AN ABSTRACT OF THE THESIS OF

Arístides Crisóstomos Petrides Jiménez for the degree of
Doctor of Philosophy in Water Resources Engineering
Presented on June 13, 2012
Title: Managed Artificial Aquifer Recharge and Hydrological Studies in the Walla Walla Basin to Improve River and Aquifer Conditions.

Abstract approved: ______________________________________
Richard H. Cuenca

This research project focuses on the Walla Walla River Basin located on the east side of the states of Oregon and Washington, USA. With the support and collaboration of the Walla Walla Basin Watershed Council, this work embraces four research topics. The first topic includes the feasibility study of artificial aquifer recharge in the Walla Walla Basin. Through development and application of a regional hydrological model, a methodology for evaluating locations of artificial aquifer recharge is presented with a test case. The second research topic evaluates the recharge rates observed from pilot test studies of artificial aquifer recharge. Scale dependence of recharge rates should be considered when excessive induced groundwater mounding forms beneath the infiltrating basins. The third topic utilizes groundwater tracers and simulation models to evaluate the hydraulic connection of springs to infiltrating basins of artificial aquifer recharge. Finally, the fourth topic as a proof of a technique, utilizes distributed temperature sensing technology with a pair of black and white coated fiber optic cables to estimate the effective exposure to solar radiation over the Walla Walla River.

Managed Artificial Aquifer Recharge and Hydrological Studies in the Walla Walla Basin to Improve River and Aquifer Conditions

by
Arístides Crisóstomos Petrides Jiménez

A THESIS

Submitted to

Oregon State University

in partial fulfillment of
the requirements for the
degree of

Doctor of Philosophy

Presented June 13, 2012
Commencement June 2013

Doctor of Philosophy thesis of Arístides Crisóstomos Petrides Jiménez presented on June 13, 2012

Approved:

Major Professor, representing Water Resources Engineering

Director of the Water Resources Graduate Program

Dean of the Graduate School

I understand that my thesis will become part of the permanent collection of Oregon State University libraries. My signature below authorizes release of my thesis to any reader upon request.

Arístides Crisóstomos Petrides Jiménez, Author

ACKNOWLEDGMENTS

This project was supported by the Walla Walla Basin Watershed Council as a part of their Water Management Initiative Program. Principle funding resources include the Oregon Watershed Enhancement Board, Washington Department of Ecology and Bonneville Power Administration.

The author would like to thank the personnel from the Watershed Council whose genuine interest to improve the hydrological conditions in the basin motivates me to work beyond the goals of this research thesis. The watershed council personnel dedicated extra time and effort, gathering the field data used in this research and made important contributions and guidance for the successful development of this project. The personnel from the watershed council include the following people: Brian Wolcott, Robert Bower, Will Lewis, Steve Patten, Wendy Harris, Troy Backer, Rick Henry, Bob Chicken, and Nella Parks.

The author would also like to express his gratitude to the graduate committee members: Yutaka Hagimoto, Lisa Madsen, Larry Enochs, J. Mark Christensen and in particular, my major advisor Richard Cuenca. Their review and guidance stimulate my learning inviting me to always perform my best.

This work is dedicated to my family whose loving support encourages me to venture in life. My wife, son and daughter: Mary, Nicolás and Anna, I love you with all my heart. To my parents and siblings who demonstrate to me that love is not bounded by space or time.

Finally, I would like to thank my friends in Corvallis who make this town a great place to live and my friends in Mexico who treat me as if I have never left my hometown.

CONTRIBUTION OF AUTHORS

This research would like to thank and recognize the individual efforts and contributions for the successful development of this project. The regional hydrological model for the Walla Walla River Basin was a collaborative effort between personnel from the Walla Walla Basin Watershed Council and the department of Biological and Ecological Engineering at Oregon State University. The following list presents the names and their contributions to the project.

- Robert Bower: Regional hydrological model boundary delineation. Co-author for chapter 3 scaling recharge rates of artificial recharge projects.
- Troy Baker: Stream segments and surface water features representation in GIS. Field verification of land uses.
- Rick Henry: Survey pumping from basalts, general coordination and model review. Co-author chapter 3
- Steve Pattern: Database of water level elevations, general information about managing aquifer recharged and field coordination for tracers experiment.
- Will Lewis: Database of surface water gauges.
- Nella Parks: Field work with black and white fiber optic.
- John Selker: Overall hydrological model review and co-author of the chapters 4 and 5.
- Jake Sherberg: Co-author chapter 2, Surface water inflows, rating tables, initial surface water diversions and model validation.
- Richard Cuenca: Overall research review, co-author of chapter 3 and 4
- Ryan Stewart: Co-author chapter 3.
- Co-authors of chapter 5: Huff, J.; Arik, A.; van de Giesen, N.; Kennedy, A. M.; Thomas, C. K

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Chapter 1 - General Introduction

1.1 Summary of projects

This research project focuses on the Walla Walla River Basin located on the east side of the states of Oregon and Washington, USA, and evaluates four major topics: the feasibility of artificial aquifer recharge to restore a depleted aquifer, the scaling effects of using pilot projects to design full-scale recharge projects, analyzing the travel time of groundwater from infiltration basins to wells and springs, and proof of the concept of using distributed temperature sensing technology to estimate effective shade over rivers.

Through the Walla Walla Basin Watershed Council (WWBWC), this project evaluates the feasibility of artificial aquifer recharge for restoration of a depleted unconfined gravel aquifer. Chapter 2 develops a regional hydrological model using the Integrated Water Flow Model (IWFM) code created by the California Department of Water Resources. The model is used to quantify the basin's water resources and evaluate water management scenarios. Model results estimate the regional water budget presented with a degree of uncertainty from simulation runs under a range of possible input values. The budget estimates a negative change in aquifer storage of 9 million m^3 per year. This yearly dropping rate of aquifer water levels is expected to increase by 30% under the water management scenario simulating the lining of all the existing irrigation canals in the model area. To mitigate the undesired effects of reducing aquifer recharge by lining the irrigation canals, a methodology for evaluating potential locations of artificial aquifer recharge is presented with the example of an evaluation of two locations. The methodology presents a transparent and rational approach for the selection of locations of managed artificial aquifer recharge projects.

In Chapter 3, recharge rates evaluated from pilot projects of managed artificial aquifer recharge are extrapolated to design full-scale projects. Field experiments at recharge facilities in the Walla Walla River Basin and a 3-dimensional computer simulation model were used to estimate recharges rates in relation to groundwater mounding and to the expansion in surface area of infiltrating basins. Results show that in the scenarios where the water table mounding does not reach the infiltrating basin floor (thereby maintaining an unsaturated zone between the water table and the infiltrating basin), the recharge rates from pilot tests scale linearly with the surface area expansion. However, where the groundwater mound reaches the bottom of the basin floor (thereby providing a full hydraulic connection between the infiltrating basins and the aquifer), recharge rates from pilot studies should be extrapolated using the perimeter of the infiltrating basin. The explanation for this effect is offered by evaluating the distribution of the water velocities at the infiltrating basin floor and the relationship between aquifer thickness and the radius of the infiltrating basins.

Chapter 4 compares analytical estimates of travel times to computer simulated and observed concentrations of groundwater tracers injected at the infiltrating basin and monitored at observations wells and springs. Bromide was selected as the most reliable tracer. During aquifer recharge operations, the inlet and outlet of the infiltration basin were closed and 30 kg of potassium bromide was injected at a rate calibrated to match the rate of drainage from the infiltrating basins. Seven wells around the infiltrating basins and two springs were sampled during this experiment. Water samples were analyzed for major anions. Travel times were calculated from the time of the tracer injection and the detected peak concentration at monitored locations. Results showed a hydraulic connection from the infiltrating basins to the targeted springs for restoration with high (60 m/day) detected groundwater velocities. Computer simulations were calibrated to show the distribution of groundwater velocities around the infiltrating basins. Lateral visualization of the simulations show that most of the recharged water flows through a thin first aquifer layer above the original water table. Travel detection times at the monitored springs cannot be explained by radial flow theory under the assumption of bulk aquifer transport of the tracer.

Water temperature is an important criterion for restoration of habitat of salmonid species (Webb 2008). Chapter 5 estimates the exposure to solar radiation (effective shade) over the Walla Walla River. The research provides successful proof of concept for the use of fiber optic cables with Distributed Temperature Sensing (DTS) technology. River conservation measurements then can evaluate the potential for riparian vegetation to reduce water river temperature in comparison to the importance of the cool groundwater seepage inflows.

1.2 Background Walla Walla River Basin and needs for study.

This research focuses on the Walla Walla River Basin located in the eastern side of the states of Oregon and Washington (Lat: 46˚00' N Log: 118˚24' W). In the basin, as pointed out by White (1969), water management decisions are not entirely made by engineering-economics relative to the optimization of resources. Instead social institutions are taken to be the instruments of water resource management influenced by the culture of the area and as manipulated by the organization and character of social guides. This research addresses various aspects of the use of hydrological modeling to support decision-making. In particular this thesis work is used to evaluate the impacts of water management by quantification of flows between the interconnections of the different components of the hydrological cycle. Management decisions can then be made with a holistic view of the system’s hydrogeological complexities. Although this research focuses primarily on the Walla Walla Basin, the results and methods could be applied elsewhere.

The Walla Walla River, a tributary of the Columbia River, is a productive spawning habitat for the endangered species of steelhead, bull trout and Columbia River salmon (Wolcott 2002). In 1998, due to low summer flows, the Walla Walla River was listed by the environmental group American Rivers as America's eighteenth most endangered river. To avoid legal prosecution, two irrigation districts in Oregon, the Hudson Bay District Improvement Company and Walla Walla River Irrigation District, and Washington's Gardena Farms Irrigation District agreed to leave 25 cfs ($61x10^3$ m^3/d) of water in the Walla Walla River on the Oregon side and 18 cfs ($44x10^3$ m^3/d) of water on the Washington side of the river (Wolcott 2002). This program, with the discontinuation of gravel mining and the approval of levee construction by Milton-Freewater, OR, allowed for restoration of riparian vegetation and reduction of river temperature.

Conservation measures to restore a portion of flow in the Walla Walla River include the improvement of irrigation systems as well as piping and lining irrigation canals (Bower 2009). Since 2004, the Walla Walla Basin Watershed Council (WWBWC) and Oregon State University have been developing a regional hydrological simulation model using the Integrated Water Flow Model (IWFM) of the California Department of Water Resources (Petrides 2008). The Walla Walla River Basin model was structured to evaluate the impacts of surface-water irrigation conservation practices on aquifer and river processes. Model results showed that lining the irrigation canals with the purpose of water conservation would significantly reduce groundwater recharge, resulting in sharp declines in aquifer levels. The model predicted that these declines would result in cessation of flow in several springs that provide critical summer habitat for endangered salmonid species.

Since 2003, the WWBWC in conjunction with Oregon irrigation districts has run pilot tests of artificial aquifer recharge to restore a depleted unconfined gravel aquifer (Bower 2004). In these conjunctive surface and groundwater management practices, water is recharged to the aquifer during months of "excess" water (non-irrigation season, Nov. through May) and saved through piping and lining during the summer months. As a result of these conservation practices, springs that were dry for almost 25 years have restored flows again (Bower 2009). Chapter 4 of this thesis investigates the hydraulic connection between springs and the pilot study of artificial aquifer recharge at the Walla Walla Facilities. The chapter compares analytical estimates of travel times to computer simulated and observed concentrations of a groundwater tracers injected at the infiltrating basin and monitor at observations wells and springs.

From the results obtained from the first pilot test of aquifer recharge in Oregon, water managers in both Oregon and Washington decided to increase the pilot test program throughout the watershed. In 2006, the Washington Department of Ecology in conjunction with Gardena Irrigation District started two new recharge projects. The first project converted an old gravel pit into two circular basins with a total volume of $600m^3$. The second project tested artificial aquifer recharge using flooded fields ($20,000m^2$). In 2008, the WWBWC constructed infiltration galleries to test the potential of different recharge methods that require smaller recharging areas. Results from the pilot studies were expected to simulate full scale projects. However, field data collected by the watershed council suggest that careful consideration should be taken when interpreting the results from such studies to avoid scale dependence of parameters. Such parameters include recharge flow rates, groundwater mounding and water quality. Pilot test studies converted to full scale artificial recharge projects saw a significant decrease in infiltration rate when the surface area was expanded.

The US Army Corps of Engineers is also evaluating the feasibility of artificial aquifer recharge to be used in combination with their proposed irrigation efficiency program (USACOE 2010). The adaptive management proposed by the Corps of Engineers provides a good initial assessment of the impacts of artificial aquifer recharge. The feasibility study does not present the engineering design calculations for their proposed shallow aquifer recharge. The basic parameters for the design of artificial aquifer recharge projects should include the storage capacity of the aquifer, travel times and direction of the proposed recharge flow. Additionally, the groundwater mound has not been estimated and could be the limiting factor for the possible amount of water recharged. Without the proper analysis, including the effects of groundwater pumping, stream/aquifer interactions and other water boundaries, artificial aquifer recharge could result in adverse effects with the possibility of flooding houses, transport of contaminants and change in the hydraulic gradient which could redirect flow to undesired locations.

The general objective of this research work is to provide water managers and decision makers the basic scientific tools for evaluating water resources projects in the Walla Walla Basin. In particular, this thesis work focuses on the use of simulation and analytical hydrological models to understand the relationships between different components of the hydrologic cycle. Description of the involved physical process of the diverse proposed water conservation alternatives is address in each chapter. Ultimately this thesis work provides a transparent and rational approach to evaluate conservation and habitat restoration programs with the goal to improve the conditions of the aquifer and the Walla Walla River.

Chapter 2 - Feasibility Study of Artificial Aquifer Recharge in the Walla Walla Basin.

Authors: Petrides1*, A.C., J. Scherberg, J. Selker, R. H. Cuenca.

Submitted to:
Walla Walla Watershed council as a technical memorandum (modified version)
June 29, 2010. Available at: http://www.wwbwc.org/
Address: 810 S Main Milton-Freewater OR 97862 USA

Abstract

This research project focuses on the Walla Walla River Basin located on the east side of the states of Oregon and Washington, USA. Through the Walla Walla Basin Watershed Council, this project evaluates the feasibility of artificial aquifer recharge for restoration of a depleted unconfined gravel aquifer. With this purpose, the project first develops a regional hydrological model utilizing the Integrated Water Flow Model (IWFM) code created by California Department of Water Resources. The model is used for quantification of the basin's water resources and evaluation of water management scenarios. Model results estimates the regional water budget presented with a degree of uncertainty from simulation runs under a range of possible input values. The budget estimates a negative change in aquifer storage of 9 million m^3 per year. This yearly dropping rate of aquifer water levels is expected to increase by 30% under the water management scenario simulating the lining of all the existing irrigation canals in the model area. To mitigate the undesired effects of reducing aquifer recharge by lining the irrigation canals, a methodology for evaluating potential locations of artificial aquifer recharge is presented with the example of evaluation of two locations. The methodology presents a transparent and rational approach for the selection of locations of managed artificial aquifer recharge projects.

2.1 Introduction and research objectives

The Walla Walla Basin Watershed Council and Oregon State University developed in 2008 a regional hydrological simulation model utilizing the FORTRAN code Integrated Water Flow Model (IWFM) of the California Department of Water Resources (Petrides 2008). The Walla Walla River Basin model was structured to evaluate the impacts of surface-water irrigation conservation practices on aquifer and river processes. Model results showed that lining the irrigation canals with the purpose of water conservation significantly reduced groundwater recharge, resulting in sharp declines in aquifer levels. The model predicted that these declines would result in cessation of flow in several springs that provide critical summer habitat for endangered salmonid species, as well as reducing potential water supplies for irrigated agriculture.

Since the development of the first previous model, new information has been gathered by the watershed council. This new information includes an expansion of the surface and groundwater monitored network, an analysis of the land use and crop identification grown in the region and a map of the thickness of the aquifer layer performed by interpolation of the geological description found in well logs. This research work as a partial fulfillment of a PhD program describes a geographical expansion of the previous hydrological model with a better definition of the surface water features and land-water demands gathered from the new information collected. The new model is improved by expanding the model domain to real physical boundaries found in the field and, a decrease of the previous node spacing for the discretization of the model area. The surface area is expanded from 70 to 231 km^2 and the node spacing decreased from 330 meters to an average of 100 meters, with an even smaller discretization of 20 meters around the infiltrating basins.

Regional Hydrological Model Objectives

The regional hydrological model was structured with three overall model objectives. First, the model assembles hydrogeological information from different sources within the basin. As information is incorporated into the model, a new level of Quality Assurance and Quality Control (QA& QC) is performed by looking at the interactions between different components of the hydrological cycle. Data are evaluated not only by their quality compared to data of similar sources but also with their integration within the model. One such example is the coupling of surface water discharge at springs and irrigation canals to the groundwater elevation levels monitored at observational wells. The model provides visualization of the spatial variability of the water resources over large areas at a spatial resolution finer than that which can be provided by observation data alone.

Second, the model furthers the understanding of the hydraulics of the system by simulating the land-water demands in the area. Quantifying the water resources of the basin is critical in the analysis of water management decision making. Water budgets with an uncertainty analysis are estimated from the simulation of water flows from all the hydrological components of the basin. As an example, groundwater pumping (previously unknown) is calculated in this project as a component in the groundwater budget.

Third, the model serves as a tool to support decision making by predicting the influence of water management practices. The model simulates the interconnection between different hydrogeological components. Proposed alternatives can be evaluated holistically. The proposed predictive scenarios to be evaluated include: recharge basins for artificial aquifer recharge, lining irrigation canals, and changing the ratio between surface water diversions and groundwater pumping to satisfy the water demands in the area.

2.2 Literature review of previous research in the area

Agricultural settlement came to the Walla Walla Basin in the late 1800's. The primary reason for the settlement was the abundance of springs and water resources. However, since then, disputes over water for agriculture and riparian vegetation have reached the Supreme Court (Mc Broom vs. Thompson 1894). The oldest reports of water resources assessment available today are Piper (1948) and Newcomb(1965). The report by Newcomb (1965) is most comprehensive assessment on the hydrogeological conditions of the basin. There have been several subsequent studies, addressing various aspects of the hydrology of the basin, including two previous models. The first model of the basin was developed by Barker and MacNish (1977). Later, Golder Associates (2007) redefined the model created by Barker and MacNish. This section discusses the limitations of these models and how in light of more recent information from studies by local agencies including: Oregon Water Resources Department(OWRD) US Fisheries and Wildlife (USFW) Washington Department of Ecology Washington Department of Fish and Wildlife (WDFW) US-Geological Survey (USGS), WWBWC and OSU, require a new conceptual model that incorporates our improved understanding of the hydrologic conditions in the area.

The goal of the Barker and MacNish model was to estimate the water budget for the basin based upon water use data and geologic records to create. Their 1973 digital model served as the starting point for numerous projects, summarized by the Pacific Groundwater Group (1995), including Cline and Kandle (1990), Collins (1987), and James et al. (1991). Barker and MacNish (1976) created a set of complementary models: one simulated the gravel aquifer, and the other simulated the underlying basalt aquifer. The 1976 model was calibrated using observed data from 1970. Barker and MacNish identified the need to re-examine this effort, using additional data and expanded model capabilities.

In 2007, Golder and Associates Inc. structured a MODFLOW based model to evaluate Aquifer Storage and Recovery (ASR) strategies proposed by the city of Walla Walla. The Golder and Associates model (2007) redefined the boundaries and distribution of aquifer parameters using new information collected since the Barker and MacNish report. This model provide estimated aquifer capacities for ASR given several pumping scenarios as well as predicted well productivity from the basalt aquifer given those scenarios (Golder and Associates, 2007). One limitation of this model is that the aquifer parameters for the gravel aquifers were calibrated for a single layer aquifer and the hydraulic horizontal conductivity was estimated based on specific capacity from well logs. The gravel aquifer is actually composed of several layers, (Lindsey, 2004) which conduct water at different rates. The presence or absence of these aquifer layers affects groundwater velocity. A single aquifer layer model would not capture the hydrologic conditions in the area since the heterogeneity is poorly defined. Model limitations, changes in water use, and new studies provide motivation to create a new model.

In the Walla Walla Basin, land use and water management practices have changed over time. Water budget estimates are constantly being refined in response to changes in land use, cropping, demographics etc. as well as new studies measuring hydrological parameters. In 1995,a report by the Washington Department of Ecology documented $7.5X10^8$ m^3(513,200 acre-feet per year) of water rights allocated in the watershed. Groundwater withdrawal permits consist of 51% of this total. Domestic, municipal, and industrial applications account for the rest of the water withdrawals. WDOE also stated that water level declines in the basalt aquifer were significant, while the gravel aquifer had been relatively stable for 30-40 years (WDOE, 1995).The area from the previous water budgets were estimated for the gravel and basalt aquifer in Oregon and Washington. Localized variations in land use and geological characteristics make it impossible to convert large scale estimates for a much smaller area. In 2008, the WWBWC-OSU team completed a previous version of the Integrated Water Flow Model (IWFM) hydrologic model. The model area encompassed a sub-section of the basin near Milton-Freewater, Oregon. This effort provided an estimated water budget and analysis the initial results of a shallow aquifer recharge project within the Hudson Bay Development and Improvement Company (HBDIC) irrigation District (Petrides, 2008).

The Washington Department of Ecology has provided new information in assessing hydrologic conditions in the area. They issued a study, which covers a The Walla Walla River, Mill Creek, and the Touchet River. This study concludes that overall, the Walla Walla River upper reaches of the watersheds are losing reaches while the lower reaches are gaining however in the section from Nursery bridge to state line, the river runs low or dry in the summer because of upstream diversion for agriculture (PGG 1995). The Walla Walla Basin Watershed Council in conjunction with several local agencies began in 2002 seasonal studies name “Seepage assessments”. The seepage assessments estimate the flow and diversions from the main stem of the Walla Walla River (WWBWC 2010). These studies concluded that the river’s re-canalization has left losing and gaining segments of streams. Bower (2009) based on a HEC-RAS model results concludes that the cold inflows of groundwater to the Main Walla Walla River help are extremely important for maintaining low water temperatures in the summer months.

In April of 2010, the US army Corp of Engineers completed a feasibility study and environmental impacts assessment of three proposed alternative programs where the primary objective was fish habitat restoration by increasing in-streams flows. The proposed alternative programs considered were: (1) the water exchange program, described in the analysis as follows: *“The water exchange in essence involves pumping water from the Columbia River through a pipeline and delivering to diversion point at GFID HBDIC and WWRID in return the irrigators will leave the Walla Walla River flows in stream for aquatic use. This system is a Bucket for Bucket exchange and only water necessary to meet in stream goal will be taken from the Columbia River”*. (2) Off-channel Storage. This alternative proposes to build a reservoir to store water during periods of high flow. This alternative studies the construction of two dams in Pine Creek near Umapine. (3) Irrigation Efficiency. This alternative program has three components: lining and piping irrigation canals, consolidation of redundant canals and on-farm application efficiency. The study evaluates the current un-lined miles of irrigation canals from 5 irrigation districts to be 86.24 miles from which only 14% is already piped or lined. The calculated seepage losses are estimated by the Corps in their first resonances report of 1997 to be 6,150 AF per year. The study team and irrigation districts wanted a better estimation and in 2004 they hired HDR to estimate the seepage loss from the unlined irrigation canals. On average, the results from the seepage study estimated 5,400AF /year However, as the Corp considered these results as questionable and based on professional judgment estimate 4,000 AF/per year with a range from 2,000 to 5,000 AF.

The feasibility of artificial aquifer recharge was also evaluated by the corps. They concluded based on the pilot test in the area that artificial aquifer recharge by spreading basins is a good alternative to be combined with their three proposed programs. The corp. evaluates the feasibility and impacts of 4 levels of artificial aquifer recharge of in conjunction with their three programs. The 4 levels of artificial aquifer recharge include: Aquifer recharge as a major subsurface reservoir, as a minor reservoir, as groundwater stabilization and recovery tools, and as mitigation of the other three proposed programs. Their final recommendation is that while gaps are addressed for AR to be employed in a larger scale, artificial aquifer recharge should be used as mitigation for the irrigation efficiency program. They estimate that 9,200 AF of shallow aquifer recharge using basin of 1 or 2 acres in size located west of the east Crockett branch of the Little Walla Walla River would have minimal impact and could benefit the groundwater resources. The goal of this proposal is to obtain up to 150 cfs in the lowest flow reach (TUM-a-LUM) during May and June and 50 to 100 cfs from July through September.

From the first modeling effort, the Walla Walla Basin Watershed Council has worked in conjunction with local agencies to collect new hydro-geological information as a part of the Water Management Initiative (WMI). The additional information added to the new model is reported by the watershed council in the following report listed in Table 2.1

Table 2.1 Documents considered from the Water Management Initiative

Document name	WMI Task
Results of the 2009 Shallow Aquifer Recharge Season at the Locher Road Site	(WMI Task 2.1)
Drilling of Additional Observation/Monitoring Wells (WMI Task 3.1-1/3.1-2) Shallow Aquifer Monitoring in the Walla Walla Basin	(WMI Task 3.2)
Aquifer Testing in the Walla Walla Basin	(WMI Task 3.5)
GIS GRID Model Development	(WMI Task 3.6)
Walla Walla Basin Surface Water Monitoring	(WMI Task 4.1)
Surface Water Monitoring in the Historic Springs Final Report	(WMI Task 4.1.b)
Seasonal Seepage Assessments on the Walla Walla River	(WMI Task 4.2)
ET/Climate Data Report	(WMI Task 4.4)
Walla Walla Basin GIS Data Model for the Analysis of Hydrology and Fisheries Data	(WMI Tasks 5.1,5.2, and 5.5)
GPS Survey Report	(WMI Task 5.3)
Assessment of Historic Trends in Land Use and Riparian Conditions in the Walla Walla Basin	WMI Task 5.4

2.3 Criteria for model selection and description of simulation models applied in this research

Water managers and decision makers need a comprehensive assessment of the available water resources in their basins in terms of quantity and the interaction of the hydrologic system components. As expressed by Xu (2004) “since the later part of the 19th century, a great deal of experience has been gained by the applicability of distributed models for assessing available water resources”. There are many simulation models available. The Texas A&M and the U.S. Bureau of Reclamation through their Hydrologic Modeling Inventory (HMI 2010) describe the most popular models from around the globe. Selecting the appropriate model eliminates the need to develop computer code from the ground up, allowing model developers to merely customize existing code. Care must be taken to select the appropriate code since a great deal of effort and expertise is required from both the model user and the model developer. This research is the continuation of the regional model developed by Petrides (2008). However, a re-evaluation of the simulation code was performed since the poor flexibility of the hydrologic model code requires creating an entire new model when the surface area is extended.

Singh (1995) proposes the following model classification for watershed models. (1) Process description: lumped or distributed, deterministic, stochastic or mixed; (2) Scale: the space scale can be small, medium or large watershed. The time scale can be event based, continuous time or large time scale and, (3) Technique of solution: numerical, analog analytical finite difference, finite element, boundary element, boundary fitted coordinate and mixed. The regional hydrological modeling class of Oregon State University taught by Dr. R.H. Cuenca adds to this classification the following variables: (1) Model platform: Unix or Windows; (2) Availability of the source code; (3) Application objective; (4) Ease of use; (5) Hydrological process simulated such as the snow component, explicit groundwater component, explicit ET computation, water quality chemistry, interception component, particle tracking, root water function.

The criterion for selecting a model is by its intent of use and the available data resources. The intent of the Walla Walla model is to be used as a planning and managing tool. The particular emphasis of the model is the groundwater component and its interaction with the basin's surface water features such as irrigation canals, springs and the main Walla Walla River. The basin, through its watershed council, has over 5 years of data monitoring the groundwater at 119 wells and 40 surface water gauges at springs and irrigation canals. The basin, however, doesn't have an estimated groundwater pumping or a detailed description of the irrigation time and method. Another key aspect of the model to be developed requires the quantification of the water demands in the area to facilitate the estimation of water budgets.

Following Singh's criterion to evaluate a model, the Walla Walla model should be representative of the spatially variable hydrological conditions in the area. Therefore, a physically based distributed model is preferred. A physically based distributed model requires that its equations and parameters are physically based in the sense that the parameters can be measured in-situ (Singh 1995). The scale of the model area can be considered medium to large (240 km^2) incorporating spatial heterogeneity into the model (Singh 1995). The scale of the model period should be continuous time, simulating the irrigation season (pumping and diversions). The method of solution should be robust enough to allow a variable grid size and efficient enough to allow a full sensitivity and calibration analysis. The finite element method allows a variable grid size where triangular and quadrilateral elements can be defined.

The Regional IWFM model simulates the interactions between surface and groundwater components. However, the engineering design of infiltrating basins of artificial aquifer recharge requires greater accuracy of the simulated flow through the vadose zone. A vadose zone model could be used to evaluate different engineering designs. Once an engineering design has been chosen, the resulting loading rates into the groundwater can be incorporated into a predictive scenario in the regional model. The IWFM can then evaluate the effects of the recharge project at larger scales and additional boundaries such as; groundwater withdrawals by pumping wells and aquifer recharge from excess agricultural applied water and unlined irrigation canals.

The simulation model used as a watershed planning tool is the Integrated Water Flow Model IWFM developed by California Department of Water Resources. The simulation model of the vadose zone used for the engineering design of artificial aquifer recharge is HYDRUS-2D/3D. This section will review the main characteristics and advantages of these models and consideration of their application in the Walla Walla Basin.

The IWFM model is considered to be a watershed management and planning model. The primary emphasis of IWFM is the simulation of groundwater as a quasi-three-dimensional system. The model allows the interaction between aquifer layers and stream-aquifer interactions by incorporating subroutines that simulate flows in streams, in the root zone, vadose zone, and lakes. An advantage of IWFM over other watershed models is the estimated water demands based on user-specified land use. Water demand can then be used to estimate groundwater pumping and surface water diversions. Model output is the groundwater head (L) at groundwater nodes and flow rates in streams (L^3/T). Post processing allows the estimation of flows through each element face calculating water budgets by the sub-regions presented for aquifer, soils, surface waters and the basin as a control volume.

The IWFM model can be described using the model classification of Singh (1995) and Cuenca (2011) model classification), IWFM can be classified as follows:

1) The process description is a semi-distributed deterministic model. The land use and surface water features vary spatially by sub-region but are kept constant inside each defined sub-region. Aquifer heterogeneity is incorporated as model

anomalies. The model is considered deterministic since the boundary and inputs are user specified based on collected data and don't vary randomly.

2) The model is considered medium size since the model area is over 200 km^2.

3) The technique of solution utilizes the Galerkin finite element method. The model solution is robust and mathematically efficient.

4) The model is coded in Fortran 77. The executable files run on a Windows platform.

5) The source code is freeware and available through the California Department of Water Resources webpage (CDWR). CDWR personnel are the model authors and have provided extensive support and advice for this application.

6) The model application objective is the efficient allocation of the basin's water resources.

7) The ease of use is considered moderate to high, requiring user and modeler specialization expertise. The model doesn't include a user interface. However, the model is geo-referenced to GIS. Model data inputs and outputs are organized and manipulated through GIS software.

(8) The hydrological processes simulated are; an explicit groundwater component. Actual Evapotranspiration, ET calculation is based on soil moistures and potential evaporation rate(Rick Allen's FAO 56). The model incorporates subroutines for water flow through the vadose zone and root water uptake.

Vadose zone model Selection: HYDRUSHYDRUS 2D/3D

The simulation code chosen for simulating the spreading basins for artificial aquifer recharge is the Microsoft windows version of HYDRUS 2D/3D (Simunek 2008). HYDRUS 2D/3D has been in the market since 2006. It is a model upgrade of HYDRUS 1D and HYDRUS 2D, originally developed at the U.S. Salinity Laboratory (Simunek 2008). HYDRUS has the capability of simulating water flow in the unsaturated and saturated zone by solving Richard's equation using the Garlekin finite element numerical method. The software has been externally reviewed by Mc Cray (2007) using the standard rating variables of the Southwest Hydrology Journal. The variables for evaluating a simulation model are: ease of use, graphical user interface, output plotting, documentation, and speed. In this review, the graphical interface was rated as an excellent feature of the model while the computational time and variable speed are rated as poor. Goyal (2009) evaluated HYDRUS 2D to simulate operations of aquifer storage and recovery with deep wells. Goyal (2009) concluded that HYDRUS performed well with model efficiencies of 94.6% and 99.9%. Since the development of HYDRUS, the software has been cited in more than 60 journal articles (Simunek 2009). The results from these articles are a positive indication of HYDRUS 2D/3D's capacity to effectively simulate the artificial recharge project by spreading.

HYDRUS 2D/3D has the following features:

1) the model includes an interactive mesh generator of finite elements and a powerful user interface.
2) the relationship between capillary pressures, head, and water content can be based on van Genuchten (1980), Brooks and Corey (1964), Durner (1994), and Kosugi (1995).
3) the root uptake is incorporated in the flow equation as a sink term with spatial root distribution of Vrugt et al. (2001b).
4) the HYDRUS code includes hysteresis by introducing the model by Scott (1983) and Kool and Parker (1987).
5) the software includes a soil hydraulic properties catalog and a Rosetta lite program for generating soil hydraulic properties.
6) a strong component of HYDRUS2D/3D is the post-processing capabilities. These capabilities included animation of graphics, observation points that can be added at anywhere in the grid, and the option for a cross-sectional view of results such as pressure head water content, or velocities (Simunek 2008).

2.4 Overview of analytical solutions for estimating groundwater mounding from managed artificial aquifer recharge projects.

This section of the research project evaluates the most sited analytical solutions for estimating groundwater mounding from managed artificial aquifer recharge projects. The objective is to select the most appropriate solution to be incorporated in the methodology for evaluating potential locations. After the first number of locations has been screen to few locations, the methodology then develops a 3 dimensional computer simulation model of the vadose zone and compares its results with the appropriate analytical solutions.

To simulate the growth and decay of groundwater mounds, a number of analytical solutions have been developed. Analytical models are important to the engineering analysis since they provide a conceptualization of the problem. Analytical solutions could be used as a fast and cost effective method for evaluating a potential site location. However, these solutions have limitations in their assumptions. Usually, the assumptions include an infinite, isotropic, homogenous aquifer with constant recharge (Bouwer 2002). These solutions differ in their ease of use and in their applicability to different boundary conditions. Some of these solutions require iterations making them difficult to implement while others only require solving simple algebraic equations. Computer models can be complex. They require more information in terms of hydro-geological parameters and user expertise than analytical solutions. Computer models are finite, requiring the definition of boundaries. A computer simulation with site specifics is more desirable than analytical solutions for mound visualization in 3-dimensions, however, the development can be expensive and it can take more time to gather the information required (e.g. Poeter 2005).

Most of the analytical solutions for groundwater mounding solve a linearized form of the Boussinesq equation (Eq. 2.1), which is the governing partial differential equation describing flow of groundwater in two dimensions,

$$T\left(\frac{\partial^2 h}{\partial x^2}+\frac{\partial^2 h}{\partial y^2}\right)+R=S\frac{\partial h}{\partial t}$$ Eq. 2.1

where the variables are defined in figure 2.1 below (Brusaet 2006). For axis asymmetrical basins, round, square or irregular area, the Boussinesq equation is often expressed in radial coordinates as:

$$T\left(\frac{\partial^2 h}{\partial r^2}+\frac{1}{r}\frac{\partial h}{\partial r}\right)+R=S\frac{\partial h}{\partial t}$$ Eq. 2.2

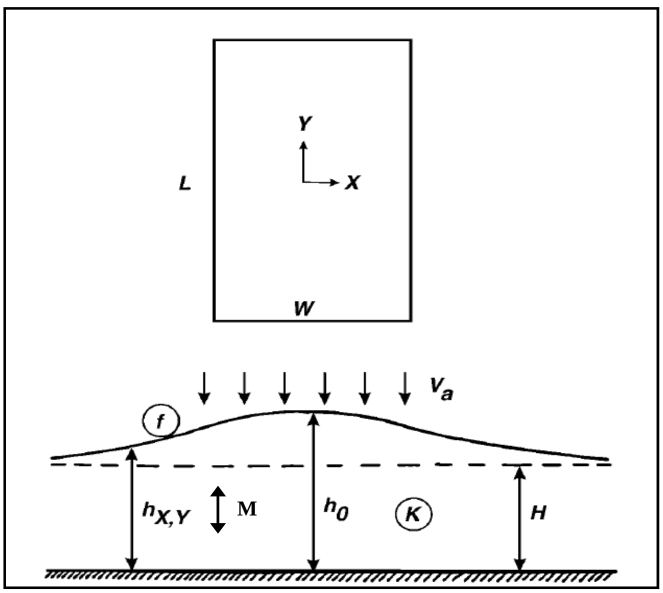

Figure 2.1 Groundwater mound formed from infiltrating basins, modified from Bouwer (2002)

Where:
W= Width of the infiltration basin (L)
L= Length of the infiltration basin (L)
Va = Infiltration rate (L/T)
H = Original saturated thickness (L)
$h_{x,y}$= Height of water table above an impermeable layer (L)
h_o = Maximum height of the water table at the center of the basin (L)
M = Height of the recharge mound above the original water table (L)
K = Hydraulic conductivity (L/T)
f = Fillable porosity (dimensionless)

Variables not in Figure 2.1 but used throughout the rest of this section.

R = Recharge Flow rate (L^3/T)

r_o = Radius of circular basin (L)

r = Radial coordinate with center of recharge basin as origin (L)

T = Transmisivity (L^2 /T)

Ln = Distance between the edge of the infiltration basin and control area (L)

H_n = Height of the water table at the control area (L)

t = Time (T)

D = Distance from the bottom of the infiltration basin to the water table (L)

Baumann (1965) developed a set of simple algebraic equations for circular basins and strip basins recharging at a constant rate. The main assumptions of the solution are a homogeneous, isotropic, horizontally infinite aquifer. The assumptions that distinguish this solution are the geometry of the mound with a nearly flat top and continuous horizontal flow from the edge of the groundwater mound to the water table. The latter assumption causes problems for calculation of the mound decay making this solution a steady state solution. The reason for this solution to be compared in this analysis is that it presents the easiest solution to be applied. Engineers and scientists can employ it readily, but need to be alert to potential errors due to the restrictive assumptions required to obtain the solution.

$$M = -H + \sqrt{H^2 - \frac{v}{\pi K}\left[\ln\left(\frac{r_o}{r}\right) + \frac{1}{r - r_o}\left(re^{\frac{r-r_o}{r}} - 2r + r_o\right)\right]} \qquad \text{Eq. 2.3}$$

The Hantush (1967) solution is probably the most popular solution as it has been compared to every new solution developed thereafter. It linearizes the Boussinesq equation by calculating aquifer transmisivity with the average of the original aquifer thickness and the new calculated aquifer height of the groundwater mound. The linearized form of Boussinesq's equation is then solved by Laplace transformation. The Hantush solution is restricted to the saturated flow ignoring flow in the vadose zone. It assumes a constant infiltration rate to be equal to the recharge rate. Another main assumption is that this solution is only valid when the rise of the groundwater mound does not exceed 50% of the initial saturated thickness. The reason for this solution to be compared in this analysis is that the Hantush (1967) solution presents the standard method for calculating groundwater mound. It is the solution recommended by the American Society of Civil Engineers (ASCE 2002) and it's the only solution available in market software packages developed for aquifer testing. Examples of software that incorporate this solution are: AQTESOLVKAQTESOLVK developed by Hydrosolve Inc. and Groundwatersoftware Inc.

$$h_{x,y,t} - H = \frac{vt}{4f}\left\{ \begin{aligned} & F\left[\left(\tfrac{w}{2}+x\right)n, \left(\tfrac{l}{2}+y\right)n\right] + F\left[\left(\tfrac{w}{2}+x\right)n, \left(\tfrac{l}{2}-y\right)n\right] \\ & + F\left[\left(\tfrac{w}{2}-x\right)n, \left(\tfrac{l}{2}+y\right)n\right] + F\left[\left(\tfrac{w}{2}-x\right)n, \left(\tfrac{l}{2}-y\right)n\right] \end{aligned} \right\}$$

Where $n = \left(\frac{4tT}{f}\right)^{-1/2}$ $\quad T = K\frac{(h_{x,y,t} - H)}{2}$ and

$$F(\alpha, \beta) = \int_0^1 erf\left(\alpha\tau^{-1/2}\right) \cdot erf\left(\beta\tau^{-1/2}\right) d\tau$$

Eq. 2.4

The Morel-Seytoux (1990) solution for circular or infinitely long strips incorporates four new aspects into the groundwater mound solutions: (1) anisotropy of the aquifer hydraulic conductivity, (2) vertical flows at the edge of the infiltrating area, (3) specific yield distinguishing between fillable and drainable porosity and (4) results for constant or time dependent recharge flow rate. The Morel-Seytoux (1990) solution offers the most theoretically rigorous solution. The major limitation as pointed out by Sumner (1999) is the one-dimensionality of the vertical flow, which does not allow for lateral spreading of the infiltrating water above the water table. The reason to incorporate this solution in our analysis is the incorporation of flow through the vadose zone that was not accounted for in the previous solutions. As pointed out by Sumner (1999), an error bigger as 20% is expected when flow through the vadose zone is omitted. This error increases with increasing depth of the water table, anisotropy, and fine texture soils.

$$Kvwt = fwq(t) + 2K\left[\sum_{ver=1}^{t-1} q(ve) + 0.5q(t)\right] + Kfw \cdot \sum_{ver=1}^{t}\left[q(ve) + q(ve-1)\right]\Delta(t - ver + 1)$$

$$q(t) = k\left[h_o - H_x\right]$$

Eq. 2.5

where q(ver) and q(t) are the vertical and horizontal flows at discrete integer values of time.

Manglik (1995) also presents a set of solutions for circular or rectangular infiltrating basins. The solutions solve the Boussinesq's equation for finite aquifers by Fourier transform. These solutions are similar to the Rao and Sarma (1981) solution with the difference that it incorporates a time varying infiltration rate and an exponential decay coefficient of infiltration with mound height. This solution has been chosen to be compared in this analysis since, as pointed out by Griffin (1987) and by Warner (1989), it is a solution that can be easily applied given that it does not require iterative solutions.

$$M = \frac{8vw}{K\pi}\sum_{1}^{\infty}\left(\frac{1}{m}\right)\sin\left[\frac{m\pi w}{2A}\right]\sin\left[\frac{n\pi l}{2B}\right]\sin\left(\frac{m\pi L_x}{A}\right)\left[\sum_{t_i}^{t_{i+1}} R_1 + R_2\right]$$

Eq. 2.6

For constant recharge rate, it reduces to:

$$M = \frac{8vw\left(w/2\right)^2}{K\pi^3}\sum_{1}^{\infty}\left(\frac{1}{m^3}\right)\sin\left[\frac{m\pi w}{2A}\right]\sin\left[\frac{n\pi l}{2B}\right]\sin\left(\frac{m\pi L_x}{A}\right)\left[1 - \exp\left(-\lambda t\right)\right]$$

Eq. 2.7

Bouwer (1999) offers a steady state solution. Steady state can be reached in aquifer recharge when equilibrium exists between recharge rate and pumping or due to a lateral boundary discharging into surface waters like rivers, lakes or springs (Bouwer 1999). This solution is chosen since it can be very useful for establishing control areas. The solution assumes a linear groundwater mound with the heights point at the center of the basin until a lateral boundary has been found.

$$H_c - H_x = \frac{i{r_o}^2}{4T}\left(1 + 2\ln\frac{r}{r_o}\right)$$

Eq. 2.8

Goo (2001) presents an easy to apply equation developed initially for storm water infiltration basins. This solution is based on potential flow theory. It simulates the flow of groundwater through the unsaturated zone in the horizontal and vertical directions. The solution also accounts for the change in storage under the infiltration basin.

$$h_{x,y,t} = H \exp\left(-{kt}/{D}\right)$$

Where

$$k = \left(\frac{v}{S} - K_{ver}\frac{D}{H}\right) \qquad D = r_0 \lambda_y \sqrt{\frac{\lambda_y \ln \lambda_y}{2(\lambda_y^2 - 1)}}$$

And

$$r = \sqrt{\frac{D}{H}} r_o \qquad \lambda_y = \frac{v}{K_y}$$

Eq. 2.9

The Hunt (1971) solution solves the Laplace equation for a three-dimensional mound in radial coordinates. Storage changes are boundary conditions that are solved by the use of dimensionless variables. This solution is chosen in this analysis since it is an easy to apply solution that does not account for the Dupuit assumptions, which can induce error when the distance between the water table and the bottom of the infiltrating pond is significant.

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$$M = \frac{z^*}{R^*}\frac{r_o v}{K}$$

Where $z^* = {H}/{r_o}$ and $R^* = {v}/{K}$

Eq. 2.10

Table 2.2. Analytical Solutions for estimating groundwater mounding from artificial aquifer recharge projects

Solution: Authors name and year	Type of solution	Main assumptions
Baumann (1965)	Steady state	Flat top and continuous horizontal flow from the edge of the groundwater mound to the water table.
Hantush (1967)	Iterative	Ignores flow through the vadose zone
Hunt (1971)	Iterative	does not account for the Dupuit assumptions
Morel-Seytoux (1990)	Iterative	one-dimensionality of the vertical flow
Manglik (1995)	Iterative	exponential decay of infiltration
Bouwer (1999)	Steady state	Known fix hydraulic control
Goo (2001)	Steady state	potential flow theory of symmetric mounds

All the analytical solutions reviewed have a formula version for circular or infinitely long strips. The solutions for circular basins can be used for square or irregular type shape geometry with the substitution of an effective radius (Bouwer 2002). The seven analytical solutions in radial coordinates are solved for the conditions presented in the Walla Walla Basin. The infiltrating basins in the Walla Walla Basin have a rectangular type shape surface geometry that is not perpendicular to the original flow of groundwater. To the contrary of what is desired, the longest side of the infiltrating basins is in the direction of groundwater flow. The groundwater mound created from this basin is certainly not characteristic of those obtained for an infinite long strip. A computer simulation of the groundwater mound contrary to the analytical solution can simulate any geometry of the infiltrating basins with the exact dimensions in relation to the original flow of groundwater.

Ground water mounding from infiltration basins has been simulated in the past using different methodologies. Examples of these simulations include: resistance network analogs (Bouwer 1962), Hele-shaw models (Marino 1967), sand tank models (Rao and Sarma 1980), and computer numerical simulation (Sumner 1999). These models assumed that groundwater mounds can be simulated in two dimensions. The two dimensional representation of the mound is a reasonable approximation only when the surface area of the infiltrating basins represents a surface rectangle with the length of at least 5 times its width, and lays perpendicular to the groundwater flow (Glover 1964 as expressed in Bouwer 2002). Conditions amenable to the two dimensional mound representation are rarely found in practice. Bauman (1965) mentions:

"*Actually few spreading mounds are strictly two or three dimensional. They are likely to be a blend that cannot be directly analyzed mathematically. However, to gain insight into these phenomena, it is necessary to consider each component separately*".

Rao and Sarma (1980) compared the analytical solutions done by Hantush (1967) and Bauman's (1965) to their own experimental results with sand tanks, and concluded that both methods give satisfactory results. The difference of the Hantush and Bauman solutions was within 2% and 5% of their simulation result, respectively. This was challenged by Sumner (1999), claiming that the Hantush solution can have up to 800% of error when he compared the Hantush solution to a computer model, simulating flow through the unsaturated and saturated zone. Also, Warner (1989) compared five analytical solutions in terms of their ease of implementation and in their approximations to a fixed imaginary set problem. The solutions analyzed were: Baumann (1952) Glover (1960) Hantush (1967), Hunt (1971) and Rao (1981). Warner (1989) provides an excellent resource for a starting point of this research. However, a limitation of his analysis is that it only used a set of fixed conditions. This research suggests that different analytical solutions should be used under different boundary scenarios. As an example, the Hunt (1971) solution questions the applicability of the Dupuit assumptions (which underlie the Boussinesq solution) arguing that at the edge of the infiltrating area, the velocity vector is the sum of the percolation rate and the velocity at the free surface. Thus, this solution should be considered when there is a relatively large aquifer thickness (e.g. the thickness of the aquifer much greater than the mound height). As for small aquifer thickness or small depth to water table compared to width or radius of the basin, most of the flow is horizontal and the Dupuit assumption can be well applied. In the latter case, the Hantush solution or any other solutions that utilize the Dupuit assumptions can be applied instead.

This project aims to show one (or more) suitable analytical solutions for evaluation of a potential site for artificial aquifer recharge. In section 2.9 a developed methodology is suggested and results from analytical solutions are compared to simulated and observed results for the Walla Walla Artificial aquifer recharge pilot test at Hudson Bay.

2.5 Regional hydrological model development

Regional hydrological models, also known as watershed models, are an assemblage of different components of the hydrological cycle. They differ from single process models (groundwater models or a surface water models) by simulating the interaction between the hydrological components of the study area. One of the challenges of this type of coupled models is the large volume of data requirements. Data processing can be quite sophisticated and time consuming (Singh 2002). Large volumes of data can be organized by geo-referencing to a Geographical Information System (GIS) model. Under this section, “Model development”, data analysis and its incorporation into the geo-reference IWFM model will be reviewed.

2.5.1 Model area

The area selected for this modeling effort encompasses an area of 231km^2 within the states of Oregon and Washington. The new model area is bounded from the southeast to northeast by the Walla Walla River as it passes through the city of Milton-Freewater, OR to the town of Touchet, WA. From the west, the Horse Heaven Hills create a diagonal model boundary (Figure 2.2). The total Walla Walla River Basin has an approximate area of 4,455 km^2. Of this, the IWFM model only focuses on the gravel aquifer portion of the basin.

The Walla Walla River is a tributary of the Columbia River. The basin is located in the eastern side of the states of Oregon and Washington (46˚00’ N 118˚32’W). River flow in the Walla Walla is driven by melting of the snow pack in the Blue Mountains whereas peak flows are produced by rain flow events. The climate is characterized as a continental climate with cold winters and moderate hot summers. The average winter temperature is 28˚ C (42˚ F) and the average summer temperature is 53˚ C (89˚ F). The annual precipitation is 25 to 38 cm/year. At least 87% of the precipitation falls in the non-irrigation season of Oct-

May. Most of the precipitation falls as rain with very little snow. Winter mean flows for the Walla Walla River are 57 m^3/sec (2,000 cfs) with low summer flows of 9 m^3/sec (300 cfs). During the irrigation season of April through November, water is diverted from the main stem of the Walla Walla into the Little Walla Walla System. This water system supplies about 200 km of irrigation canals. The principle irrigation districts are: Walla Walla River Irrigation District serving 14.5 km^2 (3600 ac); Hudson Bay District serving 27.9 km2 (6900 ac); and Gardena Farms Irrigation District #13 serving 28.3 km2 (7000 ac).

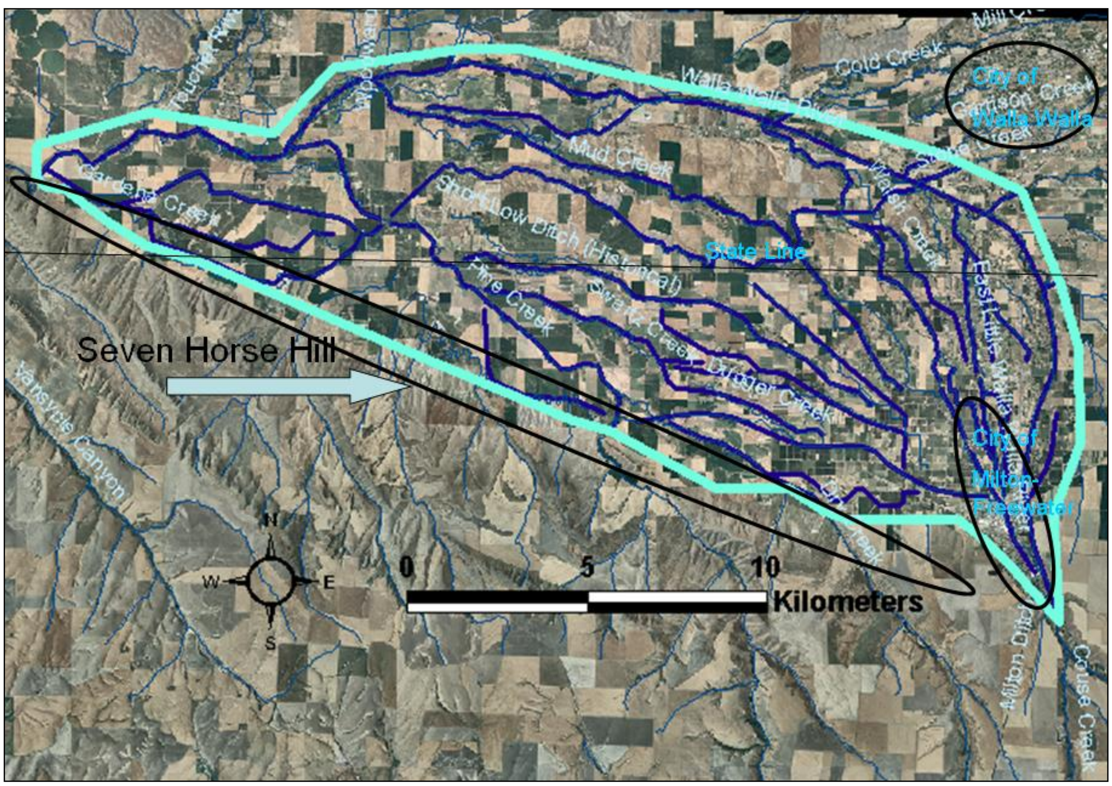


Figure 2.2 Model area output of GIS map. Blue lines represent the major surface water features represented in the model.

2.5.2 Model grid

The IWFM model simulates groundwater flow as a quasi-three-dimensional system utilizing the Garlekin finite element method. The finite element method superimposes nodal points over the model domain. A model element (triangular or quadrilateral) is formed by nodes at its edges. The set of differential equations that govern the flow equation are solved for groundwater head at each nodal point. Hydrological parameters based on field conditions are entered into the model by nodes or element numbers. Spatial scale is then measured as node spacing or element area. A larger spatial scale means that that the watershed response will be less sensitive to spatial variation (Singh 1995). A variable grid size allows us to include higher density of nodes in areas where more information has been gathered and/or for a process of interest. For example, the groundwater mound beneath infiltrating basins for artificial aquifer recharge has a four times smaller gap between nodes than the rest of the model domain (Figure 3.2).

The model grid is geo-referenced to a GIS project representation by an X-Y coordinate system using the projection “North American 1983 UTM zone 11”. The model grid was generated through a GIS based grid-generator named Triangle (Shewchuk 2007). Triangle generates the nodes based on water features and the shape of the model domain. The user specifies the average grid spacing and areas where it is desired to have smaller node spacing. Triangle also numbers the nodes and elements following the finite element rules of IWFM. In total, for a model area of 231 km^2,there are 36,486 elements, 18,520 nodes, and for 220 km of streams there are 2,015 stream nodes for a total of 67 stream reaches. The chosen average distance between nodes is 100 meters. This node spacing was chosen for having at least 3 to 4 nodes per parcel of land (300-400 m^2parcels), allowing water routing options between parcels of land and the numerous irrigation canals. This model achieves a higher resolution compared the previous IWFM model that used a node spacing of 330 m. The small grid size and high number of nodes makes the model flexible for future changes in surface water features and land uses. One disadvantage of small grid size is running/processing time and amount of information required to develop and manipulate the model. The current running time for the complete IWFM sub-routines is 3.5 hrs. with the new solver included in IWFM version 3.2. The model run time in the IWFM version 3.0 is 27 hours.

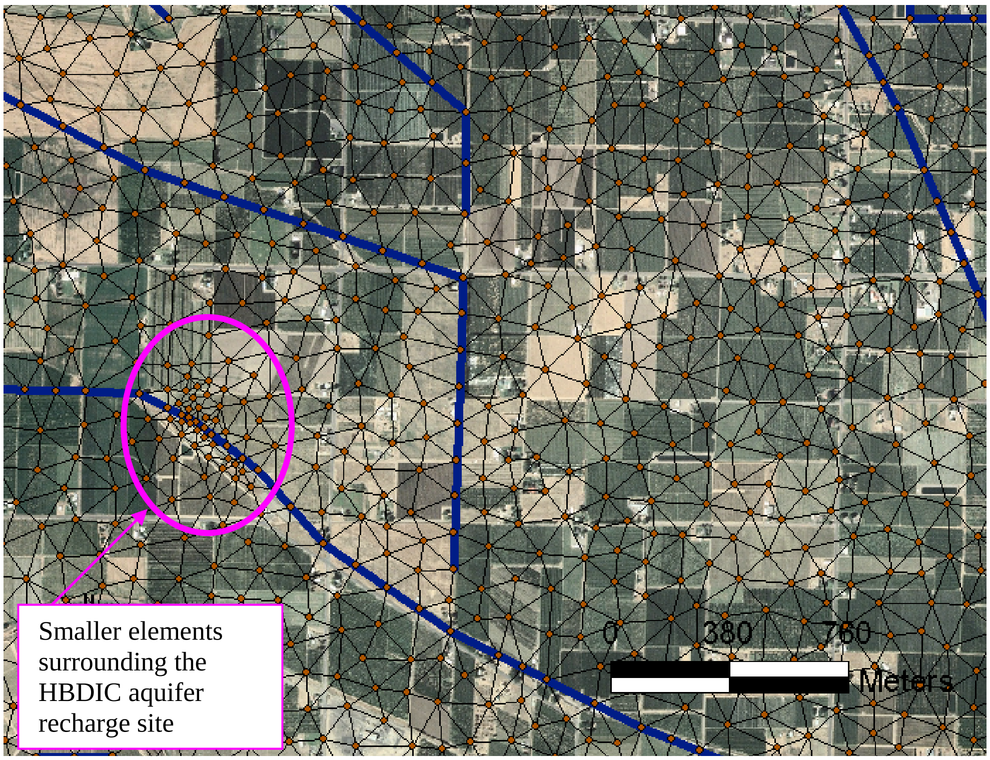


Figure 2.3. Triangular elements of variable areas compose the grid mesh. Note smaller elements around the artificial aquifer recharge projects. Average spacing between nodes is 100m.

2.5.3 Topography and hydro-geological layers

This modeling effort simulates water flow through the gravel aquifer and its interactions with the surface water features of the Walla Walla River Basin. The gravel aquifer is composed of 3 hydro-geological layers. The thickness of these layers is entered per node into the IWFM model. Surface topography also entered per node into the model is gathered from a USGS Digital Elevation Model (DEM). Sources of error from the DEM and the error estimation of the thickness of the geological layer are explored in this section. The section ends by establishing an indirect connection between the gravel aquifer and a deeper basalt aquifer.

The Walla Walla Basin Watershed Council hired GSI Water Solutions to estimate and map the thickness of the geological layers beneath the surface of the basin (Figure 2.4). Based on well log reports, they identified 4 geological layers before meeting the Columbia River Basalt Group (Lindsay 2008). The layers are described order from the surface. (1) Touchet Beds have an average thickness between 0 to 10 m, are formed by Pleistocene loess, felsic silt and felsic to basaltic fine to medium sand.(2) Young alluvial gravel has an average thickness of0 to 20 m. It was formed by Holocene to Pliocene sand and gravel not well constrained. (3) Old gravel with an average thickness of 30 to 60 m formed by Miocene to Pliocene conglomerate sand and gravel with deposits of silt and clay. (4) Blue Clay has an average thickness of 50m; it is formed by Miocene fines clay and silts. The fourth layer Named Blue Clay serves as a confining layer between the Gravel and the Basalt Aquifer. This model only considers the first three layers as part of the gravel aquifer acting as a single unconfined to partially confined aquifer. The water table mostly resides in the Old gravel. However, during recharge season, vertical and horizontal flows are found in the Young gravel and the Touchet beds.

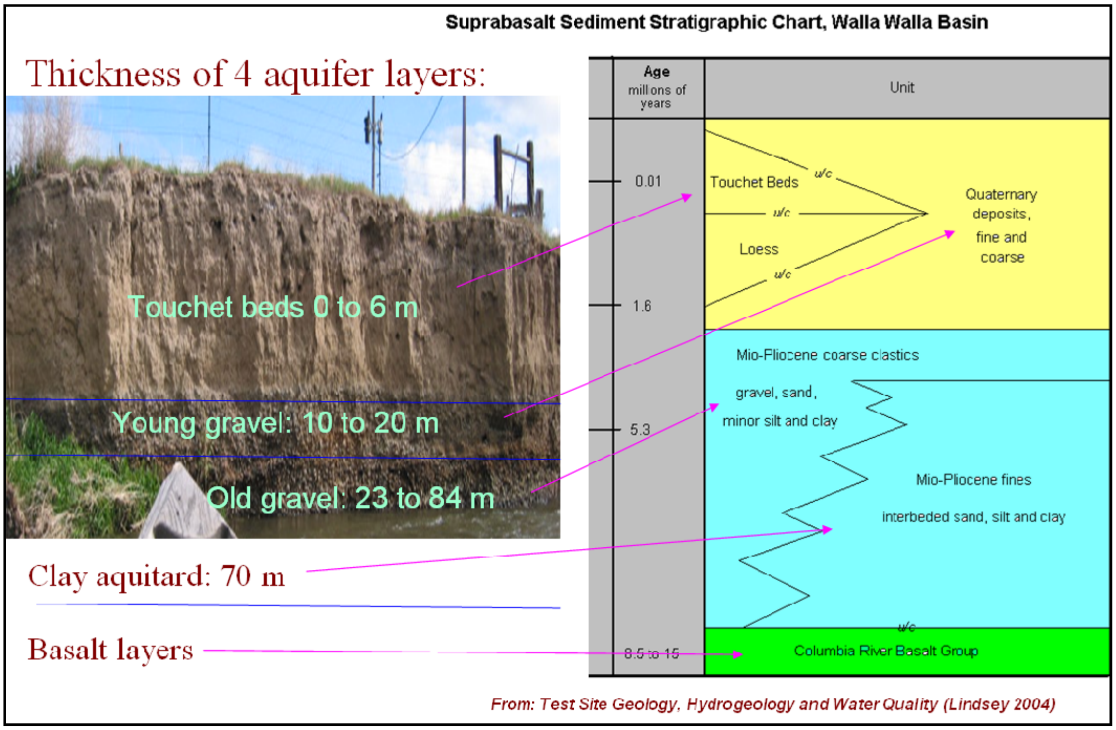


Figure 2.4 Hydro-geological layers in the model area

Mapping the thickness of the supra-basalt layers was done by GSI water solutions (Lindsey 2008) interpolating from the soil descriptions found in well logs. The interpolated maps were developed by Kriging methods in GIS and provided to the WWBWC in GIS layer shape files (Figure 2.3). From personal communication to Lindsey (WWBWC 2010 meeting), the expected error of these maps is estimated to be have a standard deviation of 15 m. The thickness of each layer is then incorporated into the IWFM model as the stratigraphy for each node. Parameters of each aquifer layer include vertical and horizontal hydraulic conductivities, specific storage, and specific yield. These parameters were initially taken from field experiments and reported values in the literature. Field experiments include pumping aquifer tests, tracer test, and seepage analysis in irrigation canals. The calibration procedure later modifies the initial estimates in the context of the model representation of the hydrological features of the basin. Initial and final values are presented in the calibration section of this document.

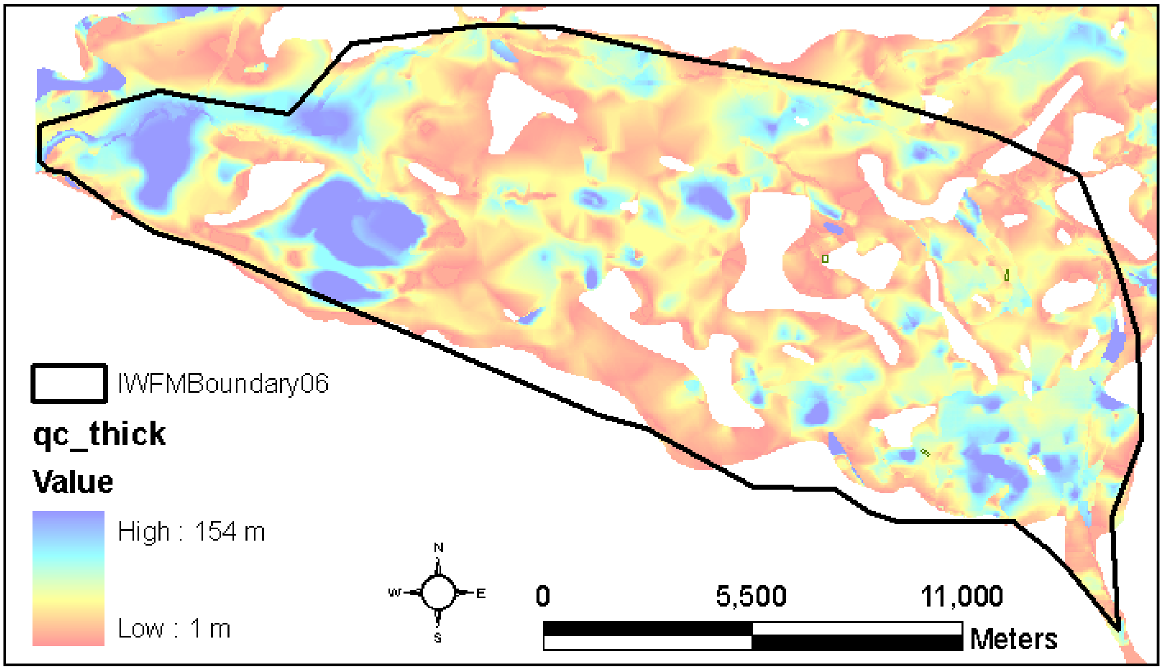


Figure 2.5 GIS layer representing the thickness of the Quaternary un-cemented gravel aquifer

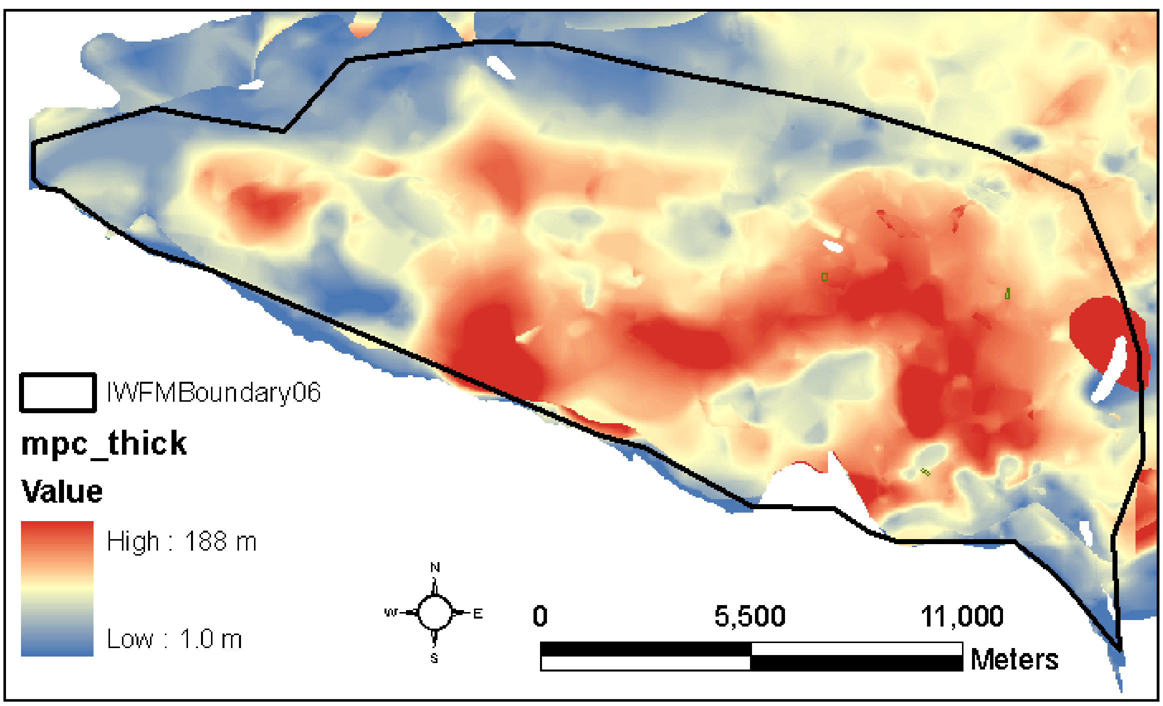


Figure 2.6 GIS layer representing the thickness of the Mio-pliocene gravel aquifer.

The surface topography elevation values are also entered per node into the IWFM model. Values of surface elevation were taken from a USGS Digital Elevation Model (DEM). USGS produced the DEMs by photogrammetric techniques from stereo-photo pairs, stereo-satellite images, or interpolation of digitized elevation data. The DEM used for this modeling effort is a 7.5 minute DEM data have a grid spacing of 10 by 10 meters on Universal Transverse Mercator (UTM) projection North America 1983. The quality of the DEM is classified as Level 1 with a standard root mean square error of 15 meters. (Moore et al 1991 as expressed in ASCE 1999)

As expressed by Moore (1991), advances with GIS technology and the increasing availability and quality of DEM have greatly expanded the application potential of DEM to many environmental computer model investigations. However, few studies assess the induced model error from the noise and low resolution of DEM. Seybert (1996) utilized the Penn State Runoff Model PRSM to compute runoff events using different qualities of DEM. Results from this research suggest that the model was more sensitive to peak flow estimates than spatial resolution. Other studies on DEM data on modeling results include Wolock and Price (1994). They estimated the effect of different resolutions of DEM on modeling results utilizing TOPMODEL. Results of this research show that the model was very sensitive to the DEM map scale and resolution. However, mapping the water table didn’t improve from a 90-m DEM to a 20-m DEM suggesting that the water table configuration may be smoother than the land surface topography. Results from the literature suggest that there are no firm guidelines to the selection of DEM selection and the impacts of the resolution and accuracy into hydrological models would depend on the type of solution and model construction.

For this modeling effort, the Walla Walla Basin Watershed Council conducted a survey-grade GPS survey of surface and groundwater monitoring sites in the summer and fall of 2009 (Patten 2010). The survey- grade GPS unit reported an accuracy of 5 centimeters under perfect conditions of operation. The watershed council surveyed all of the monitoring wells, head of springs, surface water gauges and bottom elevation of inflow streams and irrigation canals. The surveyed elevation data was considered into the IWFM model at each representing node. For the rest and majority of nodes (model area) not surveyed, the 10 by 10 meter DEM was used to estimate surface elevation.

2.5.4 Basalt aquifer interaction

The Columbia River Basalt Group lies beneath the model domain. This basalt aquifer is a source of water used for agricultural and urban demands (USGS 2010). The IWFM model of the Walla Walla Basin does not simulate this aquifer or a direct interaction between the gravel aquifer. The Mio-Pliocene fines (Blue clays) of 50 meter in thickness are considered to be an aquiclude confining layer for the basalt aquifer. No hydraulic connection is suspected between the gravels and the basalt aquifer (Newcomb 1965). There is an indirect connection between the aquifers by over applied irrigated water pumped from the basalt aquifer. The IWFM model takes into consideration the surface water demand satisfied by pumping of the basalt aquifer by importing water from a hypothetical source outside the model area. Rick Henry from the Watershed Council provides initial estimates (Figure 2.7) of utilization from the aquifer from personal communication between the irrigation districts. The timing and flow of the basalt water was then restricted to the estimated areas and then calibrated in IWFM through zone water demands. The Water Balance section presents calibrated results. Although this modeling effort provides a reasonable estimate of the utilization, further precision in the description of the basalt unit would be desirable.

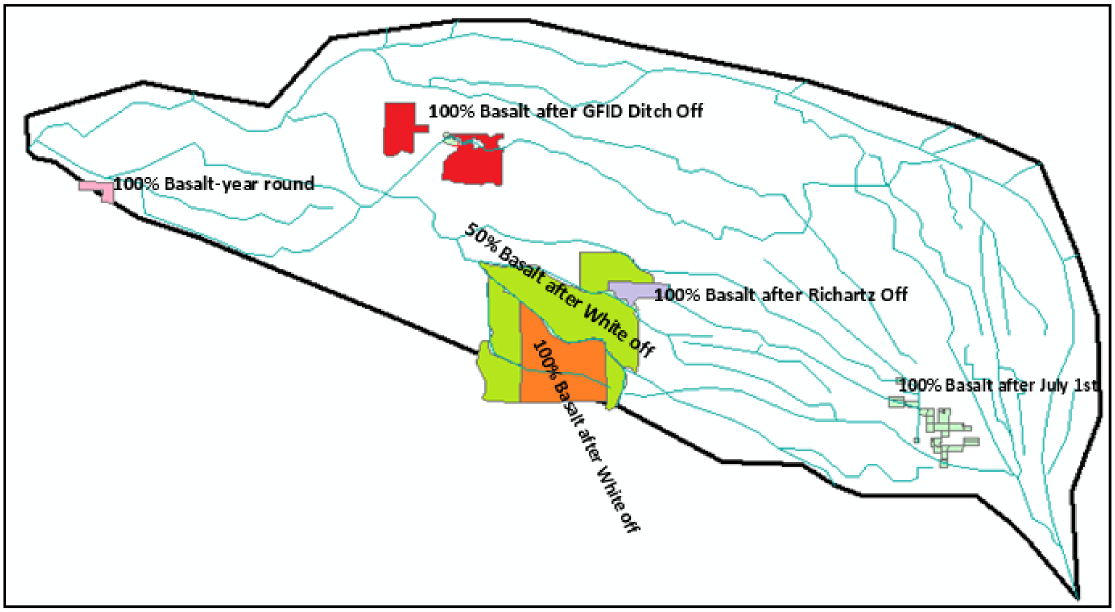


Figure 2.7 Rick Henry estimation of basalt water utilization over the model domain.

2.5.5 Model boundaries and initial conditions

The finite element approach used in IWFM simulates the movement of groundwater from a set of initial conditions and boundary conditions. Initial conditions are set for all nodes at the beginning time step while boundary nodes are set for the entire period of simulation. Values can be set at boundary nodes as either fluxes or groundwater elevations (internal boundaries are not considered in IWFM) for each time step in the simulations. Initial conditions refer to groundwater elevations at every node in the model area at the first time step of the simulation period. IWFM will then map groundwater head (water table elevation) at every node for each time step. Computations are based on the boundary values and complexities of the basin including agricultural and municipal water demands and water redistribution within the model area.

A map of the potential groundwater head elevation over a given area is known as a water table map, which is typically plotted as contours, similar to a topographic map. Haitjema (2005) has shown that for unconfined aquifers that are highly permeable, the water table is considered “recharge controlled” and the top of the water table does not replicate the topography or land surface. In the model area, the Walla Walla gravel aquifer is a highly productive aquifer with hydraulic conductivities that vary from 15 to 70 m/day. The challenge here in mapping the water table within the gravel aquifer comes from complexities in the basin related to groundwater withdrawals, redistribution, and key land surface features controlling the shape of the water table (springs and creeks). A water table map for an unconfined aquifer was initially made by interpolating values of water levels from observation wells, then performing an analysis of key hydro-geologic features in the basin, incorporating and modifying the interpolated values during the calibration process. Common methods of interpolation include splining, inverse distance weighting and ordinary Kriging. Water table maps that are only based on the interpolation of water levels from observation wells and do not incorporate the key hydro-geologic features may underestimate the water table elevation by up to 40% Buchanan (2005).

The Walla Walla Basin Watershed Council maintains monitoring locations at a number of observation wells and pumping wells using automatic pressure transducers. The automatic pressure transducers record the water level in the wells daily and in some cases, hourly. The database of water elevation records includes 100 wells in Washington and Oregon. Personnel from the WWBWC collect data from all the pressure transducers every three months and perform a manual measurement with a water level electronic tape to confirm data accuracy. OSU, in conjunction with the WWBWC, performs quality assurance and quality control for the well database and water table maps.

The simulation period for this model is from January of 2007 through December of 2009. Monthly water tables have been created for this period to be used as initial and boundary conditions. The physical water boundaries present in the area are the Horse Heaven Hills to the south and the Walla Walla River to the north and west. Figures 2.8 and 2.9 show the well network maintained by the WWBWC and the resulting water table map.

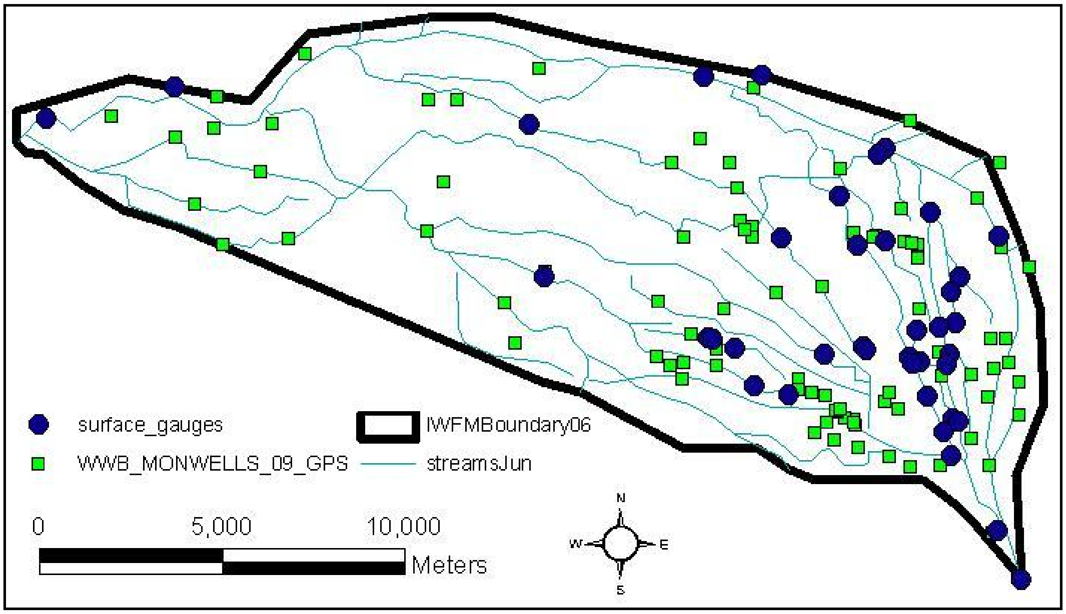


Figure 2.8 Monitored locations in the model domain. Green squares represent the monitored wells; blue dots represent surface gauges in streams blue lines.

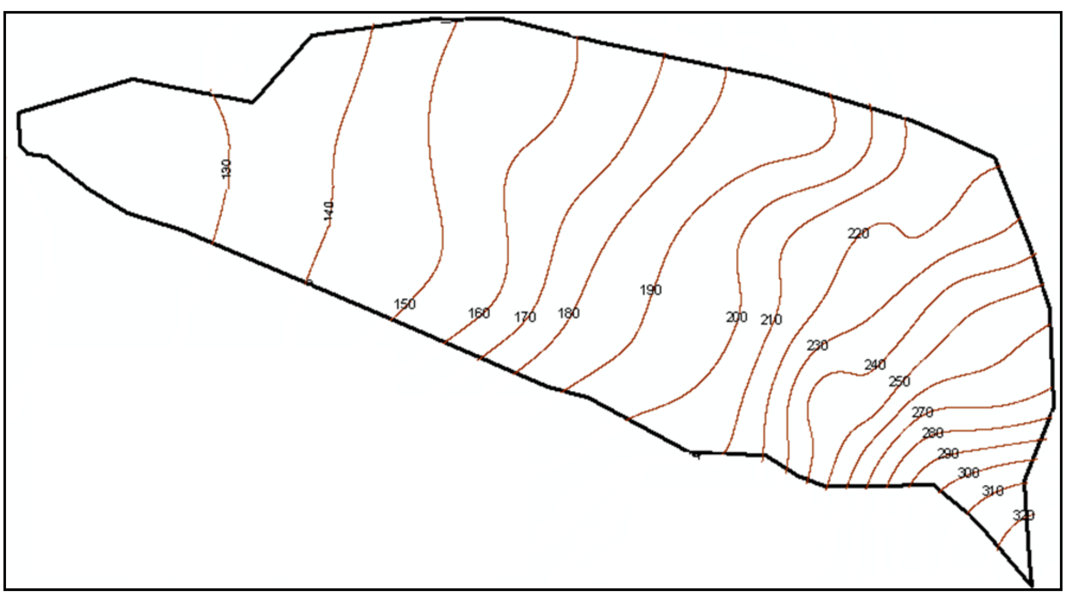


Figure 2.9 Water table elevations (m) for the month of August 2009.

2.5.6 Water demands calculated from land use.

The Integrated water flow model IWFM has the capability of estimating the water demand of a model element based on user specified crops and land uses. The user specifies per element the percentage or area covered by 4 possible types of land use: Agricultural, Urban, Native and Riparian. The user also specifies the acreages of each simulated crop in the agricultural native and riparian land uses per model sub-region. Urban water demand is entered per sub-region into the model dividing between the percentage of water use indoors and outdoors. The model also evaluates the soil moistures and root depth uptakes of the crops to estimate the model element water demand. Minimum soil moisture required for the crops is entered per day following the growing seasons. If the water demand is not supplied by precipitation and surface water diversions (diversions are user specified), the IWFM model supplies the water demand by pumping from the aquifer. This method provides a scientific approach to estimate pumping from the region where gravel pumping wells are not metered or regulated by local agencies. All the modeling results including water budgets will be presented with their degree of uncertainty. Groundwater pumping will be analyzed base on the range of possible values of water supplied by surface water diversions.

The Walla Walla Basin Watershed council hired the services of Margot Irvin as a private contractor to delineate the land and crop usages in the model domain. Irvin used GIS tools to delineate area of crops, water channels, roads and urban areas. The background maps source are from summer aerial photos of the 2006 National Agricultural Image Program (NAIP) run by the Natural Resources Conservation Service (NRCS) from the United States Department of Agriculture (USDA). These maps differentiate between 20 different crops and land uses. (Table 2.3)

Table 2.3. Land use classifications employed in basin model.

Crops	Other land uses
Alfalfa	City
Apples	Rural areas
Fallow ground	Industrial
Cherries	Riparian
Grapes	Urbanized (commercial)
Lawn	Water surfaces
Nursery	Bare soil
Peaches	
Native grasses	
Plums	
Pasture	
Row crops	

Troy Baker from the WWBWC reviewed Irving's maps delineation by comparing it to a land survey of the model area. The land survey randomly selected element numbers and when visiting the area, estimated the percentages of grown crops. Baker modified the land use map delineation according to the land survey and personal knowledge from interviews of local growers. The model does not account for crop rotation and assumes the same crops are grown during the simulation period. Figure 2.10 shows that it is estimated that 79% of the area is designated for agricultural purposes. Figure 2.11 shows the spatial distribution of the 20 land uses over the model area.

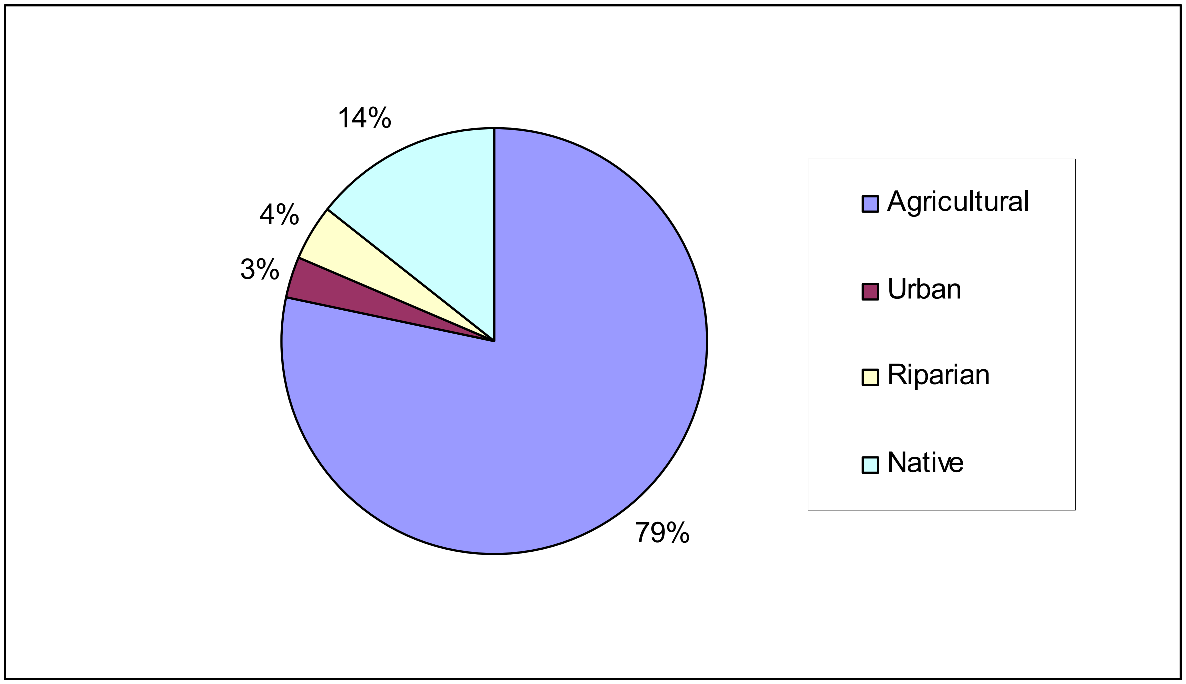


Figure 2.10 Land uses over the model domain incorporated in IWFM

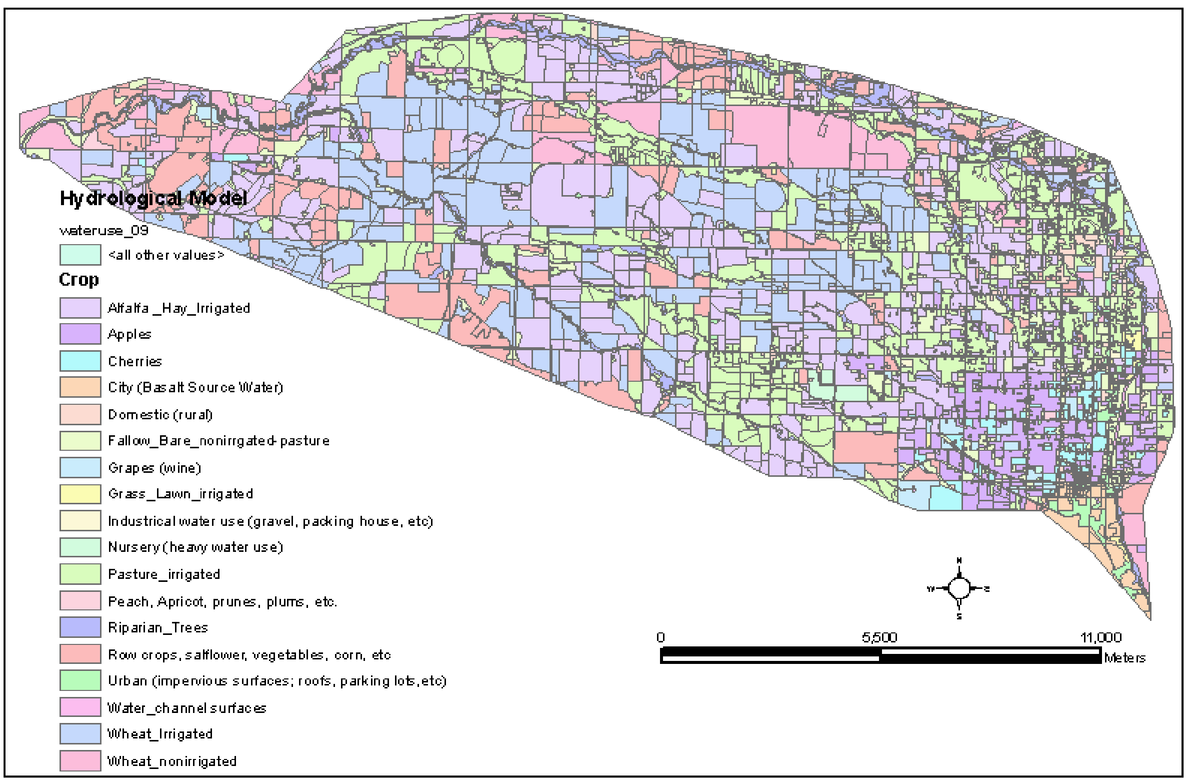


Figure 2.11 Land use distribution in model area.

2.5.7 Streams representation in IWFM

The Walla Walla IWFM Model has the most detailed water features representation of the available simulation models in the basin. The model simulates 220 km of streams. The stream representation in IWFM includes irrigation canals, springs and rivers. Each stream node overlays a groundwater node with an average separation of 100 m. Simplification of the stream network include that the natural meandering of the stream didn't exceed 100 meters, making the streams to be represented in straighter segments. A second simplification lumped canals that ran parallel with a separation less than 300 meters apart. Only 5% of canals were lumped together. A third simplification is number of stream and river inflows that flow into the model area. The surface water simplifications were made based on the available data gathered by the WWBWC and OSU. The flexibility that the small 100 meter grid size allows for easy modifications of the stream network as data is collected in the future. Figures 2.12A and 2.12B show the comparison between the map canals from the WWBWC and the stream network representation in IWFM.

Jake Scherberg, a recent master's student from OSU, calculated the rating tables for stream flow, diversions, bypasses points and surface water inflow. Rating tables are used in IWFM to calculate flow (m^3/day) based on water head elevation (m) and the cross sectional area (m^2) of the stream bed. Surveys and gauge installations from the WWBWC provide the initial estimates that were later calibrated to match observed flow data. Surface water diversions are the flow estimates taken from specific stream nodes to be applied to the parcels of land. Scherberg calculated surface diversion based on gauge data, personal communication with the council and irrigator districts. Future model enhancements include a closer review of the diversion calculations. The model, however, because land use water demand is calculated per sub-regions (instead of per element) is not sensitive to a close representation or small variations to surface water diversions.

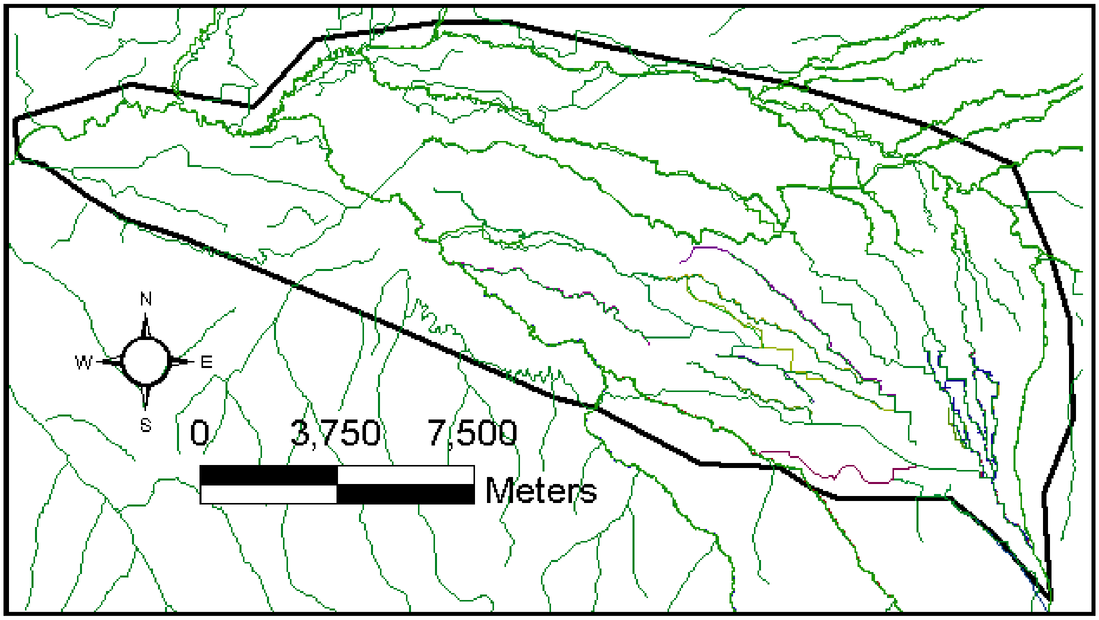


Figure 2.12A. Streams and channels in the model area, a sub-region of the Walla Walla Basin.

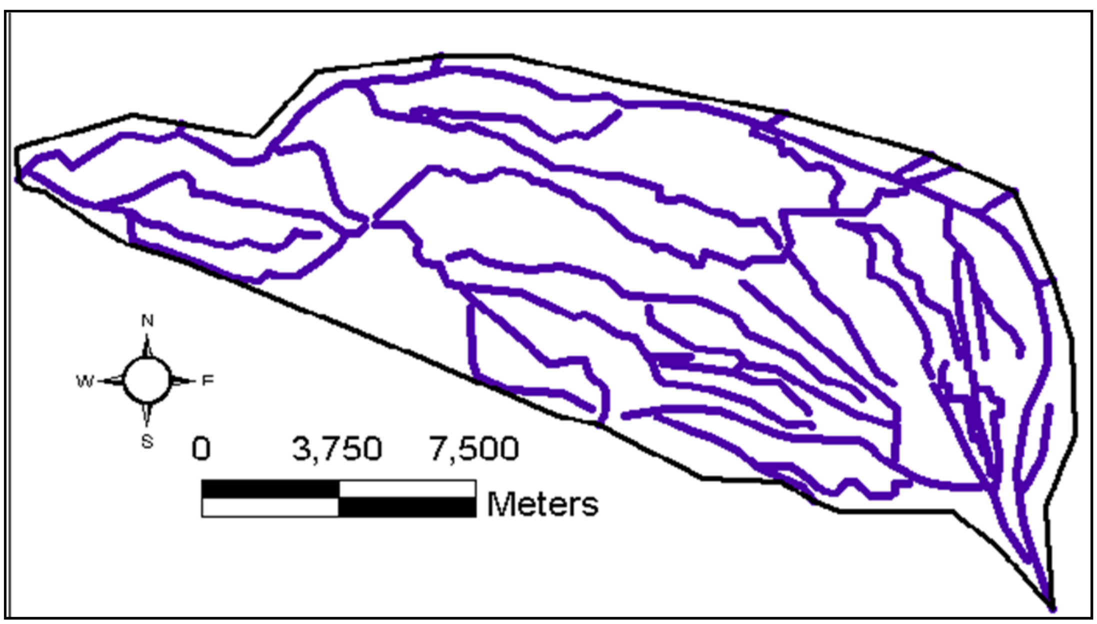


Figure 2.12B Streams as they are currently represented in the IWFM model.

2.5.8 Evapotranspiration and precipitation

As part of the task 4.4 (Table 2.1) of the WMI Monitoring Program Phase II, the WWBWC collects and maintains data from six climate stations in the basin. From these, three are $ET_0$106 Weather Stations (Campbell Scientific, Inc.) installed in different differing crop types. (1) Pasture/alfalfa field that is being irrigated regularly (West Umapine) Hudson Bay area N5094153 E379764 (2) Irrigated Orchards (Lefore) Milton-Freewater, OR N5089363 E379764 (3) Irrigated cow pasture, (Bullock) canyon confluence of north and south fork N5083399 E399347. Reference evapotranspiration and precipitation are collected from these ETo stations. The other three climatic stations are part of the AgWeatherNet (AWN) of the Washington State University. At the moment, precipitation was the only information gathered from these stations. Figure 2.13 shows the location of these stations.

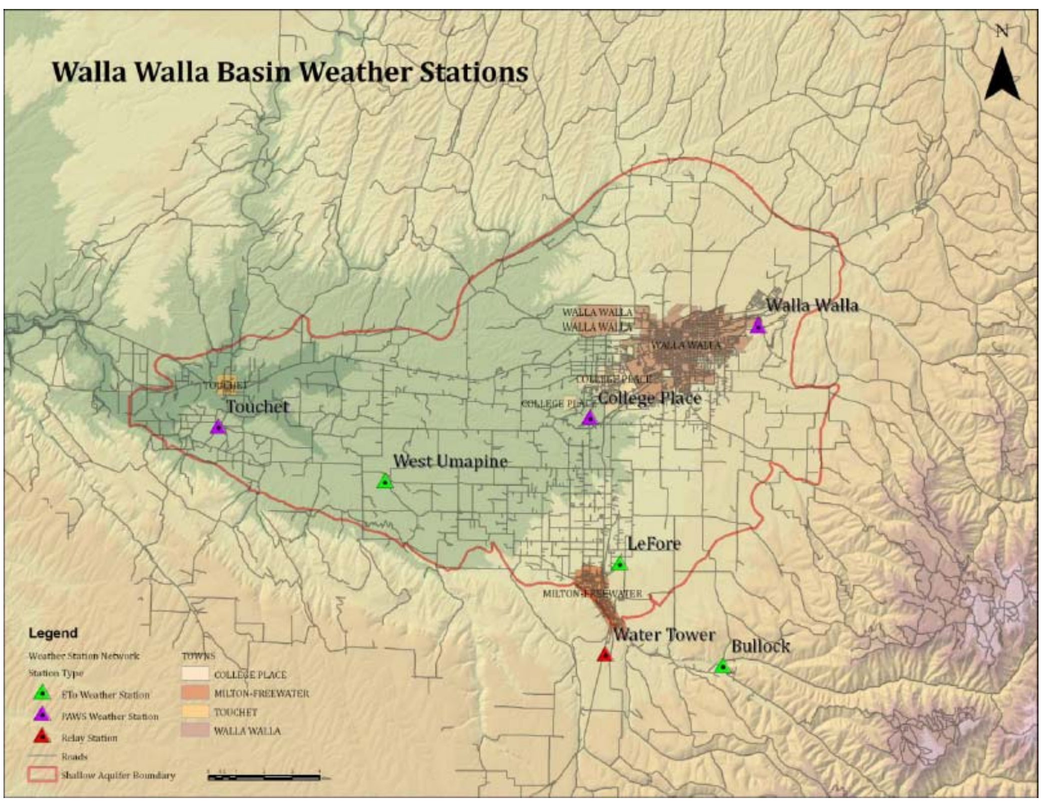


Figure 2.13 ET_o Stations in the Walla Walla Basin. Picture from: Final Report ET Climate Data Task 4.4(2009) Troy Baker, WWBWC.

The IWFM Model estimates actual evapotranspiration (ET_{act}) by calculating the soil water stress coefficient (K_s) at each time step and multiplying it to the field measured Et_0 and the crop specific parameter K_c (Allen, et al. 1998). The referenced evapotranspiration is imported directly from the ET_0 stations. It is a measure of the potential evaporation of an irrigated field of alfalfa. The crop coefficient (K_c) was taken from the FAO56 recommendations (Allen, et al. 1998). It is a parameter that varies with crop type, developmental stage, and season. The factor Ks incorporates the effect of soil moisture shortage on the crop evapotranspiration rate. This modeling effort provides a reasonable estimate of evapotranspiration in the Walla Walla Basin in a scientific defendable manner. Calculation results will be compared to the estimates by Agrimet, weather stations managed by the US Bureau of Reclamation, and incorporated into the water budget uncertainty analysis.

$$ET_{act}= K_s K_c ET_0 \qquad \text{Eq. 2.11}$$

Where
ET_0 = reference crop evapotranspiration
K_s = water stress coefficient.
K_c= crop coefficient
ET_{act}= actual evapotranspiration.

2.5.9 Soils

Soils parameters in the IWFM model are used to estimate runoff, infiltration and flow through the unsaturated zone. The estimation of runoff in IWFM is by the SCS curve number developed by the National Resources Conservation Service (Dingman 2002). Infiltration is calculated in IWFM as the difference between precipitation and runoff. The infiltrated water travels downward through the unsaturated zone delayed by the thickness of the soil layer. As soon as the unsaturated zone reaches field capacity, it recharges the groundwater. The groundwater recharge is called “Net deep percolation”. Figure 2.14 shows the conceptual model representation of the soils in IWFM.

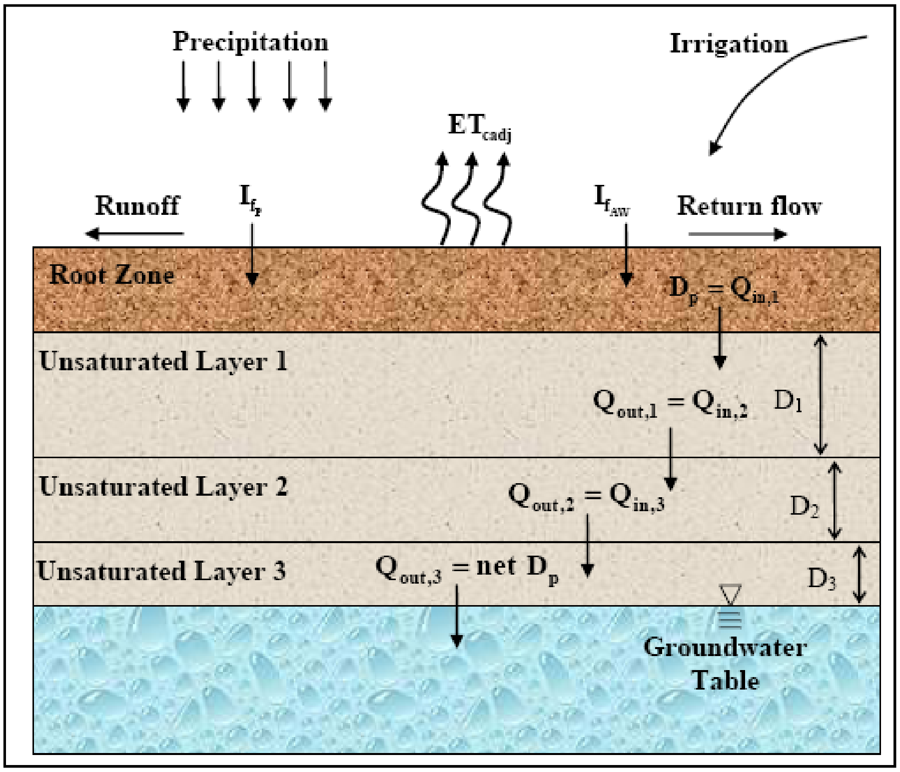


Figure 2.14 Soils budget flows. Picture from IWFM Theoretical Manual ver. 3.0

In the Walla Walla River Basin there are soils descriptions made by the USDA surveys (USDA 1987). Based on those descriptions, initial values of soils parameters and curve numbers were selected from literature reported values of similar types of soils (Dingman, 2002). The curve numbers were converted to metric units following the procedure suggested in the IWFM user manual 3.0. Table 2.4 shows the dominant soil types in the mode subareas. Figure 2.15 shows the model subareas. The model sub-areas were defined in collaboration with the watershed council. They represent areas inside the model domain with similar land uses. For example, sub-region 2 encompasses the city of Milton free-water, sub-region 3 represented the irrigated orchards area with high conductivity soils, sub-region 4 are the springs.

Table 2.4 Soil properties determined by analysis of NRCS reports for input into the model.

Soil Parameters				Curve numbers			
Sub-region	Hydraulic conductivity (M/day)	Field Capacity (volume water / unit volume soil)	Porosity	Agricultural	Urban	Native Vegetation	Riparian Vegetation
1	17.48	0.054	0.43	75	83	61	55
2	14.24	0.092	0.45	70	85	69	66
3	12.73	0.087	0.45	81	83	77	66
4	1.84	0.232	0.46	88	85	84	77
5	0.62	0.225	0.48	82	82	84	77
6	0.53	0.222	0.47	82	82	84	77
7	0.56	0.177	0.49	82	82	84	77

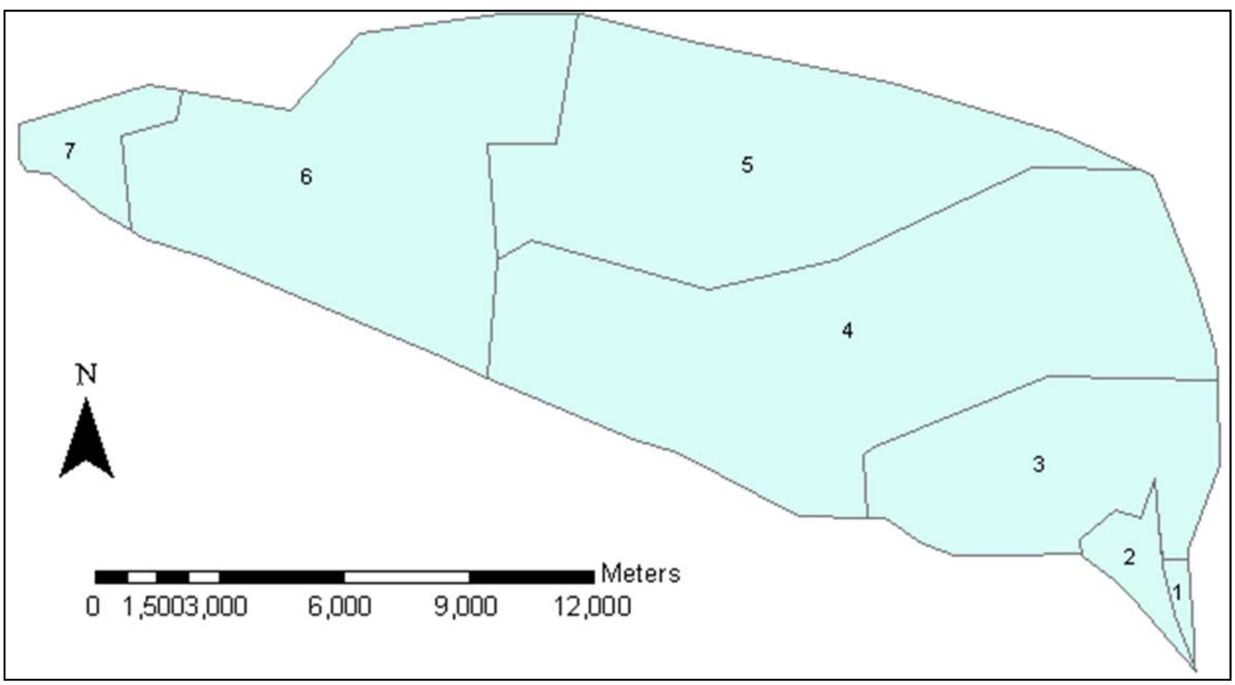


Figure 2.15 Model sub-areas used for model water budget calculations

2.5.10 Infiltrating basins for aquifer recharge modeled as lakes

There are three current projects of artificial aquifer recharge in the model area. The purpose of these projects has been to evaluate the feasibility of artificial aquifer recharge for the purpose of restoration and as a conjunctive management approach to mitigate irrigation efficiencies. The first report in the basins that suggests artificial aquifer recharge to restore the aquifer is "Geology and Groundwater Resources of the Walla Walla River Basin Washington – Oregon" (Newcomb 1965). In this report, Newcomb mentions that a couple of pilot tests in the area have reported successful results. Newcomb, however, does not present any additional information or data about these projects. The principal reasoning for artificial recharge is to use the gravel aquifer as a storage reservoir since 80% of the precipitation falls in the winter months when the demand for agricultural is minimal and the aquifer presents optimal conditions with high infiltrating rates and large storage capacity. Since 2004, artificial aquifer recharge has been tested in the basin. A brief description of the projects testing the feasibility and engineering design next and are summarized in Table 2.5 over the last 3 years of operation.

1) Hudson Bay aquifer recharge project, managed by the HBDIC and the WWBWC. The project diverts water from the White Ditch Irrigation Canal into rectangular infiltration basins with a surface area of 10,000 m^2 (2.5 acres).The permit for this project allows a maximum diversion of 1.4 m^3/sec (50 cfs) depending on the flows in the Walla Walla River. The project, however, has been only able to recharge a maximum of 0.5 m^3/sec (17 cfs). The basins used in this project have been expanded three times in order to recharge as much water as possible. Initial analyses have shown that the infiltration rate does not increase linearly with an increase of infiltration area, but rather appears to scale more closely to the perimeter of the project, reflecting the impact of mounding of groundwater below the project (Rastogi 1998). The results of the recharge rates obtained with different areas available for infiltration will be used to improve our

understanding of the engineering design of future infiltrating basins. More of this can be found in chapter 4 of this document entitled “Engineering considerations of infiltrating basins for aquifer recharge”.

2) Locher Road aquifer recharge basin, managed by Gardena Farms Irrigation District, diverts water into an abandoned gravel pit with constructed rectangular and circular basins. As with the Hudson Bay Aquifer Recharge Project, this project has expanded from its pilot scale, showing a non-linear relationship between recharge capacity and infiltration area. Washington Department of Ecology and the Gardena Farms Irrigation District hired GSI Water Solutions to monitor the operations of this project. This includes installation of observations wells, water quality monitoring and aquifer testing with a fully penetrating well of the gravel aquifer (Lindsey 2010).

3) Hall Wetland aquifer recharge. This project recharges the gravel aquifer by flood irrigating a 5 acre field pasture. This site does not have constructed infiltrating basins. This project is very valuable as a comparison of the possible methodologies for artificial recharge.

Table 2.5 Current recharge projects included in the IWFM model.

Project	Average Infiltrating Area ($X10^3$ m^2)	Average recharge rate ($X10^3$ m^3/day)	Total recharge volume ($X10^3$ m^3) 2007 to 2009
HBDIC	9.0	33.0	11,100
Locher Road	6.0	17.5	300
Hall Wetland	20.0	0.6	350

The Walla Walla IWFM model simulates artificial aquifer recharge with the IWFM module package for simulation of lakes. Figure 2.16 shows the processes simulated in the lake module of IWFM. Water can be taken from a stream segment (irrigation canal) into lakes (infiltrating basins). The lake then recharges water into the gravel aquifer at a rate specified by the user. The Walla Walla IWFM model is structured to recharge all the water diverted from the stream segment. The rate at which the basins recharge the aquifer is taken from field gauges located at the recharge projects. The advantage of using a regional hydrological model to evaluate the effects of aquifer recharge is the ability of the model to account for changes in hydrological boundaries. Such boundaries include pumping from domestic and agricultural wells in the area; additional groundwater recharged from nearby unlined irrigation canals; the effects to gaining river segments and the groundwater discharge to springs. Post-processing analysis can evaluate with the surface topography elevation values the grid cell areas that can be flooded by recharge activities.

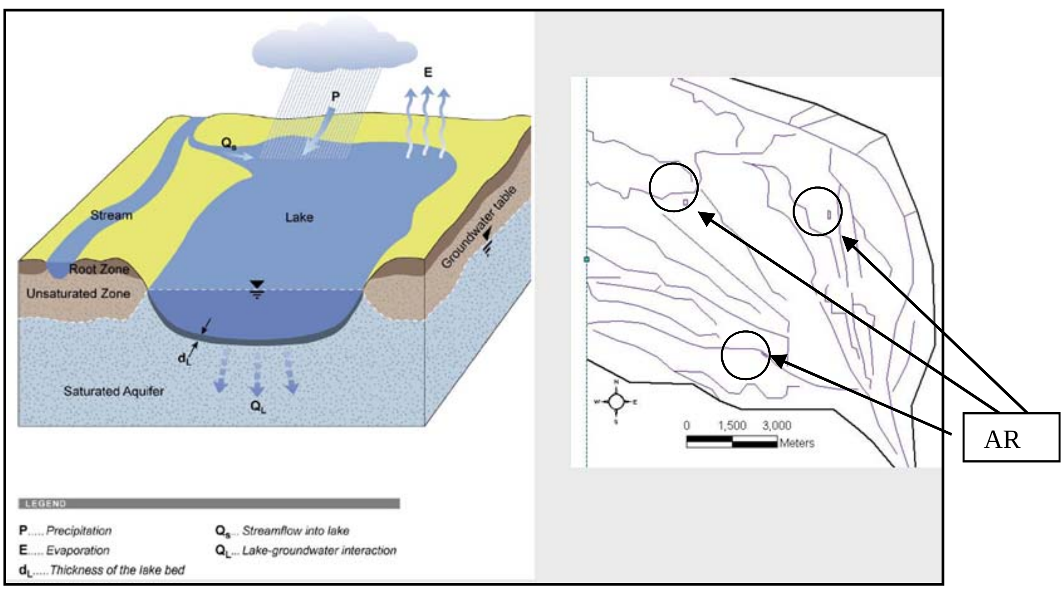


Figure 2.16 Artificial aquifer recharge sites are modeled as lakes in IWFM. This model includes three projects in Oregon; Hall Wetland, Locher Road and Hudson Bay.

The IWFM model once calibrated can be used to evaluate the feasibility of proposed aquifer recharge projects. The process of selecting suitable locations could involve predictive runs with IWFM under diverse circumstances. The procedures used for site selection is presented in the section entitled: “Proposed methodology for evaluation of locations for artificial aquifer recharge basins” under section 2.9 entitled “Examples of application of the IWFM model for Decision Support System”. Engineering considerations in the evaluation of infiltrating basins also includes the estimation of groundwater mounds created under the projects. The IWFM model does not attempt to be used for the estimation of possible infiltrating rates from different basin geometries. Therefore, this project proposes a separate, highly resolved 3D simulation of the vadose zone coupled with IWFM. A vadose zone model could be structured with a finer scale (in space and time) to fully capture the mound and estimate the achievable recharge rates given the groundwater mound formations. The IWFM model will then be used to estimate the regional impacts considering the hydrological boundaries previously mentioned.

2.5.11 Model development summary

The Walla Walla IWFM model domain is 231 km^2. The model was structured with a detailed resolution of the areal water resources utilizing an average node spacing of 100-m; for a total of 18,520 nodes in two aquifer layers and one soil layer. The surface water features include three artificial aquifer recharge projects modeled as lakes and 220 km of irrigation canals and springs that are simulated as stream segments for a total of 2015 steam nodes. The Walla Walla River Basin model is the most concentrated application yet to be developed for IWFM in terms of the number of nodes and water features definition. As an example, the California Central Valley application of IWFM was structured with 1,393 groundwater nodes and 432 stream nodes for an area of 51,800 km2.

Model Features:

- Model area: 231 km
- Total number of nodes: 18,520
- Number of elements: 36,484
- Aquifer layers: 2
- Artificial aquifer recharge model as IWFM-lakes: 3
- Total length of irrigation canals model as streams: 220 km
- Stream nodes: 2,015
- Node spacing: 100 m

2.6 Model calibration and sensitivity analysis

Hydrological models are conceptual representations of the main hydrological features in a basin (Sorooshian and Gupta 1995). The development of a hydrological model up to its calibration consists of assembling hydrological information gathered in the field to support our conceptual model and spatial representation. For the purpose of this research model error is defined as the difference between the estimated (simulation) and the measured (observed) value of groundwater head elevations (length) and the estimated and observed surface water flows (flow rates). The sources of the model error arise from the wrong conceptual representation, neglecting certain processes that are assumed unimportant, or the unintentional neglect of unknown or unconsidered process (ASCE 1999). Model parameters essentially represent a user-controllable form of model formulation error (Singh 1995). Model calibration consists of selecting the input parameters that minimize the error from our conceptual model. The processes involved during model calibration are presented in this chapter.

First, the development of the Objective Function (O.F.) to be minimized during parameter estimation is described. The O.F. employs weights to observations for the purpose to include data from different sources of uncertainty and different kind of observations. A dedicated section is presented here to introduce a special weighting factor used to avoid statistical evaluation bias by an un-even sampling distribution of observations throughout the study area. Second, once weights of observation and the O.F. have been established, a sensitivity analysis is performed to evaluate the model importance of 29 parameters. The sensitivity of the model to parameters is compared by scale composite sensitivities (Hill 2009). Finally, from the sensitivity analysis, the model parameters with the highest scale sensitivities are selected for parameter estimation. A simplification of the Gauss-Newton equation is suggested for parameter estimation procedure.

2.6.1 Objective function and weights assigned to observations

The objective function is a dimensionless numerical representation of the difference between model-simulated output and observed values (Sorooshian 1995). Model parameterization selects those values of parameters that minimize, or optimize, the objective function. The objective function used during our calibration procedure is a weighted least-square objective function (Eq. 2.12). A single objective function combines gathered information from surface water gauges and monitored observation wells by the use of weights. In a multi-parameter model, a single objective function is preferred over a multi-objective function when the sensitivity of parameters is being evaluated (Hill 2007). Also, during parameterization, correlation between parameters decreases with the use of a single objective function.

$$O.F. = \sum_{t=1}^{n} w_{gw} \bullet (h_t^{obs} - h_t^{sim})^2 \pm \sum_{t=1}^{n} w_{sw} \bullet (q_t^{obs} - q_t^{sim})^2 \qquad \text{Eq. 2.12}$$

where:

O.F. = Dimensionless objective function to be minimized during parameterization.

w_{gw}=Groundwater weight associated to the monitored observation ($1/m^2$)

h^{obs} = Observed water level elevation at monitored wells (m)

h^{sim} = Model Output simulated ground water head at selected model nodes (m)

w_{sw}=Surface water weight associated to the monitored surface gauge ($1/m^3$)

q^{obs} = Observed surface water flow at monitored locations (m^3)

q^{sim} = Model Output simulated surface flow at selected surface water nodes (m^3)

The uses of weights in the objective function serve two purposes. First, they indicate the importance of the observation relative to its uncertainty. Second, weights make the equation dimensionless so observations from different sources (groundwater and surface) with different scales and magnitudes can be combined into a single equation (Hill 2009). The weight factors are defined for the purpose of this research as the inverse variance of the measurement error (Eq. 2.13). The variance of the measurement error is estimated as the summation of all the variances from the possible sources of uncertainty (Eq. 2.14). The sources of uncertainty considered in this research are shown in table 2.6 for groundwater monitoring points and in Table 2.7 for surface-water gauges.

$$\omega_i = {}^{1}\!/_{\sigma_i^2} \qquad \text{Eq. 2.13}$$

$$\sigma_i^2 = \sum_{1}^{e} \sigma_e^2 \qquad \text{Eq. 2.14}$$

where

ω_i = Weight associate of the “i” observation

σ_i^2 = Variance of the error of the observation *i*

σ_e^2 = Variance from the uncertainty source *e*

The variance associated from each source of uncertainty can be estimated by squaring the standard deviation estimated from the known coefficient of variation Eq. 2.15. The coefficient of variation and/or error associated from each source of uncertainty was assigned in conjunction with the Walla Walla Basin Watershed Council.

$$\sigma_e = C.V. \bullet \mu \qquad \text{Eq. 2.15}$$

$$\sigma_e = \frac{err}{1.96} \qquad \text{Eq. 2.16}$$

Where

$C.V$ = Coefficient of variation

μ = Mean observation value

err = Estimated measurement error from the uncertainty source

Table 2.6 Sources of uncertainty for groundwater wells:

Source of uncertainty	Brief Description
Barometric adjustment	Gauges in observation wells that don't adjust to barometric variation
X,Y location GPS	The associated error of GPS used for surveying well locations.
Elevation GPS	The associated error of GPS used for surveying well locations
Well perforation depth	Well logs are used to estimate the perforation depth.
Non-model local process (e.g. used of pumping well as observation wells)	The measurement only represents a local process not included in the model and is not representative of the behavior of the entire aquifer.
Node location vs. Well location	The location of the nodes is not based on the well location The model uses the closest node to the well.
Instrumentation error	Error associated with the pressure transducer error factory specs
Set up error of instrumentation	Error associated with a wrong installation procedure.
Instrumentation maintenance	Battery replacement and other instrumentation maintenance requirements
Density of observations	Un-even distribution of observations. Discuss in the next section 3.4.2

Table 2.7 Sources of uncertainty for surface water gauges:

Source of uncertainty	**Brief Description**
Instrumentation	Transducers factory specs
Gauge Location	Representative placement within the stream
Instrument Maintenance	Proper Instrument maintenance
Facilities maintenance	Proper maintenance to post and structures
Flow Capture	Is all flow measured by the gauge?
Rating Curves	Stage discharge relations created by few points that may miss high events
Atmospheric conversion	Not all the transducer are adjusted for atmospheric variations
Timing	The record time of the transducers is incorrect
Lumped streams	The IWFM model simplification of streams lumped 5% of the stream segments
Channel obstructions	Seasonal vegetation, freezing etc.
Others	Experience of installations

Table 2.8 Coefficients of Variation associated with the classification of surface-water gauges (from personal communication to WWBWC)

Source of uncertainty	**Coefficient of variation**
Grade 1	10%
Grade 2	17%
Grade 3	27%
Grade 4	36%

Groundwater wells measurements of observational error were calculated using the standard deviations or coefficients of variations from Table 2.6. For surface-water gauges, measurements of the uncertainty were calculated based on the coefficient of variation classification of Table 2.7. The variance from each source of uncertainty was estimated in conjunction with the WWBWC. Coefficient of variation values were gathered from reported literature values of similar analysis and from personal experience of the field technicians responsible for maintenance and operation of the installed gauges. Stream and wells that had an automatic recording gauge were compared to manual measurements taken thought the year. (OSU-WWBWC meetings of 2009)

In practice it is generally impossible to identify all errors that contribute to an observation. The values reported in this research are approximate values that are useful to evaluate the existing monitoring network in the Walla Walla Basin. As pointed out by Hill (2009), parameter estimations are not sensitive to moderate variation in the weights "*nearly identical results are typically obtained given weighting within a range that reasonably represents the likely observation error. If the weighting is changed beyond reasonable ranges, large variation in regression can occur causing the regression to lose meaning and become arbitrary".* This research project doesn't consider to a full extent, classification of gauges and estimation of sources of error is necessary at this point. The Walla Walla Watershed Council in collaboration with state and federal institutions maintain an extensive network of monitored locations. These are available to the public at the watershed council's webpage http://www.wwbwc.org/. Some of the gauges data are available to download in real-time.

2.6.2 Sensitivity analysis

The IWFM Walla Walla Model is a multi-parameter physically based hydrological model. The model is considered to be a physically based model since most of its parameters can be measured in the field (Sing 1995).

Hydrological parameters are highly heterogeneous in the field at different scales. For simplicity the model averages or lumps parameter values for the entire model area. These values are thought to be representative values that best suits the simulated conceptual model. Hydrological parameters are then the considered the modeler's controllable form of its conceptual model error (Sing 1995). The sensitivity of the model to parameters is reviewed in this section. The goal is to identify the parameters that are the most influential (highest sensitivities) on model results to be in the process of parameter estimation.

To assess what parameters are the most influential on model results, we need an overall metric of performance. Parameter sensitivities are defined by Hill (2007) as "derivatives of dependent variables with respect to model parameters" They can be calculated by central or forward differences as shown in Eq. 2.17. The sensitivity of the parameter "*p*" by the simulated value "*h*" calculated by forward differences

$$\frac{dh}{dp} = \frac{h(p+\Delta p) - h(p)}{\Delta p} \qquad \text{Eq. 2.17}$$

Parameters in the model from different hydrological process differ in their magnitudes and units of measurement. Parameters with bigger magnitudes are more likely to produce a higher effect on model outputs. To allow comparison between sensitivities of different parameters Hill (2007) proposes the following dimensionless statistic.

$$dss = \frac{dh}{dp} p\omega^{1/2} \qquad \text{Eq. 2.18}$$

The dimensionless scale sensitivities analysis is useful for spatial model evaluation of observations. Hill (2007) point outs that observations with large dss are likely to provide more information about the parameter *p* compared to observations associated with small *dss.* To evaluate the entire model sensitivity with respect to the parameter "*p*" using the total amount of information provided, the composite scale sensitivity (CSS) can be used as the summation of the sensitivities from all monitored locations including surface and groundwater data.

$$css = \sum_{1}^{NO} (dss^2 / NO)^{1/2}$$

Eq. 2.19

Where NO is the total number of observations in space and time.

The use of surface and groundwater data in the calculation of scale sensitivities diminishes the correlation between model parameters (Petros 2009). If a high correlation between model parameters is observed, they cannot be estimated uniquely. In hydrological models high correlation between parameters R>0.85 are commonly observed (Sing 1998, Hill 2007). Hill (2007) expresses "experience has shown that unique estimates sometimes can be obtained even with absolute correlation close to 1.00" The sensitivity analysis improves the evaluation of model parameters by including all available data. A parameter that may only be sensitive when surface data is used may not be sensitive when surface and groundwater are combined. By including data from different hydrological components, a new level of analysis is performed evaluating the impacts of model parameters in their interactions (*i.e*. the effects aquifer parameters have on surface waters).

The IWFM-Walla Walla Model is a multi-parameter model. There an infinite number of possibilities for parameter vectors. In theory, surface and groundwater parameters can be selected independently per model node as anomalies in the model area. For simplicity, lumped and average values are used in the model. Most of the parameters considered in IWFM for the simulation of groundwater flow are physically based or expressed in other words, parameters that can be measured in the field. Parameters that are non-physical include those for runoff calculations (Curve numbers) water supply and demands (water reuse) and fraction of soil water that becomes deep percolation. Table 2.9 shows the 25 model parameters chosen for the sensitivity analysis. A brief definition is provided in the table, for more information on these parameters please refer the IWFM user manual (Drogul 2009). For ease of comparison, a bar graph is shown in Figure 2.11 with the estimation of composite scale sensitivities.

Table 2.9 Variables considered for the sensitivity analysis.

Parameter abbreviation	Parameter Definition	Initial values
Ks1	Hydraulic conductivity of the first aquifer layer corresponding to the Quaternary unit	90.5
Ks2	Hydraulic conductivity of the second aquifer layer corresponding to the mio-pliocene unit	30.5
EspStor1	Specific Storage aquifer layer 1	20.5
EspStor2	Specific Storage aquifer layer 2	20.5
Espyield1	Specific yield aquifer layer 1	0.2
Espyield2	Specific yield aquifer layer 2	0.2
VertK1	Vertical hydraulic conductivity of the first aquifer layer	6.3
VertK2	Vertical hydraulic conductivity of the second aquifer layer	3.5
Kverttouch	Vertical hydraulic conductivity of the Touchet unit layer	4.5

uzporosity	Porosity of the unsaturated zone	0.3
UZ K vert	Vertical hydraulic conductivity of the unsaturated zone	3
FracPerv Area	Fraction of pervious area from urban areas	0.75
Basalt pump	Estimated imports from basalt pumping as a percentage of total water permits	0.5
Basalt R	Estimated area for apply basalt usages	0.5
Re-use	Aquifer Re-use percentage of water apply to fields	0.95
Soils-FC	Soils field capacity	0.16
Frac Deep	Fraction of excess soil moisture that will become deep percolation	0.7
SprEle	Head of spring elevations	245.6
Porosity Soils	Total Porosity of soils	0.46
QK rivers	Hydraulic conductivity of streams lying in the quaternary unit	1
Spring K	Hydraulic conductivity of springs	1
K river touchet	Hydraulic conductivity of streams lying in the touchet unit	0.7
CN-A	Curve number for calculation of runoff (A,B,C)	72
CN-B	Curve number for calculation of runoff (A,B,C)	81
CN_C	Curve number for calculation of runoff (A,B,C)	88

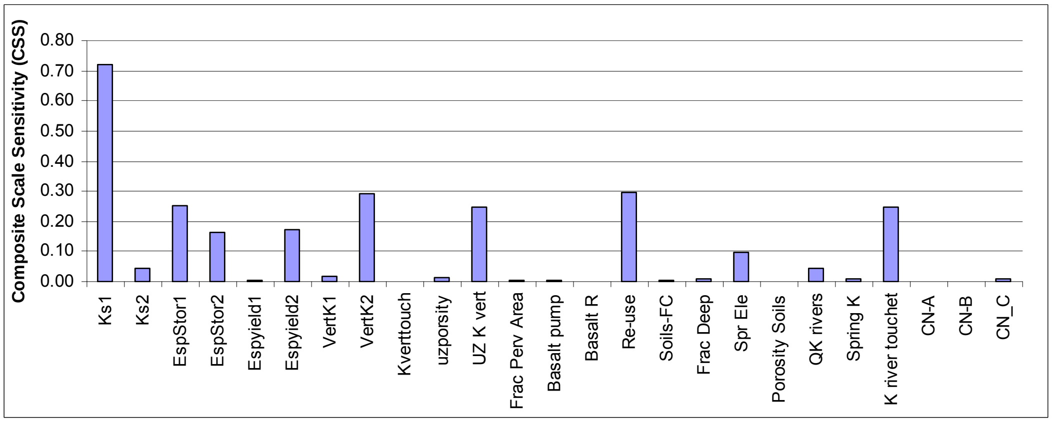


Figure 2.17 Composite Scale Sensitivities (CSS) for 27 parameters of the IWFM-Walla Walla model

Figure 2.17 presents the results from the sensitivity analysis. The parameters with the highest CSS are the parameters most influential on model output. We selected the ten most influential parameters for model calibration (parameter estimation). It was to be expected that the aquifer parameters of the three hydro-geological units were the most influential on the model results determining groundwater elevations and surface flows. The surface parameters most influential for model calibration are the head of spring's surface elevations, stream bottom conductivities and the water–supply parameter "Re-use". This last parameter determines the amount of water that is to be routed to a stream node if the element to which water has been diverted exceeds the demand at the time-step that water was applied. The initial estimate of 95% reuse factor was considered since the amount of water diverted is calculated by surface water gauges and personal communication between the irrigation district and the WWBWC.

The sensitivity analyses help us evaluate all the model parameters with respect to the overall model outputs. Parameters are not evaluated just for the hydrological process for which they are incorporated into the model but as their overall impact on model results. In this sense the sensitivity analysis determines what model parameters are important to predict or estimate. The process of parameter estimation is described next. Due to the High (3.5 to 4.5 hours) computation running times and the limited availability of time and resources, the sensitivity analysis provides an un-bias methodology to pick the most influential parameters for the time consuming parameter estimation.

2.6.3 Parameter estimation

The process of optimization of model parameters that minimize the objective function is called parameterization or parameter estimation. There are manual and automatic methods for parameter estimation. Manual methods are subjective relaying on user expertise and knowledge of the model area to adjust the model parameters. Automatic methods employ algorithms that, based on an iteration criterion, modify and evaluate model parameters values. Commercially there is software flexible enough to adapt to different simulation models. The most widely used software today are UCOED (USGS) and PEST (Parameter Estimation Inc.). These types of software linearize the objective function applying the Taylor series expansion and utilize gradient base methods to proceed to a minimum value. Gradient base methods utilize the parameter sensitivities obtained from several model runs to approximate the next parameter value.

Automatic calibration methods are preferred to manual calibration since they limit the subjectivity of the modeler and have the potential to creating a greater confidence to model predictions. The caveat for automatic methods is the high number of runs necessary per parameter estimate. The computational power required can be overwhelming for multi-parameter models with long computational times. In the case of the IWFM-Walla Walla model with runs times of 3.5 to 4.5 hrs. (on a Pentium-II P.C. computer) a super computer will be required for automatic methods to be applied. Automatic calibration methods take thousands of simulations per parameter (Sing 1998). The approach taken for this modeling effort is a combination of manual and automatic parameterization for the estimation of 10 model parameters. The chosen parameters were selected from a sensitivity analysis comparing 25 model parameters.

The parameter estimation procedure employed a simplification of the Gauss-Newton equation solved for one parameter (Eq. 2.20). The correct application of the Gauss-Newton equation requires the model to be linear with respect to parameters. In this research we do not attempt to linearize the objective function, instead, we use Eq. 2.20 for determining the directional change and manually adjust to the next parameter value. Eq. 2.20 was not used to find final parameter values but rather used as a guidance to evaluate the new value change from the results obtained from two model runs. The only difference between runs is the value of one parameter change at a time. The criterion to stop the evaluation of parameter values was set when changes in parameters did not improve the model standard error more than 0.5%. As is the case with gradient based methods, local minimum was observed to be reached with different combination of model parameter values reaching different final minimal values. Also, it was observed that different final values were reached depending on the initial starting parameter values. The strong correlation between the aquifer parameters makes hard to impossible estimate a single parameter at a time. It is difficult to state with confidence that the best set of parameters for the model have been found. Duan et al [1992] and Sing (1999) reported that for a rainfall –runoff model of Sacramento model hundreds if not thousands of local optimal solution on their response surfaces exist.

$$\frac{(h_{obs} - h_{p2}) * (P_2 - P_1)}{h_{p2} - h_{p1}} = P_{obs} - P_2 \qquad \text{Eq. 2.20}$$

h_{obs} = Observed water level elevation at observational well

h_{p1} = simulated head at observation node from the first run of parameter "p_1"

h_{p2} = simulated head at observation node from the second run with "p_2"

P_{obs} = Parameter estimation result

P_1 = Parameter value utilized for the first simulation run

P_2 = Parameter value utilize in the second simulation

Table 2.10 Parameters estimated during calibration

Variable	Acronym	Initial Value	Final Max.	Range of plausible values
Hydraulic conductivity Quaternary, lyr 1	Ks1	90.5	110	60–150
Hydraulic conductivity Mio-Pliocene , lyr 2	Ks2	30.5	38	27-60
Specific Storage lyr1	EspStor1	0.002	0.0026	0.001-0.004
Specific Storage lyr 2	EspStor2	0.002	0.0015	0.001-0.004
Specific yield aquifer 1	Espyield1	0.2	0.25	0.15-0.3
Specific yield aquifer 2	Espyield2	0.2	0.15	0.15-0.3
Vertical conductivity 1	VertK1	6.3	11	6 -15
Vertical conductivity 2	VertK2	3.5	3.8	3 - 6
Vertical cond. of soils	UZ K vert	3.0	3.8	2.3 -4.3
Agricultural Re-use water	Re-use	0.95	0.70	0.50 - 0.95
Streams leakage over Touchet	K river Touchet	1.2	1.5	0.75 – 3.0
Streams leakage over quaternary	QK rivers	2.0	2.5	1.0 – 4.0

The initial and final calibrated parameters values are shown in Table 2.10. Given the correlation between parameters, different combinations of parameters values can reach different local minima. For this, the bound of parameter values that when incorporated in the model produce a change of less than 10% in the model error standard deviation are presented in Table 2.10. The plausibility of the optimized parameter depends on the model accuracy and representation of local hydrological processes. These parameters values are thought to be a representative values for the entire model domain and constant over time. The IWFM model is structured to represent the study site with enough detail to allow its use for evaluation of water management alternatives. The purpose of the model is not to be used to investigate actual field hydrological parameters or hydrological processes at a small scale (<100m). Considering a fully distributed hydrological model thousands of input value combinations are possible. To represent the heterogeneity of parameters found in the field zonation or spatial interpolation model are suggested for future steps. At this stage a homogenous single value of parameters throughout the aquifer layers is considered

2.7 Model validation and transfer to final users

An assessment of the hydrological model is provided during model validation. The goal is evaluate the effectiveness of a model at meeting its initial purpose. There are a variety of approaches to demonstrate that the model is capable of providing reasonable results. In one approach, modelers split the available data in two sets. One set for model development and calibration and the second set for model validation (Klemes 1986, Singh 1995, Singh 2006). A second approach proposes a validation method that focuses on testing the process for which the model has been created for (Ewen and Parking 1996). The idea behind the latter approach is to evaluate the hydrological model processes by running hypothetical scenarios under a range of possible input parameter values. Differences between estimated and actual input data values are usually described in terms of uncertainty (Hill 2007). Model uncertainty can then be tested by running predictive scenarios with a single maximum and a single minimum value bounded by the main parameters affecting the process in question. The width of the bound and the resulting changes will then be evaluated.

The IWFM-Walla Walla model was developed and calibrated using data gathered from 2007 to 2009. Future modeling work (separate from this research) will validate the model by incorporating new hydrological data gathered from 2010. In this research project, the result of the statistical measurements of goodness of fit after the incorporation of the 2010 data will be shown as statistical validation of goodness of fit. Given the purpose of this research and data availability, the approach taken to validate the model was to evaluate the uncertainty of simulation runs for the hydrological process and the hypothetical scenarios this model has been created for. The description of the next steps taken for this analysis is a modification of the suggested steps of Ewen and Parkin (1996).

These are the steps followed during model validation and uncertainty analysis:

1. Specify the objective hydrological processes for the model to simulate: *a) estimate water budgets, b) predict the effects of lining the irrigation canals and c) evaluate locations for artificial recharge.*
2. Collect from literature, and when possible in the field, the parameter's values needed to construct the model*. Examples include: leakage rates of irrigation canals, recharge rates of infiltrating basins, aquifer hydraulic conductivities.*
3. Data collection performing quality assurance and quality control: *estimation of weights based on the uncertainty of the observation.*
4. Model development. *IWFM text file units and GIS project.*
5. Specify the features to be predicted: *Total amount of groundwater recharged. Groundwater head levels at observation wells,*
6. Instrument the test catchment: *The WWBWC in collaboration with OSU installed piezometers at the infiltrating basins and set up loggers at observation wells and irrigation canals.*
7. Run simulations using the collected input data
8. Calibrate model parameters. *Sensitivity and parameter estimation*
9. Evaluate model goodness of fit: *Statistical procedure evaluating model error to groundwater head elevations and surface-water flows.*
10. Review model development evaluation by interest groups: *the preliminary model development was presented to the Watershed council in June 2010. In December of that year, the model was presented to the Walla Walla Management Initiative Technical Review Team (WMI TRT 2010).* The model *transferred to its final users with hands-on workshops were developed in 2011*
11. Set bounds for the specified hydrological modeled process: the selection of scenarios and parameter bounds were evaluated during *workshops between the modeler and the WWBWC.*
12. Evaluate parameter bounds and simulated results of predictive runs.
13. Update the model. *Modeling it's a never ending process. Model updates occurred throughout model development, calibration, and validation.*

Model assessment occurred at different stages during the extent of this research. First, during model development, the processes and model assumptions were reviewed by experienced professors at OSU. The main advisors of this research include Dr. John Selker and Dr. Richard Cuenca. A calibration and validation period then followed. During this period, presentations to the WWBWC and their Technical Review Team (TRT) were performed. The discussion following these presentations had two clear messages. First, it is desirable for a model to actually be used by the final model users. Second, without model transparency and its degree of uncertainty, the model can't be used as a decision support tool.

OSU and the watershed council heard these concerns and developed a methodology that included model presentations, meetings and workshops to transfer the model from model developers to its final users. This methodology is presented in this thesis in the appendix section. During the process of model transfer, a new level of model validation was accomplished by evaluating the model performance for the process it has been created for during the model workshops. Also, model transparency was achieved by reviewing model assumptions, model data needs, and shortcomings. Finally, these activities help communicates model results to water managers, regulators, irrigators, modeling consultants and concerned citizens.

The incorporation of new hydrological data will add a new level of model validation. For the purpose of this research, model validation includes the model evaluation goodness of fit to historic conditions, as well as the activities for model transferring evaluating the models capacity to simulate the processes for which it was created allowed for the fulfillment of initial model objectives. Additionally, model credibility is increase by presenting it results in terms of model uncertainty.

2.7.1Statistical measurements of model goodness of fit

The ability of the model to simulate the hydro-geological conditions in the Walla Walla Basin is evaluated by how well the model fits the data observations. This section of the thesis reviews the data used for evaluation and the identification and definition of data outliers. We then evaluate the model goodness of fit by statistical measurements and graphical analysis identifying areas with acceptable and poor model performance.

The hydrological data used for model evaluation is the groundwater elevations at monitored wells and surface water gauges located in streams, springs and the main stem of the Walla Walla River. The WWBWC install and maintain this network of monitored locations a make them available to the public to download from the watershed council's internet page: http://www.wwbwc.org/. The well network has a total of 98 monitored wells in the model area and 25 stations for surface water gauges (Figure 2.18). About 80% of this locations count with automatic pressure transducers recording daily water levels.

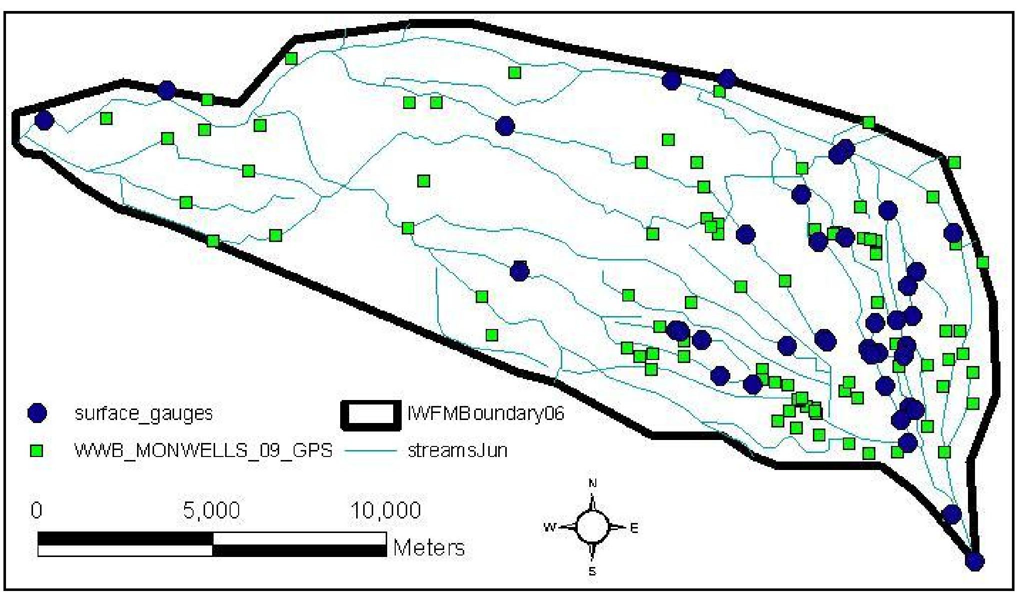


Figure 2.18 Monitored locations. Green squares represent the monitored wells. Blue dots represent surface gauges.

Outliers are defined by Ramsey (2002) as “an observation judged to be far from its group”. This research recognizes to type of outliers for those point that have either high “leverage” or “high influence”. An observation that lies far away from its group in the x direction is considered to have high “leverage”. An outlier that dominates the regression is considered to have high “influence”. Figure 2.19 shows the representation of these two types of outliers. The effects of data outliers is more pronounce in automatic methods for calibration than in manual methodology. However, for both methodologies outliers can dominate the statistics estimates used for model evaluation. The statistics used for model evaluation are resistant to outliers. Perhaps the only change when considering outliers is the increase goodness of fit reach by the model. Because of these undesired effects, outliers in the data were identified by use of the “leverage” and “Cook’s D” statistics (Hesel, D. and Hirsh, R. 1992). Leverage was calculated by Eq. 2.21 and Cook’s D by Eq. 2.23.

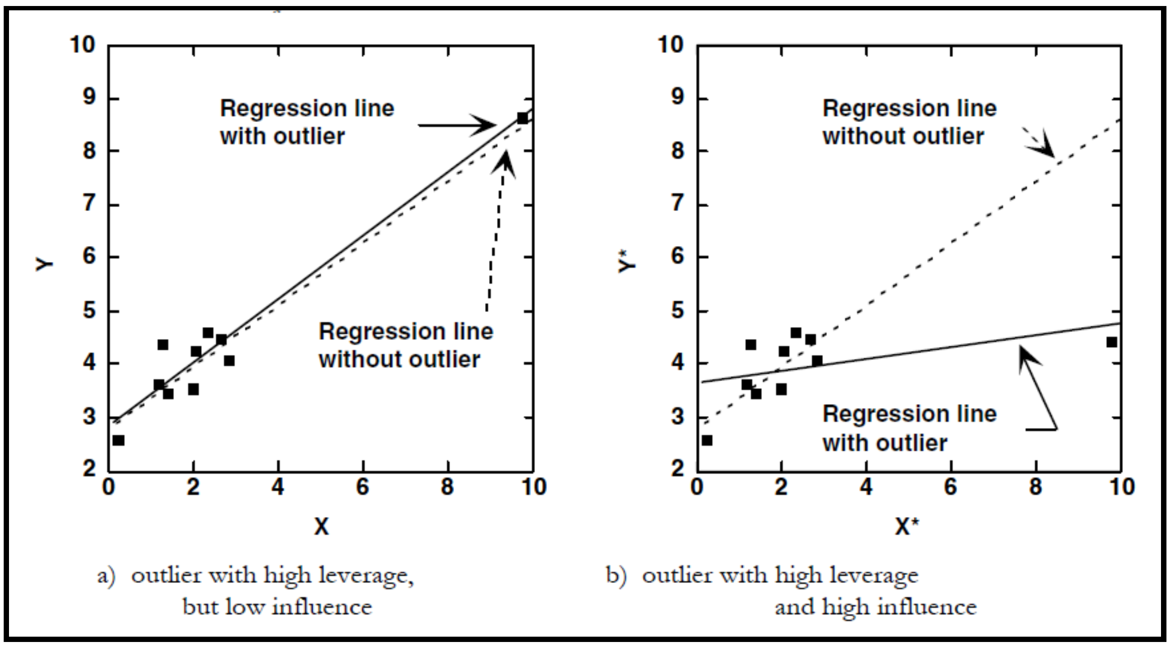


Figure 2.19 Representation of two types of outliers in regression. Figure taken from: Statistical Methods in Water Resources (Hesel, D. and Hirsh, R. 1992)

$$hii = \frac{1}{n} + \frac{\left(x_i - \bar{x}\right)^2}{SS}$$ Eq. 2.21

where:

hii = leverage

n= number of observations

x_i = Observation i

$\bar{x}$ = Average of Observations

and

SS = Sums of squares

$$SS = \sum_{1}^{n} \left(x_i - \hat{x}\right)^2$$ Eq.2.22

$$CookD = \frac{\left(x_{sim} - x_{obs}\right)^2 hii}{Var(x) * (1 - hii)^2}$$ Eq. 2.23

Where: CookD = Cook's D statistic

Xsim = simulated value

Xobs= observed value

Var(x) = Variance of x

An observation point value that has a hii>3/n is consider to have high leverage. However, as shown in Figure 2.19, a point can have a high leverage but if it does not fall outside the regression line, the point is not considered to have a high influence. The statistic Cooks'D can be used to test if a point has high influence. A point is considered to have high influence if CookD>F(p+1,np) where p is the number of parameters (Hesel and Hirsh 1992). An observation point was considered an outlier if it has a high leverage or a high CookD value and when graphically displayed does not seem to be correlated to neighboring observation point values. We considered all the points individually and take them as points deemed to be non-representative of the truth due to instrument or other identifiable error. Only 3% of the groundwater observations were considered to be outliers and were not included in the statistics used for model evaluation.

For surface water data an analysis of outliers was not perform. Ideally the model area will have a surface water gauge at the inflow and outflow of each stream segment and every inflow into the model domain. At the moment the WWBWC is expanding the number of surface water gauges and the evaluating the existing ones. OSU and the WWBWC selected the gauges use for model validation for which the statistics of Nash and Sutcliffe (1977) were calculated. For the rest of the locations, the sum of square error is presented in this section to give the model developer and extra info for calibration showing discrepancies between diversion, runoff calculations and aquifer interaction. However the statistic cannot be used by itself for model evaluation since the user controls the observation to which the simulation is compared.

The statistical measures for model evaluation are divided into groundwater and surface water. During model calibration, for the estimation of parameter sensitivities and parameter estimation, a series of weights were estimated for each type of observation (section on model calibration in this thesis). Weighted residuals can combine model misfit for surface and groundwater observations since the associated weights resulting in dimensionless numbers. Weighted residuals are also presented since they provided a useful tool for model evaluation; large values of weighted residuals indicate a poor model fit. For surface water, evaluation of model fit the coefficient of efficiency Eff and R by Nash and Sutcliffe (1970) was employ. The coefficient R represents the fraction of the variation in the observed values explained by the simulation model. The coefficient Eff represents the percentage of fit to a linear relation of the observed values. The correlation coefficient can vary between-1 to 1 showing perfect inverse and perfect direct relationships between measured and modeled values.

$$Eff = \frac{\sum_{t=1}^{n}(x_i - \bar{x})^2 - \sum_{t=1}^{n}(\hat{x} - x_i)^2}{\sum_{t=1}^{n}(x_i - \bar{x})^2} \qquad R = \sqrt{Eff} \qquad \text{Eq. 2.24 and 2.25}$$

Where:

x_i=measured value

$\hat{x}$ = estimated value

$\bar{x}$ =mean value of x_i

n= total number of observations

The statistical evaluation of goodness of fit to observations for the groundwater component of the model includes de following statistics: Sums of Square Errors (SS), Variance (Varx), Standard Deviation (SD), and as mention before, the weighted squared residual is presented for both surface and groundwater data. The formulas used to calculate this statistics are presented next, the units for the SS and Varx are m^2 and for SD are m. Table 2.11 presents the statistics calculated for the calibrated model.

$$Varx = \frac{1}{n-1}SS \qquad \text{Eq. 2.26}$$

$$SD = \sqrt{Varx} \qquad \text{Eq. 2.27}$$

Table 2.11 Statistical measurements of model goodness of fit to observations

Statistic	Value
Groundwater	
Sum of Squares Error (SS)	14,807
Total number of observations (n)	1,949
Variance (Varx)	8
Standard Deviations (SD)	2.76
Surface Water	
Nash and Sutcliffe (Eff)	96%
Nash and Sutcliffe (R)	98%
Sum of Square Error for selected locations	$5.9X10^6$
Sum of Square Error for all gauges	$3.5X10^8$
Weighted Objective function GW + SW	140,004

The graphical analysis of overall model fit includes the display of standard deviation for each observation in the model domain. Using GIS the map (figure 2.20) can help visualize locations systematic miss fit. If the model presents spatially- correlated error is an indication of poor model representation. This can be due by omitting or miss representing a local hydrological process. Figure 2.20 shows the model error distribution across the model domain. In this figure we can see that across the model area the standard deviation average 2 meters. However, for wells around the HBDIC artificial aquifer recharge project and wells close to the head of springs, the model standard deviation increases to 2.8 m. A second method for graphical analysis includes the time series comparison between simulated and observed values at a single location. This type of graphical analysis is useful to evaluate temporal trends of model fit. Figures 2.21 to 2.23 show selected observation wells. Figures 2.24 and 2.25 show model fit to surface gauges.

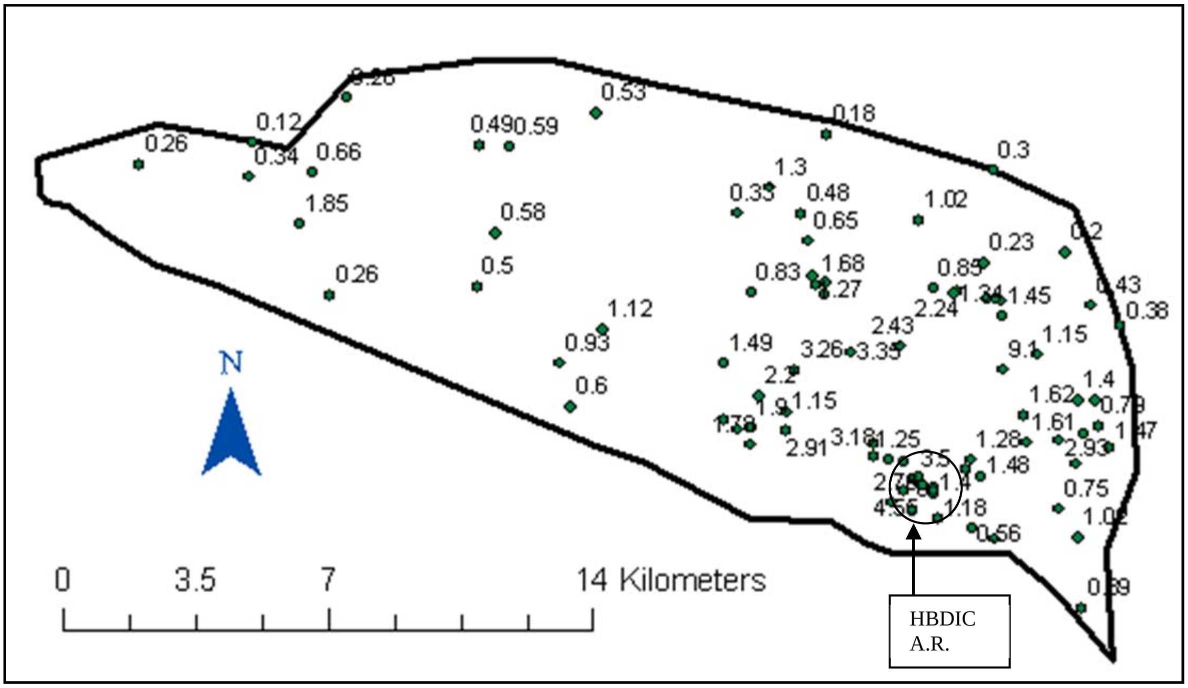


Figure 2.20 Standard deviation at all monitored observation wells.

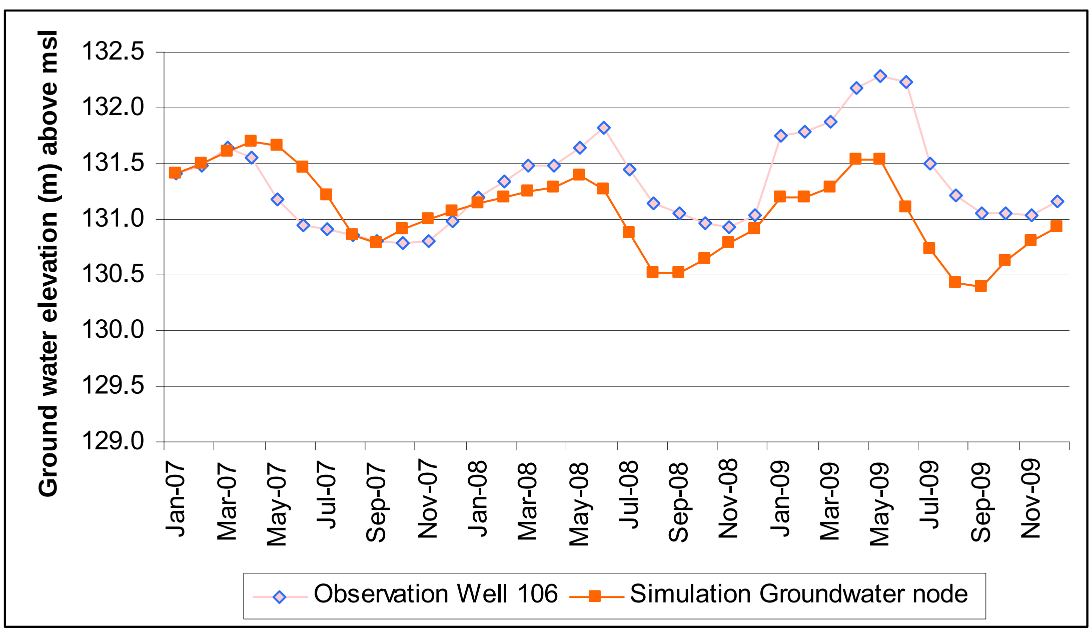


Figure 2.21 Temporal model fit to observation at groundwater well 106.

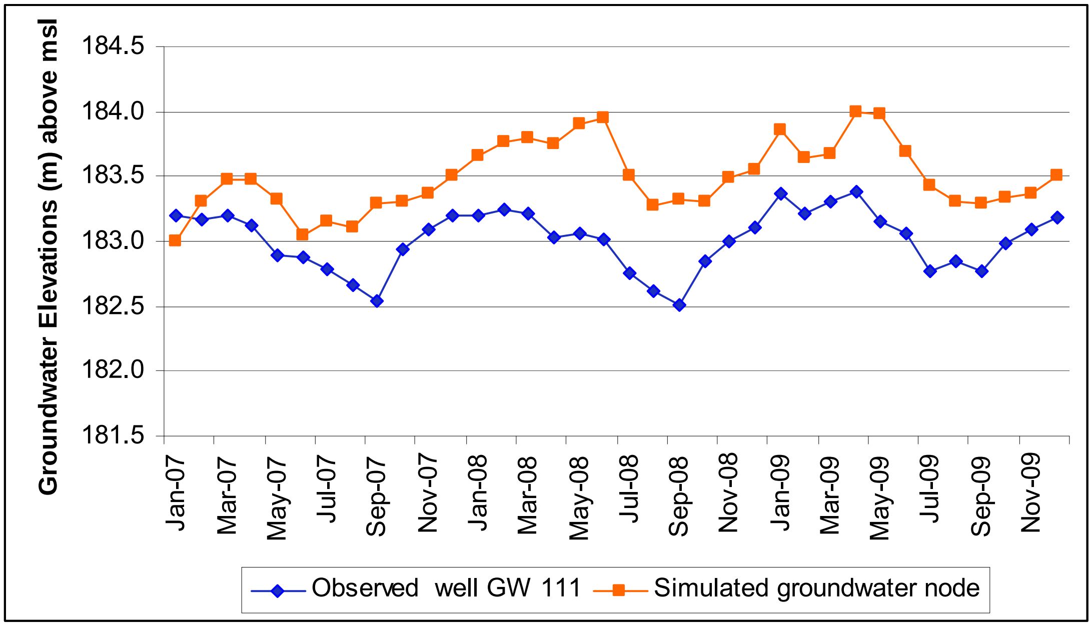


Figure 2.22 Model fit to observation at groundwater well 106.

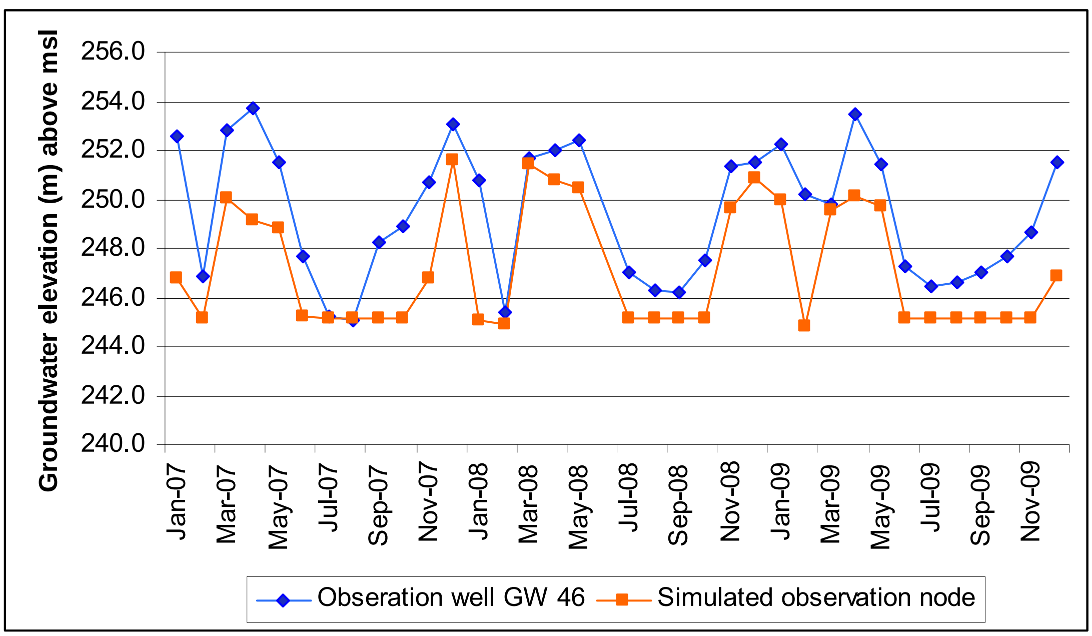


Figure 2.23 Observation node 140 meters away from the HBDIC artificial aquifer recharge project.

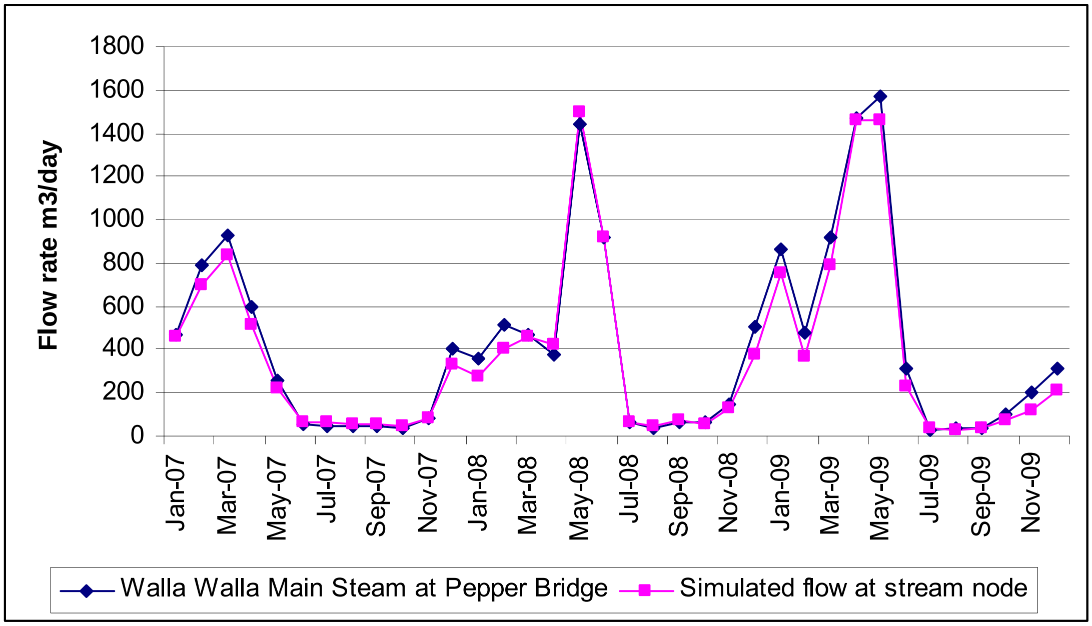


Figure 2.24 Model simulation fit of the Walla Walla River.

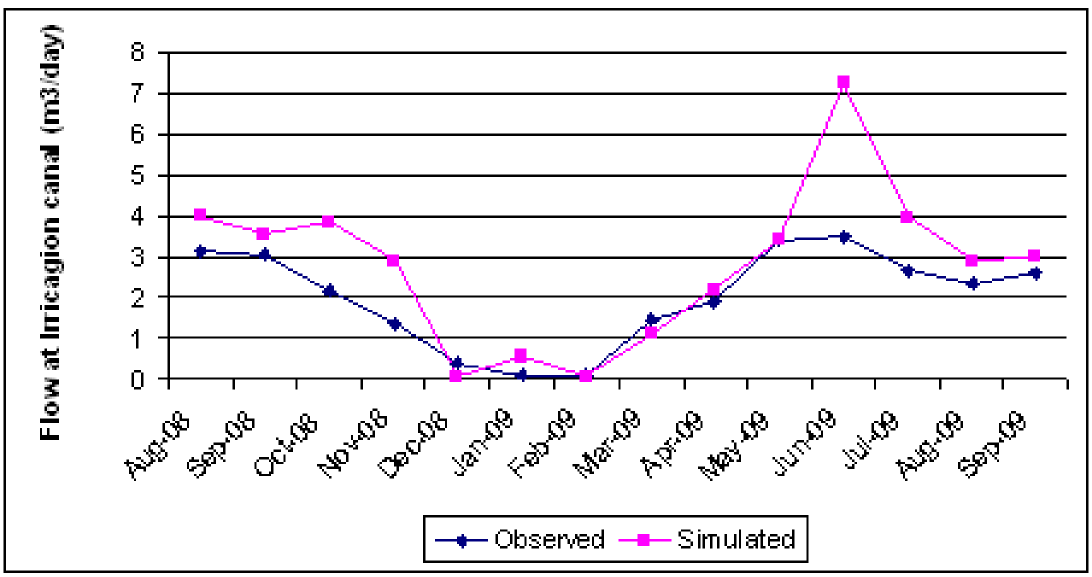


Figure 2.25 Model simulations at Little Walla Walla Canal.

The evaluation of the model goodness of fit to observation was described in this section. The analysis includes statistical and graphical procedure. The statistical procedure presents the Nash-Sutcliffe evaluation for surface waters and the model standard deviation for groundwater. The statistical analysis provides information that can be used to compare different models. The overall standard deviation of the model is 2.8 meters. Graphical analysis was used to locate model areas of poor fit. If the model presents a spatial-correlated error it is an indicative of model bias. The map of standard deviation distribution across the model domain in the well location showed that around the aquifer recharge project of the HBDIC the model statistics performs the worst. A close examination also shows that for wells close to the area of the head of the springs the model under predicts the groundwater elevation. For the rest of the model area the model SD is less than 1.5 meters. Considering the overall standard error with the wells next to the springs and the artificial aquifer recharge project thee research consider the model SD an acceptable model error range considering that the source of the information for aquifer thickness (interpolated well logs) and surface elevation (DEM) has a 15 to 20 meters of estimated standard error. Other sources of model uncertainty are evaluated in the estimation of water budgets.

The surface water evaluation of model fit to observation employs selected gauges for the statistical evaluation. The future Masters in Science thesis of Jake Scherberg, responsible for incorporating surface inflows and initial diversions estimates into the model will explain in detail the selection process for surface gauges. For the purpose of this research we observed that the model simulation fits to the Walla Walla River and the selected gauges for irrigation canal EFF of 9.1. However, the model when evaluated by graphical analysis shows that model has a better fit to low flows and poor fit to high flows especially for irrigation canals and springs (Figures 2.24 and 2.25). Due to the low number of reliable gauges in springs this analysis lacks a formal evaluation of model fit to spring flows.

2.7.2 Water budget estimation with uncertainty analysis

The estimation of the water budget is critical for the effective management of the regional water resources. One of the outputs of this hydrological model effort is the quantification of flows between the interconnections of the different components of the hydrological cycle. This section of the research will review the water budget calculation from the calibrated IWFM model. Results of the estimated water budget will be presented with their degree of uncertainty bounded by the range of possible input values and the range of results from simulations under the input values.

A water budget can be thought of as a mass balance of regional hydrological components. The IWFM modeling software has the capability of calculating the fluxes per sub-region of the model generating water budgets for the following hydrologic components: streams, root zone moistures, groundwater, and land water use demands. A detailed description of this calculation can be found in the IWFM user manual available free on-line through the California Department of Water Resources found on their webpage in the IWFM section.
http://baydeltaoffice.water.ca.gov/modeling/hydrology/

In this research, for simplicity, we combined the IWFM budget outputs to form a single equation of water budget. Eq. 2.28 is the water budget equation used in this research modified from Dingman (2002).

$$P + GW\ in + Qin - (Qout + ET + GWout) = \Delta S \qquad \text{Eq.2.28}$$

Where P is precipitation, GW is the net inflow of groundwater through the model boundaries, Q is net surface water flows and ΔS is change in storage. The units of the variables are m^3/yr calculated as the cumulative daily flows average of the three years of simulation. Given of the informative nature of the variables, we also include the following variables in the tables presented for water budgets: deep percolation which is the actual amount of water recharging the aquifer and groundwater pumping from the gravel aquifer and surfaces water diversions which is the amount of water applied to satisfy the land-water use demand. Water from the basalt aquifer was not simulated in this research and was considered to be an input of water coming from outside of the model area. The rate and surface distribution of the pumped basalt water (section 2.5.4) was kept constant in these uncertainty scenarios. To illustrate the usefulness of these incorporated variables, Figure 2.26 presents a plot comparing the evaporative demand to the supply water by surface (diversions) and the supplied water supplied by the aquifer (groundwater pumping).

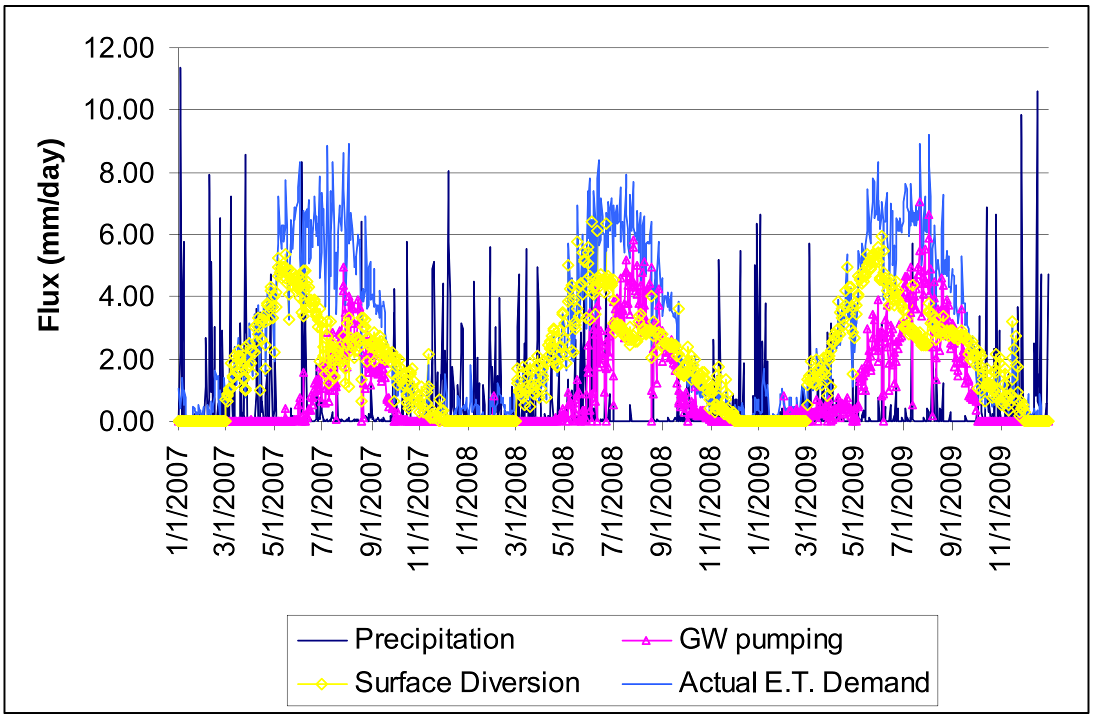


Figure 2.26 Time series comparison of actual evaporative demand and the supply of surface water diversions, groundwater pumping and precipitation.

Figure 2.26 presents the time series supply and demand water for the total model domain. One may observed from this graph is that 85% of the annual precipitation occurs over the winter months where the evaporative demand is at its lowest point. The evaporative demand is then supplied by surface water diversions (66%) and groundwater pumping (34%). Another way of visualizing results that facilitates the analysis of water budgets is by calculating the cumulative annual flows. Figure 2.27 shows the percentage of water supplied by surface diversion and groundwater.

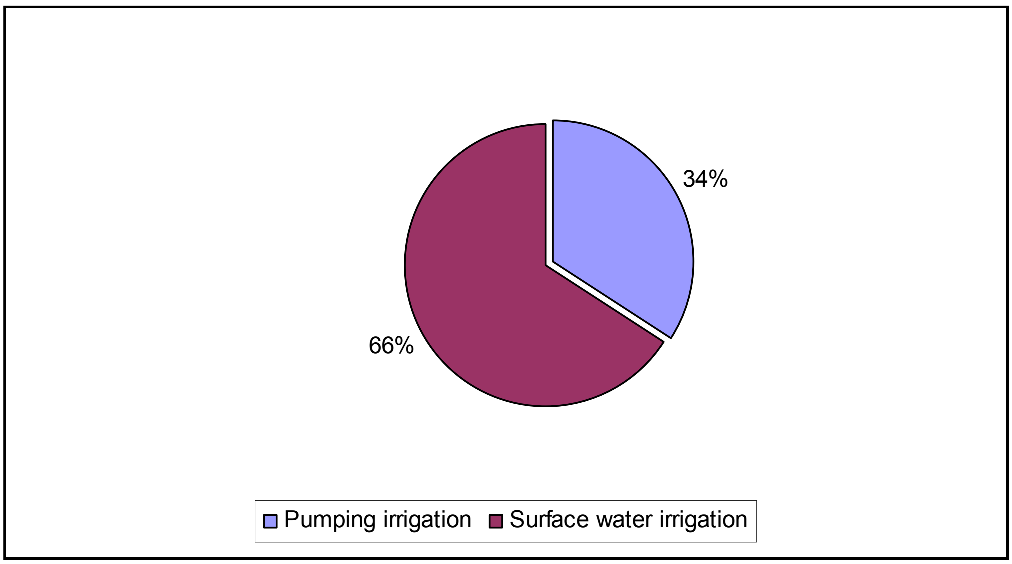


Figure 2.27 Cumulative averages comparing the source of surface water demand.

Hydrological models that lack an analysis of their degree of uncertainty cannot be used to support water management decisions and are often doubted and dismissed because their lack of transparency (Ewen and Parkin 1996). This research, as a part of its validation, presents regional water budgets and predictive model runs with their degree of uncertainty. Hill (2007) proposes evaluating model uncertainty by running hypothetical scenarios under a range of possible input parameter values. Model uncertainty is then evaluated by the width of the bound and the changes in the resulting simulated values. In the water budget calculations, the sources considered for uncertainty analysis include: precipitation, reference ET, surface water inflows, leakage from irrigation canals, and ratio between surface water diversions and ground water pumping to supply water demand. Uncertainty of the water budget estimation will be presented after evaluating individual variable simulations with a new possible range of values compared to the estimations by the calibrated model. At the end of each variable analysis, brief concluding remarks will be presented.

Table 2.12 Groundwater budgets uncertainty variable: Precipitation
Cumulative million m^3/year average from 2007-2009

Source of Uncertainty / Budgets Estimates	Simulation runs made with Precipitation from WWBWC stations A	Simulation runs made with Precipitation from Prism stations B	Ration of A/B (%)
Precipitation	40.8	34.7	85.0%
Actual ET	182.3	181.4	99.5%
Deep Percolation	51.2	48.7	95.0%
Pumping	60.3	61.5	102.0%
Model surface inflows	680.8	680.8	100.0%
Model surface outflow	633.8	631.9	99.7%
Diversion	122.0	124.4	102%
Change in Groundwater Storage (In-Out)	-30.1	-28.7	95%
Overall GW Budget discrepancies IN - OUT	-0.6	-0.6	100%
Model Fit to groundwater observations, Standard deviation (SD) meters	2.76	2.76	No change
Model Fit to surface flows Nash-Sutcliffe (EFF)	96%	96%	No change

Concluding remarks of uncertainty from precipitation estimates; the calibrated model utilizes the precipitation data gathered by the WWBWC from local stations. Another possible source of reliable information is the average monthly precipitation from PRISM (Parameter-elevation Regressions on Independent Slopes Model), a climate mapping system, developed by Dr. Christopher Daly from the Climate group at Oregon State University. PRISM is the USDA's official climatological data recognized world-wide as the highest-quality spatial climate data sets currently available (Daly 2007). The precipitation estimates from PRISM for the Walla Walla Basin are 85% lower than estimates from the local stations. Precipitation in the Walla Walla Basin doesn't represent a major component in the water budget. As a comparison, precipitation is about 9% of the total inflow of water. The major change of reducing precipitation by 15% is a reduction on deep percolation by 5%. The model statistics of model fit do not change.

Table 2.13 Groundwater budgets uncertainty for: Evapotranspiration
Cumulative million m^3/year average from 2007-2009

Source of Uncertainty / Budgets Estimates	Reference ET from WWBWC stations A	Reference ET from Agrimet Stations B	Percentage of Change A/B (%)
Precipitation	40.8	40.8	100%
Actual ET	182.3	211.9	116%
Deep Percolation	51.2	40.8	80%
Pumping	60.3	66.7	111%
Model surface inflows	680.8	680.8	100%
Model surface outflow	633.8	622.5	98%
Diversion	122.0	100.9	83%
Change in Groundwater Storage (In-Out)	-30.1	-31.9	94%
Overall GW Budget discrepancies IN - OUT	-0.6	-0.6	100%
Model Fit to groundwater observations, Standard deviation (SD) meters	2.77	2.82	0.05 (m)
Model Fit to surface flows Nash-Sut (EFF)	96%	97%	+1%

Remarks regarding evapotranspiration uncertainty: the calibrated version of the model utilizes the reference evapotranspiration from the information collected by the WWBWC. The reference ET_o from these stations can be compared to the estimates made by the Pacific Northwest Cooperative Agricultural Weather Network (AGRIMET) managed in cooperation with the Bonneville Power Administration and the U.S. Bureau of Reclamation. The

AgriMet network consists of over 70 agricultural weather stations located throughout the Pacific Northwest. The closest station from the model area is the station located at Kennewick, WA, 40 km away from the model area. The reported values for reference evaporation from this station are 50% higher than the reference evaporation estimated from the local stations. The simulated uncertainty scenario ran the simulation with reference ET_o 50% higher, resulting in an increase of actual evaporation rate of 16% higher than from the calibrated value. This increase in evaporation generated a decrease in net deep percolation of 20%. The model increases the standard error by 5 cm and improves the model surface statistic by 1%. The actual evaporation rate is a critical component in the water budget and simulation modeling calculations. Reference evaporation in comparison to the surface inflow water is 4.5%. The major source of uncertainty in the estimation of actual evaporation is the crop coverage area. During the review made by the WWBWC of the work performed by the external consultant, it is estimated that crop coverage has a coefficient of variation of 50%. This analysis did not explore uncertainty in crop coverage or irrigation practices.

Table 2.14 Groundwater budgets uncertainty of Surface Inflows

Cumulative million m3/year average from 2007-2009.

Source of Uncertainty / Budgets Estimates	Original Version A	Maximum Surface Inflows B	Minimum Surface Inflows B	Percentage of Change A/B (%)
Precipitation	40.8	40.8	40.8	100%
Actual ET	182.3	182.3	182.3	100%
Deep Percolation	51.3	51.4	51.2	100%
Pumping	59.6	59.2	60.1	98%
Model surface inflows	680.8	841.9	561.4	124%
Model surface outflow	652.5	809.6	536.5	128%
Diversion	126.0	126.8	125.3	104%
Change in Groundwater Storage (In-Out)	-30.1	-30.7	-29.8	97%
Overall GW Budget discrepancies IN - OUT	-0.6	-0.6	-0.6	100%
Model Fit to groundwater observations, Standard deviation (SD) meters	2.77	2.75	2.84	0.9 (m)
Model Fit to surface flows Nash-Sutcliffe (EFF)	96%	83%	92%	13%

The surface inflows to the model area include: The Walla Walla River main stem, Birch Creek, Yellow Hawk Creek, Stone Creek, Garrison Creek, Mill Creek, Dry Creek, Upper Dry Creek, Upper Pine Creek, The Touchet River, and Woodward Canyon. The uncertainty scenarios runs were made with a 35% increase and a 35% decrease of all inflows excluding the Walla Walla River, since its gauges are considered to be well calibrated. The surface inflows increase made a 0.9 m change in the groundwater SD and a 13% change in the model surface Nash-Sutcliffe eff. The overall increase in surface water inflows was 24%.

Table 2.15 Groundwater budgets of Leakage from irrigation canals

Source of Uncertainty / Budgets Estimates	Original	Minimum Leakage rates A	Maximum Leakage rate B	Percentage of Change A/B (%)
Precipitation	40.8	40.8	40.8	100%
Actual ET	182.3	179.5	182.3	98%
Deep Percolation	51.3	49.6	50.8	98%
Pumping	59.6	55.2	59.4	93%
Model surface inflows	680.8	680.8	680.8	100%
Model surface outflow	652.5	648.6	649.3	99%
Diversion	126.0	124.4	126.0	98%
Change in Groundwater Storage (In-Out)	-30.1	-30.9	-30.1	100%
Overall GW Budget discrepancies IN – OUT	-0.6	-0.6	-0.6	100%
Model Fit to groundwater observations, Standard deviation (SD) meters	2.76	2.91	2.79	0.15 (m)
Model Fit to surface flows (EFF)	96%	97%	95%	2%

Concluding remarks regarding uncertainty of the leakage from irrigation canals: the conductance rate is calculated by the thickness and hydraulic conductivity of the stream bed. If the conductance is higher, the stream segment is able to transmit (lose or gain) water faster to the aquifer. Two uncertainty scenarios were run, one with a 50% increase of all the conductance rates and a second run with a 50% decrease of the conductance rates. The major degree of observed change is in the pumping rate which decreased by 7% between the minimum and maximum leakage rates runs. For the overall change in leakage, the model outflow change is 1% of uncertainty and 2% in total model deep percolation.

Table 2.16 Uncertainty analysis abstract for water budget estimates.

Budget variable	Percentage of Range of possible input values	Model output Uncertainty as percentage of the range of possible values	Min. value Million m^3/yr.	Max Values Million m^3/yr.	Percentage of uncertainty in relation to the total surface water inflows
Precipitation	15%	15%	34.7	40.8	0.6%
Reference Eto	50%	16%	182.3	211.9	4.5%
Deep percolation	45%	20%	40.8	51.3	0.7%
Pumping	20%	12%	60.3	67.2	1.2%
Diversions	20%	12%	122	137	2.3%
Leakage	50%	2%	0.2	3.0	2.4%
Surface Inflows	30%	24%	680.8	841.9	24%

Overall budget uncertainty concluding remarks

The IWFM-Walla Walla model is used as a means to quantify the hydrological resources of the basin. Therefore, the water budget is presented with its degree of uncertainty from model simulation runs of possible input values. Results show that the major source of uncertainty into the model is presented from the un-gauged surface inflows. The estimated uncertainty from this source is 40%. Future research by the WWBWC will evaluate the inflows to the model domain. The model can then be easily updated by the watershed council since council personnel learned and practiced that scenario in the model workshop given by the author. Actual evaporation is the second biggest uncertainty variable in the estimated water budget. Actual evaporation is estimated by the measured potential evaporation, crop coverage and applied water. The coefficient of variation of input values is as high as 50%. However, with the advantage of the local ET_o stations and the comparison to previous estimations, the model uncertainty is considered be within 16% of actual values.

The water budgets estimation shows that surface water diversions from the Walla Walla River to the series of irrigation canals accounts for 56 to 68% of the evaporative demand. The remaining demand is satisfied by groundwater pumping, which accounts for 32 to 44%. The model uncertainty of the ratio between pumping and surface diversion is 12% of the observed range (68 – 56%). Most of the aquifer recharge occurs through the unlined irrigation canals and the excess water applied during irrigation since precipitation only accounts by 10% of the deep percolation. Deep percolation overall is 40% of the water diverted from the Walla Walla to the irrigation canals. Previous estimates (Petrides 2008) calculate that the irrigation canals lose from 15 to 35% of their water to recharge to the gravel aquifer. The recharge of the aquifer by the unlined irrigation canals is explored in detail in the next section 3.6 entitled “Examples of applications of the Walla Walla hydrological model for decision support”. The leakage uncertainty scenarios showed that only about 2% of uncertainty of these rates is due to the uncertainty in the conductance estimation of the irrigation canals. The biggest uncertainty (40%) is found in the water distribution carried by the system of irrigation canals. Future suggested research includes a survey of the network of diversions and irrigation canals throughout the model area so the amount of water carried by each section of canals is known with a higher level of certainty.

The water budget analysis shows that in all the uncertainty modeled scenarios, the overall groundwater change in storage is 30 million m^3/year including groundwater flow from boundaries and discharge from springs. Another useful comparison is the estimated amount of groundwater pumped and the amount of deep percolated water to be 9.1 million m^3/year. Assuming a porosity of 30%, for the model area of 231 km^2, the negative change in storage is equivalent to a drawdown loss of 13 cm/year in water level elevation of the aquifer (Eq. 2.29).

$$\frac{Changeinstorage(m^3 / year)}{Porosity * ModelArea(m^2)} = aquiferdrop(m / year) \qquad \text{Eq. 2.29}$$

For comparison of the estimated drop rate in the aquifer, we examine the water level of the oldest monitored wells in the area (Figure 2.28). From eleven monitored wells that have been recorded for up to the past 70 years, we can observe the declining levels of the aquifer (with the irrigation canals un-lined) to be 5 meters. This loss of aquifer storage in average over the past 70 years is equivalent to a 7 cm/year drawdown in groundwater elevations. Section 2.8.2 explores the volume of water necessary to restore aquifer levels. With the variability observed in groundwater elevations over the past 70 years, the estimated average of 13 cm/year for the simulation period compares well to the historic drop in groundwater elevation. The next section, 2.8.1, explores the increased aquifer dropping rate in the management scenarios of lining the irrigation canals.

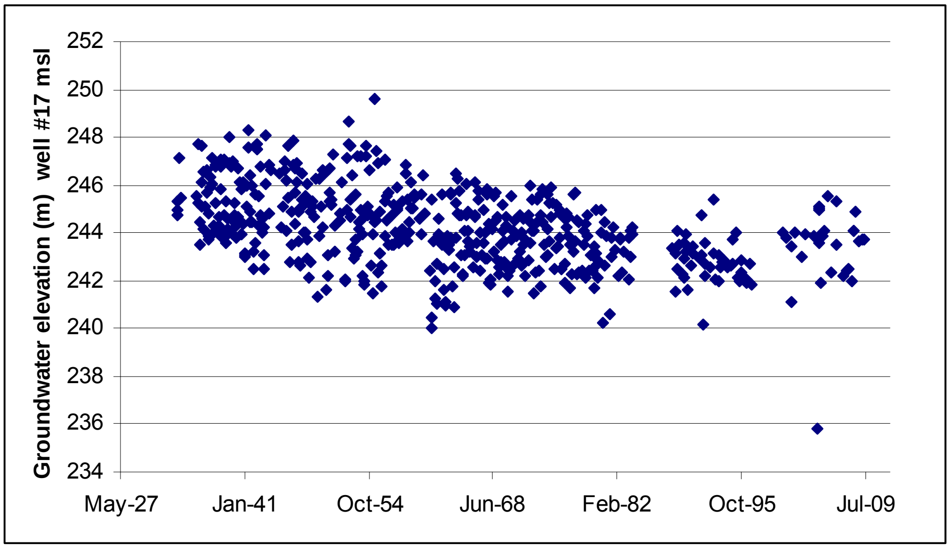


Figure 2.28 Long time monitored groundwater elevations in the basin

2.8 Model results and application examples.

The IWFM-Walla Walla model was structured with the purpose of evaluating water management scenarios and in that sense, served as a tool for decision support. The model is not useful by itself without the incorporation of the final users' judgment and experiences. These other forms of information that accompany the model outputs transform a simple computer code into a decision support tool. Gasching (1981) describes the benefits of including other forms of information such as rules of thumb, intuition, and empirical knowledge to augment the predictive power of a decision support system. During the last hands-on training workshops developed during the model transfer (section 2.7, Appendix 1) to it final users, the most important water management scenarios questions were identified. The WWBWC, in cooperation with OSU, developed the IWFM model units to simulate water management scenarios such as lining irrigation canals and evaluating locations for managed artificial aquifer recharge. This section demonstrates how these units were developed, utilizing the local field experience of WWBWC personnel. Results are presented with their degrees of uncertainty from the possible range of input values.

The scenarios to be evaluated with the IWFM model are the activities related to "water conservation irrigation efficiencies" proposed from the COE (2010) report and local irrigation districts. These include the Impacts on groundwater from lining of irrigation canals and the evaluation of locations of infiltrating basins for artificial aquifer recharge to mitigate the unwanted effects of lining canals. Due to time and budget constraints, this research couldn't evaluate all the locations proposed by the WWBWC for artificial aquifer recharge. The purpose of these scenarios is to demonstrate the IWFM-Walla Walla model capabilities as a tool to be incorporated in the decision support system in conjunction with the expert evaluation and experience of local hydrologists and water technicians.

2.8.1 Effects of planned irrigation efficiency practices

The need for hydrological models comes from the complex interrelations between different hydrological components in the basin. The model allows us to analyze the hydrology of the basin as a system. This section evaluates the effects that lining irrigation canals would have on the groundwater elevations and flow in springs. Two management scenarios will be evaluated. The first scenario assesses the impacts from lining all the irrigation canals in the basin. The second scenario will look at the impacts from the planned lining of irrigation canals proposed for the 2011 irrigation season. The irrigation canal segments were identified by the WWBWC during an IWFM model-transfer workshop .Figure 2.29 shows all the possible canal segments for lining. Figure 2.30 shows the proposed irrigation canals segments for lining.

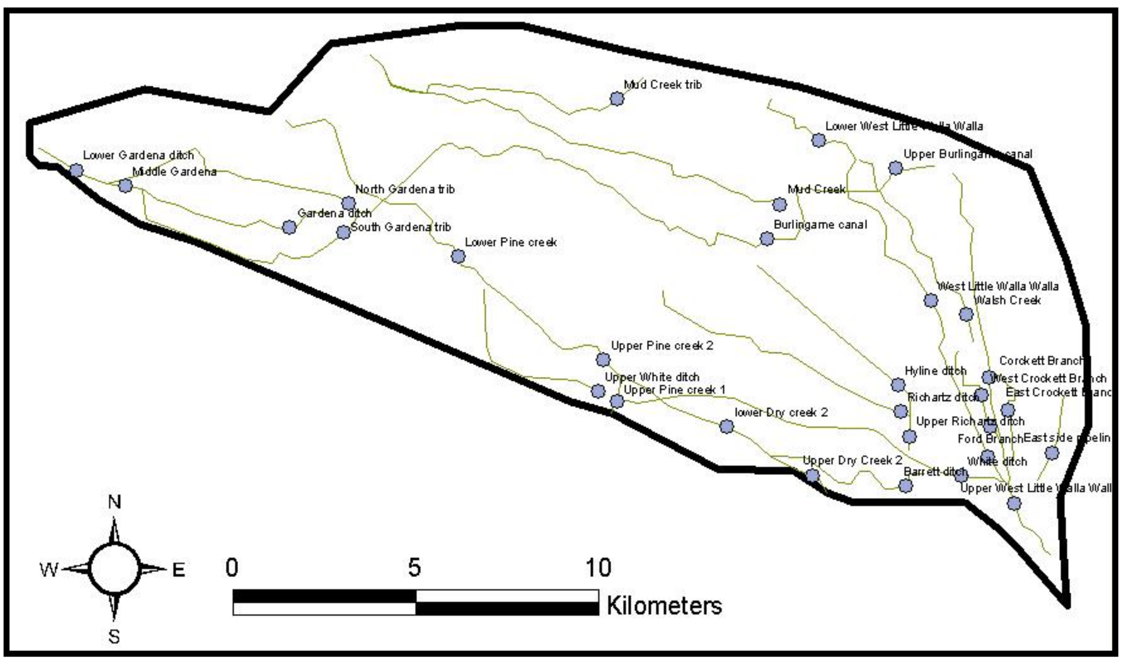


Figure 2.29 All canals possible for lining evaluated in the first scenario

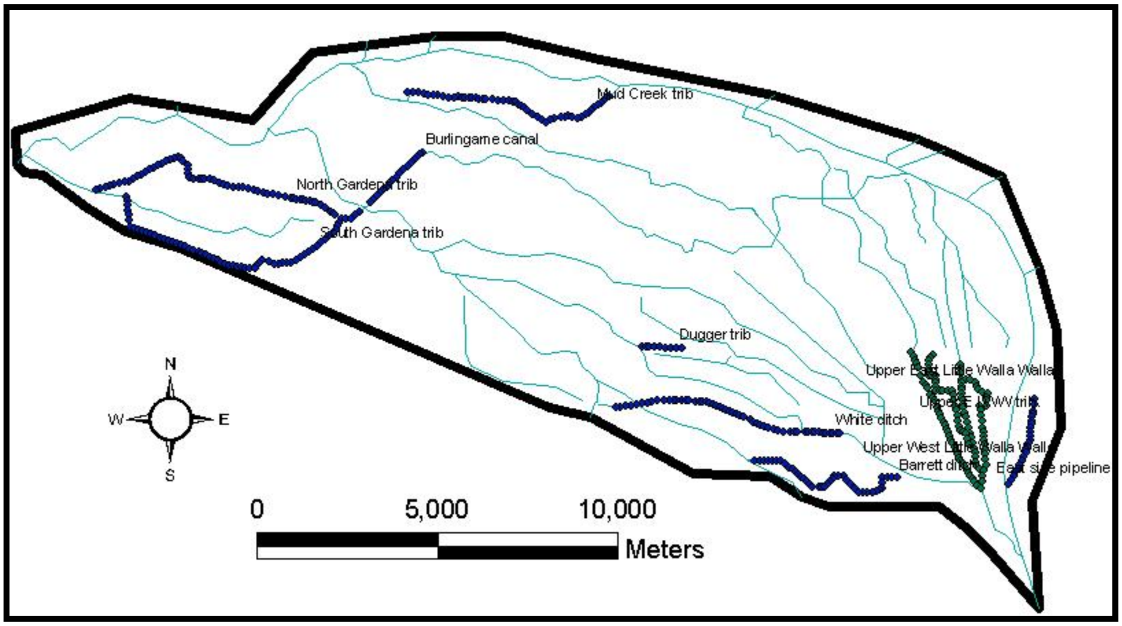


Figure 2.30 All proposed canals to be lined identified by the WWBWC.

Results from the calibrated water budgets present the amount of water currently recharged by each segment of unlined irrigation canals. The flow interaction with the aquifer is calculated by IWFM as the difference in head from the simulated water table elevation and the elevation head at the surface canal multiplied by the individual canal leakage coefficient. Water flow gains or losses from the irrigation canals are presented from model runs under the calibrated, minimum and maximum possible values of stream conductance expected in the field. This estimated water recharge is shown per segment in table 2.17 as a yearly average from the three years of simulation.

Table 2.17 Stream/Aquifer interactions; Cumulative leakage flow in a year (m^3/yr.)

Canal Name	Canal leakage Calibrated	Canal leakage Min.	Canal leakage Max.
Upper West Little Walla Walla	$3.5 X10^6$	$1.8 X 10^6$	$5.2 X 10^6$
Little Walla Walla Ditch	$5.2 X 10^4$	$2.7 X 10^4$	$6.9 X 10^4$
West Little Walla Walla	$6.3 X 10^4$	$6.3 X 10^3$	$2.1 X 10^4$
Upper Dry Creek 2	$3.5 X 10^5$	$1.8 X 10^5$	$5.2 X 10^5$
Lower Dry Creek 2	$5.2 X 10^5$	$2.7 X 10^5$	$7.9 X 10^5$
Upper White Ditch	$7.9 X 10^4$	$4.4 X 10^4$	$1.0 X 10^5$
Richartz Ditch	$1.4 X 10^5$	$6.9 X 10^4$	$2.0 X 10^5$
Upper Burlingame Canal	$9.6 X 10^5$	$3.8 X 10^5$	$1.5 X 10^6$
Burlingame Canal	$1.7 X 10^4$	$9.3 X 10^3$	$2.4 X 10^4$
Gardena Ditch	$2.5 X 10^4$	$1.6 X 10^4$	$2.7 X 10^4$
Lower Gardena Ditch	$1.3 X 10^5$	$7.4 X 10^4$	$1.7 X 10^5$
North Gardena Tributary	$3.3 X 10^5$	$1.7 X 10^4$	$4.7 X 10^6$
South Gardena Tributary	$1.7 X 10^3$	$9.3 X 10^4$	$2.4 X 10^3$
Mud Creek Tributary	$2.4 X 10^5$	$2.0 X 10^5$	$2.5 X 10^5$
Dugger Creek Tributary	$1.5 X 10^3$	$7.4 X 10^4$	$2.2 X 10^3$
White Ditch all	$2.5 X 10^5$	$1.3 X 10^5$	$3.5 X 10^6$
Barret Ditch	$1.0 X 10^4$	$5.5 X 10^5$	$1.6 X 10^4$
Ford Branch	$1.5 X 10^5$	$8.8 X 10^6$	$1.9 X 10^5$
West Crockett	$7.9 X 10^5$	$4.1 X 10^5$	$1.1 X 10^4$
Crockett Branch	$2.5 X 10^4$	$1.3 X 10^4$	$3.3 X 10^5$
East Crockett	$6.0 X 10^6$	$3.8 X 10^6$	$8.8 X 10^6$
Total amount of water recharged to the aquifer	$1.4 X 10^7$	$7.8 X 10^6$	$1.9 X 10^7$
% of Total amount diverted from the Walla Walla River	28%	17%	37%

2.8.2 Evaluation of potential site for artificial aquifer recharge basins

To mitigate the un-wanted the reduction in aquifer recharge by lining the irrigation canals, the US Army Corps of Engineers is evaluating the feasibility of artificial aquifer recharge to be used in combination with their propose irrigation efficiency program (USACOE 2010). They estimate that 1200 (m^3) 9,200 AF/year of shallow aquifer recharge using basins of 1 or 2 acres in size located west of the east Crockett branch of the Little Walla Walla River would have minimal impact and could benefit the groundwater resources. The goal of the USACOE 2010 proposal is to obtain up to 4.3 m^3/sec (150 cfs) in the lowest flow reach of the Walla Walla at TUM-a-LUM Bridge during May/June and up to 1.4 to 2.8 m^3/sec (50 to 100 cfs) from July through September**.** The TUM-a-LUM Bridge of the Walla Walla River is shown in Figure 3.36.

Location A in Figure 2.31 is the suggested location in the COE 2010 plan as identified by the WWBWC. This project evaluated the effects of location A and of location B as an alternative possible location to demonstrate the benefits from being at a greater distance from a surface water boundary, in this case the Walla Walla River. Both locations have the same surface area and are simulated under two recharge flow rates of 0.3 and 0.8 m^3/sec (10 and 30 cfs). The recharge flow rates simulated are based on water availability and the recharge rates observed at the HBIDC pilot project for same simulation period. The high simulation rate gives a three-fold increase of the HBDIC project. Table 2.19 shows the total amount of water recharged and the simulated surface flows in the Big Spring and the Walla Walla River at the locations shown in Figure 2.31.

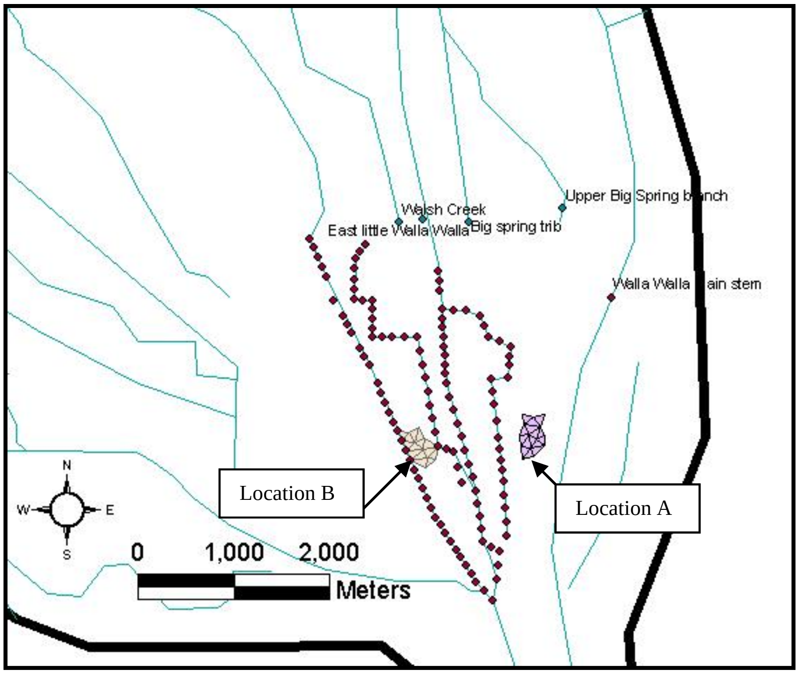


Figure 2.31 Alternative locations of infiltration basins to minimize the unwanted effects of lining irrigation canals. Proposed canals to be lined are shown with red dots.

Table 2.17 Water budget estimates following scenario 1 and 2

Simulated Scenario	Big Spring Flow Ground water gained (m^3/year)	Walla Walla River Flow (m^3/year)	Total amount of water recharge per season (m^3)
Calibrated model; Current conditions	2.72 X 10^5	1.29 X 1 0^8	No water recharge
Lining all canals: scenario #1 Figure 3.31	2.38 X 10^5	1.28 X 10^8	No water recharge
Lining proposed canals; Scenario #2 figure 3.32	2.66 X 10^5	1.29 X 10^8	No water recharge
Lining proposed canals with recharge activities at Location A; Low flow of 0.3 m^3/sec	2.72 X 10^5	1.29 X 10^8	3.59 X 10^6
Lining proposed canals with recharge activities at Location A; Low flow of 0.8 m^3/sec	2.74 X 10^5	1.30 X 10^8	8.30 X 10^6
Lining proposed canals with AR at Location B low flow of 0.3 m^3/sec	2.77 X 10^5	1.29 X 10^8	3.59 X 10^6
Lining proposed canals with AR at Location B high flow 0.8 m3/sec	2.90 X 10^6	1.29 X 10^8	8.30 X 10^6

The water budget estimates the total amount of water recharged from the unlined irrigation canals to be 1.4 X 10^7 m^3/yr. (uncertainty range from 7.8 X 10^6 to 1.9 X 10^7). The total amount of recharged water by the unlined irrigation canals is equivalent to 28% (range from 17% to 37%) of the total water diverted from the Walla Walla River to be carried by irrigation canals. In other words, the irrigation canals “lose” on average 28% of the water that they carried to recharge the unconfined gravel aquifer. From personal communication with representatives from the irrigation districts and the WWBWC, this percentage of irrigation canal loss seems to be a reasonable estimate.

If the lining of all the irrigation canals occurs without managed artificial aquifer recharge, the total amount of decreased aquifer recharge is equivalent to increase the total groundwater pumping by 23%. Applying Eq. 2.21 assuming a porosity value of 30%, the total drop in aquifer levels is 20 cm/yr. To mitigate this unwanted effect of the proposed irrigation efficiencies, management artificial aquifer recharge projects is proposed to be used in combination with proposed lining of the irrigation canals. Section 2.7.2 explores the historic aquifer dropping rate over the past 70 years and the estimated amount necessary for restoration.

Two locations of artificial aquifer recharge were evaluated using the Walla Walla-IWFM Model. Location A is proposed by COE (2010) to mitigate the reduced recharge by the planned lining of irrigation canals (Figure 2.30 and Figure 2.31). Location B was chosen to serve as an example of a location to be at a greater distance from the Walla Walla River. Location A is 400 m away from the river while Location B is 1.2 km away. Table 2.17 shows the flow rate gained from groundwater at the Walla Walla River and the head of Big Spring under the scenarios of lining all the irrigation canals and the proposed segments for lining with and without the combination of lining canals and the proposed managed aquifer recharge at either location A or B. The scenarios showed that most of the water from location A flows to the Walla Walla River without impacting the flow at Big Spring. Location B provides a longer steady flow to both Big Spring and The Walla Walla River. The next section in this research proposes a quantitative parameter measurement to evaluate a location for artificial aquifer recharge where a location with the highest estimated storage capacity is preferred.

2.9 Suggested methodology for evaluation of potential site for artificial aquifer recharge basins

The engineering design of artificial aquifer recharge proposed in this project begins by evaluating the hydraulic capacity for recharge of a specific location and the impact of estimated loading rates on the locally increased water table elevation. For this evaluation, the proposed methodology suggests first to estimate the site hydro-geological parameters necessary to solve the analytical solution presented by Bower (1999). In the first instance, the location that maximizes the volumetric recharge rate calculated by the analytical solutions would be preferable over other locations. As an example, the pilot test of the Hudson Bay Aquifer Recharge Project (HBDIC) is presented.

Although, the analytical solutions present an easy and transparent way to evaluate a potential site for aquifer recharge, they do not incorporate all the hydrological boundaries and water stresses. Computer model simulations are uniquely capable incorporating these parameters. This methodology achieves the evaluation of a potential site with a combination of a vadose zone model (HYDRUS) and the Walla Walla regional hydrological model IWFM. The vadose zone model is used to estimate the recharge rates obtained at a potential location. These recharge rates are calculated based on the simulation of the groundwater mound created beneath the basins with the site specifics. The regional hydrological model IWFM is then utilized to evaluate the effects on the groundwater table by the recharge loading rates estimated by the vadose zone model.

This research stresses the need to scientifically evaluate a potential location for artificial aquifer recharge projects. The evaluation of the location is a fundamental component in the engineering design. The success of the project depends on its engineered designed. In the Walla Walla Basin, aquifer recharge has been proven to be a potential solution to enhance habitat for fish, restore flow to springs and provide “cool” base flow to streams and the Walla Walla River

(Bower 2009). Artificial aquifer recharge projects also have the potential for disaster if they have not been properly engineered. Adverse results can include flooding houses and redirecting flow (change in hydraulic gradient) with contaminants or temperature increase. It will only take one aquifer recharge project with disastrous results in the area for stake holders and water managers to classify all aquifer recharge projects as “hazardous projects” with such big “un-knowns” that are impossible to be designed. Such comments have already been publicly expressed and published by the U.S .Corp of Engineers at Walla Walla in their feasibility report 2010.

Suggested Methodology

The first step of the proposed methodology is to evaluate the feasibility of a proposed location by means of an analytical solution. For this, we propose the following 5 steps necessary to estimate the analytical solution presented by Bower (1999) and illustrated in Figure 2.37.

1. Estimate the distance to a surface water discharge (Ln) based on the groundwater flow direction. Regardless of the project purpose, restoration, storage reservoir or as a mitigation of irrigation efficiencies, it is important to understand some basic notions of the site hydrogeological conditions for groundwater flow direction and velocity. This step could be easily estimated from a map of the water table. A location with the longest Ln is desirable since the idea is to use the aquifer as a storage or delay reservoir where water can stay as long as it can in the aquifer.
2. Estimate the distance from the bottom of the basin to the water table (H) during the period for which AR is proposed. This parameter is necessary to evaluate storage capacity and for estimating the groundwater mound growth beneath the basin. The excessive groundwater mound growth can determine the achievable recharge rates. A location with the highest distance to the water table H is desired.

3. Estimate the required surface area (A) of the infiltrating basin based on the available water for recharge and the initial estimates of infiltrating rates as A = Q/i where Q is the desired recharge flow rate (m^3/day) and i is the reported recharge rate (m/day) from reported values of pilot AR projects in the area.

4. In the case of irregular basin geometries, estimate the effective radius "r" for circular basins. We can also use width of the basin if it is considered an elongated rectangle 90˚ in direction perpendicular of the original flow of the WT.

5. Calculate the maximum infiltration rate achievable at that location by Eq. 2.21 or 2.22 in the case of irregular geometry basins. A location with the maximum achievable rate is preferred.

A formula for rectangular basins is presented in equation 2.30 to be used when the length of the basin is at least 4 times its W, and at the same time the length of the basin is perpendicular to the water table. In the case that the geometry does not meet these requirements, the formula for circular basins is recommended to avoid complications. The procedure to estimate the effective radius from any geometry is presented in Eq. 2.32.

$$Hc - Hn = \frac{iW}{2T}\left(\frac{W}{4} + Ln\right) \qquad \text{Eq. 2.30}$$

$$H = \frac{iR^2}{4T}\left(1 + 2\ln\frac{Rn}{R}\right) \qquad \text{Eq. 2.31}$$

Where R

Equivalent radius is $R = \sqrt{\frac{A}{\pi}}$ Eq. 2.32

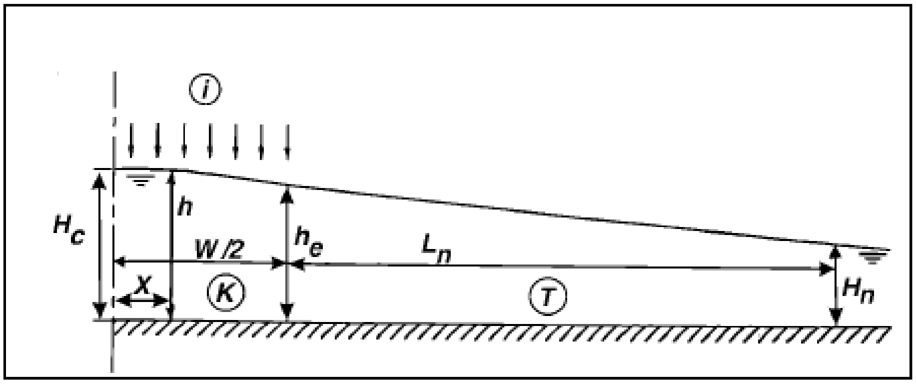


Figure 2.32 Aquifer recharge for infinite long strips Bower (1999).

The gradient calculated from the water-table elevations indicate that the flow of groundwater from the HBDIC location is flowing in the direction of the Johnson and Dugger Creek Springs. The distance from the infiltration basin to the springs is Ln= 1,600 m (1 mile). At this location, the distance from the bottom of the basins to the water table for the proposed recharge season is H= 14 m. The available area for infiltration of the basin is calculated to be 10,000 m^2, a rectangle of 200L*50W meters. The equivalent radius for a circular basin is by Eq. 2.31 to be r = 56.5 m.

Applying equation 2.33 and solving for *I* yields

$$\frac{4*30*60*14}{56.41^2} * \frac{1}{1+2\ln\left(\dfrac{1600}{56.41}\right)} = 4.1(m/day) \qquad \text{Eq. 2.33}$$

The estimated recharge rate by Eq. 2.22 compares well to reported rates from HBDIC recharge project (Bower 2009). For ease of comparison, the total recharge volumetric flow rate Q can then be calculated with the estimated recharge rate i and the infiltration basins area. In the case of the HBDIC, 4.1 (m/day) *10,000 (m^2) = 41,200(m^3/day) equivalent to 16.8 cfs the reported recharge rates for the HBDIC is 17 CFS (Bower 2009).

The analytical solution in this methodology is an initial estimate that by its ease of calculation presents a fast and transparent approach method which could be used for evaluation of potential Mar locations. The location that presents the highest combination of the parameters Hn, L and available area, is the preferred location. As an example from the two proposed locations in section 2.7.1, location B with the same area (99,000m^2 or 24.5 acres) and distance to water (14m) is preferred over location A given the lateral distance to a water boundary of location B is 1.4 km compared to 400m of location A to the Walla Walla River (Figure 2.31).

Once a location has been chosen, a simulation model of the vadose zone can be used to evaluate the engineering design and the effects the increased groundwater elevations will have on surrounding surface water bodies. The vadose zone model constructed with the site specifics can simulate the rate of growth and dimensions of groundwater mound. As the groundwater mound reaches the surface, achievable infiltrating recharge rate diminishes (see chapter 3 scaling of recharge flow rates from pilot test of artificial aquifer recharge). For the HBDIC a HYDRUS 2D/3D model has been structured to estimate the recharge rate achievable from different geometries of infiltrating basins. The original purpose of the model is to evaluate the decrease of recharge rate with the expansion of the pilot test. Figure 2.33 shows the simulation model cross section. The model is fully described in section 2.2 Engineering consideration of infiltrating basins for management aquifer recharge.

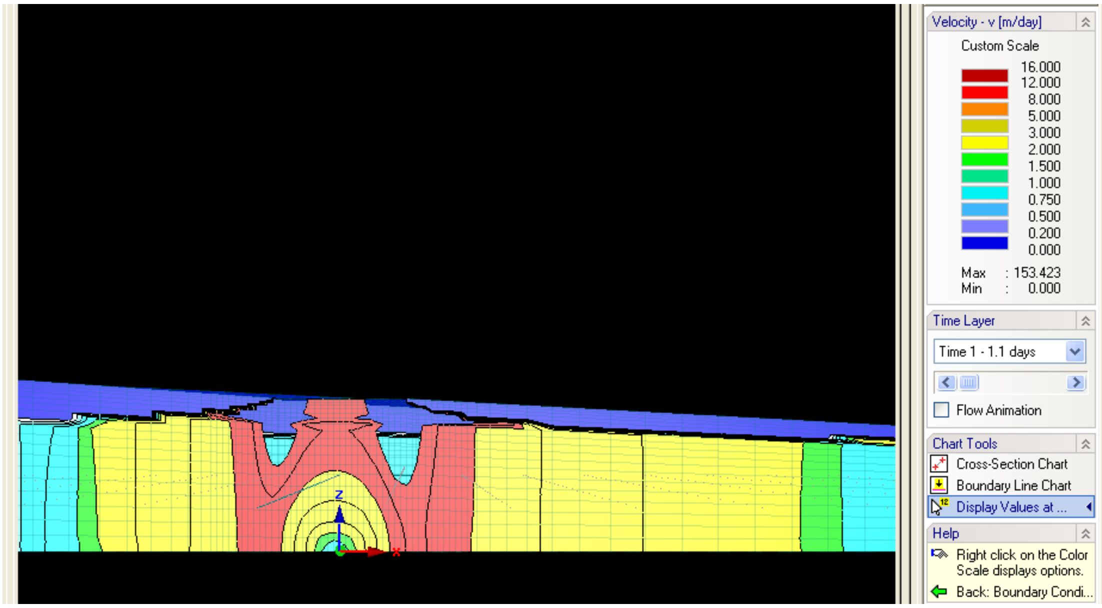


Figure 2.33 HYDRUS 2D/3D simulation model of the HBDIC Infiltrating basin. Figure show groundwater velocities (m/d) around the project basins.

The vadose zone model provides the regional hydrological model (IWFM) with the volumetric recharge rates attainable for each site from the geometries of infiltrating basins. IWFM is then used to estimate the elevation of the groundwater table and its effects on surface water features, including the accumulation of water at the ground surface. As an example, the IWFM model was used to simulate artificial aquifer recharge at two locations to mitigate the reduced amount of water recharging due to the planned lining of irrigation canals. Section 2.8.1 shows the differences in flow at a down gradient springs and the Walla Walla River from simulations under two recharge flow regimes. Due to the high (15 meters) distance to water and the thickness of the aquifer, no flooded areas are associated with these two projects. Most of the recharged water at these locations flows to the Walla Walla River that serves as a lateral boundary constraining the maximum storage capacity in the area. Travel times of the pressure wave at location A to the Walla Walla River are shown to be 2.5 days and for location B is 9 days. The pressure wave induced by the aquifer recharge projects increases the water table elevation. Here the gain of water in the river is higher from groundwater flux. The actual particles of water from the aquifer recharge take much longer to travel through the bulk thickness of the aquifer.

Another example of the use of IWFM to evaluate locations of artificial aquifer recharge is the modeled simulation of the HBIDC project and the springs of Johnson Creek Spring and Dugger Creek Springs located 1.6 km away from basins. The HBDIC project is described under chapter 3; for more information please refer to HBIDC Annual Report presented by Bower (2010). Bower and Petrides (2009) utilized the IWFM model to evaluate the feasibility of artificial aquifer recharge at HBDIC to restore flow in springs that were dry for almost 25 years. The model compared well to the observed increase in flows in the springs. Later results from tracer test and HYDRUS simulations presented in the next section 4, showed similar hydraulic connections reported with IWFM.

2.10 Discussion and conclusions

The IWFM-Walla Walla Model presents a tool to support decisions evaluating water management practices holistically. The model is first used to evaluate the water resources in the basin. Water budgets are presented (section 2.7) with their degree of uncertainty from model simulation runs and the range of possible input values. A key element identified during this analysis was the negative change in groundwater storage due to excessive pumping to satisfy the agricultural evaporative demand. The difference between groundwater pumping and the amount of net deep percolation is 9.1 million of m^3/year. Under the simulated water management scenarios of lining all the irrigation canals, the amount of water that is no longer recharging the aquifer is equivalent to 30% of the total water carried by the irrigation canals. The budgets also estimate that 34 to 44% of the evaporative demand is supplied by pumping the gravel aquifer. Lining the irrigation canals is equivalent to increasing groundwater pumping by 23% and reducing the aquifer levels an average of 20 cm/yr.

This research project presents a methodology for evaluating potential locations of artificial aquifer recharge basins. As an example, locations that are currently proposed in the basin to restore spring flows and a portion of the Walla Walla River are evaluated in section 2.8. Results show that Location B is preferred over location A because of the close proximity to the Walla Walla River and the achieved increased flows at the down gradient springs. The proposed methodology incorporates the use of Bouwer’s (1999) analytical solution to estimate the recharge rates (steady state) and groundwater mounding at a possible location. As an example, the HBIDC project is calculated by the analytical solutions and compared to the observed recharge rates. The methodology then suggests for sites that have been selected after the first screen to structure a vadose zone model with the site specifics. The vadose zone model estimates the variable recharge rates as water table rises. The variable recharge rate then is incorporated in the regional model to estimate its effects in relation to topography and other boundaries, such as pumping and recharge from leaky canals and excess irrigation.

Additional results from the efforts of structuring the Walla Walla Model are: (1) further understanding of regional hydraulic interactions, such as pumping estimates, evapotranspiration, water uses, leakage from irrigation canals, and surface flows in canals and springs; and (2) structuring the model in conjunction with the WWBWC helped to evaluate the placement and quality of the network of monitored locations of groundwater and surface water flows. The model assembled the available hydrological information in the area and provided a new level of quality assurance and quality control. Data needs and areas of future research were identified throughout the model development and shared with the WWBWC and interested organizations. Finally, the Walla Walla Model helped to promote the conservation of water resources in the basin.

2.10.1 Limitations for the regional hydrological model and recommendations for further work

The regional hydrogeological model presented in this research uses the IWFM code. The data needs for this type of model are large and because of the lack of a user interface, the model requires additional software, such as GIS for data organization and model manipulation. The modeled aquifer parameters are averaged or lumped over space and time. The spatial complexity of the physical features of hydrologic systems is unknown, but expected to have high natural variability. This model was created for the evaluation of management decisions and not solely for academic or scientific purposes. The parameters estimated during this research are only applicable to this conceptual model representation.

The ability of the model to reproduce historic conditions was based on 3 years of collected data. The monitored network maintained by the WWBWC includes 3 to 5 years of data collection. Future model validation will incorporate additional new information. Prediction runs of future impacts of management decisions have been presented with their degree of uncertainty. However, they are based only on current boundary conditions. Future model review by local professionals, such as irrigation district personnel, could corroborate the amount of flow diverted in the simulated irrigation canals and the percentage of pumping drawn from basalt wells.

This modeling effort is an ongoing project expected to be continued by the WWBWC. Expansion of the project to the entire extent of the gravel aquifer is being considered. The model activities necessary to complete this task are much simpler than starting a new hydrological model from scratch. However, given the poor flexibility of IWFM, the model expansion requires not only incorporating new information that was previously outside the original model domain, but also reconstructing the model grid and re-calibrating model parameters to adjust for the new better-established model boundaries.

Chapter 3 - Scaling recharge flow rates from pilot to full scale managed artificial aquifer recharge projects.

Authors: Petrides1*, A.C., R. Stewart, R. Henry, R. Bower, R. H. Cuenca.

Submitted to:
Journal: Water Resources Research.,
Journal Address: Journal of the American Geophysical Union (AGU) 2000 Florida Avenue N.W. Washington, DC 20009-1277 USA

Abstract

Recharge rates evaluated from pilot projects of surface managed artificial aquifer recharge are extrapolated to develop design criteria for full scale projects. Field experiments at recharge facilities in the Walla Walla River Basin, Oregon, USA, and a 3-dimensional computer simulation model were used to estimate recharges rates in relation to groundwater mounding and to the expansion in surface area of infiltrating basins. Results show that if the water table mounding does not reach the infiltrating basin floor (thereby maintaining an unsaturated zone between the water table and the infiltrating basin), the recharge rates should be estimated from pilot tests scale linearly with the basin's surface area expansion. However, if the groundwater mound reaches the bottom of the basin floor (thereby providing a full hydraulic connection between the infiltrating basins and the aquifer), recharge rates should be extrapolated using the perimeter of the infiltrating basin. The explanation for this effect is offered by evaluating the distribution of the water velocities at the infiltrating basin floor and the relationship between aquifer thickness and the radius of the infiltrating basins.

3.1 Introduction

Pilot tests of infiltrating basins are commonly used to evaluate the feasibility of artificial aquifer recharge projects, also known in the literature as surface Managed Aquifer Recharged (MAR). The analysis of the pilot test results should consider the spatial scale dependence of the various measured parameters. Usually these parameters include recharge flow rates, groundwater mounding, and water quality (American Society of Civil Engineers ASCE. 2002). The recharge flow rate, *Q* (L3/T), observed during a pilot test is commonly assumed to increase linearly with the infiltrating surface area. However, due to groundwater mounding underneath the infiltrating basins, field experiments in the Walla Walla Basin and elsewhere (Jones 1974, Szalay 1957 and Suter 1956 as expressed by Zomordi 1988) have documented a non-linear relationship between *Q* and surface area, *A* (L2).

A commonly reported confusion (Morel-Seytoux 1990, Zomordi 1991, Sumner 1999) when designing full scale recharge projects is the use the infiltration rates, *i* (L/T), instead of the recharge rates, *r*(*t*) (L/T), from pilot basin projects or infiltration tests. Infiltration is the rate at which water enters the soil, leaving the surface of the infiltrating basin. On the other hand, recharge, is the rate at which water enters the saturated aquifer zone from the partially unsaturated zone (Figure 3.1). When the infiltration rate *i* is used to estimate the volumetric flow rate (*Q* = *A* * *i*), the extrapolation of *Q* from pilot projects to design for full-scale operations is expected to be linearly related to the surface area of the full scale infiltrating basin (ASCE 2002). The linear extrapolation is because *i* is assumed to be a scale independent (constant in time and space) parameter. On the other hand, recharge rate *r(t)* is scale dependent on the growth and decay of the water table mounding. Morel-Seytoux (1989) relates the infiltration rate to the recharge rate by:

$$\frac{dz_{(t)}}{dt} = \frac{i}{(\tilde{\theta} - \theta_o)} - \frac{2r_{(t)}}{(\tilde{\theta} - \theta_o)R} \qquad \text{Eq. 3.1}$$

Where, *dz(t)/dt* is the position of the water table under a circular spreading basin of radius *R* at time *t*. *θo* is the water content in the unsaturated zone above the water table and *θ* is the saturated water content. In the absence of infiltration basin clogging (physical, chemical or biological), infiltration rates *i* differ from the recharge rates *r(t)* particularly during the initial period of groundwater mound build up (Morel-Seytoux 1985). At later times, as recharge operations progress and groundwater mounding reaches a quasi-steady state (Guo 2003), the infiltration rate can be considered equivalent to the recharge rate.

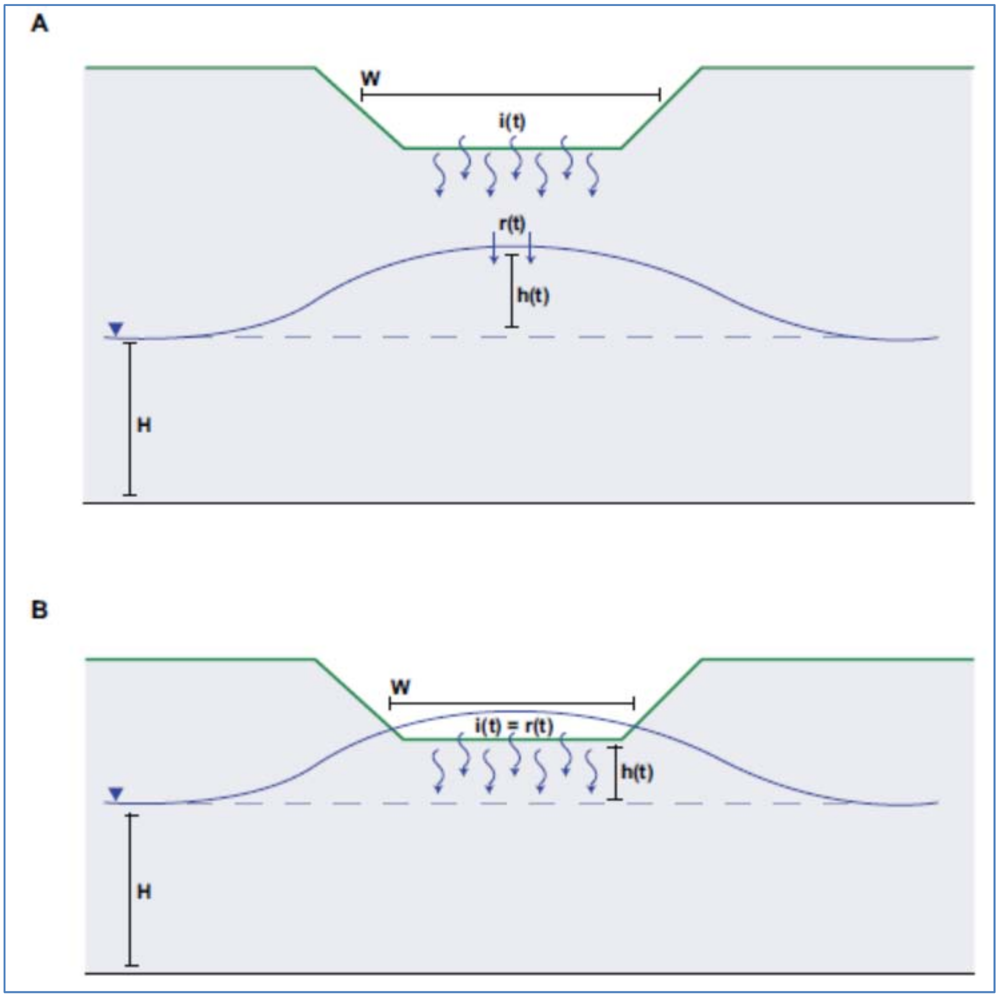


Figure 3.1. Difference between infiltration rate i(t) to recharge rate r(t) from; (A) infiltrating basins where the groundwater mound doesn't reach the surface and , (B) infiltrating basins with a full hydraulic connection to the aquifer

Recharge rates have also been shown to be dependent on the specific geometry of the groundwater mound formed beneath the basin area (Morel-Seytoux 1985). To provide additional accuracy in predicting the influence of mounding on recharge rate, it is necessary to establish estimates for both, the rate of growth and physical dimensions of the mound. Several analytical solutions have been offered to estimate the growth of groundwater mound (Bauman 1952, Hantush 1967, Hunt 1971, Morel-Seytoux 1990, Rai and Manglik 1995, Bouwer 1999, Guo 2001). Due to their ease of implementation, analytical solutions provide an inexpensive method for evaluating a potential groundwater mound build up and recharge rates. These solutions however, have limitations in their assumptions and applicability. For example, analytical solutions typically assume an infinite, isotropic, homogenous aquifer with constant recharge (Bouwer 2002). Conversely, computer simulations can be made specific to individual sites, and therefore include non-ideal conditions. For instance, computer simulations can incorporate real site boundary conditions and allow for a variable recharge rate. Examples of computer simulations of groundwater mounding from infiltration basins include Sumner (1999), Bouwer (2002), and Manglik 2004. Poeter (2005) used the vadose zone computer model HYDRUS 2D (Simunek et al., 1999) to simulate groundwater mounds in order to evaluate the feasibility of infiltrating basins. Rastogi (1998) through computer simulations of different infiltration basin geometries, showed that groundwater mounds are geometry and scale dependent.

Computer simulations can be used to study the variations of recharge rate and the distribution of infiltration rates (entry velocities) over the spreading basin area. As Huisman and Olsthoorn (1983) pointed out, entry velocities over the spreading basins surface area are not uniform. When the infiltrating basin establishes a full hydraulic connection with the aquifer, entry velocities change from a unit gradient flow to the flow gradient of the aquifer (Forheheimer 1930, Bouwer 2001, Kacimov 2007). To estimate the entry velocities distribution, Huisman and Olsthoorn (1983) provide an unpublished equation (Eq. 3.2) cited to be obtained from personal communication of the authors to Verruijt (1979). Equation 3.2, utilizing methods of conformal mapping, estimates the entry velocities (v_e) radially starting from the center of the basin.

$$v_e = v_a \frac{w}{H} \frac{\cosh(\frac{1}{2}\pi x/H)}{\sqrt{\sinh^2(\frac{1}{2}\pi w/H) - \sinh^2(\frac{1}{2}\pi x/H)}} \qquad \text{Eq. 3.2}$$

Where v_e is the entry rate calculated at a radial distance x from the center of the basin, v_a is the average infiltrating rate ($v_a = Q/\pi R^2$), w is the width of the spreading basin, and H is the aquifer saturated thickness. The distribution of entry velocities across the infiltrating basin surface area is graphically displayed in Figure 3.2 (modified from Huisman and Olsthoorn, 1983). The entry rate ve is smallest at the center (x=0) of the basin and increases radially to the outer circumference. This effect becomes more pronounced as the saturated thickness decreases in relation to the basin surface area. At the center of basin, the entry velocities become negligible as the basin width increases to over 4 times the saturated thickness of the aquifer (Figure 3.3).

In this research, the analytical solution of Verruijt (1979) and computer simulations are calibrated to field experiments in the Walla Walla Basin Oregon, USA, (Bower 2009). The simulations were employed to estimate the recharge rate Q from rectangular infiltrating basins of increasing surface areas as a recharge pilot site was expanded. These infiltrating basins geometry were modified by expanding their surface area in both, their length (chosen to be their side parallel to the original groundwater flow) and, their width (chosen to be side perpendicular to the original groundwater flow direction). Two scenarios were evaluated for each infiltrating basin geometry; (I) Infiltrating basins where the groundwater mound reaches the bottom of the basins, and (II), infiltrating basins that the groundwater mound does not reach the surface due to a low conductivity clogging layer. This methodology section provides a description of the Walla Walla field site and the structure of the simulation model.

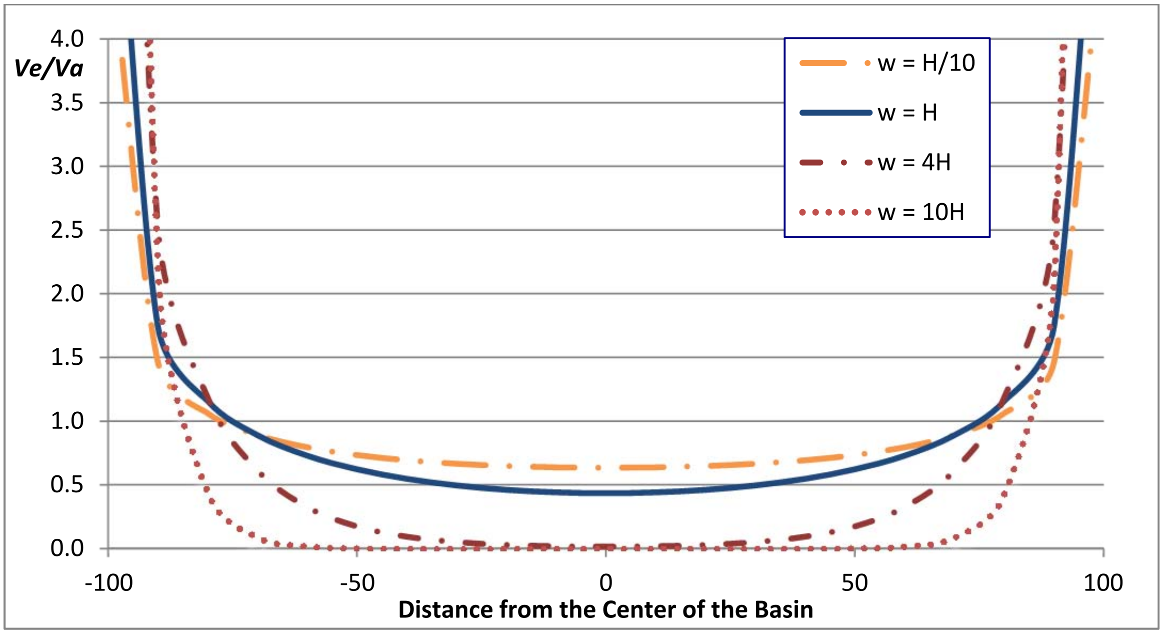


Figure 3.2. Distribution of entry rates over the spreading basin surface area as the width w increases over the aquifer thickness H. Figure modified from Huisman and Olsthorrn (1983).

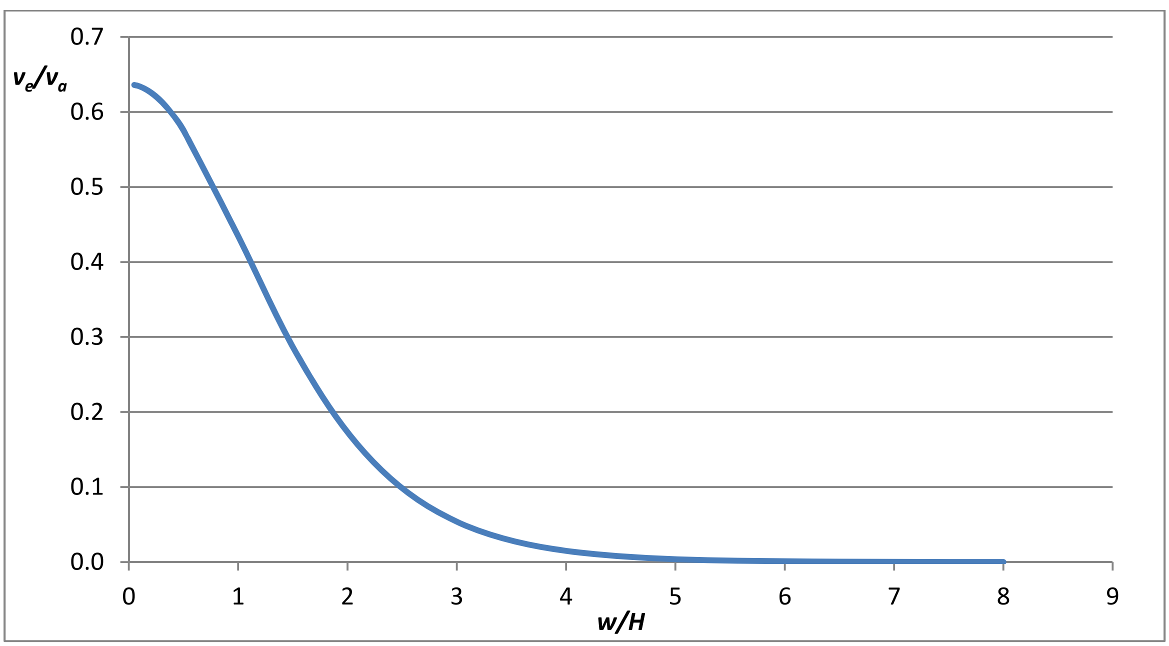


Figure 3.3. Entry rates at the center of the basin. Infiltrating rate decreases as basin's width increases over the thickness of the aquifer.

3.2 Methodology

The Walla Walla Basin Watershed Council (WWBWC), in conjunction with Hudson Bay District Improvement Company (Irrigation district) and Oregon State University, have previously evaluated pilot and full scale projects to artificially recharge an unconfined alluvial aquifer system1 (Lindsey 2004) with the purpose of aquifer and spring restoration. The original pilot test in 2004 consisted of diverting water from an irrigation canal into three rectangular, trapezoidal shaped infiltration basins (Figure 3.4). The side walls were constructed with a 2:1 slope and a ponding depth of 1.5 meters (H1) were maintained during their operation. The unconfined aquifer below the spreading basins consists of two highly permeable (K1=65 m/d; K2=30 m/d) layers of unconsolidated gravels and silts (Figure 3.5). The thickness of the first upper layer decreases linearly across the aquifer extent from 12 m to 3 m. The second lower layer composes the rest of the aquifer with an average thickness of 60 m. The average water table elevation before recharge operations is 11 m below the infiltrating basins and has an initial groundwater gradient of 0.01 m/m. To evaluate the performance of this pilot project, the WWBWC installed and instrumented piezometers at observational wells with 15-minute interval pressure transducers. These observational points are located in the infiltrating basin and eight surrounding wells in site proximity from the edge of the basins from 150 m to 1,200 m.

[1] Mio-Pliocene unit of unconfined gravels with 15% silts

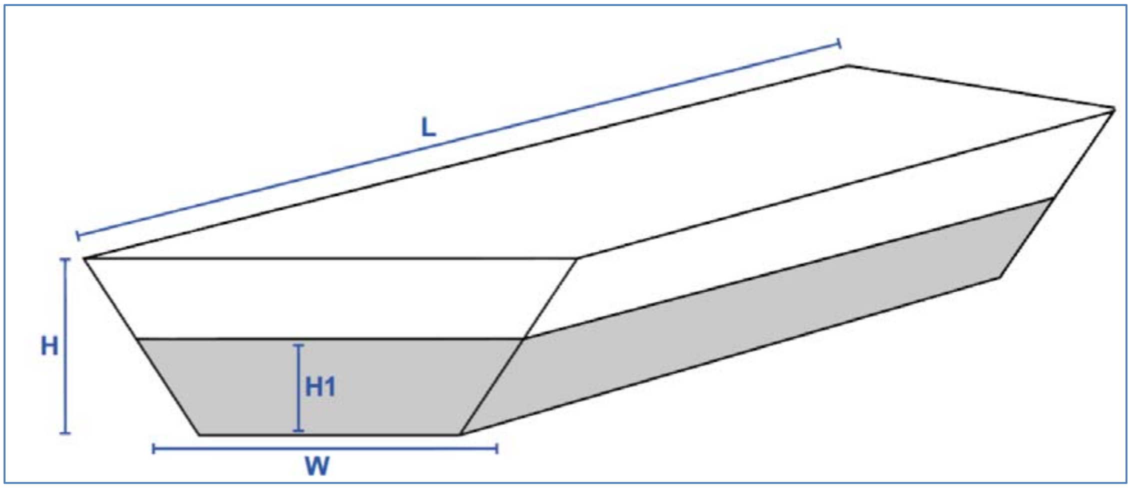


Figure 3.4.Infiltration basin design in the Walla Walla Basin. Slope 2:1, H =1.5m, W=50m, L=200m

The WWBW reported a decreased recharge rate when the surface area of the infiltrating basin was expanded (Table 3.1) between each recharge season (November through May) (Table 3.1) through a three staged, five year site expansion (Bower 2009). In 2004, the first year of operations, the combined surface area of the infiltration basins was 1,400 m^2, which provided a recharge rate of 0.2 m^3/sec. In 2005, after one year of operation, the pilot test was expanded in area to 4,500 m^2, which provided a recharge rate 0.34 m^3/sec. Even though the surface area of the basins more than tripled in size, the volumetric flow rate only increased 1.5 times over the original rate. For the years 2005 to 2007 the basins were maintained with the same surface area and achieved similar recharge rates. Subsequently in 2008 and 2009, the area was expanded 6,000 m^2, and 10,000 m^2 respectively. The recharge flow rate for these years only increased close to double the initial volumetric rate with a seven-fold increase in infiltrating area (Table 3.1).

Table 3.1.Observed volumetric recharge rates at the Walla Walla Basin.

Year	Surface Area (m^2)	Recharge Flow Rate (m^3/sec)	Infiltration rate (m/day)	Increase in Surface Area (A/A_o)	Increase in Recharge rate (Q/Q_o)
2004	1400	0.2	13	1.0	1.0
2005	4500	0.34	7.5	3.2	1.7
2006	4500	0.37	8	3.2	1.9
2007	4500	0.37	8	3.2	1.9
2008	6000	0.42	6	4.3	2.1
2009	10,000	0.46	4	7.1	2.3

Computer simulations of the infiltrating basins were performed using the Microsoft windows version of HYDRUS 2D/3D (Simunek 2008). Originally developed at the U.S. Salinity Laboratory (Simunek 2008), HYDRUS allows for simulation of water flow in the unsaturated and saturated zone by applying the Galerkin's finite element numerical method to solve the Richard's equation (Brutsaert 2005). As pointed out by Sumner (1999), omitting flow through the unsaturated zone in simulation models of infiltrating basins of managed artificial aquifer recharge can lead to significant errors (up to 800%) in the estimation of recharge rates. The three-dimensional capability of HYDRUS 2D/3D allowed testing and analysis of the variation of recharge rate from the site's infiltration basins expansion of surface area through modification of both the horizontal side length (*L*) and/or the lateral side width (*W*).

The surface area simulated in the model was 9.75 km^2 (3 km by 3.25 km). The infiltrating basins were located on the uppermost layer (the model surface) in the center of the model, and were represented as a single basin through use of a variable flux boundary condition. Springs, represented by constant head boundary conditions, were located 1.6 km north of the infiltrating basins at the model's edge. The aquifer thickness decreased linearly north to south from 70 to 35 m (Figure 3.5). Grid size varied from 1 m at the infiltrating basins to 100 m near the model boundaries. Time steps ranged from 0.01 h to 1 h for a total simulation period of 40 days. No flux boundary conditions were used for the non-infiltrating basin upper nodes and model edge's nodes.

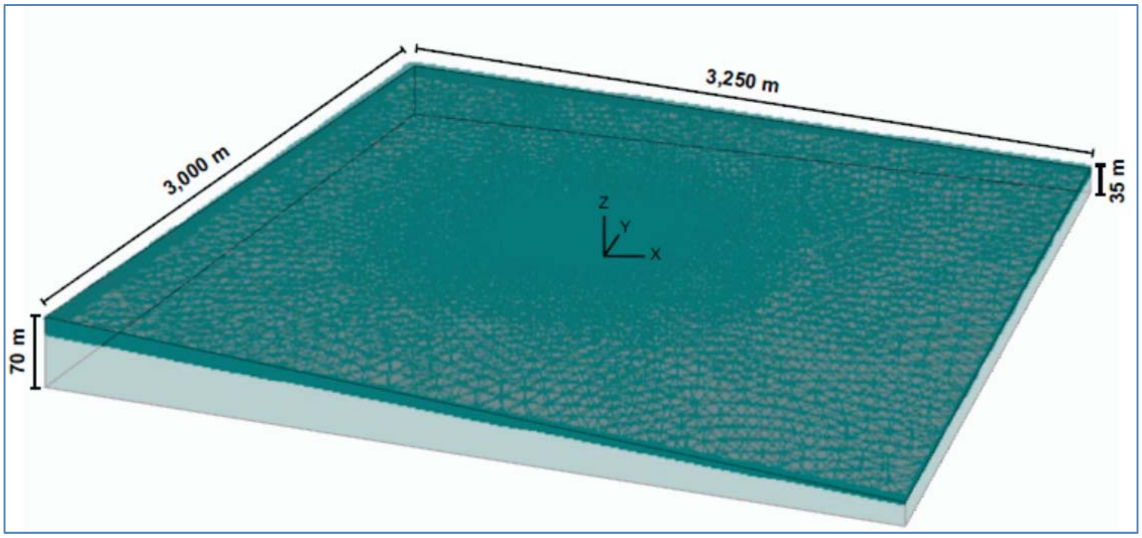


Figure 3.5. HYDRUS 2D/3D Model of the Walla Walla recharge site area 9.75 km2. Mesh grid size varies from 1 meter at the region near the infiltrating basin to 100 meters near the boundaries. The unconfined gravel aquifer is modeled by of two geological material layers represented by the colors green and light gray.

The simulation model was calibrated to ensure the base conceptual model in HYDRUS 2D/3D, which is capable of accurately simulating the development and decay of the groundwater mound. The calibrated model was attained by minimizing the discrepancy of simulated and observed groundwater head at 17 monitored locations from the 2009 recharge operations. The monitored locations included both within and around the infiltrating basins piezometers, from the center of the basin to the head of the springs (Table 3.2). Parameters adjusted during calibration included hydraulic conductivities (k) of the two layered aquifers, residual water content (θ_r), saturated water content (θ_s) and the van-Genuchten (1980) fitting parameters α and n. Figure 3.6 shows the match between simulated and observed water levels at the closest (150 m) observation well. For the compilation of all 17 observation points, the total Nash-Sutcliffe (1977) efficiency statistic was used as calibration objective. Once the model demonstrated its ability to simulate the growth and decay of the water table mounding to an acceptable level (Nash and Sutcliffe coefficient of 0.77), the surface area of the infiltrating basins was modified for each simulation.

Table 3.2 Monitoring wells used to calibrated the infiltrating basin at the Walla Walla recharge project

Monitoring well	Distance from the center of the basin in the ***x*** direction (m)	Distance from the center of the in the ***y*** direction (m)	Standard Error (m)	Nash & Sutcliffe
1	0	0	1.2	0.9
2	36	5	1.0	0.9
3	71	2	1.3	0.8
4	100	0	1.9	0.7
5	111	127	2.1	0.5
6	-121	14	2.5	0.7
7	157	6	2.0	0.7
8	175	21	1.5	0.8
9	234	-404	0.8	0.9
10	326	-807	0.4	0.9
11	-465	-31	0.7	0.8
12	-778	1198	1.5	0.7
13	1,041	74.3	0.3	0.7
14	1,385	81	0.5	0.7

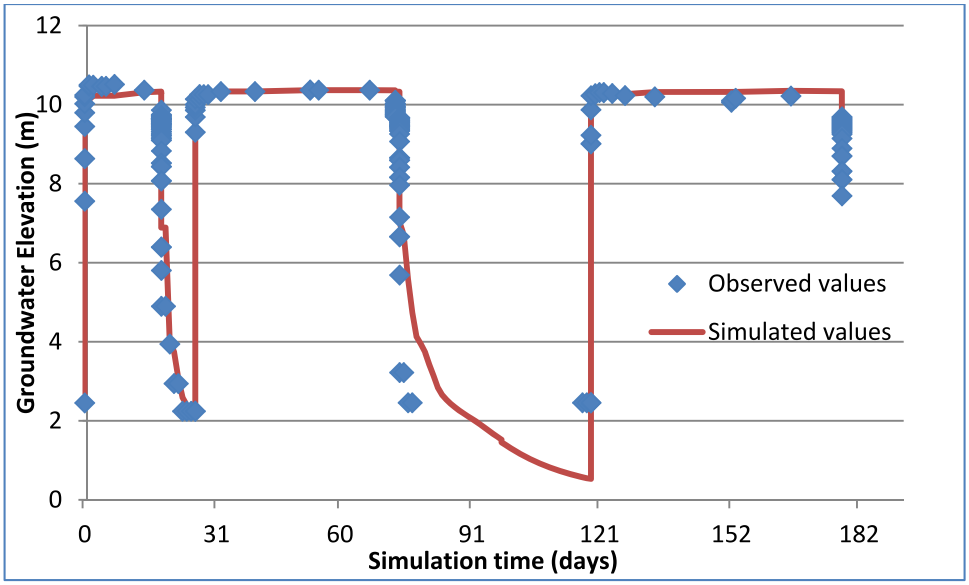


Figure 3.6, Recharge operations, calibration graph matching observed and simulated values of groundwater head at 150 meters from the edge of the basins.

The measured and simulated cumulative recharged (Q_c) volumes after 40 days of operation were used for comparison of results (Equation 3.3).

$$Q_c = \sum_{t=1}^{t=40} Q_{(t)} \qquad \text{Eq. 3.3}$$

The period of 40 days was chosen to match the length of average operations in the Walla Walla Basin. The water level data at the observational points suggest that after the first 10 days of operations, the recharge rate reaches a quasi-steady state.

The simulation of the scenarios that the groundwater mound does not reach the surface was achieved by including an extra layer of fine material (comparable to simulating a clogging layer) on the bottom of the infiltrating basins. The clogging layer was simulated as having a thickness of 1 m, and a hydraulic conductivity of 1.1 m/day (98% lower than the hydraulic conductivity of the original aquifer layer). The reduced infiltration rate caused the groundwater mound not to reach the surface, thereby maintaining an unsaturated zone between the bottom of the basin and the top of the water table. The same geometries of infiltrating basins utilized in the non-clogged scenarios were simulated, and the cumulative recharge after 40 days was used for comparison between simulations.

3.3 Results and discussion

The Walla Walla Basin pilot tests present an excellent opportunity to study managed artificial aquifer recharge. The Walla Walla Basin Watershed Council (WWBWC) has tested different engineering designs while observing the recharge rate and groundwater response via a network of monitored piezometers and observational wells. The unconfined gravel aquifer underlying the region presents the most desirable hydro-geological conditions for artificial recharge (very coarse grained aquifer with thick unsaturated and saturated zones, high hydraulic conductivity, and high porosity). Since the water used for recharge is of excellent quality and the high hydraulic conductivity, no clogging effects have shown to impair the infiltrating rates from the basins. However, when the pilot test project has been expanded, the total amount of recharged water has not increased linearly with the increase of surface area (table 3.1). Using the results from this pilot project, a three-dimensional computer simulation was calibrated using the model HYDRUS 2D/3D.

The computer simulations using the HYDRUS 2D/3D model code were used to calculate the recharge volume from rectangular infiltrating basins of different sizes. The rectangular infiltrating basins were modified by extending their Length (*L*) and their width (*W*). For each infiltrating basin geometry, two scenarios were evaluated; (1) Infiltrating basins where the developed groundwater mound reaches the bottom of the basin floor and, (2) Infiltrating basins where the groundwater mound does not reach the surface *(i.e*. due to the presence of a clogging layer) maintain a partially un-saturated aquifer area between the developed water table mound and the infiltrating basin floor. Tables 3 and 4 show the percentage increase of Q_c relative to the geometric case base of the 50 by 50 m geometry infiltrating basin.

Table 3.3 Results for scenarios where the groundwater mound reaches the bottom of the infiltrating basins. Results are shown as a percentage of cumulative recharge as compared to the 50 by 50 m geometry basin. There is on average a 4% increase *Q* when the infiltrating basin area is expanded by its Length *L* rather its width *W*.

L\W	50	10	25	100	200	400
50	100.0%	72.8%	88.4%	116.9%	142.0%	180.0%
10	79.9%	40.8%				
25	92.2%		77.9%			
100	120.7%			135.6%		
200	144.9%				180.1%	
400	180.4%					243.9%

Table 3.4. Clogged basin scenarios where the groundwater mound doesn't reach the bottom of the infiltrating basins. Results are shown as a percentage of cumulative recharge in relation to the 50 by 50 m geometry basin. There is in average a 7% major increase *Q* when the infiltrating basin area is expanded by its Length *L* rather its width *W*.

L\W	50	10	25	100	200	400
50	100.0%	24.0%	57.2%	171.2%	340.9%	651.4%
10	25.4%	7.5%				
25	64.9%		31.8%			
100	180.9%			376.0%		
200	357.9%				1055.7%	
400	689.2%					1554.2%

Water velocity vectors were also estimated by HYDRUS 2D/3D. Shown in Figure 3.7, for the basin geometries of 50m by 200m, the distribution of infiltration rates across the basin's floor differs between clogged basins and fully hydraulically connected basins. For the scenarios simulating fully hydraulic connected basins with groundwater mounding reaching the surface (bottom of the basins), entry water velocities across the basins floor resulted in radially distributed velocities. In these scenarios, the lowest infiltration rates are found at the center of the basin and the maximum infiltration rates are found at the basin's outer perimeter. The cumulative volumetric recharge water (Q_c) is therefore mostly infiltrated at the outer perimeter. Figure 3.8 shows for fully hydraulic connected basins a linear relationship between Q_c and the infiltrating basin's perimeter. For the scenarios simulating infiltrating basins that the developed groundwater mound does not reach the surface (as in the case of clogged basins), the entry rate velocities are uniformly distributed throughout the bottom of the infiltrating basins (Figure 3.7). Figure 3.9 shows for clogged basins a linear relationship between Q_c and the basin's surface area.

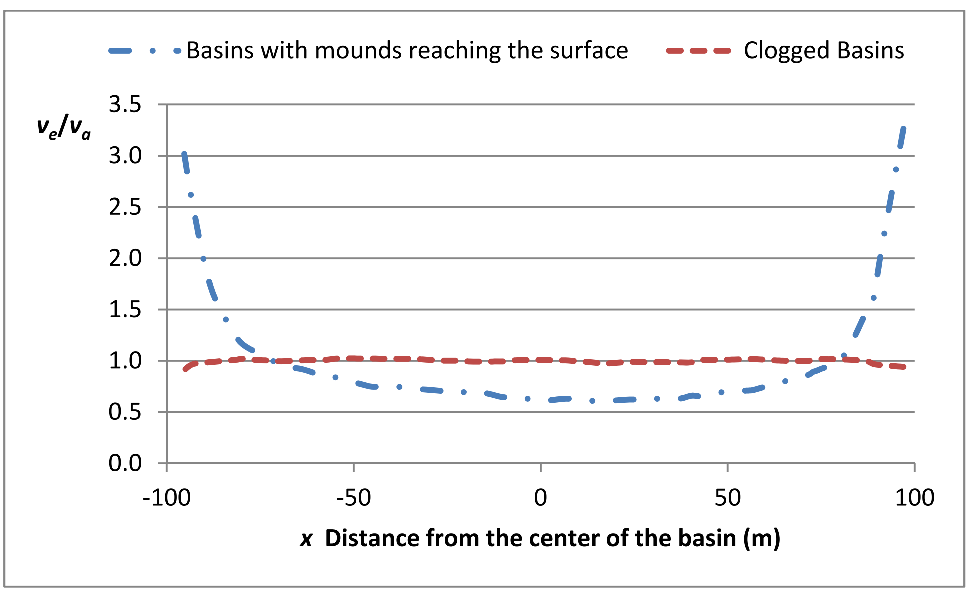


Figure 3.7. Simulated infiltrating results for the 50 by 200 (m) basins. Distribution of infiltration rates over spreading basins for clogged basins (red) and for basins where the groundwater mound reaches the surface (blue).

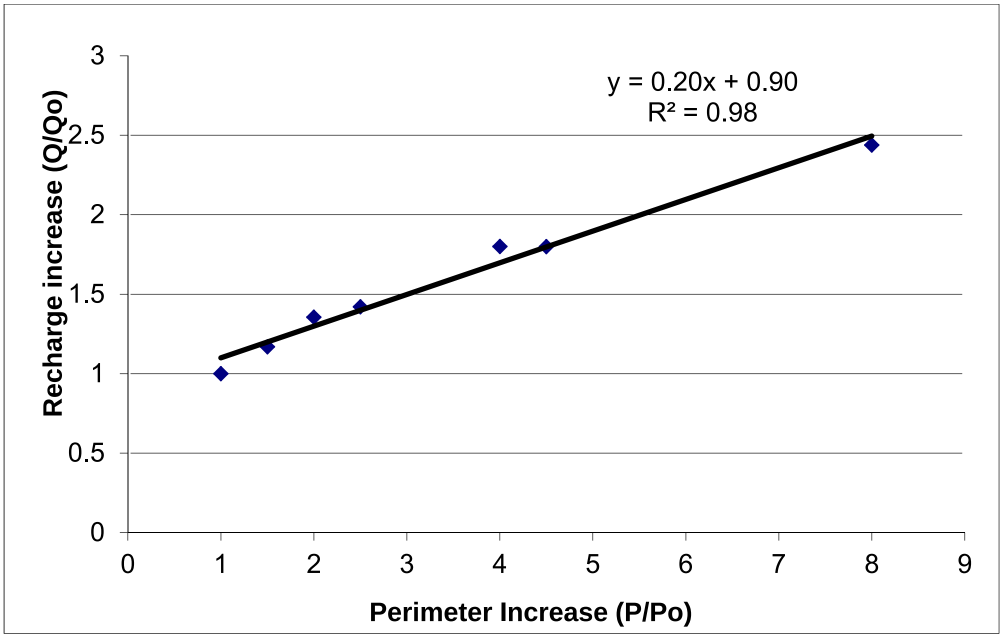


Figure 3.8. Correlation between the perimeter increases of the basins to the increase in recharge flow rate in the case where the groundwater mound reaches the surface.

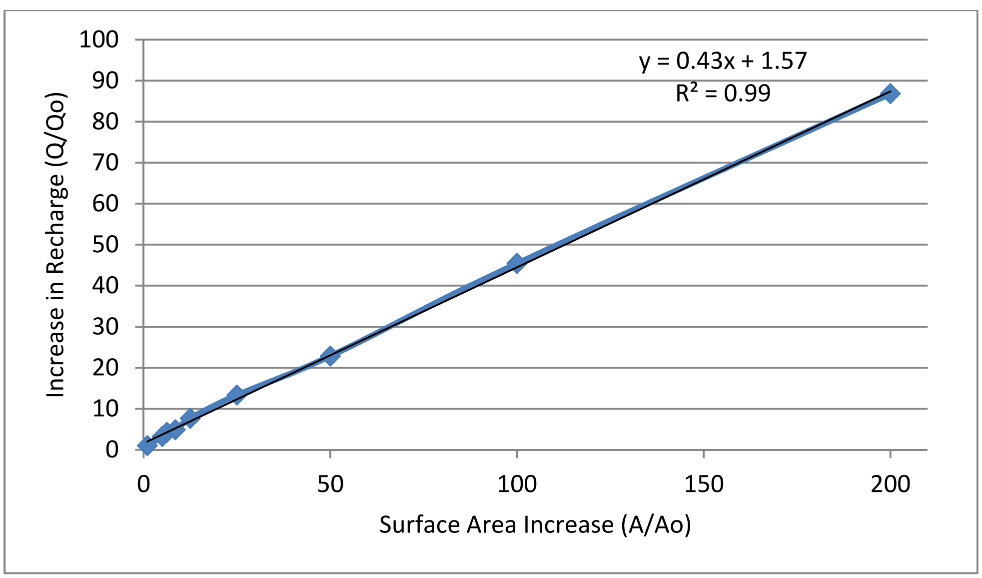


Figure 3.9. Correlation between the area increases of the infiltrating basins and the increase in recharge flow rate, for scenarios where the groundwater mound does not reaches the basin bottom.

The distribution of entry water velocities changes from a uniformly distributed average infiltrating rate to a radially distributed infiltrating rate when the groundwater mound reaches the bottom of the basins, and a fully saturated hydraulic connection is established between the infiltrating basin and the underlying aquifer. These simulated results compare well to the effects described in the literature (Bauman 1952, Morel-seytoux 1999, Bouwer 2001), which describes when a partially unsaturated zone is maintained between the basins and the water table mound, infiltrating (entry) rates are controlled by the hydraulic conductivity of the basin floor. In this case, flow through the unsaturated zone occurs under a unit gradient. Once a full hydraulic connection is established, the entry rates are dictated by the aquifer's water table gradient and its ability to transmit water (Forchheimer 1930, Huisman and Olsthoorn 1983, Kacimov 2000). The distribution of entry rates across the basin's surface area can be explained by a flow-net analysis (Huisman and Olsthoorn 1983). For a full hydraulic connected basin, as the thickness of the aquifer decreases with respect to the basin surface area (Figure 3.10), the flow lines become more bent and elongated from the center to the outer circumference, reducing the entry rates in the center of the basin.

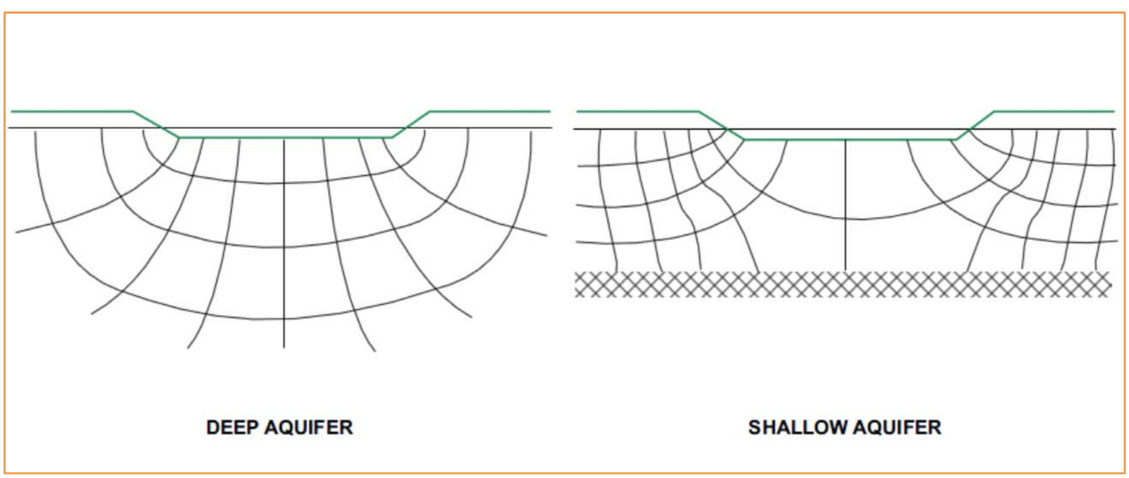


Figure 3.10. Flow-net analyses comparing a deep and a shallow aquifer. Flow lines become more bent and elongated from the center to the outer circumference as the thickness of the aquifer decreases.

The entry rates distribution for the simulated scenarios that the groundwater mound reaches the bottom of the basin compared well to the theoretical equation proposed by Verruijt (1979) as expressed by Huisman and Olstrom (1983). The differences between the simulated results and the analytical estimations were caused by the analytical assumption of a flat initial water table base over an aquifer of infinite length. The computer simulated infiltrating basins were developed with a 90° orientation relative to the regional water-table gradient. Results from these simulated scenarios, (Tables 3.3 and 3.4) show that the total amount of recharged water is greater when the basins are increased perpendicular (increasing the length "*L*" of the basins) to the direction of the original groundwater gradient rather than vertical (width "*w*" of the basin). The average increase in recharged water is 4% for a fully connected basin and 7% for clogged basins.

The increase in recharge by expanding the length of the basin is expected to be augmented with a combination of the following parameters: steeper aquifer hydraulic gradient, higher hydraulic conductivity and anisotropy ratio, and greater ratio of the infiltrating basin width *W* to aquifer thickness *H*. For the purpose of an efficient infiltrating basin design, an elongation of the basin length "*L*" is preferred over an elongation of the basin width "*W*". In addition, if the water table mounding is expected to reach the bottom of the basins, the average infiltrating rate will decrease with the basin's expansion of the surface area. The amount of recharge will not improve significantly over basins where their width "*W*" or effective radius "*R*" have surpassed 4 times the saturated thickness *H* of the aquifer, as estimated by Verruijt (1979) and as graphically displayed in Figure 3.3. This effect is exemplified in Figure 3.11, which shows a non-linear increase of recharge rate with the expansion of surface area for basins with a direct hydraulic- connection between the surface and the underlying aquifer.

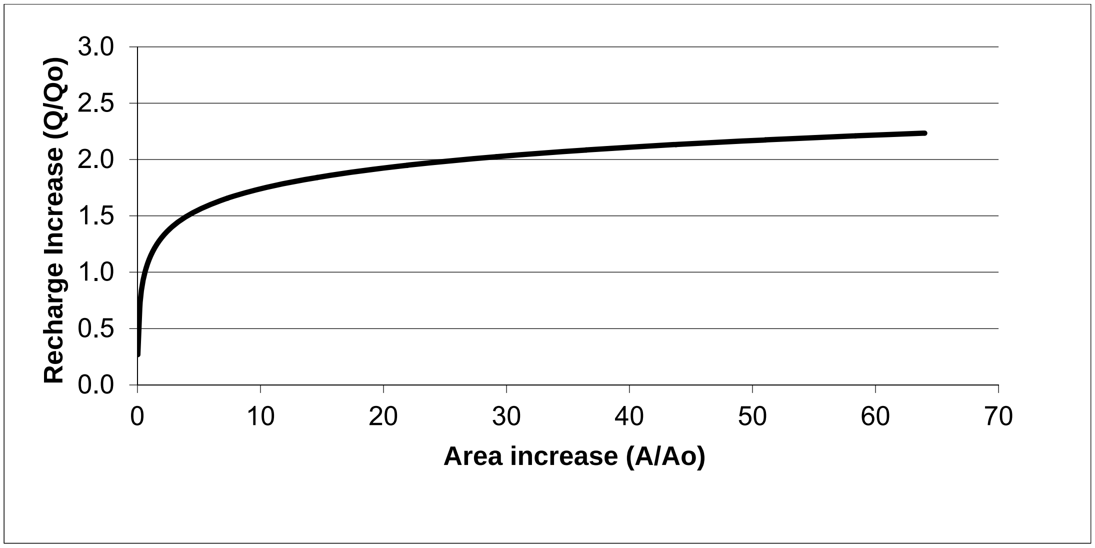


Figure 3.11 Simulated recharge rates for basins where the groundwater mound reaches the surface.

3.4 Conclusions

Infiltrating basins for managed artificial aquifer recharge projects are commonly evaluated by pilot projects with a smaller surface area than the full-scale projects. In order to extrapolate recharge rates from pilot tests, it is necessary to evaluate the maximum height of the fully-developed groundwater mound. In cases where the groundwater mounding doesn't reach the bottom of the infiltrating basins (such as when a thick clogging layer is present), infiltrating rates are kept constant throughout the basin's floor and recharge rates could be extrapolated linearly to the expansion of the infiltrating basins surface area. However, in the cases where the groundwater mound reaches the surface, infiltrating rates (entry velocities) vary across the infiltrating basin surface area. The rates are lower in the center and increase radially to their maximum rate at the basin's outer circumference (perimeter). Since most of the water infiltrates at the basin outer circumference, recharge rates should be linearly extrapolated to the increased infiltrating basin's perimeter.

Simulated results for fully connected basins compared well to the analytical solution offered by Verruijt (1979) as expressed by Huisman and Olsthoorn (1983). The entry rate at the center of the basin reduces as the infiltrating basin width increases over the aquifer thickness. The design for the most efficient infiltrating basin geometry could employ the Verruijt (1979) equation considering both the groundwater mound dynamics and the basin orientation in relation to the regional groundwater gradient.

Chapter 4 - Groundwater tracers from artificial aquifer recharge basins in the Walla Walla Basin

Authors: Petrides1*, A.C., Steven Patten, J. Selker, R. H. Cuenca.

Submitted to: The Walla Walla Basin Watershed Council as technical manuscript (modified version). June 29, 2010. Available at: http://www.wwbwc.org/
Address: 810 S Main Milton-Freewater OR 97862 USA

Abstract

The goal of this experiment was to determine the travel times of groundwater recharge delivered through infiltration basins from the Hudson Bay Aquifer Recharge Project to down-gradient monitoring points including two springs located 1.6 km from the recharge project. Bromide was selected as the most reliable tracer from a variety of possible chemicals and naturally occurring isotopes due to back ground levels. During aquifer recharge operations, the inlet and outlet of the infiltration basin were closed and 30 kg of potassium bromide were injected at a rate calibrated to match the rate of drainage from the infiltrating basins. Seven wells around the infiltrating basins and two springs were sampled during this experiment. Water samples were analyzed for major anions. Travel times were calculated from the time of the tracer injection and the detected peak concentration at monitored locations. Results showed a hydraulic connection from the infiltrating basins to the targeted springs for restoration with high (60 m/day) detected groundwater velocities. Computer simulations were calibrated to show the distribution of groundwater velocities around the infiltrating basins. Lateral visualization of the simulations show that most of the recharged water flows through a thin first aquifer layer above the original water table. Travel detection times at the monitored springs cannot be explained by radial flow theory under the assumption of bulk aquifer transport of the tracer. A plug flow equation is reviewed from the transport equations presented by Huisman and Olsthoorn (1983).

4.1 Introduction

The Walla Walla Basin Watershed Council (WWBWC), in conjunction with the department of Biological and Ecological Engineering at Oregon State University, has been studying the feasibility of an artificial aquifer recharge program in the Walla Walla Basin. Aimed at restoring groundwater levels, springs and stream flows, the pilot projects of artificial aquifer recharge have proven to be successful in the basin (Bower 2009). An example of such projects is the Hudson Bay Irrigation District Company Aquifer Recharge project (HBIDC). Springs down gradient from the project after 25 years of flow cessation appeared to have restored flows (Bower 2009). This research project studies the possible hydraulic connection between the spring and the infiltrating basins of aquifer recharge. The study compares observed water level response in the springs to estimated travel times obtained from; (A) analytical solutions, (b) computer simulations and the results from a tracer test.

4.1.1 Study location: Hudson Bay Irrigation District Company Aquifer Recharge project (HBIDC).

The HBIDC project operates during the months of “excess” water (non-irrigation season, November through May) by diverting water into rectangular recharge basins through the existing network of irrigation canals. The rectangular spreading basins have a combined surface area of 10,000 m^2 (Bower 2009) achieving a steady state recharge rate of $0.5 m^3/sec$. The underlying gravel aquifer is composed of two geological unit layers. The first unit is made of un-cemented silt gravels of the quaternary period with an estimated thickness of 10 meters. The second unit from the Mio-pliocene era is composed of cemented silty sand gravels with an estimated thickness of 60 meters (Lindsey 2004). The water table generally resides in the Mio-pliocene unit throughout the year, except when the artificial recharge project is in operation. The horizontal hydraulic conductivity of the Mio-pliocene unit was estimated to be 32 m-day^{-1} (Petrides 2008). This estimate was obtained from four aquifer tests conducted in 2007, and calibrated with a finite element hydrological model. Under natural conditions with a typical hydraulic gradient, longitudinal and transverse dispersivity for gravel aquifers can be expected to be in the range of 1 m and 0.1 m respectively (Freeze & Cherry 1979).

Water quality has been monitored and reported throughout the basin. Since the beginning of operations in 2004, the HBIDC project and(Bower 2009) and Locher Road aquifer recharge project (Lindsey 2007) water samples have been taken from the infiltrating basins, irrigation canals, wells throughout the basin, and down-gradient springs. Water samples are tested by independent laboratories (Bower 2009) for numerous constituents, including bromide, the tracer used in this study (table 4.1). Based on the water quality results at monitored locations, aquifer recharge has not shown any undesirable effects on water quality in the basin.

Table 4.1 Water quality throughout the basin.

Water quality characteristic	Hudson Bay Groundwater quality	Surface water source for Hudson Bay	State line Groundwater quality	Surface water diversion at Locher Road
pH	7.5	7.8	7.24	8.02
Nitrate-N (mg/L)	2 – 2.5	0.2	5.68	0.38
Total Dissolved Solids TDS (mg/L)	120 to 150	20.3	253	95
COD (mg/L) Chemical Oxygen Demand	11-55	21	8.0	8.0
Electrical conductivity (ms/cm)	69.5	50.5	401	95
Turbidity (NTU)	N.D.	N.D.	0.89	6.28
Hardness (mg/L)	N.D.	N.D.	217	45.2
Soluble reactive Phosphorous (mg/L)	0.19 -0.206	0.21	0.043	0.043
Total Coliform (per 100 ml)	Present	Present	Absent	Present
E-Coli (per 100 m)	Absent	Present	Absent	Present
Herbicides and pesticides EPA SOCS	Di (ethyllhexyl)-phthalate	No data	Bromacil and Di (ethyllhexyl)-phthalate	No data
Chloride (ppm) **	1.1	0.6	6.6	40 (Lindsey2007)
Bromide (ppm) **	No detection	No detection	0.003	No data

* Data source Hudson Bay: Bower 2004 and for Locher Road: Lindsey (2007)

**Bromide & chloride concentrations analyzed at the Institute for Water and Watersheds laboratory (July 2008) using Ion chromatographic techniques.

4.1.2 Tracers evaluation

After consideration of a variety of materials commonly used as groundwater tracers, the anion Bromide (Br^-) was chosen to be used as a tracer for this experiment. The reviewed groundwater tracers for this experiment include:

1) Isotopes. Isotopes are natural tracers that can be used to tag water with a (+/-) standard deviation of 2 years (Mc Dermott 2008). In this experiment, due to expected high gradients and relatively short distances between the injection and measurement points, natural isotope tracers would not provide enough signals to differentiate the water from the aquifer recharge project from water which leaked from irrigation canals.

2) Fluorescent Dyes. These allow detection at very low concentrations and sometimes are used to test the retardation factor of various organic compounds (Field 1995). Rhodamine and Sodium fluorescein were suggested to the watershed council as tracers in conjunction with bromide. The watershed council decided against the use of dyes over potential concerns with water users down gradient from the recharge project who could potentially have visible levels of dye in their water.

3) Synthetic gases are used as an alternative when background concentrations of Cl^- and Br^- are high (Mc Dermott 2008). Synthetic gases were not chosen because of the high hydraulic conductivity in the unconfined gravel aquifer being tested, the synthetic gases could easily escape from the aquifer before reaching the targeted springs, therefore eluding detection.

4) Inorganic Anions. Chloride and Bromide are considered conservative tracers. In this experiment, chloride was ruled out since it was present at significant and variable concentrations throughout the basin. Bromide, however, had not been detected in three years of groundwater sampling.

Ultimately, bromide was chosen for this experiment based on excellent health safety characteristics, very low background concentrations (allowing for lower applied mass), lack of chemical interaction with the media, reasonable cost of purchase, and ease of analysis. Bromide is known as the foremost "conservative tracer," meaning it characteristically travels at the same speed as the water it is dissolved in (rather than "sticking" to the aquifer sediments and moving more slowly than the water itself): Levy and Chambers (1987), among others, showed that bromide does not undergo chemical or interactions within the aquifer media, such as adsorption or retardation and is not subject to biological alterations. The safety of bromide as a tracer in aquifers is well established. According to Flury and Papritz (1993), concentrations below 1 mg/L have no toxic effects on aquatic organisms. Other examples of experiments using bromide as a tracer are available from the USGS Bromide Stream Tracer Reference List (9). In the USGS list, concentrations of 10 ppm were frequently employed, being also known to be without any toxic effects at these concentrations. For a tracer to be useful, its presence must be evident even at very low concentrations. There was no detectable bromide in the targeted study area of the gravel aquifer over three years of groundwater sampling in any of the artificial recharge sites or the targeted springs. The low natural concentration of bromide allows small quantities of tracer to tag great volumes of water.

4.1.3 Analytical solutions to estimate travel times from aquifer recharge basins.

The engineering design of infiltrating basins for aquifer recharge should include the basic notions of the amount of water that can be stored in the aquifer and the travel time and direction of the recharged water. This research investigates proposed analytical solutions for estimating the detection time from traced water of infiltrating basins for aquifer recharge. The travel time (T) is calculated from the observed detection time of arrival of the peak concentration from the time of the tracer injection. Travel times can then be used to solve for the aquifer parameters such as effective porosity and aquifer thickness.

Travel times of groundwater from artificial aquifer recharge projects are difficult to estimate analytically due to the unknown horizontal and vertical distributions of groundwater velocities during the operation of the infiltrating basins. McDermott (2008) found that travel times estimated by analytical solutions correlated well with depth (using a vertical distribution of hydraulic conductivities) but not with horizontal distance from the infiltrating basins. This research reviews the analytical solutions for travel times form aquifer recharge projects comparing them to computer simulations. Tracer experiments and hydrological simulations with computer models have been used in the past to understand the velocity distribution of infiltrating basins for aquifer recharge (McDermott 2008, Clark 2004, Poeter 2005). For artificial aquifer recharge projects, tracers have been used to study hydraulic connectivity, flow paths, groundwater velocity and mixing between native and recharge water (Clark 2007). Results from these experiments were used to estimate effective porosity and hydraulic conductivities of the aquifer. Stephens (1998) showed that effective porosity predicted from textual data and moisture content can be 50 to 90% greater than field calibrated values obtained from tracer tests.

Groundwater travel times are calculated by combining Darcy's law and the continuity equation under the assumptions of homogeneous aquifer with constant recharge and water boundaries far away from the recharge location. Huisman and Olsthoorn (1983) present the following derivation for steady on-dimensional flow of groundwater from infiltrating basins of aquifer recharge.

Darcy's law $q = -kH \dfrac{\partial h}{\partial x}$ Eq. 4.1

Continuity $q = q_0 = Constant$

Combining $\partial h = -\dfrac{q_0}{kH} \partial x$ Eq. 4.2

Integrating $h = \dfrac{-q_0}{kH} x + C$ Eq. 4.3

Where:

q = groundwater flow per unit of aquifer (m^2/d)

K = Hydraulic conductivity (m/d)

H = Aquifer thickness (m)

h = Water level elevation (m)

C = integration constant (m)

With the limits of integration at far distance *L* from the recharge area the piezometric level is maintained at its original value h_0

At $x=L$ $C = h_0$

$s_0 = h_{\max} - h_0 = \dfrac{q_0}{kH} L$ Eq. 4.4

Where: S_0 = Drawdown of water level (m)

The detection time *T*, in the aquifer is:

$T = \dfrac{nHL}{q_0}$ Eq. 4.5

Where, $q_0 = wv_e$

w = width of the infiltrating basin

v_e = Infiltrating rate

Substituting equation 4.5 into equation 4.4 yields a relationship between the maximum head change s_0 and travel time.

$$s_0 = \frac{nL^2}{kT} \qquad \text{Eq. 4.6}$$

Gelhar (1993) proposes the use of the average head change observed from the center of the basin to the head elevation estimated at L distance.

$$T = \frac{nL^2}{k\bar{h}} \qquad \text{Eq. 4.7}$$

Equation 4.7 presents a radial flow equation for groundwater to flow from the infiltrating basins. Radial flow doesn't take into consideration the original gradient of the underlying water table. The equation also should be used with careful consideration of the flow pathways for water to flow through the subsurface. The flow paths are estimated through geologic cross sections constructed from well logs. In the area, several clay lenses have been identified. However, their extent and thickness is unknown. Old channel beds have also been identified in the gravel aquifer providing areas of high hydraulic conductivity. The Hanford gravel aquifer (200 Km from the Walla Walla) resembles the same hydro-geologic description of the Quaternary alluvial un-cemented gravel aquifer in the Walla Walla Basin. The reported hydraulic conductivities for the quaternary alluvial formation range 150 m/day (GSI 2004)

4.2 Materials and methods

4.2.1Experimental set up; Injection and sampling methods

The goal of this experiment was to measure the time it takes for water to travel from the infiltrating basins to the springs during aquifer recharge operations. To differentiate water from the recharge basins from ambient groundwater flow in the aquifer, the chemical potassium bromide was added to the water percolating through the infiltrating basins. The concentration of bromide was then measured at observation wells and ultimately in springs located 1.6 km away from the recharge project. Figure 4.1 shows the injection point and the sampling locations.

Figure 4.1 Groundwater sampling points (green points) where tracer

concentration was measured. Point of injection (POI) was the infiltrating basin #2 of the HBDIC aquifer recharge project (red marker).

The quantity of bromide necessary to tag the groundwater while maintaining concentrations well below those toxic to the aquatic organisms was calculated using one volume of water recharged by the infiltration basins. Infiltration basin # 2 was chosen for injection of the tracer because it was the only basin with an effective mechanism for controlling the inflow and outflow of water. This basin has a volume of 1600m^3, and it was determined in a previous study (Bower 2010) that the basin takes 6.5 hours to drain. The infiltration rate is determined by dividing the basin volume by the time it takes to drain. In this case, the infiltrating flow rate was calculated to be 70 liters/sec.

The total volume recharging the aquifer from the four basins is 8,700m^3. It is assumed that the tracer concentration is diluted as the infiltrating water from this basin mixed with the groundwater flowing in the aquifer over the entire aquifer thickness. For the peak concentration of bromide expected at the springs to be less than 1 ppm, the maximum concentration targeted for bromide was 10 ppm in 1 of the 4 recharge basins. The following steps were followed to calculate this concentration and the amount of tracer required.

Step 1: Estimate the concentration of the bromide solution, adding this solution at the same rate that water drains from the recharge basin.

$$\frac{Q_r * C_{tag}}{Q_p} = C_{in} \qquad \text{Eq. 4.8}$$

where:

Q_r= Recharge flow rate (L^3 /T)

C_{tag} = Target tracer concentration (M/L^3)

Q_p= Injection flow rate (L^3 /T)

C_{in} = Tracer concentration in release solution (M/L^3)

For this experiment Q_r = 70 (liter/second), C_{tag}=10 (mg/liter), Q_p=0.95 (L/min) were obtained after conversions and C_{in} = 43,750 (mg/liter) was obtained.

Step 2: Estimate the mass of tracer required

$$V_c * C_{in} = M_t \qquad \text{Eq. 4.9}$$

Where:

V_c= Volume of container with the release solution (L3)

C_{in} = Tracer concentration in release solution (M/L3)

M_t = Mass of Bromide tracer (M)

For this experiment, we used a 400-liter container and the target tracer concentration of 10 ppm for the infiltrating basin #2. Using these values, *M* of bromide was estimated as 17.5 kg = 400(liters) * 43,750 (mg/liters).

Step 3: Estimate the amount of Potassium Bromide needed to obtain the desired mass of bromide in solution. This is obtained by the ratio of the molecular weight of Bromide (Br;79) to Potassium Bromide (KBr;119) as:

$$M_t * (119/79) = Mkbr \qquad \text{Eq. 4.10}$$

where:

Mt = Mass tracer (M)

Mkbr = Mass Potassium Bromide

For this experiment the Mass of Potassium Bromide required was estimated as 27 kg

Step 4: Account for the saturation concentration of KBr in the release solution with regard to temperature using the following curve

At 0°C ----53.5g; 25°C---70g; 100°C---102g. The level of saturation of KBr in the release solution for this experiment is 11% assuming a 20°C at the time of the experiment.

Following the injection, samples were withdrawn from a set of monitoring points (figure 4.1). Personnel from the Walla Walla Basin Watershed Council, in conjunction with OSU team, set up ISCO automatic sample collectors at seven locations (figure 4.1). These were setup at five wells and two springs. Figure 4.2 shows an ISCO set-up at GW_118.

Figure 4.2 Tracer injections at Hudson Bay aquifer recharge project. 400-liter container pumped at 950ml/min using a peristaltic pump.

Sampling the groundwater tracer at the springs proved to be a challenge. As soon as the water emerges from the ground to form a spring (now considered surface water) it travels much faster than the groundwater. Johnson and Dugger Creek also have irrigation ditches that divert excess water into the spring-fed channels further downstream. Measuring the spring water far from its source will cause the tracer to be diluted. This in turn will distort the resulting breakthrough curve (a graph of concentration versus time at a given location) making data analysis far more difficult. The automatic samplers were initially placed at secure locations downstream from spring headwaters to avoid vandalism. However, these locations were not optimal for obtaining reliable results from collected samples. Table 4.1 in the results section shows the distance from the point of injection to the sampling locations.

The ISCO samplers each hold 24 collection bottles. Each was programmed to pump every hour, beginning on the day prior to the injection. Watershed Council personnel collected samples daily, transferred them into 30 ml bottles with plastic caps and labeled them. Randomly selected samples of groundwater were then tested using a Hachsension2 (model number 09060C002349)Bromide Ion-selective electrode as a qualitative indicator of presence of the tracer. These tests only determined the presence of bromide above the detection limit of 0.01 ppm.

The samples were sent to OSU for analysis. All of the groundwater samples were analyzed using Ion chromatographic methods (Dionex model ICS-1500-System-AS40). The detection limit of the ion chromatography is 0.001 ppm for major anions. These include: fluoride, chloride, bromide, nitrate, phosphate and sulfate. Although this experiment was originally intended to only test bromide concentrations, two other constituents appeared to warrant further analysis. Nitrates were low, measuring between ‘below detection limit’ and 0.01 ppm in the surface water used for aquifer recharge. After 800 meters of subsurface travel, the groundwater samples contained up to 10 ppm of the nitrate anion [note: this is well below the drinking water standard which is 10 ppm of nitrate-nitrogen. 10 ppm nitrate-nitrogen (NO_3-N) are equivalent to 44.3 ppm nitrate anion (NO_3^-)]. Sulfate was the second constituent that deemed worthy of further analysis, as it was determined there might be a possible correlation of time spent in the aquifer and sulfate concentration. The concentration of sulfate observed in the surface water and wells near the aquifer recharge project was 0.8ppm. Concentrations at the wells near the springs averaged 6.0 ppm.

Figure 4.3 Automatic sampler setup at observation well Gw_118.

4.2.3 Computer simulations of travel times and groundwater velocities from infiltrating basins, model development

Artificial aquifer recharge by spreading basins has been simulated in the past by a number of methodologies. These include: resistance network analogs (Bouwer 1962), Hele-shaw models (Marino 1967), sand tank models (Rao and Sarma 1980), and computer numerical simulation (Sumner 1999). With the new technological advances in computer science, numerical simulation of flow through variably saturated media for models larger than 1 km^2 are now possible to simulate with considerably small resources. HDYRUS 2D/3D is a Windows based computer model that solves the Richards equations for saturated-unsaturated flow and convection-dispersion equations for heat and solute transport (Simunek 2007). HYDRUS 2D/3D has a powerful graphical user interface that allows visualization of groundwater head and chemical concentration changes in time per node in the model domain.

The research uses the previous HYDRUS model developed for the analysis of scaling infiltrating rates from pilot projects of artificial aquifer recharge analysis (as discussed in Chapter 3) to simulate the transport of the groundwater tracer. The model was calibrated using groundwater levels measured at 17 locations for the same period of the tracer experiment. In general, the model simulates an area of 9.75 km^2. The infiltrating basins are simulated in the center of the model domain and the springs discharge at the lateral extent of 1.6 km away from the basin at the edge of the model boundary. The unconfined aquifer thickness decreases linearly from 70 to 35 m and consists of two highly permeable materials. The thickness of the first layer also decreases in the model structure linearly from 12 m to 3 m (Figure 4.9). The initial conditions are set with an average water table elevation before recharge operations (11 meters below the infiltrating basins surface). To best resemble the experimental setup, the simulation begins 20 days prior to the solute tracer injection. The groundwater mound reaches a quasi-steady state at about 10 days after beginning of operations (10 days before the solute injection).

4.3 Results

4.3.1 Groundwater tracer detection

The tracer was injected on January 19, 2010 at 11:54 am. Figures 4.4and 4.5 show the concentration of bromide sampled at observation wells GW_46, GW_118, and GW_65 The latter is located near the headwaters of Johnson Creek Spring and Dugger Creek Spring

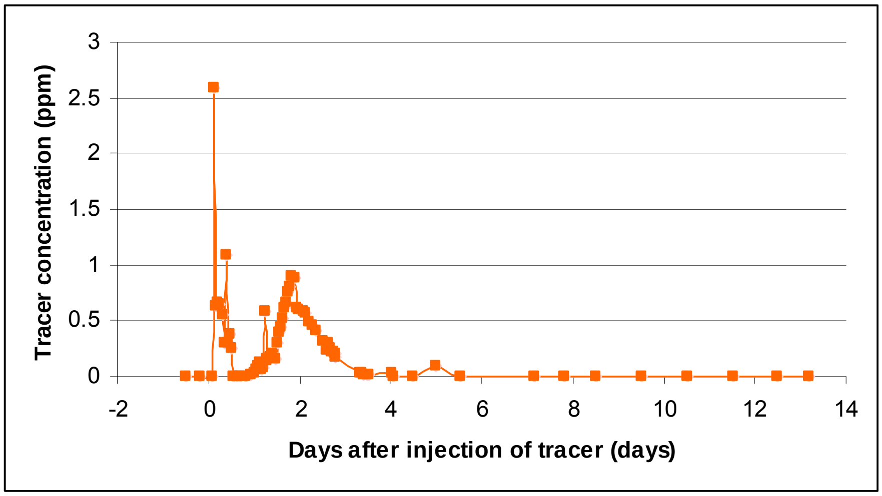


Figure 4.4 Bromide concentrations at GW_46. This sampling point is located 140 meters northwest from the point of injection.

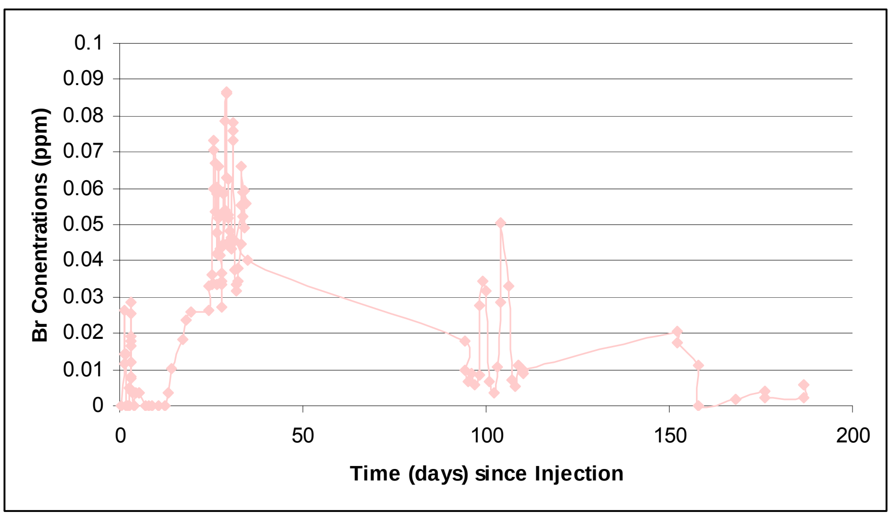


Figure 4.5 Bromide concentrations at GW_65. This sampling point is located at 1,360 meters northwest from point of injection.

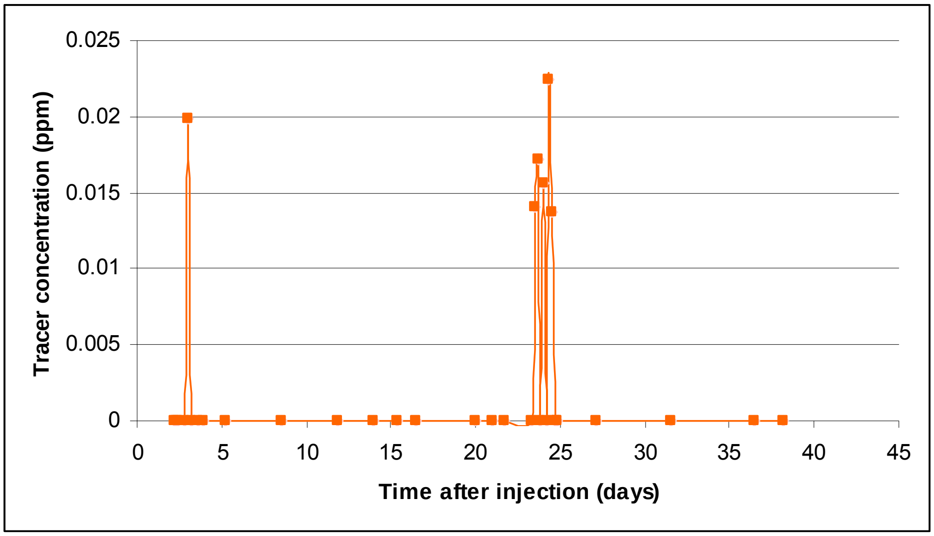


Figure 4.6 Bromide concentrations at Johnson Creek Spring. This sampling point is located at 1,600 meters northwest from point of injection.

The lag time for the tracer (second peak) detected at the springs and wells corresponds to an average water velocity of 60 m/day. Travel times and the estimated effective porosity and aquifer thickness are shown in Table 4.2.

Table 4.2 Observation points and distance from the point of injection. "Lateral" indicates a sampling location not directly down-gradient of the infiltration basin.

Location Name	Distance from the injection (m)	Time of peak arrival (days)	Inferred velocity to location (m/day)	Calculated Aquifer thickness H (m)	Calculated porosity (volume fraction)
Well GW_46	140	1.8	76	61	0.33
Well GW_48	140 (lateral)	3.2	43	108	0.59
Well GW_118	974	24.1	40	116	0.63
Well GW_65	1360	25.2	54	72	0.39
Johnson Creek Spring	1777	24.5	73	64	0.35
Dugger Creek Spring	1688	24.5	69	68	0.37

Aquifer thickness *H* was calculated from Equation 4.8 assuming a porosity of 0.3 in a constant thickness confined aquifer with a groundwater mound fully saturated the aquifer and the initial vadose zone. The calculation for well GW-46 is shown as an example:

Assuming a porosity of 0.3 and q_0= entry velocity * width of the basin

q_0=7 m/day * 200 m = 1400 m^2/day

Solving equation 4.5 for *H* and using the values from Table 4.2

$$H = \frac{T * q_0}{n * L} = \frac{(1.84)*(1400)}{(0.3)*(140)} = 61.3(m)$$

Effective porosity, *n*, is calculated in the same manner, by assuming a fixed aquifer thickness *H* of 55 m (geo-statistical reference, Lindsey 2004). Solving equation 4.5 for *n* yields:

$$n = \frac{T * q_0}{H * L} = \frac{(1.84)*(1400)}{(55)*(140)} = 0.33$$

The observed groundwater gradients from the infiltrating basins to the monitored wells shown in Table 4.3 along with the estimated hydraulic conductivities (K) from the radial flow equation (Eq. 4.7)

Table 4.3 Estimated hydraulic conductivities from the observed arrival times

Location Name	Distance from the injection (m)	Time of peak arrival (days)	Inferred hydraulic conductivity K (m/day)	Hydraulic gradient %
Well GW_46	140	1.8	58	7
Well GW_48	140 (lateral)	3.2	33	7
Well GW_118	974	24.1	214	2
Well GW_65	1360	25.2	577	1
Johnson Creek Spring	1777	24.5	703	1
Dugger Creek Spring	1688	24.5	634	1

The hydraulic conductivity shown in the third column of Table 4.3 was calculated by solving Eq. 4.8 and the observed travel times. As an example, hydraulic conductivity for GW 46 was calculated as follows:

$$k = \frac{n * L^2}{T * \bar{h}} = \frac{(0.3)*(140)^2}{(1.8)*(55)} = 58.1$$

The hydraulic gradient (*i*) is the observed change in groundwater head elevation from the infiltrating basins (Δh) divided by its distance (Δz) to the monitored well. As an example, the estimated gradient for GW46 was calculated as follows:

$$i = \frac{\Delta h}{\Delta z} = \frac{262 - 252}{140} = 0.071$$

4.3.2 Results of simulation analysis

The HYDRUS 2D/3D simulation model was already calibrated to observed groundwater elevation at 17 locations for the tracer test period (see Chapter 4.2). Due to long run times of the simulation model with solute tracking enabled (for example, a 180 day simulation with solute transport takes HYDRUS approximately 7 days to calculate, whereas the same simulation without solute tracking would only require 4 hours to calculate), the model was not recalibrated for solute data. The calibration was deemed sufficient for purposes of the tracer experiment. Furthermore, the model domain was cut in half (symmetric problem figure 4.8) to obtain faster simulation runs (this reduced run times to 3 days).

For the length of the simulation, the model was able to detect the tracer at GW 36 (located 140 meters from the POI) with a close approximation for the time of detection of the peak of the concentration. Figure 4.7 shows the match of the observations to the simulated experiment. The simulation, however, was not able to predict the tracer at wells with a radial distance from the basin longer than

1km. The model was helpful to conceptualize the distribution of groundwater velocities throughout the model domain. Figures 4.8 and 4.9 show cross-section views of simulated velocities.

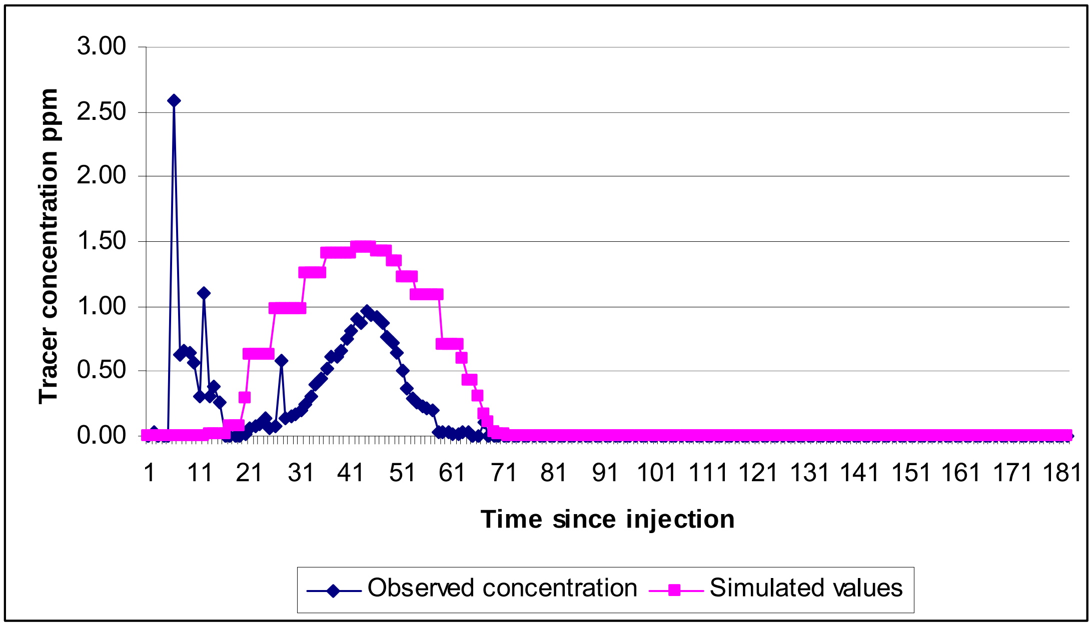


Figure 4.7 HYDRUS 2D/3D simulations of the tracer test at observation node140 meters away from the infiltrating basin injection.

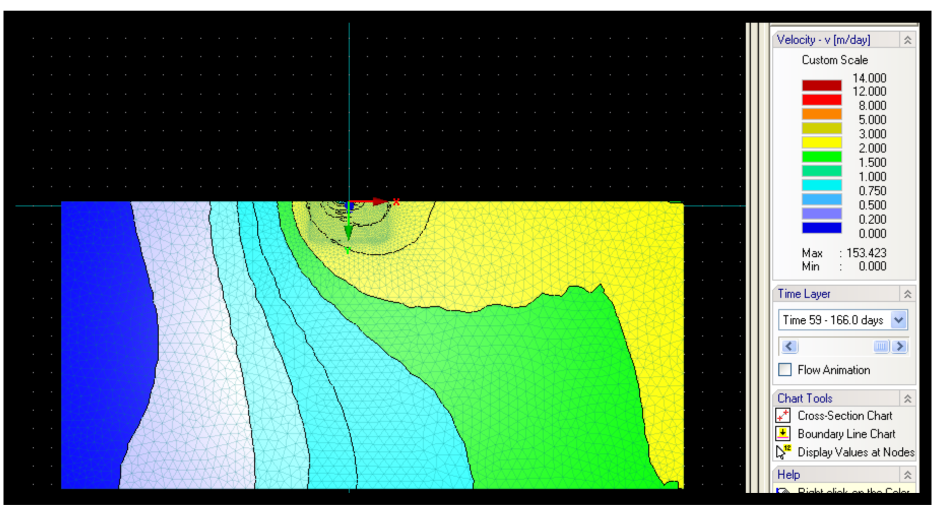
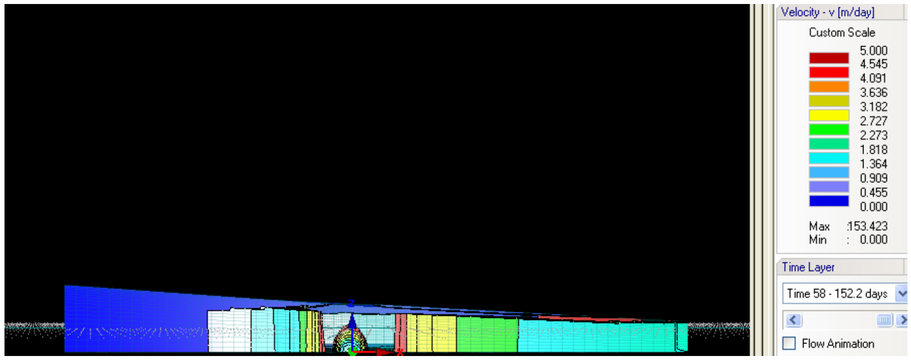
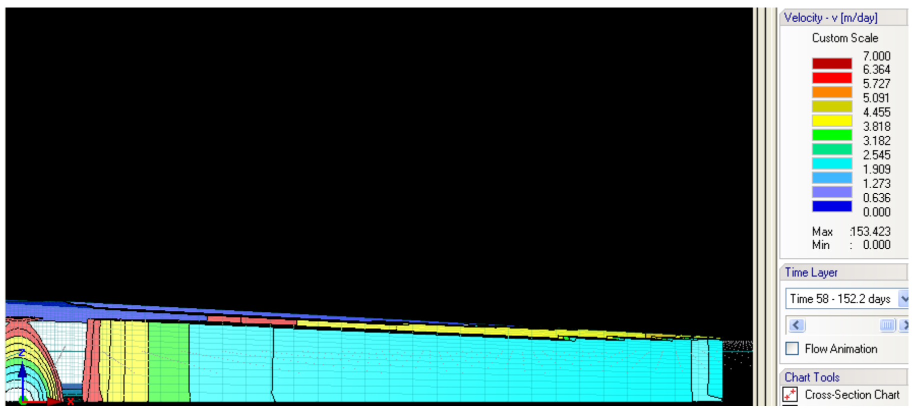


Figure 4.8 HYDRUS 2D/3D output screen of groundwater velocities lower boundary view in the X(width)-Y(Length) direction. “View of the bottom boundary”



A) Model domain cross section. Infiltrating basins are located in the center of the model domain



B) Cross-section of the right half of model domain. The graphics show a much higher flow velocity through the first layer of the aquifer.

Figure 4.9 HYDRUS 2D/3D output screen of groundwater velocities from the infiltrating basins (center of the model domain) to the boundary springs at the edge of the model domain. The screen shows cross-sectional view (lateral view) in the Z (depth)-X (width) direction.

4.4 Discussion and conclusions

The goal of this experiment was to prove a hydraulic connection between the infiltrating basins of artificial aquifer recharge and the targeted springs for flow restoration. The tracer bromide was chosen from a variety of natural and chemical tracers considered. Three years of groundwater and surface water samples prior to the experiment did not detect bromide in the area. The amount of bromide tracer necessary to tag the water at concentrations below toxicity levels was estimated to be 30 kg. The tracer was injected at the center of the basins at the same rate the infiltrating basins were draining. The tracer injection occurred on January 19th, 2010. Due to budget constraints, groundwater samples were collected only for 180 days after injection at 7 monitored locations equipped with automatic ISCO samplers.

The groundwater samples were analyzed for major anions utilizing an Ion chromatographic machine located at Oregon State University. The chromatographic machine has a manufacture reported detection limit of 0.001 ppm for bromide. The tracer bromide was detected at all sample locations; however, only location GW_46 (located 140 m down-gradient from the POI) had a Gaussian concentration curve typically taken to be indicative of the bulk aquifer transport processes expected in the tracer experiment. Travel times were estimated from the time of injection to the detected time of the peak of the concentration. The corresponding average groundwater velocity was calculated to be 60 m/day. The estimated groundwater velocity at all of the locations was within 1 standard deviation of the mean value, indicating a correlation between sample locations. The high velocities observed at distant wells and springs (located more than 1 km from the POI) indicate significant preferential flow within the groundwater system (i.e., small fractions of the system carrying much of the flow, as might occur along gravel “stringers” of former stream channels). The groundwater hydraulic gradient is 7% for the wells at 140 meters and 1% from the middle of the infiltrating basins to the head of the springs. Although high hydraulic conductivities have been reported for similar hydrogeological aquifer conditions (Hanford site), the detection times at the springs cannot be explained with the observed groundwater gradients and the analytical solution of radial flow Eq. 4.10.

Computer simulations of the tracer transport were made with HYDRUS 2D/3D. The model showed a close match of time of arrival but not of concentration at the observation well GW_65. Simulations indicated that based on the estimated hydraulic properties and calculated hydraulic gradients, the tracer would not have reached a radius of 1 km away from the infiltrating basins in 181 days under the assumption of bulk aquifer transport of the tracer. Due to the long running times required to perform the tracer simulations (4 days) the computer simulation was difficult to calibrate to the observed travel times detection. The model however gives a useful insight of the distribution of groundwater velocities. Figure 4.9 shows that most of the water recharged from the infiltrating basins flows through the first aquifer layer above the original water table. The velocity at this first aquifer layer is 3 to 4 times higher than the mean velocity of the entire aquifer thickness. The simulation model also shows the distribution of groundwater velocities around the infiltrating basins. From the bottom view of the model domain (Figure 4.8), we can observe that the radial flow assumption is violated and that groundwater velocities are axis-asymmetrical to the groundwater gradient of the original groundwater table. Travel time of tracers under this condition would be shorter to wells down gradient from the initial groundwater flow than in wells perpendicular or up gradient of the infiltrating basins. Future research is needed to explore an analytical solution that incorporates the effects of axis-symmetrical flow and the time to flow through the unsaturated zone currently omitted in Eq. 4.10 assuming a full saturated mound between the aquifer and the bottom of the infiltrating basins.

Chapter 5 - Shade estimation over streams using distributed temperature sensing

Authors: Petrides, A. C.; Huff, J.; Arik, A.; van de Giesen, N.; Kennedy, A. M.; Thomas, C. K.; Selker, J. S.

Journal: Water Resources Research.,
Journal Address: Journal of the American Geophysical Union (AGU) 2000 Florida Avenue N.W .Washington, DC 20009-1277 USA
Issue Vol. 47, No. 7, W07601 http://dx.doi.org/10.1029/2010WR009482
13 July 2011

Abstract

The characterization of temporal and spatial distribution of sunlight is essential for understanding energy transport in natural ecosystems. Fiber Optic Distributed Temperature Sensing (DTS) allows meter-resolution measurements of temperature at sub-minute resolution. The difference in temperature due to absorption and reflection of a pair of helically twisted black and white fiber optic cables was measured with a DTS to document areas exposed to sunlight over the Walla Walla River. A high correlation (R2=0.99) was found between DTS-based results and manual field observations of effective shade. These preliminary results provide a proof of the concept that this method can be used for estimating the effective shade at fine spatial resolutions. Potential short-comings and the need for a more quantitative physical model are suggested for further research.

5.1 Introduction

The use of Distributed Temperature Sensing (DTS), with fiber optic cables to measure temperature with high spatial (1 m) and thermal (<0.1°C) resolution over continuous spans (>10 km), has created great potential to study environmental dynamics (Selker et al., 2006a). DTS fiber optics recently have been used to study streams, wetlands, mineshafts, lakes, snow packs and soil moisture (Selker et al., 2006b, Lowry et al., 2007,Moffett et al., 2008, Tyler et al., 2009, Sayde et al., 2010). This note presents a proof-of concept of a technique to quantify meter-by-meter exposure to solar radiation using differences in temperature between white- and black-jacketed fiber optic cables due to their difference in short-wave albedo.

Effective shade is the percentage of the total solar radiation available above a canopy that does not reach the surface of interest. Effective shade is of great importance in stream temperature modeling since solar radiation represents the most significant input of energy(Ringold et al., 2003, Westhoff et al., 2007). Several methods have been employed to measure effective shade over streams (see Boyd and Kasper, 2004). Methods include manual delineation (e.g., Solar Pathfinder, Linden, TN, USA.), analog recorders (e.g., Campbell-Stokes Pattern Sunshine Recorder, Nova Lynx, Grass Valley, CA, USA) as well as canopy closure using hemispherical canopy photography (e.g., WinScanopy, Regent Instruments, Canada).

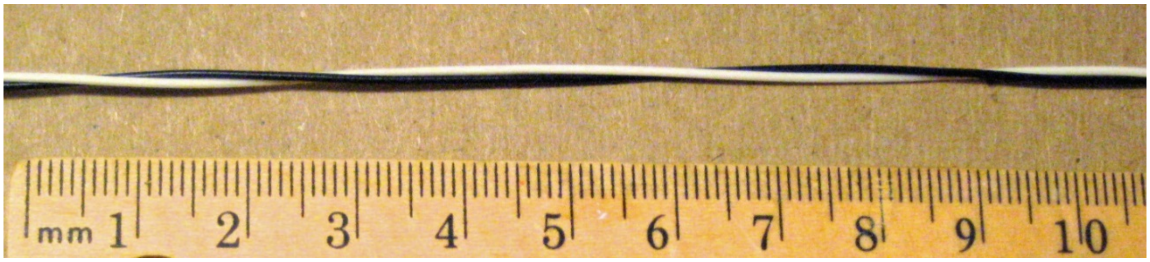
The aforementioned methods for estimating effective shade lack fine spatial resolution. They also provide static values rather than documenting time-varying conditions. Effective solar radiation is not uniformly distributed in natural systems, particularly when most of the provided shade comes from vegetation. Effective shade changes during the day and across seasons, due to changes in canopy closure, leaf area (e.g. deciduous trees) and solar angles. The objective of this paper is to demonstrate the feasibility of this approach to temporal delineation of solar radiation over river reaches.

5.2 Material and Methods

An experiment was conducted on the Walla Walla River (45°59’N 118°22’W) as it passes through Milton-Freewater, Oregon, USA, a third order stream on the north east side of the state. The stream has a well-developed woody-species riparian system with stream bank of about 2 m in height. These conditions generally lead to very low winds speeds above the stream. The average wind speed for the day was 1.4 m s-1. The near stream vegetation is mainly deciduous, with 5% of dry-land boarding the river free of vegetation (large gravel bars), 10% grass/herbs, 20% shrubs, 65% trees. The dominant tree species are cottonwood, white alder and willow (US Corp of Engineers, 2008). Air temperatures were measured with fiber optics connected to a SensorTran model 5100 DTS controller, recording for 24 hrs. at 11 min and 0.5 m resolution. SensorTran reported a DTS accuracy of less than 0.25 deg C for a distance of

4km and 0.5 deg C for a distance of 10km, both being averaged over a 5 min time period. The fiber optic cable assembly consisted of two (one black and one white) 900 μm diameter sub-cables helically wound with 10 wraps per meter. The helical winding assured symmetric environmental conditions for the two sub-cables.

Each sub-cable consisted of a sheathed multi-mode fiber (250 μm O.D. elastomeric tight buffered 50/125 μm multimode graded index fiber) with an aramid strength member contained in a polyurethane outer jacket (part number: 56 AFL Telecommunications, Duncan, NC) (figure 5.1). The installed cables were supported on 1 m tall “pig-tailed” electric fence posts (Supplement figure 5.2). The posts were placed 20 meters apart from each other in the center of the river. The actual height of the cables above the stream was on average 65 cm. During the experiment, the cables were severed by wildlife, shortening the original installation for the cable length from 1.2 km to 260 m. Solar radiation, reference air temperature, and wind data were obtained from a weather station of the Pacific Northwest Cooperative Agricultural Weather Network (Agrimet) located approximately 1,600 m from the sampling site. This station is equipped with Air Temperature Thermistor Model 44030, YSI, Inc., Pyranometer Model LI-200, Licor, Inc. and Wind Monitor Model 05103, R. M Young, Inc. For the first 260 m, four manual observations of effective shade were made with a Solar PathfinderTM as single-point measurements. The locations were chosen randomly and GPS surveyed with an average separation distance of 50 m along the river. The single-point measurements were taken the same day that the DTS system was running. The Solar PathfinderTM works by delineating the panoramic view in a convex plastic dome. The percentage of the available solar radiation is then calculated using a specific latitude sun path diagram for each month in the northern hemisphere. More information on the PathfinderTM can be found at http://www.solarpathfinder.com.



5.1. Photograph of fiber optic cable used in the experiments. The individual black and white cable elements are 900 μm O.D., and they are twisted to provide, on average, one full helical rotation per 0.1 m

5.2. Photograph of Arístides Petrides deploying the cable in the Walla Walla River, having just passed the cable through a “pig-tail” support post.

Generally, DTS measurements require in situ calibration of offset, gain, and slope parameters. In this experiment, we were particularly concerned with the accuracy of the differences in temperature between the white and black cables rather than absolute temperatures, thus precise offset values were not required. The global offset was computed based on measurements of night-time air temperature from nearby weather stations. For the purposes of this analysis, an area was considered to receive direct beam solar radiation when solar radiation measured at the near-by weather station exceeded100 W/m2.

First, the period of the day was established during which there was a difference between the black and white cables. In this case, this was the period between 8:00 and 16:00 on 16September 2008. The minimum and maximum total (diffuse + direct beam) solar radiation during this period was 194 and 704 W m-2, respectively. The average and standard deviation of temperature difference between the black and white cables was 0.39and 0.45 °C over the course of the daylight period. The white cable can become warmer than the black cable due to random noise of the DTS at low irradiances (<100W/m2) at times and/or locations where there is no direct exposure to solar radiation. However, negative values did not exceed two standard deviations. The range of temperature differences recorded was 2.83 to -0.83 °C for the entire cable length.

Second, the effective shade was estimated by the average temperature difference every 2meters (spatial average of 4 subsequent sections of 0.5 meters measured by the DTS, approximating the effective spatial resolution of the instrument) and at 44 minute intervals (temporal average of 4 readings of 11 minutes). The average temperature difference (ΔC) between black and white cables over an area was calculated as:

$$\Delta C = \frac{1}{n}\sum_{1}^{n}(T_B - T_A) \qquad \text{Eq. 5.1}$$

Where: = Average temperature difference [°C]

TB = Temperature of the black cable at x location [°C]

TA = Temperature of the white cable at x location [°C]

n = Number of periods in the time of the day during which there was a significant difference in temperature (here taken in 44 minute increments).

5.3 Results and Discussion

The temperature difference between the black and white fibers revealed a highly dynamic pattern with changes in time and space (Figure 5.3a). Manual observations of shade distribution were linearly correlated with the average temperature difference between fibers (regression coefficient R2 of 0.998, Figure 5.4). The effective shade was estimated by the temperature average over the daylight hours for each 2 m segment of the fiber optic deployment and indicated a fine-scale structure of shade distribution in this vegetated riparian system (Figure 5.3b).

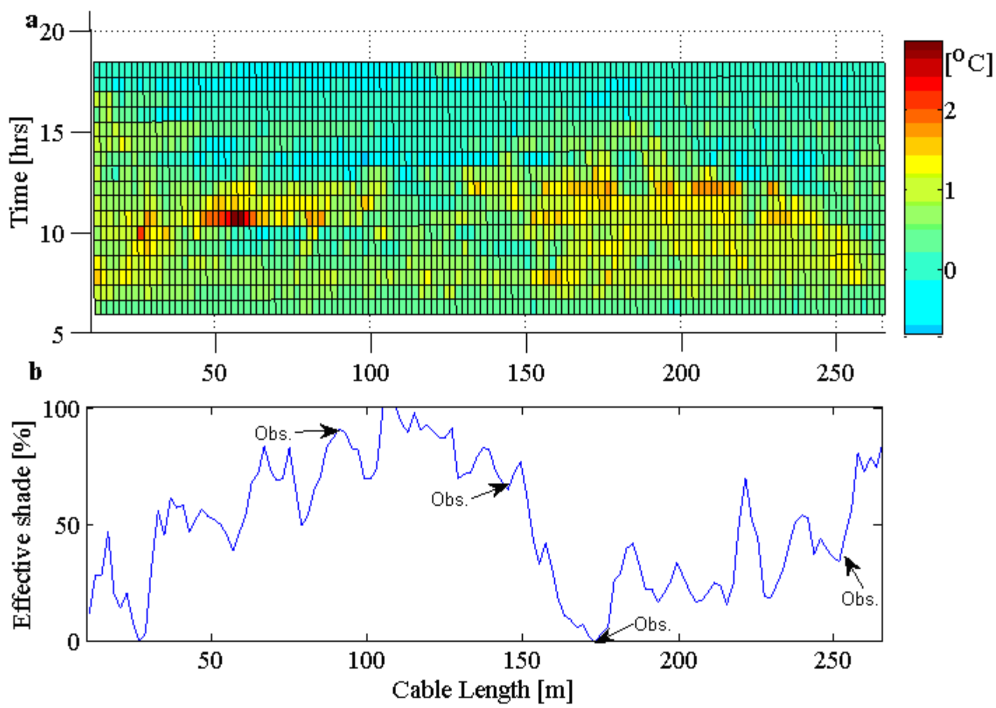


Figure 5.3. (a) Distribution of difference in temperature, average in time over 30 minutes (vertically) and averaged over 2 meters in space (horizontally) on the Walla Walla River. (b) Effective shade along the river segment. Four single-point measurements obtained manually with Solar Pathfinder are indicated.

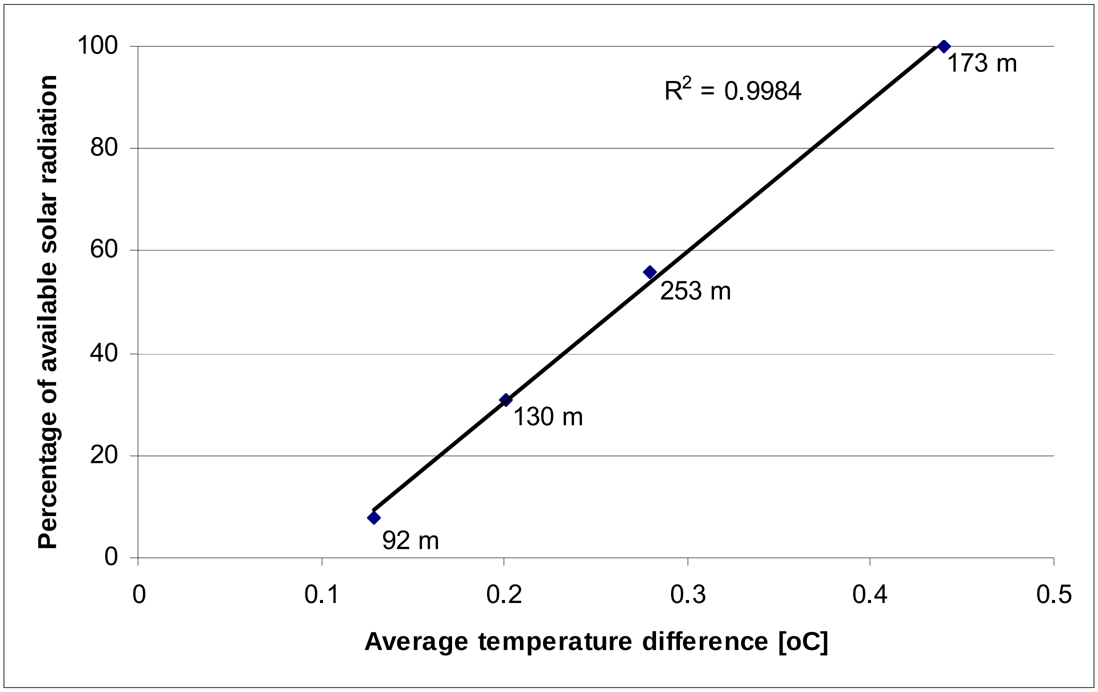


Figure 5.4.Comparison between Solar Pathfinder measurements and average difference in the fiber optic cables. The locations of manual observations are labeled in meters and shown in Figure 1b. The linear least-squared fit between the data is shown as a line, with an R2 of 0.998.

This field experiment demonstrated that a DTS fiber optic system is able to provide high temporal and spatial resolution delineation of solar exposure. This was achieved by using a duplex cable made up of black and white sub-cables. The average temperature differences due to differences in sunlight absorption between the black and white cables was found to be linearly related to the measured percentage of shade, providing a relationship for interpretation of the spatially and temporally distributed data.

Measuring the effective shade distribution using fiber optic cables has the advantage of providing meter and minute resolution measurements of shade distribution over long segments (>200 m). The data provided at such fine resolution is useful for stream temperature modeling and other biological assessments. This work describes a proof of concept. One potential short-coming of this method is its sensitivity to changes in wind speed along the fiber as it modulates the effective transfer coefficient of sensible heat to the air and thus the temperature difference between the fibers. However, solar radiation will be the dominant driver of temperature differences leading only to small uncertainty in estimation of effective shade. Further research and a more quantitative physical model are required to precisely estimate these effects. In this experiment we selected the lightest weight cables available that could be employed, however, we are interested in determining the relationship between cable diameter and response to see if there may be an optimal geometry. Here, we employed helically wound cables, which had the advantage of simplicity of handling, and assurance that the cables were in identical locations, however, having the cables touch, and shade each other reduced the signal strength. The effect of having the black and white cables touching versus having an air-gap separation deserves further study because of undetermined effects of heat transfer between the cables. Other areas of useful development would include the influence of cable composition and pigment; the effects of solar aging and the accumulation of particulate matter on the cables; and methods to enhance durability and reduced need for maintenance. For the foreseeable future, we believe that the method will require collocated wind, temperature, and solar radiation measurement at a minimum of one full-sun location, and ideally at an additional full-shade location. The method is not inexpensive, with the DTS unit costing >$20,000 and the cable about $2 m-1. Installation and operation are labor intensive at about one person day per 500 m. Also a continuous power supply is required. Finally, data analysis is involved and time consuming. In spite of the aforementioned research requirements, this method opens the potential for studies that could benefit from the measurement of effective shade at fine spatial resolutions. It should be noted that newer DTS equipment now can provide the level of precision obtained here in temperature integrating over 44 minutes every second and on 0.25 m resolution in place of the 2.0 m resolution obtained here (e.g., see CTEMPS.org),pointing out the rapid rate of improvement of performance in this quickly evolving branch of instrumentation.

Acknowledgments

This research was in part supported by the Walla Walla Watershed Council, the Oregon Watershed Enhancement Board, and the National Science Foundation (NSF EAR- 0711594). Field work at Walla Walla site was provided by Troy Baker and Nella Parks.

Chapter 6 - General Conclusions

This research thesis focuses on the conservation and restoration of water resources in the Walla Walla Basin. Due to the depletion of an unconfined gravel aquifer, the Walla Walla River and springs flows have been affected with decreased summer flows. New water management practices are being evaluated through local institutions and organizations. This PhD work serves as an independent scientific evaluation. Its purpose is to serve as a transparent tool to support decision. Although this work focuses in the Walla Walla Basin, its general findings and suggested methodologies can be applied elsewhere. This section of the document summarizes the major findings of the work completed throughout the PhD program.

The thesis starts with the development of a regional hydrological model and a general methodology to evaluate locations for artificial aquifer recharge. Besides the use of the model for the evaluation of water management scenarios, through its development, additional outcomes were achieved. First, the model assembles the available local hydrological information gathered from several organizations and provided a new level of quality control and assurance. Second, the model expanded the physical understanding basin’s hydrology by simulating the interaction between different components of the hydrological cycle. Finally, the model successfully proved to serve as a tool to evaluate water management scenarios. Two scenarios of lining irrigation canals were evaluated and quantified the decrease in the otherwise groundwater recharge. The model was also utilized in a suggested methodology to evaluate the feasibility of artificial aquifer recharge projects. The methodology first evaluates through an analytical solution the feasibility of the project. A brief overview of the most cited analytical solutions of groundwater mounding are presented. Once the analytical solution screens the first locations, a vadose zone model is suggested to determine the recharge rate achieved at field specific boundary conditions. If the location evaluated still meets the project needs, then the regional effects of artificial recharge of groundwater are evaluated in the regional IWFM model. An example case is presented for each step of the suggested methodology.

Pilot tests of infiltrating basins are commonly used to evaluate the feasibility of artificial aquifer recharge projects. They represent the most tangible option for evaluating locations. However, special consideration must be taken when parameters observed at small-scale projects are used to engineer full-scale projects. Chapter 3 of this thesis studies the scaling of recharges rates from pilot test to full-scale projects. Utilizing the data collected at the Walla Walla Basin recharge facilities, a 3-dimensional model was structured to evaluate recharge scenarios under different boundary conditions. Results show that when the induced groundwater mound does not reach the bottom of the basins, a linear relation can be established between infiltrating basin surface are and volumetric recharge rate. However, when the groundwater mound reaches the bottom of the basins, recharge rates should be scaled to the perimeter of the full-scales basins. A physical explanation of this effect is offered by exploring Verruijt's (1979)equation and comparing its results to the calibrated computer simulations.

The WWBWC explored pilot tests of artificial aquifer recharge in the Walla Walla Basin as a means for restoring flows in springs that ultimately flow into the Walla Walla River. Thanks to this pilot test effort, springs that were dry for more than 25 years have restored flows (Bower 2004). The connection between the pilot test and the restored flow in the springs was not well understood. Analytical solutions estimated that travel times from the infiltrating basin to the springs located 1.5 km away would take much longer time (years) than the observed restore flows. It was then believed that the restore flows were a result of the pressure wave induced by the pilot project and not the actual particles of artificially recharged water. In Chapter 4 of this thesis, a tracer test and computer simulations were employed to evaluate the connection between the pilot test projects and the springs. A series of chemicals was evaluated and bromide was chosen as a groundwater tracer due to its low, non-detected background concentration. Results show that water from the recharge basin travels in the top layer of the aquifer that has three times higher hydraulic conductivity than the bulk average of the aquifer. Due to the induced increase in water table gradient and the fast layer of hydraulic conductivity, the bromide tracer showed short travel times (weeks)to the springs proving an hydraulic connection of flow from the infiltrating basins. This chapter concludes with a warning to the assumptions of establishing analytical solutions that assume a full mix of the tracer with native groundwater that travels through the entire thickness of the aquifer.

Restoring flows in the springs and groundwater seepage is critical in the Walla Walla Basin to maintain cool summer flows into the Walla Walla River. Water temperature is an important criterion for restoration in the Walla Walla River, which serves as habitat for endangered salmonid species. Chapter 5 of this thesis estimates the exposure of the Walla Walla River to solar radiation. The thesis chapter is a proof of the concept of the use of black and white jacketed fiber optic cables that by the difference in observed temperature by a Distributed Temperature Sensing (DTS) technology, the effective shade over 24 hours can be estimated. The DTS technology allows meter-resolution measurements of temperature at sub-minute resolution. This application of DTS technology, although expensive (cost of the cable and labor intensive), can be used for temperature modeling inputs and other potential studies that could benefit from the measurement of effective shade at fine spatial resolutions.

This thesis addresses four major topics. First, it evaluates the feasibility of artificial aquifer recharge in the Walla Walla Basin by developing a regional hydrological model. Specific locations are screened on the basis of results from analytical solutions of groundwater mounding and vadose zone modeling. Second, it studies the scale dependence of recharge rates from pilot infiltrating basins of artificial aquifer recharge. Third, it evaluates the hydraulic connection between springs and infiltrating basins for artificial aquifer recharge by transport simulation models and a filed groundwater tracer experiment. Finally, it addresses the exposure of solar radiation over the Walla Walla River. This last chapter is proof of the concept of using distributed temperature sensing technology to estimate effective shade.

In addition to the thesis chapters, two projects are presented in the appendix to evaluate if future, more in depth formal research would be of adequate. The projects include; (a) bias in the parameter estimation to areas with high density of observations and/or areas with high correlated model errors. (b) Transferring a complex model from developers to final users. The conclusions from these additional studies suggest that further formal research on these topics would be a valuable contribution to the related literature.

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APPENDICES

A.1 Transferring a complex model: experiences of the regional hydrological model activities in the Walla Walla River Basin.

Need for the study

Hydrological models are complex; they required large amount of hydrological data to describe interrelations between physical processes and hydraulic features. The detail of representation and number of parameters are simultaneously perceived as a main strength and weakness (Freeze 1971, Parkin et al 1996). Even though hydrological models are usually available through governmental institutions, universities, and water institutions such as watershed councils, hydrological models tend to remain in the domain of the model developer and tend to be applied within a consulting framework (Taylor, Cameron, & Haines, 1998). Some institutions, such as the Walla Walla Basin Watershed Council, make great efforts to provide public access to their hydrological model and to the hydrological data used for its development. Some of the hydrological information is available for download through the internet albeit requiring the use of databases and GIS technologies.

While hydrological models and the hydrological data for model development are currently available for public use, hydrological models are inaccessible to decision makers and field hydrologist that are not modeling specialists (Taylor, Cameron, & Haines, 1998) and/or have not been instructed in the use of the model. Hydrological models have the potential to be very useful to society (Xi & Sing 2001). However, they are often doubted because of their complexity, uncertainty of data used in their calibrations/validation, and their lack of transparency. In this part of the research we describe a methodology to facilitate the transferring of complex hydrological models to their final users. In this section, will describe the experiences encountered during the development and validation of Walla Walla Hydrological Model.

Study objectives

The objectives of transferring a complex model to its final users are to increase model transparency and validate the hydrogeological data used for its development. Increasing the communication between the model developer and the final user allows for a better understanding of the model needs and its flexibility for manipulation to suite different types of applications. The final user validates the models general assumptions and applicability to its final use. The modeler is then able to incorporate the final user experience in the area. Transferring the model to its final users insures project continuation after model calibration and validation by achieving the goal of using the model for the purposes for which it has been created for. Finally, by the process of model transfer, public participation is encouraged inviting the use of the model as a way for organizations and institutions to share information. The transparency of the model structure helps decision makers and/or stakeholders utilize the model effectively in environmental decisions.

The objectives of this section of the thesis are to identify the key elements necessary for successful model transferring. From the experience gathered in the transfer of the Walla Walla model, some benefits and challenges of these activities have been identified. In the Walla Walla project, a series of model presentations and meetings between their technical review team (final users) and model developer set the goals and purpose of model use. The regional hydrological model created requires user instruction of model development and user expertise to make the output results useful. This research will attempt to specify the necessary level of knowledge required for model use.

Literature review: transferring a complex hydrological model

Technology transfer is defined by Charles and Howells (1996) as the diffusion of a complex bundle of knowledge which surrounds a level and type of technology. In this research, technology transfer relates to the education and training activities for the utilization of a complex hydrological model. Hydrological modeling requires user expertise in a wide variety of topics in hydrology and related science. Because hydrological models utilize large amount of information, knowledge of software database and Geographical Information Systems (GIS) is necessary. GIS is utilized for data organization and manipulation for model inputs. Also, for geo-reference models, GIS is utilized to retrieve model information.

There are several hydrological simulation codes available today (Xi 2007). Using an existing code eliminates the need for model programming. However, to suit diverse applications, many modelers customize existing codes, for which knowledge in computer programming its necessary. Some of the computer simulation codes in the market have a professionally developed user interfaces that facilitates visualization and model manipulation. Knill (1993) expressed the need for models to be user-friendly including visualization tools for model results. Kingston, Carver, Evans, and Turton (2000) suggest the development of an adaptive user interface that provides multiple interfaces for different skill levels of users. For example, hydrologists can be presented with a more technical interface than casual users. Models that lack a user interface relay on GIS for creation and input of data sets (DeBarry et al., 1999). GIS becomes the user interface to the model.

The spatial visualization aid of GIS in conjunction with communication technology to support and reinforce learning and teaching in geography and natural science has gradually increased over the past two decades (Castleford 1998). Case studies that have experimented with educational technology have seen the extensive availability of hydrological models for visualization and understanding of the relationships between hydrological components (Aghakouchak 2010). During the model transferring process, the model maybe used as a teaching tool to aid the understanding of the hydro-geological conditions in the basin. Field hydrologists and water managers perform adaptive management by looking at the interactions between different components of the hydrologic cycle. Field technicians re-evaluate their field data collection system by linking their hydrologic data collected to its final use and applications. For example, a field hydrologist who measures flow at irrigation canals can then learn the necessary parameters required for simulating surface water flows and its interactions with the groundwater aquifer.

According to Sol (1987), data analysis tools and models are not useful by themselves without a link to professional experience. Difficulty in linking data, analysis tools and models across organizations is one of the barriers to be overcome in developing a tool to support decision making. The model supports information exchanged and knowledge shared from different organizations. Experiences from field hydrologists and water managers from across organizations are taken into consideration by the model developer during the activities of model transferring. Andreu (1995) stresses the benefits of incorporating local knowledge and experiences into a decision support system model to aid in the management of hydrological systems. The model and the transferring process facilitate the communication between different organizations and stakeholders. Public participation helps the modeler then simulating aspects of hydrological processes that are of public interest.

Model transferring methodology

This research describes the experiences encountered by transferring the Walla Walla Hydrological Model to its final users. The methodology followed in this research was not meant to be used to infer conclusions to larger populations. The reader would rather benefit from the identification of benefits and challenges of the description of experienced activities described in the following section.

The Walla Walla hydrological model was developed in cooperation with students of Oregon State University and Walla Walla Basin Watershed Council personnel. The purpose of the model is to serve as a tool for evaluating the hydrological impact of water management scenarios. The model development was funded through various organizations and state institutions. These include Oregon Department of Water Resources, Washington Department of Ecology and local irrigation districts. The funding organizations, irrigation districts and interested residents of Walla Walla were gathered to form a Technical Review Team for the model project. Meetings and presentations of the model were performed to the TRT. Through the watershed council, the model and its data were made available for their review. During these public presentations, the idea of transferring the model was developed, inviting all the interested parties to participate in training hands-on workshops.

Four workshops were offered from the developer to members of the watershed council and interested participants from the TRT. The series of workshops was tailored to match the experience, skills, knowledge, and interests of the group. Topics included a review of model development, beginning with model theory and general assumptions of the Walla Walla conceptual model. The structure and manage of the hydrological data was then covered. Finally, participants applied their knowledge and built skills by developing model scenarios. The group identified specific scenarios which they were interested testing with the model.

Personal interviews and questionnaires of the participants were employed to assess their background knowledge and experience with hydrological models. A secondary objective of the questionnaires was to focus the topic structure of the workshops to accommodate the interest of the participants. Questionnaires can be found in appendix B. Figure A1.1 shows the primary area of interest to Field Hydrologists. Personal interviews were conducted informally throughout the workshops and primarily consisted of basic questions targeting user interest and prior experience with hydrological models. The final users consisted of Field Hydrologists and GIS specialists with little to no experience with hydrological models. From the personal interview and questionnaires the final users demonstrated a bigger interest in model theory and model assumptions rather than just model applications.

For comparison, the developer performed the same series of workshops at OSU in Dr. Cuenca's Regional Hydrological Modeling class offered in winter 2011. The same questionnaires and personal interviews were completed with the students. Figure A1.2 shows the primary area of interest to the group of graduate level students in the modeling class (8 students). The students stated in their informal interviews and questionnaires that they were mostly interested in model applications rather than in model theory or data management. The questionnaires and personal interviews were performed to the available small number of participants in an informal manner. Due to the not random nature of the experiment with a small sample population, inferences cannot be drawn for the entire population grad-students of hydrology. However, the experiences gained during this process are worth sharing. The primary hypothesis from this effort is that final model users required a different learning structure of workshops than grad-students. An area of opportunity for future research will explore the best educational techniques for hydrological model to be learned and applied differentiating the type of final users.

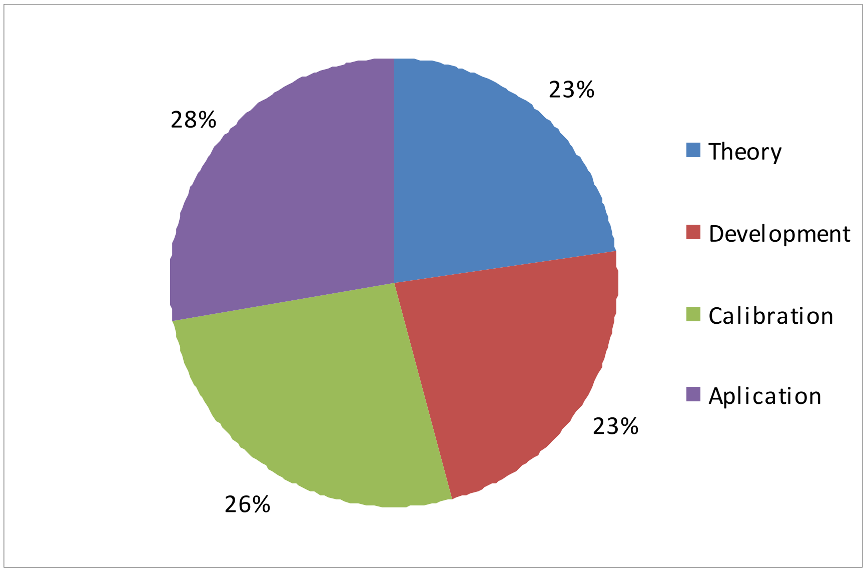

Figure A1.1 Survey of final users: Field Hydrologists. (n=7)

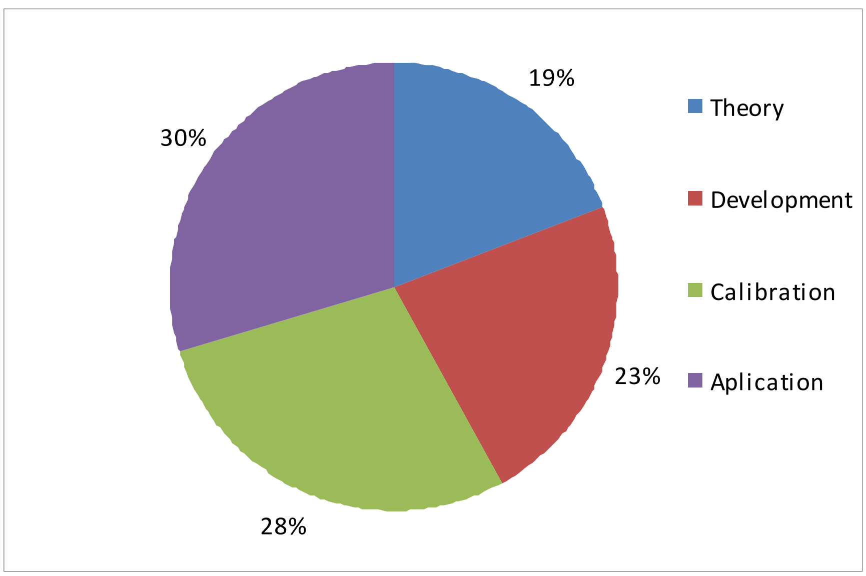

Figure A1.2 Students surveys: Dr. Cuenca's Regional Hydrological Modeling class at OSU 2010 (n=8)

Discussions Walla Walla Model Transfer

The objective of this part of the research was to identify the key components necessary for the effective transferring of a complex hydrological model from developers to it final users. For this, we have drawn from the experiences encountered in the Walla Walla River Basin through a series of suggested activities. The challenges and benefits encountered during these activities hope to be useful for future developers and to expose the need for future research in this social aspect of hydrological modeling. Finally, this research hopes to promote the transferring of complex model to final users.

The activities followed for the Walla Walla modeling transfer can be divided in three major categories: (1) presentations and meetings; (2) guided practice, workshops with hands on training, and (3) consultation and follow up. During the first activities of presentation and meetings, the final users group was gathered and the objectives and agenda of the modeling transfer were established. The group consisted of field hydrologists and GIS specialists with some albeit limited background with hydrological models. Four hands on training workshops were developed during which the entire model development was reviewed and applied to the actual final users of the model. The workshops training presented the core of the activities necessary of the model transfer. For the success of this training, it was necessary to design the workshops based on the interest and background knowledge of the participants. Surveys and interviews compared the interest between student in a formal class setting and the participant's final users of the model. From the results of the interviews and questioners, the workshops were designed specifically for each group. Individual practice and consultation follow up ensure the project completion and initiate the proposed steps for projected model continuations. The technical nature of model use limits the user's potential success when working independently.

The major challenges encountered during this effort were the current model's lack of flexibility, customizability and accessibility by a range of end users. IWFM doesn't have a developed user interface that allows the visualization and analysis of model results. The lack of a user interface requires the use of external GIS software for model manipulation and administration of data. The model not only requires knowledge of hydrology and hydrological modeling theory but also in depth level of knowledge in the use of GIS software is required. Customization and real model applications are labor intensive and time consuming. A certain level of hydrogeological theory is required for incorporating new data. There is the need for collaboration between modelers and field hydrologists collecting the data used for model development. Finally, finding the necessary resources is a challenge. Normally during the stages of model planning and cost estimation, the activities of model transfer are usually calculated into the budget.

There are many benefits identified from the activities related to model transfer. Primary it transfers sufficient information to assure the model will be used appropriately, and that the limitations of the model are well understood. Further the user understands the operational aspects of altering the model, running this model, and interpreting the results obtained. An additional benefit includes a *promotion of communication* between modelers and final users. The modeler develops a better understanding of the needs of the model by identifying with the final users intended uses. The final user also develops a better understanding of the complexities associated developing the model. Transferring helps incorporate the final user expertise in the area by reviewing the model data needs and conceptual representation. Assumptions of local hydrological process thought to be un-important by the modeler are reviewed and information that would not be other ways incorporated because of the need of local knowledge interpretation. *Modeling transferring in the end provides model transparency* and encourages the use of the model to support decision making. Finally, it helps promote project continuation and identification of new areas for research.

The outcome of the surveys an interview with the final users identified the major areas of interest. For the WWBWC a procedure detailing step by step how to (insert here the)The WWBWC was able to run the model and perform basic model modifications, such as incorporating new survey surface elevations.

A.2 - Induced calibration bias by cluster observations and spatially correlated model error

The purpose of this study was to review the related literature and evaluate by a brief experimental analysis if a future more in-depth study would be valuable. This chapter is divided into two sections. The first section is a research proposal identifying the research objectives and a brief literature review. The second section is utilizes the Walla Walla Basin model as a test example to evaluate the research hypothesis. Results from this analysis suggest that a formal research with a controlled synthetic case experiment will be of great scientific value for models that are calibrated by automatic methods

Research Proposal

Research Objectives: Process-based simulation models incorporate spatially correlated errors by the omission or poor representation of a physical local process. For model calibration, the spatial distribution of observations has the potential to induce parameterization bias towards locations with high density of observations. To minimize these unwanted sampling effects, weight factors to observations have been developed. The objective of this research is to evaluate the effects in parameter estimation over a synthetic known hydrological model by (1) un-weighted systematic evenly-space observations, (2) un-weighted random clustered sampling (3) Volume-base weights of systematic and clustered observations (4) Process-based weight factors to systematic and clustered observations.

Proposed methodology:

- Using the synthetic model example used by M, Hill and others "A controlled experiment in ground-water flow model calibration: Ground Water, vol. 36, no. 3, p. 520-535." We could generate several networks of monitored observations. One network could be implemented with a systematic placement of observations in space. A second network of observations could be implemented by randomly select model nodes. My suggestion would be to generate this network of observations with 50 monitored wells (5% of the 1,000 model nodes).
- Using pest or an automatic calibration method estimated over the different networks with and without eights the parameters Streambed conductance, Horizontal hydraulic conductivity, lakebed, areal recharge confining unit leakage and vertical anisotropy
- Conclude the analysis by comparing the value of the estimated parameters.

Suggested Title: Analysis and correction of induced calibration bias introduced by clustered distribution of observations

Preliminary work to evaluate research needs and feasibility for a formal research work

Process-based simulation models such as hydrological models are usually calibrated and evaluated by matching observations to simulated results (model errors). During calibration, parameters are estimated to minimize model error as an objective function (Hill 2007). For inverse modeling calibration methods such as nonlinear regression (Hill 1998) model error is considered randomly distributed thought the model area. However, because the omission and/or poor representation of a local process, model errors are spatially correlated (Xu 2007). Other sources that induce spatially-correlated model errors include the unknown heterogeneous distribution of physically based parameters (i.e. clay lenses or geological faults) (Cooley 2005). Optimal parameter values will then likely to be bias towards area of high density errors.

The spatial clustering of observations will also induce parameterization and model bias (Isaaks 1989, Deutsch 2002, Burns 2012). The spatial and temporal distributions of the network of observations are rather design a priori of the covariance error (Di Zio 2005) or for the purpose of model calibration (Singh 2002). Environmental sampling designs includes between others; convenience sampling (available wells), focus sampling (interest of a local process) and efficient designed networks, placing observations by the used of geo-statistical methods (van Groenigen 2000). Convenience and focus sampling methods lead to an un-even spatial distribution of observations. Areas with a high density of observation (clustered areas) have the potential to dominate the model statistical evaluation (Kennedy 2008) and dues the parameter estimates could be bias to a certain model location. To alleviate this unwanted effects, weight factors to observation have been developed (Hudak 1992, Deutsch 2002, Burns 2012).

Geo-statistical weights factors for de-clustering spatial observations are estimated either as the distance to the closest observation (Hudak 1992) or based on the density of observations in a control volume (Isaaks and Srivastava, Ch. 10 1989). Process-based method for de-clustering observations estimates the weight factor based on the importance of the observation in relation to the simulation (Burns 2012). Burns (2012) compare both methods for de-clustering observations over a synthetic analytical example. The goals of this research is to evaluate the effects in parameter estimation over a synthetic known hydrological model by (1) un-weighted systematic evenly-space observations, (2) un-weighted random clustered sampling (3) Volume-base weights of systematic and clustered observations (4) Process-based weight factors to systematic and clustered observations.

Methodology

The Walla Walla-IWFM Model is used as the example to compare model evaluation and parameter estimation between random allocations of wells and evenly distributed (systematic) wells by a design network of observations. The Walla Walla Basin Watershed Council (WWBWC) maintains a network of 98 observational wells in the area. These wells have been selected given the availability of existing pumping wells and the land-owner willingness to cooperate with the council monitoring program. Figure A2.1 shows the distribution of observations throughout the model domain. The utilization of existing wells yields an un-even distribution of observations where about 20% of the observational wells fall in a radius of 2.5 km^2 corresponding to 9% of the total surface area of the watershed. For the purposes of this research, this network of observations will be called “random allocation” of observations.

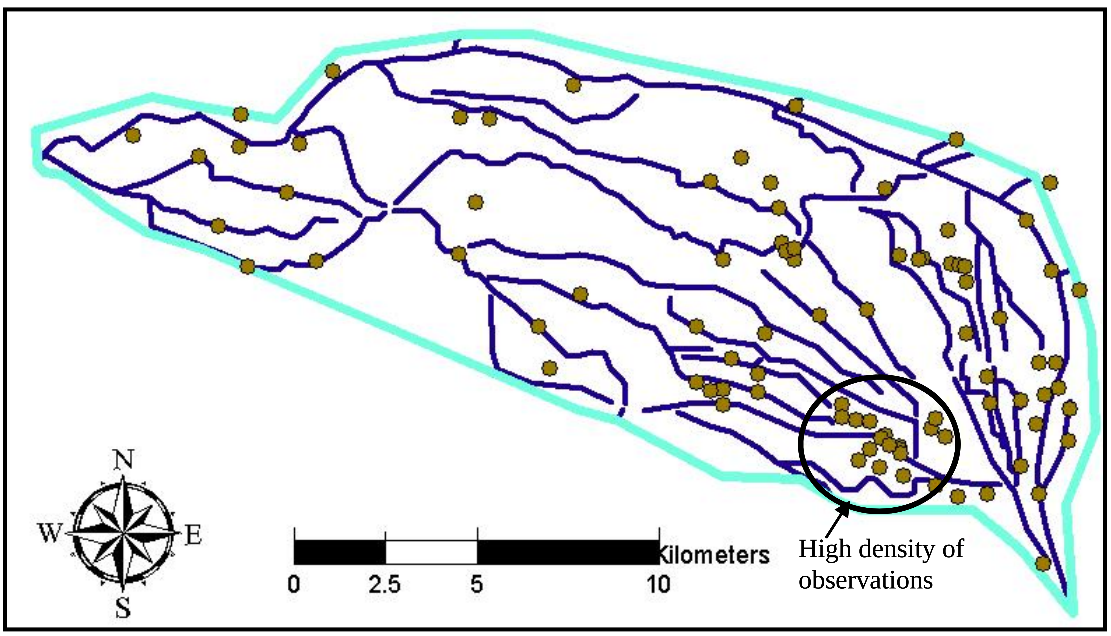

Figure A2.1 Monitored well observations in the Walla Walla Basin n=100, 20% of the observation fall in a radius of 5km^2

Utilizing Geographical Information Systems (GIS), we can evenly distribute the same number of observations throughout the model domain. Figure A2.2 shows the representation of the observational wells with an equal distance apart of 1km ensuring an even coverage of the region. The network distribution of this wells will be refer to as “systematic sampling” for the purposes of this research since the observations are systematically placed with an even distance apart of 1km.

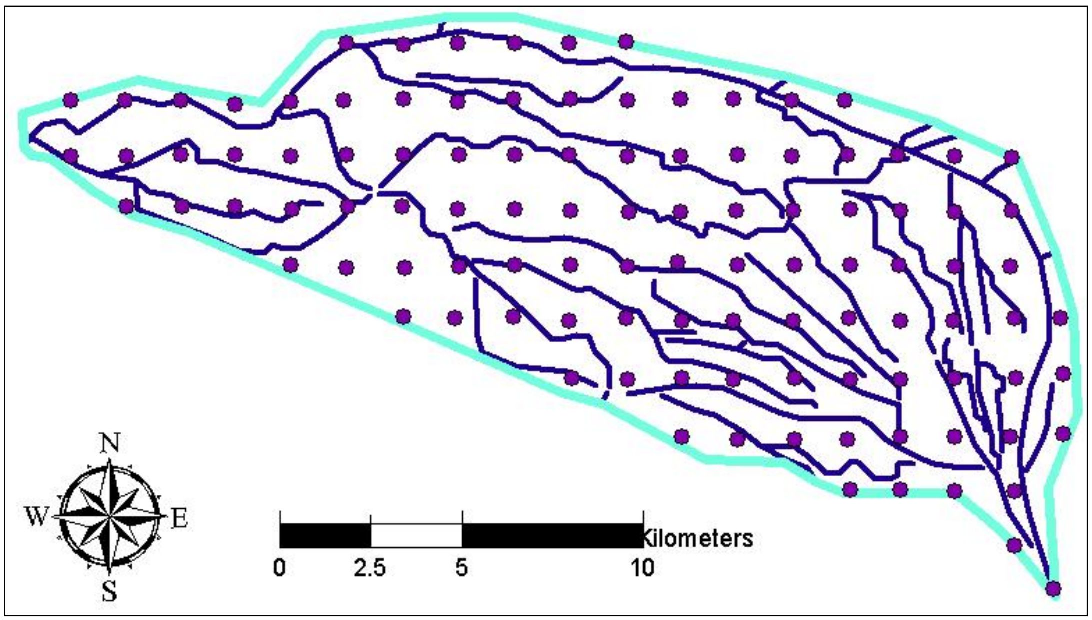

Figure A2.2 Systematic placement of observations n=100, evenly spaced with a separation of 1km.

As mentioned before, the objective is to compare the evaluation of model goodness of fit and parameter estimation between the systematic sampling and random allocations of wells. For this comparison, we used the IWFM-Walla Walla model and the WWBWC well-network distribution of observation wells. For the purpose of this analysis, the model error is estimated by the difference between the simulated value of groundwater elevations and the value at that location obtained from a water table map.

The statistical tool to be used for comparison is the sum of square errors calculated by equation A2.1

$$SD = \sqrt{\frac{1}{n-1}\sum_{1}^{N}\left(h_{obs} - h_{sim}\right)^2} \qquad \text{Eq. A2.1}$$

For the comparison between parameter estimated by random allocation of wells and parameters estimated from a systematic sampling will utilize a manual calibration with the Gauss-Newton equation simplified to solve one model parameter after two model runs (eq. 3.8). The criterion to stop the iterations of model calibration is when the change in the sum of squared error doesn't improve more than 0.5% over the new parameter estimate.

$$\frac{(h_{obs} - h_{p2}) * (P_2 - P_1)}{h_{p2} - h_{p1}} + P_2 = P_{obs} \qquad \text{Eq. A2.2}$$

Where;

h_{obs} = Observed water level elevation at observational well

h_{p1} = simulated head at observation node from the first run of parameter "p"

h_{p2} = simulated head at observation node from the second run of new value of the parameter

P_{obs} = New Parameter estimated from the head observation h_{obs}

P_1 = Parameter value at the first simulation run

P_2 = Parameter value utilize in the second simulation

Equation A2.2 estimates the new parameter value base on model error and change in parameter estimate by the change in model base on the parameter. Equation A2.2 can be expressed as:

New parameter estimate = last parameter estimate + (model error)(change in parameter estimate)/ Change in simulated head

For the comparison between systematic sampling and random allocation of wells we use a single set of systematic sample shown in table A2.1. For the random allocation of wells we used the well-network in the Walla Walla Basin and a set of 50 samples of 100 random locations. Random locations were chosen using the random function in Microsoft excel choosing sets of 100 model nodes.

The weight w of an observation was calculated for the Walla Walla well network based on the density of neighboring wells in a radius of 2.5 km. Utilizing GIS analysis tools we can estimate the density of observations in a 2.5 km area of the existing wells in the Walla Walla Basin. The weights are then divided by the number of neighboring wells within a 2.5 km radius, where the well in question is included in the count. The selection of the influence radius is critical in the weight estimation. For the Walla Walla Basin, the total model area was divided by the number of observations. As an example, figure A2.3 shows that in the case of wells GW_68, GW_67 and GW_120 the suggested weight “w” will be 1/3.

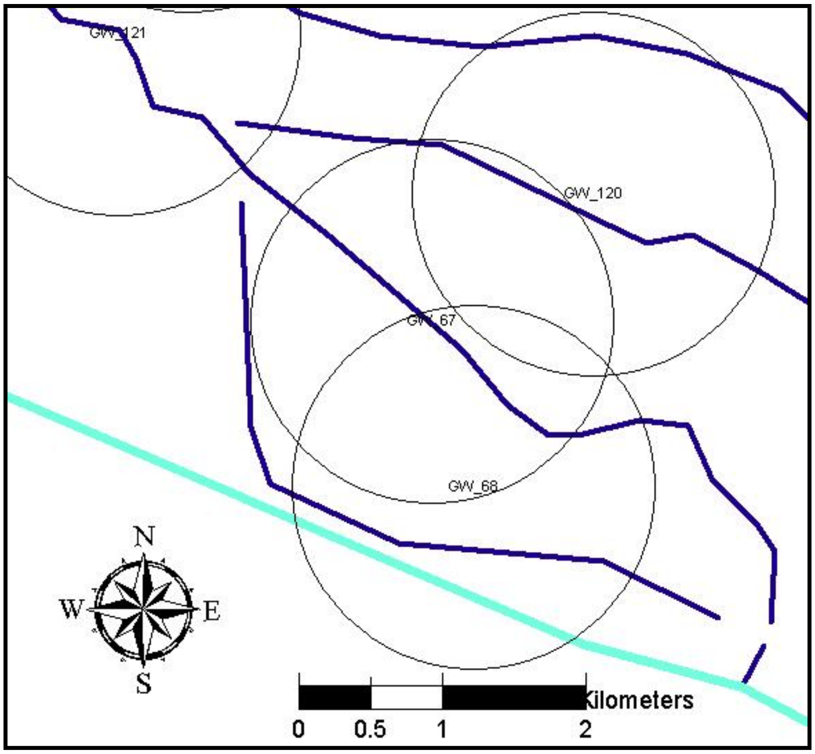

Figure A2.3 Example of three overlapping monitored wells for weight estimation.

Results

The model goodness of fit was evaluated using the estimated standard deviation from simulated values of groundwater elevations and the water table elevation map assumed to be the true value of observation. Table 3.9 shows the total standard error estimated by the Walla Walla well network with and without the suggested weight factor, the systematic well network and 50 random samples of nodes. For the 50 random samples, the table includes the average, minimum and maximum values obtain during the experiment.

Table A2.1 Comparison between Systematic and Random allocation of observations to evaluate model goodness of fit under spatially correlated model error

	Systematic sampling (even coverage)	Random allocation of wells	Existing well –network in the Walla Walla Basin	Existing network in the Walla Walla with weights
# of observations	100	100	89	89
Standard Error STD	2.5	2.6	3.8	2.5
Max STD		3.3		
Min STD		2.2		

The density of observation is also hypothesis to induce bias in the parameter estimation process during model calibration. Table A2.2 shows the results obtained from calibration under the random allocation of wells in the Walla Walla Network and the systematic sampling evenly distributing the number observations throughout the model domain.

Table A2.2 Compared parameter estimation between the Walla Walla monitored well-network and systematic sampling wells

	Initial Parameter values Run	Final Parameter values	Standard deviation SD of model error
Random allocation	k1=100	150	3.56
	k2=33	60	3.53
Systematic samples	k1=100	110	2.09
	k2=33	38	2.08

Conclusions and discussion

This research explores the idea that the distribution of observations in the case of spatially correlated model errors impacts the evaluation of the model goodness of fit and the parameter estimation during the process of model calibration. To test this hypothesis, a systematic approach evenly spacing the network of observations through the model domain was created with the same number of monitored observations in the Walla Walla Basin. Weights factors to observations for the Walla Walla Basin were calculated using GIS spatial analyst tools by dividing the density of wells that fall a radius of 2.5 km. This research compares the model goodness of fit and parameter estimation obtained from the created systematic well network and the random allocation of wells, including the Walla Walla network with and without the weights to observations. The analysis assumes that the model error is the difference between simulated model values and the values obtained at that location from an established water table map.

The results from this research suggest that for model evaluation, random allocation of wells have the potential to give either very low or high values of standard error. If the random allocation of wells by chance has a higher density of wells that fall in an area where the model fit is especially poor, the calculated standard deviation statistic will lead to a bigger unrepresentative mismatch. This is the case of the Walla Walla network of wells when compared to the systematic sampling approach. After 50 random sets of 100 nodes, the average standard deviation yields the same value as that obtained by a systematic sampling distribution. The well network in the Walla Walla using the suggested factor weight to observations yields the same standard error obtained from the systematic sampling distribution.

The parameter estimation was also compared between systematic and random sampling. The parameters minimizing the model misfit under these two scenarios are different. Results show that an area with higher density of observation can dominate the statistic or objective function to be minimized during the model parameterization. In the case of the Walla Walla model, a higher hydraulic conductivity around the infiltration basins yields an overall better model fit. However, that value of hydraulic conductivity is not representative of the entire model domain.

This short lateral research was not originally considered a part or a fundamental component of the overall dissertation research plan of the PhD program. These are preliminary results that yield the basis for a future formal research where more rigorous comparisons can be made by employing automatic calibration methods. At the moment, these results suggest that model evaluation goodness of fit to observations can be bias to areas of high density of observations in the case of spatially correlated model errors. This un-even distribution can be alleviated by applying weight factors to observations based on the density of neighboring wells. In the case of the Walla Walla Model where 20% of the monitored wells lie around the artificial aquifer recharge project. The overall evaluation statistic of model error standard deviation was initially weighted by the ability of the model to simulate the artificial aquifer recharge project. To avoid bias in the parameter estimation during model calibration, the weight factors of systematic sampling describe in this section were employed.