AN ABSTRACT OF THE THESIS OF

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The gravel aquifer of the Oregon side of Walla Walla River Basin has a strong hydrologic connection to surface water through a series of springs, unlined irrigation canals, the Walla Walla River, numerous wells and, since 2004, artificial recharge to the shallow aquifer using infiltration basins.

The finite element Integrated Water Flow Model (IWFM) developed by California Department of Water Resources was used to quantify all of the major hydrologic features of the basin. Using the information provided by the Walla Walla Basin Watershed Council, irrigation districts, and previous studies conducted at Oregon State University and by consultants, the model was setup and calibrated, and a water budget simulation was performed for the years of 2003 to 2006.

It is shown that close to 96 percent of the land use water demand goes to agriculture growing 16 major crops. 60 percent of the water comes from surface water diversions flowing through unlined irrigation canals, which themselves lose 28 percent of their inflow to the unconfined gravel aquifer. The calibrated and validated model was used to simulate the flow of the Johnson Creek Springs, which were shown to have increased flow due to the artificial recharge project.

Modeling Surface Water and Groundwater Interactions near Milton-Freewater, Oregon

by
Arístides Crisóstomos Petrides Jiménez

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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.

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To Dr. Richard Cuenca and Dr. John Selker who reviewed and followed the development of this model very closely, helping me understand all of the hydrological processes involved. And to my wife, Mary, and my family, who are my inspiration.

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1. Introduction

1.1 Watershed Background

The Walla Walla Watershed, which is shared by the states of Oregon and Washington, has gained public attention for its successful river restoration efforts in recent years. Listed in 1998 as America's 18th Most Endangered River, the Walla Walla River has seen successful changes occur in both the main stem that serves as a habitat for many endangered species, and in the restoration of water levels in the shallow aquifer gravel system on the Oregon side of the watershed. With the collaboration of concerned groups, numerous studies have been funded in the area, and with the negotiation of irrigation districts, the watershed has been able to apply water resource management programs.[1]

In the year 2000, two irrigation districts in Oregon, the Hudson Bay District Improvement Company and Walla Walla River Irrigation District, and one in Washington, Gardena Farms Irrigation District, agreed to leave 25 cfs (61×10^3 m^3/d) in the Walla Walla River on the Oregon side and 18 cfs (44×10^3 m^3/d) on the Washington side (WWBWC, 2002). This program together with the discontinuation of gravel mining and the approval of the U.S. Army Corps of Engineers for construction of a levee by Milton-Freewater, OR and the Flood Control District, allowed for restoration of riparian vegetation and reduction of the temperature of the river. Previously dry in the summer, today the Walla Walla River not only maintains flows throughout the year, but also creates aquatic habitats by restoration of the "natural" meandering of the river.

In 2004, on the Oregon side of the Walla Walla River Basin, the Walla Walla Basin Watershed Council (WWBWC), in partnership with the Hudson Bay District Improvement Company, created a pilot project to recharge the shallow aquifer by diverting water in the winter (November through May) from the

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[1] For more information on this and other related programs visit wwww.wwbwc.com

irrigation canals into three man-made infiltration basins. As a result of this restoration practice, aquifers levels have increased, making this pilot program a significant contributor to the optimization of regional water resources. This recharge project not only maintains a stable flow in the old springs that drain into the Walla Walla River, but also helps the local economy by leaving more water in the aquifer that can be pumped for irrigation purposes, in the case of an increased demand, or used in the case of a drought season.

A hydrologic simulation model for the Milton-Freewater region has been developed and calibrated using information gathered by the WWBWC since the year 2002 and other scientific research projects. This extensive work included the installation of monitoring wells, monitoring of irrigation canals and surfaces flows, installation and maintenance of climatic stations, determination of hydrogeology stratigraphy (Lindsay, 2003), land use coverage, and estimation of aquifer parameters including hydraulic conductivity and porosity.

1.2 Project Goals

In order to visualize the extended effects of water resource management projects and new potential scenarios, it is necessary to understand the interactions and functions of the processes involved in this complex hydrologic system. Modern technological advances have made possible the development of a hydrologic model capable of simulating the interactions and processes between surface water and groundwater. Results from the hydrologic model simulation will help in the decision making process by producing a water budget analysis and visualization of several management scenarios. The model chosen for this project was the Integrated Water Flow Model (IWFM), developed by the California Department of Water Resources, given its strong conceptual and physically based method of simulating groundwater flow and its interactions with surface water, and utilizing a finite element method that makes it computationally efficient. More information about the model is provided in section 2.2 IWFM History and Applications

2. Literature review and conceptual model setup

An introduction to previous studies and the description of the IWFM hydrologic model are presented in this section. The purpose is to give the reader a clear understanding of the objectives and capabilities of the model used in this thesis and to show the extent of previous studies and their limitations. First will be a review of previous hydrologic models developed in the area, followed by the history of IWFM, and finally, the studies providing new information to develop a new conceptual model.

2.1 Literature review

Two previous models have been developed for the Walla Walla Basin, the model developed by Barker and MacNish (1977) and the model developed by Golder Associates (2007) redefining the model created by Barker and MacNish. In this section we will discuss some of the limitations of these models and how given new information provided from studies by WWBWC and OSU, a new conceptual model is necessary to improve our understanding of the hydrologic conditions in the area.

The Walla Walla basin area as the eastern part of the state of Oregon has an important demand of water dedicated to agricultural production. As an example of the amount of water required for irrigation, the Oregon Department of Water Resources, based on water rights data, has estimated the annual irrigation water demand for the Walla Wallla Basin to be 5.7 to 1.1 X 10^5 acre-feet (70.7x10^6 m^3 to 14.1 x10^7 m^3) assuming water use of 1.5 and 3.0 acre-ft/acre/year (1.8 x10^3 to 3.7x$10^3$$m^3$) respectively and 262 to 1,656 acre-feet (32.3 x10^4 m^3 to 2.0 x10^6 m^3) for domestic use (Wozniak , 2007). Quoting Wozniak (2007) “the water rights track potential use, these figures are likely to be greater than actual use in any given year”. So, though Wozniak (2007) provides helpful estimates, the pumpage figures are highly uncertain, and do not distinguish between separate aquifers or indicate which existing wells are used.

A hydrologic model can be set up to estimate groundwater pumping and water diversions by land use water requirements or by user specification of water needs (see pumping by element 2.3.13) Barker and MacNish (1973) estimate 25,000 acre-feet/yr (30.8 X 10^6 m^3/yr) of pumpage from the gravel aquifer. This number was calibrated in the model and first determined by examining electrical records. The digital model of Barker and MacNish (1973) served as the starting point for numerous projects that are summarized by the Pacific Groundwater Group (1995) which includes Cline and Kandle (1990), Collins (1987), and James et al. (1991). In June, 2007 Golder Associates employed these results in a modeling effort to evaluate Aquifer Storage and Recovery (ASR) strategies proposed by the city of Walla Walla. Golder Associates (2007) redefined the Barker and MacNish (1973) model boundaries and distribution of aquifer parameters from new information collected. Using the redefined model they tested the effects of different ASR operations on the basalt aquifer system. Aquifer parameters for the gravel aquifers were calibrated for a single layer aquifer based on previous studies, and the hydraulic horizontal conductivity was estimated based on specific capacity from well logs. It is important here to mention that the gravel aquifer is compose on several layers (Lindsey, 2004) conducting water at different rates. A single aquifer layer model will not capture the hydrologic conditions in the area, where the presence or absence of the aquifer layers determine how fast water moves underground.

Based on the understanding of how water moves, an hydrological model can be used to estimate water budgets for their model area. These water budgets can be an extremely useful tool for the management of water resources but, they are not scalable in space and time. Land use and water management practices change dramatically over time and space. Newcomb (1965) not only described the geological conditions and water resources in the Walla Walla Basin, but also quantified the major components of the surface and groundwater budgets by direct observation of flow in springs, rivers and analysis of three years of pumpage.

Water management changes influenced by land use, cropping, demographics etc. and new studies measuring hydrological parameters resulted in a constant refinement of the water budget estimates. For example, the study by Metcalf (2003) measured the conductivities of stream beds along the Walla Walla River and irrigation canals. The Irrigation system analysis prepared by HDR engineering (2004) measured irrigation canal losses. Robert Bower's (2005) estimation of infiltration rates, and other studies like Lindsey (2003) describing the geological conditions in the area, give us new and better explained hydrological conditions, requiring a new approach to build a different conceptual model.

The area for which the previous water budgets were realized includes the gravel and basalt aquifer in Oregon and Washington. It is impossible to simply convert these estimates for a much smaller area because of land use and the geological characteristics. The proposed model intends to provide a water budget analysis for a much smaller area where much of the new studies have been developed and the system of monitoring gauges run by the WWBWC has been set up.

2.2 Integrated Water Flow Model IWFM : History and Applications

The model chosen to simulate the water resources of Milton-Freewater is the Integrated Water Flow Model (IWFM) developed by the California Department of Water Resources, given its capability for modeling groundwater and its interactions with other hydrologic processes. It is a comprehensive physically-based water resource allocation model that is mathematically complex and computationally efficient by application of the finite element method.

IWFM was first developed by Young Yoon from UCLA in 1976 as a new groundwater model. Major revisions and enhancements followed from 1979 to 1983 with basin-wide applications by Young Yoon / Boyle Engineering. In 1987 stream routing was added and the application to Central Valley California was done by Young Yoon / Boyle Engineering. Surface water and land surface processes were added with its release in 1990. From that time many improvements have been made, such as the tile drain simulation (Ali Teghavi, 1996) and land subsidence (Ali Teghavi, 1997). In 2000, consolidation of all previous versions was done by Ali Teghavi and Saquib Najimus, but it was not until December 2002, with the name Integrated Groundwater and Flow Model 2, IGSM2, that the model was first made available in the public domain. The California Department of Water Resources changed the name in September of 2005 to Integrated Water Flow Model (IWFM). IWFM, written in FORTRAN 95, is in continuous development by Emin Can Drogul and Tariq Kadir with the California Department of Water Resources and its modeling support branch of the Bay-Delta Office.[2]

IWFM has the capability to simulate quasi three-dimensional groundwater flow for a combination of confined and unconfined aquifers. As described in Appendix A: Digital Simulation Theory, IWFM uses the Garlekin finite element method with the Newton-Raphson method for linearization of the system of simultaneous equations. The capability of simulating surface and groundwater interactions, and the land use water demands based on user specified crops, makes this an ideal model for the Walla Walla River basin. IWFM has been used (as we can see in Table 2.1) for areas as large as the California Central Valley (C2VSIM) simulating an area of 51,800 km^2 ,. Distributed physically-based hydrologic simulation models like MIKE-SHE of the Danish Hydraulic Institute, or HEC–HMS of the U.S. Army Corps of Engineers, would require infeasible amounts of computational time to model regions of the extent covered by previous applications of IWFM.

[2] For more information, visit http://baydeltaoffice.water.ca.gov/modeling/hydrology/IWFM/index.cfm

Table 2.1 Some characteristics of previous applications of IWFM:

Project Name	Development team	Simulated Area km^2	Number of Aquifer layers	Number of Groundwater nodes per layer	# of Stream nodes	Simulated period
California Central Valley Simulation Model (C2VSIM)	California Department of Water Resources	51,800	3	1,393	432	1921 – 2003 (monthly)
West side of the California San Joaquin Valley (WESTSIM)	U. S. Bureau of Reclamation	6,291	7	2,602	351	1970 – 2000 (monthly)
California Merced River Basin (MercedSim)	Lawrence Berkeley National Lab	5,491	7	2,224	423	
California Butte Basin Groundwater Model	CDM	3,289	9	3770	753	
California Solano County Model	West Yost & Assoc	906	1	257	5	

2.3 Conceptual model formation from previous studies, related literature and studies performed by the WWBWC and OSU

The purpose of the Milton-Freewater hydrologic model is to have a tool to aid in the water resources management decision making process. To accomplish this goal, a water budget is presented per storage unit or flow process, per sub-region and as a total water budget. In this section we will review the processes and output from IWFM and the rationale for each component of the conceptual model for the basin.

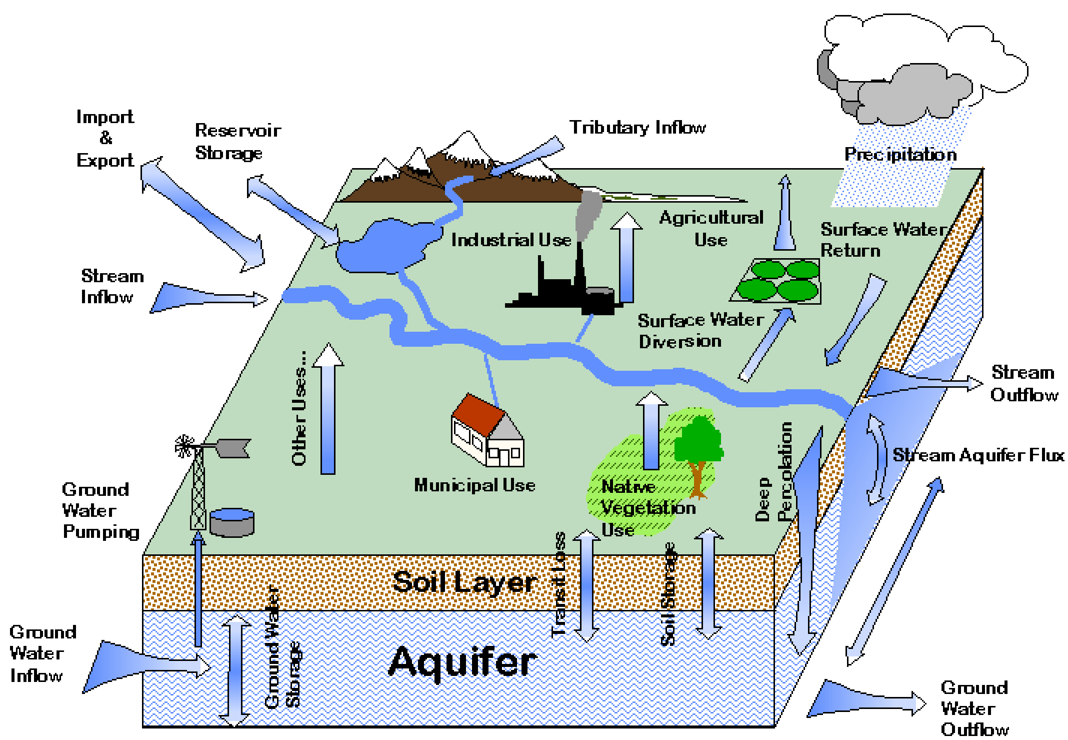


Fig. 2.1 Conceptual Water Budget from Colorado Division of Water Resources, Office of the State Engineer.

The water balance equation for a section of a watershed can be written as:

$$P + G_{in} + Q_{in} - (Q_{out} + ET + G_{out}) + Q_{external} = \Delta S \qquad \text{eq.2.1}$$

$$Q_{out} = P - ET \pm \Delta S$$

Where:

P = Precipitation (m/day)

ET = Evapotranspiration (m/day)

Q_{in} = Inflow from streams (m^3/day)

Q_{out} = Outflow from streams (m^3/day)

$Q_{external}$ = Other Inflow of surface water (m^3/day)

G_{in} = Subsurface inflow to model area (m^3/day)

G_{out}=Subsurface flow leaving the model area (m^3/day)

ΔS = Change in storage (m^3/day)

The IWFM model provides a water budget analysis per each term or storage unit in Equation 4.1 separately. The following table summarizes the files generated by the IWFM budget analysis and zone budget analysis which presents the calculated fluxes per subregion as well as for the total model.

The following section describes the information known in the model area from previous studies and how IWFM considers these components in the simulation. The information will be organized based on table 2.2, beginning with the variables required for the soils budget, (precipitation) continuing with the variables required for the streams budget and finalizing with the variables required for the groundwater budget (pumping). The hydrologic budget files have overlapping variables that depending on the budget will have a negative or positive sign indicating that they are putting water into the budget or taking water out. For example, deep percolation in the soils budget will have a net negative sign since water is leaving the soils and a positive sign in the groundwater budget since it is recharging the groundwater aquifers. These overlapping variables will only be explained once to facilitate understanding for the reader.

Table 2.2 Components of the IWFM water budget per storage unit.

Water Budget file	Variables estimated for subregion on the Budget file
Soils water budget	Precipitation, runoff , applied water, return flow, infiltration, Actual ET, Deep percolation, change in storage
Streams (including: irrigation canals and springs) two files, per river section and as a total system	Upstream inflow, Downstream outflow, Runoff Return Flow, Gain from GW, Gain from recharge project, Diversions.
Ground water	Deep Percolation, beginning and ending storage, gains from stream, general recharge and recharge from recharge project, boundary inflow, subsidence, pumping and net subsurface inflow.
Recharge project (Lake file)	Beginning and ending storage flow from by-pass, Precipitation, evaporation, groundwater recharge and surface elevation.
Land water use	Area classification, Potential CUAW, supply requirement per area, pumping, diversion, shortage, re-use , region imports and exports
Sub group details	Dividing land, stream and GW this file identifies agricultural, urban supply requirements return flow deep percolation, Runoff, gains from gw pumping and subsidence per subgroup.
Reach or stream section.	Same variables as in the stream file but here are specified per stream reach
Zone budget	GW Storage in & out, Streams inflow & outflow, Subsidence in & out, Net Deep Percolation Specified Head BC, Diversion Recoverable Loss recharge project Pumping by Element subsurface flow and Overall Zone Error

2.3.1 Precipitation:

As mentioned by Dingman (2002) "All water enters the land phase of the hydrologic cycle as precipitation. Thus in order to assess, predict and forecast hydrologic responses, hydrologists need to understand the amount, rate, duration and quality of precipitation distributed in space and time." In the model area, the watershed council has run three ETo stations (see section 2.3.5 actual ET) with the capability of measuring precipitation. We conclude that a simple interpolation with only three points will not be as complete as using the publicly available information from Oregon Climate Services based on a much larger interpolation area with many reliable gauges. A section of this interpolated mapping can be selected for our specific location.

For precipitation, we used the monthly averages from the Oregon Climate Service precipitation 2006 PRISM Group, Oregon State University. Using the query of Lon -118.42 Lat 45.962 from the Database webpage at http://mistral.oce.orst.edu/www/mapserv we found the following values shown in Table 2.3.

Table 2.3 Monthly average precipitation (mm/month) obtain from PRISM used in the model.

Year 2003	Monthly average mm/month										
Jan	Feb	Mar	April	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
65.5	45.3	41.8	52.4	28.6	0.9	0.3	4.6	21.8	7.8	25.9	80.9
Year 2004											
Jan	Feb	Mar	April	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
74.8	40.5	9.0	33.1	72.3	46.6	4.4	30.1	13.7	19.4	47.8	30.7
Year 2005											
Jan	Feb	Mar	April	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
11.7	0.3	35.4	27.3	67.4	35.1	4.4	0.3	6.9	28.6	39.0	45.2
Year 2005											
Jan	Feb	Mar	April	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
82.2	18.0	49.1	70.3	26.3	43.2	3.7	1.0	14.2	18.4	64.0	64.9

IWFM partitions the precipitation in the following ways: Precipitation that falls directly on the streams and lakes contributes to stream flows and lake storage immediately; precipitation that falls on the ground surface infiltrates into the soil at a rate dictated by the type of ground cover, physical characteristics of the soil and soil moisture content (see infiltration). A detailed description of the processes simulated by IWFM is presented below.

2.3.2 Direct Runoff and Return flow (overland flow)

The portion of water that neither evaporates nor infiltrates generates surface flows that contribute either to stream flow or it can infiltrate into the soil further down the slope. Emett (1978) and Abraham et al. (1986) describe a method for measuring overland flows distinguishing overland flows that are saturated from above and saturation overland flows that are saturated from below. For example, when the groundwater table intercepts the ground surface and contributes to surface flows in a series of springs. IWFM distinguishes between return flow and direct runoff in the following way: Flow generated by agricultural irrigation and urban water is modeled as return flow, the rest of processes generating overland flow are called direct runoff. Direct runoff and return flows, after being computed by IWFM using the SCS curve number method, are immediately carried to a pre-specified stream location.

In order to assign the stream node affected by runoff from each element, a basic 2D model of the area was generated with the Integrated Ground Water model (IGW) to observe the possible water divides (Fig. 2.2). Seven water divides were identified by observing the vectors generated by the groundwater flow. Then, each stream node was assigned the corresponding flow vectors by elements. IGW is a free-ware finite difference groundwater model obtained from the Department of Civil and Environmental Engineering at Michigan State University. The set up for this model was based on the interpolation of groundwater elevations.

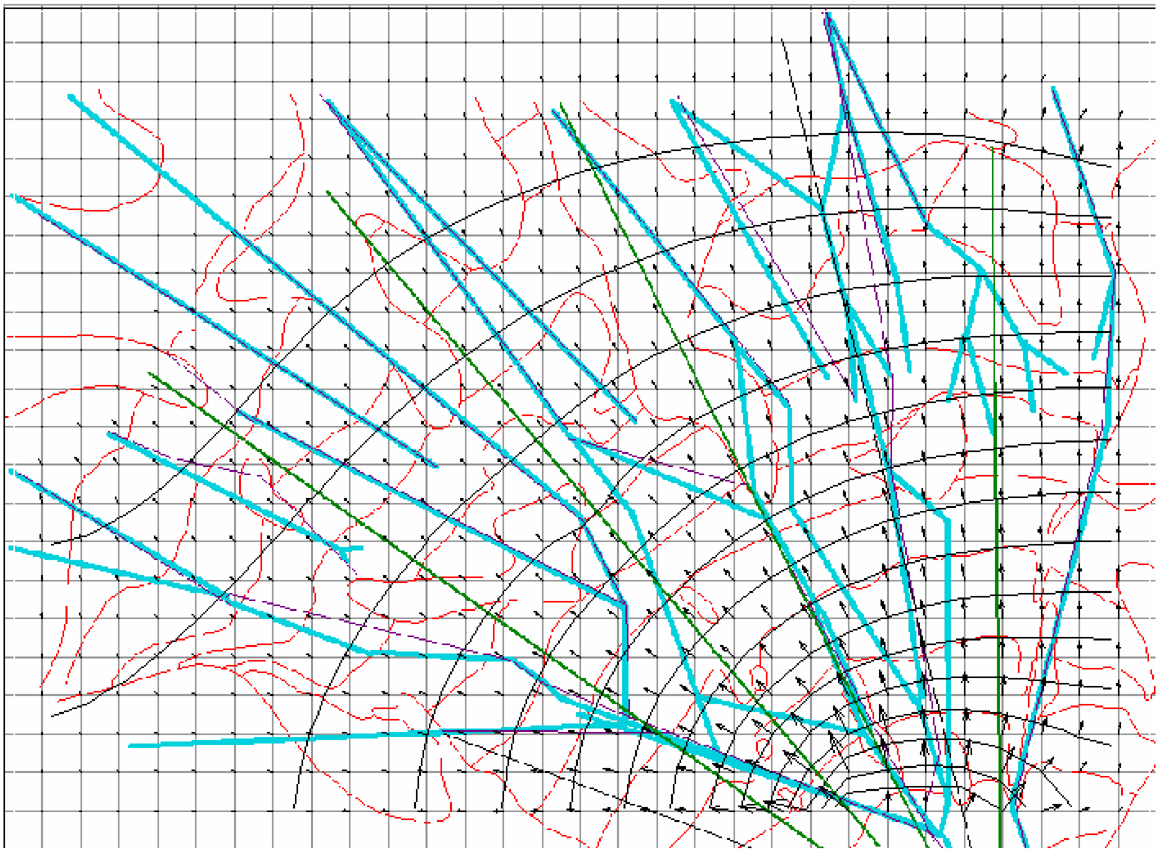


Fig 2.2 Water divides generated from IGW model, arrows represent vectors of Groundwater flow, red lines equipotential heads, blue lines rivers and springs, green lines water divides.

The SCS curve number method was developed by the National Resources Conservation Service (Dingman 2002) and used in IWFM to calculate the surface runoff. The SCS method relates the effective rainfall and water storage capacity via an empirical relation:

$$S_{max} = 1000/CN \quad -10$$

Where S_{max} = Watershed storage capacity (inches)

CN = Curve number based on soils group

Equation 2.2

From the United States Department of Agriculture soil survey of Umatilla County, Oregon, the following soils were identified in the model area:

Ellisforde silt loam 1 to 7 percent slopes, Freewater very cobbly loam 0 to 3 percent slopes, Hermiston silt loam 0 to 3 percent slopes, Oliphant silt loam, 0 to 3 percent slopes, and Umapine silt loam 0 to 3 percent slopes. All of these soils falls into the A soil group, having a low overland flow and high infiltration rate. Assuming for agriculture a straight row crop with an average antecedent condition II, the curve numbers used from (Table 9-12 of Dingman 2002) are 72 for A, 81 for B, 88 for C and 81 for D. IWFM adjusts for antecedent wetness by comparing the soil moisture to the difference between field capacity and wilting point.

In the theoretical manual of IWFM ver. 3 (Drogul 2002), usage of the curve numbers, CN, listed in the original documentation of the SCS method produces values of the retention parameter, Smax, in units of inches (USDA, 1985). In order to use the CN equation to compute Smax in units other than inches, one needs to modify CN values. Rearranging equation 2.2 to convert CN from inches to meters we used the following conversion equation.

$$S^{*}_{max} = \frac{S_{max}}{39.37} = \frac{\frac{1000}{CN} - 10}{39.37} \qquad \text{Equation 2.3}$$

And $$\frac{1000}{CN*} - 10 = \frac{\frac{1000}{CN} - 10}{39.37}$$

Equation 2.4

Rearranging

$$CN^{*} = \frac{39370CN}{383.7CN + 1000}$$

Where

S^{*} = Storage capacity in meters

CN^{*} = Curve number in meters

2.3.3 Applied water and Return flow calculation

Applied water is the quantity of water used by agricultural and urban needs. Runoff previously described is computed by IWFM for each land use and soil type. To calculate the runoff from urban settings, IWFM multiplies the applied water times the fraction of applied water that becomes return flow, $\%R_f$ which is specified by the user (see Calibration parameters 3.4).

The amount of return flow from agricultural lands is computed based on the water balance using the soil moisture content at the root zone, infiltration of precipitation, agricultural applied water, crop evapotranspiration and the fraction of water that becomes deep percolation (Figure 2.3). Return flow is taken to be zero if the mass balance is negative.

$$Rf_{ag} = Dr\,\theta r / \Delta t + I + AW_{ag} + Etc_{adj} + D_p + D_r\,\theta f / \Delta t \qquad \text{Eq.2.5}$$

Rf_{ag} = return flow of agricultural applied water, (L/T);

D_r = rooting depth of the crops, (L);

θr = soil moisture content of the root zone, (dimensionless);

θf = field capacity of the root zone, (dimensionless);

Δt = time period over which the return flow is computed, (T);

I = infiltration of precipitation, (L/T);

AW_{ag} = agricultural applied water, (L/T);

ETc_{adj} = crop evapotranspiration, (L/T);

Dp = deep percolation, (L/T).

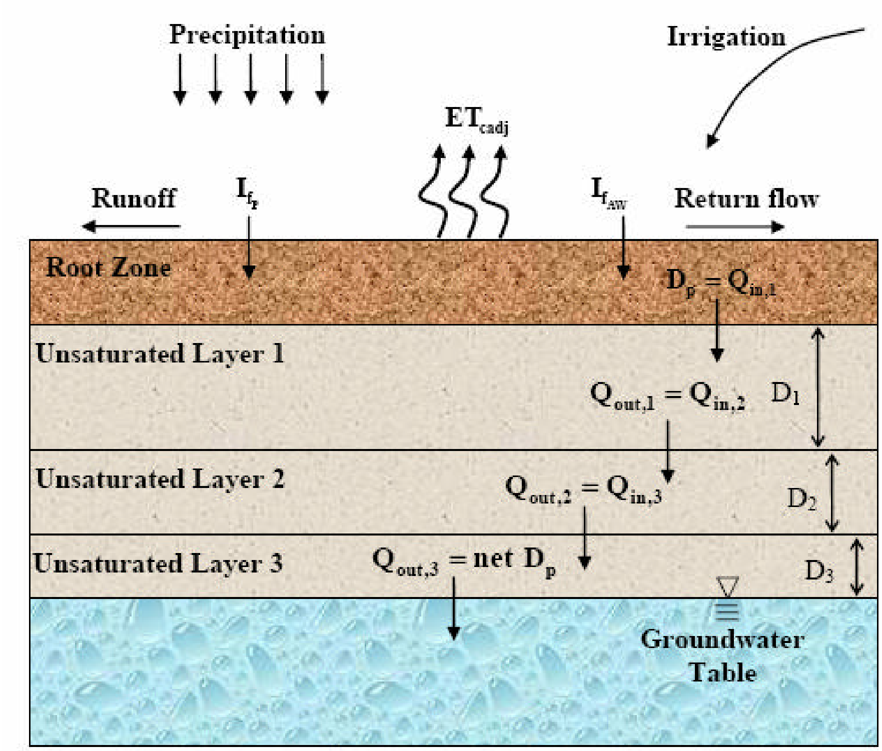


Fig 2.3 Soils budget flows, from IWFM Theoretical Manual ver. 3.0

2.3.4 Infiltration

The amount of precipitation that will enter into the soil layers and will be available for plant uptake and deep percolation is calculated as the difference between precipitation and runoff.

2.3.5 Actual ET

The combination of the processes of evaporation (liquid water to vapor) and transpiration (plant respiration) is called evapotranspiration (ET). ET is a function of weather parameters (solar radiation, air temperature, humidity, and wind speed), crop factors (resistance to transpiration, crop height, crop roughness and crop rooting characteristics), and environmental conditions (soil salinity, land fertility, presence of hard or impenetrable soil horizons, cultivation practices, irrigation method, soil water content etc) (Allen 1998).

According to Allen et al (1986), Actual ET is calculated as:

$$ET_{act} = K_s K_c ET_0 \qquad \text{eq. 2.6}$$

Where

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

ET_0 is defined as the evapotranspiration rate from a reference surface that has an adequate amount of water. The reference surface is a hypothetical grass or alfalfa reference crop with specific characteristics (Allen 1986). K_s is the factor that incorporates the effect of soil moisture shortage on the crop evaporation rate. K_c is a single crop coefficient that takes into account the effect of both crop transpiration and soil evaporation. ET_{act} is the crop evapotranspiration under non-standard conditions modified by the water stress coefficient K_s.

The Walla Walla Basin Watershed council has run three ET_0 stations (Fig 2.4) capable of estimating the reference ET_0 based on climatic parameters put into the FAO-56 Penman Monteith equation (Allen et al. 1985). Laura Jensen from the Department of Bioresource Engineering at OSU worked with the information provided from the ET_0 stations and interpolated between them to provide a grass evapotranspiration coefficient per sub-region in the model area. The ET_0 stations are Campbell Scientific, Inc. ET_0 106 Weather Station installed in:

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

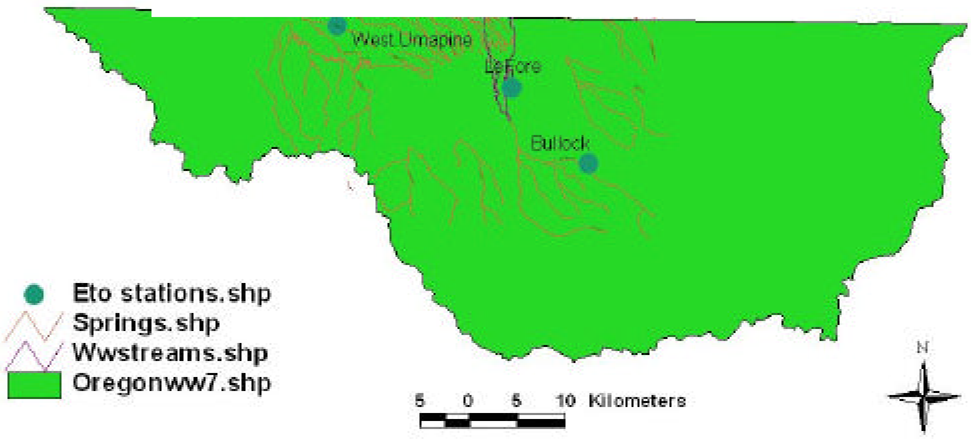


Fig. 2.4 ET_0 station locations.

The calculation for ET_0 multiplied by the crop coefficient, K_c, are done separately from IWFM and incorporated into the model in a time series. IWFM calculates the water stress coefficient, K_s, which incorporates the effect of soil moisture shortage by the following rule: When the volumetric soil moisture content in the root zone is below 50 percent of the field capacity, K_s is the fraction between the volumetric soil moisture content and field capacity; if volumetric soil moisture content is above 50 percent of field capacity, then K_s is considered to be one. If there is no vegetation coverage we see that when volumetric soil moisture content in the root zone is below 50 percent of field capacity, K_s in this case is the fraction between volumetric soil moisture content and field capacity.

$$K_s = \begin{cases} \theta_r / 0.5\theta_f & \text{if } \theta_{wp} < \theta_r < .5\theta_f \\ 1 & \text{if } \theta_r > 0.5\,\theta_f \end{cases}$$

Eq. 2.7

When there is no coverage the following expression is used:

$$K_s = \begin{cases} \theta_r / \theta_f & \text{if } \theta_{wp} < \theta_r < .5\theta_f \\ 1 & \text{if } \theta_r > 0.5\,\theta_f \end{cases}$$

Eq. 2.8

To calculate actual evapotranspiration, the WWBWC has created extensive crop coverage and land use database based on geographic information systems GIS that identifies not only the urban, riparian, industrial and agricultural land use, but also identifies 16 different crops grown in the model area. This database has been created from aerial photos and with the aid of local growers to identify each crop and the time and method of irrigation which will help to estimate the K_c per season. The K_c was based on the single crop coefficient procedure described by Allen (1998) where the season is divided into four periods: initial where the K_c is small, crop development where a rapid growth of K_c is occurring, mid-season K_c is at its maximum point and constant, and late season when the leaves begin to age and K_c decreases until it reaches the end of season value. (Table 2.4 and Table 2.5)

Table 2.4 Crop coefficients considered in the simulation.

#	Crop Name	Kc ini	Kc mid	Kc end
1	Apples	0.6	0.95	0.75
2	Cherries	0.6	0.95	0.75
3	Plums	0.55	0.9	0.65
4	Apricot	0.55	0.9	0.45
5	Grapes (wine)	0.3	0.7	0.9
6	Alfalfa hay	0.4	0.95	0.7
7	Grass hay irrigated (sudan)	0.5	0.9	0.85
8	Peas	0.5	1.15	0.7
9	Wheat	0.4	1.15	0.3
10	Wheat dry	ETo		
11	Pasture Grazing	0.4	0.85	0.85
12	Bare soil	Eto		
13	Urban (highway, buidings,structures,etc)	CN		
14	Water surfaces	CN		

Table 2.5 Lengths of crop development stages (days)

#	Crop Name	Initial	Development	Mid season	Late season
1	apples	30	50	130	30
2	Cherries	30	50	130	30
3	Plums	30	50	130	30
4	Apricot	30	50	130	30
5	Grapes (wine)	20	50	75	60
6	Alfalfa hay	10	30	25	10
7	Grass hay irrigated (sudan)	10	30	25	10
8	Peas	35	25	30	20
9	wheat	20	30	60	30
10	wheat dry	20	60	70	30
11	Pasture Grazing	10	20		

2.3.6 Deep percolation

IWFM defines deep percolation as the moisture that leaves the root zone and enters the unsaturated zone. The moisture travels downward through the unsaturated zone and as soon as the unsaturated zone reaches field capacity, it recharges the groundwater. The groundwater recharge is called "Net deep percolation". IWFM has two methods that the user can choose to calculate deep percolation. The first method calculates deep percolation as a net fraction of soil moisture that is above field capacity. The second method calculates the unsaturated hydraulic conductivity of the root zone as a non-linear function of the saturated hydraulic conductivity and the pore size distribution. The first method is recommended by users of IWFM given that the physically-based approach cannot be used due to the size of the simulation time step being inconsistent with the characteristic time scale of the deep percolation flow process, or when values of soil parameters used in the physically-based approach are not available. The first method was used in the simulation model.

When soil moisture is above file capacity $\theta_r > \theta_f$ then

Deep percolation = $f_{DP} * D_r (\theta_r - \theta_f)$

f_{DP} = Fraction of deep percolation (0 means entire soil moisture above field capacity becomes return flow, 1 entire soil moisture above field capacity becomes deep percolation.)

D_r = Thickness of the root zone (meters)

Eq. 2.9

2.3.7 Soil change in storage

The movement of water through the unsaturated layers is delayed by the thickness of the soil layer and the soil moisture in each layer. The amount of water that flows from each layer is calculated by:

$Q_{out} = K_s (\theta_m/\eta_{tm})^4$

Eq. 2.10

K_s = saturated hydraulic conductivity (m^3/day)

θ_m = soil moisture at soil layer (percentage)

η_{tm} = total porosity at soil layer (percentage)

2.3.8 Upstream and Downstream flows

Irrigation canals and springs are modeled as rivers, losing water or gaining depending on the present hydrologic conditions. The WWBWC has gauged the main system of the Walla Walla River and the main section or irrigation canals and springs on the model domain as shown in Fig 2.6. Using this information the following segments of rivers are modeled:

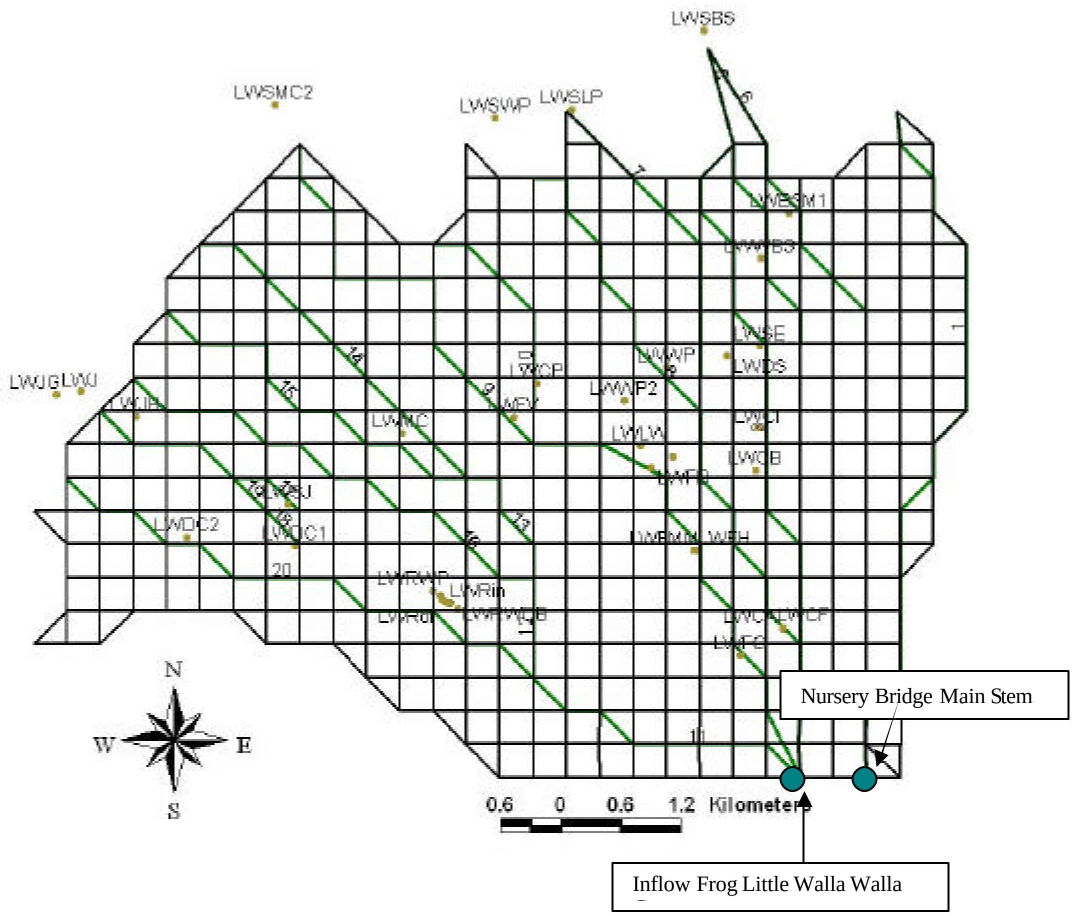


Fig 2.5 Surface gauges; dots represent point of measurement; green lines are rivers, irrigation canals, and springs include in the model, number represents model river segments.

Table 2.6 River segments and average flows simulated in the model.

Model river segment	River, irrigation canal or spring name	Mean Seasonal flow
1	Walla Walla River Main Stem	Peak occurs in February, Winter flow average 408cfs Summer average 29 cfs
2	Crocket system; east and west crocket until crocket branch to little Walla Walla west prong	Summer average 19 cfs
3	Crockett until East prong of Little Walla Walla	Summer average 3.4 cfs
4	Big spring center branch (ballou south of ballou road)	Summer Flow based on 2006 3.18 cfs
5	Big spring northen + east prong Little Walla Walla	Summer Flow based on 2006 6.26 cfs
6	Big spring east branch	Summer Flow based on 2006 6.26 cfs
7	Lewis spring	End of summer 1.26 cfs
8	West spring Little Walla Walla	End of summer 0.7 cfs
9	Ford system and Ford branch that converts into middle branch mud creek	Summer average 21 cfs at the beginning of the system
10	Mud creek Spring	End of summer 1.15 cfs
11	White Ditch from frog until division to Highline	Summer average 67 cfs at the beginning of the system
12	White Ditch division to highline and Richards	Summer average 17 cfs at the beginning of the system
13	Highline (pipe section)	Summer average 11 cfs at the beginning of the system
14	Highline Ditch	Summer 2.35 cfs at the end
15	Little mud creek spring	End of summer 2.19 cfs
16	Richards Ditch	Summer 1.78 cfs at the end
17	North Fork Johnson spring	ND
18	South Johnson spring	Based on Flow 2006 0.53cfs
19	Johnson spring	Peak Flow of 2006 0.82cfs
20	Last segment of White Ditch	

The inflow of surface water into our model area occurs at two locations: the first point is the main steam of the Walla Walla River at Eastside Diversion and Nursery Bridge gauge and the second point is by the Little Walla Walla system at the Frog Diversion (Fig. 2.6) where the flow of water gets divided into three irrigation canals: White Ditch (HBIDC), Ford and Crokett. Robert Bower (personal communication) estimated the inflow of water from gains and losses of his gauge system using the following methodology:

1. Used OWRD gauges: Little WW diversion, South Fork and North Gauges to create a synthetic Nursery Bridge Flow. This did not account for gains and losses between those gauges and Nursery Bridge.
2. Used WWBWC Gauges: Nursery Bridge gauge data was compiled for the model period. The gauge only has irrigation and low flow season data due to inability for high flow in stream measurements.
3. Used WDOE Gauge: Pepper Bridge gauge data was compiled for the model period. Does not account for channel bed gains/losses between it and Nursery Bridge gauge.
4. OWRD gauges were compared against WDOE gauge.
5. WDOE Gauge was compared against WWBWC data. Regression relationship was built based on that comparison. Data was averaged for a 7 day period to remove noise of peak flow timing and outliers.
6. Regression equation used to create synthetic, year round data for Nursery Bridge.
7. Flow gains/losses were calculated for OWRD upriver gauges to Nursery Bridge and Nursery Bridge to WDOE Pepper Bridge. (positive values gains: tributary and groundwater inflow, negative values losses: diversions, channel bed infiltration).

2.3.9 Streams Gains from Groundwater

Two previous projects, Bower (2005) and HDR Engineering (2004), have studied the gains and losses of the connected system (see Figure 2.7) between the gravel aquifer and the Walla Walla River and the irrigation canals. The Milton-Freewater hydrologic model has the capability to simulate these interactions and adjust to the water requirements of the land use if it is specified by the user to meet the diversion and bypass flows computed in the following way. First let us look at the continuity equation of the stream flows:

$$0 = Qin - Qout \qquad \text{Eq. 2.11}$$

$$Qin = Q_i + R_f + S_r + Q_{ws} + Q_{brs} + Q_{lko} + Q_h \qquad \text{Eq. 2.12}$$

$$Qout = Q_{bdiv} + Q_{sint} + Q_{si} \qquad \text{Eq. 2.13}$$

$$Q_{bdiv} = Q_b + Q_{div} \qquad \text{Eq. 2.14}$$

Where:

Q_i = flow from upper stream node

R_f = return flow from agricultural irrigation and urban water use

S_r = direct runoff due to rainfall excess and subsurface flows (water table above ground)

Q_{ws} = inflow from tributaries

Q_{brs} = inflow from bypasses

Q_{lko} = inflow due to overflow from the recharge project

Q_h = inflows other than those listed above

Q_{sint} = rate of water exchange between the stream and the groundwater

Q_{si} = net flow that contributes to flow at the downstream node

Q_{div} = flow that is diverted for agricultural and urban water use

Q_b = outflow that is diverted as bypass flow

Q_{bdiv} = flow that is diverted or bypass

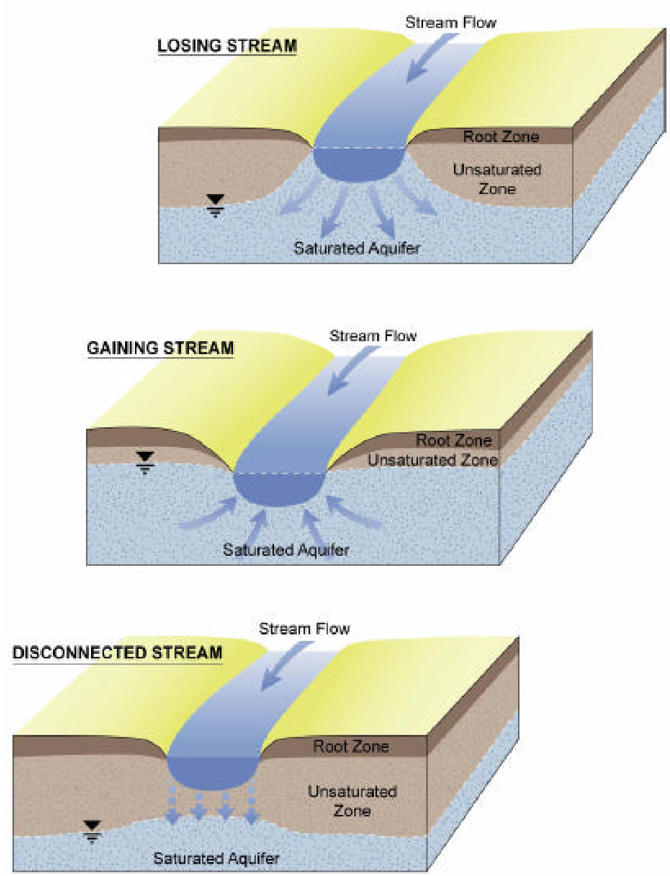


Figure 2.6 Stream connections to ground water (IWFM 2002).

The term Q_{sint} in the continuity flow equation reflects the flow of the interaction between stream-groundwater and is calculated as,

$$Q_{sint} = C_{si} \left[h_s - h_g \right] \qquad \text{Eq. 2.15}$$

where:

C_{si} = conductance of the streambed material at stream node (m^2/day)

h_s = stream head (m)

h_g = groundwater head (m)

The conductance C_{si} reflects the properties of the stream bed materials as well as its geometry.

$$C_{si} = (K_{si}/d_{si}) * L_i W_i \qquad \text{Eq. 2.16}$$

K_{si}= hydraulic conductivity of the stream bed material (m/day)

d_{si}= thickness of the stream bed (m)

L_i = length of the stream segment (m)

W_i = wetted perimeter (m)

Two M.S. projects were developed to find the saturated hydraulic conductivity, K_{si}. Metcalf (2003) estimated the hydraulic conductivity of the main stem of the Walla Walla as well as the leakage rate from the irrigation canals to be 2.5 m/day similar to that reported by Leek (2006). Metcalf (2003) based her study on an infiltration test and Leek (2006) based their analysis on a series of slug tests. Even though we expect this number to vary for different irrigation canals and river sections, we do not expect to see a large variation. The thickness of the stream bed and the wetted perimeter are still to be determined for all the rivers, springs and irrigation canals. It is important to have an accurate estimate of the wetted perimeter since the amount of water predicted to be transmitted from the stream is linearly related to this factor. Further results from these calculations are central to some of the main questions for which this model was created. For example, the scenario where the irrigation canals has been lined and the conductance term C_{si} = 0 means that the canal does not longer recharge the groundwater.

2.3.10 Groundwater gains from Hudson Bay recharge project

The recharge project run by the WWBWC, in conjunction with Hudson Bay District Improvement Company (HBDIC), consists of diverting water from an irrigation canal into three man-made infiltration basins that lie under the young gravel aquifer (see Aquifer System 2.4). Water that overflows the infiltration basins returns to the irrigation canal.

One of the objectives of the Hudson Bay aquifer recharge project is to test the possibility to store water in the unconfined aquifer and stabilize springs flows. A model goal is to predict the net gain to GW from the recharge project and its effects on the shallow aquifer. All the data gathered by the WWBWC and presented in its public annual reports are available on line at http://www.wwbwc.org/Projects/ .

Table 2.6 summarizes the available recharge data from the project which operates under the limited licenses request (#758) which allows for a diversion of 64 cfs ($1.5X10^5$ m^3/day) in November, 95 cfs ($2.3X10^5 m^3$/day) in December and January, and from February to May 15, 150 cfs ($3.6X10^5$ m^3/day) if there is adequate flow in the Walla Walla river to honor the existing water rights. In Table 2.7 we can see the results from the infiltration test measured at each infiltration basin (Bower 2007).

Table 2.7 HBDIC Recharge project operation summary:

Event	Time	Total flow diverted from White Ditch cfs	Total volume of water diverted (m^3) (time * flow)
1	5-week period April 8th until May 15th, 04	9,907	$8.77 X 10^8$
2	December 2004 to February 05	4,602	$3.09 X 10^8$
3	March 2005	91	$2.52 X 10^5$
4	March to May 2005	7,865	$6.68 X 10^8$
5	December to January 2006	14,370	$1.84 X 10^9$
6	March to May 2006	19,684	$3.68 X 10^9$
	total	56,522	$7.37 X 10^9$

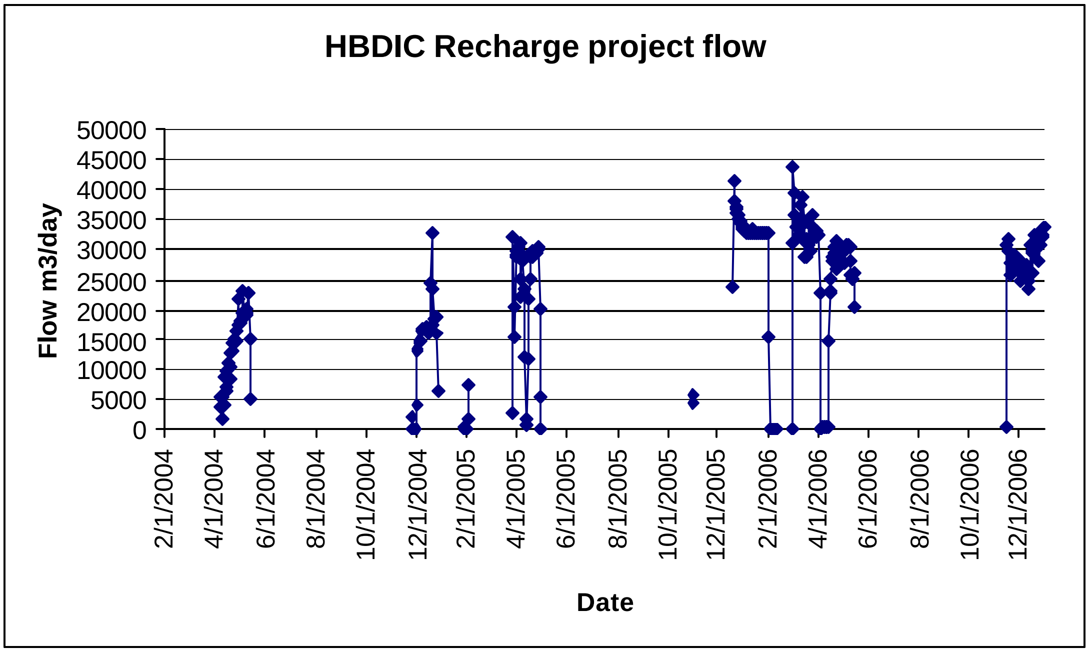


Figure 2.7 Total flow recharge, data from (Bower, 2006)

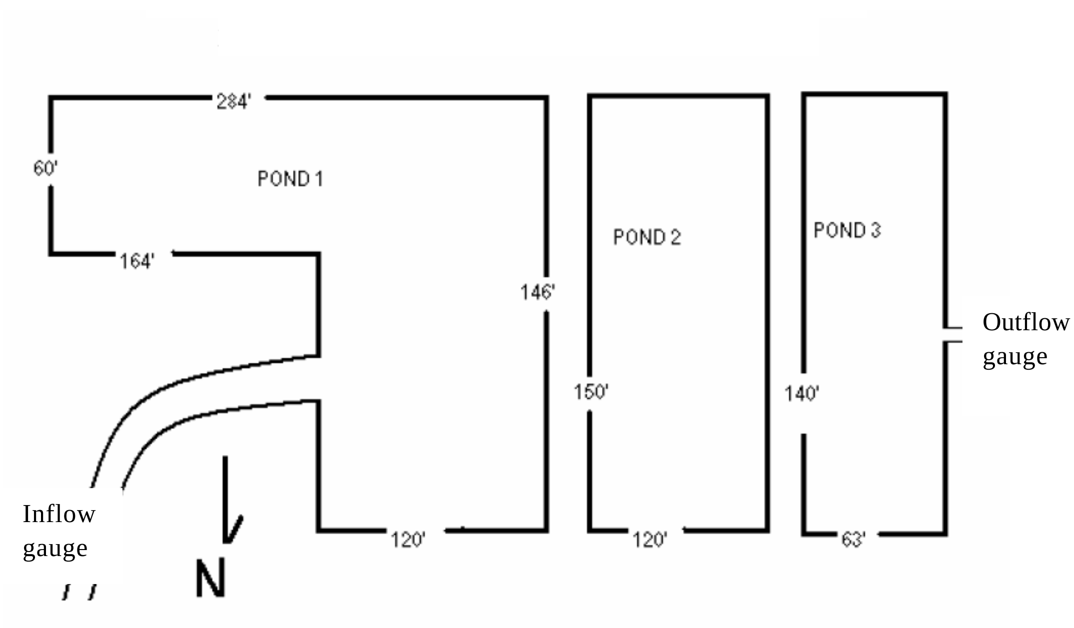


Figure 2.8 Infiltration Basins Bower (2005)

Table 2.8 Infiltration rate tested at Hudson Bay Aquifer Recharge data from personal communication from Robert Bower.

	Surface area (m^2)	Volume (m3)	Time required to drain (hrs)
Infiltration Pond 1	2121	3379	6.4
Infiltration Pond 2	1319	1612	6.1
Infiltration Pond 3	797	1080	5.8
Intake + overflow ditch	263	182	x
Total	4500	6253	6.1
Infiltration rate (m/day)		7.0	
Recharge rate (m^3/day)		24,604	

The Milton-Freewater Hydrologic model using IWFM will model the infiltration basins as lakes given their similar behavior. The conservation equation for lake storage can be expressed as:

$$S_{lk} = (P_{lk} - EV_{lk} - Q_{lkint})*A_{lk} - Q_{brlk} - Q_{inlk} + Q_{lko} \quad \text{Eq.2.17}$$

S_{lk} = Lake Storage (m^3)

P_{lk} = Precipitation over lake (m/day)

EV_{lk} = Evaporation lake surface (m/day)

Q_{lkint} = Groundwater interaction (m^3/day)

A_{lk} = Lake Area (m^2)

Q_{brlk} = inflow from diversion and bypass flows (m^3/day)

Q_{inlk} = inflow from upstream lakes (m^3/day)

Q_{lko} = outflow in case of overflow (m^3/day)

The lake ground water interaction as with rivers is calculated as

$$Q_{lkint} = C_{lki}(h_{lk} - h_{gw}) \quad \text{Eq.2.18}$$

h_{lk} = Lake surface elevation (m)

h_{gw} = groundwater head (m)

C_{lki} = conductance of the lake (m^2/day) bed material at lake node as well as with the river conductance the lake conductance is calculated as :

$C_{lki} = (K_{lki}/d_{lki})\ A_{lki}$

K_{lki} = hydraulic conductivity of the lake bed material (m/day)l

d_{lki} = thickness of the lake bed material (m)

A_{lki} = Area of the lake (m^2)

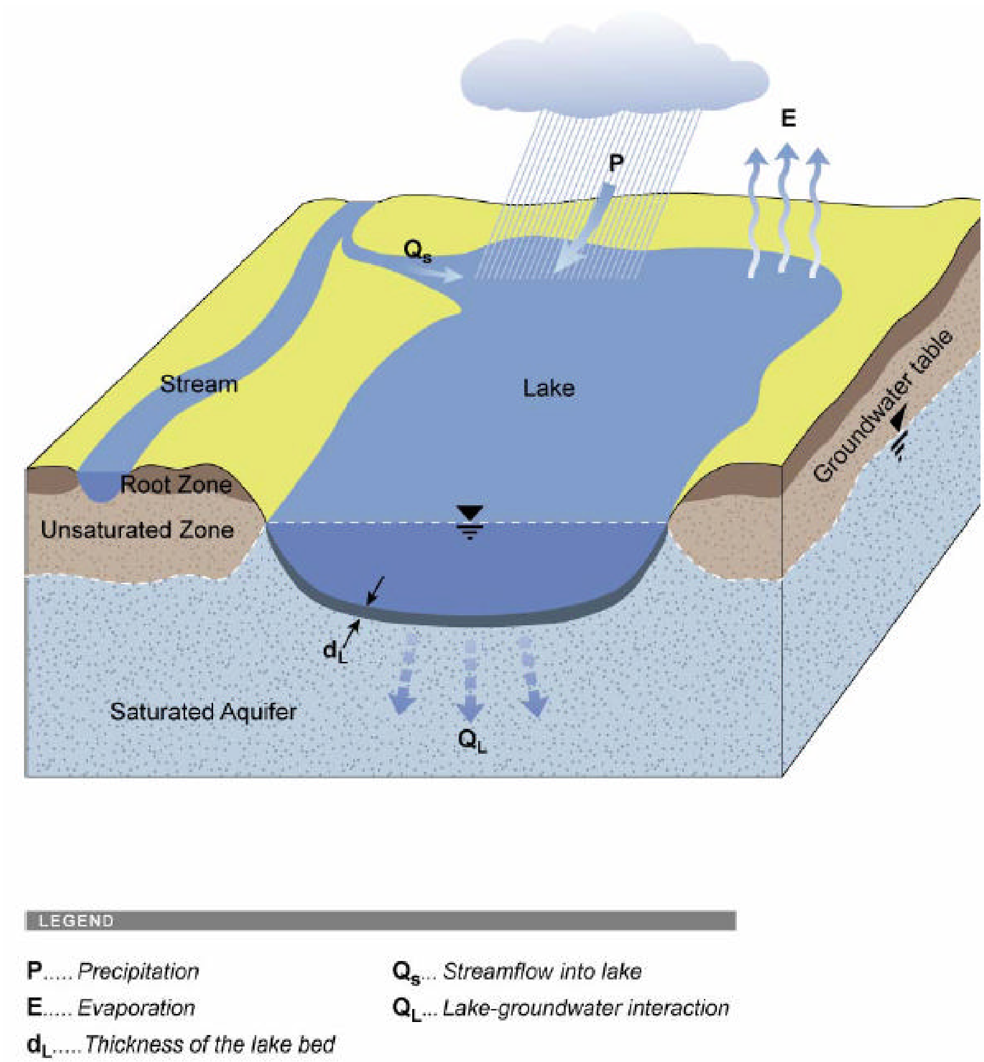


Fig 2.9 Diagram of Lake Budget from IWFM theoretical manual (2002)

2.3.11 Diversions

In IWFM diversions are defined as the irrigation canals that divert water from the point of diversion to the agricultural needs or point of use. Bypasses on the other hand, are defined as locations where flow is diverted or taken from any stream location to another stream location by any method that does not lose water. Diversions and Bypasses can be user defined or calculated by the model. For the Milton-Freewater hydrologic model we specified the rate at which we expect water to be applied by looking at the WWBWC gauges minus the infiltration rates or losses of the canals. The model will first check if there is enough flow at the stream node. If not, the amount diverted would be the maximum flow in the canal unless otherwise specified. For calibration purposes we have specified that if there is enough flow then we can compute the water diversion by the total demand of the specified location, for example, agricultural needs for irrigation of a certain crop base on our GIS Data Base coverage.

2.3.12 Subsidence

Even though we are not using land subsidence for the Milton-Freewater hydrologic model, IWFM has the capacity to simulate Land subsidence as the term used to calculate the amount of storage loss due to soil structure transformation. The elastic parameter would allow for a permanent or temporary change in storage due to a water stress. For example if there is too much pumping from a shallow aquifer, all the space that water occupied would then be empty and able to collapse. Land subsidence is calculated as the change in head due to subsidence in relation to the vertical compaction of interbeds. So, the total compaction is the sum of elastic and inelastic change in interbed thickness

$$\Delta b_{si} = -\Delta h S_{si} b_o \quad \text{and} \quad \Delta b_{se} = -\Delta h S_{se} b_o$$

Eq. 2.19

Where

Δb_{si} = inelastic change in interbed thickness, positive for compaction and negative for expansion

Δb_{se} = elastic change in interbed thickness, positive for compaction and negative for expansion

Δh = change in head

S_{si} = inelastic specific storage

S_{se} = elastic specific storage

b_o = initial thickness of the interbed

2.3.13 Pumping

IWFM model has the capability to estimate pumping using two different approaches, pumping by element and pumping per well. Pumping per well means that the user explicitly defines the location, depth and flow rate of each well in the model area. When pumping by element, the user can either specify the amount of water that is being pumped per element or the model can calculate the amount being pumped per element based on the water demand. In the Milton-Freewater simulation model both approaches were used. First, we let the model estimate the amount of water required by the crop demand of the element unmet by surface diversions, using the crop distribution coverage. We then compared this number with the water right permits of Oregon Department of Water Resources GIS database by the maximum rate permissible per groundwater permit in the element. The modeled pumping rate was set to whichever of these was smaller, The California Department of Water Resources has used the IWFM approach in the same way the in the calculation of its pumped water throughout a much larger area with the California Central Valley Simulation Model (C2VSIM) (Bush 2007). Once a model has been calibrated, IWFM has been proved to be successful in calculating the amount of water being pumped (Bush 2007).

2.3.14 Basalt pumping wells:

Based on the geology description, an impervious silty clay layer of 200 ft (60.96m) separates the gravel aquifer system from the basalt aquifers. A hydrologic connection arises from the irrigation and domestic basalt wells that take water from deep basalt aquifers to irrigate the crops and pastures grown on the gravel aquifer. These wells were identify by Lindsey (2007).

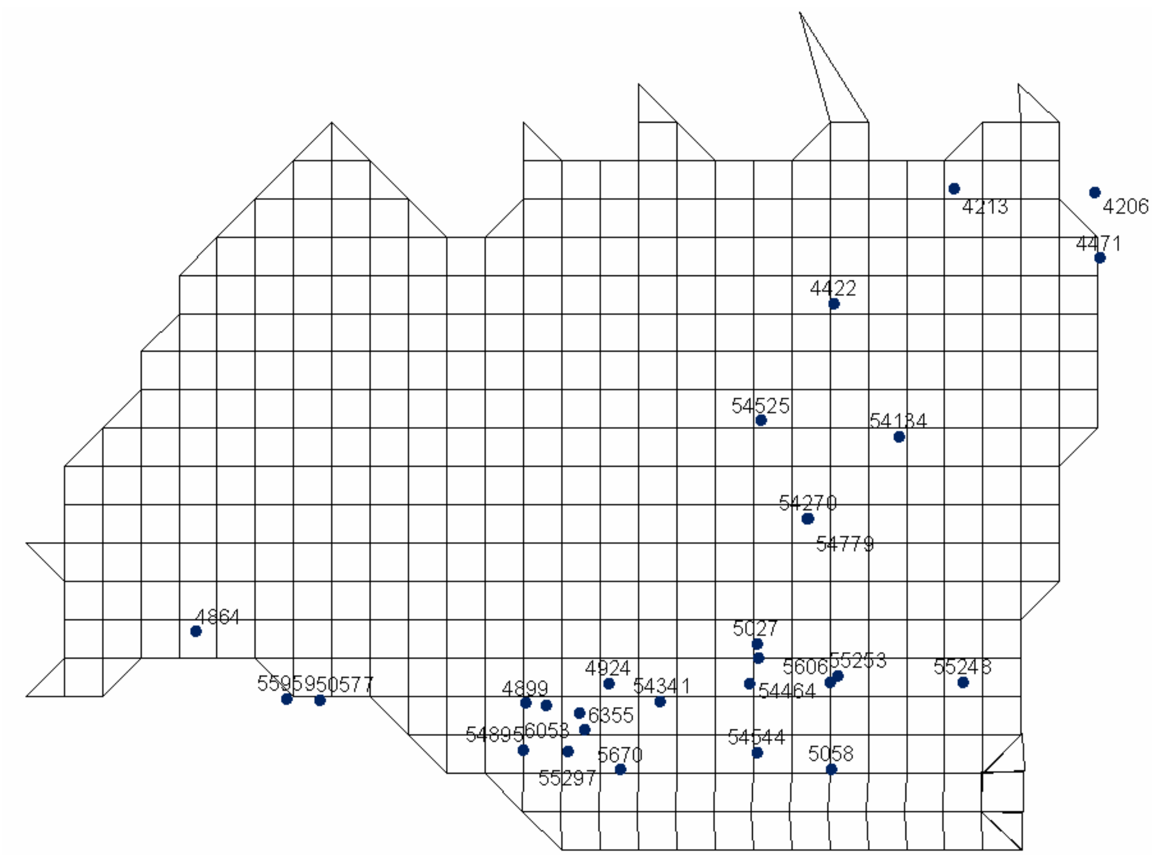


Figure 2.10 Basalt Wells over the model Area with well ID numbers Table 2.8

Table 2.9 Basalt well locations and pumping rate.

Well ID	mo/yr drilled	UTM northing	UTM easting	Use I=Irrigation D=Domestic	Max rate (gpm) from well logs normal operation from July to September
U4206	Jun-54	5094204	393562	I	700
U4213	Jun-33	5094215	392375	I	300
U53775	May-00	5083257	398616	D	60
U4471	Sep-82	5093606	393619	I	220
U54270	Mar-81	5091358	391101	I	196
U54525	2-May	5092226	390705	I	20
U54779	3-Mar	5091358	391090	I	2500
U4864	Dec-54	5090401	385847	I	500
U50577	Mar-97	5089813	386919	I	75
U55959	7-Jun	5089831	386618	I	500
U4899	Jun-33	5089798	388672	I	200
U54895	3-Aug	5089381	388640	I	135
U4924	Mar-88	5089955	389402	D	45
U50478	Feb-97	5089769	388859	I	400
U54341	1-Aug	5089787	389837	I	125
U55297	4-Dec	5089370	389046	I	125
U5670	May-92	5089213	389498	W	60
U6053	Jul-94	5089702	389149	I	200
U6355	Aug-95	5089552	389188	I	200
U5027	Dec-87	5090284	390670	D,I	150
U54464	2-Feb	5089953	390599	I	3100
U55253	4-Sep	5090002	391364	I	500
U54544	2-Jun	5089370	390650	I	450
U55999	8-Aug	5090172	390672	I	600
U55248	4-Aug	5089978	392440	I	500
U5606	Nov-91	5090018	391375	I	353

2.4 Aquifer system

The system of aquifers in the Walla Walla watershed can be divided into two major systems: the unconfined system and the confined system. The unconfined system is created by three geological layers described below. They are simulated as single variably saturated aquifers in order to account for the heterogeneity found in the area as the saturated hydraulic conductivity K diminishes with distance from the surface.

1. Touchet Beds: Pleistocene Cataclysmic flood deposited and Palouse formation, Pleistocene loess, felsic silt and felsic to basaltic fine to medium sand (Kennedy/Jenk 2003). Figure 2-12. The first non-calibrated saturated hydraulic conductivity value used in the simulation model was 3.8 m/day (from EPA, 1986).

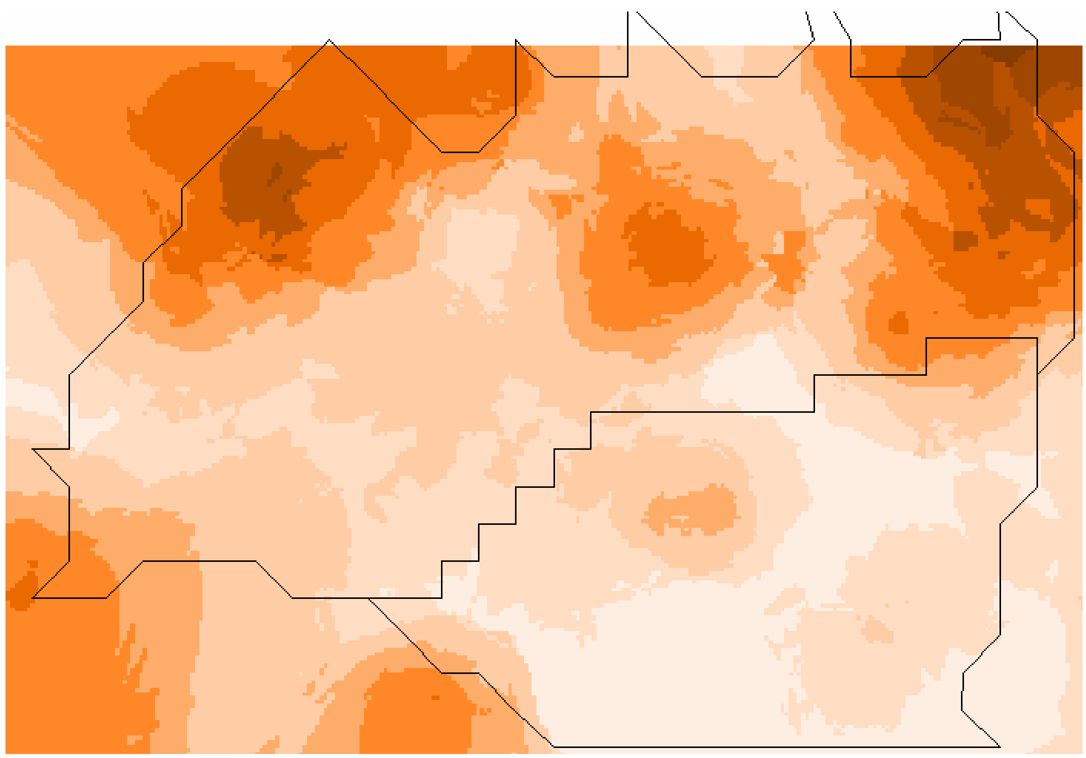


Figure 2-12 Thickness of Touchet beds created from attachment 6, well data table of Kennedy/Jenk consultants 2003 and with a GIS interpolation using the spline method in Arcview 3.3

2. Young alluvial gravel; Holocene to Pliocene sand and gravel not well constrained same as with the first layer, these uncemented gravels vary in thickness across the model area.

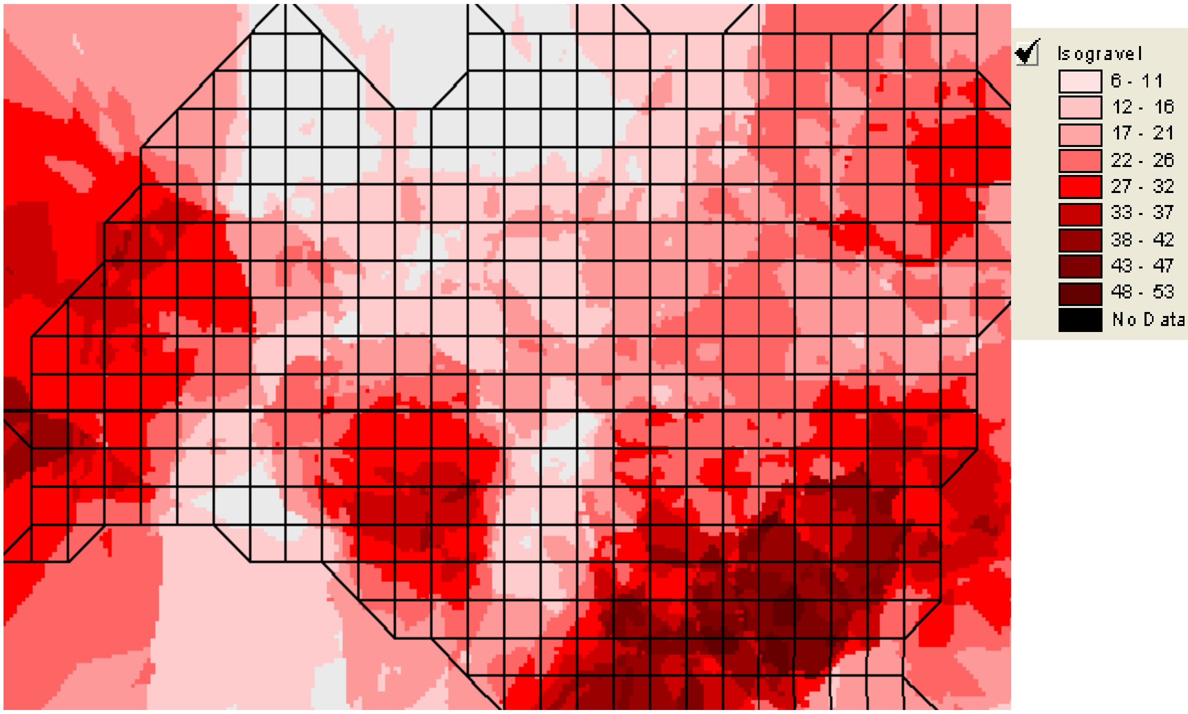


Figure 2-13 Thickness in feet for the unconsolidated gravel aquifer using well information of the report from Kennedy/Jenk consultants 2003 were imported into arcview 3.3 and spatial analyst was applied to interpolate using spline method.

3. Old gravel; Miocene to Pliocene conglomerate sand, silt and clay. At the time of the model development, there was not complete information about the thickness of this aquifer layer. Lindsey(2003) reported based on well logs a range from 22.86 to 76.2 feet thick. Based on this report, a surface of the Mioplecene Isopach was generated in GIS and shown in Figure 2-14. Future modeling will benefit more accurate descriptions and extents of the unconfined gravel aquifers.

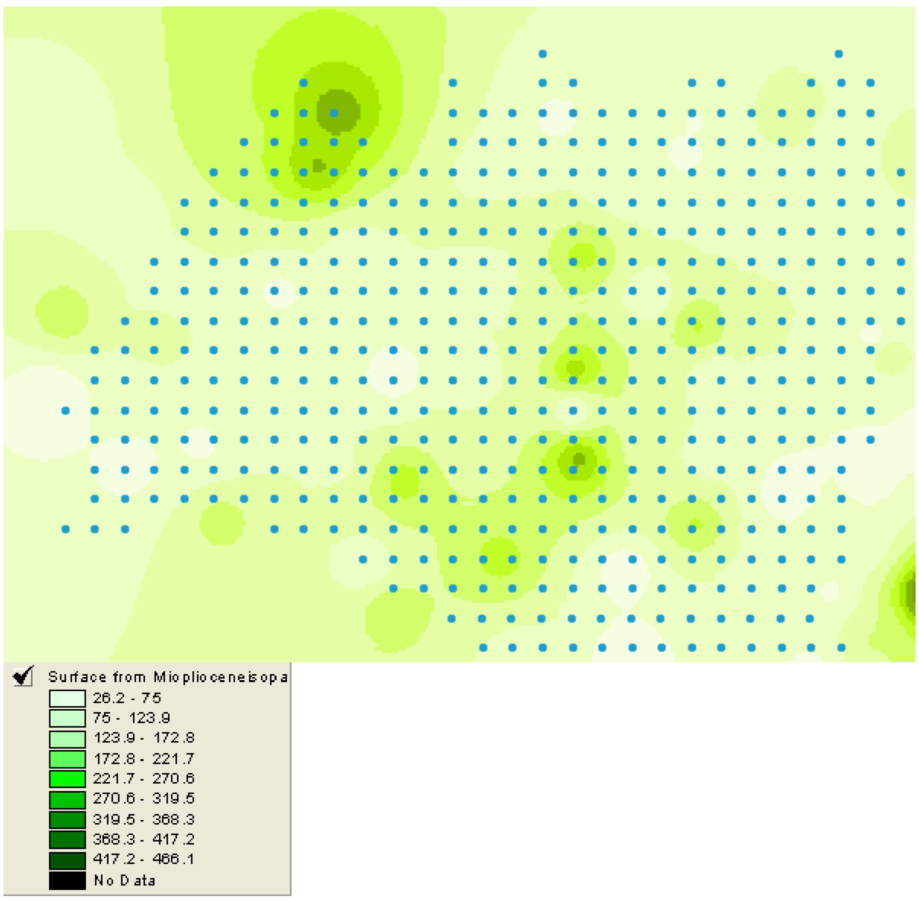


Figure 2-14 Old gravel thickness surface spline geometric interpolation arcview 3.3

The Milton-Freewater simulation model was created to simulate the movement of water in the unconfined system. Evidence suggests that there is a connection or flux of water between the confined and unconfined systems by two means. One is by pumping water from the basalt aquifer to irrigate the agricultural fields; excess water that is not transpirated by the crops or evaporated from the surface is available for infiltration and runoff. This infiltrated water will recharge the unconfined aquifer system. This mechanism is captured in the model developed here. The second connection is given by the old non-casing wells. Some of these wells were hand-made with one meter in diameter and some of the wells are abandoned. Location and description of abandoned wells are not accounted for in the model.

The Milton-Freewater hydrologic model is setup for 4 aquifer layers, Touchet silty soil, uncemented gravels, cemented gravels, and ballast aquifer system. The fourth layer is passive, not adding to the storage of groundwater. Further description of the basalt aquifer system that underlies the model area and its hydrological characteristics would allow for the incorporation of this aquifer to the existing model using the existing setup.

3. IWFM Development and Application to Walla Walla River Basin

3.1 Model Area and grid setup

The model area (fig3.1) extends from the city of Milton-Freewater, Oregon (Lon -118.42, Lat 45.962) to the state line with Washington. It is approximately 43.45 km^2 or 10,737.19 acres. For model development, the area has been divided into 416 irregular elements; the regular size for each element is 0.108 km^2 or 25.8 acres. The side of a regular quadrilateral element is 329 meters in length.

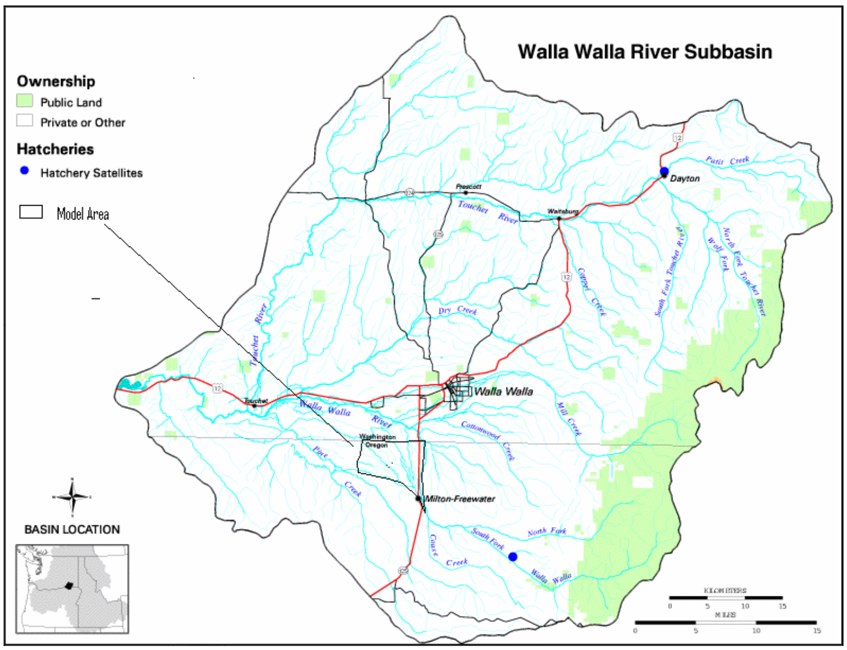


Figure 3.1 Location of Model Area

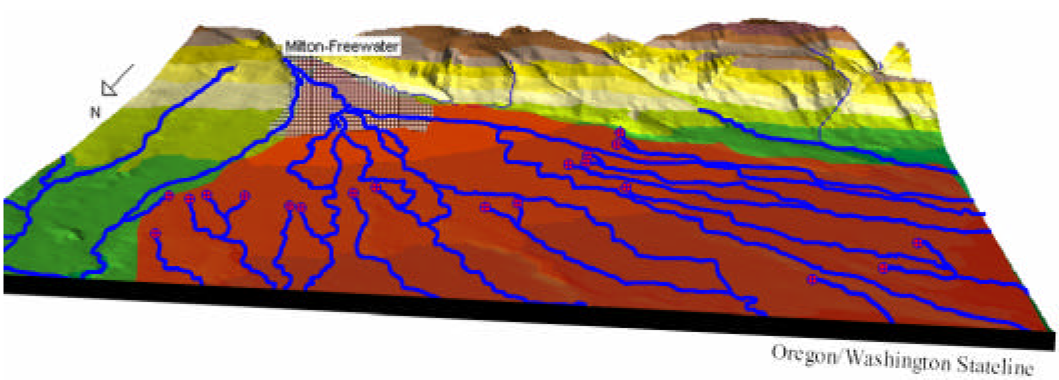
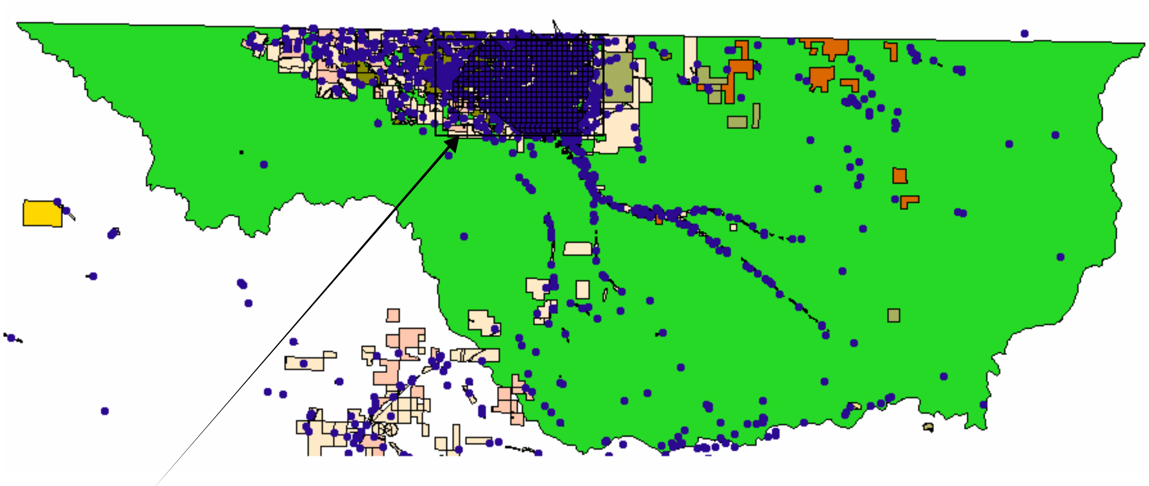


Figure 3.2 Model Area (in red) view from north boundary

The model area is a small portion (9%) of the Walla Walla River basin watershed that is shared by the states of Washington and Oregon in the Pacific Northwest of the United States of America. This first simulation attempts to represent one of the most productive areas of the watershed in Oregon. Close to 80% of the surface and ground water rights permits are in the model area (fig. 3.3)



Model Area.

Fig. 3.3, Oregon Side of the Walla Walla basin watershed. Each blue dot represents a point of diversion for a water right; each polygon represents a point of use for a specific water right.

3.2 Model Boundaries

The model boundaries for this project (fig 3) were chosen by 1) site hydrogeology and 2) information from observation wells and previous studies according to the existing data provide by WWBWC in the following manner.

Boundary 1: The main stem of the Walla Walla River is set as a no flow boundary. There is sufficient information (Metcalf 2003), that describes the interaction between the main stem of the Walla Walla River and the shallow gravel aquifer. In this connected system, (fig 3.4) the aquifer will gain and lose water to and from the river. Given that there is no lateral input or output of water from outside of the model area into this section other than the river itself, the model boundary is considered no a flow boundary.

Boundary 2: Horse Heaven Hills. From (Lindsey 2004) we can clearly see a water divide created by the site hydrogeology. Given this condition set at the base of the Horse Heaven Hills, this is a no flow boundary type and we assume there is no water flowing in to or out of this section of the model area.

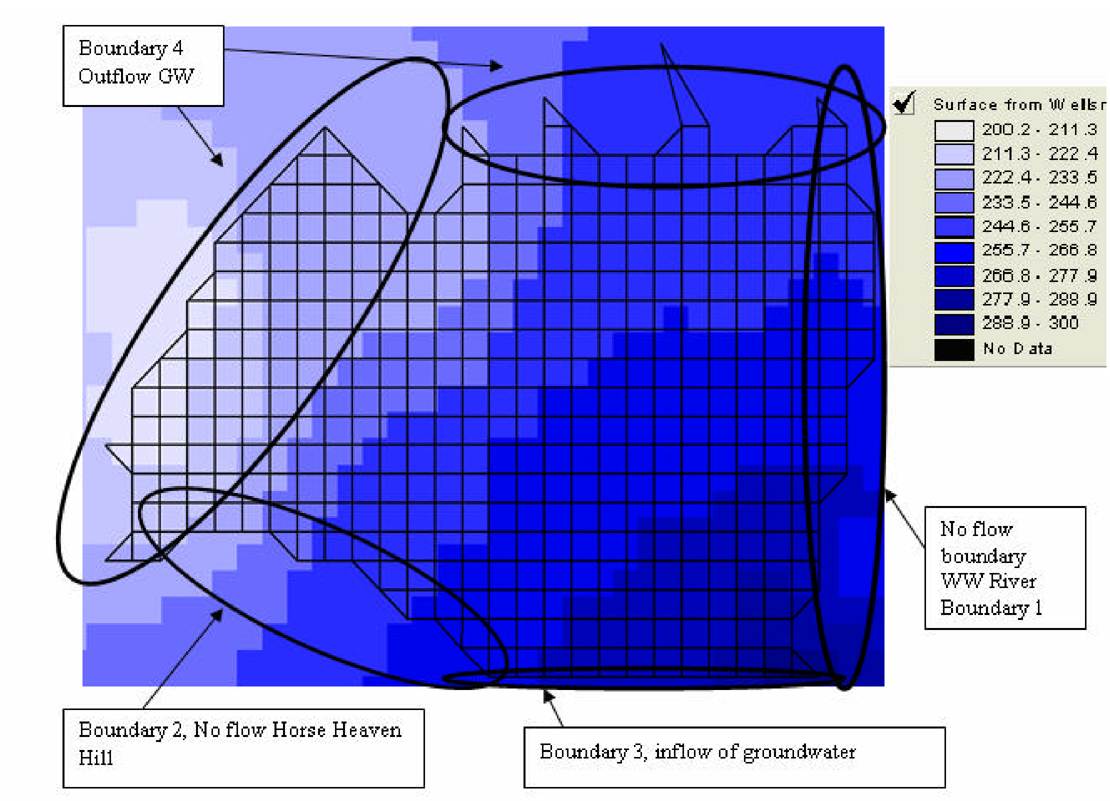


Fig 3.4 Model boundaries water table elevation using WWBWC well net and GIS interpolation.

Boundaries 3 and 4: The WWBWC has a network of observation wells (total of 30 wells) across the model area. At the base of our model, Boundary 3, we have used GW wells number 9, 27 and 23 to linearly interpolate (see initial conditions section) the height of the water table. Knowing the stratigraphy of the geology at this location, we can calculate the flow of water into our model area by:

$$q = T \, \partial h/\partial n = f(x,y,t) \qquad \text{eq. 3.1}$$

Where

q = specified flux at the boundary, (m^2/day);

T = transmissivity, (m^2/day);

h = groundwater head at the boundary, (m);

n = distance that is measured perpendicular from the boundary, (m);

f(x,y,t) = known function for all points on the part of the boundary where flux is specified, (m^2/day).

The main source of the groundwater is the upstream losses of the Walla Walla River to the shallow aquifer. Given this set of conditions: where there is not a constant head, the Drichilet boundary is discarded and with the information known by the interpolation of the groundwater elevations, we set our model boundary as a Specified head where the input is the height of the water table at each node on the boundary. The stratigraphy is dictated by the geology. The function generated by the model is the flux for each element based on the hydraulic head known for each node (water levels from well net of the WWBWC). The aquifers parameters are calculated from our aquifer tests (pump tests) and the hydrogeology characterization (Lindsey, 2003). IWFM lets the user choose between calculating the specified flux by the user outside of the model or by entering the groundwater head and letting the model calculate the flux using equation 3.1.

At the upper limit of the model, Boundary 4, we have established the same conditions as in model Boundary 3. However, in this case, water is leaving the model area. (see Fig 3.4)

3.3 Initial conditions

IWFM requires that the head values at each node at time zero be provided as well as the boundaries conditions throughout the entire length of simulation. Based on the data taken form the system of surface gauges and well loggers run by the WWBWC, the static observations made for the year 2003 were linearly interpolated using the spline method in the Geographic Informatics Software (GIS) Arcview 3.3. The spline method fits a two dimensional minimum curvature surface as suggested by Theobald (2001). The spline method is best for surfaces that exhibit smooth gentle variations such as water table heights or precipitation. (Figure 3.5)

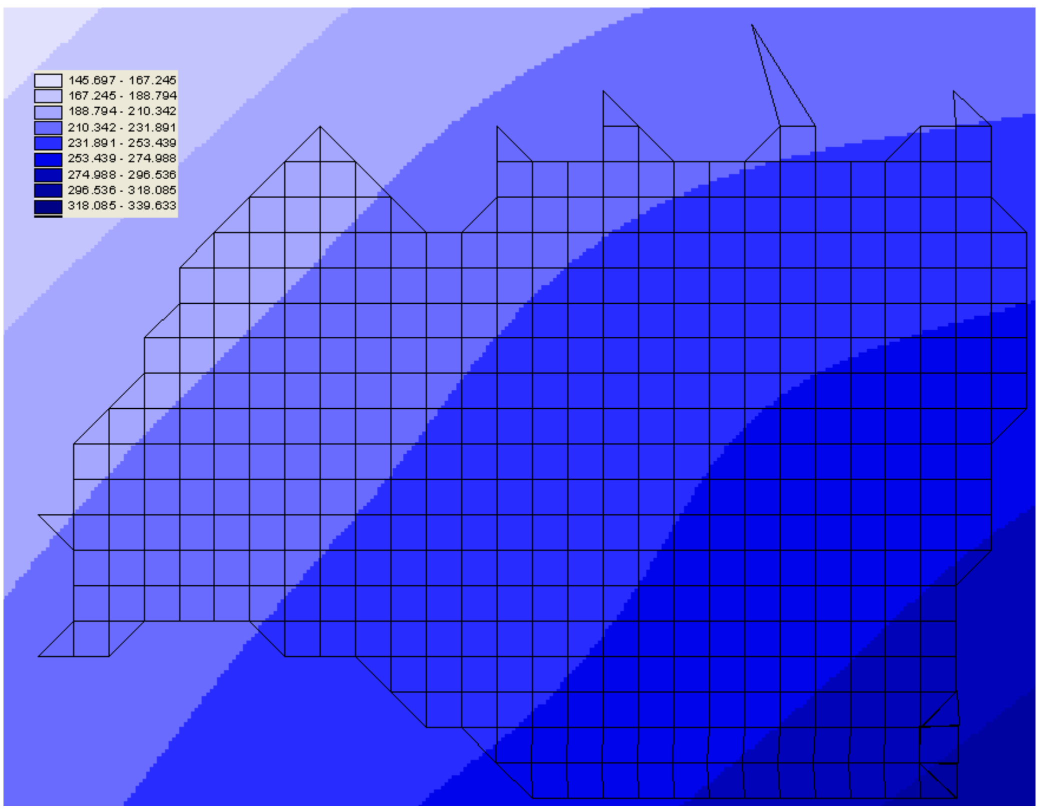


Fig 3.5 Surface from groundwater elevations created from GIS Arcview 3.3 using static measurement of wells taken by WWBWC that serves as initial conditions for our simulation model.

3.4 Hydrological Parameters.

The parameters used in the model were calibrated and adjusted from previous studies cited above. Here we present the rationale for the values used in the simulations of this thesis.

3.4.1 Aquifer parameters

The three aquifer layers that are modeled in this simulation have different hydraulic properties. The horizontal hydraulic conductivity for the Mio-pliocene cemented gravel aquifer was taken from the three pump tests made near the recharge basin project. The hydraulic conductivity for the Quaternary un-cemented gravels and the Touchet beds where taken from the literature (EPA 1986) and calibrated for the entire model area. Vertical hydraulic conductivities were assumed to have an anisotropy ratio of 1/10, concurring with the observation made by Star Davis and Rand Leek over the Walla Walla River and the irrigation canals.

Table 3.1 Calibrated vertical and horizontal hydraulic conductivities

Layer	Horizontal Hydraulic Conductivity (m/day)	Vertical Hydraulic Conductivity (m/day)
Touchet Beds Silt and Sand	6.5	2.5
Quaternary layer of Uncemented gravels	60.3	6.3
Mio-pliocene conglomerate Cemented gravels	32.0	3.2

Soil Parameters

As described in chapter 2, components of the water budget and data estimation, in the soils budget section, the model area could be divided into two sub-regions based on the description made by the Soil Survey of Umatilla County area, Oregon (USDA 1985). The main soil in sub-region 1 is Freewater very cobbly loam. For sub-region 2 the main soil is a combination of Ellisforde, Oliphant, Umapine and Hermiston silt loam with 0 to 3 % slopes. The parameters used for these soils are described in Table 3.2

Table 3.2 Calibrated soils parameters

Subregion	Field capacity	Total porosity	Fraction of excess soil moisture that becomes deep percolation	Curve number CN Agricultural	CN Urban	CN Native	CN riparian vegetation
1	0.14	0.43	0.35	98.4	99.07	99.41	99.52
2	0.22	0.47	0.35	98.4	99.04	99.41	99.52

3.4.3 Crop water use parameters

The percentage of land use specified as agricultural, urban, native and riparian vegetation varies for every element. The agricultural vegetation used in the model has the irrigation characteristics shown in Table 3.3. Crop coefficients for actual evapotranspiration calculation development days were described in the section 2.3.5 actual ET.

Table 3.3 Crop parameters, maximum root depth, and minimum soil as a fraction of field capacity, and irrigation efficiency (Allen, 1998)

crop #	crop name	abbreviation	Max root depth	Minimum soil moisture requirement	Crop irrigation efficiency
1	Alfalfa	ALF	1.5	0.45	0.57
2	Apples	APL	2	0.5	0.64
3	Cherries	CHR	2	0.5	0.64
4	City (urban)	CTY	0.5	0.35	0
5	Domestic (Rural)	DOM	0.5	0.55	0.7
6	Grapes	GRP	1.5	0.55	0.71
7	Grass and Lawn	GRS	1	0.5	0.57
8	Industrial	IND	0.5	0.35	0.75
9	Nursery	NUR	1	0.65	0.75
10	Fruit Orchards	ORC	1.5	0.5	0.64
11	Pasture	PST	1.5	0.4	0.7
12	Row Crops	ROW	0.6	0.65	0.64
13	Wheat, Non-irrigated	WHD	1	0.35	0
14	Wheat, Irrigated	WHT	1.5	0.45	0.57
15	Water Surfaces	WTR	0.5	0.25	0
16	Urban	URB	1	N/A	N/A
17	Bare soil	BRS	0.5	N/A	N/A
18	Riparian Trees	RIP	2	N/A	N/A

4. Model Results and Discussion

The results for this modeling effort can be summarized in the following sections: 1) The quantification of all major hydrologic features of the model area presented in the water budget; 2) Calibrated hydrological parameters in the model that can serve for future projects or theoretical engineering analysis over the area; 3) Example management scenario of lining the irrigation canals and the effects of the aquifer recharge testing over Johnson Creek Spring.

4.1 Water Budgets

A water budget represents the inflows and outflows of water as well as the change in storage; here we present the water budget divided for groundwater, soils, land and water use, and as a total flow budget. The water budgets analysis for the gravel aquifer system of the Oregon side of the Walla Walla basin are presented based on the average of the years from 2003 to 2006. These years provide a good representation of drought and wet years being that the year 2005 was a record drought year.

4.1.1 Groundwater Budget

The groundwater budget for this project only considers the gravel aquifer separated into the three layers previously discuss; Touchet beds, Uncemented gravels, and Cemented gravels. The simulation model does not account for the basalt aquifer but only as water input for irrigation and a small percentage for domestic use (see section 2.3.14) Table 4.1 show the results for this simulation model. The sign of each variable represents: positive for inflow and negative for outflow, for example, the ending storage (negative sign) represents the groundwater storage at the end of the simulation period. Gains from streams and irrigation canals represent the amount of water that the Walla Walla river and the irrigation canal lose to recharge the groundwater, since the they are adding water to the ground the sign is positive. The looses to springs and streams is the water that springs, irrigation canals and rivers gain from the gravel aquifer. We can observe that the amount of water recharging the aquifer from surfaces river segments is 4 times bigger that the amount of water that the aquifer is feeding to them.

Table 4.1 Groundwater budget average of the years from 2003 to 2006

Ground water Budget		
Variable	Sign	Flow m^3/year
Beginning of Storage	(+)	1.41E+11
Ending Storage	(-)	1.41E+11
Net Deep Percolation	(+)	8.08E+06
Gain from Streams	(+)	2.47E+07
Looses to springs and streams	(+)	-6.53E+06
Gain from artificial recharge project	(+)	1.45E+06
Boundary Inflow	(+)	-1.53E+07
Subsidence	(+)	0.00
Subsurface Irrigation	(+)	0.00
Tile Drain Outflow	(-)	0.00
Pumping	(-)	3.92E+06
Discrepancy	(=)	-17.55

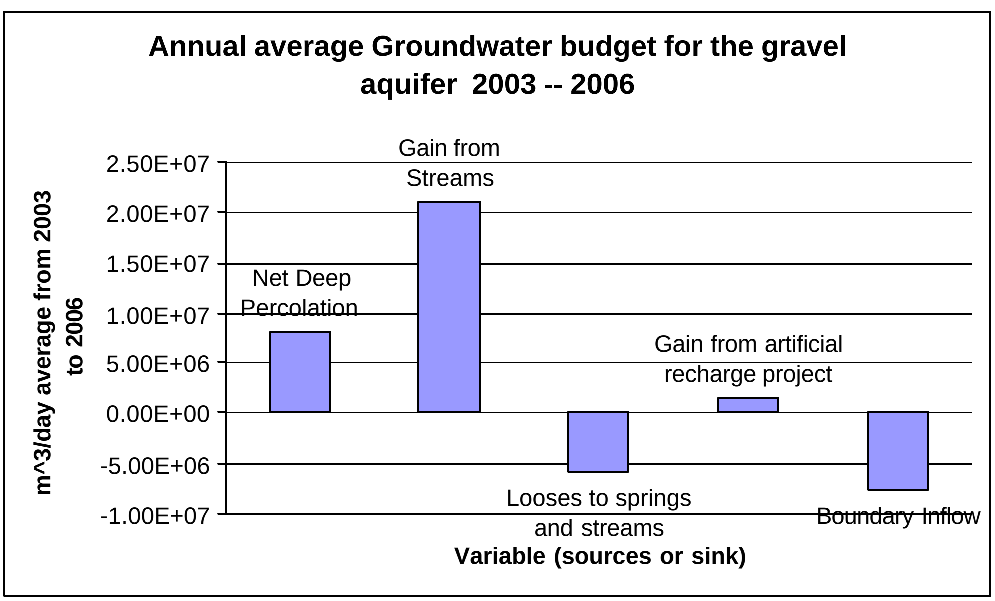


Figure 4.1 Graphic representation for the groundwater budget.

4.1.2 Soils Budget

The Soils Budget present not only the inflow of water flowing in and out of this storage unit but also provides information on processes that are used to compute soil moisture in the root zone. These are; Precipitation, Runoff, Applied Water, Reused Water, Return Flow. explanation of these variables are found in section 2

Table 4.2 Average annual soils water budget from 2003 to 2006

Soils Budget Agricultural, urban or native		
Variable	Sign	Flow m^3/year
Precipitation		1.69E+07
Runoff		7.34E+05
Prime Applied water		1.98E+07
Reused Water		1.38E+06
Total Applied water		2.12E+07
Return Flow		8.87E+05
Beginning Storage		3.01E+09
Net gain from Land expansion	(+)	0.00
Infiltration	(+)	3.50E+07
Actual ET	(-)	3.19E+07
Deep Percolation	(-)	3.07E+06
Ending Storage	(=)	3.01E+09

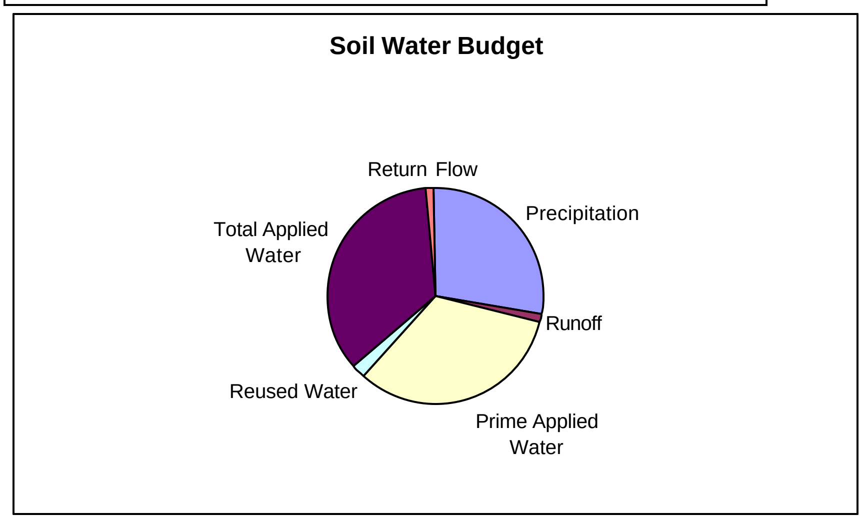


Figure 4.2 Graphic representation of the soil water budget.

4.1.3 Land and Water Use Budget

The Land and water use budget provides information about the amount of water require for agriculture and urban use. Surface water diversion where adjusted to math observation gauges run by the WWBWC and irrigation districts. The amount of pumping was adjusted to meet the water demands if this was not meet by surface water diversions. Basalt wells were used as an import of water from and external sources outside or model area.

Table 4.3 Urban and agricultural water demands and sources

Land and Water Use Budget		
Variable	Sign	Flow m^3/day
Agricultural Area	square meters	1.18E+10
Agricultural Supply requirement	**	1.86E+07
Pumping gravels	(-)	3.85E+06
Diversion	(-)	9.43E+06
Basalt wells	(-)	4.45E+06
Shortage	(=)	-2.39E+03
Re-use	**	3.81E+05
Urban Area	square meters	1.91E+09
Urban Supply requirement	**	1.18E+06
Pumping gravels	(-)	7.15E+04
Diversion	(-)	7.80E+05
Basalt wells	(-)	3.41E+05
Shortage	(=)	1.74E+03
Re-use	**	9.96E+05
** (Does not add or subtract water from the water budget)		

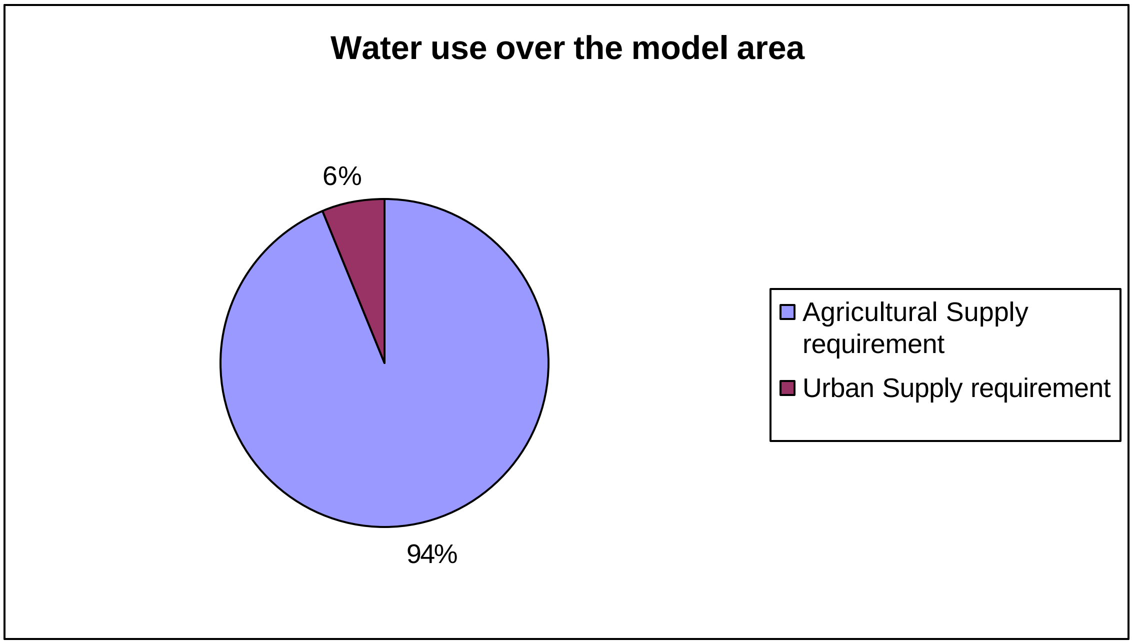


Figure 4.3 Water use over the model area

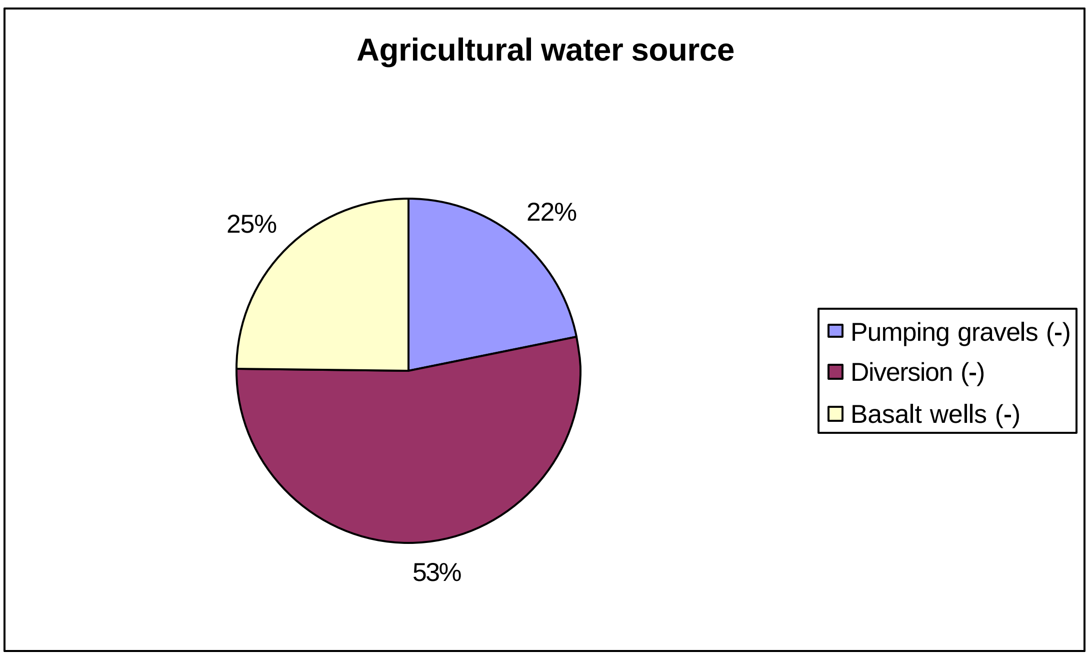


Figure 4.4 Pie chart of agricultural water use source

4.1.4 Total flow Water Budget

IWFM has the capability to post-process the subsurface flows to present a zone or total water budget. This characteristic is a helpful tool to quantify the major hydrologic components and the effects of water management practices. Table 4.4 show the total flows over the model area the percentage presented as an overall error is the ratio between inflow and out flows.

Table 4.4 Average annual total zone budget flows (m^3 /year) from 2003 to 06.

Zone Budget		
	IN	OUT
GW Storage	4.33E+07	5.13E+07
Streams	3.59E+07	1.21E+07
Net Deep Percolation	6.59E+06	0.00
Specified Head BC	4.34E+07	5.51E+07
Diversion Recoverable loss	1.21E+06	0.00
Bypass Recoverable loss	0.00	0.00
Recharge project	1.45E+06	5.18E+01
Pumping by element	0.00E+00	3.67E+06
totals	1.32E+08	1.22E+08
Overall zone error		**7.30%**

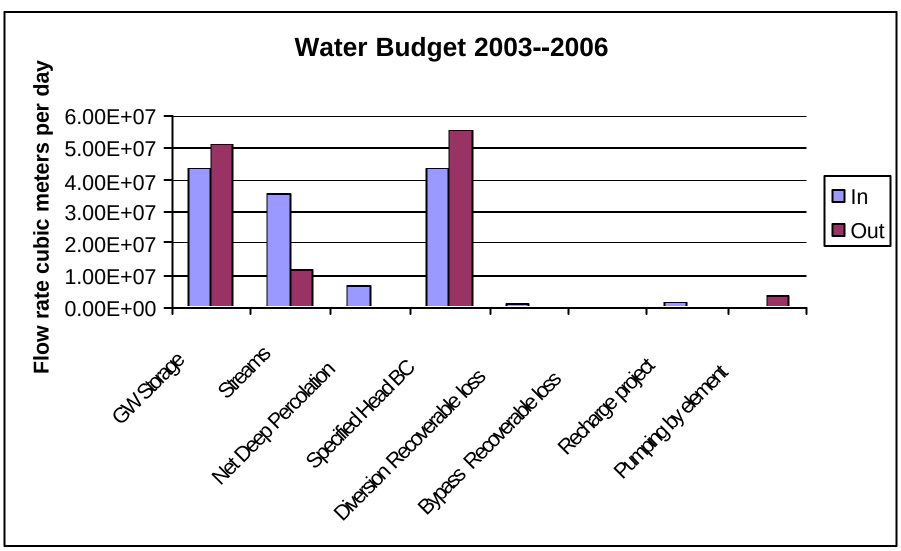


Fig 4.5 Graphic representation of flow per storage unit.

4.2. Calibration and validation

In Section 2.3, we specified all the variables used in each of the water budget analyses. Next we describe the parameters that were adjusted during calibration. We have divided the data into two sets, 2003 and 2004 for calibration, and 2005 and 2006 for validation. As expressed by Wang 1982, validation is the ability to demonstrate that the model is capable of simulating an historical hydrologic event for which data are available. The calibration step consists of adjusting the input data until computed head values match the field observed values static measurement made by WWBWC during March 2004 and March 2005. The combination of parameters and boundary conditions necessary for the model to produce the field measurements will be found by statistical comparisons. The following statistical parameters are used in the comparison:

ME: mean square error
MAE: mean of absolute values of error
RRMSE: Relative root mean square error
Eff= Coefficient of efficiency goodness of fit
R=Correlation coefficient
Cov = Covariance

A range of field values for the parameters that require calibration are determined by field observation, reported literature values and trial an error. A goodness of fit model is used to determine the accuracy of the model to simulate expected predictions.

The Mean square error ME and Mean absolute values of error MAE are describe by D. Wackerly (2002) as:

$ME = (X_{obs} - X_{sim})^2$ and $MAE = \left|X_{obs} - X_{sim}\right|$

Where Xobs = observed elevation of groundwater at a specific node or well and Xsim = simulation results for groundwater elevation per node.

The relative root mean square error and the coefficient of efficiency Eff by Nash and Sutcliffe (1970) are determined as:

$$RRMSE = \sqrt{\frac{\sum_{i=1}^{n}(x_i - \hat{x})^2}{n-1}} \qquad Eff = \frac{\sum_{t=1}^{n}(x_i - \bar{x})^2 - \sum_{t=1}^{n}(\hat{x} - x_i)^2}{\sum_{t=1}^{n}(x_i - \bar{x})^2}$$

Where

x_i= measured value

$\hat{x}$ = estimated value

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

n=number of observations

and the correlation coefficient R can be estimated as

$$R = \sqrt{Eff}$$

The coefficient R represents the fraction of the variation in the observed values explained by the simulation model and the coefficient Eff represents the percentage to fit a linear relation to the observed values. In other words it represents how well our model fits the observed values. The correlation coefficient can vary between -1 to 1 showing perfect inverse and perfect direct relationships.

Calibration was done separately for surface water and groundwater as explained later. For groundwater, observation wells were matched to simulated groundwater elevations. For surface water, flows in the rivers, canals and springs were matched to surface gauges. During these steps, the parameters shown in Table 4.5 were varied. Final values for the main parameters are shown in section 3.4. In Appendix C, the reader can find the complete files used in the final model run scenarios.

Table 4.5 variables adjusted during calibration

Variable	Definition
CN	curve number runoff
%Rf	Fraction of applied water that becomes retuned flow urban settings.
Dr	rooting depth of the crops, (L);
?r	soil moisture content of the root zone, (dimensionless);
?f	field capacity of the root zone, (dimensionless);
?t	simulation time step
AWag	agricultural applied water, (L/T);
ET0	grass or alfalfa evapotranspiration, (L/T);
Kc	Crop coefficient
fDp	deep percolation fraction between 0 and 1; 0 means entire soil moisture above field capacity becomes return flow and 1 means entire soil moisture above field capacity becomes deep percolation.
Dr	Thickness of root zone.
Ssi	inelastic specific storage
Sse	elastic specific storage
bo	initial thickness of the interbed
Clki	conductance of the lake bed material
Clki	conductance of the river bed material
Csi	Stream conductance determine by (Ksi/dsi) * LiWi
Ksi	Hydraulic conductivity of the stream bed material
Dsi	thickness of the stream bed
Li	length of the stream segment
Wi	wetted perimeter
Clki	Lake conductance determine by (Klki/dlki) Alki
Klki	hydraulic conductivity of the lake bed material
Dlki	thickness of the lake bed material (recharge project)
Alki	Area of the lake (recharge project)
Rt	Rating tables for springs, rivers and canals

4.2.1 Calibration of Groundwater

Calibration of the groundwater was done by two methods; 1) matching continuous data taken in observation wells and comparing to specific nodes and 2) by comparing the interpolations generated using GIS tools from the observations of all the wells managed by the WWBWC to the simulated groundwater elevations values of all nodes excluding the boundary nodes which are fixed and therefore show a perfect correlation value of 1.0

The first method for calibration considered observation wells that had a continuous pressure gauge recorder for the elevation of the groundwater table. An example is shown in Figure 4.6 with the observation well# 20 and the simulation for the node 268 for the year of 2003 to 2004. We can see that the model really mimics the water table elevation for the length of the simulation, the discrepancy is in average between 0.25 meters with the maximum of 0.68 at month 18 of the simulation period..

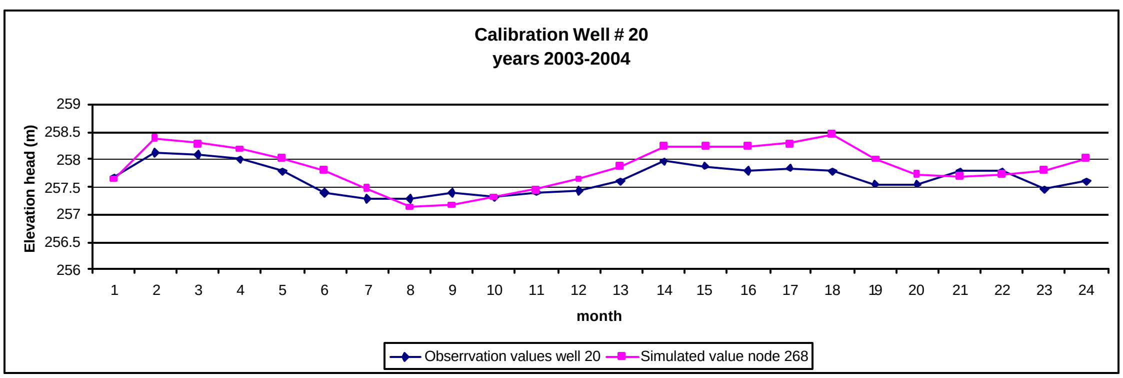


Figure 4.6 Calibration of groundwater using the comparison between well # 20 and node 268 average values per month groundwater elevation.

Calibration of Groundwater using the second method by all nodes and the interpolations of the groundwater elevations provide a better way to compare the overall performance of the model avoiding local misinterpretation of water table elevations due to pumping wells and other factors infiltration ponds, observation well close to unlined irrigation canal. As we can see from Table 4.6 the and figures 4.7 and 4.8 The observed Vs simulated heads compared well to a lineal model with correlation coefficient of 96 and 95 percent fit in the two different season chosen for calibration summer and winter.

Table 4.6 Statistics generated from the Calibration of Groundwater using all node and the observations made in June and December 2004

Calibration day using all nodes	06/3/ 2004	12/15 /2004
n	389	389
SSE	10936.03	12138.9
ME	28.11	31.21
MAE	3.94	4.45
RRMSE	5.31	5.59
EFF	0.91	0.9
R	0.96	0.95
Covariance	347.05	332.15
standard deviations Obs	18.2	17.46
standard deviations model	19.8	19.81

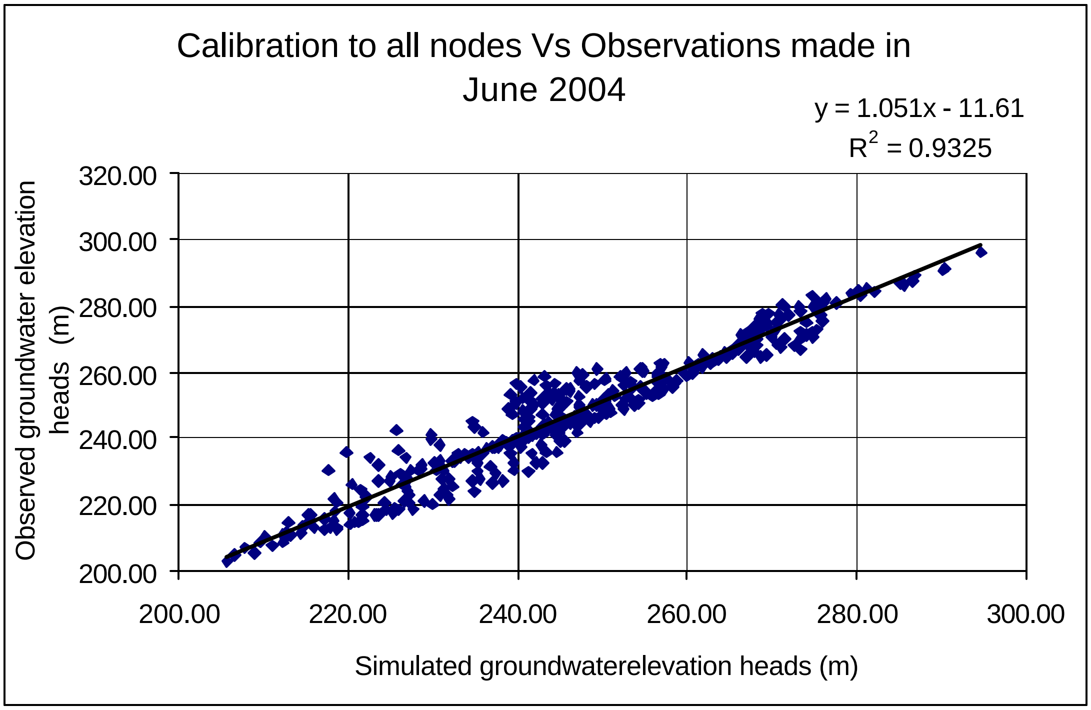


Figure 4.7 Calibration using all nodes for June 2004

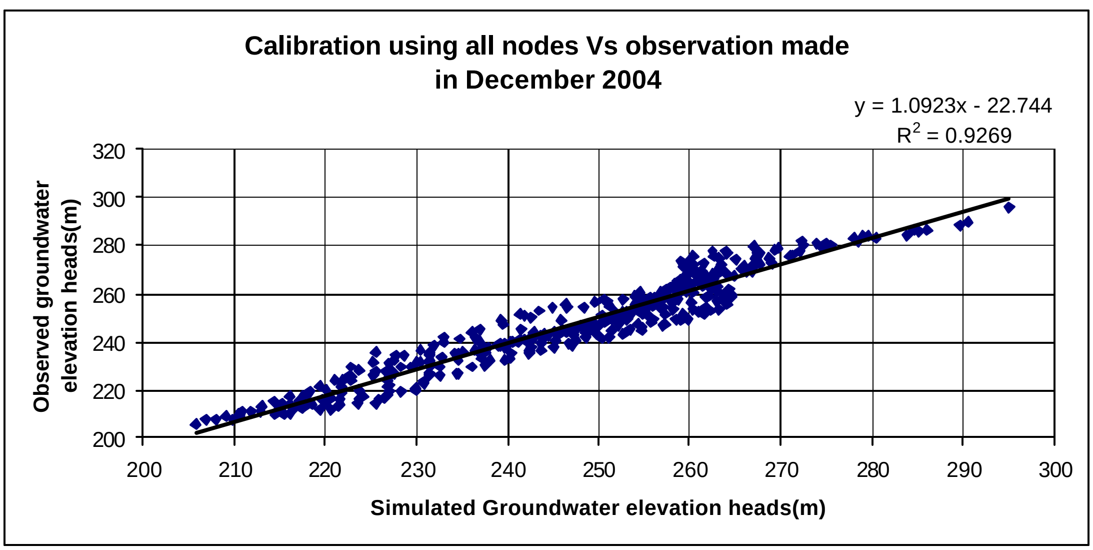


Figure 4.8 Calibration using all nodes for December 2004

The Ground water validation using 2005 and 2006 years is shown in Table 4.7 and in figures 4.9 and 4.10 the correlation coefficient for both years is 0.96 and 0.95 showing a good percentage to fit a linear model. As mention before during model validation none of the parameters were varied to math observed elevation heads.

Table 4.7 Statistic generated from the Validation of Groundwater July 06

Validation day using all nodes	7/7/2005	7/19/2006
n	389	389
SSE	10015.23	11032.48
ME	25.75	28.36
MAE	3.73	4.15
RRMSE	5.08	5.33
EFF	0.93	0.9
R	0.96	0.95
Covariance	370.28	305.55
standard deviations Obs	19.01	16.6
standard deviations model	20.15	19

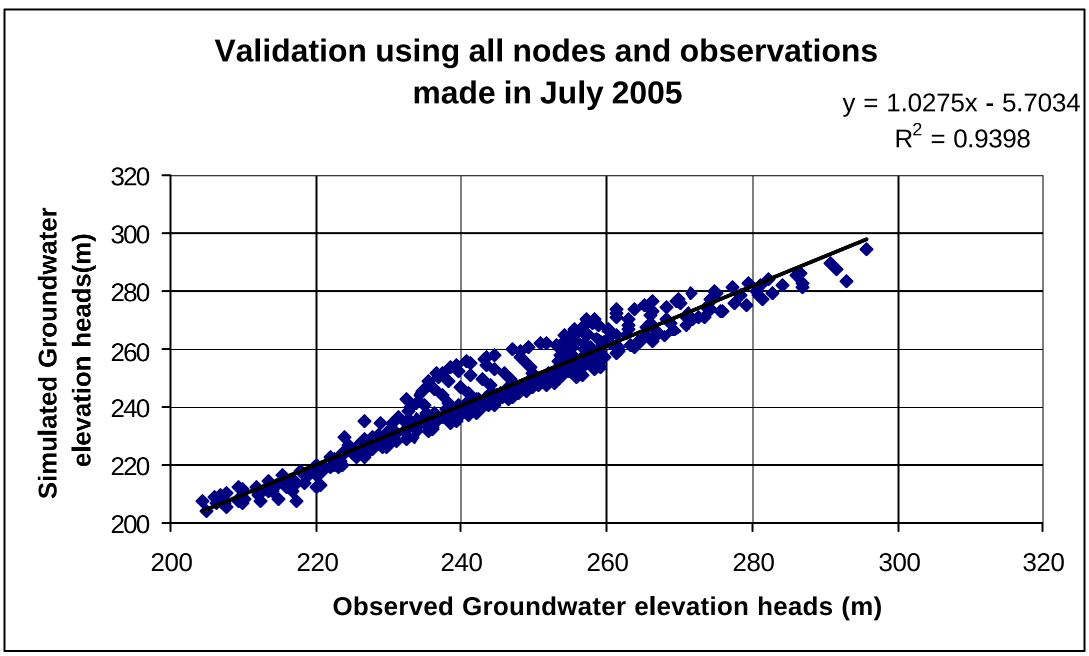


Figure 4.9 Validation using all nodes for July 2005

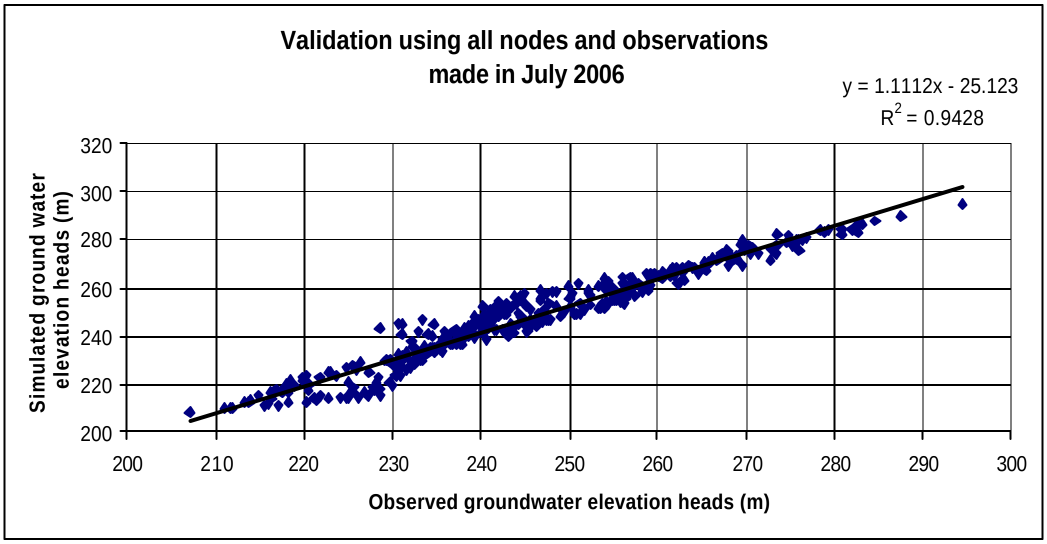


Figure 4.10 Validation using all nodes for July 2006

4.2.2 Calibration of Surface Water

Calibration to surface water was done separately for each stream segment, that is the Walla Walla River, all the irrigation canals and all the springs. For the Walla Walla River, continuous gauge data made the calibration and validation easy to visualize. For springs and irrigation canals, surface water flows were matched to static measurements over different periods.

The Walla Walla River was calibrated and validated using the gauges at Peeper bridge represented in the model by river node 21and the gauge at Tumalum represented with the river node 11. figures 4.11 and 4.12 show the calibration and validation for the Main stem of the Walla Walla river again the data was partition in half for calibration and validation showing a good fit of the model with a correlation coefficient of 0.92 Table 4.8

Table 4.8 Walla Walla River statistics

Walla Walla River using 2004 water year	
n	443.00
SSE	3.37E+12
ME	7.60E+09
MAE	2.30E+04
RRMSE	8.73E+04
EFF	0.85
R	0.92

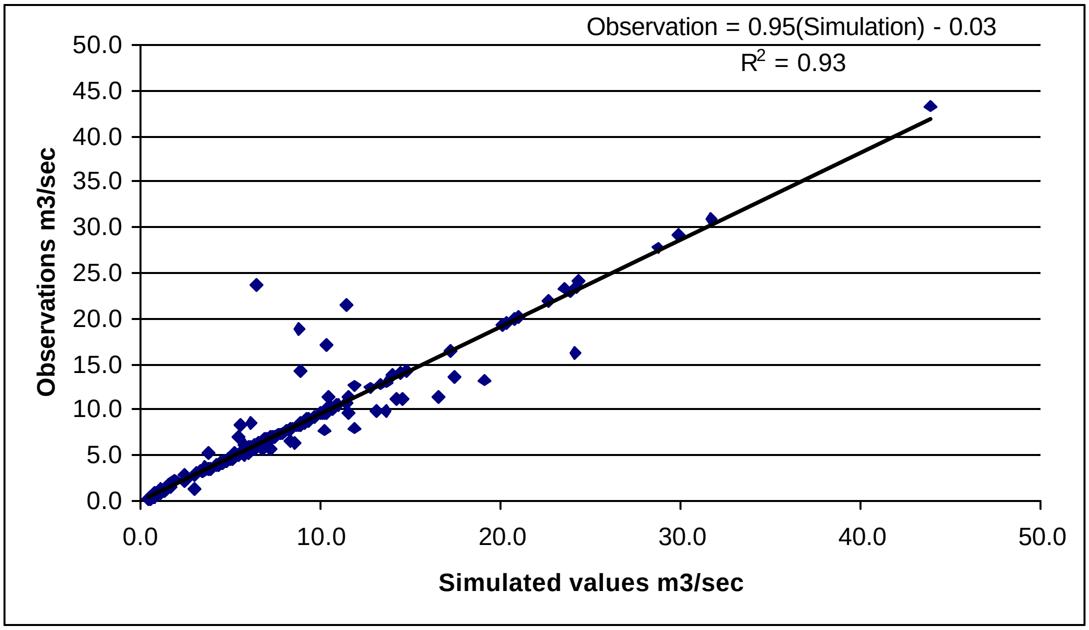


Figure 4.11 Walla Walla River Calibration for 2003 to 2004

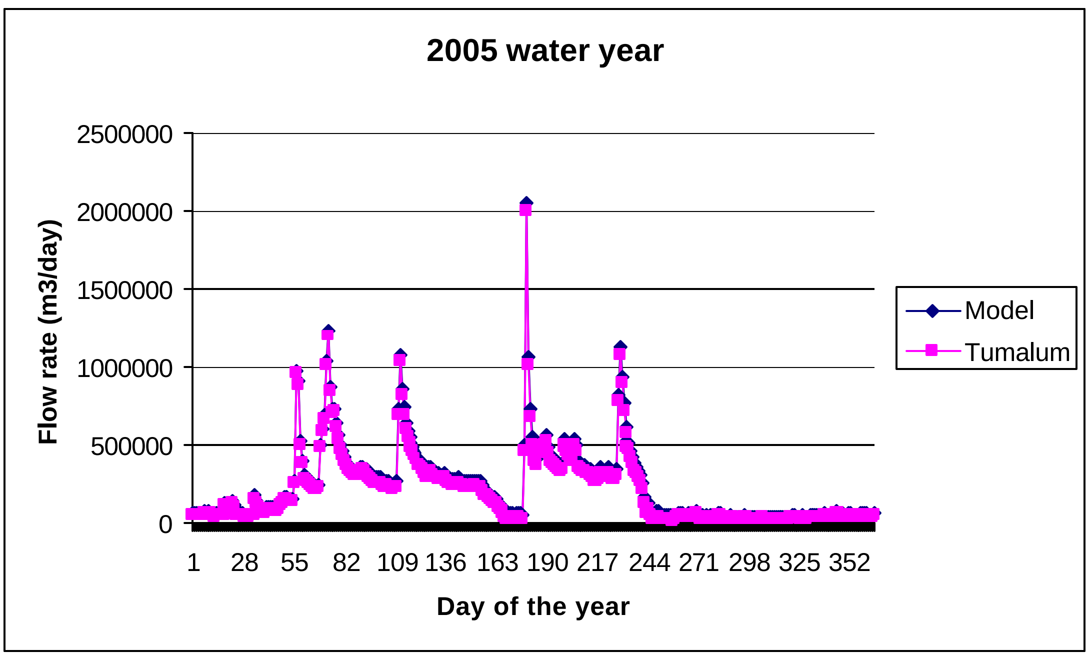


Figure 4.12 Walla Walla River validations for the years 2005 to 2006

4.3. Modeled Scenarios

Two scenarios were simulated and compared to the flow in Johnson Creek Spring in order to observe the extended effects of artificial recharge practices. The first scenario was run without the Hudson bay aquifer recharge project variables were kept constant. The second scenario was run with all the irrigation canals lined. This was done by setting the conductance term of the stream bed material, $C_{si,}$ to zero. The results of these two scenarios were compared with the flow at Johnson Creek Spring with the model running under current conditions. Figure 4.13

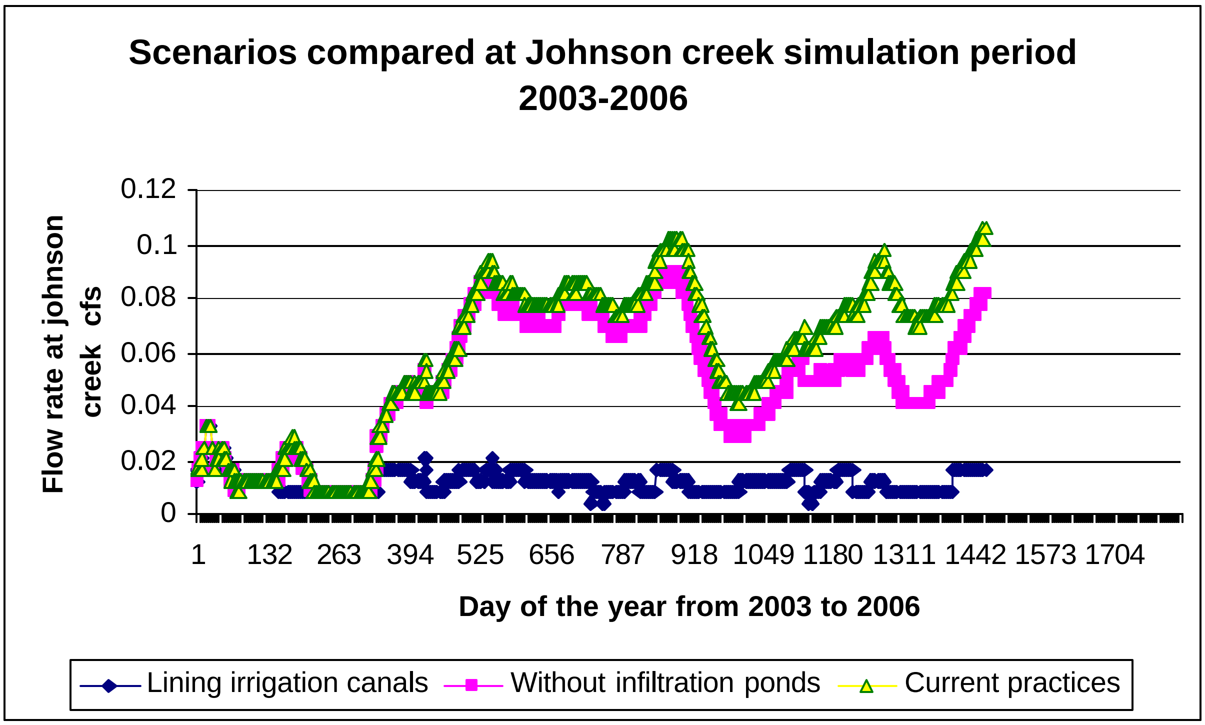


Figure 4.13 Scenarios compared at Johnson creek

Model prediction for surface flows in Johnson Creek Spring showed that a severe impact in flow can be caused by lining the irrigation canals. The incremental flow in Johnson Creek observed in the current condition scenario and without the infiltration recharge project scenario concurs with the observations made by the WWBWC: about 20% to 40% of incremental flow is observed after the second year of the project operation.

4.4 Model limitations and recommendations for further work

The objective of this project was to establish a hydrologic simulation model to quantify the major hydrologic features of part of the Walla Walla River basin. Model scenarios would help visualize and quantify different management practices. These could include: lining of specific irrigation canals, growth in size or number of infiltration ponds, change in cropping, climate variation, etc. A finer model grid descritization over interest areas like infiltration ponds would help in the correct simulation of these scenarios, especially the infiltration ponds that have a much smaller area than the actual grid size.

Model boundaries were generated from the average head of observed groundwater elevation values of each month for the entire simulation period. Data gathered for these wells have been analyzed using digital elevation models and constant pressure gauges that require further inspection of real well elevation with respect to sea level and calibration and verification of these pressure gauges. If this model were to expand in model area, a better definition of naturally occurring boundaries would help in the model set up.

The available data for the geological description of the aquifer system at the time of the model development described the three aquifer layers mentioned in this report. New geological maps and descriptions have been made recently over the model area. This and new aquifer testing in different locations over the model area could improve our understanding of anomalies in hydraulic conductivities.

Surface water data requires further development for rating tables and flow measurement in springs and irrigation canals. During model calibration, the model was very sensitive to changes in stream bottom elevation, stream width, and stage discharge table. In order to capture the real situation more stream nodes and river sections should be surveyed and continuous gauge data could be used to calibrate and validate irrigation canals and springs.

5 Conclusions

The overall statistics shown in the model calibration and validation give satisfactory results for the groundwater component of the model with correlation coefficients of 95 and 96 percent (as shown in section 4.2.1 Groundwater model calibration). Two methods for model calibration were shown. One is to use a single well (normally a pumping well for irrigation with a pressure gauge) and to match its corresponding node. The second method used the simulation results for groundwater elevation head at each node and compared them to the interpolation from the observation wells groundwater elevation surveyed by the WWBWC.

The preferred method to compare observed versus simulated values was the second method using all the groundwater nodes. Discrepancy between real versus simulated head at specific well locations compared to model nodes is expected. First, pumping wells create local drawdown of the water table; and therefore a nearby observation well would not capture the real water table elevation. Second, there are possible misrepresentations in the conceptual model, such as missing a pumping well or irrigation diversion. Third, local anomalies of the aquifer parameters such as perched water tables and clay lenses may not be accounted for by the simulation model.

The Walla Walla River statistics (section 4.2.2 Surface water calibration) show that the model is capable of simulating the river connection to groundwater for the years of 2003 to 2006 with a correlation coefficient of 92% for flow measured at the outflow (river node 22, Pepper Bridge gauge). Variability is expected between simulated versus observed values in the Walla Walla River due to water diversions not accounted for in the model and by man-made irrigation structures that return excess applied water from irrigation to the Walla Walla River that otherwise would return to an irrigation canal or to the Big Spring. This modeling effort tried to recreate the observations made by: Stejeskal (2003), Bower (2007) and Metclaf (2003).

These previous studies focused on the interactions between surface and groundwater for the Walla Walla River from the levee section located at Milton-Freewater to the state line between Oregon and Washington.

The average flows for each spring were matched to single observations made by the WWBWC (personal communication). The observations were made for certain months and most of them in the past two years. There was not enough information for all the spring flow measurements to compare to model simulations. Further model development for stream bottom elevations and the development of stage discharge rating tables are necessary to fully capture the development of flow for the complete water year for the entire system of springs. This project did not match the flows through the springs for the entire year, but instead simulated average flows.

It is important to mention that the inflow of water for the springs is not adjusted by the user but instead by the model groundwater elevation output and set up of the spring bottom elevation, wetted perimeter and stage-discharge table (section 2. 3.9 River gains from Groundwater). The parameters that were adjusted for calibration were the hydraulic conductivity of the stream bed and the stage-discharge rating table. Variation between simulated and observed values are expected due to diversions not accounted for in the model and the redirection of flow that otherwise would flood houses in the section where old springs went dry that now have flow. A finer model discretization is necessary to capture the water year development of flow through all the springs modeled. Additionally, a detailed analysis of the spring hydrological parameters is required with continuous data for calibration and validation.

The irrigation canals had to be simplified for the simulation given their complexity and proximity. In some cases, (ford system) ditches that were running parallel a few feet away from each other, that had the same water diversion source and later joined as a single irrigation canal, were modeled as single river segments. In this case, parallel segments or small canals diversions were not considered.

Irrigation canals modeled as river segments were adjusted by surface water diversions based on agricultural need per element and rating tables to match the observations in the gauges measured by the WWBWC (personal communication) and the irrigation districts, Hudson Bay District Improvement Company, Dayton Ditch Company and Gardena Farm Ditch Company. These gauges consisted of weirs or flumes with depth probes that also measured water temperature and ambient temperature. These data were incorporated into a water budget for streams. The results show that the irrigation canals lose an average of 27% of their inflow, a number equivalent to that calculated by HDR Engineering (2004).

The amount of water recharged by these irrigation canals is necessary to maintain adequate water table levels. As an example, a first scenario with the irrigation canals lined was simulated and compared to the flows at Johnson Creek Spring (4.3 Modeled Scenarios) with current leaky irrigation canal conditions and infiltration basin. A second scenario was run without the shallow aquifer recharge project. The first scenario showed a complete cessation of flow at Johnson Creek Spring providing agreement that flow in the spring is occasionally stopped by the recharge of the unconfined aquifer up gradient. Most of the recharge, 80%, is from the irrigation canals and after the third year of simulation (second year of the project), 20% of the flow is from the shallow aquifer recharge.

Continued declining levels of the basalt aquifer (Golder Associates 2007) will increase the cost of pumping by increasing the depth from which wells withdraw water. Based on the land use water demands for agricultural and domestic areas, surface water satisfies 53% of the demand leaving 47% of the water demand coming from the pumping wells. Based on water rights, it is estimated that 25% of the pumping is coming from the basalts aquifers and

22% from the unconfined shallow gravel aquifer. Reducing pumping from the basalts will cause a drop of the water table. Pumping from the gravel aquifers will need to be increased to satisfy the water demand in this area.

To sustain the groundwater demand, new water management practices like the shallow aquifer recharge are required. This results in not only a positive impact on flow at Johnson Creek Spring, but also an increase of the groundwater recharge by up to 18% (when the project runs at 12 cfs - $0.34m^3$/sec). This suggests that aquifer recharge could be a good management tool for future protection of agricultural demand by storing water in the shallow aquifer and recharging groundwater levels to maintain acceptable flows in springs and rivers. The location and number of additional infiltration basins were not addressed in this preliminary modeling effort. But this model could be used to visualize the effects of new infiltration basins in the model area.

The Water Budget components for actual evapotranspiration, pumping and land water use show that 96% of the water used goes to agricultural practices with irrigation efficiencies at a maximum of 65%. Even though most of the water, 53%, comes from surface water diversions, a large portion is dependent on groundwater both from the basalts (25%) and from the shallow gravel aquifer, which is 22% of the applied water.

The overall error shown by the water budget is 7.5%, measured as the ratio of inflow versus outflow from the model. This overall error indicates that the conceptual model for the shallow aquifer with a strong hydrologic connection between surface and groundwater, a series of springs, unlined irrigation canals and the Walla Walla River is fundamentally sound. Groundwater storage and specific head boundaries are expected to change with new information provided by geologic reports. These reports will show the true thickness of the aquifer layers. The Mio-pliocene conglomerate layer, which lies above the impervious silty clay layer above the Basalt aquifers, is expected to be thicker than the value used in this model. The amount of infiltration, runoff, applied water, actual ET and deep percolation are independent of the aquifer thickness of the third layer; therefore they are not expected to change with the same agricultural and urban water demands.

The groundwater and the soil water budgets showed very small (less than 0.1%) discrepancy between the input and output components. Therefore, we have good agreement with simulation of major hydrologic features like actual ET and pumping from the gravel aquifers. These values have never been calculated for this area based on the more complete information applied in this modeling effort. Previous studies (Barker and MacNish 1974) show uncertainty in their conceptual model of the gravel aquifers (Golder Associates, 2007), and are based on a much larger model area with fewer observations for surface water and groundwater. Scaling of these models to the model area used in this project is impossible due to land use and hydro-geological conditions. The reader should also be careful when comparing the aquifer hydrological parameters. This model used different values for each aquifer layer with a thickness varying across the model area.

This modeling effort was made possible in part by the helpful participation of Eminc. C Drogul of the California Department of Water Resources, who during my two visits reviewed the software code for the necessary characteristics to simulate the Walla Walla River Basin. The attached files in Appendix C are all the files necessary to run the Milton-Freewater Hydrological Model for the preprocessor, simulation and water budget postprocessor subroutines in IWFM version 3.0. The scenarios of lined canals and canceling the infiltration basins are also included. IWFM is available to the public from the California Department of Water Resources webpage. The source code is also available to modify and compile IWFM source code in Compaq Visual Fortran version 6.6.

Future development of this model includes the possibility of a user interface with geographic information systems and a grid generator that will greatly influence the decision of the users who wish to start a new application. IWFM was a logical choice to simulate the characteristics found in the Walla Walla Basin and for modeling the interactions between surface and groundwater, and computing agricultural and urban water demands. The required amount of effort necessary to setup the application with IWFM will depend greatly on the information available for each basin and the grid size chosen for the simulation model. This project was completed in two and a half years, from grid setup to model calibration and validation. Most of the time was spent collecting and analyzing the information from well logs, stream gauges, and aquifer testing.

The hydrologic model produced by this project can be used to test the aquifer response to water management practices, as well as changes in climatic parameters. In order for the model to be used to understand individual hydrological processes, it is necessary to understand the scale at which these processes occur and, if warranted, modify the grid size at specific locations.

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APPENDICES

Appendix A Digital Model Simulation Theory

We understand a model as a representation of a real system with specific objectives. A regional hydrological model tries to mimic the flow of water at a specific location dividing the model into different units or different pools of processes, for example, groundwater, surface water and atmosphere. The processes and theory behind the simulation model are described next.

1) Darcy's Law and the continuity equation

The core equation for a simulation model is the main flow equation which for a groundwater model it's found by combining Darcy law and the continuity equation

Henry Darcy 1803 -1857

1 Henry Darcy established that for a given type of soil, the volume discharge Q is directly proportional to the head drop h_2-h_1 (Total head is defined as the height of water above see level) and to the cross section area A, but is Inversely proportional to the length difference $x_1 - x_2$. The proportionality constant K is hydraulic conductivity.

Darcy's Law

$Q/A = K * \Delta H/\Delta x$

Darcy's Law in three dimensions, let Q/A=q

q_x= -K (dh/dx), q_y= -K (dh/dy) & q_z= -K (dh/dz)

Eq. A.1

2. Continuity Equation for a steady-state flow assumes water is not compressible and stored in an elemental unit and considering there are no sources or sinks. The amount of water flowing into an elemental unit of volume is equal to the amount of water flowing out of the elemental volume in the three dimensions. In the case of homogeneous isotropic medium, the net change in discharge rate is:

$$dq_x/dx + dq_y/dy + dq_z/dz = 0$$

Eq. A.2

3. Laplace's Equation: combines Darcy's Law and the continuity equation into a single second –order partial differential equation.

$$d^2h/dx^2 + d^2h/dy^2 + d^2h/dz^2 = 0$$

Eq. A.3

4. Poisson's equation: Precipitation and discharge from a well are examples of positive and negative recharge of ground water. In this case Laplace's equation is no longer equal to zero.

a) For a confined aquifer:

Let us consider a two-dimensional flow the Laplace's equation becomes

$$d^2h/dx^2 + d^2h/dy^2 = -R(x,y)/T$$

Eq. A.4

Where: R = Recharge rate = -Q/A

And T= Aquifer thickness b * hydraulic conductivity K

And R(x,y)= $-Q/\Delta x \Delta y$ in the case of a pumping well, recharge will have a negative sign and the cross section area A would be $\Delta x \Delta y$ (we need to account for the drawdown, or area affected by pumping)

b) For an unconfined aquifer:

Let us assume the flow is horizontal and that the hydraulic gradient is equal to the slope of the free surface (Dupuit Assumptions)

For a two-dimensional flow in an unconfined aquifer Poisson's equation becomes

$$K/2\ (d^2h^2/dx^2 + d^2h^2/dy^2) = -R$$

Eq. A.5

To get the same solution for a confined aquifer we can substitute $v=h^2$ so:

$$d^2v/ dx^2 + d^2v/ dy^2 = -2R/K$$

Eq. A.6

The finite difference form of Poisson's equation for unconfined aquifer with the substitution is in the same form that for a confined aquifer but after a solution has been obtained in terms of v, the head is obtained by taking the square root of v.

5. Unsteady Flow in the continuity equation involves the change of head over time. The rate at which water is released from storage is $\Delta V_w/\Delta t$ where ΔV_w is the volume of water released from storage.

If we define Storage S as

$$S=-\Delta V_w/(\Delta x \Delta y \Delta h)$$

Eq. A.7

Poisson's equation for a confined aquifer not at steady state becomes

$$d^2h/dx^2 + d^2h/dy^2 = [(S/T) * (dh/dt)] - [R(x,y,t)/T]$$

Eq. A.8

For an unconfined aquifer, the governing equation becomes Poisson's equation plus the incorporation of the storage coefficient S. In an unconfined aquifer the storage coefficient is also called specific yield.

$$K/2 (d^2h^2/dx^2 + d^2h^2/dy^2) = S(dh/dt) - R(x,y,t)$$

Eq. A.9

If we let $v=h^2$ then $(dv/dt) = (dh^2/dt) = 2h(dh/dt)$

So for an unconfined aquifer the governing equation is:

$$K/2 (d^2v/dx^2 + d^2v/dy^2) = (S/2vv)(dv/dt)-R(x,y,t)$$

Eq. A.10

6. Flow equation used in IWFM

IWFM incorporates into our transient unsteady flow equation the flow between layers of aquifers, flow from streams, lakes, subsurface irrigation (tile drains), pumping and land subsidence.

$$\frac{\partial S_s h}{\partial t} + \bar{\nabla} \cdot \vec{q} = I_u q_u + I_d q_d + q_o - q_{sd} + \delta\left(x - x_s, y - y_s\right)\frac{Q_{sint}}{A_s} + \delta\left(x - x_{lk}, y - y_{lk}\right)\frac{Q_{lkint}}{A_{lk}} + \delta\left(x - x_{td}, y - y_{td}\right)\frac{Q_{td}}{A_{td}}$$

Eq. A.11

where

Ss = storativity, (dimensionless). It is equal to the storage coefficient So, for a confined aquifer and specific yield, Sy, for an unconfined aquifer;

h = groundwater head, (L);

q = specific discharge field, (L2/T);

qu = rate of flow into the aquifer layer from the upper adjacent layer, (L/T);

Iu = indicator function for top aquifer layer, (dimensionless); =

1 if layer is not top aquifer layer

0 if layer is top aquifer layer

qd = rate of flow into the aquifer layer from the lower adjacent layer, (L/T);

Id = indicator function for bottom aquifer layer, (dimensionless);=

1 if layer is not bottom aquifer layer

0 if layer is bottom aquifer layer

d = dirac delta function, (dimensionless);

xs = x-coordinate of a stream location, (L);

ys = y-coordinate of a stream location, (L);

Qsint = stream-groundwater interaction (see the discussion on stream flows), (L3/T);

As = effective area of the stream through which stream-groundwater

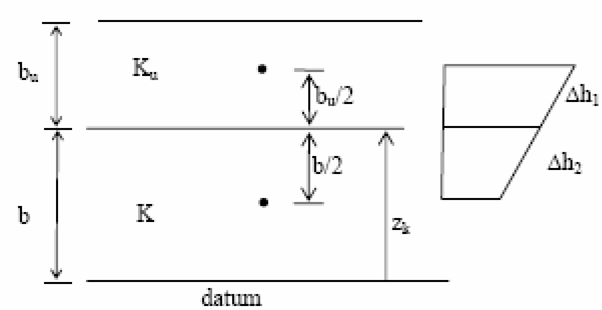
interaction occurs, (L2);
xlk = x-coordinate of a lake location, (L);
ylk = y-coordinate of a lake location, (L);
Qlkint = lake-groundwater (see the discussion on lakes), (L3/T);
Alk = effective area through which lake-groundwater interaction occurs,(L2);
xtd = x-coordinate of a tile drain or subsurface irrigation system, (L);
ytd = y-coordinate of a tile drain or subsurface irrigation system, (L);
Qtd = tile drain outflow from or subsurface irrigation inflow into the
groundwater system, (L3/T);
Atd = effective area through which tile drain outflow or subsurface irrigation inflow is occurring, (L2);
qo = other sources/sinks such as pumping, recharge, subsurface inflow
from adjacent small watersheds, etc., (L/T);

qsd = rate of flow into storage due to the compaction of interbeds, (L/T);
del operator, (1/L);
x = horizontal x-coordinate, (L);
y = horizontal y-coordinate, (L);
t = time, (T

Flow between aquifers

The interaction between aquifers can be divided into flow between aquifers when they are separated by an aquitard (interaction with confined aquifers) and flow between aquifers that are not separated by and aquitard, which in the model for Milton_Freewater this occurs between the first three layers. Interaction between touchet bed and young gravel and the interaction between the young gravel aquifer and old gravel aquifer. IWFM considers the flow or interaction between aquifer layers and surface water in the following manner:

Aquifers not separated by an aquitard.



$$\Delta h = \Delta h_1 + \Delta h_2$$

Eq. A.12

$$q_u = -\frac{K_u}{b_u/2}\Delta h_1 = -\frac{K}{b/2}\Delta h_2 = -L_u \Delta h$$

Where: Ku = vertical hydraulic conductivity of the aquitard between the aquifer layer and the upper adjacent layer, (L/T);

bu = thickness of the aquitard between the aquifer layer and the upper adjacent layer, (L);

Δh = head difference between the top and the bottom of the aquitard,(L);

Lu = leakage coefficient between the aquifer layer and the upper adjacent layer, (1/T).

The difference between the aquifers height can be expressed as:

$$\Delta h = \begin{cases} h - h_d & \text{if } h \geq z_k \ ; \ h_d \geq z_k \\ z_k - h_d & \text{if } h = z_k \ ; \ h_d \geq z_k \\ h - z_k & \text{if } h \geq z_k \ ; \ h_d < z_k \\ 0 & \text{if } h = z_k \ ; \ h_d < z_k \end{cases}$$

Where: zk = elevation of the interface between the adjacent aquifer layers, (L).

Substituting the last two equations, the Leakage coefficient L_u in the form of two aquifers not separated by an aquitard becomes

$$L_d = \frac{1}{0.5\left(\frac{b_d}{K_d} + \frac{b}{K}\right)}$$

Eq. A.13

and substituting into the main flow equation:

$$0 = \frac{\partial S_s h}{\partial t} - \vec{\nabla}\left(T\ \vec{\nabla} h\right) + I_u L_u \Delta h^u + I_d L_d \Delta h^d - q_o + q_{sd} - \delta\left(x - x_s, y - y_s\right)\frac{Q_{sint}}{A_s} - \delta\left(x - x_{lk}, y - y_{lk}\right)\frac{Q_{lkint}}{A_{lk}} - \delta\left(x - x_{td}, y - y_{td}\right)\frac{Q_{td}}{A_{td}}$$

Eq. A.14

This is the flow equation that IWFM uses where Δh^u and Δh^d are introduced to differentiate between the head difference between the aquifer and the upper adjacent layer. This equation serves for any case of confined and unconfined aquifers. The model is able to distinguish based on the input data provided for the stratigraphy characteristics of the aquifer.

The solution to this partial differential equation requires the specification for the boundaries and (see initial conditions section and model boundaries) initial conditions. The previous equations mentioned in the model budget that take into account the effect of rivers, lakes and subsidence also need to be solve simultaneously in order to effectively process the interaction among them. Since most of the equations are non-linear, obtaining an analytical solution is impossible. IWFM utilizes the Garlekin finite element method to approximate the solution to these systems of equations.

The Garlekin Finite Element Method

The Garlekin Finite element method is a hybrid of the finite element and finite difference approximation to solve for the unknown value of groundwater heads from our main flow equation at specific node locations. As has been previously mentioned, the non-linear conditions force us to approximate the solution. To do this we have divided the entire model area into small sub areas called elements. Each element is defined by three to four nodes; our points where information is provided about the total groundwater head (pressure head + elevation head). The size of the elements can be varied but the numbering of the nodes has to be consistent. For example we can define a lake into one or several elements depending in the lake and element sizes. In fig 6.1 a lake is defined into one single triangular element. In the other hand, a river is just defined by segments or branches which only require the specification of an upstream and downstream node.

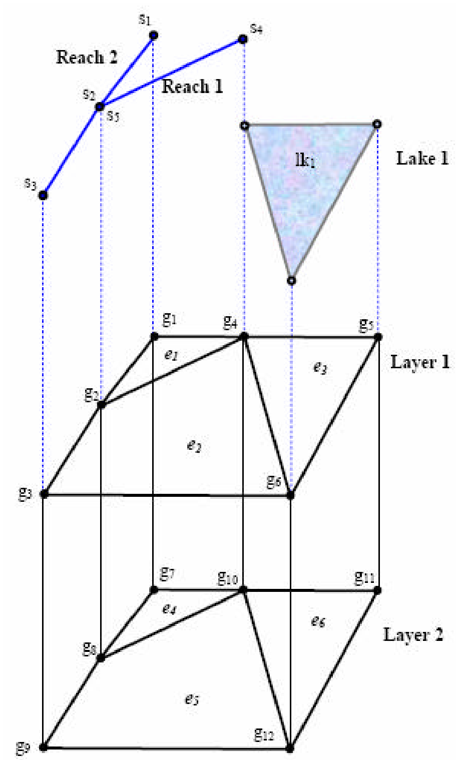


Fig. A.1Element configuration from IWFM theoretical manual

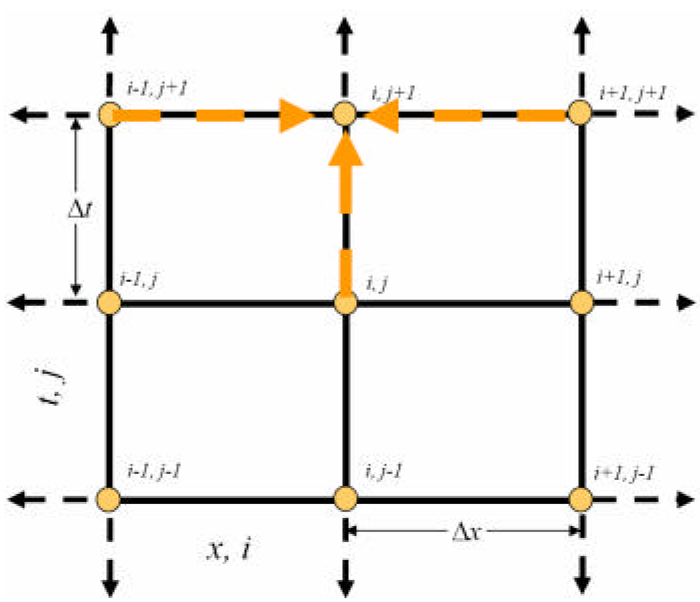


Fig A.2 Finite Differences Implicit Method; from Biosystems Modeling class OSU Prof John Bolte

The size of the elements as mentioned by the IWFM literature should be based on observed or predicted groundwater head gradients throughout the model domain. Therefore, in areas where the flux is large, the size of the elements should be smaller than those located in areas of relatively small flow gradients. Given the convenience of setting a regular grid with same sides length of quadrilateral element, the approach to develop the grid mesh was by using the software Interactive Groundwater version 3.5.8 (IGW3.5.8) as a grid generator over our model domain. Based on the information provided by the well database of the watershed council and the Report of the geology stratigraphy by Kevin Lindsey, a grid size of 329 m was generated producing 416 elements, 31 triangular (to capture the boundary geometry of the basins), and 385 quadrilateral. The total number of nodes are 445 in 4 layers (see stratigraphy geology). It worth mentioning that previous IWFM applications have been using much bigger element sizes making this model an attractive solution where there is not enough information without losing convergence problems. The Milton-Freewater simulation model is one of the first attempts to simulate on such a small scale. However for some of the processes that we are trying to simulate a much smaller grid size will be desired to see the detailed effect; for example the water mountain generated from the infiltration basin after a recharge event. In the future, California Department of Water Resources is evaluating the idea of a grid generator inside IWFM. In this way we could change the grid size and look closer at specific location. For this project the information generated from the model will be enough to evaluate our basic questions formed by the different management scenarios.

Once the grid is generated, our main flow equation is to be solved for each element based on our nodal values. The Garlekin finite element method leads to a set of algebraic equations by applying a weighted function to head from each node also called shape function that depends only on the geometric characteristics of the finite element for example:

$h(x,y,t) = \sum_{j=N} \omega(x,y)\ h_j(t)$

Eq.A.15

where

$h(x,y,t)$ = approximation of head at x,y location at time t

$\omega(x,y)$= shape function

$h_j(t)$= head at time t

N = total number of nodal points

The shape function would vary between a triangular or quadrilateral element. The shape function for a linear triangular element is calculated by a linear interpolation. For example if we want to know the head value at the center of a triangular element from which the distance to any of the three nodes is the same, the head at that point would be the summation of one third of each node given the same weight to each node.

$$h^e(\text{center}) = 1/3h_i + 1/3h_j + 1/3h_k$$

Eq.A.16

For any situation the linear interpolation formula would be

$$h^e(x,y) = \omega_i(x,y)\ h_i + \omega_j(x,y)\ h_j + \omega_k(x,y) h_k$$

Eq.A.17

where;

$$\omega_i(x,y) = \frac{1}{2A}\left[\left(x_i y_k - x_k y_j\right) + \left(y_j - y_k\right)x + \left(x_k - x_j\right)y\right]$$

$$\omega_j(x,y) = \frac{1}{2A}\left[\left(x_k y_i - x_i y_k\right) + \left(y_k - y_i\right)x + \left(x_i - x_k\right)y\right]$$

$$\omega_k(x,y) = \frac{1}{2A}\left[\left(x_i y_j - x_j y_i\right) + \left(y_i - y_j\right)x + \left(x_j - x_i\right)y\right]$$

and

$$A = \frac{1}{2}\left[\left(x_i y_j - x_j y_i\right) + \left(x_k y_i - x_i y_k\right) + \left(x_j y_k - x_k y_j\right)\right]$$

A is the area of the triangle define by ijk and ? are the shape functions.

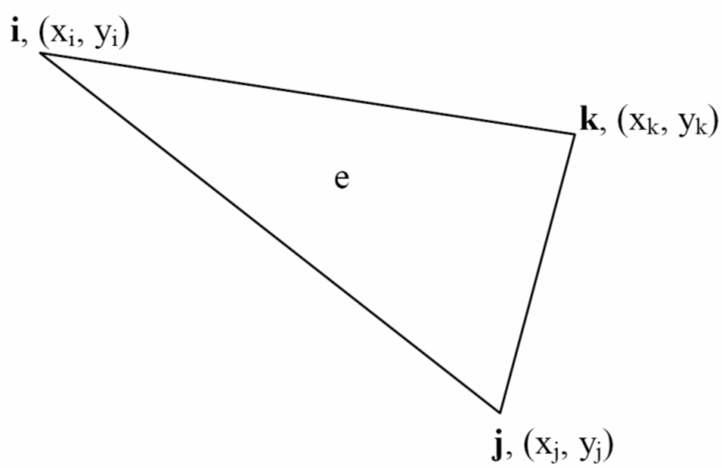


Figure A.3 Triangular element IWFM Manual

For a bilinear quadrilateral element the coordinates (x,y) are to be transformed into (ξ, η) coordinates and by Lagrange polynomials the shape functions are then calculated as:

Eq.A.18

$$\begin{aligned}
\omega_1(\xi,\eta) &= \tfrac{1}{4}\left[(\xi-1)(\eta-1)\right] \\
\omega_2(x,y) &= \tfrac{1}{4}\left[(\xi+1)(-\eta+1)\right] \\
\omega_3(x,y) &= \tfrac{1}{4}\left[(\xi+1)(\eta+1)\right] \\
\omega_4(x,y) &= \tfrac{1}{4}\left[(-\xi+1)(\eta+1)\right]
\end{aligned}$$

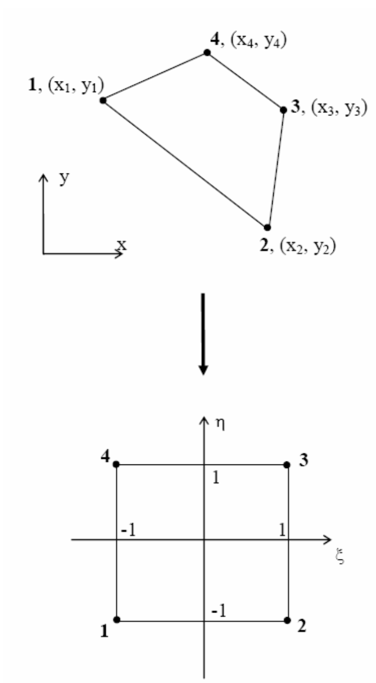


Fig A.4 Example of transformed coordinates

The main flow equation A.14 can then be re-expressed in a finite element notation as

$$
\begin{aligned}
0 = &\iint_{\Omega} \frac{\partial S_i h_i}{\partial t} \varpi_i d\Omega - \iint_{\Gamma} q_{\Gamma} \varpi_i d\Gamma + \sum_{j=N^*(m-1)+1}^{N^*m} \iint_{\Omega} Th_j \left(\frac{\partial \varpi_i}{\partial x} \frac{\partial \varpi_j}{\partial x} + \frac{\partial \varpi_i}{\partial y} \frac{\partial \varpi_j}{\partial y} \right) d\Omega + H(m-2) \iint_{\Omega} L_{i-N} \Delta h_i^{u} \varpi_i d\Omega \\
&+ [1 - H(m - N_L)] \iint_{\Omega} L_{i+N} \Delta h_i^{d} \varpi_i d\Omega - \iint_{\Omega} q_o \varpi_i d\Omega + \iint_{\Omega} q_{sd} \varpi_i d\Omega - \iint_{\Omega} \delta(x - x_s, y - y_s) \frac{Q_{sint}}{A_s} \varpi_i d\Omega \\
&- \iint_{\Omega} \delta(x - x_{lk}, y - y_{lk}) \frac{Q_{lkint}}{A_{lk}} \varpi_i d\Omega - \iint_{\Omega} \delta(x - x_{td}, y - y_{td}) \frac{Q_{td}}{A_{td}} \varpi_i d\Omega
\end{aligned}
$$

Where

Eq.A.19

Γ = boundary of the spatial domain

N = number of aquifers

Ω= model domain

The rest of the variables were describe for equation A.11 and equation A.14

Our new flow equation in finite element terms uses the inflow to the boundary, the second and third integral of the equation, to solve in multilayer aquifer system, setting a matrix of ordinary differential equations of N x NL. The finite difference method is then applied in a fully implicit way to solve in time converting our finite element flow equation into an hybrid with the finite difference method as:

$$0 = \sum_{e=N(m-1)+1}^{N*m} \iint_{\Omega} \frac{S_{Si}^{t+1}(h_i^{t+1} - TOP_i) + S_{Si}^{t}(-h_i^{t} + TOP_i)}{\Delta t} \omega^e{}_i d\Omega^e - \iint_{\Gamma} q^{t+1}{}_{\Gamma^e} \omega^e{}_i d\Gamma^e + \sum_{j=N*(m-1)+1}^{N*m} \iint_{\Omega} T^e h_j \left(\frac{\partial \omega^e{}_i}{\partial x} \frac{\partial \omega^e{}_j}{\partial x} + \frac{\partial \omega^e{}_i}{\partial y} \frac{\partial \omega}{\partial} \right.$$

$$+ H(m-2) \iint_{\Omega} L_{i-N} (\Delta h_i^u)^{t+1} \omega^e{}_i d\Omega^e$$

$$+ [1 - H(m - N_L)] \iint_{\Omega} L_{i+N} (\Delta h_i^d)^{t+1} \omega^e{}_i d\Omega^e - \iint_{\Omega} q^{t+1}{}_o \omega^e{}_i d\Omega^e + \iint_{\Omega} q^{t+1}{}_{sd} \omega^e{}_i d\Omega^e - \iint_{\Omega} \delta(x - x_s, y - y_s) \frac{Q^{t+1}{}_{sin\,i}}{A_s} \omega^e{}_i d\Omega^e$$

$$- \iint_{\Omega} \delta(x - x_{lk}, y - y_{lk}) \frac{Q^{t+1}{}_{lk\,int}}{A_{lk}} \omega^e{}_i d\Omega^e - \iint_{\Omega} \delta(x - x_{sd}, y - y_{sd}) \frac{Q^{t+1}{}_{sd}}{A_{sd}} \omega^e{}_i d\Omega^e$$

where:

Eq.A.20

TOP_i = top elevation of the aquifer at node i

Δt= length of time step

The index e represents the element number that if it is using the node i from which the integrals are defined then it has a non-zero value, other wise it will not be calculated. The only terms that are spatial functions are: ω^e $(T^e)^{t+1}$ and d(x-x_S, y-y_S) , d(x-x_{lk}, y-y_{lk}).

The rest of the terms are either constant over an element or a function of time, assuming that the boundary flow it's already available. Then by simplification, taking the transmissivity $(T^e)^{t+1}$ constant over an element but different from element to element, the shape functions and the dirac d functions of stream and lake locations are the only terms that remain in the integrals (IWFM version 3 Theoretical manual).

$$\iint_{\Omega} \omega^{e}{}_{i}\, d\Omega^{e}$$

$$\iint_{\Omega} \delta(x - x_{s}, y - y_{s})\, \omega^{e}{}_{i}\, d\Omega^{e}$$

$$\iint_{\Omega} \delta(x - x_{lk}, y - y_{lk})\, \omega^{e}{}_{i}\, d\Omega^{e}$$

$$\iint_{\Omega^{e}} \left(\frac{\partial \omega^{e}{}_{i}}{\partial x} \frac{\partial \omega^{e}{}_{j}}{\partial x} + \frac{\partial \omega^{e}{}_{i}}{\partial y} \frac{\partial \omega^{e}{}_{j}}{\partial y} \right) d\Omega^{e}$$

Eq. A21

The solution to these three integrals will vary between triangular and quadrilateral elements. After solving for these equations, IWFM also applies the finite element technique by multiplying the shape functions to the vertical flow, subsidence, stream groundwater interaction, lake-groundwater interaction as well as boundary and initial conditions equation explained in the water budget components. The system of equations produced by the hydrological processes has to be solved simultaneously where the three unknowns at the next time step are groundwater head h^{t+1}, stream surface elevation hs^{t+1} and lake elevation h_{lk}^{t+1}.
This system of equations is represented in a matrix form as:

$$[X]\{H^{t+1}\}+\{F\}=0 \quad \text{Where}$$

H^{t+1} is the vectors of unknowns for each stream node (NR), lakes(NLK), aquifers(N) times the number of aquifer layers (NL), a total system of NR+NLK+(NL*N). The Taylor series expansion is used by the Newton-raphson method in order to linearize the equation.

Appendix B. Aquifer Testing

In cooperation with the Walla Walla Basin Watershed Council four aquifer tests were performed to determine the hydraulic properties of the unconfined aquifer system. Duration, dates and commentaries can be found in Table A.1.

Table A.1 Number, date and duration of aquifer testing

Aquifer test #	Date of operation	Duration (hrs)	Average flow rate (gpm)	commentaries
1	June 22, 2006	9	76	Irrigation Ditch running at 12m from pumping well HBDIC
2	August 28, 2006	30	33	Ditch Off pumping noise
3	February 7, 2007	5	78.5	Ditch Off, no pumps around observation well
4	February 8, 2007	73	82.21	Ditch Off, no pumps around observation well

Location and methodology:

The artificial recharge project ran by the WWBWC in conjunction with Hudson Bay District Improvement Company (HBDIC), has set up 4 observation wells around the infiltration basins to monitor the behavior of the groundwater recharge (more on water budget parameters, lakes). The characteristics of the observation wells are the following:

The Pumping well have a 4 inch diameter installed to a total depth of 67.75 ft (20.65m), screened interval extends from the bottom 50.75 ft (15.5m)

Observation well #2 at 109.35 ft (33.33m) from pumping well; 2 inch diameter installed to a total depth of 60 ft (18.29m) screened from the bottom 45 ft (13.7m)

Observation well #3 at 994 ft (303m) from pumping well; 2 inch diameter installed to a total depth of 71 ft (21.64m) screened from the bottom 55 ft (16.7m)

Observation well #4 at 397 ft (121.3m) from pumping well; 2 inch diameter installed to a total depth of 61 ft (18.5m) screened from the bottom 45 ft (13.7m)

A submergible pump (Make: Berkeley Pump Company, Model #4CM-9) of 3Hp was installed at intake depth of 58ft (17.68m). The pump was supplied by a propane feed Electric motor brand Kohler running at 300Volts.

The pumping well and observation wells were monitor manually and by a pressure transducer every half a minute for the first 10 minutes, then every minute for the next hour, then every 5 minutes for the next two hours and finally every 30 minutes for the rest of the test duration.

For each aquifer test, a step drawdown test was performed to ensure a constant pumping rate without compromising the submergible pump. The step drawdown consisted of pumping at different rates for 30 minutes each and observed the drawdown generated, comparing between the several tests to determine which was the maximum rate possible for the constant pump test.

Pumping test analysis and results

The unconfined gravel aquifer has an enormous variation during the water year due to pumping, turning unlined irrigation canals on and off and artificial recharge from the infiltration basins. Figure A.5 shows the normal variation in water elevation observed in the well used as a pumping well.

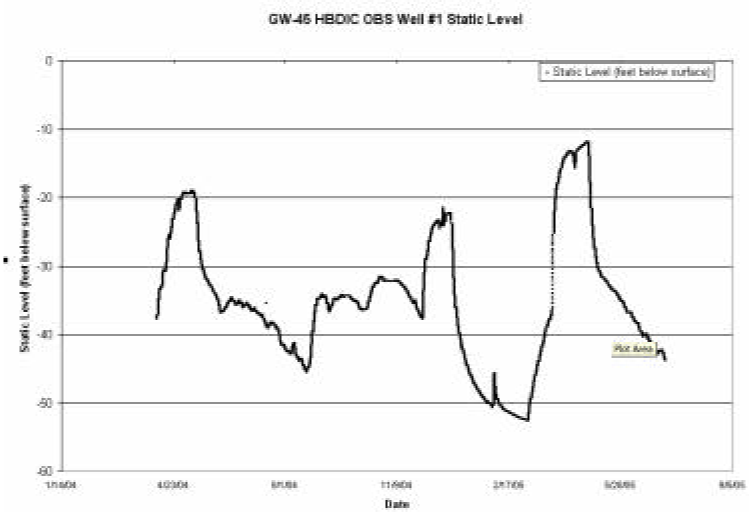


Fig A.5 variation of the water level in observation well next to the infiltration basins.

To capture the real aquifer characteristics from the aquifer testing these variations had to be considered. Three methods where used to consider the aquifer variation, these are the following:

Method 1: Estimate the expected draw down and compare the volume of aquifer displace using radial distance from the well.

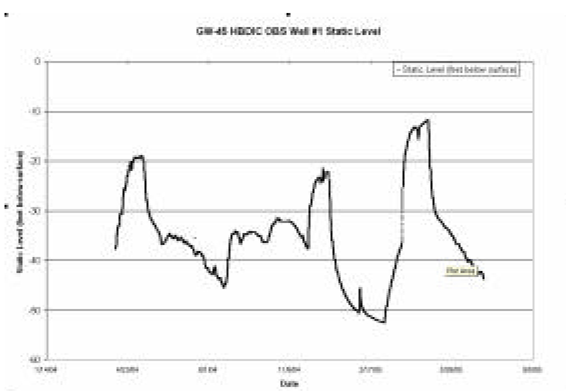
Draw down expected (m) = Known Pumping rate Q(m^3 /min) * Time pump has been run t (min) / Area from pumping well to observation well A(m^2) * porosity of Aquifer.

For example if the pump well has been running for 240 min with a constant flow rate of 16.5 gpm or $6.25*10^{-2}$ m^3/min we expect that an observation well located 33.33 meters from the pumping well would have the following drawdown

Drawdown expected $$\Delta h = \frac{Q*t}{\Pi r^2 n}$$ Eq. A21

Δh =($6.25*10^{-2}$ m^3/min * 480 min)/[(3.1416*($33.33m)^2$ * 0.27) = $3.18*10^{-2}$ m or 1.25 inch

Method 2: Comparing years 2004, 2005 and 2006, we can say that after the infiltration basins stop the operation, the water mound will be stabilize at the same rate. Graphs A.6 and A.7 show the static water level trends after the artificial recharge. This rate can be subtracted to the observed drawdown to get the actual effects of the pumping test.



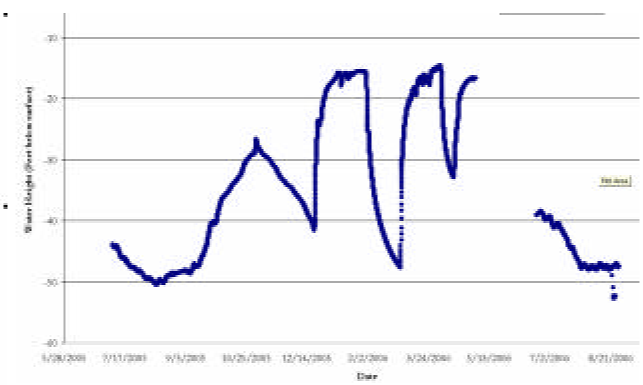


Figure A.6 and A.7 observation wells for years 2004-2005 and 2005-2006 respectively

Method 3: Looking for a fixed boundary. The WWBWC has set up a network of observation wells over the entire model area. Wells that are far enough from the pumping well show the same trends as the observation wells but without the effects of the pumping. This trend can be subtracted from our observation wells to get actual drawdown

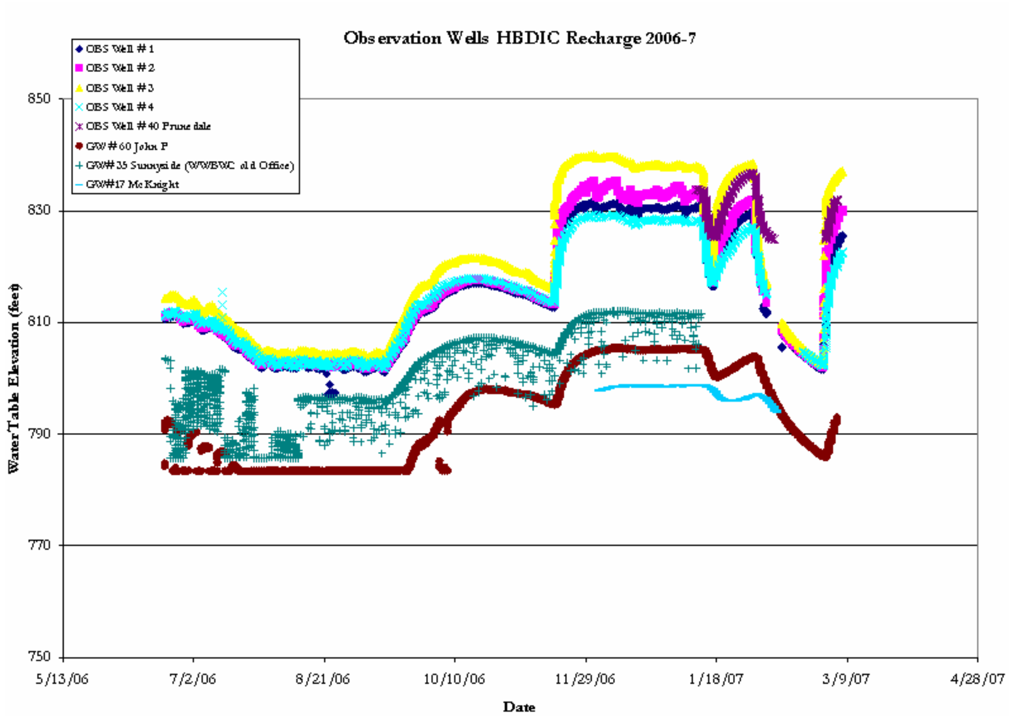


Figure A.8 Water level trends and effects of recharge and irrigation canals farther from the pumping well source WWBWC Bob Bower

The data was analyzed manually and by the Software AQTSOLV for windows Version 4.02 – Professional Developed By Glenn M Duffield, Hydrosolve, Inc. This Software has the capability of testing different methods for confined and unconfined aquifers. The method that best describes our aquifer conditions is the solution derived by Neuman (1974) for unsteady flow to a partial penetrating well in an unconfined aquifer with delayed gravity response. Data from the software manual show that the analytical solution is solved by the following sets of equations:

$$s = \frac{Q}{4\pi T}\int_0^\infty 4yJ_0(y\sqrt{\beta})\left[u_0(y) + \sum_{n=1}^\infty u_n(y)\right]dy$$

$$\beta = \frac{r^2K_z}{b^2K_r}$$

$$\sigma = \frac{S}{S_y}$$

$$t_s = \frac{Tt}{Sr^2}$$

EqA.22

Where

1. b is aquifer thickness [L]
2. K_r is radial hydraulic conductivity [L/T]
3. K_z is vertical hydraulic conductivity [L/T]
4. Q is pumping rate [L^3/T]
5. r is radial distance [L]
6. s is drawdown [L]
7. S is storativity [dimensionless]
8. S_y is specific yield [dimensionless]
9. t is time [T]
10. T is transmissivity [L^2/T]

And for a partial penetrating well the drawdown is found by the following:

$$u_0(y) = \frac{[1-\exp(-t_s\beta(y^2-\gamma_0^2))][\sinh(\gamma_0 z_{2D}) - \sinh(\gamma_0 z_{1D})]}{[y^2+(1+\sigma)\gamma_0^2 - (y^2-\gamma_0^2)^2/\sigma]\cosh(\gamma_0)} \cdot \frac{\sinh(\gamma_0(1-d_D)) - \sinh(\gamma_0(1-l_D))}{(z_{2D}-z_{1D})(l_D-d_D)\sinh(\gamma_0)}$$

$$u_n(y) = \frac{[1-\exp(-t_s\beta(y^2+\gamma_n^2))][\sin(\gamma_0 z_{2D}) - \sin(\gamma_0 z_{1D})]}{[y^2-(1+\sigma)\gamma_n^2 - (y^2+\gamma_n^2)^2/\sigma]\cos(\gamma_n)} \cdot \frac{\sin(\gamma_n(1-d_D)) - \sin(\gamma_n(1-l_D))}{(z_{2D}-z_{1D})(l_D-d_D)\sin(\gamma_n)}$$

The gamma terms are the roots of the following equations:

$$\sigma\gamma_0 \sinh(\gamma_0) - (y^2-\gamma_0^2)\cosh(\gamma_0) = 0 \qquad \gamma_0^2 < y^2$$

$$\sigma\gamma_n \sin(\gamma_n) + (y^2+\gamma_n^2)\cos(\gamma_n) = 0 \qquad (2n-1)(\pi/2) < \gamma_n < n\pi \quad n \geq 1$$

Eq. A.23

where

1. d_D is dimensionless depth to top of pumping well screen (d/b)
2. J_0 is Bessel function of first kind, zero order
3. l_D is dimensionless depth to bottom of pumping well screen (l/b)
4. z_D is dimensionless elevation of piezometer opening above base of aquifer (z/b)
5. z_{1D} is dimensionless elevation of bottom of observation well screen above base of aquifer (z_1/b)
6. z_{2D} is dimensionless elevation of top of observation well screen above base of aquifer (z_2/b)

The assumptions for this model solution are:

1. aquifer has a infinite real extent
2. aquifer is homogeneous and has uniform thickness
3. aquifer potentiometric surface is initially horizontal
4. pumping well is fully or partially penetrating
5. aquifer is unconfined with delayed gravity response
6. flow is unsteady
7. diameter of pumping well is very small so that storage in the well can be neglected

Results for the first aquifer test (06/20/06)

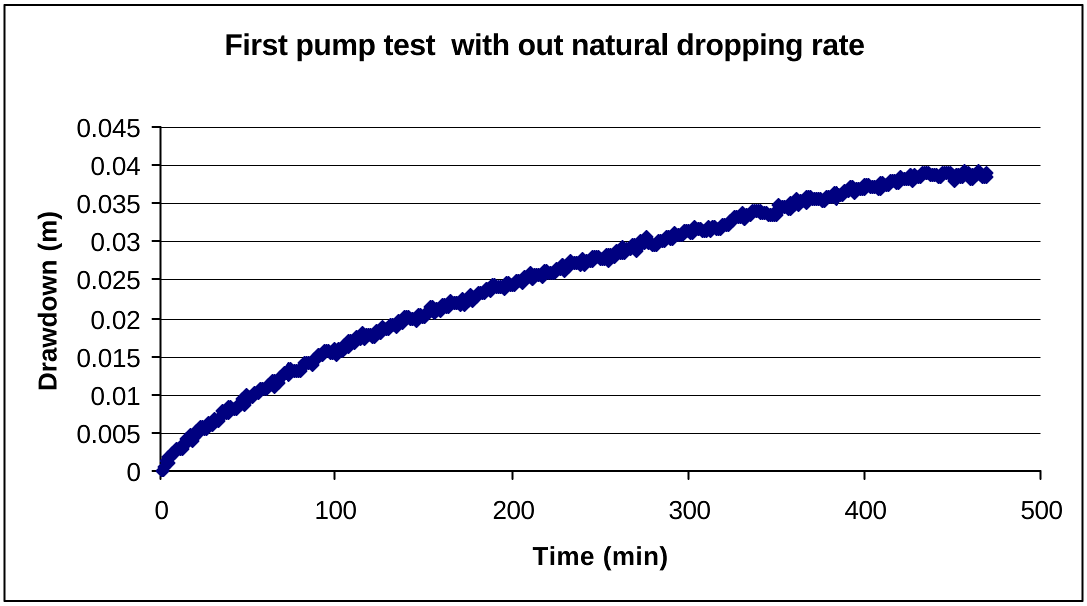


Figure A 9 Drawdown from the first pump test

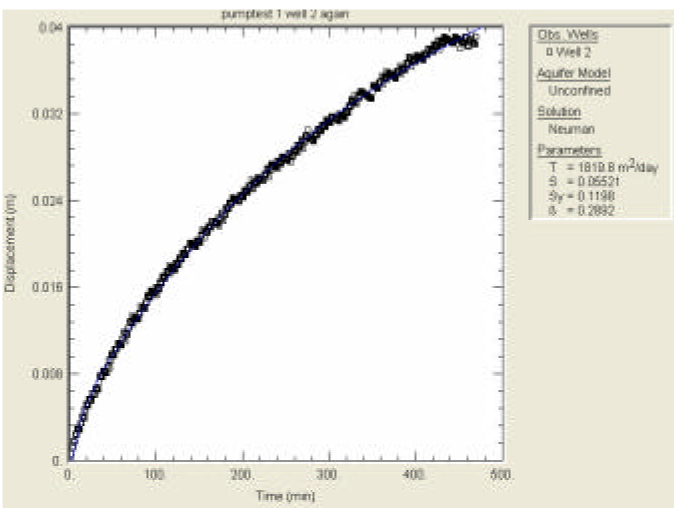


Figure A 10 Results from AQTESOLVPRO

Table A2. Results from the first pump test

Method	Transmisivity	Hydraulic conductivity m/day
Theis	1953.3	36.17
Neuman	1819.8	33.70
Moench	1819.9	33.70

Results for the second pump test (08/28/06):

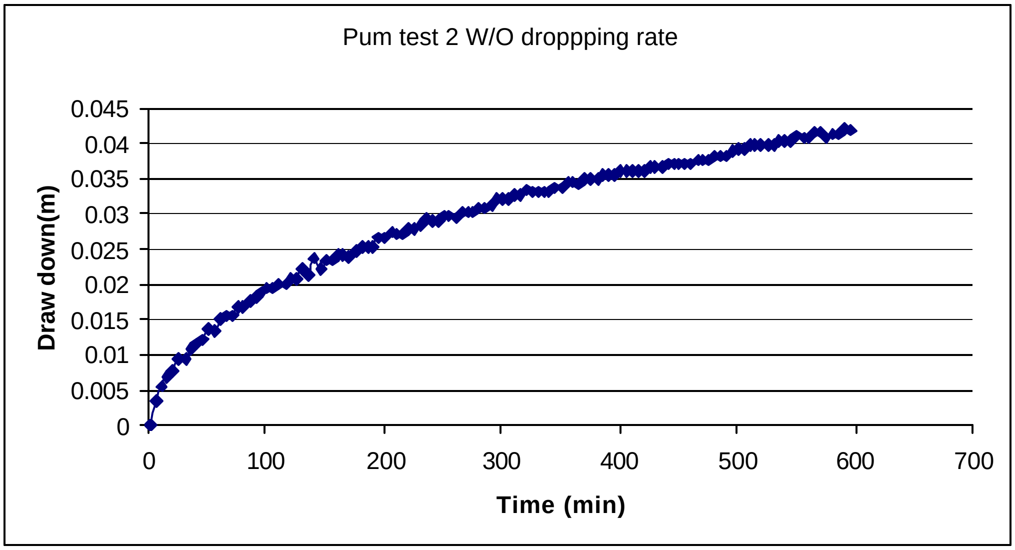


Fig. A11 Results from the 2nd aquifer test

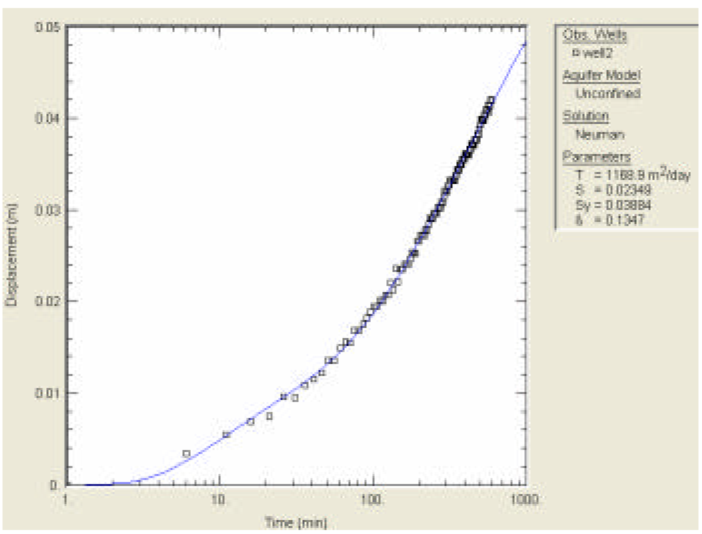


Figure A 12 Results from AQTESOLVPRO 2nd aquifer test

Table A.3 Results from the second aquifer test

Method	Transmisivity	Hydraulic conductivity m/day
Theis	1099.3	20.36
Neuman	1168.9	21.65
Moench	1169.1	21.65

Results for the third pump test (02/07/07):

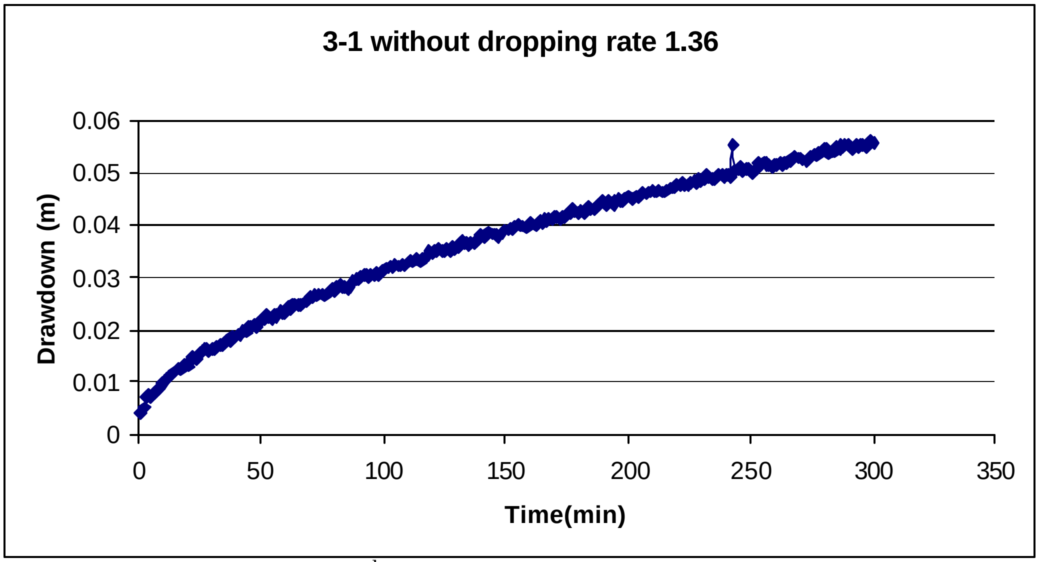


Fig. A13 Results from the 3rd aquifer test

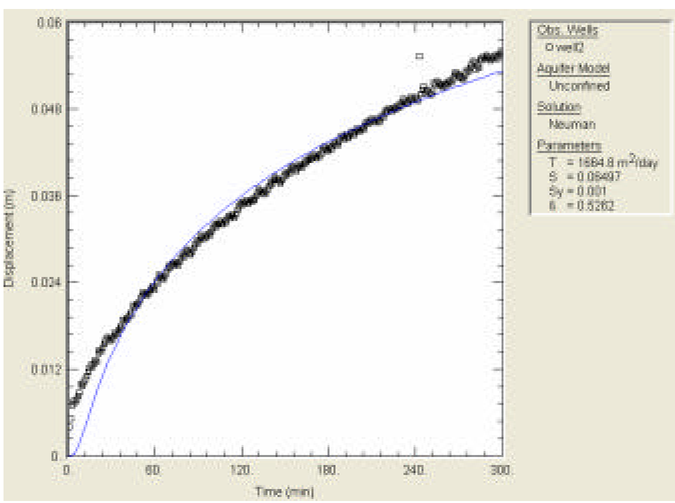


Figure A 14 Results from AQTESOLVPRO 3nd aquifer test

Table A.4 Results from the second aquifer test

Method	Transmisivity	Hydraulic conductivity m/day
Theis	1720.9	31.87
Neuman	1664.8	30.83
Moench	1664.8	30.83

Results for the forth pump test (02/08/07):

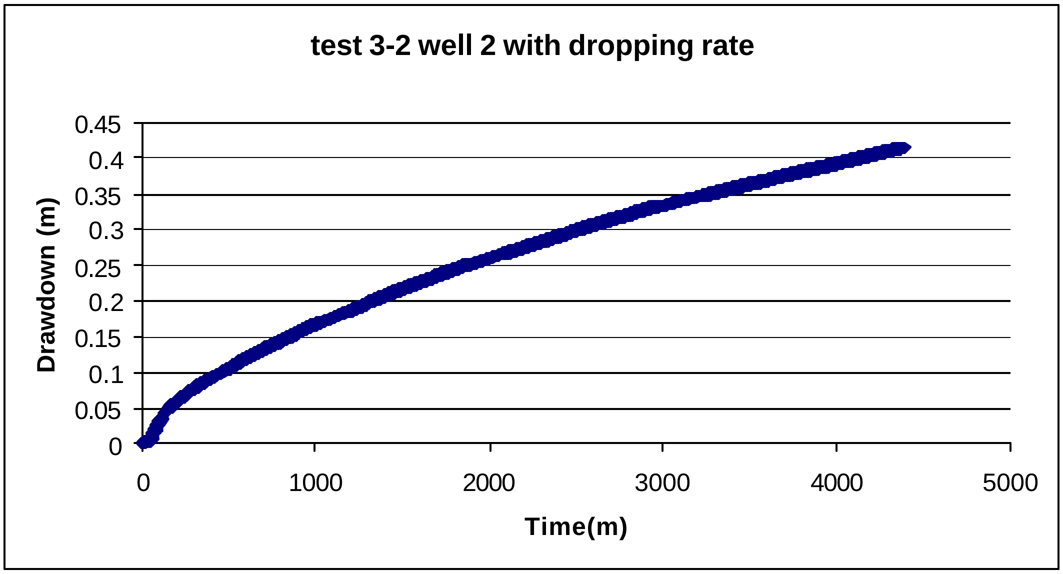


Fig. A15 Results from the 3-2 aquifer test

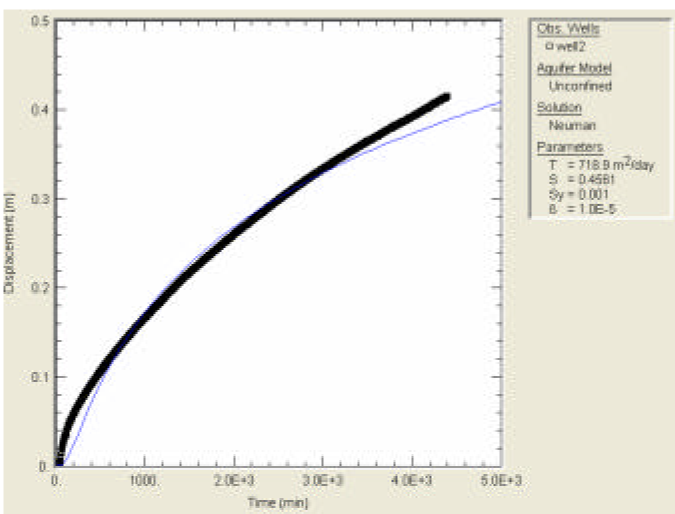


Fig A.16 Results from AQTESOLVPRO aquifer test # 3-2

Table A5 Results from the aquifer test #3-2

Method	Transmisivity	Hydraulic conductivi ty m/day
Theis	430.4	7.97
Neuman	718	13.30
Moench	719	13.31

Conclusions and final recommendations Aquifer testing

Considering the noise or natural variation in the water table was achieved by comparing the rate at which the aquifer was dropping in far away wells (method 3), and then comparing this rate to the observation well, verifying the rate by the simple calculation showed in method number 1. For the third and fourth pump tests, extra information was available by comparing previous years and events of the infiltration basins for artificial aquifer recharge. Using the software AQSOLVE it was possible to test several solutions and match the best model available. The Neuman (1974) solution was the best for all the pump tests. The best results were found in the first aquifer test where there were not other pumps around, and the aquifer was fairly stable. The irrigation canal that was running at a distance of 12m from our pumping well didn't affect our results since the water table was originally 14 meters below ground level. Aquifer testing # 2 has the most variability, the irrigation canal is off and most of the water for irrigation comes from wells around the testing site. Aquifer test # 3 gives very similar results to the first aquifer test, the water level at the observation wells were in the same range. For aquifer test number 4 the water level was at a much lower elevation than test number 1 and 3, the water drawn by the pump test came from a slower more compact part of the aquifer, the hydraulic conductivity decreases with depth from the ground surface. The 4 aquifer testing here presented are the only reported tests perform to the gravel unconfined aquifer; values for hydraulic conductivity that have been reported have been made by modeling efforts or geology descriptions, these are:

Barker and Mac Nish 1976 Gravel Aquifer model pag 29 gives the following ranges 4m/day to 20 m/day, 20m/day to 40 m/day, 40m/day to 65.84 m/day

Kevin Lindsey test site geology, hydrogeology and water 3.6 to 183 m/day

Golder associates, City of Walla Walla extended area groundwater flow model. Reported 15.24m/day for most of the area and 30.48m/day for areas next to streams

Literature values show similar results; Bear 1972 for unconsolidated sand & gravel show values in the range of 10^1 m/day and EPA1986 for a coarse sand to coarse gravel with moderate silt content is 30m/day, high silt content 27m/day.

We expect a variability of the aquifer properties over the model area, extra aquifer testing are proposed for different locations. Given the model sensitivity to this parameter a more detail analysis could improve future modeling work. The range of values for Hydraulic conductivity used in this model are 22m/day to 34 m/day more information on aquifer parameters can be found in section 9 parameter used in the model.

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Appendix C: CD with Milton-Freewater Hydrologic Model and scenarios files of IWFM, and model outputs. Database in excel with all the information input in to the model.