flow were determined using recorded storms on 9 catchments in Australia (Table 2). Total IMP was measured from maps, directly connected IMP was determined from plots of rainfall and runoff depths.

Table 2. Urban Catchments used to calibrate WBNM2000.

Catchment	National station number	Area (hectare)	URB	Total IMP	Directly connect IMP	Number of events	Rainfall range (mm)
Curtin	410745	2700	0.57	0.17	0.17	14	13-79
Mawson	410753	445	0.78	0.26	0.21	11	26-80
Long Gully	410746	494	0.16	0.05	0.05	14	26-65
Giralang	410763	94	0.70	0.25	0.22	14	5-79
Maroubra	213300	57	1.00	0.52	0.16	20	4-102
Strathfield	213304	231	1.00	0.50	0.29	20	2-139
Fishers Ghost	213006	235	0.96	0.36	0.25	20	8-86
Jamison Pk.		21	1.00	0.36	0.21	20	1-54
Vine St.	230109	64	1.00	0.37	0.31	11	14-108

The overland flow surface was split into a directly connected impervious area and the remaining pervious plus supplementary impervious areas. The natural catchment lag relation was used to allocate lag time to these latter areas since the flow velocity on grassed surfaces is relatively low.

For the impervious surfaces, a lag relation was developed by determining the ratio of lag times on impervious and pervious surfaces. This was found to be close to 0.10. The resulting relation is built into WBNM2000 and calculates a lag time for each impervious area depending on its size. Because lag times are automatically reduced for impervious surfaces, the same lag parameter is applicable to both natural and urban catchments.

Figure 1 shows parameter values derived for the catchments of table 2, as well as a range of natural catchments. Generally, urban parameter values have more scatter than natural catchments due to reduced damping of the runoff. For this reason, while the "flashiness" of urban hydrographs is reproduced quite well, estimated peak discharges can differ from recorded values, typically by 30% in this study.

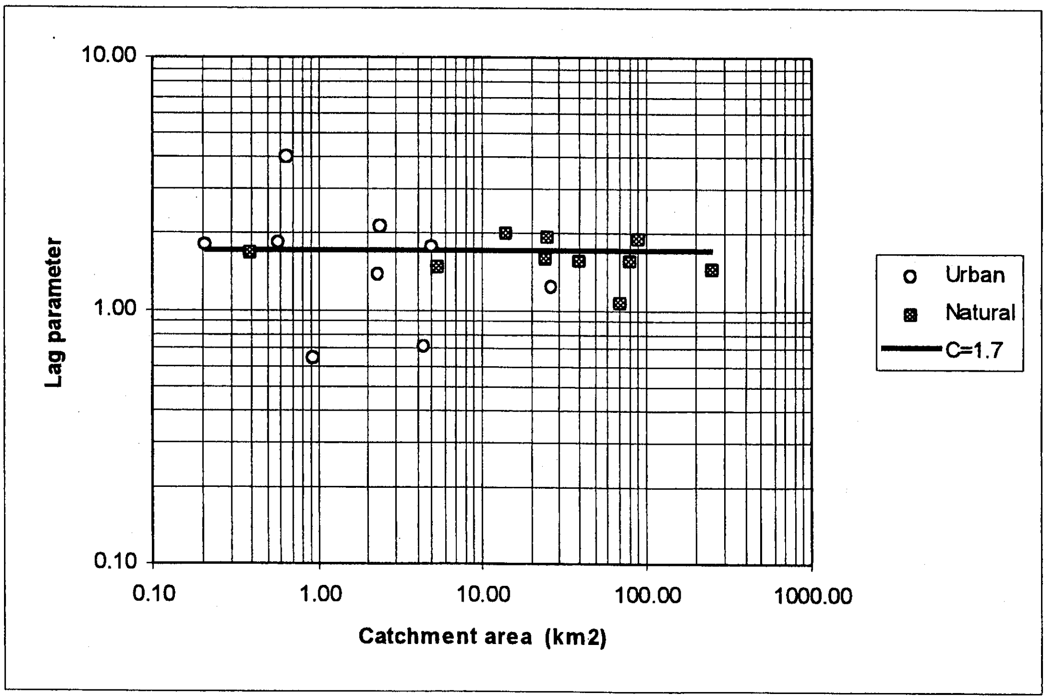

Figure 1. Lag Parameters for Urban and Natural Catchments.

Stream channel routing

Overland flow hydrographs are added into stream channels depending on the structure of the stream network. Three options are available for channel routing:

(a) Nonlinear routing using a Channel Factor selected to reflect the increased flow velocities in the "improved" channel.

(b) Muskingum routing, with parameters K and x selected depending on the translation and attenuation properties of the reach.

(c) Delay, in which the hydrograph is delayed through the reach by a specified time, but without attenuation.

Onsite detention storage

Every subcatchment has an optional onsite detention storage which takes its local runoff. Inflow to the onsite detention can be runoff from the impervious surfaces, or from the pervious surfaces, or any combination of the two. The onsite detention storage needs an elevation-storage volume-discharge relation, and can be a high early discharge type if desired. WBNM2000 will calculate the elevation-discharge relation if the dimensions of the outlet pipe and overflow weir are specified.

Mainstream detention basins

Larger flood detention basins can be placed on the stream channels. Flood routing procedures are the same as for the onsite detention basins, and require an elevation-storage volume-discharge relation. Again, the elevation-discharge relation is automatically calculated if the dimensions of the culvert and weir are specified.

The most accurate culvert hydraulics procedures available are those of the US Department of Transportation [17]. Boyd [18] fitted equations to these charts and incorporated them into WBNM in 1994. These relations continue to be used in WBNM2000.

Flow diversions

An important feature of urban catchments, which must be included in a successful urban catchment model, is the activation of the major drainage system during large floods. Flow paths during these floods may be substantially different to those in smaller floods.

Flow diversions will generally occur at mainstream structures whose capacity is exceeded, such as blocked culverts. WBNM2000 allows up to five major flowpaths for surcharging flows from each subcatchment. Because surcharging occurs at structures, flow diversions in WBNM2000 also occur at structures. These can be mainstream flood detention basins, blocked culverts, or at a point of reduced channel capacity.

For each major flowpath, an elevation-diverted discharge relation is required, plus the destination of the diverted flow. If several flow diversions to different destinations are occuring from the one location, each commencing at a different elevation, separate elevation-discharge relations are used.

Embedded design storms

The design storm intensity and temporal patterns in Australian Rainfalll and Runoff, while detailed and comprehensive, are for short bursts within longer storms. The burst duration is selected to be critical for the particular catchment. This raises two issues. Since the design rainfall is for a burst within a longer storm, rainfall initial losses should be for a burst rather than for a complete storm. Initial losses will be lower for bursts than for complete storms, because of prior wetting of the catchment, so initial losses derived for complete storms are likely to be too large, and will underestimate runoff.

Secondly, when design bursts are applied to an event based model, the assumption is that flows on the catchment are initially zero. In practice, flows will be greater than zero due to rainfall prior to the intense burst. An example of this occurred in the extreme storms experienced in the Wollongong region in August 1998 (AEP near to 1 in 100 years). The critical duration for many of these catchments is 120 minutes and an intense burst of this duration occurred on the evening of 17 August. However, this was preceded by 12 hours of solid rain, and streams in the area were already running at near to full.

The presence of flood detention on an urban catchment can exacerbate this problem, since the storages may in practice be part full from the prior rainfall, but assumed to be initially empty for modelling.

The preceding discussion indicates that application of design bursts without consideration of the rainfall in the complete storm may underestimate flooding. Several Australian studies have addressed this matter. Phillips [19] suggested that design bursts be placed within longer duration recorded storms, while Rigby [20] suggested that design bursts of a particular AEP be embedded within longer duration design storms. The advantages of the second approach are that the same AEP can be used for the critical burst and for the longer duration storm, and both burst and longer duration storm rainfall data are readily available in Australian Rainfall and Runoff.

In 1994 WBNM incorporated ARR design rainfall data. This has been extended in WBNM2000 to include embedded design storms. The input data file simply requires values of the 9 rainfall coefficients from Australian Rainfall and Runoff, plus the AEP and duration of the burst and the longer duration storm. WBNM2000 automatically calculates design rainfall intensities, applies an area reduction factor and distributes the rainfall into the appropriate design temporal pattern.

WBNM2000 also calculates Probable Maximum Precipitation storms for Probable Maximum Flood estimates, using Bureau of Meteorology procedures [21].

Conclusions

When modelling urban catchments it is desirable that the model differentiate between runoff from pervious and from impervious surfaces. This allows better modelling of the range of storm events, small ones in which most runoff comes from the directly connected impervious surfaces, and large ones in which the runoff comes from both impervious and pervious surfaces. Separating runoff from impervious and pervious surfaces also is convenient for modelling onsite detention storage on sites, since in many cases the impervious runoff will be directed to the onsite detention. Finally, separating pervious and impervious runoff allows extension of the model to water quality considerations, since the pollutant loads will be different from these two surfaces.

It is preferable to define the degree of urban development in terms of the impervious fraction rather than the urban fraction, since this better differentiates between land use types, such as commercial and residential developments. Separating overland flow routing from stream channel routing allows modelling of various kinds of urban development. The stream channel alone can be changed, or the land use alone, or some combination of the two.

The emphasis in developing WBNM2000 has been to provide a tool which incorporates these features and which allows modelling of the many possible scenarios on urban catchments.

WBNM2000 is freely available at http://www.uow.edu.au/eng/research/water.html or email michael_boyd@uow.edu.au.

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