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**STEADY-STATE AND TRANSIENT
HYDROLOGIC, THERMAL AND CHEMICAL MODELLING
OF A FAULTED CARBONATE AQUIFER
USED FOR AQUIFER THERMAL ENERGY STORAGE,
CARLETON UNIVERSITY, OTTAWA, CANADA**

by

DIANA M. ALLEN, Hon B.Sc., M.Sc.

A thesis submitted to
the Faculty of Graduate Studies and Research
in partial fulfilment of
the requirements for the degree of

Doctor of Philosophy

Ottawa-Carleton Geoscience Centre

and

Department of Earth Sciences

Carleton University

Ottawa, Ontario

June, 1996

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Chair, Department of Earth Sciences

Thesis Supervisor

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Carleton University

July, 1996

ABSTRACT

The complex hydraulic behaviour observed within faulted carbonate aquifers, which results from the preferred orientation of fractures and the development of secondary permeability, has, in general, only been described in qualitative terms. Rarely have attempts been made to model groundwater flow in these complex systems. The purpose of this study was to examine the static and dynamic hydraulic, thermal and chemical characteristics of a carbonate aquifer, which is used for Aquifer Thermal Energy Storage (ATES), at Carleton University in Ottawa, and to produce a mathematical model that would represent the hydraulic and thermal regimes of the aquifer.

A detailed examination of bedrock topography and stratigraphy, in combination with borehole and surface geophysics, was undertaken to define the aquifers and delineate the major faults. The application of microgravity profiling improved the success rate for drilling high yielding wells during the second phase of the groundwater project.

The static hydraulic head distribution, thermal depth profiles and aquifer chemistry are inter-related and are controlled, to a large extent, by faulting. Results from two extensive aquifer testing programs were analyzed. These results suggest that the groundwater regime beneath the Carleton University campus is dominated by linear fracture flow associated with major faults.

Computer codes were written to solve the steady-state and transient groundwater flow and heat transport equations, using the finite element method. The steady-state model could be calibrated under static conditions; however, transient hydraulic test data were

ultimately used to calibrate the model, and to simulate well responses measured during constant discharge and reinjection tests.

The model consists of a series of highly transmissive, homogeneous and isotropic fault zones that transect two homogeneous and isotropic aquifers. A low permeability barrier, associated with the faults, separates the two aquifers. The fault zones are modelled as equivalent porous media. Uncertainties in the boundary conditions limit the accuracy of the model.

Thermal simulations indicated that heat moves radially away from reinjection wells, except when these wells are located very close to major faults. Both analytical calculations and numerical simulations indicate that thermal breakthrough between pumping and reinjection wells in the ATEs system at Carleton should not occur within a heating or cooling season.

ORIGINAL RESEARCH CONTRIBUTIONS

This research project has attempted to integrate a variety of geological and hydrogeological data sets for Carleton University that were acquired during the development of an aquifer thermal energy storage (ATES) system for the campus. As part of this investigation, the following original contributions have been made to hydrogeological- and ATES-oriented research:

- 1) The use of hydraulic data from an extensive aquifer testing program to characterize and quantify groundwater flow in a faulted carbonate aquifer.
- 2) The development and testing of a conceptual model, using transient hydraulic test data for calibration, that simulates well interactions in a complex hydrogeologic system.
- 3) The estimation of thermal front advancement rates in a multi-well ATES system with interspersed pumping and reinjection wells.
- 4) The use of small-scale microgravity profiling for delineating major faults which coincide with buried bedrock valleys.

ACKNOWLEDGEMENTS

I would like to begin by acknowledging the effort of so many people, who had a dream for Carleton University. This dream was to implement one of the largest aquifer thermal energy storage applications in the world and to create a showpiece for innovative environmental technology that would be the envy of large institutions around the world. These people include Gil Belair, who has since retired as Special Projects Manager at Carleton University; Brian Beazer, the former head of Physical Plant at Carleton University; Nick Haitas of N. Haitas and Associates Ltd, the design engineer for the Commons Building retrofit and Phase 2 expansion of the system; Dr. David Brown, the former President of Carleton University Development Corporation; and finally, Dr. Fred Michel, from the Department of Earth Sciences at Carleton University.

I also want to express my deep appreciation to my supervisor, friend and colleague, Dr. Fred Michel, who has supported my research and has had confidence in my work from the outset. His energy and enthusiasm have made the difficult times easier. I also greatly appreciate the opportunity I was given to work with Carleton Energy Services Inc. (CESI) in the capacity of a groundwater modeller and site hydrogeologist during my research.

I would like to thank the two other members of my thesis committee, Dr. John Blenkinsop (Carleton University), who has offered continued support throughout my research program, and Dr. Michel Robin (University of Ottawa), whose excellent teaching skills made numerical methods both fun and easy to learn. I would also like to thank Ildi Munro for her editorial assistance.

I would like to acknowledge the assistance of Anthony Lambert, Jacques Liard and Carey Gagnon (Geological Survey of Canada), whose preliminary suggestions for carrying out the gravity profiling were invaluable. They also provided training and offered the use of the gravity meter for this research. As well, I thank Warner Miles (Geological Survey of Canada) for providing me with geophysical maps, Bill Hyatt and Johnathan Mwenifumbo of the Borehole Geophysics Section (Geological Survey of Canada) who carried out the geophysical logging of both the Phase 1 and Phase 2 wells, and Heather Wilson who did a superb job in collecting invaluable data prior to the implementation of Phase 1 of the Carleton Groundwater Project. This research would not have been possible without these steady-state data. I also thank Angela Gulley, the site geologist during Phase 2 drilling and testing, for her contributions to collecting data.

I would also like to thank Vic Allen (Geological Survey of Canada) for helping me with the temperature logging and with the installation of the temperature cable and data logger and Anne Wilkinson for her tutorial in making thermal conductivity cells for the divided bar.

I would like to express my appreciation to Mark Hinterman for assisting with the leveling for the gravity profiling, to Brian Moorman (Carleton University) for his assistance in interpreting the ground-penetrating radar profiles for a conference presentation and to Tom Feininger (Geological Survey of Canada) for his technical and editorial revisions of the gravity paper.

This research was funded, in part, by a grant from the Natural Sciences and Engineering Research Council of Canada (NSERC) and financial support was also

provided to me throughout my research program by NSERC and the Faculty of Graduate Studies and Research at Carleton University. The Lacoste and Romberg model D gravity meter, the software for tidal corrections and the use of the thermal conductivity laboratory were offered by the Geological Survey of Canada (GSC). Software for gravity modelling was supplied by the Department of Geophysics at McGill University and the steady-state finite difference groundwater modelling program, FLOWPATH, was supplied by Waterloo Hydrogeologic Software, Inc.

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1. INTRODUCTION

1.1 Carbonate Aquifers

1.1.1 Hydrogeology of Carbonate Aquifers

Carbonate aquifers, consisting predominantly of limestones and dolostones, have long been described as displaying “unusual behaviour”. Rovey and Cherkauer (1994) described groundwater flow in carbonates as being “radically different than other types of aquifers”. To date, much of the research conducted has been of a qualitative nature, and rarely have attempts been made to model carbonate aquifer systems. Most quantitative analyses consist of a comparison of aquifer test data to various analytically-based models.

In general, carbonate rocks exhibit a wide variety of hydraulic responses which depend upon the degree to which the pores and fractures are interconnected. By definition, a fractured rock consists of solid rock, with some **primary porosity**, cut by a system of cracks, microcracks, joints, fracture zones and shear zones that create **secondary porosity**. As a result, the **secondary permeability** can increase the **effective hydraulic conductivity** of a fractured system by up to five orders of magnitude (Anderson and Woessner, 1992).

The term “carbonate aquifer” is often prefixed with “fractured”, and in some cases “carbonate” or “limestone” aquifer is used synonymously with “karst”. Often, carbonate aquifers are vuggy and fractured, and these vugs and fractures may be sealed by secondary minerals such as calcite, as a result of carbonate precipitation, or they may be open or enlarged, as a result of carbonate dissolution. Because the rates of precipitation or

dissolution are temperature-dependent, the thermal regime exercises an important control on the hydrologic regime.

When exposed to long term erosion by naturally flowing groundwater within the carbonate units, enlargement of these cavities may occur. Variations in the rate of dissolution and the period over which dissolution takes place ultimately determine the hydrologic character of the carbonate aquifer. This variation results in secondary permeability that may be 1) developed on a small-scale (micrometre-wide fractures) and influences only the local hydrogeology; or 2) developed extensively, in a regional sense, particularly if regional faulting and folding is the source of fracturing; or 3) developed at the surface, such as in karstic terrane; or 4) developed at depth as conduits or large channels, which are identified during drilling and aquifer testing. The movement of groundwater, in each case, is enhanced and may take place in an unpredictable manner. For example, Rojstaczer (1987) stated that “the hydraulic connection between wells exhibits a complex spatial variability typical of carbonate terrane”.

1.1.2 Faults and Fractures in Carbonate Terrane

The ability of faults and fractures to serve as barriers or conduits for groundwater flow has been observed for decades. One example of this type of behaviour is the preferential connection between widely spaced wells. In Laramie, Wyoming, Huntoon and Lundy (1979) identified fractured high transmissivity zones, in an interbedded sandstone and limestone aquifer system, that are approximately two orders of magnitude greater than the unfractured zones. These high transmissivity zones correlate with regional structural

features, namely faults. Enhanced fracture permeability was observed several tens of feet on either side of the faults.

In the Mackenzie Valley, Michel (1986) found that spring discharges were preferentially aligned along regional fault structures, which served as conduits for groundwater discharge from carbonate and evaporite units. In the Ash Meadows area of Nevada, the complex hydrogeology is manifested both as zones or directions of minimal hydraulic connection as well as zones of enhanced hydraulic connection (Rojstaczer, 1987). The former were attributed to fault-controlled structural discontinuities, and the latter were attributed to preferential movement of groundwater through large-scale fractures or conduits.

1.1.3 Hydrogeologic Models for Carbonate Aquifers

Limestones and dolostones are present in geological environments, such as sedimentary basins, which are deep and laterally extensive. Regional-scale deformation subsequent to deposition and lithification often gives rise to structural features that can be traced for many kilometres. Therefore, fractures in a preferred orientation, for example, along a fault zone, will affect large-scale regional flow if these features are continuous and extensive. Horizontal bedding planes may provide additional secondary permeability; however, these planes must show lateral continuity in order to affect the large-scale flow regime. When there is a uniform distribution of fractures or solution cavities, the degree of interconnection between these spaces determines the spatial movement of water.

Many researchers quantify flow in carbonate aquifers using “fracture” or “conduit” flow models. Thrailkill (1974), for example, presented modelling results for a limestone

aquifer in Kentucky, in which the aquifer was represented and simulated as a network of pipes. Both laminar and turbulent flow were considered in this study. More commonly, **fracture flow** and **double porosity flow** models are applied to the data obtained from aquifer testing. Fracture flow models assume that most of the water movement occurs in discrete fractures, while double porosity flow models assume that water is transmitted via the fractures but is released from storage within the rock mass.

There is a general consensus that a continuum exists between the fractures and the rock matrix and that Darcian mathematics (laminar flow) can be applied when dissolution is minor and the fracturing is dense and interconnected enough to reach an equivalent porous medium scale. For this reason, some researchers choose to model these aquifers using the **EPA** (equivalent porous media) approach, which assumes that at the scale for the model, the fractures and the matrix behave as a unit and are indistinguishable from one another. These models can be adapted to incorporate any lateral or vertical heterogeneities.

From the point of view of contaminant or heat transport within a fractured carbonate unit, both the primary and the secondary permeability will contribute to the overall flow regime. Because the transport of heat or contaminants depends on the advective forces, the velocity of the water, which is determined by the product of the pressure gradient and the hydraulic conductivity, will be a major factor under normal flow conditions.

In crystalline rocks, the primary porosity and permeability are low, such that hydraulic conductivities, with water as the fluid, are in the order of 10^{-10} m/sec (Freeze and

Cherry, 1979). The ratio of the fracture hydraulic conductivity to the matrix hydraulic conductivity is so great that the secondary permeability is the dominant factor in determining the rate at which water flows. In a limestone, the “primary” hydraulic conductivity is in the order of 10^{-7} m/sec (Freeze and Cherry, 1979). In this case, there is a smaller ratio and, therefore, matrix flow is an important contributor to the overall groundwater movement.

The aquifer beneath the study area, namely, the Carleton University campus in Ottawa, Ontario (Figure 1.1), consists of a thick sequence of Paleozoic sediments. These are predominantly limestones and dolostones with some sandstones and shales. The aquifer is used by Carleton University to store energy for the purpose of heating and cooling some of the buildings on campus.

The geological nature of this aquifer is of prime importance in the characterization of groundwater flow and transport, and in the development of a conceptual model for use in any quantitative investigations. The overall hydrogeologic regime of the aquifer will ultimately depend on the nature of the permeability, and this must be assessed prior to the development of a conceptual model. This requires a sound knowledge of the geology.

1.2 Aquifer Thermal Energy Storage (ATES)

1.2.1 Principles of ATES Systems

Aquifer thermal energy storage (ATES) is a rapidly expanding technology that has been applied around the world for more than a decade. Early applications for cold storage in China date back to the mid-1960's. More recently, operating ATES systems have been developed in China, Canada, The United States, The Netherlands, Sweden and France.

Figure 1.1 : Map Showing Location of Carleton University in the Ottawa Valley

Advances in the technological aspects of ATEs have been considerable since the early experiments conducted in the United States of America during the late 1970's (Molz *et al.*, 1978). Research associated with the thermal and hydrologic aspects of various applications continues throughout the world, and this research spans a number of different disciplines, ranging from biochemistry to computer modelling of heat transport. International ATEs conferences are now held on a regular basis every three years.

In all ATEs applications, heat or cold energy is stored in the aquifer on a seasonal basis. In climates where seasonal heating and cooling are required, a simple system would consist of two wells. One well is a production well and supplies groundwater to a heat exchanger; the second is a reinjection well through which waste energy is returned to the aquifer for storage. The thermal stability of the aquifer is preserved and the hydraulic character of the flow system remains unaltered when water is continuously pumped from and reinjected into the aquifer. However, small local variations do occur in the vicinity of the wells.

The current design (Phase 1) of the Carleton University Groundwater Energy System applies ATEs technology to directly cool building air in the summer months and to preheat building air in the winter months without the use of heat pumps. The ambient temperature of the groundwater is approximately 9°C, and the climate in the Ottawa area is such that the load on the buildings can be balanced on an annual basis. A brief history of the Carleton University project is provided in Appendix 1.

Carleton University has undertaken steps towards the further expansion and redesign of the system. The start of this research coincided with the preliminary

investigations for the siting of new production wells for the Phase 2 expansion. Since that time, drilling of test and production wells and twin production wells for Phases 2 and 3 has been completed.

1.2.2 Environmental Considerations

Unlike some ATES applications, all of the groundwater used at Carleton is reinjected back into the aquifer. This was of primary concern to the hydrogeologists involved in the early design stages of the system because the aquifer from which the groundwater is taken is confined. Unlike Carleton, the University of Alabama discharges the heated waste water to the adjacent river rather than reinjecting it back into the aquifer (Midkiff and Brett, 1994). This type of application is only feasible in unconfined aquifers that have a high natural rate of recharge.

From an environmental point of view, the system at Carleton University is 'environmentally conscious'. There is a decrease in the demand for non-renewable energy; there is a reduction in CO₂ emissions compared to burning fossil fuels; the need for costly chillers with CFCs is eliminated in those buildings served by groundwater; and there is a reduction in the consumption of hydroelectric power. In all, the impact on the environment is controlled and minimal.

1.3 Previous Geological and Hydrogeological Investigations at Carleton

In 1985, a preliminary examination of the geology and subsurface hydrologic and thermal regime was undertaken and submitted as an unpublished report to Carleton University (Michel, 1985). Included in this report was a summary of the stratigraphy of

the Ottawa area, along with brief lithological descriptions of the overburden and bedrock formations in the region. A map of bedrock elevations beneath the Carleton campus and two overburden profiles were included. Michel (1985) identified a series of faults on the basis of bedrock topography and stratigraphy, and delineated several potential water-bearing intervals in the subsurface based on geophysical well logs, porosity estimates, and the presence of bedding plane fractures and open vugs in core. Temperature well logs (Figure 1.2) were also included as a means of identifying the optimum depth range for 9.0 to 10.0°C water.

Three unpublished reports were prepared by Water and Earth Science Associates Ltd. (WESA, 1985, 1986, 1988). These described the individual drilling and testing programs conducted from 1985 to 1988. Based on the results of the drilling and aquifer testing, and the results of chemical analyses, the Phase 1 production and observation wells (Figure 1.3) were divided into two groups: P2, RC, P3, P5, P6, Elba1 and Elba2 in the Eastern Block (Group I); and P1, M1, Tory and P4 in the Central Block (Group II). WESA (1986) determined that the Central and Eastern Blocks had poorly-developed hydraulic connection and that this was attributed to the hydraulic effects of a major fault.

This preliminary research formed the basis for further detailed hydrogeological investigations by Wilson (1990). The scope of this study included a chemical and isotopic analysis of the well waters, the interpretation of geophysical well logs to identify stratigraphic relationships and water-bearing units, and the monitoring of hydraulic heads prior to the implementation of Phase 1 of the Carleton University Groundwater Energy

Figure 1.2 : Temperature-Depth Profiles for Water Wells at Carleton University.
Observatory Crescent Borehole BH-0-8 Profile Shown for Comparison.

Figure 1.3: Location Map for Phase I Production Wells and Observation Wells

system. The wide range in aquifer transmissivity values (60 to 470 m²/day) calculated by WESA (1988) was attributed to the heterogeneity of the fracture network (Wilson, 1990).

1.4 Purpose of Study

During the test and production well drilling of Phase 1, it became evident that well yields were spatially variable, and that this variability was related to the well's proximity to a fault. However, the nature of this relationship was not known. Aquifer testing carried out in 1988 suggested that the excellent hydraulic connection between the P2 well and the Elba2 well, as exhibited by the instantaneous response during pumping (Figure 1.4), was attributed to a rapid pressure response along faults. The delayed response of certain wells (e.g., well P1 in Figure 1.4) was attributed to the presence of a series of fault blocks, each in poor hydraulic connection to the other. Wilson (1990) stated "It was found that certain faults acted as negative (impermeable) boundaries while others formed zones of preferential permeability".

A thorough understanding of the inter-relationship between wells is essential to predict how wells will respond to one another when some are pumping water and others are reinjecting water. Analytical models, which use the calculated hydraulic parameters derived from aquifer testing, namely, transmissivity and storativity, are highly reliable in simple aquifer systems. The complexity of the Carleton aquifer system does not, in theory, lend itself to such models which may, nevertheless, serve as estimates of the hydraulic behaviour. In addition, analytical models are available for determining heat transport in simple ATES systems, but these do not strictly apply in complex hydrogeologic regimes.

AQUIFER TEST DATA CONSTANT DISCHARGE TEST AT WELL P2

Figure 1.4 : Graph Showing Constant Discharge Test Data for Well P2 and for Observation Wells P1 and Elba2.

The Carleton University ATES site offers a unique opportunity to examine groundwater flow in a complex carbonate aquifer system. The purpose of this research was to investigate the hydrologic, thermal and chemical regimes of the Carleton University aquifer, both under steady-state and transient conditions, and to attempt to develop a physically-based hydrologic and thermal conceptual model that could simulate the observed complex well interactions. This conceptual model is represented mathematically, in order that groundwater flow and heat transport between multiple pumping and reinjection wells in the ATES system can be quantified.

1.5 Research Objectives

The objectives of this research are presented systematically to reflect the various stages involved in the development of a model for the physical system. Both the undisturbed and modified aquifer regime, under natural and operating conditions were evaluated. The natural aquifer regime has been modified by the operating ATES system since 1990, consequently, all investigations conducted prior to the implementation of the system were reviewed. Hydraulic head data (Wilson, 1990), obtained for the natural aquifer system, were used in hydrologic modelling, and early temperature logs (Michel, 1985; Wilson, 1990) were used to describe the thermal regime. In addition, the chemical analyses presented by Wilson (1990) were re-interpreted, and the aquifer test data for the RC, Elba1, Elba2 wells and the Phase 1 wells were, in some cases, re-analyzed as part of this work. Because the system was operating during the research program, there were practical restrictions on the form of research-related testing that could be carried out.

The objectives were:

1. To develop a conceptual model for the aquifer system at Carleton.
2. To formulate and solve a set of mathematical equations which can simulate the observed hydraulic interactions between wells on campus.
3. To assess the efficiency of the proposed ground water energy system by modelling transport between multiple pumping and reinjection wells.

1.6 Scope of Work

The scope of the study included a sequence of field, laboratory and computer modelling work that was carried out in order to meet the objectives outlined in the previous section.

The local geology and structural geology were reviewed. This review included a stratigraphic interpretation based on lithologic and geophysical borehole logs, and a correlation of the geology and the available bedrock elevation data with faulting. A small-scale microgravity survey was also carried out and tested as a possible way of identifying faults associated with buried bedrock valleys.

The static hydrologic, thermal and chemical regimes were examined by reworking the existing static hydraulic head data (from Wilson, 1990) and by interpreting thermal borehole logs and water chemistry data. Chemical sampling and thermal logging of accessible wells, during operation of the ATEs system, were carried out to identify possible physical and chemical changes in the aquifer that are associated with the operation of the system. Simple chemical mixing models were generated.

Aquifer test data from Phases 1 and 2 were analyzed, and the hydraulic parameters were calculated. The interpretation of these data formed the basis for developing a conceptual model for the faulted aquifer. Laboratory work was also undertaken to measure thermal conductivities of bedrock chip samples that could be used in transport modelling.

A computer code, based on the finite element method, was written to carry out simulations for groundwater flow and transport under steady-state and transient conditions. The conceptual model for the aquifer was tested by attempting to calibrate the mathematical model with existing static and transient hydraulic data. Transport simulations were superimposed on the hydrologic model to examine the thermal behaviour of the multi-well pumping and reinjection ATES system.

1.7 Thesis Overview

This thesis presents data from a variety of sources that have been compiled and interpreted, and used to develop a model that can simulate the complex well interactions that are commonly observed in faulted carbonate aquifer systems. Geological data (Chapter 2) are used to delineate a series of faults, that are believed to influence the local hydrogeology of the site. The static hydraulic head data (Chapter 3), acquired prior to the implementation of the ATES system at Carleton, are used to approximate the steady-state flow regime, and borehole temperature logs (Chapter 3) are used as an aid for interpreting the chemical data from the aquifer.

The interpretation of aquifer test data is a major focus in this thesis (Chapter 4). Various analytical methods are assessed in an attempt to determine a suitable physical

representation for the aquifer. This representation is presented as a possible conceptual model (Chapters 5 and 7) that is tested using both steady-state and transient calibration techniques (Chapter 8). The model is also used to assess heat transport between multiple pumping and reinjection wells in the aquifer during operation of the proposed ATES system at Carleton (Chapters 6, 7 and 8). The rate and pattern of thermal front migration provides estimates of thermal breakthrough times at pumping wells, which are then compared with estimates obtained from an analytical model.

The appendices include the history of the Carleton University Groundwater Energy System (A.1), the bedrock elevation data (A.2), the theory, methodology and results of gravity surveying (A.3), the borehole temperature logs (A.4), chemical data and Piper diagrams (A.5), the aquifer test data for all of the production wells and selected monitoring wells (A.6). The complete set of aquifer test data is contained in four unpublished internal reports (WESA, 1985, 1986, 1988; CESI, 1994).

Also included in the appendices are the mathematical formulations and computer codes for groundwater flow and heat transport (A.7), thermal conductivity data (A.8) and the model calibration results (A.9).

2. BEDROCK GEOLOGY

2.1 Regional Geology and Structural Geology

The Carleton campus is located in the Ottawa Valley, a structurally-controlled basin. Precambrian metamorphic rocks are overlain by a thick sequence of gently-dipping Paleozoic sedimentary rocks which consist primarily of carbonates with some shales and sandstones. These rocks have been vertically displaced and locally deformed by a series of steeply-dipping Precambrian-age faults that were reactivated most recently during the Late Cretaceous to Early Tertiary (Kumapareli and Saull, 1966).

Figure 2.1 is a generalized stratigraphic section of the Ottawa area. Detailed lithological descriptions for each of the formations are provided by Wilson (1946, 1956), Williams and Rae (1983), Williams and Telford (1986), Bélanger and Harrison (1980) and Steele-Petrovich (1986). The nomenclature used in this thesis is consistent with that used by Wilson (1956). Table 2.1 provides a brief lithologic description of the formations and shows both Wilson's (1956) fauna-based and Williams' and Rae's (1983) lithologically-based interpretation of the formational contacts.

The alignment of the Ottawa Valley is controlled by the east-west trending Ottawa-Bonnechere graben. The Carleton University campus is situated on the western limb of a smaller graben formed partially by the Gloucester Fault (Figure 2.2), which passes directly through the campus (Figure 2.3). The Gloucester Fault is a near-vertical fault, down-thrown on the northeast side, with movement ranging from zero displacement at a hinge point just northwest of Hull, Quebec, to a maximum of 500 metres to the

GENERALIZED STRATIGRAPHIC SECTION OF THE OTTAWA REGION

Figure 2.1: Generalized Stratigraphic Section of the Ottawa Area

Table 2.1: Paleozoic Stratigraphy of the Ottawa Valley

Age	Group	Formation (after Wilson, 1956)	Formation (after Williams and Rae, 1983)	Brief Description (after Michel, 1985)	Thickness (Michel, 1985)
Upper Ordovician		Queenston	Queenston		<7m
		Carlsbad	Carlsbad	Grey shale with some dolomitic layers.	<68m
		Billings	Billings	Black shale; fossiliferous; brown shale at base.	<30m
		Eastview	Lindsay	Grey limestone with interbedded shales.	<10m
		Cobourg		Sublithographic to coarsely crystalline limestone.	<48m
Middle Ordovician	Trenton	Sherman Fall	Verulam	Sublithographic to crystalline limestone; shaley limestone interbedded with calcarenite.	<30m
		Hull	Bobcaygeon	Interbedded shaley fossiliferous limestone and calcarenite.	<37m
		Rockland		Grey, sublithographic limestone with shaley partings.	<13m
		Leray		Brownish to dark grey massive calcarenite.	<9m
	Black River	Lowville	Gull River	Sublithographic, argillaceous limestone with some shale and calcarenite intervals.	<67m
		Pamelia		Shaley dolostone, sandy at base.	<9m
			Shadow Lake		
		St. Martin's	St. Martin's	Impure limey dolostone.	<3m
		Rockcliffe	Rockcliffe	Greenish grey sandstone/siltstone.	<51m
Lower Ordovician		Oxford	Oxford	Rusty dolostone with interbeds of limestone and shale; calcite-filled vugs.	<65m
		March	March	Transitional dolomitic sandstone.	<6m
Cambro- Ordovician		Nepean	Nepean	Medium- to coarse-grained, cream coloured quartz sandstone.	<80m
		Covey Hill	Covey Hill	Basal conglomerate	sporadic
Precambrian				Metamorphic Rocks

Figure 2.2 : Location of the Major Faults in the Ottawa-St. Lawrence Lowlands (Modified after Wilson, 1990)

Figure 2.3 : The Gloucester fault zone (modified after Williams et al. 1984)

southeast at Russell, Ontario (Figure 2.3). Because of an extensive overburden cover, the only on-site exposure of this fault is in the rapids of the Rideau River, adjacent to the campus.

2.2 Stratigraphy of the Carleton University Site

Michel (1985) conducted a preliminary investigation of the subsurface geology of the campus by collating and synthesizing all existing geotechnical records for the buildings, roadways and utilities with the existing geological literature. This work was expanded by Wilson (1990) who added a correlation of stratigraphic and geophysical borehole logs obtained from water wells drilled during Phase 1 of the Carleton Groundwater Project.

Wilson (1990) subdivided the campus into three major fault blocks, with some sub-blocks (Figure 2.4, from Wilson, 1990). Block I is located in the east and northeast portions of the campus and includes wells P2, RC, P3, P5, P6, Elba1 and Elba2. Block II is located in the central portion of the campus and includes wells P1, P4 and M1. The stratigraphy of well P4 (Block IIa) is slightly offset from P1 and M1 (Block IIb) and indicates additional displacement between these wells. Wilson designated the western portion of the campus as Block III, which contains the Tory well.

During the development of Phase 2 and Phase 3 of the Project, seven additional wells were drilled (Figure 2.5). Geological and borehole geophysical logs were obtained for each of these wells.

Figure 2.4: Map Showing Location of Wilson's (1990) Fault Blocks
(Modified after Wilson, 1990)

Figure 2.5: Map Showing the Location of Test and Production Wells on Campus

The stratigraphic interpretation by Wilson (1990) has been modified to include additional information from Phase 2 and Phase 3 wells. In particular, a series of natural gamma logs were compared to the Lebreton Street borehole gamma log (well DH-0-5, as shown in Figure 2.3), which is located at 601 Lebreton Street in Ottawa. This well begins in the lower portion of the Sherman Fall Formation and ends in the Precambrian.

Peaks in the natural gamma log were compared to those in the Carleton boreholes. Upper formation contacts at Carleton (Billings Formation to Sherman Fall Formation) were established on the basis of geological information and geophysical signature. In most cases, the results agree with geological interpretations of chip samples obtained during drilling (Wilson, 1990; CESI, 1994, 1995).

Tables 2.2a and b show overburden and formation thicknesses at each well in the Central and Eastern Blocks, respectively. Shallow wells, Elba1, RC and Tory have not been included because of limited information. Formation thicknesses are fairly consistent between wells in similar blocks and agree, in general, with the average thicknesses of formations in the Ottawa area (Table 2.1). The use of borehole geophysics has made it possible to define more clearly these contacts and make adjustments to lithologic logs where necessary.

Table 2.2a: Formation Thicknesses Based on Geophysical and Geological Well Logs for Central Block II Wells

Formation	P1	T31	T32	T33	P4	M1	P21	P23	T24
Overburden	6.0m	N/A	1.5m	8.2m	2.4m	10.0m	15.5m	11.4m	20.7m
Billings									
Eastview									
Cobourg									
Sherman Fall									
Hull	> 4m				> 8m		> 19m	>8m	
Rockland	23m		> 9m		21m		22m	24m	
Leray	5m		10m		6m	> 4m	9m	5m	
Lowville	30m		72m	> 44m	68m	44m	>24m	>26m	>22m
Pamelia & St. Martin	12m		11m	11m	11m	>11m		N/A	9m
Rockcliffe	53m		>12m	50m	> 5m	lower log N/A		N/A	55m
Oxford	> 4m			> 18m					>31m
March									
Nepean									

Table 2.2b: Formation Thicknesses Based on Geophysical and Geological Well Logs for Eastern Block I Wells

Formation	P2	P3	P5	P6	Elba2	P22
Overburden	9.1m	11.5m	22.9m	13.7m	14.0m	21.0m
Billings	> 11m	> 13m	> 7m	> 9m	> 14m	
Eastview	6m	6m	6m	6m	6m	
Cobourg	32m	29m	31m	35m	32m	35m
Sherman Fall	16m	18m	19m	17m	17m	16m
Hull	37m	34m	19m	34m	35m	34m
Rockland	> 11m	25m	>11m	> 4m	> 4m	> 10m
Leray		7m				
Lowville		> 9m				
Pamelia & St. Martin						
Rockcliffe						
Oxford						
March						
Nepean						

The stratigraphic profile (Figure 2.6) shows most of the major wells on campus. It was constructed using both the geological information (such as overburden thickness) and borehole natural gamma logs summarized in Tables 2.2a and b. The profile shows the relative stratigraphic positions of wells on campus and the positions of two major faults, identified as the Eastern Fault and the Central Fault. The elevation of the bedrock surface is shown for each well, and an elevation of 60.0 m.a.s.l. has been used as a zero depth datum. Consequently, the true undulating ground surface topography ($\pm 3\text{m}$) is not apparent.

Despite the large vertical exaggeration of the profile in Figure 2.6, it is apparent that certain minor displacements may be explained by dipping or undulating bedding rather than a fault. It is evident that certain displacements are greater at depth than in later strata cut by the same fault, which suggests that fault movement is synchronous with successively younger strata. The apparently large variation in the thickness of the Lowville Formation, with a uniformly thick overlying sequence of Leray and Rockland strata and underlying Pamela, St. Martin and Rockcliffe, further suggests that local displacement at the time of or prior to deposition occurred. As the Gloucester Fault is thought to be a major crustal adjustment with deep origin in the Precambrian (Michel, 1985), multiple reactivation and isostatic movement of the fault are likely to have occurred during the crustal loading and erosion of Paleozoic strata.

WELL STRATIGRAPHY

Figure 2.6: Stratigraphic Profile Showing Relative Positions of Wells and Major Faults

From the bottom of the observed stratigraphic sequence upwards, the profile is interpreted as follows:

- The Rockcliffe Formation appears to be of consistent thickness throughout the Central Block. Wells were not drilled to sufficient depth in the P4/P23 sub-Block IIb nor in the Eastern Block (I) to confirm whether or not this trend is consistent across the entire campus.
- The Pamela and St. Martin Formations are generally thin, except at P1 where they are slightly thicker.
- The thickness of the Lowville Formation, as determined from all wells which intersect the formations above and below the Lowville (namely M1, P1, T31, P32, P23, P4 and possibly T33), is highly variable. The full thickness of the Lowville is present at wells P4, P32 and P23. At P1 the Lowville is less than half of its full thickness. At M1, the entire section is present (identified in natural gamma log) but is compressed to half its normal thickness. The formations above and below the Lowville, the Leray and Rockcliffe, respectively, are of uniform thickness throughout the area. This indicates that prior to, or during deposition of the Lowville, P1 and M1 were at a higher elevation relative to the rest of the wells. P1 may have had intermittent deposition in a shallow shelf environment, reflecting periods of submergence and emergence. At M1, immediately adjacent to P1, there may have been continuous deposition but as thinner units. As these two wells are centrally located, they would have had to be uplifted by some mechanism to create a horst, or the surrounding sediments must have undergone

subsidence during deposition, with P1 and M1 being supported from beneath the lower sedimentary sequence.

During the interval between deposition of the Upper Lowville and the Hull, there may have been at least one episode of faulting. The evidence for this is that well P5 has a thinner unit of Hull Formation than is found anywhere else on campus. Low gamma counts in this interval make it difficult to determine whether the Upper Hull was missing due to erosion or whether the full sequence is present but compressed as in well M1.

- The final episode of faulting is associated with regional tectonism during the Mesozoic. At this time, the graben structure, defining the Ottawa Valley, was formed and most major faults were active. The major displacement between the Central Block (II) and the Eastern Block (I) developed at this time. Limited stratigraphic information from Block III suggests that the displacement between the Central Block (II) and Block III was small.
- Subsequent to the most recent episode of faulting, the bedrock surface has been eroded and infilled by Quaternary sediments of variable thickness.

The stratigraphy suggests that there may have been episodes of faulting in the region prior to the Mesozoic. These may be associated with loading stress developed during deposition of the sedimentary sequence or may have resulted from reactivation of deeper faults.

Evidence of deep faulting, of possible Precambrian age, is the high horizontal gradient in Bouguer gravity associated with the Gloucester Fault and a portion of the

Horizontal Gradient of Bouguer Gravity Anomalies

Figure 2.7: Horizontal Gradient of Bouguer Gravity Anomalies for the Ottawa Area. Ottawa River Outlined.

Rideau River adjacent to the campus (Figure 2.7). The Gloucester Fault corresponds with the linear, high gradient zone entering the figure from the southeast. The Rideau River intersects this feature at approximately $75^{\circ}40'W$ and $45^{\circ}22'N$. Large-scale gravity anomalies are generally associated with deeper rock, namely the Precambrian, and high horizontal gradients are an indication that significant deep-seated movement has occurred. Reactivation of Precambrian-aged faults at this location may explain the various episodes of faulting observed beneath the Carleton campus.

2.3 Bedrock Topography and Structure

2.3.1 General

Bedrock elevations for the Carleton campus have been compiled from data in geotechnical investigation reports prepared by McRostie and Associates Ltd. (1961 to present) and records from water wells (Michel, 1985; Wilson, 1990; CESI, 1994). Michel (1985) and Wilson (1990) constructed bedrock topographic maps in order to identify trends in the bedrock surface.

In general, the bedrock surface is undulating and enhances north-westerly striking lineaments (Michel, 1985; Wilson, 1990). A close correlation is found between the interpreted positions of the faults and channel-shaped depressions in the bedrock topography. These channels or valleys are thought to be remnant stream or river courses that followed fault lineaments. The geomorphology of the area, prior to glaciation, would be similar to that observed at present at Hog's Back Falls, located just to the south of the Carleton campus. At the Hog's Back exposure (described in more detail by Simpson, 1978), the bedrock has been displaced on either side of the falls. Sharp cliffs mark the

edges where erosion by the river is still taking place. The present course of the Rideau River is controlled by the position of major faults in the area.

At the Carleton campus, the overburden covers sharp cliff-like features, identified in the bedrock topography by Michel (1985), that delineate the position of major faults where substantial displacement of bedrock units has occurred with subsequent stream erosion in the fault zone. The valleys indicate that a zone of softer brecciated or highly fractured rock, up to 30 metres wide, may exist along the faults. At the surface, this zone would have undergone significant erosion, whereas at depth, it may be preserved. Therefore, fractures produced during faulting would not likely be confined to the area immediately adjacent to the fault plane, but would extend into the solid rock mass. Rock, with fine one centimetre-spaced fractures filled with secondary calcite, is seen in geotechnical cores from near the proposed faults.

The extent to which the fractures within or adjacent to the faults may be open is uncertain. A recent study of fractures in the Ottawa area by Forgrave (1995) indicated that many of the north-south trending fractures are open, whereas the east-west trending ones are often sealed by secondary mineralization, primarily calcite. Secondary minerals such as chlorite and pyrite may also precipitate within the fractures associated with these faults. The implications of secondary mineralization to the hydrogeology of the area are discussed further in Chapter 3.

The bedrock depressions adjacent to the scarps were subsequently infilled by Quaternary sediments, which has minimized any surface topographic expression. There is less than 15 metres of topographic relief across campus, with altitudes ranging from 59 to

73 metres above sea level (m.a.s.l.). The Quaternary sediments range in thickness from less than 1 metre to over 50 metres. Drill hole data indicate that a unit of dense glacial till directly overlies the bedrock in most areas of the campus, except in the buried valleys, where it is underlain by fluvial sands intermixed with gravel and boulders. Fine-grained Champlain Sea sediments cover the glacial sediments and reduce the pre-existing topographic relief.

2.3.2 Fault Interpretation on the Basis of Geology

Michel (1985) and later Wilson (1990) interpreted the lineaments, identified from bedrock topography and the displacements associated with local stratigraphic variations, as a series of northwest striking, sub-parallel displacements forming a major fault zone through the campus (Figure 2.4). Wilson (1990) confirmed that most displacements were down toward the northeast.

Based on stratigraphy, it was determined that most of the displacement had occurred along the Eastern and Central Faults. The Central Fault, separating P4 from P1 and M1, has an estimated vertical displacement of 60 to 75 metres, and the Eastern Fault has an estimated vertical displacement of 120 metres. Because most of the displacement occurs along this Eastern Fault, it is referred to as the main break of the Gloucester Fault (Michel, 1985). The Western Fault, separating Blocks (II) and (III) has only minor vertical displacement, although extensive erosion has occurred on the western side.

In addition to some minor faulting, which is generally confined to the central portion of the campus, there is evidence of a major southwest-striking cross-fault between wells P2 (Eastern Block) and P4 (Central Block). In the central part of the campus,

beneath the Engineering complex, the uppermost bedrock has been identified as Lowville Formation. Units are progressively younger toward the south; the Hull Formation forms the rapids in the Rideau River and Sherman Fall Formation outcrops on the south side of the Rideau River. The presence of Billings Formation shale north of the Engineering complex suggests that the cross-fault has a bedrock displacement on the order of 100 metres.

The cross-fault is inferred to pass through this area although its exact orientation and location cannot be determined on the basis of bedrock elevation and borehole stratigraphy alone. Fracture orientations for the Hog's Back area (Simpson, 1978) showed two major sets of fractures at roughly N 10° W and N 25° W. Only a few fractures at 90° to these were observed. This suggests that the orientation of major cross-faults, as interpreted by Michel (1985) and Wilson (1990), may not correspond with major fracture orientations in the immediate vicinity of the campus (Figure 2.4).

Although there is no indication of any significant displacement identified in the stratigraphy, the bedrock topography and the preferential hydraulic connection between the Elba wells and well P2 (Figure 1.4) suggest that a minor cross-fault extends northeastward from the main fault toward the Elba wells. The existence of several minor northeast-striking cross-faults in the central portion of the campus is also inferred from the stratigraphy.

2.3.3 The Interpolated Bedrock Map and Bedrock Surface

During the feasibility study for Phase 2, the bedrock topography and stratigraphy were reviewed to recommend sites for new production wells. It was decided that the new

wells had to be placed as close to the faults as possible to obtain high yields. Two of these wells were to be sited along the major cross-fault between Block I and Block II, and two were to be sited along the Eastern Fault.

To verify the orientation of both the major faults and the cross-faults on campus, the bedrock map was updated with new bedrock depth information obtained since 1985. An objective approach was desired for the interpretation of the data, therefore, a computer contouring package was employed. The database, giving x- and y-coordinates relative to a (0,0) origin, which corresponds to the UTM coordinates (445000E /5025400N), and a review of interpolation methods are included in Appendix 2. The density of data is not uniform and, therefore, some areas lack good control. The uneven distribution and clustering of the data, made it necessary to use an interpolation algorithm to create a uniformly spaced grid. The commercially available software *SURFER* was employed to this end. The area used for the interpolation was reduced slightly and emphasizes the immediate campus area (Figure 2.8a).

Both kriging and inverse distance interpolation schemes were employed with variable success. Triangulation produced inferior results. Octant searches generally produced less favourable results than nearest neighbour searches, with the latter enhancing the valley structures in the central portion of the campus. Inverse distance interpolation methods tended to create "bull's eye" maps, regardless of search radius and method of point selection. Maps produced by kriging showed a greater correlation with Michel's (1985) original bedrock map but enhanced the northwest-southeast trending fault structures.

Figure 2.8a: Map Showing the Area Used for Interpolation for Generating the Bedrock Map Shown in Figure 2.8b.

Figure 2.8b: Map Showing Bedrock Topography of Carleton Campus

A revised, computer-generated, contour map of the bedrock surface is shown in Figure 2.8b. The data base consisted of a total of 235 points and are distributed throughout the mapped area. The density of data points is greater in the main portion of the campus where many geotechnical holes were drilled for the buildings. The contour map was produced using kriging with a nearest neighbour 10 point search over a radius of 1074 metres (default radius for data set). Detail was lost when the search radius was reduced to 400 metres. Elevation of the bedrock surface is given in metres above sea level (m.a.s.l.). The elevation of the bedrock surface ranges from less than 26 to slightly over 60 m.a.s.l.

Notable features of this map include:

- 1) A bedrock high (60 m.a.s.l. contour) is located between 900 and 1100 metres in the x-coordinate direction and between 350 and 450 metres in the y-coordinate direction. Well P1 is located in this area. There may be some relationship between the thinner Lowville Formation in this area and the higher bedrock elevation. It is beyond the scope of this thesis to examine this aspect of the geology, although a study of this type could be considered for future research.
- 2) The bedrock surface elevation drops significantly southwest of the Western Fault, which coincides with the head of the rapids in the Rideau River. The bedrock beneath the southwest portion of the campus is believed to be Lower Lowville, based on core obtained from the northwest corner of the Library. The formation exposed in the rapids beneath the Heron Road Bridge, southwest of the campus, is Upper Lowville. The bedrock is believed to dip gently to the east in this area and the rapid decrease in

bedrock elevation seen on campus is largely attributed to erosion. The extension of the Western Fault to the northwest coincides with a major change in ground-surface elevation on the west edge of the Rideau Canal and Dows Lake. This has been interpreted to coincide with an east-dipping normal fault (Williams *et al.*, 1984), although, at least beneath the Carleton University campus, there is little evidence to suggest that any significant displacement has occurred. The canal bank is not fault-controlled, but was fluvially eroded during the Quaternary. Faulting along the margin of Dow's Lake is related to the Eastern Fault on campus.

- 3) There is a large valley extending westward which is intersected by well P5. This large broad valley likely contains the east-west cross fault, although its precise orientation remains uncertain.
- 4) A sharp incised valley is centered around coordinates (900, 600) in Figure 2.8b, with bedrock highs located to the east (Eastern Block well P3) and to the west (Central Block wells P4 and T1). Production well P22 is located on the edge of this valley, but is in the Eastern Block stratigraphically. The sharp contours are a clear representation of a laterally confined, linearly-oriented valley which coincides with a major fault.

The bedrock surface maps (Figures 2.9a and 2.9b) were generated using a normal 10 point search with the same search radius as was used above. Matrix smoothing was performed to eliminate the jagged features associated with some of the contours. Some features of these contour maps, in particular the deep incised valleys, are enhanced with this surface representation of the data. It should be noted that a vertical exaggeration of

BEDROCK SURFACE MAP FOR CARLETON UNIVERSITY
VIEW 1

Figure 2.9a: Bedrock Surface Map Looking Northwest

BEDROCK SURFACE MAP FOR CARLETON UNIVERSITY VIEW 2

Figure 2.9b : Bedrock Surface Map Looking Northeast

approximately 2.75x was used. Both the contour and the surface maps (Figures 2.8b and 2.9) represent only one interpretation of the bedrock surface beneath the campus and other possibilities exist.

Despite having produced these maps to identify the valleys associated with faults, there was still some question as to the precise orientation of some of the faults; in particular, the major east-west cross fault. The precise siting of new test and production wells along major faults greatly improves the possibility of obtaining high yields. “Missing” the fault by only a few metres can drastically reduce the well yield. P3 is an example of one such well, which, despite being located some 10 metres away from the main break in the fault, is essentially dry. The following section describes some of the surface geophysical methods that were employed to better identify bedrock valleys and, hence, delineate the major faults on campus.

2.3.4 Surface Geophysical Methods

A series of geophysical methods were employed to aid in the delineation of bedrock valleys and faults during the interval between the completion of drilling for Phase 1 (1989) and the initiation of drilling for Phase 2 (1993).

During the 1960's, a small-scale seismic survey was conducted around the area of the Tory Building. The results were inconclusive due to the presence of subsurface frost layers and poor ground conditions.

Seismic refraction profiles were constructed for a number of lines on campus (Epp, 1989). This survey suggested the presence of a single refractor at depth. The velocity of the upper layer was 232 m sec^{-1} and for the refractor, 305 m sec^{-1} . Arrival times indicated

a very rapid change in depth over a very short distance. The velocity contrast was not large, indicating that the refractor was caused by only a minor change in density, possibly the water table, or more likely, due to a vertical change in subsurface till type. Stratigraphic sections obtained from geotechnical boreholes were used for comparison purposes. In general, the results of this survey were poor.

Ground penetrating radar profiles, constructed along thirteen lines, gave improved results, although noise and diffraction-related irregularities in the data make these profiles difficult to interpret. A few profiles indicated variations in the bedrock surface (Allen *et al.*, 1994) and possibly the presence of faults at depth. It was difficult to assess whether the irregular signals, which extended vertically downwards in narrow bands below the bedrock surface, were due to reverberations or reflections off brecciated material within the fault zone.

As discussed in the previous section, the overburden within the bedrock valleys consists primarily of unconsolidated glacial till and fluvially-deposited sands and gravel. The density contrast between these sediments and the underlying consolidated bedrock is sufficient for detection using standard gravimetry. A small-scale microgravity survey, consisting of nine profiles, was carried out during the spring of 1993 to identify bedrock valleys which have been infilled by less dense material than the surrounding rock mass. The purpose was to determine whether standard gravimetry could be used to delineate buried bedrock valleys in an area which is largely unsuitable for the detection of these features using other standard geophysical techniques. Appendix 3 summarizes the results of the microgravity profiling. The theory of gravimetry, survey techniques, gravity

corrections, density estimates and a discussion of the results are covered. All gravity profiles are included.

2.3.5 Bedrock Interpolated Profiles

Bedrock profiles, corresponding to all nine gravity profiles, were generated from the interpolated bedrock map (Figure 2.8b). The profiles were constructed by approximating the endpoints of the respective gravity profiles and using the *SLICE* utility in *SURFER* to interpolate between them. It should be noted that often the gravity lines are curved (these followed pedestrian paths where possible). The profiles constructed from *SLICE* are inherently straight. Therefore, some discrepancy is expected.

Graphs showing the interpolated bedrock elevation and the corresponding variation in gravity along all of the profiles are provided in Appendix 3. A ten metre interval was used, which effectively smoothed the gravity data and eliminated some of the smaller features. As well, the scale for the gravity profiles was adjusted to coincide with variations in bedrock topography. However, the relative variation remains consistent with the data.

Minor undulations in the bedrock surface, as well as larger trends, are apparent in the gravity data, and sharp-faced valley walls are emphasized. In contrast, the interpolated surface tended to be smoothed and only indicated sharp changes where many data points are available. Therefore, along two out of the nine profiles, inconsistencies were observed. This exercise emphasizes the discrepancy which may arise when interpolation algorithms are utilized for detailed analysis of geological features. Where sufficient data are available, the results can be favourable. However, when interpolation is used, it should be used with

caution. The less than favourable correlation between gravity results and interpolated bedrock surface, along some profiles, confirms this statement.

Application of the gravity technique greatly improves the knowledge of the subsurface geological structure and enables a fairly accurate representation of the bedrock topography to be produced (Allen and Michel, in press). Where little information on bedrock elevation is available, standard gravimetry can be effectively used to identify bedrock valleys, and faults if the valleys are associated with faulting.

2.4 Fault Validation

The orientations of the faults (Figure 2.10), in particular, the position of the main east-west cross-fault between the central portion of the campus and the north-western portion of the campus, are more clearly defined than in the earlier work of Wilson (1990) (Figure 2.4). These faults have been identified using the location of bedrock valleys, the stratigraphic offsets and the preferential hydraulic connection between certain wells (eg. Elba2 and P2). Dashed lines (in Figure 2.10) show extensions of the faults that have not been confirmed on the basis of stratigraphy, bedrock topography or gravity.

The extension of the Eastern Fault to the northwest of P2 remains in question. On the basis of the gravity results, a prominent bedrock valley is present adjacent to P2. It is likely that the displacement along this fault tapers out toward the northwest, because most of the displacement is taken up along the main east-west cross-fault. However, the excellent hydraulic connection between Elba2 and P2 suggests that fracturing extends at least to P2 and possibly as far northwest as P6.

Figure 2.10 : Location of Major Faults at Carleton University

The evidence that a major fault structure closes to the north, includes exposures of Cobourg Formation in front of the Geological Survey of Canada (GSC) building at 601 Booth Street, and in the CPR railway tunnel to the northwest, near the intersection with Prince of Wales Drive. Billings Formation is encountered beneath the Natural Resources Canada (NRCan) building off Carling Avenue. This fault closure forms the northern end of a small graben, which contains the Eastern Block identified at Carleton, and is probably linked to the Eastern Fault and the main cross-fault. East of the campus, the eastern limb of this graben extends toward Billings Bridge.

The extension of the Central Fault toward P5 is questionable. However, the presence of a large bedrock valley, which was confirmed during drilling of P5, may be associated with a fault (Figures 2.8a and b).

Minor faulting within the Central Block is identified on the basis of stratigraphy. No geophysical techniques were used to confirm the location of these faults since none appear to be associated with bedrock valleys. This faulting may have pre-dated or may have been synchronous with the final faulting episode. This cannot be confirmed on the basis of the available information.

If the cross-fault by the Elba wells is present, it is either only very minor or passes to the south of these wells. There is no significant displacement to the north and no information to confirm or negate the presence of displacement to the south. A dashed line is placed here to indicate the uncertainty.

Finally, the southeast extensions of all faults remains uncertain. The presence of Hull Formation in the rapids and Sherman Fall Formation on the opposite side of the

Rideau River suggests that other cross-faults may be present along the southeast edge of the campus.

Since 1989, the knowledge base has continued to expand as more wells have been drilled on campus. Interpretations of the geology, in particular, the faults, have varied over the years (Michel, 1985; Wilson, 1990; and this research) and will likely continue to change as more information is gathered. The geological interpretations in this chapter form the basis upon which hydrogeological and thermal conceptual models are developed. The Carleton campus is situated within a complex geological setting that consists of a thick sequence of predominantly carbonate rocks that are transected by several major faults and cross-faults. The extensions of these faults beyond the campus remain questionable and this will ultimately result in difficulties in trying to determine physical bounds for the models. Nevertheless, there is some evidence to suggest that the Eastern Fault and its related cross-fault do close to the north of the study area.

3. HYDROLOGIC, THERMAL AND CHEMICAL REGIMES

3.1 Hydrogeology of the Ottawa Area

3.1.1 Regional Aquifers

Regional shallow groundwater studies in the Ottawa Region and parts of the St. Lawrence Lowlands were conducted by Brandon (1960), Scott (1967), Sobanski (1969) and Charron (1978). A compilation of these regional investigations, along with a GIS (Geographic Information System)-based hydrogeological study, was undertaken by Geo-Analysis and J. L. Richards (1992) on behalf of the Regional Municipality of Ottawa-Carleton (RMOC) to advise on a number of issues related to rural development on private services (*i.e.*, septic systems and wells). Information contained in these studies identified geologic units within the region that are considered as potential aquifers.

The productivity of the Precambrian aquifer, which is present near surface in the western part of the region, is highly variable and is largely controlled by secondary permeability (*i.e.*, fractures). In general, transmissivities are low and yields rarely meet domestic supply requirements. The lower Paleozoic aquifers consist of the Pamelia, St. Martin, Rockcliffe, Oxford, March and Nepean Formations, of which, the Nepean Formation has the highest yields of the bedrock aquifers in the area; in excess of the domestic requirement (Geo-Analysis and J. L. Richards, 1992). The Nepean Formation is more permeable near the top and within the basal Covey Hill Formation, when present (Brandon, 1960). Wilson (1990) quoted a range of yields between 2.6 and 10.5 L/s based on regional studies of this aquifer.

Wells drilled in the March and Oxford Formations yield variable quantities of water. Sobanski (1969) indicated that it was quite common to find poor producing wells immediately adjacent to high producing wells. Sporadic dry holes have also been drilled in this aquifer (Geo-Analysis and J.L. Richards, 1992). The Rockcliffe Formation has yields that are just sufficient for domestic purposes. Fracturing and jointing of the near-surface bedrock increases locally the ability of these aquifers to produce water.

Water yields in the St. Martin through to the Eastview Formations, which consist predominantly of limestone with interbedded shale, are of comparable magnitude. In general, water-bearing seams coincide with calcareous shale layers. Massive limestone units are typically low yielding and of low transmissivity, and the water is often brackish. In the Ottawa area, both the Billings and Carlsbad Formations typically produce brackish to saline groundwater (Briggs, 1995), and water supplies are commonly withdrawn from the overburden (Geo-Analysis and J.L. Richards, 1992).

In general, the bedrock aquifers in the region are considered to have relatively low transmissivities (Geo-Analysis and J.L. Richards, 1992), and results of pump tests cited in the regional surveys indicate specific capacities rarely greater than 1 L/s/m of drawdown after 10 hours of pumping (after Wilson, 1990). Charron (1978) noted that zones of improved water yield are present and that the occurrence of higher capacity wells may be indicative of permeable zones associated with faulting. Four distinct areas of higher specific capacity wells, defined as greater than 2 L/s/m of drawdown, are located approximately along the trend of the east-west portion of the Gloucester Fault east of Russell. The wells used in these studies were relatively shallow. Brandon (1960)

speculated that fractures associated with faults may be open in shallow environments, but are sealed at depth. As a result, he concluded that deep drilling to obtain higher yields was not a viable option in the immediate Ottawa-Hull area because of the numerous faults present.

Regional aquifers in the overburden consist of unconsolidated sediments of glacial, glacio-fluvial, marine or fluvial origin. The thickness of these aquifers ranges from 0 to more than 100 metres. Areas of poor well yield commonly occur near areas of good well yield. Nonetheless, pumping in these aquifers indicates that they can serve as long term supplies of potable water. A basal gravel unit, commonly present in buried valleys, provides a reliable groundwater supply for much of the area. The quality of the water from this gravel is good except when it is overlain by clay or underlain by shale.

3.1.2 Fractures and Well Yield

Fractures contribute much to the hydraulic conductivity of the aquifer. Thus, water is more easily transmitted toward wells that intersect these fractures, and the well yields are greater than those for wells that intersect fewer fractures. Within localized areas, well yield, assessed on the basis of pumping rate and duration of testing recorded in water well records (Ministry of the Environment, 1976), is highly variable. In some areas, sufficient water is found within the overburden, while an adjacent well might need to be drilled deep into the bedrock in order to obtain sufficient water for a domestic supply.

The variation in well yield can be attributed to a number of factors. First, for very shallow wells, overburden grain-size can be extremely variable. The Ottawa Valley overburden consists of till, sand and gravel, and marine clays. These units vary spatially, in

both a lateral and vertical sense. For example, a good supply well can be drilled in an extensive sand and gravel unit associated with an esker. Where overburden is thin or clay-rich, a bedrock well may be required.

Second, the bedrock geology, and thus the aquifer transmissivity, is highly variable in the region. This is due primarily to large-scale faulting in the region which has resulted in different rock sequences being juxtaposed. The hydrogeology may be distinctly different on either side of a fault as a direct result of this lithological variation. In addition, fractures associated with bedding planes or other horizontal surfaces, such as unconformities, may locally enhance flow. Determining groundwater flow in structurally complex areas, such as the Ottawa Valley, can be difficult.

Third, the fracturing, which is associated with faulting, may be intense at fault block intersections. The extent to which these fractures are open, and thus able to transmit water, both in shallow and deep environments, depends on the degree to which secondary mineralization has occurred as well as on the stress field in the area. This latter factor may act to either open or close the fractures oriented in a particular direction. A recent study by Forgrave (1995) indicated that many of the fractures in the Ottawa limestones, exposed in outcrop and oriented in an east-west direction, are sealed by secondary mineralization. However, fractures at depth may not be sealed.

The effect of faulting and fracturing on the hydrogeology is not well known for the Ottawa area. There is evidence to suggest that well yields are extremely variable and that this variability is associated with the complex geology. Whether faulting and/or fracturing is only dominant in shallow environments is also questionable.

3.1.3 Aquifer Recharge

Recharge in the Ottawa region takes place throughout the year, with a maximum occurring in the early spring (Geo-Analysis and J.L. Richards, 1992). High levels of evapotranspiration during the late spring to late summer limit the amount of recharge during these months and water levels decline. Water from heavy rainfalls during the summer months is generally lost through evaporation, evapotranspiration and runoff. Some recharge may take place during the fall when the temperature drops in the region. During the winter months, recharge is minimal or absent due to frozen ground conditions.

Sobanski (1969) used the streamflow data at Spencerville (just south of Kemptville) in the South Nation River basin to characterize recharge to a local aquifer. Although his calculations show that recharge is low in this area, they were consistent with observations made for other areas. Geo-Analysis and J. L. Richards (1992) stated “based on drainage studies in the region, the deep aquifer drainage basins are semi-independent of surface (overburden) drainage basins”. It was noted that several groundwater basins underlie surface water divides and that bedrock aquifers play an important role in the overall water budget of the region through their connection with major rivers.

The geology of the Ottawa region plays the dominant role in determining the pattern of regional aquifer recharge. In general, Precambrian rocks outcrop to the west, and the rocks become progressively younger to the east. Recharge to the individual aquifers, identified on the basis of formations or lithological variations, is likely controlled by their area of subcrop beneath the overburden or outcrop at the surface. Numerous faults disrupt the continuity of the formations and, thus, have a dominant effect on the

overall recharge pattern. The faults may act as barriers to flow, creating zones of higher pressures and possibly upwelling in areas adjacent to them or they may act as spillways and channel water to and away from areas. They may also provide direct routes to recharge and discharge areas at the surface.

Geo-Analysis and J. L. Richards (1992) identified an area, west of Ottawa (Richmond), where recharge to the bedrock aquifer may be taking place via faults connected with the overburden. In recognition of this, it is possible that recharge areas may exist in areas not generally associated with topographically-induced recharge.

The recharge area for the Carleton aquifer regime cannot be determined directly from regional studies of groundwater recharge patterns because of the complexity introduced by faulting. The following section will discuss both regional and local flow regimes in an effort to identify key recharge areas.

3.1.4 The Regional Flow Regime

Several regional studies have identified flow directions within their respective areas. Geochemical analyses by Charron (1978) defined the area along the shores of the Ottawa River as the regional discharge zone for the central part of the St. Lawrence Lowland. Based on these analyses, Charron (1978) identified a northward regional flow direction to the east of Ottawa. West of the Gloucester Fault, the direction of flow is to the north toward Lac Dechènes (Brandon, 1960).

The compilation of water well records undertaken for the Private Individuals Services Study for the Ottawa-Carleton Region by Geo-Analysis and J. L. Richards (1992) showed that flow direction is highly variable in the bedrock aquifers. A summary

representation of regional flow directions (Figure 3.1) was determined by Geo-Analysis and J.L. Richards (1992). The urban area to the north, which includes the Carleton campus, was not incorporated into this study.

The regional flow regime is clearly affected by fault boundaries and hence, geological contacts. These discontinuities probably give rise to local to intermediate flow systems. Even within large fault blocks, the flow direction is variable. For example, hydraulic head data for close to 3,000 wells in the Manotick area of Eastern Ontario (approximately 30 kilometres south of Ottawa), were compiled and used to characterize the aquifer regime (Sager-Kinsman, 1995). Sager-Kinsman (1995) attributed the preferred direction of groundwater movement in the Manotick area to topographic variations of the Nepean Formation and the underlying Precambrian basement. The Oxford, March and Nepean aquifers were observed to act as a single hydrogeological unit.

Using incremental potentiometric maps (*i.e.*, the potentiometric surface plotted for horizontal slices of aquifer), Sager-Kinsman (1995) identified an area with a low downward hydraulic gradient, but characterized the flow regime as sub-horizontal. There are at least two superimposed flow regimes in the Manotick area; one at a local- or intermediate-scale, and one at an intermediate- or regional-scale (Sager-Kinsman, 1995). Sager-Kinsman (1995) also associated a major local potentiometric low with an area in which there is significant change in lateral bedrock density (a strong horizontal gradient in Bouguer gravity (0 to 2.175 mgal/km)). Sager-Kinsman (1995) stated "It is not impossible that a fault located below the river itself could be the cause of drainage at depth in that

Figure 3.1: Regional Flow Direction Map for The Ottawa-Carleton Region (modified after Geo-Analysis and J. L. Richards, 1992).

area...It is suggested that any structural drain here is an open fracture or joint rather than a fault, since there appears to be no displacement.”

The variable flow directions, which dominate many areas of the Ottawa region (Figure 3.1) and which result in complex recharge-discharge patterns, are best explained by regional faulting. The Carleton campus, by its faulted nature, lies within one of the most complex of these environments. Faults and associated fractures may control the natural movement of groundwater under steady-state conditions and may ultimately control the movement of water under pumping and reinjection conditions.

3.2 Hydrologic Regime at Carleton

3.2.1 General

Several hydrogeological investigations have been conducted at Carleton over the last decade (Appendix 1). During early aquifer testing programs (WESA, 1985; 1986), it became evident that certain wells were preferentially connected. The wide variation in calculated aquifer hydraulic properties was the first indication that the aquifer is not homogeneous. The highly variable response times for the observation wells, which were monitored during aquifer testing, were attributed to the hydraulic effects of a major fault or series of faults passing through campus. However, as with many studies in carbonate aquifers, the results were discussed in a qualitative manner and, to date, a physically-based mathematical treatment of the observations has been lacking.

The large groundwater supply within the fault zone at the Carleton University site has not been recognized in other parts of the Ottawa Valley. All wells on the campus (Figure 3.2), ranging from 30 to 160m in depth, are artesian, suggesting that the bedrock

Figure 3.2: Map Showing the Location of Test and Production Wells on Campus

aquifers are not hydraulically connected to the Rideau River and Dows Lake. The complexity of the hydraulic responses between wells at the Carleton site is primarily a manifestation of the fracture network inherent to the faulted bedrock.

3.2.2 Static Hydraulic Head Measurements

Hydraulic head measurements were made on all of the Phase 1 production and monitoring wells at Carleton from September 1988 to August 1989 (Wilson, 1990). All water level measurements were made using a standard electric-contact water-level tape and were measured to the nearest centimetre. An extension tube and measuring tape were used for flowing wells. The well head elevations were measured with an accuracy of 0.001 metre relative to a geodetic benchmark on campus. All of the wells, with the exception of the M1 multilevel piezometer, are uncased for most of their depth.

The 1988-1989 monitoring program began at the end of the Phase 1 aquifer testing program and lasted until project construction prevented access to the wells (Wilson, 1990). Measurements were made on a daily, biweekly and then a monthly basis. The set of daily readings were recorded in the early evening of each day. Diurnal variations were measured in late July and early August, 1989. Wilson (1990) used these data to determine the hydraulic gradient across the campus, to characterize the temporal and spatial trends in hydraulic head, and to correlate the temporal trends with climatic conditions.

Diurnal head fluctuated with time of day. In the author's view, insufficient temporal data are available to confirm the relationship with precipitation, temperature and barometric pressure. Higher heads were recorded on rainy days, but no distinct

relationship was observed between hydraulic head and barometric pressure variations (Table 3.1).

Table 3.1 : Diurnal Variations in Hydraulic Head (in metres) for Selected Wells and Barometric Pressure, Temperature and Precipitation Readings for Summer 1988. Original Data From Wilson (1990).

Well	July 25 1700h	July 26 0800h	July 26 1430h	July 26 1730h	July 27 0830h	July 27 1730h	July 28 0830h	July 28 1700h
P1	60.291	60.266	60.291	60.291	60.271	60.331	N/A	N/A
P6	61.294	61.254	61.294	61.299	61.264	61.329	61.299	61.349
Elba2	61.280	61.240	61.280	61.285	61.255	61.310	61.280	61.335
Pressure (mbar)	100.5	100.6	100.4	100.1	99.9	99.4	99.6	99.8
Temperature (°C)	33.2	25.4	32.5	33.2	27.0	19.6	17.7	22.3
Precipitation (mm)	0.0	0.0	0.0	0.0	0.0	36.6	0.0	0.0

Hydraulic heads were also measured on a daily basis from September to November, 1988 (Wilson, 1990). These data were not analyzed by Wilson (1990). Daily variations in hydraulic head correspond with precipitation events. As temperature and barometric pressure were related to precipitation events (*i.e.*, on rainy days, the barometric pressure was generally low and so was the temperature), it is suggested that the well water levels respond primarily to recharge conditions and that the relationship between water levels and both temperature and barometric pressure is of a secondary nature.

Maximum monthly head variations in wells (during 1988/89) ranged from 0.5 metres to 0.8 metres. In general, well hydrographs followed seasonal recharge fluctuations related to precipitation events and evapotranspiration rates, and responded to these with

measurable variations in hydraulic head (Wilson, 1990). Table 3.2 provides ground-surface elevations, and both the average yearly hydraulic head value and the December 1, 1988 head value for each well. Wilson (1990) used the December 1988 data to construct a map showing representative hydraulic heads for the Carleton campus (Figure 3.3). Although variations occur, both daily and monthly, the pattern persists.

Table 3.2 : Ground Surface Elevations and Hydraulic Head Measurements (Dec 1, 1988 and Average for the Year). Original Data From Wilson (1990).

Well	Ground Surface Elevation (m.a.s.l.)	Hydraulic Head for December 1, 1988 (m.a.s.l.)	Hydraulic Head Yearly Average (m.a.s.l.)
P2	62.937	61.13	61.21±0.25
P3	59.598	>60.51	>60.78±0.37
P5	63.169	61.15	61.20±0.23
P6	65.469	61.16	61.27±0.23
Elba2	59.770	61.13	61.25±0.20
Elba1 (shallow)	61.770	61.00	61.06±0.23
RC (shallow)	61.290	61.13	61.17±0.21
P1	63.316	60.37	60.35±0.16
P4	61.225	60.30	60.30±0.17
T1	61.251	60.28	60.29±0.13
Tory (shallow)	67.165	60.01	59.95±0.15
T3 (overburden)	68.70	62.01	62.03±0.20

The static head measurements for the Eastern Block wells are always higher (approximately 0.7 metres) than the Center Block wells. The hydraulic head in the Center Block wells (P1 and P4) are consistently at least 0.4 metres higher than that measured at the Tory well. The higher hydraulic head at T3 was attributed by Wilson (1990) to the fact that the well is connected to the overburden aquifer (the well is ungrouted). The

Figure 3.3: Map Showing Static Hydraulic Head Measurements in December 1988 and Steady-State Flow Direction (Data From Wilson, 1990)

consistency in the magnitude of the variations is an indication that the fault blocks respond equally to external driving forces and thus, the three fault blocks are part of the same aquifer system.

The complete data set reported by Wilson (1990) was reviewed to identify any relationships between the deep and the shallow wells on campus. The Tory well is relatively isolated and is, therefore, difficult to characterize. However, the static heads for the two remaining shallow wells, RC and Elba1, and the 95 metre test well, T1, are consistently less than the static heads in their associated deeper (122 metre) production wells (Table 3.2). The static head at Elba1 is 0.19 metres lower than at Elba2; the static head at RC is 0.04 metres lower than at P2, and the static head at T1 is 0.01 metres lower than at P4. The larger difference between the static head values at Elba1 and Elba2 may be explained by connection of Elba1 (due to poor casing conditions) with the low hydraulic head overburden (the well is near the Rideau River). WESA (1985) indicated that there was significant ground seepage around Elba1 during its brief step discharge test.

Although the shallow wells are located at varying distances away from their respective deeper wells (e.g., the distance between Elba1 and Elba2 is approximately 42 metres; between P2 and RC, 34 metres; and between P4 and T1, 3 metres), the geology is consistent between adjacent shallow and deep wells. Therefore, the lower hydraulic heads measured in the shallow wells suggests that there may be a slight vertical component of upward flow in the aquifer. This conclusion is supported by the fact that during drilling, wells only became flowing artesian with depth.

A distinct spatial variation of the hydraulic head was apparent across the study area (Figure 3.3). In addition, the consistency of hydraulic heads suggests that these flow directions remain relatively unchanged throughout the year. The gradient between wells within the same block also remains relatively constant throughout the year. Excluding the shallow wells and well T3 (because of its possible connection to the overburden), the gradient is extremely low (Table 3.2) within the Eastern Block. Within the Central Block, the gradient is slightly higher (Table 3.2), although only two wells are available. The gradient across the Eastern Fault and cross fault (*i.e.*, between the Eastern and Central Blocks) is significantly higher.

3.2.3 The Multilevel Piezometer Installation M1

The history of the M1 well, which is located adjacent to the Mackenzie Building, is given in Appendix 1. Figure 3.4 is a sketch of the well instrumentation. Completion depths for the separate piezometers correspond to the depths of the open-ended tubing. All levels, except the two bottom-most levels M1-B and M1-1, are effectively sealed from each other.

Piezometers M1-PVC, M1-B and M1-1 sample the Oxford Formation below the Rockcliffe, and piezometers M1-2 and M1-3 are located within the Rockcliffe Formation, which extends from approximately 65 metres to 113 metres depth at the M1 well location (from slightly below the M1-4 piezometer to slightly above the M1-1 piezometer). Piezometer M1-4 intersects both the top of the Rockcliffe Formation and the Pamela and St. Martin Formations; and piezometers M1-5 and M1-6 are within the lower part of the Lowville Formation.

While instrumented with piezometers, static head measurements were collected both prior to the implementation of the Phase 1 system (Wilson, 1990) and during its operation (this research). The following is a summary of the static head measurements obtained prior to the implementation of Phase 1. These data (Tables 3.3 and 3.4) were collected at the same time as the well data, discussed above, and a complete data set is reported in Wilson (1990).

Table 3.3 : Monthly Hydraulic Head Measurements for the M1 Multilevel Piezometer Installation During 1988/89. Original Data From Wilson (1990).

Installation Level	Depth (metres)	Maximum (m.a.s.l.)	Minimum (m.a.s.l.)	Difference (metres)	December 1/88 values (m.a.s.l.)	Yearly Average (m.a.s.l.)
M1-PVC	159.0	64.69	>63.56	<1.13	>64.09	>63.7
M1-B	137.2	65.65	64.66	0.99	64.84	65.04±0.53
M1-1	122.0	65.53	64.34	1.19	64.39	64.81±0.57
M1-2	109.7	63.57	60.24	3.33	60.61	61.36±1.27
M1-3	82.3	62.63	61.58	1.05	61.98	62.24±0.30
M1-4	61.0	61.00	60.49	0.51	60.76	60.77±0.15
M1-5	38.1	60.75	60.18	0.57	60.45	60.44±0.18
M1-6	15.2	59.93	59.44	0.49	--	59.61±0.30

The same long-term variations, as found for the wells, were identified by Wilson (1990). These were primarily associated with seasonal fluctuations in precipitation as discussed above. Wilson (1990) noted that the deeper intervals tended to fluctuate more over time than the shallower intervals. However, the M1-2 piezometer (Table 3.3) has a greater difference and a higher standard deviation than the other piezometers. The M1-B, M1-1 and M1-3 intervals exhibit moderate fluctuations.

Short-term (daily) fluctuations measured at M1 over a two 2 week interval during November, 1988 (Table 3.4) show greater consistency. Standard deviations are relatively

low (<0.09), as expected for a shorter monitoring period, and sufficient data are available to calculate differences.

Table 3.4 : Daily Hydraulic Head Measurements in the M1 Multilevel Piezometer Installation for November, 1988. Original Data From Wilson (1990).

Installation Level	Depth (metres)	Maximum (m.a.s.l.)	Minimum (m.a.s.l.)	Difference (metres)	Average (m.a.s.l.)
M1-PVC	>137.2	--	>63.56	--	>63.56
M1-B	137.2	64.70	64.64	0.06	64.67 $\pm$ 0.03
M1-1	122.0	64.39	64.29	0.10	64.34 $\pm$ 0.04
M1-2	109.7	60.68	60.63	0.05	60.66 $\pm$ 0.02
M1-3	82.3	62.09	62.00	0.09	62.04 $\pm$ 0.03
M1-4	61.0	60.96	60.86	0.10	60.91 $\pm$ 0.04
M1-5	38.1	60.75	60.59	0.16	60.65 $\pm$ 0.06
M1-6	15.2	59.79	59.67	0.12	59.75 $\pm$ 0.05

Wilson (1990) also identified a negative relationship between barometric pressure and hydraulic head. A negative correlation is often observed in confined aquifers and accounts for the decrease in hydraulic head measured in the piezometer or well in response to an increase in barometric pressure. The greater correlation between barometric pressure variations and water level fluctuations in the multilevel, as compared to that observed in the wells, is likely the result of variations in precipitation. In the case of the former, precipitation was minimal during the monitoring period and thus, the piezometers could act as undisturbed barometers.

Vertical movement within the aquifer is clearly indicated. Using the average monthly values for M1-B to M1-6, the calculated gradient is 0.045 directed upward, which is consistent with the direction of other vertical gradients measured in wells on campus. At M1-2, lower hydraulic heads are often observed (there is no evidence to

suggest that this piezometer is incorrectly installed). This results in flow reversal between M1-3 and M1-2 (*i.e.*, flow is directed downward between these two intervals).

Table 3.5 : Hydraulic Gradients Between the Multilevel Intervals

Piezometer Level	Hydraulic Gradient
M1-PVC to M1-B	N/A
M1-B to M1-1	0.015
M1-1 to M1-2	0.280
M1-2 to M1-3	-0.032
M1-3 to M1-4	0.069
M1-4 to M1-5	0.014
M1-5 to M1-6	0.014

The hydraulic gradients (Table 3.5) between M1-B and M1-1, M1-4 and M1-5, and M1-5 and M1-6 are approximately the same. A very high gradient is present between M1-1 and M1-2, because the latter consistently has a lower hydraulic head. The negative gradient between M1-2 and M1-3 is the result of a low hydraulic head at M1-2. Finally, the gradient between M1-3 and M1-4 is slightly elevated relative to the top and bottom-most intervals.

The difference in magnitude between the hydraulic gradients measured in the P2/RC and P4/T1 wells (approximately 0.01) and that measured in the multilevel (0.045) may result from the multilevel measuring a true hydraulic gradient whereas, the wells are measuring a gradient within an open borehole which intersects all of the formations. Water originating from high hydraulic head zones in an open well is free to exit in low hydraulic head zones. The composite hydraulic head in the well adjusts itself in accordance with the transmissivities of the bridged zones (Michalski and Klepp, 1990). In the multi-level, the

high pressure intervals, which are present at depth and which are reflected in the high hydraulic heads of the lower piezometers, can be observed.

In her conclusions, Wilson (1990) identified the base of the Rockcliffe Formation as a major water-bearing zone. A similar result was observed during the drilling of three test holes at Algonquin College on Woodroffe Avenue in Nepean (a few kilometres west of the Carleton campus). During drilling of the M1 well, water-bearing zones were encountered at 106.7 metres and 112.8 metres. The water-bearing units also lie within the lower Rockcliffe Formation and are within the interval sampled by the highly variable M1-2 piezometer. Further, the relatively unstable hydraulic heads within this interval, exhibited over the course of the year (Table 3.3), support the theory that the Rockcliffe Formation is a complex aquifer having zones, particularly at its base, which can undergo significant variations in hydraulic head. However, it must also be recognized that despite its variable nature, all levels of the aquifer respond consistently to variations in precipitation /recharge.

Wilson (1990) stated that "the vertical variation in hydraulic head in the multilevel installation suggests poor vertical communication between the horizontal high permeability zones." An unsubstantiated connection was made between zones of high hydraulic heads and high permeability zones. In fact, the piezometer with the lowest average hydraulic head, M1-2 (102 to 118 metres), samples the previously-identified main water-bearing zone (112.8 metres). A thorough review and interpretation of the aquifer test data (Chapter 4) obtained for the multilevel well provides additional information on vertical connection within the aquifer regime.

It is suggested here that the lower intervals, M1-PVC, M1-B and M1-1, are different because they are within a different aquifer, namely the Oxford Formation, and not because they are associated with significant fracturing. The well drilling logs do not indicate any significant fracturing (or water-bearing zones) in the lower portion of the well. In addition, the gradients between the levels are low and uniform. Due to the Oxford Formation's higher pressures, water moves upward to the contact with the Rockcliffe Formation, where it encounters a low pressure and perhaps easily-drained horizon. The low hydraulic head at this contact zone implies that water is directed toward it from both below and above the zone, with a higher gradient in the former direction. Its variability suggests that, like a river or stream, the interval takes on water or loses water depending on factors, such as natural recharge. Since most of the water in this zone is probably derived from the Oxford Formation, stresses within the Oxford aquifer will be the dominant control on the hydraulic pressures in this zone. The contact zone is likely weathered and may be significantly fractured or may contain numerous solution cavities.

The erratic head record at M1-2 suggests that the contact zone between the Rockcliffe and Oxford Formations is highly sensitive to increases and decreases in stored water. The storativity of an aquifer is a measure of the amount of water released from storage per unit decline in hydraulic head. Since the head is variable within this specific interval, the storativity must be fairly low. If the interval is highly fractured, as suggested by the well record, then water will be transmitted readily along, but the quantity of water 'stored' within the fractures will be low. The connectiveness of the fractures strictly controls from how far away water is derived.

Hydraulic head values above the Rockcliffe Formation remain relatively stable and gradients are low and of uniform magnitude. Thus, the upper levels, M1-6 and M1-5, are probably associated with a different aquifer. The slightly elevated gradient between M1-3 and M1-4 may be explained by the fact that M1-4 samples two separate aquifers, one with a high gradient and one with a low gradient.

3.2.4 Hydraulic Head Monitoring - 1992 to Present

Hydraulic head measurements were made periodically during this research. These data were obtained from several observation wells on campus and some of the new test and production wells as they became available. The Phase 1 production wells, consisting of P1, P2, P4, P5 and P6, were often inaccessible, either because they were in operation or because no access port was present. Flow meters and pressure sensors in these wells have failed, therefore, there was no prior record of Phase 1 operating conditions within the wells themselves. The new hydraulic data merely serve as a record of water levels under operating system conditions.

It was evident at the beginning of this monitoring program, that the data were insufficient and questionably reliable. Well flow rates and the actual configuration of wells has been altered significantly from that proposed for the original system by WESA (1988). The end result, in terms of monitoring, is that the system has never been operated for any length of time in a configuration that can be identified. Log books in the Maintenance Building are blank, for months at a time, while repairs were being made. At times, water was circulating through the pipes but the heat exchangers were not in use. There was no concise record of any maintenance schedules or which wells were operational and at what

rates the wells were being pumped or reinjected. In addition, some of the Phase 1 wells were operational during the drilling programs for Phases 2 and 3.

All hydraulic head measurements are included in Tables 3.6a to 3.6c. Water levels in wells were measured with a standard electric tape with a precision of ± 0.005 metres. Because the diameter of the PVC tubing in the multilevel well, M1, was too small to accommodate the diameter of the electric probe, these water levels were measured using a section of coloured wire, which, when inserted into the tubing and then retracted showed the approximate level of the water. The dry section of the wire was then measured with a meter tape. It is estimated that the precision of these readings is on the order of ± 0.02 metres. Wilson (1990) used a small-diameter electric tape for prior measurements at M1 (± 0.01 m). This tape was unavailable for this research.

Table 3.6a contains the monitoring data acquired at various wells, including the multilevel well, M1, from late April, 1993 to mid-May 1993. Measurements were made every three to five days. Since the Maintenance log books did not record any information on the status of the system during the period between November 1992 and July 1993, it is impossible to be sure what these measurements represent. The maintenance staff thought that the system was off, but they were unsure.

The records for Elba1, P3, RC, M1 and T3 indicate that over the three week period, hydraulic heads declined, and are also consistent in magnitude to those measured by Wilson (1990) prior to system implementation. The rate of decline is highest in late April, levels off slightly in early May, and declines further toward mid May. In early May, Parks Canada begins to fill the canal, which usually takes between 3 to 10 days. During

Table 3.6a: Hydraulic Head Data for April 1993 to Mid-May 1993

WELL	26 April 93	28 April 93	30 April 93	03 May 93	05 May 93	10 May 93	12 May 93	17 May 93
Elba1	61.55		61.45	61.33	61.32	61.36	60.58	--
Elba2								59.73
P3		>60.51	>60.51	60.49	58.51	>60.51	59.82	
RC	61.59		61.42	61.41	61.44	61.44	60.75	--
M1-PVC								
M1-B		flowing	flowing	flowing	flowing	flowing		
M1-1		flowing	flowing	flowing	flowing	flowing		
M1-2		63.97	63.91	63.90	63.86	63.55		
M1-3		62.61	62.47	61.56	62.58	62.56		
M1-4		61.60	61.47	61.31	61.26	61.36		
M1-5		61.29	61.02	60.97	61.01	61.14		
M1-6		60.39	60.36	60.28	60.26	60.37		
P1		60.81						
P4		62.31						
T3		62.63	62.63	62.57	62.57	62.58	61.67	
Tory	60.32				59.99			

November 1992 to July 1993
No pumping

May 10/93
Started to fill canal
takes 3 to 10 days

Pumps at MR1 shut off Sept 13/92
Sept 24/92 MR2 shut down
Logs book stop recording data in November until July

Table 3.6b: Hydraulic Head Data Prior to Drilling of
Production Well P22 (November 1993)

WELLS	5 Nov 1993	8 Nov 1993	11 Nov 1993
Elba1	61.23	61.32	
Elba2	60.91	60.99	61.06
RC	61.15		

Well Operation Details
P1 pumping into P4 and still was during P22 pu
Probably pumping P6 into P5 since water levels
when P6 was shut of before P22 pump test.

Table 3.6c: Hydraulic Head Data Following Drilling of
Phase 3 Wells, T31, T32 and T33 (October
and November 1994)

WELLS	24 Oct 1994	27 Oct 1994	7 Nov 1994
T31	59.63	59.63	61.12
T32	58.83	58.82	58.79
T33	58.67	58.65	56.81
Tory	58.45	58.45	58.39
T24	60.33	60.21	60.81
T3	60.60	60.57	60.36
M1	58.36	58.40	57.97
P23	>63.20	>63.20	66.12
RC	60.00		
T4		flowing	
Elba1		60.08	60.87
Elba3		59.98	>60.62

Well Operation Details
Oct 24 P1 to P4 at 26.7 l/sec
P2 to P5 at 50 l/sec

Nov 4 P1 to P4, P5 and P2 shut off
P1 to P4 at 1579 l/min
P6 at 2.25 l/min
P5 at 100 l/min
P2 at 55.1 l/min

this time, water levels in the Rideau River fall, exposing the rapids near the campus. There is a slight drop in water levels on May 12, which is two days after filling of the canal had begun. The similarity in magnitude of hydraulic heads measured during this research program and earlier by Wilson suggests that the system was probably not in operation at the current measurement time, or that the disturbance due to pumping and reinjection was minimal. In the former instance, if the system had been shut down in November of the previous year (1992), the aquifer would have had sufficient time to re-equilibrate to non-pumping conditions. In the latter instance, the effect due to pumping and reinjection would be restricted to areas immediately around the wells in operation.

In contrast to the regular variation in shallow overburden pressures and well hydraulic heads, the M1 water levels measured during this study are more variable. The upper levels, M1-6 to M1-4, all respond in a way similar to that observed in the open wells. The deeper intervals, from M1-3 to M1-PVC, could not be measured, since flowing conditions were present. Both M1-3 and M1-2 do not increase.

The data contained in Table 3.6b show the variations in hydraulic head over a brief interval of time just prior to drilling the production well P22 (November 1993). Production well P21 had already been drilled (late September to October, 1993). At this time, P1 and P4 were probably in operation. Well P1 was being pumped at a rate of close to 25 L/s (330 IGPM), and P4 was receiving water at the same rate. In addition, P6 was probably pumping into P5, at a rate closer to 60 L/s (800 IGPM). Just prior to the pump test at P22 (December, 1993), well P6 was shut off. Water levels rose in the monitoring wells RC, P22, P3 and Elba2 just prior to this pump test.

The hydraulic heads measured following drilling of the new test wells for Phase 3 (T31, T32 and T33) are recorded in Table 3.6c. On October 24, 1994, P1 was pumping at a rate of 26.7 L/s into well P4. Well P2 was pumping water at a rate of 50 L/s into well P5. These flow rates were recorded from a computerized monitoring system in the Maintenance Building. This system was implemented in conjunction with a modification to the groundwater heat exchange system in the Commons Building in August 1993.

The hydraulic heads for wells located within the Central Block, namely T31, T32, T33, Tory, T3 and M1, are all slightly lower than those measured by Wilson (1990). These lower heads are likely the direct result of pumping at P1, although further analysis of aquifer test data is required to be able to predict the effect this well has on the system, particularly when reinjection is taking place at P4. On November 4, 1994, P5 and P2 were shut off. On November 7, the Eastern Block wells, Elba2 and Elba1, recorded higher water levels, on the order of 0.8 metres. In the Central Block, the hydraulic heads all decreased.

Table 3.7 shows the depth to water measured in three shallow overburden piezometers located on the west side of the Herzberg Laboratories (Figure 3.5). These wells were installed prior to the construction of the expansion to the Herzberg Laboratories. A steady decline in water level is exhibited by these shallow piezometers over the two week period. This decrease is probably related to a lack of precipitation over the time interval. The rise in hydraulic head that was observed at Elba1, around May 10th, is not observed here.

Figure 3.5: Map Showing the Locations of Shallow Piezometers on Campus

Table 3.7 : Shallow Piezometer Depths and Water Level Depths Measured
Between April 28 and May 12, 1993.

Piezometer	Depth (metres)	April 28/93 (m)	April 30/93 (m)	May 3/93 (m)	May 5/93 (m)	May 10/93 (m)	May 12/93 (m)
1	2.47	2.16	2.22	2.30	dry	dry	dry
2	6.90	1.53	1.67	1.81	1.88	1.97	2.02
3	8.5	2.09	2.22	2.37	2.50	2.69	2.74
		sunny +10°C	cloudy +15°C	sunny +17°C	cloudy +18°C	sunny +25°C	sunny +18°C

Four shallow two-level piezometers (Figure 3.5), which are slightly deeper than those discussed above, were installed following the implementation of Phase 1 when concerns were raised by the University about the possible connection of the production wells to the shallow subsurface. Overburden pressures were measured during this research at the two depths (Table 3.8).

Table 3.8 : Pressures Measured in Shallow Two-Level Piezometers on campus.

Piezometer	Depth (metres)	April 28/93 (KPa)	April 30/93 (KPa)	May 3/93 (KPa)	May 4/93 (KPa)	May 5/93 (KPa)	May 6/93 (KPa)	May 7/93 (KPa)
PZ1- 1	7.62	137	154	223	190	183	189	123
- 2	5.49	115	130	212	175	167	172	102
PZ2 - 1	8.23	138	132	208	179	173	180	125
- 2	7.01	136	131	210	178	172	180	127
PZ4 - 1	9.14	100	117	170	178	186	179	145
- 2	6.70	95	112	168	175	181	175	143
PZ5 - 1	4.88	97	130	140	178	158	179	150
- 2	2.44	81	119	136	173	153	176	145
Temperature outside		sunny +10°C	cloudy +15°C	sunny +17°C	cloudy +20°C	cloudy +18°C	sunny +20°C	cloudy +9°C

Table 3.8 cont'd :

Piezometer	May 8/93 (KPa)	May 10/93 (KPa)	May 12/93 (KPa)	May 19/93 (KPa)	May 31/93 (KPa)	Aug 26/93 (KPa)
PZ1- 1	228	243	211	138	157	192
- 2	224	238	203	118	138	178
PZ2 - 1	209	247	205	130	138	182
- 2	212	237	205	130	137	181
PZ4 - 1	187	196	207	106	125	172
- 2	190	194	205	101	120	168
PZ5 - 1	156	204	203	108	127	N/A
- 2	156	204	202	98	120	N/A
temperature outside	sunny +28°C	sunny +25°C	sunny +18°C	cloudy +9°C	cloudy +13°C	sunny +25°C

Pressures in these piezometers are variable, and this variability is probably closely linked to atmospheric pressure variations. Overburden pressures were usually higher on sunny days and lower on cloudy days. There does not appear to be any definite relationship between the filling of the canal and the overburden aquifers; however, a decline in overburden pressure does begin to take place around May 12 and reaches a low around May 19.

Apart from the physical hydraulic head data acquired, several observations were made by the author concerning well connection and hydraulic heads within the Central Block. The hydraulic head at P21 is estimated at >63.0m (ground surface elevation) since this well normally flows. The preferential connection between wells P21 and P23 became evident during drilling. While P23 was being drilled, P21 would cease to flow. This high degree of connection was also observed during aquifer testing (Chapter 4). On one occasion (November, 1994), the drilling contractor connected an extension pipe to well

P23 so that a static water level could be measured. The hydraulic head at well P23 was 65.94 metres.

High static heads at wells P21 and P23 may be the result of reinjection at well P5 or at well P4. Alternatively, they may be the result of lower transmissivity upper aquifer horizons (there were no identifiable water exit points in the temperature logs for these wells). In this instance, the hydraulic head in the open borehole will equilibrate at an elevation which is higher than that for other wells which have dominant water exit points. Wells P21 and P23 will flow because the hydraulic heads exceed ground surface elevation.

3.2.5 The Local Flow Regime

To date, no study has determined the flow direction in the immediate Ottawa area, but it is unlikely that the campus flow direction would be consistent with that of the surrounding areas due to the extensive faulting. Therefore, a specific flow direction for the campus is difficult to determine on the basis of regional information. Wilson (1990) suggested that all of the wells cited in these regional surveys were shallow wells and that the flow directions associated with the shallow regional flow systems may not coincide with those of deeper flow systems such as Carleton. However, there is no evidence to suggest that flow within the shallow bedrock aquifers is separate from that within the deeper aquifers. In fact, the shallow wells, Elba1, RC and Tory, all exhibit magnitude variations in hydraulic head, on a monthly, daily and diurnal basis, which are consistent with the magnitude variations in hydraulic head in the deeper wells. The flow systems may be linked as in the Manotick area (Sager-Kinsman, 1995).

Wilson (1990) placed the study area in a transition zone along the southern margin of a regional discharge zone. Groundwater static hydraulic head measurements at the M1 well and a low upward vertical gradient between the shallow well, T1, and the nearby deep well, P4 formed the basis for this conclusion. These conditions imply that local aquifer discharge conditions are present; however, it does not mean that the campus is strictly situated within a regional discharge area.

The hydraulic heads, measured in the wells at Carleton, which are of approximately equal magnitude in the eastern and northern portions of the study area (*i.e.*, in the Eastern Block) and which decrease toward the Central Block, indicate a south/southwest flow direction for the campus (Figure 3.3). However, this flow direction is reversed relative to the majority of the flow directions in the region (Figure 3.1). The flow direction is also opposite to the direction of dip of the beds which have a northerly strike and shallow dip. This implies that boundary conditions must exercise a dominant control over the flow regime.

This southwest flow direction is toward the Rideau River. Faulting, on both a regional and local scale, is suspected to have a dominant influence on the way water is transferred from the recharge areas, via the fault blocks, to the destination in a discharge area. Local sinks, derived from faulting, can effectively take control of this movement on a number of scales. If the Rideau River acts as a sink, as it does in the Manotick area (Sager-Kinsman, 1995), then groundwater will be diverted toward it.

Vertical connectivity between aquifers is important from the point of view of modelling the Carleton aquifer, because it ultimately affects the choice between using a

two-dimensional (plan-view) or a three-dimensional model. A full three-dimensional view or model of the aquifer is only possible if sufficient data are available at discrete depths for a large number of points. At Carleton, only one multi-level piezometer is available. Nevertheless, there is a wealth of data available for this multilevel well. In addition, there are temperature logs for all production wells and some of the monitoring wells (discussed in Section 3.4).

In the authors view, the hydraulic head data provide sufficient evidence that the aquifer can be modelled as a two-dimensional system. That is, that all horizons of the aquifer, whether defined by highly fractured intervals or by intervals of different lithological composition, are interconnected to such a degree that the aquifers behave largely as a single hydrogeological unit.

The observed hydrogeologic regime at Carleton may be controlled by several factors. These factors are presented on the basis of the distribution of static hydraulic heads (both horizontal and vertical) for the campus.

- 1) The fault blocks consist of different lithologic units, therefore, the transmissivities may be different in each block. The difference in transmissivity may essentially isolate one fault block from another hydrogeologically and, thus, control both the flow direction and the magnitude of the hydraulic gradients between fault blocks. The lower transmissivity aquifers will constitute hydrogeologic barriers causing water to be diverted at their boundaries. The presence of hydrogeologic barriers may explain the fact that higher yielding wells, at Carleton, are associated with fault block intersections.

- 2) In adjacent fault blocks, water-bearing or fractured horizons, such as the Rockcliffe-Oxford Formation contact, may not coincide across the fault, and movement of groundwater will be restricted and slow.
- 3) The faults may be controlling the flow regime to such an extent that all other variations in lithology become insignificant. The degree of well interconnection, as exhibited in Figure 1.4, suggests that the faults play a major role in controlling the movement of water in the subsurface. Intense fracturing along faults may explain the higher transmissivities in faulted areas such as the Carleton campus, and may also explain the variable well yields in the Ottawa region. However, the poor connection exhibited between certain wells suggests that the faults are at least partially-impermeable features.

3.3 The Thermal Regime

3.3.1 The Thermal Regime of the Ottawa Area

The thermal regime of the Ottawa Valley has been largely unexplored. This is due, in part, to the absence of deep boreholes in the region. Prior to the drilling of the Carleton boreholes, the only deep boreholes consisted of the Observatory Crescent borehole (BH-0-8, Figure 2.3), the Russell borehole (Figure 2.2), and one other borehole located in Franktown near Carleton Place, Ontario (west of the study area, Figure 2.2). Since water movement can be quite significant in the upper Paleozoic sediments, a good estimate of the thermal gradient is only possible where boreholes penetrate the deep Precambrian rocks.

Jessop and Judge (1971) discussed the results of temperature logging in five boreholes in southern Canada. They described the drilling of the Observatory borehole which is summarized as follows; 14 metres of overburden, 220 metres of Ordovician sediments, about 100 metres of Precambrian schists and weathered gneisses, and Precambrian gneisses to a final depth of 602 metres. Temperature logs for the uppermost sediments indicated water movement. Rocks between 240 and 300 metres were described as very fragile and porous and, therefore, reliable thermal conductivity measurements could not be made in this interval. Only measurements below 300 metres were used to calculate heat flow.

The mean thermal conductivity, which was measured for 47 samples in the interval of 330 to 360 metres, was $2.37 \text{ W m}^{-1} \text{ }^{\circ}\text{C}^{-1}$ ($5.67 \text{ mcal cm}^{-1} \text{ sec}^{-1} \text{ }^{\circ}\text{C}^{-1}$). The mean temperature gradient in this interval was $14.2 \text{ }^{\circ}\text{C km}^{-1}$. The calculated heat flow was 33.7 mW m^{-2} . Beck (1977, 1982) discussed the effects of glaciation on the observed thermal regime and showed techniques used to correct the calculated heat flow values, derived solely from gradient and thermal conductivity measurements. Beck (1977) suggested that corrections to heat flow, on the order of several mW m^{-2} , could be anticipated for wells less than 400 metres deep. The corrected heat flow value for the Observatory hole was 42 mW m^{-2} (Jessop and Judge, 1971).

Recent climatic warming has also disturbed the shallow temperature regime (<100 metres). The upper portions of temperature curves have been shifted to warmer temperatures in response to surface warming, while the bottom portions remain unchanged due to the slow propagation of heat. The result is a bow-shaped temperature

plot, which features a temperature minimum and gradient reversal at approximately 60 to 70 metres (Drury, 1984). Lewis (1975) suggested that such reversals are commonly observed in boreholes in Eastern Canada and probably reflect a climate change near the beginning of the present century.

Near surface effects are clearly evident on the thermal profile for the Observatory well (Figure 1.2). The temperature minimum, 8.8°C, is located at approximately 70 metres depth. Above this depth, the temperature increases to reflect recent warming. Shallow temperatures, less than 10 metres depth, reflect seasonal variations in ground temperature and the proximity of buildings (Michel, 1985).

3.3.2 Borehole Temperature Logging at Carleton University

All of the production wells, and several of the monitoring wells, which were drilled on the Carleton campus have been logged for temperature over the course of the past decade. Three different logging systems were used at different times, and by different groups, to measure temperatures in these boreholes.

Early temperature logs for the Carleton shallow wells, Elba1, RC and Tory, were made by the former Earth Physics Branch of Energy, Mines and Resources, Canada using a portable temperature logging system (described in Appendix 4). The temperature profiles for the three shallow wells at Carleton begin to converge with that of the deep Observatory borehole (Figure 1.2). These early logs confirmed that the wells drilled at Carleton would likely intersect the desired thermal envelope (around $9.5^{\circ}\text{C} \pm 0.5^{\circ}\text{C}$) between 42 and 122 metres depth.

In addition to these logs, temperatures were measured in the Elba2 well following the constant discharge test at that well (WESA, 1986). These four original wells, Elba1, RC, Tory and Elba2, were re-logged, subsequent to drilling for Phase 1. The Phase 1 wells, P1, P2, P4, P5 and M1, were also logged at later dates and these are included in Appendix 4. Wells P3 and P6 were not logged.

All of these logs were acquired following a brief period of equilibration in the borehole subsequent to drilling. When the period of thermal re-equilibration, following drilling, is insufficient in duration, "passive" fractures become apparent (Drury, 1989). Even with sufficient re-equilibration time, high pressure zones associated with discrete fractures or fracture zones may completely dominate the thermal profile (Jessop, 1990).

Temperature logs at Carleton are influenced by water circulation within the boreholes. Wells P2, P5, Elba2 and, to some extent, M1, all exhibit water flow within the open boreholes (Appendix 4). This effect is manifested as a near isothermal temperature field (*i.e.*, close to zero gradient) for much of the profile. The Central Block wells, P1 and P4, are less affected by water circulation and, hence, the thermal profiles retain the bowed appearance typical of Eastern Canada (Lewis, 1975).

To examine this relationship for the Carleton wells, Wilson (1990) interpreted temperature logs, obtained from a continuous logging system during the fall and winter of 1989, for all of the Phase 1 production wells (P1, P2, P4, P5 and P6), as well as P3 and Elba2. This logging system was operated by the Borehole Geophysics Section of the Geological Survey of Canada (GSC). The added vertical spatial resolution, combined with the results of density, full gamma ray spectra, induced polarization (IP), and resistivity

borehole logs, provided sufficient detail to identify the particular fractures whose hydraulic pressures tended to dominate the temperature regime. A brief description of this logging system is provided in Appendix 4 and by Bristow and Conoway (1984).

Wilson (1990) provided a thorough description of all of these continuous borehole temperature logs. Water inflow and outflow locations were identified on each profile using both the temperature and temperature gradient profiles. Flowing groundwater, entering or exiting a borehole, often results in a prominent spike in the temperature gradient profile (Conaway, 1987). Wilson (1990) attributed these to discrete narrow zones or fractures within the wells.

Wilson (1990) only identified up to a couple of water entry points and a single water exit point for each profile. These were based on the location of dominant spikes. However, many of the logs show several water entry points. In particular, at well P2 (Appendix 4 in Wilson, 1990) there are many prominent spikes in the bottom third of the well.

The density logs for many of these boreholes (Wilson, 1990) show extensive zones where intense fracturing occurs. In well P2, low density spikes, associated with fractures, occur along the full length of the borehole, while water entry points, identified by Wilson (1990) on the basis of temperature gradient logs, occur only at three distinct points at the bottom of the hole. Water entry and exit points are strictly associated with zones or fractures of high and low hydraulic potential, respectively. The thermal logs give no indication of the transmissive properties for other fractures which are intersected by the borehole, but which have insufficient hydraulic head to affect the thermal profile.

In addition to the logs obtained and analyzed by Wilson (1990), which characterize the static or non-operating groundwater system at Carleton, several temperature logs were obtained in conjunction with this current research. Selected wells, including P3, RC and Elba2, were logged during August and September 1992, using a different continuous logging system which is operated by the Terrain Sciences Division of the Geological Survey of Canada (Figure 3.6). It is still uncertain whether the groundwater system was operating at the time these logs were taken, since the maintenance log books were not complete.

The profile for the RC well is dominated by near-surface effects, because this well is shallow. The profile for Elba2 is almost identical to that obtained for this well during 1988. The temperature log for well P3 does show slight warming compared to the earlier 1989 logs (Wilson, 1990). In 1989, the isothermal portion of the well was approximately 9.2°C, while the log for 1992 shows the isothermal section to be approximately 9.5°C. The discrepancy may be related to the different logging systems and, in particular, to the curves used for calibrating the thermistors (V. Allen, Geological Survey of Canada, Personal Communication). Alternatively, the discrepancy may be related to the operation of the groundwater system. Well P3 is located close to well P4, which may have been reinjecting warm water during summer cooling mode.

Figure 3.6: Continuous Borehole Temperature Logs for RC, Elba2 and P3 (1993)

The final sets of temperature data were acquired following both the Phase 2 and Phase 3 drilling programs. The Borehole Geophysics Section of the Geological Survey of Canada logged all of the new wells using gamma and temperature probes. The temperature logs for the Phase 2 wells are included in Appendix 4 for reference. Temperature logs for the Phase 3 test wells, T31, T32 and T33 are not available.

3.3.3 Groundwater Temperatures During Drilling

During the drilling of the first deep well, Elba2, WESA (1986) noted that groundwater temperatures in the fracture zones with the highest pressures (artesian zones) were approximately 9.5°C. These measurements were made on water samples collected during drilling using a standard thermometer.

Temperatures were also measured during the step discharge test on the Elba2 well. These data are shown in Table 3.9. At the lower discharge rate (73 IGPM), water temperatures remained relatively high throughout the duration of the step. The temperature of the water dropped by slightly more than 0.5 of a degree when the discharge rate doubled, and remained the same for the remainder of the test. The accuracy of the thermometer is estimated at ± 0.1 to 0.2°C.

Table 3.9: Temperature Data Collected During the Step Discharge Test at Elba2 (Data From WESA, 1986)

Discharge Rate (IGPM)	Duration of Step (minutes)	Temperature (°C)
73	25	11.0
	50	11.2
143	30	10.5
	40	10.5
202	30	10.5
500	30	-----

These temperature data suggest that water is derived from different source regions as the duration of the test increases. However, it is impossible to predict whether flow in the productive portions of the aquifer is the cause of this, or whether connected fracture zones, which have a non-uniform distribution within the aquifer, are the pathways for cooler water. Alternatively, leakage into the productive portion of the aquifer, from above or below, may be taking place. Regardless of the direction of the source, there is a distinct cooling over time.

Temperatures were also measured during the constant discharge tests at RC and Elba2 (Tables 3.10 and 3.11, respectively). Figures 3.7 and 3.8, which correspond to these data, are a graphical representation of the results.

The early temperatures measured during the constant discharge test at RC are consistent with the observations made during the step discharge at Elba2. Initially, the temperatures at RC are high. After approximately one hour, the water temperature decreases and remains low until around 12 to 13 hours into the test, at which point it rises

Table 3.10 : Temperature Data Collected During the Constant Discharge Test at RC (Data From WESA, 1985).

Time (hours)	Temp (°C)	Time (hours)	Temp (°C)	Time (hours)	Temp (°C)	Time (hours)	Temp (°C)	Time (hours)	Temp (°C)
0	11.0	5	10.1	10	10.1	15	11.0	20	10.9
1	11.0	6	10.1	11	10.1	16	11.4	21	11.1
2	10.5	7	10.1	12	10.3	17	10.9	22	11.1
3	10.3	8	10.2	13	10.6	18	11.0	23	10.1
4	10.2	9	10.1	14	11.0	19	10.8	24	10.8

Table 3.11 : Temperature Data Collected During the 72 Hour Constant Discharge Test at Elba2 (Data From WESA, 1986).

Time (hours)	Temp (°C)	Time (hours)	Temp (°C)	Time (hours)	Temp (°C)	Time (hours)	Temp (°C)	Time (hours)	Temp (°C)
0	10.9	15	10.1	30	10.5	45	10.1	60	10.4
1	10.3	16	10.2	31	10.5	46	10.5	61	10.3
2	10.5	17	10.2	32	10.6	47	10.6	62	10.3
3	10.5	18	10.4	33	10.6	48		63	10.4
4	10.5	19	10.4	34	10.6	49	10.6	64	10.3
5	10.5	20	10.4	35	10.2	50	10.6	65	10.3
6	10.5	21	10.3	36	10.1	51	10.5	66	10.3
7	10.5	22	10.3	37	10.2	52	10.6	67	10.2
8	10.7	23	10.3	38	10.2	53	10.6	68	10.2
9	10.6	24	10.2	39	10.0	54	10.8	69	10.3
10	10.4	25	10.3	40	10.0	55	10.8	70	10.3
11	10.4	26	10.3	41	10.0	56	10.8	71	10.4
12	10.4	27	10.4	42	10.0	57	10.8	72	10.3
13	10.2	28	10.6	43	10.0	58	10.7		
14	10.2	29	10.6	44	9.9	59	10.6		

steadily to temperatures similar to those at the beginning of the test. It appears, that at low discharge rates, there is a sufficient supply of higher temperature water in the vicinity of the well. As pumping continues, or as the rate of pumping increases, water is forced to come from different parts of the aquifer which are at lower temperatures. The increase in temperature observed midway through the test may be attributed to yet another portion of the aquifer which is accessed during pumping or to leakage.

Temperatures vary significantly during the constant discharge test at Elba2. The range in temperature is 9.9°C (at 44 hours) to 10.9°C (at the beginning of the test). A distinct pattern is difficult to identify. However, these data indicate that temperatures remain at $10^{\circ}\text{C} \pm 0.3$ during the 72 hour test.

TEMPERATURE AND CONDUCTIVITY CONSTANT DISCHARGE AT RC

Figure 3.7: Temperature and Electrical Conductivity Variations at RC

TEMPERATURE AND CONDUCTIVITY CONSTANT DISCHARGE ELBA2

Figure 3.8: Temperature and Electrical Conductivity Variations at ELBA2

During the 1993/1994 Phase 2 testing program, temperature and conductivity measurements were recorded at the pumping wells (CESI, 1994, 1995). Tables 3.12a, b and c summarize the results for each of the 48 hour constant discharge tests at wells P21, P22 and P23, respectively.

Table 3.12a : Temperature and Conductivity Measurements During Aquifer Testing at P21 (Data From CESI, 1994).

Time (hours)	Temp (°C)	Conductivity (μ S/cm)
0.5	12	2000
2.5	10	1520
5	11	2000
7	11	1900
24	9	2050
25	12	2000
29	12	2020
48	12	2000

Table 3.12b : Temperature and Conductivity Measurements During Aquifer Testing at P22 (Data From CESI, 1994).

Time (hours)	Temp (°C)	Conductivity (μ S/cm)
2.5	10	1600
4.5	10	1560
7	10	1610
9	10	1610
24	10	1610
28	9	1610
30	10	1600
46	9	1650
48	10	1520

Table 3.12c : Temperature and Conductivity Measurements During Aquifer Testing at P23 (Data From CESI, 1994).

Time (hours)	Temp (°C)	Conductivity (μ S/cm)
0	11	1830
1	10	1780
4.5	12	1970
8.5	12	1960
24	11	1870
33	10	1810

Discharge rates for these tests were between 700 (at well P21) and 800 IGPM (at wells P22 and P23). Although these temperatures were only measured to the nearest degree, several observations can be made. During the two constant discharge tests at wells P21 and P23, which are located in the Central Block, temperatures were up to 1°C higher than those measured during the original Elba2 and RC tests prior to operation of Phase 1. At the time of these Phase 2 tests, well P4 was reinjecting warm water. The actual reinjection temperature is not available; however, it is greater than the temperature recorded at the pumping well, P1 (13.82°C), because these two wells were operating in summer (cooling) mode. The higher temperatures during this test may be the result of reinjection at well P4.

Overall, the aquifer temperatures at P22 are lower than those measured at wells P21 and P23. Well P22 is located in the Central Block but at the edge of the Eastern Fault. This well is also located very close to the reinjection well P4. Clearly, warm reinjection water has not influenced the temperatures measured during this constant discharge test.

3.3.4 Temperature Profiles During Drilling and Testing

Elba2 Profiles During Drilling of Elba3

The Elba2 well was logged on four occasions, during and following drilling of the nearby 122 metre deep production well, Elba3, between April 25, 1994 and April 29, 1994 (Figure 3.9). The first three profiles, labelled (1) to (3) on Figure 3.9, were obtained during shallow bedrock drilling. The fourth profile (4) was obtained on the last day of drilling. The final two logs, labelled (5) and (6) on Figure 3.9, were acquired approximately two weeks after drilling. The most recent log for this well, prior to drilling Elba3, was shown in Figure 3.6.

Many small perturbations are recorded on the logs (Figure 3.9). The range in temperature for these plots is 8.6°C to 8.9°C. For profiles (1), (2) and (3), these perturbations intensify between approximately 50 and 90 metres depth and reflect shallow drilling. These perturbations are related to “passive” fractures (Drury, 1989), which indicate that water is either entering or exiting the borehole. Once thermal equilibrium has been attained, evidence of these passive fractures is less apparent. Below 90 metres, there is a near isothermal segment followed by additional perturbations (110 to 125 metres).

Profile (4) was obtained on the last day of drilling. Perturbations above approximately 75 metres are no longer dominant. The perturbations between 75 and 85 metres are still present, and the earlier isothermal segments below 105 metres have disappeared.

The drilling process (grinding and hammering) warms up the aquifer in the vicinity of the well. Profiles (4) to (6) show a gradual return to equilibrium temperatures.

Figure 3.9: Temperature Logs for Well Elba2 During Bedrock Drilling of Elba3.
Drilling Began on April 25, 1994 and Ended on April 29, 1994

P21 Profiles During Aquifer Testing at P23

Well P21 was logged during the 33 hour constant discharge test at well P23 and on two different occasions following the test (Figure 3.10). Profiles (1), (2) and (3) were obtained during testing and profiles (4) and (5) were obtained 3 days and 4 months after, respectively.

Profiles (1), (2) and (3) show minor perturbations between 29 and 36 metres, and between 57 and 74 metres. The profiles show frequent isothermal segments, separated by distinct shifts in temperature. These shifts correspond to water entry or exit points. The most prominent shift occurs at 70 metres, where warm water, from depth, probably exits the well. Perturbations, between 45 and 65 metres, in profiles (4) and (5), are related to passive fractures. These were masked by the strong vertical water flow within the well during testing.

3.3.5 Data Logger and Temperature Cable Installation at Elba2

In addition to this temperature logging, an RBR XL-800 series, submersible, 8-channel logger was suspended in the Elba2 well, from a well cap equipped with a hook assembly. This instrumentation was used to monitor temperatures in the well during system operation. Unfortunately, the system was probably not in operation at this time (no maintenance logs to confirm), but the maintenance staff did indicate that some of the wells on campus may have been turned on and off for unscheduled system checks.

A temperature cable with eight thermistors, placed at depths corresponding to 63, 68, 73, 78, 83, 88, 98, and 118 metres in the borehole, was connected to the base of the logger. The data logger and cable were provided by the Terrain Sciences Division of the

WELL P21 DURING CONSTANT DISCHARGE TEST AT P23

Figure 3.10: Temperature Logs for Well P21 During Constant Discharge Test at Well P23. Test Began at 11:00 a.m. Jan. 14, 1994 and Ended at 8:00 p.m. on Jan. 16, 1994

Geological Survey of Canada. The data logger was installed on November 18, 1992 at 8:14 a.m. Logging was to commence at 6:00 p.m. on the same day. The sampling period was set at 12 hours. The data logger was pulled from the borehole in May 1993 to check its operation. Unfortunately, this first attempt to gather data proved unsuccessful since the seal on the logger had failed and no data were recorded. A second data logger was installed in mid-May 1993 and was removed at the beginning of November 1993. Drilling was scheduled to commence at Elba2 shortly thereafter. The results are shown in Figure 3.11.

There are two prominent shifts in the temperature data. The first one occurs around the last week of May and lasts until the end of the first week in June. This shift occurs at all recording levels within the borehole. The sensors at 98 and 118 metres depth show the least variation, while the shallower sensors, 63, 68, 73 and 78, show the most prominent shift. The intermediate depth sensors show an intermediate response. The sensors in the well are all around $9.8^{\circ}\text{C} \pm 0.1^{\circ}\text{C}$ in late May with the shallower sensors measuring temperatures at the lower end of this range (consistent with the temperature profile).

The shift in temperature in late May and then in early June is best explained, in the absence of system operation, by a drop in water level in the well, followed by a rise in water level, respectively (temperature increases with depth in the borehole) (V. Allen, Geological Survey of Canada, Personal Communication). The fact that the bottom level sensors, at 88, 98 and 118 metres, show the least disturbance is explained by their

Figure 3.11: 8-Channel Data Logger Temperature Output for Elba2 (May 1993 to November 1993)

approximate coincidence with water entry zones. The temperature of the water remains relatively stable during this time.

The pressures measured in the shallow piezometers around the campus were also lower at this time. From May 12, 1993 to May 19, 1993, there was a significant drop in pressure. The pressure remained low at least until May 31, 1993. The next measurement was not made until August. It is speculated that the changes in water level, observed at Elba2 during this time, are related either to the filling of the canal, which had begun around May 10 and was complete around May 20 (10 days later), or to pumping at well P2 for maintenance testing. There are insufficient data to confirm either explanation. A similar, but opposite, shift in temperature is observed for late September.

Another concern is the general cooling trend exhibited by the 78 metre sensor. This depth corresponds approximately to the Sherman Fall Formation / Hull Formation boundary and to the depth where significant temperature perturbations were recorded during drilling of Elba3. The decrease in temperature at this depth may be related to operation of the groundwater system. A more straightforward explanation is a faulty thermistor connection that is resulting in near-linear drift.

Finally, a high frequency variation in the temperature was recorded at the 118 metre sensor in late October. Drilling for Phase 2 began on September 20, 1993 at the T21 site near Robertson Hall. The temperature fluctuations at 118 metres are of about 0.2°C magnitude. The oscillations begin around October 12 and continue until the logger was removed on November 3. These oscillations may be related to bedrock drilling activity, which would have alternately started and stopped at the beginning and end of each day.

From the drill records, it is difficult to identify which well(s) was responsible for the response.

3.3.6 The Thermal Regime at Carleton

The natural thermal regime of the Carleton aquifers is difficult to determine because water flow within the open boreholes effectively “erases” the natural geothermal gradient. However, the excellent thermal data do provide additional information about the hydraulic character of the aquifer, which may not have been apparent in undisturbed thermal logs (*i.e.*, those with uniform geothermal gradients). The thermal regime of the Carleton aquifer system and some of the implications of the data are summarized as follows:

- The geothermal gradient measured in the Observatory borehole (Jessop and Judge, 1971) is 14.2°C/km. Using this value for gradient, the range in temperatures over a 100 metre interval will be approximately 1.4°C. This difference agrees well with the observed range (9.4°C to 10.5°C) for wells logged under static conditions (Appendix 4). The average aquifer temperature is approximately 10°C.
- In general, the least productive wells, namely P1, P4, P3 and T24, have bow-shaped temperature profiles. This shape is obtained because water flow within the boreholes is not significant. In contrast, higher yielding wells, namely P2, P5, P6, Elba2, P22, M1 and possibly P23 and P21, have near isothermal profiles. Based on these results, it appears that the least productive wells are not connected to the higher hydraulic pressure fractures at depth. Since the high productive wells are located on or adjacent to faults, the high hydraulic pressures must be associated with the faults.

- On the basis of temperature shifts and gradient spikes, Wilson (1990) identified individual fractures, or narrow discrete zones, in which water is free to enter and exit the borehole. Many of the large fractures, identified on the density logs, do not correspond with the main water entry and exit points identified by Wilson (1990). Within densely fractured zones, which may be several metres wide, thermal anomalies are likely to be smoothed (Drury, 1989). In such an aquifer, minor variations in the gradient may be overlooked and attributed to borehole eddies. For example, Wilson (1990) attributed large sections of the thermal gradient versus depth profiles to borehole eddies when, in fact, they may be associated with minor fracture zones. The presence of passive fractures in the temperature logs acquired during drilling and testing indicate that many more fractures or “fractured zones” are present.
- The continuity of fractures from one area of the campus to the other is also debatable, and there is no evidence (no packer test results) to suggest that fractures identified from density and temperature logs at certain depths in a particular well are hydraulically connected to fractures identified in other wells. The connection between wells, as exhibited in the aquifer test data (Chapter 4), strongly suggests vertical planar fracture (fault) flow, which results from matrix aquifer and fault flow.

Hydrogeological testing within the Lac du Bonnet Batholith (reported in Drury, 1984) indicated that water was flowing from near the surface to a single, large open fracture within a borehole. However, there was no indication of such flow in the temperature logs. Hydrogeological testing in another borehole revealed a large number

of fractures within a 5 metre interval which appeared to be closed. Temperature logs within this borehole indicated strong inflow signatures in the fracture zone. Drury (1984) attributed many of these inconsistencies to the nature of the hydrogeologic testing.

3.4 Chemical Regime

3.4.1 Regional Hydrogeochemistry

Early geochemical studies on groundwaters in the Ottawa region are included in reports by Brandon (1960) and Charron (1978). More recently, hydrogeological consultants in the Ottawa area have undertaken similar studies, but have based their results primarily on former work (Geo-Analysis and J. L. Richards, 1992). Smaller-scale investigations have been conducted in the Manotick area (Sager-Kinsman, 1995) and in Gloucester, east of Ottawa (Briggs, 1995).

Geo-Analysis and J. L. Richards (1992) concluded that most parts of the Ottawa region have potable groundwater and that randomly-distributed pockets of poorer quality natural water do exist. It is suggested that these pockets are better described as being “geologically-controlled”. For example, brackish groundwater is present in the Carlsbad and, possibly, the Billings Formation aquifers in parts of Eastern Ontario (Briggs, 1995).

The mineral content of groundwater generally increases toward the eastern part of the region, as subcrop lithology changes from sandstone to carbonate to shale formations (Charron, 1978). Charron (1978) and Geo-Analysis and J. L. Richards (1992) associated areas of poorer water quality to discharge zones on the basis of regional flow directions. Regional geochemical studies (Geo-Analysis and J.L. Richards, 1992) also suggested that

groundwater geochemistry changes significantly across fault boundaries and that this indicates poor connection between fault blocks. A similar observation was made at Carleton (Wilson, 1990).

Wilson (1990) constructed a Piper diagram showing the general chemical character of regional groundwaters from various regional bedrock aquifers. The data for this diagram were derived from Brandon (1960). Waters from the Carlsbad and Billings Formations are low in calcium (Ca^{2+}) and magnesium (Mg^{2+}), but very high in sodium and potassium ($\text{Na}^+ + \text{K}^+$). Bicarbonate (HCO_3^-) and sulphate (SO_4^{2-}) are elevated compared to chloride (Cl^-). In contrast, waters from the Eastview and Ottawa Formations are high in Ca^{2+} , moderately high in Mg^{2+} and low in $\text{Na}^+ + \text{K}^+$. SO_4^{2-} is low and the major anion is typically HCO_3^- , with possibly high Cl^- . The St. Martin, Rockcliffe, Oxford and March Formations show no dominant cation and are usually more enriched in HCO_3^- . Finally, Nepean Formation groundwaters are extremely high in both Ca^{2+} and HCO_3^- .

3.4.2 Previous Hydrogeochemistry Results for Carleton Aquifers

A comprehensive analysis of the groundwater composition of the Carleton bedrock aquifers was undertaken by Wilson (1990). Wilson included both the chemical analyses for waters collected during pump testing (data after WESA, 1988) and for samples collected at discrete depths during a thorough chemical sampling program.

In 1989, Wilson sampled well water at different depths. These data characterize the groundwaters for the non-pumping system (*i.e.*, steady-state conditions). Wilson (1990) divided the waters into two categories based on differences in pH, major ion chemistry and TDS values. Type I groundwaters were characterized as having no

dominant cation, with approximately equal concentrations of HCO_3^- and Cl^- , an average electrical conductivity of $1100 \mu\text{S/cm}$, a pH of 7.8, and TDS values ranging from 500 to 924 mg/l . Water samples from wells P2, P3, P5, P6 and Elba2 (Eastern Block) were described as Type I waters. The second type water, Type II, was dominated by Na^+ and Cl^- ions, with minor concentrations of Ca^{2+} , Mg^{2+} , SO_4^{2-} and HCO_3^- , and had an average conductivity of $2268 \mu\text{S/cm}$, a pH of 8.2 and TDS values ranging from 612 to 1852 mg/l .

Ionic relationships were plotted and used to verify the source of the different constituents. For Type I waters, halite dissolution and carbonate dissolution were considered the likely dominant sources of ions (Wilson, 1990). The loss of Ca^{2+} in Type I waters was explained by calcite precipitation; high saturation indices supported this conclusion. Higher magnesium concentrations were attributed to dissolution of magnesium-enriched limestones and elevated strontium was attributed to strontianite dissolution. In well P2, higher Na^+ concentrations were measured for the water sample taken at 122 metres. Wilson (1990) explained this anomalous chemistry as resulting from cation exchange, but did not consider the source of this water.

Na^+ is the dominant cation in Type II waters. Wilson (1990) attributed this to cation exchange, which has been active for a considerable time at P1. This may be due to a longer residence time or an abundance of shale units within the aquifer. The one exception was at M1, at a depth of 122 metres. At this depth, a higher level of Ca^{2+} was attributed to increased levels of carbonate dissolution, compared to that in the other Type II waters. TDS values for well P4 were noticeably lower than those for P1 or M1. In fact, water

chemistry shows lower concentrations of Na^+ and Cl^- than measured at either P1 or M1. Well P4 falls on a line between Type I and Type II waters.

Isotopic analyses, including oxygen-18, deuterium, tritium and carbon-14, indicated that study area waters are recharged from local precipitation during climatic conditions similar to the present (Wilson, 1990). Type I waters had lower $\delta^{18}\text{O}$ and $\delta^2\text{H}$ values than Type II waters, but not as low as waters from P6. Wilson (1990) attributed the variation between Type I and Type II isotopic signatures to variations in recharge temperatures. As well, P6 waters generally plotted below the local meteoric water line. Wilson (1990) attributed this to evaporation and suggested that surface water infiltration was present at this well. Mean residence times were estimated using tritium dating. For Type II waters, no measurable tritium was detected. This indicates a mean residence time of at least 45 years (Wilson, 1990). Tritium levels in Type I waters suggested that these were recharged either before 1957 or after 1971 (Wilson, 1990).

Several observations can be made on the basis of Piper diagrams (Figure 3.12; data plotted as milliequivalents).

- In the Central Block (open squares), well P1 has the highest concentrations of Na^+ and K^+ , and also very high Cl^- . The composition of water does not vary with depth. Therefore, all data for this well plot as a single point on Figure 3.12. The composition of M1 is relatively constant with depth and all but one sample (at 122 metres) plot as one point. Na^+ and K^+ are slightly lower than at P1, but Cl^- is approximately the same. At P4, there is little variation with depth and all data plot as a single point.

Figure 3.12: Piper Diagram Showing 1989 Chemistry (Data From Wilson, 1990)

- In the Eastern Block (solid squares), there is a wider scatter in the data, particularly for the cations. At wells P2, all samples except the sample at 120 metres, plot as a single point on the cation portion of the Piper diagram. The cation composition at 120 metres falls part-way between P1 and the remaining P2 samples. In fact, all of the deep samples from the Eastern Block well plot closer to the Central Block well samples. Well P3 shows the greatest cation scatter. Samples at 109 and 150 metres have different chemical compositions from samples at 25 and 55 metres. There is little variability in the anions for all Eastern Block samples.

Observations of the groundwater chemistry lead to several key conclusions that are modifications to Wilson's (1990) original interpretations of the chemical regime at Carleton.

- In wells which have moderate to strong vertical flow, including several Eastern Block wells and M1, water chemistry from shallow intervals represents the extreme case of mixing with water from lower intervals. For example, major water entry points were identified on the temperature logs for P2 at 90 and 95 metres and just above 120 metres (Wilson, 1990). In well P2, there is little chemical variation with depth in samples above 120 metres. Below 120 metres, the chemical composition is different. At Elba2, major water entry points were identified at 84 and 112 metres (Wilson, 1990). The chemistry of the well water is dominated by that of the inflowing water at 84 metres and samples from all depths display the same ionic composition. At P5, the data are clustered, with only a slight shift towards a more Na^+ -rich composition at 120

metres (below the major water entry point at 108 m). At P6, the data are somewhat clustered. The major water entry point is at 120 m, which is considerably below the lowest sampling point at 80 m. A major water entry point is located slightly above 120 metres in well M1. Water chemistry in all of these wells (above water entry points) represents mixing of aquifer waters with inflowing water of different chemical composition. Thus, the true chemical composition of the aquifer, above the water entry points, is masked.

In wells which have lower vertical flow, such as P1, P3 and P4, mixing of water still occurs. At P1, the water entry point is below 130 metres, therefore, all samples have a similar chemical composition. At P3, minor water inflow points occur at 100, 112 and 150 metres, respectively. This explains the large scatter in chemistry observed with depth in this well. In P4, water inflow is just above 110 metres. There is only a minor variation in the 110 metre sample, therefore, it was plotted on Figure 3.12 with the other data.

- The water chemistry in each of the wells on campus represents varying degrees of mixing between Type II water and Type I water. There is a greater variability among the Eastern Block wells. Central Block wells, P4 and M1, which are located close to the edge of the Central Block, have an intermediate composition. It is suggested that wells located at the edge of the Central Fault block derive their water from both the Central Block and the Faults.
- Samples below the water inflow depths in each well plot closer to the Type II water and have high Na^+ concentrations. The samples above the water inflow points have

lower Na^+ and slightly higher Ca^{2+} and Mg^{2+} concentrations, which indicate a different source. It is suggested that the inflowing water may be fault-derived, since many of these Eastern Block wells are located along major faults. Intense fracturing would be expected along a fault zone. Therefore, wells in the vicinity of these faults, which intersect a greater proportion of fractures, will likely have a chemical composition similar to that of the fault water.

- The fault water may originate from a source region which is distinctly different in chemical composition to the chemistry of the aquifer, or the chemical processes/kinetics may be different. As well, these waters may be possibly of a younger age. In fact, the isotopic ages of the groundwaters from each block suggest that the Eastern Block water is younger than the Central Block water (Wilson, 1990). The low sodium compositions of some of these waters is further evidence in support of this hypothesis. If the source of Na^+ is cation exchange, as suggested by Wilson (1990), the low sodium water may reflect a lower residence time.
- The water chemistries for Eastern and Central Block aquifers are largely the same and this composition may be similar to the Type II water. Inflowing water, along the fault zones, is responsible for the “apparent” differences.

In support of the argument for a uniform matrix aquifer chemical composition, which is preferentially modified by fault water entering at specific intervals within the borehole, the following chemical variations under stress conditions (*i.e.*, pumping) may be examined.

- During aquifer testing, P4 had an average TDS value of 1400 mg/l, while the static average TDS value is approximately 660 mg/l (Wilson, 1990). In the static case, the upward gradient within the well has a dominant effect on the chemistry of the borehole water. During pumping, the induced gradient within the borehole draws water from all levels and the chemical composition of the water reflects that of the overall aquifer, namely that of the higher TDS water.
- Slight increases in electrical conductivity (Figures 3.7 and 3.8) with time, during aquifer tests in the Eastern Block wells, RC and Elba2, respectively, demonstrate that once the water stored in the fracture network associated with the faults is depleted by pumping, water of higher conductivity then comes from the aquifer itself.
- In the Central Block aquifer tests, the electrical conductivities remain high throughout the test (Table 3.12). If the Eastern Block wells have a different average aquifer chemistry, then under pumping conditions, the electrical conductivities for the Central Block wells should decrease at late pumping times. The fact that the chemical composition remains relatively constant, (*i.e.*, similar range in TDS) during a pump test in the Central Block, suggests that either these wells do not derive a large percentage of water from the Eastern Block, or that the chemistry of the Eastern Block is similar to the Central Block. There is a slight decline in TDS after the first 1 to 2 hours of pumping in P21 and P23 (Table 3.12a and c) which indicates that water may first come from the matrix aquifer near the well (Central Block), then from the fractures associated with the Eastern Fault (fresher water), and finally, from the matrix aquifer on the opposite side of the fault (Eastern Block).

Although the two fault blocks were considered separate by Wilson (1990), (“the geochemical regimes appear to be bounded by the study area’s major faults”), there is evidence that a continuum exists between the two blocks, and the chemical composition of the aquifers on either side of the fault(s) may not be significantly different. There may be some variation in actual aquifer chemical composition with depth; however, any variation cannot be identified due to water flow within the wells.

3.4.3 Chemical Composition of Mixed Water Under Operating Conditions

To identify any changes in chemistry likely to take place during full-scale operation of the proposed ATES system, a water sampling program was initiated during the fall of 1992. Water samples (1 litre) were collected, using a PVC grab sampler with ball and socket seals, at various depths in three of the wells sampled by Wilson (1990), and later during aquifer testing and well drilling carried out for Phase 2 development (CESI, 1994). The results of the chemical analyses (Bondar-Clegg Laboratories in Ottawa) are plotted on Piper diagrams and are provided in Appendix 5.

Figure 3.13 is a Piper diagram showing Wilson’s (1990) average well chemistry for non-pumping conditions (collected in 1989) (solid squares) and the chemistry for wells, P3, RC and Elba2, sampled after approximately two and one half years of operation (February 1990 to November 1992). During November 1992, the groundwater system was not operating. Temperature logs for P3, Elba2 and RC (Figure 3.6), which were acquired at this time, confirm that the system was not operating, since the logs are very similar to the original temperature logs reported by (Wilson, 1990). Well RC was not

Figure 3.13: Piper Diagram Showing 1989 and 1992 Chemistry (1989 Data From Wilson, 1990)

sampled by Wilson (1990); however, electrical conductivity measurements acquired during early testing of this well are similar to those of well P2. The results of the sampling program are interpreted as follows.

- Water inflow is still detectable in the temperature logs; however, the chemistry does not show the original distinct variations with depth. The new chemistry for all three wells are closely clustered on the Piper diagram (Figure 3.13).
- The chemical compositions of the waters for these three wells have changed significantly since 1989. Data are now closely clustered on the Piper diagram (Figure 3.13). The change is probably the result of operation of the groundwater system, which had been active for approximately two years when the 1992 sampling program was undertaken, although it was not active at the time of sampling.
- The new chemical composition for each of the samples obtained in these wells is very close to the original chemical composition of wells P4 and M1. Both P4 and M1 are located relatively close to fault boundaries and, as previously described, had an intermediate chemical composition in 1989. Such a composition reflects fresher fault water mixing with Type II matrix aquifer water. If the chemical composition of the Eastern Block and the Central Block are similar, then the new composition for these wells may be indicative of similar mixing.
- Prior to operation of the ATEs system, well P3 exhibited chemical variability with depth (Figure 3.12); however, the groundwater samples obtained after nearly two years of operation show little variation with depth, and plot as a single point on a Piper diagram (Figure 3.13). These results suggest that water now entering this borehole, at

all levels, is of similar composition. The implication of these results is that mixing may be taking place somewhere other than the borehole, and that water of a similar composition is entering the well at all depths.

A key point that remains uncertain is the proportion of the mixing that takes place as a direct result of pumping and reinjection beneath ground surface, and the proportion of the mixing that takes place in the physical piping network above ground surface.

The design of the Phase 1 pumping configuration had wells P1, P2 and P6 operating in one mode (pumping or reinjection), while wells P4 and P5 operated in the alternate mode (reinjection or pumping), depending on the season. During summer operation, wells P1 and P4 were designed to operate at the same rate, therefore, water circulating through wells P4 and P1 would have been self-contained and recirculation would have taken place within the Central Block. However, it has been suggested that P5 was not capable of handling the full design reinjection capacity and that overflow was diverted to P4 (N. Haitas and Associates, Personal Communication). If such a diversion occurred, then it is certain that some above ground mixing took place.

The chemical composition of water collected during the drilling of Elba3, and during the aquifer testing at P21, P22 and P23, are included in Appendix 5. All of these samples have chemical compositions similar to the bulk chemistry for the operating system.

During aquifer testing of P21, P22 and P23 and during the drilling of Elba3, well P1 was pumping and P4 was reinjecting at a rate of approximately 25 L/s. As the

chemistry for all of these wells lies very close to both the original composition of well P4, and to the newly established bulk compositions as exhibited by the Eastern Block wells sampled in Fall 1992, it is impossible to determine whether the chemical composition of the water sampled in these wells is due to ongoing pumping or whether it is a true reflection of the aquifer chemistry. Wells P21, P22 and P23 are very close to the fault and mixing within the fault zone may be taking place in the vicinity of these wells. Alternatively, reinjected water at well P4 may be making its way to each of these sampled wells, thus giving an altered composition to that present in the aquifer under static conditions.

3.4.4 Saturation Indices

Table 3.13 shows the range of calculated saturation indices at 10°C for the Carleton groundwaters sampled in 1989 and in 1992/1994. At zero, the water is in equilibrium with the mineral in question. Above zero, the water is super-saturated, and below, it is under-saturated.

The chemistry of the well water, sampled after the groundwater system had been operating for some time, has changed significantly from the original composition (Figure 3.13). The saturation indices (Table 3.13), have remained relatively consistent with some exceptions. For the Type I waters, the saturation indices for calcite, aragonite, magnesite and strontianite have dropped, while for dolomite and siderite, they appear to have gone up slightly. There was little change in the saturation index for gypsum.

Table 3.13 : Saturation Indices at 10°C for Carleton Groundwaters (1989 Data Compiled From Wilson (1990)).

	1989		1992 & 1994	
MINERAL/PHASE	TYPE I	TYPE II	TYPE I	TYPE II
CaCO ₃ Calcite	0.2 - 0.7	0.0 - 0.6	-0.1 - 0.3	0.4
CaCO ₃ Aragonite	-0.1 - 0.4	-0.3 - 0.3	-0.4 - 0.2	0.3
CaMg(CO ₃) ₂ Dolomite	0.4 - 1.3	0.1 - 1.3	0.8 - 1.7	2.0
MgCO ₃ Magnesite	-0.2 - 0.3	-0.3 - 0.4	-0.7 - (-0.2)	-0.1
SrCO ₃ Strontianite	2.6 - 3.0	2.5 - 3.1	0.1 - 0.5	0.5
FeCO ₃ Siderite	-1.4 - 0.5	-1.0 - 0.2	-0.5 - 1.3	-0.1
FeOOH Goethite	0.0 - 2.5	0.2 - 4.3	N/A	N/A
FeS Mackinawite	1.6 - 3.7	1.3 - 3.0	N/A	N/A
CaSO ₄ Gypsum	-2.0 - (-1.5)	-2.0 - (-1.4)	-2.0 - (-1.8)	-1.8

For the Type II waters, only two wells were sampled, P21 and P23. Both of these wells are located close to faults and were sampled during pump tests. Therefore, these values may not be representative of the Central Block wells. Only one value for the saturation index is reported in Table 3.13, as the two calculated values were close. Calcite, aragonite, magnesite and siderite may have increased slightly. The water is much more saturated with respect to dolomite and less saturated with respect to strontianite. There has been little change in the saturation index for gypsum.

Saturation indices for each of the wells sampled in 1992 and 1994 were calculated at 10°C and at 25°C. Although the latter is considered to be well above the temperature that the aquifer would ever attain during operation of the groundwater system, the

saturation indices for the water do not change considerably within this temperature range. In going from 10°C to 25°C, the saturation index for calcite increases by 0.07, for aragonite by 0.07, for dolomite by 0.22, for siderite by 0.04 and for strontianite by 0.02. For gypsum, the saturation index drops by 0.03. Over a range of 5°C, that expected for an operating system, the saturation indices will change marginally. The saturation index is an indication of the potential for dissolution or precipitation, but does not provide sufficient information to determine whether precipitation or dissolution will take place. Further research is required to detail the geochemical implications of system operation.

3.4.5 Hydrochemical Modelling

Geochemical modelling, using the water mixing option in *SOLMINEQ*, was undertaken to determine the proportion of mixing which has taken place as a result of operation of the groundwater system at Carleton. Two waters, typical of the Central Block (well P1 at 55 metres) and of the Eastern Block (well P6), prior to operation (*i.e.*, 1989 values), were mixed in increasing proportions. An incremental step of 10 percent was used. The Piper diagram in Figure 3.14 shows the results of mixing. The starting composition for the water is that of P6; the first point corresponds to 90% P6 water and 10% P1 water. As the proportion of Central Block water increases, there is a shift to higher Na⁺-type water, while the relative proportions of Mg²⁺ and Ca²⁺ decrease slightly. In terms of anion variations, the water becomes more enriched in Cl⁻, significantly less enriched in HCO₃⁻ and only slightly more enriched in SO₄²⁻. In general, the range over which the cations vary is greater than the range over which the anions vary.

Figure 3.14: Piper Diagram Showing Results of Chemical Mixing Between P1 and P6-Type Waters.

One concern is the cation/anion balance, that was calculated at each step during the mixing sequence. Early in the mixing sequence, the balance had an error of 5.7%; however, as mixing advanced, this error became larger, up to 11 %. It is unclear why this error became larger, but it may be significant in terms of the final mixing lines plotted on Figure 3.14. Cumulative analytical errors may provide one explanation.

Also included on Figure 3.14 (open circle) are the bulk average cation and anion percentages for the wells sampled in 1992 (Elba2, RC and P3). These points are close to the mixing line between P1 and P6. This implies that the wells sampled after the groundwater system had been in operation for a few years are tending toward a composition close to that of P4, which is located within a natural zone of mixing at the edge of two major fault blocks. The operating wells must be evolving toward this composition as a result of groundwater mixing, directly within the fault zone or outside of the fault zone, and possibly some water-rock interaction. In either situation, the resulting chemistry has not been altered significantly from that which could arise under natural mixing conditions.

3.4.6 The Chemical Regime At Carleton

The chemical compositions of well waters at Carleton are strongly influenced by mixing that results from water circulation within the open wells. Water enters the wells at depth, through fracture zones intersecting the boreholes, and moves up the wells. As this water rises up the well, it mixes with water entering from other formations and yields a fairly uniform composition with depth. As a result of this mixing, the natural aquifer chemical composition is masked.

In the Eastern Block wells, the water which enters the wells through fractures at depth is younger and fresher than the waters in the matrix aquifers. This younger water has chemical, and possibly isotopic, composition that is distinctly different than that of the Central Block. However, below the water entry points, the chemical composition of the Eastern Block waters is closer to the Central Block. This suggests that the chemical compositions for the two blocks (matrix aquifers) are similar, and that the chemical composition of the shallow waters in the open wells is a mixture of matrix aquifer water with recent recharge.

The fresh water probably enters the aquifer system via the faults; however the source area cannot be identified. The results from chemical modelling suggest that the two end members are Type II waters (Central Block) and Type I waters (Eastern Block). The isotopic composition for P6 suggests that the well may be receiving recharge directly. The temperature logs for this well also show that P6 has some downward flow (Wilson, 1990). Recharge may take place at higher elevations, west of the campus, and water may be entering the deep aquifer system at Carleton through faults that are connected to deep overburden west of the campus.

In the operating groundwater system, the chemical composition for three wells, shows similar mixing. These wells also plot along the mixing line between P1 and P6. Furthermore, the bulk composition for these wells is close to the natural composition for P4, a well which is influenced by natural mixing between fault blocks.

3.5 Summary

In general, the hydrologic, thermal and chemical regimes are influenced strongly by faulting. The faults appear to influence, and possibly control, the movement of water under steady-state conditions and transient conditions; however, the nature of this connection is still uncertain.

Early aquifer test data confirmed that there is preferential connection between certain wells and very poor connection between other wells. More data have become available since these early tests, and these require further investigation. This is the focus of the following chapter.

This information is needed to define quantitatively the hydraulic and thermal regimes, so a physically-based model for the Carleton campus can be produced. In addition, this information may provide valuable insight into the hydraulics of faulted regimes.

4. AQUIFER TEST RESULTS

4.1 Aquifer Characterization

4.1.1 General

To this point, the description of the Carleton aquifer has been primarily qualitative; focusing on static hydraulic, thermal and chemical conditions (Chapter 3) in relation to the location of major faults, defined using geological and geophysical techniques (Chapter 2). Not only do the faults physically subdivide the campus into different geological blocks, but they appear to control both the direction of water movement under steady state conditions and the spatial variability of well yield. The distinct thermal and chemical character of the aquifer(s) is also a manifestation of these faults, under static conditions.

Very little emphasis has been placed on examining the behaviour of the aquifer system under stress conditions, with the exception of reviewing the early aquifer testing. These early aquifer test data indicate that there is excellent hydraulic connection between wells located along major faults and poor hydraulic connection between wells located on different sides of a fault. However, these data are insufficient to define quantitatively the physical effect of faulting on the overall hydrogeology of the system. To determine this effect, a rigorous program of aquifer testing needs to be implemented, and the results analyzed and interpreted.

4.1.2 Aquifer Testing

An aquifer testing program generally consists of pumping one or more wells individually and measuring the response in one or more observation or monitoring wells

nearby. The static water level is recorded in the pumping well and in all monitoring wells prior to commencement of the test.

When dealing with open wells, the water level measured in the well is an average of the hydraulic heads of the various intervals over which the well is open (Michalski and Klepp, 1990). For example, consider a monitoring well which is screened over a series of aquifers in which the hydraulic conductivity of the intervals varies over a few orders of magnitude. If a nearby well, also open over the same interval, is pumped, the drawdown will be greater in the intervals with the highest hydraulic conductivity. In contrast, the intervals with lower hydraulic conductivities will experience less drawdown.

When conducting an aquifer test, it is preferable to have a well distributed set of monitoring wells or piezometers and several multilevel monitoring points, to examine both the lateral and vertical heterogeneity within the aquifer. For budgetary reasons, this is not always possible and often, only open wells are available. However, there are instances where this information is not necessarily critical to the evaluation of the aquifer. Some of these include:

- Situations in which the aquifer is known to be ideal. That is, it is homogeneous and isotropic, of infinite aerial extent (in terms of pumping influence) and of uniform thickness.
- Situations in which only water supply is of interest. Drilling records will generally provide sufficient information to identify the prime water-bearing units.

- Situations where vertical heterogeneity or anisotropy result in discrete horizons where minimal influence is anticipated. Special leakage solutions may be applied to simple cases where variable head loss has taken place.

If the aquifer is known to be laterally heterogeneous or anisotropic, wells or piezometers used as monitoring points should be located at varying distances away from the pumping well and should also be distributed to achieve a good spatial representation of the aquifer. Specific aquifer testing programs can be carried out to best extract information from the aquifer. In fact, there are particular configurations for monitoring wells which have been suggested for aquifer testing programs in situations where the aquifer is known to have anisotropy or hydrogeological barriers (reported in Kruseman and de Ridder, 1990). However, hydrogeologists often do not have control over the placement of wells to take advantage of these suggestions.

If the aquifer is vertically heterogeneous or anisotropic, as is often the case in fractured crystalline or highly stratified rocks, specific rock intervals can either be tested or monitored using a packer assembly (described in Driscoll, 1986) or multilevel installation, such as that at the M1 well at Carleton, in order to obtain vertical resolution. When this type of assembly or installation is not available, a piezometer nest (closely spaced piezometers of varying depths) may be employed. Obtaining good vertical resolution may be of particular importance for problems involving contaminant or heat transport. However, if the aquifer is very heterogeneous, there will likely be insufficient spatial data to achieve the desired resolution. In this case, some degree of careful

extrapolation or interpolation will be required in order to best represent or characterize the aquifer.

4.1.3 Analytical Methods of Analysis

Every hydrogeologic model employed for data analysis is patterned after some prototype geologic model. These models are routinely used for calculating the transmissivity and storativity for the aquifer. One such model is the typical confined aquifer of large aerial extent, which is assumed to be isotropic and homogeneous, overlain by a confining layer over which there is a water table. If the confining layer is impermeable and has a specific storage of zero, then all of the water removed from the aquifer will come from storage in the aquifer, and the drawdown, observed at any point in the aquifer, will vary with time. The drawdown versus time response, when plotted on semi-log paper (time on the log scale), is a straight line (small values of u - Appendix 6), except at early time. Theis (1935) and, later Cooper and Jacob (1946) developed procedures, based on simple analytical models, for analyzing the hydraulic responses of wells to pumping. These are discussed in Section 4.4.

A slightly more complex environment, fashioned after the basic model, is one in which the confining layer has a non-zero permeability but still contains an incompressible fluid and an incompressible matrix (*i.e.*, $S_c = 0$). Under these conditions it may be possible to invoke the transfer of water across the confining layer. The drawdown-time plot will reflect this additional water source and will show a downward deflection at some point in time. This is referred to as the leaky response (Hantush and Jacob, 1955). It should be

noted that a similar type of deflection is recorded if pumping is taking place nearby a stream or lake and surface water enters the aquifer.

Successive complexities may be added to the geologic environment and the model modified to incorporate the specific conditions present in the aquifer. For example, models exist for unconfined aquifers (Neuman, 1972), anisotropic aquifers (Hantush and Thomas, 1966), aquifers which are near barriers or lines of recharge, irregularly shaped aquifers and fractured aquifers. As well, specific types of tests and analytical procedures have been developed to deal with large diameter wells, partially penetrating wells and tests involving single boreholes (summarized by Kruseman and de Ridder, 1990). The literature contains numerous new methodologies, which have been developed according to specific aquifer conditions.

To date, there has not been much emphasis in the literature on aquifer testing in carbonate rocks. As discussed in the following sub-section, these aquifers are often very complex and, thus, the simple prototype models may not be applicable for analyzing aquifer test data.

4.1.4 Aquifer Testing in Carbonate Aquifers

The hydraulic regime of carbonate aquifers has long been the subject of research, although much of this research has been of a qualitative nature. For example, unusual hydraulic behaviour, in terms of well connection and well yield variability, has been widely documented (refer to Chapter 1, Section 1.1 on carbonate aquifers). In general, where strictly quantitative analyses are considered, the data are often few in number. Rarely do

these data satisfy the requirement for adequate spatial representation of the aquifer under a variety of stress conditions.

Aquifer testing in a carbonate aquifer is more complex than in a typical prototype aquifer for several reasons. First, the degree of secondary permeability may be spatially variable, leading to an overall heterogeneity or anisotropy. Heterogeneity and anisotropy may be related to the spatial distribution and connectiveness of fractures and faults, and possibly to the presence of layering with well-developed permeability along bedding planes. Mineral precipitation or dissolution may effectively seal or open fractures. Second, there may be some intrinsic permeability associated with the matrix material which contributes to the overall storage and transmissivity properties of the aquifer. In this situation, the aquifer may qualify as being of the double porosity type.

The complexities inherent to carbonate aquifers require that an aquifer testing program be developed to determine the spatial extent of the aquifer, the degree to which secondary permeability is present, and the extent to which this secondary permeability affects the flow regime. The aquifer testing program should include the testing of several wells, located in different portions of the aquifer. In addition, monitoring data should be collected at a number of observation points including, if possible, multilevel installations. These data should be examined and analyzed using a variety of analytical approaches, since dominant processes and geometries may be different in each well.

4.1.5 Hydrologic Models

To generate a predictive hydrologic model that can be used to describe quantitatively the relationship between faulting and groundwater flow at Carleton, it is

necessary to be able to calculate expected changes in the hydraulic head distribution that will take place anywhere in the aquifer, and at any given time under different stress conditions. The model may be either analytical or numerical. An analytical model can provide a mathematically exact solution to the problem, but, in general, it is more difficult to represent and account for complex boundary and initial conditions, and a non-uniform distribution in the aquifer hydraulic properties. For this reason, analytical models are seldom used for modelling anything but simple systems. For more complex systems, such as aquifers with multiple faults, a numerical model is often the best technique for approximating the actual hydrologic conditions because numerical techniques allow for greater flexibility in representing non-uniform systems. Nevertheless, numerical techniques are still only as accurate as the conditions and properties that control them, and as the uncertainty in these parameters increases, so does the integrity of the model. Of course, no model can precisely mimic the behaviour of a natural system. Slight variations in the geology, for example, can contribute to heterogeneities on many scales. Ultimately, the model is only as accurate as the data supplied to it.

In developing any model of an aquifer, both the transmissivity (T) and storativity (S) must be defined at each point in the active aquifer. For steady state models, only T is required since there is no change in storage in the aquifer, whereas, under transient conditions, both T and S are required. If the aquifer is of uniform vertical composition and thickness, a 2-dimensional model can be applied quite adequately. In this case, the transmissivity and storativity are directly used since they take into account the aquifer thickness. Alternatively, if the aquifer is vertically heterogeneous, the transmissivity and

storativity values will vary with depth. In this more complex geological environment, a 3-dimensional model may be required, and hydraulic conductivity and specific storage must be estimated from values of transmissivity and storativity. The former are obtained by dividing T and S by the saturated thickness of the aquifer.

The aquifer test results for Carleton University present a unique source of data for a quantitative assessment of a carbonate aquifer regime. The primary focus of the remainder of this chapter is to identify and quantify the hydraulic parameters from data obtained during aquifer testing in the carbonate aquifer beneath Carleton. The results from aquifer testing are used to develop a conceptual model for the flow of water in the complex carbonate terrane at Carleton. This hydraulic information is necessary to quantify heat transport in the aquifer and, ultimately, to evaluate the aquifer thermal energy storage (ATES) system at Carleton.

4.2 Aquifer Testing at Carleton University

4.2.1 Phase 1 Testing Program

An aquifer testing program was conducted in conjunction with Phase 1 development of the Carleton Groundwater Project during the period between May 23 and September 25, 1988. Earlier aquifer testing of RC, Elba1 and Elba2 was discussed in the previous chapter. The supervision of the Phase 1 testing program, the collection of aquifer test data, and the analysis and interpretation of these data were carried out by Water and Earth Science Associates (WESA). All of the aquifer test data were re-analyzed during this study because only one analytical method of analysis was employed by WESA (1988). A short description of the aquifer tests for each well is provided in Appendix 6 (Tables

A6.1a and A6.1b), and the results of the analysis are discussed in Sections 4.3 through 4.8.

A total of five step discharge tests, six constant discharge tests, six recovery tests, four step reinjection tests and five constant reinjection tests were conducted as part of Phase 1. The wells included, P1, P2, P4, P5 and P6. Water intake was set at a depth of 61 metres (200 ft), except during the last reinjection test (P2 to P1), when the pump intake was set at 33.5 metres (110 ft). Water pumped during the non-reinjection aquifer tests was discharged into storm sewers. Water discharge rates were measured at surface with an orifice weir and manometer and cross-referenced with a pitot tube. Water levels in wells were measured with a standard electric contact tape and all measurements were recorded from the top of the casing. For each of the constant discharge and recovery tests, water levels were recorded in the production well itself and in all accessible observation wells.

Step discharge tests were conducted on each of the wells. Water yields were increased until the maximum possible discharge was attained. Steps were on the order of one hour each. These data were used to calculate the short term yield for the well and to determine the optimum pumping rate for the constant discharge test (WESA, 1988).

Step discharge tests were normally followed by a constant discharge test and a recovery test at each well. All of the tests were carried out shortly after drilling of each well. Fewer observation wells were available for the wells drilled earlier in the program. The constant discharge tests were run for 24 hours at all new production wells, except at P2, where a 72 hour constant discharge test was run.

The balance of the Phase 1 testing program included a series of reinjection tests. The purpose of these tests was to examine for reinjection efficiency as well as to establish the degree of well connection. In total, four step discharge reinjection tests and five constant reinjection tests were carried out during Phase 1.

4.2.2 Phase 2 Testing Program

A second phase of aquifer testing was carried out during December 1993 and January 1994. The supervision of this aquifer testing program, the data collection, analysis and interpretation were conducted by Carleton Energy Services Inc. (CESI). The author of this thesis was sub-contracted by CESI to participate in the aquifer testing program, to analyze these data and to provide an interpretation in the form of a final report (CESI, 1994). A short description of each aquifer test conducted during Phase 2 is included in Table A6.1a, while the data are analyzed and the results discussed in Sections 4.3 through 4.8.

Two step discharge tests, two 48 hour constant discharge tests, one 33 hour constant discharge test, and three recovery tests were run during this time. The wells tested included P21, P22 and P23. Water intake depths for the aquifer tests were set at 60 metres. Discharge rates were measured at ground surface using a manometer fitted to the discharge piping, which contained a 7 inch orifice plate and a valve control. All water was discharged to storm sewers.

Drawdown and recovery data were obtained with a standard electric tape and all measurements were recorded from the top of the well casing. In addition to the new production wells (P21, P22, and P23), the existing production well P6 and the observation

wells RC (located adjacent to P2), TORY, Elba1, Elba2, P3, T24 and M1 (when possible), were monitored. Of the existing Phase 1 production wells, P1 and P4 were in production and, therefore, could not be monitored. Wells P5 and P2 could not be monitored due to the absence of access ports. The lack of an access tube for monitoring created measurement problems in the pumping wells due to turbulence in the borehole and, thus, poorer than expected results (CESI, 1994).

Step discharge tests were conducted on two of the three production wells, P21 and P22. Water yields were increased in steps and the drawdown monitored in both the pumping well, where possible, and in selected observation wells on campus. The duration of a step was one hour and the number of steps was site specific. Although the results of the step tests were poor, the response at P22 and P21 during testing, and the observed drawdowns at selected observation wells, formed the basis for selecting the optimum discharge rate for the constant discharge tests of P21, P22 and P23. Time constraints and cold weather required CESI to cancel the preliminary step test procedure at well P23.

Step discharge tests were followed by a 48 hour constant discharge test (33 at well P23) and a 24 hour recovery test at each well. Discharge and recovery data were collected for all accessible production and observation wells.

Reinjection testing was not carried out during this development phase. For a reinjection couplet to work efficiently, the two wells must be located in the same fault block. Either P21 and P23, or P22 and Elba2, were considered possibilities; however, their physical separation made this logistically impractical. P21 and P22 could not act as a

reinjection couplet, neither could Elba2 and P21, because they are located in separate fault blocks.

4.3 Step Discharge Test Analysis

The drawdown in a well is composed of two parts and the sum of these parts is equal to the total observed drawdown in a well (Equation 4.1a). The first part is due to laminar flow of water in the aquifer toward the well and is referred to as the formation loss, s_f . The second part of the drawdown, the well loss (s_w), results from turbulent (non-linear) flow. The well loss is equivalent to the product of the well loss constant, C , and the square of the pumping rate, Q^2 (Equation 4.1b from Domenico and Schwartz (1990):

$$s = BQ + CQ^2 \quad 4.1a$$

$$s_w = CQ^2 \quad 4.1b$$

In Equation 4.1a, B is a constant that represents the theoretical linear aquifer losses and linear well losses resulting from, for example, skin effects. Thus, as the pumping rate increases, the proportion of the total drawdown due to well loss increases.

A step discharge test is one in which the well is pumped during successive periods at pumping rates which are below the full capacity of the well. To analyze the results of these tests, the drawdown (difference between the water level at a given time and the initial static water level) is plotted against total test time on semi-logarithmic paper (time is plotted on the logarithmic scale). This was done for each of the wells tested at Carleton University and the graphs are provided in Appendix 6. Step discharge data at well P5 are shown in Figure 4.1.

STEP DISCHARGE TEST AT P5

Figure 4.1: Step Discharge Test Data for Well P5 (Data From WESA, 1988)

In general, all of the graphs display the characteristic curve shapes typical of step test data, with the exception of well P22. This well was step tested in two parts since the motor ceased to function part way through the test. Although shutdowns do not constitute ideal conditions in which to conduct a step test, the test was continued at the higher rates for two successive steps. In addition, the water level tape was short-circuiting due to turbulence, while measurements were being made in the pumping well (CESI, 1994). This, as opposed to the shutdown, is probably the main source of error in these measurements.

The specific capacity, S.C., is defined as the ratio of the pumping rate for a particular step to the incremental drawdown for that step (Domenico and Schwartz, 1990). The specific capacity, for each of the steps, was calculated by extrapolating the curve through the plotted data of each step to the end of the next step. The increments of drawdown, $\Delta s_{w(i)}$, were then determined by taking the difference between the observed drawdown at a fixed time interval, taken from the beginning of each step, and the corresponding drawdown on the extrapolated curve of the following step.

The specific capacity is a measure of the productivity of the well and the aquifer. In general, the specific capacity decreases as pumping continues and as the pumping rate increases. This is due, primarily, to well losses associated with such factors as imperfections in well construction, but may also be due to natural irregularities in the aquifer. In general, the specific capacity is considered a measure of the efficiency of the well. For a 100% efficient well, the non-linear well loss is equal to zero, $s_w=0$.

Table 4.1 shows the calculated specific capacities for each well and each step for all of the production wells, with the exception of well P23, where no step drawdown test was carried out. The time interval used was 60 minutes.

Table 4.1: Specific Capacities for Production Wells Calculated for Each Step Interval

WELL	STEP	PUMPING RATE (L/s)	SPECIFIC CAPACITY L/s/m	WELL	STEP	PUMPING RATE L/s	SPECIFIC CAPACITY L/s/m
P1	1	15	1.4	P5	1	17	8.5
	2	26.5	2.2		2	31	12.4
	3	38	1.9		3	46	15.3
P2	1	15.1	10.1		4	61	15.3
	2	30.3	17.8		5	75.6	11.6
	3	45.5	28.4	P6	1	45.5	15.2
	4	60.6	75.8		2	64.7	32.4
	5	75.8	75.8		3	75.6	25.2
	6	90.9	27.5		4	82.2	74.7
P4	1	16.9	N/A	P21	1	37.9	3.79
	2	25.8	N/A		2	45	10.0
	3	32.3	8.1		3	52.5	21.0
	4	37.7	9.4	P22-TEST 1			N/A
	5	44.6	4.9	P22-TEST 2			N/A
	6	53.4	N/A				

The specific capacity may decrease from one time step to the next because it is a function of time, therefore, it is improper to calculate an average specific capacity for a well. Rather, the specific capacities may be used to determine the optimum pumping rates for a given well or to compare different wells. For example, wells P1 and P4 have much lower specific capacities than wells P2 and P6 (Table 4.1). In terms of optimum yield, well P4 displays a decrease in efficiency above 37.7 L/s. In contrast, well P2 could be pumped at rates less than approximately 75.8 L/s with no loss in efficiency.

The calculated specific capacities (Table 4.1) do not decrease with time, nor do they remain constant. At well P6, the high specific capacity calculated for step 4 may be associated with a poor control for extrapolation of the curve, since the specific capacity for step 3 had already dropped relative to that for step 2. Some wells also show a lower specific capacity at low yields which increases as the pumping rate increases. The analytical model assumes that the aquifer is ideal and does not account for heterogeneities within the aquifer, such as faulting.

The specific capacities were used to determine an optimum pumping rate for each constant discharge test, which followed the step discharge test. The pumping rates for the constant discharge tests are included in Appendix 6 (Table A6.1a).

The second measure of well performance, the well loss constant, C , is unique to each well. From the semi-logarithmic graphs of drawdown versus time, the cumulative drawdown $\Delta s_{w(n)}$ was calculated for each step at each pumping rate, Q_n , from $\Delta s_{w(n)} = \Delta s_{w(1)} + \Delta s_{w(2)} + \dots + \Delta s_{w(n)}$ (Appendix 6). The values of $\Delta s_{w(n)}/Q_{(n)}$ were plotted versus the pumping rate, $Q_{(n)}$ on an arithmetic scale. The relationship should be linear, and the slope of the line is equivalent to the well loss constant, C . The regression analyses (Appendix 6) were used to calculate well loss constants for each of the wells (Table 4.2).

Table 4.2: Well Loss Constants

WELL	WELL LOSS CONSTANT $C \text{ (sec}^2 \text{ m}^{-5}\text{)}$
P1	1.8E+4
P2	-3.9E+1
P4	2.1E+4
P5	1.9E+3
P6	1.3E+3
P21	4.1E+3

The higher the value of the well loss constant, the larger the amount of drawdown resulting from well loss (Equation 4.1). Well P2 has the lowest value (near zero). Wells P21, P5 and P6 have slightly higher values, which means that these wells are more efficient than both wells P1 and P4.

4.4 Conventional Methods of Analysis for Constant Discharge Tests

There is no prototype which is comparable to the aquifer regime at Carleton, therefore, a simplified model had to be employed to calculate a first estimate of the aquifer hydraulic parameters. This model was also used as a comparative tool in order to characterize the unusual behaviour of the Carleton aquifer, relative to ideal aquifers. Although the aquifer system at Carleton does not, in general, meet with the assumptions inherent to standard analytical methods of analysis, the data, or at least a portion of the data, appear to follow a Theis type response. The assumptions and theory for the Theis and Jacob methods are discussed in Appendix 6.

4.4.1 Theis - Methodology and Results

The method of solution for Theis consists of plotting the drawdown versus time on logarithmic paper for each of the observation wells and the pumping well. The Theis type curve (a graph of $W(u)$ versus $1/u$) is plotted on a separate logarithmic graph of the same scale. The field curve is superimposed on the type curve, with the coordinate axes of the two curves kept parallel while matching field data to the type curve. Any point on the overlapping sheets is arbitrarily selected. This point is defined by four coordinate values: $W(u)$, $1/u$, s and t . Equation A6.1 (Appendix 6) may be solved for transmissivity by

reading off the coordinate value for s and $W(u)$. Equation A6.2 (Appendix 6) may then be solved for storativity by reading off the coordinate values for t and $1/u$ and the value for transmissivity already determined from Equation A6.1.

This type of analysis is often very subjective, particularly when the match is not ideal. Therefore, the Theis curve-matching method is seldom used in data analysis (*i.e.*, to calculate T and S). A favoured approach is to plot some of the data on logarithmic graphs and to observe whether the data do follow the Theis curve. If there is a good match then a faster, less subjective method of analysis, namely the Jacob Method (discussed in Appendix 6 and Section 4.4.3) can be used. The Jacob method linearizes the well function at large pumping times or small pumping well-observation well distances (Appendix 6).

All of the Carleton aquifer test data for constant discharge tests were plotted on logarithmic graphs. These are provided in Appendix 6. The data, where available for the multilevel piezometer, M1, are included on a separate graph. Figure 4.2 shows the results for the constant discharge test at well P1. Included in the plot are the data for the pumping well, P1, and two observation wells, Elba2 and Tory. Well P1 is located in the Central Fault Block (Figure 4.3); the Tory well is also located in the Central Fault Block, but may be partially separated from P1 by a fault (the Western Fault). The Elba2 well is located in the Eastern Block (Figure 4.3). This well is separated from well P1 by two faults (the Eastern Fault and the Central Fault). Not all of the data have been plotted on this graph. Missing are the data for Elba1, RC and P2 (all Eastern Block wells). All of these data coincide approximately to the curve for Elba2 (refer to graph in Appendix 6).

Figure 4.2 also shows the matched Theis curves. For the pumping well, the match is good. Very early time data display slightly less drawdown than that predicted by Theis.

CONSTANT DISCHARGE TEST WELL P1 - THEIS CURVE MATCH

Figure 4.2: Graph Showing Theis Curve Matching to Constant Discharge Test Data at Well P1 (Data From WESA, 1988)

Figure 4.3 : Location of Wells and Major Faults at Carleton University

This may be due to wellbore storage and will be discussed further in the following section. The match for the Elba2 well and, thus, the other Eastern Block wells, is also good. The results for the Tory well also show a fairly good match.

In terms of response time and total drawdown, the results are unusual. The Tory well is located approximately 282 metres from the pumping well and the delay time and drawdown for the Tory well are in accordance with this distance. The Elba2 well is located 412 metres away from P1. The distance is greater and, as expected, the response time is greater and the drawdown less. However, the P2 and RC data, not plotted on this graph, coincide approximately with the Elba2 data curve. Well P2 is located 247 metres away from P1, and RC is located 253 metres away from P1. Both of these wells are closer to the pumping well than the Tory well, yet their drawdown is less and the response time is greater. This behaviour will be examined in greater detail in Section 4.4.3.

The second data set that is discussed is for the constant discharge test at well P2. These data are shown in Figure 4.4. Data for the same wells as those in Figure 4.2 have been plotted. Missing are the pumping well, P2, whose curve plots very slightly above Elba2; the Eastern Block wells, Elba1, P3, RC, P5 and P6, which all plot near the Elba2 data curve; and the other Central Block well, P4, which plots alongside the P1 data (refer to graph in Appendix 6). A departure from the Theis curve, at early time, is apparent in all except the Tory well (discussed later).

Wells RC, P5 and Tory (Appendix 6) begin to respond to pumping at P1 and exhibit drawdown according to their respective distances (35, 176 and 524 metres) from the pumping well. Wells P1 and P4 (Appendix 6) are both closer to the pumping well than

CONSTANT DISCHARGE TEST WELL P2 - THEIS CURVE MATCH

Figure 4.4: Graph Showing Theis Curve Matching to Constant Discharge Test Data at Well P2 (Data From WESA, 1988)

most of the other Eastern Block wells, yet their response time is later and their total drawdown is significantly less. A similar cross-block delayed response was previously observed, during the P1 constant discharge data.

All of the aquifer test data display an unusual response behaviour. There is no clear pattern relating to the response time or total drawdown to radial distance from the pumping well in this case. When one of the Central Block wells is pumped (P1 or P4), the other Central Block wells respond according to their distance away from the pumping well (radial flow), and the data for the Eastern Block wells are clustered. When one of the Eastern Block wells is pumped, the response time and the total drawdown is similar in all other Eastern Block wells and generally varies according to their respective distances from the pumping well (*i.e.*, the further the observation well, the slightly longer the response time and the less the drawdown). During these tests, the Central Block wells are delayed and show less drawdown, regardless of their distance to the pumping well. The Tory well invariably has the greatest delay and the smallest drawdown. When one of either P21 or P23 is pumped, the same general response pattern is observed, with the exception that the alternate well (*i.e.*, P23 when P21 is pumped) responds immediately and with a similar drawdown.

Data for the M1 multilevel and, during Phase 2 testing, the M1 well are not as clear. The response of this well is highly variable and a complete section (4.4.4) is dedicated to examining the data from this installation.

4.4.2 Wellbore Storage

The purpose of this section is not to present the theoretical background for analyzing wellbore storage, but to look at the possibility that wellbore storage may be the cause of the early departures from the Theis curve noted in the previous section. All of the production wells, P1, P2, P4, P5, P6, P21, P22 and P23, which have been tested at Carleton, are 12 inch (30.48 cm) diameter wells. The Elba2 well is a 10 inch (25.40 cm) diameter well. The monitoring well, M1, was a multilevel installation during Phase 1 testing, but was drilled out during Phases 2 and 3 development to a diameter of 15 inch (38.10 cm). The other monitoring wells, RC, Elba1, Tory, T24, T1 and T3, are all 6 inch (15.24 cm) diameter wells, and well P3 is a 12 inch diameter well.

The standard methods of analysis, including Theis, assume that the diameter of the well is negligible and, thus, wellbore storage is not a factor. For a well in which wellbore storage is not negligible, the early time data will plot as a straight line of unit slope on a log-log plot of drawdown versus time (*i.e.*, a 45 degree angle if the log intervals are the same for the x- and y-coordinate axes). In relation to Theis, the early data plot on a curve which is less steep than the Theis curve, since water is derived from the wellbore as opposed to from the formation.

Wellbore storage may account for some of the very early time departure (less than 5 minutes) in the pumping well data. This is particularly evident in the graphs for constant discharge at wells P4 and P6 (Appendix 6). When any Eastern Block well is tested, the early slopes for the other Eastern Block wells are less steep than Theis; however, in none of these are the slopes close to unit value. In general, the slope is between 0.5 and 0.75.

The Central Block wells, in these tests, all tend to follow Theis. When one of the Central Block wells is pumped, both the Eastern Block wells and the remaining Central Block wells are in close agreement with Theis. Only during the P21 pump test does P23 have a steeper slope. Therefore, it can be concluded that the early time departures, exhibited by some of the test results, are not related to wellbore storage, except perhaps in the very early time (less than 5 minutes).

4.4.3 Jacob - Methodology and Results

In an ideal aquifer, data corresponding to small values of u (*i.e.*, $u \ll 0.01$) will fall along a straight line in a semi-log plot of drawdown versus time, and a linear regression can be used effectively to calculate transmissivities and storativities (Jacob method, Appendix 6). In real aquifers, very early time data, particularly at the pumping well, may be dominated by wellbore storage effects and late time data may show the onset of steady state conditions. Wellbore storage and boundary effects are not accounted for in the Jacob method.

Since the Jacob method of analysis is straightforward, it is the preferred method used by the hydrogeological consulting industry for calculating aquifer parameters. WESA (1985, 1986, and 1988) used the Jacob method to analyze all of the aquifer test data acquired prior to Phase 1 and during Phase 1 development. CESI (1994) also used this method of analysis to remain consistent with WESA; however, departures from ideal responses were taken into account. Alternative methods of analysis were examined by CESI (1994). All of the aquifer test data were plotted on semi-logarithmic paper and these graphs are provided in Appendix 6, following the log-log plots.

Figure 4.5 shows the semi-log plot of some of the constant discharge data for well P1. (data for the remaining wells are included in Appendix 6). Wellbore storage is clearly present at the pumping well for $t \leq 1$ minute. After this time, the graph is close to linear, until approximately 200 minutes when it begins to level off (this was not as clear on the log-log graph). This levelling off is seen in most of the data and is discussed in Section 4.5.4. There are a few minor undulations in the straight line portion which, to make mention of briefly, coincide with the times when the Tory well (10 minutes) and Eastern Block wells (200 minutes) begin to respond to pumping.

Constant discharge data for P2, plotted on a semi-log scale (Figure 4.6), provide new insight into the unusual behaviour that was noted in the early responses in the Eastern Block wells. Under purely radial flow conditions, the late drawdown data plot as a straight line on a semi-log scale. The constant discharge data for all of the Eastern Block wells, including both the pumping well and the observation wells, do not fall on a straight line for small values of u . In fact, most of these data plot on broad curves. Radial flow does take place, but is delayed. This particular type of response is uncharacteristic of Theis, which suggests that although a period of radial flow does occur, there must be some other factor which is causing not only the unusual response time behaviour between wells in different blocks, but also the unusual non-linear portion of the semi-log graphs.

Despite the irregularities noted above, it is critical that some quantitative estimates of the aquifer hydraulic parameters be made in order to be able to quantify the type of response that would be expected under different stress conditions (*i.e.*, pumping and reinjection). To this end, transmissivity and storativity values for production wells P1, P2,

**CONSTANT DISCHARGE TEST-JACOB METHOD
WELL P1**

Figure 4.5: Graph Showing Constant Discharge Test Data at Well P1
Jacob Straight line method. (Data From WESA, 1988)

CONSTANT DISCHARGE TEST- JACOB METHOD WELL P2

Figure 4.6: Graph Showing Constant Discharge Test Data at Well P2
Jacob Straight line method. (Data From WESA, 1988)

P4, P5, P6 P21, P22, P23 and Elba2 were calculated using a regression analysis that incorporated only the linear portions of the data curves. There were several major inconsistencies between the T and S values determined by WESA (1988) for the Phase 1 wells and those recalculated as part of this research. This was due primarily to WESA's choice of the portion of the curve used in the regression analysis. WESA used a non-varying, pre-determined time interval for their regression analyses. The time interval was generally between 100 and 1000 minutes. Because of the delayed response times, the non-linear responses at small values of u in the Eastern Block wells, and the late time slope changes, the linear portion of these graphs was often not within the 100 to 1000 minute time interval. In the current re-analysis, specific emphasis was placed on selecting only the linear portion of the data curves, which is consistent, of course, with the methodology inherent to Jacob.

Table 4.3 summarizes the transmissivities for all of the available wells calculated from the data for each of the nine major constant discharge tests at the production wells. Table 4.4 provides the storativities calculated using these same data.

Several values are missing from the tables.

- Well drilling during Phase 1 began at P1 and ended at P6. Aquifer testing was conducted at P1, for example, before wells P4, P5 and P6 were drilled. Two tests were conducted at P2 and the second test was after all Phase 1 wells had been drilled.
- Some of the Phase 1 wells had no access ports and, therefore, could not be monitored during Phase 2 testing. As well, the M1 well had been partially re-drilled so that no multilevel data were acquired.

Table 4.3: Transmissivity Values Calculated Using Jacob Method

		TRANSMISSIVITY (M ² /DAY)									
PUMPING WELL:	OBSERVATION WELL:	CENTRAL BLOCK				EASTERN BLOCK					
		P1	P4	P21	P23	P2	P5	P6	ELBA2	P22	
P1		62	63			249	311	663			
P4			56			306	266	236	264		
T1			57				265				
P21				56	56					292	
P23				52						290	
TORY		137	132	140	130	290	344	292	302	382	
T3						659		456			
T24				252	231					659	
M1 WELL					211					1336	
M1/PVC		906									
M1/B		427	557			536	1178	1137			
M1/1		325	449			1152	1190	1059			
M1/2		67	63			188	392	668			
M1/3		56	98			373	374	339			
M1/4		46	87			303	301	285			
M1/5		57	61			275	216	236			
M1/6		280	270			574	533	492			
P2		388	286			173	173	157			
RC		396	288	314	350	176	173	173	166	214	
P5						188	167	152			
P6				340	375	203		151	163	248	
P3			270	319	338		171	168	188	210	
ELBA1		403	272	310	341	179	177	166	187	215	
ELBA2		383	265	304	333	172	172	152	158	208	
P22				309	329						

Table 4.4: Storativity Values Calculated Using Jacob Method

PUMPING WELL:	STORATIVITY								
	CENTRAL BLOCK			EASTERN BLOCK					
	P1	P4	P21	P23	P2	P5	P6	ELBA2	P22
OBSERVATION WELL:									
P1	0.056	4.2E-05			0.00064	0.00054	2.3E-06	0.00018	
P4		0.749			0.00029	0.00023	0.00015		
T1		0.0017				0.00023			0.00055
P21			8.2	2.4E-06					0.00044
P23			2.5E-06						0.00057
TORY	5.7E-05	5.6E-05	6.9E-05	7E-05	0.00025	0.00025	0.00013	0.00034	
T3					0.00061		0.00034		
T24			7.6E-05	0.0011					0.00112
M1 WELL				0.00163					0.01198
M1/PVC	0.071								
M1/B	0.0034	0.00145			0.0287	0.0067	0.0019		
M1/1	0.0011	0.0004			0.037	0.0048	0.0011		
M1/2	0.0064	0.00022			0.0024	0.0056	0.00245		
M1/3	0.0018	2.3E-05			0.00097	0.0011	0.00035		
M1/4	0.00039	3.7E-05			0.00048	0.00092	0.00027		
M1/5	0.0005	0.0003			0.00047	0.00267	0.0012		
M1/6	0.0087	0.00042			0.00184	0.00195	0.00063		
P2	0.001	0.00041			163.4	0.00019	8.4E-05		
RC	0.00093	0.00035	0.00022	0.00175	0.00417	0.00028	6.7E-05	4E-05	6.6E-05
P5					0.00016	4.98	0.00015		
P6			0.00014	0.00054	0.00013		45.3	4E-05	3E-05
P3		0.0036	0.001	0.00103		4E-05	1.7E-05	1.5E-05	0.00077
ELBA1	0.00035	0.00053	0.00312	0.00032	2.8E-05	2.1E-05	1.5E-05	0.00021	0.00012
ELBA2	0.00037	0.00058	0.0071	0.00032	2.7E-05	1.8E-05	1.9E-05	53.4	0.00011
P22			0.00058	0.00065					

- An attempt was made during pumping to monitor water levels in the discharge well; however, in general, these attempts were unsuccessful. Data for the pumping well are critical for quantifying the hydraulic response accurately, therefore, monitoring the pumping well during recovery became a priority. These data are presented in a later section.

In addition to missing data, there were other problems associated with each of the Phase 1 and Phase 2 testing programs, that could not be corrected due to University requirements.

- Some of the wells were flowing at the beginning of a test. With the exception of Elba2, where a header pipe was used to obtain a static water level, all other flowing wells were assigned water levels of 0.0 metres rather than negative values. This results in a slight decrease in the calculated drawdown in the well. Therefore, transmissivities may be somewhat higher than actual values. This slight discrepancy (less than 1 metre) will not alter the values enough to cause appreciable variability in the results.
- Immediately prior to the step test conducted at P22, P6 was pumping and the water was being reinjected into P5. The system had clearly not returned to equilibrium in the intervening hour between shutdown of P6 and the start of testing. In fact, water levels were slowly dropping at the beginning of the test. This has probably reduced the accuracy of the step test results at P22.
- During the Phase 2 aquifer testing program, wells P1 and P4 continued to operate. Well P1 was pumping and reinjecting water into P4. Prior to the step test at P22, and during the constant discharge test, the piezometer, T1, located adjacent to P4, was

opened and found to be under high pressure. The operation of the system will probably have had an affect on the aquifer test results; however, it is difficult to estimate how significant this effect is.

The calculated values for transmissivity range from 46 m²/day to 1190 m²/day (Table 4.3). This range of values demonstrate effectively the variability in the well responses at Carleton. Transmissivity values can be used, in general, to gauge the productivity of an aquifer. An aquifer with a high transmissivity is able to bring in water from remote portions of the aquifer. In contrast, an aquifer with a low transmissivity results in deep drawdown cones surrounding pumping wells, since all of the water is derived from the aquifer in the direct vicinity of the pumping well. Had only well P2 been pumped and only observation well P1 been monitored, it may have been concluded that the aquifer around P1 was slightly more transmissive ($T=249\text{m}^2/\text{day}$) than the aquifer around well P2 ($T=173\text{m}^2/\text{day}$). An aquifer type other than that which adheres to the assumptions for Theis may not even be considered, particularly since a Theis-type response (*i.e.*, a straight line semi-log graph) is obtained for well P1.

Consider the alternate situation, in which P1 is pumped and P2 is the monitoring well. In this case, the transmissivity calculated at P1 is only 62 m²/day, whereas at P2 it is 388 m²/day. When an aquifer is modelled, a transmissivity and storativity (or hydraulic conductivity and specific storage) must be defined for each portion of the aquifer. On the basis of these aquifer tests results, it is difficult to determine which transmissivity value is valid for each of wells P1 and P2.

Storativity values reported in Table 4.4 include the values for the pumping wells, which have been calculated using r equal to the well radius. Storativity values for observation wells range from approximately $2.5\text{E-}6$ to $1.0\text{E-}3$. Wells located at large distances to the pumping wells may have very low calculated storativities, as S is calculated from the intercept of the drawdown curve with the x-axis, the transmissivity, and the square of the distance from the pumping well. Normally, the drawdown is less and the response time is greater the further away from the pumping well. However, at Carleton, the same amount of drawdown and nearly identical response times are observed in wells that are located along faults. Ultimately, these particular conditions determine the range of storativity values calculated. It is the author's contention that the storativity values do not, in fact, represent the true aquifer storativity. The values are truly fitting parameters, which simulate the effect of the faults by taking into consideration the response time, the transmissivity and the distance from the pumping well. To assess what the true storativity of the aquifer might be, it is necessary to remove both the distance dependence and any effect which may be directly linked to delays associated with faulting.

4.4.4 The Multilevel Installation M1

The multilevel well, M1, was monitored during each of the constant discharge tests conducted during Phase 1. Both semi-log and log-log plots of drawdown versus time are included in Appendix 6.

Well M1 is located in the Central Block, near well P1. Depths where major increases in water yields were noted in the drill logs include 106.7m and 112.8m. These correspond to the M1-2 level piezometer (Figure 4.7), which samples over the interval

CONSTANT DISCHARGE TEST AT WELL P1 OBSERVATION WELL M1

Figure 4.8: Data for Multilevel Installation M1 During Constant Discharge Test at Well P1 (Data From WESA, 1988).

102 to 118 metres and intersects the base of the Rockcliffe Formation. During the constant discharge test at P1 (pumping rate 32.2 L/s), all of the various levels at M1 responded (Figure 4.8).

The greatest response was observed in the M1-3 to M1-5 interval (from 27m to 98m inclusive), which corresponds to the upper Rockcliffe to lower Lowville Formations. Of these levels, M1-4 and M1-5 were quick to respond at less than 10 minutes. M1-3 also responded around this time but the rate of change of drawdown was much lower for the same period of time. The change in drawdown in M1-4 and M1-5 began to level off at approximately 100 minutes, giving M1-3 a chance to catch up. At the end of the test, the drawdown in these three piezometers was of the same magnitude (approximately 10 metres).

In contrast, the total drawdown measured in M1-6, M1-1, M1-B and M1-PVC was less than 4m, with M1-PVC being the lowest at less than 1m. The M1-6 level piezometer is within the Lowville Formation and the three bottom ones are within the Oxford Formation. Response times also vary with M1-6 at slightly greater than 10 minutes, M1-1 at 10 minutes, M1-B at 10 minutes and M1-PVC at slightly greater than 100 minutes. Drawdown in the M1-2 piezometer fell somewhat between those of the other piezometers, approximately 7m, and the response time was over 30 minutes.

When the other Central Block well, P4, was pumped, M1 responded in much the same way, but with different responding levels (refer to Appendix 6). In this case, M1-3 and M1-4 were quick to respond and had final drawdowns of approximately 10 metres. Also, there was a much more distinct change in slope at 100 minutes. M1-2 and M1-5

were delayed, but ended at similar drawdowns. The response times and drawdowns at M1-6, M1-1 and M1-B were similar to those measured during the P1 constant discharge test (less than 4 metres).

During the constant discharge tests conducted in the Eastern Block wells, the overall drawdown was significantly less in all intervals within M1. The M1-6, M1-1 and M1-B intervals all tended to have lower final drawdowns. The connectiveness of the other intervals was again variable.

The transmissivities and storativities calculated at M1, using the Jacob method of analysis for each of the Phase 1 constant discharge tests, are provided in Tables 4.3 and 4.4, respectively. Since radial distance to the pumping well remains constant at M1 during each test, these data can be used as indicators for determining the relationship between transmissivity and well response. A close examination of the graphs and the data suggests that higher values of T are associated with low levels of drawdown and low values of T with greater drawdown.

Storativities are less obvious since these depend on transmissivity as well as response time. Storativity ranges from 0.071 to $3.7E-5$. Since neither the response time nor the transmissivity remain constant between intervals, a storativity for the whole aquifer could not be defined. However, in the M1-3 to M1-5 interval, T values and response times were fairly consistent. Using this interval, the average S for a Central Block pumping well is $5.1E-4$ and for an Eastern Block pumping well is $9.4E-4$.

One of the primary concerns is the degree of connection present between the various aquifers. This issue was raised in the previous chapter. If the aquifer consists of a

few thin permeable horizons, then advective heat transport will be restricted to these horizons. The active aquifer thickness will be significantly less than the total depth intersected by the wells. Alternatively, if these horizons are vertically connected, and respond similarly to pumping in both fault blocks, then the active thickness of the aquifer will be greater.

Clearly, the M1-3 to M1-5 intervals can be considered as part of the main aquifer body since all of these respond to pumping in either block and the transmissivity values are relatively consistent within each block. The M1-2 interval is moderately well connected to both blocks since its T values are only slightly higher than those of M1-3 through M1-5. Intervals sampled by M1-6, M1-1, M1-B and M1-PVC are also connected to both blocks, although this connection is not as well developed. What this means is that there is a definite interval between 27 and 98 metres which may, in fact, extend down to 118 metres, where the aquifer is responding to pumping in either of the two fault blocks. This is a total of at least 71 metres of active aquifer.

4.4.5 Discussion

Using both the calculated transmissivities and the graphs for all constant discharge tests as guides, it is evident that response times are generally fast (less than 10 minutes) and transmissivities are low (less than $200 \text{ m}^2/\text{day}$) when observation wells are located within the same block as the pumping well. However, the response times are not consistent with a Theis-type aquifer since they do not, in general, increase the farther away the monitoring well is from the pumping well. At early times, following a brief period in

which wellbore storage is observed, the data depart from Theis and the rate of increase in drawdown is lower.

For monitoring wells located on the opposite side of a fault to the pumping well, the response times are generally delayed relative to those within the same fault block and the total overall drawdown at the end of the test is significantly lower than would be expected under true radial flow conditions. This indicates partial hydraulic connection. However, radial-type flow conditions are maintained, as exhibited by the good correlation with the Theis curve. The transmissivities are accordingly higher for cross-fault monitoring wells.

The M1 data show, at least for the Central Block, that these responses are consistent with depth, except in the top 30 metres (M1-6) and in the bottom 38 metres (M1-1 to M1-PVC). Since most wells are cased for the top 30 metres, the poor connection exhibited at M1-6 can be anticipated. The poor connection in the bottom intervals at M1 are probably related to the fact that M1 is significantly deeper (159 metres) than the Phase 1 pumping wells. All of these wells are 122 metres deep, except P1, which is 137 metres deep.

It would appear from the delay times that the different fault blocks are acting somewhat independently in the short term, yet as the duration of the test increases, this independence is lost and the system becomes linked hydrogeologically, hence, the response in the cross-block wells.

Although Theis appears to be applicable for much of the data, there are inconsistencies. The aquifer test data for Carleton show that there are two periods of non-

radial flow; one at early time, which cannot be accounted for by wellbore storage effects, and one at late time, which may be related to leakage or other boundary conditions. The presence of these departures from radial flow suggests that the assumptions inherent to the Theis and Jacob models have not been met.

One of the basic assumptions for Theis is that the aquifer is laterally continuous (*i.e.*, there are no boundaries within the area of influence of pumping). Because of the numerous faults present on campus and the apparent delays that are observed between wells located in opposite fault blocks (different T and S values), the Carleton aquifer is to be considered non-homogeneous on a large scale, and may be, at least, partially bounded. The following section explores several other analytical methods of analysis which are, to varying degrees, more applicable to the aquifer test data acquired at Carleton because they deal with fractured aquifers.

4.5 Analytical Models for Fractured Flow Regimes

There are several alternative analytical models which may be used to describe flow in fractured aquifers. Many of these models have been developed by the oil industry, since they routinely fracture the aquifers in order to increase the yields in their production wells. There has also been much research on naturally fractured rock, particularly for characterizing the groundwater regimes at potential nuclear waste disposal sites and at contaminated sites. Much of this research has employed rigorous mathematical modelling to focus on the finer details of fracture flow, such as the importance of the connectivity of fractures to the overall flow regime, and the nature of the fracture surfaces to controlling

flow. Rarely have large-scale aquifer test data been used to examine the flow regimes in fractured aquifers.

As a result, the literature is confusing. In fact, Streltsova-Adams (1978) stated: “Published work on well tests in fractured reservoirs clearly indicates the lack of a unified approach, which has led to contradictory results in analyzing the drawdown behaviour”. Gringarten (1982) stated: “A careful inspection of the published analytical solutions indicates that they are essentially identical. Apparent differences come only from the definition of the various parameters used in the derivation”. There are references to fracture flow in many published journals, as well as in textbooks dedicated to aquifer test data analysis. Many of these appear to be concerned with crystalline aquifers, where primary porosity is of little importance. Double porosity flow models, which will be examined in Section 4.5.3, do assume some degree of primary porosity (or permeability), yet they also assume a relatively small-scale uniformity in the fracture network and, hence, a radial flow regime. In carbonate aquifers, there is some primary permeability, small as it is, which is often modified by secondary features such as bedding plane fractures or fractures associated with structural features (*i.e.*, faults and folds). A uniform fracture pattern may not always be present in carbonate aquifers. This is particularly true of karstic limestones where conduit type flow is often reported to occur.

Sections 4.5.1, 4.5.2 and 4.5.3 examine three different analytical methods for dealing with various fractured aquifers. The applicability of each of these methods to the Carleton data is discussed within each section. The theory has been summarized from Kruseman and de Ridder (1990).

4.5.1 Single Vertical Fracture Model

The conceptual model of radial flow conditions is widely recognized. The discharging well, in such a system, creates a cone of depression in the surrounding aquifer in which flow lines converge upon the well from all directions and the equipotential lines are concentric on the center of the well. Drawdown decreases with distance from the pumping well and the velocity increases toward the well.

If a well intersects a single vertical planar fracture, as shown in Figure 4.9a, which has a permeability sufficiently large that the ratio of the fracture permeability to the aquifer permeability approaches infinity, then the aquifer's unsteady response to pumping will differ significantly from that predicted by Theis. The solution to this type of flow problem has long been the subject of debate. In fact, the author contends that, to date, there has not been a definitive study on the different types of flow regimes in fractured aquifers. The underlying concept to the various analytical solutions is that the aquifer is homogeneous, isotropic and of large lateral extent. The aquifer is also confined above and below by impermeable horizons. A single plane, or vertical fracture, of relatively short length and infinite hydraulic conductivity, dissects this aquifer. The fracture is considered to be of zero width so that storage (storativity) can be neglected.

This type of aquifer representation allows the system to be modelled as an "equivalent" anisotropic, homogeneous porous medium. Since the pumping well intersects the fracture midway and since this fracture is of infinite hydraulic conductivity, the drawdown in the fracture will be uniform over its entire length (*i.e.*, there is no hydraulic gradient within the fracture).

Figure 4.9: Schematic Diagram Showing a Well that Intersects a Plane Vertical Fracture of Finite Length and Infinite Hydraulic Conductivity (after Kruseman and de Ridder, 1990)

At early pumping times, flow toward the fracture is one-dimensional (Figure 4.9b) and is best referred to as linear flow. The flow in the aquifer is horizontal and perpendicular to the fracture (parallel flow). The equipotential lines define a trough of depression, rather than a cone of depression as in radial flow regimes, and the axis of this trough coincides with the 'extended' well. Drawdown in this type of system is a function of perpendicular distance away from the fracture, rather than distance away from the pumping well.

As pumping continues, the flow pattern changes from parallel flow to pseudo-radial flow (Figure 4.9c) regardless of the fracture's hydraulic conductivity (Gringarten *et al.*, 1975). During this period, most of the well discharge originates from areas farther removed from the fracture. Kruseman and de Ridder (1990) stated "Often, uneconomic pumping times are required to attain pseudo-radial flow, but once it has been attained, the classical methods of analysis can be applied". It should be noted, that several authors have not recognized this late time shift to pseudo-radial flow (for example, Jenkins and Prentice, 1982). In fact, Sen (1986) suggested that the late time flow period, rather than the early time flow period, is dominantly a linear one. The discussions that followed the publication of Sen's (1986) paper took readers on a mathematical roller coaster ride as various researchers attempted to settle the apparent theoretical disagreements. The final verdict favoured Jenkins and Prentice (1982), whose simplified approach was consistent with previous research (Gringarten and Witherspoon, 1972; Gringarten *et al.*, 1975).

Since drawdown is a function of perpendicular distance to the fracture, rather than radial distance to the pumping well, it is important to consider the positioning of

monitoring wells for analyzing the aquifer test data. If all observation wells are located very close to the fracture, anywhere along its length, then the drawdown in these wells will be similar. Since the fracture is considered an 'extended' well, the response times in these wells will be instantaneous. Wells located at varying distances away from the fracture will undergo varying amounts of drawdown in addition to having different response times.

Gringarten and Witherspoon (1972) obtained the following general solution for the drawdown in an observation well located a distance, r' , away from a pumping well situated on a vertical planar fracture of half length, x_f :

$$s = \frac{Q}{4\pi T} F(u_f, r') \quad 4.2$$

where

$$u_f = \frac{Tt}{Sx_f^2} \quad 4.3$$

and

$$r' = \frac{\sqrt{x^2 + y^2}}{x_f} \quad 4.4$$

In Equations 4.2 to 4.4, it is apparent that the drawdown in an observation well depends not only on the parameter, u_f , (*i.e.*, on the aquifer hydraulic parameters T and S , the vertical fracture half-length, x_f , and the pumping time, t) but also on the geometrical relationship between the location of the observation well and that of the fracture. In Equation 4.4, x and y are the distances between observation well and pumping well, measured along the x - and y -axes, respectively.

Gringarten and Witherspoon (1972) derived simplified expressions for the drawdown in three simple cases; one for an observation well located along the x-axis ($r'=x/x_f$), one for an observation well located along the y-axis ($r'=y/x_f$), and one for an observation well located along a line 45° degrees from the axis of the fracture ($r'=1.4142x/x_f$). These expressions can be plotted as a family of curves for each particular well position (*i.e.*, for different values of r').

If the relative position of the observation well is known, drawdown values versus time can be plotted on log-log paper and the curve matched to one of the type curves. As for the Theis method, this curve-matching process yields a value of T and S for the aquifer. If the position of the observation well is not known relative to the fracture, data from at least two observation wells are required and the curve-matching process becomes one of trial-and-error.

Gringarten and Ramey (1974) and Gringarten *et al.* (1975) derived a similar set of expressions for a pumping well which intersects a single vertical planar fracture in an otherwise, homogeneous, isotropic, confined aquifer. The equation is actually the reduced form of Equation 4.2 for $r'=0$ and results in a single type curve for the pumping well (Figure 4.10a). At early pumping times, when the drawdown in the well is governed by horizontal, laminar flow in the aquifer (*i.e.*, the linear flow period), the data plot as a straight line on a log-log graph with a slope of 0.5. The parallel flow period ends at approximately $u_{wf}=1.6 \times 10^{-1}$ (Gringarten *et al.*, 1975). Furthermore, if the fracture is elongated and has a relatively low transmissivity, the parallel flow period may last

GRINGARTEN ET AL.'S TYPE CURVE FOR A PUMPING WELL LOCATED ON A VERTICAL FRACTURE

Figure 4.10a: Gringarten *et al.*'s Type Curve for the Pumping Well Located along a Vertical Fracture (Data From Gringarten *et al.*, 1975)

relatively long (Kruseman and de Ridder, 1990). Through a process of curve-matching, Equations 4.2 and 4.4 can be solved for T and the product Sx_f^2 .

The pseudo-radial flow period starts at approximately $u_{vf}=2$ (Gringarten *et al.*, 1975). During pseudo-radial flow, the drawdown in the pumping well varies according to the standard Theis equation for radial flow plus a constant and can be approximated by the expression (Gringarten and Ramey, 1974):

$$s_w = \frac{2.3Q}{4\pi T} \log \frac{16.69Tt}{Sx_f^2} \quad 4.5$$

For large pumping time, the data can be analyzed by plotting drawdown versus time on semi-log paper and solving Equation 4.5 in a method similar to Jacob's straight line method described earlier.

The type curves for the vertical fracture flow model (Figure 4.10b) clearly indicate that the drawdown response in an observation well differs from that in a pumped well. As long as the observation well does not intersect the same fracture as the pumped well, the log-log plot of drawdown versus time in the observation well does not yield an initial straight line of slope 0.5. In fact, far enough from the pumped well (*i.e.*, $r' > 5$), the drawdown response becomes identical to that for radial flow to a pumped well in the Theis equation.

Figure 4.11 shows a semi-log plot of the type curve for the vertical fracture flow model. In contrast to the linear Jacob graph, the vertical fracture flow model predicts a broad curvature in the early time data. This type of response was identified for several of the wells at Carleton. Jenkins and Prentice (1982) also noted that an arithmetic plot of s versus $t^{1/2}$ will yield a straight line if linear flow conditions are present. Figure 4.12 shows

GRINGARTEN-WITHERSPOON'S TYPE CURVES FOR A VERTICAL FRACTURE WITH AN OBSERVATION WELL ON X-AXIS

Figure 4.10b: Gringarten-Witherspoon's Type Curves for a Vertical Fracture with an Observation well Located on the X-axis at $r'=1.2$ and $r'=1.5$ (Data From Gringarten and Witherspoon, 1972).

SEMI-LOG GRAPH SHOWING GRINGARTEN ET AL.'S TYPE CURVE FOR A PUMPING WELL LOCATED ALONG A VERTICAL FRACTURE

Figure 4.11: Gringarten *et al.*'s Semi-log Type Curve for the Pumping Well Located along a Vertical Fracture - (Data From Gringarten *et al.*, 1975)

CONSTANT DISCHARGE DATA P21 OBSERVATION WELL P21

Figure 4.12: Drawdown Versus Square Root of Time for Well P21
During Constant Discharge Test.(data after CESI, 1994)

the constant discharge test data for pumping well P21 plotted in this fashion. The linear portion extends until approximately 200 minutes. At this time, there is a significant break in slope where the rate of increase in drawdown is reduced (discussed in section 4.5.4). It is difficult to determine whether the data plot as a straight line on the semi-log graph at late time or whether there is a slight curvature. In the former instance, the linear flow regime will persist after the break in slope. A convex upward late curve may indicate radial or pseudo-radial flow.

An analysis of all of the well data for Carleton has led to the general conclusion that the fracture flow model is a definite improvement over the Theis model for several of the pumping wells and many of the observation wells. If the position of the faults are taken to coincide with these theoretical planar vertical fractures, then, in addition to well yield variability, the aquifer test data suggest that several of the wells are located directly on or very close to these features.

Pumping Well Data

The vertical fracture flow model provides a fairly good fit to the pumping well data; however, the late time changes in slope (*i.e.*, a flattening of the data curve), which are evident on many of the data curves (Figure 4.13), are not accounted for in this model. The following points summarize the results of log-log curve matching.

- The early time slopes (log-log scale) for wells P21 and P2 are close to 0.5 (Appendix 6), as predicted by the model. The mid-time data show a slightly higher rate of change of drawdown with time than that predicted by the model, suggesting that drawdown is continuing to increase. At late time, the data curves flatten out. Although pumping

CONSTANT DISCHARGE DATA FOR PUMPING WELLS

Figure 4.13: Semi-log Graph Showing Constant Discharge Data for Production Wells (Data From WESA, 1988 and CESI, 1994)

well data for well P23 are not available, the response at this well is likely similar to that of P21, since it responds immediately to pumping at P21 and has a similar final drawdown. Although an alternative model will be proposed for these wells in the following sub-section, the fit to the vertical fracture flow model is reasonably good.

- The Theis model is more appropriate for pumping wells P1 and P4 (Appendix 6), although there is still a late time change in slope. Very early time data reflect wellbore storage effects, as discussed earlier, making curve-matching more difficult. Neither of these wells is located immediately on or adjacent to a major fault, which suggests that the vertical fracture flow model is probably not the most appropriate model. The poor well yields obtained in these two production wells is further evidence of their distance to major faults.
- Finally, the pumping well data for wells P5, P6 and Elba2 (Appendix 6) do not appear to fit this model. In all of these wells, wellbore storage dominates the early time response. From time less than 10 minutes to approximately 30 minutes (variable), the curve is generally flat. After this time, there is a gradual increase until the characteristic late-time flattening occurs. Clearly, the Theis model is not applicable to these data. However, there is a reasonably good fit to the vertical fracture flow model if the slight early time flattening is disregarded. Obviously, there is some other effect which is being manifest in these data. Possible causes will be addressed at the end of this section.

Observation Well Data

In terms of observation well data, the vertical fracture flow model is a very applicable model and accounts accurately for the unusual responses for observation wells located in different fault blocks. The data for the constant discharge test at well P2 are considered (Figure 4.14).

- The pumping well, P2, is located very close to or right on the Eastern Fault (Figure 4.3) and has the highest overall drawdown. In the Eastern Fault Block, observation well RC responds first, then P6, then P5, then P3, then Elba2 and finally Elba1.
- The Theis-like response (minor early time curvature in semi-log plot) in the observation wells located in the Central Block (*i.e.*, wells P1, P4 and Tory) suggests that these wells appear to be, in a hydrogeological sense, located far enough away from the Eastern Fault that radial or pseudo-radial flow conditions dominate. Well P4 is much closer to the Eastern Fault than well P1, yet the drawdown response is almost identical to that for well P1.

Transmissivity values were calculated using the single vertical fracture model for all of the pumping wells, even though curve-matching was, in some cases, difficult. Well P1 and P4 had the lowest transmissivities of 57 and 41 m²/day. These values are very close to those calculated using Jacob (refer to Table 4.3). The transmissivity value for well P21 (and probably P23) is 61 m²/day. This value is consistent with that calculated using the Jacob method (refer to Table 4.3). Finally, the transmissivity values for all of the Eastern Block wells range from 208 m²/day (Elba2) to 230 m²/day (P5). Of these wells, only P2

CONSTANT DISCHARGE DATA FOR PUMPING WELL P2 AND OBSERVATION WELLS

Figure 4.14: Semi-Log Graph Showing Constant Discharge Test Data for Pumping Well P2 and Observation Wells (Data From WESA, 1988 and CESI, 1994)

appears to be located directly on a fault. The difference in geology between the Central (T=41 to 60 m²/day) and Eastern Fault Blocks (T=208 to 230 m²/day) may account for the observed delays in cross-block observation wells.

Storativity values were more difficult to calculate, since an estimate of the fault half-length, x_{vf} , is required. Estimates of fault half lengths would be purely speculative, since their extent, beyond the campus, is uncertain. As well, their “hydraulic” length may be very different from their physical length, particularly if mineralization is present.

The clustering of well responses suggests that the faults are acting as highly permeable features, with an assumed infinite hydraulic conductivity. The immediate responses for wells located within the same fault block as the pumping well, and in close proximity to the fault, are consistent with the early time concept of an extended well. The slight delays observed in certain wells are related to their perpendicular distance from the fault, rather than their radial distance away from the pumping well. This is consistent with a linear flow regime in the aquifer and semi-log graphs for wells located within the linear flow time period have a characteristic broad curvature.

Describing the Carleton aquifer regime as being dominated by a linear flow regime is a major step towards developing an understanding of how the flow takes place in this complicated aquifer system. Although the vertical fracture flow model appears to be appropriate for the Carleton data, the author suggests that there are still some minor discrepancies, particularly in the pumping well data.

4.5.2 Single Vertical Dyke Model

The vertical dyke flow model is closely related to the vertical fracture flow model described in the previous section. Kruseman and de Ridder (1990) reviewed several methods of analysis for dealing with aquifers in which a permeable dyke is present. The author refers the reader to this reference for the mathematical expressions. This section summarizes the assumptions and limitations of this model and provides a comparative analysis to the Carleton aquifer test data.

In their introduction, Kruseman and de Ridder implied that this model is meant for dykes in the igneous sense. The dykes are highly jointed, fractured or weathered, thus, increasing their hydraulic conductivity several times over that of the country rock. Although not considered by Kruseman and de Ridder (1990), the author suggests that these models can be applied to sedimentary rocks in which faults act as the permeable conduit to groundwater flow. Faults often consist of a zone of brecciated rock on either side of the failure plane, which, hydrogeologically, can be expected to increase the permeability of the formation if secondary mineralization is not present.

Igneous dykes can vary in length from a few kilometres to several hundred kilometres and can have a width varying from less than one metre to tens of metres. Thus, the assumptions of short length and zero width, as for the vertical fracture model, do not hold. Therefore, the vertical dyke model assumes that the dyke is of infinite length and has a finite thickness and finite hydraulic permeability. The enhanced permeability is derived from an extensive fracture network, which is absent at depth due to increased confining

pressures and at the surface due to intensive weathering. Therefore, the water contained in the dyke is confined.

If a well is located along the dyke, and is completed within the fractured portion of the dyke, then three periods of flow can be identified. At early times, all of the pumped water originates from storage in the dyke, and none from the aquifer. This situation is described as the linear flow period. The data plot as a straight line with a slope of 0.5 on a log-log plot. At intermediate times, all of the water is supplied from the aquifer, and none is from storage in the dyke. This is the parallel flow period and the data plot again as a straight line but with a slope of 0.25. Cinco Ley *et al.* (1978), as referenced in Kruseman and de Ridder (1990), identified a similar slope for fractures with finite hydraulic conductivity. At late time, flow in the aquifer is pseudo-radial. The late data will plot as a straight line on a semi-log graph. It must be emphasized that the transitions from one type of flow to another are not abrupt, and that time-drawdown plots (whether log-log or semi-log) will yield curved segments (Figure 4.15a). On a semi-log graph the non-linear early time data extends into mid-time, thus, enhancing the broad curvature of the plot (Figure 4.15b).

The method of determining the hydraulic parameters for both the dyke and the aquifer is one of curve matching. However, the dyke parameters can only be determined as a function of the width of the dyke. The aquifer transmissivity can be calculated using the Jacob approach for late time data. A close examination of the constant discharge test data (log-log) for pumping wells P2, Elba2 and P21 (Figure 4.16), which are considered to be located on or near to faults, suggests that the drawdown in these wells would continue to

BOONSTRA AND BOEHMER'S TYPE CURVE FOR A PUMPING WELL LOCATED ALONG A VERTICAL DYKE

B

Figure 4.15a: Boonstra and Boehmer's Type Curve for a Pumping Well Located along a Vertical Dyke (Parameters defined in Boonstra and Boehmer, 1987)

**BOONSTRA AND BOEHMER'S SEMI-LOG TYPE CURVE FOR A PUMPING
WELL LOCATED ALONG A VERTICAL DYKE**

Figure 4.15b: Boonstra and Boehmer's Semi-log Type Curve for a Pumping Well Located along a Vertical Dyke (Parameters Defined in Boonstra and Boehmer, 1987)

Figure 4.16: Log-Log Graph Showing Constant Discharge Data for Production Wells (Data From WESA, 1988 and CESI, 1994)

rise in a way consistent with the vertical dyke model if the late time change in slope was absent. There is also a pronounced curvature in the semi-log plots for these three wells (refer to Figure 4.13).

The mid-time flattening, observed at several of the pumping wells, and the late time change in slope are still not accounted for with the vertical dyke model. Each of these will be discussed separately in the following two sections.

4.5.3 Double Porosity Flow Model

Barenblatt *et al.* (1960) first considered fractured rock formations as consisting of two overlapping continuum media, porous and fissured, and each filling the entire flow domain. The matrix rock is defined as having some primary porosity. This rock mass has randomly distributed and arbitrarily oriented fractures, which have a secondary porosity of the same order of magnitude as the matrix rock. Streltsova (1976) distinguished this system of blocks and fissures to be purely fractured when it consists entirely of the continuous fracture porosity, and as being doubly porous when it is controlled by the fracture and block hydraulic properties of the same order of magnitude. In the latter instance, the total permeability is due primarily to the fractures and the storativity to the matrix.

The concept of double porosity flow is based on the assumption that there are two superimposed flow fields, each with their own respective drawdown equation. When the pressure in the aquifer is lowered, there is an immediate pressure reduction within the fracture network. In the pseudo-steady state interporosity flow model, it is assumed that there is a re-equilibration of this pressure differential between the matrix and the fissures

with no resulting decline in head in the matrix (Streltsova-Adams, 1978). In this situation, the drawdown becomes constant at intermediate times, during the transition period from purely fracture flow to block and fissure flow. At late times, the flow is described by the Theis equation since the flow is from the block and fissure system. Bourdet and Gringarten (1980) showed that the double porosity flow model only applies in a limited area around the pumping well. Outside of that area, the drawdown behaviour is equivalent to a Theis-type model. Alternative models consider transient block-to-fracture flow (Boulton and Streltsova, 1977).

The characteristic drawdown curves, for an aquifer having double porosity, consist of two Theis curves, one for the fissures and one for the fissure/block system, which are separated by a transition flow period where there is little or no drawdown. The log-log family of curves are, in fact, similar to those for an unconfined aquifer (Neuman, 1972). The semi-log graph shows characteristic “asymptotic” drawdown lines (*i.e.*, parallel lines separated by the interflow period). It must be emphasized that the double porosity model is a radial flow model and any immediate responses between wells would be due to a strictly fracture flow model as described in the previous two sub-sections.

The constant discharge data for all of the wells (Figure 4.13) were examined. The following observations, concerning the possible presence of double porosity flow, were made.

- In the log-log graphs, double porosity should be manifest as asymptotic drawdown curves, convex side up with little drawdown in between. In earlier analyses, it was noted that pumping wells P5, P6 and Elba2 exhibited a mid-time curve flattening, which

was not consistent with either the single vertical fracture or single vertical dyke flow models. This flattening is similar to the transition period expected in doubly porous aquifers; however, it does not appear on the semi-log plots and there is a gradual increase in drawdown at the well.

- Asymptotic drawdown lines in the semi-log data may only be present in the very late data at the M1-4 and M1-5 level piezometers during the constant discharge test at P1. However, in this case, the early drawdown behaviour is not consistent with the double porosity flow model, since broad data curvatures are present and the period of block and fissure flow is not sufficiently long to determine if two parallel lines exist. In addition, this type of curve is only present for two out of the eight piezometers and in a radial flow regime one would expect all levels to respond in the same manner.

Wilson (1990) stated “many of the pump test drawdown versus time curves produced by WESA (1988) exhibited features, such as asymptotic drawdown lines and changes in slope, which are also characteristic of fractured aquifers”. Wilson (1990) did not indicate which of the data were used to justify this statement; however, the author could not identify any definite “asymptotic drawdown curves” in any of the data at Carleton. In addition, “changes in slope” are not necessarily related to fractured aquifers. It is more common to observe changes in slope resulting from barriers or leakage. However, in one study, a change in slope in the drawdown versus time graph was observed in a carbonate aquifer (Eagon and Johe, 1972, reported in Streltsova-Adams,

1978). This was related to dewatering effects and was manifest as a sharp increase in slope, not a decrease in slope as observed at Carleton.

On the basis of the these observations, it is suggested that double porosity flow does not take place at a noticeable scale at Carleton and that vertical fracture or vertical dyke flow are much more prevalent.

4.5.4 Late Time Data

One of the recurring problems in analyzing these data is the late time change in slope, which was observed in almost all of the constant discharge data. The change is most pronounced on the semi-log plots, but is also present in the log-log curves discussed previously.

- In the Central Block pumping wells, P1, P4 and P21, the change in slope occurs at around 100 minutes (Figure 4.13). At P4 and P21, the transition is sharp, whereas, at P1 it is gradual. During these tests, the Central Block observation wells all show a similar response (refer to Appendix 6). For example, wells T1 and P1 have a corresponding change in slope at much the same time. The change in slope at the Tory well during this test was slightly delayed relative to the other Central Block wells. During these tests, the Eastern Block wells display no apparent change in slope (refer to Appendix 6). The apparent absence of a late time change in slope in the Eastern Block wells is likely due to insufficient drawdown for the effect to be manifested in these wells.
- During the aquifer testing in the Eastern Block, the late time change in slope at the pumping wells occurs at approximately 1000 minutes (Figure 4.13), with the exception

of well P5, where there was no noticeable change in slope. The change in slope was observed at approximately the same time for both the Eastern Block and the Central Block observation wells during these tests (refer to Appendix 6), although the effects were less pronounced in the latter wells.

- At the M1 multilevel, the response is variable. When either a Central Block or an Eastern Block well is pumped, the change in slope is only observed in those piezometers which respond early in the test and which have the greater drawdown at the end of the test. The greater the amount of drawdown, the earlier the change in slope. This type of behaviour is consistent with either leakage or a constant head boundary.

There are several possibilities for the origin of this decrease in the rate of drawdown.

- One of the first explanations considered by the author for this leakage was that a new source of water was coming from the opposite fault block. In this sense, the aquifer in the pumped fault block would eventually find the water in the opposite fault block. This argument appears to hold fairly well when pumping in the Central Block is considered. The changes in slope appear to correspond to the times that drawdown begins to increase in the Eastern Block wells. When the situation is reversed, and pumping takes place in the Eastern Block, the Central Block wells begin to respond before the leakage effects are noted.
- An alternative explanation for the leakage involves a source of water above or below the active part of the aquifer. In this sense, the aquifer can be considered as a true leaky

system. An examination of the M1 multilevel data suggests that, although this explanation is supported by most of the data, there are some inconsistencies which suggest that this form of leakage is not taking place. The M1 data acquired during the P1, P2, P5 and P6 constant discharge tests showed that the M1-6, M1-1, M1-B and M1-PVC levels all have a delayed response and much less drawdown, compared to the M1-5 to M1-2 interval, at the end of the test. In fact, the time of the change in slope, observed at the other levels, coincides with the onset of drawdown at these upper and lower level piezometers. This suggests that M1-6 and M1-1 to M1-PVC may be leaking into the main aquifer. However, during the constant discharge test at P4, there was some change in slope in the upper (M1-6) and lower (M1-B) levels. The change in slope, similar to that observed in the pumping wells, suggests that leakage is occurring at these levels. Obviously, the upper levels cannot both provide a source of water and experience leakage.

- Finally, leakage can be considered to be the result of a constant head boundary(s) situated nearby. This boundary may be connected through the faults on campus, and it may be associated with faults which do not enter the domain, but which are present at a distance of approximately one kilometre from the campus to the north, west and east (*i.e.*, the continuation of the Eastern Fault north of Dows Lake and across to Billings Bridge). In both cases the constant head boundary is only felt at high pumping rates or long pumping times.

4.5.5 Derivative Analysis for Aquifer Test Data

Curve-matching and straight-line procedures are used most frequently to determine the hydraulic parameters of an aquifer. However, in many cases, more than one type curve can be identified that provides a reasonably good match to a drawdown versus time data curve. For example, double porosity and multi-layer models predict similar transient drawdown-time curves.

Bourdet *et al.* (1984) introduced the use of derivative curves for the analysis of well-test data, although the technique had been widely applied in the oil industry for many years prior (pressure derivative analysis). One of the major advantages of using this technique, in addition to standard analytical techniques, is that it can be used as a diagnostic tool and can identify the type of flow regime that is present. For example, a radial flow regime has no characteristic behaviour in a type curve; however, the logarithm of the derivative of drawdown (hydraulic head) has a well known value and a definite shape when plotted against the logarithm of time.

The improvement in hydraulic test analysis through the use of derivatives is attributed to the sensitivity of the derivative response to small variations in the rate of pressure change that occurs during testing, which would otherwise be less obvious with standard pressure change (hydraulic head) versus time analysis (Spane and Wurstner, 1993). The enhanced sensitivity of the derivative facilitates its use in identifying the presence of wellbore storage, boundaries, and the establishment of radial flow conditions within the test data.

Figure 4.17 shows a few simple sketches of the various log-drawdown derivative versus log-time curves and the standard log-drawdown versus log-time curves that will result in different aquifer environments. In 4.17a, the aquifer is confined, of infinite aerial extent and wellbore storage is present. Wellbore storage produces a characteristic “hump” in the drawdown derivative plot, which increases in amplitude and duration as the associated wellbore storage value increases. In the absence of wellbore storage, there is no “drop” in the derivative before infinite-acting radial flow commences. Infinite-acting radial flow conditions are indicated during testing when the change in pressure (drawdown) at the point of observation increases in proportion to the logarithm of time. This is also indicated when the derivative curve becomes horizontal. Test data displaying this derivative pattern can be analyzed using Jacob’s straight line method.

The presence of non-radial flow conditions caused by leakage, vertical flow or the presence of boundaries is denoted on the derivative plot by a diagnostic response pattern that deviates significantly from the horizontal radial flow-line region of the graph. On a regular log-drawdown versus log-time graph these changes are subtle. Figure 4.17b shows the effect of a constant head boundary and Figure 4.17c shows the effect of a no-flow boundary.

A simple fixed end-point algorithm that could calculate the pressure derivative was included in the spreadsheets that contained the water level data as a function of time for several of the constant discharge tests at Carleton. Although not as sophisticated as the software program *DERIV* (Spane and Wurstner, 1993), which allows the user the flexibility of selecting the type of derivative algorithm that is used (*i.e.*, fixed end-point or

Figure 4.17: Characteristic Log-Log Drawdown Versus Time and Derivative Plots for Various Hydrogeological Boundary Conditions in Confined Aquifers (after Spaine and Wurstner, 1993).

least-squares fit) and the abscissal length (L-spacing) over which the derivative is averaged, this simplified algorithm is nonetheless effective. This algorithm calculates the first derivative of the drawdown, with respect to the natural logarithm of the change of time, using the point immediately before and immediately after the point of interest, and averages the two values. The limited number of data points at Carleton precluded the use of an L-spacing greater than the time interval between measurements. This resulted in a less smooth curve, such that even the slightest deviation will be visible. Equation 4.6 shows the mathematical expression used to calculate the pressure derivative (Spane and Wurstner, 1993).

$$(dP / dX)_i = [(\Delta P_1 / \Delta X_1)\Delta X_2 + (\Delta P_2 / \Delta X_2)\Delta X_1] / (\Delta X_1 + \Delta X_2) \quad 4.6$$

where the subscript 1=point(s) before the point of interest, i ; subscript 2=point(s) after the point of interest, i ; and X =natural logarithm of the time, t . It is further noted that the definition of t is a function of the test type (*i.e.*, constant discharge or recovery) and the variable changes accordingly. For recovery tests, t is defined as the total test time divided by the time since the pump was shut off.

Figure 4.18 shows both the drawdown-time curve and the derivative-time curve for the observation well Elba2 during the constant discharge test at well P22. No wellbore storage is identified on the derivative plot. In addition, the early time data also show that the flow regime is non-radial until approximately 100 minutes into the test (slope approximately 0.5). Radial flow continues (near zero slope) until approximately 300 minutes when a constant head boundary is detected (downward deflection).

CONSTANT DISCHARGE TEST WELL P22 OBSERVATION WELL ELBA2

Figure 4.18: Graph Showing Data for Elba2 Well During Constant discharge Test at Well P22. Drawdown versus Time and Derivative Plot (Data From CESI, 1994).

Figure 4.19 shows the drawdown-time and derivative-time curves for the Elba2 well during the constant discharge test at this well. In contrast to the previous graph, wellbore storage is present until approximately 6 minutes into the test. From 6 minutes until the constant head boundary becomes apparent at 300 minutes, there is no definite period of radial flow, although the curve does tend to flatten slightly. This may be due to poor smoothing of the derivative curve, but is more likely an indication that a non-linear flow regime still dominates (*i.e.*, the early parallel flow period has persisted).

The use of derivatives in analyzing some of the Carleton constant discharge data made the task of identifying the different flow regimes more straightforward than was possible using only the drawdown-time curves. This is of particular importance in aquifers which are thought to behave non-linearly as a result of either wellbore storage, parallel flow, or boundaries. Through the use of this technique it was possible to return to the semi-log plots used in the Jacob analysis and re-interpret these on the basis of selecting the true linear portion of the curve. Many of the transmissivities and storativities calculated by WESA (1988) using the Jacob approach were very different from those calculated using the information derived from the derivative plots in this research, since there was always some discrepancy in the selection of the data on the straight-line portion of the curve. The derivative analysis permitted a more consistent approach to selecting these data.

4.6 Recovery Testing

Recovery testing was an important part of all of the aquifer testing programs conducted at Carleton. Immediately after the pump is shut off for the constant discharge test, the aquifer will begin to recover. This is manifested as a rise in water levels in all of

PUMP TEST DATA CARLETON ELBA2 OBSERVATION WELL ELBA2

Figure 4.19: Graph Showing Constant Discharge Data for Elba2 Well.
Drawdown versus Time and Derivative versus Time (data
after CESI, 1994).

the wells and is referred to as the residual drawdown, s' . The residual drawdown is defined as the difference between the original water level prior to pumping and the water level measured at a time t' after the cessation of pumping. Recovery testing serves as an independent check on the transmissivity value calculated from the constant discharge data.

The analysis of recovery data is based on the Principle of Superposition. It is assumed that after the pump has been shut off, the well continues to be pumped at the same rate as before, and that an imaginary injection well with a recharge of equal magnitude is superimposed at the pumping well. The Theis recovery equation, which is valid for confined aquifers, is defined as (after Theis, 1935):

$$s' = \frac{Q}{4\pi T} \{W(u) - W(u')\} \quad 4.7$$

where

$$u = \frac{r^2 S}{4Tt} \quad \text{and} \quad u' = \frac{r^2 S'}{4Tt'} \quad 4.8$$

In Equation 4.8, r is the radial distance from the pumping well, S and S' are the storativity during pumping and the storativity during recovery, T is the aquifer transmissivity, t is the total time (pumping plus recovery time) and t' is the recovery time. Often, the storativity values will be slightly different during and after pumping as a result of aquifer compression. However, when u and u' are sufficiently small (*i.e.*, $u, u' < 0.01$), T is constant, and the pumping and recovery storativities are equal (*i.e.*, $S=S'$), Equation 4.7 can be approximated as:

$$s' = \frac{2.3Q}{4\pi T} \log \frac{t}{t'} \quad 4.9$$

In a homogeneous, isotropic, confined aquifer which is unbounded, a plot of s' versus t/t' on a semi-log scale will yield a straight line, for large values of time. The slope of this line is used to obtain a value for T . This recovery analysis does not provide a value for storativity.

Because the Carleton aquifer is faulted and has been shown to exhibit linear flow under pumping conditions, Equation 4.9 will not strictly apply. Several researchers have considered the analysis of recovery data in linear-type flow regimes. For example, Sen (1986) derived a set of curves that could be employed to calculate aquifer parameters. This research suffered from mathematical errors, which proved the technique invalid (Hsieh, 1986; Chia-Shyun Chen, 1986). Jenkins and Prentice (1982) also derived expressions which could be employed in analyzing recovery data. They indicated that recovery data plotted as residual drawdown, s' , versus $(t^{1/2}-t'^{1/2})$ on an arithmetic scale would yield a straight line. Unfortunately, they lacked the recovery data to validate their expressions.

Recovery graphs showing residual drawdown, s' , versus t/t' , plotted on semi-log scales, are provided in Appendix 6 for all of the pumping wells. Figure 4.20 shows the recovery data observed at well P23 after pumping at well P23. The non-linear early time data observed during constant discharge testing is also apparent in the recovery data, except that the curvature is present in the latter portion of the curve (*i.e.*, early recovery). The late recovery curvature (from $t/t' = 2$ to 7) is due to leakage, as identified in the constant discharge data. Using the linear portion of the curve, a transmissivity of 30.3 m²/day is obtained from Equation 4.9.

RECOVERY DATA P23 OBSERVATION WELL P23

Figure 4.20: Graph showing Residual Drawdown versus t/t' for Well P23
Following the Constant Discharge Test at Well P23 (Data From
CESI, 1994)

Figure 4.21 shows recovery data for observation well Elba2, following the constant discharge test at P23. Linear flow is present in the early recovery data (*i.e.*, large values of t/t') and there is also a delayed recovery response in this well. Leakage effects are clearly absent.

Since constant discharge data for pumping wells P22 and P23 were not available for analysis, analysis of recovery data provided the only estimate of T for these wells. In general, the transmissivity values calculated from the Theis recovery method, using only the linear portion of the curve, were consistent with those calculated using the Jacob analysis. This is to be expected since both methods employ the straight line method.

4.7 Reinjection Testing

If water is discharged into an injection well, a cone of recharge will be formed, which is similar in shape but the reverse of a cone of depression surrounding a pumping well. In a homogeneous, isotropic, non-bounded, confined aquifer, the Theis equation can be used to calculate the theoretical positive head. At Carleton, the aquifer is partially bounded, thus, the theoretical cone of injection should not be consistent with Theis.

An integral part of the theoretical groundwater modelling relies on the results obtained from aquifer tests. Each well must be assessed in terms of its performance as both a pumping well, injection well and observation well. WESA (1988) noted that the literature provided a number of examples of design guidelines for injection wells. The NWWA and the State of Michigan (McCray, 1983) indicated that for design purposes injection wells should not be operated at more than 50% to 80% of their rated production

RECOVERY DATA P23 OBSERVATION WELL ELBA2

Figure 4.21: Graph Showing Residual Drawdown versus t/t' for the Elba2 Well Following the Constant discharge Test at Well P23 (Data From CESI, 1994).

discharge. The consensus was that there was an immediate disparity between injection well and pumping well efficiency.

To assess this disparity and to identify preferential or poor hydraulic connections between wells, WESA conducted four step discharge/reinjection tests and three constant discharge/reinjection tests, in addition to the standard constant discharge tests in 1988. The reinjection testing program formed a major part of the aquifer testing program for Phase 1 development. Details concerning these tests were provided earlier in Section 4.2. Table A6.1b (in Appendix 6) lists the various well pairs which were tested, the pumping/reinjection rates, the test type (step or constant reinjection), and the water levels or reinjection pressures which were measured at the end of the test period.

4.7.1 Reinjection Test Results

The primary outcome of the reinjection test program was the determination, in a somewhat quantitative way, of the inter-well connection and reinjection efficiency (WESA, 1988). Wells P2 and P5 were identified as being very well connected, because the reinjection test between P2 and P5 achieved the greatest discharge/reinjection rate (60.8 L/s) of all the reinjection tests conducted, and the drawdown was the smallest. In fact, during the reinjection test, where P5 was the pumping well and P2 the reinjection well, the water level at P2 did not rise significantly above the static level (2.19m) and the drawdown in the pumping well stabilized at 9.26m below static. In the reverse test (*i.e.*, P2 into P5), the pumping well (P2) drew down a total of 1.32m, while the reinjection pressure at P5 levelled off at approximately 10 metres.

In contrast, reinjection test results between other well pairs were less favourable. ReInjection pressure and drawdown were significantly higher in the constant reinjection test from P4 to P1 (41.7 L/s) than those measured in the P2/P5 tests. As well, reinjection testing between P2 (pumping) and each of P1 and P4 (reinjection) could only be done at low rates (15.1 L/s). This indicated that wells P2 and P5 were better connected hydrologically than, for example, P2 with either P1 or P4.

In general, the results of the reinjection tests indicated that the available drawdown in a well and, therefore, its long term yield is significantly influenced by the magnitude and location of reinjection. The study by WESA (1988) showed that if reinjection and withdrawal occur within a well connected system, the overall rates may be increased over rates where no mutual reinjection occurs. The drawdown in a well is significantly reduced when water is being reinjected into another well.

4.7.2 ReInjection Efficiency

P2 is probably the most efficient well on campus, in terms of its pumping capacity and reinjection capacity. It was determined that while P5 was 55% as efficient in injection mode of operation as in pumping mode, P2 was 92% as efficient (WESA, 1988). Although reinjection was not carried out between P2 and P6 or P5 and P6, it is likely that a similar efficiency range would have been obtained, since well P6 is located within the same fault block and shows excellent connection during constant discharge testing.

WESA (1988) used the results from reinjection testing to calculate fictitious aquifer storativity values, which would reproduce the reinjection results and, therefore, could be used in an analytical model to estimate real operating conditions. Of course, such

models do not account for well inefficiency. Normally, the reinjection well displays greater positive head than the theoretical amount.

Transmissivity values for wells remained essentially unchanged, while storativity values for the observation wells, in some cases, were increased by one order of magnitude to reflect the reinjection test data. For example, if the Jacob analysis for constant discharge data at well P5 yielded a storativity of 10^{-4} , when P2 was pumping, and the reinjection efficiency of P5 was poor, the storativity was raised to 10^{-3} . In an analytical model, it would appear that when P2 was pumping, the drawdown would be less in well P5, so that if P5 were reinjecting, its positive injection head would be greater.

Storativity values for P2, as an observation well, invariably remained consistent with the calculated field data, while those values for the other wells, in general, increased to reflect their lower ability to receive injected water. In the author's view, this was an innovative and effective method of dealing with well inefficiency in a pumping/reinjection system of wells.

A reinjection program was not included in the aquifer testing program for Phase 2. This was due primarily to budgetary constraints, as well as to the logistical problems associated with setting up reinjection equipment across the campus. CESI (1994) noted that since wells P22 and Elba2 are located in the Eastern Block, it is likely that these wells will respond to reinjection in a similar way to P2 and P5. Wells P21 and P23 are located in the Central Block, in terms of their lithology; however, the immediate hydrologic response observed in one while the other is pumping is suggestive of an equally good connection between these two wells. A similar approach to that employed by WESA (1988) for data

modification for the purpose of analytical modelling was used by CESI (1994). Details concerning this analytical modelling approach (WESA, 1988; CESI, 1994) are presented in Chapter 5.

4.8 Summary

This chapter examined the complex hydraulic behaviour of the Carleton aquifer system under transient conditions. Various analytical flow models, and their related type curves, were compared to the transient constant discharge data for several pumping wells and their respective observation wells. The results of these analyses have led to the characterization of the Carleton aquifer system as one which consists of two distinct aquifers (different T values for the Eastern and Central Blocks, and different static heads - Chapter 3), which are separated by a highly permeable fault (vertical fracture or dyke flow model). These faults are considered to be the dominant hydrogeological features which control the movement of water between the aquifers.

When a pumping well is located in close proximity to one of these faults, the flow regime is predominantly linear as opposed to radial at early time. This type of flow regime results in a characteristic immediate response to pumping in observation wells situated along the fault. In addition, the magnitude of the drawdown and the shape of the drawdown versus time curves, plotted as log-log or semi-log graphs, are consistent between these observation wells.

The measured drawdowns and the response times in this non-radial flow regime are a function of the perpendicular distance away from the vertical planar fracture(s), rather than the radial distance away from the pumping well. Because of the different

formations present on either side of the faults, the response times for observation wells located in the opposite fault block are greater than expected from purely radial flow and these delays result in smaller measured drawdowns at these wells.

5. HYDROGEOLOGIC MODELLING

5.1 Hydrogeologic Modelling

Hydrogeologic or groundwater-flow models are used to calculate the hydraulic head within the aquifer. They are based on the partial differential equation (*pde*) for groundwater flow with appropriate boundary and initial conditions. This mathematical equation may be solved analytically or numerically. When applying analytical methods, a functional representation for the solution of the *pde* is sought (e.g., the hydraulic head as a function of time and position in an aquifer). Usually, analytical solutions are very accurate, since the systems they represent are necessarily simple and, hence, the physical system is well defined.

Alternatively, numerical methods may be applied to solve the *pde*. Although there are various numerical techniques for solving this equation, the finite difference and finite element methods are most commonly used. Invariably, the complexity of these methods requires the use of a computer for carrying out the calculations. In contrast to analytical methods, numerical methods do not require as restrictive assumptions. Solutions to problems involving anisotropic, heterogeneous aquifers, with complex time varying boundary conditions, can be approximated with a degree of accuracy dependent on the type of numerical method employed, the complexity of the boundary and initial conditions, and the computational precision of the computer.

Mathematical models, used to make predictions, assume that the material properties and the boundaries of the groundwater system have been accurately defined.

However, to determine uniquely the parameter distribution for a field problem would require so much expensive field testing that it is seldom feasible either economically or technically. It is inherently impossible to obtain a unique solution for a large set of simultaneous equations having more unknowns than equations.

If the model is required for predicting some physical change in the system in response to an external stress, such as estimating changes in hydraulic heads in an aquifer in response to pumping, then the model should be calibrated against known conditions in the aquifer under a variety of stress conditions. More often than not, calibration is carried out using steady state field data. If transient data are used, rarely is more than one set of stressed conditions considered.

Ideally, a three-dimensional groundwater flow and heat transport model was desirable for Carleton. To do this, sufficient data were required to design, calibrate and validate the model. With the exception of the one multilevel piezometer, that was available for a short period of time, there were no hydrogeologic data to generate a three-dimensional model. Suggestions have been made to conduct packer tests on some of the wells; however, to obtain adequate hydrogeological coverage, several wells would have to be tested and monitored. Such testing would be very expensive and the results from the tests would be very difficult to analyze, since the linear flow regime complicates the aquifer test results. In addition, no reliable vertical temperature resolution was available as a result of strong upward water flow in boreholes and the absence of permanently-installed temperature cables.

To estimate all of the vertical parameters could introduce too much uncertainty in a three-dimensional solution. Such a solution may appear to describe the flow in three-dimensions, but it could never be calibrated with the present data set. This lack of vertically-resolved data in the aquifer suggests that a two-dimensional model is more appropriate for the current data set for Carleton, although even in two dimensions there are still a number of assumptions that have to be made. The conceptual models and their mathematical representations, presented in the remainder of this chapter, are applicable for two-dimensional domains. The implications of using a two- as opposed to a three-dimensional problem domain are addressed later in this thesis.

The objective of this chapter is to develop a conceptual model for the Carleton aquifer. Chapters 6 and 7 will emphasize thermal energy storage in aquifers and incorporate heat transport in the model. Appendix 7 reviews some basic concepts of mathematical modelling and outlines steps for the modelling protocol. Section 5.2 describes an analytical approach which was used by WESA (1988) and later, CESI (1994), to calculate the hydraulic heads at the pumping and reinjection wells. Section 5.3 introduces the numerical approach, presents the governing equations, and outlines several options for representing fractured carbonate systems. Section 5.4 includes the design of the conceptual model, as determined from the geological and hydrogeological information, and the transfer of the conceptual model into grid format (discretization). Section 5.5 deals primarily with the determination of the aquifer parameters and Section 5.6 discusses the boundary and initial conditions for the models. Finally, Section 5.7 deals with the

selection of time steps for the transient models. Model calibration and results for all of the models (steady-state, transient and heat transport) are the focus of Chapter 8.

5.2 The Analytical Flow Model - A Direct Use of Aquifer Test Data

5.2.1 Theoretical Considerations

Analytical models are particularly useful as a first estimate for describing the flow characteristics of an aquifer, and, therefore, are used regularly to analyze aquifer test data. Since these data are analyzed according to the assumptions and limitations inherent to each analytical model, the aquifer parameters derived from curve matching represent the conditions present in the model aquifer. As long as the conceptual model for the aquifer is accurate, and there is relatively good agreement between the actual aquifer response and what the model predicts, analytical models can be used for predication with a fair degree of confidence. However, the model does not necessarily provide a unique solution.

Of course, if one is interested in utilizing the aquifer parameters, T and S , to predict how an aquifer will behave during a stressed condition, it would not be wise to employ an analytical model which is different than that employed to calculate the parameters in the first case. Use of a different model would, in effect, compound any errors which were initially made. Analytical models are not easily adaptable to heterogeneous aquifers, and this added limitation requires that the spatial dependence of the heterogeneity be incorporated into the mathematical expressions. In addition, analytical models generally assume simple boundary conditions (for example, an aquifer of infinite aerial extent), and thus, are relatively limited in their application. The mathematics become increasingly more complex, and the equations become more difficult to solve

analytically, as the complexity of the boundary conditions or the heterogeneity of the aquifer increases.

Nevertheless, analytical models can be very powerful and fast tools for predicting the aquifer's behaviour. WESA (1988), and later CESI (1994), employed a very simple analytical technique for modelling the Carleton aquifer. The variations in hydraulic head at each well, as a result of a system of pumping and reinjection wells all operating at different rates, were calculated. The model is based on the Theis Principle of Superposition, which states simply that the overall influence of several pumping (or reinjection) wells is additive. The expression which defines this statement is:

$$\Delta s = \frac{Q_1}{4\pi T} W(u_1) + \frac{Q_2}{4\pi T} W(u_2) + \frac{Q_3}{4\pi T} W(u_3) + \dots + \frac{Q_n}{4\pi T} W(u_n) \quad 5.1$$

and

$$u_x = \frac{r_x^2 S}{4Tt} \quad 5.2$$

where Q is positive if a well is pumping and negative if it is reinjecting, and r_x is the radial distance from the point of interest to each of the pumping/reinjection wells.

5.2.2 Parameter Estimation

The Principle of Superposition is based on the Theis equation, which is clearly not valid for Carleton. The Theis model assumes that one single value for T and one for S can be chosen to represent the aquifer, and as we have seen in the aquifer test data, it is difficult to determine, directly from the aquifer test data, what these values are because the aquifer is decidedly heterogeneous. Nevertheless, Principle of Superposition was used to calculate estimates of the hydraulic heads in multiple pumping and reinjection wells in various configurations, and to estimate pumping and reinjection rates for each well

(WESA, 1988; CESI, 1994). After four years of operation of the Phase 1 system, there were no indications that the system was operating inefficiently (CESI, 1994).

The greatest rate of change of drawdown, during the constant discharge tests, corresponds with the linear portion of the semi-log drawdown versus time curves. The transmissivity values, determined from this portion of the curve (Jacob method) were calculated for each of the production wells and all of the available observation wells (Table 4.3). Storativity values were calculated (Table 4.4) from the x-intercepts (Jacob method). The pumping well storativity, calculated using the pumping well radius, is many orders of magnitude higher than the true aquifer storativity, but is representative of the pumping well and reproduces the response of the well if that well is pumped.

The aquifer parameters were calibrated using data from reinjection aquifer tests (WESA, 1988). Different combinations of transmissivity and storativity values were used until the field data were recreated. Only very slight modifications were made to the transmissivities, while storativities were often increased by an order of magnitude or more. This calibration approach has the added benefit that well inefficiency is incorporated directly into the model data by way of modifying the aquifer parameters specified for each well. As noted in the previous chapter, well reinjection inefficiency is manifested as an inability of the well to accept water at the same rate as it can be theoretically withdrawn. The result is, for a two well pumping-reinjection couplet, a higher than predicted hydraulic head in the reinjection well, and a lower than predicted hydraulic head in the pumping well. If the storativity, calculated from the constant discharge data for the reinjection well,

is increased, the effect will be to simulate lower drawdown due to pumping at that well. During a reinjection simulation, it will appear that the hydraulic head is higher.

The hydraulic simulations required several T and S values for each well. One set of values represented the hydraulic parameters of the well itself as a pumping well. Other sets represented the well as a monitoring well during pumping at other production wells. The concept, although not strictly applicable, made use of the fact that the aquifer parameters are different at each well (Tables 4.3 and 4.4). Thus, in a sense, the model simulated the behaviour of the wells, but did not utilize actual aquifer parameters to calculate the response. Furthermore, the model could only approximate responses at wells where hydraulic data were available. In a non-linear flow regime, this method cannot calculate the hydraulic heads in the aquifer, it can only approximate them at the wells themselves.

Tables 5.1a and b show the calculated values of transmissivity and storativity used by CESI (1994) in the Phase 2 simulations. Shaded blocks show estimated values.

Table 5.1a: Transmissivity Values (m^2/day) used in CESI's (1994) Simulations

Pumping Wells:	P1	P4	P21	P23	P2	P5	P6	P22	E2
Observation Wells:									
P1	70	70	230	230	450	380	420	330	250
P4	70	66	231	231	250	163	245	330	220
P21	230	230	92	78	317	317	340	292	315
P23	230	231	85	80	350	350	187	292	331
P2	430	286	317	350	175	153	140	214	181
P5	380	330	317	350	153	163	120	220	160
P6	420	300	340	375	160	170	140	248	216
P22	330	330	308	329	214	220	248	242	208
E2	250	265	315	331	178	159	189	208	169

Table 5.1b: Storativity Values used in CEST's (1994) Simulations

Pumping Wells:	P1	P4	P21	P23	P2	P5	P6	P22	E2
Observation Wells:									
P1	0.064	1.0E-4	1.0E-4	1.0E-3	9.0E-4	5.0E-3	1.0E-3	1.0E-4	1.0E-5
P4	1.0E-4	5.5E-1	1.0E-3	1.1E-3	1.0E-3	3.0E-4	1.0E-4	1.2E-3	1.0E-5
P21	1.0E-4	1.0E-3	1.033	4.0E-7	2.1E-4	2.0E-4	1.3E-4	5.5E-4	1.8E-3
P23	1.0E-3	1.1E-3	2.1E-7	1.0	1.8E-3	1.2E-3	5.4E-4	4.4E-4	3.2E-4
P2	9.0E-4	4.1E-4	2.1E-4	1.8E-3	159.4	3.0E-4	1.0E-4	6.6E-5	2.0E-5
P5	5.0E-3	4.0E-4	2.1E-4	1.2E-3	3.0E-3	10.0	2.0E-4	6.6E-5	3.0E-5
P6	1.0E-3	4.0E-4	1.3E-4	5.4E-4	1.0E-3	2.0E-4	20.0	3.0E-5	5.0E-6
P22	1.0E-4	1.0E-3	5.8E-4	6.5E-4	6.6E-5	6.6E-5	3.0E-5	1.0E-2	1.1E-4
E2	1.0E-5	5.8E-4	1.8E-3	3.2E-4	2.6E-5	2.2E-5	2.8E-5	1.1E-4	41.9

Some wells, notably P5, P2, P1 and P4, were inaccessible during the Phase 2 aquifer testing program. Not only is their response to the pumping of the new production wells lost, but also the response of the new wells to these wells pumping is not available. The following approximations were made.

- The aquifer parameters at P2 were approximated by those for RC, located nearby. Based on the results of past aquifer tests, these two wells generally respond in a similar way to pumping, both within the same fault block and in the opposite fault block.
- The aquifer parameters at P1 and P4 were difficult to estimate since these wells were operating during Phase 2 testing. The estimates were based on T and S values for T24 and the M1 well. Given the low rates at which both P1 and P4 would be pumped or injected as production wells, their response is perhaps not as critical as some of the higher yielding wells. It would be useful; however, to observe the responses in these two wells while operating the new system.

- The aquifer parameters at P5 were perhaps the most critical to lose. Unfortunately, this well was not fitted with an observation port during Phase 1 development. There is no other well located nearby that could be monitored during aquifer testing. Hence, the response was taken to be similar to that of RC for lack of a better comparison.

5.2.3 Simulations

The computer program used by CESI (1994) was tailored to the Carleton aquifer system and utilized the values in Table 5.1 to calculate the drawdowns and positive heads in all production wells as a result of pumping and reinjection. Different combinations of wells were tested (*i.e.* the selection of pumping wells and reinjection wells varied). For each combination of wells, the program calculated the drawdown and positive head for different production rates. The total system operating rate remained constant, while the magnitude of the pumping rates and reinjection rates at the various wells changed. The difference between the maximum positive head and the maximum drawdown was a measure of the performance of the system. In this way, both the configuration and the pumping/reinjection rates were optimized.

The computer program was modified from WESA's (1988) program to include a maximum of nine production wells (Phase 2) as opposed to five (Phase 1). The optimum configuration of wells was selected not only on the basis of minimizing drawdown and positive head but also on several other factors. These included:

- A configuration of wells which would minimize thermal breakthrough between pumping and reinjection wells. Since the separations of the wells are known, the maximum pumping/reinjection rates between identified well couplets could be

calculated. This placed restrictions on the maximum production rates used in the simulations. Chapters 6 and 8 discuss the thermal aspects of the problem in greater detail.

- The optimum configuration of wells and the rates at which they operate were selected to minimize the amount of water which would have to be piped from one region of the campus to another, and yet meet the water demands of the various buildings.
- The maximum pumping rates for several of the existing Phase 1 wells were to remain within the specified limits for the pumps within these wells.

Of the 126 possible combinations of wells, one configuration was selected. This configuration met with the restrictions imposed by the thermal model calculations (Chapters 6 and 8) and well capacities (CESI, 1994). The pumping rates and calculated drawdown and positive head at wells are presented in Table 5.2. Negative rates represent pumping conditions at the production well, while positive rates represent reinjection. The total amount of water available during each season is 265 L/s (4204 USGPM). The results can be applied to either the heating or the cooling configuration. During the alternate season, the pumping/reinjection configuration is reversed and the drawdown and positive heads take on the opposite sign.

Table 5.2: Optimum Configuration of Wells and their Respective Production Rates in L/s for Phase 2 (Data From CESI, 1994).

Well:	P1	P4	P21	P23	P2	P5	P6	P22	E2
Rate (L/s)	+10.0	-10.0	-65.0	+65.0	-60.0	+95.0	-65.0	-65.0	+95.0
Calculated Drawdown/ Positive Head (m)	+19.9	-5.4	-6.7	+4.4	-1.2	+19.1	+2.0	-6.7	+15.1

These pumping rates will be used to test the well configuration for thermal efficiency (heat transport simulations - Chapter 8). Table 5.3 shows the well configuration selected by WESA (1988) for Phase 1. Included are the optimum pumping/reinjection rates and the calculated drawdowns and positive heads at each of the five Phase 1 production wells. In this design, the production rates for the heating and cooling seasons are not the same.

Table 5.3: Optimum Configuration of Wells and their Respective Production Rates in L/s for Phase 1 (Data From WESA, 1988).

	Well:	P1	P2	P4	P5	P6
HEATING SEASON	Rate: (L/s)	25	65	-30	-95	35
	Calculated Drawdown/ Positive Head (m)	13.2	-4.8	-19.1	-19.5	-3.4
COOLING SEASON	Rate: (L/s)	-30	-45	30	95	-50
	Calculated Drawdown/ Positive Head (m)	-20.9	-0.7	14.7	17.4	-6.6

5.2.4 Limitations of the Analytical Method

The analytical model described above has some definite limitations. In fact, there is no physical basis for applying the Principle of Superposition in a heterogeneous aquifer. However, because the system is being used for both pumping and reinjection, the variation in hydraulic head should be relatively small under steady-state operating conditions. Therefore, it is anticipated that the model might provide approximate hydraulic head

values for the wells under operating conditions, despite the uncertainty of the model's integrity.

One of the definite disadvantages of applying this type of model was that the heat transport could not be directly incorporated into the model. A separate analytical model, describing heat transport, was used by WESA (1988) and CESI (1994). This model is described in Chapter 6.

5.3 Numerical Flow Models

When using numerical methods, a discrete approximation for the solution (*i.e.*, values of the field variable computed at a set of specified points within the aquifer at a set of specified times) is sought. The number and location of points and the choice of times and time intervals are determined in advance. Numerical methods can be very accurate; however, the accuracy of the solution is ultimately dependent upon the type of numerical method used, the proper design of the grid, and the complexity of the boundary and initial conditions.

The finite difference method (*FDM*) computes a value for the head at each node on the grid. The node is located at either the intersection of four rectangular cells or at the center of a rectangular cell surrounded, in turn, by other cells. The former representation is referred to as being mesh-centered and the latter as block-centered. The value of the variable (in this instance the head) at the node is calculated directly from the finite difference representation of the partial differential equation (*pde*). The value is actually a weighted average of the heads of the nodes that surround the node in question.

FDMs are perhaps the most widely used methods for commercial purposes, simply because the algorithms are straightforward to write and the finite difference grids are much simpler to generate and modify using a front end graphical interface. Finite difference methods use a rectangular grid and this grid format is restricted for use with rectangular or prismatic aquifers. It is inherently difficult to incorporate irregular or curved aquifer boundaries, anisotropic and heterogeneous aquifer properties or sloping layers into the numerical model without introducing numerous mathematical and computer programming complexities (Istok, 1989).

The finite element method (*FEM*) has gained popularity over the past several years because of its much improved accuracy in solving problems with complex boundary conditions and problems involving transport. In the *FEM*, the domain is divided into a finite number of elements, each having its own discrete size and shape. The *FEM* is not restricted to rectangular elements, therefore, irregularly shaped boundaries are easily incorporated into the model. The variable is evaluated at the nodes, which are located at the intersections of the elements. The *pde* serves to modify the rules of interpolation between the nodes.

The *FEM* offers several advantages over the *FDM*. As noted above, the method easily incorporates both irregularly shaped domain boundaries and geological or property boundaries. The method is also more tolerant of grid size changes. For example, in the *FDM*, a block should not be any more than 1.5X the size of any adjacent block. In the *FEM*, geological features, such as faults, and pumping wells can be incorporated easily. Faults are a particular nuisance in finite difference models since the aspect ratio of the

blocks should also be restricted to no more than 4:3. Because the *FEM* is based on interpolation, the variation in properties from one element to the next is, in a sense, weighted. Therefore, it is permitted to have adjacent elements which have very different geological properties (*i.e.*, variation of an order of magnitude or more). It is for this reason that the *FEM* is better suited to solving more complex hydrogeological problems in which heterogeneity is a dominant factor.

For most modelling purposes, the finite difference (*FD*) method is satisfactory. In fact, the groundwater industry routinely uses finite difference algorithms to solve simple groundwater problems. There are a number of commercial software packages, most notably *MODFLOW* (United States Geological Survey) and *FLOWPATH* (Waterloo Hydrogeologic Inc.), which employ the finite difference method of solution. More recently, *VISUAL MODFLOW* (Waterloo Hydrogeologic Inc.) has been developed. These software packages are relatively simple to use; however, their main disadvantage is that they operate on a “black box” approach. The user enters the required information, for example, constant head nodes, pumping rates and recharge rates, and the “black box” generates a beautifully contoured map showing variations in hydraulic head. In most cases, calibration is carried out by comparing a single set of head measurements, at one specified time (usually steady state) to calculated ones. As well, only a few software applications have been made available in recent years to model heat transport (*HEAT*, Waterloo Hydrogeologic Inc.).

The author decided to write software that could be easily adapted to generate data in various formats, as well as to solve the transport equations. The *FEM* was selected as

the method of solution. However, because of its simplicity, *FLOWPATH* was used to generate a preliminary model of the steady state flow regime at Carleton (Section 5.5). The calibration data were the steady state undisturbed aquifer hydraulic heads presented in Chapter 3 (after Wilson, 1990).

5.3.1 Mathematical Representations of Fractured Carbonate Aquifers

A fractured aquifer consists of solid rock with some primary porosity cut by a system of cracks, microcracks, joints, fracture zones, and shear zones, that create secondary porosity and form a network for flow when interconnected. In order to describe the flow through fractured rock, information on the primary permeability of the rock matrix and the secondary porosity created by the network of fractures is required. Igneous and metamorphic rocks generally have primary hydraulic conductivities that range from 10^{-8} to 10^{-11} cm/s (Freeze and Cherry, 1979). Secondary permeability can effectively increase the hydraulic conductivity of a fractured crystalline rock system by up to five orders of magnitude (Gale, 1982). This increase is dependent upon the type of material and the number, width, and interconnection of the fractures.

Groundwater flow in fractured carbonate rocks is unlike that in fractured crystalline rock. The large variability in flow patterns and well yields, observed on a regional scale, is often explained by a relatively impermeable matrix rock modified by jointing and/or secondary dissolution along well defined (although perhaps not observable) high permeability features. However, it is equally accepted that many carbonate rocks have a matrix which is permeable. The problem, as noted earlier, is one of scale. The small scale flow variability may or may not have a noticeable effect on the overall flow regime,

depending on the scale of observation. The problem domain consists of two overlapping flow regimes, each of which has its own particular influence depending on the scale of observation.

In consideration of the complexity of these aquifers, formulating a conceptual model of a fractured carbonate system requires either a gross simplification or a detailed description of the aquifer properties controlling flow. Fractured systems are typically modelled using one or more of the following conceptual models: 1) discrete fractures, 2) equivalent porous medium, 3) dual porosity. With the exception of conduit flow in karst terrane, which can be considered an extreme case, fracture flow models assume that both the fracture apertures and flow velocities are small so that Darcy's Law applies.

Discrete Fracture Flow Models

The small-scale flow variability is dependent upon the nature of the fracturing. When the fracturing is sparse, the orientation, size and connectivity of the fractures become an important consideration in describing the flow regime. In these situations, discrete fracture models may be employed. A discrete fracture model (*DFM*) is typically applied to fractured media in which the primary rock has a very low primary permeability, such as crystalline rock. The model assumes that water moves only through the fracture network. Each fracture consists essentially of two parallel plates separated by a constant aperture ($2b$) and of width w . The flow Q_f is defined by:

$$Q_f = 2bwk_f(dh/dl) \quad 5.3$$

where dh/dl is the hydraulic gradient and k_f is the hydraulic conductivity. The hydraulic conductivity of the fracture is calculated from (Anderson and Woessner, 1992):

$$k_f = \rho g (2b)^2 / 12u \quad - \quad 5.4$$

where ρ is the fluid density, u is the viscosity, and g is the gravitational acceleration. Q_f is, therefore, proportional to the cube of the fracture aperture.

Because a model based on this type of conceptual design requires a description of the fracture geometry, it is often difficult to employ. In fact, models based on this type of approach are computationally intensive and have been, in general, applied to research problems (Tsang and Tsang, 1987; Long *et al*, 1982, 1985). The scale of the Carleton system, and the uncertainty of its subsurface fracture geometry, removed discrete fracture flow modelling as a viable method.

A similar approach, based on the discrete fracture flow model, has been developed in recent years to deal with the intricacies of fracture flow while alleviating the need for detailed information on individual fractures. This is based on stochastic principles and is used to represent fracture distributions within either a non-porous or porous rock matrix. Again, some form of uniform fracture distribution is required to use these models. Computer software is available that will generate randomly oriented fractures or fracture fields with some preferred orientation. The fracture network at Carleton is neither uniform nor it is strictly random. There are several distinct zones of fracturing, which are essentially self-contained. Therefore, this type of stochastic fracture model was not applicable at Carleton.

Equivalent Porous Medium Models

When the fracturing is dense, the fractured aquifer can be represented as an equivalent porous media (*EPM*) by replacing the primary and secondary porosity and

hydraulic conductivity distributions with a continuous porous medium having so-called *equivalent* or *effective* hydraulic properties (Anderson and Woessner, 1992). This permits the standard models of groundwater flow, which apply to porous media having continuous interconnected pore space, to be used, but in this case, the fractures are the interconnected pores. An *EPM* approach assumes that the fractured medium can be treated as a continuum and that a representative elementary volume (*REV*) of material, characterized by effective hydraulic parameters, can be defined.

The hydraulic conductivity, specific storage, and porosity are selected so that the flow pattern in the *EPM* is similar to the flow pattern in the fractured medium. This can be accomplished using aquifer test results or from field estimation of fracture apertures, lengths, and interconnections, and from the unfractured rock permeabilities. Berkowitz *et al.* (1988) suggested that if estimates of equivalent hydraulic parameters are made in the field, a reasonable flow model based on *EPM* concepts may be constructed. Other researchers (Cacas *et al.*, 1990) have concluded that an *EPM* approach is generally not valid for fractured systems. They noted that the *EPM* approach may adequately represent the behaviour of a regional flow system, but poorly reproduces a local one.

Although there is much controversy surrounding the use of *EPM* models for fractured aquifers, once the effective parameters are defined, standard finite difference or finite element codes, used for continuous porous media, may be used (e.g., Guerhart, 1984). Of course, the difficulty in applying the *EPM* approach arises in determining the appropriate size of the *REV* necessary to define the effective hydraulic parameters. When

fractures are few in number, with large separations, and the unfractured rock has a low hydraulic conductivity, the *EPM* approach may not be appropriate even with large *REV*s.

Double Porosity Flow Models

An alternative approach used to describe flow in fractured rock involves the concept of double porosity flow. This concept was introduced in Chapter 4. If the rock matrix containing the fracture network has significant primary permeability, a double porosity model (*DPM*) may be used. In this conceptual model, flow through the fractures is accompanied by exchange of water and solute to and from the surrounding porous rock matrix. In this case, both the hydraulic properties of the solid rock matrix and the fracture network must be defined prior to modelling. Aquifer tests may indicate whether a system behaves as a dual porosity system, and results can be interpreted to estimate hydraulic conductivities (Streletsova-Adams, 1978). Double porosity was not apparent in the aquifer test data for Carleton, therefore, this method was not explored further.

Karst Flow Models

Groundwater systems in limestone, dolomite, marble, evaporites and other soluble rocks may behave as typical fracture flow systems or may be influenced by conduit flow. In soluble rocks, fracture apertures and primary porosity are enlarged with time, creating a system of paths with higher hydraulic conductivity (Ford and Williams, 1989). In this situation, extreme variability in hydraulic conductivity can occur when the fracture networks are enlarged by solution. In the case of karst development, conduit flow may occur. In fact, many studies use conduit-type flow or fracture models to explain the complex hydraulic behaviour in carbonate aquifers. In general, soluble rock, with high

primary porosity, develops a diffuse flow system with few karst features, whereas rock with little primary porosity develops a conduit flow system with karst (Ford and Williams, 1989).

Modelling karst is complicated by the difficulty in characterizing hydrogeologic properties, including conduit geometry. Three modelling approaches have been used to simulate flow in karst (Anderson and Woessner, 1992):

- If the fracture or conduit size is restricted, groundwater flow in the porous and fractured rock will be governed by Darcy's Law so that one of the three conceptual models for fractured rock described above may be applied to the problem. For example, Thrailkill (1974) describes the application of a *DFM* to simulate flow in karst formations having pipes or conduits.
- For mature karst, a "black box" approach is used. Equations are derived which characterize the discharge from springs and these equations may or may not include the standard aquifer parameters. The aquifer response functions are developed to reproduce input and output responses, namely recharge and springflow. Typically, springflow is used to represent an integrated response of the groundwater system to recharge or stress.
- The aquifer response functions are used to derive equivalent porous media parameters for use in *EPM* models.

All of the modelling techniques discussed above require detailed information about the nature of the fracture or conduit network and even using this information, the

conceptual models require assumptions which grossly oversimplify the actual fractured system. Field techniques to collect these data have not been perfected, thus, predictions made with these models will continue to be influenced by large uncertainties and will require field validation. Although most modelling of fractured systems involves the use of the *EPM* approach, it is evident that there are a number of important practical problems for which this conceptual model is inappropriate and, in fact, some researchers have made attempts to combine approaches in order to achieve more realistic results. However, in consideration of the difficulties inherent to applying *DFMs* to fractured aquifers, where little information is available on fracture size and orientation, the *EPM* approach was used to represent the faults in the Carleton aquifer.

5.3.2 The Governing Equation and Mathematical Formulation

There are two conceptual views of groundwater systems - the aquifer viewpoint (two-dimensional) and the flow system viewpoint (three-dimensional). The aquifer viewpoint is based on the concept of confined or unconfined flow, in which groundwater flow is assumed to be strictly horizontal through the aquifer and vertically through the confining layer (confined case)(Anderson and Woessner, 1992). In this viewpoint, the aquifer hydraulic conductivity is integrated in the vertical dimension to give an average transmissivity for the saturated thickness of the aquifer. Since transmissivity and storativity are derived directly from the analysis of aquifer test data, these properties can be defined for the whole aquifer, and no estimate of aquifer thickness is necessary. In spatially heterogeneous aquifers, transmissivity can take on different values that vary with direction (x and y).

In unconfined aquifers, the transmissivity will vary according to the height of the water table (*i.e.*, according to the saturated thickness) and in leaky aquifers, the leakage across the confining layers can be represented as a source/sink term, which either adds or extracts water from the aquifer. In this situation, the actual hydraulic heads in the confining beds are not calculated, only their effect is simulated. To gain resolution in the vertical direction, it is necessary to model the aquifer from the flow system viewpoint.

At Carleton, there were limited data available to characterize the aquifers in three-dimensions. Therefore, a three-dimensional model would have been difficult, if not impossible, to calibrate. For these reasons, a two-dimensional model was considered. The mathematical formulation and the computer implementation were based on the equation for three-dimensional flow (*i.e.*, the flow system viewpoint) so that the code could be used for other problems. Hydraulic conductivities and specific capacities are replaced in two-dimensions with transmissivities and storativities, and the z-component drops out of the equations.

Equation 5.5 is the general form of the governing groundwater equation. For steady-state flow, this equation is modified to Laplace's equation, in which the right-hand-side becomes equal to zero. A leakage term may also be added to represent leakage in two-dimensional problems.

$$\frac{\partial}{\partial x} \left(K_x \frac{\partial h}{\partial x} \right) + \frac{\partial}{\partial y} \left(K_y \frac{\partial h}{\partial y} \right) + \frac{\partial}{\partial z} \left(K_z \frac{\partial h}{\partial z} \right) = S_s \frac{\partial h}{\partial t} - R^* \quad 5.5$$

In Equation 5.5, R^* is a general source/sink term that, if positive, defines the volume of inflow to the system per unit volume of aquifer per unit of time. R^* takes on a negative

value to represent outflow (as in pumping). In the aquifer view point, R^* is simply the volume of inflow/outflow per unit time.

The *method of weighted residuals* is one method that is used to solve boundary and initial value problems. This method, summarized in Appendix 7, and described in detail by Istok (1989), is widely used in groundwater flow and solute transport modelling and has been used in this thesis. The mathematical formulation of Equation 5.5 resulted in a system of algebraic equations that could be solved for the value of the field variable at each node in the mesh.

5.3.3 Finite Element Codes

Istok (1989) discussed the finite element method and provided several computer code listings for solving the groundwater flow equation under a variety of conditions; steady-state/transient, confined/unconfined, as well as for contaminant transport. The codes were designed in a modular format, namely, as a series of subroutines, which could be assembled to provide the user with a tailor-made modelling code. The codes were written in FORTRAN-77 and were intended for use on a microcomputer.

Although Istok (1989) stated that the programs were successfully compiled on a variety of mainframe and microcomputers, apparently the programmer is restricted in choice of compiler. Both Microsoft's FORTRAN compiler and Salford Software's FTN77/486 compiler (Salford Software Ltd., 1993) were used to compile the unedited code. It was found that significant changes had to be made to the code in order to compile and execute successfully. Many of these were, in a sense, cosmetic and dealt with the syntax for commands. However, there were some fundamental differences in what the

compilers would accept. These included EQUIVALENCE commands, that resulted in a vast restructuring of the programs. In particular, the definition of global variables and the passing of arguments into subroutines had to be modified. The code was also modified, to a large extent, by incorporating the heat transfer equations (refer to Chapter 6). In essence, only the backbone or structure of Istok's code was retained.

In total, three separate programs were written for this thesis (*of course, many versions of each were actually written*). The first program, GW1, calculates the steady-state solution for groundwater flow, the second, GW2, calculates the transient solution, and the third, GW3, calculates the combined heat transport and groundwater flow solution (refer to Chapter 6). These programs can be used for a variety of problems and are not restricted to the Carleton campus, since the mesh characteristics, initial and boundary conditions, etc. are all entered by means of a user-defined input file. All input data are contained in one input file and output data are written to one or more files, which have different output formats. For example, one output format contains x- and y-coordinates and the value(s) of the field variable(s) at each point. Another format contains time dependent variables at specified points.

A description of each of the codes (steady-state and transient), which were used to solve the groundwater flow equation by the finite element method, is provided in Appendix 7.

5.4 Discretization of the Problem Domain

5.4.1 General

The first step in any numerical method is to draw the grid (finite differences) or mesh (finite elements) such that the problem domain can be represented by a discrete number of elements and the solution calculated at a discrete number of points (nodes). In fact, it is advantageous to complete this exercise even before the code is selected, since it allows the modeller to select a code which is best suited to the available information. As stated in previous sections, there is little point in undertaking a three-dimensional model if the vertical resolution is not available. Since it has already been determined that the finite element method will be implemented, this section focuses on both the design of the conceptual model and on the construction of the finite element mesh.

5.4.2 Building the Conceptual Model

The purpose of building a conceptual model is to simplify the field problem and organize the associated input data so that the system can be analyzed more readily (Anderson and Woessner, 1992). The first concern is to define the physical boundaries of the model. It is important that the mesh cover a large enough area so that edge effects are not encountered, unless, of course, it is desirable to simulate the effect of the boundary. For example, if a constant head boundary (Dirichlet type) is placed too close to the active part of the mesh (for example, where pumping is taking place), the solution may show less effect on the drawdown than actually occurs, particularly at steady state. At the same time, the mesh should not extend beyond a distance where significant extrapolation of the available data will be required. Mesh design is discussed in Section 5.4.3.

Numerical models also require boundary conditions to be specified, and in accordance with this, either a flux or a head must be specified at each node around the problem domain. For the Carleton aquifer system, the problem of specifying boundary conditions was rather a difficult one, particularly since little information is known about the geological structure and the hydrogeology except in the immediate vicinity of the campus. Incorporating the faults proved to be one of the fundamental problems in modelling a domain of limited lateral extent. The faults are important geological features, but it was uncertain if they could represent model boundaries. Boundary conditions for the hydrogeologic model are discussed in Section 5.6.

The other fundamental problem concerned the faults themselves and how these geological features could be represented in a numerical model. The linear extent of the faults beyond the campus are uncertain, although there is evidence (Chapter 2) to suggest that the Eastern Block extends approximately one kilometre to the north and east of the campus. Geologically, the Gloucester Fault Zone extends northward several tens of kilometres to Pink Lake in the Gatineau Hills. However, the presence of cross-faults, such as the main cross-fault observed on campus, may interrupt or divert groundwater along this fault zone. In addition, specific intervals within the fault zone may have undergone varying degrees of movement that could result in discrete sections which could be essentially unconnected hydrogeologically.

Several authors have examined the hydraulic responses of fractured systems and have determined suitable conceptual models to describe qualitatively the movement of groundwater. In some regional studies of carbonate aquifers, it is recognized that single

fractures or faults are the prevalent features or mechanisms for controlling groundwater circulation. For example, Siddiqui and Parizek (1971) identified fracture trace intersections as the most favourable location for obtaining high yielding wells. This type of conceptual model was suitable for Carleton University, where the faults are the dominant control on the hydrologic regime.

The faults which pass through the Carleton campus are discrete geological features, which can be interpreted as planar entities of discrete width (vertical dyke flow model - Chapter 4). Because it is difficult to describe both the width of the brecciated zone surrounding the plane of breakage, and the extent to which individual fractures extend away from the plane of breakage, the faults are of undefined width. The aquifer test data indicated that these faults are perhaps best represented as vertical planar dykes, which have higher hydraulic conductivity than the surrounding aquifers. Of course, the faults may be anisotropic or heterogeneous. Anisotropy could be the result of a preferred orientation in fractures, or the degree of brecciation within the zone immediately surrounding the plane of failure. On the basis of the information available, it is virtually impossible to estimate the precise width of the fault.

However, it is possible to represent a fault hydrogeologically. For a two-dimensional model, it is assumed that there is no vertical anisotropy or heterogeneity. The aquifer is modelled using the aquifer view point and cells are assigned discrete values for transmissivity and storativity (T and S take into consideration the thickness of the aquifer). Any variation in thickness could, for example, be modelled in two-dimensions, by varying T and S accordingly. In representing a linear feature, such as a fault, increasing the

transmissivity or hydraulic conductivity for a fixed cell width effectively decreases the cell width without having to modify the mesh. In cases where the fault is of unknown width, this proves very useful. Storage changes will be negligible if the cell width is small. Using this interpretation, the faults can be characterized as a porous medium having a high hydraulic conductivity.

There are two ways that a fault can be represented using the *EPM* approach (Townley and Wilson, 1980). The first way is to use unconnected nodes to represent an impermeable fault. The second way, which is inherently more flexible, is to use small elements with assigned transmissivities. The transmissivity determines whether the fault is permeable or not. The aquifer test data at Carleton suggest that the faults act as low permeability boundaries between the aquifers on either side. The delays in response times observed between the Eastern Block and the Central Block during constant discharge tests are a manifestation of the barrier effect of the faults. At the same time, the faults are highly permeable along their lengths. There are several ways of incorporating these apparently dissimilar properties:

- 1) To consider the entire domain to be heterogeneous and anisotropic and to use the T and S values calculated directly from the aquifer test data. Of course, as noted in Chapter 4, it is difficult to assign a unique value of T and S to each element in the mesh (refer to Tables 4.3 and 4.4).
- 2) To represent the faults strictly as high permeability, homogeneous and isotropic features, in an otherwise homogeneous and isotropic aquifer regime.

- 3) To represent the aquifers on either side of the permeable faults (as in 2) with different transmissivities and storativities. The aquifers may be homogeneous and isotropic, but the system itself is heterogeneous and anisotropic.
- 4) To represent the faults with an anisotropic transmissivity. If the mesh is oriented such that the main faults are aligned parallel with the x- and y-coordinate directions, then by varying T_{xx} and T_{yy} for the elements which form the faults, the anisotropy can be taken into consideration. The implications of an anisotropic fault zone remain questionable at this time; however, the method may achieve the desired results.
- 5) To consider the faults as strictly geologic features, which separate different aquifers. The preferential connection exhibited between wells, in this representation, is due to horizontal planar fractures within the distinct aquifers. Of course, this approach does not explain either the variable well yields or the identical responses of shallow wells and adjacent deep wells to pumping.

There is sufficient indication in the results of aquifer testing to suggest that representations 1) and 2) do not apply (refer also to Section 5.5). The modelling results for each of representations 3) through 5) are provided in Chapter 8. To be able to test each one of these representations, the mesh needed to be designed to allow for maximum flexibility.

5.4.3 The Finite Element Grid

When preparing the mesh, it is important to remember that the precision of the solution obtained and the level of computational effort required to obtain a solution will be

determined to a great extent by the number of nodes in the mesh (Istok, 1989). Finer meshes (more nodes) add precision to the solution; however, as discussed later, the size of the arrays in the programs increases significantly and the limit of the computer can be surpassed. It is common practice to design a mesh where smaller elements are used in areas where the field variable is expected to change rapidly (high horizontal gradient). Pumping wells and faults are two examples of this type of situation.

The primary objective in the design of the finite element mesh used in this research was to achieve a certain degree of flexibility without using too many elements. Since the mesh was hand drawn, it was necessary to rely on regularly shaped elements, such as rectangles or triangles, which retained some consistency in their length to width ratio. The faults (shaded in Figure 5.1) were represented as narrow bands (two triangular-shaped elements forming a rectangle). The aquifers on either side of the faults were represented by slightly larger double-triangle blocks. An example of the element configuration is provided in Figure 5.1.

The node numbers range from 1 to the total number of nodes in the mesh (*i.e.*, there are no skips in the numbering). Each node is also defined by its nodal coordinates

(Figure 5.1). Three nodes are used to define each element (triangular) and each element is also given a unique value ranging from 1 to the total number of elements in the mesh.

Several basic rules were followed in constructing the finite element mesh. These included: placing nodes at well locations, placing nodes close together in those parts of the problem domain where the field variable was expected to change rapidly (for example, at wells or near faults), placing nodes along the interface between two different materials, avoiding gaps and overlaps between elements, and using a transition region to achieve a gradual change in element size. To improve the efficiency of the program, the nodes were numbered consecutively up the shortest dimension of the mesh (Figure 5.1). This was done to minimize the semi-bandwidth of the resulting system of linear equations.

The final mesh design (Figure 5.2) consists of 6120 elements and 3184 nodes and features a series of mutually perpendicular fault zones. The element widths telescope from a 1:1 width to length ratio (20m:20m) in the matrix aquifers down to a 1:4 width to length ratio (5m:20m) in the fault zone. The term 'matrix aquifer' refers to the unfaulted (or sparsely fractured) aquifer surrounding the fault zones. Elements of intermediate width surround the fault area and have a ratio of 1:2 (10m:20m). The fault elements extend from one domain boundary to the opposite edge and are perfectly parallel and perpendicular.

In modelling faulted areas, it is inherently difficult to construct a mesh that can incorporate the faults, while minimizing the effects of boundaries on the interior of the domain. This is particularly true for Carleton because the faults are of indefinite length (at least, they appear to extend beyond the campus). As the fault elements do extend to the edges of the domain, it is anticipated that boundary conditions will strongly influence the

Figure 5.2: Finite Element Mesh for the Computer Models

solution. Nevertheless, the influence of the boundary conditions may be advantageous, particularly as late time leakage was observed during aquifer testing (Section 5.6).

Limiting the size of the mesh was necessary to avoid extrapolation of the problem domain beyond the immediate campus area, where there was little control on geological and hydrogeological properties, as well as to limit the number of equations that had to be solved (a computer memory problem). For these reasons, the mesh coincides with the general shape of the campus, but extends at least 140 metres beyond production wells. During aquifer test simulations, the zone of influence for the pumping well may approach the edge of the domain; however, during simultaneous pumping and reinjection simulations, the zones of influence for all wells is expected to remain within the bounds of the domain.

With this mesh design, the model can, at best, only simulate the system's internal hydraulic behaviour, with boundary conditions for the model that are selected on the basis of trying to curve match the observed responses of wells with simulated responses during pumping. In such a model, the hydrogeology external to the system may not be physically-based.

Well Placement and Faults:

In reality, the main faults (Western Fault, Central Fault and Eastern Fault) are not parallel, and the cross-faults are not perpendicular to the main faults (refer to Figure 4.3). However, it was necessary to use this configuration to simplify the mesh construction. In particular, it would be extremely difficult to determine the x- and y-coordinates for an

unevenly spaced grid, for roughly six thousand elements, without using a mesh generator software package.

Bearing in mind that one objective of this research was to model a faulted limestone aquifer and describe the behaviour of the faults, both in terms of their hydraulic response and their thermal response, the approximation made to the actual position of the faults was not expected to have significant consequences. The only concern was the proximity of wells to the faults. For example, well P21 is located very near to the Eastern Fault, but is in the Central Block (*i.e.*, it lies to the west of the Eastern Fault). This same fault also passes very near well P2. The Central Fault is at an angle to the Eastern Fault and passes close to well P21. It also passes near well P23, such that excellent hydraulic connection is observed between wells P21 and P23. The result is a wedge-shaped fault block whose point is near well P21 (refer to Figure 4.3). Since the grid incorporates parallel faults, the hydraulic response for wells P21 and P23 could be better achieved by displacing the wells on the mesh from their actual positions. The position of a well away from a fault is more significant to its hydrogeologic response than its proximity to an aquifer matrix well. That is, moving the well to account for the parallel faults has little effect on its connection to other aquifer matrix wells. To this end, well P21 was moved to the west, and well P23 was moved to the east by approximately 20 metres. The positions of the wells on the finite element mesh, excluding the Phase 3 wells because there were no aquifer test data to use in the calibration, are shown in Figure 5.3.

5.5 Parameter Estimation - A First Approximation

Assigning values to the hydraulic parameters can be a very difficult task, particularly if little control is available (*i.e.*, when there is a poor distribution of known values for the transmissivity and storativity). For a homogeneous and isotropic aquifer, there is one value for transmissivity and one for storativity, which uniquely characterize the entire aquifer. In this case, data derived from one aquifer test can be used to estimate the hydrogeologic parameters. However, as the aquifer becomes more heterogeneous, there are several combinations of T and S which can potentially reproduce an aquifer response. Since it is impossible to determine these parameters exactly for each portion of the aquifer, it is necessary to estimate them on the basis of known information. This information often includes aquifer test data, but can also include stratigraphic information, particularly in three-dimensional models. For models which are to be used for predictive purposes, it is critical that a good representative sample of aquifer parameters be acquired. Since it is impossible to sample the aquifer on a continuous basis, point or discrete values are depended upon. In many instances, interpolation algorithms, such as kriging, are used to interpolate (and extrapolate to a limited extent) the known hydraulic parameter field. In the author's view, such algorithms should be used with caution since they can create artifacts from poor sample points and they can smooth out features which may be, in reality, discrete anomalies. The choice of algorithm and the limits of the algorithm must be carefully scrutinized.

In general, a range of values for each hydraulic parameter, determined from aquifer test results, is specified. This range is essentially a probability distribution, whose mean

value is of interest for the purpose of constructing a model. As with any statistical distribution, the greater the sample size, the better defined is the actual mean value. However, as the sample size increases, so does the possible range of values. If the range is great, then it becomes increasingly difficult to calibrate the model. The actual value used in the model is arrived at through the calibration process. Calibration can be looked upon as fine-tuning the model. It should not be used to change drastically the values estimated from field data.

The aquifer test data for Carleton were used to determine preliminary estimates of T and S for the model. The early time aquifer test data (drawdown versus log time), for several wells, are non-linear due to faulting, and represent a parallel or linear flow regime (Chapter 4). The late time data show a change in slope due to leakage. At mid-time, the flow regime becomes approximately radial (the linear portion on a semi-log graph) and water is derived from the matrix aquifer surrounding the faults. The values for T and S , derived from the Jacob analysis probably reflect the surrounding aquifer, at least partially.

For the Carleton aquifer system, there is a large range in transmissivity and storativity values calculated from these test data (refer to Tables 4.3 and 4.4). This large range is primarily the result of faulting. High values of transmissivity are calculated along faults, as a result of the excellent connection, and even higher values are calculated across fault boundaries. The high transmissivities measured across the faults are the result of significant delays in response times for wells located on the opposite side of a fault to the pumping well (Chapter 4). These delays may be caused only by a difference in hydraulic properties between the Central and Eastern Blocks, and they may be associated with the

nature of the hydraulic properties of the fault zone. In the first case, a series of high hydraulic conductivity (transmissivity) isotropic and homogeneous faults could be superimposed on two unfaulted homogenous matrix aquifers to reproduce both the preferential hydraulic connection along the faults and the delay times across the faults. In the second case, the faults would require hydraulic properties that represent, for example, anisotropy within the fault zone. These two representations for the aquifer properties (two conceptual models) are tested in this research (Chapter 8). The preliminary estimates of T and S for both the matrix aquifers and the faults, presented in this section, are based on the first conceptual model.

To estimate the transmissivity and storativity of the unfaulted aquifer, the values calculated at observation wells, which were away from faults and which were located within the same fault block as the pumping well, were used. For example, in the Central Block, wells P1 and P4 are not located along faults. From Table 4.3, the only available T values are for pumping P1, observed at P1 ($62 \text{ m}^2/\text{day}$), pumping P4, observed at P1 ($63 \text{ m}^2/\text{day}$) and pumping P4, observed at P4 ($56 \text{ m}^2/\text{day}$). The average value is $60 \pm 4 \text{ m}^2/\text{day}$. This average value also corresponds roughly to the M1-2 through M1-5 intervals during pumping of wells P1 and P4 (refer to Table 4.3). The average storativity value was calculated using the same approach. However, since storativity is undefined for a pumping well, only one value was available. This value is for pumping P4, observed at P1 ($4.2\text{E-}5$). These two aquifer values are probably good estimates of the Central Block aquifer T and S , since the aquifer test data for these wells show the least amount of non-linearity and the straight line portion of the drawdown versus log time graph is clearly defined.

For the Eastern Block, many of the wells are located along faults and, as a result, most of the early constant discharge test data are highly non-linear at early time. This non-linearity also extends into mid-time. Therefore, the straight line portion was more difficult to define. Nevertheless, the straight line portion is representative of the aquifer response, rather than the response of the faults. Using the same approach as used in the Central Block, but for wells P2, RC, P3, P5, P6, Elba1 and Elba2 (24 values), the average T is $171 \pm 13 \text{ m}^2/\text{day}$. Values for P22 were not included because they were higher. As noted in Chapter 4, reinjection at P4 during the P22 aquifer test may have created unrealistic aquifer test results. The average S value for the Eastern Block is $2.8\text{E-}4$. Of course, this value may be unrealistic because response time does not increase as radial distance increases from the pumping well.

Section 5.4.2 provided several aquifer representations which could be tested in the modelling. The third representation, that is, the one for a heterogeneous aquifer (two fault blocks having different T and S values) with highly transmissive faults separating them, will be considered initially. The preliminary estimates of the hydraulic properties of the aquifers were:

Table 5.4: Preliminary Estimates of Aquifer Parameters for Model

Central Block	Eastern Block	Faults
$T=60 \text{ m}^2/\text{day}$	$T=171 \text{ m}^2/\text{day}$	$T=1000 \text{ to } 10000 \text{ m}^2/\text{day}$
$S=4.2\text{E-}5$	$S=2.8\text{E-}4$	$S<1.0\text{E-}5$

The choice of parameters for the faults was somewhat arbitrary, since there are no data to confirm the values. Analytical models for vertical fractures assume T is infinite in the fracture and S is zero (refer to Chapter 4). Although both T and S , in the vertical dyke

model, have finite values, these are expected, respectively, to be at least an order of magnitude greater than and less than the aquifer. Calibration should provide control over their magnitude.

Figure 5.4 shows the initial representation for the aquifer along with the assigned aquifer parameters. Later modifications to this representation, as determined through calibration, are provided in Chapter 8. The Central Block is unshaded and the Eastern Block is shaded light gray. Faults (black) are isotropic and homogeneous and are 5m wide.

5.6 Boundary and Initial Conditions

5.6.1 Boundary conditions

The correct selection of the boundary conditions is a critical step in model design and this step is often the source of greatest error. In a steady-state model, the boundary conditions determine the flow pattern. Water flows in one part of the domain and exits from another. Therefore, the calculated hydraulic head distribution for a steady-state flow model is a direct consequence of boundary inflow and outflow. The boundaries for the model must be selected so that the physics of the system are represented at the boundaries before any stresses, such as pumping, are applied.

When a stress is applied to the model, the boundary conditions should not influence the solution. If the area of interest is small relative to the size of the domain modelled, and if the simulations are stopped before perturbations reach the boundaries, the area of influence of a pumping well will not reach the perimeter of the modelled area. Under these conditions, either no flow or constant head boundaries can be used.

Figure 5.4: Diagram Showing Property Sets Used in Preliminary Model

Usually, natural hydrogeologic boundaries, such as groundwater divides, constitute no-flow boundaries. Similarly, rivers flowing along one edge of the domain (unconfined), can be assigned variable but constant head values, as long as the level of the river is expected to remain constant. Because the aquifer at Carleton is confined, surface water bodies cannot be used as constant head boundaries. In addition, groundwater divides cannot be easily identified around the campus. Nevertheless, groundwater flow patterns for the region and the Carleton campus do suggest that faults separate areas of different flow geometries (Figure 3.1). Although the faults may be semi-permeable, at least at Carleton, they are probably the only geological features that can constitute hydrogeologic boundaries.

The problem with using faults as boundaries is that many of the faults on campus are essential to the model. For example, the Eastern Fault cannot constitute a boundary because there are wells located near it on both sides. Similarly, the Central Fault cannot constitute a boundary because it transects the Central Block. Furthermore, a problem arises if the faults on campus are treated as no-flow boundaries. In this case, when the area of influence of a well approaches a fault, the fault will act as an impermeable boundary, thereby increasing the modelled drawdown. A no-flow boundary, corresponding to a fault on campus, may give unrealistic solutions because the faults are semi-permeable.

One alternative is to select the boundaries of the domain in the opposite fault block relative to the pumping wells. In this way, the effect of the model boundary will be minimized, while retaining the important hydrogeological characteristics of the fault. This approach was used for the western boundary of the Central Block, because the hydraulic

connection west of the Western Fault is expected to be minimal (refer to Figure 4.3). The approach could not be applied along the northeast, north and east sides of the domain because there are no faults within approximately one kilometre of the campus on all three sides. Using these distant faults would require additional hydrogeologic data beyond the bounds of the campus. These data are not available. Therefore, the domain was restricted to the actual campus.

During constant discharge testing, leakage, which appears as a constant head (derivative analysis of aquifer test data), was observed at late time. The corresponding change in slope at late time may correspond to water entering the system along one or more of the faults which transect the campus. Alternatively, the water may come from adjacent fault blocks, which are separated from the Carleton system by (distant) faults. Cross-block leakage is observed between the Central and Eastern Blocks on campus, therefore, it may also occur between the Eastern Block and blocks located to the northwest, north and east, and possibly to the south.

The source of the leakage cannot be determined on the basis of the aquifer test results. Therefore, it was necessary to use various modelling approaches (steady-state and transient simulations) in an attempt to establish boundary conditions that could reproduce the observed static hydraulic head distribution in addition to the transient responses of wells during pumping. This implies that the boundary conditions for the model will be non-unique, and perhaps not physically based.

The flow direction for the campus, established from the static hydraulic head distribution (Figure 3.3), is predominantly toward the southwest. One possible

configuration for this flow pattern consists of groundwater entering the flow domain along the top (northern and eastern) boundaries (Figure 5.4), encountering the Central Block (delineated by the Eastern Fault and the major cross-fault), gradually making its way into the Central Block, and then leaving the domain along the southwest boundary. The boundary conditions used to represent this type of flow regime could consist of either two constant flux boundaries, one along the northern edge and one along the southwestern edge; two constant head boundaries, one along each of these edges; or a combination of constant flux and constant heads. These various combinations of flow boundaries can be applied to the domain and the resulting static head distribution compared to the observed values.

5.6.2 Steady State Solution Using *FLOWPATH* - Finite Difference Method

Attempts were made initially to use the steady-state finite element portion of the thesis software to examine boundary conditions. This proved to be a cumbersome undertaking, as the output data file was too large to scan for trends. Importing the data file into a plotting package, such as *SURFER* (Golden Software, Inc., 1994), was extremely time consuming. The simplest way to establish the model boundary conditions was to use a commercial software package. This approach permitted easy modification to boundary conditions (graphical interface) and also displayed graphically the results of the simulation.

The software package, *FLOWPATH* (Waterloo Hydrogeologic Inc.), was used to establish the approximate boundary conditions. The software uses the finite difference method to calculate the hydraulic head distribution at steady-state. It also features an

excellent graphical interface, which permits the modeller great flexibility in constructing and modifying the finite difference grid, and allows for all types of boundaries (Dirichlet, Neuman and Cauchy (head-dependent)), sources and sinks for groundwater (*i.e.*, wells), different material properties, recharge/evaporation variations, etc. The solutions are plotted as equipotential lines and/or as velocities, and a graph showing observed versus calculated hydraulic heads at selected points may be plotted.

To use this software, the finite element mesh (Figure 5.2) had to be redrawn (within the *FLOWPATH* program) as a finite difference grid. Since rectangles were divided into equal area triangular elements, this conversion was straightforward. The finite difference grid looked very similar to the finite element mesh shown in Figure 5.2, with the exception that the grid was rectangular. Dimensions for rectangles were retained. Once the boundary conditions were established, or at least a first approximation to them determined, the steady-state finite element program was run. The results of the *FLOWPATH* simulations are provided in Chapter 8.

5.6.3 Initial Conditions

Ultimately, a transient model for the Carleton campus was desired so that the hydraulic and thermal interactions, between pumping and reinjection wells in the operating ATEs system, could be determined. Initial conditions must be specified for any transient solution (Section 5.6.3) and it is common to use the static hydraulic head distribution at steady-state for these initial conditions. Therefore, the static hydraulic head distribution must be determined. Since only a few spot measurements were available, it was necessary to interpolate and extrapolate these in order to provide values for each node in the mesh.

The most common method of doing this is to solve the steady-state groundwater flow equation for each node in the mesh using a set of boundary conditions.

New initial conditions can be specified to reflect different starting conditions for the transient simulations; however, the accuracy of the initial conditions diminishes as the departure from steady-state increases. For example, in examining system reversal (*i.e.*, the change from heating season to cooling season or *vice versa*), the initial conditions are not the undisturbed hydrologic and thermal regime. The previous pumping/reinjection configuration has resulted in a hydraulic high around the reinjection wells, which at an instant in time, becomes a hydraulic low (pumping). Likewise, the thermal front is positioned somewhere away from the former reinjection well. If there are few monitoring points, the initial conditions cannot be determined from field data. They must be calculated from a former simulation of the system. The error in the new solution becomes greater.

This increasing error with time is a fundamental problem with any type of model because of the uncertainty in the large number of parameters (hydraulic and thermal properties, initial conditions, and boundary conditions) that must be defined. It is for this reason that models should never be used to predict what will happen in a true system much beyond the limits of the calibration time period. A model, which has only been calibrated against steady-state data, should be used strictly to model low frequency variations in the hydrogeologic regime, such as changes in recharge or perhaps long-term adjustment of the aquifer to pumping. A model, calibrated against different hydraulic conditions that, for example, result from stressing the aquifer (*i.e.*, pumping), becomes a more reliable tool for making predictions. This type of model could be used to examine the

effect of different pumping configurations. As the quality of calibration increases, so does the ability of the model to predict into the future.

For a heating and cooling system, pumping and reinjection wells undergo a reversal in flow direction twice over the course of one year of operation. To model the long term (several years) behaviour of the system would require both hydrologic and thermal data for a period of at least one year. Even if these data are available, the small-scale departures of the model from real conditions within the aquifer may result in a solution that is no longer representative of the aquifer after the first year of operation. In this instance, extension of the model over several cycles of operation would likely cause large departures from reality.

A second point for consideration, in defining the initial conditions for the simulations, is that two aquifer testing programs were conducted on campus. The first of these was prior to 1989, when the system had not yet been put into commission. All aquifer tests conducted prior to 1989 were presumably conducted from a steady-state, undisturbed condition. It was stated in Chapter 4 that during the 1994 aquifer testing program some of the wells had been shut down immediately before testing and two (P1 and P4) remained operational during aquifer testing at wells P21, P22 and P23. Since this configuration had been used for several months, it was presumably under steady-state conditions. However, it is obvious that the initial conditions for the 1989 tests and the 1994 tests were not the same, and so the models should have different initial conditions. For this reason, a greater emphasis was placed on the calibration results for the pre-1989 aquifer test data.

5.7 Time Discretization for Transient Model

5.7.1 Numerical Instability and Oscillation

In addition to discretizing the spatial domain, a transient numerical model requires the time domain to be discretized. Just as it is computationally intensive to solve for the field variable at every point in space, it is often impractical to solve for the field variable at time intervals which are significantly less than the time period of interest. However, there are circumstances where added temporal precision is necessary. Ladbury (1996) used over 2000 hours of supercomputer time to generate a transient model of the Earth's geomagnetic field (a working geodynamo). This model simulated approximately 38,000 years of the Earth's history in 1 week time intervals. Although this time interval seems very small, it was necessary for model stability. Simulating complex systems, particularly those with non-linear behaviour, requires careful selection of the time intervals used in the model runs.

Normally, simulations begin with small time steps in order to retain accuracy in the time period where the largest variation in the field variable is anticipated. Of course, the size of these time steps is relative to the overall simulation time. For a 1 day simulation, time steps may be initially on the order of seconds or minutes, with the time interval increasing as the simulation proceeds. This procedure is followed during aquifer testing when hydraulic heads are monitored in increasing time intervals. For transient simulations covering a period of 100 years, the initial time intervals may be on the order of days and may increase to months and years.

The objective in choosing time steps is to maintain a balance between computational intensity (*i.e.*, long simulations) and numerical accuracy. The size of the time step required to obtain useful results depends on the size and shape of the elements in the mesh, the values of specific storage and hydraulic conductivity for each element, whether a lumped or consistent formulation (Appendix 7) is used to compute the element capacitance matrices, the boundary conditions, and the value of the relaxation factor or time-weighting function (Appendix 7), ω (Istok, 1989). The first type of problem that can occur is a violation of “reality”. In this situation, hydraulic heads may increase near point sinks (*i.e.*, near pumping wells). This can be avoided by choosing ω and Δt so that the coefficient matrix, $[M]$, is diagonally dominant.

The second type of problem is instability. Instability occurs when the difference between the true solution and the numerical solution grows extremely large in a few time steps. Choosing $\omega \geq 1/2$ produces an unconditionally stable solution (Istok, 1989). The third type of problem is numerical oscillation. Numerical oscillation occurs when the computed values of the field variable fluctuate about the true solution. From one time step to the next, the numerical solution is alternatively above and below the true solution. The problem of numerical oscillation can be avoided if the criterion for stability is satisfied (Istok, 1989):

$$\Delta t < \frac{\alpha}{1 - \omega}, \quad \omega < 1 \quad 5.6$$

where α is the *smallest* number (for any element) that satisfies the equation (the smallest number of α will occur in the smallest element in the mesh):

$$\left[[C^{(e)}] - \alpha [K^{(e)}] \right] = 0 \quad 5.7$$

where $[C^{(e)}]$ and $[K^{(e)}]$ are defined in Appendix 7. Solving equation 5.7 for α using a consistent formulation of the element capacitance matrix $[C^{(e)}]$ is computationally intensive and will not be carried out. However, it has been demonstrated that for $\omega=1$, no numerical oscillations will occur (Istok, 1989). Numerical oscillations can usually be prevented by decreasing the time step according to the following criterion for rectangular elements:

$$\Delta t_c = \frac{S^{(e)} a^2}{4T^{(e)}} \quad 5.8$$

where Δt_c is the critical time step at which no oscillations should take place and a is a characteristic dimension of the element ($a=\Delta x=\Delta y$). To avoid both instability and numerical oscillation, ω was set equal to 1 for all of the simulations.

Problems with instability and oscillation are common in unsaturated flow problems and in problems involving solute transport. A similar instability may occur in heat transport simulations. The most common phenomenon is “numerical dispersion”, which refers to artificial dispersion caused by errors associated with the discretization of the problem domain. Anderson and Woessner (1992) suggested that in order to minimize such errors, the grid should be designed so that the Peclet number ($P_e=\Delta l/\alpha$, where Δl is a characteristic nodal spacing and α is the dispersivity (Chapters 6 and 7)) is less than 1. It is usually recommended that the grid be designed so that $\Delta l < 4\alpha$ (Anderson and Woessner, 1992).

In the unfaulted aquifers, the element size is 20m x 20m. The dispersivity (as discussed in Chapter 7) was a difficult parameter to quantify since its magnitude varies

with the scale of the domain. In general, the larger the scale, the higher the dispersivity because of increased heterogeneity. Given that the domain size is roughly 1 sq. km., the dispersivity could be as high as 10-100m. Within this range, the element size is satisfactory. For lower dispersivities, the elements in the aquifer are not small enough. Along the faults, the element length remains at 20m and the width is reduced to 5m. A similar range in dispersivity gives satisfactory results for elements of this size.

Likewise, care should be taken when discretizing time so that the Courant number ($C = v \Delta t / \Delta l$) is less than or equal to one (v is the velocity of the water). That is, the time step should be chosen so that $\Delta t < \Delta l / v$. The difficult value to assess in this expression is the velocity. Along the faults, the velocity may be very high, particularly at early pumping times. If an initial gradient of 30m/300m (0.1) is established along a fault, and the hydraulic conductivity is near 10^{-2} m/s, the average linear velocity, for an aquifer with a porosity of 10%, will be 10^{-2} m/s. This velocity will quickly decrease as a result of the excellent connection between wells along a fault (*i.e.*, the gradient will be reduced significantly). Using these values, the initial time steps should be no greater than 30 minutes, which is a manageable time step.

5.7.2 Time Steps for Simulations

The final consideration in this chapter is the actual time step intervals over which the simulations were run. Model calibration was carried out by examining the transient response of both the pumping well and several observation wells located in different fault blocks. In addition to total drawdown at the end of the test, the delay in response time and the drawdown at various times were considered. Since the shape of the drawdown versus

time data was of primary interest, the time increments had to be selected to take advantage of the exponential relationship. Therefore, the time steps used for calibration of the transient groundwater flow model were similar to those used in actual aquifer tests (Table 5.5).

Table 5.5: Time Steps Used for Model Calibration

Time Step	Duration (Interval) (seconds)	Number of Time Steps	Cumulative Simulation Time (seconds)
1	120 (2 min)	5	600 (10 min)
2	1200 (20 min)	10	12600 (3.5 hours)
3	7200 (2 hours)	10	86400 (1 day)

Following the calibration, which included 10 constant discharge tests, several reinjection tests were simulated. Similar time steps were used.

Although the heat transport component of this research has not yet been discussed (Chapters 6 and 7), the time discretization is established here. For the heat transport simulations, the time frame was extended such that simulations were run over an entire heating or cooling season. It is recognized that the accuracy of the solution may be in question because the simulation continues beyond the limits of calibration (Section 5.6.3). However, since no attempts were made to reverse the system during simulation, the accuracy has likely been maintained. In the preliminary heat transport simulations, similar time steps as those established during calibration were used. Beyond twenty-four hours, the time step increment was increased to 1 day, for a total simulation time of 180 days. The time steps used for heat transport are shown in Table 5.6.

Table 5.6: Time Steps Used for Heat Transport Simulations

Time Step	Duration (Interval) (seconds)	Number of Time Steps	Cumulative Simulation Time (seconds)
1	600 (10 min)	6	3600 (1 hour)
2	3600 (1 hour)	23	86400 (1 day)
3	86400 (1 day)	179	1.5552E+7 (180 days)

5.8 Summary

The background to both the analytical and numerical approaches, that were used to model the Carleton aquifer system, was presented in this chapter. In both instances, the conceptual model assumes that both the aquifers and the faults can be represented as Equivalent Porous Media (*EPM*). Heterogeneity of the system in the numerical model is incorporated by assigning different hydraulic properties to each of two matrix aquifers, corresponding with the Central and Eastern blocks, and by assigning high transmissivities to isotropic fault zones. The framework for hydrogeologic models, including spatial discretization, time discretization, and initial conditions, was also established in this chapter. However, the boundary conditions could not be established because of the lack of geological information for areas surrounding the campus. Boundary conditions appear to be represented by constant heads, as leakage is observed in constant discharge test data; however, the source for leakage cannot be identified. On this basis, constant head boundaries are placed around most of the perimeter of the domain, and both steady-state modelling and transient modelling will be used (Chapter 8) to determine whether this representation for the boundaries can reproduce the observed hydraulics of the system.

6. THERMAL MODELLING

6.1 ATES Applications

6.1.1 Background

The concept of heat storage in aquifers dates back over thirty years, when cold storage applications were initiated in China in 1965. These early ATES (Aquifer Thermal Energy Storage) systems were implemented to combat the lack of summer cooling in factories, but problems related to ground subsidence and groundwater pollution occurred. By 1984, there were 492 cold storage wells in Shanghai using a total of 29 million cubic metres of water annually (Morofsky, 1994). Over ninety percent of these wells were used for both injection and extraction.

In response to the 1970's energy crisis, active research in Aquifer Thermal Energy Storage (ATES) was initiated in the United States. A series of experiments, involving the recovery of warm water from storage, were conducted at Auburn University in Alabama (Molz *et al.*, 1978; Sykes *et al.*, 1982; Parr *et al.*, 1983) and at Texas A&M (Davidson *et al.*, 1979). Although research has continued on an international basis since those early experiments, many of the research papers now focus on case histories and success stories, advances in hydrogeological, chemical and thermal modelling of ATES systems, and site specific problems and solutions. The years of research have shown that ATES can be successfully implemented under a variety of site conditions, yet the decision-makers and designers of Heating Ventilation and Air Conditioning (HVAC) systems are slow to embrace this technology (Brett and Midkiff, 1994).

Sweden is amongst the leaders in ATES techniques (Andersson and Sellberg, 1992; Probert *et al.*, 1994; Claesson *et al.*, 1994), along with The Netherlands (van Mourik, 1992; van Loon, 1992; Willemsen *et al.*, 1994), Germany (Hahne, 1992; Koch and Ruck, 1992), France (Iris *et al.*, 1991; Sauty *et al.*, 1992), the United States (Midkiff *et al.*, 1992; Hoyer and Sterling, 1992; Vail *et al.*, 1992; Midkiff and Brett, 1994; Jenne *et al.*, 1994; Wilke, 1994; Winters *et al.*, 1994) and Denmark (Sorensen and Reffstrup, 1992; Reffstrup *et al.*, 1994). In fact, in The Netherlands, government-based funding is available for feasibility studies aimed at incorporating ATES into energy programs. These programs have been implemented by the government to promote the technology (Willemsen *et al.*, 1994). Although Canada is not at the forefront of this technology, several large-scale applications have been implemented. Included in these are the Scarborough Centre in Scarborough, Ontario (Morofsky *et al.* 1992; Mirza *et al.*, 1994), the Sussex Hospital in Sussex, New Brunswick (Cruickshanks, 1994; Williams, 1994), the Winpak building in Winnipeg, Manitoba (Morofsky, 1994) and Carleton University in Ottawa, Ontario (Minty and Morofsky, 1988).

Canada is actively involved in developing this technology, and is a contributing member to a group of scientists involved with investigating ATES. Under the auspices of Annex VII of the International Energy Agency (IEA) (Chant, 1991; Chant and Morofsky, 1994), the research and development program is specifically targeted on thermal storage for cooling. The primary objective of Annex VII is to identify, analyze and document systems which maximize energy savings and environmental benefits from thermal energy storage applications. Both the environmental and economic aspects of ATES relating to

energy conservation and subsequent reduction in air emissions have been evaluated by Cruikshanks *et al.* (1993).

Canada shares the results of basic research and technical expertise with Denmark, Finland, France, Germany, Japan, The Netherlands, Sweden, Switzerland and the United States of America. Those countries that do not share our extreme climate swings are not as interested in the IEA project; however, several of these, including the United Kingdom, are actively involved in higher temperature geothermal energy.

6.1.2 ATES System Requirements

Energy requirements for single buildings or for building complexes are extremely variable since they depend not only on the actual building characteristics (e.g., square footage), but also on the climate. In the southern United States, for example, there is little demand for heating and most of the energy is consumed through air conditioning. Therefore, it is essential that the energy requirements for the building(s) be determined prior to any exploration for a suitable aquifer. This portion of the system design is generally undertaken by an HVAC (Heating, Ventilation and Air Conditioning) engineer and, therefore, will be discussed only in the most general terms.

Very simply, given the heating and cooling load for the building(s), the HVAC engineer calculates the volume of groundwater, at a specific temperature, that is needed to provide this energy. A plate heat exchanger system is then designed that will transfer the energy from the groundwater to the building and vice versa. In the heating season, energy is transferred from the groundwater to the building, and in the cooling season, energy is transferred from the building to the groundwater.

The basic operation of such a system requires two plate heat exchangers. The first heat exchanger transfers energy from building air to circulating water (or glycol). The second heat exchanger transfers energy from the water (or glycol) to the groundwater. The temperatures of the media on either side of the heat exchangers are necessarily different. Thus, a temperature differential is established. Conventional chillers, which operate on this basic principle, use once-through municipal water. The “waste” water is discharged to a river or to a storm sewer.

In most parts of Canada and the United States, the heating load in the winter is comparable to the cooling load in the summer. If the heat that is extracted from the buildings in the summer can be stored and later recovered, then over the course of one year, the total energy consumption should be reduced. All that is required is a suitable storage medium and a method to transfer the energy to and from it.

The aquifer is the storage medium and the groundwater is the medium used for transfer. Once the requirement of groundwater volume per unit time is established, the search for a “suitable” aquifer is initiated. What constitutes a “suitable” aquifer depends on the energy requirements. Several considerations for selecting a “suitable aquifer” are outlined below.

- All ATES systems require the ‘suitable’ aquifer to be located directly beneath (or at least, very near to) the building or group of buildings to be serviced (Hall and Raymond, 1992). The aquifer may be confined or unconfined and in consolidated or unconsolidated deposits, but it must have hydraulic properties conducive to supplying a

sufficient volume of water. This means that aquifers should have moderate to high hydraulic conductivities (or transmissivities) and adequate well yields.

- The natural hydraulic gradient must, in general, be low in order to insure that the thermal energy is contained. The aquifer must also be of a sufficient volume. An aquifer of small volume, for instance one that is too thin, will be unable to provide sufficient storage capacity without 'thermal breakthrough' occurring between the warm and cold wells. Thermal breakthrough means that the thermal front, generated at the reinjection well, arrives at the pumping well before the respective heating or cooling season has been completed. For instance, if cold water is extracted from an aquifer and used for summer cooling while the warm water is reinjected into the aquifer, the thermal front (warm) progresses outward with time from the reinjection well. If this front reaches the pumping well before the end of the summer, warm water will be extracted because thermal breakthrough has occurred. There are varying degrees of thermal breakthrough, some of which can adversely affect the operation of the system. The implications of thermal breakthrough and the rates of thermal breakthrough calculated using analytical methods will be addressed in the following section.
- The groundwater must be of a suitable chemical quality to minimize corrosion, scaling and well clogging. In cold or 'cool' ATEs applications (*i.e.*, less than 20°C), the temperature change in the aquifer is not likely to induce adverse biochemical or physical effects; however, precipitation of carbonate, in particular, may occur. Several researchers have made significant advances in providing solutions to corrosion and

scaling problems (Koch and Ruck, 1992) and some have developed computer software to examine the hydrogeochemistry of aquifers used for ATES (Jenne *et al.*, 1994).

- The ambient temperature of the aquifer is also of concern. In the southern United States, groundwater temperatures are high relative to those in Canada. These ambient groundwater temperatures (18°C) are insufficient for cooling during the summer months. Therefore, water has been cooled (to 6°C in a cooling tower) during the winter months (Midkiff *et al.*, 1992) and stored in the aquifer for use the following summer. Recent modifications to this system include a spray pond in replacement of the cooling tower (Midkiff and Brett, 1994). In southern Canada, the ambient groundwater temperature falls within a range of approximately 6-10°C. This range is ideal for meeting both the cooling requirements and the partial or pre-heating requirements for all buildings.

Hall and Raymond (1992) characterize a suitable aquifer for ATES as one that provides:

1. A sufficient water supply, that can be successfully withdrawn and reinjected, to meet the requirements of the system. This condition is determined by first identifying the aquifer by means of test drilling, and then carrying out a complete hydrogeologic evaluation of the aquifer by aquifer testing. This permits the evaluation of the hydraulic conductivity for the aquifer and the specific capacity of the wells.

2. An effective hydrologic environment for thermal energy storage. This is partly determined by assessing the natural groundwater velocity, as well as modelling the heat transport associated with water flow in the aquifer.
3. A sufficient effective porosity, such that the heated or cooled groundwater can effectively release heat into storage in the aquifer rocks.
4. Minimal adverse effects on the aquifer and water chemistry due to temperature changes. This is determined by sampling water for chemical analysis, and then calculating the chemical changes expected as a result of any temperature changes.

At the outset of the Carleton Project, thermal logging of several shallow wells was conducted. In addition, hydraulic testing of these shallow wells and drilling of a deep exploratory well (Elba2) were undertaken. It was concluded that sufficient water was available for a large-scale ATES application and that this water could be used effectively and safely (from a chemical point of view) as the medium for thermal transfer between campus buildings and the underlying confined aquifers (WESA, 1985, 1986, 1988). Although consideration was given to the fractured nature of the system, the design of the system, from a hydrogeological point of view, was based primarily on the results of two analytical models, which were employed separately to simulate groundwater flow and heat transport in the aquifer.

As discussed in Chapter 5, analytical groundwater models may be used, with relative confidence, to model the groundwater flow regime in simple or ideal aquifers. The character of the movement of groundwater within complex hydrogeological environments

may not be adequately described by these models. Similarly, analytical heat transport models can be applied to determine the rate of heat movement in simple aquifer configurations in which radial flow ensues. For a system in which fracture flow dominates, true radial flow conditions no longer exist and radial flow models, in theory, no longer apply.

One of the tasks for this research was to evaluate both numerical and analytical models for heat transport. This was accomplished by considering the ATES application at Carleton and by comparing the results generated from a numerical model to those from an analytical model (Chapter 8). The former model was more sophisticated and was expected to produce a more realistic picture of the character of heat transport. However, in terms of the long-term operation of the ATES system at Carleton, the analytical model was assessed. Since this model was used both by WESA (1988) and by CESI (1994) to calculate the expected thermal breakthrough times for the wells, this aspect of the research may have some practical implications.

In the remainder of this chapter, theory of heat transport is reviewed, and features of both simple analytical and numerical heat transfer models are discussed. The applicability of analytical and numerical solutions are evaluated for the Carleton campus. This discussion is followed by a thermal aquifer characterization and the mathematical formulation for the heat transport equations.

6.2 Heat Transport Theory

6.2.1 The Concept of Thermal Energy Storage

Thermal energy can be stored in a variety of media. In an aquifer, the energy is stored in the rock and water which comprise the aquifer volume. As water flows through the aquifer, it relinquishes energy to the rock and water in the aquifer storage volume. A thermal equilibrium between the water and the rock is quickly established. Therefore, the water temperature and rock temperature at any location are identical. Since thermal energy must be shared between the rock and water, the energy's motion is generally slower than the motion of the water. In a permeable material, a 'thermal energy front', follows the water front at a slower velocity.

The mathematical development for heat transport in porous media is based on conservation of energy, as applied to a small volume element of rock and water. The conduction of energy in the y- and z-coordinate directions is assumed negligible for the basic derivation (*i.e.*, one-directional flow and transport). A volume element of aquifer, with water flowing at an average linear velocity, v (m/s), into the element on the left, and exiting on the right can be used to derive the basic equations.

Figure 6.1: Volume Element Showing Heat Energy In and Out

The heat energy of the water flowing in, E_{in} , is defined as:

$$E_{in} = v\rho\phi A c_w \Delta t T_x \quad 6.1$$

and has the units of Joules. The energy of the water flowing out, E_{out} , is defined as:

$$E_{out} = v\rho\phi A c_w \Delta t T_{x+\Delta x} \quad 6.2$$

where ρ is the density of water (kg/m^3), ϕ is the porosity of the aquifer, A is the area (m^2) of the element through which the water enters ($\Delta y \Delta z$), c_w is the specific heat or mass heat capacity of water ($\text{Joules kg}^{-1} \text{ } ^\circ\text{C}^{-1}$) and Δt is the time interval (sec). T_x ($^\circ\text{C}$) is the temperature of the water at a distance x , and $T_{x+\Delta x}$ is the temperature at a distance Δx away. Since energy must be conserved, the difference between these two equations represents the amount of energy stored in or released from the volume:

$$E_{in} - E_{out} = E_{stored} \quad 6.3$$

Storage of energy is a function of the porosity, plus other water and rock physical and thermal properties. The volumetric thermal capacity, C ($\text{Joules m}^{-3} \text{ } ^\circ\text{C}^{-1}$), of the aquifer is:

$$C_{aquifer} = (c\rho)_{rock} (1-\phi) + (c\rho)_{water} \phi \quad 6.4$$

where c is the specific heat or mass heat capacity, ρ is the density and ϕ is the porosity.

The energy stored in the aquifer is defined, therefore, as:

$$E_{stored} = C_{aquifer} A \Delta x (T_{t+\Delta t} - T_t) \quad 6.5$$

The relationship between the rate of water transfer and the rate of heat transfer describes the thermal front velocity. The following section expands on this concept.

6.2.2 The Thermal Front

If warm (or cold) water is injected into an aquifer, the distance by which the thermal front lags behind the water flow front is determined by how much energy is released to the aquifer as the water passes through a given volume of the aquifer. The higher the heat capacity of the matrix, the more energy that is required to raise its temperature. At high heat capacities, the thermal front follows the water front at a slower velocity than it would if the aquifer heat capacity were lower. The mathematical characterization of this statement is obtained by combining Equations 6.1, 6.2 and 6.5 in accordance with Equation 6.3. This gives an expression in one-dimension for the change in energy being equal to that put into or removed from storage in an ideal aquifer:

$$(\phi v \rho c)_{\text{water}} A \Delta t (T_x - T_{x+\Delta x}) = (C)_{\text{aquifer}} A \Delta x (T_{t+\Delta t} - T_t) \quad 6.6$$

Dividing through by $A \Delta x \Delta t$ and taking the limit as Δx and $\Delta t \Rightarrow 0$ gives,

$$(v \rho c)_{\text{water}} \frac{\partial T}{\partial x} = (C)_{\text{aquifer}} \frac{\partial T}{\partial t} \quad 6.7$$

If the aquifer is at the initial condition $T=T_o$ (at $t=0$) and a final condition $T=T_f$ (for $t=\infty$), then the solution to the differential equation is:

$$T=T_o \text{ for } x > \frac{(\rho c)_{\text{water}}}{C_{\text{aquifer}}} vt \quad 6.8$$

$$T=T_f \text{ for } x < \frac{(\rho c)_{\text{water}}}{C_{\text{aquifer}}} vt \quad 6.9$$

In Equations 6.8 and 6.9, vt is the distance that the water front has moved, whereas x is the distance the thermal front has moved. In a system where the water does not move through a channel of constant area, for example in a radial flow system, these distances do

not have the above ratio. However, a direct relationship exists between the thermal front and water front velocities at any point:

$$v_{thermal} = \frac{(\rho c)_{water}}{C_{aquifer}} v_{water} \quad 6.10$$

The ideal solution to Equation 6.10 has the water front moving out from an injection well at a rate determined from the injection flow rate. Under radial flow conditions in a uniform aquifer, the velocity of the water decreases exponentially away from the well. This decrease is the result of the increase in the area through which the water flows with increasing distance from the well. A thermal front follows at a lower velocity. The temperature change at the front is assumed to be a step function. For short duration storage, Equation 6.10 is accurate; however, for large storage times the square wave deteriorates (Gringarten and Sauty, 1975). The thicker the aquifer, the closer a square wave represents the thermal front.

In real, rather than ideal, systems, energy losses exist. These include conduction losses to the confining layers, conduction losses in the aquifer itself ahead of the thermal front, water mixing losses (dispersion), and convection losses due to natural flow in the aquifer. Some of these irregularities have been addressed by various researchers. The more complicating factors that are taken into consideration when producing a model, the more complex become the governing equations. As the complexity of the problem domain increases, and as more modes of heat transfer are taken into consideration, it becomes more difficult to represent the system by equations that can be solved analytically.

WESA (1988) and CESI (1994) used the analytical solution by Gringarten and Sauty (1975) to evaluate the thermal behaviour of the Carleton system. This particular

model assumes uniform regional flow in the aquifer and gives solutions for the non-steady state temperature behaviour in a pumping/reinjection well field (a well doublet is considered). The theory is reviewed in detail in Section 6.3.

6.2.3 Energy Losses

Energy losses from the aquifer may occur in several ways. All losses which occur are independent of whether storage water is cold or hot.

- Heat conduction to the confining layers, and to the aquifer ahead of the thermal front.
- Free convection thermal mixing. Free convection occurs when the water density changes and buoyancy forces begin to act.
- Natural flow in the aquifer, which transports heat away from the storage area. This loss is a function of the natural flow velocity and aquifer porosity.

Some analytical models, including the models presented by Gringarten and Sauty (1975) and Gringarten *et al.* (1982), account for conduction losses to the overlying caprock, as well as those ahead of the thermal front. Buoyancy forces are important in large-scale and/or slow moving regional flow models and, therefore, is usually taken into consideration (Smith and Chapman, 1983; Woodbury and Smith, 1985; Molson *et al.*, 1992).

The effect of natural groundwater flow within an ATEs site has been examined by various researchers. Regional gradients affect the amount of heat that may be stored and the period over which the storage will be effective. Aquifers with moderate hydraulic conductivities in the horizontal direction (K_h), are preferred because this insures low

regional flows. However, hydraulic conductivities must be high enough to obtain an adequate yield. For energy to be efficiently stored in the aquifer, it is necessary that most of it remains in the aquifer near the injection point(s) and not advected large distances by the natural regional groundwater flow. In some applications, well configurations have been modified and additional reinjection wells placed downstream to counter high natural flow rates (Jenne *et al.*, 1992).

When several wells are placed close to each other, the cumulative drawdown in the vicinity of the pumping wells, and the cumulative injection head in the vicinity of the reinjection wells, is greater than if the individual pumping and reinjection wells are interspersed. Although more difficult to model, this latter configuration of wells leads to less overall drawdown and reinjection head for a particular system production rate and, thus, reduced velocities.

6.3 Analytical Methods of Solution

6.3.1 Analytical Models

The analytical models for heat transport are extensions of radial flow models, because the heat transport is dependent upon the rate of fluid flow. Most analytical models assume radial flow conditions and solve the heat transport equation for such flow conditions. In a simple configuration, two wells are situated a distance, D , apart. One well is a production or pumping well and the other is a reinjection well. In some applications, a single well is used for simultaneous pumping and reinjection with the depths of withdrawal and reinjection taking place at different aquifer depths. Single well systems have been

successful on a small scale; however, doublet systems or two well-field systems spaced 30 to 500m apart are more common.

Several researchers have developed analytical models which are appropriate for simple well configurations (*i.e.*, single wells and well doublets). These analytical models provide enough information to predict thermal breakthrough times for the particular well configuration in question. With the exception of a few large-scale applications, ATES has been used traditionally on a small scale (primarily domestic). Single wells and well couplets generally provide sufficient water for the operation of the system. Therefore, there has been little need to develop more sophisticated modelling approaches to ATES systems.

When large-scale ATES systems are designed, they invariably employ simple multi-well configurations, such as a cluster of pumping wells offset from a cluster of reinjection wells, or similarly, a line of pumping wells and a line of reinjection wells. In the former instance, the configuration approximates that of a well couplet, except that several wells constitute the “extraction” well and several wells constitute the “reinjection” well. In accordance with ATES technology, the temperature of the groundwater in the vicinity of a well field is increased or decreased over the course of a season. The area of influence for pumping and reinjection, and the natural groundwater velocity are two important, independent factors that determine the rate of migration of the thermal plume.

With these simple configurations, analytical models can be used to approximate the response. In addition, field experience and years of monitoring these larger projects help to define the rate of thermal front migration more accurately. In practice, well placement is

often dependent upon the space available for drilling, but more often it is because a simple configuration of wells is easier to model both hydrologically and thermally. As noted in the previous section, these simple configurations can lead to less efficient ATEs systems.

Much of the recent progress in computer modelling of thermal storage has been made in conjunction with research projects. In particular, numerical solutions for heat transport involving density dependent flow have been generated for a test site (unconfined aquifer) in Ontario (Palmer *et al.*, 1992; Molson *et al.*, 1992). In this instance, the test site was small and well defined, and the model could be calibrated accurately using the large data set available. For practical purposes, this type of model, and other three-dimensional numerical models, are far too complex to implement, because they require more spatial information than is available at Carleton.

6.3.2 Analytical Formulation - Two Well System (after Gringarten and Sauty, 1975)

Gringarten and Sauty (1975) developed an analytical solution for non-steady state temperature distribution around wells during the reinjection of heat-depleted water into aquifers with uniform regional flow. This basic analytical model has been extended to calculate well spacings at Carleton for which no thermal breakthrough should occur.

The simplifying assumptions to the model are as follows:

1. The aquifer is horizontal and of uniform thickness, h . The caprock and the bedrock, above and below the aquifer, are impermeable to flow and are of infinite extent in the vertical direction.

2. Flow is in steady-state with the production/reinjection flow field superimposed on a regional uniform flow field. The injection rate, Q , is constant and equal to the production rate. All wells fully penetrate the aquifer.
3. Both the aquifer and caprock have an initial temperature, T_o . At time $t=0$, the injection water temperature is T_i and it is maintained constant thereafter. Thermal equilibrium between the water and the aquifer rock is attained instantaneously and the rock and water are of equal temperatures, T_w , anywhere in the aquifer.
4. Horizontal thermal conductivity is neglected and vertical thermal conductivity is infinite in the aquifer itself. In the caprock, the vertical conductivity is finite. The upper boundary of the caprock is held at the constant initial temperature T_i .
5. There is no heat transfer coefficient between the aquifer and caprock.
6. Aquifer and caprock volumetric heat capacities remain constant. Viscosity and density of the water are also assumed to be independent of temperature.

These assumptions allow each stream channel (defined as an elementary stream channel) leaving a particular injection well to be treated independently. The differential equation governing the water temperature, T_w , within a stream channel is obtained by writing a heat balance on one element of the stream channel between the areas S and $S+dS$ from the injection well (Figure 6.2).

In Figure 6.2, the caprock (+) and bedrock (-) conductive contributions are:

$$\pm dS \cdot K_R \frac{\partial T_R}{\partial z} \bigg|_{z=\pm \frac{h}{2}} \quad 6.11a$$

Figure 6.2: Heat Balance on a Stream Channel Element
(Modified From Gringarten and Sauty, 1975)

where K_R is the thermal conductivity of the caprock and bedrock. The heat transported into the element is:

$$\phi \rho_w c_w h dS dt dw T_w(S) \quad 6.11b$$

and the heat stored in the element is:

$$\rho_a c_a \frac{\partial T_w}{\partial t} h dS dt \quad 6.11c$$

The heat transported out of the element is obtained by substituting $(S+dS)$ in place of S in Equation 6.11b. The differential equation is similar to that published by Carslaw and Jaeger (1959) for one-dimensional heat flow. The solution to the differential equation is (Gringarten and Sauty, 1975; Carslaw and Jaeger, 1959):

$$\frac{T_0 - T_w(S, t)}{T_0 - T_i} = \operatorname{erfc} \left[\frac{(\rho_w c_w)^2}{K_r \rho_r c_r} \left(\frac{q}{S} \right)^2 \left(t - \frac{\rho_a c_a}{\rho_w c_w} \frac{hS}{q} \right) \right]^{\frac{1}{2}} \quad 6.12$$

where q is the rate of flow within the aquifer and erfc is the complementary error function. K_r is the caprock (vertical) thermal conductivity and $\rho_r c_r$ is the volumetric heat capacity of the caprock. The other parameters are as defined earlier.

For the case in which there is no heat exchange by conduction between the aquifer and the caprock ($K_r=0$), Equation (6.11) reduces to a step function (Equation 6.10), which corresponds to a piston-like thermal front displacement with a velocity equal to $\rho_w c_w \phi / \rho_a c_a$ times that of the hydrodynamic front. The water temperature is T_0 ahead of the thermal front and T_i behind it. If $K_r > 0$, there is a transition zone behind the thermal front in which the temperature varies from T_0 to T_i .

If the streamlines leaving a recharge well reach a discharge well, the water temperature at that discharge well within an elementary stream channel is obtained by

considering the total stream channel area, S_{max} , between the two wells. The temperature at the production well can be expressed in terms of dimensionless parameters. The dimensionless temperature, expressed as a function of four dimensionless parameters, is defined as (after Gringarten and Sauty, 1975):

$$T_{wD}(\alpha, Q_D, \lambda, t_D) = \frac{T_0 - T_w(t)}{T_0 - T_i} \quad 6.13$$

where α is the angle between the aerial (planar horizontal) flow direction and the x-axis along which are the recharging and discharging wells. The recharge well is downstream when $\alpha=0$. The interflow between the wells, Q_D , is defined as:

$$Q_D = Q / h D v_o \quad 6.14$$

where Q is the production rate for the well(s) (m^3/sec), h is the aquifer thickness (m), D is the well separation for a two well system (m) and v_o is the Darcy velocity of the natural flow whose orientation is at an angle α to the x-axis. The parameter λ , is defined as:

$$\lambda = (\rho_w c_w / \rho_a c_a) (K_r / K) (Q t / D^2 h) \quad 6.15$$

and accounts for the heat transfer by conduction within the aquifer and caprock. Finally, the parameter t_D is defined as:

$$t_D = (\rho_w c_w / \rho_a c_a) (Q t / D^2 h) \quad 6.16$$

and represents the breakthrough time. For the case of a deep confined aquifer, in which interflow, between the pumping and reinjection well, dominates aerial flow ($Q_D > 10^4$), the temperature, T_{wD} , at the production well plotted against dimensionless time, t_D , for a number of different values of λ produces a family of curves (Gringarten and Sauty, 1975).

For $\lambda = \infty$, there is no heat transfer to the caprock and the breakthrough temperature is at a

maximum. For values of $\lambda > 10$, thermal front breakthrough at the production well occurs at $t_{wD} = 1.04$, and for $0 < \lambda < 10$, it occurs at $\lambda(t_D - 1) = 0.5$ (determined from type curves, Gringarten and Sauty, 1975). The maximum interflow is independent of λ , and is also equal to the maximum temperature change, T_{wDmax} , of the water at the production well.

If there is to be no change in the produced water temperature, for a period of length Δt , the spacing must be at least equal to:

$$D^2 = \frac{2 * Q * \Delta t}{\left[\left(\phi + (1 - \phi) \frac{\rho_r c_r}{\rho_w c_w} \right) h + \left(\left(\phi + (1 - \phi) \frac{\rho_r c_r}{\rho_w c_w} \right)^2 h^2 + 2 * \frac{K_r \rho_r c_r}{(\rho_w c_w)^2} * \Delta t \right)^{1/2} \right]} \quad 6.17$$

Equation 6.17 is the general form of the solution and takes into account a limited amount of heat transfer into the caprock. The amount of heat conducted into the caprock is a function of the thermal conductivity, K_r , of the caprock. If there is no heat transfer to the caprock, Equation 6.17 becomes:

$$t_D = 1.04 = \frac{\rho_w c_w}{\rho_A c_A} \frac{Q t}{D^2 h} \quad 6.18$$

Equation 6.18 was used to calculate the theoretical time interval, t , during which no thermal breakthrough will occur at the pumping well, given that no transfer of heat occurs between the aquifer and the caprock. Therefore, Equation 6.18 represents the more conservative estimate.

In free surface aquifers, where regional flow usually dominates, it is possible to locate the wells so that there is no interflow at all. In confined aquifers, Equations 6.17 and 6.18 can be used to approximate the well separation at which no thermal

breakthrough should occur over a specified time interval. Equally, given a specified well separation, the production rate for a well couplet may be calculated for which no thermal breakthrough should occur.

The ATES system at Carleton consists of multiple pumping and reinjection wells arranged in a non-uniform manner, thus, the interflow patterns diverge from those predicted for single well couplets. Drawdown and reinjection cones reflect the activity in all of the surrounding wells. Hence, the Principle of Superposition must again be considered as a rough first approximation. In a single well couplet system, the maximum interflow (assuming negligible regional flow and a uniform aquifer) will be along a straight line between the two wells. This type of behaviour can be illustrated using particle tracking for a simple two well couplet. In the case of multiple wells, the interflow is distributed amongst all of the surrounding wells. Consider a single pumping well surrounded by several reinjection wells. The velocity of the particles is averaged out in all directions, therefore, the heat transport in any particular direction will also be minimized.

The reduction in well interflow, due to the interspersion of pumping and reinjection wells, may be balanced by the increase in well interflow due to the preferential connection along the faults. For wells located along a fault, the inter-well velocities and, thus, the thermal breakthrough times, will be higher than those calculated for an aquifer of uniform hydraulic conductivity. Wells located along faults will have to be separated by larger distances than predicted by the model, in order to prevent thermal breakthrough. In contrast, wells on opposite sides of a fault can be placed closer together because of the poor hydraulic connection between different fault blocks. The faults introduce

'irregularities' to the assumed radial flow pattern which governs the analytical heat transport solution. This implies that the analytical solution should be used with caution and with great consideration for the location of wells relative to the faults. This is discussed in greater detail in Chapter 8.

Another factor, discussed in the following chapter, is the high porosities within the fault zones. These may partially counteract the effect of preferential connection, since they result in higher aquifer heat capacities.

Clearly, the fact that several assumptions are not strictly met at Carleton suggests that the analytical model falls short of being able to predict the heat transport characteristic of a faulted regime. The complexity of the system, from a hydrogeological point of view, suggests that a numerical approach is better suited to solving the problem.

6.4 Numerical Methods of Solution

6.4.1 The Selection of Equations

The fractured nature of the aquifer system at Carleton, and the high production rates for the operating wells, may cause groundwater velocities and, therefore, thermal front velocities, to be much higher than those measured in large regional systems. Furthermore, buoyancy forces, acting in the vertical direction, may be very small compared to the advective forces driving the heat flow. In addition, a two-dimensional domain has been selected (Chapter 5) for modelling groundwater flow because of the lack of information in vertical space. Therefore, a major assumption that density and viscosity are independent of temperature has been made. It is recognized that buoyancy forces, and

the density and viscosity temperature dependence, may be very important in the slowly advecting portions of the aquifer.

The governing equations for the transport problem are the Darcy equation, the continuity equation for the fluid, and the continuity equation for the heat. Molson *et al.* (1992) considered both density and viscosity to be temperature dependent, and incorporated this dependency by solving for groundwater flow and heat transport in an iterative manner. The equations for groundwater flow and heat transport are coupled in their method of solution. The equations used in this research were solved independently and, therefore, are analogous to the solute transport equations defined with different variables.

6.4.2 The Governing Equations

Heat is transferred by conduction, convection and radiation. The familiar heat conduction equation is (reviewed in Domenico and Schwartz, 1990):

$$H = -\kappa \text{ grad}T \quad 6.21$$

where H is defined as the heat flux ($\text{J s}^{-1}\text{m}^{-2}$), κ is the thermal conductivity ($\text{Js}^{-1}\text{m}^{-1} \text{ }^{\circ}\text{C}^{-1}$) and $\text{grad}T$ is the gradient of the temperature in three-dimensional space:

$$\text{grad}T = \frac{\partial T}{\partial x} + \frac{\partial T}{\partial y} + \frac{\partial T}{\partial z} \quad 6.22$$

It is assumed that the solids and the fluids are at the same temperature, therefore, T is the average temperature of both phases. Because both fluids and solids are conductors, it is necessary to define an aquifer thermal conductivity, κ_{aquifer} , such that Equation 6.21 becomes:

$$H = -\kappa_{aquifer} gradT \quad 6.23$$

In two phase mixtures, consisting of water and solids, the thermal conductivity can be described by considering the volume fractions and conductivities of the individual phases. Thermal conductivity for a solid and liquid phase is defined by the geometric mean equation (Johansen, 1972, 1973, 1975):

$$\kappa_{aquifer} = \kappa_f^\phi \cdot \kappa_s^{(1-\phi)} \quad 6.24$$

where ϕ is the porosity, κ_f is the fluid thermal conductivity, and κ_s is the solid thermal conductivity. Slattery (1972) suggested that Equation 6.24 may be amended to account for tortuosity (a non-parallel arrangement of fluids and solids) in the porous medium.

For a moving fluid, a convective effect known as thermal dispersion occurs (Domenico and Schwartz, 1990). Thermal dispersion occurs on a microscopic scale as a result of irregularities, induced by heterogeneities, in the porous structure. It has the same effect, when viewed macroscopically, as an increase in the effective thermal conductivity (Domenico and Schwartz, 1990). In the following development, dispersion will be incorporated into the effective thermal conductivity κ_{eff} . Experiments have shown that the effective thermal conductivity increases with the velocity of the fluid (Green and Perry, 1961). In fact, it is the dispersion component of the effective thermal conductivity which dominates this relationship.

Heat transfer also occurs by convection and is associated with the movement of a fluid (Domenico and Schwartz, 1990). When the flow field (heat) is caused by external forces, the transport is said to occur by **forced convection**. This is the case when groundwater movement takes place in the absence of density gradients such that Darcy's

Law applies. **Free convection** occurs when the flow of the fluid is due exclusively to density variations caused by temperature gradients. In the case of forced convection, buoyancy forces are assumed to be negligible. When buoyancy forces dominate, the velocity field and the temperature field are interdependent and the equations must be solved iteratively. Since the density and viscosity are assumed to remain constant, there is no need to couple the groundwater and heat transport equations and solve for the hydraulic head and temperature using an iterative process and set convergence criteria. The two equations may be solved independently.

The energy or heat flux associated with a moving fluid mass is obtained by taking the energy per unit area per unit time. Equation 6.1 then becomes:

$$\mathbf{H} = \mathbf{v}\phi\rho_w c_w T \quad 6.25$$

where $\mathbf{v}$ is the average linear velocity of the moving fluid mass. Both $\mathbf{H}$ and $\mathbf{v}$ are vectors. Combining the conduction (Equation 6.23) and convection (Equation 6.25) components and writing the energy flux $\mathbf{H}$ in terms of its vector components gives:

$$H_x = -\kappa_{effx} \frac{\partial T}{\partial x} + \phi\rho_w c_w T v_x \quad 6.26a$$

$$H_y = -\kappa_{effy} \frac{\partial T}{\partial y} + \phi\rho_w c_w T v_y \quad 6.26b$$

$$H_z = -\kappa_{effz} \frac{\partial T}{\partial z} + \phi\rho_w c_w T v_z \quad 6.26c$$

where it is assumed that the principle directions of κ_{eff} are aligned with the x- and y-coordinate axes. For isotropic conditions, in which κ_{eff} is the same in all directions:

$$\mathbf{H} = -\kappa_{eff} \frac{\partial T}{\partial x} + \phi\rho_w c_w T \mathbf{v} \quad 6.27$$

Using a representative elementary volume (*REV*), with inflows through three sides, the divergence of the energy field, expressed as $\text{div}\mathbf{H}$, is given as:

$$\text{div}\mathbf{H} = \text{div}[-\kappa_{\text{eff}} \text{grad}T + \phi \rho_w c_w T \mathbf{v}] \quad 6.28$$

A negative divergence indicates that there is a net loss of heat from the *REV*. If there is no net loss or gain of energy from the volume, the divergence is zero and the temperature of the volume remains constant (*i.e.*, steady state). If there is a net loss or gain of energy, the temperature of the volume changes according to its heat capacity. It is assumed that this net balance is proportional to the temperature change and that the constant of proportionality is the heat capacity (as in the analytical discussion above). The heat capacity of the aquifer is defined as in Equation 6.4 and the change in heat storage in the volume is equal to the divergence:

$$\text{div}\mathbf{H} = -\text{div}[\kappa_{\text{eff}} \text{grad}T + \phi \rho_w c_w T \mathbf{v}] = \rho_a c_a \frac{\partial T}{\partial t} \quad 6.29$$

If it is assumed that κ_{eff} , ϕ , ρ_w and c_w are constants, then Equation 6.29 may be written as:

$$\kappa_{\text{eff}} \text{div}(\text{grad}T) - \phi \rho_w c_w [\mathbf{v} \cdot \text{grad}T + T \cdot \text{div} \mathbf{v}] = \rho_a c_a \frac{\partial T}{\partial t} \quad 6.30$$

where the divergence of the product of a scalar field and a vector (*i.e.*, $\text{div}(T\mathbf{v})$) is written as the sum of two dot products (square brackets in Equation 6.30). The continuity equation for the fluid is defined as:

$$-\text{div}(\rho \mathbf{q}) + Q = 0 \quad 6.31$$

where $\mathbf{q}$ is the Darcy velocity and Q is a point source or sink mass flux term ($\text{kg m}^{-3} \text{ s}^{-1}$).

Molson *et al.* (1992) expressed as Q as:

$$Q = \sum_{k=1}^N Q_k(t) \rho_w \delta(x'_k, y'_k, z'_k) \quad 6.32$$

where N is the number of sources/sinks, $Q_k(t)$ is the time-dependent injection or withdrawal rate ($\text{m}^3 \text{s}^{-1}$), δ is the Dirac delta function (m^{-1}), and x'_k , y'_k and z'_k are the point source/sink coordinates. Expanding Equation 6.31 using the chain rule gives:

$$-\rho \operatorname{div} \mathbf{q} - \mathbf{q} \cdot \operatorname{grad} \rho + Q = 0 \quad 6.33$$

Assuming an incompressible fluid (*i.e.*, the density of the fluid is a constant):

$$\operatorname{grad} \rho = \frac{\partial \rho}{\partial h} \quad 6.34a$$

and

$$\operatorname{grad} h = 0 \quad 6.34b$$

then,

$$\operatorname{div} \mathbf{q} = + \frac{Q}{\rho} \quad 6.35$$

Dividing through by the porosity and incorporating the results of Equation 6.32 leads to an expression for the divergence of the velocity field (the second term in the square brackets of Equation 6.30) in terms of the average linear velocity (Molson *et al.*, 1992):

$$\operatorname{div} \mathbf{v} = \frac{Q_k}{\phi} \delta(x'_k, y'_k, z'_k) \quad 6.36$$

Equation 6.30 is the general form of the conduction-convection equation. It should be noted that the conductive effect of thermodispersion is incorporated in the parameter κ_{eff} . The effective thermal conductivity is due in part to the thermal conductivity of the aquifer, $\kappa_{aquifer}$, and in part to dispersion associated with water flow and is represented by:

$$\kappa_{eff} = \kappa_{aquifer} + \rho_w c_w \phi D_{ij} \quad 6.37$$

where D_{ij} is the hydrodynamic dispersion tensor which is used in the advective-dispersive solute transport equation. Molson *et al.* (1992) gave expressions for this tensor as a function of the dispersivities, α_L , α_{TH} and α_{TV} (m), the average linear velocity and the diffusion coefficient, D^* (m^2s^{-1}). Assuming that the main components of the dispersion tensor are aligned with the coordinate axes, such that $D_{xy}=D_{yx}=D_{yz}=D_{zy}=D_{zx}=D_{xz}=0$, and assuming D^* is negligible for heat transport (Molson *et al.* (1992) used a value of $6.0 \times 10^{-10} \text{ m}^2\text{s}^{-1}$), the dispersion tensor in the x, y and z directions is given by:

$$D_{xx} = \alpha_L \frac{v_x^2}{\bar{v}} + \alpha_{TH} \frac{v_y^2 \bar{v}}{v_x^2 + v_y^2} + \alpha_{TV} \frac{v_z^2 v_x^2}{v_x^2 + v_y^2} \frac{1}{\bar{v}} \quad 6.38a$$

$$D_{yy} = \alpha_L \frac{v_y^2}{\bar{v}} + \alpha_{TH} \frac{v_x^2 \bar{v}}{v_x^2 + v_y^2} + \alpha_{TV} \frac{v_z^2 v_y^2}{v_x^2 + v_y^2} \frac{1}{\bar{v}} \quad 6.38b$$

$$D_{zz} = \alpha_{TV} \bar{v} + (\alpha_L - \alpha_{TV}) \frac{v_z^2}{\bar{v}} \quad 6.38c$$

In Equations 6.38, $\bar{v}$ is the magnitude of the average linear velocity vector, α_L is the longitudinal dispersivity, α_{TH} is the transverse horizontal dispersivity, and α_{TV} is the transverse vertical dispersivity. Istok (1989) presented a similar set of expressions for evaluating the components of the dispersion tensor.

The heat sources and sinks behave in much the same way as a fluid source or sink (*i.e.*, a fluid flux). If heat is added to or removed from a system, either along a boundary (surface or line) or through a well, then the temperature distribution will be affected. In a three-dimensional domain, heat is added or removed from nodes located anywhere in three-dimensional space. In the case of a well, the total heat added to an aquifer must be

distributed or divided along the length of the well. For a two-dimensional problem, domain the source extends vertically downward and is represented as a line source. By convention, source terms are positive and sink terms are negative. In regional aquifers, the geothermal heat flux, which is directed upward from the center of the Earth, is an important heat source. In this situation, the bottom boundary of the domain (a surface in 3-D or a line in 2-D) would represent a source. The well source term for three-dimensional space takes on the form (Molson *et al.*, 1992):

$$Q = \rho_w c_w \sum_{k=1}^N Q_k(t) T_k(t) \delta(x'_k, y'_k, z'_k) \quad 6.39$$

in which $Q_k(t)$ is the time-dependent injection or withdrawal rate ($\text{m}^3 \text{s}^{-1}$) and $T_k(t)$ is the time-dependent injection temperature. Incorporating Equation 6.36, and the source term in Equation 6.39, into Equation 6.30 results in a total time-dependent source term of the form:

$$\rho_w c_w \sum_{k=1}^N Q_k (T_k - T) \delta(x'_k, y'_k, z'_k) \quad 6.40$$

The final differential equation, which is used in the numerical formulation, is:

$$(\kappa_a + D_{ij} \phi \rho_w c_w) \text{div} (\text{grad} T) - \phi \rho_w c_w \mathbf{v}_i \text{grad} T + \rho_w c_w \sum_{k=1}^N Q_k (T_k - T) \delta(x'_k, y'_k, z'_k) = \rho_a c_a \frac{\partial T}{\partial t} \quad 6.41$$

where the subscripts i and j are used to denote the x, y and z components. If the Darcy velocity, q , is used in place of the average linear velocity, since this value is calculated for

each node directly from the groundwater flow model, then the porosity drops out of the first and second terms of the above equation since $v=q/\phi$. Velocity is also used in the calculation of D_{ij} . Equation 6.41 becomes:

$$\left(\kappa_a + D_{ij} \rho_w c_w\right) \operatorname{div}(\operatorname{grad} T) - \rho_w c_w \mathbf{q}_i \operatorname{grad} T + \rho_w c_w \sum_{k=1}^N Q_k (T_k - T) \delta(x'_k, y'_k, z'_k) = \rho_a c_a \frac{\partial T}{\partial t} \quad 6.42$$

The heat transport equation (6.42) is solved independently of the groundwater flow equation, although the velocities obtained from the latter are used in calculating the former.

6.4.3 Numerical Formulation - The FEM Approach

The heat transport formulation uses the method of weighted residuals as in the groundwater flow formulation (refer to Appendix 7). The approximate solution for the temperature within an element e is of the form:

$$T^{(e)} = \sum_{j=1}^n N_j^{(e)} T_j^{(e)} \quad 6.43$$

where $N_j^{(e)}$ are the interpolation functions for each node within element e , T_j are the unknown temperatures for each node in the element, and n is the number of nodes in the element. The contribution of the element e to the residual at a node i , to which the element is joined, is defined in two-dimensions as:

$$R_i^{(e)} = - \iint_{A^{(e)}} W_i^{(e)} \left[\kappa_{ax}^{(e)} \frac{\partial^2 T^{(e)}}{\partial x^2} + \kappa_{ay}^{(e)} \frac{\partial^2 T^{(e)}}{\partial y^2} + \rho_w c_w \left(\frac{\partial}{\partial x} \left(D_{xx}^{(e)} \frac{\partial T^{(e)}}{\partial x} + D_{xy}^{(e)} \frac{\partial T^{(e)}}{\partial y} \right) + \frac{\partial}{\partial y} \left(D_{yx}^{(e)} \frac{\partial T^{(e)}}{\partial x} + D_{yy}^{(e)} \frac{\partial T^{(e)}}{\partial y} \right) \right) - \rho_w c_w \left(q_x^{(e)} \frac{\partial T^{(e)}}{\partial x} + q_y^{(e)} \frac{\partial T^{(e)}}{\partial y} \right) + \right]$$

$$\rho_w c_w Q_k^{(e)} (T_k^{(e)} - T^{(e)}) - \rho_a^{(e)} c_a^{(e)} \frac{\partial T}{\partial t} \Big] dx dy \quad - \quad 6.44$$

In the Galerkin formulation, the weighting function, W_i , is identical to N_i , for node i , and the limits of integration are chosen to represent the surface area of element e (in the 2-dimensional case). If it is assumed that $\kappa_a^{(e)}$, $D_{xx}^{(e)}$, $D_{xy}^{(e)}$, $D_{yx}^{(e)}$, $D_{yy}^{(e)}$, $c_a^{(e)}$, and $\rho_a^{(e)}$ are constant within an element, but can vary from one element to another, Equation 6.44 becomes:

$$\begin{aligned} R_i^{(e)} = & - \iint_{A^{(e)}} N_i^{(e)} \left[\kappa_{a_x}^{(e)} \frac{\partial^2 T^{(e)}}{\partial x^2} + \kappa_{a_y}^{(e)} \frac{\partial^2 T^{(e)}}{\partial y^2} \right] dx dy - \\ & \iint_{A^{(e)}} N_i^{(e)} \left[\rho_w c_w^{(e)} \left(D_{xx}^{(e)} \frac{\partial^2 T^{(e)}}{\partial x^2} + D_{xy}^{(e)} \frac{\partial^2 T^{(e)}}{\partial x \partial y} + \right. \right. \\ & \left. \left. D_{yx}^{(e)} \frac{\partial^2 T^{(e)}}{\partial y \partial x} + D_{yy}^{(e)} \frac{\partial^2 T^{(e)}}{\partial y^2} \right) \right] dx dy + \iint_{A^{(e)}} N_i^{(e)} \left[\rho_w c_w \left(q_x^{(e)} \frac{\partial T^{(e)}}{\partial x} + q_y^{(e)} \frac{\partial T^{(e)}}{\partial y} \right) \right] dx dy - \\ & \iint_{A^{(e)}} N_i^{(e)} \rho_w c_w Q_k(t)^{(e)} [T_k(t)^{(e)} - T^{(e)}] dx dy + \iint_{A^{(e)}} N_i^{(e)} \left[\rho_a^{(e)} c_a^{(e)} \frac{\partial T^{(e)}}{\partial t} \right] dx dy = 0 \quad 6.45 \end{aligned}$$

Because the approximate solution is a linear function of x , the second derivative of T , with respect to x , y or z , is undefined. However, the approximate solution does have a continuous first derivative, so Equation 6.44 can be evaluated if it is rewritten in terms of the first derivatives, as:

$$\begin{aligned} R_i^{(e)} = & - \iint_{A^{(e)}} N_i^{(e)} \left[\frac{\partial}{\partial x} \left(\kappa_{a_x}^{(e)} \frac{\partial T^{(e)}}{\partial x} \right) + \frac{\partial}{\partial y} \left(\kappa_{a_y}^{(e)} \frac{\partial T^{(e)}}{\partial y} \right) \right] dx dy - \iint_{A^{(e)}} N_i^{(e)} \left[\rho_w c_w \frac{\partial}{\partial x} \left(D_{xx}^{(e)} \frac{\partial T^{(e)}}{\partial x} \right) + \right. \\ & \left. \rho_w c_w \frac{\partial}{\partial x} \left(D_{xy}^{(e)} \frac{\partial T^{(e)}}{\partial y} \right) + \rho_w c_w \frac{\partial}{\partial y} \left(D_{yx}^{(e)} \frac{\partial T^{(e)}}{\partial x} \right) + \rho_w c_w \frac{\partial}{\partial y} \left(D_{yy}^{(e)} \frac{\partial T^{(e)}}{\partial y} \right) \right] dx dy \end{aligned}$$

$$\begin{aligned}
& + \iint_{A^{(e)}} N_i^{(e)} \left[\rho_w c_w \left(q_x^{(e)} \frac{\partial T^{(e)}}{\partial x} + q_y^{(e)} \frac{\partial T^{(e)}}{\partial y} \right) \right] dx dy + \iint_{A^{(e)}} N_i^{(e)} \left[\rho_a^{(e)} c_a^{(e)} \frac{\partial T^{(e)}}{\partial t} \right] dx dy \\
& - \iint_{A^{(e)}} N_i^{(e)} \rho_w c_w \left[Q_k(t)^{(e)} (T_k^{(e)} - T^{(e)}) \right] dx dy = 0
\end{aligned} \tag{6.46}$$

Green's Theorem is used to integrate by parts such that for each of the two terms in the first integral of Equation 6.46:

$$\iint_{A^{(e)}} N_i^{(e)} \left[\frac{\partial}{\partial x} \left(\kappa_{a_x}^{(e)} \frac{\partial T^{(e)}}{\partial x} \right) \right] dx dy = \int_{S^{(e)}} N_i^{(e)} \left[\kappa_{a_x}^{(e)} \frac{\partial T^{(e)}}{\partial x} \cdot l_x \right] ds - \iint_{A^{(e)}} \left[\kappa_{a_x} \frac{\partial T^{(e)}}{\partial x} \frac{\partial N_i^{(e)}}{\partial x} \right] dx dy \tag{6.47}$$

where the first part of the equation is a boundary line integral, ds , and the second part is an integral over the domain. A similar expression is obtained for the y-coordinate direction, such that Equation 6.47 becomes:

$$\iint_{A^{(e)}} \left[\kappa_{a_x}^{(e)} \frac{\partial T^{(e)}}{\partial x} \frac{\partial N_i^{(e)}}{\partial x} + \kappa_{a_y}^{(e)} \frac{\partial T^{(e)}}{\partial y} \frac{\partial N_i^{(e)}}{\partial y} \right] dx dy - \int_{S^{(e)}} N_i^{(e)} \left[\kappa_{a_x}^{(e)} \frac{\partial T^{(e)}}{\partial x} \cdot l_x + \kappa_{a_y}^{(e)} \frac{\partial T^{(e)}}{\partial y} \cdot l_y \right] ds \tag{6.48}$$

In Equation 6.48, l_x is the direction cosine of the vector normal to the boundary. The term $(\kappa_{a_x} \partial T / \partial x \cdot l_x)$ is simply the projection of the heat flux, $-Q_x$ (outward), onto a unit vector, $\mathbf{n}$, that is normal to the boundary. It can be equally written as:

$$\left(\kappa_{a_x} \frac{\partial T^{(e)}}{\partial x} + \kappa_{a_y} \frac{\partial T^{(e)}}{\partial y} \right) \cdot (l_x + l_y) \tag{6.49a}$$

$$[\kappa] \text{grad} T \cdot \mathbf{n} = -Q_{\text{thermal}} \cdot \mathbf{n} \equiv -Q_{\text{thermal}_n} \tag{6.49b}$$

From the original trial solution (Equation 6.43), we obtain an equation of the form:

$$\sum_{e=1}^E \iint_{A^{(e)}} \kappa_{a_x} \left[\left\{ \sum_{j=1}^n T_j^{(e)} \frac{\partial N_j^{(e)}}{\partial x} \right\} \frac{\partial N_i^{(e)}}{\partial x} \right] + \kappa_{a_y} \left[\left\{ \sum_{j=1}^n T_j^{(e)} \frac{\partial N_j^{(e)}}{\partial y} \right\} \frac{\partial N_i^{(e)}}{\partial y} \right] dx dy$$

$$+ \int_{S^{(e)}} [N_i^{(e)} \cdot Q_n] ds = 0 \quad 6.50$$

by summing the contributions from each of the elements. The T_j values are not a function of x or y , they are nodal, therefore, they can be taken out of the differential operator. The summations on j may also be moved out of the integrals, since the integral of a sum is equal to the sum of the integrals. The first two terms of Equation 6.50, which include that portion of the heat transfer by conduction, may be written in matrix format. For each element in the two-dimensional domain, the preliminary matrix is defined as:

$$[D^{(e)}] = \iint_{A^{(e)}} \begin{bmatrix} \frac{\partial N_1^{(e)}}{\partial x} & \frac{\partial N_1^{(e)}}{\partial y} \\ \frac{\partial N_n^{(e)}}{\partial x} & \frac{\partial N_n^{(e)}}{\partial y} \end{bmatrix}_{n \times 2} \begin{bmatrix} \kappa_{a_x}^{(e)} & 0 \\ 0 & \kappa_{a_y}^{(e)} \end{bmatrix}_{2 \times 2} \begin{bmatrix} \frac{\partial N_1^{(e)}}{\partial x} & \dots & \frac{\partial N_n^{(e)}}{\partial x} \\ \frac{\partial N_1^{(e)}}{\partial y} & \dots & \frac{\partial N_n^{(e)}}{\partial y} \end{bmatrix}_{2 \times n} dx dy \quad 6.51$$

The interpolation functions N_i and N_j were evaluated for triangular elements in Appendix 7. Equation 6.51 is only a partial matrix equation, the remaining terms in Equation 6.46 must be included before the element conductance-dispersive matrix can be evaluated.

The final term in Equation 6.49 represents the heat flux across the element's surface. For elements on the exterior of the mesh, this term is to represent specified rates of heat flow (Neuman boundary condition). The term is given the symbol $F_i^{(e)}$, which is positive if heat is entering the system and negative if heat is leaving the system. If no flows are specified, or alternatively, at heat impermeable boundaries, $F_i^{(e)}$ is zero. For elements on the interior of the mesh, the term $F_i^{(e)}$ for adjacent elements has the opposite sign, thus, cancelling out the contribution to $F_i^{(e)}$ for the nodes they share.

For the advective/dispersive terms in Equation 6.46 (*i.e.*, those that contain the dispersion coefficients (D_{ij}) and the components of the velocity (q_i)), the matrix formulation is similar to that for solute transport (Istok, 1989). In two-dimensions, the matrix, for a non-uniform flow field in which the velocities q_x and q_y are non-zero, becomes:

$$\begin{aligned}
 [D^{(e)}(t)]_{nxn} = & +\rho_w c_w \iint_{A^{(e)}} \begin{bmatrix} \frac{\partial N_1^{(e)}}{\partial x} & \frac{\partial N_1^{(e)}}{\partial y} \\ \frac{\partial N_n^{(e)}}{\partial x} & \frac{\partial N_n^{(e)}}{\partial y} \end{bmatrix}_{nx2} \begin{bmatrix} D_{xx}^{(e)}(t) & D_{xy}^{(e)}(t) \\ D_{yx}^{(e)}(t) & D_{yy}^{(e)}(t) \end{bmatrix}_{2x2} \begin{bmatrix} \frac{\partial N_1^{(e)}}{\partial x} & \dots & \frac{\partial N_n^{(e)}}{\partial x} \\ \frac{\partial N_1^{(e)}}{\partial y} & \dots & \frac{\partial N_n^{(e)}}{\partial y} \end{bmatrix}_{2xn} dxdy + \\
 & \rho_w c_w \iint_{A^{(e)}} \begin{bmatrix} N_1^{(e)} & N_1^{(e)} \\ \vdots & \vdots \\ N_n^{(e)} & N_n^{(e)} \end{bmatrix}_{nx2} \begin{bmatrix} q_x^{(e)} & 0 \\ 0 & q_y^{(e)} \end{bmatrix}_{2x2} \begin{bmatrix} \frac{\partial N_1^{(e)}}{\partial x} & \dots & \frac{\partial N_n^{(e)}}{\partial x} \\ \frac{\partial N_1^{(e)}}{\partial y} & \dots & \frac{\partial N_n^{(e)}}{\partial y} \end{bmatrix}_{2xn} dxdy \quad 6.52
 \end{aligned}$$

where, as in the previous formulation, the second derivative is undefined for the dispersive term and Green's Theorem is used to integrate the expression. Since the first part of Equation 6.52 is similar in form to Equation 6.51, they can be added directly together. The second part of Equation 6.52 is the advective portion of the heat transport equation.

The final term in Equation 6.46 is the flux (or source) contribution :

$$-\iint_{A^{(e)}} N_i^{(e)} \rho_w c_w [Q_k(t)^{(e)} (T_k^{(e)} - T^{(e)})] dxdy \quad 6.53$$

which can be divided into two components. The first part is added to the R.H.S. flux vector $\{F_i^{(e)}\}$:

$$\{F_i^{(e)}\} = -\rho_w c_w \iint_{A^{(e)}} N_i^{(e)} Q_k(t)^{(e)} T_k(t)^{(e)} dxdy \quad 6.54a$$

and for a point source/sink has the form:

$$\{F_i^{(e)}\} = -\rho_w c_w Q_k(t) T_k(t) \quad 6.54b$$

The second part is added to the L.H.S. coefficient matrix $[D(t)^{(e)}]$ and has the form:

$$+\rho_w c_w \iint_{A^{(e)}} N_i^{(e)} Q_k(t)^{(e)} T^{(e)} dx dy \quad 6.55a$$

$$+\rho_w c_w \iint_{A^{(e)}} N_i^{(e)} Q_k(t)^{(e)} \left[\sum_{j=1}^n N_j^{(e)} T_j^{(e)} \right] dx dy \quad 6.55b$$

Equations 6.51, 5.52 and 6.55b contribute the advective-dispersive and conductive components of the heat transfer equation and may be combined into an advective-dispersive matrix for each element:

$$\begin{aligned} [D^{(e)}(t)] = & \iint_{A^{(e)}} \begin{bmatrix} \frac{\partial N_1^{(e)}}{\partial x} & \frac{\partial N_1^{(e)}}{\partial y} \\ \frac{\partial N_n^{(e)}}{\partial x} & \frac{\partial N_n^{(e)}}{\partial y} \end{bmatrix}_{nx2} \begin{bmatrix} \kappa_{ax} + \rho_w c_w D_{xx}^{(e)}(t) & \kappa_{ay} + \rho_w c_w D_{xy}^{(e)}(t) \\ \kappa_{ax} + \rho_w c_w D_{yx}^{(e)}(t) & \kappa_{ay} + \rho_w c_w D_{yy}^{(e)}(t) \end{bmatrix}_{2x2} \begin{bmatrix} \frac{\partial N_1^{(e)}}{\partial x} & \dots & \frac{\partial N_n^{(e)}}{\partial x} \\ \frac{\partial N_1^{(e)}}{\partial y} & \dots & \frac{\partial N_n^{(e)}}{\partial y} \end{bmatrix}_{2xn} dx dy \\ & + \rho_w c_w \iint_{A^{(e)}} \begin{bmatrix} N_1^{(e)} & N_1^{(e)} \\ \vdots & \vdots \\ N_n^{(e)} & N_n^{(e)} \end{bmatrix}_{nx2} \begin{bmatrix} q_x^{(e)} & 0 \\ 0 & q_y^{(e)} \end{bmatrix}_{2x2} \begin{bmatrix} \frac{\partial N_1^{(e)}}{\partial x} & \dots & \frac{\partial N_n^{(e)}}{\partial x} \\ \frac{\partial N_1^{(e)}}{\partial y} & \dots & \frac{\partial N_n^{(e)}}{\partial y} \end{bmatrix}_{2xn} dx dy \\ & + \rho_w c_w \iint_{A^{(e)}} \begin{bmatrix} N_1^{(e)} \\ \vdots \\ N_n^{(e)} \end{bmatrix}_{nx1} [Q_k(t)]_{1x1} \begin{bmatrix} N_1^{(e)} & \dots & N_n^{(e)} \end{bmatrix}_{1xn} dx dy \quad 6.56 \end{aligned}$$

The flux, $Q_k(t)$, is a nodal value, and for a two dimensional problem, it is a line source or sink. No interpolation is necessary; the value of $Q_k(t)$ at each node is added to the global advective-dispersive matrix.

The storage term, as defined by the second to last term in Equation 6.46, may be evaluated in a similar way to the groundwater flow storage term. This term involves a time derivative. To evaluate the integral of the time derivative, the interpolation functions and the values of the time derivatives at the nodes are used. There are two commonly-used formulations; the lumped and consistent formulations (Istok, 1989). The consistent formulation is used here. Just as the trial solution was represented as the sum of the interpolation functions times the actual solution, the derivative of the trial solution is defined as:

$$\frac{\partial T^{(e)}}{\partial t} = \sum_{j=1}^n N_j^{(e)} \frac{\partial T_j}{\partial t} \quad 6.57$$

The storage term in Equation 6.46, when written in matrix format becomes:

$$[S^{(e)}(t)]_{n \times n} = + \iint_{A^{(e)}} \begin{bmatrix} N_1^{(e)} \\ \vdots \\ N_n^{(e)} \end{bmatrix}_{n \times 1} [\rho_a^{(e)} c_a^{(e)}]_{1 \times 1} [N_1^{(e)} \dots N_n^{(e)}]_{1 \times n} dx dy \quad 6.58$$

A global advective/dispersion matrix ($[D^{(e)}]$) and a global storage matrix ($[S^{(e)}]$) can be obtained by combining the element matrices for all the elements in the mesh. The matrix equation which needs to be solved is represented as:

$$[D]_{global} \{T\} + [S]_{global} \{T^0\} = \{F\}_{global} \quad 6.59$$

where

$$\{T\} = \begin{Bmatrix} T_1 \\ T_2 \\ \vdots \\ T_n \end{Bmatrix} \text{ and } \{T^0\} = \begin{Bmatrix} \frac{\partial T_1}{\partial t} \\ \frac{\partial T_2}{\partial t} \\ \vdots \\ \frac{\partial T_n}{\partial t} \end{Bmatrix} \quad 6.60$$

Finally, general time weighting (finite difference) can be used to solve Equation 6.59. This is given by:

$$([S] + \omega \Delta t [D])\{T\}_{t+\Delta t} = ([S] - (1 - \omega) \Delta t [D])\{T\}_t + \Delta t ((1 - \omega)\{F\}_t + \omega\{F\}_{t+\Delta t}) \quad 6.61$$

where ω is the relaxation factor which has a value between zero and one. For reasons discussed in the previous chapter, a value of one was selected for ω . The initial temperature vector, $\{T\}_0$, was specified and Equation 6.61 was solved for values of the temperatures at each node in the mesh in transient time. At the end of each time step, the calculated temperatures became the initial temperatures for the following time step. This process was repeated for each time increment. Because the velocities for each element changed after each time increment, the conductance/advective-dispersive matrix had to be re-evaluated.

Both the element conductance/advective-dispersive matrix ($[D^{(e)}]$) and the storage matrix ($[S^{(e)}]$) can be evaluated for any type of element, once the interpolation functions and their derivatives for each node in the element are specified. The storage matrix for a triangular element is given by:

$$[S^{(e)}] = \frac{A^{(e)}}{12} [\rho_a^{(e)} c_a^{(e)}] \begin{bmatrix} 2 & 1 & 1 \\ 1 & 2 & 1 \\ 1 & 1 & 2 \end{bmatrix} \quad 6.62$$

where $A^{(e)}$ is the area of the element. The conductance/advective-dispersive matrix consists of more terms and is given by:

$$\begin{aligned}
 [D^{(e)}] = & \frac{\kappa_{ax}^{(e)} + \rho_w c_w D_{xx}^{(e)}}{4A^{(e)}} \begin{bmatrix} b_i^2 & b_i b_j & b_i b_k \\ b_j b_i & b_j^2 & b_j b_k \\ b_k b_i & b_k b_j & b_k^2 \end{bmatrix} + \frac{\rho_w c_w D_{xy}^{(e)}}{4A^{(e)}} \begin{bmatrix} b_i c_i & b_i c_j & b_i c_k \\ b_j c_i & b_j c_j & b_j c_k \\ b_k c_i & b_k c_j & b_k c_k \end{bmatrix} \\
 & + \frac{\rho_w c_w D_{yx}^{(e)}}{4A^{(e)}} \begin{bmatrix} c_i b_i & c_i b_j & c_i b_k \\ c_j b_i & c_j b_j & c_j b_k \\ c_k b_i & c_k b_j & c_k b_k \end{bmatrix} + \frac{\kappa_{ay}^{(e)} + \rho_w c_w D_{yy}^{(e)}}{4A^{(e)}} \begin{bmatrix} c_i^2 & c_i c_j & c_i c_k \\ c_j c_i & c_j^2 & c_j c_k \\ c_k c_i & c_k c_j & c_k^2 \end{bmatrix} \\
 & + \frac{\rho_w c_w q_x^{(e)}}{6} \begin{bmatrix} b_i & b_j & b_k \\ b_i & b_j & b_k \\ b_i & b_j & b_k \end{bmatrix} + \frac{\rho_w c_w q_y^{(e)}}{6} \begin{bmatrix} c_i & c_j & c_k \\ c_i & c_j & c_k \\ c_i & c_j & c_k \end{bmatrix}
 \end{aligned} \tag{6.63}$$

where the values of $b_{i,j,k}$ and $c_{i,j,k}$ are as follows:

$$\begin{aligned}
 a_i &= x_j y_k - x_k y_j & a_j &= x_k y_i - x_i y_k & a_k &= x_i y_j - x_j y_i \\
 b_i &= y_j y_k & b_j &= y_k - y_i & b_k &= y_i - y_j \\
 c_i &= x_k - x_j & c_j &= x_i - x_k & c_k &= x_j - x_i
 \end{aligned}$$

and the pairs (x, y) are the coordinates of the vertices of a triangle. The area of the triangle is obtained by evaluating the determinant of the matrix A (det A):

$$\det A^{(e)} = \frac{1}{2} \begin{vmatrix} 1 & x_i^{(e)} & y_i^{(e)} \\ 1 & x_j^{(e)} & y_j^{(e)} \\ 1 & x_k^{(e)} & y_k^{(e)} \end{vmatrix} \tag{6.64}$$

6.4.4 The Revised Code

The groundwater flow computer code GW2 (Chapter 5) was modified to solve the heat transport equation. The revised computer code is called GW3. The modifications are described in Appendix 7. The thermal parameters and the initial and boundary conditions, which are required to solve the heat transport problem are discussed in Chapter 7.

7. THERMAL PROPERTIES, BOUNDARY AND INITIAL CONDITIONS

7.1 General

The purpose of this chapter is to define all of the heat transport parameters which are required to solve heat transport equations in both the analytical and numerical models. These parameters include the thermal properties of the aquifer, the aquifer thickness, the initial conditions (*i.e.*, the initial starting temperature distribution for transient simulations), the boundary conditions (*i.e.*, Dirichlet and Neuman Type boundary conditions), and heat sources and sinks (*i.e.*, injection and pumping wells).

7.2 Thermal Properties

7.2.1 Thermal Conductivity

Heat is a form of energy, measured in joules (J), which is said to “flow” from higher to lower temperatures (Williams, 1982). An equivalent mathematical restatement of this is that heat flows as a result of a temperature gradient. In a purely conductive thermal regime, the heat flow or flux increases in proportion to the thermal gradient according to the equation (Chapter 6):

$$H = -\kappa \text{grad}T \quad 7.1$$

The rate of heat transfer depends upon the thermal property of the material, known as the thermal conductivity, κ . For steady-state conduction problems, only the thermal conductivity needs to be specified. In transient conduction problems, the thermal diffusivity, c/κ , must be known.

Heat flow in porous media is not limited to “pure conduction”. Heat flow occurs in association with the movement of water (and/or air and vapour in unsaturated conditions). In fact, temperature gradients themselves can initiate water movement. The advance of a heating front, as a result of movement of water, is termed advection. In addition to pure heat conduction, small-scale heterogeneities in the aquifer give rise to mixing or a smearing-out of the thermal front. This mixing is termed dispersion (Section 7.2.7). The combination of pure conduction and conduction as a result of dispersive effects can be incorporated into a single parameter, defined as the effective thermal conductivity, κ_{eff} (Chapter 6).

The thermal conductivity is a measure of the quantity of heat that will flow through a unit area of the substance per unit time, under a unit temperature gradient (*i.e.*, $\text{J s}^{-1} \text{m}^{-2} \times (\text{K m}^{-1})^{-1}$, or $\text{W m}^{-1} \text{K}^{-1}$), where K is degrees Kelvin and W is watts (J s^{-1}). Materials with high thermal conductivities transmit heat more readily than materials with low thermal conductivities. For example, pure quartz has a high thermal conductivity of $8.0 \text{ W m}^{-1} \text{K}^{-1}$ (Weast, 1983). A value of $5.6 \text{ W m}^{-1} \text{K}^{-1}$ was measured for quartzite (Judge, 1973). Therefore, rocks with a high quartz content will be good conductors of heat. In contrast, water has a lower thermal conductivity of $0.56 \text{ W m}^{-1} \text{K}^{-1}$. If a quartz sandstone is saturated with water (*i.e.*, all of the pores are water-filled), then, the thermal conductivity of the sandstone will be lower than that predicted for pure quartz because a portion of the rock, depending on the porosity, contains water.

Most soil and rock materials present considerable difficulties for the determination of thermal conductivity. These arise because of the composite nature of the materials.

Conduction of heat occurs at different rates through the water, air and rock constituents. As well, the composition of the rock or soil does not necessarily remain constant with time. Similar materials will have different water contents at different times of the year or in different locations. Grain to grain contact (which acts to increase the thermal conductivity of a material) may also vary depending on the degree of compaction. Therefore, it follows that water table fluctuations play an important role in determining the thermal conductivity of unconfined aquifers.

Comprehensive studies by Johansen (1972; 1973; 1975) led to the conclusion that the conductivity of saturated materials is fairly well represented by the geometric mean of the material components:

$$\kappa = \kappa_1^{V_1} \kappa_2^{V_2} \kappa_3^{V_3} \dots \quad 7.2$$

where κ is the thermal conductivity of the material, κ_1 , κ_2 , *etc.* are the thermal conductivities of the constituents (including water) and V_1 , V_2 , *etc.* are the volume fractions of each of the constituents. Johansen applied this equation to soils, but it could be applied equally to either consolidated or unconsolidated aquifers and aquitards. The equation gives large errors when there is a large difference between the lowest and the highest conductivity. In unsaturated conditions, the equation can no longer be applied since the thermal conductivity of air is very low ($0.022 \text{ W m}^{-1} \text{ K}^{-1}$).

At Carleton, the aquifer regime is confined and saturated. The aquifers are composed primarily of limestone, dolostone and shale. Secondary porosity and permeability have been developed, particularly within the fault zones and these zones of enhanced permeability may extend, somewhat, into the matrix aquifers on either side. The

thermal conductivity of the rock aquifers can be estimated from Equation 7.2, if the porosity of the aquifers is known and the thermal conductivity of the rock material is known.

The apparatus and methodology employed to measure the thermal conductivities of the rocks is described in Appendix 8. A complete set of results for this work is also included. In Section 7.2.2, the porosities of the aquifers are discussed, and the final aquifer thermal conductivity estimates are presented in Section 7.2.3. It should be emphasized that thermal conductivities are rarely, if ever, measured for aquifer heat transport problems. Usually, these are assigned “typical values” for bulk rock types, such as limestones. An added bonus was the calculated grain densities, which were used in gravity modelling (discussed in Chapter 2). Table 7.1 summarizes the results and provides a single estimate of thermal conductivity for each dry rock formation.

Table 7.1: Average Thermal Conductivities for Each Formation at Carleton

Formation	Rock Type (average thickness)	Thermal Conductivity (W m ⁻¹ K ⁻¹)	Formation	Rock Type (average thickness)	Thermal Conductivity (W m ⁻¹ K ⁻¹)
Billings	shale (N/A)	1.789	Lowville	argillaceous limestone (35m)	1.762
Eastview	limestone (6m)	2.026	Pamelia & St. Martin	shaley dolostone (11m)	1.713
Cobourg	limestone (31m)	2.118	Rockcliffe	siltstone (53m)	2.156
Sherman Fall	limestone (some calcarenite) (17m)	2.308	Oxford	dolostone (N/A)	2.656

Table 7.1: Cont'd

Hull	limestone (calcarenite) (34m)	2.510	March	sandy dolostone (N/A)	3.059
Rockland	limestone (22m)	2.147	Nepean	sandstone (N/A)	3.291
Leray	limestone (calcarenite) (6m)	2.286	Basement	metamorphic	2.099

The thermal conductivities are typical of values reported elsewhere. For example, Judge (1973) reported a value for shale of $1.5 \text{ Wm}^{-1}\text{K}^{-1}$, which is similar to the value of $1.8 \text{ Wm}^{-1}\text{K}^{-1}$ obtained for the Billings Formation. Values for limestone can be quite variable. Touloukhian *et al.* (1970) reported a value of $1.0 \text{ Wm}^{-1}\text{K}^{-1}$ and Judge (1973) reported a value of $2.9 \text{ Wm}^{-1}\text{K}^{-1}$. Values for limestone in Table 7.1 range from approximately 1.8 (Lowville Formation) to 2.5 (Hull Formation) $\text{Wm}^{-1}\text{K}^{-1}$. Calcarenites are slightly higher than sublithographic limestones and argillaceous limestones. Dolostones (Oxford Fm.) have relatively high thermal conductivities. The lower value in the Pamela and St. Martin Formations are a reflection of the high percentage of shale. Because thermal conductivity increases significantly as the quartz content increases, high values were measured for the March and Nepean Formations.

The average thermal conductivity for the dry rock formations intersected at Carleton (Eastview through Rockcliffe Formations, inclusive) is $2.17 \pm 0.30 \text{ Wm}^{-1}\text{K}^{-1}$. Since the March and Nepean Formations and the Precambrian basement were not intersected by the wells at Carleton, those values were excluded. The Billings Formation was excluded because it is very thin in the Eastern Block and the wells are cased through

this interval. The Oxford Formation was also excluded because it is only intersected in one production well, P1 (< 4 m).

The average thermal conductivity for all of these formations, weighted according to their thickness, is $2.131 \text{ Wm}^{-1}\text{K}^{-1}$. However, this dry rock thermal conductivity does not really characterize the active aquifers, because some of these formations, in particular, the Hull, the Pamela, the St. Martin and the Rockcliffe Formations, are more permeable due to intense fracturing (Wilson, 1990). If the thermal conductivity is weighted according to the thicknesses of these formations only, κ is $2.23 \text{ Wm}^{-1}\text{K}^{-1}$. This latter value was used to calculate the aquifer thermal conductivities used in the simulations.

7.2.2 Porosity

The porosity of the aquifer will vary depending upon the formation, as well as the proximity to faults where intense fracturing is expected to occur. As well, there may be some zones where secondary mineralization may be present. This mineralization will reduce the overall porosity of the aquifer in these zones.

The porosity, ϕ , of an aquifer is the ratio of the volume of the void space to the volume of the solid material. In terms of the aquifer thermal properties, the porosity determines the volume fraction of aquifer which contains water. Since the aquifer is fully saturated, the porosity is a measure of the total water content. Both the aquifer thermal conductivity (Equation 7.2) and heat capacity (Equation 7.5) are a function of the porosity.

Estimates of the porosity were obtained from drill core from the Observatory Crescent borehole (BH-0-8). This borehole is approximately 200 metres away from any

defined faults and can be expected to be relatively unfractured. Porosity measurements were made on discs cut from the drill core (Michel, 1985). The core extends from the upper Lowville Formation into the Precambrian basement providing porosity estimates for only a segment of the stratigraphic column. The average porosity for the upper Lowville Formation to base of the Oxford Formation is approximately 3.4%. The Lowville and Oxford Formations have lower porosities, 1.4%, while the Pamela, St. Martin and Rockcliffe Formations have higher porosities, 5.5%. It is expected that the upper formations, namely the Eastview to Leray Formations, have an average porosity of approximately the same magnitude, 3%. From the borehole density logs for wells intersecting this interval, the Hull Formation may have a higher porosity, and the Sherman Fall, Cobourg and Rockland Formations may have lower porosities. For the unfractured aquifers on either side of the faults, a porosity of 3% was employed.

Within the fault zones, the number of fractures and, therefore, the aquifer porosity may be somewhat higher. The density log for well P2, which is located directly on a major fault, indicates dense fracturing along the entire length of the borehole. From this log, the porosity was estimated as approximately 10%. This value was used for aquifer material within the fault zones.

7.2.3 Aquifer Thermal Conductivities

Thermal conductivity was required for both analytical calculations of thermal breakthrough time, and for the numerical heat transport simulations. In the former calculations, the caprock heat transfer from the aquifer to the overlying material constitutes a portion of effective conductive heat transfer. The pure conduction

component in the direction of water flow is considered negligible. For a uniformly thick confined aquifer, only a small portion of the heat will be transferred to the caprock. The portion of heat that is lost, due to caprock heat transfer, effectively decreases the thermal breakthrough time. In the analytical simulations, the thermal conductivity in the vertical direction was assigned a value which represents the unfractured matrix aquifer.

In the numerical model generated in this study, heat transfer occurs only in two-dimensional space, and thermal losses or gains from the rock above and below are assumed small. Therefore, the thermal conductivity in the z-direction is not applicable. In real terms, this means that there is effectively no caprock heat transfer by conduction. All of the heat conduction takes place in a horizontal thick plane and is of equal magnitude at all levels within this plane. The only difference in the value of the thermal conductivity across the domain is as a result of variations in porosity. Both the matrix aquifer and the fault zone aquifer can be considered to be homogeneous so that thermal conductivity will vary only between the two aquifer types and not within each aquifer. The aquifers can also be considered isotropic such that the magnitude of the thermal conductivity is the same in the x- and y-directions.

Table 7.2 presents the values of aquifer thermal conductivity, both in the fault zones and in the surrounding unfractured rock matrix. These values were calculated from Equation 7.2 using the porosity estimates of 3% and 10%, which correspond to unfractured matrix and fault zone, respectively, and the weighted value for dry rock thermal conductivity of $2.23 \text{ Wm}^{-1}\text{K}^{-1}$, calculated in Section 7.2.1.

Table 7.2: Aquifer Thermal Conductivities for Carleton Heat Transport Model

Aquifer Type	Porosity, ϕ (%)	Aquifer Thermal Conductivity, κ_x, κ_y ($\text{Wm}^{-1}\text{K}^{-1}$)
Matrix	3	2.11
Fault Zone	10	1.92

7.2.4 Heat Capacity

The amount by which the temperature of a body changes in response to an applied temperature differential, is determined by its heat capacity, C . Heat capacity is best explained by considering a body of material which is initially at some uniform temperature T_0 . If the surface of the body is raised to a different temperature, T_1 , a temperature gradient is established. Heat moves into the body, and the temperature of the body changes and continues to change until steady-state conditions are attained.

The amount by which the body temperature increases or decreases depends upon the ease with which heat flows into the body (*i.e.*, the thermal conductivity), and the temperature change associated with the loss or gain of heat by the material (*i.e.*, the heat capacity). The dual dependence is reflected in the quantity referred to as the diffusivity, which is defined as the ratio of the thermal conductivity to the heat capacity, κ/C .

In an ATEs system, water is reinjected at a well and moves away from this well as a result of a hydraulic gradient. The temperature of the injected water is T_i , which is necessarily different from the ambient groundwater temperature. Therefore, a thermal gradient is established within the aquifer. The water relinquishes its heat (in the case of warm water injection) to the aquifer, and the aquifer responds with an increase in temperature. The equilibrium temperature of the aquifer is determined by the aquifer heat capacity.

Thermal diffusivity can be measured indirectly by measuring temperature or it can be calculated from measurements of thermal conductivity and from estimates of heat capacity. The heat capacities of several rock materials, including sandstones, shales and dolomitic limestones have been measured (Weast, 1983). These values are usually presented as mass heat capacities, c ($\text{J kg}^{-1} \text{ } ^\circ\text{C}^{-1}$), that can be converted to volumetric heat capacities, C ($\text{J m}^{-3} \text{ } ^\circ\text{C}^{-1}$), by multiplying through by the rock density, ρ . A value of 2630 kg m^{-3} was chosen to represent the density of the rock material. Using typical mass heat capacity values for dolomitic limestone, the volumetric heat capacity for the dry rock is estimated as $2.1 \times 10^6 \text{ J m}^{-3} \text{ } ^\circ\text{C}^{-1}$.

The equation which describes the combined heat capacity of a substance is:

$$C = \sum_{i=1}^n V_i C_i \quad 7.3$$

where C is the effective heat capacity, V_i are the volume fractions of each constituent (including water) and C_i are the heat capacities of the constituents.

7.2.5 Aquifer Heat Capacities

The heat capacity of a saturated aquifer, C_{aquifer} , is greatly altered by the heat capacity of water ($C_{\text{water}} = 4.174 \times 10^6 \text{ J m}^{-3} \text{ } ^\circ\text{C}^{-1}$). The aquifer heat capacities, for each of the unfractured matrix and the fault zone rocks, were calculated from Equation 7.3 and the estimates of porosity reported in Section 7.2.2. The volumetric fractions were replaced by the porosity according to the expression:

$$C_{\text{aquifer}} = \phi C_{\text{water}} + (1 - \phi) C_{\text{rock}} \quad 7.4$$

where C_{rock} is the heat capacity of the rock matrix. The volumetric heat capacities for the rock matrix aquifer zones at Carleton have similar magnitudes (Table 7.3).

Table 7.3: Aquifer Volumetric Heat Capacities for Carleton Heat Transport Model

Aquifer Type	Porosity, ϕ (%)	Aquifer Heat Capacity, C ($\text{J m}^{-3} \text{ } ^\circ\text{C}^{-1}$)
Matrix	3	2.16×10^6
Fault Zone	10	2.31×10^6

7.2.6 Aquifer Thickness

Apart from the thermal properties of the aquifer material, variations in the aquifer thickness can have a significant effect on the character of heat transport in the domain, particularly when these variations are large. The aquifer thickness has a direct impact on the value of the heat capacity. Since heat capacity is defined for a unit volume of material, two-dimensional models must integrate variations in thickness into the value of heat capacity used for the simulations. Thinner aquifers have less thermal storage capacity, and the amount of “recoverable heat” is lower. However, if the aquifer is composed of several discrete thin aquifers separated by aquitards, then heat transfer in the vertical direction, through the aquitards (caprock heat transfer), become significant, and contributes to the overall loss of heat from the aquifer. The question becomes one of identifying the active hydraulic portion of the aquifer, as this dictates the intervals where heat transport due to advection dominates.

There has been some debate over this issue as it relates to the Carleton ATES system. The two extremes are; first, that the aquifer consists of a single confined aquifer unit and, second, that the aquifer is actually a series of discrete fractures (*i.e.*, bedding

planes) separated by impermeable intervals. In the latter case, the bedding plane fractures control the groundwater movement and, thus, the heat transport. An intermediate representation is that the aquifer consists of a series of highly fractured and permeable intervals, separated by less permeable units.

There are several arguments against the discrete horizontal fracture aquifer model.

- The geophysical logs do not show any lateral continuity of fractures. Therefore, the hydraulic connection between widely spaced wells cannot be explained by this model.
- The aquifer test data indicate a regime dominated by vertical, as opposed to horizontal, fracture flow (refer to Chapter 4), and shallow wells measure similar hydraulic responses to deep wells.
- The aquifer test data for the M1 multilevel piezometer suggest that hydraulic connection is not associated with dominant or identifiable fractures, and indicate that a significant portion of the aquifer responds to pumping.

Equally, the aquifer is probably best not described as a single unit. There are some intervals within each well which are clearly more densely fractured than others. The dense fracturing, noted on borehole logs for wells located along faults, is not an indication that fracturing is widespread within the geologic units. This dense fracturing is concentrated in zones around faults. Away from major faults, such as in the Central Block, where fracturing is less intense, the aquifer probably does consist of discrete intervals which have higher hydraulic conductivities, for example, the contact between the Rockcliffe and Oxford Formations.

Relative decreases in density and resistivity, and increases in Induced Polarization (IP), from borehole logs (Wilson, 1990), indicate that cumulatively, these fractured or more permeable intervals constitute a large percentage of the uncased open well. In Chapter 3, a differentiation was made between what constitutes an individual fracture, that may influence the temperature profile from the point of view of water entry, and what constitutes a permeable interval that contributes to the overall groundwater flow.

The final consideration is that the rock types forming these aquifers do possess some primary permeability. In terms of heat transport, this is significant because although water may move rapidly in the densely fractured areas, such as within the fault zones, away from these zones, the movement of water will be slower and more heat transfer will take place between the rock and the water.

Clearly, aquifer thickness is of critical importance. The thermal model depends strongly on the choice of aquifer thickness, because the calculations are based on maximum interflow between wells. The smaller the aquifer thickness, the more rapid thermal breakthrough. Calculations for the analytical model were carried out using an aquifer thickness of 60 metres. This value represents approximately 60% of the total length of uncased open well, and was used by WESA (1988) and by CESI (1994) for calculations of thermal breakthrough times. In the numerical model, simulations were run using two values for aquifer thickness, 60 metres and 6 metres. Each value was incorporated into the aquifer heat capacity value and was used to calculate the velocity in each element.

7.2.7 Dispersivity

Dispersion, associated with heat transport (and with contaminant transport), is due to small variations in the velocity of the water (*i.e.*, deviations from the average linear velocity), which are the result of homogeneities inherent to the porous media. Dispersion results in a spreading-out of heat ahead of and behind the thermal front.

The amount of dispersion that takes place in a particular environment is governed by the hydrodynamic dispersion coefficient, D ($\text{m}^2 \text{s}^{-1}$). The dispersion coefficient is a tensor and was represented by Equations 6.38a, b and c (after Molson *et al.* (1992)). The magnitude of the dispersion coefficient depends primarily on the average linear velocity of the water, which is modified in each of the principal coordinate directions by parameters called dispersivities. If the main components of the dispersion tensor are aligned with the principal coordinate directions, the dispersion coefficient in each of the principal coordinate directions can be calculated from the longitudinal, σ_L , horizontal, σ_{TH} , and transverse, σ_{TV} , dispersivities (m), and the average linear velocity (m s^{-1}) in each direction.

There is much debate in the literature concerning dispersion. The dispersivity values are, in a sense, correction factors to account for the fact that it is impractical, and perhaps even impossible, to define the velocity distribution in detail (Anderson and Woessner, 1992). Dispersivities have traditionally been estimated from trial-and-error model calibration and from tracer tests (Gelhar *et al.*, 1985) and should be considered site specific. Results from field experiments support the general finding from micro-scale experiments that transverse dispersivities (both horizontal and vertical) are at least an

order of magnitude lower than longitudinal dispersivities. At the Borden aquifer test site, the longitudinal dispersivity was $\sigma_L=0.1\text{m}$, the transverse horizontal dispersivity was $\sigma_{TH}=0.01\text{m}$, and the transverse vertical dispersivity was $\sigma_{TV}=0.005\text{m}$ (Molson *et al.*, 1992). However, the scale of their model is small compared to that used in this research and the rate of heat and fluid injection is also small in comparison.

One of the biggest problems associated with quantifying dispersivities is the so-called “scale-effect”, whereby dispersivity seems to increase in a down gradient direction. At small scales (micro-dispersion), the heterogeneities are not as apparent as at larger scales (macro-dispersion). Therefore, dispersivity values will tend to increase as the scale of the observation or, for example, as the distance travelled by a contaminant plume increases. Domenico and Schwartz (1991) suggested that it is probable that longitudinal dispersivity values in excess of 10 metres exist, but values are not nearly as large as some studies indicate. They suggested that methodology problems give rise to higher than expected values.

A further problem, which is not well understood, is caused by channelling of groundwater along paths of high hydraulic conductivity, such as fractures. This channelling extends the mixing front away from the source, but only along particular paths. Van der Kamp (1982) suggested that micro-dispersion constitutes a negligible component to thermal smearing unless there is relatively rapid flow through a particularly coarse-grained (or presumably fractured) material.

In large-scale problems, such as at Carleton, most of the thermal smearing can be attributed to macro-dispersion. On a scale of 10 to 100 metres, longitudinal dispersivities can be as high as 0.1 to 7 metres (Van der Kamp, 1982).

Dispersivities for the Carleton campus had to be estimated from published values. It was suggested that tracer tests be conducted between wells located along faults in order to estimate rates of dispersion; however, the system was operational from the beginning of this research and University administrators were unwilling to shut the system down for such tests. Table 7.4 presents the dispersivity values used in the thermal transport simulations. Vertical dispersivities are not included since the model at Carleton is two-dimensional.

Table 7.4: Dispersivity Values Used in Thermal Transport Models at Carleton.

	Dispersivity (m)
Longitudinal, σ_L	1.00
Transverse Horizontal, σ_{TV}	0.10

7.3 Boundary and Initial Conditions

7.3.1 Heat Transport Boundary Conditions

The boundary conditions in a heat transport problem specify the inflow and outflow of heat from the domain. The physical boundaries to the problem domain will necessarily coincide with the hydrologic boundaries, if both groundwater flow and heat transport are modelled together. Boundary nodes can be assigned either a specified temperature, representing Dirichlet or Type I boundary conditions, or they can be assigned a specified flux which includes advection, representing Cauchy or Type III boundary conditions. Neuman type boundaries include zero flux or “no flow” across the boundary.

In three-dimensional problems, the bottom surface of the domain may be represented by a constant flux (geothermal heat flux), which is conductive and is directed upwards. The geothermal heat flux is negligible compared to the strong advective component of heat transport associated with active pumping wells. In regional flow problems, where heat transport is taking place slowly but at a large scale, the geothermal flux must be taken into consideration. In two-dimensional simulations, it is assumed that this component of upward heat flow remains constant with time and that the thermal gradient over the thickness of the aquifer is small.

At the top surface, variations in air temperature induce ground surface temperature fluctuations, which can have a significant impact on the upper thermal regime, particularly if ground freezing is taking place. Consideration of the upper boundary conditions is important when modelling flow and heat transport in unconfined aquifers, as the upper surface is exposed to the air. Molson *et al.* (1992) incorporated phase changes associated with freezing and changes in the degree of saturation into their model, because the aquifer they consider is shallow and unconfined.

At Carleton, near surface thermal disturbances associated with nearby buildings and variations in the mean annual ground temperature only propagate a few tens of metres below the ground surface (refer to temperature logs in Appendix 4). It is unlikely that these will have any significant effect on the heat transport associated with thermal energy storage, since the casing in most of the wells extends at least 30 metres. The active portion of the aquifer lies below the thermally active upper layer. This, and the fact that flow takes

place within a confined aquifer, makes it possible to model the thermal regime in two-dimensions.

Since heat transport associated with advection and dispersion is expected to dominate over purely conductive heat flow, the rate at which heat enters or leaves the domain is a function of the velocity of the water. Velocities are highest near sources or sinks of groundwater, reinjection and pumping wells, respectively, and decrease outward from these wells. The advective and dispersive forces are small at the boundaries of the domain if these boundaries are placed far enough from any active wells.

In contrast to a homogeneous and isotropic aquifer, there are distinct zones, corresponding to the faults, which exhibit preferential hydraulic connection. Since advection and dispersion are greater within these zones, there is a distinct possibility that heat transport within the domain is affected by the boundary conditions imposed at the ends of the fault zones, particularly for pumping wells located along faults. Of course, the pumping and reinjection configuration encourages interflow between wells, which counteract groundwater inflow from the surrounding regions of the aquifer. For pumping wells located at the perimeter of the configuration, there remains the possibility that water is derived from distant hydraulic sources and that these may be thermal sources as well. Observations made regarding the natural variation in groundwater temperatures, measured during aquifer tests, may give some indication of sources of heat to the aquifer.

The steady-state chemical data suggest that the two aquifers on either side of the Eastern Fault have similar chemical compositions. The chemistry of the Eastern Block wells is modified by a source of fresher, fault-derived water, which is driven up the

boreholes by high artesian pressures at depth. The observed steady increase in electrical conductivity with time during each of the constant discharge tests at Elba2 and RC is consistent with this observation and suggests that water is derived initially from the fault zone and later from the surrounding aquifer. This shift in source would be expected in a flow regime dominated by vertical fracture or dyke flow (Chapter 4). Based on the hydraulic evidence, radial flow conditions do occur at later times during aquifer tests. Therefore, for a pumping well located along a major fault, a significant fraction of water is derived from the surrounding aquifer and not from a source located along the fault.

Since the velocities of the thermal fronts are much lower than the velocities of the hydraulic fronts, and the velocities of the disturbances to the thermal regime, produced by a spatially confined pumping and reinjection system, are not expected to propagate a significant distance from the active portion of the system, the selection of thermal boundary conditions is not particularly critical. Nevertheless, two simulations are run using constant temperature nodes (Dirichlet type) and zero flux nodes (Neuman type) around the perimeter of the domain. The constant temperature nodes were given values of 10°C, which is the average aquifer temperature measured during aquifer tests.

7.3.2 Initial Conditions

The initial conditions for the thermal simulations were difficult to determine because there are no temperature data for the operating ATEs system. For the hydraulic simulations, there are data for both long- and short-term variations in the steady-state hydraulic head. These data are representative of initial conditions before the Phase 1 system was put into operation. In addition, there is an excellent source for transient data,

which represent stressed conditions within the hydraulic regime. These data include constant discharge test results, reinjection test results, recovery test results and periodic hydraulic head measurements made during system operation.

The only source for steady-state temperature data was the thermal logs, but these logs are disturbed by high artesian pressures at depth. No permanent temperature cables were installed in any of the wells and most of the temperature sensors at the well heads have either failed or have not operated consistently during the six years of operation. In terms of thermal simulations, this lack of initial temperature data means that it is impossible to calibrate the model with any degree of confidence. The validity of the thermal results relies heavily on the accuracy of the hydraulic model.

In addition, there has never been a period of time when the system has been operating at full capacity and according to the specified pumping configurations. Wells have been arbitrarily turned on and shut off, with little or no official record of the imposed changes. At times, only parts of the Phase 1 system have been operating, and at others, the system has been shut down altogether for maintenance purposes. As well, the temperature differential across the heat exchangers has not been maintained at a constant level. During the peak demand seasons of summer and winter, the system operates at high temperature differentials. In the spring and fall, the wells may continue to pump and reinject, thereby advancing the thermal fronts, but no temperature differential is introduced.

Since there is no information about the temperature field and how it has varied over time, a constant initial temperature throughout the domain was assumed. Since discharge water temperatures, measured during each of the aquifer tests at Elba2 and RC,

remained relatively constant at approximately 10°C, the initial temperature field was represented as a uniform temperature field at 10°C.

In practical terms, this choice of initial conditions means that the thermal models for both the Phase 1 and Phase 2 pumping/reinjection configurations (WESA, 1988; CESI, 1994) are not necessarily accurate representations of the thermal regime. As an example, consider the possibility that pockets of warm water are located in various portions of the aquifer as a result of improper use of the Phase 1 system. These pockets cannot be delineated, therefore, they cannot be included in the initial temperature distribution for any Phase 2 simulations. Nevertheless, the character of heat transport, in relation to the faults, can be examined, and approximations of rate of thermal front migration can be made for the operating system.

7.4 Sources and Sinks

One final consideration for the thermal models is the representation of sources and sinks of heat. In an ATEs system, these are the reinjection and pumping wells, respectively. During summer operation, warmer water is reinjected into the aquifer and a warm thermal front advances away from the reinjection well(s) toward the pumping well(s). This warmer temperature constitutes a positive heat source. In the winter, colder water, reinjected into the aquifer, constitutes a negative heat source (positive sink). At Carleton, summer pumping wells become winter reinjection wells.

In the simulations, only positive heat sources were used. These simulations apply equally to negative heat sources. As well, no attempt was made to reverse the operation of the system, since there was no means to calibrate the model.

Wells can be either Dirichlet, Neuman or Cauchy nodes. In a two-dimensional problem domain, a Dirichlet node represents a line source or sink for heat, that is maintained at a constant temperature. In contrast, Neuman nodes are used in situations when a small amount of heat is added to the system over a brief period of time. The Borden experiment, described by Palmer *et al.* (1992) and by Molson *et al.* (1992), is an example of a time-release, low-volume experiment.

At Carleton, an assumption was made to treat the wells as constant line sources or sinks for heat because the volume of water, which is pumped from and reinjected into each well, is very large. It can be assumed that the temperature of the reinjected water is uniform and that the entire length of the well is maintained at a constant temperature. Thermal simulations were run using reinjection temperatures of 14°C over a six month period. Since the system is not expected to operate at full capacity over the course of a heating season, these simulations represent extreme operating conditions.

8. SIMULATION RESULTS

8.1 Steady-State Hydrologic Models

8.1.1 General

Steady-state simulations were carried out using both the finite difference software *FLOWPATH* (Waterloo Hydrogeologic Software) and the finite element software GW1 (thesis software) to try and establish boundary conditions that could represent the observed static hydraulic head distribution, and to generate an initial head distribution for the transient simulations.

For each of the *FLOWPATH* (Finite Difference) simulations (Figures 8.1 through 8.3), the x-axis is oriented 30°W of true North. The Central Block extends along the x-axis from coordinates (0, 30) to (580, 30), and along the y-axis from coordinates (0, 30) to (0, 525). The Eastern Block makes up the remainder of the domain. These two fault blocks are separated by 5-metre wide homogeneous fault zones (referred to as 'faults' for simplicity), oriented at 90 degrees to each other. The north-south trending fault (Eastern Fault) extends to the boundaries of the domain. The major east-west cross-fault intersects the Eastern Fault (east end) and the domain boundary (west end). The small cross-fault, extending out from the Eastern Fault toward the Elba2 well at (680, 250), is also included.

For the FEM (Finite Element Method) simulations, the mesh (Figure 5.4) is similarly oriented. Fault elements, corresponding to sections or extensions of faults on the mesh, which are not hydraulically active (*i.e.*, those segments which do not represent actual faults) were assigned transmissivity values equal to those of the surrounding

aquifer. As stated in Chapter 5, the 'fault' elements can be activated or de-activated by changing the property set assigned to the elements contained within them.

The hydrogeologic system at Carleton was simulated using several different values of the hydraulic parameters for the aquifers and faults. The results of these simulations using *FLOWPATH* are discussed in order of trial.

8.1.2 Campus1 Simulation: Two Homogeneous and Isotropic Aquifers Separated by Isotropic High Permeability Faults.

In this first simulation, the aquifer system consisted of two homogeneous and isotropic aquifers, separated by high permeability isotropic faults. Aquifer transmissivities, determined directly from aquifer test results, were set at 60 m²/day and 171 m²/day for the Central and Eastern Blocks, respectively. Fault transmissivities of 10000 m²/day were estimated. For all simulations, an aquifer thickness of 60 metres was assumed. This resulted in hydraulic conductivities for the aquifers of 1 m/day (1.2 E-5 m/s) for the Central Block, 2.85 m/day (3.3E-5 m/s) for the Eastern Block and 167 m/day (1.9E-3 m/s) for the faults.

Constant head boundaries in the Eastern Block (Figure 8.1a) were established along the top, partway down along the left, and along the right boundaries (61.2m), and along the bottom boundary (61.1m). In the Central Block, both the bottom and the lower portion of the left boundaries were assigned constant heads (59.95m). No-flow boundaries were imposed along the irregular left side of the domain. These arbitrary boundary conditions were used in order to try and simulate the southwestern direction of

FLOWPATH 5.1 Copyright 1989-1994 Waterloo hydrogeologic software	
Model Dimensions No. rows : 72 No. columns : 62 No. pumps : 0 No. inject : 0 No. const. heads : 181 No. const. flux : 0 No. river nodes : 0 No. drain nodes : 0	
Hydraulic Heads (m) Min : 59.950 Max : 61.200	
Units : [m][day]	
Data Set : CAMPUS1

Figure 8.1b: Calculated Versus Observed Hydraulic Heads for Campus 1 Simulation (Steady-State Data From Wilson, 1990)

groundwater flow established in Chapter 3, although the representation is not necessarily unique.

Widely spaced contours for the simulated static hydraulic heads (Figure 8.1a), in the top portion of the domain, correspond to the higher transmissivity Eastern Block aquifer. These contours become more closely spaced as water moves into the Central Block. Flow lines converge within the Eastern Block, and water appears to exit the domain in the southwest, in response to the 59.95m constant head boundary.

The faults are only partially manifested in the hydraulic head distribution. Hydraulic head contours become irregular (sudden lower gradient) in the portion of the domain where the Elba Fault is truncated. This irregularity is due to the enhanced permeability along the fault compared to the lower permeability of the matrix aquifer at the truncation point (800, 250). The apparent lack of manifestation of the faults may also be the result of poor resolution in the contouring package employed by *FLOWPATH*.

As the domain cannot extend to infinity in the southern direction, there is a single point discontinuity at the bottom point intersection of the Eastern and Central Blocks. This node is located at the domain boundary, and is on the western edge of the 5 metre wide fault zone. As a result, there is a 0.15m difference in hydraulic head from the fault to the Central Block. This discontinuity is strictly the result of the imposed model boundary conditions, and does not represent a realistic situation. In fact, the calculated water balance for the model (approximately $2040 \text{ m}^3/\text{day} \pm 0.5\%$) shows that most of the water ($1560 \text{ m}^3/\text{day}$) enters the system at the node to the right of the discontinuity and exits ($-1986 \text{ m}^3/\text{day}$) at the node on the left. The discontinuity forces the model to simulate the

observed discontinuity between the static hydraulic heads of the Eastern Block and those of the Central Block, but as a result, it creates an entirely unrealistic water balance for the model.

This aquifer representation was unable to reproduce the steady-state hydraulic head distribution (Wilson, 1990). The simulated hydraulic heads in the Eastern Block were 0.35m below the observed heads (Figure 8.1b), while the simulated hydraulic heads in the Central Block were higher than the observed heads (approximately 0.5m). The RMS error for this simulation was unacceptably high (0.33m). In an attempt to generate the static hydraulic head distribution, the aquifer and fault transmissivities, and the boundary conditions were varied. The results for these numerous simulations are not shown; however, the attempts were generally unsuccessful.

The simulation results for this conceptual model indicate two problems. First, there is not a large enough gradient between the fault blocks, and second, the hydraulic heads in the Eastern Block are not high enough. Apart from the hydraulic head discontinuity created along the bottom boundary, the boundary conditions appear to generate the southwest flow direction. However, on the basis of poor simulation results, an alternative representation for the aquifer properties was sought. One possible modification to the conceptual model, that may better simulate the static hydraulic head distribution, was to consider the faults as anisotropic.

8.1.3 Campus2 Simulation: Two Homogeneous and Isotropic Aquifers Separated by Anisotropic High Permeability Faults

The steep hydraulic gradient between the Eastern and Central Blocks could not be reproduced by varying either the boundary conditions or the aquifer transmissivities on either side of the fault, therefore, one recourse was to consider the faults as anisotropic hydrogeologic features (Allen and Michel, 1994). This second representation for the Carleton aquifer regime consisted of two homogeneous and isotropic aquifers (the Central and Eastern Block aquifers) separated by anisotropic fault zones.

There is only indirect evidence from the aquifer test data to suggest that these faults may be anisotropic. This evidence is the delays in response times that were measured in observation wells located across a fault from the pumping well. Anisotropy implies that there must be some sort of preferred orientation to either the fractures or the brecciated material within the fault zone, which leads to excellent conductivity along the faults. In a direction perpendicular to the fault plane, the conductivity is much lower. Geologically, it is possible that a fault breccia or gouge will behave anisotropically, as clay-like or platy mineralization may be present within the fault. Slickensides might impart this type of hydrogeological anisotropy to the material.

The transmissivities for the Central and Eastern Blocks are the same values as those used in the previous models; namely, 60 m²/day and 171 m²/day, respectively. The faults were assigned transmissivities of 10000 m²/day in their longitudinal directions, and 0.3 m²/day perpendicular to the fault plane. This latter value was selected to represent a mixture of limestone with clay minerals. These transmissivities correspond to hydraulic

conductivities of 167 m/day and 0.005 m/day, respectively, for a 60 metre thick aquifer. The boundary conditions were the same as those used in Campus1.

The simulated contoured hydraulic heads (Figure 8.2a) show that flow is predominantly toward the southwest. The flow converges at the boundary to the Central Block, as a result of the anisotropic permeability of the Eastern Fault, and the constant head boundaries along the top, right and bottom boundaries of the Eastern Block. Around the perimeter of the Central Block and within the fault zone, there is a very steep hydraulic gradient, which is the result of very low transmissivities within this zone. The hydraulic heads in the Eastern Block are lowest at the northeastern corner of the Central Block, and the hydraulic head decreases through the Central Block in a steady fashion to reach a minimum of 59.95m at the bottom and lower left boundaries.

The water balance for this simulation is more realistic than in the previous model (Campus1). Approximately 173 m³/day enters and leaves the system through the constant head boundaries surrounding the Eastern Block. A larger percentage of the flow enters and leaves the system at the ends of the Eastern Fault.

The RMS error (0.08m) is low for this simulation (Figure 8.2b), which suggests that this aquifer representation is capable of reproducing the observed hydraulic head distribution under steady state conditions to within slightly more than 0.10m. This simulation demonstrates that anisotropic faults may be included in the conceptual model for the aquifer system to generate a hydraulic head distribution which is close to the observed static head distribution. The simulation also showed that a set of boundary conditions can be arbitrarily selected to achieve good results. On this basis, it could be

Figure 8.2a: Simulated Hydraulic Head Distribution for Two Homogeneous and Isotropic Aquifers Separated by Anisotropic Faults - Campus 2.

FLOWPATH 5.1

Copyright 1989-1994
waterloo
hydrogeologic
software

Model Dimensions

No. rows : 72
No. columns : 62
No. pumps : 0
No. inject : 0
No. const. heads : 181
No. const. flux : 0
No. river nodes : 0
No. drain nodes : 0

Hydraulic Heads (m)

Min : 59.950
Max : 61.200

Units : [m][day]

Data Set : CAMPUS2

Figure 8.2b: Calculated Versus Observed Hydraulic Heads for Campus 2 Simulation (Steady-State Data From Wilson, 1990)

concluded that the model is satisfactory. However, in practice, the model is sometimes tested by imposing a stress, such as pumping, and observing the responses at one or more wells.

8.1.4 Campus3: Steady-State Simulation for Pumping at Elba2. Two Homogeneous and Isotropic Aquifers Separated by Anisotropic Faults

If a model can simulate the response of a well to pumping, it could be considered accurately calibrated or perhaps validated. The Campus2 model was further tested by simulating the 48 hour constant discharge test at the Elba2 well (coordinates 225, 680), which is located approximately 200 metres from the domain boundaries, and very close to a cross-fault.

In this simulation, most of the water enters the domain from the constant head nodes surrounding the Eastern block, as in the previous simulation. However, due to pumping, more water enters the system at the bottom (south end) and the top (north end) of the Eastern Fault. This suggests that the faults contribute to the overall water balance for the system, but that the constant head nodes around the perimeter of the aquifer matrix remain the main source for water.

The distribution of simulated hydraulic heads calculated at steady-state during pumping at Elba2 is shown in Figure 8.3. The discharge rate for the pumping well was 60 L/s (800 IGPM). The minimum hydraulic head for this simulation was observed at the pumping well (41.21m). Assuming an initial hydraulic head of 61.10m at this well (from Campus2), the total simulated drawdown at steady-state is 19.89m.

At Carleton, the constant discharge tests were not continued into steady-state. However, the steady-state drawdown at Elba2 can be estimated by extrapolation of the

Figure 8.3: Simulated Steady-State Hydraulic Head Distribution During Constant Discharge Test at Elba2

constant discharge data. This calculated steady-state drawdown is approximately 18 metres. In consideration of this magnitude of drawdown, the values obtained from the simulation is within an acceptable range for the pumping well. If aquifer test data are only available for the pumping well, this model might be considered calibrated, and might be used for making predictions. However, at Carleton there is an abundance of observation well data with which the simulation can be further restrained.

8.1.5 Steady-State Model Problems

The major problem with the steady-state pumping simulation (Figure 8.3) is that the model did not effectively reproduce the observed drawdowns in the remainder of the aquifer during pump testing. This problem could not have been identified if constant discharge data were only available for one well in the aquifer.

In the pumping simulation, the Eastern Block hydraulic head distribution at steady-state was poor. The initial heads for the Eastern Block wells were approximately 61.10m, and the simulated hydraulic heads during pumping ranged from about 47 to 60 metres, depending on proximity to the pumping well. The hydraulic heads were not low enough, in the vicinity of the faults, to agree with observed values. In the Central Block, the simulated hydraulic heads remained high during pumping in the Eastern Block, and were more consistent with the measured values.

In contrast to the response that is expected in a linear flow regime (Chapter 4), there appears to be no significant connection in the model between wells located at great distances but along faults from the pumping well. The poor connection may be explained by the following:

- The Elba2 well is positioned incorrectly on the grid relative to the faults. As pointed out in earlier chapters, there is little information on the lateral extent of the fault zones. Equally, it is difficult to establish the exact positioning of a well relative to the highly fractured portion of the fault.
- The boundary conditions dominate the model. The closer a well is to a constant head boundary, the sooner the rate of drawdown is reduced. Because the faults extend directly to the boundaries, the boundary conditions become even more closely linked to the interior of the domain. If the ends of the faults coincide with constant head nodes, the amount of drawdown in any well near a fault may be significantly reduced.
- The hydraulic properties of the matrix aquifers or the faults may be incorrectly defined, and result in insufficient connection between the wells in the Eastern Block.

In an attempt to deal with the boundary problem, several additional steady-state pumping simulations were carried out to examine the behaviour of the system under slightly different boundary conditions. These included changing some of the constant head boundaries to constant flux boundaries and observing the responses. These simulations did not improve the model results.

As it is impossible to model the Carleton system at an infinite scale, and more importantly, because the high permeability faults and the aquifers result in rapid transmission of water from great distances, the selection of boundary conditions is particularly critical. The model is constrained by the boundary conditions, and the

incorrect choice of boundary conditions may result in unrealistic behaviour at late time, particularly at steady-state.

The presence of certain time-dependent processes during pumping, such as leakage, precluded the exclusive use of the steady-state model. It was impossible to detect, from the steady-state hydraulic heads during pumping of Elba2, how much, and more importantly, at what time, late time leakage reduced drawdown, because transient data are not available from steady-state models. Semi-logarithmic graphs of drawdown versus time (Chapter 4) suggest that leakage does occur at late time. Therefore, it is desirable to incorporate boundaries that will affect both the transient and steady-state conditions in the aquifer. The boundary conditions can and should be expected to play a dominant role in the hydraulic head distribution, if the boundaries are located within the zone of influence for a pumping well.

The steady-state results for the static and dynamic systems suggest that much of the perimeter of the domain can be represented in terms of constant heads. However, the poor results for the steady-state pumping simulation suggest that one or several components of the system are incorrectly specified, and that the integrity of the conceptual model is questionable. Although the steady-state program was very easy to use, it could not provide sufficient information to calibrate the model properly. Therefore, the transient response of the system to pumping was examined.

Several components of the conceptual model required more careful examination. These included the aquifer parameters, the position of the faults, the proximity of the boundaries, and the connectiveness of the boundaries to the faults. By examining the

transient responses of several wells to pumping, it was possible to identify which boundaries and parameters affect the flow in the aquifer. Because transient simulations require initial conditions that are representative of the aquifer under non-pumping conditions, an initial head distribution was generated using the steady-state Finite Element Method (*FEM*) code, GW1, written for this research (Appendix 7).

8.1.6 The FEM Steady-State Model GW1

The steady-state program, GW1, solves the steady-state groundwater flow equation for a set of specified boundary conditions and produces a distribution of static hydraulic heads that is similar to that produced using *FLOWPATH*. Minor differences occur as a result of the numerical method of solution (*i.e.*, finite differences in *FLOWPATH* and finite elements in GW1). As discussed in Chapter 5, the *FLOWPATH* finite difference grid and the finite element mesh were designed to be geometrically consistent. One notable exception is that the grid uses rectangular elements and the mesh uses two adjacent triangular elements within each rectangular element.

The boundary conditions and the aquifer parameters used in the steady-state *FEM* model were identical to those used in the Campus2 model (*i.e.*, constant head boundaries around much of the perimeter of the domain, and anisotropic faults). This conceptual model for the aquifer provided base conditions from which further calibration using steady state data could be carried out. Modifications to this conceptual model were made, and the final conceptual model was re-tested under steady-state conditions.

The steady-state distribution of hydraulic heads, generated from GW1, was used as the initial conditions for the transient simulations. The hydraulic head output data file was

copied to the input data file for the transient program, GW2. No contour map of the hydraulic heads was produced, since it would be very similar to the contour map shown in Figure 8.2a.

8.2 Transient Hydrologic Models

8.2.1 General

Several hundred transient simulations were carried out using GW2. The primary objective of this exercise was to produce a calibrated model that could be used to simulate both groundwater flow and heat transport in a faulted carbonate aquifer. The calibrated model could then be used to determine whether heat energy can be stored effectively in such an aquifer system.

The aquifer test data at Carleton constitute a unique data set for faulted carbonate aquifers. Pumping well and observation well data are available along the faults as well as in opposite fault blocks. Constant discharge data are available for test periods ranging from 24 to 48 hours. Recovery data and reinjection test data are also available. Phase 1 aquifer test data include constant discharge tests and recovery tests for wells P1, P2, P4, P5, P6 and Elba2. As well, there are data for several reinjection tests, including those between wells P2 and P5 and wells P4 and P1. Phase 2 tests included constant discharge and recovery tests at wells P21, P22 and P23. Pumping well data are only available for well P21. No reinjection tests were performed during Phase 2.

Apart from boundary conditions, which are discussed in the following sub-section, the aquifer parameters were considered somewhat flexible in terms of the values that could be assigned. However, the parameters could not vary much from the values that are

representative of confined aquifers. For example, hydraulic conductivity (or transmissivity) should remain in the range of 10^{-7} m/sec (6.0×10^{-6} m²/sec) to 10^{-1} m/sec (6.0 m²/sec) to be representative of unfractured to fractured limestones and dolostones, or of gravels (brecciated fault material) (Freeze and Cherry, 1979). An aquifer thickness of 60 metres was assumed. Storativity values should remain in the range of 10^{-6} to 10^{-4} (confined aquifers).

Preliminary estimates of storativity, obtained from the aquifer test data (Chapter 4), are summarized in Table 5.4. The storativity for the Central Block was estimated as $4.2\text{E-}5$ and was determined from one measurement. The storativity for the Eastern Block was estimated as $2.8\text{E-}4$. This estimate is highly questionable since it reflects the excellent connection between widely separated wells. The value may not be representative of the Eastern Block aquifer. Because of the uncertainty in these estimates, aquifer storativity was permitted to vary significantly during modelling, but was kept within an acceptable range for a confined aquifer. The storativity for the faults was estimated as $<1\text{E-}5$, to represent a lower storage capacity than the aquifer matrix.

The model was calibrated using the constant discharge test data for the Phase 1 wells only, for several reasons. First, these data are relatively consistent (*i.e.*, all data were obtained within a short period of time, and before the system was put into operation). Second, Phase 2 data are suspect, since pumping and reinjection was ongoing between wells P1 and P4 during these tests. As well, the static conditions may have been significantly altered as a result of system operation over a period of approximately four years. It is unlikely that steady-state conditions were attained prior to the initiation of the

Phase 2 testing program, therefore, the initial conditions in the model will not be strictly valid.

Given the available information, and the inherent complexity of the model, the aim for this research was to calibrate the model for Carleton using the constant discharge data from Phase 1, such that all of the following were reproduced:

- The time-dependent drawdown in all pumping and observation wells to within approximately 5 metres. The author considers this a reasonable level of confidence, given the uncertainty of fault width, and the approximation made to the actual orientation of the faults in the mesh. At least five observation wells per test were included. These wells were located both in the fault block where pumping was taking place, and in the opposite fault block.
- The relative delays in response times for wells located along faults and for wells located across fault boundaries.
- The observed change in slope in the semi-log graphs at late time that indicates leakage.
- The general semi-log curve shape throughout the test (*i.e.*, no unusual irregularities in the curves).

In addition to the constant discharge tests for Phase 1, two reinjection tests were simulated. The first was from P5 to P2, and the second was from P4 to P1. These latter simulations were undertaken as a somewhat independent check of the accuracy of calibration.

8.2.2 Boundary Conditions

In the steady-state simulations, the boundary conditions were identified as one important aspect of the conceptual model that needed to be more clearly defined. During the Eastern Block constant discharge tests, leakage, manifested as a decrease in slope in the semi-log graph of drawdown versus time, becomes noticeable approximately 15 hours into the test. Of course, the onset of leakage is dependent upon the rate of discharge. The higher the discharge rate, the earlier the effect is noted. One exception is well P5, which displays little, if any leakage, despite being pumped at a rate identical to other Eastern Block wells (refer to Appendix 6).

In contrast, the Central Block wells show a distinct change in slope relatively early in the test. The break in slope occurs around 100 minutes. No leakage is noted in the observation wells in the opposite fault block during any of the Central Block tests. However, leakage does occur at approximately the same time for the Central Block wells as for the Eastern Block wells during the Eastern Block pumping tests. This similar time suggests that the source area for leakage is within the Eastern Block.

The vertical fracture (dyke) flow model appears to be valid for the Carleton aquifer (Chapter 4). In this model, flow is initially derived along the fault (parallel flow period). At intermediate times (pseudo-radial to radial flow), water comes from the matrix aquifer surrounding the fault. If this radial flow period is maintained, then the main source for leakage must be situated at some distance from each of the Eastern Block wells. Furthermore, if this model is correct, it would seem unlikely that the flow regime would revert to a non-linear one at late time. This implies that leakage into the system may not be

necessarily associated with the Eastern Fault, but may be associated with other faults situated at some distance away from the perimeter of the domain.

The preliminary transient simulations used the same boundary conditions as the steady-state simulations, although they had still not been verified. One constant head boundary, extending from the top left edge of the domain, across the top boundary and down the right side, was maintained at 61.2 metres. A 61.1 metre constant head formed the southern boundary to the Eastern Block, as well as the southern end of the main Eastern Fault. A 59.95 metre constant head formed the bottom and western lower left boundaries of the Central Block.

The simulated drawdowns, in these preliminary transient simulations, were too small compared to the observed drawdowns. The breaks in slope in the semi-log graphs, which reflect leakage, were reproduced; however, these occurred too early in the simulation and tended to be flatter than the observed curves (refer to Appendix 6). Several approaches were used to overcome both the insufficient drawdown problem and the early leakage response time. Each met with varying success.

- 1) The storativity of the aquifer was decreased. This resulted in an earlier leakage response time, but no significant increase in drawdown.
- 2) The faults were truncated, just short of the boundary. This provided additional delay time, but failed to provide an adequate drawdown response. Truncating the faults can be justified on the basis of geological information for the campus (Chapter 2 and Figure 2.10).

3) The boundary to the domain was extended artificially by reducing the transmissivity of the outer elements. For example, the Central Block aquifer had a transmissivity of $60 \text{ m}^2/\text{day}$. To make the boundary appear to be ten times as far, the transmissivity for the outer element was reduced by a factor of ten (*i.e.*, to $6 \text{ m}^2/\text{day}$). Storativity values were adjusted in accordance with the “extended aquifer” to account for additional water available from the boundary aquifer zone. If the aquifer boundary is located physically ten times as far, and each boundary element has a width of 20 metres, then nine additional elements, equaling a total of 200 metres of aquifer, is represented by the one boundary element. This means that aquifer storativity for those boundary elements should be increased by a factor of ten to represent additional storage.

The results of the third representation were very successful. These boundary elements were assigned different property set numbers, and each block and fault was assigned a different boundary transmissivity and storativity. For the Eastern Block, the boundary elements, situated adjacent to the 61.2m constant head nodes were assigned a value of $T=0.0001 \text{ m}^2/\text{sec}$, and a value of $S=1.5\text{E-}5$ (Table 8.1). The constant head is effectively moved away 740 metres. The boundary elements along the bottom portion of the domain (south end of the Eastern Block and the Central Block) were assigned a value of $T=0.00075 \text{ m}^2/\text{sec}$, and a value of $S=1.5\text{E-}5$ (Table 8.1), which are the same as the values for the Central Block.

These boundary elements essentially acted as fine tuning elements and controlled the amount of leakage and the time delay for the onset of leakage. The constant head

boundaries for most of the Eastern Block appear to be located approximately 0.75 km away from the edge of the domain. This places these boundaries very close to the faults that extend on the other side of Dows Lake, and which define approximately the west, north and east sides of the entire Eastern block beyond the campus. Although these boundaries have been imposed to reproduce the observed hydraulic heads under transient conditions, they may have some physical basis.

Finally, the hydraulic head distribution for the multi-well configuration (Section 8.4.4), showed that flow was essentially perpendicular to these boundaries, and that no distortion of the equipotential lines occurred. Therefore, it appears that the technique of assigning low transmissivities to elements in a zone around the perimeter of the mesh can be used to extend artificially the domain of a model.

8.2.3 Fault Anisotropy

Several transient simulations were run initially to re-test the conceptual model that used isotropic faults with transmissivities that were high in both the principal directions. These could represent highly brecciated or fractured fault zones (in three-dimensions). For these simulations, the aquifer properties within each fault block were modified and the calculated drawdown curves were compared to actual drawdown curves. The results were poor, as expected from the unsatisfactory results obtained from the steady-state model. However, the simulations did indicate that the faults must have high hydraulic conductivities in order to reproduce the immediate responses between wells located along the faults.

Later simulations incorporated anisotropic faults. These used high transmissivities along the length of the fault and retained a transmissivity similar to, or even lower than, that of the surrounding aquifers in the transverse or across-fault direction. One problem became apparent at the outset. At the intersections of the faults, the water was essentially blocked. For example, if a fault has a high hydraulic conductivity in a north-south direction, then, any east-west fault that intersects this fault is not in hydraulic connection with it, because the principle directions of anisotropy are perpendicular. To counter this problem, the fault intersection elements (two triangles forming a 5m x 5m square) were assigned high isotropic hydraulic conductivities. In this way, water could be easily channelled from one fault to another.

Fault storativities were permitted to vary significantly for successive simulations; from 10^{-8} to 10^{-5} . The value of fault storativity did not appreciably affect the model results because of the narrow width of these features in relation to the rest of the aquifer. Therefore, the storativity for the faults was kept at the upper limit of 10^{-5} for the remainder of the simulations. This value suggests that the faults do not store any more or any less water than the surrounding aquifer, a point which could be argued, but which cannot be resolved with the available information. However, it should be emphasized that the fault zones are not strictly planar fractures and, so, should have some inherent storage capacity.

Relative delays, and lower total drawdown in observation wells, could be simulated under only certain conditions. The first involved reducing the transmissivity of the pumping block. This resulted in excessive drawdown in the pumped block, particularly around the pumping well. To counter this effect, the storativity was increased in the

pumping block. This approach appeared to work exceptionally well for uni-directional pumping simulations (*i.e.*, always simulating pumping in one block and observing in the opposite block). However, when the pumping direction was reversed, the results no longer agreed with observed data. Therefore, it was determined that all pumping tests should be simulated with each change in aquifer properties. This, of course, meant at least six separate simulations per change in aquifer properties.

Despite the success at using this method in the steady-state models, the transient simulations were largely unsuccessful because they were unable to reproduce consistently the relative delays in response time measured in observation wells located on the opposite side of a fault to the pumping well. As a result, the transient drawdowns for these wells were not within the desired calibration range. The aquifer and fault parameters were varied, in some cases by orders of magnitude, but these changes did not improve the model results.

8.2.4 The Low Permeability Zone

The calibration results were greatly improved by replacing the anisotropic faults with a low permeability zone on the inside edge (*i.e.*, the Central Block side) of both the Eastern Fault and the main cross-fault. Elements, equivalent in size and shape to those forming the faults, were assigned an arbitrary low transmissivity (final value 0.00001 m²/sec) and a lower storativity (10^{-6}). This boundary extended to the domain boundary elements. The faults reverted to their former isotropic representation.

The constant discharge test simulations improved significantly as a result of including the hydrogeologic barrier. Total transient drawdown at the pumping well and at

the observation wells were within approximately 5 metres of the observed values. In addition, the relative delay times, for observation wells located on the opposite side of a fault to the pumping well, were reproduced. Simulated delay times were much closer to observed delay times than in previous simulations. Minor discrepancies can be attributed to several factors:

- incorrect positioning of the faults relative to an individual well;
- incorrect fault width;
- incorrect barrier width or transmissive and storage properties.

The mesh included a 5 metre wide fault zone. The actual faults may be narrower and the rate of water flow across them may be faster. Alternatively, the faults may be wider, and the rate of water flow slower. The actual fault widths cannot be determined from the available information. The barrier was also given an arbitrary width of 5 metres. For the purpose of this research (*i.e.*, simulating well interactions), reproducing the relative delay times that are observed between wells located on opposite sides of a fault was considered more important than trying to produce a perfectly-calibrated model.

There are several arguments in favour of introducing the hydrologic barrier into the system, apart from those related to model calibration. As noted in Chapter 3, several researchers concluded that deep drilling within the Ottawa region would likely result in low yield water wells (Brandon, 1960). The presence of hydrologic barriers, associated with faulting in the region, may explain the low well yields reported in regional studies. In addition, calcite-filled fractures have been noted in outcrop within the Ottawa region

(Forgrave, 1995). Wilson (1990) also indicated that calcite was present in drill cuttings from the upper fault zones.

The presence of the barrier may be the result of precipitation of calcite within the fault zone due to mixing of two water types. The large displacement between the Central and Eastern Blocks has resulted in the juxtaposition of two lithologically distinct fault blocks. Although the chemical composition of the type waters in each block is similar (Chapter 3), the contrast may be sufficient to cause precipitation. Alternatively, the fresh fault water, which modifies the apparent chemistry of the Eastern Block wells, may also lead to precipitation within the fault zone. It is difficult to determine whether calcite forms this barrier, although R. Brown (Department of Earth Sciences, Carleton University, Personal Communication) suggests that precipitation of calcite in open cavities created by faulting is common within fault zones situated in carbonate rocks.

The model included only one barrier on the Central Block side of the Eastern Fault and the cross-fault. A further point for consideration is that a similar barrier may be present on the opposite side of the fault. This might help to explain why well P3, located along the Eastern Fault, has such a poor yield. Well P3 is located as close to the Eastern Fault as well P22 on the opposite side, yet its yield was an order of magnitude lower. Unfortunately, there are no constant discharge data for well P3 that could be used to test this hypothesis.

8.2.5 Transmissivity and Storativity Values

The best estimates of the hydraulic properties, for each property set, are provided in Table 8.1. Each element in the domain was assigned a property set (Figure 8.4). The

aquifer regime remained divided into two primary fault blocks (1 & 2), separated by faults (3). High transmissivity fault intersections were included (early simulations) to permit water transfer between anisotropic faults intersecting at ninety degrees (4). The domain was then surrounded, in part, by boundary elements (5 & 6) with lower transmissivities, that represent distant constant head boundaries. A low-permeability zone or barrier was also included on the Central Block side of the main Eastern Fault and major cross-fault (7). The colour-coded property sets appear in the legend accompanying Figure 8.4.

Table 8.1: Element Property Sets, Aquifer Type Description and Aquifer Parameters

Property Set Number	Aquifer Description (general type)	Transmissivity (m ² /sec) (m ² /day)	Storativity
1	Central Block	0.00075 (64.8)	1.5E-5
2	Eastern Block	0.0037 (319.7)	1.0E-5
3	Fault	1.0 (86400)	1.0E-5
4	Fault intersection	1.0 (86400)	1.0E-5
5	Eastern Block boundary zone	0.0001 (0.86)	1.5E-5
6	Central Block boundary zone	0.00075 (64.8)	1.0E-5
7	Fault block barrier	0.00001 (0.86)	1.0E-6

An important conclusion is that the best estimates of the transmissivity and storativity for the aquifers (Table 8.1), which were determined through calibration, are

Figure 8.4: Property Sets Used in Final Model

slightly higher than the preliminary estimates based on the aquifer test results (Table 5.4). The latter could not simulate the hydrologic regime under transient conditions because they are more representative of the barrier than of the aquifers.

In the vertical fracture (or vertical dyke) flow model, the transmissivity of the aquifer is calculated from the slope of the linear portion of the drawdown versus log time curve. This linear portion represents radial or pseudo-radial flow conditions, following a period of linear or parallel flow. One assumption for this model is that the aquifer on either side of the fracture is homogeneous. This particular analytical model cannot detect the presence of a barrier.

At Carleton, the low-permeability zone, identified through modelling, causes the steep upward slope in the semi-log graphs of drawdown versus time. In Chapter 4 (Figure 4.11), this slope was attributed to the shift to radial flow, not to a barrier. Therefore, the transmissivities, calculated from the slopes of the graphs, are too low, and they are not representative of the aquifer.

The actual aquifer transmissivities are better represented by a much shorter portion of the semi-log plot which immediately follows linear flow, but precedes the barrier effect. This effect is difficult to illustrate because most wells are located close to faults and, therefore, the barrier is quickly felt during pumping. However, in the constant discharge test for well P5 (Appendix 6), the curve does not swing upwards at as fast a rate as that for a well such as P2, which is located right along a fault. This is because P5 is situated away from the main Eastern Fault and the barrier. The transmissivity for the Eastern Block aquifer, when calculated from the portion of this curve (*i.e.*, before it starts to swing up

sharply), is very near to the value of 320 m²/day (0.0037 m²/sec) used in the simulations. This value is approximately 150 m²/day higher than the value calculated from the aquifer test data.

A transmissivity of 65 m²/day was used to represent the Central Block in the simulations. This value is very close to the value derived from the P1 and P4 constant discharge test data. The reason for the close agreement between the transmissivity determined by simulating well responses and that determined directly from the aquifer test results is that both wells are located far away from the barrier, and are on the same side of the fault as the barrier. The radial flow period is relatively long. However, there is a dominant late-time change in slope in the P4 data, which represents leakage across the barrier. When more water becomes available from the Eastern Block, the rate of drawdown declines. Without the transient simulations, it would have been very difficult to attribute the steep portion of the constant discharge data to a barrier.

8.2.6 Constant Discharge Test Simulation Results

Constant discharge test simulations were run for each of the six Phase 1 wells (P1, P2, P4, P5, P6 and Elba2). The simulated data for all the pumping wells and several observation wells are plotted along with the observed data for each test in Appendix 9.

Figures 8.5a to 8.5d show simulated drawdown versus time and the actual drawdown versus time for pumping well P2 and three observation wells (P1, Tory and Elba2) during the constant discharge test. The withdrawal rate was 60.2 L/s. The pumping well data (Figure 8.5a) show a good correlation throughout most of the test. The

CONSTANT DISCHARGE TEST AT P2 Observation Well P2

Figure 8.5a: Simulated Data and Actual Data for Constant Discharge Test at Well P2 - Observed at Well P2.

CONSTANT DISCHARGE TEST AT P2 Observation Well P1

Figure 8.5b: Simulated Data and Actual Data for Constant Discharge Test at Well P2 - Observed at Well P1.

CONSTANT DISCHARGE TEST AT P2 Observation Well TORY

Figure 8.5c: Simulated Data and Actual Data for Constant Discharge Test at Well P2 - Observed at Well Tory.

CONSTANT DISCHARGE TEST AT P2 Observation Well E2

Figure 8.5d: Simulated Data and Actual Data for Constant Discharge Test at Well P2 - Observed at Well Elba2.

simulated upward curvature begins slightly later than observed; however, the transient drawdown values remain consistently within 1.5 metres of the observed values.

Simulated data for each of the two Central Block wells, P1 and Tory (Figures 8.5b and 8.5c) also remain consistently within 1.5 metres of the observed values. More importantly, the relative response delay times (*i.e.*, the time at which the upward curvature begins), for each of these cross-boundary wells, are reproduced. At early time, the difference is within a few tens of minutes, but because the response is exponential (plotted on log scale), the time difference appears to be more pronounced later in the simulation (*i.e.*, there is a longer time interval). The delay remains consistent with exponential time.

Simulated data for the Elba2 well (Figure 8.5d) are also consistent with observed data. The actual data (Appendix 6) show that Elba2 responds at almost the same time, but slightly after wells RC and P3, respectively. Thus, the further along the Eastern Fault, the longer the delay time. This relative delay time, for all of the fault wells in each simulated test, was reproduced.

The late time leakage, or downward deflection in the drawdown versus time curve, was also reproduced. Figures 8.5a through 8.5d all show simulated breaks in slope at late time which are consistent with observed data.

In Figure 8.6, the residuals are plotted as a function of time for each of the pumping wells. Wells P1, P2, P4 and P5 were consistently within 2 metres of the observed values. In most simulations, the observation well responses were equally as good (refer to Appendix 9 for plotted data). Very early time variations can be attributed to wellbore storage effects. Wellbore storage will cause actual drawdown in the well to be less than

RESIDUALS BETWEEN SIMULATED AND ACTUAL PUMP TEST DATA FOR PUMPING WELLS

Figure 8.6: Graph Showing Residuals Between Simulated Data and Actual Constant Discharge Test Data for Phase 1 Pumping Wells

that expected from theoretical and, hence, simulated values. As well, early time data are also plagued by non-constant pumping rates. Although a constant discharge rate is sought, sometimes it takes a few minutes to achieve a constant rate. This rate can also vary slightly throughout the test, if not monitored, and will invariably cause the drawdown to differ from that expected if a constant discharge rate is maintained.

Wells P6 and Elba2 were the poorest of the simulations. Simulated drawdown in well P6 was consistently 4-6 metres above the actual drawdown; simulated drawdown in Elba2 was consistently 4-6 metres below the actual drawdown. The reason for this discrepancy is probably related to the inaccurate positioning of the wells relative to the faults (refer to Chapter 5). For example, by placing well P6 just slightly farther away from the main Eastern Fault, its simulated drawdown would be reduced. Likewise, Elba2 could be placed slightly closer to the fault to achieve similar, but opposite, results. It was considered more important to reproduce relative responses between wells rather than actual drawdowns.

The use of transient data for calibration was a very effective method both for achieving the desired responses at wells, in terms of the magnitude of drawdown and delay time, and for observing the effect on the transient data if parameters were varied.

Transmissivities for the aquifers could be varied by approximately $5\text{-}10\text{m}^2/\text{day}$ and storativities could be varied by $2.0\text{E-}6$ without affecting the calibration. The hydraulic properties for the faults and the barriers were more flexible, and order of magnitude estimates were made. The model was slightly more sensitive to changes in the

transmissivities of the boundary elements, as these controlled the amount of leakage into the system.

8.2.7 ReInjection Test Simulation Results

Prior to examining the thermal behaviour, two simulations were run to examine the behaviour of the system under reinjection conditions. Because the model had already been calibrated using the constant discharge data, the reinjection tests were expected to follow suit.

The first reinjection test was for well P5 pumping and well P2 reinjecting. The discharge rate was 60.2 L/s, the same as that used in the actual test. The drawdown (positive values) and the reinjection head (negative values) versus time are shown in Figure 8.7. The simulation was run for approximately 2 months to observe the steady-state hydraulic head distribution. The actual test was run for approximately one day (1410 minutes). In addition to the pumping and reinjection well, the simulation included four other observation wells, Elba2, P1, P4 and Tory. Table 8.2 shows actual drawdown and positive head values for each of the six wells. Positive head values for the reinjection well were calculated from pressure measurements (in P.S.I.).

At early time, the simulated drawdown (Figure 8.7) for the pumping well, P5, increases from approximately 8 metres at 10 minutes to just over 10 metres at 100 minutes. Thereafter, the drawdown remains steady at approximately 10.5 metres. The actual data have a similar magnitude. Between 4 and 6 minutes, there is a rapid increase in drawdown from 4 to 13 metres. The water levels then rise to reach a drawdown of approximately 9 metres at the end of the test.

**SIMULATED REINJECTION TEST DATA
WELL P4 TO P1 @ 42L/SEC**

Figure 8.7: Simulated ReInjection Data for P4 to P1 Test at 42 L/sec.

Table 8.2: Actual Data for Reinjection Test from Well P5 to Well P2. Discharge Rate 60.2 L/s (Data From WESA, 1988).

Time (minutes)	drawdown (+) / Positive Head (-) (metres)					
	P5	P2	E2	P1	P4	Tory
2	3.57	-0.10	-0.10	-0.02	0.01	0
4	3.82	-0.20	-0.20	0.00	0.02	0
6	12.99	-0.30	-0.30	0.02	0.03	0
8	11.89	-0.40	-0.32	0.04	0.03	0
10	11.85	-0.50	-0.36	0.06	0.04	0
110	10.09	-0.79	-0.64	0.21	0.04	-0.06
450	9.60	-1.22	-1.12	0.00	-0.06	-0.30
1410	9.35	-1.89	-1.83	-0.31	-0.41	-0.58

Simulated data for the reinjection well, P2, also agree well with the actual data. At 10 minutes, the simulated reinjection head is approximately 1 metre, compared to the actual value of 0.5 metres. The simulated positive head remains relatively constant at just over 1 metre for the remainder of the test. The final positive head in the reinjection well during the test measured 1.89 metres.

Simulation data for all four of the observation wells agree moderately well with actual data. Simulated data show a steady increase in drawdown in these four wells to just under 2 metres. Actual data are more variable. Elba2, for example, measures a positive head of just under 2 metres at the end of the test. This well, as noted in an earlier section, generally showed less drawdown in the constant discharge simulations because it was placed slightly too far from the fault. This would also account for the discrepancy in the reinjection tests. Well P1, P4 and Tory all measured slight positive heads at the end of the test (Table 8.2), whereas, simulated data indicate slight drawdowns. However, the difference between the two is only about a metre.

The second simulated reinjection test was between P4 and P1, where P4 was pumping at a rate of 42 L/s and reinjecting into well P1. These two wells are located in the Central Block and were, in fact, operated for several years as a thermal couplet, albeit in the opposite direction. Figure 8.8 shows the simulated drawdown (+) and positive head (-) values for each of same six wells used in the previous simulation. Table 8.3 shows the actual drawdown (+) and positive head (-) values for this test.

Table 8.3: Actual Data (Some Interpolation Necessary) for Reinjection Test From Well P4 to Well P1. Discharge Rate 42 L/s (Data From WESA, 1988)

Time (minutes)	drawdown (+) / Positive Head (-) (metres)					
	P5	P2	E2	P1	P4	Tory
2					4.85	0.03
4					5.20	0.01
6				-6.00	6.40	-0.04
8				-8.17	9.10	-0.10
10			0.00	-10.90	11.95	-0.18
110		0.04	0.13	-6.80	14.89	-0.65
450		0.13	0.19	-29.96	15.05	-0.50
1410		0.57	0.65		21.92	0.57

The measured drawdown in the pumping well, P4, was 21.92 metres at 1410 minutes. This compares well with the simulated value of 22 metres. The measured positive head (converted from reinjection pressure, measured in P.S.I.) in the reinjection well, P1, was 29.96 metres at 450 minutes. This is slightly higher than the simulated value of 25 metres. Eastern Block observation wells, Elba2 and P2 (P5 unavailable), measured just over 0.5 metre of drawdown at the end of the test, compared to approximately 2 metres in the simulation. The head variation measured in the Tory well is similar in magnitude and pattern to that of the simulation.

SIMULATED REINJECTION TEST DATA WELL P5 TO P2 @ 60.2L/SEC

Figure 8.8: Simulated Reinjection Data for P5 to P2 Test at 60.2 L/sec.

The reinjection tests are probably more difficult to simulate, since the simulations cannot account for reinjection efficiency. If a well has a lower reinjection capacity, such as well P1, then the measured reinjection head will be higher than the simulated value. This could explain the poorer results in the second simulation. Of course, if the water is more difficult to reinject into a well, then all observation wells, including the pumping well will be affected to varying degrees, depending upon their proximity or connection to the reinjection and pumping wells.

In the author's opinion, the reinjection test simulation results were encouraging. These simulations were considered to be a test of the accuracy of the model calibration, and it was concluded that the model is probably accurate in its predictions of drawdown and reinjection head to within a couple of metres of the actual values. From an engineering point of view, this is considered reasonable for design purposes. From a hydrogeological point of view, and with due consideration to the complexity of the aquifer system, the results are very good.

8.2.8 Return to Steady State

Once the transient simulations were completed, the input parameters were used to recalculate the static hydraulic heads using the steady state program, GW1. During the transient calibration process, constant head nodes were placed farther away by changing the hydraulic properties of the elements surrounding most of the domain. The added distance of these nodes was required to simulate the well interactions on campus. For these boundary conditions to be considered representative of the system, they must generate the steady state static head distribution.

The steady-state solution was very similar to the original simulation results using both *FLOWPATH* and GW1 (Section 8.1.4). Although these original simulations used anisotropic faults to simulate the apparent discontinuous drop in hydraulic head across the Eastern Fault, the effect of anisotropy produces a similar result to including a low-permeability zone adjacent to the fault.

The simulated hydraulic heads at each of the deep Eastern Block wells (Wilson, 1990) are between 61.10 and 61.20 metres. Actual static heads for these wells range from 61.13 (E2, P2) to 61.16 (P6) (Figure 3.3). The model produced a high gradient across the Eastern Fault and cross-fault, with hydraulic heads within the Central Block ranging from 60.30 to 59.90 metres. Actual hydraulic heads for the deep Central block wells range from 60.37 (P1) to 60.01 metres (Tory) (Figure 3.3).

8.2.9 Discussion

The unexpected results from the transient simulations, in terms of the final aquifer representation (the hydrologic barrier) and the hydraulic parameters (higher values than calculated from aquifer testing), suggest that a more sophisticated approach, than that currently used in industry, should be applied to groundwater modelling. This approach, which includes calibrating the model using both steady-state and transient data, is necessary when there is geological and hydrogeological evidence to suggest that the aquifer may be non-uniform. The incorrect choice of hydraulic parameters for a numerical model, or for an analytical model, can have some serious implications if the transport of contaminants or heat are involved.

Furthermore, the complexity of the boundary conditions, that are needed to reproduce the observed transient hydraulic test data, demonstrate the need for additional physical information on the aquifer region beyond the model domain. Although the boundary conditions used for the model lack a strong physical basis, there are indications that a form of constant head boundary is present west, north and east of the campus.

The author recognizes that the information necessary to undertake this type of detailed modelling may not always be available. Seldom are data from multiple pumping and observation wells obtained, and seldom is the time available for hydrogeologists to achieve a perfectly calibrated model. Nevertheless, when faults are suspected to exist on the basis of geological information, and when aquifer testing suggests that preferential connection between widely spaced wells, and poor connection between nearby wells, exist, then the possibility of hydrologic barriers associated with the faults should be considered.

8.3 Analytical Heat Transport Model Results

8.3.1 Maximum Pumping Rates for Zero Thermal Breakthrough

The results of analytical modelling, which was carried out by the author on behalf of Carleton Energy Services Inc., are presented in this section. The modelling results were included as part of the final Phase 2 Drilling and Testing Report to the University (CESI, 1994) and were used to determine an optimum pumping and reinjection configuration for the Phase 2 expansion of the Carleton Groundwater Energy System. An analytical model was employed because of its relative ease of use and because no adequate numerical

model was available which could simulate the complex hydrogeological conditions at Carleton.

The thermal calculations were done initially to identify maximum pumping rates for individual well couplets, such that thermal breakthrough within a six month period would be minimized. Based on these calculations, the wells were grouped into summer (pumping/reinjecting) and winter (reinjecting/pumping) wells. Analytical modelling to calculate optimum pumping rates followed. These results were presented in earlier chapters.

The maximum pumping rates for well couplets, at which no thermal breakthrough is expected, were calculated using the general analytical heat transfer equation for well couplets (Equation 6.17; Gringarten and Sauty, 1975). The results are shown in Table 8.4. The couplet is identified in columns 1 and 4, and the separation between the wells is shown in columns 2 and 5. Well M1 was included in the calculations since it was targeted as a possible production well at a later time.

An aquifer thickness of 60 metres was used, and the rate of injection was equal to the rate of pumping. Calculations were based on a six month period, which is the maximum heating or cooling season. The aquifer properties were evaluated in Chapter 7. Caprock heat transfer, using a thermal conductivity of $2.2 \text{ Wm}^{-1}\text{K}^{-1}$, results in a slight (approximately 2%) increase in the maximum pumping rates relative to those calculated without caprock heat transfer. For example, between wells P1 and P2, the maximum flow rate is increased by 2.64 L/s.

Table 8.4: Maximum Pumping Rates Calculated for Well Couplets Using An Analytical Thermal Model (Model After Gringarten and Sauty, 1975).

Well Couplet	Well Separation (metres)	Maximum Production Rate (L/s)	Well Couplet	Well Separation (metres)	Maximum Production Rate (L/s)
P1 - P2	251	151	P22 - Elba2	201	97
P1 - P4	145	50	P22 - P2	290	202
P1 - P5	248	147	P22 - P23	191	87
P1 - P6	442	468	P22 - P4	69	11*
P1 - Elba2	404	391	P22 - P5	389	363
P1 - M1	46	5*	P22 - P6	533	681
P1 - P21	358	307	P22 - M1	250	150
P1 - P22	207	103	P23 - Elba2	388	361
P1 - P23	76	14*	P23 - P2	175	73
P2 - P4	283	192	P23 - P4	152	55
P2 - P5	168	68*	P23 - P5	213	109
P2 - P6	244	143	P23 - P6	381	348
P2 - Elba2	465	519	P23 - M1	74	13*
P4 - P5	357	305	M1 - Elba2	448	481
P4 - P6	518	643	M1 - P2	230	127
P4 - ELBA2	259	161	M1 - P4	196	92
P5 - P6	210	106	M1 - P5	204	100
P5 - Elba2	591	837	M1 - P6	394	372
P6 - Elba2	709	1205			
P21 - Elba2	160	61*			
P21 - P2	495	588			
P21 - P23	381	348			
P21 - P4	229	126			
P21 - P5	587	826			
P21 - P6	747	1338			
P21 - P22	213	109			
P21 - M1	400	384			

* values are low and indicate that these couplets should be able to operate in the same mode (*i.e.*, both should pump or inject) at rates exceeding the calculated values.

According to the calculations, most well couplets can be pumped/reinjected at fairly high rates. Those identified with an asterisk, namely P1/M1, P1/P23, P2/P5,

P21/Elba2, P22/P4 and P23/M1, are of primary concern and are given special consideration in the following sub-section.

8.3.2 Interpretation

Based on the results of the analytical model, a number of restrictions were placed on the possible configurations for pumping and reinjecting. The separation distance between P4 and P22, for example, allowed for a maximum pumping/reinjection rate for the couplet of 11 L/s. Likewise, the maximum pumping/reinjection rate for P23 with either M1 or P1 was calculated as 14 L/s. To prevent thermal breakthrough, both wells should be either pumped or reinjected together, if the rates exceed the calculated flow rates. Similarly, the maximum pumping/reinjection rate between P23 and P4 is 69 L/s. This should not present a problem since P4 is incapable of pumping or reinjecting at rates higher than about 30 L/s.

Maximum allowable rates were permitted, in some cases, to be higher than those calculated, due to fault block interconnection, or lower, due to fault connectivity. According to Table 8.4, wells P22 and P23 are restricted to 86 L/s, and wells P21 and Elba2 are restricted to 60 L/s. However, given that both of these well couplets are located in separate fault blocks and, thus have poor hydraulic connection, the heat transport will likely be minimized.

In contrast, one well couplet that may not behave according to predictions in the simplified model is P23-P21. Both of these wells are located in the Central Block, which has traditionally been the lower yielding area of the campus. Aquifer test data also show that these two wells are extremely well connected by the Central Fault. To achieve

hydraulic stability, these two wells should be used in opposition. It was anticipated that thermal breakthrough may be a problem between these two wells (CESI, 1994).

To satisfy the thermal constraints imposed by the Phase 1 well separations, as well as the requirements for groundwater flow, the following configuration of wells was recommended by CESI (1994). One group of wells includes two Central Block wells, P1 and P23, plus P5 and Elba 2 from the Eastern Block. The second group of wells consists of P4 and P21 from the Central Block, and P22, P2 and P6 from the Eastern Block.

This configuration of wells satisfies the following constraints:

- 1) to allow P23 and P1 to work together to satisfy their separation constraint,
- 2) to allow P4 and P22 to work together to satisfy their separation constraint,
- 3) to keep a hydrologically balanced Central Block by pumping or injecting P21 and P23 in opposition, as well as pumping or injecting P1 and P4 in opposition,
- 4) to keep a hydrologically balanced Eastern Block by staggering the injection and pumping wells, thus distributing the water evenly throughout (*i.e.*, Elba 2 is in opposition to P22),
- 5) to maintain the restrictions imposed by pump and pipe size in existing Phase 1 wells P2, P5 and P6. Since these wells cannot operate at higher than 63 L/s, 95 L/s and 69 L/s, respectively, it would be impossible to change the configuration for these wells (*i.e.*, P2 and P6 against P5) and still meet the water requirements for Phase 2 (265 L/s).

The configuration of wells, which was recommended to the University for the Phase 2 expansion of the system, is provided in Table 8.5 (also provided in Table 5.2). This configuration was designed to optimize well production, while maintaining hydraulic stability (minimizing drawdown and positive hydraulic head) and minimizing thermal breakthrough between wells.

Table 8.5: Optimum Configuration of Wells and their Respective Production Rates in L/s for Phase 2 (Data From CESI, 1994).

Well:	P1	P4	P21	P23	P2	P5	P6	P22	E2
Rate (L/sec)	+10.0	-10.0	-65.0	+65.0	-60.0	+95.0	-65.0	-65.0	+95.0
Calculated Drawdown/ Positive Head (m)	+19.9	-5.4	-6.7	+4.4	-1.2	+19.1	+2.0	-13.1	+15.1

8.3.3 Discussion

The analytical thermal model is considered to produce only approximate estimates of thermal breakthrough between wells for three reasons. First, the model calculates thermal breakthrough for well couplets and does not allow for interspersed wells. Second, the model assumes radial flow between wells and calculates thermal breakthrough by assuming maximum interflow. Third, the analytical thermal model cannot incorporate faults. Nevertheless, this model was considered by CESI to be an appropriate method for designing a well configuration, but only if consideration was given to well interconnection. For example, CESI (1994) recognized the possibility of early thermal breakthrough between certain wells, particularly P21 and P23, and recommended that these wells not operate at rates higher than 65 L/s.

To explore the possibility that early breakthrough might occur in the Carleton system, as a result of faulting, it was necessary to develop a model that could simulate heat transport. Using this model, it would be possible to assess the thermal performance of the ATES system, and to compare simulated thermal breakthrough times to estimates of thermal breakthrough time calculated using the analytical model.

8.4 Numerical Heat Transport Model

8.4.1 General

Several heat transport simulations were carried out using the thesis heat transport software, GW3. Contour maps for each simulation were produced using *SURFER* (Golden Software, Inc.) with no interpolation between nodes. Two well couplet reinjection simulations were performed (Section 8.4.2). These included pumping at P1 and reinjection at P4, and pumping at P2 and reinjection at P5. Both of these configurations were used for ATES at Carleton, for brief periods of time, although neither was used consistently for any length of time. Actual flow and temperature measurements at the pumping and reinjection wells are available. These monitoring data are provided in Section 8.4.3.

The configuration for the ATES system recommended to the University by CESI (1994) was also simulated (Section 8.4.4). This system has not yet been implemented, therefore, there are no data to calibrate the model.

Aquifer thickness is the most difficult parameter to estimate for Carleton, and there has been considerable debate concerning this issue. Consequently, each of the heat transport simulations was done using aquifer thicknesses of 6 and 60 metres, in order to

determine the effect of varying aquifer thickness on thermal breakthrough time. Because aquifer thickness is a major factor in determining both the average linear velocity and the aquifer heat capacity (Chapter 7), the order of magnitude variation in this parameter can be used to represent extreme conditions for the model. Varying aquifer thickness constitutes a sensitivity analysis, because the model is most sensitive to changes in this parameter. Furthermore, this variation effectively simulates changes in the amount of dispersion, because thermal smearing can be expected to be less in thinner aquifers.

8.4.2 Simulation Results - Selected Well Couplets

P1-P4 Well Couplet

The first heat transport simulation was for the well couplet P1-P4, where P1 is pumping at a rate of 26 L/s and reinjecting into well P4 at the same rate. This production rate was maintained for several years, therefore, monitoring data may be used to partially calibrate the results (Section 8.4.3). Temperatures and hydraulic heads were simulated over a six month period. The initial aquifer temperature was 10°C and the reinjection temperature at well P4 was 14°C.

In the simulation for the 6 metre thick aquifer (Figure 8.9a), the thermal front has advanced radially between 50 and 80 metres from the reinjection well. Temperatures at the pumping well, P1, remain at 10°C, indicating that thermal breakthrough has not occurred. In fact, the thermal front is approximately 40 metres away from the pumping well, P1, at the end of the simulation time.

In the simulation for the 60 metre thick aquifer (Figure 8.9b), the thermal front has not advanced as far from the reinjection well (25 to 50 metres). Temperatures at the

P1-P4 WELL COUPLET TEMPERATURE CONTOURS AFTER 6 MONTHS

Figure 8.9a: Contour Map of Temperatures at the End of 6 Months for the P1-P4 Well Couplet.
Production Rate is 26 L/s. Aquifer Thickness 6 Metres.

P1-P4 WELL COUPLET TEMPERATURE CONTOURS AFTER 6 MONTHS

Figure 8.9b: Contour Map of Temperatures at the End of 6 Months for the P1-P4 Well Couplet.
Production Rate is 26 L/s. Aquifer Thickness 60 Metres.

pumping well remain at 10°C, indicating that thermal breakthrough has not occurred. In fact, the thermal front is approximately 80 metres away from the pumping well, P1, at the end of the simulation time.

Both of these simulation results demonstrate effectively the approximate radial distribution of heat in the system, which is consistent with the hydrologic interpretation of the aquifers (*i.e.*, away from faults, radial flow takes place). The contour maps also show evidence of heat transport along the Central Fault, which separates wells P1 and P4. This fault channels heat away from the pumping well, thus, restricting thermal breakthrough. The fault not only acts as a partial hydraulic barrier but as a thermal barrier as well.

The effect of increasing the aquifer thickness is to reduce the rate of thermal front migration and, hence, to decrease the possibility that thermal breakthrough will occur at the pumping well. For both of these simulations, thermal breakthrough does not occur within the six month simulation time.

P2-P5 Well Couplet

The second simulation was for well couplet P2-P5. In this simulation, P2 is pumping at a rate of 60 L/s and reinjecting into well P5 at the same rate. Temperatures and hydraulic heads were simulated over a six month period. The initial aquifer temperature was 10°C and the reinjection temperature at well P5 was 14°C.

In the simulation for the 6 metre thick aquifer (Figure 8.10a), the thermal front has advanced radially between 80 and 110 metres from the reinjection well, and is approximately 85 metres away from the pumping well, P2, at the end of the simulation.

P2-P5 WELL COUPLET TEMPERATURE CONTOURS AFTER 6 MONTHS

Figure 8.10a: Contour Map of Temperatures at the End of 6 Months for the P2-P5 Well Couplet.
Production Rate is 60 L/s. Aquifer Thickness 6 Metres.

P2-P5 WELL COUPLET TEMPERATURE CONTOURS AFTER 6 MONTHS

Figure 8.10b: Contour Map of Temperatures at the End of 6 Months for the P2-P5 Well Couplet.
Production Rate is 60 L/s. Aquifer thickness 60 metres.

Temperatures at the pumping well remain at 10°C, indicating that thermal breakthrough has not occurred.

In the simulation for the 60 metre thick aquifer (Figure 8.10b), the thermal front has advanced approximately 60 metres in all directions, and is located approximately 110 metres away from the pumping well at the end of the simulation. Temperatures at the pumping well remain at 10°C, indicating that thermal breakthrough has not occurred.

There is no fault separating wells P2 and P5, therefore, the distribution of heat is not influenced by linear flow; it is essentially radial. In addition, because the hydraulic head differential between these two wells remains at a low level, there is a fairly low hydraulic gradient. This reduces significantly the rate of advancement of the thermal front in the direction of the pumping well.

8.4.3 Thermal Model Calibration

No rigorous program was undertaken by the University to monitor the ATES system, and consequently, there are no monitoring data for the early days of system operation. In addition, temperature sensors and flow meters failed to operate at the well heads. In 1992/1993, as part of the Commons Building retrofit work and proposed Phase 2 expansion of the system, a more efficient data acquisition system was implemented. This system provides computer-updated measurements of flow, pressure and temperature at wells P2 and P5. The Commons Building is now directly cooled in the summer and is pre-heated in the winter using groundwater, and groundwater for this portion of the project is derived from the operation of wells P2 and P5.

Despite efforts to computerize the system, sensors are not regularly calibrated and measurements are often suspect. For example, well P4 always indicated a reinjection temperature of 18.04°C, despite changing temperatures at well P1. It is unlikely that the temperature differential changed in such a manner to produce a constant reinjection temperature, therefore, the sensor must be malfunctioning. In addition, after months of collecting data at well P5, it became apparent that the temperature sensor was incorrectly calibrated. This was brought to the attention of the maintenance staff; however, a recalibration did not take place until several weeks later. In addition to poor temperature data, flow measurements are rarely the same for a pumping well and its companion reinjection well. Again, this is the result of poorly calibrated sensors.

Another problem is the inconsistent use of the system. On warm days in the winter, the system automatically shuts off, or temperature differentials are reduced. System shut-off also happens on cooler days in the summer. Since the thermal model assumes constant production rates and constant temperature differentials at peak load conditions, the actual rate of advancement of the thermal front may be significantly lower than that predicted by the model.

The most important problem is that there is no base level for the calibrations because wells have not been operated consistently. Shutdowns for maintenance purposes, arbitrary changes in both the configuration and production rates from those proposed for Phase 1, and continued operation in one mode (for example, several years of pumping at P1 and reinjection at P4), have all resulted in modifications to the thermal regime which cannot be modelled. Therefore, the initial conditions for the model cannot be reproduced.

With all of these short-comings, it has been extremely difficult to identify trends in the data as well as to calibrate the thermal model. Table 8.6 shows the monitoring results over a period of approximately two years. The author places little faith in the accuracy of these data for the reasons stated at the beginning of this section.

Table 8.6: Flow Rates and Temperatures for Production Wells from Summer 1994 to Spring 1996. Blank Spaces Indicate Wells are not in Operation, * are not avail.

Date	P1		P2		P4		P5		P6	
	flow (L/s)	temp (°C)	flow (L/s)	temp (°C)	flow (L/s)	temp (°C)	flow (L/s)	temp (°C)	flow (L/s)	temp (°C)
07/94	pump	*	pump	*	inj	*	inj	*		
09/94	pump	*	pump	*	inj	*			inj	*
24/10/94	-26.7	13.85	-53.5	12.80	26.5	18.04	48.8	15.38		
4/11/94	-26.3	13.88			26.32	18.04				
1/12/94	-24.7	13.92			*	*				
14/12/94	-24.3	13.80	-74.4	11.99	24.2	18.04			25.5	9.43
10/01/95	pump	*	-66.5	11.60	inj	*			*	8.56
18/01/95	pump	*	-47.0	11.54	inj	*			*	9.47
23/01/95	pump	*	-41.2	11.50	inj	*			*	9.48
09/05/95	-22.1	14.06	-70.9	*	22.2	18.04	65.8			
15/05/95	-22.1	14.07	-55.7	14.45	22.1	18.04	50.4	17.32		
17/05/95	-22.1	14.08	-59.2	14.30	22.1	18.04	52.9	17.41		
21/05/95	-22.1	14.13	-58.5	14.02	22.0	18.04	52.3	17.43		
2/06/95	-22.1	14.17	-58.6	13.77	22.1	18.04	51.4	18.37		
12/06/95	-22.1	14.17	-56.7	13.60	22.1	18.04	50.6	18.25		
20/06/95	-22.4	14.16	-58.1	13.45	22.2	18.04	51.3	17.37		
27/06/95	-22.1	14.25	-57.7	13.38	22.2	18.04	50.8	18.74		
4/07/95	*	*	-57.1	13.34	*	*	51.2	17.92		
13/07/95	*	*	-57.4	13.27	*	*	51.1	17.77		
20/07/95	-22.1	14.46	-57.0	13.23	22.2	18.04	47.3	18.27		
27/07/95	-22.0	14.55	-56.6	13.17	22.0	18.04	50.2	18.53		
17/08/95	-22.0	14.81	-56.3	13.08	22.0	18.04	49.5	18.67		
24/08/95			-56.2	13.06			50.2	16.55		
15/09/95			-55.6	12.95			50.3	15.42		
21/09/95			-55.5	12.93			50.1	15.40		
28/09/95			-61.1	12.85			54.9	14.90		
3/10/95			-60.9	12.82			54.6	14.86		

Table 8.6: Cont'd

Date	P1		P2		P4		P5		P6	
	flow (L/s)	temp (°C)	flow (L/s)	temp (°C)	flow (L/s)	temp (°C)	flow (L/s)	temp (°C)	flow (L/s)	temp (°C)
12/10/95			-73.5	12.70			65.2	14.69		
19/10/95			-73.1	12.65			63.3	14.65		
26/10/95			-73.0	12.61			65.2	14.62		
3/11/95			-72.5	12.55			64.6	14.56		
9/11/95			-72.4	12.52			64.9	14.45		
16/11/95			-72.1	12.48			65.0	14.48		
23/11/95			-71.8	12.42			64.5	14.47		
6/12/95			-71.6	12.33			64.6	13.88		
14/12/95			-72.3	12.29			65.2	10.62		
22/12/95			-71.9	12.25			64.4	13.58		
4/01/96			-71.6	12.18			64.2	12.18		
11/01/96			-71.4	12.14			63.8	9.76		
19/01/95			-71.2	12.08			63.0	9.50		
25/01/96			-71.8	12.12			63.5	9.91	sensors recalibrated	
1/02/96			-71.4	12.20			63.6	9.27		
8/02/96			-66.6	12.14			59.9	10.98		
15/02/96			-66.7	12.10			60.1	9.53		
22/02/96										
29/02/96			-62.3	12.08			60.2	11.35		
6/03/96										
14/03/96			-61.7	11.33			59.9	10.98		
22/03/96			-61.0	11.30			59.7	10.15		
4/04/96			-61.0	11.28			59.3	10.13		

These data could not be used for a proper calibration. However, two important observations were made.

- During the period from October 1994 to August 1995, well P1 continued to pump at a rate of 22 L/s and reinject into well P4 at the same rate. These two wells had been in operation at approximately the same rate for several years prior to this. Although no monitoring data are available, the temperature differential was likely 3 to 4°C. These

two wells were providing cooling only to the Mackenzie and Minto Engineering buildings for this entire period, and measured temperatures increased gradually and consistently at the pumping well from 13.85°C to 14.81°C (0.96°C) over the 10 month period.

Thermal breakthrough has already occurred between these two wells because of their continued operation for several years. However, the rate of increase in temperature at the pumping well is approximately 0.1°C per month. This amount of breakthrough will not reduce significantly the efficiency of the system (N. Haitas, Personal Communication). If the P1-P4 couplet had been used for only one heating season or cooling season, instead of for several years, it is likely that no thermal breakthrough would have occurred.

- During the period from May 1995 to the present, well P2 has been pumping at an average rate of approximately 62 L/s and reinjecting into well P5. In contrast to the P1-P4 couplet, which was only used for cooling, this well couplet has been operating in the same direction for summer cooling and winter heating over the 10.5 month period. Despite being used for cooling, temperatures have continued to decrease at a rate of approximately 0.05°C per week. Over a six month period, the total decrease is 1.28°C. The speculated reason for this decrease is that the aquifer was initially quite warm from previous reinjections of warm water during cooling in this part of the campus (several years of cooling only). Cooler fault water is being drawn in along the Eastern Fault toward the pumping well P2. There is no evidence of breakthrough between these

wells, although if thermal breakthrough did occur in the summer cooling mode, the effect has been masked by the steadily-decreasing temperatures at well P2.

In each of the two well couplet simulations, namely well P1 to well P4 (26 L/s), and well P2 to well P5 (60 L/s), no thermal breakthrough occurred during the numerical simulations, and none was predicted from WESA's (1988) analytical calculations. On the basis of the limited monitoring data for these well couplets, it is suggested that both the analytical and numerical models have provided conservative estimates of breakthrough times for these well couplets because actual operating conditions are not as severe as simulated in the models.

However, neither of these couplets is located along a fault, such as P21 and P23. In order to estimate breakthrough in this case, the model must be applied to the multi-well configuration.

8.4.4 Multi-well Configuration (CESI, 1994) - Simulation Results

The final set of simulations were for the CESI (1994) configuration. This configuration includes nine production wells; five of which pump during the cooling season, and four of which pump during the heating season. The configuration was designed to be completely interchangeable, that is, it does not matter which groups of wells are used for which season. The cooling season has been simulated for both an aquifer thickness of 6 metres and 60 metres. The total rate of production is 265 L/s. Wells P2, P6, P21, P22 and P4 are pumping and wells P1, P5, P23 and Elba2 are reinjecting (refer to Table 8.5 for individual well rates). This configuration has not yet been implemented, therefore, no monitoring data are available.

For these simulations, a data file, consisting of hydraulic heads and temperatures at each node in the mesh, was produced for simulation times of 1 month, 3 months, and 6 months. *SURFER* (Golden Software, Inc.) was again used, without interpolation, to contour the results. Figure 8.11a shows the temperature distribution at the end of the first month for the 6-metre thick aquifer.

The rate of advancement of the thermal front from well P5 has increased in comparison to that of the P2-P5 couplet simulation, because the rate of injection has increased from 60 L/s to 95 L/s. The thermal front advancing away from P1 is small, as a result of the low injection rate (10 L/s). The thermal front around Elba2 is distorted because of the Elba Fault, which lies just to the north of this well.

The most interesting aspect of this map is the linear advancement of the thermal front away from well P23 and toward well P21. These two wells are located along the Central Fault, and exhibited excellent hydraulic connection during testing. Figure 8.11b shows the advancement of this front after 3 months. Thermal breakthrough has not yet occurred at well P21. In addition, the thermal front from well P1 is beginning to merge with the one from well P23. Figure 8.11c shows the final distribution of temperature at the end of the six month period. Thermal breakthrough has still not occurred at well P21; the front is located approximately 30 metres to the west and is confined to the fault zone.

Several observations can be made concerning the shape of the thermal fronts. In Figure 8.11b, the thermal front projects outward from the Central Fault toward P4. In the P4-P1 couplet simulations, the heat was trapped at the fault due to the barrier, and thermal breakthrough did not occur even though the wells were operating at 26 L/s. In the CESI

MULTI-WELL CONFIGURATION TEMPERATURE CONTOURS AFTER 1 MONTH

Figure 8.1.1a: Contour Map of Temperatures at the End of 1 Month for the Multi-well Configuration (CESI, 1994).
Total Production Rate is 265 L/s. Aquifer Thickness 6 Metres.

MULTI-WELL CONFIGURATION TEMPERATURE CONTOURS AFTER 3 MONTHS

Figure 8.11b: Contour Map of Temperatures at the End of 3 Months for the Multi-well Configuration (CESI, 1994)
Total Production Rate is 265 L/s. Aquifer Thickness 6 Metres.

MULTI-WELL CONFIGURATION TEMPERATURE CONTOURS AFTER 6 MONTHS

Figure 8.11c: Contour Map of Temperatures at the End of 6 Months for the Multi-well Configuration (CESI, 1994).
Total Production Rate is 265 L/s. Aquifer Thickness 6 Metres.

simulation, the rate is significantly lower, however, the thermal front is observed to break through the fault and extend toward P4. This heat is probably coming from well P23.

Figures 8.12a, b and c show the temperature contours at 1 month, 3 months and 6 months for the 60 metre thick aquifer. As expected, the thermal fronts remain much closer to the reinjection wells. Thermal breakthrough has not occurred within the 6 month simulation time. From the simulations, some general statements can be made concerning the relative importance of faults to the overall thermal front geometry and the rate of thermal front migration in relation to the faults and the aquifer thickness.

- The faults act primarily as hydraulic conduits. When a well couplet is situated directly along a fault, thermal breakthrough occurs more rapidly than if the couplet is situated away from a fault. Heat does move into the surrounding aquifers along both sides of the fault, but at a slower rate.
- For wells situated slightly off a major fault, the general shape of the thermal front is radial. The front is flattened slightly at the fault face and does not extend readily across the fault (e.g., Elba2 at six months).
- Heat does not remain trapped within a fault, but can be transported away from a fault toward a pumping well (e.g., well P4 at six months).
- The effect of increasing the aquifer thickness from 6 to 60 metres is to reduce significantly the rate of migration of the thermal front. The lack of monitoring data makes it difficult to determine the exact thickness of the aquifer by calibration. However, even when a thin aquifer was used in the simulations, there was no thermal breakthrough within a six month period.

MULTI-WELL CONFIGURATION TEMPERATURE CONTOURS AFTER 1 MONTH

Figure 8.12a: Contour Map of Temperatures at the End of 1 Month for the Multi-well Configuration (CESI, 1994).
Total Production Rate is 265 L/s. Aquifer Thickness is 60 Metres.

MULTI-WELL CONFIGURATION TEMPERATURE CONTOURS AFTER 3 MONTHS

Figure 8.12b: Contour Map of Temperatures at the End of 3 Months for the Multi-well Configuration (CESI, 1994).
Total Production Rate is 26.5 L/s. Aquifer Thickness is 60 Metres.

MULTI-WELL CONFIGURATION TEMPERATURE CONTOURS AFTER 6 MONTHS

Figure 8.12b: Contour Map of Temperatures at the End of 6 Months for the Multi-well Configuration (CESI, 1994).
Total Production Rate is 265 L/s. Aquifer Thickness is 60 Metres.

8.5 Comparison of Analytical and Numerical Heat Transport Models

The main advantage of the numerical model is that the advancing thermal front can be simulated within a non-homogeneous aquifer. This is particularly important in a complex aquifer regime, such as a faulted carbonate aquifer. The numerical model demonstrates that although the faults act as hydrologic and thermal conduits, the rate of heat transport is not sufficient to cause the thermal front to advance significantly beyond the fault zone at the specified production rates. Therefore, wells located slightly away from the major faults will not be expected to exhibit significant thermal breakthrough. In addition, even with an order of magnitude variation in aquifer thickness, there is no thermal breakthrough at the pumping wells when simulated using peak operating conditions for a six month season.

Nevertheless, analytical models can be used effectively in determining well spacing, or alternatively, maximum pumping rates for wells used in ATEs configurations. However, it is critical that a thorough understanding of both the geology and the hydraulics of the system be attained prior to designing a configuration of wells. Careful consideration must be given to well connection. Wells connected by high permeability fractures or faults should be operated at rates below that predicted by the analytical model, unless the wells are situated slightly away from the main conductive conduits. In the latter situation, additional time is required for heat to move away from the fault or fracture to the pumping well. Alternatively, wells located across hydraulic boundaries can be operated at rates exceeding those predicted by the analytical models, particularly if a hydraulic barrier is present.

It should be emphasized that both the analytical and numerical thermal models assume constant production for the entire six month period. In real terms, the system will likely not operate at maximum capacity for this length of time. The position of the thermal front after four months is probably more representative of actual operating conditions for one heating or cooling season.

The practical implication of these results is that the groundwater system designed for Carleton University should operate with hydrologic and thermal efficiency. There are no indications of serious thermal breakthrough problems with the Phase 2 configuration, and even if breakthrough does occur, the monitoring data show the rates of increase in temperature at the pumping wells will be minimal.

9.0 SUMMARY AND CONCLUSIONS

To model the faulted carbonate aquifer underlying the Carleton campus, it was necessary to integrate geological information with an extensive hydrogeological database. Borehole lithology logs were used in combination with detailed subsurface investigation techniques, including bedrock topographic mapping, downhole geophysical logging and surface geophysical exploratory methods, to identify stratigraphic offsets related to faulting, and to delineate the faults. In particular, microgravity profiling was shown to be an effective technique for imaging the bedrock surface when local exposures are lacking.

Static hydraulic head and transient aquifer test data from several wells on campus were critical for characterizing the static and dynamic flow regimes, and for identifying zones within the aquifer where higher well yields and preferential connection between wells could be anticipated. Analysis of the aquifer test data for Carleton has shown that hydraulic testing should be undertaken at wells located both along faults and on opposite sides of faults. This type of spatial resolution is necessary to successfully identify the inter-relationship between faults and the surrounding aquifers, and to aid in the design of a conceptual model.

Chemical data, acquired prior to and during operation of the ATES system, were used to model mixing that takes place between the aquifers, both in the natural environment and during system operation. These data were carefully scrutinized because mixing, associated with water movement in the open boreholes, masks the true chemical composition of the aquifer. Thus, chemical variations, observed with depth, are not

representative of the chemistry of the aquifer in the immediate vicinity of the well. Downhole temperature logs were used to identify water entry and exit points in the open borehole, and thus, to define the zone within the borehole where mixing takes place.

A thorough analysis of aquifer test data from two extensive aquifer testing programs was necessary to provide the estimates of the hydraulic properties of the aquifer, to characterize the flow regime during testing, and to identify leakage induced by pumping at late time. The faulted aquifers at Carleton often exhibit non-radial flow conditions at early time during constant discharge testing. When wells are situated close to highly transmissive faults, the flow regime is more appropriately described as linear. The linear flow regime results in a trough of depression that decreases in depth away from, and perpendicular to the faults.

One conceptual model for the faulted carbonate aquifer at Carleton consists of a series of highly transmissive, homogeneous and isotropic fault zones, which transect two homogeneous and isotropic aquifers. Both the faults and the aquifers can be represented as equivalent porous media. Boundary conditions for the conceptual model were difficult to define because there are insufficient geologic and hydrogeologic data for areas surrounding the campus. Therefore, a set of boundary conditions were selected that could generate the observed static hydraulic head distribution, and reproduce the late time leakage observed in transient hydraulic test curves. These boundary conditions are indicative of leakage through faults that are present in the distant surrounding area, but additional geological information is required to confirm both their location and their hydraulic significance.

To reproduce the transient response times and drawdown in all pump test and reinjection test simulations, a low-permeability barrier, located on the edge of the fault zone, was incorporated into the conceptual model. The presence of this hydrogeological barrier at Carleton suggests that similar features may be present in other faulted carbonate aquifers, and that their existence should be considered when response times, for observation wells situated across faults, are delayed.

Although a steady-state model for the Carleton aquifer effectively reproduced the static hydraulic head distribution without pumping, it was unable to simulate the hydraulic head distribution under pumping conditions. This result suggests that steady-state models should be used with caution when modelling complex aquifers. Transient calibrations, using aquifer test data from more than one well, should be implemented in these circumstances.

Transient two-dimensional heat transport models show that the thermal fronts migrate radially away from reinjection wells that are slightly offset from the faults, but linearly away from reinjection wells located along the faults. The thermal models for the multi-well configuration, designed for the Phase 2 expansion of the Carleton ATEs system, show that there is no thermal breakthrough between wells operating at maximum conditions in a 6 metre thick aquifer. In reality, the aquifer is probably 60 metres thick, and the system does not operate continuously at maximum conditions. Therefore, thermal breakthrough should not occur at the pumping wells within a cooling or heating season.

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Appendix 1

THE CARLETON GROUNDWATER ENERGY SYSTEM

A1.1 Project Background

The Carleton University campus is located in Ottawa, Ontario, Canada (Figure A1.1). The campus is bordered by Bronson Avenue, the Rideau River, the Rideau Canal, and the south end of Dow's Lake.

In 1985, Carleton University initiated a project to develop a groundwater energy system for the heating and cooling of campus facilities. The initial intention was to obtain a groundwater supply for the cooling of the computing facilities located in the Administration Building (Robertson Hall) (Figure A1.2). Temperature logging of three existing shallow wells was carried out in order to evaluate the shallow thermal regime. An optimum temperature of 9.5°C was sought for the cooling application. The temperature-depth plots for the Elba1, Tory and Residence Commons (RC) wells on campus, as well as for a deep well located at the Dominion Observatory on Observatory Crescent, Ottawa, are shown in Figure A1.3. Differences in the shallow near-surface temperatures of the Carleton wells were attributed to the proximity of buildings on campus (Michel, 1985). However, at depth the profiles converge and reflect the local geothermal gradient. The ground temperature within the depth range of 42 to 122 metres (defined as the thermal envelope) was considered to be favourable for use in a cooling application.

A1.2 Feasibility Studies (Pre-Phase 1)

In 1985, Water and Earth Science Associates Ltd. (WESA) was contracted by Carleton University to conduct aquifer tests on two of the existing shallow wells, and thus determine the potential yield of these wells. Both the Elba1 and the Residence Commons

Figure A1.1 : Map Showing Location of Carleton University in the Ottawa Valley

Figure A1.2: Map Showing the Location of Test and Production Wells on Campus

Figure A1.3 : Temperature-Depth Profiles for Water Wells at Carleton University.
Observatory Crescent Borehole BH-0-8 Profile Shown for Comparison.

(RC) wells were tested (Figure A1.2). A preliminary step test at Elba1, consisting of three complete steps and one incomplete step test, was abandoned due to sand production and turbidity. The maximum pumping rate attained was 6.42 l/sec (84.7 IGPM) with a drawdown of 24.08 metres at 150 minutes. Further aquifer testing at this well was not undertaken.

At the RC well, a step discharge test, consisting of five complete steps and one incomplete step, indicated that the well could be tested at a constant discharge rate of 9.5 l/sec (125 IGPM). The maximum drawdown at the RC well during the constant discharge test measured 10.65 metres after 24 hours of pumping (WESA, 1985). Drawdown in the Tory and Elba1 wells was recorded at 0.45 and 4.55 metres, respectively. Once the pump was shut off, drawdown continued at the Tory well to a maximum of 0.49 metres. A considerable time lag of over 5 hours was observed in the response time of the Tory well. The Tory well did not begin to recover immediately when the pump was shut off. In contrast, the response in the Elba1 well was immediate. Both wells are located at roughly the same distance from the pumping well (500 metres).

Aquifer parameters were calculated for both the pumping well and the two observation wells using the Jacob method of analysis. The transmissivity was calculated at 103.3 m²/day for the RC well, 51.6 m²/day for the Elba1 well, and 107.9 m²/day for the Tory well (WESA, 1985). Storativities were calculated for the observation wells only, since the Jacob method of analysis does not lend itself to calculating the storage coefficient for the pumping well. Storativities for the Elba1 and Tory wells, were 1.134E-5 and 4.08E-4, respectively. Recovery testing was also undertaken immediately following

pump shut-off. The transmissivity for the RC well was calculated at 48.29 m²/day from the recovery data.

Michel (1985) concluded from these initial pump test results that some of the faults located beneath the campus do not form impermeable barriers to groundwater flow, while others form partial barriers. In particular, the rapid response of the Elba1 well (<10 minutes) relative to the much later response at the Tory well (300 minutes) supported this statement. However, these test results were insufficient to prove whether the variation in response time could be attributed to other factors, such as lithological differences, rather than, strictly speaking, the faults. WESA (1985) stated that the poor hydraulic connection with the Tory well, observed during this test, is consistent with a sealed or partially sealed fault and that this type of behaviour is characteristic of others measured in the area. WESA (1985) suggested that the excellent connection between the RC and Elba1 wells could be explained by the presence of an open and conducting section of the Gloucester Fault (*i.e.*, the Eastern Fault). It was concluded that increases in aquifer transmissivity on the order of three to four times would be expected if the shallow wells were drilled a further 30 to 45 metres into the bedrock, as a result of increased aquifer intersection, and that these new wells would be sufficient to meet the demands of a groundwater energy system.

In the summer of 1986, WESA was contracted by Carleton University to supervise the drilling of a 25.4 cm (10 inch) diameter test well. The Elba2 well, located slightly east of the existing shallow Elba1 well, was drilled to a final depth of 121.9 metres. Yields were extremely high during drilling; approximately 700 IGPM.

At a depth of 32.6 metres, the first indication of an artesian aquifer was noted. Water-bearing seams are noted throughout the drill log, however, these are more frequent in the lower units, namely the Sherman Fall and Hull Formation (depth interval 82 to 122 metres). Flowing conditions persisted throughout the drilling, except at two intervals, 100.6 metres and 106.7 metres, where lower hydrostatic heads were measured and the well temporarily ceased to flow. At a depth of 111.25 metres, the drilling technique was changed to incorporate a tricone bit, because the percussion bit became ineffective with the high yields. After drilling was completed, flowing conditions persisted at a rate of approximately 200 to 300 IGPM.

The Elba2 well was step tested at rates ranging from 72 to 500 IGPM. During this test, water discharge temperatures decreased from 11.2°C to 10.5°C and these decreases appeared to coincide with the increases in pumping rate. Following a period of recovery, the well was tested at a rate of 450 IGPM for a period of 72 hours. This is the longest aquifer test of the entire suite of tests conducted at Carleton. Drawdown and recovery data were collected from the pumping well (Elba2) and the three available observation wells (RC, Elba1, and Tory). Total drawdown in the pumping well was 11.27 metres at the end of the test. The drawdown was widespread and the response times were again consistent with those observed during testing of the RC well. Total drawdown measured 8.34 metres at RC, 4.06 metres at Tory and 10.05 metres at Elba1. The delay at Tory was only 1 hour as opposed to the 5 hour time delay observed during the RC constant discharge test. The higher pumping rate may explain this discrepancy.

Aquifer test data were analyzed using the Jacob and Theis methods of analysis (WESA, 1986). A classical response was observed at Tory, Elba1 and Elba2, however, it was noted that leakage occurred at RC. Irregularities in the storativity values were not explained. Electrical conductivity and temperature were measured throughout this test. These data are discussed in Chapter 3.

A short term yield of approximately 700 IGPM was estimated from the results of the step test. This figure may over- or under-estimate the actual yield due to the fractured nature of the aquifers (WESA, 1986). Long term drawdown (*i.e.*, at the end of a 20 year period), without reinjection, was calculated at 41 metres for a pumping rate of 450 IGPM. This value assumes that no boundaries are present and that recharge to the system is negligible. If recharge is taken into consideration, a much lower drawdown could be expected (WESA, 1986).

The Elba2 well was later tested at a rate of 800 IGPM for a period of 48 hours. During this test, several deeper wells (*i.e.*, those drilled during Phase 1 and not yet converted to operating production wells) were monitored as observation wells. The final drawdown in the pumping well measured 16.93 metres. Transmissivity and storativity were calculated at 188.4 m²/day and 2.59E-6, respectively. Results of this test are discussed in more detail in Chapter 4 on Aquifer Test Results.

The higher than expected water yields for Elba2 prompted the University to review the project and consider the possibility of providing heating and cooling for several buildings on campus, with the additional possibility that eventually all buildings could be connected to the system (Figure A1.2). Some of the campus mechanical systems were due

for renewal within a short time frame and, therefore, groundwater heat exchange technology was considered a viable option. The revised project was then designed for implementation in phases.

A1.3 The 1987/1988 Phase 1 Development Program

In 1987, Water and Earth Science Associates (WESA) was contracted by the University to oversee the drilling of production wells, which would provide a groundwater supply at a rate of 121.2 l/sec (1600 IGPM). The system would be designed to operate in a closed loop (*i.e.*, no exposure to the atmosphere) with some of the wells pumping water through heat exchangers and heat pumps (winter only) and returning the water to separate wells for reinjection. The primary objective was to minimize the amount of thermal breakthrough between the reinjection and pumping wells while minimizing drawdown levels and positive reinjection pressures. The drilling results described in this section are summarized from WESA (1988).

The first official test well for Phase 1 was drilled in the Fall of 1987, although it was drilled prior to the call for proposals for Phase 1 drilling and testing. This well, located north of the Mackenzie Building (Figure A1.2) and designated as M1, was drilled to assess the hydrogeological potential of the central portion of the campus after concerns were raised by Dr. Michel in the Earth Sciences Department of Carleton University that the supply of water in the Central Block may not be as great as that found in the Eastern Block.

The diameter of M1 is 15.2 cm for its entire 160 metre depth. Discrete intervals were identified as having increased flow during drilling. These were at 106.7 and 112.8

metres, respectively. Well yield, at the end of drilling, exceeded 280 IGPM. This well was not pump tested, but was converted to a permanent multilevel piezometer installation later in 1988. The instrumentation included eight separate piezometers (Figure A1.4). The bottom most (M1-PVC) was a PVC (polyvinyl chloride) tube. All other piezometers were constructed of polyethylene tubing. Seals between the piezometers were a neat cement grout, except the bottom-most seal, which was composed of peltonite tablets (WESA, 1988). Backfill material between the seals consisted of washed pea-stone gravel. Wilson (1990) noted that the two bottom piezometers (M1-1 and M1-B) may have been improperly sealed and that these two may be hydraulically connected. Data obtained from this installation, both prior to implementation of the system and afterwards, is discussed in Chapter 3.

On the basis of the water found at M1, the University decided to proceed with Phase 1 development. The production well was moved slightly toward the Engineering Complex (Figure A1.2) in order to take advantage of the proximity of the tunnel system (which would later house the piping network for the groundwater system). The larger diameter production well, P1, did not meet with expectations. The projected flow during drilling of this well was estimated at 800 IGPM (roughly three times the yield of the test well), however, the actual yield upon completion was closer to 345 IGPM (25.7 l/sec), with a final depth of 137.2 metres.

The sensitivity of well placement to well yield became evident early on in this phase of development and the geological complexity of the site was given added consideration. Well yield was identified as being directly related to placement of a well

MULTILEVEL PIEZOMETER INSTALLATION WELL M1

along a fault. Based on the data from RC, the second well, P2, was located near the Commons Building in the Residence Complex (Figure A1.2). The estimated yield, at the final depth of 122 metres, exceeded 1000 IGPM (75.7 l/sec). Unfortunately, the next production well, P3 (near the Athletics Complex), which was drilled along a line extending from P2 to Elba2, was unsuccessful. The yield of this deep well (151 metres) was low compared to the other campus wells. At the bottom, it amounted to 80 IGPM (6.1 l/sec). This well was retained as a monitoring well (P3/M2).

From this point forward, it was decided that a preliminary (6 inch) test or pilot hole should be drilled before an expensive production well was drilled. The first official test hole, T1, was located west of the Maintenance Building (Figure A1.2). Water-bearing fracture zones were intersected at 62.5 and 70.1 metres and the yield increased to approximately 240 IGPM (18.2 l/sec). The favourable results at this pilot hole prompted the drilling of the production hole, P4, 3 metres north of the test hole (Figure A1.2). Bedrock was encountered at a shallower depth (2.4 metres compared with 9.75 metres at the test hole), and the yield was estimated at only 600 IGPM (45.4 l/sec) upon completion of this 122 metre deep well. It was noted (WESA, 1988) that most of the water was derived from fractures located within the uppermost 67 metres. Deeper drilling only added about 50 IGPM to the total yield.

The next hole, T2/P5, was located near the northwest corner of Russell House in the Residence Complex (Figure A1.2). A major fracture zone was encountered at a depth of 109.7 metres and water yield increased from 400 IGPM (30.3 l/sec) to between 800 and 1000 IGPM (60.6 and 75.7 l/sec).

Well T3 was drilled north of the Tory Building, just south of Library Road (Figure A1.2). Bedrock was encountered at a depth of 42.7 metres and the yield at 97.5 metres was only 20 to 30 IGPM (1.5 to 2.3 l/sec). The well site was abandoned.

Well T4/P6 was located beside the Residence parking lot. The water yield was estimated at 1200 IGPM (90.9 l/sec) at the bottom of the 122 metre well.

In total, five production wells (P1, P2, P4, P5 and P6) and four monitoring wells (M1, P3, T1, and T3) were completed during Phase 1, between April 1 and August 25, 1988. The production wells were drilled with a diameter of 30.5 cm (12 inches) for 61 metres through the overburden and upper bedrock. Steel casing was pressure grouted from the bottom of the casing to ensure a proper seal. The remainder of each well was drilled with a diameter of 25.4 cm (10 inches). All wells were at least 122 metres deep. Wells drilled along proposed faults during Phase 1 have estimated long-term yields ranging from 50 l/sec to 69 l/sec (WESA, 1988), whereas, wells located on bedrock highs, which are slightly offset from the faults contained in bedrock valleys, have much lower yields (23 to 25 l/sec). For example, well P3 was drilled east of the Eastern Fault and its yield was less than 6 l/sec.

A complete well testing program was also completed. All new production wells (P1, P2, P4, P5 and P6) were step tested prior to constant discharge testing at a rate determined from the step test results. Recovery testing was also undertaken immediately following pump shut off. In addition, a series of six reinjection tests were conducted on some of these wells. All the wells located on campus were monitored as observation

points during these tests, thus a significant data base was created. The results of these aquifer tests are discussed in detail in Chapter 4.

On the basis of this testing program, WESA (1988) undertook model simulations which led to a recommendation on a well configuration. This well configuration was derived using an analytical model. This model incorporated the actual aquifer test data for the Phase 1 wells using the Principle of Superposition (discussed in Chapter 5) and simulated the response of these production wells to various rates and pumping/reinjection configurations. The optimum configuration was assessed for thermal breakthrough time and temperature using an analytical thermal model. The recommended configuration consisted of the wells and rates as outlined in Table A1.1.

Table A1.1 : Simulated Optimum Discharge/Injection Scenarios for Phase 1. Pumping rates and Drawdown are Negative Numbers, Reinjection Rates and Positive Head are Positive Numbers. (Data From WESA, 1988)

	Production Well	P1	P2	P4	P5	P6
SUMMER	Rate (l/sec)	-30	-45	30	95	-50
	Water Level (metres)	-20.9	-0.7	14.7	17.4	-6.6
WINTER	Rate (l/sec)	25	65	-30	-95	35
	Water Level (metres)	13.2	-4.8	-19.1	-19.5	-3.4

In conjunction with the Phase 1 drilling program, Wilson (1990) initiated her research on the hydrogeology of the Carleton campus. Her research focused on an examination of both the geological and hydrogeological characteristics of the aquifers using borehole geophysics, aqueous geochemistry and the well logs. A monitoring program to measure hydraulic heads in the wells, prior to implementation of the operating

system, was also undertaken. Using these data, Wilson (1990) identified the potential aquifers beneath the campus, expanded the geological database and provided the base data from which to undertake this current research. When this current research was initiated, the system was fully operational, and had been for about three years. Wilson's static water levels have proven invaluable to this study, and have subsequently been used to develop the steady state hydrologic model of the aquifer. The steady state hydrologic, thermal, and chemical regimes are discussed in detail in Chapter 3.

Phase 1 of the Carleton University Groundwater Energy Project, formerly called the Carleton University Heat Pump Project, was officially opened in February 1990. The system was designed to provide heating and cooling to approximately forty percent of the buildings on campus, primarily in the Residence Complex and Engineering Complex.

A1.4 The 1993/1994 Phase 2 Development Program

The accurate positioning of wells along faults prior to drilling was considered the most effective means of obtaining sufficient quantities of water for the successful implementation of Phase 2. In addition, all new production wells drilled, during this and any subsequent phases, were to be preceded by test or pilot holes. During the intervening years, between the implementation of Phase 1 and the initiation of Phase 2 planning, a series of geophysical techniques were employed, with variable success, to delineate the faults. These are referred to in Chapter 2.

The portion of the campus slated for expansion of the system included the area east of the railway tracks. In particular, Robertson Hall, the Maintenance building, the Elba greenhouses, the new Carleton Technology Training Center (CTTC) building and

possibly the Athletics Complex. The previous studies on bedrock topography and stratigraphy, in conjunction with results from gravity profiling, identified four possible sites for new production wells, which were expected to be located on or near faults. A target groundwater supply of 1600 IGPM (125 l/sec) was sought for this phase of development. New wells were to be incorporated into the existing configuration of wells operating in Phase 1.

Carleton Energy Services Inc. (CESI) was contracted by Carleton University to provide drill and test site supervision throughout the drilling program and to gather and interpret aquifer test data obtained from testing. The author of this thesis was contracted through CESI to interpret these data and to design a configuration of wells with optimum production/injection rates suitable for the expanded system. The following is a summary of the drilling results as reported in CESI (1994).

The first test well, T21, was located immediately south of Robertson Hall (Figure A1.2). The aim was to locate this well as close to the edge of the buried bedrock valley as possible, in order to intersect the main Eastern Fault, which was identified on the basis of bedrock topography and gravity profiling. The bedrock surface was encountered at a depth of 15.6 metres, indicating that the valley had indeed been encountered. This well intersected bedrock stratigraphically similar to well P4 in the Central Block. Major water-bearing fractures were encountered at 61 metres and at 71 metres. Test well yields were in excess of 200 IGPM at the completed depth (97.0 metres) and resulted in the conversion of this well to a production well. The total depth of the production well, P21, was 91.8 metres and the yield at the bottom of the hole was estimated at 900 IGPM (50 l/sec). The

well diameter decreased from 30.5 cm (12 inches) at a depth of 70 metres to 25.4 cm (10 inches). The well was flowing when drilling was complete. The ground surface elevation at this site is approximately 63.0 m.a.s.l., making this static water level significantly above that measured for the other wells.

The drillers experienced a great deal of difficulty while drilling this well. Blocks of bedrock at the bottom of the unconsolidated material (probably talus blocks) caused the drill rod to snag frequently. This difficulty was experienced by a separate drilling contractor when the twin production well, P21b, was drilled during Phase 3 (discussed later).

Test well T22 was located near the northeast corner of the Maintenance Building (Figure A1.2). The depth to the bedrock surface is 21.9 metres, significantly deeper than at well P21. Well T22 intersected formations intermediate between the Eastern Block and the Central Block, namely the Cobourg Formation. The bedrock has been displaced downward at this location relative to at P4, however, it has not dropped quite as far as at wells Elba2, P2 or P3. This implies that although P22 is closest stratigraphically to the Eastern Block wells, it may display a slightly different response than that characteristic of, for example, Elba2.

Major water-bearing fracture zones were intersected at 95 metres and 111.2 metres. A flow rate of over 200 IGPM in the 129.3 metre test hole led to the conversion of the test hole to a 30.5 cm (12 inch) diameter production well, P22. At the completion depth of 71.0 metres, the estimated yield was 850 IGPM (65 l/sec).

Test well T23 was located in the open ground immediately south of the Renfrew Residence (Figure A1.2). The depth to the bedrock surface was slightly less than at the previous two wells (11.44 metres). Stratigraphically, this well is located within the same interval as wells P4 and P21.

Main water bearing fracture zones were intersected at 69.8 metres, 99.1 metres and 107.0 metres. Test well flow rates exceeded 250 IGPM (18.9 l/sec) at the completion depth (122.0 metres). The production well, P23, was drilled to a final depth of 115.9 metres. At the bottom of the hole, the yield was estimated at greater than 1000 IGPM (75.75 l/sec). While this well was being drilled, flow at P21 ceased. This was the first indication that there was an excellent connection between these two wells. This was later confirmed during aquifer testing (Chapter 4).

Perhaps the most perplexing event during the course of the Phase 2 development program occurred while well P23 was being logged using the borehole geophysics logging system of Geological Survey of Canada. All new wells were logged to obtain, among other information, both a temperature and natural gamma profile down the borehole. At P23, the temperature probe became lodged in the borehole and would not pass below a depth of approximately 70 metres, which coincided with the depth of the first major fracture zone. The drilling contractor was engaged to clear out the well because it was suspected that a portion of the well had collapsed. The borehole geophysics probe again became lodged at this depth. On two other occasions, the drilling contractor cleared out the hole, bringing the drill stem well above and well below the level of the blockage. Each time the way proved to be clear. No obstacles could be felt by the drilling contractor.

However, the probe would not pass through. This matter did not resolve itself, and the geophysical profile ends at the blockage depth. Subsequently, the twin well P23b has been logged.

Test well T24 was located to the south of the Lanark Residence, beside the access road (Figure A1.2). Bedrock was encountered at a depth of 20.7 metres. At the completion of the test well, at a depth of 83.0 metres, the yield was between 150 and 180 IGPM (11.4 and 13.6 l/sec). A yield of approximately 600 IGPM (45.5 l/sec) was projected for this well if it were converted to a 30.5 cm (12 inch) diameter production well. The University decided to complete this well as a monitoring well rather than as a production well. The well intersected Central Block lithologic units.

An examination of the gravity data, presented in Chapter 2, shows that this well is within a valley, or least on the edge of one. The fault passes to the north, based on stratigraphic evidence. However, it is still uncertain how far away this fault is. It must pass north of T24, north of P23, but south of wells P5 and P2. The other question that remains to be answered is whether the fault passes beneath the center axis of the valley or beneath the side walls. A more thorough geophysical program in a natural setting (*i.e.*, no buildings) would help to determine the answer. Since well placement along a fault is a critical factor for increasing well yield, this information would be invaluable.

The final well drilled as part of Phase 2 was the Elba3 well. This new 38.1 cm (15 inch) diameter production well is located immediately northeast of the existing Elba2 well (Figure A1.2). Elba3 was drilled because the original 25.4 cm (10 inch) well (Elba2) was of insufficient diameter to house a pump capable of fully utilizing the well capacity. It was

decided that Elba3 would be the pumping well, and during the alternate season, Elba2 would be the reinjection well. Because the two wells are located essentially at the same spot, they are expected to act as a single unit, hydrogeologically. That is, all fractures, lithological units and thus, aquifer physical parameters should be the same.

Major water-bearing fracture zones were encountered at a depth of 83.2 metres, 89.6 metres and 103.3 metres, increasing the yield from 750 IGPM (56.8 l/sec) at 83.2 metres to in excess of 3000 IGPM (227 l/sec) at the final depth of 122 metres.

The drilling program for Phase 2 was completed between October 1993 and May 1994. A total of five permanent wells were drilled (P21, P22, P23, T24 and Elba3). All production wells, except Elba3, were cased with at least 30 metres of Schedule 40 new pipe casings with an inside diameter of 30.48 cm (12 inches). These casings were installed and pressure grouted from the bottom. Elba3 has an inside casing diameter of 38.1 cm (15 inches) and is cased to a depth of 36.6 metres. Well T24 has a 25.4 cm (10 inch) casing grouted to a depth of 31.7 metres.

As with Phase 1, a complete aquifer testing program was carried out following the drilling of the new production wells, P21, P22 and P23. Elba3 was not aquifer tested because the original well, Elba2, had been tested on two previous occasions. The results of these tests are presented and interpreted in Chapter 4. This suite of tests constitutes the most recent acquisition of aquifer test data.

The expanded Phase 2 system was designed to incorporate the five original production wells (P1, P2, P4, P5 and P6) with the new wells P21, P22, P23 and Elba3. The optimum configuration was designed, in the same manner as that of the original

system, namely, using analytical methods to determine hydraulic heads and breakthrough temperatures. The design was undertaken in conjunction with this research by the author and submitted to CESI for submittal to the University (CESI, 1994). The final configuration, selected by the engineer, provided a sufficient quantity of water for the Phase 2 requirements. This configuration was modified slightly from that contained in CESI's 1994 report, and was submitted separately. The analytical modelling method was employed in order to retain some consistency with the original methodology of WESA (1988), but more importantly, because the original system had not failed. An alternative modelling method, using a numerical approach, was not selected because the aquifer was sufficiently complex and was the topic of ongoing research. A more thorough discussion of this and other models is made in Chapters 5 and 8.

A1.4.1 The 1994/1995 Phase 3 Development Program

Immediately following the Phase 2 development program, the Phase 3 drilling program, was initiated (Fall, 1994). Phase 3 was to add additional buildings, hence more well capacity, to the Phase 2 system. In order to optimize the well capacities of the Phase 2 wells, the twin system was introduced. Additional wells at new locations were also required to fulfill the water requirements for Phase 3.

The new 38.1 cm (15 inch) diameter twin wells, P5b, P21b, P22b and P23b, located immediately adjacent to the existing production wells, P5, P21, P22 and P23, respectively, would house the pumps and the original wells would be used for reinjection. Hydrogeologically, the twin pair would act as a single well. The drilling program for these

twin wells needed to precede work on the piping network, well chambers and building retrofits.

CESI was contracted by Carleton University as the hydrogeological consultant during Phase 3. The drilling results for the new test and production wells, summarized in this section, are reported in CESI (1995). The drilling results for the four twin wells will not be discussed because well yields, bedrock elevation, fracture-zone intervals etc., are all similar to those of the original production wells.

The drilling program was to include an evaluation of the hydrogeological potential of the central portion of the campus. This was to be accomplished by drilling a number of test holes with the possibility of converting these to production wells if sufficient supplies were found. As noted in previous sections, the hydrogeological potential of the Central Block is variable. M1 was an excellent test well, however, its companion production well P1 is not. Because the majority of the campus buildings are situated within the Central Block, a sufficient supply in this area was sought to avoid extensive piping.

Three 15.24 cm (6 inch) diameter test holes were drilled in the central portion of the campus. Well T31 is located on the grassy mound south of the Unicenter Building and east of the Tory Building (Figure A1.2). Bedrock was relatively shallow (7.0 metres). Major water-bearing zones were encountered at 33.5 metres, 45.0 metres, and between 119.2 and 120.1 metres. At the bottom of the test hole (147.5 metres), the yield was estimated at close to 200 IGPM (15.2 l/sec). This well had a sufficient yield to warrant its conversion to a production well (CESI, 1995). The casing in this test well remains ungrouted.

Well T32/P32 is located alongside the Rideau River at the corner of the parking lot beside the Herzberg Laboratories (Figure A1.2). Bedrock at this well (2.1 metres depth) was extremely shallow, as expected, because its position coincides with the head of the rapids in the river. Major water-bearing zones were encountered at 38.1 metres, 42.2 metres, 56.7 metres and between 98.5 and 99.0 metres. The yield was estimated at greater than 200 IGPM. A 38.1 cm (15 inch) production well was drilled over-top of the test well to a final depth of 95.1 metres. This well was not developed, therefore, no estimates of yield were the production well were obtained.

The final test hole, T33, is located at the southwest corner of the grassy area north of the Unicenter and west of the Mackenzie Building (Figure A1.2). A preliminary test hole had been located just north of the final well position, but drilling was abandoned because the bedrock was too deep. Well T3, located across the road, had been abandoned during Phase 1 because of its low yield. Well T3 is considered to across the Tory-Herzberg Fault because the depth to bedrock is 42.7 metres. The drill rig was moved in order to try and intersect this fault.

Bedrock in well T33 was at a depth of 9.1 metres. Water-bearing zones were encountered at 25.3 metres, 52.1 metres, 58.8 metres, between 83.8 and 84.4 metres and at 108.5 metres. The total yield at a final depth of 123.4 metres was estimated at 87 IGPM (6.6 l/sec). Despite extensive fracturing, which was manifested on many occasions by a significant drop of the drill rod, the yield was surprisingly low. It is the author's contention that long term and ongoing pumping at well P1 is the cause of this low yield. This is

discussed in Chapter 4, following an analysis of the aquifer test data. The casing in this test well remains ungrouted.

One further production well was drilled during this development phase. The M1 well had been converted to a multilevel piezometer installation at the end of Phase 1. Although not mentioned above, during the Phase 2 drilling program, initial steps were taken to convert this well to a production well. The piezometers and the cement grout between the levels had been drilled out and the well was backfilled with gravel. During Phase 3, the well was drilled as a 38.1 cm (15 inch) diameter production well to a final depth of 121.4 metres. At the bottom of the hole, the yield was estimated at 200 IGPM (15.2 l/sec); this being less than the estimated yield for the original 15.24 cm (6 inch) test well. Again, long term and ongoing pumping at well P1 is believed to be the cause of this low yield.

During the course of this drilling program, the University administrators decided that using groundwater was not economically viable, and the debate over this issue continues. The optimum project design, that would provide thermal balance, was costly to implement and the University felt that such an expenditure could not be justified in a time of fiscal restraint. Despite efforts on the part of the engineering firm (N. Haitas and Associates Ltd.) and the hydrogeological consultants (CESI) to reduce the scope of the project and thus stay within a manageable budget, all contracts were terminated and work was halted.

As a result of the University's decision, a number of the wells were not developed (*i.e.*, they were not flushed to remove drill cuttings and improve well yield). The twin

production wells P21b and P22b were not developed, and neither was the new production well P32. Likewise, no aquifer testing was conducted during this phase of development. To date, none of the work has been completed and the wells remain as they were at the termination of the drilling program.

The preceding discussion was intended to document the history and the critical data related to the drilling on campus. Sufficient details have been presented to enable the reader to grasp a portion of the complexity of the system as it pertains to well yield.

A1.5 Ground Surface Elevations for Wells

Table A1.2a : Measured and Estimated Ground Surface Elevations for Wells on Campus. Estimated Elevations are Shaded.

Well	G. S. Elevation (m.a.s.l.)	Casing Height above G. S. (metres)	Well	G. S. Elevation (m.a.s.l.)	Casing Height above G. S. (metres)
P1	63.316	N/A	T3	68.679	0.72
P2	62.937	N/A	Tory	67.165	below g.s.
RC	61.290	1.00	M1	63.115	0.85
P3	59.598	0.91	P21	63.0	N/A
P4	61.225	N/A	P22	61.4	N/A
T1	61.251	0.98	P23	63.2	0.65
P5	63.169	N/A	T24	63.2	0.38
P6	65.469	N/A	T31	68.1	0.55
Elba1	61.770	0.33	T32	61.0	0.42
Elba2	59.770	1.79	T33	65.3	0.59
Elba3	60.0	0.62			

Table A1.2b : Geodetic Benchmark Elevations in the Vicinity of Carleton University.

Identification Number	Location	Elevation (m.a.s.l.)
GEODETIC 75 380 1A	Canal - Hartwell Locks	70.920
GEODETIC 87-1	West side, North end - CPR bridge over Rideau River	59.764
GEODETIC BM # 67U517	East side of Maintenance Building - near main entrance	61.489
GEODETIC BM # 209.34	East end Renfrew House - standpipe	63.807

Appendix 2
BEDROCK INTERPOLATION

A2.1 Review of Interpolation Methods

A regularly organized array of points is easier to contour than a set of randomly scattered data points. This is true whether a computer or an experienced geologist does the contouring. Likewise, different interpretations of the same set of data can be made depending on the method of interpolation. Even if different geologists have the same interpretation of an area, the final contour map will be different for method of contouring.

- Mechanical contouring proportions the spacing of contours between adjacent control points.
- Equal-spaced contouring assumes a uniform angle of dip over all or part of the mapped area, and results in highs or lows that may be artifacts of the method rather than actual structural relief.
- Parallel contouring produces maps with parallel or near-parallel contours which do not have to have uniform slopes.
- Interpretative contouring (performed by the experienced geologist) combines geologic knowledge of the structural and depositional style of the area and imagination.

Computer contouring packages employ gridding or triangulation algorithms in order to create a uniformly-spaced set of data. Different weights may be applied to the data depending on the method of interpolation chosen, therefore, the final contour maps, produced by each method, can look very different.

The most frequently used data structure for geologic mapping is the grid: an array of nodes with uniform X and Y increments. Estimates of the attribute being mapped (*i.e.*,

the Z values) are assigned to these grid nodes through interpolation between data points. The integrity of the data may or may not be preserved. Extrapolation of the data beyond the area in which there is real data is also possible.

The second type of data structure is the triangular network or TIN (triangulated irregular network). This structure is formed by connecting neighbouring data points into sets of triangles; each data point is the vertex for the several adjacent triangles. The observed Z values are assigned to these nodes. Triangular sub-nodes may be generated, with the Z values calculated by interpolation. Triangular mesh systems are good at interpolation, but generally only project beyond data when “dummy” points are specified at the perimeter of the region.

Unrealistic extrapolations can occur in two common situations. First, clustered data points containing different Z values often produce unusual contours. In this case, large surface gradients are carried past the data points causing the surface to overshoot the data. Projection away from the data can extend local trends as unrealistic planar surfaces well beyond what is reasonable. Second, the extrapolation of naturally bounded data can cause a generated surface to exceed the limits of the natural bound.

The low-order-polynomial surface-fitting algorithms, namely least-squares algorithms, can produce unwanted extrapolations. These are best used when a uniform data base exists. Calculated node values, made with moving-weighted-average algorithms, such as simple distance-weighted algorithms or kriging, cannot exceed the range of the data. They do not project trends, but construct an average. The value of that local average depends on the parameters that regulate the number of points required for the search as

well as the search radius. When a large number of points, and a large search radius are employed, the calculated values tend toward a global average. In contrast, fewer points enhance local averages.

The difference between kriging and the other weighting methods is the method of calculation of the weights of the surrounding points. In the inverse distance method, for example, the contribution of each observation point depends on its geographical distance to the node and the sum of all distances involved. Once the sampling search is done, each weight is computed separately.

The calculations involved in kriging are more demanding, as more factors enter the estimation and all weights are the solution of a system of simultaneous equations. Kriging weights depend not only on the distance between a node and observation, but on the distances among observations as well.

Several bedrock data interpolations were undertaken, using a variety of search methods and search criteria, to achieve a geologically feasible representation of the bedrock surface. The results of these interpolations are discussed in Chapter 2.

BEDROCK ELEVATION DATA

BEDROCK ELEVATION DATA USED FOR BEDROCK INTERPOLATION MAP

X-COORDINATE (METRES)	Y-COORDINATE (METRES)	BEDROCK ELEVATION (M.A.S.L.)
110	83	57.90
142	100	57.90
165	145	57.00
100	385	53.60
185	320	51.66
0	515	56.80
0	600	57.60
25	560	57.60
40	425	53.60
65	490	54.60
170	580	55.20
175	565	49.80
140	620	56.70
190	235	55.50
210	280	54.50
230	270	54.90
250	260	54.90
310	355	52.70
315	380	53.60
315	395	53.30
320	375	53.30
380	580	48.25
370	430	51.20
210	695	52.40
245	650	51.20
260	720	48.80
275	750	46.60
305	755	49.70
320	685	49.10
375	730	48.80
400	710	50.60
395	965	45.70
440	335	51.92
465	305	51.80
440	495	53.60
495	470	53.60
500	595	52.40
530	510	54.20
535	585	55.20
435	625	50.90
550	445	51.97
530	435	53.05
535	415	52.53
565	420	53.35
560	450	52.62
595	400	54.30
595	430	54.20
590	470	52.43
520	650	50.30

X-COORDINATE (METRES)	Y-COORDINATE (METRES)	BEDROCK ELEVATION (M.A.S.L.)
560	630	52.40
575	695	53.30
505	955	45.70
525	915	46.30
525	805	50.30
580	840	49.70
650	315	40.53
630	840	49.70
660	860	49.70
625	445	52.70
625	540	54.20
630	410	53.00
630	465	53.90
840	500	54.20
665	465	53.60
675	435	52.90
685	470	53.90
700	435	51.90
700	550	53.40
710	400	36.90
710	465	54.20
760	400	48.10
615	700	52.10
690	700	51.20
695	675	48.70
695	715	51.20
705	670	48.70
720	620	51.50
720	640	49.70
755	685	51.20
765	700	49.70
770	770	51.80
780	630	51.80
795	690	49.70
810	450	51.90
860	390	55.50
870	380	57.30
885	365	57.30
885	620	40.10
900	610	54.86
905	608	57.67
908	615	56.48
912	625	55.72
915	612	58.85
918	605	59.77
915	590	60.99
915	598	59.14
910	608	57.26
925	592	59.91
915	605	58.64
920	608	59.16

X-COORDINATE (METRES)	Y-COORDINATE (METRES)	BEDROCK ELEVATION (M.A.S.L.)
919	600	59.71
925	602	59.40
930	596	58.00
935	608	57.27
890	375	60.30
890	400	60.30
900	360	58.20
900	385	60.00
905	355	53.90
910	350	53.90
845	370	53.40
910	370	60.00
915	345	51.50
920	360	58.20
925	350	53.90
925	380	60.60
930	370	59.70
930	400	61.60
935	350	53.90
935	365	59.10
945	350	55.20
945	355	55.20
945	375	60.30
950	395	61.60
970	390	62.20
975	400	61.60
980	390	61.90
985	395	61.90
990	390	61.00
805	940	52.10
955	815	47.00
810	450	51.92
815	680	50.00
820	620	47.20
835	675	49.00
880	665	48.80
885	620	40.06
910	640	38.70
920	650	52.10
930	610	57.00
935	605	57.30
955	745	44.30
985	790	42.70
985	665	51.50
995	630	56.40
835	445	50.00
845	475	45.70
850	455	56.10
855	435	55.20
870	475	55.20
870	540	57.60

X-COORDINATE (METRES)	Y-COORDINATE (METRES)	BEDROCK ELEVATION (M.A.S.L.)
880	410	59.10
880	420	57.90
880	500	50.60
880	570	59.40
890	430	57.00
890	450	55.60
895	470	55.60
900	565	58.40
905	410	60.60
905	430	58.50
915	475	57.60
915	570	51.80
920	425	59.40
920	580	59.40
925	490	57.90
925	500	55.80
925	555	50.00
940	470	60.00
940	530	54.60
945	520	54.90
955	420	60.30
955	430	61.60
960	425	61.30
960	435	57.90
980	435	61.60
995	445	61.00
995	500	57.90
1000	465	59.40
1010	370	59.70
1025	360	59.40
1025	375	60.96
1035	395	60.00
1035	320	57.90
1040	320	19.80
1040	380	59.70
1060	355	59.40
1080	365	59.70
1110	390	59.40
1020	265	37.50
1000	420	60.00
1005	425	60.65
1020	430	61.30
1030	415	60.35
1030	575	51.80
1050	600	59.10
1060	580	57.30
1065	405	61.30
1080	700	47.56
1100	480	58.50
1115	530	58.80
1120	445	59.40

X-COORDINATE (METRES)	Y-COORDINATE (METRES)	BEDROCK ELEVATION (M.A.S.L.)
1120	475	59.70
1125	495	57.00
1125	555	56.70
1135	440	58.20
1135	465	54.90
1140	485	57.30
1155	500	57.60
1165	430	57.60
1165	550	58.50
1170	595	56.00
1180	595	55.50
1185	415	59.70
1195	445	59.70
1200	505	57.30
1200	545	56.70
1020	640	53.30
1050	640	55.50
1055	660	55.80
1060	675	59.10
1065	665	56.10
1085	660	55.80
1145	480	57.00
1155	440	57.00
1165	400	56.00
1165	445	57.60
1170	410	57.30
1240	350	29.56
1380	265	38.40
1215	450	56.70
1215	470	59.70
1230	500	58.80
1115	420	59.56
1235	490	59.00

Appendix 3
MICROGRAVITY PROFILING

A3.1 The Gravity Method

Gravimetry is a potential field method in which local variations in the densities of near surface rocks and materials can cause minute measurable changes in the main gravity field. Gravity surveys have been used extensively and successfully for delineating buried valleys (Hall and Hagnal, 1962; Spangler and Libby, 1969; Rankin and Lavin, 1970; Ibrahim and Hinze, 1972). When buried valleys contain permeable materials such as sand and gravel, generally of lower density than their surrounding rocks, a gravity survey can be used to delineate the valley (Carmichael and Henry, 1977; Stewart, 1980; Wilson *et al.*, 1983; Hansen, 1984).

The gravity method has been used not only to locate buried bedrock valleys, but also to define the configuration of the bedrock surface, particularly where some geological control was available (Klasner, 1964; Hall and Hajnal, 1962). Ibrahim and Hinze (1972) combined gravity data and well data to compute bedrock topography. Stephenson *et al.* (1993) used high resolution seismic and gravity surveys to identify Quaternary deformation across a fault in Utah. Gravity surveys have also been conducted in karst terrane (Colley, 1963; Omnes, 1977; Barrows and Fett, 1985) in an attempt to locate sink holes. Denne *et al.* (1984) combined remote sensing with geophysical investigations to study large (greater than 120 metres deep and greater than 120 kilometres in length) buried pre-glacial drainage-ways.

A recent study by Pool and Eychaner (1995) documented water level variations in an aquifer and associated gravity variations. A temporal gravity survey was undertaken to measure changes in aquifer storage. Changes in storage reflect variations in the volume of

water stored in the subsurface. For an unconfined aquifer, storage change occurs primarily within the interval of water level change. The change in density associated with the increasing or decreasing volume of stored water with time can be measured using relative gravity. Measured gravity changes of about 13 microgal represented storage changes of about 0.30 metres of water. Although this study was undertaken over a large area with station spacing on the order of kilometres, the details of the survey demonstrated that gravity meters are now sufficiently precise to measure variations in the gravitational attraction which are on the order of a few tens of microgals.

A3.2 Theory of Gravimetry

The gravitational acceleration, g (m s^{-2}), in scalar form, caused by the gravitational field produced by a spherical body is given by the equation (Telford *et al.*, 1976):

$$g = -\frac{GM}{R^2} \quad \text{A3.1}$$

where G is the universal gravitational constant ($6.67 \times 10^{-12} \text{ m}^3 \text{ s}^{-1} \text{ kg}^{-1}$), M is the mass of the body (kg) and R is the radius of the body (m). The gravitational effect of a tabular body can be approximated using a relation that is independent of distance, provided the lateral extent of the body is much greater than the distance to the center of the body. This relation is known as the Bouguer slab approximation. Telford *et al.* (1976) presented the mathematical expression for the change in gravity associated with a change in density in a slab of large lateral extent:

$$\Delta g = 2\pi \cdot G \cdot \Delta \rho \cdot b \quad \text{A3.2}$$

In this expression, $\Delta\rho$ is the density contrast (kg m^{-3}) between the slab and the surrounding material and b is the thickness of the slab. The equation is commonly used in exploration to estimate the density or thickness of an anomalous body such as a sedimentary basin and has been used to estimate the storage variations in an aquifer (Pool and Eychaner, 1995). Gravitational anomalies are commonly on the order of a few mgals (where $1 \text{ mgal} = 10^{-3} \text{ cm s}^{-2}$), although anomalies in the microgal range have been reported.

Gravity anomalies from density variations between glacial drift and the surrounding formations are superimposed on the anomalies caused by changes in bedrock topography. Thus, in large-scale surveys it is necessary to isolate residual anomalies caused only by bedrock topography. In a few instances, the buried valley anomalies were isolated from other sources by subtracting trends determined from "small-scale" gravity studies (Wilson *et al.*, 1983). In those cases, station spacing was on the order of kilometres. Maps were used as an aid for regional-residual separation from observed gravity.

Density contrasts in shallow formations, because of their nearness to the surface, may have a marked effect on gravity observations. Hall and Hajnal (1962) noted that a buried valley need not be cut into bedrock to provide sufficient density contrast to give rise to a measurable gravity anomaly, thus, this method may be applied to valleys cut into glacial till or other unconsolidated sediments.

The zone of anomalous density in a buried valley commonly occupies most or all of the valley cross-section (Hall and Hajnal, 1962). In all but the most youthful valleys, the low density zone will have a width several times the thickness. The maximum anomaly height will be approximately given by the Bouguer slab approximation given in Equation

A3.2, with b equal to the thickness of the valley and $\Delta\rho$ equal to the density contrast between the surrounding bedrock and the lower density overburden.

A3.3 Survey Techniques

The Carleton University campus occupies a total area of just over 1 km². Based on bedrock topography, the widths of the bedrock valleys associated with the faults were expected to be less than 50 m. Thus, the scale of this gravity survey was much smaller than those referred to above. Using an estimated density contrast of 0.3 g/cm³ to represent the contrast in density between a limestone or shale bedrock and a glacial till, sand or clay overburden, and an estimated minimum valley depth of approximately 10 metres, the calculated anomaly was expected to be on the order of 0.125 mgal.

Given this magnitude for the anomalies, it was critical that the relative station elevations were known accurately. It was determined, therefore, that elevations be obtained with an accuracy of at least ± 1.0 cm, which corresponds to an error of ± 0.003 mgal at each station. The station elevations were measured with at least 0.01 metre accuracy relative to a geodetic benchmark situated on campus. Profile locations were selected on the basis for the need to define the bedrock surface for well siting. The new wells needed to be located fairly close to the existing tunnel system so that pipes for the groundwater system, could be easily accessed. This made the design of the gravity survey more difficult, as a tunnel would greatly affect the results if it were close to a gravity station. Profiles were cut short or diverted in order to avoid both the tunnel and building problems.

A Lacoste and Romberg Model D “microgal” gravimeter was used for this investigation. The instrument and the training required to operate the meter were supplied by the Geophysics Division of Natural Resources Canada. The gravimeter is a null-reading device which uses a test mass attached to a highly sensitive spring to measure the gravitational field. Measurements are made by placing the instrument on the ground, levelling the meter, and adjusting the spring length with a screw to balance the spring tension with the gravitational force of the Earth. A manual screw is used for coarse adjustment of the spring length and an electrostatic nulling device (Valliant *et al.*, 1986) is used for fine adjustment. The addition of an electrostatic nulling device made it possible to achieve the precision required for this survey. A consistent three minute interval was used between the time when the instrument was unclamped, adjusted using the coarse screw and then nulled using the electrostatic device. In order to eliminate variations in gravity introduced by the effect of non-linear spring hysteresis, the readings had to be taken during precisely the same interval from the time that the instrument was unclamped. To further reduce the effect of spring hysteresis, the coarse screw was adjusted and the electrostatic device allowed to operate for approximately the full three minutes.

The Lacoste and Romberg Model D gravity meter is a relative gravimeter and is used to measure the relative variation in the gravitational attraction from a base station. This is in contrast to an absolute gravity meter, which is inherently more precise, but is expensive and not as portable. Relative gravity surveys are conducted in a closed-loop format. A traverse is generally made from a base station of known gravity to stations of unknown gravity, followed by a return base station to determine the closure error. Several

closed loops may be performed during a survey for the purpose of evaluating meter drift and the repeatability and accuracy of the measured differences in gravity.

The technique used at Carleton was a 'leap-frog' technique. Rather than completing an entire profile and then returning to a base station to repeat a measurement once (closed-loop), a series of measurements were made at four or five stations (stations 1 to 5) and then a measurement was made at the first of those stations (station 1). The survey continued on from the last occupied station a further four or five stations (stations 5 to 9) and then back to the first of that set (station 5). In this way, base stations along the survey line were established and measurements were made at these stations up to three times. At the completion of the entire profile the first station of the survey (station 1) was reoccupied.

Nine gravity lines were run, as shown in Figure A3.1. The stations were spaced 5 metres apart and profiles were typically 70 to 100 metres long. For the most part, the surveys extended along pathways or roads. The gravimeter tended to slide when placed on the grass and these areas were avoided when possible. Since many of the profiles crossed roads, the surveys were conducted in the early morning on the weekend in order to avoid the heavy traffic on campus. Curious joggers thumping the pavement right next to the meter often made it difficult to achieve a stable reading. The wind proved to be the most difficult to deal with. It was often necessary to try and shield the meter from gusts in order to obtain stable readings. Variations in gravimeter drift, as discussed in the following

Figure A3.1: Location of Gravity Profiles and Wells

section, are partially attributed to air temperature variations. Variations in the temperature of the instrument housing produce a non-linear instrument drift. This can be partially overcome by shielding the instrument from the sun, but the air temperature during the course of a May day is often large enough to create a disturbance.

Non-linear drift, caused by mechanical offsets (tars) in instrument readings, can occur if the instrument is subjected to unnecessary jars. Steps were taken to minimize possible jarring by using a padded instrument case. The meter was removed from the case at each station and then repacked carefully when the reading at the station was completed. Since all of the individual profiles were completed within one day and the stations were spaced close together, there was no need to transport the meter in a vehicle.

A3.4 Gravity Corrections

Gravity readings taken at stations during a survey will generally be influenced by topography and other factors. Corrections for variations in latitude, elevation and terrain are normally applied to the data to reduce them to the values they would have on some datum equipotential surface. An additional correction due to earth tides is also required. At the Carleton University site, neither latitude nor terrain corrections to the data were required. There is negligible variation in latitude over the survey area; and the terrain is generally flat. Corrections for variations in the barometric pressure are sometimes applied, however, as the duration of an individual survey was less than a few hours, this correction was not applied.

Gravity varies inversely with the square of distance, therefore, it is necessary to correct for changes in elevation between stations so that all of the measured values are

reduced to a common datum. This correction is referred to as the free-air correction and it takes no account of the additional material between the datum plane and the station. The free air correction dg_{FA} is obtained by differentiating the basic expression for the gravitational acceleration produced by the Earth (Equation A3.1):

$$\frac{dg_{FA}}{dR_e} = -\frac{2GM_e}{R_e^3} = -\frac{2g}{R_{eq}} \approx -0.3085 \text{ mgal / m} \quad \text{A3.3}$$

where R_{eq} is the equatorial radius of the Earth. As the negative sign implies, the correction is added to the field reading when the station is above the datum plane and subtracted when below it. Variations in ground surface elevation to within 1 cm are required in order to obtain accurate measurements of variations in the gravitational field to within 0.003 mgal.

The free-air correction, as noted above, does not take into account the material which lies between the datum plane and the station. The Bouguer correction (dg_B) accounts for the attraction of this material. If the station were centrally located on a plateau of large horizontal extent and uniform thickness and density, the gravity readings would be increased by the effect of this slab. Assuming the slab is of infinite horizontal extent and of uniform density, ρ , the variation in gravitational attraction resulting from changes in elevation above and below the datum is determined from:

$$\frac{dg_B}{dR_e} \approx \frac{dg_B}{dR_{eq}} = 2\pi G\rho \text{ mgal / m} = 0.0419\rho \text{ mgal / m} \quad \text{A3.4}$$

The Bouguer correction is applied in the opposite sense to free-air, that is, it is subtracted when the station is above the datum and added when below. Density estimates are discussed in Section A3.5.

Instruments for measuring gravity are sensitive enough to record the changes in g caused by movement of the sun and moon. These variations have amplitudes as large as 0.3 mgal (Telford *et al.*, 1976). The amplitude of these earth waves or tides is only a fraction of that of ocean tides and these variations depend on latitude and time of day. The change in gravity owing to earth tides (dg_T) is quite significant and although this variation can be accurately measured, it is commonly calculated and applied as a correction to the data. The computer software *MARCOR*, supplied by the Geophysics Division of Natural Resources Canada, calculates the tidal corrections for different latitudes at intervals of 10 minutes. A simple linear interpolation was used to obtain correction factors for the intervening time periods. Because the variation due to earth tides is a smooth function of time, it is often taken out in the instrument drift correction. However, earth tides and gravimeter drift were treated separately in this survey.

The final correction applied to the data was to account for gravimeter drift. The origin of meter drift and the measures used to control it and define its magnitude were discussed previously. An average linear drift was calculated from variations in gravity for all repeat stations. A stronger weight was placed on the base station reading which closed the loop. Drift ranged from 0.5 to 1.0 microgal/min. For the particular D-meter used, this estimate of drift is somewhat high. The duration of this survey was much shorter than those for larger scale surveys (these are on the order of days or weeks, or in the case of measuring tides - months). Drift estimates for previous surveys of these types are estimated over long cycles. Short term drift for a mechanical instrument of this type may be very different.

When all of the preceding corrections have been applied to the measured gravity readings, the *Bouguer gravity* for the station is obtained. Changes in the Bouguer gravity relative to the base station, which is designated as having zero gravity, are calculated according to the expression:

$$dg_B = g_{observ} \pm dg_{Latitude} + dg_{FA} - dg_B + dg_T - g_r \quad A3.5$$

where g_r is the corrected gravity reading at some particular base station.

A3.5 Density Estimates

Density estimates for bedrock and the overburden were determined from laboratory measurements of bedrock chip samples, obtained during drilling, using the Parasnis method (1952) and from published values. Grain density is one of the parameters calculated by the software used in analyzing thermal conductivity measurements made on the divided bar apparatus. This is discussed further in Chapter 7. The densities of the various formations are shown in Table A3.1. Porosities of the lower formations were measured on core samples and an average porosity for the formations is estimated at approximately 3%.

From Table A3.1, the calculated average in-situ density is $2.30 \pm 0.08 \text{ g/cm}^3$. Although this average value is calculated using different rock types, it characterizes a bulk sedimentary rock for the Carleton campus. Telford *et al.* (1976) quoted an average value of 2.50 g/cm^3 for sedimentary rocks. Table A3.2 shows ranges values for the different rock types along with the calculated averages.

Table A3.1: Grain Density Measurements and In-situ Density Calculations for the Various Formations Using an Average Porosity of 3%.

Formation	Grain Density (g/cm ³)	In-situ Density (g/cm ³)
Billings	2.17	2.13
Eastview	2.35	2.31
Cobourg	2.35	2.31
Sherman Fall	2.43	2.39
Hull	2.37	2.33
Rockland	2.28	2.24
Leray	2.42	2.38
Lowville	2.29	2.25
Pamelia / St. Martin	2.31	2.27
Rockcliffe	2.43	2.39

Table A3.2 : Bulk Densities of Common Sedimentary Rocks
(From Telford *et al.*, 1976)

Rock Type	Range (g/cm ³)
Sandstone	1.61 - 2.76
Shale	1.77 - 3.20
Limestone	1.93 - 2.90
Dolostone	2.28 - 2.90

Application of Nettleton's method (discussed in Telford *et al.* (1976)) for determining the overburden density for use in the Bouguer correction was unsuccessful due to the limited surface topographic variation along the profiles. In this method, Bouguer gravity is plotted using various estimates of the overburden density. The smoothest of these profiles, that is, the one which least reflects the surface topography, provides the best estimate of density. The application of this method requires that the density beneath the area profiled be uniform. All profiles in this survey were along lines which were expected to have variations in density.

An alternative method for determining the in-situ density was formulated by Parasnis (1952). This technique, also described in Telford *et al.* (1976), uses an analytical approach somewhat similar to Nettleton's standard graphical procedure.

When attempting to determine an average bulk or in-situ density over a survey area, the Bouguer anomaly $dg_{bouguer}$ is considered to be a random error of mean value zero. In particular, for a single profile, Equation A3.6 resembles a straight line of the form $y=mx+b$.

$$(g_{observ} - g_{base} + .094h) - dg_{bouguer} = 0.0419h\rho \quad A3.6$$

Here g_{observ} is the measured drift and tide corrected gravity at a station, $dg_{bouguer}$ is the Bouguer correction, ρ is the in-situ density, h is the elevation difference relative to a datum and g_{base} is the base station reading. If the expression in the brackets is plotted against the right hand term, and a best fit line is drawn through the points and through the origin, its slope will be ρ . Obviously, not all the points will fall on this line unless the ground is uniform and the Bouguer anomaly is everywhere zero. Therefore, the best fit is obtained by a least squares fit to the data.

Only one profile could be used for this analysis because most surveys were conducted along sections where the elevation of the bedrock surface was known to vary (Figure A3.2). An in-situ density for the River Profile (A - A') of 2.47 g/cm^3 was calculated. This value compares well with that quoted from Telford *et al.* (1976) for sedimentary rocks, and is probably more representative of the bedrock than the average value obtained from the bedrock chip samples.

Figure A3.2: Graph Showing a Portion of the Gravity Data from the River Profile Used in Parasnisi' Method of Calculating In-situ Density

Densities of unconsolidated sediments have been estimated in various studies. Although the composition of glacial till is variable, these estimates may only be viewed as possible ranges for overburden density. Stewart (1980) reported values ranging from 1.8 to 2.2 g/cm³ for alluvial and lacustrine deposits. Wilson *et al.* (1983) reported densities of 2.17, 2.06 and 1.9 g/cm³ for till, sand/silt/clay and sand deposits, respectively. Telford *et al.* (1976) reported average values of 1.92 g/cm³ for soil, 2.21 g/cm³ for clay (range of 1.63 to 2.6 g/cm³), 1.92 g/cm³ for silt, and 2.0 g/cm³ for sand and gravel. Given this range of values, and given that the overburden consists largely of dense till, a density of 2.17 g/cm³, as reported in Wilson *et al.* (1983), was selected to represent the overburden valley fill. Using the average bedrock density of 2.47 g/cm³, the density contrast is approximately 0.30 g/cm³. This density contrast was used in the modelling to determine the form and magnitude of the features causing the anomalies. Results are discussed in the following section.

A3.6 Gravity Results - Profiles

Nine gravity profiles are provided at the end of Appendix 3. These profiles are discussed below. Also included at the end of Appendix 3 are the interpolated bedrock and gravity data for each profile. The bedrock profiles are discussed in Chapter 2.

River Profile A-A'

A single valley, corresponding to one fault, was anticipated prior to the survey, however, two distinct valleys are shown. One is centered around the station at 60m and the other is around 95m. The dip at the beginning of the profile may represent the eastern

edge of another valley. There is also some variation in the depth to bedrock in the central portion of the graph. This profile also shows the location of production well P21, which was drilled on the edge of the valley near Robertson Hall (refer to Figure A3.1). The stratigraphic sequence present at well P21 is similar to that at well P4. Valleys are approximately 20 to 30 metres wide and are characterized by gravity anomalies on the order of 0.05 mgal. The locations of both the Eastern and Central Faults are included in this profile.

The second negative spike within the second valley (identified with an arrow) is associated with a large culvert which passes beneath the adjacent road. A detailed map showing the location of water mains, sewers and storm sewers, provided by the University, was used to identify any features which may be associated with variations in gravity along a line.

Residence-Gym Profile B-B'

This profile shows little variation in the first 20 metres. From 20 to 40 metres, there is a steep, 0.05 mgal, rise followed by a more gradual increase between 40 metres to the end of the line, with a small negative peak due to a sewer. The Eastern Fault intersects the profile around the 30 metre mark.

Elba Building-Garage Profile F-F'

This profile was intermediate between the River and the Gym-Residence Profiles. The early part of the graph is dominated by the presence of a tunnel. (The tunnel system connects all of the campus buildings. The tunnels are approximately 3 metres wide by 3 metres high). A 0.02 mgal drop in gravity occurs between 15 and 25 metres. This

coincides with the second valley identified on profile A - A' (Note that the profile directions are reversed). There is a gradual drop in the readings from 35 metres to the end of the profile. This may be the bedrock valley associated with the Central Fault. A culvert is located at 50 metres.

Gym-Maintenance Profile G-G'

There is a well defined valley in this profile, which is of the same orientation as the previous profile. This valley is the same one identified on profiles A - A' and F - F'. The position of production well P22 is shown on this graph. It was drilled at the edge of the valley structure.

These first four profiles delineate, with some certainty, the major bedrock valley which extends from the Rideau River and along University Drive, northward. As it was fairly certain that a valley did extend along this stretch, these profiles confirmed that gravimetry was a viable method. There is indication of another valley, possibly associated with the Central Fault, west of Robertson Hall, although this has not yet been confirmed.

The next two profiles were surveyed in an attempt to determine the orientation of the cross-fault.

Residence-Engineering Profile C-C'

This profile was run along the pedestrian pathway east of Campus Avenue from the Commons building to just past the intersection with Library Road. A large broad valley is indicated beneath this portion of the campus. This valley extends between stations at approximately 15 metres and 100 metres, but has only a 0.035 mgal anomaly associated

with it. The south side of this profile indicates a steep bedrock surface, whereas, the north shows a more gradual rise in bedrock elevation.

Residence-Canal Profile D-D'

The edge of the valley between the Residence and Engineering complexes is reflected by an anomaly of 0.15 mgal. Profiles C-C' and D-D' were used to orient the cross-fault. Well T24 is also shown. This well intersected Lower Lowville Formation at 27 metres and is classified as a Central Block well. It is suspected that the bottom of the valley identified in the gravity profile is located further to the north, near well P5. The cross-fault may intersect this profile at around 10 metres or further to the north.

The next two profiles were carried out in order to try and find the fault which subdivides the Central Block. This is the fault which separates wells P23, P4 and P21 from the wells P1, M1 and T33.

Engineering-P4 Profile H-H'

The profile is rather short and extends from the railway tracks near P4 westward for 45 metres. There is evidence of a bedrock valley between 0 and 20 metres. This may correspond to the Central Fault. The drop in readings at 45m is attributed to another tunnel beneath the ground surface.

Campus Avenue-Commons Building Profile C-I

This profile extended from just east of the Commons Building, on the east side of Campus Avenue, to the front of the Commons Building. The 0.1 mgal anomaly may be the result of a broad valley, perhaps associated with the Eastern Fault.

Gym-Elba Profile E-E'

This final profile extends from north to south on either side of the Elba2 well. A large variation in gravity is observed, with a prominent bedrock high between two steep sided valleys on either side. It should be pointed out that readings on the right hand side of this profile are questionable because the gravity meter was placed on a rather steep surface and was constantly being levelled. The results of this one profile, therefore, are inconclusive, although there is an indication of a bedrock valley on the right side.

The resulting gravity profiles clearly showed variation; however, the depths associated with these features could not be determined unless the actual density contrast was accurately known.

A3.7 Gravity Results - Modelling

In order to approximate the shape and magnitude of the features causing the anomalies, the computer software program *GRAV2D*, version 1.0, written by the Department of Geophysics at McGill University, was applied. The algorithm uses a simple polygon method for generating a two dimensional model and calculating the resulting gravity anomaly. Two of the nine gravity profiles, the River profile and the Gym-Maintenance profile, were modelled using a density contrast of 0.3 g/cm^3 . Results for these two profiles indicated that the anomalies may be caused by changes in the bedrock surface that mimic the observed corrected Bouguer gravity profile. Chi-squared goodness of fit tests gave values of 0.28 and 0.20 for the two models, respectively. Valley depths modelled using a density contrast of 0.3 g/cm^3 ranged from 10 to 20 metres. A sensitivity

analysis was undertaken using a density contrast of 0.2 g/cm^3 . Valley depths, modelled using this value, were 20 percent deeper.

All production wells drilled during Phase 2 are believed to have intersected bedrock valleys. At P21, the depth to the bedrock was 15.5 metres; at P22 it was 21.0 metres; at P23 it was 11.4 metres; at T24 it was 20.7 metres; and at Elba3 it was 18.0 metres.

GRAVITY PROFILES

GRAVITY SURVEY RIVER PROFILE A - A'

GRAVITY SURVEY
RESIDENCE-GYM PROFILE B - B'

GRAVITY SURVEY RESIDENCE-ENGINEERING PROFILE C - C'

GRAVITY SURVEY RESIDENCE-CANAL PROFILE D - D'

GRAVITY SURVEY
GYM-ELBA PROFILE E - E'

GRAVITY SURVEY
ELBA BLDG - GARAGE PROFILE F - F'

GRAVITY SURVEY
GYM - MAINTENANCE PROFILE G - G'

GRAVITY SURVEY
ENGINEERING-P4 PROFILE H' - H

GRAVITY SURVEY
CAMPUS AVE-COMMONS BLDG PROFILE C-I

BEDROCK AND GRAVITY PROFILES

BEDROCK ELEVATION AND BOUGUER GRAVITY RIVER PROFILE A - A'

BEDROCK ELEVATION AND BOUGUER GRAVITY RESIDENCE-GYM PROFILE B' - B

— BEDROCK ELEVATION —+— BOUGUER GRAVITY

BEDROCK ELEVATION AND BOUGUER GRAVITY RESIDENCE-ENGINEERING PROFILE C - C'

BEDROCK ELEVATION AND BOUGUER GRAVITY RESIDENCE-CANAL PROFILE D - D'

BEDROCK ELEVATION AND BOUGUER GRAVITY GYM-ELBA PROFILE E - E'

— BEDROCK ELEVATION —+— BOUGUER GRAVITY

BEDROCK ELEVATION AND BOUGUER GRAVITY ELBA BLDG-GARAGE PROFILE F - F'

BEDROCK ELEVATION AND BOUGUER GRAVITY GYM-MAINTENANCE PROFILE G - G'

BEDROCK ELEVATION AND BOUGUER GRAVITY ENGINEERING-P4 PROFILE H' - H

— BEDROCK ELEVATION —+— BOUGUER GRAVITY

BEDROCK ELEVATION AND BOUGUER GRAVITY CAMPUS AVE-COMMONS BLDG PROFILE C-I

Appendix 4

BOREHOLE TEMPERATURE LOGGING

A4.1 General

Temperature logging of boreholes and the interpretation of these profiles is one area of geothermics which has gained attention in both the geophysical and hydrogeological communities in recent years. In the past, accurate downhole temperature measurements were routinely made in shallow and deep boreholes in an attempt to quantify the geothermal heat flux. An excellent review was provided by Jessop (1990).

The temperature of the water, or any other fluid in a borehole, is an accurate measure of the formation temperature if the borehole and its fluid remain undisturbed for a sufficient period of time, such that an equilibrium is established between the two. The temperature of the ground should vary continuously with depth, and except in unusual instances, should also increase linearly with increasing depth. Although the temperature is a continuous function of depth, the gradient of temperature is discontinuous wherever sudden change in the thermal conductivity occurs. The average geothermal gradient may be calculated by taking the difference in temperature between two points and dividing this value by their separation. Of course, the true gradient will be extremely variable at a small scale (*i.e.*, centimetres to metres) due to small heterogeneities within the formation. On a slightly larger scale (tens of metres to hundreds of metres), distinct changes in the gradient result from variations in the thermal conductivity from one formation to the next.

In the absence of lithologic variations, or other factors which affect the thermal conductivity (*i.e.*, the material is homogeneous), the gradient at depth (below the depth where surface temperature perturbations affect the temperature distribution) will be uniform and positive. If these criteria are strictly met, there are two instances in which

variations in the thermal profile would be anticipated. The first concerns the development of microconvection cells within larger diameter boreholes. These minor perturbations may appear as small-scale fluctuations in the gradient. The second, and undoubtedly the most critical, results from water movement through fractures or permeable formations. In the latter situation, the conductive heat flow regime may be severely altered, to such an extent that a reliable gradient cannot be calculated.

Convection by groundwater in upper crustal rocks has been one of the most difficult obstacles to overcome for researchers trying to measure terrestrial heat flow. As early as 1876, water circulation within boreholes was recognized as having a significant effect on the determination of heat flow (Everett, 1876 as quoted in Jessop, 1990). The purely conductive temperature regime exhibited unusual perturbations, caused either by water circulation within the formation or within the borehole itself, which were the result of free or forced convection.

A borehole creates an effective new path for water. If there was any preferential movement of water before the hole was drilled, the flow pattern will most probably be modified. The flow and pressure patterns in the area will change in response to this new channel, and the temperature of the surrounding rock will adjust to the water temperature over a long period. Temperature patterns in a borehole may be interpreted to reveal even the slightest amount of water movement within a well. Borehole temperature logs may reveal many points at which water enters the borehole, at high hydraulic pressure, through fractures or permeable zones.

Around the 1960's, hydrogeologists and geophysicists began publishing observations on the effect of groundwater movement on the temperature distribution (Bredehoeft and Papadopoulos, 1965). More recent studies have attempted to quantify this relationship and models for coupled groundwater and heat transport in aquifers are now normally used to this end (Smith and Chapman, 1983; Stegena, 1983).

A simple flow system in a borehole would consist of water entering at one point, flowing up or down the hole, and exiting at another point. Ramey (1962) considered both liquid and gas flows for such a system. Ramey (1962) presented equations which could be used to calculate type curves for various flow rates within a borehole. A curve-matching exercise is then employed to determine the rate of vertical flow based on the observed temperature profile and the type profile.

Jessop (1990) showed several temperature-depth plots which exhibit varying degrees of water inflow and outflow. In general, the higher the velocity of the moving water within the borehole, the more isothermal is the temperature profile. Depending on the nature of the driving forces, there may be one or several distinct water entry and exit locations. The direction of water movement will change depending on whether the borehole is situated within a recharge or discharge area. As recharge areas generally make up a larger proportion of the landmass, downward flow is likely more predominant than upward flow. Of course, high and low pressure zones may also exist within either of these environments. In recent years, continuous temperature logging has added to the success in identifying slight offsets in the temperature profiles which are indicative of water entry or

exit points (Mwenifumbo, 1993). These offsets are also manifested as spikes in the temperature gradient profiles.

In any well where water movement is significant, a reliable estimate of the gradient can only be made below the depth of the deepest water entry or exit point. The reader can appreciate the difficulty faced by researchers who attempt to calculate gradients and heat flows in areas where water circulation is dominant. One method, which has been successful at determining gradients in areas where significant vertical movement is anticipated, involves the design of a drilling program which permits temperatures in the borehole to be measured regularly throughout the course of drilling. Specific attention is paid to measuring bottom-hole temperatures, which are thought to represent the true rock temperature to within 10 mK (Lewis and Jessop, 1981).

Various methods have been employed to identify the precise location of fractures. Often, successive temperature logs are acquired, once drilling is complete, in order to identify "passive" fractures intersected by the borehole. These passive fractures accept heat provided by the drilling fluid and dissipate this heat over a longer period of time than unfractured portions of the well. Thus, they appear as anomalies on successive temperature logs, which generally decrease in amplitude and increase in width with time.

If a fracture was permeable before drilling, it will probably continue to cause flow in any borehole that intersects it. Jessop (1990) suggested that such fractures will likely manifest themselves in the post-drilling thermal profiles, which distinctly indicate vertical movement. It is recognized, however, that where such flows are high, it may be impossible to detect any earlier effect. Drury (1989) reviewed the thermal signatures of discrete

fractures and fracture zones and uses thermal data, acquired from boreholes situated in the Lac du Bonnet Batholith in the Canadian Shield, to map fracture systems. The author recognizes that flow within a purely crystalline rock mass is dissimilar to that in carbonate rocks, since water will flow only within fractures in the former.

In addition to fractures, the presence of permeable formations may also significantly alter the observed thermal regime. Carbonates, in particular, have widely variable sizes and shapes of pores, and of the sedimentary-type rocks, these are also more likely to have unconnected voids which do not form continuous pathways for water flow. Vertical variation in lithology is also common since limestones are often interbedded with shale horizons, which can be a few centimetres to several metres thick. On a large scale, the aquifer system, on the whole, can be highly variable both in the horizontal and vertical directions.

Temperature profiles in aquifers may show distinct horizons which have higher hydraulic conductivities. As a result of the wide range in permeability associated with the different horizons, aquifer leakage may be quite significant, particularly if viewed on a large scale. Keys and Brown (1978) described a program consisting of reinjection testing and temperature logging, which was used to map subsurface variations in a visually uniform sand of the Ogallala Formation in Texas. Bredehoeft and Papadopoulos (1965) presented an analytical solution to Stallman's (1960) general equation for steady-state flow of heat and groundwater in a vertical direction. This solution has been used frequently to calculate such factors as aquifer recharge rate. For example, Sorey (1971) and Boyle and Saleem (1979), respectively, described techniques used to measure vertical

movement of groundwater and, thus estimate leakage rates out of and recharge rates into aquifers.

A4.2 Borehole Temperature Logging At Carleton

A4.2.1 The Portable Logging System

The portable logging system was operated by the former Earth Physics Branch of Energy, Mines and Resources Canada. This system was used to measure temperatures in several of the wells at Carleton, including the three shallow wells, RC, Elba1 and Tory, and a few Phase 1 wells (Figure A4.1). These logs are provided at the end of this appendix.

In this system, the thermistor (temperature sensitive resistor) is housed in a brass probe, which is connected by a cable to a portable winch. Changes in resistance are measured using a Wheatstone Bridge. The thermistor is calibrated to an accuracy of 1 mK. After correction for lead resistance, resistive leakage in the cable, and stray potentials in the circuit, the absolute accuracy is about 3 mK. Temperatures, measured this precisely, are inherently sensitive to the slightest perturbation in the borehole. For this reason, temperature is normally the first log carried out when a complete geophysical suite of borehole logs is required. Temperatures are measured at discrete depths and the depth interval for measurement can be modified depending on the vertical resolution required.

A4.2.2 Early Temperature Logs (1985-1988)

The first borehole temperature logs at Carleton were obtained in 1985 for three shallow wells, RC, Elba1 and Tory. Elba1 was re-logged in 1988. Temperatures were

Figure A4.1: Map Showing the Location of Test and Production Wells on Campus

measured every 2 or 3 metres. The 1985 temperature profiles were shown, along with the Observatory Crescent borehole log (BH-0-8 in Figure 2.3), in Figure A1.4. However, because of their shallow depth, very little information can be gained from these logs.

Temperatures were measured in Elba2, following the constant discharge test (August 27, 1986) and again in 1988. In the 1986 log, the profile is near isothermal between 20 and 90 metres, however, several minor temperature shifts are observed. A small rise (approximately 0.2°C), corresponding to the water entry point, is observed at a depth of approximately 90 metres.

The near-isothermal character of the depth profile is due to the artesian pressures in the subsurface aquifers, which cause the water to flow up the borehole. The temperature of the water flowing up the borehole is approximately 9.55°C . This temperature is the same as the minimum temperature recorded during previous temperature logs for the shallow wells on campus. Below 85 metres, temperature increases more rapidly with depth to close to 9.8°C at 120 metres.

The 1988 log is much more smoothed. Above 30 metres, temperatures are dominated by near-surface effects. From 30 to 90 metres, isothermal conditions (9.5°C) exist. Below 90 metres, the temperature increases, to reach 9.7°C at 120 metres.

Well M1 was logged two times prior to its conversion to a multilevel piezometer. The first log was on October 15, 1987, immediately after it was drilled. The high temperature at the top of the borehole reflects the warmer summer air temperatures. Below 10 metres depth, the temperature field is approaching isothermal, although there is

some minor variation $\pm 0.5^{\circ}\text{C}$. Below a depth of 118 metres, the temperature increases by 0.7°C . The gradient below this depth is estimated at approximately $14.8^{\circ}\text{C}/\text{km}$.

The second log was on April 22, 1988. From 25 to 120 metres, the log isothermal. The abrupt change in temperature at 118 metres occurs at the boundary between the Rockcliffe Formation and the Oxford Formation, and corresponds to the depth at which the M1-2 and M1-1 piezometers are sealed from each other.

Temperature logs were also obtained for the production wells, P1, P2, P4 and P5, in 1988. The temperature log for P6 is unavailable. Wells P2 and P5 show a similar isothermal section as Elba2 and M1. Water temperatures are 9.2°C and 9.25°C , respectively. There is a slightly higher gradient in P5. The temperature logs for P1 and P4 are distinctly bow-shaped. This reflects a lower hydraulic gradient up the borehole. This bow-shape is characteristic for wells in Eastern Canada (Lewis, 1975).

A4.2.3 The Continuous Borehole Logging System

This logging system is operated by the Borehole Geophysics Section of the Geological Survey of Canada. It was used to measure temperatures in all production and several test wells on campus (Figure A4.1). A similar system, used for this research, is operated by Terrain Sciences Division of the Geological Survey of Canada.

The temperature probe for this system consists of a 10 cm long tip of thermistor beads with a 1 sec time constant and an overall temperature resolution of 0.1 mK, after wireline telemetry and conversion of the thermistor response at the surface (Mwenifumbo, 1993). The probe is connected to a 1 kilometre cable and to a minicomputer-based acquisition system. Depths are measured with an accuracy of 0.001 metres by an optical

shaft encoder located on the well-head pulley assembly. A high degree of spatial resolution is obtained using logging speeds of 3 or 6 metres per minute with data sampled every 0.015 or 0.03 metres, respectively. The thermistor measures changes in resistance which are converted to true temperatures using a calibration curve particular to the thermistor in use. Deconvolution techniques (discussed by Conaway, 1977) are used to process the data derived from continuous measuring. Temperature gradients are normally calculated using the Savitzky-Golay derivative operator (Madden, 1978).

Mwenifumbo (1993) shows the reproducibility of temperature data recorded during downhole and uphole runs at two wells, one of which was located at Carleton. In general, uphole temperatures are on the order of 0.016°C warmer than the downhole logs at one well. Although, logging in both the downhole and uphole directions is usually carried out using the continuous logging system, the former is more accurate.

A4.2.4 Continuous Temperature Logs (1989-1994)

In 1989, the Borehole Geophysics Section of the Geological Survey of Canada logged all of the Phase 2 production wells, P1, P2, P4, P5 and P6, and the observation well, P3 several times. These logs were provided and discussed by Wilson (1990).

In conjunction with this research program, Terrain Sciences of the Geological Survey of Canada re-logged each of wells P3, RC and Elba2 in the fall of 1993. These wells were logged in order to examine the thermal regime of the aquifer while the groundwater system was operating. Unfortunately, the groundwater system was not operating at this time. The logs are similar to the earlier logs obtained prior to implementation of the system and are shown in Figure 3.6.

In 1994, the Borehole Geophysics Section logged each of the five Phase 2 wells, P21, P22, P23, T24 and Elba3, as well as Elba2. These temperature profiles are included at the end of this appendix. A complete log for P23 could not be obtained because of blockage within the borehole at approximately 70 metres.

Several logs were obtained for well P23. These logs were acquired during and following the constant discharge test at well P21, and are discussed in detail in Chapter 3. The log acquired a few months after the test is similar to the one for P21. The temperatures are relatively high in these wells.

The log for T24 is very similar to that for P1. The temperature reversal is apparent on this log because a strong upward movement of water is not present. The temperature of this well is much lower than P21 and P23 at around 8.75°C.

The logs for P22 are typical of the Eastern Block. The first log was taken immediately following the aquifer test at P23. The temperatures in the well are approximately one degree warmer during the test than a few months later.

In 1995, the Borehole Geophysics Section logged the three Phase 3 wells, T31, T32 and T33. These logs have not been included in this thesis.

BOREHOLE TEMPERATURE LOGS

TEMPERATURE PLOT FOR ELBA1 1988

TEMPERATURE PLOT FOR ELBA2 WELL
AUGUST 27, 1986

TEMPERATURE PLOT FOR ELBA2 WELL 1988

TEMPERATURE PLOT FOR M1
OCT 15, 1987

TEMPERATURE PLOT FOR M1
APRIL 22, 1988

TEMPERATURE PLOT FOR P1 1988

TEMPERATURE PLOT FOR P2 1988

TEMPERATURE PLOT FOR P4 1988

TEMPERATURE PLOT FOR P5 1988

Carleton University Hole P-21 Preliminary Plot

94.05.15.11.44.43

524

Carleton University
Hole P-22
Preliminary Plot

Carleton University Hole P-23 Preliminary Plot

Carleton University
Hole ELBA-3
Preliminary Plot

526

ML-TMP-ELBA3-CA411-TD1-BZY

94.05.16.11.17.18

527

Carleton University Hole T-24 Preliminary Plot

Appendix 5
CHEMISTRY DATA AND PIPER DIAGRAMS

Table A5.1: Chemical Data for 1993 Sampling Program

Well	Depth	Ca	K	Mg	Mn	Fe	Na	Sr	pH
	(m)								
RC	25	43.9	7.45	30.2	<0.05	4.00	222	20.50	7.94
E2	30	33.7	7.75	26.6	<0.05	3.77	226	17.00	7.77
E2	55	34.0	7.67	27.3	<0.05	3.98	228	17.00	7.75
E2	85	34.2	7.88	27.5	<0.05	20.8	227	17.20	7.63
E2	108	35.7	7.65	26.6	<0.05	2.45	229	17.60	7.58
P3	25	37.8	8.18	30.6	0.12	5.89	200	18.40	7.96
P3	55	40.5	8.33	31.0	<0.05	0.55	207	18.50	7.92
P3	109	36.7	8.79	30.1	<0.05	5.06	220	17.40	7.83
P3	150	39.0	8.43	30.6	<0.05	2.50	207	17.60	7.88
E3	drilling	47.7	8.90	31.0	0.01	0.21	235	11.82	7.83
P21	testing	48.1	8.40	31.3	<0.01	0.22	281	16.26	7.93
P22	testing	39.8	8.70	28.7	<0.01	0.10	244	10.10	7.76
P23	testing	46.0	7.70	30.2	<0.01	0.18	275	16.52	7.93

Table A5.1: Cont'd

Well	Depth	Cl	CO3	HCO3	N-NH3	N-NO3	SO4	Alk.	Cond.
	(m)	ppm	ppm	ppm	ppm	ppm	ppm	ppm	uMH/cm
RC	25	318	<1.00	288	0.85	<0.10	103	236	1530
E2	30	332	<1.00	281	0.44	<0.10	99	230	1460
E2	55	306	<1.00	296	0.27	<0.10	93	243	1400
E2	85	309	<1.00	294	0.40	<0.10	93	241	1490
E2	108	319	<1.00	292	0.51	<0.10	93	239	1470
P3	25	288	<1.00	293	0.44	<0.10	86	240	1370
P3	55	309	<1.00	283	0.42	0.15	88	232	1460
P3	109	325	<1.00	278	<0.10	0.18	93	228	1470
P3	150	302	<1.00	288	0.40	<0.10	88	236	1450
E3	drilling	330	<1.00	307	n/a	<0.10	130	252	1651
P21	testing	390	<1.00	309	n/a	<0.10	86	253	1948
P22	testing	323	<1.00	325	n/a	<0.10	113	266	1671
P23	testing	351	<1.00	309	n/a	<0.10	147	253	1822

P22(Constant Discharge Test)

Appendix 6

THE AQUIFER TESTING PROGRAMS AT CARLETON

Table A6.1a: Summary of Results From Aquifer Testing Programs - Step, Constant Discharge and Recovery Tests

Pumping Well	Type of test	Date	Static W.L. (m) (below Top of Casing)	Duration (hours)	Pumping Rate (L/sec)	Water level (m) at end of test (below Top of Casing)
RC	Step	June 24/85	0	.5	2.6	1.15*
				.8	4.28	2.60*
				.66	6.4	4.35*
				.66	8.01	6.2*
				.66	10.5	9.4*
RC	Constant	June 26/85	0	24	9.5	10.65*
RC	Rec	June 27/85	0	24		5.4*
ELBA1	Step	June 25/85	0	.66	2.6	6.5*
				.77	3.7	10.8*
				1.2	5.2	18.0*
				.33	6.4	24.0*
ELBA2	Step	Aug 7,8/86	.25	1	5.45	N/A
				1	10.8	1.24
				.66	15.3	1.69
				1	22.7	2.1
				1	30.3	3.48
				1	37.9	4.95
ELBA2	Constant	Aug 11-15/86	-1.03	72	34.1	10.24
ELBA2	Rec	Aug 11-14/86	-1.03	23		1.85
ELBA2	Constant	Oct 12-14/90	-0.75	48	60.6	16.18
	Rec	Oct 15-16/90	-0.75	24		1.48

Table A6.1: Cont'd

Pumping Well	Type of test	Date	Static W.L. (m) (below Top of Casing)	Duration (hours)	Pumping Rate (L/sec)	Water level (m) at end of test (below Top of Casing)
P1	Step	May 23/88	5.38	1	15	15.67
				1	26.5	28.48
				2	38	51.8
P1	Constant	May 24-25/88	6.90	24	32.2	43.42
P1	Rec	May 26/88	6.90	7.7		4.26
P2	Step	May 16	3.61	1	15.1	4.57
				1	30.3	5.84
				1	45.5	7.6
				1	60.6	9.4
				1	75.8	11.38
				1	90.9	13.87
P2	Constant	May 17-20/88	4.77	72	60.6	17.85
P2	Rec	May 21/88	4.77	22		6.34
P2	Constant	Sept 22/88	5.01	6	60.6	7.13
P4	Step	June 27/88	2.66	1	16.9	14.94
				1	25.8	22.65
				1	32.3	30.00
				1	37.7	36.01
				1	44.6	47.10
				.25	53.4	57.9
P4	Constant	June 28-29/88	2.80	24	41.7	42.24
P4	Rec	June 30/88	2.80	9		4.84

Table A6.1a: Cont'd

Pumping Well	Type of test	Date	Static W.L. (m) (below Top of Casing)	Duration (hours)	Pumping Rate (L/sec)	Water level (m) at end of test (below Top of Casing)
P5	Step	July 19/88	4.42	1	17	6.21
				1	31	9.35
				1	46	13.42
				1	61	18.76
				1	75.6	25.4
P5	Constant	July 20-21/88	4.83	24	61.0	25.15
P5	Rec	July 22/88	4.83	9.5		8.18
P6	Step	Aug 28/88	2.57	1	45.5	12.25
				1	64.7	15.44
				1	75.6	19.59
				1	82.2	22.26
P6	Constant	Aug 29-30/88	6.64	24	61	22.69
P6	Rec	Aug 31/88	6.64	8		10.60
P21	Step	Jan 6/94	FLW	1	37.9	15.57*
				1	45	22.63*
				1	52.5	26.93*
P21	Constant	Jan 7-9/94	FLW	48	52.5	38.42*
P21	Rec	Jan 9-11/94	FLW	24		0.56*
P22	Step	Dec 15/93		1	37.9	~20*
				1	45.5	~21*
				1	52.5	21.69
				1	60	23.89
P22	Constant	Dec 16/93	1.02	11	60	N/A
P22	Constant	Dec 20-22/93	1.02	48	60	31.6
P22	Rec	Dec 22-23/93	1.02	24		2.77

Table A6.1a: Cont'd

Pumping Well	Type of test	Date	Static W.L. (m) (below Top of Casing)	Duration (hours)	Pumping Rate (L/sec)	Water level (m) at end of test (below Top of Casing)
P23	Constant	Jan 14-15/93	0.7	33	60	>27.31
P23	Rec	Jan 15-16/94	0.7	24		1.35

* Indicates Drawdown (in metres) from static water level.

Table A6.1b: Summary of Results From Aquifer Testing Programs - ReInjection (Data From WESA, 1988) (all water levels measured from ground surface)

Pumping Well	Reinject Well	Type of test	Date	Static Pump. Well	Static Inj. Well	Duration (hours)	Pumping Rate (l/sec)	Water level Pump. Well	Water level Inj. Well
P1	P4	Step	June 25/88	4.18	1.59	1	16.9	10.41	
						1	25.8	16.66	
						1	32.6	22.22	
						.06	37.7	36.40	
P4	P1	Step	June 26/88	1.14	3.15	1	16.9	8.46	10 PSI
						1	25.8	11.43	16.66 PSI
						1	32.6	12.99	22.22 PSI
P4	P1	Const.	July 4-7/88	2.75	1.86	22	41.7	18.56	44-50 PSI
P5	P2	Step	July 21/88	6.80		1	16.8	8.68	
						1	30.8	10.5	
						1	45.8	14.8	
						1	60.8	16.03	
						.5	75.5	23.5	
P5	P2	Const.	July 22-23/88	5.68	3.87	35	60.8	14.94	1.68m

Table A6.1b: Cont'd

Pumping Well	Reinject Well	Type of test	Date	Static P.W.	Static I.W.	Duration (hours)	Pumping Rate (l/sec)	Water level P.W.	Water level I.W.
P2	P5	Step	July 26/88	2.52		1	16.8	6.82	
						1	30.8	6.93	
						1	45.8	7.21	5 PSI
						1	60.8	7.96	16 PSI
						1	75.5	8.32	25 PSI
						.08	90.9	6.68	
P2	P5	Const.	July 27-30/88	2.47	2.73	22	60.8	3.79	13 PSI
P2	P1	Const.	July 23,24/88	2.8		21	15.1	5.22	38 PSI
P2	P4	Const.	July 24,25/88	3.48		21	26.5	5.25	40 PSI

A6.1 Assumptions for Ideal Aquifers

The earliest and simplest of the prototype models was developed by Theis (1935).

Theis developed a set of mathematical expressions (based on heat flow theory) to calculate the expected drawdown in an ideal aquifer as a result of pumping. From this model, several others were developed by various researchers which modified the basic Theis equation in order to accommodate aquifers in which not all of the assumptions were met. There are five basic assumptions which apply to many of the conventional methods of analysis. These include (after Kruseman and de Ridder, 1990):

- The aquifer is assumed to be of infinite aerial extent and horizontal.

- The aquifer is assumed to be homogeneous and isotropic with respect to its hydrogeological properties and of constant thickness over the area influenced by the test.
- Prior to the pump test, the potentiometric surface is horizontal over the area influenced by the test.
- The aquifer is pumped at a constant discharge rate, Q .
- The pumped well fully penetrates the aquifer and, thus, receives water from the entire thickness of the aquifer by horizontal flow.

In addition to these five basic assumptions, there are four more which apply particularly to the Theis type model. These include (after, Kruseman and de Ridder, 1990):

- The aquifer is confined.
- Flow to the well is in unsteady state, *i.e.*, the change in drawdown is measurable and the hydraulic gradient changes with time.
- The water is removed from storage and is discharged instantaneously with a resulting decline in head.
- The diameter of the well is very small, *i.e.*, the storage in the well can be neglected.

Of course, in the real world, most of these assumptions will not be met. Nevertheless, the models they support are a good first approximation to what may actually be happening in the aquifer. Because the Theis response is for an ideal aquifer, it is useful

to compare it to the response of the aquifer under investigation in order to identify any departures from it. For example, Theis models assume no wellbore storage. If the diameter of the well is large, wellbore storage may be apparent at early time and the actual data will depart from that predicted by Theis at early time. Alternatively, there may be some leakage into the aquifer from an overlying or underlying formation. In this case, less drawdown than that predicted by Theis will take place in the aquifer, and this will be evident upon comparing the data to the Theis model.

A6.2 Theis Method of Analysis - Theory

The basic observation made by Theis (1935) was that the rate of decline of head, multiplied by the storage coefficient and summed over the area of influence, was equal to the discharge. Because the water must come from a reduction of storage within the aquifer, the head will continue to decline as long as the aquifer is effectively infinite and there is no recharge to the system. Therefore, theoretically, there can be no steady-state. However, the rate of decline in head decreases continuously as the area of influence expands and will eventually be so small that it becomes negligible (*i.e.*, steady state conditions).

The equation which defines the decline in head (drawdown) is given by:

$$s = \frac{Q}{4\pi T} \int_u^\infty \frac{e^{-y}}{y} dy = \frac{Q}{4\pi T} W(u) \quad \text{A6.1}$$

where u is defined as:

$$u = \frac{r^2 S}{4Tt} \quad \text{A6.2}$$

s (m) is the drawdown measured in a piezometer measured at a distance r (m) from the pumping well, Q (m³/sec) is the constant pumping rate, S is the storativity, T is the transmissivity and t is the time (sec) since pumping started. $W(u)$ is called the *Theis well function*. It is a tabulated function that is equivalent to the following expression:

$$W(u) = -0.5772 - \ln u + u - \frac{u^2}{2 * 2!} + \frac{u^3}{3 * 3!} - \frac{u^4}{4 * 4!} + \dots \quad \text{A6.3}$$

Of the variables comprising u , the storativity and transmissivity may be considered constant for a given set of conditions, and the distance and time considered variables. Values of $W(u)$ may be plotted against values of u or $1/u$ in such a way that, for a given time, the plotted curve reveals the exact profile of a cone of depression as a function of the distance, r , from a pumping well; or such that for any fixed distance, r , the plotted curve reveals the exact shape of the drawdown curve as a function of time, t . This type of curve is referred to as a “Theis type curve”.

Equation A6.1 is the solution to the general diffusion equation for water flow, written in polar coordinates. Using Equations A6.1 and A6.2, the drawdown at any point in an ideal aquifer can be calculated for any time due to pumping if the transmissivity and storativity are known for the aquifer. Alternatively, these equations may be used to determine the transmissivity and storativity from a set of aquifer test or pumping data, which includes measurements of drawdown at wells or piezometers at different times. If the aquifer is homogeneous and isotropic, then the contribution to the overall drawdown will be the same over the entire thickness of the aquifer. Therefore, either a piezometer or a well will give the necessary information.

A6.3 Jacob Method of Analysis - Theory

The Jacob Method (Cooper and Jacob, 1946) is a simplification of the Theis method of analysis. Equation A6.3 showed the expanded form of the solution to the well function, $W(u)$. From Equation A6.3 it is apparent that values for u decrease as the time of pumping, t , increases and the distance from the pumping well, r , decreases. Therefore, it follows that for observation wells placed close to the pumping well, and after a sufficiently long pumping time, the terms beyond the $\ln u$ term in Equation A6.3 become so small that they can be neglected. For values of u less than 0.01, the drawdown equation can be represented as (Cooper and Jacob, 1946 as described in Kruseman and de Ridder, 1990):

$$s = \frac{Q}{4\pi T} \left(-0.5772 - \ln \frac{r^2 S}{4Tt} \right) \quad \text{A6.4}$$

The Theis equation reduces to the familiar form of the Jacob equation for unsteady flow in a confined aquifer (Cooper and Jacob, 1946):

$$s = \frac{2.3Q}{4\pi T} \log \frac{2.25Tt_0}{r^2 S} \quad \text{A6.5}$$

Equation A6.5 is that of a straight line when plotted on semi-logarithmic paper (time on the log scale) since Q , T and S are constants. The slope of the line is equivalent to calculating the drawdown over one log cycle. Therefore, the transmissivity can be calculated from:

$$T = \frac{2.3Q}{4\pi m} \quad \text{A6.6}$$

where m is the slope of the line. The storativity can be calculated knowing the x-intercept (t_0) of the expression. When the drawdown, s , is zero, the expression which we are taking the log of must be equal to one (since $\log 1=0$). Storativity is determined from:

$$S = \frac{2.25Tt_0}{r^2} \quad \text{A6.7}$$

Of course, since we are dealing with determining the slopes and intercepts of straight lines, the subjectivity of the analysis is removed. Data can be plotted quickly and a computer spreadsheet (*i.e.*, Lotus, Excel, Quattro Pro) can be used to carry out a linear regression analysis on the data.

It should be noted that the same set of assumptions are valid for Jacob as for Theis with the added exception that values of u must be less than 0.01.

Table A6.2a: Cumulative Drawdown for Calculation of Well Loss Constants

STEP DISCHARGE TESTS WELL LOSS CONSTANTS					
WELL	STEP	PUMPING RATE Q (L/S)	DRAWDOWN FOR TIME STEP (M)	CUMMULATIVE DRAWDOWN (DELTA Sw)	DELTA Sw/Q
P1	1	15.00	10.50	10.50	0.70
	2	26.50	12.00	22.50	0.85
	3	38.00	20.00	42.50	1.12
P2	1	15.10	1.50	1.50	0.10
	2	30.30	1.70	3.20	0.11
	3	45.50	1.60	4.80	0.11
	4	60.60	0.80	5.60	0.09
	5	75.80	1.00	6.60	0.09
P4	6	90.90	3.30	9.90	0.11
	1	32.30	4.00	4.00	0.12
	2	37.70	4.00	8.00	0.21
P5	3	44.60	9.00	17.00	0.38
	1	17.00	2.00	2.00	0.12
	2	31.00	2.50	4.50	0.15
P6	3	46.00	3.00	7.50	0.16
	4	61.00	4.00	11.50	0.19
	5	75.60	6.50	18.00	0.24
	1	45.50	3.00	3.00	0.07
	2	64.70	2.00	5.00	0.08
P21	3	75.60	3.00	8.00	0.11
	4	82.20	1.10	9.10	0.11
	1	37.90	10.00	10.00	0.26
	2	45.00	4.50	14.50	0.32
	3	52.50	2.50	17.00	0.32

**Table A6.2b: Regression Analysis Output for Determining Well Loss Constant from
Slope of Cumulative Drawdown Versus Pumping Rate for Step Discharge Tests**

**REGRESSION ANALYSES FOR WELL LOSS CONSTANT C
C IS EQUIVALENT TO SLOPE OF LINE
IN CUMMULATIVE DRAWDOWN VERSUS PUMPING RATE GRAPH**

P1			P5		
Regression Output:			Regression Output:		
Constant	0.407065		Constant	0.08145	
Std Err of Y Est	0.049115		Std Err of Y Est	0.010116	
R Squared	0.973182		R Squared	0.963387	
No. of Observations	3		No. of Observations	5	
Degrees of Freedom	1		Degrees of Freedom	3	
X Coefficient(s)	0.018192	ms ² /L ²	X Coefficient(s)	0.001931	ms ² /L ²
Std Err of Coef.	0.00302		Std Err of Coef.	0.000217	
P2			P6		
Regression Output:			Regression Output:		
Constant	0.101891		Constant	0.003429	
Std Err of Y Est	0.009485		Std Err of Y Est	0.008127	
R Squared	0.016993		R Squared	0.906961	
No. of Observations	6		No. of Observations	4	
Degrees of Freedom	4		Degrees of Freedom	2	
X Coefficient(s)	-3.9E-05	ms ² /L ²	X Coefficient(s)	0.001291	ms ² /L ²
Std Err of Coef.	0.00015		Std Err of Coef.	0.000292	
P4			P21		
Regression Output:			Regression Output:		
Constant	-0.56629		Constant	0.119596	
Std Err of Y Est	0.020045		Std Err of Y Est	0.023849	
R Squared	0.988249		R Squared	0.756393	
No. of Observations	3		No. of Observations	3	
Degrees of Freedom	1		Degrees of Freedom	1	
X Coefficient(s)	0.021083	ms ² /L ²	X Coefficient(s)	0.00407	ms ² /L ²
Std Err of Coef.	0.002299		Std Err of Coef.	0.00231	

STEP DISCHARGE TEST DATA

STEP DISCHARGE TEST WELL P1

STEP DISCHARGE TEST WELL P2

STEP DISCHARGE TEST WELL P4

STEP DISCHARGE TEST WELL P5

STEP DISCHARGE TEST WELL P6

STEP DISCHARGE TEST WELL P21

STEP DISCHARGE TEST WELL P22 - TEST 1

STEP DISCHARGE TEST WELL P22 - TEST 2

**CONSTANT DISCHARGE TEST DATA
LOG-LOG GRAPHS**

CONSTANT DISCHARGE TEST WELL P1

CONSTANT DISCHARGE TEST WELL P1

CONSTANT DISCHARGE TEST WELL P2

—●—	E1
—●—	P1
—●—	P3
—●—	RC
—●—	TORY
—●—	E2
—●—	P2
—●—	P4
—●—	P5
—●—	P6

CONSTANT DISCHARGE TEST WELL P2

M1	—●—
M2	—●—
M3	—●—
M4	—●—
M5	—●—
M6	—●—
MB	—●—

CONSTANT DISCHARGE TEST WELL P4

CONSTANT DISCHARGE TEST WELL P4

CONSTANT DISCHARGE TEST WELL P5

E1	—•—
E2	—•—
P1	—•—
P3	—•—
RC	—•—
TORY	—•—
P2	—•—
P4	—•—
P5	—•—

CONSTANT DISCHARGE TEST WELL P5

CONSTANT DISCHARGE TEST WELL P6

CONSTANT DISCHARGE TEST WELL P6

CONSTANT DISCHARGE TEST WELL E2

CONSTANT DISCHARGE TEST WELL P21

—●—	P21
—●—	P23
—●—	E1
—●—	E2
—●—	P3
—●—	P6
—●—	RC
—●—	TORY
—●—	P22
—●—	T24

CONSTANT DISCHARGE TEST WELL P22

—●—	P23
—●—	E1
—●—	E2
—●—	M1 WELL
—●—	P3
—●—	P6
—●—	RC
—●—	TORY
—●—	P21
—●—	T24

CONSTANT DISCHARGE TEST WELL P23

**CONSTANT DISCHARGE TEST DATA
SEMI-LOG GRAPHS**

CONSTANT DISCHARGE TEST WELL P1

CONSTANT DISCHARGE TEST WELL P1

CONSTANT DISCHARGE TEST WELL P2

E1	—•—
P1	—•—
P3	—•—
RC	—•—
TORY	—•—
E2	—•—
P2	—•—
P4	—•—
P5	—•—
P6	—•—

CONSTANT DISCHARGE TEST WELL P2

CONSTANT DISCHARGE TEST WELL P4

CONSTANT DISCHARGE TEST WELL P4

CONSTANT DISCHARGE TEST WELL P5

—●—	E1
—●—	E2
—●—	P1
—●—	P3
—●—	RC
—●—	TORY
—●—	P2
—●—	P4
—●—	P5

CONSTANT DISCHARGE TEST WELL P5

CONSTANT DISCHARGE TEST WELL P6

CONSTANT DISCHARGE TEST WELL P6

CONSTANT DISCHARGE TEST WELL E2

CONSTANT DISCHARGE TEST WELL P21

—●—	P21
—●—	P23
—●—	E1
—●—	E2
—●—	P3
—●—	P6
—●—	RC
—●—	TORY
—●—	P22
—●—	T24

CONSTANT DISCHARGE TEST WELL P22

CONSTANT DISCHARGE TEST WELL P23

RECOVERY TEST DATA

RECOVERY P1 OBSERVATION WELL P1

RECOVERY P2 OBSERVATION WELL P2

RECOVERY P4 OBSERVATION WELL P4

RECOVERY P5 OBSERVATION WELL P5

RECOVERY P6 OBSERVATION WELL P6

RECOVERY E2 OBSERVATION WELL E2

RECOVERY P21 OBSERVATION WELL P21

RECOVERY P22 OBSERVATION WELL P22

RECOVERY P23 OBSERVATION WELL P23

Appendix 7

MATHEMATICAL FORMULATION AND COMPUTER IMPLEMENTATION FOR GROUNDWATER FLOW AND HEAT TRANSPORT

A7.1 Selection of the Groundwater Model

Mathematical models of groundwater flow are by necessity simplified descriptions of reality. It is impossible to translate all of the complex processes involved on a microscopic scale into a single set of equations that apply equally well to all situations on a macroscopic scale. This is particularly true of fractured crystalline aquifers where the orientations, widths, and densities of the fractures are spatially variable.

The key problem becomes one of identifying a representative elementary volume (*REV*) of aquifer, which can be regarded as homogeneous, and applying a suitable model to represent flow conditions present at the scale of that volume. If the objective is to model the flow of water at the scale of an individual fracture, then obviously the dimensions of the problem domain are small and the choice of a *REV* is constrained by the dimensions of the fracture. In contrast, if the objective is to determine the effect of falling water levels in a river to the groundwater regime in a regional aquifer, a much larger domain is used and the size of the *REV* increases significantly.

Scale dependence is an area of groundwater research which has been gaining much attention in recent years. The relative scale of the faults to the individual fractures at Carleton, in terms of their influence on groundwater movement, is one example of this type of scale dependency problem and are considered briefly, therefore, in this thesis.

In addition to defining a *REV* of an aquifer, which defines both the properties of the aquifer and the scale of the problem domain, it is necessary to select a model type which is suited to solving the problem. In particular, the mathematical equations differ according to whether the flow is through porous media or fractured material, and

according to whether a steady-state or transient solution is sought. As well, the number of dimensions for the simulation must be identified. If insufficient data are available to define the problem domain in three-dimensions, as well as to calibrate the model, then a two-dimensional model may be more appropriate. For problems involving solute, multi-phase, or heat transport a different set of differential equations must be solved.

A7.2 Modelling Protocol

Anderson and Woessner (1992) presented a simple flow chart (their Figure 1.1) which identifies the steps in a modelling protocol. Similar steps have been identified by the American Society for Testing and Materials (ASTM, 1993). These steps are summarized as follows.

- Establish the purpose of the model. The purpose determines the governing equation to be solved and the method of solution to be employed.
- Develop a conceptual model of the system. Assemble field data to characterize the geology and hydrogeology of the site to provide preliminary estimates of the hydraulic parameters and boundary conditions.
- Select the mathematical model according to the factors mentioned in the previous subsection.
- Select the computer code to be used to solve the equations. Verify the code, if necessary, by comparing a numerical solution generated by the model to a solution generated by an analytical model.
- Design the model by taking the conceptual model and putting it in a form suitable for mathematical modelling. This includes designing the grid (for numerical methods),

selecting time steps (for transient problems), setting boundary and initial conditions and selecting values for aquifer parameters (T and S or K and S_s) and hydrologic stresses (*i.e.*, pumping or reinjection).

- Calibrate the model to insure that it can reproduce field-measured heads and flows. Calibration is usually done by trial-and-error adjustment of the parameters. A sensitivity analysis is also carried out to estimate the effect of uncertainty in the calibrated model.
- Verify the model by using the calibrated model to reproduce a second set of field data.

Only once these steps have been taken can the model be used for predictive purposes. Some researchers question strongly the use of models as tools for making predictions, since a model, by its very nature, is an approximation to a natural system. As such, the models cannot be proven or validated, but only tested and invalidated (Konikow and Bredehoeft, 1992). Further, these researchers suggest that the terms ‘validate’ and ‘verify’ should be abandoned and that more meaningful terms such as ‘history matching’ or ‘model evaluation’ be used in their place. As new data become available, a post-audit may be conducted. The results of this post-audit may require that the conceptual model be refined or parameters modified.

A7.3 Mathematical Formulation for the Finite Element Method

A7.3.1 The Method of Weighted Residuals

In the method of weighted residuals, an approximate solution to the boundary value or initial value problem is substituted into the governing differential equation. Upon

doing so, an error or residual occurs at each node in the problem domain. The method of weighted residuals, then, forces the weighted average of these residuals to equal zero. The approximate solution is of the form:

$$\phi(x, y, z) = \sum_{i=1}^n N_i(x, y, z) \phi_i \quad A7.1$$

where N_i is an interpolation function (one per node), ϕ_i is the unknown value of the field variable at node, i , and n is the number of nodes in the mesh.

If one considers a differential equation of the form:

$$L(\phi(x, y, z)) - F(x, y, z) = 0 \quad A7.2$$

where L is the differential operator, ϕ is the field variable and F is a known function, then, upon substitution of Equation A7.1 into A7.2, the differential equation is no longer satisfied and there is a distinct residual at each node.

$$L(\phi(x, y, z)) - F(x, y, z) = R(x, y, z) \neq 0 \quad A7.3$$

The residual at certain nodes cannot be made to equal zero, otherwise, the residual may become unacceptably large elsewhere in the problem domain. The weighted average of the residuals at the nodes is defined as:

$$\int_{\Omega} W(x, y, z) R(x, y, z) d\Omega = 0 \quad A7.4$$

where $W(x, y, z)$ is a weighting function and Ω represents the element domain. In a two-dimensional problem, Ω is an area; in a three-dimensional problem, Ω is a volume.

Substituting Equation A7.3 into A7.4 gives:

$$\int_{\Omega} W(x, y, z) [L(\phi(x, y, z)) - F(x, y, z)] d\Omega = 0 \quad A7.5$$

which can be evaluated if the approximate solution, ϕ , is defined mathematically in a piecewise fashion over the domain. The value of ϕ , within any element e , $\phi^{(e)}$, was given by Equation A7.1. Thus, for any type of element, the approximate solution can be defined according to an interpolation function. For one-dimensional problems, linear bar elements are commonly used; for two-dimensional problems, linear triangles or linear rectangles are used. The interpolation functions are relatively simple for these simple types of elements and have been derived elsewhere (Segerlind, 1984).

The form of the weighting function W must also be specified. In the Galerkin Method, the weighting function is identical to the interpolation function used to define the approximate solution such that:

$$W_i(x, y, z) = N_i(x, y, z) \quad \text{A7.6}$$

Once the form of the approximate solution and the weighting function are specified, the integral in Equation A7.5 can be evaluated to obtain a system of linear equations of the form:

$$[K]\{\phi\} = \{F\} \quad \text{A7.7}$$

that can be solved for the values of the field variable at each node in the mesh.

As an example, consider the equation for steady-state, saturated groundwater flow. The approximate solution for the hydraulic head within an element e is defined as:

$$h^{(e)} = \sum_{i=1}^n N_i^{(e)} h_i \quad \text{A7.8}$$

The contribution of the element e to the residual at a node i to which the element is joined is defined as:

$$R_i^{(e)} = -\iiint_{V^{(e)}} W_i^{(e)}(x, y, z) \left[\frac{\partial}{\partial x} \left(K_x^{(e)} \frac{\partial h^{(e)}}{\partial x} \right) + \frac{\partial}{\partial y} \left(K_y^{(e)} \frac{\partial h^{(e)}}{\partial y} \right) + \frac{\partial}{\partial z} \left(K_z^{(e)} \frac{\partial h^{(e)}}{\partial z} \right) \right] dx dy dz \quad A7.9$$

where $V^{(e)}$ is the volume of the element. The equation was multiplied by a (-1) for later convenience. Using the Galerkin Method (A7.6) and assuming the values of the hydraulic conductivity in the three-coordinate directions, K_{xx} , K_{yy} and K_{zz} are constant within an element, but can vary from one element to another, the residual at node i from the element e is expressed as:

$$R_i^{(e)} = -\iiint_{V^{(e)}} N_i^{(e)} \left[K_x^{(e)} \frac{\partial^2 h^{(e)}}{\partial x^2} + K_y^{(e)} \frac{\partial^2 h^{(e)}}{\partial y^2} + K_z^{(e)} \frac{\partial^2 h^{(e)}}{\partial z^2} \right] dx dy dz \quad A7.10$$

Since the second derivative of the approximate solution is not defined for most types of elements, Equation A7.10 can be integrated by parts using Green's Theorem and expressed in terms of the first derivative:

$$R_i^{(e)} = -\iiint_{V^{(e)}} N_i^{(e)} \left[K_x^{(e)} \frac{\partial^2 h^{(e)}}{\partial x^2} + K_y^{(e)} \dots + K_z^{(e)} \dots \right] dx dy dz = \iiint_{V^{(e)}} \left[K_x^{(e)} \frac{\partial N_i^{(e)}}{\partial x} \frac{\partial h^{(e)}}{\partial x} + K_y^{(e)} \dots + K_z^{(e)} \dots \right] dx dy dz - \int_{S^{(e)}} N_i^{(e)} \{ (-q) \cdot l_x + (-q) \cdot l_y \} ds \quad A7.11$$

Green's Theorem divides the integral into two parts consisting of a boundary line integral over the surface of the element $S^{(e)}$, and a volume integral. The former integral cancels out over the domain in the absence of a specified boundary flux. In Equation A7.11, q is a flux vector (directed outward), and l_x and l_y are unit vectors in the x- and y-coordinate directions, respectively. The approximate solution in Equation A7.11 is then written in terms of the interpolation function (A7.8) giving:

$$\sum_{e=1}^E \iiint_{V^{(e)}} K_x^{(e)} \left[\left\{ \sum_{j=1}^n h_j^{(e)} \frac{\partial N_j^{(e)}}{\partial x} \right\} \frac{\partial N_i^{(e)}}{\partial x} \right] + K_y^{(e)} \left[\left\{ \sum_{j=1}^n h_j^{(e)} \frac{\partial N_j^{(e)}}{\partial y} \right\} \frac{\partial N_i^{(e)}}{\partial y} \right] + K_z^{(e)} \left[\left\{ \sum_{j=1}^n h_j^{(e)} \frac{\partial N_j^{(e)}}{\partial z} \right\} \frac{\partial N_i^{(e)}}{\partial z} \right] dx dy dz$$

$$+ \int_{S^{(e)}} N_i^{(e)} \cdot (q_n) ds = 0 \quad \text{A7.12}$$

In Equation A7.12, q_n is the projection of the flux vector onto a unit vector, n , that is perpendicular to the surface. Note also that h_j is not a function of x , y or z , it is simply an array of the unknown field variable (*i.e.*, the hydraulic head). Also, Equation A7.12 is summed over all of the elements in the three-dimensional domain to obtain the contribution to the residual from each element. This sum is set equal to zero in the method of weighted residuals. Equation A7.12 can be re-written as:

$$\sum_{j=1}^n h_j^{(e)} \left[\sum_{e=1}^E \iiint_{V^{(e)}} \left\{ K_x^{(e)} \frac{\partial N_j^{(e)}}{\partial x} \frac{\partial N_i^{(e)}}{\partial x} + K_y^{(e)} \frac{\partial N_j^{(e)}}{\partial y} \frac{\partial N_i^{(e)}}{\partial y} + K_z^{(e)} \frac{\partial N_j^{(e)}}{\partial z} \frac{\partial N_i^{(e)}}{\partial z} \right\} dx dy dz \right]$$

$$+ \int_{S^{(e)}} N_i^{(e)} \cdot q_n ds = 0 \quad \text{A7.13}$$

where the expression in the braces is integrated over the volume of the element and is identified as the element conductance matrix, $[K_{ij}^{(e)}]$.

$$[K^{(e)}]_{nxn} = \iiint_{V^{(e)}} \begin{bmatrix} \frac{\partial N_1^{(e)}}{\partial x} & \frac{\partial N_1^{(e)}}{\partial y} & \frac{\partial N_1^{(e)}}{\partial z} \\ \vdots & \vdots & \vdots \\ \frac{\partial N_n^{(e)}}{\partial x} & \frac{\partial N_n^{(e)}}{\partial y} & \frac{\partial N_n^{(e)}}{\partial z} \end{bmatrix}_{nx3} \begin{bmatrix} K_x^{(e)} & 0 & 0 \\ 0 & K_y^{(e)} & 0 \\ 0 & 0 & K_z^{(e)} \end{bmatrix}_{3x3} \begin{bmatrix} \frac{\partial N_1^{(e)}}{\partial x} & \dots & \frac{\partial N_n^{(e)}}{\partial x} \\ \frac{\partial N_1^{(e)}}{\partial y} & \dots & \frac{\partial N_n^{(e)}}{\partial y} \\ \frac{\partial N_1^{(e)}}{\partial z} & \dots & \frac{\partial N_n^{(e)}}{\partial z} \end{bmatrix}_{3xn} dx dy dz$$

A7.14

The sum of the element conductance matrices forms a global conductance matrix, $K_{ijglobal}$.

The integral term is referred to as the flux term at the boundary of the system. It only

applies at the boundary since it cancels out within the domain. Equation A7.14 can be expressed in matrix format as:

$$[K]_{global} \{h\} + \{F\}_{global} = 0 \quad A7.15$$

The element conductance matrices can be determined by evaluating the integral expressions in Equation A7.14. Of course, these element conductance matrices will depend upon the shape of the element. The basis functions, $\partial N_j / \partial x$, $\partial N_i / \partial x$, in each coordinate direction must be evaluated. For triangular elements (two-dimensions), basis functions are often used. The solution to the integral expressions (Segerlind, 1984) for triangular elements used in solving saturated groundwater flow problems is provided at the end of the appendix. Both the element conductance matrix and the capacitance matrix are presented.

The boundary line integral in Equation A7.13 is weighted according to the length, L , of the element between nodes:

$$F_i = \sum_{e=1}^E q_n^{(e)} \frac{L}{2} \quad A7.16$$

The flux, specified along the surface of the element, is divided between the nodes at the corners or edges, depending on the distance between nodes. If a problem domain has a flux boundary, then the flux at each node is specified and included in the general system of equations. Pumping and/or reinjection wells located at nodes also have a specified flux; negative is withdrawal and positive is injection. These nodes are considered specified flux Neuman or Type II boundary nodes.

In some instances, the actual value of the field variable may be known and, thus, specified somewhere in the domain. Initial value problems require at least one constant head or Dirichlet Type I boundary node to be specified. These Type I boundary conditions essentially reduce the vector of unknowns $\{h\}$ by the number of specified values.

For steady-state groundwater flow, the system of linear equations represented by Equation A7.14 must be solved. The global conductance matrix is first assembled from the individual element conductance matrices. Any boundary fluxes (Neuman Type) are represented as a right-hand-side (RHS) vector, which modifies the solution. Dirichlet nodes (known values of the variable) are used to reduce the size of the global system of equations. For steady-state problems, at least one Dirichlet node must be specified. The system of linear equations can then be solved using any one of several methods. The Choleski Method (Cook, 1981), which is a decomposition method, was used to solve the system of linear equations generated in the model. The method is described in slightly more detail in the following section.

In contrast to steady-state models, transient models require information regarding the initial state of the domain. In general, transient models start off with some head distribution which is modified with time until a steady-state is achieved. In the transient model, the steady-state head distribution is often first calculated using a steady-state model. Once established, this field can be modelled under transient conditions.

The transient groundwater equation includes a storage term which changes over time. The integral formulation of the transient groundwater equation is similar to that for the steady-state equation. Substitution of the storage term into Equation A7.9 gives:

$$R_i^{(e)} = -\iiint_{V^{(e)}} N_i^{(e)} \left[K_x^{(e)} \frac{\partial^2 h^{(e)}}{\partial x^2} + K_y^{(e)} \frac{\partial^2 h^{(e)}}{\partial y^2} + K_z^{(e)} \frac{\partial^2 h^{(e)}}{\partial z^2} \right] dx dy dz + \iiint_{V^{(e)}} N_i^{(e)} S_s^{(e)} \frac{\partial h^{(e)}}{\partial t} dx dy dz$$

A7.17

In the second integral, the time derivative of the approximate solution must be defined. This approximation is done with interpolation functions and the value of the time derivative at the nodes. Two types of formulations can be used; the lumped element formulation and the consistent element formulation. The consistent element formulation is used in this formulation.

The consistent element formulation uses the same interpolation functions, N_i , to define the time derivative of the approximate solution for hydraulic head within an element such that the second term in Equation A7.17 becomes:

$$\iiint_{V^{(e)}} N_i^{(e)} S_s^{(e)} \frac{\partial h^{(e)}}{\partial t} dx dy dz = \iiint_{V^{(e)}} N_i^{(e)} S_s^{(e)} \left[N_1^{(e)} \dots N_n^{(e)} \right] \left\{ \begin{array}{c} \frac{\partial h_1^{(e)}}{\partial t} \\ \cdot \\ \frac{\partial h_n^{(e)}}{\partial t} \end{array} \right\} dx dy dz \quad \text{A7.18}$$

The element capacitance matrix $C_{ij}^{(e)}$, is then defined as:

$$[C^{(e)}]_{n \times n} = \iiint_{V^{(e)}} \begin{bmatrix} N_1^{(e)} \\ \cdot \\ N_n^{(e)} \end{bmatrix}_{n \times 1} [S_s^{(e)}]_{1 \times 1} \begin{bmatrix} N_1^{(e)} & \dots & N_n^{(e)} \end{bmatrix}_{1 \times n} dx dy dz \quad \text{A7.19}$$

The global capacitance matrix $[C]$ is obtained by combining the element capacitance matrices for all elements in the mesh in the same way that the global conductance matrix was obtained. If $\{h^0\}$ is used to represent the vector of the time derivative of the field variable in Equation A7.18 and $\{h\}$ is used to represent the field variable, the system of linear equations to be solved can be expressed as:

$$[C]_{global} \{h^0\} + [K]_{global} \{h\} = \{F\}_{global} \quad A7.20$$

Equation A7.20 is a system of ordinary differential equations which can be easily solved using any one of a variety of solution methods for systems of linear equations (Istok, 1989). The solution provides values of h and $\partial h / \partial t$ at each node in the mesh. For solutions in transient time, the general time weighting scheme (of the finite difference method) is used to solve Equation A7.20. This is given as:

$$([C] + \omega \Delta t [K]) \{h\}_{t+\Delta t} = ([C] - (1 - \omega) \Delta t [K]) \{h\}_t + \Delta t ((1 - \omega) \{F\}_t + \omega \{F\}_{t+\Delta t}) \quad A7.21$$

where ω is the time weighting function or relaxation factor. If ω is equal to 1, the weighting scheme uses the implicit finite difference (backward difference) method; if ω is equal to 0, the weighting scheme uses the explicit (forward difference) method; and if ω is equal to 1/2, the Crank-Nicholson method (central difference) is used. The solution procedure begins by specifying the initial values of the field variable $\{h\}$ at time $t=t_0$. The system of linear equations is solved to obtain values at the end of the first time step, $\{h\}_{t+\Delta t}$. These new values become the initial values of $\{h\}$ for the next time step.

A7.4 Computer Implementation

The computer code was written in modular format. Different versions of the main program allow for different types of flow problem (*i.e.*, steady-state or transient). For example, in the transient program (GW2), the main program loops through time increments which are specified in a particular subroutine (Subroutine INITIAL). Each program also permits the use of a variety of element types, although subroutines for three-dimensional elements were excluded from the working program for this research. Bar

elements were retained in order to validate the code against hand-calculated solutions and published results (Istok, 1989). Istok's (1989) programs also made use of a file containing array dimensions, defined common blocks, and global variables, which were shared by the various subroutines. This file was included (using \$INCLUDE) during compilation and eliminated the need to pass long strings of variables into subroutines. It also provided a simple means to change the array dimensions. However, as a result of the numerous changes made to the code, each subroutine had to be modified in accordance with the changes made to the \$INCLUDE file.

The following sections describe the three computer codes which were used to solve the steady-state, GW1, and transient, GW2, groundwater flow equations, and the transient heat transport equation, GW3.

Steady-State Groundwater Program GW1

Figure A7.1 is a flow chart which shows the structure of the steady-state computer code, GW1. The main program begins by reading the dimensions of the domain from the input file. This sets up the dimensions of the matrices which will be used to store variables.

Subroutine NODES reads, in free format, the node number, the node number increment, and the coordinates of the node. The operation of this subroutine is controlled by the domain dimensions. The subroutine has the capability to "generate" the node numbers and coordinates for "missing" nodes in a sequence identified by the node number increment. The node numbers do not have to be entered sequentially and the node number increments can be assigned any positive value. The coordinates for each node are stored in vectors. Subroutine ELEMENT reads the element number, the element type, the element

PROGRAM GW1
STEADY-STATE GROUNDWATER FLOW

Figure A7.1: Flow Chart Showing Structure of the Steady-State Groundwater Flow Program, GW1.

number increment, and the node numbers for each element. As for the nodes, “missing” elements can be generated. The node number type, which is defined arbitrarily, identifies the number of nodes expected by the program. For example, element type 4, for these programs, is a triangular element. Therefore, the program anticipates three node numbers. Node and element numbering and their correct sequencing proved to be one of the most tedious and critical factors. Node numbers contained in the data file had to be entered in a consistent manner (*i.e.*, clockwise or anti-clockwise), otherwise the program would crash. This became increasingly difficult as the size of the mesh increased. It is unfortunate that the new software package *Argus Mesh Maker Pro*, which creates finite element meshes and the input data files, was not available at the early stages of this research.

Subroutine MATERL reads the element number and the material set for each element. Then, the subroutine reads the material set number and its associated properties. For two dimensional steady-state problems, two properties are read; namely K_x and K_y . The material properties for missing elements can also be generated as long as the material set number between the end point elements is assumed to remain constant.

Subroutine BOUND reads the boundary conditions for the problem domain. Dirichlet and Neuman nodes (pumping wells and boundary fluxes) can be specified for individual nodes in the mesh. In two dimensional problems the flux has units of volume. The value of the field variable must be specified for at least one node in the mesh. The number of degrees of freedom are calculated for the problem by subtracting the number of Dirichlet nodes from the total number of nodes in the mesh.

Subroutine ASMBK is the longest and most complex subroutine. First, the element conductance matrices are computed in a set of subroutines, one subroutine for each element type. The subroutine then assembles the global conductance matrix $[K]$, and the specified groundwater flow matrix $\{F\}$. Each of these is modified to account for specified values of the field variable. The global conductance matrix is assembled and modified in vector storage in the array M . The modified, global specified flow matrix is stored in array $\{B\}$. Both arrays are passed to the subroutines DECOMP and SOLVE to solve for the unknown field variables according to $[M]\{X\}=\{B\}$.

The semi-bandwidth for the modified system of equations is computed by adding one to the maximum difference in node numbers for any two nodes within any element. To improve the efficiency of solution, it is always preferable to minimize the semi-bandwidth. This is usually accomplished by numbering the nodes in the mesh in a consecutive manner in a direction which corresponds to the least number of elements. For example, for a mesh containing 7×3 (x- and y-direction) elements, the nodes along the x-axis would be numbered 1,4,7,10,13, 16. The operation of subroutine ASMBK is demonstrated with an example at the end of the appendix.

Subroutine DECOMP performs triangular decomposition on the matrix of known coefficients $[M]$ in vector storage by the Choleski Method. This method makes use of the fact that any square matrix can be expressed as the product of a lower triangular matrix $[L]$ and an upper triangular matrix $[U]$ and by assuming the diagonal elements of $[L]$ and $[U]$ are the same. These matrices overwrite the original matrix. The known coefficient matrix may be either symmetric or non-symmetric, but it must be in vector storage.

Subroutine SOLVE solves the system of linear equations by a process called back substitution to give the values of the field variable at all the nodes. It is beyond the scope of this thesis to describe the matrix manipulations used in the Choleski Method, however, these can be found elsewhere (Istok, 1989; Press *et al.*, 1992)

Transient Groundwater Program GW2

Figure A7.2 is a flow chart which shows the structure of the transient computer code. The transient program calls all of the subroutines discussed above and two additional ones. However, subroutine ASMBK is modified, as described below, to incorporate the process of assembling the global capacitance matrix $[C]$ as well.

The primary difference in structure between the steady-state and transient programs is the respective large and small time steps, and of course, in a transient problem, the initial conditions for the problem. Subroutine INITIAL, which is called immediately after subroutine BOUND, reads the time control parameters and the initial estimates of the field variable over the domain. The relaxation factor, ω , in Equation A7.21, is read first, followed by the time steps and the time intervals. Next, a list of times and the values of the time function for each time is read. The time weighting function is used to specify the proportion of the total flux that acts on the domain at any time. For situations in which the flux increases or decreases over time, such as in a steady increase in pumping rate, the time function is used as a weighting factor. Finally, the initial values of the field variable are read for each node.

The program then loops over the specified time steps. These time steps do not have to be of equal length (*i.e.*, Δt can be different), nor do the number of increments per

PROGRAM GW2 TRANSIENT GROUNDWATER FLOW

Figure A7.2: Flow Chart Showing Structure of the Transient Groundwater Flow Program, GW2.

step have to be the same. For the first time step, the global matrices are assembled in ASMBKC. This subroutine is similar to ASMBK (in GW1); the difference being the addition of the global capacitance matrix $[C]$ into the transient flow equation, $[M]=[C]+\omega\Delta t$. ASMBKC calls subroutines to calculate the element conductance matrices, one for each element type. The global flow matrix is also assembled and later modified. The matrix of known coefficients $[M]$ is then decomposed in DECOMP.

Since the solution to the problem is transient, the values for the field variable have to be retained from the previous time step and used in solving the following time step. This is accomplished in the subroutine RHS, which computes the right-hand-side vector in Equation A7.21. The choice of ω determines the weight of the field variable values from the previous and following time steps. Finally, the set of linear equations are solved in subroutine SOLVE, which is the same as that used in GW1. The program then returns and loops over the next time step. If the size of the time step changes, the global matrices are reassembled in the subroutine ASMBKC. Otherwise, a new RHS vector is calculated immediately and used to solve for that time step. The global conductance and capacitance matrices only need to be re-computed if Δt changes (refer to LHS of Equation A7.21).

Transient Heat Transport Program, GW3

The computer code GW3, used for solving the transient groundwater flow equation, was modified to incorporate the heat transport equations and, thus, solve for the temperature at each node in the mesh in addition to the hydraulic head. Figure A7.3 is a flow chart showing the general structure of the code. The beginning of the program has remained unchanged from the groundwater flow code. The subroutines NODES and

**PROGRAM GW3
TRANSIENT GROUNDWATER FLOW
AND HEAT TRANSPORT**

Figure A7.3a: Flow Chart Showing Structure of the Transient Groundwater Flow and Heat Transport Program, GW3.

Figure A7.3b: Flow Chart Showing Structure of the Transient Groundwater Flow and Heat Transport Program, GW3 (continued).

ELEMENT are only called once during execution. A new subroutine MATERLB was added, which reads the element number and the thermal set number for each element. The subroutine then reads the thermal set number and its associated thermal properties. The rock thermal conductivity in the x and y directions, the longitudinal and transverse dispersivities, the aquifer porosity, and the rock heat capacity are read from the input data file. Subroutine MATERLB is called immediately after subroutine MATERL.

Two additional subroutines, BOUNDB and INITIALB were also added to the code. These two subroutines read the boundary conditions (Dirichlet and Neuman Type) and initial conditions, respectively, for the temperature problem domain. Boundary conditions are discussed in greater detail in the following chapter. Because the model domain is planar two-dimensional, there is no geothermal flux in the z-direction (*i.e.*, there is no temperature gradient in the vertical direction).

For each time step, the hydraulic head is calculated at each node in the mesh. The velocities in the x- and y-coordinate directions are evaluated and used to calculate the dispersion coefficients (D_{ij}) in subroutine DISPERS. These D_{ij} values are calculated using Equations 6.38a, b and c. The element conduction/advection-dispersion ($[D^{(e)}(t)]$) and storage ($[S^{(e)}]$) matrices are computed, and the global matrices are assembled in subroutine ASMBKCH. These matrices are solved in vector storage. Also, the global flux vector $\{F\}$ is assembled. The matrices are decomposed into upper and lower triangular matrices and then solved using the Choleski method, as in the groundwater part of the code. At the end of each time step, the nodal temperatures are written to file. The element matrices for the heat transport are re-calculated and re-assembled for each time step, because the velocity

changes with time. The final temperatures and hydraulic heads for one time step become the initial conditions for the following time step.

A7.5 Element Conductance Matrix for Triangular Elements

The interpolation functions for the linear triangle were derived by Segerlind (1984) and were summarized by Istok (1989). Figure A7.4 shows a triangular element.

Figure A7.4: Linear Triangular Element

In Figure A7.4, the interpolation functions are :

$$N_i^{(e)}(x, y) = \frac{1}{2A^{(e)}} (a_i + b_i x + c_i y) \quad \text{A7.22a}$$

$$N_j^{(e)}(x, y) = \frac{1}{2A^{(e)}} (a_j + b_j x + c_j y) \quad \text{A7.22b}$$

$$N_k^{(e)}(x, y) = \frac{1}{2A^{(e)}} (a_k + b_k x + c_k y) \quad \text{A7.22c}$$

where:

$$\begin{aligned} a_i &= x_j^{(e)} y_k^{(e)} - x_k^{(e)} y_j^{(e)} & a_j &= x_k^{(e)} y_i^{(e)} - x_i^{(e)} y_k^{(e)} & a_k &= x_i^{(e)} y_j^{(e)} - x_j^{(e)} y_i^{(e)} \\ b_i &= y_j^{(e)} - y_k^{(e)} & b_j &= y_k^{(e)} - y_i^{(e)} & b_k &= y_i^{(e)} - y_j^{(e)} \\ c_i &= x_k^{(e)} - x_j^{(e)} & c_j &= x_i^{(e)} - x_k^{(e)} & c_k &= x_j^{(e)} - x_i^{(e)} \end{aligned}$$

A7.23

and the area of the element is computed from the determinant:

$$A^{(e)} = \frac{1}{2} \begin{vmatrix} 1 & x_i^{(e)} & y_i^{(e)} \\ 1 & x_j^{(e)} & y_j^{(e)} \\ 1 & x_k^{(e)} & y_k^{(e)} \end{vmatrix} \quad \text{A7.24}$$

Taking the first derivatives of the interpolation functions and using the integration formula derived by Segerlind (1984):

$$\int_{A^{(e)}} (N_i^{(e)})^a (N_j^{(e)})^b (N_k^{(e)})^c dA = \frac{a!b!c!}{(a+b+c+2)!} 2A^{(e)} \quad \text{A7.25}$$

where a , b and c are integers, the integral expression in Equation 5.14 (Chapter 5) (representing the element conductance matrix) can be evaluated. the final form of the element conductance matrix for a linear triangular element is :

$$\begin{bmatrix} K^{(e)} \end{bmatrix}_{3 \times 3} = \frac{K_x^{(e)}}{4A^{(e)}} \begin{bmatrix} b_i^2 & b_i b_j & b_i b_k \\ b_j b_i & b_j^2 & b_j b_k \\ b_k b_i & b_k b_j & b_k^2 \end{bmatrix} + \frac{K_y^{(e)}}{4A^{(e)}} \begin{bmatrix} c_i^2 & c_i c_j & c_i c_k \\ c_j c_i & c_j^2 & c_j c_k \\ c_k c_i & c_k c_j & c_k^2 \end{bmatrix} \quad \text{A7.26}$$

The element capacitance matrix for a linear triangular element evaluated using the consistent formulation (integral expression in Equation 5.18) is defined as:

$$\begin{bmatrix} C^{(e)} \end{bmatrix}_{3 \times 3} = \frac{S_s^{(e)} A^{(e)}}{12} \begin{bmatrix} 2 & 1 & 1 \\ 1 & 2 & 1 \\ 1 & 1 & 2 \end{bmatrix} \quad \text{A7.27}$$

A7.6 Example Problem for Subroutine ASMBK

Consider a one-dimensional finite element mesh which contains four nodes (NUMNOD=4) and three elements (NUMELM=3) as shown in Figure A7.5.

Figure A7.5: Bar Element Showing Node Numbers (no brackets) and Element Numbers (brackets)

The hydraulic conductivities are the same for each element $K_x^{(1)}=K_y^{(2)}=K_z^{(3)}=0.02$ m/day.

The lengths of each element are: $L^{(1)}=5\text{m}$, $L^{(2)}=6\text{m}$, $L^{(3)}=4\text{m}$.

The value of the hydraulic head is specified at nodes 1 and 4, therefore, the number of Dirichlet nodes (NDN) is 2. The problem is to compute the hydraulic heads at nodes 2 and 3. The number of degrees of freedom for the problem is equal to $\text{NDOF}=\text{NUMNOD}-\text{NDN}=2$ and the semi-bandwidth, $\text{SBW}=1+1=2$. All elements are the same, namely a linear bar, which is characterized by two nodes. The element conductance matrix for each element are as follows:

for element 1

$$E=1 \quad KE = \begin{bmatrix} 0.0040 & -0.0040 \\ -0.0040 & 0.0040 \end{bmatrix} \quad \text{A7.28a}$$

for element 2

$$E=2 \quad KE = \begin{bmatrix} 0.0033 & -0.0033 \\ -0.0033 & 0.0033 \end{bmatrix} \quad \text{A7.28b}$$

for element 3

$$E=3 \quad KE = \begin{bmatrix} 0.0050 & -0.0050 \\ -0.0050 & 0.0050 \end{bmatrix} \quad \text{A7.28c}$$

The global system of equations for this problem is:

$$\begin{bmatrix} 0.0040 & -0.0040 & 0 & 0 \\ -0.0040 & 0.0073 & -0.0033 & 0 \\ 0 & -0.0033 & 0.0083 & -0.0050 \\ 0 & 0 & -0.0050 & 0.0050 \end{bmatrix} \begin{Bmatrix} h_1 = 10 \\ h_2 \\ h_3 \\ h_4 = 0 \end{Bmatrix} = \begin{Bmatrix} 0 \\ 0 \\ 0 \\ 0 \end{Bmatrix} \quad \text{A7.29}$$

which can be modified to give:

$$\begin{bmatrix} 0.0073 & -0.0033 \\ -0.0033 & 0.0083 \end{bmatrix} \begin{Bmatrix} h_2 \\ h_3 \end{Bmatrix} = \begin{Bmatrix} -0.04 \\ 0 \end{Bmatrix} \quad \text{A7.30}$$

Equation A7.26 is stored in three vectors in ASBMK: M, X and B.

$$M = \begin{Bmatrix} 0.0073 \\ -0.0033 \\ 0.0083 \end{Bmatrix} \dots\dots\dots X = \begin{Bmatrix} h_2 \\ h_3 \end{Bmatrix} \dots\dots\dots B = \begin{Bmatrix} -0.04 \\ 0 \end{Bmatrix} \quad \text{A7.31}$$

which are passed to subroutines DECOMP and SOLVE to obtain h_2 and h_3 .

Appendix 8
THERMAL CONDUCTIVITIES

A8.1 The Divided Bar Apparatus

The thermal conductivity of earth materials can be measured in three different ways (Williams and Smith, 1989). They are the Guarded Hot Plate (GHP), the Divided Bar (DB) and the Thermal Probe (TP) methods. Thermal Probe (or Needle Probe) methods employ line sources of heat which are inserted into the sample. The rate of dissipation of heat is measured. The Needle Probe apparatus has been used successfully for measuring thermal conductivities of such materials as ocean sediments. The Divided Bar apparatus, at the Geological Survey of Canada, Natural Resources Canada, was used in this research. Typically, this apparatus is used for measuring thermal conductivities of rock discs cut from drill cores. The methodology has been adapted for measuring thermal conductivities of unconsolidated material.

Consider a uniform solid bar of length L , which is maintained at some temperature T_1 at one end and at another lower temperature, T_2 , at the other end. The flow of heat along the bar depends on the difference between the two end temperatures, the distance between them, the cross-sectional area, A , and the thermal conductivity of the bar. Of course, the flow of heat also depends on the time involved. Equation A8.1 defines the flow of heat, Q , in J s^{-1} .

$$Q = A\kappa \frac{T_1 - T_2}{L} \quad \text{A8.1}$$

The Divided Bar, shown in schematic form in Figure A8.1a, has a comparable experimental arrangement (Birch, 1950). A cylindrical sample is placed between two

Figure A8.1: a) A Cross-Section of the Divided Bar Apparatus. b) A Thermal Conductivity Cell.

uniform cylindrical plates whose thermal conductivities are accurately known. Constant temperatures T_1 and T_2 are established and after a short while a steady state condition (*i.e.*, a constant flow of heat from one side to the other) is attained. The flow of heat, q_s , through the sample is necessarily the same as the flow of heat through the plates, q_p . The temperatures T_3 and T_4 , at the faces of the sample, will assume values such that:

$$q_p = A\kappa_p \frac{T_1 - T_3}{l_p} = A\kappa_s \frac{T_3 - T_4}{l_s} = q_s \quad \text{A8.2}$$

If the thickness of the sample, l_s and the plates l_p are equal, then:

$$\frac{\kappa_s}{\kappa_p} = \frac{T_1 - T_3}{T_3 - T_4} \quad \text{A8.3}$$

Assuming perfect insulation on the sides of the bar, the temperature difference over each section is directly proportional to the thermal resistance of that section. It is not necessary to know the amount of heat flowing along the bar since the method is purely comparative (Jessop, 1970). Of course, perfect insulation of the sides of the bar is impossible to achieve when processing large numbers of samples. These losses are kept to a minimum by keeping the length of the bar small in comparison to its diameter, and also by keeping the thermal resistance within as short a length as possible. Jessop (1970) suggested that good measurements are possible if the thermal resistance of each half-bar or plate is of the same order of magnitude as the samples being measured. To meet these requirements, the divided bar apparatus at the Geological Survey of Canada uses low conductivity glass discs sandwiched between brass discs. This configuration enables thermocouples to be inserted into tiny drill holes in the sides of the brass plates, as shown in Figure A8.1a.

Applying the basic principle that the temperature drop is proportional to the thermal resistance, the basic equation for the divided bar is (after Jessop, 1970):

$$\frac{V_2}{V_1 + V_3} = \frac{R_2 + d_s/K_s}{R_1 + R_3 + d_s/K_s} \quad \text{A8.4}$$

where V_1 , V_2 and V_3 are the temperature drops across zones 1, 2 and 3, respectively, R_1 , R_2 , and R_3 are the unwanted thermal resistances, K_g and K_s are the conductivities of the reference discs and of the sample, d_s is the thickness of the sample, and d_g is the total thickness of the two reference discs. Simplifying:

$$d_g V_2 / (V_1 + V_3) = d_s (K_g / K_s) + R K_g \quad \text{A8.5a}$$

where

$$R = R_2 - (R_1 + R_3) d_s K_g / d_g K_s \quad \text{A8.5b}$$

Measurements are generally made on a single disc of adequate thickness (approximately 10 mm), and the thermal resistance, R , is estimated. The value of R is usually about $100 \mu\text{Km}^2\text{W}^{-1}$ (Jessop, 1970).

The calibration of the divided bar is achieved in exactly the same way as the measurements, except that instead of calculating from a known bar resistance to an unknown sample, the results are calculated from a known standard to an unknown bar resistance. To minimize errors, the calibration standards should be as close as possible to the samples being measured in both conductivity and size. The material normally used is fused quartz.

Typically, the samples measured in the divided bar are rock discs cut from drill core. Unfortunately, drill core is not always available; sometimes only rock chip samples

can be retrieved from drilling efforts. Sass *et al.* (1971) described a method for measuring thermal conductivity from rock fragments. This procedure was employed for this research.

The procedure consists of packing rock fragments uniformly using a spatula into a cell as shown in Figure A8.1b. The sample is first crushed using a mortar and pestle in order to achieve a fine grain size. The optimum cell sample is one that is well packed (*i.e.*, little void space). The sample and cell are weighed dry, saturated with water (to which a small amount of detergent has been added) and weighed again. The difference between the dry and saturated weights determines, ϕ , the volume fraction of water. The cell is placed in the divided bar apparatus and its effective thermal conductivity is determined as if it were a disc of solid rock.

The interpretive model must provide an estimate of the thermal conductivity of the solid rock, K_r , from measurements, K_{cell} and ϕ , for a water-saturated aggregate composed of fragments of rock. The model has two parts; first an estimation of the conductivity of the aggregate inside the cell, and second, an estimation of the solid rock conductivity. The first part of the model considers the thermal resistance of the full cell to be represented by the thermal resistance of the aggregate and the plastic cell in parallel. For the second part, the conductivity of the aggregate is usually represented by the geometric mean of the conductivities of its constituents as in Equation 7.2, where V_1 and V_2 are replaced by ϕ and $(1-\phi)$.

A8.2 Methodology

Representative rock chip samples were selected at different depths corresponding to the different formations for each of wells P1, P2, P3, P4, P5, P6, Elba2, T1 and M1. The wells and depths are given in Table A8.1.

The wells are located in each of the main fault blocks and, thus, intersect all of the rock formations present. The divided bar work was carried out prior to the Phase 2 drilling program, which began in the fall of 1993. Therefore, none of the new wells were included in the analysis. Samples for three wells located at Algonquin College in Nepean, were also included. These wells intersect the Rockcliffe, Oxford, March and Nepean Formations, as well as the Precambrian basement.

The cell samples were prepared according to the procedure described above, with particular care taken in packing the crushed fragments uniformly into the cells. The cells consisted of two brass discs on top and on bottom of a plastic cylinder. The bottom brass disc was glued to the plastic cylinder with epoxy. Each cell had its own unique physical dimensions and these were measured accurately using a vernier calipers. Once the sample was packed into the cell, the top brass plate was held on with paraffin. The most difficult part of carrying out these measurements was sample preparation, since it was imperative to achieve a good contact between the rock fragments and the top and bottom brass plates composing the cell.

Prior to conducting measurements, the divided bar was calibrated using a set of standards (quartz discs). Once calibrated, the samples themselves were then placed in the divided bar. A glycerin-water mixture was spread on the top and bottom plates to insure

good contact with the bar. (The divided bars are in boxed chambers in order to maintain constant ambient temperatures. A constant pressure is also maintained on the bar presses throughout the measurements.) Prior to recording the temperatures, the samples were allowed to stabilize for a period of time to insure steady-state conditions were present. The sample was then removed from the bar, the paraffin was taken off and the sample was weighed. The sample was placed in a drying oven and weighed again after a period of approximately 24 hours.

The entire system, apart from sample preparation and weighing, was computer-automated. The temperature readings were automatically used to calculate the thermal conductivities (Table A8.2) once the sample wet and dry weights were entered into the computer.

Table A8.1: Sample Depths and Corresponding Formations for Each Well

Sample Number and (Well)	Depth in metres (feet)	Formation
100-1-1 (P1)	30.5 (100')	Rockland
100-1-2	30.5 (100')	Rockland
100-1-3	65.5 (200')	Lowville
100-1-4	91.4 (300')	Lowville
100-1-5	135.6 (445')	Rockcliffe
100-2-1 (P2)	30.5 (100')	Cobourg
100-2-2	61.0 (200')	Sherman Fall
100-2-3	91.4 (300')	Hull
100-3-1 (P3)	36.6 (120')	Eastview
100-3-2	61.0 (200')	Cobourg
100-3-3	91.4 (300')	Hull
100-3-4	118.9 (390')	Rockland
100-3-5	144.1 (495')	Leray/Lowville
100-4-1 (P4)	7.6 (25')	Hull
100-4-2	15.2 (50')	Rockland
100-4-3	21.3 (70')	Rockland

Table A8.1: Cont'd

100-4-4	82.3 (270')	Lowville
100-4-5	121.9 (400')	Pamelia & St. Martin
100-5-1 (P5)	47.2 (155')	Cobourg
100-5-2	76.2 (250')	Sherman Fall
100-5-3	106.7 (350')	Rockland
100-6-1 (P6)	76.2 (250')	Sherman Fall
100-6-2	91.4 (300')	Hull
100-6-3	91.4 (300')	Hull
100-6-4	121.9 (400')	Rockland
100-7-1 (E2)	15.2 (50')	Billings
100-7-2	21.3 (70')	Billings
100-7-3	45.7 (150')	Cobourg
100-7-4	76.2 (250')	Sherman Fall
100-7-5	106.7 (350')	Hull
100-7-6	121.9 (400')	Rockland
100-8-1 (T1)	30.5 (100')	Rockland
100-9-1 (M1)	61.0 (200')	Pamelia & St. Martin
100-9-2	91.4 (300')	Rockcliffe
100-9-3	91.4 (300')	Rockcliffe
100-9-4	121.9 (400')	Oxford
200-1-1 (AC1)	91.4 (300')	Nepean
200-1-2	102.1 (335')	Basement
200-1-3	102.1 (335')	Basement
200-2-1 (AC2)	23.8 (78')	Rockcliffe
200-2-2	23.8 (78')	Rockcliffe
200-2-3	76.2 (250')	Oxford
200-2-4	125.9 (413')	March
200-2-5	137.1 (453')	Nepean
200-2-6	147.2 (483')	Nepean
200-3-1 (AC3)	100.6 (330')	Oxford
200-3-2	100.6 (330')	Oxford

Table A8.2a: Thermal Conductivities for Formations Intersected in Each Well at Carleton University

Formation	P1	P2	P3	P4	P5	P6	E2	M1 (&T4)
Billings							1.279 2.298	
Eastview			2.026					
Cobourg		2.084	2.173		2.001		2.213	
Sherman Fall		2.316			2.374	2.107	2.433	
Hull		2.363	2.723	2.562	2.594	2.290 2.359	2.678	
Rockland	1.769 1.831		1.775	1.975 2.069		2.058	1.988	(1.563)
Leray			2.286					
Lowville	1.870 1.493			1.923				
Pamelia & St. Martin				1.586				1.841
Rockcliffe	2.088							2.256 2.045
Oxford								2.363

Table A8.2b: Thermal Conductivities for Formations Intersected in Each Well at Algonquin College

Formation	AC1	AC2	AC3
Rockcliffe		2.310 2.082	
Oxford		2.280	2.878 3.102
March		3.059	
Nepean	3.054	3.517 3.301	
Basement	2.377 1.821		

Appendix 9
TRANSIENT HYDROLOGIC MODEL CALIBRATION GRAPHS

CONSTANT DISCHARGE TEST AT WELL P1 SIMULATED AND ACTUAL DATA

CONSTANT DISCHARGE TEST AT WELL P2 SIMULATED AND ACTUAL DATA

CONSTANT DISCHARGE TEST AT WELL P4 SIMULATED AND ACTUAL DATA

CONSTANT DISCHARGE TEST AT WELL P5 SIMULATED AND ACTUAL DATA

CONSTANT DISCHARGE TEST AT WELL P6 SIMULATED AND ACTUAL DATA

CONSTANT DISCHARGE TEST AT WELL E2 SIMULATED AND ACTUAL DATA

CONSTANT DISCHARGE TEST AT WELL P21 SIMULATED AND ACTUAL DATA