C-CALM TREATMENT MODULES:

Models for Catchment Sediment Transport, Ponds and Raingardens

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C-CALM TREATMENT MODULES: Models for Catchment Sediment Transport, Ponds, Wetlands and Raingardens

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Executive Summary

NIWA is currently developing a spatial decision support system (SDSS) within ArcMAP to estimate annual loads of suspended sediments, copper and zinc from urbanised catchments called C-CALM (Catchment Contaminant Annual Loads Model). This report discusses the modelling context for the C-CALM project and presents three model modules for the development of performance rules for water treatment. The work is being carried out under subcontract to Landcare Research and is funded by the Foundation for Research Science and Technology. The model is being developed in response to a need for a standard tool that can be used by Regional and Territorial governments to determine the impacts of urbanisation on local receiving waters. The need for such a tool was expressed in the Landcare Research LIUDD FRST contract C09X0309 in 2003. A workshop for stormwater managers held by NIWA in June 2006 showed there is support amongst potential users for a national tool. The delegates stated that operational models currently used (e.g., the widely used StormWater Management Model, SWMM, US EPA and Model Of Urban SEwers MOUSE, DHI) are too demanding of data requirements, set-up and run times and user expertise for this purpose. They specified that the model should be simple and intuitive to use with minimal data and data handling requirements. Delegates also stated that the proposed model should be developed within a Geographical Information System (GIS) to enable geo-visualisation of contaminant sources and sinks both to aid decision making and to improve communication with other stakeholders.

This report:

- Overviews the background to the C-CALM project.
  (Section 1)

- Provides information about the modelling context for C-CALM within GIS including the structure of the SDSS and intended users and applications.
  (Section 2)

- Presents the informing modules which will be used to develop the performance rule library for ponds/wetlands and raingardens. General model assumptions are also discussed.
  (Section 3)

- Summarises the modules and gives information on how they will be used to develop the performance rules.
  (Section 4)

C-CALM will use the Auckland Regional Council's spread sheet annual contaminant loads model (CLM, Timperley, 2007) as a basis for load estimation dependant on land use and surface type. C-CALM is one of several GIS models currently under development for stormwater contaminant modelling in the Auckland area using CLM as a starting point (e.g., Krpo, 2007; Peng and Young, 2007). However, C-CALM differs both in that it aims to be applicable across the country and there will be provision of tools for the creation of future land use scenarios. One of the main innovations of C-CALM is that it will have variable BMP efficiencies to reflect the fact that contaminant removal is a function of BMP size and design, sediment grain size and catchment characteristics. Rather than modelling the stormwater network explicitly in the GIS which would increase model complexity, a compromise is to develop a set of performance rules with variable removal efficiencies that can inform the SDSS.

The original intention was to provide the C-CALM SDSS with a set of performance rules that had been developed using continuously run conceptual models of the devices used for water treatment. The SDSS would be able to query these rules rather than explicit continuous simulation modelling of stormwater flow pathways and treatment within the GIS. This would enable C-CALM to have a sound theoretical modelling basis without the complexity, data and user expertise required of operational urban drainage models. However; New Zealand data suitable for model development was only available with a sufficient length of time for two ponds and a single raingarden. The models for sediment and particulate metal removal from these devices are discussed in this report. The removal efficiencies for dissolved metals from ponds, wetlands and raingardens were not simulated due to the complexity of the processes involved – the values for C-CALM are addressed in Semadeni-Davies (2008). Out of necessity, C-CALM has had to evaluate the efficiencies of other treatment options based on literature values for other treatment options (*ibid*). The rules themselves will be presented in a later report in accordance with the project work schedule.

At present, modelling modules have been developed for catchment runoff / wash-off simulation and treatment in ponds and raingardens. All the modules are run with five minute time-steps.

- Catchment runoff and sediment.
  This module couples a hydrological rainfall / runoff model to sediment accumulation and wash-off equations and is used to provide input flows to the treatment modules. The module has been tested on stormwater flows and sediment sampling from four catchments with varying areas and land use.

- Settling in ponds and wetlands.
  This module couples a simple continuity equation for pond storage to settling equations to simulate flow and water treatment. Wetlands are treated in the same way as ponds albeit with greater hydraulic efficiency. The module has been tested on two stormwater ponds.

- Raingardens and bio-retention units.
  This module couples a simple water balance for raingardens to a time-based contaminant depletion equation. The module has been tested on data from one raingarden. Bio-retention units are assumed to have the same removal efficiency as raingardens with the difference in removal due to scale.

The informing modules have been developed with a modelling interval of 5 minutes. Given that the purpose of developing these modules is to provide a means of creating a treatment library, the models are deliberately simple with few calibration parameters. That is, they represent generic conditions for archetypical landuse and treatment types. While increasing model detail and complexity will probably increase model performance, it is important to keep the task of developing a treatment library in focus. Providing further options for model parameters will both increase the run-time for the performance rule sensitivity analysis and the memory needed for the C-CALM SDSS (and therefore query times).

In addition to presenting the treatment modules, general assumptions about contaminant characteristics are also detailed in this report. It is found that contaminant concentrations and characteristics vary in both time and space. Sediments, for example, can have a range of fall velocities related to both the grain size and density. Moreover, both the partitioning and fractionation of metals with respect to sediments is highly complex and is dependant on water chemistry and sediment properties. The heterogeneity of contaminants has major implications for stormwater modelling. The C-CALM modules have the following simplifying assumptions:

- Fall velocities will be based on the NURP settling column classes (US EPA Nationwide Urban Runoff Program, Driscoll *et al.*, 1986), the original distribution will be scaled up and down as a proxy for finer and coarser sediments.

- The fractionation of particulate metals is divided in the same relative proportions of sediments in the particle size distribution (PSD), and

- Removal of dissolved metals is too complex for simple generic modelling and is influenced by the physical, biological and chemical make-up of the treatment facility, the physical and chemical characteristics of suspended sediments in the stormwater and the chemistry of the stormwater. Instead, literature values will be chosen (see Semadeni-Davies, 2008).

1. Background

Many cities in New Zealand are located on natural harbours or estuaries and the health of these environments is closely linked to the quality of contaminants transported in urban stormwater. Sediments are a particular concern, not only as high yields can potentially damage benthic communities by smothering or changing substrate grain size, but because contaminants, especially metals, tend to bind to sediments. Williamson and Morrisey (2000) for instance found an increase in the metal content of sediments in Auckland estuaries with urbanised catchments. In order to safeguard these receiving environments, there has been a move by regional and local governments to require treatment of stormwater at both the sub-catchment (e.g., source and site control) and the catchment (e.g., end-of-pipe) levels.

The relationship between total suspended sediments (TSS) and other contaminants means that sediment removal is a primary method of stormwater treatment. Over recent years there has been a trend away from purely reticulated stormwater systems towards the installation of devices for contaminant removal and control of stormwater volumes. These are usually designed to perform both functions by storing and treating stormwater. Most devices therefore use some form of detention to both reduce and attenuate peak flows and remove contaminated sediments. The main removal methods are settling and filtration. The performance of these devices is highly dependent on their design and the characteristics of the contaminants to be removed. These devices can complement each other and a stormwater system may include a number in series - so-called treatment trains - where hydraulics and removal processes of up-stream devices have an impact on the processes of those downstream. How treatment trains function is thus extremely complex and is dependant on the number, type, configuration and dimensions of devices present and rate and volume of stormwater flows between devices (slope, travel distance, channel characteristics etc).

While there has been some work to establish links between urbanisation, water management and environmental health including both catchment monitoring and harbour sediment sampling, the long-term impacts of continued urbanisation are difficult to assess. Two key questions are:

- what is the rate of long-term contaminant delivery to receiving environments?

- how will different stormwater management options change this rate?

At present, there is no standard tool to answer these questions. The need for such a tool was expressed in the Landcare Research LIUDD FRST contract C09X0309 in 2003. A workshop for stormwater managers held by the NIWA in June 2006 showed there is support amongst potential users for the development of a national tool. The delegates stated that operational models currently used are too demanding of data requirements, set-up and run times and user expertise for this purpose. They specified that the model should be simple and intuitive to use with minimal data and data handling requirements. While the ARC CLM (contaminant loads model - a spreadsheet spatially lumped model for annual contaminant loads) is widely available and is used across the country, delegates wanted the proposed model to act as a spatial decision support system (SDSS, see Densham, 1991) by building it within a GIS platform. It was noted that most councils have GIS databases for storing spatial data including impervious surfaces, catchment boundaries, land use zones, storm- and wastewater pipe networks, location and type of stormwater treatment devices, roads and streams.

An SDSS incorporating this information would enable contaminant sources and sinks to be easily identified. This will aid planners and stormwater engineers in finding environments most at risk and determining where water treatment devices would be best located to reduce that risk. It could also allow new land use scenarios to be created and simulated quickly without changing the contaminant model set-up. Presenting results visually in maps would also enable local water managers to better communicate the impacts of urbanisation and water treatment to other stakeholders. This point is particularly important as without showing the possible impacts of urbanisation on receiving waters, stormwater treatment has often been seen by developers as an optional extra more akin to landscaping rather than as an important part of sustainable urban design.

A key requirement of the model is that it should have a flexible and realistic representation of stormwater treatment options without the need to model the stormwater network explicitly. Instead, the C-CALM SDSS will be provided with a library of so-called performance rules for commonly used treatment options under different base-line conditions. The SDSS will query this library with respect to the spatial information provided in the GIS to modify calculated contaminant loads for treatment. The library of performance rules will contain removal efficiencies for the following treatment options: street-sweeping, ponds and wetlands, filters, vegetative bio-filters (raingardens, swales and infiltration strips), catch-pits (with and without inserts) and porous paving.

The original intention was to develop the performance rule library using continuously run conceptual models of the devices used for water treatment. This approach would enable C-CALM to have a sound theoretical modelling basis without the complexity, data and user expertise required of operational urban drainage models. The idea is to change input data (e.g., regional rainfall) and parameters (e.g., dimensions of the treatment facility) systematically in a series of model runs to build up a matrix of performance rules for each device which would be held in the GIS library. However; New Zealand data suitable for model development was only available with a sufficient length of time for two ponds and a single raingarden. Out of necessity, C-CALM has had to evaluate the efficiencies of other treatment options based on literature values for other treatment options (see Semadeni-Davies, 2008).

This report presents the informing modules which will be used to develop the performance rules for sediment and particulate metal removal from ponds, wetlands and raingardens. In addition, a simple conceptual model for catchment flow and sediment transport is given. This model will be used to provide input data for the treatment modules. Removal efficiencies for dissolved metals were not simulated as they are too complex for simple generic modelling, that is, removal is influenced by the physical, biological and chemical make-up of the treatment facility, the physical and chemical characteristics of suspended sediments in the stormwater and the chemistry of the stormwater.

The rules themselves will be presented in a later report in accordance with the project contract and work-plan.

1.1 Layout and Objectives

This report:

- Provides information about the modelling context for C-CALM within GIS including the structure of the SDSS and intended users and applications.
(Section 2)

- Presents the informing modules which will be used to develop the performance rule library for ponds/wetlands and raingardens.
(Section 3)

- Summarises the modules and gives information on how they will be used to develop the performance rules.
(Section 4)

Section 3 contains the bulk of this report and has the following objectives:

- Overview the general modelling assumptions, notably with respect to sediment particle size distribution (PSD) to be used in the development of the performance rules.

- Develop and test a coupled rainfall/runoff and accumulation/wash-off catchment model. This modelling module will be used to provide input data for the treatment modules. Inputs are rainfall and potential evapotranspiration. Parameters include the ratio of permeable to impervious surfaces, surface and drainage characteristics, and sediment accumulation and wash-off rates. Outputs are discharge and sediment concentration and load. The module is tested using data from three Auckland stormwater catchments with different land uses.

- Develop and test a coupled flow and treatment (settling) model for ponds and wetlands. Inputs are inflow, evaporation and sediment concentration. Parameters include the size and hydraulic efficiency of the treatment basin, the size of the outlet, and sediment fall velocities. Outputs are discharge and sediment concentration and load. The module is tested for two ponds with different designs.

- Develop and test a coupled flow and treatment (settling) model for raingardens. Inputs are inflow, evapotranspiration and sediment concentration. Parameters include the area and depth of the raingarden, the grain size of the filter media and the ratio of deep percolation to discharge. Outputs are discharge and sediment concentration and load. The module is tested for a raingarden.

All the modules are simulated with time-steps of 5 minutes. The models are deliberately simple with few calibration parameters. That is, they represent generic conditions for archetypical landuse and treatment types.

2. The C-CALM model

2.1 Modelling context

An SDSS called C-CALM (Catchment Contaminant Annual Loads Model) is being developed by NIWA using the ArcMAP platform. The work is being carried out as a sub-contract under Landcare Research Ltd and is overseen by a six-member advisory group composed of scientists and end users. Contaminants to be modelled are total suspended solids (TSS), zinc and copper.

C-CALM will relate annual contaminant loads per unit area to land cover according to the relationships found by Timperley *et al.* (2005). These relationships form the basis of a lumped spreadsheet model (CLM, contaminants load model) developed by the Auckland Regional Council (ARC) which is has also been applied elsewhere in New Zealand (Timperley, 2007) such as Wellington (personal communication, Juliet Milne, Greater Wellington Regional Council, Jan 2008). Auckland territorial authorities (TLAs) are required by the ARC to run the model as part of their Integrated Catchment Management Plan (ICMP) preparation, however, whether the model is run at the catchment or sub-catchment level depends on the TLA and, in some cases, the environmental consultants engaged by the TLA.

The spreadsheet model works by relating annual contaminant loads to the land surface coverage. Land cover classes include roofing material, roads (classed by traffic), other imperious surfaces (e.g., paving), construction sites and vegetation (several classes such as pasture and orchard). In some cases, the land cover is further split into slope classes. Streams are included within the model; sediment load generated in channel is related to stream length and width. The percentage area covered by each cover type is simply multiplied by a scalar (representing the annual load per unit area, for example) and the total catchment area to give the annual contaminant load that can be expected to be generated by that cover. The total contaminant load is then the sum of the loads from the different cover classes. Users are required to either input known land cover information, or where this information is not known, make assumptions about the relative proportions of land cover classes based on land use and age of the development. Thus industrial catchments are assumed to have a greater roof cover than residential catchments: moreover, these roofs are more likely to be unpainted galvanised iron and therefore a source of dissolved Zn. Water treatment in the CLM for each land cover is simulated by reducing the load by a pre-defined removal efficiency (e.g., 75% removal of TSS from ponds assuming adequate sizing and design based on literature values, e.g., ARC TP 108, 1999; Schueler, 1997, 1992). Within a catchment area, spatial information is lumped though quasi-distribution can be achieved by running the spreadsheet model for a series of sub-catchments and pooling the results – however, this approach is unable to link sub-catchments in a network. C-CALM will both offer more flexible treatment options and improved spatial representation.

Embedding the spreadsheet model within a GIS platform offers a new suite of applications by allowing spatial distribution of model inputs and outputs and a tool for geo-visualisation. The value of GIS for urban water management has been covered in detail by Shamsi (2005). He notes that the technology has applications as diverse as storage and mapping of spatial data including drainage network elements to modelling water flows through those elements. C-CALM has been described above as a SDSS to be built within ArcMAP. An SDSS is an interactive computer based model designed to support a user or group of users to make land use decisions and to solve semi-structured spatial problems. An SDSS also provides a tool for more effective communication between stakeholders. Densham (1991), amongst others, has listed the components of an SDSS as:

- A database management system to input, store and analyse large quantities of spatial data,

- The ability to represent complex spatial relationships and structures common to spatial data.

- A library of analytical sub-routines that can be used to query the spatial data to forecast the possible outcome of decisions,

- Display and report capability using a variety of forms (i.e., cartographic displays, tables and plots of spatial data and forecasts), and

- An interface to aid users to interact with the system and assist in the analysis of outcomes. The interface should be powerful and easy to use by following REAL principles (reliability, efficiency, attitude, learnability)

Central to the functionality of an SDSS is the provision of tools for geo-visualisation of inputs and outputs. Geo-visualisation (Dyke *et al.*, 2005; MacEachren *et al.;* 2001) for decision making requires:

- The ability to overview (pan) spatial data to identify change and/or areas of interest;

- The ability to zoom into the detail of an area of interest or out to the wider spatial pattern;

- The ability to filter redundant information;

- The ability to interact with or query the spatial data to change the information displayed; and

- The ability to extract and report on spatial data and spatial relationships.

ArcMAP software was chosen as the platform for C-CALM rather than creating a standalone product as ArcMAP is widely used by regional and local government in New Zealand. ArcMAP has powerful tools for spatial data storage, management, analysis and display. A C-CALM interface including tools for creating land use and treatment scenarios and toggling between display options can be coupled to ArcMAP as a toolbar. Users will also be able to use standard GIS functions included in ArcMAP to complement the options for analysis and display included with C-CALM. Coupling water quality models to ArcMAP is not without precedent. Indeed NIWA has recently developed a model for simulating the impacts of rural land use change on annual sediment and nutrient loads in river networks called CLUES (Catchment Land Use for Environment Sustainability, Semadeni-Davies *et al.,* 2007*;* Woods *et al.*, 2006) for the Ministry of Agriculture and Forestry (MAF) in collaboration with Lincoln Ventures, Harris Consulting, AgResearch, HortResearch, and Landcare Research.

While there are some existing GIS planning packages for simulating water treatment (e.g., StormTac available from SWECO: Larm, 2000), these are not widely available and have not been created for New Zealand conditions. C-CALM is one of several GIS models currently under development for stormwater contaminant modelling in the Auckland area using the ARC spreadsheet model cited above as a starting point (e.g., Krpo, 2007; Peng and Young, 2007). However, C-CALM differs both in its aim to be applicable across the country and the provision of tools for the creation of future land use scenarios. Unlike its predecessor, C-CALM will have variable BMP efficiencies to reflect the fact that contaminant removal is a function of BMP size and design, sediment grain size and catchment characteristics.

Figure 1 gives a schematic overview of how the models presented in this report and the literature derived treatment efficiencies (Semadeni-Davies, 2008) link via the performance rules to the C-CALM SDSS. For removal of sediments and associated particulate metals from ponds, wetlands and raingardens, the following modelling steps are indentified:

1. Development and testing of continuous conceptual models for flow and contaminant transport using New Zealand catchment and water treatment data. This report presents this stage of the modelling process.

2. Development of performance rules by carrying out a series of runs with different input data (i.e., representative regional rainfall and evapotranspiration) and parameter sets to determine the effect of different stormwater control options on long-term removal efficiencies. An example is given for ponds in Figure 2 where the parameters catchment type (slope, and impervious area with respect to landuse), sediment settling characteristics (PSD), pond specific area (percentage area relative to the area contributing to flow) and hydraulic efficiency of the device. The modules will be run with long-term climate data (e.g., 10 years, subject to availability) from the NIWA national climate database. This body of work will be presented in a later report in accordance with the project work-plan.

3. Incorporation of performance rules into the C-CALM SDSS to allow quasi-representation of flexible water treatment options. The SDSS will query the rules to find the long term removal efficiency for a given set of drivers either defined from the GIS database or by the user.

Figure 3 shows the steps involved in running the SDSS indicating points where users interact with the model interface. C-CALM will use user-defined stormwater catchments and sub-catchments so that the results will be compatible with other stormwater management tasks undertaken by the user. This will also allow users the flexibility to break catchments up into areas at the scale of interest. Furthermore, it is NIWA's experience that automated catchment delineation tools (e.g., based on slope or node geometry) can lead to catchment boundaries that do not match the TLA stormwater boundaries. Each sub-catchment will be assigned a treatment node by C-CALM – this node will be representative of the entire sub-catchment and will not be tied to a specific location. Users will be required to list the treatment devices present, their relative order and which land covers (and relative proportion of those land covers) they will treat. Users will then be required to build the drainage network by linking the nodes either to another node downstream or to a catchment outlet. C-CALM will then calculate the total contaminant load for each sub-catchment which will be reduced according to the relevant removal efficiency from the performance rules.

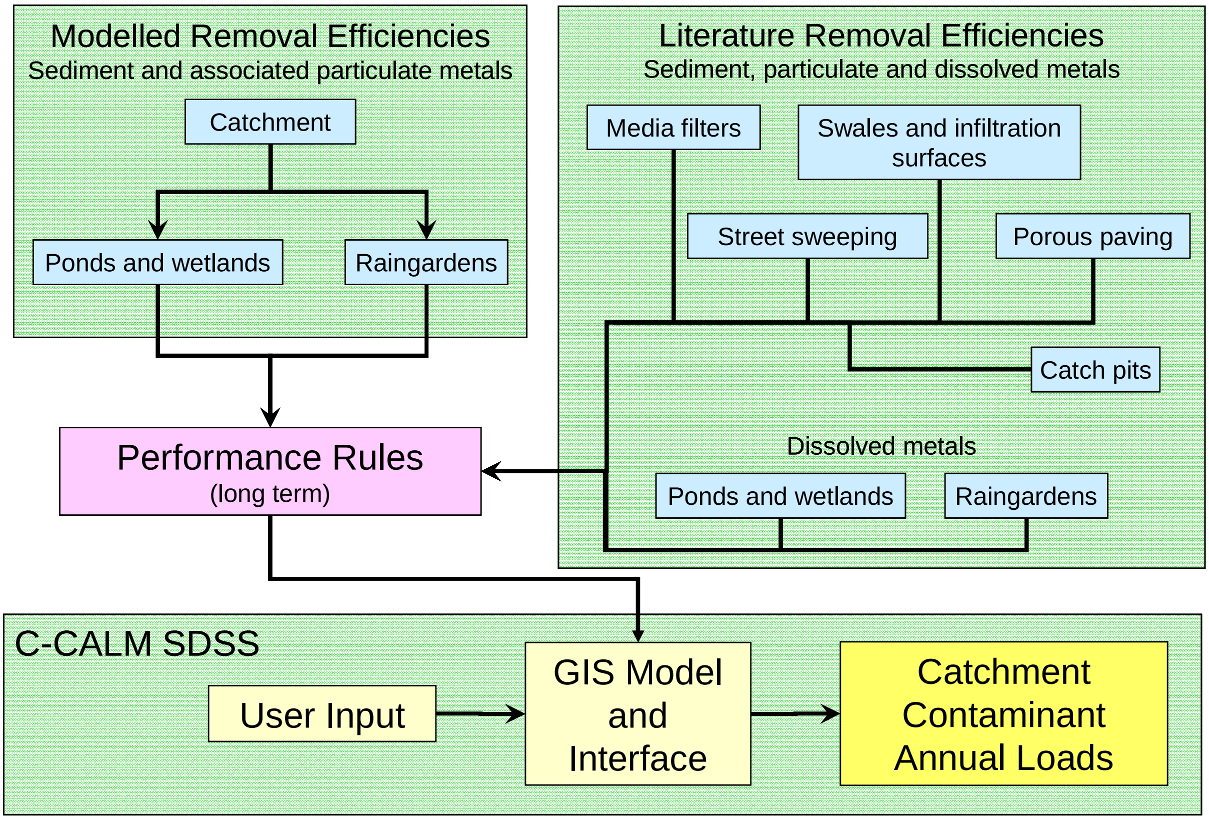


Figure 1 **Schematic overview of the C-CALM model.**

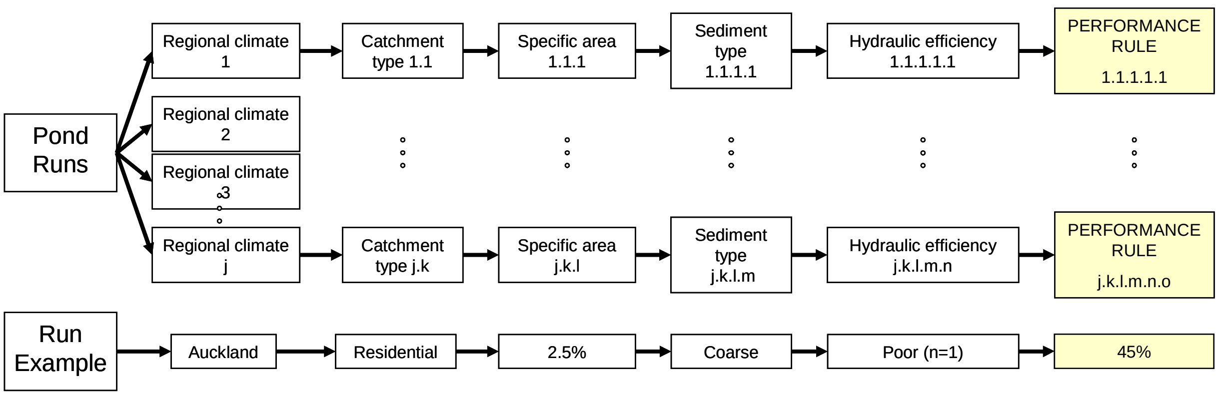


Figure 2 **Building the performance rules using multiple model runs showing an example for ponds.**

A primary concern of C-CALM is the reliability of a simplistic representation of the complex hydraulic and treatment processes in operation within the stormwater network. In an earlier phase of this project, Elliott *et al.* (2006, 2009) showed that it is possible to successfully aggregate these processes within a spatially distributed urban drainage model. They aggregated treatment devices and flow pathways within the

MUSIC model (CRCCH, 2005) for a 0.83 km^2 catchment. An initial total of 810 drainage nodes with uniform flow characteristics in the model were aggregated to 55, seven and, finally a single node. The parameters governing the performance of modelled treatment devices (i.e., dimensions of the devices) were up-scaled proportionally to the greater contributing area upon spatial aggregation. They found that there was some loss of model skill with respect to peak flow, but aggregation had little effect on the predictions of water quality, mean discharge and baseflow when there were uniform soil properties and sizing of devices relative to the source area. That is, treatment devices designed according to the same criteria for contributing areas with similar flow characteristics can be aggregated. Their findings have direct relevance to C-CALM as the model will allow users to aggregate treatment devices within a subcatchment and with similar design into a single device. .

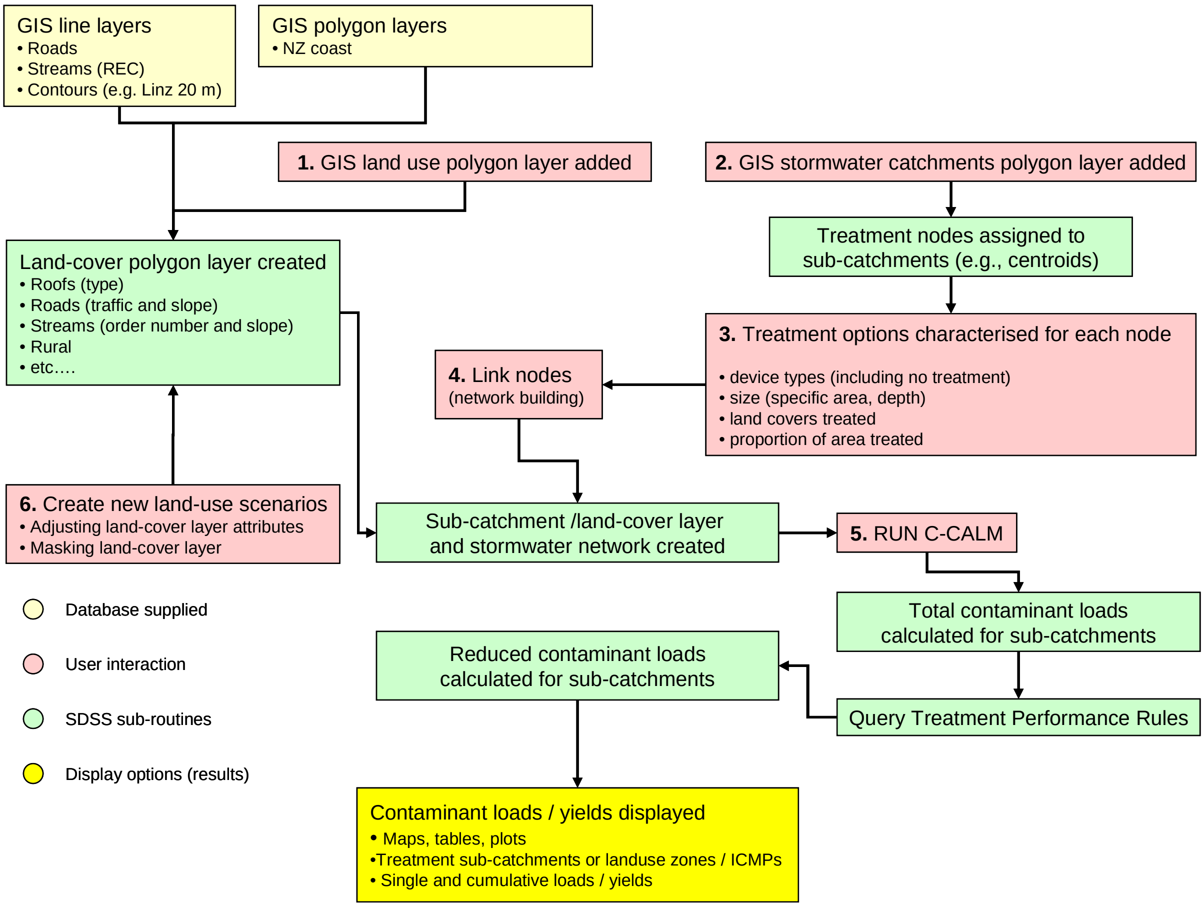


Figure 3 **The C-CALM SDSS showing the links between the spatial database, analytical tools, user interaction, and display of results. Steps the user takes to run C-CALM are numbered, note that evaluation of landuse change and treatment options will require the user to reset the model inputs.**

2.2 Intended users and applications

The C-CALM SDSS is being developed as a planning tool rather than an operational model for stormwater management. Fundamental to the C-CALM project is the difference between urban stormwater planning, design and operation. Operation refers to the day to day decisions relating to stormwater management. Such applications require detailed spatial representations of surface flow pathways, treatment devices and the reticulated stormwater network within physically-based drainage models which are typically run with time-steps of five minutes or less. Design refers to the sizing of stormwater individual network components according to a set of pre-defined criteria (see review of methods in Clar *et al.*, 2004). Finally, the purpose of planning is to reduce the long-term impacts of changes in land use and water management on the wider environment. These modelling tasks are related and often include similar modelling routines, however, the spatial and temporal resolution and the degree of model complexity can differ significantly.

Each region in the country has different mandates and requirements when it comes to planning for water management. For instance, in the Auckland region, Territorial Authorities (TLAs) receive part funding from the Auckland Regional Council (ARC) to prepare ICMPs. These identify the risks posed by stormwater runoff and outline how those risks can be minimised though stormwater management and treatment. Whilst they are not statutory documents, ICMPs are intended to provide long-term strategic direction to stormwater management at the district level. The ICMP is intended to act as a blue-print for development within the catchment area and the primary intention is to avoid, remedy or mitigate adverse effects of stormwater on the receiving environments. The area covered by an ICMP is typically managed in smaller drainage units often known as stormwater management units (SMUs) which are either based on natural or engineered sub-catchment boundaries.

Stormwater managers must grapple with a range of possible development scenarios at the SMU level, including water treatment, which could have an impact on the direction laid out in the ICMP. Take the following situations:

- A land-owner has approached the TLA about the possibility of re-developing an industrial park into a brown-field residential estate of high density housing.

- Developers would like the TLA to rezone farmland on the rural-urban fringe for commercial development or life-style blocks.

- A pollution sink has been identified in a local estuary and the TLA is required to find the source and remedy the situation.

- The Regional Authority requires resource consents and ICMPs to be updated for any new development.

- The Regional Authority wishes to assess city wide sediment and contaminant loads to identify problem catchments.

In each case, the users need to know the long-term impact of land use change and stormwater management on receiving environments. They need the information quickly and do not have the resources available for explicit modelling in an operational model. It is for this type of basic application that C-CALM is being developed.

Depending on data availability, treatment devices initially to be included in C-CALM include detention basins (i.e., wet ponds and wetlands), media filters, swales and infiltration strips, and porous paving. Sound water management should follow the principles of triple-bottom-line (TBL, Elkington, 1997) which gives equal weight to economic, social and environmental factors for a sustainable outcome (Taylor and Fletcher, 2005). Thus the choice of device and its dimensions at a particular site is related to:

1. The contaminant source land use and land cover type

2. The type, concentration/load and physical characteristics of the contaminants to be treated

3. The required removal efficiency

4. The size of the area to be served (i.e., source, site or catchment control)

5. Cost effectiveness over the life time of the device

6. Impact on receiving environment

7. Existence of historical devices which can be modified for treatment (e.g., sedimentation ponds remaining after bulk earthworks)

8. Land value

9. Land availability

10. Site access (including the need to keep emergency corridors open)

11. Proximately to building foundations and other infrastructure

12. Stakeholder needs (e.g., aesthetic value, cultural mores, health and safety considerations)

At this stage, the C-CALM SDSS will have ability to aid decisions based on the first four factors. For the factors five and six, C-CALM will provide data that can be coupled to the Landcare Life-cycle costing model (see Ira *et al.*, 2007) and to ecosystem models such as the harbour circulation and sediment transport model being developed by NIWA. As C-CALM will be developed within GIS, it is conceivable that factors seven to eleven could be incorporated in the future.

3. The Informing Modules

This section outlines the models to be used to develop the performance rule library. General model assumptions are discussed in Section 3.1. The modules are a catchment rainfall/runoff, sediment accumulation/wash-off model (Section 3.2); a hydrological flow and settling pond/wetland model (Section 3.3); and a raingarden water balance and sediment removal model (Section 3.4). All the modules are run with a five-minute time interval.

The treatment devices modelled were largely dictated by the availability of local data. The data requirements for testing of all the modules were that:

- The hydrological data sets for model development were six months or longer.
- Inflow and outflow data should be available for the same events.
- At least three flow events were sampled and analysed for suspended solids.
- The devices modelled were typical of stormwater treatment facilities found in New Zealand.

The modules have been tested for Mission Bay, Tamaki and Auckland CBD for the catchment module, detention ponds in Te Atatu and Silverdale, and the raingarden at the Henderson vehicle testing station.

Given that the purpose of developing these modules is to provide a means of creating a treatment library, the models are deliberately simple with few calibration parameters. That is, they represent generic conditions for archetypical landuse and treatment types. While increasing model detail and complexity will probably increase model performance, providing further options for model parameters will both increase the run-time for the performance rule sensitivity analysis and the memory needed for the C-CALM SDSS (and therefore query times).

Performance rules for other treatment devices and for dissolved metals are based on literature values and are presented in Semadeni-Davies (2008). The C-CALM performance rules will be updated as suitable data become available.

3.1 General assumptions

The quality of stormwater is generally related to the chemistry of urban sediments. The density and the size distribution of particles affect the transport of the solids and associated pollutants (Characklis and Wiesner, 1997). The affinity of contaminants to sediments in urban stormwater has meant that sediment removal is seen as the key to effective water treatment with settling and filtration being the main methods. Removal of dissolved metals is not modelled here and is discussed more fully with respect to C-CALM in Semadeni-Davies (2008).

Although the modules have been tested using measured data where possible, the performance rules have several simplifying assumptions.

- Particle fall velocities are based on the NURP settling column classes (US EPA presentation of the Nationwide Urban Runoff Program, cited in Driscoll *et al.,* 1986); the original distribution is scaled up and down as a proxy for finer and coarser particle size distributions (PSD).

- The fractionation of particulate metals is divided in the same relative proportions of sediments in the PSD - this means the removal efficiency for metals held by each particle size band will also be the same as the sediment removal efficiency for that band.

It should be noted that none of the data sets used for model development contained information about sediment characteristics. Clearly, these assumptions will limit the ability of C-CALM to simulate the removal of sediments and associated particulate metals, however, given the wide spatial and temporal variation of the characteristics of sediments and heavy metals in stormwater caused by differences in source, water chemistry and rainfall dynamics, they offer a workable way forward for the derivation of the performance rules. The discussion below outlines some of the complexities associated with the assumptions.

3.1.1 Particle size distribution and fall velocity

Particulate matter in storm water ranges from nanometre-sized colloidal organic material to millimetre-sized sand, silt and gravel - more than six-orders of magnitude (Makepeace, 1995). While fall velocity is dependant on grain size, other factors such as sediment concentration, particle density, shape and texture, flocculation, and water temperature (i.e., viscosity and density) also have an impact on settling. Actual settling rates are often significantly lower than theoretical fall velocities, particularly with fine particles, due to the influence of water turbulence caused by wind and aquatic fauna (e.g., CRCCH, 2005).

While it is preferable to use measured fall velocities in settling models, this information is not always available. In a review of sediment settling velocity studies, Semadeni-Davies (2007) found that there was more-or-less an even split in the literature between measured values from column settling experiments and fall velocities calculated from PSD. These calculations are sensitive to the parameterisation of sediment properties in the absence of supporting data – for instance, grains are often assumed to be smooth spheres with a constant density equivalent to mineral sands. In either case, there has been criticism in the literature (e.g. Fan, 2004; Lin, 2003) about water sampling methods, particularly when samples have been taken using pre-1990's automatic samplers, which may have introduced a bias towards the collection of fines.

Analytical method notwithstanding, there is wide spread of both reported fall velocities (Figure 4) and PSD (Figure 5) in New Zealand and internationally. The NURP breakdown of fall velocity into five classes each containing 20% of the sediment mass has become a standard internationally (Table 1), and is used, for example, as the basis for design criteria in Auckland (ARC TP 4, 1993). The NURP values were derived from a large number of column settling experiments and represent an average for the United States. Fan (2004) revised PSD and fall velocity values for the US EPA in a study of contaminant inflows to sanitary sewers (including stormwater inflows to combined systems). He reported generally coarser sediments than NURP. Two sets of sediment grain sizes are given by Fan (2004) to reflect the impact of street sweeping which removes coarse sediments (>125 μm) leaving fines on the road surface available for wash-off. CRCCH (2005) present a hypothetical fall velocity distribution based on the PSD of sediment samples from Melbourne and Brisbane, their velocity calculations assume a variable density based on grain size.

Table 1 NURP fall velocity distribution (Driscoll *et al.*, 1986)

Band	Particle mass in stormwater (%)	Settling velocity	
		($ft\ h^{-1}$)	($m\ h^{-1}$)
1	0-20	0.03	0.009
2	20-40	0.3	0.091
3	40-60	1.5	0.457
4	60-80	7	2.134
5	80-100	65	19.812

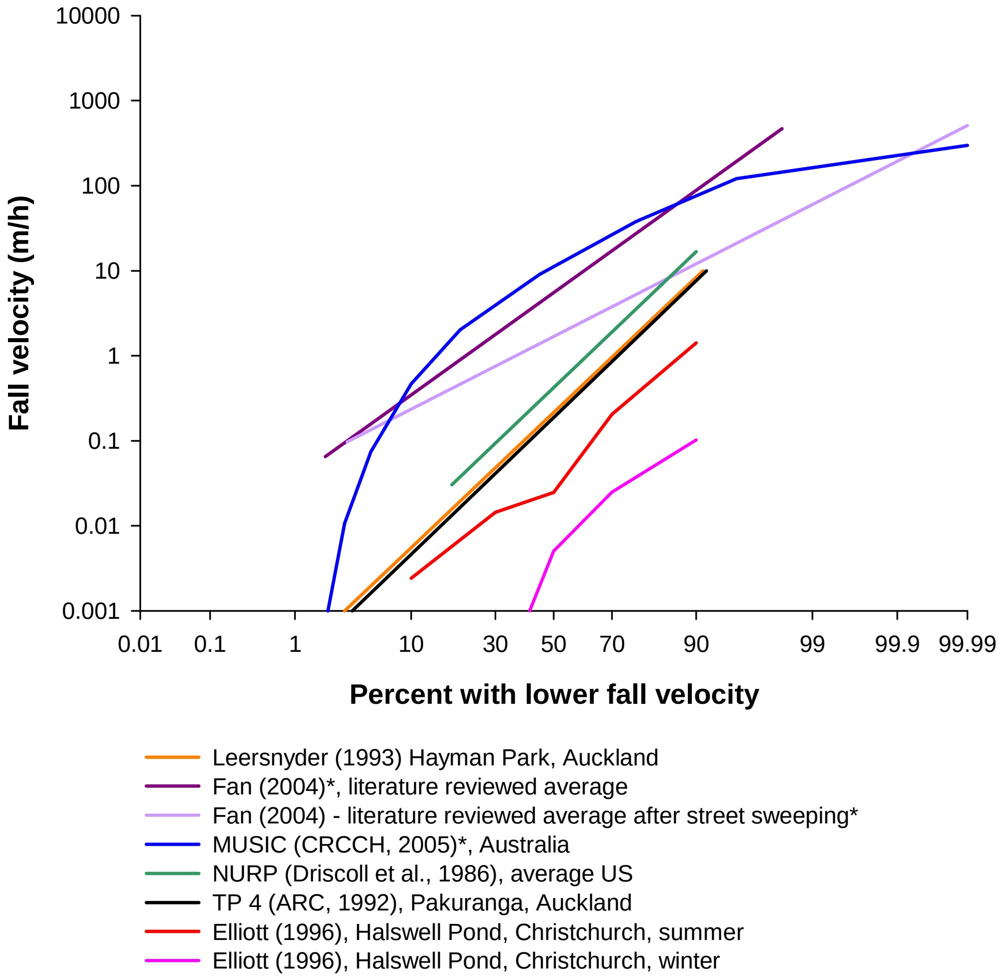


Figure 4 **Summary of probability distributions for sediment settling velocities reported in local and international literature. The fall velocities are both measured and calculated (*).**

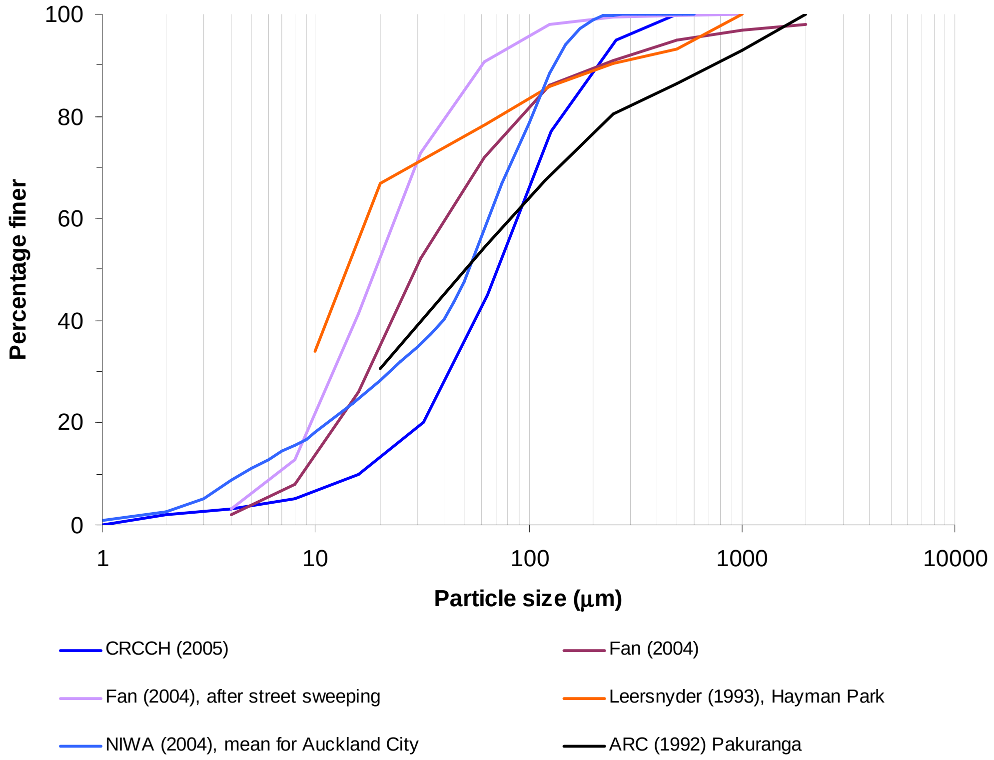


Figure 5 **Summary of sediment particle size distributions in local and international literature (compiled by Semadeni-Davies, 2007)**

In Auckland, NIWA collected some 930 stormwater samples which were analysed for suspended sediment concentration and PSD for Metrowater Ltd and the Auckland City Council (Reed and Timperley, 2004; Timperley *et al.*, 2004a and b). The samples where taken from eight catchments during between seven and 16 rainfall events. A summary of the results is given in Figure 6. It was found that even within Auckland City there is a geographical spread related to soils and land use with Mission Bay having relatively coarse sediments and Oakley Creek fines. Moreover, within the catchments themselves, sediments became progressively finer downstream (probably due to removal by settling of coarse sediments in the reticulated network and catch basins). Additionally, PSD varied from event to event due to differences in accumulation times and wash-off characteristics. Semadeni-Davies (2007) pointed out that in the wider Auckland Region, North Shore City, which has clayey soils, is likely to have finer sediments. While the volcanic soils in South Auckland could lead to coarser sediments (e.g., Pakuranga: ARC, 1992), Leersnyder (1993) found that the PSD and settling rates for sediments from the Hayman Park pond in Manukau were consistent with sediments finer than NURP.

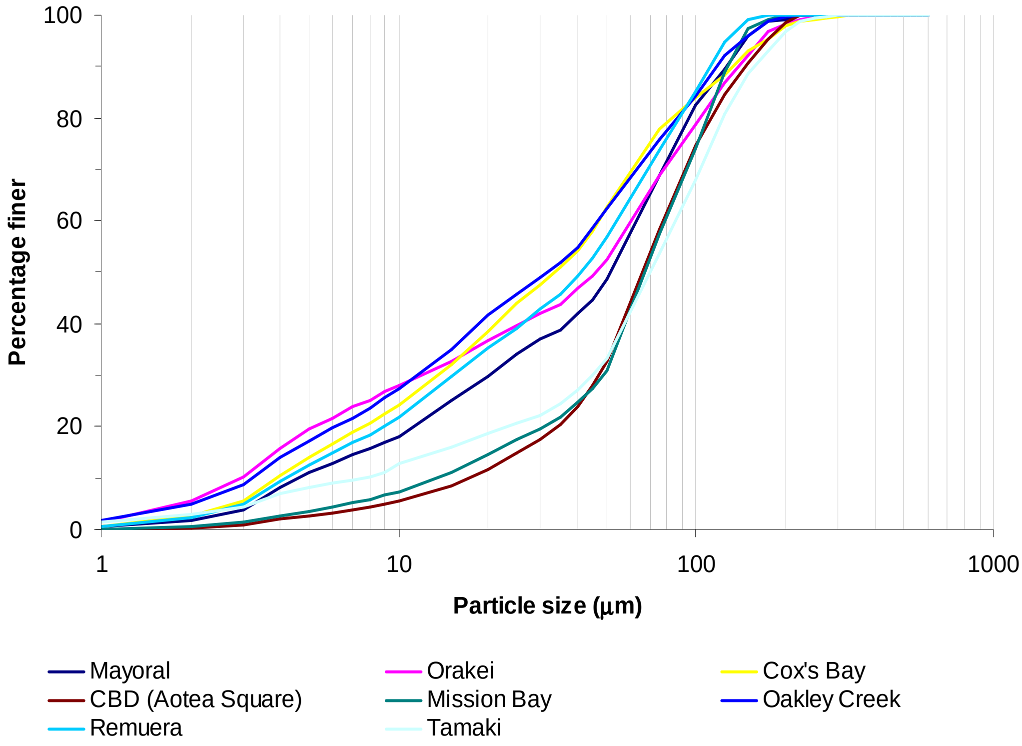


Figure 6 **Particle size distributions found in Auckland (derived from Reed and Timperley, 2004; Timperley *et al.*, 2004 a and b)**

Settling rates in other parts of the country are likely to be just as disparate. To illustrate, Elliott (1996) carried out settling column tests for sediments in inflow to the Halswell Junction pond in Christchurch. He found that the settling velocities were 10 to 100 times less than the average settling velocities reported by NURP. Moreover, there was a seasonal difference with the finest particles sampled in winter, it was suggested that this was due to wet conditions aiding the dispersal of fines. The sediment also settled more slowly than the Manukau sediments (Leersnyder, 1993). This slow settling may be a characteristic of the loess soils in the Christchurch area. The median settling velocity was 0.003 m h^{-1} to 0.02 m h^{-1} which, back calculating using Stoke's Law, corresponds to spherical sand particles of about 1 μm to 2.5 μm (clay). Elliot (1996) found that 90% of the sediment had a settling velocity corresponding to a particle of diameter less than 5 μm to 20 μm.

There is a similarly wide range of particle characteristics important for settling reported in the literature. The following discussion takes density as an example. Stahre and Urbonas (1990) found that it is helpful to split stormwater particles into two groups which are roughly related to size, the first (silts) with densities between 1000-1160 kg m^{-3}, and the second (coarser grains such as sands) greater than 1160 kg m^{-3}. Other authors have stated that density is also a function of sediment type, generally, inorganic particles have a higher density than organics (Karamalegos *et al*, 2005). Butler *et al.* (1996) found that organics in stormwater had a density range of between 1100 and 2500 kg m^{-3}. Sansalone and Triboullard (1999) stated that abraded vehicular matter from tyres has a large range in density (1600 – 4000 kg m^{-3}) and particle diameter (1-104 μm) respectively. Lin (2003) found that organic matter in stormwater had a greater grain size, consisted of leaves and other plant materials and had a density range of between 1400 and 2300 kg m^{-3}. Unlike Stahre and Urbonas (1990), he found that the particles less than 425 μm had a more or less constant density of around 2500 kg m^{-3} which is similar to mineral sands. Andral *et al.* (1997) found relatively less variation with size, even so, there was a definite increase in density with particle size up to 500 μm. The MUSIC model (CRCCH, 2005) assumes a variable density with particle size, based on Lawrence and Breen (1998) *that ranges from 1100 to 2600 kg* m^{-3} for particles between 1 and 256 μm respectively.

In light of the above discussion and the need for C-CALM to be simple and intuitive, the pragmatic approach adopted here is to base settling velocities and PSD on the NURP findings. The settling velocities used to develop the performance rules take the five NURP bands as representative of a medium PSD. The velocities are scaled down by factors of ten and two respectively to simulate fine sediments, and up by factors of two and ten to simulate coarse sediments (Table 2). PSD is calculated from the fall velocity using Stokes' Law (grain size < 100 μm) or Rubey's Equation (grain size > 100 μm) assuming a variable density (i.e., CRCCH, 2005). For local data reported as fall velocities, the scaled medium fine fall velocity distribution is close to those reported for South Auckland (ARC, 1992; Leersnyder, 1993) while the fine distribution is close to the summer fall velocities reported at the Halswell pond in Christchurch. For local data reported as PSD, the coarse PSD is similar to the average for Auckland City (Reed and Timperley, 2004; Timperley *et al.*, 2004 a and b).

Table 2 **Fall velocities and PSD used to develop the performance rules**

Band	Particle mass in stormwater (%)	Density* (kg/m^3)	Medium Grain - NURP		Fine Grain		Medium Fine Grain		Medium Coarse Grain		Coarse Grain	
			Velocity (m h^{-1})	Grain size (µm)	Velocity (m h^{-1})	Grain size (µm)	Velocity (m h^{-1})	Grain size (µm)	Velocity (m h^{-1})	Grain size (µm)	Velocity (m h^{-1})	Grain size (µm)
1	0-20	1300	0.009	4	0.001	1	0.005	3	0.018	6	0.091	9
2	20-40	1600	0.091	9	0.009	3	0.046	6	0.182	12	0.914	27
3	40-60	1900	0.457	16	0.046	5	0.229	11	0.914	23	4.572	51
4	60-80	2300	2.134	29	0.213	9	1.067	20	4.268	41	21.336	91
5	80-100	2650	19.812	78	1.981	25	9.906	55	39.624	110 (94**)	198.12	384**

* Densities taken from CRCCH (2005)
** Calculated using Rubey's equation

3.2 Catchment Surface Flows

The purpose of this module is to provide realistic inflow and sediment wash-off data to the treatment modules; it is not intended to be used for actual prediction of flows for urban catchments. The inputs to the module are rainfall depth and evapotranspiration (disaggregated from Penman daily values), and the outputs are the runoff flow rate and sediment concentration and load. The module couples flow generation from impervious surfaces with build-up and wash-off of surface sediments (see Figure 7), and the simulation routines are very similar to those for surface runoff processes in the US EPA SWMM.

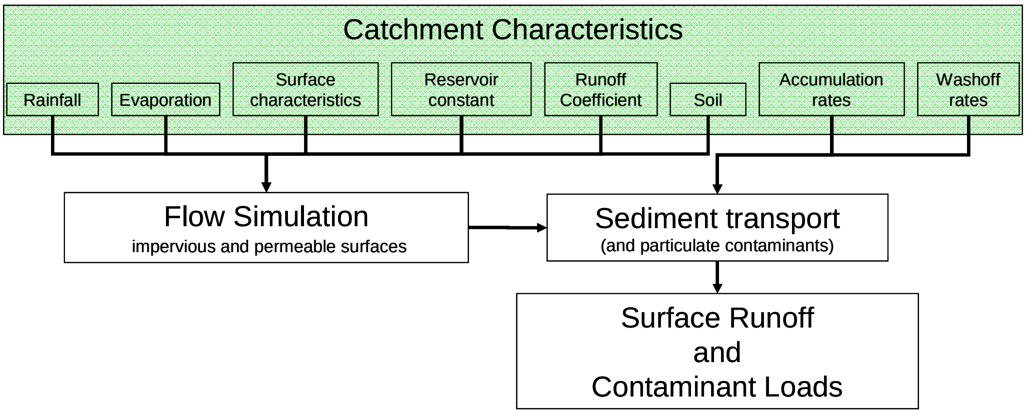


Figure 7 Modelling scheme for surface flows of stormwater and sediments (and associated contaminants).

It is assumed that overland flow pathways from surfaces are direct to the stormwater reticulated network (e.g., over roads and footpaths to gutters or via roof down-drains). This means that the time of concentration is considered to be of the same order as the model time-step of 5 minutes. The catchment is assigned a length of 1 m, and the width and area for a surface contributing to flow are also set at 1 m; in this way, the catchment is conceptually likened to section of road and guttering contributing flow to a reticulated network. Once the flow reaches the network, it is instantaneously routed to the catchment outlet – that is, pipe and channel flow through the network is not modelled. It is recognised that the simplicity of this method could lead to faster response, and greater quickflows, than seen in catchments where flow is attenuated in the network as stormwater moves from source to outlet. However, to include routed flow through the network would require detailed information on the configuration, dimensions and materials of the network, which is outside the scope of this project.

The relative area for each surface type (impervious or permeable) is conceptually modelled as a wide channel with a single outlet; the depth of water accumulated on the surface is the depth of water in the channel, and the overland flow to the reticulated network is the flow from the outlet. The overland flow is related to the rate of change in the volume of water stored in the channel by

$$\frac{dV_s}{dt} = A_s \frac{dh_s}{dt} = A_s\left(P - ET\right) - Q_s \qquad \text{Equation 1}$$

where the subscript s denotes the surface type, V_s (m^3) is the volume of stored water, h_s (m) is the depth of stored water, A_s (m^2) is the relative area for the given surface type, P ($m\ s^{-1}$) is the precipitation depth, ET ($m\ s^{-1}$) is the evapotranspiration rate, and Q_s ($m^3\ s^{-1}$) is the overland flow. For an urban catchment with both impervious and permeable surfaces, Equation 1 is calculated separately for each surface type, and the total discharge is the sum of the flow from both.

For each surface type, overland flow is calculated using a simplification of the kinematic wave equation:

$$Q_s = W_s \alpha h_s^m \qquad \text{Equation 2}$$

where W_s is the width of the catchment impervious or permeable surfaces respectively, and α and m depend on the flow resistance formula adopted. The derivation of Equation 2 can be found in numerous urban hydraulics texts books including Butler and Davies (2000), and Akan and Houghtalen (2003); the equation is generally considered to be adequate for overland flow, and is the same as that used to determine surface flows to gutters by SWMM (albeit with the width, slope and α coefficient calibrated together as a single combined variable).

For overland flow from permeable surfaces:

$$m = 3, \text{ and } \alpha = \frac{8gS}{C\nu}$$

where g is the acceleration due to gravity (9.8 $m\ s^{-2}$), S is the catchment slope, C is the Chezy coefficient and ν is the kinematic viscosity of water ($m^2\ s^{-1}$). The Chezy coefficient is calculated from Manning's n, assuming Reynold's number Re = 500:

$$C = \frac{500^{1/6}}{n} \qquad \text{Equation 3}$$

where in this case, n is assigned a value of 0.1 (the effective roughness for grass, Akan and Houghtalen, 2003).

For flow over impervious surfaces:

$$m = \frac{5}{3}, \text{ and } \alpha = \frac{\sqrt{S}}{n}$$

where Manning's n is assigned a value of 0.02 (the effective roughness for concrete and asphalt, Akan and Houghtalen, 2003). Note that over impervious surfaces, a depression storage of 0.7 mm (equivalent to the depth of water required for flow initiation from asphalt, Niemczynowicz, pers comm.) is removed per rainfall event. Depression storage is assumed to evaporate rapidly (that is, ET in Equation 1) and is set to zero after 3 hours.

Baseflow from permeable surfaces, denoted Q_{base}, (m^3 s^{-1}) is simulated as a single non-linear reservoir. Baseflow is governed by the soil moisture storage, SMS (m), and a recession constant k (s^{-1}), such that

$$Q_{base} = kA_{perm}\text{SMS} \qquad \text{Equation 4}$$

where A_{perm} (m^2) is the relative area of the permeable surfaces. Soil moisture storage (SMS) in Equation 4 is calculated using the continuity equation:

$$\frac{d\text{SMS}}{dt} = P - \frac{Q_{base}}{A_{perm}} - ET \ . \qquad \text{Equation 5}$$

It is assumed that all water at the soil surface for a given time-step is able to infiltrate unless the soil is saturated. While SWMM simulates infiltration (Horton or Green-Ampt options), MOUSE uses the same assumption as here for permeable surfaces. Catchment-wide Horton (infiltration limited) overland flow form permeable surfaces is a rare phenomena outside of arid areas or cold regions with frozen soils. Here, the upper limit of SMS is set by the depth of the soil layer assumed to be contributing to stormwater drainage, denoted d (m), and the available water capacity AWC (the equivalent proportion of the soil depth that can store water). Excess water is assigned to the surface as saturated overland flow, and is routed to the catchment outlet using Equation 2. The default parameters are adjusted for loam soils.

Evapotranspiration is calculated as a linear function of the potential evapotranspiration rate PET (m s^{-1}) and the soil moisture storage SMS:

$$ET = \text{PET}\left(\frac{\text{SMS}}{\text{AWC}\ d}\right). \qquad \text{Equation 6}$$

Note that ET has a maximum value of PET. Daily PET calculated by NIWA for Auckland airport with the Penman method is disaggregated and used as a model input. The daily PET is disaggregated by dividing it into five-minute blocks for the nominal daylight hours from 07:00 to 19:00. Evapotranspiration in urban areas is highly heterogeneous due to the complexity of interaction between diverse land covers, topography and the atmosphere. Variable cloud cover, shading, reduced sky-view, and multiple reflections (and absorption) of solar and longwave radiation mean that the radiation fields over urban areas are extremely complex (Semadeni-Davies and Bengtsson, 1998; Semadeni-Davies *et al.,* 2001). No attempt has been made to adjust the PET for local conditions. Generally, urban drainage engineers are mostly concerned with peak flows from impervious surfaces (flows from permeable surfaces are a minor consideration, usually in the form of sewer infiltration) which means that monthly normal PET values are favoured by commercially available urban drainage models.

The concentration of TSS is simulated using build-up and wash-off equations. The effects of catch pits and street-sweeping on TSS are not simulated, but will be taken into account in C-CALM contaminant load calculations (Semadeni-Davies, 2008). Sediment in stormwater is assumed to originate only from impervious sources: sediment from stream erosion is not simulated, as to do so would require detailed channel routing and erosion modelling, and wash-on from point sources (such as construction sites) is also not simulated. The rate of sediment accumulation available for wash-off is determined by a constant accumulation rate, which is related to local sources and land use (particularly traffic), and a loss rate which is proportional to the accumulated mass (Sartor and Boyd, 1972):

$$\frac{dM_{sed}}{dt} = aA_{imper} - bM_{sed} \qquad \text{Equation 7}$$

where M_{sed} is the total mass of accumulated sediments (kg), a is the accumulation rate (up to 5 kg ha^{-1} day^{-1}), A_{imper} (m^2) is the relative area of the impervious surfaces, and b is the removal rate (between 0.2 – 0.4 day^{-1}, Novotny and Chesters, 1981). This equation implies that there is an equilibrium between sediment accumulation and loss after several days.

The mass rate of sediment wash-off W (kg s^{-1}) (equivalent to the flow of sediment in the stormwater) is related to the intensity of the overland flow from impervious surfaces, such that:

$$W = \begin{cases} 0, & I < Threshold_{depth}, \\ wIM_{sed}, & I \geq Threshold_{depth}, \end{cases} \qquad \text{Equation 8}$$

where w is the wash-off constant (m^{-1}), and I is the runoff depth rate ($m\ s^{-1}$) equal to the overland flow from impervious surfaces Q_{imper} (calculated from Equation 2), divded by the relative area of impervious surfaces A_{imper}. Note that the runoff depth rate must be greater than a threshold value $Threshold_{depth}$ (set to 0.2 mm $5min^{-1}$, or 3×10^{-6} $m\ s^{-1}$) to initiate sediment wash-off.

For each time step, the TSS concentration ($kg\ m^{-3}$) is equal to the mass rate of sediment wash-off W, divided by the total flow rate (overland flow Q_s from both impevious and permeable surfaces, plus the base flow Q_{base} from permeable surfaces).

The choice of method was dictated by the need in C-CALM for a simple conceptual method with low data requirements. It is known as the exponential wash-off method and is commonly used in urban drainage models (e.g., SWMM). DHI MOUSE (MOUSE TRAP) and the Wallingford Software InfoWorks models have variants which are calculated from rainfall intensity rather than runoff depth and include several calibration coefficients. None-the-less, the modelling assumptions are the same. It is known that the method is prone to error (Sutherland and Jelen, 2003*;* Huber pers comm. and 2007); for instance, a first-flush is always predicted. The method also implies that the rate of wash-off is constant, whereas in reality, wash-off varies with, amongst other factors, the type and size of the sediments available, and the rainfall intensity (e.g., splash erosion of bare soil). Sutherland and Jelen (2003), who put forward recommendations for SWMM, suggest that a second coefficient be used to simulate changes in sediment availability. They state that with all else held constant; sediment availability will increase with increasing rainfall intensity and runoff volume, and will decrease with increasing initial loading, particle sizes and pavement roughness.

While there are other methods for simulating sediment concentration, those investigated for possible use in C-CALM do not offer advantages over the exponential wash-off method, given that the purpose is to provide input data to the performance rule modules. The STORMQUAL model that has been used by NIWA for various applications in Auckland (Timperley and Reed, 2005, Reed and Timperley, 2004) calculates the fraction of accumulated sediments (accumulation is simulated in a similar way to Equation 7) that is washed off as a function of the total catchment discharge for the time interval. This method assumes that wash-off for permeable surfaces is the same as for impervious surfaces – the rationale was to allow for quasi-simulation of soil and stream bed erosion. Additionally, calibration required unrealistic assumptions regarding sediment supply and accumulation rates. In some cases, the catchment is effectively supplied with unlimited sediment – that is, the initial accumulated mass was parameterised to a sufficiently high value such that wash-off tracks discharge. STORMQUAL often results in high TSS concentrations for baseflow.

Another alternative is to set a constant TSS concentration such that load is dependant only on flow rates. This method takes neither antecedent rainfall conditions (i.e., time available for accumulation) nor reduced wash-off as an event progresses into account. The method is available to users in SWMM and MUSIC (CRCCH, 2005). MUSIC also includes an option for stochastic simulation of TSS concentration whereby the value for successive time-steps is taken from a log-normal distribution of event mean concentrations (EMC, total event load over total flow volume). The distributions of EMCs were derived for a range of land uses by Duncan (1999). This method does not link the variation in concentration to flow characteristics or antecedent conditions. Moreover, there is no link between successive concentrations. Hence, while the distribution of simulated concentrations may approximate real concentrations over the long-term, the distribution of load may not be correct. The method also precludes the ability to compare simulated concentrations to corresponding sampled concentrations.

3.2.1 Module testing

The surface flow and contaminant transport were tested against NIWA collected flow and TSS observations made for catchments representative of different land uses: commercial/city (Auckland CBD), residential (Mission Bay), and industrial (Tamaki). Data collection is detailed in Timperley and Reed (2005). Data availability for each catchment is summarised in Table 3, note that the catchments did not have on-site rain gauges. For each catchment, the models were run for an initiation period of several months.

Table 3 Summary of data used to test the surface flow module

	Mission Bay	Auckland CBD	Tamaki
Simulation period	Dec 2000 – 30 November 2001	1 February 2001 – 28 Feb 2002	1 Dec 2001- 16 July 2002
Calibration Period	Dec 2000 – June 2001	Feb 2001-July 2001	Dec 2001 – Feb 2002
Monitoring site	Beside Aotea Reserve	Between Aotea Centre and Ferguson Building	University of Auckland Tamaki Campus, Glen Innes
Rain gauge location	Okahu Bay (ARC)	Albert Park (Metrowater)	Constructed from Okahu Bay, Rowe St and Pakuranga (ARC)
Number of events sampled	13	16	13

Literature values were used for the proportion of impervious surfaces to permeable surfaces (i.e., analogous to the runoff coefficient) and accumulation and wash-off rates, these were related to land use (summarised in Butler and Davies, 2000). This was done to test the applicability of simple model parameters to generic modelling of discharge. The parameters are listed in Table 4, the soil parameters are set for loam. Obviously, better fits for both flow and sediment transport could be obtained by calibrating for each catchment. However, the performance rules will be limited in the number of parameters that can be adjusted, hence the use of generic parameters.

With the combination of soil available water capacity (0.16 m) and recession constant (consistent with a hydraulic conductivity of 1 cm/h) used in the model applications discussed below, there was negligible overland flow produced from the permeable surfaces for the catchments simulated.

Discharge

The module was able to simulate stormwater discharges for the catchments with reasonable accuracy (Table 5).

For the Mission Bay catchment, the model had very good fit, though a tendency to slightly underestimate peak flows meant that the total flow simulated was underestimated by around 12%. Figures 8 – 10 give examples of model fit for the catchment.

The discharge simulations for the CBD are given in Figures 11 to 13. Due to the high proportion of impervious surfaces, discharge at the CBD responds rapidly to rainfall and is not buffered by base-flow. Total flow was underestimated by around 8%. Discharge at the CBD was better simulated for the validation period than the calibration period. This is largely explained by anomalies between rainfall and recorded flow. A comparison of the observed flow record against rainfall shows that despite the close location of the rain gauge (about 2 km) to the catchment outlet, there were several rainfall events which did not lead to recorded flow. Similarly, there were some flow peaks without observed rainfalls or with only modest rainfalls. The anomalies seen on 23rd February, 2nd April and 2nd May in particular have an undue impact on overall model fit during the calibration period. If these events are removed from the simulation, the correlation (R^2) increases to 0.65. While there were similar anomalies during the validation period (e.g., 5 September 2001 and 30 December 2001 had recorded rainfalls but no flow), their impact on the overall model fit was not as great.

Figures 14 to 16 give the results of the flow simulation for Tamaki. Note that the rainfall data used for the simulation was constructed from gauges at Okahu Bay, Onehunga (Rowe St) and Pakuranga which could explain some of the difference in discharge. There are some flow events that are not represented in the rainfall records such as 8 May 2002. Like the CBD, it can be expected that some high rainfalls at the gauges may not have resulted in high flow peaks. Total flow was overestimated by 12 %.

All three catchments had anomalies between rainfall and the discharge volume recorded. Indeed, some flow events occurred with no recorded rainfall and vice versa. Auckland rainfall is characterised by localised showers which could explain some of the differences between the simulated and recorded discharge.

The overestimation of flow peaks for all the catchments is likely due to the fact that flow is not routed through the reticulation network. In reality, flow peaks would be attenuated due to differential travel times from different parts of the catchment with different flow pathways and surface types.

Table 4 Model parameters for the catchment surface flow module.

	Mission Bay	Auckland CBD	Tamaki
Area (m^2)	451630	301140	340000
Fraction impervious	0.5	0.95	0.8
Average slope	0.02	0.02.	0.02
Depression storage (m)	7×10^{-4}	7×10^{-4}	7×10^{-4}
Depression dry-time (h)	3	3	3
Available water capacity (volume fraction)	0.16	0.16	0.16
Soil depth (m)	1	1	1
Recession constant (s^{-1})	2.7×10^{-6}	2.7×10^{-6}	2.7×10^{-6}
Sediment accumulation rate ($g/m^2/s$)	6.94×10^{-4} (2 kg/ha/day)	1.22×10^{-3} (3.5 kg/ha/day)	1.74×10^{-3} (5 kg/ha/day)
Sediment removal rate (s^{-1})	2.35×10^{-6} (0.2 /day)	2.35×10^{-6} (0.2 /day)	2.35×10^{-6} (0.2 /day)
Wash-off constant (mm^{-1})	0.2	0.2	0.2
Runoff depth threshold (mm/5 mins)	0.1	0.1	0.1

Table 5 **Summary of model fit for discharge.**

Catchment	Correlation (R^2)			Total flow volume (m^3)				Peak Discharge (m^3/5mins)			
	Calibration	Validation	Entire	Calibration		Validation		Calibration		Validation	
				OBS	SIM	OBS	SIM	OBS	SIM	OBS	SIM
Mission Bay	0.73	0.865	0.75	198838	169853	162123	146663	1094	1150	324	383
CBD	0.54	0.67	0.60	148987	114463	176762	184137	588	882	432	717
Tamaki	0.71	0.57	0.61	48931	44792	107690	131082	442	561	736	904

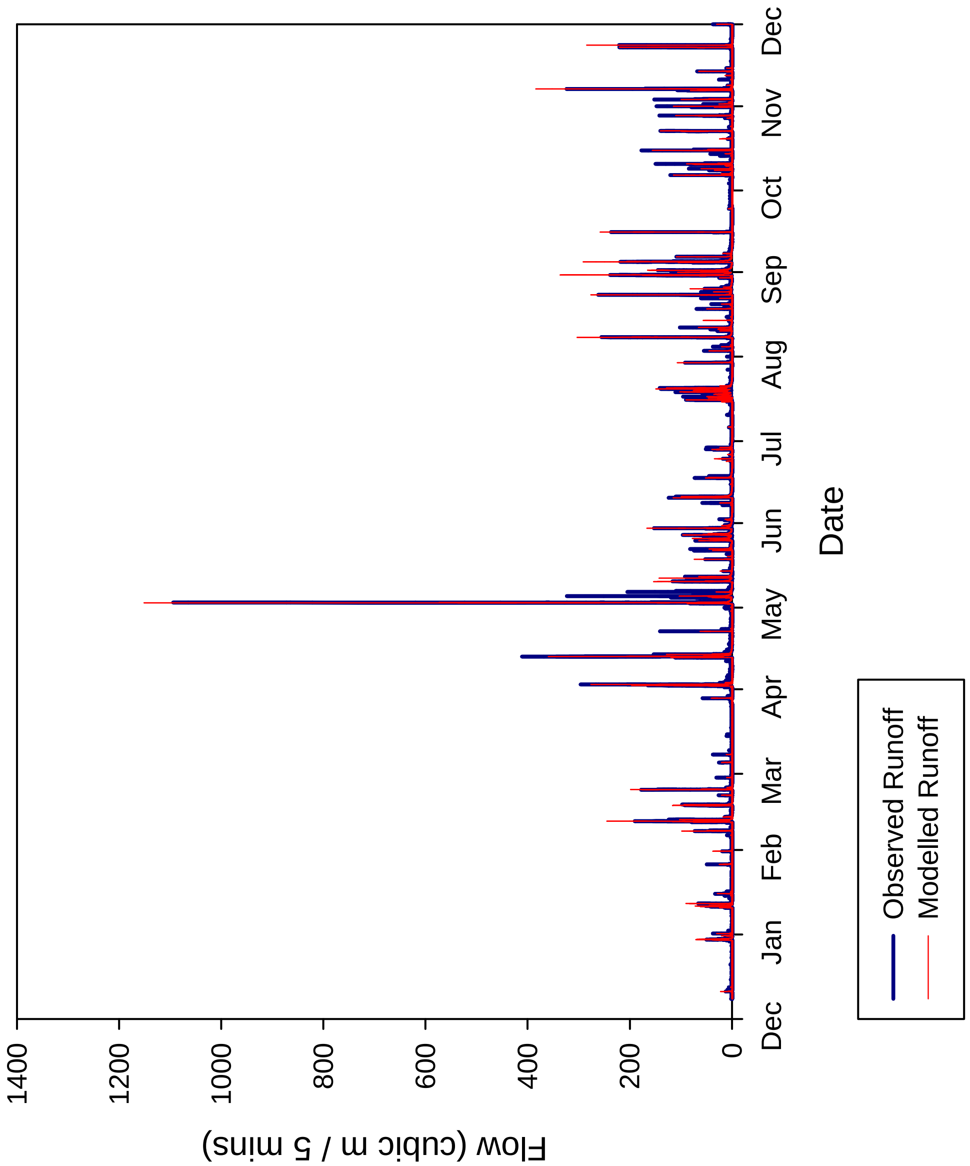


Figure 8 **Observed and simulated stormwater runoff for Mission Bay – December 2000 to November 2001**

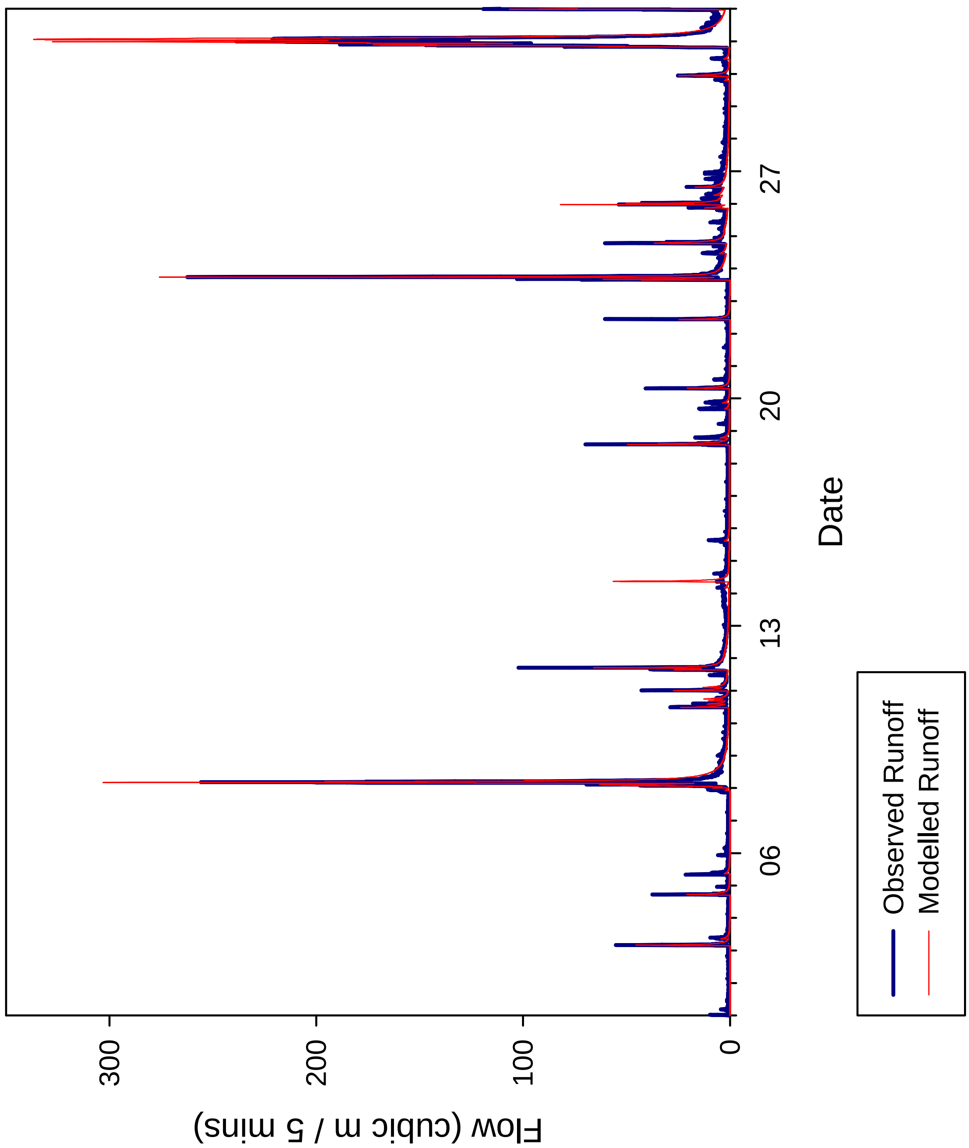


Figure 9 **Observed and simulated stormwater runoff for Mission Bay – August 2001**

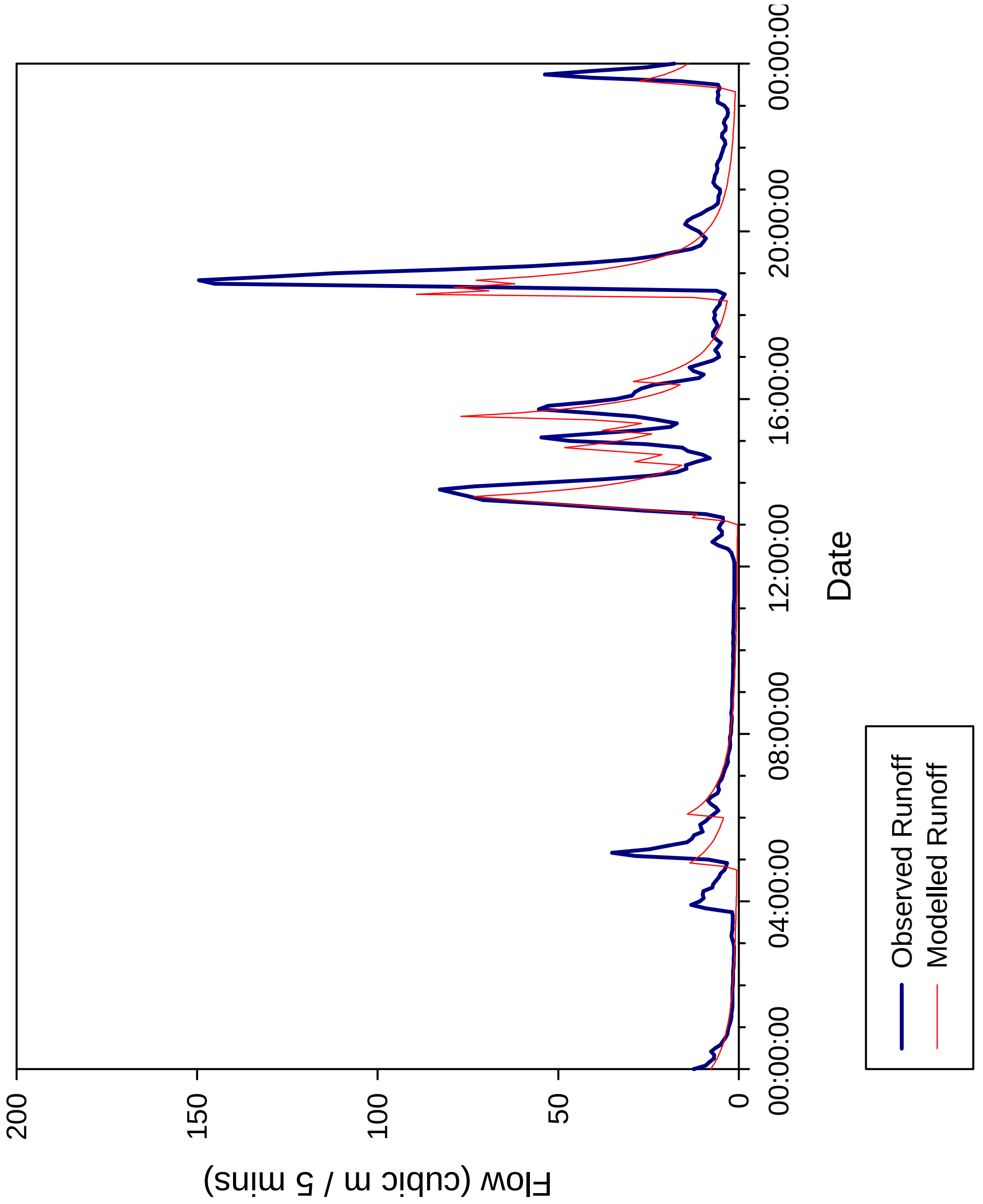


Figure 10 **Observed and simulated stormwater runoff for Mission Bay – 10 October 2001**

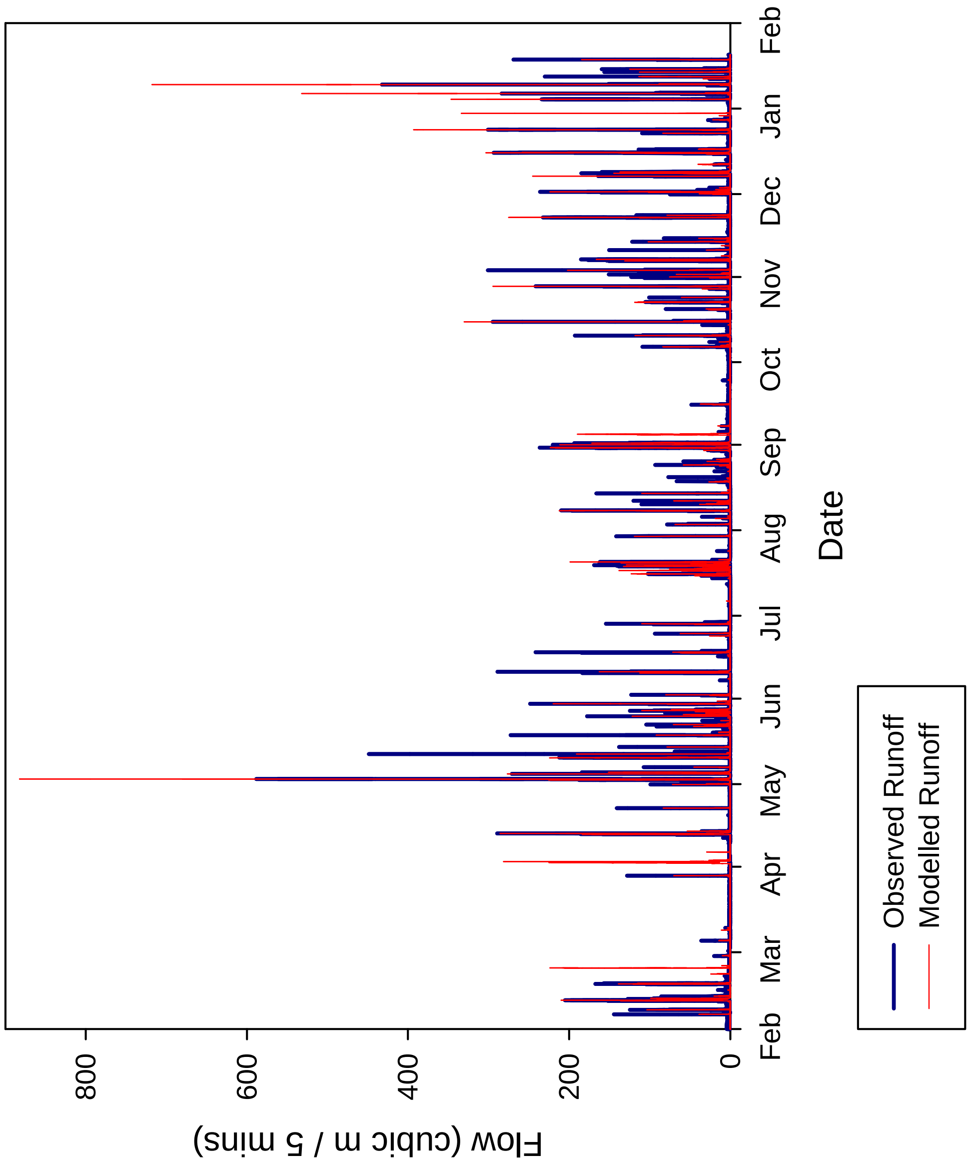


Figure 11 **Observed and simulated stormwater runoff for Auckland CBD, 2001**

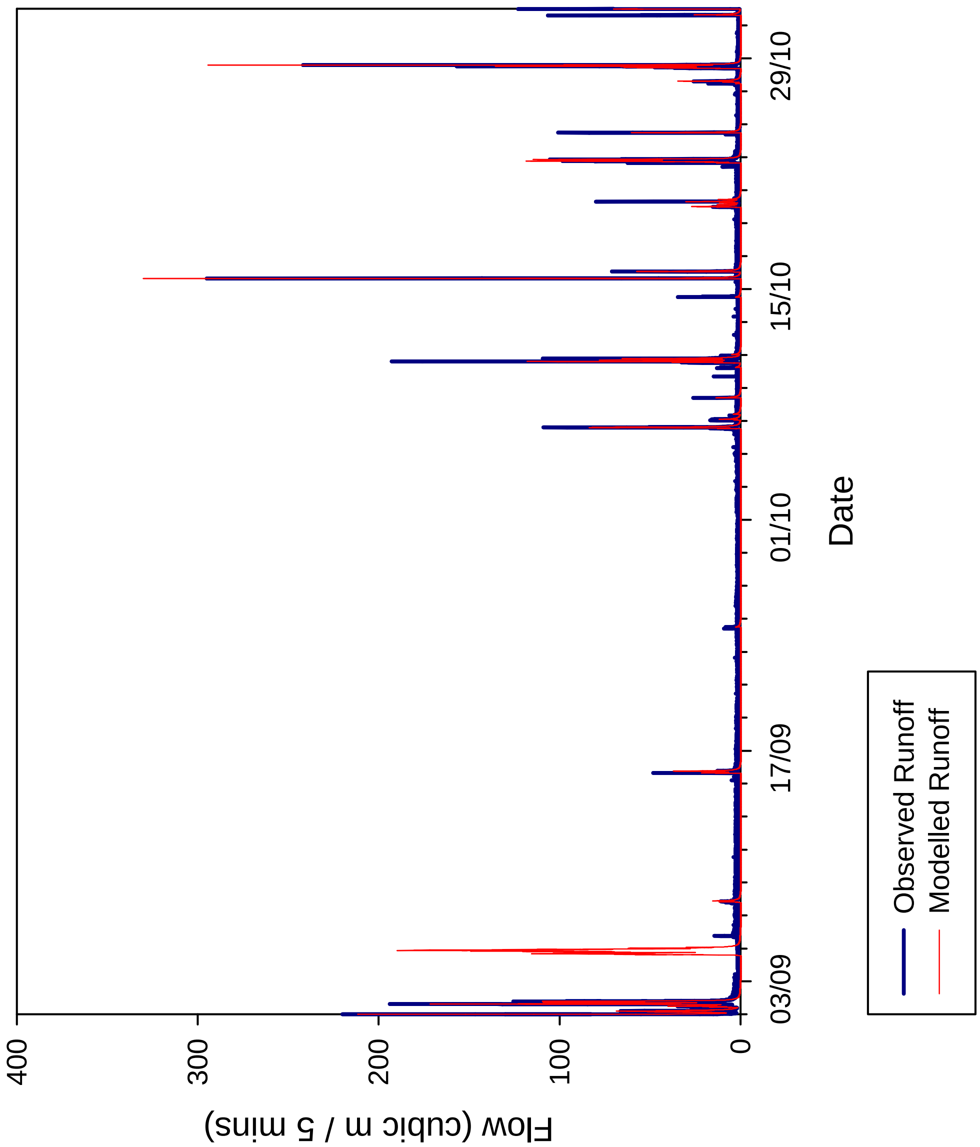


Figure 12 **Observed and simulated stormwater runoff for Auckland CBD, September 2001, note that the flow event predicted for 4 September is due to a heavy rainfall – there was no flow recorded for this event.**

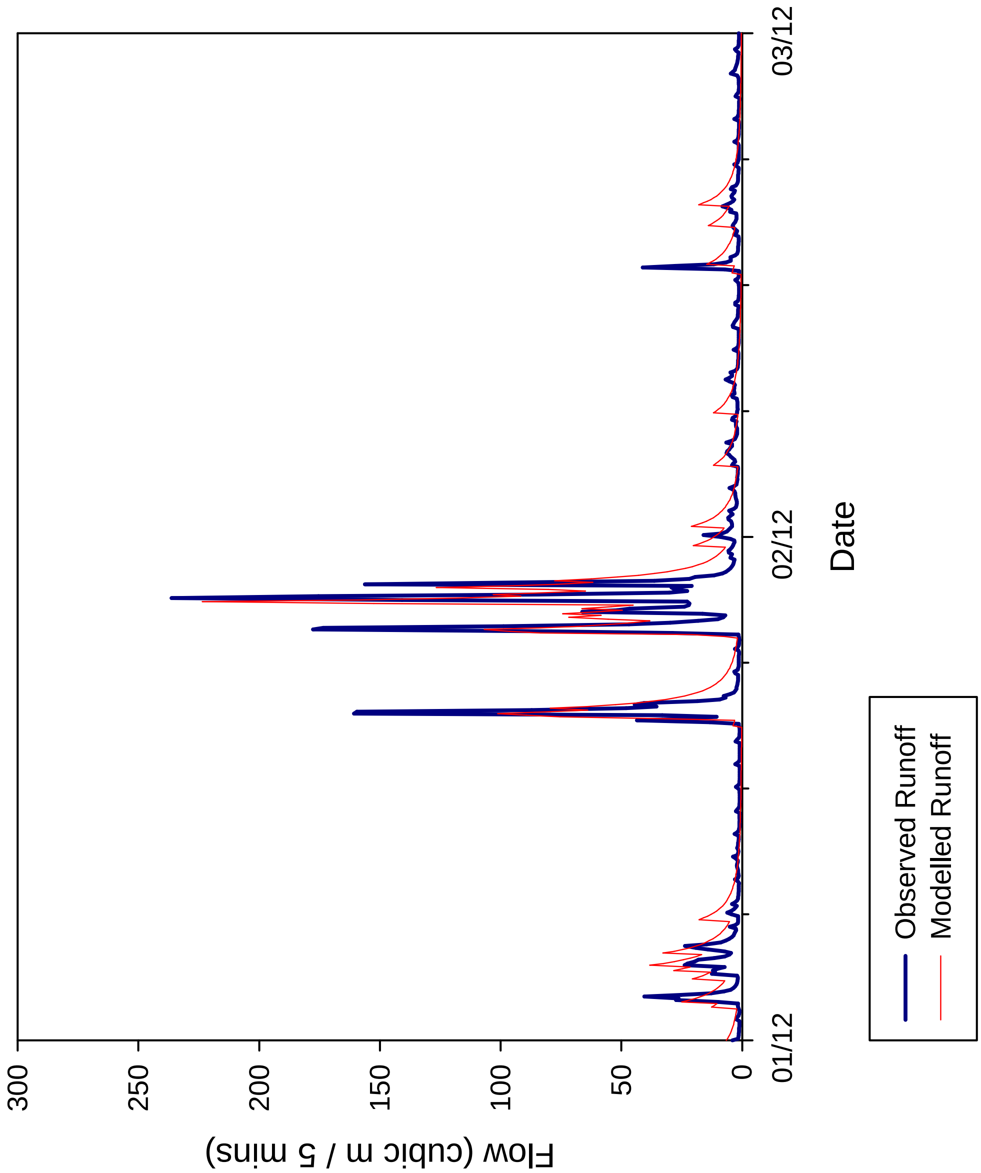


Figure 13 **Observed and simulated stormwater runoff for Auckland CBD, detail for 1-3 December 2001.**

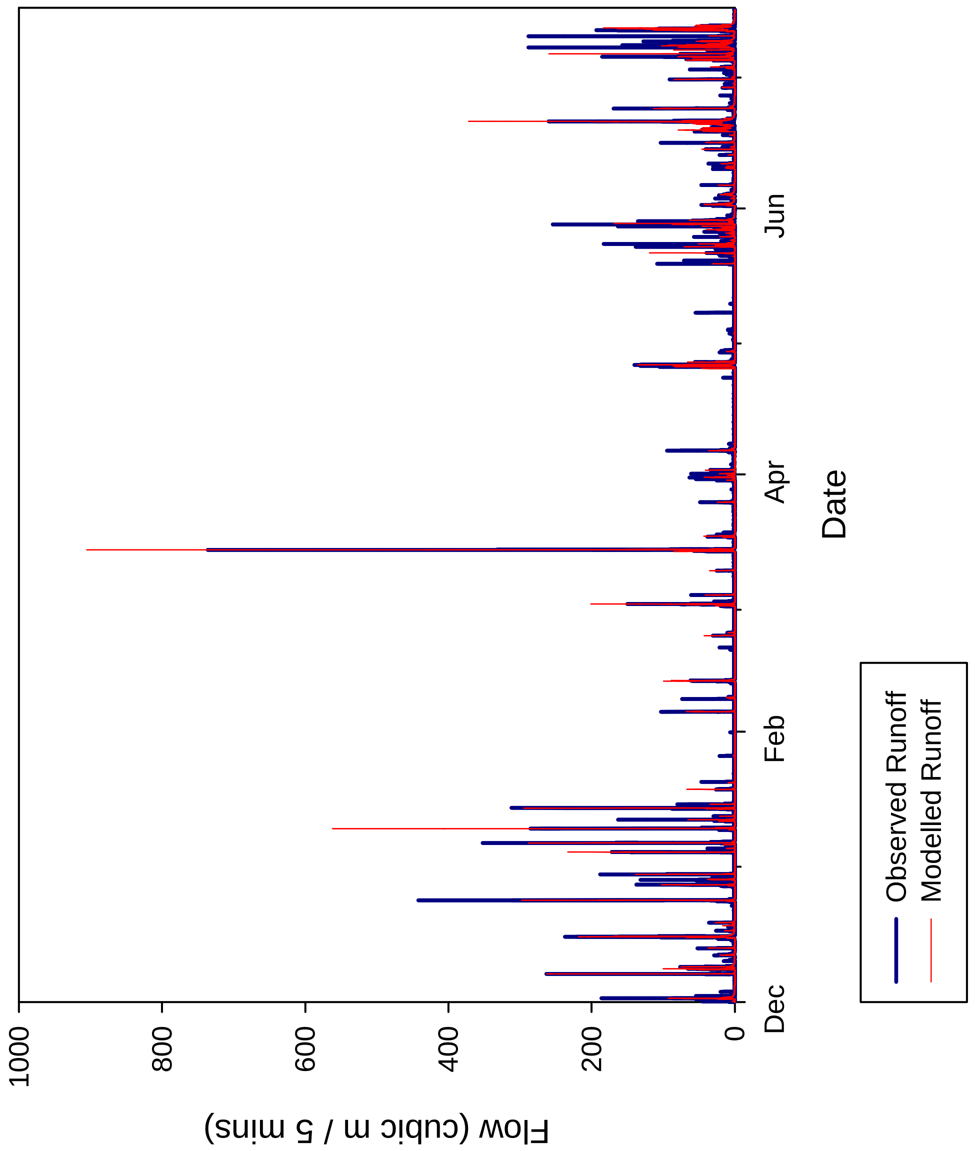


Figure 14 **Observed and simulated stormwater runoff for Tamaki – December 2001 to July 2002**

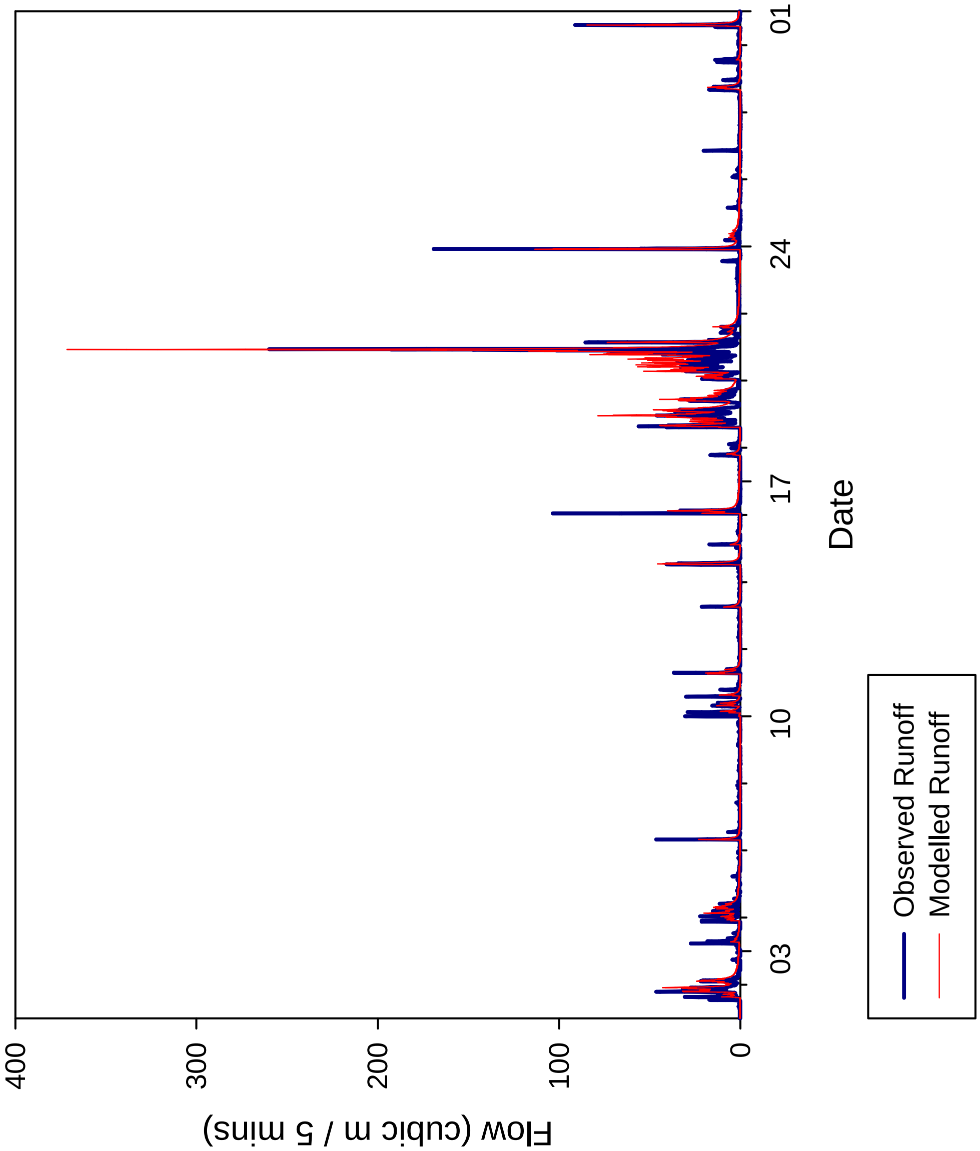


Figure 15 **Observed and simulated stormwater runoff for Tamaki –June 2002**

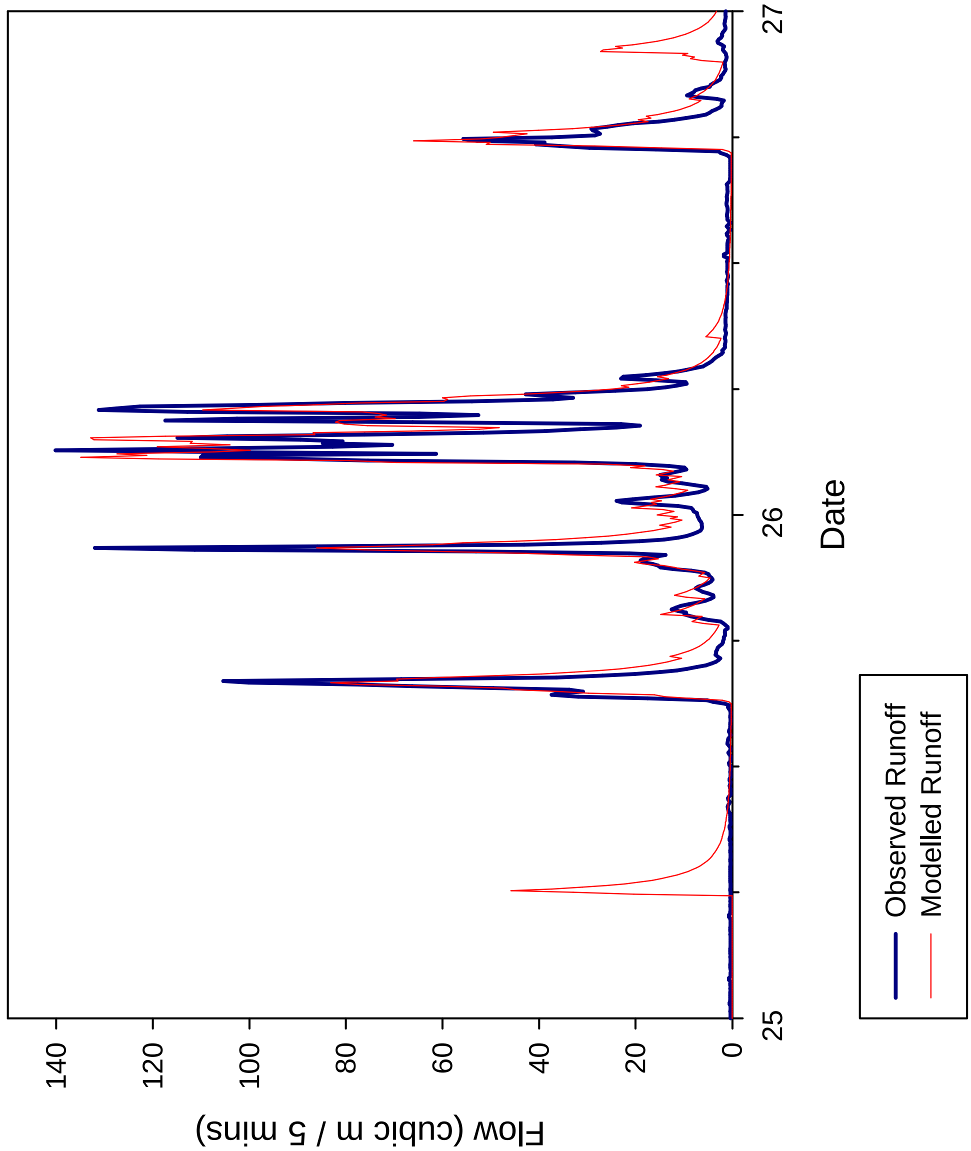


Figure 16 **Observed and simulated stormwater runoff for Tamaki – 25-26 April 2002. Note the model predicts two flow peaks not present in the flow record.**

TSS concentration and load

Figures 17 - 20 show model results for both TSS concentration and load for Mission Bay. The simulation of point concentration was variable with some events having similar concentrations and loads to those recorded and others being greatly underestimated. For instance, there is an increase in concentration for the events sampled in October 2001 compared to those taken earlier in the sampling programme despite similar peak discharges and accumulation times. These increases are not predicted by the model, which, due to the modest discharge, simulated low TSS concentrations and loads. This catchment has some open water streams as part of the net-work which may partially explain the high rates of sediment transport. Indeed, contrary of expectations, Mission Bay had higher sediment concentrations and EMCs than the CBD (commercial) and Tamaki (industrial) catchments for some events. The observed and simulated EMCs for events where sufficient water quality samples were taken to allow calculation are given in Table 6. Note that not all time steps during the sampled events have associated TSS concentrations. With the exception of the October events, the simulated and observed EMCs have similar ranges in values.

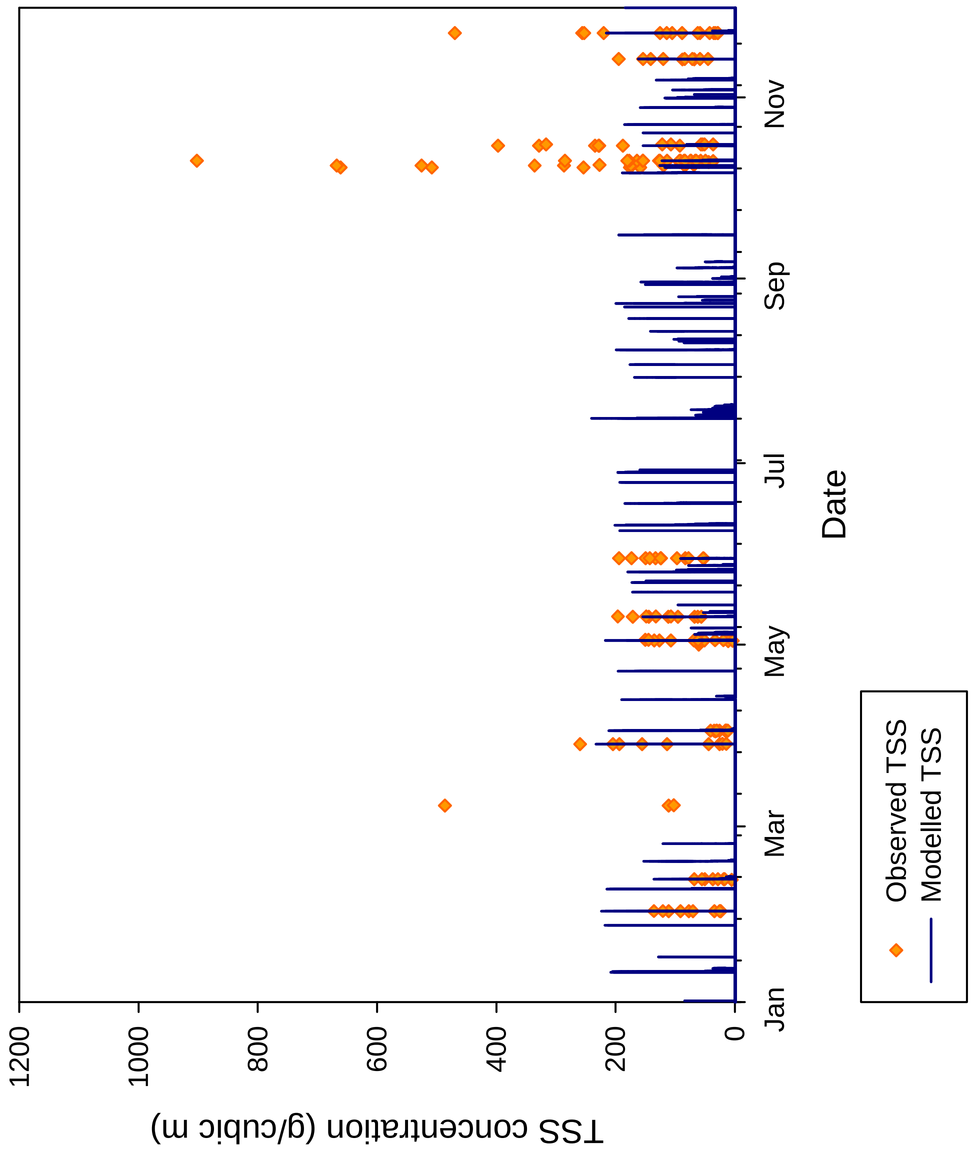


Figure 17 **Observed and simulated TSS concentration for Mission Bay – January to November 2001**

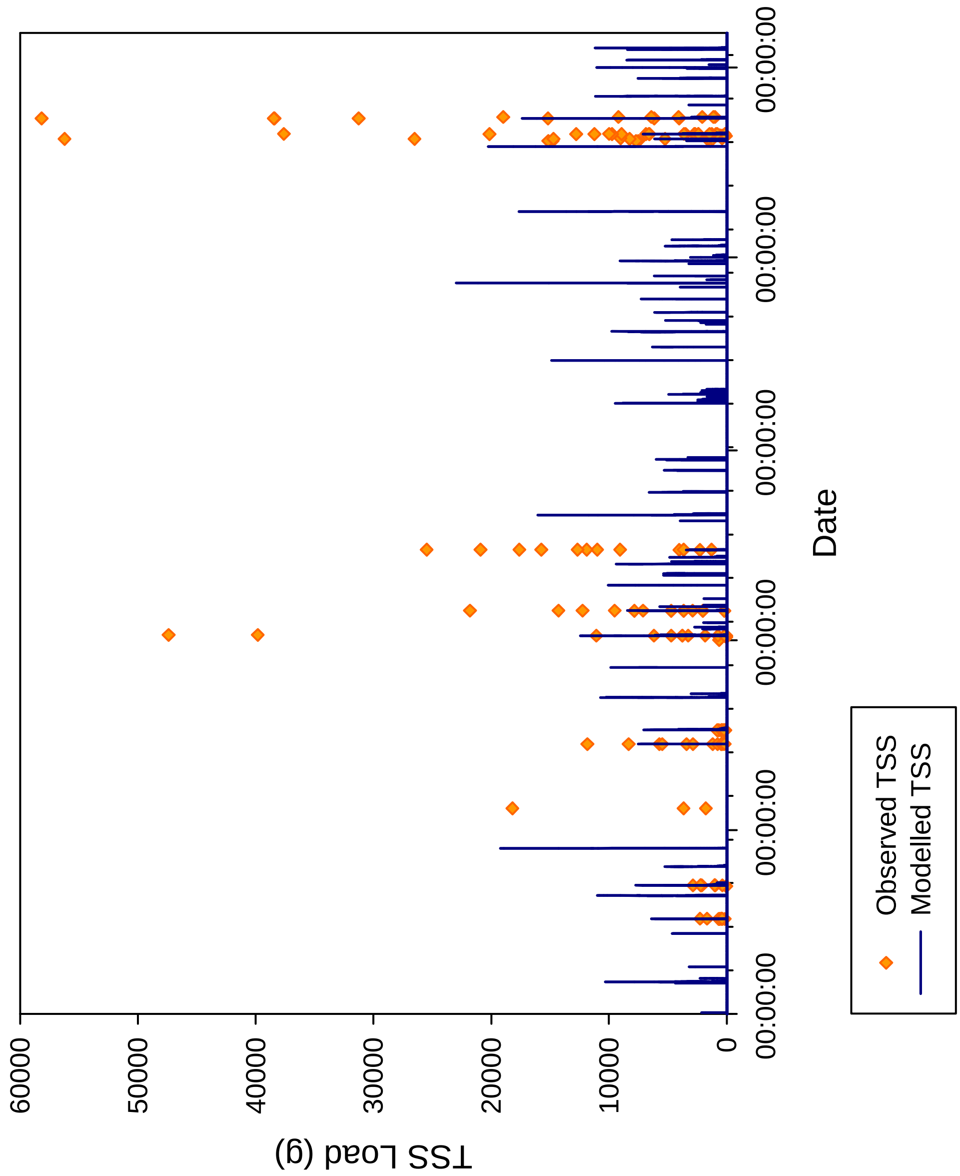


Figure 18 Observed and simulated TSS load for Mission Bay – January to November 2001

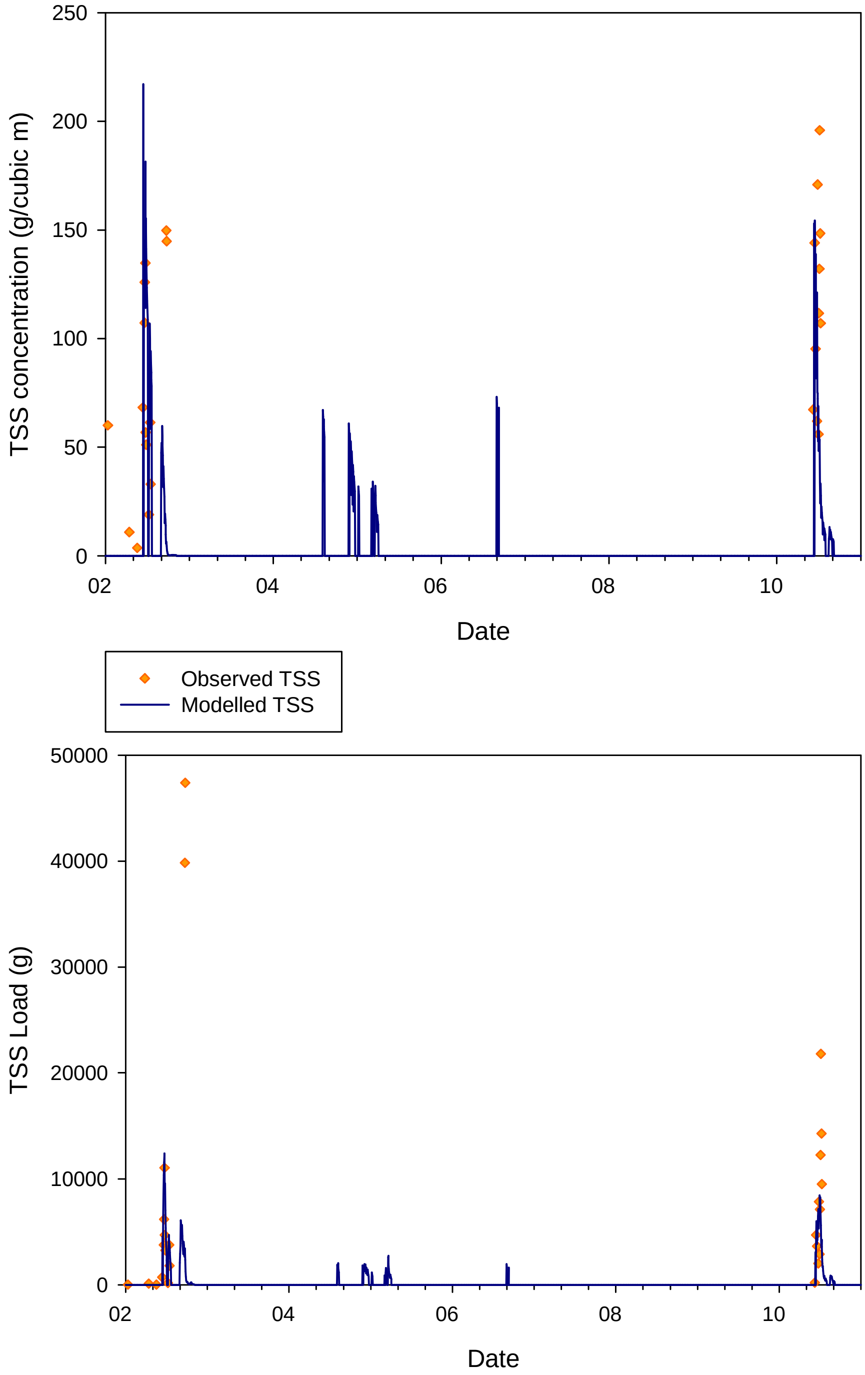


Figure 19 **Observed and simulated TSS concentration (top) and load (below) for Mission Bay – 2 to 10 May 2001**

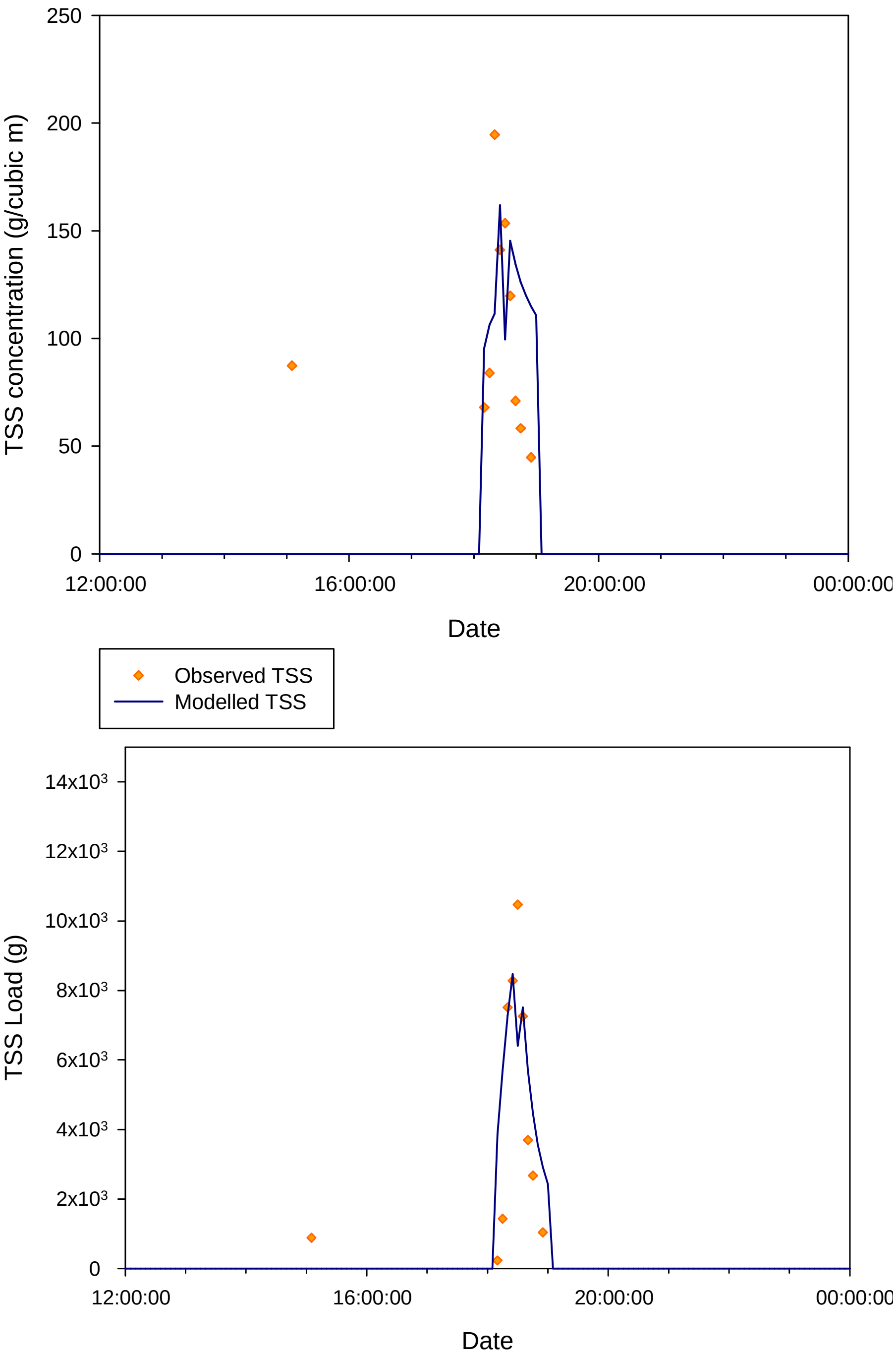


Figure 20 **Observed and simulated TSS concentration (top) and load (below) for Mission Bay – 13 November 2001**

Table 6 **Observed and simulated event mean concentrations for events where calculation was possible – Mission Bay. Note that not all time steps during the sampled events have associated TSS concentrations.**

Event	Observed EMC	Simulated EMC
31/01/2001	62	176
11/02/2001	38	61
28/03/2001	104	183
2/05/2001	78	121
10/05/2001	126	68
30/05/2001	132	25
8/10/2001	346	112
9/10/2001	397	96
10/10/2001	130	73
15/10/2001	263	107
16/10/2001	119	67
13/11/2001	115	120
21/11/2001	119	102

Results for the CBD are shown in Figures 21 to 24 for both concentration and load. Whereas the point concentrations simulated the CBD catchments were in the correct order of magnitude, the assumption of a first flush has led to the concentrations and loads being underestimated in some cases where the maximum concentration occurred with peak flows. EMCs for events where water quality samples where taken over the entire event are given in Table 7. It can be seen that the results are variable with some events having overestimations and other under estimations. However, the magnitude of the EMCs is comparable. The high sediment concentrations seen in January 2002 may be due to wash-on from construction sites; this was observed on several occasions when sediment traps overflowed.

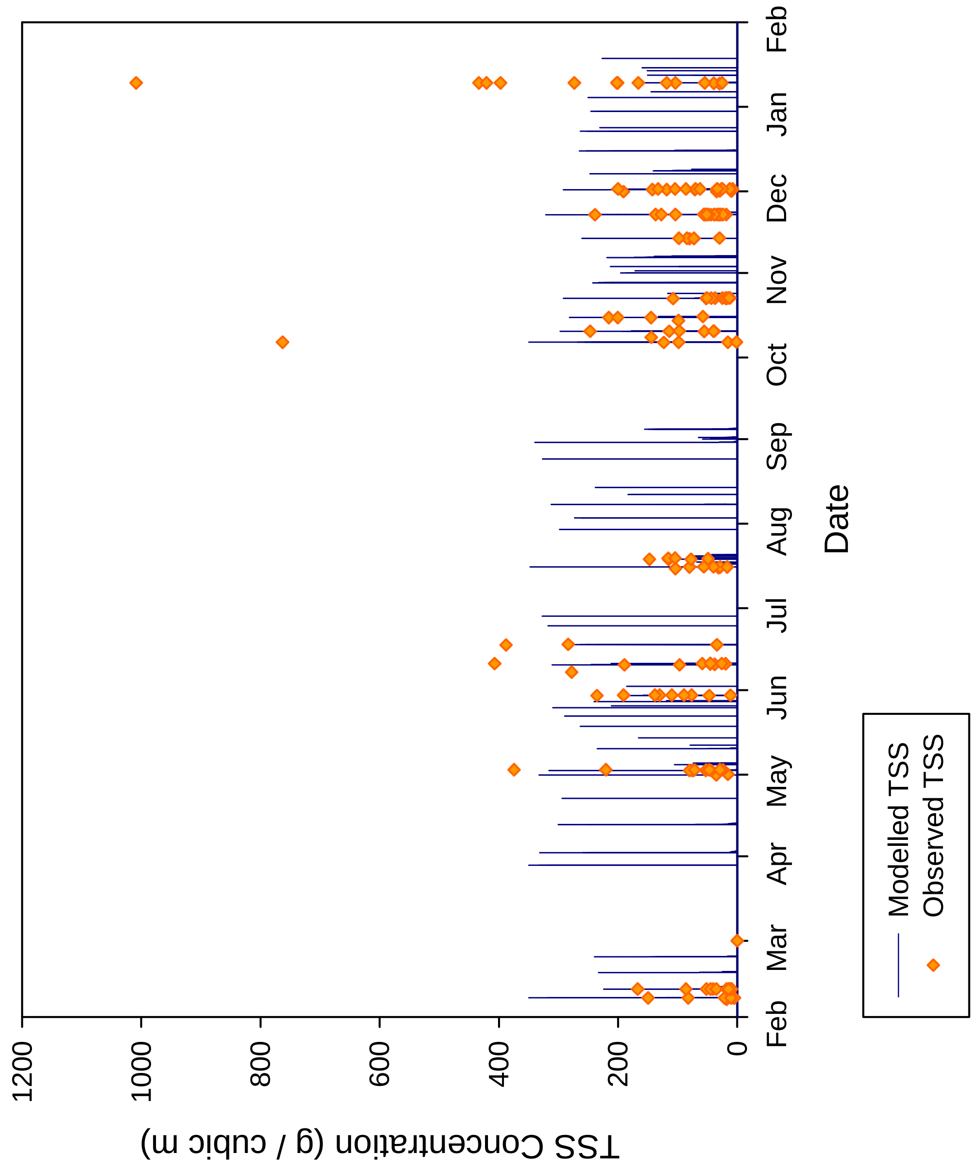


Figure 21 **Observed and simulated TSS concentration for Auckland CBD – February 2001 to January 2002**

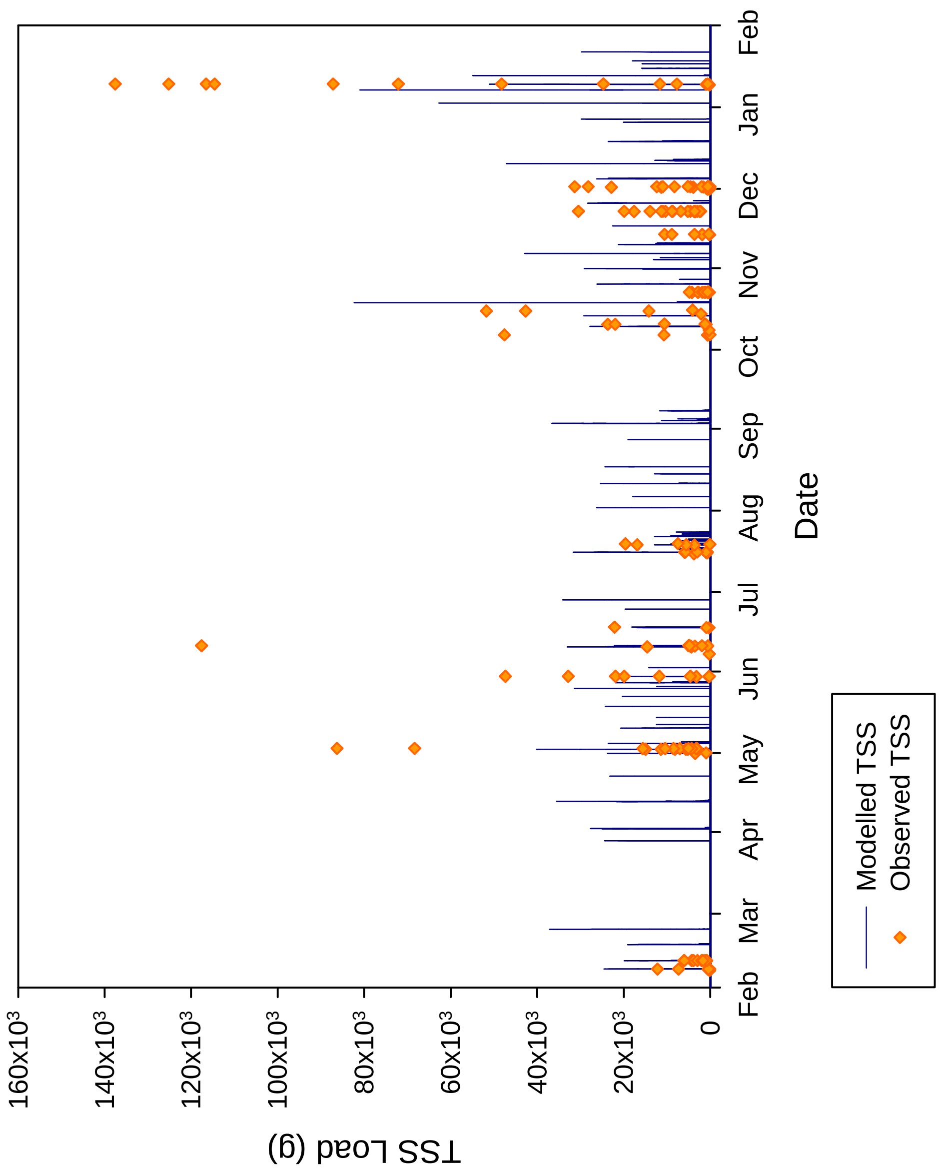


Figure 22 Observed and simulated TSS load for Auckland CBD - 2001

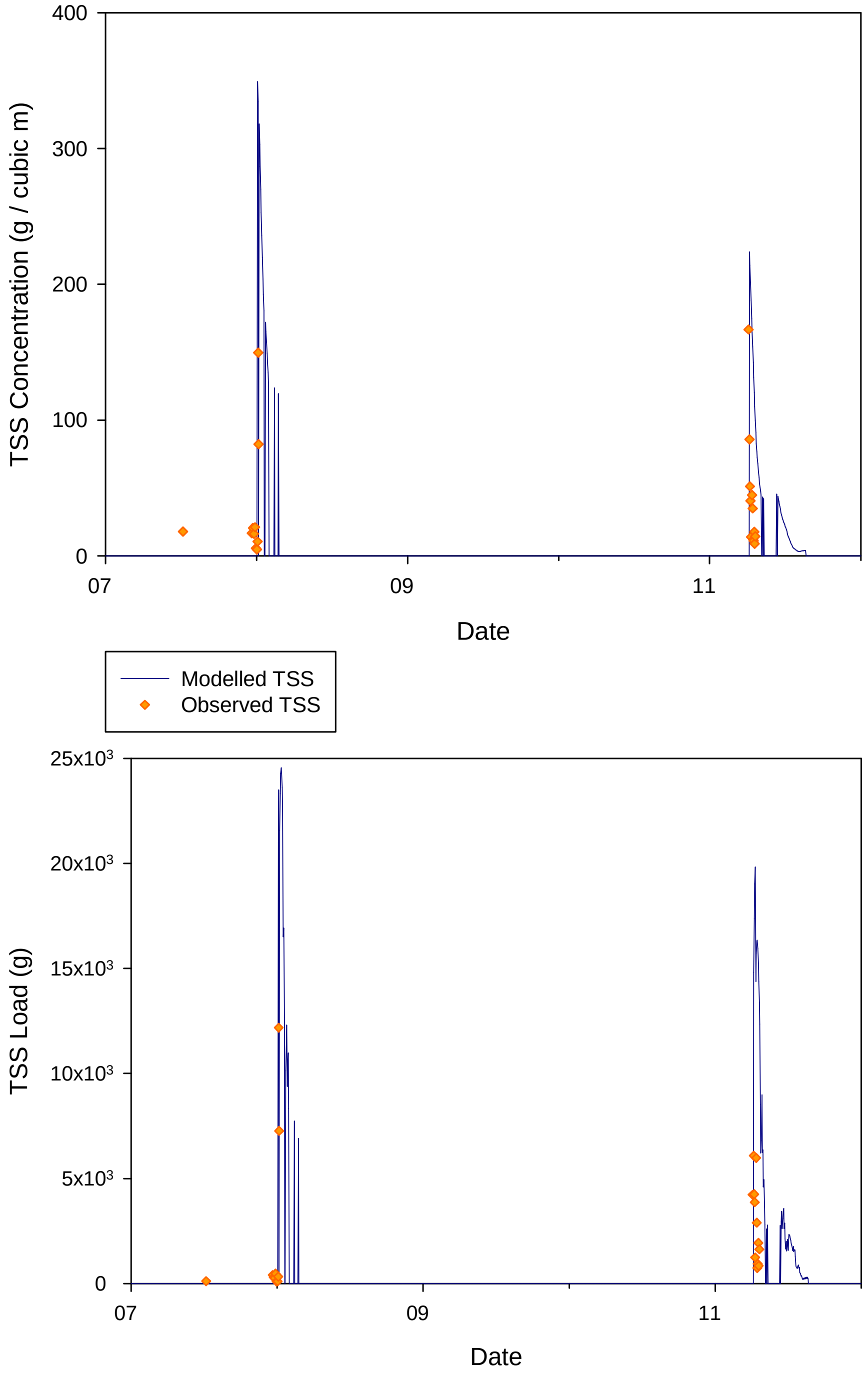


Figure 23 Observed and simulated TSS concentration (top) and load (below) for Auckland CBD – 7-9 Feb 2001

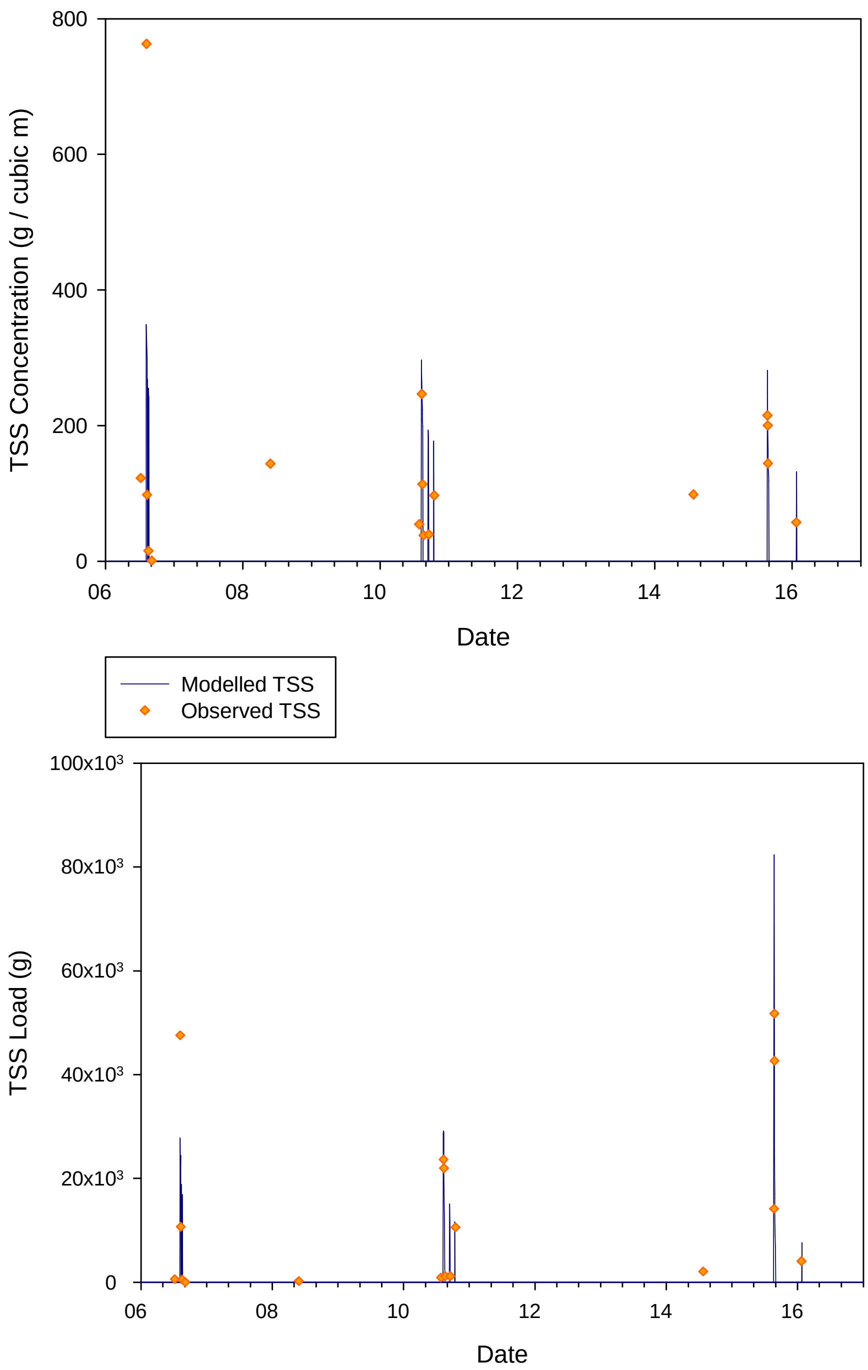


Figure 24 **Observed and simulated TSS concentration (top) and load (below) for Auckland CBD – 6-17 October 2001**

Table 7 **Observed and simulated event mean concentrations for events where the entire flow hydrograph was sampled – Auckland CBD. Note that not all time steps during the sampled events have associated TSS concentrations.**

Event	Observed EMC	Simulated EMC
2/05/2001	92	16
30/05/2001	125	98
16/07/2001	47	202
6/10/2001	277	214
10/10/2001	125	232
15/10/2001	175	199
22/10/2001	29	114
22/11/2001	59	70
1/12/2001	56	88
9/01/2002	27	53

On the whole, TSS concentration and load were modelled well for Tamaki as can be seen in Figures 25 to 28. However, some events such as the 28 and 29 May 2002 had differences in the timing of simulated and recorded peak flows which meant that there was also a mismatch in the timing of the pollutograph. The events on 19, 23 and 27 May had some samples with high sediment concentrations that were not simulated due to underestimation of predicted flows. EMCs for events where there were adequate water samples for calculation are given in Table 8. Despite the mismatch in the point concentrations, there is good agreement between the simulated and observed EMCs for the events (within 25% of the observed value) with the exception of 27 and 28 May where EMC was underestimated.

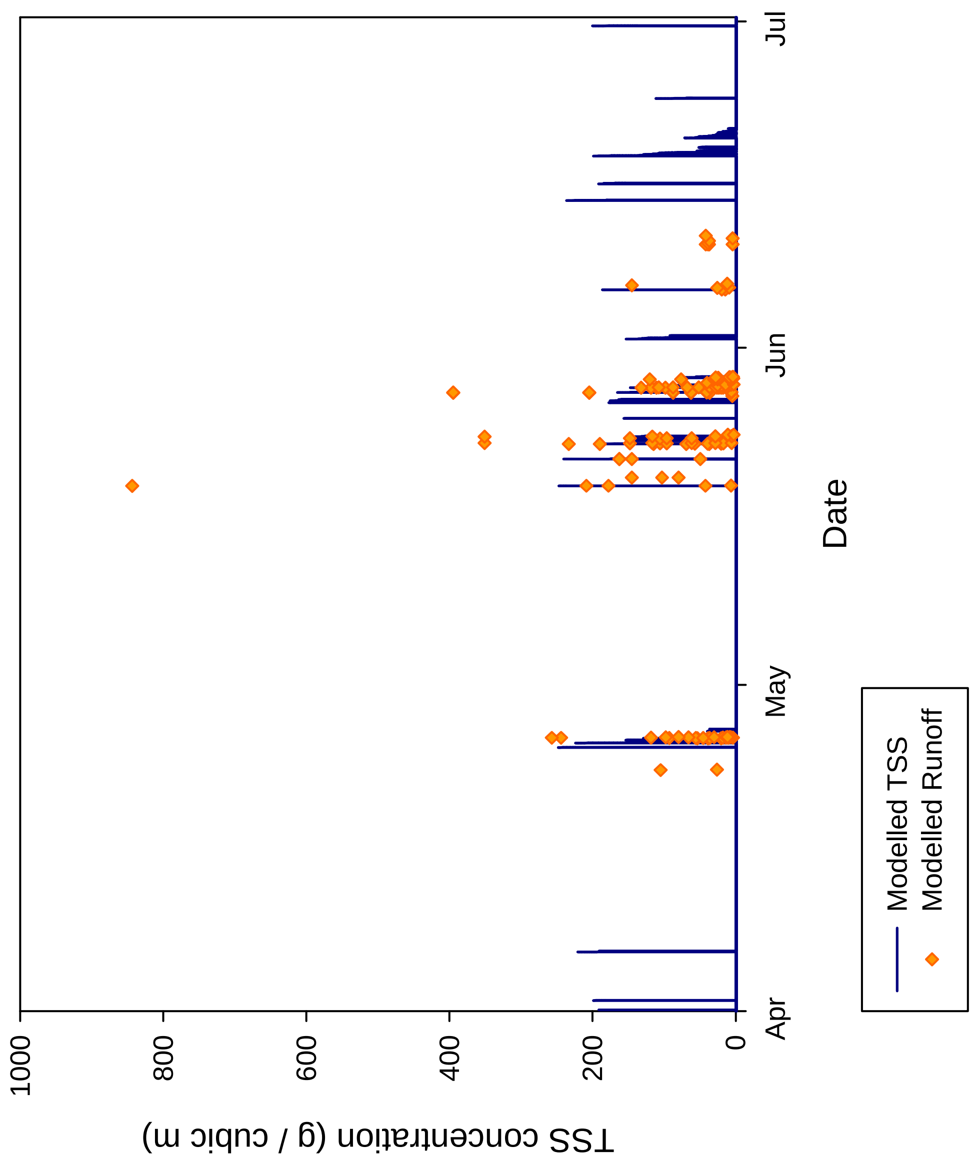


Figure 25 **Observed and simulated TSS concentration for Tamaki – April to June 2002**

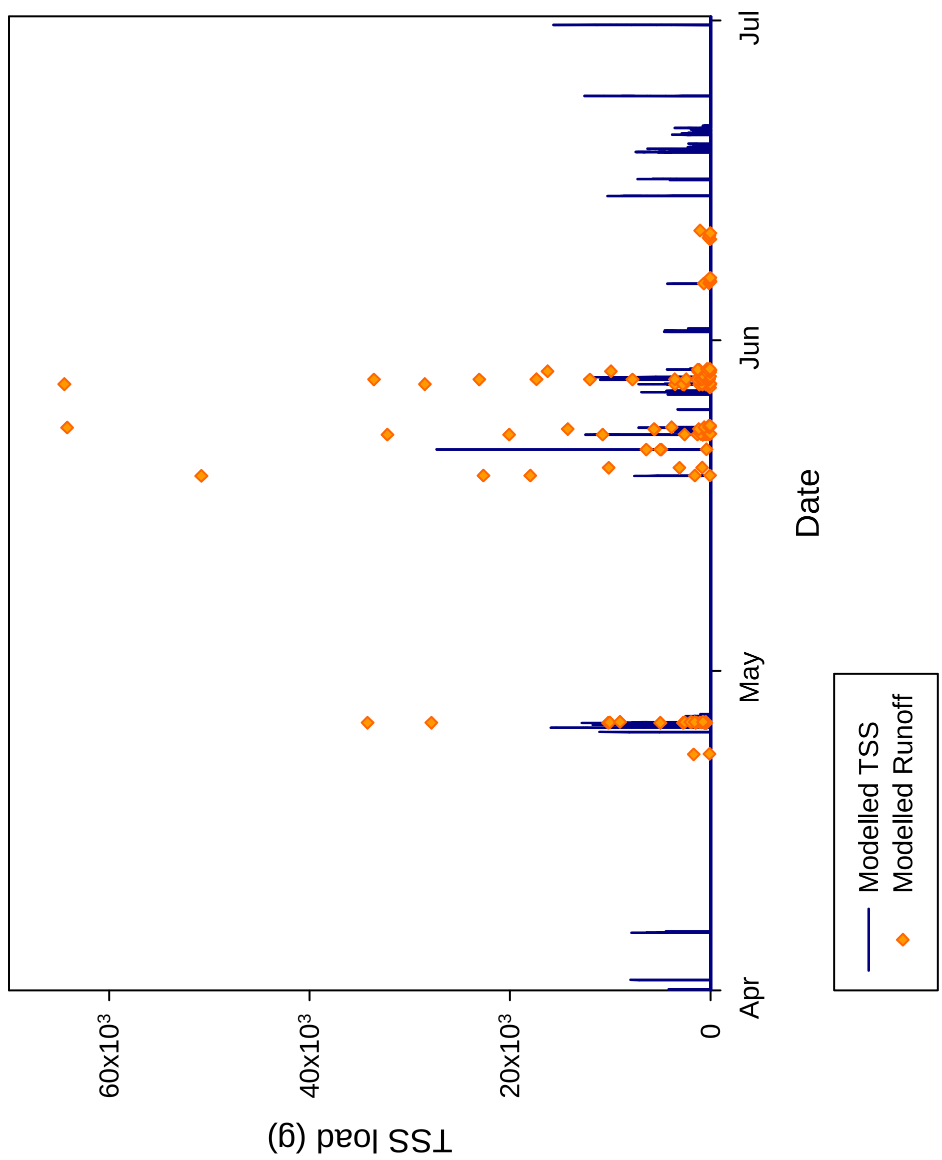


Figure 26 **Observed and simulated TSS load for Tamaki – April to June 2002**

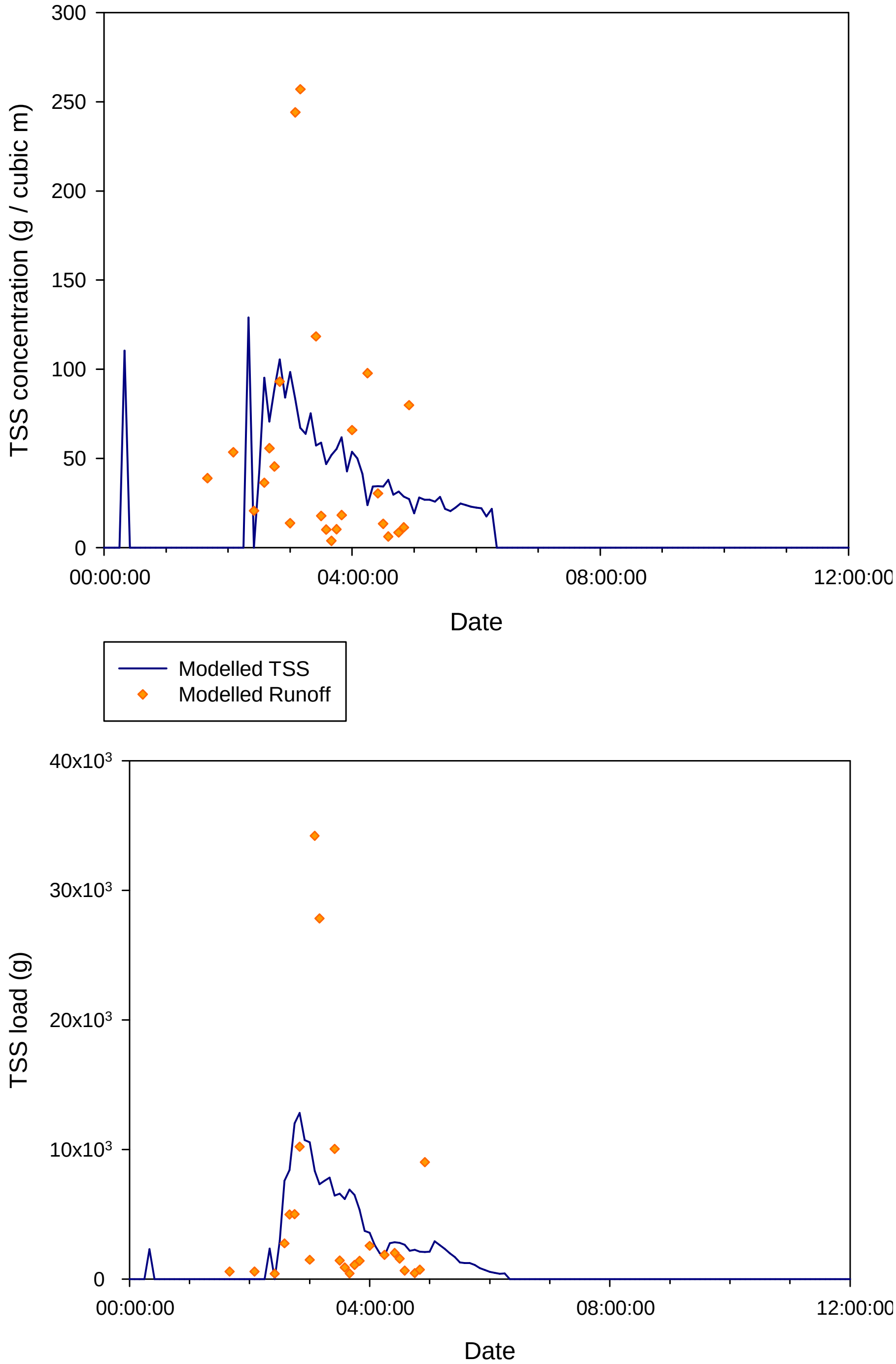


Figure 27 **Observed and simulated TSS concentration (top) and load (below) for Tamaki – 26 Apri1 2002**

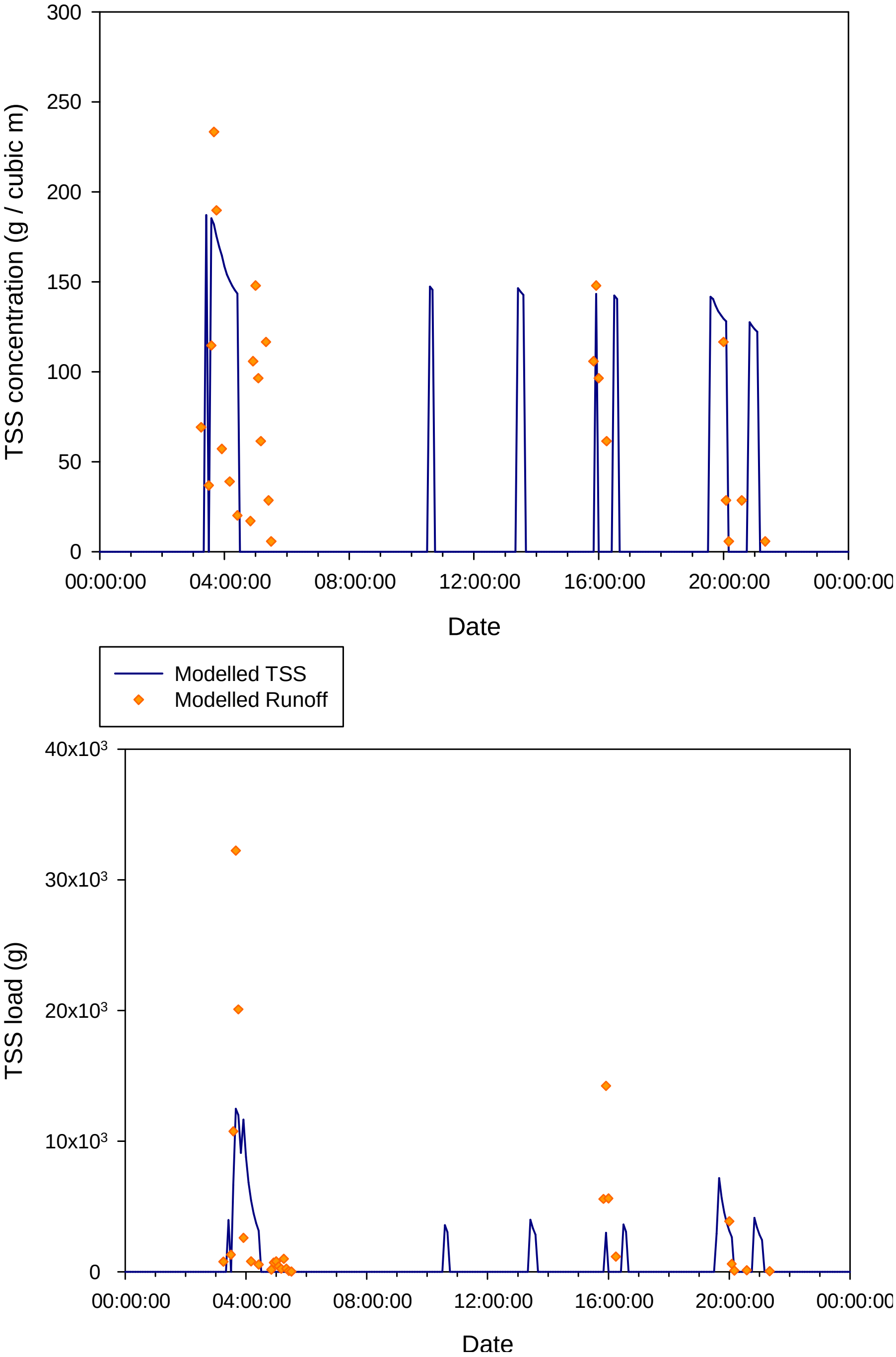


Figure 28 **Observed and simulated TSS concentration (top) and load (below) for Tamaki – 23 May 2002**

Table 8 Observed and simulated event mean concentrations for events where the entire flow hydrograph was sampled – Tamaki. Note that not all time steps during the sampled events have associated TSS concentrations.

Event	Observed EMC	Simulated EMC
26/04/2002	63	59
19/05/2002	298	242
23/05/2002	157	151
27/05/2002	227	152
28/05/2002	86	50
29/05/2002	53	40

The catchments showed variable fit with respect to the different sampled events simulated for concentration and load which was due to both error in estimates of discharge and the assumptions of the build-up wash-off method. The exponential wash-off routine used here assumes constant accumulation, reduction and wash-off rates. Conceptually, this means that the concentration and load during an event is a function of the flow rate and the length of time between events. Consequently, a first flush is always simulated. Wash-on and sediment from point sources are not simulated as to do so would require detailed knowledge of the catchment to be modelled as well as routines for soil and stream erosion including flow routing. The variable TSS concentration seen at Mission Bay in particular cannot adequately be explained by the simple model presented here. The high loads seen in the samples for some of the events are probably due to the presence of streams in the stormwater system. Similarly, anomalies seen in the CBD concentration and load could be due to wash-on from construction sites.

Splitting the catchments up into component land covers with calibrated accumulation and wash-off parameters and including water routing to the outlet would probably improve the fit but will add to model complexity. Given the project aim is to provide input data to treatment devices, the module gives concentrations and loads in the same order of magnitude as those seen in stormwater samples. It can therefore be considered suitable for use in the performance rules, with the caveat that it may not be accurate for catchments with a high degree of stream erosion, construction or some other point sediment sources.

3.3 Detention and settling in Ponds and Wetlands

Wet detention ponds and constructed wetlands consist of a permanent pool of water into which stormwater is directed. Water is detained until it is displaced by the next volume of stormwater. The primary purpose is to slow stormwater delivery to receiving waters for flood control and improve water quality. While it is detained in the pond natural, physical, chemical and biological processes treat the stormwater. In a well maintained pond with an adequate retention time, settling removes up to 50-90% of the TSS, and with it the bulk of particulate contaminants (Schueler, 1992, cited in US EPA, 1999 a); for instance, the removal efficiencies of total Pb and Zn can range between 70-80% and 40-50% respectively. There is little data on water treatment in New Zealand stormwater ponds and wetlands; Elliott (1996) did find that the reduction in contaminant concentrations at the Halswell Pond in Christchurch were in this range (e.g., 64% for TSS, 42% for Zn and 48% for Cu), however the reduction in load was much greater due to significant water losses to the pond (i.e., bottom infiltration and evaporation).

Wetlands have their own set of design criteria and removal processes. The conventional wisdom is that as well as allowing settling, constructed wetlands offer increased water treatment over detention ponds due to the presence of vegetation. According to the US EPA (1999 b), wetland plants:

- Increase flow pathways and therefore detention times;
- Filter litter, debris and other floatables carried in stormwater;
- Filter particulates as the water flows through root masses;
- Provide surfaces for microbial growth therefore increasing biological uptake; and
- Provide surfaces for bonding of dissolved contaminants.

While there have been studies which show increased rates of removal for nutrients (e.g., Bavor *et al.*, 2001), settling remains the primary treatment for particulate metals in wetlands (Somes *et al.*, 2000; Walker and Hurl, 2002). Settling in wetlands is usually modelled in the same way as in ponds within urban drainage models (e.g., MUSIC), albeit with increased hydraulic efficiency ratings as a proxy for more convoluted flow paths (e.g., Persson *et al.*, 1999; Persson, 2000). This is the approach that will be used for the C-CALM performance modules.

Anderson *et al.* (2002) give a list of critical issues for stormwater pond function, ranging from accessibility for maintenance, to pond dimensions and public perception. They note that as knowledge of stormwater quality has changed so has pond design. Long-term detention pond performance can be estimated based on geographical location and the ratio of pond surface area to the contributing source area. The basic rule-of–thumb for pond design is that the ratio of water surface to drainage area should be at least 1:100 (US EPA, 1999 a). In his PhD research, German (2003) carried out a literature search on pond function, and investigated the removal efficiencies of several ponds in Sweden. Part of his early work is reported in Pettersson *et al.* (1999). He found a relationship between specific area and removal efficiency, however, the increase in efficiency with pond size plateaus after 250 m^2 ha^{-1} impervious catchment surfaces.

The primary determinant of retention time is pond volume. Water should remain in the pond at least 24 hours for settling of large particles, but the longer the retention time the better, particularly if nutrients need to be removed through biological uptake. Indeed, the retention time of a pond intended to reduce downstream eutrophication may be three or more times that of pond solely intended for settling. Choice of pond depth is a trade-off between public safety, pond hydraulics and biological activity: while it is important to maintain a depth sufficient to avoid re-suspension of bed sediments, the pond should not be so deep that thermal stratification or anoxic conditions develop, as both have an effect on biological uptake. Butler and Davies (2000) suggest that ponds should ideally be around 1.5-3 m deep for effective treatment, though in practice, ponds are usually shallower.

Another aspect of pond design is the length-to-width ratio: the longer the flow pathway, the better the removal efficiency (Petterson, 1999; Persson, 2000). The US EPA (1999 a) state that a ratio of 2:1 or more will decrease the possibility of short-circuiting and increase retention time allowing for greater settling. Baffles and islands can also be used to extend the flow pathway (Persson, 2000), though poor placement of these can introduce dead-areas which reduce the active pond volume and retention time (e.g., Semadeni-Davies, 2007). Other studies of the removal efficiency of detention ponds and constructed wetlands, particularly for metals, include Walker and Hurl (2002), Somes *et al.* (2000), Lee *et al.* (1997) and Barbosa and Hvitved-Jacobsen (1999).

3.3.1 Module components

Wet detention ponds and wetlands are simulated with the same module. Flow through the facility is calculated using the continuity equation:

$$\frac{dV_{pond}}{dt} = Q_{in} - Q_{out} + A_{pond}\left(P - ET\right)$$ Equation 9

where V_{pond} is the volume of water stored in the pond (m^3), Q_{in} and Q_{out} (m^3 s^{-1}) are the pond inflow and outflow respectively, A_{pond} is the pond surface area (m^2), P is the precipitation depth (m s^{-1}) and ET is evapotranspiration (m s^{-1}). Note that the pond surface area changes with time as the volume of water in the pond increases and decreases. Inflow, precipitation depth and evapotranspiration (open water rate) are model inputs. Outflow is calculated as a function of both the pond depth and the type and configuration of the outlet.

The pond is conceptually modelled as a trapezoidal basin (see Figure 29), where the hydraulic head, h (m), is a geometric function of the storage volume:

$$h = \frac{s}{2L}\sqrt{\left(wL\right)^2 + \frac{4LV_{pond}}{s}} - \frac{ws}{2} - h_0$$ Equation 10

where w is the pond base width (m), L is the pond length (m), s is the slope of the pond banks and h_0 is invert level of the outlet (m). The base width is calculated from the bank slope, the width at the outlet invert level and the invert height as:

$$w = W - \frac{2h_0}{s}.$$

The initial detention volume is assumed to be at pond fill (i.e., h = 0).

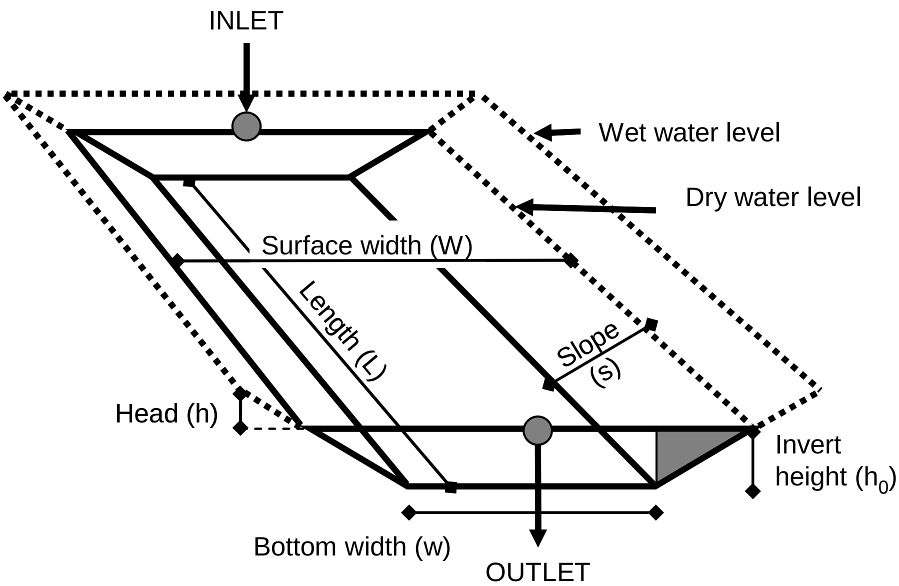


Figure 29 **Trapezoid conceptual layout of a detention pond**

Outflow is dependant on the type of outlet device. For simplicity, a sharp-crested weir outlet is assumed, in which case the outflow is calculated as:

$$Q_{out} = C_d \frac{2}{3} l_{weir} \sqrt{2g} h^{1.5} \qquad \text{Equation 11}$$

Where C_d is the weir coefficient (0.6, e.g., Butler and Davies, 2000), l_{weir} is the width of the weir and g is the gravitational acceleration. In the case of the two ponds modelled below, the outflow configurations are more complex though.

Krishnappen and Marsalek (2002) state that the methods currently used to evaluate settling in detention ponds are typically based on two approaches: (a) the ideal settling tank concept, and (b) two or three dimensional computational fluid dynamics (CFD) models. While CFD models provide a better representation of flow distribution in ponds, they are complex and require detailed knowledge of pond bathymetry. Operational urban drainage models which simulate settling generally use the former approach, which commonly known as the continuously stirred tank reactor model (CSTR). This is the method chosen here.

Driscoll *et al.* (1986) presented a basic methodology for the design and analysis of wet detention ponds based on CSTR modelling. The methodology is widely used for both pond design and operation and is discussed in detail by Persson and Wittgren (2003). It assumes that there are two types of settling, quiescent and dynamic. A pond operates under dynamic conditions when the storage of the pond is increasing with runoff entering the pond and with the stage rising, and when the storage is decreasing when the pond stage is lowering. Quiescent settling occurs during the dry period between storms when previous flows are trapped in the pond. The relative importance of the two settling periods depends on the size of the pond, the volume of each runoff event, and the inter-event time between the rains.

Settling during dynamic conditions is based on Hazen theory (Fair and Geyer, 1954) where ponds are approximated as a series of successive tanks with flow from one tank to the next. The more tanks, the less mixing or short circuiting between sections of the pond. The proportion of sediment removed from the water column (R_s) is calculated as:

$$R_s = 1 - \left[1 + \frac{1}{n} \frac{A_{wet} v_s}{Q_{in}}\right]^{-n} \qquad \text{Equation 12}$$

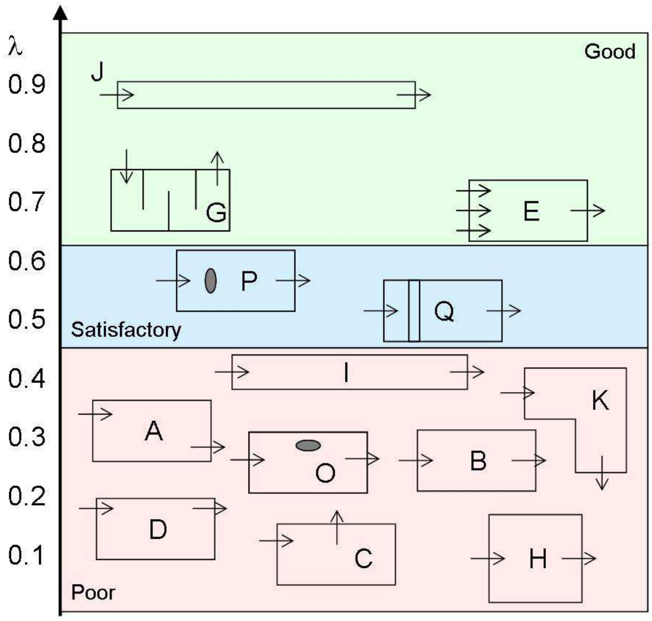
where n is the conceptual number of tanks, Q_{in} is inflow rate ($m^3\ s^{-1}$), A_{wet} is the water surface area (m^2) and v_s is the particle settling velocity ($m\ s^{-1}$). The ratio v_s/Q_{in} gives the nominal detention time in the pond.

The water surface area is given by

$$A_{wet} = L\left(w + \frac{2(h + h_o)}{s}\right). \qquad \text{Equation 13}$$

The fall velocity for each of the particle-size bands are taken from Table 2 depending on the PSD chosen. The NURP fall velocity bands were used as a default for module testing.

In an exceptional pond with perfect plug the number of tanks n tends to infinity. In a poorly designed pond (n = 1), there is only one tank with continuous mixing horizontally and vertically, turbulence and short circuiting. Generally, n ranges from 1 to 8 in CSTR models of stormwater ponds. Hydraulic modelling (2-D MIKE-21) of hypothetical pond configurations was used to find an approximate relationship between pond configuration, hydraulic efficiency (λ, the ratio of the time to peak concentration at the outlet and the nominal flow detention time) and n (Persson *et al.*, 1999; Persson, 2000). The simulated values for λ and n are given in Figure 30 below and show how the relative locations of the inlet and outlet, the width to length ratio, and the presence of berms, baffles and islands can change hydraulic efficiency. Short circuiting, for instance, not only reduces detention time, it results in pond dead areas which reduce the effective storage at the facility.



Configuration	Number of tanks, n
A	1.4
B	1.4
C	1.1
D	1.2
E	4.2
G	4.2
H	1.1
I	1.7
J	10
K	1.6
O	1.4
P	2.6
Q	2.4

Figure 30 **Relationship between pond configuration, hydraulic efficiency and the number of CSTR tanks. Configurations O and P include an island, G has 3 baffles and Q a berm. (after Persson *et al.*, 1999; Persson, 2000; Persson and Wittgren, 2003)**

Removal during quiescent flow is calculated as a function of the settling velocity, time since the last flow event and pond depth as:

$$R_s = 1 - e^{-v_s t/(h+h_o)} \quad . \qquad \text{Equation 14}$$

This is the quiescent settling formulation recommended by Driscoll *et al.* (1986).

A simple hydrograph separation is used to determine periods of dynamic and quiescent settling. Incoming baseflow is determined by

$$Baseflow_{(t)} = \frac{k_b Baseflow_{(t-1)} + (1-k_b)Q_{in}}{2-k_b} \qquad \text{Equation 15}$$

where k_b is a dimensionless recession constant. If the proportion of baseflow is greater than stormflow, then settling is said to be quiescent.

For each time step, the module first calculates the stored sediment mass in the pond water. The change in sediment storage is calculated as the sum of the stored mass and the incoming sediment load, less the settled mass and outgoing sediment load from the previous time-step. Thus, a running tab is kept of both the accumulated mass and concentration of sediment (calculated as the mass of sediment divided by the stored pond volume) in the pond for each size band. The removal efficiency is calculated for each sediment size band using the CSTR formulae above. Outflow water is assigned the stored concentration and the load of sediment leaving the pond calculated as the product of this concentration and the outflow volume for the time-step.

3.3.2 Module testing

Flow and water quality data suitable for module development were available from two local ponds: a highway detention pond treating road runoff near Silverdale (NIWA), and a sub-urban pond in a Te Atatu Peninsula housing development (Landcare Research). The characteristics of the ponds are summarised in Table 9. Inflow and outflow are available for each pond with 5 minute time-steps although the way in which water samples were taken varied. The TSS concentration of time-steps without a water sample during sampled events were linearly interpolated. Model inputs are measured inflow and TSS concentration.

The main outlet structure in both ponds is a standpipe, these can be modelled as weirs with a length equivalent to the circumference of the pipe. However, the ponds also have extended detention with flow regulated by a secondary outlet that allows the pond to drain slowly between events maximising storage (Figure 31). The Silverdale pond standpipe has a slot weir and the Te Atatu pond standpipe a circular orifice. The outflow equation presented in Section 3.3.1 was therefore adjusted to include these secondary outlets.

The Silverdale pond test thus includes two separate weir equations with different weir invert levels and widths, whereas the Te Atatu pond simulates three flow situations (orifice uncovered, orifice covered and standpipe topped). The Te Atatu pond standpipe has a debris screen and the combined width of the bars was subtracted from the standpipe circumference. However, there is a possibility that the bars could cause a flow restriction that was not simulated. The aim of the module is to provide a generic pond model for the development of the performance rules. Hence, the ponds were simulated twice (i.e. with and without low-flow regulation via the secondary outlets) to see how sensitive the results are to the simulation of the outlet configuration.

Pond Hydrology

Table 10 summarises the performance of the pond flow model for both ponds.

Flow was well simulated at the Silverdale pond (Figures 32 to 34). The event on 12 March, which was not recorded at the outlet due to a problem with the stage recorder, was excluded from the flow model evaluation. The total outflow volume was underestimated by 7 %. The module was able to simulate both low flows via the slot and high flows over the standpipe including an extreme event which occurred on 28 March 2007. This event caused widespread flooding across Northland, and, at Silverdale, the standpipe was overtopped.

Table 9 **Summary of characteristics for the test ponds simulated by the pond treatment module**

	Silverdale	Te Atatu
Simulation period	12 Dec 2006 – 29 April 2007	1 Dec 2003 – 31 Aug 2004
Calibration period	Dec 2006 - Jan 2007	Dec 2003 – Feb 2004
Area at outflow level	264 m^2	960 m^2
Volume	218 m^3	277 m^3
Outflow configuration	Standpipe (1.2 m diameter, invert 1.2 m) Slot weir on standpipe (8 cm wide, invert 90 cm from pond base).	Standpipe (1.8 m diameter, invert 1.05 m) with debris screen. Orifice on standpipe (15 cm diameter, invert 0.35 cm from pond base)
Estimated n (from Figure 30)	2	1
Sampled events at inlet	3 (26 samples in total)	37 (134 samples in total)
Sampled events at outlet	3 (20 samples in total)	39 (340 samples in total)
Number of events modelled	3	12
Comments on sampling	Time based sampling triggered by stage. Sampling interval 5 minutes, however, the interval between analysed samples ranged from 15 minutes to 2 hours depending on sampling location on hydrograph. The flow for one event was not recorded due to an problem with the stage recorder.	Time based sampling, with samples taken hourly on the hour. Some inflow events have only one inflow or outflow sample and others few samples with sampling intervals of several hours. Some events were not sampled concurrently at the inlet and outlet.

Figure 31 **The outlet standpipes for the Silverdale (above, high and low flow conditions) and Te Atatu (bottom, low flow) ponds showing the secondary outlets for extended detention.**

Table 10 Summary of model fit for discharge.

Catchment	Correlation (R^2)			Total flow volume (m^3)				Peak Discharge ($m^3 s^{-1}$)			
	Calibration	Validation	Entire	Calibration		Validation		Calibration		Validation	
				OBS	SIM	OBS	SIM	OBS	SIM	OBS	SIM
Silverdale (excludes 12 March 2007)	0.89	0.95	0.93	604	645	1415	1240	0.014	0.008	0.060	0.053
Te Atatu	0.81	76	0.78	35421	32655	56301	61403	0.744	1.222	0.381	0.309

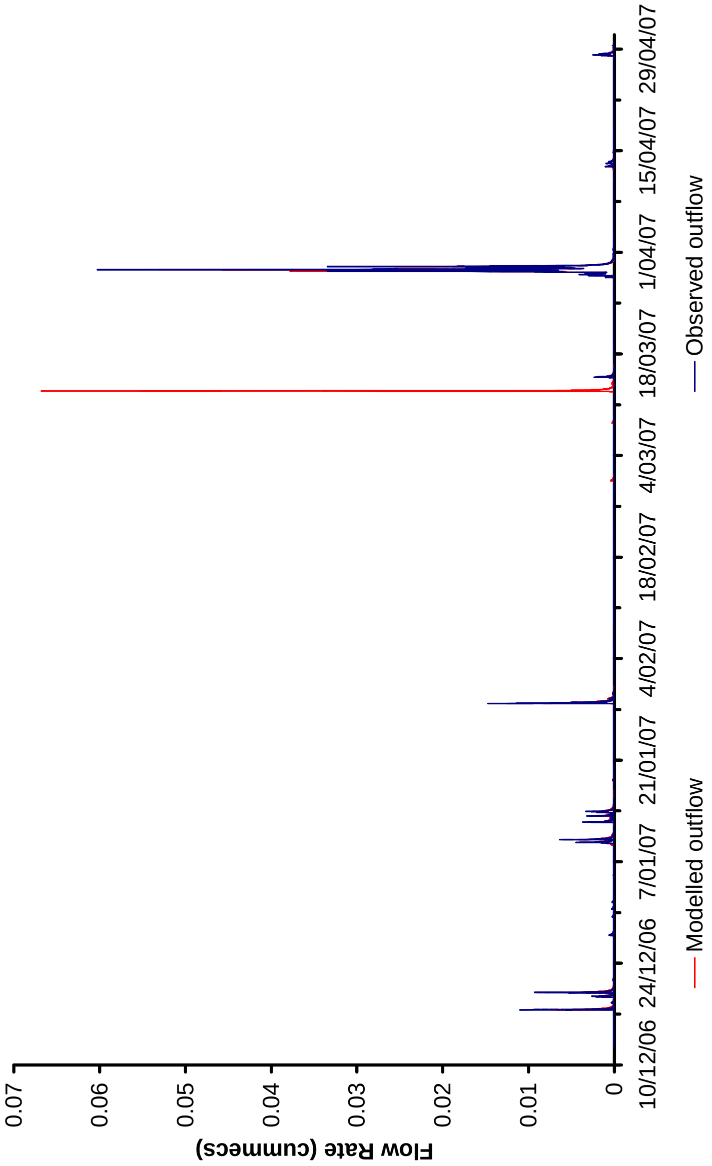


Figure 32 **Simulated and observed outflow from the Silverdale Pond -. 28 to 30 March 2007 (note, stage was not recorded for the event on 12 March)**

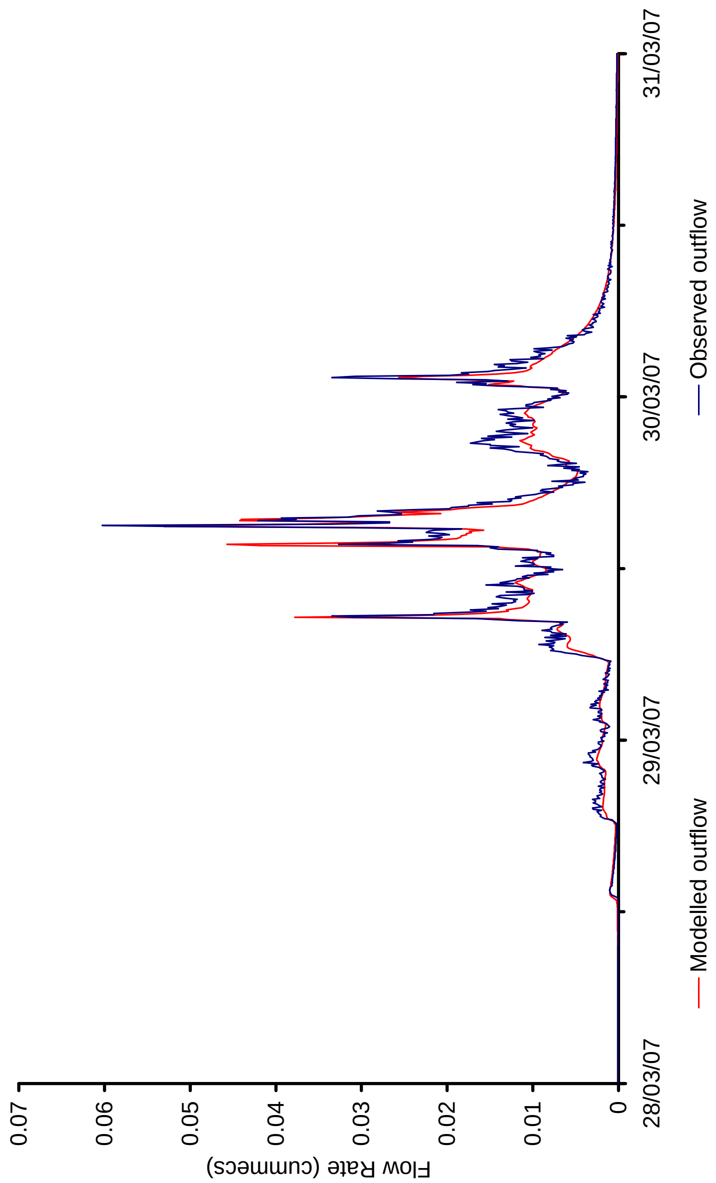


Figure 33 **Simulated and observed outflow from the Silverdale Pond for high flow conditions -. 28 to 30 March 2007**

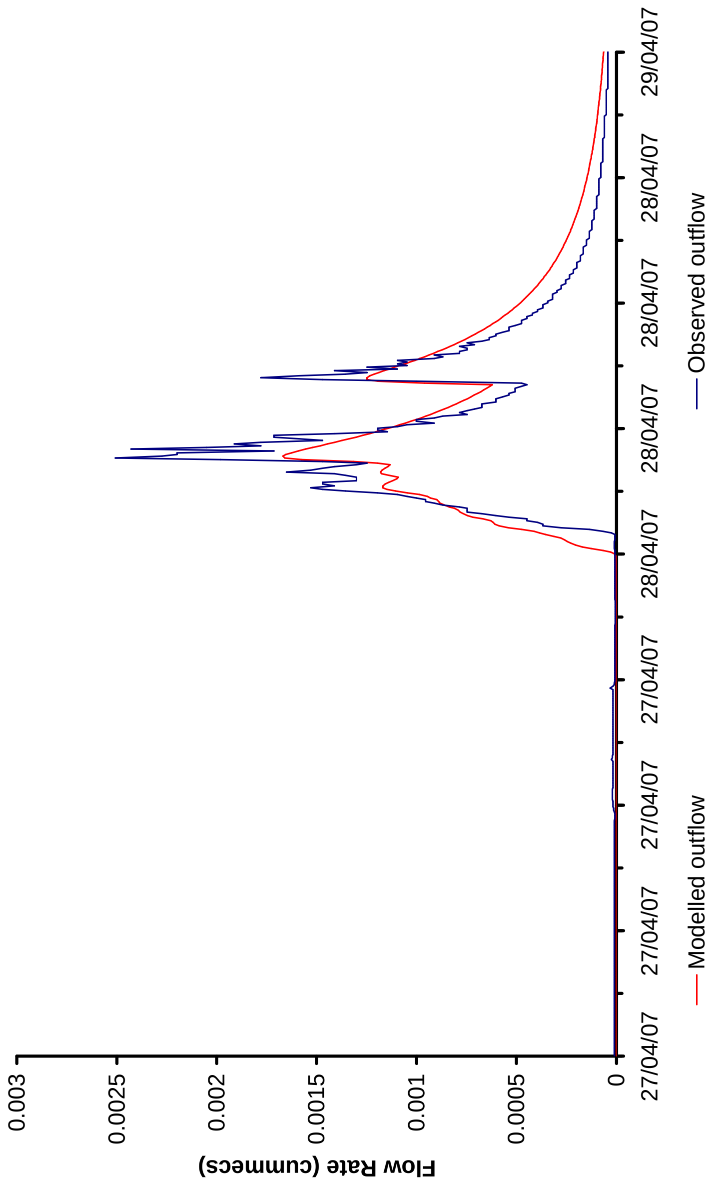


Figure 34 **Simulated and observed outflow from the Silverdale Pond for low flow conditions – 28 April 2007**

Flow at the Te Atatu pond was reasonably well simulated (R^2=0.79). The total flow volume was overestimated by 3%. Both low flows the orifice and high flows over the standpipe were simulated, however, the transition from one flow situation to the next was too abrupt. On several field-visits reported by Trowsdale and Fletcher (2005), the orifice was blocked with debris such as floating sticks and weeds causing the pond level to rise to the standpipe outflow. This would cause a reduction in the observed outflow during between events and higher peak flows during events. There is a tendency for the model to overestimate low flow peaks, while some high flows are underestimated. The latter may be an artefact of the flow record as discussed above. Examples of model fit for flow are given in Figures 35 to 37.

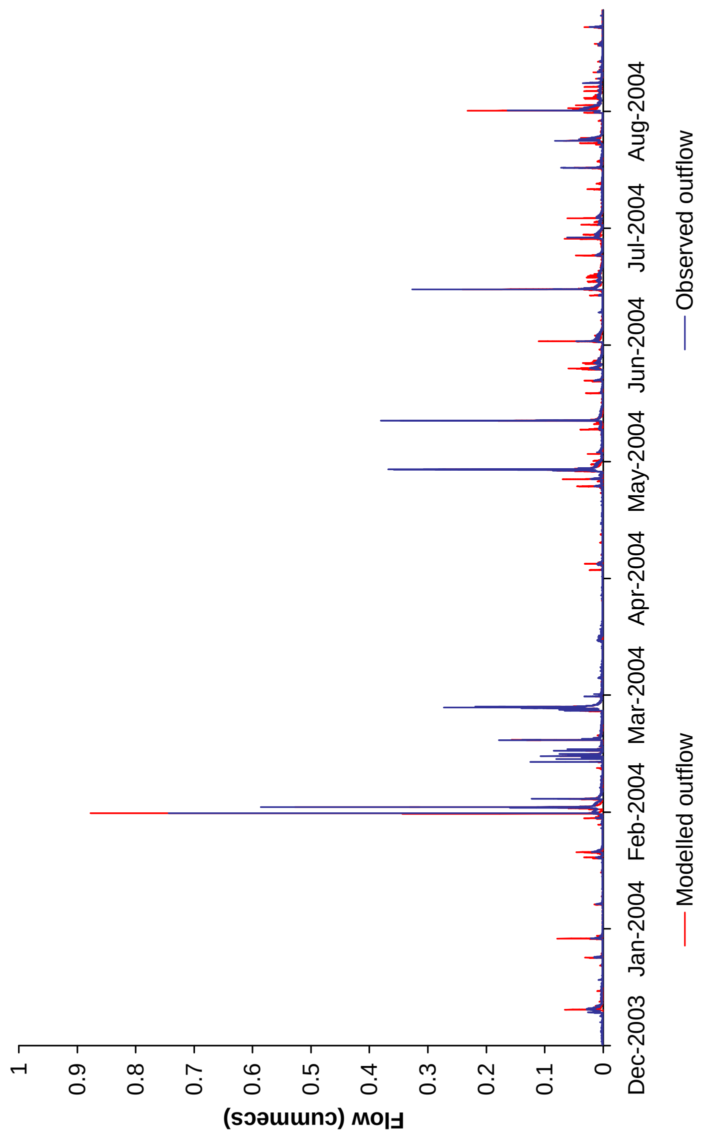


Figure 35 **Simulated and observed outflow from the Te Atatu Pond -. December 2003 to August 2004**

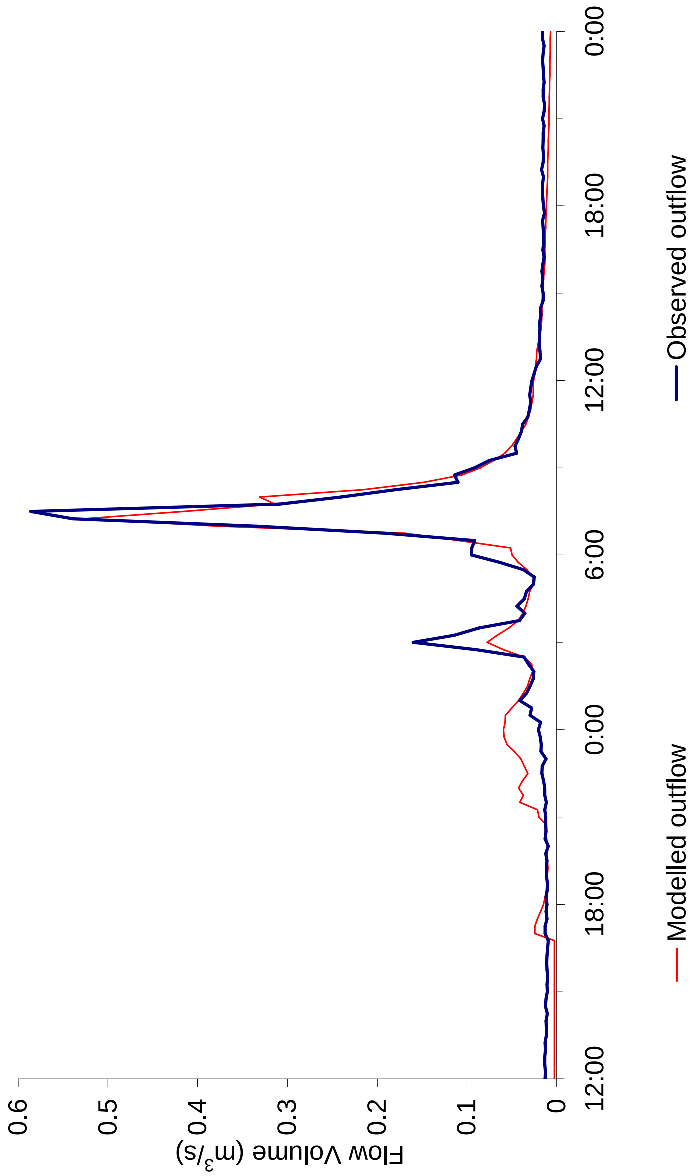


Figure 36 **Simulated and observed outflow from the Te Atatu Pond during high flow conditions -. 1 to 2 February 2004**

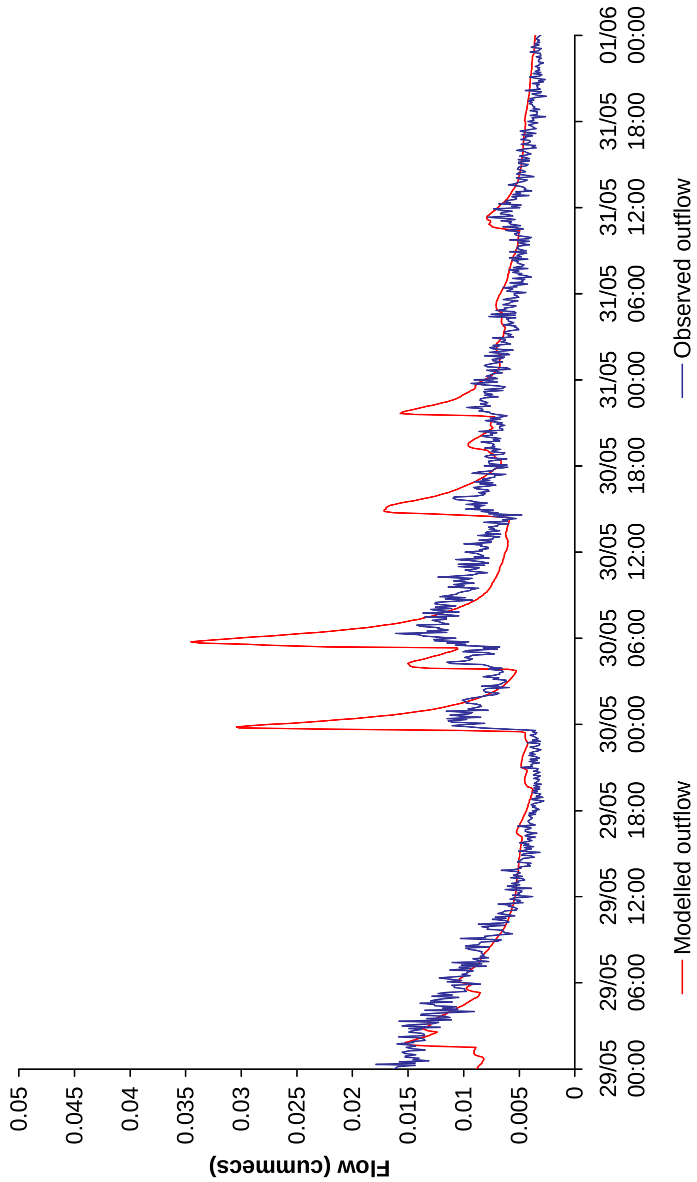


Figure 37 **Simulated and observed outflow from the Te Atatu Pond during low flow conditions -. 29 to 31 May 2004**

Settling

Three events were sampled at the Silverdale pond. A total of 137 kg of sediment was calculated for the inlet based on the inflow samples. The first event was a short, high flow event (12 March). Unfortunately, stage was not recorded during this event, hence load was calculated for the observed outflow concentrations using the modelled outflow. The second event was due to an extreme rainfall that lasted 48 hours (28-30 March 2007). The sampler carousels were changed during this event, the incoming concentrations for the intervening period were linearly interpolated. The third event (28 April) was at the other end of the spectrum and was in response to a minor rainfall. Sediment removal was poorly simulated for the Silverdale pond (Figure 38) using the NURP fall velocity distribution which represents medium sized grains in Table 2. The total removal efficiency calculated was 73% compared to 56% observed, though the first event had very similar removal. The total removal (55 %) was very close to that observed when the fall velocity distribution was changed to that of a fine PSD, to take into account the clay soils of North Shore. With the slower fall velocities, the fit for the events on 28-30 March were well simulated, however the removal for the first event was overestimated. The third event, which was very minor with low incoming TSS concentration, had poor fit for both simulations. The improvement in model fit with the adjustment to the grain size classes shows just how important it is to allow C-CALM users to choose between PSD options. Table 11 gives a summary for the three sampled events with calculations using both the NURP and fine grain PSDs.

Table 11 **Summary of model performance for the Silverdale pond. Simulated load was only calculated for those times when observations were available.**

		NURP (medium grain PSD)	Fine PSD
12 March 2007	Load in (kg)	57.7	57.7
	Observed load out (kg)	12.01	12.01
	Simulated load out (kg)	13.7	20.3
28 30 March 2007	Load in (kg)	74.8	74.8
	Observed load out (kg)	47.2	47.2
	Simulated load out (kg)	23.7	40.5
28 April 2007	Load in (kg)	4.6	4.6
	Observed load out (kg)	1.4	1.4
	Simulated load out (kg)	0.1	0.2

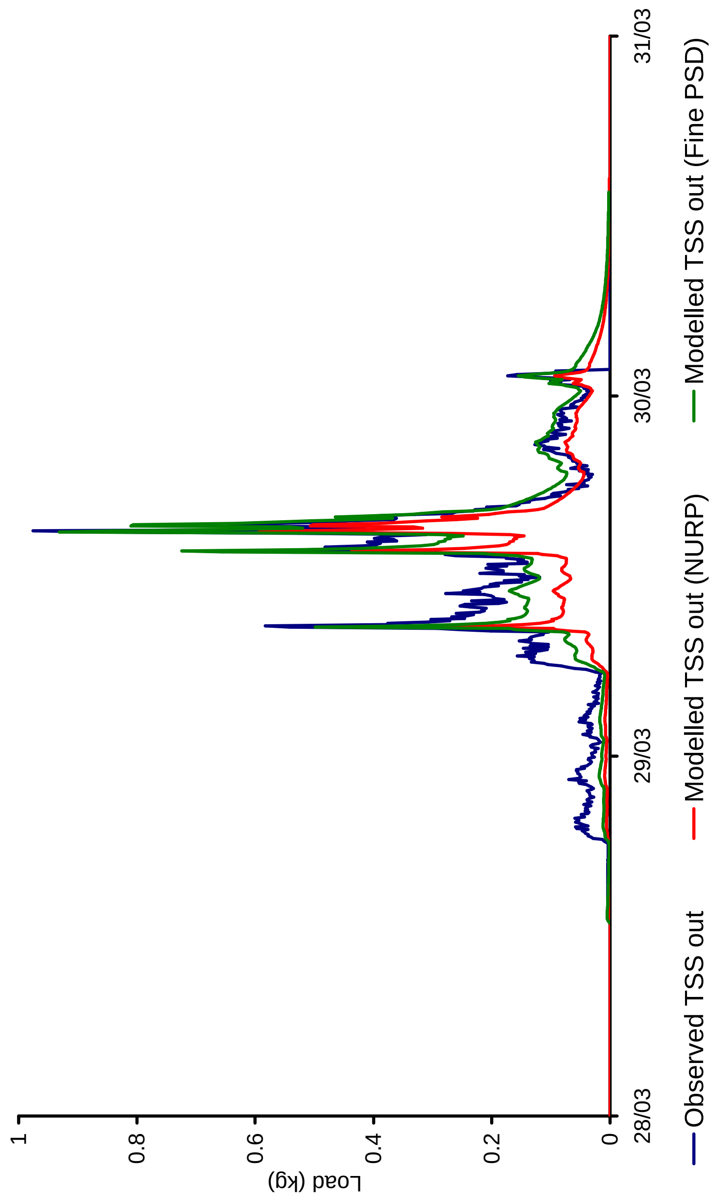
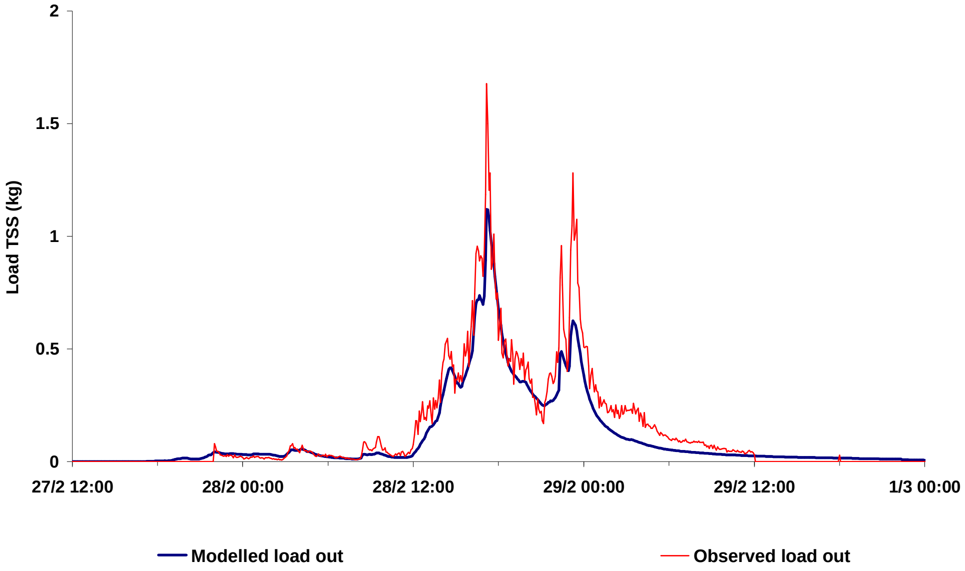


Figure 38 **Sediment load (5 minute intervals) at the outflow simulated with the NURP fall velocity distributions for the Silverdale pond**

It is difficult to assess the performance at the Te Atatu pond given the large sampling intervals during events and the small number of inflow samples taken with respect to outflow. Unfortunately, not all of the data available could be used, as some events sampled were not concurrent at the inlet and outlet and others had only one inflow sample. Of some 40 events sampled, 11 events (collated into five groups of consecutive events) were found to be suitable for modelling, even so, several of these had very few inflow samples which could lead to an underestimation of incoming sediment loads if the entire inflow event is not captured. The grouped events are summarised in Table 12. There is a possibility that inflow phenomena such as first flush were not captured by the sampling programme which would cause the removal efficiency to be underestimated. The event of 28 May 2004, for instance, has just two inflow samples taken three hours apart whereas 14 outflow samples were taken at two hourly intervals. If the peak incoming TSS concentration occurred between the inflow sampling intervals, the load entering the pond would be greater than that estimated using linear interpolation. For a couple of events, the outflow TSS load was greater than inflow, this could be an artefact of the sampling programme or could be due to untreated water from an earlier event. The latter explanation is not likely as in each case there were several days available for settling between events. For the 11 events simulated, there is good agreement between the modelled (31 %) and observed removal efficiency (26 %). Figures 39 and 40 show that the model is able to track the observed sediment load at the outlet.



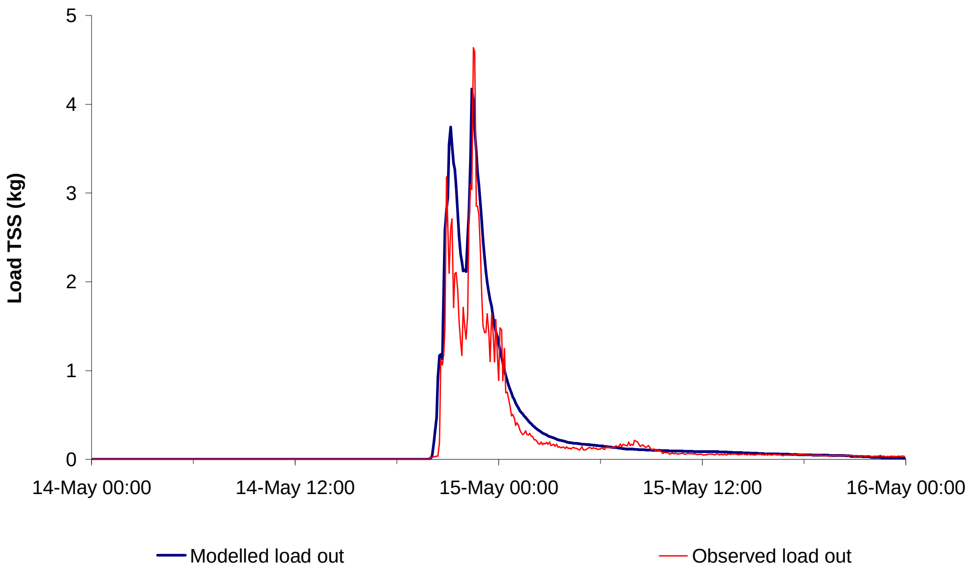


Figure 39 **Sediment load (5 minute intervals) at the outflow simulated with the NURP fall velocity distributions for the Te Atatu pond – 27 to 29 February (top) and 14 to 15 May (below)**

Table 12 Summary of model performance for the Te Atatu pond by flow period

	Load in (kg)	Observed load out (kg)	Simulated load out (kg)
4 February	29.3	44.7	18.0
27 – 29 February	86.0	101.4	75.9
14 May	187.7	118.76	152.1
6 August	37.1	6.2	6.4
8 – 9 August	49.9	19.0	16.4

Simplifying the outflow structure

There is a wide variety of not only outflow types (e.g., weirs, stand-pipes, and orifices – both above and below the water level) and dimensions but also combinations of outlets. For instance, the Silverdale and Te Atatu ponds had a standpipe with a secondary outlet for extended detention. Moreover, some outlets can have gross pollutant traps which impede flow. It is not possible for a model like C-CALM to cover all the options available. It is therefore proposed that outflow be modelled from a single outlet structure (nominally a sharp-crested weir). For this reason, the pond module has been run with a simplified outlet to test impact of this simplification.

Removing the slot weir from the model set-up for the Silverdale pond has a negligible impact on the simulated total outflow and correlation. However, flows became more peaky, with both high and low peak flows being overestimated. Removal efficiency simulated with the fine sediment fall velocities increased to 57% (from 55 %), which is compatible with the recorded value of 56%.

The impact of removing the orifice flow simulation for the Te Atatu pond was minimal, though some of the high flow events did have increased peaks – presumably due to decreased storage capacity. The effect on modelled removal efficiency was also negligible.

As the effect on both flow volumes and removal efficiency was fairly minor, the simple representation given in Equation 9 should suffice for the performance rules.

3.4 Raingardens (and bio-retention units)

Raingardens and bioretention units are increasingly being seen in New Zealand's urban landscape. The distinction between raingardens and bioretention units is subjective, with the former referring to larger devices constructed in situ, and the latter usually used to describe smaller housed units isolated from surrounding soil. The removal processes involved in each are essentially the same, and henceforth in this module the term raingarden will be used to cover both. A raingarden is very similar in design to a media filter; the difference is that raingardens may allow biological uptake of dissolved contaminants (especially nutrients) and evapotranspiration of stormwater. Also, unlike media filters, raingardens may not be lined allowing some interaction with local ground water, which means that there is a potential for deep percolation of contaminants that reach the base of the raingarden.

Davis *et al.* (2003) investigated bioretention of dissolved Pb, Cu and Zn, both in the laboratory using two specially constructed cells, and in the field using two existing raingardens (one around a year old and one around ten years old). The two cells in the lab were of differing dimensions (107 cm long × 76 cm wide with media depth 61 cm, and 305 cm × 152 cm with media depth 91 cm), and each was a box filled with sandy loam and planted with creeping juniper. The two field sites were different in both their planting and filter media. In each case, artificial stormwater was applied to the surface, and effluent collected. They found that both the lab and field units were able to retain nearly all of the dissolved metals, and that the field raingardens have an expected lifetime of at least 15 years. While influent pH, flow duration and density and water quality were varied, these factors had little impact on removal efficiency. Depth of the media bed, on the other hand, did influence removal and the best removal rates were for deeper beds. Since the removal processes were not discussed however, it is not possible to deduce the relative contribution of bio-uptake to removal.

In another lab-scale test of bioretention units with a mixed medium of sand, mulch and soil, Sun and Davis (2006) found that uptake of metals by plants is relatively low compared to retention in the media. Retention in the medium is in turn related to both the physical and chemical properties of that medium. For example, in lab scale experiments carried out in New Zealand with local material, Zanders *et al.* (2003, cited in Taylor, 2005) and Taylor and Simcock (2006) tested a number of substrate mixes including sand, pumice, mulches, scoria and soil; they found that natural sandy soils such as a pumice and topsoil blend performed well. The ARC TP 10 (2003) guidelines suggest a sandy loam with a surface mulch of woodchip or bark; this seems to be fairly typical for New Zealand raingardens, though there are exceptions such as the use of topsoil at the Albany raingarden installed by the North Shore City Council.

As stated above, raingardens are similar in design to media filters. While there are several hydraulic models in the literature for media filters (e.g., Błażejewski and Murat-Błażejewska, 2003), a continuous flow model specifically for raingardens could not be found. Likewise, with respect to water treatment, no models could be found which distinguish between physical removal of contaminants (i.e. settling and sieving) and chemical removal (e.g., sorption, surface precipitation). Nor could models be found for particulates and dissolved contaminants. There are physically-based models for metal removal by sorption, but these are unsuited to the conceptual approach used here (e.g., the Langmuir absorption equation). Conceptually, operational stormwater treatment models such as MUSIC (CRCCH, 2005) relate the degree of water treatment to the physical properties of the filter media (i.e., area, depth and median grain size), and the detention time. The same method will be followed here. Raingardens are often preceded by a small pre-settling basin for temporary water storage and removal of coarse sediments and floatables. While the model does allow water to build-up on the raingarden when the filter media is saturated, settling during surface detention is not modelled explicitly – though water storage will increase detention time and therefore contaminant removal.

3.4.1 Module components

Evapotranspiration is a key component of raingarden function, thus one cannot assume that the filter media is always wet. Partial wetting and drying of pore spaces leads to changes in the hydraulics, so simple physically-based methods used for media filters (such as Darcy's law for a single wetting front) cannot be applied. While there are advanced physically-based models for solute transport in unsaturated soils (e.g., Persson *et al.*, 2001), these are outside the scope of this project as they rely on detailed knowledge of the properties of the soil matrix (texture, grain size, porosity, presence of macropores etc.).

Raingarden through-flow is conceptually modelled as a runoff plot with a defined area and depth; using a water balance or continuity approach, discharge is the residual of inflow less deep percolation and evapotranspiration. The governing equation is:

$$\frac{d\mathrm{SMS}}{dt} = P + \Delta Pond + F_i - \left(ET + R + F_o + B_{pass}\right) \qquad \text{Equation 16}$$

here SMS is soil moisture storage$P$ is precipitation, $\Delta Pond$ is thechange in water depth ponded the surface, F_i is inflow, F_o is outflow, ET is evapotranspiration, R is deep percolation to groundwater (which may include some artificial drainage) and B_{pass} is bypass water. The degree of loss to percolation will be a user defined parameter. All terms are expressed in depth (m).

As shown in Figure 40, through-flow is modelled as a linear reservoir with two soil layers relating to the upper root and lower soil layers respectively. The raingarden is assumed to be isolated from neighbouring soil with no horizontal flow (i.e., baseflow) and the medium is assumed to be homogenous with no preferential flow pathways. Capillary rise is assumed to be negligible and is not simulated.

The soil layers have the same flow parameters (they are assumed to have the same physical properties) but different depths, and the vegetation is assumed to have a negligible effect on the flow parameters. The soil moisture storage (SMS) for both layers is the product of the available water capacity (AWC) and the soil depth.

Inter-flow from the upper layer to the lower layer (m s^{-1}) is calculated as a function of the SMS of the upper layer:

$$F_{\mathrm{int}\,er} = k_u \mathrm{SMS}_u \qquad \text{Equation 17}$$

where k_u is a recession coefficient (s^{-1}) and SMS$_u$ is the soil moisture storage of the upper layer.

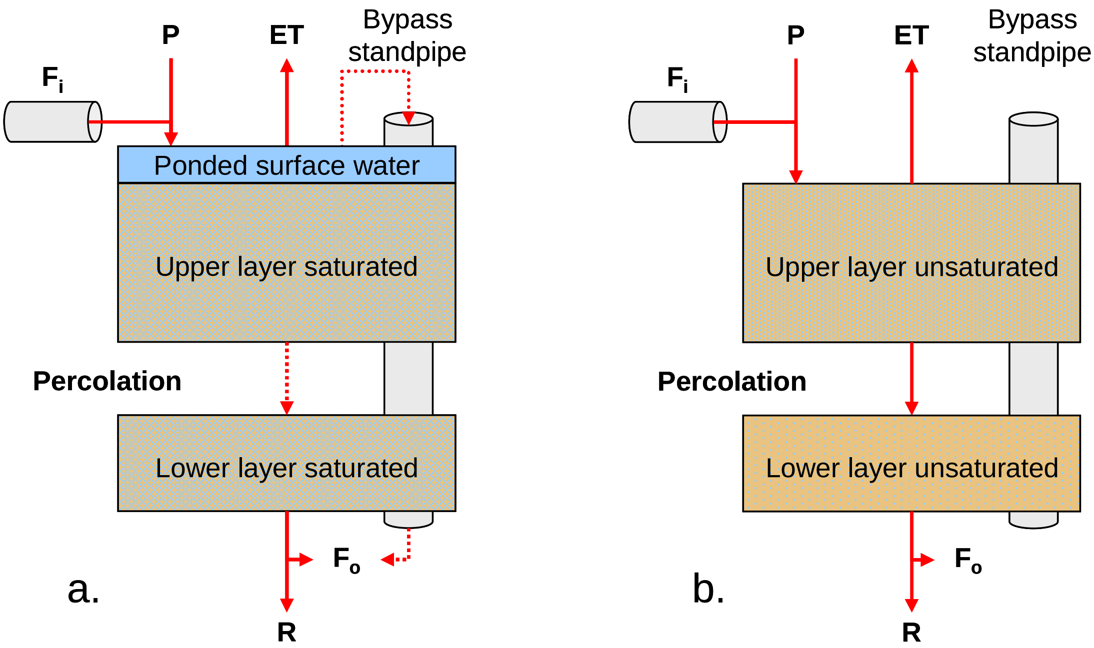


Figure 40 **The raingarden water balance scheme showing the situation where a. the layers are at full storage with surface ponding and b. the layers are unsaturated. Terms are the same as those given for Equation 16**

Evapotranspiration is only calculated for the upper root layer and is a linear function of the potential rate and soil moisture storage of the upper layer:

$$ET = \mathrm{PET}\left(\frac{\mathrm{SMS}_u}{\mathrm{AWC}_u d_u}\right) \qquad \text{Equation 18}$$

where PET is the potential evapotranspiration rate (m s^{-1}), AWC_u is the available water capacity of the upper layer (dimensionless), and d_u is the depth of the upper layer (m). Note that *ET* has a maximum value of PET. The disaggregation of daily PET from the NIWA climate database is discussed in Section 3.2.

Similar to interflow between layers, drainage from the lower layer (m s^{-1}) is calculated as:

$$F_{lower} = k_l \mathrm{SMS}_l \qquad \text{Equation 19}$$

where k_l is the recession coefficient (equal to k_u) and SMS_l is the soil moisture storage of the lower layer. This drainage from the lower layer is then separated into deep percolation to ground water (*R*) and raingarden outflow (F_o).

If the lower layer is at full capacity, flow from the upper layer is limited and is given a maximum value set to the same as the discharge from the lower layer (i.e., steady state). If both layers are at capacity, the excess water is able to pond on the surface and the depth of accumulated water is added to rainfall and inflow for the next time-step. Once the depth surpasses the invert level of the standpipe, the contribution of inflow to by-pass is calculated using the weir equation for a sharp crested weir (Equation 9). By-pass is added directly to F_o.

Although there is a potential for settling of coarse sediments on the soil surface, settling is not explicitly modelled. Instead, for simplicity, all of the removal processes are modelled as a single step where the sediment removal is related to the flow detention time (days), and the median particle size of the raingarden filter media (mm), using an empirical relationship of the form:

$$\log TSS_{OUT} = a - b\log\left(\frac{Detention\ time}{Particle\ size}\right) \qquad \text{Equation 20}$$

where TSS_{OUT} (%) is the percentage of the incoming TSS that leaves the raingarden, and *a* and *b* are coefficients. This formulation is taken from the MUSIC model (CRCCH, 2005) for soil filters, and the coefficients are the same as those used in MUSIC. TSS has not been split into size fractions for this module on the basis that there is no established method for doing so, and it is assumed that the removal efficiency is the same across the PSD.

The concentration of the sediment leaving the raingarden is calculated as the removed load divided by the outflow. For each new event, the detention time is calculated as the cumulative time from the rise of the inflow hydrograph when surface flows first enter the device. If by-pass occurs, the incoming contaminant load is then split between water treatment in the raingarden and no-treatment in by-pass water according to the ratio between the inflow and by-pass volumes for that time-step.

3.4.2 Module testing

The model has been developed and tested using flow and water quality data collected by NIWA at the Henderson Vehicle Testing Station raingarden for the ARC from June 2006 to July 2007 (Reed and Pattinson, 2007). The events sampled and the number of samples per event are summarised in Table 13. The raingarden takes stormwater from the car park and the roof of a neighbouring building (total area 3800 m^2). The area is roughly square (350 m^2) with a single inlet and outlet. The soil is a coarse pumice, gravel and sand mix. Much of the outflow from the raingarden is lost which could either be due to deep percolation or an underlying drain, while the rest is drained with a single outflow which leads to the reticulated stormwater network. The raingarden is set in a shallow basin to allow ponding on the surface during heavy rainfalls. This means that there could be some settling of coarse particles on the soil surface. In addition to drainage, the outflow has a 1.2 m diameter standpipe raised in rip-rap some 25 cm above the soil surface that can allow by-passes under extreme conditions, however, no by-pass was observed during the monitoring programme.

Flow and water quality were measured from the inflow and outflow (see Figure 41 for configuration) as listed below:

- Onsite rainfall (0.2 mm tipping bucket aggregated into 5 minute intervals).
- Raingarden inflow recorded using a 120° v-notch, sharp-crested weir with a float and counterweight driven stage recorder. Flow to the raingarden was diverted into a small, temporary, ply-wood holding bay.
- Raingarden outflow recording in the standpipe using a v-notch, sharp-crested weir with a float and counter weight driven stage recorder.
- Concentrations of total suspended soils (TSS), dissolved and particulate copper and zinc at the inlet and outlet. Timed samples (20 minute intervals) were taken with an ISCO automatic water sampler triggered by stage. All

samples were analysed within 48 hours of collection. A total of seven events was sampled (Table 13).

Figure 41 **Monitoring instruments were installed at the raingarden inlet (left) and outlet (right) to record inflow and outflow and to sample stormwater for chemical analysis. Photo by Pete Pattinson, 2006.**

Table 13 **Sampled events and number of samples taken**

Sample Number	30 Nov 2006	18-19 Dec 2006	9-10 Jan 2007	12 Mar 2007	28 Mar 2007	27-28 Apr 2007	20 June 2007
Inlet	12	12	12	12	12	11	23
Outlet	12	12	8	12	12	8	17

The simulation period for the raingarden was split into two sections for model development and calibration (1 June 2006 – 14 January 2007) and testing (15 January – 6 July 2007). The model parameters are summarised in Table 14. The ratio between recharge and raingarden outflow was calibrated here as 40%. The high proportion of water lost to outflow, apparently to recharge, suggests that there may be some other form of drainage from the raingarden. Unfortunately, plans for the raingarden have not been located. As flow and treatment from raingardens often includes a recharge component, it is essential that the performance rules allow users are able to specify whether deep percolation occurs and at what rate with respect to outflow.

Raingarden hydrology

The water balance method is able to simulate outflow from the raingarden well. Table 15 gives the correlation and total volume for the calibration and evaluation periods. The correlation for the entire period was 0.85. Figures 42 and 43 shows the time series for observed and simulated flows from the raingarden. There is a slight tendency to underestimate flow peaks. No by-pass events were observed or simulated.

Table 14 Summary of parameters for the raingarden module calibrated to the Henderson vehicle testing station

Flow sub-routine	
Available water capacity (proportion)	0.16
Depth of upper layer (m)	0.5
Depth of lower layer (m)	0.4
Recession coefficient of upper layer (s^{-1})	0.38
Recession coefficient of lower layer (s^{-1})	0.38
Area raingarden, (m^2)	350
Ratio recharge to outflow	0.4
Removal sub-routine	
Median grain size of filter medium (mm)	3
Removal parameter a	0.52
Removal parameter b	0.39
By-pass sub-routine	
Standpipe invert level (m)	0.25
Standpipe diameter (m)	1.2
Weir width (m)	3.77
Weir coefficient	0.6

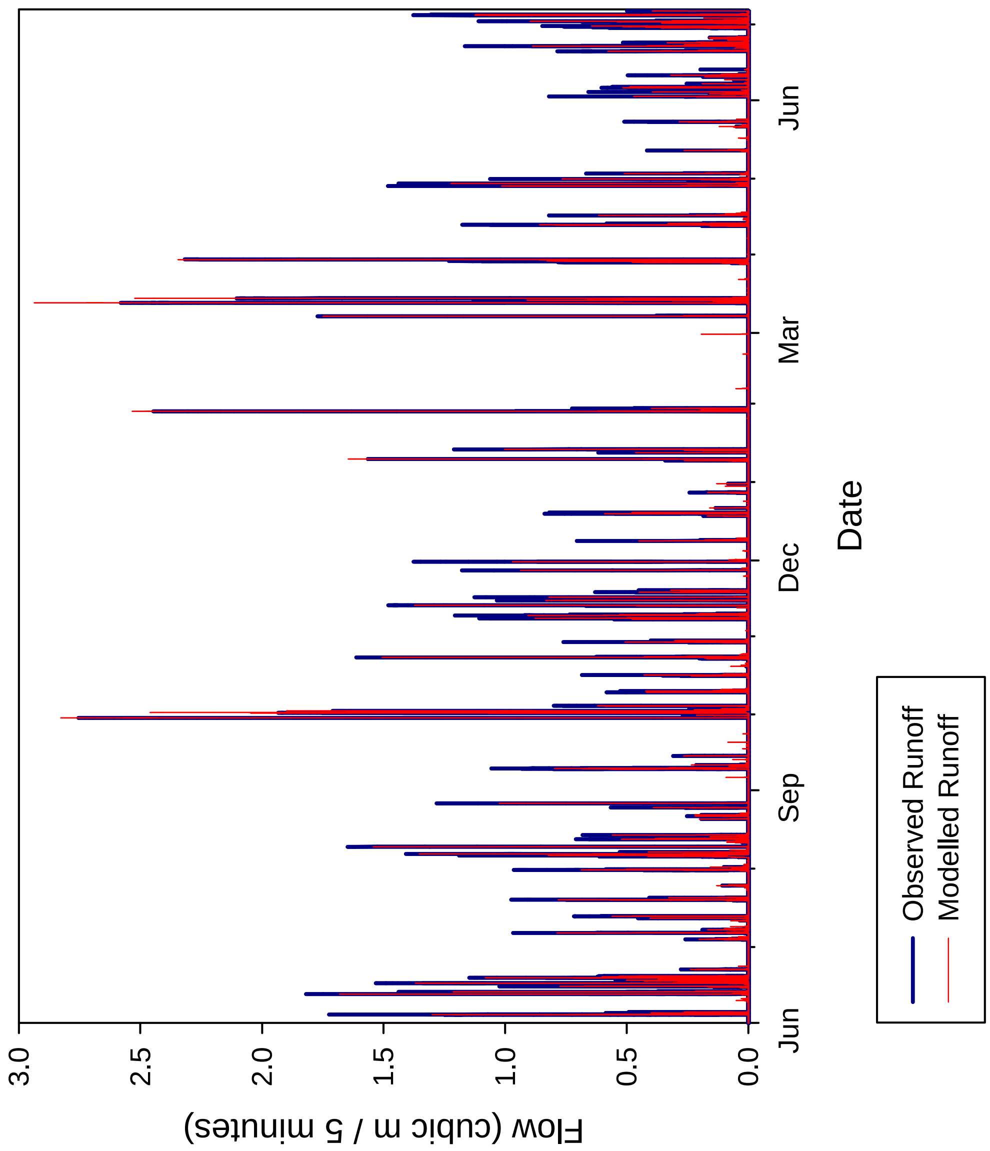


Figure 42 **Simulated and observed flow from the Henderson vehicle testing station raingarden -June 2006 to July 2007)**

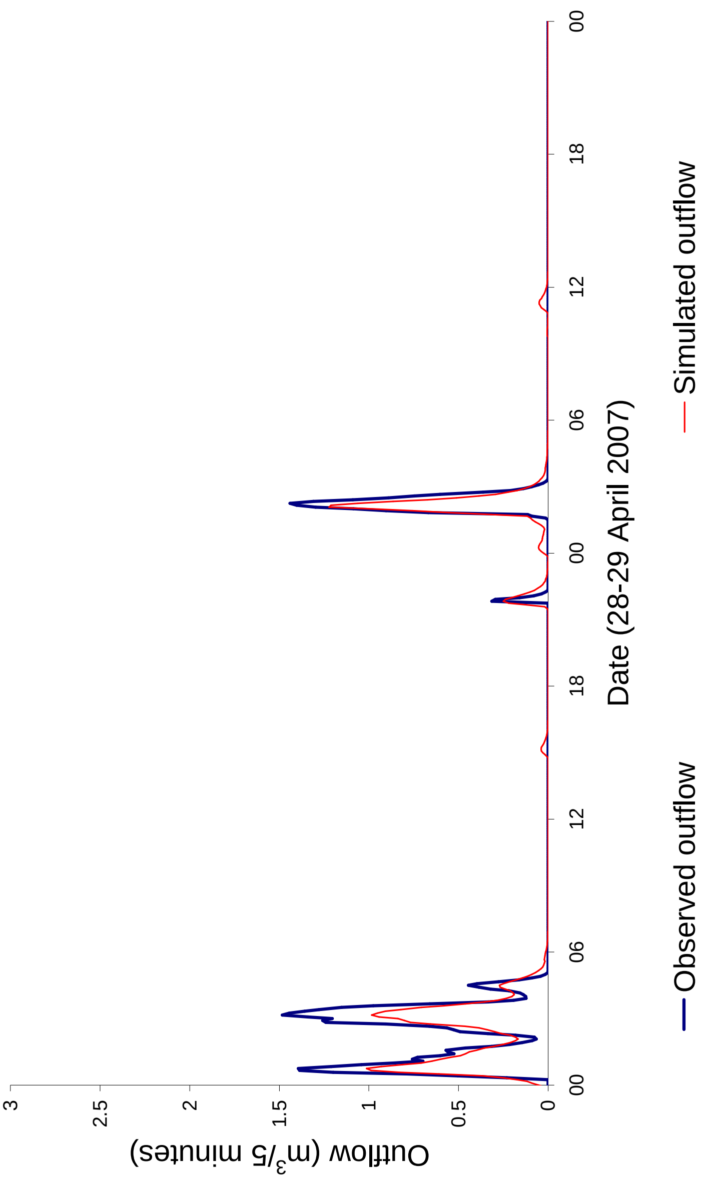


Figure 43 **Simulated and observed flow from the Henderson vehicle testing station raingarden – 28 to 29 April 2007**

Table 15 **Performance summary for the flow routines for the Henderson raingarden**

	Observed		Modelled	
	Calibration	Evaluation	Calibration	Evaluation
Correlation (R^2)	NA	NA	0.86	0.85
Total flow volume (m^3)	1080	638	1071 (difference -9)	660 (difference 22).

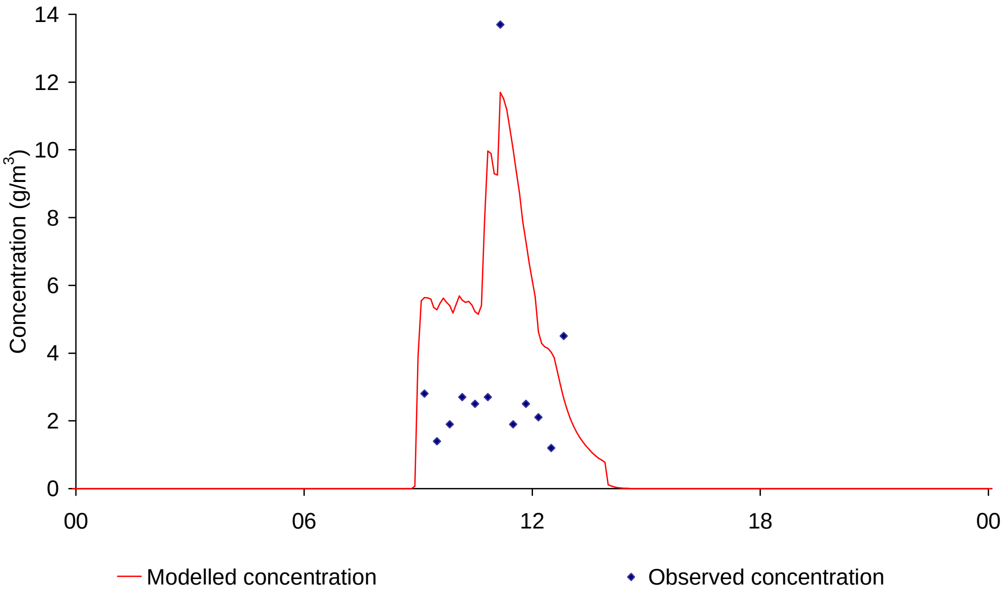
Sediment removal

Assuming linear interpolation between sampled timesteps, the total removal efficiency was 88% for the observed loads and 85% for the simulated loads. However, evaluation of the time series showed that model performance was variable with some events well simulated and others poorly. Sediment concentration and load are plotted for three events in Figures 44 and 45. Table 16 summarises the loads in and out of the raingarden by event. The high sediment load simulated for 30 November 2006 compared to that observed is partially explained by a slight overestimation of outflow for that event.

In terms of C-CALM, the model is adequate for the prediction of long term removal efficiencies.

Table 16 Summary of model performance for the Henderson Vehicle Testing Station raingarden

	Load in (g)	Observed load out (g)	Simulated load out (g)
30 November 2006	698	86	229
18 December 2006	66	2	10
19 December 2006	38	16	8
9 January 2007	44	5	13
12 March 2007	1936	183	222
28 March 2007	31	0	2
28 April 2007	523	66	36
20 June 2007	213	62	12



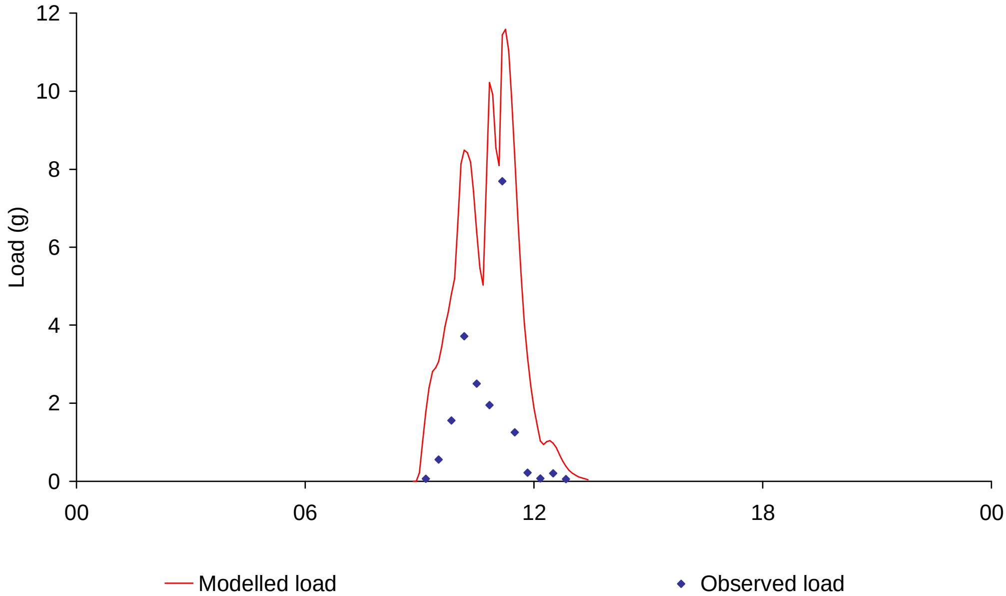
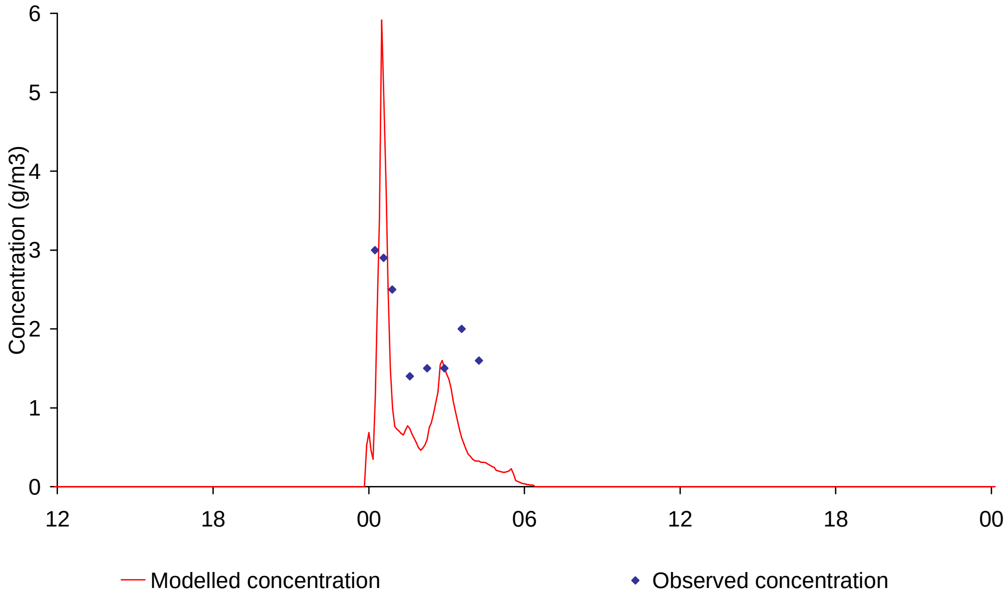


Figure 44 Simulated and observed sediment concentrations (above) and loads (below) for the Henderson vehicle testing Station raingarden - 30 November 2006



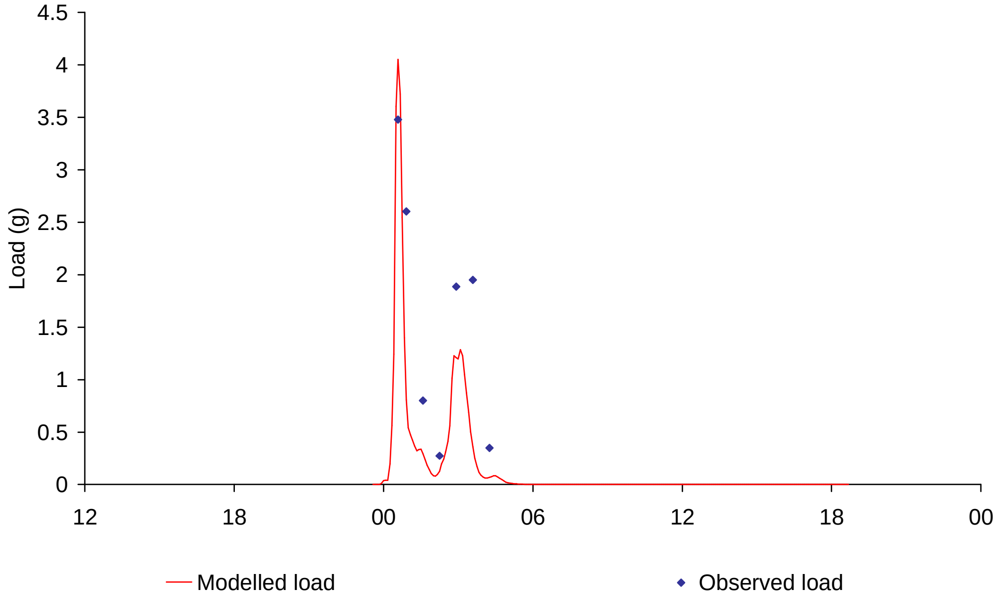


Figure 45 **Simulated and observed sediment concentrations (above) and loads (below) for the Henderson vehicle testing Station raingarden – 28 April 2007**

4. Discussion and Future Work

This report has presented the modelling context for C-CALM including the rationale behind the development of performance rules for stormwater treatment devices as a proxy explicit simulation within the SDSS. The main part of the report has been the description and testing of three informing modules which will be used to develop the performance rules. The idea is to couple the catchment surface flow model (i.e., input) to the pond/wetland and raingarden models and to run the modules for different sets of environmental drivers to obtain a matrix of removal efficiencies. The treatment matrixes will then be available as a library within the SDSS. The SDSS will take spatial data from the land cover geo-data base and user defined treatment options to create a query for the library. The library will then return the appropriate removal efficiency.

The modules had variable success with individual events but gave overall good fit with respect to both flows and sediment load and concentration. This success has enabled us to keep the modules as simple as possible with generic parameters rather than site specific and event specific calibration. Increasing the number of parameters increases both the run time of the modules and the memory needed in the SDSS library. The latter could effect the time needed to run queries in the GIS which may lead to user impatience with C-CALM.

4.1 Performance rule development

The performance rules will work with the scaling principle that stormwater treatment is a function of the specific area (or volume) of the device with respect to the catchment contributing area (10 ha for ponds and wetlands, 0.5 ha for raingardens) rather than the actual dimensions of individual devices. This choice was made to reduce the number of model runs needed to develop the performance rules. The precedence for doing this is the work done by Elliot et al (2006; 2007), they showed that aggregating treatment and stormwater network elements can successfully be used to reduce the complexity of stormwater drainage models where catchment hydrological conditions are the same.

A necessary step, not yet carried out, in developing the performance rules will be coupling the informing modules to access error propagation.

A number of key parameters which will be need to be included in the performance rules model runs have been identified as part of the module development presented here. These are listed below.

Catchment Surface Flows to Settling Ponds and Wetlands

The parameters below give a total of 388,800 unique combinations.

Table 17 Parameter set for the generation of the performance rule library for settling ponds and wetlands in C-CALM.

Catchment Parameters:	
Regional rainfall and evapotranspiration:	Auckland/Northland/Waikato, Bay of Plenty/East Cape/Hawke's Bay, Taranaki/Manuwatu, Wellington/Tasman, Marlborough, Canterbury, Otago/Southland, West Coast
Land use (i.e. build-up and wash-off rates) and impervious percentage of surface area:	Residential – 20%, 40% and 60% Commercial – 60%, 75% and 90% Industrial – 60%, 75% and 90%
Average catchment slope:	0.005, 0.01, 0.02 and >0.02 (Baldwin St in Dunedin is around 0.3)

Pond (and Wetland) Parameters:	
Specific area: (ratio of pond surface area relative to the total catchment area)	50, 100, 150, 200 and >250 m^2 ha^{-1}
Invert level:	0.5, 1.0 and >1.5 m
Width (or width equivalent) of outlet weir:	1, 2 and 3 m
Extended detention:	Yes, or no (if yes, slot weir width is set to 10% of the outlet weir width, depth = 30cm)
Hydraulic rating:	1 (poor), 3.5 (good) and 8 (excellent)
Particle size distribution:	Fine, medium fine, medium, medium coarse, and coarse

Raingardens / Bioretention

The parameters below give a total of 11,520 unique combinations.

Table 18 Parameter set for the generation of the performance rule library for raingardens in C-CALM.

Catchment Parameters:	
Region:	Auckland/Northland/Waikato, Bay of Plenty/East Cape/Hawke's Bay, Taranaki/Manuwatu, Wellington/Tasman, Marlborough, Canterbury, Otago/Southland, West Coast
Land use (i.e. build-up and wash-off rates):	Residential, commercial and industrial
Average catchment slope:	0.005, 0.01, 0.02 and >0.02

Raingarden (or Bioretention Unit) Parameters:	
Specific area: (ratio of raingarden surface area relative to the total catchment area)	100, 200, 400 and 600 m^2 ha^{-1}
Depth:	0.5, 1.0 and 1.5 m
Bypass:	Yes, or no (if yes, generic parameters for the bypass outflow weir)
Deep percolation to groundwater:	0% (isolated from groundwater), 10%, 20%, 40% and 50%

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