Evaluating the Potential for Low Impact Development to Mitigate Impacts of Urbanization on Groundwater Dependent Ecosystems using MIKE SHE

by

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ABSTRACT

EVALUATING THE POTENTIAL FOR LOW IMPACT DEVELOPMENT TO MITIGATE IMPACTS OF URBANIZATION ON GROUNDWATER DEPENDENT ECOSYSTEMS USING MIKE SHE

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Groundwater dependent ecosystems (GDEs), including wetlands and river baseflow systems, are a topic of substantial scientific study. The degradation of GDEs due to urbanization has been well documented. An altered hydrologic regime, through increased impervious area resulting in a flashier hydrologic regime with lower troughs, higher peaks, and quicker changes, has been recognized as a main factor affecting ecological condition. Yet studies on GDEs rarely include a hydrologic modelling component.

In this study, the conjunctive hydrologic model MIKE SHE was used to simulate the Lovers Creek subwatershed near Barrie, ON. The hydrologic regime was simulated for pre-development (natural), current (urbanized), and various low-impact development (LID) land use scenarios. The results were linked to the ecological condition via the $T_{Qmean}$ metric, which has been used in the literature to relate the hydrologic and ecological conditions of streams.

The highest percentage LID scenario restored, on average, 11% of the reduction in $T_{Qmean}$ that occurred from pre-development to urbanized conditions, indicating that LID has the potential to protect GDEs in urbanized watersheds. It is expected that the effect of LID would be amplified if considered on a more local scale within a predominantly high density urban area. Recommendations for future modelling efforts to evaluate GDEs and represent LID are made.
ACKNOWLEDGEMENTS

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1. **INTRODUCTION**

Groundwater dependent ecosystems (GDEs) are important for overall environmental management and have become the subject of considerable scientific investigation, even prompting dedicated editions of the *Australian Journal of Botany* (Volume 54, Issue 2, 2006) and *Freshwater Biology* (Volume 54, Issue 4, 2009). GDEs provide many ecosystem services such as flood attenuation, water quality treatment, and supporting biodiversity (Eamus et al., 2006; Ramsar, 2012). The hydrologic regime, either a natural flow regime in rivers (Poff et al., 1997) or the hydroperiod in wetlands (Reinelt et al., 1998), is a crucial factor controlling the structure and function of these ecosystems. Development of land and water resources alters the hydrologic regimes of GDEs. Urbanization, agriculture, and groundwater abstraction are often reported to alter the hydrologic regime by increasing runoff and reducing groundwater recharge, groundwater levels, groundwater discharge, and baseflow (Booth & Bledsoe, 2009; Walsh et al., 2005; Acreman et al., 2000; Zektser et al., 2005). Alteration of the hydrologic regime as a result of urbanization can cause loss of GDE structure and function. The total and effective imperviousness (TI and EI) of a watershed have been linked to ecological condition (Walsh et al., 2005; Booth & Bledsoe, 2009; Wenger et al., 2009). Low-impact development (LID) serves to directly reduce the amount of EI in a watershed, and therefore has potential as a strategy to mitigate the impacts of urbanization on GDEs (Walsh et al., 2005; Wenger et al., 2009). Strategies for the effective management of GDEs that recognize the links between altered hydrologic conditions and ecological responses have been recommended in scientific literature (Poff et al., 2010). Hydrologic modelling tools have become sophisticated enough to adequately represent the interconnected nature of the surface water and groundwater and accurately predict
hydrologic conditions of GDEs under various land use and water management strategies (Thompson et al., 2004). Nevertheless, most studies on GDEs are completed by biologists and ecologists and do not include a hydrologic modelling component.

This study used the integrated hydrologic model MIKE SHE to simulate hydrologic conditions in the Lovers Creek subwatershed and linked the altered hydrologic regime to ecological responses using information gained from the literature review on the hydrologic requirements of GDEs. It examined the impact of urbanization and LID on streamflow and groundwater discharge to the stream in the Lovers Creek subwatershed near Barrie, Ontario. A MIKE SHE model was developed to simulate pre-development, current (urbanized), and various LID land use scenarios. This paper discusses the procedure and results as well as the difficulties encountered. It provides recommendations for future efforts to develop and use integrated hydrologic modelling as a tool for management of GDEs.
2. LITERATURE REVIEW

2.1. GROUNDWATER-DEPENDENT ECOSYSTEMS

Groundwater dependent ecosystems (GDEs) are ecosystems whose ecological health relies on the availability of groundwater resources. They include, but are not limited to, river systems such as aquatic, hyporheic, and riparian ecosystems as well as wetlands that are dependent on groundwater influx all or part of the time (Boulton & Hancock, 2006; Murray et al., 2003; Sophocleous, 2007). GDEs can be classified into three different categories based on their required groundwater regime: 1. Subsurface groundwater-dependent ecosystems (SGDEs); 2. Ecosystems dependent on the surface expression of groundwater; and 3. Ecosystems dependent on the subsurface presence of groundwater (Eamus et al., 2006).

The first category includes aquifer and cave ecosystems where stygobites (obligate groundwater fauna) reside in subterranean areas with well-developed voids, typically in karstic systems, but they have also been found in alluvial and fractured rock aquifers (Humphreys, 2006). The hyporheic zone of rivers can also be considered a SGDE, since it often supports stygobites (Eamus et al., 2006); however, this eco-tone is dominated by exchanges between surface water and groundwater and provides functions important to surface ecosystems as well (Boulton & Hancock, 2006), so it may also be included in the second category of GDEs. SGDEs, as a sub-category of GDEs, have themselves been a growing topic of research (Humphreys, 2006; Tomlinson & Boulton, 2008).

The second category of GDEs includes some types of wetlands and river baseflow systems (Eamus et al., 2006). The surface expression of groundwater may be continuous, as is the case for stream ecosystems that flow year round, or periodic, as may be the case for
floodplains that require inundation to maintain fish spawning and nursery habitats. The ecosystem does not necessarily have to use the water while it is above the surface. In many cases, the water may soak back into the root zone of the soil and be available for plant roots (Eamus et al., 2006).

The third category of GDEs primarily includes terrestrial communities with plants whose roots access the groundwater through the capillary fringe (Eamus et al., 2006). This category includes phreatophytes, which are deep-rooted plants that access the groundwater directly. Some types of wetlands may also be dependent on subsurface presence of groundwater. If the water table is near the surface, plants can access the water. Even where the water table drops below the rooting zone of wetland plants, groundwater may still supply soil moisture to the rooting zones via unsaturated zone processes.

2.1.1. Degrees of Dependency

GDEs can possess varying degrees of dependency on the groundwater regime. In general this dependency ranges from obligate to facultative (Murray et al., 2003; Eamus et al., 2006), but it is virtually impossible to assess because of complex environmental factors and natural variability (Boulton & Hancock, 2006). The degree of groundwater dependency may vary between plant and animal communities within an ecosystem. This makes predicting ecological changes from a loss of groundwater availability or change to groundwater regime complicated and difficult.

The degree of dependency also varies temporally, with some GDEs relying on groundwater availability all the time, and others only occasionally. The frequency and timing of groundwater availability, therefore, are important variables. Furthermore, even obligate GDEs need not be reliant on groundwater availability all the time; those with relatively infrequent
dependency (e.g. 6 months availability every 10 – 20 years) or frequent but short-lived dependency (e.g. 1 month of every year) may be considered obligate (Eamus et al., 2006). In general, obligate GDEs require groundwater availability either continuously, seasonally, or episodically (Eamus et al., 2006). A consequence of the complex nature and timing of dependency is that loss of groundwater availability may not be evident for several years, when impacts start to become evident on the surface (Eamus et al., 2006).

2.1.2. Ecosystem Response Functions

GDEs and their plant and animal communities not only have different degrees of dependency on groundwater availability, they also exhibit different responses to changes in the groundwater regime. Two alternative types of ecosystem response functions were proposed by Murray et al. (2003) to relate the health of a GDE to the groundwater availability: step (or threshold) and linear. A step response function for a particular measure of ecosystem health would show little change as groundwater availability decreases or increases until some threshold level is reached, after which a substantial change occurs. A linear response function for a particular measure of ecosystem health would show a gradual decline as groundwater availability decreases.
Figure 1: Example of linear and step response functions of ecosystem health and groundwater availability defined by Murray et al. (2003).

The sensitivity to groundwater levels varies, with some sites showing minimal ecological impacts even with large changes and other sites showing significant ecological impacts even with minimal changes to the groundwater levels (Acreman et al., 2000). This may be indicative of, for example, plant communities being sensitive to changes in the groundwater level, or groundwater levels being near a threshold. Thus, when predicting the effect of a change in groundwater availability on a GDE, it is important to understand the relationship between the GDE and groundwater availability. Although the depth to groundwater table does not directly influence ecosystem communities, it is a more useful parameter than the factors that do (e.g. soil water content, nutrient availability, soil aeration) because it is related to these other controlling variables and is more easily measured (Wheeler et al., 2004). Furthermore, using the depth to the groundwater table allows links to be made to hydrologic models that generate this output.

2.1.3. Ecosystem Services

GDEs provide many ecosystem services including water management, flood and sediment attenuation, water purification, regulate runoff and water supply, bank and shoreline stabilization, support of a high biodiversity of flora and fauna, and pollution control (Eamus et
al., 2006; Murray et al., 2006; Kløve et al., 2011b; Ramsar, 2012). Aquifer and cave ecosystems, the hyporheic zone, and river floodplains all have roles in nutrient cycling, while Type 3 GDEs that depend on the subsurface presence of groundwater affect air quality and carbon dioxide sequestration (Kløve et al., 2011b). Figure 2, from Murray et al. (2006) summarizes the ecosystem services of the three different types of GDEs.

<table>
<thead>
<tr>
<th>Ecosystem function</th>
<th>GW</th>
<th>BG</th>
<th>AG</th>
<th>Examples of ES provided by function</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gas regulation</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>UV protection by atmospheric gases, maintenance of air quality</td>
</tr>
<tr>
<td>Climate regulation</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>Maintenance of favourable climate</td>
</tr>
<tr>
<td>Disturbance prevention</td>
<td>?</td>
<td>✓</td>
<td>✓</td>
<td>Storm and flood protection</td>
</tr>
<tr>
<td>Water regulation</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>Water transport, drainage and irrigation</td>
</tr>
<tr>
<td>Water supply</td>
<td>✓</td>
<td></td>
<td>✓</td>
<td>Provision and storage of water for consumption, minimisation of evapotative losses</td>
</tr>
<tr>
<td>Soil retention</td>
<td>✓</td>
<td></td>
<td></td>
<td>Prevention of salination and erosion</td>
</tr>
<tr>
<td>Soil formation</td>
<td>✓</td>
<td></td>
<td></td>
<td>Maintenance of productive soils and amble land</td>
</tr>
<tr>
<td>Nutrient regulation</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>Maintenance of healthy and productive ecosystems</td>
</tr>
<tr>
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<td>✓</td>
<td></td>
<td>Pollution control, filtering</td>
</tr>
<tr>
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<td></td>
<td>✓</td>
<td>Pollination of wild plants and crops</td>
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<tr>
<td>Biological control</td>
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<td></td>
<td>✓</td>
<td>Control of pests and diseases</td>
</tr>
<tr>
<td>Refugium function</td>
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<td>✓</td>
<td>✓</td>
<td>Maintenance of commercially important populations</td>
</tr>
<tr>
<td>Nursery function</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>Breeding sites maintain game stocks</td>
</tr>
<tr>
<td>Food</td>
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<td>Provision of fish, game, fruits</td>
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<tr>
<td>Raw materials</td>
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<td>✓</td>
<td>✓</td>
<td>Provision of building materials, energy and mineral resources</td>
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<tr>
<td>Genetic resources</td>
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<td>✓</td>
<td>✓</td>
<td>Improve crop resistance to pathogens and pests</td>
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<td>Medicinal resources</td>
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<td>✓</td>
<td>✓</td>
<td>Drugs and pharmaceuticals</td>
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<td>✓</td>
<td>✓</td>
<td>Resources for fashion, pets, worship</td>
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<td>✓</td>
<td>Enjoyment of scenery</td>
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<td>Recreation</td>
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<td>✓</td>
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<td>Eco-tourism and outdoor sports</td>
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<tr>
<td>Cultural and artistic information</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>Use of nature as motive in books, film, advertising etc.</td>
</tr>
<tr>
<td>Spiritual and historic information</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>Use of nature for religious or historic purposes</td>
</tr>
<tr>
<td>Science and education</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>Use of nature for research and education</td>
</tr>
</tbody>
</table>

**Figure 2: Services provided by the three different types of groundwater dependent ecosystems (Murray et al., 2006).**

Flood attenuation is provided by absorption and storage of precipitation and runoff in surface depressions and soils and reduced runoff velocity. Water purification and pollutant removal may be done by plants. For example *Azolla* (water fern) is able to absorb and store heavy metals (Ramsar, 2012). Wetlands support biodiversity and provide habitat for many species of plants and animals dependent on groundwater. Stygofauna, which exhibit obligate groundwater dependency, are often found in river baseflow systems near springs where there is GW-SW interaction (Boulton & Hancock, 2006).
2.1.4. Growing Recognition of GDEs

Several countries have undertaken dedicated projects to study and protect groundwater resources and GDEs. The PASCALIS (Protocol for the Assessment and Conservation of Aquatic Life in the Subsurface) project in Europe is a notable example. Sampled ecosystems in France, Spain, Portugal, Italy, Slovenia, and Belgium indicated several new stygofauna (groundwater invertebrates) and high biodiversity in many groundwater resources (Boulton A. J., 2009). The Agriculture and Resource Management Council of Australia and New Zealand (ARMCANZ) has also produced a policy statement defining principles for provisions of water to protect water dependent ecosystems based on the best available scientific information on the required water regimes to maintain ecological values (ARMCANZ, 1996; Nevill, 2008). The European Water Framework Directive and Habitat Directive recognizes the importance of achieving good ecological status and may be applied to the protection and conservation of GDEs (Kløve et al., 2011). Despite significant research into GDEs, the hydrologic requirements of a specific GDE often remain unknown. While researchers continue to uncover and provide valuable information regarding the hydrologic requirements of GDEs, governments and managers need to concurrently develop tools that can help to meet defined targets. LID and integrated modelling are both tools that can be used for this purpose. Integrated hydrologic modelling may be used to predict whether the hydrologic requirements of a GDE can be met into the future, and LID may be a means of meeting requirements despite ongoing development within the watershed. However, this can only be achieved in conjunction with research into the hydrologic requirements of GDEs.
2.2. **THE HYDROLOGIC REGIME AND ECOLOGICAL CONDITION**

The concept of a natural hydrologic regime is not new; Poff et al. (1997) discussed the concept relating to rivers, describing it as the spatial and temporal variability of streamflow and claiming it to be vital to the ecological health of a stream. They characterize the regime by the magnitude, frequency, duration, timing (predictability), and flashiness (rate of change) of stream discharge. Similarly, wetlands have a natural hydrologic regime that may be defined by either surface water or groundwater levels, or both, and characterized in terms of magnitude, frequency, duration, timing, and flashiness. Reinelt et al. (1998) examined hydrologic conditions of wetlands using the three parameters of water depth (magnitude), water level fluctuation (relates to magnitude, frequency, and flashiness), and length of summer dry period (relates to duration and timing), which they used to define the hydropereiod of a wetland. Streamflow can be considered a “master variable” that controls the ecological integrity of riverine ecosystems (Poff et al., 1997) and the hydoperiod of a wetland is critical for controlling wetland ecology (Reinelt et al. 1998). In either case, the natural hydrologic regime controls the ecological condition of the ecosystem and changes to this regime cause degradation of GDEs.

Changes to hydrologic regime can also result in biological degradation directly or indirectly through influences on other processes. Booth and Bledsoe (2009) identified that chemical, physical, and biological processes dictate stream biological responses and can be classified into five groups: habitat structure, flow regime, water quality and toxicity, energy source, and biotic interactions. Fischenich (2003) identified physical aspects such as hydrology and sediment movement as driving forces for chemical and biological processes. Therefore, while confounding impacts to ecological condition may occur as a result of changes to chemical
and biological processes, the hydrologic regime is a particularly important factor influencing ecological condition.

The ecological response to hydrologic alteration depends on the type of ecosystem, the hydrologic regime to which it is accustomed, and the tolerance of its species to hydrologic alteration. The biota, in turn, may also affect the physical and hydrologic processes of the ecosystem, though this tends to be to a lesser extent (Fischenich, 2003). These can be very difficult relationships to quantify because of environmental complexity within an ecosystem and difficulties in accurately determining hydrologic alteration, but in general, hydrologic alterations illicit a decline in the ecological condition. A review of the scientific literature by Poff & Zimmerman (2010) revealed that 92% of studies reported decreased ecological condition with hydrologic alteration; macroinvertebrates and riparian vegetation showed a mixed response, both increasing and decreasing in response to elevated and reduced flows; fish abundance consistently declined in response to increased and reduced flow magnitude. Neither high nor low portions of a hydrologic regime can be ignored as extreme events influence and determine species composition and processes (Poff & Zimmerman, 2010). Increased frequency and magnitude of flood flows in river systems cause erosion, incision, and stream widening, which reduces the low flow water depth and causes a loss of longitudinal connectivity and/or habitat area.

### 2.3. Degradation of GDEs

Anthropogenic threats to GDEs come from land use changes and development of water resources. The hydrologic regime of GDEs may be altered by many human activities including urbanization, groundwater abstraction, and agriculture. In addition to changing the hydrologic regime, these activities may also affect GDEs through degradation of water quality, loss of riparian buffers, and other impacts.
Conversion of natural land cover to urban and agricultural land uses modifies the water balance for a watershed, creating less evapotranspiration, more runoff, and less infiltration. Furthermore, water quality issues tend to coincide with hydrologic changes. Runoff from agricultural and urban areas carries a variety of contaminants to receiving waters. Development of water resources and increases in water takings from both surface and groundwater sources, which tend to coincide with land use change, further stress the hydrologic balance. Groundwater abstraction causes drawdown, lowering the groundwater table and reducing or reversing the hydraulic gradient between the saturated zone and a GDE, causing a loss of groundwater availability in the GDE.

2.3.1. Effects of Urbanization

It is recognized within scientific literature that an altered hydrologic regime as a result of urbanization is a major source of degradation of ecological conditions in streams. Urbanization alters runoff-generating processes as a result of increased impervious surface area, vegetation clearing, soil compaction, and ditching and draining (Booth & Bledsoe, 2009). In addition, stormwater drainage networks that are more efficient than natural pathways alter runoff conveyance processes. These changes create a system which has an increased volume of stormwater runoff and higher peak flows, as well as increased flashiness, as peak storm flows occur (and recede) more quickly (Walsh et al., 2005). Furthermore, decreased infiltration and subsequent reductions in groundwater recharge and baseflow are also prevalent. The effects of urbanization on streamflow are well documented and the phrase “urban stream syndrome” (Walsh et al., 2005; Meyer, Paul, & Taulbee, 2005) has been used to describe the consistently degraded ecological condition of streams in urbanized landscapes. Other, non-hydrologic
symptoms include “elevated concentrations of nutrients and contaminants, altered channel morphology and stability, and reduced biotic richness” (Walsh et al., 2005).

An important factor contributing to the urban stream syndrome is the drainage connection (DC), or the proportion of the total impervious area that is directly connected to the stream (or wetland) via a drainage network of stormwater pipes and channels. This raises the effective imperviousness (EI), which is the amount of impervious area that has a direct hydraulic connection with the receiving stream or wetland; precipitation is converted into runoff that is efficiently transported to the receiving water. In contrast, precipitation on impervious areas that are disconnected may have the opportunity to infiltrate and be slowly routed to the stream or wetland as groundwater discharge or interflow.

The total imperviousness (TI) of a watershed is often used as an indicator of degradation of urban streams and has been shown to be highly correlated with the ecological condition of a stream. Watersheds with low TI may have streams with a wide range of ecological conditions, but watersheds with a high TI possess streams with uniformly degraded ecological conditions (Wenger et al., 2009). A shortcoming of using TI as a metric, however, is that it includes impervious surfaces that drain to pervious areas and so may not contribute directly to storm runoff (Booth & Bledsoe, 2009). This shortcoming can be accounted for by utilizing the EI, which is often a better indicator of ecological condition than TI since it includes only those surfaces with direct hydraulic connection to streams (Wenger et al., 2009). Walsh et al. (2005) studied the ecological condition of 15 streams and found that the ecological condition degraded as EI increased up to 0.14, beyond which no further degradation was observed.

It is well-established that urbanization increases the magnitude, frequency, and duration of a streamflow regime (Poff et al., 1997; Bledsoe & Watson, 2001; Booth et al, 2004; Walsh et
al., 2005; Beechie et al., 2009; Booth & Bledsoe, 2009). The peak storm discharge can be increased by a factor of 2 to 5; the duration of a specific flow magnitude can be 5 to 10 times longer; the frequency of sediment-moving and habitat-forming discharges is increased by a factor of 10 or more (Booth & Bledsoe, 2009). This can have serious consequences on the stream ecology. The increased frequency and magnitude of flows causes the stream channel to widen, incise, or both, which can then be a source of excess sediment further downstream (Bledsoe & Watson, 2001).

The changes to stream form have consequences for its ecological condition. As streams widen or incise and low flows become more frequent, longitudinal connectivity of the stream may be lost (i.e. the depth of water in the stream will be reduced), which may affect fish species. As banks erode, prime habitat areas for biological communities may be lost. An incised stream channel may lower the groundwater table, which may result in a reduction of soil moisture in riparian wetlands and a loss of wetland species (Loheide & Booth, 2011). Even the loss of a thalweg may create harmful hydraulic conditions for some species, resulting in a loss of available habitat.

Baseflow is a critical attribute for many GDEs and their associated ecological functions. In urban areas it is often altered from pre-urbanized conditions (Walsh et al., 2005; Booth & Bledsoe, 2009), but the response is not consistent. Urbanization reduces baseflow due to reduced groundwater infiltration, but this may be counteracted by leaking infrastructure (Walsh et al., 2005) or irrigation using water imported from other catchments or from deep aquifers (Booth & Bledsoe, 2009). Unless these counteracting effects are considerable, however, it is likely that urban streams will have a reduced groundwater discharge and baseflow compared to the natural hydrologic regime.
Reduced groundwater discharge to streams leads to reduced flow through the hyporheic zone, which is an integral part of fluvial ecosystems and a mechanism for exchange processes that are important to ecological condition. Many species are dependent on an active, healthy hyporheic zone for survival. Many invertebrates live in the riffle section of streams and depend on the hyporheic zone for habitat and sustenance and many fish indirectly rely on the hyporheic zone to provide invertebrates for a food supply (Smith, 2005). Furthermore, fish may also rely more directly on hyporheic exchanges for refuge from environmental stressors (e.g. groundwater discharge may regulate stream temperature within tolerance ranges for fish) or for reproduction (e.g. salmonids lay eggs in hyporheic sediments) (Smith, 2005). Furthermore, reduced groundwater discharge will lead to more frequent low flow events during prolonged dry periods, which can impact the longitudinal connectivity of the stream and access to refuge areas.

Degradation of GDEs also occurs through the loss of riparian buffers to streams. Riparian areas provide many important functions to a stream. They store sediment and water in their soil, which reduces flood damage downstream; they cycle nutrients and remove pollutants from overland flow; they maintain biodiversity and wildlife habitat through regulating stream temperature, structure, and sedimentation (National Research Council (U.S.), 2002). Ecologically healthy stream corridors maintain hydrologic connection between the stream channel and the riparian area to provide these services.

Human alterations of watershed, including hydrologic changes, have resulted in significant degradation of riparian areas. Water withdrawals, both surface and groundwater can have serious impacts on riparian areas due to lowering the water table in those areas (National Research Council (U.S.), 2002). Urbanization also degrades riparian areas, primarily through the alteration of the hydrologic regime. Decreased groundwater flow and stream baseflow combined
with higher peak storm flows and the resulting increase in channel cross-sectional area may reduce or remove connectivity of the stream channel and riparian area. Also, in urban watersheds, overland flow tends to be concentrated through smaller portions of the riparian area, reducing its ability to regulate water temperature and remove pollutants (National Research Council (U.S.), 2002).

In urban areas the changes to hydrologic regimes are ubiquitous. Urbanization is known to dramatically alter the hydrologic regime and is a significant source of degradation to GDEs. Thus, to protect GDEs in urban areas, it is important to maintain the natural hydrologic regime of the ecosystem. LID is a potential tool that may be used to achieve such goals.

2.3.2. Effects of Groundwater Abstraction

Groundwater abstraction for use in agriculture, industry, or urban water supply has also been shown to cause degradation of many rivers around the world, usually through the reduction of groundwater discharge that provides baseflow to the stream (Acreman et al., 2000; Petts et al., 1999; Zektser, Loaiciga, & Wolf, 2005; Kirk & Herbert, 2002; Sophocleous M., 2000). Agricultural irrigation accounts for a significant proportion of total groundwater abstraction (Murray et al., 2006), which reduces groundwater discharge to streams and wetlands and the overall availability of groundwater to aquatic ecosystems.

Protection of GDEs requires proper management of groundwater abstraction, allowing for the provision of environmental flows (Richter, 2010). Past river and groundwater management strategies have not achieved this goal. The documented loss of river flows and concomitant loss of healthy aquatic ecosystems is evidence of that failure. Groundwater abstraction permits historically did not consider the effect on surface water ecosystems and while this is changing, environmental uses of water sometimes remain a low priority (Zektser, Loaiciga, & Wolf, 2005).
Furthermore, well defined concepts of groundwater management can sometimes be applied inappropriately. Safe yield is a concept often applied in the management of groundwater resources and is defined as the “quantity of water that can be pumped regularly and permanently without dangerous depletion of the storage reserve” (Alley & Leake, 2004). This concept is often applied as an annual balance between groundwater abstraction and recharge (Sophocleous M., 2000), but without that groundwater abstraction, the groundwater would be available to discharge to GDEs, so balancing abstraction and recharge is insufficient for ecosystem protection. Therefore, unless accompanied by assumptions about acceptable effects of groundwater withdrawals, it may be unsustainable as a long term management strategy (Bekesi, McGuire, & Moiler, 2009).

2.3.3. Hydrologic Metrics

The hydrologic regime can be related to ecological condition through the use of hydrologic metrics that suitably describe the hydrology including magnitude, frequency, flashiness, duration, and timing of flow or water levels. Relationships developed between hydrologic metrics and ecological indicators (e.g. $T_{\text{Qmean}}$ vs. B-IBI) provide a mechanism to examine the impact of hydrologic changes on the ecological condition of a GDE.
Urban stream degradation can be inferred from hydrologic metrics computed from daily streamflow data. $T_{Q\text{mean}}$ and $T_{0.5\text{-yr}}$ are two metrics used by Booth et al. (2004). The $T_{Q\text{mean}}$ is the fraction of days in a single year for which daily discharge exceeds the annual mean daily discharge, $Q_{\text{mean}}$. $T_{0.5\text{-yr}}$ is the fraction of days in a multiple year period for which mean daily discharge exceeds the discharge that is equalled or exceeded on average, twice each year (Booth et al., 2004). $T_{Q\text{mean}}$ reflects the degree of flashiness of a stream hydrograph (Booth, 2005).

Lower values of $T_{Q\text{mean}}$ indicate a flashier, urbanized streamflow regime since the annual mean discharge is not significantly altered, but duration of peak storm event flows are reduced (Booth et al., 2004). $T_{0.5\text{-yr}}$ is similar to $T_{Q\text{mean}}$ but compares daily discharge to the 0.5 year return period discharge rather than the daily mean. This is done because the discharge with a frequency of 2 events per year has geomorphic and biological significance (Booth et al., 2004). The $T_{0.5\text{-yr}}$ metric also reflects the impact of urbanization on the stream hydrology because high flows will occur more frequently, but not increase in duration. The use of these streamflow metrics provides a mechanistic link to understand the causes of biological degradation in urban settings (Booth et al., 2004).
The $T_{Q_{\text{mean}}}$ and $T_{0.5\text{yr}}$ metrics compare the duration and flashiness of a flow regime over the reference flows of $Q_{\text{mean}}$ and $Q_{0.5\text{yr}}$. For analysis of GDEs that rely on groundwater discharge and low water levels, a lower reference flow may be more appropriate. Unfortunately, no such metric has been used in the past and no relationship to ecological condition has been determined. Despite its limitation for evaluating effects at low flows, the $T_{Q_{\text{mean}}}$ metric was considered to be applicable, if not ideal, for evaluation of hydrologic changes to GDEs.

2.4. POTENTIAL RESTORATION AND MITIGATION

2.4.1. Processed-Based Restoration

Restoration of streams in urban catchments has traditionally been attempted by physical enhancement of the stream channel; however, these methods often fail in the long-term because they do not account for hydrology and natural processes (Tompkins & Kondolf, 2003), they do not manage for spatial and temporal variability, and they address the symptoms of degradation but not the causes (Beechie et al., 2009). Process-based restoration techniques have been proposed by several authors (Tompkins & Kondolf, 2003; Fischenich, 2003; Beechie et al., 2009) as a possible alternative to form-based techniques. Process-based restoration seeks to restore the hydrologic regime of a stream ecosystem, recognizing that restoring the function of hydrodynamic character will affect the other functions (e.g. biological communities and process, surface/subsurface water exchange, and chemical processes and nutrient cycles) necessary for a healthy stream ecosystem (Fischenich, 2003). Shortcomings of process-based stream restoration techniques are that they are unpredictable since they rely on the natural processes of the stream, less visible, and short term results are not guaranteed (Tompkins & Kondolf, 2003). LID can be
considered as a process-based stream restoration technique, since it works to restore the natural hydrologic processes of the watershed.

2.4.2. Low Impact Development

Low impact development (LID) is a stormwater management strategy that seeks to mitigate the hydrologic impact caused by urbanization, specifically on the volume and rate of stormwater runoff (LSRCA, 2010), which are important variables in urban streams (Walsh et al., 2005; Wenger et al., 2009). In contrast to conventional stormwater management strategies that treated stormwater as a waste product to be quickly transported away, LID considers stormwater as a resource and tries to manage it as close to its source as possible (USEPA, 2012a). LID accomplishes its goal through storage, infiltration, and evapotranspiration of stormwater at or near its source.

Common types of LID include green roofs, permeable pavement, and bioretention cells. These different LID technologies, while striving for the same objective, do not have the same effect on the hydrologic cycle. Green roofs consist of layers of vegetation and growing medium on top of a conventional roof and allow for storage and enhanced evapotranspiration of stormwater, reducing peak storm flow and runoff volume (CVCA & TRCA, 2010). Permeable pavement is an alternative to traditional pavement that allows for infiltration of stormwater through the pavement surface and detention in subsurface aggregate layers (CVCA & TRCA, 2010). Bioretention cells are vegetated areas that allow for infiltration and storage of stormwater. Both permeable pavements and bioretention may be designed with an underdrain if native soils have low permeability. Runoff volume may be reduced by 85% for bioretention cells with no underdrain and 45% for bioretention cells with an underdrain (CVCA & TRCA, 2010), providing a significant benefit to the water balance of urban areas. Walsh et al. (2005) found that LID maintained hydrologic and ecologic conditions
for all but the largest rain events (~4%) while conventional urban design had significant impacts for
any size of precipitation events. LID has potential to be an effective process-based stream restoration
tool by disconnecting impervious surfaces in urban areas from the drainage network and thereby
reducing the EI of the catchment, which has been related to the ecological condition of urban streams
(see Section 2.3.1).

2.5. MANAGEMENT TO PROTECT AND RESTORE GDEs

Groundwater and surface water are not separate resources; they represent a single,
interconnected resource (Winter et al., 1998). Despite the heavily connected nature of
groundwater and surface water, the two have traditionally been managed separately, leading to
unsustainable management of freshwater resources (Richter, 2010; Zektser, Loaiciga, & Wolf,
2005). To protect and restore stream ecosystems effective groundwater management must be
established which is best accomplished by managing groundwater and surface water with an
integrated strategy. A major step forward in developing an integrated strategy is the recognition
that abstractions from rivers or other surface water bodies and abstractions from their
contributing groundwater systems represent depletions of the same source (Sophocleous M.,
2000).

Several methods for effective management of riverine and wetland ecosystems have been
proposed in scientific literature. The Ecological Limits of Hydrologic Alteration (ELOHA) (Poff
et al., 2010), the sustainability boundaries approach (Richter, 2010), and the minimum
acceptable flow volume (MAFV) (Petts et al., 1999) approach are a few examples. The ELOHA
framework proposes the development of flow alteration-ecological response relationships to
determine if hydrologic changes will have significant consequences. Richter (2010) proposes
that targeted percentage changes that allow for specific hydrologic circumstances to occur at the
frequency and duration with which they are required by the ecosystem (e.g. if a flood flow of 100 m$^3$ s$^{-1}$ were required by the ecosystem once per year between the months of April and May, what percent change from the baseline condition is possible that still allows for this hydrologic need). The MAFV approach (Petts et al., 1999) was developed to determine an acceptable volume of groundwater abstraction in groundwater-dominated stream ecosystems. It determines an ecological acceptable flow regime and represents it as a flow-duration curve.

2.5.1. Hydrologic Modelling as a Management Tool

Surface water and groundwater are a single resource with complex interconnections across space and time both within and between watersheds (Winter et al., 1998). GDE are not immune to changes in surface water hydrology. Management and protection of GDEs requires knowledge of the groundwater regime in the area; but they can also be sensitive to surface water inputs. Wetlands may require groundwater to sustain plant life, but flooding from surface water sources may also be necessary to the ecosystem. Conversely, flooding may be detrimental to the ecosystem of wetlands. Groundwater fed stream baseflow may support GDEs in low flow periods; but inputs from surface water may provide high flows necessary for lateral connection of the floodplain to maintain riparian communities. Groundwater and surface water, therefore, both need to be considered in the management of GDEs.

Any approach to management of GDEs would also benefit from hydrologic modelling. In the papers outlining management strategies for riverine and GDEs (Petts et al., 1999; Poff et al., 2010; Richter, 2010) the use of hydrologic models was recommended to provide streamflow data for future forecasts and in locations where historical flow records are not available. Petts et al. (1999) identify that a calibrated numerical model such as MODFLOW or MIKE SHE is required to forecast hydrologic conditions into the future and support management decisions;
Poff et al. (2010) suggest that hydrologic modelling is an important tool for computing hydrographs and the amount of allowable flow alteration; and Richter (2010) recommended the use of models where historical flow records were unavailable. Acreman and Miller (2007) identify that the use of hydrologic models is required for accurate and specific identification of hydrologic changes in wetlands. They suggest several modelling efforts requiring various amounts of effort and resources, but identify that full spatial and temporal representation of hydrologic patterns requires some form of sophisticated model such as MODFLOW (Acreman & Miller, 2007).

2.5.2. Rationale for Integrated Hydrologic Modelling

GDEs may be influenced by both the surface water and the groundwater regimes so the use of only a groundwater or surface water model may not be sufficient as it overlooks the interconnected nature of the watershed hydrology. Even using two models, a surface water model and a groundwater model, that use the output from one as input into the other (e.g. infiltration from the surface water model as recharge in the groundwater model), the spatial and temporal variability of the groundwater-surface water interactions may be lost. These interactions can be significant factors to the health of a GDE, especially since GDEs may or may not rely on the surface expression of groundwater, or may do so temporally, and for short or long durations (Eamus et al., 2006). Integrated hydrologic modelling allows the spatial and temporal interactions of groundwater and surface water to occur, making it a useful predictor of the hydrologic regime of GDEs.

MIKE SHE has been used in many circumstances including: the assessment of land use change on watershed hydrology in Korea (Im et al., 2009); the impact of fire induced land use change on streamflow responses in California (McMichael & Hope, 2007); prediction of
streamflow in a flashy, mountainous watershed in Hawaii (Sahoo et al., 2006); and to the Elmley Marshes, a wet grassland area in England where the performance of a MIKE SHE/MIKE 11 model to wetland areas was evaluated (Thompson et al., 2004). While these studies included groundwater flow and interaction between surface water and groundwater, the output was focused towards streamflow (Sahoo et al., 2006; McMichael & Hope, 2007) or a general water balance (Im et al., 2009). Groundwater discharge and baseflow in the stream were simulated, but the output was not specifically considered in any of the studies. Good calibration of MIKE SHE was obtained in each of these studies, yet difficulties were encountered. Calibration of an integrated hydrologic model could last indefinitely as parameterization increases, so these types of models should be calibrated for the specific objective and application to which it is applied (Sahoo et al., 2006).

The Thompson et al. (2004) had a large focus on linking MIKE SHE and MIKE 11, which was not easily or routinely done at the time. However, it did focus on groundwater levels as part of its calibration and output, which was necessary because of its focus on a wetland area. The Thompson et al. (2004) study was, therefore, the most similar study to the application of MIKE SHE to groundwater dependent ecosystems; though it did not link the hydrologic simulations to ecological condition. It demonstrated that a good fit with observed groundwater and surface water data could be obtained and that MIKE SHE was a useful modelling tool when both surface water and groundwater may be having an effect on a wetland area.

2.5.3. Challenges and Limitations to Integrated Modelling

Integrated hydrologic modelling is a useful tool for evaluating anthropogenic impacts to GDEs, but it is not without its shortfalls. The number of available integrated models is far less than the number of available surface or groundwater models. The cost is often much higher (e.g.
MIKE SHE), or the user friendliness is lower (e.g. GSFLOW) compared with surface water models that are both inexpensive and user-friendly (e.g. HEC-HMS). The data requirements are greater, since data on both the surface and subsurface of the watershed is required. The simulation time and/or the processing power required are greater, since more calculations are required per time step. These shortfalls can be overcome, however, as data is more easily obtained by newer technologies or published online by governments, as computational power is steadily increasing, and as the existing integrated models improve and new ones become available. However, as surface water and groundwater resources are increasingly being recognized as a single interconnected source and as environmental legislation protecting water sources and ecological integrity are being strengthened, the use of integrated hydrologic models is expected to grow.

Another shortfall of hydrologic models being used to evaluate ecological conditions is an issue of scale. Scales of hydrologic efficiency in model must be large enough to run model simulations in a reasonable amount of time given limits on computation power, but this may be too large a scale to match ecological sensitivity. Hydrologic models are typically run at spatial scales of tens of meters for small catchments or hundreds of meters for large catchments. For MIKE SHE applications in scientific literature, Thompson et al. (2004) used a “relatively fine” grid resolution of 30 × 30 m on an 8.7 km² catchment; Sahoo et al. (2006) used 30 × 30 m grid on a 2.7 km² catchment; McMichael & Hope (2007) used a 270 × 270 m grid on a 34 km² watershed; Im et al. (2009) used 200 × 200 m on a 258 km² catchment; and Vásquez et al. (2002) ranged the grid size from 300 × 300 m to 1200 × 1200 m on a 586 km² catchment. Obviously, the grid resolution of hydrologic models is related to the size of the model domain and the computational power available to the modeller, which increases with time. Ecological responses
may be on much finer spatial scales and influenced by local conditions (Wheeler et al., 2004). An added complication to the disconnect in the scales of hydrology and ecology is that finer resolution hydrologic models do not necessarily produce more accurate hydrologic results. Vásquez et al. (2002) tested the performance of the MIKE SHE code with a range of hydrologic grid sizes and found that a grid size of 600 × 600 m produced the best model performance for the given model type, structure, and data inputs of their application.

One method to alleviate this would be to run the hydrologic model at spatial scales as fine as possible. To maintain reasonable computational time, the total extent of the model domain should be reduced. For example, a hydrologic model of an entire watershed may not be useful for evaluating ecological conditions, but a model of smaller domain and scale could then be developed within the watershed model.

2.6. SUMMARY OF LITERATURE REVIEW

Special issues of journals dedicated to GDEs suggest there is a growing recognition of GDEs in the scientific community. GDEs seem to be a topic of greater interest amongst biologist and ecologists than amongst hydrologists and engineers. Much is known about types of GDEs, the species that populate them, the services they provide, common threats and processes of degradation. Less is known about the specific hydrologic requirements of types of GDEs or of species that inhabit GDEs. Several authors have postulated how ecological conditions may degrade with a loss of groundwater availability. This led to ideas on degrees of dependency and ecosystem response functions, but specific relationships between GDE health and hydrologic change remain undetermined. Furthermore, studies on the use of hydrologic modelling as a tool for evaluating GDEs are few. The literature recognizes that increased urbanization in the watershed may be responsible for ecological degradation, relating ecological condition to total or
effective impervious area, but does not provide mechanistic, hydrologic links or estimates of acceptable hydrologic change. Many studies discuss the ecological importance of the groundwater regime and/or threats to this regime, but few have specifically examined the impact of urbanization on the groundwater regime, the effectiveness of low-impact development to mitigate these impacts, or the capability of integrated modelling tools for the analysis of these ecosystems.
3. STUDY OBJECTIVES

This study used an integrated hydrologic model, MIKE SHE, to evaluate the hydrologic changes associated with urbanization and LID scenarios in an urbanized watershed, and related these changes to GDEs. The two main objectives were:

1. Develop a MIKE SHE modelling technique to represent LID within a watershed and predict hydrological response at a scale of ecological relevance, and
2. Evaluate the effectiveness of LID to restore a hydrologic regime to GDEs in an urban setting.

The first study objective was met by developing a method to represent LID within MIKE SHE. MIKE SHE lacks modules for many processes that occur in urban settings (e.g. pipe network flow) and has no defined way of modelling LID. The effect of LID on the hydrologic cycle was mimicked through adjusted parameters for overland flow, storage, and infiltration parameters. The second study objective was met by developing a MIKE SHE model of the Lovers Creek subwatershed, simulating pre-development, current (urbanized), and various LID land use scenarios ranging from 5% to 20% conversion of high density urban land use and comparing the hydrologic regime of each scenario.

Furthermore, the issue of scale is a significant factor as well. Hydrologic models tend to run on spatial scales of several meters and temporal scales of several hours, especially if an entire subwatershed is being modelling, while ecological systems may be sensitive to hydrologic scales of much smaller spatial and temporal extent.
4. METHODOLOGY AND MODEL DEVELOPMENT

4.1. STUDY METHODS

The goal of this research was to examine the effectiveness of LID on GDEs. AquaResource Inc., a division of Matrix Solutions, who completed MIKE SHE and FEFLOW models of the Barrie region for a Tier 3 Water Budget Study, provided the MIKE SHE model as a starting point for this research. The regional Barrie Tier 3 model had a total model domain of approximately 800 km² and was run at a spatial grid of 200 × 200 m. This was considered too large a grid to be representative of ecological conditions, which may be affected at a more localized scale. So, for this research, the Lovers Creek subwatershed model was built, reducing the overall model domain and enabling the creation of a finer grid resolution, while still performing simulations in a reasonable time. Furthermore, Lovers Creek was recognized as a watershed with important ecological functions and one that will undergo further urbanization in the future, escalating the impact of urbanization on GDEs and increasing the risk of ecological damage.

The impact of LID was examined by developing MIKE SHE models of the Lovers Creek watershed for pre-development, current (urbanized), and several LID land use scenarios. The output from these simulations was compared with respect to peak flows, baseflow, groundwater discharge, and the $T_{Q_{mean}}$ metric. Sections 4.4 and 4.5 discuss in more detail the development of the base MIKE SHE model and LID scenarios.
4.2. **Model Choice**

While MIKE SHE was used for this research, one other conjunctive hydrologic model was originally considered: GSFLOW. GSFLOW is developed by the United States Geological Survey (USGS, 2012). It is free of charge, but is a command line-based model with no graphical user interface. Input data and output data is stored in text and spreadsheet files. A customized Microsoft Excel spreadsheet is included with the download of the model that aids the user in evaluation of model output. However, this model represented a steep learning curve and would have also been difficult to implement. MIKE SHE, developed by DHI Water & Environment, is a fully integrated, physically-based model of the entire land phase of the hydrologic cycle that offered an intuitive user-friendly interface as well as a powerful computation engine.

Furthermore, AquaResource Inc. had a previously built MIKE SHE model of the Barrie region from which input data could be extracted. AquaResource Inc. also had extensive experience with MIKE SHE and provided training and support with the Lovers Creek model as needed.

DHI also produces MIKE URBAN and MOUSE, which are both designed to simulate water movement in urban settings. MIKE URBAN can simulate 2-D overland flow, pipe flow, weirs, pumps, and pollutants, which makes it a useful model for simulating sewer networks, stormwater drainage systems, and water distribution systems (DHI, 2011). MIKE URBAN also makes use of the public domain software produced by the United States Environmental Protection Agency (EPA) SWMM5 and EPA-NET. MOUSE, which stands for MOdel of Urban SEwers, is designed to simulate pipe flow and surface runoff, as well as water quality and sediment transport through urban sewer systems, making it a useful tool for modelling any drainage network that contains both free surface and pressurized flow (University of Texas at Austin). Both MIKE URBAN and MOUSE integrate with the ESRI ArcView GIS software for
fast processing of model data. While it is possible to link MIKE SHE to MIKE URBAN or MOUSE to more precisely simulate urban hydrology, neither MIKE URBAN nor MOUSE have specified tools for modelling LID. Thus, an approach similar to the one used in this study would still be required to simulate LID. Furthermore, linking MIKE SHE to MIKE URBAN would require more data (e.g. on the urban stormwater drainage network, manhole locations, pipe sizes, and discharge locations) as well as an experienced MIKE SHE/MIKE URBAN modeller, since the task is not a user-friendly procedure (see Section 6.6.2). Therefore, it was preferable for this research to use only the MIKE SHE model to represent urbanization and LID.

4.3. MIKE SHE

MIKE SHE is a fully integrated model that simulates the entire land-based phase of the hydrologic cycle (DHI, 2011a). It is developed, maintained, and distributed by DHI. It is a deterministic, fully distributed, physically-based model that utilizes a modular structure for the different processes of the hydrologic cycle including interception/evapotranspiration, snow melt, overland/channel flow, unsaturated zone, saturated zone, and river-aquifer exchange flow (Thompson et al., 2004). MIKE SHE is also coupled to the one-dimensional hydraulic model MIKE 11 to model river and stream networks. MIKE SHE represents the model domain using an orthogonal gridded network.
4.3.1. Precipitation and Snowmelt

MIKE SHE takes precipitation as a main input to generate runoff, infiltration, groundwater flow, and river flow. Precipitation can be in the form of rainfall or snowfall and is defined in the model as a distributed time series of precipitation rate (mm·hr⁻¹) or amount (mm) that can be uniformly distributed throughout the model domain or vary based on the locations of available climate stations.

Snowmelt is simulated using a simple degree-day method where the rate of snowmelt is dependent on a degree day coefficient ($DDC$, mm·day⁻¹·°C⁻¹), threshold melting temperature ($T_{\text{threshold}}$, °C) and the air temperature in the current time step ($T$, °C), and calculated by Equation 1.

$$q_{\text{snow}} = DDC \times (T - T_{\text{threshold}}) \times \Delta t$$

*Equation 1: Snowmelt computed by MIKE SHE model.*
The threshold melting temperature is usually 0 °C and the degree day coefficient is typically 1 – 5 mm·°C⁻¹·day⁻¹ and is often considered to be a calibration parameter. More complicated methods incorporating thermal melting may also be selected in MIKE SHE, but the simple degree method is less data intensive and more easily calibrated.

4.3.2. Evapotranspiration

Evapotranspiration (ET, mm) is computed by MIKE SHE from a reference evapotranspiration distributed time series data set. ET is dependent on the root depth (\(z_{RD}\), mm), leaf area index (LAI), and soil properties including an infiltration capacity and soil moisture contents at wilting (\(\theta_W\)), field capacity (\(\theta_{FC}\)), and saturated conditions (\(\theta_S\)). The actual soil moisture content, \(\theta\), varies linearly with depth within a range between \(\theta_{min}\) and \(\theta_{max}\), which are based on these parameters as seen in Figure 5.

![Figure 5: Moisture content in the unsaturated zone for the Two-Layer Water Balance method (DHI, 2011b)](image)

MIKE SHE first computes the interception storage using the LAI and canopy interception parameter (\(C_{int}\), mm), which is commonly 0.05 mm, using Equation 2.

\[
I_{max} = C_{int} \times LAI
\]

Equation 2: Interception storage calculation in MIKE SHE.
Where $I_{\text{max}}$ is the maximum amount of interception storage (mm). The actual interception storage, $I_{\text{act}}$ (mm), is then computed by Equation 3.

$$I_{\text{act}} = \min(I_{\text{max}}, P \cdot \Delta t)$$

*Equation 3: Actual interception storage calculation in MIKE SHE*

Where $P$ is the amount of precipitation (mm) and $\Delta t$ is the calculation time-step. In the Two-Layer Water Balance method, $ET$ is only allowed from the upper layer, which extends from the ground surface down to the higher of the water table or $ET$ extinction depth, while the lower layer extends from the bottom of Layer 1 of the subsurface down to the water table (DHI, 2011b). The $ET$ extinction depth (mm) is the maximum depth from which water can evaporate and is defined as the depth of the root zone plus the capillary fringe. If water from the root zone is removed by ET it may be replaced through capillary action, if the capillary fringe reaches the root zone from the water table (DHI, 2011b).

Actual ET ($E_{\text{act}}$, mm) is computed as the sum of ET from the canopy ($E_{\text{can}}$, mm), from ponded water ($E_{\text{pon}}$, mm), from the unsaturated zone ($ET_{\text{UZ}}$, mm), and from the saturated zone ($ET_{\text{SZ}}$, mm). $E_{\text{can}}$ is computed from Equation 4.

$$E_{\text{can}} = \min(I_{\text{act}}, E_{\text{p}} \cdot \Delta t)$$

*Equation 4: Canopy evaporation calculation in MIKE SHE.*

$E_{\text{can}}$ is then deducted from $I_{\text{act}}$ to maintain water balance. If $E_{\text{can}}$ does not satisfy the potential ET ($E_{\text{p}}$) then $E_{\text{pon}}$ is computed by Equation 5.

$$E_{\text{pon}} = \min(d_{oc}, (E_{\text{p}} - E_{\text{can}}) \cdot \Delta t)$$

*Equation 5: Evaporation from ponded water calculation in MIKE SHE.*

Where $d_{oc}$ is the amount of ponded water storage (mm) and is subsequently updated by subtracting $E_{\text{pon}}$. If the potential evapotranspiration demand is still not satisfied, then ET from the unsaturated zone is computed using Equation 6.
Equation 6: Evapotranspiration from the unsaturated zone calculation in MIKE SHE.

\[ E_{UZ} = \min(V_{UZ}, E_p - E_{can} - E_{pon}) \]

Where \( V_{UZ} \) is the volume of water in the unsaturated zone that is available for ET (mm) and is computed by Equation 7.

Equation 7: Volume of water in the unsaturated zone calculation in MIKE SHE.

\[ V_{UZ} = (\theta_{act} - \theta_{min}(z_d)) \cdot z_d \]

If the potential evapotranspiration demand is still not satisfied, then ET from the saturated zone is computed by Equation 8.

Equation 8: Evapotranspiration from the saturated zone calculation in MIKE SHE.

\[
E_{SZ} = \begin{cases} 
E_p \cdot \Delta t - E_{can} - E_{pon} - E_{UZ} ; & \text{when } z_{WT} < z_{RD} \\
E_p \cdot \Delta t \left( \frac{z_{RD} + z_{CF} - z_{WT}}{z_{CF}} \right) - E_{can} - E_{pon} - E_{UZ} ; & \text{when } z_{RD} \leq z_{WT} < z_{RD} + z_{CF} \\
0 ; & \text{when } z_{WT} \geq z_{RD} + z_{CF}
\end{cases}
\]

There are three cases that are used to compute \( ET_{SZ} \). If the depth to the water table \( (z_{WT}) \) is less than the depth of the root zone \( (z_{RD}) \), then \( ET_{SZ} \) will be maximized and the full potential evapotranspiration will be realised. If the depth to the water table is greater than the depth of the root zone, but less than the depth of the root zone plus the capillary fringe \( (z_{RD} + z_{CF}) \), then \( ET_{SZ} \) will be a fraction of that maximum, decreasing linearly until the water table falls below the depth of the root zone plus capillary fringe, after which \( ET_{SZ} \) is 0.0 mm. Finally, the actual evapotranspiration \( (E_{act}) \) is computed by summing the values from the each section, as in Equation 9.

Equation 9: Actual evapotranspiration calculation in MIKE SHE.

\[
E_{act} = E_{can} + E_{pon} + E_{UZ} + E_{SZ}
\]
4.3.3. Overland Flow Simulation

The overland flow module in MIKE SHE simulates the transport of water above ground after infiltration and ponding have occurred. There are two means of defining overland flow in MIKE SHE: a finite difference method and a simplified overland flow routing method. Only the finite difference method is discussed here, since it was used in this study.

The finite difference method of overland flow simulation in MIKE SHE uses the diffusive wave approximation to the St. Venant equations. The general form and the diffusive wave approximation of the St. Venant equations are shown for the \( x \)-direction in Equation 10 and Equation 11.

\[
S_{fx} = S_{o_x} - \frac{\partial h}{\partial x} - \frac{u}{gh} \frac{\partial u}{\partial x} - \frac{1}{gh} \frac{\partial u}{\partial t} - \frac{Qu}{gh}
\]

*Equation 10: General form of St. Venant Equations*

\[
S_{fx} = S_{o_x} - \frac{\partial h}{\partial x} = -\frac{\partial z_g}{\partial x} - \frac{\partial h}{\partial x} = -\frac{\partial z}{\partial x}
\]

*Equation 11: One-direction diffusive wave approximation of the St. Venant equations.*

Where \( S_{fx} \) is the friction slope of the water surface, \( S_{o_x} \) is the slope of the ground surface, \( h \) is the flow depth above the ground surface (m), \( z_g \) is the elevation of the ground surface (m), and \( z \) is the elevation of the water surface (m), \( u \) is the velocity of overland flow (m\,s\(^{-1}\)), and \( g \) is acceleration due to gravity (m\,s\(^{-2}\)). The diffusive wave approximation method “ignores momentum losses caused local and convective acceleration and lateral inflows perpendicular to the flow direction” (DHI, 2011b).

MIKE SHE also uses the Manning’s equation to define the friction slope. However, it uses the Manning’s \( M \), which is the reciprocal of Manning’s \( n \). This relationship is defined by Equation 12.
\[ S_f = \frac{v^2}{M^2 h^{4/3}} \]

*Equation 12: Friction slope calculation in MIKE SHE.*

Where \( v \) is the velocity of overland flow and \( M \) is Manning’s \( M \) and is equal to \( 1/n \). Combining these equations and solving for flow in a finite difference form produces Equation 13.

\[ Q = \frac{M\Delta x}{\Delta x^{1/2}} (Z_U - Z_D)^{1/2} h_u^{5/3} \]

*Equation 13: Overland flow calculation in MIKE SHE.*

Where \( Q \) is the flow, \( \Delta x \) is the cell size, \( Z_U \) and \( Z_D \) are the upstream and downstream water elevations, and \( h_u \) is the depth of water minus the detention storage. Equation 13 is the governing equation solved by the overland flow module of MIKE SHE.

MIKE SHE provides two alternative methods for solving this equation: successive over-relaxation or an explicit solution. The SOR method uses a linear matrix of \( N \) equations with \( N \) unknown water levels which is iteratively solved using a modified Gauss-Seidel method (DHI, 2011b). This method can possibly incur water balance errors, which MIKE SHE corrects by reducing the calculated outflows from a grid cell, if necessary, according to Equation 14.

\[ \sum |Q_{out}| \leq \sum (Q_{in}) + 1 + \frac{\Delta x^2 h(t)}{\Delta t} \]

*Equation 14: Water balance correction for overland flow in MIKE SHE*

The explicit solution solves the finite difference form of the diffusive wave equation on a cell by cell basis, computing the flow based on the head values using an explicit finite difference scheme in time. This method is less stable than the SOR method, requiring the flow to be slow, relative to the cell size (e.g. a flood wave cannot cross a grid cell in one time step) (DHI, 2011b). To maintain stability, the model requires the Courant number to be less than 1, as computed according to Equation 15.
Equation 15: Courant number governing numerical stability of overland flow in MIKE SHE

\[ C = \frac{dQ/dA}{dx/dt} = \frac{1}{dx} \times \frac{dQ}{dh} \times \frac{dt}{dx} \]

The criteria of \( C < 1 \) required much shorter time steps than were required using the SOR solver. For this reason, the SOR method was used in this modelling study.

4.3.4. Unsaturated Zone Flow

MIKE SHE can model the unsaturated zone using three methods: 1. One-dimensional Richards equation; 2. Gravity flow; and 3. the Two-Layer Water Balance method. MIKE SHE only simulates vertical flow through the unsaturated zone. The Two-Layer Water Balance method was selected for use in this model because it is less data intensive and more easily calibrated. Although the Richard’s Equation provides a more deterministic representation of the unsaturated zone, it is highly data intensive, requiring not only subsurface soil properties, but also relationships between \( K, \theta \), and \( \psi \), which were not known.

The Two-Layer Water Balance method aims to calculate the amount of actual evapotranspiration from the root zone and the amount of water that recharges to the saturated zone. Refer to section 4.3.2 for details on how evapotranspiration is computed. Once ET has been computed, the remaining water in the UZ is considered groundwater recharge and is time delayed to reach the water table based on the infiltration capacity of the soil. The method is primarily suited to areas where the water table is shallow; however, the parameters can be calibrated to accurately represent evapotranspiration and recharge in most conditions (DHI, 2011b).
4.3.5. Saturated Zone Flow

MIKE SHE represents the saturated zone using a fully three-dimensional representation of the subsurface using the Darcy’s Law to compute groundwater flow in all directions (see Equation 16).

\[
\frac{\partial}{\partial x} \left( K_{xx} \frac{\partial h}{\partial x} \right) + \frac{\partial}{\partial y} \left( K_{yy} \frac{\partial h}{\partial y} \right) + \frac{\partial}{\partial z} \left( K_{zz} \frac{\partial h}{\partial z} \right) - Q = S_s \frac{\partial h}{\partial t}
\]

*Equation 16: Three-dimensional Darcy’s Law for saturated zone flow.*

Where \( K_{xx}, K_{yy}, \) and \( K_{zz} \) are the saturated hydraulic conductivities in the \( x, y, \) and \( z \) axes, \( Q \) represents any source or sink terms from the saturated zone, \( S_s \) is the storage coefficient, which is the specific yield for unconfined conditions and the specific storage for confined conditions, and \( h \) is the hydraulic head where \( h = h(x,y,z,t) \). An implicit (backward in time) finite difference technique is used to solve Darcy’s Law for the hydraulic head. Two different solution techniques can be selected by the user: a successive over-relaxation method or a preconditioned conjugate gradient solution method. The preconditioned conjugate gradient solution was used in this study.

The preconditioned conjugate gradient (PCG) solver was developed by the USGS for use in their three-dimensional groundwater program MODFLOW. The solver used by MIKE SHE is the same solution technique that is used by MODFLOW. The potential flow, \( Q, (m^3 \cdot s^{-1}) \) is computed by Equation 17.

\[
Q = \Delta h \cdot C
\]

*Equation 17: Potential flow in the saturated zone computed by the PCG solver.*

Where \( \Delta h \) is the head difference (m) and \( C \) is the conductance \( (m^2 \cdot s^{-1}) \). \( C \) is computed for horizontal flow by Equation 18 and for vertical flow by Equation 19.

\[
C_{i-1/2} = \frac{K_{i-1,j,k} \cdot K_{i,j,k} \cdot (\Delta z_{i-1/2,j,k} + \Delta z_{i,j,k})}{(K_{i-1,j,k} + K_{i,j,k})}
\]

*Equation 18: Horizontal conductance for PCG solver of saturated zone flow.*
Equation 19: Vertical conductance for PCG solver of saturated zone flow.

\[ C_v = \frac{\Delta k Z_k \Delta z_{k+1}}{2K Z_k \Delta z_{k+1}} \]

Where \( K \) is the horizontal conductivity of the cell (\( \text{m} \cdot \text{s}^{-1} \)) and \( \Delta z \) is the saturated thickness of the cell (m).

Corrections must be applied to calculated flow if the cell above or below the current cell becomes dry (dewatered). Equation 20 calculates the correction to flow, \( q_c \) (\( \text{m}^3 \cdot \text{s}^{-1} \)).

\[
q_c = \begin{cases} 
    C v_{k+1/2} (h_{k+1} - z_{top,k+1}) & \text{when cell below is dewatered}; \\
    C v_{k-1/2} (z_{top,k} - h_k) & \text{when cell above is dewatered}. 
\end{cases}
\]

Equation 20: Corrections to vertical flow to account for dewatering of cells.

This correction is added to the right-hand side of the finite difference equation using the last computed head.

The saturated zone also produces drainage flow that may be used to define natural and artificial drainage systems that cannot be defined in MIKE 11. SZ drainage flow is computed as a linear reservoir model (Equation 21) using the depth of groundwater above a user-defined SZ drainage level and a specified time constant typically between \( 1 \times 10^{-6} \) or \( 1 \times 10^{-7} \) s\(^{-1}\) (DHI, 2011b).

\[ q = (h - Z_{dr}) \cdot C_{dr} \]

Equation 21: Saturated zone drainage flow calculation in MIKE SHE.

Where \( h \) is the head in the SZ, \( Z_{dr} \) is the drain level and \( C_{dr} \) is the drain conductance or time constant. SZ drainage flow is directed from a source cell to a recipient river link or boundary condition in the current time step. The assumption is that the time step for overland flow is longer than the time it takes for drainage flow to reach the recipient river link.

MIKE SHE directs drainage flow according to a developed source-recipient system based on the adjacent drain levels. Grid codes may be used to limit areas where MIKE SHE searches.
for lower drain levels. Distributed drainage options may also be used to directly link SZ drain to a specified MIKE 11 river link, or to a MIKE URBAN manhole, a link which will be discussed further in Section 6.6.2.

4.3.6. River-Aquifer Exchange Flow

The hydrologic modules of MIKE SHE are coupled to the one-dimensional hydraulic model MIKE 11 to compute channel flow and water level. MIKE 11 uses the fully dynamic St. Venant equations to simulate river flow. It is capable of simulating several different types of hydraulic structures as well, including weirs, gates, and culverts, as well as overbank spilling and flooding. However, the most important feature for this study focusing on GDEs was the fully dynamic coupling of the river and aquifer flow between MIKE 11 and MIKE SHE.

Figure 6: Conceptual model of exchange flow between river and saturated zone in MIKE SHE (DHI, 2011a)

Rivers in MIKE 11 are located along the borders of the orthogonal grid of MIKE SHE. River-aquifer exchange flow is then considered from both sides of the river with the adjacent cells of the saturated zone module. The river is considered to be a line source or sink to the groundwater module along the cell boundary. Thus, smaller grids produce a better representation of the river network. However, MIKE SHE grid cells can only couple to one river
link, so channels in the network must be spaced further apart than the MIKE SHE grid to prevent run time errors.

River-aquifer exchange flow is calculated using Equation 22.

\[ Q = C \cdot (h_{\text{grid}} - h_{\text{river}}) \]

*Equation 22: Exchange flow between river and saturated zone calculation in MIKE SHE.*

Where \( C \) is a conductance factor \((m^2 \cdot s^{-1})\) and \( h_{\text{grid}} \) and \( h_{\text{river}} \) are the head elevations in the adjacent MIKE SHE grid cell and the river, respectively. If \( h_{\text{grid}} \) is less than \( h_{\text{river}} \), then \( h_{\text{river}} \) is used instead. The conductance factor, \( C \), can be computed in three different ways by MIKE SHE: 1. Aquifer Only Conductance, 2. River Bed Only Conductance, and 3. Aquifer + Bed Conductance. All three methods of computing the conductance factor are described here because during model calibration and development, all three methods were tested and tried.

Aquifer only conductance (Equation 23) assumes that the river is in full contact with the subsurface aquifer and that there is no low permeability lining along the river bottom. Thus, the only head loss computed for exchange flow between the river and the aquifer comes from the grid cell itself. This set up is typical of gaining or fast moving streams (DHI, 2011b).

\[ C = \frac{K \cdot da \cdot dx}{ds} \]

*Equation 23: Conductance factor calculation for aquifer only exchange flow.*

Where \( K \) is the horizontal hydraulic conductivity of the grid cell, \( dx \) is the grid size used in the SZ component of MIKE SHE, \( ds \) is distance from the grid node to the middle of the simplified triangular cross section, and \( da \) is the vertical surface available for exchange flow. There are three ways of calculating \( da \), depending on the depth of the river and the water levels of the river and water table. If the water table in the SZ is above the river water level, then \( da \) is the saturated aquifer thickness above the bottom of the river bed; if the water table is below the river level, then \( da \) is the depth of water in the river; and if the river is deep enough to penetrate
multiple layers of the saturated zone, then \( da \) is calculated independently for each layer and the exchange flow within each layer is calculated independently.

River Bed Only Conductance (Equation 24) assumes that there is a river bed lining and that the head loss across this lining is significantly greater than the head loss across the grid. This is a common assumption in groundwater models, including MODFLOW (DHI, 2011b).

\[
C = L_c \cdot w \cdot dx
\]

*Equation 24: Conductance factor calculation for river bed only exchange flow.*

Where \( L_c \) is a leakage coefficient (T\(^{-1}\)), \( w \) is the wetted perimeter of the simplified triangular cross-section, and \( dx \) is the grid size.

Aquifer plus River Bed Conductance assumes that neither the head loss across the grid and the river lining can be neglected. In this case, \( C \) is calculated using Equation 25, which is the harmonic mean of conductance computed by Equation 23 and Equation 24.

\[
C = \frac{1}{\frac{1}{K} \cdot \frac{1}{dx} + \frac{1}{L_c \cdot w \cdot dx}}
\]

*Equation 25: Conductance factor calculation for combined exchange flow.*

4.4. **Input Data**

4.4.1. **Model Domain, Topography, and Land Use**

The domain of this modelling study encompassed the 60 km\(^2\) Lovers Creek subwatershed near Barrie, ON (Figure 7). Lovers Creek drains north into Kempenfelt Bay of Lake Simcoe. Approximately one-third of the model domain is covered by the City of Barrie along Kempenfelt Bay. This represents the majority of the high-density urban land use within the watershed and is where LID retrofitting would have the most significant impact. Only two small urban areas are found in the southern half of the subwatershed, which is predominantly agriculture. A golf course lies in the center of the subwatershed and there is also a small quarry. Near the southern
border of the subwatershed is the Lovers Creek Swamp, a provincially significant wetland. Figure 7 shows a satellite image of the model domain, the Lovers Creek subwatershed boundary, the river network used in this study, and the location of the Lovers Creek gauge used for model calibration (orange dot). Figure 8 depicts the land use of the Lovers Creek subwatershed. Dark grey represents the high density urban land use, which is along the northern part of the watershed within the City of Barrie. Purple areas represent wetland land uses, which is predominantly the Lovers Creek Swamp area in the southern part of the watershed. Another wetland area is seen along the southern edge of the City of Barrie, which is also part of Lovers Creek Swamp.

![Figure 7: Lovers Creek model domain and location map.](image-url)
4.4.2. Simulation Period

The simulation period chosen was from 1989/10/01 to 2010/09/30, or water year 1990 – 2010. The land use data, from 2007 and 2008, was applied to this simulation period. The calibration and validation data periods were from 2001 up to 2010, which were included in the simulation period.

The 20 year span was considered sufficient to generate a hydrologic regime, including inter-annual variability, for Lovers Creek. It was considered to have sufficient variability in precipitation data to create both dry and wet years for analysis. The average annual precipitation during this period at the Barrie WPCC station was 926 mm yr\(^{-1}\), with a minimum of 754 mm yr\(^{-1}\) occurring in 1992, and a maximum of 1334 mm yr\(^{-1}\) occurring in 1996 (Table 1). Water years 1987 and 1988 (1987/10/01 – 1989/09/30) were also simulated, but were considered a warm-up period to minimize impacts of initial conditions on model results and were not included in any model analysis.
MIKE SHE utilizes a variable time step, adjusting the $\Delta t$ as necessary to maintain numerical stability in each model of code. The maximum allowable time step must be set by the user. This model utilized a maximum allowable time step of 6, 12, and 24 hrs in the overland flow, unsaturated flow, and groundwater flow modules respectively. However, the model generally ran on much smaller time steps on the order of 30 min. While there was some variability in total simulation time for each scenario modelled, the total simulation time of the model was approximately 30 hrs.

4.4.3. Climate Data

The climate data used in this model was the same data used by the Barrie Tier 3 MIKE SHE model, with a few adjustments. The data was available from two Environment Canada climate stations, Barrie WPCC (ID: 6110557) and Cookstown (ID: 6111859). This data set included hourly precipitation data, daily rainfall and snowfall, and daily maximum and minimum temperatures. Hourly data sets for precipitation, temperature, and reference evapotranspiration were generated for use as model input based on these data sets. Each of these data sets was distributed according to the Voronoi areas (Thiessen polygons) of the nearby climate stations (Figure 9). Table 1 summarizes the climate data used as input from the two climate stations.
Figure 9: Lovers Creek division of climate data based on Voronoi areas.

The Barrie WPCC Station (6110557) is located near the top left corner of Figure 9 (orange dot); the Cookstown station (6111859) is not visible as it is further south outside the boundaries of the figure.
Hourly precipitation data was available for the Barrie WPCC station from 1950/01/01 – 2010/09/30, but data from the Cookstown station was only available until 2006. Therefore, the data from the Barrie WPCC station was used for the entire model domain for the period of 2007 – 2010. This assumption introduces some error into the model, as the Cookstown station consistently has a lower annual precipitation than the Barrie station. The average difference between the two stations was -12%, which was considered to be an acceptable error.
Daily maximum and minimum temperature data was available from these two climate stations. An hourly temperature data set was prepared assuming the daily minimum and maximum occurred at 3:00AM and 3:00PM, respectively, and followed a sinusoidal curve. The hourly temperature data set is used as input into the MIKE SHE model and the annual average is summarized in Table 1. Table 2 summarizes the monthly averages of the hourly temperature data for the duration of the simulation period.

AquaResource Inc. generated an hourly potential evapotranspiration data set using the WDMUtil program within the BASINS software, which is distributed by the (USEPA, 2012b). The Hamon method was utilized within the WDMUtil program to compute potential evapotranspiration. This method takes input data of daily minimum and maximum temperature to determine humidity, the latitude of the climate station to determine sunlight exposure, and a monthly coefficient to compute the potential evapotranspiration. Table 1 and Table 2 summarize the potential evapotranspiration data for the simulation period.

Table 2: Average monthly precipitation, potential evapotranspiration, and temperature for the two climate stations from water year 1990 – 2010

<table>
<thead>
<tr>
<th>Month</th>
<th>Average Precipitation (mm)</th>
<th>Average Potential Evapotranspiration (mm)</th>
<th>Average Temperature (°C)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Barrie</td>
<td>Cookstown</td>
<td>Barrie</td>
</tr>
<tr>
<td>Jan</td>
<td>53.4</td>
<td>44.0</td>
<td>20.0</td>
</tr>
<tr>
<td>Feb</td>
<td>47.3</td>
<td>41.8</td>
<td>26.2</td>
</tr>
<tr>
<td>Mar</td>
<td>37.6</td>
<td>36.2</td>
<td>50.0</td>
</tr>
<tr>
<td>Apr</td>
<td>44.4</td>
<td>47.0</td>
<td>106.7</td>
</tr>
<tr>
<td>May</td>
<td>51.0</td>
<td>48.3</td>
<td>190.8</td>
</tr>
<tr>
<td>Jun</td>
<td>55.6</td>
<td>51.4</td>
<td>290.5</td>
</tr>
<tr>
<td>Jul</td>
<td>57.8</td>
<td>54.0</td>
<td>320.1</td>
</tr>
<tr>
<td>Aug</td>
<td>63.8</td>
<td>55.1</td>
<td>258.4</td>
</tr>
<tr>
<td>Sep</td>
<td>61.2</td>
<td>52.9</td>
<td>161.9</td>
</tr>
<tr>
<td>Oct</td>
<td>48.9</td>
<td>46.3</td>
<td>81.9</td>
</tr>
<tr>
<td>Nov</td>
<td>70.8</td>
<td>63.0</td>
<td>42.5</td>
</tr>
<tr>
<td>Dec</td>
<td>47.1</td>
<td>39.2</td>
<td>23.1</td>
</tr>
</tbody>
</table>
Snowmelt parameters such as the threshold melting temperature and degree day coefficient were assumed to be uniform throughout the model domain. The hourly air temperature, however, was varied based on the climate stations Voronoi areas, and thus snowmelt computations varied throughout the model domain. Again, the temperature data from the Barrie WPCC station was used for the entire model domain from the period of 2007-2010. This introduced some error into the model as annual average temperature was approximately 8% lower for the Cookstown station.

Snowmelt was assumed to be uniform throughout the model domain and thus did not use Voronoi areas (Thiessen polygons) or a distributed grid to define. Table 3 summarizes the snowmelt data used in the model.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Threshold Melting Temperature (°C)</td>
<td>0</td>
</tr>
<tr>
<td>Degree Day Coefficient (mm·day⁻¹·°C⁻¹)</td>
<td>5</td>
</tr>
<tr>
<td>Minimum Snow Storage for Full Coverage (mm)</td>
<td>5</td>
</tr>
<tr>
<td>Maximum Wet Snow Fraction (-)</td>
<td>0.001</td>
</tr>
<tr>
<td>Initial Total Snow Storage (mm)</td>
<td>0</td>
</tr>
<tr>
<td>Initial Wet Snow Fraction (-)</td>
<td>0</td>
</tr>
</tbody>
</table>

The degree day coefficient was initially 3 °C as was used in the Barrie Tier 3 MIKE SHE model, but this was changed during the model calibration process. None of the other parameters were altered from the Barrie Tier 3 model completed by AquaResource Inc.

4.4.4. Overland Flow Data

The topography of the model domain was specified using a 5 m resolution Digital Elevation Model (DEM) provided to AquaResource Inc. by the Lake Simcoe Region Conservation Authority (LSRCA). Figure 10 shows the DEM buffered 500 m beyond the
Lovers Creek model domain. All input data that was distributed throughout the model domain had this buffer to ensure that no areas contained undefined parameters. The lowest elevation in the model is 218 m at the Kempenfelt Bay water level. Along the western border of the model domain the elevations are highest and consistently above 300 m.

![Digital Elevation Model at 5 m resolution of the Lovers Creek subwatershed.](image)

MIKE SHE’s overland flow module requires Manning’s M, depression storage, a paved runoff coefficient, and an initial water depth. Manning’s M, depression storage, and the paved runoff coefficient were distributed throughout the model domain based on the eight land use classes which were created from 2008 land use data provided to AquaResource Inc. by the LSRCA. The eight land use classes, as well as the values for these parameters, are listed in Table 4 and their distribution throughout the watershed is shown in Figure 8. The initial water depth was assumed to be 0 m throughout the model domain. The spatial distribution of land use within MIKE SHE is defined via a unique grid code identifier (Table 4).
Table 4: Summary of overland data by land use class.

<table>
<thead>
<tr>
<th>Land Use (Grid Code ID)</th>
<th>Manning’s n (m$^{-1/3}$s)</th>
<th>Manning’s M (m$^{1/3}$s$^{-1}$)</th>
<th>Depression Storage (mm)</th>
<th>Paved Runoff Coefficient (-)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Water (1)</td>
<td>0.06</td>
<td>16.7</td>
<td>10.0</td>
<td>-</td>
</tr>
<tr>
<td>Low Density Urban / Rural Areas (2)</td>
<td>0.15</td>
<td>6.7</td>
<td>2.0</td>
<td>-</td>
</tr>
<tr>
<td>High Density Urban (3)</td>
<td>0.09</td>
<td>11.1</td>
<td>2.0</td>
<td>0.3</td>
</tr>
<tr>
<td>Hay / Pasture / Idle / Transitional (4)</td>
<td>0.37</td>
<td>2.7</td>
<td>5.0</td>
<td>-</td>
</tr>
<tr>
<td>Row Crops / Intensive Agriculture (5)</td>
<td>0.37</td>
<td>2.7</td>
<td>2.0</td>
<td>-</td>
</tr>
<tr>
<td>Forests / Mixed Woods (7)</td>
<td>0.42</td>
<td>2.4</td>
<td>10.0</td>
<td>-</td>
</tr>
<tr>
<td>Wetlands (10)</td>
<td>0.42</td>
<td>2.4</td>
<td>9.0</td>
<td>-</td>
</tr>
<tr>
<td>Pits / Quarries (12)</td>
<td>0.05</td>
<td>20.0</td>
<td>1.0</td>
<td>-</td>
</tr>
</tbody>
</table>

The paved runoff coefficient was only defined for the high density urban land use because it represents the fraction of ponded water that drains to storm sewers and other surface drainage features in paved areas (DHI, 2011b). This coefficient defines the “fraction of the overland flow that will be drained via the SZ drainage network in the current time step” (DHI, 2011b), which sends the water directly to the river link via SZ drainage flow.

4.4.5. Unsaturated Zone Data

The unsaturated zone was modelled using the Two-Layer Water Balance method within MIKE SHE and represents the uppermost zone of the subsurface where infiltration, evapotranspiration, and percolation may all occur. The Two-Layer Water Balance method focuses on computing the amount of evapotranspiration and recharge to the saturated zone. The Green and Ampt method was used to calculate infiltration into the soil profile.

The implementation of the Two-Layer Water Balance method within MIKE SHE requires an evapotranspiration surface depth, a distributed soil profile, the leaf area index, and a rooting depth. The distributed soil profile was the same as the Barrie Tier 3 model, which was defined using the quaternary geology from the Ontario Geological Survey (OGS, 2003). It defines the soil parameters of water content at saturation, field capacity, and wilting point, the saturated
hydraulic conductivity, and soil suction at the wetting front. Simple by-pass flow or full macro pore flow can also be defined for each soil type, but these were not included in the model since supporting data was not available, the Richards Equation must be used for UZ flow simulation (for full macro-pore flow), and to remain consistent with the Barrie model. The soil parameters are listed in Table 5 and their distribution is shown in Figure 11. These parameters were determined by AquaResource Inc. through calibration of the Barrie Tier 3 MIKE SHE model. The ET surface depth was assumed to be 0.1 m throughout the model domain. This depth defines the thickness of the capillary fringe and extends the thickness of Layer 1 in the Two-Layer Water Balance method used for ET.

The Two-Layer Water Balance method also requires the leaf area index (LAI) and root depth (RD) to be defined. These two parameters were defined within the model domain using the eight land use classes using MIKE timeseries files (*.dfs0) to define the changes through time at a monthly discretization. Tables Table 6 and Table 7 summarize the values for these parameters based on land use. These values were determined by AquaResource Inc. in the Barrie Tier 3 model.

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>Water content at saturation (-)</th>
<th>Water content at field capacity (-)</th>
<th>Water content at wilting point (-)</th>
<th>Saturated hydraulic conductivity (m·s⁻¹)</th>
<th>Soil suction at the wetting front (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sand</td>
<td>0.46</td>
<td>0.23</td>
<td>0.07</td>
<td>4 × 10⁻⁶</td>
<td>-0.25</td>
</tr>
<tr>
<td>Gravel</td>
<td>0.30</td>
<td>0.20</td>
<td>0.04</td>
<td>6 × 10⁻⁶</td>
<td>-0.20</td>
</tr>
<tr>
<td>Silt/Till</td>
<td>0.56</td>
<td>0.46</td>
<td>0.27</td>
<td>4 × 10⁻⁸</td>
<td>-0.20</td>
</tr>
<tr>
<td>Clay</td>
<td>0.56</td>
<td>0.46</td>
<td>0.27</td>
<td>1 × 10⁻⁸</td>
<td>-0.20</td>
</tr>
</tbody>
</table>
Figure 11: Distribution of simplified soil layers for use in the Two-Layer Water Balance.

Table 6: Leaf area index for each land use class defined by month of year

<table>
<thead>
<tr>
<th>Month</th>
<th>Wetlands/Water</th>
<th>Low Density Urban/Rural Areas</th>
<th>High Density Urban</th>
<th>Hay/Intensive Agriculture</th>
<th>Hay/Pasture/Idle/Transitional</th>
<th>Forest/Mixed Woods</th>
</tr>
</thead>
<tbody>
<tr>
<td>Jan</td>
<td>2.0</td>
<td>2.0</td>
<td>0.8</td>
<td>2.5</td>
<td>2.5</td>
<td>5.0</td>
</tr>
<tr>
<td>Feb</td>
<td>2.0</td>
<td>2.0</td>
<td>0.8</td>
<td>2.5</td>
<td>2.5</td>
<td>5.0</td>
</tr>
<tr>
<td>Mar</td>
<td>2.0</td>
<td>2.0</td>
<td>0.8</td>
<td>2.5</td>
<td>2.5</td>
<td>5.0</td>
</tr>
<tr>
<td>Apr</td>
<td>2.5</td>
<td>2.5</td>
<td>1.1</td>
<td>3.5</td>
<td>3.5</td>
<td>6.0</td>
</tr>
<tr>
<td>May</td>
<td>3.0</td>
<td>3.0</td>
<td>1.1</td>
<td>4.0</td>
<td>4.0</td>
<td>6.0</td>
</tr>
<tr>
<td>Jun</td>
<td>4.0</td>
<td>4.0</td>
<td>1.5</td>
<td>5.0</td>
<td>5.0</td>
<td>7.0</td>
</tr>
<tr>
<td>Jul</td>
<td>4.0</td>
<td>3.0</td>
<td>1.5</td>
<td>5.0</td>
<td>5.0</td>
<td>7.0</td>
</tr>
<tr>
<td>Aug</td>
<td>4.0</td>
<td>3.0</td>
<td>1.5</td>
<td>5.0</td>
<td>5.0</td>
<td>7.0</td>
</tr>
<tr>
<td>Sep</td>
<td>4.0</td>
<td>3.0</td>
<td>1.4</td>
<td>4.5</td>
<td>4.5</td>
<td>6.0</td>
</tr>
<tr>
<td>Oct</td>
<td>3.0</td>
<td>2.5</td>
<td>1.1</td>
<td>4.0</td>
<td>4.0</td>
<td>6.0</td>
</tr>
<tr>
<td>Nov</td>
<td>2.5</td>
<td>2.0</td>
<td>1.1</td>
<td>3.5</td>
<td>3.5</td>
<td>5.0</td>
</tr>
<tr>
<td>Dec</td>
<td>2.0</td>
<td>2.0</td>
<td>0.8</td>
<td>2.5</td>
<td>2.5</td>
<td>5.0</td>
</tr>
</tbody>
</table>
Table 7: Rooting depth (mm) for each land use class defined by month of year

<table>
<thead>
<tr>
<th>Month</th>
<th>Wetlands / Water</th>
<th>Low Density Urban / Rural Areas</th>
<th>High Density Urban</th>
<th>Hay / Pasture / Idle / Transitional</th>
<th>Row Crops / Intensive Agriculture</th>
<th>Forest/Mixed Woods</th>
</tr>
</thead>
<tbody>
<tr>
<td>Jan</td>
<td>200</td>
<td>750</td>
<td>750</td>
<td>50</td>
<td>50</td>
<td>1000</td>
</tr>
<tr>
<td>Feb</td>
<td>200</td>
<td>750</td>
<td>750</td>
<td>50</td>
<td>50</td>
<td>1000</td>
</tr>
<tr>
<td>Mar</td>
<td>200</td>
<td>750</td>
<td>750</td>
<td>50</td>
<td>50</td>
<td>1000</td>
</tr>
<tr>
<td>Apr</td>
<td>200</td>
<td>750</td>
<td>750</td>
<td>52</td>
<td>52</td>
<td>1041</td>
</tr>
<tr>
<td>May</td>
<td>200</td>
<td>750</td>
<td>750</td>
<td>116</td>
<td>116</td>
<td>2000</td>
</tr>
<tr>
<td>Jun</td>
<td>200</td>
<td>750</td>
<td>750</td>
<td>521</td>
<td>521</td>
<td>2000</td>
</tr>
<tr>
<td>Jul</td>
<td>200</td>
<td>750</td>
<td>750</td>
<td>1000</td>
<td>1000</td>
<td>2000</td>
</tr>
<tr>
<td>Aug</td>
<td>200</td>
<td>750</td>
<td>750</td>
<td>1000</td>
<td>1000</td>
<td>2000</td>
</tr>
<tr>
<td>Sep</td>
<td>200</td>
<td>750</td>
<td>750</td>
<td>1000</td>
<td>1000</td>
<td>2000</td>
</tr>
<tr>
<td>Oct</td>
<td>200</td>
<td>750</td>
<td>750</td>
<td>968</td>
<td>968</td>
<td>1960</td>
</tr>
<tr>
<td>Nov</td>
<td>200</td>
<td>750</td>
<td>750</td>
<td>196</td>
<td>196</td>
<td>1000</td>
</tr>
<tr>
<td>Dec</td>
<td>200</td>
<td>750</td>
<td>750</td>
<td>100</td>
<td>100</td>
<td>1000</td>
</tr>
</tbody>
</table>

4.4.6. Saturated Zone

The saturated zone in this MIKE SHE model used data from the Barrie Tier 3 study. That study utilized both a MIKE SHE model and a more detailed model of the subsurface using FEFLOW. The FEFLOW model contained 9 subsurface layers of aquifers and aquitards; however, to minimize computation time and given the limited computation power, the simplified 3-layer structure from the Barrie Tier 3 MIKE SHE model was used for the Lovers Creek model developed in this research. This simplified 3-layer structure consisted of an upper aquifer, a middle aquitard, and a deep aquifer (Figure 12).
Figure 12: Simplified layer structure using the MIKE SHE model vs. The FEFLOW model.

The upper aquifer in MIKE SHE used lumped properties from the top 4 layers of the FEFLOW model including the unconfined layer, aquifers A1 and A2 and the aquitard C1. This layer is where the majority of the interaction between surface water and groundwater occurs and where exchange flow between the saturated zone and the river network occurs. The middle layer (Layer 2) is a confining aquitard. The third layer represents a deep aquifer system consisting of two aquifer layers (A3 and A4) and two aquitard layers (C3 and C4). Groundwater flow through this layer represents much longer, regional flow paths. This simplified subsurface geology in the MIKE SHE Barrie Tier 3 study was kept for the Lovers Creek model to maintain reasonable run times.

The saturated zone was defined spatially using 50 × 50 m grids of the lower level of each layer, and the horizontal and vertical saturated hydraulic conductivity, while the specific yield and specific storage of each layer was assumed to be uniform throughout the model domain. Figure 13 shows the saturated hydraulic conductivity used in Layer 1 of the model and Table 8 summarizes the parameters of the saturated zone. The specific yield and specific storage of all
three layers were assumed to be 0.2 and $1.0 \times 10^{-5} \text{ m}^{-1}$. Values for all parameters in the saturated zone were kept the same as the Barrie model to remain consistent.

The $50 \times 50 \text{ m}$ grids were created to match the alignment of the overland flow finite difference grid. However, they were created from $100 \times 100 \text{ m}$ grids used in the Barrie Tier 3 study so, despite being $50 \times 50 \text{ m}$, the values are only accurate to a $100 \times 100 \text{ m}$ discretization. Other grids files (e.g. land use classes) were defined from available unprocessed shape files within GIS. Only the saturated zone was missing these files and had to be defined from the larger grid.

Table 8: Summary of subsurface zone data.

<table>
<thead>
<tr>
<th>MIKE SHE Layer</th>
<th>Lower Level (m)</th>
<th>Horizontal Conductivity (m s$^{-1}$)</th>
<th>Vertical Conductivity (m s$^{-1}$)</th>
<th>Specific Yield</th>
<th>Specific Storage (m$^{-1}$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shallow Aquifer</td>
<td>177 – 230</td>
<td>$7.0 \times 10^{-6} – 4.4 \times 10^{-4}$</td>
<td>$1.0 \times 10^{-9} – 8.3 \times 10^{-6}$</td>
<td>0.2</td>
<td>$1 \times 10^{-5}$</td>
</tr>
<tr>
<td>Confining Layer</td>
<td>151 – 199</td>
<td>$1.0 \times 10^{-8} – 4.6 \times 10^{-6}$</td>
<td>$1.0 \times 10^{-9} – 4.6 \times 10^{-7}$</td>
<td>0.2</td>
<td>$1 \times 10^{-5}$</td>
</tr>
<tr>
<td>Deep Aquifer</td>
<td>108 – 145</td>
<td>$5.0 \times 10^{-6} – 1.2 \times 10^{-3}$</td>
<td>$5.0 \times 10^{-7} – 1.2 \times 10^{-4}$</td>
<td>0.2</td>
<td>$1 \times 10^{-5}$</td>
</tr>
</tbody>
</table>

Figure 13: Horizontal and vertical saturated hydraulic conductivity used in Layer 1 of the saturated zone.
Boundary conditions used along the entire border of the model domain were known head values. A constant head value of 218 m was used in all three layers at the border along Kempenfelt Bay. Along the rest of the border the average head values in each layer of the SZ from the Barrie Tier 3 study were used. The model was also run using the time-varying SZ heads in each layer from the Barrie Tier 3 model, but there was no difference in model results.

The average depth of the first layer was determined to be about 70 m and in some areas it was over 100 m thick. This had significant influences on the results, particularly in exchange flow between the SZ and the river network. Initially, exchange flow was computed using aquifer plus river bed conductance (Equation 25), but the hydraulic conductivity was not considered to be representative of near surface conditions since it represented conditions over such a large depth. Ultimately, the exchange flow was computed using River Bed Only conductance (Equation 24).

The drainage layer in the Lovers Creek model was changed from the Barrie Tier 3 model. Initially, the same grid file used in the Barrie model to specify the level of the drainage layer in the SZ was also used in the Lovers Creek model. However, that drainage layer was lower than the known head boundary conditions used in the model at some locations, particularly in the southern portions of Lovers Creek subwatershed, near Kempenfelt Bay. This was adding a significant amount of water to the model as inflow across the boundary and flowing through the SZ drainage layer to the stream network causing severe differences between observed and simulated results. The drainage layer was changed to be 1 m below the ground surface for subsequent runs, which is a typical value suggested by DHI (DHI, 2011b). The drainage time constant was also subsequently changed to $1 \times 10^{-7} \text{ s}^{-1}$ during model calibration.
4.4.7. River Network

The river network used in the Lovers Creek model was the same network used in the Barrie Tier 3 study. It was a simplified river network that used only rivers greater than 500 m apart, a Strahler classification greater than 2, and a stream length greater than 700 m. However, new cross-sections were created for the Lovers Creek model at intervals of every 200 – 400 m. New cross-sections were 70 m wide (35 m on each side) so as not to extend beyond the border of the grid cell they are placed within. Cross-section bank elevations were adjusted to correct for discrepancies with the overland flow elevations. These discrepancies occur because the cross-section elevations were taken from the 5 m DEM and the distributed model averaged elevations across the 50 × 50 m grid. The midpoint of each cross section was reduced by 0.3 m to create an inner channel (Figure 14), which was meant to account for low flow and prevent ‘drying out’ of the channel, which may cause numerical instabilities in the hydraulic model.

Figure 14: Sample cross-section in MIKE 11 editor.
The lack of detail in the cross-section definition is a source of uncertainty and error in the model. This will particularly affect the head in the river, which may then also affect the amount of computed exchange flow between the SZ and the river.

Figure 15: Lovers Creek river network in MIKE SHE model.

4.5. **LID Scenario Development**

The MIKE SHE model runs on an orthogonal gridded network, and this study used a 50 × 50 m grid. In practice, LID areas are not likely to be as large as 50 × 50 m. This is one limitation of using a gridded hydrologic model at this scale for this type of study. LID areas,
therefore, were modelled as ‘conglomerate’ LIDs, so each LID cell may represent multiple LID retrofits in that area. LID scenario simulations were developed by modifying the base scenario of the current, urbanized conditions model.

Two different types of LID land use cells were created to contrast different hydrologic processes that LIDs influence: infiltration and retention. In reality, a specific LID is not an ‘infiltration’ LID or a ‘retention’ LID, but many types of LID provide enhanced infiltration (e.g. permeable pavement, infiltration trenches) and many provide enhanced stormwater retention (e.g. green roofs, rain barrels with water reuse) and many also provide both of these effects (e.g. bioretention cells). However, since individual LIDs cannot be represented in an orthogonal, gridded representation of the watershed, no specific type of LID was modelled, only the enhanced processes each would provide to examine the overall effectiveness of LID cells with regard to restoring a natural hydrologic regime to an urban stream and GDE. Furthermore, each of these LIDs may or may not provide enhanced evapotranspiration, through either increased vegetative cover or having an increased period of time available for evaporation. For example, a green roof may provide enhanced evaporation by allowing the water to be stored on the roof for longer while it drains and enhanced transpiration through increased vegetative cover, while permeable pavement likely would not have a significant impact on evapotranspiration. The enhanced infiltration LIDs were assigned the unique grid code value of 13 within MIKE SHE, while the enhanced evapotranspiration LIDs were assigned a value of 14. Five different LID scenarios were developed included four that used the infiltration based LID (Grid Code 13) and one using the retention based LID (Grid Code 14). The four LID13 scenarios used represented various percent conversions of High Density Urban land use to LID land use, which were 5%, 10%, 15%, and 20%. These scenarios were named LID13-05, LID13-10, LID13-15, and LID13-
20, respectively. No conversion beyond 20% was done since the four different percent conversions were thought to provide enough simulations to make a meaningful comparison between LID and urban land use. All LID scenarios simulated contained only one type of LID to allow for comparison between the two types. Only one scenario, at the 20% conversion case, was completed with the LID 14 land use cells (LID14-20) for comparison to the 20% conversion to LID 13 (LID13-20) scenario.

4.5.1. Generation of New Model Inputs

New land use grids, created for each LID scenario modelled, placed LID cells at an approximately even distribution throughout the high density urban (HDU) land use of the model domain. These new land use grids were created by taking advantage of the ability to copy MIKE SHE grids and paste them into Microsoft Excel. A spreadsheet tool in Excel was developed to modify and create new land use grids, which were then copied back into MIKE SHE and saved as new land use grid input files.

LID cells (grid code = 13, 14) were created by modifying the land use grid code in cells that contained HDU land use (grid code = 3) and were the lowest elevations based on the 50 × 50 m DEM within their “block” of cells. A block was defined as a group of 20 cells of 5 rows by 4 columns. Blocks were defined this way to create LID scenarios with approximately 5%, 10%, 15% and 20% conversion of HDU to LID land use. Within each block, the four lowest elevation HDU cells were marked for conversion and ranked 1 – 4 based on their respective elevations. The number of HDU cells within each block was also recorded and a spreadsheet was created that converted HDU cells to LID cells based on the following rules:

1. In the 5% LID scenario, the 1st ranked cell (i.e. the lowest elevation HDU cell within each block) was converted to LID land use if the number of HDU cells
within the block was greater than 8. This created a total of 460 LID cells, for a total of 5.1% conversion of HDU to LID land use.

2. In the 10% LID scenario, the 1st and 2nd ranked cells were converted if the number of HDU cells within the block was greater than 11 and only the 1st ranked cell was converted if the number of HDU cells within the block was greater than 6. This created 901 LID cells for a 10.1% conversion.

3. In the 15% LID scenario, the 1st, 2nd, and 3rd ranked cells were converted if the number of HDU cells within the block was greater than 15; the 1st and 2nd ranked cells were converted if the number of HDU cells was greater than 7; and only the 1st ranked cell was converted if the number of HDU cells was greater than 4. This created a total of 1340 LID cells for a 15.0% conversion.

4. In the 20% LID scenario, the 1st, 2nd, 3rd, and 4th ranked cells were converted if the number of HDU cells was greater than 17; the 1st, 2nd, and 3rd ranked cells were converted if the number of HDU cells was greater than 12; the 1st and 2nd ranked cells were converted if the number of HDU cells was greater than 7; and only the 1st ranked cell was converted if the number of HUD cells was greater than 2. This created a total of 1790 LID cells for a 20.0% conversion.

The creation and use of blocks was used to ensure an approximately even spatial distribution of the LID cells. This is certainly true for the 5% LID case, since each block only received a maximum of 1 LID cell. However, for the 10%, 15% and 20% LID scenarios, small groups of LID cells occasionally formed pockets because the four lowest, previously HDU cells within the block were adjacent to one another. The spatial distribution of LID cells remained approximately even as these pockets of LID cells occur within a single block and thus are
relatively evenly distributed themselves and they only occurred occasionally given the right circumstances.

In the original land use grid (including the 500 m buffer around the Lovers Creek boundary) there were a total of 33,491 cells; of these, 8,939 (27%) were high density urban. In the 20% conversion scenario a total of 1790 LID cells were created, 5.3% of the total land use in the Lovers Creek subwatershed. Table 9 summarizes the total land use percentages of each of the LID scenarios.

<table>
<thead>
<tr>
<th>Conversion Scenario</th>
<th>5%</th>
<th>10%</th>
<th>15%</th>
<th>20%</th>
</tr>
</thead>
<tbody>
<tr>
<td>No. of LID cells</td>
<td>460</td>
<td>901</td>
<td>1340</td>
<td>1790</td>
</tr>
<tr>
<td>Percent of total land use</td>
<td>1.4%</td>
<td>2.7%</td>
<td>4.0%</td>
<td>5.3%</td>
</tr>
</tbody>
</table>

Figure 16 provides an example for a single block of cells. The example shows the conversion of HDU to LID cells for the 20% conversion scenario. In the original grid, there are 13 HDU land use cells (top of figure) and the four lowest are located in the top left corner (highlighted cells). The minimum elevation, however, is in the top right corner of the block (bolded and underlined). Based on the conversion rules described above, for the 20% scenario, with 13 HDU cells, the three lowest cells (ranked 1, 2, and 3) get converted to LID cells (bottom of figure) for a total of 23.1% HDU to LID conversion. While this is more than the 20%, the rules were created so that the average conversion rate of all blocks would be approximately 20% conversion scenario showing which cells were converted and the change to the topography grid. The topography was also modified to ensure overland from along and through the HDU cells, which would be the case if an actual LID site were designed and installed. The modifications
were based on the 1 – 4 ranking of elevations used in the creation of LID land use grids. The base scenario (current urbanized conditions) 50 × 50 m topography grid was modified by lowering the elevation of any LID cell to be lower than the previous lowest elevation. The elevation of the 4th ranked cell, if converted, was changed to be 0.25 m below the previous minimum; the elevation of the 3rd ranked cell, if converted, was changed to be 0.5 m below the previous minimum; the elevation of the 2nd ranked cell, if converted was changed to be 0.75 m below the previous minimum, and the elevation of the 1st ranked cell was changed to be 1.0 m below the previous minimum (Figure 16). This provided a modest slope of 0.5% between LID cells, which allowed for the possibility of some overland flow between LID cells while also ensuring that runoff from nearby cells flowed toward the LID areas. However, this method did not consider the elevations of cells in adjacent blocks, so it remains possible that some LID cells on the border of a block have higher elevations than non-LID cells in an adjacent block and would therefore not receive runoff from those cells. However, it was assumed that this method lowered the elevation of LID cells sufficiently to receive the majority of runoff from nearby cells. Figure 17 shows the created land use grid for the LID13-20 scenario, represented 20% conversion of HDU to LID land use type 13.

![Table](Table.png)

*Figure 16: Example block of cells showing conversion of HDU to LID land use for the LID13-20 scenario.*
4.5.2. Generation of other input data

Each LID scenario modelled required modifying more than just the land use grid and topography grids. New timeseries files were created for LAI and RD, modified grids for topography, Manning’s M, depression storage, paved runoff coefficient, evapotranspiration surface depth, and the soil profile for the Two-Layer Water Balance representation of the UZ.
A new timeseries was generated for each type of LID land use using a monthly discretization of values similar to that of existing land uses. Table 10 summarizes the LAI and RD timeseries used for both the infiltration and retention LIDs simulated. The LAI and RD values were chosen to be slightly less than the Forest/Mixed Woods land use to represent the presence of vegetated land cover, but not to the same extent that would be evident in a forested area.

<table>
<thead>
<tr>
<th>Month</th>
<th>Leaf Area Index (-)</th>
<th>Rooting Depth (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>January</td>
<td>4.0</td>
<td>300</td>
</tr>
<tr>
<td>February</td>
<td>4.0</td>
<td>300</td>
</tr>
<tr>
<td>March</td>
<td>4.0</td>
<td>300</td>
</tr>
<tr>
<td>April</td>
<td>4.5</td>
<td>500</td>
</tr>
<tr>
<td>May</td>
<td>5.0</td>
<td>500</td>
</tr>
<tr>
<td>June</td>
<td>6.0</td>
<td>800</td>
</tr>
<tr>
<td>July</td>
<td>6.0</td>
<td>800</td>
</tr>
<tr>
<td>August</td>
<td>6.0</td>
<td>800</td>
</tr>
<tr>
<td>September</td>
<td>6.0</td>
<td>800</td>
</tr>
<tr>
<td>October</td>
<td>5.0</td>
<td>500</td>
</tr>
<tr>
<td>November</td>
<td>4.5</td>
<td>500</td>
</tr>
<tr>
<td>December</td>
<td>4.0</td>
<td>300</td>
</tr>
</tbody>
</table>

New grids for Manning’s M, depression storage, and the paved runoff coefficient were also generated for each LID scenario. Table 11 summarizes the values of these parameters for both types of LIDs modelled. Both types of LID land uses were assigned the same, relatively low, value for Manning’s M of 3.5, which was chosen based on conditions being similar to agricultural or pastured land uses (M = 2.7), but providing slightly less resistance to runoff while still providing reduced runoff capacity compared to urban conditions (M = 11.1). It is difficult to say for certain what the overall Manning’s M value would be for a conglomerate representation of several LIDs that may have a significant variability in their values (e.g. permeable pavement may have a relatively high value for Manning’s M whereas a green roof may have a relatively
low value). Furthermore, each type of LID being modelled (infiltration and retention) may contain low or high values of Manning’s M, which is why a single value was considered appropriate for both types. The relatively low Manning’s M value is meant to provide a slower runoff velocity which may be the result of increased vegetated cover or reduced slope of an LID site, which may occur at either infiltration or retention type LIDs.

New grids for the depression storage were also created. The infiltration based LIDs were assigned the high value of 8.0 mm, which is nearest the value of wetlands in the current conditions model (9.0 mm) to allow for some ponding and promote infiltration. Retention type LIDs were assigned a higher value of 15.0 mm as they would be designed to retain the water and allow for evapotranspiration to occur. The value of 15.0 mm was considered sufficiently high to provide this function given the value for forested conditions was 10.0 mm. Both LID land uses were assigned a value of 0.0 for the paved runoff coefficient, since this parameter represents the fraction of ponded water draining to the stream via a drainage network.

<table>
<thead>
<tr>
<th>Land Use</th>
<th>Manning’s M (m^{1/3} \cdot s^{-1})</th>
<th>Depression Storage (mm)</th>
<th>Paved Runoff Coefficient (-)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Infiltration LID</td>
<td>3.5</td>
<td>8.0</td>
<td>0</td>
</tr>
<tr>
<td>Retention LID</td>
<td>3.5</td>
<td>15.0</td>
<td>0</td>
</tr>
</tbody>
</table>

The Two-Layer Water Balance method of modelling UZ flow in MIKE SHE requires the surface soil properties to be defined for computing infiltration. The base scenario contained the four soil profiles distributed throughout the model domain: sand, gravel, silt/till, and clay (Table 5). A new soil profile was generated for the infiltration LIDs that allows for enhanced infiltration, particularly via the higher $K_{sat}$. The retention LIDs were assigned the same values for infiltration parameters as that of gravel in the current conditions model to allow for some
infiltration that they may provide, but not at the enhanced level of the infiltration LIDs. Figure 18 shows the distribution of the soil profiles used in the 20% conversion scenario; the red dots on the figure indicate the LIDs which contain the enhanced infiltration parameters.

Table 12: Unsaturated zone soil profile parameters for LID cells

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>Water content at saturation (-)</th>
<th>Water content at field capacity (-)</th>
<th>Water content at wilting point (-)</th>
<th>Saturated hydraulic conductivity (m·s⁻¹)</th>
<th>Soil suction at the wetting front (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Infiltration LID</td>
<td>0.25</td>
<td>0.15</td>
<td>0.04</td>
<td>1 × 10⁻⁵</td>
<td>-0.2</td>
</tr>
<tr>
<td>Retention LID</td>
<td>0.30</td>
<td>0.20</td>
<td>0.04</td>
<td>6 × 10⁻⁶</td>
<td>-0.2</td>
</tr>
</tbody>
</table>

Figure 18: Distribution of soil layers in the 20% LID scenario.
5. MODEL CALIBRATION AND VALIDATION

5.1. CALIBRATION

5.1.1. Calibration Data

The model was calibrated using an observed data set of daily discharge from a Lake Simcoe Region Conservation Authority gauge located in Lovers Creek at Tollendale near the mouth of the Lovers Creek subwatershed (Figure 7). Observed discharge data was available from 2001 – 2009; however, the 2009 data was not corrected for the impacts of winter ice and freezing and was therefore not representative of actual conditions. The data from 2005 – 2008 was also suspect. Annual flow volumes for the gauge increased from an average of 275 mm·yr$^{-1}$ (0.52 m$^3$·s$^{-1}$) for the period of 2001 – 2004 to an average of 523 mm·yr$^{-1}$ (0.99 m$^3$·s$^{-1}$) for 2005 – 2008. A T-test analysis indicated that the mean annual daily precipitation from 2005 – 2008 (2.44 mm·day$^{-1}$) was less than the mean precipitation in 2001 – 2004 (2.66 mm·day$^{-1}$), but not significantly (p < 0.05). However, both potential evapotranspiration and streamflow were significantly greater in 2005 – 2008 (2.22 mm·day$^{-1}$ and 0.99 m$^3$·s$^{-1}$) than 2001 – 2004 (1.99 mm·day$^{-1}$ and 0.52 m$^3$·s$^{-1}$). A simple water balance analysis was also conducted and showed that total discharge (as indicated by the gauge) plus evapotranspiration were greater than precipitation for the years 2005 – 2008, an unlikely, though not impossible, scenario. It could be that the saturated zone is losing storage and acting as the source of extra streamflow. Unfortunately, data on groundwater levels was not available to determine if it may be sourcing streamflow during this time. However, this argument does not explain why this would be consistently happening in the years 2005 – 2008 and not in 2001 – 2004 or why the discharge in those years is significantly
greater than the 2001 – 2004 discharge. It is possible that the local geometry and cross-section at
the location of the gauge was modified, which changed the stage discharge relationship and the
rating curve on which the gauge was based. Therefore, the data from 2005 – 2008 was also
considered to be unusable for model calibration. Thus, only the daily discharge data from the
Lovers Creek gauge from the period of 01/01/2001 – 31/12/2004 was used for model calibration.
This was consistent with the calibration data from the AquaResource Inc. Barrie model, which
also used the Lovers Creek gauge data from this period.

5.1.2. Calibration Methodology

The model was run several times from a period of 01/01/2000 – 31/12/2004, using a 1-
year ‘warm-up period’ (2000) so the initial conditions would not impact the simulated results.
Several tools for assessing the model’s fit to the observed data were used. A visual comparison
was made using graphs of the annual and monthly flow volumes, daily streamflow, and a flow
duration curve.

Several calibration statistics (also called model efficiency measures) were computed on
the daily and monthly streamflow data to provide a quantitative assessment of model
performance. These included the coefficient of determination, \( r^2 \) (Equation 26), the Nash-
Sutcliffe efficiency index, \( E \) (Equation 27), the logarithmic Nash-Sutcliffe efficiency index, \( \ln E \),
and a modified form of \( E \), called \( E_1 \) (Equation 28), defined by Krause et al. (2005), as well as the
root mean squared error, \( RMSE \), and the mean absolute error, \( MAE \). The efficiency measures, \( \ln E \) and \( E_1 \) were sensitive to low flows compared to \( E \), which is based on squared differences and
is more sensitive to peak flows of the hydrograph (Krause, Boyle, & Bäse, 2005). \( E_1 \) and \( \ln E \)
are both sensitive to low flows, but they do not lose all sensitivity to peak flows (Krause, Boyle,
& Bäse, 2005).
\[ r^2 = \left( \frac{\sum_{i=1}^{n} (O_i - \bar{O})(P_i - \bar{P})}{\left( \sum_{i=1}^{n} (O_i - \bar{O})^2 \right)^{1/2} \left( \sum_{i=1}^{n} (P_i - \bar{P})^2 \right)^{1/2}} \right)^2 \]

*Equation 26: Calculation of the coefficient of determination.*

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

*Equation 27: Calculation of the Nash-Sutcliff Efficiency Index*

\[ E_1 = 1 - \frac{\sum_{i=1}^{n} |O_i - P_i|}{\sum_{i=1}^{n} |O_i - \bar{O}|} \]

*Equation 28: Calculation of the modified Nash-Sutcliffe Efficiency Index.*

Where \( O \) and \( P \) are the observed and predicted streamflow data. These efficiency measures were computed on both daily and monthly streamflow values. For the calibration to daily data, there were 1,461 data points, and for the monthly data there were 48.

Model calibration was completed manually, with each successive simulation having modified input parameters that were adjusted based on the results of the previous simulation. In this way specific inputs could be targeted to produce desired results. For example, during one simulation it was observed that the simulated hydrograph had spring melt that was occurring too slowly, simulating lower initial spring melt that lasted further into the summer, which prompted an increase in the degree day coefficient controlling snowmelt, ultimately from 3 to 5 mm·day\(^{-1}\)·°C\(^{-1}\). Table 13 summarizes the parameters modified during the calibration process.
Table 13: Parameters used and tested in model calibration

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Original Value</th>
<th>Final Calibration Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Drainage level</td>
<td>MIKE SHE dfs2 grid file</td>
<td>1 m below ground surface</td>
</tr>
<tr>
<td>Drainage level time constant (s⁻¹)</td>
<td>$1 \times 10^{-6}$</td>
<td>$1 \times 10^{-7}$</td>
</tr>
<tr>
<td>Degree day coefficient (mm·day⁻¹·°C⁻¹)</td>
<td>3</td>
<td>5</td>
</tr>
<tr>
<td>Exchange flow conductance</td>
<td>Aquifer + Bed</td>
<td>River Bed Only</td>
</tr>
<tr>
<td>Exchange flow river bed leakage coefficient (s⁻¹)</td>
<td>$1 \times 10^{-6}$</td>
<td>$8 \times 10^{-7}$</td>
</tr>
<tr>
<td>Reference evapotranspiration</td>
<td>$0.9 \times \text{PET}$</td>
<td>$1.0 \times \text{PET}$</td>
</tr>
<tr>
<td>Paved runoff coefficient</td>
<td>0.3</td>
<td>0.3</td>
</tr>
<tr>
<td>Saturated zone specific yield</td>
<td>0.2</td>
<td>0.2</td>
</tr>
<tr>
<td>Cross-section width (m)</td>
<td>100</td>
<td>70</td>
</tr>
<tr>
<td>MIKE 11 branch type</td>
<td>Fully dynamic</td>
<td>Kinematic routing</td>
</tr>
</tbody>
</table>

An auto-calibration of the model was planned, but not ultimately carried out. Manual calibration was started initially as a means to bring the simulation reasonably close to observed results so that an auto-calibration could refine the model further. An auto-calibration that begins from a simulation that is too far from an acceptable fit with observed results may run hundreds of simulations (taking several weeks to complete) and may or may not converge on a solution, depending on the selection of parameters and the criteria used to assess the model’s fit.

Furthermore, when an attempt was made to set up an auto-calibration routine, some of the traditionally important or sensitive parameters in hydrologic models could not be used in the calibration. This was because of the way they had been specified within MIKE SHE. For example, the hydraulic conductivities in the saturated zone, often a sensitive parameter with significant uncertainty and therefore a good candidate for calibration, were specified in MIKE SHE using a DHI two-dimensional grid file (*.dfs2). These types of files, and the data they contain, cannot be modified by the auto-calibration tool in MIKE SHE. Other data sets that were candidates for calibration but suffered from this same problem included Manning’s roughness, depression storage, and the paved runoff coefficient grids. For these reasons, an auto-calibration
was not run on the model. Later discussion with a DHI representative indicated that the saturated zone parameters could be included in an auto-calibration routine, if they are defined in MIKE SHE by geological units and not geological layers. The overland flow Manning’s $M$ and depression storage cannot be specified this way and so, cannot be included in an auto-calibration of MIKE SHE.

5.1.3. Calibration Results

The final calibrated model performed fairly well, accurately predicting observed streamflow at the Lovers Creek gauge under most circumstances, although there are some notable differences between the simulated and observed data sets, which are described below.

The average annual streamflow for the calibration period was a close match for all years except 2001 (Figure 19). Annual volume errors for 2002 – 2004 were all less than 20% and 2003 had only a 3% error. However, 2001 had an error of 49%, which was the result of a poorly simulated summer period, where observed mean monthly flows for June – September were 0.18 m$^3$·s$^{-1}$ or less, but simulated mean monthly flows were all greater than 0.30 m$^3$·s$^{-1}$. The largest error occurred in August with observed and simulated mean monthly flows of 0.08 and 0.31 m$^3$·s$^{-1}$, respectively. However, the strong performance of the other years suggested the model was performing fairly well.

The mean monthly flow volume from 2001 – 2004 for observed and simulated data was computed and is compared in Figure 20. Observed and simulated results, for the most part, compare well with errors less than 20%; however, July, August, September, and October show poor results, with substantial errors of 54%, 138%, 24%, and 92% respectively. Table 14 shows a sample of data from a portion July of 2002. Neither low flows nor high flows are being accurately represented by the data in this table. Simulated low flows are higher than observed
and the response of the simulated data to precipitation inputs is greater and of longer duration. This explains the discrepancy in October as well, which had more frequent days of moderate precipitation, but since low flows were higher the response to these moderate precipitation events was greater. However, September displayed a similar trend, but still had a much better fit between observed and simulated streamflow volumes. Despite these few months of inaccurate flow volumes, the calibration was thought to be good.

![Mean annual streamflow at Lovers Creek gauge.](image1)

**Figure 19:** Mean annual streamflow at Lovers Creek gauge.

![Mean monthly flow volume for observed and simulated data from 2001 – 2004.](image2)

**Figure 20:** Mean monthly flow volume for observed and simulated data from 2001 – 2004.
Table 14: Sample of July data showing poor fit for low and high flows.

<table>
<thead>
<tr>
<th>Date</th>
<th>Precipitation (mm)</th>
<th>Streamflow (m·s⁻¹)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2002-07-11</td>
<td>0.0</td>
<td>0.14</td>
</tr>
<tr>
<td>2002-07-12</td>
<td>0.0</td>
<td>0.13</td>
</tr>
<tr>
<td>2002-07-13</td>
<td>0.0</td>
<td>0.12</td>
</tr>
<tr>
<td>2002-07-14</td>
<td>0.0</td>
<td>0.12</td>
</tr>
<tr>
<td>2002-07-15</td>
<td>0.0</td>
<td>0.14</td>
</tr>
<tr>
<td>2002-07-16</td>
<td>0.0</td>
<td>0.14</td>
</tr>
<tr>
<td>2002-07-17</td>
<td>0.0</td>
<td>0.13</td>
</tr>
<tr>
<td>2002-07-18</td>
<td>0.0</td>
<td>0.14</td>
</tr>
<tr>
<td>2002-07-19</td>
<td>0.0</td>
<td>0.14</td>
</tr>
<tr>
<td>2002-07-20</td>
<td>0.0</td>
<td>0.13</td>
</tr>
<tr>
<td>2002-07-21</td>
<td>2.0</td>
<td>0.13</td>
</tr>
<tr>
<td>2002-07-22</td>
<td>25.8</td>
<td>0.16</td>
</tr>
<tr>
<td>2002-07-23</td>
<td>6.9</td>
<td>0.67</td>
</tr>
<tr>
<td>2002-07-24</td>
<td>0.3</td>
<td>0.30</td>
</tr>
<tr>
<td>2002-07-25</td>
<td>0.0</td>
<td>0.23</td>
</tr>
<tr>
<td>2002-07-26</td>
<td>3.3</td>
<td>0.21</td>
</tr>
</tbody>
</table>

The daily streamflow hydrographs (Figure 21) comparing observed and simulated discharge at the Lovers Creek gauge show that the simulated flows matched the observed trends in the flows well. The simulated flows do appear to be slightly flashier, responding to smaller storm events with daily peak flows that are not present in the observed data. This is especially true for 2001; other years display this trend, but not as often as the year 2001. Daily peak storm flows are simulated well, except for a few circumstances where simulated daily peak flows are noticeably less than observed (e.g. 15/06/2002 where observed daily flow is 15.0 m³·s⁻¹ and simulated daily flow is 3.4 m³·s⁻¹). Low flows appear to be simulated reasonably well, except for the aforementioned tendency of the simulated flows to contain daily peaks from small precipitation events. However, the low flows are generally being over estimated in the simulated results, as evidenced by the flow duration curve which shows a very close fit between observed and simulated results until approximately the 70% exceedence discharge, when the observed and
simulated discharge data appear to diverge (Figure 22), indicating that the low flows were not well represented by the model. The calibration plot of observed vs. simulated discharge (Figure 23) shows a clear threshold at approximately 0.17 m$^3$·s$^{-1}$, which the simulated discharge never fell below, further indicating that low flows may not be properly represented in the model.
Figure 21: Hydrograph of daily streamflow for years 2001 – 2002 (top) and 2003 – 2004 (bottom).
Numerical stability of the MIKE 11 hydraulic model may be a reason for the poor fit of low flow data. In MIKE 11, numerical instability may arise if rapid drying out of a river occurs. MIKE SHE uses the “maximum fraction of H-point volume” parameter to limit the amount of exchange flow from the river to the saturated zone in any one time step. This parameter represents the fraction of total streamflow volume that may be lost to the saturated zone in a
single time step. The default value of 0.9 was used in this study. The MIKE 11 simulation gave warning messages (Figure 24) about having reduced the amount of outflow from the river to the SZ in some time steps and cross-sections so as to prevent drying out of the river. The reduction in exchange flow from the river to the SZ may be partly to blame for the poor simulation of low flows in the model.

Figure 24: Warning message from completed MIKE SHE simulation indicating reduced exchange flow between river and saturated zone to avoid drying river.

Several measures of efficiency were computed for both the daily data set and the monthly mean data set. Table 15 summarizes these statistics. The monthly data set is useful for examining overall trends, since daily streamflow data may fluctuate significantly, but efficiency measures are artificially high when computed on mean monthly flows because the variability is averaged out. Both daily and monthly statistics should be considered when determining a model’s closeness of fit.

The daily and monthly \( r^2 \) values of 0.38 and 0.66 indicate that the model explains 38% of the variability in daily discharge and 66% of the variability in monthly discharge. The increase in \( r^2 \) in the monthly data set is due to the reduced variability within the monthly data set, but may also indicate that the model performs well for prediction of longer term processes. The regular and modified Nash-Sutcliffe coefficients for the daily and monthly data indicate that the model fits fairly well to both high flow and low flow conditions. The values for \( E_i \) are lower, which may indicate that the model is performing less well during low flow conditions, but it is more difficult to achieve high values with this measure (Krause et al., 2005), so even lower numbers may indicate a reasonable fit.
Table 15: Model efficiency measures computed for daily and monthly discharge data sets.

<table>
<thead>
<tr>
<th>Model Efficiency Measures</th>
<th>( r^2 )</th>
<th>( E )</th>
<th>( \ln E )</th>
<th>( E_1 )</th>
<th>( \text{RMSE} ) (( m^3 \cdot s^{-1} ))</th>
<th>( \text{MAE} ) (( m^3 \cdot s^{-1} ))</th>
</tr>
</thead>
<tbody>
<tr>
<td>Daily</td>
<td>0.38</td>
<td>0.36</td>
<td>0.34</td>
<td>0.23</td>
<td>0.79</td>
<td>0.31</td>
</tr>
<tr>
<td>Monthly</td>
<td>0.66</td>
<td>0.64</td>
<td>0.40</td>
<td>0.34</td>
<td>0.52</td>
<td>0.32</td>
</tr>
</tbody>
</table>

Despite the lack of an auto-calibration, the manual calibration was considered to be fair. An auto-calibration would refine the results of this calibration further and likely produce more accurate predictions of streamflow at the Lovers Creek gauge. Although the daily streamflow predictions have significant uncertainty associated with them, comparisons between different simulations should still produce meaningful results, particularly for longer term trends and processes.

In general, the calibration statistics presented slightly lower than calibration statistics reported in the literature, but still indicate a reasonable model performance. Sahoo (2006) reported correlation coefficient, \( R \), values of 0.7 or higher (\( r^2 \) of 0.49) and Thompson et al. (2004) had \( r^2 \) values in the range of 0.40 to 0.80 for the calibration of MIKE SHE to monthly groundwater level data, compared to an \( r^2 \) value of 0.66 to monthly streamflow data in this study. Im et al. (2009) calibrated to daily streamflow data and had higher values of \( r^2 \) and \( E \) at 0.79 and 0.54, respectively, compared to 0.38 and 0.36 in this study. Furthermore, Im et al. (2009) reported that other applications of MIKE SHE produced \( R \) values ranging 0.51 to 0.80 (\( r^2 \) from 0.26 to 0.64) and model efficiency (Nash-Sutcliffe) from -0.85 to 0.48. Im et al. (2009) also reported a \( \text{RMSE} \) of 7.07 \( m^3 \cdot s^{-1} \), and a \( \text{MAE} \) of 2.53 \( m^3 \cdot s^{-1} \), which were 1.3 and 0.5 times the mean observed streamflow of 5.55 \( m^3 \cdot s^{-1} \), respectively. By contrast, in this study, the daily \( \text{RMSE} \) of 0.79 \( m^3 \cdot s^{-1} \) and \( \text{MAE} \) of 0.31 \( m^3 \cdot s^{-1} \) were 1.5 and 0.6 times the mean observed streamflow of 0.52 \( m^3 \cdot s^{-1} \). Thus, the calibration statistics reported in this study are slightly lower than other values reported in literature for calibration of MIKE SHE values, but are still
considered reasonable for the purposes of comparing the impact of land use change on the streamflow regime. Typically values of Nash-Sutcliffe greater than 0.6 indicate a reasonable fit and values greater than 0.8 indicate a good fit, while values greater than 0 indicate that the model has some utility (Nash & Sutcliffe, 1970). Overall, the results of the calibration were considered to be fair.

The results of the calibration should also be interpreted within the objectives of this modelling study. It was not the intention of the study to exactly predict the daily streamflow values. Rather the objectives require the development of a model that reasonably predicts daily discharge and allows for the comparison of hypothetical land use scenarios.
5.2. Model Validation

5.2.1. Validation Data

There was a general lack of available observed data with which to compare simulated results. The 2005 – 2009 streamflow timeseries data was considered unusable and the 2001 – 2004 data was used for calibration, which left only a small set of observed spotflow data for model validation. This observed data included baseflow measurements at 21 locations throughout the watershed between June and October of 2008 and a set of spotflow measurements at the Lovers Creek gauge between 2003 and 2011. These spotflow measurements were taken manually by LSRCA employees and represent a single snapshot of streamflow in time. The baseflow measurements were spotflow measurements that were conducted after a period of at least 72 hours with no precipitation, while the spotflow measurements at the gauge were taken during a range of low and high flows in an attempt to acquire flow data for a representative range of flows. Five of the baseflow sites were excluded because they were not located on the simplified stream network used in the MIKE SHE model, which left 16 locations each with 2 or 3 baseflow measurements, for a total of 43 baseflow measurements. The spotflow data at the Lovers Creek gauge contained 123 measurements from 2003 – 2011, but only data within the simulation period and not in the calibration period was considered for model validation. This left 70 spotflow measurements for model validation.

It must be understood that the validation data contains considerable uncertainty. Not only is the total quantity of data limited, but the measurements represent a single moment in time and space, not a continuous timeseries of observations, and therefore may not be representative of actual conditions. Also, since they are measured at a particular time of day (e.g. 2:20PM), they
may not be accurate predictors of mean daily discharge, since the daily discharge may vary significantly in any day. For the baseflow data, this error may be reduced, since the absence of rain for at least 72 hours prior to the measurement may make for more consistent conditions in the stream on that day, but it does not eliminate this error altogether.

There was an additional source of uncertainty in that the locations of the baseflow data did not match up with the ‘Q-points’, which are locations along the MIKE 11 river network at which MIKE 11 outputs discharge data. Q-points are located halfway between cross-sections in the MIKE 11 model; as such they are separated by the same distance as the cross-sections, which was in the range of 200 – 400 m. The nearest Q-point to each baseflow measurement was used for comparison. The difference between flow measurements at the two surrounded Q-points was found to be small, 0.005 m$^3\cdot$s$^{-1}$ on average. Only one location had a substantial difference, and it was only for one day on which baseflow measurements occurred. Location LV19, located on River 1 at a chainage of 5807 m, was approximated by the Q-point located at 1:5884 m. The next was approximately 300 m upstream at 1:5578 m. On 19/08/2008, the baseflow measurement at LV19 was 0.72 m$^3\cdot$s$^{-1}$, the simulated daily discharges at 1:5578 and 1:5884 m were 0.64 and 0.75 m. A linear interpolation of this change in flow estimated the flow at 1:5807 m to be 0.72 m. This was the only point where the assumption of using the nearest Q-point had an impact.

Given the results of the model calibration and the uncertainty associated with the validation data, it was not expected that the model would match too closely with either the baseflow or the spotflow data. Figure 25 shows the location that the baseflow measurements were taken throughout the watershed.
5.2.2. Validation Results

The simulated discharge data matched relatively well with observed baseflow data (Figure 26), with an $r^2$ of 0.62; however, this needs to be interpreted carefully. Although the $r^2$ value was reasonable and suggested a good fit, the slope of the regression line was only 0.53, when an ideal value is 1.0. This suggests that the simulated discharge is consistently under-predicting the measured discharge. Furthermore, the baseflow measurements for sites located in the upstream half of the watershed are located in the bottom left corner of the plot, from 0.0 – 0.1 m$^3$·s$^{-1}$. The simulated discharge at many of these points is 0.0 m$^3$·s$^{-1}$, which further suggests the consistent under-prediction of the simulated discharge in these areas. When considering the
relatively high $r^2$ value with the consistent under-prediction it can be said that the overall model performance is relatively poor.

The simulated discharge matched less well to the spotflow data than it did to the baseflow data, with an overall $r^2$ value of 0.07, suggesting a very poor fit (Figure 27). There are three data points in this plot that are causing considerable variability in the simulated data, and if these three days are removed the result is considerably better with an $r^2$ value of 0.26 (Figure 28). One of these data points was not shown on the graph as the simulated streamflow was greater than 10 m$^3$·s$^{-1}$.

Each of these days (29/09/2005, 11/09/2007, and 22/09/2010) had significant amounts of precipitation fall in the previous 48 hours (38.3, 38.1, and 32.7 mm respectively), so the variability in discharge may be considerable, such that the spotflow measurements may not accurately represent the mean daily discharge. Even so, the results show a relatively poor fit between the simulated and spotflow data as the model is again consistently under-predicting the mean daily discharge.

![Figure 26: Measured baseflow vs. simulated discharge.](image)
Overall, it can be said that the model performance for the validation period is poor, but this was the expected result, given the overall performance of the model during the calibration period and the considerable uncertainty associated with the validation data.

Figure 27: Measured spotflow vs. simulated discharge.

Figure 28: Measured spotflow vs. simulated discharge with three outliers removed.
5.3. **Overall Assessment of Model Reliability**

In any modelling exercise, the overall model reliability must be assessed before it can be used to make predictions, which is done through the calibration and validation process. On the qualitative scale of ‘poor’, ‘fair’, ‘reasonable’, ‘good’, and ‘excellent’, the calibration results of this model were considered to be fair, while the validation results were considered to be relatively poor. This indicates the model may not be a reliable tool in the prediction of discharge within the Lovers Creek subwatershed. Nonetheless, the model was considered to be useful for comparison of hypothetical scenarios with different land use and stormwater management practices within the watershed, to examine the impact of urbanization and the mitigation potential of LID.

If, in the future, this model were to be used for predictive purposes for actual discharge, its overall performance should be improved. This could be achieved in a number of ways. First, more calibration data is necessary. The reliability of a watershed model that is calibrated only to the streamflow at the mouth of the watershed is never 100%, even if the model efficiency measures displayed a perfect fit, since this only assesses the goodness of fit at one location within the watershed. MIKE SHE simulates both surface water and groundwater and both types of data should be used for calibration. Thompson et al. (2004) claimed that the use of MIKE SHE required calibration to groundwater and surface water data. Second, to improve on the performance of the model in this study, better data should be used to represent the subsurface and stream network. The lack of detail in the MIKE 11 cross-sections became a source of difficulty in the modelling process and is a source of error in the model results. A more detailed subsurface geology, especially in the upper layers near the ground surface and at locations near the stream network, needs to be included in the model. The 9-layer structure used by
AquaResource Inc. in the FEFLOW model may be sufficient, but it may also be helpful to exclude the lowest layers to save simulation time, since the model took approximately 30 hrs to run. Using the main aquitard (layer 2 in this model) as the bottom of the subsurface. Third, an auto-calibration should be run. This would require the re-processing of much of the input data, but an auto-calibration routine, run from this already fairly well calibrated simulation, might refine the model into a more reliable version.
6. RESULTS AND DISCUSSION

6.1. STREAMFLOW RESULTS AT MOUTH OF LOVERS CREEK

The Lovers Creek subwatershed was modelling using MIKE SHE under pre-development, current (urbanized), and five LID scenarios. The computed streamflow regimes were compared with respect to low flows, high flows, the $T_{Q_{\text{mean}}}$ metric, and groundwater discharge to determine the impact of these land use changes. The $T_{Q_{\text{mean}}}$, which is the fraction of days in a year that had a daily discharge greater than mean annual discharge, was computed and compared for each model scenario, with lower values indicating a reduced Benthic Index of Biologic Integrity (B-IBI). The percent reduction of $T_{Q_{\text{mean}}}$ from pre-development to current conditions that is restored by each LID scenario was computed using Equation 29. This value indicates the percentage improvement in $T_{Q_{\text{mean}}}$ for the LID scenario.

$$\% \text{ Restored} = \frac{T_{Q_{\text{mean}}}(\text{LID}) - T_{Q_{\text{mean}}}(\text{Current})}{T_{Q_{\text{mean}}}(\text{Pre – development}) - T_{Q_{\text{mean}}}(\text{Current})}$$

Equation 29: Calculation of the percent of pre-development $T_{Q_{\text{mean}}}$ restored by LID scenario

The streamflow results at the Tollendale gauge showed that the current conditions scenario had increased peak storm flows, but also a substantially increased baseflow regime, compared to pre-development conditions (Figure 29). It was expected that the baseflow would be reduced by the increase in urban land use through reduced infiltration and groundwater recharge. The underlying assumption associated with this expectation is that the average annual streamflow volume does not change with urbanization, but there is a redistribution of flow from baseflow to event flow. However, in the Lovers Creek subwatershed, a significant decrease ($p < 0.01$) in annual evapotranspiration occurred from the pre-development mean of 628 mm yr$^{-1}$ to
the current conditions mean of 539 mm yr\(^{-1}\). The reduced volume of ET leaves more water available to supply streamflow, resulting in the higher baseflow volumes under current land use conditions.

The shift in the water balance and increase in baseflow under urban conditions was especially true in dry years compared to wet years. For example, in the dry water year of 1992, there was a total of 772 mm of P and 567 mm of ET in the pre-development scenario, leaving a surplus (P minus ET) of only 205 mm to supply streamflow and in the current conditions scenario, 502 mm of ET, leaving a surplus of 270 mm to supply streamflow, a 32% increase over pre-development conditions. In the wet year of 1996, there was 1212 mm of P and 645 mm of ET in the pre-development scenario, for a surplus of 567 mm, and only 574 mm of ET in the current conditions scenario, for a surplus of 638 mm, a 13% increase over pre-development conditions. Table 16 summarizes the modelled ET and surplus of P over ET for pre-development and current land use scenarios for the dry and wet water years of 1992 and 1996, respectively. In both wet and dry years, ET is reduced under current conditions, leaving more water available to supply streamflow, but this has more of an impact in dry years where ET represents a higher fraction of total P.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Dry Water Year, 1992</th>
<th>Wet Water Year, 1996</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Pre-development</td>
<td>Current</td>
</tr>
<tr>
<td>Precipitation (mm)</td>
<td>772</td>
<td>772</td>
</tr>
<tr>
<td>Evapotranspiration (mm)</td>
<td>567</td>
<td>502</td>
</tr>
<tr>
<td>Surplus (mm)</td>
<td>205</td>
<td>270</td>
</tr>
</tbody>
</table>

Despite having higher baseflow, the streamflow regime in the current land use scenario was flashier than the pre-development scenario (Figure 29), resulting in lower values for \( T_{Q_{mean}} \). This was true for each year in the simulation period of water year 1990 – 2010, with the
exception of water year 2003, which saw no change in \( T_{Q_{\text{mean}}} \) (Table 17). The lower \( T_{Q_{\text{mean}}} \) values for the current conditions scenario are indicative of the urbanized watershed with quicker streamflow responses to precipitation inputs (Booth, 2005).

The two 20% LID scenarios (LID13-20 and LID14-20) are also displayed on Figure 29, but the differences between streamflow in these two simulations were negligible, with an average absolute difference of 0.005 \( m^3\cdot s^{-1} \) or 0.5%. They appear as a single line on the hydrograph. The 5%, 10%, and 15% scenarios (LID13-05, LID13-10, and LID13-15) are not shown on Figure 29; but they plot between the current conditions and 20% LID hydrographs.

The LID13-20 and LID14-20 scenarios were very similar to the current conditions scenario. There are, however, two notable differences: the daily peak storm flows were reduced and the baseflow was increased (Figure 29). The LID13-20 and LID14-20 scenarios resulted in a decrease in peak storm flows of approximately 15% and an increase in baseflow of approximately 4%, compared to the current scenario. These results were expected as the LID types modelled are designed to reduce the volume of stormwater runoff and increase infiltration into the soil and groundwater recharge. The results provide some evidence that LIDs may mitigate impacts of urbanization on GDEs. However, the 4% increase in baseflow is small and would not likely result in significant ecological benefits.

The \( T_{Q_{\text{mean}}} \) results (Table 17) also indicate that the LID scenarios may provide some ecological benefit to urban streams. In Table 17, the percent reduction of \( T_{Q_{\text{mean}}} \) from pre-development to current conditions that is restored by LID is presented in brackets beside each \( T_{Q_{\text{mean}}} \) value for the LID land use scenarios. LID13-20 and LID14-20 restored an average of 30% of the drop in \( T_{Q_{\text{mean}}} \) from pre-development to urban conditions (Table 17). For example, water year 1992 had a pre-development \( T_{Q_{\text{mean}}} \) of 0.30, current conditions \( T_{Q_{\text{mean}}} \) of 0.23, an
LID13-20 \(T_{\text{Qmean}}\) of 0.24 and an LID14-20 \(T_{\text{Qmean}}\) of 0.24, for an 11% and 15% restored, respectively.

The general trend of reduced B-IBI with reduced \(T_{\text{Qmean}}\) reported by Booth et al. (2004) was expected to hold true for Lovers Creek, but it was considered unlikely that the specific relationship was transferable to the Lovers Creek subwatershed. For this reason, the B-IBI was not computed, but it is recognized from the literature that it would be expected to decrease as \(T_{\text{Qmean}}\) is reduced. Thus, the decrease in \(T_{\text{Qmean}}\) from pre-development to current conditions likely indicates a decrease in the ecological condition and the increase from the current to the LID scenarios would likely indicate an increase in ecological condition of the stream. Overall, these increases in \(T_{\text{Qmean}}\) are moderate and they may not yield significant changes to ecological conditions of GDEs.
Figure 29: Hydrograph of streamflow at Tollendale Gauge for simulated scenarios under dry conditions (water year 1992, top) and wet conditions (water year 1996, bottom).
Table 17: Summary of T_{Qmean} values at the Tollendale Gauge

<table>
<thead>
<tr>
<th>Water Year</th>
<th>Pre-dev</th>
<th>Current</th>
<th>LID13-05</th>
<th>LID13-10</th>
<th>LID13-15</th>
<th>LID13-20</th>
<th>LID14-20</th>
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</thead>
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<tr>
<td>Min.</td>
<td>0.16</td>
<td>0.16</td>
<td>0.15</td>
<td>0.16</td>
<td>0.16</td>
<td>0.16</td>
<td>0.16</td>
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<tr>
<td>Median</td>
<td>0.27</td>
<td>0.20</td>
<td>0.19</td>
<td>0.20</td>
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<td>0.21</td>
<td>0.21</td>
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<tr>
<td>Mean</td>
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<td>Max.</td>
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<td>0.25</td>
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<td>1990</td>
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<td>0.25</td>
<td>0.25</td>
<td>0.25</td>
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</tr>
<tr>
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<td>2009</td>
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As expected, each successive LID scenario with increased area was more effective than the last. LID13-05 had lower $T_{Q_{\text{mean}}}$ values, on average, than the current conditions scenario. This is because it did not substantially alter the mean annual flow, $Q_{\text{mean}}$, (from 0.53 to 0.52 m$^3$·s$^{-1}$) and was only slightly successful in increasing the low flows (+1.0%) and decreasing peak flows (-2.2%). Low flows that were determined as being less than or equal to the flow that was equalled or exceeded 90% of the days within the simulation period ($Q_{90}$) and high flows were determined as being greater than or equal to the flow that was equalled or exceeded 10% of the days within the simulation period ($Q_{10}$). This resulted in a streamflow regime that was relatively unchanged from current conditions, but with peaks from some small storms that were reduced to below $Q_{\text{mean}}$, which slightly increased the number of days with discharge less than $Q_{\text{mean}}$ and therefore decreased $T_{Q_{\text{mean}}}$. For example: on 07/03/1991 there was 11.4 mm of precipitation, the daily discharge was 0.55 m$^3$·s$^{-1}$ for current conditions which was greater than the 0.50 m$^3$·s$^{-1}$ which was the mean annual daily discharge for water year 1991, and the daily discharge was 0.48 m$^3$·s$^{-1}$ for the LID13-05 scenario, which was less than the mean annual daily discharge of 0.50 m$^3$·s$^{-1}$. By contrast, LID13-20 was more effective at increasing low flows (+4.4%) and reducing peak flows (-12.9%). This resulted in a less flashy hydrograph with a higher and more consistent baseflow regime and high peak storm flows occurring less frequently. Therefore, each of the LID scenarios was more effective at reducing the peak storm flows than it was at increasing the baseflow. Table 18 summarizes the changes to low and high flows of each LID scenario versus the current conditions.

The hydrologic changes simulated by the LID scenarios were relatively minor, but they should be taken in context of the watershed and the total amount of LID. The original land use had only 27% high density urban, so the 20% conversion scenario converted only 5.3% of the
total watershed to LID. The streamflow results are being influenced by the presence of other significant land uses (e.g. agricultural) which were unchanged between the all current and LID scenarios. It is expected that the impact of LID would be amplified in a more urbanized watershed or on a more local scale within a predominantly high density urbanized area.

The differences between the two different 20% LID scenarios were negligible, with a mean absolute difference of 0.005 $\text{m}^3\cdot\text{s}^{-1}$ or 0.5%. This is likely due to the constraints of modelling LID using MIKE SHE. There were limited parameters available to simulate the hydrologic processes affected by LID. The most significant differences between LID13-20 and LID14-20 were the amount of depression storage (8.0 vs. 15.0 mm, respectively) and the saturated hydraulic conductivity used for infiltration ($1.0 \times 10^{-5}$ and $6 \times 10^{-6}$ m$\cdot$s$^{-1}$, respectively). This allowed the LID14-20 to reduce peak storm flows slightly more (16.0% vs. 14.5%) due to its ability to store a larger volume of storm water, but the LID13-20 was slightly better at increasing low flows (4.2% vs. 4.0%) due to increased ability to infiltrate water and recharge the groundwater regime.

The streamflow results at the Tollendale gauge suggest that LID was an effective stormwater management strategy by reducing peak event flows, but also increased low flows, suggesting that it may be of some use for the protection of GDEs. Too great an increase in low flows may as detrimental as a loss of groundwater availability to GDEs; however, this increase in low flows is relatively small, and may not translate into significant ecological changes.

<table>
<thead>
<tr>
<th>LID Scenario</th>
<th>LID13-05</th>
<th>LID13-10</th>
<th>LID13-15</th>
<th>LID13-20</th>
<th>LID14-20</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mean Increase in Low Flows (%)</td>
<td>1.0%</td>
<td>2.2%</td>
<td>3.0%</td>
<td>4.4%</td>
<td>4.3%</td>
</tr>
<tr>
<td>Mean Decrease in Peak Flows (%)</td>
<td>2.2%</td>
<td>5.2%</td>
<td>8.4%</td>
<td>12.9%</td>
<td>13.6%</td>
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</tbody>
</table>
6.2. **GROUNDWATER FLOW AND LEVEL RESULTS**

The general groundwater flow and level results for the pre-development and current land uses can be seen in Figure 30 through Figure 33. These figures show the groundwater flow in the x-direction (positive from west to east, left to right) for Layer 1 of the saturated zone of the MIKE SHE model. Groundwater flow arrows are overlaid and the groundwater levels are shown adjacent to provide the general direction and gradient of groundwater flow. It should be noted that these are snapshots at a particular time, during spring conditions in representative dry and wet years. For the purposes of this discussion, they were selected to illustrate the general groundwater flow paths of the Lovers Creek subwatershed and differences between pre-development and current land use conditions. Groundwater flow in the watershed for all conditions simulated was generally from south to north, but on the west side of Lovers Creek, the groundwater flow was predominantly from west to east, draining towards Lovers Creek, while on the southeast portion of the watershed it is draining mainly south, alongside Lovers Creek.

Given the reduced ET and increased baseflow, it was expected that the groundwater flow and GW discharge to the stream may have increased from pre-development to urban land uses. However, the difference in groundwater flow was minor (Figure 30 vs. Figure 31 and Figure 32 vs. Figure 33). In the spring of the dry year (1992), there may be slightly more groundwater flow west of Lovers Creek near where Rivers 9 and 10 meet Lovers Creek. This is likely a direct result of land uses changes from pre-development to current conditions. Urban land use in this section of the Lovers Creek subwatershed is sparse, but agricultural land use is prominent. Current, agricultural land use would reduce the overall ET in this area compared to the pre-development, forested conditions, but would not increase overall imperviousness, once again indicating a shift in the hydrologic cycle that favours groundwater and streamflow. In the spring
of the wet year (1996), there is no notable difference between the groundwater flows from pre-
development to current land use conditions (Figure 32 and Figure 33).

There is one location of notable interest, which is the junction of Rivers 4 and 1. The
groundwater flow on both the east and west sides of Lovers Creek seem to be concentrated
towards this position. This is also an area of the Lovers Creek Swamp. As such, this location is
a likely candidate for sustained groundwater discharge to the stream and the potential to support
a GDE.

Figure 30: Groundwater flow in the x-direction and head elevation in the saturated zone for pre-development land use on 26/03/1992, the spring of a dry year.
Figure 31: Groundwater flow in the x-direction and head elevation in the saturated zone for current land use on 26/03/1992, the spring of a dry year.

Figure 32: Groundwater flow in the x-direction and head elevation in the saturated zone for pre-development conditions on 28/03/1996, the spring of a wet year.
6.2.1. **Groundwater Discharge to the Stream**

Since streams are a type 2 GDE that require surface expression of GW, it is important to consider the GW discharge to the stream. The overall GW discharge to the stream can be seen in Figure 34, which shows the entire watershed and river network on 28/03/1996. Again, this figure represents a snapshot in time, but is meant to describe the general trends in groundwater discharge to the stream for the watershed. Little detail is visible on this figure, but it is evident that the middle stretch of Lovers Creek, between Rivers 12 and 3, had the most significant volumes of groundwater discharge. This is consistent with the groundwater flow results which saw groundwater flowing westward towards Lovers Creek at this location (Figure 33). Other sections of the river network and the upper tributaries to Lovers Creek had only negligible amounts of SZ exchange flow with the river, on the order of $1 \times 10^{-5} \text{ m}^3\text{s}^{-1}$ or less, either positive or negative, over the 50 m length of cells connecting the river to the SZ. This middle stretch of Lovers Creek did have consistent groundwater discharge; it was not just in the spring of a wet year where groundwater levels and flow are relatively high (Figure 35). This section of Lovers
Creek was not heavily impacted by urban land use, but by agricultural land uses, which are predominant in that area. In the high density urbanized portion of the watershed (north of River 4), very little groundwater discharge occurred for any scenario simulated.

Figure 34: GW exchange flow with the river under current land use on 28/03/1996 with groundwater flow vectors also shown.
The junction of Rivers 4 and 1 was identified from the groundwater flow results as a location that may have significant GW discharge and the potential to support a GDE. This location is within the Lovers Creek Swamp area and could be expected to have groundwater discharge to the stream, or groundwater levels near the ground surface elevation to support the swamp ecosystem. However, in the GW discharge results, it was found that under all land uses, this area only had negligible GW discharge to the stream, on the order of $10^{-4}$ m$^3$·s$^{-1}$ (Figure 36). Under dry conditions of 1992, the median GW discharge at this location was $8 \times 10^{-5}$ m$^3$·s$^{-1}$, and under the wet conditions of water year 1996, the median GW discharge was $1 \times 10^{-4}$ m$^3$·s$^{-1}$, neither of which was enough to be considered a significant source of streamflow. However, GW from the upstream portion of the watershed does converge at this location. A closer examination of the GW and river exchange flow in the urban area suggests that the GW is flowing beneath the river, without interacting with the stream. The average hydraulic gradient between the SZ and
the river at this location is +0.32 m, which may not be enough to overcome the headloss across the grid cell and the river lining to generate significant GW discharge.

The GW discharge to the stream is dependent on the cross-section elevations. In the resistance term of the river-aquifer exchange flow computation (Equation 23, Equation 24, and Equation 25), \( w, ds, \) and \( da \) may be affected by the cross-section elevations. However, these parameters are determined by the simplified triangular cross-section that is used by MIKE SHE for exchange flow, not from the MIKE 11 cross-sections used for the surface water hydraulics. Thus, the thalweg elevation of the cross-section may be more important for the exchange flow computation than the overall shape of the MIKE 11 cross-section. The cross-section elevations in this model were sampled from the 5 m resolution DEM, which does not provide any in-stream detail or an accurate thalweg elevation. The in-channel cross-section, from left bank to right bank, is being simulated as a triangular cross-section with an arbitrary depth of 0.3 m. This could be having a significant impact on the GW discharge results. To test this, another simulation was set up to run current conditions and LID13-20 with modified cross-sections that dropped the thalweg to an assumed depth of 2.0 m. Lowering the thalweg of the cross-sections should reduce the head in the river, increase the gradient, and promote GW discharge to the stream. Therefore, two scenarios were run with the adjusted cross-sections under current and LID13-20 land uses: “Current-AdjXS” and “LID13-20-AdjXS”. These scenarios were only run for dry (1992) and wet (1996) conditions, but not the entire simulation period to save on time. Results from these scenarios with adjusted cross-sections are discussed in Section 6.4.
6.3. **STREAMFLOW AT END OF RIVER 4**

The overall streamflow at the junction of River 4 and River 1 was examined using the last Q-point in River 4, at chainage 1565 m (4:1565). The streamflow at this location was analyzed because the groundwater flow results indicated the potential for groundwater discharge to the
stream at this location. However, the results from this analysis indicated that it was unlikely to support a GDE. The majority of days in both dry and wet conditions had no flow (Figure 37). In fact, the median annual streamflow was zero for all water years from 1990 to 2010 for the pre-development, current, and LID13-20 land use scenarios suggesting that there was no baseflow at this location for any land use scenarios simulated, including pre-development conditions. This corroborates the conclusion from Chapter 6.2.1 that the GW discharge to the stream at this location was too small to generate streamflow. It was thought that the results with the adjusted cross-sections might have more substantial GW discharge to support baseflow, which is discussed in Chapter 0.

The pre-development, current, and LID13-20 land use scenarios had an average of 273, 241, and 245 days (75%, 66%, and 67% of the year), respectively with a mean daily discharge of zero (“no-flow days”). Table 19 summarizes the number of days with no flow at the junction of River 4 and River 1 for dry (1992) and wet (1996) conditions. In dry conditions, the current and LID13-20 scenarios reduced the number of no-flow days at this site by 16% and 14%, respectively. In wet conditions, these dropped to 8% and 9% for current and LID13-20 respectively. The decrease in the number of no-flow days was caused by an increase in the response to small precipitation events that generated runoff and streamflow in the current and LID13-20 scenarios, but not in the pre-development scenario. For example, 10/04/1992 had 9.1 mm of precipitation and mean daily streamflow of 0.000, 0.005, and 0.004 m$^3$·s$^{-1}$ for pre-development, current, and LID13-20 land uses, respectively. There was less of a reduction in no-flow days under wet conditions because there were more days with sufficient precipitation to generate streamflow in the pre-development scenario resulting in fewer no-flow days in wet conditions for all land uses. The lack of baseflow at this location was evident for both dry and
wet conditions, but was more severe in the dry conditions, which had a greater number of no-flow days (Table 19). The GW discharge at this location, which was on the order of $10^{-5}$ m$^3$s$^{-1}$, was not sufficient to generate streamflow.

Both the current and LID13-20 scenarios increased the peak daily storm flows over pre-development conditions (Table 20); however, the LID13-20 was successful in reducing the peak daily storm flows versus the current conditions (Figure 37). In dry conditions, the current land use increased the five largest peak storm flows by an average of 59% from 0.15 to 0.23 m$^3$s$^{-1}$, while the LID13-20 scenario increased the five largest peak storm flows by an average of 12%, to 0.16 m$^3$s$^{-1}$. In wet conditions, current land use increased the five largest peak storm flows by an average of 48%, from 0.61 to 0.90 m$^3$s$^{-1}$, while the LID13-20 scenario increased the flows by an average of 28%, to 0.78 m$^3$s$^{-1}$. The LID13-20 was more effective, by percentage, at reducing the peak storm flows in dry conditions, but this was because of the smaller values of peaks; the absolute reduction in peak flows was greater in wet conditions. The $T_{Q_{\text{mean}}}$ was not a meaningful metric to evaluate the regime changes at this location because the large number of no flow days prevented the redistribution of daily Q data.
Table 19: Number of no-flow days at the junction of River 4 and River 1.

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Pre-development</td>
<td>300</td>
<td>211</td>
</tr>
<tr>
<td>Current</td>
<td>251 (-16%)</td>
<td>194 (-8%)</td>
</tr>
<tr>
<td>LID13-20</td>
<td>258 (-14%)</td>
<td>192 (-9%)</td>
</tr>
</tbody>
</table>

Figure 37: Streamflow at junction of River 4 and River 1 for dry (1992, top) and wet (1996, bottom) conditions.
Table 20: Summary of peak storm flows and percent increase over pre-development land use at the junction of River 4 and River 1 for dry (1992) and wet (1996) water years.

<table>
<thead>
<tr>
<th>Date</th>
<th>48 hr Precipitation (mm)</th>
<th>1992 Peak Flows</th>
<th>Streamflow (m³/s⁻¹)</th>
<th>Pre-Development</th>
<th>Current</th>
<th>LID13-20</th>
</tr>
</thead>
<tbody>
<tr>
<td>1991-12-09</td>
<td>1.0</td>
<td>0.21</td>
<td>0.32</td>
<td>0.21 (54%)</td>
<td>0.21</td>
<td>(2%)</td>
</tr>
<tr>
<td>1992-04-17</td>
<td>22.0</td>
<td>0.07</td>
<td>0.14</td>
<td>0.10 (46%)</td>
<td>0.10</td>
<td>(46%)</td>
</tr>
<tr>
<td>1992-08-03</td>
<td>28.8</td>
<td>0.33</td>
<td>0.42</td>
<td>0.31 (26%)</td>
<td>0.31</td>
<td>(-7%)</td>
</tr>
<tr>
<td>1992-08-29</td>
<td>30.7</td>
<td>0.10</td>
<td>0.12</td>
<td>0.07 (46%)</td>
<td>0.07</td>
<td>(-37%)</td>
</tr>
<tr>
<td>1992-09-18</td>
<td>33.5</td>
<td>0.02</td>
<td>0.16</td>
<td>0.13 (644%)</td>
<td>0.13</td>
<td>(644%)</td>
</tr>
<tr>
<td>AVERAGE:</td>
<td>23.2</td>
<td>0.15</td>
<td>0.23</td>
<td>0.16 (12%)</td>
<td>0.16</td>
<td>(12%)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>50.7</td>
<td>0.20</td>
<td>0.39</td>
<td>1.37</td>
<td>0.35</td>
<td>0.74</td>
</tr>
<tr>
<td>47.8</td>
<td>0.91 (355%)</td>
<td>0.82 (111%)</td>
<td>1.48 (8%)</td>
<td>0.48 (35%)</td>
<td>0.82 (11%)</td>
</tr>
<tr>
<td>20.3</td>
<td>1.37</td>
<td>1.48 (8%)</td>
<td></td>
<td>0.48 (35%)</td>
<td></td>
</tr>
<tr>
<td>13.3</td>
<td>0.82 (11%)</td>
<td></td>
<td></td>
<td>0.34 (-4%)</td>
<td>0.72 (-1%)</td>
</tr>
<tr>
<td>64.7</td>
<td>0.74</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>AVERAGE:</td>
<td>34.3</td>
<td>0.61</td>
<td>0.90 (48%)</td>
<td>0.78 (28%)</td>
<td></td>
</tr>
</tbody>
</table>

6.4. **RESULTS FROM SIMULATIONS WITH ADJUSTED CROSS-SECTIONS**

The area of where Rivers 4 and 1 merge was identified in Section 6.2 as a possible location where groundwater discharge may be occurring. It was also expected that groundwater discharge at this location may be occurring since it is within the boundaries of the Lovers Creek Swamp. However, Sections 6.3 and 6.2.1 indicated that very little groundwater discharge was occurring at this location. These results may be impacted by the assumptions surrounding the generation of cross-sectional data, specifically the assumption of 0.3 m deep thalweg. The lack of GW discharge and baseflow was thought to be due to a small value or lack of hydraulic gradient between the SZ and the river caused by an inaccurate estimate of the thalweg of the river cross-sections. Therefore, a new MIKE SHE cross-sections input data file with an adjusted river depth of 2.0 m was created and two scenarios with adjusted cross-sections were simulated for current and LID13-20 land use grids. Dry (1992) and wet (1996) conditions were simulated
to examine the impact of the adjusted cross-sections on the GW discharge and streamflow at the junction of Rivers 4 and 1.

### 6.4.1. Groundwater Discharge

The adjusted cross-sections substantially increased the groundwater discharge to the river at the junction of River 4 and River 1 for both the current and LID13-20 land use scenarios (Figure 38). The mean groundwater discharge in water year 1992 for current land use was $8.1 \times 10^{-5}$ m$^3$ s$^{-1}$ and increased to $4.3 \times 10^{-4}$ m$^3$ s$^{-1}$, an increase of $3.5 \times 10^{-4}$ m$^3$ s$^{-1}$ (425%). For the LID13-20 scenario, the adjusted cross-sections increased the mean groundwater discharge from $8.2 \times 10^{-5}$ to $4.2 \times 10^{-4}$ m$^3$ s$^{-1}$, a difference of $3.4 \times 10^{-4}$ m$^3$ s$^{-1}$ (413%). This increase was caused by a drop in the water surface elevation in the river (Figure 39), which increased the hydraulic gradient between the saturated zone and the river and was caused by the drop in river thalweg. Despite this significant increase in GW discharge, it is still less than 0.001 m$^3$ s$^{-1}$, which was the precision of the daily discharge output, so it still may not generate sufficient baseflow at this location. There was still no discernible difference in GW discharge between the current and LID13-20 land use simulations (Figure 40). The GW discharge results at this location also indicated a SZ-river interaction that was more variable in time.

The GW discharge at the junction of Rivers 4 and 1 was not appreciably different between current and the LID13-20 land use, for either the initial cross-sections or the adjusted cross-sections (Figure 40). However, for the adjusted cross-sections, the GW discharge under the LID13-20 scenario is slightly lower than for current conditions (Figure 38). This was true for both dry (1992) and wet (1996) conditions, but was slightly more prevalent under wet conditions (-3%) than dry conditions (-2%). However, the difference between current and LID13-20 were
negligible. The reduction in GW discharge from current to LID13-20 was not observed using the initial cross-sections.

The adjusted cross-sections not only substantially increased the amount of GW discharge, but also increased the temporal variability of GW discharge at this location (Figure 38), suggesting that it became more responsive to precipitation inputs under the adjusted cross-sections. This was a result of the increased variability in the river water level (Figure 39) which caused greater variability in the hydraulic gradient between the SZ and the river. The increased variability was evident in both dry and wet conditions, but was more prevalent under wet conditions as the water level responded to the higher levels of precipitation inputs.
Figure 38: Groundwater discharge to river at River 4:1565 for dry (top) and wet conditions (bottom).
Figure 39: Water level at River 4:1565 for dry (1992) and wet conditions (bottom).
6.4.2. Streamflow at Junction of Rivers 4 and 1

When simulated with the initial cross-sections, there was no baseflow at the junction of Rivers 4 and 1, and most days had a simulated daily discharge of zero. But the adjusted cross-sections increased the GW discharge at this location, so there was potential for consistent baseflow under the adjusted cross-sections. However, despite the increase GW discharge being substantial, the total GW discharge was still on the order of $10^{-4}$ m$^3$·s$^{-1}$ and was not enough to
generate baseflow in the stream. The streamflow results from the current and LID13-20 land uses simulated with the adjusted cross-sections showed only slight differences from the results simulated with the initial cross-sections (see Section 6.4.2). As in the case of the initial cross-sections, there was no noticeable difference between current and LID13-20 scenarios (Figure 41). Peak storm flows were reduced. The five highest peak flows were reduced by an average of 32% in dry conditions and 14% in wet conditions (Table 21). The number of no-flow days was reduced from the simulations with initial cross-sections by 22% in dry conditions and 80% in wet conditions; however, the number of days with a simulated daily discharge less than 0.005 m$^3$·s$^{-1}$ (and therefore rounded to less than 0.00 m$^3$·s$^{-1}$) was unchanged for the adjusted cross-sections for dry and wet conditions (Table 22). Therefore the baseflow was not substantially increased by the simulations with adjusted cross-sections.
Figure 41: Streamflow at end of River 4 (4:1565) with adjusted cross-sections for dry (1992, top) and wet (1996, bottom) conditions.
### Table 21: Summary of peak storm flows simulated with adjusted cross-sections for dry and wet conditions.

<table>
<thead>
<tr>
<th>Date</th>
<th>Current</th>
<th>LID13-20</th>
<th>Current-AdjXS</th>
<th>LID13-20_AdjXS</th>
</tr>
</thead>
<tbody>
<tr>
<td>1991-12-09</td>
<td>0.32</td>
<td>0.21</td>
<td>0.32</td>
<td>0.21 (-35%)</td>
</tr>
<tr>
<td>1992-04-17</td>
<td>0.14</td>
<td>0.10</td>
<td>0.14</td>
<td>0.09 (-36%)</td>
</tr>
<tr>
<td>1992-08-03</td>
<td>0.42</td>
<td>0.31</td>
<td>0.44</td>
<td>0.32 (-27%)</td>
</tr>
<tr>
<td>1992-08-29</td>
<td>0.12</td>
<td>0.07</td>
<td>0.11</td>
<td>0.06 (-46%)</td>
</tr>
<tr>
<td>1992-09-18</td>
<td>0.16</td>
<td>0.13</td>
<td>0.17</td>
<td>0.13 (-24%)</td>
</tr>
<tr>
<td><strong>AVERAGE:</strong></td>
<td><strong>0.23</strong></td>
<td><strong>0.16</strong></td>
<td><strong>0.24</strong></td>
<td><strong>0.16 (-32%)</strong></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Date</th>
<th>Current</th>
<th>LID13-20</th>
<th>Current-AdjXS</th>
<th>LID13-20_AdjXS</th>
</tr>
</thead>
<tbody>
<tr>
<td>1995-10-06</td>
<td>0.91</td>
<td>0.78</td>
<td>1.16</td>
<td>1.01 (-13%)</td>
</tr>
<tr>
<td>1995-11-11</td>
<td>0.82</td>
<td>0.68</td>
<td>0.93</td>
<td>0.78 (-16%)</td>
</tr>
<tr>
<td>1996-01-19</td>
<td>1.48</td>
<td>1.38</td>
<td>1.61</td>
<td>1.47 (-9%)</td>
</tr>
<tr>
<td>1996-02-24</td>
<td>0.48</td>
<td>0.34</td>
<td>0.51</td>
<td>0.34 (-33%)</td>
</tr>
<tr>
<td>1996-09-16</td>
<td>0.82</td>
<td>0.72</td>
<td>0.90</td>
<td>0.81 (-11%)</td>
</tr>
<tr>
<td><strong>AVERAGE:</strong></td>
<td><strong>0.90</strong></td>
<td><strong>0.78</strong></td>
<td><strong>1.02</strong></td>
<td><strong>0.88 (-14%)</strong></td>
</tr>
</tbody>
</table>

Table 22: Number of days with daily discharge of 0.000 and less than or equal to 0.005 m$^3$·s$^{-1}$ at River 4:1565 using adjusted cross-sections.

<table>
<thead>
<tr>
<th>Land Use Scenario</th>
<th>No-Flow</th>
<th>Daily Discharge less than 0.005 m$^3$·s$^{-1}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Current</td>
<td>251</td>
<td>194</td>
</tr>
<tr>
<td>LID13-20</td>
<td>258</td>
<td>192</td>
</tr>
<tr>
<td>Current-AdjXS</td>
<td>197 (-22%)</td>
<td>44 (-77%)</td>
</tr>
<tr>
<td>LID13-20-AdjXS</td>
<td>204 (-21%)</td>
<td>35 (-82%)</td>
</tr>
</tbody>
</table>

**6.4.3. Streamflow at Mouth of Lovers Creek**

The impact of the adjusted cross-sections on the streamflow results was also evaluated at the mouth of Lovers Creek. Using the initial cross-sections, the LID13-20 scenario reduced peak flows versus current conditions by 12.9% and slightly increased the low flows by of 4.4% (Table 18). These changes created a less flashy flow regime and increased the $T_{Qmean}$ metric by an average of 11% (Table 17). But these differences between the LID13-20 and current flow...
regimes were not substantially different from each other; the 11% increase in $T_{Q_{\text{mean}}}$ was from an average of 0.20 to 0.21. The impact of the assumption of cross-section depth which limited GW discharge to the stream may have reduced the total impact that LID13-20 may have had on the flow regime at this location.

The adjusted cross-sections increased the baseflow at this location in dry conditions, but did not have an impact on the baseflow in wet conditions (Figure 42). This was likely because in wet conditions, the amount of baseflow was limited by the leakage coefficient, and not by a lack of hydraulic gradient, whereas in dry conditions, the smaller hydraulic gradient was a limiting factor to GW discharge.

The performance of the LID13-20 scenario over the current conditions scenario was not improved when simulated with the adjusted cross-sections. Table 23 summarizes the $Q_{\text{mean}}$ and $T_{Q_{\text{mean}}}$ results for both current and LID13-20 scenarios with initial and adjusted cross-sections in 1992 and 1996. In dry conditions, $T_{Q_{\text{mean}}}$ was increased by 4% with initial cross-sections, from 0.23 to 0.24, but only by 1% with the adjusted cross-sections, from 0.21 to 0.22. In wet conditions, $T_{Q_{\text{mean}}}$ was increased by 3% by LID13-20 over current conditions with initial cross-sections, from 0.23 to 0.24, and with adjusted cross-sections, LID13-20 land use reduced the $T_{Q_{\text{mean}}}$ by 2%, from 0.23 to 0.23.

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>$Q_{\text{mean}}$ (m$^3$·s$^{-1}$)</td>
<td>0.35</td>
<td>0.40</td>
</tr>
<tr>
<td>Days &gt; $Q_{\text{mean}}$</td>
<td>83</td>
<td>78</td>
</tr>
<tr>
<td>$T_{Q_{\text{mean}}}$</td>
<td>0.23</td>
<td>0.21</td>
</tr>
<tr>
<td></td>
<td>(+4%)</td>
<td>(+1%)</td>
</tr>
</tbody>
</table>
The results from the simulation with the adjusted cross-sections had increased low flows (less than $Q_{90}$) by 22% in dry conditions and 7% in wet conditions and increased high flows (greater than $Q_{10}$) by 16% and 9% in dry and wet conditions, respectively. This led to a decrease in the $T_{Q_{\text{mean}}}$ under the adjusted cross-sections. While peak storm flows increased, the rate of change of did not, so the recession time after peak storm flows to below $Q_{\text{mean}}$ was shorter. For example, 16/04/1992 had a storm of 21.8 mm of precipitation, which led to 4 days with $Q > Q_{\text{mean}}$ in initial cross-sections, but only 3 days with $Q > Q_{\text{mean}}$ for adjusted cross-sections (Table 24). This was because the recession of the peak storm flow occurred as quickly with the adjusted cross-sections as with the initial cross-sections, but because the peak flows and $Q_{\text{mean}}$ were higher, the flows became lower than $Q_{\text{mean}}$ more quickly. Further, a second, smaller rainfall event that occurred shortly after on 21/04/1992 created 2 days with $Q > Q_{\text{mean}}$ for initial cross-sections, but only 1 day for adjusted cross-sections. This was because the increase in $Q_{\text{mean}}$ in the adjusted cross-sections simulation prevented smaller precipitation events from creating streamflow greater than $Q_{\text{mean}}$. 
Figure 42: Hydrograph at mouth of Lovers Creek for current and LID13-20 land uses with initial and adjusted cross-sections for dry (1992, top) and wet (1996, bottom) conditions.
Table 24: Streamflow data for April storm event.

<table>
<thead>
<tr>
<th>Date</th>
<th>Precipitation (mm)</th>
<th>Daily Discharge (m$^3$·s$^{-1}$)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Current</td>
</tr>
<tr>
<td>1992-04-15</td>
<td>0.0</td>
<td>0.273</td>
</tr>
<tr>
<td>1992-04-16</td>
<td>21.8</td>
<td>0.370</td>
</tr>
<tr>
<td>1992-04-17</td>
<td>0.2</td>
<td>2.496</td>
</tr>
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</table>

Highlighted cells indicate $Q > Q_{\text{mean}}$.

6.5. **EXAMINATION OF GW DISCHARGE AT JUNCTION OF RIVER 4 AND RIVER 1**

The lack of change in baseflow at the junction of River 4 and River 1 when simulated with the adjusted cross-sections suggested that there was another limiting factor to GW discharge. MIKE SHE uses Equation 30, which is described in Section 4.3.6 and is reprinted below to compute exchange flow between the SZ and the river.

$$Q = C \cdot \Delta H = \left( \frac{1}{K \cdot d_a \cdot dx \cdot \frac{1}{2} \cdot w \cdot dx} \right) \cdot \Delta H$$

*Equation 30: Computation of exchange flow between SZ and river in MIKE SHE.*

Where $\Delta H$ is the hydraulic gradient between the SZ cell and the river, computed by $H_{SZ} - H_{RIV}$ [m]; $ds$ is the distance from the grid node to the middle of the simplified triangular cross-section [m]; $da$ is the vertical surface available for exchange flow [m]; $dx$ is the SZ grid cell size [m]; $w$ is the wetted perimeter of the simplified triangular cross-section [m]; $K$ is the horizontal saturated hydraulic conductivity of the grid cell [m·s$^{-1}$]; and $L_c$ is the leakage coefficient of the river bed liner [s$^{-1}$]. The parameter $C$ is called the conductance factor [m$^2$·s$^{-1}$] and can be thought
of as providing the opposite of resistance and head loss along the flow path of the SZ grid cell and the river bed lining.

The impact of modifying $\Delta H$, $K$, and $L_c$ was assessed through an analysis of Equation 30 and its parameters. Although $K$ and $L_c$ could both reasonably be assigned a value within a very large range spanning several orders of magnitude, the impact of each on the amount of exchange flow computed by Equation 30 is limited by the value of the other. They work in tandem to resist flow between the SZ and the river. Both $K$ and $L_c$ contribute to the conductance factor, C, and influence Q through it. As $K$ and $L_c$ increase, C and Q increase, but there are limits to how much C and Q, defined by Equation 30, can be affected by $K$ and $L_c$. Mathematically speaking, the limit of Q as $K$ independently approaches infinity is $L_c \cdot w \cdot dx \cdot \Delta H$ and the limit of Q as $L_c$ independently approaches infinity is $K \cdot da \cdot dx/ds \cdot \Delta H$. Practically speaking, as $K$ increases, the resistance to exchange flow and head loss across the SZ grid cell becomes negligible and the river liner is the only source of head loss from the SZ to the river. And as $L_c$ increases, the resistance to exchange flow and head loss across the river liner becomes negligible and the SZ grid cell is the only source of head loss from the SZ to the river.

In this study, however, it was assumed that the $K$ of the first layer of the SZ was not representative of the near surface conditions, since that layer was, on average, 70 m thick and $K$ typically has substantial spatial variability. Thus, this study used the ‘River Bed Only’ version of Equation 30, shown in Equation 31, which only considers head loss across the river lining, assuming that head loss is negligible in the SZ (the limit as $K$ approaches infinity).

$$Q = C \cdot \Delta H = (L_c \cdot w \cdot dx) \cdot \Delta H$$

Equation 31: Exchange flow between SZ and river using river bed only conductance

In this form $L_c$ and the $\Delta H$ are the controlling factors of GW discharge in this study. The wetted perimeter, $w$, would also have an impact, but it is a property of the geometry of the
simplified triangular cross-section used for exchange flow, the depth in the river, and the head in the SZ and cannot be controlled by the user. The adjusted cross-sections increased $\Delta H$ through a reduction in $H_{RIV}$, but $L_c$ remained at a constant $8.0 \times 10^{-7}$ s$^{-1}$, the value obtained during model calibration.

A brief examination of Equation 31 near the junction of Rivers 4 and 1 was completed to determine how much GW discharge could occur at this location given uncertainties in the $\Delta H$ and $L_c$ parameters. For 15/02/1996, under current land use, the amount of exchange flow was $1.7 \times 10^{-5}$ m$^3$ s$^{-1}$ and with the adjusted cross-sections, this was increased to $7.2 \times 10^{-4}$ m$^3$ s$^{-1}$.

Values of $dx$ and $L_c$ were known to be 50 m and $8.0 \times 10^{-7}$ s$^{-1}$ respectively. $H_{SZ}$ and $H_{RIV}$ at this location and time were determined from model output to be 241.6 m and 241.26 m respectively, under current land use with initial cross-sections, and 241.49 m and 239.67 m respectively, under current land use with adjusted cross-sections. Thus, $\Delta H$ values were 0.40 m and 1.82 m for current land use with initial and adjusted cross-sections. The value of $w$ could not be estimated since it is function of the geometry of the simplified triangular cross-section that MIKE SHE uses for exchange flow and it would also change with the adjusted cross-sections. Values of 10.7 and 9.9 m were found to compute GW discharge that matched the MIKE SHE output; however, since there was no way of estimating $w$ without known values of GW discharge, it was assumed that $w$ remained constant at 10.3 m, introducing some error into these estimates of GW discharge in this analysis. Thus, at this location and time, the GW discharge to the stream was computed as $Q(\Delta H, L_c) = \Delta H \times L_c \times 10.3$ m $\times$ 50 m, which is a linear equation with respect to both $\Delta H$ and $L_c$.

Since $Q$ was a linear function of $\Delta H$ and $L_c$, the derivative of this function represents the sensitivity of $Q$ to changes in each of these parameters. The derivative with respect to $\Delta H$,
\( \frac{\partial Q}{\partial \Delta H} \), was equal to 515 \( L_c \), and the derivative with respect to \( L_c \), \( \frac{\partial Q}{\partial L_c} \), was equal to 515 \( \Delta H \). Since \( \Delta H \) could only be computed to three decimal places (nearest mm), and \( L_c \) is typically on the order of \( 10^{-4} \) s\(^{-1}\) or less, \( |\Delta H| \) is much greater than \( L_c \), and \( \frac{\partial Q}{\partial L_c} \) is much greater than \( \frac{\partial Q}{\partial \Delta H} \). Therefore, the GW discharge computed at this location was more sensitive to absolute changes in \( L_c \). However, since absolute changes to \( \Delta H \) may be of a greater magnitude it also may produces substantial changes in \( Q \). Percentage changes in either \( L_c \) or \( \Delta H \) produce an identical percent change in the computed groundwater discharge, since the equation is a linear function with respect to each of these parameters. Therefore, both \( \Delta H \) and \( L_c \) have a significant impact on the amount of computed GW discharge.

Given the current value of \( L_c \) (\( 8 \times 10^{-7} \) s\(^{-1}\)) determined duration model calibration and reasonable limits on the possible range of values for \( \Delta H \) (0.0 to 5.0 m), the maximum possible groundwater discharge at this location was estimated to be 0.002 \( m^3 \cdot s^{-1} \), which was not considered sufficient quantity to be a source of streamflow. One of two conclusions may be drawn from this: 1. the results are accurate, \( L_c \) is an appropriate estimate, and there is, in fact, negligible GW discharge at this location; or 2. the results are inaccurate, \( L_c \) is not appropriate estimate, and there should be more GW discharge at this location. Given that this location is a focal point of groundwater flow (see Section 6.2) and this location is surrounded by the Lovers Creek Swamp, it is not unreasonable to think that there should be more GW discharge at this location in the model, though it is not a requirement to support a wetland area.

Overall, it was concluded that given the current value of \( L_c \), it was impossible to get sufficient GW discharge to generate baseflow at this location within reasonable values of \( \Delta H \). GW discharge at any specific location and time was sensitive to changes in both \( L_c \) and \( \Delta H \). Further, the \( K \) could not be used in the computation of GW discharge because it was considered
unlikely to be representative of hydraulic properties near the ground surface. These three conclusions point to an overall high degree of uncertainty in the amount of GW discharge. Therefore, in order to produce accurate estimates of GW discharge to the stream, the uncertainty in each of these parameters should be reduced as much as possible. This was not done in this research as ΔH contains significant uncertainty as a result of the cross-section elevations being sampled from the 5 m DEM; the K represented lumped hydraulic properties of several tens of meters of subsurface material; and the Lc was a calibration coefficient used only on the streamflow at the mouth of Lovers Creek.

6.6. POTENTIAL MODELLING IMPROVEMENTS

6.6.1. Improved Input Data and Modelling Technique

There are several ways in which this study, or any study involving complex hydrologic modelling, could be improved. Perhaps the top of the list is the quality of input data, especially in consideration with the objectives of the study. In the end, a model is only as reliable as the quality of the input data used.

The cross-section data used in this model was sampled from the 5 m DEM and was a significant source of uncertainty, especially in computing the exchange flow between the SZ and the river (refer to Chapter 0). The streamflow within the river network may be accurately represented without detailed cross-sectional data, but the arbitrary depth of the channel was shown to have a significant impact on the total amount of GW discharge. When examining the impact on GDEs, it is crucial to accurately represent the GW discharge. Therefore, it is crucial to know, at the least, an estimate of the thalweg elevation of each cross-section. This would allow for the river depth and \( H_{RIV} \) to be based on this elevation and the simulated depth of flow.
Ideally, an entire river cross-section would be known so that the bottom elevation and stage-discharge relationship could be known, not assumed from a triangular cross-sectional shape. However, this also requires a substantial increase in time, cost, and effort to acquire this level of data. The 5 m resolution DEM may provide accurate elevations for the floodplain of the cross-section, but it does not provide an accurate estimate of the channel elevation, which has a significant impact on $H_{RIV}$, $\Delta H$, and GW discharge to the stream.

A second area where better quality input would make a substantial improvement is the representation of the subsurface conductivity, specifically near the ground surface and adjacent to the river networks, as it has a substantial impact on the overall computed GW discharge. Currently, the first layer of the SZ has an average thickness of 70 m. This thick layer structure reduced computation time, but the lumped hydraulic properties, namely $K$, may not accurately represent the properties near the ground surface. Sahoo et al. (2006) found that streamflow results were not sensitive to estimates of $K$ in the upper portions of the mountainous watershed in Hawaii, because very little water infiltrated at this point. However, for watersheds that are not in mountainous regions, such as Lovers Creek, MIKE SHE results are likely sensitive to changes in the $K$, and it is often used as a calibration parameter (Thompson et al., 2004; Im et al., 2009).

When the focus is on GDEs, and specifically GDEs that require surface expression of GW, it is crucial to have accurate estimates of GW discharge. The importance of an accurate estimate of $K$ for GW discharge and streamflow estimation depends on the watershed and the amount of interaction the SZ has with the stream.

The subsurface also does not need to be represented down to bedrock. The total depth of all 3 layers of the SZ in this model was about 150 m, but the groundwater in the lower portions of that subsurface flows into and out of the watershed through the boundaries and does not
interact with the stream or wetlands. A smaller section of the subsurface could be considered, with more detail in the near surface and an aquitard layer at a more reasonable depth could be used as the bottom layer. In this way, the important part of the subsurface for consideration of GDEs could be modelled with sufficient detail and discretization, but the computation time can still be kept to reasonable limits.

A lack of both quantity and quality of calibration and validation data also had a negative impact on this modelling study. This MIKE SHE model was only calibrated to streamflow at the mouth of the watershed, which is not a sufficient data set to yield confidence in the performance of the model in various points within the watershed. Even if the calibration at the mouth of the river network was near perfect, the results in the upstream portion of the watershed would not be calibrated and could not be considered reliable. MIKE SHE simulates both the surface water and groundwater portions of the hydrologic cycle; as such, data from both of these areas should be used for model calibration. Continuous streamflow data at multiple locations along the river network should be used in conjunction with groundwater level data at multiple locations within the watershed for a more reliable integrated hydrologic model.

### 6.6.2. Improved Representation of Urbanization and LID within MIKE SHE

The way that LID was represented in MIKE SHE in this study was sufficient, but not ideal and could be improved. The lack of a pipe network was the most extreme difference between the model and reality and effects the results of the urbanized and LID simulations. There are two ways in which a pipe drainage network could have been simulated better in this model using MIKE SHE. The first option involves using the MIKE SHE SZ drainage options to define fast water movement through an artificial drainage network from MIKE SHE to the MIKE 11 river network. The second option would be to link MIKE SHE with MIKE URBAN and
MOUSE, which are two other hydrologic modelling software packages produced by DHI and specifically designed to model water movement in a city environment. MIKE URBAN package uses the MOUSE engine for pipe flow. Note that coupling MIKE SHE was originally developed for MOUSE, but MOUSE has since been incorporated into MIKE URBAN, so the terms MOUSE and MIKE URBAN may be interchangeable in this section.

The first method of using the SZ drainage to better define urbanization can be done through the SZ drainage options. Note that the SZ drainage options must also be used when coupling the SZ drainage to MIKE URBAN. There are four options for defining SZ drainage: 1. Drainage routed downhill based on adjacent drain levels; 2. Drainage routed based on grid codes; 3. Distributed drainage options; and 4. Drainage not routed, but removed from model.

The first option computes SZ drainage based on the drain levels in each cell, which is essentially the same as overland flow draining downhill, except that drain levels in the SZ do not necessarily follow the surface topography. The second option is the same as the first, except that separate areas of the watershed are sectioned off and SZ drainage is not allowed to occur between them. This was the option used in this study; however, the entire Lovers Creek subwatershed was given the same drain code, so the first option was effectively used. In contrast, in the larger, regional Barrie MIKE SHE model developed by AquaResource Inc., SZ drainage within the Lovers Creek subwatershed could not drain to adjacent watersheds, since they had different drain codes. The third option is the option that could be used for better representing urbanization and LID within MIKE SHE. This option allows for using either of the first two options or two extra options in different areas within the watershed. SZ cells with a distributed drainage code of 1 behave as if they were defined using SZ drainage option 1; SZ cells with a distributed drainage code of 2 behave as if they were defined using SZ drainage
option 2; SZ cells with a distributed drainage code of 3 are routed to a specific location along the MIKE 11 river network, simulating a fast response to precipitation inputs through an artificial drainage network. Note that the recipient location must be a defined H-point in MIKE 11 (i.e. it must be a location where MIKE 11 computes the river water level, which are the same points that have defined cross-sections). The fourth option can be used if MIKE SHE is being linked to MIKE URBAN as this defines a specific manhole to which the SZ drainage will occur in a similar way to the third option.

The third option could be used to define MIKE SHE grid cells that drain to specific locations along a MIKE 11 river link. If the stormwater drainage network of an urban area were known, then the recipient location on the MIKE 11 river network for each urban land use cell in that area could be defined. Further, the first or second SZ drainage options could be used on LID cells, so that SZ drainage from these cells do not follow the artificial stormwater drainage network of the urban area. However, this option may have a limited impact on the SZ drainage results because of how SZ drainage is computed in MIKE SHE. SZ drainage is not computed from one cell to the next, rather, the SZ drainage pathway for each cell is found based on adjacent drain levels and the SZ drainage flow is routed directly from the source cell to the recipient location, which may be a cell with a low drain level, a MIKE 11 river link, or a boundary. The assumption here is that the time step length is longer than the time it takes for the drainage flow to reach its destination. Thus, if SZ drainage is already being routed directly to a MIKE 11 river link, the use of routing it to a specific MIKE 11 H-point may not provide a substantial difference in the amount of SZ drain flow to that location.

Defining LID using this method could potentially be done by using artificially low drain levels in LID cells within urban areas so that SZ drainage flow is directed to these cells and not
further onto the MIKE 11 river network. This would represent the detention of stormwater and delay of drainage flow. However, it could create ponding conditions, in which case the water would be added back to the overland flow module of MIKE SHE. Nonetheless, a combination of the method used in this study, where LID cells are given a modified, reduced topography relative to adjacent urban cells, are given a relatively high depression storage to represent ponding at the surface and delay of overland flow, and a reduction in the SZ drain level may provide a more accurate representation of the urban and LID land uses within MIKE SHE.

The second, and likely the best, option to better define urbanization and LID within MIKE SHE would be to link MIKE SHE to MIKE URBAN. This would allow for the simulation of the effect of the urban stormwater drainage network and sewer systems directly. Coupling these two models can be done through creating two stand alone models, one MIKE SHE model and one MIKE URBAN model, and linking them through prescribed locations where they interact. The coupling between MIKE SHE and MIKE URBAN is done in a similar manner as coupling between the MIKE SHE SZ and the MIKE 11 river network, by exchange flow between the two packages, which is computed by Equation 32.

\[
Q = C \cdot (H_{SHE} - H_{MOUSE})^k
\]

*Equation 32: Coupling of MIKE SHE and MOUSE.*

Where \( Q \) is the exchange flow between MIKE SHE and MIKE URBAN; \( C \) is the exchange coefficient, similar to the conductance factor for MIKE SHE and MIKE 11 coupling; \( H_{SHE} \) is the head in the MIKE SHE grid cell; and \( H_{MOUSE} \) is the head in the MIKE URBAN pipe network; and \( k \) is a head difference exponent, usually 1, but sometimes 1.5 if a weir type formulation were to be used.

There are several different circumstances in which MIKE SHE and MIKE URBAN may interact. These use different definitions of the exchange coefficient, \( C \), and are described below.
**MIKE SHE SZ to MIKE URBAN Links**

\[ C = C_L \cdot R_H \cdot L \]

*Equation 33: Exchange coefficient for flow from MIKE SHE to MIKE URBAN links.*

This would simulate exchange flow between the SZ of MIKE SHE and the pipe network of MIKE URBAN, through aging infrastructure with leaky pipes or open channels, in which case it is similar to the MIKE 11 coupling. \( C_L \) is the leakage coefficient of the pipe, similar to the leakage coefficient of the river lining, \( L_c \), that was used for MIKE 11 coupling [s\(^{-1}\)]; \( R_H \) and \( L \) are the hydraulic radius and length of the pipe or drainage channel [m]. The leakage coefficient may be computed in one of two ways: one that includes just the leakage coefficient and another that incorporates the surrounding hydraulic conductivities of the SZ, analogous to ‘River Bed Only’ and ‘Aquifer + Bed’ conductance when MIKE SHE was coupled to MIKE 11.

**MIKE SHE Overland Flow to MIKE URBAN Links**

\[ C = C_L \cdot L \]

*Equation 34: Exchange coefficient for MIKE SHE overland flow to MIKE URBAN links*

This would simulate exchange flow between the Overland Flow module of MIKE SHE directly to MIKE URBAN channels. This type of flow is limited to open channels with a ‘CRS’ designation in MIKE URBAN (i.e. a natural channel with an ‘open’ cross-section). This type of coupled exchange flow can also take two forms depending on the \( k \) exponent, which can be equal to 1 or 1.5. If \( k \) is equal to 1 it is a drain for flow going from the open MIKE URBAN channel onto the MIKE SHE surface grid, and if \( k \) is equal to 1.5 then this is a weir formulation.
**MIKE SHE Overland Flow to MIKE URBAN Manholes**

\[ C = C_L \]

*Equation 35: Exchange coefficient from overland flow to manholes*

This form would be used to simulate MIKE SHE overland flow drainage into catch basins and a covered pipe network.

**MIKE SHE SZ Drainage Flow to MIKE URBAN Manholes**

If SZ drainage is used in MIKE SHE then it may also drain into MIKE URBAN via manholes. This option would utilize the fourth option of a distributed drainage option in the MIKE SHE drainage flow editor and would perform similarly to the way that MIKE SHE drainage flow would get routed to a specified H-point, except the H-point is replaced by a specified manhole in the MIKE URBAN model. The water would then be routed through the MIKE URBAN drainage network, which would outlet to a MIKE 11 river link and couple with MIKE SHE. This option provides a more realistic representation of SZ drainage through an artificial drainage network than simply routing the water directly from the source cell to the recipient MIKE 11 H-point or river link.

**MIKE SHE Paved Areas to MIKE URBAN Manholes**

If the paved area option was used in the land use definition of MIKE SHE then the flow generated by the paved areas can be discharged to a MIKE URBAN manhole, rather than directly to a river link. This is similar to the difference between SZ drainage being routed directly to the MIKE 11 river or through the MIKE URBAN drainage network to the MIKE 11 river link.
**MIKE URBAN Outlets to MIKE SHE**

MIKE URBAN may also outlet water to MIKE SHE. However, MIKE URBAN is not able to directly discharge water to MIKE SHE’s overland flow module. A work around developed by DHI suggests to create a dummy manhole at the end of the MIKE URBAN pipe, followed by a dummy pipe with a small diameter, forcing most of the water out of the manhole and making it available for the overland flow module of MIKE SHE.

All these available options for linking MIKE SHE to MIKE URBAN allow for urbanization to be better defined, but since they all link directly to the stormwater drainage network, there is not a clear way to define the reduced runoff from LID. Thus, a combination of defining LID land use cells, reducing the topography of these cells to direct overland runoff, a high depression storage to represent water storage and ponding, defining a low Manning’s M to represent reduced overland runoff, and defining a low SZ drain level to direct SZ drainage to these cells could be used. All in all, urbanization could be better represented, but there are still limitations to how well LID can be defined, even in a linked MIKE SHE and MIKE URBAN model.

Despite the ability to couple MIKE SHE and MIKE URBAN and the better representation of urban conditions it would provide, there remain some barriers to its use. First, a licence for both MIKE SHE and MIKE URBAN must be obtained. Each of these packages is its own software, designed for its own purpose, and users may not have regular need of both packages. The added cost of having a licence for both software packages may be too great to justify the benefits of creating a coupled model. Second, the added data requirements of coupling MIKE SHE and MIKE URBAN also increase costs of doing a coupled model. Unless a coupled model can produce cost-effective results, the added cost of the data collection, resources,
computational time may not be justified. A third, albeit less severe barrier, is that currently the use of SZ drainage to specific MIKE 11 H-point or linking MIKE SHE to MIKE URBAN requires the use of the “extra parameters” menu in MIKE SHE and the preparation of a specially formatted, tab-delimited text file to define the link locations and parameters. This is certainly a surmountable barrier, but creating the file that links the two models adds difficulty for the user, especially considering the ease and organization of other areas of a MIKE SHE model development process.

6.6.3. Reduced Model Domain and Targeted Output Area

A modelling study on GDEs may be improved by a reduction in the model domain and identification of a specific area for which output should be analysed. While this study looked specifically at the Lovers Creek subwatershed, which was a smaller part of the regional Barrie MIKE SHE model, the 60 km$^2$ model domain was still quite large for the purposes of examining alterations to GDE. The lack of a specific location where model output could be analysed and compared to the requirements of a target species limited the value of the analysis.

There are two major concerns with simulating a larger model domain: there is a discrepancy between the discretization of the hydrologic model and the scale at which ecological conditions may be relevant. The regional MIKE SHE Barrie model completed by AquaResource Inc. had a 200 × 200 m grid and MIKE 11 cross-sections that were spaced 500 m apart. Many ecological processes could be lost within grid cells at scales that size. By contrast, the Lovers Creek model developed in this research had a 50 × 50 m hydrologic grid resolution and approximately 200 – 300 m spacing of MIKE 11 cross-sections. By comparison, Thompson et al. (2004), who modelled a specific marsh in southeast England, used a 30 × 30 m grid on an 8.7 km$^2$ catchment. The smaller grid size reduced, but did not eliminate, the difference between
scales of hydrologic efficiency and ecological relevance. Many ecologic models use hydrologic output (e.g. water depth, streamflow, inundation duration) as model input. Therefore it is important to provide hydrologic output at scales as close to ecological relevance as possible, while still maintain efficiency and accuracy of hydrologic conditions.

A target area for model output may have helped to focus the modelling effort. The target area would be a specific GDE of potential concern. Examples include a specific reach of the river network that has been identified as, or is at least suspected to be, an area of importance for supporting GDEs in the stream through GW discharge and hyporheic exchange; or a wetland area that is known to contain species dependent on groundwater discharge to provide a period of inundation; or a wetland that is known to rely on a shallow groundwater table for a source of water for its plant communities. A targeted output area would provide two things: 1. A specific area or species whose hydrologic requirements are known or could reasonably be determined or estimated; and 2. A specific location where MIKE SHE output could be analyzed. Knowing the hydrologic regime required for sustaining a particular GDE provides a target for comparison of the simulated MIKE SHE output and allows evaluation of potential ecological responses.

Further, MIKE SHE is a powerful hydrologic model that generates vast amounts of output. Sifting through this output for information that is relevant to a GDE (e.g. streamflow, GW discharge, groundwater flow, depth to groundwater table) for many locations in a watershed is a daunting task. Having a targeted area, with known hydrologic data requirements where analysis of model output could be focused could reduce the time and effort required for analysis of model output.

Using this research as an example, if it were known or pre-determined that the location of the junction of River 4 and River 1 were a potential GDE and if the hydrologic requirements of
this GDE were determined based on species composition, then a MIKE SHE model could have been developed to specifically examine the impact of urbanization and LID on this GDE. This model could potentially be developed around the location of River 4, rather than the entire Lovers Creek subwatershed, again using the regional Barrie model to set boundary conditions. In this way the model domain could have been reduced to boundaries around River 4, approximately 15 km$^2$. To keep the same number of grid cells as was in the Lovers Creek model and maintain computation time, the hydrologic grid resolution could have been reduced to 25 × 25 m, approaching a scale of greater ecological relevance. Furthermore, the analysis of model output could have been focused on that specific location. The flow regime at multiple Q- and H-points within this location could have been analyzed, as opposed to just at the end of River 4 and the Tollendale Gauge. If the target output area were not the river, but the wetland surrounding it, then the depth to the water table may have been the variable of interest, not streamflow or groundwater discharge. The lack of focus on a specific area and variable for model output limited the value of analysis that was completed.

6.6.4. Improved Connection between Hydrologic and Ecological Data

Finally, this research would benefit from an improved understanding of the connection between hydrologic, biologic, and ecological data. This would require the involvement and cooperation of professionals from several disciplines and their coordinated efforts. This research used the MIKE SHE hydrologic model to simulate the hydrology and then attempted to link the hydrology to the ecological conditions. This was done through the $T_{Q_{\text{mean}}}$ statistic, which was been related to B-IBI of streams in previous scientific literature. However, it is possible that the B-IBI is not the most appropriate index of ecological condition to be used for GDEs. Furthermore, specific information about the biologic species in a GDE would provide
information pertaining to their hydrologic requirements and/or allow for the creation of a hydrologic indicator specific to that GDE or that type of GDE.

For hydrologic modelling of GDEs to be useful, a link must be made between the data that is important to a GDE and data that is generated by MIKE SHE. The hydrologic requirements or tolerance limits of a GDE must be put into parameters that can be generated by MIKE SHE (e.g. depth to groundwater table instead of soil moisture content). The difficulty in this step is that individual GDEs may have vastly different hydrologic requirements, degrees of groundwater dependency, and sensitivity to changes in groundwater inputs. This link can be made through hydrologic metrics that are related to ecological condition (e.g. $T_{Q_{\text{mean}}}$), but uncertainties remain on the transferability and validity of such relationships in different climates and different types of GDEs. The development of target hydrologic regimes and tolerance limits (whether it be for streamflow, water depth, or depth to groundwater table) for species within a GDE would be useful for comparison of hydrologic simulation results.
7. CONCLUSIONS AND RECOMMENDATIONS

This study used MIKE SHE to evaluate the effect of land use changes on the hydrologic regime of GDEs. The results indicated potential for LID to restore ecological conditions to a stream, yet the model contained uncertainty that must be considered when interpreting the results. Improvements to the modelling effort were identified that could reduce this uncertainty in future attempts at studies of this nature.

Land use changes impacted the overall water balance of Lovers Creek by reducing evapotranspiration, which left more water available to generate streamflow. This caused a significant (p < 0.01) increase in the amount of baseflow in Lovers Creek for both current and LID scenarios simulated. Coincident with the increase in baseflow was an increase in the peak storm flows and degree of flashiness in the streamflow regime. Differences in types of LID (retention or infiltration) were not evident, but this was affected by the extent of urbanization and LID implementation in the model domain.

This research suggests that LID may be an effective means of protecting GDEs in urban streams where an altered flow regime is a main stressor of degradation and indicates that MIKE SHE can be useful tool for the analysis of long term impacts of land use changes on GDEs. The 20% conversion scenario increased low flows (daily discharge less than \( Q_{90} \)) by 4.4% and decreased high flows (daily discharge greater than \( Q_{10} \)) by 12.9%. These changes led to an increase in the \( T_{Q_{\text{mean}}} \) metric, from a current land use value of 0.20 to the 20% LID13 scenario of 0.21, which restored 11% of the reduction in \( T_{Q_{\text{mean}}} \) experienced by the change from pre-development (0.26) to current conditions. This is likely to lead to an increase in the Benthic Index of Biologic Integrity (B-IBI) and protect the ecological condition of GDEs in the Lovers
Creek subwatershed. While these changes are relatively minor, they should be taken in context of the land use over the entire watershed. The 20% conversion of high density urban to LID represented only a 5.3% conversion to LID for the whole watershed. Other land uses (e.g. agricultural) which were not changed between scenarios influenced the results. It is expected that the impact of LID would be amplified in a more urbanized watershed or on a more local scale with HDU as the predominant land use.

These results must be interpreted recognizing the uncertainty and limitations of the study, in both the hydrologic responses and relationship to ecological data. The model calibration indicated only a ‘fair’ performance of the model, with daily and monthly Nash-Sutcliffe efficiency measures of 0.36 and 0.64, respectively. The specific relationship between $T_{Q\text{mean}}$ and B-IBI determined by Booth et al. (2004) was not considered transferable to the Lovers Creek subwatershed. As such, the amount of increase in the B-IBI expected from the increase in $T_{Q\text{mean}}$ was unknown and it may or may not provide significant biological benefits.

Groundwater discharge in the Lovers Creek river network was minimal for most locations in the river network and at all times throughout the simulation period. These results were affected by assumptions regarding cross-sections and parameters controlling the exchange flow between the SZ and the river. A wetland area near the junction of Rivers 4 and 1 was thought to have potential to support a GDE because it represented a focal point of groundwater flow and was within a wetland area, but the model suggested that groundwater discharge at this location was minimal.

The modelling process revealed many areas where improvement could be made for better hydrologic modelling of GDEs. Improvements in hydrologic modelling of GDEs could be made through improvements to the integrated models used or in the data used for model input,
analysis, and comparison. Current integrated hydrologic models, such as MIKE SHE, are useful for hydrologic modelling of GDEs and do not require major improvements, although minor improvements are recommended. A knowledge gap that exists between the GDEs, their species of flora and fauna, and their hydrologic requirements is a substantial hurdle to interpreting output of hydrologic models for ecological response modelling.

The current version of MIKE SHE is capable of generating output that is sufficient for hydrologic analysis of GDEs and does not require major improvements for this type of study. However, if impacts of urbanization and LID are considered pertinent to a GDE of concern, then minor improvements in the representation of these land uses would be beneficial. These minor improvements are related to how an artificial stormwater drainage network, such as would be found in urban areas, can be represented in MIKE SHE and to how LID cells could be defined. Urbanization can be well represented if MIKE SHE is coupled with MIKE URBAN, but this is an expensive and time consuming option. The option of routing SZ drainage in urban land use cells to specific river cross-sections based on known pipe networks and outlets is a recommended alternative as this allows for representation of the artificial drainage network without the need of licences for both MIKE SHE and MIKE URBAN. Representing LID remains difficult as various model parameters must be used to mimic the effect that LID has on the hydrologic cycle. A recommended way is to use the method used in this study combined with a drop in the SZ drainage layer to break the link between SZ drainage flow and a receiving stream and creating a low point that allows for water storage in the soil zone.

Improvements to the hydrologic modelling of GDEs would be best made through improvements in the data used for model input and comparison. Currently there is a knowledge gap in terms of the hydrologic requirements of a GDE. What is the degree of dependency of the
GDE to groundwater? What is the sensitivity of a GDE to changes in groundwater inputs? If a GDE expresses a threshold sensitivity relationship to groundwater inputs, where does it currently sit on the graph? Is it near the threshold or in the stable range? What is the frequency and duration of groundwater inputs that is required to sustain a GDE (e.g. 1 month every year, 6 months every 10 years)? Without knowledge of the hydrologic regime required to sustain and protect a GDE, the hydrologic modelling effort is of limited use.

Specific suggestions for hydrologic modelling of GDEs are made based on the difficulties experienced in this research. Recommended information to have for a hydrologic modelling effort of this kind include:

- A specific GDE location that is under threat or of concern in order to focus the modelling effort and analysis of model output.
- A target hydrologic regime determined from hydrologic requirements of species within the GDE.
- Detailed input data, at least in the nearby area surrounding the GDE of interest
  - Cross-section data (if a river), especially the river thalweg and channels within a wetland area.
  - Accurate estimates of the hydraulic conductivity in the saturated zone, particularly the near surface conductivity in the area of the GDE of interest.
- Reduced size of model domain and grids to maximize hydrologic efficiency and effectiveness and to minimize the gap between scales of hydrologic and ecological sensitivity. This may require the preparation of two models, one large to cover the regional groundwater flow patterns, and a subsequent smaller one to focus on
the GDE. Boundary conditions for the smaller model may be taken from output of the larger model.

This study represented a first attempt to use an integrated model, MIKE SHE, for the purpose of examining hydrologic changes to GDEs. The results should be considered preliminary, but the model shows potential for use with future refinements in input data and modelling procedure and LID showed potential to restore ecological condition to GDEs in urban watersheds.
8. WORKS CITED


