University of Technology Sydney

COUPLED ONE and TWO-DIMENSIONAL MODELING in URBAN CATCHMENTS – REDUCING UNCERTAINTY in FLOOD ESTIMATION

Author - -Stephen Don Gray

Supervisor – Assoc. Prof. James Ball

Certificate of Authorship

I certify that the work in this thesis has not previously been submitted for a degree, nor has it been submitted as part of requirements for another degree.

I also certify that the thesis has been written by me. Any help that I have received in my research work and the preparation of this thesis itself has been acknowledged. In addition, I certify that all information sources and literature used are indicated in the thesis.

Production Note:
Signature removed prior to publication.

Stephen Gray B. E.(Res Eng)

ACKNOWLEDGEMENTS

ACT Electricity and Water

Australian Capital Territory Government

Allen Goyen is acknowledged for his PhD dissertation and the data he collected and so readily offered to share which facilitated this study

Supervisor Assoc Prof James Ball

Pavel Kozarovski, Grantley Smith and Robert Carr for ideas relevant to the genesis of this study

Strathfield City Council

Burwood City Council

Richard Dewar

DHI Water and Environment

University of New South Wales

University of Technology Sydney

The Australian Government

TABLE OF CONTENTS

List of Figures

List of Tables

ABSTRACT

COUPLED ONE and TWO-DIMENSIONAL MODELING in URBAN CATCHMENTS – REDUCING UNCERTAINTY in FLOOD ESTIMATION

KEY WORDS: Urban Stormwater, spatial, coupled, two-dimensional, rainfall, runoff, models, hydrology, hydraulics, conceptual, distributed, calibration, validation, pipe, design, drainage

ABSTRACT: A recent trend in urban stormwater modeling projects is the application of coupled one and two dimensional models whereby a two dimensional model routes rainfall excess overland and interfaces with a one dimensional representation of a pipe drainage system.

Two principle advantages are sought in utilising the 1d/2d model. These are:

- 2d routing of surface flow means that flow paths do not need to be known prior to model run; and
- The 2d surface flow model can replace conceptualised lumped hydrology with a physical process based distributed approach.

Numerous studies have been carried out which demonstrate the 1d/2d methodology. Few however have been able to demonstrate model performance against gauged data. Also few such applications have separated out hydrological response from different areas in the urban catchment, such as road, roof and yard response.

This study aims to test the 1d/2d coupled modeling approach on a data set which includes numerous gauged events which separate out three main hydrological processes: roof, road and yard runoff. The data set was compiled as part of PhD dissertation work undertaken by Goyen (2000) for a catchment in the A.C.T, Australia.

It is found that the 1d/2d model system examined, given specific inclusions in the methodology, does demonstrate an ability to reproduce gauged flows extremely well without need for variation of model parameters other than proportional loses applied.

Chapter 1: Introduction

Background to Urban Stormwater Modeling

In order to control surface runoff resulting from rainfall events, drainage schemes are often designed and implemented in conjunction with development. Typically a model system that utilizes both hydrologic and hydraulic models (stormwater model) will be used to examine the proposed drainage networks performance relative to a design standard and results from this will feedback to design.

Stormwater modeling may also be carried out for an existing drainage network. In this case the objective may be to define the existing flood hazard for safety reasons, to identify areas poorly drained in order to undertake mitigation works or to examine how flooding in the area has been changed by development in the catchment since the drainage system was originally designed. An example of this is a suburban catchment that is becoming increasingly impervious as urban density rates increase (the quarter acre block becomes a duplex for example). Council may request a modeling study to be carried out in order to assess how increasing levels of catchment imperviousness will impact flooding.

The generic components of a standard coupled stormwater model are shown in Figure 1. Note that in some instances where pipe capacity only is being examined, the surface flow model may be excluded however in this case the magnitude of the event that may be modeled is necessarily limited to those near pipe flow capacity. Also typically in such a stormwater modeling system the hydrological model is lumped[1] and conceptual[2], whilst the hydraulic

---

[1] "lumped" means that it covers a wider area which may or may not be homogenous in landuse or features and then develops a flow hydrograph at the outlet. See Figure 2 and Figure 4 for a graphical description of lumped and distributed catchment modeling.

[2] "conceptual" meaning that no directly measureable relationship exists between model parameters and physical features of the modeled system. It will be argued in Chapter 2 that conceptual models (also called black box and

models for surface and pipe flow are physical process based[3] and 1d/quasi-2d[4]. All of the models applied herein are deterministic[5]. In this approach the hydrological model is used to develop sub-catchment hydrographs which are then applied to the pipe model. If applied flow is in excess of pit inlet or pipe flow capacity, surcharge occurs and water enters the surface flow model.

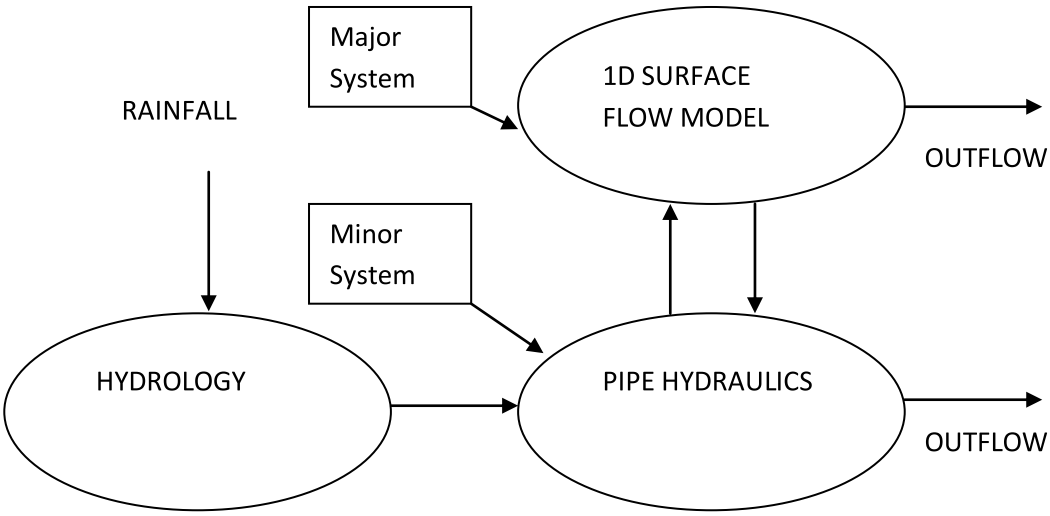

Figure 1: System Components in Typical Stormwater Model[6]

---

transformational models) rely more heavily on calibration due to the lack of physicality of their parameters than do physically based models.

[3] "Physical process" is a description taken from a paper by O'Loughlin et al (1996) and it is used to describe models which are based on physical equations.

[4] "1d/quasi 2d" simply implies that if the user chooses to, the 1d model can be schematised such that flow can move in multiple directions from a particular point and is not always limited to follow one direction. A 2d model differs in that no schematization is required to achieve this effect. If hydraulic characteristics allow (conveyance, slope, roughness), the flow can inherently move in one of four orthogonal directions.

[5] "deterministic" means that with the same input you will achieve the same output each time, i.e. not stochastic.

[6] The surface flow component is alternatively described as the "Major" drainage system. Similarly the subsurface flow system may be called the "Minor" system.

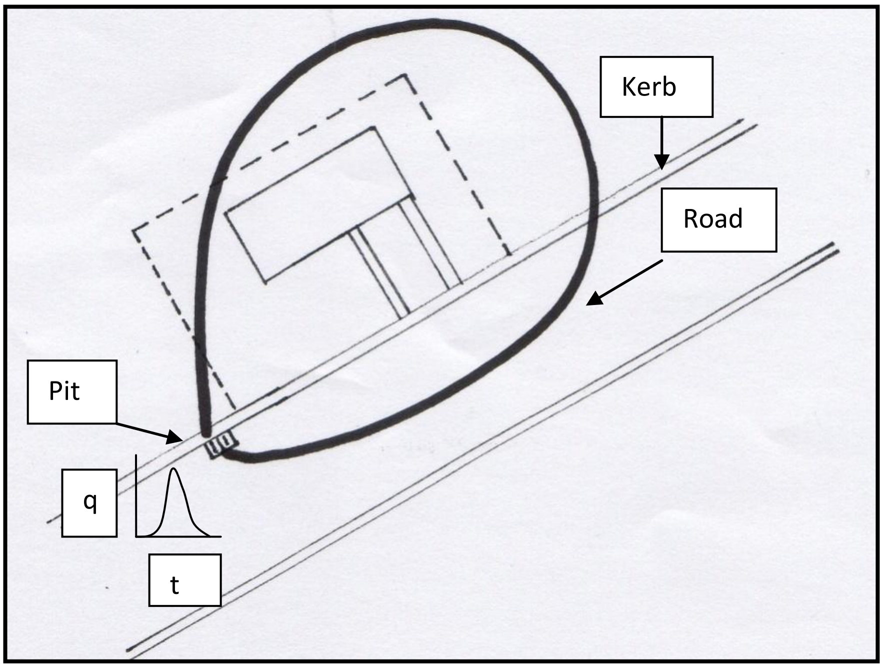

Figure 2: Lumped Sub-catchment example

Figure 2 is a demonstration of the typical hydrological model utilised in the urban stormwater modeling system of Figure 1. It shows that a single lumped catchment routes rainfall excess over the entire sub-catchment area (outlined in thick black) to the downstream point to generate a single flow hydrograph at sub-catchment outlet. Thus various features of the housing lot are “lumped” together. These include the dual strip driveway, house roof, yard areas and elements of the kerb.

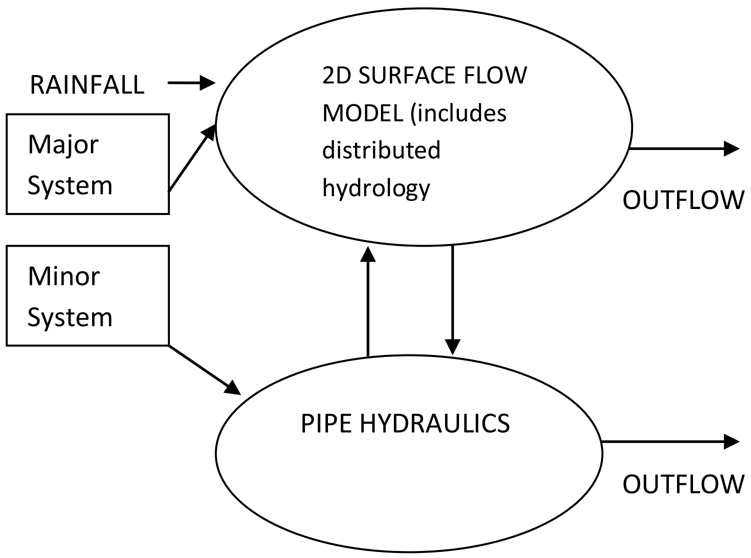

Figure 3: System Components in 1d/2d coupled Stormwater Model

Figure 3 describes the 1d/2d approach and fundamentally differs from Figure 1 in that rainfall is applied directly to the 2d surface flow model. This is not always the case and examples of 1d/2d applications where sub-catchment hydrographs have been applied to the 2d surface model will be shown in the next Chapter. Rainfall excess applied onto the 2d surface model is the form of application in this work however. Thus there is a distributed and physical process based model being used to produce pit inflow hydrographs. Note that rainfall excess is applied and so this implies that losses are subtracted from the rainfall prior to its application. As in the previous description of the modeling system, should the developed pit inflow hydrograph be in excess of pit inlet or pipe flow capacity, water will be routed and/or detained (depending on local topographic conditions) within the surface flow model.

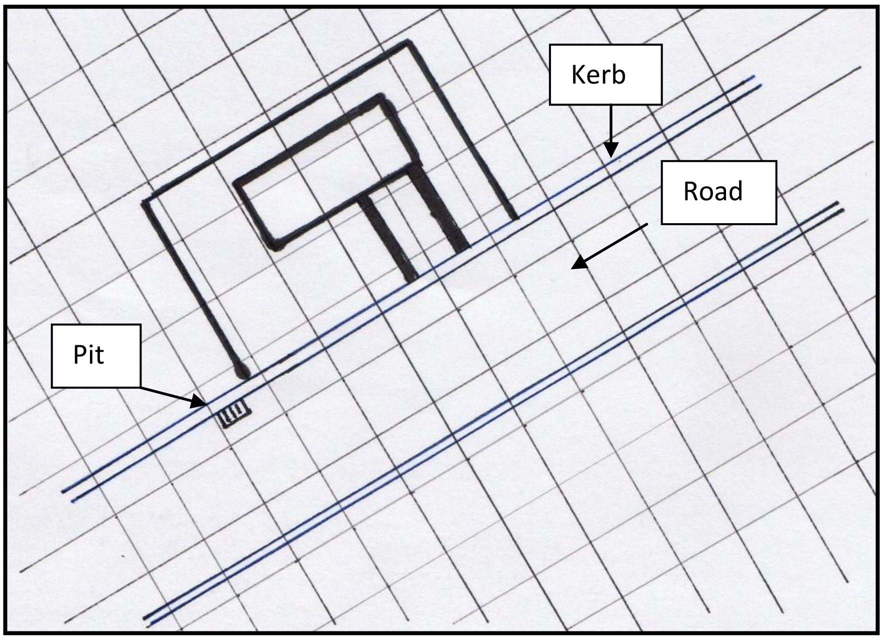

Figure 4: Distributed Sub-catchment example

In Figure 4 the surface flow model component of the urban stormwater modeling system shown in Figure 3 is depicted. No sub-catchment is discretised, instead the entire catchment is broken into an arrangement of grid cells. Rainfall excess is applied to these and then flow routes based on slope and roughness. Flow hydrographs are generated at each grid cell and if topography and roughness allows, flow may accumulate in the kerb cells and route to the pit inlet.

1d/2d models in urban stormwater modeling

Since the 1970's municipal drainage design manuals in The United States have called for the interaction between the minor and major drainage systems to be taken into consideration when designing urban drainage systems and events up to the 1 in 100 Year ARI event have been modeled. These requirements drove the development of coupled models (O'Loughlin et al 1996), the history of which will be covered in greater detail in Chapter 2.

Given the relatively recent ability of personal computer's to facilitate highly distributed physical process modeling of overland flows there is an emerging trend towards the utilization of coupled 1d and 2d models for stormwater modeling in projects.

In utilising the 2d component in the overall 1d/2d modeling system as proposed herein the two principle advantages sought are that:

- 2d routing based on a detailed topographic dataset means that flow paths are not required to be schematised prior to the event and that flow unable to enter a pit may flow downstream to become surface flow or enter another pit at which sufficient capacity does exist; and
- by applying rainfall excess to the 2d model utilising a diffusive wave description for overland flow, lumped conceptual routing or lumped physical process routing (kinematic wave for example) is replaced with highly distributed physical process routing. This avoids the need for estimation of routing parameters, a topic which will be covered in some detail in the literature review section of this work.

In the 1d/2d coupled urban stormwater models the relatively novel component is the inclusion of the 2d surface model, particularly in the application of rainfall excess to the 2d surface model.

It is envisaged that in situations where a model is applied to ungauged catchments and hence calibration is not possible, that by utilising distributed physical process based models for hydrograph generation, peak flow estimation error will be reduced.

This work does not provide conclusive evidence that this 1d/2d modeling does in fact reduce error for ungauged catchment peak flow estimation. To achieve this a "blind test[7]" would be required on an additional catchment or catchments. What it does aim to do however is discussed in the following section.

Research Goals

The goals of this work are to:

1. demonstrate the 1d/2d methodology on an unusually detailed and small scale dataset (particularly with respect to gauging);
2. establish or confirm significant components of methodology for the 1d/2d approach when applied to urban modeling; and
3. indicate whether or not it is likely that the 1d/2d methodology could in fact be used to reduce flow estimation error for ungauged catchments.

Background to Goal 1

Although the 1d/2d coupled modeling system is being used (in forms that do slightly vary) in research and in project work, few studies have compared results against a multi-event calibration data set. Additionally, although different units of hydrological response exist in urban catchments (such as roads, yards, roofs) very few applications have examined the ability of 1d/2d models to emulate the various hydrological response elements. Both of these

[7] In a blind test the model would be applied to a gauged catchment but flow estimation would be carried out without recourse to calibration/validation. Flow estimation results would then be compared to the gauged data.

absences in the research are due to a lack of suitable observed data and also due to the only recent ability to run such models in reasonable time frames.

Goyen (2000), in his PhD work, carried out gauging over a three year period (pipe flow and pluviometer gauging) for two adjacent micro-catchments within a larger urban catchment in the suburb of Giralang, in north Canberra ACT. The gauging work was carried out such that three individual areas of hydrologic response could be separated, these being roof, road and yard runoff. The availability of excellent model build data, including Lidar, makes the site an ideal one for the testing of the 1d/2d coupled modeling system.

This study utilizes Goyen's (2000) data and approach in splitting hydrological response into a triumvirate of roof, road and yard. This study also takes a more physical, less aggregated approach to modeling an urban stormwater system, as recommended by Goyen (2000) for improvement of urban storm water modeling accuracy.

Tests will consist of comparisons to gauged events (validation) of a single event calibrated model. As indicated, a further goal is to refine the modeling methodology by reference to previous studies findings as well as iterative optimization in this study.

Background to Goal 2

Being relatively new in project application, it is the case that guidelines on how 1d/2d models should be applied in practice are not readily available. In reviewing literature on the subject and demonstrating the modeling technique against a uniquely detailed data set, it is hoped that some guidelines for application can be established.

Background to Goal 3

If the use of distributed physical process models can improve flow estimation accuracy in the absence of available calibration data then this could be a real boon, as most urban areas lack the gauged data required for the calibration/validation process. As will be discussed in Chapter 2, some practitioners have found that in applying the 1d/2d modeling system, calibration requirement is lessened.

Thesis Layout

The layout of this dissertation is as follows.

Chapter 2 provides a historical context for the work described herein by discussing how urban stormwater modeling has developed since its inception in 1850 when Mulvaney (Mulvaney, 1850) presented his Rational Method model which linked rainfall and specific catchment attributes to anticipated flow. The generation of sub-catchment hydrographs is focussed on, particularly routing techniques. Inadequacies in current methodology are also discussed. The brief history of coupled 1d/2d models in urban stormwater modeling is then addressed and a summary of findings from the literature review with respect to methodology and expectations is made.

Chapter 3 provides details of the site and data, based on which the modeling reported herein has been carried out. Note that the site was developed by Goyen (2000) in that it was he, with the assistance of others, who developed the in-pipe gauging of discharge as well as located high resolution pluviometers which have in turn made this study possible. A great credit is owed to Goyen (2000) and those that worked with him in order to collect the data, some part of which is utilised in this study. Also presented are the events that were selected for calibration and validation of the modeling system applied herein.

Chapter 4 defines the methodology employed. The models utilised are discussed as are methods of preparing data for input utilising GIS manipulation of raster data. Model build including the coupling of the 1d and 2d elements is discussed

Chapter 5 presents model results for calibration and validation runs. Plots are provided as are statistics which describe the fit between modeled and observed data sets.

Chapter 6 discusses the model results and seeks to identify where strengths and weaknesses of the approach utilised occur. In the discussion further research opportunities are identified.

Chapter 7 presents the conclusions of the research presented herein and ties these back to the original problem as articulated in Chapter 1 – Research Goals.

Chapter 2: Modeling Background

Historic Context and Development of the 1d/2d coupled Modeling System

Introduction

The recent identification of 1d/2d modeling systems as the new benchmark (Phillips et al, 2005) in urban modeling is partly the product of necessity i.e. the New South Wales Floodplain Development Manual made overland flow modeling a requirement of all urban storm water studies (DNR, 2005) and partly the product of possibility. Spatial data sets, Lidar data and computational power make possible the use of two-dimensional surface flow models at resolutions suitable for urban modeling.

In the past less spatial data has been available and computational capacity was not adequate for high resolution two-dimensional surface flow modeling of urban areas. There have however, since the 1980's at least, been attempts to couple 1d and 2d models and utilize their respective advantages in order to model specific scenarios (Smith, 2006). That is, coupled 1d/2d modeling is not particularly new in concept or in application (Smith, 2006).

This section aims to provide details of the evolution of the development of 1d/2d coupled models. From the early beginnings in the late 1980's where coarse grids were used with applied hydrographs developed by external hydrological models to today, where rainfall excess is applied to high resolution grids over large areas without recourse to lumped conceptualized hydrology. In describing the progressive development of 1d/2d model applications in urban stormwater particular emphasis will be placed on the comparisons made between model results and observed data as well as the specific techniques that are described as being important to achieving reasonable results. Also focused on will be any attempts made by researchers to observe the model systems ability to emulate specific elements of the overall hydrological response.

In order to contextualise the use of the 1d/2d coupled model in urban stormwater modeling and the possible advantages it is necessary to first look at the history of urban stormwater modeling in general.

History of Urban Stormwater Modeling

Numerous summations of the history of urban drainage design exist. However none were found to be as comprehensive as the one by Goyen (2000), which itself heavily relies on O'Loughlin et al (1996) and Chow (1964). The ensuing section then is based mainly on Goyen's (2000) work.

O'Loughlin et al (1996) makes the observation that with respect to estimating flows of a given probability, ideally flows would be derived based on an observation of historical flows at the location of interest. Obviously however this is not always possible, especially when at first such estimates were required. This has driven design flow estimation to be rainfall based. As such models were required to turn a location specific rainfall into a corresponding flow (corresponding with respect to probability of occurrence).

The first established model for estimating peak discharge was the Rational Method. Originally developed by Mulvaney (1850) and his brothers for rural areas, Kuichling (1889) and Lloyd-Davies (1906) then refined it for use in urban areas. The Rational Method is empirical and it utilises catchment area, rainfall intensity and some localised empirically based co-efficient (which also accounts for the frequency of event being estimated) together in order to provide a peak flow estimate only. In some variations slope may also be incorporated. In 1945, McIllwraith noted that some forty empirically based equations existed around the world for the estimation of peak flows, with the Rational Method remaining the most popular by far (O'Loughlin et al 1996).

Goyen (2000) however notes that in the early 1900's, a smaller body of literature began to develop which promoted the estimation of entire event hydrographs. The first of these methods were developed by Ross (1921) and Hawken (1921) and was the Time-Area routing method.

The next noteworthy developments are found in America where Hicks (1944) produced a method for estimating design peak hydrographs from storms in the Los Angeles region. The method was essentially taking a design rainfall and then subtracting losses and applying detention lagging to contributions from surface, gutter and conduits. Final hydrographs from individual components were then superimposed to produce an output hydrograph (Goyen, 2000).

Similar work was undertaken in Chicago in 1960 by Tholin and Kiefer. They simplified the drainage network of an area, representing it as a series of main drains with sets of laterals flowing in at half mile intervals. Each of the simplified units represented an area of two to five acres. Losses were subtracted and the routing was carried out according to a storage routing procedure developed by Izzard (1946). As per Hicks various sources of detention were then implemented such as street gutter detention. From this, they were able to develop the inflow hydrograph for the pipe system. Roof runoff was dealt with separately and in an approximate fashion in order to avoid laborious input (Chow, 1964). The methodology was found to be in loose agreement to gauged data for two separate events over a 13.9 acre catchment (Goyen, 2000).

In the previous two examples it can be seen that various components which provide lag or detention to runoff inputs to the stormwater pit are incorporated in a manual and step by step fashion. Also, simplifications are made in order to avoid laborious computations in this pre-computer age.

Between 1951 and 1963 similar work was being carried out at John Hopkins University (Goyen, 2000). The triangular pit inflow hydrograph had a peak derived from the Rational Method and a base width that was twice the value of T, where T was equal to the period of time from the initiation of rainfall to the end of the maximum intensity of the rainfall burst. Once in the drain T was lengthened according to the time of wave travel for the drain distance (based on length and velocity) and as such, given the fixed volume, the peak flow was attenuated (Goyen, 2000).

In 1962 Watkins published work on the TRRL model, which utilised time-area routing in order to develop inlet hydrographs for stormwater pits, with runoff coming from impervious areas only (Goyen, 2000). This model was further developed by numerous contributors to eventually become ILSAX (O'Loughlin 1986). Later versions of the model considered inlet hydrographs developed from pervious areas as well as the impervious surfaces (although considered separately and then superimposed at the inlet). Upon entering the pipe system, hydrographs are time shifted based on pipe length and velocity and attenuation of peak occurs.

In the early 1970's (Goyen, 2000) a storage routing method was introduced to SWMM for estimation of urban sub-catchment hydrographs (such as might be applied to inlet pits). A single non-linear reservoir was used and its width (the reservoir was conceptualised as a rectangle) was equal to the width of overland flow. In contrast to the models previously discussed, in-pipe/channel lagging was then carried out based on the full momentum and continuity equations being solved. Recently, kinematic wave approaches have also been incorporated into SWMM for inlet hydrograph estimation. Such an approach, with respect to urban sub-catchment hydrograph estimation has also been proposed by other researchers such as Rovey et al (1977) and Ferguson and Ball (1994).

In 1978 in England an International Conference was held (Kidd, 1978) that sought to compare the available methods for urban sub-catchment hydrograph generation. Models were applied to impervious surfaces only and in a lumped fashion for eighteen catchments which ranged in size from 78 m^2 to 20,000 m^2. Various routing methods and loss models were trialled. The routing methods used were:

- Linear reservoir;
- Non-linear reservoir;
- Non-linear reservoir with additional lagging;
- The NASH cascade;
- Muskingham; and
- Time-area methods.

Loss methods trialled were:

- Initial loss followed by continuing loss;
- Initial loss followed by a constant proportional loss; and
- A variable proportional loss whereby the initial period of loss was highest and infiltrative capacity was able to be reduced as the storm burst persisted (based on the Hortonian method).

Generally support was found for the non-linear reservoir approach and proportional losses. With respect to routing models it was however the case that any of the applied methods could produce reasonable results (Goyen, 2000).

The preceding history is mainly concerned with approaches to urban sub-catchment hydrograph generation. It was during the 1970's that routing methods developed via the rural branch of hydrological research intended for whole catchment studies crossed over for use in whole of urban catchment hydrology work (Goyen, 2000). As such, the history of regional hydrology which eventually contributed to urban stormwater modeling (as presented above) will now be examined. Again, the work presented below is heavily indebted to Goyen's (2000) PhD work.

History of Regional Hydrology

Note this section will not attempt to examine the history of the development of distributed deterministic models. Instead those developments in regional hydrology which led to the models that have typically, in Australia, been used in urban stormwater hydrology, are examined.

As previously stated, rural hydrology peak flow estimation initiated with the work of Mulvaney (1850). However, the first work on total hydrograph estimation comes from the unit hydrograph method developed by Sherman in 1932. Following this we have Clark who in 1945 proposed the lagging of rainfall excess and then the routing of it through a linear reservoir. Nash (1960) proposed a model which too used the linear reservoir approach, but in Nash's model there were ten cascading linear reservoirs. In 1964 Laurenson continued along this line but used a non-linear rather than a linear reservoir. During the same period Singh (1962), Diskin (1962) and Kulandaiswamy (1964) also researched the advantages of non-linear storages over linear methods. Their studies supported the use of non-linear storages for the large rural catchments studied (Goyen, 2000).

In the 1970's, pioneers such as Aitken (1973 and 1975) then applied these rural hydrological methods to urban catchments (Goyen, 2000). In examining a number of gauged Australian catchments, Aitken (1975) developed a relationship by regression which tied catchment properties for Laurenson's (1964) ten cascading non-linear storage model to the estimation of the average storage delay parameter or B_{av}. The specific relationship was

$$B_{av} = 0.285.A^{0.52}.(1+U)^{-1.972}.S_c^{-0.499}$$

where A = area, U the fraction of urbanisation and S_c the channel slope.

This development is significant as it was seen as means for estimating model parameters based on catchment characteristics at locations where gauging was not available to do so by a calibration/validation approach. Laurenson (1964) said that "…the greatest potential usefulness of the runoff routing procedure developed was in the field of flood estimation for ungauged catchments".

In 1976 Goyen and Aitken developed a networked version of Aitken's (1975) modeling procedure. In this, total catchment techniques were transferred to a sub-catchment basis in a similar arrangement to the urban drainage design systems presented earlier. Sub-catchments were networked together by a series of links such that pipes, channels and creeks could be incorporated as routing elements. The overall procedure eventually became the software known as RAFTS in 1980 (Goyen, 1991). Subsequent modifications included the ability to split out pervious and impervious fractions of sub-catchments. Estimation of average storage delay for pervious and impervious fractions of sub-catchments was done on the basis of a modified version of Aitken's (1975) work which included parameters for impervious percentage rather than degree of urbanisation and also a surface roughness.

Other developers produced RORB (Laurenson et al 1985) which also utilised a non-linear storage routing model and was later on modified for use in urban applications (Goyen, 2000).

Subsequently Bufill (1989) carried out research which found that the storage routing parameters for such models were poorly correlated to increases in catchment area. That is, Bufill (1989) found that overall catchment lag did not increase as catchment area increased, but at some point, would increase relatively slowly for a given increase in area (Goyen, 2000). As such Bufill proposed a model with parallel "clusters" of impervious areas that would drain directly to a main branch. Catchment lag parameters (in this case k_c[8]) would then refer to the lag of a single cluster, rather than the lumped catchment under investigation.

Performance Limitations of Available Methods

Both Aitken's (1975) and Bufill's (1989) work developed regression relationships for estimating storage delay parameters applicable to the different modeling structures they were working with, and then blind tested these on gauged catchments and compared estimates of flow peak to gauged flow peak. Results were not without significant error (Goyen, 2000).

Scale effects for these models have also been examined and for the same catchment it is demonstrated (Goyen, 2000) (IEAust, 1987) that degree of discretisation impacts estimates of peak flow. Results show that some models will, for a higher degree of discretisation, increase peak flows, and others will further route and hence reduce estimated peak flows.

Further, Goyen (2000) states that the models used by Aitken and Laurenson which then became the basis for the popular packages RAFTS and RORB respectively have developed regression relationships necessarily from large (greater than 100 km^2) catchments and thus are not particularly suited to smaller sub-catchment work.

Based on the above Goyen (2000) states that "…the main barrier to providing more reliable urban stormwater runoff analysis and design rests almost wholly within the sub-catchment hydrologic processes that supply inputs to the hydraulic models of the main pipe and channel carriers". Specifically he concludes that there are indications that parameter constants applied in the models in current usage are not constants or that the definitions of current processes is

[8] K_c is similar to the average storage delay parameter (B_{av}) previously described in RAFTS

incomplete. He states that “Re-examination of the basic conceptual structure of the current sub-catchment models may be required” (Goyen, 2000).

Summary

Computer modeling is a relatively recent phenomenon when it comes to urban drainage design. Until the early 80’s when the use of micro-computers became widespread, most urban drainage designs were done by hand piece meal. So a small area requiring design may have been focussed on, rather than whole of catchment holistic type design being carried as would be best practice today. Whole of catchment modeling is the preferred way simply because it ensures that upstream developments to do not lead to worsening of flood conditions in the lower areas of the catchment.

Goyen (2000) and O’Loughlin et al (1996) contend that since computer modeling became wide spread, there has been little development of fundamental theory in hydrology. Focussing on hydrological methods that facilitated hydrograph estimation, the research work was done between the 1930’s and 1970’s. Since then some modifications have been made, for example co-ordination of different hydrological methods into the one software package or an improved ability to split pervious and impervious areas and hence to cope with future scenario modeling. But there has been no fundamental improvement in the way hydrological processes are represented as algorithms in the models being applied.

Goyen (2000) insists that more research is necessary as various studies have shown significant issues with the current approaches. To summarise Goyen’s work he says the following:

- Scaling issues exist with the specific problem being that depending on how many sub-catchments you discretise for a specific study, your peak flow rate will vary. Some models may find a lower peak flow rate, others the opposite;

- Parameters appear to be sensitive to storm burst properties rather than simply descriptive of catchment characteristics only; and
- Despite Laurenson's (1964) hope that the main advantage of the model he had developed would be its usefulness in ungauged catchments, subsequent studies have shown large amounts of error when the models are applied blind to gauged catchments i.e. routing parameters are estimated based on physical catchment properties. Two studies are principally mentioned by Goyen, Aitken (1975) and Bufill (1989). Both have an undesirable level of error as they attempt to match gauged flows using physical catchment characteristic derived model parameters.

Goyen's (2000) PhD Work then develops an alternate method which takes on-board Bufill's (1989) idea of a parallel "cluster" that contributes to main drain flow and further develops this against a highly detailed and specifically collected data set, including gaugings. This work whilst still utilising conceptualisation, is far more detailed and is effectively a semi-distributed approach. The validation and blind test demonstrate very good success in matching gauged flows and it appears to be a genuine advance on the work carried out by Aitken (1975) and Bufill (1989).

Given, however, the current ability to utilise physical process models at high resolution, the author seeks to test a methodology of coupled 2d and 1d modeling. A 2d surface model is used, with rainfall excess applied directly to it, in order to eliminate conceptualised routing of sub-catchment hydrology.

The next section seeks to summarise the work to date on coupled 1d/2d modeling in urban areas in order to place the work of this research within the correct context.

History of coupled 1d/2d modeling in application to Urban Stormwater Modeling

Introduction

In the following section the aim is to trace the development of 1d/2d coupled modeling systems. The following is not supposed to be exhaustive in describing all systems that have ever been developed for the purpose of urban stormwater modeling. As such many popular software suites have been omitted simply because in being developed they were not particularly novel but instead refinements perhaps of existing modeling systems.

It is also noteworthy that the limiting factor in many proposed solutions to the coupled model issue has been the availability of computational power at the specific time of development. That is the theoretical basis has existed, but the machine to make the run in a reasonable time frame has not. For example, it is the case that only recently (arguably in the latter half of the first decade of the 21st Century) has a common desktop personal computer had the capacity to simulate 2d routing for a multiple hour rainfall event over a typically sized urban catchment on a grid size of sub-5 m. As such the timeline following which indicates a movement towards the usage of rectilinear grids for describing surface flow does not do so because of theoretical advances necessarily, but simply due to earlier hardware limitations.

History of development

As discussed in the previous section, in the 1970's urban drainage philosophy altered such that the major/minor drainage system was focussed on and events up to the 1 in 100 Y ARI were considered (O'Loughlin et al, 1996).

The major/minor drainage philosophy was simply this, smaller events could be dealt with via the kerb/gutter, flow to pipe inlets and then the buried pipe system (or drainage channels). The design capacity for this system might for example be the 1 in 5Y or 1 in 10Y ARI. Where the minor system began to fail due to insufficient capacity, the major system was to take over. The major flow paths were the streets. The idea was that flow could be moved away from the area of interest without damaging houses and buildings. Adoption of this design philosophy meant that designs needed to be fairly complex. Piped systems might flow in different directions to the surface systems they were connected to. Flow from pipes may, in certain conditions, surcharge into the major system, then be carried overland only to re-enter the piped system at some other point where sufficient capacity in either inlet or pipe was available. It was recognized that although these models needed to be linked in order to facilitate flow transfer, they were also required to be able to act independently, as surface flow directions were not necessarily likely to follow sub-surface flow directions.

Initial applications, such as SWMM for example, applied runoff developed from a hydrologic engine to the pipe system. If pipe capacity was already utilised, then surcharge was possible, however surcharged water remained at the inlet node, until such time as pipe capacity was available for entry of the detained surcharge to the pipe system.

What was required really were models which were able to route based on the actual on-ground conditions and incorporate water levels computed dynamically when determining calculation of flow direction for the surface model.

The first models capable of achieving this goal were 1d/1d models. That is, a 1d pipe model was coupled to a 1d surface flow model[9]. According to the literature this was first done in 1982 (Smith, 2006) by Kassem (1982). Efforts continued on 1d/1d modeling in the 1980's and 1990's with major advances made by numerous individuals. The development of such models is best dealt with by Smith (2006) in his comment on a paper published in the Journal of Hydrology by Schmitt et al (2005). An obvious limitation with this method was that surface flow paths had to

[9] It is assumed here that a 1d model can, if schematized so, act in a quasi-2d fashion.

be pre-described and hence had to be known to the modeler prior to the model run being carried out. In flat areas it is not always possible to predict flow paths, particularly when during events afflux at various structures may alter hydraulic gradients and cause unpredictable flow paths.

1d/1d applications were developed and applied and in some cases compared to gauged data. Observations made were that results did vary significantly when surface runoff in excess to inlet capacity was allowed to route downstream rather than arbitrarily pond and that GIS was a significant tool for this work in that its inherent spatial capability was likely to be instrumental in facilitating data input. GIS was also used as a modeling tool whereby several researchers used it for routing flow to inlet pits and routing excess flow overland to downstream pits (Smith 1993). A limitation to this technique was that flow paths were pre-described based on analysis of topographical and other features (such as street alignments). Flow was not dynamically routed, i.e. free to flow according to gravity and topography. Findings from these studies indicated the importance of using a fine grid (in the order of 1-5 m Elgy et al 1993) in order to properly represent urban features. In the case of Elgy et al (1993) the street area was lowered in the grid by 0.5 m.

For example in 1999 (Djordjevic et al) a GIS was used to construct overland flow path connectivity for a coupled 1d/1d model. In this study minor depressions in the source topographic data were filled and the streets stamped into the DTM in order to ensure its use as a preferred flow path. A 1 – 2 m grid was recommended.

One of the first dynamic 1d/2d model applied to urban stormwater modeling can be attributed to Takanishi et al (1991). The model used in the application was developed by Iwasa et al (1987). In Takanishi et al (1991) a 100 m grid surface flow model was coupled to pipe and river flow for the purposes of modeling inundation in the Japanese city of Nagasaki. Water levels were applied as boundary conditions as it was fundamentally a storm surge study. The model was able to coarsely reproduce flooding extents from a 1957 flooding event. This work also demonstrated the need to adjust the topography in order to ensure that important flow paths such as street flow were emphasised.

Hsu et al in 2000 made another application looking at storm surge induced flooding. In this application a 120 m grid was coupled to a sewer model in order to replicate surcharge induced flooding associated with a recorded typhoon in Taipei. Spatial inundation extent results were shown to indicatively correspond to those observed based on inundation mapping.

Also in 2000 Inoue et al published a paper describing an application of three different types of model to urban flood modeling. Finite difference grid, curvilinear element and 1d-quasi 2d modeling solutions were compared for an urban flooding situation. Inoue et al's conclusion is that in the finite difference model flow blockage due to buildings is difficult to incorporate (presumably as the assumption is that grid sizes must be relatively large in order to have achievable run times). It is found that both the curvilinear element and the 1d-quasi 2d system are superior, with the 1d-quasi 2d system providing the most satisfactory results. What is not commented on is the time required to establish each of the models.

In 2002 Schmitt (2002) stated that 2d representation of the overland flow in urban stormwater modeling was essential. Presumably Schmitt was making the observation that a 2d model did not require pre-determined flow paths and as such was ideally suited to modeling overflow in urban catchments.

Also in 2002 Boonya-Aroonnet et al compared pipe/detention[10] models to 1d/1d models to 1d/2d models. This showed that the 1d/2d model performed best but most notably so for the larger events where the 2d overland flow layer became emphasised. In the paper the need for detailed topographic data was emphasised and a grid size of 5 m was recommended.

In 2003 Alam developed a model of coupled 1d pipe and 2d surface flow or Dhaka in Bangladesh. In this instance flow hydrographs were developed using an exterior hydrological model and then applied to the pipe model. Surcharge from the pipe system was then modeled in the 2d surface flow component.

Schmitt et al (2004) presented an application of a modeling system which combined many modules one of which was a dynamic coupling of 1d pipe and 2d surface flow. This system could apply lumped conceptual methods or in specific areas could use more physical disaggregated methods, as per user choice. Importantly this study focused on identifying different components of hydrological response and utilizing an appropriate model for replication. For example house roof response was modeled in a separate module which utilized a unit hydrograph. Schmitt et al (2004) highlighted the need for detailed topographical information in order to allow for key flow control structures to be resolved adequately. There was no calibration carried out in the study and instead verification of the model was carried out using synthetic hydrographs.

In 2004 Carr presented a conference paper which outlined the application of a 1d/2d modeling system to a catchment in Auckland NZ. This study compared model results to a 1d approach (1d pipe model coupled to a 1d quasi 2d overland flow model) as well as comparing the results

[10] "detention" meaning simply that water that surcharges from the pipe system is stored at the inlet node (based typically on a user defined or default stage/storage relationship) until such time as sufficient pipe capacity exists for it to enter the pipe system.

to gauged data. Not all inlet pits were modeled and this likely reduced the quality of the model results. It was however shown to be an improvement on the 1d/1d approach with respect to peak flow estimation. Interestingly it was also reported that model setup time was, compared to the 1d/1d approach, markedly reduced. This application used rainfall applied directly to the grid, with losses being subtracted based on the spatially defined loss settings such that a spatial rainfall excess was developed.

In 2004 Mark et al presented a paper which looked at the limitations of the traditional 1d/1d approach and discussed some of the issues with the 1d/2d approach. Note was made of the need to "burn" the kerb into the surface flow model to ensure that these important flow paths performed adequately. Also it was indicated that the required grid resolution was in the order of 1-5 m if the appropriate features were to be resolved.

In 2004 Syme et al presented a paper where a pipe model was linked to a 1d river model and then the system interfaced with a 2d surface model. No calibration was carried out, a 4 m grid was used and there was no mention of specific treatment of the 2d surface for flow path emphasis etc.

Halie's (2005) thesis provided an excellent summary of coupling approaches for 1d and 2d models which, although focused on the coupling of 1d river and 2d floodplain models, highlighted some issues of importance to urban storm water modeling. A key point made was that DEM's, as they are re-sampled to coarser levels lose significant amounts of data of key importance to hydraulic modeling.

A paper in 2005 by Phillips et al compared the 1d/2d model to a 1d/1d and pure 2d approach for an area in Penrith, NSW, Australia. Not only were 1d/1d and full 2d approaches compared to the 1d/2d system, but also within the 1d/2d approach different grid sizes were used and impact on results examined. It is noteworthy that in this study Manning' 'n' roughness values of 0.02, 0.15 were used for roads and yards respectively. The high yard value was used on the basis of fences etc obstructing and attenuating yard flows. It was determined that the 1d/2d system provided the best system in that small continuous channels features could be resolved efficiently in the 1d whilst overland areas where flow paths were ill defined were best dealt with in 2d. It was also found that varying the grid size had significant impacts on resultant water levels. No calibration was carried out in this study.

In 2006 Smith et al presented a paper which discussed the application of a 1d/2d modeling system to investigate the impact of drainage augmentation and road works on flooding in the city of Dubbo, NSW, Australia. This paper makes the comment that due to the requirement by the NSW Floodplain Manual (DNR, 2005) that in urban stormwater studies overland flow paths should be modeled and also due to the increased availability of facilitating data, the 1d/2d model system was becoming the new benchmark in urban flood studies. In this study the 1d/2d system was specifically applied because Lidar data was available and also there was a concern, that due to minor chainages in the drainage system, surcharged water might move overland via pathways that had not been previously identified. Model calibration was not carried out nor were results compared to any observations. The study did find, by sensitivity testing, that model results were significantly impacted by model resolution.

In 2006 Carr and Smith presented a paper which documented two studies that had utilized slightly varied forms of the 1d/2d coupled system. Details for the first application are already summarized in Carr (2004), the second example was an application in a Sydney coastal catchment where spatial calibration data (flood marks) were available. The model was run with an applied rainfall excess for a calibration event and it was found that results were a close match, with a mean error of 0.17 m, median 0.13 m, maximum error of 0.45 m and a best result (out of a set of seven marks) of 0.01 m. It was noted that:

- The pit inlet co-efficient controls entry to the sub-surface and so this is a calibration parameter for events only in which pipe capacity is not reached;
- If pipe capacity is reached, then the overland flow parameters (roughness only) become dominant and these can be readily informed by observation and standard values; and so
- Apart from loss assumptions, very little calibration was required.

It was found however, given the 3 m grid utilized, that where errors were larger than the mean, this tended to be linked to the presence of features that were not resolved in the grid. The study concluded that the methodology applied meant that the behaviour of the catchment could be represented in the model with a minimum of calibration. Also setup and presentation time effort was reduced due to GIS facilitation of input and output.

In 2006 Xing et al published results from a coupled 1d/2d approach that utilised a flexible mesh approach. The flexible mesh approach was developed by the authors and it incorporated pipe/channel elements into the side of the flexible mesh element. Over an area of 60 km^2 786 elements were defined. Calibration for an event of extreme intensity (Tropical Storm Allison) gave good results. Significantly, run time was such that the 60 km^2 area could be modeled for a 24 hour event. Plots comparing model results to observed results show a good shape correlation. Roughness values applied were:

- Mannings ‘n’=0.08 for residential areas (houses and other obstructions were incorporated as lumped roughness rather than individually owing to element size);
- Mannings ‘n’=0.06 for agricultural areas; and
- Mannings ‘n’=0.05 for undeveloped areas.

In 2007 Gordon presented an application of the 1d/2d system which included rainfall excess on grid and comparison of results to a substantial and spatial observed data set. The grid size used was 15 m which was ill-matched to some parts of the study site in that urban roads were modeled with a typical width of approximately half the grid size. Results from the study were good and no iterative calibration was carried out. Result error is reported at within +/- 0.075 m for 72 separate distributed locations. Another highlight of the study reported is the quick model setup time. Results were compared to a previous study which had used a more typical 1d/1d approach (utilizing XP-SWMM) and found to compare favourably but with considerable less effort expended on model setup.

In 2007 Dey et al applied a 50 m grid to an urban area of 2.7 km^2. Significantly Dey states that in order to ensure accuracy of flood prediction and get simultaneous interaction between surface and sub-surface, overland flow is simulated in the 2d and not in the runoff block of the applied SWMM model. Model results were compared to observed at eight locations and the mean error found was 0.13 m. A single approximately 1 in 100Y ARI event was examined.

Although not a coupled model application, work by 2008 Hunter et al is of interest. The study compared six different fixed grid 2d models in application to a small (400 m^2) urban catchment utilising a 2m grid. Interestingly in the comparison it is noted that diffusive finite grid models will require a smaller time step than models solving the full shallow water equations and hence require greater computational time. It is concluded that diffusive wave solutions may, in circumstances where inertial effects lead to greater spatial extent of flooding, lead to an underestimation of flooding extent. As such the diffusive solution is perceived to be more appropriate to a flatter area.

Summary of lessons learnt from previous studies

- DEM resolution needs to be high in order to resolve the features of import to urban stormwater flow;
- few 1d/2d models have been applied and then compared to observed data. Takanishi et al (1991), Hsu et al (2000), Carr (2004) and Carr and Smith (2006), Xing et al (2006), Dey et al (2007) and Gordon (2007) are the only research identified which compared model results to observations. Of these only Carr (2004) and Xing et al (2006) compare model results to a gauged hydrograph and this for both is for one event only;
- Schmitt et al (2004) stands out as the only example which attempts to apply different model modules to represent different units of response as deemed appropriate. None of the studies reviewed has data which facilitates the calibration or validation of model performance against different elements of hydrological response;
- Many studies have emphasized the importance of "stamping" or "burning" the kerb and gutter into the grid. It is likely that this is the case as the kerb/gutter is so important to urban storm water drainage yet is unlikely to be resolved by even the highest resolution topographical data sets available today;
- Calibration requirement is reported to be reduced (Carr and Smith 2006, Dey et al 2007 and Gordon 2007) and it is also reported that there are few parameters to manipulate in any calibration should it be carried out. These models all use the 2d surface model for hydrological routing. Hunter (2008) emphasises that given Lidar, roughness is the key calibration parameter;
- Model build effort is reduced when compared to traditional methods which involve partitioning the urban area into hydrological sub-catchments and also schematizing the 1d overland flow paths;

- GIS is very important for facilitating data input and presentation of results and
- The 2d model is used for generating overland flow in order to get better accuracy than traditional methods of sub-catchment hydrograph generation and in order that interaction is accounted for between sub-surface and surface elements (Dey et al 2007).

Chapter 3: Site Location, Characteristics and Data Description

Site Description

The study sites, named Lot 12 and Lot 14[11], are adjacent to each other in the north Canberra suburb of Giralang, Australian Capital Territory (ACT), Australia. Lot 12's catchment is 12 house roofs all of which drain direct to trunk drain. Lot 14 drains a catchment of approximately 1.5 ha, which includes house roofs, road, paved areas and yard. The wider area that Lots 12 and 14 lie in is itself a well gauged one square kilometre catchment, having three proximate pluviographs (each within a kilometre of each other) and an in-pipe outlet gauge with a continuous discharge record of approximately forty years.

The general area of the study sites within the ACT is shown on the left in Figure 5, with the image on the right showing the location of the sites within the wider Giralang area.

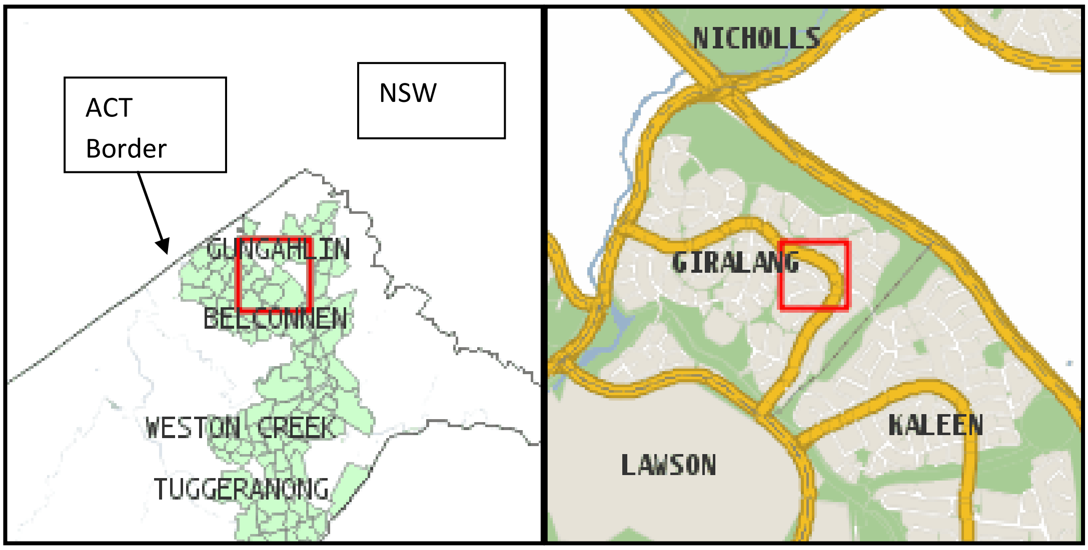

Figure 5: Site Location[12]

[11] Names are based on the number of houses in each of the study catchments

[12] Maps are produced from ACTMAPi data, supplied by the ACT Planning and Land Authority, Copyright ACT Government 2008.

Figure 6 shows the approximate location of Lots 12 and 14, within the larger one square kilometre catchment in Giralang. The 3d image has been produced using Lidar data. Note that the wider catchment boundary is also shown along with a cadastral overlay.

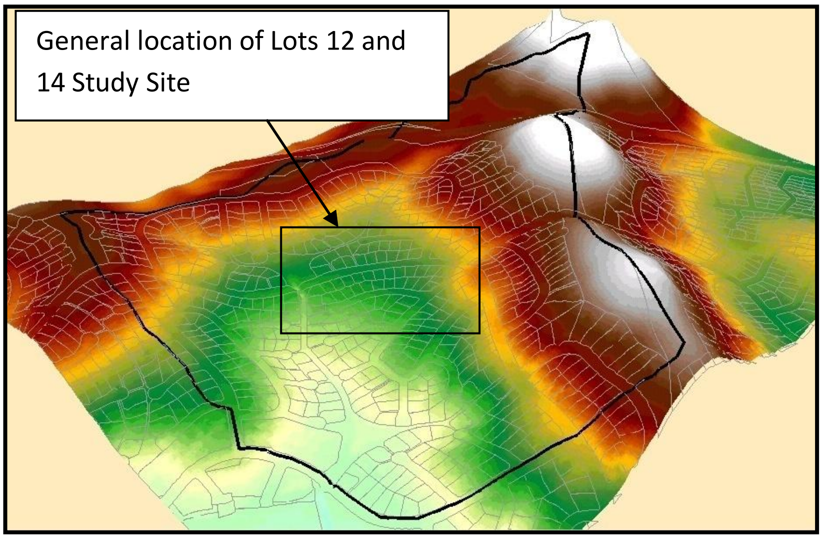

Figure 6: 3d catchment with Cadastre and catchment boundary for wider Giralang catchment

Figure 7: Street Map of Study Area with study areas shown

Figure 7 and Figure 8 show Lot's 12 and 14 in some detail. In Figure 5 house roofs draining to the in-pipe Lot 12 gauge are marked whilst the catchment outline for Lot 14 is shown in black. Figure 8 shows the location of Lot 12 and 14 gauges, trunk drainage and inlet pits. Gauges are shown as red dots, trunk drains as orange lines and inlets are noted with a label and green dot. It is these inlet pits which serve as the interface between the 1d and 2d models used in this work.

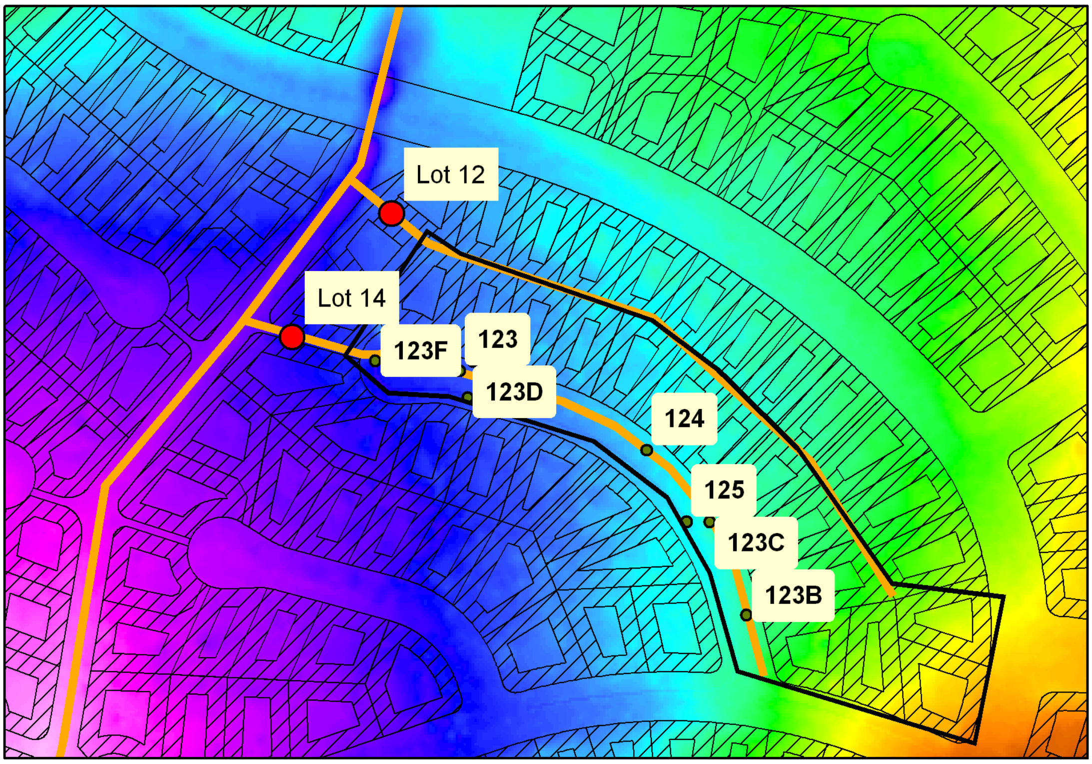

Figure 8: Study Catchments - Lot 12 and Lot 14 with Cadastre and Inlet Pits

The study catchments, Lot 12 and 14, were established by Goyen (2000). Goyen implemented in-pipe gauging at the terminal ends of two adjacent catchments, one draining only house roofs (Lot 12) and the other draining roof, road and yard areas (Lot 14). Goyen also established a high resolution pluviograph at a location close to the centroid of the combined Lot 12 and Lot 14 catchments. Refer to Goyen (2000) for details of gauge setup and validation etc.

Lot 12 Description

Lot 12 is a catchment of twelve house roofs only with a total area of 0.207 Ha or a mean roof size of 0.0175 Ha each. House roofs are pitched and mainly of masonry tile type. Roof slopes are in the order of 10%. All roofs drain to perimeter gutters which then drain to downpipes connected to the municipal drain by underground pipe. The receiving trunk municipal drain has a diameter of 300 mm.

Lot 14

Lot 14 is a catchment of total area 1.51 Ha that contains a road (Gundulu Place) and 14 houses including roofs and yard areas.

Photos of Lot 14 which depict typical conditions are shown below as Figure 9 to Figure 11.

The principal features of the study catchments are as follows:

- Yard areas are not entirely pervious. Paved areas including driveways and recreational spaces are distributed over house allotments, with most impervious spaces within yard areas being proximate to the road and hydraulically connected in the form of driveways;
- Overall slopes are flat, at grades of approximately 1%;
- Most front and back yards have lawns but gardens also feature, some of which are ringed by brick or concrete edging. Barbeque areas also feature;
- There is a relatively large variation in imperviousness from property to property. At a minimum, a property may have a dual strip driveway, roof area and concrete path in the backyard adjoining the house, whilst other areas are grassed or planted. In contrast, some of the properties have large portions of the front or back yard concreted, complete concrete driveways and back/front yard impervious patio areas.

Imperviousness variation from property to property are approximately 30% to 50%; and

- The Gundulu Place road is a bitumen sealed road with kerb and gutters and inlet pits to municipal drains of diameter 450 mm placed at approximately 100 m centres along kerb and gutter alignments.

Figure 9: Property in Lot 14 at western edge with high impervious percentage

Figure 9 shows front yard fencing, a combination of pervious and impervious surfaces in front yard, extensive sloped and connected pervious areas as well as a pitched roof. Note that the area behind the front fence was mainly concrete and so this property had an overall imperviousness of ~ 40 – 50%.

Figure 10: Example of pervious easement and front garden area.

Figure 10 shows a different front yard to the one shown in Figure 9. In this front yard very little pervious area exists and for small rainfall events very little contribution from the "yard" area can be expected. Note also that a typical inlet pit is pictured in the foreground.

Figure 11: Lot 14 - Looking from upstream to downstream

Figure 11 shows Gundulu Place from the upstream looking towards the downstream (south to north). The photo was taken at the intersection of Gundulu Place with Spica St (see Figure 7). Note the fully bituminized road, that yard areas are raised above the road and drain to the kerb/gutter system that lies on the roads perimeter. Also in the background of the photo can be seen driveways in yard areas that flow to the gutter. Gutter width is approximately 0.35 m.

Catchment characteristics for Lot 14 as modeled are shown below in Table 1.

	Area ($m^{2)}$
Impervious Road[13] (m^2)	2,598
Impervious Roof (m^2)	2,536
Pervious Yard (m^2)	9,958
Total Area (m^2)	15,092
Pervious %	66%
Impervious %	34%

Table 1: Lot 14 Catchment Characteristics Derived from Land Use Map Analysis

Data Review and Collection

Various types of data were collected and where necessary were transferred to appropriate format, these were;

- Rainfall (gauged in 0.2 mm increments) for Lot 12 and 14 events[14] as listed in Table 2. Plots of rainfall aggregated into five minute blocks for presentation purposes are also shown in Figure 12;
- In-pipe gauged flow for Lots 12 and 14 corresponding to the events for which rainfall data is presented in Figure 12;

[13] Note that paved yard surfaces (such as driveways etc) are included in the road category.

[14] The total number of events which Goyen (2000) gauged over the period 1993-95 is much larger than the set of events selected for use in this study. Refer to Goyen (2000) for further details.

- Spatial Data – this included cadastre, general development layers such as roads, scanned drainage plans on a cadastre background, 1m DTM obtained via Lidar and aerial imagery; and
- Drainage Asset Data which gave the location of manholes and pipes for the entire catchment. Note that pit locations were not provided exhaustively by this data set but were established by field survey.

Selected Events

Events were chosen from the available set based on Goyen's (2000) work. The period over which such events were available was 1993-1995. A surprising number of relatively large events occurred in this three year interval.

Large and small events were selected for modeling as the small events test the performance of the model in replicating flow from impervious areas whilst larger events test the performance of the model in replicating combined surface flow behaviour. Any event modeled had to be of interest to drainage system designers and so approximately 1 in 1Y ARI events were chosen at the lower end. The largest event gauged was a one in eight year event which occurred on 5th January 1995 and this was utilized in calibration of yard response. The other events used are shown over the page in Table 2.

A feature of the selected events was that all but one experienced rainfall in the preceding period. The mean runoff coefficient for Lot 12 events was 0.83, the minimum was 0.61 (for the April 1993 event where the catchment was dry beforehand) and the maximum was 1.15. This is due to downpipe blockage and this issue (i.e. persistent base flow from the small and impervious Lot 12 catchment) will be further discussed in Chapter 6. It is interesting to note then losses for the entirely impervious catchment of Lot 12 are an average of approximately 20% but get as high as approximately 40%.

Lot 14 with two thirds of its area being pervious exhibits substantially lower runoff coefficients than Lot 12 as would be expected. The mean value is 0.55, the minimum value is 0.31 and the maximum value is 0.87. Once again the minimum value is for the April 1993 event which was dry beforehand. The maximum value is from the 28 January 1995 event which was preceded by a heavy rain burst. Rainfall hyetographs are shown in Figure 12 and on these plots total rainfall depth, peak intensity and mean intensity and event duration are indicated. Note that peak intensities have been calculated based on five minute intervals simply since these intensities based on five minute intervals are likely to more meaningful to the reader.

Event	c/v	Start	End	Obs Peak	ARI	Ground	Gauged Vol	rainfall	runoff
				(m3/s)	1 in xY	Before	(m3)	(mm)	ratio
LOT 12									
May-95	c	13/05/95 10:00	13/05/95 13:00	0.018	1.1	wet	15.8	11.2	0.65
Early Jan 93	v	03/01/93 18:15	03/01/93 22:00	0.037	6	wet	91.2	50.8	0.83
Jan-93	v	03/01/93 21:00	04/01/93 00:30	0.042	8	wet	67.4	27.2	1.15
Apr-93	v	05/04/93 15:54	05/04/93 18:00	0.033	2.5	dry	17.2	13	0.61
Jan-95	v	05/01/95 17:45	05/01/95 20:20	0.033	7	wet	36.6	18.4	0.92
LOT 14									
Jan-95	c	05/01/95 17:45	05/01/95 20:20	0.251	7	wet	159.6	18.4	0.56
May-95	c	13/05/95 10:00	13/05/95 13:00	0.096	1.1	wet	60.5	11.2	0.35
Early Jan 93	v	03/01/93 18:15	03/01/93 22:00	0.184	6	wet	380.9	50.8	0.49
Jan-93	v	03/01/93 21:00	04/01/93 00:30	0.234	8	wet	297	27.2	0.71
Apr-93	v	05/04/93 15:54	05/04/93 18:00	0.164	2.5	dry	62.7	13	0.31
28-Jan-95	v	28/01/95 17:00	28/01/95 21:00	0.209	7.5	wet	373	28	0.87

Table 2: Events modeled

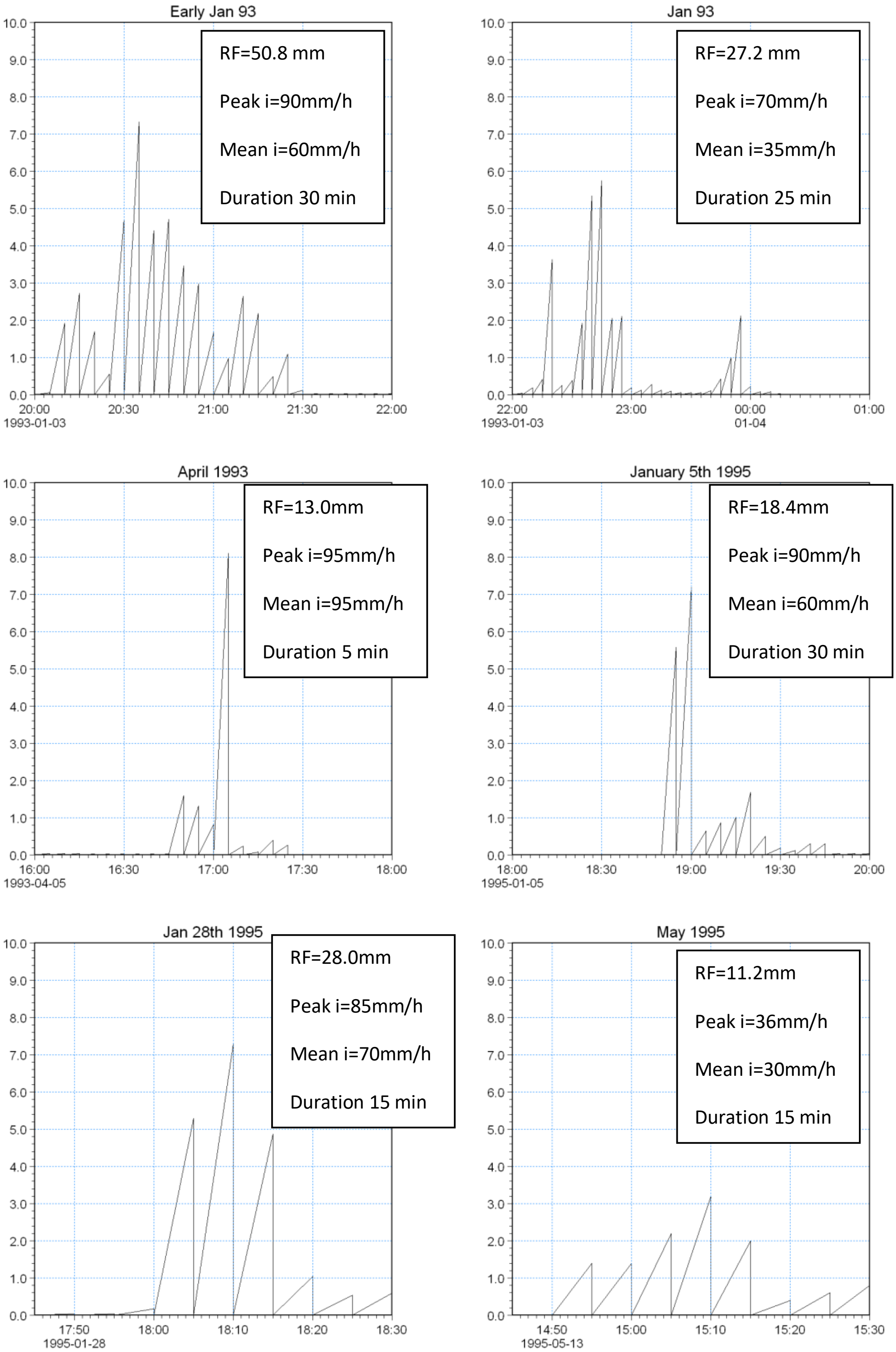

Figure 12: Event Rainfall Depths in 5 min increments

Chapter 4: Methodology

Introduction

Core to the study research goals was the need to build coupled 1d/2d models and to assess performance against gauged data. As such the methodology has been entirely about facilitating this work. In order to build the models, data was required. So the first step in the study was collecting and converting data. Following the collection of the data, the model had to be built. Being a coupled model, both 1d and 2d portions of the model were required to be built. Data collected in the previous step facilitated this work. However in some cases major amounts of editing/processing were required to be carried out in order to make the data usable for model build purposes. Following build the models were calibrated and again, data collected earlier facilitated this although it did require conversion to suitable format.

The calibration method was as follows:

- The model representing Lot 12 was calibrated first. Since Lot 12 contains a catchment area entirely composed of house roofs, this facilitated calibration of the roof response process. Lot 12 performance based on the calibrated parameter set was then tested against four validation events;
- Parameters for roof representation were then transferred to the Lot 14 model and roof plus road response was then calibrated to. This was achieved by using an event with a small rainfall depth and it was assumed (based on the runoff co-efficient for this event) little to no yard contribution to runoff; and
- Finally, Lot 14 roof, road and yard calibration was carried out by using the largest event (with respect to peak flow) from the event set established by review of available events presented in Goyen (2000).

Following calibration, validation runs were carried out. Four events were used for validation, and in these events no model parameters other than the proportional loss rate in 2 of the 4 validation events were varied.

Plots were then assembled and comparisons made between modeled and observed hydrographs.

Data

Data collected has been described in the previous Chapter.

Model Build

Introduction

The approach in the modeling was to utilize the available data set and the adjacent catchments in order to separate out flow components so that the model could be calibrated in increments. The first part of the model built was that part which emulated roof response. The Lot 12 setup was used to achieve this. Following this, road response and then yard response were modeled using the Lot 14 setup. By utilizing this methodology it was hoped that the total model built would be representative of these three processes and as such be able to emulate a wide range of events, from small events dominated by impervious surface contribution to large events where pervious yard areas provided some proportion of volume if not peak flow.

Following the model calibration/build process, it was anticipated that the model would then be applied to the event set and validated.

Another important element of the modeling approach was that although methods were adapted to produce results that were a good match between observed and simulated, a primary goal was to not compromise one of the chief advantages in the coupled 1d/2d modeling system which is ease/speed of setup. Optimizations were to be generic and not require laborious data input.

The idea of building a model that aimed to emulate different comments of the hydrograph and then in sum match the total flow hydrograph came from both my supervisor and from reviewing Goyen's work (2000).

Models Used

The models used in this study are proprietary models commercially available from DHI Water Environment and Health Pty Ltd. These models are MOUSE and SHE. MOUSE is a 1d quasi 2d unsteady flow pipe model, which incorporates a semi-distributed hydrological model based on the kinematic equation. SHE is a distributed deterministic physical process based hydrological model that is capable of a wide variety in its application. In this instance, it was used in order to route rainfall excess, to apply spatial proportional loss maps to applied rainfall, and to apply spatial roughness maps to the catchments studied. The 2d routing solution in SHE is non-inertial (i.e. diffusive) and it can be utilised to route rainfall excess. These models will simply be referred to as the 1d and 2d models in the ensuing text.

The overall system utilized then is a 1d/2d modeling system, where the 2d models surface flows and 1d models pipe flows. Pit inlets are embedded in the 2d surface and allow transfer of flow from 2d to 1d and vice versa.

Model Elements

Note that the demarcation of physical feature to model used for emulation is as follows:

Pipe – 1d quasi 2d dynamic pipe flow model based on full set of St Venant Equations;

Roof – Kinematic Equation Based Semi-Distributed Hydrological Model;

Road – 2d surface flow model using non-inertial form of the St Venant equations;

Yard – 2d surface flow model using non-inertial form of the St Venant equations; and

Pits – weir calculation that interfaces dynamically (i.e. per time step) between the 1d and 2d model components.

1d Pipe Model Build – Lot 12 and 14 Catchments

The 1d pipe model included roof areas in Lots 12 and 14, as well as pipes connecting roof runoff to municipal drainage. Municipal drainage was also modeled, including the main trunk drain in order to move water away from the gauging locations and avoid artificial backwatering of flow. With respect to municipal drainage components the model was populated using an excel database provided by Goyen (2000). This database provided information on pipe dimension, length, invert, ground level, connectivity and location. Editing was done in order to produce a format of database acceptable to the 1d pipe model for ready import. Figure 13 below shows a plan description of the overall 1d pipe model layout. Figure 14 shows a zoomed plan view of the Lot 14 and 12 pipe models. Note that nodes representing individual house roofs are connecting to the municipal trunk drain.

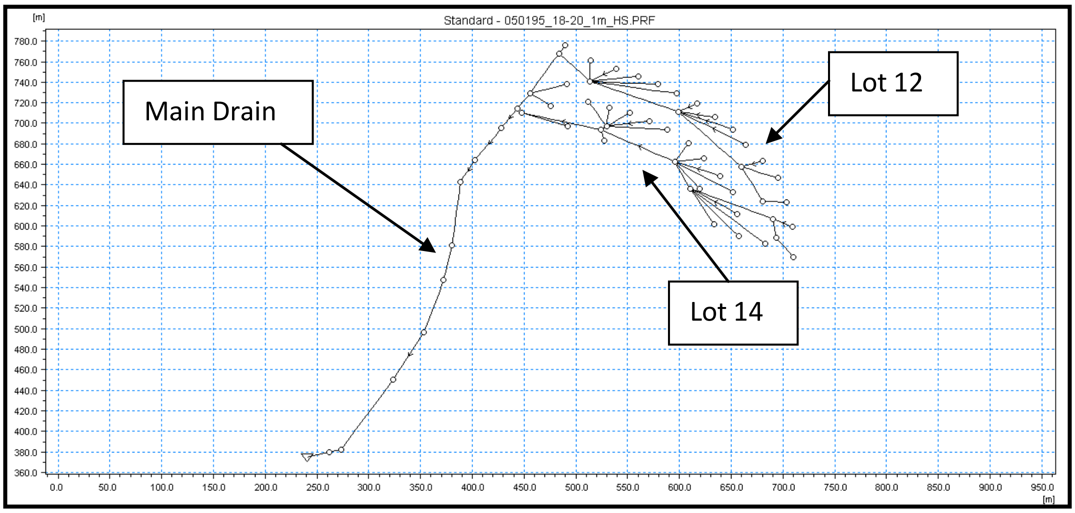

Figure 13: Lot 12 and Lot 14 Pipe Model Plan

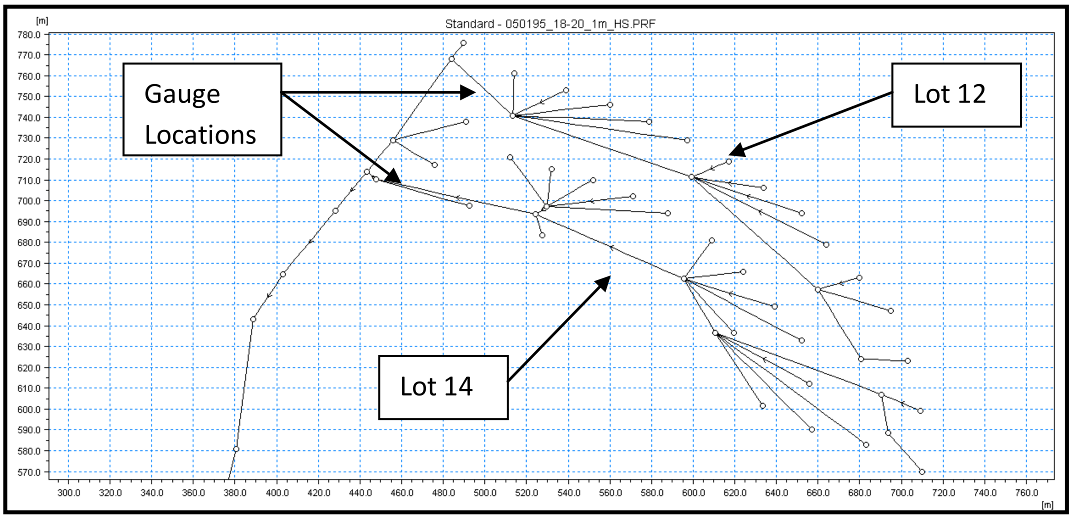

Figure 14: Lot 12 and Lot 14 Pipe Model Plan Zoom

Modeling of roof hydrology and hydraulics was incorporated into the 1d pipe model. Early results showed that 2d routing at a 1 m resolution could not capture the fast flow response of a graded roof surface. It was also considered burdensome and ill-fitting with the overall methodology (quick setup being one of the main advantages of the system) to introduce sloped surfaces to the roof area in 2d and to test the models ability to emulate roof runoff (at a 1 m grid spacing) in this way. Further it was considered that the percentage contribution from roofs (to peak flow) would be relatively low at around 10% and as such, a peak error from the roof would become insignificant at the Lot 14 gauge location.

Roof hydrology was modeled utilizing the kinematic wave hydrological model incorporated into the 1d pipe model called URBAN Model B. This uses a kinematic channel in order to route flow and relies on input of slope, area, length and Mannings roughness. Discharge is calculated as

$$Q=(1/n).B.I^{1/2}.y_r(t)^{5/3}$$

where Q is flow (m^3/s), n is Mannings n, I is slope, B is breadth (B=Area/Length) and $y_r(t)$ is the rainfall excess per time step.

Individual roofs were defined as nodes connected to municipal drainage. Calibration of the roof input was carried out using the May 1995 event. The parameters utilized are shown below in Table 3.

Area (ha)	0.018
Length (m)	20
Slope	0.04
Roughness (n)	0.02
PL (%)	33

Table 3: Hydrological Parameters applied to each roof

Two-Dimensional Model Build – Lot 14 Catchment

A 2d surface flow model, with inlet pits that connected to the 1d pipe model, was constructed for Lot 14.

The main steps to constructing the 2d surface flow model are summarized as:

- produce DEM;
- adjust DTM with break lines for kerb/gutter alignment;
- produce map of pervious and impervious areas; and
- produce roughness and loss maps and
- insert coupling points.

Digital Elevation Model

The principle input to the 2d model is the Digital Terrain Model (DTM). A 1 m DTM sourced from Lidar data was used in this study. This was the highest resolution that could be used as the Lidar data consists of a mass of points at approximately 1 m centres and was also the resolution the original data was supplied in.

At a 1 m resolution, delineation of kerb and gutter was shown by initial calibration/build runs to be inadequate. As such, some manipulation of the source DTM was required. Run results (presented in Chapter 5) demonstrate the degree of influence each of these have on results and these modifications are listed below:

- Stamping in a defined flow path in order to emulate gutter characteristics, note given the 1 m grid and the fact that site gutters are in fact approximately 0.35 m in width means this was done rather coarsely[15]; and
- Filling of DTM depressions. This was achieved using an artificial period of intense rainfall well before gauged rain was applied to the model.

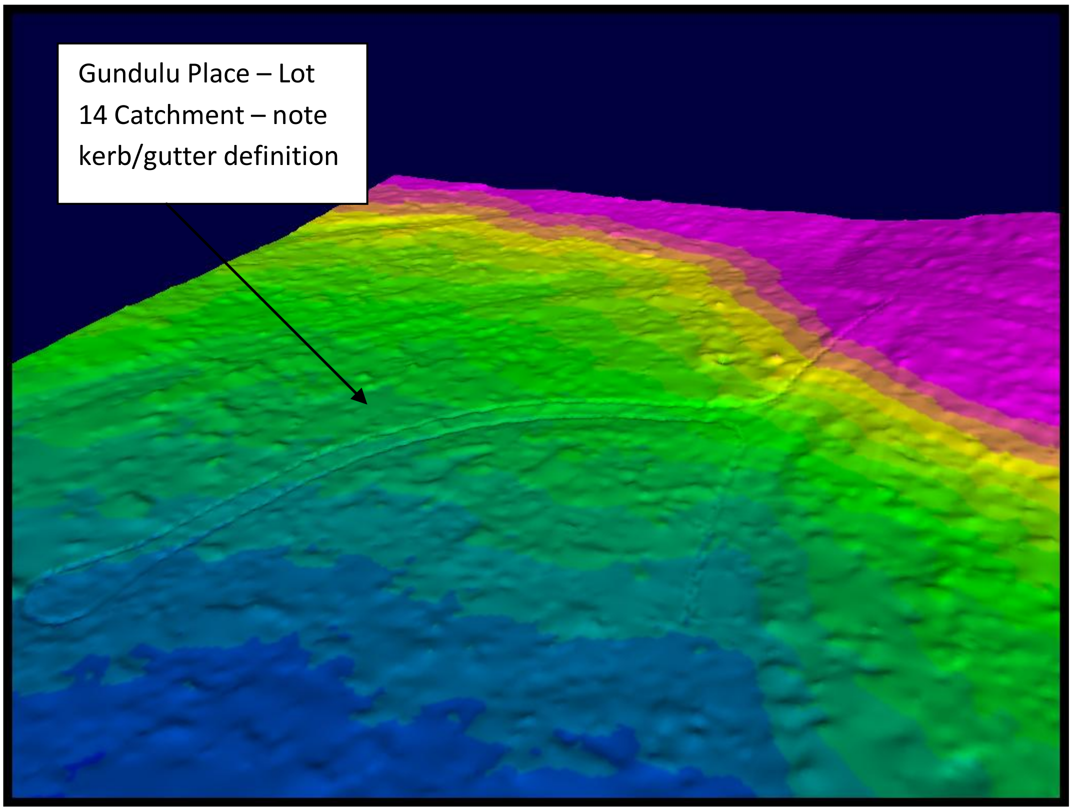

Figure 15: 3d view of Lot 14 Catchment and Surrounds

[15] It is common place in the research literature for writers to insist on the necessity of "stamping" or "burning" in the street or kerb/gutter flow feature.

The DTM did not represent houses. These were not, as they are in some applications, extruded from the surface. This seems appropriate given the large capacity of the drainage systems and the relatively small events modeled. Were surface flows that cut from street to street via house blocks occurring, it would be critical to include houses as objects capable of blocking overland flow. For similar reasons fencing between lots and inter-catchment flows were not considered.

Impervious/Pervious Map

Cadastral data was used to classify areas as pervious or impervious. The pervious/impervious map used is shown below in Figure 16. The intent here was to match overall average numbers based on aerial imagery analysis, not to produce a detailed representation at an intra lot scale. This integrated with the overall approach aimed at ease and speed of model setup utilising GIS.

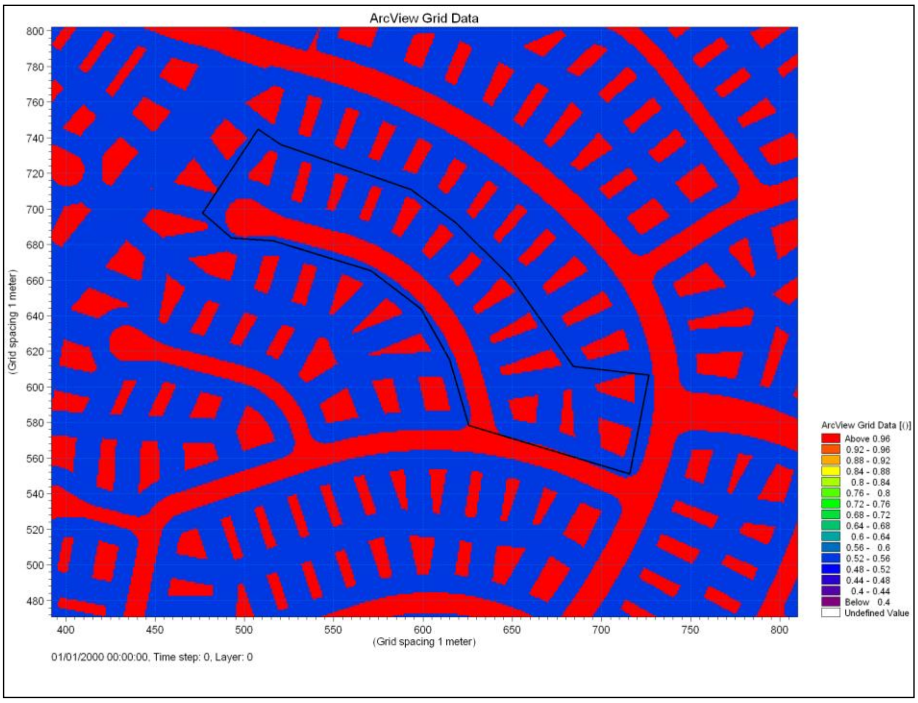

Figure 16: Impervious/Pervious Map – red areas are impervious

(note Lot 14 catchment outlined in black)

Impervious surfaces were comprised of roof and road, pervious surfaces yard. Roofs were created using a buffer based on the Cadastral lot boundary description iteratively until average roof area matched those found by analysis of aerial imagery. This use of GIS functionality avoided the need for time intensive digitization of individual lot impervious areas and was deemed to provide adequate data for model runs.

Road areas in the model were created solely on the basis of cadastral data (areas not within lots were defined as road). This exaggeration of road area (as vegetated easements approximately 3 m wide do exist in this Canberra suburb) was found to approximately equate with those impervious areas on lots not explicitly described (when compared to analysis of aerial imagery).

The area not defined as roof or road was specified as pervious yard.

Losses

The impervious/pervious spatial description was then utilized for producing a loss map.

Constant losses were utilized for impervious areas modeled (road and roof) whilst losses for pervious areas were varied on an event basis. It was deemed outside the scope of this application to utilize a continuing loss model such that the manipulation of losses per event was not necessary. This issue did relate to the need to avoid excessive run times and also a study focus on routing rather than loss models and the issue of antecedent conditions.

Note that proportional losses for the roof areas (in the 2d model) were set to 100%, as roof hydrological response was modeled in the 1d pipe model and roof areas were drained directly by pipe to municipal drainage.

Road proportional losses were set to zero, so all rainfall onto road was considered excess. This initial approximation was validated by model results and was also considered appropriate because:

- all but one of the six events used experienced rainfall immediately prior to the storm event rainfall burst; and
- initial losses for road surfaces are typically small and so omitting this process seemed unlikely to produce serious issues.

Proportional loss values for the yard were varied in all events in order to produce a peak and flow match. Proportional loss rates were used rather than an initial continuing loss approach for two reasons. The first was convenience, the second was that in previous studies, proportional loss approaches have been found to be superior to initial/continuing loss approaches in facilitating accurate emulation of a wide range of events Goyen (2000).

An example loss map is shown below in Figure 17.

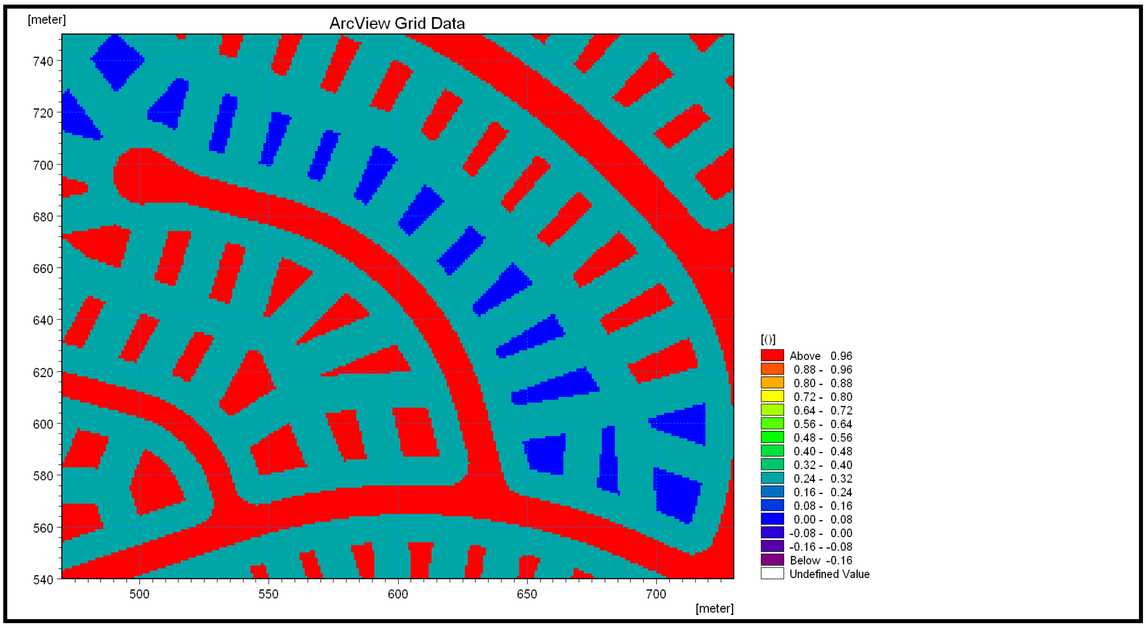

Figure 17: Loss Map for 2d Surface Flow Model

Roughness

Spatial roughness was defined utilising, as a base, the spatial map of pervious and impervious areas shown in Figure 16. Based on calibration, pervious areas were given a constant roughness of n = 0.04, impervious areas n = 0.008. This provided adequate results for the set of comparisons undertaken. Both values were arrived at by iterative runs during the road and yard calibration runs respectively. An example roughness map is shown below in Figure 18.

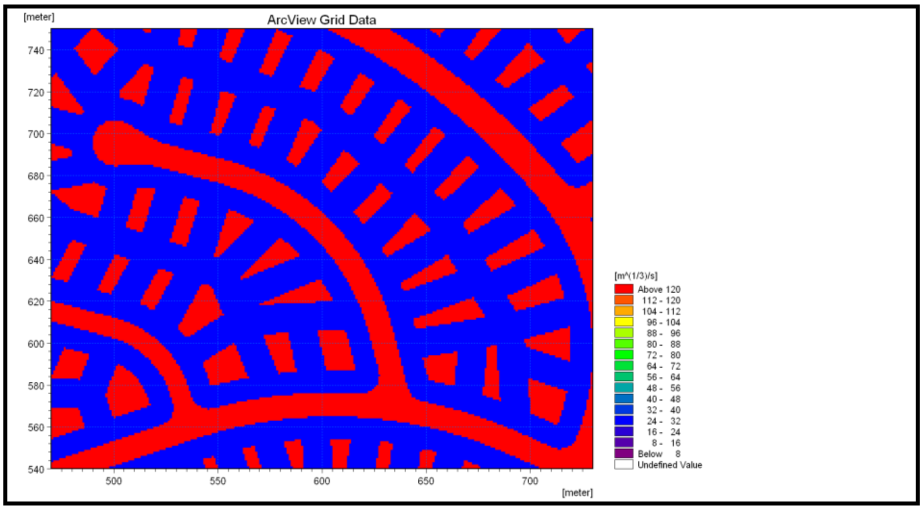

Figure 18: Roughness Map for 2d Surface Flow Model

The values found by iterative calibration do differ, especially for yard, with the recommended roughness values for surface flow shown in Table 4. Table 4 shows "typical" Mannings 'n' roughness values for different surface types with respect to overland flow. The comparison between these values and those identified by modeling will be discussed in Chapter 6.

Surface Type	Manning *n*	
	Recommended	Range
Concrete/Asphalt**	0.011	0.01-0.013
Bare Sand**	0.01	0.01-0.06
Bare Clay-Loam** (eroded)	0.02	0.012-0.033
Gravelled Surface**	0.02	0.012-0.03
Packed Clay**	0.03	0.02-0.04
Short Grass**	0.15	0.10-0.20
Light Turf*	0.20	0.15-0.25
Lawns*	0.25	0.20-0.30
Dense Turf*	0.35	0.30-0.40
Pasture*	0.35	0.30-0.40
Dense Shrubbery and Forest Litter*	0.40	0.35-0.50

* From Crawford and Linsley (1966) – obtained by calibration of Stanford Watershed Model.

** From Engman (1986) by Kinematic wave and storage analysis of measured rainfall runoff data.

Table 4: Values of Manning's 'n' for Overland Flow

Coupling the 1d and 2d models

Coupling of the models is achieved by designating inlet point location, inlet point invert level and inflow capacity factors which then dynamically utilise head at the inlet to compute actual inlet flow. Inlet invert levels were matched to final DTM values for the corresponding grid points. The specification of inlet capacity is defined by a series of values specified in the coupling file, an example of which is shown below in Table 5.

[MOUSE_COUPLING]	C	OLexp
COUPLINGMMSHE = '"123"'	0.63752	0.5
COUPLINGMMSHE = '"124"'	0.63752	0.5
COUPLINGMMSHE = '"125"'	0.63752	0.5
COUPLINGMMSHE = '"123b"'	0.63752	0.5
COUPLINGMMSHE = '"123c"'	0.63752	0.5
COUPLINGMMSHE = '"123d"'	0.63752	0.5
COUPLINGMMSHE = '"123f"'	0.63752	0.5

Table 5: Coupling Parameters

The pit inflow is calculated based on the following equation

$$Q = C.\Delta H^{\mathrm{OLexp}}$$

Where ΔH is the water depth at the inlet cell, C is conductance, and $OLexp$ is the overland flow exponent.

Conductance (C) is informed by the physical characteristics of the inlet pit in question.

In order to conserve mass, prior to transferring a given volume (as described based on the above equation for computing flow) the model checks that sufficient mass is available in the transfer cell. If not, then the input discharge is modified to equate to what volume is available.

In practice the events modelled did not produce surface flows at inlets that exceeded inlet capacity.

A specific representation of the interaction that occurs at a pit is given below in Figure 19. Here surface water that has routed via the 2d model is shown entering the pipe model. In Figure 20 surcharge from the 1d to the 2d model is depicted.

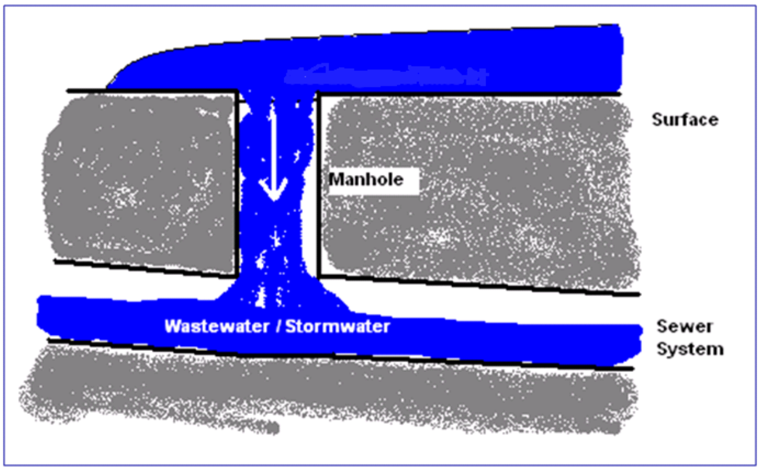

Figure 19: Surface flow entering the pipe system

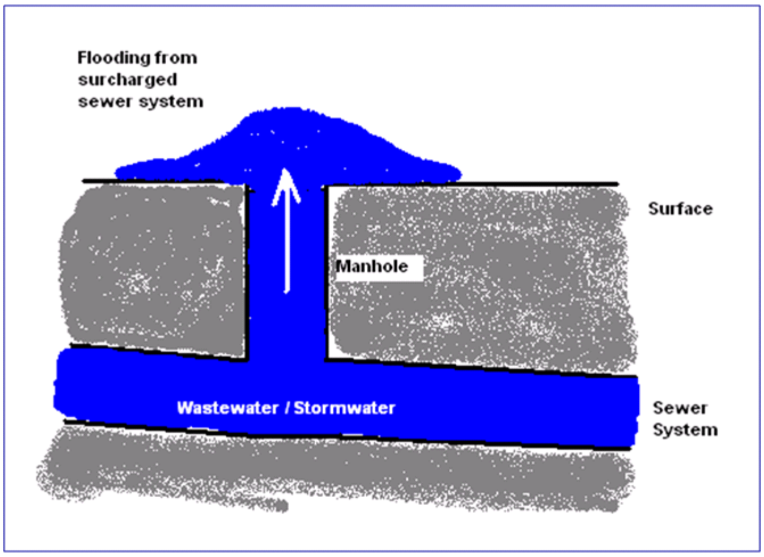

Figure 20: Flow surcharging from the pipe system to the surface

Coupling locations utilized are shown below in Figure 21. Note these locations were informed by field survey.

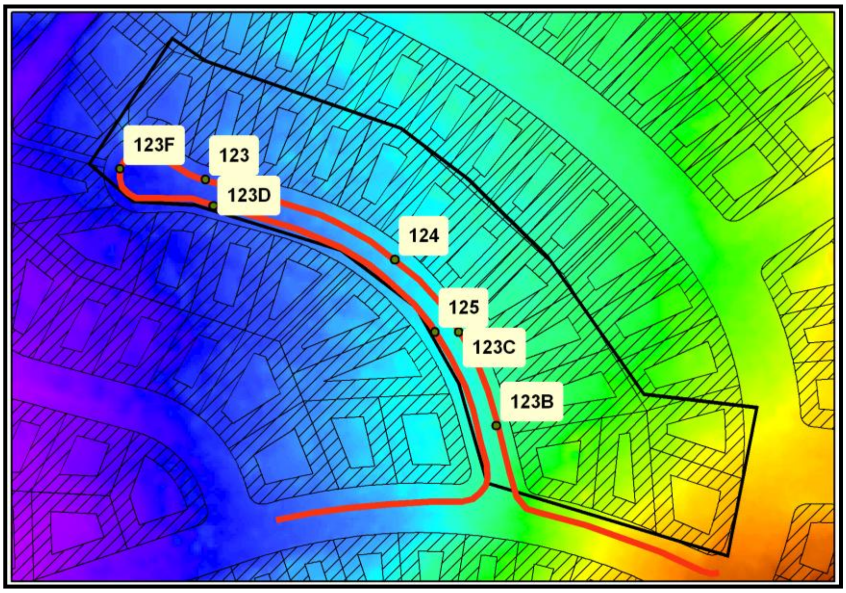

Figure 21: Lot 14 Catchment with Coupling Point Locations

Note that both models were run on the same time step of 0.2 seconds.

Mass Balance Check

A standard check carried out whenever numerical models are being run is a mass balance check. A mass balance check is especially useful when it comes to checking the overall performance of coupled modeling systems and spatial modeling systems. This is because if there is an overall mass issue, then further trouble shooting can be carried out to identify what might be the cause. However, if the overall mass balance check shows good conservation of mass then the model can be assumed to be running in a stable fashion throughout the model domain.

The procedure to carry out a mass balance check may vary depending on the specific modeling system being applied, however it can be stated generically as

$$Mass\ Balance\ \% = \frac{(Volume\ Output) - (Volume\ Input)}{Volume\ Input} \times 100$$

If the result is positive then the model has gained mass, if the result is negative then the model has lost mass. The level of acceptable mass error may vary depending on application, however a general rule of thumb is if the error is greater than 10% then model setup needs to be reviewed and/or run using a smaller time step.

In a model setup such as the one illustrated here the input volume will equate to the rainfall depth applied to the area taking into account any losses which may be applied to the input rainfall depth. The output volume can be calculated by adding the volumes that:

- flow out of the pipe model;
- remain in the pipe model at run end;
- flow out of the 2d surface domain; and
- remain in the 2d domain at end of model run.

Results of this check are presented in the next section.

A further check of the stability of the modeling system is to look at maximum velocity plots of the wider 2d area.

Chapter 5: Results

Preface to Results

- Result statistics are presented for each run within the hydrograph comparison plot. These statistics are as follows (example below is from Lot 12 calibration plot as presented in Figure 22);
- rf depth = 11.2 – the total rainfall depth for the event modeled in mm;
- PL = 33% - the proportional loss applied;
- R^2 = 0.91 – the coefficient of determination (R^2) is a statistic that will give some information about the goodness of fit of a model. In regression, R^2 is a statistical measure of how well the regression line approximates the real data points. An R^2 of 1.0 indicates that the regression line perfectly fits the data (Everitt, 2002);
- CE = 0.78 – The Nash-Sutcliffe model efficiency coefficient (CE) is used to assess the predictive power of hydrological models. Nash-Sutcliffe efficiencies can range from minus infinity to one. An efficiency of one corresponds to a perfect match of modeled discharge to the observed data. An efficiency of zero indicates that the model predictions are as accurate as the mean of the observed data, whereas a CE of less than zero occurs when the observed mean is a better predictor than the model. Essentially, the closer the model efficiency is to 1, the more accurate the model is (Nash et al, 1970);
- Qpe = -0.3% - the percentage difference in peak flow estimation between observed and modeled. A negative number indicates that the model underestimated peak discharge; and

- Ve = 2.6% - the percentage difference in volume. Again a negative number indicates that the model underestimated hydrograph volume.

The result statistics are then collected and presented en masse in Table 6 (Lot 12 runs) and Table 7 (Lot 14 runs).

Lot 12 (ROOF)

One event was used for calibration and this was the May 95 event shown below in Figure 22. Subsequently four events were used for validation without change of any parameters including losses. Goodness of fit statistics are summarised in Table 6. May 95 was used as the roof calibration event because it is a small rainfall event (total rainfall is 11.2 mm) and as such, it was envisaged to be used for road calibration. Given that road calibration would include roof response as well, it made sense to use the road calibration event for roof calibration. This ensured that the flow coming in from the roof was as close to reality as possible.

CALIBRATION

May 13 1995 11:00 - Peak Intensity approximately 36mm/h. Mean Intensity 30 mm/hr for 15 minutes. Intensities based on 5 minute data.

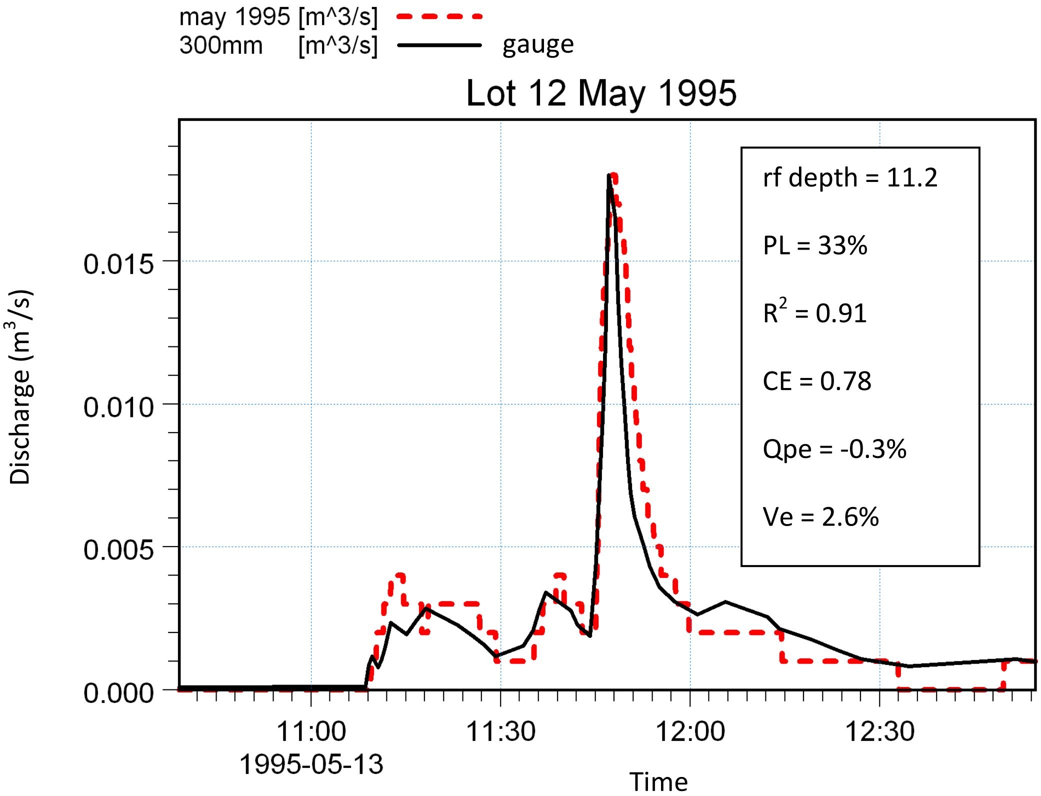

Figure 22: Lot 12 May 95 Calibration Plot

The model hydrograph is not responsive enough in the falling limb however generally fit is good to excellent. Volume and peak flow matches are excellent. A proportional loss of 33% was applied for this event, as were the kinematic channel characteristics used in all subsequent validation runs. Characteristics derived from calibration are shown in Chapter 4 Table 3.

Although overall flow volume fit is excellent it can be seen in the plot that some overestimation of volume exists in the area of peak to mid-flows whilst the base flow has been underestimated.

VALIDATION

January 3 1993 19:00

Peak Intensity 90 mm/h. Mean Intensity 60 mm/hr for 30 minutes. Intensities based on 5 minute data.

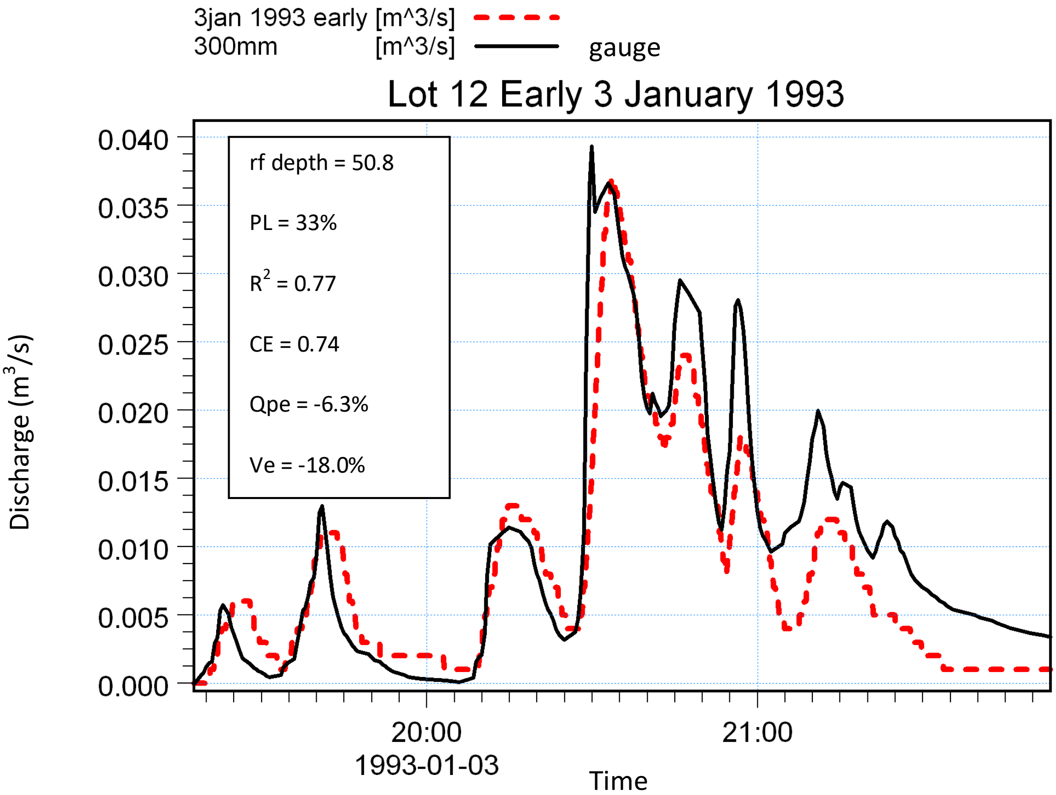

Figure 23: Lot 12 Early Jan 93 Validation Plot

Generally the rising limb is not steep enough whilst mid flows (less than 0.03 m^3/s) following the peak are underestimates and recession and base flow is too low. The overall shape fit however is good and the peak is well matched. Overall volume is significantly low at 18% under. Overall low to mid peaks are well matched prior to peak flow but then substantially underestimated post peak flow. It is interesting to note here that in reality the proportion of loss is highly variable even for this “impervious” surface i.e. terra cotta roof tiles.

It seems from the plot that the applied proportional loss in this case works to achieve a good peak match but then fails to produce an ideal volume match. This fits with the modeling priority of matching the peak flow and the cause of this is discussed in the next Chapter. The main cause is however that loss is not in reality constant over the event. Proportional losses are highest at event start and lowest towards event end.

January 3 1993 22:00

Peak Intensity 70 mm/h. Mean Intensity 35 mm/hr for 25 minutes. Intensities based on 5 minute data.

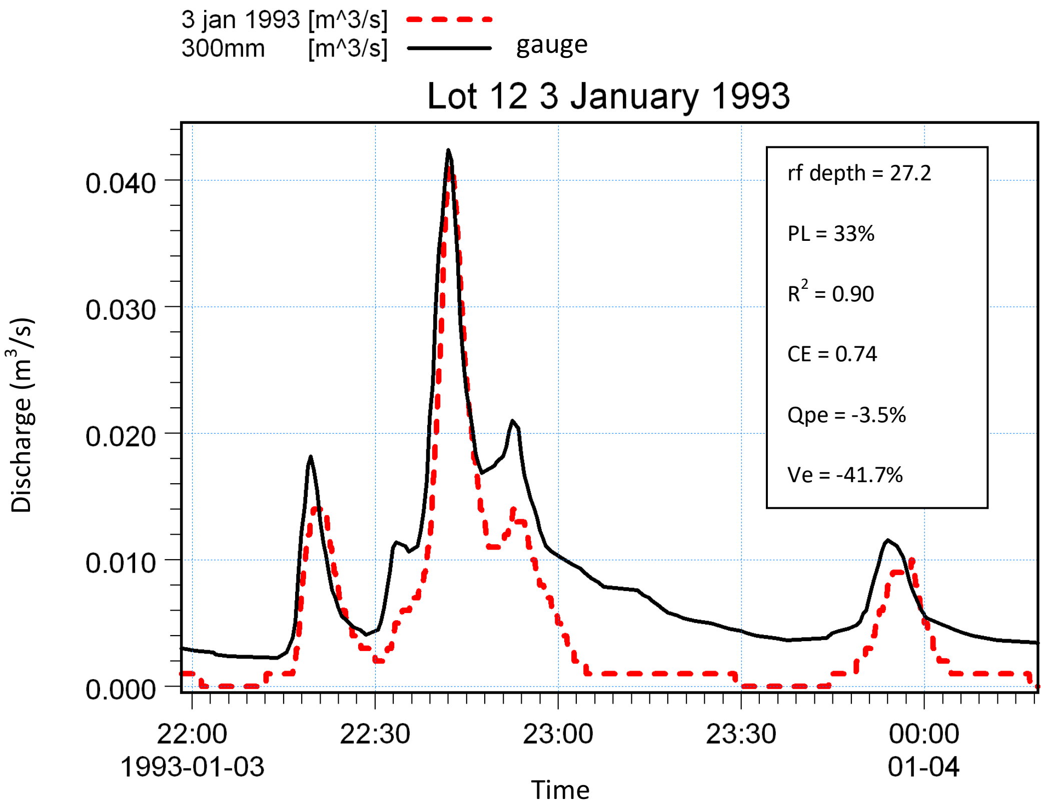

Figure 24: Lot 12 Jan 93 Validation Plot

Peak is very well matched however most flows below 0.02 m3/s are underestimated. Overall volume is very low at 42% underestimated. The rising and falling limbs of the main peak above 0.018 m3/s are excellently matched . As with the previous validation event, calibration parameters have achieved the main goal which is a match of peak flow, however volume is significantly underestimated.

April 5 1993 17:00

Peak Intensity 95 mm/h. Mean Intensity 95 mm/hr for 5 minutes. Intensities based on 5 minute data.

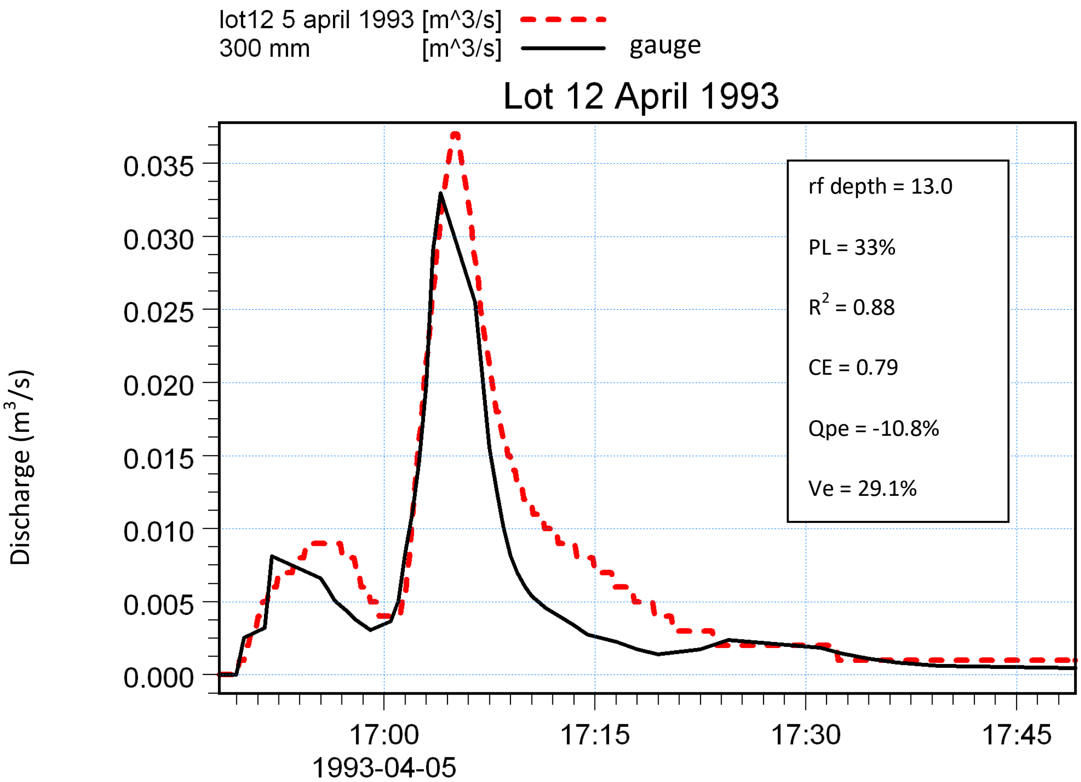

Figure 25: Lot 12 April 9 Time *Plot*

The general shape of hydrograph response is good, especially in the rising limb of the main peak however the peak flow is overestimated by approximately 10% and following this the recession is high and not steep enough for mid to low flows. The volume overestimate is substantial at 29%. It is noteworthy that this is the one event of the five modeled in Lot 12 which was dry beforehand. Obviously in reality losses for this event are higher than the herein applied constant proportional loss rate of 33%. This would explain not only the volume overestimate but also the peak error.

January 5 1995 19:00

Peak Intensity 90 mm/h. Mean Intensity 60 mm/hr for 30 minutes. Intensities based on 5 minute data.

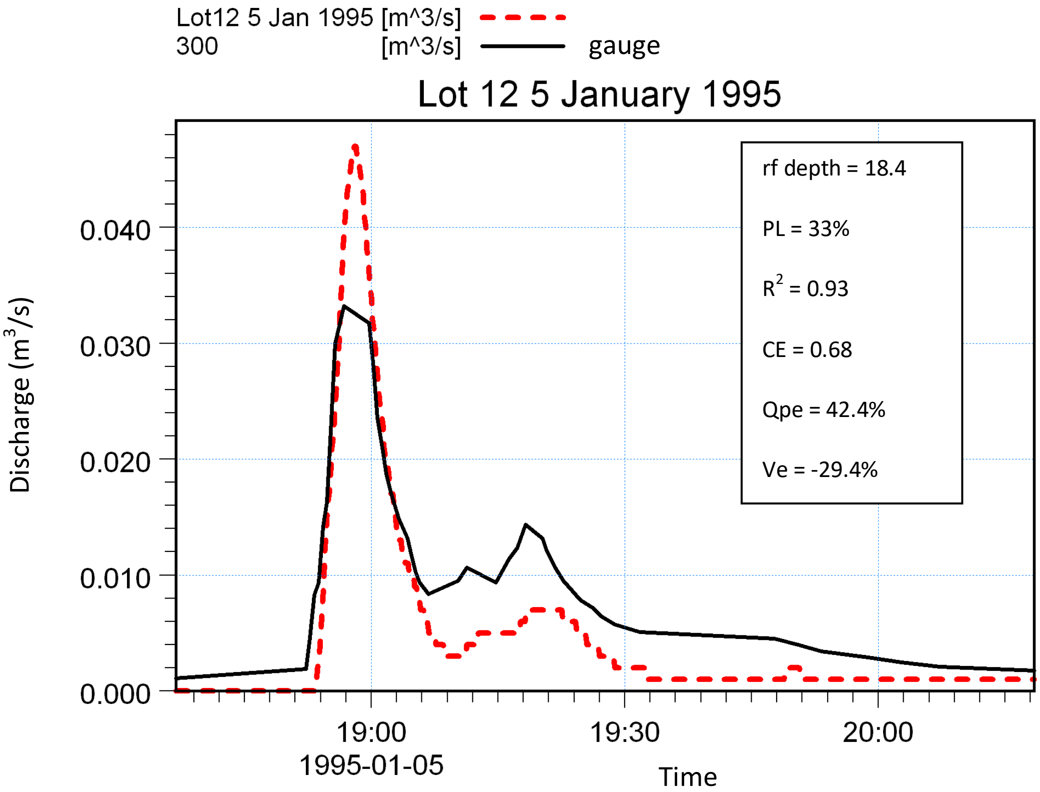

Figure 26: Lot 12 January 1995 Validation Plot

In the rising limb modeled response is a good match until a flow rate of 0.032 m^3/s is reached at which point the model continues to a peak of 0.047 m^3/s whilst the gauged flow peak is 0.033 m^3/s, an overestimate of 42%. Subsequent flow is significantly underestimated, although the falling limb match between 0.032 m^3/s and 0.01 m^3/s is excellent. The overall volume is 29% underestimated. One interpretation of the results could be that the roof system transferred peak flow into extended base flow, a process which the model failed to emulate. This will be further discussed in the next Chapter.

		Calibration	Validation	Validation	Validation	Validation
Fit Statistics	Units	May-95	Early Jan-93	Jan-93	Apr-93	Jan-95
		Lot 12	Lot 12	Lot 12	Lot 12	Lot 12
Correlation coefficient R^2		0.91	0.77	0.90	0.88	0.93
Max. positive difference	m^3/s	0.01	0.01	0.00	0.01	0.01
Max. negative difference	m^3/s	0.00	-0.02	-0.01	0.00	-0.01
Volume observed	m^3	15.76	91.21	67.42	17.20	36.62
Volume modelled	m^3	16.17	74.77	39.31	22.21	25.85
Volume error	%	2.59	-18.02	-41.69	29.13	-29.41
Peak observed value	m^3/s	0.01805	0.0395	0.0425	0.033	0.033
Peak modelled value	m^3/s	0.018	0.037	0.041	0.037	0.047
Peak error	%	-0.3	-6.3	-3.5	10.8	42.4
Goodness for Time to centroid	%	11.23	11.79	13.44	-2.06	17.34
Coefficient of Efficiency		0.78	0.74	0.74	0.79	0.68
Observed TS. Peak 1	m^3/s	0.02	0.04	0.04	0.03	0.03
Simulated TS. Peak 1	m^3/s	0.02	0.04	0.04	0.03	0.04
Magnitude Error. Peak 1	%	2.36	6.86	2.79	5.62	-17.38
Observed TS. Time to Peak 1	hours	0.79	1.18	0.54	0.25	0.10
Simulated TS. Time to Peak 1	hours	0.80	1.25	0.55	0.25	0.10
Timing Error. Peak 1		-2.60	-5.68	-1.91	0.00	0.00

Table 6: Lot 12 Fit Statistics Compiled

For four validation events:

- Mean Peak Error is +/- 15.8% (~ 7% if worst event is removed)
- Mean Volume Error is +/- 29.0% (~ 26% if worst event is removed)
- Mean R^2 is 0.87 (~ 0.9 if worst event is removed)
- Mean C.E is 0.74 (~0.76 if worst event is removed)

Lot 14 (ROOF, ROAD and YARD)

CALIBRATION ROAD

May 13 1995 11:00 am

Wet Prior to Event - Peak Intensity 36mm/h. Mean Intensity 30 mm/hr for 15 minutes.

Intensities based on 5 minute data. Event runoff coefficient is 0.35.

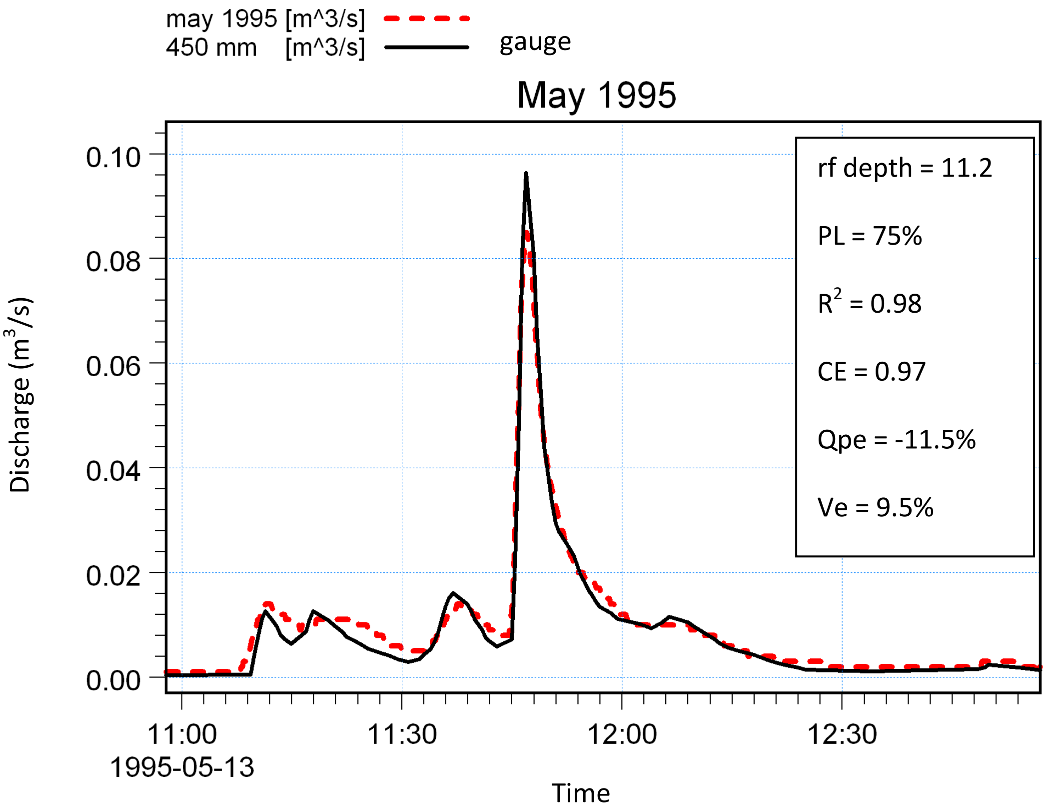

Figure 27: Lot 14 Validation Plot for Event May 1995

First impression is a very good shape but a 10% miss on peak (under). Given the fact that this is a small event roof results are worth referring to. Roof peak discharge is achieved. As such it seems that the failure to emulate the peak originates in road response. Overall volume for this event is slightly overestimated.

The match in shape is extremely good. The rising limb is an excellent match until a flow rate of 0.08 m^3/s and the falling is again excellent below 0.075 m^3/s. In order to convert the coarse (relatively) grid into the kerb/gutter system an n of 0.008 was used. Lower values for n actually reduced the R^2 and co-efficient of efficiency values as overall shape match declined and additionally, the impact on peak flow was not significant. Also it was considered that n values that were too low might detract from the models ability to model other validation events of various magnitudes, particularly larger. Overall however this was a good match which indicated that the roof/road flow components were being well emulated. Note n value is, compared to n values typically used for shallow flow over a smooth concrete surface, excessively smooth (see Table 4). Further sensitivity runs later conducted did however demonstrate the insensitivity to roughness. This result will be discussed at length in the next Chapter.

The very high proportional loss value used here is based on iterative calibration.

CALIBRATION YARD

January 5 1995 19:00

Wet Prior to Event - Peak Intensity approximately 90 mm/h. Mean Intensity 60 mm/hr for 30 minutes. Intensities based on 5 minute data. Event runoff coefficient is 0.56.

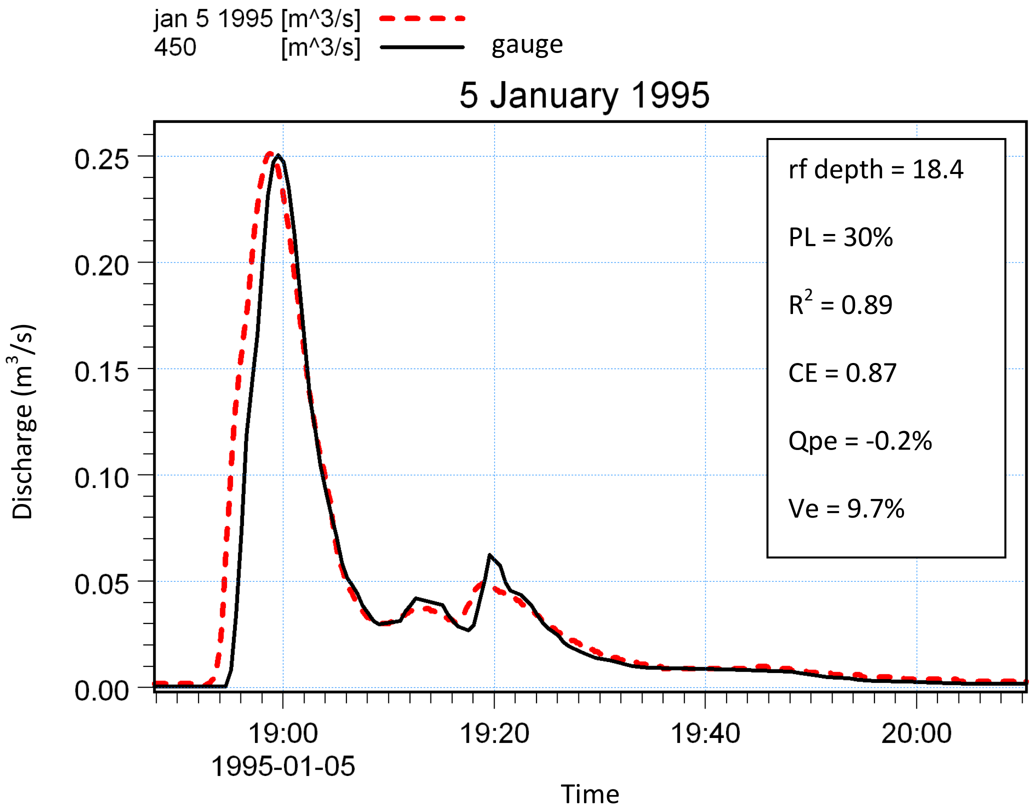

Figure 28: Lot 14 Calibration Plot for Event 5 Jan 1995

Overall shape match is excellent as is magnitude of peak. Volume is 10% high. Note the early start but excellent shape fit.

Falling limb is excellent, rising limb shape is excellent and base flow emulation is excellent. One minor peak not well matched.

Low base flow is explicable if roof is looked at where too much of the runoff volume is in peak and not enough in base flow.

The volume overestimate can be observed to correspond to the early rise of the model versus the observed.

Figure 29 shows a plot of surface flow in the vicinity of the inlet pit “123d” (refer to Figure 21 and inlet pit marked as “123d”). The 2d plot describes surface flooding as well as showing velocity vectors whilst the two plots above show rainfall and inlet discharge plotted against the lot 14 and lot 12 gauge results.

Figure 30 to Figure 33 are plots showing the entirety of Lot 14 at different points during the Jan 1995 event. Note that 2d plots of surface flooding are synchronized with rainfall and gauged hydrograph plots for Lot 14 and 12 gauges.

Figure 30 depicts a point in time prior to the event but after pre-wetting of the 2d surface. Due to the pre-wetting some flow can be seen moving to the kerb/gutter. Note also the defined nature of the kerb/gutter owing to the “stamping” of this feature into the DTM.

In Figure 31 rainfall has fallen, flow has moved to the kerb/gutter and there are some relatively high velocities apparent. Water is now entering the inlet pits. In Figure 32 water has drained off the 2d into the 1d and peak flow is now being gauged at the end of the Lot 14 trunk drainage system. In Figure 33 velocities are low, the event has passed and the residual water on the 2d surface looks similar to the pre-wet picture of Figure 30.

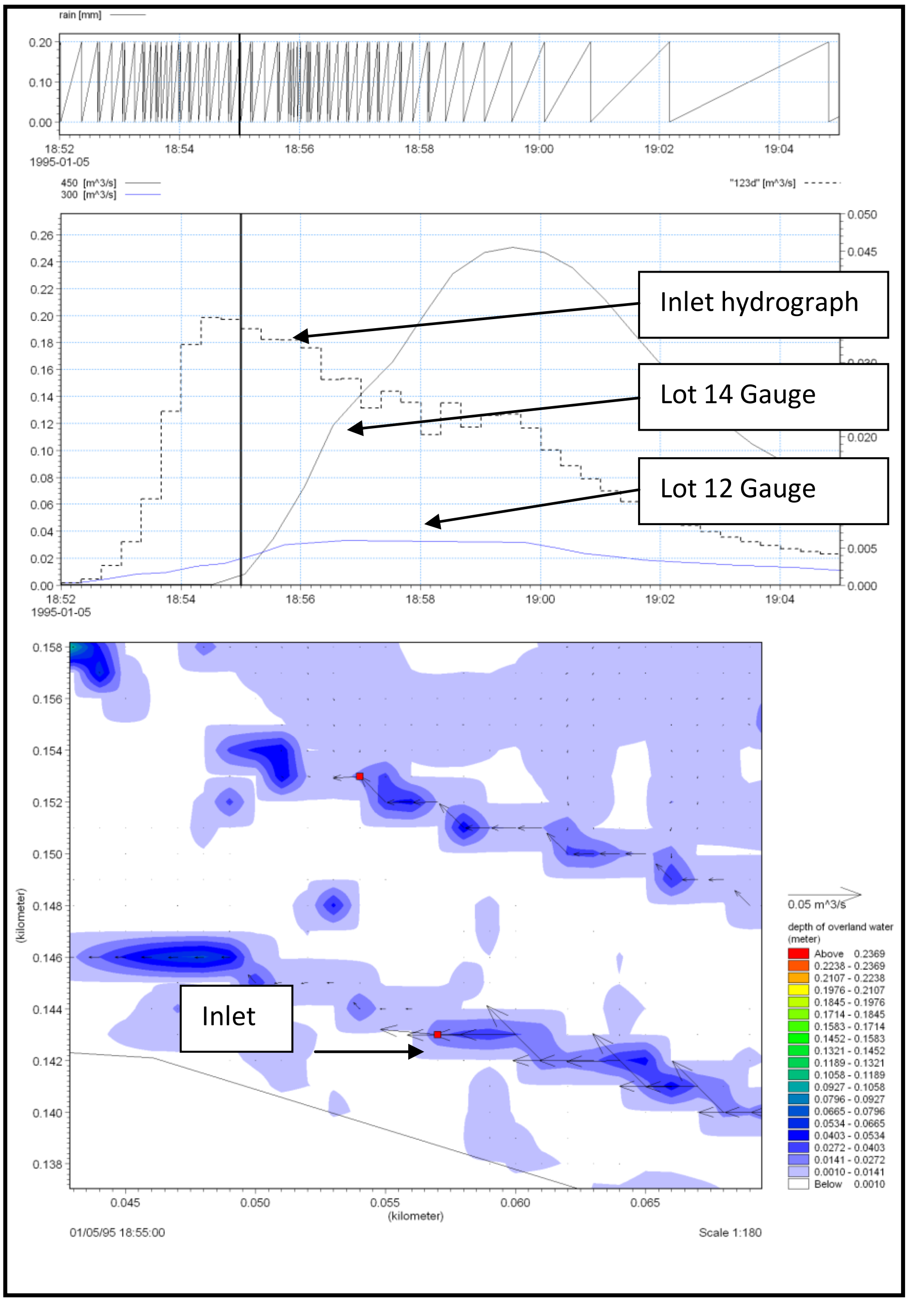

Figure 29: Rainfall, inlet hydrograph at pit and 2d plot of surface flow for 5 Jan 1995

In the above plot (from top to bottom) is shown 1. Rainfall, 2. the flow entering the inlet pit versus the gauged flow from both Lot 12 and Lot 14 and 3. A 2d plot indicating the depth of overland flow (with velocity vectors indicating the speed of that flow) and showing the location of the inlet pit.

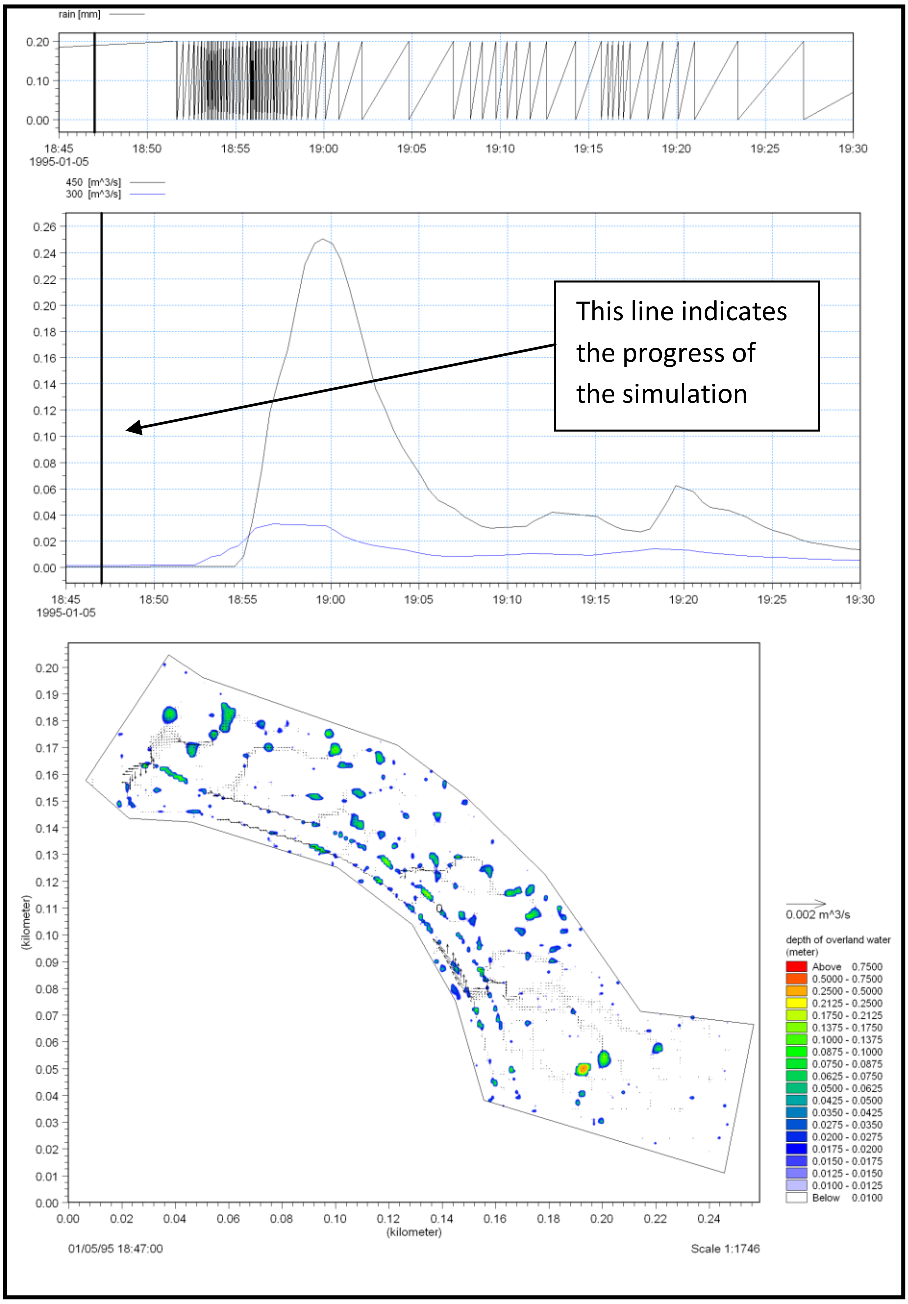

Figure 30: Prior to the event - note pre-wet water on ground

In the above plot (from top to bottom) is shown 1. Rainfall, 2. The gauged hydrograph for both Lot 12 and 14 (blue and black respectively) and 3. A 2d plot with inundation indicated via depth and also flow speed indicated via arrows. This is prior to the event and so the inundation indicated is due to the pre-wetting of the surface. Note the black line indicating the simulations progress in time.

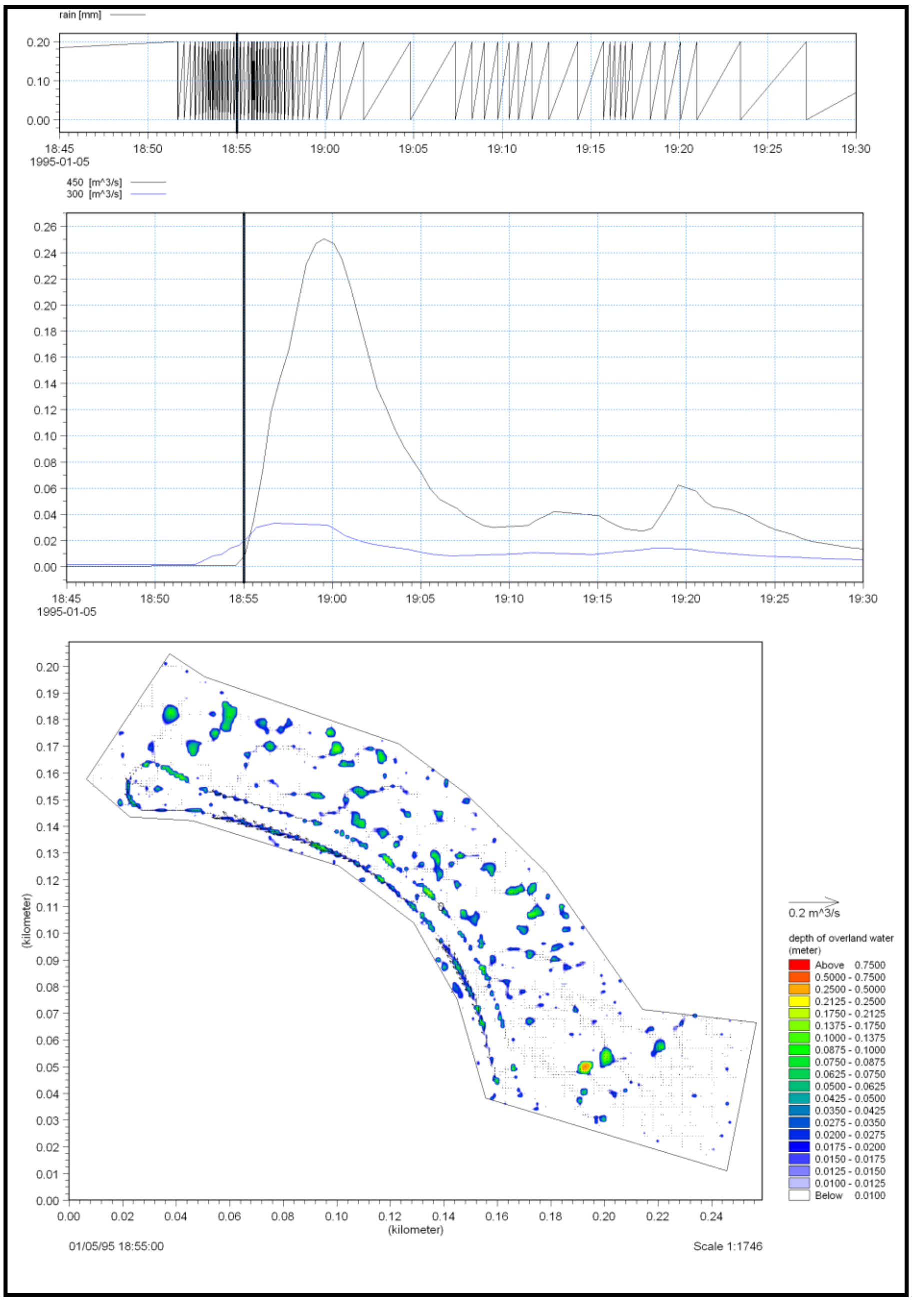

Figure 31: Immediately prior to hydrograph rise

In the above plot (from top to bottom) is shown 1. Rainfall, 2. The gauged hydrograph for both Lot 12 and 14 (blue and black respectively) and 3. A 2d plot with inundation indicated via depth and also flow speed indicated via arrows. As is indicated by the vertical black line, the simulation is now half way through the main rainfall burst. Some accumulation of flow in the kerb drain can be noted.

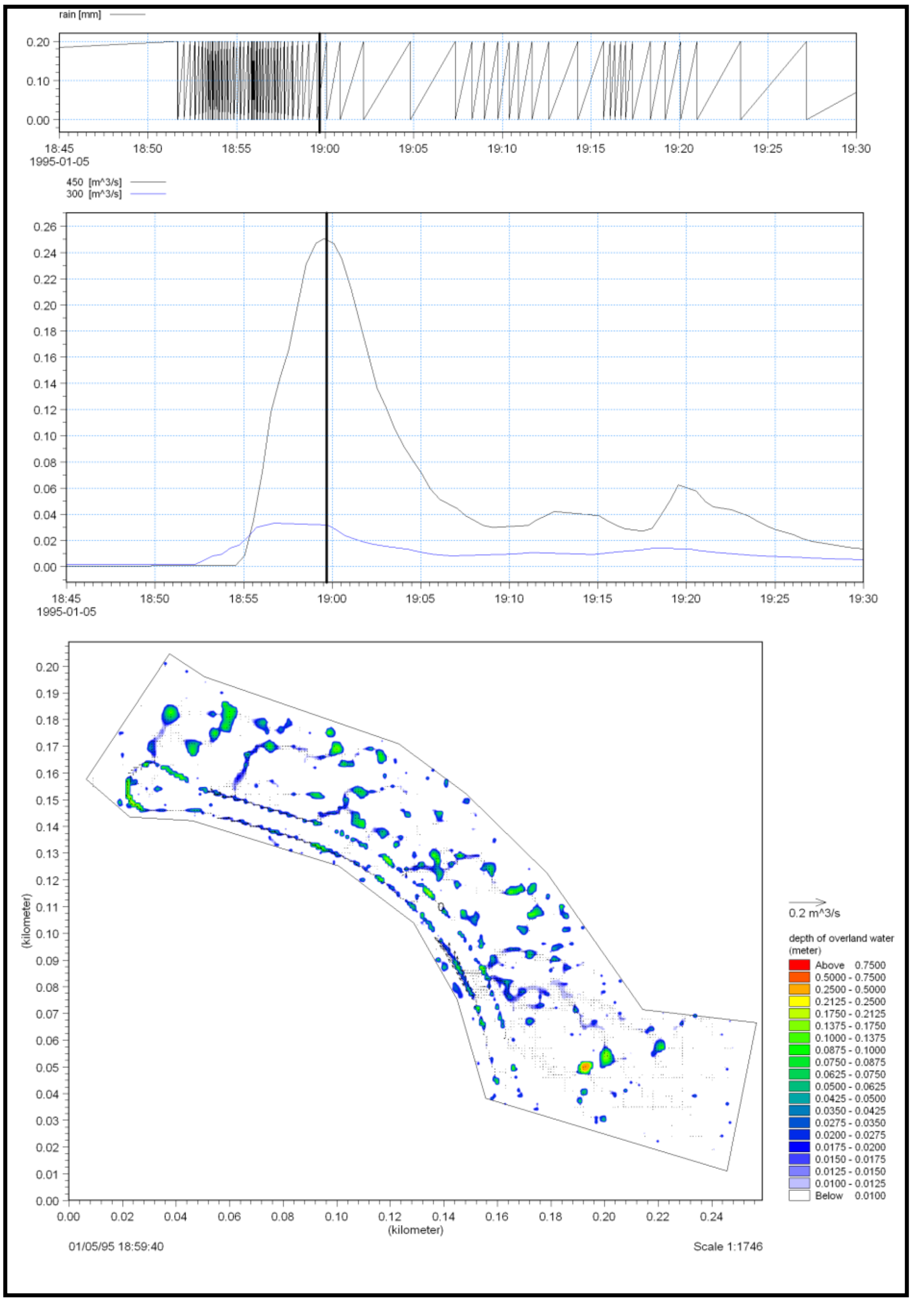

Figure 32: Overland flow shown at time of peak flow Lot 14

In the above plot (from top to bottom) is shown 1. Rainfall, 2. The gauged hydrograph for both Lot 12 and 14 (blue and black respectively) and 3. A 2d plot with inundation indicated via depth and also flow speed indicated via arrows. As is indicated by the vertical black line, the simulation is now past the main rainfall burst and at the peak of the event as gauged. Overland flow to the kerb drain can be noted.

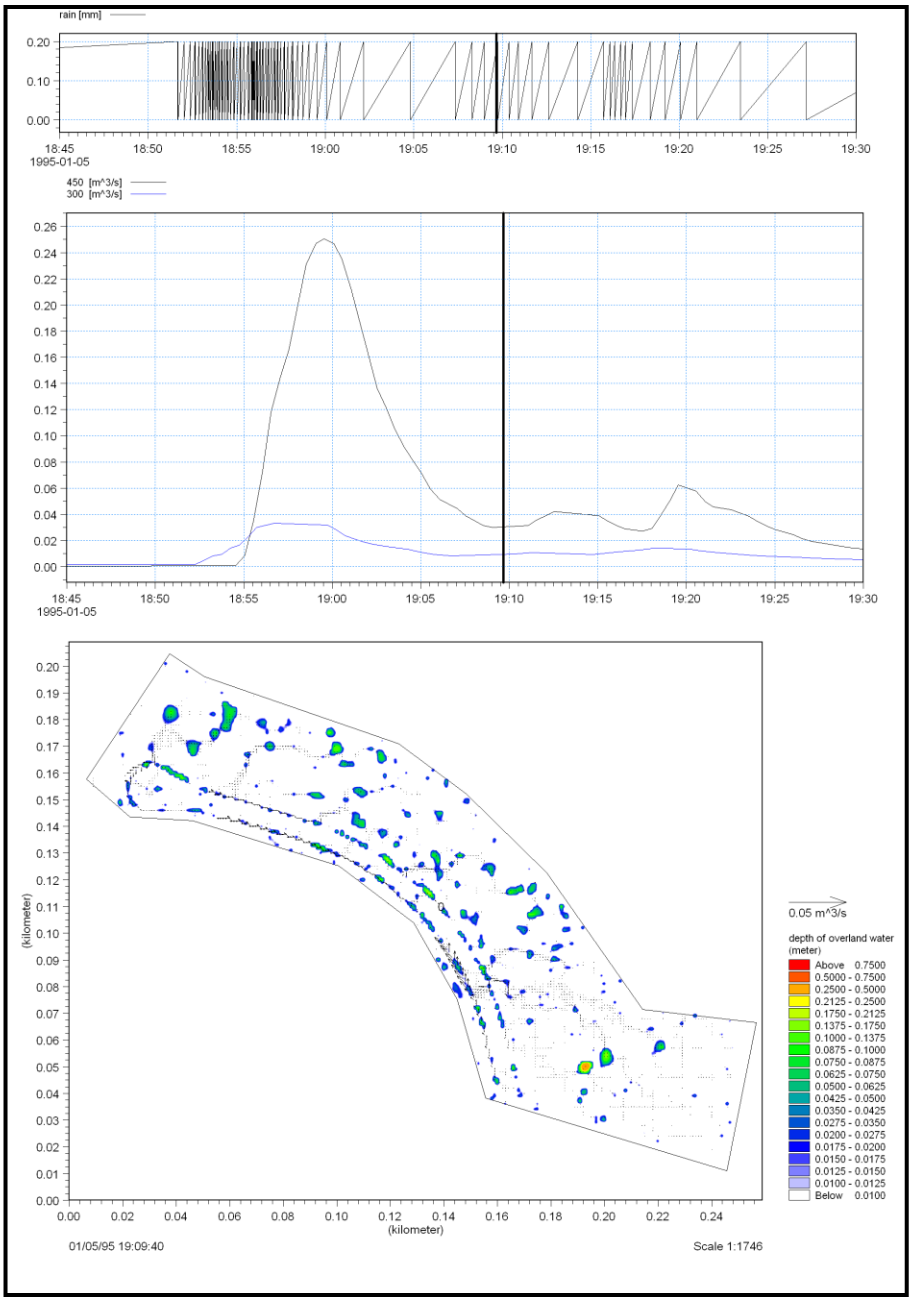

Figure 33: Following Main Peak

In the above plot (from top to bottom) is shown 1. Rainfall, 2. The gauged hydrograph for both Lot 12 and 14 (blue and black respectively) and 3. A 2d plot with inundation indicated via depth and also flow speed indicated via arrows. As is indicated by the vertical black line, the simulation is now past the gauged peak and mapped depth can be seen to be ebbing.

VALIDATION

Early January 3 1993 19:00

Wet Prior to Event - Peak Intensity 90 mm/h. Mean Intensity 60 mm/hr for 30 minutes.

Intensities based on 5 minute data. Event runoff coefficient is 0.49.

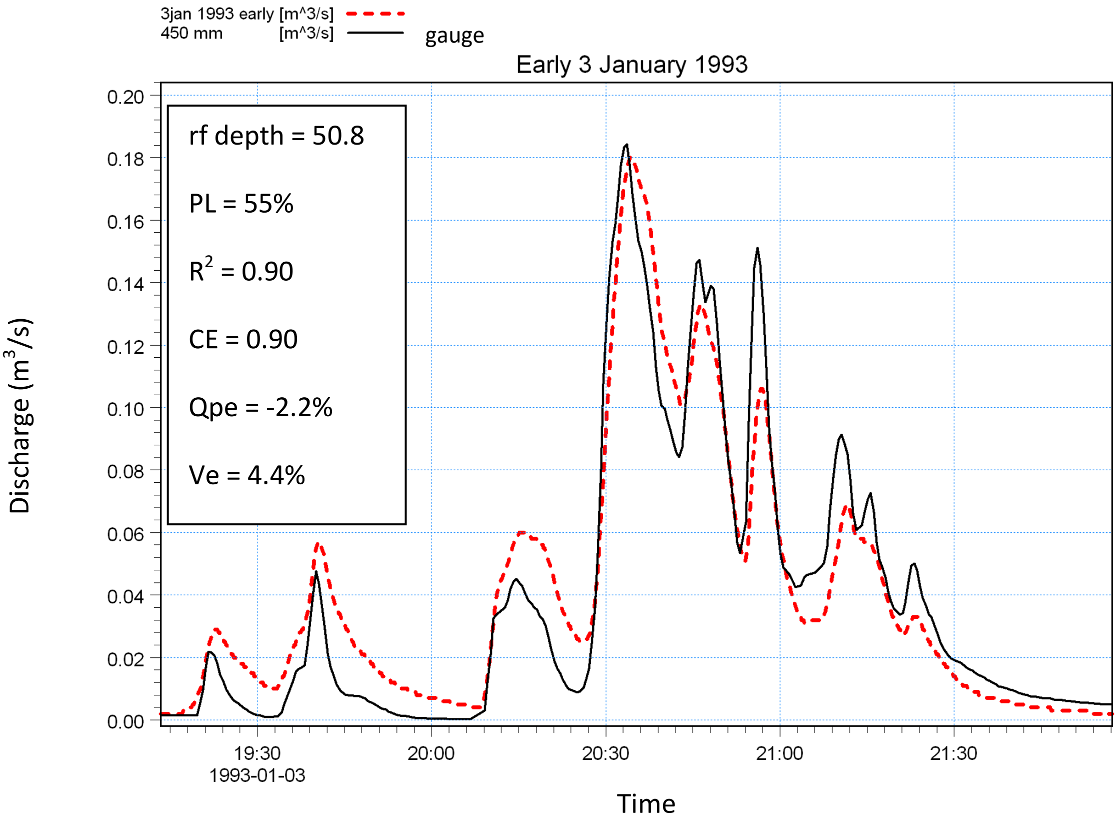

Figure 34: Lot 14 Validation Plot for Event Early Jan 1993

First impression is that response is good, overall shape is good, rising and falling limbs are good and peak match is very good. Volume is 4% out. It can be observed that minor peaks are overestimated prior to peak flow but then underestimated following the peak.

A higher proportional loss value was used in this event, which has the greatest rainfall depth of all events modelled.

January 3 1993 22:00

Wet Prior to Event - Peak Intensity 70 mm/h. Mean Intensity 35 mm/hr for 25 minutes.

Intensities based on 5 minute data. Event runoff coefficient is 0.71.

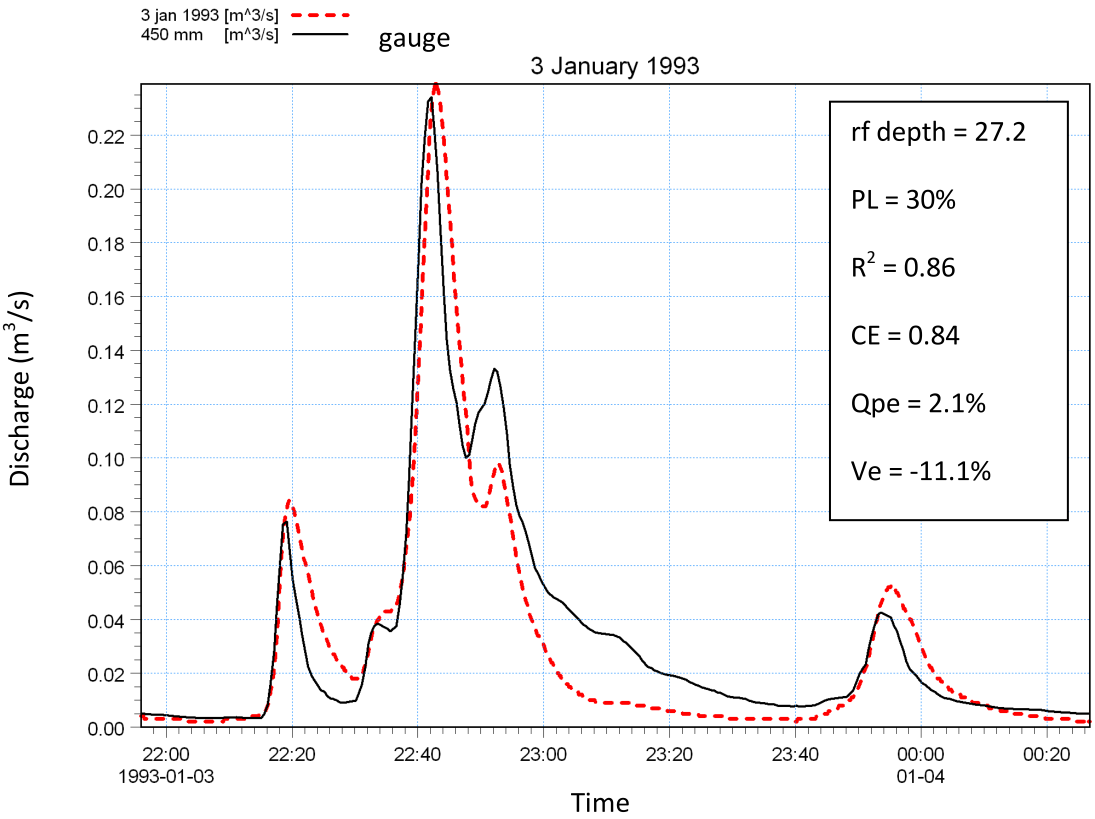

Figure 35: Lot 14 Validation Plot for Event Jan 1993

Peak match is excellent and as per last plot initial peaks prior to main peak are overestimated and peak immediately following main peak is underestimated (as is recession). Overall volume is low by approximately 10% and it is noteworthy that there was a good deal of rainfall prior to this event.

This event used the same parameter set as the Jan 5 1995 event in that proportional loss value used is 30%.

April 5 1993 17:00

Dry Prior to Event - Peak Intensity 95 mm/h. Mean Intensity 95 mm/hr for 5 minutes.

Intensities based on 5 minute data. Event runoff coefficient is 0.31.

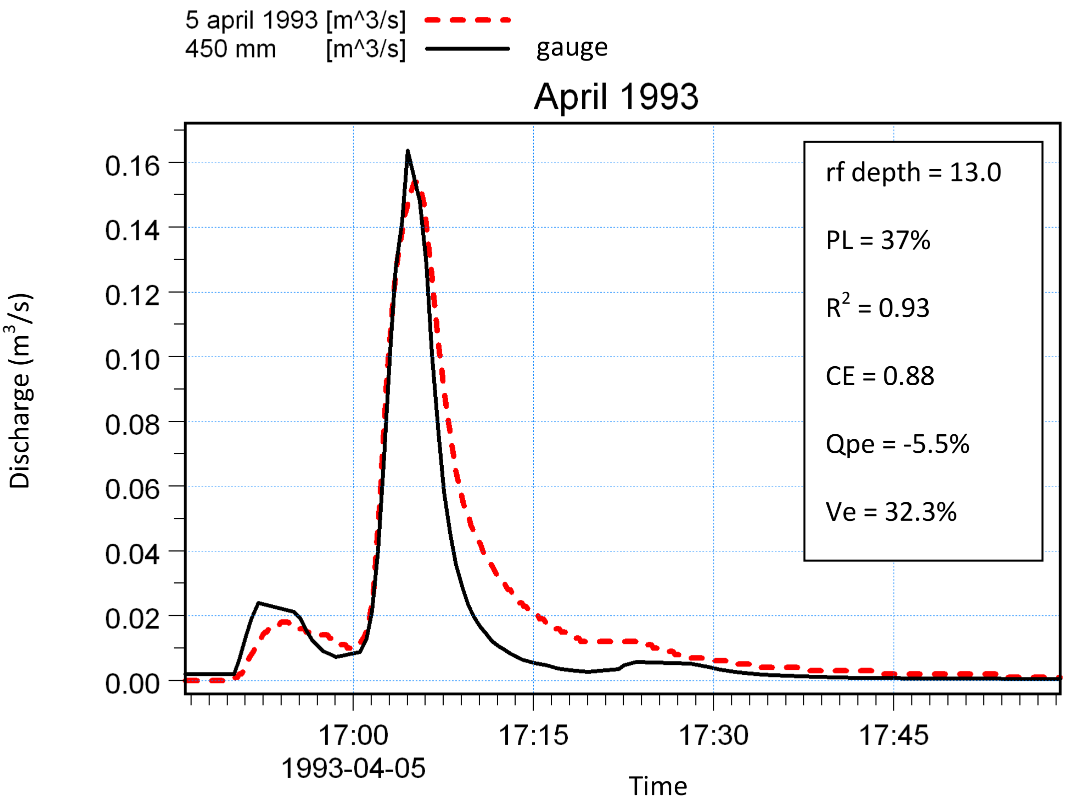

Figure 36: Lot 14 Validation Plot for Event April 1993

In order to best match shape peak match was slightly sacrificed. Volume is too high by 30%. There is too much base flow. Recession limb is overly lagged i.e. not peaky enough. Note this is a relatively small event compared to others in the set.

A slightly higher proportional loss value was used than 30% at 37%, however this event was the only in the entire set which did not have rainfall prior to the event, i.e. the antecedent condition was dry.

January 28 1995 18:00

Wet prior to event - Peak Intensity 85 mm/h. Mean Intensity 70 mm/hr for 15 minutes.

Intensities based on 5 minute data. Event runoff coefficient is 0.87.

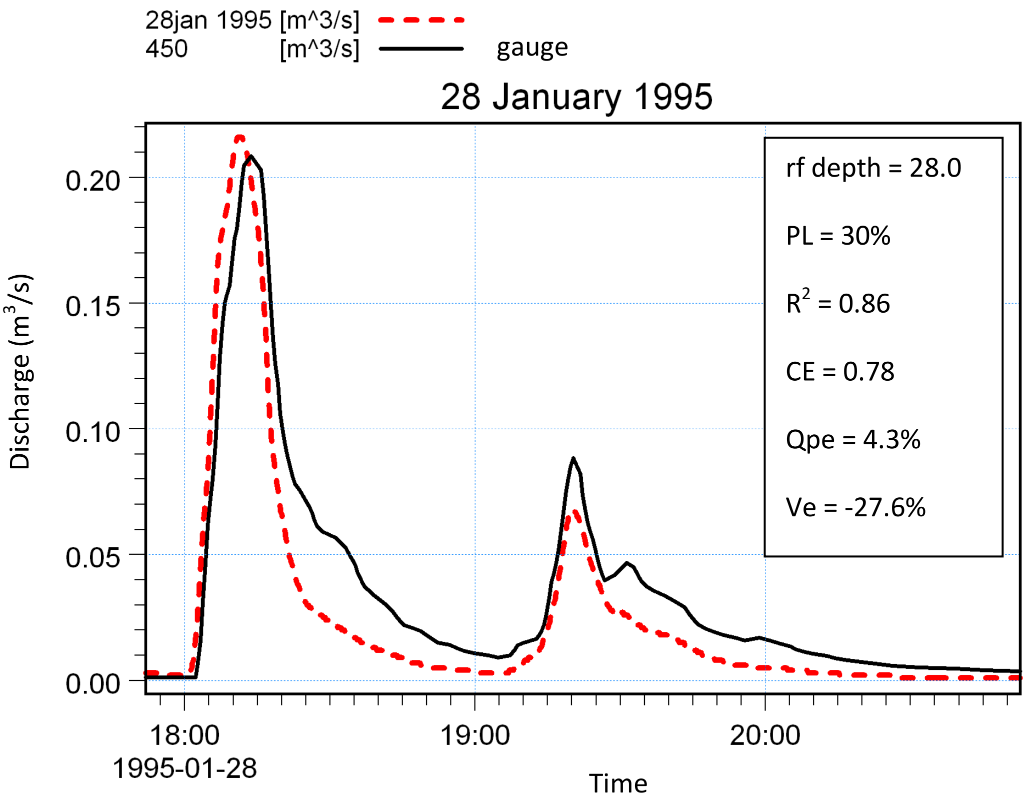

Figure 37: Lot 14 Validation Plot for Event Jan 28 1995

Response is good, peak is okay, base flow is too low and rising limb is too steep as is falling limb. Volume is 27% too low.

This event used the same parameter set as the Jan 5 1995 event in that proportional loss value used is 30%.

Summary of Results for Lot 14

Fit Statistics	Units	5jan-95	early jan 93	Jan-93	Apr-93	28jan-95	01-May-95
Correlation coefficient R^2		0.89	0.90	0.86	0.93	0.86	0.98
Max. positive difference	m^3/s	0.10	0.03	0.06	0.03	0.05	0.01
Max. negative difference	m^3/s	-0.02	-0.06	-0.04	-0.02	-0.05	-0.01
Volume observed	m^3	159.60	380.86	297.51	62.69	372.97	60.53
Volume modelled	m^3	175.02	397.41	264.47	82.96	269.98	66.29
Volume error	%	9.66	4.35	-11.11	32.34	-27.62	9.51
Peak observed value	m^3/s	0.25	0.18	0.23	0.16	0.21	0.10
Peak modelled value	m^3/s	0.25	0.18	0.24	0.15	0.22	0.08
Peak error	%	-0.2	-2.2	2.1	-5.5	4.3	-11.5
Goodness for Time to centroid	%	17.15	10.20	5.52	-20.76	25.63	6.58
Coefficient of Efficiency		0.87	0.90	0.84	0.88	0.78	0.97
Observed TS. Peak 1	m^3/s	0.25	0.18	0.23	0.16	0.21	0.10
Simulated TS. Peak 1	m^3/s	0.25	0.18	0.24	0.15	0.22	0.08
Magnitude Error. Peak 1	%	0.12	2.49	-1.54	6.79	-3.71	12.45
Observed TS. Time to Peak 1	hours	0.25	1.33	0.65	0.26	0.24	0.78
Simulated TS. Time to Peak 1	hours	0.24	1.35	0.67	0.27	0.20	0.78
Timing Error. Peak 1		9.33	-1.35	-2.66	-6.91	21.51	0.00

Table 7: Lot 14 Calibration, Validation and Sensitivity Run Statistics

For four validation events

- Mean Peak Error is +/- 3.5%
- Mean Volume Error is +/- 18.9%
- Mean R^2 is 0.89
- Mean C.E is 0.85

SENSITIVITY RUNS

All sensitivity runs were run using the January 5 1995 event.

No pre-wet of yard or road and no break line (green solid line)

Initial response is good however at 0.05 m^3/s (where roof peak cuts out) match becomes very poor. Rising limb above this flow level is too flat, peak is 40% of actual, base flow is too low and second and third peaks are indistinct. Overall volume is low by 55%.

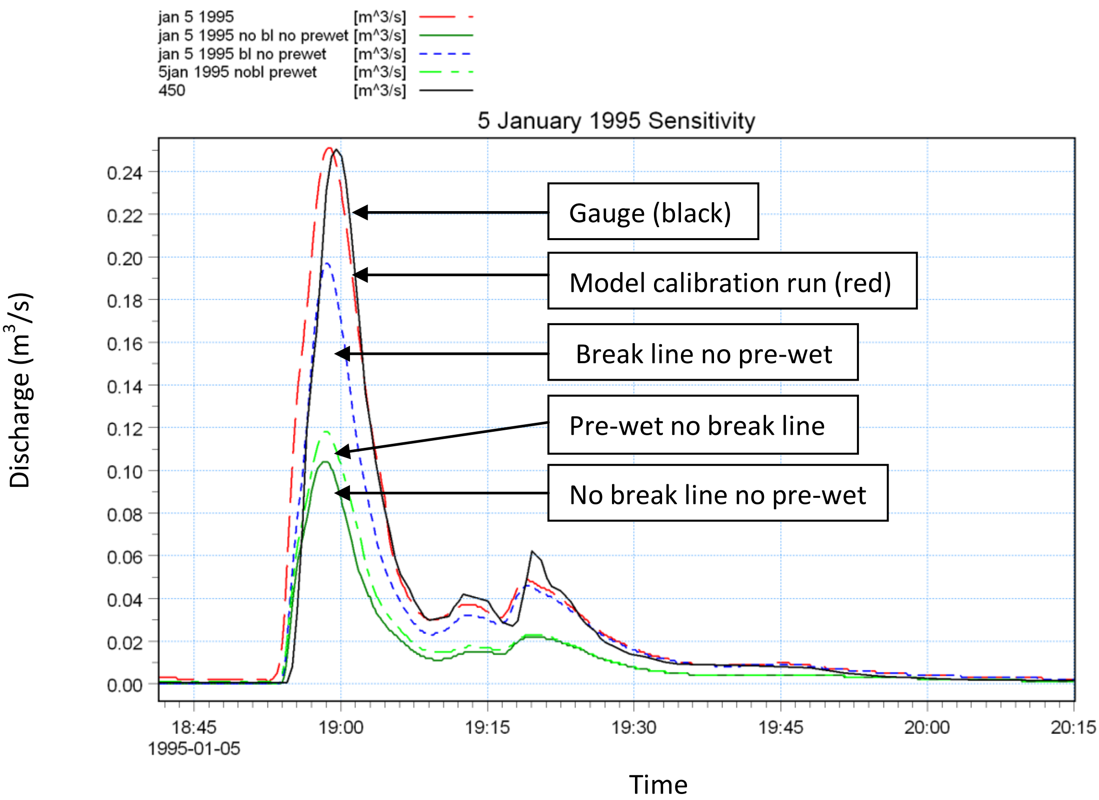

Figure 38: Sensitivity to Pre-Wet and Break line

No pre-wet of yard or road (blue dash line)

Volume is much better compared to the “no break line” run at only 14% too low. Peak representation is also a relative improvement however is low at 15% underestimated. General shape is good. This highlights the break lines effectiveness in creating a continuous hydraulic path similar to kerb and gutter. It also highlights however that “holes” in the break lined path, create artificial storage.

Pre-wet but no break line (green dot dash line)

Match is very similar to first sensitivity run and this highlights the importance (most critical) or the break line.

Fit Statistics	Units	05-Jan-95	05-Jan-95	05-Jan-95
		no prewet or breakline	no prewet	no breakline
Correlation coefficient R^2		0.83	0.91	0.86
Max. positive difference	m^3/s	0.03	0.05	0.04
Max. negative difference	m^3/s	-0.16	-0.08	-0.15
Volume observed	m^3	159.60	159.60	159.60
Volume modelled	m^3	71.43	136.61	81.57
Volume error	%	-55.24	-14.40	-48.89
Peak observed value	m^3/s	0.25	0.25	0.25
Peak modelled value	m^3/s	0.10	0.20	0.12
Peak error	%	-58.5	-21.4	-53.0
Goodness for Time to centroid	%	15.18	6.92	18.30
Coefficient of Efficiency		0.33	0.85	0.46
Observed TS. Peak 1	m^3/s	0.25	0.25	0.25
Simulated TS. Peak 1	m^3/s	0.10	0.20	0.12
Magnitude Error. Peak 1	%	58.45	21.39	53.01
Observed TS. Time to Peak 1	hours	0.23	0.25	0.25
Simulated TS. Time to Peak 1	hours	0.21	0.23	0.23

Table 8: Sensitivity to Break line and Pre-Wet Statistics

Relatively Rough Road (n of 0.013 as compared to final adopted value of 0.008)

Similar to calibration result except that less peaky and peak flow is further underestimated (18% compared to 12% for n = 0.008). Based on this couldn't say that the result is all that sensitive to road roughness, although it does help. Change in roughness of 67% (0.013 to 0.008) produces a 6% improvement in peak match, with shape unaffected (slight improvement of 0.01 for both R^2 and CE).

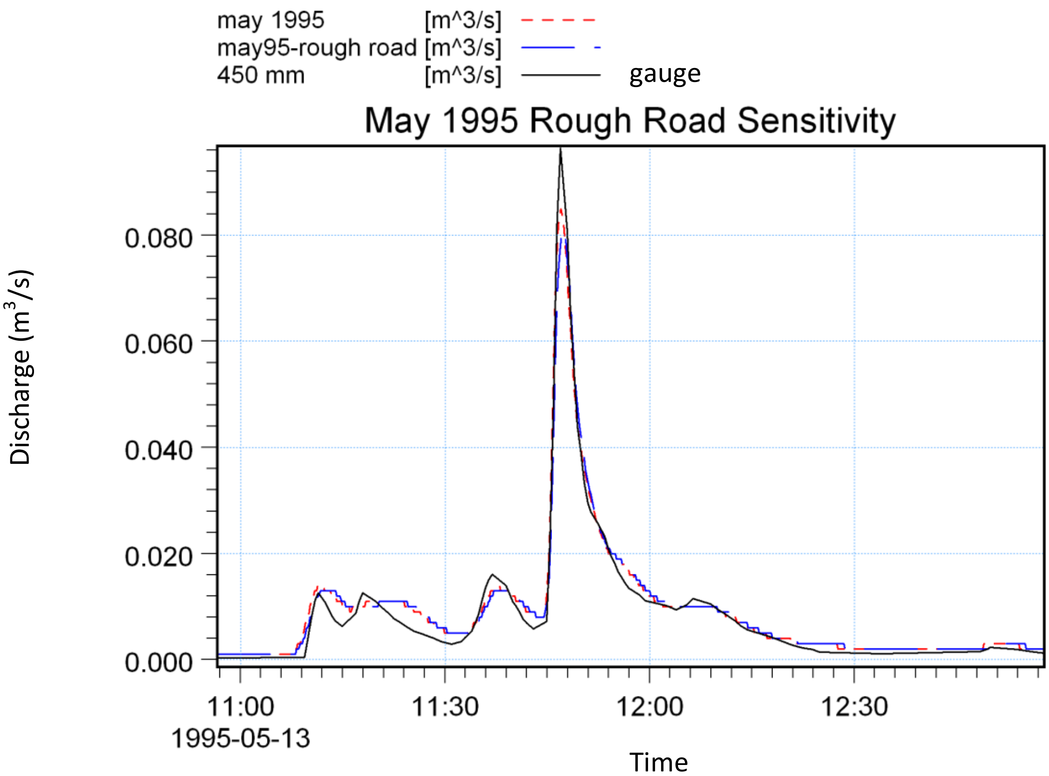

Figure 39: Sensitivity to roughness change in road

Rough Yard (n = 0.1 rather than 0.04)

Significant impact on peak (18% worse) and shape declines. Note that attenuation occurs in that peak flow is transferred to the recession limb.

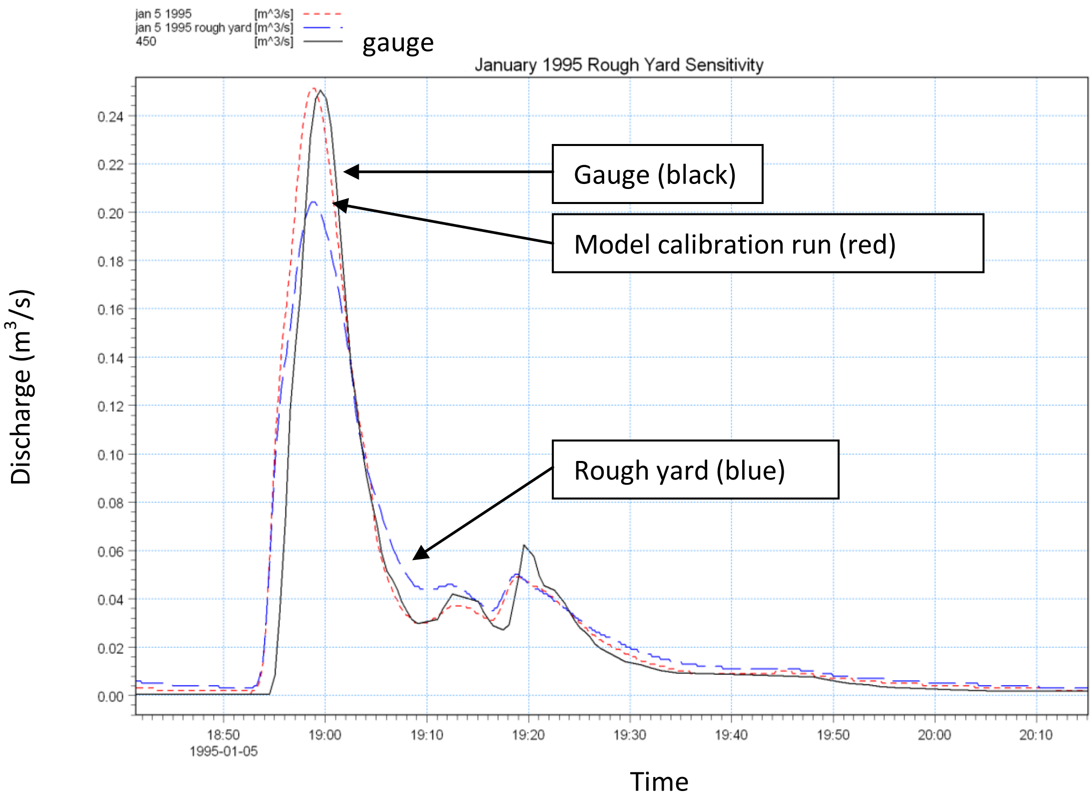

Figure 40: Sensitivity to roughness Change in Yard

Fit Statistics	Units	05-Jan-95	01-May-95
		Yard M=10	Road M=75
Correlation coefficient R^2		0.89	0.97
Max. positive difference	m^3/s	0.09	0.01
Max. negative difference	m^3/s	-0.06	-0.02
Volume observed	m^3	159.60	59.65
Volume modelled	m^3	174.53	65.03
Volume error	%	9.36	9.02
Peak observed value	m^3/s	0.25	0.10
Peak modelled value	m^3/s	0.20	0.08
Peak error	%	-18.8	-18.4
Goodness for Time to centroid	%	9.41	5.14
Coefficient of Efficiency		0.88	0.96
Observed TS. Peak 1	m^3/s	0.25	0.10
Simulated TS. Peak 1	m^3/s	0.20	0.08
Magnitude Error. Peak 1	%	18.77	18.37
Observed TS. Time to Peak 1	hours	0.12	0.68
Simulated TS. Time to Peak 1	hours	0.11	0.68

Table 9: Sensitivity to Roughness Statistics

Mass Balance Check

To confirm that the model is running well and no mass is being made or lost a mass balance check was carried out, the methodology for which was outlined in Chapter 4.

Event					
	5-Jan-95				
Rainfall Depth					
	48.4	mm (inclusive of pre-wet)			
Catchment					
	Total	15,150	m2		
	Pervious	9,958	m2		
	Road	2,598	m2		
	Roof	2,538	m2		
Proportional Losses					
	30%	for pervious			
Input Flow					
	Rainfall Volume		1d	30.9	m^3
	Rainfall Volume		2d	463.1	m^3
	sub-total			494.0	m^3
Output Flow					
	Final Volume		1d	0.5	m^3
	Final Volume		2d	46.1	m^3
	2d Outflow		2d	71.8	m^3
	1d Outflow		1d	379.8	m^3
	sub-total			498.1	m^3
Volume Difference					
				1%	gain

Table 10: Mass Balance Check for 5 Jan 1995 Event

The finding then is that 1% mass is being made. This is trivial and as such the model can be said to be performing adequately with respect to mass conservation which in turn implies that it is running in a stable fashion. Graphical results also show reasonableness with respect to velocities and flood depths.

Chapter 6: Discussion

Preface

Results for Lot 12 and Lot 14 calibration and validation runs are discussed in the ensuing sections. Sensitivity runs are also discussed as are potential future studies following on from this work.

Lot 12

Results for Lot 12 indicate a reasonable match over the range of validation events. Mean peak error is approximately 16%, although this statistic is skewed by one particularly poor match, 5 Jan 1995, which has an error of ~ 42%. Generally peak flows for Lot 12 are less than 20% of Lot 14 peak flows (the main focus of this work since this utilizes the 1d/2d model setup) and so a 42% error becomes an 8% error in peak flow for Lot 14. This is higher than is desirable however following calibration of Lot 12 with May 95 (which had an excellent match) it was not considered reasonable in terms of the procedure being followed to alter model parameter settings.

This fit into the overall methodology and the ideal behind it, which was to calibrate for a specific portion of response (in this case roof) and then to apply those settings for further runs. Losses were not varied for validation events in Lot 12 as they were for Lot 14. The thinking behind this was that given Lot 12 consists of only impervious surfaces, loss modification per event should be less important.

A difficulty with Lot 12 which may or not be specific to this study site is the presence as noted by Goyen (2000) of rubble in the pipes which connect roof down pipe flows to trunk drainage. This attenuated flows from roofs such that the peak was reduced and base flow extended over periods of time one would not normally associate with small impervious catchments. The loss settings along with the parameter values used in the lumped kinematic hydrological model applied attempted to emulate this attenuation impact on the peak specifically. The calibration event is however the smallest event of the set with 11.2 mm of rainfall and a relatively low peak intensity of 36 mm/h (over 5 minutes). As such one could intuit that the "attenuation" parameters (combination of loss and kinematic settings) although suitable for this small low intensity event were not ideally suited to the 5 Jan 1995 event which was short and extremely intense.

In looking at the brevity of the burst we can see that the two worst replicated events differ from the calibration event in that they are very brief. 5 Jan 1995 and April 1993 occur over 10 minutes and 5 minutes respectively (with respect to the main burst). The other events are half an hour at least.

It is quite apparent that having been calibrated to the May 1995 event, the parameters have not coped well with replicating events which are different in nature to the calibration event. This perhaps demonstrates an issue with conceptual modeling espoused by Goyen (2000), which is that routing parameters are often affected by the temporal patterns of applied rainfall bursts.

The important deliverable from Lot 12 modeling was a reasonable emulation of flow from the roof such that in modeling other units of hydrological response, some confidence could be had that errors were in the new units being looked at, not in roof emulation. This was achieved and aided by the fact that the roof contribution is not a major input to overall flow.

Note that the May 1995 event was specifically used in calibration of roof so that the match for peak flow and overall response would be optimal. This was done because the May 1995 event was also be used to calibrate road response for Lot 14. May 1995 was in turn selected as the road calibration event due to it having a low intensity and low overall rainfall depth and hence it was likely that all gauged flow was from impervious surfaces. Note that Goyen (2000) determined that the May 95 event was likely to consist of impervious surface response only. The runoff co-efficient for the May 95 event is 0.35. This compares favourably with the fact that 34% of the catchment is impervious. and so little runoff from yard areas for the event seems a valid assumption.

Lot 14

The two calibration events for Lot 14 have been well reproduced, the 5 Jan 95 event more so than the smaller May 95 event, and the validation events have been well replicated also. Upon examination of the results however, some issues do appear. These relate to the:

- catchment routing responsiveness using the 2d 1m grid;
- loss values utilised, particularly for the road surface; and
- sensitivity testing carried out.

Routing in the 2d 1m grid

In examining the calibration of road response, which was done via the May 95 event, it appears that the 1m grid struggles to match the hydraulic efficiency of the actual kerb/gutter system. The evidence of this is that despite using a very low roughness value (n of 0.008) and zero road losses, the peak was unable to be matched. Further it can be seen in an event of similar magnitude, the April 93 event, again the gauged response is unable to be matched. The observation of insufficient response from the surface model is supported by the sensitivity run which demonstrated little impact on peak flow estimation for a relatively large change in roughness.

The cause here is the relative coarseness of the grid relative to kerb drain, and also the fact that in the model flow may not always be orthogonal to the grid. When flow is non-orthogonal, it may be forced to take a zigzagging path and this has the impact of increasing the distance over which the flow travels. In having an increased distance to flow, the energy gradient is reduced. As such, the unrealistically low roughness value used can be seen as a means of compensation. The reduced energy losses due to the very low roughness compensate for the longer flow path and allow an approximation of the actual energy gradient. Through this specific example it can be seen that model parameters are functions of model setup and hence whilst maintaining an appearance of theoretical correctness (the approximation of the actual energy gradient in this case) may deviate from typical values.

Loss values - Road

It is likely that the May 95 event result should not have been taken as confirmation of the suitableness of using zero losses for the road area. In fact in hindsight, given that a proportional loss of 33% was used for Lot 12 calibration/validation runs and also given the mean value of runoff coefficient of 0.83 from Lot 12 gaugings, it appears likely that applying zero losses for road response was a coarse approximation.

Similarly if the runoff coefficient from the Lot 14 April 93 event is examined (value of 0.31), given that impervious area does account for 34% of the catchment area and given that proportional loss values for yard were changed in order to enhance the validation match, the decision to use zero losses for the road area appears to be in error.

Note that the runoff coefficient for the Lot 12 April 93 event is 0.61. This value might be artificially low due to the blockage in down pipes as discussed previously in this Chapter, but nevertheless, some element of loss is highly likely to have occurred.

Complicating the matter of an appropriate loss value for roads is the fact that for the events with the lowest runoff coefficients, it appears that, as stated above, the surface flow model used is at its limit with respect to emulating hydraulic conditions. So the artificially low road loss values used are in some way compensation for the attenuating 1m grid representation of the kerb/gutter system.

Loss values – Yard

Given that the volume response from roads has likely been consistently overestimated it follows that presumably yard response has been inaccurately reduced by the use of the proportional loss values applied. Evidence that this is the case appears in the fact that for four of the six events volume is an overestimate. The largest overestimate comes from the April 93 event which was dry beforehand and for which the assumption that road losses were zero is the most erroneous.

Proportional losses for pervious areas were, in some cases, varied for the different runs, with the main purpose of altering proportional loss being an achievement of peak flow match and shape match. Proportional losses have been varied between 75% for May 1995 (a low intensity low overall rainfall depth event that included very little contribution from impervious surfaces) and 30% (used for three of the events, one calibration and two validation). The mean proportional loss applied was 38% (including only validation events). If all 4 yard and road events are considered (i.e. excluding the May 95 and April 93 event) then the mean proportional loss value used is 36%.

In looking at the proportional losses used (again excluding the May 1995 and April 93 event), it can be discerned that the odd one out is the value of 55% used for the Early Jan 1993 event. If this is excluded from the set of events then the other values in the set are all 30%. However, as stated above, the 55% proportional loss value for the Early Jan 1993 event does stand out. In seeking an explanation one does observe that the rainfall depth for the Early Jan 1993 event is the largest of all events modeled at 50 mm. Also, the event is the longest event modeled, in that a rainfall intensity of 60 mm/hr is sustained over a period of 30 minutes. An explanation then for the relatively high proportional loss value that was required to be used is that in this event, more of the yard began to contribute to gauge runoff than in other shorter duration, lower rainfall depth events. That is, for events with lower rainfall depths and shorter durations, the entire yard does not (in reality or in the model) become a contributor to yard response.

Again in examining the match between observed and modeled hydrographs for the Early Jan 1993 event, despite the need to vary proportional losses in order to compensate for a model setup which has seemingly failed to emulate an on ground process (i.e. the initial loss in some yard areas that may, given some amount of rainfall depth, then begin to contribute flow), it can be seen that the overall shape match is good. The time of peak is well represented albeit slightly late. This is confirmed by the excellent R^2 and CE values. Certainly though, looking at the plot in Figure 34, it can be seen that a loss model which allowed for saturation would be of benefit (such as the Hortonian method). So instead of having a constant loss, losses would be higher at the front end of the event (note overestimates of flow by model prior to peak) and then lower at the front end (note underestimates of flow by model following peak).

It would be of interest to leave proportional loss as a constant value for the run set and instead manipulate initial loss values in yard areas, perhaps in just the rear yard area. This might yield a setup which was able to use a constant or at least less variable proportional loss value. Further work might utilise a more physical model that utilised a continuous water balance in order to eliminate the need to vary losses per event.

One interpretation of the fact that pervious loss values were fairly constant for all events might be that yard contribution to flow, particularly peak flow, was small to negligible. In examining the overall volume of gauged hydrographs it appears that for all events modeled some yard response occurred. An indication of this can be found in the gauged data established runoff coefficients for Lot 14 events. The lowest runoff coefficient is 0.31 for the April 93 event. The mean value for the Lot 14 events is 0.55. Given that only approximately 34% of the catchment is impervious it seems likely that even for the April 93 event (assuming losses from the dry road were not zero which as established above is likely the case) some yard contribution did occur.

Further evidence can be found in the 5 Jan 1995 plot sensitivity to yard roughness change plot (Figure 40) which shows that a change in yard roughness attenuated the hydrograph response. Also the validation plots for 3 Jan 93 and 28 Jan 95 show low slope portions of gauged recession hydrograph that seem consistent with runoff that would be sourced from yard area. Again though it is the runoff coefficients which provide the strongest evidence of yard contribution as both events have runoff coefficients that are well above unity (0.71 and 0.87 respectively) indicating that yard areas must have contributed significant volume to the gauged hydrographs.

Significantly however the model does quite poorly in emulating this low slope recession flow that as stated above, is indicative of yard response. In short it appears that the model is not correctly attenuating the yard runoff flow. This may be due to the lack of inclusion of blockages such as fences and houses which become significant for these larger events. It may also be because the yard roughness value used in the modeling is quite low (too smooth). And this result is in turn due the fact that the yard calibration event did not in fact feature enough yard flow response in order to make yard roughness calibration meaningful.

The question that follows from this is how much impact does yard response in the model have on peak flow, or is peak flow entirely determined by runoff from impervious surfaces? As will be discussed below, it is unfortunate in this regard that sensitivity testing to applied loss values has not been carried out in this study. However it can be positively asserted that yard response was a contributor to peak flow for the Early 3 Jan 93 event, as the value applied of 55% for proportional loss, was done so in order to ensure that peak flow estimation was reasonable. It is then noteworthy that the Early 3 Jan 93 event is a long duration high rainfall event which was preceded by other small rainfall events. Also the event peak for the Early 3 Jan 93 event occurs an hour after initial rainfall.

Sensitivity testing - roughness

Given the above discussed limit to emulation of catchment routing response, it is likely that the May 95 event was a poor choice for testing sensitivity of road roughness value. Selection of a larger event would have been of greater value to the study. The results indicated a lack of sensitivity to roughness value used. This may not be the case however for a larger event with greater flow velocities.

It also appears that the use of the 5 Jan 95 event for yard roughness sensitivity testing was a sub-optimal choice. This is because although it has the largest peak in the event set, it does not have the largest sustained runoff or the highest runoff coefficient from the event set. A better choice would have been the 28 Jan 1995 event with 28 mm of rainfall and a runoff coefficient of 0.87. In comparison the 5 Jan 95 event had 18.4 mm of rain and a runoff coefficient of 0.56. Nevertheless the yard runoff sensitivity testing did indicate some sensitivity to the roughness value used and as can be seen in the plot (Figure 40), a higher roughness value for pervious areas (yard) led to a reduced peak (16% reduced for a 150% change in roughness) and transference of volume from the peak to hydrograph recession. That is, the higher roughness value attenuated the hydrograph response.

Sensitivity testing - losses

In hindsight it is considered unfortunate that loss sensitivity runs were not carried out in order to document the sensitivity of the different events to changes in losses applied. Some sensitivity to loss changes was observed in the case of the proportional losses applied to yard and hence the reason for these being manipulated in order to achieve suitable shape and peak matches between modeled and observed hydrographs. This is not however documented. As for road losses, the initial assumption was made that these would be zero for the May 95 event and when results seemingly confirmed this assumption, no further changes were made.

Sensitivity runs – pre-wet and break line

Sensitivity runs show the importance of the break line, although this is also demonstrated positively by the fact that rising limbs for all of the events are well replicated. That the break line is vital to achieve good results is understandable given that a 1 m grid is being used in order to emulate flow paths such as the kerb/gutter arrangement. At best, physically the orthogonal 1 m grid is a poor replication of the continuous and approximately 0.35 m wide kerb/gutter. Also key here is that in the construction of the road the kerb/gutter is the preferred flow path, with the road being shaped to achieve this. The general shape is of course a crown in the middle of the road (highest point) which then slopes down to the kerb/gutter. Again it is a challenge to emulate this even using Lidar data and a 1 m grid. Stamping in of the kerb/gutter is essential then, as it means that there is a preferred flow path in the DTM and that this preferred flow path is lower than the rest of the road area in the DTM. Here it is noteworthy to reiterate the finding of the literature review presented in Chapter 2, that since the beginning of development of joint surface and subsurface models, a constant refrain has been the need to "stamp" the main overland flow path into the DTM or other topographical data representing surface flow.

The sensitivity results also do indicate that the pre-wet aids the break line in achieving performance which is a good simulacrum of observed behaviour, particularly in the initial rising limb of the event and in achieving a peak flow and volume match as well. From the sensitivity run using the 5 Jan 1995 event close inspection of Figure 38 shows that without the pre-wet hydrograph response starts one minute later. This is no doubt due to the early runoff filling holes in the DTM surface.

It could be argued that rather than applying a pre-wet in the sensitivity run, the peak and volume could be matched if the PL value was changed. However the pre-wet does facilitate, emulation of the rising limb.

Impact on results

Naturally the conclusion that losses for road have been consistently underestimated in the Lot 14 runs leads to the question, how does this impact the models ability to emulate larger events, as perhaps impervious area response is exaggerated. The answer is it seems that there is some exaggeration shown. For the second two largest events (with respect to flow), 28 Jan 95 and 3 Jan 93, it appears that yard response has been artificially reduced in order to ensure that peak flow reproduction is reasonable. This can be seen from the volume results which in both cases show significant underestimates at -28% and -11% respectively. The plots of both shown in the previous Chapter indicate significant amounts of yard flows in the gauged hydrograph recession which the model does not emulate. As for the largest event with respect to flow, the 5 Jan 95 event, the road response works well and produces a good match, however perhaps the early start of the modeled hydrograph rise is explained by the preceding arguments.

The calibration process

It could be said that the model build process was the calibration process. This statement is supported by the following observations:

- There wasn't any iterative calibration of parameters which dramatically impacted results. Build steps were identified, for example the "stamping" in of the kerb/gutter alignment, but this is not a matter of degree. It is either "stamped" in or it is not. This becomes a generic and automatic part of the methodology; and
- Roughness values were iterated upon but following completion of the modeling and sensitivity testing it can be seen that these iterations were attempts only and did not

dramatically changing model responsiveness. The roughness values could just as easily have been pre-informed based on values shown in Table 4. As previously indicated however it is likely that the sensitivity runs carried out were sub-optimal from the point of view of establishing true sensitivity of model parameter settings.

Results indicate that the diffusive non-inertial form of the St Venant equation was adequate for routing of flow in the 2d surface, albeit perhaps at its absolute limit (using the 1m grid) for the smaller events in the modeled in the event set.

How the 2d surface flow model performs in the instance of larger events is not known, as in this application pipe capacity is not exceeded by any of the modeled events. For events where pipe capacity is exceeded, surface flow which moves through areas of houses, fences etc may become more important and as such it may be preferable to apply a 2d surface model that solves the full St Venant equation.

Following on from the above point the overall model setup has been driven by the application in the sense that the 2d model is in this case being used to route flows to inlet pits only, i.e. as a means of developing inflow hydrographs for pits only. As such effort has not been expended on the DTM such that houses stand proud of the surface, nor has any effort been made to look at representation of fences etc as permeable obstructions to flow.

Chapter 7: Conclusions

Firstly the results of the study are compared to the research goals put forward in Chapter 1.

With respect to demonstrating the 1d/2d methodology on a detailed data set this work is a success. A very good fit is found between modeled and observed discharge over a reasonable range of discharges. Validation runs achieved very good results for fit without the need to vary any model values other than pervious area proportional loss rates. Plotted hydrographs comparing gauged to modeled flow indicate a modeling system that is performing very well.

The second research goal was to establish or confirm components of methodology. The idea of "stamping" the kerb/gutter alignment into the DTM has been thoroughly confirmed by sensitivity testing. Also, pre-wetting in order to fill holes in the DTM, has been confirmed to be of use particularly in ensuring that the rising limb of flow is well emulated. Additionally a general methodology has been documented that may be of some use to others working with coupled urban stormwater models.

The third goal was to indicate whether or not it was likely that the 1d/2d methodology could in fact be used to reduce flow estimation error for ungauged catchments. In regards to this it can be said that the modeling approach demonstrates that there are few parameters requiring calibration and that results are relatively insensitive to roughness values. The example set is small however and validation of the method on an external catchment is required in order to confirm the effectiveness of the method.

With respect to the calibration process it is the case however that the model utilised herein when compared to a lumped conceptual approach for generation of pit inlet hydrographs is more complex and subject to many more potential errors in setup. As such, in the absence of calibration data, checks would have to be made that estimates seemed reasonable. The mass balance check is a good start for this process but comparisons against estimates made for similar catchments or against a modelers "idea" of what flows should be would also be informative.

It was also found that due to the scale at which some key hydrological processes occur they cannot be included in the 2d surface flow model. It was found that house roofs were more efficiently modeled, with respect to model setup time and computational effort, using a lumped kinematic model with a sub-catchment for each roof

Further Work

The sensitivity testing could be greatly improved if different events had been utilised other than the ones that were. A further study could address this issue.

Also the main difficulty with respect to the 1d/2d model systems success in application to ungauged catchments is the setting of losses. In particular pervious surface losses, which can, depending on the catchment antecedent condition, change significantly from event to event. A real step forward would therefore be integration of a physical process loss model that not only gave a reasonable estimate of the antecedent condition but that also which allowed for a varying level of infiltration during the event including total saturation

Other sites need to be looked at, particularly sites with lower design capacities in the minor systems and with spatial data available for comparison to model predictions. This could help achieve a standard methodology for dealing with porous but attenuating fencing, house blockage and standard yard features such as impervious patio areas.

Sites where inlet capacity rather than pipe capacity is limiting would also be useful as this could establish the suitability of the pit inflow description applied.

..

References

Aitken, A.P. (1973) Hydrologic Investigation and Design in Urban Areas – A Review, AWRC Technical Paper No. 5, Dept, Canberra

Aitken, A.P. (1975) Hydrologic Investigation and Design of Urban Stormwater Drainage Systems, AWRC Technical Paper No. 10, Department. of the Environment and Conservation, Research Project No. 71/22, Canberra

Alam, J.Md., 2003. Two-dimensional urban flood modeling for real time flood forecasting for Dhaka City, Bangladesh. AIT Thesis no. WM-02-06. Asian Institute of Technology, Thailand.

Boonya-Aroonnet, S., Weesakul, S., Mark, O., 2002. Modeling of urban flooding in Bangkok, Ninth International Conference on Urban Drainage, Portland, Oregon, USA.

Bufill, M.C. (1989) Effects of Urbanisation on Floods, PhD Thesis, University of Wollongong.

Carr, R. 2004. "Mike Storm – Dynamically linked 2d overland and 1d pipe stormwater modeling" 2004 NZWWA conference Rotorua.

Carr and Smith 2006. "Linking 2D and pipe hydraulic models at fine spatial scales" Urban Drainage Modeling and Water Sensitive Urban Design 2006

Chow, V.T. (1964) Handbook of Applied Hydrology, McGraw-Hill, New York.

Clark, C.O. (1945) Storage and the Unit Hydrograph, Transactions ASCE, Vol 110, pp. 1419-1446.

Crawford N.H. and Linsley, R.K. (1966) Digital Simulation in Hydrology: Stanford Watershed Model IV, Technical Report 39, Department of Civil Engineering, Stanford University.

Dey, A.K., Kamioka, S., 2007. An Integrated modeling approach to predict flooding on urban basin, In Water Science and Technology, Vol 55, No 4, pp 19-29.

Diskin, M.H. (1962) A basic study of the linearity of the rainfall-runoff process, Ph.D Thesis directed by V. T. Chow, University of Illinois, Urbana, Illinois

Djordjevic, S., Prodanovic, D., Maksimovic, C., 1999. An Approach to Simulation of Dual Drainage, In Water Science and Technology, Vol 39, No. 9, pp 95-103.

Elgy, J., Maksimovic, C., Prodanovic, D., 1993. Matching standard GIS packages with urban storm drainage simulation software. HydroGIS 93: Applications of Geographic Information Systems in Hydrology and Water Resources (Proceedings of the Vienna Conference, April, 1993), IAHS Publication No. 211, 151–160.

Everitt, B.S. (2002). Cambridge Dictionary of Statistics (2nd Edition). CUP.

Ferguson, D. and Ball, J. E. (1994) Implementation of the Kinematic Wave in the Runoff Block of SWMM, Research Report No. 183, University of New South Wales

Gordon, R. (2007) Next Generation Hydraulic Modeling for Inundation Mapping and Flood Warning in Complex Urban Systems, Masters Thesis, RICE University.

Goyen, A.G. and Aitken, A.P. (1976) A Regional Stormwater Drainage Model, Hydrology Symposium, Institution of Engineers Australia, Sydney, National Conference

Goyen, A.G., Phillips, B.C. and Black, D.C. (1991) Recent Advances in Flood Estimation using RAFTS-XP, International Symposium on Hydrology and Water Resources, Institution of Engineers Australia, 2-4 October, Perth

Goyen, A.G.. (2000) Spatial and Temporal Effects on Urban Rainfall / Runoff Modeling, PhD Thesis, University of Technology Sydney.

Haile, A.T. (2005) Integrating Hydraulic Models and High Resolution DEM (Lidar) for Flood Modeling, Masters Thesis, International Institute for Geo-Information and Earth Observation, Netherland.

Hawken, R.W.H. (1921) An analysis of maximum runoff and rainfall intensity, Transactions, Institution of Engineers Australia, Volume 2, pp. 193-215

Hicks, W. I. (1944) A method of computing urban runoff, Transactions, ASCE, Volume 109, pp. 1217-1253

Hsu, M.H., Chen, S.H., Chang, T.J., 2000. Inundation simulation for urban drainage basin with storm sewer system. Journal of Hydrology 234, 21–37.

Hunter, N. M. P. D. Bates S. Neelz G. Pender I. Villanueva N. G. Wright D. Liang R. A. Falconer B. Lin S. Waller A. J. Crossley D. C. Mason Benchmarking 2D hydraulic models for urban flooding Water Management Volume: 161 Issue: 1 Page 13 – 30.

Inoue, K., Kawaike,K., Hayashi, H. 2000. Journal of Hydroscience and Engineering. "Numerical Simulation Models on Inundation Flow in Urban Area" Vol 18, No.1 May 119-126.

Institution of Engineers, Australia. (1987) Australian Rainfall and Runoff - A Guide to Flood Estimation (edited by D.H. Pilgrim and R.P. Canterford), Volumes 1 and 2, Revised Edition, November, 1987, Canberra

Iwasa, Y., Noguchi, M., Nakamura, T. 1987. AIRH-XXII Congress – IAHR. 4th International Conference on Urban Storm Drainage. "Simulation or Urban Storm Drainage Involving River and Overland Flows" Lausanne.

Izzard C.F. (1946) Hydraulics of Runoff from Developed Surfaces, U.S. National Research Council, Higway Research Board, Proceedings, Vol 26, P.129-150 (with discussion)

Kassem, A.M.A., 1982. Development and application of a simultaneous routing model for dual drainage. PhD dissertation,

Kidd C.H.R. (1978) Rainfall runoff processes over urban surfaces. Report 53, Institute of Hydrology, Wallingford.

Kuichling, E. (1889) The relationship between the rainfall and the discharge of sewers in populous areas, Transactions, ASCE, Vol. 10, No.1, pp 1-60

Kulandaiswamy, V.C. (1964) A basic study of the rainfall excess-surface runoff relationship in a basin system, Ph.D Thesis directed by V.T. Chow, University of Illinois, Urbana, Illinois

Laurenson, E.M. (1964) A Catchment Storage Model for Runoff Routing, Journal of Hydrology, Volume 2, pp 141-163

Laurenson, E.M. and Mein, R.G. (1985) RORB - Version 3 Runoff Routing Program User Manual, Monash University, Department of Civil Engineering, March

Lloyd-Davies, D.E. (1906) The elimination of stormwater from sewerage systems, Proceedings, Institution of Civil Engineers, Volume 164, No.2, pp. 41-67+plate

Mark, O., Weesakul, S., Apirumanekul, C., Boonya Aroonnet, S., Djordjevic, S. 2004. Journal of Hydrology “Potential and limitations of 1D modeling of urban flooding” Volume 299 (2004) 284–299.

Mulvaney, T.J. (1850) On the use of self-registering rain and flood gauges in making observations of the relations of rainfall and of flood discharges in a given catchment, Proceedings, Institution of Civil Engineers, Ireland, Vol. 4, pp. 18-31

Nash, J.E. (1960) A unit hydrograph study with particular reference to British catchments, Proceedings, Institution of Civil Engineers, Vol. 17, pp. 249-282

Nash, J. E. and J. V. Sutcliffe (1970), River flow forecasting through conceptual models part I — A discussion of principles, Journal of Hydrology, 10 (3), 282–290.

NSW Floodplain Development Manual, 2005. Department of Natural Resources, NSW.

O'Loughlin, G.G. (1986) The ILSAX Program for Urban Stormwater Drainage Design and Analysis, New South Wales Institute of Technology, Civil Engineering Monograph No. 86/3

O’Loughlin, G., Huber, W., Chocat, B., 1996. Rainfall-runoff processes and modeling. In Journal of Hydraulic Research, Volume 34, No.6, pp 733-751.

Phillips, B.C., Yu, S., Thompson, G.R., DeSilva, N., 2005. 1D and 2D Modeling of Urban Drainage Systems using XP-SWMM and TUFLOW, In 10th International Conference on Urban Drainage, Copenhagen Denmark.

Ross, C.N. (1921) The calculation of the flood discharge by the use of time contour plan, Transactions, Institution of Engineers Australia, Volume 2, pp. 85-92

Rovey, E.W., Woolhiser, D.A. and Smith, R.E. (1977) A Distributed Kinematic Model of Upland Watersheds, Hydrology Papers (93), Colorado State University, Fort Collins, Colorado

Schmitt, T.G., Schilling, W., Sægrov, S., Nieschulz, K.-P., 2002. Flood risk management for urban drainage systems by simulation and optimisation, Ninth International Conference on Urban Drainage, Portland, Oregon, USA.

Schmitt, T.G., M. Thomas and N. Ettrich, 2005. Assessment of urban flooding by dual drainage simulation model RisUrSim. Water Science & Technology Vol 52 No 5 pp 257–264

Singh, K.P. (1962) A non-linear approach to the instantaneous unit hydrograph, Ph.D Thesis directed by V. T. Chow, University of Illinois, Urbana, Illinois

Smith, M.B., 1993. A GIS-based distributed parameter hydrologic model for urban areas. Hydrological Processes 7, 45–61.

Smith, M.B. Comment on `Potential and limitations of 1D modeling of urban flooding' by O. Mark et al. [J. Hydrol. 299 (2004) 284-299]., Journal of Hydrology Volume 321, Issues 1-4, 30 April 2006, Pages 1-4.

Smith, M.B. Comment on `Analysis and modeling of flooding in urban drainage systems', Journal of Hydrology Volume 317, Issues 3-4, 20 February 2006, Pages 355-363.

Smith, J., Phillips, B.C., Yu, S. 2006. Modeling Overland Flows and Drainage Augmentations in Dubbo. 46th Floodplain Management Authorities Conference Lismore.

Syme, W.J., Pinnell, M.G., Wicks, J.M., 2004. Modeling Flood Inundation of Urban Areas in the UK Using 2D/1D Hydraulic Models, In 8th Natinal Conference on Hydraulics in Water Engineering, Australia.

Takanishi, S., Noguchi, M, and Nakamura, T., 1991. Simulation of urban stormwater by 'Numerous'. Proceedings of the XXIV IAHR Congress held in Madrid Spain, September 9-13, A-99 - A108.

Tholin, A.L. and Keifer, C.J. (1960) The hydrology of urban runoff, Transactions, ASCE, Volume 125, pp. 1308-1379

Watkins, L.H. (1962) The Design of Urban Sewer Systems, Research into the Relation Between Rate of Rainfall and Rate of Flow in Sewers, U.K. Department of Scientific and Industrial Research, Road Research Laboratory, Technical Paper No. 55

Xing, F., Dehui, S., 2006. An Integrated One-Dimensional and Two-Dimensional Urban Stormwater Flood Simulation Model, In Journal of the American Water Resources Association, Vol 42, 3, pp 713-724.