
by

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ABSTRACT


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The analysis of large-scale watershed processes and development of an efficient and integrated modelling platform is the focus of this research. The research focused on developing a series of modelling tools that can be used in the simulation of the overall response of a watershed based on a localized or distributed hydrologic event. This is achieved through the introduction of a hybrid modelling concept using a combination of empirically based lumped hydrologic processes and a physics-based distributed model representation. The watershed simulation model (GFLOOD) was developed to account for the complexity of the watershed including the variations in climate, soils, topography, and landuse conditions across the watershed. GFLOOD stands for Guelph Flood Forecasting Model, a river basin or watershed scale flow prediction model.

Two major modelling components of the GFLOOD model are the time parameters (time of concentration ($T_c$) and recession constant ($K$)) and the channel routing component. Each of these modelling components is evaluated separately. The equations developed in this study for estimating the time parameters can be used as an initial estimate for $T_c$ and $K$ for ungauged basins, and through calibration and/or sensitivity analysis the values of $T_c$ and $K$ can be finalized. The Saint Venant equations for flood routing are solved by transforming the momentum equation into a partial differential equation which has six parameters related to cross-sectional area and discharge of the channel, left floodplain and right floodplain. The simplified dynamic model was further modified to account for transmission losses, evaporation losses and bank storage within the channel. The model was compared with the solutions of the general dynamic wave model, diffusion wave model and the more complex dynamic wave model. The comparison shows that there is good agreement between the results of the simplified dynamic model and the other
models however, the simplified dynamic model is easier to formulate and compute than the other models. The complete GFLOOD model was applied to the Welland River Watershed within Southern Ontario. The model was evaluated for its ability to predict streamflow and water levels along the main branch of the Welland River.
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Chapter 1 – Introduction

Worldwide flooding is the leading cause of losses from natural hazards and is responsible for a greater number of damaging events than any other type of natural event. At least one third of all losses due to nature’s forces can be attributed to flooding. In recent decades flood damages have been extremely severe and it is evident that the frequency and intensity of floods are increasing (Loster, 1999). In Ontario, flooding is the leading cause of public emergency (Ministry of Municipal Affairs and Housing, 2007).

The application of flood control measures to reduce flood damages are often based on peak flows, water levels and their recurrence interval. In order to design these measures appropriately, engineers require information such as the peak discharge, peak water level and time to peak for large storm events. This information is required in a variety of design applications including storage reservoirs, in-channel works, dams, spillways, bridges and culverts. As a result, some form of hydrologic and/or hydraulic analysis is required to obtain the necessary information in order to make the right design decisions.

Standard methods of analysis in engineering design include empirical and/or statistical approaches. In Ontario, the Ministry of Natural Resources (MNR) Technical Guideline on Floodplain management (2002) recommends that a flood frequency analysis be undertaken as a primary method of analysis. A flood frequency analysis predicts flow rates and water levels within certain return periods. Extrapolation is sometimes done in order to predict hydrologic and hydraulic parameters beyond the record length. This is typically done when the structures service life extends beyond the record length. In turn, historical flow and water level data is required in order to conduct flood frequency analysis.

Many streams are ungauged and do not have flow or water level records. Muzik (1993) and Ajward (1996) have indicated that even when stream gauges are in place the records are too short to accurately predict extreme events and estimate hydraulic parameters. When historical flow and water level records are unavailable the MNR Technical Guideline on Floodplain
Management (2002) further recommends the use of a watershed simulation model as secondary method of analysis and regionalization as a tertiary. A watershed simulation model incorporates physical characteristics of the watershed to predict flows and water levels. A properly calibrated and validated watershed model can provide a synthetic time series of flows and water levels for a given precipitation input. If streamflow and water level data for the watershed is unavailable for the purposes of calibration and validation, then regionalization may be used to calibrate and validate the model, or as an alternative method of analysis.

In numerical simulation, the response of a watershed to an event is described using mathematical operations. The availability of field data is a factor that can affect the complexity of the model. Physically based watershed models can provide realistic estimates of flows and water levels, since they represent actual field conditions, provided that their parameters have been properly defined through calibration and validation. In many watershed models, the methods used for defining the watershed parameters may be dependent upon empirical equations, which were developed using hydro-meteorological data and lumped basin parameter values such as slope, flow length and drainage area. These empirical relationships may not accurately reflect the hydro-meteorological conditions for basins within Ontario, which can lead to errors in runoff estimates. Furthermore, the present empirical equations do not allow for visual or manual inspection of the accuracy of the variables used in the equations (Green and Nelson, 2002).

Watershed modelling for estimating flows and water levels typically involves two steps; first, a hydrologic model is used to obtain the inflow hydrograph or flow to the channel. Second, the flow or inflow hydrograph must then be input to a steady state or unsteady state hydraulic model in order to predict the water levels at key sites along the river. This process can be relatively time consuming and depending on the application of the model it can inefficient. For example, in operational flood forecasting regardless of how accurate a model may be in predicting the flood magnitude, if a forecast is consistently late it serves no useful purpose (Singh, 1989). In turn, a hybrid modelling approach combining both hydrologic and hydraulic modelling capabilities can simplify the current process. In addition, a hybrid model would allow for both models to complement each other and overcome many of their own limitations. For example, hydraulic routing models have the ability to produce output describing both water level and discharge
hydrographs at the outlet and along the reach. However, many hydraulic models do not have the capability to model rainfall – runoff processes and this information has to be input either from field observations or another model. A hydrologic model does have this capability, however, it can only provide discharge hydrographs as output, and this hydrograph output is generated at the outlet of the basin. The provision of a single discharge hydrograph at the basin outlet should not necessarily be perceived as a limitation in hydrologic models. However, if a modeller were interested in determining the change in velocities or water levels along a river, a single discharge hydrograph at the outlet would not be sufficient information. Therefore, a model’s limitations become a function of its intended application, which in turn, relates back to its own capabilities.

Hydrologic models using hydrologic routing techniques are limited in that they neglect backwater effects and assume that the flood wave is similar in shape and magnitude to previous flood waves for which stage and discharge observations are available for calibrating the hydrologic routing parameters. Such an assumption may be true if the flood wave was kinematic in nature; however, if the flood wave was dynamic and the reach was affected by floodplain storage and backwater effects, such an assumption would not hold true. Hydraulic models compute the flow rate and water level simultaneously instead of separately, so that the model more closely approximates the actual unsteady non-uniform nature of flow propagation in the channel. Furthermore, hydraulic models can be applied to dynamic problems, such as dam breaches and ice jam releases, which cannot be handled by traditional hydrologic modelling approaches. Therefore, a hybrid hydrologic-hydraulic model would have a range of capabilities that would be able to solve a variety of hydrologic and hydraulic engineering problems.
Chapter 2 – Literature Review

Watershed models can vary with respect to process and scale. In order to review the hydrologic and hydraulic processes that make up these models, it is first necessary to review and understand how these models are classified with respect to process and scale and how user interfaces such as geographical information systems (GIS) and remote sensing techniques can improve their accuracy. Process and scale are especially important, since they will determine the overall structure of the model (Singh, 1995).

2.1 Classification of Watershed Models

There are different types of watershed simulation models, and although these models share common similarities because their underlying assumptions are the same, many of these models are distinctly different. As a result, watershed simulation models are classified according to different criteria which encompass both process and scale.

A watershed simulation model has five (5) main components, (1) input, (2) watershed characteristics, (3) governing laws and equations, (4) initial and boundary conditions, and (5) output. Depending on the type of watershed model and its intended application these components can be combined in a variety of ways. The processes that make up the watershed model contribute to the model’s output. A description of these processes in conjunction with the watershed characteristics, the model can be described as being lumped or distributed, deterministic or stochastic or mixed.

A lumped model is expressed by differential equations and it does not take into account the spatial variability of the processes, input, boundary conditions and watershed characteristics. Examples of lumped models are HEC-1 (HEC, 1981); HYMO (Williams and Hann, 1973); RORB (Laurenson and Mein, 1993); SSAR (USACE, 1987); and the Tank Model (Sugawara, et al., 1974).
Distributed models take into account the spatial variability of processes, inputs, boundary conditions and watershed characteristics. However, this is a relatively generalized description of distributed models. Many of the components and processes in these models are lumped and are therefore, not considered to be fully distributed but are rather semi- or quasi- distributed. Examples of distributed models include SWMM (Metcalf and Eddy, Inc., et al. 1971), GAWSER (Schroeter et al., 2000), HSP-F (Bicknell et al, 2001), WATFLOOD (Kouwen, 2000), HEC-HMS (USACE, 2000), and MIKE-SHE (Refsgard and Storm, 1995), EUROSEM (Morgan et al., 1998) and PCSWMM (James et al., 2006).

Watershed models are classified according to the processes that make-up the model and how they are those processes are described. If all of the components of a model are deterministic, the model is deterministic. Similarly, if all of the components are stochastic, the model is stochastic. When the model components are described by a mix of deterministic and stochastic components, the model is a hybrid model.

A majority of the models currently in use are deterministic, and virtually no model is fully stochastic (Singh and Woolhisers, 2002). There are examples in which some parts of the model are stochastic and other parts are fully deterministic. Such models are referred to as quasi-deterministic or quasi-stochastic. An example of a quasi-stochastic model is the API method. The Ministry of Natural Resources of Ontario Flood Forecasting Branch uses the API (Antecedent Precipitation Index) Method for estimating potential runoff within a region or basin for a specific day during the year provided the amount of precipitation or available water for runoff is known. However, the API method presumes that there is a certain amount of soil-moisture storage for each specific day of the year. If this value can be obtained through independent measurements at the time of the forecast then the API value is updated accordingly and the runoff potential estimated accurately (source Ministry of Natural Resources Flood Forecasting and Warning website: www.mnr.gov.on.ca).

Similarly, an example of a quasi-deterministic flood forecasting model is the GRIFFS model for the Grand River Watershed. The Grand River Integrated Flood Forecast System (GRIFFS) is a real-time streamflow forecasting model including rainfall – runoff modelling and combined rainfall – snowmelt modelling. The hydrologic routines in GRIFFS are based on GAWSER (Guelph All –
Weather Events Runoff) model. The model is run using hourly input data and parameters are adjusted during the start-up period so that the first rise in the hydrograph is modelled correctly. Current work involves incorporating of real-time GIS-based radar – rainfall data into the forecast model using NEXRAD data from the National Weather Service (NWS) (Grand River Conservation Authority, 2007).

An example of a hybrid model is the simulation models implemented in Italy for flood forecasting which are based on two different approaches; black – box statistical modelling based on the Constrained Linear System (CLS) model (Todini, 1978), and a conceptual semi-distributed model based on a version of the Xinanjiang model described by Franchini and Pacciani (1991). The model predictions are directly corrected (or updated) by means of a Kalman filtering technique or by the MISP (Mutually Interactive State Parameter Algorithm) (Todini, 1978). The model forecasts are updated at each time step based on the differences between observed and forecasted flows at previous N-1 steps (usually N<6).

Another example, of a hybrid model can be found in Wagner et al. (2005). They investigated the identification of a simulation model for operational applications with respect to flood forecasting. A probability-distributed soil moisture model was coupled with a linear parallel routing scheme, and conditioned on rainfall-runoff observations from three catchments in the southeast of England. Using an abstraction control program, which requires accurate simulation of the intermediate flow range, it was shown that using the traditional root mean square error (RMSE) fit criterion, produces operationally sub-optimal predictions. This was also true in the identification period, when applied to a testing period, and to proxy catchment data. Using a second case study of the Leaf River in Mississippi (USA), where the focus changed to predicting flood peaks over a specified threshold, also suggested that the relevant flood threshold should govern the objective function choice. It was concluded that, due to limitations in the structure of the employed model, it would be counter-productive to try to achieve a good all-round representation of the rainfall-runoff processes, and that a more empirical approach to identification may be preferred for specific forecasting problems.
Watershed simulation models can also be classified based on the time scale of the models. The time scale can be defined as a combination of two time-intervals (Diskin and Simon, 1979; Singh, 1995). One of the time intervals is used for input and internal computations. The second time interval is used for model output and calibration. Based on this description, watershed simulation models can be defined as either continuous based and/or event based.

Singh (1989) refers to the time interval used for data collection and input as the sampling time interval ($STI$) or the computational time step ($Δt$). The $Δt$ for data collection and input will depend on the time of concentration ($T_c$) or lag time of the basin. The intended purpose or application of the model can also factor into the selection of $Δt$. For example, for the purposes of flood forecasting and warning, if $Δt$ is equal to or greater than $T_c$ of the basin it is ineffective. By the time the forecast is made and the warning issued the flood has already passed and the forecast and warning have served no purpose. If $Δt$ is less-than $T_c$ then several forecasts can be made and updated as data becomes available, and warnings can be issued more frequently. Similarly, if the model is used to estimate the design flood peak flow, a high $Δt$ value could lead to a low estimation of the flood peak. Since the $STI$ is greater than $T_c$, thereby missing the peak. As a rule of thumb, $Δt$ or $STI$ is selected between one-third and one-sixth of $T_c$.

The spatial scale can be used as a criterion to classify models into small watershed, medium sized watershed, and large watershed models. Singh (1995) defines watersheds with areas of 100 km$^2$ or less as small, those with areas 100 to 1000 km$^2$ medium and those with areas larger than 1,000 km$^2$ large. This type of classification is rather subjective, since with respect to runoff generation, in addition, to the size of the basin, the runoff within the watershed will also be influenced by land cover and its underlying soils. For example, large watersheds will have mix of land cover and soils with a defined channel network, making them heterogeneous. Whereas small watersheds tend to be more homogeneous with respect to land cover and soils and the channel network is not as defined. Large watersheds tend to be impacted more by long duration, less intense, high volume events, such as a snowmelt or snowmelt plus rainfall events, whereas small watersheds tend to be effected by short duration, high intensity events of the thunderstorm type (Singh 1995; Chaplot, et al., 2005, Camorani, et al., 2005). The level of homogeneity or heterogeneity within a watershed is dependent upon the drainage area of the watershed. For a small watershed heterogeneity it is not as evident, for
a medium sized watershed, the heterogeneity is present but it is in the medium size range (±2 standard deviation around the mean), whereas it is quite large for large watersheds (Singh, 1995). A watershed can be heterogeneous in one characteristic and homogeneous in another. For example, the dominant land use in a watershed could be agricultural; however, the underlying soils can vary spatially, thereby, influencing the runoff potential in the watershed. When classifying watersheds with respect to scale all physical characteristics must be taken into consideration (Camorani, et al., 2005, Liu, et al., 2005, Liu et al., 2005 and Liu and Schmedt, 2005).

2.1.1 Scaling and its Effects on Watershed Simulation Models

Song and James (1992) reviewed five (5) scales used in hydrologic simulation: laboratory scale, hill slope scale, catchment scale, basin scale, continental and global scales (Singh, 1995). The vast majority of watershed simulation models used for flood forecasting are either catchment scale or basin scale. For example, the GAWSER model which is part of the Grand River Integrated Flood Forecasting System (GRIFFS) is a catchment scale model (Schroeter et al., 2000, Grand River Conservation Authority, 2007). The US National Weather Service uses the NWS flood forecasting model to administer the national flood warning program, the NWS model, is a catchment scale model. Other catchment scale models include the UBC model and WATFLOOD model which are popular in Canada for hydrologic simulation. The RORB and WBN models are commonly employed for flood forecasting in Australia. The TOPMODEL and MIKE-SHE models are commonly employed in European countries for flood forecasting and the HBV model is the standard model for flood forecasting in Scandinavian countries (Singh and Woolhiser, 2002). In other parts of the world, such as Thailand, flood forecasting is carried out using a Discrete Linear Cascade Model (DLCM), which is applied at the basin scale (Uthaisang, 1993). Catchment scale models add topography in simulation of surface runoff and geology in simulation of baseflow, and often divide larger catchments into smaller homogeneous parts. Basin scale models employ storage and translatory routing schemes in combining catchment runoff.

With respect to flood forecasting, heterogeneity would be greater at the basin scale as opposed to the catchment scale. This heterogeneity can be traced to three sources; the physical characteristics of the watershed, the boundaries separating the physical characteristics and the differences in the
factors that control hydrologic processes. The modeller must determine what level of scale would be most appropriate for flow and water level computations. As the spatial scale changes from a small area to a large area, the watershed response becomes less sensitive to spatial variations of input as well as watershed characteristics (Singh, 1995). The spatial extent, however, is constrained by the differences in physical, vegetative, and topographic features that can cause the pattern of hydrologic response to vary. For example, Chapolt (2005) examined the impact of DEM mesh size and soil map scale on runoff. Results from the Lower Walnut Creek (21.8 km², central Iowa) showed that an upper limit to DEM mesh size of 50 m is required in order to simulate watershed runoff accurately. Decreasing the mesh size beyond this threshold does not substantially affect the computed runoff flux; however, depending on the DEM mesh size considered, a detailed soil map had to be considered in order to accurately estimate the runoff loads. This, then, leads to defining a scale as a unit or subwatershed within which the hydrologic response can be treated homogeneous. This scale must not be too small to be dominated by local physical features, nor so large as to ignore significant hydrologic heterogeneity caused by spatial variability of the watershed characteristics. Therefore, the optimal scale in a watershed is determined by the collective working of multiple processes that generate hydrologic response and the availability of hydrologic data.

2.1.2 Remote Sensing Applications to Watershed Simulation Models

An issue in hydrologic and hydraulic modelling is the availability of adequate data to quantitatively describe the watershed processes accurately. Schultz (1988) and reported by Singh (1995) identified the following eight points in using remote sensing technology in aiding the development of a watershed model:

“It produces aerial measurements in place of point measurements; all information is collected and stored at one place; it offers high resolution in space and/or time; data are available in a digital form; data acquisition does not interfere with data observation; data can be gathered for remote areas that are otherwise inaccessible; and once the remote sensing networks are installed, data measurement is relatively inexpensive.”
Singh (1995) makes the following comments regarding remote sensing technology:

“Remote sensing and satellite data can be used for flood forecast modeling, and are useful, especially where data collection sites and cost-effectiveness of data collection are important on one hand, and increasing requirement for data on the other. Satellites with hydrologically useful data are NOAA series, TIROS N, SPOT, Landsat, GOES, GMS, and Meteosat. Remote sensing permits obtaining data from a location far away from the user or modeller. Sensing means observation of the average value of a variable over some areal extent by examining the electromagnetic energy. Measurement techniques are either passive or active. Passive measurement techniques determine the amount of reflected sunlight or amount of natural emissions from the target at various wave lengths. Active measurement techniques direct an artificially generated signal at a target and measure the reflected signal. Several different remote sensing sensors provide hydrological information including aerial photography, scanning radiometers, spectrometers, and microwave radars. Since all information is obtained in the form of electromagnetic signals, each signal must be related to a specific hydrologic variable. In addition, the needed resolution in space and time is obtained with certain types of sensors only. Remote sensing data from satellites are useful in flood forecast modeling in two ways. First, satellite data can be used to better define soils and land cover over a watershed, which are needed to determine infiltration, evaporation and runoff. Second, remote sensing measures data over space rather than at a point, and can therefore, be used to correct errors in input data based on point measurements such as is with rainfall, evaporation, and the like.”

One of the key advantages to using remote sensing technology in flood prediction is that it can produce the soil moisture and evapotranspiration data through satellite thermal infrared imaging. These data can be employed to model water exchange between land surface and atmosphere (Papa et al., 2003), and to couple land-surface hydrology and atmospheric models (Anderson et al., 2002).

The remote sensing technology can be employed in a variety of applications in hydrology and flood forecasting. For example, radars are employed for rainfall measurements (NEXRAD, WSR-88D weather radar data, NOAA). Satellite data can be used for estimation of area and intensity of
rainfall. Temperature estimates obtained from thermal infrared and soil moisture from microwave observations can be applied to develop mathematical models of evapotranspiration. Soil moisture is most adequately obtained through remote sensing. Inventories of water bodies such as lakes, dams, rivers and wetlands are most easily obtained by remote sensing imagery (Kouraev, et al., 2004). Furthermore, snow and ice conditions can also be mapped with the use of satellites.

Improvements in the capability to observe hydrologic data through remote sensing technology has led to a range of hydrologic applications, and especially within flood forecasting. Some of these applications are demonstrated in real-time flood prediction by a distributed model with radar rainfall measurements as input, development of distributed hydrologic models based on a digital terrain model and a geographic information system with the use of Landsat satellite imagery (Anderson et al., 2002 and Tate et al., 2002). These applications permit the use of space technology in such areas as weather forecasting, forecasting seasonal/short term snowmelt runoff, development of reporting services for drought assessment/forecasting and flood damage assessment.

2.1.3 Application of GIS to Watershed Simulation Models

Geographic information systems (GIS) are tools that store, analyze, retrieve, manipulate and manage large amounts of spatial data. Flood forecasting and flood forecast models require large amounts of accurate and representative data. What differentiates GIS from other information management systems is that each piece of data is geo-referenced. Furthermore, GIS differs from other mapping software applications such as computer-aided-design (CAD), in that GIS has topology built into its geo-spatial data. In essence, CAD models “real world” objects whereas GIS models the “world”.

Through the use of GIS and associated software, such data can be compiled and processed with relative ease. The watershed and its associated hydrology are inherently spatial. The physical characteristics of the watershed such as soils, land use and topography vary spatially. In distributed flood forecasting models, these characteristics can be used directly. However, the watershed would have to be characterized into homogeneous sub-catchments.
Distributed watershed simulation models require large volumes of data. A hydrologic unit is characterized by many data, including topography, land use, soils, geology and climate. The various flood forecasting models may require the same data stored in different files, thus creating storage redundancy within a traditional file structure system. Many of the data issues and problems that are encountered in hydrologic modelling can be resolved through the use and application of GIS. Its ability to extract, overlay, and delineates watershed characteristics permits integration with distributed flood forecasting models (Garbrecht et al., 2001). Application of GIS to flood simulation requires planning and extensive data manipulation, involving three major steps: spatial database construction; integration of spatial model layers; and the GIS and model interface. The first step is the most time consuming. The second and third steps are less tedious due to advances GIS capabilities and programming extension GIS data models that work in conjunction with hydrologic models for example, HEC-GeoHMS/HEC-HMS (USACE, 2003).

Construction of an appropriate of an appropriate geo-spatial data base is the most difficult and time consuming undertaking for the purposes of flood forecast modelling. There exist large volumes of data on soils, geology, land use, vegetative cover and hydrometric data in digitized form. Some of the more common problems encountered within the existing digital data include: different formats of digital data and the use of different coordinate and projection systems.

Integration of GIS with flood forecasting models accomplishes a number of significant functions including: calibration and modification of watershed parameters and the ability to simulate and compare different forecast and forecast models. GIS improves the ability to incorporate spatial details beyond the existing capabilities of watershed and/or forecast models. Thereby resulting in improved accuracy of forecasts, less duplication, easier map storage, more computational flexibility, and ease of data sharing, timeliness, greater efficiency, and higher product complexity when using GIS in conjunction with watershed and/or flood forecast models (Ogden et al., 2001).

The use of distributed watershed models along with GIS interfaces and modules have given modellers and forecasters the ability to incorporate spatial details beyond the existing capabilities of traditional watershed models. Distributed hydrologic models that fully use spatial data from
GIS are reaching maturity, although questions regarding their appropriate application remain in the research arena. There is a growing body of evidence that no hydrologic model is universally applicable (Ogden et al., 2001). The variety of runoff production mechanisms and the wide range of space scales and timescales studied have necessitated creation of a number of different spatially distributed model formulations.

The increasing availability of spatially distributed topographic soils, land-use, land-cover, and precipitation data including remotely-sensed data such as weather radar data (i.e. NEXRAD data) provides the prime motivation for the development, verification, and eventual acceptance of GIS modules and distributed hydrologic models capable of taking full advantage of these new data within flood simulation. However, new developments must acknowledge the uncertainties inherent in the data and subsequent model parameter assignments (Singh and Woolhiser, 2002). Lumped parameters at the basin scale made sense when data were largely read from paper maps. However, the increasing use of GIS to store watershed characteristics data is forcing hydrologic modellers and specifically forecasters to spatially aggregate data to the lumped sub-catchment scale because the application of lumped models on very small sub-catchments is not feasible. This is not to say that all hydrologic predictions require a distributed model. Lumped models will remain valuable tools with many applications. The development of distributed models will allow flood simulation to be made with a level of detail and accuracy that include the impacts of spatial variability of watershed characteristics and precipitation where necessary, as well as provide spatially distributed output of hydrologic variables.

### 2.2 Time Parameters

Time parameters are used in hydrology to measure the response of a watershed to a hydrologic event. The time of concentration ($T_c$) and the recession constant ($K$) are two widely used parameters for estimating the peak discharge and flow hydrograph in hydrologic designs and analyses. Synthetic unit hydrographs have typically been used to develop streamflow hydrographs for a watershed basin. Measured streamflow or rainfall data for a particular point of analysis is rarely available. A fundamental part of the development of an overland flow hydrograph is the use of synthetic unit hydrographs such as the SCS (USDA, 1985), Williams
and Haan (1973), or Clark (1945) unit hydrographs. All three of these unit hydrograph methods (as well as most others) require the estimation of $T_c$ and $K$. With the advent of GIS and the ability to process vector and raster data, the ability of estimating travel times from watershed geo-spatial data has been realized. These methods are dependent on empirical equations which were developed using hydro-meteorological data and lumped basin parameter values such as slope, flow length and drainage area. In Ontario, there are 504 active streamflow stations and over 800 active climate stations; however, very few equations or empirical relationships have been developed using hydro-meteorological data and basin parameters from Ontario (Environment Canada, 2012). The current empirical relationships do not accurately reflect the hydro-meteorological conditions for basins within Ontario nor do they allow for computations to be made using the actual physical parameters of the basins.

2.2.1 Time of Concentration

The time of concentration ($T_c$) of a watershed is the time needed for runoff to travel from the most remote point in a watershed to the watershed outlet (Kirpich 1940; Bell and Kar, 1969; NRCS, 1972; McCuen et al, 1984; Haan et al, 1994; and Garg 2001). The time of concentration is a function of the topography, surficial geology and land use within the watershed. Bondelid et al, (1982) showed that 75% of the total error in estimating the peak flow can be attributed to errors in the estimation of $T_c$. Different methods and empirical equations have been developed over the years to estimate $T_c$. Empirical equations for estimating $T_c$ can be classified into two groups, basins where overland flow dominates and basins where channel flow dominates. Equations developed for basins where overland flow dominates include equations by Izzard (1946), Henderson and Wooding (1964), Morgali and Linsley (1965), Aaron and Erborge (1973), Wong (1995), Wong (1996a), Wong (1996b), Wong and Chen (1997) and Wong (2002). Equations developed for basins where channel flow dominates include equations by Williams (1922), Kirpich (1940), Johnstone–Cross (1949), and Haktanir and Sezen (1990). Common input parameters among the different types of empirical equations include: basin slope, flow length, Manning’s roughness coefficient or flow retardance factor and rainfall intensity (McCuen 1998). The application of empirical equations to regions other than those used in their development has shown that the majority of them are unreliable (Kibler and Aron, 1983; McCuen et al, 1994;
Goitom, 1989). However, empirical equations and regionalization do serve a purpose, especially when observed field is available for a watershed or region, which in turn, provides some level of reliability within the equations.

2.2.2 Recession Constant

The *recession curve* is the specific part of the flood hydrograph after the crest (and the rainfall event) where streamflow diminishes, referred to as baseflow. The slope of the recession curve flattens over time from its initial steepness as the “quick flow” component passes and baseflow becomes dominant. A *recession period* lasts until stream flow begins to increase again due to subsequent rainfall. Hence, recession curves are the parts of the hydrograph that are dominated by the release of water from natural storages, including surface storage, subsurface (inter) flow storage and groundwater flow. The recession constant $K$ is commonly used as an indicator of the extent of baseflow and is the product of the recession constants for 3 individual components including the surface storage recession constant ($K_{surf}$), the subsurface (inter) flow recession constant ($K_{subs}$) and the groundwater flow recession constant ($K_{gw}$), (Nathan and McMahon, 1990). The typical ranges of daily recession constants for streamflow components, namely runoff (0.2-0.8), interflow (0.7-0.94) and groundwater flow (0.93-0.995) do overlap (Nathan and McMahon, 1990). However, high recession constants (>0.9) tend to indicate dominance of baseflow in streamflow. *Recession segments* are selected from the hydrograph and can be individually or collectively analyzed to gain an understanding of these discharge processes that make up baseflow. Graphical approaches have traditionally been taken (Barnes 1939; Knisel 1963; Singh and Stall 1971; and Brustaert and Nieber 1977), but more recently analysis has focused on defining an analytical solution or mathematical model that can adequately fit the recession segments (James and Thompson 1970; Jones and McGilchrist 1978; Britles 1978; Vogel and Kroll 1996; Smaktin 2001; and Sujono et al., 2004). A review of studies relating to baseflow recession was performed by Hall (1968) and Tallaksen (1995).
2.3 Channel Routing

Flow routing is a procedure to determine the time and magnitude of flow at a point on a watercourse from known or assumed hydrographs at one or more points upstream. When applied to flows in irregularly-shaped channels, for example, rivers with inundated floodplain zones, the exercise becomes very complex. If channel sections are treated as a single composite section, discharges are generally under-estimated. If the channel and floodplain are treated separately, then system discharges are generally over-estimated (Wormleaton and Hadjipanos, 1982). Successful applications of dynamic flood routing models are dependent not only on proper initial and boundary conditions being established, but may also require certain modifications to the original data sets (Kouwen, 1984).

The flow of water through the channel network of a watershed is a distributed process because the flow rate, velocity, and depth vary in both space and time throughout the watershed. Estimates of the flow rate or water level at important locations in the channel network can be obtained using a distributed flow routing model. This type of model is based on partial differential equations, the Saint Venant equations (de Saint Venant, 1871) for one-dimensional flow that allow flow rate and water level to be computed as functions of space and time, rather than time alone as in the lumped hydrologic routing models.

The computations of flood water levels are needed because these levels delineate the floodplain and determine the height of structures such as bridges and culverts; the computation of flood flow rate is also important; first, because the flow rate determines the water level, and second, because the design of any flood storage structure such as a detention pond or reservoir requires an estimate of its inflow hydrograph. The alternative to using a distributed flow routing model is to use a lumped model to calculate the flow rate at the desired location, than compute the water level by assuming steady non-uniform flow along the channel at the site. The advantage of a distributed flow routing model over this alternative is that the distributed model computes the flow rate and water simultaneously instead of separately, so that the model more closely approximates the actual unsteady non-uniform nature of flow propagation in the channel.
Distributed flow routing models can be used to describe the transformation of storm rainfall into runoff over a watershed to produce a flow hydrograph for the watershed outlet, and then to take this hydrograph as input at the upstream end of a river or pipe system and route it to the downstream end. Distributed models can also be used for routing low flows, such as irrigation water deliveries through a canal or river system. The flow process in either of these applications varies in all three (3) space dimensions; for example, the velocity in a river varies along the river, across it, and also from the water surface to the river bed. However, for many practical applications, the spatial variation in velocity across the channel and with respect to depth can be ignored, so that the flow process can be approximated in only one direction of flow. The Saint Venant equations, first developed by Barre de Saint-Venant in 1871, describe the one-dimensional unsteady open channel flow.

The Saint Venant Equations are partial differential equations that must be solved using numerical methods. Methods for solving partial differential equations may be classified as direct numerical methods and characteristic methods. In direct methods, finite difference equations are formulated from the original partial differential equations for continuity and momentum. Solutions for the flow rate and water surface elevation are obtained for incremental times and distances along the stream or river. In characteristic methods, the partial differential equations are first transformed to a characteristic form, and the characteristic equations are solved analytically.

In numerical methods for solving partial differential equations, the calculations are performed on a grid placed over the x-t plane. The x-t grid is a network of points defined by taking distance increments of length $\Delta x$ and time increments of duration $\Delta t$. The distance points are denoted by index $i$ and the time points by index $j$. A time line is a line parallel to the x-axis through all the distance points at a given value of time. Numerical schemes transform the governing partial differential equations into a set of algebraic finite-difference equations, which may be linear or nonlinear.

The finite-difference equations represent the spatial and temporal derivatives in terms of the unknown variables on the current time line, $j+1$, and the preceding time line, $j$, where all the values are known from previous computations. The solution of the Saint Venant Equations
advances from one time line to the next. A finite-difference method may employ either an explicit scheme or an implicit scheme for the solution. The main difference between the two is that in the explicit method, the unknown values are solved sequentially along a time line from one distance point to the next, while in the implicit method the unknown values on a given time line are all determined simultaneously. The explicit method is a simpler method however it requires small values of $\Delta x$ and $\Delta t$, to maintain computational stability and convergence of the numerical procedure. Convergence means that as $\Delta x$ and $\Delta t$ approach zero, the results of the finite-difference technique approach the true solution. Stability means that errors at any stage of the computation are not amplified but are attenuated as the computation progresses. The explicit method is convenient because results are given at the grid points, and it can treat slightly varying channel geometry from section to section, but it is less efficient than the implicit method. The implicit method is mathematically more complicated; although it is more stable for large computational steps with little loss of accuracy and hence works much faster than the explicit method.

Methods have also been developed to provide users with simple tools that are robust yet efficient including the level-pool, Muskingum, kinematic wave, diffusion wave and Muskingum-Cunge (Maidment, 1993). These simple and efficient methods enable one to gain insight into the main features of flood propagation in river channels, but at the same time, avoid the numerical complexity of dynamic routing models. Van Der Linden and Woo (2003) pointed that complex models require more input information than can be afforded. A suitable level of model complexity must be sought so that the model matches both the availability of data and the spatial and temporal scale at which the major hydrological processes occur. However, in many cases the hydrologic models fail to perform. Blackburn and Hicks (2002) suggested that the use of a hydraulic model has the potential for modelling more dynamic flood events such as dam breaks and ice jam releases, which cannot be handled by traditional hydrologic modelling approaches. Practical applications of hydrodynamic mathematical modelling in solving combined sewer outflow problems under surcharges with real-time control have been presented by Park and Johnson (1998) and Duchesne et al. (2001). Hicks et al. (1997) applied and solved the one-dimensional equations for channels of variable widths, while Hicks (1996) successfully
performed a hydraulic flood routing exercise for the Peace River in Canada with minimal channel data.

2.4 Watershed Processes and Scale Issues

The complexity of a watershed is influenced by the temporal and spatial nature of the surface and subsurface domains of the watershed flow pathways. The overland surface flow, channel flow and the subsurface and groundwater flows represent the different flow pathways within the watershed. These distinct flow pathways experience entirely different time and space scales (Aral and Gunduz, 2003; and Therrien et al., 2012). These differences have a direct impact on the numerical discretization of these sub-processes and on the overall compatibility with each other (Gunduz, 2004).

Significant time scale dissimilarities exist between the overland surface flow and the subsurface and groundwater flows. For example, the overland surface flow and open channel flow is a much faster pathway requiring computational time steps in the order of minutes or seconds, the subsurface flow and groundwater flow are much slower requiring time steps in the order of days or even months. Differences also exist between the open channel flow and overland surface flow, where, the open channel flow tends to be faster than the overland surface flow. These differences often create problems in the numerical solution procedure and are further exacerbated when they are solved in an integrated fashion (Aral and Gunduz, 2003; and Therrien et al., 2012). The dissimilarities in the temporal scales of the different pathways are even more evident in the values of their time parameters. For example, the values of the surface storage recession constant \( K_{surf} \) and \( T_c \), are used to compute the overland surface flow into the channel. The subsurface flow recession constant \( K_{sub} \) and the groundwater flow recession constant \( K_{gw} \) are used to determine the subsurface flow and the shallow groundwater flow into the channel. The values of \( K_{surf} \) and \( T_c \) are several orders of magnitude smaller than the values of \( K_{sub} \) and \( K_{gw} \). Similarly, differences exist between the overland flow travel time and the channel flow travel time, where the overland flow travel time is several orders of magnitude greater than the channel flow travel time. Furthermore, issues arise in the spatial discretization of the various domains. For example,
channel flow pathways are several orders of magnitude smaller than their surface flow counterparts (Aral and Gunduz, 2003).

In addition, to the varying scale requirements of the sub-processes, there is also the issue of selecting a computation time step ($\Delta t$) that is representative of all watershed pathways. The variability in temporal scale between the different watershed processes must be accounted for in the implementation and application of watershed models. This leads to the problem of assigning a time frame for the overall analysis of the processes. For long-term simulations the existence of the more dynamic processes as opposed to the more static and the time step in either becomes an issue. For example, the overland flow process exists over a very short time frame in comparison to other processes, such as open channel flow or subsurface and groundwater flow. In addition, the spatial extent of the overland flow process in comparison to other pathways is bounded. For example, it is very difficult to discern the spatial extent of the overland flow pathway for a localized event. Therefore, the long-term simulation of a large-scale watershed presents numerical difficulties when overland flow is included in the watershed model (Aral and Gunduz, 2003; and Therrien et al., 2012). As a result, a variable time step may be necessary to account for the differences in time scales between the different watershed processes.

### 2.5 Summary

Watershed models are defined by their processes and scale. Some processes are described by differential equations based on hydrologic and hydraulic laws, and other processes are expressed by empirical equations. Two major modelling components, that affect the estimation of the peak discharge and the peak water level are the time parameters, which include, the time of concentration ($T_c$) and the recession constant ($K$); and the channel routing component of a watershed model. Recent advances in computer hardware and software, including increased speed and storage and GIS/spatial analysis software, have allowed large-scale simulation of watershed processes to become feasible. Available models with these capabilities are generally limited by both spatial and temporal scales. This is attributed to the uncertainty inherent in both the data and model parameter assignments. Users must therefore, be aware of the variability in spatial and temporal
scales of the different watershed processes when implementing and applying these watershed simulation models.
Chapter 3 – Scope and Objectives

3.1 Scope

The research work presented within this document will further enhance Canada’s scientific capacity for long-term flood control and emergency response planning. Floods in Ontario can result from snowmelt, spring rainfall storms, summer thunderstorms, tropical storms, or ice jams causing damages in the order of millions of dollars. Over the last three decades enormous urban sprawls have occurred in Ontario. Impervious surfaces decrease the amount of water that infiltrates into the ground, increase the volumes of stormwater runoff and increase the frequency of flood events. If planning of urban development is done carefully, losses due to floods can be mitigated to a great extent. New policies can be brought into effect to balance the urban and industrial development as a measure to keep the direct runoff under control and hence mitigate the chances of occurrences of floods during high intensity storms.

The time of concentration \((T_c)\) and the recession constant \((K)\) are two widely used parameters for estimating the peak discharge and overland flow hydrograph in hydrologic designs and analyses. Bondelid et al. (1982) indicated that as much as 75% of the total error in estimates of peak discharge could result from errors in the \(T_c\) estimation. Current methods for estimating the time of concentration \((T_c)\) and recession constant \((K)\) are dependent on empirical equations, which were developed using hydro-meteorological data from the United States and lumped basin parameter values such as slope, flow length and drainage area. These empirical relationships do not accurately reflect the hydro-meteorological conditions nor do they allow for computations to be made using the actual physical parameters of the basins. Furthermore, they do not allow for visual or manual inspection of the accuracy of the variables used in the equations (Green and Nelson, 2002). Therefore, equations are needed to estimate time of concentration and recession constant for a given basin that are regionally based and are representative of the hydro-meteorological conditions for that region and are dependent on the physical parameters of the basin.
Flow routing is a procedure to determine the time and magnitude of flow at a point on a watercourse from known or assumed hydrographs at one or more points upstream. When applied to flows in irregular shaped channels, for example, rivers with inundated floodplain zones, the exercise becomes very complex. If channel sections are treated as a single composite section, discharges are generally underestimated. If the channel and floodplain are treated separately, then system discharges are generally over-estimated (Wormleaton and Hadjipanos, 1982). Successful applications of dynamic flood routing models are dependent not only on proper initial and boundary conditions being established, but may also require certain modifications to the original data sets (Kouwen, 1984).

In practice, flood routing is rarely achieved by directly solving the complete Saint-Venant equations (de Saint Venant, 1871). Methods have been developed to provide users with simple tools that are robust yet efficient. These simple and efficient methods enable one to gain insight into the main features of flood propagation in river channels to avoid the numerical complexity of dynamic routing models. However, such simplified methods are limited in their application. Therefore, a more computationally efficient routing model is required that solves the complete Saint-Venant equations and also considers other sub-processes within the channel routing subroutine such as transmission losses, evaporation losses and bank storage.

3.2 Objectives

The overall objective of the research was to test the hypothesis that creating an Ontario–based method for estimation of model time constants, and incorporating an improved flood routing subroutine would produce a more accurate, versatile and computationally-efficient watershed model applicable to gauged and ungauged watersheds in Ontario. The sub-objectives are to (i) develop a methodology for predicting the time of concentration \((T_c)\) and recession \((K)\) for estimating the lateral inflow to a reach by accounting for the hydro-meteorological conditions and physical characteristics of the sub-basin; (ii) develop a methodology to account for the lateral subsurface and ground-water inflows into the channels; and to account for the lateral seepage outflows into the banks from the channels; and (iii) implementation of the model to a
watershed that considers the available hydro-meteorological data, and spatial and temporal scales at which the major hydrologic and hydraulic processes occur.

Through the use of a case study this research will further evaluate the performance of a hydrologic – hydraulic model for the purpose of estimating peak flows and water levels. This research will follow through a simulated process in a real environment to quantify the level of efficiency and accuracy that may be achieved when using a coupled hydrologic – hydraulic model. Given the results from these simulations, it will be possible to provide recommendations, methodologies and new tools to better estimate peak discharge values, peak water levels, time-to-peak discharge values, and time-to-peak water levels for the purposes of flood management.

A product of this research will be the development of a watershed-scale water quantity simulation model that can be utilized for flood forecasting and flood risk management. The watershed scale simulation model will be developed by evaluating and modifying existing hydrologic and hydraulic modelling techniques. The hydrologic model will be developed to simulate the rainfall – runoff processes and the hydraulic model will be used to route the flood flows along the channels. The tools developed as part of this study will be used to investigate the risk due to flooding. The main elements of the simulation model will include data collection, analysis and dissemination of results. In addition to GIS data that characterizes the watershed, on-line flow monitoring, and precipitation data will be collected and analyzed as part of the research project. The benefits achieved through this work include:

(i) The ability to produce output describing both water level and discharge hydrographs at and between points of interest along a watershed;
(ii) The ability to simultaneously model and evaluate dynamic problems, such as dam breaches and ice jam releases, which cannot be handled by traditional hydrologic or hydraulic modelling approaches alone;
(iii) The ability to model watersheds with floodplains affected by floodplain storage, flow regulation and flood protection measures with a degree of accuracy;
(iv) The ability to provide detailed representation of the physical watershed, including environmental data, channel morphology and surface topography, through the use of geographical information systems (GIS) and digital elevation models (DEMs); and

(v) The ability to evaluate the impacts of natural and man-made control features on water surface profiles upstream of major/minor crossings adjacent to residential communities.
Chapter 4 – Methodology

4.1 Modelling Approach

Watershed simulation models require four types of input: (1) climate data (rainfall, snowfall, and other variables needed to predict snowmelt and evaporation); (2) transfer and storage processes (for example infiltration and/or interception); (3) physical watershed characteristics (watershed areas, slopes, channel locations and dimensions); and (4) initial state variables (such as soil-water storage and groundwater storage conditions). According to these inputs, model output includes runoff hydrographs at the watershed outlet, water level hyetographs, maximum peak discharges, maximum peak water levels and total runoff volumes.

The relationship between the precipitation and resulting runoff is quite complex and is influenced by many factors related to the catchment and climate. However, despite these complexities watershed simulation models do serve a purpose and offer several advantages over other methods for predicting flows and water levels.

The most obvious advantage of the watershed simulation model is that it can be used at locations at which there is little or no streamflow data. However, the calibration of the model depends on the availability of some data within the watershed or an adjacent watershed with similar physical characteristics. A second advantage of watershed models compared to computational methods that only determine peak flow rates is their ability to generate entire hydrographs or hyetographs, which may be critical in applications for which flood volumes are important. Finally, with watershed simulation models it is possible to account explicitly for changes in land use.

The problems associated with the scales of sub-processes are probably the most significant; and it is possible to couple all processes if cost and data availability do not impose significant limitations (Gunduz, 2004; and Therrien et al., 2012). However, for large-scale applications such as catchment modelling, the small-scale requirements of overland flow and open-channel flow domains exhibit severe limitations on efforts of fully integrating the system (Aral and Gunduz, 2003). A hybrid approach is more suitable in which distributed and lumped parameter models are
essentially linked and blended to obtain a semi-distributed watershed model (Gunduz, 2004; and Therrien et al., 2012). In such models, the overland flow, subsurface flow and groundwater flow are replaced with their lumped parameter semi-empirical counterparts in an effort to simplify the overall analysis. When issues such as computational limitations, proper mathematical formulation of physical processes, and data requirements are addressed accurately and sufficiently, these systems would ultimately be included in the analysis.

Based on the above discussion, a hybrid watershed modelling system approach is adopted, where the distributed parameter model of open channel flow is integrated with the semi-empirical quasi-lumped parameter models of overland flow, subsurface flow and shallow groundwater flow to yield a computationally efficient hybrid watershed model. In this hybrid model, the semi-empirical lumped parameter modelling of overland flow, subsurface flow and groundwater flow are provided with the watershed simulation model GFLOOD (Appendix A). GFLOOD is an acronym that stands for Guelph Flood Forecasting Model, a river basin or watershed scale flow prediction model. GFLOOD is based on the platform developed for the Greater Toronto Area (GTA) Conservation Authorities “Flood Forecasting and Warning Spreadsheet Model, (FFOR)” developed by Environmental Water Resources Group Limited (2000). For channel routing the GFLOOD model uses a one-dimensional dynamic wave channel routing sub-model based on the complete Saint-Venant Equations. This gives the model the added advantage of being able to deal with a variety of dynamic flooding problems including ice jam releases and dam breaches.

4.1.1 Overview of the Guelph Flood Forecasting Model (GFLOOD)

GFLOOD is an acronym that stands for Guelph Flood Forecasting Model, a river basin or watershed scale flow prediction model. GFLOOD was developed to determine the impact of flooding on communities in complex watersheds with varying climate, soils, topography and land use conditions over long and short periods of time. GFLOOD is capable of simulating flows on watersheds ranging in size from small (less than 100 km$^2$) to large (greater than 1000 km$^2$).

To satisfy these objectives, the GFLOOD model is physically based. Rather than relying on regression equations to describe the relationship between input and output variables, the
GFLOOD model requires specific information about weather, soils, vegetation and land use occurring within a watershed as input to model the processes associated with water movement. The benefits of this approach are watersheds with no stream gauge information can be modelled; and the impact of alternative input data on water quantity, such as changes in climate, vegetation and land use can be evaluated.

The GFLOOD model can simulate floods using minimal data commonly available through public and open sources. The model is also computationally efficient and is capable of simulating a variety dynamic flood scenarios. Furthermore, the model can be used to study the long-term impacts of flooding on a community. For example, many of the problems addressed by users in flood studies involve estimating the probability of exceedance of an extreme event and its impact on the community. This can either be undertaken by simulating a statistically significant design event or running a long-term continuous simulation and generating a hydrologic time series of data. The latter requires meteorological data set spanning over several decades.

GFLOOD is based on the platform developed for the Greater Toronto Area (GTA) Conservation Authorities “Flood Forecasting and Warning Spreadsheet Model, (FFOR)” developed by Environmental Water Resources Group Limited (2000). In the late 1990s, Conservation Authorities identified a need for a model to predict floods along their respective watersheds. The original spreadsheet model contained a snowmelt subroutine, along with the Antecedent Precipitation Index Method for calculating runoff volumes, and the isochronal method for hydrograph convolution and the determination of flood flows. Despite its relative ease of use, FFOR was limited in its application. For example, the FFOR model was limited to modelling basins with time of concentrations less than 24 hours. In addition, the application of the antecedent precipitation index (API) method to urban watersheds for estimating runoff volumes resulted in poor model performance and results. Furthermore, FFOR utilized no channel or reservoir routing limiting its prediction of flood flows to the basin outlet.

As a result, modifications were made to the original model to extend its capabilities, these included: a) a meteorological time series sub-model for varying climatic conditions; b) a
snowmelt sub-model; c) the Green and Ampt two layer soil-water accounting model for calculating runoff volumes; d) a hydrograph transformation procedure based on the linear reservoir method; and e) channel routing subroutine that uses a simplified version of the unsteady flow routing model based on the complete Saint Venant Equations.

GFLOOD allows a number of different processes to be simulated within a watershed. For modelling purposes the watershed may be subdivided into a series of subwatersheds, sub-basins or sub-catchments. Modelling the watershed as a series of sub-basins is beneficial particularly when different areas of the watershed are dominated by land uses and soils dissimilar enough to impact hydrology. The sub basins are further sub divided into a series of elevation bands, a maximum of 10 elevation bands per sub basin, thereby taking into consideration the orographic variations in precipitation and temperature across the sub basin.

Information for each sub basin is organized into the following categories: climate; hydrologic response units (HRUs); subsurface flow and/or groundwater; and the main channel or reach draining the sub basin. Hydrologic response units are lumped land areas within the sub basin that are comprised of land cover, soils and stormwater management strategies.

Similar to other watershed models, in GFLOOD, the water balance is the driving force behind everything that happens within a watershed. The hydrologic cycle as simulated by the model must conform to what is happening within the watershed. The simulation of a hydrograph within a watershed can be separated into two major components, the hydrologic phase and the hydraulic phase. The hydrologic phase includes all of the water that has fallen from the sky onto the ground and controls the amount of water to the main channel of the sub basin. The second phase, the hydraulic phase, controls the route water follows to the sub basin outlet (Figure 4.1).

4.1.2 Time Parameters

4.1.2.1 Introduction

In this hybrid modelling approach the equations for estimating the time of concentration ($T_c$) and the recession constants ($K$) are derived based on the results of a study to estimate time
parameters associated with unit hydrographs of the Credit River Watershed for the purposes of modelling streamflows along the Credit River. The study was undertaken for 9 basins of the Credit River Watershed. The values of the surface storage recession constant ($K_{surf}$) and the time of concentration ($T_c$) are used to compute the overland surface flow into the channel. In addition, the subsurface flow recession constant ($K_{subs}$) and the groundwater flow recession constant ($K_{gw}$) are used to determine the subsurface flow and the shallow groundwater flow into the channel. The time of concentration ($T_c$) is estimated using parameters derived from digital elevation models and geographic information system software, and an empirical equation derived from multiple-linear regression.

The physical factors thought to have a major effect on the $T_c$, are defined here as the time interval from the centre of gravity of the runoff to the point of inflection on the hydrograph. Using 9 gauging stations along the Credit River, the basins parameters have been combined into an equation that defines $T_c$. The records of 14 events from the 9 gauging stations along the Credit River have been analysed and the actual $T_c$ for the 9 basins was compared with those indicated by the equation. Estimates of $T_c$ based on the derived equation are compared and analysed against estimated values of $T_c$ using Williams Equation (1922), Kirpich (1940), Johnstone–Cross (1949), and Haktanir and Sezen (1990). These equations were selected to study since they use only a few readily available watershed parameters for engineering practice. In addition, these equations were also developed on watersheds where channel flow dominates as is the case for the Credit River watershed. The empirical equation for the time of concentration is validated by the comparing observed and estimated values of $T_c$ using data derived by Kennedy and Watt (1967) from basins across Southern Ontario.

The equations for estimating the recession constant ($K$) were derived using multiple-linear regression. The physical factors thought to have a major effect on the recession constant ($K$), are defined here as the time interval from the centre of gravity of the runoff to a point where runoff ceases to predominate. Using 9 gauging stations along the Credit River, through regionalization the parameters have been combined into a series of equations that define the surface storage recession constant ($K_{surf}$), subsurface (inter) flow recession constant ($K_{subs}$) and groundwater flow recession constant ($K_{gw}$). The records of a number of isolated storms from 9 gauging stations
along the Credit River have been analysed and the actual recession constants for the 9 basins compared with those indicated by the equations. In addition, estimates of the surface storage recession constant ($K_{surf}$) based on the derived equations are compared and analysed against estimated values for $K$ using Williams Equation (1968) and Williams and Haan (1973).
4.1.2.2 Selection of a Test Watershed

The Credit River Watershed is a mid-sized watershed situated in Southern Ontario, Canada (total drainage area is approximately 1000 km²). This watershed was selected to test methods to estimate time parameters because there is a very extensive data set available for this watershed on meteorological and hydrological measurements, the physiography of the watershed is well defined, and previous studies of time properties have been completed. The Credit River watershed contains 22 subwatersheds with over 1500 km of streams and creeks. Basin characteristics related to basin slope and size are often used in empirical equations to estimate $T_c$ and $K$. Physical characteristics used by previous investigators in the estimation of the recession constant $K$ include main channel length, main channel slope and elongation ratio (Williams, 1968 and Williams and Hann, 1973). In order to apply empirical equations to estimate $T_c$ and $K$, several basin characteristics were developed for 9 basins along the Credit River. These study basins are associated with 9 Water Survey of Canada (WSC) streamflow gauging stations. In addition, in order to derive the necessary regression equations for estimating the time of concentration, surface storage, sub-surface and groundwater flow recession constants, the basin characteristics for an additional 7 gauging stations operated by Credit Valley Conservation (CVC) were developed. Figure 4.2 illustrates the location of the, the major flow paths, the 9 WSC streamflow gauging stations (indicated by a red target) and the 7 CVC streamflow gauging stations (indicated by a blue diamond) along the Credit River Watershed.

Basin parameters were developed using GIS software, ESRI, Arcview 3.3 with spatial analyst extension and the Ontario Flow Assessment Technique (OFAT) data model (MNR 2001; Chang et al, 2002). The use of GIS can greatly improve the speed and accuracy of how these parameters are estimated and travel times computed (Fang et al, 2008). Basins were delineated using a 10-m MNR digital elevation model (DEM) before basin parameters were abstracted. A similar approach was undertaken by Noto and Loggia (2007) and Cleveland et al, 2008, who used a digital elevation, model (DEM) to estimate the flow paths and travel times within a basin. The locations of the gauging stations were treated as the outlet or pour point of the basin. Twenty one characteristics for each individual basin were determined; however, only total drainage area, main channel length, main channel slope, basin width, maximum flow distance and elongation
ratio were used for application of the regression equation and the empirical equations for estimating the time of concentration \( (T_c) \) and the derivation of the regression equations for estimating the recession constants \( K_{surf}, K_{subs} \) and \( K_{gw} \). Basin width is the ratio of contributing drainage area to the basin length, which is the sum length of a limited number of sequential line segments following the geometric centerline of the watershed from the watershed outlet to the basin divide. Main channel slope is the ratio of the basin divide elevation minus the outlet elevation to the main channel length (Asquith and Slade 1997). The elongation ratio of a basin represents the ratio of the diameter of a circle with the same area of the watershed to the maximum flow length of the watershed (Williams, 1968). The elongation ratio is related to basin shape. Basin shape is typically not used directly in hydrologic design methods; however, parameters that reflect basin shape are used occasionally and have a conceptual basis. Watersheds have an infinite variety of shapes, and the shape supposedly reflects the way that runoff will “build-up” at the outlet. As the elongation ratio approaches 1.00, the watershed is considered to be more circular. As a result, a circular watershed would result in runoff from various parts of the watershed reaching the outlet at the same time. An elliptical watershed having the outlet at one end of the major axis and having the same area as the circular watershed would cause the runoff to be spread out over time, thus producing a smaller flood peak than that of the circular watershed.

The physical parameters including location (latitude and longitude), basin area (km\(^2\)), main channel length (km), main channel slope (m/km), maximum flow distance (km), and elongation ratio, for the 9 basins (Figure 4.2) studied are listed in Table 4.1. In accordance with Figure 4.2, the 7 streamflow gauging stations operated by CVC are located between the 9 WSC streamflow gauging stations and therefore, their basin parameters fall within the ranges listed in Table 4.1.

Eight out of the nine basins lie in the upper two-thirds of the Credit River watershed, and are predominantly rural (cropland and pasture land, with pockets of forest) with developed areas covering less than 10\% of the total basin area. The exception is the Erindale Streamflow Gauge Station (02HB002) which is located in the lower third of the watershed (Figure 4.2). Although the developed area for this basin is 21\% of the total drainage area, most of this development is concentrated in the lower third of the basin.
Streamflow Records (1998 to 2005) of 15-minute streamflow data from the selected streamflow gauge stations were obtained from Water Survey of Canada and Credit Valley Conservation. For two of the gauge stations within the Credit River watershed, Credit River at Erindale (02HB002) and Credit River Alton Branch above Alton (02HB019), 1-hour flow data were taken for the period 1984 to 1991; since, both stream gauge stations were discontinued in the early nineties. The time of concentration at each of the 9 WSC streamflow gauging stations was determined using the 15-minute and 1-hour flow records. The time of concentration was estimated from the hydrographs by measuring the time between the end of the precipitation event and the inflection point on the hydrograph. Hourly precipitation records from nearby Meteorological Service of Canada Climate stations were plotted against the 15-minute and 1-hour flow records and incorporated into the analysis. Fourteen (14) storm hydrograph events were analysed for each gauging station. Different events were selected for different seasons throughout the year. Each of the events produced runoff volumes in excess of 10 mm. The Ministry of Natural Resources Technical Guidelines on Floodplain Management (1987 and 2002) recommend a minimum of 25 mm of runoff when selecting events for model calibration and/or model parameter derivation. However, given the difficulty in identifying such extreme events throughout the year, a compromise was reached to select events that generated half-bank to bank-full conditions (i.e. 10 mm or greater).

Plots of average channel discharge versus average channel velocity indicate that the maximum velocity is achieved at top of bank (Appendix C). As a result, the minimum travel time along the channel would occur when flows and water levels are at top of bank. In addition, each of the events selected were uniform across the watershed and had single peak hydrographs with no runoff excess after the peak. The number of events used for the different basins including event characteristics for the hydrographs, such as event occurrence, event duration, peak flow, average flow and runoff depth are listed in Table 4.2. Event characteristics for the rainfall hyetographs, such as storm duration, average intensity, peak intensity and rainfall depth are listed in Table 4.3.

For the Credit River above Alton Branch gauge station (02HB019) and the Erindale gauge station (02HB002); the time of concentration values were abstracted from the simulated hydrographs of the calibrated Guelph All-Weather Sequential Events Runoff (GAWSER) model.
for the Credit River watershed, which was developed as part of the Credit River Adaptive Management Strategy (Credit Valley Conservation, 2001). The model was simulated from October 1st, 1997 to December 31st, 2005. The same events that were used to determine the observed mean time of concentration values from the hydrometric data for the seven operational gauge stations were also used to determine the time of concentration values for the Credit River above Alton Branch gauge station and the Credit River at Erindale gauge station using the simulated hydrograph events.

Using the available hourly hydrometric data (1984-1991), for the Credit River above Alton Branch gauge station and Credit River at Erindale gauge station, the observed mean time of concentration values were determined for both stations. In total 4 events were used to determine the observed time of concentration values for the Alton branch gauge and Erindale gauge stations. Table 4.4 lists the event characteristics for both the observed hydrographs and hyetographs including event occurrence, event duration, rainfall depth, snowfall depth, runoff volume and peak flow. The observed mean time of concentration values based on the hydrometric data for the Erindale and Alton Branch stations were compared against the mean time of concentration values that were abstracted from the simulated hydrograph events for both stations. The differences between the two values were less than 5%. The observed mean time of concentration and standard deviation for each of the 9 Water Survey of Canada gauge stations are listed in Table 4.5.

4.1.2.3 Developing Prediction Equations

An equation for estimating the time of concentration was derived using forward stepwise multiple-linear regression. The derivation of the equation is described in detail in Chapter 5, Sections 5.1 and 5.2 of this thesis. The estimated values of the time of concentration for the 9 WSC streamflow gauge stations were compared against the observed values and estimates using the following empirical equations: Williams (1922), Kirpich (1940), Johnstone-Cross (1949) and Haktenir-Sezen (1990). These equations are listed in Table 4.6 for SI units. The following statistical criteria were used to evaluate the performance of the empirical equations: coefficient of determination ($r$); coefficient of efficiency ($E$); modified coefficient of efficiency ($E_1$); root
mean square error (RMSE); systematic root mean square error (RMSE_s); unsystematic root mean square error (RMSE_u); coefficient of residual mass (CRM); and relative bias (RBIAS); and percent difference.

The coefficient of determination is a measure of the association between observed and simulated model outputs (Aitken, 1973), and is defined by the following equation:

\[
\begin{array}{r}
\end{array}
\]

\[
\begin{array}{c}
\end{array}
\]

Where \(O_i\) is the observed output at time step \(i\), \(P_i\) is the predicted output at time step \(i\), \(\bar{O}\) is the mean of the observed outputs and \(\bar{P}\) is the mean of the predicted outputs. The coefficient of determination will always be less than unity. A value of \(r\) close to one indicates good results. Even though this is a good measure of association between observed and simulated values, it does not indicate systematic errors. The coefficient of efficiency of a model is defined as the proportion of the variance of the observed outputs accounted for by the model (Nash and Sutcliffe, 1970).

\[
E = 1 - \frac{\sum_{i=1}^{n}(O_i-P_i)^2}{\sum_{i=1}^{n}(O_i-\bar{O})^2}
\]

The value of this statistic will always be less than unity. If the results from the model are highly correlated with the observed values but biased, then the value of \(E\) will be less than \(r\). The main criticism of the Nash and Sutcliffe criterion is based on the fact that a measure of accuracy should not be dependent on the number of ordinates considered in the analysis. The modified efficiency coefficient (\(E_1\)) is the ratio between the mean square error and the potential error; it was introduced to overcome the insensitivity of \(E\) and \(r\) to differences in the observed and predicted means and variances.

\[
E_1 = 1 - \frac{\sum_{i=1}^{n}|O_i-P_i|}{\sum_{i=1}^{n}|O_i-\bar{O}|}
\]
The root mean square error (RMSE) is a dimensional objective function, which is proposed to overcome the dilemma of the dependency of the objective function on the number of output ordinates. The Root Mean Square Error (RMSE) is defined as (WMO, 1992):

\[ \text{RMSE} = \sqrt{\frac{\sum_{i=1}^{n}(P_i - o_i)^2}{n}} \]  
(4.4)

The Systematic Root Mean Square Error (RMSEs) assess whether the model errors are predictable. The variable \( \hat{P}_i \) is the predicted output from the linear regression of the observed measurements on the simulated outputs of the model.

\[ \text{RMSE}_s = \sqrt{\frac{\sum_{i=1}^{n}(\hat{P}_i - o_i)^2}{n}} \]  
(4.5)

Unsystematic root mean square error (RMSEu) identifies those errors that are not predictable by the model.

\[ \text{RMSE}_u = \sqrt{\frac{\sum_{i=1}^{n}(P_i - \hat{P}_i)^2}{n}} \]  
(4.6)

The coefficient of residual mass (CRM) is a measure of the deviation of the simulated outputs from the observed outputs.

\[ CRM = \frac{\sum_{i=1}^{n} o_i - \sum_{i=1}^{n} P_i}{\sum_{i=1}^{n} o_i} \]  
(4.7)

The \( RMSE, \text{RMSE}_s, \text{RMSE}_u \) and \( CRM \) are valuable indices because they indicate error in the units (or squared units) of the constituent of interest, which aids in analysis of the results. \( RMSE, \text{RMSE}_s, \text{RMSE}_u \) and \( CRM \) values of 0 indicate a perfect fit. Singh et al, (2004) state that \( RMSE \)
values less than half the standard deviation of the measured data may be considered low and is appropriate for model evaluation. The relative bias (\textit{RBIAS}, \%), measures the average tendency of the simulated data to be larger or smaller than their observed counterparts (Gupta et al, 1999). The optimal value of \textit{RBIAS} is 0.0, with low-magnitude values indicating accurate model simulation. Positive values indicate model overestimation bias, and negative values indicate model underestimation bias (Gupta et al, 1999). \textit{RBIAS} is calculated with equation 4.8:

\[
RBIAS(\%) = 100 \times \frac{1}{N} \sum_{i=1}^{N} \frac{P_i - O_i}{O_i}
\]  

(4.8)

Where \textit{RBIAS} is the deviation of data being evaluated, expressed as a percentage.

The regression equation for \textit{T}_c was validated by comparing observed and estimated values of \textit{T}_c using data derived by Kennedy and Watt (1967). Data obtained by Kennedy and Watt (1967) are representative of basins in southern Ontario of intermediate size (60 – 320 km\(^2\)). The physical parameters for the basins analyzed by Kennedy and Watt (1967) including basin name, station ID, main channel length (km), main channel slope (m/km) and elongation ratio are listed in Table 4.7. Kennedy and Watt (1967) derived the observed lag time for each of the basins listed in Table 4.7. \textit{T}_c is computed from lag time based on the NRCS relationship \textit{T}_L=0.6\textit{T}_c (NRCS 1972, 1986). According to Tables 4.1 and 4.7, the sizes of the basins analyzed by Kennedy and Watt (1967) fit within the range of basin sizes used to derive the empirical equation for time of concentration along the Credit River. The regression equation was further evaluated by comparing the estimated \textit{T}_c values with those determined by: Williams (1922), Kirpich (1940), Johnstone-Cross (1949) and Haktenir-Sezen (1990) for the basins analyzed by Kennedy and Watt (1967). Equations 4.1 to 4.8 were used to evaluate the performance of the empirical equations.

The procedure for determining the recession curve for the selected basins was determined by analysis of the recession segments of several storm hydrographs. Fourteen (14) storm hydrograph events were analysed for each basin. Different events were selected for different seasons throughout the year. As with the events selected for estimating the time of concentration, each of the events used to estimate the recession constant produced runoff volumes in excess of
10 mm. The same events used to determine the time of concentration values were also used to
determine the recession constants for the 9 WSC gauge stations.

In addition, each of the events selected had single peak hydrographs with no runoff excess after
the peak. The recession constant, $K$ was determined by plotting the recession curves for each of
the historical events on semi-log graphs. A straight line drawn through the points of the recession
curves permitted the determination of $K$. The recession curve is defined by the linear reservoir
equation (Boussinesq, 1877) in its logarithmic form:

$$\ln(Q(t)) = \ln(Q(t_0)) - K(t - t_0)$$  \hspace{1cm} (4.9)

Where $K$ is the recession constant, and $Q(t_0)$ represents the flow at the inflection point or the time
of concentration. The plotting of the curves revealed that there are three distinct recession
constant values along each of the curves. The surface storage recession constant ($K_{surf}$), the
subsurface flow recession constant ($K_{subs}$) and the groundwater flow recession constant ($K_{gw}$).
Subsurface runoff refers to water that flows within the soil matrix and is pushed to the streams
(i.e. interflow); and groundwater runoff refers to water that is returned to the stream thru the
groundwater flow regime. The subsurface flow and groundwater flow make up the total baseflow
within the stream channel. For each basin there was significant variation in the recession
constant values for both the subsurface flow and groundwater flow. This was primarily due to the
fact that a variety of events were used for different seasons. Because of this high variability, it
was decided to derive relationships for these two additional components. The mean surface
storage, subsurface flow, and groundwater flow recession constant values for each subwatershed
including standard deviation are listed in Table 4.8. The subsurface flow recession constants
were determined by averaging the surface storage recession constants and the groundwater flow
recession constants, due to the difficulty in discerning $K_{subs}$, using the graphical method.
Forward stepwise multiple linear regression was used to derive relationships for the surface
storage recession constant ($K_{surf}$), the subsurface flow recession constant ($K_{subs}$) and the
groundwater flow recession constant ($K_{gw}$). The derivation of these equations is described in
detail in Section 4.2.2.
For the Credit River above Alton branch gauge station and the Credit River at Erindale gauge station, the recession constant values were abstracted from the simulated hydrographs of the GAWSER model for the Credit River watershed. The same events used to determine the time of concentration and recession constant values for the operational gauge stations were also used to determine the recession constant values for the Alton branch and Erindale gauge stations. Using the available hourly hydrometric data (1984-1991), for the Credit River above Alton Branch gauge station and Credit River at Erindale gauge station, the observed mean recession constant values were determined for both stations. The same events used to determine the observed mean time of concentration values for the Alton branch gauge and Erindale gauge stations were also used to determine the observed mean recession constant values. The observed mean recession constant values based on the hydrometric data for the Erindale and Alton Branch stations were compared against the mean recession constant values that were abstracted from the simulated hydrograph events for both stations. The differences between the two values were less than 5%.

The estimated values of the surface storage recession constant \( K_{surf} \) for the 9 WSC streamflow gauge stations were compared against the observed values and estimates using the following empirical equations: Williams (1968) and Williams and Haan (1973). Williams (1968) developed an expression for the surface storage recession constant, equation 4.10.

\[
K = 0.002044 \cdot L^{0.52} \cdot S^{-1.263} \cdot E^{1.78} \quad (4.10)
\]

Where \( K \) is the surface storage recession constant (hours), \( L \) is the length of the main stem along the channel from the most distant point on the watershed to the outlet (mi), \( S \) is the slope of the main stem of the channel from the most distant point on the watershed to the outlet (ft/ft), and \( E \) is the elongation ratio of the basin. Williams and Hann (1973) developed expressions for estimating \( K \), for rural basins, Equations 4.11a and 4.11b.

\[
K = 27.0 \cdot A^{0.231} \cdot S^{-0.777} \cdot \left(\frac{L}{W}\right)^{0.124} \quad S < 2\% \quad (4.11a)
\]

\[
K = 16.1 \cdot A^{0.24} \cdot S^{-0.84} \quad S > 2\% \quad (4.11b)
\]
Where $A$ is the drainage area ($\text{mi}^2$), $S$ is the slope of the main stem of the channel from the most distant point on the watershed to the outlet (ft/mi), $L$ is the length of the main stem along the channel from the most distant point on the watershed to the outlet (mi) and $W$ is the width of the basin in (mi), which is determined by dividing the drainage area of the basin $A$, by the length $L$. Equations 4.10, 4.11a and 4.11b have been used in numerous watershed and subwatershed studies across Southern Ontario and specifically within the Credit River watershed (Credit Valley Conservation, 2007). As a result, equations 4.1 to 4.8 were used to evaluate the performance of the regression equation for $K_{surf}$ and compare the results to equations 4.10, 4.11a and 4.11b.

### 4.1.3 Open-Channel Flow Routing

To evaluate opportunities to improve streamflow routing in the selected model the capabilities of the simplified dynamic model for routing flows in open channels are examined. A new form of the momentum equation was derived in order to solve the Saint-Venant’s equations for flood routing in trapezoidal and triangular open channels with varying widths. The model is derived from the general dynamic equation by assuming the derivative $\frac{\partial S_f}{\partial x}$ to be negligible with respect to other terms of the equation. The proposed simplified dynamic model can be called a dynamic cascade, which is defined as a sequence of discrete channel segments in that the dynamic wave equations are used to describe the flood routing (Keskin, 1997). In the solution, a simple numerical algorithm of a cascade, similar to a kinematic one (Keskin, 1997; and Kibler and Woolhiser, 1970) is used. This methodology for deriving the momentum equation builds upon the work first proposed by Keskin (1997).

In this formulation, the momentum equation transforms to a partial differential equation which has two parameters related to cross-sectional area and discharge of the channel. The simplified dynamic model was further modified to account for transmission losses, evaporation losses and bank storage along the length of the channel. The simplified dynamic model has been solved by using an explicit finite difference scheme in which the operator is in the form of a tiling diagram (Keskin, 1997). In the computation procedure after computing the discharge from the momentum equation, the cross-sectional area will be obtained from the continuity equation for given point
for the channel. The results are compared with the solution of the dynamic wave model, and the diffusion wave model. The mathematical formulation of the simplified dynamic model is described in detail in Section 4.3. The performance of the simplified dynamic model was tested and evaluated along a reach and event(s) selected from the literature (Keskin, 1997; Akan and Yen, 1981). The characteristics of the channel, floodplain and event are given as follows.

The length of the channel is \( L = 2000 \) m, the bottom slope of the channel \( S_0 = 0.0005 \), and its Manning’s roughness coefficient \( n = 0.0138 \) for the channel. The depth of flow at bankfull for the channel, \( y_{bnk} = 1 \) m. The bottom slope of the floodplain and Manning’s “n” value were arbitrarily selected since, these parameters were not provided in the original test reach from the literature. In turn, the bottom slope of the floodplain was set to be equivalent to the channel, and its Manning’s roughness coefficient was set to \( n = 0.08 \) for the floodplain. A trapezoidal channel and a triangular shaped channel are used in the evaluation; the bottom width of the trapezoidal channel \( B_w = 5 \) m and the inverse side slopes (z) of the trapezoid and triangle have been varied between one (1) and four (4). The shape of the floodplain is trapezoidal, the bottom width of the floodplain is five (5) times the bankfull width of the channel and the inverse side slopes of the left (\( z_lf \)) and right (\( z_{rf} \)) are both equal to 4. For the triangular inflow hydrograph, the hydrograph is selected as \( Q(x, t) = Q(0, 0) = Q_o = 3 \) m\(^3\)/s, \( Q(0, 10) = Q_p = 12 \) m\(^3\)/s and \( Q(0, 20) = 3 \) m\(^3\)/s. For a trapezoidal inflow hydrograph, the hydrograph is selected as \( Q(0, 0) = 3 \) m\(^3\)/s, \( Q(0, 10) = 12 \) m\(^3\)/s, \( Q(0, 15) = 12 \) m\(^3\)/s and \( Q(0, 20) = 3 \) m\(^3\)/s.

A comparison was made between the simplified dynamic model and the more complex dynamic wave model of the FLDWAV model (Fread and Lewis, 1998). The comparison was made using the same reach and inflow hydrographs selected from the literature as used by Akan and Yen (1981) and Keskin (1997). The diffusion wave approximation to the Saint Venant equations is also used for channel routing applications due to its simplicity and ease of solution. The diffusion wave sub-routine within the FLDWAV model was used to simulate the outflow hydrographs for the diffusion wave approximation using the same reach and inflow hydrographs from the literature. Comparisons were made between the outflow hydrographs for the diffusion wave approximation and simplified dynamic model. The FLDWAV model solves the Saint-Venant equations using a four point implicit solution.
The performance of the simplified dynamic model on computed hydrographs is further evaluated for two reaches along the Credit River. The simplified dynamic model’s performance is compared against two other distributed routing methods, the diffusion wave and the more complex dynamic wave routing models. The National Weather Service’s FLDWAV model, version 2-0-0 (Fread and Lewis, 1998) is used to route flows along both reaches of the Credit River using the dynamic wave and diffusion wave routing methods. For the dynamic wave method the FLDWAV model solves the complete Saint-Venant equations using a weighted four-point implicit solution. The model solves the continuity and momentum equations for each \( \Delta x \) reach along the channel, along with equations describing the upstream and downstream boundary conditions simultaneously for each time step. The system of non-linear algebraic equations is solved using an iterative technique, such as the Newton-Raphson method. The flow may be either subcritical or supercritical or a combination of each varying in space and time from one to the other; fluid properties may obey either the principles of Newtonian flow or non-Newtonian flow. The hydrograph to be routed may be user-specified as an input time series, or it can be developed by the model via user-specified breach parameters.

A user-specified option is provided within the FLDWAV model to utilize a simplified distributed routing model, known as the diffusion wave (zero-inertia) model. It is based on the continuity equation along with an approximation of the momentum equation that omits the first two terms, the inertial terms, local acceleration and convective acceleration. The FLDWAV model solves the governing equations using a four-point implicit finite-difference solution, which is applied to the continuity and momentum equations directly. The diffusion simplified routing model considers backwater effects; however, its accuracy is also deficient for very fast rising hydrographs, such as those resulting from dam failures, hurricane storm surges, or rapid reservoir releases, which propagate through mild to flat sloping waterways with medium to small Manning’s “n” values. The range of application (with expected modeling errors less than 5 percent) for the diffusion wave models are given by Fread (1983a and 1992), Ponce et al. (1978) and Sinha et al. (1995).
The first reach extends from Melville to Cataract and is approximately 10 km in length with an average channel bed slope of 0.0025 m/m, and a Manning’s ‘n’ value of 0.051 for the channel section was selected. This reach is characterized by wide floodplains and wetland features throughout. Dominant flows come from Subwatershed 19 (headwaters of the Credit River), shallow groundwater discharges and Shaw’s Creek (Subwatershed 17). Modest amounts of flow are contributed from riparian floodplains when shallow groundwater discharges and surface flows appear from Caledon Creek (Subwatershed 16). Flow along this reach increases downstream as the bedrock valley and surface valley deepens towards Cataract.

The second reach, is located along the mid-potion of the watershed. It extends from Cheltenham to Norval and is approximately 23 km in length with an average channel bed slope of 0.0023 m/m, and a Manning’s ‘n’ value of 0.040 for the channel section. This portion of the watershed is dominated by both the Niagara Escarpment and the Oak Ridges Moraine, two prominent physiographic features within the Credit River watershed. This reach along the mid-potion of the Credit River is characterized by exposed rocks and steep cliffs due to the presence of the Niagara Escarpment. Resulting ‘quick’ discharge releases to the main channel through subsurface and shallow groundwater flow. However, the presence of trees and woodlots along the edge of the escarpment has slowed this process down significantly (Credit Valley Conservation, 1990). The bed of the main channel is made up of both sand and gravel; as a result, a portion of the flow is lost to the deep groundwater aquifer along this reach.

Streamflow data for routing the flows along the two reaches in the Credit River was obtained from Water Survey of Canada (WSC). Flow data in 15-minute interval was obtained for the following stations: Boston Mills (02HB018), Silver Creek (02HB008), Norval (02HB025), Melville (02HB013) and Cataract (02HB001) for the period January 1, 1998 to December 31, 2005. For the Alton Branch station (02HB019) flow data was obtained in 1-hour increments for the period June 1, 1983 to December 31, 1991. Though the Alton Branch gauge was discontinued in the early 90’s however, the historical data still remains on record. A typical water year in Southern Ontario is between November 1st and October 31st of the following year. Water levels along the reaches of the Credit River are typically at normal levels during the mid to late fall period; with average antecedent conditions.
Figure 0-2 Credit River Watershed, flow paths, subwatersheds, Water Survey of Canada stream gauge stations and Credit Valley Conservation stream gauge stations
Table 0-1 Locations and physical parameters for the Water Survey of Canada gauge stations along the Credit River watershed

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<th>Station Name</th>
<th>Station ID</th>
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<th>LONG (°)</th>
<th>Basin Area (km²)</th>
<th>Main Channel Length (km)</th>
<th>Main Channel Slope (m/km)</th>
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Table 0-2 Event hydrograph characteristics for Water Survey of Canada gauge stations

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<td>2.4</td>
</tr>
<tr>
<td>Cataract</td>
<td>14.2</td>
<td>6.8</td>
</tr>
<tr>
<td>Boston Mills</td>
<td>24.9</td>
<td>11.9</td>
</tr>
<tr>
<td>Norval</td>
<td>36.6</td>
<td>17.5</td>
</tr>
<tr>
<td>Erin Branch</td>
<td>3.3</td>
<td>1.6</td>
</tr>
<tr>
<td>Black Creek</td>
<td>2.1</td>
<td>1.0</td>
</tr>
<tr>
<td>West Branch</td>
<td>9.8</td>
<td>4.7</td>
</tr>
<tr>
<td>Orangeville</td>
<td>1.1</td>
<td>0.7</td>
</tr>
<tr>
<td>Cataract</td>
<td>3.1</td>
<td>2.1</td>
</tr>
<tr>
<td>Boston Mills</td>
<td>5.5</td>
<td>3.7</td>
</tr>
<tr>
<td>Norval</td>
<td>8.0</td>
<td>5.4</td>
</tr>
<tr>
<td>Erin Branch</td>
<td>0.7</td>
<td>0.5</td>
</tr>
<tr>
<td>Black Creek</td>
<td>0.5</td>
<td>0.3</td>
</tr>
<tr>
<td>West Branch</td>
<td>2.2</td>
<td>1.5</td>
</tr>
<tr>
<td>Orangeville</td>
<td>3.7</td>
<td>1.7</td>
</tr>
<tr>
<td>Cataract</td>
<td>10.8</td>
<td>4.8</td>
</tr>
<tr>
<td>Boston Mills</td>
<td>18.9</td>
<td>8.4</td>
</tr>
<tr>
<td>Norval</td>
<td>27.7</td>
<td>12.4</td>
</tr>
<tr>
<td>Erin Branch</td>
<td>2.5</td>
<td>1.1</td>
</tr>
<tr>
<td>Black Creek</td>
<td>1.6</td>
<td>0.7</td>
</tr>
<tr>
<td>West Branch</td>
<td>7.4</td>
<td>3.3</td>
</tr>
</tbody>
</table>
Table 0-3 Event hyetograph characteristics for the Credit River watershed

<table>
<thead>
<tr>
<th>Event</th>
<th>Start Date and Time</th>
<th>End Date and Time</th>
<th>Storm Duration (h)</th>
<th>Average Peak Intensity (mm/h)</th>
<th>Average Intensity (mm/h)</th>
<th>Rainfall Depth (mm)</th>
<th>Event Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mar. 19-22, 1998</td>
<td>3/19/98 18:00</td>
<td>3/22/98 0:00</td>
<td>36</td>
<td>2.6</td>
<td>0.7</td>
<td>24.6</td>
<td>Rainfall Event</td>
</tr>
<tr>
<td>May 11-16, 1998</td>
<td>5/11/98 0:00</td>
<td>5/15/98 0:00</td>
<td>72</td>
<td>4.2</td>
<td>0.7</td>
<td>48.8</td>
<td>Rainfall Event</td>
</tr>
<tr>
<td>Apr 20-22, 2000</td>
<td>4/20/00 0:00</td>
<td>4/29/00 0:00</td>
<td>72</td>
<td>12.6</td>
<td>0.7</td>
<td>47.2</td>
<td>Rainfall Event</td>
</tr>
<tr>
<td>May 12-16, 2000</td>
<td>5/12/00 0:00</td>
<td>5/17/00 0:00</td>
<td>24</td>
<td>18.7</td>
<td>2.4</td>
<td>59.3</td>
<td>Rainfall Event</td>
</tr>
<tr>
<td>May 18-23, 2000</td>
<td>5/18/00 0:00</td>
<td>5/23/00 0:00</td>
<td>12</td>
<td>11.3</td>
<td>2.0</td>
<td>23.9</td>
<td>Rainfall Event</td>
</tr>
<tr>
<td>Jun 14-17, 2000</td>
<td>6/13/00 0:00</td>
<td>6/18/00 0:00</td>
<td>48</td>
<td>11.4</td>
<td>1.4</td>
<td>68.6</td>
<td>Rainfall Event</td>
</tr>
<tr>
<td>Jun 25-29, 2000</td>
<td>6/25/00 0:00</td>
<td>6/29/00 0:00</td>
<td>8</td>
<td>8.2</td>
<td>3.5</td>
<td>28.0</td>
<td>Rainfall Event</td>
</tr>
<tr>
<td>Apr 03-05, 2002</td>
<td>4/3/02 4:00</td>
<td>4/6/02 0:00</td>
<td>6</td>
<td>1.5</td>
<td>0.9</td>
<td>5.6</td>
<td>Snowmelt plus rainfall Event</td>
</tr>
<tr>
<td>Apr 11-12, 2002</td>
<td>4/11/03 7:00</td>
<td>4/14/03 13:00</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>Snowmelt Event</td>
</tr>
<tr>
<td>Nov 28-Dec 02, 2003</td>
<td>11/28/03 0:00</td>
<td>11/2/03 0:00</td>
<td>24</td>
<td>2.2</td>
<td>1.4</td>
<td>33.0</td>
<td>Rainfall Event</td>
</tr>
<tr>
<td>Mar 05-10, 2004</td>
<td>3/4/04 0:00</td>
<td>3/9/04 0:00</td>
<td>4</td>
<td>5.3</td>
<td>3.3</td>
<td>13.2</td>
<td>Snowmelt plus rainfall Event</td>
</tr>
<tr>
<td>Jan 13-18, 2005</td>
<td>1/13/05 0:00</td>
<td>1/18/05 0:00</td>
<td>48</td>
<td>2.3</td>
<td>0.9</td>
<td>19.8</td>
<td>Snowmelt plus rainfall Event</td>
</tr>
<tr>
<td>Jul 17-21, 2005</td>
<td>7/17/05 0:00</td>
<td>7/21/05 0:00</td>
<td>2</td>
<td>4.2</td>
<td>2.2</td>
<td>4.3</td>
<td>Rainfall Event</td>
</tr>
<tr>
<td>Aug 19-23, 2005</td>
<td>8/19/05 0:00</td>
<td>8/22/05 0:00</td>
<td>6</td>
<td>30.7</td>
<td>6.9</td>
<td>41.4</td>
<td>Rainfall Event</td>
</tr>
</tbody>
</table>
Table 0-4 Statistical quantities of mean (μ) and standard deviation (σ) for observed values of time of concentration (Tc)

<table>
<thead>
<tr>
<th>Station Name</th>
<th>Number of Events (N)</th>
<th>Time of Concentration (Tc, hours)</th>
<th>μ</th>
<th>σ</th>
</tr>
</thead>
<tbody>
<tr>
<td>Credit River at Orangeville</td>
<td>14</td>
<td>20.2</td>
<td>2.2</td>
<td></td>
</tr>
<tr>
<td>Credit River at Cataract</td>
<td>14</td>
<td>25.0</td>
<td>3.9</td>
<td></td>
</tr>
<tr>
<td>Credit River at Boston Mills</td>
<td>14</td>
<td>29.4</td>
<td>3.5</td>
<td></td>
</tr>
<tr>
<td>Credit River at Norval</td>
<td>14</td>
<td>33.2</td>
<td>2.2</td>
<td></td>
</tr>
<tr>
<td><strong>Credit River at Erindale</strong></td>
<td>14</td>
<td>40.4</td>
<td>2.4</td>
<td></td>
</tr>
<tr>
<td>Credit River at Erin Branch</td>
<td>14</td>
<td>14.1</td>
<td>3.4</td>
<td></td>
</tr>
<tr>
<td><strong>Credit River above Alton Branch</strong></td>
<td>14</td>
<td>17.6</td>
<td>3.7</td>
<td></td>
</tr>
<tr>
<td>Black Creek below Acton</td>
<td>14</td>
<td>18.1</td>
<td>3.1</td>
<td></td>
</tr>
<tr>
<td>Credit River West Branch at Norval</td>
<td>14</td>
<td>23.5</td>
<td>4.0</td>
<td></td>
</tr>
</tbody>
</table>

**Note 1:** The time of concentration values were estimated from simulated hydrographs derived from the GAWSER model of the Credit River watershed.

Table 0-5 Event characteristics for the Credit River above Alton branch gauge station and the Credit River at Erindale gauge station

<table>
<thead>
<tr>
<th>Event</th>
<th>Duration (d)</th>
<th>Rainfall Depth (mm)</th>
<th>Snowmelt (mm)</th>
<th>Gauge Station Name</th>
<th>Runoff Volume (mm)</th>
<th>Peak Intensity (m³/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>March 13 to April 6, 1989</td>
<td>24</td>
<td>51</td>
<td>246</td>
<td>Credit River above Alton Branch</td>
<td>62</td>
<td>7.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Credit River at Erindale</td>
<td>41.4</td>
<td>49.1</td>
</tr>
<tr>
<td>June 18 to June 30, 1989</td>
<td>12</td>
<td>91</td>
<td>--</td>
<td>Credit River above Alton Branch</td>
<td>17.6</td>
<td>2.8</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Credit River at Erindale</td>
<td>11.2</td>
<td>32.5</td>
</tr>
<tr>
<td>March 9 to March 21, 1990</td>
<td>12</td>
<td>28.6</td>
<td>135.8</td>
<td>Credit River above Alton Branch</td>
<td>60.7</td>
<td>11.4</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Credit River at Erindale</td>
<td>47.7</td>
<td>117.3</td>
</tr>
<tr>
<td>April 1 to April 25, 1991</td>
<td>24</td>
<td>117.4</td>
<td>120.6</td>
<td>Credit River above Alton Branch</td>
<td>96.8</td>
<td>9.6</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Credit River at Erindale</td>
<td>63.3</td>
<td>57.8</td>
</tr>
</tbody>
</table>

Table 0-6 Empirical equations used to estimate the time of concentration (Tc) (minutes) for subwatersheds of the Credit River (Fang et al, 2008)

<table>
<thead>
<tr>
<th>Method</th>
<th>Equation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Williams (1922)</td>
<td>( T_c = 16.32 L_c A^{0.4} / (D S_c^{0.2}) )</td>
</tr>
<tr>
<td>Kirpich (1940)</td>
<td>( T_c = 3.97 L_c^{1.37} S_c^{-0.385} )</td>
</tr>
<tr>
<td>Johnstone – Cross (1949)</td>
<td>( T_c = 3.258 (L_c / S_c)^{0.5} )</td>
</tr>
<tr>
<td>Haktanir – Sezen (1990)</td>
<td>( T_c = 26.85 L_c^{0.841} )</td>
</tr>
</tbody>
</table>

**Note:** The channel length \( L_c \), watershed equivalent diameter \( D \), and watershed width \( W \) are in km, area \( A \) is in km², and \( S_c \) (channel slope) is in (m/m).
Table 0-7 Physical basin parameters for Water Survey of Canada gauge stations across Southern Ontario (Kennedy and Watt, 1967)

<table>
<thead>
<tr>
<th>Basin Name</th>
<th>Station ID</th>
<th>Main Channel Length (km)</th>
<th>Main Channel Slope (m/km)</th>
<th>Elongation Ratio</th>
<th>Observed Lag Time (h)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Canagagigue Creek</td>
<td>02GA023</td>
<td>25.6</td>
<td>4.29</td>
<td>0.460</td>
<td>15.9</td>
</tr>
<tr>
<td>Cold Creek</td>
<td>02HC023</td>
<td>9.82</td>
<td>7.25</td>
<td>0.881</td>
<td>10.9</td>
</tr>
<tr>
<td>Conestoga Creek</td>
<td>02GA017</td>
<td>36.4</td>
<td>2.59</td>
<td>0.250</td>
<td>20.0</td>
</tr>
<tr>
<td>Don West River</td>
<td>02HC005</td>
<td>25.4</td>
<td>5.83</td>
<td>0.419</td>
<td>13.2</td>
</tr>
<tr>
<td>Dufferin Creek</td>
<td>02HC006</td>
<td>25.3</td>
<td>8.89</td>
<td>0.719</td>
<td>14.1</td>
</tr>
<tr>
<td>Etobicoke Creek</td>
<td>02HC002</td>
<td>42.0</td>
<td>4.16</td>
<td>0.345</td>
<td>20.7</td>
</tr>
<tr>
<td>Oakville Creek</td>
<td>02HB005</td>
<td>21.7</td>
<td>7.22</td>
<td>0.480</td>
<td>11.0</td>
</tr>
<tr>
<td>Parkhill Creek</td>
<td>02FF003</td>
<td>43.1</td>
<td>1.41</td>
<td>0.292</td>
<td>30.8</td>
</tr>
<tr>
<td>Trout Creek</td>
<td>02GD009</td>
<td>27.4</td>
<td>1.68</td>
<td>0.474</td>
<td>17.0</td>
</tr>
<tr>
<td>West Humber River</td>
<td>02HC008</td>
<td>36.7</td>
<td>4.35</td>
<td>0.440</td>
<td>15.5</td>
</tr>
</tbody>
</table>

Table 0-8 Statistical quantities of mean (μ) and standard deviation (σ) for observed values of surface storage recession constant (K_{surf}), subsurface recession constant (K_{subs}) and groundwater flow recession constant (K_{gw})

<table>
<thead>
<tr>
<th>Station Name</th>
<th>Number of Events (N)</th>
<th>K_{surf} (h)</th>
<th>K_{subs} (h)^2</th>
<th>K_{gw} (h)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Credit River at Orangeville</td>
<td>14</td>
<td>13</td>
<td>2.5</td>
<td>57</td>
</tr>
<tr>
<td>Credit River at Cataract</td>
<td>14</td>
<td>23</td>
<td>10</td>
<td>73</td>
</tr>
<tr>
<td>Credit River at Boston Mills</td>
<td>14</td>
<td>28</td>
<td>9.4</td>
<td>76</td>
</tr>
<tr>
<td>Credit River at Norval</td>
<td>14</td>
<td>32</td>
<td>13</td>
<td>110</td>
</tr>
<tr>
<td>Credit River at Erindale</td>
<td>14</td>
<td>39</td>
<td>10</td>
<td>165</td>
</tr>
<tr>
<td>Credit River at Erin Branch</td>
<td>14</td>
<td>22</td>
<td>9.8</td>
<td>36</td>
</tr>
<tr>
<td>Credit River above Alton Branch</td>
<td>14</td>
<td>12</td>
<td>5.9</td>
<td>79</td>
</tr>
<tr>
<td>Black Creek below Acton</td>
<td>14</td>
<td>2.9</td>
<td>1.3</td>
<td>45</td>
</tr>
<tr>
<td>Credit River West Branch at Norval</td>
<td>14</td>
<td>17</td>
<td>7.3</td>
<td>63</td>
</tr>
</tbody>
</table>

Note 1: The recession constant values were estimated from simulated hydrographs derived from the GAWSER model of the Credit River watershed and verified through historical time series data.

Note 2: The subsurface flow recession constant values were determined by averaging the surface storage recession constants and the groundwater flow recession constants.

4.2 Mathematical Formulation of Time Parameters

A step forward regression procedure was adopted for determining the multiple regression equations for the time of concentration. The curvilinear regression technique was used to determine the form of the regression equation. The method of least squares was used to evaluate the “best fit” line between the observed and predicted values of the time constants. The method of least squares is generally used because it minimizes the sum of the square of the differences between the sample criterion values and the estimated criterion values (Part 630 Hydrology National Engineering Handbook Chapter 18: Selected Statistical Methods, 2000). In this case
the criterion variable is the observed time of concentration (h) and the predictor variables are the main channel length (km), main channel slope (m/km) and the elongation ratio (dimensionless).

Each variable was plotted against each other to determine if a linear or non-linear data trend exists. The linear correlation coefficients between each pair of variables were determined. The linear correlation coefficient for each pair of variable was computed using equation 4.12 (Part 630 Hydrology National Engineering Handbook Chapter 18: Selected Statistical Methods, 2000).

$$r = \frac{\sum_{i=1}^{N} (x_i - \bar{x})(y_i - \bar{y})}{\left[ \sum_{i=1}^{N} (x_i - \bar{x})^2 \sum_{i=1}^{N} (y_i - \bar{y})^2 \right]^{\frac{1}{2}}}$$

(4.12)

Where \(X_i\) and \(Y_i\) are values of the \(i^{th}\) observation of the two variables \(X\) and \(Y\), respectively; \(\bar{X}\) and \(\bar{Y}\) are the means of the two sample variables; and \(N\) is the number of common elements in the sample. A multiple regression equation based on the main channel length (\(L_c\), km), main channel slope (\(S_c\), m/km) and elongation ratio (\(E\), dimensionless) was developed using a step forward regression procedure. The step forward regression starts with the most important predictor as the only variable in the equation. The most important of the remaining predictors is added and partial F-test is computed. If this predictor is significant, the next most important of the remaining predictors is added and the process repeated. When a non-significant predictor is found, the previous equation that does not include that predictor is used (Part 630 Hydrology National Engineering Handbook Chapter 18: Selected Statistical Methods, 2000). The values of the regression coefficients were found by a stepwise multiple-linear regression analysis of \(\log T_c\), on the logarithms of the independent variables. The Excel 2012 statistical package, regression statistical function was used to calculate the regression coefficients for each step of the regression analysis. The total and partial F-test ratios were also calculated (equations 4.13 and 4.14), to evaluate each predictor variable and determine the total equation significance.

The standard error of estimate, which is denoted by \(S_e\) and is computed by equation 4.13 (Part 630 Hydrology National Engineering Handbook Chapter 18: Selected Statistical Methods, 2000):
\[ S_e = \left( \frac{\sum_{i=1}^{N}(Y_i - \hat{Y}_i)^2}{df} \right)^{0.5} \]  

(4.13)

Where \( \hat{Y}_i \) is the predicted value; \( Y_i \) is the observed value of the \( i \)th observation on the criterion variable or \( T_c \), and \( df \) is the degrees of freedom. The degrees of freedom equal the number of independent pieces of information required to form the estimate or \( T_c \). For a regression equation, this equals the number of observations in the data sample \( N \) minus the number of unknowns estimated from the data. The degrees of freedom for the regression equation were determined to be 5 degrees of freedom.

The significance of the predictor variables (i.e., \( L_c, S_c \) and \( E \)) and the total equation were evaluated by using F-tests. Two F-tests were used, the partial F-test \( (F_p) \) and the total F-test \( (F_t) \). The partial F-test checks the significance of the predictor variables that are added from the regression equation. The total F-test checks the significance of the entire regression equation. The partial F-test is computed by equation 4.14 (Part 630 Hydrology National Engineering Handbook Chapter 18: Selected Statistical Methods, 2000):

\[
F_p = \frac{(r_p^2 - r_{p-1}^2)}{(1-r_p^2) \frac{1}{(N-p-1)}}
\]  

(4.14)

Where \( r_p \) and \( r_{p-1} \) are the coefficients of determination for the \( p \) and \( p-1 \) predictor models. The equation is significant if the computed F is greater than the value found in an F distribution table. The degrees of freedom needed for uses of the F table are 1\((df_1)\) and \( N-p-1 \) \((df_2)\) or 1 and 5 respectively. The total F-test is computed by equation 4.15 (Part 630 Hydrology National Engineering Handbook Chapter 18: Selected Statistical Methods, 2000):

\[
F_t = \frac{r_p^2}{p} \frac{1}{(1-r_p^2) \frac{1}{(N-p-1)}}
\]  

(4.15)

Where \( p \) is the number of predictors in the equation and \( r_p^2 \) is the coefficient of determination for the \( p \) predictor equation. The same procedures as described previously were used for all time
constants including the surface-storage recession constant, subsurface flow recession constant and groundwater flow recession constant.

### 4.3 Mathematical Formulation of Simplified Dynamic Model

The Saint Venant equations, first developed by Barre de Saint-Venant in 1871, describe the one-dimensional unsteady open channel flow. The following assumptions are necessary for the derivation of the Saint Venant equations: The flow is one-dimensional; depth and velocity vary only in the longitudinal direction of the channel. This implies that the velocity is constant and the water surface is horizontal across any section perpendicular to the longitudinal axis. Flow is assumed to vary gradually along the channel so that hydrostatic pressure prevails and vertical accelerations can be neglected (Chow, 1959). The longitudinal axis of the channel is approximated as a straight line. The bottom slope of the channel is small and the channel bed is fixed; therefore the effects of scour and deposition are negligible. Resistance coefficients for steady uniform turbulent flow are applicable so that relationships such as Manning’s equation can be used to describe resistance effects. The fluid is incompressible and of constant density throughout the flow. The complete derivation of the Saint Venant equations is described in Chow and Maidment (1988).

The conservation form of both the continuity and momentum equations are defined by equations 4.16 and 4.17 respectively (Aral and Gunduz, 2003). The momentum equation in equation 4.17; is based on the complete dynamic wave form of the unsteady non-uniform Saint-Venant equations.

**Continuity:**

\[
\frac{\partial Q}{\partial x} + \frac{\partial (A + A_0)}{\partial t} - q_{L1} - q_{L2} = 0 \tag{4.16}
\]

**Momentum:**

\[
\frac{\partial Q}{\partial t} + \frac{\partial }{\partial x} \left( \frac{Q^2}{A} \right) + gA \frac{\partial h}{\partial x} + gA(S_{ec} + S_f) + M_{L1} + M_{L2} = 0 \tag{4.17}
\]
Where $x$ is the longitudinal coordinate representing the distance along the channel, $t$ is the temporal coordinate, $A$ is the active cross-sectional area of flow (m$^2$), $A_o$ is the inactive (off-channel storage) cross-sectional area of the channel and/or floodplain (m$^2$), $Q$ is the discharge (m$^3$/s), $q_{L1}$ is the lateral seepage flow per channel length (positive for inflow and negative for outflow, m$^3$/s), $q_{L2}$ is the lateral overland flow per channel length (positive for inflow and negative for outflow, m$^3$/s), $g$ is the gravitational acceleration (m/s$^2$), $h$ is the water surface elevation in the river (m), $M_{L1}$ is the momentum flux due to lateral seepage inflow/outflow, $M_{L2}$ is the momentum flux due to lateral overland inflow/outflow and $S_f$ and $S_{ec}$ are channel or floodplain boundary friction slope and contraction/expansion slope, respectively. The momentum flux due to lateral seepage and overland flows, contraction/expansion slope, and channel/flood plain boundary friction slope are evaluated as:

$$M_{L1} = \begin{cases} 
0 & \text{for seepage inflow} \\
\frac{-q_{L1}}{2A} & \text{for seepage outflow}
\end{cases}$$

(4.18)

$$M_{L2} = \begin{cases} 
-\beta v_x q_{L2} & \text{for overland inflow} \\
\frac{-q_{L2}}{A} & \text{for overland outflow}
\end{cases}$$

(4.19)

$$S_{ec} = \frac{k_{ec}(\frac{Q}{A})^2}{2g \Delta x}$$

(4.20)

$$S_f = \frac{n^4 |q| Q}{A^2 R^3}$$

(4.21)

Where $v_x$ is the velocity of the overland flow in the direction of open channel flow, $K_{ec}$ is the expansion/contraction coefficient, $\Delta x$ is the reach length, $n$ is the manning’s coefficient for open channel, $\beta$ is the momentum coefficient for velocity distribution and $R$ is the hydraulic radius. If the overland flow is assumed to enter the channel perpendicular to the channel direction then the momentum flux for overland flow is equal to zero. The hydraulic radius is defined as the ratio of cross-sectional area to wetted perimeter but it is also approximated as the ratio of cross-sectional area to top width for large rivers. In the routing model, the channel and/or floodplain section is
treated as a composite section (Fread, 1998). The lateral flow that provides the link between the open channel flow model and the overland flow, subsurface flow and shallow groundwater flow models is defined as:

$$q_{L1} = \frac{Q_{subs} + Q_{gw}}{L_{ch}}$$ (4.22)

$$q_{L2} = \frac{Q_{surf}}{L_{ch}}$$ (4.23)

Where $Q_{surf}$ is the overland surface inflow, $Q_{subs}$ is the lateral subsurface inflow, $Q_{gw}$ is the shallow groundwater inflow and $L_{ch}$ is the channel reach length.

### 4.3.1 Selection of Numerical Solution Scheme

In order to obtain an analytical solution to flood routing in open channels, various simplified approximations to the Saint Venant Equations have been proposed, due to the difficulty in directly solving the complete equations. Common techniques using simplifications to the full momentum equation are kinematic wave routing (Ağiralioğlu, 1981, 1988) and diffusion wave routing (Akan and Yen, 1981; Gonwa and Kavvas, 1986). The accuracy of these approximations to the momentum effects in flood routing are discussed in detail in Ponce et al. (1978) and Sinha et al. (1995). Their application to flood routing is dependent upon the physical characteristics of the channel and the flood wave. For example, if inertial and pressure forces are unimportant, the gravitational force of the flow is balanced by the frictional resistance force, channel slope is steep, backwater effects are negligible, and kinematic waves govern the flow. However, if the pressure forces are important but inertial forces remain unimportant a diffusion wave model is applicable. When both inertial and pressure forces are important, such as in mild-sloped rivers, and backwater effects from downstream disturbances are not negligible, then both the inertial force and pressure force terms in the momentum equation are needed. Under these circumstances the dynamic wave routing method is required, which involves numerical solution of the full Saint Venant Equations. In turn, the Saint Venant’s equations, without any simplification, have been solved by many numerical methods for cylindrical and irregular cross-sections and are currently
in use (Amein and Fang, 1970; Fread, 1973; Koussis, 1976; Lambreti and Pilati, 1996). However, many of these solutions are computationally complex and require a lot of computer effort.

In this study, an explicit solution scheme developed by Keskin (1997) and utilized by Raimundo and Patrícia (2005) to solve the Saint Venant Equations was applied. However, Keskin’s formulation for solving the Saint Venant equations was applied to a rectangular open channel with constant top-width. The numerical solution proposed here expands upon Keskin’s original formulations for solving the Saint Venant equations to include trapezoidal and triangular open channels. In addition, these same formulations for solving the Saint Venant equations are further applied to include routing of flows in meandering rivers with wide floodplains. The overall structure and flow chart of the one-dimensional unsteady flow model is illustrated in Figure 4.3.

In explicit models, simplified versions of equations 4.24 and 4.25 are expressed in the following form to enable an explicit solution of their finite difference approximations:

\[
\frac{\partial q}{\partial x} + \frac{\partial A}{\partial t} - q_{L1} - q_{L2} = 0 \tag{4.24}
\]

\[
\frac{\partial q}{\partial t} + \frac{\partial}{\partial x}\left(\frac{q^2}{A}\right) + gA\left(\frac{\partial y}{\partial x} + S_t - S_o\right) + M_{L1} + M_{L2} = 0 \tag{4.25}
\]

In which \(A\) is the total cross-sectional area of the channel (active and inactive or off-channel storage areas). Also, the effect of lateral inflow both seepage flow \((q_{L1})\) and overland flow \((q_{L2})\) and the resulting momentum fluxes \((M_{L1}\) and \(M_{L2}\)) are included in equations 4.24 and 4.25 respectively. In equation 4.25, \(S_o\) is the bed slope and \(y\) is the water depth, all other terms are as previously defined.

For a trapezoidal cross-section, the cross-sectional area can be written as

\[
A = (B_w + yz)y \tag{4.26}
\]
Where $B_w$ is the bottom-width of the cross-section, $y$ is the depth of flow in the channel and $z$ is the inverse of the channel side slope. Assuming the bottom width of the channel is constant the partial derivative of equation 4.26 can be obtained as follows:
\[
\frac{\partial A}{\partial x} = B_w \frac{\partial y}{\partial x} + 2yz \frac{\partial y}{\partial x} \quad (4.27a)
\]

\[
\frac{\partial A}{\partial x} = (B_w + 2yz) \frac{\partial y}{\partial x} \quad (4.27b)
\]

\[
\frac{\partial y}{\partial x} = \frac{1}{B} \frac{\partial A}{\partial x} \quad (4.27c)
\]

In equation 4.27c \(B\) is the wetted top width of the total cross-sectional area of the channel.

For a triangular cross-section, the cross-sectional area can be written as

\[
A = y^2z \quad (4.28)
\]

The partial derivative of equation 4.28 can be obtained as follows:

\[
\frac{\partial A}{\partial x} = 2yz \frac{\partial y}{\partial x} \quad (4.29a)
\]

\[
\frac{\partial y}{\partial x} = \frac{1}{B} \frac{\partial A}{\partial x} \quad (4.29b)
\]

Equation 4.29b is equivalent to equation 4.27c, however, in equation 4.27c \(B = B_w + 2yz\); in equation 4.29b \(B = 2yz\); and therefore, the wetted top width \(B\) of the channel in equations 4.27c and 4.29b is not constant but will vary with the depth of flow. Substituting equation 4.27c or equation 4.29b into equation 4.25 and rearranging the equation one obtains

\[
\frac{\partial Q}{\partial t} + 2Q \frac{\partial Q}{\partial x} + \left(\frac{gA}{B} - \frac{Q^2}{A^2}\right) \frac{\partial A}{\partial x} + gA(S_f - S_o) + M_{L1} + M_{L2} = 0 \quad (4.30)
\]

The flow velocity in SI units is obtained from Manning’s formula:

\[
V = \frac{1}{n} R^{2/3} S_f^{1/2} \quad (4.31)
\]
Where, $R$ is the hydraulic radius, $S_f$ is the friction slope and $n$ is Manning’s friction coefficient.

For a trapezoidal or triangular channel the following relationships are given:

$$ R = \frac{A}{P} \quad (4.32) $$

**Trapezoid:**

$$ P = B_w + 2y\sqrt{1 + z^2} \quad (4.33a) $$

**Triangle:**

$$ P = 2y\sqrt{1 + z^2} \quad (4.33b) $$

The partial derivatives for equations 4.32, 4.33a and 4.33b are as follows:

$$ \frac{\partial P}{\partial x} = 2\sqrt{1 + z^2} \frac{\partial y}{\partial x} \quad (4.34) $$

$$ \frac{\partial R}{\partial x} = \frac{1}{P} \left( 1 - \frac{2\sqrt{1 + z^2}A}{PB} \right) \frac{\partial A}{\partial x} \quad (4.35) $$

$$ \frac{\partial v}{\partial x} = \frac{2}{3} n R^{-1/3} S_f^{1/2} \frac{\partial R}{\partial x} + \frac{1}{2} n S_f^{-1/2} R^{2/3} \frac{\partial S_f}{\partial x} \quad (4.36) $$

The partial derivatives $\frac{\partial P}{\partial x}$, $\frac{\partial R}{\partial x}$ and $\frac{\partial v}{\partial x}$ are the same for both a trapezoidal and triangular channel cross-section. The values for the wetter perimeter ($P$), the wetted top width ($B$) and the hydraulic radius ($R$) of the channel in equations 4.35 and 4.36 will vary based on the type of channel cross-section and the depth of flow. If $\frac{\partial S_f}{\partial x}$ is very small in comparison to other terms in equation 4.36, then it can be neglected and equation 4.36 can be written as

$$ \frac{\partial v}{\partial x} = \frac{2}{3} n R^{-1/3} S_f^{1/2} \frac{\partial R}{\partial x} \quad (4.37) $$
The discharge is defined as \( Q = VA \), and the partial derivative of the discharge can be obtained as

\[
\frac{\partial Q}{\partial x} = \frac{\partial V}{\partial x} A + \frac{\partial A}{\partial x} V
\]

(4.38)

By substituting equations 4.34, 4.35 and 4.37 into equation 4.38, the following equation is obtained:

\[
\frac{\partial A}{\partial x} = \frac{1}{\sqrt[3]{3 \frac{4R}{3B} 1 + z^2}} \frac{\partial Q}{\partial x}
\]

(4.39)

Substituting equation 4.39 into equation 4.25, the following equation is obtained (Keskin, 1997),

\[
\frac{\partial Q}{\partial t} + \alpha \frac{\partial Q}{\partial x} + \beta = 0
\]

(4.40)

\[
\alpha = 2 \cdot \frac{Q}{A} + \frac{\frac{\partial A}{\partial x} \cdot \frac{Q^2}{A^2}}{\sqrt[3]{3 \frac{4R}{3B} 1 + z^2}}
\]

(4.41)

\[
\beta = g \cdot A \cdot (S_f - S_o) + M_{L1} + M_{L2}
\]

(4.42)

The friction slope, \( S_f \), can be obtained from Manning’s friction formula as follows:

\[
S_f = \frac{Q^2 n^2}{A^2 R^{1/3}}
\]

(4.43)

The momentum equation, equation 4.25, is transformed to equation 4.40, which has two parameters related to cross-sectional area and discharge of the channel (Keskin, 1997). If \( z \) was set equal to zero in equation 4.41, then equations 4.40 and 4.41 would be also be applicable to a rectangular channel cross-section. Therefore, equation 4.41 is equivalent to Keskin’s original formulation of alpha (\( \alpha \)) and could be applied to a trapezoidal and/or triangular channel cross-section, provided that the \( z \) term, the inverse of the channel side slopes, is taken into account.
consideration as in equation 4.41. Furthermore, equation 4.40 can be solved easily by using a numerical solution subject to initial, internal and external boundary conditions.

In this hydrodynamic model there are two dependant variables. The first refers to the cross-sectional area $A(x, t)$, along the channel, for each interval of time. The second one refers to the flow field $Q(x, t)$ along the channel, for the same previous conditions. Two dependent variables are necessary to solve the differential equations: equations 4.24 and 4.40 will compose the model. To solve the differential equations an explicit solution scheme will be used and defined through the relationship:

$$ \frac{Q_{i}^{j+1} - Q_{i}^{j}}{\Delta t} + \alpha_{i}^{j} \frac{(Q_{i}^{j} - Q_{i-1}^{j})}{\Delta x} + \beta_{i}^{j} = 0 \quad (4.44) $$

Making a re-arrangement of equation 4.44, it is possible to find:

$$ Q_{i}^{j+1} = Q_{i}^{j} - \frac{\Delta t}{\Delta x} (\alpha_{i}^{j}) (Q_{i}^{j} - Q_{i-1}^{j}) - \beta_{i}^{j} \Delta t \quad (4.45) $$

Where $\alpha_{i}^{j}$ and $\beta_{i}^{j}$ are defined, respectively:

$$ \alpha_{i}^{j} = 2 \frac{Q_{i}^{j}}{A_{i}^{j}} + \frac{g A_{i}^{j}}{R_{i}^{j}} \left( \frac{Q_{i}^{j}}{A_{i}^{j}} \right)^{2} \quad (4.46) $$

$$ \beta_{i}^{j} = g A_{i}^{j} \left[ \frac{(Q_{i}^{j})^{2}}{A_{i}^{j}(R_{i}^{j})^{7/3}} - S_{o} \right] + M_{L1,i}^{j} + M_{L2,i}^{j} \quad (4.47) $$

With the value of $Q$ calculated for the next time step, it is possible to calculate the cross-sectional area $A(x, t)$, through:

$$ A_{i}^{j+1} = A_{i}^{j} - \frac{\Delta t}{\Delta x} (Q_{i}^{j+1} - Q_{i-1}^{j+1}) + \Delta t \cdot (q_{L1} + q_{L2}) \quad (4.48) $$
The depth of flow within the channel is estimated using one of the following equations:

**Rectangle:**

\[ y_{l+1} = \frac{A_{l+1}}{B_{wl}} \]  \hspace{1cm} (4.51)

**Triangle:**

\[ y_{l+1} = \sqrt{\frac{A_{l+1}}{z_l}} \]  \hspace{1cm} (4.52)

**Trapezoid:**

\[ y_{l+1} = \sqrt{\frac{A_{l+1}}{z_l} + \frac{B_{wl}}{2z_l} - \frac{B_{wl}}{2z_l}} \]  \hspace{1cm} (4.53)

Unsteady flow in a natural river which meanders through a wide floodplain is complicated by differences in geometry and hydraulic characteristics between the river channel and the floodplain. The flow is further complicated by the meandering of the main channel causing a portion of the total flow to proceed downstream along the more direct path afforded by the floodplain rather than along the meandering channel. This process is further enhanced by the greater longitudinal slope of the floodplain; however, it is diminished by the greater hydraulic roughness associated with the floodplain. This is further complicated by portions of the floodplain that act as dead storage areas or ineffective flow areas, where the flow velocities are negligible.
The GFLOOD model contains a modified form of the Saint Venant equations for an alternative method for routing floods in meandering rivers with floodplains. The one-dimensional Saint Venant equations are modified such that the flow in the meandering channel and floodplain are identified separately. The Saint Venant equations are modified (Fread, 1976) as follows:

\[
\frac{\partial (K_{cn}Q)}{\partial x_{cn}} + \frac{\partial (K_{lf}Q)}{\partial x_{lf}} + \frac{\partial (K_{rf}Q)}{\partial x_{rf}} + \frac{\partial (A_{cn}+A_{lf}+A_{rf})}{\partial t} - q_{L1} - q_{L2} = 0
\] (4.54)

\[
\frac{\partial Q}{\partial t} + \frac{\partial (K_{cn}^2 Q^2/A_{cn})}{\partial x_{cn}} + \frac{\partial (K_{lf}^2 Q^2/A_{lf})}{\partial x_{lf}} + \frac{\partial (K_{rf}^2 Q^2/A_{rf})}{\partial x_{rf}} + gA_{cn} \left( \frac{\partial y_{cn}}{\partial x_{cn} + S_{fcn} - S_{ocn}} \right) + gA_{lf} \left( \frac{\partial y_{lf}}{\partial x_{lf} + S_{lf} - S_{olf}} \right) + gA_{rf} \left( \frac{\partial y_{rf}}{\partial x_{rf} + S_{rf} - S_{orf}} \right) + M_{L1} + M_{L2}
\] (4.55)

The parameters \((K_{cn}, K_{lf}, K_{rf})\) proportion the total flow \((Q)\) into the channel, left floodplain and right floodplain, respectively. These are defined (Fread, 1976) as follows:

\[
K_{cn} = \frac{1}{(1+k_l+k_r)}
\] (4.56)

\[
K_{lf} = \frac{k_l}{(1+k_l+k_r)}
\] (4.57)

\[
K_{rf} = \frac{k_r}{(1+k_l+k_r)}
\] (4.58)

\[
k_l = \frac{Q_{lf}}{Q_{cn}} = \frac{n_{cn} A_{lf} (R_{lf})^{2/3}}{n_{lf} A_{cn} (R_{cn})^{2/3}} \left( \frac{\Delta x_{cn}}{\Delta x_{lf}} \right)^{1/2}
\] (4.59)

\[
k_r = \frac{Q_{rf}}{Q_{cn}} = \frac{n_{cn} A_{rf} (R_{rf})^{2/3}}{n_{rf} A_{cn} (R_{cn})^{2/3}} \left( \frac{\Delta x_{cn}}{\Delta x_{rf}} \right)^{1/2}
\] (4.60)

Equations 4.57 and 4.58 represent the ratio of flow in the channel section to flow in the left and right floodplain sections; the flows are expressed in terms of the Manning equation, the energy
slope approximated by the water surface slope \( \frac{\Delta y}{\Delta x} \). The friction slope terms in equation 4.55 are similar to equation 4.43. Equations 4.54 and 4.55 are solved using the same explicit finite difference expressions used to solve equations 4.24 and 4.25. The coefficients \( K_{lf} \) and \( K_{rf} \) are considered to be zero until the water elevation is sufficient to produce wetted top widths, \( B_{lf} \) and \( B_{rf} \), greater than 0.3 m. Therefore, any terms associated with the left or right floodplains in equations 4.54 and 4.55 are set to zero, as are the derivatives associated with the terms. This avoids numerical difficulties such as division by zero, during the numerical simulation.

When the volume of water in the reach exceeds the maximum amount that can be held by the channel, the excess water spreads across the floodplain. When flow is present in the floodplain, the calculation of the flow depth \( y \), cross-sectional flow area and wetting perimeter is the sum of the channel and floodplain components:

**Channel component:**

\[
A_{cn} = A_{ch} + B_{bnk}(y_c)
\]

(4.61)

**Left Floodplain component:**

\[
A_{lf} = B_{wlf} y_{lf} + \frac{1}{2} z_{fl} y_{fl}^2
\]

(4.62)

**Right Floodplain component:**

\[
A_{rf} = B_{wrf} y_{rf} + \frac{1}{2} z_{rf} y_{rf}^2
\]

(4.63)

\[
A = A_{cn} + A_{rf} + A_{lf}
\]

(4.64)

Where \( A_{ch} \) is the cross-sectional area of the channel at bankfull, \( B_{bnk} \) is the wetted top width of the channel at bank full, \( z_{lf} \) and \( z_{rf} \) are the inverse side slopes of the left and right floodplain components, and \( y_c \) is the depth of flow above top of bank within the channel or the depth in the floodplain. Equation 4.61 assumes that the channel component above top of bank is rectangular in shape. The depth of water in the floodplain is defined by the left and right floodplain components, \( y_{lf} \) and \( y_{rf} \), respectively. As a result,
\[ y_c = y_{lf} = y_{rf} \quad \text{(4.65)} \]

The total depth of water within the channel portion is defined by equation 4.61. Assuming the wetted top width of the channel at bankfull is constant, the partial derivative of equation 4.61 can be obtained as follows:

\[ y_{cn} = y_{bnk} + y_c \quad \text{(4.66)} \]

\[ A_{cn} = A_{ch} + B_{bnk}(y_{cn} - y_{bnk}) \quad \text{(4.67)} \]

\[ \frac{\partial y_{cn}}{\partial x_{cn}} = \frac{1}{B_{bnk}} \frac{\partial A_{cn}}{\partial x_{cn}} \quad \text{(4.68)} \]

By substituting equation 4.68 into equation 4.55 and rearranging the equation one obtains

\[ \frac{\partial Q}{\partial t} + 2 \frac{\kappa_{cn}^2 Q}{A_{cn}} \frac{\partial Q}{\partial x_{cn}} + \left( \frac{g A_{cn}}{B_{bnk}} - \frac{\kappa_{cn}^2 Q^2}{A_{cn}^2} \right) \frac{\partial A_{cn}}{\partial x_{cn}} + \frac{\partial (\frac{\kappa_{lf}^2 Q^2}{A_{lf}})}{\partial x_{lf}} + \frac{\partial (\frac{\kappa_{rf}^2 Q^2}{A_{rf}})}{\partial x_{rf}} + g A_{cn} (S_{fcn} - S_{ocn}) + g A_{lf} \left( \frac{\partial y_{lf}}{\partial x_{lf}} + S_{flf} - S_{olf} \right) + g A_{rf} \left( \frac{\partial y_{rf}}{\partial x_{rf}} + S_{frf} - S_{orf} \right) + M_{L1} + M_{L2} = 0 \quad \text{(4.69)} \]

For the channel component the following relationships are given for the channel velocity \( V \), hydraulic radius \( R \) and the wetted perimeter \( P \).

\[ V_{cn} = \frac{1}{n} R_{cn}^{2/3} S_{f_{cn}}^{1/2} \quad \text{(4.70)} \]

\[ R_{cn} = \frac{A_{cn}}{P_{cn}} \quad \text{(4.71)} \]

\[ P_{cn} = B_{wch} + 2y_{bnk} \sqrt{1 + z_{ch}^2} \quad \text{(4.72)} \]
Where $B_{wch}$ and $z_{ch}$ are the bottom width of the channel and inverse side slope of the channel respectively. Given the above relations, the following partial derivatives may be listed

\[
\frac{\partial R_{cn}}{\partial x_{cn}} = \frac{1}{p_{cn}} \frac{\partial A_{cn}}{\partial x_{cn}} \quad (4.73)
\]

\[
\frac{\partial v_{cn}}{\partial x_{cn}} = \frac{2}{3} \frac{1}{n_{cn}} R_{cn}^{-1/3} S_{cn}^{1/2} \frac{1}{p_{cn}} \frac{\partial R_{cn}}{\partial x_{cn}} \quad (4.74)
\]

In equation 4.74, the term $\frac{\partial S_{cn}}{\partial x_{cn}}$ is very small in relation to the other terms and was therefore neglected. Defining the discharge for the channel component as $Q_{cn} = V_{cn} A_{cn}$, the following partial derivative can be obtained:

\[
K_{cn} \frac{\partial Q}{\partial x_{cn}} = \frac{\partial v_{cn}}{\partial x_{cn}} A_{cn} + \frac{\partial A_{cn}}{\partial x_{cn}} V_{cn} \quad (4.75)
\]

Substituting equations 4.73 and 4.74 into equation 4.75 and rearranging the equation one obtains

\[
K_{cn} \frac{\partial Q}{\partial x_{cn}} = \frac{5}{3} V_{cn} \frac{\partial A_{cn}}{\partial x_{cn}} \quad (4.76)
\]

The same formulations are used to define the left and right floodplain components in equation 4.69. Assuming the bottom width of the floodplain is constant, the partial derivatives of equations 4.62 and 4.63 can be obtained as follows:

**Left floodplain:**

\[
\frac{\partial A_{lf}}{\partial x_{lf}} = B_{lf} \frac{\partial y_{lf}}{\partial x_{lf}} \quad (4.77)
\]

\[B_{lf} = B_{wlf} + z_{lf} y_{lf} \quad (4.78)\]

**Right floodplain:**

\[
\frac{\partial A_{rf}}{\partial x_{rf}} = B_{rf} \frac{\partial y_{rf}}{\partial x_{rf}} \quad (4.79)
\]
\[ B_{rf} = B_{wrf} + z_{rf}y_{rf} \]  \hspace{1cm} (4.80)

Where \( B_{lf} \) and \( B_{rf} \) are the wetted top widths for the left and right floodplain components. Substituting equations 4.77 and 4.79 into equation 4.69 and rearranging the equation one obtains:

\[
\frac{\partial Q}{\partial t} + 2 \frac{K_{cn}^2 Q}{A_{cn}} \frac{\partial Q}{\partial x_{cn}} + \left( \frac{g A_{cn}}{B_{bnk}} - \frac{K_{cn}^2 Q^2}{A_{cn}^2} \right) \frac{\partial A_{cn}}{\partial x_{cn}} + 2 \frac{K_{lf}^2 Q}{A_{lf}} \frac{\partial Q}{\partial x_{lf}} + \left( \frac{g A_{lf}}{B_{lf}} - \frac{K_{lf}^2 Q^2}{A_{lf}^2} \right) \frac{\partial A_{lf}}{\partial x_{lf}} + 2 \frac{K_{rf}^2 Q}{A_{rf}} \frac{\partial Q}{\partial x_{rf}} +
\]

\[
\left( \frac{g A_{cf}}{B_{rf}} - \frac{K_{rf}^2 Q^2}{A_{rf}^2} \right) \frac{\partial A_{cf}}{\partial x_{rf}} + g A_{cn} (S_{fcn} - S_{o cn}) + g A_{lf} (S_{fcf} - S_{olv}) + g A_{rf} (S_{fcrf} - S_{orw}) + M_{L1} + M_{L2} = 0 \]  \hspace{1cm} (4.81)

For the right and left floodplain components the following relationships are given for the hydraulic radius \((R)\) and wetted perimeter \((P)\):

**Left floodplain:**

\[ R_{lf} = \frac{A_{lf}}{P_{lf}} \]  \hspace{1cm} (4.82)

\[ P_{lf} = B_{wlf} + y_{lf} \sqrt{1 + z_{lf}^2} \]  \hspace{1cm} (4.83)

**Right floodplain:**

\[ R_{rf} = \frac{A_{rf}}{P_{rf}} \]  \hspace{1cm} (4.84)

\[ P_{rf} = B_{wrf} + y_{rf} \sqrt{1 + z_{rf}^2} \]  \hspace{1cm} (4.85)

Given the above relations, the following partial derivatives may be listed:

**Left floodplain:**
\[ \frac{\partial P_{lf}}{\partial x_{lf}} = \sqrt{1 + z_{lf}^2} \frac{\partial y_{lf}}{\partial x_{lf}} \] (4.86)

\[ \frac{\partial R_{lf}}{\partial x_{lf}} = \frac{1}{p_{lf}} \left( 1 - \frac{A_{lf} \sqrt{1 + z_{lf}^2}}{p_{lf} B_{lf}} \right) \frac{\partial A_{lf}}{\partial x_{lf}} \] (4.87)

\[ \frac{\partial v_{lf}}{\partial x_{lf}} = \frac{2}{3} \frac{1}{n_{lf}} R_{lf}^{-1/3} S_{lf}^{1/2} \frac{\partial R_{lf}}{\partial x_{lf}} \] (4.88)

**Right floodplain:**

\[ \frac{\partial P_{rf}}{\partial x_{rf}} = \sqrt{1 + z_{rf}^2} \frac{\partial y_{rf}}{\partial x_{rf}} \] (4.89)

\[ \frac{\partial R_{rf}}{\partial x_{rf}} = \frac{1}{p_{rf}} \left( 1 - \frac{A_{rf} \sqrt{1 + z_{rf}^2}}{p_{rf} B_{rf}} \right) \frac{\partial A_{rf}}{\partial x_{rf}} \] (4.90)

\[ \frac{\partial v_{rf}}{\partial x_{rf}} = \frac{2}{3} \frac{1}{n_{rf}} R_{rf}^{-1/3} S_{rf}^{1/2} \frac{\partial R_{rf}}{\partial x_{rf}} \] (4.91)

Similar to equations 4.37 and 4.74, the terms \[ \frac{\partial S_{lf}}{\partial x_{lf}} \] and \[ \frac{\partial S_{rf}}{\partial x_{rf}} \] are very small in comparison to the other terms and therefore, are neglected from equations 4.88 and 4.91. The discharge for the left and right floodplains are defined as \( Q_{lf} = V_{lf} A_{lf} \) and \( Q_{rf} = V_{rf} A_{rf} \) respectively, and the following partial derivatives can be obtained:

**Left floodplain:**

\[ K_{lf} \frac{\partial Q}{\partial x_{lf}} = \frac{\partial v_{lf}}{\partial x_{lf}} A_{lf} + \frac{\partial A_{lf}}{\partial x_{lf}} V_{lf} \] (4.92)

**Right floodplain:**

\[ K_{rf} \frac{\partial Q}{\partial x_{rf}} = \frac{\partial v_{rf}}{\partial x_{rf}} A_{rf} + \frac{\partial A_{rf}}{\partial x_{rf}} V_{rf} \] (4.93)
By substituting equations 4.87 and 4.88 into equation 4.92 and substituting equations 4.90 and 4.91 into equation 4.93 and rearranging the equations one obtains

**Left floodplain:**
\[
\frac{\partial A_{lf}}{\partial x_{lf}} = \frac{K_{lf}}{V_{lf}} \frac{\partial Q}{\partial x_{lf}} \quad (4.94)
\]

**Right floodplain:**
\[
\frac{\partial A_{rf}}{\partial x_{rf}} = \frac{K_{rf}}{V_{rf}} \frac{\partial Q}{\partial x_{rf}} \quad (4.95)
\]

Substituting equations 4.76, 4.94 and 4.95 into equation 4.81, equation 4.96 can be obtained as follows:
\[
\frac{\partial Q}{\partial t} + \alpha_{cn} \frac{\partial Q}{\partial x_{cn}} + \alpha_{lf} \frac{\partial Q}{\partial x_{lf}} + \alpha_{rf} \frac{\partial Q}{\partial x_{rf}} + \beta_{cn} + \beta_{lf} + \beta_{rf} = 0 \quad (4.96)
\]

In which,
\[
\alpha_{cn} = \frac{2K_{cn}^2Q}{A_{cn}} + \frac{3}{5} \left( \frac{\frac{gA_{cn}}{B_{hn}} \frac{K_{cn}^2Q^2}{A_{cn}}}{Q} \right) \quad (4.97)
\]
\[
\alpha_{lf} = \frac{2K_{lf}^2Q}{A_{lf}} + \frac{\frac{gA_{lf}}{B_{lf}} \frac{K_{lf}^2Q^2}{A_{lf}}}{Q} \quad (4.98)
\]
\[
\alpha_{rf} = \frac{2K_{rf}^2Q}{A_{rf}} + \frac{\frac{gA_{rf}}{B_{rf}} \frac{K_{rf}^2Q^2}{A_{rf}}}{Q} \quad (4.99)
\]
\[ \beta_{cn} = gA_{cn}(S_{f, cn} - S_{o, cn}) + M_{L1} + M_{L2} \] (4.100)

\[ \beta_{lf} = gA_{lf}(S_{f, lf} - S_{o, lf}) \] (4.101)

\[ \beta_{rf} = gA_{rf}(S_{f, rf} - S_{o, rf}) \] (4.102)

The friction slope terms in equations 4.100, 4.101 and 4.102, can be obtained from the Manning friction formula or equation 4.43. The momentum equation, equation 4.55 is transformed to equation 4.96, which has two parameters related to cross-sectional area and discharge of the channel. Therefore, equation 4.96 can be solved relatively easily by using a numerical solution subject to initial and boundary conditions.

In order to solve the governing equations, an explicit finite difference method similar to the one used to solve the governing equations for the channel, equations 4.44 thru 4.50, will be used and defined through the following relationship:

\[ Q_{i}^{j+1} = Q_{i}^{j} - (Q_{i}^{j} - Q_{i-1}^{j})\left(\alpha_{lc, i}^{j} \frac{\Delta t}{\Delta x_{cn}} + \alpha_{lf}^{j} \frac{\Delta t}{\Delta x_{lf}} + \alpha_{tr, i}^{j} \frac{\Delta t}{\Delta x_{rf}}\right) - \Delta t(\beta_{lc}^{j} + \beta_{lf}^{j} + \beta_{tr, i}^{j}) = 0 \] (4.103)

\[ A_{i}^{j+1} = A_{i}^{j} - (Q_{i}^{j+1} - Q_{i-1}^{j+1}) \Delta t \left(\frac{K_{cn}}{\Delta x_{cn}} + \frac{K_{lf}}{\Delta x_{lf}} + \frac{K_{tr, i}}{\Delta x_{rf}}\right) + \Delta t(q_{L1} + q_{L2}) \] (4.104)

\[ A_{lc}^{j+1} = K_{cn}A_{l,c}^{j+1} \] (4.105)

\[ A_{lf}^{j+1} = K_{lf}A_{l,f}^{j+1} \] (4.106)

\[ A_{tr, i}^{j+1} = K_{tr, i}A_{t, r, i}^{j+1} \] (4.107)
\[ Q_{i cn}^{j+1} = K_{cn} Q_{i}^{j+1} \]  
\[ Q_{i lf}^{j+1} = K_{lf} Q_{i}^{j+1} \]  
\[ Q_{i rf}^{j+1} = K_{rf} Q_{i}^{j+1} \]  

Where

\[ a_{i cn}^{j} = \frac{2K_{cn}^2 Q_{i}^{j}}{A_{i cn}^{j}} + \frac{3}{2} \frac{gA_{i cn}^{j} \kappa_{cn}^2 Q_{i}^{j2}}{B_{n k}^{j2} A_{i cn}^{j2}} \]  
\[ a_{i lf}^{j} = \frac{2K_{lf}^2 Q_{i}^{j}}{A_{i lf}^{j}} + \frac{\left( gA_{i lf}^{j} \kappa_{lf}^2 Q_{i}^{j2} \right)}{B_{l f}^{j2} A_{i lf}^{j2}} \]  
\[ a_{i rf}^{j} = \frac{2K_{rf}^2 Q_{i}^{j}}{A_{i rf}^{j}} + \frac{\left( gA_{i rf}^{j} \kappa_{rf}^2 Q_{i}^{j2} \right)}{B_{r f}^{j2} A_{i rf}^{j2}} \]  

\[ \beta_{i cn}^{j} = gA_{i cn}^{j} \left( S_{fcn_{i}}^{j} - S_{ocn} \right) + M_{L1} + M_{L2} \]  
\[ \beta_{i lf}^{j} = gA_{i lf}^{j} \left( S_{f lf_{i}}^{j} - S_{olf} \right) \]  
\[ \beta_{i rf}^{j} = gA_{r f_{i}}^{j} \left( S_{frf_{i}}^{j} - S_{orf} \right) \]
If the volume of water in the reach segment has filled the channel and is in the floodplain, the depth of flow is calculated using the following equation:

\[
y^{i+1}_l = y_{bnk,l} + \sqrt{\frac{2(A^{i+1}_l - A_{ch,l})}{(z_{lf,l} + z_{rf,l})} + \left(\frac{B_{wld,l}}{(z_{lf,l} + z_{rf,l})}\right)^2 - \left(\frac{B_{wld,l}}{(z_{lf,l} + z_{rf,l})}\right)}
\]  

(4.117)

And

\[
B_{wld,l} = B_{wlf,l} + B_{wrf,l} + B_{bnk,l}
\]  

(4.118)

In equation 4.117, \(y^{i}_l\) is the total depth of flow within the channel and floodplain; \(A^{i}_l\) is the total cross-sectional area of flow for the channel and floodplain; and \(B_{wld,l}\) is the bottom width of the floodplain.

Each pair of \(\alpha^{i}_l\) and \(\beta^{i}_l\) values can be readily calculated from equations 4.46 and 4.47 for the channel alone, and equations 4.111 through 4.116 for the channel and floodplain using the known initial and boundary data at starting point \((i, j)\) then one can obtain \(Q^{i+1}_l\) from equation 4.45 or equation 4.103. In addition, using \(Q^{i+1}_l\), \(A^{i+1}_l\) can be calculated from equation 4.48 or equation 4.104. Furthermore, the flow area \((A)\) and discharge \((Q)\) for the channel, left floodplain and right floodplain are calculated using equations 4.105 thru 4.107 and equations 4.108 thru 4.110 respectively. This technique is repeated for successive values of \((i, j)\). In the proposed simplified dynamic model discharge leaves the downstream boundary and enters the upstream boundary of the next segment and establishes the upstream boundary condition for flow at that segment.

Explicit models, although relatively simple compared to implicit models, have a restriction in the size of the computational time step \((At)\) in order to achieve numerical stability. In the Stoker scheme the maximum permissible time step \((At)\) is given by the following inequality (Garrison et al., 1969):
\[ \Delta t = \frac{\Delta x}{\sqrt{V + \frac{g A^2}{2c_1} g n^2 |V| \Delta x}} \]  \hspace{1cm} (4.119)

In which \( n \) is the Manning roughness coefficient, \( c_1 = 1.0 \) in metric units and \( c_1 = 2.21 \) in English units, and \( R \) is the hydraulic radius. The first two terms in the denominator are associated with the well known Courant condition for stability of explicit schemes in frictionless flow. The third term accounts for the effects of friction. An inspection of equation 4.119 indicates that the computational time step is substantially reduced as the hydraulic depth \((A/B)\) increases. Therefore, in large rivers it is not uncommon for time steps on the order of a few minutes or even seconds to be required for numerical stability even though the flood wave may vary gradually. In addition, in explicit schemes there is the requirement of equal \( \Delta x \) distance steps. This is typically solved by sub-dividing each reach into a series of equally spaced sub-reaches and the outflow hydrograph from the upstream reach becomes the inflow hydrograph for the downstream reach.

### 4.3.2 Initial Conditions

In order to start the transient solution, initial values of the unknowns including, discharge and water surface elevation are to be specified along the one-dimensional open channel domain. The initial conditions can be obtained from, field data, or the solution of steady, non-uniform flow equation. In any case, the initial conditions are given as:

\[ Q(x, 0) = Q_o(x) \]  \hspace{1cm} (4.120)

\[ h_{wl}(x, 0) = h_{wl_o}(x) \]  \hspace{1cm} (4.121)

Where \( Q_o \) and \( h_{wl_o} \) represent the discharge and the water surface elevation within the channel at the beginning of the simulation, respectively. Using a known point of discharge from field data along the watershed or an adjacent watershed of similar physiology the model, computes the baseflow or initial flow along the channel reaches of a watershed using the following relationship:
Where $Q_2$ is the unknown point of discharge along a watershed, $Q_1$ is the known point of discharge along the watershed, $A_2$ is the cumulative drainage area of the unknown point of discharge, $A_1$ is the cumulative drainage area of the known point of discharge and $n$ is a user specified value that is regionally specific and obtained through regional regression analysis. For Ontario values of $n$ can be found in the “Floodplain Management for Ontario Technical Guidelines” by the Ministry of Natural Resources, 1986.

Once the baseflows or initial discharge values in the stream channels or reaches have been determined, the discharge at each cross-section along the length of the channel can be solved as follows:

$$Q_{i+1} = Q_i + (q_{L1} + q_{L2}) \cdot \Delta x_i \quad i = 1, 2, 3, \ldots N - 1 \quad (4.123)$$

Where $Q_i$ is the assumed steady flow at the upstream boundary at time $t=0$ and $q_{L1}$ is the known average lateral seepage inflow and $q_{L2}$ is the known average lateral overland inflow along each $\Delta x$ reach at $t=0$. The water surface elevations, $h_{wl,i}$, is computed according to the following steady flow simplification of the momentum equation:

$$\left( \frac{Q^2}{A} \right)_{i+1} - \left( \frac{Q^2}{A} \right)_i + g \cdot \left( \frac{A_{i+1} + A_i}{2} \right) \cdot \left( h_{wl,i+1} - h_{wl,i} + \Delta x_i \cdot \left( \frac{s_{f,i+1} + s_{f,i}}{2} \right) \right) = 0 \quad (4.124)$$

The computations proceed in the upstream direction ($i=N-1, \ldots, 3, 2, 1$) for subcritical flow and in the downstream direction for supercritical flow. The starting water surface elevation $h_{wl,N}$ can be specified or obtained from the appropriate downstream boundary condition for the discharge $Q_N$ obtained from equation 4.123.
4.3.3 External Boundary Conditions

The routing model is capable of modelling a network of channels or a single channel or reach. The tree-like network is composed of several upstream and internal channels and a single downstream channel. Therefore, for a dendritic tree-like network of channels, the model can accommodate several upstream boundary conditions and a single downstream boundary condition. In this regard, the model does not solve looped channel networks. Reaches which are not affected by downstream flows and are modelled as a single channel, require only an upstream boundary condition.

Values of the unknowns at external boundaries of the channel must be specified in order to obtain solutions to the Saint Venant Equations. At the upstream boundary, a discharge or stage hydrograph can be used as the boundary condition. The hydrograph is expressed as a time series of discharge or stage and is not affected by downstream flows. For an explicit numerical solution, which is forward in space and backward in time, only one boundary condition is necessary to be specified. Within the GFLOOD model the user can specify more than one boundary condition. All values of the discharge variable $Q(x, t)$, can be determined if another equation is provided for each boundary and solved in connection with the specified boundary condition. The extra equation for both external boundaries is derived by integrating the continuity equation, for the first ($i=1$) and the last ($i=N-1$) computational time and distance steps as (Jing and Fread, 1997):

$$\Delta t \left( \overline{Q_{i+1}} - \overline{Q_i} \right) + \Delta x \left[ \overline{(A + A_o)^{j+1}} - \overline{(A + A_o)^j} \right] - 2\overline{q}\Delta x \Delta t = 0.$$  

Where $i=1$ for the upstream boundary, and $i=N-1$ for the downstream boundary ($N$ is the total number of computational sections); the bar stands for a temporal averaging, and the underline stands for a spatial averaging, for example, $(A + A_o) = 0.5[(A + A_o)_i + (A + A_o)_{i+1}], Q = 0.5(Q^j + Q^{j+1})$; and $q$ is the time-averaged lateral inflow or outflow within the computational distance step. The above equations can also be applied to upstream and downstream reaches of a hydraulic structure (internal boundary such as a dam or bridge), together with an appropriate internal boundary structure and water surface elevations both upstream and downstream of the structure.
The boundary condition, at the downstream boundary can be defined as a discharge or stage time-series hydrograph. In addition, it is also possible to define the downstream boundary condition as a single-valued rating curve, a looped rating curve or a critical depth section. The single valued rating curve relates a particular stage value to a corresponding discharge value and can be expressed by using linear interpolation within a table of stage-discharge data:

\[
Q_{N,t} = Q^k + \frac{Q^k-Q^{k+1}}{h_{wl}^{k+1}-h_{wl}^k} \cdot \left( h_{wl,N} - h_{wl}^k \right)
\]  

(4.125)

Where \(Q^k, Q^{k+1}, h_{wl}^{k+1}\) and \(h_{wl}^k\) are consecutive tabular data sets of the rating curve and \(h_{wl,N}\) is the stage at the downstream boundary. The model also accepts a looped rating curve as the downstream boundary condition. A looped rating curve relates a stage value to several possible discharge values depending on the hydraulic conditions of the channel and can be expressed using the Manning’s Equation:

\[
Q_{N,t} = \frac{1}{n} \cdot A \cdot (R)^{2/3} \cdot S_f^{1/2}
\]  

(4.126)

Where \(S_f\) is given by the modified momentum equation as:

\[
S_f = -\frac{1}{gA} \frac{\partial Q}{\partial t} - \frac{1}{gA} \frac{\partial (Q^2/A)}{\partial x} - \frac{\partial h_{wl}}{\partial x}
\]  

(4.127)

Furthermore, it is possible to use critical depth as the downstream boundary condition when the most downstream point of the modelling domain is a controlling structure such as a weir or a waterfall. In this particular case, the critical depth is related to the critical discharge using the following equation:

\[
Q_{N,t} = \sqrt{\frac{g}{B}} A^{3/2}
\]  

(4.128)

Where \(B\) is the cross-sectional top-width of the channel.
4.3.4 Internal Boundary Conditions

Any two or more channels intersecting within a channel network form a junction where internal boundary conditions are specified to satisfy the mass and energy balances. The proposed model does not allow for looped networks and requires that there is always a single outflow channel from a junction. The mass balance equation at a junction can therefore, be specified as:

$$\sum_{k=1}^{m} Q_k - Q_o = \frac{\partial S}{\partial t}$$  \hspace{1cm} (4.129)

Where $m$ is the total number of inflowing channels to the junction, $Q_k$ is the discharge at the end of the $k^{th}$ inflowing channel to the junction, $Q_o$ represents the discharge at the beginning of the outflowing channel from the junction, and $dS/dt$ corresponds to the change in storage within the junction. For many applications, it is common practice to assume that the change in storage within a junction is negligible compared to the change in storage within a channel (Akan and Yen, 1981; Fread, 1993; Jha et al., 2000; and Aral and Gunduz, 2003). As a result, the mass balance equation reduces to a simple continuity equation. The energy equation at the junction is written as:

$$(h_{wl})_k + \frac{V_k^2}{2g} = (h_{wl})_o + \frac{V_o^2}{2g} + h_T \quad k = 1, 2, ..., m$$  \hspace{1cm} (4.130)

Where $(h_{o1})_t$ and $V_k$ are the stage and flow velocity at the end of the $k$th inflowing channel to the junction, $(h_{ojo})_t$ and $V_o$ are the stage and flow velocities at the beginning of the outflowing channel from the junction, and $h_T$ is the total headloss in the junction. When all the flows in all the branches joining a junction are subcritical and the head lost in the junction is negligible, the equation simplifies to:

$$(h_{wl})_k = (h_{wl})_o \quad k = 1, 2, ..., m$$  \hspace{1cm} (4.131)

Equation 4.131 is commonly used in modelling channel networks (Akan and Yen, 1981; Fread, 1993; Jha et al., 2000; and Aral and Gunduz, 2003).
Highway and railroad bridges are treated as internal boundary conditions within the routing model. Because the flow through a bridge, spillway, breach or waterfall is rapidly varying rather than gradually varying therefore, the Saint Venant Equations no longer apply. Two equations are required to define an internal boundary condition, because two unknowns ($Q$ and $h$) are added at the internal boundary. For example, when modelling a bridge crossing and/or opening $Q_{br}$, flow over the embankment $Q_{em}$, and flow through a breach, $Q_{b}$, the two internal boundary conditions are:

$$Q_{i}^{j+1} = Q_{br} + Q_{em} + Q_{b}$$  \hspace{1cm} (4.132)$$

And

$$Q_{l+1}^{j+1} = Q_{l}^{j+1}$$  \hspace{1cm} (4.133)$$

The breach flow $Q_{b}$, can be computed using a combination of the formulas for a broad-crested rectangular weir, gradually enlarging as the breach widens, and a trapezoidal weir for the breach end slopes (Fread, 1980):

$$Q_{b} = 3.1B_{w} \cdot t_{b} \cdot C_{v} \cdot K_{s} \cdot \frac{(h-h_{b})^{1.5}}{T} + 2.45 \cdot z \cdot C_{v} \cdot K_{s} \cdot (h-h_{b})^{2.5}$$  \hspace{1cm} (4.134)$$

Where $t_{b}$ is the time after the breach starts forming, $C_{v}$ is the correction factor for velocity of approach, $K_{s}$ is the submergence correction for tail water effects on weir outflow, and $h_{b}$ is the elevation of the breach bottom. Flow through a bridge opening is defined by a rating curve or by an orifice equation:

$$Q_{br} = C_{b} \sqrt{2gA_{i+1}^{l+1} \cdot (h_{i}^{j+1} - h_{l+1}^{j+1})^{1/2}}$$  \hspace{1cm} (4.135)$$

Where $C_{b}$ is the bridge coefficient; the embankment overflow $Q_{em}$ is defined by a broad crested weir formula:
\[ Q_{em} = K_{em} \cdot L_{em} \cdot C_{em} \cdot \left( h_{t+1} - h_{em} \right)^{3/2} \]  

(4.136)

Where \( C_{em}, L_{em}, \) and \( K_{em} \) are the discharge coefficient, length of embankment, and submergence correction factor, and \( h_{em} \) is the elevation of the top of the embankment. Reservoir outflow consists of both the breach outflow \( Q_b \) and spillway outflow \( Q_s \):

\[ Q = Q_b + Q_s \]  

(4.137)

The breach outflow can be computed using equation 4.135, and the spillway outflow can be computed using the following equation (Fread, 1980):

\[ Q_s = C_s \cdot L_s \cdot (h - h_s)^{1.5} + \sqrt{2g} \cdot C_g \cdot A_g \cdot (h - h_g)^{0.5} + C_d \cdot L_d \cdot (h - h_d)^{0.5} + Q_t \]  

(4.138)

Where \( C_s \) is the uncontrolled spillway discharge coefficient, \( L_s \) is the uncontrolled spillway length, \( h_s \) is the uncontrolled spillway crest elevation, \( C_g \) is the gated spillway discharge coefficient, \( A_g \) is the area of gate opening, \( h_g \) is the centre-line elevation of the gated spillway, \( C_d \) is the discharge coefficient for flow over the dam crest, \( L_d \) is the length of the crest, \( h_d \) is the dam crest elevation, and \( Q_t \) is a constant outflow or a time series \( Q(t) \) of outflows specified by the user. Breach formation or the growth of the opening in the dam as it fails is specified by the slope of the breach \( z \), and the terminal width \( B_w \) for the bottom of the breach. The model assumes the breach bottom width starts at a point and enlarges at a linear rate until the terminal width is attained at the end of the failure time interval \( T \).

For culvert analysis, the user must specify a head loss rating curve for the culvert. The head loss rating curve can be developed using the Federal Highway Administration nomographs which are based on laboratory testing results. The results of the model are further enhanced if the culvert flow characteristics are analyzed externally (using FHWA methods) and imported as a head loss rating curve. The following equation is used to determine the headwater elevation for a culvert:
\[ HW = h_o + H - S_o \cdot L \]  
(4.139)

Where \( HW \) is the headwater elevation (m), \( h_o \) is the tail-water elevation (m), \( H \) is the total headloss through the culvert, \( S_o \) is the culvert barrel slope (m/m), and \( L \) is the length of the culvert (m).

For close conduits, the user has two options; first, if it is a single conduit that is open on both ends, the user can treat it as a culvert. For the second option, the flow is treated as pressurized flow from one section to another and/or as the flow changes with time. When the flow, passing through a section of closed conduit of any shape, completely submerges the section; the flow properties change from those of free-surface to pressurized flow. In the latter type of flow, disturbances in the flow are propagated at velocities many times greater than those for free surface flow. A technique which enables the Saint-Venant equations to properly simulate pressurized flow is included within the GFLOOD model. It follows the method known as the Preissmann slot, first described by Cunge and Wegner (1964) and described by Fread (1984b) for application of the Saint-Venant equations to unsteady flows in a network of storm sewers.

In this method, closed conduits are rectangular in shape, and the width \( B \), bottom elevation and top elevation are user-specified. When the top width diminishes to zero at the top of the closed conduit it is actually user-specified to have a very small top width \( (b^*) \) which extends vertically upward for at least one or more metres. Within the GFLOOD model this top width is extrapolated for elevations larger than the user-specified elevation; hence, the extrapolated top width is always \( b^* \) for all elevations since the user-specified top width is \( b^* \). Thus, by expressing the top width in this manner for closed conduits, the Saint-Venant equations properly simulate either free-surface or pressurized flow. In GFLOOD, flows may be simulated which are always free surface in some reaches where the sections are open while in other reaches with closed conduit sections, the flow may be initially pressurized or with time change from free surface to pressurized flow and vice versa.
4.3.5 Consideration of Transmission Losses and Channel Water Balance

The simplified dynamic model was modified to include transmission losses along the stream. Transmission losses (gains) include evaporation, seepage, diversions and (return flow). The methodology presented here, is the same methodology used in the SWAT model for accounting for transmission losses for a reach (Neitsch et al, 2005). Evaporation losses from the channel are calculated as follows (Neitsch et al, 2005):

\[ E_i = \text{coef}_{ev} \cdot E_o \cdot L_i \cdot \bar{W} \cdot f_r \Delta t \]  

(4.140)

Where \( E_i \) is the evaporation from the reach \((i)\) for the time step \((m^3)\), \(\text{coef}_{ev}\) is an evaporation coefficient, \(E_o\) is potential evaporation \((mm)\), \(L_i\) is the reach length \((km)\), \(\bar{W}\) is the average channel top width at cross-sections \(i\) and at \(i+1\) \((m)\), and \(f_r \Delta t\) is the fraction of the time step in which water is flowing in the reach. The evaporation coefficient is a calibration parameter and may vary between 0.0 and 1.0. A value of 0.69 is typically used (Maidment and Chow, 1988; Neitsch et al, 2005). The fraction of the time step in which water is flowing in the reach is calculated by dividing the travel time by the length of the time step. Replacing the first two terms in Equation 4.140, with the amount of precipitation, \(PCP_o\) \((mm)\) for the time step, one can determine the total volume of precipitation added to the reach during that time step. The travel time in each reach is calculated using the following equation:

\[ TT_i = \frac{\Delta x_i}{\bar{V}} \]  

(4.141)

Where \(TT_i\) is the travel time within reach \(i\) and \(\bar{V}\) is the average velocity between cross-sections \(i\) and \(i+1\).

Seepage losses from the reach were estimated with the following equation (Neitsch et al, 2005):

\[ T_{\text{loss},i} = K_{\text{eff},i} \cdot TT_i \cdot P_i \cdot L_i \]  

(4.142)
Where $T_{loss,i}$ are the reach $(i)$ seepage losses (m$^3$), $K_{eff,i}$ is the effective hydraulic conductivity of the reach (mm/hr), $TT_i$ is the flow travel time (hr), $P_i$ is the wetted perimeter (m), and $L_i$ is the reach length (km). Typical values of $K$ for various alluvium materials can range from 127 mm/hr (clean gravel and large sand) to 0.025 mm/hr (bed material with high silt clay content, Lane 1983). For perennial streams with continuous groundwater contribution, the effective conductivity will be zero. Table 4.9 lists hydraulic conductivity values for various bed materials. Transmissions losses from the main channel are assumed to enter bank storage or the deep aquifer. Dividing $T_{loss,i}$ by the reach length ($L_i$) and the travel time ($TT_i$) one can obtain the lateral seepage outflow from the reach ($q_{L1}$).

The amount of water entering bank storage on a given time step is calculated by using the following equation (Neitsch et al, 2005).

$$bnk_{in} = T_{loss,i} \cdot (1 - fr_{trans}) \quad (4.143)$$

Where $bnk_{in}$ is the amount of water entering bank storage (m$^3$), $T_{loss,i}$ is the seepage loss (m$^3$), and $fr_{trans}$ is the fraction of seepage losses partitioned to the deep aquifer.

The volume of water entering the reach from bank storage is calculated as follows (Neitsch et al, 2005):

$$V_{bnk} = bnk \cdot (1 - exp[-\alpha_{bnk}]) \quad (4.144)$$

Where $V_{bnk}$ is the volume of water added to the reach via return flow from bank storage (m$^3$), $bnk$ is the total amount of water within bank storage (m$^3$) and $\alpha_{bnk}$ is the bank flow recession constant or constant of proportionality.

Water storage within the reach at the end of the time step is calculated by using the following equation (Neitsch et al, 2005):

$$V_{stored,2} = V_{stored,1} + V_{in} - V_{out} - T_{loss,i} - E_i + PCP_i + div + V_{bnk} \quad (4.145)$$
Where $V_{stored}$ is the volume of water in the reach at the end of the time step (m$^3$), $V_{stored}$ is the volume of water in the reach at the beginning of the time step (m$^3$), $V_{in}$ is the volume of water flowing into the reach during the time step (m$^3$), $V_{out}$ is the volume of water flowing out of the reach during the time step (m$^3$), $T_{loss,i}$ is the volume of water lost from the reach via seepage outflow through the bed and sides (m$^3$), $E_i$ is the evaporation from the reach for the time step (m$^3$), $PCP_i$ is the amount of precipitation added to the reach for the time step (m$^3$), $div$ is the volume of water added or removed from the reach for the time step through diversions (m$^3$), and $V_{bnk}$ is the volume of water added to the reach via return flow from bank storage (m$^3$). The model treats the volume of outflow ($V_{out}$) as the net amount of water removed from the reach. As transmission losses, evaporation and other water losses for the reach segment are calculated, the amount of outflow to the next amount will equal the value obtained from equation 4.145. The total volume within the channel is estimated by summing all of the individual reach segment volumes within the channel. Equations 4.140 thru 4.145 are solved periodically by incorporating a variable time step. The flow routing equations are solved using a computational time step ($\Delta t$) of minutes and/or seconds and transmission loss equations, are solved using a time step ($\Delta t$) of days. Thereby, ensuring the computational efficiency of the equations.

<table>
<thead>
<tr>
<th>Bed Material Group</th>
<th>Bed Material Characteristics</th>
<th>Hydraulic Conductivity</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Very High Loss Rate</td>
<td>Very clean gravel and large sand</td>
<td>&gt; 127 mm/hr</td>
</tr>
<tr>
<td>2 High Loss Rate</td>
<td>Clean sand and gravel, field conditions</td>
<td>51-127 mm/hr</td>
</tr>
<tr>
<td>3 Moderately High Loss Rate</td>
<td>Sand and gravel mixture with low silt-clay content</td>
<td>25-76 mm/hr</td>
</tr>
<tr>
<td>4 Moderate Loss Rate</td>
<td>Sand and gravel mixture with high silt-clay content</td>
<td>6-25 mm/hr</td>
</tr>
<tr>
<td>5 Insignificant to Low Loss Rate</td>
<td>Consolidated bed material; high silt-clay content</td>
<td>0.025-2.5 mm/hr</td>
</tr>
</tbody>
</table>

**4.4 Evaluation of the Simplified Dynamic Routing Model**

Two separate reaches of the Credit River watershed were used to evaluate the simplified dynamic model. In addition, a distributed routing model, the diffusion wave model was tested on
these reaches and the results compared against the simplified dynamic model and the more complex dynamic wave model. For distributed routing models, parameters are obtained from channel cross-sections and other physical data. Several routing methods have been developed based on the complete Saint Venant equations for gradually varied unsteady channel flow (de Saint-Venant, 1871). In the present study, simplified dynamic model, along with the diffusion wave and dynamic wave models found within the FLDWAV model were applied in a series of simulation exercises involving two reaches of the Credit River Watershed. In addition, the simplified dynamic model was modified to account for transmission losses, due to seepage outflow, evaporation losses, diversions and return flow along the channel reaches.

For the reach between Melville and Cataract, the reach was divided into 4 sub-reaches. All 4 sub-reaches were characterized as triangular shapes based on field surveyed cross-sections for each sub-reach. For the first reach the side slope was 2:1 (H:V); for the second and third reach the side slope was 5:1(H:V); and for the fourth reach the side slope was 15:1 (H:V). For the simplified dynamic model each sub-reach was further subdivided into 11 sub-reaches to maintain computational stability and ensure convergence of the numerical solution, $\Delta x = 462, 234, 171$ and 148 m for sub-reaches 1, 2, 3 and 4 respectively. In addition, $\Delta t$ values were set equal to 20 s, 20 s, 15 s and 15 s for sub-reaches 1, 2, 3 and 4 respectively, to maintain computational stability and to satisfy the courant condition. Since the FLDWAV program used an implicit numerical solution technique to solve the Saint Venant equations for both the diffusion wave and dynamic wave models, the courant condition does not apply here. However, within the FLDWAV manual it is recommended that the courant condition be used as a criterion for selecting the computational distance steps ($\Delta x$) in order to avoid truncation errors in the solution (Fread and Lewis, 1998). In addition, there is an automatic function within the FLDWAV model to compute the computational time step using the following criteria: the time of rise in the hydrograph ($T_r$) is divided by a factor ($M'$), where $6 \leq M' \leq 40$, usually a value of 20 is sufficiently large to produce computational time steps sufficiently small so as to minimize truncation errors (Fread and Lewis, 1998). As a result, the time step ($\Delta t$) is first automatically computed by the FLDWAV model and then the distance step ($\Delta x$) is computed automatically within the model using the following relationship: $\Delta x = \frac{\hat{c} T_r}{M'}$, in which $\hat{c}$ is the bulk wave celerity, and all other terms have been previously defined.
For the floodplain component, the side slope for the floodplain was set to 4:1 (H:V), for all sub-reaches. The Manning’s “n” value was set equal to 0.04 for the channel portion and 0.1 for the floodplain portion for each reach. The Manning’s “n” values were obtained from the routing reaches of the calibrated Guelph All-Weather Sequential Events Runoff (GAWSER) model for the Credit River watershed that was developed as part of the Adaptive Water Management Strategy for the Credit River (Credit Valley Conservation, 2002). For the simplified dynamic model, to account for seepage losses along the reach, the hydraulic conductivity of the reach (K) was set equal to 2.5 mm/h (bed material has a high silt-clay content, refer to Table 4.9); the fraction of seepage losses partitioned to the deep aquifer, (frns) was set equal to 0.25; and the bank flow recession constant (a) was set equal to 0.1 (Credit Valley Conservation, 2007). The upstream boundary condition was defined by the inflow hydrograph time series from Water Survey of Canada’s Melville gauge station (station no. 02HB013, Figure 3.2). Lateral inflow along the reach was estimated by the inflow hydrograph time series from Shaw’s Creek and Caledon Creek. The inflow hydrograph time series for Shaw’s Creek and Caledon Creek were based on flow transpositioning and water balance analyses using Water Survey Canada’s Melville and Cataract gauge stations and historical data from Water Survey of Canada’s now defunct Alton Branch gauge station (station no. 02HB019; Brown et al., 1974; Moin and Shaw, 1985; Singer et al., 1994; Schroeter, 1999).

For the reach between Boston Mills and Norval, the reach was subdivided into 7 sub-reaches. The first two sub-reaches were characterized as triangular shapes with a side slope of 10:1 (H:V). The subsequent sub-reaches were characterized as a trapezoidal section with a bottom width of 24 m and a side slope of 2:1 (H:V). These characterizations were based on field surveyed cross-sections for each sub-reach. The upstream boundary condition was defined by the inflow hydrograph time series from Water Survey of Canada’s at the Boston Mills gauge station (station no. 02HB018, Figure 3.2). For the simplified dynamic model each sub-reach was further subdivided into 11 sub-reaches to maintain computational stability and ensure convergence of the numerical solution, \( \Delta x = 324, 384, 180, 653, 111, 477 \) and 144 m for sub-reaches 1, 2, 3, 4, 5, 6 and 7 respectively. In addition, \( \Delta t \) values were set equal to 20 s, 20 s, 20 s, 20 s, 15 s, 12 s and 9 s for sub-reaches 1, 2, 3, 4, 5, 6 and 7 respectively to maintain computational stability and to
satisfy the courant condition. For the floodplain component, the side slope for the floodplain was set to 4:1 (H:V), for all sub-reaches. The Manning’s “n” value was set equal to 0.033 for the channel potion and 0.08 for the floodplain potion for each reach. The Manning’s “n” values were obtained from the routing reaches of the calibrated Guelph All-Weather Sequential Events Runoff (GAWSER) model for the Credit River watershed that was developed as part of the Adaptive Water Management Strategy for the Credit River (Credit Valley Conservation, 2002). For the simplified dynamic model, to account for seepage losses along the reach, the hydraulic conductivity of the reach (K) was set equal to 6 mm/h (Credit Valley Conservation, 2007). The fraction of seepage losses partitioned to the deep aquifer (fr_trans) was set equal to 1.0, since all seepage losses along this reach occur through the bed of the river and are partitioned to the deep aquifer (Credit Valley Conservation, 2007). The bank flow recession constant (α) was set equal to zero, since there was no return flow to bank storage (fr_trans=1.0) and a time series hydrograph was used to account for lateral seepage inflows along the reach. The downstream boundary condition was defined by the measured rating curve from Water Survey of Canada’s Norval gauge station (station no. 02HB025, Figure 3.2). Lateral inflow near the outlet at Norval was quantified by the inflow hydrograph time series from Water Survey of Canada’s Silver Creek gauge station (station no. 02HB008).

The assumptions used to represent a hydraulic phenomenon into mathematical relationships (or models) inevitably results some degree of inaccuracy. Some errors (inaccuracies) also occur for approximating a differential equation (mathematical equation) by finite differences. Truncation errors also occur in the numerical solution technique. Therefore, it is generally accepted that no single simulation model output will be identical in all respects to the physical phenomenon it aims to represent. However, it is required that this output be sufficiently close to its physical counterpart for the model simulation to be considered acceptable. The principle of goodness-of-fit is a measure of the degree to which the output conforms to the corresponding observed data. Goodness-of-fit techniques may range from purely subjective graphical (visual) methods to purely objective using statistical methods. These relationships usually portray the difference between the simulated and observed variables. Prior to any calibration or model application, the user should establish criteria for comparison. However, if too many criteria are used and are frequently switched, the assessment of the model performance becomes difficult.
Related to model simulation assessment is the graphical method of evaluation, plotting the observed and simulated hydrographs on the same graph. A visual comparison is then made of peak flows and hydrograph shapes. The importance of the graphical method should not be overlooked. Although it is usually thought to be highly subjective and difficult when comparing the performance of similar models, it provides a general appreciation of the model’s capabilities. However, it may be misleading in some cases when a visual check is made for the rising and recession parts of the hydrograph. A visual check may appear to have an excellent fit, but the actual relative error at a particular point might be high. In these instances the user can only resort to statistical goodness-of-fit techniques.

A statistical approach was followed to evaluate the performance of the four routing models. The main objective was to examine the closeness of simulated output more with the observed output by minimizing the objective function. In general, different goodness-of-fit criteria are weighted in favour of different hydrograph components, and may give more weight to a certain aspect of disagreement between observed and simulated results. Thus, there is no general criterion, and the one ultimately selected should depend on the objective of the modelling exercise (Chatila, 1992). The calibration and validation procedure along with the statistical criteria used to evaluate the performance of the different routing methods are described in the proceeding section.

The evaluation of a model’s behaviour and performance is commonly made and reported through comparisons of simulated and observed variables. Single and multi-response, efficiency criteria are commonly used by hydrologists to provide an objective assessment of the “closeness” of the simulated behaviour to the observed measurements (Beven, 2001). While there are a few efficiency criteria such as the Nash-Sutcliffe efficiency, coefficient of determination, and index of agreement that are frequently used in hydrologic and hydraulic modeling studies and reported in the literature, there are a large number of other efficiency criteria to choose from. Each criterion may place different emphasis on different types of simulated and observed behaviours. The selection of the best efficiency measures should reflect the intended use of the model and should concern model quantities which are deemed relevant for the study at hand (Janssen and Heuberger 1995). Krause et al, (2005), recommends using a combination of different efficiency
criteria complemented by the assessment of the absolute or relative error. The goal should be to provide good values for a set of measures, even if they are lower than single best realisations, to include the whole dynamics of the model results (Krause et al., 2005).

The calibration procedure adopted for calibrating the different routing models was a systematic approach to manual calibration. The systematic manual calibration relied on the measured and estimated values of the model parameters (Manning’s ‘n’ value) available from Credit Valley Conservation Authority. This ensured that a physically – meaningful set of initial parameters (Manning’s ‘n’ value) was used for the calibration. In the next step, a calibration scheme was defined, which systematically changed the value of a given parameter (Manning’s ‘n’ value) while keeping the remaining parameters constant. A 10% increase and/or decrease step was used to linearly change parameter values until the soft limits were reached. The soft limits were defined as the 25% - 175% of the initial parameter value (initial ±75%), which encompassed all reasonably expected values.

For the purposes of model testing and evaluation the following tools and criteria were used to test and evaluate the performance of the different routing models: comparison graph between simulated and observed measurements and statistical goodness-of-fit measurements. The comparison graph between simulated and observed measurements provides a plot of the observed and simulated output of variables at a specified location. For the reach between Melville and Cataract comparisons were made between the observed and simulated streamflows at the Cataract gauge station (Water Survey of Canada station, 02HB001). For the reach between Boston Mills and Norval comparisons were made between the observed and simulated streamflows at the Norval gauge station (Water Survey of Canada station, 02HB025).

A comprehensive summary of statistical performance measures used for the evaluation of the different routing models are provided in the literature by Sorooshian et al. (1983); American Society of Civil Engineers (1993); Gupta and Sorooshian (1998); and Moriasi et al. (2007). For the purposes of this study, nine (9) different statistical goodness-of-fit measures were used to evaluate the performance of the simplified dynamic model. These statistical goodness-of-fit measures included the coefficient of determination ($D$); coefficient of efficiency ($E$); modified
coefficient of efficiency ($E_1$); root mean square error ($RMSE$); systematic root mean square error ($RMSE_s$); unsystematic root mean square error ($RMSE_u$); coefficient of residual mass ($CRM$); relative bias ($RBIAS$); and percent difference in runoff volumes ($%Vol$). The first eight statistical goodness-of-fit criteria are described in detail in Section 4.2.1.3. The percent difference in runoff volumes ($%Vol$) is a measure of the differences between the observed and simulated runoff volumes for each ordinate of the hydrographs. Each ordinate of the computed hydrograph is given equal weighting. A percent ($%Vol$) difference in runoff volume of zero is considered to be a perfect fit; all computed hydrograph ordinates equal the observed ones. However, this is seldom the case. Percent ($%Vol$) difference in runoff volumes is calculated with Equation 4.146:

$$%Vol = \frac{\sum_{i=1}^{n} Vol_{P,i} - \sum_{i=1}^{n} Vol_{O,i}}{\sum_{i=1}^{n} Vol_{O,i}}$$

(4.146)

When testing the performance of a model a combination of graphical techniques and statistical indices should be used for model evaluation (Moriasi, 2007).

Model validation is a process of testing the model’s ability to simulate observed data other than those used for the calibration, with acceptable accuracy. During this process, calibrated model parameters are not subject to change and their values are kept constant during the simulation process. For the validation procedure in this study, the Manning’s “n” values for the different routing models were kept constant. The validation output for the different simulation events was assessed by flow comparison graphs and the statistical goodness-of-fit measures described above.

4.5 Application of the GFLOOD Model to the Welland River Watershed

The hybrid hydrologic – hydraulic model GFLOOD was applied to the Welland River Watershed. The entire Welland River Watershed is 880 square kilometres in size and flows in an easterly direction from its headwaters in Ancaster to its physical outlet at the Niagara River. The lower section of the river, from Port Davidson in the Township of West Lincoln downstream, has a very small gradient (only 4 m drop over 80 kilometres) and is prone to flooding. Many hydraulic structures (dams, bridges, weirs, and two siphons that carry the river under the
Welland Shipping Canal) have been built on this river and the hydraulics of flow (e.g. backwater effects and flooding) are further complicated due to fluctuating water levels in the Niagara River to support hydro initiatives.

The main objective of this study was to test and validate the hybrid model GFLOOD along the main branch of the Welland River between Binbrook Dam and the New Siphon. The GFLOOD was calibrated and validated for three (3) different periods of record: November 1st, 1999 to October 31st, 2000, November 1st, 2000 to October 31st, 2001, and November 1st, 2004 to October 31st, 2005. The main elements of the study included data collection, analysis and dissemination of results. In addition to GIS data that characterizes the watershed, on-line flow monitoring, and precipitation data were collected and analyzed as part of this study.

4.5.1 Description of the Welland River Watershed

The Welland River watershed drains an area of approximately 880 square kilometres in size. Located above the Niagara Escarpment the river flows in an easterly direction from its headwaters in Ancaster to its physical outlet at the Niagara River. The river itself is approximately 135 kilometres in length. The River falls approximately 82 metres in elevation over its entire course. The most significant vertical drop is a 78 metre drop which occurs over the first 55 kilometres with only a 4 metre drop on the lower 80 kilometres of the River. This slight gradient results in a meandering, sluggish river from Port Davidson in the Township of West Lincoln downstream. The streambed profile of main branch of the Welland River is illustrated in Figure 4.4.

The watershed is characterized by smooth, moderately sloping topography within the Haldimand Clay Plain. With the exception of some low moraine ridges in the north and a few sand plains in the Wainfleet and Fonthill areas, the drainage characteristics of the watershed vary from excellent to very poor with poor drainage being most prevalent. Some swampy areas are found around Wainfleet and Humberstone. The most common soil is clay or clay loam. Three man-made modifications to the river are of considerable interest. They are:
1. Old Welland Canal
2. New Welland Canal
3. Hydro operations at the Niagara River

Two inverted siphons have been built to convey the flow of the Welland River water beneath the Old and New Welland Ship Canals. These structures flow full under pressure and create backwater pools during floods in a manner similar to dams. Originally, the Welland River drained directly into the Niagara River at Niagara Falls. However, its flow is now diverted entirely into the Queenston-Chippawa Power Canal. In fact, the lower portion of the Welland River between the Chippawa Grass Island Pool and the Queenston-Chippawa Power Canal now flows in reverse, drawing Niagara River water to the Power Canal.

![Main Branch Welland River (Binbrook Dam to New Siphon) Diagram](image)

*Figure 0-4 Streambed profile of the main branch of the Welland River*

In addition to these man-made impacts, the Niagara Peninsula Conservation Authority has constructed a dam and reservoir in the headwaters of the Welland River (Binbrook Dam and Reservoir). The Reservoir was constructed for flood control and flow augmentation purposes. The NPCA owns and operates two weir structures on the system as well - the Oswego Creek
Weir in the Town of Dunnville and the Port Davidson Weir on the main River channel on the Wainfleet / West Lincoln border. These weirs were originally constructed for the purposes of supplying a water source for firefighting in rural areas and for irrigation. The watershed is divided into 18 subwatersheds, which are listed in Table 4.10.

The dominant land use within the Welland River watershed is agriculture. ‘Water’ constitutes the smallest area (<1%) in the watershed. Water is necessary for irrigation purposes (agricultural and recreational), assimilation of pollutants (from pollution control plants and industry), drinking water (i.e. private wells), sustaining fisheries, wildlife and terrestrial resources, and recreational pursuits. The population of the watershed relies heavily on this water resource to carry out daily activities.

Table 4.11 provides land use information for both the Welland River watershed, as a whole, and the Oswego Creek watershed (Philips Engineering Limited, 2001). Although there is some variance in percentages between the subwatershed and the watershed as a whole, there is a definite correlation in land use at the macro-scale (Welland River) and micro-scale (Oswego Creek). This correlation exists between all subwatersheds.

<table>
<thead>
<tr>
<th>Subwatershed</th>
<th>Drainage Areas (km²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Welland River East</td>
<td>20 km²</td>
</tr>
<tr>
<td>Lyon’s Creek</td>
<td>47 km²</td>
</tr>
<tr>
<td>Thompson Creek</td>
<td>10 km²</td>
</tr>
<tr>
<td>Welland River West</td>
<td>205 km²</td>
</tr>
<tr>
<td>Tee Creek</td>
<td>30 km²</td>
</tr>
<tr>
<td>Coyle Creek</td>
<td>43 km²</td>
</tr>
<tr>
<td>Drapers Creek</td>
<td>10 km²</td>
</tr>
<tr>
<td>Beaver Creek</td>
<td>72 km²</td>
</tr>
<tr>
<td>Sucker Creek</td>
<td>11 km²</td>
</tr>
<tr>
<td>Mill Creek</td>
<td>20 km²</td>
</tr>
<tr>
<td>Moore Creek</td>
<td>13 km²</td>
</tr>
<tr>
<td>Little Wolf Creek</td>
<td>10 km²</td>
</tr>
<tr>
<td>Wolf Creek</td>
<td>15 km²</td>
</tr>
<tr>
<td>Buckhorn Creek</td>
<td>23 km²</td>
</tr>
<tr>
<td>West Wolf Creek</td>
<td>14 km²</td>
</tr>
<tr>
<td>Oswego Creek</td>
<td>181 km²</td>
</tr>
<tr>
<td>Elsie Creek</td>
<td>25 km²</td>
</tr>
<tr>
<td>Forks Creek</td>
<td>131 km²</td>
</tr>
<tr>
<td><strong>TOTAL</strong></td>
<td><strong>880 km²</strong></td>
</tr>
</tbody>
</table>
### Table 0-11 Landuse information Welland River Watershed

<table>
<thead>
<tr>
<th>Landuse</th>
<th>Welland River</th>
<th>Oswego Creek</th>
</tr>
</thead>
<tbody>
<tr>
<td>Agriculture (high and low density)</td>
<td>61%</td>
<td>63%</td>
</tr>
<tr>
<td>Scrubland</td>
<td>18%</td>
<td>25%</td>
</tr>
<tr>
<td>Forest</td>
<td>12%</td>
<td>9%</td>
</tr>
<tr>
<td>Transportation</td>
<td>4%</td>
<td>3%</td>
</tr>
<tr>
<td>Residential</td>
<td>1%</td>
<td>&lt;1%</td>
</tr>
<tr>
<td>Recreational</td>
<td>&lt;1%</td>
<td>&lt;1%</td>
</tr>
<tr>
<td>Water (natural and man-made)</td>
<td>&lt;1%</td>
<td>&lt;1%</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>100%</strong></td>
<td><strong>100%</strong></td>
</tr>
</tbody>
</table>

#### 4.5.2 Factors Influencing Surface Water

The purpose of the surface water characterization is to describe the dominant watershed characteristics that influence surface water flow. Water in the creek is the result of precipitation that has fallen on the watershed over time. Water resulting from precipitation gains entry to the creek following three main paths: by directly falling on the creek surface, by running over the land surface to the creek (surface runoff) or by infiltrating into the ground and reappearing as groundwater discharge (springs or seeps) along the creek.

It is important to note that not all of the precipitation that falls on the watershed makes it to the creek. A portion of the precipitation that falls, returns to the atmosphere by evaporation from open water sources, or is used by plants through transpiration. A portion of the water infiltrates into the ground, and may leave the watershed and may discharge or is used by plants in an adjacent watershed.

The path water follows in a watershed will determine to a great extent how the watershed responds to precipitation. The local climate and physiography (surficial geology, topography and land use) are dominant factors that influence how water is delivered to the streams and rivers that form a watershed. Streamflow is the response to how water is delivered to the streams and creeks forming the drainage network of a watershed. Each of these factors needs to be considered when describing the surface water characteristics of a watershed.
Climate Setting

The climate of Southern Ontario is characterized as having warm summers, mild winters, a long growing season, and usually reliable precipitation. The climate within Southern Ontario differs somewhat from one location to the other and from one year to the next. Spatial variations are caused by the topography and varying exposure to the prevailing winds in relation to the Great Lakes.

The Regional Municipality of Niagara operates several rain and temperature gauges (seasonal gauges between April and November) in the Welland watershed. Climate data from the Welland Sewage Treatment Plant (STP) and the McLeod Pumping Station (PS) are available from the Region for the period 1995-2001 (hourly format) inclusive. In addition, there are 17 Meteorological Service of Canada (formerly Atmospheric and Environment Service) stations throughout Niagara Region. Long-term monitoring of climate data has been undertaken at the following Meteorological Service of Canada Stations, Hamilton Airport, Vineland RCS, St. Catharine’s Airport, Niagara Falls and Port Colborne. Hourly rainfall and climate data is available for the period 2000 to 2005, for each respective climate station (Meteorological Services Canada, 2005).

An analysis of historic records gives an average annual precipitation of 926 mm of which 16% (147 cm) appears as snowfall within the Welland River Watershed. Figure 4.5 illustrates the average annual precipitation for the above selected climate stations. Total annual precipitation tends to be greatest along the Lake Erie Shoreline (Fort Erie, Port Colborne) and least along the Lake Ontario Shoreline (Niagara-on-the-Lake, Lincoln and St. Catharines). The greatest precipitation amounts occur during the months of June, July August, September, November and December. The results are illustrated in Figure 4.6. The maximum temperature occurs during the month of July (23°C) and the minimum temperature occurs during the month of January (-5°C). The results are illustrated in Figure 4.7.

The mean annual evapotranspiration in the watershed is about 540 mm as deduced from isoheytal maps for southern Ontario (Brown et al., 1974; OMNR, 1984), which has been verified
from water balance analyses using observed streamflow data by Singer et al. (1994). Although August and September tend to be the wettest months, the annual maximum streamflows usually occur in the February to April period resulting from snowmelt or rainfall on frozen ground, or a combination of both. Although the precipitation is generally distributed throughout the years, during the summer period there is a net deficit in the amount of precipitation that falls and is lost through evapotranspiration. The potential evapotranspiration amounts (e.g. lake evaporation) are higher than the total precipitation input for May through August. Based on these results climate of the Welland River Watershed can be described as having warm summers, mild winters and a relatively long growing season (approximately 220 days) and reliable precipitation.

Figure 0-5 Average annual precipitation amounts for the Welland River Watershed (1971 to 2000)
Figure 0-6 Average monthly precipitation amounts Welland River Watershed (1971 to 2000)

Figure 0-7 Average monthly temperature values for the Welland River Watershed (1971 to 2000)
Streamflow

The effect of fluctuating water levels near the outlet creates significant reversal in the flow direction in the lower Welland River, which produces backwater conditions up to some 65 km along the Welland River. For this reason no streamflow gauges are located on the Lower Welland River. However, water level monitoring has occurred on the Lower Welland River for over 20 years, and streamflow monitoring has occurred along the Welland River over the past 40 years. A gauge installed by Water Survey of Canada (WSC) at Merritts Church in late 1957 (Station No. 02HA007) was renamed to “Welland River below Caistors Corners”. The total drainage area to the gauge is 230 km$^2$; this represents 26% of the total drainage area of the Welland River. Table 4.12 provides a summary of the available hydrometric data for the Welland River Watershed.

Although, many of these stations lie outside the period of record for hourly climate information (2000-2005); the hourly water level data can be beneficial in validating the overall hydraulic model. In October 1988 a second streamflow gauge became operational on a tributary of the Welland River, Water Survey of Canada Station 02HA024, Oswego Creek at Canboro. This station is located at the outlet of Oswego Creek, which is downstream of station 02HA007, Welland River below Caistor Corners. The total drainage area to the gauge is 80.7 km$^2$; this represents 10% of the total drainage area of the Welland River and 35% of the total drainage area of WSC station 02HA007.

Figure 4.8 illustrates the monthly streamflow at the Water Survey of Canada gauge 02HA007 (Welland River below Caistor Corners). According to the graph the peak discharge occurs in March and the low flow discharge occurs during the summer months July and August. Figure 4.9 illustrates the monthly streamflow at the Water Survey of Canada gauge 02HA024 (Oswego Creek at Canboro). According to the graph the peak discharge occurs in March and the low flow discharge occurs during the summer months between July and September.

The Oswego Creek at Canboro gauge and the Welland River below Caistors Corners gauge have been in operation concurrently since 1988. This makes it possible to ascertain the contributions
from different parts of the watershed relative to the total flow at the outlet to the Welland River from Oswego Creek. For example, Figure 4.10 gives the accumulative mean monthly flows in Oswego Creek to the Caistor Corners gauge, showing the relative contribution from the gauge along Oswego Creek. From Figure 4.10, the flows are highest during the spring freshet in March and late autumn and lowest during the summer months.

The influence of topography, geology and climate on the flows along the length of the Welland River is evident as one looks at the relative contributions from different parts of the watershed. For instance, in a watershed that is totally homogenous in terms of topography, geology and climate, one would expect the contributions from each part of the watershed relative to the total flow at the outlet to be in the same proportion as their contributing drainage areas. In this regard, the homogeneity in the Welland River for this portion of the watershed is clearly evident. For example, the drainage area for the Oswego Creek gauge represents 35% of the total drainage area of WSC station 02HA007, and yet it contributes to 30% of the flow.

Hydrographs of peak flows recorded in April of 2000 during the spring freshet at the Caistor Corners gauge and Oswego Creek gauge show a rise period of two (2) days (Figure 4.11). According to the graph the peaks are occurring at relatively the same time, indicating that the event is affecting more than one part of the watershed. However, less severe events, especially during the summer or fall period may produce a three day rise period as illustrated in Figure 5.9 for the September 27 to October 2, 2003 event. According to Figure 4.12 the peak flow at Oswego Creek is occurring 24 hours after the peak flow at Caistor Corners. In both examples the recession leg of the hydrographs is less steep and could last three (3) to four (4) days before the flow returns to normal.
### Table 0-12 Hydrometric stations along the Welland River Watershed

<table>
<thead>
<tr>
<th>Station</th>
<th>Period of Record</th>
<th>Data (format)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Welland River Below Caistor Corners</td>
<td>1957-present</td>
<td>Flow (Daily)</td>
</tr>
<tr>
<td></td>
<td>2000-2005</td>
<td>Flow (hourly)</td>
</tr>
<tr>
<td>Oswego Creek at Canboro</td>
<td>1988-present</td>
<td>Flow (Daily)</td>
</tr>
<tr>
<td></td>
<td>2000-2005</td>
<td>Flow (hourly)</td>
</tr>
<tr>
<td>Big Forks Creek</td>
<td>1990-1992</td>
<td>Water Levels (hourly)</td>
</tr>
<tr>
<td>Welland River at Wellandport</td>
<td>1992-1993</td>
<td>Flows (daily/hourly)</td>
</tr>
<tr>
<td>Welland River at Old Syphon</td>
<td>2001-2008</td>
<td>Water Levels (hourly)</td>
</tr>
<tr>
<td>Material Dock Grass Island Pool</td>
<td>1989-2006</td>
<td>Water Levels (hourly)</td>
</tr>
<tr>
<td>Welland River at Montrose Gauge</td>
<td>2000-2001</td>
<td>Water Levels (hourly)</td>
</tr>
<tr>
<td></td>
<td>2004-2005</td>
<td></td>
</tr>
<tr>
<td>Welland River at Binbrook Dam</td>
<td>1980-present</td>
<td>Water Levels (Daily)</td>
</tr>
</tbody>
</table>

---

**Welland River Below Caistor Corners**

![Graph showing monthly daily flows for the Welland River below Caistor Corners gauge station](image)

**Figure 0-8 Statistical monthly daily flow values for the Welland River below Caistor Corners gauge station**
**Figure 0-9** Statistical monthly daily flow values for the Oswego Creek at Canboro gauge station

**Figure 0-10** Accumulated mean monthly flow values at Caistor Corners and Oswego Creek at Canboro
Figure 0-11 Hydrographs for Welland River at Caistor Corners and Oswego Creek at Canboro for April 20-24, 2000

Figure 0-12 Hydrographs for Welland River at Caistor Corners and Oswego Creek at Canboro for September 27 to October 2, 2003
Further evidence for climate influences on the streamflow response of the Welland River can be seen in Figure 5.13, which gives the time-series of annual flows at the Caistors Corners gauge for the period 1958 to 2008 and the Oswego Creek gauge for the period 1989 to 2008. Here we see lower peak flows during the drought of the late 1980’s and higher peaks during the early 1990’s. An examination of the time of occurrence of maximum flows indicated that within the period of record for the Caistors Corners gauge (1958 to 2008) and the Oswego Creek gauge (1989 to 2008); 75% of the annual maximum flows in the Welland River occurred during the ‘spring freshet’ in the months of February, March and April, when flood flows result from snowmelt or a combination of rain and snowmelt on frozen ground conditions (Figure 5.14). 20% of the highest flows have occurred in December and January when early winter thaws and significant rainfalls contribute to high flows. Flood flows in the late summer and early fall period are typically caused by tropical storm systems, a period when the infiltration capacity for most soils is reduced to 25 to 30% of their mid-summer values. During this time the runoff potential is at its highest without a snow pack. Recall that the highest 24 hour rainfall totals that has occurred in the Welland River was during the August to November period.

Figure 5.15 gives the time-series annual minimum daily flows at the Caistors Corners gauge for the period 1958 to 2008 and the Oswego Creek gauge for the period 1989 to 2008. Generally, this plot shows some of the same climate variability that was evident in a similar plot for annual maximum flows. The highest minimum flow occurred in 1987 and the lowest minimum flows occurred in 1988 during the drought of the late 80’s, with recovery in baseflow in 1992. An examination of the time of occurrence of minimum low flows indicated that within the period of record for the Caistors Corners gauge (1958 to 2008) and the Oswego Creek gauge; 75% and 100% respectively of the annual minimum flows in the Welland River occurred during the summer months (June thru August; Figure 5.16).

4.5.2.1 Analysis of Recorded Peak Flows

A statistical analysis was carried out on 51 years (1958-2008) of annual maximum instantaneous discharge data recorded at the Caistor Corners gauge to estimate the frequency of high flows generated by 230 km² area of the Welland River Watershed. The annual maximum instantaneous
flows published by Water Survey of Canada were analyzed with the Consolidated Frequency Analysis (CFA88) program maintained and distributed by the Water Resources Branch of the Inland Waters Directorate of Environment Canada. Prior to 1969, flow records did not exist for maximum instantaneous flows, only maximum daily flows. From 1969 onward flow records included both maximum daily values and maximum instantaneous flows recorded within the 24 hour period. In addition, for the period 1969 to 2008 flow records of maximum instantaneous discharge values are missing for a number of years. The data set on maximum instantaneous flows from 1969 to 2008 contains 35 values out of a total of 39 years of record. Therefore, in order to fill in the data gaps for the period of record for maximum instantaneous flows and to extend the record back to 1958; a correlation exercise was undertaken for those years where both maximum daily and maximum instantaneous flows were available. The results of that correlation exercise and regression analysis are illustrated in Tables 4.13 to 4.15 and Figures 4.17 and 4.18.

![Time Series Maximum Daily Flows -- Welland River](image)

*Figure 0-13 Time series of maximum daily flows for the Welland River*
Figure 0-14 Occurrence of maximum daily flows for the Welland River

Figure 0-15 Time series of minimum daily flows for the Welland River
Figure 0-16 Occurrence of minimum daily flows Welland River Watershed

Table 0-13 Regression statistics for the correlation between maximum daily and maximum instantaneous flows for Water Survey of Canada Gauge 02HA007, Welland River Below Caistor Corners, Period of Record 1958-2008, 51 years of record

<table>
<thead>
<tr>
<th>Regression Statistics</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Multiple R</td>
<td>0.982</td>
</tr>
<tr>
<td>R Square</td>
<td>0.964</td>
</tr>
<tr>
<td>Adjusted R Square</td>
<td>0.962</td>
</tr>
<tr>
<td>Standard Error</td>
<td>3.199</td>
</tr>
<tr>
<td>Observations</td>
<td>21.000</td>
</tr>
</tbody>
</table>

Table 0-14 ANOVA parameters for the correlation between maximum daily and maximum instantaneous flows for Water Survey of Canada Gauge 02HA007, Welland River Below Caistor Corners, Period of Record 1958-2008, 51 years of record

<table>
<thead>
<tr>
<th>ANOVA Parameters</th>
<th>df</th>
<th>SS</th>
<th>MS</th>
<th>F</th>
<th>Significance F</th>
</tr>
</thead>
<tbody>
<tr>
<td>Regression</td>
<td>1</td>
<td>5155.47</td>
<td>5155.47</td>
<td>503.67</td>
<td>3.822x10^-33</td>
</tr>
<tr>
<td>Residual</td>
<td>19</td>
<td>194.48</td>
<td>10.2357</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Total</td>
<td>20</td>
<td>5349.95</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 0-15 Statistics for the correlation between maximum daily (X variable) and maximum instantaneous flows (Y) for Water Survey of Canada Gauge 02HA007, Welland River Below Caistor Corners, Period of Record 1958-2008, 51 years of record

<table>
<thead>
<tr>
<th></th>
<th>Coefficient</th>
<th>SE</th>
<th>t-stat</th>
<th>P-value</th>
<th>Upper 95%</th>
<th>Lower 95%</th>
<th>Upper 95%</th>
<th>Lower 95%</th>
</tr>
</thead>
<tbody>
<tr>
<td>Intercept</td>
<td>3.317</td>
<td>2.272</td>
<td>1.46</td>
<td>0.161</td>
<td>-1.437</td>
<td>8.072</td>
<td>-1.437</td>
<td>8.072</td>
</tr>
<tr>
<td>X-var.</td>
<td>1.042</td>
<td>0.046</td>
<td>22.443</td>
<td>0.000</td>
<td>0.945</td>
<td>1.139</td>
<td>0.945</td>
<td>1.139</td>
</tr>
</tbody>
</table>
Based upon the above regression analysis the equation used to relate the maximum instantaneous flows to the maximum daily flows is as follows:

\[ Q_{ins} = 1.042 * Q_{MaximumDaily} + 3.317 \]  

(4.147)

A similar analysis was undertaken for the Water Survey of Canada gauge station 02HA024, Oswego Creek at Canboro in order to estimate the high flows along this tributary of the Welland
River Watershed. Oswego Creek at Canboro gauge station has 18 years of data including maximum instantaneous flows and maximum daily flows. It should be noted however, that the results of the frequency analysis can be used only as approximate when extrapolated to the 50 or 100-year events given the short period of record at the Oswego Creek at Canboro gauge station. Out of those 18 years of record, 2 years have missing data for the maximum instantaneous flows. Therefore, in order to fill in the data gaps for the period of record for maximum instantaneous flows; a correlation exercise was undertaken for those years where both maximum daily and maximum instantaneous flows were available. The results of that correlation exercise and regression analysis are illustrated in Tables 4.16 to 4.18 and Figures 4.19 and 4.20.

Table 0-16 Regression statistics for the correlation between maximum daily and maximum instantaneous flows for Water Survey of Canada Gauge 02HA024, Oswego Creek at Canboro, Period of Record 1989-2008, 16 years of record

<table>
<thead>
<tr>
<th>Regression Statistics</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Multiple R</td>
<td>0.799</td>
</tr>
<tr>
<td>R Square</td>
<td>0.638</td>
</tr>
<tr>
<td>Adjusted R Square</td>
<td>0.593</td>
</tr>
<tr>
<td>Standard Error</td>
<td>5.654</td>
</tr>
<tr>
<td>Observations</td>
<td>10.000</td>
</tr>
</tbody>
</table>

Table 0-17 ANOVA parameters for the correlation between maximum daily and maximum instantaneous flows Water Survey of Canada Gauge 02HA024, Oswego Creek at Canboro, Period of Record 1989-2008, 16 years of record

<table>
<thead>
<tr>
<th>ANOVA Parameters</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>df</td>
<td>SS</td>
</tr>
<tr>
<td>Regression</td>
<td>1</td>
</tr>
<tr>
<td>Residual</td>
<td>8</td>
</tr>
<tr>
<td>Total</td>
<td>9</td>
</tr>
</tbody>
</table>

Table 0-18 Statistics for the correlation between maximum daily (X variable) and maximum instantaneous flows (Y) for Water Survey of Canada Gauge 02HA024, Oswego Creek at Canboro, Period of Record 1989-2008, 16 years of record

<table>
<thead>
<tr>
<th>Coefficient</th>
<th>SE</th>
<th>t-stat</th>
<th>P-value</th>
<th>Upper 95%</th>
<th>Lower 95%</th>
<th>Upper 95%</th>
<th>Lower 95%</th>
</tr>
</thead>
<tbody>
<tr>
<td>Intercept</td>
<td>3.787</td>
<td>4.715</td>
<td>0.803</td>
<td>0.445</td>
<td>-7.085</td>
<td>14.659</td>
<td>-7.085</td>
</tr>
<tr>
<td>X-var.,</td>
<td>0.992</td>
<td>0.264</td>
<td>3.757</td>
<td>0.006</td>
<td>0.383</td>
<td>1.602</td>
<td>0.383</td>
</tr>
</tbody>
</table>
Based upon the above regression analysis the equation used to relate the maximum instantaneous flows to the maximum daily flows for the Oswego Creek gauge is as follows:

\[ Q_{\text{ins}} \tan \theta = 3.767 + 0.992 \times Q_{\text{MaximumDaily}} \]  \hspace{1cm} (4.147)

Once the annual maximum instantaneous flow rates were determined for both stations the next step would be to carry out a frequency analysis using the CFA88 computer program. The input data for the CFA88 computer program consists of a series of annual maximum data that is
described by the year and month of occurrence and the annual maximum peak streamflow rate. In addition, the program requires the Water Survey of Canada gauge number, the drainage area in km$^2$, and the number of data points and whether historic information is being used.

The output includes the following:

- The ranked input series of annual maximum flood flows with high and low outliers and empirical probabilities;
- Estimates of population statistics and distribution parameters;
- Streamflow rates for various return periods; and
- Plots of the frequency curves and a display of non-parametric data.

The program allows the user to conduct the following:

- Enter, modify (add, delete or change) and save data sets for future use;
- Perform non-parametric tests for homogeneity, trend, independence and randomness;
- Perform tests for low and high outliers; and
- Determine T year events for the straight forward case, samples with historic information, and samples with low outliers, samples with zero values. In addition, any data sets can be combined.

The following distribution probability density functions can be computed with the CFA88 computer program:

1. Generalized Extreme Value Distribution Type I (GEV);

$$f(x) = \alpha \exp\left[-\alpha(x - \mu) - \exp^{-\alpha(x - \mu)}\right]$$  \hspace{1cm} (4.148)

2. Three Parameter Log Normal Distribution (3PLN);
\[ f(x) = \frac{1}{(x - \varepsilon)\sigma_y \sqrt{2\pi}} \exp \left[-\left(\frac{\ln(x - \varepsilon) - \mu_y}{\sigma_y}\right)^2 / 2\sigma_y^2 \right] \]  

(4.149)

3. Log Pearson Type III (LP3); and

\[ f(x) = \frac{(\ln x - \varepsilon)^{\lambda - 1}}{\beta} \exp \left[-\frac{\ln x - \varepsilon}{\beta}\right] \frac{\exp \left[-\frac{\ln x - \varepsilon}{\beta}\right]}{\beta} x \Gamma(\lambda) \]  

(4.150)

4. Wakeby Distribution density function

\[ x = -a(1 - F)^b + c(1 - F)^{-d} + e \]  

(4.151)

Where \( F \) is the probability not exceeding \( x \), \( e \) is a location parameter, \( a \) and \( c \) are scale parameters, and \( b \) and \( d \) are shape parameters.

The Cunnane plotting position that follows is used to plot the data on probability paper:

\[ P(X \geq x) = \frac{m - 0.4}{N + 0.2} \]  

(4.152)

\[ T = \frac{1}{P(X \geq x)} = \frac{N + 0.2}{m - 0.4} \]  

(4.153)

Where \( m \) equals the series rank of the value in ascending order, \( N \) is the sample size and \( T \) is the return period. The CFA88 program determines return period values for the four (4) frequency distributions mentioned above. The CFA88 computer program does not select a frequency distribution and the resultant return period annual maximum peak flow rates. The user must determine which frequency distribution best fits the data. Typically, a distribution is selected by reviewing the distribution statistics with the statistics of the data values and by comparing how closely the data plots with the distributions. The computer program prints out the data values and
plots the four distributions and ranked data values. Moin and Shaw (1986) found that for most
gauge stations in Southern Ontario the 3 Parameter Log Normal Frequency Distribution best fits
the data when the coefficient of skew is determined to be positive. If the coefficient of skew is
negative the Log Pearson Type III Frequency Distribution is used in lieu of the 3-Parameter Log
Normal Distribution. The results of the frequency analysis for each of the Distributions for the
Welland River below Caistor Corners gauge station are tabulated in Tables 4.19 and 4.20.

Table 0-19 Distribution statistics for the Water Survey of Canada Gauge 02HA007, Welland River below Caistor
Corners, Period of Record 1958-2004, 46 years of record

<table>
<thead>
<tr>
<th>Series</th>
<th>Mean</th>
<th>Standard Deviation</th>
<th>Coefficient of Variance</th>
<th>Coefficient of Skew</th>
<th>Coefficient of Kurtosis</th>
</tr>
</thead>
<tbody>
<tr>
<td>x-series</td>
<td>52</td>
<td>18.2</td>
<td>0.350</td>
<td>0.865</td>
<td>0.621</td>
</tr>
<tr>
<td>Lnx-series</td>
<td>3.892</td>
<td>0.347</td>
<td>0.089</td>
<td>-0.088</td>
<td>0.060</td>
</tr>
</tbody>
</table>

Table 0-20 Single station frequency analysis for the Water Survey of Canada Gauge 02HA007, Welland River Below
Caistor Corners, Period of Record 1958-2008, 51 years of record

<table>
<thead>
<tr>
<th>Return Period</th>
<th>Exceedance Probability</th>
<th>GEV</th>
<th>3PLN</th>
<th>LP3</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>0.50</td>
<td>49</td>
<td>49</td>
<td>49</td>
</tr>
<tr>
<td>5</td>
<td>0.20</td>
<td>65</td>
<td>66</td>
<td>66</td>
</tr>
<tr>
<td>10</td>
<td>0.10</td>
<td>76</td>
<td>76</td>
<td>76</td>
</tr>
<tr>
<td>20</td>
<td>0.05</td>
<td>89</td>
<td>90</td>
<td>89</td>
</tr>
<tr>
<td>50</td>
<td>0.02</td>
<td>99</td>
<td>100</td>
<td>98</td>
</tr>
<tr>
<td>100</td>
<td>0.01</td>
<td>109</td>
<td>110</td>
<td>107</td>
</tr>
</tbody>
</table>

The results of the frequency analysis for each of the Distributions for the Oswego Creek at
Canboro gauge station are tabulated in Tables 4.21 and 4.22.

Table 0-21 Distribution statistics for Water Survey of Canada Gauge 02HA024, Oswego Creek at Canboro, Period of
Record 1989-2008; 16 years of record

<table>
<thead>
<tr>
<th>Series</th>
<th>Mean</th>
<th>Standard Deviation</th>
<th>Coefficient of Variance</th>
<th>Coefficient of Skew</th>
<th>Coefficient of Kurtosis</th>
</tr>
</thead>
<tbody>
<tr>
<td>x-series</td>
<td>25.9</td>
<td>14.7</td>
<td>0.55</td>
<td>1.19</td>
<td>1.59</td>
</tr>
<tr>
<td>Lnx-series</td>
<td>3.12</td>
<td>0.55</td>
<td>0.18</td>
<td>-0.16</td>
<td>0.08</td>
</tr>
</tbody>
</table>
Table 0-22 Single station frequency analysis for Water Survey of Canada Gauge 02HA024, Oswego Creek at Canboro, Period of Record 1989-2008, 16 years of record

<table>
<thead>
<tr>
<th>Return Period</th>
<th>Exceedance Probability</th>
<th>GEV</th>
<th>3PLN</th>
<th>LP3</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>0.50</td>
<td>24</td>
<td>23</td>
<td>23</td>
</tr>
<tr>
<td>5</td>
<td>0.20</td>
<td>36</td>
<td>36</td>
<td>36</td>
</tr>
<tr>
<td>10</td>
<td>0.10</td>
<td>44</td>
<td>46</td>
<td>45</td>
</tr>
<tr>
<td>20</td>
<td>0.05</td>
<td>55</td>
<td>59</td>
<td>57</td>
</tr>
<tr>
<td>50</td>
<td>0.02</td>
<td>63</td>
<td>70</td>
<td>67</td>
</tr>
<tr>
<td>100</td>
<td>0.01</td>
<td>70</td>
<td>81</td>
<td>76</td>
</tr>
</tbody>
</table>

Based on the results of Tables 4.20 and 4.22, the three (3) Parameter Log Normal Distribution best fits the data for both the Welland River at Caistor Corners gauge station and Oswego Creek at Canboro gauge station. Therefore, the high flows along the Welland River Caistor Corners and Oswego Creek at Canboro gauge stations for the 2 thru 100-year return period flow rates are listed in Table 4.23.

Table 0-23 Two (2) thru 100-year peak flow rates for the Welland River at Caistor Corners gauge station and Oswego Creek at Canboro gauge station

<table>
<thead>
<tr>
<th>Return Period</th>
<th>Welland River at Caistor Corners (m³/s)</th>
<th>Oswego Creek at Canboro (m³/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2-year</td>
<td>49</td>
<td>23</td>
</tr>
<tr>
<td>5-year</td>
<td>66</td>
<td>36</td>
</tr>
<tr>
<td>10-year</td>
<td>76</td>
<td>46</td>
</tr>
<tr>
<td>20-year</td>
<td>90</td>
<td>59</td>
</tr>
<tr>
<td>50-year</td>
<td>100</td>
<td>70</td>
</tr>
<tr>
<td>100-year</td>
<td>110</td>
<td>81</td>
</tr>
</tbody>
</table>

A comparison of unit area flood flows for the two locations within the Welland River watershed is presented in Figure 4.21. According to Figure 4.21, the unit area flood flows are much higher in Oswego Creek at Canboro relative to the Welland River at Caistors Corners. Although the 2 and 5 year unit area flood flows for the Oswego Creek gauge are similar in magnitude to those for the Caistors Corners gauge, the 10 to 100 year unit area flood flows are much higher. These higher flood flows likely reflect differences in precipitation amounts. Although Oswego Creek and the Welland River upstream of Caistors Corners receive equal amounts of snowfall (161 cm versus 158 cm, respectively); however, Oswego Creek gets about 1003 mm of precipitation.
annually, whereas the Welland River upstream of Caistors Corners receives about 10% less precipitation (910 mm).

4.5.2.1 Analysis of Recorded Low Flows

The objectives of the low flow frequency analysis are to determine the minimum flow rates for various durations and return periods for the Welland River below Caistor Corners gauge station and Oswego Creek at Canboro gauge station. The low flow analysis determines streamflow rates that can be expected to occur on average once every 1.005 through 500 years. A separate low flow frequency analysis was carried out for each duration of 1, 7, 15, 30 and 60 days. The minimum streamflow rates for the various return periods are best determined using data recorded over long periods (50 to 100 years) from the basin where the predicted runoff rates are required. Less accurate results would be expected for shorter periods of record or flow rates recorded on adjacent basins. It appears that the mainly clay based Welland Watershed produces extreme low flow periods, generally during the summer months of June, July, and August.

The analyses involved first the determination of a series of annual minimum mean daily flow rates for durations 1, 7, 15, 30, and 60 days and then performed a frequency analysis for each
data series. A spreadsheet was used to extract the minimum 1, 7, 15, 30 and 60 day flow values that occur within each calendar year of the recorded data. The values are then averaged to generate an annual minimum mean daily flow rate for each duration. The minimum values must occur over a consecutive period of time. For example, each non leap year will contain 358 seven (7) day mean daily flow rates. The minimum seven (7) day mean daily rate for each year will be used in the frequency analysis.

The frequency analysis was carried out with the aid of the Low Flow Frequency Analysis Package (LFA) computer program maintained and distributed by the Water Resources Branch of Inland Waters Directorate of Environment Canada. The input data for the LFA program consists of a series of annual minimum data. The output includes the following:

- The ranked input and cumulative probabilities;
- Population statistics; and
- Estimates of the minimum annual mean daily flow rates for the selected return periods.

Each annual series of minimum runoff rates are ranked in ascending order. The Cunnane plotting position is then used to assign a cumulative probability occurrence to runoff value.

The LFA program fits the annual minimum data series to a Gumbel III distribution and determines values for return periods ranging from 2 to 100 years. The following equation describes the Gumbel III Distribution density function:

\[
f(x) = \frac{k}{\nu - \xi} \left( \frac{\omega - x}{\omega - \nu} \right)^{k-1} \exp \left[ -\left( \frac{\omega - x}{\omega - \nu} \right)^k \right]
\]

The Gumbel III distribution has been considered to be acceptable for drought analysis by the Water Resources Branch although numerous distributions are available with which to determine the return period values. The results of the low flow frequency analysis are tabulated in Tables 4.24 and 4.25 for the Welland River below Caistor Corners gauge station and the Oswego Creek at Canboro gauge station respectively.
Statistical analysis further showed that on the average, 42 days in every year, the flow drops to 0.001 m³/s or below specifically during the drought years of the 1960s. Although such low flows may only occur for a day or so, once every two or three years on average such dry conditions can last seven consecutive days.

There are no low flow records available on the Welland River flow downstream of the Caistor Corners gauge with the exception of the gauge station on Oswego Creek at Canboro. At this station zero flows have been recorded over several weeks during the months of July, August and September for the years 2001 and 2002. It can be safely assumed that the few tributaries and drains discharging to the Welland River downstream of Oswego Creek have experienced similar extreme low flow conditions.

A comparison of unit area low flows in Oswego Creek with the Caistors Corners gauge on the Welland River is presented in Figure 4.22. The unit area low flows for the Caistors Corners gauge are higher due to the fact that many of the tributaries including Oswego Creek experience extreme low flow conditions particularly during the summer months of July and August where the flow has been observed to be zero.

The low flow or dry weather flows can be characterized by examining the flow duration curves for the two gauges within the study area. Figure 4.23 gives the ‘all year’ flow duration curves for the Oswego Creek and Caistors Corners gauges. Generally, flows less than the 10% duration represents the flood flow portion of the curve. According to Figure 4.23, it is difficult to compare the curves for each gauge because the Caistors Corners gauge flows are higher than the Oswego Creek values. Consequently, the unit area flow curves are presented in Figure 4.24. As suggested by Schroeter and Boyd (1998), the flow duration curves are highly correlated with the physiography of an area. Remarkably, the unit area flow duration curves for the Caistors Corners and Oswego Creek gauges are very similar, which suggests the overall watershed area upstream of each gauge location in terms of physiography and climate are also very similar.
Table 0-24 Annual minimum mean daily flow series, Water Survey of Canada Gauge 02HA007, Welland River below Caistor Corners, Period of Record 1957-2008, 51 years of record

<table>
<thead>
<tr>
<th>Return Period (Years)</th>
<th>Average Minimum 7-day Flow Rate (m$^3$/s)</th>
<th>Average Minimum 15-day Flow Rate (m$^3$/s)</th>
<th>Average Minimum 30-day Flow Rate (m$^3$/s)</th>
<th>Average Minimum 60-day Flow Rate (m$^3$/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.010</td>
<td>0.124</td>
<td>0.149</td>
<td>0.252</td>
<td>0.594</td>
</tr>
<tr>
<td>1.015</td>
<td>0.109</td>
<td>0.132</td>
<td>0.222</td>
<td>0.521</td>
</tr>
<tr>
<td>1.110</td>
<td>0.059</td>
<td>0.072</td>
<td>0.119</td>
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</tr>
<tr>
<td>1.250</td>
<td>0.043</td>
<td>0.053</td>
<td>0.086</td>
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<td>2</td>
<td>0.019</td>
<td>0.024</td>
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<tr>
<td>100</td>
<td>0.000</td>
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</table>

Table 0-25 Annual minimum mean daily flow series, Water Survey of Canada Gauge 02HA024, Oswego Creek at Canboro, Period of Record 1989-2008, 16 years of record

<table>
<thead>
<tr>
<th>Return Period (Years)</th>
<th>Average Minimum 7-day Flow Rate (m$^3$/s)</th>
<th>Average Minimum 15-day Flow Rate (m$^3$/s)</th>
<th>Average Minimum 30-day Flow Rate (m$^3$/s)</th>
<th>Average Minimum 60-day Flow Rate (m$^3$/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.010</td>
<td>0.015</td>
<td>0.019</td>
<td>0.021</td>
<td>0.049</td>
</tr>
<tr>
<td>1.015</td>
<td>0.013</td>
<td>0.016</td>
<td>0.019</td>
<td>0.043</td>
</tr>
<tr>
<td>1.110</td>
<td>0.007</td>
<td>0.009</td>
<td>0.010</td>
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</tr>
<tr>
<td>1.250</td>
<td>0.005</td>
<td>0.007</td>
<td>0.008</td>
<td>0.017</td>
</tr>
<tr>
<td>2</td>
<td>0.002</td>
<td>0.003</td>
<td>0.004</td>
<td>0.008</td>
</tr>
<tr>
<td>5</td>
<td>0.000</td>
<td>0.000</td>
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</tr>
</tbody>
</table>
Comparison Unit Area Low Flows

[Graph showing low flows vs return period for Oswego Creek and Caistors Corners.]

Figure 0-22 Unit area low flows for the Welland River Watershed

Flow Duration Curves

[Graph showing flow duration curves for Oswego Creek and Caistors Corners.]

Figure 0-23 Flow duration curves for the Welland River Watershed
4.5.3 Travel Time Assessment Welland River

Travel times were calculated for the Welland River between Port Davidson and the Old Siphon in the City of Welland using detailed hydrographs from the Welland River below Caistor Corners gauge station and Oswego Creek at Canboro gauge station. Hourly flow values were available from both stations between January 1, 2000 and December 31, 2004. However, significant errors due to backwater influences were identified at the Oswego Creek at Canboro gauge station for the year 2004, as a result, corrected discharge values were only available for both stations between January 1\textsuperscript{st}, 2000 and December 31\textsuperscript{st}, 2003. In addition, hourly stage hydrograph values were available for the Old Siphon hydrometric station between June 9\textsuperscript{th}, 2001 and June 15\textsuperscript{th}, 2006. However, due to the backwater influence of the Old Siphon structure, a rating curve was never developed for this station and therefore, only water levels and/or stage data was available at this site.

The events selected for the travel time assessment were based on the following criteria: 1) Their uniformity across the watershed; for example, the events were not isolated local events which effected only one part of the watershed but were uniform across the entire Welland River Watershed; and 2) Each event produced a significant amount of rainfall, for example, greater than 25 mm within 24 hours. This value was selected in accordance with the Ministry of Natural
Resources Technical Guidelines for Floodplain Management (2002). Two rainfall events which fit these criteria include the May 12-17, 2002 event and the May 1-5, 2003 event.

The travels times were determined by first summing the detailed hydrographs for specific events for the Welland River below Caistor Corners gauge station and Oswego Creek at Canboro gauge station. The resultant hydrograph represents the inflow hydrograph at Port Davidson. The time-to-peak for the resultant hydrograph at Port Davidson was then extracted. In turn, the time-to-peak for the stage hydrograph at the Old Siphon was also extracted and the travel time for the reach was determined by subtracting the two (2) time-to-peak values. This analysis only looks at the travel times of the hydrographs from the upstream end of the Welland River to the downstream end and neglects the influences of backwater coming from Hydro operations near the outlet of the Welland River. The results are illustrated in Table 4.26.

Based on the results of Table 4.26, the rise period in the hydrograph at Port Davidson is approximately two (2) days and the rise period in the hydrograph is approximately three (3) days at the Old Siphon in the City of Welland. The travel time of the hydrograph between Port Davidson and the Old Siphon can range between 12 and 24 hours. During less severe events, especially during the summer or fall period, the rise in the hydrographs may be longer, approximately three (3) days at Port Davidson and four (4) days at the Old Siphon. In addition, longer travel times between Port Davidson and the Old Siphon are also expected.

Table 0-26 Travel times between Port Davidson and Old Siphon within the City of Welland for the May 12-17, 2002 rainfall event and the May 1-5, 2003 rainfall event

<table>
<thead>
<tr>
<th>Events</th>
<th>Time-to-Peak Port Davidson (Hours)</th>
<th>Time-to-Peak Old Siphon (Hours)</th>
<th>Travel Time Between Port Davidson and Old Siphon (Hours)</th>
</tr>
</thead>
<tbody>
<tr>
<td>May 12-17, 2002</td>
<td>56</td>
<td>78</td>
<td>22</td>
</tr>
<tr>
<td>May 1-5, 2003</td>
<td>50</td>
<td>62</td>
<td>12</td>
</tr>
</tbody>
</table>

4.5.4 Lower Welland River

The Niagara River at one time provided a natural outlet approximately 3 km upstream from the Horseshoe Falls for the very mild sloping Welland River. During the past 80 years, a
considerable amount of construction work has been carried out near the outlet of the Welland River. Together with the present water diversion for power they have brought about some impacts on water levels in the lower Welland River. Because of the extreme mild Welland River slope, a detailed review of the outlet and the Lower Welland River is required before any analysis of upstream flows and water levels can be undertaken.

To study the effects of hydro operation on the Welland River, hydrometric data sets were provided by Niagara Peninsula Conservation Authority and Ontario Power Generation (formerly Ontario Hydro) through Niagara Peninsula Conservation Authority for the following locations:

I. Niagara River at Material Dock water level data (200-2001, semi-hourly and 1989-2006 hourly)
III. Chippawa Canal at Beck Crossover flow data (2000-2001)
IV. Chippawa Canal at Beck Crossover Tunnel Test January 2000, water level data
V. Measured velocity and flow values at the Welland River/Queenston-Chippawa Canal, April 11, 2002 and November 9, 2001

Additional historical water level and flow information was obtained from previous studies specifically the 1985 Flood Damage Reduction Study by Dillon Engineering which provides a technical summary of historical water level and flow information for each of the above locations dating back to 1939.

4.5.4.1 Niagara River Levels

Flow and water levels in the Niagara River are controlled by Lake Erie at the upstream end, and by conditions in the Chippawa-Grass Island Pool.

The Niagara River water level in the Chippawa-Grass Island Pool (Figures 4.25 and 4.26) controls the level in the lower portion of the Welland River. Hence, the effects of the Power Authorities’ operation on water levels in the Pool were considered in detail. Since power
development on the Niagara River began, there have been five time periods during which major changes in the Chippawa-Grass Island pool have occurred. These are as follows:

I. Prior to the construction of the submerged rock-filled weir in the period 1942-1944. The weir, about 502.92 m in length, was constructed 3 km upstream from the crest of the Horseshoe Falls. Its purpose was to compensate for the lowering effect of increased power diversions from the Chippawa-Grass Island Pool (Figure 5.27);

II. After construction of the submerged rock filled weir but before construction of the gated concrete control structure which took place from 1954 to 1963 (Figure 5.28);

III. After construction of the gated control structure, when the Chippawa-Grass Island Pool was regulated according to the first International Niagara Board of Control (INBC) Directive (1955). The 1955 Directive required that the Pool level be regulated close to the Pool’s pre-1953 hydraulic regime. This directive was in effect until early 1973;

IV. After the Chippawa-Grass Island Pool regulation was modified by the second INBC Directive (1973). The 1973 Directive requires that the Pool level be maintained as near as practicable at the Pool’s long-term mean level, with some flexibilities and tolerances permitted; and

V. After the Chippawa-Grass Island Pool regulation was modified by the third INBC Directive (1993). The 1993 directive requires that the Pool level be maintained at elevation 171.16 m (IGLD) with a view to ameliorating adverse high or low water levels in the Pool. If for any reason the material dock gauge is inoperative, the equivalent elevation at Slater’s point shall be controlling.

The water level analysis is summarized for the various time periods in Table 4.27. The historical water level data was taken from the 1985 Flood Damage Reduction Study of the Welland River by Dillon Engineering and supplemented with additional up-to-date data provided by Niagara Peninsula Conservation Authority. The data does not represent conditions for a given event on a given day, rather, various time periods were analyzed to determine characteristic trends
regarding flows and water levels for each individual period. The summary table allows for a comparison of trends developed for each period, with those of the base (unregulated) period (pre-1942).

The data in this table indicates that average levels in the Niagara River were generally 0.1 – 0.5 m lower during the unregulated (pre-1942) period than during any other period analyzed. However, the flows were also lower for the pre-1942 period than that for any other period (Dillon Engineering, 1985). This partially accounts for the trend toward higher levels for the post-1942 period. The exception is a flow value measured in 1999 which was 3950 m$^3$/s below the minimum requirement for flows in the Niagara of 4250 m$^3$/s in accordance with the 1993 INCB Directive.

The important period to compare with the pre-1942 data is the most recent period (post – 1993), since complaints of flooding have been quite frequent through Wainfleet in the last 20 years. For this analysis maximum, mean and minimum monthly water levels have been used. Generally, since 1993, the monthly maximum water level has fluctuated between 171.38 m and 171.58 m IGLD, rarely exceeding 171.5 m IGLD. The pre-1942 levels fluctuated between elevations 171.0 and 171.3 m IGLD, rarely exceeding elevation 171.5 m IGLD (Dillon Engineering, 1985). Therefore, the post-1993 maximum Niagara River levels at Material Dock are about 0.3 m above the level for the period 1939-1942. It should be noted that flows in the post-1993 period had also been higher with the exception of a flow value measured in 1999 which was 3950 m$^3$/s below the minimum requirement for flows in the Niagara River of 4250 m$^3$/s in accordance with the 1993 INCB Directive.

The post-1993 monthly mean has been held close to elevation 171.16 m IGLD as required by the 1993 INBC Directive, while the pre-1942 mean fluctuated between elevations 170.7 m and 171.0 m IGLD. There is a 0.36 m difference between the averages of the means for the two periods. If the higher flows occurring during the post-1993 period were taken into account, there would actually be negligible difference between the long term means for the two periods. Finally, the monthly minimum values for post-1993 conditions fluctuate within a band of 0.1 m compared to a 0.3 m band of fluctuation for pre-1942 conditions. Suggesting that the control structure has
eliminated the large band of fluctuation associated with the pre-1942 conditions and is therefore beneficial to the Niagara River and Welland River riparian interests.

The following conclusions can be drawn from the Niagara River historical water level analysis recorded at Material Dock:

I. Pre-1942 data provides a reasonable base (unregulated) condition against which data from other periods can be compared.

II. Water levels analyzed for the period 2000 to 2001 tend to be 0.1 to 0.5 m higher on the average than pre-1942 levels. This is mainly due to the higher flows associated with the more recent period.

III. Although increased Chippawa-Grass Island Pool level (CGIP) in the Niagara River (associated with development for power production) could potentially aggravate flooding problems on the Welland River, data analyzed for various time periods show the increase in the pool level to be minor (0.1 to 0.5 m based on monthly maximum, minimum and monthly means) caused mainly by higher flows.

High water levels which may have been experienced in the pool under natural conditions have actually been reduced by the operation of the Power Authorities due to the International Niagara Board of Control (INBC) restriction of a maximum level of 171.61 m and the requirement for maintaining the long term average at elevation 171.16 m IGLD.
Figure 0-25 Schematic plan of the Lower Welland River downstream of the new siphon to the material dock (Recreated from International Niagara Board of Control, 107th Semi-Annual Report, 2006)
Figure 0-26 Power entities operations in the Niagara River (Recreated from International Niagara Board of Control, 107th Semi-Annual Report, 2006)

Figure 0-27 Submerged rock-filled weir and material dock at Niagara River (Photo taken June 2006)
Fluctuations in Niagara River water levels in the CGIP depend on the outflow from Lake Erie, the operation of the Control structure and the extent of power diversions on both sides of the border. Changes in water levels caused by river flow are generally gradual, however for hydro generation and the requirement to maintain minimum flows over Niagara Falls during daytime frequently results in rapid cyclical changes in water levels in the Niagara River at the junction of the Welland River. Figure 4.29 shows a typical hourly fluctuation of levels in the Pool. During the daylight hours, pool levels are generally lower, and diversions reduced to enable higher flows over Niagara Falls (minimum 2832 m$^3$/s as per INBC Directive). At night, when the flow requirement over the Niagara Falls is reduced, the Pool level is raised to facilitate more power diversions and/or to increase the storage in the Pool for the next day.
Table 0-27 Niagara River material dock water levels, summary of data analysis

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Comment</td>
<td>Prior to construction of submerged rock weir</td>
<td>After submerged weir but prior to gated concrete control structure</td>
<td>After construction of gated concrete control structure (regulation under 1955 INBC directive)</td>
<td>After 2nd INBC directive</td>
<td>After 3rd INBC directive</td>
</tr>
<tr>
<td>Reason for chose time period</td>
<td>To illustrate conditions before major power developments</td>
<td>Determine water level increase caused by weir</td>
<td>Determine how closely water levels are being regulated to 1955 directive</td>
<td>Determine how closely water levels are being regulated to 1973 directive</td>
<td>Determine how closely water levels are being regulated to 1993 directive</td>
</tr>
<tr>
<td>Mean Monthly flow range (m³/s)</td>
<td>4530 to 5660</td>
<td>5100 to 6130</td>
<td>5660 to 6800</td>
<td>5850 to 7010</td>
<td>5240 to 6560</td>
</tr>
<tr>
<td>Maximum Level</td>
<td>171.7</td>
<td>172.0</td>
<td>172.2</td>
<td>172.0</td>
<td>171.6</td>
</tr>
<tr>
<td>Maximum daily flow rate (m³/s)</td>
<td>6540</td>
<td>6770</td>
<td>7620</td>
<td>9760</td>
<td>7970</td>
</tr>
<tr>
<td>Maximum level rarely exceeded</td>
<td>171.5</td>
<td>171.7</td>
<td>171.6</td>
<td>171.6</td>
<td>171.5</td>
</tr>
<tr>
<td>Minimum level recorded</td>
<td>170.0</td>
<td>170.4</td>
<td>170.4</td>
<td>170.4</td>
<td>170.8</td>
</tr>
<tr>
<td>Minimum level frequently exceeded</td>
<td>170.4</td>
<td>170.7</td>
<td>170.6</td>
<td>170.5</td>
<td>170.83</td>
</tr>
<tr>
<td>Average of monthly means</td>
<td>170.8</td>
<td>171.1</td>
<td>171.1</td>
<td>171.0</td>
<td>171.16</td>
</tr>
</tbody>
</table>

Notes:

I. Elevations in metres-IGLD. Conversion to geodetic elevation GD=IGLD+0.021
II. Levels recorded at material dock gauge refer to daily levels (Dillon Engineering, 1985)
III. Water levels pre-1993 obtained from 1985 Flood Damage Reduction Study of the Welland River by Dillon Engineering
IV. Flows obtained from published Water Survey of Canada records at the Queenston gauge.
$4.5.4.2 \text{ Welland River at Montrose Gauge}$

The analyses in this section concern water level data at the Montrose gauge, located at the junction of the Welland River and the Chippawa Power Canal (Figure 4.25). Knowledge of water levels at this location is important since they represent starting water surface elevations required to assess the hydraulics of the lower Welland River. Water levels and flow rates at the Welland River/Chippawa Power Canal junction at Montrose gauge are controlled by the combined effects of Cross-over Point water levels located approximately 12.5 km downstream and the Niagara River level located approximately 6 km upstream. The cross-over discharge varies with water levels through the generating stations and the Niagara River level. The average annual discharge at the cross-over point was estimated to be 1569 m$^3$/s ($+/-$) 274 m$^3$/s. According to the 1985 Flood Damage Reduction Study by Dillon Engineering the long term water level at the Cross-over point was estimated to be 164.6 m IGLD. This value was based on several years of water level data analysis at the cross-over point. According to the 1985 Flood Damage Reduction Study this level was regarded as optimum since lower levels can cause problems at the intakes, while higher levels reduce the flow to the plants. In turn, a tunnel test in January of 2000 revealed that the water levels in tunnels 2A and 2B ranged between 169.5-169.9 m and 169.4-169.8 m respectively. As described previously the long term Material Dock mean level is maintained at elevation 171.16 m IGLD. Therefore, the difference between Material Dock and
the tunnels 2A and 2B levels can vary between 1.26 m and 1.76 m. The difference in head between the Material Dock and the Cross-over point level is approximately 6.56 m.

Water level data from the Montrose gauge was analyzed for the period from January 1, 2000 to December 31, 2000. Several observations can be drawn from this data when compared to Material Dock data:

I. Maximum levels at the Montrose gauge generally fluctuate between elevations 170.93 m and 171.19 m IGLD, compared to fluctuations between elevations 171.38 m and 171.58 m IGLD at the Material Dock gauge;

II. Montrose mean levels fluctuate within a narrow band about elevation 170.8 m IGLD compared to Material Dock fluctuations about elevation 171.16 m IGLD.

III. Minimum Montrose levels fluctuate between elevations 170.35 m and 170.86 m IGLD compared to Material Dock fluctuations between elevations 170.78 m and 170.93 m IGLD;

IV. The water level at the Montrose gauge (Welland River inlet to Chippawa Power Canal) is generally 0.33 to 0.45 m lower than the Material Dock (Niagara River) water level. This varies with degree of weed growth, silting and debris in the channel and canal, and also varies with the relative difference between the Crossover and Material Dock water surface elevations.

Item IV) indicates a headloss of not more than 0.45 m from the Niagara River, through the 6 km long Welland River intake, to the Welland River / Chippawa Power Canal junction.

**4.5.4.3 Chippawa Power Canal**

The Chippawa Power Canal has an effect on the Welland River levels at Montrose. In the exceptional case, such as the months of October 1964, when the Canal was not in operation,
water levels at the Montrose gauge were almost identical to the Niagara River water levels at the Material Dock (Dillon Engineering, 1985). Prior to November 1, 1965, the Canal diversion flow was limited to 453 m$^3$/s, which created a drop in water levels between Montrose and Material Dock of approximately 15 cm (Dillon Engineering, 1985). The subsequent improvements to the Canal have increased the diversion flow to approximately 595 m$^3$/s, resulting in an additional 12 cm drop in water levels at Montrose, caused by the frictional resistance of the Channel (Dillon Engineering, 1985). Recorded water levels indicate a headloss from the Material Dock gauge to the Crossover gauge of 4.82 m, while the headloss from the Material Dock to the Montrose gauge is only 0.37 m. Since the 1993 INBC directive, Montrose water levels have fluctuated between maximum and minimum elevations of 171.19 m and 170.35 m IGLD respectively, with a mean level of about 170.80 m IGLD.

**4.5.4.4 Welland River Flow Reversal**

In the normal period of operation of the Chippawa Power Canal, the average flow of 595 m$^3$/s is diverted from the Niagara River (Dillon Engineering, 1985). During the peak of spring event, a portion of this 595 m$^3$/s is supplied from runoff in the Welland River. Based on the measured flow rates provided in the previous section the percentage of flow from the Welland River was approximately 14%. For example, the total flow in the Power Canal was measured to be 539 m$^3$/s, the total flow in the west channel of the Power Canal was measured to be 65 m$^3$/s, this value in turn, represents 14% of the total flow in the Power Canal.

The Welland River flow reversal was accomplished in the early 1920’s by dredging the last 6 km of the Welland River and providing the Chippawa Power Canal as the new outlet. The following passage was taken out of the 1985 Flood Damage Reduction Program Study by Dillon Engineering and provides a comparison of pre- and post-flow reversal water levels expected at the Welland River/Chippawa Power Canal junction is given below for typical flow conditions.

“Prior to the Welland River flow reversal, the channel invert extended from the existing Welland River Chippawa Canal junction to the Niagara River on a very mild slope. The natural Niagara River water level controlled the outlet capacity of the mild sloping
Welland River. The hydraulic grade line originated at the Niagara River, and to facilitate flow in the Welland River a positive grade in the Welland River was needed which increased the water surface elevation upstream of the Welland River. The magnitude of the increase was dependent on the flow in the Welland River and the Niagara River levels. Therefore, at the location of the now existing Chippawa Power Canal, the water level in the Welland River had to be higher than the starting level in the Niagara River. The chosen Niagara water level of 170.8 IGLD represents a typical natural level at Material Dock prior to 1942.

After the Welland River flow reversal, the hydraulic conditions described above were altered, both on the Welland and the Niagara Rivers. Various controls and regulations on the Niagara River have been enacted to maintain a current monthly mean of 171.16 IGLD. Compared to the pre-1942 monthly mean of 170.8 IGLD, this regulated water level represents an increase of 0.36 m. Currently on average 573 m$^3$/s is being withdrawn from the Chippawa Grass Island Pool via the Welland River, which results in a headloss of 0.36 m between the Material Dock and the Montrose gauges. Since the flow direction has been reversed and the hydraulic grade line must drop in the direction of flow, the water level at Montrose is now lower than the Niagara River water level. This is in contrast to the higher water level which would have been experienced at Montrose prior to the flow reversal. Therefore, the effect of the higher Niagara River water levels due to artificial regulation is offset by the new downward sloping hydraulic grade line. The result is a water surface elevation at Montrose which is similar to that which would have experienced under natural conditions.

Table 4.28 illustrates the difference in head between the Montrose gauge station and the Grass Island Pool at Material Dock on a monthly basis. The monthly means are based on two (2) years of data (2000-2001). Using these two boundary conditions an unsteady state hydraulic model (HEC-RAS version 3.1.3) was set-up between the material dock and the Montrose gauge to determine the average monthly intake values from the Grass Island Pool. Table 4.29 lists the calculated mean monthly intake values from the Grass Island Pool using the HEC-RAS unsteady state model. According to Table 4.29 the average annual intake from the Grass Island Pool is approximately 470 (+/-) 40 m$^3$/s.
In 2006, Ontario Power Generation through Niagara Peninsula Conservation Authority provided an additional two (2) years of flow data at the cross-over point Sir Adam Beck Power Generating Station (2000-2001). The flow data represents the total combined flow from the two existing tunnels and the power canal. Table 4.30 lists the mean monthly flows at the cross-over point. In 1991, Hatch-Energy (formerly Hatch-Acres) developed a polynomial equation relating the total diversion flow to the head loss from the Grass Island Pool (GIP) to the cross-over. This equation, as provided by AMEC (formerly Philips Engineering), has the following form:

\[ Q_{div} = (N_t \times K + C_0 + C_1 \times R_x + C_2 \times R_x \times R_x) \times R_y - Q_{res} \]  \hspace{1cm} (4.155)

Where:

- \( N_t \) = the number of new tunnels
- \( K \) = 3,800 additional tunnel(s) coefficient for two new tunnels of 500 cms flow capacity each
- \( R_x \) = \((W_L_{gip} + W_L_{xover})/2\)
- \( R_y \) = \(\sqrt{W_L_{gip} - W_L_{xover}}\)
- \( W_L_{gip} \) = Grass Island Pool water level (ft, IGLD)
- \( W_L_{xover} \) = Cross-over water level (ft, IGLD)

\( C_0, C_1, \) and \( C_2 \) are coefficients of the original polynomial as follows:

- \( C_0 \) = -5,406,132.387
- \( C_1 \) = 19,385,56439
- \( C_2 \) = -17,32954545
- \( Q_{res} \) = seasonal flow restriction

An important term in this equation is \( Q_{res} \), the seasonal flow restriction, which reflects the differences between the equation values and actual measured diversion flows. The value of this term varies from 0 to 85 m\(^3\)/s. The monthly water levels at the cross-over location have been estimated through the Hatch-Energy equation (4.155) by solving for \( W_L_{gip} \). The results are
illustrated in Table 4.31. The Niagara Peninsula Conservation Authority provided additional water level information for the Cross-over point for the years 2000 and 2001. A comparison of the water levels at the Cross-over point between the hydrometric data and the calculated values using the Hatch-energy equation are listed in Table 4.31.

Based on the values listed in Table 4.31 the percent (%) difference between the measured and calculated water levels using the Hatch-Energy equation is less than 0.3 percent (%). The monthly flows in the tunnels were than calculated using the orifice equation (Maidment and Chow, 1988) and the computed water levels at the cross-over point and the measured water levels at the material dock:

\[ Q = 0.6A\sqrt{2gh} \]  

\[ Q = \text{Flow, ft}^3/\text{s} \]
\[ A = \text{Area of the opening, ft}^2 \]
\[ g = \text{Acceleration due to gravity 32.2 ft/s}^2 \]
\[ h = \text{headloss, ft} \]

On April 11, 2002, the recorded flow in the Power Canal was estimated to be 539 m$^3$/s, from this value the total flow from the tunnels was estimated to be 932 m$^3$/s (1471 (mean monthly combined flow at the cross-over) – 539); the headloss was determined to be 13.63 ft (Mean monthly water level at Material Dock – Computed Water level at Cross-over point); substituting into equation 4.156 and solving for A; the value for A was determined to be 1359 ft$^2$. In turn, the combined flows from the tunnels were than solved for using equation 4.156. Table 4.32 lists the computed flows in the tunnels.

Having computed the mean monthly flows in the tunnels, flows in the Power Canal can be determined by subtracting the mean combined flows at the Cross-over from the flows in the tunnels. In turn, once the flow in the Power Canal is determined the flows within the Welland River at the outlet can be determined by subtracting the flow in the Power Canal by the intake flow from the Grass Island Pool. The results are illustrated in Table 4.33. The foregoing analysis
shows that the flows within the Power Canal and the Tunnels remain relatively consistent, along with the intake flow from the Grass Island Pool, whereas the flow in the Welland River at the outlet can vary from 119 m$^3$/s to 5 m$^3$/s.

<table>
<thead>
<tr>
<th>Month</th>
<th>Material Dock (m, IGLD 1985)</th>
<th>Montrose Gauge (m, IGLD 1985)</th>
<th>Difference in Head (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>January</td>
<td>171.09</td>
<td>170.68</td>
<td>0.41</td>
</tr>
<tr>
<td>February</td>
<td>171.11</td>
<td>170.73</td>
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<tr>
<td>March</td>
<td>171.14</td>
<td>170.75</td>
<td>0.39</td>
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<tr>
<td>April</td>
<td>171.18</td>
<td>170.83</td>
<td>0.34</td>
</tr>
<tr>
<td>May</td>
<td>171.17</td>
<td>170.83</td>
<td>0.34</td>
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<tr>
<td>June</td>
<td>171.21</td>
<td>170.86</td>
<td>0.35</td>
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<tr>
<td>July</td>
<td>171.19</td>
<td>170.84</td>
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</tr>
<tr>
<td>August</td>
<td>171.17</td>
<td>170.82</td>
<td>0.35</td>
</tr>
<tr>
<td>September</td>
<td>171.17</td>
<td>170.83</td>
<td>0.34</td>
</tr>
<tr>
<td>October</td>
<td>171.18</td>
<td>170.83</td>
<td>0.34</td>
</tr>
<tr>
<td>November</td>
<td>171.15</td>
<td>170.77</td>
<td>0.38</td>
</tr>
<tr>
<td>December</td>
<td>171.15</td>
<td>170.78</td>
<td>0.37</td>
</tr>
<tr>
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<td>171.16</td>
<td>170.79</td>
<td>0.36</td>
</tr>
</tbody>
</table>

Table 0-29 Average monthly intake at the Grass Island Pool (GIP)

<table>
<thead>
<tr>
<th>Month</th>
<th>Flow (m$^3$/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>January</td>
<td>521</td>
</tr>
<tr>
<td>February</td>
<td>511</td>
</tr>
<tr>
<td>March</td>
<td>516</td>
</tr>
<tr>
<td>April</td>
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<tr>
<td>May</td>
<td>485</td>
</tr>
<tr>
<td>June</td>
<td>487</td>
</tr>
<tr>
<td>July</td>
<td>489</td>
</tr>
<tr>
<td>August</td>
<td>492</td>
</tr>
<tr>
<td>September</td>
<td>485</td>
</tr>
<tr>
<td>October</td>
<td>485</td>
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<tr>
<td>December</td>
<td>500</td>
</tr>
<tr>
<td>Annual</td>
<td>484</td>
</tr>
</tbody>
</table>
### Table 0-30 Mean monthly flows and water levels at the cross-over point

<table>
<thead>
<tr>
<th>Month</th>
<th>Flow (m³/s)</th>
<th>Water Level (m, IGLD 1985)</th>
</tr>
</thead>
<tbody>
<tr>
<td>January</td>
<td>1706</td>
<td>165.140</td>
</tr>
<tr>
<td>February</td>
<td>1675</td>
<td>165.450</td>
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<td>1705</td>
<td>165.150</td>
</tr>
<tr>
<td>April</td>
<td>1471</td>
<td>167.005</td>
</tr>
<tr>
<td>May</td>
<td>1527</td>
<td>166.640</td>
</tr>
<tr>
<td>June</td>
<td>1528</td>
<td>166.630</td>
</tr>
<tr>
<td>July</td>
<td>1496</td>
<td>166.845</td>
</tr>
<tr>
<td>August</td>
<td>1479</td>
<td>166.995</td>
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<td>165.040</td>
</tr>
<tr>
<td>December</td>
<td>1693</td>
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<tr>
<td>Annual</td>
<td>1562</td>
<td>166.345</td>
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### Table 0-31 Comparison of mean monthly water levels at the cross-over point

<table>
<thead>
<tr>
<th>Month</th>
<th>Water Levels Computed Values (m, IGLD 1985)</th>
<th>Water Levels Measured (m, IGLD 1985)</th>
</tr>
</thead>
<tbody>
<tr>
<td>January</td>
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<td>165.16</td>
</tr>
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<td>167.005</td>
<td>166.88</td>
</tr>
<tr>
<td>May</td>
<td>166.640</td>
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<td>June</td>
<td>166.630</td>
<td>166.33</td>
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<tr>
<td>July</td>
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<td>166.54</td>
</tr>
<tr>
<td>August</td>
<td>166.995</td>
<td>166.53</td>
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<td>166.83</td>
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<tr>
<td>October</td>
<td>167.055</td>
<td>166.60</td>
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<td>165.040</td>
<td>164.51</td>
</tr>
<tr>
<td>December</td>
<td>166.865</td>
<td>164.53</td>
</tr>
<tr>
<td>Annual</td>
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<td>165.89</td>
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### Table 0-32 Mean monthly flows in the tunnels and headloss between the material dock and the cross-over point

<table>
<thead>
<tr>
<th>Month</th>
<th>Headloss between Material Dock and Cross-over point (m)</th>
<th>Flow in Tunnels (m³/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>January</td>
<td>6.22</td>
<td>1122</td>
</tr>
<tr>
<td>February</td>
<td>5.70</td>
<td>1074</td>
</tr>
<tr>
<td>March</td>
<td>5.97</td>
<td>1100</td>
</tr>
<tr>
<td>April</td>
<td>4.29</td>
<td>933</td>
</tr>
<tr>
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<td>4.69</td>
<td>974</td>
</tr>
<tr>
<td>June</td>
<td>4.88</td>
<td>994</td>
</tr>
<tr>
<td>July</td>
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<td>970</td>
</tr>
<tr>
<td>August</td>
<td>4.65</td>
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<td>4.35</td>
<td>938</td>
</tr>
<tr>
<td>October</td>
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<td>963</td>
</tr>
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<td>November</td>
<td>6.64</td>
<td>1160</td>
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<tr>
<td>Annual</td>
<td>5.27</td>
<td>1030</td>
</tr>
</tbody>
</table>
Table 0-33 Mean monthly flows in the Chippawa Power Canal and the Welland River

<table>
<thead>
<tr>
<th>Month</th>
<th>Flow Power Canal (m³/s)</th>
<th>Intake from Niagara River (m³/s)</th>
<th>Flow from Welland River (m³/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>January</td>
<td>583</td>
<td>521</td>
<td>62</td>
</tr>
<tr>
<td>February</td>
<td>600</td>
<td>511</td>
<td>89</td>
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<tr>
<td>March</td>
<td>605</td>
<td>516</td>
<td>89</td>
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<tr>
<td>April</td>
<td>539</td>
<td>489</td>
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<tr>
<td>May</td>
<td>553</td>
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<td>510</td>
<td>46</td>
</tr>
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<td>536</td>
<td>500</td>
<td>36</td>
</tr>
<tr>
<td>Annual</td>
<td>544</td>
<td>493</td>
<td>51</td>
</tr>
</tbody>
</table>

4.5.4.5 Effect of Lake Erie

Any water level changes on Lake Erie are reflected on the Niagara River and hence the backwater in the Welland River. Lake Erie is subject to long-term seasonal and annual changes in water level and to short term changes in water level caused by wind set-up, barometric pressure variations and by seiches. The average or normal elevation of the Lake varies irregularly from year to year, and during each year is subject to consistent seasonal rise and fall, reaching the lowest stage during the winter months (Dillon Engineering, 1985). The annual lake level variations are primarily dependent upon the precipitation and evaporation that occur over the Great Lakes Watershed. The mean monthly Lake levels for Lake Erie are listed in Table 4.34 (International Niagara Control Board, Published 107th Semi-Annual Report, 2006).

In addition, to the seasonal and annual fluctuations, Lake Erie is subject to short term effects such as wind set up which raises the level at one end while lowering it at the other. Lake Erie, the shallowest of all of the Great Lakes, has appreciable wind set-up with ranges of 3.7 to 4.3 m having been recorded (Dillon Engineering, 1985). The effect of wind set up can be significant even in the Welland River where, for example, on 20 November 1964 an increase in the Lake Erie water levels caused by wind set-up resulted in a 0.76 m water level increase in the Lower Welland River (Dillon Engineering, 1985; US Army Corps of Engineers Detroit District Office,
Given that there is a 3 m headloss between Lake Erie and the Grass Island Pool, water levels in the Welland River can rise anywhere between 0.7 and 1.3 m.

Lake Erie is also subject to occasional seiches or standing waves of irregular amount duration. Sometimes these result from variations in barometric pressure which may produce changes in water surface elevations ranging from a few inches to several feet (International Niagara Control Board, Published 107th Semi-Annual Report, 2006). Lake level variations due to wind set up and seiches are of a short-term nature, usually of one or two day duration, whereas seasonal and annual variations in lake level, however, are of a long-term nature (International Niagara Control Board, Published 107th Semi-Annual Report, 2006). During the winter month’s ice jamming along the Niagara River can cause increases in water levels along the Welland River of approximately 0.6 to 0.9 metres (International Niagara Control Board, Published 107th Semi-Annual Report, 2006). However, the presence of the ice boom (Figure 4.25) at the inlet of the Niagara River from Lake Erie has reduced the potential for ice jamming within the Niagara River.

It is important to note that due to hydro operations including forecasting and control procedures, the effect of wind set up and ice jams on the Welland River have been considerably reduced. This point is further verified by the foregoing analysis. An exceedance probability analysis was undertaken for the water levels for Lake Erie at Buffalo for each respective month. The values listed in Table 4.35, represent the difference in elevation between the maximum instantaneous water level and the mean monthly water level. They do not represent any particular storm event but only provide a probability of exceedance for a particular water level. These values were then added to the mean monthly values for Lake Erie and the results are illustrated in Table 4.36. The average headloss between Lake Erie and the Grass Island Pool is approximately 3 m, the expected exceedance probabilities of water levels at the Grass Island Pool are illustrated in Table 4.37. However, based on the continuous hourly water level data (1989-2006) and a historical maximum recording of 172.45 m (taken from 1985 Flood Damage Reduction Program Study) at the Grass Island Pool the actual exceedance probabilities for the water levels at the Grass Island Pool on an annual basis are illustrated in Table 4.38.
### Table 0-34 Mean monthly lake levels (1918-2005), Lake Erie

<table>
<thead>
<tr>
<th>Month</th>
<th>Water Levels (m, IGLD 1985)</th>
</tr>
</thead>
<tbody>
<tr>
<td>January</td>
<td>174.24</td>
</tr>
<tr>
<td>February</td>
<td>174.27</td>
</tr>
<tr>
<td>March</td>
<td>174.30</td>
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<tr>
<td>April</td>
<td>174.39</td>
</tr>
<tr>
<td>May</td>
<td>174.41</td>
</tr>
<tr>
<td>June</td>
<td>174.33</td>
</tr>
<tr>
<td>July</td>
<td>174.23</td>
</tr>
<tr>
<td>August</td>
<td>174.16</td>
</tr>
<tr>
<td>September</td>
<td>174.08</td>
</tr>
<tr>
<td>October</td>
<td>174.01</td>
</tr>
<tr>
<td>November</td>
<td>173.89</td>
</tr>
<tr>
<td>December</td>
<td>173.86</td>
</tr>
</tbody>
</table>

### Table 0-35 Exceedance probability of water levels (m) for Lake Erie at Buffalo

<table>
<thead>
<tr>
<th>Month</th>
<th>5-year</th>
<th>10-year</th>
<th>20-year</th>
<th>50-year</th>
<th>100-year</th>
</tr>
</thead>
<tbody>
<tr>
<td>January</td>
<td>1.40</td>
<td>1.58</td>
<td>1.80</td>
<td>1.92</td>
<td>2.04</td>
</tr>
<tr>
<td>February</td>
<td>1.01</td>
<td>1.25</td>
<td>1.58</td>
<td>1.83</td>
<td>2.04</td>
</tr>
<tr>
<td>March</td>
<td>1.22</td>
<td>1.46</td>
<td>1.80</td>
<td>2.01</td>
<td>2.23</td>
</tr>
<tr>
<td>April</td>
<td>1.04</td>
<td>1.28</td>
<td>1.62</td>
<td>1.89</td>
<td>2.13</td>
</tr>
<tr>
<td>May</td>
<td>0.67</td>
<td>0.79</td>
<td>0.98</td>
<td>1.10</td>
<td>1.22</td>
</tr>
<tr>
<td>June</td>
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<td>0.73</td>
<td>0.85</td>
<td>0.91</td>
<td>1.01</td>
</tr>
<tr>
<td>July</td>
<td>0.58</td>
<td>0.67</td>
<td>0.76</td>
<td>0.82</td>
<td>0.88</td>
</tr>
<tr>
<td>August</td>
<td>0.61</td>
<td>0.70</td>
<td>0.79</td>
<td>0.88</td>
<td>0.94</td>
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<td>0.94</td>
<td>1.13</td>
<td>1.34</td>
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<td>1.37</td>
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<td>1.86</td>
<td>2.04</td>
</tr>
<tr>
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<td>1.77</td>
<td>2.04</td>
<td>2.23</td>
<td>2.44</td>
</tr>
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<td>1.80</td>
<td>2.10</td>
<td>2.32</td>
<td>2.53</td>
</tr>
</tbody>
</table>

### Table 0-36 Exceedance probability of water levels (m, IGLD) for Lake Erie at Buffalo

<table>
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<tr>
<th>Month</th>
<th>5-year</th>
<th>10-year</th>
<th>20-year</th>
<th>50-year</th>
<th>100-year</th>
</tr>
</thead>
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<td>175.79</td>
<td>175.91</td>
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<td>175.56</td>
<td>175.81</td>
<td>176.02</td>
</tr>
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<td>175.52</td>
<td>175.86</td>
<td>176.07</td>
<td>176.29</td>
</tr>
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<td>175.49</td>
<td>175.83</td>
<td>176.10</td>
<td>176.34</td>
</tr>
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<td>175.09</td>
<td>175.28</td>
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</tr>
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<td>175.24</td>
<td>175.34</td>
</tr>
<tr>
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<td>174.99</td>
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<td>175.43</td>
<td>175.71</td>
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</tr>
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<td>175.79</td>
<td>176.09</td>
<td>176.31</td>
<td>176.52</td>
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</table>
### Table 0-37 Exceedance probability of water levels (m, IGLD) for Grass Island Pool

<table>
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<tr>
<th>Month</th>
<th>5-year</th>
<th>10-year</th>
<th>20-year</th>
<th>50-year</th>
<th>100-year</th>
</tr>
</thead>
<tbody>
<tr>
<td>January</td>
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<td>172.57</td>
<td>172.79</td>
<td>172.91</td>
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</tr>
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<td>172.81</td>
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<td>172.83</td>
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<tr>
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<td>172.34</td>
</tr>
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<td>172.08</td>
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<td>172.20</td>
</tr>
<tr>
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<td>171.95</td>
<td>172.04</td>
<td>172.13</td>
<td>172.19</td>
</tr>
<tr>
<td>September</td>
<td>172.10</td>
<td>172.29</td>
<td>172.50</td>
<td>172.68</td>
<td>172.84</td>
</tr>
<tr>
<td>October</td>
<td>172.22</td>
<td>172.43</td>
<td>172.71</td>
<td>172.92</td>
<td>173.10</td>
</tr>
<tr>
<td>November</td>
<td>172.51</td>
<td>172.76</td>
<td>173.03</td>
<td>173.22</td>
<td>173.43</td>
</tr>
<tr>
<td>December</td>
<td>172.51</td>
<td>172.79</td>
<td>173.09</td>
<td>173.31</td>
<td>173.52</td>
</tr>
</tbody>
</table>

This analysis also confirms the 1993 Directive by the International Niagara Board of Control to meet a long-term water level within the Grass Island Pool of 171.16 m (IGLD, 85).

### 4.5.5 Welland Canal and Siphons

The Welland Canal company was created in 1824 by William Hamilton Merritt in part to provide a faster method to transport cargo from cities in Lake Erie to cities in Lake Ontario. The first canal opened in 1829 running from Port Dalhousie on Lake Ontario where it continued south to Port Robinson then east to Chippewa via the Welland River where the river meets up with Niagara River. Throughout the years numerous improvements were made to make the canals more efficient and to minimize impacts on the surrounding areas such as the Welland River. The final part of the Welland Canal began construction in 1913 and was completed in 1932. In 1967,
the Welland Canal was rerouted in order to bypass the City of Welland. This construction was completed in 1972 and the work was known as the New Welland Canal (as opposed to the Old Canal and the Old Siphon which is located within the heart of the City of Welland). The new canal is composed of only eight locks, seven running through the Niagara escarpment and the eighth lock acting as a control lock located at Port Colborne.

The Welland River flows underneath both the Old and New Welland canals through a series of complex hydraulic structures called siphons. The City of Welland bypass (the New Canal) required the construction of the New Siphon. As a result of the Welland Canals, the complexity of the Welland watershed has greatly been increased. Various flows have been altered and backflows are recorded after large rainfalls at both siphons. These backflows create conditions that augment the risk of flooding in the regions upstream to the siphons. The siphons also create an area where sediments can collect and as a result the siphons have become very inefficient. This sedimentation also increases flooding hazards and add complexity to forecasting flooding scenarios. Two inverted siphons have been built to divert the flow of Welland River water beneath the Old and New Welland Ship Canals. These structures flow full under pressure and create backwater pools during floods in a manner similar to dams.

4.5.5.1 Old Siphon

The Welland River siphon in the City of Welland was constructed by the St. Lawrence Seaway Authority during the period from 1925 to 1930. The inverted syphon culvert under the canal, on a realigned route of the river, was designed to provide a greater canal depth to accommodate heavier water traffic. The old structure is located in the City of Welland, north of Main Street. It consists of six 6.7 m (22 ft.) diameter tubes in parallel beneath the canal, connected at each end to six vertical shafts which rise to the river bed. The siphon headwall on each side of the canal is 5.5 m (18 ft. thick). The massive concrete structure is lightly reinforced and supported on timber piles driven to rock. In 1969, the City of Welland used the southernmost tube to route three sanitary sewers across the canal. These sewers were encased in concrete, reducing the area of flow in this tube. The total available cross sectional area of the siphon tubes, assuming no other blockages, is about 207 m².
The 1985 Flood Damage Reduction Study calculated the theoretical head loss rating curves for the Old Siphon. The head loss coefficients were selected on the basis of the physical characteristics of the syphon barrels. No detailed information is available on the loss coefficients used in the original siphon design. The theoretical head loss rating curves for the old syphon are plotted in Figure 5.27. According to Figure 5.27, there is little difference between the head loss values for the rating curves for “all tubes clear” and “6th tube plugged”.

The reduction in flow capacity as indicated by this curve is due to the concrete encasement within the southern tube, constructed in 1969, to route sanitary sewer pipes across the Welland Canal. The conditions inside the old siphon were probably changed during the first few years of operation, as the original area of flow through the siphon was greater than the area of flow in adjacent natural channel reaches. However, some long-term accumulation of silt has probably reduced the area of flow through the Old Siphon to something comparable to the area of flow in the adjacent river reaches. Therefore, the man-made structure has probably been modified over the years by the river flow regime and the siphon culvert has become a part of the river.

On 1 May and 30 May 1967, the Public Works Canada Western Region investigated the silting condition of the syphon, as part of the information needed for designing the new siphon downstream at Port Robinson. The exploratory investigation had been carried out by a diver lowered down into No. 2 and vertical shafts at the discharge end of the syphon. It revealed that a silty mud had filled up the bottom of the shafts, the sump pits, about 1.2 m to 1.5 m deep or about 0.3 to 0.6 m above the bottom of the horizontal culverts. However, inside the end of the No. 5 culvert, there was a large pile of debris at least 1.8 m high, consisting of numerous logs, rocks, concrete, pipes, etc. In each end of Nos. 2 and 6 culverts, there was only a mound of soft and silty mud, at least 1.8 m high, preventing the diver from moving further into the culverts. The other three culverts, the upstream end of the siphon and the condition inside all culverts were not investigated, as the diver was no able to move in the current, soft mud, and the water was very dirty and dark.

On the basis of the foregoing, the estimated reduction in the area of flow through the siphon may be as much as 25%. However, the sediments in the siphon bottom have been reported to be in a
semi-suspended state, and that it has further suggested that much of the silt may be flushed out during high flow periods (Dillon, 1985).

### 4.5.5.2 New Siphon

The New Siphon was constructed on the Welland River by the St. Lawrence Seaway Authority in 1973. The New Siphon, located near Port Robinson, conveys the Welland River under the new ship canal, east of the City of Welland. The New Siphon consists of four 5.2 m X 8 m rectangular barrels in parallel. The flared inlet and outlet of each of the syphon barrels are constructed on a slope of approximately 2.4 H: 1V, giving better inlet and outlet conditions compared to the old culvert. The total available cross section area of the New Siphon, assuming no blockages, is approximately 169 m². Comparing the theoretical rating curves of the Old and New Siphons (Figure 4.30), the New Syphon can be seen to have approximately 75% excess capacity for a constant head differential. Thus, the new structure is more efficient than the old. The maximum head loss estimated by high water marks during the 1979 spring runoff event was 0.15 m, at the Old Siphon and 0.09 m at the New Siphon.

![Head Loss Rating Curves for Syphons](image)

*Figure 0-30 Headloss rating curves for Old and New Siphons (Dillon Engineering, 1985)*
4.5.6 Historical Flooding along the Welland River

The purpose of the review of historical flood levels along the Welland River is to compare the historical flood data with calculated flood levels. A review of past reports on Welland River floods (Department of Energy and Resources Management, Niagara Peninsula Conservation Report 1964; Flood Damage Reduction Study, 1985) provides two flood levels above G.S.C., datum for the 1954 October Hurricane Hazel event:

Elevation 174.65 m at Beckett’s Bridge
Elevation 173.55 m at Old Syphon outlet

The indicated flood levels were a result of the Hurricane Hazel storm; however the storm was not at its peak intensity and concentration as it passed over Niagara. In 1985 as part of the Flood Damage Reduction Study, the Conservation Authority, with assistance from local municipal officials, obtained additional information on flood levels from local residents. All information furnished by the local residents was converted to geodetic elevations by a field survey by a consultant (Dillon Engineering, 1985). This survey was carried out by the Authority and used temporary benchmarks identified on photo based contour maps (Dillon Engineering, 1985). The seven sites surveyed by the Authority range from the City of Welland to Port Davidson. One of the seven (7) sites is on Big Forks Creek of Chambers Corners.

Site  
1  173.76 m, City of Welland, Niagara Street  
2  173.85 m, City of Welland, Prince Charles Street  
3  173.91 m, City of Welland, Lincoln Street West  
4  174.93 m, City of Welland, Chambers Corners  
5  174.06 m, Becketts Bridge  
6  174.73 m, Riverview Golf Course  
7  175.23 m, Port Davidson

The 1 in 100-year levels determined during the Flood Damage Reduction Study, 1985 by Dillon Engineering are compared against the historical flood levels (Table 4.39). In previous discussions with Authority staff it was established that the 1 in 100-year event was the Regulatory event for the Welland River. The regulatory flood or standard project flood for Southern Ontario is typically Hurricane Hazel, or the greater of the 100-year storm event or
Hurricane Hazel. In 1988, Niagara Peninsula Conservation Authority petitioned the Province to reduce the Regulatory storm event from Hurricane Hazel (the current standard project flood at that time) down to the 100 year storm event. It is important to note that in the 1985 Flood Damage Reduction Study, Hurricane Hazel was determined to be approximately 1.5 m (5 ft) above the historical flood levels (Table 4.39).

According to Table 4.39 the historical flood levels were approximately 0.3 m below the estimated 100-year flood levels downstream of Becketts Bridge. In conclusion this brief comparison helped to identify the historical high water marks along the Welland River. When compared to the estimated flood levels for the 100-year of the Flood Damage Reduction Study (1985), the historical water levels came close to but stayed below the 100-year flood levels (Table 4.39).

Within the last seven (7) years conservation authority staff have made similar observations. For example, during the April 2-5\textsuperscript{th}, 2005 snowmelt event, authority staff had observed water levels to be within 0.3 to 0.6 m of the estimated 100-year water levels (Flood Damage Reduction Study, 1985). In addition, during the December 29\textsuperscript{th}, 2008 snowmelt event authority staff had observed water levels to be within 0.3 m of the estimated 100-year water levels at Port Davidson. A review of streamflow records at the Caistors Corners gauge station has revealed there to be an upwards trend in high flow events within the last several years. Furthermore, flooding along the Welland River typically occurs during two (2) periods throughout the year, during the spring freshet and the late fall period, during the months of November and December.

According to the 1985 Flood Damage Reduction Study, the 100-year event is a 5-day snowmelt plus rainfall event. The 1, 2, 3, 4 and 5-day accumulated runoff volumes for the Caistors Corners gauge station are listed in Table 4.40. The accumulated runoff volumes at the Caistors Corners gauge station are representative of average runoff conditions within the Welland River Watershed. According to Table 4.40, the 5-day accumulated runoff volume is 116 mm. The total drainage area at the Old Siphon is approximately 872 km\textsuperscript{2}, assuming zero drainage, and all of the runoff were to accumulate in the lower Welland River, the absolute maximum water level would be 178 m. This value is 2.5 m higher than the estimated 100-year water levels and 1 m higher.
than estimated water levels for Hurricane Hazel. Based on the reported 100-year water levels and flows in the 1985 Flood Damage Reduction Study, the total volume of water within the channel and/or floodplain was estimated to be on average 26,160,000 m$^3$. The total outflow volume was estimated to be on average approximately 59,296,000 m$^3$ and the initial total volume of water within the channel was estimated to be on average approximately 6,104,000 m$^3$. Based on the foregoing information the estimated 100-year total inflow runoff volume to the lower Welland River was estimated to be 79,352,000 m$^3$, 21,800,000 m$^3$ less than the observed 100-year 5-day accumulated runoff volume. Therefore, based on the foregoing analysis the reported 100-year water levels in the Flood Damage Reduction Study (1985) could potentially be an under estimation of the actual 100-year water levels within the Welland River.

<table>
<thead>
<tr>
<th>Site</th>
<th>Historical Flood Level (m)</th>
<th>100-year Flood Level (FDRP, 1985)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Niagara Street</td>
<td>173.76</td>
<td>174.14</td>
</tr>
<tr>
<td>Prince Charles Street</td>
<td>173.85</td>
<td>174.28</td>
</tr>
<tr>
<td>Lincoln Street West</td>
<td>173.91</td>
<td>174.34</td>
</tr>
<tr>
<td>Chambers Corners</td>
<td>174.93</td>
<td>174.93</td>
</tr>
<tr>
<td>Becketts Bridge</td>
<td>174.06</td>
<td>175.03</td>
</tr>
<tr>
<td>Riverview Golf Course</td>
<td>174.73</td>
<td>175.39</td>
</tr>
<tr>
<td>Port Davidson</td>
<td>175.23</td>
<td>175.50</td>
</tr>
</tbody>
</table>

### Table 0-40 Observed accumulated runoff volumes (mm) at the Caistors Corners gauge Station

<table>
<thead>
<tr>
<th>Return Period</th>
<th>1-Day</th>
<th>2-Day</th>
<th>3-Day</th>
<th>4-Day</th>
<th>5-Day</th>
</tr>
</thead>
<tbody>
<tr>
<td>2-Year</td>
<td>15</td>
<td>27</td>
<td>36</td>
<td>43</td>
<td>48</td>
</tr>
<tr>
<td>5-Year</td>
<td>20</td>
<td>37</td>
<td>50</td>
<td>59</td>
<td>66</td>
</tr>
<tr>
<td>10-Year</td>
<td>24</td>
<td>44</td>
<td>59</td>
<td>70</td>
<td>78</td>
</tr>
<tr>
<td>25-Year</td>
<td>28</td>
<td>52</td>
<td>71</td>
<td>84</td>
<td>94</td>
</tr>
<tr>
<td>50-Year</td>
<td>32</td>
<td>58</td>
<td>79</td>
<td>94</td>
<td>105</td>
</tr>
<tr>
<td>100-Year</td>
<td>35</td>
<td>64</td>
<td>88</td>
<td>104</td>
<td>116</td>
</tr>
</tbody>
</table>

#### 4.5.7 GFLOOD Model Set-Up

The purpose of setting up a new watershed model for the Welland River was to test and validate the hybrid model GFLOOD by developing a tool that can estimate discharge within the river by accounting for the physiographic characteristics of the watershed and the complex hydraulics present along the main branch of the Welland River. Previous modelling studies carried out on the Welland River assumed that computations based on steady state conditions are applicable for
the River. These methods represent conditions where flow patterns and magnitude do not vary with time. Such calculations were based on mathematical tools which do not reflect the special conditions of the River. Subsequently, the uniqueness of the Welland River was recognized and attempts were made to make allowance for the vast storage potential of the River.

In 1985, as part of the Flood Damage Reduction Study, Dillon Engineering developed an unsteady state hydraulic model using the DWOPER computer model (predecessor for FLDWAV). The DWOPER computer model was found to be more suitable for estimating flood levels along the lower part of the Welland River, primarily due to backwater effects from hydroelectric operations on the Niagara River. In turn, the hybrid GFLOOD model which integrates a hydrologic model with an unsteady state hydraulic model (one-dimensional unsteady flow model) was used to estimate the discharge and water levels along the main branch of the Welland River between Binbrook Dam and Montrose.

The process for developing a hydrologic – hydraulic model for the Welland River Watershed is as follows:

- Select a rainfall / runoff simulation model appropriate for the Study area (Hybrid hydrologic – hydraulic model: GFLOOD was selected);
- Collect data and build model input files;
- Calibrate the model for two different time periods and/or events over the course of a hydrologic year (November to October); and
- Validate the model for a different time periods and/or events over the course of a hydrologic year (November to October).

Given the homogeneous nature of the watershed, runoff volumes within the Welland River were estimated from the Caistors Corners gauge station and the Oswego Creek gauge station. The equations for determining the time parameters, which were introduced in Chapter 3, along with the linear reservoir routing method were used to convolute and estimate the overland flow hydrographs, which would act as lateral inflow to the hydraulic modelling component of the
GFLOOD model. The input parameters required to estimate the runoff hyetographs and the parameters used by the linear reservoir method for estimating overland flows are as follows:

- Hourly climate information including rainfall, snowfall, temperature and potential evaporation;
- Basin physical characteristics;
- Watercourse characteristics; and
- Storage facility characteristics.

The basin physical characteristics required to calculate the overland flow hydrographs include the following:

- Runoff volumes;
- Drainage areas;
- Impervious areas; and
- Hydrograph base times that are composed of time of concentration and recession constant values.

4.5.7.1 Hourly Climate Information

Hourly climate information including rainfall, snowfall, temperature and potential evaporation for the Welland River watershed was determined using the climate model WeatherGEN (Appendix B). The WeatherGEN model accepts as input from Meteorological Services of Canada (MSC) climate stations daily precipitation, hourly rainfall, maximum temperature, minimum temperature, wind speed and relative humidity values. In addition, long-term monthly means for the following parameters were also entered into the WeatherGEN model: rainfall, snowfall, maximum temperature, minimum temperature, wind speed, maximum relative humidity, minimum relative humidity and number of wet days per month. The WeatherGEN model is used to calculate the hourly snowfall, hourly temperature, relative humidity, wind speed, solar radiation and potential evaporation. The WeatherGEN model is also used to
calculate missing weather data, such as hourly rainfall, and replace missing values using a “filling-in” procedure.

In watershed modelling, meteorological inputs can vary significantly with location. To account for these variations, the GFLOOD model accepts inputs on the basis of separate Zones of Uniform Meteorology (ZUM). A ZUM (Schroeter et al, 1992) is defined as “a portion of a watershed throughout which one set of meteorological measurements can be used to calculate snowmelt and runoff”. The ZUMs are further refined or ‘downscaled’ to the subcatchment level using the inverse distance squared method. The distance between the centroid of each sub basin and the climate stations is calculated using equation 4.157.

\[ d_i^2 = (x_i - x_{STN})^2 + (y_i - y_{STN})^2 + (z_i - z_{STN})^2 \]  

(4.157)

Where \( d_i \) is the distance between the sub-basin and the climate station; \( x_i, y_i \) and \( z_i \) are the northing, easting and elevation of the sub-basin centroid; and \( x_{STN}, y_{STN} \) and \( z_{STN} \) are the northing, easting and elevation of the climate station. A subroutine within the GFLOOD model has been set-up to calculate the precipitation, snowfall, temperature and potential evaporation for each sub basin by incorporating each of the climate stations surrounding the watershed using the inverse distance squared method (equation 4.158).

\[ P = \frac{\sum_{i=1}^{N} P_i}{\sum_{i=1}^{N} \frac{1}{d_i^2}} \]  

(4.158)

Where \( P_i \) is the climate parameter value and \( d_i \) has been previously defined.

The following Meteorological Service of Canada (MSC) stations were used to represent each ZUM “Zones of Uniform Meteorology” within the watershed: Hamilton Airport station, Vineland RCS station, ST Catharines Airport station and Port Colborne station. The ZUMs were established using Thiessen Polygon Method, Figure 4.31 illustrates the discretization of ZUMs.
within the watershed. Each MSC station(s) represent a specific ZUM, for example, Hamilton Airport and Hamilton RBG stations represent ZUM 1, Vineland RCS station represents ZUM 2, ST Catharines Airport station represents ZUM 3 and Port Colborne station represents ZUM 4. In addition, to the above weather parameters, the following information is inputted into the WeatherGEN model for each station: UTM coordinates, station latitude, angular velocity of the earth’s rotation \( (\omega, \text{rad/hr}) \), scaling factor \( (b_H) \) that controls the degree of deviation in relative humidity caused by the presence or absence of precipitation, and an empirical coefficient for estimating the short wave solar radiation \( (k_r) \). Table 4.41 lists the different parameters used in the WeatherGEN model for the Welland River.

The inverse distance squared methodology was used to estimate a weighted average meteorological time series for each sub-basin within the Welland River. The distance between the centroid of the sub-basin and each climate station was calculated using equation 4.157. A weighted average meteorological time series for each sub-basin incorporating all climate stations within the watershed was determined using equation 4.158. The following climate parameters were determined for each sub-basin: hourly rainfall, hourly snowfall, hourly temperature and hourly potential evaporation. The WeatherGEN model was simulated for the following time periods: November 1\(^{st}\), 1999 to October 31\(^{st}\), 2000; November 1\(^{st}\), 2000 to October 31\(^{st}\), 2001; November 1\(^{st}\), 2001 to October 31\(^{st}\), 2002; November 1\(^{st}\), 2002 to October 31\(^{st}\), 2003; November 1\(^{st}\), 2003 to October 31\(^{st}\), 2004; and November 1\(^{st}\), 2004 to October 31\(^{st}\), 2005.

4.5.7.2 Model Discretization

The Welland River watershed was subdivided into 28 sub-basins for the hydrologic modelling component. Points of interest were used as the starting points for the discretization of the watershed. The points of interest were based on land use, confluence of major tributaries, and physiography. The drainage area for each sub-basin was abstracted from the 2006 Ministry of Natural Resources Water Resources Information Program (WRIP) geo-spatial data layers, provided by Niagara Peninsula Conservation Authority and are based upon Ontario Base Mapping (OBM) 1:10,000 scale mapping. Drainage boundaries were determined from field investigations, topographic mapping with scales of 1:10,000 and 5 m contour intervals and aerial
photographs. The sub-basins for the Welland River watershed are illustrated in Figure 10.29 and listed in Table 10.34. The hydrologic modelling parameters that were used to construct the lateral inflow and upstream hydrographs were extracted from the geo-spatial data layers provided by Niagara Peninsula Conservation Authority using the HEC-GeoHMS and ArcHydro data models. Parameters extracted included drainage area, overland flow lengths, basin lengths, basin widths, basin slopes, basin curve numbers, reach lengths and reach slopes.

Hydrograph characteristics were developed on the basis of individual basin properties and several analyses of recorded runoff events, which include: the surface-storage recession constant ($K_{surf}$); and the time of concentration ($T_c$). In gauged watersheds $K_{surf}$ and $T_c$ can be determined from recorded runoff events.

Utilizing the recorded data from the Caistor Corners gauge station the time of concentration ($T_c$) was estimated from five different recorded hydrograph events. $T_c$ was taken to be the time in hours from the start of basin response to the inflection point in the recession limb of the hydrograph. The surface storage recession constant ($K_{surf}$) was determined by the following equation:

$$K = \frac{t}{2.3\log\left(\frac{q_o}{q_t}\right)}$$

Where:

- $t =$ time interval in hours between two points on the recession limb of the hydrograph
- $Q_o =$ discharge at the first point in $m^3/s$
- $Q_t =$ discharge at the second point in $m^3/s$

The resulting watershed parameters determined for the five recorded runoff events are presented in Table 4.42. Watershed parameters were further developed for each sub-basin (reference Figure 4.32) and reach in the Welland River watershed based on individual hydrologic characteristics and on the results of the analyses of the recorded runoff events. The time of concentration for the
sub-areas and the reach local inflows have been determined based on estimated velocities from recorded runoff events according to the following equation:

\[ T_c = \frac{L}{V} \]  

Where \( L \) is the hydraulic length in metres; and \( V \) is the estimated velocity in m/s. Average velocities were estimated by dividing the basin length by the time of concentration from the recorded runoff hydrographs of the Caistor Corners gauge station. The average velocity of the reviewed recorded runoff events was approximately 0.43 m/s (Table 4.42). The estimated velocities for the sub-basins were based on this analysis in addition to taking into consideration individual characteristics such as the overland flow length and the sub-basin slope. The velocities for the sub-basins were estimated to be in the order of 0.23 to 0.64 m/s. The lengths, velocities and the resultant \( T_c \) and \( K_{surf} \) for the hydrographs of the sub-areas as utilized in the hydrologic modelling component of GFLOOD are illustrated in Figure 4.32. The surface storage recession constants \( (K_{surf}) \) for the sub-basins were estimated using the regression equation for \( K_{surf} \) (Chapter 5, Results and Discussion). The observed values for \( T_c \) and \( K_{surf} \) for the sub-basins were compared against the computed values. The results are listed in Table 4.43.

According to Table 4.43, the calculated values of \( K_{surf} \) and \( T_c \) are approximately 2-3 times higher on average than the observed values of \( K_{surf} \) and \( T_c \) respectively. The observed values of \( K_{surf} \) and \( T_c \) assume an average constant reference flow velocity for all sub-basins, whereas the calculated values of the time of concentration and surface storage recession constant are based on the physical properties of the sub-basins. The physical properties of the sub-basins include overland flow length, basin slope and elongation ratio. The physical properties of the sub-basins are listed in Table 4.44. The observed values for \( K_{surf} \) and \( T_c \) were used as an initial estimate and then were adjusted based on the calibration procedure described in Section 4.5.8.1.
Figure 0-31 Zones of uniform meteorology (ZUMs) for the Welland River Watershed

Table 0-41 Input parameters for the WeatherGEN model of the Welland River

<table>
<thead>
<tr>
<th>Station</th>
<th>UTM Coordinates</th>
<th>Station Latitude</th>
<th>( \omega ) (rad/hr)</th>
<th>( b_H )</th>
<th>( k_r )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hamilton Airport</td>
<td>586707 4779875 236</td>
<td>43.1717</td>
<td>0.2618</td>
<td>0.9</td>
<td>0.16</td>
</tr>
<tr>
<td>Hamilton RBG</td>
<td>590599 4792885 102</td>
<td>43.2833</td>
<td>0.2618</td>
<td>0.9</td>
<td>0.16</td>
</tr>
<tr>
<td>Vineland RCS</td>
<td>630026 4782417 79</td>
<td>43.1833</td>
<td>0.2618</td>
<td>0.9</td>
<td>0.16</td>
</tr>
<tr>
<td>ST Catherines Airport</td>
<td>648948 4784656 97</td>
<td>43.2000</td>
<td>0.2618</td>
<td>0.9</td>
<td>0.16</td>
</tr>
<tr>
<td>Port Colborne</td>
<td>642911 4749345 175</td>
<td>42.8833</td>
<td>0.2618</td>
<td>0.9</td>
<td>0.16</td>
</tr>
</tbody>
</table>

Table 0-42 Watershed base time parameters based on historical runoff events at Caistors Corners gauge station

<table>
<thead>
<tr>
<th>Recorded Event</th>
<th>Surface Storage Recession Constant ( K_{surf} ) (hours)</th>
<th>Time to Peak ( T_p ) (hours)</th>
<th>Time Concentration ( T_c ) (hours)</th>
<th>Average Velocity (m/s)</th>
<th>Watershed Parameter ( K/Tp )</th>
</tr>
</thead>
<tbody>
<tr>
<td>April 20-24, 2000</td>
<td>23.00</td>
<td>35.00</td>
<td>47.00</td>
<td>0.37</td>
<td>0.66</td>
</tr>
<tr>
<td>May 1-5, 2003</td>
<td>28.00</td>
<td>48.00</td>
<td>62.00</td>
<td>0.28</td>
<td>0.58</td>
</tr>
<tr>
<td>December 3-8, 1970</td>
<td>20.70</td>
<td>22.00</td>
<td>35.83</td>
<td>0.49</td>
<td>0.94</td>
</tr>
<tr>
<td>December 6-10, 1972</td>
<td>27.30</td>
<td>20.00</td>
<td>29.00</td>
<td>0.60</td>
<td>1.37</td>
</tr>
<tr>
<td>December 13-17, 1972</td>
<td>33.20</td>
<td>20.00</td>
<td>44.00</td>
<td>0.40</td>
<td>1.66</td>
</tr>
</tbody>
</table>
Table 0-43 Sub-basin estimated and calculated recession constant values ($K_{surf}$) and time of concentration ($T_c$) values

<table>
<thead>
<tr>
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Figure 0-32 Sub-Basins of the Welland River watershed
**4.5.7.3 External and Internal Boundary Conditions**

External boundary conditions are required at all model limits for the hydraulic modelling component of the GFLOOD model. For this study, a discharge hydrograph and a water level hyetograph were the two (2) mechanisms used to drive the model, and the following two (2) boundaries have been defined: Binbrook Dam – Upstream flow hydrograph boundary condition and Montrose – Downstream water level hyetograph boundary condition. It is important to note that water levels at the Grass Island Pool fluctuate between 171.58 and 170.58 m (GSC) in accordance with the 1993 directive of the International Niagara Board of Control for maximum and minimum water levels at the Grass Island Pool, the average being 171.14 m (GSC). The fluctuating water levels at the Grass Island Pool are reflected in the recorded water levels at Montrose based on the available hydrometric data (2000-2001) and (2002-2005).

Internal boundary conditions include lateral inflows, levees, locks, bridges and dams. Lateral inflow hydrographs from tributaries have been defined at: West Wolf Creek; Elsie Creek; Wolf Creek; Mill Creek; Beaver Creek; Suckers Creeks; Oswego Creek; Big Forks Creek; Coyle Creek; Draper Creek; and Two unnamed tributaries. The discharge hydrographs within the hydraulic modelling component of the GFLOOD model were generated using the runoff volumes from the Caistors Corners gauge station and Oswego Creek gauge stations in conjunction with the linear reservoir methodology for estimating direct runoff to the streams or main channel. The model was set-up to simulate the overland flow hydrographs for the 28 sub-basins as defined for the Welland River watershed. The model uses the runoff volume for each sub-basin and the linear reservoir methodology to simulate the direct runoff hydrograph to the main channel. All channel flood routing was carried out by the hydraulic model of GFLOOD, which is based on the one-dimensional unsteady flow model. For each calibration and validation event the surface flow and baseflow were separated using the method by Nathan and McMahon (1990). However, given that the watershed lies upon the Haldimand clay plain, the watershed is predominantly surface flow driven, with baseflows accounting for less than 3% of the total flow to the main branch of the Welland River. Runoff volumes were then estimated for each event for both surface flows and baseflows. The rainfall hyetographs for each event and sub-basin were then prorated to determine the runoff hyetograph for the event and sub-basin. The time parameter equations along
with the linear reservoir method were then used to determine the direct runoff to the river for both surface flows and baseflows. Two (2) separate overland surface flow hydrographs were set-up for each sub-basin; one for pervious areas and one for directly connected impervious areas. For the directly connected impervious areas all water input was assumed to be runoff.

Bridges were model as hydraulic control points within the GFLOOD model. The openings for each of the bridges including the floodplain and ineffective flow areas (off-channel storage) were modelled within GFLOOD. A table of elevation versus top width was used to model the bridge crossings. Detailed channel information was made available through the MIKE-11 modelling files for the Welland River. The cross-sections in the MIKE-11 model were extracted from the triangulated irregular network (TINs) for the Welland River watershed. The Mike-11 crossings represent the channel and floodplain portions of the cross-sections. The Mike-11 cross-sections were imported into the HEC-RAS model and were exported to HEC-RAS. The geometric pre-processor for unsteady flow computations in HEC-RAS was used to create the table of top widths and elevations for each bridge opening. This process was used not only to compute the table of top widths and elevations for the bridge openings but also for the cross-sections between the bridges. In channel and bridge sections where MIKE-11 files were not available for the model crossings, HEC-2 files provided by Niagara Peninsula Conservation Authority were used instead.

To compute water levels at the siphons, there are two (2) different methods that have been previously used: the siphons were modelled as a dam or bridge and were represented by a head loss rating curve (Dillon Engineering, 1985); or the siphons were represented by a combined rating curve at the downstream end (Klohn Crippen Berger, Dam Break Analysis Binbrook Dam, 2003). The use of a singled valued rating curve at the downstream end is a conservative assumption provided that the water level in the Grass Island Pool remains constant at 171.14 m (GSC). In this study the downstream boundary was defined by a water level hyetograph at Montrose. Given that the downstream boundary condition has been defined at Montrose, the option of using a singled-value rating to represent the siphons is no longer applicable, since, only one downstream boundary condition can be defined within the GFLOOD model.
In the 1985 Flood Damage Reduction Study the siphons were modelled as bridges and headloss rating curves were used as internal boundary conditions at the siphons within the DWOPER model. This method has a tendency to over-estimate upstream water levels. However, the simplified dynamic model may be used to simulate unsteady flows which can change from free surface flow to pressurized flow from one section to another and/or as the flow changes with time. When the flow, passing through a section of closed conduit of any shape, completely submerges the section; the flow properties change from those of free-surface to pressurized flow. In the latter type of flow, disturbances in the flow are propagated at velocities many times greater than those for free surface flow. In turn, the siphons were modelled as closed conduits with upstream and downstream cross-sections defined and the conduit geometry adjusted until the headloss rating curves between the upstream and downstream cross-sections matched the theoretical headloss rating curves developed in the 1985 Flood Damage Reduction Study. A 25% blockage was assumed for the Old Siphon due to build-up of sediment and debris.

4.5.7.4 Selection of Computational Distance and Time Step Criteria

It is most important that computational distance steps $\Delta x$ in the finite-difference Saint Venant Equations, be properly selected in order to avoid computational difficulties and to achieve an acceptable level of numerical accuracy. When the computational distance step chosen is too large, the resulting truncation error the difference between the true solution of the partial differential Saint-Venant Equations, and the approximate solution of the finite-difference Saint-Venant Equations, may be so large that the computed solutions of $h_i$ and $Q_i$ are totally unrealistic, for example, the computed flow depths have negative values. This causes the program to abort since a negative depth or cross-sectional area is raised to a power which is necessary when computing the friction slope ($S_f$). Large truncation errors can also cause irregularities in the computed hydrograph as manifested by spurious spikes in the rising and/or falling limbs.

Three criteria are used to select the computational distance steps. The first of the criteria is related to contracting and/or expanding cross sections by Samuels (1985); the second criteria was developed by Fread and Lewis (1993) which relates the computational distance step to the wave
celerity, time of rise in the hydrograph and an empirical value \( m \) (equation 4.162), which can range between 5 and 40, typically \( m \) is equal to 20; and the third criteria developed by (Fread, 1988) is related to significant changes in the channel bottom slope. Wherever the channel bottom slope abruptly changes, smaller computational distance steps, say one-fifth to one-half of those required by the second criteria (equation, 4.162) are determined. Also, wherever the flow changes from subcritical to supercritical or vice versa, the computational distance step should be smaller.

The criteria used for selecting the distance between computational points or sections was based on the Courant Condition:

\[
\Delta x = c \Delta t 
\]  

(4.161)

Where \( \Delta x \) is the distance between cross-sections, \( \Delta t \) is the time step and \( c \) is the flood wave velocity, equal to 1.5 times the average channel velocity. The value for \( \Delta t \) is determined from equation 4.162. Assuming an average channel velocity of 1 m/s and a time step of 1-hour; results in a distance of approximately 5400 m between cross-sections. Sections used to represent the main branch of the Welland River fall within these criteria.

Equally important to the computational distance steps \( \Delta x \) are the computational time steps \( \Delta t \). Their proper selection prevents the occurrence of numerical difficulties due to excessive truncation errors in the finite-difference approximate solution of the Saint-Venant equations. Also, if the computational time steps are too large, the user-specified hydrograph or in the case of a dam break, the hydrograph generated by the breaching of the dam will not be accurately characterized, for example, if the time steps are too large, the peak of the hydrograph can be ignored as the time steps \( \Delta t \) step through and actually bypass the hydrograph peak. To ensure small truncation errors and to properly treat the hydrograph peak, the following criteria are used to select \( \Delta t \): (1) the selected time step \( \Delta t \) is evenly divisible into the smaller of either the time of rise of the user-specified hydrograph or the time of failure of the breach; usually the latter is sufficiently small such as to also cause the time step to coincide with the peak of the user-specified hydrograph; (2) the time of rise \( (T_r) \) of the user-specified hydrograph or the time of
failure of the breach is divided by a factor \( m \), where \( 5 \leq m \leq 40 \); usually a value of 20 is sufficiently large to produce computational time steps sufficiently small so as to minimize truncation errors. The selection of time step for computation of flows and levels was based on the following equation:

\[
\Delta t = \frac{T_r}{m}
\]

(4.162)

Where \( \Delta t \) is the time step, \( T_r \) is the time of rise in the hydrograph and \( m \) is previously defined. The time of rise for the 100-year design event is approximately 48 hours. Assuming \( m \) to be equal to 20, a time step of 2.5 hours would result. This value was found to be unacceptable as it would mask some high peaks. However, for some basins according to Table 4.43, the time-of-concentration could be as low as 2.4 hours therefore, the time step should be at a minimum of 30 minutes to one-hour. However, in order to ensure stability in the computations of the unsteady state hydraulic model; a time step of 1-minute was selected for the routing computations and for the overland hydrograph computations.

4.5.7.5 Selection of Routing Reaches

For the main branch of the Welland River the hydraulic model channel bottom width, channel inverse side slope ratio, the inverse side slope ratio of the left and right floodplain, left and right floodplain bottom widths, top-of-bank elevation, and bottom elevation were defined for each section. Cross sections should represent the channel and floodplain portions which convey flow and should not include areas where flow velocities are small or nil; these areas are modeled as off-channel storage. Cross-sections were taken where changes in channel slope, width or roughness occur. Cross-sections in the upper reaches of the Welland River were taken where changes in slope occur.

All the data for the sections including locations were taken from 1:2000 or 1:5000 scanned topographic maps, Digital Elevation models (DEMs), Digital Terrain Models (DTMs), HEC-RAS models, HEC-II models, dam break models and bathymetry data supplied by Niagara Peninsula Conservation Authority. In the case where the section is not large enough to contain
the flow, the hydraulic model internally extends the end points of the section, vertically, to convey the flow downstream.

Bridges and siphons represent hydraulic control points along the main branch of the Welland River. Three cross sections are used to represent each bridge crossing, where applicable, one cross-section was located 0.8 km upstream, and one cross-section was located 1.7 km downstream to represent fully expanded flow, and one cross-section was located along the centre-line of the bridge to represent contracted flow. Although bridge sections were modeled along the main branch of the Welland River, the top decks of the bridges were not modelled. Four cross-sections were used to represent each siphon. Where applicable, one cross-section was located 0.3 - 0.8 km upstream, and one cross-section was located 1.0 - 1.7 km downstream to represent fully expanded flow, one cross-section was located on the upstream face of the bridge and one cross-section was located on the downstream face of the bridge, in order to represent contracted flow.

Off-Channel storage was represented by the bottom-width of the inactive portion of the floodplain and the inverse side slope ratio of the inactive portion of the floodplain. A review of flood plain maps and previous reports including the Flood Damage Reduction Study were used to determine the areas where flow velocities were expected to be small or nil and were specified as off-channel storage. Cross-sections which included off-channel storage were input directly into HEC-RAS and were indicated as ineffective flow areas; the model then calculated the off-channel storage between cross-sections using the pre-processor in the unsteady state computation program.

The number and location of reaches used to model the main branch of the Welland River are listed in Table 4.45. In Table 4.45 XT represents the cumulative distance along the main branch of each reach, $\Delta x$ is the distance between cross-sections along each reach, and storage represents the off-channel storage in each reach. The value of $\Delta x$ is set to satisfy the Courant Condition in the unsteady state hydraulic model. The Manning’s “$n$” values are listed in Table 4.46. A Manning’s “$n$” value is set for channel portion, left floodplain portion and right floodplain portion along each $\Delta x$ reach. A composite Manning’s “$n$” value is calculated for each stage.
There is one dam along the main branch of the Welland River, Binbrook Dam. The surface area – stage values for the Binbrook reservoir are tabulated in Table 4.47 and the rating curve for the spillway for the Binbrook Dam is tabulated in Table 4.48. The surface area – stage values for the Binbrook reservoir and the rating curve for the spillway for the Binbrook dam were provided by Niagara Peninsula Conservation Authority. The hydraulic model was used to route flows through the Binbrook Reservoir using level-pool routing and along the main branch of the Welland River using the simplified dynamic model.

The Froude number is calculated for each $\Delta x$ reach. For subcritical flow, computations proceed in the upstream direction if the downstream condition influences flows and water levels. For supercritical flow computations must proceed in the downstream direction. In addition, reaches along the Welland River were identified as not being impacted by the downstream conditions or the backwater from the Niagara River. In this case, computations proceed in the downstream direction under subcritical flow conditions (i.e., reach between Binbrook Dam and Port Davidson). If a flood wave was to propagate downstream between Port Davidson and the New Siphon, computations would also proceed in the downstream direction since the impacts at the New Siphon due to the backwater from the Niagara River, would be minimal, approximately 2-3 cm increase in water surface elevations (Dillon Engineering, 1985).
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<th>Δx (km)</th>
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<td>Boyle Road</td>
<td>73.85</td>
<td>5.00</td>
<td>5000</td>
<td>Bridge</td>
</tr>
<tr>
<td>22</td>
<td>Beckett’s Bridge</td>
<td>78.20</td>
<td>0.80</td>
<td>1500</td>
<td>Bridge</td>
</tr>
<tr>
<td>23</td>
<td>O’Reilly’s Bridge</td>
<td>84.60</td>
<td>0.10</td>
<td>5000</td>
<td>Bridge</td>
</tr>
<tr>
<td>24</td>
<td>Lincoln St.</td>
<td>90.00</td>
<td>0.30</td>
<td>1700</td>
<td>Bridge</td>
</tr>
<tr>
<td>25</td>
<td>Old Siphon</td>
<td>92.10</td>
<td>0.00</td>
<td>-</td>
<td>Siphon</td>
</tr>
<tr>
<td>26</td>
<td>New Siphon</td>
<td>97.60</td>
<td>1.00</td>
<td>-</td>
<td>Siphon</td>
</tr>
</tbody>
</table>
Table 0-46 Manning’s “n” values along the main branch of the Welland River

<table>
<thead>
<tr>
<th>Reach No.</th>
<th>Location</th>
<th>Channel</th>
<th>Right Floodplain</th>
<th>Left Floodplain</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Binbrook Dam</td>
<td>0.025</td>
<td>0.035</td>
<td>0.035</td>
</tr>
<tr>
<td>2</td>
<td>Harrison Road</td>
<td>0.035</td>
<td>0.060</td>
<td>0.060</td>
</tr>
<tr>
<td>3</td>
<td>Golf Course</td>
<td>0.035</td>
<td>0.060</td>
<td>0.060</td>
</tr>
<tr>
<td>4</td>
<td>RR 56</td>
<td>0.040</td>
<td>0.060</td>
<td>0.060</td>
</tr>
<tr>
<td>5</td>
<td>Hwy 56</td>
<td>0.040</td>
<td>0.060</td>
<td>0.060</td>
</tr>
<tr>
<td>6</td>
<td>Trimble Road</td>
<td>0.035</td>
<td>0.055</td>
<td>0.055</td>
</tr>
<tr>
<td>7</td>
<td>Above Landfill</td>
<td>0.040</td>
<td>0.060</td>
<td>0.060</td>
</tr>
<tr>
<td>8</td>
<td>Hall Road</td>
<td>0.030</td>
<td>0.045</td>
<td>0.045</td>
</tr>
<tr>
<td>9</td>
<td>Woodburn Road</td>
<td>0.030</td>
<td>0.052</td>
<td>0.052</td>
</tr>
<tr>
<td>10</td>
<td>Sinclairville Road</td>
<td>0.030</td>
<td>0.051</td>
<td>0.051</td>
</tr>
<tr>
<td>11</td>
<td>Westbrook Road</td>
<td>0.040</td>
<td>0.045</td>
<td>0.045</td>
</tr>
<tr>
<td>12</td>
<td>Below Westbrook</td>
<td>0.040</td>
<td>0.045</td>
<td>0.045</td>
</tr>
<tr>
<td>13</td>
<td>Caistorville</td>
<td>0.030</td>
<td>0.055</td>
<td>0.055</td>
</tr>
<tr>
<td>14</td>
<td>Abingdon Road</td>
<td>0.030</td>
<td>0.055</td>
<td>0.055</td>
</tr>
<tr>
<td>15</td>
<td>Warner Road</td>
<td>0.030</td>
<td>0.055</td>
<td>0.055</td>
</tr>
<tr>
<td>16</td>
<td>Church Road</td>
<td>0.030</td>
<td>0.055</td>
<td>0.055</td>
</tr>
<tr>
<td>17</td>
<td>Attercliffe Road</td>
<td>0.030</td>
<td>0.055</td>
<td>0.055</td>
</tr>
<tr>
<td>18</td>
<td>Port Davidson</td>
<td>0.030</td>
<td>0.055</td>
<td>0.055</td>
</tr>
<tr>
<td>19</td>
<td>Below Oswego Creek</td>
<td>0.030</td>
<td>0.055</td>
<td>0.055</td>
</tr>
<tr>
<td>20</td>
<td>Wellandport</td>
<td>0.030</td>
<td>0.055</td>
<td>0.055</td>
</tr>
<tr>
<td>21</td>
<td>Boyle Road</td>
<td>0.030</td>
<td>0.055</td>
<td>0.055</td>
</tr>
<tr>
<td>22</td>
<td>Beckett’s Bridge</td>
<td>0.030</td>
<td>0.055</td>
<td>0.055</td>
</tr>
<tr>
<td>23</td>
<td>O’Reilly’s Bridge</td>
<td>0.030</td>
<td>0.055</td>
<td>0.055</td>
</tr>
<tr>
<td>24</td>
<td>Lincoln St.</td>
<td>0.030</td>
<td>0.055</td>
<td>0.055</td>
</tr>
<tr>
<td>25</td>
<td>Old Syphon</td>
<td>0.030</td>
<td>0.055</td>
<td>0.055</td>
</tr>
<tr>
<td>26</td>
<td>New Syphon to Montrose</td>
<td>0.030</td>
<td>0.055</td>
<td>0.055</td>
</tr>
</tbody>
</table>

Table 0-47 Surface area -- stage values for the Binbrook reservoir (Niagara Peninsula Conservation Authority)

<table>
<thead>
<tr>
<th>Surface Area (km²)</th>
<th>2.9</th>
<th>2.7</th>
<th>2.3</th>
<th>1.5</th>
<th>1.0</th>
<th>0.7</th>
<th>0.4</th>
<th>0.0</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stage (m, GSC)</td>
<td>201.78</td>
<td>201.17</td>
<td>199.64</td>
<td>198.10</td>
<td>196.60</td>
<td>195.07</td>
<td>193.55</td>
<td>190.80</td>
</tr>
</tbody>
</table>

Table 0-48 Rating curve for the spillway for the Binbrook dam (Niagara Peninsula Conservation Authority)

<table>
<thead>
<tr>
<th>Reservoir Elevation (m)</th>
<th>Discharge Capacity (m³/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>198.8</td>
<td>0.0</td>
</tr>
<tr>
<td>199.0</td>
<td>1.7</td>
</tr>
<tr>
<td>199.2</td>
<td>5.0</td>
</tr>
<tr>
<td>199.4</td>
<td>10.4</td>
</tr>
<tr>
<td>199.6</td>
<td>17.4</td>
</tr>
<tr>
<td>199.8</td>
<td>38.4</td>
</tr>
<tr>
<td>200.0</td>
<td>73.3</td>
</tr>
<tr>
<td>200.2</td>
<td>111.4</td>
</tr>
<tr>
<td>200.4</td>
<td>155.1</td>
</tr>
<tr>
<td>200.6</td>
<td>203.5</td>
</tr>
<tr>
<td>200.8</td>
<td>259.5</td>
</tr>
<tr>
<td>201.0</td>
<td>321.1</td>
</tr>
</tbody>
</table>
4.5.8 Calibration and Validation of the GFLOOD Model for the Welland River

Computer simulation models usually require the calibration and validation of parameters which cannot be accurately measured to simulate the rainfall – runoff processes. Calibration is recommended to adjust those input parameters until a good agreement is obtained between measured and simulated output hydrographs. Calibration is a systematic process where model results match acceptably the observed data. The quantitative measure of the match is described by the objective function. In hydrologic and hydraulic modelling this function measures the degree of variation between the computed and observed hydrographs and hyetographs respectively. The calibration process finds the optimal parameters that minimize the objective function. This process can be either manual or automated (optimization). Madsen et al. (2002) provides a comparison of different strategies for the calibration of watershed models. Novel approaches to watershed model calibration can be found in Yu and Yang (2000), Madsen (2000), or Eckhardt and Arnold (2001). Model validation is a process of testing the model’s ability to simulate observed data other than those used for the calibration, with acceptable accuracy. During this process, calibrated model parameters are not subject to change as their values are kept constant.

4.5.8.1 Calibration Procedure

The calibration procedure adopted for calibrating the GFLOOD model for event simulation was a systematic approach to manual calibration. The systematic manual calibration relied on the measured and estimated values of the model parameters available from Niagara Peninsula Conservation Authority. This ensured that a physically – meaningful set of initial parameters was used for the calibration. In the next step, a calibration scheme was defined, which systematically changed the value of a given parameter while keeping the remaining parameters constant. A 10% increase and/or decrease step was used to linearly change parameter values until the soft limits were reached. The soft limits were defined as the 25% - 175% of the initial parameter value (initial ±75%), which encompassed all reasonably expected values. The definition of the soft limits was also confronted with the information on the parameter values available in the literature (Schroeter and Associates, 1996; USACE, 2000; and Nietsch et al, 2005).
The input data for the event model are available in an hourly time step. Since some small sub-basins in the study area have times of concentration as low as 2.5 hours, a computational time step of 15-minutes was selected for the hydrologic modelling portion of the GFLOOD model. For the hydraulic modelling component the program was set-up to allow the model to select the appropriate computational time-step based on the Courant Condition for routing flows along the channel.

The following tools were used to assess the calibration of the GFLOOD model: the flow comparison graphs or the water level comparison graphs showing the modelled and observed hydrographs and/or hyetographs at the calibration locations; and statistical performance criteria previously described in Section 4.1.2.3. Each optimization output was assessed according to the prescribed statistical criteria. If the output was acceptable, the calibration process was completed, otherwise the initial optimization parameters were adjusted and the process was repeated. Since there are no hydrometric stations that represent the outlets of single sub-basins, the calibration process started with hydrometric stations representing more than one sub-basin at the outlet. At which point, the parameters of ungauged contributing sub-basins were also estimated. In the final stage individually calibrated sub-basins were linked into one model and the calibration finalized.

Calibration of the Welland River GFLOOD model was undertaken at two locations; Welland River at Caistors Corners (total flow station) and Welland River upstream of the Old Syphon (water level station). Given the physiographic differences between the upper and lower portions of the Welland River watershed, two sets of parameters were set-up for the upper and lower portions of the Welland River watershed.

Two (2) events were selected for calibrating and two (2) events were selected for validating the GFLOOD model to the Caistors Corners gauge station and the overall flow hydrograph. In addition, two (2) events were selected for calibrating and two (2) events were selected for validating the FLDWAV model to the Old Syphon gauge station and the overall water level hyetograph. The criteria used to select the events for calibration and validation are as follows: data availability; hourly streamflow data was made available for the Caistors Corner gauge between January 1, 2000 to December 31, 2005 by Water Survey of Canada; Hourly water level
data for the Old Syphon was provided by Niagara Peninsula Conservation Authority for the period January 1, 2001 to December 31, 2008; Hourly water level data for the Grass Island Pool was provided by Niagara Peninsula Conservation Authority (from Ontario Power Generation) for the period January 1, 1989 and December 31, 2006; Hourly rainfall data was provided by Meteorological Services of Canada for the period January 1, 1998 to December 31, 2005 for the following climate stations: Hamilton Airport, Vineland RCS, ST Catharines Airport and Port Colborne. The event(s) generated between 10 and 25 mm of runoff; this criterion is similar to the criterion set out by the Ministry Natural Resources Technical Guidelines for Floodplain management (1986) for the purposes of hydrologic model calibration; and the event(s) were uniform across the watershed; for the purposes of calibration, events were selected on the basis that they were uniform across the watershed and were not isolated to one region. These events typically occur during the mid spring season or late April to May period and since ground conditions are still relatively saturated they tend to yield high runoff values. Event(s) of the thunderstorm type which are isolated and occur during the summer or late June to mid-September period were discounted for the purposes of calibration. During this period antecedent conditions are relatively dry and such events tend to yield low runoff values. Therefore, they become unsuitable for calibration of a model used to estimate flood flows. However, it is important to note that for some events precipitation amounts were concentrated in the upper watershed and no precipitation was observed within the lower watershed. In this case, the rainfall for the lower watershed was turned off in the model.

4.5.8.2 Model Validation Procedure

In the validation procedure for the GFLOOD model adopted in this study, only the input parameters describing basin conditions (i.e. measured rainfall volumes and measured runoff volumes) were changed over time. All other parameters of the GFLOOD model were kept constant during the model validation. The validation output was assessed by flow comparison graphs, water level comparison graphs, scatter graphs, residual graphs, and statistical goodness-of-fit measures described in Section 4.1.2.3.
Based on the above criteria the events used for the purposes of calibration and validation are as follows: for the flow Hydrograph at the Caistors Corners Station; April 20-24, 2000 (calibration event); May 13-17, 2002 (calibration event); and November 27-December 4, 2003 (validation event); and May 23-27, 2004 (validation event). For the water level hyetograph at the Old Siphon gauge station; May 13-17, 2002 (calibration event); May 1-6, 2003 (calibration event); September 27 – October 1, 2003 (validation event); and May 23-27, 2004 (validation event).

4.5.9 Sensitivity Analysis

Sensitivity analysis is a method to determine which parameters of the model have the greatest impact on the model results. It ranks model parameters based on their contribution to overall model error in model predictions. Sensitivity analysis can be local and global (Haan, 2002). In the local sensitivity analysis, the effect of each input parameter is determined separately by keeping other model parameters constant. The result is a set of sensitivity functions, one for each model parameter. In the global sensitivity analysis all model inputs are allowed to vary over their ranges at the same time. Global sensitivity is based on the use of probabilistic characteristics of the input random variables.

Three types of coefficients can be used in local and global sensitivity analyses. The absolute sensitivity coefficient, $SA$, is defined as (Haan, 2002):

$$SA = \frac{\partial O}{\partial P}$$ (4.163)

Where $O$ is the model output and $P$ represents a particular input parameter. The absolute sensitivity coefficients are affected by units of output and input and therefore, cannot be used for the comparison of parametric sensitivities. The relative sensitivity, $SR$, is defined as (Haan, 2002):

$$SR = \frac{\frac{\partial O}{\partial P}}{\frac{\partial P}{\partial O}} = \frac{\partial O}{\partial P} \frac{P}{O}$$ (4.164)
The relative sensitivity coefficients are dimensionless and thus can be compared across parameters. Finally, the deviation sensitivity, $SD$, is quantified as the changes in the output $\Delta O$ (McCuen, 2003):

$$SD = \Delta O = \frac{\partial O}{\partial P} \Delta P \approx \frac{\Delta O}{\Delta P} \Delta P$$  \hspace{1cm} (4.165)$$

The deviation sensitivity has the same units as the variable $O$. The partial derivatives of Equations 4.163 to 4.165 can be approximated by numerical derivatives as (Haan, 2002):

$$\frac{\partial O}{\partial P} \approx \frac{O_{P+\Delta P} - O_{P-\Delta P}}{2\Delta P}$$  \hspace{1cm} (4.166)$$

Where $\Delta P$ is the change in parameter value from its base value (usually 20% or 25% of $P$).

In this study, a local sensitivity analysis was adopted for evaluating the GFLOOD model. The final set of parameters of the calibrated model was deemed as the baseline/nominal parameter set. Then the model was run repeatedly with the starting baseline value for each parameter and multiplied, in turn, by 0.75 and 1.25, while keeping all other parameters constant at their nominal starting values. The hydrographs resulting from the scenarios of adjusted model parameters were then compared with the baseline model hydrograph. The performance measures defined in Section 4.1.2.3 were used as sensitivity functions. Since these measures are dimensionless, the normalized value of the absolute sensitivity coefficient (equation 4.163) was used to compare the results from different sensitivity scenarios. The sensitivity procedure was applied at two locations; the Caistors Corners gauge station to evaluate the effects on flows and at the Old Syphon gauge station to evaluate the effects on water levels using the rainfall data from the May 23-27, 2004 event. Both of these locations are representative of the different hydro-climatic regimes within the watershed.
Chapter 5 – Results and Discussion

5.1 Modelling of Time of Concentration

Figure 5.1 is a plot of the time of concentration versus the main channel length. Similar plots are done for all combinations of variable pairs (Appendix D). The plot indicates a linear trend exists between the observed time of concentration (h) and the main channel length (km).

The linear correlation coefficients for each pair of variables are listed in Table 5.1. According to Table 5.1, the main channel length is highly correlated with the observed time of concentration indicating a direct relationship between the two variables. In addition, the correlation coefficient between the observed time of concentration and the elongation ratio is approximately -0.67 indicating an inverse relationship between the two variables. However, the correlation coefficient is relatively low between the observed time of concentration and the main channel slope, indicating the two variables may be independent of each other. Furthermore, the correlation coefficients between the predictor variables are relatively low indicating that the predictor variables are independent of each other. However, the correlation coefficient between the main channel length and elongation ratio was determined to be approximately -0.71, indicating an inverse relationship between the two variables. It is recommended that the use of correlated variables should be avoided in regression analysis (Wang and Huber, 1967).

Two highly correlated predictors will explain the same part of the criterion variation. The predictor with the highest criterion correlation is usually retained. High correlation between predictor variables may cause irrational regression coefficients. In addition, the potential accuracy of estimates is rarely increased significantly by including a predictor variable that is highly correlated with one or more other predictor variables in the equation. Despite this, the elongation ratio was included in the analysis, since it is not related to the main channel length, but to the length of the basin parallel to the flow path.

However, when a watershed is relatively elongated the difference between the actual basin length and the length of the main channel is relatively small. According to Table 4.7, the elongation
ratios for the Credit River watershed for the different basins range between 0.352 and 0.672, indicating that the watershed is more elliptical than circular. Therefore, by including the elongation ratio into the regression equation it is expected that the increase in accuracy of the predicted values of the time of concentration will be relatively minor.

The results of the stepwise regression analysis are given in Tables 5.2 and 5.3. Table 5.2 shows the regression coefficients for each step of the regression, and Table 5.3 shows the regression equation data for each step. The following statistical data were calculated to evaluate the regression equations: the coefficient of determination (equation 4.12); the standard deviation of the logarithms of the observed time of concentration values; the standard error of estimate (equation 4.13); the sum of squares to degrees of freedom ratio for regression and residuals; and the sum of the residuals of the logarithms for the observed and predicted time of concentration values.

The results of the regression analysis yielded the following three equations:

\[ T_c = 6.799 L_c^{0.391} \]  \hspace{1cm} (5.1)

\[ T_c = 8.252 L_c^{0.389} S_c^{-0.124} \]  \hspace{1cm} (5.2)

\[ T_c = 8.549 L_c^{0.344} S_c^{-0.170} E^{-0.264} \]  \hspace{1cm} (5.3)

According to equations 5.1, 5.2 and 5.3, the time of concentration varies directly with the main channel length and inversely with the main channel slope and elongation ratio. Therefore the relationships between the observed time of concentration and all three predictor variables are rational. The coefficient of determination \((r^2)\) values for equations 5.1, 5.2 and 5.3 were determined to be 0.92, 0.93 and 0.95. Equations 5.1, 5.2 and 5.3 explain 92%, 93% and 95% of the variation in the time of concentration respectively. According to Table 5.3, for each equation, the standard error of estimate is smaller than the standard deviation of the logarithms of the time of concentration. All equations are significant based on the total F-test at the 5-percent level. The least significant variable is the main channel length based on a 5-percent level F with 1 and 7
degrees of freedom. From a standard F table, for these degrees of freedom, F is equal to 5.5914. The partial F value required to enter the main channel length variable is 65.0164. Equation 5.1 explains 92% of the variation \( (r^2) \) in the logarithm of the time of concentration, and addition of the remaining variables only raises this to 95%. Equation 5.1 would be considered the best of the three equations, because the regression coefficients are rational, and including additional variables does not significantly decrease the standard error of estimate. The slope of the main channel and elongation ratio in equations 5.2 and 5.3 are not really needed, but are included to illustrate multiple predictor models. However, given that the intent was to derive an equation with multiple predictor variables, equation 5.2 was selected for testing and evaluation. Other watershed characteristics that were tested as predictors of \( T_c \) included drainage area, basin width, average basin slope, moisture content, percentage of wetland and water coverage and maximum flow distance. None of these parameters significantly improved the accuracy of equation 5.2.

The sum of the residuals of the logarithms of the observed and predicted time of concentration values from Table 5.3 is 0.000. A plot of the residuals with the logarithms of the predicted time of concentration values is shown in Figure 5.2. Figure 5.2 shows no correlation between the logarithms of the predicted time of concentration values and the residuals. The residual variation is also constant over the range of the logarithms of the predicted time of concentration values.

The predicted time of concentration values and the residual values for equation 5.2, along with the observed values are listed in Table 5.4. Plots of the residuals for equation 5.2 are illustrated in Figure 5.3. The greatest amount of under-prediction (negative residual) occurs near a time of concentration of 15 hours. Two data points (Credit River at Erin Branch and Black Creek below Acton) in the region account for 56% of the sum of residuals squared. The greatest amount of over prediction (positive residuals) occurs near a time of concentration of 18 hours. Large residual values positive or negative may be a problem when the regression equation is used in the upper range of time of concentration values.
Table 0-1 Linear correlation coefficients between each pair of variables, including time of concentration \((T_c)\), main channel length \((L_c)\), main channel slope \((S_c)\) and elongation ratio \((E)\)

<table>
<thead>
<tr>
<th></th>
<th>Observed Time of Concentration (T_c) (h)</th>
<th>Main Channel Length, (L_c) (km)</th>
<th>Main Channel Slope, (S_c) (m/km)</th>
<th>Elongation Ratio, (E)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Observed Time of Concentration (T_c) (h)</td>
<td>1.000</td>
<td>0.966</td>
<td>-0.167</td>
<td>-0.666</td>
</tr>
<tr>
<td>Main Channel Length, (L_c) (km)</td>
<td></td>
<td>1.000</td>
<td>-0.072</td>
<td>-0.708</td>
</tr>
<tr>
<td>Main Channel Slope, (S_c) (m/km)</td>
<td></td>
<td></td>
<td>1.000</td>
<td>-0.239</td>
</tr>
<tr>
<td>Elongation Ratio, (E)</td>
<td></td>
<td></td>
<td></td>
<td>1.000</td>
</tr>
</tbody>
</table>

Figure 0-1 Variable plot, observed time of concentration, \(T_c\) (h) versus main channel length, \(L_c\) (km), Credit River watershed

Table 0-2 Regression coefficients for the predictor variables, main channel length \((L_c)\), main channel slope \((S_c)\) and elongation ratio \((E)\), that have high correlations with the observed time of concentration \((T_c)\)

<table>
<thead>
<tr>
<th>Equation</th>
<th>Variable</th>
<th>Intercept</th>
<th>(B_1)</th>
<th>(B_2)</th>
<th>(B_3)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>(L_c)</td>
<td>0.832</td>
<td>0.391</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>(L_c, S_c)</td>
<td>0.917</td>
<td>0.389</td>
<td>-0.124</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>(L_c, S_c, E)</td>
<td>0.932</td>
<td>0.344</td>
<td>-0.170</td>
<td>-0.264</td>
</tr>
</tbody>
</table>
Table 0-3 Statistical quantities of coefficient of determination ($r^2$), standard error of estimate ($S_e$), sum of squares per degrees of freedom ($SS/df$), total F-test ($F_t$), partial F-test ($F_p$) and sum of residuals for each step of the regression process

<table>
<thead>
<tr>
<th>Equation</th>
<th>Variable</th>
<th>$r^2$</th>
<th>$\Delta r^2$</th>
<th>$S_e$</th>
<th>SS/df Regression</th>
<th>SS/df Residuals</th>
<th>$F_t$</th>
<th>$F_p$</th>
<th>Sum of Residuals</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>$L_c$</td>
<td>0.9155</td>
<td>0.0453</td>
<td>0.1599/1</td>
<td>0.0021/7</td>
<td>65.0164</td>
<td>65.0164</td>
<td>0.000</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>$L_c$, $S_c$</td>
<td>0.9318</td>
<td>0.0440</td>
<td>0.0793/2</td>
<td>0.0019/6</td>
<td>34.1436</td>
<td>1.1918</td>
<td>0.000</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>$L_c$, $S_c$, $E$</td>
<td>0.9452</td>
<td>0.0432</td>
<td>0.0537/3</td>
<td>0.0019/5</td>
<td>22.9993</td>
<td>0.9803</td>
<td>0.000</td>
<td></td>
</tr>
</tbody>
</table>

Note 1: Standard deviation ($S_y$) of the logarithms of the observed time of concentration values, $T_c$ (h)

Figure 0-2 Plot of the residuals versus the logarithms of the predicted time of concentration values, $T_c$ (h)
Figure 0-3 Plot of the residuals versus the predicted time of concentration values, \( T_c \) (h)

Table 0-4 Observed time of concentration values, predicted time of concentration values and residuals for the 9 WSC gauge stations along the Credit River watershed

<table>
<thead>
<tr>
<th>Station Name</th>
<th>Observed Time of Concentration, ( T_c ) (h)</th>
<th>Predicted Time of Concentration, ( T_c ) (h)</th>
<th>Residuals</th>
</tr>
</thead>
<tbody>
<tr>
<td>Credit River at Orangeville</td>
<td>20.15</td>
<td>19.34</td>
<td>-0.82</td>
</tr>
<tr>
<td>Credit River at Cataract</td>
<td>24.98</td>
<td>22.96</td>
<td>-2.02</td>
</tr>
<tr>
<td>Credit River at Boston Mills</td>
<td>29.35</td>
<td>28.57</td>
<td>-0.78</td>
</tr>
<tr>
<td>Credit River at Norval</td>
<td>33.22</td>
<td>34.26</td>
<td>1.04</td>
</tr>
<tr>
<td>Credit River at Erindale</td>
<td>40.45</td>
<td>39.35</td>
<td>-1.10</td>
</tr>
<tr>
<td>Credit River at Erin Branch</td>
<td>14.12</td>
<td>16.97</td>
<td>2.85</td>
</tr>
<tr>
<td>Credit River above Alton Branch</td>
<td>17.58</td>
<td>18.43</td>
<td>0.84</td>
</tr>
<tr>
<td>Black Creek below Acton</td>
<td>18.11</td>
<td>15.52</td>
<td>-2.58</td>
</tr>
<tr>
<td>Credit River West Branch at Norval</td>
<td>23.46</td>
<td>25.24</td>
<td>1.77</td>
</tr>
</tbody>
</table>

The time of concentration \( (T_c) \) values for the stream gauges along the Credit River were estimated using equations by Williams (1922), Kirpich (1940), Johnstone – Cross (1949), Haktanir – Sezen (1990) and the regression equation, equation 5.2. The results are illustrated in Figure 5.4. The equations developed by Williams, Kirpich, Johnstone – Cross and Haktanir – Sezen were derived from basins where channel flow dominates. According to the Credit River Water Management Strategy (Credit Valley Conservation, 1990 and 2007) the total length of streams and creeks within the Credit River watershed is over 1500 km, baseflow index is approximately 0.48 (ratio baseflow to total flow) and channel flow being the dominant flow
process. Therefore, the equations developed by Williams, Kirpich, Johnstone – Cross and Haktanir – Sezen for estimating $T_c$ would be applicable to a watershed such as the Credit River. Whereas, equations such as those developed by Izzard (1946), Su and Fang (2004) and Wong (2002) are used for estimating $T_c$ where overland flow dominates. However, in order to ensure a fair comparison between the empirical equations and the regression equation, the overland flow $T_c$ values were estimated for each of the nine stream gauges along the Credit River. The overland flow $T_c$ values were then added to each of the $T_c$ values estimated using equations by Williams, Kirpich, Johnstone – Cross and Haktanir – Sezen. The overland flow $T_c$ values for the Credit River were estimated using the kinematic Darcy–Weisbach time of concentration formula developed by Wong and Chen (1997), assuming no upstream inflow, $T_{ov} = \frac{L_{slp}^0.6}{S_w^{0.6}n}$. Where $L_{slp}$ (m) is the basin slope length, $n$ is Manning’s roughness coefficient and $S_w$ (m/m) is the watershed slope. The above equation incorporate a 2-year, 24-hour rainfall intensity of 2.2 mm/h based on published intensity duration IDF curves for the Credit River watershed (Meteorological Services Canada, 2012). The physical parameters of the basins used to estimate the overland flow $T_c$ values are listed in Table 5.6.

Figure 5.4, graphically indicates, that there is good correlation between the observed and predicted values for the time of concentrations ($T_c$) using the regression equation, equation 5.2. According to Figure 5.4, $T_c$ estimated by Kirpich, Johnstone – Cross, Williams and Haktanir – Sezen equations give a lower bound of $T_c$ estimates, particularly when the drainage area is less than about 40 km$^2$. When the area is greater than 50 km$^2$ and less than about 400 km$^2$, $T_c$ estimated by William’s equation and Haktanir – Sezen equation also give a lower bound of $T_c$ estimates respectively. The time of concentration ($T_c$) estimated by the Kirpich equation was similar to $T_c$ as estimated by Johnstone – Cross equation (Figure 5.4). The Johnstone – Cross (1949) equation combines channel length and channel slope into a single parameter ($L/S$). Figure 5.4 shows the Johnstone – Cross equation to underestimate $T_c$ values for all basins. The Haktanir – Sezen equation (1990) requires only the main channel length as an input watershed parameter for estimating $T_c$, the watershed lag. The watershed time of concentration is then estimated by dividing the watershed lag by 0.6 (NRCS, 1972, 1986). $T_c$ estimated using the Haktanir – Sezen equation is very similar to $T_c$ estimated using William’s equation as shown graphically in Figure 5.4.
Table 5.5 provides nine (9) statistical quantities for evaluating the performance of the empirical equations and the regression equation in estimating the time of concentration ($T_c$). These statistical quantities were calculated using equations 4.1 thru 4.8. According to Table 5.5, the regression equation outperformed the empirical equations for estimating the time of concentration. This is expected since the regression equation was derived using the hydro-meteorological and physiographic data of the Credit River watershed. According to Table 5.5, for the regression equation, the coefficient of determination ($r^2$) was determined to be 0.95, the coefficient of efficiency ($E$) 0.95 and the modified coefficient of efficiency ($E_I$) 0.77. The values for $r^2$ and $E$ suggest that the observed and predicted values of $T_c$ for the regression equation are highly correlated and given that $r^2 = E$, the results are not bias. According to Table 5.5 the relative bias ($RBIAS$) for the regression equation is 0.4%. For Williams, Kirpich, Johnstone – Cross and Haktanir – Sezen equations, the values for $E$ are less than $r^2$, suggesting bias within the results. The empirical relationships developed by Williams, Kirpich, Johnstone-Cross and Haktanir–Sezen have a relative bias ($RBIAS$) for underestimating the time of concentration (Table 5.5). The relative bias values for Williams and Haktanir – Sezen equations, are -23% and -21% respectively. The relative bias values for Kirpich and Johnstone -- Cross equations, are -43% and -53% respectively. An acceptably low value for the root mean square error ($RMSE$) would be approximately half the standard deviation of the measured or observed values (0.5 · $\sigma_{measured} = 4.2$). According to Table 5.5, only the regression equation meets this criterion. The $RMSE$ values for the empirical equations including Williams, Kirpich, Johnstone – Cross and Haktanir – Sezen were found to be greater than half the standard deviation of the observed values. The same applies for the other statistical criteria including the systematic root mean error ($RMSE_s$), unsystematic root mean square error ($RMSE_u$) and the coefficient of residual mass ($CRM$). According to Table 5.5, only the regression equation was able to produce acceptably low values of these three statistical criteria.

In order to validate the regression equation, the equation was applied to ten (10) additional basins within Southern Ontario. The data for these 10 additional basins were abstracted from Kennedy and Watt (1967). The names, station identification numbers, and physical parameters for these basins are listed in Table 5.7. Kennedy and Watt (1967) estimated the watershed lag of the basins and not the watershed time of concentration. The predicted watershed lag was estimated by
multiplying the predicted watershed time of concentration values by 0.6 (NRCS, 1972, 1986). The times of concentration \( T_c \) values were estimated using the regression equation for the additional data set, Kennedy and Watt (1967). In addition, time of concentration values were also estimated using the four (4) empirical equations. In order to ensure a fair comparison between the empirical equations and the regression equation, the overland flow \( T_c \) values were estimated for each of the 10 basins using the kinematic Darcy–Weisbach time of concentration formula, Wong and Chen (1997), assuming no upstream inflow. The physical parameters for estimating the overland flow \( T_c \) values are listed in Table 5.7. The predicted values for the regression equation and empirical equations were compared against the observed values and the results are shown in Figure 5.5.

According to Figure 5.5, there is good correlation between the observed and predicted values for the time of concentration \( T_c \) values for the regression equation. Williams, Kirpich and Haktanir–Sezen equations, give a lower bound of the watershed lag time for drainage areas greater than 100 km² (Figure 5.5). The equation by Johnstone–Cross also gives a lower bound of the watershed lag time. According to Figure 5.5, the equation by Johnstone–Cross underestimates the watershed time of concentration and/or lag time for all basins.

Table 5.8 provides 9 statistical quantities for validating the performance of the regression equation and the empirical equations. These statistical quantities were calculated using equations 4.1 thru 4.8. According to Table 5.8, the regression equation outperformed the empirical equations for estimating the watershed time of concentration values and subsequently the watershed lag times. According to Table 5.8, for the regression equation, the coefficient of determination \( r^2 \) was determined to be 0.81, the coefficient of efficiency \( E \) 0.79 and the modified coefficient of efficiency \( E_I \) 0.50. The values for \( r^2 \) and \( E \) suggest that the observed and predicted values of \( T_c \) for the regression equation are correlated. However, \( r^2 > E \), and therefore, the results are considered to be bias. According to Table 5.8 the relative bias (\( RBIAS \)) for the regression equation is -2.1%. For Williams equation \( E < r^2 \), however, the relative bias is equivalent to the regression equation (-2.1%). However, according to Figure 5.5, Williams’ equation underestimates the watershed lag time for basins with drainage areas greater than 100 km². According to Table 5.8, for Williams’ equation, the coefficient of determination \( r^2 \) was
determined to be 0.67, the coefficient of efficiency ($E$) 0.63 and the modified coefficient of efficiency ($E_1$) 0.37. The values for $r^2$ and $E$ suggest that the correlation of the observed and predicted values of $T_c$ for Williams’ equation is fair to poor. Similarly the same is true for Haktanir – Sezen’s equation, the relative bias is low (-3.4%), however, the correlation between the predicted and observed values is relatively poor (Table 5.8). For Kirpich and Johnstone – Cross equations, the values for $E$ are significantly less than $r^2$, suggesting bias within the results. The empirical relationships developed by Kirpich and Johnstone-Cross have a relative bias ($RBIAS$) for underestimating the watershed time of concentration and/or the watershed lag time (Table 5.8).

The relative bias for Kirpich’s equation is -22%. The relative bias value for Johnstone – Cross equation is -32%. The $RMSE$ values for the empirical equations including Williams, Kirpich, Johnstone – Cross and Haktanir – Sezen were found to be greater than half the standard deviation of the observed values ($0.5 \cdot \sigma_{measured} = 2.9$). According to Table 5.8, only the regression equation met this criterion. The same applies for the other statistical criteria including the systematic root mean error ($RMSE_s$), unsystematic root mean square error ($RMSE_u$) and the coefficient of residual mass ($CRM$). According to Table 5.8, only the regression equation was able to produce acceptably low values of these three statistical criteria.

Overall, $T_c$ estimated with the regression equation provides a relatively good estimate with an average relative bias of less than 2%. In comparison to the four empirical equations tested, the regression equation provides a more reliable estimate of the time of concentration for watersheds within Southern Ontario. In addition, an examination of equation 5.2, reveals that it is similar in form to the four empirical equations tested, with the main channel length ($L_c$) and the main channel slope ($S_c$) being the common parameters. Williams’ equation also incorporates the drainage area ($A$) parameter. Furthermore, equation 5.2 was developed for basins where channel flow dominates, similar to the four empirical equations and is therefore, not only similar in form but also in function to Williams, Kirpich, Johnstone – Cross and Haktanir – Sezen equations. However, unlike the four empirical equations, equation 5.2 was developed from basins within the Credit River watershed, and is therefore, more applicable to watersheds which have similar hydro-meteorological conditions to the Credit River, such as those within Southern Ontario.
Equation 5.2 can be further calibrated and validated for a larger data set. In total, there are over 800 active meteorological stations across Ontario and 504 active stream gauges. This is not to suggest that one equation can be developed for the entire province. However, the stream gauges and climate stations can be grouped into separate regions across the province. The above methodology can be applied to each region and an equation for estimating the time of concentration can be developed for that specific region.

Figure 0-4 Observed time of concentration values ($T_c$) versus predicted time of concentration values determined with Williams (1922), Kirpich (1940), Johnstone – Cross (1949), Haktanir – Sezen (1990) and the regression equations

Table 0-5 Statistical quantities of coefficient of determination ($r^2$), coefficient of efficiency (E), modified coefficient of efficiency ($E_1$), root mean square error (RMSE), systematic root mean square error (RMSE$_S$), unsystematic root mean square error (RMSE$_U$), coefficient of residual mass (CRM) and relative bias (RBIAS) for $T_c$ estimates using different empirical equations and the regression equation

<table>
<thead>
<tr>
<th>Equations</th>
<th>$r^2$</th>
<th>E</th>
<th>$E_1$</th>
<th>RMSE</th>
<th>RMSE$_S$</th>
<th>RMSE$_U$</th>
<th>CRM</th>
<th>RBIAS</th>
</tr>
</thead>
<tbody>
<tr>
<td>Regression Equation</td>
<td>0.95</td>
<td>0.95</td>
<td>0.77</td>
<td>1.7</td>
<td>0.4</td>
<td>1.7</td>
<td>-0.004</td>
<td>0.4</td>
</tr>
<tr>
<td>Williams 1922</td>
<td>0.95</td>
<td>0.51</td>
<td>0.24</td>
<td>5.6</td>
<td>1.9</td>
<td>5.3</td>
<td>-0.204</td>
<td>-23.4</td>
</tr>
<tr>
<td>Kirpich 1940</td>
<td>0.95</td>
<td>-0.94</td>
<td>-0.61</td>
<td>11.1</td>
<td>11.0</td>
<td>1.1</td>
<td>-0.429</td>
<td>-43.1</td>
</tr>
<tr>
<td>Johnstone-Cross 1949</td>
<td>0.83</td>
<td>-2.47</td>
<td>-1.06</td>
<td>14.8</td>
<td>14.8</td>
<td>0.9</td>
<td>-0.552</td>
<td>-53.3</td>
</tr>
<tr>
<td>Haktanir-Sezen 1990</td>
<td>0.95</td>
<td>0.58</td>
<td>0.31</td>
<td>5.2</td>
<td>4.7</td>
<td>2.2</td>
<td>-0.185</td>
<td>-21.5</td>
</tr>
</tbody>
</table>
Table 0-6 Locations, physical basin parameters and overland flow time of concentration values for the Water Survey of Canada gauge stations along the Credit River watershed

<table>
<thead>
<tr>
<th>Station Name</th>
<th>Station ID</th>
<th>LAT (°)</th>
<th>LONG (°)</th>
<th>Manning’s Overland Flow “n” Value</th>
<th>Basin Slope Length (km)</th>
<th>Basin Slope (m/m)</th>
<th>Rainfall Intensity 2-year 24-h (mm/h)</th>
<th>Overland Flow Travel Time (h)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Credit River at Orangeville</td>
<td>02HB013</td>
<td>43.89°</td>
<td>80.06°</td>
<td>0.325</td>
<td>1.82</td>
<td>0.0300</td>
<td>2.2</td>
<td>6.94</td>
</tr>
<tr>
<td>Credit River at Cataract</td>
<td>02HB001</td>
<td>43.84°</td>
<td>80.02°</td>
<td>0.325</td>
<td>1.82</td>
<td>0.0300</td>
<td>2.2</td>
<td>6.94</td>
</tr>
<tr>
<td>Credit River at Boston Mills</td>
<td>02HB018</td>
<td>43.77°</td>
<td>79.93°</td>
<td>0.325</td>
<td>1.82</td>
<td>0.0300</td>
<td>2.2</td>
<td>6.94</td>
</tr>
<tr>
<td>Credit River at Norval</td>
<td>02HB025</td>
<td>43.65°</td>
<td>79.86°</td>
<td>0.325</td>
<td>1.82</td>
<td>0.0300</td>
<td>2.2</td>
<td>6.94</td>
</tr>
<tr>
<td>Credit River at Erindale</td>
<td>02HB002</td>
<td>43.54°</td>
<td>79.66°</td>
<td>0.325</td>
<td>1.82</td>
<td>0.0300</td>
<td>2.2</td>
<td>6.94</td>
</tr>
<tr>
<td>Credit River at Erin Branch</td>
<td>02HB020</td>
<td>43.77°</td>
<td>80.09°</td>
<td>0.325</td>
<td>1.79</td>
<td>0.0613</td>
<td>2.2</td>
<td>5.54</td>
</tr>
<tr>
<td>Credit River above Alton Branch</td>
<td>02HB019</td>
<td>43.85°</td>
<td>80.08°</td>
<td>0.325</td>
<td>1.99</td>
<td>0.0316</td>
<td>2.2</td>
<td>7.20</td>
</tr>
<tr>
<td>Black Creek below Acton</td>
<td>02HB024</td>
<td>43.63°</td>
<td>80.01°</td>
<td>0.325</td>
<td>1.01</td>
<td>0.0218</td>
<td>2.2</td>
<td>5.37</td>
</tr>
<tr>
<td>Credit River West Branch at Norval</td>
<td>02HB008</td>
<td>43.65°</td>
<td>79.87°</td>
<td>0.325</td>
<td>2.20</td>
<td>0.0256</td>
<td>2.2</td>
<td>8.15</td>
</tr>
</tbody>
</table>

Note 1: The first five (5) gauges are located along the main branch of the Credit River and therefore the overland flow travel time is the same for these gauges.

Table 0-7 Station ID, physical basin parameters and overland flow time of concentration values for Water Survey of Canada gauge stations across Southern Ontario (Kennedy and Watt, 1967)

<table>
<thead>
<tr>
<th>Basin Name</th>
<th>Station ID</th>
<th>Manning’s Overland Flow “n” Value</th>
<th>Basin Slope Length (km)</th>
<th>Basin Slope (m/m)</th>
<th>Rainfall Intensity 2-year 24-h (mm/h)</th>
<th>Overland Flow Time of Concentration (h)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Canagagigue Creek</td>
<td>02GA023</td>
<td>0.325</td>
<td>2.13</td>
<td>0.00634</td>
<td>2.2</td>
<td>12.14</td>
</tr>
<tr>
<td>Cold Creek</td>
<td>02HC023</td>
<td>0.325</td>
<td>2.99</td>
<td>0.01200</td>
<td>2.2</td>
<td>12.31</td>
</tr>
<tr>
<td>Conestoga Creek</td>
<td>02GA017</td>
<td>0.325</td>
<td>0.89</td>
<td>0.00328</td>
<td>2.2</td>
<td>8.78</td>
</tr>
<tr>
<td>Don West River</td>
<td>02HC005</td>
<td>0.325</td>
<td>1.75</td>
<td>0.00724</td>
<td>2.2</td>
<td>10.39</td>
</tr>
<tr>
<td>Dufferin Creek</td>
<td>02HC006</td>
<td>0.325</td>
<td>5.13</td>
<td>0.01130</td>
<td>2.2</td>
<td>17.31</td>
</tr>
<tr>
<td>Etobicoke Creek</td>
<td>02HC002</td>
<td>0.325</td>
<td>1.97</td>
<td>0.00564</td>
<td>2.2</td>
<td>12.00</td>
</tr>
<tr>
<td>Oakville Creek</td>
<td>02HB005</td>
<td>0.325</td>
<td>1.97</td>
<td>0.01004</td>
<td>2.2</td>
<td>10.10</td>
</tr>
<tr>
<td>Parkhill Creek</td>
<td>02FF003</td>
<td>0.325</td>
<td>1.44</td>
<td>0.00191</td>
<td>2.2</td>
<td>13.78</td>
</tr>
<tr>
<td>Trout Creek</td>
<td>02GD009</td>
<td>0.325</td>
<td>2.41</td>
<td>0.00168</td>
<td>2.2</td>
<td>19.52</td>
</tr>
<tr>
<td>West Humber River</td>
<td>02HC008</td>
<td>0.325</td>
<td>2.79</td>
<td>0.00435</td>
<td>2.2</td>
<td>16.00</td>
</tr>
</tbody>
</table>
Table 0-8 Statistical quantities of coefficient of determination ($r^2$), coefficient of efficiency ($E$), modified coefficient of efficiency ($E_1$), root mean square error (RMSE), systematic root mean square error (RMSE_s), unsystematic root mean square error (RMSE_U), coefficient of residual mass (CRM) and relative bias (RBIAS) for comparing observed and predicted values of $T_c$ estimated using the regression equation and empirical equations for the 10 additional basins across Southern Ontario.

<table>
<thead>
<tr>
<th>Equations</th>
<th>$r^2$</th>
<th>$E$</th>
<th>$E_1$</th>
<th>RMSE</th>
<th>RMSE_s</th>
<th>RMSE_U</th>
<th>CRM</th>
<th>RBIAS</th>
</tr>
</thead>
<tbody>
<tr>
<td>Regression Equation</td>
<td>0.81</td>
<td>0.79</td>
<td>0.50</td>
<td>2.6</td>
<td>1.6</td>
<td>2.0</td>
<td>-0.035</td>
<td>-2.1</td>
</tr>
<tr>
<td>Williams 1922</td>
<td>0.67</td>
<td>0.63</td>
<td>0.37</td>
<td>3.4</td>
<td>2.6</td>
<td>2.2</td>
<td>-0.049</td>
<td>-2.1</td>
</tr>
<tr>
<td>Kirpich 1940</td>
<td>0.46</td>
<td>-0.14</td>
<td>-0.05</td>
<td>6.0</td>
<td>5.5</td>
<td>2.2</td>
<td>-0.252</td>
<td>-22.2</td>
</tr>
<tr>
<td>Johstone-Cross 1949</td>
<td>0.26</td>
<td>-0.91</td>
<td>-0.45</td>
<td>7.7</td>
<td>7.4</td>
<td>2.2</td>
<td>-0.357</td>
<td>-32.3</td>
</tr>
<tr>
<td>Haktanir-Sezen 1990</td>
<td>0.39</td>
<td>0.32</td>
<td>0.22</td>
<td>4.6</td>
<td>4.0</td>
<td>2.3</td>
<td>-0.081</td>
<td>-3.4</td>
</tr>
</tbody>
</table>

Figure 0-5 Observed time of concentration values ($T_c$) versus predicted time of concentration values determined with the Williams (1922), Kirpich (1940), Johnstone – Cross (1949), Haktanir – Sezen (1990) and the regression equations for ten (10) additional basins across Southern Ontario.

5.2 Modelling of Recession Constants

5.2.1 Surface Storage Recession Constant ($K_{surf}$)

An expression relating recession constant to basin physical characteristics should be relatively simple. This requirement, together with the small size of the sample available, limited the number of independent variables to three. In addition it is essential that this expression be compatible with the known principles of hydraulics.
The average surface storage recession constant values were used to yield the surface storage recession constant \( (K_{surf}) \) prediction equation. The \( K_{surf} \) values were related to the same physical characteristics used in equations 4.10, 4.11a and 4.11b. The values of the exponents were found by a forward stepwise multiple linear regression analysis of \( \log K_{surf} \) on the logarithms of the independent variables. The same procedure that was used to undertake the regression analysis of the time of concentration was also used to undertake the regression analysis of the surface storage recession constant.

A data point (Credit River at Erin Branch) was excluded from the regression analysis. The Erin Branch basin has one of the longest surface storage recession values, 22 hours (Table 4.8) and one of the steepest main channel slopes 6.34 m/km in comparison to the other basins. The dominate landuse in this basin is similar to other basins, rural and/or agricultural (cropland and pasture land) and the percentage of wetland and water coverage is relatively low less than 1%. However, a review of soil maps for the Credit River at Erin Branch gauge station revealed that the underlying soils within the basin are predominantly loam (64%) and clay (36%). Both types of soils have high water retention values and given that the recession constant is a function of storage, the predominance of water retaining soils within the basin can have an impact on the recession constant. A review of municipal site plans at a scale of 1:500 also revealed that a number of stormwater management ponds exist upstream of the stream gauge at Erin Branch. These stormwater management ponds were not previously identified on the Ontario Flow Assessment Techniques (OFAT) geo-spatial maps; because their scale is much coarser (1:10,000) than the municipal site plans and therefore were not included in the OFAT geo-spatial maps. The presence of these ponds upstream of the Credit River at Erin Branch gauge station, detain the flow for an extended period of time and then gradually release it, thereby increasing the time of concentration to the gauge. Furthermore, a review of measured flow, velocity and depth data at the Erin Branch station provided by Water Survey of Canada, revealed significant scatter in the data and no relationships could be deduced from the available data (Figure 5.6). Based on these findings it was concluded that the Credit River at Erin Branch gauge station should be excluded from the analysis when developing a relationship between the recession constants and the physical characteristics of the basins.
Each variable was plotted against each other to determine if a linear or non-linear data trend exists. Figure 5.7 is a plot of the surface storage recession constant versus the main channel length, \( L_c \) (km). According to Figure 5.7 there is a direct curvilinear relationship between the surface storage recession constant and the length of the main channel. Plots of the other combinations of variable pairs are shown in Appendix D. The linear correlation coefficients for each pair of variables are listed in Table 5.5. The results of the stepwise regression analysis are in Tables 5.6 and 5.7. Table 5.6 shows the regression coefficients for each step of the regression, and Table 5.7 shows the regression equation data for each step. The results of the regression analysis yielded the following equations:

\[
K_{surf} = 0.916L_c^{0.888} \quad (5.4)
\]

\[
K_{surf} = 0.021L_c^{0.924}S_c^{-0.678} \quad (5.5)
\]

\[
K_{surf} = 0.2801L_c^{1.134}S_c^{-0.275}E^{1.586} \quad (5.6)
\]

Where the variables \( L_c \) is the main channel length (km), \( S_c \) is the main channel slope (m/m) and \( E \) is the elongation ratio (dimensionless) for the basin. According to equations 5.4, 5.5 and 5.6, the surface storage recession constant varies directly with the main channel length and inversely with the main channel slope. Therefore the relationships between the observed surface storage recession constant and the main channel length and main channel slope are rational. In equation 5.6, the surface storage recession constant varies directly with the elongation ratio. However, according to Table 5.5 and Figure 5.8, the surface storage recession constant varies inversely with the elongation ratio. Therefore, the direct relationship between the surface storage recession constant and the elongation ratio in equation 5.6 is not rational and should only be included if the increased significance is meaningful.

The coefficient of determination \( (r^2) \) values for equations 5.4, 5.5 and 5.6 were determined to be 0.78, 0.86 and 0.93. Equations 5.4, 5.5 and 5.6 explain 78\%, 86\% and 93\% of the variation in the surface storage recession constant respectively. According to Table 5.7, for each equation, the standard error of estimate is smaller than the standard deviation of the logarithms of the surface
storage recession constant. All equations are significant based on the total F-test at the 5-percent level. The least significant variable is the main channel length based on a 5 percent level F with 1 and 6 degrees of freedom. From a standard F table, for these degrees of freedom, F is equal to 5.9874. The partial F value required to enter the main channel length variable is 20.9098. Equation 5.4 explains 78% of the variation (r^2) in the logarithm of the surface storage recession constant, and the addition of the remaining variables raises this to 93%. Equation 5.6 was selected as the best because equation 5.6 is a good predictor of the surface storage recession constant and the additional variables decreased the standard error of estimates from 0.777 to 0.0696. Other watershed characteristics that were tested as predictors of K_{surf} included drainage area, basin width, average basin slope, moisture content, percentage of wetland and water coverage and maximum flow distance. None of these parameters significantly improved the accuracy of equation 5.6.

The sum of the residuals of the logarithms of the observed and predicted surface storage recession constant values from Table 5.7 is 0.000. A plot of the residuals with the logarithms of the predicted surface storage recession constant values is shown in Figure 5.9. Figure 5.9 shows no correlation between the logarithms of the predicted surface storage recession constant values and the residuals. The residual variation is also constant over the range of the logarithms of the predicted surface storage recession constant values.

The predicted surface storage recession constant values and the residual values for equation 5.6, along with the observed values are listed in Table 5.8. Plots of the residuals for equation 5.6 are illustrated in Figure 5.10. The greatest amount of under prediction (negative residual) occurs near a surface storage recession constant value of 13 hours. Two data points (Credit River West Branch at Norval and Credit River at Norval) in the region account for 63% of the sum of residuals squared. The greatest amount of over prediction (positive residuals) occurs near a surface storage recession constant of 37 hours. Large residual values positive or negative may be a problem when the regression equation is used in the upper range of surface storage recession constant values.
Figure 0-6 Plot average flow versus average velocity for the Credit River at Erin Branch stream gauge station

Figure 0-7 Plot surface storage recession constant, $K_{surf}$ (h) versus main channel length, $L_c$ (km)
Table 0-9 Linear correlation coefficients between each pair of variables, including surface storage recession constant ($K_{surf}$), main channel length ($L_c$), main channel slope ($S_c$) and elongation ratio ($E$)

<table>
<thead>
<tr>
<th>Variable</th>
<th>Surface Storage Recession Constant, $K_{surf}$ (h)</th>
<th>Main Channel Length, $L_c$ (km)</th>
<th>Main Channel Slope, $S_c$ (m/m)</th>
<th>Elongation Ratio, $E$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Surface Storage Recession Constant $K_{surf}$ (h)</td>
<td>1.000</td>
<td>0.918</td>
<td>-0.119</td>
<td>-0.368</td>
</tr>
<tr>
<td>Main Channel Length, $L_c$ (km)</td>
<td></td>
<td>1.000</td>
<td>0.035</td>
<td>-0.677</td>
</tr>
<tr>
<td>Main Channel Slope, $S_c$ (m/m)</td>
<td></td>
<td></td>
<td>1.000</td>
<td>-0.504</td>
</tr>
<tr>
<td>Elongation Ratio, $E$</td>
<td></td>
<td></td>
<td></td>
<td>1.000</td>
</tr>
</tbody>
</table>

Table 0-10 Regression coefficients for the predictor variables, main channel length ($L_c$), main channel slope ($S_c$) and elongation ratio ($E$), that have high correlations with the observed surface storage recession constant ($K_{surf}$)

<table>
<thead>
<tr>
<th>Equation</th>
<th>Variable</th>
<th>Intercept</th>
<th>$B_1$</th>
<th>$B_2$</th>
<th>$B_3$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>$L_c$</td>
<td>-0.038</td>
<td>0.888</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>$L_c$, $S_c$</td>
<td>-1.683</td>
<td>0.924</td>
<td>-0.678</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>$L_c$, $S_c$, $E$</td>
<td>-0.553</td>
<td>1.134</td>
<td>-0.275</td>
<td>1.586</td>
</tr>
</tbody>
</table>

Table 0-11 Statistical quantities of coefficient of determination ($r^2$), standard error of estimate ($S_e$), sum of squares per degrees of freedom (SS/df), total F-test ($F_t$), partial F-test ($F_p$) and sum of residuals for each step of the regression process

<table>
<thead>
<tr>
<th>Equation</th>
<th>Variable</th>
<th>$r^2$</th>
<th>$\Delta r^2$</th>
<th>$S_e$</th>
<th>SS/df Regression</th>
<th>SS/df Residuals</th>
<th>$F_t$</th>
<th>$F_p$</th>
<th>Sum of Residuals</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>$L_c$</td>
<td>0.777</td>
<td>0.7770</td>
<td>0.1821</td>
<td>0.6933/1</td>
<td>0.0332/6</td>
<td>20.9098</td>
<td>20.9098</td>
<td>0.000</td>
</tr>
<tr>
<td>2</td>
<td>$L_c$, $S_c$</td>
<td>0.857</td>
<td>0.0798</td>
<td>0.1598</td>
<td>0.3822/2</td>
<td>0.0255/5</td>
<td>14.9667</td>
<td>2.7890</td>
<td>0.000</td>
</tr>
<tr>
<td>3</td>
<td>$L_c$, $S_c$, $E$</td>
<td>0.927</td>
<td>0.0696</td>
<td>0.1280</td>
<td>0.3822/3</td>
<td>0.0255/4</td>
<td>16.8111</td>
<td>3.7910</td>
<td>0.000</td>
</tr>
</tbody>
</table>

Note 1: Standard deviation ($S_e$) of the logarithms of the observed surface storage recession constant values, $K_{surf}$ (h)
Figure 0-8 Plot surface storage recession constant, $K_{surf}(h)$ versus elongation ratio, $E$

Figure 0-9 Plot of the residuals versus the logarithms of the predicted surface storage recession constant values, $K_{surf}(h)$
Table 0-12 Observed surface storage recession constant values, predicted surface storage recession constant values and residuals for the 8 WSC gauge stations along the Credit River watershed

<table>
<thead>
<tr>
<th>Station Name</th>
<th>Observed Surface Storage Recession Constant, $K_{surf}$ (h)</th>
<th>Predicted Surface Storage Recession Constant, $K_{surf}$ (h)</th>
<th>Residuals</th>
</tr>
</thead>
<tbody>
<tr>
<td>Credit River at Orangeville</td>
<td>13.31</td>
<td>10.77</td>
<td>-2.54</td>
</tr>
<tr>
<td>Credit River at Cataract</td>
<td>22.73</td>
<td>25.92</td>
<td>3.20</td>
</tr>
<tr>
<td>Credit River at Boston Mills</td>
<td>28.13</td>
<td>29.05</td>
<td>0.92</td>
</tr>
<tr>
<td>Credit River at Norval</td>
<td>31.46</td>
<td>36.76</td>
<td>5.30</td>
</tr>
<tr>
<td>Credit River at Erindale</td>
<td>38.46</td>
<td>39.45</td>
<td>0.99</td>
</tr>
<tr>
<td>Credit River above Alton Branch</td>
<td>12.20</td>
<td>9.728</td>
<td>-2.47</td>
</tr>
<tr>
<td>Black Creek below Acton</td>
<td>2.858</td>
<td>4.120</td>
<td>1.26</td>
</tr>
<tr>
<td>Credit River West Branch at Norval</td>
<td>16.58</td>
<td>12.61</td>
<td>-3.96</td>
</tr>
</tbody>
</table>

Figure 0-10 Plot of the residuals versus the predicted surface storage recession constant values, $K_{surf}$ (h)

The surface storage recession constant ($K_{surf}$) estimated using Williams’ equation (1968), Williams and Haan equation (1973) and the regression equation (equation 5.6) are shown in Figure 5.11. Figure 5.11, graphically indicates, that there is good correlation between the observed and predicted values for the surface storage recession constant ($K_{surf}$) using the regression equation. According to Figure 5.11, $K_{surf}$ estimated by the Williams equation (1968) and the Williams and Hann equation (1973) give a lower bound of $K_{surf}$ estimates.
Table 5.13 provides nine (9) statistical quantities for evaluating the performance of the regression equation (equation 5.6) for estimating the surface storage recession constant. In addition, statistical quantities for William’s equation (1968) and Williams and Haan equation (1973) are also included in Table 5.13. According to Table 5.13, as expected, the regression equation (equation 5.6) outperformed Williams’ equation and the Williams and Haan equation. According to Table 5.13, the observed and predicted $K_{surf}$ values for equation 5.6 are highly correlated. However, the coefficient of efficiency ($E$) is less than the coefficient of determination ($r^2$), suggesting bias within the results, however, the difference between the two statistical quantities is 0.03 and the relative bias ($RBIAS$) was determined to be 2.2%, therefore, the differences are insignificant. The root mean square error ($RMSE$) is less than half the standard deviation of the measured values ($0.5 \cdot \sigma_{measured} = 5.8$); indicating a relatively low value for the $RMSE$. The regression equation (equation 5.6) was the only equation that met this criterion. The same applies for the other statistical criteria including the systematic root mean error ($RMSE_s$), unsystematic root mean square error ($RMSE_u$) and the coefficient of residual mass ($CRM$). Both the Williams equation and the Williams and Haan equation showed significant bias in their predicted values in comparison to the observed values, -81% and -48% respectively (Table 5.13). As a result, the correlation between the observed values and the predicted values for each of these equations is very poor.

A comparison of surface storage recession constant values predicted by Williams’ equation (equation 4.10) and the Williams and Haan equation (equations 4.11a and 4.11b) and the regression equation (equation 5.6) suggests a preference for the equation 5.6. This relationship contains the same number of independent variables as with equations 4.10 and 4.11b and fewer than equation 4.11a and yields a better goodness-of-fit with the observed data. The basin length, slope and elongation ratio of equation 5.6 can be determined easily through the use of GIS data; and results in a better fit then equations 4.10, 4.11a and 4.11b. With regards to equations 5.6, 4.11a, 4.11b and 4.10 it is apparent from Figure 5.11 that the discrepancy between predicted and observed values is greatest for equations 4.10, 4.11a and 4.11b. A review of the elongation ratios that were used to develop equation 4.10 (Williams’ equation, 1968), indicated that the elongation ratios for the basins ranged between 0.60 and 0.92, with an average basin elongation ratio of 0.75 and standard deviation of 0.1 (Williams, 1968). Therefore, the basins used to develop equation...
4.10 and subsequently equations 4.11a and 4.11b were circular in shape, with high relief, steep ground slopes and runoff from various parts of the basin reaching the outlet at the same time. Whereas in the Credit River, the basins are more elongated (0.35-0.67, Table 4.1); therefore, causing the runoff to be spread out over time, thus producing a smaller flood peak and a longer surface storage recession constant. As a result, equations 4.10, 4.11a and 4.11b would not be applicable to basins that are elongated, such as, the Credit River, since they were developed for watersheds that were circulatory in shape. Similarly, equation 5.6 would not be applicable to watersheds that are circulatory in shape, since it was developed using basins that are elongated.

![Graph showing surface storage recession constant comparison](image)

**Figure 0-11** Observed surface storage recession constant \( (K_{\text{surf}}) \) versus predicted surface storage recession constant values determined with Williams (1968), Williams and Haan (1973) and the regression equation (equation 5.6)

**Table 0-13** Statistical quantities of coefficient of determination \( (r^2) \), coefficient of efficiency \( (E) \), modified coefficient of efficiency \( (E_l) \), root mean square error \( (\text{RMSE}) \), systematic root mean square error \( (\text{RMSE}_S) \), unsystematic root mean square error \( (\text{RMSE}_U) \), coefficient of residual mass \( (\text{CRM}) \), and relative bias \( (\text{RBIAS}) \) for estimating the surface storage recession constant using equation 3.20, Williams equation (1968) and Williams and Haan equation (1973)

<table>
<thead>
<tr>
<th>Equations</th>
<th>( r^2 )</th>
<th>( E )</th>
<th>( E_l )</th>
<th>( \text{RMSE} )</th>
<th>( \text{RMSE}_S )</th>
<th>( \text{RMSE}_U )</th>
<th>CRM</th>
<th>RBIAS</th>
</tr>
</thead>
<tbody>
<tr>
<td>Regression Equation (equation 5.6)</td>
<td>0.96</td>
<td>0.93</td>
<td>0.73</td>
<td>3.0</td>
<td>1.4</td>
<td>2.6</td>
<td>0.016</td>
<td>2.2</td>
</tr>
<tr>
<td>Williams (1968)</td>
<td>0.06</td>
<td>-2.59</td>
<td>-0.87</td>
<td>20.7</td>
<td>20.6</td>
<td>1.8</td>
<td>-0.857</td>
<td>-81.4</td>
</tr>
<tr>
<td>Williams and Haan (1973)</td>
<td>0.70</td>
<td>-1.02</td>
<td>-0.38</td>
<td>15.5</td>
<td>15.4</td>
<td>1.5</td>
<td>-0.617</td>
<td>-48.3</td>
</tr>
</tbody>
</table>
5.2.2 Subsurface Flow Recession Constant ($K_{subs}$)

The data from the 9 streamflow gauging stations was also used to develop an equation for predicting the subsurface flow recession constant ($K_{subs}$). The following watershed characteristics were used to predict $K_{subs}$: maximum flow distance ($MFD$); watershed slope ($S_w$); and hydraulic conductivity ($K_{eff}$). The same events that were used to estimate the surface storage recession constant were also used to determine the subsurface flow recession constant. The values of the exponents were found by a forward stepwise multiple linear regression analysis of $\log K_{surf}$, on the logarithms of the independent variables. The same procedure that was used to undertake the regression analysis of the time of concentration and surface storage recession constant was also used to undertake the regression analysis of the subsurface flow recession constant.

Each variable was plotted against each other to determine if a linear or non-linear data trend exists. Figure 5.12 is a plot of the subsurface flow recession constant versus the maximum flow distance, $MFD$ (km). According to Figure 5.12 there is a direct curvilinear relationship between the subsurface flow recession constant and the maximum flow distance. Plots of the other combinations of variable pairs are shown in Appendix D. The linear correlation coefficients for each pair of variables are listed in Table 5.14. The results of the stepwise regression analysis are in Tables 5.15 and 5.16. Table 5.15 shows the regression coefficients for each step of the regression analysis, and Table 5.16 shows the regression equation data for each step. The results of the regression analysis yielded the following equations:

$$K_{subs} = 12.781 MFD^{0.529}$$  \hspace{1cm} (5.7)

$$K_{subs} = 6.243 MFD^{0.513} S_w^{-0.221}$$  \hspace{1cm} (5.8)

$$K_{subs} = 4.972 MFD^{0.646} S_w^{-0.247} K_{eff}^{-0.179}$$  \hspace{1cm} (5.9)

Where the variables $MFD$ is the maximum flow distance (km), $S_w$ is the watershed slope (m/m) and $K_{eff}$ is the hydraulic conductivity (mm/h) for the basin. According to equations 5.7, 5.8 and 5.9 the subsurface flow recession constant varies directly with the maximum flow distance and
inversely with the watershed slope and the hydraulic conductivity. However, according to Table 5.14 the subsurface flow recession constant varies directly with the hydraulic conductivity. Therefore the relationships between the observed subsurface flow recession constant and the maximum flow distance and watershed slope are rational and the relationship between the observed subsurface flow recession constant and the hydraulic conductivity is irrational and the variable $K_{eff}$ should only be included in equation 5.9 if the increased significance is meaningful.

The coefficient of determination ($r^2$) values for equations 5.7, 5.8 and 5.9 were determined to be 0.81, 0.83 and 0.85 respectively. Equations 5.7, 5.8 and 5.9 explain 81%, 83% and 95% of the variation in the subsurface flow recession constant respectively. According to Table 5.16, for each step of the regression analysis, the standard error of estimate is smaller than the standard deviation of the logarithms of the subsurface flow recession constant. All steps of the regression analysis are significant based on the total F-test at the 5-percent level. The least significant variable is the watershed slope based on a 5 percent level F with 2 and 6 degrees of freedom. From a standard F table, for these degrees of freedom, F is equal to 5.143. The partial F value required to enter the watershed slope variable is 13.4149. The maximum flow distance and the watershed slope explain 83% of the variation ($r^2$) in the logarithm of the subsurface flow recession constant, and the addition of the hydraulic conductivity raises this to 85%. Equation 5.8 would be considered the best because equation 5.8 is a good predictor of the subsurface flow recession constant and the addition of the hydraulic conductivity only marginally increases the ($r^2$) value to 85%. However, equation 5.9 was selected as the predictor equation in order to illustrate a multiple predictor model. Other watershed characteristics that were tested as predictors of $K_{subs}$ included the drainage area, baseflow index, moisture content and percentage of wetland and water coverage. None of these parameters significantly improved the accuracy of equation 5.9.

The sum of the residuals of the logarithms of the observed and predicted subsurface flow recession constant values from Table 5.16 is 0.000. A plot of the residuals with the logarithms of the predicted subsurface flow recession constant values is shown in Figure 5.13. Figure 5.13 shows no correlation between the logarithms of the predicted subsurface flow recession constant
values and the residuals. The residual variation is also constant over the range of the logarithms of the predicted subsurface flow recession constant values.

The predicted subsurface flow recession constant values and the residual values for equation 5.9, along with the observed values are listed in Table 5.17. Plots of the residuals for equation 5.9 are illustrated in Figure 5.14. The greatest amount of under prediction (negative residual) occurs near a subsurface flow recession constant value of 87 hours. Three data points (Credit River at Erindale, Credit River at Alton Branch and West Credit River at Norval) in the region account for 95 percent of the sum of residuals squared. The greatest amount of over prediction (positive residuals) occurs near a subsurface flow recession constant of 56 hours. Large residual values positive or negative may be a problem when the regression equation is used in the upper range of subsurface flow recession constant values.

![Plot subsurface flow recession constant, $K_{subs}$ (h) versus maximum flow distance, $MFD$ (km)](image_url)
Table 0-14 Linear correlation coefficients between each pair of variables, including subsurface flow recession constant ($K_{subs}$), maximum flow distance ($MFD$), watershed slope ($S_w$) and hydraulic conductivity ($K_{eff}$)

<table>
<thead>
<tr>
<th></th>
<th>Subsurface Flow Recession Constant, $K_{subs}$ (h)</th>
<th>Maximum Flow Distance, $MFD$ (km)</th>
<th>Watershed Slope, $S_w$ (m/m)</th>
<th>Hydraulic Conductivity, $K_{eff}$ (mm/h)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Subsurface Flow Recession Constant, $K_{subs}$ (h)</td>
<td>1.000</td>
<td>0.943</td>
<td>-0.288</td>
<td>0.520</td>
</tr>
<tr>
<td>Maximum Flow Distance, $MFD$ (km)</td>
<td></td>
<td>1.000</td>
<td>-0.230</td>
<td>0.654</td>
</tr>
<tr>
<td>Watershed Slope, $S_w$ (m/m)</td>
<td></td>
<td></td>
<td>1.000</td>
<td>-0.193</td>
</tr>
<tr>
<td>Hydraulic Conductivity, $K_{eff}$ (mm/h)</td>
<td></td>
<td></td>
<td></td>
<td>1.000</td>
</tr>
</tbody>
</table>

Table 0-15 Regression coefficients for the predictor variables, maximum flow distance ($MFD$), watershed slope ($S_w$) and hydraulic conductivity ($K_{eff}$), that have high correlations with the observed subsurface flow recession constant ($K_{subs}$)

<table>
<thead>
<tr>
<th>Equation</th>
<th>Variable</th>
<th>Intercept</th>
<th>B₁</th>
<th>B₂</th>
<th>B₃</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>MFD</td>
<td>1.105</td>
<td>0.529</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>MFD, $S_w$</td>
<td>0.795</td>
<td>0.513</td>
<td>-0.221</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>MFD, $S_w$, $K_{eff}$</td>
<td>0.697</td>
<td>0.646</td>
<td>-0.248</td>
<td>-0.179</td>
</tr>
</tbody>
</table>

Table 0-16 Statistical quantities of coefficient of determination ($r^2$), standard error of estimate ($S_e$), sum of squares per degrees of freedom ($SS/df$), total F-test ($F_t$), partial F-test ($F_p$) and sum of residuals for each step of the regression process

<table>
<thead>
<tr>
<th>EQN</th>
<th>Variable</th>
<th>$r^2$</th>
<th>$\Delta r^2$</th>
<th>$S_e$ 0.1980$^1$</th>
<th>SS/df Regression</th>
<th>SS/df Residuals</th>
<th>$F_t$</th>
<th>$F_p$</th>
<th>Sum of Residuals</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>MFD</td>
<td>0.8084</td>
<td>0.8084</td>
<td>0.0927</td>
<td>0.21933/1</td>
<td>0.02518/7</td>
<td>29.5313</td>
<td>29.5313</td>
<td>0.000</td>
</tr>
<tr>
<td>2</td>
<td>MFD, $S_w$</td>
<td>0.8269</td>
<td>0.0185</td>
<td>0.0951</td>
<td>0.17056/2</td>
<td>0.00908/6</td>
<td>14.3291</td>
<td>0.6411</td>
<td>0.000</td>
</tr>
<tr>
<td>3</td>
<td>MFD, $S_w$, $K_{eff}$</td>
<td>0.8544</td>
<td>0.0275</td>
<td>0.0956</td>
<td>0.12200/3</td>
<td>0.00592/5</td>
<td>9.7794</td>
<td>0.9446</td>
<td>0.000</td>
</tr>
</tbody>
</table>

Note 1: Standard deviation ($S_e$) of the logarithms of the observed subsurface flow recession constant values, $K_{subs}$ (h)
Figure 0-13 Plot of the residuals versus the logarithms of the predicted subsurface flow recession constant values, $K_{sub}$ (h)

Table 0-17 Observed subsurface flow recession constant values, predicted subsurface flow recession constant values and residuals for the 9 WSC gauge stations along the Credit River watershed

<table>
<thead>
<tr>
<th>Station Name</th>
<th>Observed Subsurface Flow Recession Constant, $K_{sub}$ (h)</th>
<th>Predicted Subsurface Flow Recession Constant, $K_{sub}$ (h)</th>
<th>Residuals</th>
</tr>
</thead>
<tbody>
<tr>
<td>Credit River at Orangeville</td>
<td>57</td>
<td>58</td>
<td>-0.6</td>
</tr>
<tr>
<td>Credit River at Cataract</td>
<td>73</td>
<td>74</td>
<td>-1.4</td>
</tr>
<tr>
<td>Credit River at Boston Mills</td>
<td>76</td>
<td>76</td>
<td>0.0</td>
</tr>
<tr>
<td>Credit River at Norval</td>
<td>110</td>
<td>110</td>
<td>0.4</td>
</tr>
<tr>
<td>Credit River at Erindale</td>
<td>165</td>
<td>145</td>
<td>20</td>
</tr>
<tr>
<td>Credit River Erin Branch above Erin</td>
<td>36</td>
<td>39</td>
<td>-3.1</td>
</tr>
<tr>
<td>Credit River above Alton Branch</td>
<td>79</td>
<td>57</td>
<td>22</td>
</tr>
<tr>
<td>Black Creek below Acton</td>
<td>45</td>
<td>45</td>
<td>0.5</td>
</tr>
<tr>
<td>Credit River West Branch at Norval</td>
<td>63</td>
<td>87</td>
<td>-24</td>
</tr>
</tbody>
</table>
5.2.3 Groundwater Flow Recession Constant ($K_{gw}$)

Data from these nine (9) stream flow gauging stations was also used to develop an equation for predicting the groundwater flow recession constant ($K_{gw}$) as a function of maximum flow distance, watershed slope and hydraulic conductivity. The same events that were used to estimate the surface storage and subsurface flow recession constants were also used to determine the groundwater flow recession constant. An equation was derived for the groundwater flow recession constant using forward step-wise multiple linear regression analysis of $\log K_{gw}$, on the logarithms of the independent variables. The same procedure that was used to undertake the regression analysis of the time of concentration, surface storage recession constant and subsurface flow recession constant was also used to undertake the regression analysis of the groundwater flow recession constant.

Each variable was plotted against each other to determine if a linear or non-linear data trend exists. Figure 5.15 is a plot of the groundwater flow recession constant versus the maximum flow distance, $MFD$ (km). According to Figure 5.15 there is a direct linear relationship between the
groundwater flow recession constant and the maximum flow distance. Plots of the other combinations of variable pairs are shown in Appendix D. The linear correlation coefficients for each pair of variables are listed in Table 5.18. The results of the stepwise regression analysis are in Tables 5.19 and 5.20. Table 5.19 shows the regression coefficients for each step of the regression analysis, and Table 5.20 shows the regression equation data for each step. The results of the regression analysis yielded the following equations:

\[ K_{gw} = 22.015MFD^{0.534} \quad (5.10) \]

\[ K_{gw} = 8.654MFD^{0.514}S_w^{-0.289} \quad (5.11) \]

\[ K_{gw} = 5.897MFD^{0.738}S_w^{-0.333}K_{eff}^{-0.301} \quad (5.12) \]

Where the variables \( MFD \) is the maximum flow distance (km), \( S_w \) is the watershed slope (m/m) and \( K_{eff} \) is the hydraulic conductivity (mm/h) for the basin. According to equations 5.10, 5.11 and 5.12 the groundwater flow recession constant varies directly with the maximum flow distance and inversely with the watershed slope and the basins’ hydraulic conductivity. However, according to Table 5.18 the groundwater flow recession constant varies directly with the hydraulic conductivity. Therefore the relationships between the observed groundwater flow recession constant and the maximum flow distance and watershed slope are rational and the relationship between the observed groundwater flow recession constant and the hydraulic conductivity is irrational and the variable \( K_{eff} \) should only be included if the increased significance is meaningful.

The coefficient of determination \( (r^2) \) values for equations 5.10, 5.11 and 5.12 were determined to be 0.75, 0.78 and 0.85 respectively. Equations 5.10, 5.11 and 5.12 explain 75%, 78% and 85% of the variation in the groundwater flow recession constant respectively. According to Table 5.20, for each step of the regression analysis, the standard error of estimate is smaller than the standard deviation of the logarithms of the groundwater flow recession constant. All steps of the regression analysis are significant based on the total F-test at the 5-percent level. The least significant variable is the maximum flow distance based on a 5 percent level F with 1 and 7
degrees of freedom. From a standard F table, for these degrees of freedom, F is equal to 5.591. The partial F value required to enter the maximum flow distance variable is 21.5186. The maximum flow distance explains 75% of the variation ($r^2$) in the logarithm of the groundwater flow recession constant, and the addition of the watershed slope and hydraulic conductivity raises this to 86%. Equation 5.12 was selected as the predictor equation in order to illustrate a multiple predictor model. Other watershed characteristics that were tested as predictors of $K_{gw}$ included drainage area, basin width, average basin slope, moisture content and percentage of wetland and water coverage. None of these parameters significantly improved the accuracy of equation 5.12.

The sum of the residuals of the logarithms of the observed and predicted subsurface flow recession constant values from Table 5.20 is 0.000. A plot of the residuals with the logarithms of the predicted groundwater flow recession constant values is shown in Figure 5.16. Figure 5.16 shows no correlation between the logarithms of the predicted groundwater flow recession constant values and the residuals. The residual variation is also constant over the range of the logarithms of the predicted groundwater flow recession constant values.

The predicted groundwater flow recession constant values and the residual values for equation 5.12, along with the observed values are listed in Table 5.21. Plots of the residuals for equation 5.12 are illustrated in Figure 5.17. The greatest amount of under prediction (negative residual) occurs near a subsurface flow recession constant value of 100 hours. Two data points (Credit River at Alton Branch and Credit River West Branch at Norval) in the region account for 82 percent of the sum of residuals squared. The greatest amount of over prediction (positive residuals) occurs near a groundwater flow recession constant of 162 hours. Large residual values positive or negative may be a problem when the regression equation is used in the upper range of subsurface flow recession constant values.
Figure 0-15 Plot groundwater flow recession constant, $K_{gw}$ (h) versus maximum flow distance, $MFD$ (km)

Table 0-18 Linear correlation coefficients between each pair of variables, including groundwater flow recession constant ($K_{gw}$), maximum flow distance ($MFD$), watershed slope ($S_w$) and hydraulic conductivity ($K_{eff}$)

<table>
<thead>
<tr>
<th></th>
<th>Groundwater Flow Recession Constant, $K_{gw}$ (h)</th>
<th>Maximum Flow Distance, $MFD$ (km)</th>
<th>Watershed Slope, $S_w$ (m/m)</th>
<th>Hydraulic Conductivity, $K_{eff}$ (mm/h)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Groundwater Flow Recession Constant, $K_{gw}$ (h)</td>
<td>1.000</td>
<td>0.914</td>
<td>-0.311</td>
<td>0.448</td>
</tr>
<tr>
<td>Maximum Flow Distance, $MFD$ (km)</td>
<td></td>
<td>1.000</td>
<td>-0.201</td>
<td>0.675</td>
</tr>
<tr>
<td>Watershed Slope, $S_w$ (m/m)</td>
<td></td>
<td></td>
<td>1.000</td>
<td>-0.121</td>
</tr>
<tr>
<td>Hydraulic Conductivity, $K_{eff}$ (mm/h)</td>
<td></td>
<td></td>
<td></td>
<td>1.000</td>
</tr>
</tbody>
</table>

Table 0-19 Regression coefficients for the predictor variables, maximum flow distance ($MFD$), watershed slope ($S_w$) and hydraulic conductivity ($K_{eff}$), that have high correlations with the observed groundwater flow recession constant ($K_{gw}$)

<table>
<thead>
<tr>
<th>Equation</th>
<th>Variable</th>
<th>Intercept</th>
<th>$B_1$</th>
<th>$B_2$</th>
<th>$B_3$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>MFD</td>
<td>1.343</td>
<td>0.534</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>MFD, $S_w$</td>
<td>0.937</td>
<td>0.514</td>
<td>-0.289</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>MFD, $S_w$, $K_{eff}$</td>
<td>0.771</td>
<td>0.738</td>
<td>-0.333</td>
<td>-0.301</td>
</tr>
</tbody>
</table>
Table 0-20 Statistical quantities of coefficient of determination \( (r^2) \), standard error of estimate \( (S_e) \), sum of squares per degrees of freedom \( (SS/df) \), total F-test \( (F_t) \), partial F-test \( (F_p) \) and sum of residuals for each step of the regression process.

<table>
<thead>
<tr>
<th>EQN</th>
<th>Variable</th>
<th>( r^2 )</th>
<th>( \Delta r^2 )</th>
<th>( S_e ) ( \text{0.2071} )</th>
<th>( SS/df ) Regression</th>
<th>( SS/df ) Residuals</th>
<th>( F_t )</th>
<th>( F_p )</th>
<th>Sum of Residuals</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>MFD</td>
<td>0.7545</td>
<td>0.7545</td>
<td>0.1097</td>
<td>0.2589/1</td>
<td>0.0120/7</td>
<td>21.5186</td>
<td>21.5186</td>
<td>0.000</td>
</tr>
<tr>
<td>2</td>
<td>MFD, ( S_w )</td>
<td>0.7833</td>
<td>0.0287</td>
<td>0.1113</td>
<td>0.1344/2</td>
<td>0.0124/6</td>
<td>10.8418</td>
<td>0.7950</td>
<td>0.000</td>
</tr>
<tr>
<td>3</td>
<td>MFD, ( S_w ), ( K_{eff} )</td>
<td>0.8547</td>
<td>0.0714</td>
<td>0.0999</td>
<td>0.0978/3</td>
<td>0.0100/5</td>
<td>9.8027</td>
<td>2.4575</td>
<td>0.000</td>
</tr>
</tbody>
</table>

Note 1: Standard deviation \( (S) \) of the logarithms of the observed groundwater flow recession constant values, \( K_{gw} \) (h)

Figure 0-16 Plot of the residuals versus the logarithms of the predicted groundwater flow recession constant values, \( K_{gw} \) (h)
Table 0-21 Observed groundwater flow recession constant values, predicted groundwater flow recession constant values and residuals for the 9 WSC gauge stations along the Credit River watershed

<table>
<thead>
<tr>
<th>Station Name</th>
<th>Observed Subsurface Flow Recession Constant, $K_{subs}$ (h)</th>
<th>Predicted Subsurface Flow Recession Constant, $K_{subs}$ (h)</th>
<th>Residuals</th>
</tr>
</thead>
<tbody>
<tr>
<td>Credit River at Orangeville</td>
<td>101</td>
<td>103</td>
<td>2.6</td>
</tr>
<tr>
<td>Credit River at Cataract</td>
<td>123</td>
<td>136</td>
<td>13.8</td>
</tr>
<tr>
<td>Credit River at Boston Mills</td>
<td>124</td>
<td>120</td>
<td>-3.8</td>
</tr>
<tr>
<td>Credit River at Norval</td>
<td>189</td>
<td>190</td>
<td>0.8</td>
</tr>
<tr>
<td>Credit River at Erin</td>
<td>292</td>
<td>266</td>
<td>-25.5</td>
</tr>
<tr>
<td>Credit River Erin Branch above Erin</td>
<td>59</td>
<td>66</td>
<td>6.5</td>
</tr>
<tr>
<td>Credit River above Alton Branch</td>
<td>156</td>
<td>104</td>
<td>-51.9</td>
</tr>
<tr>
<td>Black Creek below Acton</td>
<td>74</td>
<td>76</td>
<td>1.9</td>
</tr>
<tr>
<td>Credit River West Branch at Norval</td>
<td>125</td>
<td>162</td>
<td>37.2</td>
</tr>
</tbody>
</table>

Figure 0-17 Plot of the residuals versus the predicted groundwater flow recession constant values, $K_{gw}$ (h)

Figures 5.18 and 5.19 graphically illustrate the correlation between the observed and predicted values for the subsurface flow and groundwater flow recession constants using equations 5.9 and 5.12 respectively. According to Figures 5.18 and 5.19 there is a linear trend among the data points. Table 5.22 provides nine (9) statistical quantities for evaluating the performance of the regression equations (equations 5.9 and 5.12) for estimating the subsurface flow and groundwater flow recession constants. According to Table 5.22, the observed values are highly
correlated with the predicted values of equations 5.9 and 5.12. In addition, the differences between the coefficients of determination and the efficiency coefficients for equations 5.9 and 5.12 are negligible and the relative biases for the two equations are small, 0.9% for equation 5.9 and 1.4% for equation 5.12. Furthermore, the root mean square error values (RMSE) for equations 5.9 and 5.12 are less than half the standard deviation of the measured or observed values and therefore, are considered to be low \(0.5 \cdot \sigma_{measured} = 19.5\) for \(K_{subs}\) and \(0.5 \cdot \sigma_{measured} = 34.9\) for \(K_{gw}\); both acceptably low values. The same applies for the other statistical criteria including the systematic root mean error (RMSEs), unsystematic root mean square error (RMSEu) and the coefficient of residual mass (CRM).

The effect of area of storage on time of runoff and flow depends on the geographical location of the storage and on the level of the water in the storage areas at the beginning of runoff (Watt, 1989). Since 14 runoff events were used to come up with an average recession constant (Table 4.8), there is an inherit variance in the results, since values for the recession constant may vary due to seasonal changes and antecedent conditions. In the current analysis, two of the gauges used were inactive and recession constants were estimated from simulated hydrographs and verified through historical data. A more thorough approach would be to use the same runoff event for all stream gauges and to determine the recession constant values for each basin, and then undertake a regression analysis to determine the exponents for the physical parameters of the basins. This process should be repeated for each runoff event and an examination of the variance of the exponent values due to changing antecedent conditions should be undertaken. This could potentially pose a problem, given the difficulty in isolating an event that is uniform across all basins.

The variance in the recession constant values is even more apparent for the groundwater flow. According to Table 4.8 there was significant variance in the recession constant values for the groundwater flow due to varying antecedent conditions for the different runoff events. There was less discrepancy between the predicted and observed recession constant values for groundwater flow than for predicted and estimated values for the subsurface flow. The soil water content parameter theta was included as a predictor in the regression analysis for the surface storage recession constant, subsurface flow recession constant and groundwater flow recession constant.
However, the improvement in prediction resulting from the addition of the soil water content parameter was only marginal. The $r^2$ value increased only slightly from 0.93 to 0.94 for equation 5.6 and from 0.85 to 0.86 for Equations 5.9 and 5.12. On the basis of these statistical results the inclusion of the soil water content theta cannot be justified. Furthermore, the intent of these empirical equations is to estimate the recession constants for surface storage, subsurface flow and groundwater flow for ungauged basins using physical parameters which cannot be directly measured. Without hydrometric data it is difficult to estimate antecedent conditions within the basin.

As more data becomes available these analyses should be expanded to include additional basins of similar physiography. In addition, the analysis can also be expanded to other regions within Ontario and across Canada. Furthermore, with the availability of more data it should be possible to derive a series of equations of the form of equations 5.6, 5.9 and 5.12 that can be applied to basins with varying landuse, slope, hydraulic conductivity and shape, and which are more certain of not being unduly influenced by one or two odd basins. Equations 5.6, 5.9 and 5.12 are unique in that they were derived using a watershed within Southern Ontario, whereas previously estimations of the recession constants for ungauged basins were undertaken using equations that were derived in the Southern United States. Aside from the differences in watershed shape, there are also significant differences in meteorology and climate between the two regions, which are driving forces behind the hydrologic processes.
Figure 0-18 Observed subsurface flow recession constant \( (K_{\text{subs}}) \) versus predicted subsurface flow recession constant estimates using the regression equation (equation 5.9)

Figure 0-19 Observed groundwater flow recession constant \( (K_{\text{gw}}) \) versus predicted groundwater flow recession constant estimates using the regression equation (equation 5.12)
Table 0-22 Statistical quantities of coefficient of determination ($r^2$), coefficient of efficiency (E), modified coefficient of efficiency ($E_1$), root mean square error (RMSE), systematic root mean square error (RMSE_s), unsystematic root mean square error (RMSE_u), coefficient of residual mass (CRM), and relative bias (RBIAS) for estimating the subsurface flow recession constant (equation 5.9) and the groundwater flow recession constant (equation 5.12)

<table>
<thead>
<tr>
<th>Equations</th>
<th>$r^2$</th>
<th>E</th>
<th>$E_1$</th>
<th>RMSE</th>
<th>RMSE_s</th>
<th>RMSE_u</th>
<th>CRM</th>
<th>RBIAS</th>
</tr>
</thead>
<tbody>
<tr>
<td>Subsurface flow Recession</td>
<td>0.88</td>
<td>0.88</td>
<td>0.70</td>
<td>12.9</td>
<td>7.1</td>
<td>10.8</td>
<td>-0.020</td>
<td>0.9</td>
</tr>
<tr>
<td>Constant (equation 5.9)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Groundwater flow Recession</td>
<td>0.93</td>
<td>0.93</td>
<td>0.75</td>
<td>23.5</td>
<td>17.8</td>
<td>23.7</td>
<td>-0.015</td>
<td>1.4</td>
</tr>
<tr>
<td>Constant (equation 5.12)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

5.3 Modelling of Channel Flow Progression (Routing)

5.3.1 Testing and Evaluation of Simplified Dynamic Model

The simplified dynamic model uses an explicit numerical solution to solve the Saint Venant equations, the courant condition applies. Therefore, for accuracy and to maintain stability within the computations, the ratio $\frac{\Delta t}{\Delta x}$ should be less than the courant criteria. The courant number for a trapezoidal section was estimated to be approximately 0.42 for $z=1$. For other values of $z$, the courant number varied from 0.39 ($z=2$) to 0.35 ($z=4$). This is based on a reference flow rate of 7.5 m$^3$/s; which was calculated from the inflow hydrograph using equation 5.13 (USACE, 2000):

$$Q_o = Q_B + \frac{1}{2} (Q_p - Q_B)$$

(5.13)

Where $Q_B$ is the baseflow (3 m$^3$/s) and $Q_p$ is the peak flow (12 m$^3$/s). As a result, in order to maintain computational stability the space interval $\Delta x=100$ m and the time interval $\Delta t=10$s was selected. Larger values for $\Delta t$ have been tested however; this resulted in the non-convergence of the solution technique of the simplified dynamic model. The computed discharge hydrographs and the triangular inflow hydrograph for a trapezoidal channel section are illustrated in Figure 5.20 for different distances along the channel, 600 m, 1200 m and 2000 m. According to Figure 5.20, outflow hydrographs have peak flows of 9.6, 7.9 and 6.5 m$^3$/s at distances 600 m, 1200 m and 2000 m respectively. The times to peak flows are 12, 15 and 20 minutes respectively.
The courant number for a triangular channel section varied between 0.43 for \( z = 1 \) and 0.33 for \( z = 4 \). To maintain computational stability, the space interval \( \Delta x \) was set equal to 100 m and the time interval \( \Delta t \) was set equal to 9 s. Larger values of \( \Delta t \) were tested however this resulted in the numerical solutions’ failure to converge. The computed discharge hydrographs for a triangular channel section are illustrated in Figure 5.21 for different distances along the channel, 600 m, 1200 m and 2000 m. According to Figure 5.21, outflow hydrographs have peak flows of 9.7, 7.9 and 6.3 \( m^3/s \) at distances 600 m, 1200 m and 2000 m respectively. The times to peak flows are 11, 12 and 16 minutes respectively.

To show the effect of the inverse channel side slope (\( z \)) and the channel length on the hydrographs and to compare the inflow and outflow hydrographs, dimensionless discharge, and time variables are designated in Table 5.23 for the triangular inflow hydrograph. Dimensionless attenuations (difference in peaks) and translations (difference in time-to-peaks) are calculated and shown in Table 5.23.

According to Table 5.23, the amount of attenuation increases as the distance increases and as the inverse side slope (\( z \)) increases for both trapezoidal and triangular channel sections. However, attenuation decreases as the number of routing steps increases. The number of routing steps is defined as the distance (\( L \)) of the reach divided by the space interval (\( \Delta x \)). In the above example, the number of routing steps is equal to 20. The maximum attenuation corresponds to one step; this is commonly used to route flows through lakes, ponds, reservoirs, wide floodplains and channels and where the flow is controlled by downstream conditions. Translation also increases with increasing distance and with increasing inverse channel side slopes (\( z \)) for both trapezoidal and triangular channel sections. For the triangular channel section the computational time step (\( \Delta t \)), was reduced by one-second in order to satisfy the Courant Condition and to ensure the convergence of the explicit numerical solution scheme. This led to an increase in the number of computational time steps, which in turn, led to a decrease in the translation of the outflow hydrographs for the triangular section. According to Table 5.23, the dimensionless translation for a triangular channel section is approximately half of the dimensionless translation of a trapezoidal channel section. The simplified dynamic model (SDM) is applicable to reaches.
where the flow is supercritical or when the effect of the downstream boundary condition can be disregarded for subcritical flows.

To demonstrate the validity and efficiency of the proposed simplified dynamic model, the derivative $\frac{\partial s_f}{\partial x}$ neglected in equation 4.36 for channel flow and in equations 4.74, 4.88 and 4.91 for floodplain flows is accounted for in the momentum equation. From the momentum equation for channel flow, equation 4.25, the following equation can be obtained for channel flow using equation 4.36 with the term $\frac{\partial s_f}{\partial x}$ as

$$\frac{\partial q}{\partial t} + \alpha \frac{\partial q}{\partial x} + \beta + \gamma \frac{\partial s_f}{\partial x} = 0 \quad (5.14)$$

Where

$$\gamma = \left( \frac{q^2}{A} - \frac{gA}{B} \right) \frac{A}{2s_f} \left( \frac{1}{3} \frac{4K}{3B} \sqrt{\frac{q^2}{4} + 1} \right) \quad (5.15)$$

For floodplain flows, equation 4.55, the following equations can be obtained for channel flow using equations 4.74, 4.88 and 4.91 with the terms $\frac{\partial s_{fcn}}{\partial x_{cn}}$, $\frac{\partial s_{flf}}{\partial x_{lf}}$ and $\frac{\partial s_{rf}}{\partial x_{rf}}$ as

$$\frac{\partial Q}{\partial t} + \alpha_{cn} \frac{\partial Q}{\partial x_{cn}} + \alpha_{lf} \frac{\partial Q}{\partial x_{lf}} + \alpha_{rf} \frac{\partial Q}{\partial x_{rf}} + \beta_{cn} + \beta_{lf} + \beta_{rf} + \gamma_{cn} \frac{\partial s_{fcn}}{\partial x_{cn}} + \gamma_{lf} \frac{\partial s_{flf}}{\partial x_{lf}} + \gamma_{rf} \frac{\partial s_{rf}}{\partial x_{rf}} = 0 \quad (5.16)$$

Where

$$\gamma_{cn} = \left( \frac{K_{cn}^2 q^2}{A_{cn}^2} - \frac{gA_{cn}}{B_{cn}} \right) \frac{A_{cn}}{2s_{fcn}(\frac{2}{3})} \quad (5.17)$$
In equations 5.15, 5.17, 5.18 and 5.19, gamma (γ) is the dependent variable and is related to cross-sectional characteristics of the channel and floodplain and the discharges.

Equations 5.14 and 5.16 and can be solved with their respective continuity equations, equations 4.24 and 4.54, using the same algorithms defined in Section 4.3.1. The results of the simplified dynamic model were compared against the results for equations 5.14 and 5.16. The comparison was undertaken for a trapezoidal channel and triangular channel section with \( z = 1 \) and using the same triangular inflow hydrograph and \( \Delta x \) and \( \Delta t \) values as in the previous analysis. The results of the analyses are illustrated in Figures 5.22 and 5.23 for the trapezoidal channel section and triangular channel section respectively.

For the trapezoidal channel the simplified dynamic model gives a peak flow of 6.5 m\(^3\)/s and a time to peak of 20 minutes, whereas the general dynamic model for a trapezoidal channel section yields a peak flow of 6.5 m\(^3\)/s and a time to peak of 20 minutes. According to Figure 5.22 there is no difference in the outflow hydrographs between the general dynamic model and the simplified dynamic model for a trapezoidal channel section. In addition, for different bottom slopes as \( S_o = 0.05, 0.005 \) and \( 0.0005 \), and for different peak flows of inflow hydrograph as \( Q_p = 12, 10 \) and \( 8 \) m\(^3\)/s, the simplified dynamic model and the general dynamic model yields almost the same runoff hydrographs for a trapezoidal channel. The results are summarized in Table 5.24. For the triangular channel the simplified dynamic model gives a peak flow of 6.3 m\(^3\)/s and a time to peak of 16 minutes, the general dynamic model for a triangular channel section yields a peak flow of 6.3 m\(^3\)/s and a time to peak of 18 minutes. According to Figure 5.23 there is good agreement in the magnitude and shape of the outflow hydrographs between the general dynamic model and the simplified dynamic model for a triangular channel section, however, there is an increase in the translation of the runoff hydrograph by 2 minutes for the general dynamic model.
Table 5.25 compares the peak flows and time to peaks for the runoff hydrographs for the simplified dynamic model and general dynamic model for a triangular channel section, for different bottom slopes ($S_o = 0.05$, 0.005 and 0.0005) and different peak flows of inflow hydrograph ($Q_p = 12$, 10 and 8 m³/s). According to Table 5.18 there are no significant differences in the results between the general dynamic model and the simplified dynamic model for a triangular channel section. The 2 minute delay is evident in the change in slopes and for the base case, however, for the change in peak flows there is no change in the results between the two models. Although, the shape of the channel does affect the translation of the runoff hydrograph, and the inclusion of the term $\frac{\partial s_x}{\partial x}$ in the model further intensifies this effect, the simplified dynamic model can be used where the general dynamic model is appropriate, provided that there are no abrupt changes to the shape of the channel along the reach. Changes in the channel shape typically occur near bridges, culverts, and barriers such as weirs, where there is a contraction or expansion of flow, in which case, the reach would need to be subdivided into several sub-reaches to account for these changes.

A comparison was made between the simplified dynamic model and the more complex dynamic wave model of the FLDWAV model (Fread and Lewis, 1998). An event was selected from the literature as used by Akan and Yen (1981) and Keskin (1997). The same triangular inflow hydrograph was used as in the previous examples. The FLDWAV model solves the Saint-Venant equations using a four point implicit solution. Figures 5.24 and 5.25, illustrate the outflow hydrographs at 2000 m for both the simplified dynamic model and the FLDWAV model for a trapezoidal channel and a triangular channel respectively. The peak flow for the simplified dynamic model for a trapezoidal channel is 6.5 m³/s and its time is 20 minutes, whereas for the dynamic wave subroutine within the FLDWAV model the peak flow is 7.3 m³/s and its time is 20 minutes. The peak flow for the simplified dynamic model for a triangular channel is 6.3 m³/s and its time is 16 minutes, whereas for the dynamic wave subroutine of the FLDWAV model the peak flow is 7.2 m³/s and its time is 18 minutes.
Figure 0-20 Inflow hydrograph and outflow hydrographs for different distances for a trapezoidal channel

Figure 0-21 Inflow hydrograph and outflow hydrographs for different distances for a triangular channel
Table 0-23 Dimensionless characteristics of inflow hydrograph and outflow hydrographs at different distances and inverse channel side slopes ($z$) for the simplified dynamic model (SDM) for trapezoidal and triangular channels

<table>
<thead>
<tr>
<th>Characteristics of Hydrographs</th>
<th>Inflow Hydrograph (x=0)</th>
<th>Outflow Hydrograph (x=600 m)</th>
<th>Outflow Hydrograph (x=1200 m)</th>
<th>Outflow Hydrograph (x=2000 m)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Trapezoid</td>
<td>Triangle</td>
<td>Trapezoid</td>
</tr>
<tr>
<td>Max. discharge, $Q_{max}$</td>
<td>12.00</td>
<td>9.6</td>
<td>9.7</td>
<td>7.9</td>
</tr>
<tr>
<td>Time to max. discharge, $t_{max}$ (min)</td>
<td>10.00</td>
<td>11.8</td>
<td>11.0</td>
<td>15.2</td>
</tr>
<tr>
<td>Dimensionless discharge, $Q_{max}/Q_p$</td>
<td>1.000</td>
<td>0.80</td>
<td>0.81</td>
<td>0.66</td>
</tr>
<tr>
<td>Dimensionless attenuation, $(Q_p - Q_{max})/Q_p$</td>
<td>0.000</td>
<td>0.20</td>
<td>0.19</td>
<td>0.34</td>
</tr>
<tr>
<td>Dimensionless translation, $(t_p - t_{max})/t_p$</td>
<td>0.0</td>
<td>0.18</td>
<td>0.09</td>
<td>0.52</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Characteristics of Hydrographs</th>
<th>Inflow Hydrograph (x=2)</th>
<th>Outflow Hydrograph (x=600 m)</th>
<th>Outflow Hydrograph (x=1200 m)</th>
<th>Outflow Hydrograph (x=2000 m)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Trapezoid</td>
<td>Triangle</td>
<td>Trapezoid</td>
</tr>
<tr>
<td>Max. discharge, $Q_{max}$</td>
<td>12.00</td>
<td>9.4</td>
<td>9.5</td>
<td>7.7</td>
</tr>
<tr>
<td>Time to max. discharge, $t_{max}$ (min)</td>
<td>10.00</td>
<td>12.2</td>
<td>11.1</td>
<td>15.8</td>
</tr>
<tr>
<td>Dimensionless discharge, $Q_{max}/Q_p$</td>
<td>1.000</td>
<td>0.78</td>
<td>0.79</td>
<td>0.64</td>
</tr>
<tr>
<td>Dimensionless attenuation, $(Q_p - Q_{max})/Q_p$</td>
<td>0.000</td>
<td>0.22</td>
<td>0.21</td>
<td>0.36</td>
</tr>
<tr>
<td>Dimensionless translation, $(t_p - t_{max})/t_p$</td>
<td>0.0</td>
<td>0.22</td>
<td>0.11</td>
<td>0.58</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Characteristics of Hydrographs</th>
<th>Inflow Hydrograph (x=3)</th>
<th>Outflow Hydrograph (x=600 m)</th>
<th>Outflow Hydrograph (x=1200 m)</th>
<th>Outflow Hydrograph (x=2000 m)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Trapezoid</td>
<td>Triangle</td>
<td>Trapezoid</td>
</tr>
<tr>
<td>Max. discharge, $Q_{max}$</td>
<td>12.00</td>
<td>9.2</td>
<td>9.2</td>
<td>7.5</td>
</tr>
<tr>
<td>Time to max. discharge, $t_{max}$ (min)</td>
<td>10.00</td>
<td>12.3</td>
<td>11.3</td>
<td>16.3</td>
</tr>
<tr>
<td>Dimensionless discharge, $Q_{max}/Q_p$</td>
<td>1.000</td>
<td>0.77</td>
<td>0.77</td>
<td>0.63</td>
</tr>
<tr>
<td>Dimensionless attenuation, $(Q_p - Q_{max})/Q_p$</td>
<td>0.000</td>
<td>0.23</td>
<td>0.23</td>
<td>0.37</td>
</tr>
<tr>
<td>Dimensionless translation, $(t_p - t_{max})/t_p$</td>
<td>0.0</td>
<td>0.23</td>
<td>0.13</td>
<td>0.63</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Characteristics of Hydrographs</th>
<th>Inflow Hydrograph (x=4)</th>
<th>Outflow Hydrograph (x=600 m)</th>
<th>Outflow Hydrograph (x=1200 m)</th>
<th>Outflow Hydrograph (x=2000 m)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Trapezoid</td>
<td>Triangle</td>
<td>Trapezoid</td>
</tr>
<tr>
<td>Max. discharge, $Q_{max}$</td>
<td>12.00</td>
<td>9.1</td>
<td>9.0</td>
<td>7.4</td>
</tr>
<tr>
<td>Time to max. discharge, $t_{max}$ (min)</td>
<td>10.00</td>
<td>12.5</td>
<td>11.3</td>
<td>16.7</td>
</tr>
<tr>
<td>Dimensionless discharge, $Q_{max}/Q_p$</td>
<td>1.000</td>
<td>0.76</td>
<td>0.75</td>
<td>0.62</td>
</tr>
<tr>
<td>Dimensionless attenuation, $(Q_p - Q_{max})/Q_p$</td>
<td>0.000</td>
<td>0.24</td>
<td>0.25</td>
<td>0.38</td>
</tr>
<tr>
<td>Dimensionless translation, $(t_p - t_{max})/t_p$</td>
<td>0.0</td>
<td>0.25</td>
<td>0.13</td>
<td>0.67</td>
</tr>
</tbody>
</table>
Figure 0-22 Outflow hydrographs for general and simplified dynamic models for a trapezoidal channel

Figure 0-23 Outflow hydrographs for general and simplified dynamic models for a triangular channel
Table 0-24 Dimensionless characteristics of inflow hydrograph and outflow hydrographs at different distances, channel slopes, and peak flows for the simplified dynamic model (SDM) and the general dynamic model (GDM) for a trapezoidal channel

<table>
<thead>
<tr>
<th>Characteristics of Hydrographs</th>
<th>Inflow Hydrograph at (x= 0)</th>
<th>Outflow Hydrograph (x= 600 m)</th>
<th>Outflow Hydrograph (x= 1200 m)</th>
<th>Outflow Hydrograph (x= 2000 m)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>SDM</td>
<td>GDM</td>
<td>SDM</td>
<td>GDM</td>
</tr>
<tr>
<td>Base Case</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Max. discharge, ( Q_{\max} ) (m³/s)</td>
<td>12.00</td>
<td>9.6</td>
<td>9.6</td>
<td>7.9</td>
</tr>
<tr>
<td>Time to max. discharge, ( t_{\max} ) (min)</td>
<td>10.00</td>
<td>11.8</td>
<td>11.8</td>
<td>15.2</td>
</tr>
<tr>
<td>Dimensionless discharge, ( Q_{\max}/Q_0 )</td>
<td>1.000</td>
<td>0.80</td>
<td>0.80</td>
<td>0.66</td>
</tr>
<tr>
<td>Dimensionless attenuation, ( (Q_0-Q_{\max})/Q_0 )</td>
<td>0.000</td>
<td>0.20</td>
<td>0.20</td>
<td>0.34</td>
</tr>
<tr>
<td>Dimensionless translation, ( (t_0-t_{\max})/t_p )</td>
<td>0.0</td>
<td>0.18</td>
<td>0.18</td>
<td>0.52</td>
</tr>
<tr>
<td>( S_o = 0.05 ) Max. discharge, ( Q_{\max} ) (m³/s)</td>
<td>12.00</td>
<td>11.7</td>
<td>11.7</td>
<td>11.6</td>
</tr>
<tr>
<td>Time to max. discharge, ( t_{\max} ) (min)</td>
<td>10.00</td>
<td>10.9</td>
<td>10.9</td>
<td>11.8</td>
</tr>
<tr>
<td>Dimensionless discharge, ( Q_{\max}/Q_0 )</td>
<td>1.000</td>
<td>0.98</td>
<td>0.98</td>
<td>0.97</td>
</tr>
<tr>
<td>Dimensionless attenuation, ( (Q_0-Q_{\max})/Q_0 )</td>
<td>0.000</td>
<td>0.02</td>
<td>0.02</td>
<td>0.03</td>
</tr>
<tr>
<td>Dimensionless translation, ( (t_0-t_{\max})/t_p )</td>
<td>0.0</td>
<td>0.09</td>
<td>0.09</td>
<td>0.18</td>
</tr>
<tr>
<td>( S_o = 0.005 ) Max. discharge, ( Q_{\max} ) (m³/s)</td>
<td>12.00</td>
<td>11.3</td>
<td>11.3</td>
<td>11.6</td>
</tr>
<tr>
<td>Time to max. discharge, ( t_{\max} ) (min)</td>
<td>10.00</td>
<td>11.7</td>
<td>11.7</td>
<td>11.8</td>
</tr>
<tr>
<td>Dimensionless discharge, ( Q_{\max}/Q_0 )</td>
<td>1.000</td>
<td>0.98</td>
<td>0.94</td>
<td>0.97</td>
</tr>
<tr>
<td>Dimensionless attenuation, ( (Q_0-Q_{\max})/Q_0 )</td>
<td>0.000</td>
<td>0.02</td>
<td>0.06</td>
<td>0.03</td>
</tr>
<tr>
<td>Dimensionless translation, ( (t_0-t_{\max})/t_p )</td>
<td>0.0</td>
<td>0.09</td>
<td>0.17</td>
<td>0.18</td>
</tr>
<tr>
<td>( Q_{\text{peak}} = 10 ) m³/s Max. discharge, ( Q_{\max} ) (m³/s)</td>
<td>10.00</td>
<td>8.0</td>
<td>8.0</td>
<td>6.7</td>
</tr>
<tr>
<td>Time to max. discharge, ( t_{\max} ) (min)</td>
<td>10.00</td>
<td>12.0</td>
<td>12.0</td>
<td>15.5</td>
</tr>
<tr>
<td>Dimensionless discharge, ( Q_{\max}/Q_0 )</td>
<td>1.000</td>
<td>0.80</td>
<td>0.80</td>
<td>0.46</td>
</tr>
<tr>
<td>Dimensionless attenuation, ( (Q_0-Q_{\max})/Q_0 )</td>
<td>0.000</td>
<td>0.20</td>
<td>0.20</td>
<td>0.54</td>
</tr>
<tr>
<td>Dimensionless translation, ( (t_0-t_{\max})/t_p )</td>
<td>0.0</td>
<td>0.20</td>
<td>0.20</td>
<td>0.58</td>
</tr>
<tr>
<td>( Q_{\text{peak}} = 8 ) m³/s Max. discharge, ( Q_{\max} ) (m³/s)</td>
<td>8.00</td>
<td>6.5</td>
<td>6.5</td>
<td>5.6</td>
</tr>
<tr>
<td>Time to max. discharge, ( t_{\max} ) (min)</td>
<td>10.00</td>
<td>12.2</td>
<td>12.2</td>
<td>15.8</td>
</tr>
<tr>
<td>Dimensionless discharge, ( Q_{\max}/Q_0 )</td>
<td>1.000</td>
<td>0.81</td>
<td>0.81</td>
<td>0.70</td>
</tr>
<tr>
<td>Dimensionless attenuation, ( (Q_0-Q_{\max})/Q_0 )</td>
<td>0.000</td>
<td>0.19</td>
<td>0.19</td>
<td>0.30</td>
</tr>
<tr>
<td>Dimensionless translation, ( (t_0-t_{\max})/t_p )</td>
<td>0.0</td>
<td>0.22</td>
<td>0.22</td>
<td>0.58</td>
</tr>
</tbody>
</table>
Table 0-25 Dimensionless characteristics of inflow hydrograph and outflow hydrographs at different distances and inverse channel side slopes (z) for the simplified dynamic model (SDM) and the general dynamic model (GDM) for a triangular channel

<table>
<thead>
<tr>
<th>Characteristics of Hydrographs</th>
<th>Inflow Hydrograph at (x= 0)</th>
<th>Outflow Hydrograph (x= 600 m)</th>
<th>Outflow Hydrograph (x= 1200 m)</th>
<th>Outflow Hydrograph (x= 2000 m)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>SDM</td>
<td>GDM</td>
<td>SDM</td>
<td>GDM</td>
</tr>
<tr>
<td><strong>Base Case</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Max. discharge, $Q_{max}$ (m³/s)</td>
<td>12.00</td>
<td>9.7</td>
<td>9.5</td>
<td>7.9</td>
</tr>
<tr>
<td>Time to max. discharge, $t_{max}$ (min)</td>
<td>10.00</td>
<td>11.0</td>
<td>11.6</td>
<td>12.3</td>
</tr>
<tr>
<td>Dimensionless discharge, $Q_{max}/Q_p$</td>
<td>1.000</td>
<td>0.81</td>
<td>0.79</td>
<td>0.66</td>
</tr>
<tr>
<td>Dimensionless attenuation, $(Q_p-Q_{max})/Q_p$</td>
<td>0.000</td>
<td>0.19</td>
<td>0.21</td>
<td>0.34</td>
</tr>
<tr>
<td><strong>S_o = 0.05</strong> Max. discharge, $Q_{max}$ (m³/s)</td>
<td>12.00</td>
<td>11.6</td>
<td>11.7</td>
<td>11.3</td>
</tr>
<tr>
<td>Time to max. discharge, $t_{max}$ (min)</td>
<td>10.00</td>
<td>10.7</td>
<td>10.8</td>
<td>11.5</td>
</tr>
<tr>
<td>Dimensionless discharge, $Q_{max}/Q_p$</td>
<td>1.000</td>
<td>0.96</td>
<td>0.98</td>
<td>0.95</td>
</tr>
<tr>
<td>Dimensionless attenuation, $(Q_p-Q_{max})/Q_p$</td>
<td>0.000</td>
<td>0.04</td>
<td>0.02</td>
<td>0.05</td>
</tr>
<tr>
<td><strong>S_o = 0.005</strong> Max. discharge, $Q_{max}$ (m³/s)</td>
<td>12.00</td>
<td>10.9</td>
<td>11.2</td>
<td>10.1</td>
</tr>
<tr>
<td>Time to max. discharge, $t_{max}$ (min)</td>
<td>10.00</td>
<td>10.8</td>
<td>11.6</td>
<td>11.9</td>
</tr>
<tr>
<td>Dimensionless discharge, $Q_{max}/Q_p$</td>
<td>1.000</td>
<td>0.91</td>
<td>0.93</td>
<td>0.84</td>
</tr>
<tr>
<td>Dimensionless attenuation, $(Q_p-Q_{max})/Q_p$</td>
<td>0.000</td>
<td>0.09</td>
<td>0.07</td>
<td>0.16</td>
</tr>
<tr>
<td><strong>Q_{peak} = 10 m³/s</strong> Max. discharge, $Q_{max}$ (m³/s)</td>
<td>10.00</td>
<td>8.2</td>
<td>8.0</td>
<td>6.7</td>
</tr>
<tr>
<td>Time to max. discharge, $t_{max}$ (min)</td>
<td>10.00</td>
<td>11.0</td>
<td>11.6</td>
<td>12.0</td>
</tr>
<tr>
<td>Dimensionless discharge, $Q_{max}/Q_p$</td>
<td>1.000</td>
<td>0.68</td>
<td>0.80</td>
<td>0.56</td>
</tr>
<tr>
<td>Dimensionless attenuation, $(Q_p-Q_{max})/Q_p$</td>
<td>0.000</td>
<td>0.32</td>
<td>0.20</td>
<td>0.44</td>
</tr>
<tr>
<td><strong>Q_{peak} = 8 m³/s</strong> Max. discharge, $Q_{max}$ (m³/s)</td>
<td>8.00</td>
<td>6.7</td>
<td>6.5</td>
<td>5.6</td>
</tr>
<tr>
<td>Time to max. discharge, $t_{max}$ (min)</td>
<td>10.00</td>
<td>11.0</td>
<td>11.6</td>
<td>12.3</td>
</tr>
<tr>
<td>Dimensionless discharge, $Q_{max}/Q_p$</td>
<td>1.000</td>
<td>0.55</td>
<td>0.81</td>
<td>0.47</td>
</tr>
<tr>
<td>Dimensionless attenuation, $(Q_p-Q_{max})/Q_p$</td>
<td>0.000</td>
<td>0.45</td>
<td>0.19</td>
<td>0.52</td>
</tr>
<tr>
<td>Dimensionless translation, $(t_{p}-t_{max})/t_p$</td>
<td>0.0</td>
<td>0.10</td>
<td>0.16</td>
<td>0.23</td>
</tr>
</tbody>
</table>
According to Figures 5.24 and 5.25, the hydrograph for the simplified dynamic model increases more rapidly in the beginning than the other dynamic model. In turn, the recession hydrograph decreases less rapidly for the simplified dynamic model than that of the dynamic model. The dynamic wave model within the FLDWAV model arrives at a base flow more rapidly than the simplified dynamic model. For a trapezoidal channel, the main difference between the two models is the magnitude of the peak flow which is approximately 12%. For a triangular channel, the differences are in the magnitude of the peak flow (approximately 12%) and their respective timings (approximately 2 minute delay in peak flows for the dynamic wave model). For the simplified dynamic model the peak is smaller and the attenuation is less than the dynamic wave model. For a trapezoidal channel with respect to the overall shape of the hydrographs there are no significant differences between the two models. However, for a triangular channel, the differences in the time to peaks for the outflow hydrographs further intensify the differences in the discharge values between the two models. However, the simplified dynamic model is dependent upon more simple governing equations and is solved by using an explicit finite different scheme, rather than the more computationally complex “full” dynamic wave equations and implicit solver found within the FLDWAV model. Furthermore, for the different bed slopes ($S_0 = 0.05$ and $0.005$), the dynamic wave model within FLDWAV failed to converge. Therefore, the simplified dynamic model can be applied where the complex dynamic wave model may not be appropriate.

The diffusion wave approximation to the Saint Venant equations is also used for channel routing applications due to its simplicity and ease of solution. The diffusion wave subroutine within the FLDWAV model was used to simulate the outflow hydrograph for the diffusion wave approximation. The diffusion wave subroutine within the FLDWAV model uses an implicit solver. As an initial starting point, the outflow hydrograph for the diffusion wave was compared to the outflow hydrograph for the dynamic wave at 2000 m for both the trapezoidal channel and the triangular channel; the results are illustrated in Figures 5.26 and 5.27. The same triangular inflow hydrograph was used as in the previous analyses. According to Figures 5.26 and 5.27, there is little difference between the two outflow hydrographs for both channel sections, suggesting that for the present example, the local and convective acceleration terms in the Saint Venant equations are negligible in comparison to the frictional force and pressure force terms.
As a result, the same conclusions that were drawn from comparing the simplified dynamic model with the dynamic wave model can be applied here. A comparison of the outflow hydrographs between the simplified dynamic model and the diffusion wave model are illustrated in Figures 5.28 and 5.29 for a trapezoidal channel and a triangular channel respectively. Furthermore, similar to the dynamic wave model, the diffusion wave model failed to converge for $S_o = 0.05$, therefore, the simplified dynamic model can be applied where the diffusion wave model may not be appropriate.

Three different methods were tested to illustrate the effect of spatial grid size on model behaviour. Three levels of spatial interval were tested with constant time interval equal to 10 s. The results of are shown in Table 5.26 for a trapezoidal channel and Table 5.27 for a triangular channel, expressed in term of peak flow and time to peak at $x = 2000$ m. As expected there was little variation in the results with respect to grid size for the diffusion wave model, simplified dynamic model and dynamic wave model. In addition, the results indicated that the convergence properties of the simplified dynamic model were just as good as the diffusion wave and dynamic wave models, which utilize an implicit numerical solution for solving the Saint Venant equations.

In order to show the effect of peak flows of triangular inflow hydrographs on runoff hydrographs, the peak flows were selected as 8 and 10 m$^3$/s instead of 12 m$^3$/s. The peak discharges of the runoff hydrograph were calculated at $x = 2000$ m by using the simplified dynamic model (SDM), with $\Delta x = 100$ m, $\Delta t = 10$ s as 6.5, 5.6 and 4.8 m$^3$/s for inflow peak flows 12, 10 and 8 m$^3$/s, respectively for the trapezoidal channel. The dimensionless attenuations, 0.46, 0.44 and 0.40 were found for the mentioned inflow peaks for a trapezoidal channel, respectively. For a triangular channel, the peak flows of the runoff hydrographs were calculated at $x = 2000$ m by using the simplified dynamic model, with $\Delta x = 100$ m, $\Delta t = 10$ s as 6.3, 5.5 and 4.8 m$^3$/s for inflow peak flows 12, 10 and 8 m$^3$/s, respectively. The dimensionless attenuations, 0.48, 0.45 and 0.40 were found for the mentioned inflow peaks for a triangular channel, respectively. However, for both a trapezoidal and triangular channel, the dimensionless translation was found to be consistent. Therefore, the inflow peak flows, and consequently the shape of the inflow
The runoff hydrographs for triangular and trapezoidal inflow hydrographs are also compared. For a trapezoidal channel, the inflow triangular hydrograph yields a peak runoff of 6.5 m$^3$/s, its corresponding dimensionless attenuation is 0.46 and its dimensionless translation is 1.02. The inflow trapezoidal hydrograph yields a peak discharge of 7.7 m$^3$/s its corresponding dimensionless attenuation is 0.36 and its dimensionless translation is 1.10 for the same channel and data as $S_o= 0.0005$, $Δx= 100$ m, $Δt= 10$ s at $x= 2000$ m. For a triangular channel, the inflow triangular hydrograph yields a peak runoff of 6.3 m$^3$/s, its corresponding dimensionless attenuation is 0.47 and its dimensionless translation is 0.6. The inflow trapezoidal hydrograph yields a peak discharge of 7.6 m$^3$/s its corresponding dimensionless attenuation is 0.37 and its dimensionless translation is 0.82 for the same channel and data. Based on the above results the form of the inflow hydrograph affects the attenuation and translation of the outflow hydrograph. Therefore, a trapezoidal inflow hydrograph will result in a smaller attenuation and a longer translation than a triangular inflow hydrograph when applying the simplified dynamic model.

Figure 0-24 Comparison of outflow hydrographs for the simplified dynamic and dynamic wave models at distance of 2000 m at triangular inflow hydrograph for a trapezoidal channel
Figure 0-25 Comparison of outflow hydrographs for the simplified dynamic model and dynamic wave model at distance of 2000 m at triangular inflow hydrograph for a triangular channel.

Figure 0-26 Comparison of outflow hydrographs for diffusion wave model and dynamic wave model at distance of 2000 m at triangular inflow hydrograph for a trapezoidal channel.
Figure 0-27 Comparison of outflow hydrographs for diffusion wave model and dynamic wave model at distance of 2000 m at triangular inflow hydrograph for a triangular channel

Figure 0-28 Comparison of outflow hydrographs for diffusion wave model and simplified dynamic model at distance of 2000 m at triangular inflow hydrograph for a trapezoidal channel
Figure 0-29 Comparison of outflow hydrographs for diffusion wave model and simplified dynamic model at distance of 2000 m at triangular inflow hydrograph for a triangular channel

Table 0-26 Calculated peak flows and time to peak by three methods for a trapezoidal channel and different $\Delta x$ at $x=2000$ m

<table>
<thead>
<tr>
<th>Model</th>
<th>$\Delta x=100$ m</th>
<th>$\Delta x=200$ m</th>
<th>$\Delta x=400$ m</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Peak flow (m$^3$/s)</td>
<td>Peak time (min)</td>
<td>Peak flow (m$^3$/s)</td>
</tr>
<tr>
<td>Diffusion Wave Model</td>
<td>7.0</td>
<td>22</td>
<td>7.0</td>
</tr>
<tr>
<td>Dynamic Wave Model</td>
<td>7.3</td>
<td>20</td>
<td>7.4</td>
</tr>
<tr>
<td>Simplified Dynamic Model</td>
<td>6.5</td>
<td>20</td>
<td>6.3</td>
</tr>
</tbody>
</table>

Table 0-27 Calculated peak flows and time to peak by three methods for a triangular channel and different $\Delta x$ at $x=2000$ m

<table>
<thead>
<tr>
<th>Model</th>
<th>$\Delta x=100$ m</th>
<th>$\Delta x=200$ m</th>
<th>$\Delta x=400$ m</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Peak flow (m$^3$/s)</td>
<td>Peak time (min)</td>
<td>Peak flow (m$^3$/s)</td>
</tr>
<tr>
<td>Diffusion Wave Model</td>
<td>6.8</td>
<td>20</td>
<td>6.8</td>
</tr>
<tr>
<td>Dynamic Wave Model</td>
<td>7.2</td>
<td>18</td>
<td>7.2</td>
</tr>
<tr>
<td>Simplified Dynamic Model</td>
<td>6.3</td>
<td>16</td>
<td>6.3</td>
</tr>
</tbody>
</table>
5.3.2 Application of the Simplified Dynamic Model to the Credit River Watershed

For the reach between Melville and Cataract, the diffusion wave, dynamic wave and simplified dynamic wave models were used to simulate the time series flows at Cataract for the calibration event May 12\textsuperscript{th}, 2000 and the validation event August 19\textsuperscript{th}, 2005. Both of these events were significant in that they caused minor and/or nuisance flooding in upper Credit River watershed where the events were concentrated. Through the calibration procedure the Manning’s ‘n’ value for all three models was determined to be 0.04 for the channel portion and 0.08 for the floodplain portion. A single-valued Manning’s roughness coefficient is an unlikely case, but feasible, given that the surface roughness along the channel and floodplain for this reach is relatively uniform. However, Manning’s “n” does vary with discharge and this is not reflected in any of the models or modelling results. Appendix E, illustrates plots of velocity versus Manning’s “n” coefficient. This is a limitation in both the FLDWAV model (Fread and Lewis, 1998) and within the simplified dynamic model. In the FLDWAV model under high flow events the propagation speed (c) of the flood wave is related to the uncertainty in the Manning “n” coefficient and as a result, Manning’s “n” coefficient directly affects rate of propagation of the flood wave. This phenomenon is more apparent in dam-break floods than in runoff events. In the FLDWAV model Manning’s “n” is defined for each channel reach and specified as a tabular piecewise linear function of stage and discharge, with linear interpolation used to obtain n values intermediate to the tabulated values. For the channel portion Manning’s “n” is usually kept constant and for floodplain flows a composite Manning’s “n” value is often used, representing both the channel and floodplain. In the simplified dynamic model, separate Manning’s “n” values are specified for the channel, the left and right floodplains. Physically, however, as the flow increases and more portions of the bank and overbank become inundated, the vegetation located at these elevations causes an increase to the resistance to flow (Maidment, 1993). In addition, the n value may be larger for small floodplain depths than for large floodplain depths due to the flattening of thick brush, weeds and tall grasses (Maidment, 1993). This effect may be reversed for wooded overbank areas where the leaves on fallen trees and branches can also impede the flow in addition to the tree trunks. Furthermore, n may also decrease with increasing discharge when the flow area in the banks is greater than the flow area in the overbank areas,
such as the case in wide channels with levees on either side for flood protection, or when the flood is confined to the channel.

Both models performed equally well for each of the simulation periods with negligible differences in the results. This is expected since under typical river routing scenarios (i.e. snowmelt or rainfall runoff events) dynamic terms are small in comparison to friction effects. Fread (1993) uses a simple criterion \( E \) error term expressed in percent, which represents the energy slope ratio of diffusion-type models to the dynamic wave model.

\[
E' = \frac{\mu' \varphi' q_p^3}{T_s S_0^{0.8} n^{0.6}} \tag{5.20}
\]

Where

\[
\varphi' = \frac{m+3}{3m+5} \tag{5.21}
\]

The \( \mu'' \) is the unit conversion factor equal to 0.0091; \( T_s \) is the time of rise (hours), in the inflow hydrograph; \( S_0 \) is the channel bottom slope (m/m); \( n \) is Manning’s coefficient; \( q_p \) is the unit width peak discharge (\( m^3/s \)); and \( m \) is the cross-sectional shape factor used to describe the channel top-width \( B \), as \( B=ky^m \), where \( k \) is a constant, \( y \) is the depth of flow, \( m=0 \) for rectangular, \( m=0.5 \) for parabolic and \( m=1.0 \) for triangular channels. From equation 5.20, it can be seen that in cases where there is a gently sloping channel and a rapidly rising hydrograph equation 5.20 cannot be satisfied and therefore, the complete Saint Venant equations are required. In the implementation of the diffusion wave, dynamic wave and simplified dynamic wave models for the reach between Melville and Cataract, given the negligible acceleration effects present for the two events, the acceleration terms in the dynamic wave model and simplified dynamic wave model would have been negligible in magnitude. Therefore, for the above reach and flow routing scenarios, this amounts to a diffusion wave model implementation.

Figure 5.30, compares the outflows hydrographs for the three different models with the observed hydrograph for the Cataract gauge. Table 5.28 lists the statistical results for the validation event
for each model. According to Table 5.28, the diffusion wave model and the dynamic wave model generated the same statistical results. This is expected since both models generated the same outflow hydrographs, and the acceleration terms in the dynamic wave model are negligible; therefore, the dynamic wave model simply becomes the diffusion wave model. According to the statistical results in Table 5.28, there was good agreement between observed and simulated outflow hydrographs of the diffusion wave model and simplified dynamic wave model. Although, the diffusion wave models’ statistical results indicate a better agreement between the observed and simulated outflow hydrographs ($r^2 = 0.87$ versus $r^2 = 0.80$) than the simplified dynamic model. However, for the simplified dynamic wave model, the results of the other statistical criteria including the coefficient of efficiency ($E$) and the modified coefficient of efficiency ($E_1$); the root mean square error ($RMSE$), which is less than half the standard deviation of the measured outflow (0.87); the percent difference in volumes; and the $RBIAS$; matched the statistical criteria for the diffusion wave model. Furthermore, the simplified dynamic model is much easier to formulate than the diffusion wave model and especially the dynamic wave model, since it depends on simpler governing equations and uses a less complex finite difference numerical solution. It should also be noted that the differences in the discharge values between the observed and simulated flow data could be attributed to errors within the observed data. For example, a review of Figure 5.30 indicates that the observed flows during the August 19th event follow a distinct prismatic pattern of highs and lows, although practical, this is physically not possible. Water Survey of Canada measures stage not discharge, to formulate their rating curves for open water; the stage is then related to a discharge value on their measure rating curve. As a result, the discharge values which Water Survey of Canada published are estimated and not measured directly. In addition, water levels for the stream gauges along the Credit River are automatically measured every 15 minutes using a Sutron data logger (Water Survey of Canada, 2012). In turn, if the water level is rising faster than the data is being collected due to an intense runoff event, such as in the case of the August 19th, 2005 storm event, much of the stage or water level data will be lost, resulting in an observed hydrograph as depicted in Figure 5.30. A comparison was undertaken between the observed outflow hydrograph and the outflow hydrographs for the simplified dynamic model with transmission losses and without transmission losses; the results are illustrated in Figure 5.31. According to Figure 5.31 the differences in discharge values between the two modelling scenarios are negligible. Evaporation losses are not
likely to be of paramount importance in a flood routing application given the time scales involved and are often neglected. Similarly, under the same application, seepage losses or recharge from bank storage would be similarly irrelevant in routing surface runoff, since they are considerably less efficient hydraulically and therefore, would be expected to involve significantly lower flow rates. Therefore, there impact on flood flows would be negligible.

For the reach between Boston Mills and Norval, the diffusion wave, dynamic wave and simplified dynamic wave models were used to simulate the time series flows at Norval for the same calibration (May 12th, 2000) and validation (August 19th, 2005) events were used as in the in the previous reach in the upper Credit River. Through the calibration procedure the Manning’s ‘n’ value for all three models was determined to be 0.033 for the channel portion and 0.100 for the floodplain portion. The channel and floodplain surface roughness along this reach is relatively uniform. Figure 5.32, compares the outflows hydrographs for the three different models with the observed hydrograph for the Norval gauge. Table 5.29 lists the statistical results for the validation event for each model. According to Figure 5.32, as expected, there is little difference between the outflow hydrographs for the diffusion wave model and the dynamic wave model. However, according to Table 5.29, the dynamic wave model performed better than the diffusion wave model, with fair to good agreement between the observed and simulated outflow hydrographs for the dynamic wave model and fair agreement for the diffusion wave model. According to Table 5.29, there was also good agreement between observed and simulated outflow hydrographs for the simplified dynamic wave model, whose performance was better than the dynamic wave and diffusion wave models. This is expected, since the reach between Boston Mills and Norval is characterized as a cascade type, a sequence of discrete channel segments, with quick discharge characteristics, and the simplified dynamic model is referred to as a dynamic cascade. In some instances, the flow along this reach can become supercritical. The numerical solution used in the simplified dynamic model is a simple one of a cascade type. This type of model becomes particularly useful if the flow is supercritical or when the effect of the secondary boundary condition is negligible at the downstream end of the channel for subcritical flows. A comparison was undertaken between the observed outflow hydrograph and the outflow hydrographs for the simplified dynamic model with transmission losses and without transmission
losses; the results are illustrated in Figure 5.33. Similar to the reach in the upper Credit River the differences in the discharge values between the two modelling scenarios are negligible.

The simulated results and the statistical analysis indicate that the results of the simplified dynamic model are the same as the results of the dynamic wave and diffusion wave models. In addition, the simplified dynamic model could be applied in some reaches where the dynamic wave and diffusion wave models may not be appropriate. A comparison of outflow hydrographs at different distances shows that the attenuation increases with an increase in the computational distance step \((\Delta x)\) and with an increase in inverse channel side slope \((z)\); and translation increases with distance, but decreases with an increase in the computational time step \((\Delta t)\). The simplified dynamic model is compared with the dynamic wave model. Findings indicated that both models yield approximately the same characteristics of outflow hydrographs under the same conditions; however, the shape of the channel can affect the characteristics of the outflow hydrograph of the simplified dynamic model. The simplified dynamic model is much easier to formulate than the dynamic wave model and is simple to calculate in comparison to the other one. The simplified dynamic model was also compared to the diffusion wave model. The results show that the simplified dynamic model generates a smaller peak flow and time-to-peak flow than the diffusion wave model. The attenuation obtained by the simplified dynamic model depends on the characteristics of the inflow hydrograph including shape and peak discharge, while the attenuation obtained using the diffusion wave or dynamic wave models is not dependent upon the shape or peak flow of the inflow hydrograph. This indicates that the simplified dynamic model can be applied to cases or routing problems where neither the diffusion wave nor dynamic wave models are appropriate.
Figure 0-30 Comparison of outflow hydrographs for the diffusion wave, dynamic wave and simplified dynamic wave models and the observed hydrograph at the Cataract stream gauge for the August 19th, 2005 event

Table 0-28 Statistical quantities of coefficient of determination ($r^2$), coefficient of efficiency (E), modified coefficient of efficiency ($E_1$), root mean square error (RMSE), systematic root mean square error (RMSE$_S$), unsystematic root mean square error (RMSE$_U$), coefficient of residual mass (CRM), percent difference in volumes (%VOL) and relative bias (RBIAS) for the August 19$^{th}$, 2005 event for the three (3) different routing models for reach between Melville and Cataract

<table>
<thead>
<tr>
<th>Routing Models</th>
<th>$r^2$</th>
<th>E</th>
<th>$E_1$</th>
<th>RMSE</th>
<th>RMSE$_S$</th>
<th>RMSE$_U$</th>
<th>CRM</th>
<th>VOL.%</th>
<th>RBIAS</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dynamic Wave</td>
<td>0.87</td>
<td>0.76</td>
<td>0.63</td>
<td>0.9</td>
<td>0.5</td>
<td>0.8</td>
<td>0.1</td>
<td>12</td>
<td>8.0</td>
</tr>
<tr>
<td>Diffusion Wave</td>
<td>0.87</td>
<td>0.76</td>
<td>0.63</td>
<td>0.8</td>
<td>0.5</td>
<td>0.8</td>
<td>-1.0</td>
<td>12</td>
<td>8.0</td>
</tr>
<tr>
<td>Simplified Dynamic Wave</td>
<td>0.80</td>
<td>0.66</td>
<td>0.68</td>
<td>1.0</td>
<td>0.4</td>
<td>1.0</td>
<td>-1.0</td>
<td>13</td>
<td>12.8</td>
</tr>
</tbody>
</table>
Figure 0-31 Comparison of outflow hydrographs for the simplified dynamic wave model with and without losses and the observed hydrograph at the Cataract stream gauge for the August 19th, 2005 event

Figure 0-32 Comparison of outflow hydrographs for the diffusion wave, dynamic wave and simplified dynamic wave models and the observed hydrograph at the Norval stream gauge for the August 19th, 2005 event
Table 0-29 Statistical quantities of coefficient of determination ($r^2$), coefficient of efficiency (E), modified coefficient of efficiency ($E_1$), root mean square error (RMSE), systematic root mean square error (RMSE_S), unsystematic root mean square error (RMSE_U), coefficient of residual mass (CRM), percent difference in volumes (%VOL) and relative bias (RBIAS) for the August 19th, 2005 event for the three (3) different routing models for reach between Boston Mills and Norval

<table>
<thead>
<tr>
<th>Routing Models</th>
<th>$r^2$</th>
<th>E</th>
<th>$E_1$</th>
<th>RMSE</th>
<th>RMSE_S</th>
<th>RMSE_U</th>
<th>CRM</th>
<th>VOL.%</th>
<th>RBIAS</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dynamic Wave</td>
<td>0.74</td>
<td>0.65</td>
<td>0.78</td>
<td>3.2</td>
<td>1.4</td>
<td>2.8</td>
<td>-0.1</td>
<td>-6.4</td>
<td>-4.7</td>
</tr>
<tr>
<td>Diffusion Wave</td>
<td>0.66</td>
<td>0.50</td>
<td>0.71</td>
<td>3.6</td>
<td>1.8</td>
<td>3.1</td>
<td>-1.0</td>
<td>-6.8</td>
<td>-3.5</td>
</tr>
<tr>
<td>Simplified Dynamic Wave</td>
<td>0.80</td>
<td>0.71</td>
<td>0.82</td>
<td>3.0</td>
<td>1.1</td>
<td>2.7</td>
<td>-1.0</td>
<td>-6.4</td>
<td>-5.4</td>
</tr>
</tbody>
</table>

Figure 0-33 Comparison of outflow hydrographs for the simplified dynamic wave model with and without losses and the observed hydrograph at the Norval stream gauge for the August 19th, 2005 event

5.4 Application of GFLOOD Model to the Welland River Watershed

5.4.1 Calibration Results of the GFLOOD Model for the Welland River Watershed

Two rainfall events were used for the calibration of the GFLOOD model at the Caistors Corners gauge station, the April 20-24, 2000 event and the May 13-17, 2002 event. The April 20-24, 2000 event represents a snowmelt plus rainfall high flow event and the May 13-17, 2002 event represents a frontal rainfall induced high flow event. Both events were found to be suitable for calibration both in terms of their magnitude and spatial extent. Both events produced unimodal
flow hydrographs at the Caistors Corners gauge station. In addition, the May 13-17, 2002 event and the May 1-6, 2003 event were used to calibrate the GFLOOD model at the Old Syphon gauge station. The May 1-6, 2003 event also represents a frontal rainfall-runoff induced high flow event. This event was found to be suitable in terms of model calibration both in terms of its magnitude and spatial extent. The performance of the GFLOOD model for each of the calibration events was evaluated at two locations Caistors Corners and the Old Syphon. The locations were selected to represent different physiographic sub-regions of the Welland River watershed, as well as to reflect the different sub-basin areas and streamflow regimes. Figures 5.34 and 5.35 compare the observed and simulated flow hydrographs at the Caistors Corners gauge station. According to the figures, in both cases the event model fits the observed hydrographs very well. The peak flows for both events have also been captured by the event model with a high degree of accuracy. The rising parts of the hydrographs are generally modelled better than the recession limbs of the hydrographs; this can be attributed to the limited ability of the event model to simulate longer dry weather periods. Figures 5.36 through 5.39 represent the scatter graphs and residual graphs for the calibration events at the Caistors Corners gauge station.

Figures 5.40 and 5.41 compare the observed and simulated water level hyetographs at the Old Siphon gauge station. According to the figures, in both cases the event model fits the observed hyetographs very well. The fluctuating water levels upstream of the Old Siphon have been captured by the event model for both events with a high degree of accuracy. During the May 1-6, 2003 event as fluctuating water levels begin to rise (40-60 hour); the event model over estimates the water levels upstream of the Old Syphon. However, the differences between the observed and simulated water levels during this event are less than 5 cm. This was also observed during the calibration of the May 13-17, 2002 event, approximately 40-60 hours after the event has begun; the event model begins to overestimate the water levels upstream of the Old Siphon. However, similar to the previous event the differences between the observed and simulated water levels are less than 5 cm. These small differences between the observed and simulated water level data can be attributed to turbulence within the observed data, and the event models limited ability to simulate instantaneous changes in water levels. Figures 5.42 through 5.45 represent the scatter graphs and residual graphs for the calibration events at the Old Siphon gauge station.
Figure 0-34 Observed and simulated hydrographs for the April 20-24, 2000, event at Caistors Corners

Figure 0-35 Observed and simulated hydrographs for the May 13-17, 2002, event at Caistors Corners
Figure 0-36 Scatter graph for the April 20-24, 2000, event at Caistors Corners

Figure 0-37 Residual graph for the April 20-24, 2000, event at Caistors Corners
Figure 0-38 Scatter graph for the May 13-17, 2002, event at Caistors Corners

Figure 0-39 Residual graph for the May 13-17, 2002, event at Caistors Corners
Figure 0-40 Observed and simulated hyetographs for the May 1-6, 2003, event at the Old Siphon.

Figure 0-41 Observed and simulated hyetographs for the May 13-17, 2002, event at the Old Siphon.
Figure 0-42 Scatter graph for the May 1-6, 2003, event at the Old Siphon

Figure 0-43 Residual graph for the May 1-6, 2003, event at the Old Siphon
Tables 5.30 and 5.31 compare the statistical performance measures obtained for each location. The coefficient of determination, $R^2$, is in all locations close to unity or greater than or equal to 0.95. The value of this measure at the Old Siphon for the May 13-17, 2002 event is a bit low (0.85) in comparison to the other events, however, it is still acceptably high. This confirms the event model’s limited ability in capturing instantaneous changes in water levels during this period, between the 40th and 60th hour after the event had begun. This is demonstrated in Figure 5.45, which shows the residual values as a function of time. The maximum deviation between

Figure 0-44 Scatter graph for the May 13-17, 2002, event at the Old Siphon

Figure 0-45 Residual graph for the May 13-17, 2002, event at the Old Siphon
observed and modelled water levels during this event is approximately 12 cm. However, despite
this deviation between observed and modelled water levels, the residual values are still
acceptably low. The efficiency coefficient $E$ and the modified efficiency $E_i$ are all acceptably
close to unity at both locations with the exception of the May 13-17, 2002 event at the Old
Siphon. Both coefficients were determined to be relatively low 0.51 and 0.37 respectively. This
can be attributed to the models sensitivity to instantaneous changes in water levels. A review of
the RMSE, including systematic and unsystematic reveals that the values for these statistical
parameters are relatively low at the Old Siphon for the May 1-6, 2003 event and May 13-17,
2003 event. For the May 13-17, 2003 event the RMSE including systematic and unsystematic
was found to equal zero. For the May 1-6, 2003 event RMSE including systematic and
unsystematic was found to be less than 0.04. For the Caistors Corners gauge station the root
mean square error for the April 20-24, 2000 event was found to be 2.66, 2.92 for the systematic
RMSE and 0.87 for the unsystematic RMSE. All three values are considered to be relatively low.

For the May 13-17, 2002 event the RMSE was found to be 5.16, 8.30 for the systematic RMSE
and 1.26 for the unsystematic RMSE. The higher values associated with this event can be
attributed to the event models limited ability in predicting low flows. The coefficient of residual
mass ($CRM$) was found in both locations to be relatively low for all events, in most cases equal
to zero. The relative bias, $RBIAS$, at the Old Siphon gauge station was determined to be very low,
0.009% for the May 1-6, 2003 event and 0.015% for the May 13-17, 2002 event. For the Caistors
Corners gauge station the relative bias, $RBIAS$, was determined to be 25% for the April 20-24,
2000 event and 52% for the May 13-17, 2002 event. The rather high values of the $RBIAS$
parameter at the Caistors Corners gauge is caused by the limited ability of the event model to
simulate low flows preceding and succeeding the flood event. This is demonstrated in Figure
5.46, which shows the RBIAS measure for the May 13-17, 2002 event at Caistors Corners as a
function of time. The RBIAS during the peak hydrograph is actually within the range of (+/-)
40%. It is the period of low flows (before and after the peak), where the model systematically
underestimates the observed streamflow. High relative errors in low flows consequently lead to
high values of relative performance measures. Figure 5.47 depicts the errors from Figure 5.46 in
absolute terms. The absolute $BIAS$ in the period of low flows is negligible, within the range of
(+/-) 1.6 m$^3$/s. The maximum $BIAS$ occurred during the start of the rising limb of the hydrograph
(up to (+) 6.5 m$^3$/s). The percent error in peak (%peak) and the percent error in time-to-peak (%Tp) values were also found to be acceptably low at both locations, in some cases both values were equal to zero. The percent difference in runoff volumes (%Vol) was also found to be low at the Caistors Corners gauge station, on average 3.0% for both calibration events.

Table 0-30 Statistical performance measures for the Caistors Corners gauge station

<table>
<thead>
<tr>
<th>Event</th>
<th>$R^2$</th>
<th>E</th>
<th>$E_1$</th>
<th>RMSE</th>
<th>RMSE$_S$</th>
<th>RMSE$_U$</th>
<th>CRM</th>
<th>RBIAS (%)</th>
<th>%Peak</th>
<th>%TP</th>
<th>%Vol</th>
</tr>
</thead>
<tbody>
<tr>
<td>April 20-24, 2000</td>
<td>0.97</td>
<td>0.95</td>
<td>0.85</td>
<td>2.66</td>
<td>2.92</td>
<td>0.87</td>
<td>0.03</td>
<td>25.7</td>
<td>-0.03</td>
<td>0.00</td>
<td>3.39</td>
</tr>
<tr>
<td>May 13-17, 2002</td>
<td>0.98</td>
<td>0.95</td>
<td>0.81</td>
<td>5.16</td>
<td>8.30</td>
<td>1.26</td>
<td>-0.03</td>
<td>52.7</td>
<td>-1.64</td>
<td>0.00</td>
<td>-2.92</td>
</tr>
</tbody>
</table>

Table 0-31 Statistical performance measures for the Old Syphon gauge station

<table>
<thead>
<tr>
<th>Event</th>
<th>$R^2$</th>
<th>E</th>
<th>$E_1$</th>
<th>RMSE</th>
<th>RMSE$_S$</th>
<th>RMSE$_U$</th>
<th>CRM</th>
<th>RBIAS (%)</th>
<th>%Peak</th>
<th>%TP</th>
</tr>
</thead>
<tbody>
<tr>
<td>May 1-6, 2003</td>
<td>0.95</td>
<td>0.87</td>
<td>0.66</td>
<td>0.03</td>
<td>0.04</td>
<td>0.01</td>
<td>0.00</td>
<td>0.000</td>
<td>-0.03</td>
<td>0.00</td>
</tr>
<tr>
<td>May 13-17, 2002</td>
<td>0.85</td>
<td>0.51</td>
<td>0.37</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.015</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
</tbody>
</table>

Figure 0-46 RBIAS of the model results for the May 13-17, 2002 event at Caistors Corners
The sensitivity of relative performance measures on the parts of the hydrograph which are less important in the event modelling is a feature that has to be accounted for in the overall evaluation of the event model. On the other hand, the use of relative measures is the only approach for comparing model performance at a station given different streamflow magnitudes.

5.4.2 Validation Results of the GFLOOD Model for the Welland River Watershed

The November 27- December 1, 2003 event and the May 23-27, 2004 event were chosen for the validation of the GFLOOD model at Caistors Corners. The September 27-October 1, 2003 event and the May 23-27, 2004 event were chosen for the validation of the GFLOOD model at the Old Siphon. All three (3) events had a fairly large spatial coverage across the watershed. And since all three events represent frontal rainfall types of events, with the exception of the November 27-December 1st, 2003 event which represents a rainfall plus snowmelt event, all three events are suitable for validation using the parameters established in the calibration exercise. The Manning’s “n” values in the lower watershed had to be adjusted to account for weed growth which is typical for the time periods of the following events: May 23-27, 2004 event and the September 27 – October 1, 2003 event. Figures 5.48 and 5.49 show the flow comparison graphs for the selected events at the Caistors Corners gauge station. Figures 5.50 and 5.51 show the water level comparison graphs for the selected events at the Old Siphon gauge station.
Figure 5.48 depicts the modelled and observed hydrographs generated by the November 27 – December 1, 2003 rainfall plus snowmelt event. The overall fit of the model is very well; only in the rising part of the hydrograph and the recession limb of the hydrograph there is some deviation between the observed and modelled flows. Figure 5.49 depicts the modelled and observed hydrographs generated by the May 23-27, 2004 rainfall event. The overall fit of the model is very well; only in the rising part of the hydrograph and the recession limb of the hydrograph there is deviation between the observed and modelled flows. There are some irregularities in the overall shape of the observed hydrograph; these irregularities are often attributed to basins response to smaller events preceding the larger event. In addition, on the recession limb of the hydrograph, modelled hydrograph recession limb tends to fall more quickly than the observed hydrograph recession limb. The recession limb of a hydrograph represents the withdrawal of water from the storage built up in the basin during the earlier phases of the hydrograph. Within the GFLOOD model a constant surface-storage recession constant \((K_{surf})\) is specified for each sub-catchment or sub-basin. However, for this specific event, the release of water from storage is much slower for the observed hydrograph than for the simulated hydrograph, indicating that the \(K_{surf}\) value for the sub-basins may higher than the value specified in the model. Figure 5.50 compares the modelled and observed water level hyetographs for the same rainfall event at the Old Siphon. According to Figure 5.50 the model fits very well with the observed water level hyetograph. There is some deviation in the peak water level, however, this is considered to be minor less than 15 cm. Figure 5.51 compares the modelled and observed water level hyetographs for the September 27 – October 1, 2003 rainfall event. According to Figure 5.51 the model fits well with the observed water level hyetograph. Tables 5.32 and 5.33 compare the statistical performance measures at each selected location for their respective events.

According to Tables 5.32 and 5.33 the GFLOOD model performed exceptionally well for all validation events at both locations and in some cases performed better for the validation events then the calibration events. Values for the coefficient of determination, \(R^2\) and the coefficient of efficiency were in the mid to high 90’s at the Caistors Corners gauge station. Values for the coefficient of determination, \(R^2\) at the Old Siphon were also in the high 90’s and high 80’s to low 90’s for the coefficient of efficiency. Values for the modified coefficient of efficiency ranged
between 0.82 and 0.76 at Caistors Corners and 0.71 and 0.77 at the Old Siphon. The lower values at the Old Siphon can be attributed to the event models sensitivity to simulating instantaneous changes in water levels upstream of the Old Siphon. Values for the root mean square error (RMSE) including systematic and unsystematic, were all relatively low at both locations. The percent error in peak (\%peak) and the percent error in time-to-peak (\%T_p) values were relatively low at both locations. The coefficients of residual mass (CRM) values were also found to be very low at both locations. The percent error in volume at the Caistors Corners gauge station was found to be a higher (13.4\%) for the November 27-December 1, 2003 event when compared to other events. However, this again can be attributed to the event models sensitivity to modelling low flows before and after the peak.

Figure 0-48 Observed and simulated hydrographs for the November 27 – December 1, 2003, event at Caistors Corners
Figure 0-49 Observed and simulated hydrographs for the May 23-27, 2004, event at Caistors Corners

Figure 0-50 Observed and simulated hyetographs for the May 23-27, 2004, event at the Old Siphon
Figure 0-51 Observed and simulated hyetographs for the September 27 – October 1, 2003, event at the Old Siphon

Table 0-32 Statistical performance measures for the Caistors Corners gauge station

<table>
<thead>
<tr>
<th>Event</th>
<th>$R^2$</th>
<th>E</th>
<th>$E_1$</th>
<th>RMSE</th>
<th>RMSE$_S$</th>
<th>RMSE$_U$</th>
<th>CRM</th>
<th>RBIAS (%)</th>
<th>%Peak</th>
<th>%TP</th>
<th>%Vol</th>
</tr>
</thead>
<tbody>
<tr>
<td>November 27 – December 1, 2003</td>
<td>0.98</td>
<td>0.95</td>
<td>0.82</td>
<td>2.48</td>
<td>3.66</td>
<td>1.57</td>
<td>-0.13</td>
<td>-16.5</td>
<td>1.50</td>
<td>3.03</td>
<td>-13.4</td>
</tr>
<tr>
<td>May 23-27, 2004</td>
<td>0.97</td>
<td>0.94</td>
<td>0.76</td>
<td>2.29</td>
<td>2.58</td>
<td>0.72</td>
<td>-0.07</td>
<td>-1.1</td>
<td>0.07</td>
<td>-6.25</td>
<td>-6.43</td>
</tr>
</tbody>
</table>

Table 0-33 Statistical performance measures for the Old Siphon gauge station

<table>
<thead>
<tr>
<th>Event</th>
<th>$R^2$</th>
<th>E</th>
<th>$E_1$</th>
<th>RMSE</th>
<th>RMSE$_S$</th>
<th>RMSE$_U$</th>
<th>CRM</th>
<th>RBIAS (%)</th>
<th>%Peak</th>
<th>%TP</th>
<th>%Vol</th>
</tr>
</thead>
<tbody>
<tr>
<td>May 23-27, 2004</td>
<td>0.98</td>
<td>0.93</td>
<td>0.77</td>
<td>0.07</td>
<td>0.09</td>
<td>0.04</td>
<td>0.00</td>
<td>0.003</td>
<td>0.06</td>
<td>-6.25</td>
<td></td>
</tr>
<tr>
<td>September 27 – October 1, 2003</td>
<td>0.97</td>
<td>0.87</td>
<td>0.71</td>
<td>0.02</td>
<td>0.04</td>
<td>0.02</td>
<td>0.00</td>
<td>0.007</td>
<td>0.02</td>
<td>3.03</td>
<td></td>
</tr>
</tbody>
</table>

In total three (3) events were used to calibrate and three (3) events were used validate the GFLOOD model for the Welland River at two different locations, Welland River at Caistors Corners and Welland River at the Old Syphon. According to Tables 5.30 through 5.33, the
overall efficiency of the GFLOOD model was estimated to be approximately 97% on average for the Upper Welland River and 95% for the lower portion of the Welland River. The GFLOOD model for the lower portion of the Welland River Watershed was determined to have a peak ratio of 1.00 and a normalized peak error of zero (0). Therefore, based on the statistical results in Tables 5.30 through 5.33, the GFLOOD model provides an accurate estimation of both flows and water levels for the purposes of event simulation. According to Figures 5.48 through 5.51, many of the deviations between measured and simulated streamflows and/or water levels were observed during the low flow periods, rising limbs or recession limbs of the hydrographs and/or hyetographs. There is a 7% error associated with streamflow measurements, however, most of these discrepancies are mainly attributed to the lack of accurate overland flow information used in this study for the small tributaries along the main branch of the Welland River. Runoff volumes for overland flows were assumed to be uniform across the sub-basins, given the homogeneous nature of the watershed. However, this may not necessarily be the case. Therefore, once proper mathematical formulation and cost-effective numerical simulation of overland flow are achieved, the reach modelling efforts would most likely yield better results.

5.4.3 Sensitivity Analysis of the GFLOOD Model for the Welland River Watershed

There are three (3) parameters of the event model that were subject to the sensitivity analysis: surface storage recession constant, $K_{surf}$; percent imperviousness, %Imp.; and Manning’s “n” value. The surface storage recession constant accounts for the surface storage within the basin including interception and depression. The surface storage recession constant is an index of the temporary storage of precipitation excess in the basin as it drains to the outlet point. As the surface storage recession constant decreases, the peak flow increases and the flood hydrograph becomes sharper (the recession limb falls faster). As the surface storage recession constant decreases, the peak flow decreases and the hydrograph becomes flatter. Figure 5.52 compares the baseline hydrograph with the hydrographs generated by the two sensitivity scenarios (+/-) 25%. According to Figure 5.52 the highest relative differences between the generated hydrographs and the baseline hydrograph are at the peak and along the recession limbs of the hydrographs, the average absolute sensitivity coefficient was estimated to be 0.72. The difference in peak flows is approximately (+/-) 10% between the baseline hydrograph and the generated hydrograph. Figure
5.53 compares the baseline hyetograph at the Old Syphon gauge with the generated hyetographs by the two sensitivity scenarios (+/-) 25%. According to Figure 5.53, the highest relative differences between the generated hyetographs and the baseline hyetograph are along the recession limb of the hyetographs. On average, the relative difference between the baseline hyetograph and the generated hyetographs is approximately (+/-) 40 cm.

The second parameter is the directly connected impervious area. The initial value for impervious area was 1.00%, a (+/-) 25% change in impervious area will have no impact on flows and water levels. Figures 5.53 and 5.54 compare the baseline hydrographs and hyetographs respectively with the generated hydrographs and hyetographs of the two (2) sensitivity scenarios.

The third and final parameter is the Manning’s “n” value; the Manning’s “n” directly effects the lag in the hydrograph or hyetograph. As a result, it is expected that the recession limbs of the hydrographs or hyetographs in comparison to the baseline hydrograph or hyetograph will show highest relative differences. Figures 5.55 and 5.56 compare the baseline hydrographs and hyetographs respectively with the generated hydrographs and hyetographs of the two (2) sensitivity scenarios for the Manning’s “n” parameter. The average absolute sensitivity coefficient was estimated to be 0.38 at the Caistors Corners gauge station and the maximum sensitivity coefficient was estimated to be 0.75. On average, the relative difference between the baseline hyetograph and the generated hyetographs at the Old Syphon station is approximately (+/-) 20 cm.
Figure 0-52 Event model sensitivity on the surface storage coefficient ($K_{surf}$) at Caistors Corners

Figure 0-53 Event model sensitivity on the surface storage coefficient ($K_{surf}$) at the Old Syphon
Figure 0-54 Event model sensitivity on the percent imperviousness (%Imp) at Caistors Corners

Figure 0-55 Event model sensitivity on the percent imperviousness (%Imp) at the Old Syphon
Figure 0-56 Event model sensitivity on the Manning’s “n” value at Caistors Corners

Figure 0-57 Event model sensitivity on the Manning’s “n” value at the Old Syphon
The hydrographs generated according to the scenarios of the change in the model parameters were also compared with the reference – baseline hydrograph by means of the performance measures introduced in Section 4.1.2.3. The results are summarized in Figures 5.58 to 5.77.

Figure 5.58 depicts the values of the coefficient of determination $R^2$, between the sensitivity outputs and the reference baseline data at Caistors Corners. Very high values of $R^2=0.99$ were found for the surface storage and Manning’s “n” value for a +25% change in the parameter values. The lowest values of $R^2$ were obtained for the scenarios corresponding to the -25% change in the surface storage and Manning’s “n” values because these parameters generate the highest differences between the hydrographs. Figure 5.59 depicts the values of the coefficient of determination $R^2$, between the sensitivity outputs and the reference baseline data at the Old Syphon. Almost functional forms, with the $R^2$ values approaching 1.0 were found for the surface storage, Manning’s “n” and percent impervious parameter scenarios.

Figures 5.60 and 5.61 depict the values of the efficiency coefficient and modified efficiency coefficient, $E$ and $E_1$ respectively, between the sensitivity outputs and the reference baseline data at Caistors Corners. Almost functional forms, with the $E$ and $E_1$ values approaching 1.0 were found for the surface storage, Manning’s “n” and percent impervious parameter scenarios. Figures 5.62 and 5.63 depict the values of the efficiency coefficient and modified efficiency coefficient, $E$ and $E_1$ respectively, between the sensitivity outputs and the reference baseline data at the Old Syphon. Almost functional forms, with the $E$ and $E_1$ values approaching 1.0 were found for the Manning’s “n” and percent impervious parameter scenarios. The lowest value of $E$ and $E_1$ were obtained for the scenarios corresponding to the change in the surface storage because this parameter generates the highest difference between the hydrographs.

Figures 5.64 through 5.67 compare the root mean square error, systematic root mean square error, the unsystematic root mean square error and the coefficient of residual mass from the individual sensitivity scenarios. The results show an almost negligible change in the parameters. Similar results were also observed for the Old Syphon gauge station and are depicted in Figures 5.68 to 5.71.
Figures 5.72 and 5.73 compares the relative BIAS obtained from the individual sensitivity scenarios at the Caistors Corners gauge station and the Old Syphon gauge station respectively. The results for all three (3) parameters at the Caistors Corners gauge station were almost zero. At the Old Syphon gauge station the change in the surface storage was approximately -10%, and for the Manning’s “n” value this change was approximately (+/-) 5%. The lowest values of RBIAS were obtained for the percent imperviousness, less than (+/-) 2.5%, because this parameter modifies only the peak ordinates of the hydrograph.

Figures 5.74 and 5.75 compares the percentage error in peak flow, %peak, of the model results generated from the sensitivity scenarios of the change in the event model parameters at the Caistors Corners gauge station and the Old Syphon gauge station respectively. Moderate values of the %peak measure were obtained by changing the surface storage, percent imperviousness and Manning’s “n” values. The changes observed in these parameters from the baseline data was approximately (+/-) 20%.

Figure 5.76 compares the percentage error in time-to-peak, %Tp, of the model results generated from the sensitivity scenarios of the change in the event model parameters at the Caistors Corners gauge station. Figure 5.77 compares the percentage error in time-to-peak, %Tp, of the model results generated from the sensitivity scenarios of the change in the event model parameters at the Old Syphon gauge station. According to Figure 5.77 a (+/-) 25% change in the parameter values can lead to a 100% change in the output results.
Figure 0-58 Coefficient of determination, D or $R^2$; of the model results generated from the sensitivity scenarios of the change in the event model parameters at Caistors Corners.

Figure 0-59 Coefficient of determination, D or $R^2$; of the model results generated from the sensitivity scenarios of the change in the event model parameters at the Old Syphon.
Figure 0-60 Coefficient of efficiency $E_i$; of the model results generated from the sensitivity scenarios of the change in the event model parameters at Caistors Corners

Figure 0-61 Modified coefficient of efficiency $E_{i1}$; of the model results generated from the sensitivity scenarios of the change in the event model parameters at Caistors Corners
Figure 0-62 Coefficient of efficiency $E_1$ of the model results generated from the sensitivity scenarios of the change in the event model parameters at the Old Syphon.

Figure 0-63 Modified coefficient of efficiency $E_1$ of the model results generated from the sensitivity scenarios of the change in the event model parameters at the Old Syphon.
Figure 0-64 Root mean square error RMSE; of the model results generated from the sensitivity scenarios of the change in the event model parameters at Caistors Corners

Figure 0-65 Systematic root mean square error RMSE; of the model results generated from the sensitivity scenarios of the change in the event model parameters at Caistors Corners
Figure 0-66 Unsystematic root mean square error $\text{RMSE}_u$; of the model results generated from the sensitivity scenarios of the change in the event model parameters at Caistors Corners

Figure 0-67 Coefficient of residual mass $\text{CRM}$; of the model results generated from the sensitivity scenarios of the change in the event model parameters at Caistors Corners
Figure 0-68 Root mean square error RMSE; of the model results generated from the sensitivity scenarios of the change in the event model parameters at the Old Syphon

Figure 0-69 Systematic root mean square error RMSE of the model results generated from the sensitivity scenarios of the change in the event model parameters at the Old Syphon
Figure 0-70 Unsystematic root mean square error RMSE_u; of the model results generated from the sensitivity scenarios of the change in the event model parameters at the Old Syphon

Figure 0-71 Coefficient of residual mass CRM; of the model results generated from the sensitivity scenarios of the change in the event model parameters at the Old Syphon
Figure 0-72 Relative BIAS RBIAS; of the model results generated from the sensitivity scenarios of the change in the event model parameters at Caistors Corners

Figure 0-73 Relative BIAS RBIAS; of the model results generated from the sensitivity scenarios of the change in the event model parameters at the Old Syphon
Figure 0-74 Percent error in peak, %peak; of the model results generated from the sensitivity scenarios of the change in the event model parameters at Caistors Corners

Figure 0-75 Percent error in peak, %peak; of the model results generated from the sensitivity scenarios of the change in the event model parameters at the Old Syphon
Figure 0-76 Percent error in time-to-peak, %T<sub>p</sub>; of the model results generated from the sensitivity scenarios of the change in the event model parameters at Caistors Corners.

Figure 0-77 Percent error in time-to-peak, %T<sub>p</sub>; of the model results generated from the sensitivity scenarios of the change in the event model parameters at the Old Syphon.
An event hydrologic – hydraulic model is aimed primarily at reproducing flood magnitudes and water levels. With respect to flood magnitudes, the event model GFLOOD is calibrated on the data from the Welland River, both the surface storage coefficient and Manning’s “n” values play a critical role in the modelling of peak hydrographs. In terms of peak water levels, the event model was found to be most sensitive to the Manning’s “n” value. When a basin is gauged, then these parameters can be readily estimated through calibration. In the case a basin is ungauged, the values of these parameters need to be carefully chosen based on the information available in the study area.

Finally, it must be remembered that the presented results were obtained by a local sensitivity analysis. Therefore, the results reflect the given combination of model parameters. Different parameter combinations may generate different values of the performance measures used in the sensitivity analysis, although a significant change in the pattern of the sensitivity results is unlikely.

5.4.4 Sensitivity of Changing Water Levels at the Chippawa Grass Island Pool

Water levels in the Chippawa Grass Island Pool (CGIP) are regulated by the International Niagara Board of Control’s 1993 Directive. The directive requires that the Power Entities operate the International Control Works to ensure the maintenance of an operational long-term average CGIP level of 171.16 m (IGLD, 85). The accumulated deviation of the CGIP’s level from March 1, 1973 through February 28, 2011 was 0.30 metre-month (0.98 foot-months) above the long-term average elevation. The maximum permissible accumulated deviation is +/- 0.91 metre-months (3.00 foot-months); (International Niagara Control Board, Report No. 117, 2011).

A comparison of water levels between CGIP Material dock gauge and the Montrose gauge, determined water levels at Montrose were on average 0.36 m below the water levels at the CGIP Material Dock gauge. The long term average water level at Montrose gauge based on hourly water levels is 170.80 (IGLD, 85) or 170.77 (GSC). A correlation analysis of water levels between the Material Dock and the Montrose gauge for the year 2000 is illustrated in Figure
5.78. According to Figure 5.78 there is a direct correlation between the water levels at the Material Dock and the Montrose gauge.

In turn, a sensitivity analysis was carried out on the computed water levels upstream of Montrose due to changing water levels at the CGIP and Montrose. The maximum water level at Montrose was determined to be 171.24 m (GSC); the minimum water level at Montrose was determined to be 170.42 m (GSC) and the average long-term water level was determined to be 170.77 m (GSC). These water levels were determined based on the long-term average water level at the CGIP and the allowable tolerances as set out by the International Niagara Board of Control, minus the average headloss between the CGIP Material Dock gauge and the Montrose gauge.

The 100-year event was used to test the sensitivity of the computed water levels upstream of Montrose due to changing water levels at the Chippawa Grass Island Pool (CGIP). The 100-year event represents the standard project flood for the Welland River watershed and it also generates the highest runoff volume within the watershed. As a result, the 100-year event would be sensitive to changes occurring at the downstream boundary condition. A review of streamflow records for the Caistors Corners gauge station revealed that the 100-year peak flow is 110 m$^3$/s (Table 4.20). According to streamflow records the maximum daily peak on March 21st, 1959 was 95 m$^3$/s; the maximum instantaneous peak was estimated to be 104 m$^3$/s using equation 4.146; in turn, this value is 5% less than the predicted 100-year peak flow at Caistors Corners. The March 21st, 1959 event does not necessarily represent the 100-year event; however, it came very close to approaching the 100-year event and therefore, for the purposes of this study was used to represent the 100-year event.

The hydrologic modelling component of the GFLOOD model was used to simulate the overland flow hydrographs for the period November 1st, 1958 to April 30th, 1959. The overland flow hydrographs of the sub-basins for the March 21st, 1959 event were isolated and added as lateral inflows to the hydraulic modelling component of GFLOOD. The hydraulic model of GFLOOD was used to simulate the water levels along the main branch of the Welland River between Binbrook dam and Montrose. The Froude number was calculated for each $\Delta x$ reach along the Welland River. For subcritical flow, computations proceeded in the upstream direction, and for
supercritical flow computations must proceeded in the downstream direction. In addition, reaches along the Welland River were identified as not being impacted by the downstream conditions or the backwater from the Niagara River. This was accomplished graphically, by taking the maximum water surface elevation at Montrose and extending it backwards until it intersected with the bed elevation of the Welland River. As a result, Port Davidson was identified as the point along the Welland River where the extent of the backwater from the Niagara River ceases. Therefore, for the reach between Binbrook Dam and Port Davidson the computations proceeded in the downstream direction. In addition, for the reach between Port Davidson and upstream of the New Siphon, flood computations also proceeded in the downstream direction. Under flood flow conditions the impacts on water levels upstream of the New Siphon, due to the backwater from the Niagara River, are negligible. Previous sensitivity analyses on water levels along the Welland River due to fluctuating water levels at the Grass Island Pool (Niagara River) have shown that the impacts to water surface elevations at the upstream end of the New Siphon are minimal, approximately 2-3 cm (Dillon Engineering, 1985).

A constant long-term mean water level of 170.77 (GSC) was assumed at Montrose. The simulation was then repeated again for a constant maximum water level at Montrose of 171.24 m (GSC) and a constant minimum water level at Montrose of 170.42 m (GSC). The results of the simulation are illustrated in Figure 5.73.

According to Figure 5.79 there is no impact to upstream water levels due to changing water levels at Montrose. Therefore, while hydro operation and manipulation of the Montrose Power Canal Gates do produce flow reversals under “normal” daily flow conditions, this is as much a function of the flat gradient on the Welland River which only falls 4 m from the siphons to Wellandport. Under 100-year flow conditions, the Power Canal serves to take flow out of the system and actually serves to reduce flood levels upstream of the siphons. Below the siphons, the predominant factor affecting 100-Year flood levels is the water levels within the Niagara River which are regulated by the International Niagara Board of Control’s 1993 Directive.
Figure 0-78 Correlation analysis of water levels between Chippawa Grass Island Pool and Montrose for the period January 1st, 2000 to December 31st, 2000

\[ y = 1.0721x - 12.711 \]
\[ R^2 = 0.8986 \]

Figure 0-79 Upstream water levels due to changing water levels at Montrose for the March 21st 1959 event
5.4.5 Sensitivity of Changing Water Levels due to Blockage at the Old Siphon

In 1985 Dillon Engineering undertook a sensitivity analysis to test the effects of the bridges and syphons on water levels. To test the cumulative effect of all bridges and syphons, the 100-year flow was simulated with all structures removed, thus assuming that these structures have been improved to such an extent that no losses occur at any crossing. According to their results the removal of hydraulic losses at all structures would lower the 100-year water levels by not more than 0.5 m. The most beneficial effect would be noticed at the upstream end of the City of Welland. The analysis demonstrated that even with an unlimited amount of investment, structural improvements would not significantly reduce the flood levels in the Welland River (Dillon Engineering, 1985).

A sensitivity analysis was undertaken to test the sensitivity of the blockage at the Old Syphon, the results are illustrated in Figure 5.80. To test the cumulative effect of the blockage at the Old Syphon, the 100-year flow was simulated based on the March 21st 1959 event. According to Figure 5.80, the removal of the hydraulic losses at the Old Siphon (All Clear case) would lower the 100-year water levels by not more than 0.17 m (maximum change), along the Welland River extending from Niagara Street to Church Road. According to Figure 5.80, the 75% blockage and 50% blockage had the most significant impact on water levels along the Welland River. However, the level of impact was relatively small despite the significant amount of blockage. It is important to note that the headloss rating curves for the Old and New Siphons were developed using a steady state model. A limitation of steady-state models and other, more simplified routing methods is their assumption that, along a reach, there is a single valued relationship between stage and discharge. This assumption is contrary to observations of natural floods on rivers of very low slope. For a low-gradient river such as the Welland River, discharge at a particular stage when the flood level is increasing is larger than the discharge at the same stage when the flood level is decreasing. Storage is not a function of outflow alone because the water surface is not always parallel to the channel bed. When the flow is increasing, the water surface has a greater slope than the channel bottom, but the opposite is the case when the flow is decreasing. Thus, the relationship between outflow and storage for a given reach is not easily observed and will be different depending on whether the hydrograph is rising or falling. When
flow is significantly restricted, such as the blockages at the Old Siphon, steady-state models estimate the energy grade required to drive the specified flow through the restriction, without regard to storage effects. In reality, however, the flow will back up behind the restriction as stored volume. Depending on the amount of runoff and the upstream storage capacity, the actual flow through the restriction and the corresponding energy grade may be considerably less than the values computed with the steady-state model. As with the Dillon study, this analysis further demonstrates that even with an unlimited amount of investment, structural improvements to the Old Siphon would not significantly reduce the flood levels in the Welland River.

![Figure 0-80 Sensitivity analysis due to blockages at the Old Siphon for the March 21st, 1959 event](image)

5.5 Limitations of the GFLOOD Model

For some applications, the GFLOOD model is subject to limitations due to its governing equations, and also due to the uncertainty associated with some of the parameters used within the model. The governing equations within the GFLOOD model for routing hydrographs are the one-dimensional Saint Venant equations. There are however, some instances where the flow may be more two-dimensional as opposed to one-dimensional.
For example, in the case where the flow, particularly a dam break induced flood, first expands onto a wide and flat floodplain after having passed through an upstream reach which severely constricts the flow. Other applications include, very wide lakes, estuaries or bays in which the computed velocities are required to be accurate for sediment modelling or other transport modelling applications and areas with very wide floodplains with very complex overbank flows controlled by a complicated pattern of road embankments and levees and flow bifurcations.

The simplified dynamic model within GFLOOD assumes the bottom width of the channel remains constant, and under high flow conditions (greater than top of bank), assumes the area of the channel remains constant. However, some floods can cause significant scour (degradation) of alluvial channels. This enlargement in channel cross-sectional area is neglected in the GFLOOD model, since the equations for sediment transport, sediment continuity, dynamic bed-form friction, and channel bed armoring are not included in the governing equations. The significance of the neglected alluvial channel degradation is directly proportional to the channel/floodplain conveyance ratio. As this ratio increases, the degradation increases, causing a significant lowering of the water surface elevations. The simplified dynamic model can be further improved by accounting for mobile bed effects; this may be accomplished by coupling the sediment conservation equation to the finite difference solution of the Saint Venant equations. The coupling can take place between time steps of the flow computation; the type of coupling between water and sediment flow depends on the rate of change of the cross-sectional area of the mobile bed with respect to time. In addition, the dynamic interaction between the changing sand bed forms and bed friction may be approximated through a continuous modification of Manning’s “n”.

For certain flood events the uncertainty associated with the Manning’s “n” value can be significant. For example, for a large flood event certain portions of the floodplain become inundated, which never have been before, and therefore, this necessitates the selection of a Manning’s “n”, without the benefit of previous evaluations of n, from measured data or the use of calibration techniques for determining n. In the simplified dynamic model, separate Manning’s “n” values are specified for the channel, the left and right floodplains. Physically,
however, as the flow increases and more portions of the bank and overbank become inundated, the vegetation located at these elevations causes an increase to the resistance to flow (Maidment, 1993). In addition, the \( n \) value may be larger for small floodplain depths than for large floodplain depths due to the flattening of thick brush, weeds and tall grasses (Maidment, 1993). This effect may be reversed for wooded overbank areas where the leaves on fallen trees and branches can also impede the flow in addition to the tree trunks. Furthermore, \( n \) may also decrease with increasing discharge when the flow area in the banks is greater than the flow area in the overbank areas, such as the case in wide channels with levees on either side for flood protection, or when the flood is confined to the channel. The uncertainty in selecting a Manning’s “\( n \)” value is currently a limiting factor within the GFLOOD model. In turn, a sub-routine for the continuous modification of the Manning’s “\( n \)” parameter can be included based on measured \( n \) values for different water surface elevations and/or discharge values.

Dam-break floods with a large amount of transported debris may accumulate at constricted cross sections such as bridge openings where it acts as a temporary dam and it partially or completely restricts the flow. In turn, the maximum magnitude of this effect can be simulated at best using the GFLOOD model by treating the downstream blockage as a dam and eventual release. There is also uncertainty associated with volume losses due to dam-break floods as they propagate downstream and inundate large floodplains where infiltration and detention storage losses may occur. Such losses are difficult to predict and are usually neglected, however, they can become significant, particularly in arid regions.

The GFLOOD model neglects the effects of ice cover, which can affect the flow and ice effects. In addition, the model does not account for localized hanging of ice dams, which can further increase the flow resistance. Severe increases in water surface elevation can occur during the break-up of ice as air temperatures increase. Ice jams can considerably reduce the cross-sectional area of flow and act as a constricted flow control section. A sub-routine within the simplified dynamic model can be set-up to account for ice cover, ice jams and ice jam releases.

The GFLOOD model also neglects the effects of landslide generated waves. Reservoirs are sometimes subject to landslides which move at high velocities into the reservoir and displace the
contents within the reservoir, and create a very steep water wave which travels up and down the length of the reservoir, and overtop an earth embankment or dam and cause its failure. A landslide-generated wave can be simulated using the simplified dynamic routing model. Using known information on the volume of landslide mass, its porosity, and the time interval over which the landslide moves into the reservoir, the landslide is deposited into the reservoir during very small time steps in the routing computations; this reduction in reservoir cross-sectional area creates a landslide generated wave in the solution of the Saint Venant equations. Wave run-up for vertical, concrete face dams can be neglected, however, for earthen dams, the wave can advance up the sloping dam face 2.5 times the height of the wave (Maidment 1993; Morris and Wiggert, 1972).
Chapter 6 – Summary of Findings

In this study, the overall objective of the research was to test the hypothesis that creating an Ontario–based method for estimation of model time constants, and incorporating an improved flood routing subroutine would produce a more accurate, versatile and computationally-efficient watershed model applicable to gauged and ungauged watersheds. The sub-objectives were to (i) develop a methodology for predicting the time of concentration ($T_c$) and recession ($K$) for estimating the lateral inflow to a reach by accounting for the hydro-meteorological conditions and physical characteristics of the sub-basin; (ii) develop a methodology to account for the lateral subsurface and ground-water inflows into the channels; and to account for the lateral seepage outflows into the banks from the channels; and (iii) implementation of the model to a watershed that considers the available hydro-meteorological data, and spatial and temporal scales at which the major hydrologic and hydraulic processes occur.

6.1 Time Parameters

6.1.1 Time of Concentration

The regression equation, equation 5.2 provides a good estimate of the $T_c$ with a relative bias of less than 2%. Equation 5.2 is similar in form and function to other empirical relationships for estimating the $T_c$, primarily those where channel flow dominates. Equation 5.2 was developed from basins within the Credit River watershed, and is therefore, more applicable to watersheds which have similar hydro-meteorological conditions to the Credit River, such as those within Southern Ontario. There are currently over 800 active meteorological stations across Ontario and 504 active stream gauge stations; regionalization could be applied to group the stream gauges and meteorological stations across the province into specific regions. Step-wise multiple linear regression can be applied to each region and an equation for estimating the $T_c$ can be developed for that specific region.
6.1.2 Recession Constants

Equations 5.6, 5.9 and 5.12 were developed to estimate the surface storage recession constant, subsurface flow recession constant and groundwater flow recession constant respectively. Equations 5.6, 5.9 and 5.12 are unique in that they were derived using basins or subwatersheds within Southern Ontario, whereas previous estimations of the recession constants for ungauged basins were undertaken using equations that were derived in the Southern United States. Aside from the differences in watershed shape, there are also significant differences in meteorology and climate between the two regions, which are driving forces behind the hydrologic processes. The regression analysis can be expanded to other regions within Ontario and across Canada. As more data becomes available, it should be possible to derive a series of equations of the form of equations 5.6, 5.9 and 5.12 that can be applied to basins with varying landuse, slope, hydraulic conductivity and shape, and which are more certain of not being unduly influenced by one or two odd basins.

6.2 Channel Routing Procedure

Using the momentum equation of the Saint Venant equations, equations were derived for trapezoidal channels with constant bottom width and triangular channels, and for floodplains to describe the flood routing. The derivations are based on the formulation by Keskin (1994 and 1997), but the equations derived in this study are more comprehensive than Keskin’s original equation, and as a result, they can be applied to rectangular, trapezoidal and triangular channel sections, and floodplains. In these derivations it is assumed that the derivative \( \frac{\partial S_f}{\partial x} \) is negligible with respect to the other terms in equations 4.36, 4.74, 4.88 and 4.91. It is shown that the results of the simplified dynamic model are the same as the results of the dynamic wave model and diffusion wave model and the simplified dynamic model could be used accurately in some routing reaches where the dynamic wave and diffusion wave models may not be appropriate. The derived equations and the continuity equation are solved by using a simple explicit finite difference scheme, defined as forward in time and backward in space. The numerical algorithm used in the model is a simple one of a cascade type, similar to a kinematic cascade. This model
can be useful if the flow is supercritical or when the effect of the secondary condition is negligible at the downstream end of the channel for subcritical flows.

In the simplified dynamic model, a comparison of outflow hydrographs at different distances shows that the attenuation increases with an increase in the computational distance step ($\Delta x$) and with an increase in inverse channel side slope ($z$). Translation increases with distance, but decreases with an increase in the computational time step ($\Delta t$). The simplified dynamic model is compared with the dynamic wave model. Findings indicated that both models yield approximately the same characteristics of outflow hydrographs under the same conditions; however, the shape of the channel can affect the characteristics of the outflow hydrograph of the simplified dynamic model. The simplified dynamic model is much easier to formulate than the dynamic wave model and is simple to calculate in comparison to the other one. The simplified dynamic model was also compared to the diffusion wave model. The results show that the simplified dynamic model generates a smaller peak flow and time-to-peak flow than the diffusion wave model. The attenuation obtained by the simplified dynamic model depends on the characteristics of the inflow hydrograph including shape and peak discharge, while the attenuation obtained using the diffusion wave or dynamic wave models is not dependent upon the shape or peak flow of the inflow hydrograph. This indicates that the simplified dynamic model can be applied to cases or routing problems where neither the diffusion wave nor dynamic wave models are appropriate.

For the simplified dynamic model, a comparison was undertaken between the observed outflow hydrograph and the simulated outflow hydrographs with and without transmission losses. The differences in discharge values between the two modelling scenarios were negligible. Evaporation losses are not likely to be of paramount importance in a flood routing application given the time scales involved and are often neglected. Similarly, under the same application, seepage losses or recharge from bank storage would be similarly irrelevant in routing surface runoff, since they are considerably less efficient hydraulically and therefore, would be expected to involve significantly lower flow rates. Therefore, the impact on flood flows would be negligible. However, floods occurring in some channels, such as those located in arid regions, can be of sufficient magnitude to affect the river flow by attenuating the peak flow, reducing the
wave peak celerity, and extending the recession limb of the hydrograph. However, time steps required for flow routing are much smaller than those used for modelling evaporative losses, subsurface flow and/or groundwater flow, and as a result variable time steps are required for solving each of the different processes within the watershed model. The watershed model can take advantage of this for greater efficiency by solving the evaporation, subsurface flow and groundwater flow equations periodically.

### 6.3 Application of the GFLOOD Model to the Welland River Watershed

The GFLOOD model was applied to the Welland River Watershed in Southern Ontario. The GFLOOD model combines the physics–based parameter models of open channel flow and semi-empirical semi-distributed–parameter models of overland flow and unsaturated groundwater flow for optimum prediction performance with manageable computational power and data requirements. The model makes full use of the complete Saint Venant Equations. A simultaneous coupling procedure is implemented to provide fully dynamic interactions between overland surface flow, the unsaturated groundwater flow or baseflow to the channel and the channel flow interface.

In order to account for the backwater from the Niagara River onto the Welland River, modifications were made to the numerical algorithm of the simplified dynamic routing subroutine within GFLOOD. A secondary equation was provided for each boundary condition and solved in connection with the specified boundary condition. The extra equation for both external boundaries is derived by integrating the continuity equation. The equation can also be applied to upstream and downstream reaches of a hydraulic structure (internal boundary such as a dam or bridge), together with an appropriate internal boundary structure and water surface elevations both upstream and downstream of the structure. The Froude number is calculated for each Δx reach, for subcritical flow, computations proceed in the upstream direction, and for supercritical flow computations must proceed in the downstream direction. In addition, reaches along the Welland River were identified as not being impacted by the downstream conditions or the backwater from the Niagara River. In this case computations proceeded in the downstream direction under subcritical flow conditions, for example, the reach between Binbrook Dam and
Port Davidson. In addition, if a flood wave were to propagate downstream between Port Davidson and upstream of the New Siphon, flood computations would also proceed in the downstream direction since the impacts due to the backwater from the Niagara River, would be minimal (approximately 2-3 cm increase in water surface elevations). A technique for simulating pressurized flow in a closed conduit, has also, been incorporated into the model and was utilized to simulate flow through the siphons.

The results of the calibration and validation exercise indicated that no single statistical goodness-of-fit criterion is sufficient to assess performance of the model for simulation of the hydrograph or hyetograph. The selection of the objective function for calibration purposes should be dependent upon the objective of the modelling exercise, as selection of criteria depends upon the hydrograph or hyetograph variables being evaluated. For example, selection of peak discharges should not be the only criterion in deciding whether the simulations are acceptable. While an excellent fit may occur around a peak value, a poor fit may occur on the rising and/or recession parts of the hydrograph. Overall, the model results indicated good predictive capability for both flows and water levels.

The model was then used to evaluate the impacts of hydraulic structures (i.e. siphons) and changing water levels at the watershed outlet due to hydro operations on upstream water levels and flows during an extreme event. The simplified dynamic model is able to account for the effects of storage on upstream discharges and water levels due to blockages or restrictions at a crossing. The model can be used as a tool to evaluate the impacts of man-made control features and hydraulic structures on water surface profiles and flows and to further evaluate the interactions between the overland surface flow and channel flow process interactions and general hydrologic patterns in watersheds.
Chapter 7 – Conclusions

7.1 Time Parameters

The regression equations developed in this study are good predictors for estimating the time constants for Ontario watersheds with a reasonable level of accuracy. Their form and function is similar to existing empirical relationships and are applicable to basins where channel flow dominates. The methods developed for estimating the time constants for Ontario watersheds provide good starting points for watershed modelling but may require some calibration for watersheds with adequate flow data to allow calibration.

7.2 Channel Routing Procedure

Using the momentum equation, an equation was derived for a prismatic channel section with varying top width to describe the flood routing. The derived equation and the continuity equation are solved by using a simple explicit finite difference scheme defined as forward in time and backward in space. The numerical algorithm used in the model is a simple one of a cascade type. The model can be useful if the flow is supercritical or when the effect of the secondary condition is negligible at the downstream end of the channel for subcritical flows.

The results of the simplified dynamic model are the same as the results of the general dynamic model and that the simplified dynamic model could be used accurately in some flood routing problems where the general dynamic model is not appropriate. The attenuation obtained using the simplified dynamic model depends on some characteristics of the inflow hydrograph, such as the geometric form of the hydrograph and its peak flow. The simplified dynamic model is compared with the more complex dynamic wave model selected from the literature. Findings indicate that both models yield approximately the same characteristics of outflow hydrographs under the same conditions. The simplified dynamic model is also compared to the diffusion wave model. The results show that the simplified dynamic model gives a smaller peak flow and time-to-peak flow than the diffusion wave model. The simplified dynamic model is easier to formulate.
and is simpler to calculate than the other two. The time of computation of the model is also very short.

A comparison was undertaken between the observed outflow hydrograph and the simulated outflow hydrographs with and without transmission losses for the GFLOOD model. The differences in discharge values between the two modelling scenarios were negligible under flood flow conditions. Transmission losses involve lower flow rates and are less efficient hydraulically when routing surface flows. Therefore, transmission losses are considered to be negligible or are only solved periodically to warrant their inclusion in the computations.

7.3 Application of the GFLOOD Model to the Welland River

The results of the calibration and validation exercise indicated that no single statistical goodness-of-fit criterion is sufficient to assess the performance of the model. The selection of the objective function for calibration and validation is dependent upon the objective of the modelling exercise. The calibration and validation results indicate good predictive capability for both flows and water levels of the GFLOOD model. The model can also be used as a tool to evaluate the impacts of man-made control features and hydraulic structures on water surface profiles and flows and to further evaluate the interactions between the overland surface flow and channel flow process interactions and general hydrologic patterns in watersheds.

The objective of this research was to test the hypothesis that creating an Ontario – based method for estimation of model time constants, and incorporating an improved flood routing subroutine would produce a more accurate, versatile and computationally-efficient watershed model applicable to gauged and ungauged watersheds in Ontario. The results of the calibration and validation exercise meet the first part of this objective. The GFLOOD model requires minimal data and very little effort to execute. The computations of the model for channel routing are less complex than other solution techniques for solving the Saint Venant equations. The additional accuracy gained when using more complex models does not justify their use to predict water levels and average flows in typical unsteady flow applications with and without floodplains. Therefore, the model’s simplified numerical solution satisfies the second part of this objective.
Chapter 8 – Recommendations

8.1 Time Parameters

The regression analysis can be expanded to include other watersheds and regions within Ontario and across Canada. There are currently over 800 active climate stations and 504 active stream gauge stations across Ontario alone. Regionalization could be applied to group the stream gauges and climate stations into specific regions across the province. Regression analysis can be applied to each region and a specific set of equations for estimating the time constants could be established.

8.2 Channel Routing Procedure

Improvements to the existing channel routing model would include: (i) the development of equations for routing flows in non-prismatic channel sections, such as circular sections including closed conduits and parabolic cross-sections; (ii) a sub-routine embedded within the routing model that would allow the Manning’s “n” value to vary with the depth of flow within the channel; (iii) accounting for mobile bed effects, by coupling the sediment conservation equation to the finite difference solution of the Saint Venant equations and eliminating the assumption that the bottom width of the channel and cross-sectional area of the channel under high flow conditions (greater than top-of-bank) are not constant; (iv) develop a sub-routine to account for ice cover, ice jams and ice jam releases; and (v) develop a sub-routine for landside generated waves.

8.3 Application of the GFLOOD Model to the Welland River

It is recommended that the GFLOOD model be applied to other watersheds within Southern Ontario to further test its’ versatility. In addition, subroutines can be developed that could automate many of the processes required by the routing sub-routine. For example, selection of a computational time step and grid spacing between sub-reaches and the implementation of a mixed flow (sub-critical/super-critical) algorithm within the model could be automated.
Furthermore, the model should also be applied in a flood forecasting and warning mode, therefore, testing its ability to predict flows and water levels in real-time.
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GLOSSARY

\( \alpha \) Regression coefficient
\( \alpha \) Coefficient
\( \alpha_{bnk} \) Bank flow recession constant
\( \alpha_{cn} \) Coefficient channel portion
\( \alpha_{lf} \) Coefficient left floodplain
\( \alpha_{rf} \) Coefficient right floodplain
\( \beta \) Regression coefficient
\( \beta \) Coefficient
\( \beta_{cn} \) Coefficient channel portion
\( \beta_{lf} \) Coefficient left floodplain
\( \beta_{rf} \) Coefficient right floodplain
\( \Delta \) Gradient of saturated vapor pressure curve (kPa/°C)
\( \Delta P \) Change in model input parameter
\( \Delta T \) Difference between maximum and minimum temperature
\( \Delta t \) Time step for channel routing (h)
\( \Delta x \) Computational distance step (m)
\( \gamma \) Psychometric constant (kPa/°C)
\( \delta \) Solar declination
\( \varepsilon \) Emissivity
\( \varepsilon' \) Net emissivity
\( \varepsilon_a \) Atmospheric emittance
\( \varepsilon_{vs} \) Vegetative or soil emittance
\( \vartheta \) Zenith angle
\( \mu_{10m} \) Average daily wind speed (m/s)
\( \mu_{windmon} \) Mean monthly wind speed (m/s)
\( \rho_a \) Density of air (kg/m³)
\( \rho_w \) Density of water (kg/m³)
\( \sigma \) Standard deviation
\( \varphi \) Geographic latitude in radians
\( \omega \) Angular velocity of the earth’s rotation in rad/hr
\( \omega t \) Hour angle
\( A \) Coefficient (1/°C)
\( A \) Drainage Area (km²)
\( A \) Total Cross-sectional flow area (active and inactive, m²)
\( A_{cn} \) Cross-sectional flow area channel component (m²)
\( A_{lf} \) Cross-sectional flow area left floodplain (m²)
\( A_{rf} \) Cross-sectional flow area right floodplain (m²)
\( AM \) Aspect-slope reduction factor
\( A_o \) In-active cross-sectional area (channel storage) of flow (m²)
\( B \) Coefficient (hour)
\( B \) Vapor transfer coefficient (m/Pa·s)
\( B \) Wetted top-width (m)
\( b_H \) Scaling factor that controls the degree of deviation in relative humidity caused by the presence or absence of precipitation
\( bnk \) Total amount of water within bank storage
\( bnk_{in} \) Amount of water entering bank storage from the reach
<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$B_w$</td>
<td>Channel bottom width (m)</td>
</tr>
<tr>
<td>$B_{wlf}$</td>
<td>Bottom-width left floodplain (m)</td>
</tr>
<tr>
<td>$B_{wrf}$</td>
<td>Bottom-width right floodplain (m)</td>
</tr>
<tr>
<td>$C$</td>
<td>Regression coefficient</td>
</tr>
<tr>
<td>$C_{em}$</td>
<td>Discharge coefficient for the embankment</td>
</tr>
<tr>
<td>$\text{coef}_{ev}$</td>
<td>Coefficient of evaporation</td>
</tr>
<tr>
<td>$\text{cov}_1$</td>
<td>Coefficient of the areal depletion curve for snow cover</td>
</tr>
<tr>
<td>$\text{cov}_2$</td>
<td>Coefficient of the areal depletion curve for snow cover</td>
</tr>
<tr>
<td>$c_p$</td>
<td>Specific heat of moist air at constant pressure ($1.013 \times 10^{-3} \text{ MJ kg}^{-1} \circ \text{C}^{-1}$)</td>
</tr>
<tr>
<td>$\text{CRM}$</td>
<td>Coefficient of residual mass</td>
</tr>
<tr>
<td>$C_r$</td>
<td>Discharge coefficient of the uncontrolled spillway</td>
</tr>
<tr>
<td>$C_v$</td>
<td>Correction factor</td>
</tr>
<tr>
<td>$D$</td>
<td>Duration of a rainfall event</td>
</tr>
<tr>
<td>$D^*$</td>
<td>Coefficient of determination</td>
</tr>
<tr>
<td>$\text{Div}$</td>
<td>Diversion flow from the reach ($\text{m}^3$)</td>
</tr>
<tr>
<td>$D_m$</td>
<td>Number of days in a month</td>
</tr>
<tr>
<td>$DS_{mx}$</td>
<td>Maximum depression storage (mm)</td>
</tr>
<tr>
<td>$\text{days}_{dry}$</td>
<td>Number of dry days in the month</td>
</tr>
<tr>
<td>$\text{days}_{total}$</td>
<td>Total number of days in a month</td>
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<tr>
<td>$\text{days}_{wet}$</td>
<td>Number of wet days in the month</td>
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<td>$df$</td>
<td>Degrees of freedom</td>
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<tr>
<td>$dn$</td>
<td>Day number of the year</td>
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<tr>
<td>$e$</td>
<td>Vapor pressure (kPa)</td>
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<td>$e_a$</td>
<td>Vapor pressure (kPa), aerodynamic method</td>
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<tr>
<td>$e_{as}$</td>
<td>Saturation vapor pressure (kPa), aerodynamic method</td>
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<td>$e_s$</td>
<td>Saturated vapor pressure (kPa)</td>
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<td>$E$</td>
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<td>Elongation Ratio</td>
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<td>$E_a$</td>
<td>Modified coefficient of efficiency</td>
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<td>Amount of evapotranspiration at time $i$ (mm, $\text{H}_2\text{O}$)</td>
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<td>Evaporation rate (mm/day) using aerodynamic method</td>
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<td>Elevation (m)</td>
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<td>Total $F$-test</td>
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<td>$\bar{fr}_{trans}$</td>
<td>Volume of transmission losses to deep aquifer ($\text{m}^3$)</td>
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<td>$\bar{fr}_{\Delta t}$</td>
<td>Fraction of time step where water is flowing in the reach</td>
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<td>$F_{t_i}$</td>
<td>Infiltration rate at time $i$, (mm $\text{H}_2\text{O}$)</td>
</tr>
<tr>
<td>$G_{lyr}$</td>
<td>Gravity drainage of the free water for the soil layer (mm $\text{H}_2\text{O}$)</td>
</tr>
<tr>
<td>$h_b$</td>
<td>Elevation of the breach</td>
</tr>
<tr>
<td>$H_{lyr}$</td>
<td>Depth of the soil layer (mm)</td>
</tr>
<tr>
<td>$H_o$</td>
<td>Extraterrestrial daily irradiation ($\text{MJ m}^{-2} \text{ d}^{-1}$)</td>
</tr>
<tr>
<td>$H_b$</td>
<td>Net long-wave radiation ($\text{MJ m}^{-2} \text{ d}^{-1}$)</td>
</tr>
<tr>
<td>$H_{day}$</td>
<td>Net short-wave radiation reaching the ground ($\text{MJ m}^{-2} \text{ d}^{-1}$)</td>
</tr>
<tr>
<td>$H(I)$</td>
<td>Hourly fraction for hour $I$</td>
</tr>
<tr>
<td>$H_{MX}$</td>
<td>Maximum solar radiation on a clear day($\text{MJ m}^{-2} \text{ d}^{-1}$)</td>
</tr>
<tr>
<td>$H_{T(I)}$</td>
<td>Cumulative hourly fraction for hour $I$</td>
</tr>
</tbody>
</table>
\( H_{\text{net}} \)  Net solar radiation (MJ m\(^{-2}\) d\(^{-1}\))
\( H_R \)  Radiant energy (MJ m\(^{-2}\) d\(^{-1}\))
\( Hs \)  Crest elevation of the uncontrolled spillway
\( h_{wl} \)  Water level (m)
\( i \)  Rainfall intensity (mm/hr)
\( I^* \)  Hour for a rainfall event (1 to 24)
\( I \)  Model sensitivity index
\( I_{on} \)  Extraterrestrial irradiance
\( I_o \)  Irradiance on a horizontal surface
\( I_{SC} \)  Solar constant (4.921 MJm\(^{-2}\)h\(^{-1}\))
\( l_v \)  Latent heat of vaporization (MJ/kg)
\( k \)  Von Karmen constant, 0.4
\( k \)  Constant
\( K_c \)  Compaction time constant (hour)
\( K_{cn} \)  Fraction of flow in channel portion
\( K_{eff} \)  Effective hydraulic conductivity (mm/h)
\( K_{em} \)  Submergence correction factor for the embankment
\( K_{lf} \)  Fraction of flow in left floodplain
\( K_{rf} \)  Fraction of flow in right floodplain
\( K_i \)  Effective hydraulic conductivity of the reach (mm/h)
\( K_s \)  Submergence correction factor
\( K_{surf} \)  Surface-storage recession constant (h)
\( K_{subs} \)  Subsurface flow recession constant (h)
\( K_{gw} \)  Groundwater flow recession constant (h)
\( L \)  Reach Length (m)
\( L \)  Maximum flow distance (km)
\( L \)  flow length (ft)
\( L_c \)  Main channel length (km)
\( L_{em} \)  Length of the embankment (m)
\( L_{hill} \)  Hill slope length (m)
\( L_i \)  Reach length
\( LIA \)  Leaf area index
\( LIA_{mx} \)  Maximum leaf area index
\( L_{us} \)  Length of the uncontrolled spillway
\( L_{slo} \)  Slope length (km)
\( MFD \)  Maximum flow distance (km)
\( M_{L1} \)  Momentum flux, seepage flow
\( M_{L2} \)  Momentum flux, overland flow
\( n \)  Manning’s roughness coefficient
\( n_{cn} \)  Manning’s roughness coefficient for the channel
\( n_{lf} \)  Manning’s roughness coefficient for left floodplain
\( n_{rf} \)  Manning’s roughness coefficient for right floodplain
\( NEWDEN \)  Density of new snow (vol/vol)
\( O \)  Model output
\( O_i \)  Observed output at time step \( i \)
\( O_o \)  Baseline or nominal model output
\( \bar{O} \)  Mean of the observed outputs
\( O_o \)  Nominal output parameter
\( O_1 \)  Output for parameter 1
$O_2$  
Output for parameter 2

$p$  
Predictor variable

$P_o$  
Nominal parameter

$P_1$  
Parameter 1

$P_2$  
Parameter 2

$P^*$  
Air pressure (kPa)

$P$  
Model input parameter

$P_o$  
Model baseline or nominal input parameter value

$P_i$  
Predicted output at time step $i$

$P_i$  
Wetted perimeter of the channel or floodplain section

$\bar{P}_i$  
Predicted output based on the regression of $P_i$ on $O_i$

$\bar{P}$  
Mean of the predicted outputs

$POR$  
Porosity of the snowpack (vol/vol)

$P_i$  
Percolation rate at time $t$, (mm $H_2O$)

$P_w$  
Probability of a wet day

$P_{(w|d)}$  
Probability of a wet day followed by a dry day

$P_{(w|w)}$  
Probability of a wet day followed by a wet day

$Q$  
Discharge ($m^3/s$)

$Q_b$  
Breach discharge ($m^3/s$)

$Q_{br}$  
Discharge through a bridge ($m^3/s$)

$Q_{cn}$  
Discharge in channel component ($m^3/s$)

$Q_{em}$  
Flow over an embankment ($m^3/s$)

$q_{L1}$  
Lateral inflow, seepage flow ($m^3/s/m$)

$q_{L2}$  
Lateral inflow, overland flow ($m^3/s/m$)

$q_v$  
Specific humidity

$P$  
Wetted perimeter (m)

$PCP$  
Precipitation volume gain ($m^3$)

$P_{cn}$  
Wetted perimeter of channel portion (m)

$P_{lf}$  
Wetted perimeter of left floodplain (m)

$P_{rf}$  
Wetted perimeter of right floodplain (m)

$r^2$  
Coefficient of determination

$r_p$  
Coefficient of determination for $p$ predictor variable

$R$  
Fraction of maximum rainfall within 1 hour

$R$  
Hydraulic radius (m)

$R_a$  
Gas constant of moist air (J/kg·k)

$RBIA$  
Relative BIAS

$R_h$  
Average relative humidity for the day

$R_{hLmon}$  
Average relative humidity for the month on dry days

$R_{hLmon}$  
Smallest relative humidity value that can be generated on a given day in the month

$R_{hUmon}$  
Largest relative humidity value that can be generated on a given day in the month

$R_{hWmon}$  
Average relative humidity for the month on wet days

$R_{hmon}$  
Average relative humidity for the month

$R_{hmon,mean}$  
Mean of $R_{hLmon}$, $R_{hmon}$, and $R_{hUmon}$

$R_{cn}$  
Hydraulic Radius channel portion (m)

$R_{lf}$  
Hydraulic Radius left floodplain (m)
$R_{rf}$ Hydraulic Radius right floodplain (m)
$R_i$ Precipitation amount at time $i$ (mm, H$_2$O)
$R_m$ Total precipitation in a month
RMSE Root mean square error
RMSE$_s$ Systematic root mean square error
RMSE$_u$ Unsystematic root mean square error
$RND_i$ Random number between 0.0 and 1.0
$R_t$ Rainfall or snowmelt rate at time $t$, (mm H$_2$O)
$R_w$ Average precipitation per wet day
$S$ Main channel slope (m/km)
$S$ Watershed slope (%)
$SA$ Absolute sensitivity coefficient
$S_B$ Basin slope (m/m)
$S_C$ Channel slope (m/m)
$SD$ Deviation sensitivity
$S_e$ Standard error of estimate
$S_f$ Boundary friction slope
$Sf_{ch}$ Frictional slope channel component
$Sf_{lf}$ Frictional slope left floodplain
$S_{mx}$ Maximum retention parameter
$Sf_{rf}$ Frictional slope right floodplain
$S_o$ Channel bed slope
$S_{olf}$ Bed slope left floodplain
$S_{orf}$ Bed slope right floodplain
SR Relative Sensitivity
$STI$ Sample time interval
$t$ Time in hours
$t_b$ Time after breach
$T_{AR}$ Air temperature (°C) at time $t$
$T_{BS}$ Base Temperature (°C)
$T_C$ Time of Concentration (Hrs)
$T_{DL}$ Total day length
$T_k$ Temperature in Kelvin (273.15 + °C)
$T_{lag}$ Lag time (Hrs)
$T_{Loss}$ Transmission losses within the reach
$T_{max,dry}$ Maximum monthly temperature on dry days
$T_{max,m}$ Maximum monthly temperature
$T_{max,wet}$ Maximum monthly temperature on wet days
$T_{min,m}$ Minimum monthly temperature
$T_{SR}$ Time in sunrise
$T_{SS}$ Time of sunset
$T_{loss}$ Transmission losses (m$^3$)
$TT$ Travel time (h)
$V$ Velocity of flow (m/s)
$V_{bnk}$ Volume of water added to reach from bank storage (m$^3$)
$V_{cn}$ Channel velocity (m/s)
$V_{lf}$ Velocity left floodplain (m/s)
$V_{rf}$ Velocity right floodplain (m/s)
$V_{in}$ Reach inflow volume (m$^3$)
\( V_{\text{out}} \)  
Reach outflow volume \( (m^3) \)

\( V_{\text{stored, 1}} \)  
Initial volume of water in reach \( (m^3) \)

\( V_{\text{stored, 2}} \)  
Final volume of water in reach \( (m^3) \)

\( \bar{W} \)  
Average wetted top-width for a reach \( (m) \)

\( x_i \)  
Northing coordinate of the sub-basin centroid \( (i) \)

\( x_{\text{STN}} \)  
Northing coordinate of the climate station \( (\text{STN}) \)

\( y \)  
Total depth of flow in reach \( (m) \)

\( y_{\text{bnk}} \)  
Depth at top-of-bank in channel \( (m) \)

\( y_{\text{cn}} \)  
Depth of flow in floodplain, channel portion \( (m) \)

\( y_i \)  
Easting coordinate of the sub-basin centroid \( (i) \)

\( y_{\text{lf}} \)  
Depth of flow left floodplain \( (m) \)

\( y_{\text{rf}} \)  
Depth of flow right floodplain \( (m) \)

\( y_{\text{STN}} \)  
Easting coordinate of the climate station \( (\text{STN}) \)

\( z \)  
Breach slope

\( z_2 \)  
Height at which wind speed was measured \( (m) \)

\( z_{\text{ch}} \)  
Inverse side-slope of channel

\( z_i \)  
Elevation of the sub-basin centroid \( (i) \)

\( z_{\text{lf}} \)  
Inverse side-slope of left floodplain

\( z_{\text{rf}} \)  
Inverse side-slope of right floodplain

\( z_o \)  
Roughness of the surface at which wind speed was measured

\( z_{\text{STN}} \)  
Elevation of the climate station \( (\text{STN}) \)
APPENDIX A

Guelph Flood Forecasting and Warning Model (GFLOOD)

GFLOOD is an acronym that stands for Guelph Flood Forecasting Model, a river basin or watershed scale flow prediction model. GFLOOD was developed to determine the impact of flooding on communities in complex watersheds with varying climate, soils, topography and land use conditions over long and short periods of time. GFLOOD is capable of simulating flows on watersheds ranging in size from small (less than 100 km$^2$) to large (greater than 1000 km$^2$).

To satisfy these objectives, the GFLOOD model is physically based. Rather than relying on regression equations to describe the relationship between input and output variables, the GFLOOD model requires specific information about weather, soils, vegetation and land use occurring within a watershed as input to model the processes associated with water movement. The benefits of this approach are watersheds with no stream gauge information can be modelled; and the impact of alternative input data on water quantity, such as changes in climate, vegetation and land use can be evaluated.

The GFLOOD model can simulate floods using minimal data commonly available through public and open sources. The model is also computationally efficient and is capable of simulating a variety dynamic flood scenarios. Furthermore, the model can be used to study the long-term impacts of flooding on a community. For example, many of the problems addressed by users in flood studies involve estimating the probability of exceedance of an extreme event and its impact on the community. This can either be undertaken by simulating a statistically significant design event or running a long-term continuous simulation and generating a hydrologic time series of data. The latter requires meteorological data set spanning over several decades.

GFLOOD is based on the platform developed for the Greater Toronto Area (GTA) Conservation Authorities “Flood Forecasting and Warning Spreadsheet Model, (FFOR)” developed by Environmental Water Resources Group Limited (2000). In the late 1990s, Conservation
Authorities identified a need for a model to predict floods along their respective watersheds. The original spreadsheet model contained a snowmelt subroutine, along with the Antecedent Precipitation Index Method for calculating runoff volumes, and the isochronal method for hydrograph convolution and the determination of flood flows. Despite its relative ease of use, FFOR was limited in its application. For example, the FFOR model was limited to modelling basins with time of concentrations less than 24 hours. In addition, the application of the antecedent precipitation index (API) method to urban watersheds for estimating runoff volumes resulted in poor model performance and results. Furthermore, FFOR utilized no channel or reservoir routing limiting its prediction of flood flows to the basin outlet.

As a result, modifications were made to the original model to extend its capabilities, these included: a) a meteorological time series sub-model for varying climatic conditions; b) a snowmelt sub-model; c) the Green and Ampt two layer soil-moisture accounting model for calculating runoff volumes; and d) a hydrograph transformation procedure based on the linear reservoir method. For channel routing the GFLOOD model uses a one-dimensional simplified dynamic routing sub-model based on the complete Saint Venant Equations. This gives the model the added advantage of being able to deal with dynamic flooding problems such as dam breaches.

**A.1 Modelling Overview**

GFLOOD allows a number of different processes to be simulated within a watershed. For modelling purposes the watershed may be subdivided into a series of subwatersheds, sub-basins or sub-catchments. Modelling the watershed as a series of sub-basins is beneficial particularly when different areas of the watershed are dominated by land uses and soils dissimilar enough to impact hydrology. The sub basins are further sub divided into a series of elevation bands, a maximum of 10 elevation bands per sub basin, thereby taking into consideration the orographic variations in precipitation and temperature across the sub basin.

Information for each sub basin is organized into the following categories: climate; hydrologic response units (HRUs); subsurface flow and/or groundwater; and the main channel or reach.
draining the sub basin. Hydrologic response units are lumped land areas within the sub basin that are comprised of land cover, soils and stormwater management strategies.

Similar to other watershed models, in GFLOOD, the water balance is the driving force behind everything that happens within a watershed. The hydrologic cycle as simulated by the model must conform to what is happening within the watershed. The simulation of a hydrograph within a watershed can be separated into two major components, the hydrologic phase and the hydraulic phase. The hydrologic phase includes all of the water that has fallen from the sky onto the ground and controls the amount of water to the main channel of the sub basin. The second phase, the hydraulic phase, controls the route water follows to the sub basin outlet.

A.2 Hydrologic Phase

The hydrologic cycle as simulated by GFLOOD is based on the water balance equation:

\[ SMC_t = SMC_o - \sum_{i=1}^{T} (R_t - Q_{surf} - E_a - w_{seep} - Q_{gw}) \]  

(A.1)

In equation A.1, \( SMC_t \) is the final soil moisture content (mm, H₂O), \( SMC_o \) is the initial soil moisture content at time \( t \) (mm, H₂O), \( t \) is the time in hours, \( R_t \) is the precipitation amount at time \( t \) (mm, H₂O), \( Q_{surf} \) is the amount of surface runoff at time \( t \), \( E_a \) is the amount of evapotranspiration at time \( t \), \( w_{seep} \) is the amount of water entering the vadose zone from the soil profile at time \( t \) (mm, H₂O) and \( Q_{gw} \) is the amount of return flow at time \( t \), (mm, H₂O).

The subdivision of the watershed into sub basins enables the model to reflect differences in evapotranspiration for the various plants and soils. Runoff is predicted separately for each hydrologic response unit (HRU) and is routed to determine the total runoff for the watershed. This increases the accuracy of the model and gives a much better physical description of the water balance.
Figure A.1, illustrates the different processes used by the GFLOOD model of the hydrologic phase of the hydrograph. The different inputs and processes used in the hydrologic phase of the model are summarized in the proceeding sections.

Figure A - 1 HRU/Sub-basins command loop
A.2.1 Climate

The climate of a watershed provides the moisture and energy inputs that control the water balance. The climate information required by GFLOOD includes daily precipitation, daily maximum and minimum air temperatures, wind speed and humidity. These values can either be read by the GFLOOD model from observed climate records or they can be simulated by the model.

In watershed modelling, meteorological inputs can vary significantly with location. To account for these variations, the GFLOOD model accepts inputs on the basis of separate Zones of Uniform Meteorology (ZUM). A ZUM (Schroeter et al, 1992) is defined as “a portion of a watershed throughout which one set of meteorological measurements can be used to calculate snowmelt and runoff”. The ZUMs are further refined or ‘downscaled’ to the subcatchment level using the inverse distance squared method. The distance between the centroid of each sub basin and the climate stations is calculated using Equation A.2.

\[ d_i^2 = (x_i - x_{STN})^2 + (y_i - y_{STN})^2 + (z_i - z_{STN})^2 \]  \hspace{1cm} (A.2)

Where \( d_i \) is the distance between the sub-basin and the climate station; \( x_i, y_i \) and \( z_i \) are the northing, easting and elevation of the sub-basin centroid; and \( x_{STN}, y_{STN} \) and \( z_{STN} \) are the northing, easting and elevation of the climate station. A subroutine within the GFLOOD model has been set-up to calculate the precipitation, snowfall, temperature and potential evaporation for each sub basin by incorporating each of the climate stations surrounding the watershed using the inverse distance squared method (Equation A.3).

\[ P = \frac{\sum_{i=1}^{N} P_i}{\sum_{i=1}^{N} \frac{1}{d_i^2}} \]  \hspace{1cm} (A.3)
Where $P_i$ is the climate parameter value and $d_i$ has been previously defined. In Canada recording rain gauge measurements are not generally made during the period November 1st to March 31st, primarily because the delicate tipping bucket mechanism used in most gauges is prone to freezing. To shield the tipping bucket mechanism from freezing the rain gauge collecting canister is ‘capped’ with a simple lid. In some instances during warm winter rainstorms, the canister ‘lids’ are removed, and hourly rainfall depths are recorded. Consequently, hourly rainfall depths are estimated primarily for the winter period (November 1st to March 31st), but also for isolated dates throughout the year.

A subroutine within the GFLOOD model has been set-up to simulate the hourly distribution of air temperature from the daily maximum, mean and minimum temperature ($T_{\text{max}}$, $T_{\text{mean}}$ and $T_{\text{min}}$, respectively) values. The hourly temperatures are assumed to follow a sinusoidal distribution with the maximum at 3:00 PM and the minimum at 6:00 AM. This distribution would correspond to the clear sunny day case, which is common for snowmelt only events, but not so typical for a rainfall, or rainfall plus snowmelt events. However, for simulating outflow hydrographs from subwatersheds, the proper temperature distribution is not as critical an input as the rainfall distribution (Schroeter, 1996). The hourly humidity, wind speed and station pressure values are read into the model directly from the observed climate records. In the absence of hourly climate records, average daily values are used in lieu. In the absence of daily climate records, monthly records of precipitation, relative humidity and wind speed are used to generate a daily time series for each parameter.

GFLOOD uses a model similar to that developed by Richardson (1981) to generate precipitation for simulations which do not read in measured data. This precipitation model is also used to fill in missing data in the measured records. The precipitation generator uses a two-state first order Markov-Chain (Geng et al, 1986) to define a day as either wet or dry by comparing a random number (0.0-1.0) generated by the model to monthly wet-dry probabilities input by the user. If a day is classified as wet, the amount of precipitation is calculated using the two-parametre gamma distribution.
If hourly rainfall is required, a randomization procedure has been setup within the GFLOOD model to estimate the hourly rainfall from daily rainfall values. A pre-defined series of temporal rainfall patterns (20 in total) based on the double exponential function, are used to define the rainfall distribution. With the double exponential function the rainfall intensity increases with time until it reaches a maximum, or peak rainfall intensity. Once the peak rainfall intensity has been reached the rainfall intensity exponentially decreases with time until the end of the storm.

Maximum and minimum air temperatures are generated from a normal distribution. A continuity equation is incorporated into the generator to account for variations in temperature for dry and rainy conditions. Maximum air temperatures are adjusted downwards for rainy conditions and upwards when simulating dry conditions. The adjustments are made so that the long-term generated average monthly maximum temperature values agree with the input averages.

A modified exponential function is used to generate daily mean wind speed given the maximum monthly wind gust and average monthly wind speed. The relative humidity model uses the hourly dew point temperature to calculate hourly relative humidity, based on the methodology developed by the National Oceanic and Atmospheric Administration (NOAA). In the absence of hourly dew point temperature, long term monthly averages for relative humidity are used to compute the daily maximum and minimum relative humidity values using a triangular distribution. The maximum value occurs at 6:00 AM and the minimum value occurs at 3:00PM. The values are further adjusted to account for wet and dry day conditions. Once the maximum and minimum daily relative humidity values have been established, a sinusoidal distribution is applied to calculate the hourly relative humidity values.

GFLOOD classifies precipitation as rainfall or freezing rain and/or snowfall. The daily snowfall amounts are read directly by the model from observed climate records. In the case where there may be missing data values, the GFLOOD model will classify precipitation as either rain or snow based on the daily average temperature. Snow cover within the GFLOOD model is computed for each sub basin using a model which takes into consideration the shading, drifting, topography and land cover. The user defines a threshold snow depth which covers 100% of the
sub-basin, as the snow depth in a sub-basin decreases below this value, the snow coverage is allowed to decline non-linearly based on an areal depletion curve.

GFLOOD employs a temperature index approach to determine snowmelt, which is based on the model developed by Schroeter and Whitely (1987ab) and Schroeter (1988). Six processes (refreeze, compaction, new snow deposition, rain deposition, snowmelt, and release of liquid water) are considered in the model. The snow pack conditions at the start of melt simulation \(t=0\) for the snow covered proportions of the sub basins are specified in terms of an initial snow depth \(IDEPTH, mm\), the initial solid water content \(ISWC, mm\) and the initial liquid water content \(ILWC, mm\). This information can be obtained from snow course survey measurements. Generally, \(ILWC\) can be set to zero if a period of below freezing temperatures preceded the snowmelt event. The values of runoff (liquid water released from the snow pack) for each time step provide input to the runoff generation routine. Within the GFLOOD model the sequence within a time step of the calculations is refreeze, compaction, new snow additions, snowmelt and rainfall deposition, and the release of liquid water.

The model allows the sub basin to be split into a maximum of ten (10) elevation bands. By separating the sub-basins into elevation bands the model is able to consider the orographic variations in precipitation and temperature values. Snowfall and snowmelt are simulated separately for each band.

### A.2.2 Water Balance

As precipitation falls from the sky, it may be intercepted by and held within the vegetation canopy or fall directly onto the ground. Water on the ground will either infiltrate into the soil profile or runoff as overland flow into the stream channel. Runoff moves relatively quickly to a stream channel and contributes to short-term stream response. Infiltrated water may be held in the soil or it may make its way through the soil-water matrix and to the surface-water system via subsurface and groundwater flow paths. The parameters that are accounted for in the GFLOOD model include: canopy storage, ground surface storage, infiltration, water movement within the soil layers, evaporation, interflow, surface runoff, subsurface and/or shallow groundwater flow and deep groundwater flow.
Canopy storage is the amount of water held by vegetative surfaces (canopy) where it is held and made available for evaporation. Canopy storage is modelled separately within the GFLOOD model. GFLOOD allows the user to enter the maximum amount of canopy storage at maximum leaf area index for the land cover. This value and the leaf area index are used to calculate the maximum storage at any time within the growth cycle of the land cover or vegetation. When evaporation is calculated water is first removed from the canopy storage.

Each sub basin is divided into impervious and pervious zones. The impervious zone includes roads and adjoining shoulders, lanes, ditches, and stream channels. The pervious area is apportioned to four hydrologic soil groups. Rainfall or snowmelt falling onto the ground on the impervious zone first fills surface depression storage and then is available as overland runoff. The amount of water falling onto the pervious zone is sub divided into overland runoff and infiltrated components. Infiltration refers to the amount of water entering the soil profile from the surface. As water enters the soil, the soil will become increasingly wet, thereby decreasing the rate of infiltration over time until it reaches a steady value. The initial rate of infiltration depends upon the moisture content of the soil prior to the introduction of water into the soil profile. The final rate of infiltration is equivalent to the saturated hydraulic conductivity of the soil. The GFLOOD model uses the Green-Ampt two-layer soil model for predicting infiltration. The Green and Ampt two-layer soil model is based on the Green and Ampt (1911) formula, and has been used in Southern Ontario on a number of watershed Studies (Schroeter et al, 2000).

A soil-moisture accounting procedure has also been added to the GFLOOD model, allowing the model to run in continuous simulation mode. Water continues to flow through the soil profile even after water infiltrating from the soil surface has ceased. The movement of water within the soil matrix is caused by the differences in water content throughout the soil profile. Once the water content within the soil profile is uniform redistribution will cease. The redistribution of water within GFLOOD is computed using Darcy’s Law. The downward movement of water from one soil layer to the next, or percolation, occurs when the field capacity of the top layer is exceeded and the bottom layer is not saturated. The flow rate within the soil matrix is governed by the saturated hydraulic conductivity of the soil. The air temperature can also effect the movement of water within
the soil matrix. If the air temperature is below 0°C, water movement within that top soil layer is not permitted.

The potential daily evaporation rate within the GFLOOD model is calculated using the combination method (Penman, 1948 and Wiesner, 1970). No evaporation is assumed during a rainfall or snowmelt period. For water that has fallen onto the ground, evaporation is deducted from the depression storage first, and then from the soil-water of the top soil layer until the available storage reaches half its maximum storage. Thereafter, half the evaporation amount is removed from the soil-water of the top layer and half from the second or bottom layer. Evaporation rates are usually set equal to zero when snow cover is present.

Interflow is the streamflow contribution which originates below the soil surface and above the zone where rocks are saturated with water. Interflow within the soil profile (0-2 m) is calculated simultaneously with water movement. A kinematic storage model is used to predict the amount of interflow within each soil layer. The model accounts for variation in conductivity, slope and soil-water content.

Surface runoff or overland flow is flow that is occurring along a sloping surface. GFLOOD simulates surface runoff volumes and flows for each hydrologic response unit (HRU). In GFLOOD surface runoff volumes are calculated using the Green and Ampt method. The Green and Ampt method calculates infiltration as a function of the wetting front and effective hydraulic conductivity. Water that does not infiltrate or is not held within surface depression storage becomes surface runoff. GFLOOD includes a provision for estimating runoff from frozen soil, where soil is defined as frozen if the air temperature is less than 0°C. Overland surface runoff volumes are converted to overland surface flows using the Clark Unit Hydrograph method (Clark, 1945) or the linear reservoir method to estimate direct runoff to the streams. The time of concentration and the surface flow recession constant are calculated using empirical relationships.

Tributary channels are minor or lower order channels branching off the main branch. They are sometimes referred to as intermittent or ephemeral streams. All of the flows in the tributary
channels is released and routed through the main channel of the sub basin. The GFLOOD model uses the attributes of the representative tributary channel to determine the time of concentration ($T_c$) and recession constant ($K$). There is no subsurface or groundwater flow contribution to the tributary channels within the GFLOOD model.

GFLOOD partitions baseflow into two (2) aquifer systems: a shallow unconfined aquifer which contributes baseflow to streams within the sub-basin; and a deep confined aquifer which contributes baseflow to streams outside the sub-basin (Schroeter, 1996; Arnold, et al, 1993). Water percolating past the bottom of the second soil layer is partitioned into two (2) fractions. Depending on the soil texture each of the fractions becomes recharge for one of the aquifers. Water may also be removed from the shallow or deep aquifer by pumping. Baseflows are routed to the main channel using the linear reservoir routing method.

A.3 Hydraulic Phase

Once GFLOOD determines the amount of water to the main channel the flow is routed through the stream network of the watershed using a command structure similar to HYMO (Williams and Hann, 1972). As water flows downstream through the channel a portion may be lost due to evaporation and transmission through the stream bed of the channel. Another potential loss is the removal of water for human or agricultural use. Flow may be supplemented by the fall of rain directly onto the channel and/or the addition of point source discharges along the channel.

For the purposes of channel routing the GFLOOD model uses a one-dimensional channel routing sub-model which is based on the simplified dynamic routing method. Runoff in a computer model for fairly large, well-defined channels is transmitted downstream by routing techniques of the hydrologic or storage routing variety. Although such hydrologic routing techniques function adequately in most situations, they have serious shortcomings when unsteady flows are subjected to backwater effects due to reservoir operations, inflows from, or outflows to, large tributaries. When channel bottom slopes are quite mild, the flow inertial effects ignored in the hydrologic technique also become important. The model is applicable to rivers of varying physical features, such as cross-section geometry, variable roughness parameters, lateral inflows, flow diversions,
off-channel storage, local head losses such as bridge contraction-expansions, dam operations, and siphons.

The simplified dynamic model is based on the complete one-dimensional equations of unsteady flow (Saint-Venant equations) which are used to route flood hydrograph(s) through a designated channel/floodplain and its tributaries. The simplified dynamic model provides more accuracy in simulating the unsteady flood wave than that provided by the hydrologic methods, as well as, other less complex hydraulic methods such as the kinematic-wave and the diffusion-wave methods. Of the many available hydrologic and hydraulic routing techniques, only the dynamic wave method accounts for the acceleration effects associated with the dam-break wave and the influence of downstream unsteady backwater effects produced by channel constrictions, dams, bridge-road embankments, and tributary inflows (Maidment, 1993).

Natural waterways as well as man-made channels are linked together to form a network of channels. The configuration may be either dendritic (tree-type) and/or looped (Islands, parallel channels connected by bypasses, etc.). Within the GFLOOD model tributary flow at a junction is treated as lateral inflow $q$ to the main stem channel. Numerical difficulties in solving the Saint-Venant equations can sometimes arise when the ratio of lateral inflow to channel flow is too large. Therefore, as the complexity of the watershed increases the computational efficiency begins to decrease, and the model becomes limited in its application. To overcome this issue a separate computational time step is used for the channel routing component than the $\Delta t$ ordinates for the lateral inflow hydrographs. For example, if $\Delta t$ for the overland hydrographs is less than $\Delta t$ for the channel routing component, than the first lateral inflow ordinate to be routed through the channel is the average of the first $N$ ordinates of the lateral inflow hydrographs. $N$, in turn, is equivalent to the ratio of the computational time step for channel routing ($\Delta t_{\text{channel routing}}$, in hours) to the computation time step for the overland hydrograph transformation ($\Delta t_{\text{overland hydrographs}}$, in hours).
APPENDIX B

Weather Generator Model (WeatherGEN)

Climate data and information is a driving force behind many physical and biological processes. Most hydrologic models require as a minimum daily weather data to run. In Canada, climate data is archived in daily format. However, missing values often occur in the raw data. For daily climate data, including daily rainfall and snowfall depths, maximum and minimum air temperature, wind speed and relative humidity, even the best available records usually have a few missing values that require “filling-in” procedures. However, in Canada, most hourly rainfall records are missing at least 40% of the values, because the gauges are not open during the winter period (November to April). Distributed hydrologic models are particularly adversely affected by the lack of hourly and/or daily data or the existence of very inaccurate data as they impart large uncertainties to the model prediction. As part of this research, a daily and hourly weather generator (WeatherGEN) numerical model that uses the currently available monthly statistics from Meteorological Services of Canada (MSC) climate stations was developed to work in conjunction with the GFLOOD model.

Appendix B describes the daily and hourly weather generator algorithm (WeatherGEN) for estimating daily and hourly rainfall; daily and hourly snowfall; daily maximum and minimum air temperature and hourly air temperature; daily maximum and minimum relative humidity and hourly relative humidity; daily and hourly solar radiation; daily and hourly wind speed; and daily and hourly potential evaporation, using data from Meteorological Services of Canada (MSC) climate stations. This data set contains, among others, monthly values of precipitation, minimum and maximum temperature, minimum and maximum relative humidity, wind speed, solar radiation and the number of wet days per month. A combination of qualitative and quantitative criteria was used to test and evaluate the model. Comparisons were made between the generated meteorological data and the recorded measured data at select climate stations. The daily meteorological data was used to generate a time series of daily and/or hourly, where applicable, meteorological data values. These values were then compared to the daily and/or hourly recorded measured data at the applicable climate stations.
B.1 Weather Generators

Many large-scale hydrologic models require daily and/or hourly weather data, but such data are not directly available. Currently there is daily weather information available for some 4000 stations across Canada (Meteorological Services Canada, 2011). The distribution of these data sets, however, is quite uneven leaving large parts of the country with very few stations. Furthermore, the quality of the daily data is not always very reliable and there are often large amounts of missing data. For this reason daily weather generators that use monthly statistics have been used to “fill-in” the missing data. But these monthly statistics must first be calculated based on the available daily data hence, making them unusable for areas with very little to no daily data.

Three major types of weather generators: non-parametric, empirical or semi-parametric, and parametric are accepted by the scientific community (Brissette et al., 2007). Non-parametric weather generators are computationally simple and do not require any statistical assumptions to be made. They work by using a nearest-neighbour re-sampling procedure known as the K-NN approach (Sharif and Burn, 2007; Brandsma and Buishand, 1998; Beersma et al., 2002; Yates et al., 2003). The nearest neighbour algorithm works by searching the days in the historical record which have similar characteristics to those of the previously simulated day, and randomly selecting one of these as the simulated value for the next day (Beersma et al., 2002). A major drawback of this approach is that unless historical daily data is available for a site or region, the model becomes virtually unusable.

Semi-Parametric or Empirical weather generators include LARS-WG and the Wilks model, SDSM (Semenov and Barrow, 1997; Wilks and Wilby, 1999). The LARS-WG differs from the non-parametric approach described above because it employs a series-approach in which the wet and dry spells are determined by taking into account the observed values and assuming mixed-exponential distributions for dry or wet series as well as precipitation amounts (Semenov and Barrow, 1997). Wilks (1998) introduced Markov-chains of higher order that have a better “memory” of the preceding weather. The model was further extended for multi-site applications.
by using a collection of single site models in which a conditional probability distribution is specified and thus spatially correlated random numbers can be generated (Mehrotra, 2006; Wilks, 1998). A drawback to these empirical approaches is that there is a subjective assumption about the type of probability distribution for precipitation amounts and spell lengths, and the spatial correlation structure is empirically estimated for use with multiple sites (King et al., 2010).

In most parametric weather generators, a Markov chain is used to determine the probability of a wet or dry day and a probability distribution is assumed to determine the amount of precipitation (Kuchar, 2004; Hanson and Johnson, 1998). Most of the parametric weather generators are extensions of Richardson’s WGEN model, which was developed in 1981 (Richardson, 1981). Some examples of parametric weather generators successfully employed using the Richardson approach are CLIGEN, WGENK, GEM, WXGEN, and SIMMENTO (Kuchar, 2004; Schoof et al., 2005; Hanson and Johnson, 1998; Soltani and Hoogenboom, 2003). A major drawback of the parametric approach is that the Markov chain does not take into account the previous days’ weather, and thus rare events, such as droughts or wet spells are not well produced (King et al., 2010; Sharif and Burn, 2007; Semenov and Barrow, 1997; Dibike and Coulibaly, 2005). Another limitation of the parametric weather generators is that an assumption must be made about the probability distribution of precipitation amounts, and different distributions do not give similar results (King et al., 2010; Sharif and Burn, 2007). In addition, the weather generators cannot be easily transferred to other basins as their underlying probability assumptions would change (King et al., 2010; Sharif and Burn, 2007). The computational effort is also significantly higher than other methods since many parameters must be estimated and statistically verified (Mehrotra et al., 2006). Furthermore, many of the above parametric weather generators have not been fully parameterized for Canadian climate stations (McKague et al., 2003).

One of the objectives of this research was to develop a methodology to simultaneously simulate flows and water levels within a stream and/or river reach using a hybrid modelling approach. In order to accomplish this objective hourly and/or sub-hourly weather data is required. The current daily weather generators use monthly weather statistics to generate daily weather data and are therefore, not directly applicable here, since the ultimate goal is to develop a time series of
hourly and/or sub-hourly meteorological data. In addition, these weather generators cannot be automated to simultaneously generate values for many stations and/or climate zones. Small watersheds, which are less than 100 km$^2$ in drainage area, will typically be bound by 1 or 2 climate zones. However, medium to large sized watersheds, greater than 1000 km$^2$ will be bound by several different climate zones. For example, the Credit River Watershed in Southern Ontario is bound by four (4) different climate zones, the Lake Ontario shoreline, the Huron slopes, the South slopes, and the Simcoe and Kawartha Lakes (Credit Valley Conservation, 2002). Furthermore, there is a lack of available daily relative humidity and solar radiation data available from Canadian Climate Stations; two parameters which are necessary for estimating potential evaporation. Therefore, a separate weather generator (WeatherGEN) was developed to work in conjunction with the hybrid hydrologic-hydraulic model GFLOOD and to utilize the available climate data from Meteorological Services of Canada (MSC) climate stations to generate a time series of meteorological hourly values for multiple climate stations and/or climate zones.

### B.2 Overview of the Weather Generator Model: WeatherGEN

The climate of a watershed is the driving force behind the hydrologic cycle; climate information provides the moisture content and energy inputs that control the water balance. The GFLOOD model requires daily values of precipitation, maximum and minimum air temperature, relative humidity and wind speed. These values may be read into the model from a file or generated using the monthly average data summarized over a number of years. The weather generator model WeatherGEN was developed to generate the necessary climate data or to fill in the gaps in the measured records.

The occurrence of rain on a given day has a major impact on relative humidity, temperature and solar radiation for the day. The weather generator first independently generates precipitation for the day. Once the total amount of rainfall for the day is generated, the distribution of rainfall within the day is computed. Maximum air temperature, minimum air temperature, solar radiation and relative humidity are then generated based on the presence or absence of rain for the day. Wind speed is generated independently. Once the daily values are entered into or determined by the model, hourly values are computed for precipitation, air temperature, relative humidity, solar
radiation and wind speed. In addition, to the above parameters the WeatherGEN model also calculates daily and hourly potential evaporation for each climate station.

In watershed modelling, meteorological inputs can vary significantly with location. To account for these variations, the WeatherGEN model accepts inputs on the basis of separate Zones of Uniform Meteorology (ZUM). Each ZUM is represented by a single climate station. The ZUMs are further refined or ‘downscaled’ to the sub-basin level using the inverse distance squared method (Equation A.2). The distance between the centroid of each sub-basin and climate station is calculated using Equation A.1. A subroutine within the WeatherGEN model has been set-up to calculate the rainfall, snowfall, air temperature and potential evaporation for each sub-basin by incorporating each of the climate stations surrounding the watershed using the inverse distance squared method (Equation A.2). Figure B.1 shows the flowchart of the WeatherGEN algorithm.

**B.2.1 Daily Precipitation**

To parameterize the WeatherGEN model, the precipitation parameters “average precipitation per wet day,” \( R_w \) and “probability of a wet day,” \( P_{(w)} \), were directly calculated for each climate station based on the available monthly values:

\[
R_w = \frac{R_m}{W_m} \tag{B.1}
\]

\[
P_w = \frac{W_m}{D_m} \tag{B.2}
\]

Where \( R_m \) is the total precipitation in a month, \( W_m \) the number of wet days in a month, and \( D_m \) is the number of days in a month. A “wet day” is defined as a day where the precipitation is greater than 0.2 mm. A wet day was determined by using the popular two-state first order Markov chain. Geng et al., (1986) analyzed seven stations with diverse climates (from below 200 mm rain per year to over 2000 mm rain per year and from 34 rainy days per year to 187 rainy days) and showed that the monthly transitional probabilities for a wet day following a dry day, \( P_{(w|d)} \), can be linearly related to \( P_{(w)} \) and the intercept is not significantly different from zero. Based on this,
the transitional probabilities for a wet day following a wet day, \( P_{(w|w)} \), can also be derived (Equations B.3 and B.4).

\[
P_{(w|d)} = 0.75P(w) \quad (B.3)
\]

\[
P_{(w|w)} = 0.25 + P_{(w|d)} \quad (B.4)
\]

In case of a wet day, the precipitation amount \( (R) \) was computed with a two-parameter gamma distribution (Equation B.5). According to Geng et al, (1986), the parameters are related to the average rainfall per wet day as shown in Equations B.6 and B.7, and the regression line explained 96.5% of the total variation of the analyzed months at different sites.

\[
P(R) = \frac{(R)^{(\alpha-1)}e^{-\frac{R}{\beta}}}{\beta^\alpha \Gamma(\alpha)} \quad (B.5)
\]

\[
\beta = -2.16 + 1.83R_w \quad (B.6)
\]

\[
\alpha = \frac{R_w}{\beta} \quad (B.7)
\]

### B.2.2 Daily Air Temperature

In certain circumstances daily temperature values may not be available; therefore, a methodology within the WeatherGEN model has been implemented to disaggregate historical monthly temperature values into daily temperature values. Daily average temperature values including minimum and maximum temperatures are sampled from a normal distribution using average monthly minimum and maximum temperatures. Since the daily temperature values were sampled using a normal distribution they require standard deviations in addition to monthly averages. For wet days, the maximum temperature is assumed to be lower than on dry days. Equations B.8 through B.13 describe the daily temperature generation parameters. The coefficients of the standard deviation computation are consistent with those of the SIMMETEO version included in WeatherMan (Pickering et al., 1994; Soltani and Hoogenboom, 2003):

\[
\Delta T = T_{max,m} - T_{min,m} \quad (B.8)
\]
Where \( T_{\text{min,}m} \) is the average minimum temperature in a month, \( T_{\text{max,}m} \) is the average maximum temperature in a month, \( \sigma \) stands for standard deviation, \( T_{\text{max,dry}} \), the average maximum temperature for dry days in a month and \( T_{\text{max,wet}} \), the average maximum temperature for wet days in a month. The scaling factor for the calculation of \( T_{\text{max,dry}} \) and \( T_{\text{max,wet}} \), that controls the degree of deviation in temperature on wet and dry days was set to 0.25 (in Equation B.11) to obtain a smoother temperature course. However, this coefficient can be adjusted, in order to adapt the model to a specific region or climate.

Once the minimum and maximum daily temperatures have been determined the WeatherGEN model proceeds to calculate the daily rainfall and snowfall. If the mean daily temperature falls below 0°C, all the precipitation for the day is assumed to be snowfall. If the mean daily temperature is above 0°C, all of the precipitation for the day is assumed to be rainfall. The mean daily temperature is the average of the minimum and maximum daily temperatures.
Figure B - 1 Flowchart of the weather generator WeatherGEN algorithm
B.2.3 Daily Relative Humidity

Once the minimum and maximum daily temperatures have been determined the WeatherGEN model proceeds to calculate the daily rainfall and snowfall. If the mean daily temperature falls below 0°C, all the precipitation for the day is assumed to be snowfall. If the mean daily temperature is above 0°C, all of the precipitation for the day is assumed to be rainfall. The mean daily temperature is the average of the minimum and maximum daily temperatures.

Daily relative humidity values are determined by WeatherGEN when the combined method is used to calculate daily potential evaporation. Daily average relative humidity values are sampled from a triangular distribution using average monthly relative humidity values. This method has been implemented in other watershed modelling applications such as the “Soil Water Assessment Tool,” SWAT (Nietsch et al, 2005).

Not all climate stations across Canada and in Ontario collect relative humidity. Only “principal” climate stations collect this parameter. For the most part, elements of climate data that is of greatest interest includes: the daily values of minimum and maximum temperatures (deg C), rainfall (mm), snowfall (cm) and total precipitation (mm). For principal stations, additional daily elements such as peak wind gusts, days with a variety of weather phenomena such as thunderstorms or freezing precipitation, and elements based on hourly elements such as wind, sunshine, and solar radiation are also available. Generally the network of volunteer stations is limited to basic daily temperature and precipitation observations. Therefore, historical monthly relative humidity values must be brought in from adjacent or neighboring “principal” climate station(s).

The triangular distribution used to generate daily relative humidity values requires four inputs: mean monthly relative humidity, maximum relative humidity value allowed in month, minimum relative humidity value allowed in month, and a random number between 0.0 and 1.0. The maximum relative humidity value, or upper limit of the triangular distribution, is calculated from the mean monthly relative humidity with the equation:
Where \( R_{U\text{mon}} \) is the largest relative humidity value that can be generated on a given day in the month, and \( R_{\text{mon}} \) is the average relative humidity for the month. The minimum relative humidity value, or lower limit of the triangular distribution, is calculated from the mean monthly relative humidity with the equation:

\[
R_{L\text{mon}} = R_{\text{mon}} \cdot \left(1 - e^{(-R_{\text{mon}})}\right)
\]

(B.15)

Where \( R_{L\text{mon}} \) is the smallest relative humidity value that can be generated on a given day in the month, and \( R_{\text{mon}} \) is the average relative humidity for the month. The daily relative humidity values are then generated using Equations B.16 through B.18.

The triangular distribution uses one of two sets of equations to generate a daily maximum relative humidity and daily minimum relative humidity.

If \( \text{RND}_1 \leq \frac{R_{U(I)} - R_{L(I)}}{R_{U(I)} - R_{L(I)}} \)

then

\[
R_{(I)} = R_{\text{mon}(I)} \cdot \frac{T_{L(I)} + \left[\text{RND}_1 \cdot (R_{U(I)} - R_{L(I)}) \cdot (R_{\text{mon}(I)} - R_{L(I)})\right]^{0.5}}{R_{\text{mon\_mean}}}
\]

(B.17)

If \( \text{RND}_1 > \frac{R_{\text{mon}(I)} - R_{L(I)}}{R_{U(I)} - R_{L(I)}} \) then

\[
R_{(I)} = R_{\text{mon}(I)} \cdot \frac{R_{U(I)} - (R_{U(I)} - R_{\text{mon}(I)}) \cdot \left[\frac{R_{U(I)} \cdot (1 - \text{RND}_1) - R_{L(I)} \cdot (1 - \text{RND}_1)}{R_{U(I)} - R_{L(I)}}\right]^{0.5}}{R_{\text{mon\_mean}}}
\]

(B.18)

Where \( \text{RND}_I \) is a random number generated between 0.0 and 1.0, \( I \) represents the daily maximum or minimum relative humidity values for the month, \( R_{\text{mon}(I)} \) represents the average
daily maximum or minimum relative humidity values for the month, $R_{hU(I)}$ is the upper limit of the triangular distribution for the daily maximum or minimum relative humidity values for the month and $R_{hL(I)}$ is the lower limit of the triangular distribution for the daily maximum or minimum relative humidity values for the month and $R_{h\text{mon,mean}}$ is the mean of $R_{hU(I)}$, $R_{hL(I)}$, and $R_{h\text{mon}(I)}$.

To incorporate the effect of clear and overcast weather on generated values of relative humidity, monthly average relative humidity values are adjusted for wet or dry conditions. The continuity equation relates average relative humidity adjusted for wet or dry conditions to the average relative humidity for the month:

$$R_{h\text{mon}} \cdot \text{days} = R_{hW\text{mon}} \cdot \text{days}_\text{wet} + R_{hD\text{mon}} \cdot \text{days}_\text{dry}$$  \hspace{1cm} (B.19)

Where $R_{h\text{mon}}$ is the average relative humidity for the month, $\text{days}$ are the total number of days in the month, $R_{hW\text{mon}}$ is the average relative humidity for the month on wet days, $\text{days}_\text{wet}$ are the number of wet days in the month, $R_{hD\text{mon}}$ is the average relative humidity of the month on dry days, and $\text{days}_\text{dry}$ are the number of dry days in the month. The wet day average relative humidity is assumed to be greater than the dry day average relative humidity by some fraction:

$$R_{hW\text{mon}} = R_{hD\text{mon}} - b_H(R_{hU\text{mon}} - R_{hL\text{mon}})$$  \hspace{1cm} (B.20)

Where $R_{hW\text{mon}}$ is the average relative humidity of the month on wet days, $R_{hD\text{mon}}$ is the average relative humidity of the month on dry days, and $b_H$ is a scaling factor that controls the degree of deviation in relative humidity caused by the presence or absence of precipitation. The scaling factor, $b_H$, is set to 0.5 in WeatherGEN. However, this coefficient may be changed in order to adapt the model to a specific region or climate. To calculate the dry day relative humidity, Equations B.19 and B.20 are combined and solved for $R_{hD\text{mon}}$:

$$R_{hD\text{mon}} = \left( R_{hW\text{mon}} - b_H \cdot \frac{\text{days}_\text{wet}}{\text{days}} \right) \cdot \left( 1 - b_H \cdot \frac{\text{days}_\text{wet}}{\text{days}} \right)^{-1}$$  \hspace{1cm} (B.21)
B.2.4 Solar Radiation

Solar radiation is a driving force in many physical processes and specifically hydrologic processes. Processes that are greatly affected by solar radiation include snow fall, snow melt and evaporation. A number of basic concepts developed by Iqbal (1983) related to the earth’s orbit around the sun are required by WeatherGEN to make solar radiation calculations. The methodology used in WeatherGEN is the same used in SWAT (Nietsch et al, 2005).

The mean distance between the earth and the sun is 1.496 x 108 km and is called one astronomical unit (AU). The earth revolves around the sun in an elliptical orbit and the distance from the earth to the sun on a given day will vary from a maximum of 1.017 AU to a minimum of 0.983 AU. Solar radiation reaching the earth is inversely proportional to the square of its distance from the sun. For engineering applications, Duffie and Beckman (1980) developed an expression for calculating the reciprocal of the square of the radius vector of the earth, also called the eccentricity correction factor, $E_o$, of the earth’s orbit:

$$E_o = \left(\frac{r_0}{r}\right)^2 = 1 + 0.033 \cos \left(\frac{2\pi d_n}{365}\right) \quad (B.22)$$

Where $r_0$ is the mean earth-sun distance (1 AU), $r$ is the earth-sun distance for any given day of the year (AU), and $d_n$ is the day number of the year, ranging from 1 on January 1st to 365 on December 31st. In the modelling computations the equation is modified to account for leap year cycles.

The solar declination is the earth’s latitude at which incoming solar rays are normal to the earth’s surface. The solar declination is zero at the spring and fall equinoxes, approximately $+23\frac{1}{2}^\circ$ at the summer solstice and approximately $-23\frac{1}{2}^\circ$ at the winter solstice. A simple formula to calculate solar declination from Perrin de Brichambaut (1975) is:

$$\delta = \sin^{-1} \left[0.4 \sin \left(\frac{2\pi}{365} (d_n - 82)\right)\right] \quad (B.23)$$
Where $\delta$ is the solar declination reported in radians, and $d_n$ is the day number of the year. The angle between the line from an observer on the earth to the sun and a vertical line extending upward from the observer is called the zenith angle, $\theta_z$. For a given geographical position, the relationship between the sun and a horizontal surface on the earth’s surface is:

$$\cos \theta_z = \sin \delta \sin \phi + \cos \delta \cos \phi \cos \omega t$$  \hspace{1cm} (B.24)

Where $\delta$ is the solar declination in radians, $\phi$ is the geographic latitude in radians, $\omega$ is the angular velocity of the earth’s rotation ($0.2618 \text{ rad h}^{-1}$ or $15^\circ \text{ h}^{-1}$), and $t$ is the solar hour. $t$ equals zero at solar noon, is a positive value in the morning and is a negative value in the evening. The combined term $\omega t$ is referred to as the hour angle. Sunrise, $T_{SR}$, and sunset, $T_{SS}$, occur at equal times before and after solar noon. These times can be determined by rearranging the above equation as:

$$T_{SR} = \cos^{-1} \left( \frac{-\tan \delta \tan \phi}{\omega} \right)$$  \hspace{1cm} (B.25)

$$T_{SS} = -\cos^{-1} \left( \frac{-\tan \delta \tan \phi}{\omega} \right)$$  \hspace{1cm} (B.26)

Total day length, $T_{DL}$ is calculated:

$$T_{DL} = 2 \cos^{-1} \left( \frac{-\tan \delta \tan \phi}{\omega} \right)$$  \hspace{1cm} (B.27)

At latitudes above $66.5^\circ$ or below $-66.5^\circ$, the absolute value of $[-\tan \delta \tan \phi]$ can exceed 1 and the above equation cannot be used. When this happens, there is either no sunrise (winter) or no sunset (summer) and $T_{DL}$ must be assigned a value of 0 or 24 hours, respectively. To determine the minimum day length that will occur during the year, Equation B.27 is solved with the solar declination set to $-23.5^\circ$ (-0.4102 radians) for the northern hemisphere or $23.5^\circ$ (0.4102 radians) for the southern hemisphere.
The solar constant, $I_{SC}$, is the rate of total solar energy at all wavelengths incident on a unit area exposed normally to rays of the sun at a distance of 1 AU from the sun. The value officially adopted by the Commission for Instruments and Methods of Observation in October 1981 is 4.921 MJm$^{-2}$h$^{-1}$. On any given day, the extraterrestrial irradiance (rate of energy) on a surface normal to the rays of the sun, $I_{on}$, is:

$$I_{on} = I_{SC}E_o$$  \hspace{1cm} (B.28)

Where $E_o$ is the eccentricity correction factor of the earth’s orbit, and $I_{on}$ has the same units as the solar constant, $I_{SC}$. To calculate the irradiance on a horizontal surface, $I_o$,

$$I_o = I_{on} \cos \theta_z = I_{SC}E_o \cos \theta_z$$  \hspace{1cm} (B.29)

Where $\cos \theta_z$ is defined in Equation 4.24. The amount of energy falling on a horizontal surface during a day is given by:

$$H_o = \int_{sr}^{ss} I_o dt = 2 \cdot \int_0^{ss} I_o dt$$  \hspace{1cm} (B.30)

Where $H_o$ is the extraterrestrial daily irradiation (MJ m$^{-2}$ d$^{-1}$), $sr$ is sunrise, and $ss$ is sunset. Assuming that $E_o$ remains constant during the one day time step and converting the time $dt$ to the hour angle, the equation can be written as:

$$H_o = \frac{24}{\pi} I_{SC}E_o \cdot \int_0^{\omega_{sr}} (\sin \delta \sin \varphi + \cos \delta \cos \varphi \cos \omega t) d\omega t$$  \hspace{1cm} (B.31)

or

$$H_o = \frac{24}{\pi} I_{SC}E_o \cdot [\omega_{sr} \sin \delta \sin \varphi + (\cos \delta \cos \varphi \sin (\omega_{sr}))]$$  \hspace{1cm} (B.32)

or
When solar radiation enters the earth’s atmosphere, a portion of the energy is removed by scattering and adsorption. The amount of energy lost is a function of the transmittance of the atmosphere, the composition and concentration of the constituents of air at the location, the path length the radiation travels through the air column, and the radiation wavelength. The model makes a broad assumption that roughly 20% of the extraterrestrial radiation is lost while passing through the atmosphere under cloudless skies. Using this assumption, the maximum possible solar radiation on a clear day, $H_{MX}$ (MJ m^{-2} d^{-1}), at a particular location on the earth’s surface is calculated as:

$$H_{o} = 37.59 \cdot E_{o} \cdot \left[ \omega T_{sr} (\sin \delta \sin \varphi) + (\cos \delta \cos \varphi \sin (\omega T_{sr})) \right]$$  \hspace{1cm} (B.33)

The extraterrestrial radiation falling on a horizontal surface during one hour is given by the equation:

$$I_{o} = I_{sc} E_{o} (\sin \delta \sin \varphi + \cos \delta \cos \varphi \cos \omega t)$$  \hspace{1cm} (B.35)

Where $I_{o}$ is the extraterrestrial radiation for 1 hour centred on the hour angle $\omega t$. An accurate calculation of the radiation for each hour of the day requires knowledge of the difference between standard time and solar time for the location. The model simplifies the hourly solar radiation calculation by assuming that solar noon occurs at 12:00 pm local standard time. When the values of $I_{o}$ calculated for every hour between sunrise and sunset are summed, they equal the value of $H_{o}$.

Net radiation requires the determination of both incoming and reflected short-wave radiation and net long-wave or thermal radiation. Expressing net radiation in terms of the net short-wave and long-wave components gives:

$$H_{net} = (1 - \alpha) \cdot H_{day} + H_{b}$$  \hspace{1cm} (B.36)
Where \(H_{\text{net}}\) is the net radiation (MJ m\(^{-2}\) d\(^{-1}\)), \(H_{\text{day}}\) is the short-wave solar radiation reaching the ground (MJ m\(^{-2}\) d\(^{-1}\)), \(\alpha\) is the short-wave reflectance or albedo, and \(H_{\text{b}}\) is the net incoming long-wave radiation (MJ m\(^{-2}\) d\(^{-1}\)).

Net short-wave radiation is defined as \((1-\alpha) \cdot H_{\text{day}}\). \(H_{\text{day}}\) is calculated using Equation B.37 (Hargreaves and Samani, 1982).

\[
H_{\text{day}} = K_r \cdot (T_{\text{max}} - T_{\text{min}})^{0.5} \cdot I_o \tag{B.37}
\]

Where \(H_{\text{day}}\) is the short-wave solar radiation reaching the ground (W m\(^{-2}\)); \(T_{\text{max}}\) and \(T_{\text{min}}\) are the daily maximum and minimum temperatures (deg C); \(I_o\) is the extraterrestrial radiation (W m\(^{-2}\)); and \(K_r\) is an empirical coefficient. A review of measured solar radiation data for Southern Ontario determined this value to be approximately 0.17. Equation B.37 is also referred to as the temperature based approach.

The WeatherGEN model calculates the net short wave and long wave radiation at the climate station. Climate stations across Canada and in Ontario are located in open areas that are free from vegetation, such as trees and tall grasses. As a result, when the snow water equivalent is greater than 0.5 mm, \(\alpha\) is equal to 0.85. When the snow water equivalent is less than 0.5 mm \(\alpha\) is equal to 0.25.

Long-wave radiation is emitted from an object according to the radiation law:

\[
H_R = \varepsilon \sigma T_K^4 \tag{B.38}
\]

Where \(H_R\) is the radiant energy (MJ m\(^{-2}\) d\(^{-1}\)), \(\varepsilon\) is the emissivity, \(\sigma\) is the Stefan-Boltzmann constant (4.903 x 10\(^{-9}\) MJ m\(^{-2}\) K\(^{-4}\) d\(^{-1}\)), and \(T_K\) is the mean air temperature in Kelvin (273.15 + \(^\circ\)C). The net long-wave radiation is calculated using Equation B.39 (Jensen et al., 1990):

\[
H_{\text{b}} = f_{\text{clld}} \cdot (\varepsilon_\alpha - \varepsilon_{\text{vs}}) \cdot \sigma \cdot T_K^4 \tag{B.39}
\]
Where $H_b$ is the net long-wave radiation (MJ m\(^{-2}\) d\(^{-1}\)), $f_{cld}$ is a factor to adjust for cloud cover, $\varepsilon_a$ is the atmospheric emittance, and $\varepsilon_{vs}$ is the vegetative or soil emittance. Wright and Jensen (1972) developed the following expression for the cloud cover adjustment factor, $f_{cld}$:

$$f_{cld} = a \cdot \frac{H_{day}}{H_{MX}} - b \quad (B.40)$$

Where $a$ and $b$ are constants, $H_{day}$ is the solar radiation reaching the ground surface on a given day (MJ m\(^{-2}\) d\(^{-1}\)), and $H_{MX}$ is the maximum possible solar radiation to reach the ground surface on a given day (MJ m\(^{-2}\) d\(^{-1}\)). The two emittances in Equation B.41 may be combined into a single term, the net emittance $\varepsilon'$. The net emittance is calculated using an equation developed by Brunt (1932):

$$\varepsilon' = \varepsilon_a - \varepsilon_{vs} = -\left(a_1 + b_1 \sqrt{e}\right) \quad (B.41)$$

Where $a_1$ and $b_1$ are constants and $e$ is the vapor pressure on a given day (kPa). Combining Equations B.39, B.40 and B.41 results in a general equation for net long-wave radiation:

$$H_b = -\left(a \cdot \frac{H_{day}}{H_{MX}} - b\right) \cdot (a_1 + b_1 \sqrt{e}) \cdot \sigma \cdot T_k^4 \quad (B.42)$$

The model uses the coefficients proposed by Doorenbos and Pruitt (1977) in Equation B.43 to compute net long-wave radiation:

$$H_b = -\left(0.9 \cdot \frac{H_{day}}{H_{MX}} + 0.1\right) \cdot \left(0.34 - 0.139 \sqrt{e}\right) \cdot \sigma \cdot T_k^4 \quad (B.43)$$

The net daily radiation is used to calculate the potential evaporation, along with temperature, relative humidity and wind speed data.
B.2.5 Wind Speed

Wind speed is required by the weather generator to calculate the potential evaporation. Mean daily wind speed is generated in WeatherGEN using a modified exponential equation (Equation B.44). This method is similar to the one used in other weather generators including WXGEN (Sharpley and Williams, 1990).

\[ \mu_{10m} = \mu_{\text{windmon}} \cdot \left( -\ln(RND_1) \right)^{0.3} \]  

(B.44)

Where \( \mu_{10m} \) is the mean wind speed for the day (m s\(^{-1}\)), \( \mu_{\text{windmon}} \) is the average wind speed for the month (m s\(^{-1}\)), and \( RND_1 \) is a random number between 0.0 and 1.0. Wind speed is typically observed at 10 m above the ground. The weather generator for disaggregating historical monthly wind speed data into daily data is used only when there is no daily or hourly data available. If the wind speed data is available, then the average daily values are computed from the hourly values.

B.2.6 Hourly Air Temperature

The WeatherGEN model computes hourly temperatures by assuming a sinusoidal interpolation function between the minimum and maximum daily temperature values. The maximum daily temperature is assumed to occur at 1500 hours and the minimum daily temperature is assumed to occur at 0600 hours. The hourly and sub-hourly temperature values are calculated using the following equations:

\[ T_{\text{mean}} = 0.5 \cdot (T_{\text{max}} + T_{\text{min}}) \]  

(B.45)

\[ T_{\text{amb}} = T_{\text{max}} - T_{\text{mean}} \]  

(B.46)

\[ T_t = \sin \left( \frac{\pi}{15(t-13.5)} \right) T_{\text{amb}} + T_{\text{mean}}, t < 6 \, AM \]  

(B.47)

\[ T_t = \sin \left( \frac{\pi}{15(t-7.5)} \right) T_{\text{amb}} + T_{\text{mean}}, t > 3 \, PM \]  

(B.48)
Where $T_{amb}$ is the ambient daily temperature, $T_{max}$ is the daily maximum temperature, $T_{mean}$ is the mean daily temperature and $T_t$ is the temperature at time $t$ in hours. Equations B.44 through B.49 are also utilized for estimating the hourly and sub-hourly relative humidity values. The maximum relative humidity is assumed to occur at 0600 hours and the minimum relative humidity is assumed to occur at 1500 hours.

### B.2.7 Hourly Rainfall and Snowfall

To account for hourly rainfall values where data may be missing, a procedure has been implemented within the WeatherGEN model to disaggregate daily rainfall values into hourly rainfall values. This process is similar to the methodology developed by Boughton (2000) at the Centre for Catchment Hydrology for disaggregating daily rainfall into hourly rainfall for design flood estimation. Ten (10) principle climate stations across Southern Ontario were used to develop the disaggregation model. The records varied in length from 40 years to 90 years. The data was initially prepared by Schroeter et al, (2000 and 2007) as part of the Source Water Protection Program of the Ministry of Natural Resources. The data were selected as 9 am to 9 am daily blocks of 24 hourly values. Only the top ten (10) rainfall patterns for each month and station were used in the model development (daily rainfalls ≥ 15 mm). The reason being is that there are substantial differences in the hourly temporal patterns between small and large daily rainfalls. In particular, it is common that all rainfall can occur in a single hour when the daily total is small, whereas this is not a common pattern with large daily totals. The purpose here is to disaggregate the larger daily rainfalls that are important in flood studies; hence it was essential to avoid the bias that would be introduced from the much more abundant small daily rainfalls. The distributed rainfall patterns represent days where the hourly rainfall values had not been estimated nor had they been deduced from daily total rainfall values.
The disaggregation model consists of four (4) parts: (i) the primary part of the model is the distribution of the fraction $R$ of the daily total that occurs in the hour of maximum rainfall; (ii) for each value of $R$, there is an average set of values for the other 23 fractions of the daily total; (iii) given the 24 hourly fractions from 1 and 2, values are clustered to maintain the average values of the highest 2-hour, 3-hour, 6-hour and 12-hour fractions; and (iv) the clusters are then arranged into random patterns, which reproduce the variations in 9 am to 9 am daily temporal patterns while retaining all of the statistics of hourly rainfalls used in parts 1 thru 3.

In application, a random number is used to select a value of the fraction $R$ of the daily total that occurs in the hour of maximum rain. $R$ is the ratio of the highest hourly rainfall to the daily total. The selection of $R$ is made from the distribution of $R$, which is derived from observed climate station records (Figures B.2 and B.3). This value of $R$ automatically determines the other 23 fractions based on the average sets of values derived from observed climate records required to maintain the average multi-hour fractions. The 24 fractions are multiplied by the daily total to give the hourly rainfalls, which are then arranged into a daily temporal pattern.

The fraction $R$ of the daily rainfall that occurs in the hour of maximum rain has a distribution of values that is a major characteristic of sub-daily rainfall statistics. A value of $R = 1.0$ means that all rainfall fell in a single hour and is the boundary of non-uniformity of rainfall during a day. Completely uniform rainfall during a day would have 0.04167 of the daily total in every hour. This is the lower bound of $R$.

The distribution of $R$ is based on a collection of 1200 days of data in the period 1950 to 2005 across all 10 climate stations. The values of $R$ for all stations were collated into 20 ranges. The distribution shows the proportion of all values in each of the ranges (Table B.1). In application, a random number is used to select a value of $R$ from the distribution shown in Table B.1. This fixes the fraction of the daily total (which is being disaggregated) that occurs in the hour of maximum rainfall.
Figure B - 2 R-value histogram of Lester B. Pearson Toronto International Airport climate Station

Figure B - 3 R-value histogram of Fergus Shand Dam climate station
Table B-1 Average distribution of R based on ten (10) climate stations

<table>
<thead>
<tr>
<th>Range of R</th>
<th>Percent in Range (%)</th>
<th>Range of R</th>
<th>Percent in Range (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.042-0.075</td>
<td>0.0</td>
<td>0.525-0.575</td>
<td>2.5</td>
</tr>
<tr>
<td>0.075-0.125</td>
<td>0.5</td>
<td>0.575-0.625</td>
<td>2.2</td>
</tr>
<tr>
<td>0.125-0.175</td>
<td>13.0</td>
<td>0.625-0.675</td>
<td>1.2</td>
</tr>
<tr>
<td>0.175-0.225</td>
<td>15.0</td>
<td>0.675-0.725</td>
<td>2.0</td>
</tr>
<tr>
<td>0.225-0.275</td>
<td>17.3</td>
<td>0.725-0.775</td>
<td>1.3</td>
</tr>
<tr>
<td>0.275-0.325</td>
<td>16.9</td>
<td>0.775-0.825</td>
<td>0.4</td>
</tr>
<tr>
<td>0.325-0.375</td>
<td>9.0</td>
<td>0.825-0.875</td>
<td>0.6</td>
</tr>
<tr>
<td>0.375-0.425</td>
<td>7.7</td>
<td>0.875-0.925</td>
<td>0.1</td>
</tr>
<tr>
<td>0.425-0.475</td>
<td>6.4</td>
<td>0.925-0.975</td>
<td>0.1</td>
</tr>
<tr>
<td>0.475-0.525</td>
<td>3.2</td>
<td>0.975-1.000</td>
<td>0.6</td>
</tr>
</tbody>
</table>

If the fraction $R$ of the daily total in the hour of maximum rain is 1.0, then all other 23 fractions must be zero. If the rainfall is completely uniform, then all 24 fractions will be $1/24$ of the daily total. If the fraction $R$ is just a little less than 1.0, then it is highly likely that the rest of the daily rainfall will occur in one or two of the other hours while the other 22 or 23 hours will have zero rainfalls. If the fraction $R$ is just a little larger than 0.04167, then the other 23 values will be slightly less than but close to 0.04167. The significance of this is that the fraction $R$ in the hour of maximum rainfall has a strong influence on the values in the other 23 hours. In addition, it can also have an impact on the duration of the event (Equation B.50).

$$D = \ln \left( \frac{1.04167 - R}{0.04167} \right)$$ (B.50)

Where $D$ is the duration of the event and $R$ is the fraction of maximum rainfall within 1 hour. After the average distribution of $R$ was established (Table B.1), the data from the 10 stations were processed to find the other 23 fractions of the daily total for each value of $R$. All 24 hourly fractions were ranked in order of magnitude (with $R$ as the largest), and the ranked series were averaged in each of the 20 ranges of $R$ shown in Table B.1. Figure B.4 shows 3 of the averaged ranked series of fractions of the daily totals. Equations B.51 and B.52 are used to determine the hourly fractions of the daily total rainfall for values of $R$ and $D$.

$$(I) = H_T(I - 1) - H_T(I)$$ (B.51)

$$H_T(I) = \left( \frac{1-R}{\ln(D)} \right) \cdot \ln(I) + R$$ (B.52)

Where $I$, is the hour (1 to 24), $H(I)$ is the hourly fraction for hour $I$, and $H_T(I)$ is the cumulative hourly fraction for hour $I$. 354
There is no dominant or common temporal pattern of rainfall within the 9 am to 9 am blocks that can be used as a single pattern. The temporal patterns show a very wide range from nearly uniform to highly variable rainfall, as shown by the frequency distributions of R. There is also no pattern in the times when high or low rain occurs. The times of day when the highest rainfall occurred were determined for each of the 10 stations. The average values of all 10 stations are listed in Table B.2. The values shown are percentages of the total that occurred in each of the 24 hours of the day. The values in Table B.2 suggest that a random selection for the time of peak rainfall together with equations B.50 through B.52 can provide an appropriate variability of the temporal patterns of hourly rainfalls within the limits of the daily total and the statistics of hourly rainfalls. The approach used within the disaggregation model is to reproduce the variability of temporal patterns rather than try to average or select a single pattern for use.

There are 24 arrangements of temporal patterns which can be used to allow for the peak hourly rainfall to occur in any hour of the day. When combined with the distribution of R, which allows for variation between uniform and non-uniform rainfall, these arrangements provide 24 x 20 = 480 different temporal patterns with variation between uniform and very non-uniform rainfall.

In application, a random number is used to select a range of R from the 20 ranges (Table B.1). This selection fixes all 24 fractions of the daily total, which form the 2, 3, 6 and 12-hour sub-totals. A second random number is then used to select from the 24 temporal arrangements. The fractions are multiplied by the daily rainfall to give the disaggregated temporal pattern of hourly rainfalls.

Hourly snowfall values within the WeatherGEN model are determined by dividing the total daily snowfall by the number of hours in a day where the hourly temperature falls below 0°C. If the hourly temperature is above 0°C, the hourly snowfall is assumed to be zero for that hour. As a result, the WeatherGEN model assumes a uniform distribution for hourly snowfall values.

The advantages of this generator over other generators is its ability incorporate directly the historical monthly climate data from Meteorological Services Canada into its computations and
the automation that it can generate many pseudo meteorological time series at the same time. Furthermore, the generator is designed in such a way that many of the coefficients and parameters used to generate the climate data can be easily changed in case the user would like to adapt them to a specific region or climate.

![Figure B - 4 Average rank series of fractions of the daily total](image)

**Table B-2 Hour of day of highest hourly rainfall**

<table>
<thead>
<tr>
<th>Hour</th>
<th>%</th>
<th>Hour</th>
<th>%</th>
<th>Hour</th>
<th>%</th>
<th>Hour</th>
<th>%</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2.2</td>
<td>7</td>
<td>5.9</td>
<td>13</td>
<td>4.9</td>
<td>19</td>
<td>4.3</td>
</tr>
<tr>
<td>2</td>
<td>4.8</td>
<td>8</td>
<td>4.1</td>
<td>14</td>
<td>4.3</td>
<td>20</td>
<td>1.5</td>
</tr>
<tr>
<td>3</td>
<td>5.5</td>
<td>9</td>
<td>4.3</td>
<td>15</td>
<td>3.7</td>
<td>21</td>
<td>5.5</td>
</tr>
<tr>
<td>4</td>
<td>6.9</td>
<td>10</td>
<td>10.0</td>
<td>16</td>
<td>2.3</td>
<td>22</td>
<td>4.4</td>
</tr>
<tr>
<td>5</td>
<td>4.8</td>
<td>11</td>
<td>3.0</td>
<td>17</td>
<td>3.1</td>
<td>23</td>
<td>1.6</td>
</tr>
<tr>
<td>6</td>
<td>4.1</td>
<td>12</td>
<td>4.4</td>
<td>18</td>
<td>2.8</td>
<td>24</td>
<td>1.9</td>
</tr>
</tbody>
</table>

**B.2.8 Potential Evaporation**

Potential evaporation or evapotranspiration (PET) was a concept originally introduced by Thornthwaite (1948) as part of a climate classification scheme. He defined PET as the rate at which evapotranspiration would occur from a large area uniformly covered with growing vegetation that has access to an unlimited supply of soil water and that was not exposed to advection or heat storage effects. Because the evapotranspiration rate is strongly influenced by a
number of vegetative surface characteristics, Penman (1956) redefined PET as “the amount of water transpired by a short green crop, completely shading the ground, of uniform height and never short of water.” Penman used grass as his reference crop, but later researchers (Jensen, et al, 1990) have suggested that alfalfa at a height of 40 to 50 cm may be a more appropriate choice.

Numerous methods have been developed to estimate potential evaporation and/or evapotranspiration (PET) these include: the Penman-Monteith method, (Monteith, 1965; Allen 1986; Allen et al, 1989), the Priestley – Taylor method (Priestly and Taylor, 1972) and the Hargreaves method (Hargreaves et al, 1985). Each of these methods varies in the amount of required data inputs. For example, the Penman-Monteith method requires solar radiation, air temperature, relative humidity and wind speed. The Priestley-Taylor method requires solar radiation, air temperature and relative humidity. The Hargreaves method requires air temperature only.

The Penman-Monteith equation combines components that account for energy needed to sustain evaporation, the strength of the mechanism required to remove the water vapor and aerodynamic and surface resistance terms. Potential evaporation in the WeatherGEN model is calculated using the combined method originally developed by Penman (1948) and is described by Equation B.53. This method is well suited for watersheds with detailed climatological data.

\[ E = \Delta \frac{\Delta + \gamma}{\Delta + \gamma} E_r + \Delta \frac{\Delta + \gamma}{\Delta + \gamma} E_a \]  

(B.53)

Where \( \Delta \) is the gradient of the saturated vapor pressure curve (kPa °C\(^{-1}\)) at air temperature \( T_{air} \), and is given by the following equation:

\[ \Delta = \frac{4098e_s}{(237.3+T)^2} \]  

(B.54)

Where \( e_s \) is the saturated vapor pressure (kPa) and \( T \) is the air temperature (°C). Saturated vapor pressure is calculated using the following equation:
\[ e_s = 611 \exp \left( \frac{17.27 \cdot T}{237.3 + T} \right) \]  

(B.55)

In Equation B.53, \( E_r \) is the evaporation rate (mm/day) computed from the net radiation and is calculated using Equation B.56:

\[ E_r = \frac{H_{\text{net}}}{l_v \rho_w} \]  

(B.56)

Where \( H_{\text{net}} \) is the net radiation (MJ m\(^{-2}\) d\(^{-1}\)); \( l_v \) is the latent heat of vaporization (MJ/kg); and \( \rho_w \) is the density of water taken as 1000 (kg/m\(^3\)). The latent heat of vaporization \( l_v \) is a function of temperature and can be calculated using the following equation by Harrison (1963):

\[ l_v = 2.501 - 2.361 \times 10^{-3} \cdot T \]  

(B.57)

In Equation B.57, \( T \) is the mean daily air temperature (°C). Evaporation involves the exchange of both latent heat and sensible heat between the evaporating body and the air. The psychrometric constant, \( \gamma \), represents a balance between the sensible heat gained from air flowing past a wet bulb thermometer and the sensible heat converted to latent heat (Brunt, 1952) and is calculated:

\[ \gamma = \frac{c_p P}{0.622 \cdot l_v} \]  

(B.58)

Where \( \gamma \) is the psychrometric constant (kPa °C\(^{-1}\)), \( c_p \) is the specific heat of moist air at constant pressure \((1.013 \times 10^{-3} \text{ MJ kg}^{-1} \cdot \text{°C}^{-1})\), \( P \) is the atmospheric pressure (kPa), and \( l_v \) is the latent heat of vaporization (MJ kg\(^{-1}\)). Calculation of the psychrometric constant requires a value for atmospheric pressure. WeatherGEN estimates atmospheric pressure using an equation developed by Doorenbos and Pruitt (1977) from mean barometric pressure data at a number of East African sites:

\[ P = 101.3 - 0.1152 \cdot E_l + 0.544 \times 10^{-6} E_l^2 \]  

(B.59)
Where $P$ is the atmospheric pressure (kPa) and $E_l$ is the elevation (m). The variable $E_a$ (mm/day) in Equation B.53 is the evaporation rate computed from the aerodynamic method and is given by the following equation:

$$E_a = B \cdot (e_{as} - e_a) \quad (B.60)$$

Where $B$ is the vapor transfer coefficient (m/Pa·s), which varies from one place to another, $e_{as}$ is the saturation vapor pressure (kPa) and $e_a$ is the vapor pressure (kPa). The saturated vapor pressure is estimated using Equation 4.55 and the vapor pressure is estimated by multiplying the saturated vapor pressure by the relative humidity. The equation for estimating $B$ was first proposed by Dalton in 1802:

$$B = \frac{0.622k^2\rho_a u_2}{pp \rho_w [ln(z_2/z_0)]^2} \quad (B.61)$$

Where $k$ is the von Karman constant set equal to 0.4, $\rho_a$ is the density of air (kg/m³), $u_2$ is the wind speed (km/hr), $p$ is the atmospheric pressure (kPa), $\rho_w$ is the density of water, which is set at 1000 kg/m³, $z_2$ is the height at which wind speed was measured at the climate station, and $z_0$ is the roughness of the surface at which the wind speed was measured.

The density of air is calculated using the ideal gas law as follows:

$$\rho_a = \frac{p}{R_aT} \quad (B.62)$$

Where $p$ is atmospheric pressure (kPa), $\rho_a$ is the density of air (kg/m³), $R_a$ is the gas constant for moist air (J/kg·k) and $T$ is the air temperature in Kelvin. The gas constant for moist air $R_a$ is calculated using the following equation:

$$R_a = 287 \cdot (1 + 0.608 \cdot q_v) \quad (B.63)$$
Where the variable $q_v$, is the specific humidity in Equation 4.63. According to Equation 4.63 the gas constant of moist air increases with specific humidity, but even for a large specific humidity, the difference between the gas constants for moist and dry air is only about 2 percent (%). The specific humidity is the mass of water vapor per unit mass of moist air and is approximated as follows:

$$q_v = 0.622 \cdot \frac{e}{p}$$  \hspace{1cm} (B.64)

Where $e$ is the vapor pressure (kPa) and $p$ is the atmospheric pressure (kPa). Equation B.53 is the basic equation for the combination method of computing evaporation, which was first developed by Penman (1948). The combination method of calculating evaporation from meteorological data is the most accurate method when all the required data are available and the assumptions are satisfied. The chief assumptions of the energy balance are that steady state energy flows prevails and that changes in heat storage over time in the water body are not significant. This assumption limits the application of the method to daily time intervals or longer, and to situations not involving large heat storage capacity, such as large lake processes. However when applied to watersheds, studies have shown that the most accurate estimates of potential evaporation with the combined method are made when evaporation is calculated on an hourly basis and summed to obtain the daily values. Calculating the evaporation with the combined method using mean daily values can potentially lead to significant errors. These errors result from diurnal distributions of wind speed, relative humidity, and net radiation that in combination create conditions which the daily averages do not replicate. The main assumption of the aerodynamic method is associated with the form of the vapor transfer coefficient $B$, in Equation B.61. Many empirical forms of $B$ have been proposed locally fitted to observed wind and other meteorological data.

When some of the required data in the combined method are unavailable, simpler evaporation equations requiring fewer variables are used (American Society of Civil Engineers, 1973; Doorenbos and Pruitt, 1977). For evaporation over large areas, energy balance considerations largely govern the evaporation rate. For such cases Priestley and Taylor (1972) found that the second term of the combination equation (Equation B.53) is approximately 30 percent of the first, so that Equation B.53 can be written as the Priestley – Taylor evaporation equation:
$E = \alpha \frac{\Delta}{\Delta + \gamma} E_r$  \hspace{1cm} (B.65)

Where $\alpha$ is equal to 1.3. Other investigators have confirmed the validity of this approach, with values of $\alpha$ varying slightly from one location to another. The Priestley-Taylor equation provides potential evaporation estimates for low advective conditions. In semi-arid or arid areas where the advection component of the energy balance is significant, the Priestley-Taylor equation will underestimate potential evaporation. Pan evaporation data provides the best indication of nearby open water evaporation where such data are available. The observed values of pan evaporation are multiplied by a pan factor between 0 and 1, to convert them to equivalent open water evaporation values. The pan factor is approximately 0.7, however, this factor varies by season and location.

**B.2.9 Elevation Bands**

Orographic precipitation is a significant phenomenon in certain areas of the world including North America and Canada. To account for orographic effects on both precipitation and temperature, the weather generator allows up to 10 elevation bands to be defined within each sub-basin. Precipitation and maximum and minimum temperatures are calculated for each band as a function of the respective lapse rate and the difference between the gauge elevation and the average elevation specified for the band. The methodology used within the Weather Generator is the same used in SWAT (Nietsch, et al., 2005). For precipitation,

$$R_{band} = R_{day} + (El_{band} - El_{gauge}) \cdot \frac{plaps}{1000}$$ \hspace{1cm} (B.66)

where $R_{band}$ is the precipitation falling in the elevation band (mm H2O), $R_{day}$ is the precipitation recorded at the gage or generated from gage data (mm H2O), $El_{band}$ is the mean elevation in the elevation band (m), $El_{gauge}$ is the elevation at the recording gage (m), $plaps$ is the precipitation lapse rate (mm H2O/km), and 1000 is a factor needed to convert meters to kilometres. For temperature,
\[ T_{mx,band} = T_{mx,day} + (El_{band} - El_{gage}) \cdot \frac{tlaps}{1000} \]  

(B.67)

\[ T_{mn,band} = T_{mn,day} + (El_{band} - El_{gage}) \cdot \frac{tlaps}{1000} \]  

(B.68)

\[ T_{av,band} = T_{av,day} + (El_{band} - El_{gage}) \cdot \frac{tlaps}{1000} \]  

(B.69)

where \( T_{mx,band} \) is the maximum daily temperature in the elevation band (°C), \( T_{mn,band} \) is the minimum daily temperature in the elevation band (°C), \( T_{av,band} \) is the mean daily temperature in the elevation band (°C), \( T_{mx} \) is the maximum daily temperature recorded at the gage or generated from gage data (°C), \( T_{mn} \) is the minimum daily temperature recorded at the gage or generated from gage data (°C), \( T_{av} \) is the mean daily temperature recorded at the gauge or generated from gauge data (°C), \( EL_{band} \) is the mean elevation in the elevation band (m), \( EL_{gage} \) is the elevation at the recording gauge (m), \( tlags \) is the temperature lapse rate (°C/km), and 1000 is a factor needed to convert meters to kilometres.

Once the precipitation and temperature values have been calculated for each elevation band in the sub-basin, new average sub-basin precipitation and temperature values are calculated:

\[ R_{day} = \sum_{bnd=1}^{b} R_{band} \cdot f_{r_{band}} \]  

(B.70)

\[ T_{mx,day} = \sum_{bnd=1}^{b} T_{mx,band} \cdot f_{r_{band}} \]  

(B.71)

\[ T_{mn,day} = \sum_{bnd=1}^{b} T_{mn,band} \cdot f_{r_{band}} \]  

(B.72)

\[ T_{av,day} = \sum_{bnd=1}^{b} T_{av,band} \cdot f_{r_{band}} \]  

(B.73)

Where \( R_{day} \) is the daily average precipitation adjusted for orographic effects (mm H\(_2\)O), \( T_{mx} \) is the daily maximum temperature adjusted for orographic effects (°C), \( T_{mn} \) is the daily minimum temperature adjusted for orographic effects (°C), \( T_{av} \) is the daily mean temperature adjusted for orographic effects (°C).
orographic effects (°C), \( R_{\text{band}} \) is the precipitation falling in elevation band (mm H\(_2\)O), \( T_{\text{mx,band}} \) is the maximum daily temperature in elevation band (°C), \( T_{\text{mn,band}} \) is the minimum daily temperature in elevation band (°C), \( T_{\text{av,band}} \) is the mean daily temperature in elevation band (°C), \( fr_{\text{band}} \) is the fraction of sub-basin area within the elevation band, and \( b \) is the total number of elevation bands in the sub-basin.

**B.2.10 Climate Change**

The impact of global climate change on water supply is a major area of research. Climate change scenarios can be simulated within the weather generator model by manipulating the climatic input that is read into the model (precipitation, temperature, solar radiation, relative humidity, wind speed, potential evaporation and weather generator parameters). A less time-consuming method is to set adjustment factors for the various climatic inputs. The weather generator model will allow users to adjust precipitation, temperature, solar radiation, and relative humidity in each sub-basin. The methodology employed in WeatherGEN is the same as that used in the SWAT model (Nietsch et al., 2005). The alteration of precipitation, temperature, solar radiation and relative humidity are as follows:

\[
\begin{align*}
R_{\text{day}} &= R_{\text{day}} + \left(1 + \frac{adj_{\text{pcp}}}{100}\right) \\
T_{\text{mx}} &= T_{\text{mx}} + adj_{\text{tmp}} \\
T_{\text{mn}} &= T_{\text{mn}} + adj_{\text{tmp}} \\
T_{\text{av}} &= T_{\text{av}} + adj_{\text{tmp}} \\
H_{\text{day}} &= H_{\text{day}} + adj_{\text{rad}} \\
R_{h} &= R_{h} + adj_{\text{hmd}}
\end{align*}
\]
Where $R_{day}$ is the precipitation falling in the sub-basin on a given day (mm H$_2$O), and $adj_{pcp}$ is the percent (%) change in rainfall. $T_{mx}$ is the daily maximum temperature (°C), and $adj_{tmp}$ is the change in temperature (°C). $T_{mn}$ is the daily minimum temperature (°C), and $adj_{tmp}$ is the change in temperature (°C). $T_{av}$ is the daily mean temperature (°C), and $adj_{tmp}$ is the change in temperature (°C). $H_{day}$ is the daily solar radiation reaching the earth’s surface (MJ m$^{-2}$), and $adj_{rad}$ is the change in radiation (MJ m$^{-2}$ d$^{-1}$). $R_{h}$ is the relative humidity for the day expressed as a fraction, and $adj_{hmd}$ is the change in relative humidity expressed as a fraction. The weather generator allows the adjustment terms to vary from month to month so that the user is able to simulate seasonal changes in climatic conditions.

**B.2.11 Forecasting**

Being able to assess the impact of predicted weather on a watershed is useful for some applications. The weather generator allows a forecast period to be defined in a simulation. During the first portion of the simulation, climatic data is read into the model from a spreadsheet or is generated by the weather generator model using long-term monthly averages as input. When the simulation reaches the first day of the forecast period (defined by FRCSTDDAY) the model replaces the monthly long-term generator averages with averages provided for the forecast period. All climatic data required by the model is generated during the forecast period.

Forecast data is provided by Environment Canada and the US National Weather Service and is summarized by region. Within the weather generator model multiple regions can be specified within one simulation. Alternative temperature and precipitation averages can be defined for the forecast period to generate daily precipitation and temperature values. For temperature, the average daily, average maximum and minimum air temperature are defined. For precipitation, the average amount of precipitation falling by month along with the wet/dry probabilities and the average number of days of precipitation expected are defined. The forecast period must be simulated a number of times to obtain a distribution of possible weather scenarios. A minimum of 20 cycles is recommended, one for each temporal pattern. The only difference between forecast scenarios is the value of the random number used to generate daily weather values.
For the weather generator model the order of the computations are as follows: Daily precipitation; daily mean, maximum and minimum temperatures; daily rainfall; daily snowfall; daily relative humidity; daily wind speed; daily solar radiation; daily potential evaporation; hourly/sub-hourly rainfall; hourly/sub-hourly snowfall; hourly temperature; hourly solar radiation; and hourly potential evaporation.
APPENDIX C

Flows versus Velocities for the Credit River Watershed

Figure C-1 Flow versus velocity for the Credit River near Orangeville stream gauge

Figure C-2 Flow versus velocity for the Credit River Erin Branch above Erin stream gauge
Figure C-3 Flow versus velocity for the Credit River near Cataract stream gauge

Figure C-4 Flow versus velocity for the Credit River at Boston Mills stream gauge
Figure C-5 Flow versus velocity at Black Creek below Acton stream gauge

Figure C-6 Flow versus velocity for the Credit River West Branch at Norval stream gauge
Figure C-7 Flow versus velocity for the Credit River at Norval stream gauge
APPENDIX D
Variable Plots for Watershed Parameters

Figure D-1 Plot of main channel length ($L_c$) versus observed time of concentration ($T_c$)

Figure D-2 Plot of main channel slope ($S_c$) versus observed time of concentration ($T_c$)
Figure D-3 Plot of elongation ratio (E) versus observed time of concentration (T_c)

Figure D-4 Plot of main channel slope (S_c) versus main channel length (L_c)
Figure D-5 Plot of elongation ratio \((E)\) versus main channel length \((L_c)\)

Figure D-6 Plot of elongation ratio \((E)\) versus main channel slope \((S_c)\)
Figure D-7 Plot of main channel length ($L_c$) versus observed surface storage recession constant ($K_{surf}$).

Figure D-8 Plot of main channel slope ($S_c$) versus observed surface storage recession constant ($K_{surf}$).
Figure D-9 Plot of elongation ratio ($E$) versus observed surface storage recession constant ($K_{surf}$)

Figure D-10 Plot of main channel slope ($S_c$) versus main channel length ($L_c$)
Figure D-11 Plot of elongation ratio ($E$) versus main channel length ($L_c$)

Figure D-12 Plot of elongation ratio ($E$) versus main channel slope ($S_c$)
Figure D-13 Plot of maximum flow distance ($MFD$) versus observed subsurface flow recession constant ($K_{subs}$)

Figure D-14 Plot of watershed slope ($S_w$) versus observed subsurface flow recession constant ($K_{subs}$)
Figure D-15 Plot of hydraulic conductivity ($K_{eff}$) versus observed subsurface flow recession constant ($K_{subs}$)

Figure D-16 Plot of watershed slope ($S_w$) versus maximum flow distance ($MFD$)
Figure D-17 Plot of hydraulic conductivity ($K_{\text{eff}}$) versus maximum flow distance ($MFD$)

Figure D-18 Plot of hydraulic conductivity ($K_{\text{eff}}$) versus watershed slope ($S_w$)

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Figure D-19 Plot of maximum flow distance (MFD) versus observed groundwater flow recession constant ($K_{gw}$)

Figure D-20 Plot of watershed slope ($S_w$) versus observed groundwater flow recession constant ($K_{gw}$)
Figure D-21 Plot of hydraulic conductivity ($K_{eff}$) versus observed groundwater flow recession constant ($K_{gw}$)
APPENDIX E

Plots of Manning’s Roughness Coefficient “n” versus Velocity

Figure E-1 Plot of velocity versus Manning’s roughness coefficient (n), for the Credit River near Orangeville stream gauge

Figure E-2 Plot of velocity versus Manning’s roughness coefficient (n), for the Credit River Erin Branch above Erin stream gauge
Figure E-3 Plot of velocity versus Manning’s roughness coefficient ($n$), for the Credit River near Cataract stream gauge

Figure E-4 Plot of velocity versus Manning’s roughness coefficient ($n$), for the Credit River at Boston Mills stream gauge
Figure E-5 Plot of velocity versus Manning’s roughness coefficient ($n$), for the Black Creek below Acton stream gauge

Figure E-6 Plot of velocity versus Manning’s roughness coefficient ($n$), for the Credit River West Branch at Norval stream gauge
Figure E-7 Plot of velocity versus Manning's roughness coefficient \( (n) \), for the Credit River at Norval stream gauge