Stormwater Industry Association 2002 Conference on Urban Stormwater Management,
Orange NSW
23-24 April 2002

TOWARDS CONTINUOUS SIMULATION: A COMPARATIVE ASSESSMENT OF FLOOD PERFORMANCE OF VOLUME-SENSITIVE SYSTEMS

George Kuczera and Peter J. Coombes
Civil, Surveying and Environmental Engineering,
University of Newcastle, Callaghan, NSW

Abstract. Continuous simulation of hydrologic systems requires long-term high-resolution climate data. With continuing advances in stochastic rainfall models continuous simulation is likely to become a practical tool for hydrologic risk assessment. This study explores whether adoption of continuous simulation is worthwhile. Two case studies compare the performance of continuous simulation and the Australian Rainfall and Runoff design storm approach. Both consider peak flow estimation in the context of volume-sensitive systems involving detention basins and rainwater tanks. The case studies demonstrate the potential for large errors when using design storms to simulate the flood performance of volume-sensitive systems. Significantly there does not appear to be an obvious way to “fudge” the design storm approach to make it reliable. Given the wide usage of detention and retention systems in urban stormwater management one has to seriously question the fundamental assumptions that underpin Australian flood estimation practice. The case for adopting rigorous joint probability approaches such as continuous simulation is considerable.

1. INTRODUCTION

The design of hydraulic systems that have to cope with natural flows of flood magnitude is risk-based. In Australia risk-based design is almost universally employed with Australian Rainfall and Runoff, hereafter referred to ARR [Institution of Engineers, Australia, 1987] being the authoritative document guiding practice. Indeed in the design of stormwater systems ARR advocates use of the major/minor design philosophy in which the engineer designs the system so that the risk of the system failing to cope with high flows is acceptable according to some criterion and, in the event of a failure, the system copes sufficiently well to avoid catastrophic and unacceptable loss.

Fundamental to risk-based design is the estimation of the probability of different modes of “failure” – here the term failure is used to refer to the occurrence of predefined events (such as surcharging or exceedance of safety thresholds) which constitute unacceptable outcomes. ARR states that the primary method for evaluating such probabilities is based on the so-called design storm approach for which “the intention is to derive a flood of selected probability of exceedance from a design rainfall of the same probability” [ARR, p6]. Central to this approach is the assumption that median values of variables other than rainfall (such as losses, baseflow, temporal patterns and hydrograph model parameters) be used. ARR admits that “rigorous proof is not possible” that the design storm approach does produce floods with the same exceedance probability as the rainfall. Indeed ARR admits that “there is a need for research to test this approach”.

These are disquieting words that introduce the design storm approach which dominates Australian flood estimation practice.

Until recent years there has been a dearth of research investigating an assumption fundamental to Australian flood estimation practice. Rahman et al. (2002) describe work undertaken by the CRC for Catchment Hydrology and advocate the adoption of explicit joint probability methods to replace the design storm approach. They demonstrate that their particular joint probability approach is more accurate than the design storm approach. However, their approach is still in the development phase requiring considerable data to calibrate probability distributions of rainfall, losses and rainfall-runoff model parameters. They note that their approach "is readily applicable to gauged catchments with good pluviograph data and limited streamflow data" [Rahman et al., 2002, p208]

A more general joint probability approach uses continuous simulation. Weinmann et al. (2000, p. 566) state that that the "continuous simulation approach is conceptually the most desirable one". Properly applied, continuous simulation can rigorously evaluate the joint probability contributions affecting flood probabilities. Continuous simulation is a Monte Carlo approach which requires simulating the flood response of a very long rainfall record and empirically deriving flood probability distributions. The key idea is that the simulation is sufficiently long to ensure that all significant joint probability interactions are sampled to yield an accurate inference of flood probabilities. Indeed ARR (p.6) comments that continuous simulation is "theoretically superior" to other approaches. However, both ARR and Weinmann et al. (2000) have argued that the lack of long-term pluviograph records limits the ability of continuous simulation to evaluate probabilities in the tails of the distributions.

We are of the opinion that the impediments to continuous simulation are disappearing. The most significant impediment has been the availability of long pluviograph records. Research in stochastic point rainfall models has reached the point where it is not fanciful to think that such models will soon be sufficiently robust for general use. For example, the DRIP stochastic point rainfall model [Frost et al., 2000; Heneker et al., 2001] has been shown to reproduce extreme rainfall statistics without calibration to such statistics. Significantly Jennings et al. (2002) present simple strategies for regionalizing the DRIP model. The regionalisation involves transferring the DRIP model calibrated to a long-term pluviograph record to a site with a short pluviograph record. Current work is extending the transfer to sites with only daily rainfall records. Therefore, for small catchments for which point rainfall inputs are deemed adequate the main impediment to continuous simulation may be overcome in the near future. Another impediment to continuous simulation has been the lack of computer power to undertake massive Monte Carlo simulation. The current generation of personal computers can perform a 1000-year simulation at 5-minute resolution in a matter of minutes when using a lumped rainfall-runoff model.

The question therefore arises: What is to be gained by moving from a design storm to a continuous simulation approach? This paper seeks to answer this question in two ways. First it offers some theoretical insight into the connection between the design storm approach and rigorous joint probability assessment. The insight makes clear under what conditions the design storm approach may yield reliable estimates of flood probability. Second it investigates how well the design storm approach evaluates the flood performance of volume-sensitive systems. With the growing use of detention and retention storage to manage stormwater in urban environments it is very pertinent to ask such a question, particularly since the ARR approach is primarily focussed on producing flood peaks with the same probability as the design rainfall. Two case studies are presented. One considers evaluating the performance of detention basin at the outlet of a 10 km^2 catchment. The second considers evaluating the performance of a rainwater tank in an urban allotment.

2. CONTINUOUS SIMULATION AND DESIGN STORMS: WHAT'S THE JOINT PROBABILITY CONNECTION

Continuous simulation of a rainfall-runoff process is fundamentally a Monte Carlo procedure for evaluating the joint probability interactions of random inputs to estimate probability distributions of desired outputs. By understanding the mechanics of joint probability the connection can be made between continuous simulation and the design storm approach. A simple example illustrates the main considerations.

Let q be the random variable corresponding to the peak discharge in a storm event, r be the random rainfall characteristics (such as average intensity), $\boldsymbol{\theta}$ be a vector of secondary random variables affecting the transformation of rainfall to runoff (such as temporal pattern and antecedent conditions) and p(r) be the probability density of the rainfall characteristic. Assume there is a deterministic relationship between the random inputs, r and $\boldsymbol{\theta}$, and the peak discharge q:

$$q = f(r,\boldsymbol{\theta}) \tag{1}$$

Typically Eqn (1) represents an event-based rainfall-runoff or a soil-moisture accounting model.

Let $p(q|\boldsymbol{\theta})$ be the probability density of q conditioned on some value of $\boldsymbol{\theta}$. Assuming a unique relationship between q and r for a given $\boldsymbol{\theta}$, it can be shown using derived distribution theory that

$$p(q\,|\,\boldsymbol{\theta}) = \left|\frac{\partial r}{\partial q}\right| p(r)$$

Using the total probability theorem yields the probability density of q

$$p(q) = \int p(q\,|\,\boldsymbol{\theta})\, p(\boldsymbol{\theta})\, d\boldsymbol{\theta} \tag{2}$$

where $p(\boldsymbol{\theta})$ is the joint probability density of $\boldsymbol{\theta}$. Continuous simulation evaluates this integral implicitly by generating samples from p(q).

Eqn (2) on its own does not offer much insight. However suppose $\boldsymbol{\theta}$ is partitioned into two components, $\boldsymbol{\theta}_1$ and $\boldsymbol{\theta}_2$, with $\boldsymbol{\theta}_1$ selected so that $p(q|\,\boldsymbol{\theta}_1, \boldsymbol{\theta}_2)$ varies linearly over the range of $\boldsymbol{\theta}_1$ values which have significant probability. It follows using a Taylor series that

$$
\begin{aligned}
p(q \mid \boldsymbol{\theta}_2) &\approx \int \left(p(q \mid \hat{\boldsymbol{\theta}}_1, \boldsymbol{\theta}_2) + \left.\frac{\partial p}{\partial \boldsymbol{\theta}_1}\right|_{\hat{\boldsymbol{\theta}}_1} (\boldsymbol{\theta}_1 - \hat{\boldsymbol{\theta}}_1) \right) p(\boldsymbol{\theta}_1 \mid \boldsymbol{\theta}_2)\, d\boldsymbol{\theta}_1 \\
&= p(q \mid \hat{\boldsymbol{\theta}}_1, \boldsymbol{\theta}_2) + \left.\frac{\partial p}{\partial \boldsymbol{\theta}_1}\right|_{\hat{\boldsymbol{\theta}}_1} \int (\boldsymbol{\theta}_1 - \hat{\boldsymbol{\theta}}_1)\, p(\boldsymbol{\theta}_1 \mid \boldsymbol{\theta}_2)\, d\boldsymbol{\theta}_1 \qquad (3) \\
&= p(q \mid \hat{\boldsymbol{\theta}}_1, \boldsymbol{\theta}_2)
\end{aligned}
$$

where $\hat{\boldsymbol{\theta}}_1$ is the average or expected value of $\boldsymbol{\theta}_1$.

Two noteworthy insights follow from Eqn (3):

1. If the flood probability response is approximately linear to random variations in $\boldsymbol{\theta}_1$, then the conditional probability density of q given $\boldsymbol{\theta}_2$ can be obtained by using average values of $\boldsymbol{\theta}_1$.

   Stochastic rainfall-runoff models are complex and data-hungry. Eqn (3) suggests that for the purpose of simulating p(q) it suffices that the stochastic model only simulate secondary random variables whose variation produces a nonlinear response in p(q). All other secondary random variables can be replaced by their expected values.

   It therefore makes sense to study the linearity of the response p(q) to various input variables with the aim of simplifying the complexity of the joint probability analysis.

2. If we incorrectly estimate the probability distribution of $\boldsymbol{\theta}$ and, in particular, its expected value, then the inferred density p(q) will be biased. One would, therefore, be wary of subjectively assigned input probability distributions p($\boldsymbol{\theta}$), particularly if the discharge q is sensitive to variations in the input $\boldsymbol{\theta}$.

   The bias due to misspecification of the input probability distribution arises even if the correct model structure is being used. If model structure is incorrect then, of course, the bias would change.

ARR makes the key assumption that apart from average intensity the total probability contribution of all other random variables affecting peak runoff can be represented by their medians – in fact Eqn (3) makes clear that one should use averages not medians. To the authors' knowledge this assumption remains largely untested. Eqn (3) offers a framework for testing the fundamental assumption underpinning the ARR design storm approach.

Regardless of the soundness of the ARR assumption about the use of average values, there remain concerns about the use of unrepresentative average parameters. In particular, the initial losses recommended in ARR appear to be biased towards antecedent conditions with above-average wetness. The work of Walsh et al. (1991) gives credence to these concerns.

3. COMPARISON OF CONTINUOUS SIMULATION AND DESIGN STORM FLOOD ESTIMATION FOR VOLUME-SENSITIVE SYSTEMS

Two case studies are considered in which flood probabilities are estimated using continuous simulation approach and the ARR design storm approach. The first case study deals with the estimation of peak outflow from a detention basin, while the second deals with estimation of peak flows from an urban allotment with a rainwater tank.

In both case studies the methodology employed is the same. A continuous simulation model of the reference system is used to simulate a very long flow record – this record will constitute the "truth". Following the method developed by Walsh et al. (1991), antecedent conditions in the design storm approach will be calibrated for the reference system so that the peak flow predicted by the design storm approach equals the "true" flow for a given ARI (average recurrence interval). The system is then modified by inclusion of volume-sensitive structures. The predictions of peak flows from the modified system will then be compared using continuous and design storm simulations.

3.1 Estimation of Peak Outflow From a Detention Basin

The objective of the first case study is to assess the accuracy of the design storm approach to simulate peak outflows from a detention basin. A detention basin uses temporary storage to attenuate and delay the flood peak. Basin behaviour is sensitive to both inflow peak and volume as well as antecedent conditons.

3.1.1 Description of continuous simulation model

Figure 1 presents a schematic of the continuous simulation model. The reference system

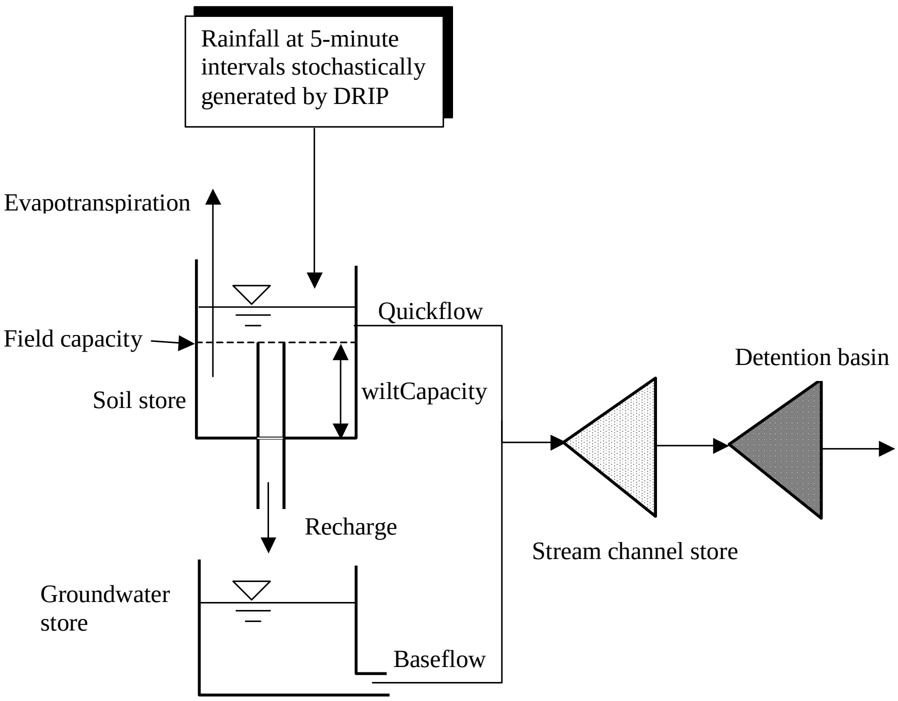

Figure 1. Schematic of continuous simulation model using DRIP and AWBM.

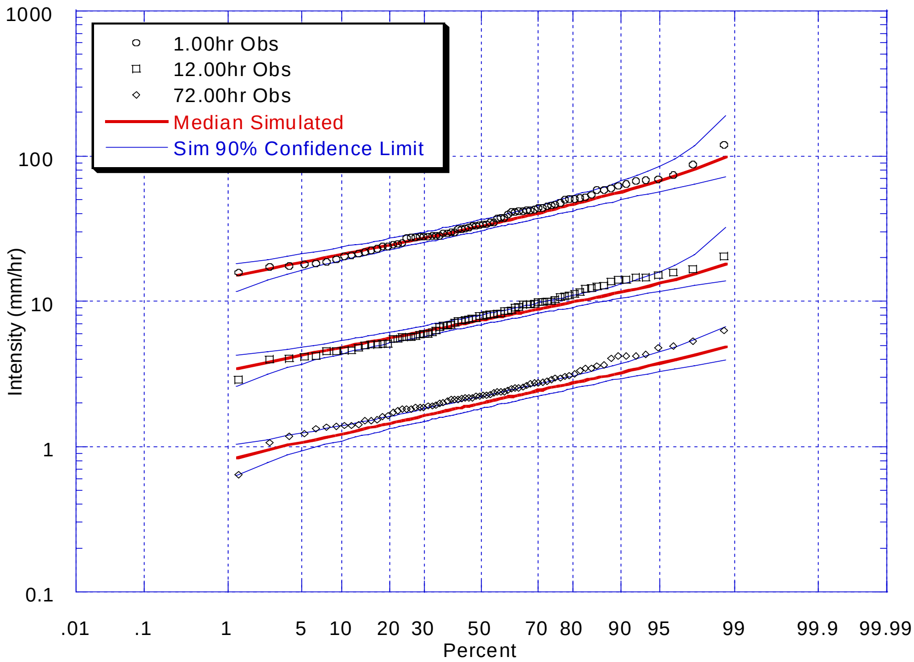

Figure 2. Sydney Observatory Hill versus DRIP simulated intensity-frequency-duration curves

consists of a natural catchment with area 10 km^2. The central feature of a continuous simulation model is the facility to simulate long climate sequences. In this study a long rainfall record at 5-minute resolution is generated by the stochastic point rainfall model DRIP [Heneker et al., 2001] calibrated to the pluviograph record at Sydney Observatory Hill. The calibration focussed on reproducing the probability distributions of storm dry spells, durations and average intensities for each month along with the distribution of temporal patterns. The DRIP calibration does not pay special attention to extreme storms. Accordingly, the comparison of the observed and DRIP simulated 1, 12 and 72-hout intensity-frequency-duration (IFD) curves in Figure 2 provides an independent check on the ability of DRIP to simulate extreme rainfalls. For all three durations the observed data lie within the 90% confidence limits indicating that the DRIP simulation is not inconsistent with the data at Sydney Observatory Hill. It needs to be stressed that because DRIP was not calibrated to reproduce observed IFD curves one would not expect the median DRIP simulation to pass through the observed IFD data. If anything DRIP slightly underestimates the observed IFD curves, the significance of which will become apparent shortly.

The rainfall is routed through a version of the AWBM model, originally developed by Boughton [1996], to produce a long-term sequence of 5-minute streamflows. The AWBM used in this study has three stores: Soil, groundwater and stream channel stores. When the soil store is above field capacity it produces quickflow and recharge as well as evapotranspiration. Although the AWBM allows for multiple soil stores with different wilting capacities to simulate partial area quickflow mechanisms, only one store was used in this study. The groundwater and stream channel stores are modelled as linear reservoirs.

The detention basin had a constant surface area of 0.1 km^2 (which is 1% of the catchment area). Outflow from the basin is over a 5-m long broad-crested weir. The

Table 1. AWBM parameters used in reference continuous simulation.

Parameter	Description	Value
wiltCap	Soil store depth below field capacity at which ET stops	100 or 20 mm
drainCap	Temporary storage depth for quickflow	20 or 0 mm
bfFrac	Fraction of groundwater discharging as baseflow	0.001 1/day
V	Stream velocity	1.5 m/s
F	Maximum recharge rate	5 mm/day

Table 2. Continuous simulation results.

AWBM parameters mm		2-year peak flow m^3/s		50-year peak flow m^3/s	
wiltCap	drainCap	No basin	With basin	No basin	With basin
100	20	73.8	38.6	200.2	105.5
20	0	100.6	47.1	234.4	117.7

invert of the weir coincides the lowest level of the basin implying the basin will not permanently retain any water.

The reference system consists of a 10 km^2 catchment with rainfall characteristic of Sydney Observatory Hill and AWBM parameters listed in Table 1. No detention basin is present in the reference system.

Twenty rainfall replications of length 1000 years and 5-minute resolution were routed through the AWBM model. The hydrographs associated with the complete storm that produced a peak flow equivalent to the 2- and 50-year ARI were extracted from the simulation – note that in DRIP dry spells of up to 2 hours may occur within the storm. Figure 3 presents the 20 hydrographs whose peak flow equalled the 2-year discharge. What is striking about the hydrographs is their differences. Marked differences arise in shape (one, two and three-peaked hydrographs), duration and volume (ranging from 41 to 223 mm).

It is not uncommon to hear reference made to say the 2-year hydrograph. One is not sure whether such a reference is made to the hydrograph having an ARI of 2 years or to a hydrograph with a peak flow equal to the 2-year discharge. The former interpretation is incorrect. Figure 3 makes absolutely clear that there is no such thing a 2-year hydrograph. It is only meaningful to refer to the peak of the hydrograph as having a 2-year ARI. The significance of the remainder of the hydrograph with regard to risk assessment is moot.

Table 2 presents the 2- and 50-year peak flows derived from continuous simulation with and without the detention basin. Simulations were performed for two sets of AWBM soil store parameters: The first set represented a catchment with moderate natural storage capacity, while the second represented a catchment with little storage capacity.

3.1.2 Calibration of design storm approach to reference system peak flows

Figure 3 reveals that for many of the hydrographs there is considerable runoff activity before the storm burst that produces the peak discharge. It is not clear what initial volumes should be assigned to the system at the commencement of the design storm.

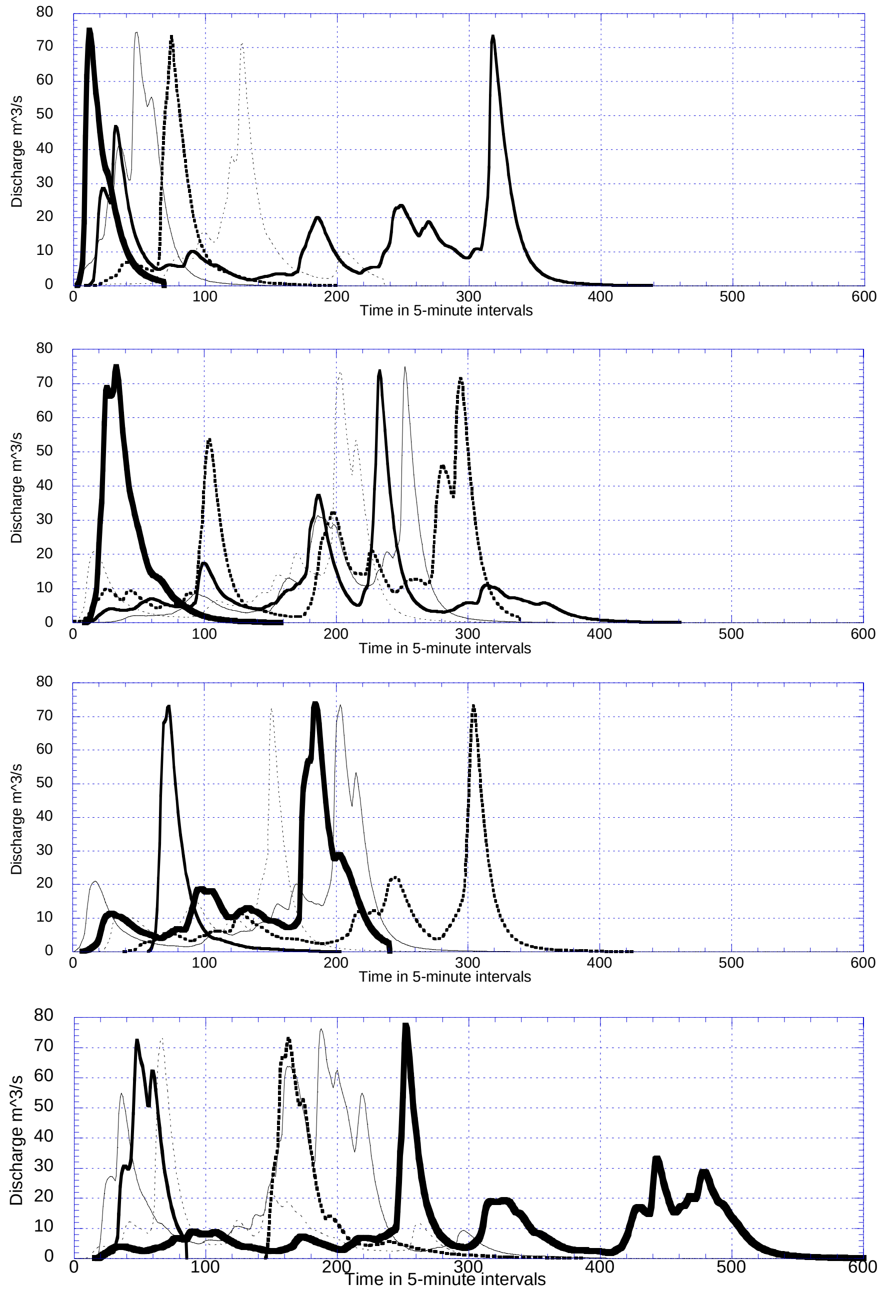

Figure 3. Twenty hydrographs with peak flow from the catchment equal to 2-year discharge.

Walsh et al. (1991) suggested that the accuracy of the design storm approach can be improved by forcing it to be consistent with discharges of known ARI. Specifically Walsh et al. performed a flood frequency analysis of gauged data, calibrated a rainfall-runoff model and then using design storms with a Y-year ARI varied the initial loss until the rainfall-runoff model produced a peak flow equal to the Y-year flood frequency peak. Walsh et al. observed that this procedure produced considerable scatter in initial loss from one catchment to another. Nonetheless, Walsh el al's approach has the apparent virtue of removing bias from the design storm approach.

We adopted Walsh et al's approach in our case study.. The continuous simulation yielded the "truth", namely the flood frequency curve. Using design storms with a Y-year ARI we searched for the initial soil store depth so that the critical storm yielded a peak flow equal to the Y-year ARI peak flow derived from the flood frequency curve. The design storms were based on the IFD data for Sydney Observatory Hill, shown in Figure 1.

Table 3 presents the calibration results. Somewhat surprisingly, the initial soil store values necessary for the design storms to reproduce the "true" peak flow from the catchment without a detention basin were above field capacity. Perusal of Figure 3 suggests why this may be the case. For most replicates, there was considerable rainfall and runoff activity prior to the main storm burst. As a result, on average, there was considerable flow at the beginning of the design storm burst. The only way the AWBM model can achieve such an outcome is by having the initial soil depth above field capacity to produce substantial quickflow from the outset.

3.1.3 Performance of detention basin

Using the calibrated initial soil store depths, design storms were routed through the catchment and detention basin. The basin was assumed to be empty at the start of the storm. Table 3 reports the peak flow from the detention basin and the relative error defined as [simulated peak – true peak]/true peak. The design storm peak flow estimates consistently underestimated the "true" peak flow by 15 to 24%. The most likely explanation is that the runoff activity leading up to the main storm burst had already significantly elevated the basin water level above the crest of the spillway.

The assumption that the detention basin was empty at the start of the design storm proved to be inadequate. Unfortunately there appears to be no rational way of assigning an initial water level – it depends on storm activity prior to the main burst.

Table 3 presents the design storm approach in the most favourable light by using calibrated initial losses. Table 4 illustrates the sensitivity of the design storm results to variations in initial soil store depth. The reduction of the initial soil store depth to field capacity and a 30 mm deficit produces very considerable underpredictions of the detention basin peak outflow.

Table 3. Calibrated initial soil store depths and design storm peak flows from detention basin assumed empty at start of design storm.

AWBM parameters		2-year design storms			50-year design storms		
wiltCap mm	drainCap mm	Calibrated initial soil depth mm	Peak flow from basin m^3/s	Relative error, %	Calibrated initial soil depth mm	Peak flow from basin m^3/s	Relative error, %
100	20	11.1	32.9	-14.7	29.2	80.0	-24.2
20	0	7.1	37.8	-19.7	24.8	93.1	-20.9

Table 4. Sensitivity of design storm peak flow to variation in initial soil store depth.

AWBM parameters		2-year design storms			50-year design storms		
wiltCap mm	drainCap mm	Initial soil depth mm	Peak flow from basin m^3/s	Relative error, %	Initial soil depth mm	Peak flow from basin m^3/s	Relative error, %
100	20	11.1	32.9	-14.7	29.2	80.0	-24.2
		0	27.3	-29.3	0	69.9	-33.7
		-30	15.4	-60.1	-30	53.8	-49.0

3.2 Allotment Analysis

The objective of the second case study is to assess the accuracy of the design storm approach at a much smaller scale than the first case study, namely the urban allotment. Our focus is the simulation of peak outflows from an urban allotment using a rainwater tank to provide storage for domestic use and detention storage for roof runoff.

3.2.1 PURRS continuous simulation model

The reference system for the allotment analysis is an allotment on a clay soil with an area of 600 m^2, a house with a roof area of 220 m^2 and a paved area of 80 m^2. The performance of the reference allotment system was modelled using the DRIP rainfall for Observatory Hill in the PURRS (Probabilistic Urban Rainwater and wastewater Reuse Simulator) model [Coombes and Kuczera, 2001; Coombes, 2002]. A schematic of PURRS is shown in Figure 4.

Twenty rainfall replications of length 1000 years and 2-minute resolution were routed through the reference system using the PURRS model. The hydrographs from the complete storm that produced a peak flow equivalent to the 2- year ARI were extracted from the simulation. Figure 5 presents the 20 hydrographs whose peak discharges equalled the 2-year discharge. Similar to the first case study the hydrographs are shown to have considerable differences in shape (one, two and three-peaked hydrographs), duration and volume (ranging from 39 to 217 mm).

The allotment reference system was modified with the introduction of a 15 kL rainwater tank that had reserved 3.75 m^3 airspace for detention. The rainwater tank is used to supply hot water, toilet and outdoor uses for a four-person household residing in the house on the allotment.

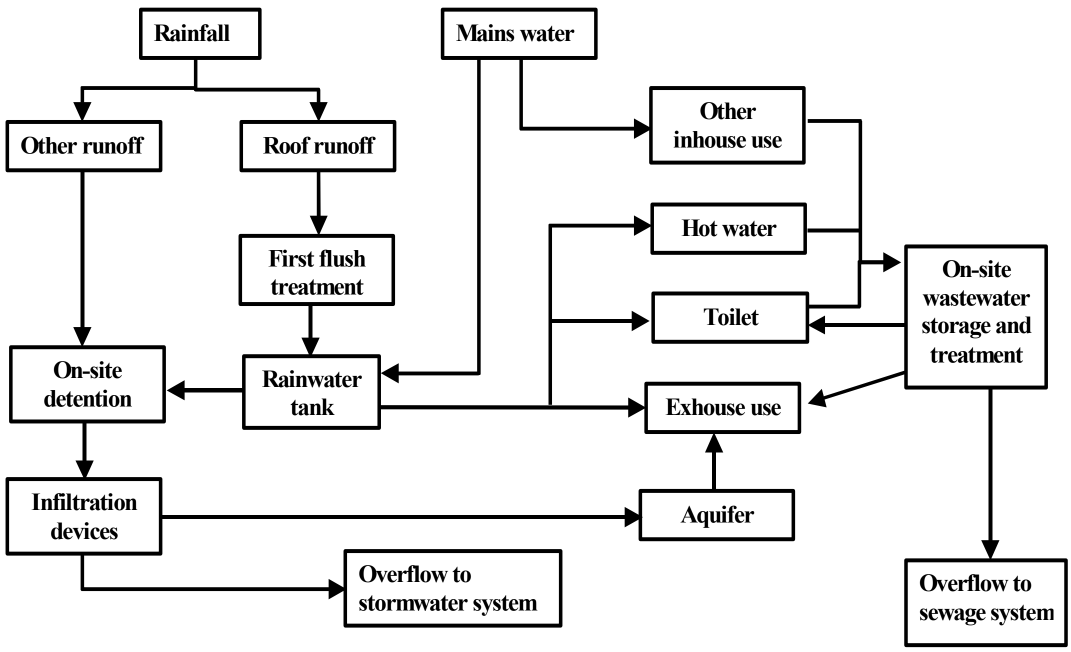

Figure 4. Schematic of the PURRS model

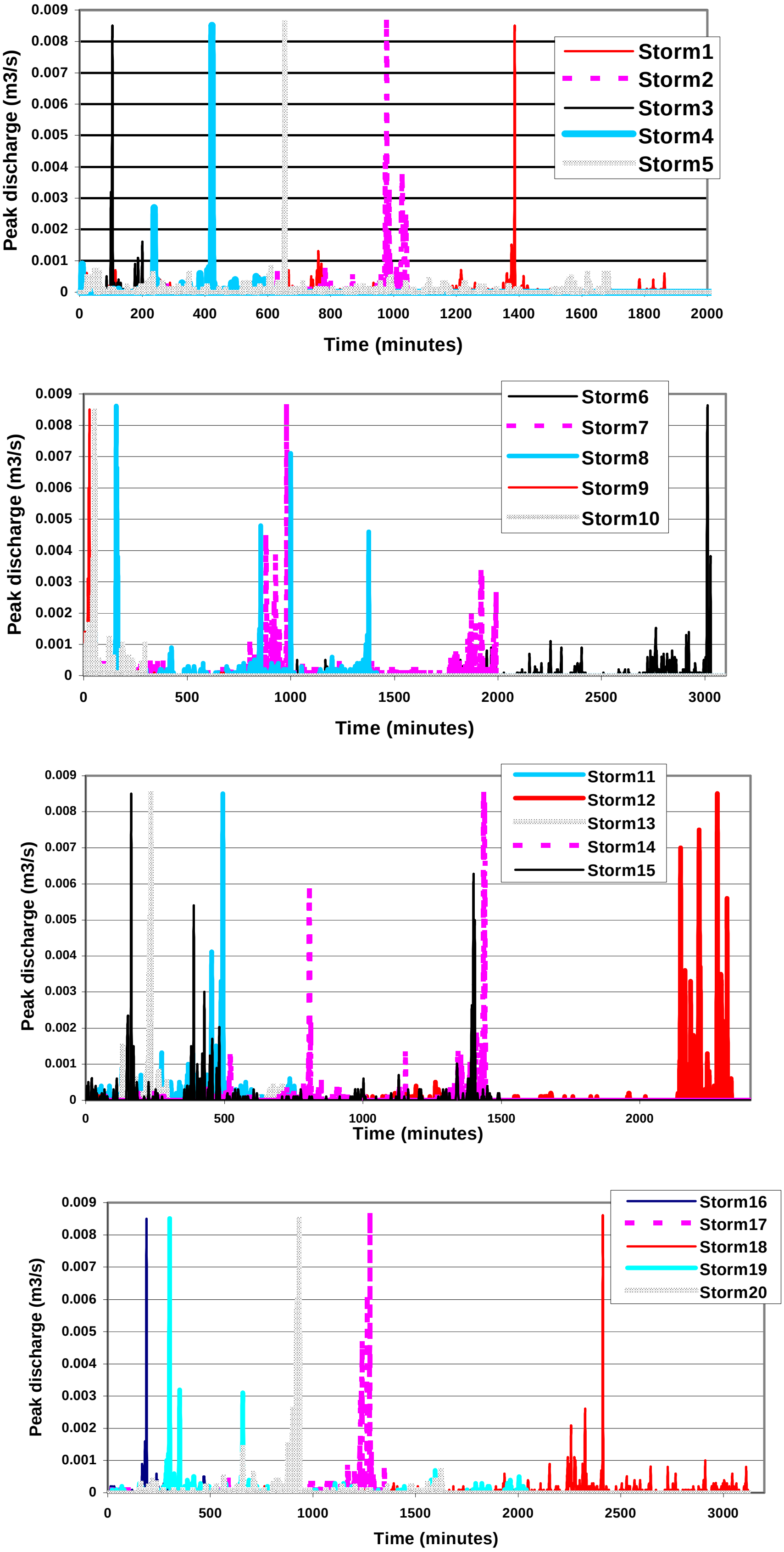

Figure 5. Twenty hydrographs with peak flow from the allotment equal to 2-year discharge.

Figure 6 illustrates the three zones of the rainwater tank. The bottom zone stores water to provide a buffer against peak irrigation-based demand; whenever the water level falls

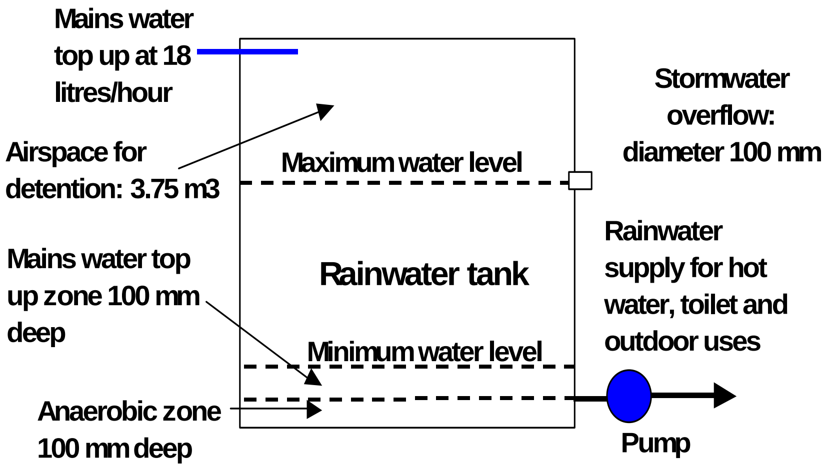

Figure 6. Configuration of the rainwater tank

Table 5. Continuous simulation results for the allotment

ARI years	IDS mm	PDS mm	Peak flow, m^3/s	
			No tank	With tank
2	1	5	0.0085	0.0065
50	1	5	0.0257	0.022

into this zone mains water top up at a rate of 18 Litre/hr occurs until the zone is filled. The middle zone provides storage for roof runoff to be used for domestic use. The top zone provides detention storage during storm events that fill the second zone. The performance of rainwater tanks during a particular storm event is dependent on the water level or airspace at the start of the storm.

Table 5 presents the 2- and 50-year peak discharges from the allotment derived from continuous simulation with and without the rainwater tank. In addition the Table reports the impervious area depression storage (IDS) and pervious area depression storage (PDS) capacities used in the continuous simulation.

3.2.2 Calibration of design storm approach to reference system peak flows

The design storm approach was calibrated to the 2- and 50-year peak flows (reported in Table 5) for the reference system, namely an allotment without rainwater tank. Because PURRS does not support design storms, the WUFS model [Kuczera et al., 2000] was used to simulate the response of the reference system to design storm inputs.

Initially the calibration was performed by assuming the impervious and pervious depression stores were empty and adjusting the antecedent moisture condition (AMC) so that the critical design storm produced a peak flow close to the no-tank flow reported in Table 5. The AMC is a measure of soil wetness at the start of the design storm with 1 representing a soil in a dry condition and 4 representing a soil in wet condition. If it was impossible to match the no-tank peak flow using AMC alone then the initial depression storage values IDS and PDS were adjusted.

Table 6 reports the results of the calibration. In the case of the 2-year design storm adjustment of AMC proved sufficient. However, in the 50-year case, even when the soil was wet (AMC equal to 4) there remained too much antecedent depression storage to match the peak flow. Accordingly, the IDS and PDS values were lowered until a match

Table 6. Design storm results.

Calibration results for the allotment without a tank				Allotment with a 15 kL rainwater tank: Initial tank volume 7.89 m^3	
ARI years	Initial IDS, mm	Initial PDS, mm	AMC	Peak flow m^3/s	Relative error %
2	1	5	1.1	0.004	-31
50	0	0	4	0.023	+4

was obtained. It is noted that the adjustment process is arbitrary because there are many combinations of AMC and initial depression storage that could achieve the same outcomes. This indeterminacy is fundamental to the design storm approach.

3.2.3 Performance of rainwater tank

Using the calibrated AMC and depression storage values, design storms were routed through the allotment with a 15 kL rainwater tank. The choice of initial volume in the tank prior to the start of the design storm is problematic. We used the value of 7.89 m^3 which represents the expected volume in the tank prior to commencement of a storm as derived by continuous simulation.

Table 6 reports with peak flows simulated by the 2 and 50-year critical design storms along with their relative errors. Of significance is the considerable jump in relative error from –31% to 4%. One could argue that the initial tank volume should coincide with 11.25 m^3, which is the volume at which spills commence. The logic is that design storms are bursts embedded in longer storms. The pre-burst activity would fill the tank close to spilling. In such a case the relative error for the 2-year peak flow would be closer to zero at the expense of substantially increased relative error for the 50-year peak. It appears that there is no simple logic to deal with what is a complex joint probability interaction – it is noted that the tank is being continually drawndown by domestic use and periodically refilled with roof runoff.

Table 7 illustrates the sensitivity of the design storm peak flows to variations in the initial conditions. The considerable sensitivity of relative errors to both initial volume in the rainwater tank and AMC highlights the fundamental difficulty of applying the design storm approach to volume-sensitive systems.

4. CONCLUDING REMARKS

Though one needs to be wary about generalizing the results of the two case studies it is nonetheless evident that there are likely to be serious estimation errors associated with the use of design storms to evaluate the performance of volume-sensitive systems. Both case studies demonstrated that even if the design storm approach uses calibrated initial conditions, the assignment of initial volumes in detention/retention systems remains problematic and can induce considerable errors in peak flow estimation. Moreover, both case studies highlight the considerable sensitivity of peak flows to variations in initial conditions.

Given the widespread usage of detention and retention systems in urban stormwater management one has to seriously question the fundamental assumptions that underpin Australian flood estimation practice. A disturbing aspect of this study is that there does

Table 7. Sensitivity analysis for variation in initial conditions

ARI years	Initial IDS, mm	Initial PDS, mm	Initial tank volume, m^3	AMC	Peak flow m^3/s	Relative error , %
2	1	5	11.25	1	0.008	+23
2	1	5	11.25	4	0.012	+85
2	1	5	5	1	0.002	-69
2	1	5	5	4	0.006	-8
50	1	5	11.25	1	0.018	-18
50	1	5	11.25	4	0.023	+4
50	1	5	5	1	0.016	-27
50	1	5	5	4	0.019	-14

not appear to be an obvious way to “fudge” the design storm approach to make it reliable.

The joint probability approach reported by Rahman et al. (2002) represents a major conceptual advance on the design storm approach. Nonetheless, it requires identification of probability distributions of all antecedent conditions. In the case of volume-sensitive detention/retention systems such distributions would be required for all or most of the storages in the system. One is inexorably drawn to the conclusion that continuous simulation linked with stochastic rainfall models offers the best prospect of rationally dealing with the complex joint probability problem that we call flood estimation.

5. REFERENCES

Boughton, W., AWBM catchment water balance model – Version 2, CRC for Catchment Hydrology, 1996.

Coombes P.J., and Kuczera G., (2001). Rainwater tank design for water supply and stormwater management. Stormwater Industry Association 2001 Regional Conference, Port Stephens, NSW.

Coombes P.J., (2002). Rainwater tanks revisited: new opportunities for urban water cycle management. Unpublished PhD. thesis. University of Newcastle, Callaghan, NSW.

Frost, A., Jennings, S., Thyer, M., Lambert, M.F. and Kuczera, G., Droughts, Floods and Everything Else In Between, 3rd Int’l Hydrology and Water Resources Sym., Inst. Eng. Aust., Perth, 2000.

Heneker, T.M., Lambert, M.F. and Kuczera, G., A point rainfall model for risk-based design, Journal of Hydrology, 247(1-2), 54-71, 2001.

Institution of Engineers, Australia, Australian Rainfall and Runoff: A Guide to Flood Estimation, Vol. 1, D.H. Pilgrim (ed), Canberra, 1987.

Jennings, S., Lambert, M.L., Frost, A. and Kuczera, G., Regionalisation of a High Resolution Point Rainfall Model, 27th Hydrology and Water Resources Sym, Inst. Eng. Aust., Melbourne, 2002.

Kuczera, G., Williams, B.J., Binning, P. and Lambert, M.L., J., An education web site for free water engineering software, 3rd Int’l Hydrology and Water Resources Sym., Inst. Eng. Aust., Perth, 2000.

Rahman, A., Weinmann, P.E., Hoang, T.M.T and Laurenson, E.M., Monte Carlo simulation of flood frequency curves, *J. Hydrology*, 256(3-4), 1976-210, 2002.

Walsh, M.A., Pilgrim, D.H. and Cordery, I., Initial losses for design flood estimation in New South Wales, Hydrology and Water Resources Sym., Inst. Eng. Aust., Perth, p. 283-288, 1991.

Weinmann, P.E., Rahman, A., Hoang, T.M.T., Laurenson, E.M. and Nathan, R.J., Monte Carlo simulation of flood frequency curves from rainfall – The way ahead, 3rd Int'l Hydrology and Water Resources Sym., Inst. Eng. Aust., Perth, p. 564-569, 2000.