Impacts of Aggregate Quarry Dewatering on Groundwater Management in Ontario

by

Brian Henderson

B. Eng., Carleton University

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Abstract

Any individual that removes more than 50,000 litres of water per day from surface or groundwater sources in Ontario must hold a Permit to Take Water (PTTW) issued by the Ministry of the Environment (MOE). As a result, aggregate quarries operating below the water table often require a PTTW in order to dewater the pit. The government of Ontario has recently prioritized the protection of drinking water through source water protection legislation managed by the Conservation Authorities (CAs) on a watershed scale. PTTW applications are assessed on an individual basis by the MOE, not by the CA responsible for water use planning. This research proposes a framework for bridging the gap between the CAs and MOE for assessing PTTW applications and their cumulative effects using numerical modelling. A case study area with five quarries was numerically modelled using Visual MODFLOW to illustrate some of the questions or concerns that arise when assessing the cumulative impacts of PTTWs.

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Chapter 1: Introduction

1.1 Introduction

Canadians are becoming more and more interested in water management as problems with contaminated water sources develop throughout the country. Perhaps the most serious case occurred at Walkerton Ontario in May 2000 where 7 people died and thousands of others became very sick due to the contamination of the town's drinking water system with *E. coli* bacteria. The protection of groundwater resources is of significant importance to many Canadians since 10 million people depend on these resources for their drinking water (Rivera, 2004).

In response to these concerns, the government of Ontario has made source water protection a priority. One of the first steps was drafting new legislation including the *Drinking Water Source Protection Act.* This act, when it passes, will put water resource planning on the shoulders of the Conservation Authorities (CAs) of the province. Each watershed, either on their own or combined with others, will have to establish a plan to protect its surface and groundwater drinking water resources. This plan will include a water budget for every source protection area.

As well as developing new legislation, the government of Ontario also changed existing legislation under the *Ontario Water Resources Act.* Of particular interest to this research were the changes to the Permit to Take Water (PTTW) program. Anyone in the province of Ontario that uses more than 50,000 litres of water a day, with a few exceptions, must hold a PTTW from the Ontario Ministry of the Environment (MOE). The application to obtain a permit is assessed by the MOE. These permits are generally considered on an individual basis with no consideration of potential cumulative impacts on the resources surrounding these water takings.

1.2 Thesis Objective and Scope

The objectives for the research were to establish a framework for assessing the cumulative impacts of PTTWs. This was to be done by first of all examining groundwater resource management in Ontario, Canada, and other countries to assess the state of groundwater management throughout the world. The PTTW process for Ontario was also investigated to better understand how applications were being assessed.

During the time of writing, the government of Ontario put forward new guidelines for assessing PTTW applications, and draft legislation for source water protection. As a result of this, the old and new approaches for assessing PTTW were both investigated and presented. The new Source Water Protection draft legislation was also investigated so that it could be included in the research.

The most important objective of the research was the formulation of a framework for assessing PTTW applications. Based on the apparent discontinuity between the government agency responsible for assessing the PTTW applications (MOE) and the agency responsible for water management (CAs) a new framework was proposed to establish how the assessment of PTTWs could be integrated with watershed management in Ontario. Included in this proposed framework is the assessment of the cumulative impacts of PTTWs on a more regular and structured basis.

The use of numerical modelling to assess the cumulative impacts on aquifers, as well as to manage water resources in general, is thought to be an important part of the solution. It is evident that numerical modelling requires a lot of data and resources. Because of this, the suggested framework also had to evaluate what would be required from a data management point of view. The proposed framework indicates which agency should be responsible for collecting and maintaining the data, and the agencies entitled to use the collected data. The agency responsible for the development of the model also had to be identified to ensure that there was consistency across the different watersheds of Ontario in terms of data management.

The final objective of the research was to evaluate a case study for an area where there were many PTTWs in fairly close proximity. West of Ottawa there is an area in which there are several small rural subdivisions, two golf courses, and recently, a large number of requests for PTTWs in order to remove the local aggregate. Local residents have raised concerns related to the quantity and quality of their drinking water resources if the aggregate quarries dewater as permitted. The water quality in the area may decrease with depth and as such is of concern.

The case study was used to illustrate the questions and concerns that arise when assessing the cumulative impacts of aggregate quarry operations on local groundwater resources. It was done using a numerical model based on a conceptual model built on the local geology and governing principles of groundwater flow and contaminant transport. Sources of information for the case study included existing modelling studies and reports as well as the PTTWs themselves and their applications. Although the case study was useful in meeting its primary objective - to illustrate the questions and concerns that may arise when assessing cumulative impacts - further refinement of the model to include local heterogeneities would improve model predictions at the local scale.

This thesis addresses the objectives through the different chapters. Chapter 2 addresses water resource management around the world, within Canada, and specifically what is happening in the province of Ontario. Chapter 3 examines both the current and former PTTW application and review process.

Chapter 4 presents the proposed framework for assessing PTTW applications. The framework looks at what agency should be assessing the applications as well as when and how cumulative effects should be considered. Specific issues that might be encountered during the numerical modelling process and at aspects unique to aggregate quarries are also examined.

Chapter 5 gives a brief introduction into some of the boundary conditions that are used in Visual MODFLOW, the software used to perform the numerical modelling. The chapter also discusses how to best simulate an aggregate quarry within the numerical model.

The case study is presented in Chapter 6. It includes the geology of the area, the conceptual model used to create the numerical model and describes how the regional numerical model was developed and calibrated. Smaller local scale numerical models were also used to simulate pump tests that had been conducted during hydrological studies of the area. Finally, some of the results of different simulations to examine the cumulative effects of quarry dewatering are presented and discussed.

Chapter 7 summarizes the research and presents conclusions and recommendations for further study.

Chapter 2: Literature Review

2.1 Water Resource Management

Water resource management has become very important for governments around the world. This is especially true in arid countries with limited water resources, where steps are being taken to ensure that water will be available for the present generation as well as for future uses. The main drive by governments throughout the world is to establish a plan for water management that is supported by both legislation and by society.

It is not surprising that areas more affected by drought conditions are much more active in water management than countries with plentiful resources such as Canada. Recently more water-rich areas are concerning themselves with water management because they realize that the quantity of available water is not endless, and that the quality of water also has to be protected.

2.1.1 Water Management Planning

Recent thinking in water management has indicated that governments need to supply a water resource plan that includes goals for the quantity and quality of water as well as the protection of the environment including aquatic life and wetlands (Miloradov *et al.*, 1995; Wilson and Droste, 2000). In doing this they must also consider future development, land management, municipal needs, and economics (Wilson and Droste, 2000). It has been realized that the hydrologic cycle can be very complicated and unique in each area studied. For this reason, prepared solutions cannot be fit to a specific problem, and the problem must be well defined in order to obtain an appropriate solution (Wilson and Droste, 2000).

The key to effective decision making is data. The more information that planners have access to, the better the solution they will come up with. This information has to come from a number of sources including special interest groups, water users, community members and technical studies (Wilson and Droste, 2000). The data requirements for some planning decisions can be enormous, but it must be presented in a way that can be easily understood by those making the decisions (Kendy, 2003; Wilson and Droste, 2000). It is also important to store the data in a location that is accessible to all concerned so that studies are not duplicated, and so that there is easy access to any required data.

One tool that is being used more frequently in the planning process is numerical modelling. It is an effective tool for helping hydrologists and planners understand the dynamic nature of an aquifer (Bredehoeft, 2002; Sophocleous, 2000). A model can be developed for a site and the outcomes of potential planning alternatives can be examined in order to come up with the best solution. The model created cannot be so simple that the results are inexact, nor can it be so complex that it takes years to construct, is very expensive and not flexible (Wilson and Droste, 2000). Boundary conditions must be appropriate and reflect what is happening in the area. Since there can be large uncertainties in the inputs for aquifer modelling, an element of risk should be incorporated into the results (DuMars and Minier, 2004; Sophocleous, 2000).

Throughout the literature, groundwater planners have been using the principle of safe yield for planning decisions. The concept of safe yield is that the amount of water that can be removed from an area without lowering the groundwater levels should be equivalent to the recharge of the system (DuMars and Minier, 2004). It has been shown that safe yield does not work to maintain groundwater resources because it does not take into account evapotranspiration, or the discharge from groundwater to streams (Bredehoeft, 2002; Bredehoeft, 1997; Kendy, 2003; Sakiyan and Yazicigil, 2004; Sophocleous, 2000; Sophocleous, 1997). One can plan using the concept of safe yield, but one will find that streams and wetlands may dry up. A different planning strategy is obviously required.

Planners should be using the concept of sustainable yield, which is a rate of withdrawal high enough to supply current needs while considering the hydrologic cycle and the needs of future generations (Sakiyan and Yazicigil, 2004). This is a difficult concept to quantify because of the complexity of interconnected systems.

Sustainable yield can be defined as follows. In any given basin before development, at steady state, the recharge to groundwater is equal to the discharge from groundwater. As pumping begins in the basin, all of the water that is pumped comes from storage (Sophocleous, 1997). As time passes, a new equilibrium will eventually exist with an increased recharge, a decreased discharge, or a combination of the two. This change in recharge or discharge is called capture (Bredehoeft, 1997; Sophocleous, 1997). The timing of the switch of water coming from storage to water coming from a change in recharge and discharge is very important for sustainable yield planning (Sophocleous, 1997). Systems can take a long time to reach a new equilibrium and it depends on physical parameters such as the diffusivity of the aquifer (defined as transmissivity divided by storativity) or the distance of the well from a source of recharge (Bredehoeft, 2002; Sophocleous, 2000). In most cases, the recharge to the basin will remain constant so the capture will be equal to the change in discharge from the basin (Bredehoeft, 1997). The capture is what planners need to focus on and manage (Bredehoeft, 1997).

Kendy (2003) has taken the concept one step further by adding that in some cases the pumping rate, like the original recharge, is irrelevant to sustainable aquifer use. In her study on the North China Plain it was found that it was the rate of evapotranspiration from plants and soils that controlled the water leaving the system. Her argument was that regardless of what was pumped, the plants on the surface only used a fixed amount of the water. The remaining water, that does not evaporate, is returned to the aquifer through infiltration. In order to increase the groundwater levels in the North China Plain, the area of irrigated land has to be reduced to reduce evapotranspiration. This example shows that each situation is different and has to be examined closely to determine where changes might be made to render the situation sustainable.

In order to implement the sustainable yield concept, a number of plans and policies should be used, including controlling new development, regulating existing development, metering water use, using water conservation and efficient irrigation, the implementation of water tariffs, artificial recharge, powerful legislation, and public education and involvement (Kendy, 2003; Sakiyan and Yazicigil, 2004). One has to bear in mind that the management options have to be flexible to meet the needs of changing environmental conditions (Sophocleous, 2000).

In planning for water management, the decisions that must be made can be very difficult. By pumping from aquifers, the natural system will be changed. Planners have to determine how much change is too much. To do this they depend on a team, with a wide variety of disciplines that can obtain and present the best possible data (Sophocleous, 2000; Wilson and Droste, 2000). Managing the groundwater resources in a basin requires an understanding of the interactions of the entire system and the transient nature of the system (Kendy, 2003; Sophocleous, 1997). Both surface waters and groundwater should be managed together because of the way that they are linked so strongly together (Sakiyan and Yazicigil, 2004; Sophocleous, 2000; Sophocleous, 1997).

The sustainable yield concept is an important tool because it is based on a mass balance that can incorporate multiple users of a given volume of water (Miloradov *et al.*, 1995; Sophocleous, 2000). One should realize that planning is an iterative and dynamic process that involves constant monitoring, analysis, prioritization and revision to work effectively (Sophocleous, 2000).

2.1.2 Water Management around the World.

Countries around the world are attempting to determine exactly who should take the responsibility for water management within their borders and at what scale. In countries including France, Kenya, Australia and the United States, surface water catchment areas are used to define the physical borders of a management area (Ivey *et al.*, 2002). More and more, water resource planners for the watersheds are managing both surface and groundwaters because they are linked through the hydrologic cycle (Ivey *et al.*, 2002; Sakiyan and Yazicigil, 2004; Sophocleous, 2000; Sophocleous, 1997). The use of watersheds as an area for management may involve planning across municipal, provincial and in some cases national borders (Ivey *et al.*, 2002). This allows a more thorough understanding of the resource.

In Guanajuato State, Mexico, the legal situation made it very hard for the government to stop individuals who were abusing their water allocations or were constructing wells without permits (Sandoval, 2004). This was due to the fact that water was administered by the federal government, leaving the State no formal method to intervene (Sandoval, 2004). The lack of control has led to a decrease in the water table of 2 to 3 meters annually, and a decrease in the quality of the water (Sandoval, 2004). Realizing that there was a serious problem, the State government knew that they had to change their groundwater management strategy to one that encouraged more efficient use of the water that was pumped out but would not affect the current wealth enjoyed by the farmers. The solution involved looking at three elements and their interactions. The technical program was used to assess the aquifers and model them numerically. The social program involved the users of the resource through an education program, the formation of several groundwater users associations who could get funding from the government for operational expenses, and by opening permanent centres where individuals could express their views on water use and management. Each of the local areas had the ability to establish their own priorities to reach the common goals of the government (Sandoval, 2004). The final element of the program was institutional where the government was able to enforce the decisions made through legislation (Sandoval, 2004).

The program in Mexico was successful because it achieved social agreements by constantly communicating with the water users and helping them find and use the best available technology. In this way, a critical mass was started where the users directly saw the benefits of their conservation (Sandoval, 2004). Other important issues learned in Mexico included the need for communication between all levels of government to ensure that all goals and proposals were reasonable and in line with each other. The program had to be financially sustainable, possibly through the collection of water rights fees. The study showed that groundwater management is a very complex issue and the integration of several different initiatives, from technical studies to social education, must be included (Sandoval, 2004).

A study in Jordan also found that the participatory approach to groundwater management was very successful (Chebaane *et al.*, 2004). Similar to the situation in Mexico, the government of Jordan was unable to control the amount of wells that were being drilled or ensuring the abstraction limits imposed were being respected. To manage the groundwater more effectively, the approach taken was first to assess the aquifers, consult with the users, and then determine what options were best and how they should be implemented (Chebaane *et al.*, 2004). The researchers found that the initial assessment was quite difficult because the individual water users did not want to answer their questions. Once the objectives of the program were understood through education and outreach, people were much more willing to participate (Chebaane *et al.*, 2004). The farmers wanted to be a part of a collaborative water management process because they were the ones who were the most affected if they ran out of water. After discussions with the farmers, it was found that they wanted to be better informed about conservation methods, and would be willing to slow down their groundwater use and explore alternatives in order to protect the resource (Chebaane *et al.*, 2004). They supported a ban on drilling new wells, the sale of illegal water, and thought that the socio-economic impacts of the management of groundwater be considered (Chebaane *et al.*, 2004). As a result of the discussions with the farmers, a number of management options were explored and analyzed by all stakeholders. It was found that a better institutional framework would have to be developed with the formation of a consultive committee representing all stakeholders involved with a private sector representative as the lead. Other initiatives required were the movement of farmers towards crops with higher economic values and lower water needs, a source of funding for the program, revision of current laws to cover any gaps that were discovered, an education campaign, and a monitoring program for groundwater levels (Chebaane *et al.*, 2004). This program was also very successful because of the participation of the water users in developing the strategy for groundwater management.

China has also developed a groundwater management strategy in its High Plains region. The management of groundwater in China has evolved from a series of fragmented plans, to an "institutionally integrated and decentralized" plan (Foster *et al.*, 2004). All levels of government are involved in the plans, but the day to day operations are carried out at the local (county) level. The higher levels of government do not participate unless there are trans-boundary issues involved. The bottom up approach allows the users to interact with the responsible agency (Foster *et al.*, 2004). There have been problems found with this approach. Each government department is only concerned with meeting their own targets regardless of the impacts on groundwater. As well, rural and municipal departments do not communicate effectively about groundwater management even though they share the resource. Another serious problem in China was that there was no link between the rates of abstraction that were allowed by a permit and actual groundwater estimates. The permits were handled manually so there was no central database of all permits. This has a major impact for water use planning as well as having access to the permits for legal purposes (Foster *et al.*, 2004).

To address these problems, the provincial government had to become more involved to coordinate groundwater management in different counties, and help by sharing the experiences of groundwater management of the different counties throughout the province (Foster *et al.*, 2004). The management strategy was to reduce the amount of water being used through engineering methods (best practices), management measures (abstraction permits) and agronomic measures (high value/low water use crops) (Foster *et al.*, 2004). Other parts of the management plan included education of water users, improved laws, a monitoring program and improved links between the permitting procedure and water saving measures (Foster *et al.*, 2004). Of course this method depended on user participation so the water management areas needed boards that would listen to the stakeholders to develop a strategy to obtain their targets.

From these examples there are a number of factors that are essential in developing groundwater management plans that will be effective. The resource must be understood as well as possible through technical studies and numerical modelling. The water users who are affected by the plan must also be well informed and support the plan. There must be avenues established so that the opinions of any water user can be heard by the planners. Finally, there must be a legislative framework in place so that the government can enforce the water management plans.

2.1.3 Water Management in the United States

Water management in the United States (US) has been active for a long time especially in the western states where water resources can be quite scarce. Groundwater accounts for approximately one third of the water supply in the western US (DuMars and Minier, 2004). Historically there have been several methods for water management that were usually controlled by the state in the form of water rights. Water in the western US was allocated based on prior appropriation, where the person who first established a right to the water was the last to be affected in times of drought. One must actually use the water for the right to exist, if the water is not being used, the right returns to the state so that it can be assigned to another (DuMars and Minier, 2004).

Groundwater management in the western US has also seen many forms but was not as regulated as surface waters. Different states have regulated groundwater in different ways including (DuMars and Minier, 2004):

- The rule of 'capture' – an individual can divert groundwater until there is none remaining under their own land, as long as no other property owners are affected.
- The rule of 'reasonable use' – an individual can divert as much groundwater as they need for use on the overlying land.
- The correlative rights doctrine – an individual has the right to a share of the water in the aquifer proportional to the amount of land they own above the aquifer.
- The law of prior appropriation – an individual can drill a well wherever they want as long as it does not interfere with an existing well that is being put to good use.

As one can imagine, each of these methods has flaws that would make the management and sustainability of groundwater in a watershed very difficult.

One example of water management in the US is in Arizona, where the state has separate laws to handle surface and ground waters. This two layer system has led to conflicts where groundwater pumping interferes with surface water rights (DuMars and Minier, 2004; Jacobs and Holway, 2004). Similar to groundwater management strategies in other countries, Arizona has found that there is a need for user participation to make their plan work (Jacobs and Holway, 2004). The government is trying to control the drawdowns of the aquifers, allocate resources to different users, and actually augment groundwater through water supply development (Jacobs and Holway, 2004). The regulations are statewide, but implementation is done on a local scale with basins or sub-basins being used for boundaries. The programs under development run a continuum from low coercion/indirect impact policies (volunteer) to high coercion/direct impact policies (regulation) but it has been found that the regulatory programs have met with more opposition and as such are used only if necessary (Jacobs and Holway, 2004). Other needs for the management of groundwaters in Arizona include the monitoring of water use, enforcement authority for those who violate their permits, and the use of conservation techniques for all users (Jacobs and Holway, 2004). The use of numerical models to look at management policies is being used in Arizona, but the modelling is being done by a government department (Jacobs and Holway, 2004). Arizona recognizes that proper groundwater management will require a combination of regulatory and non-regulatory methods that will continuously evolve over many years as new information is gathered.

2.2 Groundwater Resource Management in Canada

Canada does not have a national strategy for groundwater management (Rivera, 2004; Rivera *et al.*, 2003). Management is controlled by the different provinces and territories and is done on a regional level. Because each province and territory is independent of each other, the management plans that do exist are quite different and have relatively inconsistent coverage (Rivera, 2004). Canadians are beginning to become concerned with the sustainability and quality of the groundwater resources. This is due to the fact that 30% of Canadians (almost 10 million people) rely on groundwater resources for their drinking water (Rivera, 2004). Of the groundwater removed from the soil, approximately 14% is used for industry, 43% is used for agriculture, and 43% is used domestically (Rivera, 2004). As a result, groundwater will likely become a strategic national resource in the future because it affects everyone in some respect (Rivera, 2004).

The most recent attempt at a national assessment of groundwater resources was done in 1967, but new initiatives are beginning both on a federal and provincial level to better understand the resource (Rivera, 2004). To perform an inventory on groundwater resources across Canada, a series of studies would have to be conducted varying from a local to an international scale. The studies would be part of national co-operative programs that could allow provinces to share technology and expertise, the identification of missing information, groundwater data and information, and training and accreditation programs (Rivera *et al.*, 2003). The studies from the different provinces would then have to be integrated into a National Groundwater Inventory (NGI) (Rivera, 2004). From the information gathered, carefully built and calibrated numerical models could be used to assess the aquifers. The models would need to be updated on a 5 year basis to ensure that they are as relevant as possible (Rivera, 2004). These models could be used in developing sustainable management plans that incorporate socio-economic and environmental issues (Rivera, 2004).

In Canada today there are no methodologies for sustainable yield in practice, but provinces are starting to take action. The lack of planning was due to the fact that groundwater is taken for granted because of the large abundance of the resource. There have not been any areas in Canada where the water resources have been overexploited to date, but studies have found that there have been negative impacts on surface waters due to groundwater use (Rivera, 2004). As a result, a partnership between the federal and provincial governments could lead to a better understanding of the groundwater resources in Canada so that they can be properly managed (Rivera *et al.*, 2003).

2.2.1 Canada Water Act

The federal legislation covering water management issues in Canada is the *Canada Water Act* (CWA). Environment Canada is the department of the Canadian federal government responsible for regulating the CWA. It is "an Act to provide for the management of the water resources of Canada, including research and the planning and implementation of programs relating to the conservation, development and utilization of water resources" (Canada Water Act, R.S. 1985). The CWA is essentially a framework to allow for water management, both quantity and quality, within Canada. It is broken up into three parts: Comprehensive water resource management, Water quality management, and a general section on regulations and enforcement. Within the CWA itself, there is no specific mention of groundwater issues. The CWA covers any water management problems with emphasis on those that are federal, inter-provincial, or if the waters are international.

2.2.1.1 Comprehensive Water Resource Management

This section of the CWA outlines who should be managing the water resources of any given area. The federal government can make an arrangement with itself and one or more provincial governments to establish a committee to manage the water resources for any area in Canada. The CWA goes on to describe what programs can be established with the provinces for waters of national interest. The programs include resource inventories, collection of water quality and quantity data, conducting research, and the formulation and implementation of water resource management and conservation plans.

In certain areas, the federal government will develop water management plans on their own. These include any federal waters, inter-jurisdictional waters, and international or boundary waters. The federal government will not begin a water management program on their own without first trying to work with provincial, or other concerned governments.

The CWA outlines the details of any inter-governmental relationship that would be developed for water resource management. Details include who is responsible for what parts of the work, who pays for the work, how the work will be presented and any other terms and conditions that need to be determined in the agreement.

2.2.1.2 Water Quality Management

The second section of the CWA governs water quality management. It is very similar to the first section in describing the process that will be undertaken to establish committees between different levels of government to manage water quality issues. The government describes and will designate specific areas that are of national concern, and will attempt to get the provincial or territorial governments concerned to enter into an agreement and form a water quality management agency.

This section of the CWA also allows the federal government to make regulations on the concentrations of substances and classes of substances in water. The federal government can prescribe the procedure that must be followed for the different agencies to develop their own water quality standards, as well as the development of the charges that can be levied if the regulations are broken.

2.2.1.3 Regulations and Enforcement

The final section of the CWA is a general section that includes the role of an inspector, offences and punishments for a contravention of the Act, and the methodology for the release of information to the public. Any information gathered through the CWA is under the direct control of the Federal Minister. It is up to the Minister's office to decide what is released and how it will be distributed or published.

This section describes in detail how the financial aspects of the CWA work for funding agencies or giving loans and grants to the provinces. It also includes a statement indicating that the Minister of the Environment has to report to parliament once a year about the operations under the CWA.

2.3 Water Resource Management in Ontario

2.3.1 Prior to 2004

Before 2004, there were few groundwater studies being done in Ontario. Only environmentally sensitive areas were given any attention in the province (Gerber and Howard, 2002). The task of managing the groundwater resources in the province fell to the conservation authorities (CAs). The CAs were originally created to form a partnership between the municipalities and the province to manage surface waters, other natural resources and to create education and outreach programs for the public (Ivey *et al.*, 2002). The role of the CAs evolved over time to include managing groundwater resources, but the extent of the initiatives were determined by the municipality and the CA board. The more the population in the watershed depended on groundwater, the more groundwater studies were done (Ivey *et al.*, 2002). The efforts of some CAs were to reorganize existing MOE data, while others engaged in short term groundwater monitoring studies (Ivey *et al.*, 2002). Another very important task taken on by the CAs was education and outreach programs for the public.

One program that was instituted by the province of Ontario was the Provincial Water Protection Fund. It was designed to allow municipalities to assess their water resources and develop protection measures (de Loë *et al.*, 2005). The program resulted in municipalities defining wellhead protection areas, identifying potential contaminant sources and drilling groundwater monitoring wells.

From the early work of the CAs, some concern arose about the capacity of the different CAs to perform groundwater management. Large and better funded CAs would be able to afford to do studies while a smaller CA could not (de Loë *et al.*, 2002; Ivey *et al.*, 2002). The CA had to build its capacity to perform the studies on several levels including technical, financial, institutional, political, and social (de Loë *et al.*, 2002). The most important factors that were associated with performing the studies were the political will, the relationships that the CA had with other agencies, and the involvement of the citizens (de Loë *et al.*, 2002; Ivey *et al.*, 2002). The most important factor was the formal or informal vertical and horizontal linkages between the CA and other communities and agencies so that expertise and information could be shared relatively easily (de Loë *et al.*, 2002). Since the CAs were originally designed for managing surface water, they may not have been able to manage if the demands for groundwater management became too high (Ivey *et al.*, 2002).

Following the Walkerton tragedy, the government decided to ensure safe drinking water for all those inhabiting the province. As a result of this, water management has become a priority within the province and as such funding will increase to the agencies involved.

The management approach for Ontario is called Watershed Based Source Protection Planning. A more detailed discussion on the steps taken can be seen in Section 2.3.4.

2.3.2 The O'Connor Inquiry

2.3.2.1 Introduction

Walkerton is a small town of 4,800 in southern Ontario that uses groundwater as the source for its drinking water. In May 2000, the water supply in the town was contaminated with a deadly strain of *E. coli*. This contamination resulted in the death of 7 people and caused more than 2,300 to become ill (O'Connor, 2002a). As a result of this incident, many questions were raised about the safety of drinking water in Ontario. The government decided to conduct an inquiry into the specific events at Walkerton as well as any policies and procedures that were in place that contributed to the incident.

2.3.2.2 Public Inquiry

Justice Dennis O'Connor led the Walkerton inquiry and presented his report in two parts. The first part was completed in January 2002 and was directed specifically to the events that led to the disaster in Walkerton. The second part, delivered in May 2002, considered a wider view of what the province of Ontario could do to better protect its water resources so that the tragedy would never be repeated.

Part 1 of the inquiry determined that the contamination of the drinking water system came from the runoff from a farm entering into a very shallow supply well. Due to several factors, the chlorine added to the water was not sufficient to destroy the contaminants and as a result the water distribution system became contaminated.

Beyond the physical causes, there were a number of other factors that contributed to the problem. Programs and policies that were in place from the MOE were not adequate to prevent the incident from occurring. The problem was that the MOE did not take appropriate action when problems were discovered. The deficiencies were mostly in the approvals, inspection, and operator certification and training programs (O'Connor, 2002a). Since these deficiencies were discovered, steps have been taken to rectify the problems.

In Part 2 of the Walkerton inquiry report (O'Connor, 2002b), Justice O'Connor put forth 93 recommendations to improve the reliability of drinking water safety in the province. He suggested a multi-barrier approach with the first barrier being the protection of the source of drinking water. In a watershed-based planning process involving both the MOE and the conservation authorities within the province, a source protection plan could be developed within an area that everyone must follow, from large municipal water suppliers, to local farmers.

Other barriers of protection for drinking water include having a set of water quality standards and technologies that treat the water and monitor its quality (O'Connor, 2002b). These standards should be continuously updated as knowledge in the field increases. Municipal water providers should have an approved operational plan, and their workers should be certified on an ongoing basis.

The report also indicated that the provincial government should have an increased role in protecting the drinking water in the province by legislation. The *Safe Drinking Water Act* would allow the government more leverage to increase enforcement on those not respecting drinking water regulations (O'Connor, 2002b).

The recommendations were also designed to provide transparency and accountability to water supply. Decisions that affect drinking water would be made more carefully if the public was made aware of the situation and that individuals would be held accountable for the decisions that were made (O'Connor, 2002b).

2.3.3 Ontario Water Resources Act

In the province of Ontario, the legislation that governs water use is the *Ontario Water Resources Act* (OWRA). It is a very comprehensive piece of legislation that covers such topics as water, wells, water and sewage works, agreements between different levels of government or other agencies, Ministry work, and other topics. The section of the legislation that is most important for water management and specifically groundwater management are sections on water and wells.

The OWRA states that the Minister of the Environment within the MOE is in charge of the supervision of all surface and groundwaters in Ontario. The OWRA defines what contaminated water is, and how it can become polluted. Under the OWRA, the Minister has the right to order a person to stop discharging polluted material, and ensure that someone who is allowed to dump certain material has measures on hand to alleviate possible impairment of water quality.

One specific section that applies to water management is Section 34, Taking of Water. The OWRA states that "no person shall take more than 50,000 litres of water in a day" (OWRA, 1990). This applies to everyone in the province except those that need water for firefighting, and water used for domestic or farm purposes.

The MOE has divided the province of Ontario into 5 administrative regions; Northern, Eastern, Central, West Central and Southwestern. Each of the regions is managed by a Director who reports to the Minister of the Environment. The Director of any of the 5 regions in Ontario can give a permit to allow for greater takings than the maximum. The Director can also order an individual to stop taking water if it is interfering with any other person's interest. The OWRA also states that the Director can refuse to issue a permit, cancel a permit at their discretion or change the conditions of a permit if the need arises. If any of the sections in the OWRA are contravened, the person who did it is guilty of an offence and is punishable under the Act.

The final relevant section to groundwater management is the part of the OWRA that refers to wells (section 35-50). If an area is designated by the provincial government, no person can install a well unless it has been approved by the Director of that area. In any other area, if the fee is paid and the application is in order, a well construction permit will be granted. The Director can refuse a permit if the proposed well would contravene the OWRA, or if there is a danger to people or property. Also the permit can be refused if there would be a reduction in the quantity of water available for any use. This part of the OWRA gets into great detail concerning the license and qualifications of the well technician who will install the well.

2.3.4 Drinking Water Source Protection Act

As a response to the report produced following the Walkerton inquiry, the Ontario government began its movement towards protecting water resources for the population. The O'Connor inquiry put forth a number of recommendations that set out how water resources should be managed in the province, including a multi-barrier approach to protecting drinking water. The first barrier in the recommendations was to protect the source of the drinking water whether it is from surface waters or ground waters (O'Connor, 2002b).

At the time of writing this thesis, only a draft version of the *Drinking Water Source Protection Act* (DWSPA) was available from the *Environmental Bill of Rights* (EBR) Environmental Registry. Although comments will undoubtedly be made concerning the legislation, very large changes will most likely not occur. The DWSPA was developed based on reports from the Technical Experts Committee (Ontario, 2004a) and the Implementation Committee (Ontario, 2004b) assigned to determine the best approach for protecting the drinking water of Ontario

The legislation divides the province into a number of source protection areas. These areas, typically watersheds, are those described under the *Conservation Authorities Act* meaning each CA will be responsible for a single source protection area. Under the DWSPA, a Conservation Authority is referred to as a source protection planning board (SPPB). The SPPB will then create a source protection planning committee (SPPC). In areas of Ontario where Conservation Authorities do not exist, the Minister of the Environment will determine the source protection areas and appoint an individual or body to serve as the SPPB. The SPPBs within an area will help the SPPC by supplying any technical information as well as administrative support that might be required (Ontario, 2004c).

Under certain circumstances, the Minister of the Environment may determine that several watershed protection areas be put together into a source protection region. One SPPB will be designated as the lead board for that region and will create the SPPC for the region. The SPPC will be responsible for the source water protection plan (SPP) for all the areas within the region. The lead board will assist the other boards as necessary through administrative and technical support, and will serve as the liaison between the Ministry and the SPPBs in the region and will ensure that the source water protection plans put forth by the committee for the different areas are not in conflict (Ontario, 2004c). An overview of the source protection planning process can be seen in Figure 2-1.

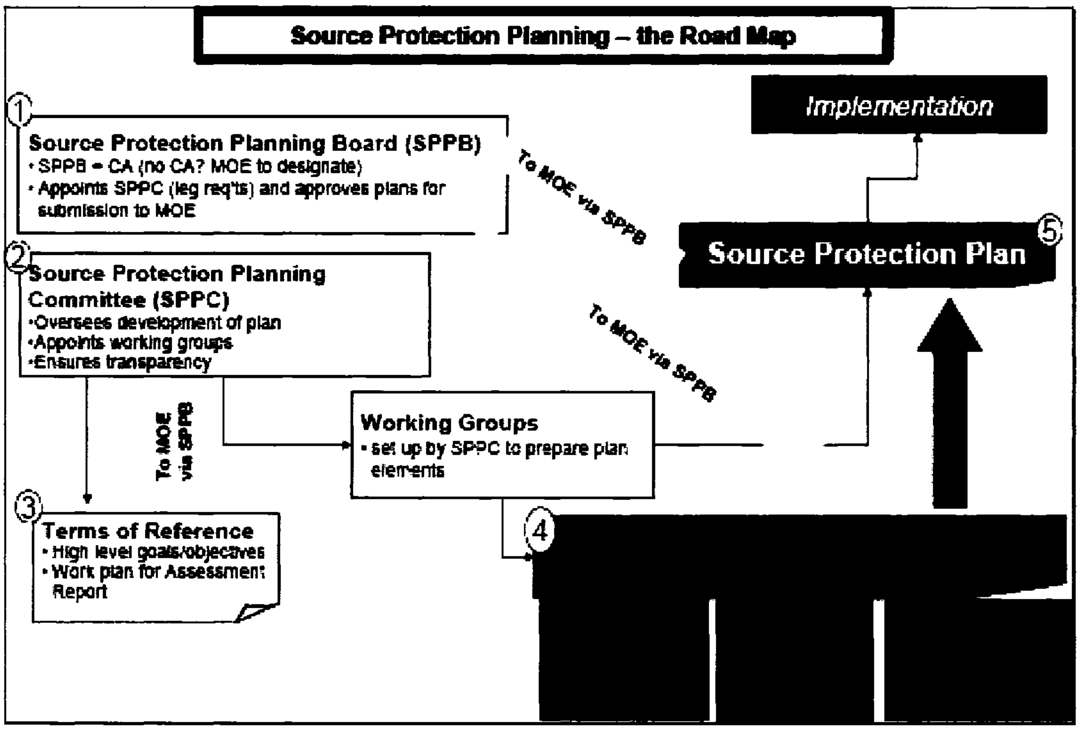


Figure 2-1: Source protection planning framework (From Ontario, 2004a)

2.3.4.1 Implementation

Source protection will be put in place in the province of Ontario by having each SPPC produce a terms of reference for their individual areas that identifies the specific watersheds for which an assessment report and protection plan will be made. The terms of reference should include (Ontario, 2004c):

- Reasons for preparing the assessment report and source protection plan.

- The boundaries of the watershed.
- A plan of work for producing the assessment report and source protection plan.
- Description of information, existing and non-existing, required for the creation of the source protection plan and assessment report. This means locating studies that already exist, and determining what technical field work is required to complete the task.
- A consultation plan listing the advisory committees required, how the public will be informed and consulted, and the methods used to resolve potential concerns of municipalities and First Nations.

Once completed, the terms of reference will be submitted to the Minister of the Environment who will amend them if necessary, and make the final version available to the public.

The second step to be performed by the SPPCs is the preparation of an assessment report. An assessment report has to be written for each watershed identified in the terms of reference. The assessment report must include (Ontario, 2004a; Ontario, 2004c):

- Identification and description of features within the watershed, specifically surface waters, aquifers, groundwater recharge and discharge zones, other hydrologically sensitive areas, wells (existing and anticipated) and wellhead protection areas (WHPA), surface water intakes (existing and anticipated) and intake protection zones (IPZ).
- Assessment of water quality and quantity especially detailed in hydrologically sensitive areas, WHPAs, and IPZs.

- A water budget indicating all the different paths where water enters and leaves the watershed including water removals permitted through the OWRA.
- An analysis of the risks present in the watershed and a classification to determine the priority for mitigating the risks during the source protection plan phase.
- A description of how gaps in information can be identified and filled in during the next phase.
- Identification of any other issues that are encountered in this phase and a description of programs that can be used to deal with the issues.

The assessment report, once completed by the SPPC, is submitted to the SPPB for the area. Once it has been approved by the source protection planning board, it is submitted to the Minister of the Environment for final approval, and is made available to the public.

The final step in source protection planning that is covered by the Drinking Water Source Protection Act is the Source Protection Plan (SPP). The SPP includes (Ontario, 2004c):

- The terms of reference and the assessment report already created for the area.
- The objectives of the plan and the standards that will be used to determine if the plan was successful.
- Determining the mandatory and voluntary measures that are needed to achieve the plan, including identifying parties that would be responsible.

Once complete, the SPP must be approved by the SPPB and finally the Minister of the Environment who will post the plan on the EBR site for public comments before approving the plan or not.

The DWSPA also indicates that if the SPPBs do not submit their terms of reference, assessment reports or SPPs within a specified period of time, they will have to justify why they have not produced the required report. If the justification is not appropriate, the Ministry of the Environment will prepare the necessary report, and the board will have to repay any funding that they were given, and supply any information that they have already obtained (Ontario, 2004c).

Finally, the SPPB must prepare an annual report to be given to the Minister indicating what progress has been achieved towards the objectives of the source protection plan (Ontario, 2004c).

Chapter 3: Ontario Permits to Take Water

The Ministry of the Environment (MOE) in Ontario is responsible for managing the water resources in Ontario. As noted previously, this task is done under the Ontario Water Resources Act (OWRA). In section 34 of the OWRA, it is stated that anyone using more than 50,000 litres of water a day, either from surface or groundwater takings, has to obtain a permit. This Permit to Take Water (PTTW) is obtained through an application process to the MOE and evaluated through the Water Taking and Transfer legislation.

3.1 Original Permitting Legislation

3.1.1 Water Taking and Transfer Legislation

The purpose of Regulation 285/99 of the OWRA, the original permitting legislation, was to ensure that there was sufficient conservation, protection, wise-use and management of the water resources in Ontario because they are important to the long-term environmental, social and economic future in Ontario. (O. Reg. 285/99)

For administrative purposes, the Ministry of the Environment has divided the province into five regions; Northern, Southwestern, West Central, Central, and Eastern. The decision to approve a permit fell to the Director of the region of the province where the application was being made. Under Regulation 285/99, the Directors of the different areas in Ontario had to look at a number of matters when considering an application for a PTTW. These matters included the protection of the natural ecosystem, how the taking of surface water would affect groundwater in the area, and how the taking of groundwater would affect the surface waters in the region. The Director also considered other users in the area, and how the taking might impact them. Other important factors considered were how the water would be used (municipal water supply, agriculture or livestock, domestic, industrial, or commercial use) and if it was in the public's best interest to issue the permit. Other aspects that the Directors concerned themselves with were the notification of other interested agencies and jurisdictions of the proposed water taking including any obligations under the Great Lakes Charter. (O. Reg. 285/99)

The Regulation also discussed the removal of water from one of the three basins in Ontario. The basins are the Great Lakes-St. Lawrence Basin, the Nelson Basin, and the Hudson Bay Basin. Water was not allowed to be transferred from one basin to any other, unless the water was used in the manufacturing and transport of a product, or if the water was packaged in containers with a volume less than 20 litres.

3.2 Original Permitting Procedures

3.2.1 Permits to Take Water – Original Review Process

The original review process for the approval of Permits to Take Water was in effect until January 2005. To apply for a permit, the applicant filled out a copy of the Application for a Permit to Take Water which was accompanied by an instructional guide (Ontario, 2000). A copy of this application can be seen in Appendix A. The application guide asked that for surface water withdrawals, the applicant had to show that their water taking would not affect water levels, stream flow, the ecology of the area, and the recreational use of water by others. They also indicated if there would be any impacts on water quality or quantity to other users (Ontario, 2000).

For groundwater takings, the guide asked that the applicant indicate how other groundwater users would be affected by the proposed withdrawal and state how the determination was made (Ontario, 2000). The guide also stated that "for large groundwater takings, areas where there are numerous users of the groundwater resources, or in areas of hydrogeological complexity, it may be necessary to have a qualified professional investigate and report on the potential impact of the proposed water taking on the groundwater resources and existing uses" (Ontario, 2000). Applicants for groundwater use had to include a monitoring program for the actual impacts of their water taking and a contingency plan to mitigate any possible interference with other users (Ontario, 2000). In a review of actual PTTW applications, it was found that there was typically little supporting documentation supplied with the applications (Golder, 2002).

When the application was reviewed, it was the MOE that was the lead agency for granting the PTTW. In some cases, other agencies such as the federal Department of Fisheries and Oceans (DFO), the Conservation Authority (CA) in the area or the Ministry of Natural Resources (MNR) also had to give their approval. It was up to the applicant to provide the approvals from the other agencies to the MOE before their PTTW could be granted (Golder, 2002).

In the past, permits were approved with very little technical justification for the quantity of the taking or the predicted effects on others. There was no need for monitoring, but if another user was impacted, the proponent would have to mitigate the problem. In the more recent past, there was significantly more technical justification required to show the potential impacts before a permit was approved (Golder, 2002).

The cumulative effects of water withdrawals were never considered by the MOE. Each permit was considered in isolation from each other. The MOE realized the potential problems that might arise from this policy and in 2002, were attempting to address the problem, but it was administratively difficult to do under Ontario Regulation 285/99 (Golder, 2002).

One final aspect of the review process for a PTTW application was the notification of the public. It was recommended in the application guide that the proponent meet with those individuals who had concerns about the water taking, and address any issues prior to submitting an application (Ontario, 2000). Once the application had been submitted, the MOE posted the proposal on the *Environmental Bill of Rights* (EBR) Environmental Registry (Golder, 2002; MOE 2000a). The public then had the opportunity to comment on the proposal for a minimum of 30 days. The Director then considered the public's comments when reviewing the application. If a member of the public commented on the proposal and they disagreed with the decision, then they had the right to appeal the decision (Golder, 2002).

3.3 Current and Proposed Permitting Legislation

3.3.1 Water Taking and Transfer Legislation

Following the O'Connor inquiry after the Walkerton tragedy, the government re-evaluated the methods it used to determine if a water taking permit should be issued or not.

As indicated under the OWRA, if more than 50,000 litres of water a day is to be withdrawn, a PTTW must be obtained from the Director representing the MOE in the appropriate area. To this end, the government of Ontario passed a new Water Taking and Transfer regulation (O. Reg. 387/04), under section 34 of the OWRA, in December 2004 that replaced the previous regulation (O. Reg. 285/99).

"The purpose of this regulation is to provide for the conservation, protection and wise use and management of Ontario's waters, because Ontario's water resources are essential to the long-term environmental, social and economic well-being of Ontario" (O. Reg. 387/04). The purpose of the Water Taking and Transfer regulation was not changed from the original document. What was improved were the details of what the Director must consider when evaluating a PTTW application.

The Director must consider 4 things when canceling, amending, imposing conditions upon, or issuing a permit (MOE, 2005).

1. The need to protect the natural functions of the ecosystem. This includes such areas as stream flow and water levels for both groundwater and surface water and how they interact with each other and any changes in quantity or quality that would result from the water taking.
2. Water availability. The Director must consider the impact of the water removal on the water balance for the area and the sustainability of aquifer yield. They must also think about both low water conditions and the type of watershed that the water is being withdrawn from. A high use watershed will have different rules than a medium use watershed. They must also determine if the water takings will interfere with any planned or existing municipal use.
3. Issues regarding the use of the water. The Director has to ensure that proper conservation techniques with the best water management practices are used at the site requesting approval. The purpose of the withdrawn water must also be considered. If the water is not currently being used by the applicant, the Director must determine if the permitted quantity of water will ever be used in the future.
4. Other issues that must be considered include the interests of any other person that is using the same water and any other matters that the Director decides to be of relevance for the permit.

In an entirely new section, the regulation goes on to explain the decision making process in high use watersheds. It only applies to applications for new or expanding water takings or water takings for specific purposes such as manufacturing with large quantities of water. The section is based upon two watershed usage maps produced by the Ministry of the Environment. Both maps break down Ontario into several tertiary watersheds as defined by the MNR. The first map (Figure 3-1) identifies high water use watersheds based on average annual conditions. The second map (Figure 3-2) identifies watersheds with potential water quantity problems based on the summer low flow conditions.

PTTW applications will be denied if they fall within a high use watershed as illustrated on the annual map (Figure 3-1). If the PTTW application is for an area with a high usage during the summer months (Figure 3-2), the new PTTW will include a condition that the water taking will cease between August 1st and September 11th at a minimum. Of course there are exceptions to the rule for several different watersheds including the St. Lawrence and Ottawa Rivers. The exceptions include PTTW applications for municipalities and the extraction of aggregates where the water taking is incidental to the extraction (O. Reg. 387/04; MOE, 2005).

The regulation finishes by addressing both the Great Lakes Charter, and the right to information by the public. Ontario must ensure that it is respecting its obligations to the Great Lakes Charter. In terms of notice and consultation with the public, the Director must notify interested parties including municipalities and the Conservation Authority for the area. The Director can post a notice for the application on the EBR Environmental Registry for the general public and make the applicant notify concerned parties (O. Reg. 387/04; MOE, 2005).

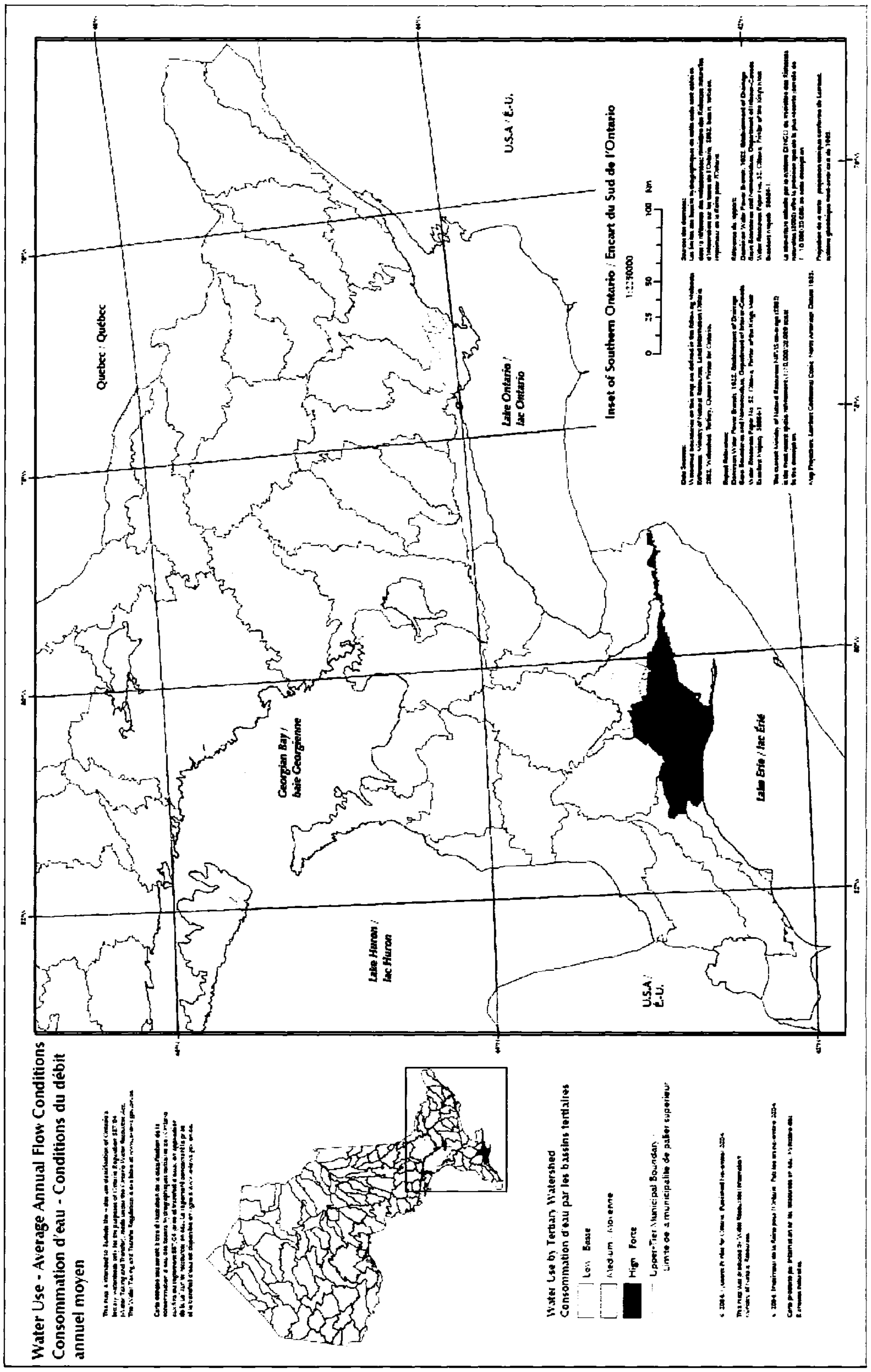


Figure 3-1: Average annual water use conditions in Ontario (From MOE, 2005)

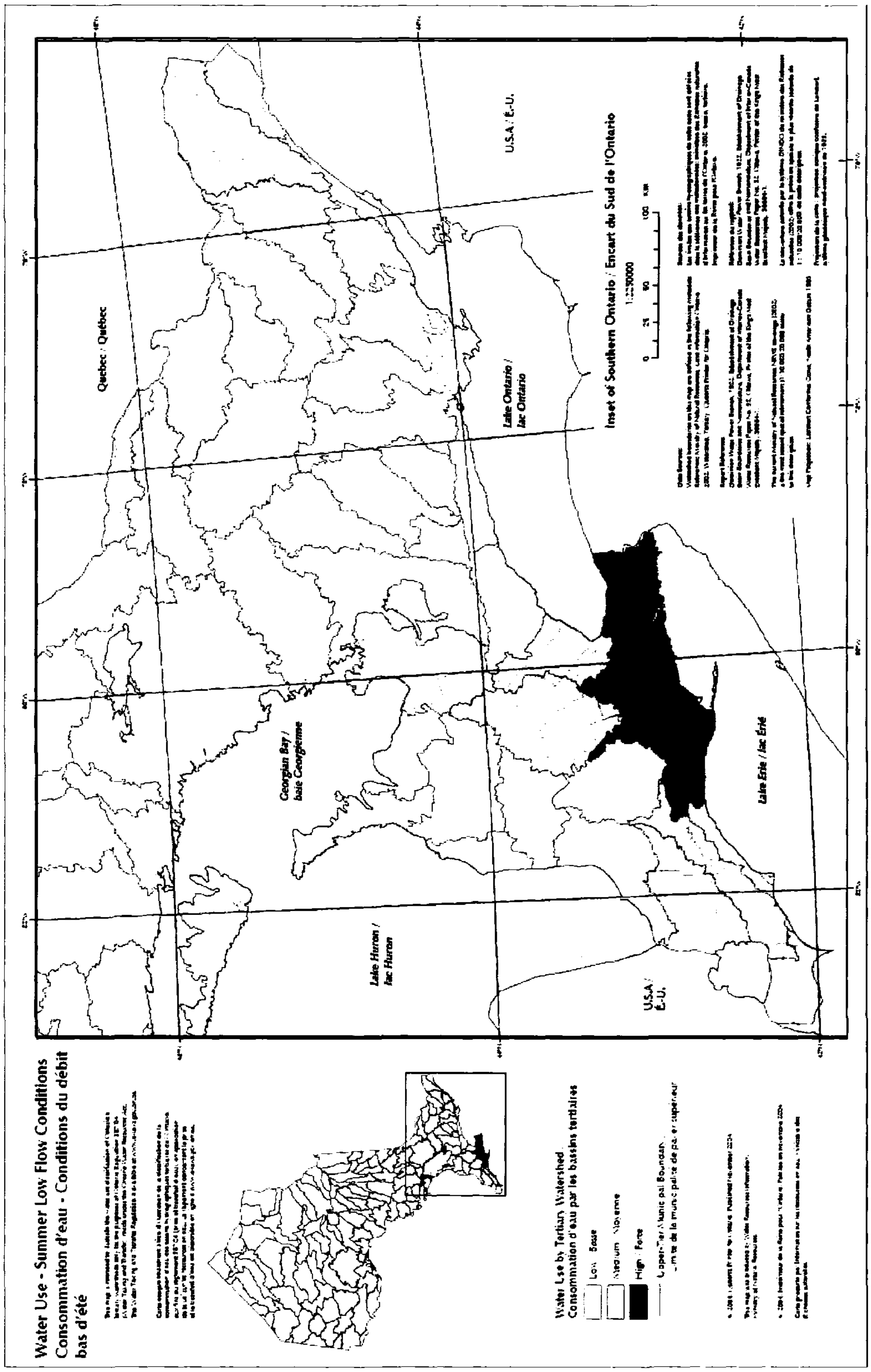


Figure 3-2: Summer low flow conditions in Ontario (From MOE, 2005)

The new regulation also states that all PTTWs if approved will have the condition that accurate volumetric readings of the water removed on a daily basis must be recorded and provided to the MOE on a yearly basis. It also states that water cannot be transferred out from any of the three basins in Ontario (Great Lakes-St. Lawrence Basin, Nelson Basin, Hudson Bay Basin) (O. Reg. 387/04; MOE, 2005).

3.4 Current Permitting Procedures

The new applications for a PTTW are different than the old ones both in the information gathered as well as the way they are evaluated. The new application form can be seen in Appendix B. To help applicants with their applications, a guide is available to answer many of the questions for the new system. A manual was also released to provide guidance to Directors and technical reviewers when considering a PTTW application. The manual was released in late December 2004, just before the new Water Taking and Transfer Regulation (O. Reg. 387/04) came into effect January 1st, 2005. A new version of the manual and the guide were released in early 2005.

The PTTW system is in place to achieve the environmental objectives that are laid out in the OWRA. The permits also help to control water quality and quantity interference problems, and to help with solving the situation if a problem does occur. As seen previously, the OWRA attempts to control the taking of water in Ontario. Along with the OWRA there exist common-law riparian rights for water resources, where land owners along rivers and lakes can use the water for their own use as long as it does not interfere with other users downstream. These riparian rights are not superceded by Section 34 of the OWRA, however they might be limited by the Act. The person taking the water would be governed by the more limiting of the two provisions (MOE, 2005).

There are takings that are exempt from section 34 of the OWRA. These include takings for firefighting, takings for ordinary household purposes, and takings for watering livestock and poultry, and takings that are less than 50,000 litres in a day. However, if there is any interference from these takings with other users, then a PTTW will have to be obtained.

The policy behind the PTTW process is based on a number of water management principles. These can be found in the OWRA, the *Statement of Environmental Values* under the *Environmental Bill of Rights*, and *Water Management Policies, Guidelines, Provincial Water Quality Objectives of the Ministry of Environment*. To ensure fair sharing, conservation and sustainability of the water resources in Ontario, six guiding principles were developed (MOE, 2005).

1. Recognize both water takers' reasonable needs for water and the natural function of the ecosystem.
2. A consistent and structured approach will be taken to manage water takings by incorporating risk management principles into the PTTW application process.
3. Encourage fair sharing of the resource by making permit holders control their takings if interference with other users of water occurs.
4. The cumulative impacts of water takings will be considered.

5. Adaptive management will respond to ever changing environmental conditions.
6. Public and local agency participation will be encouraged in the PTTW application process.

The introduction of risk management in the PTTW application process has allowed for the development of a classification system for the different applications. Simply, the applications that have a high potential for impact will be subjected to increased scrutiny and scientific evaluation. The three categories are summarized in Table 3-1.

3.4.1 Types of Permits

3.4.1.1 Category 1

Category 1 permits are water takings with low risk of causing undesirable environmental impacts. They are typically existing water withdrawals and will be considered to be Category 1 as long as "the Director is satisfied that the continued taking is not likely to have adverse environmental impact/interference, or has not previously requested the permit holder to submit additional studies" (MOE, 2005). New applications will not be classified as Category 1 unless they meet very specific criteria.

Category 1 applications do not require any technical studies to be submitted for evaluation purposes to the MOE. Detailed information concerning the source and other factors such as conservation practices being used must be included with the application.

The conditions that can be placed on the PTTW will be standard for all permits so that there will be limited interference between water users as well as to minimize any environmental impacts (MOE, 2005).

Table 3-1: Different categories of PTTWs (From MOE, 2005)

Groundwater
Category 1
Renewal (same or lesser amount, same purpose, same location, same source, no past interference/ impacts, and no scientific study required as part of renewal).
Ponds (e.g. irrigation and agriculture) • not connected to nor receiving water from surface water; • and <4m deep and >100m from the nearest stream or wetland; • or <7m deep and >250m from the nearest stream or wetland
Category 2
Short-term, non-recurring taking less than 7 days (e.g. pumping test or hydrostatic test).
Short-term, non-recurring taking less than 30 consecutive days and less than 400,000 litres/day (e.g. construction dewatering and dust suppression).
Category 3
All groundwater takings that do not meet Category 1 or Category 2 criteria.

3.4.1.2 Category 2 and 3

Category 2 PTTWs are generally for short term water takings whereas Category 3 PTTWs are for new or increased water takings of longer duration. Both have a higher risk of environmental impacts, and potential for interference with other users. Permits that fall into these Categories include (MOE, 2005):

- Those that are for new or increased water takings.
- Existing takings where further information concerning interference with the ecosystem or other takings was requested by the MOE for technical review.
- Existing permits that the Director has deemed to be interfering with the environment or others users.

Category 2 applications are scoped permits and are submitted with a scientific evaluation performed by a qualified person to ensure that the water taking is for a limited period of time, either less than 7 days or less than 30 days with a pump rate of less than 400,000 L/d. The application has to indicate that takings will not impact other users or the environment. Any approvals required by other agencies must be included with the application. Due to the short duration of the Category 2 permits, many of the potential impacts can be dealt with through proper water taking design, monitoring and following the conditions set out by the qualified person. The application will be screened by the MOE to ensure that it does not contravene regulation 387/04 and evaluate the scientific report to ensure that it is complete and accurate (MOE, 2005).

Category 3 applications require the submission of scientific studies prepared by qualified persons that show the potential impacts of the water taking. The MOE will carefully review the scientific studies to ensure their completeness and accuracy. The application will also undergo the same evaluations as the Category 1 and 2 applications before a decision is reached. Table 3-2 illustrates the requirements of the applicants as well as the evaluation of the MOE for the different Categories of PTTW (MOE, 2005).

The MOE will help the applicant before they submit their application. The pre-submission consultation (PSC) will help those applying for a Category 2 or 3 permit determine how potential environmental problems from their water withdrawal might affect other agencies such as the MNR, the DFO and any CAs that fall within the affected area. The applicants will also have to consult with local municipalities to determine if there are any wellhead protection areas in the vicinity of the PTTW or any other future planned water uses.

3.4.1.3 Qualified Person

The MOE defines a qualified person for evaluating groundwater withdrawals as a licensed Professional Geoscientist under the Professional Geoscientists Act or a licensed Professional Engineer under the Professional Engineers Act who is competent through training and experience to engage in practices that would also constitute the practice of professional geoscience (MOE, 2005; Professional Geoscientists Act, 2000). Other qualified persons such as geotechnical engineers may be required depending on the type of study being done.

Table 3-2: Submission and screening process for different categories of PTTWs (From MOE, 2005)

Category	Applicant Submits	Ministry Action
1	• Completed Application Form. • Information required by conditions of previous permit.	• Ministry staff will check the information submitted by the applicant for completeness. • Technical Screening: Check whether existing permit requirements and screening criteria are met. Check to ensure conformity with O. Reg. 387/04 requirements (e.g., High Use Watersheds, Great Lakes Charter, water conservation and complete required notifications).
2	• Completed Application Form. • Information required by conditions of previous permit. • Scientific evaluation completed by a qualified person.	• Ministry staff will check the information submitted by the applicant for completeness. • Technical Screening (as in Category 1). • Ministry staff will check the scientific evaluation (schedule 2 and/or 3) prepared by a qualified person for completeness and may undertake audits to determine if the requirements are being met.
3	• Completed Application Form. • Information required by conditions of previous permit. • Scientific study (hydrogeological and/or hydroecological study) completed by a qualified person.	• Ministry staff will check the information submitted by the applicant for completeness. • Technical Screening (as in Category 1). • Ministry staff will conduct a scientific review of studies prepared by a qualified person.

In the case of surface water takings, experts are defined as an individual with a bachelor degree specializing in hydrology, biology, physical geography, or water resource management or engineering. Qualified people must perform work that falls in line with their education and experience (MOE, 2005).

3.4.2 Evaluation of Permits

If the Director is considering a permit under section 34 of the OWRA, they must abide by the items to be considered as laid out in Regulation 387/04. The Director must look at the natural functions of the ecosystem including potential impacts of water levels and stream flows. They must consider the availability of water, looking specifically at the water balance of the area including other takings such as municipal water supplies. The use of the water must also be defined and any conservation practices that the user will implement should also be indicated. The final consideration would be the potential impacts on other water users in the area. These considerations must be applied to all applications with the difference being that Category 1 applications will get a standard set of regulations, and Category 2 and 3 applications must be accompanied by appropriate studies and information prepared by a qualified person.

The evaluation of a PTTW for groundwater follows procedures that were based on past experience, those used in other jurisdictions, and a science-based assessment of the potential effects. The three different Categories of PTTW are considered somewhat differently.

3.4.2.1 Category 1

After an application for a category 1 PTTW has been submitted, the MOE will first of all ensure that the application has all of the necessary information. They will then ensure that the PTTW conforms to all parts of Regulation 387/04 during the technical screening. This includes the Great Lakes Charter, high use watersheds and the use of water conservation. Any approval from other agencies (DFO, MNR, CA) will also be required with the application before it is considered for approval.

3.4.2.2 Category 2

The evaluation of Category 2 applications is based on the report of the qualified person. The report must indicate the potential effects on surface waters, other water uses such as other users or previously defined wellhead protection areas (WHPAs), and any contaminants or sensitive areas near the proposed taking. Category 2 permits are typically for short term pumping, and as such it has to be clearly indicated if the permit is for remediating contaminated groundwaters. Since the water is being pumped to the surface, it has to be shown that there will not be any flooding, erosion or impacts to surface water. In most cases, a waste water approval under section 53 of the OWRA is required for the discharge of the pumped water. The qualified person should make specific recommendations on the taking, discharge and any monitoring necessary.

The MOE will look at all of the information submitted for completeness. They will also check the scientific evaluation and can undertake audits to ensure that all of their needs are being addressed. If the permit is approved, the conditions will include the use of buffers around other water users and sensitive or contaminated areas, as well as ensuring that the discharged water has very little impact on the environment.

3.4.2.3 Category 3

The Category 3 evaluation is also known as a detailed hydrogeological study. This study must be performed by a qualified person and will have a full scientific review by the MOE. The applicant can contact the MOE before the studies have started to discuss exactly what requirements must be met including other studies that might have to be conducted beyond the hydrogeological study including hydrological and hydroecological studies.

The hydrogeological study will typically include the following (MOE, 2005):

- Calculation of the area of influence from the water taking based on 20 years of pumping at the maximum daily rate for the maximum requested number of days.
- Determination of any other water users, sensitive areas, or potential contaminant sources within the area of influence and assess the impact on these receptors to determine whether there will be significant interference problems.
- Look at the connectivity between groundwater and surface water and evaluate potential impacts including changes in base flow, reduction of artesian conditions

and possible water quality changes (including temperature). This could involve using tracer tests or installing piezometers in the beds of nearby surface water bodies and monitoring them during pumping tests.

- Determine what measures should be taken to avoid impacts on other users and the environment. Preventative actions, monitoring programs, trigger limits and contingency plans should be defined.

The hydrogeological studies protect the environment by examining issues such as the effect of the water taking on base flow, water quality and habitat. The studies also look at the interconnection between the surface and groundwaters and any changes in flow patterns. The Director can also request the assessment of the sustainable yield of an aquifer to determine if the aquifer can support the new water taking without any impacts on the environment.

All precautions are taken during the application process but if a new water taking interferes with an older taking, with very few exceptions, the new one will have to restore the supply, or reduce their taking so that there is no longer any interference (MOE, 2005). If there was not enough data within the application to predict the extent of possible interference, new PTTWs would have conditions placed on them requiring the monitoring of water levels in observation wells. The data would have to be forwarded to the MOE for analysis (MOE, 2005).

3.4.2.4 Other Considerations during Evaluation

In most cases, the applicant is only responsible for the evaluation of the hydrogeological conditions within the area of influence of their proposed water taking. If more information is required in order to evaluate the cumulative impacts of water takings the Director may initiate larger watershed or sub-watershed studies to be performed. The studies on the change in water balance or sustainable yield due to the withdrawal will be initiated if (MOE, 2005):

- there is a pattern of significant decline in hydraulic head in the aquifer over the previous 5 years;
- the watershed is classified as high or medium water use for summer low flow conditions;
- there is a high density of permitted takings within an area.

The MOE would use the best available science to consider the applications with such tools as numerical models, water budgets based on analytical methods, and water use maps for Ontario.

Water conservation and water use are both important in the PTTW application process. The application must include a statement concerning the efficient use of water and that best management practices are in use or are planned. Water use is considered in the PTTW process because it allows the Director to gather information about the amount of water that will be required by the applicant throughout the year, any other approvals that might need to be obtained such as water discharge permits, and the identification of water uses that actually remove water from the watershed (MOE, 2005).

A notice of the PTTW application is placed on the EBR Environmental Registry so that people who are concerned with the application can come forward and provide information to the Director for consideration. Municipalities and CA are also notified if they are close to the area of the taking. Municipalities outside of the immediate area may also be told if they are likely to be affected by the taking.

Water volumes that are pumped out once a PTTW has been granted must be recorded on a daily basis and forwarded to the MOE once a year. The phase in for the reporting of the data occurs in 3 phases with certain industries coming on line at different times from July 1, 2005 to January 1, 2007.

Although very complicated, the permitting procedure for water removal is a beginning by the government of Ontario to start monitoring and managing its water resources more closely. A summary of the process for evaluating a PTTW can be seen in Figure 3-3.

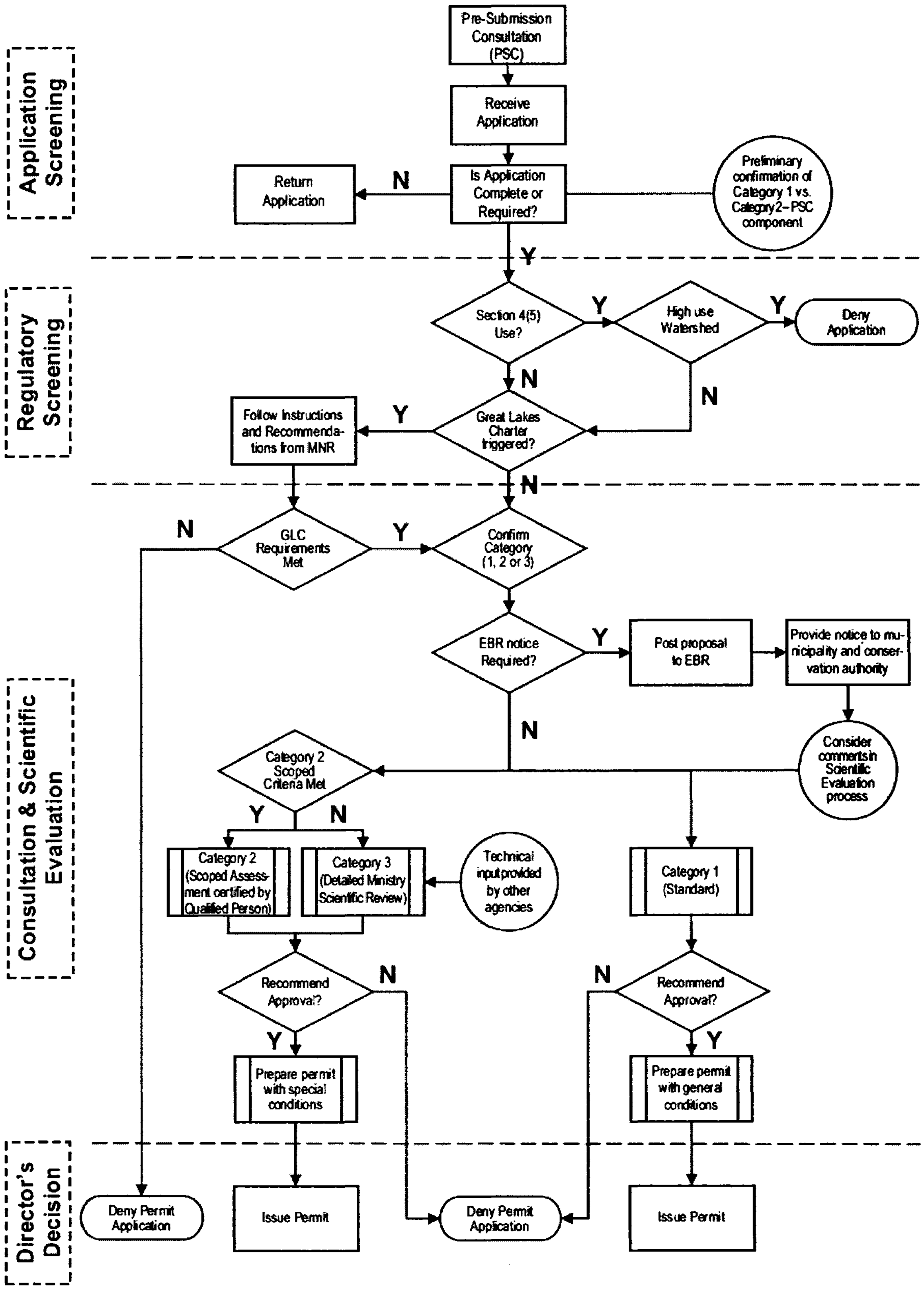


Figure 3-3: Evaluation process for PTTW applications (From MOE, 2005)

Chapter 4: Assessing Cumulative Impacts of Permits to Take Water

4.1 Proposed Methodology for Assessing Cumulative Impacts

Water resource management depends on the interpretation of a great deal of data to be effective. Due to the large need for data, the task of water management becomes one of data management. The data must be as accurate and up to date as possible so that water managers can use it as a tool for effective decision making.

Assessing the cumulative impacts of water withdrawals involves the development of procedures and legislation to support the process. Due to the complexity of the problem, numerical modelling will have to be used to determine cumulative impacts and as such, technical issues must also be addressed to ensure that all PTTW applications throughout Ontario are considered on an equal basis by being modelled as realistically as possible. Numerical modelling is especially applicable to groundwater resources because the effects of removing water from aquifers may not be evident for a long period of time.

Aggregate quarries, below the water table, with their permits for relatively large water pumping rates can potentially cause the water table to fall (Ostrander *et al.*, 1998). But since a quarry is typically only active during the summer months, the effects of dewatering may only be seasonal and may not cause as significant a decrease of the water table as if the aggregate quarries were constantly pumping at their maximum allowed rates.

Assessing the cumulative impacts of PTTWs involves identifying who is responsible for evaluating the applications, how the impacts should be assessed, and when cumulative effects should be considered. This should be done for every PTTW application received in the province of Ontario.

During the formulation of this framework many sources were consulted including an interview with Brian Stratton, Brad Carew, and Bruce Reid from the Source Water Protection group at the Rideau Valley Conservation Authority, Peter Jordan and Colin Heard representing the North West Goulbourn Community Association where the residents could possibly be affected by the cumulative impacts of aggregate quarry dewatering, Stephen Maude from the Land and Water Policy Branch of the Ministry of the Environment, as well as past and present legislation and local PTTW applications and permits obtained for the case study area.

4.1.1 Who Is Responsible For PTTW Application Assessment?

The question of what organization should do the assessment for the cumulative impacts of PTTWs can be quite complicated. Presently, the Ministry of the Environment (MOE) is responsible for issuing the PTTWs within Ontario (Ontario, 2005; MOE, 2005; O. Reg. 387/04). If the draft *Drinking Water Source Protection Act* (DWSPA) is passed, then it will be the responsibility of the different Conservation Authorities (CAs) within the province to manage the water within their watersheds or source water protection areas. With the current system, those who are responsible for water resource management do not make decisions concerning the removal of large quantities of water from their watershed.

Presently, the Director of the region where the PTTW application is being made only has to notify the CA of the area that the application is being made. It is up to the CA to decide if any comments should be submitted concerning the application. CAs who consider water resource management as a priority will submit comments on the PTTW application whereas a CA with different priorities or fewer human resources may not. The comments can indicate whether a different permit is required from the CA, an interpretation of any existing watershed plans for the area, and a more detailed sensitivity analysis on the biology or water quality of a particular site if the data exist (Stratton *et al.*, 2005).

There is no formal delineation of a procedure for the MOE to handle the information submitted by the CA, it is merely considered by the Director during the decision making process. Certainly if the information submitted by the CA to the MOE indicated a potential problem, it would be investigated further before the PTTW was approved.

Water managers should make the decisions concerning the use of water within their regions. For this to occur within Ontario, either the MOE will have to undertake water management for the whole province, or the CA should be given the role of assessing PTTW applications within their source water protection areas. It would appear that since the mandate of the CA will include source water protection under the DWSPA, they will be in a position where they can determine if the PTTW fits into the source protection plan (SPP) of the watershed.

There would have to be some large changes in the existing legislation to allow this to happen. All of the CAs would have to take their roles as water planners very seriously throughout the province. Funding to the CAs would also have to increase. Legislation would have to be changed to make the CA the decision making body for PTTW within the OWRA. This may be impossible to do under the present legislation because the OWRA is administered by the MOE. It is well beyond the scope of this research to determine the legislation necessary to make this change; however it would have to be done in order to ensure the CA has a legal foundation for decision making.

The proposed framework involves establishing a more structured method for the input concerning the PTTW application from the CA to the MOE. This would include an assessment of how the proposed PTTW would fit into the source protection plan for the watershed. The issuing of the permit would become contingent on the approval of the appropriate CA rather than just a notification of the CA. Ontario Regulation 387/04 would have to be amended to include the new connection between the DWSPA and the OWRA. The MOE Director of the region would still make the final decision concerning the PTTW based on the decision of the CA. This method would keep the political aspect of the PTTW process handled by the MOE separate from the scientific basis for decision making used by the CA.

Similar programs have been implemented by the government of Ontario for private water and sewage works and non-municipal drinking systems. The Transfer of Review program operates for municipal and private sewage works where works with low technical complexity, and a low risk to human and environmental health will be reviewed by the designated municipal authority instead of the MOE (MOE, 2000b). The designated municipal authority reviews the application and presents their recommendation for approval or not to the MOE. The agreements between the designated authority and the MOE are different and very well defined for every participating municipality. Under the proposed framework, this type of program could be developed for the CAs with the MOE to give greater opportunity for the CAs to assess the PTTW applications. The MOE would still have to assess any PTTW applications in certain areas such as where there is the possibility of interference along the boundary of a source protection area, or where the CA might be the applicant for the PTTW.

Since the CAs are currently funded by municipalities, care would have to be taken to ensure that the assessment performed for assessing PTTWs was done based scientific methods since development is a priority within many municipalities. Increased funding from the province and the need for the Director of the MOE to give the final decision on PTTW applications would help to minimize any conflicts that develop.

Within the framework, the CA would also have to have the ability to add conditions to the permit so that it fits within the SPP. Similar to what exists for the MOE currently, the CA would require the ability to order the withdrawal to temporarily stop when it is required. This would eliminate any delays that would occur if the CA had to ask the Director to order the withdrawal to cease. A change in Ontario Regulation 387/04 under the OWRA would be required to ensure that the CA had the proper legal foundation for their involvement in the PTTW process.

Another change required for the PTTW process is to issue permits with a shorter time frame and very strict rules on monitoring. With a shorter time frame, any changes that need to be made to the permit based on the monitoring results can be done when the PTTW is to be renewed (Gartner Lee *et al.*, 2002).

4.1.1.1 Assessing the Cumulative Impacts

The cumulative effects of PTTWs could be done by a number of different bodies including the MOE, the CA, or the proponent. Some of these agencies are a more reasonable choice than others based on their view of the outcome of the assessment. Both the MOE and the CA are government agencies with a mandate to protect the environment. They should both be able to perform an unbiased study if the resources were made available for the task. The proponent could also perform the study, but the results may be biased in their favour.

The CA could be a logical choice within the proposed framework for considering the cumulative impacts especially if they were the authority assessing the PTTW applications. When the draft DWSPA is passed, the CA will be responsible for the management of the water resources in their watershed. To do this, they have to prepare terms of reference, an assessment report, and a source protection plan. The assessment report involves the identification of any sensitive areas, WHPAs, and IPZs as well as the creation of a water budget. All of these tasks can lead to the building of a numerical model for the watershed based on their conceptual model. This numerical model could then be used to assess the cumulative impacts of any new PTTW applications on the watershed as a whole. The water resource planners in the watershed could then use the results of the model as a tool when making their decisions. Because of their activity in watershed planning, the CA would be continuously gathering data that could be added to the numerical model to ensure that it is as up to date and as accurate as possible.

The MOE could also perform the task if they had the data gathered by the CA available to them. The data would have to be available without restriction from the CA so that the MOE could build and update their model. This extra step may prove to be difficult at first, but the protocols would come into place after a period of time. It seems redundant however to build two models with the same dataset for essentially the same purpose. Since the CA must construct its model to satisfy its role in the DWSPA, is there really a need to construct the same model at the MOE for the analysis of the cumulative impacts of PTTW on the watershed. One area where it may be advantageous for the MOE to have the models from the different watersheds is where cross-boundary studies are performed. In this case, models encompassing the data from two or more watersheds would be combined to produce a larger model. The MOE may want to perform this task rather than assigning it to one of the CAs or a committee of several CAs.

The proponent conducting the cumulative impacts study may not be the best choice for the assessment because the data would have to be made available to them so that they could create a regional numerical model. The cost of such an undertaking may be prohibitive to the proponent due to the potential size of the project (Gartner Lee *et al.*, 2002). The proponent would hire an expert to create the model and this could lead to any number of different software and conceptual models being used. In order to prevent this, the government (MOE or CA) would have to establish very specific methods for the creation of the model and investigate the underlying conceptual models very carefully when looking at the results from the proponent. When the proponent submitted their results, the agency responsible for granting the PTTW application would likely want to incorporate the new data into their own model to assess the cumulative impacts. This leads to several models of the same area using the same dataset which wastes the resources of all parties involved.

The proponent could however do a desktop study of the cumulative effects of their proposed PTTW. Looking at the following items (Gartner Lee *et al.*, 2002):

- Peak taking rate vs. estimated minimum existing resource
- Average taking rate vs. estimated average existing resource
- Downstream takings that might be affected
- Other natural uses of water

From this information, the proponent should be able to determine if the proposed taking is reasonable or not. It will still be up to the CA, after modelling different situations, to determine if the proposed taking is sustainable or not.

4.1.2 How Should Cumulative Effects be Assessed?

It is very difficult to determine the cumulative effects on water quality and quantity due to a PTTW if the only data available is limited to the site-specific data required to assess the individual PTTW application. A study has to be done on a regional scale to assess if a change in water use will have negative impacts on water quality and quantity. With all of the different hydrological complexities associated with such a large area, simple analytical solutions cannot be used (Arnold *et al.*, 2003; Frind *et al.*, 2003, Gerber and Howard, 2002). It is for this reason that the assessment of cumulative effects should be done with a calibrated numerical model that has the most current and accurate information over an area large enough to obtain meaningful results. The use of numerical models allows water managers to determine where potential problems might occur for existing water withdrawals, and can help determine appropriate maximums for new water withdrawals so they do not impact other users or known contaminated sites (Vieux *et al.*, 1998).

The development of a numerical model to help in understanding and quantifying the flow in an area follows a number of steps. These can be seen in Figure 4-1 where the development of an understanding of the flow in an area begins by gathering data, then building a conceptual model, followed by the development of numerical flow and transport models (Mercurio *et al.*, 1999; Refsgaard and Henriksen, 2004; Sharpe *et al.*, 2002). This method depends on expertise in a number of areas including hydrology and geostatistics to develop the final understanding of the water processes in an area (Sharpe *et al.*, 2002).

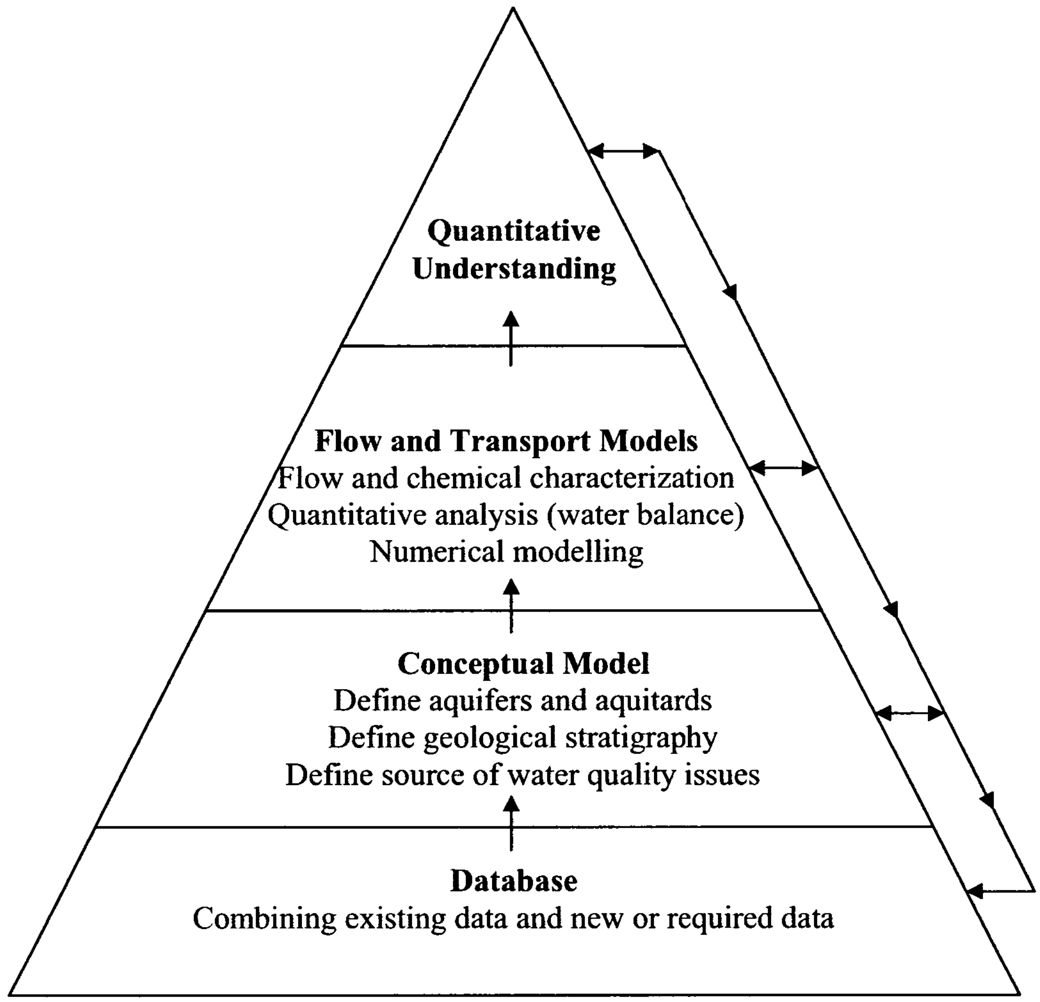


Figure 4-1: Steps required to model watersheds (Adapted from Sharpe *et al.*, 2002)

Numerical models are only as accurate as the reliability and coverage of the data that are used to create them (Beckers and Frind, 2001; Bhatt, 1993; Frind *et al.*, 2002; Vieux *et al.*, 1998). The better the data, the greater the confidence in the results obtained from a model. Uncertainties exist in the results from a model due to the precision of the input data. If the input data lack precision, the uncertainties in the predictions of the model will be greater than a model created from reliable data (Bhatt, 1993; Beckers and Frind, 2001). In the case of hydrological information, parameters can change from well to well within the same well field. At a local scale these heterogeneities should be simulated as closely as possible but at a larger scale, assumptions can be made to average out the differences. All known information should be inputted into the model to give the best representation of the real world situation (Frind *et al.*, 2002).

A point may be reached however where a model based on extensive data will give the same results as a more simple representation, but requiring greater resources necessary to create and run the complex model. Numerical models have to be created with their final use in mind. For example a regional scale model of groundwater flow does not need the same level of detail as a model used to establish the protection zone around a well.

One very difficult task that must be accomplished by the water resource planners is to establish what the acceptable change to the water table might be due to an additional PTTW. The planners can use the numerical model to evaluate different pumping scenarios by examining the changes in water levels. The difficult question to answer is how much is too much? What change can be considered sustainable? It is up to the planners in each CA to determine what is acceptable. It is only at that time that cumulative effects can be truly evaluated.

4.1.2.1 Database Management

The dataset required for establishing a reasonable numerical model for an entire watershed would obviously be very large and the dataset needed for the whole province of Ontario would be significantly larger. These data must be stored and maintained, but determining which organization is best suited for keeping the data becomes an issue. As mentioned under the proposed framework, the CAs are responsible for collecting the data in their watersheds. After compiling all of the existing data for their SPP, the CAs would be in the unique position of knowing where the gaps in the data might exist. For this reason, the CAs might be a good choice for the storage of all the data. On the other hand, the province through the MOE has an interest in the data as well and may want to have a certain amount of control and access to the data.

One solution is where each CA is responsible for the collection and upkeep of their data, but the MOE has a protocol that must be followed for the collection and storage of data to ensure consistent databases across the different watersheds. This method can be seen in Figure 4-2. Unfortunately surface water and ground water divides are rarely in the same place and as such, the boundaries for an aquifer may extend beyond the boundaries of the watershed (Gerber and Howard, 2002). If the CAs follow the data collection and storage protocols set out by the MOE, there should be no difficulty to build a model across the boundary between two watersheds without having to reconfigure one data set to match the other. This would also facilitate the eventual modelling of the water resources of the entire province of Ontario if that is required by the MOE.

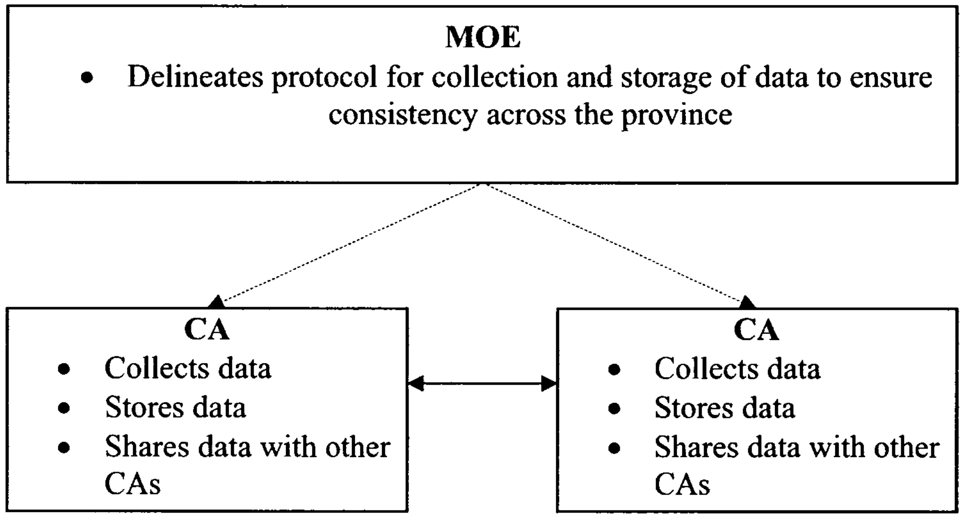


Figure 4-2: Conservation Authority data model

A better solution that fits in with the proposed framework involves the data being maintained by the MOE for the province within a central database. The CAs would update the data on a regular basis with any new information that they have obtained. This should not be confused with the numerical models that exist for each watershed. The MOE serves as a central warehouse for all of the data used for creating the numerical models, and the CAs use the data from their watersheds to develop a numerical model.

When the CA gathers new data they are stored in the central database through a website where it can be accessed by the CA that gathered it, or by a neighboring CA that requires it. Similar to the previous example, strict protocols would have to be established for the storage of data so that it can easily be updated in the MOE database. This data storage method can be seen in Figure 4-3. New data could be incorporated into the numerical model for the watershed as it becomes available to allow for continuous improvement of the planning tool.

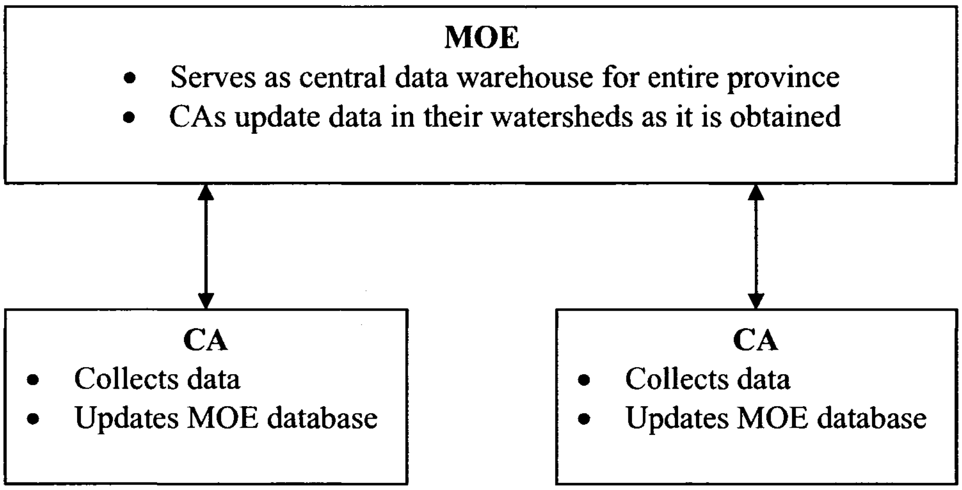


Figure 4-3: Ministry of the Environment data model

Some of the required data for assessing water quantity includes: topography, surface water levels and piezometric levels, stream and river flow rates, surficial geology, bedrock geology, hydraulic conductivity, faulting, rainfall, transpiration, evaporation, runoff, groundwater recharge areas, groundwater discharge areas, and any existing withdrawals including PTTWs and estimated water use from domestic and agricultural wells. Information on water quality including contaminants and other water quality parameters should be gathered and monitored so that it can be used in the future.

Data sources include the information gathered from any hydrogeological studies, installation of private wells, new subdivision assessments and others. CAs should be proactive in establishing monitoring wells to keep up to date with water quantity and quality in their watersheds. This may require developers to include monitoring wells in their subdivision plans, or quarry operators to drill additional wells around their property so that the CA can use them to monitor the water taking, and gather useful background information.

Although the MOE has the capacity to construct a numerical model, under the proposed framework, there may be no need as long as the different CAs have calibrated models. The MOE may be interested in a much coarser model of the entire province to get an overall picture of the water resources. Certainly this would be possible once all of the watershed scale models are completed and the data are shared with the MOE.

The watershed numerical model would be continuously updated by the CA as new information is gathered. The use of monitoring wells can show the variation in water levels and water quality over time. Effort must be made to collect these data, but it would be worthwhile in demonstrating the fluctuations in these parameters through the different seasons. This information could be introduced into the numerical model to examine transient conditions which would lead to better predictive capability. The daily actual water takings under the different PTTWs could also be used in the model to make it reflect a more realistic situation rather than just assuming an average amount for withdrawal or using the maximum allowable pumping rate.

The most important part of the data management is communication between all parties involved. All of the different federal, provincial, and municipal government agencies have to make what data they gather available to the CA or the MOE to ensure that the data are all in one place. Since the data might be coming to the MOE from different sources, it is also important for the MOE to inform individual CAs about new information that they may not be aware of.

Within the proposed framework, access to the data should be available to everyone in Ontario because they are gathered using public funds. The actual distribution of the data may be more difficult because of the potential size of the database. Perhaps the simplest way to make the data available is through a website administered by the MOE where people can download the information that they need. It seems unlikely that there will be a large demand for the information by the general public because they will most likely not be as interested in the raw data as in the models or results from the models that lead to planning decisions. Those that will be interested in the information are those conducting research, proponents who are looking at obtaining PTTWs, or their opponents.

4.1.2.2 Conceptual Model

The numerical model should be based on a conceptual model that is in three dimensions (3-D) and incorporates as much geological information as possible (Martin and Frind, 1998; Refsgaard and Henriksen, 2004). The conceptual model should incorporate any large scale heterogeneities (Frind *et al.*, 2002). One has to thoroughly understand the system before they can predict what might occur with changes to the system because the level of interconnectedness within a groundwater system determines the response to any change in a given area (Martin and Frind, 1998; Ophori, 1991). It is the conceptual model that drives the numerical model meaning that if the numerical model gives a result that is not expected according to the conceptual model, then the inputs to the numerical model should be examined to ensure that they represent the conceptual model. A good conceptual model will also help to interpret data in areas with questionable data quality or a lack of data (Sharpe *et al.*, 2002).

The history of the formation of the stratigraphy for the area as well as the erosional and other geological processes that might have occurred helps to develop a better understanding of the subsurface (Martin and Frind, 1998; Sharpe *et al.*, 2002). A better knowledge of the stratigraphy will lead to an improved understanding of the hydrogeological units in the area. From this, and data gathered in the field, a conceptual hydrogeological model can be formulated. This model should consider aquifers as well as the aquitards and any windows that might exist between the two (Gerber and Howard, 2000).

4.1.2.3 Numerical Modelling

As was discussed in an earlier section, under the proposed framework, the modelling of the water resources should be done by the CAs since they are responsible for the SPP in their watersheds. Numerical modelling can lead to a better understanding of groundwater flow (Sharpe *et al.*, 2002). The models themselves will have to consider groundwater and surface water as well as the interactions between the two because a change in one will affect the other (Beckers and Frind, 2000; Gartner Lee *et al.*, 2002; Sophocleous, 2002). This could be difficult, as the present day models are somewhat limited in this respect.

Interactions between the surface and the subsurface occur through infiltration into or out of saturated soils and through lateral flow through unsaturated soils (Sophocleous, 2002). On a large scale, the flow between surface and ground waters is controlled by the hydraulic conductivity of the soil and the difference in heads in the stream and the subsurface. Although this sounds simple, the direction of this flow can change depending on the season and will buffer the runoff in the river (Sophocleous, 2002). When a stream and an aquifer are hydraulically connected, the flow is a function of the water level in the river and the head in the aquifer. This relationship is non-linear where the flow between the stream and the aquifer will eventually reach a maximum rate when the level of the water table below the stream is roughly greater than twice the stream width (Sophocleous, 2002). When the lowering of the water table has no effect on flow from the stream, the stream and the water table can be said to be hydraulically disconnected.

For situations where pumping of groundwater exists near a water body, there are in fact two processes that lead to a decline in base flow to the stream. The first is the groundwater that is removed at the well that might have eventually been released to the stream if it wasn't intercepted. The second is the reversal of flow from the surface water body to the well because of the change in gradient due to pumping (Sophocleous, 2002).

This effect can occur at any well, but may take a significant amount of time to develop if the well is far from the surface water body.

The numerical model required to incorporate groundwater and surface water interactions must look at flow between the two, which is difficult to quantify (Sophocleous, 2002). A 3-D numerical model would be the best way to understand the process, but the stream-aquifer interface has to be described with high resolution which can be difficult in large domains with large heterogeneities (Sophocleous, 2002; Beckers and Frind, 2000). Attempts have been made in recent studies to consider these interactions. Using MODFLOW, Gerber *et al.* (2004) modelled upper reaches of streams with a 'drain' boundary condition so that if the water table surrounding the stream dropped below the elevation of the stream, it would go dry. Although a somewhat simplistic representation of what happens in reality, planners could use this information when considering PTTW applications.

One area where a large scale numerical model has been developed is the Oak Ridges Moraine (ORM) in Southern Ontario (Gerber *et al.*, 2004; Gerber and Howard, 2002; Gerber and Howard, 2000; Holysh *et al.*, 2004; Lemon *et al.*, 2004; Sharpe *et al.*, 2002). A flow model and a geological model were developed for the ORM project. The flow model can be used to assess the effects of new wells, determine if a well is sustainable, predict changes to ground water discharge areas, and delineate WHPAs (Lemon *et al.*, 2004). Similar to what is proposed for assessing PTTWs, the ORM project is focused on data with the goals of incorporating geological and field data, creating an understanding of groundwater and surface water interaction, and developing a regional scale numerical flow model that could still be used to make local planning decisions (Holysh *et al.*, 2004). There is a centralized database to collect geology, groundwater, surface water, and climate data that is linked with all of the partners so that it is always up to date (Holysh *et al.*, 2004). One interesting aspect that the project included was the combination of expert knowledge with geostatistical methods. The experts were able to interpret data in ways that geostatistical techniques cannot handle (Holysh *et al.*, 2004; Sharpe *et al.*, 2002). The success of the ORM project indicates that with enough care, all of the watersheds in Ontario can have a similar resource for making water resource planning decisions.

By far the most difficult task in using the numerical model as a tool for water resource planning is determining what is sustainable in the watershed. The model may be the most accurate model possible, but decisions have to be made as to what is acceptable in terms of drawdowns or changes in surface water levels. Besides the water quantity, how are water quality issues going to be evaluated? A first very necessary step is establishing a baseline for water quality and quantity at any given point. Decisions must be made based on factors set out in the SPP to determine if there is an adverse effect from the PTTW. Each CA will have to determine their levels of sustainability as they may be different in every case.

4.1.3 When Should Cumulative Effects be Considered?

The simple answer to this question is that cumulative effects should always be considered when reviewing PTTW applications. In the Permit to Take Water (PTTW) Manual (MOE, 2005), the current situation for assessing cumulative impacts is somewhat vague as to when the Director of one of the regions of Ontario will require the impacts to be studied. It is also confusing as to who is responsible, the MOE or the proponent, for conducting the assessment.

Under the proposed framework for assessing PTTWs, the CAs should investigate the proposed taking within their model as accurately as possible. This will ensure that the cumulative effects of the proposed taking with existing takings will be considered. If the effects are considered sustainable by the CA and they fit within the SPP, then the CA should recommend to the MOE that the application be granted. If the effects are not sustainable, then the CA should recommend that the permit not be given. In some cases, conditions might be placed on the permit to render it more sustainable within the watershed. These conditions might include coordinating water use with other PTTW holders in times of potential water shortage.

One exception under the proposed method, to the assessment of all PTTW applications for cumulative impacts would be the very short term, Category 2 PTTWs used for dewatering construction sites. These permits, unless they are for large volumes of water, will not have a lasting impact on the watershed as a whole. Monitoring should continue to be a condition of this type of PTTW and all others, to ensure that there are no shorter term impacts on neighboring water users or natural systems.

4.1.4 Funding for Cumulative Impact Studies

The assessment of the cumulative impacts of water withdrawals will cost money in terms of human resources because all of the CAs must have sufficient resources to perform the proposed responsibilities. The added cost may be borne by the proponent or the MOE or by a combination of the two. It would seem that the proponent should be the one to pay for this assessment because they are the ones who eventually gain from having their PTTW approved. The province of Ontario also gains from assessing the cumulative impacts because the sustainability of the water resources in the watershed is investigated more thoroughly. This increased knowledge leads to increased source water protection in the province.

The cost would be easiest to implement with a raised price for the application for a PTTW. A proponent who wants to have a permit would most likely be able to bear the increased cost of the permit. It would also ensure that the proponent was very serious about obtaining a PTTW rather than just investigating if such a permit could be obtained. If the funding was to come from the MOE, it would have to be through increased funding to the CA. The public at large may not see the advantage of the cumulative assessment and how they might benefit, and as such, might object to a small increase in taxes. On the other hand, by making the proponent pay, the cost will be passed on to the general public in increased cost for the products produced by the applicant.

4.2 Specific Technical Issues for Modelling Cumulative Impacts from PTTWs

4.2.1 How Will the Numerical Model be Used?

Could the same model be used for both watershed planning decisions and for assessing PTTWs? In most cases, it is reasonable to assume that this would be the case. Since the assessment of PTTW applications is directly related to water resources planning, the same tools and information should be used in both decision making processes. They cannot be independent of each other. The resources required to maintain two numerical models representing the same area would be large and unnecessary. Equal access to the use of the model should be given to all decision makers.

One area where differences may be required is for the scale of the model. Planners may need to have a larger picture of all the resources in a watershed whereas the effects of a PTTW might be more localized to a smaller region within the watershed. This can be solved with a regional scale model having a small enough discretization to show localized impacts. This has successfully been accomplished in the ORM study (Holysh *et al.*, 2004) with a 100m discretization.

4.2.2 Scale of Numerical Model

The scale chosen for modelling the cumulative impacts is an important factor and would depend on the level of detail required for making decisions. On the whole, a well calibrated regional scale model should be sufficient to see any potential conflicts that might arise due to the PTTW (Gartner Lee *et al.*, 2002). The geographical area that the model initially covers may not be the entire watershed due to a lack of population in some areas or lack of information. A goal should be to eventually include the entire watershed in the model.

The domain of the model has to be such that the impacts due to the withdrawal of water will not reach out as far as the boundaries of the model. This might require a model for one watershed to include information from the adjacent watershed. The data for implementing this type of scenario has to be shared between the two responsible CAs. Good communication between the two CAs with data and information sharing would ensure that this was not a problem.

A regional scale model makes a number of assumptions that removes details from the simulation (Arnold *et al.*, 2003). A more local scale model might be used for assessing specific problems, but a higher level of detail in the data might be required compared to a watershed scale model so that the more complex local scale differences can be observed.

The scale selected can sometimes be difficult to defend especially when considering fractured bedrock aquifers. The assumption that fractured rock can be considered as an equivalent porous medium may be challenged (Bradbury and Muldoon, 1994). This is especially true at a smaller scale and if there is a low density of fractures in the rock because flow in this situation is hard to predict since flow can occur in a single fracture and it is hard to know the length and direction of that fracture.

Flow in bedrock with high fracture density is much easier to predict because flow will occur within the dense fracture network. This pattern of flow will become even more evident at larger scales where it can be more reasonably modelled as an equivalent porous medium with an anisotropic hydraulic conductivity (Bradbury and Muldoon, 1994; Rayne *et al.*, 2001). However, porous media approximations tend to underestimate flow through fractured bedrock (Bradbury and Muldoon, 1994). This increases the level of uncertainty associated with the results of numerical modelling and should be taken into consideration when making water resource planning decisions.

4.2.3 Transient vs. Steady State

One very important aspect considered by the proposed method for modelling the impacts due to several PTTWs is modelling either steady state or transient conditions. Since water withdrawals are rarely steady, the transient simulation would be a better representation. The transient simulation requires more data than the steady state model and as such is more difficult to implement, but gives more accurate results of changes over time as opposed to an average condition.

In some cases, long term average flow conditions can be used in modelling aquifers because the transient, seasonal fluctuations tend to even out over a long period of time, especially on a regional aquifer scale (Frind *et al.*, 2002). On the other hand, the effects of variations in seasonal pumping may not even out over time. This implies that modelling a water withdrawal as a steady state problem would not be accurate over time. In a comparison of steady state and transient methods using a numerical model for the delineation of a WHPA, Ramanarayanan *et al.*, (1995) found that the capture zones determined by a transient analysis were larger than those of a steady state simulation. The reasons for this were the higher drawdowns from the transient situation during pumping. The same amount of water was removed from the system in both cases, but the pump rates were higher over a shorter period of time compared to the steady state situation. This was compounded by the fact that, in the case studied, the recharge did not have enough time to re-establish the water table before the pumping began the following season. It is important to realize that a thorough understanding of both temporal and spatial flow must exist in order to properly address water resource management decisions (Gerber *et al.*, 2004).

4.3 Aggregate Quarry Operations

The effects of aggregate quarries on the watershed take several forms. If the quarry is below the water table, dewatering it will cause a lowering of the water table (Johnson, 2004; Ostrander *et al.*, 1998). This can cause changes to both river base flows and water levels in wetlands (Ostrander *et al.*, 1998). Since the water is typically returned to the surface, there can be water quality problems and temperature problems in the runoff (Johnson, 2004). The volume of water that needs to be removed from a quarry depends on the surrounding level of the water table, the depth of the quarry, the geology surrounding the quarry, the hydraulic conductivity, and other hydrogeologic conditions (Arnold *et al.*, 2003; Johnson, 2004). The equations developed for well drawdowns cannot be used for large excavations, so drawdowns and required pumping rates are best determined by 2-D and 3-D numerical models (Johnson, 2004). Analytical solutions cannot model either heterogeneous or transient situations (Arnold *et al.*, 2003).

Aggregate quarries are not always bad news for the local water resources. In a study in Nevada, Huzar *et al.* (2001) determined that the water that was being pumped to surface was having a benefit to the local rivers. In Nevada, water is a very serious issue in times of drought. It was estimated that the water being pumped to the surface was worth almost $864 million a year at the current market prices (2001) and could be used for municipalities or irrigation. It was also found that the pumping of the water to the rivers was mitigating the volatility of the water levels, which was seen as a benefit to water users downstream of the quarries. It was also found that there was an economic benefit from the increased flows from recreational use. These beneficial effects will not last for ever due to several reasons. The most obvious reason is that the quarries will only operate when they are profitable and will eventually close. When they close it is more than likely that the dewatering will stop and the pit will be allowed to fill with groundwater instead of pumping to surface. If water resource managers find that the economic benefits of continuing to pump the groundwater to surface offsets the cost of pumping the water, then dewatering can continue. This practice will cause the flows in the rivers to decrease over time due to a slow lowering of the water table, even though the flows are relatively constant throughout the year.

The first half of Figure 4-4 illustrates the steps that an aggregate quarry operator would take when considering the dewatering of their proposed quarry. It is also what the MOE currently uses to consider the PTTW applications. The first step the proponent takes is to establish if the quarry will extend below the water table and as such will any dewatering be necessary. The second step is to determine if the hydrogeological conditions are well known or if further investigations such as pump testing might be required. The operator then has to estimate the effects of the quarry due to dewatering. Initial calculations based on water balances and pump tests should be done at this stage to give an estimate on the quantity of water that will have to be removed from the quarry and the extent of the areal impacts on the water table.

The second half of Figure 4-4 illustrates the proposed cumulative impacts study that will be undertaken by the CA for each new application for quarry dewatering. Essentially, the CA determines if the proposed taking is sustainable within the SPP for the watershed. They do this through the use of their 3-D numerical model developed for the watershed.

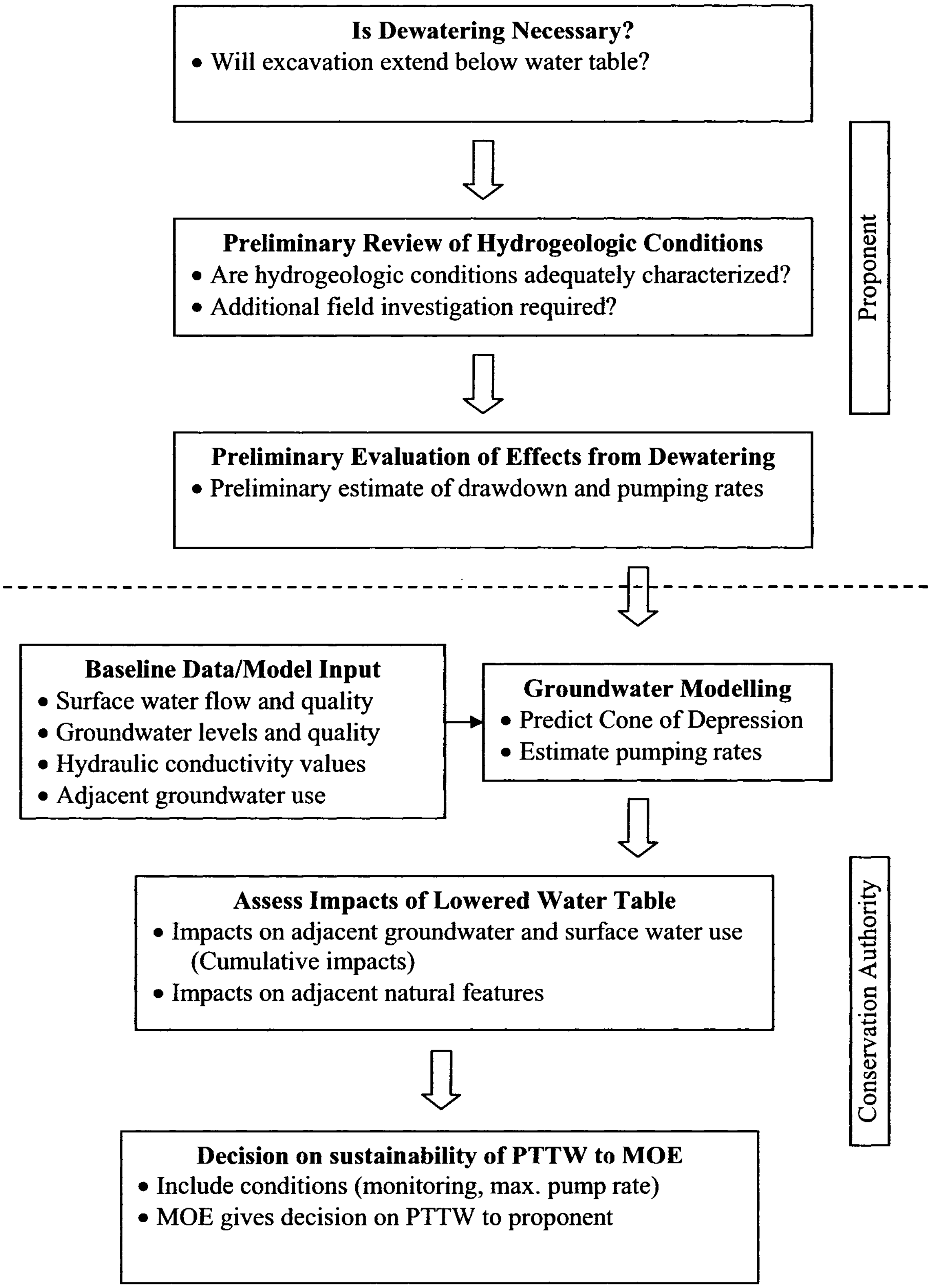


Figure 4-4: Dewatering decision tree (Adapted from Johnson, 2004)

Steady state models will give a worst case scenario result for the effects of the PTTW on the water resources. In the case of aggregate quarries, different pumping rates will lead to very different results. For example if a steady state simulation is done with the maximum allowable dewatering rates, there will be a very large impact on the water table causing shifts in hydraulic gradients (Gerber *et al.*, 2004). The same effect might be seen with an average pumping rate but to a much lesser extent.

The use of a transient simulation for aggregate quarry operations is much more reasonable due to the way that they operate. Currently, the maximum pumping rate that the quarry is allowed via their PTTW is for a volume based on the final state of the quarry. The quarry obviously does not reach its maximum depth quickly. It can take decades before the quarry is at its maximum depth. This should be reflected in the modelling for assessment of the cumulative impacts of the quarry operation with other PTTW in the area. The effects of a quarry development can be modelled by slowly lowering the bottom of the quarry and increasing the size of the quarry over time.

Another issue that should be considered is the actual amount of water that would be pumped out of the quarry. The rate of dewatering is typically not steady over time for aggregate quarries. In Ontario, a significant number of aggregate quarries do not operate in the winter months and the quarry is allowed to fill with groundwater. Come spring, when operations in the quarry are to begin, the water is pumped out in order to allow access to the working surface. This pumping is done relatively quickly with large flow rates. The large maximum allowed pump rates that are given for PTTW in aggregate quarries are to accommodate the initial emptying of the working surface at this time. Once the quarry is empty, a relatively smaller pump rate is required to dewater the excavation.

Monitoring is required to determine the actual pump rates on a daily basis for the aggregate quarry (Gartner Lee *et al.*, 2002). The new conditions on the PTTW ensure that this information is reported on an annual basis. This information can be used in a transient simulation to model the effects of an existing aggregate quarry more realistically and to assess the impacts with other existing PTTW holders. In the case of a PTTW application for an aggregate quarry, very little information is known about the actual pumping rates that will be required to keep the quarry dewatered when it is operational. A numerical model may be used to obtain a realistic value for the pumping rates required. From this, the PTTW application can be either accepted with conditions for monitoring water levels and flows or denied due to non sustainable impacts from the quarry.

The use of a transient simulation will allow water resource planners to see more realistic effects from aggregate quarry dewatering based on the time of year. Once the use of a transient model has been decided upon, one has to determine the most appropriate time step. A yearly average is not enough because of the widely ranging pump rates. More reasonable time frames might be monthly, weekly, or daily as data resources and processing time allows.

Chapter 5: MODFLOW

5.1 Introduction

In 1984, the United States Geological Survey (USGS) released a new three dimensional finite difference model to simulate groundwater flow called MODFLOW. The original program was updated and improved in 1988. Similar to the original MODFLOW, it was a program that could easily be modified, used and maintained (McDonald and Harbaugh, 1988). Since computing power was not what it is today, the program had to be efficient with both computer memory and running time. It worked on the principles of 'modules' that could be pulled into the main program if they were required. The model included modules on wells, recharge, rivers, drains, general head boundaries, and evapotranspiration (McDonald and Harbaugh, 1988).

Since 1988, MODFLOW has gone through a number of versions with a more recent version released as MODFLOW-2000 (Harbaugh *et al.*, 2000). The new version has added modules, solvers and also allowed users to model contaminant transport.

MODFLOW is a finite difference model that models 3-D flow through saturated porous media. It is used very frequently around the world. The program itself is very simply constructed but the input and output files are FORTRAN formatted ASCII or text files. The development of the groundwater flow equation which is used by MODFLOW can be seen in Appendix C and further details on the assumptions and equations used in MODFLOW can be seen in Appendix D.

Many software development companies have created pre and post processors that allow users to build the data files and see the output in a more user friendly environment. One such software package is Visual MODFLOW by Waterloo Hydrogeologic Inc. (WHI). For this project, version 3.1.0.86 of Visual MODFLOW was used with MODFLOW-2000.

5.2 Boundary Conditions available in MODFLOW

Of the many boundary conditions available in MODFLOW, three can be used to simulate the dewatering of a quarry. A brief description of each follows.

5.2.1 Constant Head

Constant head (CH) cells are assigned a head that will always remain constant (McDonald and Harbaugh, 1988) and they serve as an infinite source or sink of water to the model domain. The direction of flow into or out of the CH cell depends on the heads of the cells around it. CH cells can be used to define boundaries of constant head such as lakes or rivers or equipotential lines.

5.2.2 Wells

MODFLOW can model a well as a flow of water either into or out of a model cell. The flow is assumed to be constant over a specified time step and is independent of both the head and the area of the cell in which the well is located. The model represents the well as a flow either into or out of the cell. The continuity equation can then be solved for the cell with the effects of the well included (McDonald and Harbaugh, 1988).

5.2.3 Drains

The drain boundary condition in MODFLOW is very different from a well but is similar to a CH boundary. The flow out of a drain is limited by the head of the water in the cell containing the drain, as well as the area of the cell. A drain works as long as the head in the aquifer (or cell) is above the elevation of the drain. If the head in the cell falls below the elevation of the drain, no water will leave through the drain (McDonald and Harbaugh, 1988).

The drain is always assumed to be half full so that the elevation specified for the drain actually represents a point in the centre of the drain. The head losses around a typical drain, due to the different hydraulic conductivities of the surrounding materials or flow through small orifices into the drain, are simulated by a conductance term that is typically adjusted when calibrating the model to represent real world situations. The flow into the drain can thus be represented by the following if the head in the cell is greater than the elevation of the drain.

$$Q_{i,j,k} = CD_{i,j,k}\left(h_{i,j,k} - d_{i,j,k}\right) \quad (5\text{-}1)$$

Where $CD_{i,j,k}$ is the conductance term associated with the drain [L^2/t], $h_{i,j,k}$ is the head in the cell [L]and $d_{i,j,k}$ is the elevation of the drain [L]. If the head in the cell is below the drain elevation, the flow will be equal to zero (McDonald and Harbaugh, 1988).

5.3 Modelling Aggregate Quarries in Visual MODFLOW

Of particular interest to this research is the modelling of aggregate quarries using Visual MODFLOW to assess cumulative impacts. The aggregate quarry is essentially a hole in the ground that is often drained in order to remove the aggregate. The flow of water into the quarry comes from groundwater seeping in as well as from precipitation and surface flow that falls into it. The site grading plan usually diverts surface flow away from the quarry and hence the groundwater flow and the precipitation need to be removed by dewatering. There are several methods that can be used within Visual MODFLOW to estimate the flow into an operational quarry. The methods include the use of a well or a well field to simulate the water withdrawal and drawdown due to dewatering at a quarry, a constant head boundary could be set at the level of the quarry floor or a drain whose elevation represents the quarry floor could also be used to simulate the dewatering at a quarry. The use of the drain boundary has been used to represent an aggregate quarry in several studies (Arnold *et al.*, 2003; S.S. Papadopulos & Associates, 2004).

For the purposes of demonstrating the differences between the different methods for modelling aggregate quarries, a simple scenario was created in Visual MODFLOW. Figure 5-1 shows the model domain that was established as a rectangular block (2000m by 2000m by 40m). The domain was divided up into 4 layers, 10m thick and a 10m by 10m grid was established over the domain so that each cell was an identical 10m by 10m by 10m grid block size.

A recharge value of 100 mm/year was established over the entire area. The hydraulic conductivity (K) of each cell block was assumed to be isotropic and equal to 1×10^{-5} m/s. A constant head boundary of 40.0 m was assigned to all layers on two opposite edges as can be seen in Figure 5-1. The remaining two edges of the model and the lower, or bottom boundary, were set as no flow boundaries.

The quarry was modelled as a single cell situated in the centre of the domain 1000m from each edge. It was placed in either the top or the second layer from the surface depending on the simulation so that the floor of the quarry could be assigned an elevation between 20m and 40m.

All of the simulations were conducted at steady state to examine the differences between the model simulations for the different boundary conditions used to simulate the quarry.

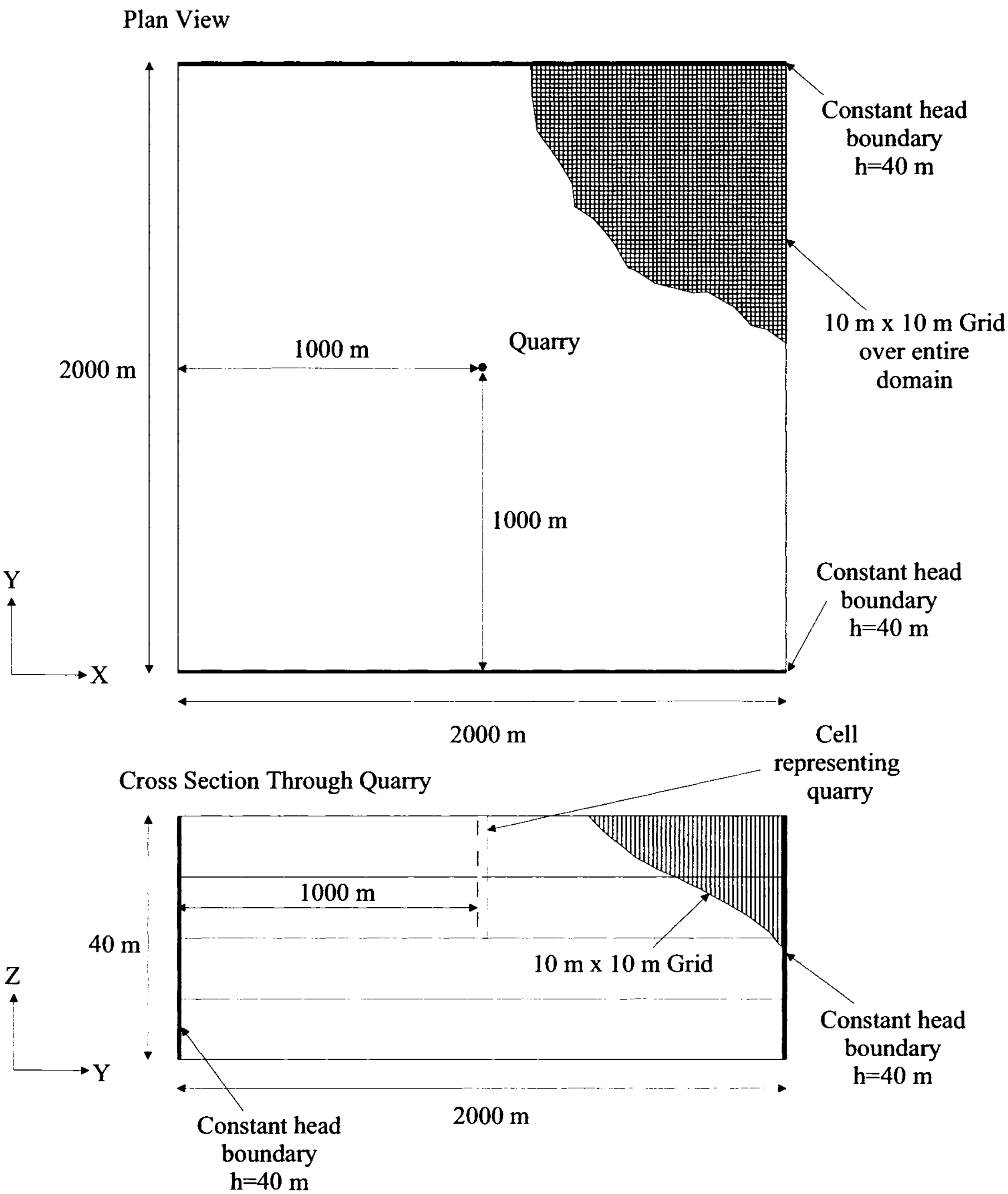


Figure 5-1: Quarry simulation domain

5.3.1 Simulation with Wells

There are two inputs that are required when entering information about a well. They are length and depth of the screened portion of the well, and the pumping rate from the well. As discussed in a previous section, the effect of a pumping well is simulated by removing the specified flow rate from the cell over which the well is screened. The same amount of water will be removed from the cell no matter where the screened portion of the well lies within the cell. More simply stated the flow rate from the well is independent of the elevation of the screened portion of the well.

When simulating a quarry with a well one has to lower the water table in the area of the quarry to be equal to or slightly below the quarry floor. The flow rate from the well will have to be established so that the head in the cell with the well is just slightly below the bottom of the quarry. The adjustment of the flow rate would have to be done on a trial and error basis until the appropriate head is obtained.

One difficulty with this approach is the potential of causing dry cells. Since MODFLOW is a saturated flow code, it cannot be used for unsaturated conditions. When running, the model essentially turns any given cell off if the head falls below the bottom of that cell. Once the cell is off, it remains off for the remainder of the simulation. To overcome this problem, MODFLOW-2000 has a rewetting package that can be used but model convergence becomes an issue when it is used. In the case of the quarry being represented as a well, if the cell containing the well was to run dry during the simulation results could become very unreliable.

Since quarries typically take a large amount of surface area, several wells would have to be used to simulate the entire quarry. This can prove to be very difficult in determining the best place for the wells as well as the pump rates necessary to keep the quarry drained.

Although the use of wells to simulate quarries is possible, there may be other boundary conditions that would be more suitable for the task. Boundary conditions, where the maximum elevation of the head in the cell is assigned are ideal for simulating a quarry because this elevation could be assigned to represent the bottom of the quarry. With this, the cell representing the quarry would always have the proper water level.

5.3.2 Simulation with Constant Head Boundaries

A second method for modelling the dewatering of a quarry is the use of constant head boundaries. For this simulation, the head of the Constant Head (CH) boundary is equivalent to the elevation of the base of the quarry. Care must be taken using CH boundaries in this way to ensure that the head in the surrounding cells is above the head in the cell representing the quarry. If the heads in the surrounding cells are below that of the quarry, the model would simulate flow from the quarry to the surrounding cells. This of course cannot occur in reality. Another factor that must be considered to allow the CH boundary to be used is the hydraulic conductivity (K) of the CH cell. The higher the K value, the more easily water will flow into the CH cell. In the case of a quarry, the water is flowing into the quarry so a large value for K should be used to simulate unrestricted flow in the quarry.

Several scenarios were run with the CH cell representing the quarry. The first scenario was simulated with the CH located in the top layer. The head assigned to the CH cell was set at three different elevations (31m, 35m, and 39m) to represent three different depths of quarry. The second scenario was the same as the first but the quarry was located in the second layer from the top and the head was changed from 21m to 29m. For both scenarios, the hydraulic conductivities were varied from $1x10^{-5}$ m/s to 1 m/s. The different flows from the CH cell for the different scenarios can be seen in Figure 5-2 and 5-3. The complete data can be seen in Appendix E.

From these results a number of observations can be made. The flow from the quarry levels off with increasing K within the CH cell. This is explained by the rate of flow to the CH boundary switching from being controlled by the K in the CH cell to the K of the formation around it. When simulating a quarry with a CH boundary, this maximum flow should be modelled with a very high K in the cell to obtain the maximum rate of flow to the quarry.

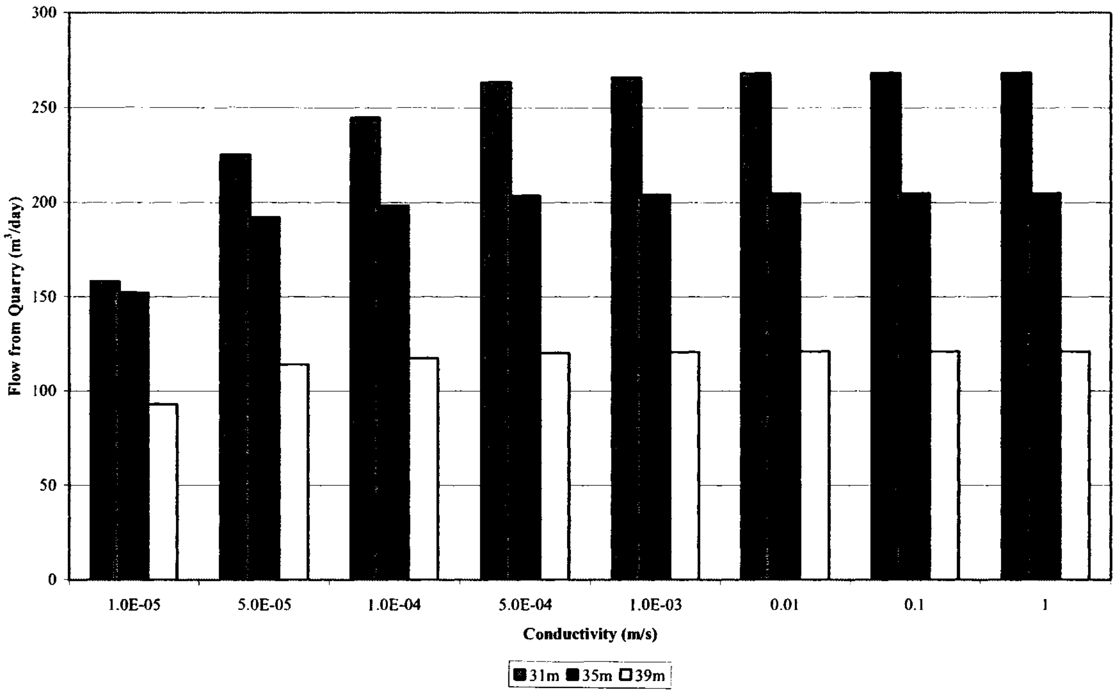


Figure 5-2: Flow from constant head cell in layer 1 with changing elevation and hydraulic conductivity

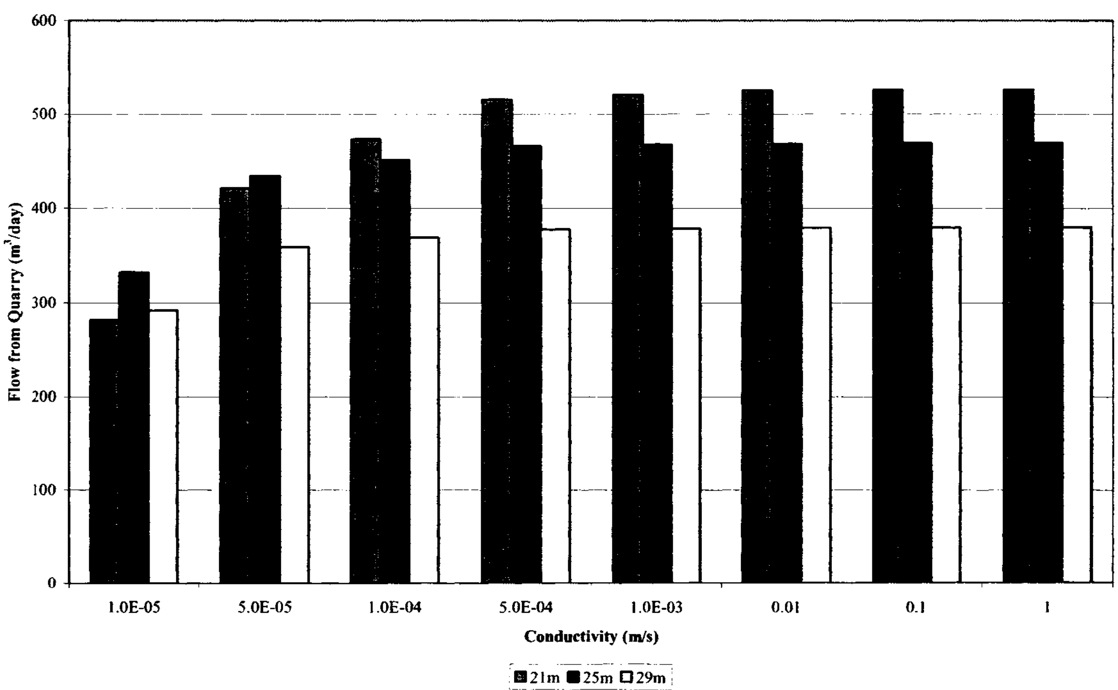


Figure 5-3: Flow from constant head cell in layer 2 with changing elevation and hydraulic conductivity

The other item of interest is the relative flows at each value of K. Conceptually, there should be higher flows into a CH cell at a lower elevation than a higher elevation due to the increased gradient. This is certainly evident when one compares the overall flows in Figure 5-2 and 5-3. Likewise as illustrated in Figure 5-2, the largest flows for each conductivity value occur when the CH boundary is at its lowest elevation. However, Figure 5-3 indicates that at lower K values, the flow from the quarry at the low elevation (21m) is less than that at the higher elevation (25m). What is happening at the lower K values is surprising and was investigated in more detail however an explanation for the phenomenon was not found.

In order to investigate this issue, the mass balance for all water entering and exiting the model domain was studied for all of the scenarios to ensure that it was reasonable. Once it was ensured that the mass balance for the entire domain was adequate, the flows through each face of the cell containing the CH boundary were examined to determine where the potential problem might be occurring. No inconsistencies were observed. Finally, the cells surrounding the quarry were looked at to ensure that they were not being deactivated through the solver iteration process. If the cells were deactivating, the flow to the quarry would be limited. It was found that all the cells were behaving as expected.

This potential problem should be investigated further by perhaps increasing the discretization surrounding the quarry. If one uses CH cells to simulate a quarry, care should be taken to model at higher K values so that the effects from this observation will not influence the results.

A problem with CH boundaries is that recharge is not applied to them. This can lead to an underestimation of the water flowing into the quarry. In modelling larger quarries with CH boundaries in multiple adjacent cells, flow can occur between the CH cells (i.e. into one cell and out of an adjacent cell), causing mass balance problems. This can be prevented by setting the cells around the edge of the quarry as constant head cells, and deactivating the cells inside the quarry. This eliminates the 'short circuiting' of flow, but it is not a good representation of the quarry because it eliminates both recharge from the surface and flow from the bottom of the quarry.

5.3.3 Simulation with Drains

A third way of representing the quarry is through the use of the drain boundary condition in MODFLOW. As stated earlier, drains are similar to constant head cells in that they are a sink for water if the head in the cell is higher than the assigned level of the drain. If the head in the surrounding cell is lower than the elevation of the drain, then there is no flow to the drain. The flow into a drain cell should be equal to the flow to a constant head cell if the conductance assigned to the drain is very high.

The drain appears to be an effective way of simulating a quarry because the elevation of the base of the quarry can be assigned to the cell, there is still recharge to the cell and the flow will always be into the drain. Since the drain module was originally created to simulate an agricultural drain, assumptions must be made to have it behave like a quarry. The conductance of the drain is used to account for the head losses that occur in the agricultural drain from the type of material directly surrounding the drain and the size of the orifices of the drain. To ensure that the drain operates freely, the conductance term assigned to the drain should be high so that the water is essentially free flowing into the drain.

Similar scenarios used to evaluate the CH approach to simulating a quarry were used to evaluate a drain representing the quarry. The elevation of the drain was varied from 31m to 39m on layer 1, and from 21m to 29m on layer 2 to simulate different quarry floor elevations. The conductance term was varied from very low (5 m^2/day) to very high values (100,000 m^2/day). An overview of the results can be seen in Figures 5-4 and 5-5, and the complete results can be seen in Appendix E.

One observation that can be seen from these figures is that the flow from the drain reaches a steady value at a conductance of approximately 1,000 m^2/d. This is the point where the flow to the drain is controlled by the surrounding formation rather than the drain itself. It is at this or larger conductances that the drain should be operated in order to simulate a quarry. Another observation is that the flow from a deeper drain is larger than that of a higher drain. This is true in all cases if the drain cell is in layer 1 (Figure 5-4), but does not always apply if the drain is in the second layer (Figure 5-5). It is only valid until a conductance value of 100 m^2/d. At that point, the flow out of the drain for the lowest elevation (21m) reaches a maximum and then decreases to a constant value with increasing conductance. This pattern is similar when the drain cell on the top layer

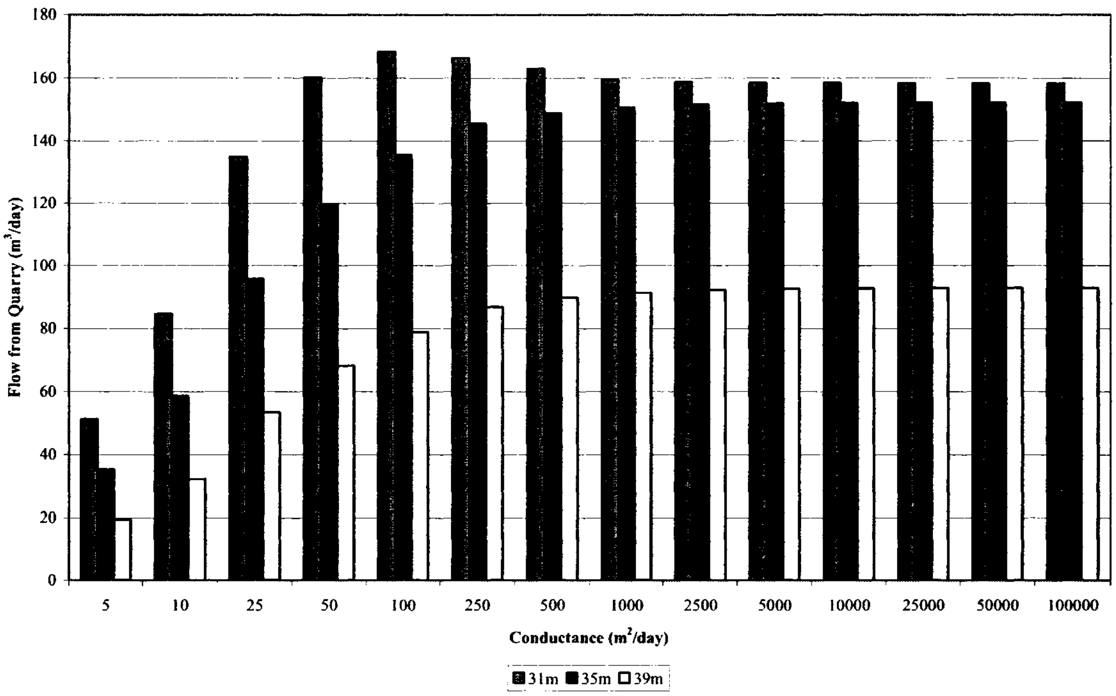


Figure 5-4: Flow from drain cell in layer 1 with changing elevation and conductance

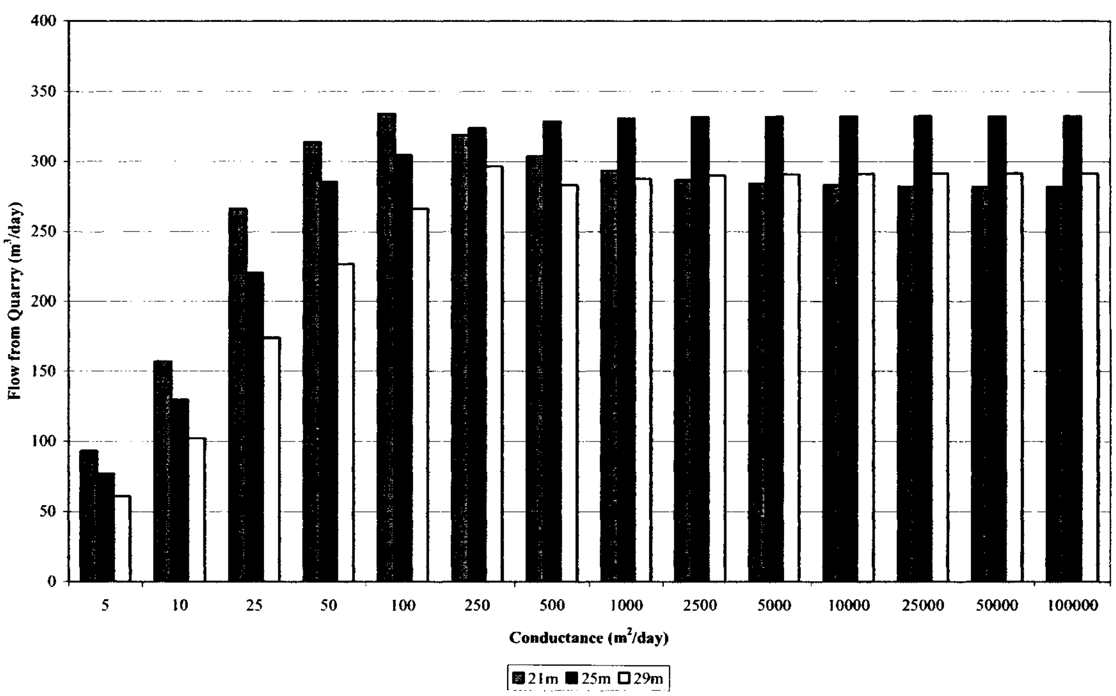


Figure 5-5: Flow from drain cell in layer 2 with changing elevation and conductance

is at its lowest elevation, but the flow does not decrease enough to fall below those flows from higher elevations.

The same pattern does not appear at any of the higher elevations within the layers, only for the lowest elevation. The reason for this pattern is not immediately obvious and though it was examined in the same way as for the CH simulations, an explanation could not be found. It is an issue that needs further research. Until that time, as long as the drain is not assigned to the lower elevations of a cell, it can reliably be used as a drain.

The flows from the drain at high conductance are very similar to those from the CH cells when the hydraulic conductivity is $1x10^{-5}$ m/s. In layer 1 the flows are approximately 160, 150, and 90 m^3/day for elevations at 31, 35, and 39m for both boundary condition types (Figures 5-2 and 5-4). The same appears in layer 2 where the flows from the quarry are 280, 330, and 290 m^3/day when the head is assigned to 21, 25, and 29m for the boundary condition (Figures 5-3 and 5-5). This is to be expected because the two boundary conditions, with their respective definitions and inputs, are representing the quarry in the same way.

The drain package can be used to simulate a quarry because in effect the quarry acts as a large drain. Water flows in from the surrounding rock and is pumped out in order to keep the working surface dry. Drains also avoid the concern raised earlier with respect to multiple adjacent CH boundary conditions. Since flow cannot occur out of a drain, the scenario of flow into and out of adjacent CH boundary conditions is avoided.

Chapter 6: Case Study

6.1 Background

North West Goulbourn, a region to the west of Ottawa, is an area in which there are several small rural subdivisions, two golf courses, and recently, a large number of requests for pumping permits to extract the local aggregate. A total of five aggregate quarries exist or are currently going through the application process to open within an area of 12.5 km^2. Each of the quarries has applied for, or already holds a PTTW in order to dewater their quarry so they can remove the aggregate. The residents of the area are concerned that the quantity of water available to them in their wells will be reduced by the dewatering. They also fear that with the large volume of water being removed from the aquifer, groundwater gradients may shift causing water of lesser quality to enter into the area and subsequently into their wells. The area was used as a case study to show the potential of cumulative impacts due to aggregate quarry operations on the local groundwater resources. This was done using a numerical model based on a conceptual model built on the local geology and governing principles of groundwater flow and contaminant transport.

Several numerical models were developed. A regional model was constructed based on the conceptual model developed from the hydrogeological information gathered. From this model, two local scale models were developed to simulate pump tests at a much finer discretization. These pump tests were used to improve the confidence of the calibration of the regional model. The regional model was then used to simulate transient effects of dewatering the quarries on the regional aquifers.

The regional model was then improved by simulating wetlands in the area with a drain boundary condition in order to better represent the measured piezometric surface, and to better quantify the effects of the quarry dewatering on the wetlands in the area. The local scale models were used as calibration for the updated regional model as well, but due to convergence errors, the updated model could not simulate the transient quarry dewatering on the regional scale. Further work on the regional scale model with wetlands simulated as drains is required for the numerical model to converge to a solution.

6.2 Regional Model

The chosen study area is an 800 km^2 rectangle roughly centred on the five quarry operations. Figure 6-1 shows the general location and outline of the area covered by the case study. The area stretches 32 km in the east-west direction and 25 km in the north-south direction covering parts of Lanark County to the west, and the City of Ottawa to the east. The communities of Kanata, Richmond, Stittsville, Carp, Carleton Place, Almonte, and portions of the City of Ottawa also fall within the borders.

The main bodies of water in the area are the Mississippi River to the west, the Jock River

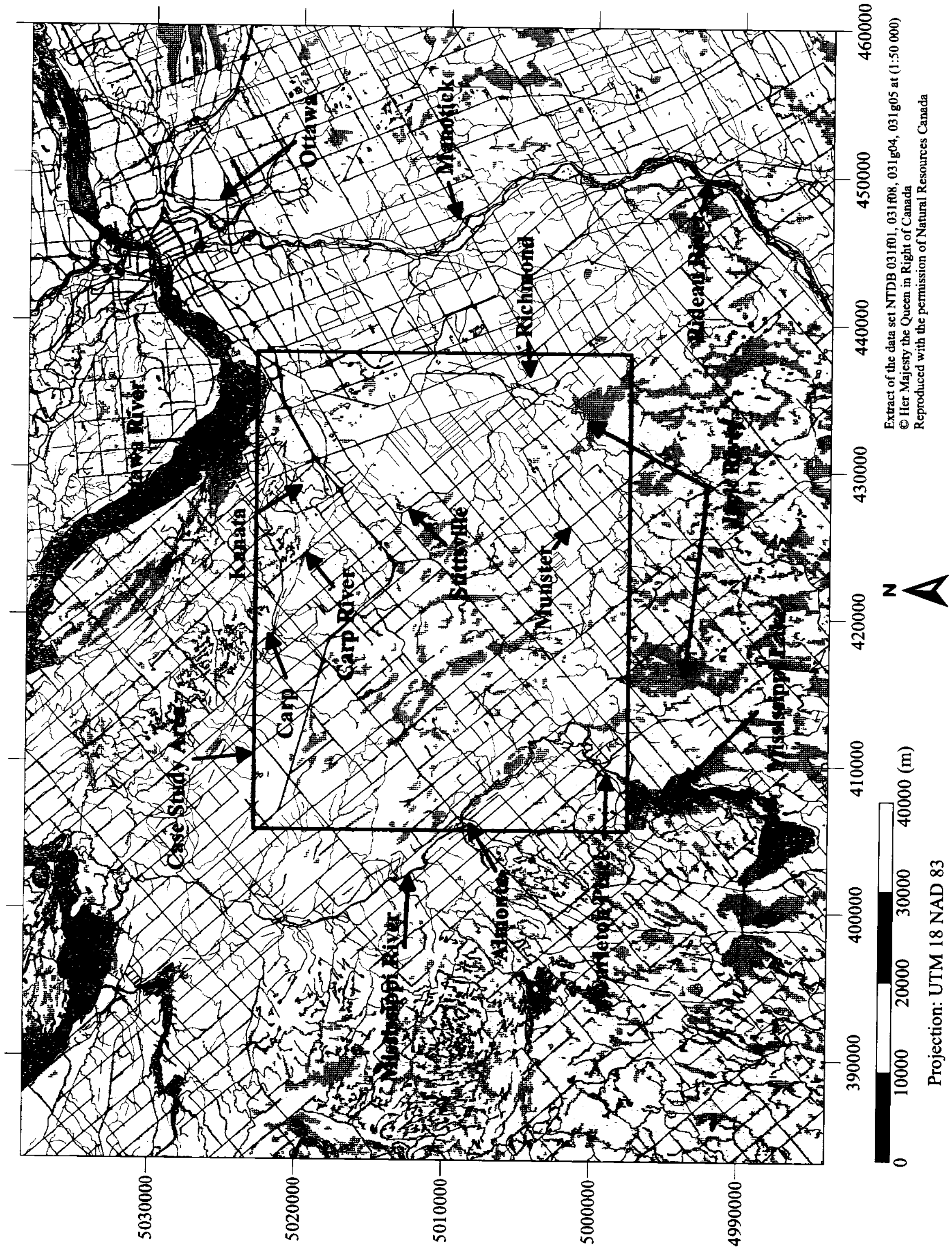


Figure 6-1: Case study area

in the south, and a very small portion of the Ottawa River is located in the north-east corner of the area. There are many other streams, small lakes, and wetlands within the boundaries.

The topography of the area ranges from 160 meters above sea level (masl) to 50 masl as can be seen in Figure 6-2. There is an elevated portion in the central part of the area with a decrease in elevation in all directions towards the different rivers. There is a major fault that runs north-west/south-east through Kanata along the Carp River. Figure 6-2 shows that there is a sudden gain of elevation to the east side of the fault.

6.2.1 Geology of Area

6.2.1.1 Bedrock Geology

There have been a number of geological processes that have led to the landscape that is currently found in the case study area. The deepest element in the area is the Precambrian rocks that underlie the entire region. Above these are younger Paleozoic rocks that vary in thickness throughout the region. The youngest sediments are found in the overburden which also varies in thickness throughout the area. The different bedrock geology can be seen in Figure 6-3 and is summarized in Table 6-1.

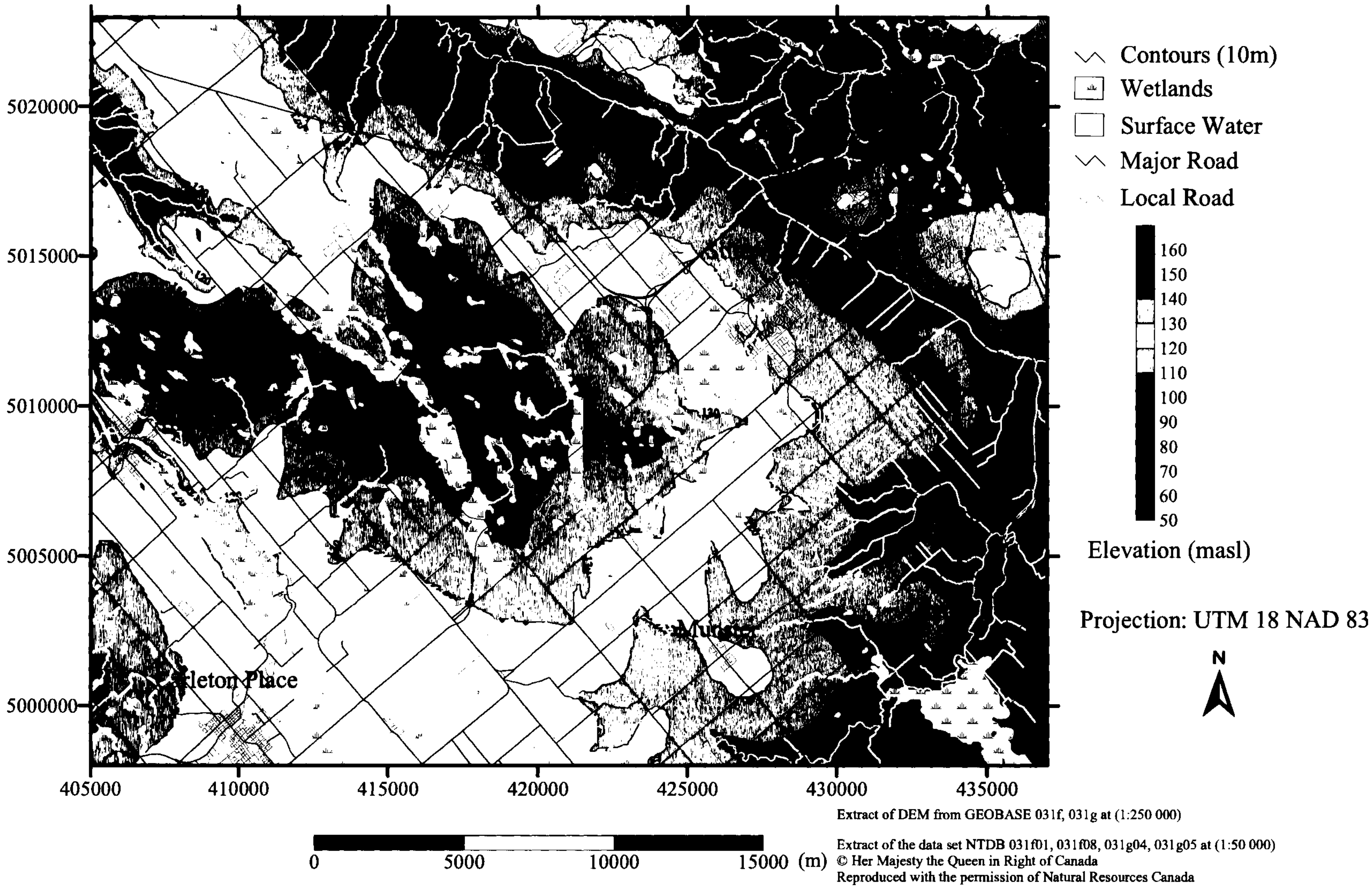


Figure 6-2: Topography for case study area

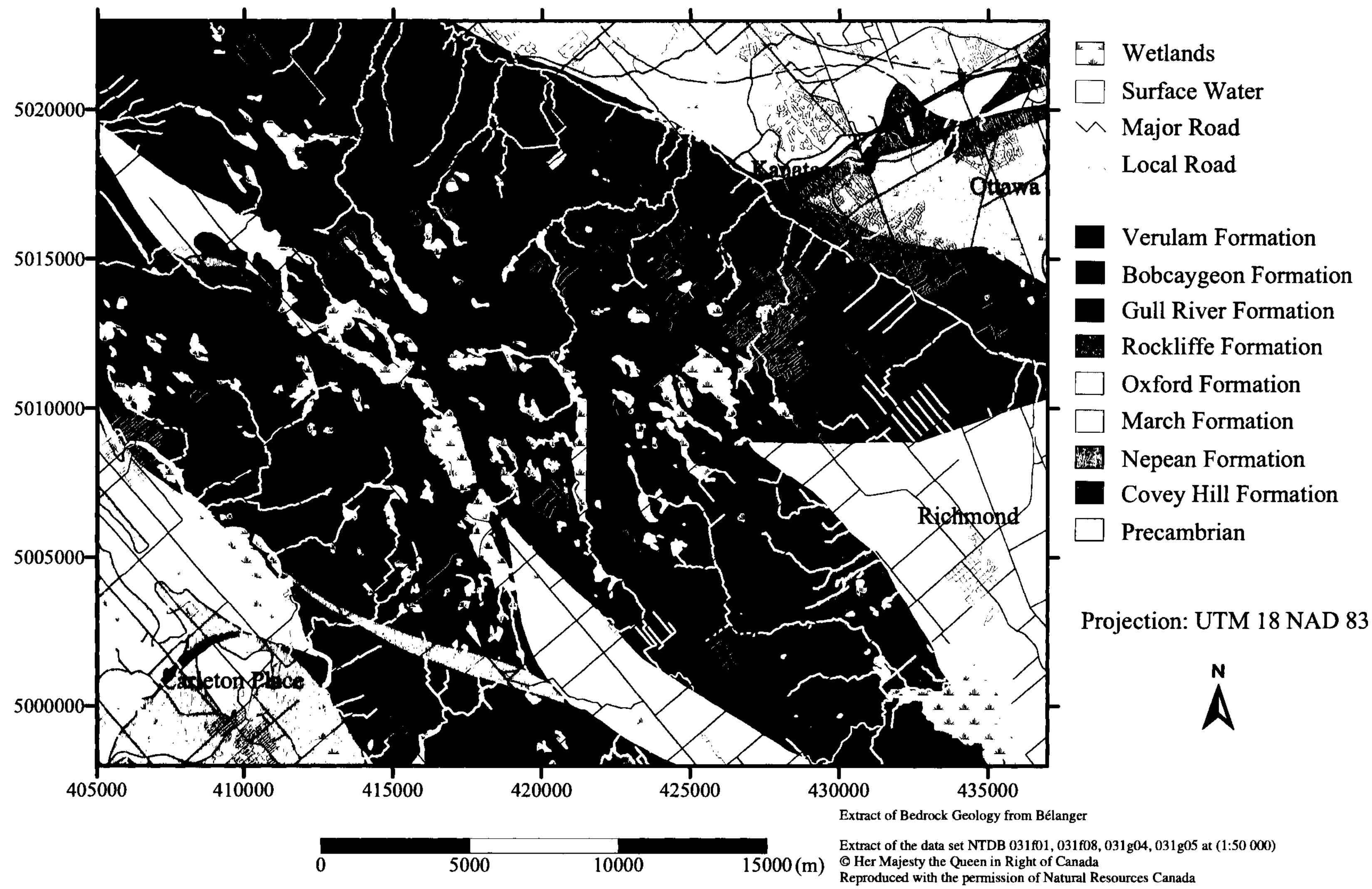


Figure 6-3: Case study bedrock geology

Table 6-1: Bedrock Formations (From Johnson *et al.*, 1992)

	Formation	Age	Thickness*	Description
Paleozoic	**Ottawa Group**			
	Carlsbad	Late Ordovician	Up to 186m	Shale, siltstone and limestone
	Billings	Late Ordovician	Up to 62m	Shale with limestone interbeds
	Lindsay	Late Ordovician	Up to 67m	Limestone with shale interbeds
	Verulam	Middle Ordovician	32 to 65m	Limestone with shaly interbeds
	Bobcaygeon	Middle Ordovician	7 to 87m	Limestone with shaly partings
	Gull River	Middle Ordovician	7.5 to 136m	Limestone to silty dolostone
	Shadow Lake	Middle Ordovician	2 to 3m	Dolostone interbedded with sandstone
	Beekmantown Group			
	Rockcliffe	Middle Ordovician	Up to 125m	Sandstone, shale and limestone
	Oxford	Lower Ordovician	Up to 200m	Dolostone with shale interbeds, more quartz sandstone at bottom of layer
	March	Lower Ordovician	6 to 64m	Interbedded Quartz sandstone, dolomitic sandstone, sandy dolostone, and dolostone
	Potsdam group			
	Nepean	Upper Cambrian	Up to 300m	Quartz-rich sandstone with conglomerate interbeds
	Covey Hill	Cambrian	Up to 13m	Poorly sorted, feldspathic conglomerate and sandstone
Precambrian		Precambrian	Basement rock	Igneous and metamorphic rocks

*Thickness ranges are province wide, not just in study area

6.2.1.1.1 Precambrian Bedrock

The basement rock of the area is part of the Precambrian shield consisting of igneous and metamorphic rocks. The Precambrian shield is part of the core of the North American continent. It was built up between 0.9 and 3.8 billion years ago as the Earth's crust thickened (Golder *et al.*, 2003). Since the rock types within the Canadian Shield are very complex, it has been divided into a number of belts that share a similar evolutionary history (Golder *et al.*, 2003). These belts are then divided into terranes that share similar rock types and ages (Golder *et al.*, 2003; Easton, 1992). The study area is located within the Central Metasedimentary Belt and consists of marble, volcanic rocks, and clastic metasedimentary rocks (Easton, 1992). Due to overlying younger Paleozoic formations, little detail is known about the exact types of Precambrian rocks that exist under the study area.

6.2.1.1.2 Paleozoic Bedrock

Overlying the Precambrian rocks are the younger Paleozoic sedimentary rocks. These were formed when an ocean covered the eastern part of North America approximately 500 million years ago (Golder *et al.*, 2003). During this time, the erosion of the Precambrian rocks caused sand and gravels to be deposited along the shore of the ocean which subsequently were transformed to the sedimentary rocks found today. Since sea levels fluctuated up and down over the Paleozoic period, the deposits changed over time leading to layers of sandstone, limestone and shale (Golder *et al.*, 2003).

The different sedimentary rocks are divided into a number of groups which can be seen in Table 6-1. The study area contains rocks from the Potsdam group, Beekmantown group and the Verulam, Bobcaygeon, Gull River, and Shadow Lake formations of the Ottawa group.

The Potsdam group is made up of the Covey Hill and Nepean formations. The Covey Hill formation is a relatively thin and non continuous layer of feldspathic conglomerates and sandstone (Johnson *et al.*, 1992). It lies unconformably on low areas of the Precambrian surface (Golder *et al.*, 2003) and may not be present over the entire study area. The Nepean formation is a thick layer composed of quartz-rich sandstones and some minor conglomerates that lie unconformably on the Covey Hill formation and the Precambrian surface (Golder *et al.*, 2003; Johnson *et al.*, 1992). The grain size in this formation tends to decrease towards the top of the layer. There are some iron-rich minerals in the formation which can lead to elevated iron content in groundwater (Golder *et al.*, 2003). Over the study area, the formation tends to pinch out on the western side, and generally dip to the east (Golder *et al.*, 2003). These formations are only seen on the bedrock surface in the north-east and south-west corners of the study area. There is also a small area north of Almonte where the layers approach the surface.

The Beekmantown group consists of the March, Oxford and Rockcliffe formations. The March formation serves as the transition from sandstone to dolostone. It lies conformably on the Nepean formation (Golder *et al.*, 2003; Johnson *et al.*, 1992). The Oxford formation is mostly dolostone with some shale interbeds up to 30 cm thick

(Golder *et al.*, 2003; Johnson *et al.*, 1992). The Oxford formation is the surface bedrock formation in several parts of the study area (Figure 6-3). The Rockcliffe formation lies unconformably on the Oxford formation and is made up of sandstone, limestone and shale with the sandstones towards the bottom and limestones toward the top (Golder *et al.*, 2003). The Rockcliffe formation appears at the surface of the bedrock in a few places within the study area, specifically to the south-east of Carleton Place, east of Almonte, and Richmond.

Only the lower four formations of the Ottawa group; Shadow Lake, Gull River, Bobcaygeon, and Verulam are present in the study area. The Shadow Lake formation is a silty dolostone with shale partings and should be present in the study area but is very thin (Golder *et al.*, 2003). It does not appear within the surface bedrock. The Gull River formation is fairly complex and has two distinct portions. The lower portion is typically limestone and silty dolostone with sandstone and shale interbeds whereas the upper portion is similar but with less dolostone (Golder *et al.*, 2003; Johnson *et al.*, 1992). This formation represents the majority of the surficial bedrock in the study area. The Bobcaygeon formation is a limestone rock with some shale and many fossils (Golder *et al.*, 2003). It lies conformably on the Gull River formation and appears in the central region of the study area but is not as prevalent as the Gull River formation. The Verulam formation is present in the north of the study area. It is the youngest of the Paleozoic rocks within the area and lies conformably on the Bobcaygeon formation. It is a limestone rock with shale beds of up to 15 cm thick and also contains fossils (Golder *et al.*, 2003).

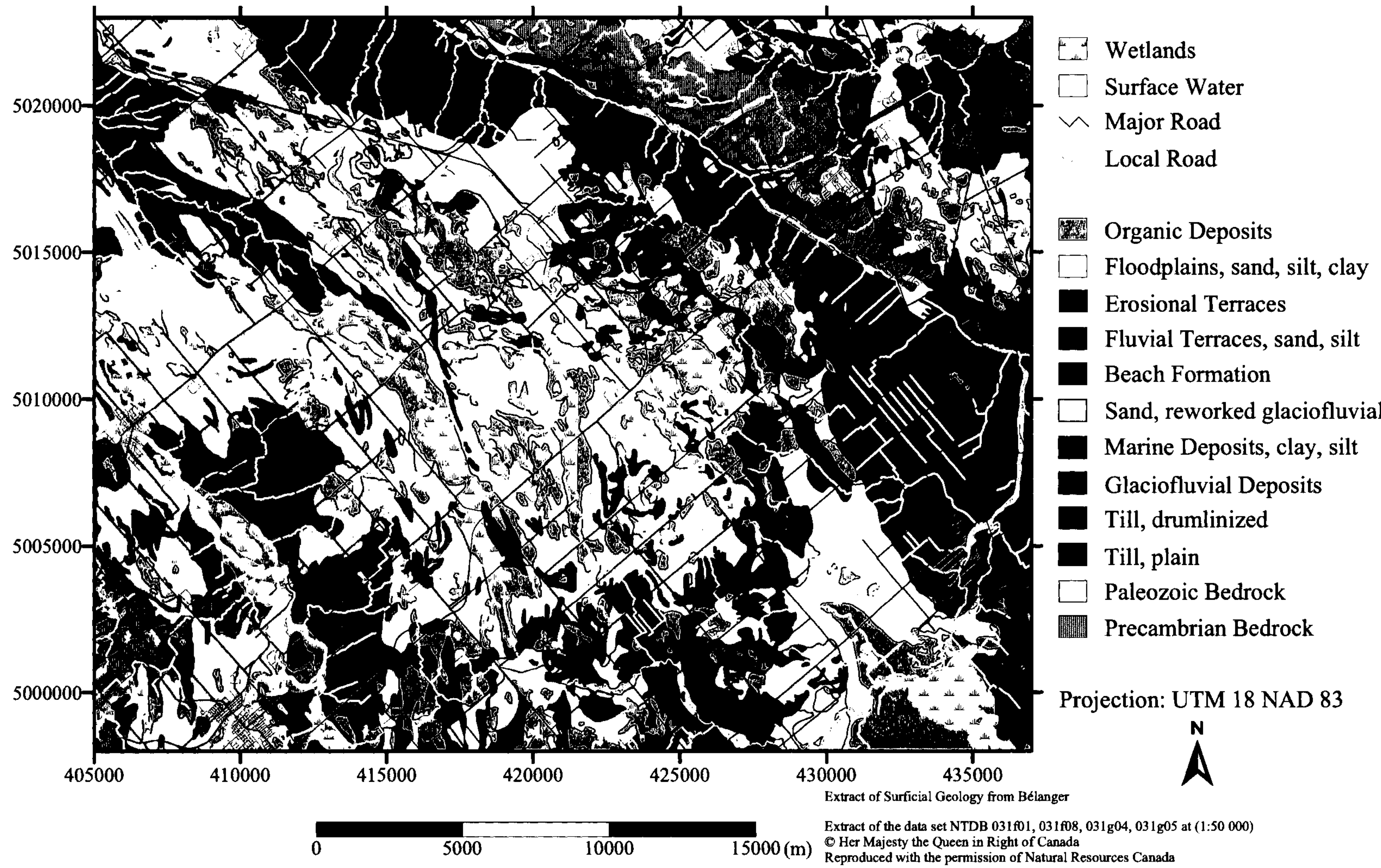
6.2.1.1.3 Faulting

The tectonic history of the study area has been active since the Precambrian era and several faults can be seen within the study area. Of particular note are the faults that run along the Carp River (Hazeldean fault), and the Mississippi River. These faults are a part of the Ottawa-Bonnechere Graben system that runs from Lake Timiskaming to Montreal (Golder *et al.*, 2003). The result of these faults is that the central portion of the study area from the north-west to the south-east dropped along these faults. This result can be seen on the bedrock geology map (Figure 6-3) where Paleozoic (younger) layers are present in the centre of the area and the Precambrian (older) rocks are exposed on the edges. The older rocks are exposed because all of the younger Paleozoic rocks above them were eroded away.

6.2.1.2 Overburden

The overburden of the area can be seen in Figure 6-4. From this map, it can be seen that there is a wide variety of surficial materials in the area from clays and tills to no overburden whatsoever. The overburden was deposited during the Late Wisconsinan glaciation that ended 10,000 years ago (Golder *et al.*, 2003). As the glaciers advanced over the area, they ground down the bedrock and carried sediment within the ice. As the ice melted, these sediments were deposited as tills in the area. During this time, the St. Lawrence River valley was blocked towards the east by other glacier lobes causing large

Figure 6-4: Case study surficial geology



lakes to form in the area as indicated by the glaciolacustrine (clay, silt, sand, and gravel) sediments in the area (Golder *et al.*, 2003).

When the glaciers retreated, the area was depressed below sea level allowing the water from the Atlantic Ocean to cover the region. This sea allowed for the deposition of clays in more quiet environments. As the ground rebounded and the water level dropped, eskers, drumlins, and tills, deposited by the glaciers, were exposed and reworked or eroded by the waves of the sea to produce the sand beaches (Golder *et al.*, 2003).

The thickness of the overburden in the area is generally very thin (< 2m) and the bedrock is exposed at the surface (Figure 6-4) (Golder *et al.*, 2003). It does however have areas of significant thickness (> 20m) in the Carp area as well as surrounding Richmond and Stittsville.

6.2.2 Hydrogeology of Area

The hydrogeology of the area is very closely linked with the geology described in the previous section. There are two types of aquifers present in the study area; bedrock and overburden. The overburden in the majority of the area is very thin and as a result the overburden aquifers are not as prevalent as the bedrock aquifers. In the study area, the overburden aquifers are localized around areas with thicker, water bearing sediments such as sands, gravels, and sandy till above the bedrock surface. Where there is clay and silty till present, the sediments serve as aquitards. Examples of aquifers include the sand aquifer in the Carp area (Golder, 2003a) and the area through Stittsville to Richmond and beyond. The hydrologic properties of the different surficial sediments, as well as the yields from wells within certain formations can be seen in Table 6-2.

Table 6-2: Hydraulic conductivity and aquifer yield of surficial material (From: Golder *et al.*, 2003)

Material	Hydraulic conductivity (m/s)	Yield*
Surficial Sand	$1x10^{-5}$ to $1x10^{-4}$	Very Good
Glaciofluvial Sand and Gravel	$1x10^{-6}$ to 1.0	Very Good
Basal Sand and Gravel/Till	$1x10^{-5}$	Good
Clay	$1x10^{-10}$ to $1x10^{-8}$	Poor
Silty Till	$1x10^{-7}$ to $1x10^{-6}$	Poor

*Poor: < 15 L/min; Good: Usually = 15 L/min; Very Good: > 15 L/min

By far the most prevalent aquifers in the study area are the bedrock aquifers which underlie the entire study area. The majority of the wells in the study area use these aquifers for their water source. The different geological formations provide different yields at wells. As can be seen in Table 6-3, the best source for water in the study area is in the deep Nepean sandstone formation.

The sandstone aquifer is made up of the Nepean and Covey Hill formations, as well as the lower portions of the March formation where sandstone is present (Golder *et al.*, 2003). It is used as a source for large capacity wells including municipal and industrial uses. The typical yield for the aquifer is 150 to 630 L/min with some values as high as

2000 L/min (Golder *et al.*, 2003). The flow of water through the aquifer is due to the relatively high porosity of the sandstone as well as fractures throughout the formations. It has been found that the aquifer is most productive at the contacts with the March formation above and the Precambrian bedrock below (Golder *et al.*, 2003).

Table 6-3: Hydraulic conductivity and aquifer yield of bedrock (From: Golder *et al.*, 2003)

Material	Hydraulic conductivity (m/s)	Yield*
Verulam		Poor to Moderate
Bobcaygeon		Poor to Moderate
Gull River		Poor to Moderate
Rockcliffe		Poor to Moderate
Oxford	$1x10^{-10}$ to $1x10^{-4}$	Good
March	$1x10^{-6}$ to $1x10^{-4}$	Good
Nepean	$1x10^{-8}$ to $1x10^{-6}$	Very Good
Covey Hill		Very Good
Precambrian	$1x10^{-8}$ to $1x10^{-4}$	Moderate

*Poor: < 15 L/min; Moderate: Barely = 15 L/min; Good: Usually = 15 L/min; Very Good: > 15 L/min

The second major bedrock aquifer is the dolostone aquifer. It consists of the Paleozoic March and Oxford formations. The average well yield for the formation is 42 L/min (Golder *et al.*, 2003). Groundwater flow through the aquifer is primarily through fractures in the rocks (Golder *et al.*, 2003).

The top four layers of Paleozoic limestone, Rockcliffe, Gull River, Bobcaygeon, and Verulam formations also serve as a source of water for individual domestic use in the study area. Although the yields tend to be less than in the lower aquifers, the cost of drilling to reach the lower aquifers forces wells to be completed in the upper aquifer.

Due to the tectonic history of the area a number of faults are present throughout the area and generally strike in a south-east direction (Golder, 2003b). Displacements of the order of 5 to 50 m have been reported in the study area (Golder, 2003b). These faults can potentially play a very large role in groundwater flow, but detailed information on faults is lacking. These faults can cause groundwater flow to stop in some areas, and serve as a conduit in others. In a report modelling the sandstone aquifer in the southern half of the case study area, it was found through the analysis of a contaminant plume, that groundwater flow is generally unimpeded across the faults (Golder, 2003b).

6.2.3 Hydrogeological Conceptual Model

A map of the measured water table elevations can be seen in Figure 6-5. This map was created for this case study from the information held in the MOE Well Water Information System (WWIS). The steps involved in creating the map were modified from the MOE Groundwater Studies protocols (MOE, 2001) and were as follows:

i. The MOE WWIS data were filtered to remove any wells with a UTM reliability code greater than '6' (error of at least 300 m).
ii. The MOE WWIS data were filtered to remove any wells that were recorded at an elevation greater than 10 m difference from the digital elevation model (DEM) (Geobase, 2005).

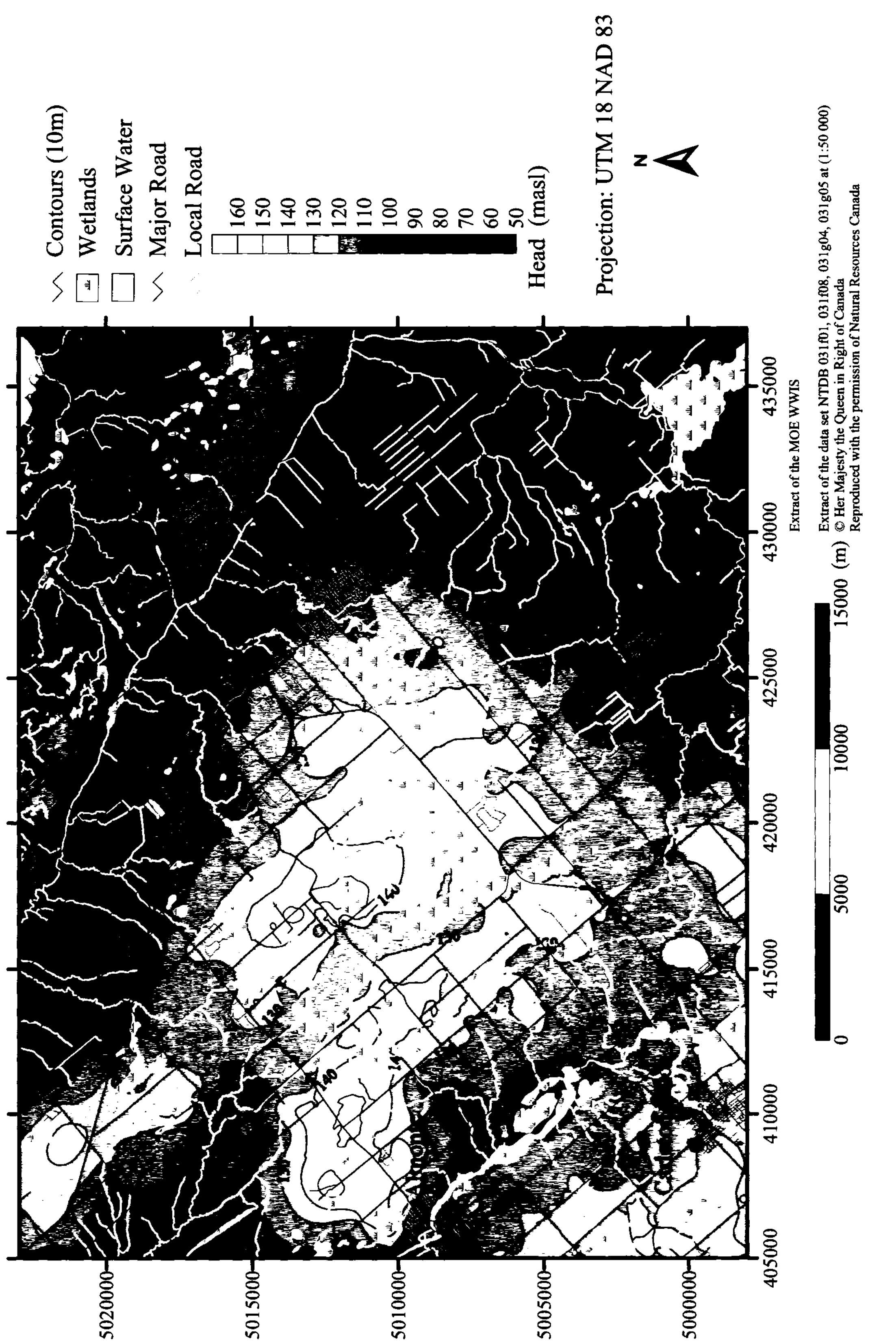


Figure 6-5: Case study piezometric surface

iii. The MOE WWIS data were filtered to remove any wells not completed into bedrock.

iv. The water levels in the remaining MOE WWIS wells were then interpolated using Surfer 8.02 (kriging) using a 100m grid. Any obvious outliers and wells having a residual greater than 10 m were removed.

v. The surface was then interpolated again to obtain a representation of the water table.

The wells were not divided into the different geological layers because there is little vertical gradient between deep and shallow bedrock units except in certain localized areas (Golder, 2003b). This implies that the piezometric levels will be fairly constant with depth throughout the Paleozoic units. The Nepean sandstone has been considered as a semi-confined aquifer because it is exposed in places but is also overlain by leaky aquifers and aquitards (Golder, 2003b).

The general flow of groundwater in the bedrock of the area can be determined from a number of different studies (Golder, 2003b; Golder *et al.*, 2003) as well as the static water levels from the MOE Water Well Information System (WWIS) (Figure 6-5). The groundwater forms a mound in the central north-west of the region with decreasing gradients in the east, south, and west directions. This essentially indicates that groundwater flows radially outward from the centre of the region where the water table is at a maximum elevation of approximately 155 masl.

The rivers in the area serve as a groundwater discharge in most cases because the equipotential lines indicate that flow is moving towards the bedrock below the rivers. Figure 6-5 also shows that the flow of groundwater towards the Mississippi and Carp rivers is from both sides indicating that the groundwater moves towards the surface to feed the river.

There are a number of wetlands throughout the study area. These may serve as groundwater recharge areas or discharge areas depending on the situation. If the bedrock piezometric water surface (Figure 6-5) and the topography of the area (Figure 6-2) are compared, it can be seen that the water elevation in the bedrock follows the topographic elevation fairly closely. Figure 6-2 also shows that the wetlands occur, in most cases, in topographically low areas. The wetlands in the region can be considered as groundwater discharge areas where the topographic elevation is lower than the water table in the bedrock. The groundwater flows to the wetland where it eventually joins the surface water streams and rivers.

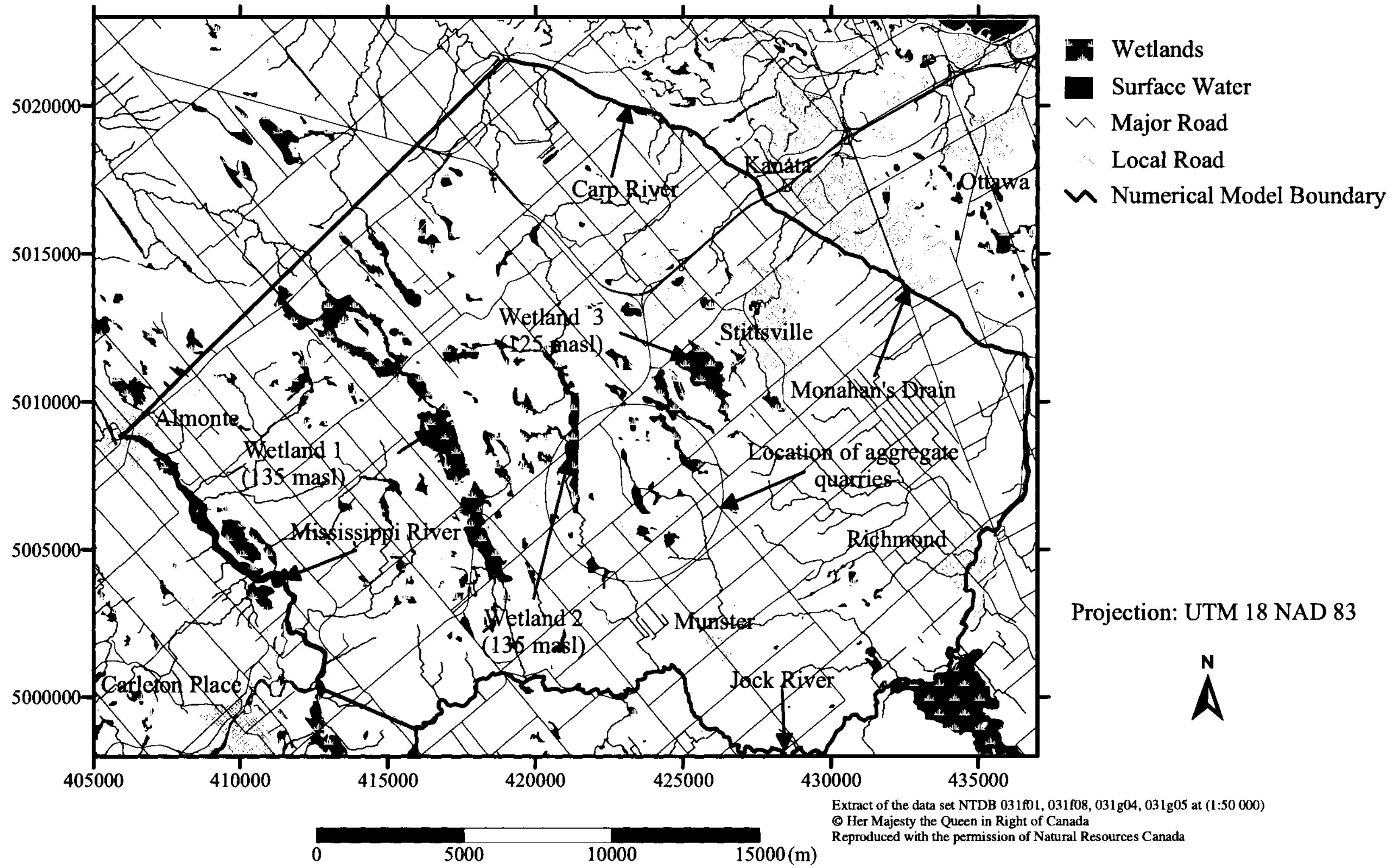
6.3 Regional Numerical Model

With the conceptual hydrogeological model in place, a numerical model, using Visual MODFLOW (3.1.0), was developed in order to quantify groundwater flows and determine the effects of dewatering the five quarries on the surrounding aquifers. The area of the numerical model was determined from known hydrological water elevations so that boundary conditions could be easily determined, and so that they would not influence the results at the centre of the model where the aggregate quarries to be studied were located. In this case, the boundary of the numerical model was determined from the rivers in the area; the Mississippi River to the west, the Jock River to the south, and the Carp River and Monahan's Drain to the east. The boundaries of the numerical model can be seen in Figure 6-6. The east and west boundaries also coincide with major faults in the area where the bedrock is exposed to the surface. Placing the boundaries in line with the rivers eliminated the need to model the complicated geology at these points.

6.3.1 Discretization

In order to model the area in three dimensions (3D), the different hydrogeologic units had to be represented in the model. This was done using a 5 layer model. The top layer represented the overburden in the area. Layers 2 to 4 represent the upper Paleozoic bedrock units (March, Oxford, Rockcliffe, Gull River, Bobcaygeon, and Verulam formations) and the 5^{th} layer represents the Nepean sandstone aquifer.

To determine the different elevations of the different layers, the borehole logs contained in the MOE WWIS database were used to obtain the tops of the layers. By its nature, kriging, the interpolation technique used, contains some uncertainty in the results it gives. These results were minimized by ensuring that the residuals are minimized, and removing obviously suspect observation points. For the purposes of the numerical model created for this case study, the uncertainty created by the interpolation does not contribute



Extract of the data set NTDB 031f01, 031f08, 031g04, 031g05 at (1:50 000)
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Figure 6-6: Boundaries of regional numerical model significantly to the results of the model. This is due to the sometimes large uncertainties associated with hydrogeological data.

6.3.1.1 Layer 1

The top layer of the model represents the overburden in the area. The top of this layer was defined by the DEM for the region (Geobase, 2005). The bottom of the layer was defined by the top of the bedrock in the area. It can be seen in Figure 6-7 and was developed by using information from the MOE WWIS and the following procedure:

i. The MOE WWIS data were filtered to remove any wells with a UTM reliability code greater than '6' (error of at least 300 m).

ii. The remaining MOE WWIS data were filtered to remove any wells that were recorded at an elevation greater than 10 m difference from the DEM.

iii. The remaining MOE WWIS data were filtered to select only the wells that were completed in bedrock.

iv. The elevation of the uppermost bedrock unit was determined from the wells completed in bedrock.

v. The bedrock surface was then interpolated with Surfer 8.02 (kriging) using a 100m grid. Any obvious outliers and wells having a residual greater than 10 m were removed.

vi. The data were interpolated again to obtain a representation of the bedrock surface.

vii. The bedrock surface was cropped using the DEM to ensure that the bedrock surface was never above the topography of the area.

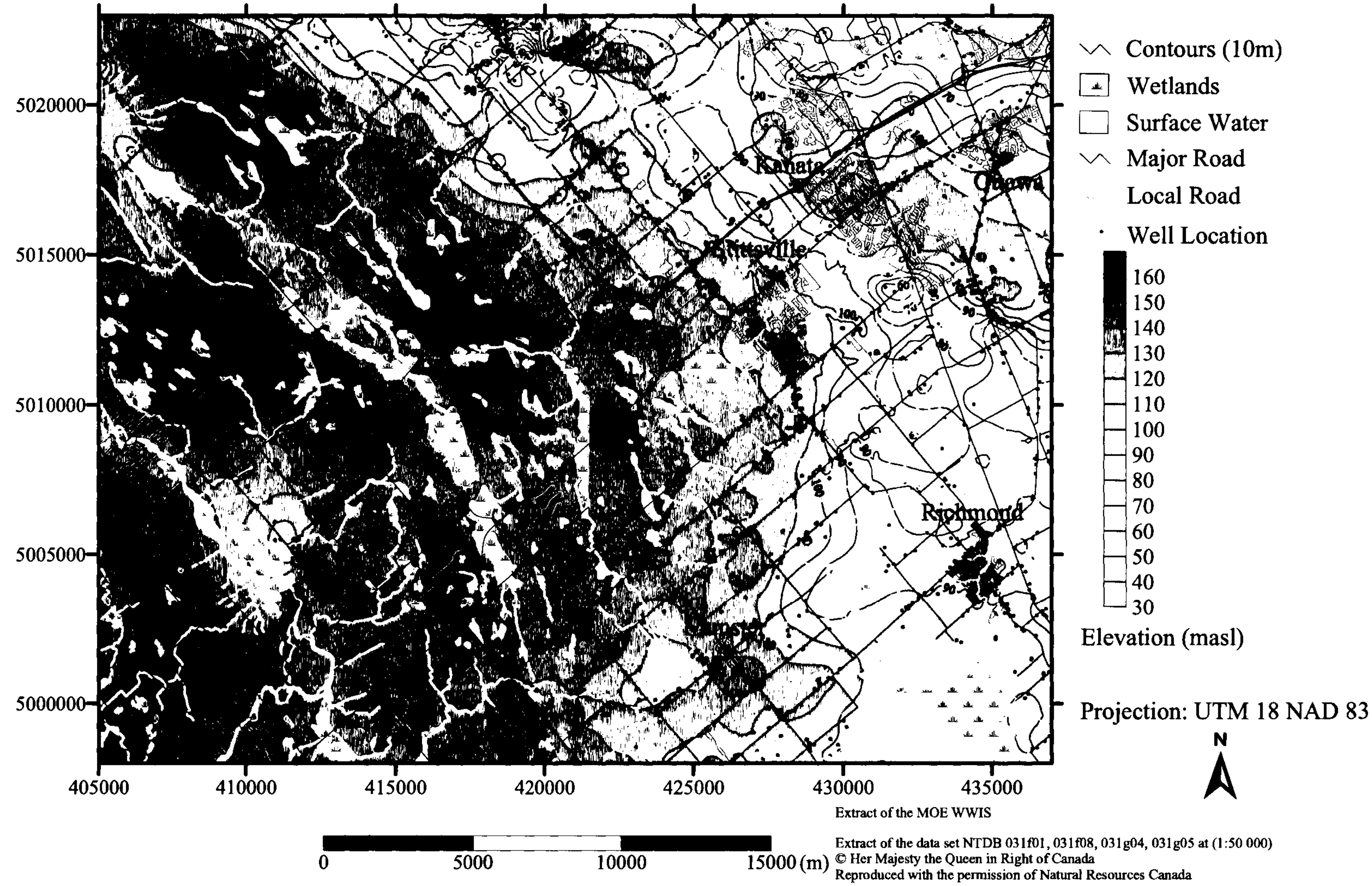


Figure 6-7: Case study elevation of bedrock surface

The thickness of the overburden can be seen in Figure 6-8. The thickness was obtained from subtracting the top of the bedrock surface from the DEM. It can be seen from the figure that the overburden is found mostly in the south-east of the numerical model. It ranges in thickness from 0 m to 50 m. Since Visual MODFLOW does not allow the thickness of a layer to be 0, the minimum thickness of the layer was set to be 0.1m in the numerical model.

6.3.1.2 Layers 2, 3, and 4

Layers 2, 3, and 4 represent the upper Paleozoic formations of the area consisting of limestone, dolostone and shale. The top of layer 2 was defined by the bottom of layer 1 (top of bedrock) and the bottom of layer 4 is defined by the top of the Nepean sandstone aquifer (Figure 6-9). The top of the sandstone aquifer was determined as follows:

i. The MOE WWIS data were filtered to remove any wells with a UTM reliability code greater than '6' (error of at least 300 m).
ii. The remaining MOE WWIS data were filtered to remove any wells that were recorded at an elevation greater than 10 m difference from the DEM.
iii. The remaining MOE WWIS data were filtered to select only the wells that were completed into sandstone.
iv. The wells to the north west of the Carp River and Monaghan Drain were removed to eliminate the effects of the fault during the interpolation. This was necessary as the interpolated top of the sandstone surface was above the top of the bedrock surface along the fault. These wells fall outside of the numerical model boundaries.

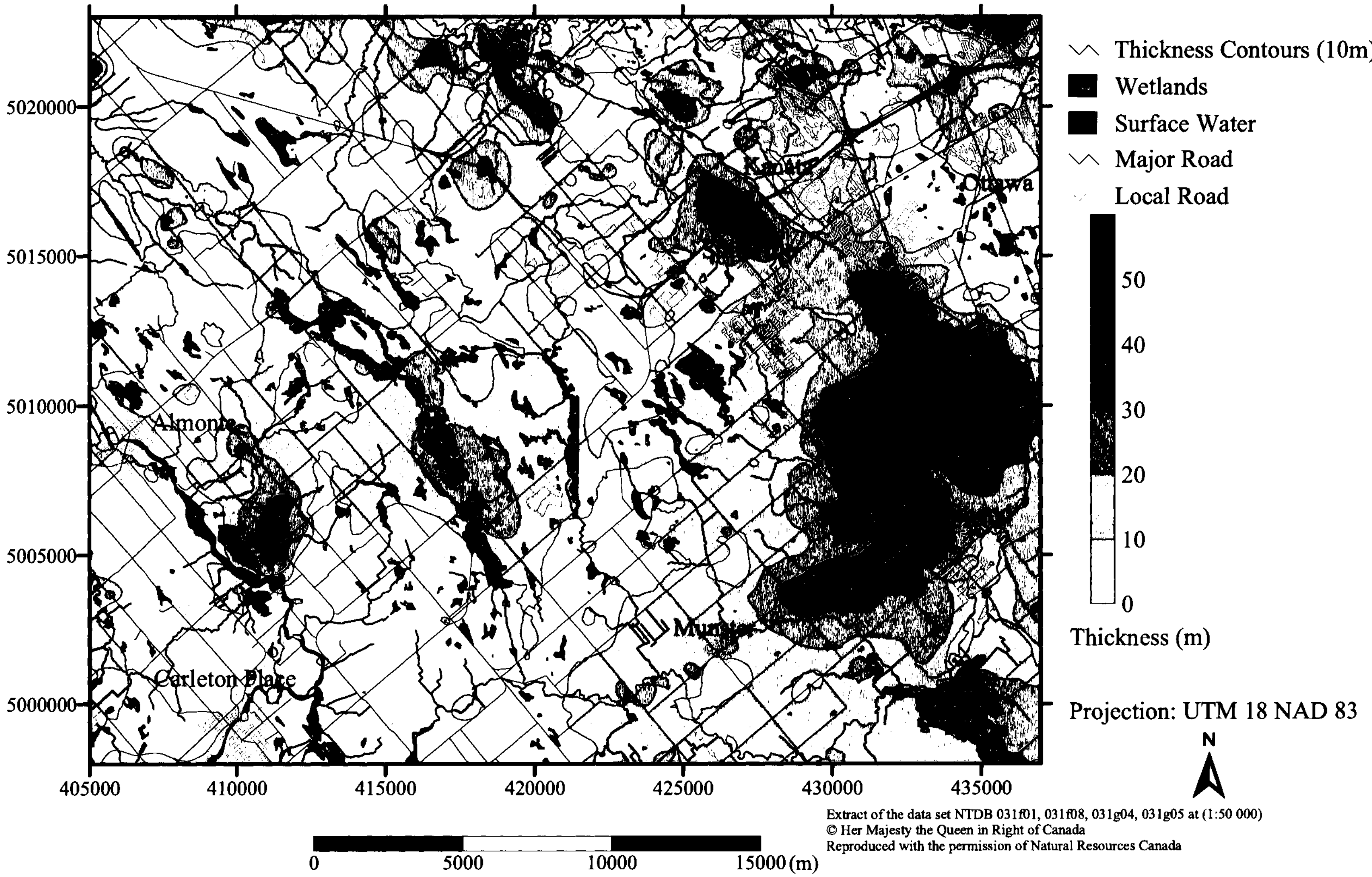
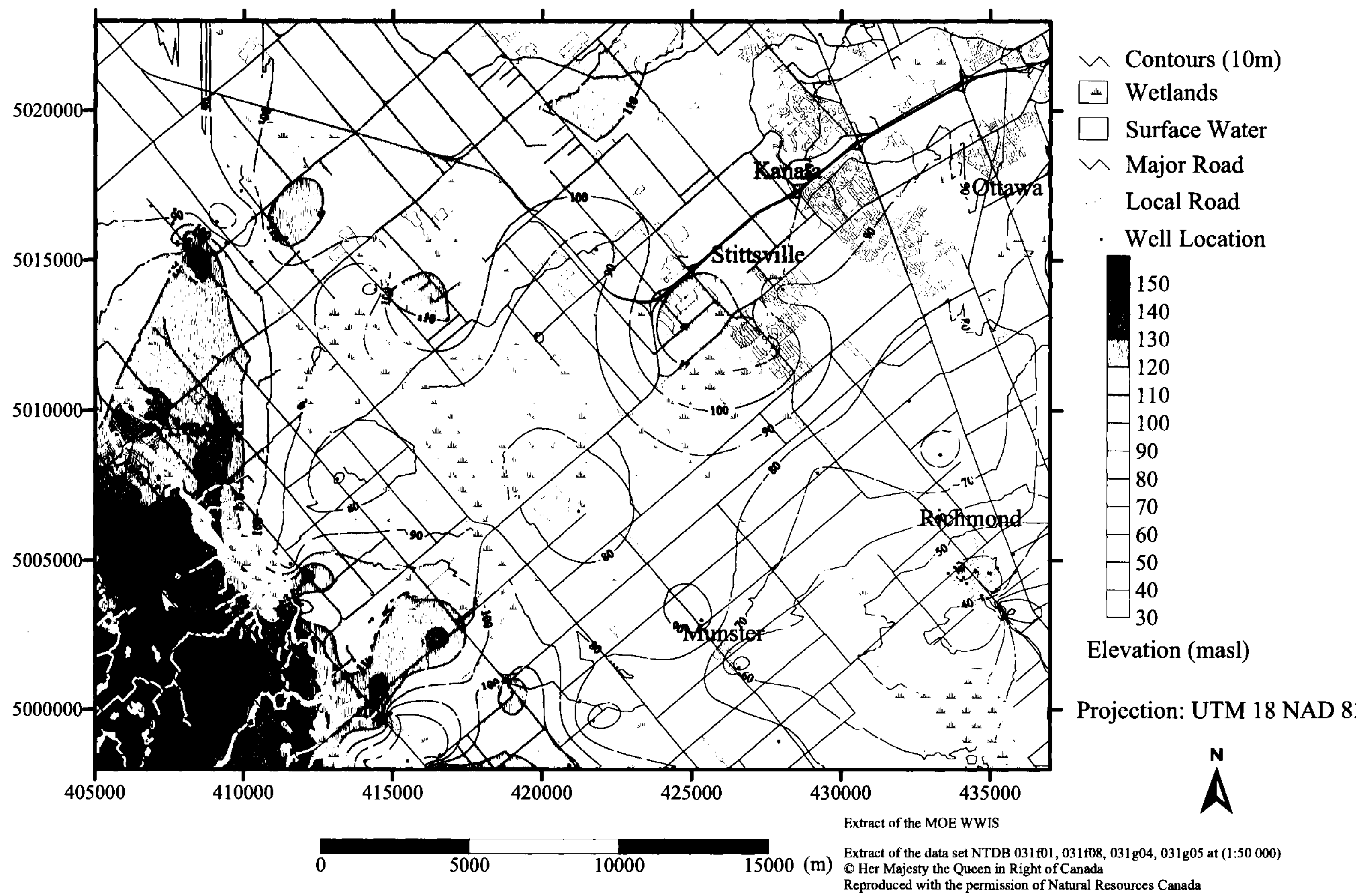


Figure 6-8: Case study area overburden thickness

Figure 6-9: Case study elevation of Nepean sandstone surface



v. The elevation of the top of the sandstone unit was determined from the wells completed in sandstone.

vi. The sandstone surface was then interpolated using Surfer 8.02 (kriging) using a 100m grid. Any obvious outliers and wells having a residual greater than 10 m were removed.

vii. The data were interpolated again to obtain a representation of the top of the sandstone surface.

viii. To account for interpolation errors, the sandstone surface was cropped using the bedrock surface to ensure that the sandstone surface was never above the top of the bedrock of the area.

The thickness of the upper Paleozoic units can be seen in Figure 6-10. It was obtained by subtracting the top of the sandstone surface from the top of the bedrock surface. The thickness of the layer ranges from 0m to 80m and seems to run from north-west to south-east. The formation is thin where there is bedrock exposed to the north-east and south-west of the region. Since the upper Paleozoic formation is so thick, it was divided into 3 equal layers for the numerical model. Visual MODFLOW will not allow layer thicknesses to be 0 so the minimum thickness for the three layers is 0.1 m in the numerical model.

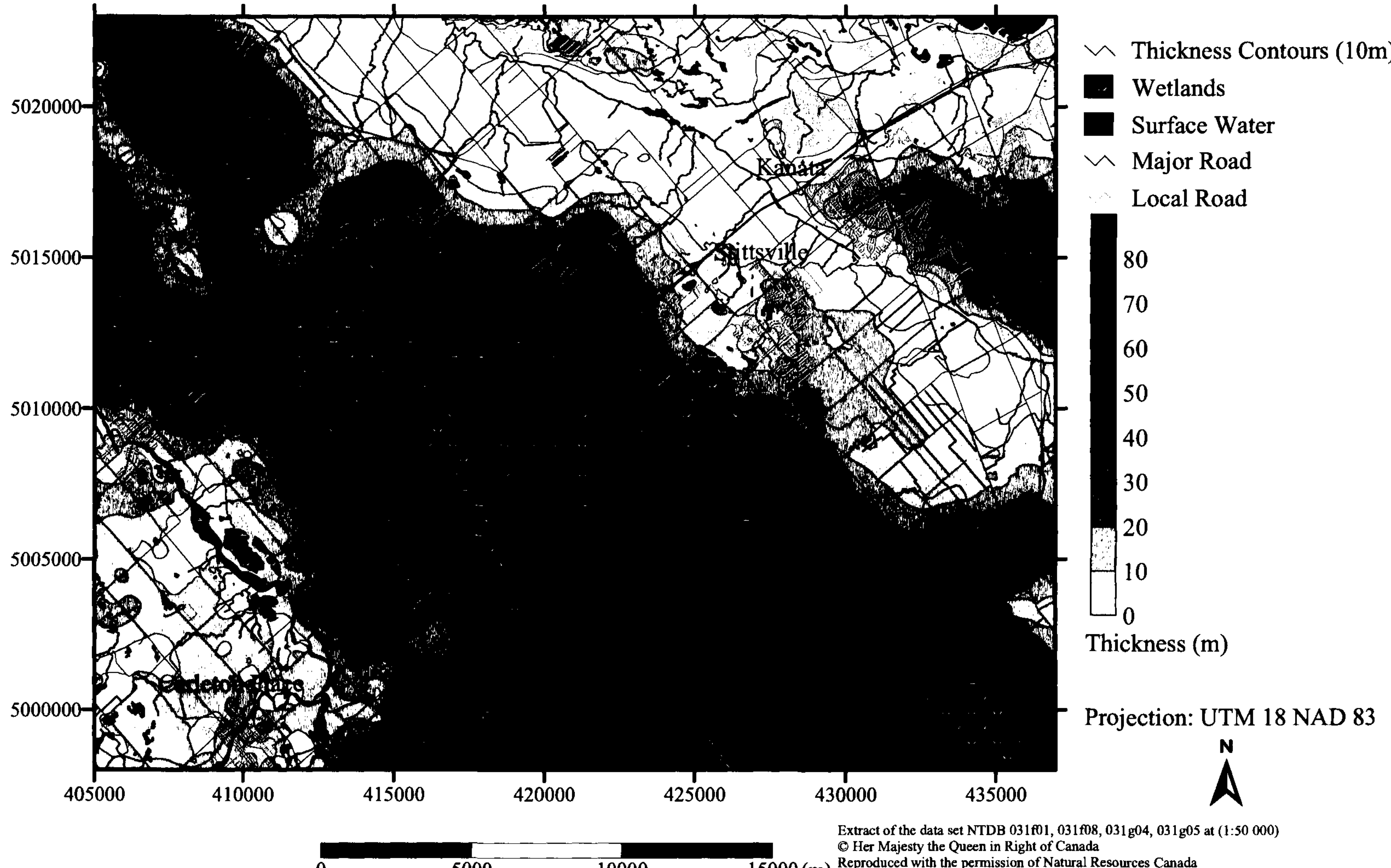


Figure 6-10: Thickness of upper Paleozoic bedrock

6.3.1.3 Layer 5

The lowest layer of the numerical model represents the Nepean sandstone aquifer. Since the basement Precambrian bedrock has a sufficiently low hydraulic conductivity (K), it can be seen as an aquitard (Golder, 2003b; Golder *et al.*, 2003), so that there is no flow into or out of the formation. The bottom of the layer is defined by the Precambrian layer, and the top is defined by the top surface of the Nepean sandstone. Since there were very few wells that reached the Precambrian rock (2 wells within the model domain), interpolating a surface would have a very large uncertainty. To control this, a uniform thickness of 50m was assigned to the Nepean sandstone unit (Layer 5). This was based on the thickness of the sandstone reported by Golder et al. (2003) and a hydrogeological report covering the southern portion of the study area (Golder, 2003b). Although the assumption of a uniform thickness of 50m is not representative of the actual thickness of the aquifer, without having more information of the Precambrian surface it is difficult to represent the thickness more accurately.

6.3.1.4 Horizontal Discretization

The entire model domain was discretized into 320 rows and 250 columns in a 100m by 100m grid in the horizontal plane. It was assumed that this level of resolution would be sufficient to model the regional aquifer. There were a total of 80,000 cells on any of the five layers with a total of 400,000 cells in the entire model.

6.3.2 Boundary Conditions

The borders of the numerical model can be seen in Figure 6-6. The model was constrained by the Jock River to the south, the Mississippi River to the west, and the Carp River and Monahan's Drain to the east. The northern edge of the boundary was formed by a line between Almonte and Carp. These boundaries were chosen because the water level of the rivers was known and could be prescribed as a constant head boundary. The area of interest surrounding the aggregate quarries fell into the centre of the domain and was far enough from the edges that the effects from the dewatering activity would not extend to the borders.

Only the major rivers on the border of the model were given constant head values. In the original model, all wetlands, lakes, and small creeks and streams within the domain were not defined. This is because the resolution of the grid was not small enough to consider the streams effectively. Wetlands were not modelled as constant head boundaries because the effect of dewatering the aggregate quarries on the surrounding wetlands was one of the considerations when creating the model.

An improvement was made to the original model by simulating the larger wetlands as drains. All other boundary conditions remained the same. This was done because it better represented the conceptual model, where wetlands were thought to be fed by groundwater. It also allowed the model to quantify the flows from the wetlands as a calibration point and would better show the impacts of dewatering the quarries on the wetlands. The three modelled wetlands were located in layer 2 and can be seen in Figure 6-6.

6.3.2.1 Constant Head Boundaries

The rivers in the numerical model were given constant head values in the top layer assuming that the levels did not fluctuate significantly throughout the year. Figure 6-11 shows the different constant head boundaries used in the model. The information on the surface water levels was obtained from hydrogeological studies (Golder, 2003b) as well as from the DEM of the area (Geobase, 2005). As can be seen the Mississippi River drops in elevation from 125 to 117 masl. The Jock River drops in elevation from 133 to 90 masl, the Carp River changes in elevation from 95 to 90 masl as it moves to the north and Monahan's Drain drops from 95 to 90 masl as it joins the Jock River. All of the other boundaries on the first layer are "no flow" boundaries due to lack of specific information concerning the flow of groundwater in the overburden. These boundaries are sufficiently far from the quarry site that they should not influence the results of the numerical model.

The constant head boundary representing these rivers is modelled on the top layer only. This implies that the river is directly connected to the bedrock. This may not be an accurate assumption for all of the rivers due to organics and other sediments on the river bed, but at the scale being studied, the general direction and quantity of flow will be adequately represented. If there was a numerical model created at a much smaller scale,

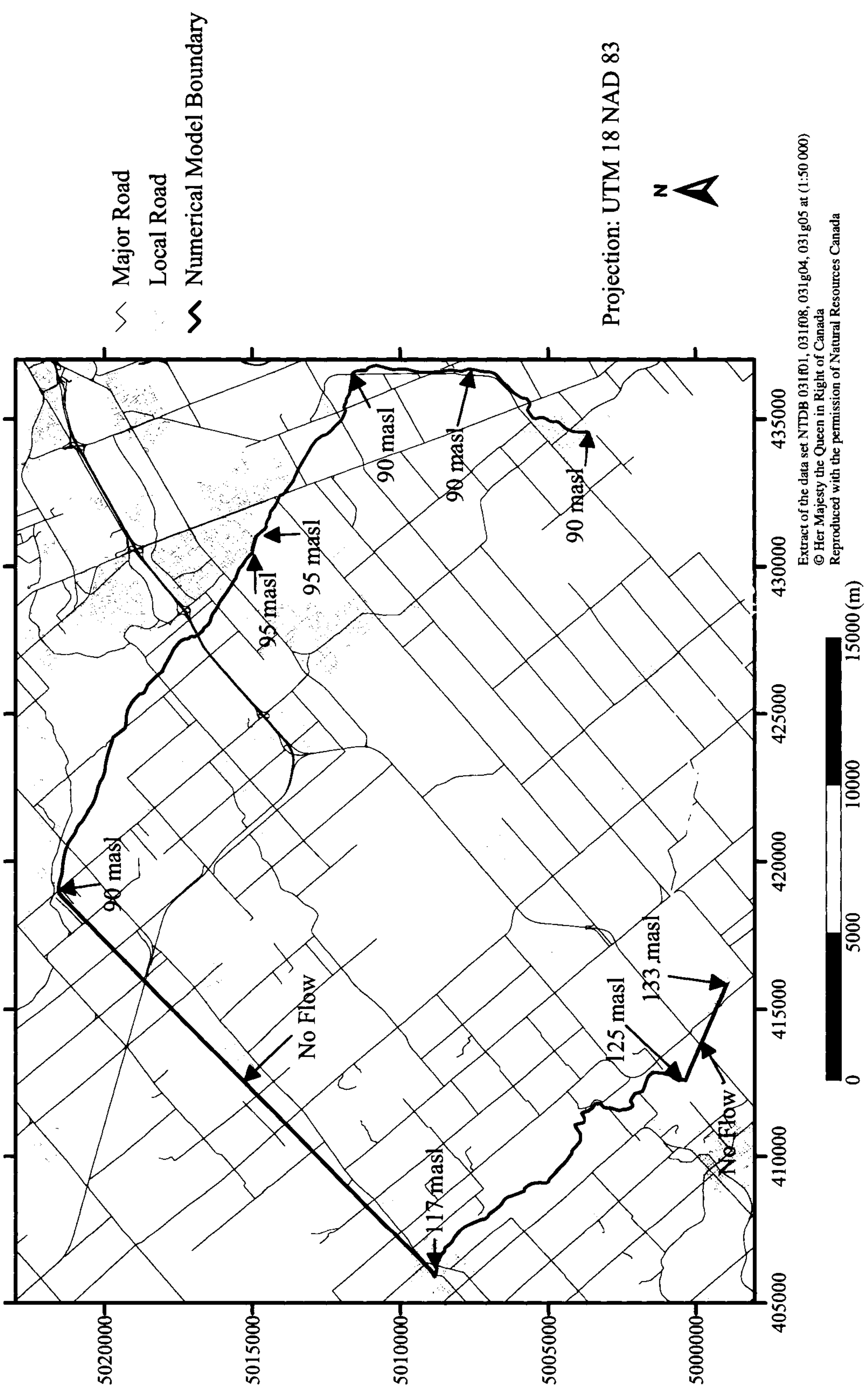


Figure 6-11: Boundary conditions for layer 1 of the regional numerical model

the sediments between the bedrock and the river would have to be considered in order to capture the interactions between surface and groundwaters.

The upper Paleozoic limestone, shale, and dolostone are assumed in this model to serve as an aquitard. Typically aquitards only allow flow in the vertical direction. For this reason, the boundaries on layers 2, 3, and 4 are all "no flow" boundaries as seen in Figure 6-12.

The fifth layer represents the Nepean sandstone aquifer where a gradient was established in the layer using constant head boundaries. These boundaries were established from existing reports (Golder, 2003b, Golder *et al.*, 2003) as well as the data from the MOE WWIS. Figure 6-13 shows the boundary to the north of the model falls in elevation from 135 to 120 towards the south-west, and from 135 to 90 towards the north-east. The boundary in the south of the model falls from 125 to 85 towards the east of the model. All other boundaries are "no flow" boundaries including the bottom of the 5^{th} layer.

6.3.2.2 Recharge

Recharge values were assigned to different surficial materials in the top layer (layer 1) of the domain. Recharge in the model represents the amount of water that enters into the top layer and subsequently to other deeper layers. It does not represent the interflow between the surficial sediments and surface water.

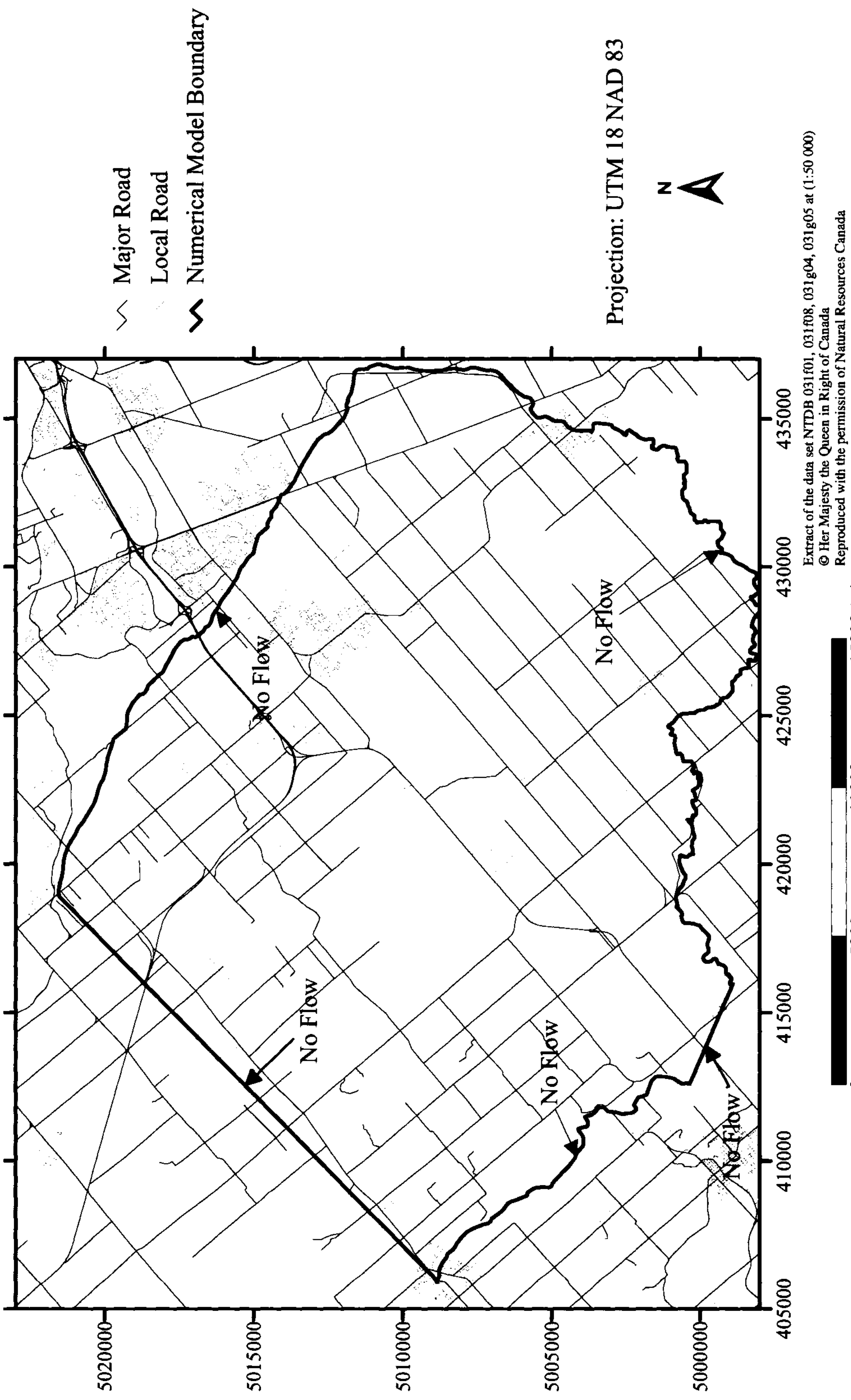


Figure 6-12: Boundary conditions for layers 2, 3, and 4 of the regional numerical model

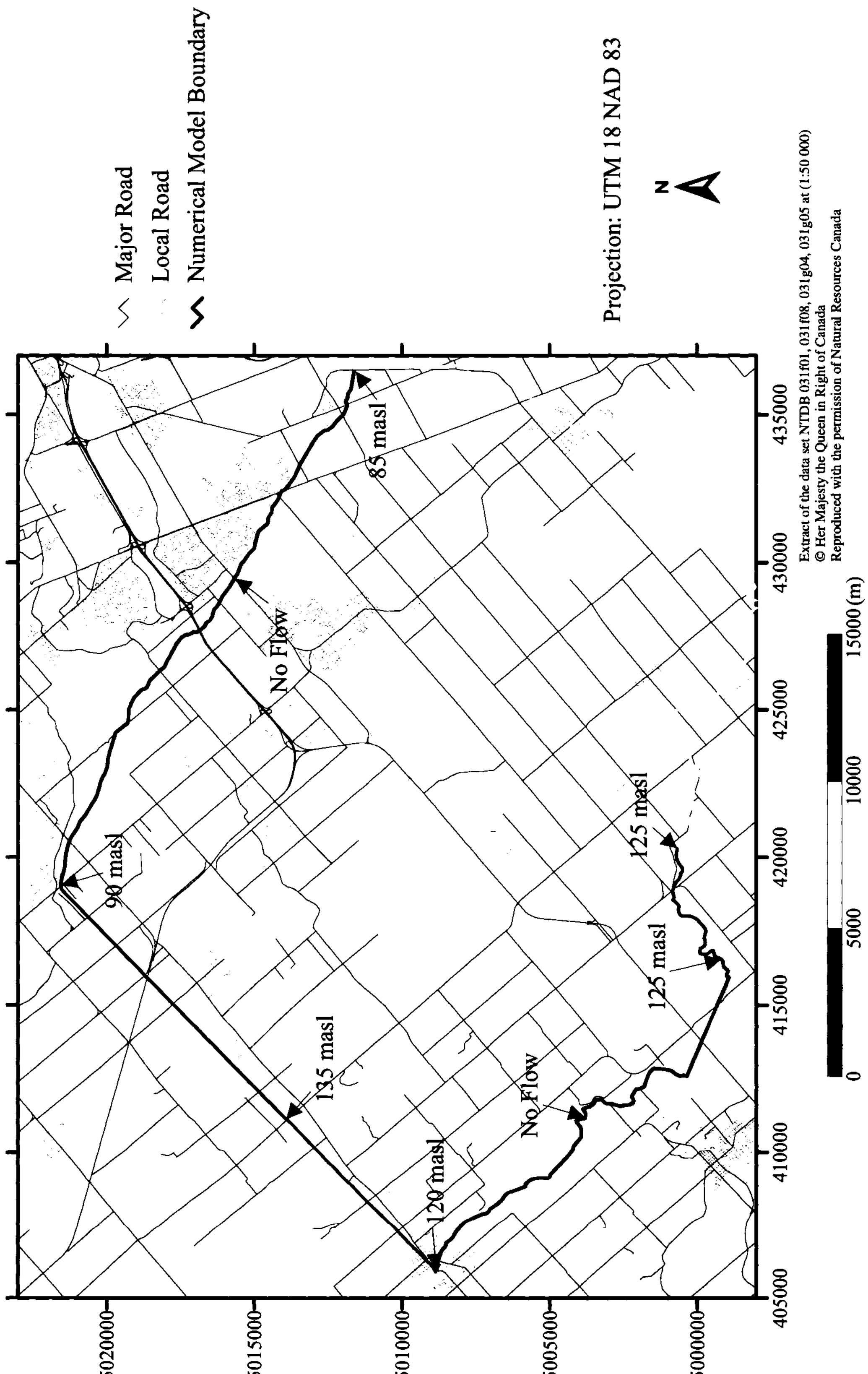


Figure 6-13: Boundary conditions for layer 5 of the regional numerical model

Since there are a large number of different surficial sediments in the area, in order to simplify the numerical model, some of the different sediments were combined if they shared similar properties. The resulting distribution of recharge units can be seen in Figure 6-14. The actual values used for recharge were one element of the calibration of the model and are discussed in section 6.4.1. Golder *et al.* (2003) estimated an average amount of infiltration to be 130 to 275 mm/yr for a very large area stretching from north of Renfrew to north of Kingston that included the study area. This estimate should only be used as a guideline since it is based on such a large geographic area with many different soil types and ground covers.

6.3.2.3 Hydraulic Conductivity and Porosity

The overburden in the area of the model is composed of many different units. In order to simplify the inputs to the numerical model, the different types of overburden with similar hydraulic properties were grouped into 5 units consisting of sand, till, clay, bedrock, and wetlands (organic sediments). The resulting distribution for the first layer of the numerical model can be seen in Figure 6-15. Several studies within the area have determined ranges for the K values for the surficial soils. The maximum and minimum values from these studies can be seen in Table 6-4. The results reported by the different studies can be seen in more detail in Appendix F.

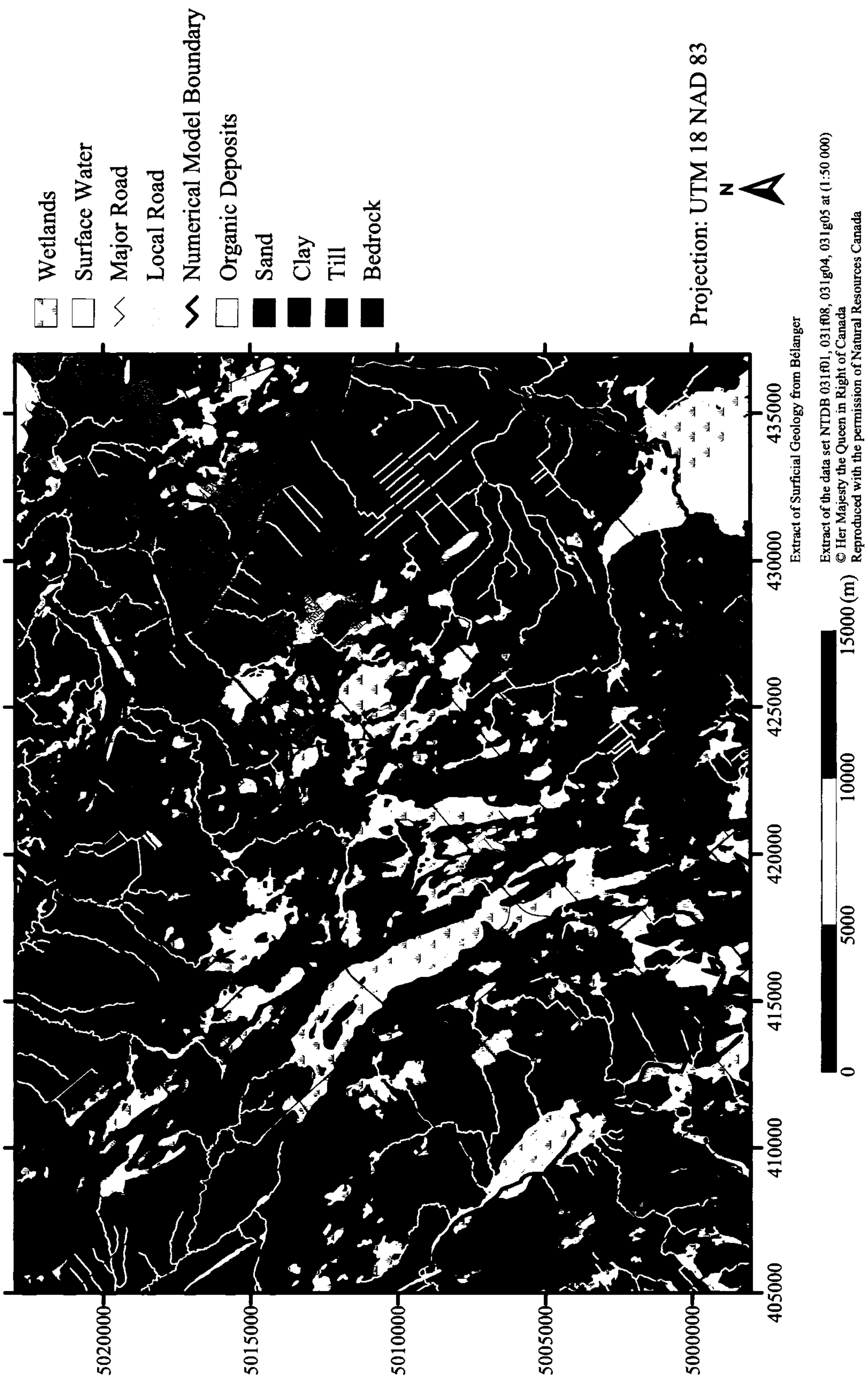


Figure 6-14: Recharge distribution for regional numerical model

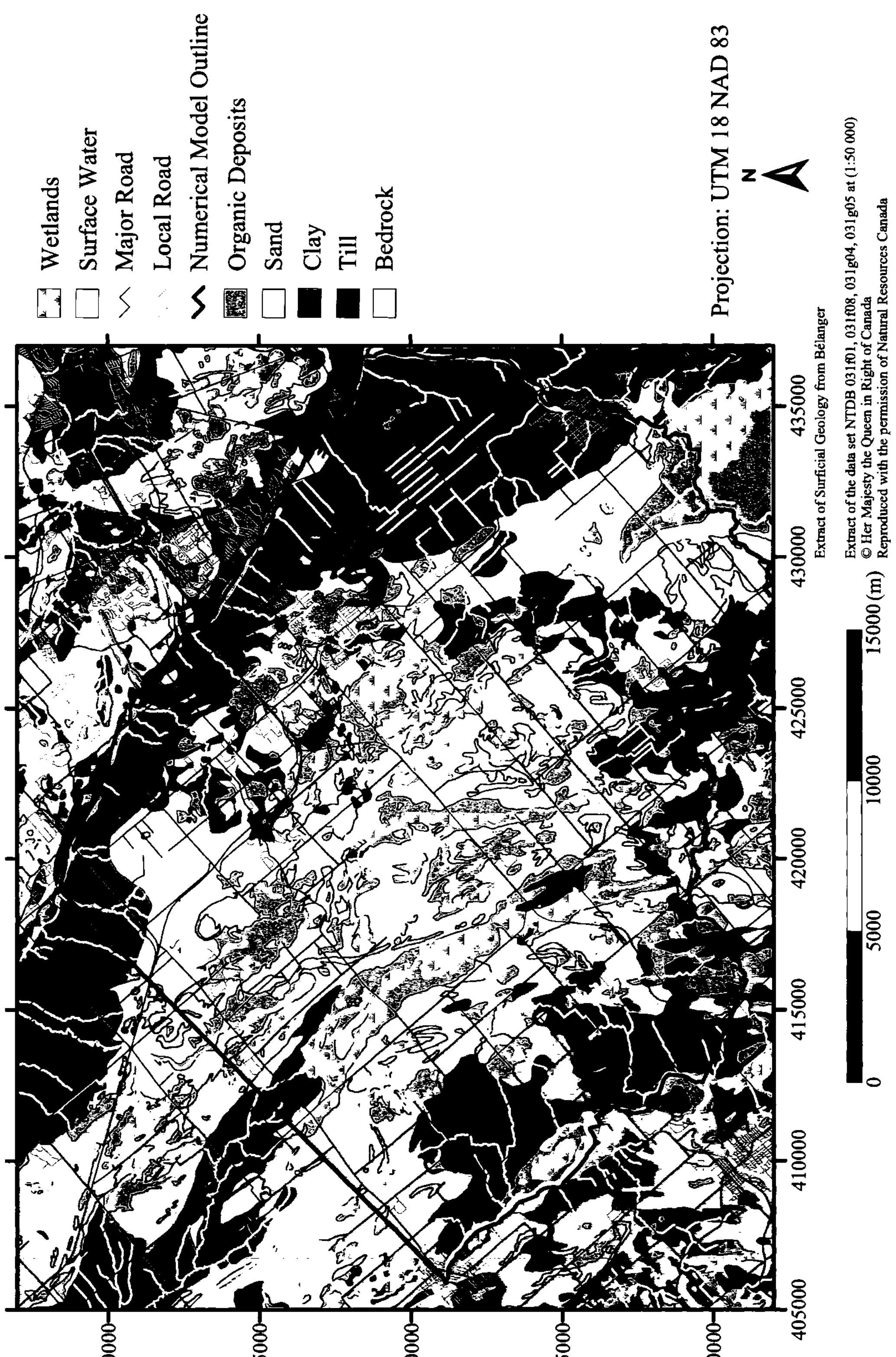


Figure 6-15: Hydraulic conductivity distribution for regional numerical model

Table 6-4: Hydraulic conductivity ranges from local studies

Material		Conductivity (m/s)
Clay	Max Min	1×10^{-6} 1×10^{-10}
Till	Max Min	1×10^{-6} 1×10^{-9}
Sand & Gravel	Max Min	1.01 1×10^{-5}
Sand	Max Min	1×10^{-4} 1×10^{-5}
Limestone and Dolostone	Max Min	9×10^{-4} 1.8×10^{-11}
Sandstone	Max Min	1.7×10^{-4} 1×10^{-8}

As is easily noticed from this table, the range for each of the values is very large due to the heterogeneity of the surficial soils. The maximum and minimum values served as an indication of the range of values to be considered for the overburden K values when calibrating the model.

The K values from different studies in the case study area associated with the Paleozoic bedrock can also be seen in Table 6-4. Similar to the surficial sediments, the Paleozoic bedrock show a wide range of values indicating the heterogeneity of the bedrock in the area. These values were used to establish a reasonable value for layers 2, 3, and 4 representing the upper Paleozoic limestone and dolostone, and layer 5 representing the Nepean sandstone aquifer. The values chosen were refined during the calibration process of the model. The local heterogeneities of K including fractures were not represented due to both scale factors and lack of specific information. The assumption made in this case is one of an equivalent porous medium in which one value of K will satisfy the entire formation. This assumption may not be valid at a finer resolution, but should be applicable in the present case. With this in mind, a single K value was used for the 3 layers representing the upper Paleozoic formations (layers 2, 3, and 4), and a different K value was used for the sandstone (layer 5).

In order to simplify the model, and due to a lack of specific information, there was no distinction made between horizontal and vertical hydraulic conductivity. There was also assumed to be no anisotropy in the K values in the horizontal direction.

One final consideration for the hydraulic conductivity in the numerical model was the presence of faults and fractures. There are two large faults under the Carp and Mississippi Rivers however they fall outside of the domain of the numerical model. Very little is known about the locations of localized faulting and the effects on groundwater movement. As described in section 6.2.1.1.3, local groundwater studies on contaminant plumes in localized areas have found that there is little effect on groundwater movement due to faulting (Golder, 2003b). Based on this observation and a lack of specific faulting information for the area, no faults were included in the bedrock for the numerical model. This assumption should be examined more closely as more information is gathered in the area because faulting and fractures can influence groundwater flow and the K values in both the horizontal and vertical directions.

The porosity for the different layers was also included in the numerical model. The porosity values were assigned to the different hydrological units with the values seen in

Table 6-5. The values given to the overburden were based on a typical conservative value of 0.25 based on a sandy silt or silty sand till (Golder, 2003b). This uniform value was given to the overburden materials because little information is known about the individual units and because the values do not play a large role in groundwater movement. If contaminant transport was to be considered by the numerical model, the porosity of the overburden, as well as the bedrock, would play a much more important role.

Table 6-5: Effective porosity assigned to different units and layers in numerical model

	Layer	Effective Porosity
Till	1	0.25
Clay	1	0.25
Sand	1	0.25
Wetland (Organics)	1	0.25
Upper Paleozoic	1, 2, 3, 4	0.001
Sandstone	5	0.02

The entire limestone upper Paleozoic formations were assigned a porosity of 0.001. This was based on reported joint spacing and the bulk hydraulic conductivity in previous studies (Golder, 2003b). The Nepean sandstone was given an effective porosity of 0.02. This larger porosity reflects the large amount of fractures present in the sandstone formation and the higher hydraulic conductivity value (Golder, 2003b).

6.4 Calibration

The numerical models were calibrated by adjusting the recharge, hydraulic conductivity values and the constant head boundary conditions until there was a reasonable match between the measured head values and those predicted by the numerical model.

The constant head boundaries in the top layer could not be changed as they represented the water levels in the different rivers. The constant head boundaries in the sandstone aquifer (layer 5) were adjusted to match measured values from the MOE WWIS.

The numerical models were calibrated at steady state using the Conjugate Gradient Solver supplied with Visual MODFLOW. Large water removals including the dewatering of the quarries were not considered during the calibration process so that a baseline could be established. This baseline could then be compared to different dewatering scenarios in order to demonstrate the effects of dewatering the quarries.

6.4.1 Hydraulic Head

The calibration results comparing the observed and calculated heads for the modelled area can be seen in Figure 6-16 and Figure 6-17 where the simulated groundwater elevations are compared to observed values. A spatial representation of the residuals from the calibration can be seen in Appendix G. The calibration data were based on 2279 wells located within the 5 layers. As a measure of the effectiveness of the calibration, Visual MODFLOW provides some descriptive statistics. Of particular interest are the absolute residual mean and the normalized root mean square (RMS) for the data. In this case, the residual is defined as the difference between the calculated and observed results.

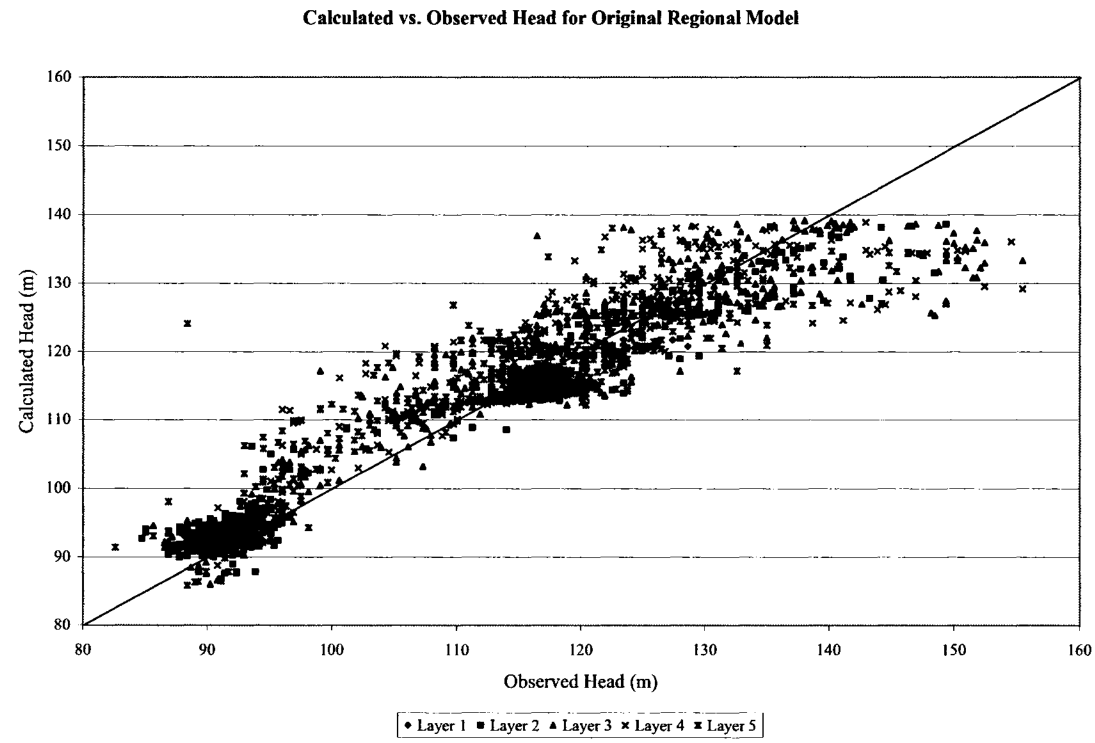


Figure 6-16: Calculated vs. observed heads for original regional model at steady state

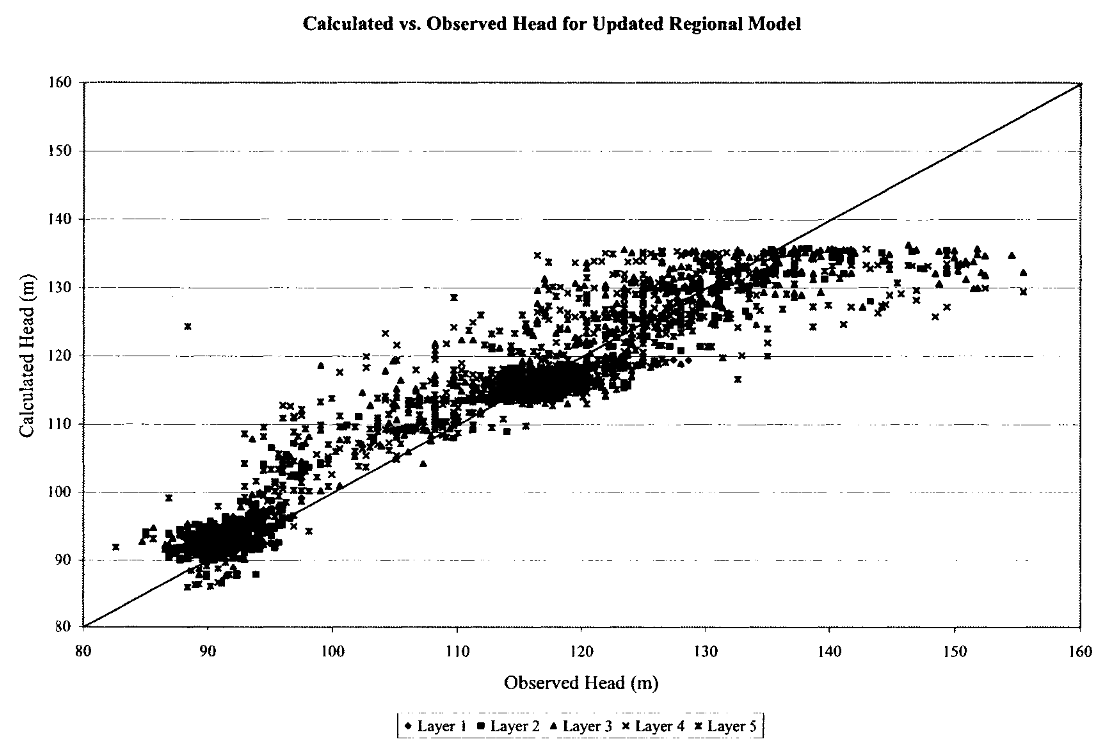


Figure 6-17: Calculated vs. observed heads for updated regional model at steady state

The absolute residual mean is calculated by the following:

$$\left|\overline{R}\right| = \frac{1}{n}\sum_{j=1}^{n}\left|R_j\right| \qquad (6\text{-}1)$$

Where $\left|\overline{R}\right|$ is the absolute residual mean, n is the number of wells and $\left|R_j\right|$ is the absolute value of the residual at each individual well. The absolute residual mean gives an indication of the average magnitude of the residuals and should be minimized.

The normalized RMS is the RMS of the data normalized by the maximum difference in observed values. It also is used to determine if the calculated and observed values are similar and is presented as a percentage. It is defined by the following:

$$\text{Normalized RMS} = \frac{\sqrt{\frac{1}{n}\sum_{j=1}^{n} R_j^2}}{\left(X_{obs}\right)_{\max} - \left(X_{obs}\right)_{\min}} \times 100\% \qquad (6\text{-}2)$$

Where n is the number of wells, R_j is the residual calculated for each well, and $(X_{obs})_{max}$ and $(X_{obs})_{min}$ are the maximum and minimum heads recorded in the area. Similar to the absolute residual mean, this value should be minimized in a calibrated model.

The absolute residual mean for the original model was 3.82 m and the normalized RMS was 7.21 %. For the updated regional model, simulating the wetlands in the area, the absolute residual mean was 3.73 m and the normalized RMS was 7.24 %. As can be seen in Figures 6-16 and 6-17, there does not appear to be a strong bias either above or below the measured values. However the calibration does not appear to be as good at the higher observed head values where the models under-predict the observed heads. In an attempt to address this situation the hydraulic conductivity of both the upper Paleozoic formations and the sandstone aquifer (layers 2, 3, 4, and 5) were decreased in order to raise the head in the areas where the model was under-predicting. Instead of just addressing the outlying points, all of the calculated heads increased over the whole domain causing the entire model to over-predict the heads. These results can be seen in Appendix G.

Due to the heterogeneity of the K values in the upper Paleozoic rocks, and the fairly large discretization (100m by 100m) and thickness of layers, the numerical model cannot match all of the observed heads. The areas where the heads are being under-predicted tend to be in areas of topographic high points. The observed heads cannot be matched at the topographic highs without flooding the topographically low areas.

Calibration using K, recharge, and constant head boundary conditions could not reflect the local regional heterogeneity. The assumption of a homogeneous medium to represent the bedrock may contribute to the calibration difficulties at areas with localized groundwater highs. Faults and fractures were not considered in the bedrock of the numerical model. These faults could serve as conduits for groundwater flow within the area contributing to local groundwater highs and lows. The heterogeneities caused by the faults could alter the flow within the area.

Another factor contributing to the calibration of the model is the inaccuracies of the MOE WWIS. The water well records are a historical record of all of the wells drilled in the province. Although every effort is made to only use the information from wells of known location and accurate elevations, measurement errors may exist in the database. Since the

MOE WWIS database is a historical record, the measurements taken reflect a snapshot of the time of the measurement at each individual well. Variations in water levels may occur over time due to droughts or floods and due to pumping. A well drilled in a drought could have a very different water level compared to the same well drilled during a period of plentiful water. These variations could reasonably be of the order of a meter or more. These variations are not reflected in the MOE WWIS data and as such an absolutely perfect calibration can never be achieved. The calibration obtained for the case study is reasonable and can be used to investigate the effects of dewatering the quarries on the aquifers.

The final values that were given to the different recharge areas can be seen in Table 6-6 and the final values determined for the hydraulic conductivity of the different units in the calibrated model can be seen in Table 6-7. These values were estimated from existing reports as well as from the model calibration process and the sensitivity analysis.

Table 6-6: Recharge assigned to different units in numerical model

	Original Regional Model	Improved Regional Model
Surficial Sediment	Recharge (mm/yr)	Recharge (mm/yr)
Till	5	5
Clay	5	5
Sand	30	100
Bedrock	200	120
Swamp (Organics)	60	60

In order to investigate the numerical models more closely, a sensitivity analysis was performed on both of the calibrated models. The details of the sensitivity analysis can be seen in Appendix H.

Table 6-7: Hydraulic conductivities assigned to different units and layers in numerical model

	Original Regional Model	Improved Regional Model
	Hydraulic Conductivity (m/s)	Hydraulic Conductivity (m/s)
Till	$1x10^{-8}$	$1x10^{-7}$
Clay	$1x10^{-8}$	$4x10^{-9}$
Sand	$5x10^{-4}$	$5x10^{-4}$
Swamp (Organics)	$5x10^{-7}$	$5x10^{-7}$
Upper Paleozoic	$5x10^{-6}$	$5x10^{-6}$
Sandstone	$1.3x10^{-4}$	$5x10^{-5}$

From the sensitivity analysis, it was found that the models were very sensitive to the hydraulic conductivity of the Nepean sandstone formation (layer 5). An increase in the K by a factor of 2 increased the calibration statistics by almost 50% while lowering the heads in the entire model. A decrease in the K by a factor of 2 increased the calibration statistics by approximately 100% in the original model and 50% in the updated model and increased the heads in both models. Other factors that influenced the models, but to a lesser degree, were the recharge values assigned to the bedrock units on layer 1 of both the original and updated numerical models, and the recharge value to the sand in the updated model. Doubling the recharge to the bedrock resulted in an increase in the heads of the models and an increase of almost 100% in the calibration statistics of the original model and 10% in the updated model. Halving the value lowered the heads in the models and increased the statistics by 30% in the original model and 10% in the updated model. Doubling the recharge to the sand in the updated model increased the calibration statistics by 18% due to an increase in the heads. Halving the calibrated value of the recharge to the sand led to an increase of only 3%. In all of the above cases, the model became less calibrated due to the changes in the assigned values further indicating that the numerical model was a good representation of the case study area.

The calculated heads for all of the layers of the updated regional model can be seen in Figures 6-18 to 6-22. These show that there is little vertical gradient between the lower sandstone and the overburden. They show that the basic flow of groundwater in the model is similar to that determined in the conceptual model (Figure 6-5). There are also some groundwater peaks in the second layer of the numerical model between the wetlands similar to those in Figure 6-5; however, the measured elevation of the mounds could not be matched by the model.

6.4.2 Local Scale Model

In order to add credibility to the updated calibrated model, a number of pump tests were simulated on a much more local scale. The area studied can be seen in Figure 6-23. The boundaries were chosen and constant head boundaries were assigned based on the results of the steady state updated regional numerical model (Figure 6-24). Since there is very little vertical gradient, the same constant head boundary conditions were assigned to each

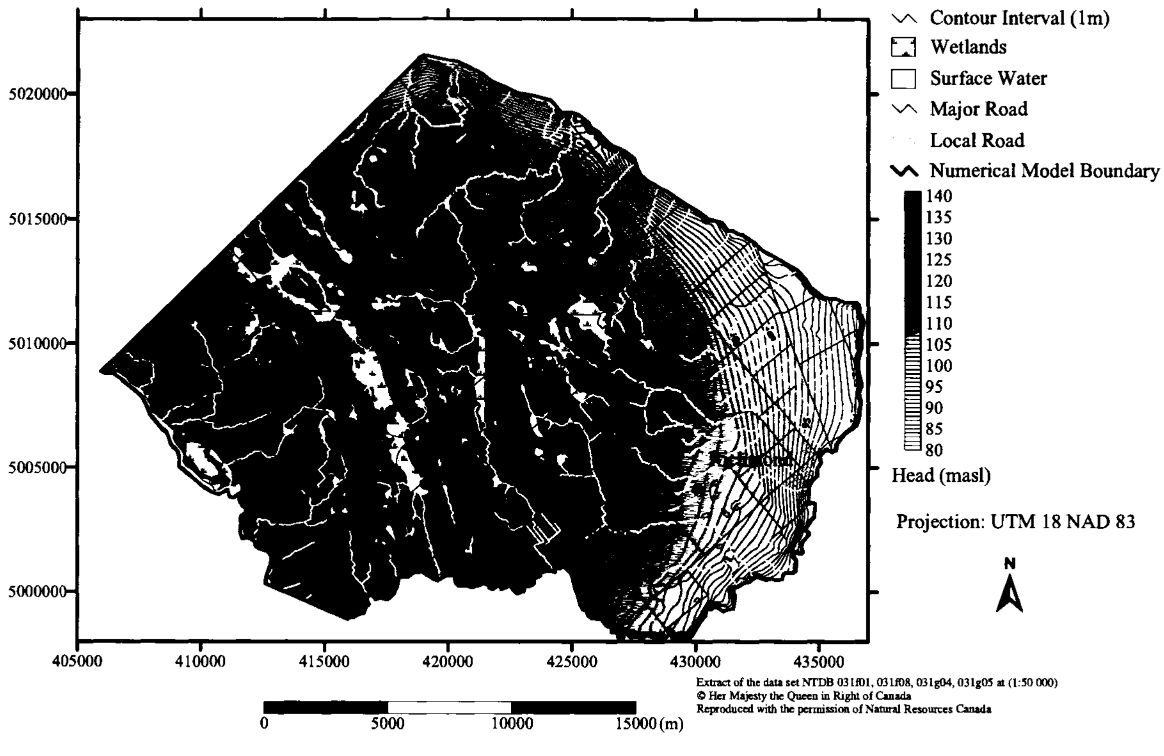


Figure 6-18: Calculated head in layer 1

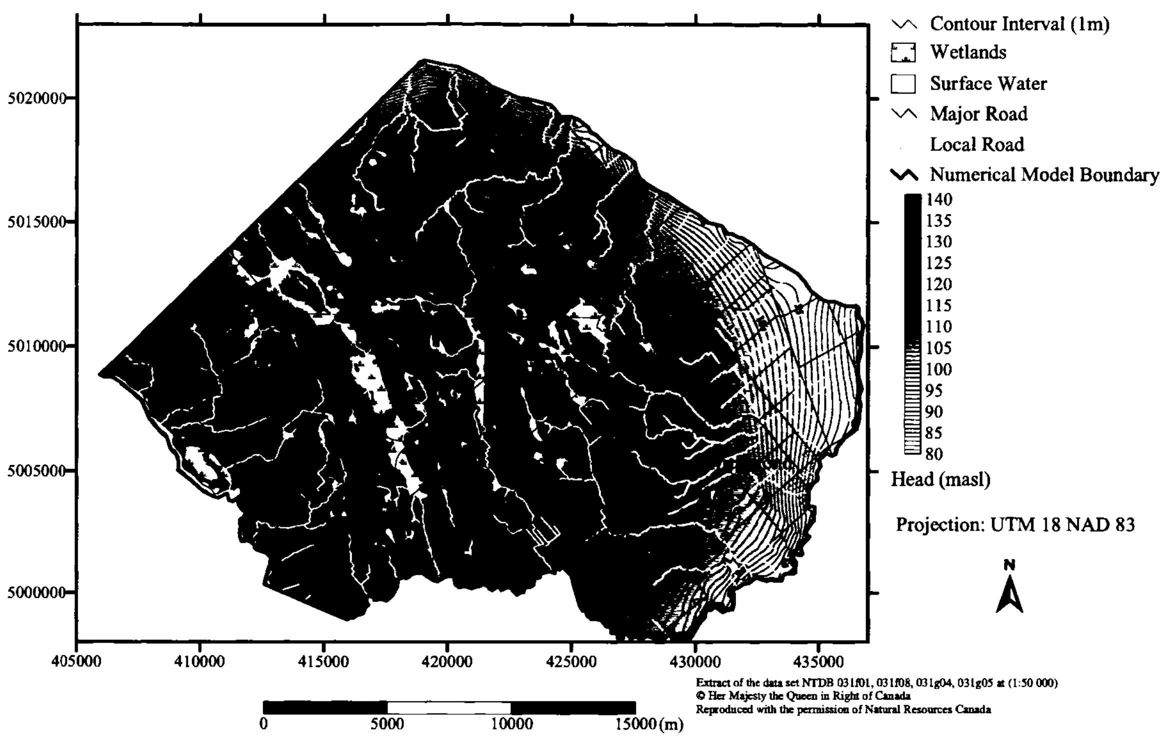


Figure 6-19: Calculated head in layer 2

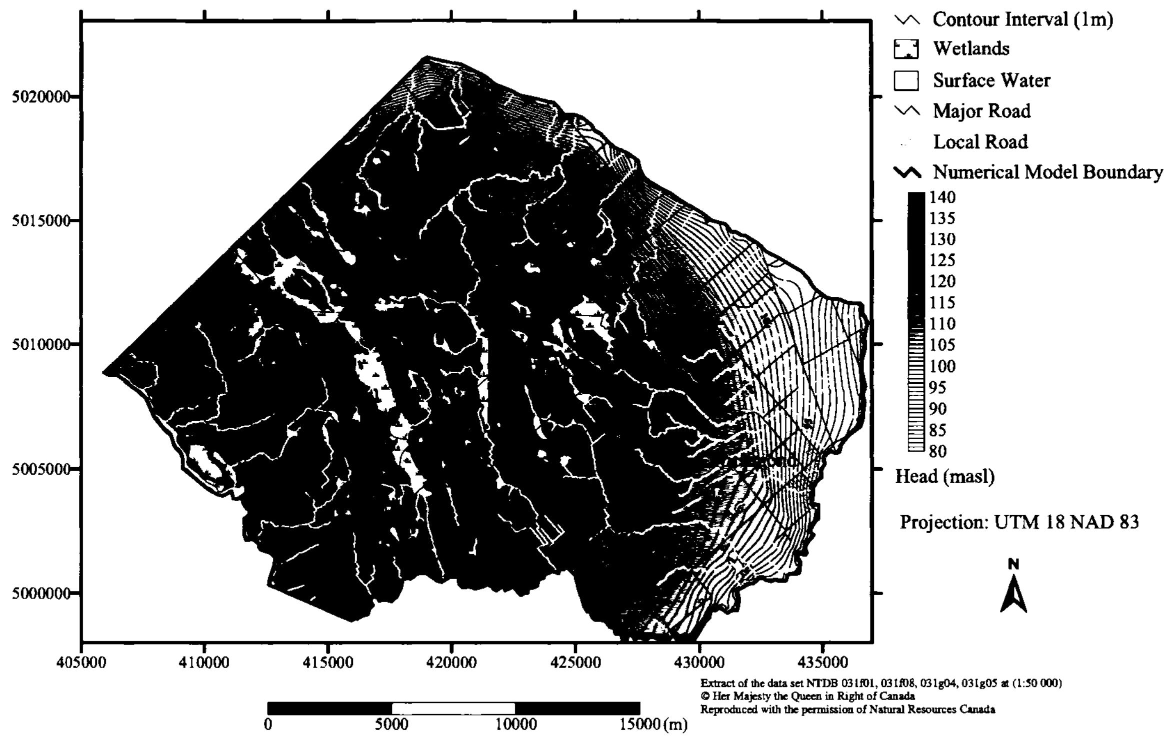


Figure 6-20: Calculated head in layer 3

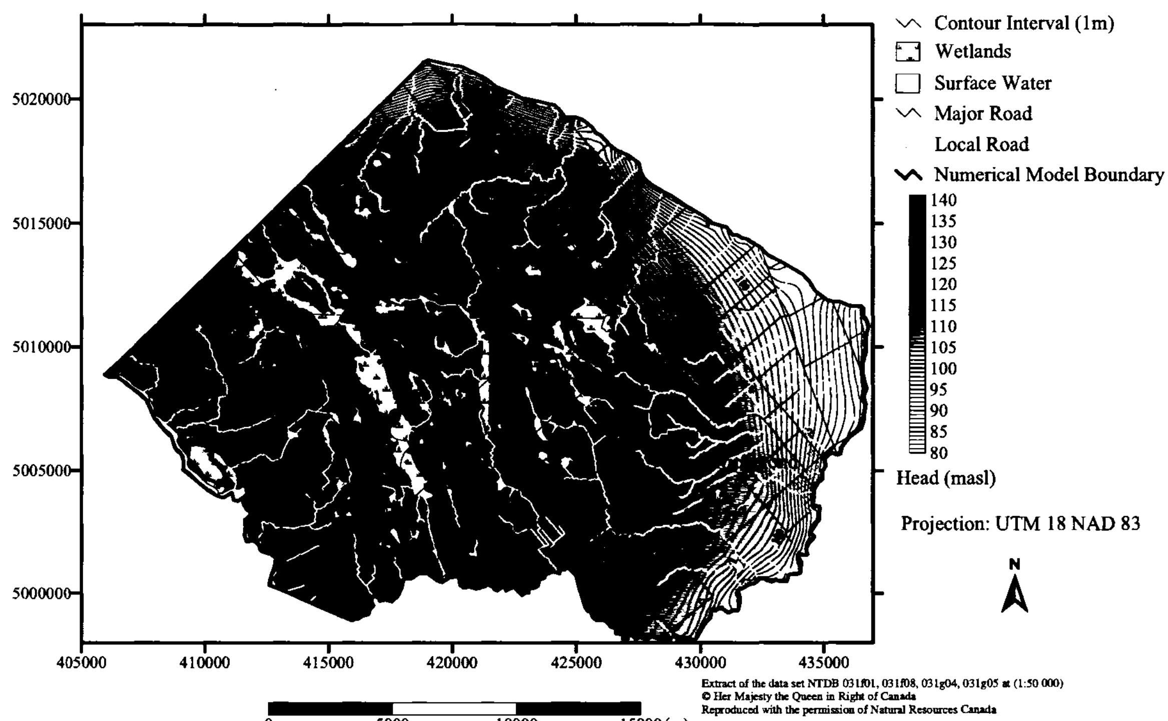


Figure 6-21: Calculated head in layer 4

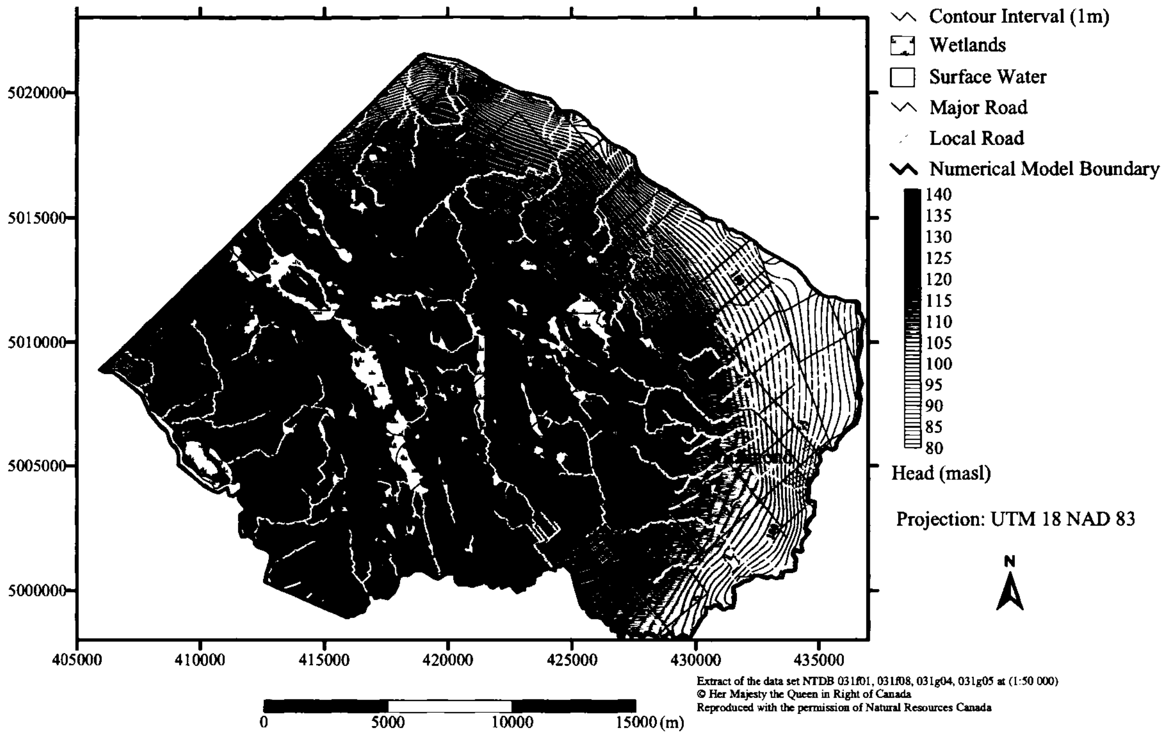


Figure 6-22: Calculated head in layer 5

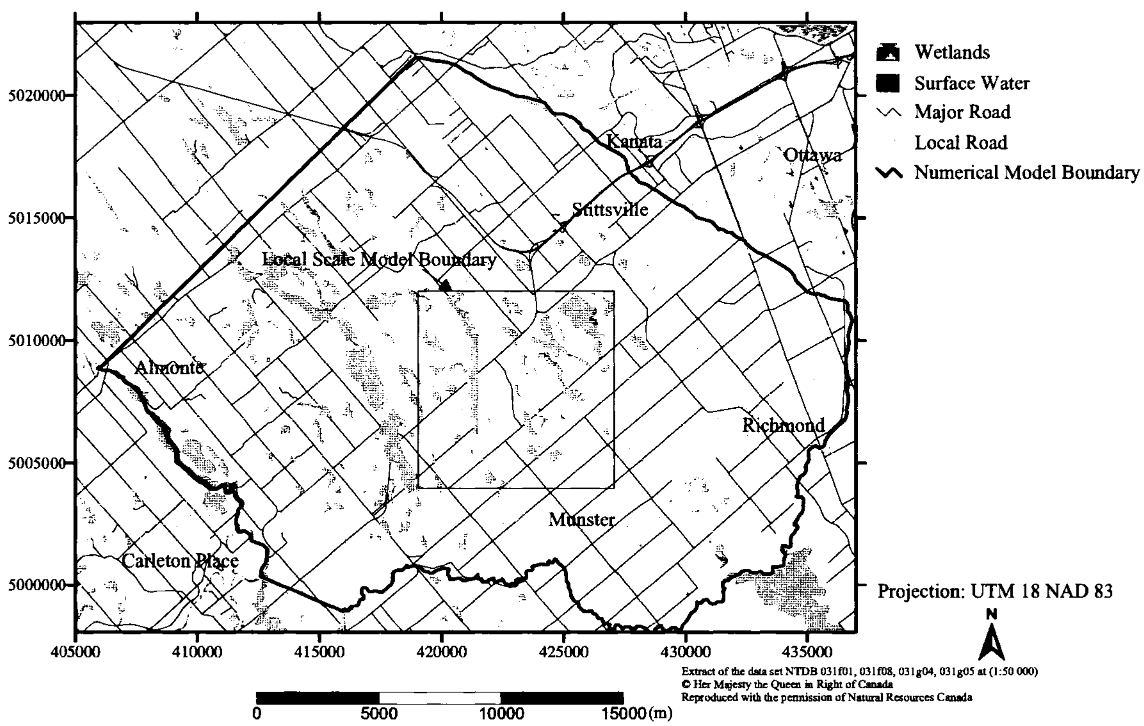


Figure 6-23: Local scale numerical model boundary

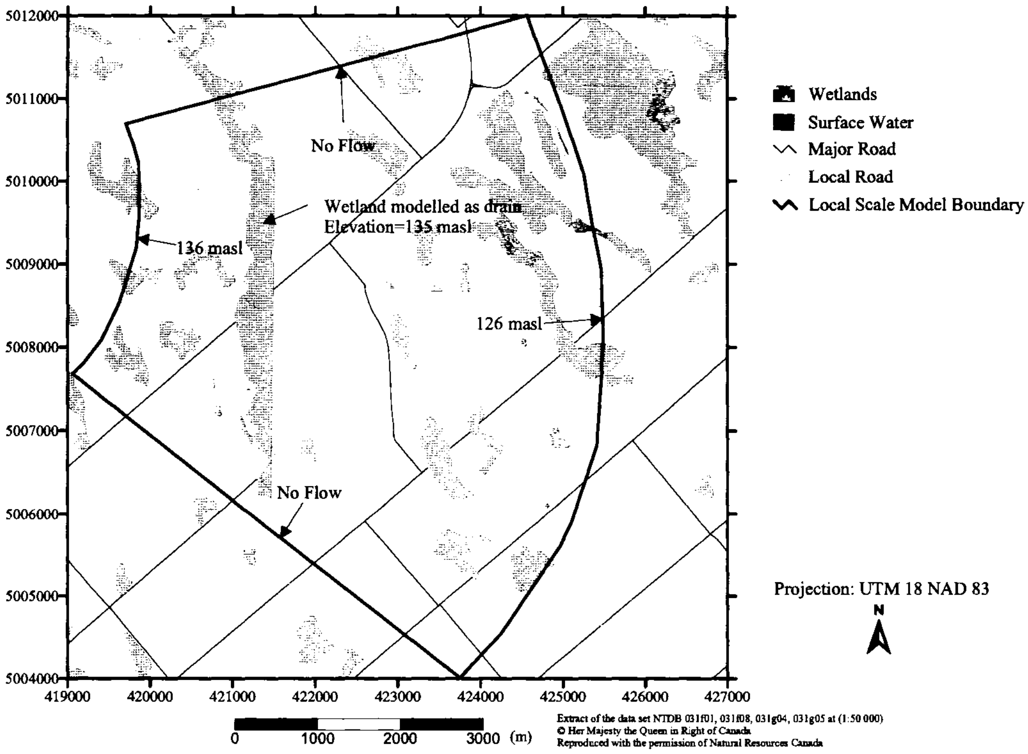


Figure 6-24: Local scale model constant head boundaries

layer of the local scale numerical model. Recharge was assigned to the uppermost layer with the same values and distribution as the calibrated regional model. The major wetland in the area was also simulated in the local scale model as a drain with an elevation of 135 masl.

Since the overburden is very thin in the area of interest, the model was simplified from the regional model by only including the 4 bedrock layers and eliminating the overburden layer. The aggregate is located in the top 2 layers of the numerical model. The thicknesses of these layers were identical to those in the regional numerical model. In order to capture the impacts of the pump tests on the different wells, a smaller discretization had to be established in the horizontal direction. A 20 m by 20 m grid was assigned to the entire model. The total number of cells on each layer was 160,000 and the total number of cells for the entire model was 640,000.

There were 2 pump tests simulated using the local scale model under transient conditions. The first was done in the Henderson quarry as part of its PTTW application. The second 'pump test' was done in the Beagle Club quarry. This second pump test was not a pump test using a well, but was the dewatering of the quarry in the spring so that it could be used throughout the summer. This was essentially seen as a pump test on a very large scale, and was very applicable to the case study. The locations of the quarries and the test wells can be seen in Figure 6-25.

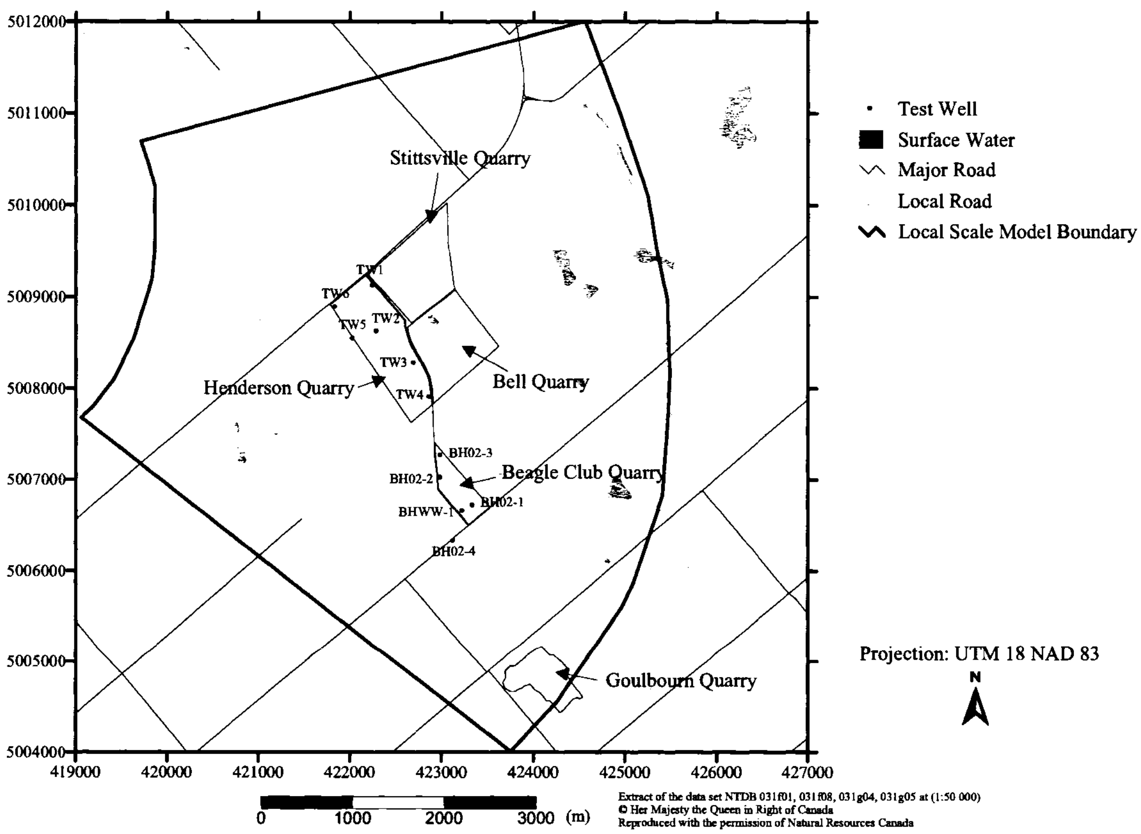


Figure 6-25: Local scale model quarry and well positions

6.4.2.1 Henderson Quarry

The pump test was simulated for the Henderson quarry based on one of the pump tests conducted to complete the PTTW application. The pump test consisted of pumping at well TW-5 for approximately 214 hours, with water levels taken at all of the wells (TW-1, TW-2, TW-3, TW-4, TW-5, and TW-6) at regular intervals. At one point during the pump test, the flow from the pump was increased from 30 L/min to 60 L/min. Although this is not usual procedure during a pump test, it does give interesting drawdown results that can be simulated with the numerical model.

The wells were simulated in Visual MODFLOW using the well package. The observation points for each of the wells were placed in the centre of the cell containing the bottom of the well. In all cases, this was layer 2. As a result of this assumption, the calculated heads from the pump test were those calculated in layer 2.

The calculated drawdowns due to the pump test were obtained by adjusting the K values for the upper Paleozoic bedrock (layers 1, 2, and 3) under transient conditions in order to match the observed drawdowns at each of the wells. The maximum calculated and observed drawdowns can be seen in Table 6-8.

The results can be seen in Appendix I. It can be seen that there is a reasonable amount of similarity between the observed and modelled drawdowns. In most cases, with the exception of TW-4, the calculated drawdowns are well within 1 m from the observed drawdowns. TW-4 had negative drawdowns during the pump test and as a result the calculated values do not agree with those observed. The differences between the observed and calculated drawdowns can be explained in a number of ways. The first is the heterogeneity of the upper limestone. There can be such a wide range of K values in a relatively small area that the assumption of an equivalent homogeneous medium cannot match observed measurements. Similar to the regional model, localized faulting may play a role in groundwater flow that is not accounted for in the numerical model.

Table 6-8: Comparison of maximum calculated and observed drawdowns for the Henderson quarry pump test

Well	Max Observed Drawdown (m)	Max Calculated Drawdown (m)
TW1	0.12	0.04
TW2	0.5	0.05
TW3	0.38	0.02
TW4	0	0.03
TW5	11.21	9.93
TW6	0.96	0.04

Another issue is that the gradient within the model can not match the observed gradient. This is due in part to the thickness of the layers providing too coarse of a mesh, and to the fact that the Henderson quarry sits on a localized mound of groundwater that is not captured by the regional model possibly due to the heterogeneities of the local bedrock not captured in the regional model. The test wells measure a gradient of 6 m from TW-2 to TW-5. This is a local gradient from the east to the west of the model that cannot be seen in Figure 6-5. Obviously this disagrees with the regional scale model where the flow of water is from the west to the east. The localized gradient does appear within the confines of the local model due to the presence of the drain representing the wetland but to a much lesser extent. This modelled gradient is only of the order of magnitude of 1m because the measured height of the localized mound cannot be reproduced. As a result of this the drawdowns calculated by the numerical model are less than those observed.

The hydraulic conductivity values established for the local scale model of the Henderson quarry pump test were $8.5x10^{-7}$ m/s for the upper Paleozoic layers and $5x10^{-5}$ m/s for the Nepean sandstone layer. The value for the upper Paleozoic layers is approximately half of one order of magnitude less than those obtained for the calibrated regional model.

6.4.2.2 Beagle Club Quarry

The 'pump test' for the Beagle Club quarry was modelled in a very similar way to that of the Henderson quarry except that there was no pumping well. The quarry was dewatered in the spring of 2004 and during this process, the water levels in the surrounding monitoring wells as well as the water level in the quarry itself were recorded at regular intervals. The flow rate from the pump was also recorded (Appendix I). Using the information, the dewatering of the quarry was simulated in Visual MODFLOW. The same initial four layer local scale model used for the Henderson quarry pump test was used.

The monitoring wells were modelled using the MODFLOW well package. Wells BH02-1, BH02-2, BH02-3, and BH02-4 were all screened in the second layer of the model whereas BHWW-1 was screened in the top layer. Each of the wells in the actual pump test were a multi level well, but the different screened elevations of the wells all fell within the same layer when they were simulated in MODFLOW. As a result of this, only one head value was calculated for each well compared to three measured values.

The quarry itself was simulated as a drain with a conductance of 10,000 m^2/hr. This would allow the free flow of water to the drain from the surrounding formation. The level of the drain was set to the recorded elevation of the water level in the quarry. As the water level dropped, the simulated elevation of the drain dropped. Before and after the draining of the quarry, the elevation of the quarry was set at 2000 masl to ensure that there would be no flow into the drain. Since the bottom elevation of the quarry was in layer 1 (the uppermost limestone layer), the drain was only assigned to layer 1 of the model.

The calibration of the local model was done the same way as for the simulation of the Henderson pump test. The drawdowns in the monitoring wells were adjusted by increasing and decreasing the K of the upper Paleozoic formations (Layers 1, 2, and 3) until the calculated values were similar to the observed values. The results from this model can be seen in Appendix I.

As can be seen from the figures in Appendix I, the drawdown data are a reasonable approximation for all of the wells except BH02-1 where the model predicts a much greater drawdown and BH02-3 where the drawdown is under-predicted. A comparison of the maximum drawdowns for the Beagle Club quarry pump test can be seen in Table 6-9.

Table 6-9: Comparison of maximum observed and calculated drawdowns for the Beagle Club quarry pump test

Well	Max Observed Drawdown (m)	Max Calculated Drawdown (m)
BH02-1A	1.21	2.19
BH02-1B	1.03	
BH02-1C	1.26	
BH02-2A	4.37	1.89
BH02-2B	2.16	
BH02-2C	2.65	
BH02-3A	2.61	1.39
BH02-3B	3.00	
BH02-3C	2.51	
BH02-4A	1.75	1.40
BH02-4B	1.43	
BH02-4C	1.47	
BHWW-1A	1.67	2.25
BHWW-1B	3.49	

Similar to the Henderson quarry pump test, matching the observed heads in the wells before the quarry was dewatered was very difficult because the water levels in the wells varied over 10 m between BH02-4 and BH02-3. This is due to the use of the regional model for the boundary conditions and from a local groundwater mound under the northern end of the quarry not reproduced by the regional numerical model. This was the other side of the mound that was illustrated at the Henderson quarry. Again the local changes in heterogeneity and faulting that cannot be captured by the relatively large discretization and application of a homogeneous porous medium in the regional numerical model cause the localized water table highs and lows to be missed.

The calibrated values for the Beagle Club quarry local model are $5x10^{-6}$ m/s and $5x10^{-5}$ m/s for the upper Paleozoic and the Nepean sandstone formations. The value obtained for the upper Paleozoic formation is the same as that obtained in the updated regional model.

6.4.3 Surface Water Balance

As a final calibration parameter for the updated regional model, the flows simulated from groundwater to the rivers surrounding the regional model were compared to actual flows in the river. As well, the drainage from the wetlands were also compared. This was used as a check to determine if the calibrated model was performing as was expected by the conceptual model. The only wetland that had values for comparison was the Huntley Wetland (Wetland 2, Figure 6-5). It is a large wetland located immediately west of the aggregate quarry sites. The detailed calculations and results obtained can be seen in Appendix J.

The flow from the groundwater to the rivers was estimated by first of all determining the flow in each of the rivers at two locations. This was done using flow metering stations monitored by Environment Canada (2001). An estimation of the groundwater flows to the river was determined by subtracting the upstream flow from the downstream flow. This gave an estimate of the total groundwater flow to the river from both sides. This value was divided by 2 to determine the flow from groundwater to one side of the river. Since the flow stations did not coincide with the boundaries of the numerical model, the average flow per kilometer was calculated by dividing the flow from one side of the river by the length of the river between the flow stations. The final value obtained was the flow to one side of the river from groundwater per kilometer of river. This value could then be compared to the value calculated by the numerical model based on the approximate length of the river in the model. The flow for the wetland is from a hydrologic study performed in the study area (CH2M Hill, 2002). The results can be seen in Table 6-10.

Table 6-10: Comparison of estimated and calculated river and wetland flows

	Estimated (m^3/s)	Modelled (m^3/s)	% Difference
Mississippi R.	1.121	0.174	-84.5
Carp R.	0.614	0.178	-71.1
Jock R.	2.206	1.871	-15.2
Huntley Wetland	0.038	0.013	-65.6

Although the numerical model under-predicts the flows to the different rivers from groundwater, most of the flows are of the same order of magnitude. Seeing as the estimation of the flows to the river based on the flow stations is fairly simplistic and makes many assumptions, the flows predicted by the calibrated model are of the same order of magnitude so can be considered reasonable.

Similarly even though the flow predicted by the model from the wetland is less than measured it is of the same order of magnitude. It is smaller because the numerical model could not reproduce the gradients adjacent to the wetland thus the flow predicted would be less. The flow from wetlands is also quite variable depending on the conditions present when the flow is measured. This inconsistency in flow may also contribute to the discrepancy between the measured and predicted flows.

6.5 Modelling Results

In order to illustrate some of the different uses of the numerical model for assessing PTTW applications, the aggregate quarries in the study area were simulated using drains in four different scenarios. The numerical model used for the steady state simulations was the updated regional model. For the transient simulations, the original calibrated regional model was used because when wetlands were incorporated into the model as drains, the transient regional models would not converge. After several attempts to determine the reason for the convergence problem, and after discussions with Waterloo Hydrogeologic Inc. (creators of Visual MODFLOW) and Christopher Neville of S.S. Papadopulos and Associates Inc. (a very experienced MODFLOW user) the problem could not be solved. It was anticipated that the problem was related to an issue with the initial heads used by the transient simulation. In order to calculate drawdowns, the initial heads assigned to all the cells in the transient model were those obtained from the solution of the regional model. It was thought that for the dry cells created in the steady state simulation, Visual MODFLOW might import them as active cells with a very large head (1×10^{30}m) and not as inactive cells. This would create problems for the solution of the transient simulation.

The first two scenarios modelled were steady state simulations where a single quarry and then all the quarries were dewatered to their maximum depth and kept empty. The third scenario was run under transient conditions where the quarries were dewatered for 6 months of the year, and left to fill for the remaining 6 months of the year. This was repeated over 30 years to see the effects on the water table in the area. In this scenario, the quarries were simulated at their maximum depths. The fourth and final scenario was again a transient simulation where the depths of the quarries increased over time, the dewatering only took place for 6 months of the year, and the quarries started their dewatering in different years, representing the different times that the quarries would have opened. For a comparison of the scenarios, a number of head observation wells were included in the model so that the drawdowns in different locations could be compared. The observation points and names of the different aggregate quarries can be seen in Figure 6-26, and the results for all of the simulations can be seen in Appendix K.

As can be seen in Figure 6-26, observation points were located in various communities around the case study area including Richmond, Munster, Stittsville, Tranquility Estates, and Lucas Lane. Observation points were also placed at increasing distances of 1 km (A), 2 km (B), 4 km (C), 6 km (D), 8 km (E), and 9 km (F), from the aggregate quarries. These were all put in place to examine the drawdown effects from dewatering the quarries.

It is important to remember that the results obtained are based on the information and assumptions that were entered in to the model as described in previous sections.

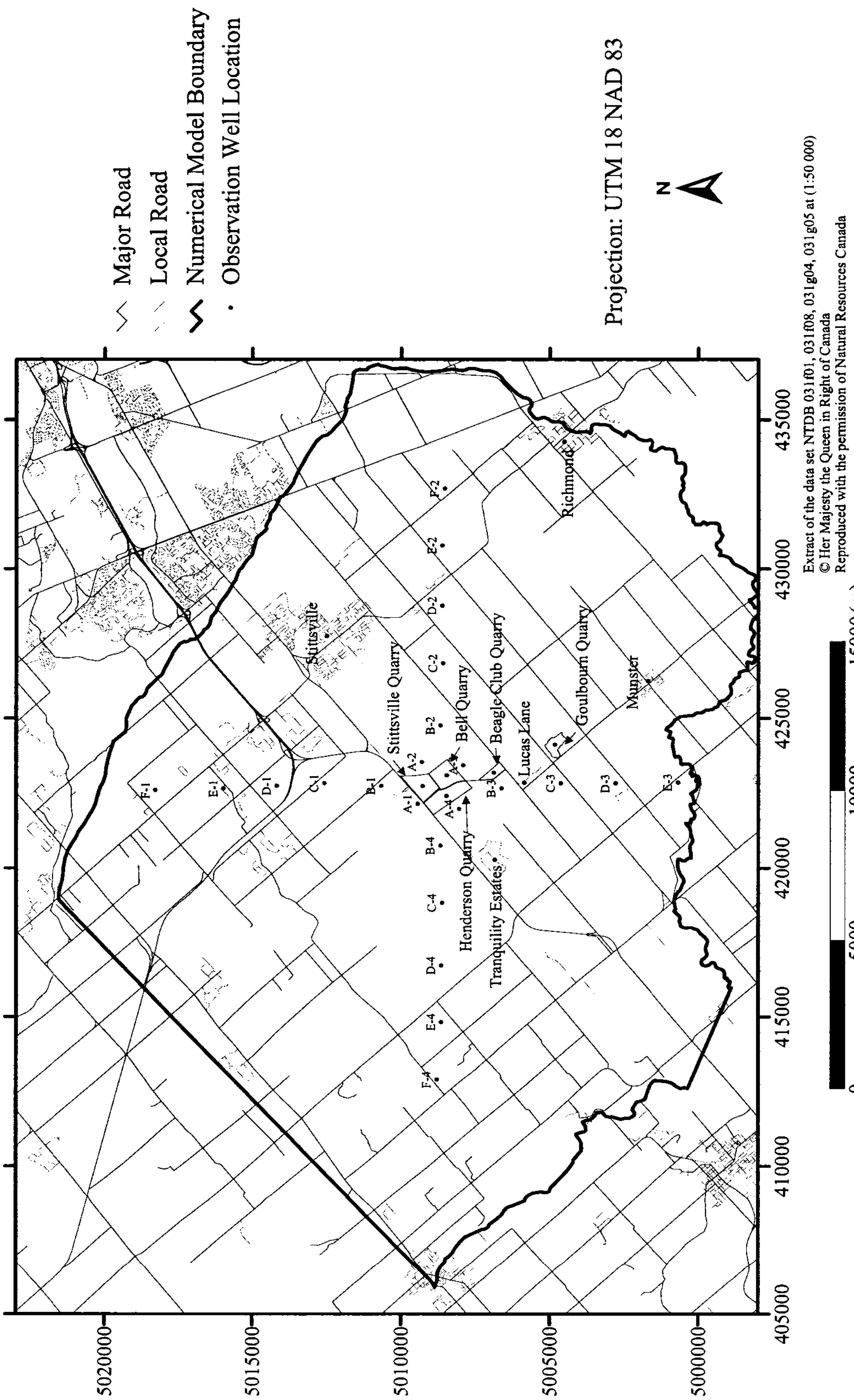


Figure 6-26: Observation well location in regional numerical model

Localized heterogeneities and faulting were not included in the regional model and the presence of these features may alter local groundwater flows. Including these heterogeneities in the numerical model may alter the results of the simulations. The results must be interpreted realizing that there is error inherent in the model from the different data inputs including hydrological layers, water well information, constant head boundaries, and pumping rates. Numerical models are in continuous need of improvement as new information is obtained and the numerical model is further refined.

6.5.1 Steady State

The first steady state scenario that was simulated was the case where only one aggregate quarry was dewatered. For the simulation, the updated regional numerical model was used and the Henderson quarry was simulated in layer 3 of the model as a drain with an elevation of 112 masl and a conductance of 10,000 m^2/day. The area covered by the drain was essentially the entire areal extent of the property owned by the aggregate quarry.

The flow entering the drain that represented the Henderson quarry was estimated by the simulation to be 11,365 m^3/day. This value is significantly higher than the 3,500 m^3/day determined in the hydrological assessment of the quarry site (Morey Houle Chevrier, 2001) and the 7,857 m^3/day on the PTTW for the site. Many factors play a part in this discrepancy including the different hydraulic conductivities and the evaluation technique since the original assessment did not include seepage from the bottom of the quarry or inflow from recharge.

The impact of the quarry dewatering on the surface water bodies in the area surrounding the site can be seen in Table 6-11. As can be seen there is an impact due to the dewatering of almost 1,000 m^3/day to the Carp River. The other two rivers are not as affected by the dewatering. Also the wetlands in the area are greatly affected by the dewatering causing two of them to go dry due to the drawdowns of the water table surrounding them. These are serious concerns that should be considered when assessing the PTTW applications. The impacts to the surface water bodies might be mitigated somewhat by modelling the impacts of the pumped water once it has reached the surface.

Table 6-11: Comparison of flows to and from the modelled water bodies

	One Quarry Dewatering		No Dewatering	
	Flow from river to domain (m^3/day)	Flow from domain to water body (m^3/day)	Flow from river to domain (m^3/day)	Flow from domain to water body (m^3/day)
Mississippi R.	30.87	13,617	27.6	13,744
Jock R.	80,506	161,400	80,491	161,630
Carp R.	0.19	14,227	0	15,348
Wetland 1		1,319.1		4,654.70
Wetland 2		0		1,125.10
Wetland 3		0		1,036.50

One further aspect that can be considered when looking at the dewatering of the quarry are the drawdowns that occur. The drawdowns for all the observation points can be seen in Table 6-12 and Figure 6-27 shows the drawdowns due to dewatering the Henderson quarry.

The largest drawdowns are near the quarry as expected and agree with the drawdowns predicted in the hydrogeological assessment. The hydrogeological assessment only considers wells very close (within 1km) to the quarry site and does not look at the regional effects. These drawdowns are considered to be acceptable for the wells in the immediate area (Morey Houle Chevrier, 2001).

Table 6-12: Calculated drawdowns from steady state simulation of the Henderson quarry

Well name	Drawdown (m)	Well Name	Drawdown (m)
A-1	14.00	E-1	0.60
A-2	11.68	E-2	1.40
A-3	14.09	E-3	0.32
A-4	15.75	E-4	0.07
B-1	8.11	F-1	-0.29
B-2	8.31	F-2	0.60
B-3	10.14	F-4	0.10
B-4	8.80	Tranquility Estates	6.00
C-1	3.92	Stittsville	2.31
C-2	4.21	Richmond	0.51
C-3	4.88	Munster	0.52
C-4	3.06	Lucas Lane	7.21
D-1	1.69	Stittsville Quarry	14.91
D-2	2.06	Henderson Quarry	21.92
D-3	2.14	Goulbourn Quarry	4.52
D-4	0.00	Bell Quarry	16.31
		Beagle Club Quarry	10.41

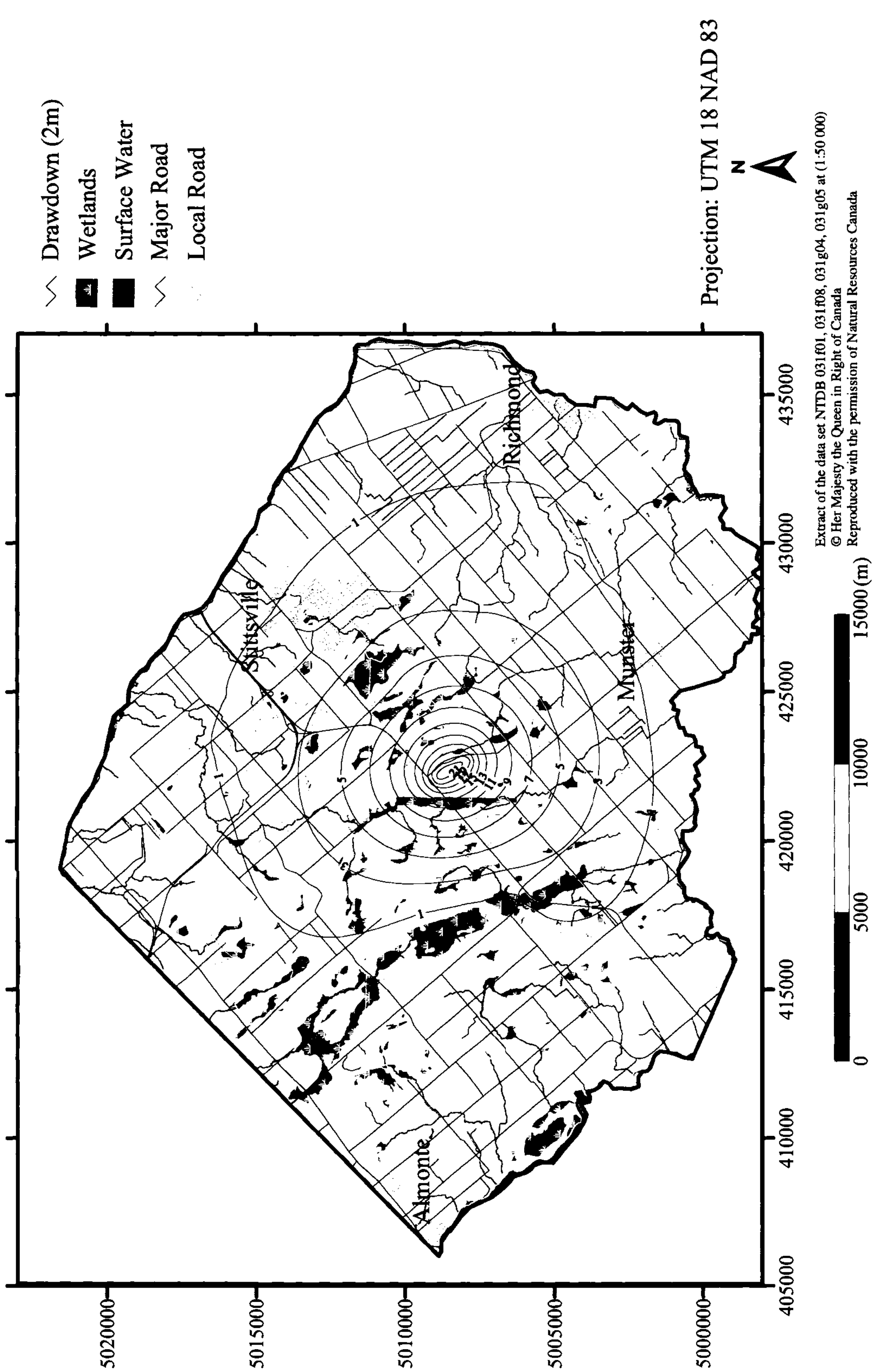


Figure 6-27: Drawdowns due to dewatering of the Henderson quarry

For the second steady state scenario, again using the updated regional numerical model, all of the aggregate quarries were modelled in Visual MODFLOW using the drain package with the elevation of the drain set at the maximum proposed depths of each quarry. The drains were situated in the layer where the final depth was located, and coverėd most of the footprint of the quarry property. This scenario would be the worst case situation for the cumulative effects caused by the dewatering of the quarries and would most likely never occur because the operators tend to allow the aggregate quarries to fill during the winter months. Also since the quarries all started their initial operations at different times, some may no longer be in use when others are just beginning to remove aggregate.

Generally, the modelled groundwater head from the calibrated regional numerical model in the area of the aggregate quarries is between 128 masl in the southern end (Goulbourn quarry) and 135 masl at the northern end (Henderson quarry). The flow of groundwater is generally downward from the surface to the Nepean sandstone aquifer, but this flow is very slow. What may occur, if the quarries dewater to their maximum depth as noted in Table 6-13, is a reversal in the flow of groundwater so that the flow is from the Nepean aquifer towards the surface.

Table 6-13: Approximate final depths of aggregate quarries in the regional model

Quarry	Final Depth (masl)	Layer of model
Beagle Club Quarry	107	3
Bell Quarry	131.5	2
Stittsville Quarry	110	3
Henderson Quarry	112	3
Goulbourn Quarry	110	3

Residents in the area have their water supplies in the upper Paleozoic formations and the water quality varies in the area. If the water in the Nepean sandstone aquifer or other areas is of lesser quality, a reversal of the vertical flow may cause the water quality of the wells in the upper Paleozoic limestone layers to deteriorate. Since the bottom levels of the aggregate quarries are below the piezometric water surface in the area, there will most certainly be a reversal of the vertical flow of water under the quarries.

The drawdowns obtained from the steady state simulation can be seen in Table 6-14. At steady state, the vertical downward flow changed to an upward flow in the vicinity directly under the aggregate quarries where the drawdown was up to 24 m. The observation wells also indicate that large drawdowns of 3 to 6 m exist up to 6 km (D observation wells) away.

The steady state flow from the quarries was estimated at 19,549 m^3/day which is significantly less than the maximum total permitted water withdrawal from all of the aggregate quarries combined (43,546 m^3/day). This is due to the fact that the quarries request a large maximum pumping rate so that the quarry can be quickly dewatered in the spring. It also accounts for precipitation and surface runoff into the quarry. Since the PTTWs were issued for each individual quarry, the cumulative effects of all the quarries are not considered. A drop in the water table will affect the amount of water flowing into the quarries.

The flow from each of the individual quarries can be seen in Table 6-15. One interesting observation from this table is that there is no flow from the Bell quarry. This is due to the relatively shallow depth of the quarry and the dewatering activities of the two quarries around it lowering the water table. Effects such as this should be investigated by water management planners to help in making planning decisions.

Table 6-14: Drawdowns from steady state simulation

Well name	Drawdown (m)	Well Name	Drawdown (m)
A-1	18.11	E-1	1.77
A-2	19.26	E-2	3.41
A-3	20.10	E-3	0.85
A-4	18.38	E-4	0.29
B-1	13.14	F-1	0.31
B-2	14.42	F-2	1.79
B-3	19.72	F-4	0.27
B-4	11.59	Tranquility Estates	9.58
C-1	6.74	Stittsville	4.39
C-2	8.57	Richmond	0.77
C-3	12.90	Munster	2.48
C-4	4.82	Lucas Lane	16.91
D-1	3.32	Stittsville Quarry	23.58
D-2	4.63	Henderson Quarry	22.21
D-3	6.07	Goulbourn Quarry	18.72
D-4	0.26	Bell Quarry	21.05
		Beagle Club Quarry	25.44

Other interesting observations in the steady state simulation of the case study area are the effect on the flow of groundwater to the rivers. The results compared to the steady state case with no quarry dewatering can be seen in Table 6-16. This table shows that in all cases the flows from groundwater to the rivers has decreased but the flow from the rivers to groundwater has remained essentially the same. The flows to the wetlands also changed significantly. In two cases, the flow to the wetlands was stopped entirely and the flow to the third wetland was reduced substantially due to the lowering of the water table.

Table 6-15: Flow from individual quarries at steady state

Quarry	Flow Out (m^3/day)
Henderson	3,609.8
Stittsville	5,819.5
Bell	0
Beagle Club	5,022.1
Goulbourn	5,097.3

Table 6-16: Comparison of flows to and from the modelled water bodies

	All Quarries Dewatering		No Quarry Dewatering	
	Flow from river to domain (m^3/day)	Flow from domain to water body (m^3/day)	Flow from river to domain (m^3/day)	Flow from domain to water body (m^3/day)
Mississippi R.	35.92	13,410	27.6	13,744
Jock R.	80,523	161,030	80,491	161,630
Carp R.	0.71	13,405	0	15,348
Wetland 1		348.60		4,654.70
Wetland 2		0		1,125.10
Wetland 3		0		1,036.50

All of these results can be seen in Figure 6-28 which shows the drawdowns calculated for the scenario using the numerical model. It can be seen that the heads have been significantly reduced under the aggregate quarries and all surrounding areas, especially to

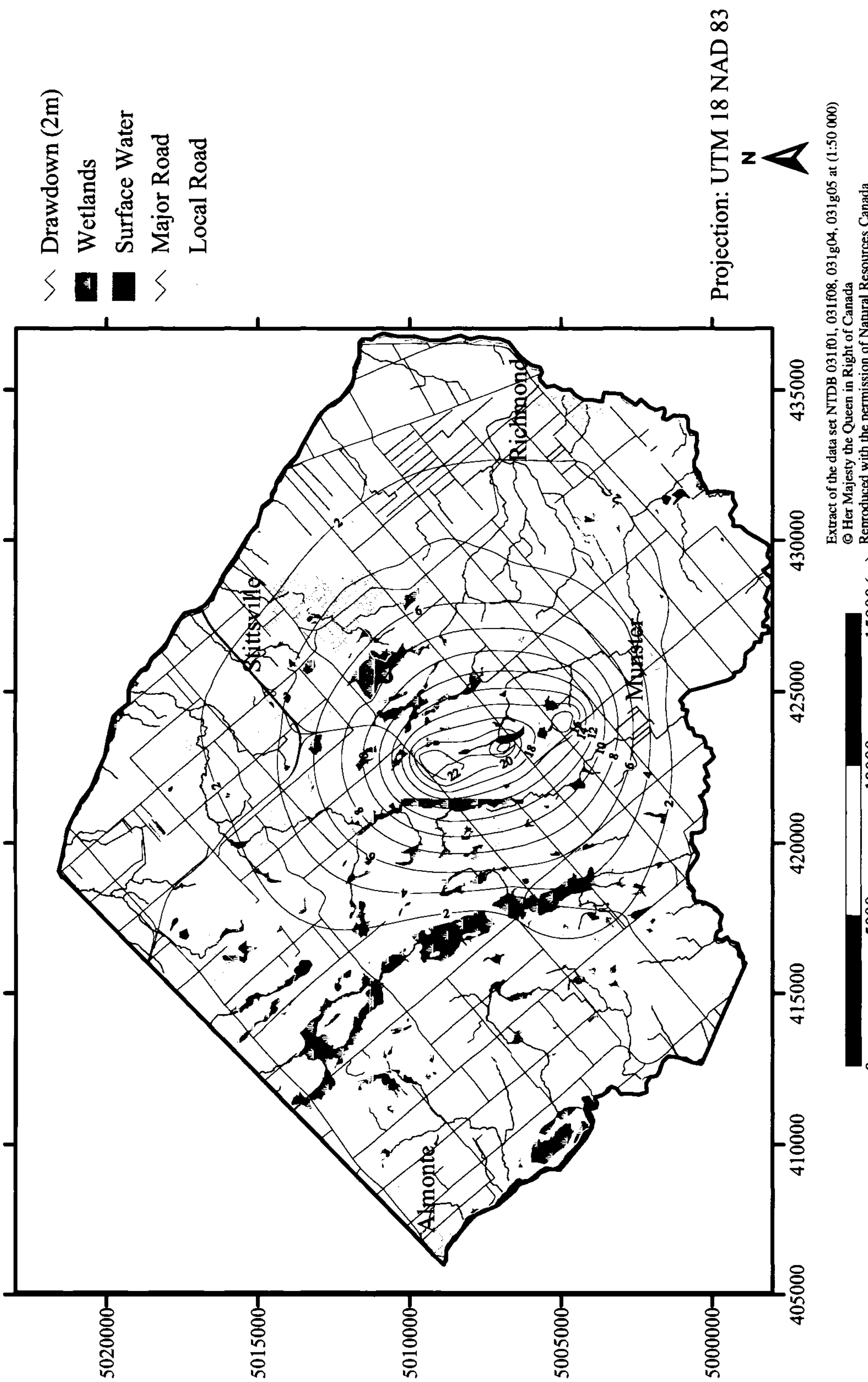


Figure 6-28: Drawdowns due to dewatering of all quarries

the south east. From the figure one can determine that the groundwater flow has been changed so that it does not move from west to east, but instead forms a ring around the area with the aggregate quarries so that the flow is all towards the aggregate quarries. In the vertical direction, the flow would be upwards from areas with high heads to the areas of lower head around the quarries.

As in the case of the first steady state scenario, the results obtained from this scenario was based on the boundary conditions and the conceptual model presented in earlier sections. There may be several heterogeneities in the area including faults or vertical fractures that were not captured by the numerical model. Including these heterogeneities in the numerical model could impact the results obtained from the model.

6.5.2 Transient Simulations

Two transient simulations were also created to illustrate the transient nature of the dewatering process and how drawdowns might be affected by this. The first transient simulation (Transient 1) demonstrated what might occur if all of the quarries were operating at their lowest level for only 6 months of the year for a number of years. For the remaining 6 months, the quarries were left to fill with groundwater. The simulation was done using the drain package in Visual MODFLOW to represent the quarries. The elevation of the drain was set at the lowest elevation of the quarry it represented, and was only active during the 6 months that the quarry was operating. The drain was inactive during the 6 months when the quarry was not operating. For this simulation all of the quarries started and stopped dewatering at the same time. The simulation was performed for 30 years to see if there were any long term drops in the water table due to the dewatering.

The second transient simulation (Transient 2) investigates the effect of the time it takes for an aggregate quarry to reach the bottom of the quarry. It was assumed that each quarry would be developed in three lifts. Each lift was one third of the total depth of the aggregate quarry. It was also assumed that each lift took 8 years to complete, so that the final depth of the quarry was reached in 24 years. This was modelled by lowering the drain elevation every 8 years during the simulation for each quarry. After the 24 years, the quarry was abandoned and allowed to fill throughout the entire year. It was also assumed that the quarries only operated during 6 months of the year thus allowing groundwater to fill the quarry during the other 6 months. The last aspect that was investigated during the simulation was the staggered development of the quarries. It was assumed for the scenario, that the aggregate quarries were developed three years apart so that for the first three years, only one was operating, for the second three years, two were operating and so on. The order of development was chosen based on the level of development of the existing quarries as the following: Goulbourn, Bell, Beagle Club, Stittsville, and Henderson. The input as well as detailed results can be seen in Appendix K.

The results show that the water table responds quickly to the different stresses placed upon it. The drawdowns respond almost immediately to when the drains are turned on and off. In the case of Transient 1, the drawdowns experienced at all of the monitoring wells are the same for each on/off cycle for each well. The water table does not return to its original position when the quarries are not dewatering. The water table is lowered by approximately 1m at most of the wells over the entire study area. This decrease in the level of the water table does not appear to worsen with time. There is not enough recharge to return the water table to its original position during the six months that the quarries are not operating. The wells that do not show any lowering of the water table are D-3, E-3, F-2, F-4, Munster, and Richmond. These wells are all sufficiently far away from the quarries (up to 10 km) so that the constant head boundaries might be influencing the rebound of the water table.

The size of the drawdowns when the drains are active is very similar to those from the steady state scenario with all the quarries dewatering. As expected, the drawdowns decrease as the distance increases from the aggregate quarry sites. Table 6-17 shows the results of the maximum drawdowns from the two transient simulations. As can be seen from the table, there is a small decrease (<1 m) in the maximum drawdowns in the Transient 2 simulation compared to Transient 1

The results from the Transient 2 simulation appear to be very similar to those of Transient 1 if only the maximum drawdowns are examined. Due to the slow development of the quarry as well as the different operational starts, the maximum drawdowns are only seen for very short periods of time. There also appears to be a decrease in the water table during the 36 years of the simulation (Appendix K) but it is

Table 6-17: Comparison of maximum drawdowns for transient simulations

	Maximum Drawdown (m)	
Well Name	Transient 1	Transient 2
A-1	18.16	18.04
A-2	16.22	15.82
A-3	16.93	16.25
A-4	17.27	16.93
B-1	13.51	13.22
B-2	13.02	12.08
B-3	16.68	15.49
B-4	12.87	12.49
C-1	8.93	8.59
C-2	8.74	7.69
C-3	11.04	10.31
C-4	7.94	7.53
D-1	6.93	6.63
D-2	5.89	5.10
D-3	5.52	5.14
D-4	5.21	4.90
E-1	5.50	5.25
E-2	4.37	3.76
E-3	0.62	0.57
E-4	3.63	3.40
F-1	4.03	3.87
F-2	2.00	1.64
F-4	1.29	1.17
Tranquility Estates	10.13	9.35
Stittsville	5.90	5.46
Richmond	0.90	0.80
Munster	2.68	2.47
Lucas Lane	14.32	13.05
Stittsville Quarry	20.75	20.74
Henderson Quarry	19.36	19.32
Goulbourn Quarry	14.38	14.32
Bell Quarry	18.20	17.86
Beagle Club Quarry	20.90	20.63

less than 1m and is quite variable with time as the different quarries come into operation and as their depths increase. The drop in the water table appears to improve as time increases and the quarries start to close.

As can be seen from these simulations, differences exist in the results obtained depending on the simulation chosen. As more information was included in the models, from steady state to transient, the maximum drawdowns were similar, but other effects such as the lowering of the water table and the different drawdowns at the observation wells with time appeared.

The results from the numerical modelling based on the conceptual model indicate that there are some very large drawdowns in the vicinity of the aggregate quarries and that the impact of these quarries when considered cumulatively extends well beyond the properties of the individual quarries. These regional effects were also seen when a single aggregate quarry was simulated using Visual MODFLOW but to not as large of an extent. The regional effects were not seen in any of the actual PTTW applications due to the local assessment performed. The observed effects may be impacted if more of the local heterogeneities were included within the numerical model as it was improved.

The information from the different simulations could be used by planners when making decisions about PTTW permits. Since there are so many variables that can be included in the numerical model, a set of guidelines needs to be developed for Ontario so that the models used to assess cumulative impacts of PTTW are similar throughout the province.

Chapter 7: Conclusions and Recommendations

This research presented a framework for assessing the cumulative effects of PTTWs on groundwater resources. The objectives that were originally set out in the thesis were accomplished and are summarized in the following section.

7.1 Conclusions

Water management, and specifically groundwater management in Canada, was until recently not a priority for the different levels of government. Recent incidents with water quality have caused water resource management to become one of the priorities of the Ontario government. The province has established, through legislation, new policies on drinking water including protecting the source of the drinking water.

One important aspect of protecting source waters is determining a water budget for a given area. In Ontario, this task as been given to the Conservation Authorities (CAs) for their watersheds. These watershed scale water budgets can be used by water management planners to identify water sources and water users within that area. The next step involves determining how the water users might affect the water sources.

Within the province of Ontario, large scale water users must have a Permit to Take Water (PTTW) to remove more than 50,000 litres per day. Applications for these permits are considered by the Ontario Ministry of the Environment (MOE) without any mandatory examination by the CAs responsible for managing the water resources in their watersheds. These applications were historically considered on an individual basis, but new policies have mentioned the consideration of cumulative effects, but not required them to be studied.

A framework for addressing the cumulative impacts of PTTWs on groundwater resources was proposed in this thesis. It involves the use of numerical modelling to help water resource planners obtain more information to make decisions. It was found that groundwater management is strongly dependent on data management. The ideas presented may not be able to be applied due to limitations in budget and other resources, but the principle ideas should be considered.

The framework involves a data warehouse managed by the MOE that houses all of the information gathered by the CAs and other agencies. The role of the MOE would be to establish protocols in information updating as well as delineating the type of information collected so that there is consistency throughout the province. Under the proposed framework, the CAs would be responsible for maintaining a numerical model for their watersheds that is up to date and could be used to examine potential cumulative effects of proposed water takings. The numerical model must be regularly updated and improved as new information from different studies is collected.

Under this scenario, the CAs would assume a leading role in assessing the PTTW applications since it is the CAs who are responsible for the water management within their watersheds. Under the proposed framework, this could follow a Transfer of Review program where the applications are reviewed by the CA on behalf of the MOE except in specific cases such as trans-boundary applications. The MOE would have the ability to make the final decision to approve the PTTW, but it would be heavily weighted on the decision reached by the concerned CA. Since under the proposed framework all of the different CAs within the province would be assessing the various PTTW applications, protocols must be established by the MOE to ensure consistency across Ontario.

The cumulative effects of water takings on groundwater resources were illustrated by a case study where a large number of aggregate quarries were concentrated in a small area. A numerical model was created for the area, and different dewatering scenarios were examined. In all cases, the cumulative effects of the aggregate quarries on the area were significant as indicated by large water table drawdowns. Gradients in the area were altered so that the direction of ground water flow changed from the original conditions. This information is helpful to watershed managers especially when drinking water sources need protection. Changing groundwater flow patterns can cause changes in both water quality and quantity.

The results also indicated that the cumulative effects of dewatering quarries can be different depending how a scenario is simulated. Transient simulations offer better insight into what can occur than a steady state simulation. There is a need to reflect the pumping conditions as accurately as possible to predict the potential effects due to dewatering the quarries.

The results from the transient simulations performed by the numerical model also indicated that the effects of dewatering aggregate quarries can be controlled by regulating when quarries can pump out the water. Conditions that regulate the pumping time could be placed on the PTTWs issued to aggregate quarries in order to minimize the effects of dewatering on the water resources in the area.

7.2 Recommendations

This thesis uncovered a number of questions that could be examined by further research. The issue of water quality is very important to drinking water users, but it is very difficult to model or predict changes that can occur. Steps should be taken to examine this situation so that better predictions for water quality can be made.

A common problem with hydrogeological studies is with the accuracy of information from well records. The existing records in Ontario, provide significant amounts of information, but can be very inaccurate depending on when the well was established and the driller responsible. An attempt should be made to clean up the well records so that a consistent and accurate set is used for any studies being conducted.

Other important issues that were uncovered were the unexpected results from the different boundary conditions in Visual MODFLOW used to simulate the dewatering of a quarry. Further attempts should be made to determine why the model gives the results that it does.

Finally, the numerical model created for the case study does not account for any of the local heterogeneities present in the case study area. The numerical model should be improved to include any heterogeneities found in the area due to faulting. Including these aspects could improve the calibration of both the regional and local scale models and the impacts from the cumulative effects presented herein. The model should also eventually include the interactions between surface water and groundwater to account for the fact that the water removed from the quarry becomes surface water which can reenter the domain through increased infiltration. Alternative software would be required however because any detailed models that account for and couple ground water and surface water are currently being developed in the research field and are not readily available or used in practice. In addition, more should be done to improve the knowledge in groundwater and surface water interactions so that they can be included in numerical models.

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Appendix A: Original Permit to Take Water Application (From Ontario, 2000)

Ontario

Ministry of the Environment

Application for Permit To Take Water

Information requested by this form is collected under the authority of the Ontario Water Resources Act, R.S.O. 1990, Chapter 0.40 (OWRA) and the Environmental Bill of Rights, Statutes of Ontario, 1993, Chapter 28 (EBR). The purpose of the Permit is to regulate water takings in order to promote efficient development and equitable use of surface and ground waters.

	☐ **New Permit** ☐ **Permit Renewal** ☐ **Permit Amendment**	Existing Permit No.
Name of Applicant		Telephone No.
Address		Postal Code

Application Particulars

Please read instructions on the Guide for Applying for Approval of Permit to Take Water ("Guide") and ensure that all sections of the application are completed in full, especially the section of Request Amount of Taking from each Source and project/application description for purposes of EBR registry.

Submit a diagram of the area of this water taking. Diagram, instructions and example are shown in the "Guide". If the taking is from a groundwater source, then a diagram indicating any wells within 500 metres of the taking must be submitted.

If there are questions concerning the application, please contact the corresponding Ministry of Environment and Regional Office listed in the "Guide".

A Source of Water

① Well(s): How many? Spring(s): How many?	② Lake, Stream or River Name (s)
③ Pond(s): How many? Type: ☐ Dugout ☐ By-Pass ☐ On-Stream ☐ Pit or Quarry	
④ Other: Type of Source	
⑤ Construction date of Source	⑥ Date of installation of Water Taking Equipment

B Location of Taking

Lot, Concession, Township or former Township and County or Region or District, or City, Town or Village with name of street and number

---

Are the proposed works located in an area of development control as defined by the Niagara Escarpment Planning and Development Act (NEPDA)

☐ Yes ☐ No (If Yes, attach copy of NEPDA permit)

C Location of Water Use

☐ **Same as B or**

Lot Concession, Township or former Township and county or Region or District, or City, Town or Village with name of street and number

D Purpose of Taking

☐ Irrigation ☐ Commercial ☐ Industrial ☐ Municipal ☐ Public Supply ☐ Recreation

☐ Drinking Water ☐ Other (please describe)

E Period of Water Taking (complete either section 1 or 2 below)

① Taking to commence on __________ and to extend for a period of _________ ☐ days ☐ weeks ☐ months ☐ years

② Seasonal taking to extend from ________ to ________ each year for __________ (number of years)

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* Ce formulaire est disponible en français

F Request Amount of Taking from each Source (if the taking involves the taking of water into storage, please state the amount of water taken into storage as well as the amount of water withdrawn from storage).

Source Number	Name of source or Description	Maximum amount taken per minute	Maximum amount taken per day	Number of ours of taking per day-maximum	Number of hours taken per day-average	Maximum number of days taking per year

Indicate unit of measure ☐ Litres ☐ Imperial Gallons ☐ U.S. Gallons

G **Project/application description for purposes of EBR registry (brief description of proposal)**

____

____

____

H **Environmental Bill of Rights requirements**

Is this a proposal for a Prescribed Instrument under EBR? ☐ Yes ☐ No

If "Yes", is it excepted from public notification? ☐ Yes ☐ No

If it is excepted from public notification, provide reason ☐ Equivalent Public Participation ☐ Emergency

☐ Environmentally Insignificant ☐ EAA or Tribunal Decision

Amendment or revocation

Documentation is support of the above noted exception must be provided (refer to "Guide").

I **Supporting information checklist. This is a list of supporting information attached to this application and is subject to the Freedom of Information and Protection of Privacy Act (FOIPOPA) and the Environmental Bill of Rights (EBR).**

SUPPORTING INFORMATION	ATTACHED?	REFERENCE	CAN BE DISCLOSED
Pre-application consultation with MOE	☐ yes ☐ no		
Documentation Provided			
Description of the proposed works	☐ yes ☐ no		☐ yes ☐ no
Environmental Study Report (ESR)	☐ yes ☐ no		☐ yes ☐ no
Preliminary Report	☐ yes ☐ no		☐ yes ☐ no
Design Report/Brief	☐ yes ☐ no		☐ yes ☐ no
Hydraulic and Process Calculations	☐ yes ☐ no		☐ yes ☐ no
Final Plans and Specifications	☐ yes ☐ no		☐ yes ☐ no
Water Supply and			
Raw Water Quality Analysis	☐ yes ☐ no		☐ yes ☐ no

Hydro geological Report	☐ yes ☐ no		☐ yes ☐ no
Other Attached Information	☐ yes ☐ no		☐ yes ☐ no

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J Statement of Applicant

I, the undersigned hereby declare that to the best of my knowledge, the information contained herein and the information submitted in support of this application is complete and accurate in every way. The applicant agrees to indemnify and save harmless the Crown in right of the Province of Ontario and its officers, employees, agents and contractors from and against all damages, loss, costs, claims, suits, injuries, demands, actions and proceedings resulting from or in any manner connected with act or omission of the applicant or any of its officers, employees, agents or contractors relating to this Application and any Permit, Renewal Permit or terms and conditions of a Permit issued in response to this Application.
I understand that it is the policy of the Director in issuing a Permit to Take Water to impose the General Terms and Conditions appearing on the Guide for Applying for Permit to take Water.

Name of Applicant or Agent/Official of Applicant (please print)	Signature of Applicant or Agent of Applicant	Date

Diagram of Location of Water Taking

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ptwform.wpd

Appendix B: Current Permit to Take Water Application (From Ontario, 2005)

Ontario Ministry of the Environment

Application for Permit to Take Water

Ce formulaire est disponible en français

For Office Use Only			
Reference Number	Payment Received	Date (y/m/d)	Initials

General Information and Instructions

General:

Information requested in this form is collected under the authority of the *Ontario Water Resources Act, R.S.O.* 1990 (OWRA) and the *Environmental Bill of Rights*, C. 28, Statutes of Ontario, 1993, (EBR) and will be used to evaluate applications for a Permit to Take Water as required by Section 34 (OWRA).

Instructions:

1. When completing this form, please refer to the "Guide to Permit to Take Water Application Form" (referred to as the Guide). Questions regarding completion and submission of the application should be directed to your local Regional Office of the Ministry of the Environment (see Guide for information).
2. This form must be completed with respect to all the requirements of the Guide in order for it to be considered as an application for approval. **INCOMPLETE APPLICATIONS WILL BE RETURNED TO THE APPLICANT.**
3. A complete application consists of:

   (1) a completed and signed application form,

   (2) all required supporting information identified in this form and the Guide, and

   (3) a certified cheque or money order, in Canadian funds, made payable to the Minister of Finance for the application fee when required. Payment may also be made by VISA, Mastercard or American Express.

   The Ministry may require additional information during the technical review of any application initially accepted as complete.
4. The original application, along with the supporting information and the application fee, must be sent to:

   The Ministry of the Environment,
   Attention: Permit to Take Water
   Director, Environmental Assessment and Approvals Branch,
   2 St. Clair Avenue West, Floor 12A, Toronto, Ontario M4V 1L5.
5. Information contained in this application is not considered confidential and will be made available to the public upon request. Information submitted as supporting information may be claimed as confidential but will be subject to the Freedom of Information and Protection of Privacy Act (FOIPPA) and EBR. If you do not claim confidentiality at the time of submitting the information, the Ministry may make the information available to the public without further notice to you. If you are identifying confidential material, please indicate why you believe the information is confidential.

1. Permit Administration

Please indicate if this application is for a:

- ☐ New Permit
- ☐ Amendment to Permit, attach photocopy of permit.
- ☐ Renewal of Permit, attach photocopy of permit.

2. Classification

Classification	Fee Required	No Fee Required
☐ Category 1	☐ $750	☐ Reason ________
☐ Category 2	☐ $750	☐ Reason ________
☐ Category 3	☐ $3,000	

3. Client Information

Client Name (legal name of individual or organization as evidenced by legal documents)	Business Identification Number
Business Name (the name under which the entity is operating or trading if different from the Client Name - also referred to as trade name)	
Client Type: ☐ Corporation ☐ Individual ☐ Partnership ☐ Sole Proprietor ☐ Federal Government ☐ Municipal Government ☐ Provincial Government ☐ Other (describe): ______	NAICS Code

4. Client Physical Address Complete A, B and C

A. Civic Address - Street Information Street Number/Name/Type/Direction/Unit/Suite Emergency (911) location number and street				
B. Municipality/Unorganized Township	County/District	Province/State	Country	Postal Code
C. Telephone Number (including area code)	Extension	Fax Number (including area code)	E-mail Address	

5. Client Mailing Address ☐ Same as Client Physical Address, or complete A and B.

A. Civic Address - Street Information Street Number/Name/Type/Direction/Unit/Suite/Emergency (911) location number and street/P.O. Box/Rural Route Number			
B. Municipality	Province/State	Country	Postal Code

6. Project Technical Contact Information ☐ Same as Client Name, or complete below

A. Name	Company		
Contact Address - Street Information ☐ Same as Client Mailing Address, or complete A, B, C and D below			
B. Street Number/Name/Type/Direction/Unit/Suite			
C. Municipality	Province/State	Country	Postal Code
D. Telephone Number (including area code)	Extension	Fax Number (including area code)	E-mail Address

7. Source(s) Information. A separate Part 7 section must be completed for each source. Note: Photocopy blank Part 7 section prior to completion if your application includes more than one source (location where water taking occurs).

Water Taking sources included in this application: (Do not include domestic use wells that do not require a permit)

Total number of wells	Total number of lake intakes	Total number of ponds	Total number of watercourse intakes

Source Location Information (if multiple sources are included in application, provide information for each source)

Street Number/Name/Type/Direction/Unit/Suite

Lot	Conc.	Part	Reference Plan

Municpality/Unorganized Township	County/District	Original Geographic Township

Geographic (GPS) Coordinates (to be provided in Datum NAD83)

Method of Collection	Accuracy Estimate	UTM Zone	Easting	Northing

Is the Client the owner of the site(s) where the water taking will occur? ☐ Yes ☐ No If "No," attach the owner's name, address and consent for the applicant to access the water taking location.

Is the Site located in the Oak Ridges Moraine Area as defined by the Ontario Regulation 01/02 made under the *Oak Ridges Moraine Conservation Act* (2001)? ☐ No ☐ Yes

Is the Site located in an area of development control as defined by the Niagara Escarpment Planning Development Act (NEPDA)? ☐ No ☐ Yes

Are you aware of any complaints or impacts resulting from water takings at this location? ☐ No ☐ Yes If "Yes," describe:

Is water packaged into a container (bottled water, tanks)? ☐ No ☐ Yes If "Yes," size of containers: ☐ 20 litres or greater ☐ less than 20 litres

If no wells are located within 500 m of the water taking location, provide the distance to the nearest well _______________ m.

Is municipal water available to all dwellings within 500m of the water taking location? ☐ Yes ☐ No ☐ Unknown

Estimated start date of taking

Taking to extend for a period of _______ ☐ days ☐ weeks ☐ months ☐ years ☐ indefinite

Is activity subject to the *Environmental Assessment Act*? ☐ No ☐ Yes If "Yes," attach approval or Notice of Completion.

List any public consultation/notification (such as public hearings, notification of First Nations etc) that has occurred related to the proposed water taking.

☐ **Well:**

Well Name / Identifier	Water Well Record Number	If not available, provide name of property owner at time of well construction

Has this well been deepened? ☐ No ☐ Yes If "Yes," Date of Deepening ____________________

Type of Well:
☐ drilled
☐ bored
☐ dug
☐ driven or jetted (sandpoints/wellpoints) If 'driven or jetted' is selected, Total number of sandpoints/wellpoints ___________,
Number of interconnected sandpoint/wellpoint systems ___________.

Can you measure the depth to water in this well?	If Yes, Depth to Static Water Level	Date Measured	Has a pumping test been done?
☐ No ☐ Yes			☐ No ☐ Yes If "Yes," attach report.

☐ **Lake:**

Lake Name

☐ **Watercourse (stream, municipal ditch, open drain, etc):**

Watercourse Name	Tributary to
Does flow in the watercourse stop at any time during the year? ☐ No ☐ Yes, If "Yes," during which months?	For what period of time?
Do you move/relocate the water intake (pump)? ☐ No ☐ Yes If "Yes," show primary intake and any secondary locations on a map.	

☐ **Pond/Reservoir:**

Pond Name/Identifier	Was pond constructed (man-made)? ☐ No ☐ Yes	If "Yes," date constructed.

Pond Size:

Average Length	Average Width	Average Depth of Water	Maximum Depth of Water

Pond Type:

Select diagram which most closely resembles your pond.

☐ online ☐ by-pass ☐ connected ☐ dugout

Source of pond water (How does water enter the pond?) (mark all that apply):

☐ seepage / springs / groundwater

☐ surface water runoff (including tile drains, does not include watercourse or open channel)

☐ pumped water (if water is pumped into a pond, complete section information for source from which water is pumped (ie: well or lake or watercourse))

☐ flowing water (watercourse, open drains, ditches, etc)

If source,

Does water flow into the pond (inflow)? ☐ Yes ☐ No	If "Yes," is there a control structure to regulate the inflow? ☐ No ☐ Yes	If "Yes," describe.
Does water flow out of the pond (outflow)? ☐ Yes ☐ No	If "Yes," is there a control structure to regulate the outflow? ☐ No ☐ Yes	If "Yes," describe.

8. Water Taking Volumes

Purpose Options

Agriculture:	irrigation of (includes frost protection): field and pasture crops; fruit orchard; market garden/flowers; nursery; sod farm; tender fruits; tobacco, other (must specify)
Commercial:	aquaculture, bottled water, golf course irrigation, mall/business; snowmaking, other (must specify)
Dewatering:	pits and quarries; construction; other (must specify)
Industrial:	aggregate washing, brewing/soft drinks, cooling water, food processing, manufacturing; pipeline testing; power generation; other (must specify)
Institutional:	school, hospital, other (must specify)
Recreation:	aesthetic, fish pond, wetland, other (must specify)
Remediation:	groundwater; other (must specify)
Water Supply:	campground, communal, municipal, other (must specify)
Miscellaneous:	dam/reservoir, heat pump, wildlife conservation, pumping test, other (must specify)

Water Source Information - Table A (Units in Litres)

Source Name	Purpose of Water Taking	Maximum rate per minute	Maximum number of hours of taking a day	Maximum volume per day	Typical volume per day	Maximum number of days of taking in a year	Earliest likely calendar date of taking	Latest likely calendar date of taking

9. Attachments

The following must be attached for all applications (Category 1, 2 and 3) to be considered complete.

☐ Map Requirements
On a 1:10 000 OBM (Ontario Base Map) (1:50 000 map only acceptable in locations where 1:10 000 is not obtainable), mark and label
- all existing and proposed water taking locations with sources corresponding with source name.
- all of the following features within 500m of each source: all environmental features including: existing wells (indicate use of existing well), springs, watercourses, wetlands, water bodies, property lines, locations and names of property owners, nearest road intersection, dwellings.

☐ Describe in detail how, where and when all water is obtained, stored, transferred, used and returned to the environment (if applicable). Details must include the source of all water takings (and corresponding source name if applicable), purpose of the water taking, period of water taking, and maximum quantity requested (see Guide for further instruction).
Note: If your application is subject to posting on the Environmental Bill of Rights Registry, this description will be used to create the Proposal Notice. The ministry may change the wording as required, to meet the EBR posting requirements.

☐ Describe how water taking needs (rates, amounts and time periods) were determined. Provide all relevant information and calculations to demonstrate the water takings requested are warranted. Calculation worksheets are available. Refer to Appendix E of guide.

☐ Attach completed water conservation Schedule 1.

Category 2 Only

☐ Completed Schedule 2 signed by a Qualified Person.

Category 3 Only

☐ Study ______________________________

10. Statement/Signature of Client (Applicant)

I, the undersigned, hereby declare that to the best of my knowledge:

- The information contained herein and the information submitted in support of this application is complete and accurate in every way and I am aware of the penalties against providing false information.
- The project technical information contact identified in Section 6 of this form is authorized to act on my behalf for the purpose of obtaining this approval.
- I have used the most recent application form (as obtained from the Ministry of the Environment Internet site at http://www.ene.gov.on.ca (or the Environmental Assessment and Approvals Branch at 1-800-461-6290) and I have included all necessary information identified on this form and in the Guidance Material.

Print Name	Signature	Date

11. Application Fee

Application Category	Application Fee Amount	
Method of Payment ☐ Certified Cheque ☐ Money Order ☐ VISA ☐ Master Card ☐ American Express		
Credit Card Information Name on Card (please print)	Credit Card Number	Expiry Date (mmm/year)
Cardholder Signature		Date

For Office Use Only			
Reference Number	Payment Received	Date (y/m/d)	Initials

Appendix C: Derivation of Groundwater Flow Equation

C.1 Darcy's Law

The most fundamental theory applicable in groundwater flow is Darcy's Law which was introduced by Henry Darcy in 1856 (Freeze and Cherry, 1979). This empirical law applies to the flow of a liquid through any porous medium such as sand.

Darcy found through a series of experiments that the Darcy flux, also known as the specific discharge (v), through a medium is proportional to the difference in head (Δh) between any two points, and inversely proportional to the distance between those two points (Δl). In order to express this mathematically, a number of definitions must be made.

Darcy flux or specific discharge is defined as:

$$v = \frac{Q}{A} \qquad \text{(C-1)}$$

Where Q is the flow through the medium [L^3/t], and A is the cross sectional area to flow [L^2]. As can be seen from a dimensional analysis, the units of Darcy flux are those of velocity [L/t]. Darcy flux represents the macroscopic flow of the liquid through the entire medium. It cannot attempt to describe the microscopic velocities passing through each individual pore space. As a result of this, an analysis done with Darcy's law treats the entire medium as a single unit to which certain properties can be assigned.

There is a sign convention applied to Darcy's law because Δh is defined as h_2-h_1, whereas Δl is the difference between point 1 and point 2. The use of the sign convention leads to the following equation

$$v = -K\frac{\Delta h}{\Delta l}, \text{or as a differential, } v = -K\frac{dh}{dl} \quad \text{(C-2)}$$

In the above equation, K is a proportionality constant that is a property of the medium through which the liquid (water) is flowing. It is defined as the hydraulic conductivity [L/t] and it will vary depending on the type of soil and liquid. K will be a large value for media such as gravel that allow large flows and a smaller value for clay soils that are sometimes used as impermeable boundaries because they transmit very low flows.

The equation shows that the driving force for flow through a porous medium is the *dh/dl* term which is known as the hydraulic gradient (*i*). Steep gradients, where there is a large head difference over a very short distance, will provide more flow than a gentle gradient.

C.2 Specific Storage

To understand the flow of water in aquifers, the concept of specific storage has to be understood. The compressibility of any soil medium is defined as follows (Freeze and Cherry, 1979):

$$\alpha = \frac{-dV_T / V_T}{d\sigma_e} \qquad \text{(C-3)}$$

Where V_T is the total volume of the medium and $d\sigma_e$ is the effective stress. Effective stress can be thought of as the stress in a saturated medium that is carried by the solid matrix of the medium. Consider any point within a saturated medium deep underground. The stress on that point from above is the total stress (σ_T). Countering σ_T are the effective stress (σ_e) and the fluid pressure (p) from the fluid in the pores. Any change in σ_T will cause a change in both σ_e and p.

$$d\sigma_T = dp + d\sigma_e \qquad \text{(C-4)}$$

In most cases, the total stress does not change significantly in a given area so $d\sigma_T = 0$ and as such,

$$-dp = d\sigma_e \qquad \text{(C-5)}$$

Since fluid pressure is governed by the density of the fluid (ρ), the gravitational constant (g) and the pressure head (ψ) in the form,

$$p = \rho g \Psi \qquad \text{(C-6)}$$

and the pressure head is a function of hydraulic head and the elevation of the point, the fluid pressure can be described by:

$$p = \rho g(h - z) \qquad \text{(C-7)}$$

Since we are considering a single point, z is constant and if equation C-7 is substituted into equation C-5, the effective stress is defined by:

$$d\sigma_e = -\rho g dh \qquad \text{(C-8)}$$

The compressibility of water (β) is defined as (Freeze and Cherry, 1979):

$$\beta = \frac{-dV_w / V_w}{dp} \qquad \text{(C-9)}$$

Where V_w is the volume of water, and p is the pressure. The negative sign is used by convention to make β a positive number.

If one compresses a saturated unit volume of aquifer, the water that would be expelled (dV_w) is equal, but opposite to the reduction in volume of the aquifer (dV_T). This can be expressed by manipulating equation C-3 such that:

$$dV_w = -dV_T = \alpha V_T d\sigma_e \qquad \text{(C-10)}$$

If one considers a unit volume, $V_T = 1$, equation C-8 is substituted for σ_e, and we assume a unit drop in head ($dh = -1$), the following will result.

$$dV_w = \alpha\rho g \qquad \text{(C-11)}$$

The compressibility of the fluid must also be considered with the compression of the aquifer. Rearranging equation C-9 gives:

$$dV_w = -\beta V_w dp \qquad \text{(C-12)}$$

Since we are only considering the water in the medium, the porosity must be taken into account such that $V_w = nV_T$. Again assuming a unit volume, $V_T = 1$, a unit drop in head ($dh = -1$) and substituting equation C-7 into equation C-12 with the assumption of z being constant, the following equation results.

$$dV_w = \beta n\rho g \qquad \text{(C-13)}$$

Specific storage (S_s) is defined as "the volume of water that a volume of aquifer releases from storage under a unit decline in hydraulic head" (Freeze and Cherry, 1979). This applies to a saturated aquifer. From this definition, the volume of water released from a unit drop in hydraulic head is the sum of equations C-11 and C-13 giving the equation

$$dV_w = \rho g(\alpha + \beta n) = S_s \qquad \text{(C-14)}$$

Specific storage is the sum of the water released due to the compressibility of the matrix and the compressibility of the fluid.

C.3 Transient Saturated Flow Equation

MODFLOW is a saturated flow model whose governing equations are based on fundamental principles including the conservation of mass and Darcy's Law. To develop the equation one must consider a cube, or elemental control volume, with the length of each side Δx, Δy, and Δz respectively and a porosity (*n*). There is a flow of a liquid with a density (ρ), and velocity (*v*) flowing through each face.

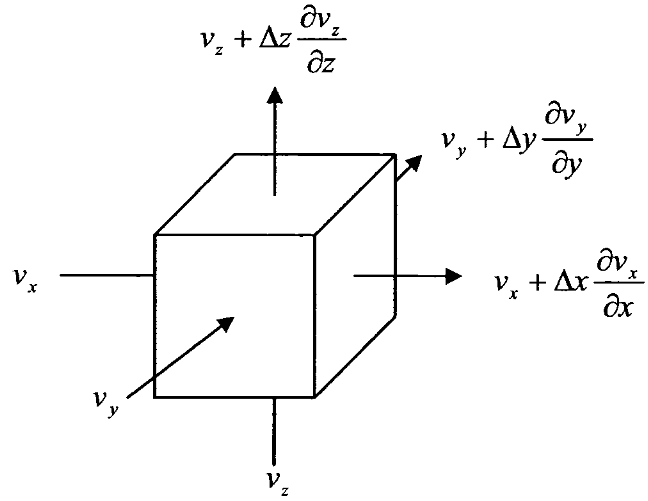


Figure C-1: Elemental Control Volume

If one applies the conservation of mass,

$$\text{Accumulation} = \text{Mass}_{\text{in}} - \text{Mass}_{\text{out}} \qquad \text{(C-15)}$$

Substituting the mass flows from Figure C-1 over a given time (Δt), the mass balance becomes:

$$\rho n\Delta x\Delta y\Delta z^{t+\Delta t} - \rho n\Delta x\Delta y\Delta z^{t} = \left(\rho v_x\Delta t\Delta y\Delta z + \rho v_y\Delta t\Delta x\Delta z + \rho v_z\Delta t\Delta x\Delta y\right) - \left(\rho\left(v_x + \Delta x\frac{\partial v_x}{\partial x}\right)\Delta t\Delta y\Delta z + \rho\left(v_y + \Delta y\frac{\partial v_y}{\partial y}\right)\Delta t\Delta x\Delta z + \rho\left(v_z + \Delta z\frac{\partial v_z}{\partial z}\right)\Delta t\Delta x\Delta y\right) \qquad \text{(C-16)}$$

This reduces to:

$$\frac{\partial(\rho n)}{\Delta t}=-\frac{\partial(\rho v_x)}{\partial x}-\frac{\partial(\rho v_y)}{\partial y}-\frac{\partial(\rho v_z)}{\partial z} \qquad \text{(C-17)}$$

When one expands the left hand side of the equation using the chain rule, one obtains the following:

$$n\frac{\partial p}{\partial t}+\rho\frac{\partial n}{\partial t}=-\frac{\partial(\rho v_x)}{\partial x}-\frac{\partial(\rho v_y)}{\partial y}-\frac{\partial(\rho v_z)}{\partial z} \qquad \text{(C-18)}$$

The two terms on the left hand side describe the change in storage in the control volume. The first term reflects the change in the density of the fluid whereas the second term describes a change in the porosity of the medium. A change in density of the fluid is related to its compressibility (β), and the change in porosity reflects the compressibility of the medium (α). From the discussion on specific storage (S_s) it was shown that changes in hydraulic head (h) result in changes of both the porosity of the medium as well as the density of the fluid. As a result, the mass rate of change for the water produced when an aquifer is compressed can be represented as $\rho S_s \partial h/\partial t$ thus resulting in the following equation (Freeze and Cherry, 1979).

$$\rho S_s\frac{\partial h}{\partial t}=-\frac{\partial(\rho v_x)}{\partial x}-\frac{\partial(\rho v_y)}{\partial y}-\frac{\partial(\rho v_z)}{\partial z} \qquad \text{(C-19)}$$

To further simplify this equation, one would apply the chain rule to develop the right hand side. Once the chain rule is applied it can be shown that the $\upsilon_x \partial\rho/\partial x$ term would be very much smaller than the $\rho\partial\upsilon_x/\partial x$ (Freeze and Cherry, 1979). With this, the density (ρ) term could be removed from the equation. When one applies Darcy's law (Equation C-2) into the equation, the equation becomes:

$$S_s\frac{\partial h}{\partial t}=\frac{\partial}{\partial x}\left(K_x\frac{\partial h}{\partial x}\right)+\frac{\partial}{\partial y}\left(K_y\frac{\partial h}{\partial y}\right)+\frac{\partial}{\partial z}\left(K_z\frac{\partial h}{\partial z}\right) \qquad \text{(C-20)}$$

This is the equation for transient flow through an anisotropic saturated aquifer with the principle direction of flow aligned with the principle axes; x, y, and z.

Appendix D: Governing Equations for MODFLOW

D.1 Equations used by MODFLOW

Ground water flow under non-equilibrium conditions in a heterogeneous and anisotropic medium, with the principal axes of hydraulic conductivity in alignment with the coordinate directions can be represented by the following equation.

$$\frac{\partial}{\partial x}\left(K_x \frac{\partial h}{\partial x}\right)+\frac{\partial}{\partial y}\left(K_y \frac{\partial h}{\partial y}\right)+\frac{\partial}{\partial z}\left(K_z \frac{\partial h}{\partial z}\right)-W=S_s \frac{\partial h}{\partial t} \qquad \text{(D-1)}$$

Where,

K_x, K_y and K_z are the hydraulic conductivity along the x, y and z axes [Lt^{-1}];

h is the hydraulic head [L];

W represents the volumetric flux of sink/source terms [t^{-1}];

S_s is the specific storage term [L^{-1}], and

t is time [t]

This equation can be used to represent groundwater flow if initial heads and boundary conditions are known. MODFLOW uses this equation to solve for the hydraulic head as a function of time at the required coordinates. When different heads are known with time at a series of points, the direction and speed of flow can be developed for a system (McDonald and Harbaugh, 1988).

In complex systems, the solution to this equation becomes very difficult and as such, numerical techniques must be used to solve the system. The finite difference method is one such technique that divides the continuous system into a series of discrete points. Partial derivatives are then replaced by the differences in heads between two adjacent points. Solving the system in this manner gives an approximate solution to the problem by developing a series of linear equations which are solved simultaneously to obtain the head at specific points at specific times.

D.2 Setting up the model

McDonald and Harbaugh (1988) describe how the discretization is developed for their MODFLOW code. An overview of this is presented here. Consider a system of a group of blocks called cells. As seen in figure D-1 these cells can be identified by their rows, columns and layers.

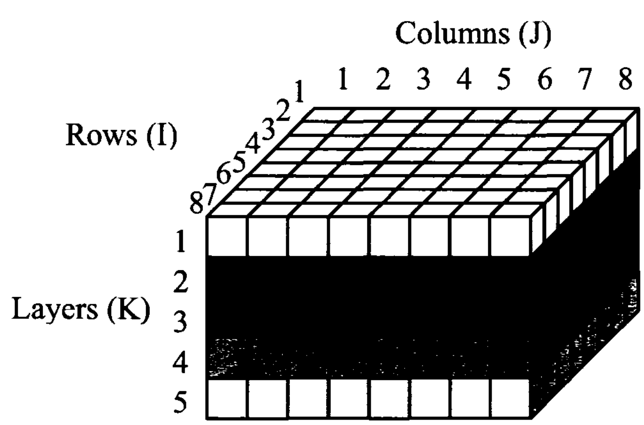


Figure D-1: Discretization used in MODFLOW (from McDonald and Harbaugh, 1988)

The sizes of the cells are chosen so that the hydraulic properties inside any given cell are somewhat uniform. The thickness of the layers usually corresponds to the different hydrogeological units present in the study area. In the centre of each cell is a node. It is at this point where the head will be calculated for the entire cell.

The heads for the cells are calculated using the finite difference equation using the assumption that the density of groundwater is constant. One simply has to apply the continuity of mass to a cell based on the surrounding cells as can be seen in Figure D-2. During development, MODFLOW assumed that the flows are positive if entering a cell.

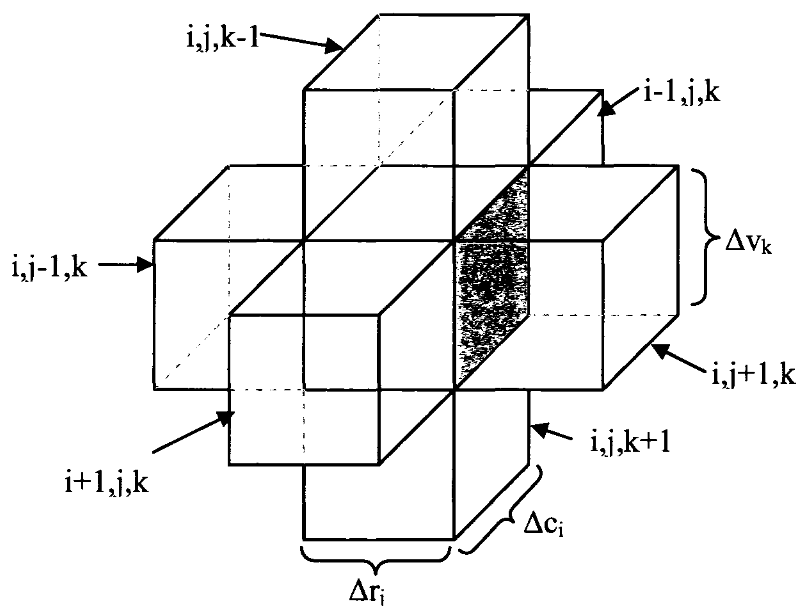


Figure D-2: MODFLOW cell naming convention (from McDonald and Harbaugh, 1988)

From Figure D-2, and applying Darcy's Law, the flow to cell i,j,k (centre) from cell i,j-1,k (left) is as follows.

$$q_{i,j-1,k} = K_{i,j-1/2,k} \Delta c_i \Delta v_k \frac{(h_{i,j-1,k} - h_{i,j,k})}{\Delta r_{j-1/2}} \qquad \text{(D-2)}$$

The use of j-1/2, as a subscript for K and Δr is due to the use of the harmonic mean of the hydraulic properties between the two adjacent nodes.

Five other equations can be written so that each face of cell i,j,k has a flow equation. To simplify these equations further, MODFLOW combines the dimensions of the grid and the hydraulic conductivity into a single term called the conductance. Conductance can be represented mathematically as:

$$\text{Conductance(C)} = \frac{\text{(Hydraulic Conductivity (K))(Cross} - \text{sectional area)}}{\text{Length of flow path}} \qquad \text{(D-3)}$$

Using this relationship, equation D-2 becomes:

$$q_{i,j-1,k} = C_{i,j-1/2,k}\left(h_{i,j-1,k} - h_{i,j,k}\right) \qquad \text{(D-4)}$$

Other sources and sinks for the cell also have to be accounted for. These flows can take on of two forms; either dependent on the head in the receiving cell and independent of all other heads in the aquifer (seepage from a river), or independent of all heads (recharge). Any external flows can be represented by the following:

$$QS_{i,j,k} = P_{i,j,k}h_{i,j,k} + Q_{i,j,k} \qquad \text{(D-5)}$$

Where

$QS_{i,j,k}$ is the sum of the external flows

$P_{i,j,k}h_{i,j,k}$ is the sum of the flows dependent on the head in the receiving cell

$Q_{i,j,k}$ is the sum of the flows independent of head

Applying the six flow equations developed for cell i,j,k in Figure D-1 and any external flows into the continuity of mass equation gives the following.

$$
\begin{aligned}
&q_{i,j-1/2,k} + q_{i,j+1/2,k} + q_{i-1/2,j,k} + q_{i+1/2,j,k} + q_{i,j,k-1/2} \\
&+ q_{i,j,k+1/2} + QS_{i,j,k} = SS_{i,j,k} \frac{\Delta h_{i,j,k}}{\Delta t} \Delta r_j \Delta c_i \Delta v_k
\end{aligned} \qquad \text{(D-6)}
$$

Where $SS_{i,j,k}$ is the specific storage, $\Delta h_{i,j,k}/\Delta t$ is a finite difference approximation of the change in head with respect to time, and $\Delta r_j \Delta c_i \Delta v_k$ is the volume of the cell.

The finite difference approximation of the change in head has to be expressed in terms of specific heads at specific times in order to satisfy the transient nature of groundwater flow. In order to perform this, one can use a backward difference approach to solve the head at a specific time (t_m) if one knows the head at a previous time (t_{m-1}). The heads at the different times can be expressed as $h^m_{i,j,k}$, and $h^{m-1}_{i,j,k}$. The finite difference approximation for the change in head in time then becomes:

$$
\left(\frac{\Delta h_{i,j,k}}{\Delta t}\right)_m = \frac{h^m_{i,j,k} - h^{m-1}_{i,j,k}}{t_m - t_{m-1}} \qquad \text{(D-7)}
$$

The continuity equation for the centre cell can therefore be written as the following with the substitution of the flow equations in the form of equation D-2.

$$
\begin{aligned}
&C_{i,j-1/2,k}\left(h^m_{i,j-1,k} - h^m_{i,j,k}\right) + C_{i,j+1/2,k}\left(h^m_{i,j+1,k} - h^m_{i,j,k}\right) \\
&+ C_{i-1/2,j,k}\left(h^m_{i-1,j,k} - h^m_{i,j,k}\right) + C_{i+1/2,j,k}\left(h^m_{i+1,j,k} - h^m_{i,j,k}\right) \\
&+ C_{i,j,k-1/2}\left(h^m_{i,j,k-1} - h^m_{i,j,k}\right) + C_{i,j,k+1/2}\left(h^m_{i,j,k+1} - h^m_{i,j,k}\right) \\
&+ P_{i,j,k} h^m_{i,j,k} + Q_{i,j,k} = SS_{i,j,k} \Delta r_j \Delta c_i \Delta v_k \frac{\left(h^m_{i,j,k} - h^{m-1}_{i,j,k}\right)}{t_m - t_{m-1}}
\end{aligned} \qquad \text{(D-8)}
$$

Everything in this equation is known except for the seven heads at time m. This problem is overcome when one realizes that the above equation can be written for each of the seven cells in Figure D-2 thus giving a system of seven equations with seven unknowns. These seven equations can be solved simultaneously to give an approximate solution for the heads in each of the cells as long as the heads are known in the cells surrounding the seven cells. This illustrates the need for assigning boundary conditions to the edges of the model so that the heads can be solved for all cells within the model.

Appendix E: Results of Modelling Quarry with Different Boundary Conditions

The global flow balance and quarry flow balance are mass balance calculations based on the following formula:

$$\text{Flow Balance} = \frac{(\text{IN} - \text{OUT})}{\frac{(\text{IN} + \text{OUT})}{2}} \times 100\% \qquad \text{(E-1)}$$

Constant Head Boundary Simulated Quarry on Layer 1:

Elevation (m)	Conductivity (m/s)	Cumulative in from recharge (m3/d)	Flow from formation to cell below (m3/d)	Flow to Quarry from all sides (m3/d)	Cumulative out from CH boundary TOP (m3/d)	Cumulative out from CH boundary BOT (m3/d)	Flow to Quarry from below (m3/d)	Cumulative out from quarry (m3/d)	Global Flow Balance (%)	Quarry Flow Balance (%)
31	1.0E-05	1084.89	81.36	55.78	462.75	463.68	102.54	158.32	0.013	-0.001
31	5.0E-05	1084.89	69.52	135.30	429.17	430.41	89.89	225.19	0.011	0.000
31	1.0E-04	1084.89	64.71	160.10	419.31	420.66	84.72	244.82	0.010	-0.001
31	5.0E-04	1084.89	59.78	183.99	409.95	411.40	79.43	263.42	0.011	0.000
31	1.0E-03	1084.89	59.09	187.18	408.72	410.18	78.70	265.88	0.010	0.002
31	0.01	1084.89	58.47	190.07	407.59	409.07	78.04	268.11	0.011	0.000
31	0.1	1084.89	58.41	190.36	407.48	408.96	77.97	268.33	0.011	0.001
31	1	1084.89	58.40	190.39	407.47	408.95	77.97	268.35	0.011	0.003
35	1.0E-05	1084.89	36.74	104.22	465.59	466.68	48.06	152.28	0.032	0.001
35	5.0E-05	1084.89	37.57	141.97	445.71	446.90	50.10	192.06	0.021	0.003
35	1.0E-04	1084.89	37.38	148.23	442.61	443.82	50.04	198.27	0.018	0.002
35	5.0E-04	1084.89	37.15	153.53	440.02	441.24	49.93	203.46	0.016	-0.001
35	1.0E-03	1084.89	37.12	154.21	439.68	440.91	49.91	204.12	0.017	-0.001
35	0.01	1084.89	37.09	154.83	439.39	440.61	49.89	204.72	0.016	-0.001
35	0.1	1084.89	37.08	154.89	439.36	440.58	49.89	204.78	0.016	-0.002
35	1	1084.89	37.08	154.90	439.35	440.57	49.89	204.79	0.017	-0.002
39	1.0E-05	1084.89	15.31	72.28	494.28	496.22	20.63	92.91	0.137	-0.001
39	5.0E-05	1084.89	18.22	89.46	483.81	485.68	24.64	114.10	0.120	0.004
39	1.0E-04	1084.89	18.64	92.23	482.43	483.99	25.23	117.45	0.094	0.001
39	5.0E-04	1084.89	18.99	94.57	481.15	482.59	25.71	120.28	0.081	-0.001
39	1.0E-03	1084.89	19.03	94.87	480.97	482.41	25.77	120.64	0.081	0.002
39	0.01	1084.89	19.07	95.15	480.88	482.24	25.83	120.97	0.074	0.001
39	0.1	1084.89	19.07	95.17	480.86	482.23	25.83	121.00	0.074	0.003
39	1	1084.89	19.08	95.18	480.86	482.23	25.83	121.01	0.073	-0.002

Constant Head Boundary Simulated Quarry on Layer 2:

Elevation (m)	Conductivity (m/s)	Cumulative in from recharge (m3/d)	Flow to Quarry from all sides (m3/d)	Flow from formation to cell below (m3/d)	Cumulative out from CH boundary TOP (m3/d)	Cumulative out from CH boundary BOT (m3/d)	Flow to Quarry from below (m3/d)	Cumulative out from quarry (m3/d)	Global Flow Balance (%)	Quarry Flow Balance (%)
21	1.0E-05	1084.89	101.30	180.66	400.76	402.17	180.66	281.96	0.000	0.000
21	5.0E-05	1084.89	267.95	153.25	330.79	332.91	153.25	421.20	-0.001	0.000
21	1.0E-04	1084.89	334.29	138.97	304.63	307.00	138.97	473.26	0.000	0.000
21	5.0E-04	1084.89	390.35	125.17	283.39	285.99	125.16	515.52	-0.001	-0.002
21	1.0E-03	1084.89	397.72	123.35	280.60	283.22	123.35	521.07	0.000	0.000
21	0.01	1084.89	404.32	121.70	278.12	280.76	121.70	526.02	-0.001	0.000
21	0.1	1084.89	404.98	121.54	277.87	280.51	121.54	526.51	0.000	0.002
21	1	1084.89	405.04	121.52	277.84	280.49	121.52	526.56	0.000	0.000
25	1.0E-05	1084.89	232.03	100.33	375.43	377.10	100.33	332.36	0.000	0.000
25	5.0E-05	1084.89	335.18	98.95	324.29	326.47	98.95	434.13	0.000	-0.001
25	1.0E-04	1084.89	354.44	97.14	315.53	317.80	97.14	451.57	-0.001	0.002
25	5.0E-04	1084.89	370.79	95.27	308.25	310.59	95.27	466.06	-0.001	0.000
25	1.0E-03	1084.89	372.83	95.01	307.35	309.71	95.02	467.84	-0.001	0.001
25	0.01	1084.89	374.64	94.78	306.56	308.92	94.78	468.42	0.092	0.214
25	0.1	1084.89	374.82	94.76	306.48	308.84	94.76	469.58	-0.001	0.000
25	1	1084.89	374.84	94.76	306.47	308.83	94.76	469.59	0.000	0.002
29	1.0E-05	1084.89	230.13	61.58	395.85	397.32	61.58	291.71	0.001	0.001
29	5.0E-05	1084.89	286.51	72.08	362.25	364.05	72.08	358.59	0.000	0.000
29	1.0E-04	1084.89	295.48	73.49	357.04	358.89	73.49	368.97	-0.001	-0.001
29	5.0E-04	1084.89	303.05	74.61	352.67	354.57	74.61	377.66	-0.001	-0.001
29	1.0E-03	1084.89	304.02	74.75	352.11	354.01	74.75	378.77	0.000	-0.001
29	0.01	1084.89	304.90	74.87	351.61	353.51	74.87	379.78	-0.001	-0.002
29	0.1	1084.89	304.99	74.88	351.55	353.46	74.88	379.88	0.000	-0.002
29	1	1084.89	305.00	74.89	351.55	353.46	74.89	379.89	-0.001	-0.001

Drain Boundary Simulated Quarry on Layer 1:

Elevation (m)	Conductance (m2/d)	Cumulative in from recharge (m3/d)	Recharge Directly to Drain (m3/d)	Flow from formation to cell below (m3/d)	Flow to Quarry from all sides (m3/d)	Cumulative out from CH boundary TOP (m3/d)	Cumulative out from CH boundary BOT (m3/d)	Flow to Quarry from below (m3/d)	Cumulative out from quarry (m3/d)	Global Flow Balance (%)	Quarry Flow Balance (%)
31	5	1084.92	0.03	7.85	40.56	515.89	516.96	10.65	51.24	0.077	-0.019
31	10	1084.92	0.03	13.47	66.43	499.65	500.33	18.22	84.68	0.024	-0.011
31	25	1084.92	0.03	27.76	98.19	474.63	475.33	36.71	134.94	0.002	-0.006
31	50	1084.92	0.03	42.86	104.45	461.96	462.78	55.69	160.18	0.000	-0.007
31	100	1084.92	0.03	57.40	95.14	457.65	458.50	73.59	168.77	0.000	-0.006
31	250	1084.92	0.03	70.50	76.87	458.85	459.69	89.49	166.39	-0.001	-0.006
31	500	1084.92	0.03	75.75	67.29	460.49	461.31	95.81	163.13	-0.001	-0.006
31	1000	1084.92	0.03	80.22	58.23	462.35	463.15	101.17	159.44	-0.001	-0.006
31	2500	1084.92	0.03	80.79	57.01	462.62	463.41	101.85	158.90	-0.001	-0.010
31	5000	1084.92	0.03	81.80	56.40	462.75	463.55	102.20	158.63	-0.001	-0.006
31	10000	1084.92	0.03	81.08	56.40	462.75	463.55	102.20	158.63	-0.001	-0.004
31	25000	1084.92	0.03	81.25	56.03	462.83	463.63	102.40	158.44	0.002	0.008
31	50000	1084.92	0.03	81.31	55.90	462.86	463.66	102.47	158.38	0.002	0.012
31	100000	1084.92	0.03	81.34	55.84	462.88	463.67	102.51	158.35	0.002	0.017
35	5	1084.92	0.03	5.42	28.00	522.82	524.85	7.35	35.38	0.172	-0.016
35	10	1084.92	0.03	8.98	46.40	512.22	513.29	12.18	58.62	0.073	-0.018
35	25	1084.92	0.03	15.98	74.29	494.19	494.78	21.52	95.84	0.010	-0.010
35	50	1084.92	0.03	22.47	89.82	482.24	482.87	29.93	119.79	0.002	-0.011
35	100	1084.92	0.03	28.00	98.50	474.34	475.03	37.02	135.55	0.000	-0.007
35	250	1084.92	0.03	32.76	102.49	469.31	470.04	43.05	145.58	-0.001	-0.010
35	500	1084.92	0.03	34.66	103.48	467.61	468.36	45.44	148.96	-0.001	-0.008
35	1000	1084.92	0.03	35.68	103.89	466.76	467.52	46.72	150.64	0.000	-0.001
35	2500	1084.92	0.03	36.32	104.10	466.26	467.02	47.52	151.65	-0.001	-0.004
35	5000	1084.92	0.03	36.53	104.16	466.09	466.86	47.79	151.99	-0.002	-0.010
35	10000	1084.92	0.03	36.64	104.19	466.01	466.77	47.92	152.15	-0.001	-0.006
35	25000	1084.92	0.03	36.70	104.21	465.96	466.72	48.01	152.22	0.002	0.015
35	50000	1084.92	0.03	36.73	104.21	465.94	466.71	48.03	152.26	0.001	0.006
35	100000	1084.92	0.03	36.74	104.22	465.93	466.70	48.05	152.27	0.002	0.015
39	5	1084.92	0.03	2.99	15.46	530.55	532.72	4.05	19.52	0.196	0.050
39	10	1084.92	0.03	4.95	25.61	524.83	526.35	6.72	32.34	0.129	0.029
39	25	1084.92	0.03	8.18	42.26	514.41	515.88	11.09	53.37	0.116	0.018
39	50	1084.92	0.03	10.44	53.95	507.35	508.53	14.17	68.15	0.082	-0.011
39	100	1084.92	0.03	12.27	62.26	502.67	503.20	16.63	78.93	0.011	-0.011
39	250	1084.92	0.03	13.93	67.97	498.80	499.26	18.83	86.84	0.002	-0.011
39	500	1084.92	0.03	14.59	70.08	497.33	497.79	19.69	89.80	0.000	-0.011
39	1000	1084.92	0.03	14.94	71.17	496.55	497.02	20.15	91.36	0.000	-0.011
39	2500	1084.92	0.03	15.16	71.83	496.08	496.54	20.44	92.31	-0.001	-0.016
39	5000	1084.92	0.03	15.23	72.06	495.92	496.38	20.53	92.64	-0.001	-0.016
39	10000	1084.92	0.03	15.27	72.17	495.84	496.30	20.58	92.80	-0.001	-0.015
39	25000	1084.92	0.03	15.29	72.24	495.79	496.25	20.61	92.86	0.002	0.019
39	50000	1084.92	0.03	15.30	72.26	495.77	496.24	20.62	92.90	0.002	0.019
39	100000	1084.92	0.03	15.30	72.27	495.76	496.23	20.63	92.91	0.002	0.019

Drain Boundary Simulated Quarry on Layer 2:

Elevation (m)	Conductance (m2/d)	Cumulative in from recharge (m3/d)	Recharge Directly to Quarry (m3/d)	Flow to Quarry from all sides (m3/d)	Flow from formation to cell above (m3/d)	Fow from formation to cell below (m3/d)	Cumulative out from CH boundary TOP (m3/d)	Cumulative out from CH boundary BOT (m3/d)	Flow to Quarry from above (m3/d)	Flow to Quarry from below (m3/d)	Cumulative out from quarry (m3/d)	Global Flow Balance (%)	Quarry Flow Balance (%)
21	5	1084.92	0.00	63.12	14.28	15.79	495.24	496.08	14.31	15.78	93.22	0.035	-0.011
21	10	1084.92	0.00	106.17	24.34	26.58	463.30	464.28	24.41	26.56	157.16	0.017	-0.008
21	25	1084.92	0.00	179.35	42.01	45.11	408.52	409.88	42.04	45.11	266.50	0.002	0.002
21	50	1084.92	0.03	238.21	0.00	75.73	384.68	386.26	0.00	75.73	313.98	0.000	-0.005
21	100	1084.92	0.03	224.72	0.00	109.28	374.60	376.28	0.00	109.28	334.04	0.000	-0.004
21	250	1084.92	0.03	173.12	0.00	146.21	381.98	383.59	0.00	146.21	319.36	-0.001	-0.001
21	500	1084.92	0.03	141.17	0.00	162.68	389.75	391.28	0.00	162.68	303.89	0.000	-0.004
21	1000	1084.92	0.03	122.11	0.00	171.55	394.88	396.36	0.00	171.55	293.69	-0.001	-0.001
21	2500	1084.92	0.03	109.81	0.00	177.00	398.32	399.76	0.00	177.00	286.84	0.000	-0.001
21	5000	1084.92	0.03	105.58	0.00	178.83	399.53	400.96	0.00	178.83	284.45	-0.002	-0.004
21	10000	1084.92	0.03	103.45	0.00	179.75	400.14	401.56	0.00	179.75	283.23	-0.001	-0.001
21	25000	1084.92	0.03	102.16	0.00	180.30	400.51	401.93	0.00	180.30	282.47	0.001	0.006
21	50000	1084.92	0.03	101.73	0.00	180.48	400.63	402.05	0.00	180.48	282.22	0.002	0.006
21	100000	1084.92	0.03	101.52	0.00	180.57	400.69	402.11	0.00	180.57	282.10	0.002	0.006
25	5	1084.92	0.00	52.10	11.79	13.03	503.17	504.18	11.81	13.03	76.95	0.057	-0.010
25	10	1084.92	0.00	87.82	19.87	21.97	477.05	477.95	19.90	21.96	129.69	0.021	-0.009
25	25	1084.92	0.00	148.31	34.95	37.19	431.60	432.77	34.99	37.18	220.49	0.006	-0.005
25	50	1084.92	0.00	192.98	44.32	48.60	398.77	400.21	44.35	48.60	285.93	0.001	0.001
25	100	1084.92	0.03	235.63	0.00	69.05	389.34	390.87	0.00	69.05	304.72	-0.001	-0.004
25	250	1084.92	0.03	238.50	0.00	85.34	379.71	381.34	0.00	85.34	323.88	-0.001	-0.003
25	500	1084.92	0.03	236.44	0.00	92.34	377.22	378.88	0.00	92.34	328.82	0.000	-0.003
25	1000	1084.92	0.03	234.57	0.00	96.21	376.22	377.89	0.00	96.21	330.81	0.000	-0.002
25	2500	1084.92	0.03	233.13	0.00	98.65	375.72	377.39	0.00	98.65	331.80	0.001	0.001
25	5000	1084.92	0.03	232.60	0.00	99.48	375.57	377.24	0.00	99.48	332.10	0.001	0.003
25	10000	1084.92	0.03	232.32	0.00	99.90	375.50	377.17	0.00	99.90	332.25	0.000	0.000
25	25000	1084.92	0.03	232.15	0.00	100.16	375.46	377.13	0.00	100.16	332.32	0.001	0.005
25	50000	1084.92	0.03	232.09	0.00	100.24	375.45	377.12	0.00	100.24	332.32	0.003	0.011
25	100000	1084.92	0.03	232.06	0.00	100.29	375.44	377.11	0.00	100.29	332.36	0.001	0.005
29	5	1084.92	0.00	41.09	9.30	10.27	511.10	512.26	9.32	10.27	60.69	0.081	-0.015
29	10	1084.92	0.00	69.25	15.67	17.32	490.73	491.58	15.70	17.32	102.28	0.031	-0.014
29	25	1084.92	0.00	117.27	27.23	29.35	455.00	455.96	27.26	29.35	173.89	0.007	-0.006
29	50	1084.92	0.00	152.43	35.96	38.23	428.54	429.71	35.99	38.23	226.66	0.001	-0.004
29	100	1084.92	0.00	179.29	41.99	45.10	408.58	409.92	42.03	45.10	266.43	-0.001	-0.005
29	250	1084.92	0.00	200.86	45.08	50.61	393.43	394.92	45.11	50.61	296.57	0.000	0.003
29	500	1084.92	0.03	225.83	0.00	57.51	400.06	401.49	0.00	57.51	283.38	-0.001	-0.003
29	1000	1084.92	0.03	228.03	0.00	59.49	397.96	399.41	0.00	59.49	287.56	-0.001	-0.005
29	2500	1084.92	0.03	229.30	0.00	60.73	396.70	398.16	0.00	60.73	290.06	0.000	-0.001
29	5000	1084.92	0.03	229.72	0.00	61.16	396.28	397.74	0.00	61.16	290.89	0.001	0.004
29	10000	1084.92	0.03	229.92	0.00	61.37	396.07	397.53	0.00	61.37	291.31	0.001	0.002
29	25000	1084.92	0.03	230.05	0.00	61.50	395.94	397.41	0.00	61.50	291.55	0.002	0.009
29	50000	1084.92	0.03	230.09	0.00	61.54	395.90	397.37	0.00	61.54	291.64	0.001	0.006
29	100000	1084.92	0.00	230.11	0.00	61.56	395.88	397.35	156.97	61.56	291.67	0.002	42.407

Appendix F: Hydraulic Conductivities from Local Studies

Table F-1: Comparison of hydraulic conductivity values for surficial sediments

Study		Clay	Till	Sand	Sand & Gravel
Golder, 2003a	Max	$1x10^{-6}$			
	Min	$1x10^{-9}$			
	Modelled	Both		$6.4x10^{-5}$	$8x10^{-4}$
Golder, 2003b	Max	$1x10^{-6}$	$1x10^{-6}$		
	Min	$1x10^{-9}$	$1x10^{-9}$		
	Modelled	$5x10^{-7}$	$5x10^{-7}$	$5x10^{-7}$	
Golder *et al.*, 2003	Max	$1x10^{-8}$	$1x10^{-6}$	$1x10^{-4}$	1.01
	Min	$1x10^{-10}$	$1x10^{-7}$	$1x10^{-5}$	$1x10^{-6}$
Freeze and Cherry, 1979	Max	$1x10^{-9}$	$2x10^{-6}$	$1x10^{-2}$	
	Min	$1x10^{-12}$	$1x10^{-12}$	$5x10^{-6}$	

All hydraulic conductivity values are in m/s

There is a wide range of hydraulic conductivity (K) values associated with the different types of sediment as can be seen in Table F-1. This table includes the maximum and minimum K values obtained from studies conducted in the study area. It also includes the value used in modelling the sediment if calculations were performed using the value. The value reported by Freeze and Cherry (1979) is a range of values that are associated with the different surficial sediments but are not necessarily those from the study area.

Table F-2 contains the different hydraulic conductivity values for both the upper Paleozoic bedrock and the Nepean sandstone. These values represent the maximum and minimums that were presented in the study. Included is the value used to represent the porous media if modelling or calculations were included in the report. The values provided by Freeze and Cherry (1979) are considered a range of hydraulic conductivity values of the particular type of rock. The values specific to the case study area fall into this range.

Table F-2: Comparison of hydraulic conductivity values for Paleozoic bedrock

Study		Upper Paleozoic	Nepean
Golder, 2003b	Max Min Modelled	$9x10^{-4}$ $2x10^{-11}$ $5x10^{-7}$	$1.7x10^{-4}$ $1.3x10^{-6}$ $4x10^{-4}$
Golder *et al.*, 2003	Max Min	$1x10^{-4}$ $1x10^{-10}$	$1x10^{-6}$ $1x10^{-8}$
Golder, 2000a	Max Min Modelled	$2x10^{-5}$ $2x10^{-8}$ $2x10^{-6}$	
Golder, 1979	Max Min	$1x10^{-8}$ $1x10^{-9}$	
Golder, 2003c	Max Min	$1.1x10^{-5}$ $5x10^{-9}$	
Golder, 2001	Max Min Modelled	$1.3x10^{-6}$ $1.8x10^{-11}$ $1.5x10^{-7}$	
Golder, 2000b	Max Min Modelled	$2x10^{-5}$ $2x10^{-8}$ $2x10^{-6}$	
Morey Houle Chevrier, 2001	Max Min Modelled	$1.2x10^{-5}$ $2x10^{-6}$ $6.6x10^{-6}$	
Golder, 2003a	Modelled	$3.9x10^{-5}$	
Freeze and Cherry, 1979	Max Min	$3x10^{-6}$ $8x10^{-10}$	$5x10^{-6}$ $1x10^{-10}$

All hydraulic conductivity values are in m/s

Appendix G: Calibration Results

Residuals presented in the following figures are based on the calculated heads for the layer by the updated numerical model and the water levels recorded in the MOE WWIS for the wells terminating in the layer.

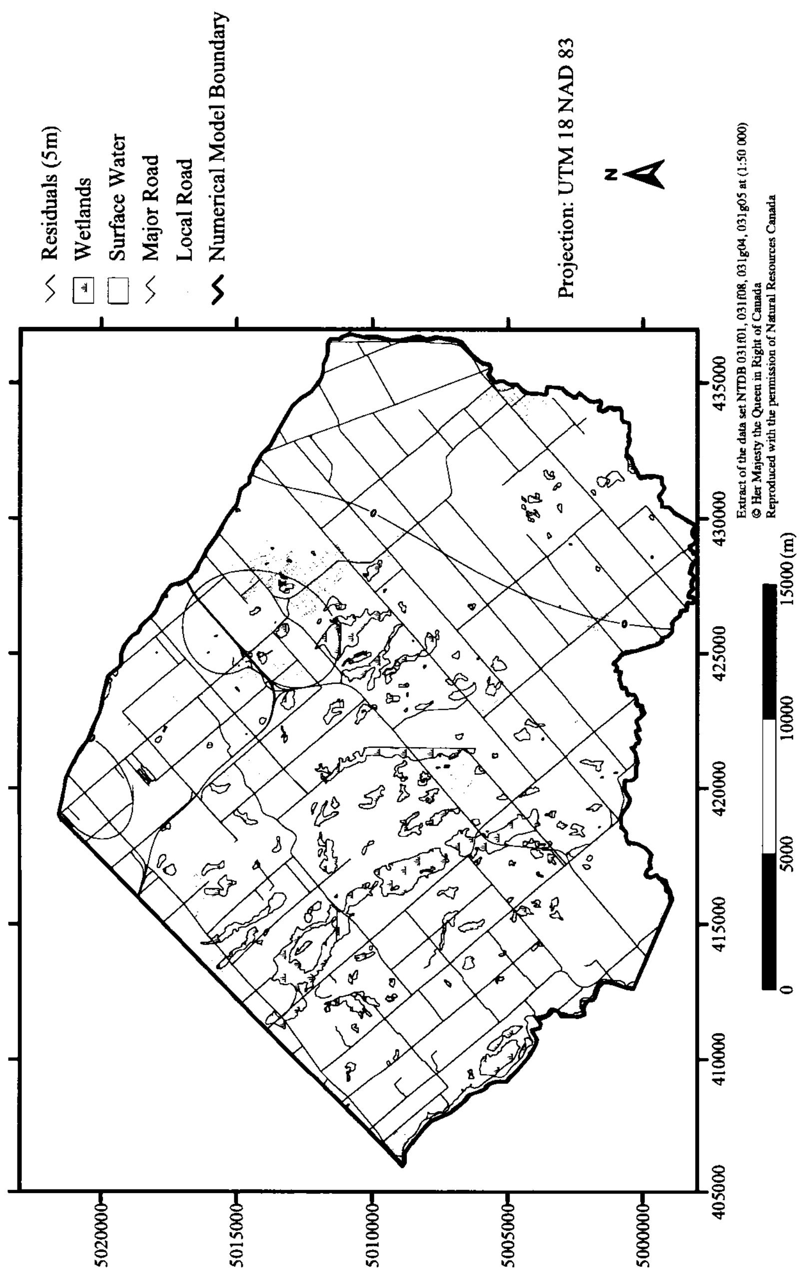


Figure G-1: Residuals for layer 1

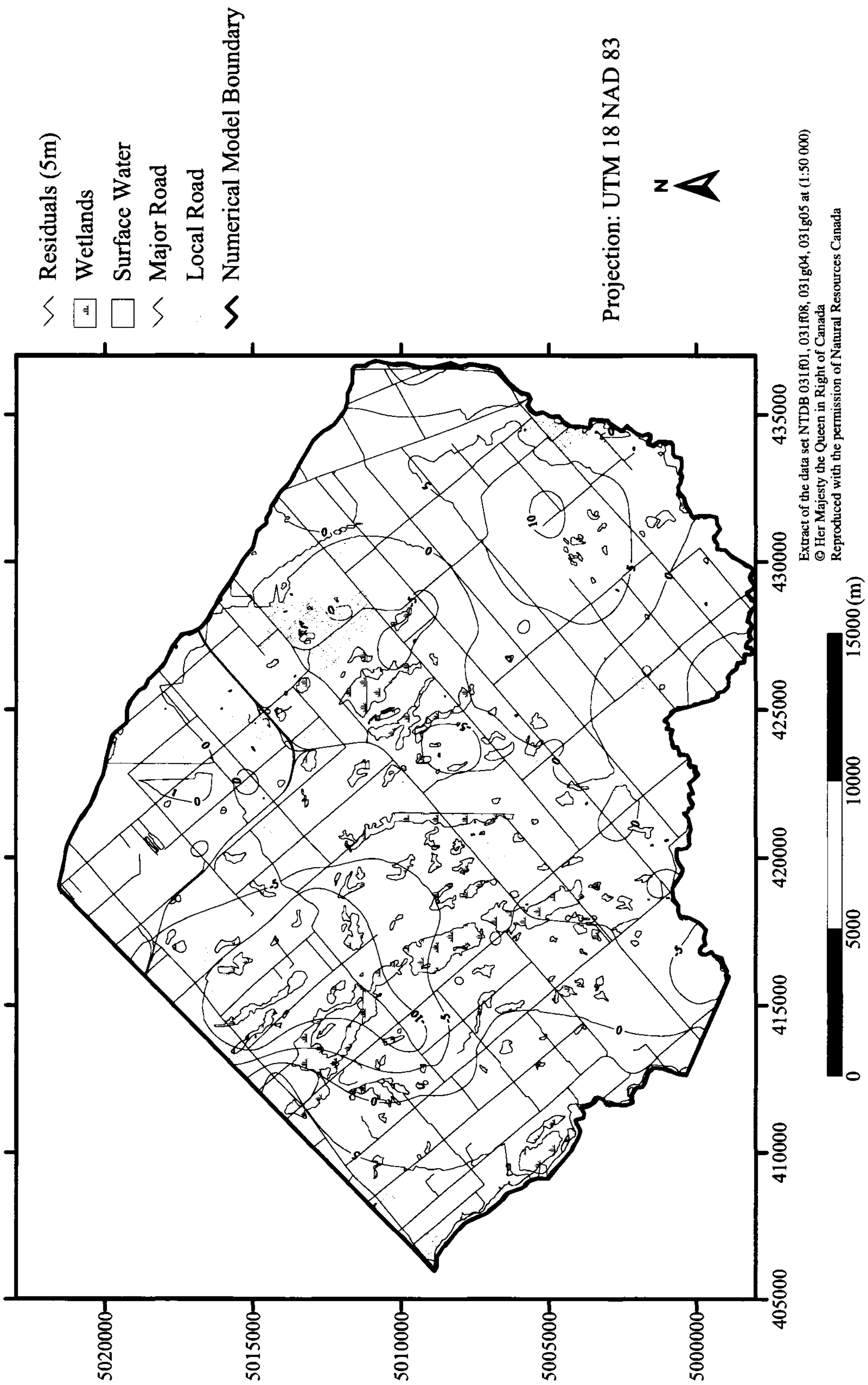
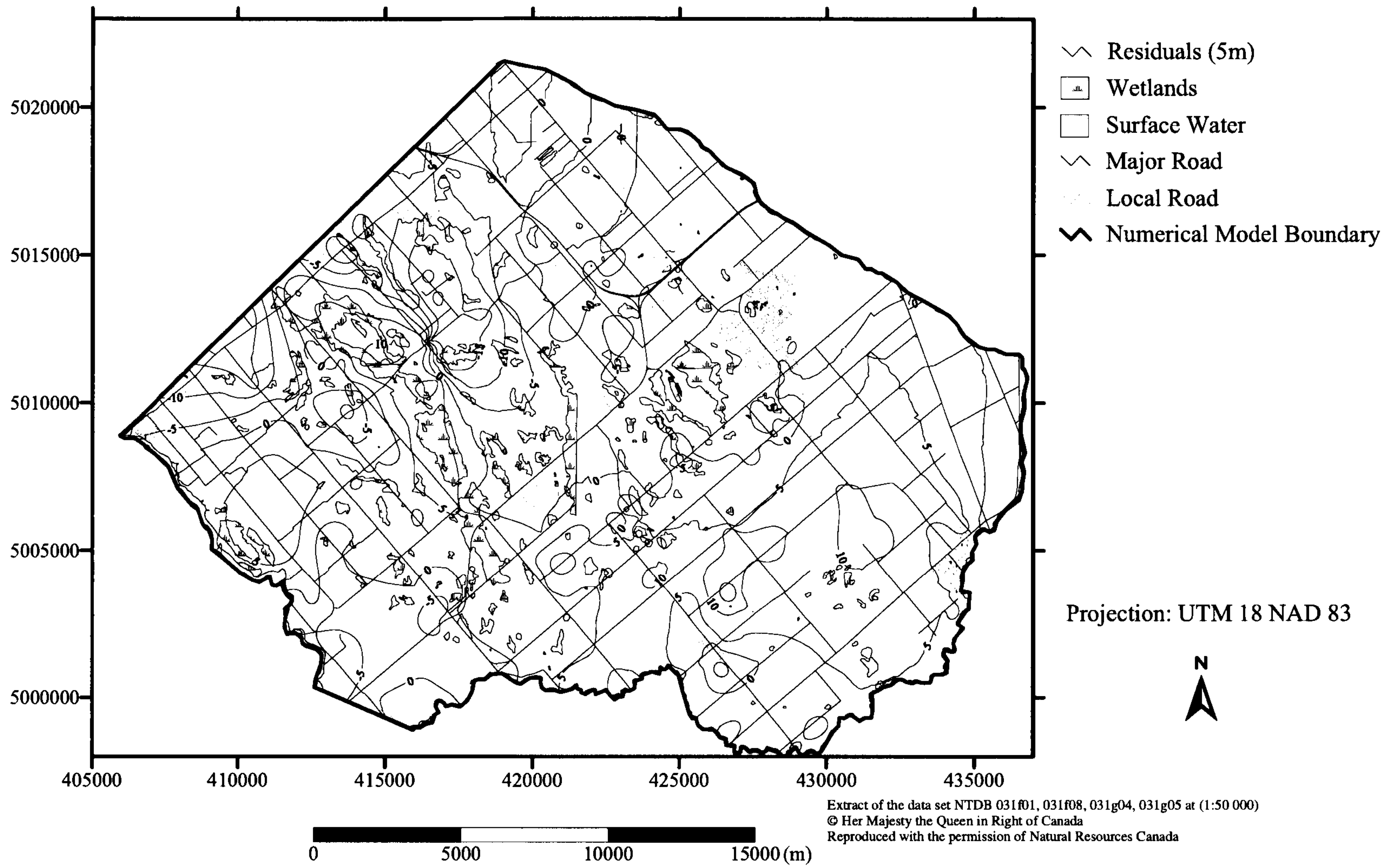


Figure G-2: Residuals for layer 2

Figure G-3: Residuals for layer 3



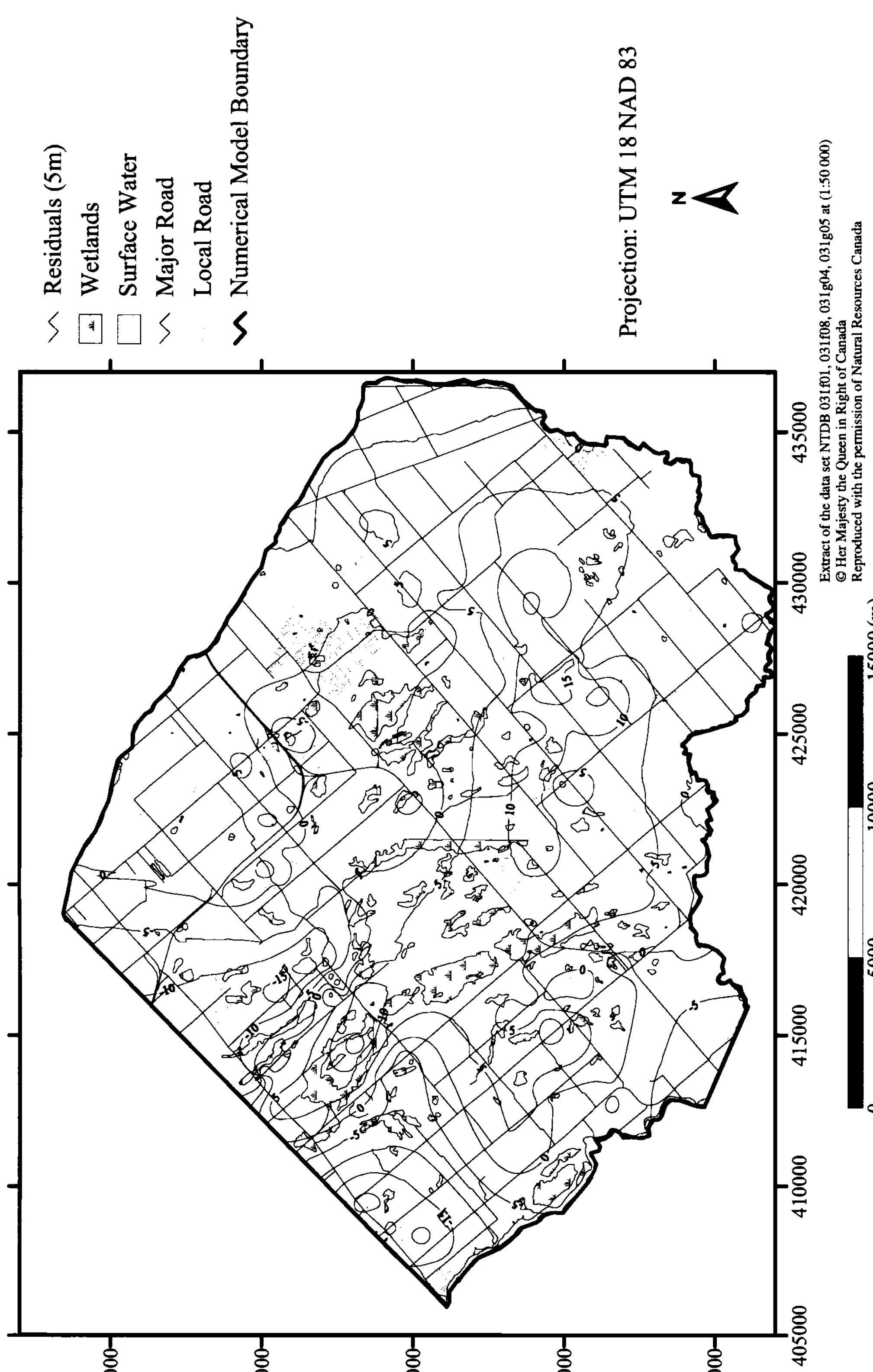


Figure G-4: Residuals for layer 4

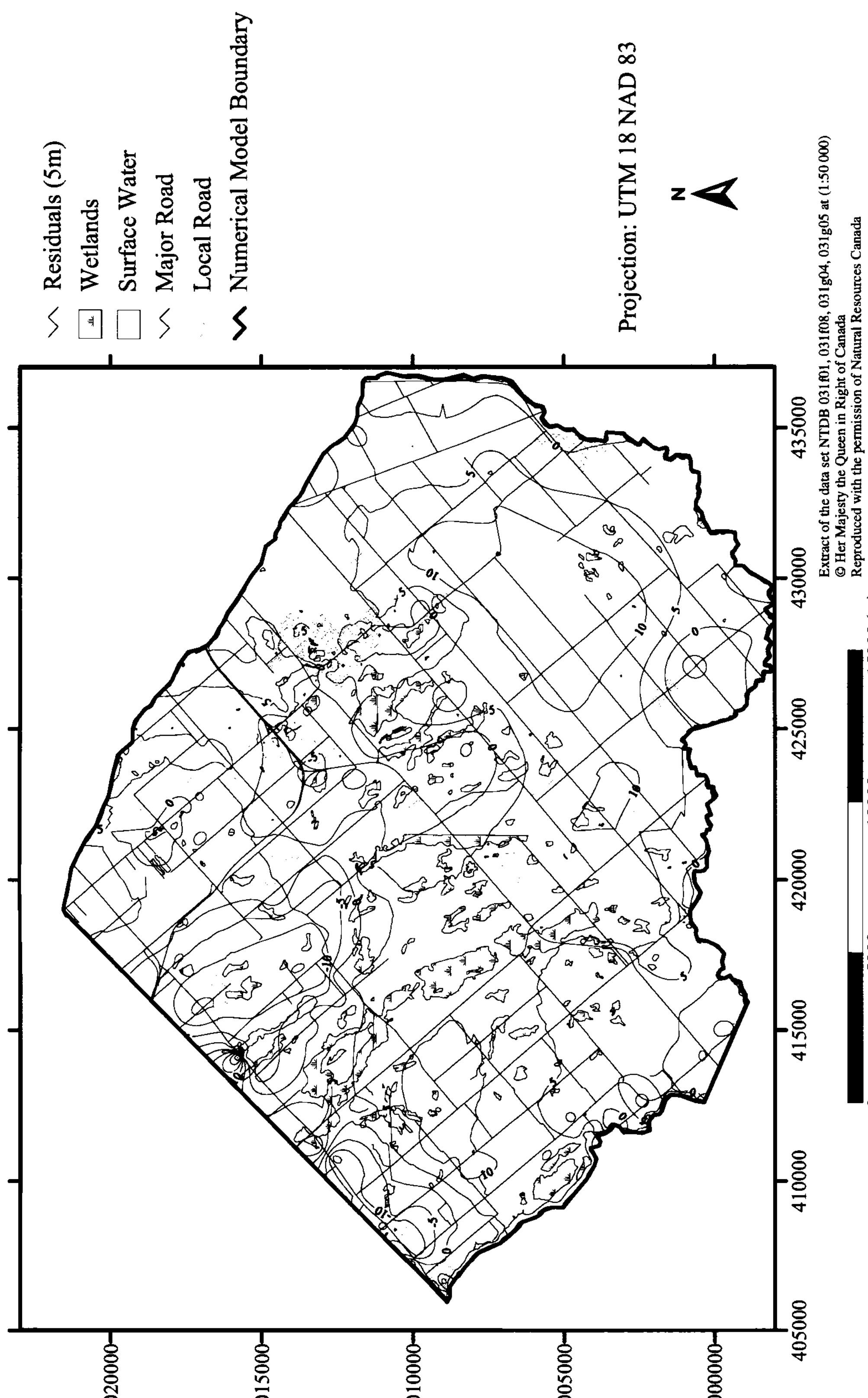


Figure G-5: Residuals for layer 5

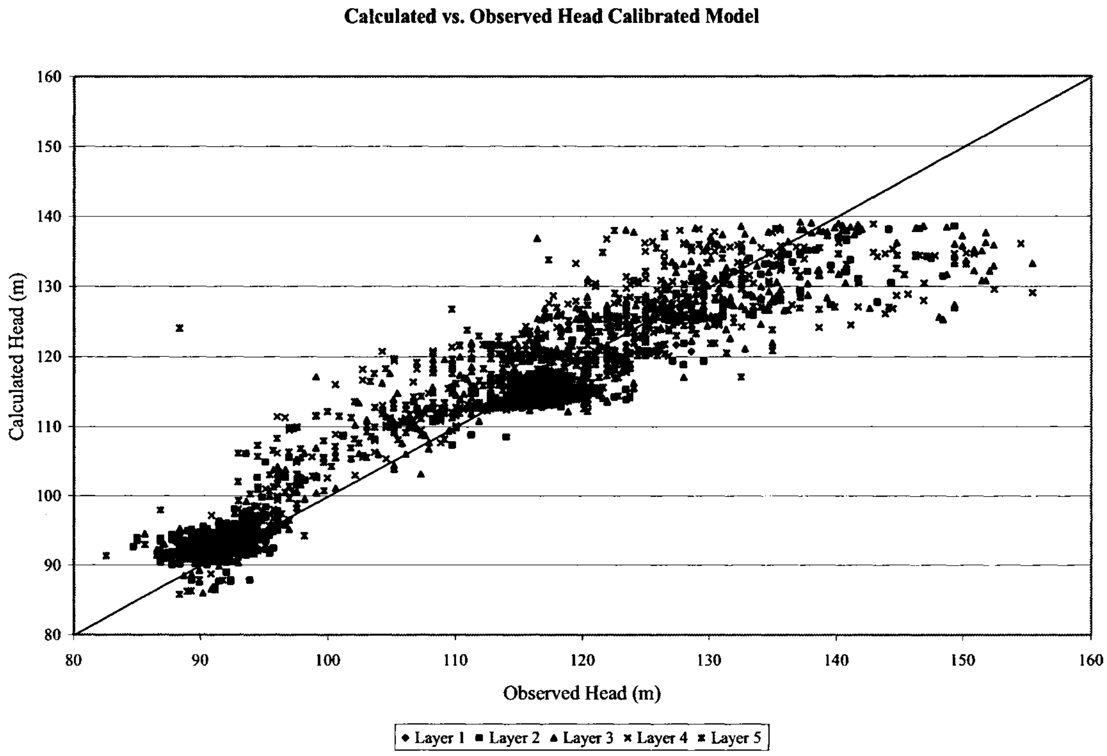
The different runs for the calibration of the original regional numerical model and the different calibration statistics obtained can be seen in Table G-1

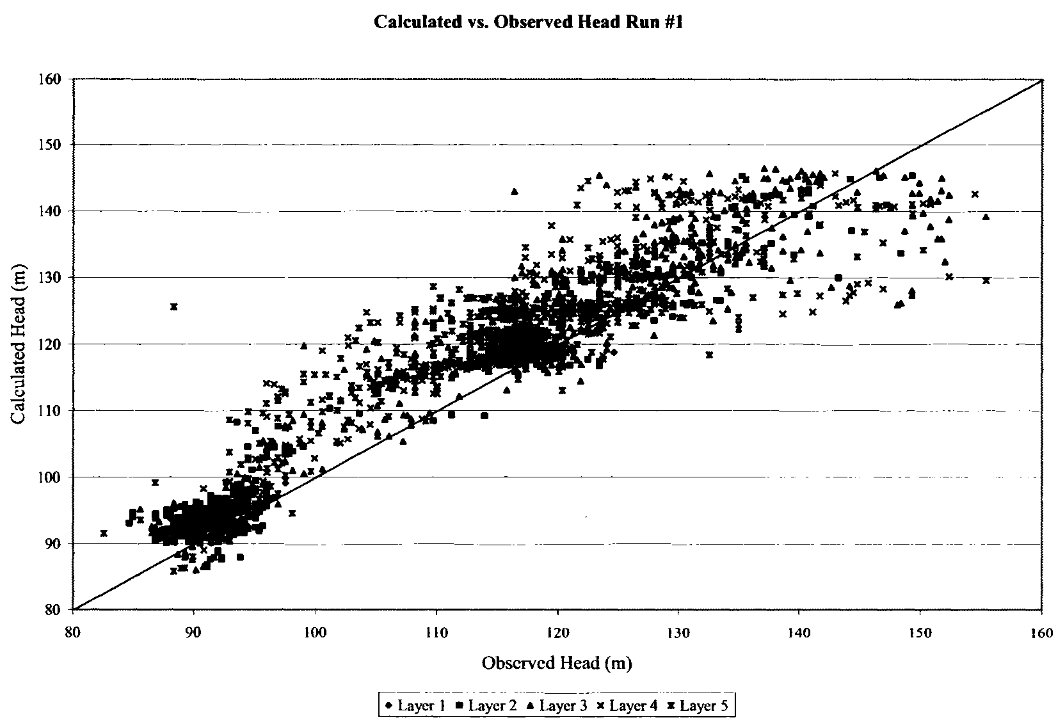
Table G-1: Results from different calibration runs

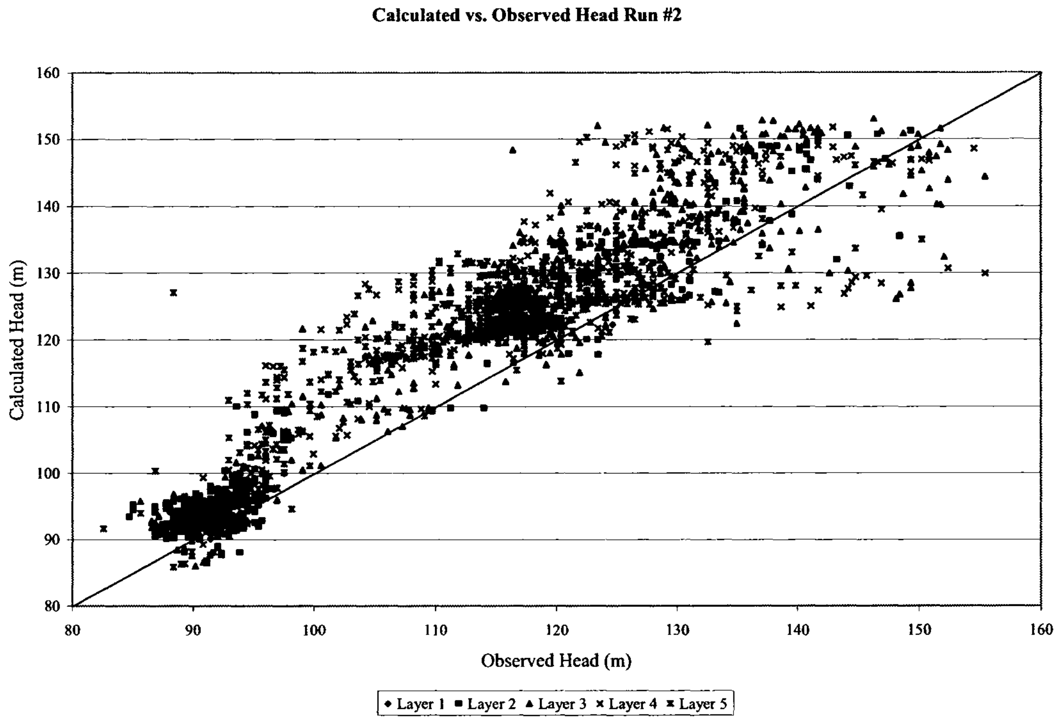
	Calibrated	Run 1	Run 2
Maximum Calculated Value (m)	138	146	153
Minimum Calculated Value (m)	86	86	86
Absolute Residual Mean (m)	3.822	4.773	6.929
Normalized Root Mean Square (%)	7.211	8.66	11.694
Upper Paleozoic Conductivity (m/s)	$5x10^{-6}$	$1x10^{-6}$	$1x10^{-6}$
Nepean Formation Conductivity (m/s)	$1.3x10^{-4}$	$1x10^{-4}$	$8x10^{-5}$

When the observed vs. calculated heads for the calibrated model is compared to those from Run 1 and 2, on can see that decreasing the hydraulic conductivity in the Nepean sandstone layer (layer 5) increases all of the heads over the entire model, not just the heads that were being under-predicted.

At the level discretization of the regional model, the local highs and lows are not able to be replicated. Areas where there are steep gradients tend to be smoothed out thus under-predicting the heads at local high areas.


Calculated vs. Observed Head Calibrated Model
Calculated Head (m)
Observed Head (m)
160
150
140
130
120
110
100
90
80
80
90
100
110
120
130
140
150
160
• Layer 1 ▪ Layer 2 ▲ Layer 3 × Layer 4 ж Layer 5


Calculated vs. Observed Head Run #1
Calculated Head (m)
Observed Head (m)
160
150
140
130
120
110
100
90
80
80
90
100
110
120
130
140
150
160
• Layer 1 ▪ Layer 2 ▲ Layer 3 × Layer 4 ж Layer 5



Calculated vs. Observed Head Run #2
160
150
140
130
120
110
100
90
80
Calculated Head (m)
80
90
100
110
120
130
140
150
160
Observed Head (m)
◆ Layer 1 ■ Layer 2 ▲ Layer 3 × Layer 4 ✱ Layer 5

Appendix H: Sensitivity Analysis

Original Model

	Calibrated Model	1	Comments	2	Comments	3	Comments	4	Comments	5	Comments	6	Comments
Conductivity (m/s):													
Limestone (Layer 1,2,3,4)	5.00E-06	1.00E-05	*2	2.50E-06	/2	No Change		No Change		No Change		No Change	
Sandstone (Layer 5)	1.30E-04	No Change		No Change		2.60E-04	*2	6.50E-05	/2	No Change		No Change	
Clay (Layer 1)	1.00E-08	No Change		No Change		No Change		No Change		2.00E-08	*2	5.00E-09	/2
Till (Layer 1)	1.00E-08	No Change		No Change		No Change		No Change		No Change		No Change	
Sand (Layer 1)	5.00E-04	No Change		No Change		No Change		No Change		No Change		No Change	
Swamp (Layer 1)	5.00E-07	No Change		No Change		No Change		No Change		No Change		No Change	
Porosity:													
Limestone (Layer 1,2,3,4)	0.001	No Change		No Change		No Change		No Change		No Change		No Change	
Sandstone (Layer 5)	0.02	No Change		No Change		No Change		No Change		No Change		No Change	
Clay (Layer 1)	0.25	No Change		No Change		No Change		No Change		No Change		No Change	
Till (Layer 1)	0.25	No Change		No Change		No Change		No Change		No Change		No Change	
Sand (Layer 1)	0.25	No Change		No Change		No Change		No Change		No Change		No Change	
Swamp (Layer 1)	0.25	No Change		No Change		No Change		No Change		No Change		No Change	
Recharge (mm/yr):													
Till	5	No Change		No Change		No Change		No Change		No Change		No Change	
Sand	30	No Change		No Change		No Change		No Change		No Change		No Change	
Clay	5	No Change		No Change		No Change		No Change		No Change		No Change	
Bedrock	200	No Change		No Change		No Change		No Change		No Change		No Change	
Swamp	60	No Change		No Change		No Change		No Change		No Change		No Change	
Statistics:													
All:				Model did not Converge									
Abs. residual mean (m):	3.82	3.85	0.84		-100.00	5.74	50.05	8.64	126.06	3.82	-0.03	3.82	0.05
Normalized RMS (%)	7.21	7.24	0.36		-100.00	10.28	42.50	14.26	97.68	7.21	-0.07	7.22	0.08
Layer 1:													
Abs. residual mean (m):	2.15	2.08	-2.98		-100.00	1.35	-37.26	3.54	64.65	2.10	-2.10	2.23	4.01
Normalized RMS (%)	6.47	6.42	-0.91		-100.00	6.05	-6.61	10.67	64.77	6.40	-1.14	6.64	2.50
Layer 2:													
Abs. residual mean (m):	2.96	2.97	0.24		-100.00	3.27	10.40	4.79	61.78	2.96	-0.03	2.96	0.00
Normalized RMS (%)	5.86	5.94	1.33		-100.00	7.62	30.11	9.34	59.46	5.86	0.02	5.86	-0.02
Layer 3:													
Abs. residual mean (m):	4.66	4.69	0.69		-100.00	6.68	43.25	8.88	90.41	4.66	-0.04	4.66	0.04
Normalized RMS (%)	9.09	9.18	1.01		-100.00	12.95	42.46	15.69	72.65	9.09	-0.01	9.09	0.01
Layer 4:													
Abs. residual mean (m):	4.93	4.97	0.65		-100.00	6.76	36.93	10.69	116.76	4.94	0.06	4.93	-0.04
Normalized RMS (%)	10.19	10.17	-0.26		-100.00	13.02	27.75	18.93	85.74	10.19	-0.02	10.19	0.01
Layer 5:													
Abs. residual mean (m):	3.51	3.57	1.48		-100.00	6.36	80.93	10.12	188.08	3.52	0.03	3.52	0.03
Normalized RMS (%)	7.00	6.99	-0.14		-100.00	10.93	56.20	16.64	137.66	6.99	-0.16	7.01	0.19

	Calibrated Model	7	Comments	8	Comments	9	Comments	10	Comments	11	Comments	12	Comments
Conductivity (m/s):													
Limestone (Layer 1,2,3,4)	5.00E-06	No Change		No Change		No Change		No Change		No Change		No Change	
Sandstone (Layer 5)	1.30E-04	No Change		No Change		No Change		No Change		No Change		No Change	
Clay (Layer 1)	1.00E-08	No Change		No Change		No Change		No Change		No Change		No Change	
Till (Layer 1)	1.00E-08	2.00E-08	*2	5.00E-09	/2	No Change		No Change		No Change		No Change	
Sand (Layer 1)	5.00E-04	No Change		No Change		1.00E-03	*2	2.40E-04	/2	No Change		No Change	
Swamp (Layer 1)	5.00E-07	No Change		No Change		No Change		No Change		1.00E-06	*2	2.50E-07	/2
Porosity:													
Limestone (Layer 1,2,3,4)	0.001	No Change		No Change		No Change		No Change		No Change		No Change	
Sandstone (Layer 5)	0.02	No Change		No Change		No Change		No Change		No Change		No Change	
Clay (Layer 1)	0.25	No Change		No Change		No Change		No Change		No Change		No Change	
Till (Layer 1)	0.25	No Change		No Change		No Change		No Change		No Change		No Change	
Sand (Layer 1)	0.25	No Change		No Change		No Change		No Change		No Change		No Change	
Swamp (Layer 1)	0.25	No Change		No Change		No Change		No Change		No Change		No Change	
Recharge (mm/yr):													
Till	5	No Change		No Change		No Change		No Change		No Change		No Change	
Sand	30	No Change		No Change		No Change		No Change		No Change		No Change	
Clay	5	No Change		No Change		No Change		No Change		No Change		No Change	
Bedrock	200	No Change		No Change		No Change		No Change		No Change		No Change	
Swamp	60	No Change		No Change		No Change		No Change		No Change		No Change	
Statistics:													
All:													
Abs. residual mean (m):	3.82	3.82	0.00	3.82	0.00	3.89	1.81	3.79	-0.73	3.83	0.24	3.81	-0.26
Normalized RMS (%)	7.21	7.21	0.00	7.21	0.00	7.25	0.50	7.24	0.35	7.21	0.01	7.21	0.04
Layer 1:													
Abs. residual mean (m):	2.15	2.15	0.00	2.15	0.00	2.17	1.12	2.12	-1.30	2.15	-0.09	2.15	0.14
Normalized RMS (%)	6.47	6.47	0.00	6.48	0.02	6.66	2.83	6.35	-1.99	6.50	0.34	6.46	-0.29
Layer 2:													
Abs. residual mean (m):	2.96	2.96	-0.03	2.96	0.00	3.05	2.90	2.87	-3.21	2.96	0.07	2.96	-0.10
Normalized RMS (%)	5.86	5.86	0.00	5.86	0.02	5.97	1.95	5.69	-2.87	5.87	0.12	5.85	-0.15
Layer 3:													
Abs. residual mean (m):	4.66	4.66	-0.02	4.66	0.00	4.69	0.69	4.67	0.26	4.67	0.11	4.65	-0.17
Normalized RMS (%)	9.09	9.09	-0.02	9.09	0.01	9.18	0.96	9.09	-0.04	9.10	0.07	9.08	-0.10
Layer 4:													
Abs. residual mean (m):	4.93	4.93	0.00	4.93	0.00	4.99	1.05	4.95	0.26	0.36	-92.62	4.91	-0.41
Normalized RMS (%)	10.19	10.19	-0.01	10.19	0.00	10.14	-0.51	10.32	1.29	10.20	0.04	10.19	-0.04
Layer 5:													
Abs. residual mean (m):	3.51	3.51	0.00	3.51	0.00	3.60	2.39	3.49	-0.80	3.53	0.37	3.50	-0.34
Normalized RMS (%)	7.00	7.00	0.00	7.00	0.00	7.01	0.07	7.07	1.06	6.99	-0.10	7.02	0.29

	Calibrated Model	25	Comments	26	Comments	27	Comments	28	Comments	29	Comments
Conductivity (m/s):											
Limestone (Layer 1,2,3,4)	5.00E-06	No Change		No Change		No Change		No Change		No Change	
Sandstone (Layer 5)	1.30E-04	No Change		No Change		No Change		No Change		No Change	
Clay (Layer 1)	1.00E-08	No Change		No Change		No Change		No Change		No Change	
Till (Layer 1)	1.00E-08	No Change		No Change		No Change		No Change		No Change	
Sand (Layer 1)	5.00E-04	No Change		No Change		No Change		No Change		No Change	
Swamp (Layer 1)	5.00E-07	No Change		No Change		No Change		No Change		No Change	
Porosity:											
Limestone (Layer 1,2,3,4)	0.001	No Change		No Change		No Change		No Change		No Change	
Sandstone (Layer 5)	0.02	No Change		No Change		No Change		No Change		No Change	
Clay (Layer 1)	0.25	No Change		No Change		No Change		No Change		No Change	
Till (Layer 1)	0.25	No Change		No Change		No Change		No Change		No Change	
Sand (Layer 1)	0.25	No Change		No Change		No Change		No Change		No Change	
Swamp (Layer 1)	0.25	No Change		No Change		No Change		No Change		No Change	
Recharge (mm/yr):											
Till	5	10	*2	2.5	/2	No Change		No Change		No Change	
Sand	30	No Change		No Change		60	*2	15	/2	No Change	
Clay	5	No Change		No Change		No Change		No Change		10	*2
Bedrock	200	No Change		No Change		No Change		No Change		No Change	
Swamp	60	No Change		No Change		No Change		No Change		No Change	
Statistics:											
All:											
Abs. residual mean (m):	3.82	3.82	-0.13	3.83	0.08	3.88	1.54	3.86	0.94	3.83	0.21
Normalized RMS (%)	7.21	7.22	0.06	7.21	-0.01	7.39	2.45	7.22	0.12	7.23	0.22
Layer 1:											
Abs. residual mean (m):	2.15	2.15	0.33	2.14	-0.19	2.29	6.43	2.08	-3.07	2.30	7.03
Normalized RMS (%)	6.47	6.47	-0.03	6.48	0.05	6.71	3.58	6.48	0.11	6.77	4.57
Layer 2:											
Abs. residual mean (m):	2.96	2.96	-0.07	2.96	0.03	3.03	2.43	2.95	-0.51	2.99	0.95
Normalized RMS (%)	5.86	5.85	-0.17	5.87	0.10	5.89	0.51	5.90	0.61	5.89	0.51
Layer 3:											
Abs. residual mean (m):	4.66	4.66	-0.02	4.66	0.00	4.73	1.57	4.66	0.00	4.66	0.06
Normalized RMS (%)	9.09	9.09	-0.03	9.09	0.02	9.13	0.47	9.15	0.63	9.09	0.00
Layer 4:											
Abs. residual mean (m):	4.93	4.93	-0.10	4.94	0.08	4.98	0.99	4.99	1.07	4.93	-0.04
Normalized RMS (%)	10.19	10.20	0.13	10.18	-0.07	10.44	2.47	10.15	-0.36	10.21	0.14
Layer 5:											
Abs. residual mean (m):	3.51	3.50	-0.28	3.52	0.17	3.56	1.22	3.60	2.33	3.51	-0.14
Normalized RMS (%)	7.00	7.01	0.16	7.00	-0.07	7.34	4.90	6.99	-0.19	7.02	0.31

	Calibrated Model	30	Comments	31	Comments	32	Comments	33	Comments	34	Comments
Conductivity (m/s):											
Limestone (Layer 1,2,3,4)	5.00E-06	No Change		No Change		No Change		No Change		No Change	
Sandstone (Layer 5)	1.30E-04	No Change		No Change		No Change		No Change		No Change	
Clay (Layer 1)	1.00E-08	No Change		No Change		No Change		No Change		No Change	
Till (Layer 1)	1.00E-08	No Change		No Change		No Change		No Change		No Change	
Sand (Layer 1)	5.00E-04	No Change		No Change		No Change		No Change		No Change	
Swamp (Layer 1)	5.00E-07	No Change		No Change		No Change		No Change		No Change	
Porosity:											
Limestone (Layer 1,2,3,4)	0.001	No Change		No Change		No Change		No Change		No Change	
Sandstone (Layer 5)	0.02	No Change		No Change		No Change		No Change		No Change	
Clay (Layer 1)	0.25	No Change		No Change		No Change		No Change		No Change	
Till (Layer 1)	0.25	No Change		No Change		No Change		No Change		No Change	
Sand (Layer 1)	0.25	No Change		No Change		No Change		No Change		No Change	
Swamp (Layer 1)	0.25	No Change		No Change		No Change		No Change		No Change	
Recharge (mm/yr):											
Till	5	No Change		No Change		No Change		No Change		No Change	
Sand	30	No Change		No Change		No Change		No Change		No Change	
Clay	5	2.5	/2	No Change		No Change		No Change		No Change	
Bedrock	200	No Change		400	*2	100	/2	No Change		No Change	
Swamp	60	No Change		No Change		No Change		120	*2	30	/2
Statistics:											
All:											
Abs. residual mean (m):	3.82	3.82	-0.08	7.71	101.73	5.17	35.22	4.04	5.63	4.01	4.95
Normalized RMS (%)	7.21	7.20	-0.10	12.92	79.21	9.27	28.55	7.64	5.88	7.40	2.62
Layer 1:											
Abs. residual mean (m):	2.15	2.07	-3.49	3.13	45.78	1.52	-29.20	2.28	6.05	2.12	-1.40
Normalized RMS (%)	6.47	6.34	-2.15	9.33	44.10	6.69	3.29	6.54	1.00	6.75	4.25
Layer 2:											
Abs. residual mean (m):	2.96	2.95	-0.47	4.23	42.64	3.41	15.16	2.94	-0.81	3.01	1.62
Normalized RMS (%)	5.86	5.85	-0.22	8.32	42.02	7.53	28.52	5.67	-3.16	6.05	3.29
Layer 3:											
Abs. residual mean (m):	4.66	4.66	-0.04	8.14	74.58	6.09	30.70	4.76	2.06	4.82	3.48
Normalized RMS (%)	9.09	9.09	0.00	14.56	60.19	11.83	30.11	9.13	0.48	9.43	3.69
Layer 4:											
Abs. residual mean (m):	4.93	4.94	0.06	9.70	96.70	6.17	25.10	5.22	5.72	5.18	4.95
Normalized RMS (%)	10.19	10.18	-0.07	17.52	71.87	11.94	17.14	10.86	6.55	10.27	0.78
Layer 5:											
Abs. residual mean (m):	3.51	3.52	0.11	8.89	152.96	5.42	54.24	3.90	11.07	3.80	8.25
Normalized RMS (%)	7.00	6.99	-0.14	14.80	111.47	9.46	35.19	7.93	13.33	7.18	2.54

	Calibrated Model	1	Comments	2	Comments	3	Comments	4	Comments	5	Comments	6	Comments
Conductivity (m/s):													
Limestone (Layer 1,2,3,4)	5.00E-06	1.00E-05	*2	2.50E-06	/2	No Change		No Change		No Change		No Change	
Sandstone (Layer 5)	5.00E-05	No Change		No Change		1.00E-04	*2	2.50E-05	/2	No Change		No Change	
Clay (Layer 1)	4.00E-09	No Change		No Change		No Change		No Change		8.00E-09	*2	2.00E-09	/2
Till (Layer 1)	1.00E-07	No Change		No Change		No Change		No Change		No Change		No Change	
Sand (Layer 1)	5.00E-04	No Change		No Change		No Change		No Change		No Change		No Change	
Swamp (Layer 1)	5.00E-07	No Change		No Change		No Change		No Change		No Change		No Change	
Porosity:													
Limestone (Layer 1,2,3,4)	0.001	No Change		No Change		No Change		No Change		No Change		No Change	
Sandstone (Layer 5)	0.02	No Change		No Change		No Change		No Change		No Change		No Change	
Clay (Layer 1)	0.25	No Change		No Change		No Change		No Change		No Change		No Change	
Till (Layer 1)	0.25	No Change		No Change		No Change		No Change		No Change		No Change	
Sand (Layer 1)	0.25	No Change		No Change		No Change		No Change		No Change		No Change	
Swamp (Layer 1)	0.25	No Change		No Change		No Change		No Change		No Change		No Change	
Recharge (mm/yr):													
Till	5	No Change		No Change		No Change		No Change		No Change		No Change	
Sand	100	No Change		No Change		No Change		No Change		No Change		No Change	
Clay	5	No Change		No Change		No Change		No Change		No Change		No Change	
Bedrock	120	No Change		No Change		No Change		No Change		No Change		No Change	
Swamp	60	No Change		No Change		No Change		No Change		No Change		No Change	
Statistics:													
All:													
Abs. residual mean (m):	3.73	3.72	-0.48	3.74	0.13	5.56	48.81	5.31	42.30	3.73	-0.05	3.74	0.16
Normalized RMS (%)	7.24	7.20	-0.61	7.27	0.40	9.79	35.19	9.28	28.09	7.24	-0.03	7.25	0.08
Layer 1:													
Abs. residual mean (m):	2.44	2.37	-2.86	2.46	0.78	1.52	-38.01	3.08	25.94	2.35	-3.97	2.66	8.72
Normalized RMS (%)	7.35	7.22	-1.75	7.37	0.26	7.21	-2.00	8.39	14.05	7.20	-2.01	7.75	5.36
Layer 2:													
Abs. residual mean (m):	2.92	2.87	-1.75	2.93	0.14	3.22	10.20	3.87	32.58	2.92	0.03	2.92	-0.03
Normalized RMS (%)	5.71	5.67	-0.81	5.71	-0.12	7.31	27.97	7.15	25.20	5.72	0.04	5.71	-0.04
Layer 3:													
Abs. residual mean (m):	4.61	4.58	-0.74	4.64	0.61	6.05	31.07	5.64	22.27	4.62	0.09	4.61	-0.07
Normalized RMS (%)	8.96	8.90	-0.67	9.00	0.44	11.48	28.05	10.42	16.29	8.97	0.07	8.96	-0.04
Layer 4:													
Abs. residual mean (m):	5.69	5.64	-0.98	5.74	0.74	6.67	17.05	7.42	30.30	5.69	-0.07	5.70	0.07
Normalized RMS (%)	11.68	11.55	-1.10	11.78	0.84	13.10	12.12	14.17	21.32	11.68	-0.03	11.68	0.03
Layer 5:													
Abs. residual mean (m):	3.20	3.22	0.59	3.19	-0.31	10.73	235.45	5.36	67.61	3.20	-0.03	3.20	0.09
Normalized RMS (%)	6.71	6.70	-0.04	6.72	0.13	6.31	-5.89	9.69	44.53	6.70	-0.12	6.72	0.13

	Calibrated Model	7	Comments	8	Comments	9	Comments	10	Comments	11	Comments	12	Comments
Conductivity (m/s):													
Limestone (Layer 1,2,3,4)	5.00E-06	No Change		No Change		No Change		No Change		No Change		No Change	
Sandstone (Layer 5)	5.00E-05	No Change		No Change		No Change		No Change		No Change		No Change	
Clay (Layer 1)	4.00E-09	No Change		No Change		No Change		No Change		No Change		No Change	
Till (Layer 1)	1.00E-07	2.00E-07	*2	5.00E-08	/2	No Change		No Change		No Change		No Change	
Sand (Layer 1)	5.00E-04	No Change		No Change		1.00E-03	*2	2.50E-04	/2	No Change		No Change	
Swamp (Layer 1)	5.00E-07	No Change		No Change		No Change		No Change		1.00E-06	*2	2.50E-07	/2
Porosity:													
Limestone (Layer 1,2,3,4)	0.001	No Change		No Change		No Change		No Change		No Change		No Change	
Sandstone (Layer 5)	0.02	No Change		No Change		No Change		No Change		No Change		No Change	
Clay (Layer 1)	0.25	No Change		No Change		No Change		No Change		No Change		No Change	
Till (Layer 1)	0.25	No Change		No Change		No Change		No Change		No Change		No Change	
Sand (Layer 1)	0.25	No Change		No Change		No Change		No Change		No Change		No Change	
Swamp (Layer 1)	0.25	No Change		No Change		No Change		No Change		No Change		No Change	
Recharge (mm/yr):													
Till	5	No Change		No Change		No Change		No Change		No Change		No Change	
Sand	100	No Change		No Change		No Change		No Change		No Change		No Change	
Clay	5	No Change		No Change		No Change		No Change		No Change		No Change	
Bedrock	120	No Change		No Change		No Change		No Change		No Change		No Change	
Swamp	60	No Change		No Change		No Change		No Change		No Change		No Change	
Statistics:													
All:													
Abs. residual mean (m):	3.73	3.73	0.00	3.73	0.03	3.77	0.96	3.74	0.27	3.74	0.05	3.73	0.00
Normalized RMS (%)	7.24	7.24	0.00	7.25	0.01	7.21	-0.43	7.35	1.41	7.24	-0.03	7.25	0.08
Layer 1:													
Abs. residual mean (m):	2.44	2.44	0.00	2.44	0.00	2.45	0.25	2.43	-0.70	2.45	0.12	2.44	-0.25
Normalized RMS (%)	7.35	7.35	-0.01	7.35	0.00	7.51	2.08	7.22	-1.82	7.38	0.39	7.31	-0.63
Layer 2:													
Abs. residual mean (m):	2.92	2.92	-0.03	2.92	0.00	3.00	2.81	2.87	-1.88	2.92	0.00	2.92	0.00
Normalized RMS (%)	5.71	5.71	-0.02	5.71	0.00	5.85	2.42	5.64	-1.38	5.72	0.02	5.71	-0.04
Layer 3:													
Abs. residual mean (m):	4.61	4.61	-0.02	4.61	0.02	4.62	0.07	4.64	0.67	4.62	0.09	4.61	-0.13
Normalized RMS (%)	8.96	8.96	-0.01	8.96	0.02	8.93	-0.35	9.08	1.34	8.97	0.03	8.96	-0.04
Layer 4:													
Abs. residual mean (m):	5.69	5.69	-0.02	5.70	0.04	5.63	-1.16	5.82	2.16	5.69	0.00	5.70	0.02
Normalized RMS (%)	11.68	11.68	-0.02	11.68	0.02	11.44	-2.04	12.01	2.82	11.68	-0.03	11.69	0.05
Layer 5:													
Abs. residual mean (m):	3.20	3.20	0.00	3.20	0.00	3.26	1.91	3.20	-0.03	3.20	0.06	3.20	0.06
Normalized RMS (%)	6.71	6.71	0.00	6.71	0.00	6.69	-0.18	6.79	1.21	6.70	-0.12	6.73	0.28

	Calibrated Model	25	Comments	26	Comments	27	Comments	28	Comments	29	Comments
Conductivity (m/s):											
Limestone (Layer 1,2,3,4)	5.00E-06	No Change		No Change		No Change		No Change		No Change	
Sandstone (Layer 5)	5.00E-05	No Change		No Change		No Change		No Change		No Change	
Clay (Layer 1)	4.00E-09	No Change		No Change		No Change		No Change		No Change	
Till (Layer 1)	1.00E-07	No Change		No Change		No Change		No Change		No Change	
Sand (Layer 1)	5.00E-04	No Change		No Change		No Change		No Change		No Change	
Swamp (Layer 1)	5.00E-07	No Change		No Change		No Change		No Change		No Change	
Porosity:											
Limestone (Layer 1,2,3,4)	0.001	No Change		No Change		No Change		No Change		No Change	
Sandstone (Layer 5)	0.02	No Change		No Change		No Change		No Change		No Change	
Clay (Layer 1)	0.25	No Change		No Change		No Change		No Change		No Change	
Till (Layer 1)	0.25	No Change		No Change		No Change		No Change		No Change	
Sand (Layer 1)	0.25	No Change		No Change		No Change		No Change		No Change	
Swamp (Layer 1)	0.25	No Change		No Change		No Change		No Change		No Change	
Recharge (mm/yr):											
Till	5	10	*2	2.5	/2	No Change		No Change		No Change	
Sand	100	No Change		No Change		200	*2	50	/2	No Change	
Clay	5	No Change		No Change		No Change		No Change		10	*2
Bedrock	120	No Change		No Change		No Change		No Change		No Change	
Swamp	60	No Change		No Change		No Change		No Change		No Change	
Statistics:											
All:											
Abs. residual mean (m):	3.73	3.74	0.08	3.73	0.00	4.44	18.89	3.87	3.78	3.75	0.51
Normalized RMS (%)	7.24	7.26	0.21	7.24	-0.08	8.28	14.33	7.29	0.62	7.28	0.43
Layer 1:											
Abs. residual mean (m):	2.44	2.45	0.29	2.44	-0.12	2.93	19.97	1.86	-23.90	2.74	12.03
Normalized RMS (%)	7.35	7.35	-0.07	7.35	0.03	8.13	10.51	7.91	7.52	7.93	7.92
Layer 2:											
Abs. residual mean (m):	2.92	2.93	0.21	2.92	-0.10	3.54	21.15	2.80	-4.31	2.96	1.27
Normalized RMS (%)	5.71	5.72	0.11	5.71	-0.05	6.68	16.96	5.61	-1.75	5.76	0.86
Layer 3:											
Abs. residual mean (m):	4.61	4.62	0.09	4.61	-0.02	5.12	10.91	4.63	0.41	4.62	0.09
Normalized RMS (%)	8.96	8.98	0.18	8.96	-0.08	9.79	9.22	8.98	0.16	8.97	0.13
Layer 4:											
Abs. residual mean (m):	5.69	5.71	0.25	5.69	-0.11	6.56	15.26	5.67	-0.42	5.71	0.21
Normalized RMS (%)	11.68	11.71	0.29	11.66	-0.14	13.14	12.47	11.41	-2.29	11.71	0.22
Layer 5:											
Abs. residual mean (m):	3.20	3.20	-0.13	3.20	0.09	4.01	25.38	3.61	12.85	3.20	0.16
Normalized RMS (%)	6.71	6.72	0.18	6.70	-0.07	8.03	19.77	7.01	4.49	6.74	0.55

	Calibrated Model	30	Comments	31	Comments	32	Comments	33	Comments	34	Comments
Conductivity (m/s):											
Limestone (Layer 1,2,3,4)	5.00E-06	No Change		No Change		No Change		No Change		No Change	
Sandstone (Layer 5)	5.00E-05	No Change		No Change		No Change		No Change		No Change	
Clay (Layer 1)	4.00E-09	No Change		No Change		No Change		No Change		No Change	
Till (Layer 1)	1.00E-07	No Change		No Change		No Change		No Change		No Change	
Sand (Layer 1)	5.00E-04	No Change		No Change		No Change		No Change		No Change	
Swamp (Layer 1)	5.00E-07	No Change		No Change		No Change		No Change		No Change	
Porosity:											
Limestone (Layer 1,2,3,4)	0.001	No Change		No Change		No Change		No Change		No Change	
Sandstone (Layer 5)	0.02	No Change		No Change		No Change		No Change		No Change	
Clay (Layer 1)	0.25	No Change		No Change		No Change		No Change		No Change	
Till (Layer 1)	0.25	No Change		No Change		No Change		No Change		No Change	
Sand (Layer 1)	0.25	No Change		No Change		No Change		No Change		No Change	
Swamp (Layer 1)	0.25	No Change		No Change		No Change		No Change		No Change	
Recharge (mm/yr):											
Till	5	No Change		No Change		No Change		No Change		No Change	
Sand	100	No Change		No Change		No Change		No Change		No Change	
Clay	5	2.5	/2	No Change		No Change		No Change		No Change	
Bedrock	120	No Change		240	*2	60	/2	No Change		No Change	
Swamp	60	No Change		No Change		No Change		120	*2	30	/2
Statistics:											
All:											
Abs. residual mean (m):	3.73	3.73	-0.21	4.17	11.57	4.16	11.33	3.81	2.09	3.81	2.04
Normalized RMS (%)	7.24	7.23	-0.19	7.95	9.76	7.67	5.84	7.40	2.15	7.29	0.65
Layer 1:											
Abs. residual mean (m):	2.44	2.30	-5.81	2.60	6.18	1.98	-19.15	2.54	4.09	2.44	-0.04
Normalized RMS (%)	7.35	7.08	-3.69	7.46	1.46	8.25	12.16	7.46	1.40	7.59	3.21
Layer 2:											
Abs. residual mean (m):	2.92	2.90	-0.65	3.10	5.95	3.14	7.53	2.98	1.85	2.92	0.07
Normalized RMS (%)	5.71	5.69	-0.42	5.88	2.87	6.35	11.10	5.74	0.51	5.77	1.00
Layer 3:											
Abs. residual mean (m):	4.61	4.61	-0.04	4.99	8.28	4.97	7.76	4.61	-0.15	4.67	1.17
Normalized RMS (%)	8.96	8.96	-0.06	9.61	7.22	9.55	6.54	8.96	-0.01	9.05	0.98
Layer 4:											
Abs. residual mean (m):	5.69	5.69	-0.09	6.45	13.19	5.76	1.12	5.85	2.67	5.71	0.23
Normalized RMS (%)	11.68	11.67	-0.10	12.97	11.00	11.52	-1.34	11.98	2.57	11.60	-0.68
Layer 5:											
Abs. residual mean (m):	3.20	3.20	-0.03	3.71	15.82	3.90	21.79	3.31	3.41	3.34	4.53
Normalized RMS (%)	6.71	6.69	-0.25	7.61	13.41	7.35	9.56	7.00	4.41	6.79	1.25

Appendix I: Local Scale Numerical Model Results

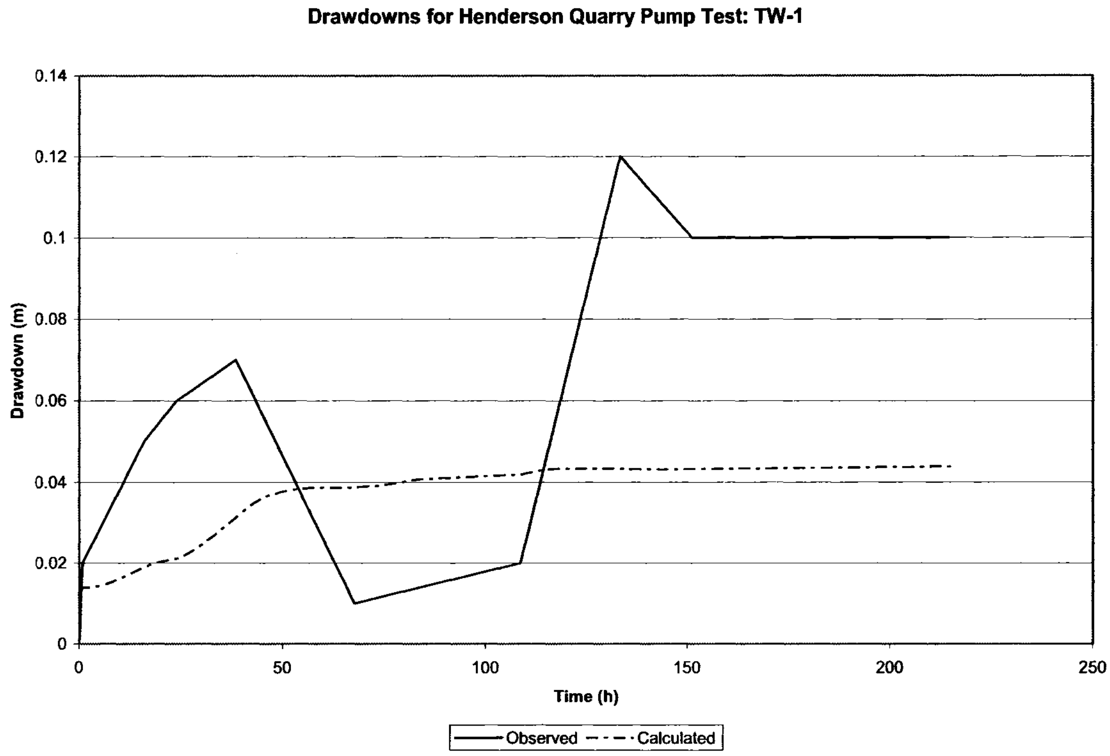
Henderson Quarry

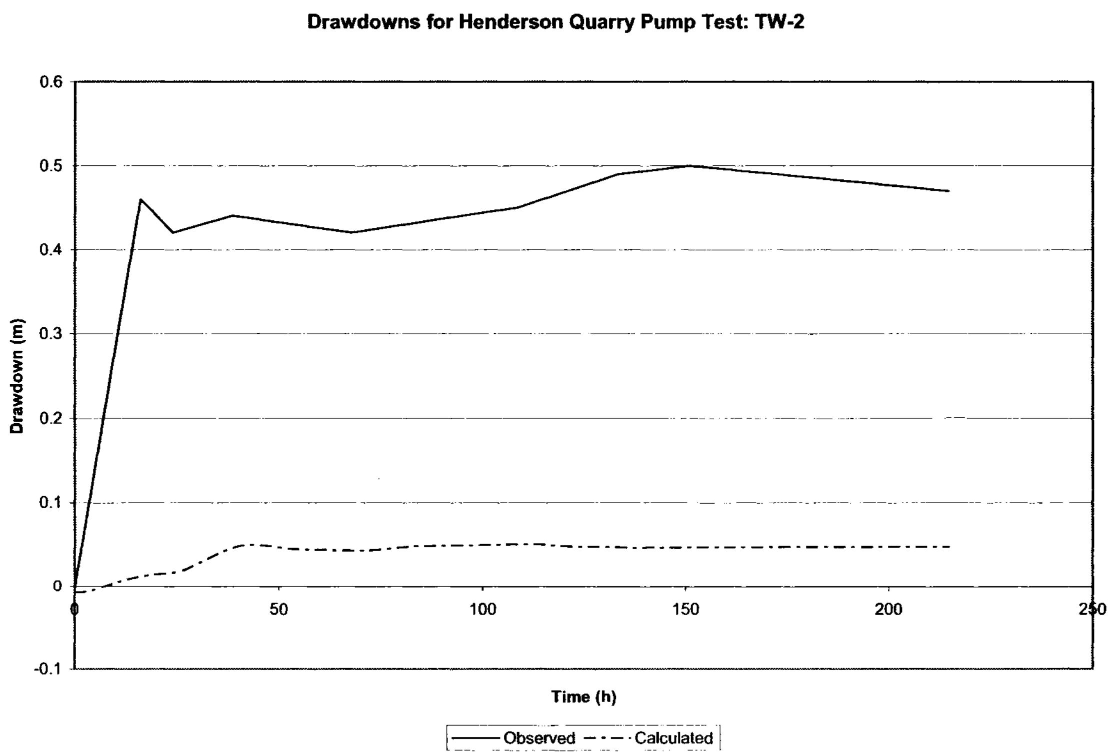
Pumping rate from well TW5

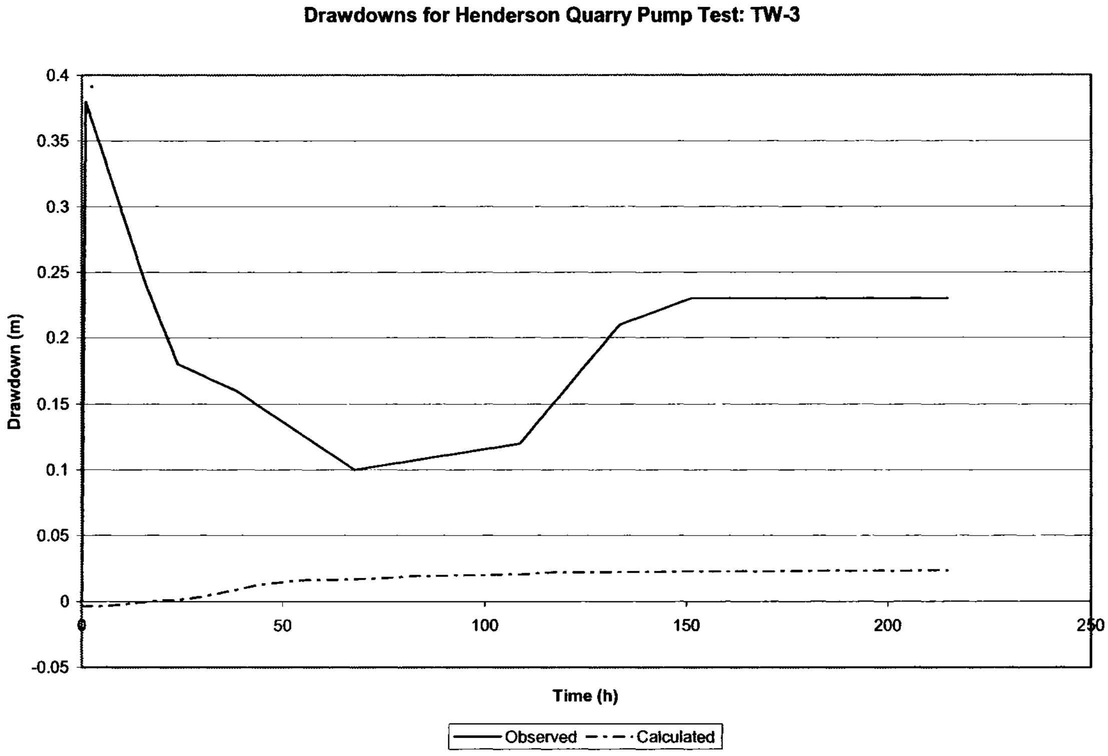
Elapsed Time (h)	Pump Rate (L/min)
0.00	30
0.67	30
16.30	29
24.25	29
38.87	60
67.67	28
108.67	40
108.75	30
133.42	28
151.17	32
214.67	32

Measured heads at observation wells for TW5 pump test

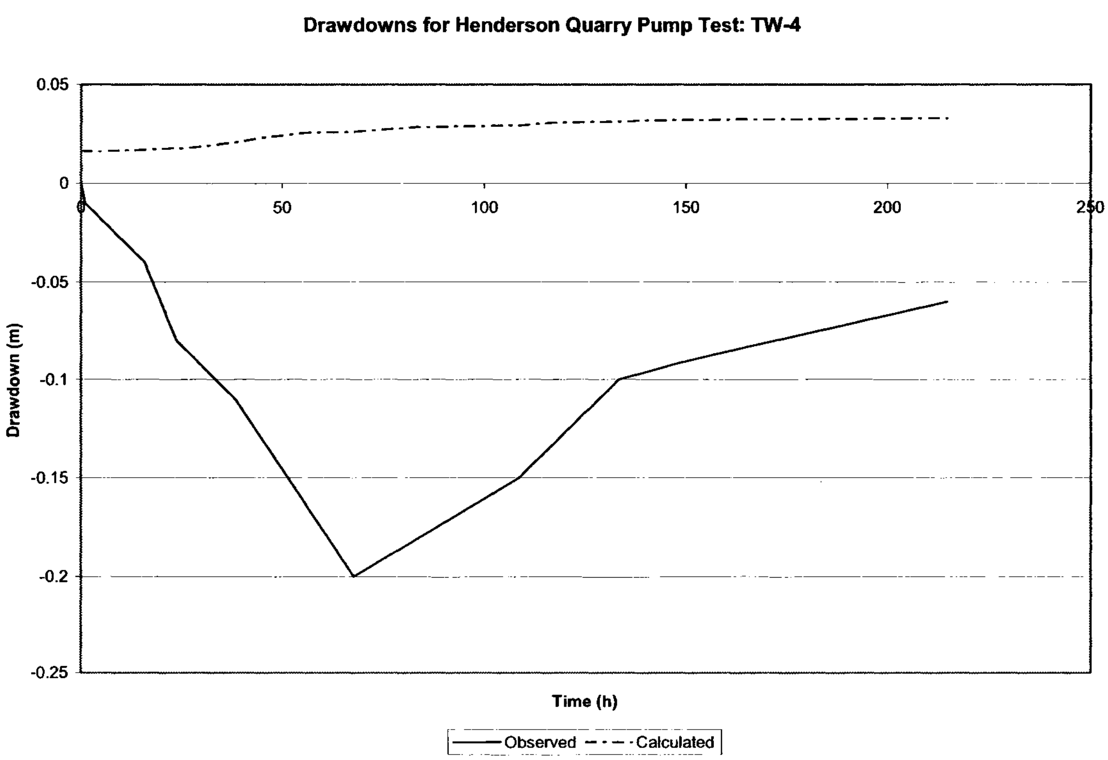
Time	TW1	TW2	TW3	TW4	TW5	TW6
0.00	139.63	141.18	141	140.22	135	135.2
0.78	139.61		140.62	140.23	131.16	135.12
16.05	139.58	140.72	140.76	140.26	130.59	134.8
24.03	139.57	140.76	140.82	140.3	130.59	134.74
38.57	139.56	140.74	140.84	140.33	123.79	134.3
67.67	139.62	140.76	140.9	140.42	131.59	134.47
108.67	139.61	140.73	140.88	140.37	128.6	134.25
133.42	139.51	140.69	140.79	140.32	131.91	134.33
151.17	139.53	140.68	140.77	140.31	131.33	134.27
214.67	139.53	140.71	140.77	140.28	131.8	134.24


Drawdowns for Henderson Quarry Pump Test: TW-1
0.14
0.12
0.1
0.08
0.06
0.04
0.02
0
Drawdown (m)
0
50
100
150
200
250
Time (h)
Observed
Calculated

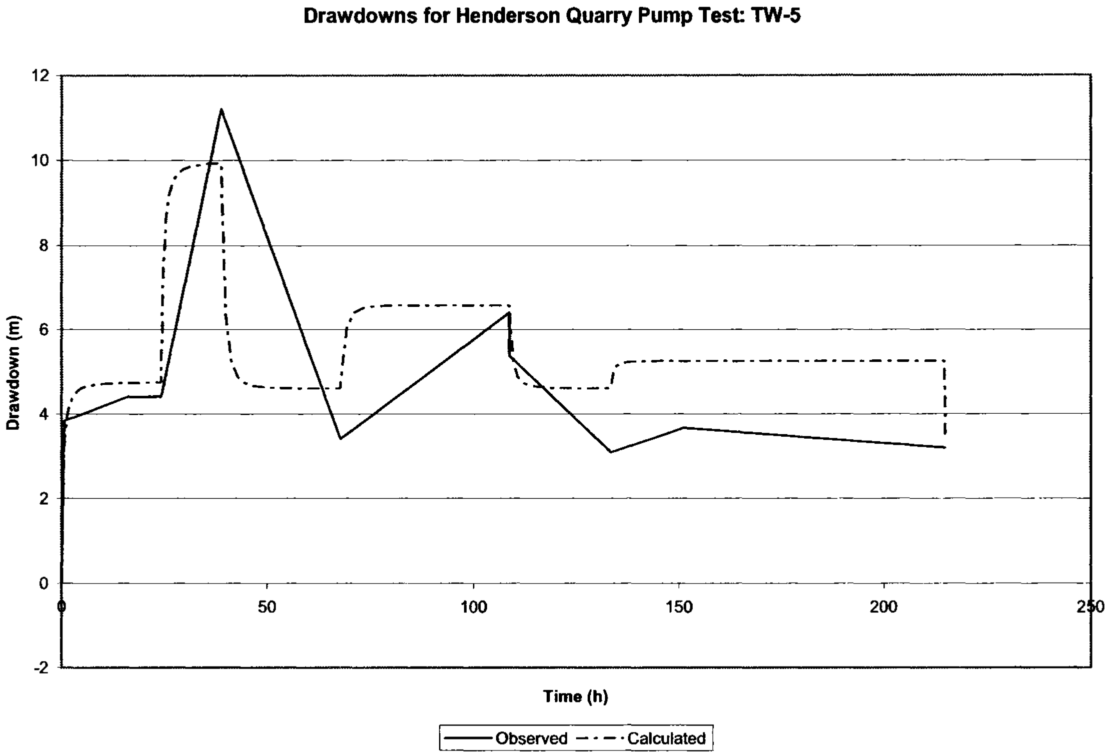

Drawdowns for Henderson Quarry Pump Test: TW-2
0.6
0.5
0.4
0.3
0.2
0.1
0
-0.1
Drawdown (m)
0
50
100
150
200
250
Time (h)
Observed
Calculated

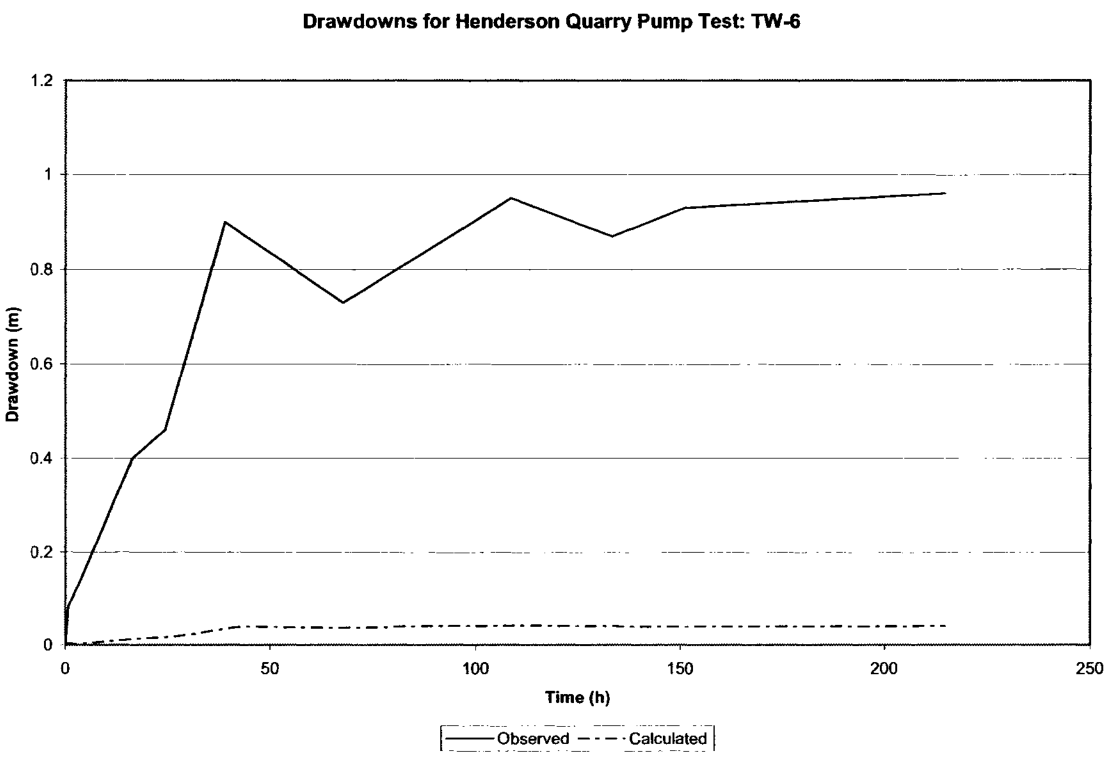


Drawdowns for Henderson Quarry Pump Test: TW-3
0.4
0.35
0.3
0.25
0.2
0.15
0.1
0.05
0
-0.05
0
50
100
150
200
250
Drawdown (m)
Time (h)
Observed
Calculated



Drawdowns for Henderson Quarry Pump Test: TW-4
0.05
0
-0.05
-0.1
-0.15
-0.2
-0.25
0
50
100
150
200
250
Drawdown (m)
Time (h)
Observed
Calculated


Drawdowns for Henderson Quarry Pump Test: TW-5
12
10
8
6
4
2
0
-2
0
50
100
150
200
250
Drawdown (m)
Time (h)
Observed
Calculated


Drawdowns for Henderson Quarry Pump Test: TW-6
1.2
1
0.8
0.6
0.4
0.2
0
0
50
100
150
200
250
Drawdown (m)
Time (h)
Observed
Calculated

Beagle Club Quarry

Pumping rate from quarry

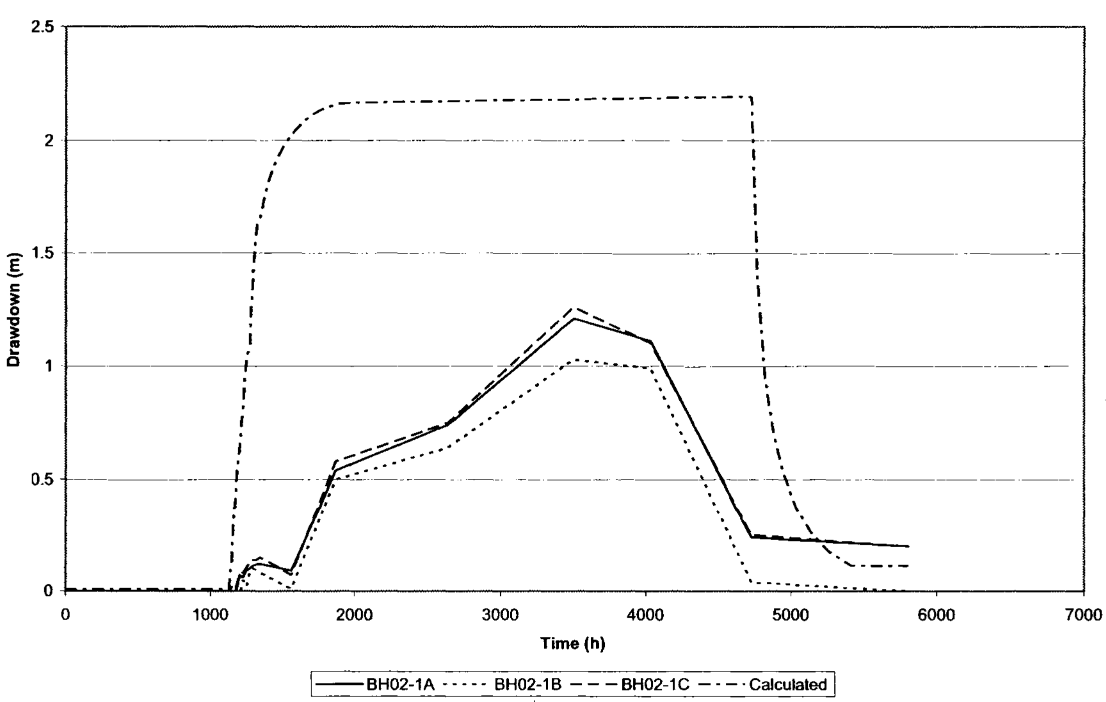
Elapsed Time (hrs)	Pump Rate (L/min)	Elapsed Time (hrs)	Pump Rate (L/min)
0		1293	2700
552		1323	
1128.5	6800	1347	2300
1130.5	7300	1563	1900
1148	7000	1872	
1173	3600	2640	
1203	2100	3504	
1226	1850	4032	
1250.5		4728	
1274.5		5808	

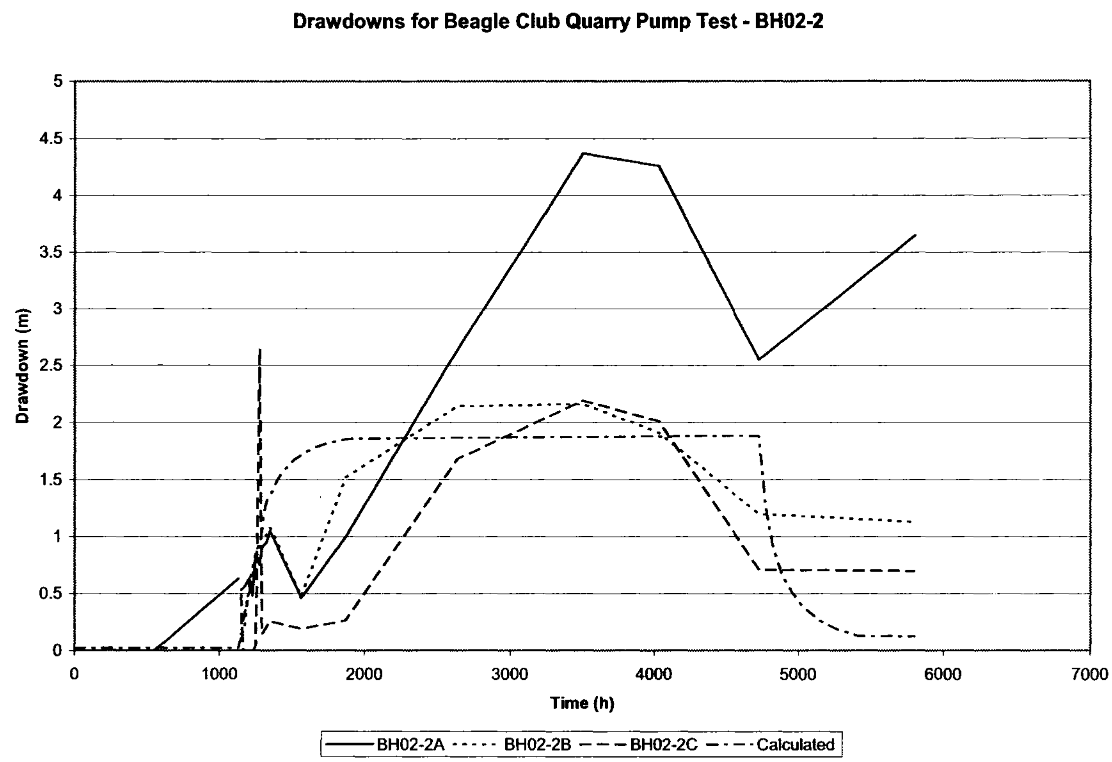
Measured heads in test wells and quarry

Time (hrs)	BH02-1A	BH02-1B	BH02-1C	BH02-2A	BH02-2B	BH02-2C	BH02-3A
0				141.43	135.64	142.76	137.27
552				141.43	135.64	142.76	137.27
1128.5	135.26	135.18	132.29	140.8	136.01	143.03	
1130.5	135.26	135.18	132.29				
1148	135.28	135.23	132.27	140.9	135.5	142.27	
1173	135.28	135.17	132.29	140.87	135.46	142.98	
1203	135.19	135.2	132.22	140.8	135.06	142.84	
1226	135.2	135.13	132.21	140.95	134.92	142.79	
1250.5	135.17	135.16	132.19	140.58	134.79	142.72	138.79
1274.5	135.16	135.09	132.16	140.68	134.69	140.11	
1293	135.15	135.08	132.15	140.52	134.61	142.62	
1323	135.14	135.09	132.15	140.48	134.63	142.56	
1347	135.14	135.1	132.14	140.38	134.56	142.5	
1563	135.17	135.17	132.22	140.97	135.17	142.57	
1872	134.72	134.68	131.71	140.43	134.12	142.49	138.17
2640	134.52	134.54	131.54	138.79	133.5	141.08	136
3504	134.05	134.15	131.03	137.06	133.48	140.57	134.66
4032	134.15	134.19	131.19	137.17	133.73	140.75	134.79
4728	135.02	135.14	132.04	138.88	134.44	142.05	136.59
5808	135.06	135.18	132.09	137.78	134.51	142.06	136.89

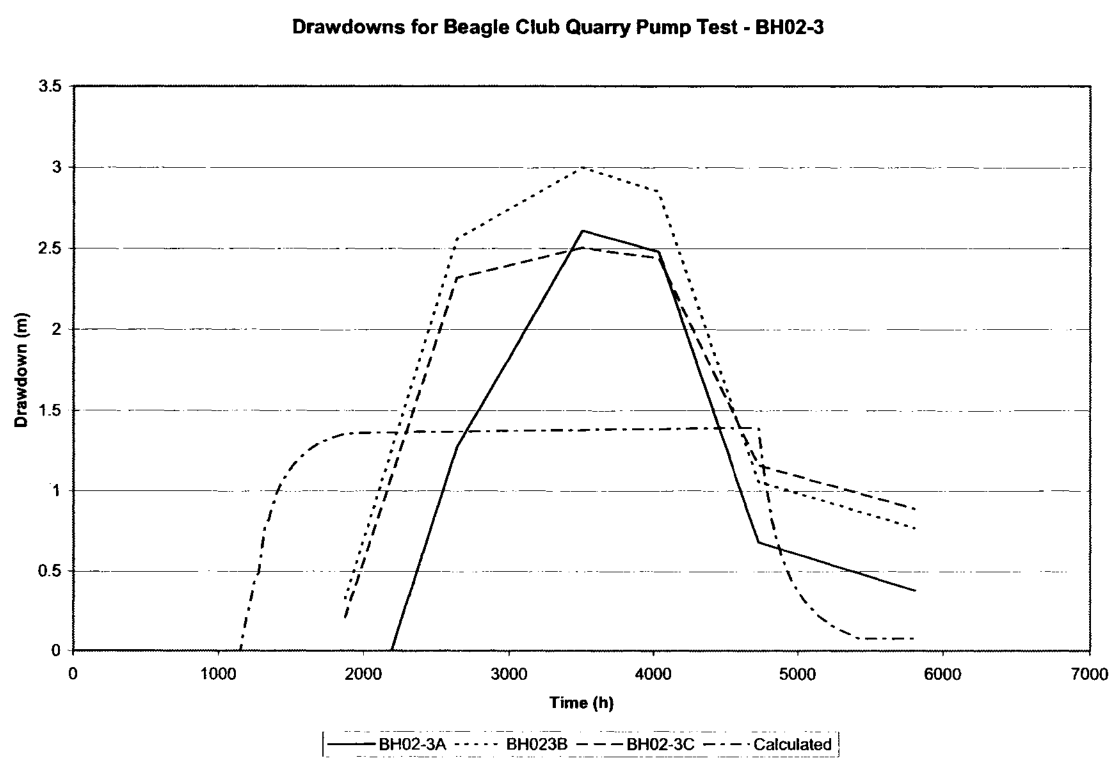
Time (hrs)	BH023B	BH02-3C	BH02-4A	BH02-4B	BH02-4C	BHWW-1A	BHWW-1B	Quarry
0	137.65	142.19	132.17	135.11	135.22	135.83	135.93	
552	137.65	142.19	132.17	135.11	135.22	135.83	135.93	
1128.5							135.49	134.66
1130.5							135.49	134.54
1148							135.49	132.99
1173						135.35	135.45	131.64
1203							135.3	130.82
1226							132.44	130.14
1250.5	137.6	142.26	132.14	135.25	135.25	135.19	135.28	129.19
1274.5						135.16		129.19
1293							135.17	127.66
1323							135.15	127.04
1347							135.15	127.04
1563							135.28	127.04
1872	137.33	141.985	131.59	134.74	134.73	134.87	134.97	
2640	135.09	139.87	130.9	134.06	134.11	134.525	134.59	
3504	134.65	139.685	130.42	133.68	133.75	134.345	134.41	
4032	134.8	139.75	130.6	133.81	133.89	134.16	134.22	
4728	136.59	141.03	132.22	134.76	134.83	134.87	135.02	
5808	136.88	141.3	131.82	134.75	134.88	134.97	135.11	

Drawdowns for Beagle Club Quarry Pump Test - BH02-1

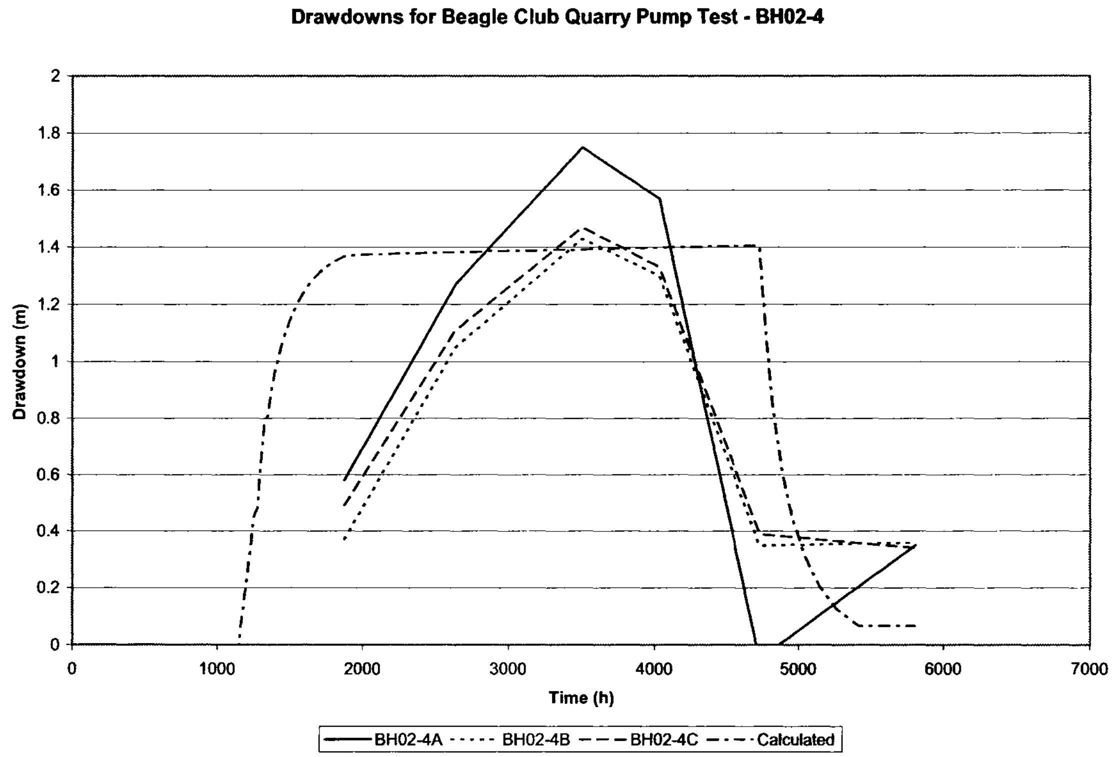


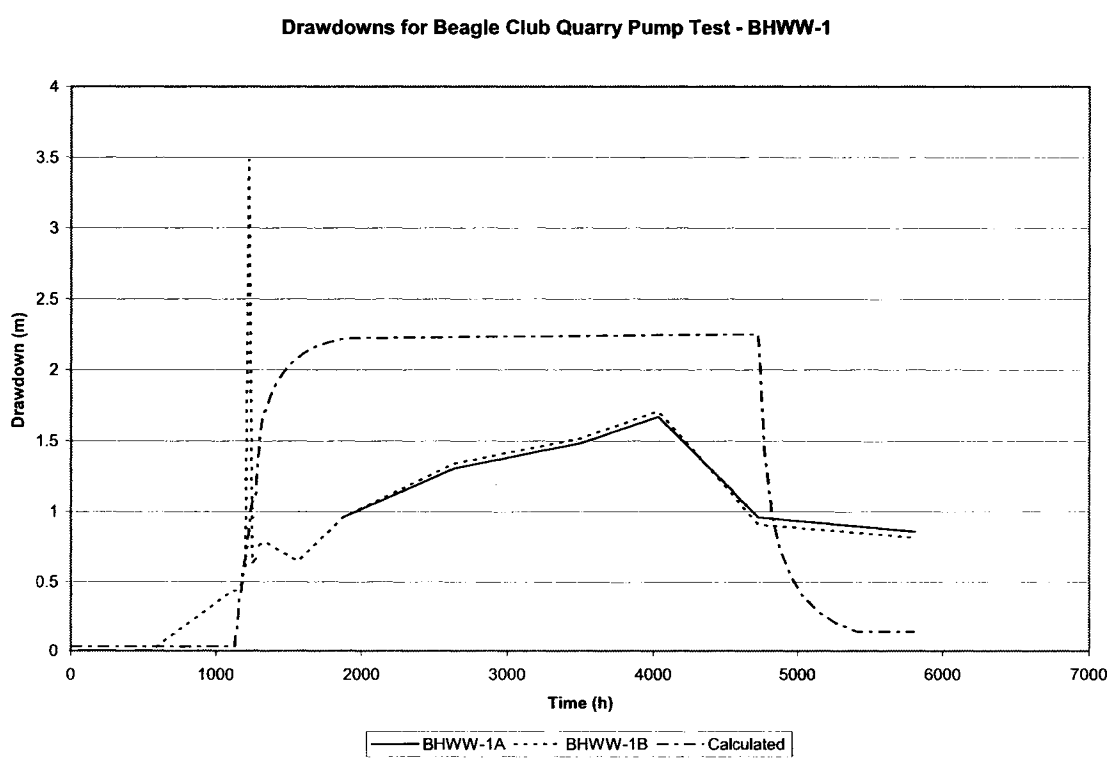


Drawdowns for Beagle Club Quarry Pump Test - BH02-2
Drawdown (m)
Time (h)
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
0
1000
2000
3000
4000
5000
6000
7000
BH02-2A
BH02-2B
BH02-2C
Calculated



Drawdowns for Beagle Club Quarry Pump Test - BH02-3
Drawdown (m)
Time (h)
0
0.5
1
1.5
2
2.5
3
3.5
0
1000
2000
3000
4000
5000
6000
7000
BH02-3A
BH023B
BH02-3C
Calculated


Drawdowns for Beagle Club Quarry Pump Test - BH02-4
Drawdown (m)
Time (h)
0
1000
2000
3000
4000
5000
6000
7000
0.2
0.4
0.6
0.8
1
1.2
1.4
1.6
1.8
2
BH02-4A
BH02-4B
BH02-4C
Calculated


Drawdowns for Beagle Club Quarry Pump Test - BHWW-1
Drawdown (m)
Time (h)
0
1000
2000
3000
4000
5000
6000
7000
0.5
1
1.5
2
2.5
3
3.5
4
BHWW-1A
BHWW-1B
Calculated

Appendix J: Surface Water Calibration Results

Mississippi River

Appleton to Galetta

Distance:	35	km
Flow at Appleton:	31.20	m^3/s
Flow at Galetta:	37.10	m^3/s
Difference in flows:	5.90	m^3/s
Flow per kilometer:	0.17	m^3/s km
Flow from one side only:	0.08	m^3/s km
Length of river in model:	13.30	km
Output of model:	15046	m^3/day
	0.17	m^3/s
Flow per kilometer:	0.01	m^3/s km

Carp River

Source to Kinburn

Distance:	32.50	km
Flow at Kinburn	2.85	m^3/s
Flow per kilometer:	0.09	m^3/s km
Flow from one side only:	0.04	m^3/s km
Length of river in model:	14.00	km
Output of model:	15348	m^3/day
	0.18	m^3/s
Flow per kilometer:	0.01	m^3/s km

Jock River

Source to Richmond

Distance:	57	km
Flow at Richmond:	6.35	m^3/s
Flow per kilometer:	0.11	m^3/s km
Flow from one side only:	0.06	m^3/s km
Length of river in model:	39.60	km
Output of model:	161580	m^3/day
	1.87	m^3/s
Flow per kilometer:	0.05	m^3/s km

Flow from Wetlands (West to East)

Wetland 1	4654.70	m^3/day
Wetland 2	1125.10	m^3/day
Wetland 3	1036.50	m^3/day

Appendix K: Results from Regional Transient Simulations

Transient 1

Input:

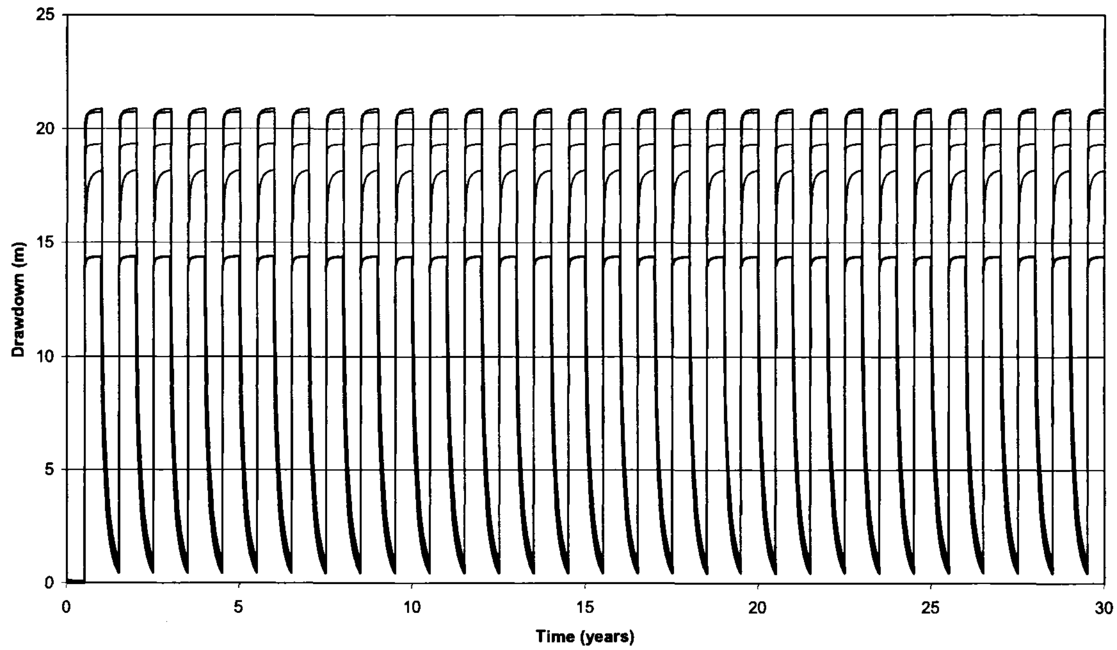
Time step is years.
Quarries running for 6 months of year, full depth.
All come on at the same time, all go off at the same time
Conductance is 10,000 m^2/day = 3,650,000m^2/year

Time Start	Stittsville Elevation	Henderson Elevation	Bell Elevation	Beagle Club Elevation	Goulbourn Elevation
0	500	500	500	500	500
0.5	110	112	131.5	107	110
1	500	500	500	500	500
1.5	110	112	131.5	107	110
2	500	500	500	500	500
2.5	110	112	131.5	107	110
3	500	500	500	500	500
3.5	110	112	131.5	107	110
4	500	500	500	500	500
4.5	110	112	131.5	107	110
5	500	500	500	500	500
5.5	110	112	131.5	107	110
6	500	500	500	500	500
6.5	110	112	131.5	107	110
7	500	500	500	500	500
7.5	110	112	131.5	107	110
8	500	500	500	500	500
8.5	110	112	131.5	107	110
9	500	500	500	500	500
9.5	110	112	131.5	107	110
10	500	500	500	500	500
10.5	110	112	131.5	107	110
11	500	500	500	500	500
11.5	110	112	131.5	107	110
12	500	500	500	500	500
12.5	110	112	131.5	107	110
13	500	500	500	500	500
13.5	110	112	131.5	107	110
14	500	500	500	500	500
14.5	110	112	131.5	107	110
15	500	500	500	500	500
15.5	110	112	131.5	107	110

Time Start	Stittsville Elevation	Henderson Elevation	Bell Elevation	Beagle Club Elevation	Goulbourn Elevation
16	500	500	500	500	500
16.5	110	112	131.5	107	110
17	500	500	500	500	500
17.5	110	112	131.5	107	110
18	500	500	500	500	500
18.5	110	112	131.5	107	110
19	500	500	500	500	500
19.5	110	112	131.5	107	110
20	500	500	500	500	500
20.5	110	112	131.5	107	110
21	500	500	500	500	500
21.5	110	112	131.5	107	110
22	500	500	500	500	500
22.5	110	112	131.5	107	110
23	500	500	500	500	500
23.5	110	112	131.5	107	110
24	500	500	500	500	500
24.5	110	112	131.5	107	110
25	500	500	500	500	500
25.5	110	112	131.5	107	110
26	500	500	500	500	500
26.5	110	112	131.5	107	110
27	500	500	500	500	500
27.5	110	112	131.5	107	110
28	500	500	500	500	500
28.5	110	112	131.5	107	110
29	500	500	500	500	500
29.5	110	112	131.5	107	110

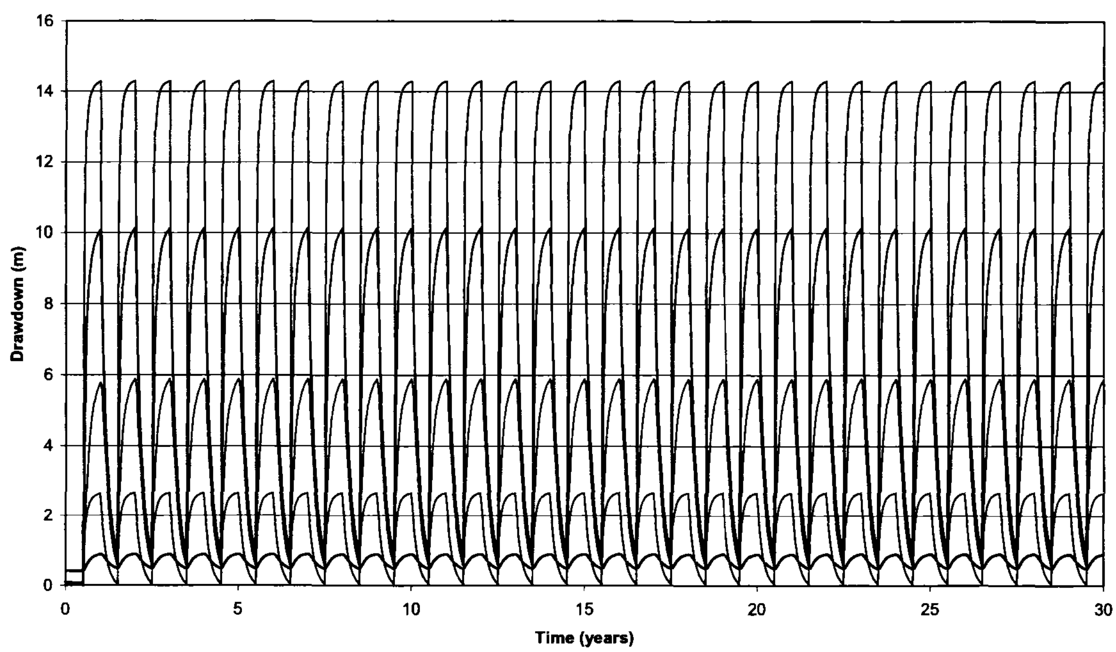
Output:

Regional Transient Simulation 1: Drawdowns

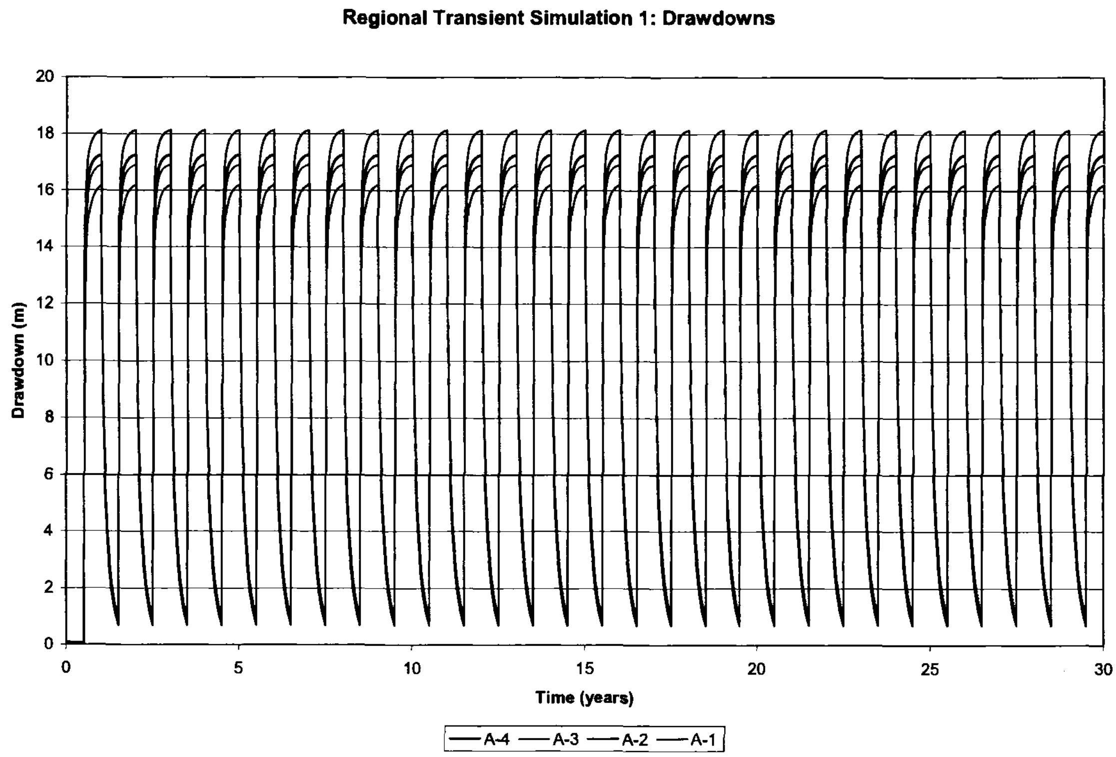


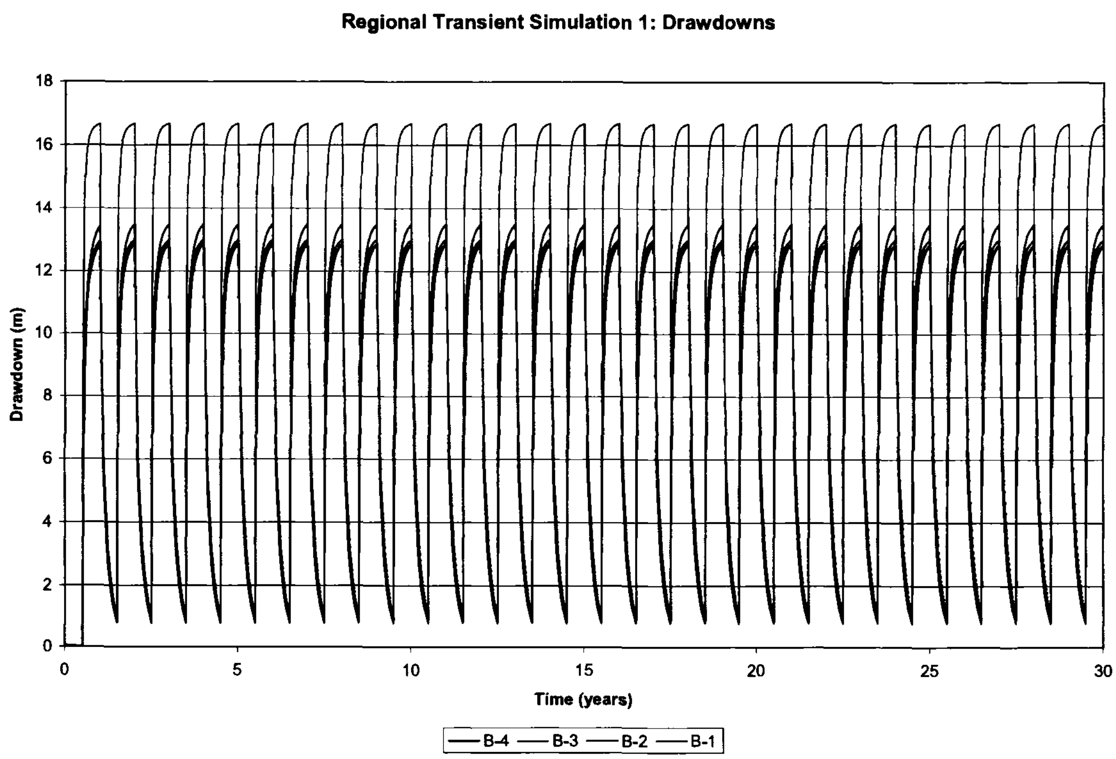
STITTSVILLE QUARRY — HENDERSON — GOULBOURN — BELL — BEAGLE CLUB

Regional Transient Simulation 1: Drawdowns

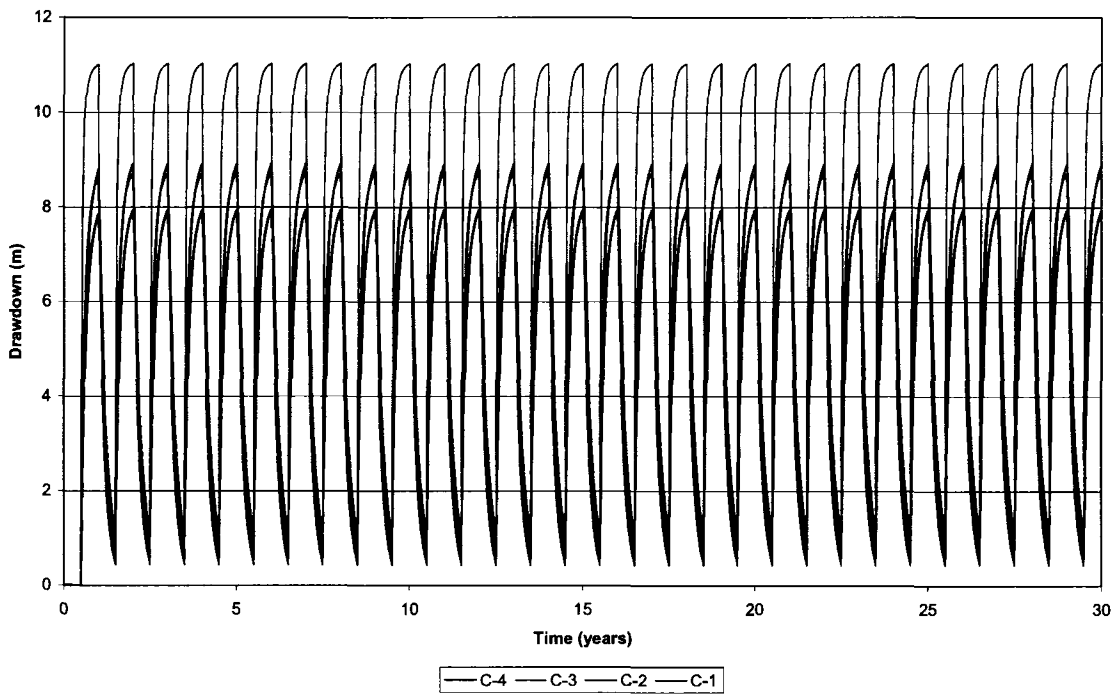


TRANQUILITY ESTATES — STITTSVILLE — RICHMOND — MUNSTER — LUCAS LANE

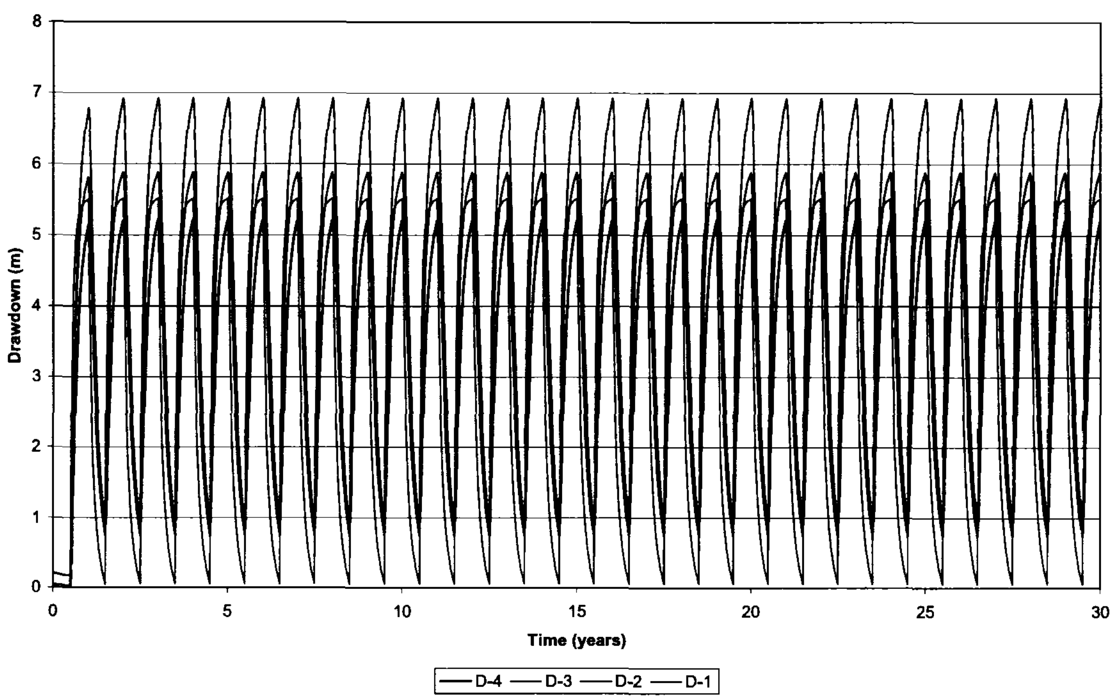

Regional Transient Simulation 1: Drawdowns
Drawdown (m)
20
18
16
14
12
10
8
6
4
2
0
0
5
10
15
20
25
30
Time (years)
A-4
A-3
A-2
A-1


Regional Transient Simulation 1: Drawdowns
Drawdown (m)
18
16
14
12
10
8
6
4
2
0
0
5
10
15
20
25
30
Time (years)
B-4
B-3
B-2
B-1

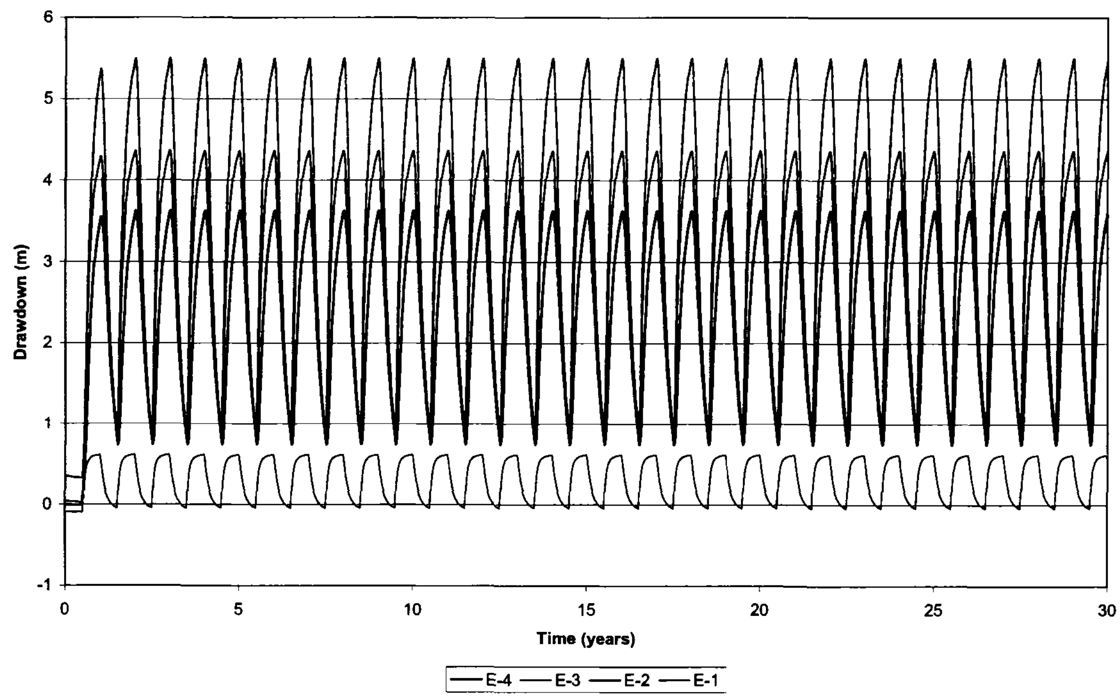
Regional Transient Simulation 1: Drawdowns



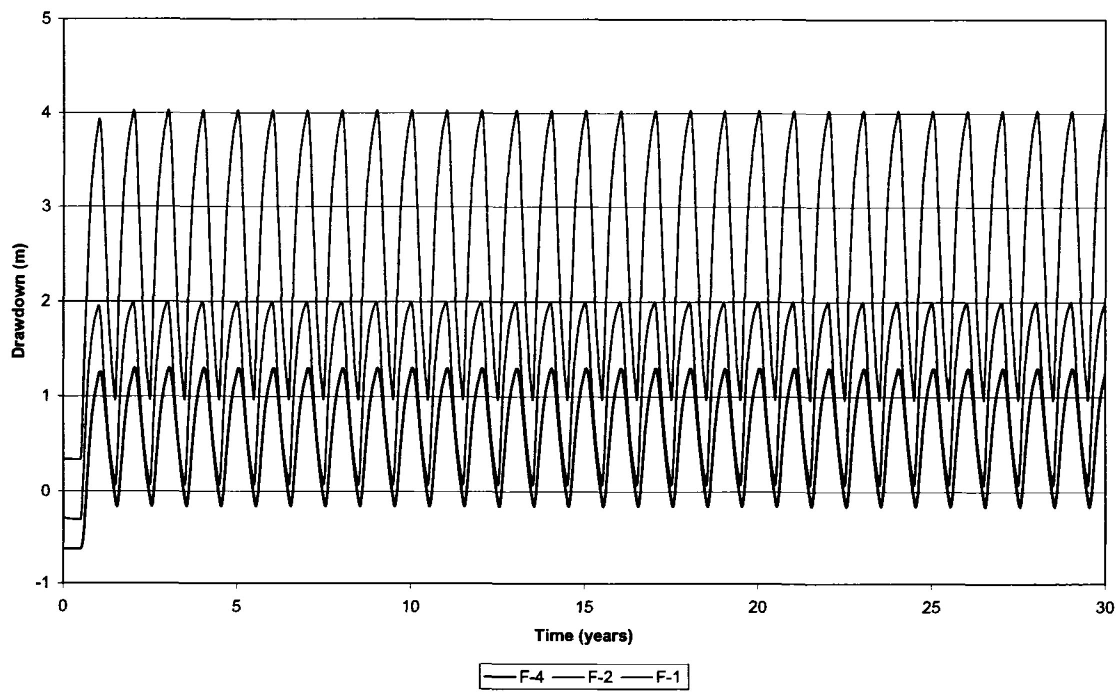
Regional Transient Simulation 1: Drawdowns



Regional Transient Simulation 1: Drawdowns



Regional Transient Simulation 1: Drawdowns



Transient 2

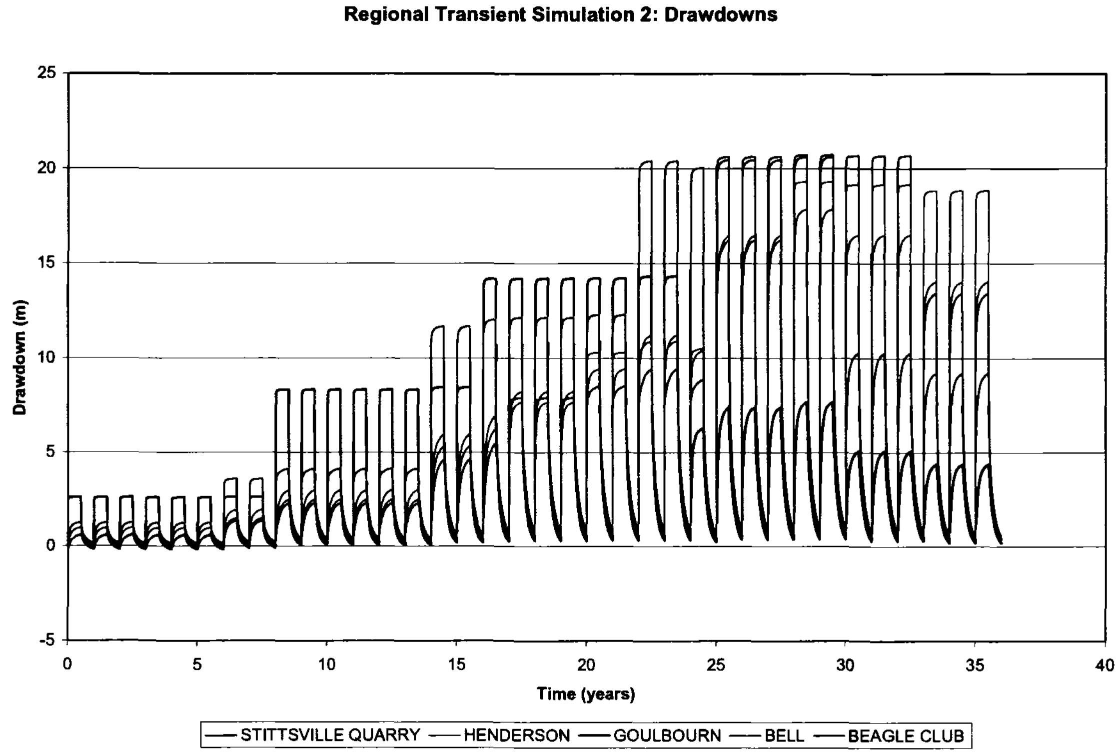
Input:

Time step is years.
Quarries running for 6 months of year, changing depth
3 lifts per quarry
Each lift takes 8 years to finish
Quarries only run for 6 months
Quarries stagger their starts by 3 years
Conductance is 10,000 m^2/day = 3,650,000m^2/year

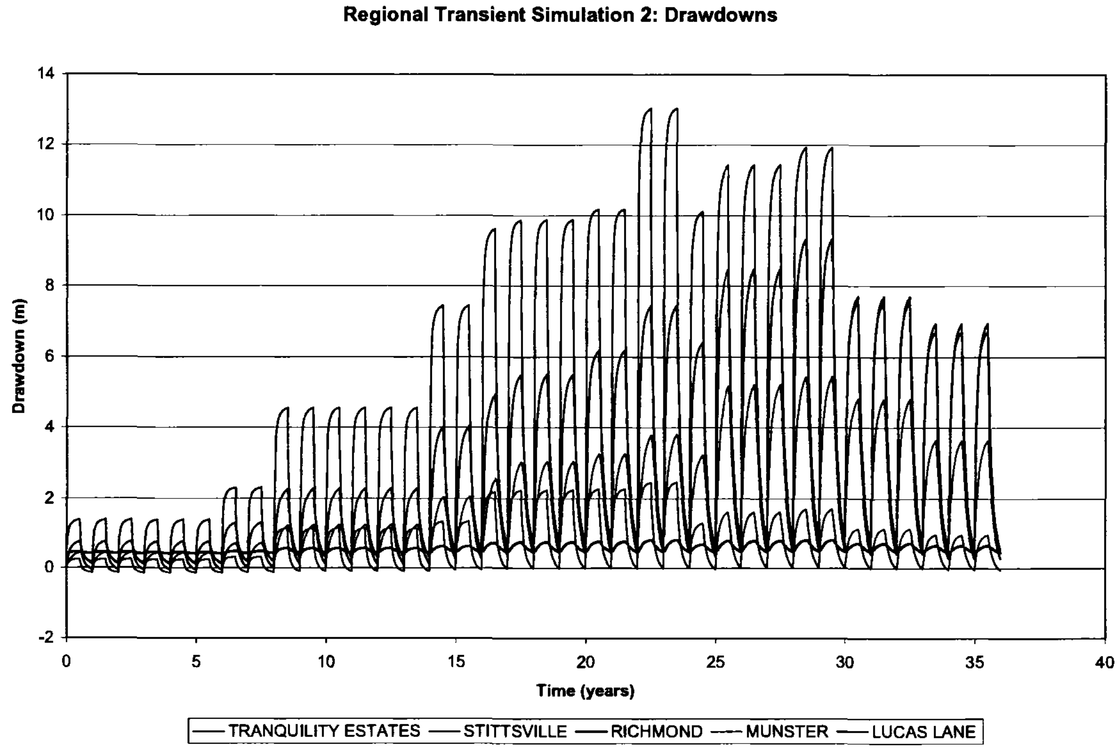
Time Start	Stittsville Elevation	Henderson Elevation	Bell Elevation	Beagle Club Elevation	Goulbourn Elevation
0	500	500	500	500	122
0.5	500	500	500	500	500
1	500	500	500	500	122
1.5	500	500	500	500	500
2	500	500	500	500	122
2.5	500	500	500	500	500
3	500	500	144	500	122
3.5	500	500	500	500	500
4	500	500	144	500	122
4.5	500	500	500	500	500
5	500	500	144	500	122
5.5	500	500	500	500	500
6	500	500	144	125	122
6.5	500	500	500	500	500
7	500	500	144	125	122
7.5	500	500	500	500	500
8	500	500	144	125	116
8.5	500	500	500	500	500
9	136	500	144	125	116
9.5	500	500	500	500	500
10	136	500	144	125	116
10.5	500	500	500	500	500
11	136	500	138	125	116
11.5	500	500	500	500	500
12	136	131	138	125	116
12.5	500	500	500	500	500
13	136	131	138	125	116
13.5	500	500	500	500	500
14	136	131	138	116	116
14.5	500	500	500	500	500
15	136	131	138	116	116

Time Start	Stittsville Elevation	Henderson Elevation	Bell Elevation	Beagle Club Elevation	Goulbourn Elevation
15.5	500	500	500	500	500
16	136	131	138	116	110
16.5	500	500	500	500	500
17	123	131	138	116	110
17.5	500	500	500	500	500
18	123	131	138	116	110
18.5	500	500	500	500	500
19	123	131	131.5	116	110
19.5	500	500	500	500	500
20	123	121	131.5	116	110
20.5	500	500	500	500	500
21	123	121	131.5	116	110
21.5	500	500	500	500	500
22	123	121	131.5	107	110
22.5	500	500	500	500	500
23	123	121	131.5	107	110
23.5	500	500	500	500	500
24	123	121	131.5	107	500
24.5	500	500	500	500	500
25	110	121	131.5	107	500
25.5	500	500	500	500	500
26	110	121	131.5	107	500
26.5	500	500	500	500	500
27	110	121	500	107	500
27.5	500	500	500	500	500
28	110	112	500	107	500
28.5	500	500	500	500	500
29	110	112	500	107	500
29.5	500	500	500	500	500
30	110	112	500	500	500
30.5	500	500	500	500	500
31	110	112	500	500	500
31.5	500	500	500	500	500
32	110	112	500	500	500
32.5	500	500	500	500	500
33	500	112	500	500	500
33.5	500	500	500	500	500
34	500	112	500	500	500
34.5	500	500	500	500	500
35	500	112	500	500	500
35.5	500	500	500	500	500
36					

Output

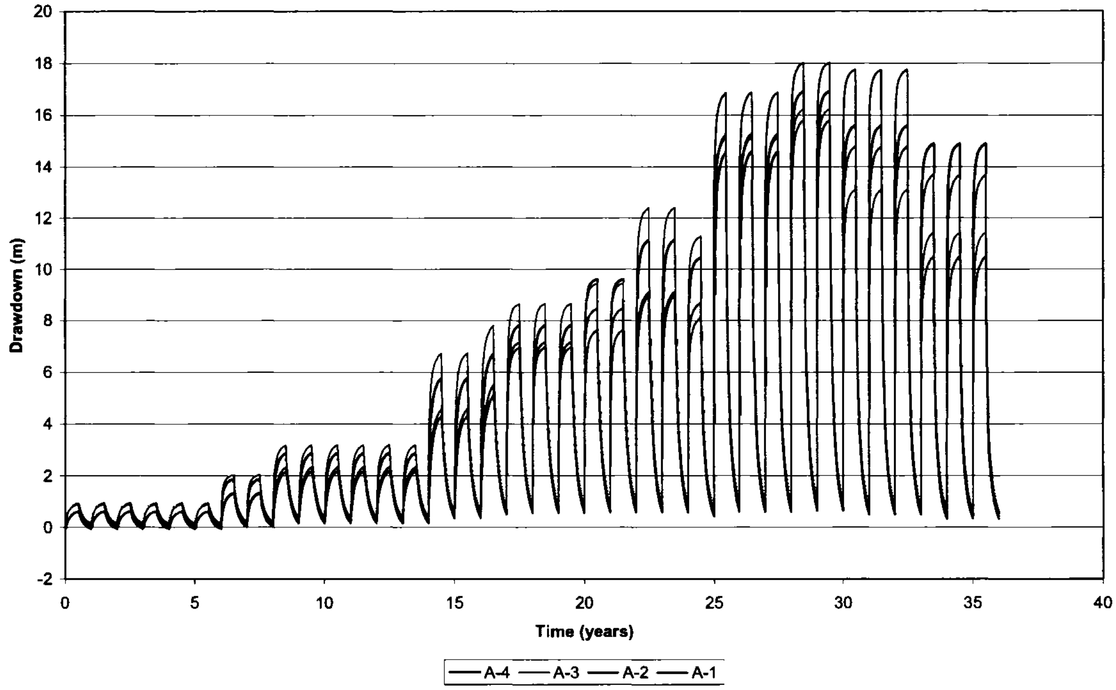


Regional Transient Simulation 2: Drawdowns
Drawdown (m)
25
20
15
10
5
0
-5
0
5
10
15
20
25
30
35
40
Time (years)
STITTSVILLE QUARRY
HENDERSON
GOULBOURN
BELL
BEAGLE CLUB

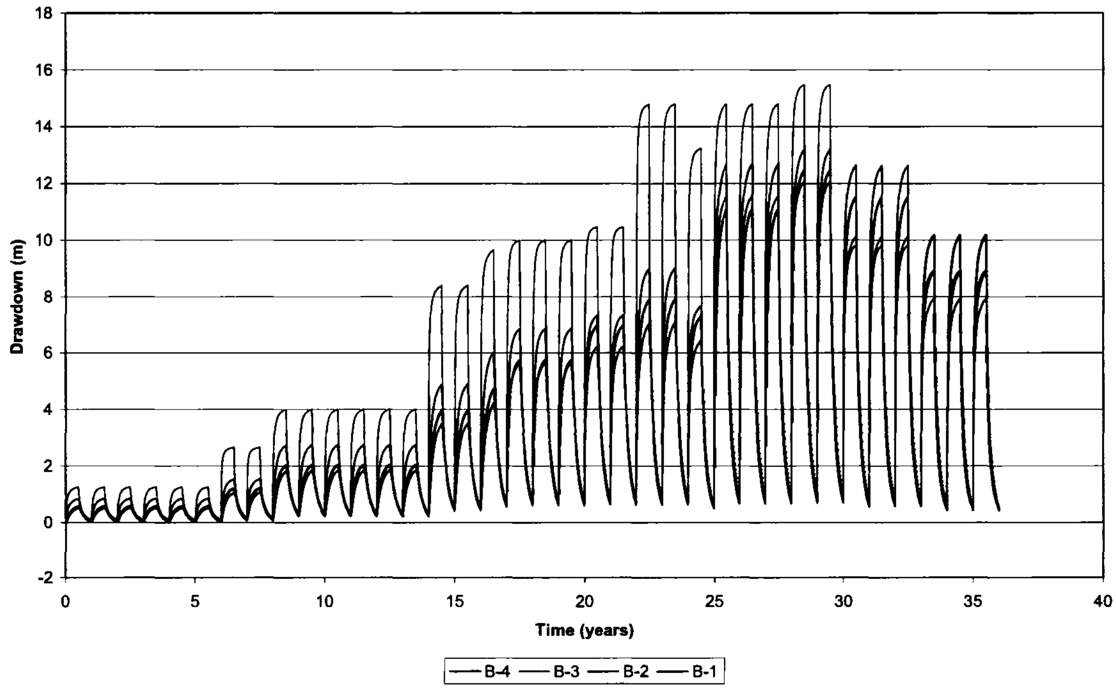


Regional Transient Simulation 2: Drawdowns
Drawdown (m)
14
12
10
8
6
4
2
0
-2
0
5
10
15
20
25
30
35
40
Time (years)
TRANQUILITY ESTATES
STITTSVILLE
RICHMOND
MUNSTER
LUCAS LANE

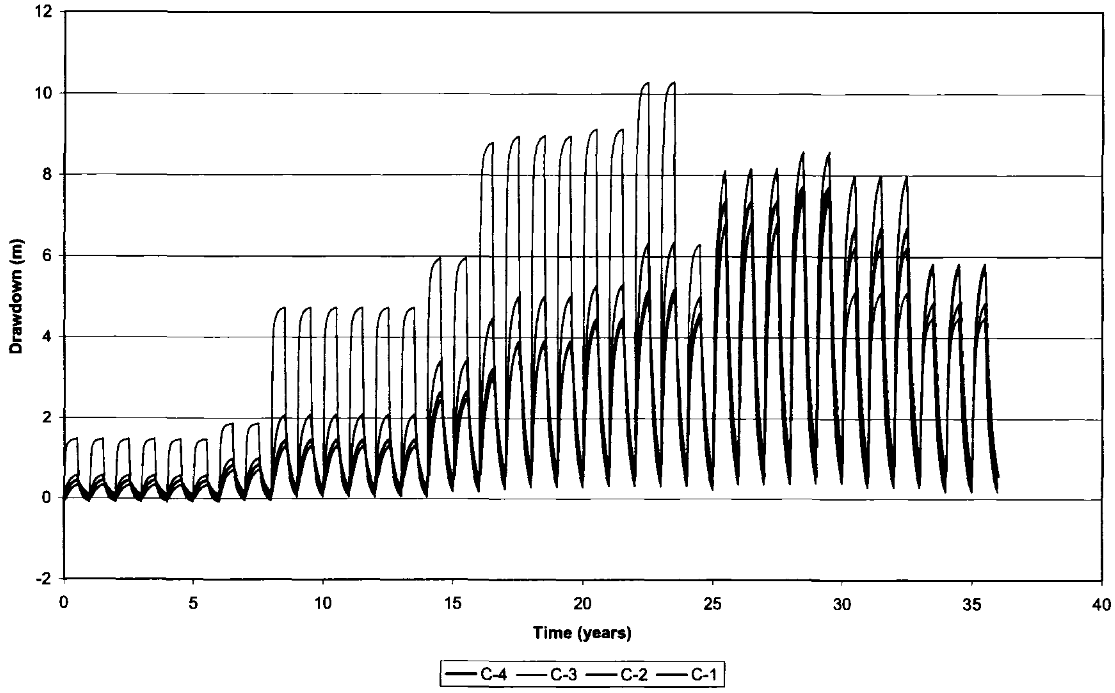
Regional Transient Simulation 2: Drawdowns



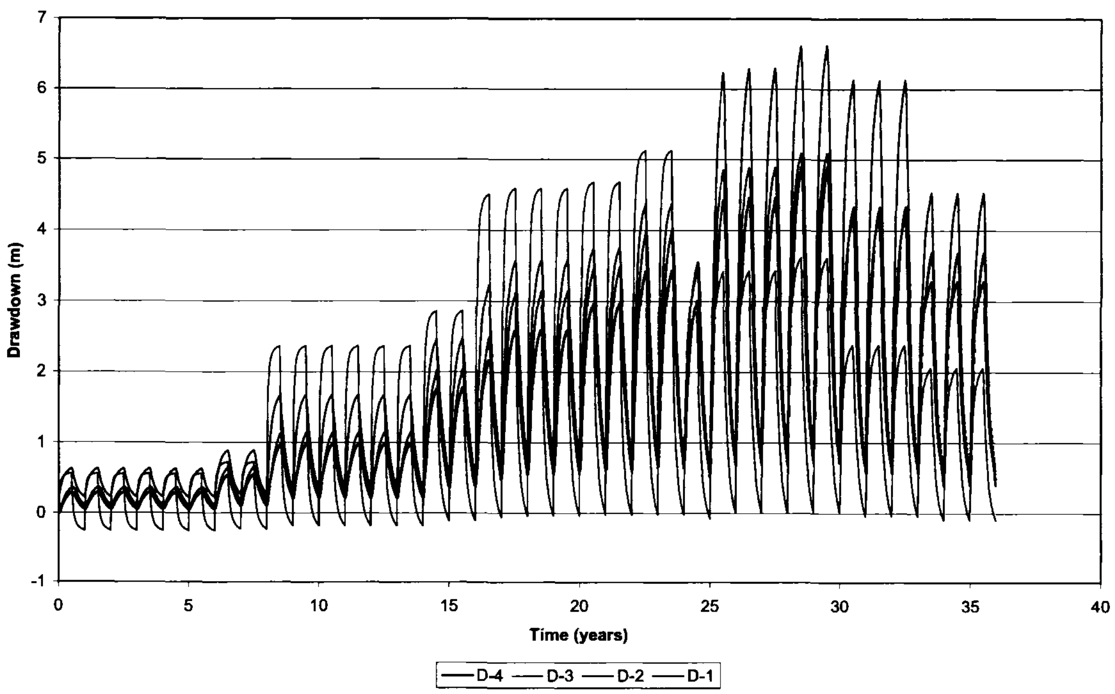
Regional Transient Simulation 2: Drawdowns

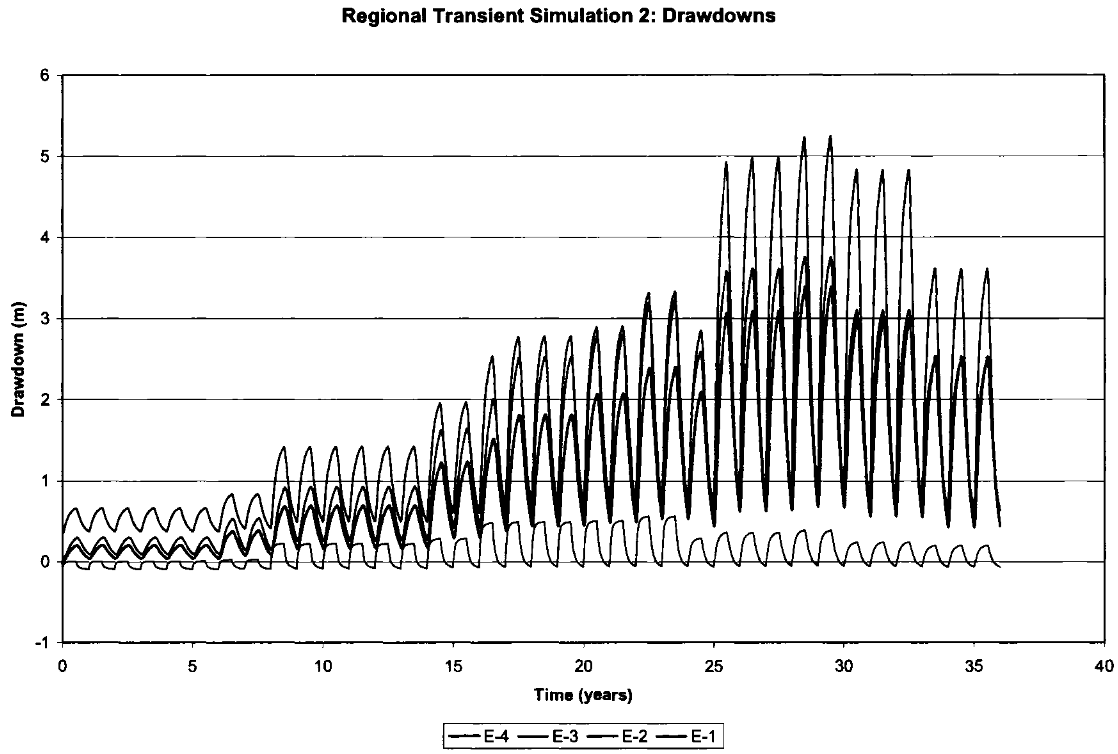


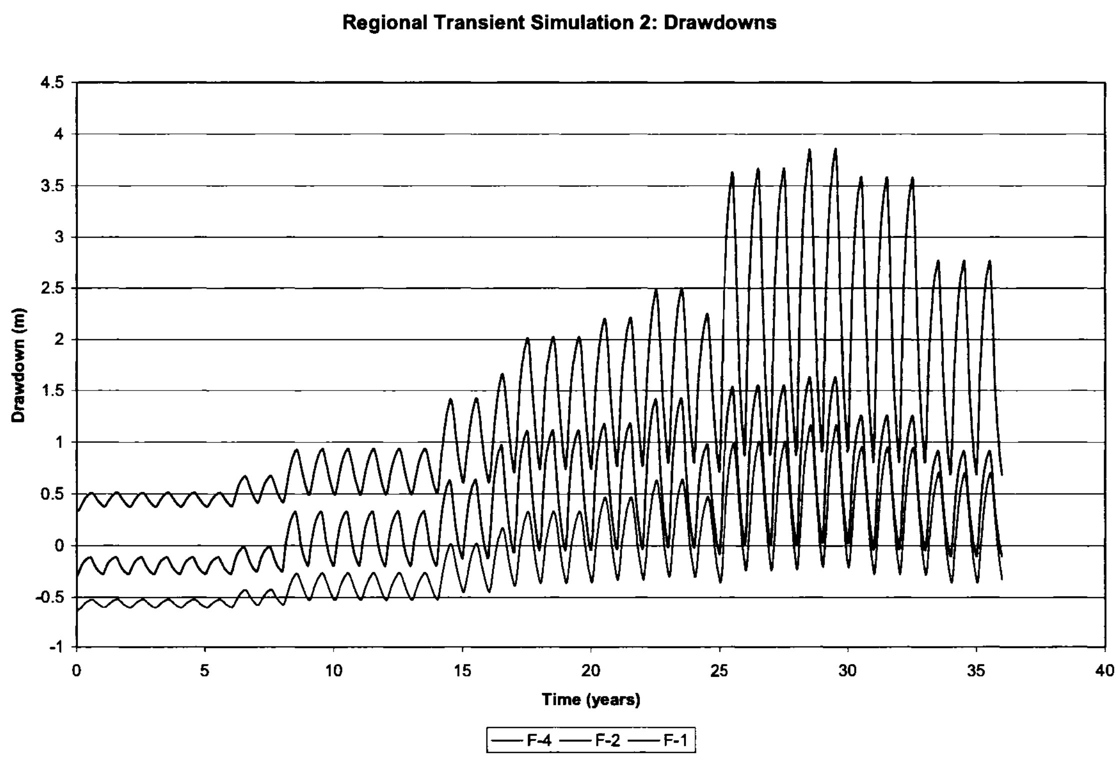
Regional Transient Simulation 2: Drawdowns



Regional Transient Simulation 2: Drawdowns




Regional Transient Simulation 2: Drawdowns
Drawdown (m)
Time (years)
E-4
E-3
E-2
E-1


Regional Transient Simulation 2: Drawdowns
Drawdown (m)
Time (years)
F-4
F-2
F-1