REPORT OF THE WILLIMANTIC RIVER STUDY

AN ANALYSIS OF THE IMPACT OF THE UNIVERSITY OF CONNECTICUT WATER SUPPLY WELLS ON THE FISHERIES HABITAT OF THE WILLIMANTIC RIVER

WILLIMANTIC RIVER WELLFIELD
MANSFIELD DEPOT, CONNECTICUT

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Prepared for:

University of Connecticut
Office of Environmental Policy
31 LeDoyt Road, U-3055
Storrs, CT 06269

Prepared by:

MILONE & MACBROOM, INC.
99 Realty Drive
Cheshire, Connecticut 06410
(203) 271-1773
www.miloneandmacbroom.com

In association with:

Environmental Research & Consulting, Inc.
126 Bancroft Road
Kennett Square, PA 19348
(610) 444-2875

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ACKNOWLEDGEMENTS

This report could not have been completed without the significant efforts and contributions of many people and organizations. The Technical Advisory Group (TAG) provided valuable input and guided this project. Members of the TAG included the following:

- Dr. Amvrossios Bagtzoglou, University of Connecticut, Department of Civil & Environmental Engineering
- Mr. Thomas Callahan, University of Connecticut, former Vice President
- Mr. Jason Coite, University of Connecticut Office of Environmental Policy
- Ms. Virginia de Lima, United States Geological Survey
- Ms. Corrine Fitting, Connecticut Department of Environmental Protection, Aquifer Protection Program
- Ms. Lori Mathieu, Connecticut Department of Public Health, Drinking Water Section
- Mr. Daniel Meade, retired from Department of Environmental Protection
- Mr. Richard Miller, University of Connecticut Office of Environmental Policy
- Mr. Robert Miller, Eastern Highlands Health District
- Mr. Brian Murphy, Connecticut Department of Environmental Protection (DEP), Fisheries Division
- Mr. Michael Pacholski, University of Connecticut, Facilities Department
- Mr. Gregory Padick, Town of Mansfield Planning Department
- Mr. Peter Pezanko, The Connecticut Water Company
- Ms. Meg Reich, Willimantic River Alliance
- Mr. Timothy Tussing, University of Connecticut, Facilities Department
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The consultant team consisted of Milone & MacBroom, Inc. (MMI) with Environmental Research and Consulting (ERC) as a subconsultant. David Murphy, P.E. (MMI), Harold M. Brundage III (ERC), and Scott J. Bighinatti (MMI) conducted the bulk of data collection, analysis, modeling, and report drafting. James G. MacBroom, P.E. and Jeanine Armstrong Gouin, P.E. of MMI provided technical and managerial oversight. Shawn Goulet, Garrett Cavaiuolo, Edward Belinsky, and James Murac of MMI assisted with field data collection and survey.

EXECUTIVE SUMMARY

The University of Connecticut (the University) owns and operates two public water supply wellfields known as the Willimantic River Wellfield and the Fenton River Wellfield for provision of public water in the Storrs area of Mansfield, Connecticut. Ongoing concerns over the relationship between wellfield operations and instream flow diminution have led the University to study the two rivers associated with the wellfields. The subject study originated in the November 6, 2006 *Memorandum of Agreement* with the Connecticut Water Planning Council in which the University agreed to conduct a study to determine whether and, if so, how water withdrawals from the Willimantic River Wellfield affect the aquatic habitat of the Willimantic River in the vicinity of the wellfield.

The Willimantic River Wellfield is located upstream (north) of Route 44 and downstream of Merrow Road. The four Willimantic River Wellfield wells are registered with the Connecticut Department of Environmental Protection for a maximum combined withdrawal of 2.3077 millions of gallons per day (3.57 cubic feet per second).

The specific objectives of this study were to:

- Develop relationships between instream flow and habitat in the Willimantic River for selected fish species
- Derive the relation between the magnitude and timing of ground water withdrawals on the stage and flow of water in the Willimantic River from Merrow Road to Mansfield Depot using existing data, new data collection, and mathematical simulation modeling
- Numerically model selected water-management scenarios to optimize water withdrawals while minimizing adverse impacts on river flow and instream habitat

The Willimantic River drainage basin encompasses approximately 225 square miles in Connecticut and a small part of Massachusetts. Discharge in the Willimantic River is affected upstream of the Willimantic River Wellfield by several registered and permitted diversions for public water supply and industry in addition to impoundments utilized for recreation and fire protection. The geology of the watershed in the vicinity of the wellfield was studied in depth by the University and the United States Geological Survey (USGS) in the late 1960s, and further research was performed at the wellfield during the Level A Aquifer Mapping fieldwork performed in the 1990s.

The Instream Flow Incremental Method (IFIM) was used to evaluate the potential effects of reductions in river flow associated with withdrawal of water at the Willimantic River Wellfield on the habitats of representative fish species in the Willimantic River. Target fish species included brook trout, brown trout, fallfish, and common shiner.

Simulation of river hydraulics and aquatic habitat was performed using computer models collectively known as Physical Habitat Simulation (PHABSIM). The hydraulic simulation models of PHABSIM are used to predict changes in depth, velocity, and wetted area at various river flows. The aquatic habitat simulation models generate a composite suitability function collectively referred to as Habitat Suitability Criteria (HSC) derived from curves representing the depth, velocity, and substrate preferences of selected target species/life stages. The aquatic habitat simulation models integrate the output of the hydraulic simulation models with the HSC to yield an estimate of habitat usability called weighted usable area (WUA).

Field data collection for the IFIM spanned 2008 and 2009. Aquatic habitat was mapped to determine the percentage of all significant mesohabitat types in the study area. Nine representative reaches of the significant mesohabitats were selected based on the aquatic habitat mapping, with representative transects selected within those reaches. Velocity, depth, substrate, cover, bed elevations, and water surface elevations were surveyed at each transect during five calibration discharges.

The USGS has operated a long-term real-time gaging station on the Willimantic River in Coventry since 1931. Flow statistics from this site have been published by the USGS. The 99% duration discharge of the Willimantic River (approximately equivalent to the 7Q10 discharge) is estimated to be 11 cubic feet per second at the subject wellfield. The published mean daily discharge values were modified to represent discharge at the Willimantic River Wellfield by correcting for water supply withdrawals, wastewater discharges, and drainage basin area. The lowest recorded mean daily discharge at the wellfield since 1958 is believed to be approximately 6.0 cubic feet per second during the prolonged drought of August 1999.

The PHABSIM output provided relationships between WUA and discharge for each target fish species. The mean daily streamflow dataset calculated for the wellfield and the WUA to discharge relationships for each target species were then used to perform habitat time-series and Uniform Continuous Under-Threshold (UCUT) analyses. These analyses evaluated the magnitude, frequency, and duration of various discharge-related habitat events for the target species. The results of the UCUT analysis are summarized in Table ES-1.

Table ES-1
Percent of Maximum WUA, Discharge, and Persistent Duration of Common, Critical, Rare, and Extreme Habitat Thresholds for Target Fish Community

Habitat Stressor Threshold	Parameter	Result
Common (Upper Subregion)	Habitat (% Max WUA) Discharge (cfs) Persistent Duration (days)	44% 27 19
Common (Lower Subregion)	Habitat (% Max WUA) Discharge (cfs) Persistent Duration (days)	34% to 49% 19 19
Critical	Habitat (% Max WUA) Discharge (cfs) Persistent Duration (days)	28% 15 13
Rare	Habitat (% Max WUA) Discharge (cfs) Persistent Duration (days)	24% 12 12
Extreme	Habitat (% Max WUA) Discharge (cfs) Persistent Duration (days)	19% 7.8 7

Note: cfs = cubic feet per second

The recommendations of this study are aimed at reducing demand through the use of conservation measures rather than setting specific production cutbacks. The results of the UCUT analyses were tied to the draft drought response plan of the University of Connecticut Water Supply Emergency Contingency Plan as shown in Table ES-2. The time lapse between each trigger level was found historically to be approximately four to six days. Future efforts will formally link these trigger thresholds to appropriate response and recovery guidelines.

Table ES-2
Recommended Willimantic River Drought Trigger Levels and Corresponding Drought Management Response

<table>
<tr><th>Drought Response Stage</th><th>Willimantic River at Wellfield Trigger Discharge</th><th>Examples of Conservation Measures</th></tr>
<tr><td>Prepare for implementation of Stage IA</td><td>Discharge ≤ 27 cfs</td><td>None / Preparation for Stage IA</td></tr>
<tr><td rowspan="2">Stage IA
(Two potential triggers)</td><td>Discharge < 27 cfs for 19 or more days</td><td rowspan="3">Voluntary: Shorter showers, condensed washing loads, elimination of nonessential consumption, raise thermostats on centrally chilled buildings</td></tr>
<tr><td>Discharge < 19 cfs</td></tr>
<tr><td>Stage IB</td><td>Discharge < 15 cfs</td></tr>
<tr><td rowspan="2">Stage II
(Two potential triggers)</td><td>Discharge < 15 cfs for 13 or more days</td><td rowspan="5">Voluntary items above become mandatory and include (but are not limited to) the following mandatory items: No flushing of hydrants, pipes, or sewer lines; no vehicle fleet washing; no use of water for street sweeping; reduce irrigation by 50%; reduce operation of research equipment cooled with domestic water; import water needed for construction dust control; no pool filling; raise thermostats of centrally chilled buildings</td></tr>
<tr><td>Discharge < 12 cfs</td></tr>
<tr><td rowspan="2">Stage III
(Two potential triggers)</td><td>Discharge < 12 cfs for 12 or more days</td></tr>
<tr><td>Discharge < 7.8 cfs</td></tr>
<tr><td>Stage IV</td><td>Discharge < 7.8 cfs for 7 or more days</td></tr>
</table>

A hydrogeologic study was performed to evaluate the effects of sustained pumping on the aquifer under various river discharges. The objective was to collect data during three different combinations of river flow regime (low to moderate, low to moderate, and low) and wellfield operation (low, moderate, and high). Each monitoring event consisted of a 72-hour constant-rate pumping test.

Data collection included water levels measured at existing monitoring wells and at 12 piezometers installed for the study and temperature monitoring at each piezometer and along the thalweg of the river. In addition, river flow was measured consistent with USGS methods at locations upstream of, downstream of, and at the USGS gaging station at the wellfield in order to determine if direct impacts to river discharge could be detected. Automatic dataloggers were used to assist with data collection and were installed in one monitoring well and in four of the piezometers.

The drawdown of ground water due to the Willimantic River wells can cause the ground water table in the vicinity of the Willimantic River to fall below the river water surface and, in some locations, the riverbed. In these cases, water will infiltrate from the riverbed into the ground water system (i.e., induced infiltration). The piezometer and temperature data provided an estimate of the area of influence of the wellfield, which is believed to extend from slightly south of the wellfield and along the stratified drift aquifer to the northwest into Coventry.

The Willimantic River in the vicinity of the Willimantic River Wellfield is a complex system that naturally has gaining and losing reaches due to the surrounding geology. A numerical model was originally constructed using the USGS program MODFLOW-2000 for the vicinity of the Willimantic River Wellfield during the Level A Aquifer Protection Area Study. The Level A model was updated in this study to more precisely model the Willimantic River and its interactions with the underlying aquifer. A pumping test conducted in 1999 and the three monitoring events performed during the hydrogeologic study herein were used to calibrate and verify the updated model.

The updated numerical model was used to simulate the timing and magnitude of pumping on the stage and discharge in the Willimantic River under various management scenarios. First, the four existing production wells and eight theoretical production well locations within the model area were simulated to determine the timing of pumping impacts. The model output suggests that the Willimantic River will have a slightly delayed response to pumping with reductions of discharge in the Willimantic River occurring as soon as nine hours after pumping begins for wells close to the river.

The existing wells and several of the theoretical wells were then simulated under 11 pumping management scenarios to determine if withdrawals can be managed to minimize adverse habitat impacts while meeting water supply demands. The model output for the management scenarios suggests that while there are combinations of wellfield withdrawals that will provide lower impact overall to instream flow through the model area the difference in river flow reduction between the existing wellfield operation and the best modeled condition is a reduction of only 0.31 cubic feet per second. It is believed that water conservation measures are more cost effective than constructing and permitting new water supply wells.

The formal recommendations of this study are divided into Demand-Based Water Conservation recommendations and Supply Management recommendations. Recommendations for Demand-Based Water Conservation include:

1. Incorporate the trigger discharges into the Drought Response Plan. Discharges measured by the USGS at the Merrow Road gaging station will be used to determine when triggers are met. The precise methodology that the University will use to activate and deactivate conservation measures will be determined outside of this study, such as in the proposed

Willimantic River Wellfield – Fenton River Wellfield Management Plan. These triggers should be revisited as appropriate when changes in supply occur.

2. Incorporate mandatory conservation measures for both on- and off-campus users, including residential, municipal, and commercial customers; and Connecticut Department of Corrections facilities.

Recommendations for Supply Management include:

1. Develop a combined Willimantic River Wellfield – Fenton River Wellfield Management Plan to manage the University's water supplies. This document should include a discussion of how the University will correlate upstream discharges to the discharge triggers for protection of fisheries habitat, a formal update to the Drought Response Plan, authorization for limited but occasional use of the Fenton River Wellfield when it would otherwise be shut down, and available supply versus system demand calculations on a monthly basis throughout the calendar year.

2. Complete the design and construction of the Reclaimed Water Facility.

3. After the Reclaimed Water Facility is operational, the University should ensure that the increment of water freed from nonpotable usage (central utility plant and athletic fields) will be partially allocated to instream needs as well as new potable demands that may arise in the future in an equitable manner.

4. Consider future ground water supplies downstream of the Willimantic River Wellfield in a location where instream flows would be higher than they are at the existing wellfield, and/or fish habitats would be less sensitive to flow reductions. If a new supply were to be developed, the most logical use relative to protection of instream flows in the Willimantic River would be to utilize the new source(s) to reduce stress on the Willimantic River habitat near the Willimantic River Wellfield.

5. Pursue interconnections with the Connecticut Water Company's Northern Region/Western System and Windham Water Works, which the University could utilize for supply during drought periods.

6. Consider provision of short-term or pulsed releases from the Staffordville Reservoir, Crystal Lake, and/or State Line Pond. This will require cooperation with the dam owners and the parties that control the impoundments and the dam outlet works.

1.0 INTRODUCTION

1.1 Background

The University of Connecticut (the University) withdraws water from two stratified drift wellfields in the town of Mansfield, Connecticut. These are known as the Fenton River Wellfield (located to the east of campus along the Fenton River) and the Willimantic River Wellfield (located to the west of campus along the Willimantic River). The four Willimantic River Wellfield wells are registered with the Connecticut Department of Environmental Protection (DEP) for a maximum withdrawal rate of 2.3077 million gallons per day (mgd). Both wellfields are integral sources of supply for the University of Connecticut, which also provides water to portions of the town of Mansfield.

As a result of ongoing concern about the environmental impacts of withdrawing water from the Fenton River Wellfield and in conjunction with the Environmental Impact Evaluation of the North Campus Master Plan, the Fenton River and its stratified drift aquifer have been extensively studied. The University's "Fenton River Study" was published in 2006 with the formal name *Long-Term Impact Analysis of the University of Connecticut's Fenton River Water Supply Wells on the Habitat of the Fenton River*. The study was conducted to determine whether and how water withdrawals from the Fenton River Wellfield affect the fisheries habitat of the Fenton River adjacent to the wellfield.

With a better understanding of the aquifer processes in the Fenton River and the impacts of ground water withdrawals, attention has turned to the Willimantic River aquifer and associated wellfield. As set forth in the November 6, 2006 *Memorandum of Agreement* with the Connecticut Water Planning Council, the University agreed to conduct a study to determine whether and, if so, how water withdrawals from the Willimantic River Wellfield affect the aquatic habitat of the Willimantic River in the vicinity of the wellfield.

Shortly thereafter, the University commissioned a *Water and Wastewater Master Plan* prepared by Milone & MacBroom, Inc. (MMI) under direction from the Connecticut Department of Public Health (DPH). The *Master Plan* evaluated various water management scenarios that included successive cutbacks on Fenton River Wellfield withdrawals, demonstrating the importance of utilizing the Willimantic River Wellfield as a public water supply. While it had long been believed that the aquifer along the Willimantic River could sustain greater withdrawals with less environmental impacts as compared to the Fenton River aquifer, the potential impacts to the habitat of the Willimantic River had not yet been quantified.

1.2 Study Objectives

Similar to the Fenton River Study, the specific objectives of this study were to:

- Develop relationships between instream flow and habitat in the Willimantic River for selected fish species
- Derive the relation between the magnitude and timing of ground water withdrawals on the stage and flow of water in the Willimantic River from Merrow Road to Mansfield Depot using existing data, new data collection, and mathematical simulation modeling
- Numerically model selected water-management scenarios to optimize water withdrawals while minimizing adverse impacts on streamflow and instream habitat

This report is organized into the following sections:

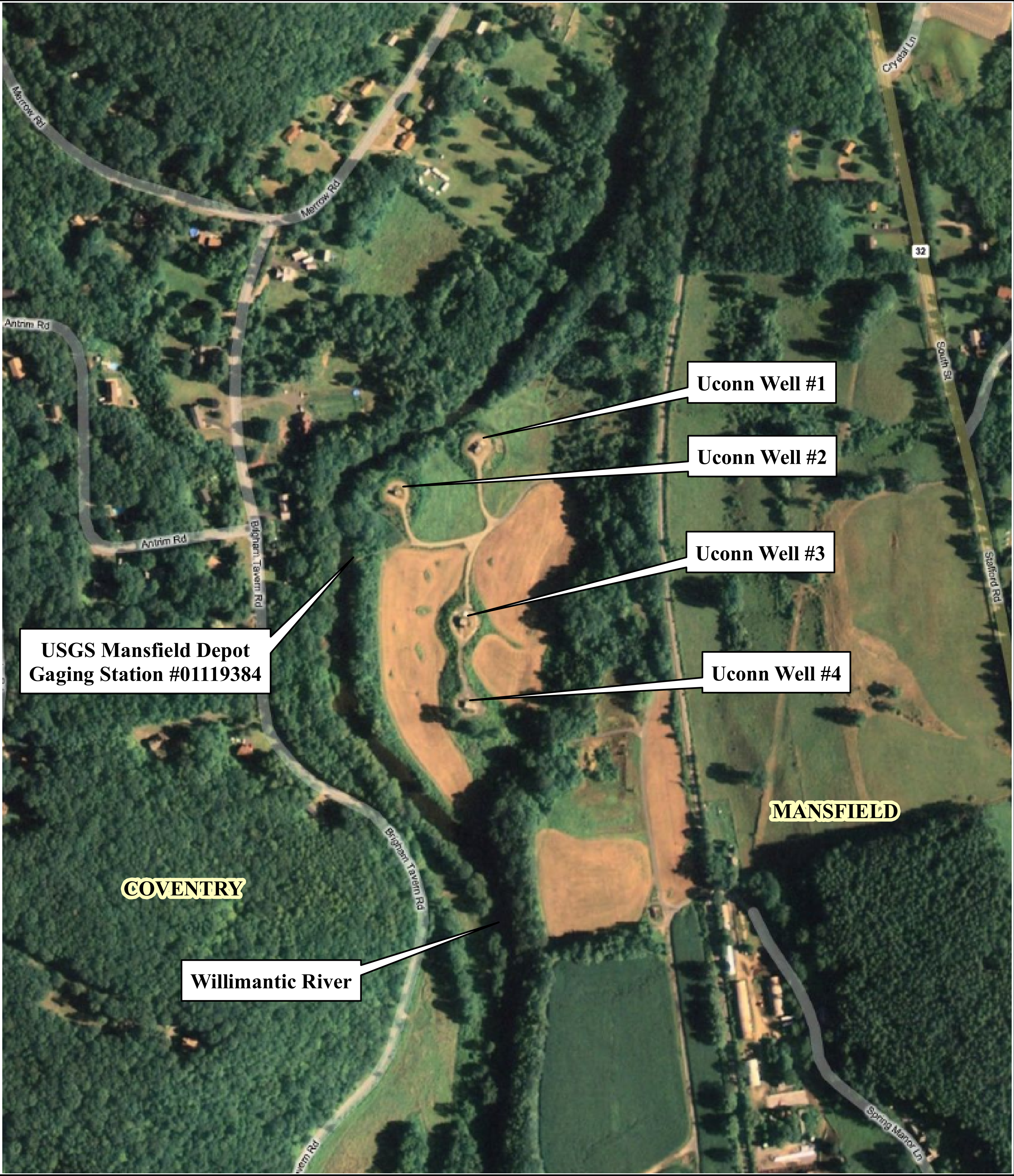
- The remainder of Section 1 describes the University's Willimantic River Wellfield.
- Section 2 discusses the known hydrology and hydrogeology of the Willimantic River aquifer in the study area.
- Section 3 describes in detail previous studies performed to quantify or qualify the aquifer conditions at the wellfield.

- Section 4 describes the methods and results of the Instream Flow Study related to fisheries habitat.
- Section 5 describes the supplemental hydrogeologic monitoring conducted to further quantify the performance of the Willimantic River Wellfield aquifer.
- Section 6 describes the update and recalibration of the Level A model for use in predictive simulations and pumping management scenarios as described in Section 7.
- Sections 8 and 9 discuss the conclusions and recommendations of the study.
- Section 10 contains a list of references utilized in the completion of this study.
- In addition, several appendices are included that provide further details of the methods and some of the data collected during this study.

This study will ultimately be used in conjunction with the results of the Fenton River Study to develop an overall wellfield management plan for the University. The proposed management plan will set forth guidance for conjunctive use of supplies as well as storage facilities and conservation methods in order to reduce adverse impacts to the Fenton and Willimantic Rivers.

1.3 Willimantic River Wellfield

The Willimantic River Wellfield is located within a large tract of state-owned lands that are located north of Route 44, south of Merrow Road, and west of Route 32 in Mansfield, Connecticut (Figure 1-1). The wellfield currently consists of four active stratified drift wells (UConn Wells #1 through 4) installed between 1958 and 1998. A more complete description of the historical and present production wells at the Willimantic River Wellfield is presented in Section 3. The wells are summarized below and in Table 1-1:



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Site Location Map
Willimantic River Wellfield

LOCATION: Mansfield, CT

MMI#: 1958-09
MXD: H:\Figure1-1.mxd
SOURCE: Microsoft Virtual Earth

N

Willimantic River Study

Map By: SJB
Date: Mar. 2009
Scale: 1"=400'

SHEET: Figure 1-1

- UConn Well #1 was installed in 1970 by the University to supplement existing supplies. The well is a 30-inch by 16-inch gravel-packed well that is 69 feet deep, with 20 feet of screen.
- UConn Well #2 was installed in 1974 by the University to supplement existing supplies. The well is a 24-inch by 14-inch gravel-packed well that is 68 feet deep with 15 feet of screen. UConn Well #3 (formerly Mansfield Training School Well #3) was installed in 1958 to replace a dug well in use since approximately 1913. The well is a 24-inch by 18-inch gravel-packed well that is 68 feet deep with 20 feet of screen.
- UConn Well #4 was installed in 1998 to replace Mansfield Training School Well #2, which was originally installed in 1948. The well is a 20-inch by 12-inch gravel-packed well that is 53 feet deep with 15 feet of screen.

Table 1-1
Summary of Well Specifications[1]

Well	Year Installed	Registration (mgd)	Dimensions	Depth[2] (ft)	Length of Screen (ft)	Well Yield (gpm)	Design Pump Capacity
UConn #1	1970	0.648	30" x 16"	69	20	750	400 gpm @ 555' TDH
UConn #2	1974	0.432	24" x 14"	68	15	361	210 gpm @ 520' TDH
UConn #3	1958	0.648	24" x 18"	68	20	725	400 gpm @ 555' TDH
UConn #4	1998	0.720	20" x 12"	53	15	560	540 gpm @ 484' TDH

Notes: 1. Well yields and design pump capacity taken from 2004 Water Supply Plan.
2. UConn Well #4 is 63 feet deep, but the bottom of the screen is at 53 feet.

Note that while the sum of the individual registrations is equal to 2.448 mgd the registered withdrawal for the Willimantic River Wellfield is only 2.3077 mgd. This is because UConn Wells #1 through #3 were registered by the University in 1982 while Mansfield Training School registered UConn Well #4 (then Mansfield Training School Well #2). The original registration for the first three wells was capped at a combined withdrawal of 1.5877 mgd. When Mansfield Training School Well #2 was transferred to the University in the early 1990s, the entire registration (0.720 mgd) was transferred, and the combined withdrawal increased by the same amount.

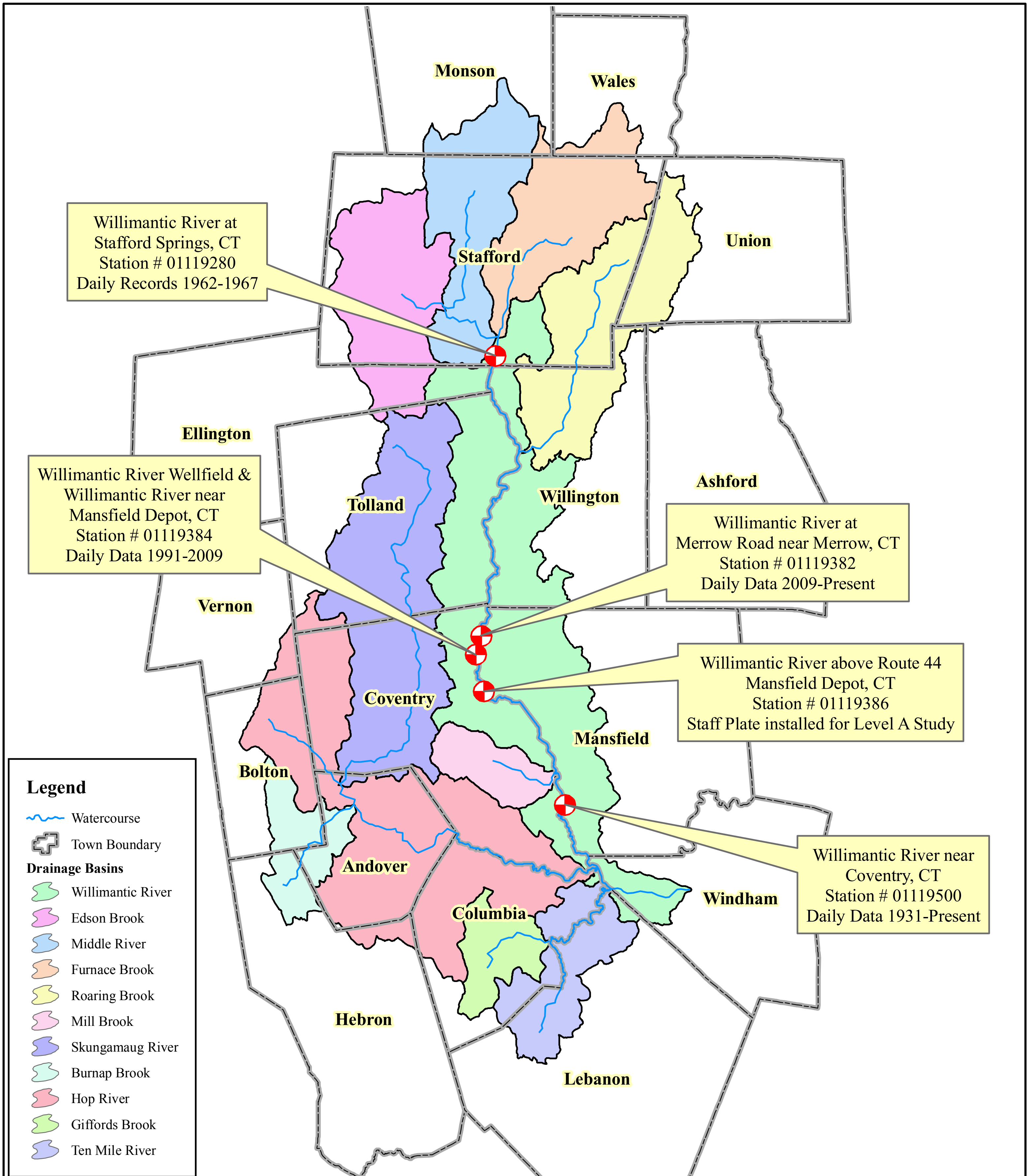
2.0 HYDROLOGY AND HYDROGEOLOGY

2.1 Drainage Basins

The Willimantic River Wellfield is located within the regional drainage basin of the Willimantic River (Basin #3100), a tributary of the Shetucket River (Basin #3800), within the Thames River major basin (#3000). The Willimantic River is formed by the confluence of Middle River and Furnace Brook in the village of Stafford Springs (Figure 2-1). The river flows generally southward from Stafford Springs to Willimantic before turning to the east where it joins with the Natchaug River to form the Shetucket River.

The Willimantic River drainage basin has a total area of 225.5 square miles. According to the United States Geological Survey (USGS), the area of the drainage basin to the USGS gage in South Coventry is 121 square miles. The upper section of the Willimantic River drainage basin that drains to the wellfield has a total area of 98 square miles, with 6.4 square miles lying in Massachusetts. Municipalities in the watershed include Andover, Ashford, Bolton, Columbia, Coventry, Ellington, Hebron, Lebanon, Mansfield, Stafford, Tolland, Union, Vernon, Willington, and Windham, Connecticut; and Monson and Wales, Massachusetts.

In 2003, the Willimantic River's 25-mile length was designated as the Willimantic River Greenway/Blueway by the State of Connecticut. The nine adjacent municipalities work with the Willimantic River Alliance to enhance recreational access for boating, fishing, and walking trails and also to protect natural resources along the river. The river is a popular canoe/kayak route for 24 miles, and several trails link numerous town and state properties along the river.



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Location Map

LOCATION: **Connecticut & Massachusetts**

MMI#: 1958-09
MXD: H:\Figure2-1.mxd
SOURCE: CT DEP, USGS

Willimantic River Study

Map By: SJB
Date: May 2010
Scale: 1" = 20,000'

SHEET: **Figure 2-1**

2.2 Surface Water Hydrology

The Willimantic River is an incised channel composed of alluvial cobbles and dense sand in the vicinity of the wellfield. The channel of the river is well developed, with a width ranging from 50 to 100 feet. The riverbanks near the wellfield are primarily state or privately owned undeveloped land. Both riverbanks are typically steep to as much as four feet, with riverbanks in some areas approaching as high as 40 feet.

In general, the riverbed is fairly flat throughout the majority of the reaches near the wellfield. A variety of riffles, runs, pools, and other habitat areas are prevalent in the river within the study area. During times of low flow in the summer and fall, depth ranges from only a few inches to several feet. The riverbed contains abundant sand, gravel, and cobbles. Organic material is found in some locations as well.

The most recent drought period on the Willimantic River was September to early October 2007. During this period, the depth of the river varied from several inches in the riffles to one to three feet in the pools and moderate to deep run habitats as seen by MMI upstream from the wellfield in Tolland and by the DEP at the wellfield. Discharge during low-flow periods are buttressed upstream by effluent releases from the wastewater treatment plant in Stafford Springs with an average flow of 1.5 cubic feet per second (cfs) from the plant during the summer months (DEP, 2001) and downstream by effluent releases from the University's wastewater treatment plant. This flow rate may not be constant over the course of each day.

Upstream of the University's wellfield, the discharge of the Willimantic River is further influenced by withdrawals from Connecticut Water Company's public water supply reservoirs in the headwater streams and by withdrawals from the Town of Tolland's public water supply wellfield in River Park. These diversions are discussed further in Section 2.5.

Flow Characteristics

Until 2009, the USGS maintained two real-time gaging stations on the Willimantic River (Appended Figure 1). The upstream gage was formerly the Mansfield Depot gage (station #01119384, watershed of 98 square miles) located at the Willimantic River Wellfield, and the downstream gage is located in Coventry (station #01119500, watershed of 121 square miles). The Mansfield Depot station was active from 1991 to 2009 whereas the Coventry station has been gaged continuously since 1931.

One of the provisional recommendations of this study from December 2008 was to establish a new gaging station at Merrow Road. The gaging station would provide the University with instream flow data that would be unaffected by pumping at the Willimantic River Wellfield, similar to the creation of the USGS gaging station (station # 01121330) on the Fenton River (a recommendation of the Fenton River Study). The Merrow Road real-time gaging station (station #01119382) was activated in September 2009.

The unregulated statistical daily flows of the Willimantic River were compiled using information provided in several sources. The bulletin *Water Resources Inventory of Connecticut, Shetucket River Basin* (Thomas, et al., 1967) was used to provide low flow statistics using limited data from the 1960s for the Willimantic River at Merrow, Connecticut. Flow duration data for the USGS gage in Coventry were recently generated by the USGS over a long-term period (74 years of data, Ahearn, 2005). The USGS-calculated flow duration statistics for the Coventry gage were then used to calculate long-term statistics for the Mansfield Depot gage at the Willimantic River Wellfield by multiplying each discharge by watershed ratio (98 mi^2 / 121 mi^2, or 0.81).

Statistical low flow discharges for the Willimantic River are presented in Table 2-1 below as reported by the USGS.

Table 2-1
Statistical Flows of the Willimantic River

Flow Duration	Willimantic River at Merrow (1960s data)	Willimantic River at Willimantic River Wellfield[1]	Willimantic River at Coventry (Ahearn, 2005)
80%	37 cfs (23.8 mgd)	44 cfs (28.3 mgd)	54 cfs (34.9 mgd)
90%	24 cfs (15.3 mgd)	28 cfs (17.8 mgd)	34 cfs (22.0 mgd)
95%	16 cfs (10.5 mgd)	20 cfs (13.1 mgd)	25 cfs (16.2 mgd)
99%	8.9 cfs (5.7 mgd)	11 cfs (6.8 mgd)	13 cfs (8.4 mgd)

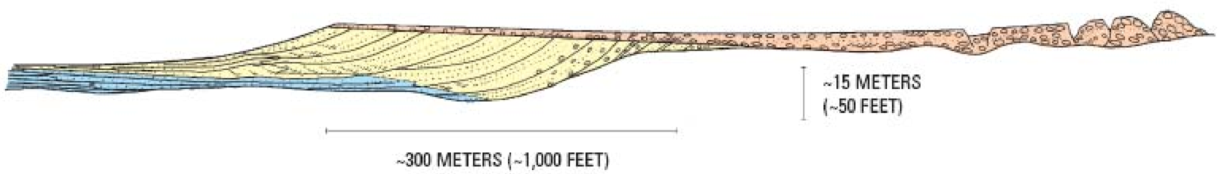
Notes: [1]Based on watershed ratio transformation from long-term Coventry gage data in Ahearn (2005).

The University may withdraw 2.3077 mgd from the Willimantic River Wellfield according to its diversion registration. This rate amounts to 34% of its 99% duration flow but only 8% of its 80% duration flow as calculated at the Willimantic River Wellfield. Much of this water is returned downstream at the sewage effluent discharge point just below the Eagleville Lake dam at Route 275 in Eagleville. Therefore, any instream flow impacts are manifested between the upstream edge of the area of influence of the wellfield and the upstream end of Eagleville Lake.

2.3 Surficial Geology

Several geologic maps are available documenting the surficial materials near the Willimantic River Wellfield. First and most recently, the report *Quaternary Geologic Map of Connecticut and Long Island Sound Basin* (USGS Scientific Investigations Map 2784, Stone et al., 2005) was recently published by the USGS. A review of this report has provided valuable information toward a better interpretation of the subsurface deposits in the vicinity of the wellfield.

The report and its maps depict the presence of "wru" (Upper Willimantic River Deposits) at the wellfield and extending upstream and downstream along the valley. This is classified as a deposit of sediment-dammed ponds. A graphic and an excerpt from the report are provided below:

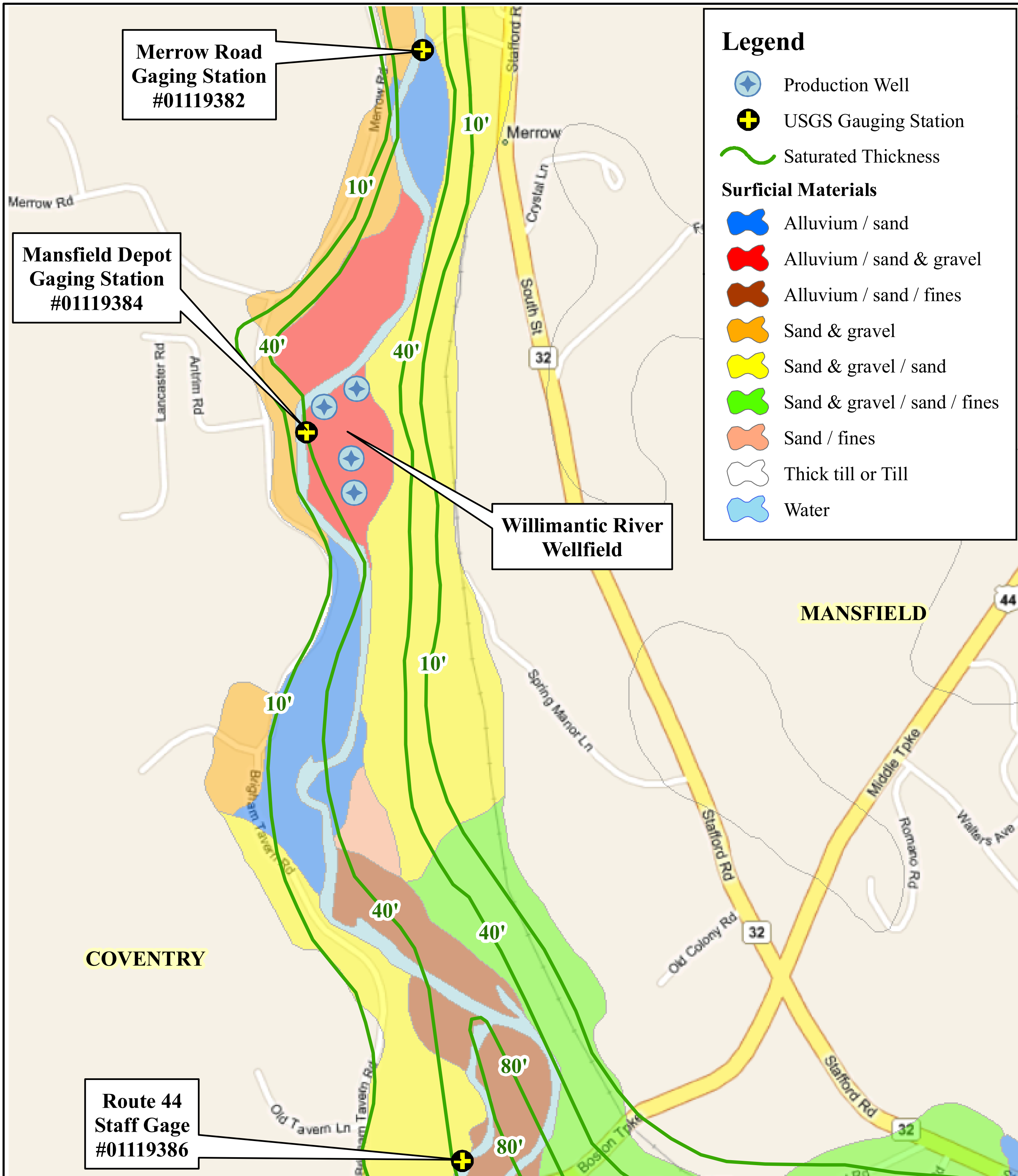


Ice-Marginal Fluviodeltaic Morphosequence (Stone et al., 2005)

> *Upper Willimantic River deposits: Series of ice-marginal fluviodeltaic deposits in section of upper Willimantic River valley that is narrow with a steep gradient. Ponding was initially behind head of Willimantic River deposits (wrl) at about 96 m (315 ft); deltaic surfaces are as high as 126 m (415 ft) in northern part of unit.*

The morphosequence suggests that the geology of the wellfield will have a thin, coarser layer of overburden underlain by finer sands and gravels, with the presence of finer lake-bottom sediments at depth under some of the deposits. The surficial geology of the deposits based on this report is shown in Figure 2-2. In general, the surficial geology is mapped as alluvium or sand and gravel overlying layers of finer materials such as sand and gravel, sand, or fines. This is consistent with deposition of the ice-marginal fluviodeltaic morphosequence depicted above.

The thickness of the saturated stratified drift at and around the wellfield is mapped as being greater than 40 feet in thickness in the *Water Resources Inventory of Connecticut, Shetucket River Basin* (Thomas et al., 1967). Refer to Figure 2-2 for a depiction of saturated thickness based on this publication. Typically, stratified drift deposits are thickest near the center of the valley and become thinner toward the edges. Drilling logs for the wellfield indicate a stratified drift thickness of up to 70 feet at the wells. Therefore, the reported geology is consistent with the mapped "greater than 40-foot" thickness.



Engineering, Landscape Architecture and Environmental Science **MILONE & MACBROOM®** **99 Realty Drive** **Cheshire, Connecticut 06410** **(203) 271-1773 Fax: (203) 272-9733** **www.miloneandmacbroom.com**	**Surficial Geology**		**LOCATION:** **Mansfield, CT**	
	MMI#: 1958-09 **MXD:** H:\Figure2-2.mxd **SOURCE:** Microsoft Virtual Earth, DEP, USGS	N **Willimantic River Study**	**Map By:** SJB **Date:** Mar. 2010 **Scale:** 1"=1,000'	**SHEET:** **Figure 2-2**

The report *Water Resources Inventory of Connecticut, Shetucket River Basin* (Thomas et al., 1967) depicts as Figure 40 a cross section of the Willimantic River Wellfield through UConn Well #3 (also known as "Ms-25" in the USGS nomenclature). Figure 40 is reprinted here as Figure 2-3. The cross section depicts a layer of coarser "bouldery river deposits" overlying generally finer layers of sand and sand and gravel at the wellfield, with areas of gravel or sand on the sidewalls of the river valley. This information is consistent with the information provided in the 2005 USGS report.

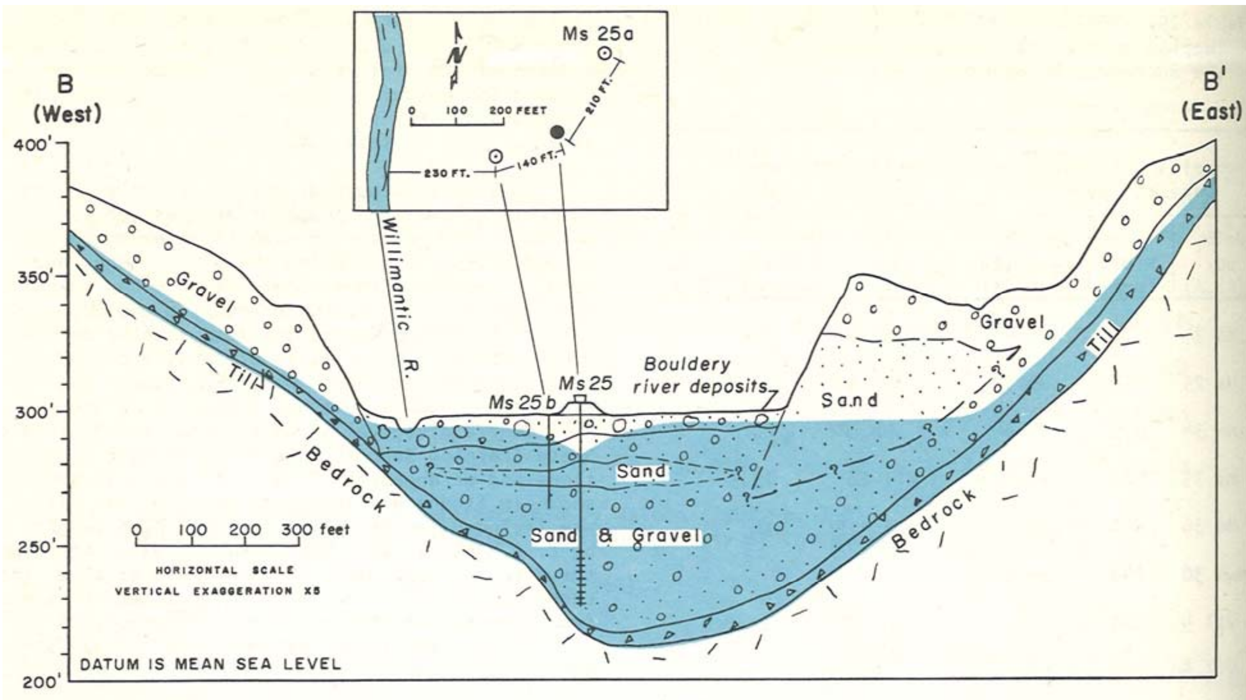


Figure 40.--Geologic cross section of the Willimantic River valley near the wells of the Mansfield State Training School, along line B-B' on plate B.

In a pumping test, Ms 25 was pumped at 418 gpm for 24 hours, causing a cone of depression to form in the water table. Average permeability of the stratified deposits was determined to be 4,170 gpd per sq ft.

Figure 2-3: Geologic Cross Section Through UConn Well #3 (Thomas et al., 1967)

Many observation wells and borings were drilled around the wellfield as a result of the various boring programs completed since the early 20th century. While the logs for many of the borings and wells are unavailable, the report *Level A Mapping for the University of Connecticut Willimantic River Wellfield and Aquifer, Mansfield, Connecticut* (Meade,

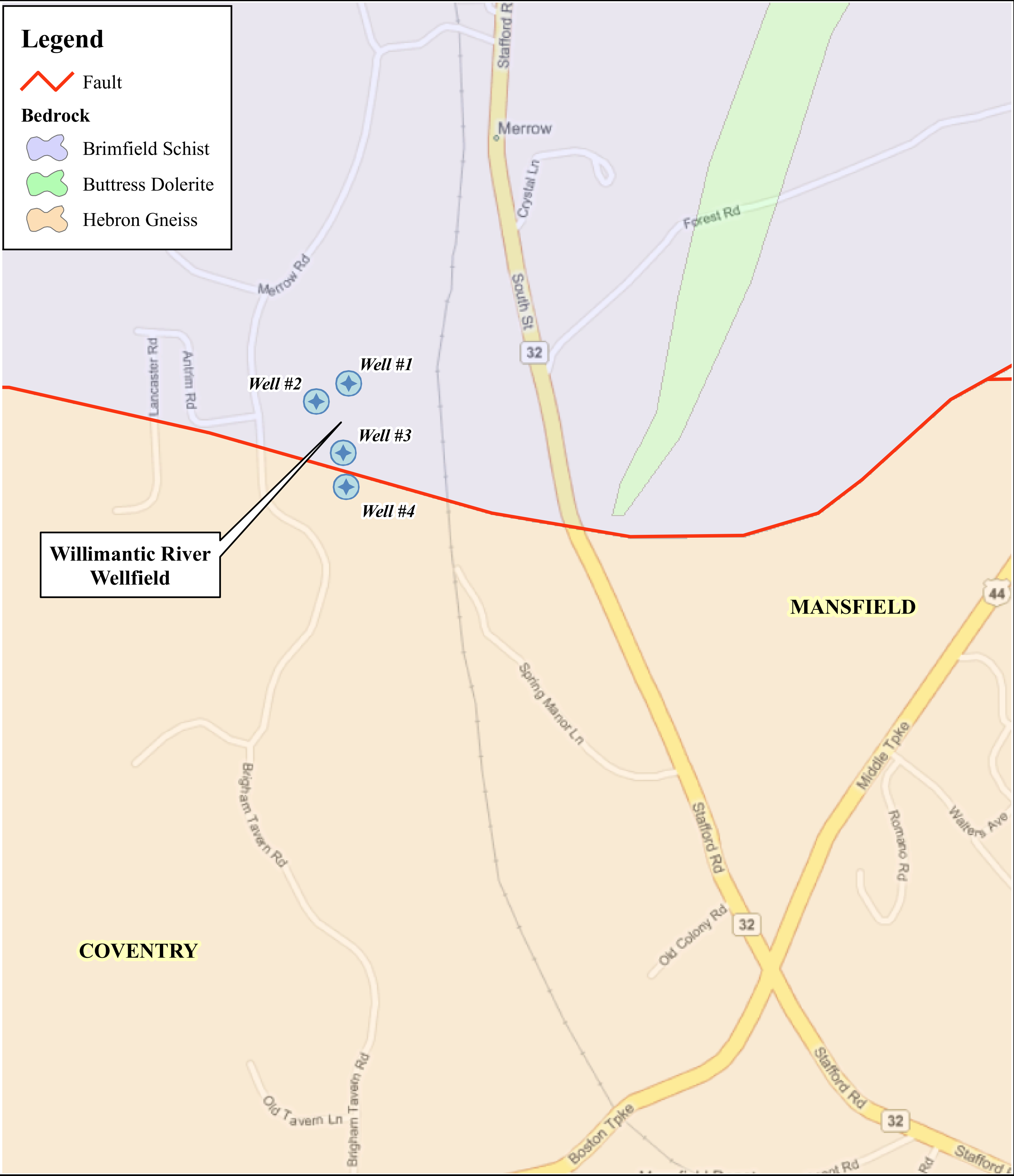
2007) describes the extensive boring program undertaken in the 1990s for the Level A Mapping Study.

A total of 10 cross sections were drawn across the Willimantic River valley at and downstream of the wellfield based on this boring program and the review of nearby private well drilling logs. The cross sections generally depict a coarser layer of "boulders and sand" overlying finer layers of sand or sand and gravel in the river valley, with areas of fines at depth located downstream of the wellfield. This is consistent with the information provided in the 1967 and 2005 USGS reports.

2.4 Bedrock Geology

The wellfield is bisected by an undefined fault line according to the *Bedrock Geologic Map of Connecticut* (Rodgers, 1985). The fault line is inactive and runs slightly south of east between UConn Well #3 and UConn Well #4. The bedrock to the north of this fault is mapped as Brimfield Schist (map unit Obr). Brimfield Schist is identified as a gray, rusty-weathering, medium- to coarse-grained, interlayered schist and gneiss. The bedrock to the south of this fault is mapped as Hebron Gneiss (map unit SOh). Hebron Gneiss is identified as an interlayered dark-gray schist and greenish gray, fine- to medium-grained calc-silicate gneiss. Refer to Figure 2-4 for an illustration of bedrock geology in the area.

As stated in the Level A Report (Meade, 2007), the Willimantic River is confined to a valley that has been cut into fractured and folded metamorphic rock. Bedrock elevations were previously interpolated as part of the Level A Mapping Study based on information in the observation and water supply well logs, from well drilling and well completion reports, and from a ground-penetrating radar study. Additional borings or geophysical studies of the aquifer have not been completed since the Level A study.



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Bedrock Geology

LOCATION: Mansfield, CT

MMI#: 1958-09
MXD: H:\Figure2-4.mxd
SOURCE: Microsoft Virtual Earth, DEP

N

Willimantic River Study

Map By: SJB
Date: Mar. 2009
Scale: 1"=1,000'

SHEET: Figure 2-4

2.5 Locations of Water Diversions

No known surface water or ground water diversions of more than 50,000 gpd are located within the areas of recharge and contribution of the Willimantic River Wellfield. According to the Connecticut DEP, several registered and permitted diversions are located in the Willimantic River watershed upstream of the Willimantic River Wellfield. These diversions are summarized in Table 2-2.

Table 2-2
Registered and Permitted Diversions Upstream of Willimantic River Wellfield

Owner	Type	Basin	Town	Location / Description	Maximum Withdrawal (mgd)
Connecticut Water Company	Registered	Willimantic River	Stafford	Stafford Well #3	0.1872
Connecticut Water Company	Registered	Roaring Brook	Stafford	Stafford Reservoir #2	0.7560
Connecticut Water Company	Registered	Roaring Brook	Union	Stafford Reservoir #3 (Mathews Pond)	9.5000
Connecticut Water Company	Registered	Roaring Brook	Union	Stafford Reservoir #4 (Moore Pond)	5.3000
Connecticut Water Company	Registered	Roaring Brook	Stafford	Stafford Well Caisson #1	0.8640
Connecticut Water Company	Registered	Roaring Brook	Stafford	Stafford Well Caisson #2	0.1872
Town of Tolland	Permitted	Willimantic River	Tolland	River Park Wellfield Wells #1 and 2	0.2200
TTM Printed Circuit Group, Inc.	Permitted	Furnace Brook	Stafford	Riverside Pond Withdrawal	0.1800
Warren Corporation	Permitted	Furnace Brook	Stafford	Furnace Brook Mill Pond Withdrawal	0.5000

Most of the listed diversions are located in the headwaters of the Willimantic River Watershed. The Connecticut Water Company (CWC) utilizes its registered surface water withdrawals for public water supply in the Stafford system. As the Stafford Water Treatment Plant has a capacity of only 1.0 mgd, the full registered withdrawal rates are not realized. The three registered wells are currently inactive (CWC, 2006).

As introduced in Section 2.2, much of the water withdrawn by CWC is returned to the Willimantic River via the Stafford Springs Water Pollution Control Facility (WPCF).

Only the fraction of water used for landscaping and other evaporative needs is lost from the river system. However, it is suspected that infiltration and inflow to the sewer system contribute a greater quantity of water to the river than the quantity lost from the public water service. The average discharge from the WPCF is 1.5 cfs, equivalent to 1.0 mgd during the summer months (DEP, 2001).

The two industrial-use withdrawals are believed to be active as the permits were renewed within the last decade. The Furnace Brook Mill Pond and the Riverside Pond are located in the villages of Stafford Springs and Stafford, respectively, along Furnace Brook. Furnace Brook forms the Willimantic River along with Middle River. The exact use of water by TTM Printed Circuit Group and the Warren Corporation is not known, but cooling is suspected.

The closest diversion to the Willimantic River Wellfield is the Town of Tolland withdrawal for public water supply, which is three miles upstream. The Tolland withdrawal is authorized for 0.22 mgd from the River Road Wellfield (or "Willimantic River Wellfield"). This withdrawal is relatively minor in comparison to the amount of withdrawal regularly utilized by the University.

During periods of very low instream flow, effluent at Stafford Springs comprises a relatively significant percentage of the flow of the river. On the other hand, the daily withdrawals from Tolland's wellfield are typically less than 0.2 mgd (0.3 cfs), comprising a very low percentage of flow in the river.

2.6 Climate, Precipitation, Evapotranspiration, and Recharge

The climate of Mansfield is characterized by warm, humid summers and cold winters. Although mean annual precipitation in the Shetucket River basin for the period 1931 through 1960 was reported at 45.9 inches per year (Thomas et al., 1967), a review of monthly precipitation data provided by CWC from Rockville, Connecticut for the calendar years 1960 through 2007 reveals that precipitation totals are now higher, with a mean of approximately 47.5 inches per year.

Precipitation totals for the calendar years 1998 through 2007 are even higher, averaging approximately 55.1 inches per year. This is consistent with studies showing that average annual precipitation has been increasing on the order of 0.96 inches per decade since the end of the 19th century (Miller et al. 1997, MacBroom, 2005).

Thus, both runoff and ground water recharge in the watershed have likely been increasing over time, neglecting other factors such as impervious surfaces and diversions. A comparison to summaries provided by the USGS in the annual reports for Connecticut indicates that, for the most part, these data comprise a reliable and realistic record of regional precipitation. Table 2-3 summarizes annual totals for calendar years based on CWC records.

Table 2-3
Precipitation Summary, Connecticut Water Company Gage in Rockville, Connecticut

Year	Calendar Year Annual Precipitation (inches)	Year	Calendar Year Annual Precipitation (inches)
1960	49.37	1984	50.00
1961	36.51	1985	42.80
1962	35.39	1986	42.37
1963	38.33	1987	42.69
1964	38.25	1988	38.13
1965	31.72	1989	49.21
1966	37.45	1990	48.79
1967	45.44	1991	46.26
1968	36.13	1992	51.01
1969	42.37	1993	47.92
1970	36.79	1994	54.16
1971	40.71	1995	46.03
1972	57.72	1996	58.59
1973	52.22	1997	43.41
1974	49.47	1998	50.96
1975	56.14	1999	51.48
1976	38.07	2000	49.11
1977	55.25	2001	40.06
1978	37.06	2002	54.57
1979	56.72	2003	59.01

Year	Calendar Year Annual Precipitation (inches)	Year	Calendar Year Annual Precipitation (inches)
1980	39.75	2004	58.41
1981	45.78	2005	71.53
1982	48.95	2006	65.34
1983	61.67	2007	50.32
Average (1960 to 2007)			*47.49*
Average of Last 10 Years (1998 to 2007)			*55.08*

Mean annual evapotranspiration in the Shetucket River Basin was reported as 20.3 inches per year for the period 1947 to 1962 (Thomas, et al., 1967) as shown in Table 2-4. Water lost to evaporation is derived from the unsaturated zone as well as from the water table. Actual evapotranspiration rates depend on temperature, length of daylight, and vegetative cover and vary widely on a monthly basis.

Table 2-4
Monthly Evapotranspiration Rates for Shetucket River Basin (Thomas, Jr. et al., 1967)

Month	Theoretical Average Monthly Evapotranspiration (inches)
January	0.18
February	0.18
March	0.60
April	1.20
May	2.50
June	3.40
July	4.00
August	3.50
September	2.45
October	1.50
November	0.70
December	0.18
Total	**20.39**

The amount of precipitation that infiltrates the ground surface is dependent on the setting. In the report *Ground-Water Availability and Water Quality at Southbury and Woodbury, Connecticut* (Mazzaferro [USGS], 1986), runoff was reportedly 35% to 53% of precipitation, and approximately 44% to 50% of the precipitation was estimated to recharge the ground water table. Estimates of recharge to stratified drift areas are provided below in Table 2-5 as taken from the report.

Table 2-5
Recharge Rates

Time Period and Average Annual Precipitation	Average Annual Recharge Estimated From Precipitation	Average Annual Recharge Estimated From Streamflows
Long-term average (1941-1970) – 43.1 inches	18.8 inches	20.1 inches
10-year average (1969-1978) – 51.4 inches	22.5 inches	25.8 inches
3-year highest (1975-1977) – 55.2 inches	24.1 inches	27.1 inches
3-year lowest (1964-1966) – 34.5 inches	15.1 inches	11.6 inches

Runoff from till can be as much as three times higher than runoff from stratified drift areas. Section 6.0 of this report will discuss precipitation and its relationship to recharge in terms of numerical modeling.

After a 10-plus year study period under the Regional Aquifer-System Analysis (RASA) program, the USGS published the report *Regional Hydrology and Simulation of Flow of Stratified-Drift Aquifers in the Glaciated Northeastern United States* in 2004. The RASA report mainly compiled and discussed information from numerous individual studies, including those listed above for Woodbury and Southbury, and a similar study conducted in the Town of Farmington, Connecticut. Although the RASA report provides some ranges of recharge rates that include higher and lower values than those listed in Table 2-5, these were for areas that were dissimilar to Mansfield. The RASA report also states that the average annual evapotranspiration for the Mansfield area is approximately 22 inches per year.

3.0 DESCRIPTIONS OF PREVIOUS STUDIES

The Willimantic River Wellfield has been in existence since approximately 1913. Since that time, several studies of the aquifer at and near the wellfield have been performed by Mansfield Training School and the University. Studies with available reports and documentation are detailed below.

3.1 Mansfield Training School Wells

According to the USGS Report *Hydrogeologic Data for the Shetucket River Basin, Connecticut* (Thomas, Jr., et al., 1967), Mansfield Training School installed a 240-inch diameter "dug" well at the Willimantic River Wellfield to a depth of 16.5 feet around the year 1913. This well is labeled as "Ms-23" in the report and more commonly known as MTS Well #1.

Mansfield Training School performed investigations in the early 1940s culminating in a 1945 report on water supply facilities and a yield test of the well. The well was supplemented by MTS Well #2 (Ms-24) in 1948 and MTS Well #3 (Ms-25) in 1958 due to insufficient yield. MTS Well #1 was reportedly taken off-line in 1961. MTS Well #3 was installed by R. E. Chapman and Company and later became UConn Well #3. MTS Well #1 was eventually officially abandoned in the 1970s.

MTS Well #2 had a reported yield of 525 gallons per minute (gpm) in 1967. The 1967 USGS Report contains a drilling log and a 24-hour pumping test of MTS Well #3 performed in 1964 during which the well was pumped at 418 gpm for 24 hours, with a maximum drawdown of nine feet, and water levels were measured at two nearby observation wells (which were abandoned by 1967). The average permeability of the aquifer was determined to be 4,170 gallons per day per square foot. MTS Well #3 was also reportedly pump tested for two days by R. E. Chapman Company at 703 gpm in 1959 (Ritsick, Vol. 2, 2004).

According to information available in the 1967 USGS Report (Thomas, Jr. et al.), the total water usage of the University water system in 1963 was 276 million gallons, equivalent to an average day demand of 756,160 gallons per day. The total water usage of the Mansfield Training School in 1964 (including residential, institutional, industrial, and agricultural uses) was 140 million gallons, equivalent to a water usage of 382,520 gallons per day with an average per capita demand of 134.3 gallons per day. The two systems were interconnected by 1964, with the turnover of the Mansfield Training School system to the University occurring in 1969 (Ritsick, 2004).

3.2 1960s Subsurface Investigations

In 1969, the University reached an agreement with Mansfield Training School in which the University was granted exclusive use of the land at the Willimantic River Wellfield to supply water to both Mansfield Training School and the University. Prior to and as a result of this agreement, the University commissioned several studies performed by Frederic R. Harris Associates including ground water/aquifer explorations, test well development, production well development, well yield tests, and aquifer yield tests at the wellfield. These investigations concluded in a series of interim reports and two final reports in 1968 and 1970.

The 1968 report entitled *Additional Water Supply – Results of Subsurface Explorations at Existing Mansfield Well Field* discusses the suitability of the Willimantic River aquifer supplying the existing wellfield as a source of water for the combined demand of the University and Mansfield Training School. Field investigations were performed by the R. E. Chapman Company under the supervision of Frederic R. Harris Associates. A total of 14 borings were drilled, of which 12 were fitted with well point piezometers to monitor water levels during an aquifer pumping test.

The boring program began in May 1968. Following the boring program, the aquifer pumping test was conducted to determine aquifer characteristics. The test likely occurred during a 15-day period in July 1968. MTS Well #3 was initially pumped at 500 gpm for 45 hours, and then MTS Well #2 and MTS Well #3 were pumped concurrently for more than 304 hours at 500 gpm and 725 gpm, respectively. Water levels were measured in both production wells and seven observation points. The resulting analysis suggested that the aquifer in the vicinity of the Mansfield Training School wells has a storage coefficient of 1.46×10^{-3} and a transmissivity of 90,000 gpd/ft, slightly different than the 1964 test, which suggested that the aquifer had a storage coefficient of 0.82×10^{-3} and a transmissivity of 242,000 gpd/ft.

The report drew the following conclusions from the various field investigations relevant to the current study:

1. The state-owned lands across the Willimantic River southwest of the wellfield are underlain by a shallow aquifer not suitable for developing a major quantity of water.
2. The Willimantic River Wellfield is limited in extent to the south and east by finer grained materials but extends northward beyond the state-owned lands.

3.3 Construction and Testing of Production Well UConn #1

The 1970 report entitled *Additional Water Supply Facilities – Construction and Testing of UConn Deep Well No. 1, Mansfield Well Field* discusses the construction and testing of UConn #1 as performed by the Able Drillers and Pump Company under the supervision of Frederic R. Harris Associates. First, two pilot borings set 100 feet apart were performed, indicating a depth to bedrock of 71.5 feet and 54.8 feet, respectively. Deep Well No. 1 (UConn Well #1) was drilled near the deeper pilot boring. Next, the well was installed with a No. 65 slot, 14-inch diameter screen 20 feet in length. Finally, the well was developed by surging and bailing, which took approximately one week.

Twelve piezometers were constructed to monitor drawdown, located three each along mutually perpendicular lines at approximate distances of 25, 50, and 100 feet from the production well. The depths of the piezometers ranged from 52 to 60 feet below the ground surface. These piezometers were used to determine aquifer parameters during a subsequent pumping test.

On September 7, 1970, MTS Well #3 began pumping at a constant rate of 500 gpm to stabilize water levels at the Willimantic River Wellfield in an effort to disregard any interference from MTS Well #3 on the pumping test. Pumping of UConn #1 commenced at 2:25 p.m. on September 10 and continued until 11:00 a.m. September 18, 1970. UConn #1 was pumped at an average of 750 gpm during the pumping test. Water levels were monitored in the 12 piezometers.

Analysis of the pump test data using the Reverse-Type Curve method suggested that the aquifer in the vicinity of UConn Well #1 has a storage coefficient ranging from of 0.003 to 0.017 and a transmissibility ranging from of 93,848 gpd/ft to 127,056 gpd/ft. Further analysis using distance-drawdown plots suggested that the coefficient of transmissibility was 148,314 gpd/ft and the coefficient of permeability to be approximately 2,557 gpd/ft^2, with a specific yield of 39 gpm per foot of drawdown.

3.4 Construction of Production Well UConn #2

UConn #2 was installed in 1974 by R. E. Chapman Company as a 24-inch by 14-inch gravel-packed well to a depth of 67.5 feet. The well was completed with 15 feet of 14-inch diameter screen and was initially yield tested at 361 gpm (Ritsick, Vol. 2, 2004) for 48 hours with a drawdown of 40.5 feet, representative of a specific capacity of 8.9 gpm/ft.

The well was redeveloped by R. E. Chapman Company in 1993. Prior to redevelopment, the well was rated at 175 gpm with 26.20 feet of drawdown, or a specific capacity of 6.7

gpm/ft. After the redevelopment, the well was rated at 191 gpm with 22.10 feet of drawdown and a specific capacity of 8.6 gpm/ft.

3.5 Level A Mapping of the Willimantic River Wellfield

Level A Aquifer Protection Area Mapping of the Willimantic River Wellfield commenced in 1993 by Mr. Daniel Meade, a hydrologist working with funding and support provided by the University and the Connecticut Department of Environmental Protection. The modeling and mapping were first submitted to the Connecticut Department of Environmental Protection in 1999 and resubmitted with revisions in May 2007, resulting in subsequent approval.

The results of the Level A Mapping project were discussed by Mr. Meade in the March 2007 report *Level A Mapping for the University of Connecticut Willimantic River Wellfield and Aquifer, Mansfield, Connecticut*. The report describes the evaluation of existing data and discusses the field data collection program; the ground water model construction, calibration, and verification; and the subsequent mapping of areas of influence, contribution, recharge, and indirect recharge to the wellfield.

Data Collection

According to the Level A report (Meade, 2007), existing data was analyzed in a report entitled *A Plan to Collect and Analyze Data for the Willimantic River Wellfield and Aquifer System*. Based on information collected and analyzed in this document, it was determined that new data collection would focus on characterizing the hydraulic properties of the aquifer, the discharge of the Willimantic River, and establishing an understanding of ground water/surface water relationships between the Willimantic River and its aquifer.

A total of five test holes and 16 new observation wells were installed in the vicinity of and adjacent to the Willimantic River Wellfield. The boring logs and median grain size from several of these wells were used to estimate hydraulic conductivity. In addition, the bedrock elevations from these borings were utilized along with ground-penetrating radar to estimate the bedrock surface beneath the wellfield. The Level A Report (Meade, 2007) contains boring logs and geologic cross sections related to the field investigations.

Ground water levels were measured in 18 observation wells on an irregular schedule from 1992 to 1995. Streamflow measurements were measured at USGS Station #01119384 at the Willimantic River Wellfield, at Merrow Road (USGS Station #01119382), and 100 yards upstream of Route 44 (USGS Station #01119386) during 1991 to 1997 to establish a stage-discharge relationship.

Ground water/surface water relationships were studied using a series of nine piezometers driven into the river bottom. The piezometers were used only to qualitatively identify areas where surface water was being induced during pumping. In addition, the report states that measurements of vertical hydraulic conductivity in the Willimantic River by both constant and falling head permeameters were performed.

Numerical Model

The Willimantic River Wellfield was modeled using MODFLOW 2000, a finite-difference model capable of simulating three-dimensional ground water flow. After calibration and verification of the Level A model, a steady-state predictive simulation was performed under recharge/discharge conditions that would produce a median annual streamflow. A 1,400 gpm pumping rate was used at the four Willimantic River Wellfield production wells to determine the areas of influence, contribution, recharge, and indirect recharge.

The model results indicated that 97.3% of the water pumped from the aquifer under this condition would come from water that was either induced from or would have discharged to the Willimantic River. The Level A numerical model is discussed in detail in Section 6.2.

3.6 1999 Safe Yield Test of Production Well UConn #4 and Aquifer Pumping Test

In 1998, Lenard Engineering, Inc. performed a test boring in the vicinity of MTS Well #2. The subsurface exploration found that the aquifer consists of coarse sand, gravel, and cobbles overlying fine to medium sand, with cobbles existing between 25 and 40 feet below the ground surface, fine sands present from 55 to 57 feet and below 61 feet, and bedrock at 71 feet. The University subsequently installed production well UConn #4 in 1998 to replace the aging MTS Well #2. UConn #4 has a 12-inch diameter, 15-foot long screen located between 38 and 53 feet below the ground surface.

A safe yield test and aquifer pumping test were conducted by Lenard Engineering, Inc. during extended low-flow conditions in August 1999. Antecedent trends were monitored for eight days from August 11, 1999 to August 18, 1999 during a period of very little to zero pumping at the Willimantic River Wellfield. The pumping of UConn #4 commenced on August 19, 1999 at 8:15 a.m. at a rate of 290 gpm. On the morning of August 20, 1999, the settings of the turbine bowls were adjusted to allow a pumping rate of 500 gpm.

Starting on August 20, 1999 at 10:30 a.m., UConn #4 was pumped for 72 hours at a rate of 489.6 gpm. Two additional wells, UConn #1 and UConn #3, were activated on August 23, 1999. UConn #1, UConn #3, and UConn #4 were pumped until 12:00 p.m. on August 28, 1999 at rates of 286.55 gpm, 281.76 gpm, and 413.97 gpm, respectively. The total yield of the three wells during the five-day test was 982.3 gpm, or 1,414,500 gpd.

During the pumping test, water levels were measured in the four production wells, MTS Well #2, 17 observation wells, and five piezometers. Automatic dataloggers were also used to record water levels in the production wells. Streamflow measurements were performed at the three USGS stations used in the Level A Study during the antecedent period and at the end of the pumping test. Precipitation was also monitored using a temporary rain gage installed at the wellfield.

Based on the results of the pumping test, Lenard Engineering, Inc. determined that the specific capacity of UConn Well #4 was 21.67 gpm/ft, with an ultimate safe yield of 560 gpm, greater than the registered diversion rate. The data from this pumping test was used to calibrate the updates to the Level A numerical model performed as part of the Supplemental Hydrogeologic Study (Section 6.4).

4.0 INSTREAM FLOW STUDY

4.1 Background

The Instream Flow Incremental Method (IFIM) was used to evaluate the potential effects of reductions in river flow associated with withdrawal of water at the Willimantic River Wellfield on the habitats of representative fish species in the Willimantic River. The IFIM was developed by the U.S. Fish and Wildlife Service's (USFWS) Cooperative Instream Flow Group as a method of evaluating the impact of alternative flow regimes on aquatic habitat (Bovee, 1982).

One element of the IFIM simulates river hydraulics and aquatic habitat using computer models collectively known as PHABSIM (Physical Habitat Simulation). The hydraulic simulation models of PHABSIM are used to predict changes in depth, velocity, and wetted area at various river flows. The aquatic habitat simulation models generate a composite suitability function collectively referred to as Habitat Suitability Criteria (HSC) derived from curves representing the depth, velocity, and substrate preferences of selected target species/life stages. The aquatic habitat simulation models integrate the output of the hydraulic simulation models with the HSC to yield an estimate of habitat usability called weighted usable area (WUA) (Bovee and Milhous, 1978).

The IFIM study for the UConn Willimantic River Wellfield consisted of the following components:

- Project scoping in cooperation with the Technical Advisory Group (TAG)
- Field mapping of aquatic habitat to locate and determine the percentages of all significant mesohabitat types in the study area
- Field data collection, including surveying riverbed elevations and characterization of substrate type across multiple transects, and measurement of water depths and velocities at these transects over a range of river flows

- Hydraulic and habitat simulation using the PHABSIM models
- Time series and UCUT (Uniform Continuous Under-Threshold) analyses to evaluate the magnitude, frequency, and duration of various flow-related habitat "events."

The study area for the instream flow assessment consisted of a 2.2-mile reach of the Willimantic River extending from the wellfield downstream to the backwater of Eagleville Lake (Appended Figure 1). This river reach is characterized by a series of shallow riffles and runs, with a few deeper pools. Riverbed materials are primarily cobble and small boulders interbedded with gravel. Areas of sand and silt with some organic matter occur in the pools and backwater areas. The riverbanks within the study area are generally steep with narrow, low-lying shelves immediately adjacent to the river in some areas. Upslope, the riparian vegetation is comprised primarily of deciduous trees, shrubs, grasses, and vines.

Electrofishing was conducted by the DEP at two locations in the Willimantic River in July 1994. The electrofishing performed near UConn Well #4 and upstream of Depot Road/Coventry Road in Coventry, Connecticut yielded a total of 1,676 specimens of 23 taxa of fish (Table 4-1) and evidenced a mix of fluvial specialist, fluvial dependent, and macrohabitat generalist species.

The most abundant species were fallfish (25.7%), common shiner (20.0%), white sucker (18.7%), redbreast sunfish (9.4%), and smallmouth bass (8.8%). A single brown trout was collected in these samples. The DEP stocks brown trout and brook trout (*Salvelinus fontinalis*) in the Willimantic River although it is unlikely that these species reproduce in the river (Brian Murphy, DEP, personal communication).

Table 4-1
Fishes Collected in the Willimantic River by the DEP, July 1994
(Source: Brian Murphy, CTDEP, personal communication)

Common Name	Scientific Name	1.3 Kilometers Downstream of Merrow Road (Near UConn Well #4)	Upstream of Depot Road and/or Coventry Road in Coventry	Total	Percent
American eel	*Anguilla rostrata*	7	4	11	0.7
Common shiner	*Luxilus cornutus*	140	196	336	20.0
Golden shiner	*Notemigonus crysoleucas*	0	19	19	1.1
Spottail shiner	*Notropis hudsonius*	0	23	23	1.4
Blacknose dace	*Rhinichthys atratulus*	65	2	67	4.0
Fallfish	*Semotilus corporalis*	158	273	431	25.7
Unidentified minnow	Cyprinidae	14	0	14	0.8
White sucker	*Catastomus commersoni*	176	137	313	18.7
Brown bullhead	*Ameiurus nebulosus*	0	2	2	0.1
Grass pickerel	*Esox americanus vemiculatus*	1	0	1	0.1
Chain pickerel	*Esox niger*	2	6	8	0.5
Brown trout	*Salmo trutta*	0	1	1	0.1
Rock bass	*Ambloplites rupestris*	0	12	12	0.7
Redbreast sunfish	*Lepomis aurites*	127	30	157	9.4
Green sunfish	*Lepomis cyanellus*	1	5	6	0.4
Pumpkinseed	*Lepomis gibbosus*	30	3	33	2.0
Bluegill	*Lepomis macrochirus*	1	2	3	0.2
Sunfish hybrid	*Lepomis* hybrid	10	0	10	0.6
Unidentified sunfish	*Lepomis* spp.	0	1	1	0.1
Smallmouth bass	*Micropterus dolomieu*	75	72	147	8.8
Largemouth bass	*Micropterus salmoides*	11	2	13	0.8
Tessellated darter	*Etheostoma olmstedi*	6	20	26	1.6
Yellow perch	*Perca flavescens*	31	11	42	2.5
Total taxa		17	20	23	
Total specimens		855	821	1676	

4.2 Materials and Methods

4.2.1 Project Scoping

A project scoping meeting was held on May 16, 2008 to discuss the selection of target species and HSC curves, field data collection methods, and the project schedule. This meeting was attended by representatives from the TAG. A field visit was conducted on June 12, 2008, during which the entire study area was walked to document/map aquatic habitat features and to determine the number and locations of transects for collection of hydraulic data. The field visit was attended by a selection of TAG representatives (DEP, MMI, ERC, and the Willimantic River Alliance).

During the field visit, aquatic mesohabitats were classified using the definitions in Table 4-2. The upstream and downstream coordinates of each habitat reach were determined using a 2005 Series Trimble GeoXT handheld Geographic Positioning System (GPS) receiver.

Table 4-2
Definitions of Aquatic Mesohabitat Types
(Source: Parasiewicz, 2007b)

Mesohabitat	Description of Characteristics
Riffle	Shallow stream reach with moderate current velocity, some surface turbulence, high gradient, and convex streambed morphology.
Rapid	Higher gradient reach than a riffle, with faster current velocity, coarser substrate, more surface turbulence, and convex streambed morphology.
Cascade	Stepped rapids with very small pools behind boulders and small waterfalls.
Glide	Moderately shallow stream channel with laminar flow. Lacks pronounced turbulence and exhibits flat streambed morphology.
Ruffle	Dewatered rapid in transition to either run or riffle
Run	Deeper stream reach with moderate current velocity but no surface turbulence (laminar flow). Streambed is longitudinally flat and laterally concave.
Fast run	Uniform fast-flowing stream channel.
Pool	Deep water impounded by a channel blockage or partial channel obstruction. Slow velocities with a concave streambed shape.
Plunge pool	Area where main flow passes over a complete channel obstruction and drops vertically to scour the streambed.
Backwater	Slack area along a channel margin caused by eddies behind obstructions, the development of sandbars during flood events, or through the abandonment of older channels.
Side arm	Channel around an island, smaller than half the width of the river, frequently at a different elevation than the main channel.

Dominant substrate type, the presence of aquatic habitat features such as undercut banks or woody debris, and adjacent riparian characteristics were recorded, and digital photographs of each reach were taken. In the present study, runs were further classified by water depth (shallow, moderate depth, or deep).

4.2.2 Selection of Target Species and Habitat Suitability Curves

Target species and HSC curves were selected cooperatively during the scoping process. Brook trout, brown trout, fallfish, and common shiner were selected as the target species. MMI and ERC decided to use the depth and velocity HSC curves for brook trout, brown trout, and fallfish developed for the Fenton River instream flow study (Warner et al., 2006) in the present study, with TAG concurrence. The Fenton River HSC curves were developed based on fish collections in the Fenton River and, because of small sample sizes, were composite curves for all life stages. Whereas the Fenton River study utilized a multivariate statistical function to describe substrate suitability, it was decided to use substrate HSC based on a simpler substrate coding system developed by Bovee (1978).

Common shiner was not considered in the Fenton River study, and HSC curves for this species (adults and juveniles) were obtained from the literature and modified as necessary in consultation with the DEP and with TAG concurrence. The final HSC curves selected for the Willimantic River study and their sources are summarized in Table 4-3 and presented in Appendix A.

Table 4-3
Target Species and HSC Curves

Species	Life Stage	Parameter	Source
Brook trout	Composite	Depth Velocity Substrate	Fenton River (Warner et al., 2006) Fenton River (Warner et al., 2006) Raleigh et al. (1986)
Brown trout	Composite	Depth Velocity Substrate	Fenton River (Warner et al., 2006) Fenton River (Warner et al., 2006) Raleigh et al. (1986)
Fallfish	Composite	Depth Velocity Substrate	Fenton River (Warner et al., 2006) Fenton River (Warner et al., 2006) modified from Trial et al. (1983b)
Common shiner	Adult	Depth Velocity Substrate	Trial et al. (1983a) modified from Trial et al. (1983a) modified from Trial et al. (1983a)
Common shiner	Juvenile	Depth Velocity Substrate	Trial et al. (1983a) Trial et al. (1983a) Trial et al. (1983a)

4.2.3 Transect Selection

PHABSIM habitat analysis relies upon assessment of hydraulic conditions (depth, velocity, etc.) measured along stream cross sections, or transects, placed at locations that represent the range of mesohabitats in the study area. The study area was described with a total of nine transects, two of which (Transects 16 and 17) were later combined for hydraulic analysis. Transect locations are shown on Appended Figure 1. The selection of transects over a range of representative habitats was performed in the field in consultation with the DEP.

4.2.4 Hydraulic Data Collection

A hydraulic data set was compiled for the instream flow study. Riverbed elevations, velocity and depth information, and substrate type along each transect were measured and characterized for use in PHABSIM.

The ends of each transect were marked with headstakes, and headstake locations and elevations were surveyed relative to a previously established baseline by MMI in the National Geodetic Vertical Datum of 1929. Distances along each transect were determined using a surveyor's tape affixed to a taut line set over the headstakes. The zero mark on the tape was set at the headstake on the right (looking downstream) side of the channel.

During the first field data collection event, riverbed elevations were surveyed using a Leica TPS1200+ Series Instrument total station, and substrate was characterized at two-foot intervals across each transect. Data was collected at intermediate stations where there was an abrupt change in elevation or substrate type. Substrate was visually classified using the Bovee code (Table 4-4).

Table 4-4
Bovee Substrate Codes Used in the Willimantic River Instream Flow Study
(Source: Bovee, 1978)

Code	Description	Size (inches)
1	organic/vegetation	----
2	mud/clay	----
3	silt	<0.002
4	sand	0.002-0.1
5	gravel	0.1-2.5
6	cobble	2.5-10
7	boulder	>10
8	bedrock	----

The Bovee code is recorded as x.y, where x is the code for the smaller of the dominant two adjacent-size substrate particle classes, and y is the decimal percentage of the larger.

Current velocities and water depths were measured across each transect on four additional dates to provide a range of flows for hydraulic modeling. Water surface elevations were measured at the ends of each transect using the total station. Velocities were measured with a calibrated Marsh-McBirney Flo-mate 2000 electromagnetic current meter according to USGS stream gaging practices (Buchanan and Somers, 1969; Rantz, 1982) (measured at 20% of depth and 80% of depth when the depth of water exceeded 2.5 feet and at 60% of depth for depths of water less than 2.5 feet). Depths were read from the wading rod of the current meter. Velocity and depth data were used to calculate river discharge according to USGS stream gaging practices (Buchanan and Somers, 1969).

4.2.5 Hydrologic Data Collection

A long-duration series of continuous daily streamflow data is necessary to perform a habitat time-series analysis (Section 4.2.8). Ideally, a period of record of greater than three decades is preferred. The USGS has collected discharge data at five locations on the Willimantic River as described below:

- The Coventry station (USGS Gage #01119500) has the longest period of record (1931 to present).
- The Mansfield Depot station (USGS Gage #01119384) has a record of 1991 to 2009, but the majority of this data is considered provisional.
- The Stafford Springs station (USGS Gage #01119280) has a record of 1962 to 1967.
- The Merrow station (USGS Gage #01119380) has a very limited period of record.
- The Merrow Road station (USGS Gage #01119382) came online in September 2009 to replace the function of the Mansfield Depot station.

The Coventry station has the longest period of record; therefore, this dataset was selected for further analysis. A series of adjustments were performed in order to correct the daily data for the Willimantic River at the Coventry gage to be realistic for a "natural" condition at the site of the wellfield. The adjustments include the following steps in this sequence:

1. Wellfield withdrawals were first added to each daily value of discharge at the Coventry gage to account for the loss of instream flow that occurs upstream of the Coventry gage.
2. Treated effluent discharges were next subtracted from each daily value to account for the gain in instream flow caused by the effluent outflow below Eagleville Lake (upstream of Route 275).
3. Finally, a watershed ratio factor was applied to scale down the Coventry daily data (after adjustment for withdrawals and wastewater outflows) to represent a "natural" flow condition at the Willimantic River Wellfield.

The three adjustments were used to create a 51-year dataset of daily mean flows extending from October 1, 1958 through September 30, 2008. This period includes the data approved for publication by the USGS for the Coventry station as updated through May 2010. Refer to Appendix B for detailed supporting documentation for the three corrections to the Coventry data set.

Adjustments for withdrawals and additions upstream of the wellfield, such as the CWC withdrawals for water supply and the Stafford Springs WPCF effluent discharges, were not included as these occur far upstream of the wellfield and are already considered a permanent component of the streamflow record at the Coventry gage.

4.2.6 Hydraulic Simulation

The current standard for calibration of the PHABSIM hydraulic models utilizes one complete set of measured velocities at the high target calibration discharge and water surface elevation/discharge measurements at an overall minimum of three calibration discharges. This combination of data allows development of stage-discharge rating curves and simulation of velocity patterns over a wide range of discharges (generally from 40% of the lowest target flow to 250% of the highest target flow) by extension of the rating curves. Hydraulic and aquatic habitat simulations were performed using RHABSIM 3.0 software (Thomas R. Payne & Associates, Arcata, CA), a commercial implementation of the USFWS' PHABSIM programs.

A hydraulic model for the study area was developed using the FIELDAT and HYDSIM modules of RHABSIM. A log-log stage-discharge relationship was developed for each transect by regressing the flow measured during five field data collection events against plotting stage (PS), where PS is defined as the water surface elevation minus the stage of zero flow (SZF) (i.e., the water surface elevation at which flow through the transect would cease).

Results from the log-log rating curve method (IFG4) were validated using an alternative method (channel conveyance Manning's stage-discharge, or MANSQ). Since the log-log rating curves met the calibration standards (A and B coefficients and percent error), provided results similar to MANSQ, and offered the flexibility to alter simulated discharges without reprocessing the model, the log-log method was selected for further hydraulic modeling.

Velocities along each transect were simulated in the HYDSIM module using the calibrated log-log rating curves and the velocities measured during the high flow field data collection effort. In this method, known velocities at all wetted vertical measurement points were used to compute a Manning's N value for each corresponding point based on velocity, depth, and energy slope. Depending on the accuracy of the discharge computed from the velocity and depth measurements (at each transect) and the best estimate of actual discharge, the computed Manning's Ns will reproduce the observed velocities. This method, commonly known as the one-velocity option, was then utilized to simulate velocities at flow intervals from 300 cfs (approximately 65 cfs higher than the high measured discharge) down to 10 cfs (about 40% of the low measured discharge).

4.2.7 Habitat Index Simulation

Habitat index computation is the process in the HABSIM module of RHABSIM that relates predicted velocity and depth and observed substrate at each vertical measurement point (i.e., the results of the hydraulic simulation) to the corresponding suitability values for these attributes (i.e., the habitat suitability criteria or HSC). The product of the suitabilities for each parameter (V x D x S) is weighted by the area each vertical represents, both in width along the transect and in length by mesohabitat type as a percentage of the study area.

The weighted values for all verticals are summed for each simulated discharge to give a single habitat index (weighted usable area, or WUA) for that discharge. WUA is expressed in square feet per thousand linear feet of stream. The pattern of WUA for all simulated discharges describes the relationship between physical habitat and stream discharge that can then be utilized to evaluate the potential impact of flow alteration.

4.2.8 Habitat Time Series/UCUT Analysis

A habitat time series was constructed for each target species from the natural discharge dataset created for the Willimantic River Wellfield based on the Coventry gaging station data for water years 1959 through 2008. As explained in Section 4.2.5, the Coventry gage records were corrected for (1) water withdrawn from the Willimantic River Wellfield, (2) outflows from the UConn WPCF, and (3) for watershed ratio.

Only discharges reported from July 8 through September 30 were included in the habitat time series. This time period, or bioperiod, corresponds to the same period used in the Fenton River Study and is generally considered the most sensitive time period in terms of the potential impacts of low flow on fish and other aquatic biota.

Habitat duration curves were constructed by determining the habitat area (WUA, derived from the habitat index simulation) that corresponded with each mean daily flow and then determining the percentage of time that the WUA was exceeded. WUAs for flows above and below the hydraulic simulation flow range (10 cfs to 300 cfs) were estimated using linear extrapolation, producing a range of WUAs for discharges ranging from five cfs to several hundred cfs dependent on species type. The linear extrapolation at the low end of instream flows is believed appropriate for this type of river with the combination of habitats observed in the field.

UCUT curves were developed using the methods originally proposed by Capra et al. (1995) and modified by Warner et al. (2006) in the Fenton River study. UCUT curves evaluate the duration and frequency of continuous events with habitat values (i.e., WUAs) lower than a specified threshold as a proportion of an entire bioperiod (Parasiewicz, 2008).

There are two types of habitat disturbances to consider during a time-series analysis. The first type includes "press" disturbances that occur during low-flow conditions and are defined by a reduction in WUA (available habitat). The second type includes "pulse" disturbances that occur during high flow events such as floods and primarily cause habitat stress through high velocities (Niemi et al., 1990; Parasiewicz 2007b). As this study is concerned with the effect of the Willimantic River Wellfield on low instream flows in the Willimantic River, it was decided to include only press disturbances (i.e., low-flow events) in the dataset used to generate UCUT graphs.

As will be discussed and demonstrated in Section 4.3.3, the WUA curves for each species (except juvenile common shiner) reach maximum WUA between 45 and 114 cfs. Discharges associated with press disturbances were defined as being equal to or lower than the discharge that generates maximum WUA. In order to correctly correlate the press disturbance data with the July 8 through September 30 bioperiod, the cumulative frequency curves were generated based on the number of discharges in the entire bioperiod data set. The benefit to only using press disturbances is that each percentage of maximum WUA can be directly correlated to one discharge (on the press side of the WUA curve) instead of two discharges (one press and one pulse discharge).

UCUT curves were constructed for each species by first assigning a WUA value to each mean daily flow in the bioperiod and then calculating the percentage of the maximum WUA that each daily value of WUA represented. A script was written in Microsoft® Excel to search sequentially through the percent maximum WUA values to test for various thresholds (e.g., 10%, 15%, 20% of maximum WUA, etc.) and then determine the continuous duration of events (in days) that WUA was below each threshold. The sum-length of all events of the same duration was then computed as a proportion of the total duration of the bioperiod hydrograph.

These proportions were plotted as a cumulative frequency, where the proportion of shorter periods was added to the proportion of all longer periods. The UCUT curves

(Appendix A) were interpreted using the criteria in Parasiewicz (2008) to identify "extreme," "rare," "critical," and "common" habitat stressor thresholds (HSTs) and durations of each particular HST per species.

4.3 Results

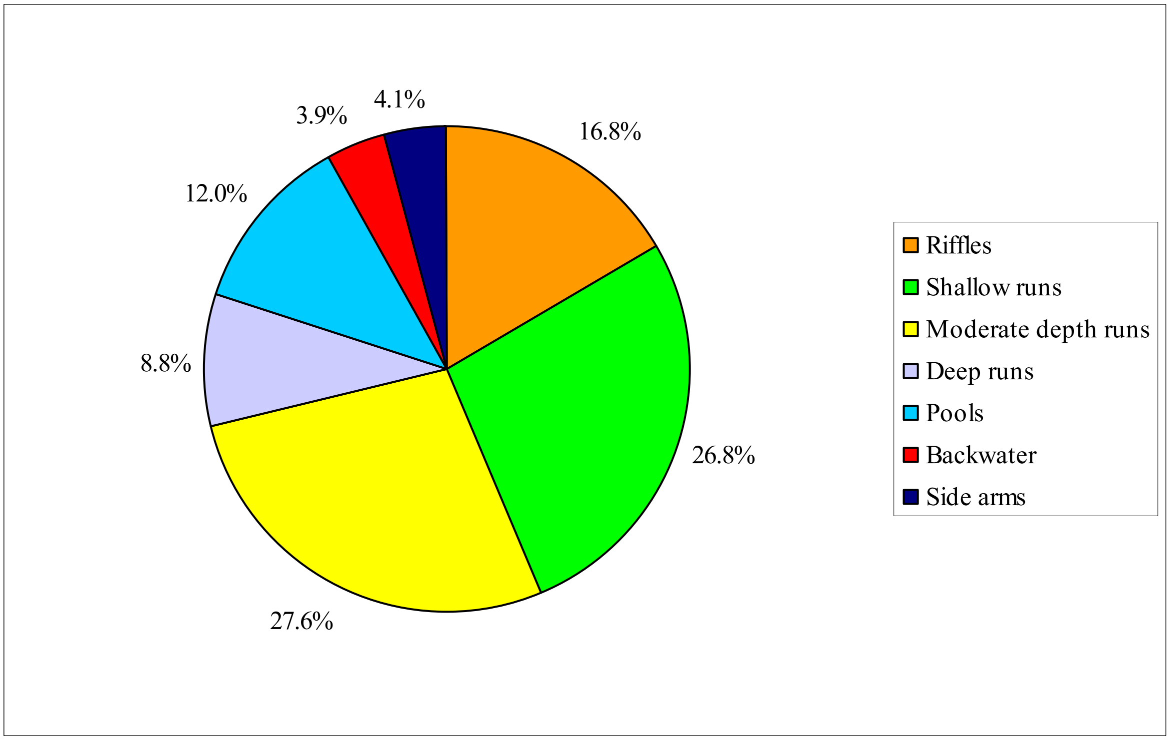
4.3.1 Habitat Mapping and Transect Selection/Weighting

The Willimantic River study area was mapped by mesohabitat type on June 12, 2008. A total of 60 habitat reaches were mapped (Appended Figure 1). Runs (63.2%) comprised the predominant mesohabitat type within the study area, followed by riffles (16.8%), pools (12.0%), side arms (4.1%), and backwaters (3.9%) (Table 4-5). The proportional distribution of mesohabitat types is illustrated in Figure 4-1.

Table 4-5
Summary of Mesohabitats Within the Study Area

Mesohabitat	Number of Reaches	Percent of Total Study Area Length	Percent of Total Study Area
Riffle	21	15.7	16.8
Shallow run	12	23.7	26.8
Moderate depth run	13	27.6	27.6
Deep run	4	11.2	8.8
Pools	3	10.3	12.0
Backwater	4	4.8	3.9
Side arm	3	6.7	4.1

Figure 4-1
Proportional Distribution of Mesohabitats Within the Study Area



Nine transects for hydraulic data collection were established (Table 4-6; Appended Figure 1). Transects 16 and 17 were combined for hydraulic analysis since they describe one cross section of river channel separated by a large island.

Table 4-6
Habitat Typing and Weighting of Transects

Transect No.	Mesohabitat Type	Percent Weighting
7	Riffle	8.99
8	Shallow run	14.63
14	Shallow run	14.63
16-17	Side arm (16), moderate depth run (17)	15.05
26	Pool	13.06
32	Moderate depth run	15.05
33	Riffle	8.93
34	Deep run	9.67

4.3.2 Hydraulic Data Collection and Simulation

Field Measurements

Hydraulic data were collected on five dates at river flows ranging from 26 cfs to 235 cfs (Table 4-7). Collection of data at lower discharges was not possible in 2008 or 2009 because of above-average precipitation maintaining instream flows in the summer and autumn.

Table 4-7
Dates of Hydraulic Data Collection and Measured River Flows

Date	Average Measured Discharge Through Study Area (cfs)
7/29/2008	235
10/24/2008	78
7/16/2008	43
9/5/2008	33
9/9/2009	26

Hydraulic simulations were initially performed over a flow range of 10 cfs to 300 cfs, with the low target flow based on the usual IFIM practice of limiting downward projections to 40% of the lowest measured flow. The low target flow was, however, reduced to five cfs in subsequent simulations. Extending the analyses downward to five cfs was justified because the relationship between WUA and flow for the target species was linear at the low end of the habitat response curve (discussed further in Section 4.3.4).

Stage-discharge regression curves for each transect are presented in Appendix A. Profiles of each transect showing riverbed elevations, simulated water surface elevations, and velocity patterns at flows from five to 300 cfs are also presented in Appendix A.

Time-Series Data Set for Natural Willimantic River Condition at Wellfield

Flow duration curves for the natural dataset at the wellfield described in Section 4.2.5 were constructed using methods presented by Searcy (1959). Flow duration discharges for particular percentages were calculated from the flow duration curves using both the entire natural wellfield data set (October 1, 1958 through September 30, 2008) and from a subset of data for the July 8 through September 30 bioperiod (data occurring in the months of July, August, and September). The results are provided in Table 4-8. The July 8 through September 30 bioperiod is a period of much lower discharges along the Willimantic River as compared to the year-round dataset.

Table 4-8
Flow Duration Statistics for Mean Daily Flow Data in Natural Dataset at Willimantic River Wellfield

Percentage of Time Indicated Discharge is Equaled or Exceeded	All Data (October 1, 1958 to September 30, 2008) Discharges, cfs	July 8 through September 30 (1959 to 2008) Discharges, cfs
1%	988	501
5%	509	213
10%	374	136
20%	263	85
30%	203	62
40%	159	48
50%	122	40
60%	92	33
70%	67	27
80%	45	22
90%	29	17
95%	21	14
99%	13	8.5

The minimum instream flow recorded on the Willimantic River in the corrected dataset was 3.2 cfs and occurred on October 1, 1978. The discharge the previous day (36 cfs) and the following day (22 cfs) suggest an anthropologic disturbance occurred that blocked or restricted flow to the Coventry gaging station on October 1, 1978. It is possible that the outlet of Eagleville Lake was temporarily restricted or otherwise blocked. The lowest instream flow in the natural dataset due to a sustained dry period is

6.0 cfs, which occurred on August 31, 1999. This discharge occurred during the drought period coincident with the Well #4 Safe Yield Test (Section 3.6).

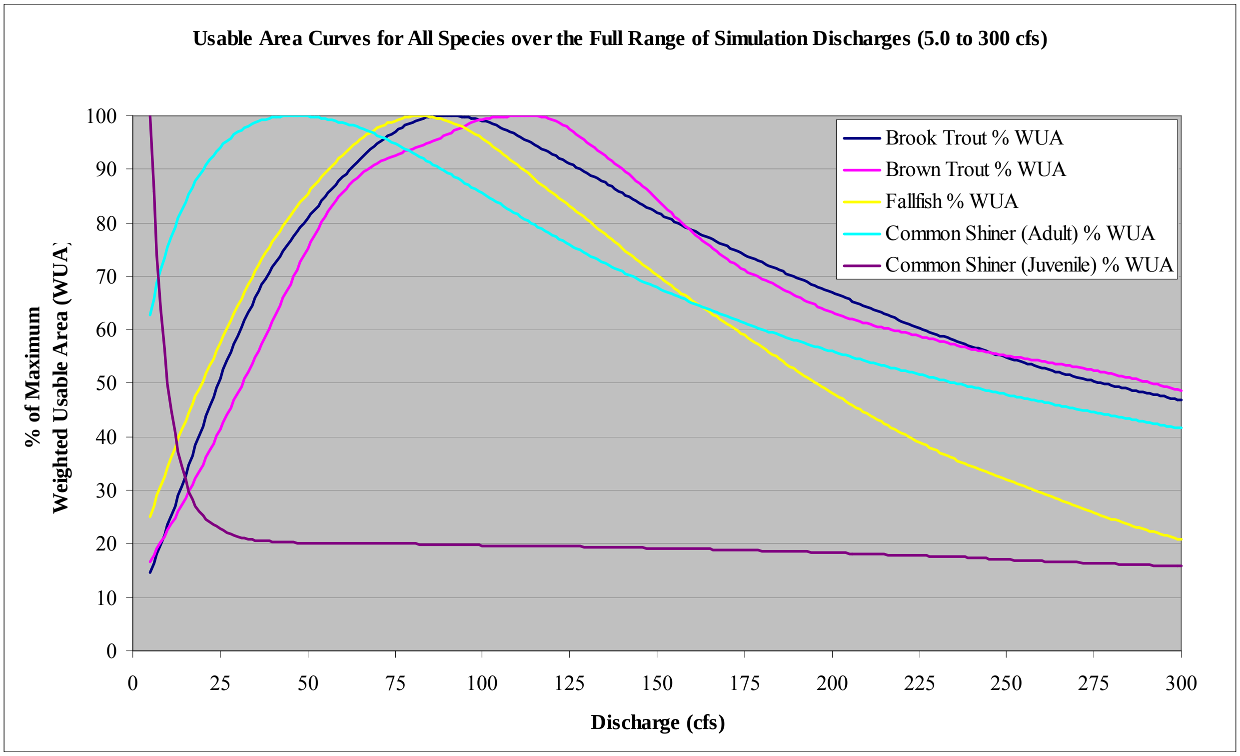
4.3.3 Habitat Index Simulation

The hydraulic modeling results were combined with the HSC curves for each target species to generate an index of relative habitat usability (i.e., WUA) for each simulated discharge. The results of habitat index simulations for each target species are summarized in Table 4-9 and presented graphically in Figure 4-2.

Table 4-9
Weighted Usable Area (ft²/1,000 ft) for Target Species

Simulated Discharge (cfs)	Total Wetted Area	Brook Trout	Brown Trout	Fallfish	Adult Common Shiner	Juvenile Common Shiner
5	61,930	4,529	3,971	4,747	13,636	2,548
10	65,833	7,323	5,379	6,475	16,400	1,272
15	68,776	10,233	6,807	8,099	18,224	814
20	70,389	13,114	8,314	9,503	19,518	644
25	71,546	15,898	9,882	10,845	20,416	583
30	72,449	18,382	11,492	12,149	21,046	544
35	73,022	20,568	13,069	13,367	21,436	527
40	73,402	22,367	14,705	14,419	21,637	521
45	73,798	23,882	16,361	15,336	21,712	517
50	74,172	25,209	17,951	16,118	21,699	514
60	75,715	27,607	20,417	17,467	21,439	510
70	76,384	29,552	21,725	18,418	20,933	509
80	77,009	30,828	22,380	18,841	20,212	510
90	77,202	31,195	23,016	18,676	19,408	507
100	77,357	30,922	23,695	18,062	18,567	502
110	77,499	30,047	23,830	17,130	17,709	500
120	77,627	28,959	23,669	16,155	16,873	498
130	77,750	27,858	22,672	15,187	16,101	496
140	77,869	26,691	21,459	14,199	15,393	492
150	77,992	25,557	20,115	13,238	14,742	489
160	78,106	24,527	18,677	12,349	14,138	486
170	78,217	23,578	17,474	11,526	13,573	482
180	78,357	22,643	16,590	10,702	13,057	477
190	78,512	21,748	15,809	9,865	12,583	473
200	78,644	20,893	15,077	9,096	12,153	467
225	78,893	18,829	14,032	7,352	11,221	453
250	79,205	17,113	13,155	6,018	10,398	434
275	79,313	15,718	12,509	4,862	9,688	418
300	79,415	14,626	11,616	3,917	9,027	402

Figure 4-2
Weighted Usable Area Curves for Target Species



Based on the data presented in Figure 4-2 and Table 4-9, brook trout, brown trout, fall fish, and adult common shiner demonstrated bell-shaped habitat response curves where WUA progressively increased with flow to maximum values at flows in the range of 45 to 115 cfs and then decreased as flow continued to increase. WUA for juvenile common shiner was low at all modeled flows and, because of the preference of this life stage for very low current velocities, was highest at the lowest modeled flow (five cfs). Maximum WUA occurred at the following flows:

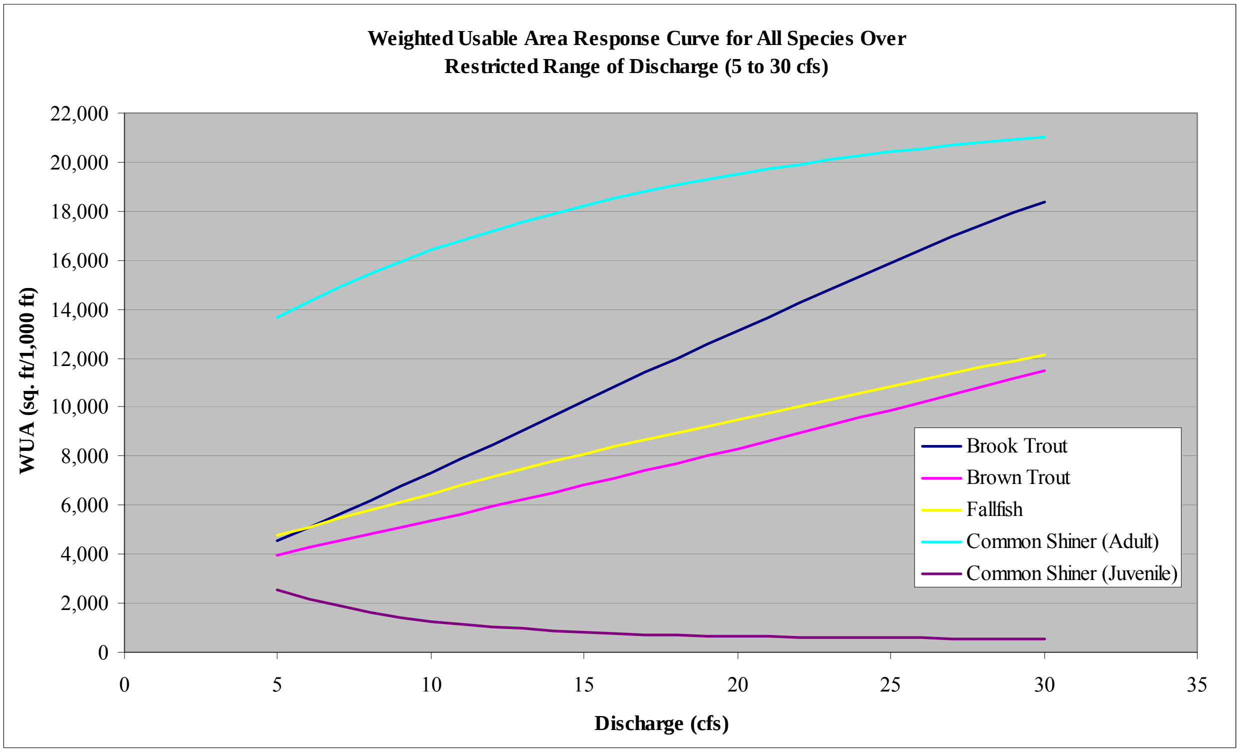
- 90 cfs for brook trout (WUA = 31,195 ft²/1,000 ft)
- 114 cfs for brown trout (WUA = 23,849 ft²/1,000 ft)
- 82 cfs for fallfish (WUA = 18,856 ft²/1,000 ft)
- 47 cfs for adult common shiner (WUA = 21,717 ft²/1,000 ft)
- 5 cfs for juvenile common shiner (WUA = 2,548 ft²/1,000 ft)

Since habitat response at low river flows is of primary interest in the Willimantic River instream flow study, WUAs were calculated in one cfs increments for flows in the range of five to 30 cfs (Table 4-10). Figure 4-3 shows that habitat responses over this restricted range of flows were essentially linear. This is consistent with the findings of the Fenton River instream flow study (Warner et al., 2006).

Table 4-10
Weighted Usable Area (ft^2/1,000 ft) Over a Restricted Range of Flows (5-30 cfs)

Simulated Discharge (cfs)	Brook Trout		Brown Trout		Fallfish		Adult Common Shiner		Juvenile Common Shiner	
	WUA	% Max WUA	WUA	% Max WUA	WUA	% Max WUA	WUA	% Max WUA	WUA	% Max WUA
5	4,529	15	3,971	17	4,747	25	13,636	63	2,548	100
6	5,085	16	4,261	18	5,107	27	14,301	66	2,187	86
7	5,639	18	4,544	19	5,459	29	14,915	69	1,888	74
8	6,198	20	4,824	20	5,801	31	15,458	71	1,632	64
9	6,758	22	5,101	21	6,138	33	15,949	73	1,433	56
10	7,323	23	5,379	23	6,475	34	16,400	76	1,272	50
11	7,894	25	5,659	24	6,813	36	16,810	77	1,143	45
12	8,467	27	5,938	25	7,152	38	17,196	79	1,040	41
13	9,048	29	6,221	26	7,483	40	17,559	81	949	37
14	9,636	31	6,513	27	7,799	41	17,900	82	878	34
15	10,233	33	6,807	29	8,099	43	18,224	84	814	32
16	10,828	35	7,106	30	8,396	45	18,522	85	759	30
17	11,409	37	7,405	31	8,687	46	18,798	87	719	28
18	11,981	38	7,704	32	8,966	48	19,057	88	687	27
19	12,549	40	8,007	34	9,237	49	19,298	89	663	26
20	13,114	42	8,314	35	9,503	50	19,518	90	644	25
21	13,680	44	8,623	36	9,773	52	19,722	91	628	25
22	14,253	46	8,935	37	10,043	53	19,913	92	614	24
23	14,809	47	9,250	39	10,310	55	20,091	93	603	24
24	15,354	49	9,565	40	10,579	56	20,258	93	592	23
25	15,898	51	9,882	41	10,845	58	20,416	94	583	23
26	16,426	53	10,202	43	11,112	59	20,562	95	574	23
27	16,935	54	10,523	44	11,375	60	20,699	95	566	22
28	17,433	56	10,841	45	11,635	62	20,825	96	557	22
29	17,913	57	11,165	47	11,893	63	20,941	96	550	22
30	18,382	59	11,492	48	12,149	64	21,046	97	544	21

Figure 4-3
Weighted Usable Area Over a Restricted Range of Flows (5-30 cfs)



Examination of the percent of maximum WUA values in Table 4-10 shows that the two trout species are the most sensitive to low flows, with brook trout being slightly more sensitive than brown trout at the extreme low end (five to eight cfs) of the curve. However, even at these extremely low flows, the WUA for brook trout ranged from 15% to 20% of maximum WUA (Table 4-10), percentages that are much higher than the habitat percentages seen for very low discharges in the Fenton River Study.

4.3.4 Habitat Time Series/UCUT Analysis

Habitat time series/UCUT analyses were conducted for brook trout, brown trout, fallfish, and adult common shiner. UCUT graphs for these species are presented in Appendix A. To be conservative, this analysis was not performed for juvenile common shiner since it demonstrated low WUAs at all modeled flows and an inverse WUA to discharge relationship. Extreme, rare, critical, and common HSTs or habitat "events" were identified from the curves based on the criteria presented below (based on Parasiewicz, 2008):

a. An "extreme" event is an indicator of natural maximum environmental stress and is the lowest possible amount (>0) of habitat occurring under natural conditions.

b. "Rare" events are infrequent and occur for only a short duration. They are usually in the lower-left corner of the UCUT graph with adjacent lines of constant percent WUA being very close to each other. The "rare" habitat curve is chosen as the highest curve (in terms of frequency of the lines of equal percent of maximum WUA) of this group.

c. The next highest curve after the "rare" curve is the "critical" curve. This habitat event occurs more frequently than the "rare" habitat, with a rapid decrease to the "rare" threshold compared to the next highest percent WUA curve. It is generally the first line to "stand out" (be more separated from its neighbors) on the graph. Lines of equal percent of maximum WUA to the right (higher frequency of occurrence) are more separated apart than the "rare" group but generally still spaced together.

d. The next outstanding curve demarcating a rapid reduction in frequency of events is assumed to mark the stage at which "common" habitat levels occur, or those that occur (or have the potential to occur) nearly every year.

e. The length of events is also considered. The "shortest" common duration is the first inflection point on each curve and is generally one to two days. The "persistent" event duration is likely to occur every few years and no more than twice in one year. It is marked by the main inflection point on the associated percent of maximum WUA curve.

Because the UCUT graph *interpretation* is affected by (1) the interval between lines of equal percentage of maximum WUA (e.g., plotting of lines that represent a difference of four percentage points of maximum WUA versus plotting of lines that represent a difference of two percentage points of maximum WUA) and (2) the scale of the x-axis (which can cause crowding or spacing of plotted lines), several sets of UCUT graphs were prepared for each species. However, it is important to note some of the observations that resulted from the examination of different plot intervals and scales:

- Plotting of the extreme event was relatively straightforward per criteria (a) above. The extreme event represents 19% of maximum WUA for brook trout and 20% of maximum WUA for brown trout. The percentage of maximum WUA is higher for fallfish and much higher for adult common shiner. These lines are clear and apparent whether the interval between lines of equal percentage of maximum WUA is 2% or 4% and whether the x-axis is compressed (zero to 30% cumulative continuous duration) or expanded (zero to 60% cumulative continuous duration). Therefore, selection of the extreme threshold for each species has very little uncertainty.

- Selecting the rare threshold for brook trout, brown trout, and fallfish was slightly more challenging than selecting the extreme threshold. Ultimately, the rare and extreme thresholds are separated by only four percentage points of maximum WUA for brown trout and by eight percentage points of maximum WUA for brook trout and fallfish. Given the close proximities of the extreme and rare event thresholds for each of these three species, the differences are more easily observed when intervals of two percentage points of maximum WUA are plotted and the x-axis focuses on an interval of zero to 30% cumulative continuous duration. The use of the two different plots for each species resulted in the selection of rare thresholds with low uncertainty.

- The spread between the extreme and rare events for adult common shiner is much greater (13 percentage points of maximum WUA) as compared to the other species.

This is consistent with the fact that the percentage of maximum WUA is much higher overall for this species at the low river flows.

- The critical and rare thresholds plot adjacent in the graphs in Appendix A for all species as required by criteria (c) above. For brook trout, brown trout, and fallfish, the spacing represents a difference of four percentage points of maximum WUA between the rare and critical thresholds. The spacing is only a difference of two percentage points of maximum WUA for adult common shiner. Also note the following:

  - Recall that the critical threshold is generally the first line to stand out on the UCUT graph. However, given the overall close spacing, it can be difficult to know whether a line is really "standing out." This is where examination of the different sets of graphs was helpful.
  - When the x-axis ranges from zero to 30% cumulative continuous duration and the plotted lines represent WUA intervals of only two percentage points instead of four percentage points, the critical threshold is much easier to delineate for brook trout, brown trout, and fallfish.
  - It is noted that with the additional lines plotted the rare and critical thresholds are no longer adjacent. However, it is also apparent that this additional line of equal percent of maximum WUA is much closer to (and partially overlaps) the rare threshold line for brook trout, brown trout, and fall fish. As the intermediate line is much more similar to the rare threshold line than the selected critical threshold line, it must not represent the critical threshold.
  - The use of the different UCUT plots for each species provides a low uncertainty to the selection of the critical thresholds.

- The common threshold for each species is meant to be the next line that stands out as the lines of equal percent duration on the UCUT graph become similar in shape and separation along the x-axis (criteria d). The common event was the most challenging to select because there appeared to be two distinct groups of lines in the region to the right of the critical threshold on the UCUT graphs for each species. The common event could be selected from the left edge of either of these two groups as noted below:

- When the x-axis is condensed (ranging from zero to 30% of cumulative continuous duration) and the lines of equal percentage of maximum WUA are separated by intervals of only two percentage points instead of four percentage points, the common threshold appears to be represented by discharges of 19 cfs for brown trout and fallfish, 17 cfs for brook trout, and 18 cfs for adult common shiner.
- When the x-axis is expanded (ranging from zero to 60% of cumulative continuous duration) and the lines of equal percentage of maximum WUA are separated by intervals of four percentage points, the common threshold appears to be represented by discharges of 25 cfs for brook trout and fallfish and 27 cfs for brown trout.
- Examination of the spacing of the lines of equal percentage of maximum WUA beyond the two potential common events for each species ultimately points to the higher set of choices for the common event. These discharges (25 cfs to 27 cfs for brown trout, brook trout, and fallfish) could be described as realistic if not slightly conservatively high given that the flow in the Willimantic River drops below 30 cfs annually, even if only for a brief period.
- More importantly, the UCUT curves appear to imply that the common event has two "subregions" to the right of the critical event. This has important implications for river management because the "higher" common threshold could represent a preliminary warning whereas the "lower" common event threshold could represent that actions should be taken to begin protecting fish habitat.

- Overall, the three lowest thresholds (extreme, rare, and critical) plot very close to each other for habitats of brook trout, brown trout, and fallfish in the Willimantic

River. In addition, the nature of the lines of equal percentage of maximum WUA changes significantly in the region from the rare threshold to some point just prior to the common threshold for each species. It is clear that fisheries habitat in the river is sensitive to changes in discharges between approximately 10 cfs and 20 cfs, just below the two subregions of the common event.

Based on the UCUT analysis, the percent of maximum WUA, discharge (flow), and the persistent duration for common, critical, rare, and extreme habitat thresholds for each target species are presented in Table 4-11. Discharge and duration values for the fish community as a whole are also provided. These are based on the highest flow in a given habitat threshold category and the lowest persistent duration, consistent with the approach used in the Fenton River instream flow study. Adult common shiner was not used to generate values for the fish community as the percentage of maximum WUA for this species was extremely high even at the extreme threshold.

Table 4-11
Percent of Maximum WUA, Discharge, and Persistent Duration of Common, Critical, Rare, and Extreme Habitat Thresholds

Habitat Stressor Threshold	Parameter	Brook Trout	Brown Trout	Fallfish	Adult Common Shiner	Fish Community
Common (Upper Subregion)	Habitat (% Max WUA)	51%	44%	57%	88%	
	Discharge (cfs)	25	27	25	18	27
	Persistent Duration (days)	19	19	19	19	19
Common (Lower Subregion)	Habitat (% Max WUA)	37%	34%	49%	88%	
	Discharge (cfs)	17	19	19	18	19
	Persistent Duration (days)	19	19	19	19	19
Critical	Habitat (% Max WUA)	31%	28%	41%	84%	
	Discharge (cfs)	14	15	14	15	15
	Persistent Duration (days)	15	15	13	16	13
Rare	Habitat (% Max WUA)	27%	24%	37%	82%	
	Discharge (cfs)	12	11	11	14	12
	Persistent Duration (days)	12	12	12	13	12
Extreme	Habitat (% Max WUA)	19%	20%	29%	69%	
	Discharge (cfs)	7.5	7.8	7.0	7.1	7.8
	Persistent Duration (days)	7	7	7	7	7

4.3.5 Hydrologic Dataset Analysis

The natural dataset for the Willimantic River at the wellfield was reviewed to see how often the common, critical, rare, and extreme instream threshold flows might occur. The threshold flows of 27, 15, 12, and 7.8 cfs were equaled or exceeded 90.8%, 98.1%, 99.2%, and 99.9% of the time respectively in the full data set and were equaled or exceeded 70.4%, 93.1%, 96.8%, 99.4% of the time during the June 8 through September 30 bioperiod. Thus, it is expected that during the months of July through September the common threshold would naturally occur approximately 30% of the time, the critical threshold would trigger 7% of the time, the rare threshold would trigger 3% of the time, and the extreme threshold would trigger 0.6% of the time.

The rearing and growth bioperiod data (July through October) in the corrected dataset was then reviewed to determine the frequency that the persistent durations were equaled or exceeded for consecutive days. The persistent duration for the common threshold (19 days) was equaled or exceeded 11 times in the 51 years of record in the natural wellfield dataset. The persistent duration for the critical threshold (13 days) was equaled or exceeded six times. The persistent duration for the rare threshold (12 days) was exceeded three times while the persistent duration for the extreme threshold (seven days) was equaled only twice. The persistent durations were therefore equaled or exceeded rarely, having only occurred (for the June 8 through September 30 bioperiod) in 22%, 12%, 6%, and 4% of the years of record, respectively.

From a wellfield management perspective, the University is concerned with knowing (1) how much time it will take for the Willimantic River to recess from trigger to trigger at baseflow conditions, and (2) how long it will take for the river to recede between triggers following a rain event when flows are already low (important for operational flexibility). The June 8 through September 30 bioperiod data in the natural wellfield dataset were then reviewed to determine the amount of time it takes for the Willimantic River to "recess" or decrease from one streamflow trigger to the next. Results are described below:

- A total of 38 recession curves was reviewed that receded from approximately 27 cfs to 19 cfs or below and that were generally unaffected by rainfall. A mean of 4.0 days was observed to recede from the "upper subregion" common threshold flow (27 cfs) to the "lower subregion" common threshold flow (19 cfs), with a maximum of 11 days and a minimum of two days. One recession curve was extended out to 15 days by light rainfall but was not included in the average above. In general, when the river was already at low flows and affected by a rain event, it took only two to three days to recede back below 19 cfs but took five to six days when the river was receding without a rainfall event.

- A total of 30 recession curves were reviewed that receded from approximately 19 cfs to 15 cfs or below and that were generally unaffected by rainfall. A mean of 3.3 days was observed to recede from the "lower subregion common" threshold flow (19 cfs) to the critical threshold flow (15 cfs), with a maximum of seven days and a minimum of two days. One recession curve was extended out to 11 days by light rainfall but was not included in the average above. Similar to the first set of recession curves, it took only two to three days for the river to recess after a rainfall event when the river was already at low flow but took four to six days when the river was receding without a rainfall event.

- A total of 12 recession curves was reviewed that receded from approximately 15 cfs to 12 cfs or below and that were generally unaffected by rainfall. A mean of 4.1 days was observed to recede from the critical threshold flow (15 cfs) to the rare threshold flow (12 cfs), with a maximum of nine days and a minimum of one day. Similar to the above sets of recession curves, it took only three days for the river to recess after a rainfall event when the river was already at low flow but took four to five days when the river was receding without a rainfall event.

- Only four recession curves were available showing a recession from 12 cfs to 7.8 cfs or below:

    - August 24 through August 31, 1995: seven days for flows to recede, but they were buttressed by a light rainfall event.
    - August 2 through August 7, 1999: five days for flows to recede following a light rain event, and flows were already very low at the beginning of the 1999 drought.
    - August 18 through August 20, 1999: only two days for flows to recede following a light rain event well into the 1999 drought.
    - August 20 through September 2, 2007: 13 days for flows to recede from 12 cfs (the rare threshold flow) to below 7.8 cfs (the extreme threshold flow). This recession is not believed to be influenced by a rain event.

In summary, the University can expect to have a buffer of at least several days between triggers for the baseflow condition:

- The Willimantic River is expected to take five or six days to recede from 27 cfs to 19 cfs during baseflow while providing two to three days of flow above 19 cfs following a rain event.

- The river is expected to take four to six days to recede from 19 cfs to 15 cfs during baseflow while providing two to three days of flow above 15 cfs following a rain event.

- The river is expected to take four or five days to recede from 15 cfs to 12 cfs during baseflow while providing three days of flow above 12 cfs following a rain event.

- Although there is limited data regarding recessions from 12 to 7.8 cfs, it is believed that the river will require at least one week to recede during baseflow conditions while providing two to five days of flow above 7.8 cfs following a rain event.

4.4 Recommendations

The discharge and duration thresholds in Table 4-11 can be used to set triggers for water conservation or flow augmentation events, thus resulting in modified withdrawals from the Willimantic River Wellfield. The thresholds in Table 4-11 have been tied to the drought trigger levels in the draft Drought Response Plan (dated August 22, 2008) of the UConn Water Supply Emergency Contingency Plan. The drought trigger levels are currently divided into five stages based on projected available supply and levels in the High Head Reservoir. A copy of the draft Drought Response Plan is included in Appendix C.

Note that during drought periods the Willimantic River Wellfield will likely be the sole source of water supply to the University as the Fenton Wellfield will most likely be shut down. As such, setting specific cutbacks to wellfield production is not feasible except through reduction of demand by conservation measures. Otherwise, even the most critical demands of water users may not be able to be met. Thus, the following recommendations focus on water conservation as opposed to specific cutbacks in production.

Recall from Section 4.3.4 that the amount of useable fisheries habitat in the river is sensitive to changes in flows between 10 cfs and 20 cfs and that the common threshold appeared to have two "subregions" on the UCUT graphs to the right of the critical event. It was noted that this has important implications for river management because the "higher" common threshold could represent a preliminary warning whereas the "lower" common threshold could represent the level at which actions should be taken to begin protecting fish habitat. Taking these findings into account, the first trigger for management of wellfield withdrawals (the "lower subregion of the common habitat threshold) should be at the high end of this range.

It is important to ensure that the five stages of drought response can only be activated in order as outlined in the draft Drought Response Plan and that the potential exists for multiple days of recession between each trigger discharge to allow water system operators and University administrators time to implement pumping reductions and conservation measures. The following management response combines the habitat threshold discharges and persistent durations in Table 4-11 with the drought trigger levels in the draft Drought Response Plan. Management response is proposed based on the following schedule:

Drought Management Begins: The "upper subregion" common threshold event (27 cfs) should serve as a cautionary condition where the water system operators would prepare to implement pumping reductions and/or the University would prepare to implement conservation measures and the drought response plan.

Stage IA: Should the discharge in the Willimantic River fall below 27 cfs for 19 days (the persistent duration of the common habitat threshold), OR if the discharge in the Willimantic River falls below 19 cfs (the "lower subregion" common habitat threshold), it would trigger *Stage 1A – Water Conservation Alert.*

Stage IB: Should the discharge in the Willimantic River fall below 15 cfs (the critical habitat threshold), it would trigger *Stage IB – Water Supply/Drought Advisory*.

Stage II: Should the discharge in the Willimantic River fall below 15 cfs for 13 days or more (the persistent duration of the critical habitat threshold), OR if the discharge in the Willimantic River falls below 12 cfs (the rare habitat threshold), it would trigger *Stage II – Water Supply/Drought Watch*.

Stage III: Should the discharge in the Willimantic River fall below 12 cfs for 12 days or more (the persistent duration of the rare habitat threshold), OR if the discharge in the Willimantic River falls below 7.8 cfs, it would trigger *Stage III – Water Supply/ Drought Warning*.

Stage IV: *Stage IV – Water Supply/Drought Emergency* would trigger if the discharge in the Willimantic River falls below 7.8 cfs for seven or more days.

Table 4-12 provides a summary of the above recommendations. Because the response stages outlined in the Drought Response Plan currently are linked to projected available supply and reservoir levels, the Drought Response Plan will need to be rewritten to include the above provisions along with appropriate response and recovery guidelines. The analysis in Section 4.3.5 will prove useful in that future plan update with regard to appropriate recovery guidelines.

Table 4-12
Recommended Willimantic River Drought Trigger Levels and Corresponding Drought Management Response

Drought Response Stage	Willimantic River at Wellfield Trigger Discharge	Habitat Stressor Threshold	Examples of Conservation Measures
Prepare for implementation of Stage IA	Discharge ≤ 27 cfs	Common (Upper Subregion)	None / Preparation for Stage IA
Stage IA (Two potential triggers)	Discharge < 27 cfs for 19 or more days	Persistent Duration of Common (Upper Subregion)	Voluntary: Shorter showers, condensed washing loads, elimination of nonessential consumption, raise thermostats on centrally chilled buildings
	Discharge < 19 cfs	Common (Lower Subregion)	
Stage IB	Discharge < 15 cfs	Critical	
Stage II (Two potential triggers)	Discharge < 15 cfs for 13 or more days	Persistent duration of Critical	Voluntary items above become mandatory and include (but are not limited to) the following mandatory items: No flushing of hydrants, pipes, or sewer lines; no vehicle fleet washing; no use of water for street sweeping; reduce irrigation by 50%; reduce operation of research equipment cooled with domestic water; import water needed for construction dust control; no pool filling; raise thermostats of centrally chilled buildings
	Discharge < 12 cfs	Rare	
Stage III (Two potential triggers)	Discharge < 12 cfs for 12 or more days	Persistent duration of Rare	
	Discharge < 7.8 cfs	Extreme	
Stage IV	Discharge < 7.8 cfs for 7 or more days	Persistent duration of Extreme	

The above recommendations are similar to how the University managed its water supply during the drought of 2007, when hydraulic and hydrogeologic limitations caused the University to take conservation measures. The 2007 conservation measures reduced production approximately 10% as compared with 2006 production for the months of July through November when the Fenton River Wellfield was offline. This information is summarized in Table 4-13.

Table 4-13
Comparison of 2007 Drought Period Production Data With 2006 Production Data

Month	2006 Production (gal)	2007 Production (gal)	Comparative Reduction (2006 to 2007)
July	42,516,000	37,358,000	12.1%
August	45,066,000	40,339,000	10.5%
September	49,683,000	46,694,000	6.0%
October	49,185,000	45,348,000	7.8%
November	41,928,000	36,600,000	12.7%
Total	**228,378,000**	**206,339,000**	**9.7%**

Final recommendations regarding discharge triggers are discussed in Section 9. Recall that the trigger discharges discussed in this section are based on a "natural" condition dataset for the section of the river flowing past the wellfield. Discharge triggers will be compared to the USGS-measured discharges at the Merrow Road gaging station to determine when a trigger is reached, similar to the method that the Old Turnpike Road station is utilized by the University on the Fenton River. Operationally, this means that the University may wish to make an adjustment to the triggers that is approximately equal to the wellfield withdrawal rate. As an alternative, the daily discharges at the Merrow Road station could be adjusted for direct comparison to the triggers. The decision has operational implications and need not be determined as part of this study.

5.0 SUPPLEMENTAL HYDROGEOLOGIC STUDY

5.1 Background

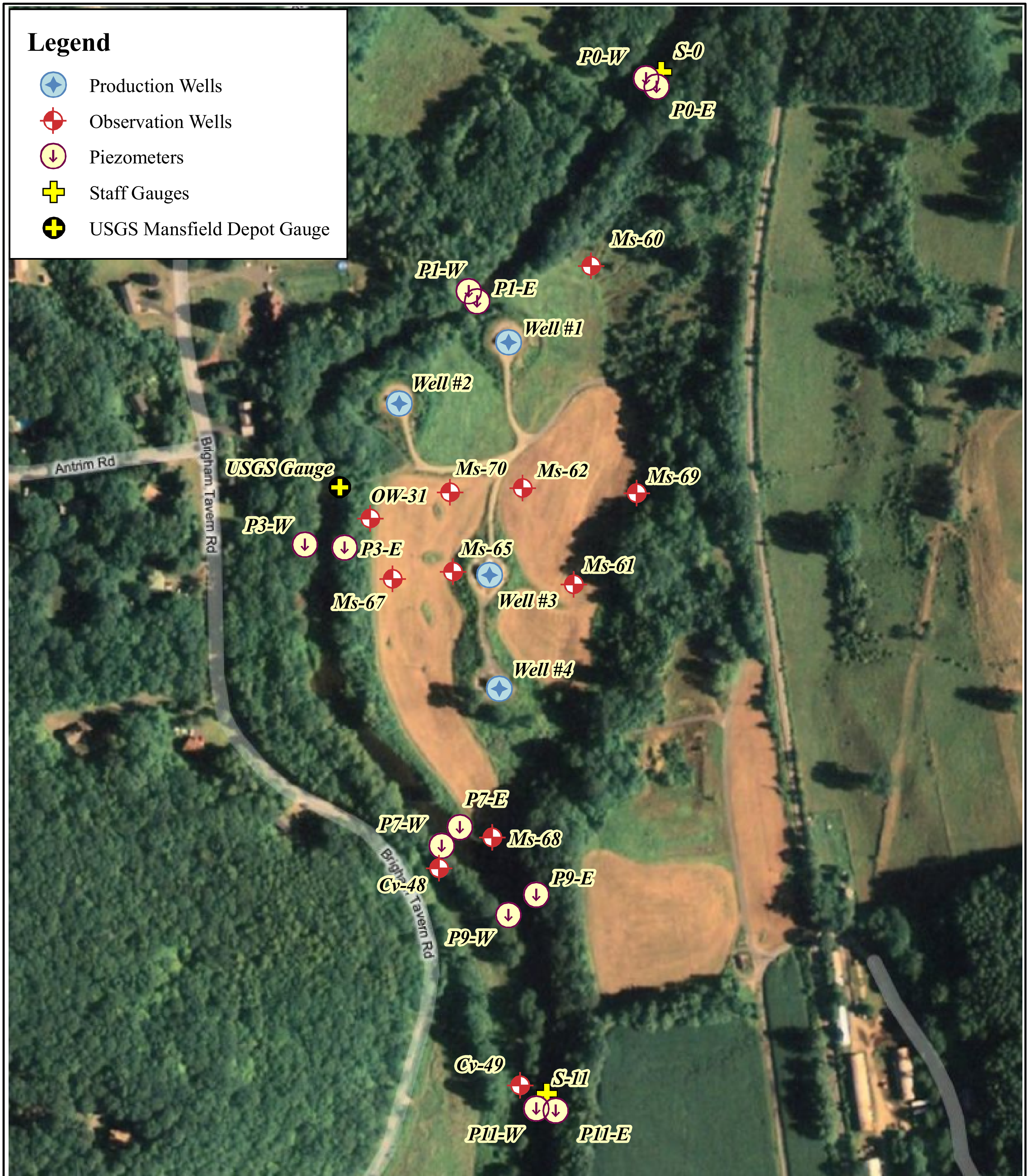
In order to provide additional hydrogeologic information to support the updates to the approved Level A model, the University conducted three 72-hour pumping tests in 2008 and 2009. Protocols for monitoring were discussed with the TAG in summer 2008. The original objective was to collect data during three different combinations of river flow regime and wellfield operation as described below:

1. Low wellfield operation (on the order of 1.0 mgd) and low to moderate river flow
2. Moderate wellfield operation (on the order of 1.5 mgd) and low to moderate river flow
3. High wellfield operation (on the order of 2.0 mgd) and low river flow

While these exact combinations of pumping and instream flows could not be met, the tests were completed under conditions that differed from one another. One monitoring event was conducted in August 2008, with the remaining two events conducted in September and November 2009.

5.2 Monitoring Network

The monitoring network for the 2008 and 2009 supplemental hydrogeologic monitoring included available observation wells present at and near the wellfield, 12 piezometers divided into six piezometer pairs (one at each bank), and temporary gaging stations that were installed for this study upstream and downstream of the wellfield. The monitoring network is depicted on Figure 5-1 and discussed in more detail throughout Section 5.2.



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MMI#: 1958-09
MXD: H:\Figure5-1.mxd
SOURCE: Microsoft Virtual Earth

N

Willimantic River Study

Map By: SJB
Date: Feb. 2010
Scale: 1"=300'

SHEET: Figure 5-1

Automatic dataloggers were monitored in two piezometer pairs (total of four) near Well #1 and between Well #2 and Well #3, and in one observation well (Ms-65) located between Well #3 and the river. A datalogger ("barologger") was also set to record barometric pressure (to correct the datalogger data at the four piezometers for barometric pressure) and air temperature. A graph of the air temperature data is presented in Appendix D. The dataloggers were set to record continuously throughout the monitoring periods, including antecedent and post-test conditions.

All elevations associated with the monitoring network were surveyed by MMI in 2009 relative to the Connecticut State Plane North American Datum of 1983 and the National Geodetic Vertical Datum of 1929. Piezometers were not surveyed in 2008. Instead, riser height information was correlated to streambed elevation at each piezometer in 2009 to estimate water elevations at the piezometers in 2008.

5.2.1 Selection of Observation Wells

As described in Section 3.0, many observation wells have been installed for various hydrogeologic studies at the Willimantic River Wellfield. Many of these observation wells are at the sides of the maintained fields buried in deep underbrush. In 2008, several observation wells were found accessible, and more were discovered and utilized in 2009. Table 5-1 summarizes these monitoring wells.

Many of the wells have names provided by the USGS during the Level A Study in the 1990s while others are monitoring wells from unknown studies that are recorded as labeled in the field. The observation wells located above are generally all located in the vicinity of the Willimantic River Wellfield. While additional observation wells were utilized further afield to the south during the 1999 pumping test, limited or no drawdowns occurred at these observation wells due to pumping at the wellfield. Thus, these observation wells were not monitored in connection with the pumping tests completed for this study.

Table 5-1
Observation Wells Utilized for Supplemental Hydrogeologic Monitoring

Observation Well	Location	Utilized in 2008	Utilized in 2009	Monitoring Point Elevation (ft NGVD 1929)	Depth to Bottom of Well (ft)
Cv-48	SW Field - N	X	X	296.47	32.3
Cv-49	SW Field - S	X	X	295.53	36.2
Ms-60	NW Corner	X	X	301.71	23.0
Ms-61	East of Well #3	X	X	298.29	36.7
Ms-62	North of Well #3		X	297.98	35.2
Ms-65	West of Well #3	X	X	297.14	56.3
Ms-67	West of Well #3	X	X	297.15	53.1
Ms-68	SW of Well #4	X	X	297*	51.1
Ms-69	NE of Well #3		X	300.40	29.5
Ms-70	NW of Well #3		X	296.3	78.6
OW-31	Near USGS Gage		X	297.21	37.2

Note: Monitoring Point Elevation from top of PVC or metal pipe inside outer casing. A "*" means elevation was estimated.

Data collection at Ms-68 was problematic because water levels were more representative of the levels in the river than in the aquifer below. It is not known if there is a discontinuous unit of finer materials buttressing the water table at this location, or if the bentonite seal around the well and above the well screen has failed, allowing water from the upper layer to penetrate the aquifer in this location. Therefore, data collected at this well in 1999, 2008, and 2009 was not used for model calibration (refer to Section 6.4).

5.2.2 Riverbed Piezometers

Piezometer pairs were manually driven into the riverbed adjacent to the Willimantic River Wellfield in an effort to monitor surface water/ground water recharge and discharge conditions. Each piezometer was 1.25 inches in diameter and consisted of a two-foot long screen connected to a variable length riser pipe by a metal coupling. Although the two-foot long screen allows for an averaging of potentiometric head over the exposed interval of aquifer, it is considered an appropriate length for two reasons. First, despite the averaging, the head over the interval is sufficiently different than the head in the surface water body in gaining or losing stream conditions. Second, the two-foot long screen protects the integrity of the project from complications such as siltation of the bottom section of the screen inside the pipe or clogging of part of the screen. If a shorter screened section were used, clogging and/or siltation could render a piezometer unusable.

One piezometer was driven into the riverbed near each bank as shown on Figure 5-1. The location of each pair was selected to assist in delineating the areas of induced infiltration. Piezometers were identified by the habitat reach they were installed in as delineated for the Instream Flow Study (Section 4.0). For example, the upstream-most piezometer pair, P0, was installed upstream of Reach 1 at the northernmost wellfield property line. The piezometers are summarized in Table 5-2.

Table 5-2
Piezometers Installed for Supplemental Hydrogeologic Monitoring

Piezometer	Location	Exposed Height in 2008 (ft)	Full Length of Piezometer (ft) in 2008	Exposed Height in 2009 (ft)	Full Length of Piezometer (ft) in 2009
P0-W	Near northern property line	3.8	6.7	3.3	7.7
P0-E		4.0	6.6	3.1	6.8
P1-W	Near Well #1	3.7	6.6	3.4	7.7
P1-E		4.0	6.7	3.6	6.5
P3-W	Between Well #2 and Well #3	3.7	6.7	3.0	7.7
P3-E		3.7	6.7	2.8	8.8
P7-W	Near Well #4	3.7	6.8	3.1	7.7
P7-E		3.9	6.7	3.2	7.7
P9-W	Downstream of riffle near Well #4	3.4	6.7	3.6	6.8
P9-E		3.4	6.7	3.1	6.7
P11-W	Downstream of wellfield	2.7	5.7	3.1	6.7
P-11E		4.0	6.7	3.3	6.8

Note: "Exposed height" is the height of pipe above the riverbed. "Full Length" is depth to bottom of inside of piezometer as measured from the top.

The piezometers installed during the 2008 monitoring were all destroyed during the high flows of early September 2008 and were reinstalled in the same locations for the 2009 monitoring events. Longer riser pipes were used in some of the locations in 2009 in an attempt to achieve deeper penetration to prevent the insides of the piezometers from going dry where drawdown beneath the river is greatest.

5.2.3 River Discharge Gaging Stations

Three gaging stations were monitored during each pumping event in order to quantify the discharge in the Willimantic River flowing past the wellfield. These gaging stations are depicted on Figure 5-1. The first station, S-0, was installed upstream of the wellfield near the northern property line. The second station is the now inactive USGS Mansfield Depot gage (Station # 01119384) at the Willimantic River Wellfield. The third station, S-11, was installed downstream of the wellfield. Similar to the piezometers, the two gaging stations installed for this study were identified based on the adjacent habitat reach.

5.3 Monitoring Events

Pumping rates were stabilized during each monitoring event for at least 72 hours in accordance with the TAG-approved protocols for the pumping tests.

5.3.1 Monitoring Event #1

This monitoring event captured a combination of "moderate" wellfield operation (1.5 mgd, 2.32 cfs) and moderate river flow between August 18, 2008 at 8:00 a.m. and August 21, 2008, when 15-minute discharge in the Willimantic River ranged between 145 cfs and 89 cfs as recorded by the USGS Mansfield Depot gaging station at the wellfield. Pumping was performed as per system demand prior to and following this pumping period. The full data collection period extended from July 29, 2008 to September 5, 2008. Refer to Table 5-3 for water elevations recorded during this period. Graphics associated with the automatic dataloggers are provided in Appendix D.

Table 5-3
Data Collection During 2008 Monitoring

Date	7/29			8/14			8/18			8/21			9/5		
Wellfield Withdrawal	1.13 mg			0.71 mg			1.47 mg (test started approximately 12 P.M.)			1.56 mg			1.60 mg		
Observation Wells	GW			GW			GW			GW			GW		
Cv-48	-			289.28			289.35			288.82			287.56		
Cv-49	-			289.11			289.08			288.80			287.96		
Ms-60	-			-			290.13			288.24			284.60		
Ms-61	-			-			286.71			284.13			278.69		
Ms-65	-			285.14			285.40			282.28			275.97		
Ms-67	-			-			286.01			283.18			277.16		
Ms-68	-			-			292.37			291.32			289.31		
Piezometers	GW	SW	Gradient	GW	SW	Gradient	GW	SW	Gradient	GW	SW	Gradient	GW	SW	Gradient
P0-E	294.45	294.33	-0.12	-	-	-	294.21	294.11	-0.10	293.84	293.74	-0.10	293.24	293.22	-0.02
P0-W	294.38	294.65	0.27	-	-	-	294.13	294.40	0.27	294.13	294.40	0.27	291.82	293.46	1.64
P1-E	293.02	293.13	0.11	292.73	292.99	0.26	292.85	292.97	0.02	292.39	292.52	0.13	291.78	291.98	0.20
P1-W	292.67	292.77	0.10	292.55	292.67	0.12	292.46	292.56	0.10	292.05	292.18	0.13	291.46	291.63	0.17
P3-E	289.11	292.32	3.21	289.30	292.26	2.96	289.71	292.18	2.47	Dry	291.93	-	Dry	291.55	-
P3-W	292.13	292.31	0.18	291.98	292.19	0.21	291.95	292.19	0.24	291.57	291.95	0.38	291.02	291.60	0.58
P7-E	289.58	290.11	0.53	-	-	-	289.49	289.92	0.43	288.92	289.74	0.82	287.84	289.36	1.52
P7-W	290.27	290.19	-0.08	-	-	-	290.11	290.07	-0.04	289.87	289.87	0.00	289.41	289.53	0.12
P9-E	289.14	289.44	0.30	-	-	-	289.08	289.22	0.14	288.71	288.98	0.27	287.85	288.49	0.64
P9-W	288.89	288.80	-0.09	-	-	-	288.64	288.66	0.02	288.51	288.47	-0.04	288.26	288.30	0.04
P11-E	288.08	287.87	-0.21	287.98	287.85	-0.13	287.88	287.72	-0.16	287.63	287.43	-0.20	287.16	287.06	-0.10
P11-W	288.79	288.73	-0.06	288.70	288.58	-0.12	288.58	288.50	-0.08	288.32	288.22	-0.10	287.88	287.80	-0.08
Staff Gauges	SW			SW			SW			SW			SW		
SG-0	295.16			295.07			-			294.64			294.05		
SG-11	289.03			288.88			-			288.57			288.00		

Notes: GW= Groundwater, SW=Surface Water, Gradient = GW-SW.
A negative gradient means that surface water is being recharged by groundwater, while a positive gradient indicates that ground water is being recharged by surface water.
Water Elevations at the piezometers are estimated from 2008 measurements of riser height and 2009 survey of the streambed at each piezometer.
Elevations are measured in feet relative to the National Geodetic Vertical Datum of 1929.

Observation Well Data

The datalogger at Ms-65 provided a good indication of how ground water levels near Well #3 are affected by pumping cycles. The heavy rains and flooding of September 7 through September 9, 2009 provided recharge to the ground water at the wellfield. However, the drawdown curve from August 18 to August 21, while pumping the wellfield at 1.5 mgd, does not appear significantly different from the other drawdowns recorded at the wellfield under normal operation (mean of 1.20 mgd from July 29 through September 5, 2008) or during slightly higher operation during the first week of September (mean of 1.66 mgd).

Piezometer Data

Measurements at piezometer pair P0 suggested that the east side of the river is buttressed by ground water flowing off the till while the west side of the river loses water to the ground water system. The west piezometer showed a weak downward gradient throughout most of the monitoring period, with a stronger gradient under lower flow conditions.

Measurements at pair P1 showed a consistent weak downward gradient (recharging ground water) throughout the monitoring period. This gradient was generally balanced across the Willimantic River at this location, likely because of the proximity to Well #1. Only on August 14, 2008 was there a significant difference in gradient, coincident with the period just following the start of the constant rate pumping test. The river is deeper in the vicinity of P1-E, which explains why ground water levels in P1-E drop to approximately one foot above the riverbed while ground water levels in P1-W drop nearly to the riverbed during the higher withdrawal rates (Table 5-3) in the relatively dry first week of September.

Measurements at pair P3 showed a much stronger downward gradient recharging ground water throughout the monitoring period. The gradient was generally weak at P3-W during the 72-hour pumping test but stronger during the low flow period in early September when pumping rates were slightly higher (1.6 mgd) and ground water levels were reduced to nearly being at the riverbed. The gradient was very strong at P3-E, likely because of its proximity to the wellfield, and ground water levels were below the bottom of the piezometer screen (three feet below the riverbed) for the last part of the monitoring period, indicating development of an unsaturated zone or a low-pressure zone consistent with the high rate of induced infiltration.

Measurements at pair P7 showed that the downward gradient was generally strong at P7-E throughout the monitoring period and strongest during the low flow period in early September. The gradient was very weak upwards with ground water discharging to surface water at P3-W, but the gradient equalized by the end of the 72-hour pumping test. P7-W showed a weak downward gradient recharging ground water during the September low-flow period.

Measurements at P9-E showed a weak downward gradient recharging ground water throughout most of the monitoring period, with a strong gradient during the September low-flow period. P9-W showed a weak upward gradient recharging surface water in July, but the gradient was about even during the pumping test period and September low-flow period. At the final pair, measurements at pair P11 showed a consistently weak upward gradient with ground water discharging to surface water throughout the monitoring period.

The storm of September 7 to September 9, 2008 overtopped all the piezometers and likely flooded low-lying areas at the wellfield. Water levels were measured by the dataloggers at piezometer pairs P1 and P3 (Appendix D). Surface water peaked at approximately 6.7 feet above the riverbed at P1-W, 7.5 feet above the riverbed at P1-E, and 5.6 feet above the riverbed at P3-W and P3-E.

Temperature Data

Manual temperature monitoring was performed at each piezometer and at the river thalweg (denoted by "-Mid") at each piezometer pair on 8/21/2008 just following the end of the 72-hour pumping test period. Monitoring was performed using an Omega® 865F Digital Thermometer attached to a two-foot long thermistor probe by a three-foot wire. Temperatures of ground water were taken by pushing the probe into the substrate to a depth of one to two inches. Results are presented in Table 5-4.

Table 5-4
8/21/2008 Temperature Monitoring

Location	Ground Water Temperature, °F	Surface Water Temperature, °F	Difference, °F
P0-E	63.6	63.4	-0.2
P0-Mid	63.3	63.9	0.6
P0-W	63.7	64.7	1.0
P1-E	63.2	64.4	1.2
P1-Mid	64.3	64.8	0.5
P1-W	63.9	65.2	1.3
P3-E	64.8	61.7	-3.1
P3-Mid	63.1	61.9	-1.2
P3-W	65.3	62.1	-3.2
P7-E	63.3	62.4	-0.9
P7-Mid	64.0	62.4	-1.6
P7-W	63.3	62.4	-0.9
P9-E	64.8	63.4	-1.4
P9-Mid	64.5	63.4	-1.1
P9-W	64.7	63.2	-1.5
P11-E	62.8	62.6	-0.2
P11-Mid	62.8	62.6	-0.2
P11-W	63.3	62.8	-0.5

Note: Difference is measured as Surface Water Temperature – Ground Water Temperature.

Manually measured ground water temperatures were generally constant throughout the monitoring area. In general, the surface water temperature was equal or greater than the ground water temperature near P0 and P1 and lower than the ground water temperature from P3 to P11. The datalogger at Ms-65 was not equipped with the capability to measure ambient ground water temperature, so no comparison to temperature beneath the wellfield could be made.

The manually measured ground water temperatures were generally colder than the water temperatures recorded by the dataloggers in the four piezometers. Three significant spikes are present on the graph in Appendix D. The first spike is that the temperature of water in each piezometer appreciably rises after the start of pumping while the second two spikes are the temperature of ground water decreasing toward the end of the pumping test. The first spike is likely related to warmer surface water being induced from the Willimantic River into the aquifer during the relatively warmer period leading into the pumping test.

The remaining spikes are likely due to the very cold air temperatures on the last two nights of the pumping test (below 45 degrees Fahrenheit, °F). For example, the air temperature the previous night before the manual temperature monitoring was performed had a low of 44.2 °F as measured by the barologger at Ms-65. As all of the piezometer gradients (except P0-E and the P11 pair) were equal or recharging ground water on this date, the lower surface water temperatures may be due to the chilly weather the previous night, the shady conditions near most of the piezometers, and the moderate river flow. If induced infiltration were occurring near these piezometers (as reflected by the gradients in Table 5-3), it is reasonable to conclude that ground water temperature would also be reduced due to the colder surface water.

Discharge Data

Discharges were measured four times during the 2008 monitoring period by MMI at staff gages S-0 and S-11. A manual discharge measurement was also performed at the Mansfield Depot gaging station in September. Results are depicted on Table 5-5. The discharge at the upstream station (near P-0) was measured at 102 cfs at 12:30 p.m., and discharge at the downstream station (near P-11) was measured at 100 cfs at 11:30 a.m. on

8/21/2008 near the end of the 72-hour pumping test. These measurements are consistent with the belief that continuous withdrawals from the production wells (in this case 1.5 mgd, or 2.3 cfs) are coincident with a similar reduction in streamflow over the reach of the wellfield.

Table 5-5
2008 Discharge Measurements Performed by MMI

Date	Discharge at S-0, Time (EDT)	Discharge at USGS Gaging Station at Mansfield Depot, Time (EDT)	Discharge at S-11, Time (EDT)	Gain, S-0 to S-11	Hydrograph Condition*
7/29	212 cfs (19:00)	-	221 cfs (16:00)	9 cfs	Receding (0.04')
8/14	189 cfs (12:15)	-	196 cfs (11:15)	7 cfs	Receding (0.01')
8/21	102 cfs (12:30)	-	100 cfs (11:30)	-2 cfs	Stable
9/5	32 cfs (18:00)	31 cfs (17:00)	35 cfs (15:15)	3 cfs	Stable

*As discerned from 15-minute real-time data for the USGS gaging station at Mansfield Depot.

However, the two cfs difference between these discharge measurements is well within the margin of error for each discharge measurement. Properly performed discharge measurements with USGS equipment generally have an associated uncertainty of 3% to 6% (Sauer and Meyer, 1992). The measurements in this study were performed with an electromagnetic current meter. Although information concerning the uncertainties of electromagnetic meters could not be found as of this writing, it is assumed that the uncertainty of each discharge measurement performed with an electromagnetic current meter would be similar to the ranges presented in Sauer and Meyer (1992).

The higher discharge measurements (taken on 7/29/2008 and 8/14/2008) were recorded under receding flow conditions, which the downstream measurement performed first. Also, a significant amount of time (approximately three hours) passed between the measurements taken on 7/29/2008. Thus, the gains shown in Table 5-5 are exaggerated by the receding condition.

The lower discharge measurements (taken on 8/21/2008 and 9/5/2008) were taken under generally stable flow conditions, with no change in stage between the time of the two measurements as recorded at the USGS gaging station at Mansfield Depot. A notable gap in time (approximately 2.75 hours) did occur between the first and last measurements taken on 9/5/2008. While the last two measurements (upstream and at the gaging station) are similar, the downstream measurement was performed much earlier. It is also possible that this measurement was elevated by outflow from the Stafford Springs WPCF.

5.3.2 Monitoring Event #2

This monitoring event captured a combination of high wellfield operation (1.80 mgd, 2.79 cfs) and low river flow between September 21, 2009 at 1:00 a.m. and September 24, 2009, when mean daily discharge in the Willimantic River ranged between 18 cfs and 23 cfs as recorded by the USGS gaging station at Merrow Road upstream of the wellfield. Data collection occurred on September 18, 2009 and September 23, 2009. The mean daily pumping rate of the Willimantic River Wellfield prior to September 18 was 1.55 mgd, indicative of a period of moderate to high water usage. Refer to Table 5-6 for water elevations recorded on these dates. Graphics associated with the automatic dataloggers are provided in Appendix D.

Observation Well Data

The datalogger at Ms-65 again provided a good indication of how ground water levels near Well 3 are affected by pumping cycles. Well #3 appeared to be pumped consistently through the end of September 2009 and then set to normal pumping cycles based on system demand in October and November prior to the third monitoring event. The drawdown curve during the second pumping test period again appears slightly steeper from the other drawdown curves recorded at Ms-65 under normal operation, which concurs with the higher pumping rate during this period.

Table 5-6
Data Collection During September 2009 Monitoring

Date	9/18			9/23		
Wellfield Withdrawal	0.35 mg (following pumping 1.47 mg on 9/17)			1.80 mg		
Observation Wells	GW			GW		
Cv-48	287.88			287.84		
Cv-49	288.16			288.11		
Ms-60	285.57			285.64		
Ms-61	280.42			280.32		
Ms-62	279.35			279.10		
Ms-65	278.93			278.20		
Ms-67	279.91			279.57		
Ms-68	289.61			289.88		
Ms-69	280.83			281.42		
Ms-70	279.54			278.40		
OW-31	280.53			280.16		
Piezometers	GW	SW	Gradient	GW	SW	Gradient
P0-E	293.50	293.47	-0.03	293.56	293.53	-0.03
P0-W	292.62	293.49	0.87	292.73	293.56	0.83
P1-E	290.85	291.87	1.02	290.94	291.90	0.96
P1-W	291.36	291.80	0.44	291.35	291.84	0.49
P3-E	Dry	291.52	-	Dry	291.54	-
P3-W	291.14	291.78	0.64	291.24	291.81	0.57
P7-E	288.66	289.49	0.83	288.67	289.52	0.85
P7-W	289.03	289.48	0.45	289.05	289.50	0.45
P9-E	287.99	288.56	0.57	288.07	288.59	0.52
P9-W	288.73	288.80	0.07	288.77	288.84	0.07
P11-E	287.71	287.54	-0.17	287.72	287.58	-0.14
P11-W	287.56	287.49	-0.07	287.62	287.53	-0.09
Staff Gages	SW			SW		
SG-0	293.87			293.93		
SG-11	287.89			287.91		

Piezometer Data

Measurements at piezometer pair P0 again suggested that the east side of the river is buttressed by ground water flowing off the till while water in the west side of the river recharges ground water. The west piezometer showed a strong downward gradient before and during the pumping test.

Measurements at pair P1 showed a consistent strong downward gradient recharging ground water during the test. Unlike the fairly consistent gradient noted in 2008, the gradient was noticeably stronger at P1-E during this monitoring event, suggesting that the higher pumping rate is affecting the east side of the river more than the west side at P1, as expected.

Measurements at P3-W showed a strong downward gradient recharging ground water throughout the monitoring period. The gradient was very strong at P3-E likely because of its proximity to the wellfield, with ground water levels as measured in nearby observation well OW-31 at 280 feet in elevation, which was 13 feet below the surface water at P3. Despite driving this piezometer to six feet below the riverbed, the presence of an unsaturated or low-pressure zone resulted in a lack of useable ground water data acquired during the 2009 monitoring period.

Measurements at pair P7 showed a strong downward gradient recharging ground water during the pumping period, with a stronger gradient being measured at P7-E consistent with the 2008 monitoring.

Measurements at P9-E showed a strong downward gradient recharging ground water during the monitoring period while P9-W showed a weak gradient recharging ground water. The gradients are stronger overall than during the 2008 monitoring, which is expected since the pumping rate was higher. As in 2008, measurements at pair P11 again displayed a consistently weak upward gradient recharging surface water throughout the monitoring period.

Temperature Data

Similar to 2008, manual temperature monitoring was performed at each piezometer and at the river thalweg (denoted by "-Mid") between each piezometer pair on 9/23/2009 near the end of the 72-hour pumping test. Temperatures of ground water were taken by pushing the probe into the substrate to a depth of one to two inches or by lowering the probe inside each piezometer until it was submerged. Results are depicted in Table 5-7.

Table 5-7
9/23/2009 Temperature Monitoring

Location	Ground Water In Piezometer Temperature, °F	Ground Water in Riverbed Temperature, °F	Surface Water Temperature, °F	Difference, °F[1]
P0-E	60.0	61.3	61.9	0.6
P0-Mid	-	61.2	62.0	0.8
P0-W	61.0	63.4	63.5	0.1
P1-E	61.2	61.6	62.8	1.2
P1-Mid	-	62.5	63.2	0.7
P1-W	61.1	62.9	64.4	1.5
P3-E	61.4 (water in tip)	63.0	63.7	0.7
P3-Mid	-	62.8	63.7	0.9
P3-W	*	62.9	63.8	0.9
P7-E	63.5	71.5[2]	73.6[2]	2.1
P7-Mid	-	63.0	64.3	1.3
P7-W	60.3	63.4	63.8	0.4
P9-E	62.0	64.6	66.6	2.0
P9-Mid	-	63.4	64.6	1.2
P9-W	61.2	63.3	64.5	1.2
P11-E	56.7	61.3	65.6	4.3
P11-Mid	-	63.1	64.7	1.6
P11-W	58.2	62.9	64.8	1.9

Notes: 1. Difference is measured as Surface Water Temperature – Ground Water Temperature as measured in the riverbed.
2. Outside of P7-E was nearly dry (puddle), so temperature was significantly higher.
*Measurement blocked by datalogger

The surface water temperature was always greater than the ground water temperature near all monitoring locations. This was expected given the low flows in the Willimantic River and the warm period leading up to the pumping test. Ground water temperatures were generally constant throughout the monitoring area but much colder in the vicinity of P11. As shown in Table 5-6, the area near P11 is coincident with ground water discharging to the river outside of the area of influence of the wellfield. Surface water temperatures were highest at P7-E and P9-E because the side of the river was nearly dry at the time of the measurement.

The datalogger utilized in Ms-65 in 2009 was able to record ground water temperature although this data was taken at a significant depth (approximately 50 feet below ground level). As shown in the Figure for Ms-65 in Appendix D, the ground water temperature near Well #3 was approximately 49 °F prior to the September 2009 constant rate pumping test, indicating that the ground water at depth is much colder than the ground water near the surface. According to Thomas et al. (1967), ground water more than 30 feet below the land surface has a relatively constant temperature, usually between 48 °F and 50 °F.

The ground water temperature at Ms-65 increased steadily throughout the test period and through the third week October 2009 before leveling off at approximately 58.5 °F. The increase was likely due to induced infiltration of river water into the aquifer during what ultimately was the driest part of 2009. The temperature of the ground water was fairly constant leading up to the third monitoring event.

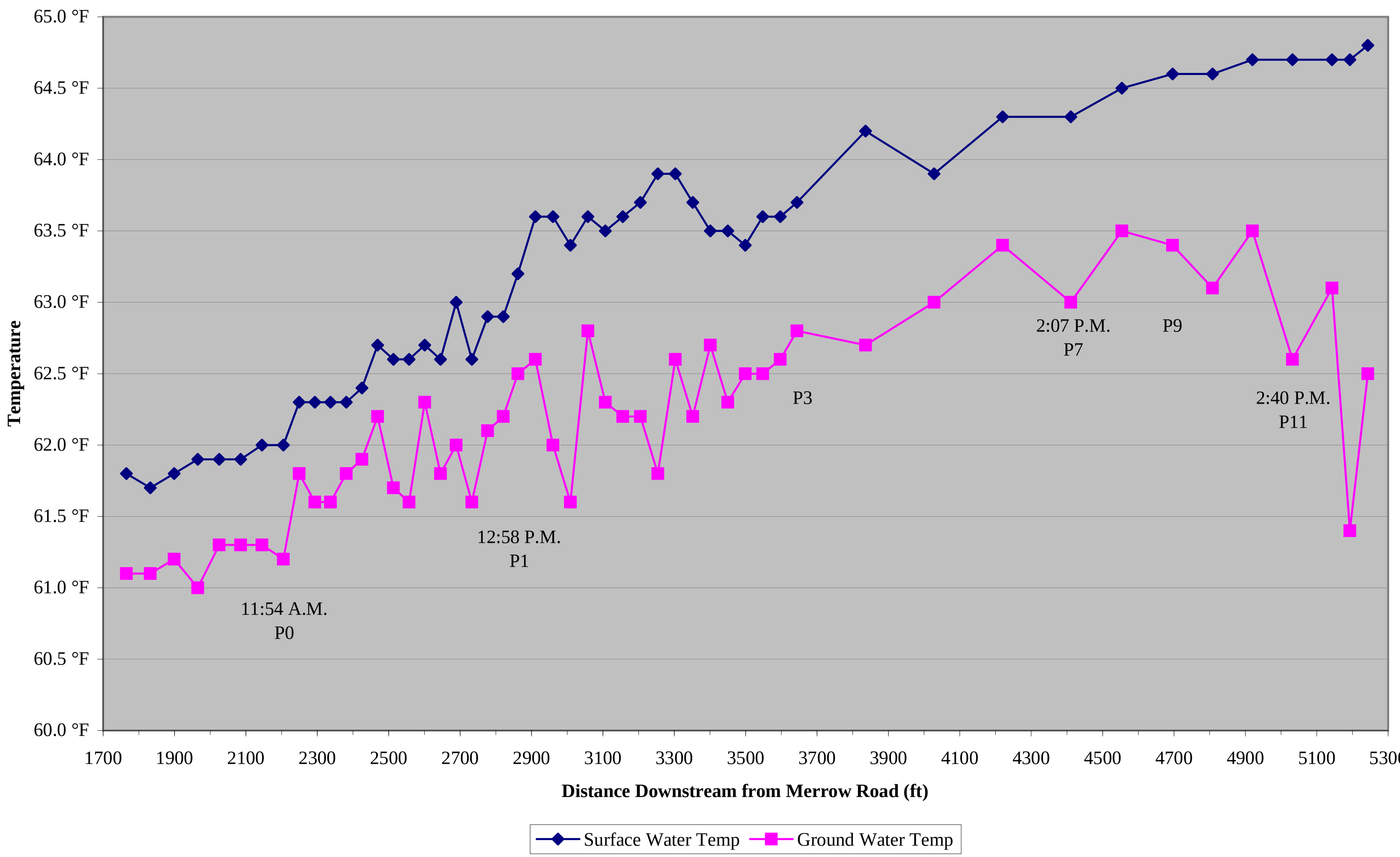
In contrast to the 2008 monitoring, the manually measured ground water temperatures were generally warmer than the ground water temperatures recorded by the dataloggers in the four piezometers. This is likely due to the unseasonably warm overnight air temperature (low of about 60 °F) that occurred the previous night. This premise is supported by the fact that nighttime spikes are present on the datalogger temperature graph leading into the constant rate pumping period, coincident with cold nighttime temperatures (31 °F to 37 °F) that occurred on the early mornings of September 19, 20, and 21, 2009. The early mornings of September 23 and September 24, 2009 had lows greater than 60 °F, leading to an increase in ground water temperature from inducement of warmer surface water.

In addition, a temperature profile down the river thalweg was performed from upstream of P0 to downstream of P11. The temperature profile is depicted graphically on Figure 5-2. Similar to the manual monitoring data presented in Table 5-7, the temperature data taken along the thalweg of the Willimantic River show that surface water was generally higher (0.5 °F to 1.0 °F) in temperature than ground water throughout the monitoring area and that ground water is much colder (greater than 1.3 °F) in the vicinity of P11. The strong deviation of colder ground water concurs with the water elevation measurements at P11 suggesting that this area is outside of the area of influence of the wellfield.

Discharge Data

Discharges were measured by MMI at staff gages S-0 and S-11 and at the USGS gage at the wellfield on 9/23/2009 near the end of the 72 hour pumping test. The discharge at the upstream station (near P0) was measured at 23 cfs at 11:00 a.m., and discharge at the downstream station (near P11) also measured at 23 cfs at 1:00 p.m. The discharge at the USGS gage was measured to be 18 cfs by MMI at 3:00 p.m. The hydrograph on the Willimantic River was generally stable during these measurements. The measurement at the USGS gage is considered to be poor due to a large depression in the center of the measurement area, which accounts for at least part of the difference in flow between the three stations.

Figure 5-2
9/23/2009 Temperature Profile along Willimantic River Thalweg



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5.3.3 Monitoring Event #3

This monitoring event captured a combination of high wellfield operation (1.97 mgd, 3.05 cfs) and moderate river flow between November 9, 2009 at 8:45 a.m. and November 12, 2009, when mean daily discharge in the Willimantic River ranged between 42 cfs and 38 cfs as recorded by the USGS Merrow Road gaging station upstream of the wellfield. Data collection occurred on November 12, 2009. Refer to Table 5-8 for water elevations recorded on this date. Graphics associated with the automatic dataloggers are provided in Appendix D.

Table 5-8
Data Collection on November12, 2009

Observation Wells	GW	Piezometers	GW	SW	Gradient
Cv-48	287.57	P0-E	293.89	293.90	0.01
Cv-49	288.13	P0-W	292.12	293.90	1.78
Ms-60	284.86	P1-E	290.85	292.22	1.37
Ms-61	279.29	P1-W	291.55	292.17	0.62
Ms-62	278.16	P3-E	-	291.78	-
Ms-65	277.06	P3-W	290.95	292.02	1.07
Ms-67	278.37	P7-E	288.21	289.73	1.52
Ms-68	289.10	P7-W	289.10	289.73	0.63
Ms-69	280.64	P9-E	287.99	288.86	0.87
Ms-70	277.24	P9-W	288.80	288.94	0.14
OW-31	278.77	P11-E	287.86	287.84	-0.02
		P11-W	287.84	287.79	-0.05
Staff Gages	**SW**				
SG-0	294.28	**Pumping Rate:**		**1.97 mgd**	
SG-11	288.15				

Observation Well Data

The datalogger at Ms-65 showed that Well #3 was generally shut down from November 6, 2009 to November 8, 2009 in preparation for the pumping test. The drawdown curve for this constant rate pumping period appears slightly steeper than the other drawdowns recorded at the wellfield under normal operation but is similar to the September pumping test. This makes sense as high pumping rates were utilized during these periods with Well #3 starting from a recovered condition.

Piezometer Data

Measurements at piezometer pair P0 indicated that the east side of the river is buttressed by ground water flowing off the till while the west side of the river recharges ground water. This condition is similar to that observed in September 2009.

Measurements at pair P1 showed a strong gradient downward with surface water recharging ground water at the end of the test. This gradient was stronger at P1-E, consistent with the previous monitoring.

Measurements at P3-W showed a strong downward gradient recharging ground water. The gradient was very strong at P3-E likely because of its proximity to the wellfield, with ground water levels as measured in nearby observation well OW-31 at 279 feet in elevation, 13 feet below the surface water at P3. As in September 2009, the inside of P3-E was dry.

Measurements at pair P7 showed a strong downward gradient recharging ground water during the pumping period, with a stronger gradient being measured at P7-E consistent with the previous monitoring.

Measurements at P9-E showed a strong downward gradient recharging ground water at the end of the 72-hour test while P9-W showed a weak downward gradient recharging ground water. The gradients are slightly stronger than those observed during the September monitoring period. As seen previously, measurements at pair P11 again displayed a weak upward gradient with ground water discharging to surface water throughout the monitoring period.

Temperature Data

Temperature monitoring was performed similar to that performed in September 2009. Results are depicted in Table 5-9. The temperature profile along the river thalweg is included in Appendix D. Ground water temperatures and surface water temperatures were similar and both cold, making interpretation difficult. Surface water was generally slightly warmer than ground water. Deeper ground water temperatures as measured at Ms-65 were higher (approximately 58.5 °F) than that observed near the surface, likely as a result of an entire summer and fall of induced infiltration. The first week of November 2009 had lows ranging from 20 °F to 35 °F, suggesting an overall cooling of surface water would have occurred.

Table 5-9
11/12/2009 Temperature Monitoring

Location	Ground Water Temperature, °F	Surface Water Temperature, °F	Difference, °F
P0-E	45.9	46.3	0.4
P0-Mid	46.0	46.0	0.0
P0-W	46.2	46.0	-0.2
P1-E	46.1	46.4	0.3
P1-Mid	46.2	46.3	0.1
P1-W	46.4	46.3	-0.1
P3-E	46.4	46.5	0.1
P3-Mid	46.2	46.4	0.2
P3-W	46.4	45.9	0.5
P7-E	46.3	46.7	0.4
P7-Mid	46.5	46.6	0.1
P7-W	46.1	46.5	0.4
P9-E	46.2	46.6	0.4
P9-Mid	46.5	46.6	0.1
P9-W	45.9	46.5	0.6
P11-E	46.7	46.8	0.1
P11-Mid	46.7	46.8	0.1
P11-W	46.7	46.8	0.1

Note: Difference is measured as Surface Water Temperature – Ground Water Temperature.

Temperature data collected at the piezometers showed a general warming trend over the constant rate pumping period. This could be due to the warmer early mornings experienced from November 10 to November 12, 2009 (above 40 °F) allowing surface water temperatures to rise into the mid 40s (°F) during this period.

Discharge Data

Discharges were measured by MMI at staff gages S-0 and S-11 and at the USGS gage at the wellfield on 11/12/2009 near the end of the 72-hour pumping test. The discharge at the upstream station (near P0) was measured at 54 cfs at 11:00 a.m., and discharge at the downstream station (near P11) measured at 56 cfs at 1:30 p.m. The discharge at the USGS gage was measured at 54 cfs by MMI at 12:30 p.m. All of these measured flows are similar, and a reduction due to pumping the wells would be difficult to separate from the data given the uncertainties associated with the measuring equipment, as discussed earlier. Furthermore, the downstream discharge may have been affected by releases from the Stafford Springs WPCF as more than an hour passed between the measurements.

5.4 Findings and Conclusions

The three hydrogeologic monitoring events provided useful data for verifying the area of influence of the wellfield and delineating the lines between areas of the river experiencing induced infiltration and those experiencing ground water discharge. Information collected at the piezometers generally showed decreasing gradients of surface water recharging ground water from the east side of the river to the west except near P0 where the ground water on the east bank is buttressed by the nearby till boundary and at P11, which is downstream of the area of influence based on data collected at the piezometers and temperature monitoring. The area of influence is believed to extend past P0 to the north and northwest into Coventry. A very strong gradient occurs at P3-E due to the proximity of the centroid of the wellfield, possibly exacerbated by the proximity of the till boundary across the river near P3-W.

The temperature data showed that there is an impact to ground water temperature from changes in surface temperature at the wellfield. This impact is stronger at piezometer pair P3 than at P1, which is reasonable since the downward gradients were stronger at P3 during pumping. The temperature data measured at Ms-65 demonstrates that the aquifer may warm over the late summer and fall when wellfield withdrawals are high and streamflows are relatively lower as compared to the winter and spring months.

Streamflow measurements were intended to characterize the pumping impact of the wellfield on the Willimantic River after 72 hours of pumping at a constant rate. Only the 8/21/2008 monitoring event showed a decrease in flow through the wellfield consistent with the ground water withdrawal. This measurement was taken during a period when the hydrograph of the Willimantic River was stable, and the two measurements were performed right after each other.

For the most part, the small increment of instream flow that is associated with wellfield production (up to three cfs) fell within the margin of error for each streamflow measurement (assumed to be 3% to 6% of each measured discharge for a well-performed measurement). Many of the measurements were also performed during times of hydrograph recession, or were spaced apart such that they could be influenced by upstream flow releases from the Stafford WPCF.

The hydrogeologic monitoring data was used for the verification of the updated Level A model (Section 6.0).

6.0 UPDATE OF LEVEL A NUMERICAL MODEL

6.1 Numerical Methods

The USGS program known as MODFLOW was used to simulate the stratified drift aquifer associated with the Willimantic River Wellfield. The study utilized the version of MODFLOW known as "MODFLOW 2000" for the aquifer system simulation and MODPATH for particle tracing and the delineation of the area of contribution.

MODFLOW is used to solve ground water flow problems by approximating a mathematical representation of the flow system. This representation is made up of a governing equation, boundary conditions that describe flux and/or head conditions, and an equation that describes initial conditions. The governing equation is derived by combining a water balance through an elementary volume with Darcy's Law for flow through a porous medium and states that fluxes through the system minus source and sinks is equal to a change in storage. Although the equation can be solved analytically for simple solutions, numerical methods are preferred for complex, real-world situations.

MODFLOW requires that the continuous hydrogeologic system be replaced with a finite set of cells. Next, the partial derivatives of the governing equation are replaced by terms that express head differences between the center points of each cell. The resulting set of linear algebraic equations can be expressed with a matrix. MODFLOW solves the matrix equations with matrix and iterative techniques. The output is a set of hydraulic heads for each point in the flow system.

The particular version of MODFLOW 2000 to be used to update the Level A model for this analysis is compiled with recent versions of Groundwater Vistas (Environmental Simulations, Inc., 2007), a windows-based platform including preprocessors and postprocessors and executable versions of MODFLOW, MODPATH, MT3D, and several other programs.

6.2 Previous Modeling Effort (2007 Level A Model)

The Level A report for the Willimantic River Wellfield was submitted to the DEP in 1999. This submission included a predictive simulation under conservative conditions of low streamflow, no ground water recharge from precipitation, and continuous pumping at a maximum rate for 180 days. In order to sustain the simulation without the aquifer running dry, the model was run using 1,200 gpm for this period.

In a letter dated March 20, 2001 to Mr. Larry Schilling of the University of Connecticut, the DEP stated that while the Level A model met the regulatory requirements for mapping the approval was going to be deferred while the DEP investigated potential changes to the Level A regulations. The analysis was subsequently revised in 2007 as requested by the DEP for consistency with amendments to the Level A Regulations. This provided an opportunity for inclusion of findings and conclusions of the Well #4 safe yield test completed by Lenard Engineering, Inc. in August 1999. The predictive simulation for this model was discussed in Section 3.5. The Level A model submitted and approved in 2007 is discussed below.

Numerical Model

The Level A model was laid out in a grid with 78 rows and 43 columns containing approximately 4,100 active model cells. The grid size of the active cells ranged from 25 feet by 50 feet to 100 feet by 400 feet. Model boundaries included the water table at the top and the interface of stratified drift with till or bedrock at the bottom and laterally. Boundaries of the model were also set at a saturated thickness of 10 feet near the lateral boundaries where mapped by the USGS.

The aquifer was modeled as a three-layer system corresponding to stratigraphic changes in the aquifer. The first layer (Layer 1) consisted of the sand and gravel, alluvium, and fine to medium sand sequence. The second layer (Layer 2) consisted of a fine to coarse sand, sand and gravel, and fine sand sequence. The bottom active layer of the model (Layer 3) was modeled entirely as a fine sand sequence.

The following properties were utilized for the Level A model following calibration:

- Horizontal hydraulic conductivity of the aquifer was found to range between 40 and 128 feet per day (ft/d), with the highest values in the vicinity of the wellfield and the lowest values at the southeastern end of the terrace on the eastern side of the aquifer based on boring logs and median grain size (Section 3.5).
- Horizontal hydraulic conductivity in Layers 1 and 2 was the same as described above, with a uniform value of 10 ft/d representing the finer layer in Layer 3.
- Horizontal to vertical hydraulic conductivity ratios were set at 150 for Layers 1 and 2 and at 100 for Layer 3.
- Specific Yield was estimated to be 0.2 and specific storage to be 0.0004.
- Recharge was calculated using a rate of 30 inches on an annual basis.
- Flux across boundaries was calculated assuming a recharge rate to till of nine inches per year, with flux cells established in both Layers 1 and 2.
- Contributions from upstream underflow in stratified drift into the model area and discharge of water from the model area downstream were calculated using Darcy's Law and modeled in Layers 1 and 2.
- Evapotranspiration was set at 56 inches per year for the growing season and two inches per year for the nongrowing season with an extinction depth of five feet.
- A combined average pumping rate of 890 gpm for the three active Willimantic Wellfield Wells in 1994–1995 was used for the calibration and verification.
- Riverbed vertical hydraulic conductivity was set at 0.001 ft/d with a 0.01-foot thickness during the summer period coincident with a "clogging layer" covering the riverbed and set at 0.5 ft/d with a one-foot thickness during periods of high flow.

- Elevation of the stream surface was assigned uniformly from USGS Station #01119382 to Station #01119386 (Merrow Road to Route 44) with a four-foot depth applied universally.

Level A Model Calibration and Verification

As the Willimantic River Wellfield reportedly could not be shut down (during the monitoring period of 1993 to 1995) for a satisfactory length of time to collect a dataset that allowed significant recovery of ground water levels at the wellfield, natural stresses were chosen for model calibration and verification. The Level A report does not discuss how the starting heads in the aquifer for the transient simulations were generated, but it is assumed that an appropriate steady-state simulation was run to generate starting heads.

A dataset collected on November 2, 1994 was selected for calibration as it was a time when both ground water and surface water levels were low. Baseflow discharge in the Willimantic River was estimated to be 40 cfs on November 2, close to the 80 percent duration discharge. A 45-day transient simulation with 35 time steps in one stress period was performed, with the 45th day being November 2, 1994.

Fourteen monitoring points were selected to calibrate the model. Of the 14 targets, the difference between measured and simulated heads was less than one foot in four wells, less than two feet in 10 wells, and less than five feet in 12 wells. The Level A Report states that simulated discharge in the model was also close to 80% duration for the Willimantic River. The mass balance error for the simulation was -0.76%.

A dataset collected on January 18, 1995 was selected for verification as it was a time when both ground water and surface water levels were high. According to the Level A Report, the discharge in the Willimantic River was approximately equal to 30% duration discharge on that date. A 45-day transient simulation with 1,000 time steps in one stress period was performed, with the 45th day being January 18, 1995. Input parameters that were changed included increases to recharge, flux across boundaries, and streambed conductance; and a decrease to evapotranspiration.

The same 14 monitoring points were selected to verify the model. Of the 14 targets, the difference between measured and simulated heads was less than one foot in six wells and less than two feet in 11 wells. The Level A Report states that simulated discharge in the model was also close to 40% duration for the Willimantic River. The mass balance error for the simulation was 0.03%.

Following the model calibration and verification, a sensitivity analysis was performed. Model input parameters were increased and decreased at least 50% where appropriate. The model was found to be most sensitive to changes in aquifer horizontal hydraulic conductivity, streambed conductance, specific storage, and boundary flux. Changes to specific yield had generally minimal impacts on the model.

6.3 Revised Model Characteristics

The MODFLOW 2000 model prepared by Mr. Daniel Meade was obtained by MMI from the DEP. The Level A model was imported into Groundwater Vistas and exported into ArcGIS, a Geographic Information System (GIS) platform. Graphics associated with the numerical model are organized as individual figures within Section 6, and numerical model files are provided on a CD as Appendix F.

6.3.1 Dimensions and Discretization

MMI accepted the model grid for use in the modeling. The model extends approximately 7,800 feet in the north-south direction and 2,100 feet in the east-west direction. The model grid is rotated 6.6 degrees west of north to align with the stratified drift axis trending generally north to south through the model area. The model domain is depicted on Figure 6-1.

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Model Domain

LOCATION:

Coventry & Mansfield, CT

MMI#: 1958-09
MXD: H:\Figure6-1.mxd
SOURCE: Microsoft Virtual Earth

N

Willimantic River Study

Map By: SJB
Date: Feb. 2010
Scale: 1"=1,000'

SHEET:

Figure 6-1

The model grid extends approximately 950 feet easterly and 2,600 feet northerly from production well UConn #1 and extends approximately 1,030 westerly and 4,400 feet southerly from production well UConn #4. This area includes much of the stratified drift aquifer associated with the Willimantic River to its eastern, southwestern, western, and northwestern edges and extends upstream (to the north) and downstream (to the south-southeast) from the wellfield. The model is sufficiently long to prevent interference between the area of influence and the model boundaries.

The ratio of spacing between adjacent rows and columns is less than 1.5, ensuring model stability. The only exception to this ratio is the one-foot thick boundary of no-flow cells that surrounds the model grid. These transitions were not adjusted in MMI's revisions to the model as it was determined that these outer transitions were not problematic.

6.3.2 Layers and Elevations

The updated numerical model has three layers consistent with the Level A model, corresponding to the primary sedimentary units of the stratified drift valley in the vicinity of the Willimantic River Wellfield. Layer 1 corresponds to the overlying layer of fine- to medium-grained sand, gravel, and boulders that generally extends down to 275 feet in elevation. Layers 2 and 3 refers to the fine- to coarse-grained sand and sand and gravel found to lie at depth in the boring logs in the stratified drift aquifer. The production wells draw water from Layer 2. Discontinuous units of finer materials were not modeled as discrete units given the heterogeneous nature of the ice-marginal fluviodeltaic deposits.

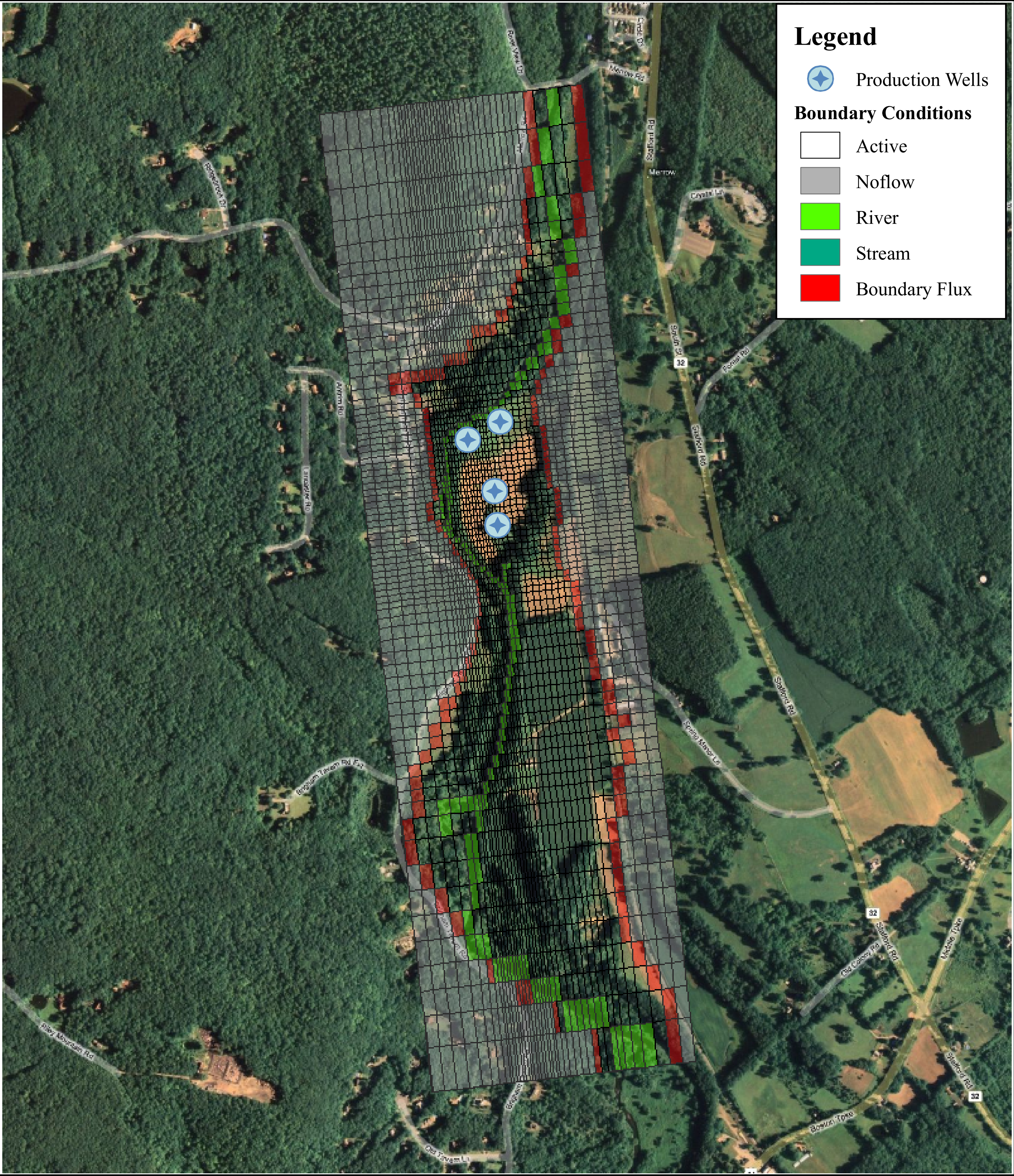
MMI accepted the top elevations and bottom elevations of the model layers in the model. Bottom elevations of each model layer were presented as Figures 8a through 8c in the Level A Report and are not reprinted here.

6.3.3 Boundary Conditions

The boundaries of the active numerical model cells were set as stated in Section 6.2. Boundary conditions within the active model cells include no-flow cells to account for the extent of glacial till; injection wells surrounding the majority of the outermost reaches of the glacial till no-flow cells; and river cells to represent the Willimantic River. Smaller tributaries to the Willimantic River in the model area (such as Winding Brook) were not simulated as these streams were dry when the instream flows of the Willimantic River were low during 2008 and 2009 monitoring and contribute insignificant flow when the river is high during wetter periods. Refer to Figure 6-2 for a depiction of model boundary conditions.

No-Flow Cells

The no-flow boundary condition applies to cells that are outside the computational domain of the model. These are termed inactive cells in MODFLOW, and head is not computed in cells designated as no-flow (Rumbaugh, 2007). No-flow cells were used to delineate areas of glacial till outside the active area of the model. As stated in Section 6.2, no-flow cells were also used for areas with a saturated thickness of less than 10 feet based on USGS mapping. No-flow boundaries in the model were accepted for use by MMI.



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Boundary Conditions

LOCATION: **Coventry & Mansfield, CT**

MMI#: 1958-09
MXD: H:\Figure6-2.mxd
SOURCE: Microsoft Virtual Earth

N

Willimantic River Study

Map By: SJB
Date: May 2010
Scale: 1"=1,000'

SHEET: **Figure 6-2**

River Package

A generic head-dependent boundary condition computes the flux of water into or out of the model and assigns the flux to that boundary cell. The conductance term in MODFLOW governs how water moves through or leaks into the aquifer from each river cell and is computed using the hydraulic conductivity of the riverbed material, the area of the river bottom within the finite-difference cell, and the thickness of the river bottom. The River Package generates a special form of the head-dependent boundary condition, which also limits the amount of water injected into the aquifer if the aquifer head drops below the bottom of the river (McDonald and Harbaugh, 1988).

River Cell Characteristics in the Level A Model

River cells were originally used in the Level A model to simulate the Willimantic River in the model area. The location of the river cells was defined based on USGS topographic mapping. The water elevation in the river was set uniformly throughout the model area from the USGS gaging station at Merrow Road (the northeast corner of the model) to the USGS gage upstream of Route 44 (well downstream of the south end of the model), which tends to smooth and simplify the riffles, runs, and pools that make up the reaches of the Willimantic River in the model area. In addition, a uniform four-foot stream depth was used for the river cells during low-flow and high-flow conditions, which is not appropriate when more detailed information becomes available as it did through this study.

The Level A model used very low values for the hydraulic conductivity of the riverbed, particularly in the summer months. The reasoning for the very low value was the presence of a "clogging layer" of fine and organic sediments that would coat the riverbed during periods of lower flow thereby inhibiting the vertical hydraulic conductivity. Field data supporting this assessment were not included in the Level A report.

River Cell Characteristics in the Updated Model

The Level A river cells were removed and new river cells were assigned to the active model area by using 2004 aerial photography to locate the Willimantic River in ArcGIS. The aerial photography was obtained from the University's Center for Land Use Education and Research (CLEAR). The length of each river cell was also assigned by measuring the appropriate distances in ArcGIS. The width of each river cell was set based on the transect widths collected during the instream flow study supplemented with distances measured in ArcGIS. The thickness of the riverbed was set to one foot in each model cell, a value consistent with many Level A studies and consistent with the high flow calibration of the Level A model.

Transect riverbed elevations from the Instream Flow Study (Section 4) and riverbed elevations from the piezometers and staff gages used in the supplemental hydrogeologic study (discussed in Section 5 of this report) were used to define the riverbed in associated river cells. The riverbed elevations in the river cells between these points were interpolated with guidance from the habitat mapping and reach information defined during the instream flow study. A graphic of the updated riverbed elevations is included in Appendix D. River stages and bottom elevations are lower in the updated model as compared to the Level A model.

In order to simulate the river under a variety of discharge conditions, several sets of river input files were developed for discharges on the Willimantic River ranging from 11 to 193 cfs, each with differing widths and water elevations for each model cell. The water elevations and widths in each river cell were created for each discharge using data collected during the instream flow and hydrogeologic studies, with interpolation between points based on guidance from the qualitative reach information defined during the instream flow study.

Riverbed hydraulic conductivity in the model was set at two feet per day and maintained for all simulations. The reasoning for this constant value warrants a detailed discussion below.

Discussion of Riverbed Hydraulic Conductivity

While it has been documented that some areas of the riverbed become clogged during periods of low flow, this largely appears to occur in pools and near the edges of the stream where velocities are lower. For example, silts and fine sands were prevalent on the riverbed in the oxbow near Reach 16, near the east bank of Reach 26, and in the backwater above Reach 9. These areas were coincident with areas of low velocity either due to backwater conditions or flow impairments (logs, debris, etc.).

In addition, compact fines and clay were prevalent in the riverbed in over half the channel and southern bank along Reaches 28 and 29 and along the east side and bank of the river near P3-E (the "dry" piezometer during 2008 and 2009 monitoring). The fine/clay stratigraphy of the banks was perhaps most notable in Reaches 33 and 34, where over six feet of exposed bank was observed, and the stratigraphy of the packed fines and clay continued to the bottom of the riverbed. The conditions that would create some of these larger zones with low hydraulic conductivity may be the result of postglacial depositional environments and not due to fines temporarily deposited on the riverbed during the summer months. However, such areas of packed fine sand and clay were limited in extent throughout the model area.

Indeed, during the periods of low instream flows in 2008 and 2009, the river bottom was generally observed as cobbled medium to coarse sand and gravel where instream flow was occurring. Most areas of the active river channel did not appear to be collecting such clogging sediments in the summer months. This is especially true of the many riffle areas along the Willimantic River, which often have high velocities and low stages (e.g., Transect 8), along with the shallow to medium runs that have sufficient velocities to prevent deposition of fine sediments. Even the deep run in Reach 26 did not collect fines except near the east bank where low stages and flow impairments created low-velocity deposition zones. However, it is important to note that MMI did not observe the Willimantic River at very low flows (eight to 20 cfs) during these studies as might have occurred during the original Level A study.

As explained at the beginning of the River Package discussion, MODFLOW uses a conductance term to govern how water moves into the aquifer from each river cell and vice-versa. For a given length and width of a cell, decreasing or increasing the thickness of the streambed will require a lower or higher hydraulic conductivity, respectively, to generate the same conductance, as shown by the conductance equation (and generalized units) presented below:

$$\textit{Conductance } [L^2/T] = \frac{\textit{Width of cell } [L] * \textit{Length of cell } [L] * \textit{Thickness of streambed } [L]}{\textit{Hydraulic Conductivity of Streambed } [L/T]}$$

For a natural channel, river width tends to decrease as stream stage decreases, creating more area of the river with the low velocities necessary for the deposition of fine and organic sediments to occur. However, these depositional areas often become backwater zones where flow is insignificant, resulting in a narrower effective width of the river.

Since (1) the site-specific field data regarding riverbed hydraulic conductivity from the Level A study were not readily available, (2) qualitative data collected during the 2008 and 2009 studies were not consistent with the assessment in the Level A report, and (3) additional measurements of streambed hydraulic conductivity were not performed during the instream flow study and supplemental hydrogeologic study, MMI elected to not use the Level A values of hydraulic conductivity and streambed thickness.

Instead, areas of negligible velocity and discharge were removed from the conductance term used in MODFLOW by reducing the width of the river in each river cell. Next, the riverbed conductivity was calibrated to two feet per day using an automatic sensitivity analysis in Groundwater Vistas. This value is well within the typical values of streambed hydraulic conductivity (0.1 to 5 ft/d) used in Level A models submitted to the DEP (DEP, 2009d). Changing individual cell dimensions to manipulate conductance provided much more cell-specific information for the conductance term than in the Level A model, where high velocity riffles were counted as low velocity and low streambed conductivity zones.

The conductance terms generated for the updated numerical model were compared to those in the two calibrations of the Level A model. The input files for the 11 cfs condition in the updated model were used for the comparison against various landmarks along the Willimantic River in the model area. Table 6-1 presents the comparison.

Table 6-1
River Cell Conductance (ft^2/d)

Reference Location	Level A Low Flow	Level A High Flow	MMI Updated Model - 11 cfs	Habitat Type
Merrow Road	6,300	10,500	13,540	Pool
S-0 / P-0	1,500	1,500	2,368	Moderate depth run
P-1 / Well 1	750	200	1,832	Riffle
Well 2	375	225	1,108	Pool
USGS Gage	530	450	2,000	Pool
P-3	765	675	6,036	Riffle
P-7	375	625	1,740	Shallow run
T-8	375	1,250	1,908	Riffle
P-9	1,125	3,750	8,424	Riffle
S-11 / P-11	750	1,250	7,090	Riffle
T-14	750	1,250	5,240	Shallow run
T-16	3,600	6,000	9,840	Sidearm (pool)
T-17	4,800	8,000	14,994	Moderate depth run
T-26	1,200	2,000	8,610	Pool

In general, the Level A high flow river cell conductance is only slightly higher (up to a factor of 3.3) than the Level A low flow conductance for each of the same river cells although some of the river conductance values are actually lower in the high flow model than in the low flow model.

The updated model prepared by MMI has a higher river cell conductance than the Level A model (1.2 to nine times as high for comparable model cells). This increase may be partially due to the more precise GIS methods available today to assist in delineating the area of river in each model cell. It is recognized, however, that the current model does not attempt to simulate an organic clogging layer, which also affects the final conductance values for the model.

6.3.4 Hydrologic Stresses, Sources, and Sinks

Data from the 1999 pumping test of production well UConn #4 was used to calibrate the updated model. Recharge from rainfall was applied to the upper active layer of the entire model. Precipitation from June 1 to August 11, 1999 (prior to the test) was 6.16 inches and was 2.92 inches during the 31 days before August 11, 1999 as measured at the University weather station in Storrs, Connecticut. The precipitation rate during the summer of 1999 prior to the test period was therefore approximately 0.008 ft/d. Recharge was set at 0.004 ft/day, or 50% of the precipitation rate for the steady-state model.

The Evapotranspiration Package was used to model evapotranspiration out of the model as in the Level A model. Evapotranspiration was applied to the upper active layer of the entire model. The mean annual evapotranspiration rate in the 2004 RASA report (22 inches per year, or 0.0050 ft/d) was used for the steady-state model. For the months of August, September, October, and November, the evapotranspiration rates are 0.0094 ft/d, 0.0066 ft/d, 0.0040 ft/d, and 0.0019 ft/d based on the monthly evaporation estimates presented by the USGS for the Shetucket River Basin (Thomas et al., 1967). The monthly values were used in the transient portions of the numerical model when appropriate. An extinction depth for evapotranspiration was set at five feet, consistent with the Level A model.

Boundary flux values were initially set at 16% of mean annual precipitation (50 inches) to adjacent areas, as modified by model cell size and the size of the upgradient area of till. This value takes into account the increased amount of runoff that occurs from till areas. This value was reduced to 12% of mean annual precipitation to account for the dry period experienced in the summer of 1999.

Withdrawals from the stratified drift aquifer were simulated as pumping cells using the Well Package. This includes the four production wells at the Willimantic River Wellfield. No other withdrawals from the stratified drift aquifer were simulated as no withdrawals in the model area are believed to exist at a rate greater than 50,000 gpd, as explained in Section 2.5.

6.3.5 Selection of Aquifer Parameters

Hydraulic Conductivity

Selection and assignment of hydraulic conductivity values followed a methodical process guided by informal automatic sensitivity analyses that increased or decreased parameter values at least up to 50% throughout the calibration process. The range of values for hydraulic conductivity determined in the Level A Study (10 to 128 ft/d) was used to input initial K values into the model. The concentric rings of hydraulic conductivity used in the final Level A model were removed, and mapping in the *Quaternary Geologic Map of Connecticut* (Stone et al., 2005) was utilized to define areas of lower and higher conductivity zones in each model layer in order to simulate geologic heterogeneities in the model.

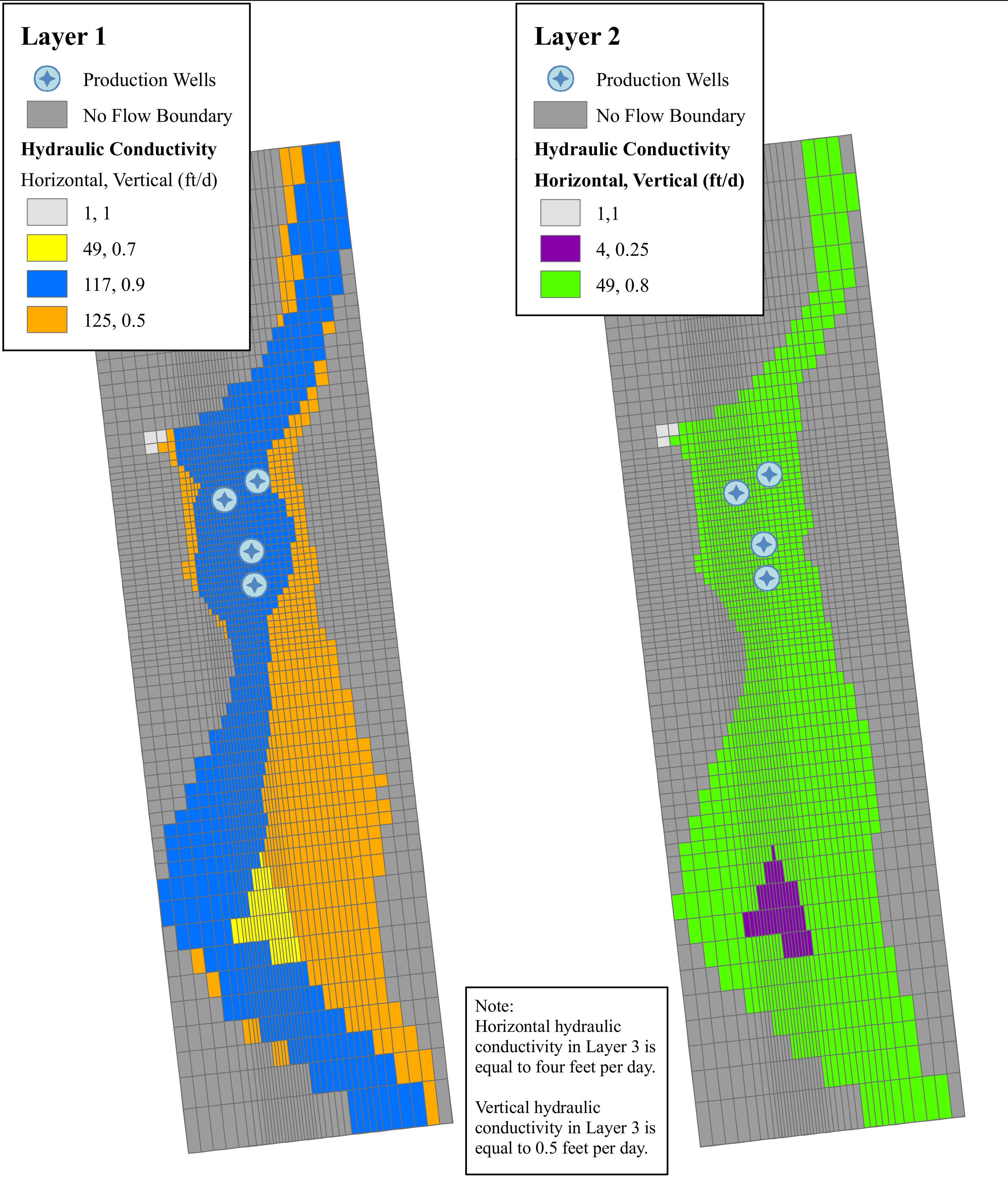
The area near where Winding Brook enters the model area was originally modeled as being active in the Level A model but was mapped as being glacial till on the more recent *Quaternary Geologic Map of Connecticut* (Stone et al., 2005). The model cells were kept as active in both Layers 1 and 2, but the hydraulic conductivity was reduced to one ft/d to reflect the relatively poor conductive nature of glacial till.

The final horizontal hydraulic conductivity values for Layer 1 include three additional zones beyond the active till zone described above. The first zone is set at 125 ft/d and is generally aligned with the sandy uplands southeast of the wellfield. The second zone is set at 117 ft/d corresponding to the central stratified drift valley associated with the Willimantic River. The third zone is set at 49 feet per day corresponding to a layer of finer sands mapped east of Transect 17. These values are believed to be reasonable given the types of deposits found in the boring logs throughout the upper model layer.

The final horizontal hydraulic conductivity values for Layer 2 include two additional zones beyond the active till zone described above. The first zone is set at four ft/d corresponding to the lower layer of the finer sands mapped east of Transect 17. The remainder of Layer 2 is set at 49 ft/d. This generally uniform value is believed to be reasonable given the types of deposits found in the boring logs for the Level A study. The final horizontal hydraulic conductivity value for the entirety of Layer 3 was simulated at a uniform four ft/d, consistent with the layer of fine sands simulated in the Level A model.

The Level A model used a horizontal to vertical hydraulic conductivity ratio of 150 to one for Layers 1 and 2 and a ratio of 100 to one for Layer 3. These values gave a range of vertical hydraulic conductivity in the Level A model from 0.26 ft/d to 0.85 ft/d. Vertical hydraulic conductivity values in the updated model were initially defined using this guideline. The values were adjusted throughout the informal automatic sensitivity analyses as delineated by the zones of horizontal hydraulic conductivity discussed above.

The final vertical hydraulic conductivity values for the updated model range from 0.25 ft/d to one ft/d, a similar range of values as used in the Level A study. These selected values are consistent with the guidance provided by Yu (1993) where it is suggested that "the vertical component of the saturated hydraulic conductivity is usually smaller (one to two orders of magnitude) than the horizontal component." Refer to Figure 6-3 for a graphical depiction of hydraulic conductivity.



Note:
Horizontal hydraulic conductivity in Layer 3 is equal to four feet per day.

Vertical hydraulic conductivity in Layer 3 is equal to 0.5 feet per day.

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Hydraulic Conductivity

LOCATION: **Coventry & Mansfield, CT**

MMI#: 1958-09
MXD: H:\Figure6-3.mxd
SOURCE: MMI

N

Willimantic River Study

Map By: SJB
Date: June 2010
Scale: 1"=1,000'

SHEET: **Figure 6-3**

Specific Storage and Specific Yield

A range of storage properties was tested as part of the informal automatic sensitivity analyses. A starting value of 0.20 was used based on that used in the Level A model. A value of 0.21 for Specific Yield was ultimately utilized as a uniform value in Layer 1 of the updated model, the average value of fine sand and the minimum value of fine gravel as presented in Johnson (1967). These types of deposits are prevalent throughout the upper layer of the model.

MODFLOW 2000 utilizes specific storage rather than storativity. Specific storage was initially set at 0.0004 as in the Level A model. The value of specific storage was adjusted slightly upwards such that transient model heads in the observation wells nearby the production wells more closely approximated the shape of the drawdown curves observed in the field. The final value of specific storage was 0.0006 in Layers 2 and 3.

6.4 Updated Model Construction and 1999 Pumping Test Calibration

The Level A model was modified in several ways for the Willimantic River Study as described above within the appropriate report sections. The model modifications are summarized below:

- Horizontal and vertical hydraulic conductivity were recalibrated and adjusted.
- Storage parameters were recalibrated and adjusted.
- Boundary fluxes were recalculated all around the model boundary in Layer 1 to a value equivalent to 12% of rainfall falling on adjacent upgradient areas.
- Boundary fluxes modeling ground water entering and exiting the model through stratified drift were removed.
- Riverbed conductivity was set to a constant two ft/d throughout the model.
- Several river input files were constructed with varying stages and widths for discharges ranging from 11 cfs to 193 cfs.

The Automatic Sensitivity Analysis in Groundwater Vistas was utilized to assist with model calibration. The purpose of these analyses was to evaluate the strength of the model parameters and provide confidence to the model. These analyses were based on measured heads collected before and during the 1999 pumping test. Adjusted parameters included recharge, hydraulic conductivity, specific storage, boundary flux, and riverbed conductivity. Model parameters were adjusted from 25% to 400% of the starting value during each analysis. The results indicated that the model was particularly sensitive to changes of the horizontal hydraulic conductivity parameters. The final parameter values utilized in the updated model resulted in acceptable model runs for the 1999, 2008, and 2009 pumping tests.

As the 1999 pumping test included extensive data collection and occurred during an extended period of very low flows on the Willimantic River, that pumping test was selected as the dataset to formally calibrate the updated model. A total of 27 observation points were monitored – four production wells, five piezometers, and 18 observation wells – although points were not necessarily monitored on all dates, and not all of the data were usable for this study. For example:

- Piezometer mapping for the 1999 test only revealed the locations of three of the five piezometers, so the locations of P-1 and P-2 were not known.
- Upon review, the datalogger data collected in production Well #3 appeared erroneous as water elevations reached above the ground surface. This data was therefore discarded.
- Other observation wells were either outside of the model (Ms-75) or were believed to reflect ground water levels associated with discontinuous units of finer materials that could not be modeled (such as Ms-68 and Ms-76).

Thus, a total of 21 targets were available to calibrate the model. Survey elevations of each observation well were measured during the supplemental hydrogeologic study. The survey data collected by MMI (in the National Geodetic Vertical Datum of 1929, or NGVD 1929) showed an average discrepancy of approximately four feet from the survey data collected by LEI. According to representatives of the University and The Connecticut Water Company, the observation well standpipes were added after the 1999 pumping test. Assuming that the LEI survey data was measured using the North American Vertical Datum of 1988, (a vertical datum that is approximately one foot lower than NGVD 1929), the discrepancy would be three feet (the approximate height of the majority of the standpipes at the wellfield).

In order to correlate the 1999 data with data collected by MMI in 2008 and 2009, the dataset for each observation well was lowered by the discrepancy (if known) or by approximately four feet (if unknown) to be in line with the 2008 and 2009 MMI surveys that were measured based on the National Geodetic Vertical Datum of 1929.

The model was set with five stress periods with one time-step per day related to the 1999 pumping test. Model stress periods are defined in Table 6-2. The first stress period was a steady-state period with calibration targets being those measured by LEI on August 11, 1999. This is the earliest data collection date and was believed to correspond with the end of normal wellfield operation prior to the pumping test. The wellfield was either shut down or had limited production on August 11, allowing ground water levels beneath the wellfield to recover. For the purpose of the model, it was assumed that the wellfield was completely shut down although that was not likely the case prior to August 16, 1999. Pumping rates during this period were determined based on monthly values for the three months preceding the test.

Table 6-2
1999 Model Stress Period Setup

Date	Model Day	Stress Period
8/11/1999	92	1
8/12/1999	93	2
8/13/1999	94	2
8/14/1999	95	2
8/15/1999	96	2
8/16/1999	97	2
8/17/1999	98	2
8/18/1999	99	2
8/19/1999	100	2
8/20/1999	101	3
8/21/1999	102	4
8/22/1999	103	4
8/23/1999	104	4
8/24/1999	105	5
8/25/1999	106	5
8/26/1999	107	5
8/27/1999	108	5
8/28/1999	109	5

Note that model day is used in MODFLOW instead of dates. The model day in these models is defined simply as the starting day of the pumping test being day 100. It is defined this way by MMI in order to be able to add stress periods prior to the start of pumping without having to change calibration targets.

The four remaining stress periods were all modeled as transient simulations. The second stress period consisted of an eight-day simulation with no pumping. During this period, the depressed ground water levels at the wellfield recovered to the pretest condition measured on August 19, 1999.

The third stress period lasted one day and corresponded to the start of the Well #4 pumping test. From August 19, 1999 to August 20, 1999, Well #4 pumped at 290 gpm. On the morning of August 20, 1999, the turbine settings in Well #4 were adjusted, and it was restarted at a rate of 490 gpm. This was the pumping rate for the fourth stress period corresponding to the 72-hour yield test of Well #4 ending August 23, 1999.

The fifth and final stress period was the five-day yield test of the wellfield. Well #4 pumped at 414 gpm during this period, Well #1 pumped at 287 gpm, and Well #3 pumped at 281 gpm for a wellfield total of 982 gpm, or approximately 1.4 mgd.

Other inputs to the model included recharge, evapotranspiration, initial heads, and river conductance. Recharge was set to 50% of precipitation for the preceding period during the first stress period and 50% of precipitation that fell during each transient stress period. Precipitation data was obtained from the National Weather Service for the weather station at the UConn Agricultural Experiment Station in Storrs, Connecticut. The recharge rates for the five stress periods were equal to 0.004 ft/d for stress period 1 (based on 50% of the 6.16 inches of rain that fell since June 1, 1999), 0.005 ft/d for stress period 2, zero for stress period 3, 0.003 ft/d for stress period 4, and 0.001 ft/d for stress period 5. Evapotranspiration was set to 0.0050 ft/d for the steady-state stress period and to 0.0094 ft/d for the transient stress periods based on the values presented in Section 6.3.4, with a five foot extinction depth as used in the Level A model.

Initial heads in the model were based on a steady-state no-pumping scenario run after the model was run with initial heads in each model layer of 300 feet. The average of the mean daily discharges in the natural dataset created for the Willimantic River at the wellfield from August 11, 1999 through August 28, 1999 was 10.7 cfs, with a high of 17.7 cfs and a low of 7.7 cfs. Thus, the 11 cfs river input file was utilized to simulate river stages and widths around the time of the 1999 pumping test.

The 1999 model was considered calibrated when the simulated potentiometric surface was consistent with topography, with high ground water levels in outlying areas to the north, east, and west and low ground water levels from the center to the south of the model. The model was also considered calibrated when the lines of simulated head responded realistically to the river cells in the model. Results are depicted for Layer 2 on August 19, 1999, August 23, 1999, and August 28, 1999 on Figures 6-4, 6-5, and 6-6.

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Simulated Heads (in Feet NGVD 1929) in Layer 2, 8/19/1999		LOCATION: Coventry & Mansfield, CT
MMI#: 1958-09 MXD: H:\Figure6-4.mxd SOURCE: Microsoft Virtual Earth	Willimantic River Study	Map By: SJB Date: June 2010 Scale: 1"=500'
		SHEET: Figure 6-4

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Simulated Heads (in Feet NGVD 1929) in Layer 2, 8/23/1999			LOCATION: Coventry & Mansfield, CT	
MMI#: 1958-09 MXD: H:\Figure6-5.mxd SOURCE: Microsoft Virtual Earth	N	Willimantic River Study	Map By: SJB Date: June 2010 Scale: 1"=500'	SHEET: Figure 6-5

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Simulated Heads (in Feet NGVD 1929) in Layer 2, 8/28/1999

LOCATION: Coventry & Mansfield, CT

MMI#: 1958-09
MXD: H:\Figure6-6.mxd
SOURCE: Microsoft Virtual Earth

N

Willimantic River Study

Map By: SJB
Date: June 2010
Scale: 1"=500'

SHEET: Figure 6-6

The 1999 model was also considered calibrated when the simulated head was within two feet of the observed head in greater than 50% of the observation wells, when the simulated head was within five feet of the observed head in greater than 70% of the observation wells, and when the simulated head was within 10 feet of the observed head in 100% of the observation wells. This is the criterion used for Level A models. As an additional check of confidence, plots of observed and simulated head were produced for each observation well to ensure a realistic simulation of observed heads. Tables and graphs related to individual calibration targets are provided in Appendix E. Refer to Table 6-3 for a summary of the residuals in the 1999 model.

Table 6-3
Residuals Associated with Model of 1999 Pumping Test

Model Day	Date	Residuals Within Two Feet	Residuals Within Five Feet	Residuals Within Ten Feet	Number of Targets
92	8/11/1999	8	6	0	14
93	8/12/1999	13	3	0	16
94	8/13/1999	12	7	0	19
95	8/14/1999	13	7	0	20
96	8/15/1999	13	7	0	20
97	8/16/1999	13	6	0	19
98	8/17/1999	16	4	0	20
99	8/18/1999	19	2	0	21
100	8/19/1999	19	2	0	21
101	8/20/1999	17	3	1	21
102	8/21/1999	18	3	0	21
103	8/22/1999	18	3	0	21
104	8/23/1999	19	2	0	21
105	8/24/1999	16	5	0	21
106	8/25/1999	16	5	0	21
107	8/26/1999	17	3	1	21
108	8/27/1999	17	3	1	21
109	8/28/1999	17	3	1	21

For model day 92, 57 % of the 14 targets had residuals within two feet, and 100% had residuals within five feet. This model day was limited by the number of available targets and the uncertainty of the well pumping rates. For model day 93, 81% of the 16 targets had residuals within two feet, with 100% having residuals within five feet. Once the wellfield begins pumping (model day 100), 80% to 90% of the target residuals fall within two feet. The target residual greater than five feet is associated with the production well UConn #4 although it is recognized that pumping wells are often inappropriate targets due to wellbore inefficiencies. The large amount of residuals within two feet, combined with the well-matched observed and simulated drawdown graphs in Appendix E, provide confidence that the model is properly calibrated.

6.5 Model Verification

The three 72-hour pumping tests conducted in 2008 and 2009 for the supplemental hydrogeologic study were used to verify the model. Two separate models were set up to simulate these three events. The first model simulates the period leading up to and following the 2008 monitoring event from August 14, 2008 until September 5, 2008, when a severe high flow event abruptly ended the summer low-flow period at the wellfield. The second model simulates the autumn of 2009 from September 18, 2009 through November 11, 2009.

Model parameters remained unchanged in these models although model inputs (stress periods, recharge, evapotranspiration, river conductance, and initial heads) were adjusted as necessary to realistically simulate conditions. These models are described below.

6.5.1 2008 Hydrogeologic Monitoring

Recall from Section 5.3.1 that the first supplemental hydrogeologic monitoring event occurred beginning August 18, 2008 and ended on August 21, 2008. The wellfield was run under normal operation before and after this monitoring event. Data collection prior to this event occurred beginning on August 14, 2008. Data was collected at a total of 19 monitoring points although data collection was limited on August 14, 2008 (nine observations), and P3-E was dry after August 18, 2008. The data at Ms-68 was not used (as described in Section 5.2.1), so a total of 18 observation points were available to verify the model.

The model was set up with seven stress periods with one time-step per day related to the period around the 2008 pumping test. Model inputs (recharge rate, river discharge, and pumping rates) are defined in Table 6-4. The first stress period was a steady-state simulation with calibration targets being those measured by MMI on August 14, 2008. Pumping rates during this period were determined based on average values for the months of June, July, and the first half of August in 2008. The remaining pumping values were based on daily production values.

Evapotranspiration was set to a constant 0.0050 ft/d for the steady-state stress period and 0.0094 ft/d for the transient stress periods, with the exception of the final stress period in September (0.0066 ft/d). A five foot extinction depth was used as in the Level A model. Recharge rates were again defined based on data obtained from the National Weather Service for the UConn Agricultural Experiment Station in Storrs, Connecticut. As shown in Table 6-4, a variety of river discharges was simulated in this model.

The 2008 model was considered verified when the simulated potentiometric surface was consistent with topography, with high ground water levels in outlying areas to the north, east, and west and low ground water levels from the center to the south of the model. The model was also considered verified when the lines of simulated head responded realistically to the river cells in the model.

Table 6-4
2008 Model Setup

<table>
<tr><th rowspan="2">Model Day</th><th rowspan="2">Stress Period</th><th rowspan="2">Date</th><th rowspan="2">Discharge (cfs)</th><th rowspan="2">Recharge (ft/d)</th><th colspan="4">Average Production (gpm)</th></tr>
<tr><th>Well #1</th><th>Well #2</th><th>Well #3</th><th>Well #4</th></tr>
<tr><td>100</td><td>1</td><td>8/14/2008</td><td>193</td><td>0.0065</td><td>234</td><td>51</td><td>225</td><td>154</td></tr>
<tr><td>101</td><td rowspan="3">2</td><td>8/15/2008</td><td>193</td><td rowspan="3">0.0024</td><td rowspan="3">125</td><td rowspan="3">30</td><td rowspan="3">125</td><td rowspan="3">70</td></tr>
<tr><td>102</td><td>8/16/2008</td><td>193</td></tr>
<tr><td>103</td><td>8/17/2008</td><td>193</td></tr>
<tr><td>104</td><td rowspan="2">3*</td><td>8/18/2008</td><td>167</td><td rowspan="2">None</td><td rowspan="2">359</td><td rowspan="2">95</td><td rowspan="2">365</td><td rowspan="2">206</td></tr>
<tr><td>105</td><td>8/19/2008</td><td>167</td></tr>
<tr><td>106</td><td rowspan="2">4*</td><td>8/20/2008</td><td>101</td><td rowspan="2">0.0002</td><td rowspan="2">373</td><td rowspan="2">97</td><td rowspan="2">377</td><td rowspan="2">215</td></tr>
<tr><td>107</td><td>8/21/2008</td><td>101</td></tr>
<tr><td>108</td><td rowspan="5">5</td><td>8/22/2008</td><td>79</td><td rowspan="5">None</td><td rowspan="5">357</td><td rowspan="5">82</td><td rowspan="5">353</td><td rowspan="5">204</td></tr>
<tr><td>109</td><td>8/23/2008</td><td>79</td></tr>
<tr><td>110</td><td>8/24/2008</td><td>79</td></tr>
<tr><td>111</td><td>8/25/2008</td><td>79</td></tr>
<tr><td>112</td><td>8/26/2008</td><td>79</td></tr>
<tr><td>113</td><td rowspan="8">6</td><td>8/27/2008</td><td>43</td><td rowspan="8">0.0006</td><td rowspan="8">383</td><td rowspan="8">88</td><td rowspan="8">387</td><td rowspan="8">216</td></tr>
<tr><td>114</td><td>8/28/2008</td><td>43</td></tr>
<tr><td>115</td><td>8/29/2008</td><td>43</td></tr>
<tr><td>116</td><td>8/30/2008</td><td>43</td></tr>
<tr><td>117</td><td>8/31/2008</td><td>43</td></tr>
<tr><td>118</td><td>9/1/2008</td><td>43</td></tr>
<tr><td>119</td><td>9/2/2008</td><td>43</td></tr>
<tr><td>120</td><td>9/3/2008</td><td>43</td></tr>
<tr><td>121</td><td rowspan="2">7</td><td>9/4/2008</td><td>33</td><td rowspan="2">None</td><td rowspan="2">451</td><td rowspan="2">106</td><td rowspan="2">455</td><td rowspan="2">253</td></tr>
<tr><td>122</td><td>9/5/2008</td><td>33</td></tr>
</table>

*Stress periods corresponding to the 72-hour test

The 2008 model was also considered verified when the simulated head was within two feet of the observed head in greater than 50% of the observation wells, when the simulated head was within five feet of the observed head in greater than 70% of the observation wells, and when the simulated head was within 10 feet of the observed head in 100% of the observation wells. As an additional check of confidence, plots of observed and simulated head were produced for each observation well to ensure a realistic simulation of observed heads. Tables and graphs related to individual calibration targets are provided in Appendix E. Refer to Table 6-5 for a summary of the residuals in the 2008 model.

Table 6-5
Residuals Associated With Model of 2008 Pumping Test

Model Day	Date	Residuals 0-2 feet	Residuals 2-5 feet	Residuals 5-10 feet	Number of Targets
100	8/14/2008	7	2	0	9
104	8/18/2008	16	2	0	18
107	8/21/2008	15	2	0	17
122	9/5/2008	13	4	0	17

All residuals were within five feet. For model day 100, 78% of the nine targets had residuals within two feet. This model day was limited by the number of available targets. For model day 104, 89% of the 18 targets had residuals within two feet. The end of the hydrogeologic monitoring event (model day 107) had 88% of the targets within two feet. Finally, the end of the simulation (model day 122) had 76% of the simulated heads fall within two feet of the observed heads.

This fact, combined with the well-matched observed and simulated drawdown graphs in Appendix E, provide confidence that the model was properly verified. While there is limited data for many of the graphs in Appendix E, the matching slope and the good fit at Ms-65 show that the model appropriately simulated the environmental conditions at the wellfield in late summer of 2008.

6.5.2 2009 Hydrogeologic Monitoring

Recall from Sections 5.3.2 and 5.3.3 that the second and third supplemental hydrogeologic monitoring events occurred from September 22, 2009 to September 24, 2009 and from November 10, 2009 to November 12, 2009. The wellfield was run under normal operation before, between, and after these monitoring events. Data collection for these two events occurred on September 18, 2009, September 23, 2009, and

November 12, 2009. Data was collected at a total of 23 monitoring points although P3-E was dry throughout the monitoring period. The data at Ms-68 was not used (as described in Section 5.2.1), so a total of 21 observation points were available to verify the model.

The model was set up with eight stress periods with one time step per day related to the period around the 2009 pumping tests. Model inputs (recharge rate, river discharge, and pumping rates) are defined in Table 6-6. The first stress period was a steady-state simulation with calibration targets being those measured by MMI on September 18, 2009. Pumping rates during this period were determined based on average values for the months of June, July, August, and the first half of September in 2009. The remaining pumping values were based on daily production values.

Evapotranspiration continued to utilize a five-foot extinction depth as used in the Level A model, but rates were varied based on the average monthly rates in Thomas et al. (1967). The recharge rates were again defined based on data obtained from the National Weather Service for the UConn Agricultural Experiment Station in Storrs, Connecticut. As shown in Table 6-6, a variety of river discharges were simulated in this model. Note that stress period five represents the entire month of October 2009.

Table 6-6
2009 Model Setup

Model Day	Stress Period	Date	Discharge (cf/s)	ET (ft/d)	Recharge (ft/d)	Average Production (gpm) Well #1	Well #2	Well #3	Well #4
96	1	9/18/2009	33	0.0050	0.0766	341	69	309	200
97	2	9/19/2009	33	0.0066	None	263	90	263	157
98		9/20/2009							
99		9/21/2009							
100	3[1]	9/22/2009	33	0.0066	0.0004	435	158	317	244
101		9/23/2009							
102		9/24/2009							
103	4	9/25/2009	33	0.0066	0.0113	414	165	414	230
104		9/26/2009							
105		9/27/2009							
106		9/28/2009							
107		9/29/2009							
108		9/30/2009							
109	5	10/1/2009	79	0.0040	0.007	302	118	292	168
110		10/2/2009							
Etc.		Etc.							
138		10/30/2009							
139		10/31/2009							
140	6	11/1/2009	79	0.0019	0.001	277	105	280	154
141		11/2/2009							
142		11/3/2009							
143		11/4/2009							
144		11/5/2009							
145		11/6/2009							
146	7	11/7/2009	43	0.0019	None	260	99	261	148
147		11/8/2009							
148		11/9/2009							
149	8[2]	11/10/2009	43	0.0019	None	449	174	455	246
150		11/11/2009							
151		11/12/2009							

1. Corresponds to 72-hour test completed in September
2. Corresponds to 72-hour test completed in November

The 2009 model was considered verified when the simulated potentiometric surface was consistent with topography, with high ground water levels in outlying areas to the north, east, and west and low ground water levels from the center to the south of the model. The model was also considered verified when the lines of simulated head responded realistically to the river cells in the model.

The 2009 model was also considered verified when the simulated head was within two feet of the observed head in greater than 50% of the observation wells, when the simulated head was within five feet of the observed head in greater than 70% of the observation wells, and when the simulated head was within 10 feet of the observed head in 100% of the observation wells. As an additional check of confidence, plots of observed and simulated head were produced for each observation well to ensure a realistic simulation of observed heads. Tables and graphs related to individual calibration targets are provided in Appendix E. Refer to Table 6-7 for a summary of the residuals in the 2009 model.

Table 6-7
Residuals Associated with Model of 2009 Pumping Tests

Model Day	Date	Residuals 0-2 feet	Residuals 2-5 feet	Residuals 5-10 feet	Number of Targets
96	9/18/2009	17	4	0	21
101	9/23/2009	19	2	0	21
151	11/12/2009	16	5	0	21

All residuals were within five feet. For model day 96, 81% of the 21 targets had residuals within two feet. For model day 101, 90% of the 21 targets had residuals within two feet. Finally, the end of the simulation and the second hydrogeologic monitoring event (model day 151) had 76% of the simulated heads fall within two feet of the observed heads. The drop in low residuals in the last model day is attributed to the entire month of October being modeled as one stress period. The fact that over 70% of the residuals for the model fall within two feet provides confidence that the model is properly verified.

While there is limited data for many of the graphs in Appendix E, the simulated slopes over the model period and the good fit at Ms-65 show that the model appropriately modeled the environmental conditions at the wellfield in autumn of 2009.

7.0 PREDICTIVE SIMULATIONS

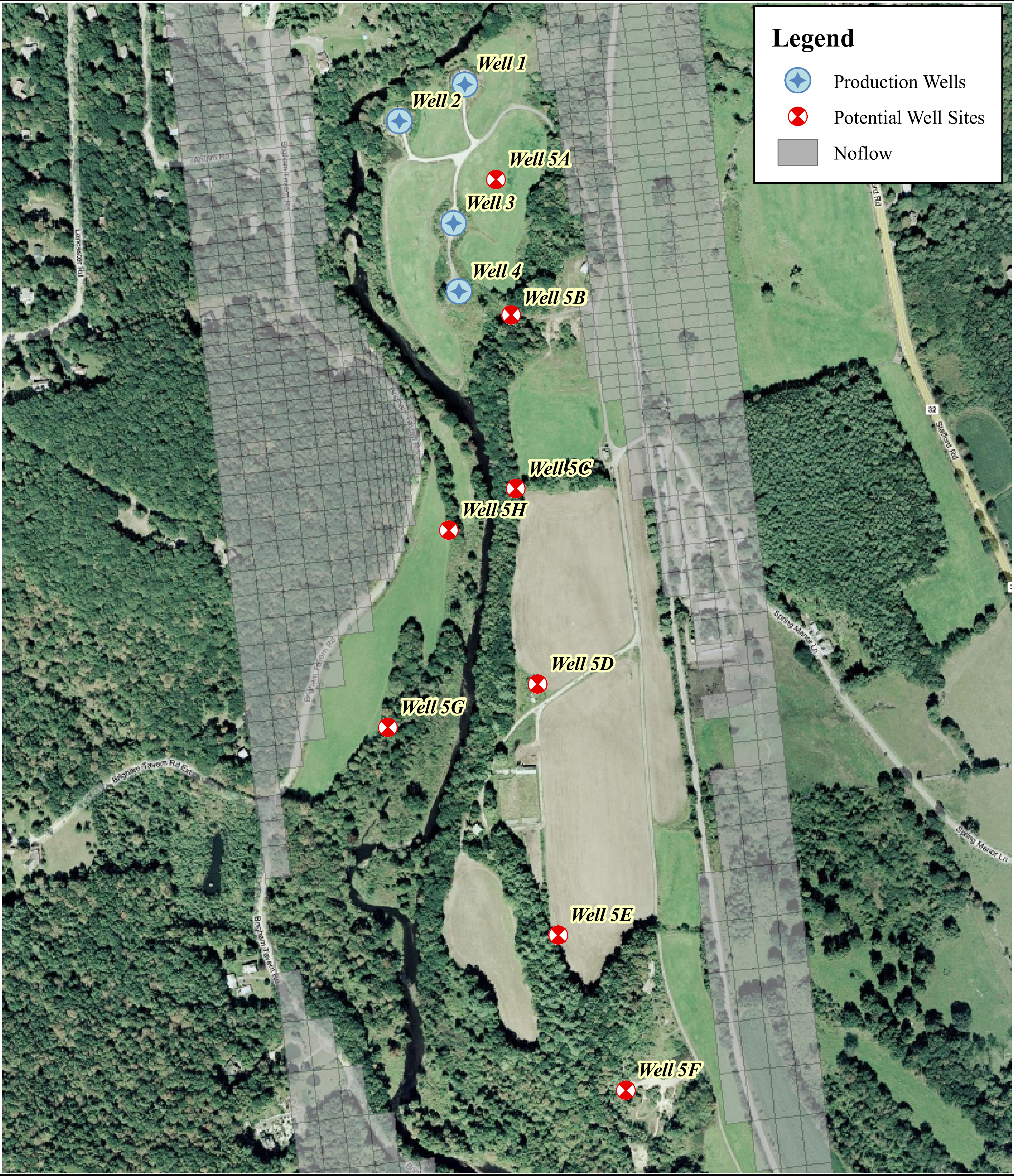
The updated numerical model was used to simulate the timing and magnitude of pumping on the stage and discharge in the Willimantic River under various management scenarios. Should the hydrogeologic analysis indicate that the relationship between pumping and reduced streamflow is not immediate and direct, then it may be possible to optimize ground water withdrawals under a set of constraints based on streamflow or stream stage.

The four existing production wells and eight theoretical production well locations within the model area were first simulated to determine the timing of pumping impacts. The existing wells and several of the theoretical wells were then simulated under various pumping management scenarios to determine if withdrawals can be managed to minimize adverse habitat impacts while meeting water supply demands.

The locations of the actual and theoretical wells used in the predictive simulations are shown on Figure 7-1. Wells 5A through 5F are located on state-owned lands on the east side of the Willimantic River while Wells 5G and 5H are located on state-owned lands on the west side of the Willimantic River. The expense of actually constructing these wells and the associated infrastructure was not considered in this study. It is important to note that the simulations performed here for the theoretical well locations should not be used to determine potential well yields in lieu of site-specific hydrogeologic investigations.

7.1 Timing of Pumping Impacts

The updated numerical model was used to simulate how long it would take a particular well to reduce flow in the Willimantic River. The model was set to have two stress periods – the first a steady-state period with no pumping to define a baseline, unaffected river condition (using the 11 cfs river input file), and the second a seven-day transient stress period with 168 time steps to provide results in hours.



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Existing and Potential Well Locations Used in Predictive Simulations

LOCATION: Coventry & Mansfield, CT

MMI#: 1958-09

MXD: H:\Figure7-1.mxd

SOURCE: Microsoft Virtual Earth

N

Willimantic River Study

Map By: SJB

Date: Feb. 2010

Scale: 1"=500'

SHEET: Figure 7-1

Aquifer parameters from the 1999 pumping test model were used to represent a conservative condition. Only one well was pumped at a time to provide an estimate without interference as it was assumed that multiple wells pumping would decrease the time to a pumping impact on the river.

The purpose of these simulations was to determine if a change in existing well location or the creation of additional wells could mitigate discharge reductions assumed to occur in the Willimantic River during low-flow periods. This mitigation would occur by operating a downstream well or a well more distant from the river over a short period to remove ground water from storage rather than ground water that would otherwise recharge the river. Higher pumping rates were used where the model simulated that the aquifer was able to support the withdrawal. A summary of the output from the various simulations to generate the time to river impact is shown in Table 7-1.

Table 7-1
Simulated Time to Reduction in Flow in the Willimantic River Due to Pumping

Well	Pumping Rate, gpm	Time Until 0.01 cfs Reduction in Flow, hr	Comment
Well 1	440	10	
Well 2	190	12	
Well 3	440	16	
Well 4	250	16	Existing Screen
Well 4	250	16	Screen at elev. 228'
Well 5A	190	28	
Well 5A	440	22	
Well 5B	190	23	
Well 5C	190	9	
Well 5D	190	20	
Well 5E	190	65	
Well 5E	440	50	Dry cell at 3.2 days
Well 5F	190	31	
Well 5G	190	23	
Well 5H	190	12	
Well 5H	440	9	

The predictive simulation model output shows that use of the existing wells will begin to reduce discharge in the Willimantic River with 10 to 16 hours of pumping. Moving the bottom of the screen in UConn Well #4 downwards to the bottom of the model layer had no effect on the timing of river impact. Wells 5A through 5H represent potential well locations, and Wells 5E and 5F show that water can be withdrawn with no impact to the river for at least 1.25 days. Well 5E provides over two days with no impacts to instream flow; however, the model indicates that the aquifer at this well cannot support a pumping rate of 440 gpm for more than 3.2 days.

Recall from Section 4.3.5 that during baseflow the Willimantic river at the wellfield is expected to require five or six days to recede from 27 cfs to 19 cfs, four to six days to recede from 19 cfs to 15 cfs, four or five days to recede from 15 cfs to 12 cfs, and at least seven days to recede from 12 cfs to 7.8 cfs. Assuming that the University is pumping the Willimantic River Wellfield at a constant rate, the timing of these recessions would not change appreciably though they are expected to be slightly faster than the number of days described above.

The results of these predictive simulations indicate that very little benefit (perhaps up to three days without discharge reduction) would be gained from moving production wells to reduce the timing of impacts. Furthermore, this analysis relies on a series of simple assumptions. For example, wells were pumped one at a time starting from nonpumping conditions, thus interference effects between wells were neglected. Multiple wells that were in production would only tend to decrease the time to discharge reduction in the river. Even if a benefit of one or two days could be gained by moving the location of the production wells, the river would still recede at the natural rates.

In summary, while it is possible that moving production wells to reduce the timing of impacts may marginally reduce discharge reduction in the river, it is believed that the cost of constructing additional wells merely to change the timing of impacts would far outweigh the benefits to fisheries habitat. The amount of discharge reduction with the entire wellfield pumping is explored in the following section.

7.2 Pumping Management Scenarios

The updated numerical model was next used to simulate how various pumping management scenarios affect discharge spatially along the reach of the Willimantic River within the model. The model was set to have two stress periods – the first a steady-state period with no pumping and the second a 31-day transient stress period. Aquifer parameters from the 1999 pumping test model were used to represent a conservative condition.

A total of 12 scenarios was run to determine the reduction in discharge due to pumping of the wellfield. Simulated pumping was set to 24 hours per day. The pumping scenarios are outlined in Table 7-2.

Table 7-2
Pumping Management Scenario Setup

Well	Scenario Pumping Rates (gpm)										
	1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**
Well 1	360	440	340	-	-	-	120	-	440	-	450
Well 2	110	170	-	-	-	-	120	-	170	-	203
Well 3	360	440	440	440	440	300	160	440	-	270	450
Well 4	210	270	270	270	160	300	120	-	270	-	500
Well 5A	-	-	170	70	180	180	120	-	-	300	-
Well 5B	-	-	100	100	100	100	100	-	-	100	-
Well 5D	-	-	-	-	170	170	120	170	-	-	-
Well 5E	-	-	-	-	170	170	120	170	-	180	-
Well 5F	-	-	-	-	100	100	100	100	-	-	-
Well 5G	-	-	-	-	-	-	120	-	-	170	-
Well 5H	-	-	-	440	-	-	120	440	440	300	-
Total Rate (mgd)	**1.50**	**1.90**	**1.90**	**1.90**	**1.90**	**1.90**	**1.90**	**1.90**	**1.90**	**1.90**	**2.31**

The first scenario (Scenario 0) was run with no wells pumping to establish a baseline of the normal gain in river flow through the model that could be compared to the pumping scenarios. The model showed that the Willimantic River gained 0.83 cfs from Merrow Road to Transect 26 at the end of the second stress period (31 days of no pumping).

Scenario 1 was set to a typical demand day pumping rate of 1.50 mgd such as that used in the 2008 hydrogeologic monitoring event. Scenarios 2 through 10 simulate various pumping rates at the existing and theoretical well locations at a total rate of 1.90 mgd, a rate greater than the average day demand projection for 2020 (Ritsick, 2004). Scenario 11 simulates each existing well pumping at its individually registered withdrawal rate, with pumping from Well #2 slightly reduced such that the total wellfield withdrawal is at the combined registered withdrawal rate of approximately 2.3077 mgd.

Pumping scenario results were compared to the no-pumping reference simulation (Scenario 0). The river flux at each river node in each of the 11 scenarios was subtracted from the river flux at the same node during the reference simulation to calculate simulated loss to instream flow. Figure 7-2 depicts the streamflow loss at each river node (which is cumulative in the downstream direction). A summary of the simulated instream flow loss near various landmarks from this study is provided in Table 7-3.

Figure 7-2
Cumulative Streamflow Loss Under Simulated Pumping Management Scenarios

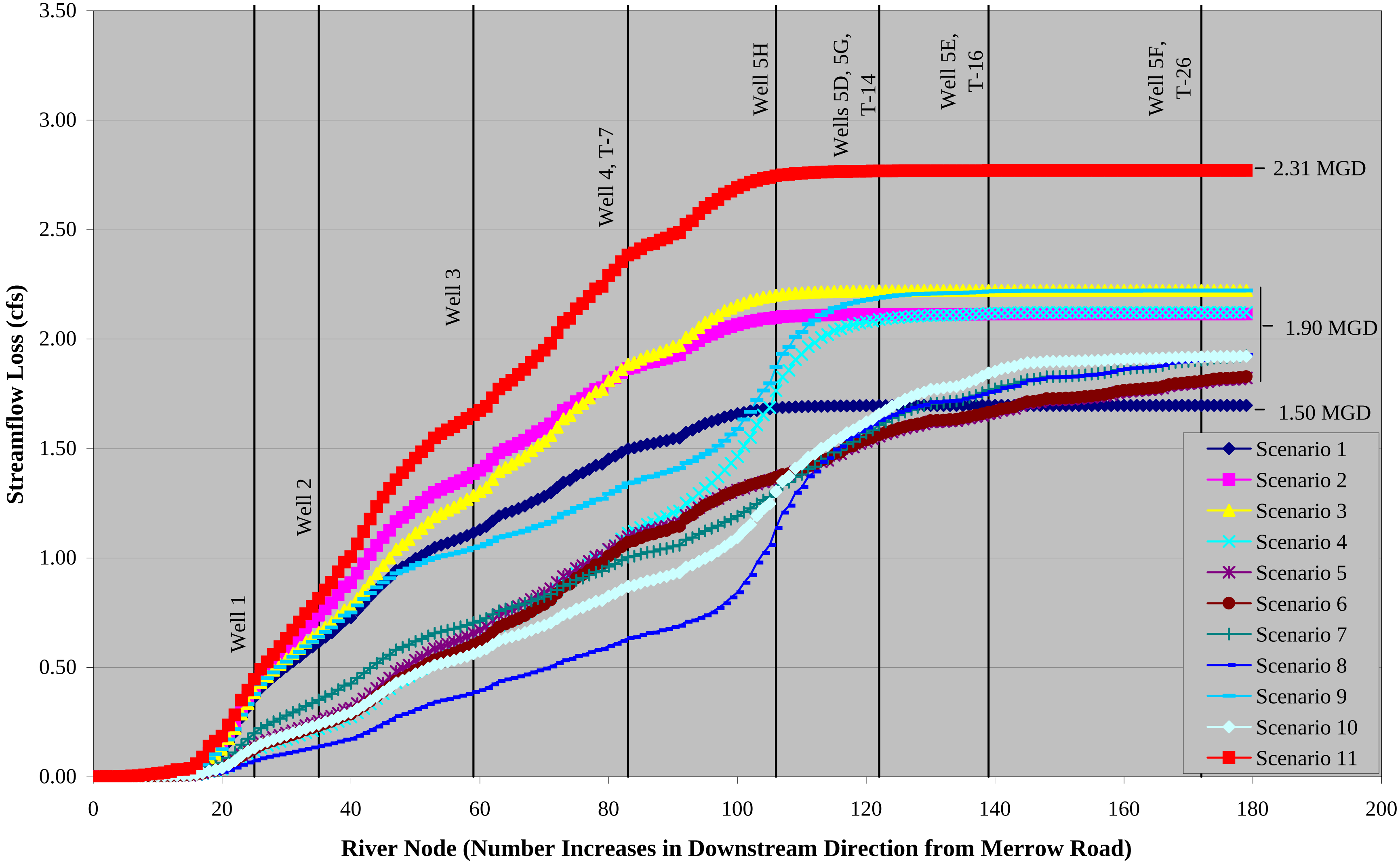

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Table 7-3
Summary of Cumulative Streamflow Loss for Pumping Management Scenarios

Land-mark	Tested Scenario (all flows in cfs)										
	1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**
Well 1	0.36	0.41	0.38	0.12	0.14	0.13	0.20	0.07	0.38	0.13	0.45
Well 2	0.62	0.74	0.66	0.22	0.25	0.23	0.35	0.14	0.64	0.24	0.82
Well 3	1.09	1.35	1.25	0.58	0.63	0.58	0.69	0.37	1.03	0.54	1.62
Well 4	1.39	1.73	1.70	0.95	0.96	0.92	0.91	0.56	1.24	0.77	2.16
T-7	1.46	1.82	1.83	1.06	1.06	1.02	0.97	0.61	1.30	0.84	2.31
T-8	1.50	1.87	1.89	1.12	1.10	1.08	1.01	0.64	1.34	0.87	2.39
Well 5H	1.68	2.09	2.19	1.65	1.35	1.35	1.26	1.02	1.76	1.22	2.73
Well 5D	1.69	2.11	2.22	2.04	1.47	1.47	1.48	1.49	2.14	1.54	2.76
Well 5G	1.70	2.11	2.22	2.08	1.53	1.54	1.57	1.60	2.18	1.62	2.77
T-14	1.70	2.11	2.22	2.09	1.57	1.57	1.62	1.64	2.19	1.67	2.77
T-16	1.70	2.11	2.22	2.11	1.65	1.65	1.74	1.74	2.21	1.81	2.77
Well 5E	1.70	2.11	2.22	2.12	1.66	1.66	1.77	1.76	2.22	1.84	2.77
Well 5F	1.70	2.11	2.22	2.12	1.77	1.78	1.87	1.87	2.22	1.91	2.77
T-26	1.70	2.11	2.22	2.12	1.80	1.80	1.90	1.90	2.22	1.92	2.77
Overall Loss (cfs)	**1.70**	**2.11**	**2.22**	**2.12**	**1.80**	**1.80**	**1.90**	**1.90**	**2.22**	**1.92**	**2.77**

The data in Figure 7-2 and Table 7-3 demonstrate that it is possible to shift some of the diminution of instream flow in a downstream direction and that there are pumping scenarios that result in lower loss to instream flow after 31 days of constant pumping at 1.90 mgd than others. The difference between the existing wellfield operation (Scenario 2) and the "best" proposed conditions (Scenarios 5 and 6) is a reduction in overall streamflow loss of 0.31 cfs (140 gpm).

While 0.31 cfs could potentially be important to protect fisheries habitat at very low flows and could potentially assist with preventing earlier occurrence of the more severe stages of the drought management response plan discussed in Section 4.4, it is also important to note that 140 gpm is 10.6% of the pumping rate of 1.90 mgd. This is approximately the same percent reduction achieved by the University through the water conservation measures enacted on campus in 2007. It is not likely that the benefits of this 0.31 cfs reduction would outweigh the costs of the five additional wells necessary to achieve Scenarios 5 and 6.

8.0 FINDINGS

The Willimantic River consistently conveys more water at the Willimantic River Wellfield than the Fenton River conveys at the Fenton River Wellfield. For this reason, it has historically been considered the more appropriate river for supporting public water supply withdrawals. The instream flow study portion of the Willimantic River Study has resulted in some distinctive findings, especially when compared to the Fenton River Study:

- The Willimantic River Wellfield is not capable of running the Willimantic River dry as the maximum legal withdrawal of 2.3077 mgd is only 3.6 cfs, and 3.6 cfs is roughly 60% of the value of the lowest instream flows believed to have occurred at the wellfield.
- From the perspective of fish habitats, a very low flow may be "rare" on the Willimantic River but not especially rare on the Fenton River.
- As a result, the UCUT curves for the Willimantic River are shifted in comparison to the UCUT curves for the Fenton River, and differentiation of the common, critical, extreme, and rare thresholds is more challenging.
- The critical threshold for the Fenton River occurs around 15% of maximum WUA whereas the critical threshold for the Willimantic River occurs around 30% of maximum WUA.
- Fish species in the Willimantic River routinely experience a relatively lower loss of habitat than fish species in the Fenton River. In other words, fish routinely enjoy relatively greater amount of habitat in the Willimantic River.
- Nevertheless, a strict interpretation of the UCUT curves for the Willimantic River would tend to call for protection to a higher standard (maintaining a greater percent of maximum WUA for each species) than the interpretation of the UCUT curves for the Fenton River.
- If cutbacks in wellfield withdrawals were linked with the common, critical, extreme, and rare thresholds, the Willimantic River would be asked to protect a proportionally

greater quantity of habitat than the Fenton River (nearly double for the critical flow) largely because it conveys more water.

- However, unlike the Fenton River where the common, critical, rare, and extreme habitat stress thresholds can be met in a matter of days from one to the next, the Willimantic River may require several days to pass through these thresholds. This will allow for a more methodical response from the University.

The hydrogeologic study portion of the Willimantic River Study has resulted in an updated numerical model that works well under a variety of wellfield pumping scenarios. Some distinctive findings include the following:

- Effects of wellfield withdrawals are manifested in reduced ground water discharge and induced infiltration within 10 to 16 hours for each existing well.
- Without significant aquifer widths and depths, and without the presence of confining or semiconfining units, the ratio of ground water withdrawals to reduced instream flow is nearly one-to-one in the short term and equal to one-to-one under continuous steady pumping conditions.
- Therefore, the relationship between wellfield withdrawals and reduced ground water discharge/induced infiltration is relatively immediate and direct.
- Minimal overall benefit can be gained by relocating wells. The time lag between pumping and impact to the river is difficult to increase by moving wells further away because the aquifer is narrow.
- A very minor (0.31 cfs) benefit to proximal riffle habitats can be gained by shifting some of the ground water withdrawals downstream, but the net effect will be the same at the downstream end of the study area over the long term.
- This low benefit to streamflow suggests that an investment in moving or replacing infrastructure to reduce the effect on instream flow may not be as cost effective as additional water conservation measures on and off campus.

9.0 RECOMMENDATIONS

Recommendations are grouped into two categories. The first set of recommendations (demand-based water conservation) will result in a linkage between instream flows recorded at the USGS gaging station at Merrow Road and the various drought response triggers in the University's draft Drought Response Plan, with the goal of preventing onset of the critical, extreme, and rare flows. This approach is analogous to the staged response plan currently in place for the Fenton River, with a significant difference. Cutbacks in pumping at the Fenton River Wellfield are currently caused by the shutdown of some or all of the wells. This "forward" approach has worked well to protect instream flow because a low flow will result in reduced pumping, but it does not allow for any flexibility. For the Willimantic River Wellfield, the reverse shall occur: shutdown of wells shall be caused indirectly by the strict control of water usage on and off campus.

The second set of recommendations (supply management) involves methods of managing existing potable water supplies and creating new potable and nonpotable water supplies to lessen the reliance on the Willimantic River Wellfield. Examples include the use of reclaimed water, an interconnection with The Connecticut Water Company, development of new wells in Mansfield, and ensuring that additional increments of water are available in the river during periods of low instream flow.

9.1 Recommendations for Demand-Based Water Conservation

1. Incorporate the Willimantic River common, critical, rare, and extreme threshold discharges into the Drought Response Plan. The threshold discharges in Section 4.4 were determined based on actual streamflow data from the USGS gage in Coventry as corrected for withdrawals, sewage treatment plant discharges, and watershed ratio. As the gage proposed to be used to determine flow triggers is at Merrow Road upstream of the Willimantic River Wellfield, the flow within the influence of the wellfield (and downstream) will be less than that measured at Merrow Road by the

withdrawal rate (assuming a conservative 1:1 ratio). Therefore, in order to determine the discharge downstream of the Willimantic River wellfield, either the USGS-measured discharge at Merrow Road could be adjusted downwards by an amount equal to the pumping rate, or a similar value could be added to each trigger to account for the pumping at the wellfield. The precise methodology that the University will use should be determined in the proposed Willimantic River Wellfield – Fenton River Wellfield Management Plan (Section 9.2).

- Drought Management Begins: The "upper subregion" common threshold event (27 cfs) should serve as a cautionary condition where the water system operators would prepare to implement pumping reductions and/or the University would prepare to implement conservation measures and the Drought Response Plan.
- Stage IA: Should the discharge in the Willimantic River fall below 27 cfs for 19 days (the persistent duration of the common habitat threshold) OR if the discharge in the Willimantic River falls below 19 cfs (the "lower subregion" common habitat threshold), it would trigger *Stage 1A – Water Conservation Alert.*
- Stage IB: Should the discharge in the Willimantic River fall below 15 cfs (the critical habitat threshold), it would trigger *Stage IB – Water Supply / Drought Advisory*.
- Stage II: Should the discharge in the Willimantic River fall below 15 cfs for 13 days or more (the persistent duration of the critical habitat threshold) OR if the discharge in the Willimantic River falls below 12 cfs (the rare habitat threshold), it would trigger *Stage II – Water Supply / Drought Watch*.
- Stage III: Should the discharge in the Willimantic River fall below 12 cfs for 12 days or more (the persistent duration of the rare habitat threshold) OR if the discharge in the Willimantic River falls below 7.8 cfs, it would trigger *Stage III – Water Supply / Drought Warning*.
- Stage IV: *Stage IV – Water Supply / Drought Emergency* would trigger if the discharge in the Willimantic River falls below 7.8 cfs for seven or more days.

These triggers should be revisited upon the commissioning of the reclaimed water facility and addition of new water supply sources (refer to Section 9.2 below).

2. Incorporate mandatory conservation measures for both on- and off-campus users, including residential, municipal, and commercial customers; and Connecticut Department of Corrections facilities.

    - For municipal customers, work directly with the Town of Mansfield to conduct water audits and develop drought response plans for municipal facilities.
    - For residential customers, establish enforceable restrictions for outdoor water usage such as lawn watering bans. Work with the Town of Mansfield to adopt ordinances allowing for enforcement and then enforce restrictions. Ensure that residential retrofit programs have reached every residence, apartment, and condominium unit.
    - For commercial customers, work directly with each individual customer. Conduct water audits for those customers using more than 1,000 gpd. Enforceable restrictions for outdoor water usage shall apply to commercial customers as well. Work with the Town of Mansfield to enforce restrictions.
    - Work directly with the Department of Corrections to conduct water audits and develop drought response plans for correctional facilities.
    - When new potable water demands develop in the future at new off-campus developments such as the downtown Storrs project, the University should ensure that these developments incorporate the most up-to-date means of water conservation possible.
    - Encourage the Town of Mansfield to adopt water conservation measures for nonpublic water users during drought advisory periods as private well owners also affect the ground water and surface water aquifer systems.

9.2 Recommendations for Supply Management

1. Develop a combined Willimantic River Wellfield – Fenton River Wellfield Management Plan to manage the University's water supplies. The combined wellfield management plan must consider and be informed by typical instream flows, recession of instream flows, weather patterns, drought occurrence, and ground water levels. In particular, this document should include and discuss:

   - A determination for how the University will monitor USGS-measured upstream discharges at each wellfield and correlate pumping rates to the habitat threshold triggers determined in both this and the Fenton River Study.
   - A formal update to the Drought Response Plan should be included, including response timing and recovery guidelines.
   - Authorization for a limited but occasional use of the Fenton River Wellfield when it would otherwise be shut down. This will allow for brief decreases in pumping at the Willimantic River Wellfield, providing short periods of relief to the fish species in the Willimantic River. Note that the draft Drought Response Plan already includes provisions for using Fenton Well D.
   - Available supply versus system demands on a monthly basis throughout the calendar year. The projected available supply should consider potential various monthly supply restrictions based upon the operating season, and the projected system demands should consider any further demand reductions anticipated from the reclaimed water facility or additional conservation measures to be instituted.

2. The University's Water and Wastewater Master Plan recommends the "development of a treated effluent water supply for non-potable uses" (Section 6.0 of the plan, recommendation number 12). The University is currently in the design phase for a Reclaimed Water Facility (RWF). The RWF will produce treated effluent for nonpotable uses such as the central utility plant, thus reducing the amount of water needed from the Willimantic River Wellfield. It is noted that the central utility plant

may use an average of 0.5 mgd during a maximum-demand month; this is equal to 0.8 cfs. Peak demands at the central utility plant may be higher.

3. After the RWF is operational, the University should ensure in the short term that the increment of water that is no longer tied up for nonpotable usage (central utility plant and athletic fields) will be allocated to instream needs as well as new potable demands that may arise in the future in an equitable manner (one-to-one ratio). In other words, if 0.8 cfs from the Willimantic River Wellfield is freed by the reuse of treated effluent, 0.4 cfs of this should be reserved for instream flow. In the long term, this recommendation may no longer apply because new sources of supply may be available (as described below) or the river basin may be managed to provide additional instream flows upstream from the wellfield (recommendation number seven below).

4. The University's Water and Wastewater Master Plan recommends that the University "pursue an additional ground water source in the Willimantic River basin to meet future off-campus water demands…." (Section 6.0 of the plan, recommendation number 13). This recommendation was also offered in the Mansfield Water Supply Plan published in 2002. For both plans, future locations of ground water supplies were envisioned downstream of the Willimantic River Wellfield in a location where instream flows would be higher than they are at the existing wellfield, and/or fish habitats would be less sensitive to flow reductions. If a new supply were to be developed, the most logical use relative to protection of instream flows in the Willimantic River would be to utilize the new source(s) to reduce stress on the Willimantic River habitat near the Willimantic River Wellfield. A more comprehensive look at balancing impacts between any new sources, interconnections, and the Fenton River should be taken if a new supply is proposed.

5. Although an interconnection with (and main extension from) the Connecticut Water Company's Northern Region/Western System is envisioned in the 2006 CWC water

supply plan, the interconnection has not progressed further than the planning stage. If the interconnection were to occur, the most logical use relative to protection of instream flows in the Willimantic River would be to utilize the pipeline for supply when the critical threshold was reached. Interconnection with (and main extension from) the Windham Water Works is another potential source of supply.

6. Consider provision of short-term or pulsed releases from the Staffordville Reservoir, Crystal Lake, and/or State Line Pond. This will require cooperation with the dam owners and the parties that control the impoundments and the dam outlet works. Refer to Appendix G for a discussion of these impoundments, potential releases available, and the implications of making releases.

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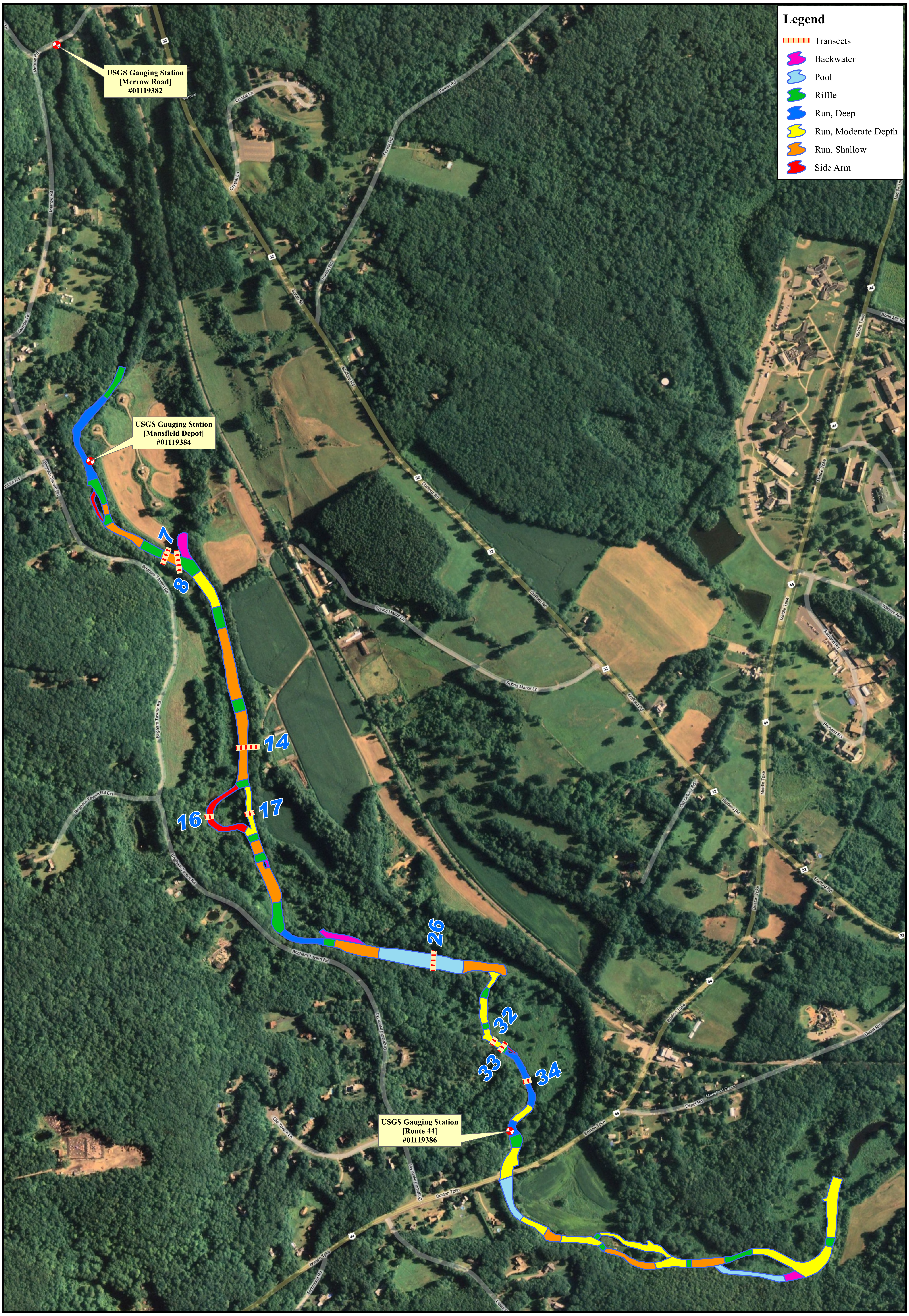
1958-09-jn1510-rpt.doc

APPENDED FIGURE



REACH LOCATION MAP

INSTREAM FLOW STUDY OF THE WILLIMANTIC RIVER
UNIVERSITY OF CONNECTICUT

COVENTRY / MANSFIELD, CONNECTICUT

DESIGNED	SJB
DRAWN	SJB
CHECKED	DM
SCALE	1" = 300'
DATE	FEBRUARY 2010
PROJECT NO	1958-09
MXD NAME	H:APPFIG1.MXD
SHEET NO.	APPENDED FIGURE 1

REVISIONS

5/3/2010

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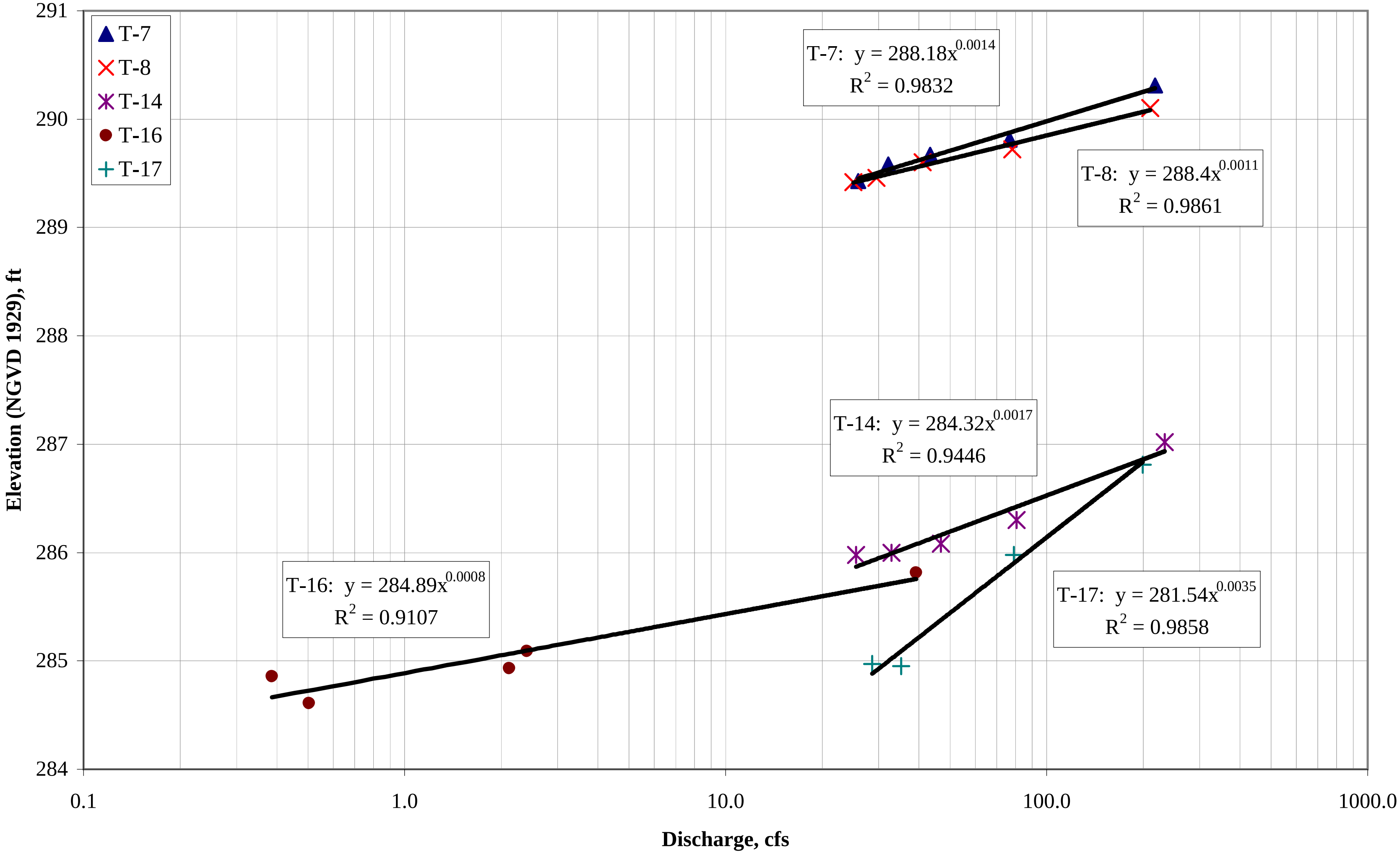
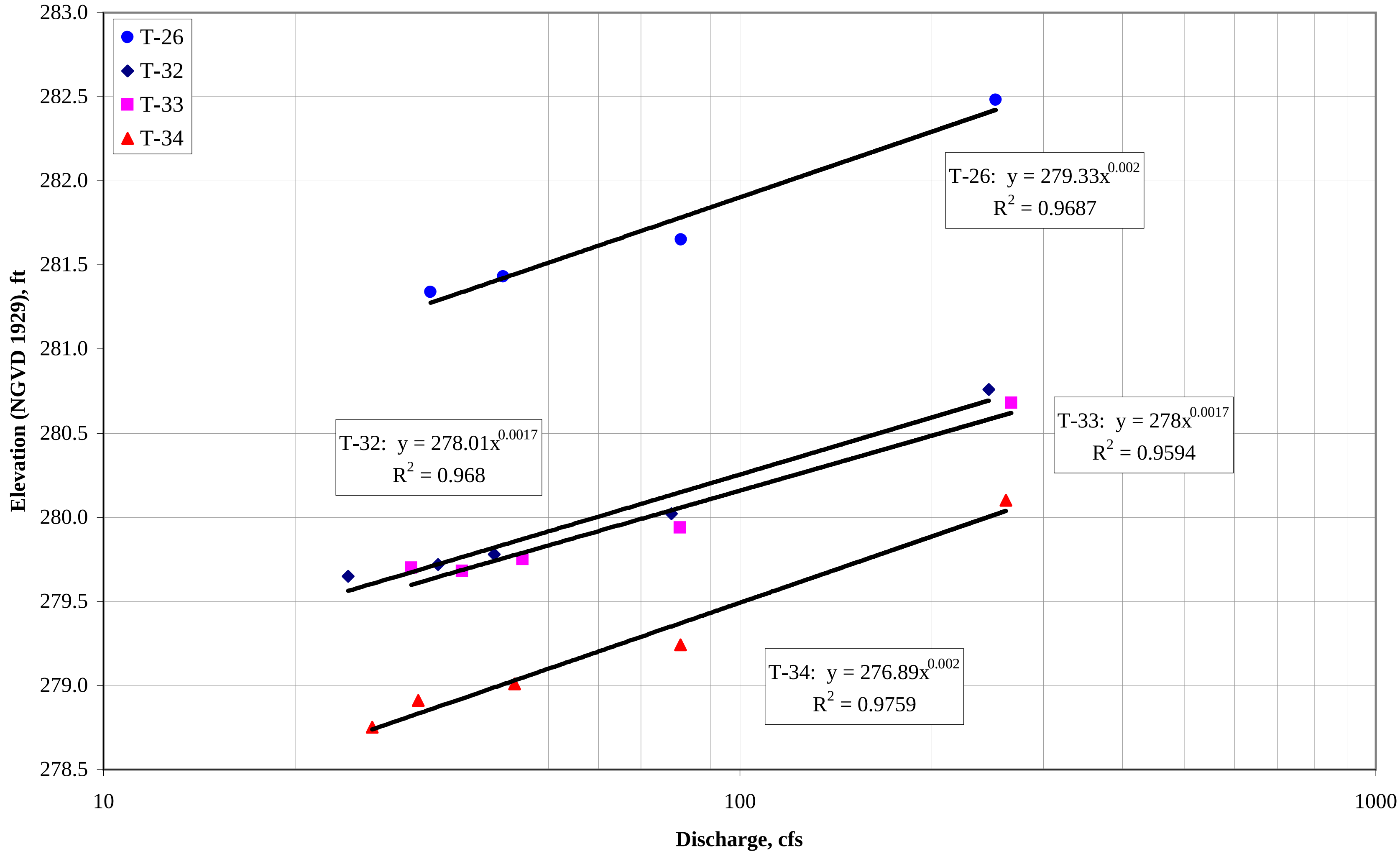
N E S W

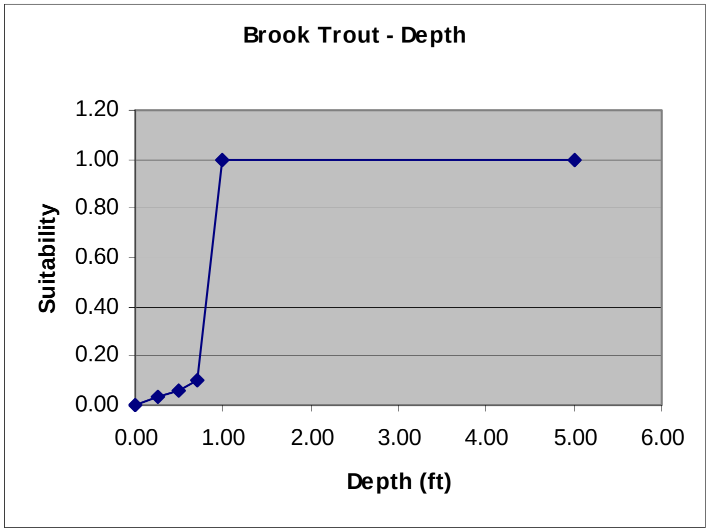
APPENDIX A
PHABSIM INPUT AND UCUT CURVES

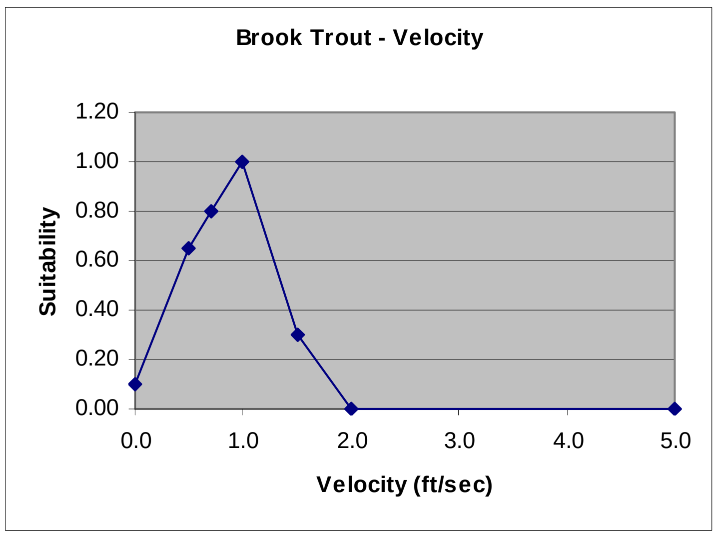
Transect Rating Curves - 2008 & 2009 Instream Flow Study
Willimantic River Wellfield
University of Connecticut

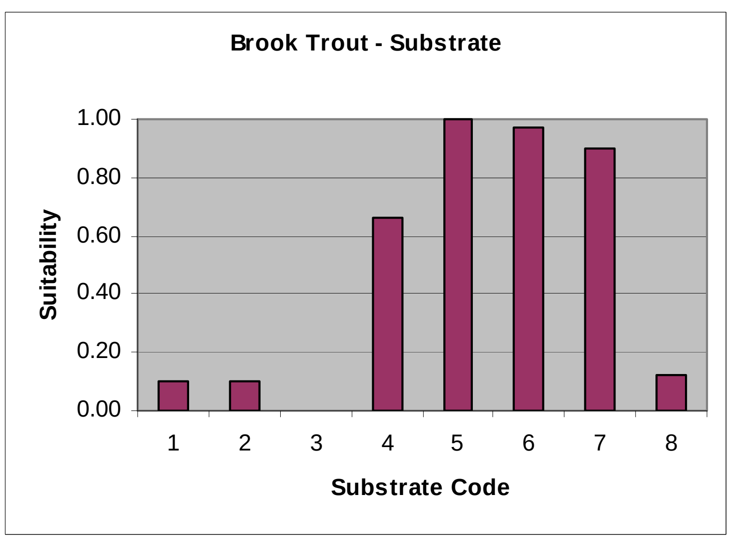


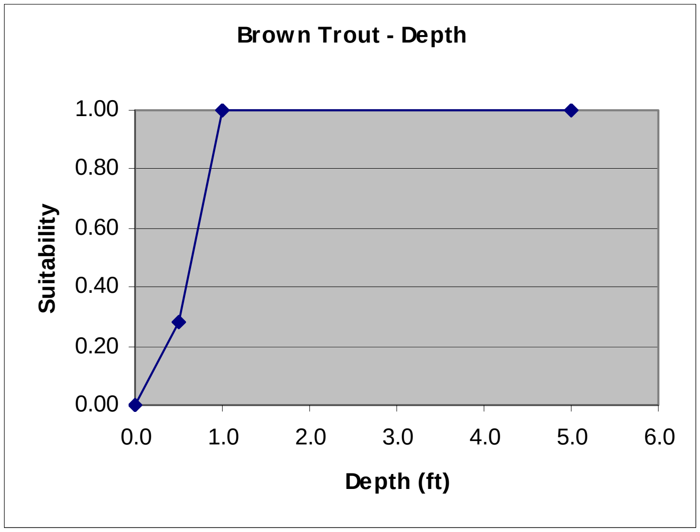
Transect Rating Curves - 2008 & 2009 Instream Flow Study
Willimantic River Wellfield
University of Connecticut

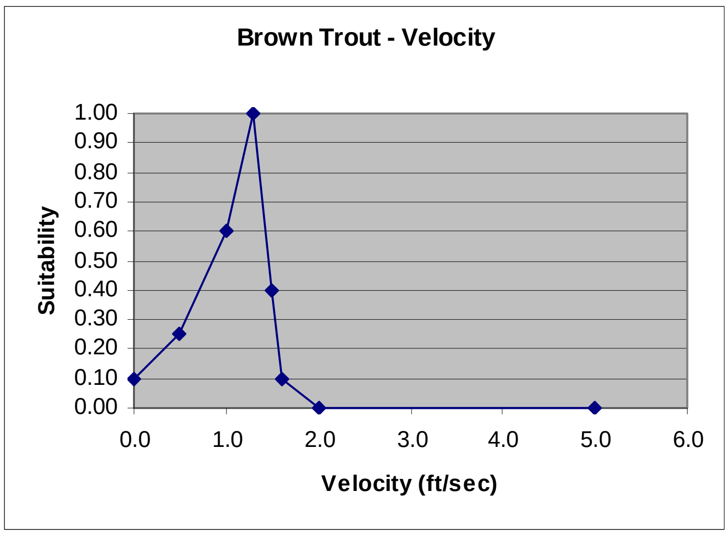


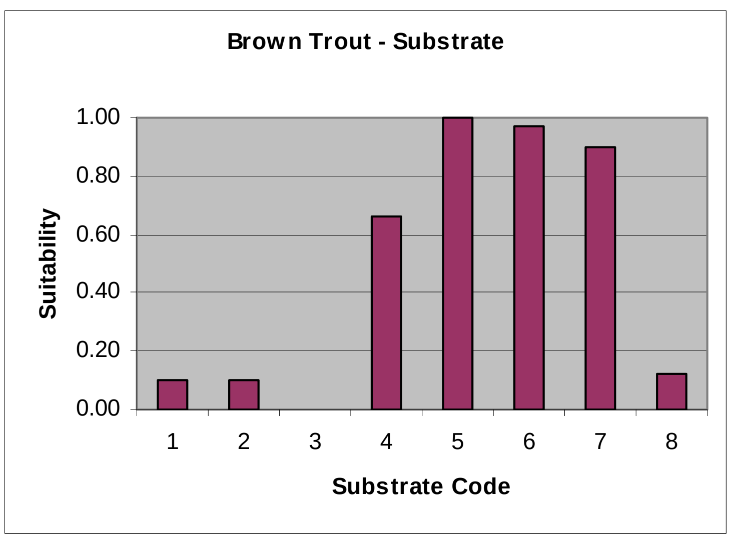

Brook Trout - Depth
Suitability
1.20
1.00
0.80
0.60
0.40
0.20
0.00
0.00
1.00
2.00
3.00
4.00
5.00
6.00
Depth (ft)

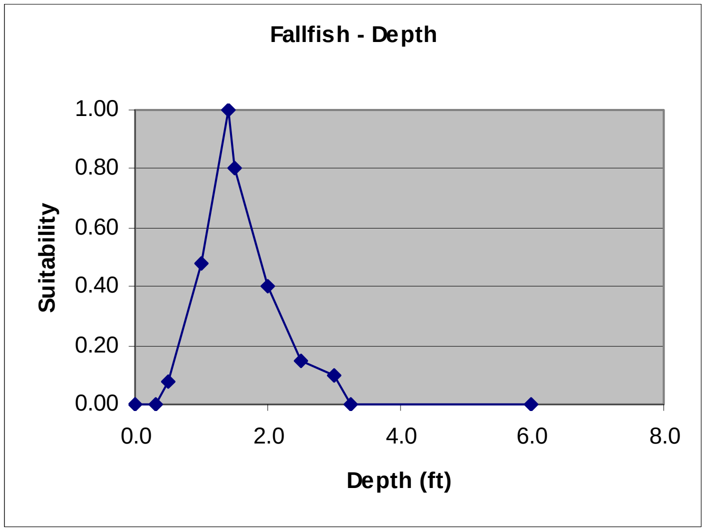

Brook Trout - Velocity
Suitability
1.20
1.00
0.80
0.60
0.40
0.20
0.00
0.0
1.0
2.0
3.0
4.0
5.0
Velocity (ft/sec)

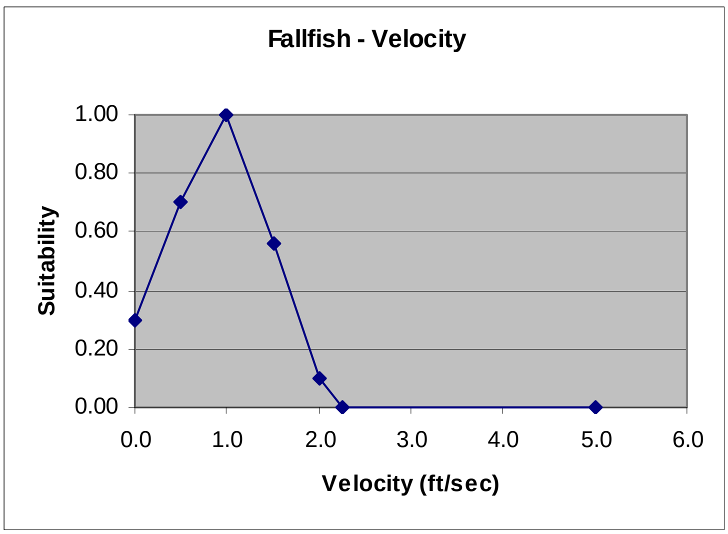

Brook Trout - Substrate
Suitability
1.00
0.80
0.60
0.40
0.20
0.00
1
2
3
4
5
6
7
8
Substrate Code

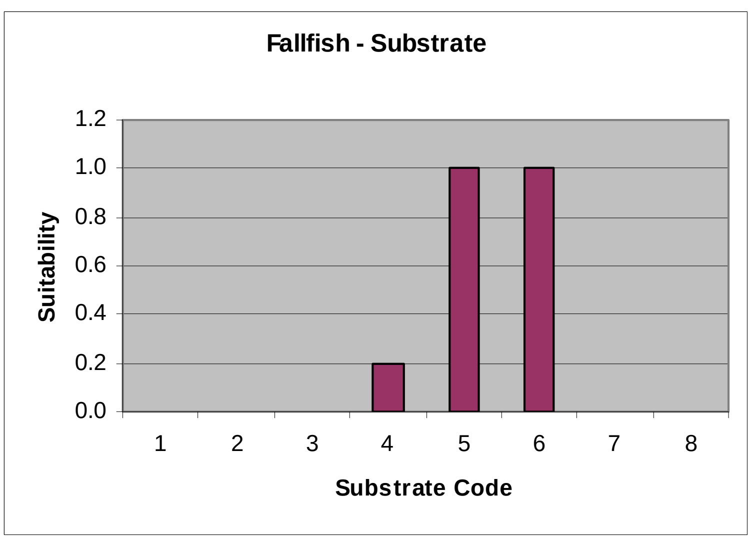

Brown Trout - Depth
Suitability
1.00
0.80
0.60
0.40
0.20
0.00
0.0
1.0
2.0
3.0
4.0
5.0
6.0
Depth (ft)

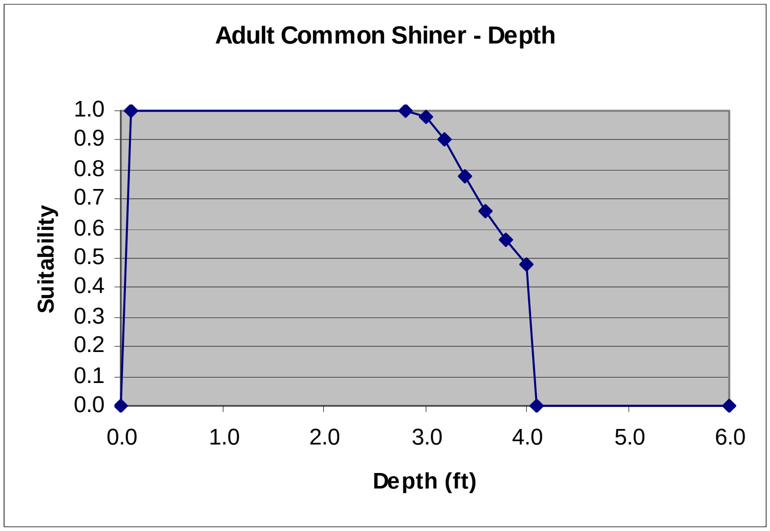

Brown Trout - Velocity
Suitability
1.00
0.90
0.80
0.70
0.60
0.50
0.40
0.30
0.20
0.10
0.00
0.0
1.0
2.0
3.0
4.0
5.0
6.0
Velocity (ft/sec)

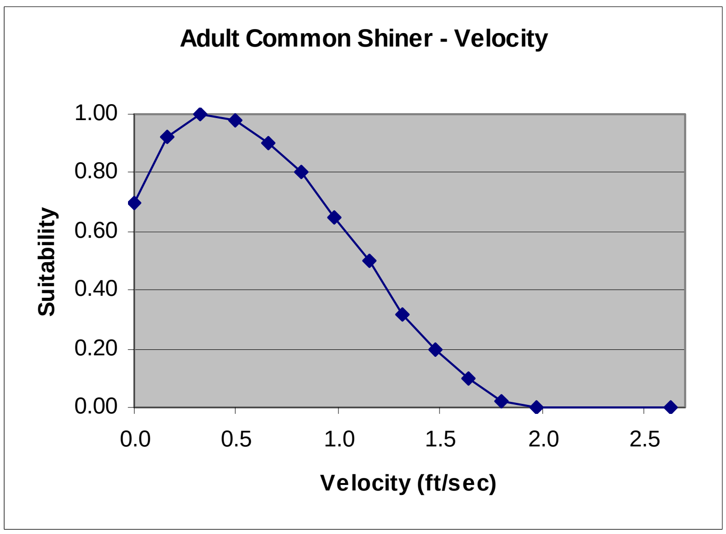

Brown Trout - Substrate
Suitability
1.00
0.80
0.60
0.40
0.20
0.00
1
2
3
4
5
6
7
8
Substrate Code

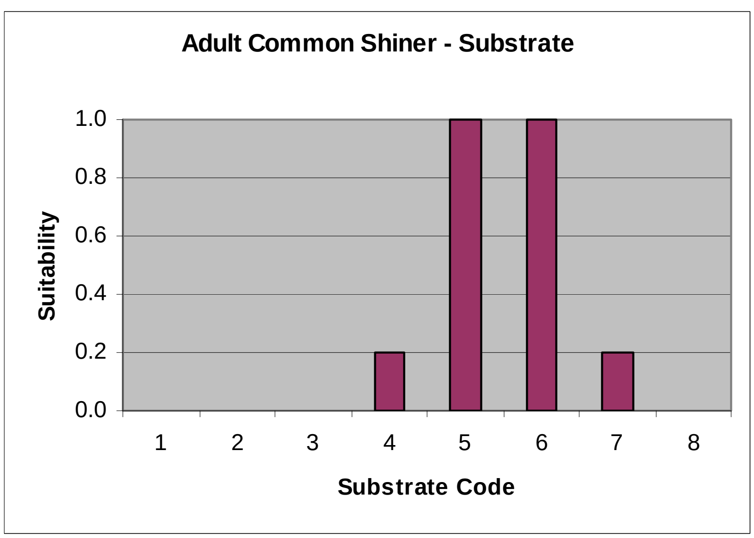

Fallfish - Depth
Suitability
1.00
0.80
0.60
0.40
0.20
0.00
0.0
2.0
4.0
6.0
8.0
Depth (ft)

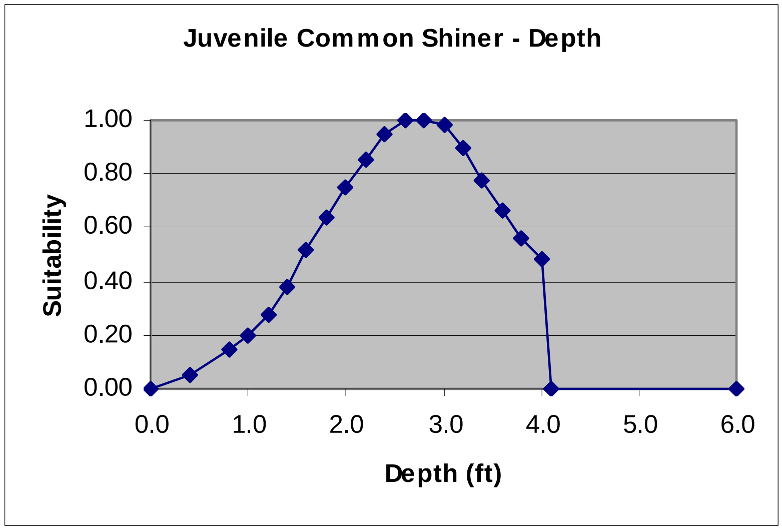

Fallfish - Velocity
Suitability
1.00
0.80
0.60
0.40
0.20
0.00
0.0
1.0
2.0
3.0
4.0
5.0
6.0
Velocity (ft/sec)

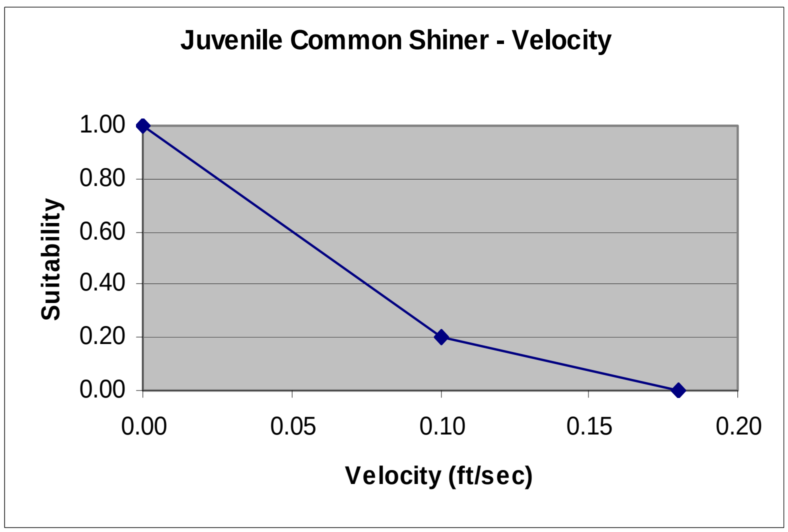

Fallfish - Substrate
Suitability
1.2
1.0
0.8
0.6
0.4
0.2
0.0
1
2
3
4
5
6
7
8
Substrate Code

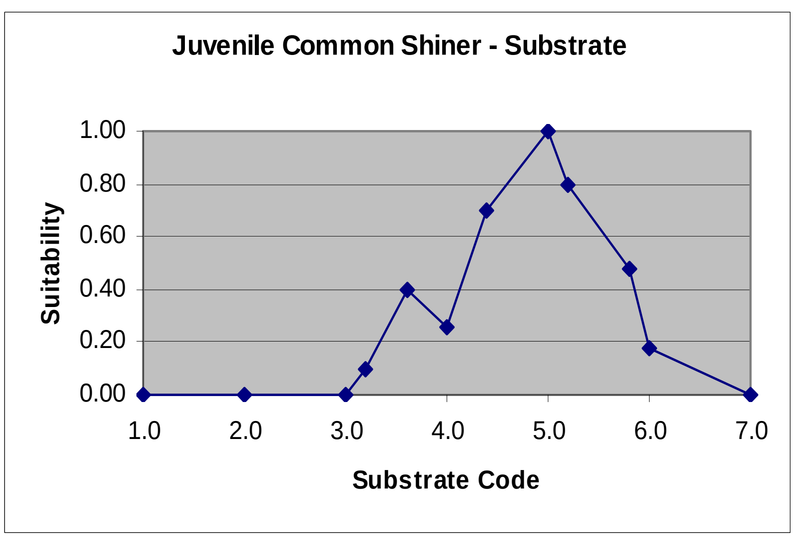

Adult Common Shiner - Depth
Suitability
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.0
0.0
1.0
2.0
3.0
4.0
5.0
6.0
Depth (ft)


Adult Common Shiner - Velocity
Suitability
1.00
0.80
0.60
0.40
0.20
0.00
0.0
0.5
1.0
1.5
2.0
2.5
Velocity (ft/sec)

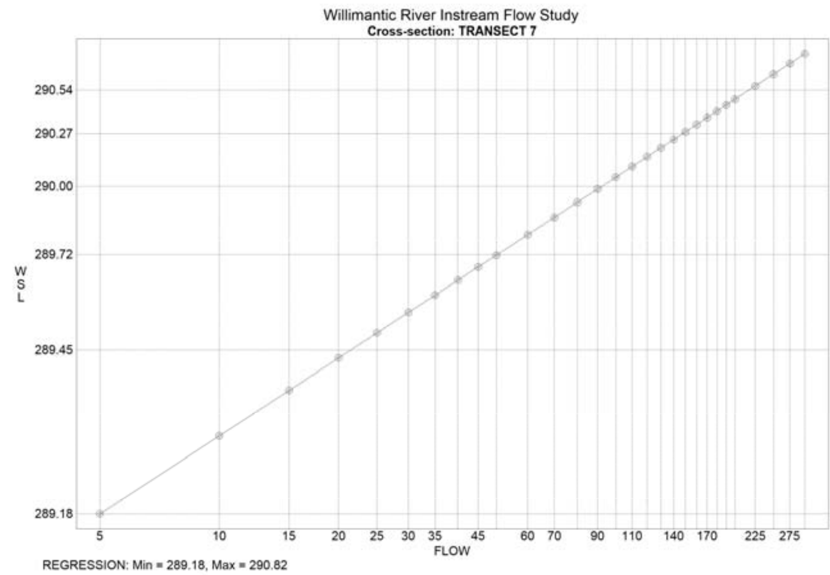

Adult Common Shiner - Substrate
Suitability
1.0
0.8
0.6
0.4
0.2
0.0
1
2
3
4
5
6
7
8
Substrate Code

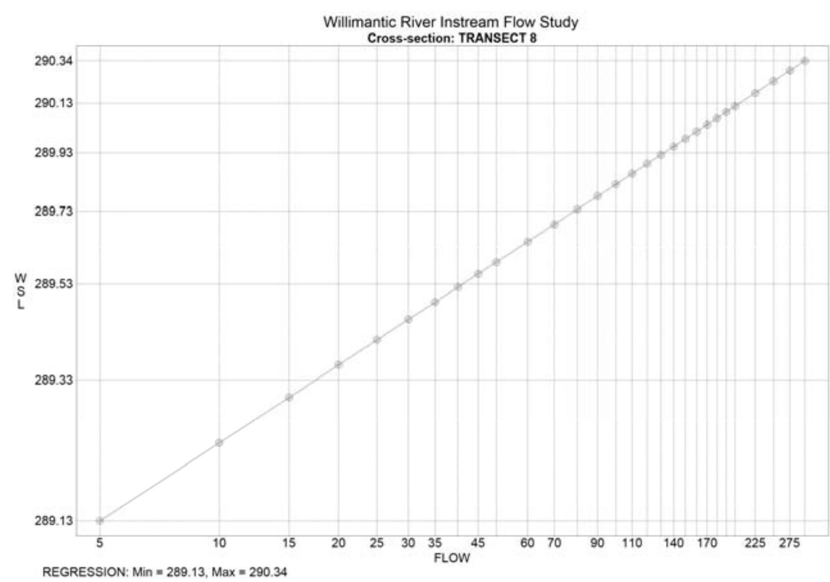

Juvenile Common Shiner - Depth
Suitability
1.00
0.80
0.60
0.40
0.20
0.00
0.0
1.0
2.0
3.0
4.0
5.0
6.0
Depth (ft)

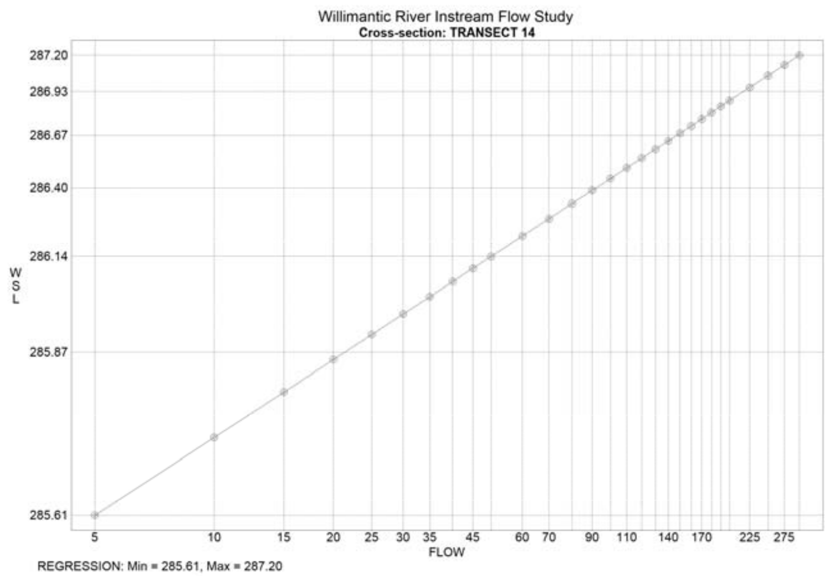

Juvenile Common Shiner - Velocity
Suitability
1.00
0.80
0.60
0.40
0.20
0.00
0.00
0.05
0.10
0.15
0.20
Velocity (ft/sec)

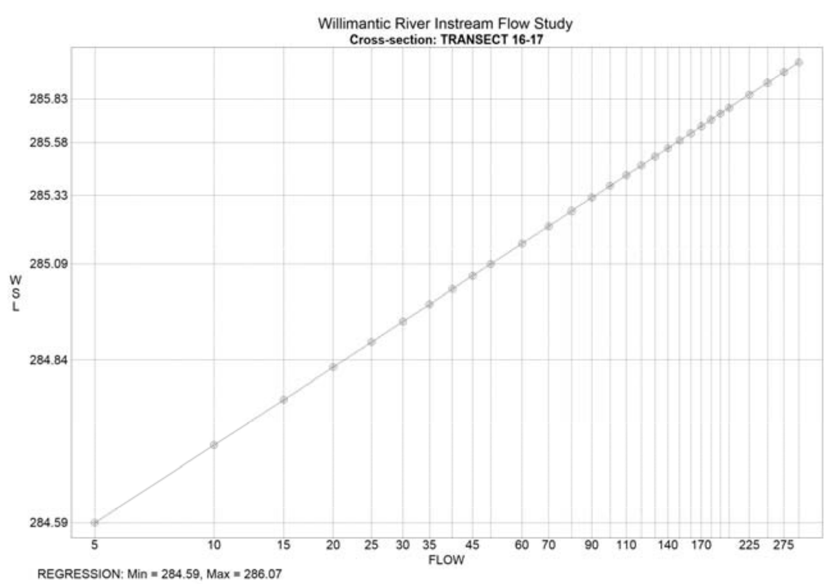

Juvenile Common Shiner - Substrate
Suitability
1.00
0.80
0.60
0.40
0.20
0.00
1.0
2.0
3.0
4.0
5.0
6.0
7.0
Substrate Code

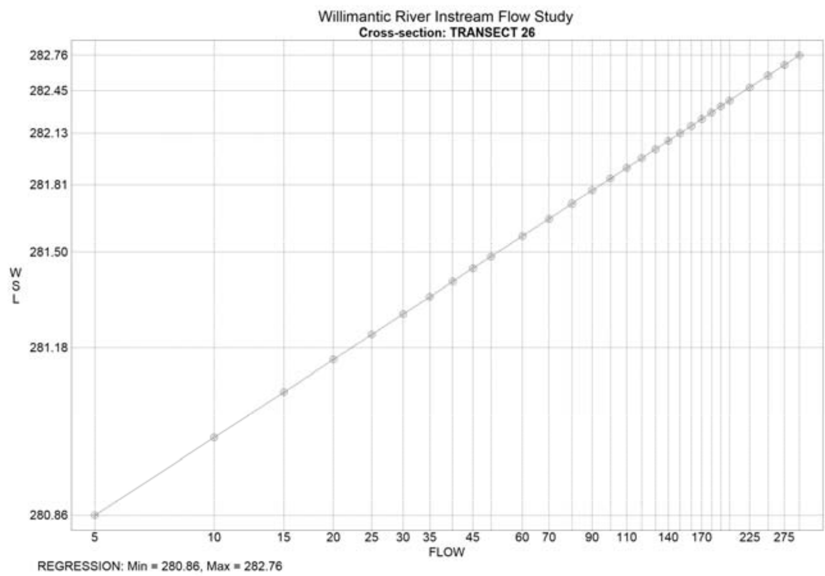
APPENDIX A
TRANSECT RATING CURVES

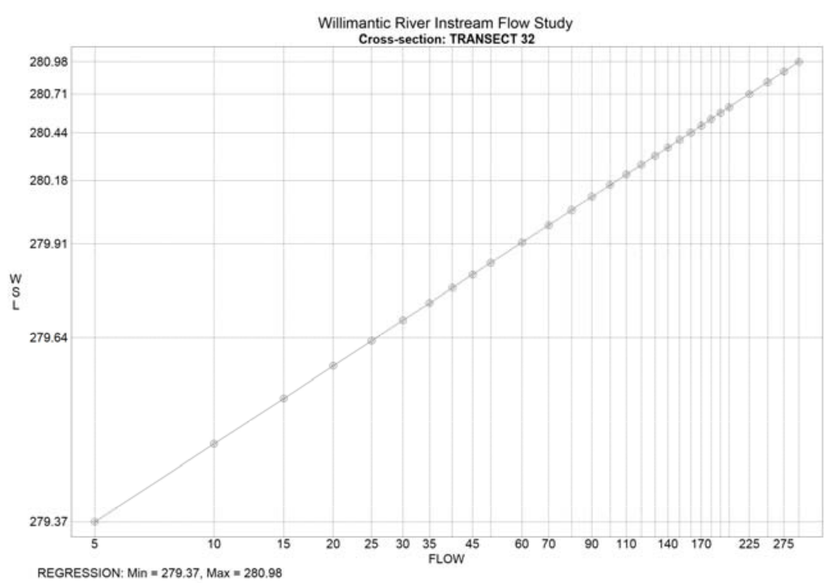


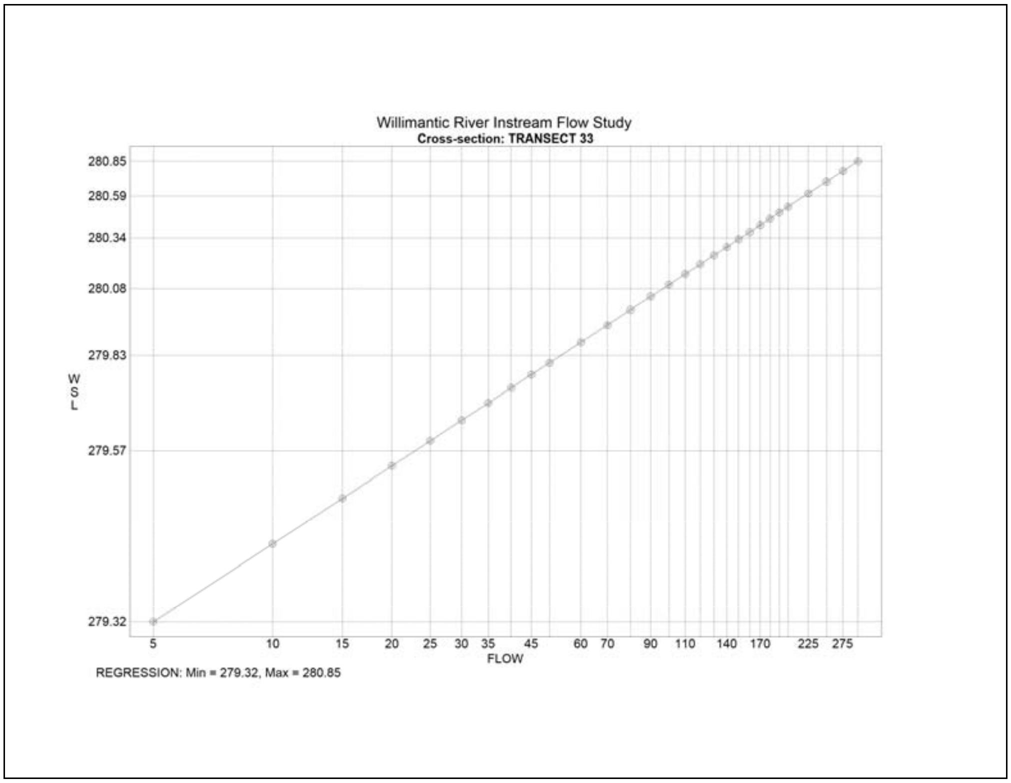



Willimantic River Instream Flow Study
Cross-section: TRANSECT 14
287.20
286.93
286.67
286.40
286.14
285.87
285.61
W
S
L
5
10
15
20
25
30
35
45
60
70
90
110
140
170
225
275
FLOW
REGRESSION: Min = 285.61, Max = 287.20

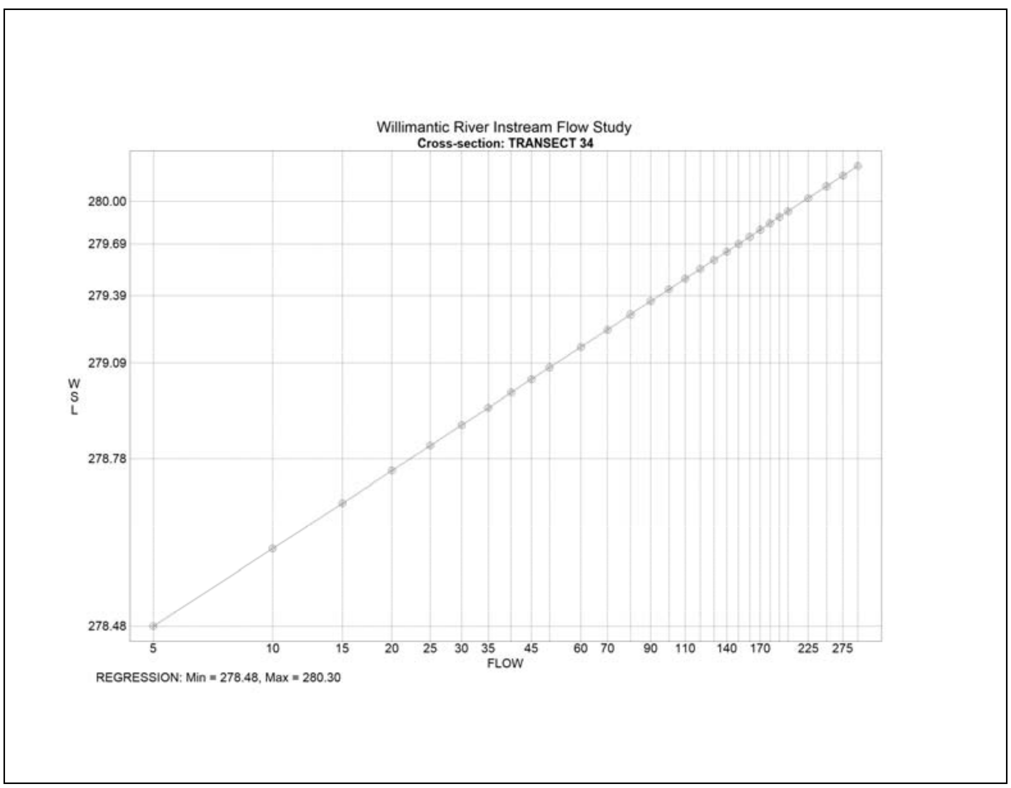

Willimantic River Instream Flow Study
Cross-section: TRANSECT 16-17
285.83
285.58
285.33
285.09
284.84
284.59
W
S
L
5
10
15
20
25
30
35
45
60
70
90
110
140
170
225
275
FLOW
REGRESSION: Min = 284.59, Max = 286.07


Willimantic River Instream Flow Study
Cross-section: TRANSECT 26
282.76
282.45
282.13
281.81
281.50
W
S
L
281.18
280.86
5
10
15
20
25
30
35
45
60
70
90
110
140
170
225
275
FLOW
REGRESSION: Min = 280.86, Max = 282.76


Willimantic River Instream Flow Study
Cross-section: TRANSECT 32
280.98
280.71
280.44
280.18
279.91
W
S
L
279.64
279.37
5
10
15
20
25
30
35
45
60
70
90
110
140
170
225
275
FLOW
REGRESSION: Min = 279.37, Max = 280.98

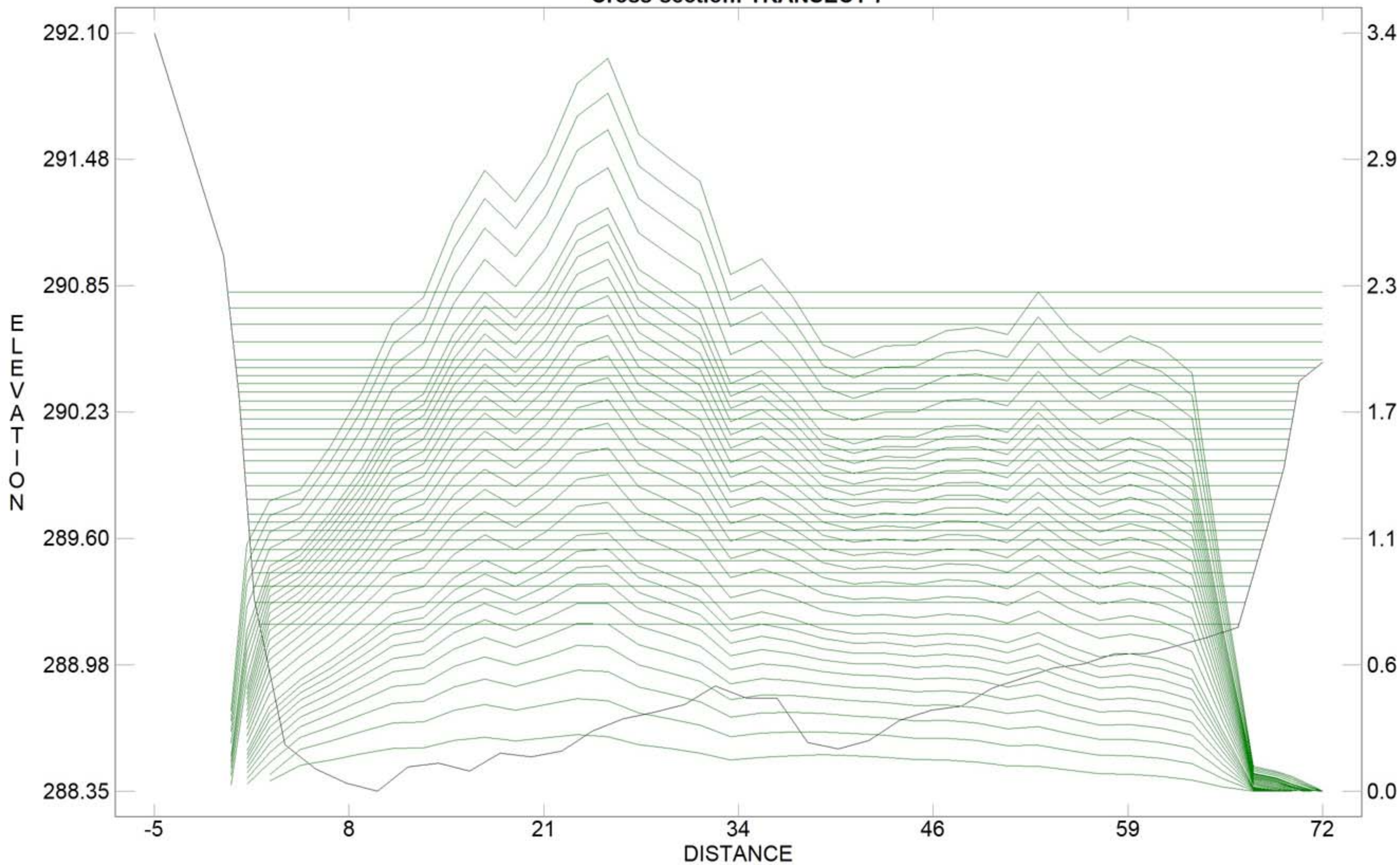


Willimantic River Instream Flow Study
Cross-section: TRANSECT 33
280.85
280.59
280.34
280.08
279.83
279.57
279.32
W S L
5
10
15
20
25
30
35
45
60
70
90
110
140
170
225
275
FLOW
REGRESSION: Min = 279.32, Max = 280.85



Willimantic River Instream Flow Study
Cross-section: TRANSECT 34
280.00
279.69
279.39
279.09
278.78
278.48
W S L
5
10
15
20
25
30
35
45
60
70
90
110
140
170
225
275
FLOW
REGRESSION: Min = 278.48, Max = 280.30

Willimantic River Instream Flow Study
Cross-section: TRANSECT 7



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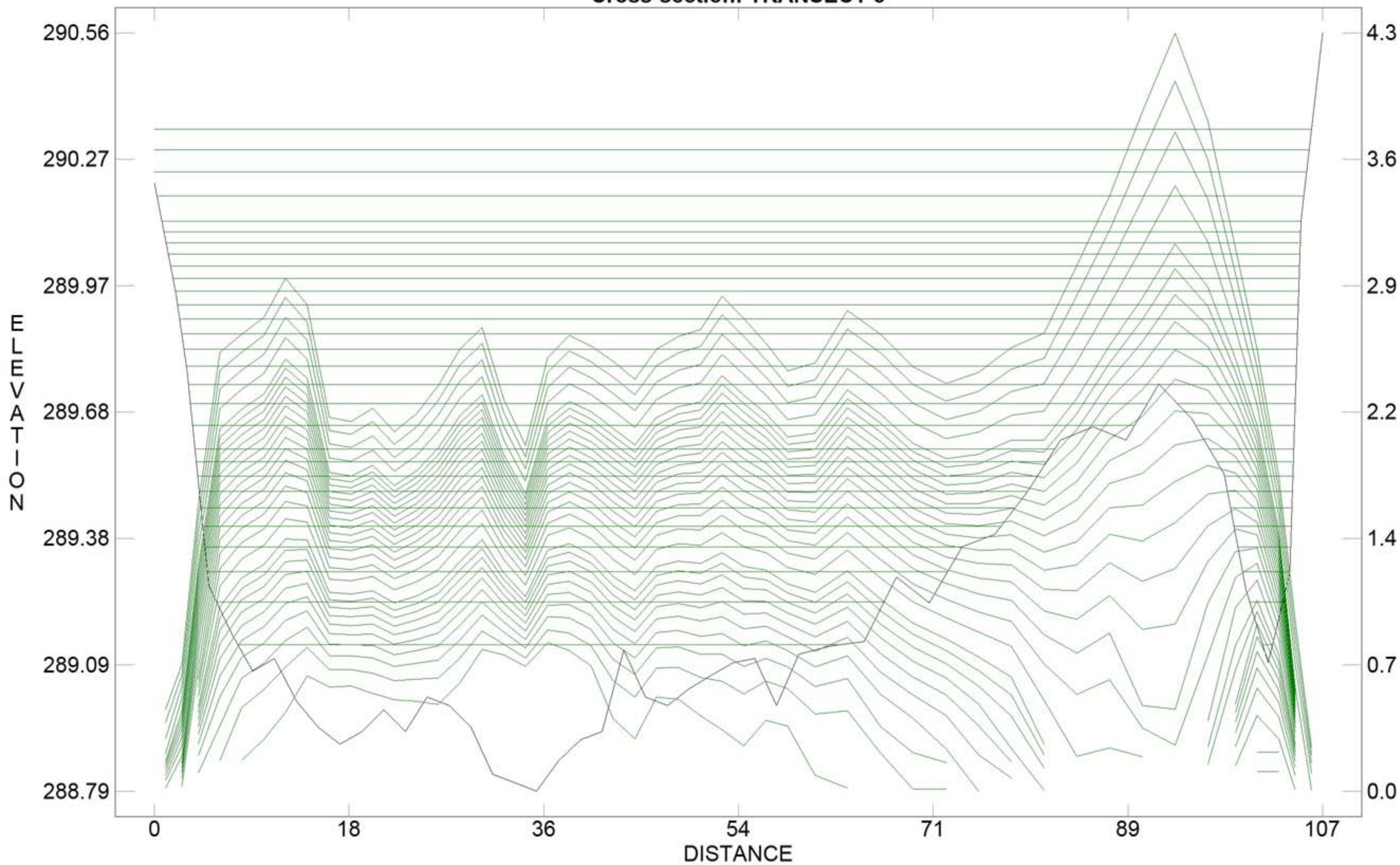
Willimantic River Instream Flow Study

Cross-section: TRANSECT 8



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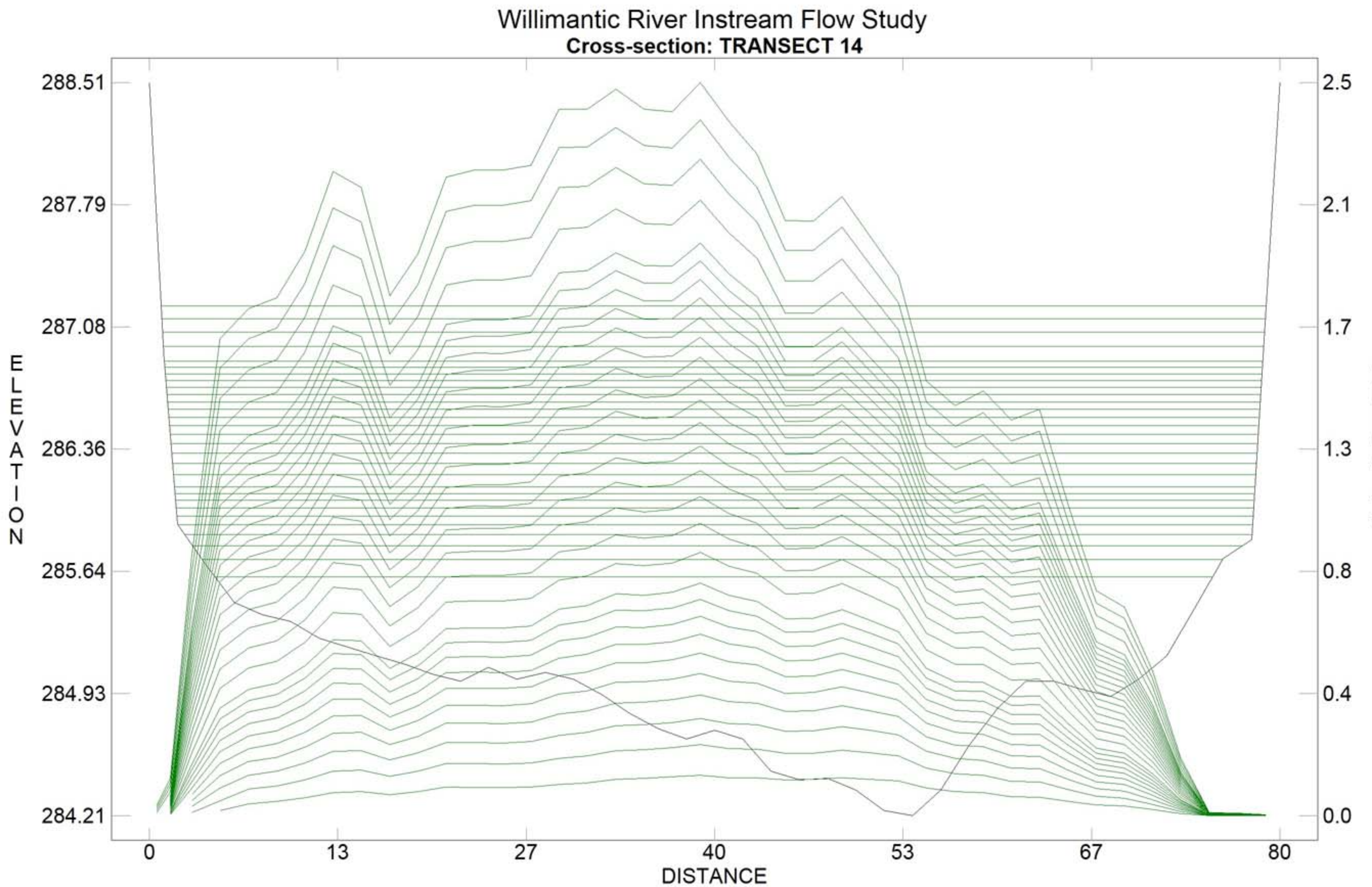
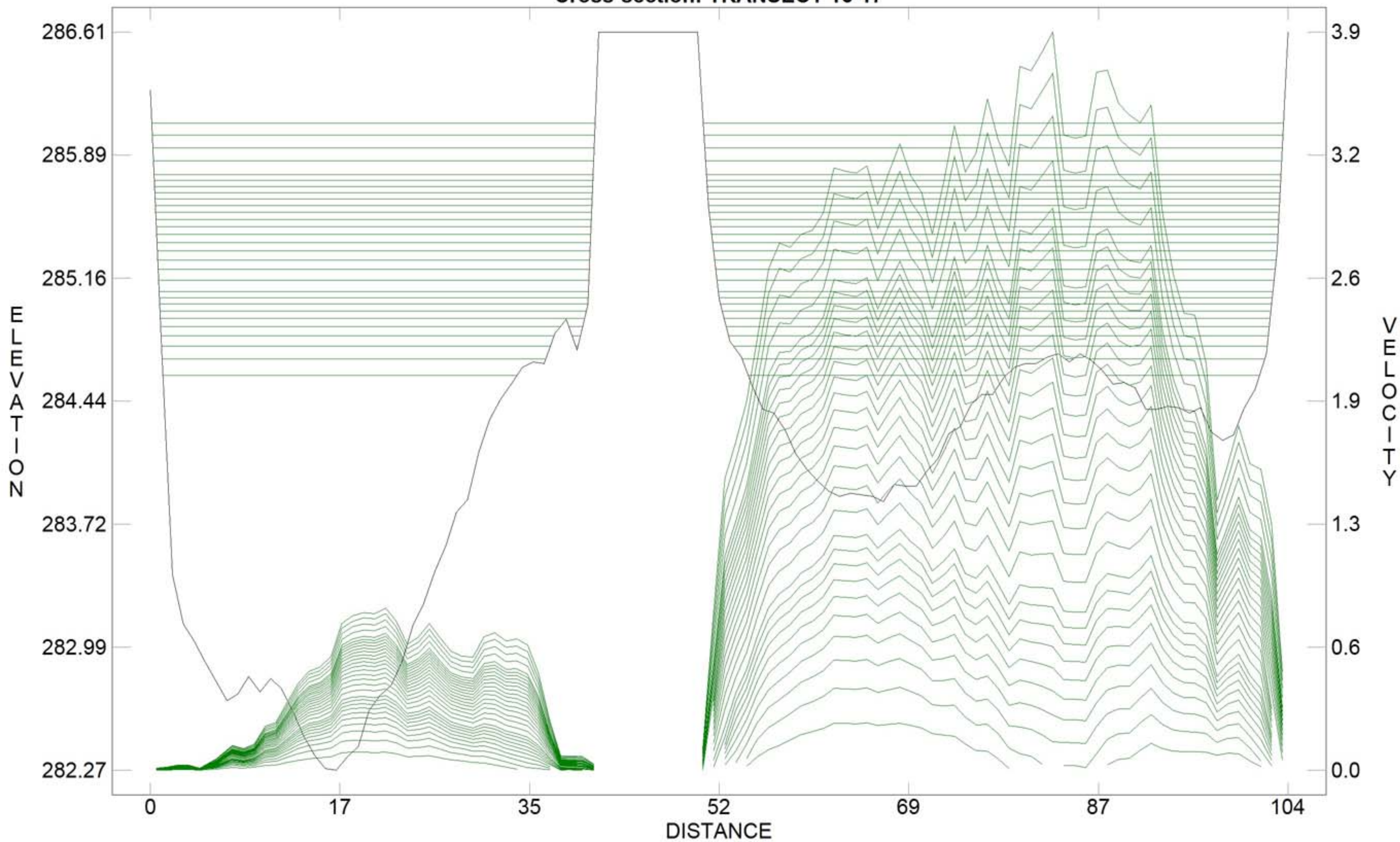
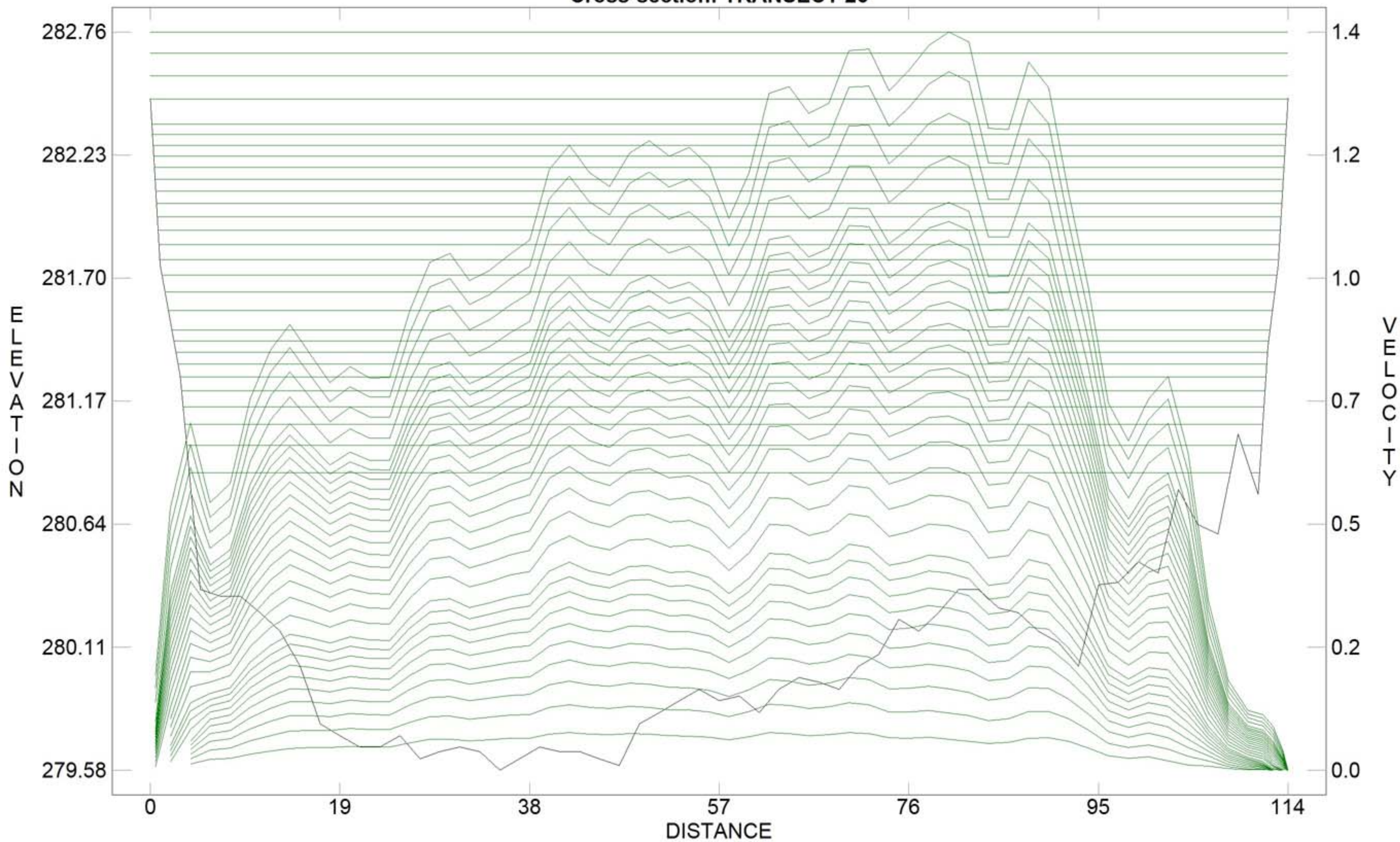


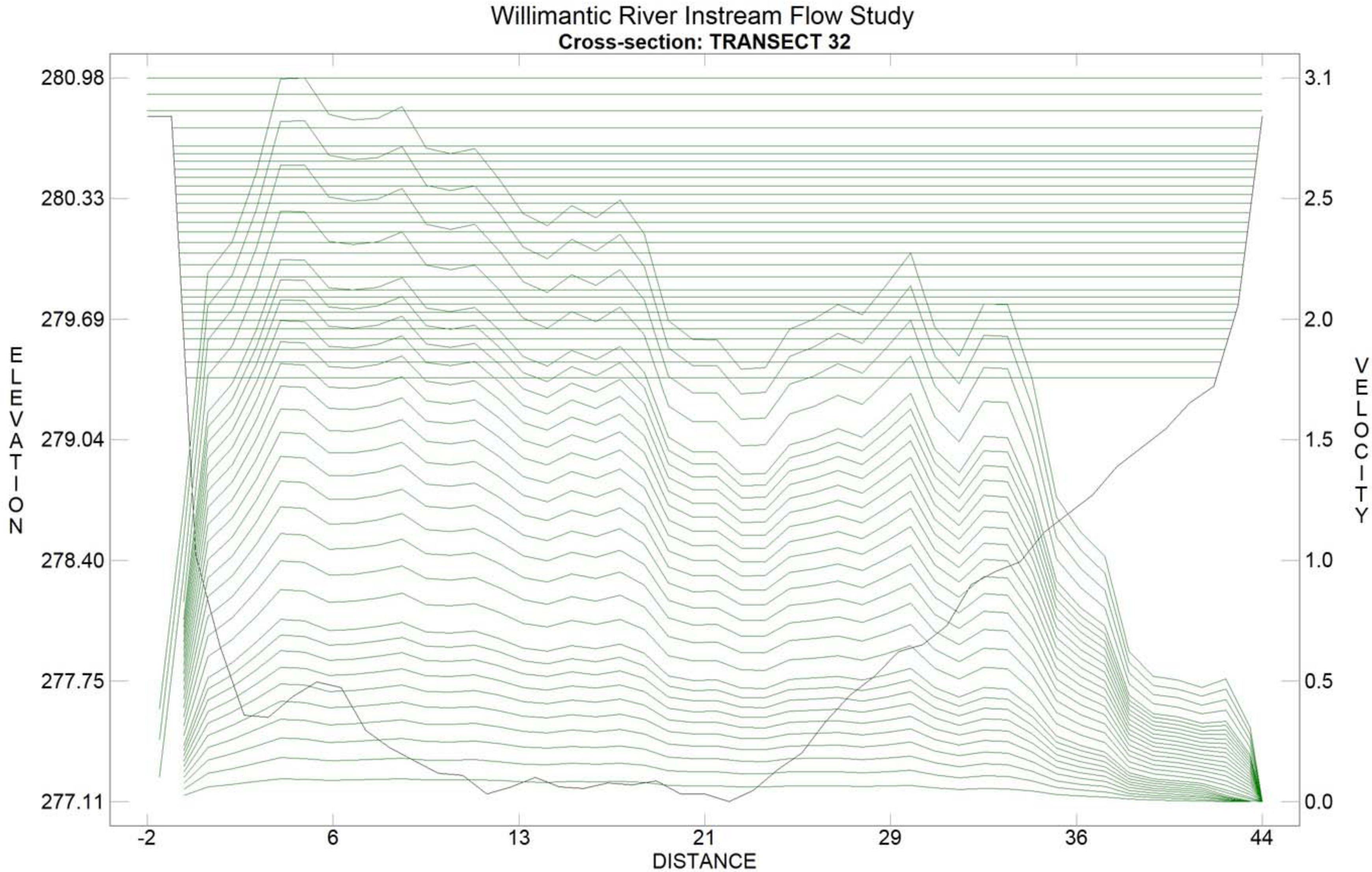
MILONE & MACBROOM®

Willimantic River Instream Flow Study
Cross-section: TRANSECT 16-17

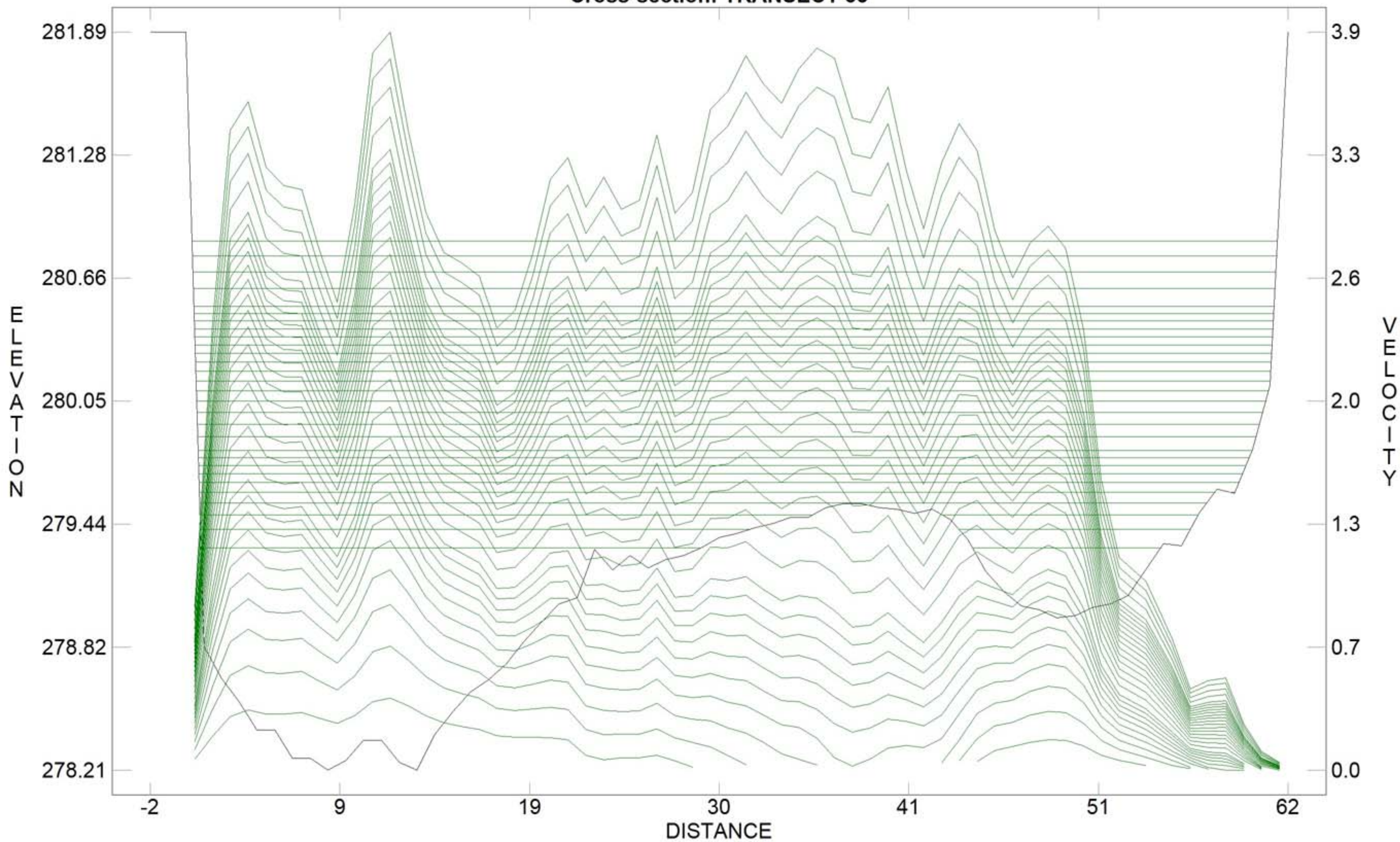


Willimantic River Instream Flow Study
Cross-section: TRANSECT 26




Willimantic River Instream Flow Study
Cross-section: TRANSECT 32
ELEVATION
280.98
280.33
279.69
279.04
278.40
277.75
277.11
VELOCITY
3.1
2.5
2.0
1.5
1.0
0.5
0.0
-2
6
13
21
29
36
44
DISTANCE

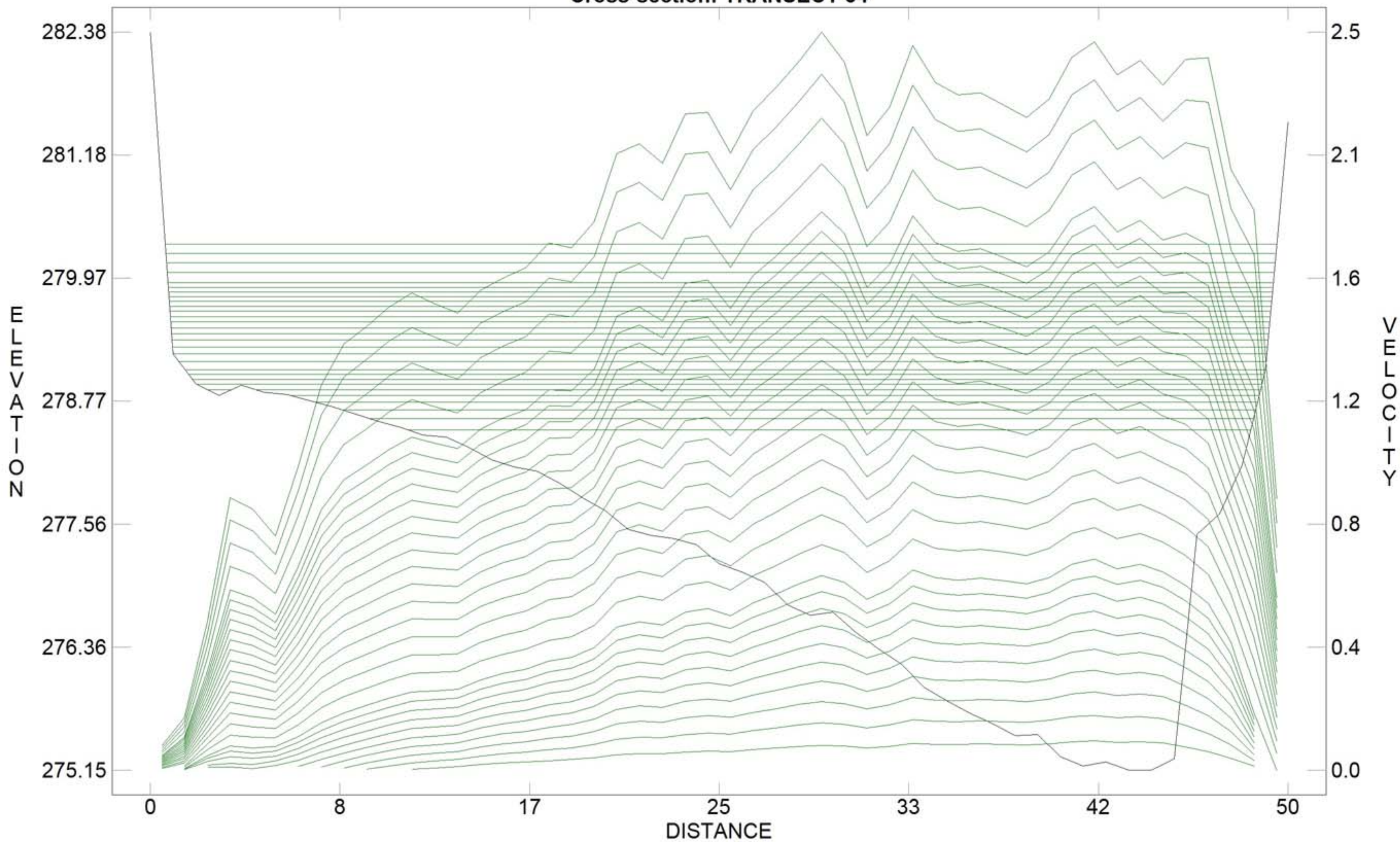
Willimantic River Instream Flow Study
Cross-section: TRANSECT 33



MILONE & MACBROOM®

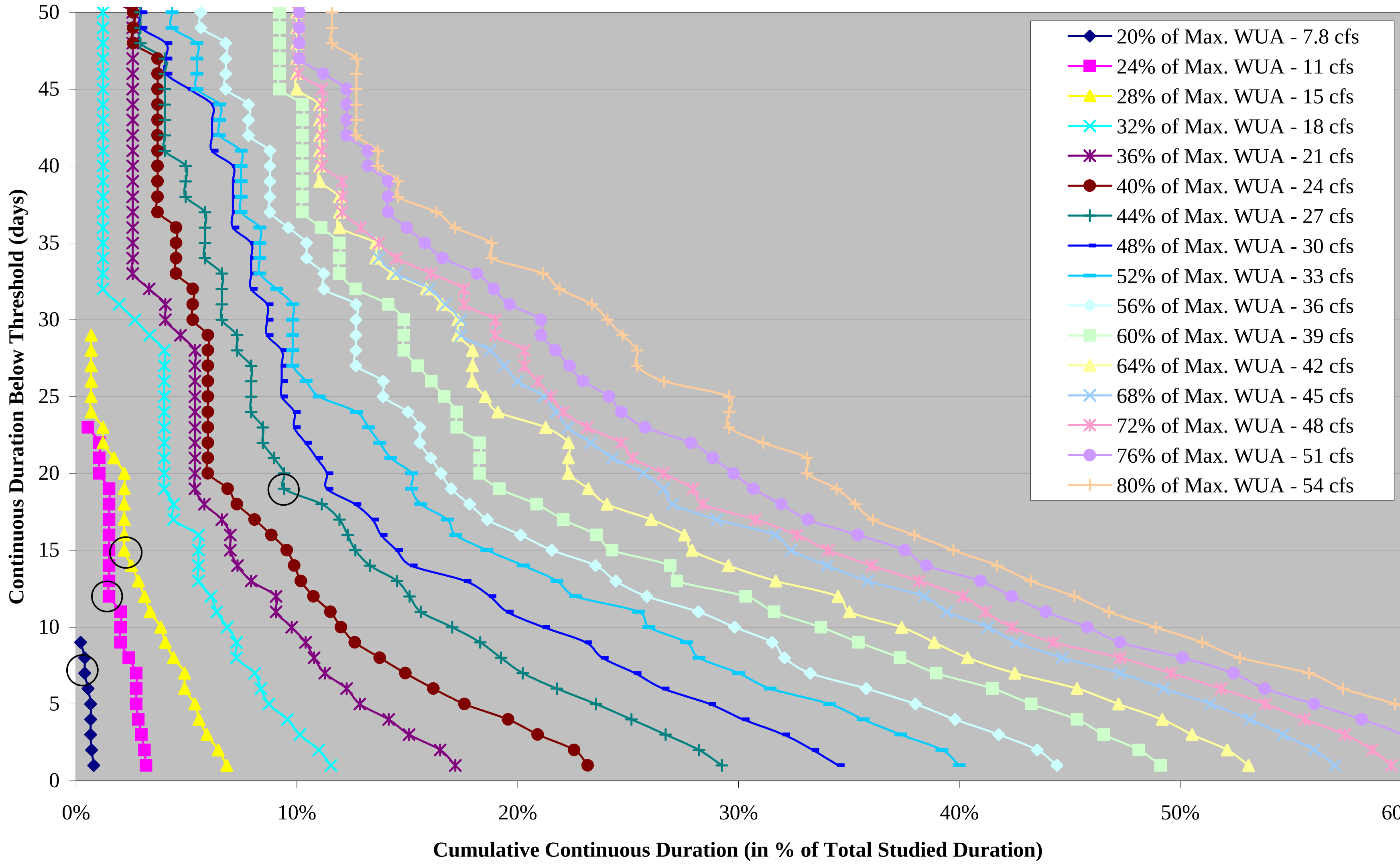
Willimantic River Instream Flow Study

Cross-section: TRANSECT 34



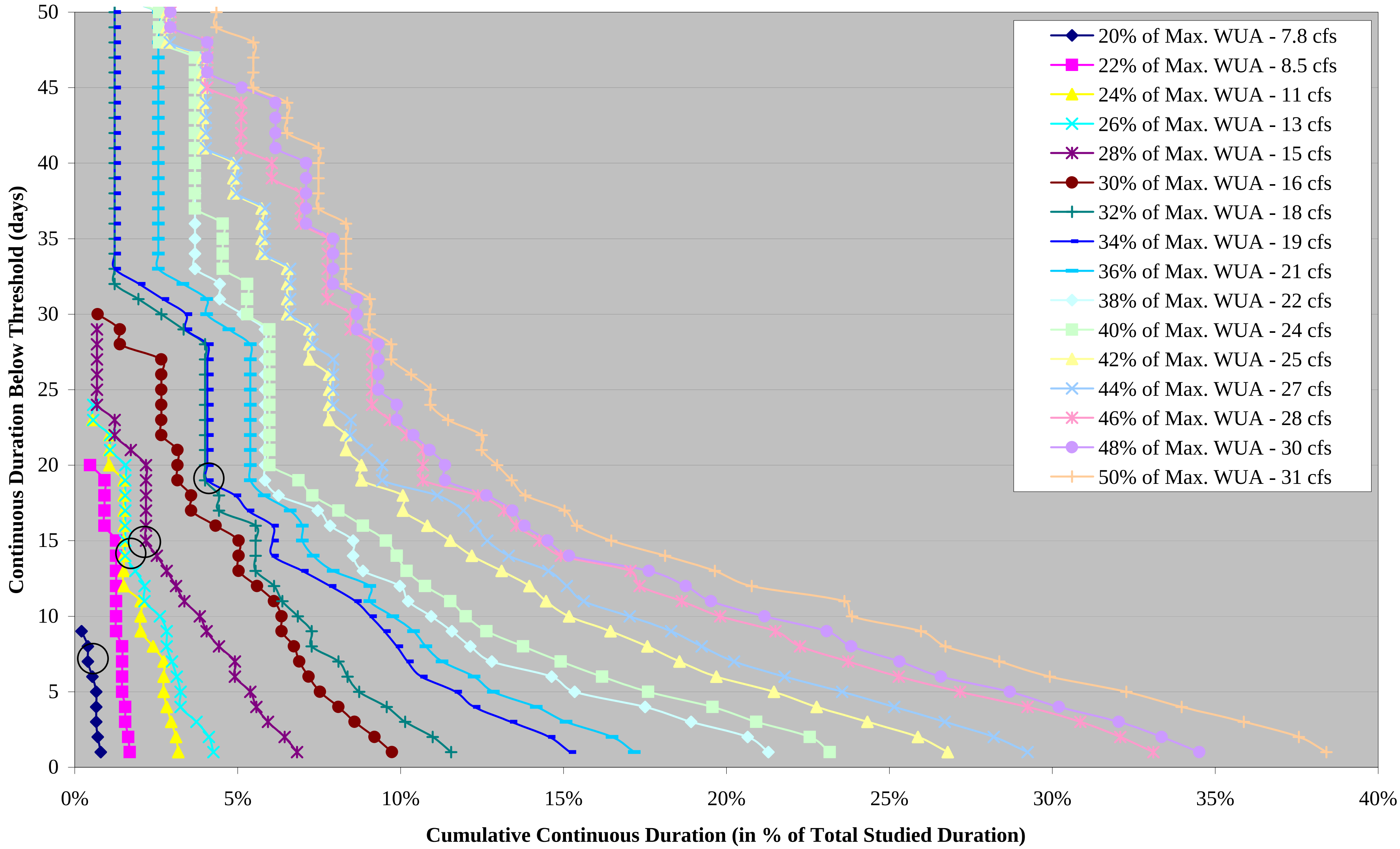
Milone & MacBroom®

Selected Habitat UCUT Curves for Brown Trout - Press Disturbances Only
July 8 - September 30, 1959 - 2008

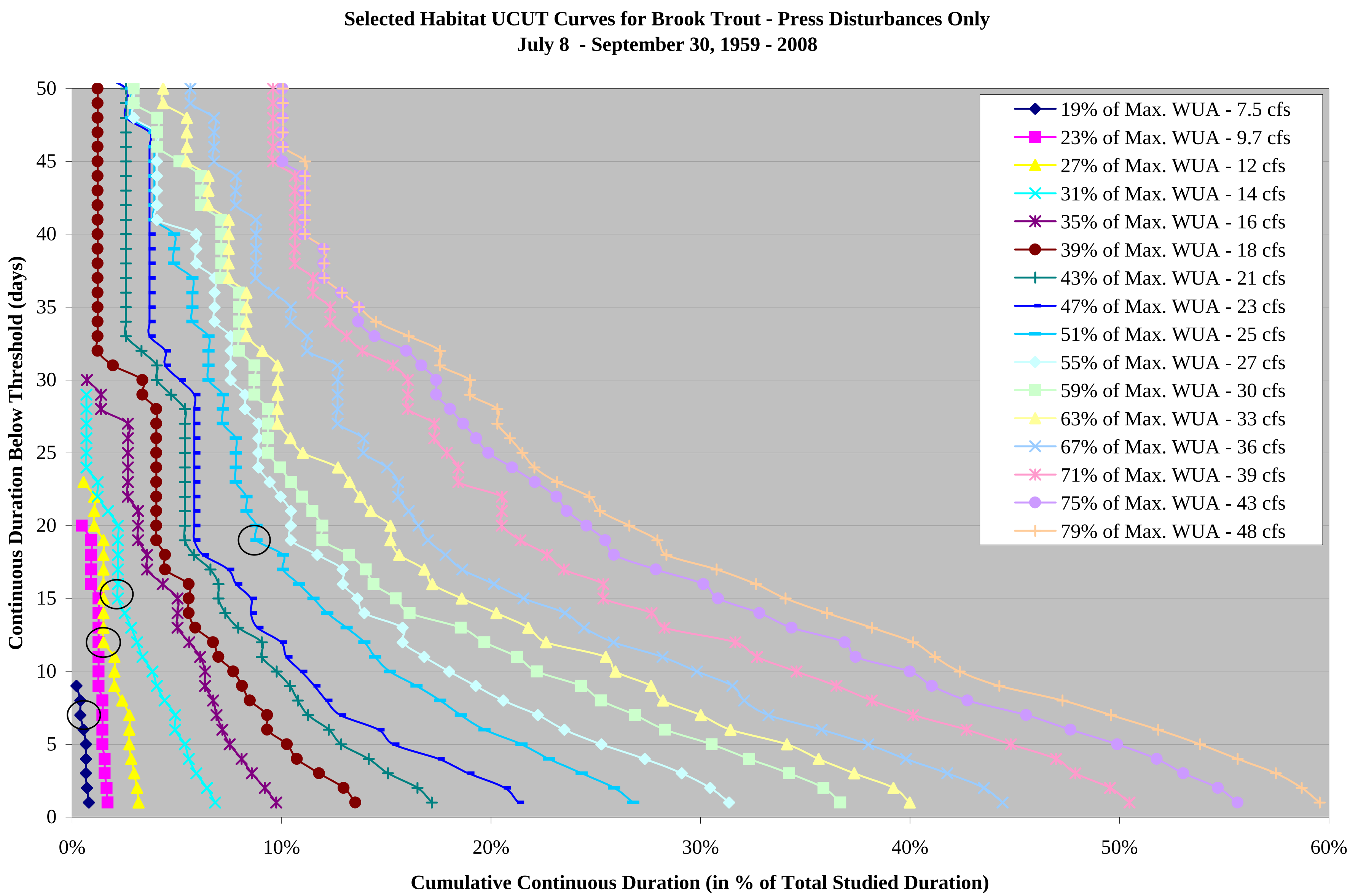


MILONE & MACBROOM®

Selected Habitat UCUT Curves for Brown Trout - Press Disturbances Only
July 8 - September 30, 1959 - 2008

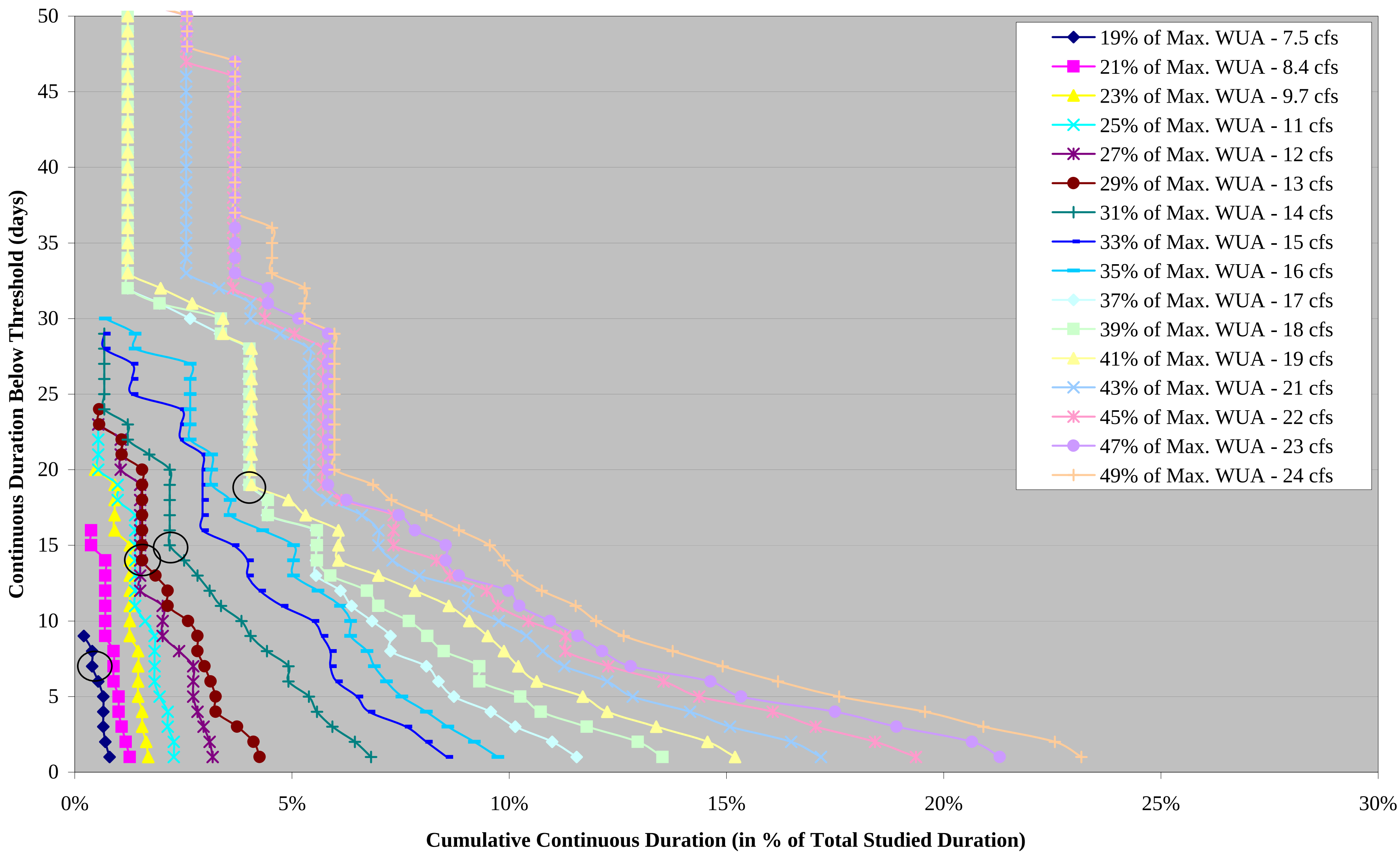


MILONE & MACBROOM®



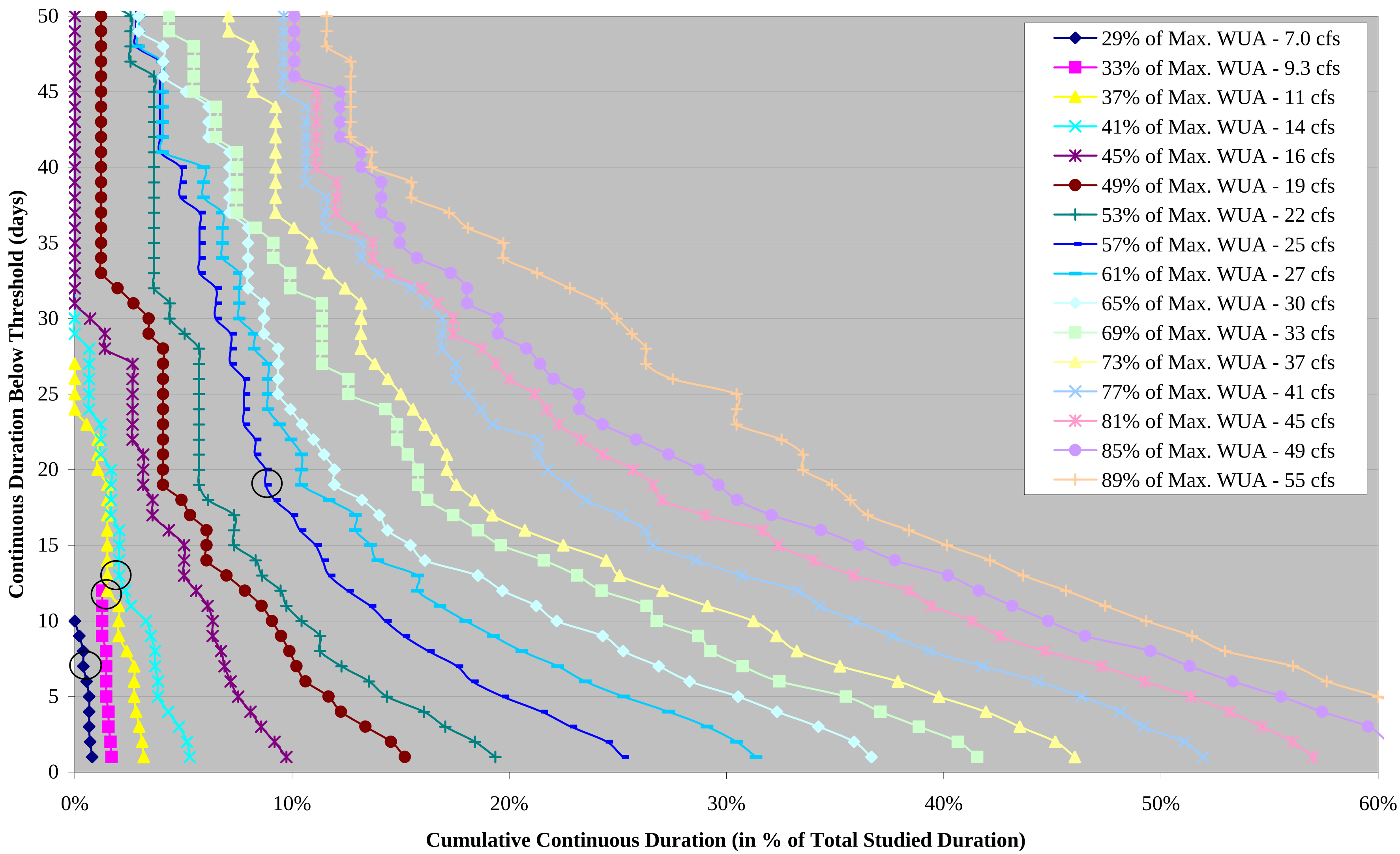
MILONE & MACBROOM®

Selected Habitat UCUT Curves for Brook Trout - Press Disturbances Only
July 8 - September 30, 1959 - 2008



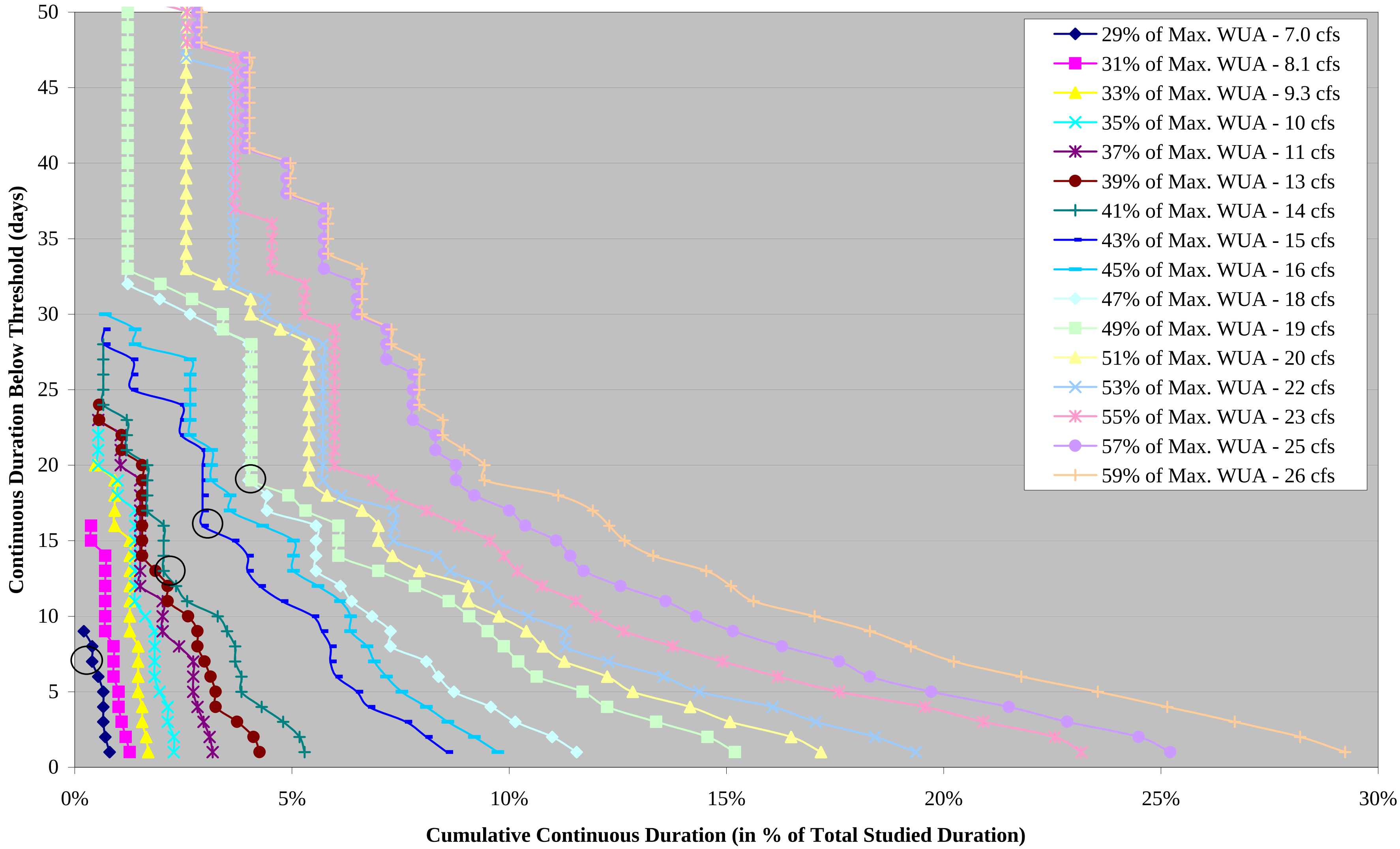
MILONE & MACBROOM®

Selected Habitat UCUT Curves for Fallfish - Press Disturbances Only
July 8 - September 30, 1959 - 2008

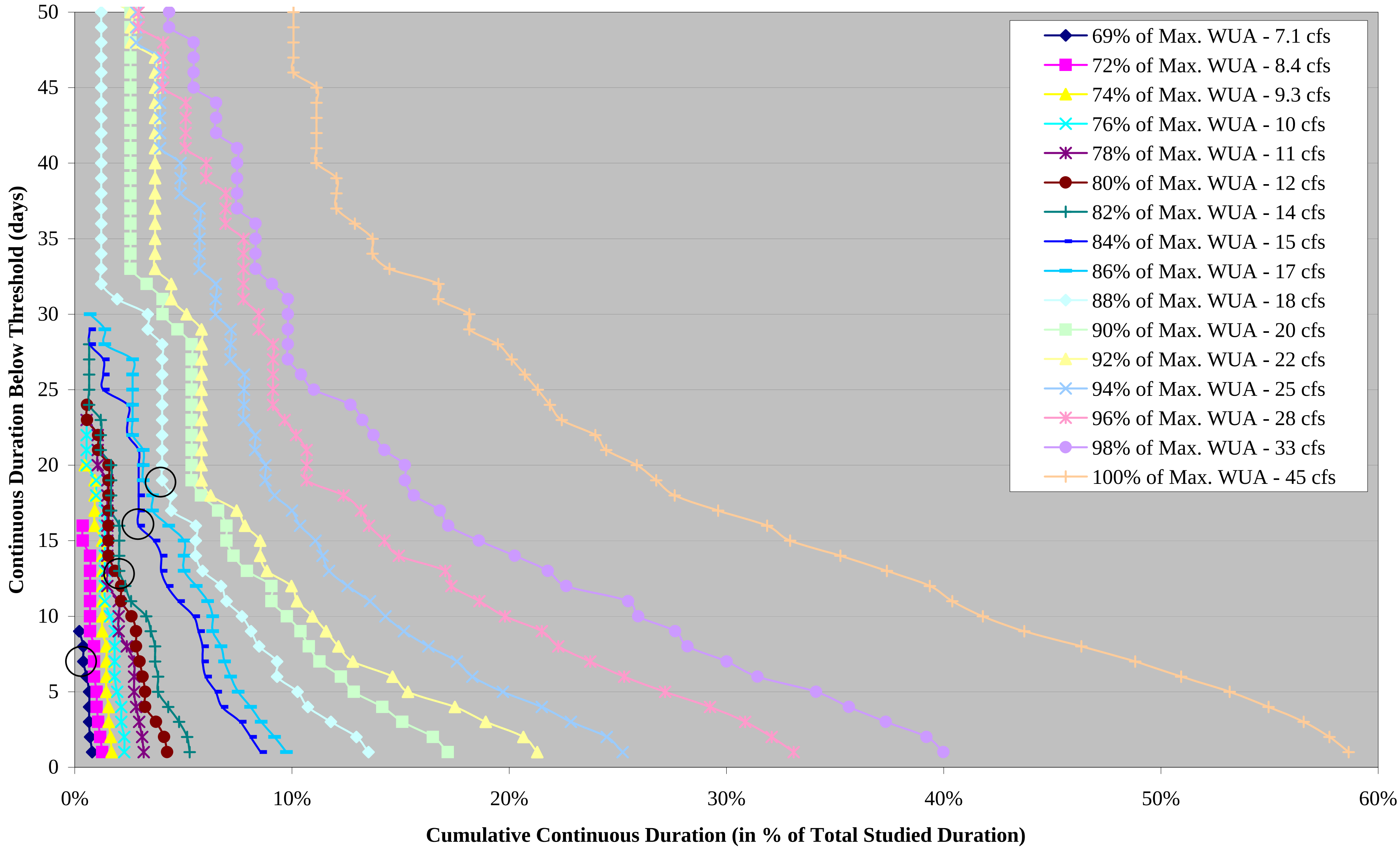


MILONE & MACBROOM®

Selected Habitat UCUT Curves for Fallfish - Press Disturbances Only
July 8 - September 30, 1959 - 2008



Selected Habitat UCUT Curves for Common Shiner (Adult) - Press Disturbances Only
July 8 - September 30, 1959 - 2008



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APPENDIX B
SUPPORTING DOCUMENTATION FOR CONVERSION OF MEAN DAILY FLOWS AT COVENTRY TO MEAN DAILY FLOWS ABOVE WELLFIELD

Appendix B
Supporting Documentation for Conversion of Mean Daily Flows at Coventry to Mean Daily Flows Above Wellfield

Correction for Public Water Supply Withdrawals from Willimantic River Wellfield

Withdrawals have been metered at the Willimantic River Wellfield since 1984. Withdrawals for the Willimantic River Wellfield and the Fenton River Wellfield have been separately recorded since 1988. From 1984 to 1988, Willimantic River Wellfield withdrawal data were combined with Fenton River Wellfield withdrawal data. If withdrawals from before 1984 were metered, they are archived in paper format and were not available for this study.

As discussed in the most recent Water Supply Plan (Ritsick, 2004), the University water system is complex and serves both on-campus and off-campus customers in both Storrs and Mansfield. It is generally assumed that the majority of the Storrs area has been served by the University since before the 1950s, but data regarding the timing of hookups for customers prior to the 1990s water supply planning effort is limited.

Given the data limitations regarding the timing and extent of historical public water usage, an indirect method of calculating historical water usage was utilized. Census data for both Mansfield and Storrs was obtained from the U.S. Census Bureau and the Windham Region Council of Governments. While the census data collected for Storrs is limited as compared to the available data for Mansfield, the data presented below shows that the ratio of population in Storrs to the population in Mansfield is almost constant at approximately 55% from 1970 through 2000.

Comparison of Mansfield and Storrs Population Statistics

Year	Mansfield Population	Storrs Population	Ratio of Storrs Population to Mansfield Population
1930	3,349		
1940	4,559		
1950	10,008		
1960	14,638		
1970	19,994	10,691	53.5%
1980	20,634	11,394	55.2%
1990	21,103	12,198	57.8%
2000	20,720	10,996	53.1%
2008	24,662*		
		Average:	**54.9%**

*Estimate for 2008 as reported by the U.S. Census Bureau

Assuming that the ratio of population in Storrs to the population in Mansfield was also approximately 55% from 1958 through 1970 and assuming that the percentage of public water users within Storrs has also been relatively constant since 1958, the population trend for Mansfield provides an "index" for estimating the rate of increase of water production at the Willimantic River Wellfield from 1958 to 1988. This appendix presents tables used to evaluate the known population and production data and estimate values for the unknown periods. Population figures for each year between the known census values were interpolated linearly.

A baseline water usage figure was necessary to project back in time through the study period. The 10-year period from 1988 through 1997 was utilized because it represents the period of water usage prior to the UConn 2000 project, which helped enact many water conservation measures at the University. The known water production values at the Willimantic River Wellfield from the years 1988 through 1997 were then divided by the Mansfield population figures to generate a "water usage factor" in gallons per capita day (gpcd). This factor does not represent actual per capita consumption[1].

[1] Note that for the purposes of this study the water usage factor is simply a generalized figure relating the Mansfield Population "index" to water production at the Willimantic River Wellfield and has no use outside of this current analysis. It is not a true reflection of water usage per capita by the University, which in reality would include all population served divided by the sum of water produced by both the Fenton and Willimantic Wellfields.

The water usage factors for each month from 1988 through 1997 were then averaged to generate monthly figures of per capita water usage. For the years 1969 through 1987, the monthly average per capita water usage estimate for each month was multiplied by the Mansfield population to estimate water production for that month. The withdrawals (in gallons) in Table 1 in this appendix for all months and years were then converted to daily withdrawal figures for each month with units of cfs (Table 2).

As reported in Section 3.1, the Mansfield Training School was a separate water system prior to 1969. Thus, the monthly average per capita estimates of water usage discussed in the preceding paragraph could not be utilized to calculate water usage prior to 1969.

A different approach was used for years prior to 1969. Population data were collected regarding the population of Mansfield Training School to estimate water usage for the period from 1958 through 1968. Rather than conducting an extensive archival search, the periodic published information regarding the population of residents and staff at the Mansfield Training School was compiled from the Connecticut State Library (2008) and other newsprint sources (Carrier, 1973; Mesce, 1982; Megan, 1993). In addition, the National Register of Historic Places nomination form from 1987 (available online from the National Park Service) contains population and anecdotal data. This information is compiled below along with estimated, linearly interpolated, and calculated figures.

Known and Estimated Population and Water Usage at Mansfield Training School, 1932 to 1969

Year	MTS Residents	MTS Staff	Total Population	Water Usage (gpd)	Water Usage (cfs)
1932	**1,070**	578	1,648	221,357	0.34
1933	1,077	582	1,659	222,852	0.34
1934	1,084	586	1,670	224,346	0.35
1935	1,092	590	1,681	225,840	0.35
1936	1,099	593	1,692	227,334	0.35
1937	1,106	597	1,703	228,828	0.35
1938	1,113	601	1,715	230,322	0.36
1939	1,121	605	1,726	231,816	0.36
1940	1,128	609	1,737	233,310	0.36
1941	1,135	613	1,748	234,804	0.36
1942	1,142	617	1,759	236,299	0.37
1943	1,149	621	1,770	237,793	0.37
1944	1,157	625	1,781	239,287	0.37
1945	1,164	629	1,792	240,781	0.37
1946	1,171	632	1,804	242,275	0.37
1947	1,178	636	1,815	243,769	0.38
1948	1,186	640	1,826	245,263	0.38
1949	1,193	644	1,837	246,757	0.38
1950	***1,200***	648	1,848	248,251	0.38
1951	1,262	682	1,944	261,095	0.40
1952	1,324	715	2,039	273,939	0.42
1953	1,386	749	2,135	286,782	0.44
1954	1,448	782	2,230	299,626	0.46
1955	1,510	816	2,326	312,469	0.48
1956	1,573	849	2,422	325,313	0.50
1957	1,635	883	2,517	338,156	0.52
1958	1,697	916	2,613	351,000	0.54
1959	1,759	950	2,708	363,843	0.56
1960	1,821	983	2,804	376,687	0.58
1961	1,883	1,017	2,900	389,531	0.60
1962	**1,945**	1,050	2,995	402,374	0.62
1963	1,897	1,024	2,921	392,444	0.61
1964	1,849	998	2,847	**382,514**	0.59
1965	1,801	973	2,774	372,584	0.58
1966	1,753	947	2,700	362,654	0.56
1967	1,705	921	2,626	352,724	0.55
1968	1,657	895	2,552	342,794	0.53
1969	**1,609**	**875**	2,484	333,689	0.52

Notes: Known figures are in **bold**, and figures estimated for this project are in ***bold italics***. All other figures are calculated or interpolated using these figures.
Water usage is calculated at 134.3 gallons per capita day based on 1964 water usage (Thomas Jr., et al., 1967)
MTS Residents and Staff compiled from various data sources. Population growth occurred in 1930 and 1950.
MTS Staff prior to 1969 is based on 1969 ratio of staff to residents (0.54 to 1).

Monthly variations of water usage at the Mansfield Training School are not known, so the annual water production figures above were applied uniformly to each month of the related year for the years 1958 through 1968. These values are shown on Table 1 of this appendix and converted to daily production values in cfs on Table 2 of this appendix.

The daily production figures for each month in Table 2 of this appendix were then used to correct the Coventry dataset for wellfield withdrawals. Thus, every day of a given month has an equal correction factor applied to each daily streamflow. This averaging is appropriate for a dataset based on mean daily gage data because day-to-day pumping fluctuations are not immediately reflected in the mean daily flow values.

Correction for Wastewater Discharges to Willimantic River

The current water pollution control facility (WPCF) on the main campus was constructed in the mid 1960s. The Mansfield Training School utilized its own WPCF up until its closure in 1993, which delivered effluent to the Willimantic River at Plains Road. The University consolidated the two systems and performed upgrades to the main campus WPCF in 1995 to increase treatment efficiency and capacity, including constructing the current outfall pipe delivering effluent to the Willimantic River downstream of Route 275. Prior to 1995, the University discharged to sand beds adjacent to Eagleville Brook, which flows to the Willimantic River just upstream of Route 275. Each of these discharge points is located upstream of the USGS gaging station at Coventry.

While wellfield withdrawals could be estimated from 20 years of data, monthly wastewater flows were only available since 2005. As such, each month has only two to four measurements as shown on Table 3 of this appendix. Given the data limitations, estimating wastewater effluent discharges based on the population of Mansfield was appropriate for the following reasons:

1. The effluent discharges from both the University and the Mansfield Training School systems have always been located upstream of the Coventry gage such that both will have historically increased discharge in the Willimantic River.

2. The population of Mansfield includes the population of the University and the population of Mansfield Training School.
3. It is unlikely that per capita rates of wastewater generation have changed significantly over the period in question.

Similar to the analysis described above, the known monthly wastewater effluent dicharge values were divided by the population figures for Mansfield (Table 1) to develop a per-person monthly "wastewater effluent discharge factor" in gpcd. As with the water usage factor described above, the wastewater effluent discharge factor is an index created for this study and has no useful meaning outside of the current analysis.

Each monthly average wastewater effluent discharge per capita factor was then multiplied by the population figure for Mansfield for each respective month of each year where data needed to be estimated (Table 3 of Appendix B). Table 4 of this appendix presents the known and estimated wastewater effluent discharge figures (in units of cfs) utilized to correct the Coventry dataset for wastewater inputs.

Locational Correction by Watershed Ratio

A simple equation is commonly used to convert a known discharge value on a stream at a gaged location to an estimated discharge value at a point someplace else on the same stream. This conversion is accomplished by first dividing the discharge value by the area of the watershed draining to that point (resulting in a per-area discharge value in cubic feet per second per square mile, or csm) and then multiplying the per-area discharge value by the area of the watershed draining to the new location on the stream.

More simply, this conversion can be performed by dividing the area of the smaller watershed by that of the larger watershed to generate a unitless number known as the "watershed ratio" that can then be multiplied by the known discharge value to calculate the needed discharge value. Searcy (1959) also outlines methods in which this conversion can be applied to ungaged sites along different streams in the same watershed or in separate watersheds. For this analysis, the watershed ratio is equal to the area of the watershed draining to the USGS gaging station at Mansfield Depot (98 square miles) divided by the area of the watershed draining to the USGS gaging station at Coventry (121 square miles) along the Willimantic River, or approximately 0.81.

During the initial development of the "natural" wellfield dataset based on the Coventry gage data, it became apparent that for daily mean discharges less than 13 cfs the USGS-reported daily mean discharge value at the USGS gaging station at Mansfield Depot was often very similar to the discharge at the Coventry gage for the same date. These similar occurrences were assumed to be due to the Willimantic River reaching a baseflow condition. As such, the initial hydraulic dataset generated for this study did not multiply by watershed ratio for discharges less than 13 cfs as recorded at the USGS gaging station in Coventry.

The data collected at the USGS gaging station at Mansfield Depot from October 1, 2005 through September 30, 2008 was approved by the USGS for publication toward the end of this study. Based on the recommendation of the TAG, the new dataset was reviewed to determine if the 13 cfs cutoff for utilizing the watershed ratio remained appropriate. Overall, there are a total of 70 daily river flow records below 13 cfs at the Coventry gage that have concurrent records at the USGS gaging station at Mansfield Depot. Of these, 44 of the values for the Mansfield Depot station are considered provisional, and 26 records are considered approved.

Table 5 of this appendix contains a comparison of the approved and provisional USGS discharges for the Mansfield Depot gaging station and a natural-condition dataset for a site at the wellfield, with and without the watershed ratio for discharges below 13 cfs at the Coventry gaging station. Note that the Mansfield Depot gaging station is located midway alongside the Willimantic River Wellfield. Thus, a correction factor equal to 50% of the average daily wellfield withdrawal for that month (50% of the monthly average withdrawal value) was added to the Mansfield Depot gaging station data to eliminate the effect of the Willimantic River Wellfield.

Upon analyzing the comparisons, it became apparent that while the provisional USGS flow data at Mansfield Depot (after adjustment for 50% of the withdrawal rate) were similar to the Coventry gage data *without* the watershed ratio applied the approved USGS data at Mansfield Depot (after adjustment for 50% of the withdrawal rate) were very similar to the Coventry gage data *with* the watershed ratio applied. The table below summarizes these findings.

Summary of Streamflow Data Comparisons

Type of Data at Mansfield Depot Gaging Station	Comparison of Mansfield Depot Gage Data and Coventry Gage Data				Total
	Excellent (difference of < 0.5 cfs)	Good (difference of 0.5 to 1 cfs)	Fair (difference of 1 to 2 cfs)	Poor (difference of > 2 cfs)	
Provisional data[2]	6 (14%)	15 (34%)	8 (18%)	15 (34%)	44[1]
Approved data[3]	21 (81%)	3 (11%)	2 (8%)	0 (0%)	26[1]

1. The total number of concurrent records below 13 cfs (as measured at the Coventry station) for the Coventry gaging station and the Mansfield Depot gaging station is 70, with 44 provisional values and 26 approved values from the Mansfield Depot gage.
2. The provisional data (corrected for 50% of the wellfield withdrawal) was compared to the natural dataset (Coventry gage data) without the watershed ratio applied.
3. The approved dataset (corrected for 50% of the wellfield withdrawal) was compared to the natural dataset (Coventry gage data) with the watershed correction applied.

Nearly 81% of the approved daily data from the Mansfield Depot gage has an excellent comparison to the natural dataset for the wellfield generated using the watershed ratio applied to the Coventry gage data. Therefore, the analysis presented in Table 5 of this appendix, along with the summary data presented above, indicate that the use of the watershed ratio is appropriate for correcting the low-flow USGS data recorded at the Coventry gaging station to represent a natural condition at the wellfield. As the approved Mansfield Depot data matched very well with the natural data set generated using the watershed ratio, the natural dataset was considered suitable for use in the Instream Flow Study.

TABLE 1- WILLIMANTIC WELLFIELD PRODUCTION (GAL) - BOLD VALUES ARE KNOWN VALUES

Year	Population	January	Gal pcd	February	Gal pcd	March	Gal pcd	April	Gal pcd	May	Gal pcd	June	Gal pcd	July	Gal pcd	August	Gal pcd	September	Gal pcd	October	Gal pcd	November	Gal pcd	December	Gal pcd
1958	13,712	18,446	134.3	18,446	134.3	18,446	134.3	18,446	134.3	18,446	134.3	18,446	134.3	18,446	134.3	18,446	134.3	18,446	134.3	18,446	134.3	18,446	134.3	18,446	134.3
1959	14,175	18,571	134.3	18,571	134.3	18,571	134.3	18,571	134.3	18,571	134.3	18,571	134.3	18,571	134.3	18,571	134.3	18,571	134.3	18,571	134.3	18,571	134.3	18,571	134.3
1960	**14,638**	18,695	134.3	18,695	134.3	18,695	134.3	18,695	134.3	18,695	134.3	18,695	134.3	18,695	134.3	18,695	134.3	18,695	134.3	18,695	134.3	18,695	134.3	18,695	134.3
1961	15,174	18,820	134.3	18,820	134.3	18,820	134.3	18,820	134.3	18,820	134.3	18,820	134.3	18,820	134.3	18,820	134.3	18,820	134.3	18,820	134.3	18,820	134.3	18,820	134.3
1962	15,709	18,944	134.3	18,944	134.3	18,944	134.3	18,944	134.3	18,944	134.3	18,944	134.3	18,944	134.3	18,944	134.3	18,944	134.3	18,944	134.3	18,944	134.3	18,944	134.3
1963	16,245	19,069	134.3	19,069	134.3	19,069	134.3	19,069	134.3	19,069	134.3	19,069	134.3	19,069	134.3	19,069	134.3	19,069	134.3	19,069	134.3	19,069	134.3	19,069	134.3
1964	16,780	19,194	134.3	19,194	134.3	19,194	134.3	19,194	134.3	19,194	134.3	19,194	134.3	19,194	134.3	19,194	134.3	19,194	134.3	19,194	134.3	19,194	134.3	19,194	134.3
1965	17,316	19,318	134.3	19,318	134.3	19,318	134.3	19,318	134.3	19,318	134.3	19,318	134.3	19,318	134.3	19,318	134.3	19,318	134.3	19,318	134.3	19,318	134.3	19,318	134.3
1966	17,852	19,443	134.3	19,443	134.3	19,443	134.3	19,443	134.3	19,443	134.3	19,443	134.3	19,443	134.3	19,443	134.3	19,443	134.3	19,443	134.3	19,443	134.3	19,443	134.3
1967	18,387	19,567	134.3	19,567	134.3	19,567	134.3	19,567	134.3	19,567	134.3	19,567	134.3	19,567	134.3	19,567	134.3	19,567	134.3	19,567	134.3	19,567	134.3	19,567	134.3
1968	18,923	19,692	134.3	19,692	134.3	19,692	134.3	19,692	134.3	19,692	134.3	19,692	134.3	19,692	134.3	19,692	134.3	19,692	134.3	19,692	134.3	19,692	134.3	19,692	134.3
1969	19,458	26,782,542	44.4	28,658,332	52.6	30,522,446	50.6	32,223,110	55.2	29,919,236	49.6	29,479,476	50.5	27,807,999	46.1	24,610,984	40.8	31,522,608	54.0	31,668,546	52.5	30,121,603	51.6	29,858,915	49.5
1970	**19,994**	27,519,742	44.4	29,447,163	52.6	31,362,588	50.6	33,110,064	55.2	30,742,774	49.6	30,290,910	50.5	28,573,425	46.1	25,288,411	40.8	32,390,280	54.0	32,540,235	52.5	30,950,712	51.6	30,680,793	49.5
1971	20,058	27,607,831	44.4	29,541,422	52.6	31,462,979	50.6	33,216,048	55.2	30,841,181	49.6	30,387,870	50.5	28,664,888	46.1	25,369,358	40.8	32,493,960	54.0	32,644,395	52.5	31,049,784	51.6	30,779,001	49.5
1972	20,122	27,695,921	44.4	30,694,099	52.6	31,563,369	50.6	33,322,032	55.2	30,939,587	49.6	30,484,830	50.5	28,756,350	46.1	25,450,306	40.8	32,597,640	54.0	32,748,555	52.5	31,148,856	51.6	30,877,209	49.5
1973	20,186	27,784,010	44.4	29,729,941	52.6	31,663,760	50.6	33,428,016	55.2	31,037,994	49.6	30,581,790	50.5	28,847,813	46.1	25,531,253	40.8	32,701,320	54.0	32,852,715	52.5	31,247,928	51.6	30,975,417	49.5
1974	20,250	27,872,100	44.4	29,824,200	52.6	31,764,150	50.6	33,534,000	55.2	31,136,400	49.6	30,678,750	50.5	28,939,275	46.1	25,612,200	40.8	32,805,000	54.0	32,956,875	52.5	31,347,000	51.6	31,073,625	49.5
1975	20,314	27,960,190	44.4	29,918,459	52.6	31,864,540	50.6	33,639,984	55.2	31,234,806	49.6	30,775,710	50.5	29,030,737	46.1	25,693,147	40.8	32,908,680	54.0	33,061,035	52.5	31,446,072	51.6	31,171,833	49.5
1976	20,378	28,048,279	44.4	31,084,601	52.6	31,964,931	50.6	33,745,968	55.2	31,333,213	49.6	30,872,670	50.5	29,122,200	46.1	25,774,094	40.8	33,012,360	54.0	33,165,195	52.5	31,545,144	51.6	31,270,041	49.5
1977	20,442	28,136,369	44.4	30,106,978	52.6	32,065,321	50.6	33,851,952	55.2	31,431,619	49.6	30,969,630	50.5	29,213,662	46.1	25,855,042	40.8	33,116,040	54.0	33,269,355	52.5	31,644,216	51.6	31,368,249	49.5
1978	20,506	28,224,458	44.4	30,201,237	52.6	32,165,712	50.6	33,957,936	55.2	31,530,026	49.6	31,066,590	50.5	29,305,125	46.1	25,935,989	40.8	33,219,720	54.0	33,373,515	52.5	31,743,288	51.6	31,466,457	49.5
1979	20,570	28,312,548	44.4	30,295,496	52.6	32,266,102	50.6	34,063,920	55.2	31,628,432	49.6	31,163,550	50.5	29,396,587	46.1	26,016,936	40.8	33,323,400	54.0	33,477,675	52.5	31,842,360	51.6	31,564,665	49.5
1980	**20,634**	28,400,638	44.4	31,475,104	52.6	32,366,492	50.6	34,169,904	55.2	31,726,838	49.6	31,260,510	50.5	29,488,049	46.1	26,097,883	40.8	33,427,080	54.0	33,581,835	52.5	31,941,432	51.6	31,662,873	49.5
1981	20,681	28,465,191	44.4	30,458,830	52.6	32,440,060	50.6	34,247,570	55.2	31,798,952	49.6	31,331,564	50.5	29,555,074	46.1	26,157,202	40.8	33,503,058	54.0	33,658,165	52.5	32,014,033	51.6	31,734,841	49.5
1982	20,728	28,529,744	44.4	30,527,904	52.6	32,513,627	50.6	34,325,237	55.2	31,871,065	49.6	31,402,617	50.5	29,622,099	46.1	26,216,521	40.8	33,579,036	54.0	33,734,495	52.5	32,086,634	51.6	31,806,809	49.5
1983	20,775	28,594,297	44.4	30,596,978	52.6	32,587,194	50.6	34,402,903	55.2	31,943,179	49.6	31,473,671	50.5	29,689,124	46.1	26,275,841	40.8	33,655,014	54.0	33,810,824	52.5	32,159,236	51.6	31,878,777	49.5
1984	20,822	28,658,850	44.4	31,761,269	52.6	32,660,762	50.6	34,480,570	55.2	32,015,292	49.6	31,544,724	50.5	29,756,149	46.1	26,335,160	40.8	33,730,992	54.0	33,887,154	52.5	32,231,837	51.6	31,950,745	49.5
1985	20,869	28,723,403	44.4	30,735,127	52.6	32,734,329	50.6	34,558,236	55.2	32,087,406	49.6	31,615,778	50.5	29,823,173	46.1	26,394,479	40.8	33,806,970	54.0	33,963,484	52.5	32,304,438	51.6	32,022,713	49.5
1986	20,915	28,787,957	44.4	30,804,201	52.6	32,807,896	50.6	34,635,902	55.2	32,159,519	49.6	31,686,831	50.5	29,890,198	46.1	26,453,798	40.8	33,882,948	54.0	34,039,814	52.5	32,377,039	51.6	32,094,681	49.5
1987	20,962	28,852,510	44.4	30,873,275	52.6	32,881,464	50.6	34,713,569	55.2	32,231,632	49.6	31,757,885	50.5	29,957,223	46.1	26,513,117	40.8	33,958,926	54.0	34,116,143	52.5	32,449,640	51.6	32,166,649	49.5
1988	21,009	**21,410,000**	32.9	**38,660,000**	63.5	**38,440,000**	59.0	**38,990,000**	61.9	**38,160,000**	58.6	**38,820,000**	61.6	**27,460,000**	42.2	**15,980,000**	24.5	**36,910,000**	58.6	**42,330,000**	65.0	**36,780,000**	58.4	**39,380,000**	60.5
1989	21,056	**24,020,000**	36.8	**35,950,000**	61.0	**33,960,000**	52.0	**39,520,000**	62.6	**39,710,000**	60.8	**35,740,000**	56.6	**37,330,000**	57.2	**28,340,000**	43.4	**37,840,000**	59.9	**39,920,000**	61.2	**36,200,000**	57.3	**30,370,000**	46.5
1990	**21,103**	**27,730,000**	42.4	**32,760,000**	55.4	**35,800,000**	54.7	**37,560,000**	59.3	**38,270,000**	58.5	**38,440,000**	60.7	**34,630,000**	52.9	**37,620,000**	57.5	**36,210,000**	57.2	**35,710,000**	54.6	**35,580,000**	56.2	**35,920,000**	54.9
1991	21,065	**38,230,000**	58.5	**31,600,000**	53.6	**35,300,000**	54.1	**33,520,000**	53.0	**34,160,000**	52.3	**35,320,000**	55.9	**31,790,000**	48.7	**32,200,000**	49.3	**31,500,000**	49.8	**31,040,000**	47.5	**30,980,000**	49.0	**30,610,000**	46.9
1992	21,026	**28,770,000**	44.1	**26,000,000**	42.6	**29,620,000**	45.4	**29,020,000**	46.0	**25,750,000**	39.5	**25,770,000**	40.9	**16,190,000**	24.8	**24,610,000**	37.8	**22,050,000**	35.0	**21,600,000**	33.1	**22,640,000**	35.9	**23,500,000**	36.1
1993	20,988	**22,400,000**	34.4	**19,320,000**	32.9	**25,040,000**	38.5	**21,900,000**	34.8	**17,860,000**	27.5	**18,520,000**	29.4	**20,540,000**	31.6	**23,160,000**	35.6	**34,570,000**	54.9	**32,940,000**	50.6	**30,900,000**	49.1	**34,630,000**	53.2
1994	20,950	**35,520,000**	54.7	**29,900,000**	51.0	**25,690,000**	39.6	**36,770,000**	58.5	**32,420,000**	49.9	**33,580,000**	53.4	**33,050,000**	50.9	**35,420,000**	54.5	**40,110,000**	63.8	**38,220,000**	58.9	**37,340,000**	59.4	**36,900,000**	56.8
1995	20,912	**39,000,000**	60.2	**35,210,000**	60.1	**37,800,000**	58.3	**39,300,000**	62.6	**36,080,000**	55.7	**34,310,000**	54.7	**36,810,000**	56.8	**12,900,000**	19.9	**36,060,000**	57.5	**36,910,000**	56.9	**34,030,000**	54.2	**35,300,000**	54.5
1996	20,873	**30,800,000**	47.6	**36,100,000**	59.6	**37,160,000**	57.4	**35,390,000**	56.5	**31,770,000**	49.1	**31,470,000**	50.3	**33,600,000**	51.9	**28,020,000**	43.3	**34,080,000**	54.4	**32,490,000**	50.2	**31,230,000**	49.9	**28,010,000**	43.3
1997	20,835	**20,740,000**	32.1	**26,800,000**	45.9	**30,630,000**	47.4	**35,170,000**	56.3	**28,530,000**	44.2	**25,730,000**	41.2	**28,640,000**	44.3	**27,500,000**	42.6	**30,650,000**	49.0	**30,630,000**	47.4	**29,000,000**	46.4	**27,520,000**	42.6
1998	20,797	**29,440,000**	45.7	**28,570,000**	49.1	**28,460,000**	44.1	**32,840,000**	52.6	**29,560,000**	45.9	**24,710,000**	39.6	**30,710,000**	47.6	**27,490,000**	42.6	**30,820,000**	49.4	**36,560,000**	56.7	**28,930,000**	46.4	**29,700,000**	46.1
1999	20,758	**28,771,500**	44.7	**28,873,500**	49.7	**30,952,100**	48.1	**29,951,800**	48.1	**28,240,600**	43.9	**28,944,600**	46.5	**29,099,900**	45.2	**21,066,900**	32.7	**40,912,001**	65.7	**39,897,073**	62.0	**33,413,282**	53.7	**37,110,061**	57.7
2000	**20,720**	**27,358,744**	42.6	**35,990,795**	59.9	**33,145,246**	51.6	**30,683,336**	49.4	**19,723,693**	30.7	**20,650,069**	33.2	**20,694,806**	32.2	**32,997,645**	51.4	**35,536,138**	57.2	**35,061,483**	54.6	**36,663,622**	59.0	**36,989,769**	57.6
2001	21,188	**32,481,429**	49.5	**38,681,138**	65.2	**36,867,718**	56.1	**36,867,718**	58.0	**36,867,718**	56.1	**32,242,441**	50.7	**32,242,441**	49.1	**32,242,441**	49.1	**39,211,579**	61.7	**40,360,469**	61.4	**34,792,903**	54.7	**33,812,440**	51.5
2002	21,656	**33,360,299**	49.7	**33,523,944**	55.3	**35,294,016**	52.6	**38,632,395**	59.5	**36,070,523**	53.7	**30,406,698**	46.8	**35,224,520**	52.5	**36,235,224**	54.0	**35,195,743**	54.2	**33,796,561**	50.3	**30,801,321**	47.4	**31,559,604**	47.0
2003	22,123	**33,406,412**	48.7	**32,236,967**	52.0	**31,777,744**	46.3	**34,745,987**	52.4	**31,198,828**	45.5	**26,984,844**	40.7	**30,072,672**	43.8	**33,317,246**	48.6	**33,946,250**	51.1	**32,154,730**	46.9	**32,354,468**	48.7	**32,354,468**	47.2
2004	22,591	**31,285,000**	44.7	**31,688,000**	48.4	**29,607,000**	42.3	**33,217,000**	49.0	**20,332,000**	29.0	**24,631,000**	36.3	**29,145,000**	41.6	**29,402,000**	42.0	**32,602,000**	48.1	**30,615,000**	43.7	**29,320,000**	43.3	**34,096,000**	48.7
2005	23,059	**33,581,000**	47.0	**32,942,000**	51.0	**33,482,000**	46.8	**32,989,000**	47.7	**31,721,000**	44.4	**29,370,000**	42.5	**31,463,000**	44.0	**33,216,000**	46.5	**36,027,000**	52.1	**38,064,000**	53.2	**34,482,000**	49.8	**30,261,000**	42.3
2006	23,527	**29,853,000**	40.9	**30,950,000**	47.0	**28,097,000**	38.5	**31,745,000**	45.0	**27,663,000**	37.9	**31,058,000**	44.0	**40,945,000**	56.1	**43,586,000**	59.8	**46,566,000**	66.0	**46,145,000**	63.3	**39,038,000**	55.3	**30,365,000**	41.6
2007	23,994	**33,157,000**	44.6	**38,492,000**	57.3	**31,496,000**	42.3	**35,683,000**	49.6	**28,298,000**	38.0	**29,775,000**	41.4	**36,051,000**	48.5	**40,167,000**	54.0	**46,539,000**	64.7	**45,214,000**	60.8	**36,600,000**	50.8	**31,987,000**	43.0
2008	**24,462**	**28,872,000**	38.1	**35,892,000**	50.6	**28,817,000**	38.0	**29,449,000**	40.1	**20,670,000**	27.3	**24,184,000**	33.0	**33,938,000**	44.8	**36,176,000**	47.7	**41,317,000**	56.3	**35,839,000**	47.3				
Average 1988-1997			44.4		52.6		50.6		55.2		49.6		50.5		46.1		40.8		54.0		52.5		51.6		49.5

TABLE 2 - WILLIMANTIC RIVER WELLFIELD PRODUCTION, CFS

Year	January	February	March	April	May	June	July	August	September	October	November	December
1958	0.54	0.54	0.54	0.54	0.54	0.54	0.54	0.54	0.54	0.54	0.54	0.54
1959	0.56	0.56	0.56	0.56	0.56	0.56	0.56	0.56	0.56	0.56	0.56	0.56
1960	0.58	0.58	0.58	0.58	0.58	0.58	0.58	0.58	0.58	0.58	0.58	0.58
1961	0.60	0.60	0.60	0.60	0.60	0.60	0.60	0.60	0.60	0.60	0.60	0.60
1962	0.62	0.62	0.62	0.62	0.62	0.62	0.62	0.62	0.62	0.62	0.62	0.62
1963	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61
1964	0.59	0.59	0.59	0.59	0.59	0.59	0.59	0.59	0.59	0.59	0.59	0.59
1965	0.58	0.58	0.58	0.58	0.58	0.58	0.58	0.58	0.58	0.58	0.58	0.58
1966	0.56	0.56	0.56	0.56	0.56	0.56	0.56	0.56	0.56	0.56	0.56	0.56
1967	0.55	0.55	0.55	0.55	0.55	0.55	0.55	0.55	0.55	0.55	0.55	0.55
1968	0.53	0.53	0.53	0.53	0.53	0.53	0.53	0.53	0.53	0.53	0.53	0.53
1969	1.34	1.43	1.52	1.61	1.49	1.47	1.39	1.23	1.57	1.58	1.50	1.49
1970	1.37	1.47	1.57	1.65	1.53	1.51	1.43	1.26	1.62	1.62	1.54	1.53
1971	1.38	1.47	1.57	1.66	1.54	1.52	1.43	1.27	1.62	1.63	1.55	1.54
1972	1.38	1.53	1.58	1.66	1.54	1.52	1.44	1.27	1.63	1.63	1.55	1.54
1973	1.39	1.48	1.58	1.67	1.55	1.53	1.44	1.27	1.63	1.64	1.56	1.55
1974	1.39	1.49	1.59	1.67	1.55	1.53	1.44	1.28	1.64	1.64	1.56	1.55
1975	1.40	1.49	1.59	1.68	1.56	1.54	1.45	1.28	1.64	1.65	1.57	1.56
1976	1.40	1.55	1.60	1.68	1.56	1.54	1.45	1.29	1.65	1.66	1.57	1.56
1977	1.40	1.50	1.60	1.69	1.57	1.55	1.46	1.29	1.65	1.66	1.58	1.57
1978	1.41	1.51	1.61	1.69	1.57	1.55	1.46	1.29	1.66	1.67	1.58	1.57
1979	1.41	1.51	1.61	1.70	1.58	1.56	1.47	1.30	1.66	1.67	1.59	1.58
1980	1.42	1.57	1.62	1.71	1.58	1.56	1.47	1.30	1.67	1.68	1.59	1.58
1981	1.42	1.52	1.62	1.71	1.59	1.56	1.48	1.31	1.67	1.68	1.60	1.58
1982	1.42	1.52	1.62	1.71	1.59	1.57	1.48	1.31	1.68	1.68	1.60	1.59
1983	1.43	1.53	1.63	1.72	1.59	1.57	1.48	1.31	1.68	1.69	1.61	1.59
1984	1.43	1.59	1.63	1.72	1.60	1.57	1.49	1.31	1.68	1.69	1.61	1.59
1985	1.43	1.53	1.63	1.72	1.60	1.58	1.49	1.32	1.69	1.70	1.61	1.60
1986	1.44	1.54	1.64	1.73	1.61	1.58	1.49	1.32	1.69	1.70	1.62	1.60
1987	1.44	1.54	1.64	1.73	1.61	1.59	1.50	1.32	1.69	1.70	1.62	1.61
1988	1.07	1.93	1.92	1.95	1.90	1.94	1.37	0.80	1.84	2.11	1.84	1.97
1989	1.20	1.79	1.69	1.97	1.98	1.78	1.86	1.41	1.89	1.99	1.81	1.52
1990	1.38	1.64	1.79	1.87	1.91	1.92	1.73	1.88	1.81	1.78	1.78	1.79
1991	1.91	1.58	1.76	1.67	1.70	1.76	1.59	1.61	1.57	1.55	1.55	1.53
1992	1.44	1.30	1.48	1.45	1.29	1.29	0.81	1.23	1.10	1.08	1.13	1.17
1993	1.12	0.96	1.25	1.09	0.89	0.92	1.03	1.16	1.73	1.64	1.54	1.73
1994	1.77	1.49	1.28	1.84	1.62	1.68	1.65	1.77	2.00	1.91	1.86	1.84
1995	1.95	1.76	1.89	1.96	1.80	1.71	1.84	0.64	1.80	1.84	1.70	1.76
1996	1.54	1.80	1.85	1.77	1.59	1.57	1.68	1.40	1.70	1.62	1.56	1.40
1997	1.04	1.34	1.53	1.76	1.42	1.28	1.43	1.37	1.53	1.53	1.45	1.37
1998	1.47	1.43	1.42	1.64	1.48	1.23	1.53	1.37	1.54	1.82	1.44	1.48
1999	1.44	1.44	1.54	1.49	1.41	1.44	1.45	1.05	2.04	1.99	1.67	1.85
2000	1.37	1.80	1.65	1.53	0.98	1.03	1.03	1.65	1.77	1.75	1.83	1.85
2001	1.62	1.93	1.84	1.84	1.84	1.61	1.61	1.61	1.96	2.01	1.74	1.69
2002	1.67	1.67	1.76	1.93	1.80	1.52	1.76	1.81	1.76	1.69	1.54	1.58
2003	1.67	1.61	1.59	1.73	1.56	1.35	1.50	1.66	1.69	1.60	1.61	1.61
2004	1.56	1.58	1.48	1.66	1.01	1.23	1.45	1.47	1.63	1.53	1.46	1.70
2005	1.68	1.64	1.67	1.65	1.58	1.47	1.57	1.66	1.80	1.90	1.72	1.51
2006	1.49	1.54	1.40	1.58	1.38	1.55	2.04	2.18	2.32	2.30	1.95	1.52
2007	1.65	1.92	1.57	1.78	1.41	1.49	1.80	2.00	2.32	2.26	1.83	1.60
2008	1.44	1.79	1.44	1.47	1.03	1.21	1.69	1.81	2.06	1.79		

TABLE 4 - WILLIMANTIC RIVER SEWERAGE OUTFLOW, CFS

Year	January	February	March	April	May	June	July	August	September	October	November	December
1958	1.13	1.19	1.12	1.32	0.83	0.77	0.75	0.79	1.04	1.11	1.04	0.90
1959	1.17	1.23	1.16	1.36	0.86	0.80	0.77	0.81	1.07	1.15	1.08	0.93
1960	1.21	1.32	1.19	1.41	0.88	0.82	0.80	0.84	1.11	1.19	1.11	0.96
1961	1.25	1.32	1.24	1.46	0.92	0.85	0.83	0.87	1.15	1.23	1.15	1.00
1962	1.30	1.37	1.28	1.51	0.95	0.88	0.86	0.90	1.19	1.28	1.19	1.03
1963	1.34	1.41	1.32	1.56	0.98	0.91	0.88	0.93	1.23	1.32	1.23	1.07
1964	1.38	1.51	1.37	1.62	1.01	0.94	0.91	0.96	1.27	1.36	1.27	1.10
1965	1.43	1.51	1.41	1.67	1.04	0.97	0.94	0.99	1.31	1.41	1.31	1.14
1966	1.47	1.55	1.46	1.72	1.08	1.01	0.97	1.02	1.35	1.45	1.36	1.17
1967	1.52	1.60	1.50	1.77	1.11	1.04	1.00	1.06	1.39	1.49	1.40	1.21
1968	1.56	1.71	1.54	1.82	1.14	1.07	1.03	1.09	1.43	1.54	1.44	1.24
1969	1.60	1.69	1.59	1.87	1.17	1.10	1.06	1.12	1.47	1.58	1.48	1.28
1970	1.65	1.74	1.63	1.92	1.21	1.13	1.09	1.15	1.51	1.62	1.52	1.31
1971	1.65	1.75	1.64	1.93	1.21	1.13	1.09	1.15	1.52	1.63	1.52	1.32
1972	1.66	1.81	1.64	1.94	1.21	1.13	1.10	1.16	1.52	1.63	1.53	1.32
1973	1.66	1.76	1.65	1.94	1.22	1.14	1.10	1.16	1.53	1.64	1.53	1.32
1974	1.67	1.76	1.65	1.95	1.22	1.14	1.10	1.16	1.53	1.64	1.54	1.33
1975	1.68	1.77	1.66	1.96	1.23	1.14	1.11	1.17	1.54	1.65	1.54	1.33
1976	1.68	1.84	1.66	1.96	1.23	1.15	1.11	1.17	1.54	1.66	1.55	1.34
1977	1.69	1.78	1.67	1.97	1.23	1.15	1.11	1.17	1.55	1.66	1.55	1.34
1978	1.69	1.79	1.67	1.97	1.24	1.15	1.12	1.18	1.55	1.67	1.56	1.35
1979	1.70	1.79	1.68	1.98	1.24	1.16	1.12	1.18	1.56	1.67	1.56	1.35
1980	1.70	1.86	1.68	1.99	1.25	1.16	1.12	1.18	1.56	1.68	1.57	1.35
1981	1.71	1.80	1.69	1.99	1.25	1.16	1.13	1.19	1.57	1.68	1.57	1.36
1982	1.71	1.80	1.69	2.00	1.25	1.17	1.13	1.19	1.57	1.68	1.57	1.36
1983	1.71	1.81	1.69	2.00	1.25	1.17	1.13	1.19	1.57	1.69	1.58	1.36
1984	1.72	1.88	1.70	2.00	1.26	1.17	1.13	1.20	1.58	1.69	1.58	1.37
1985	1.72	1.82	1.70	2.01	1.26	1.17	1.14	1.20	1.58	1.70	1.58	1.37
1986	1.72	1.82	1.71	2.01	1.26	1.18	1.14	1.20	1.58	1.70	1.59	1.37
1987	1.73	1.83	1.71	2.02	1.26	1.18	1.14	1.20	1.59	1.70	1.59	1.38
1988	1.73	1.89	1.71	2.02	1.27	1.18	1.14	1.21	1.59	1.71	1.59	1.38
1989	1.74	1.83	1.72	2.03	1.27	1.19	1.15	1.21	1.60	1.71	1.60	1.38
1990	1.74	1.84	1.72	2.03	1.27	1.19	1.15	1.21	1.60	1.71	1.60	1.38
1991	1.74	1.83	1.72	2.03	1.27	1.19	1.15	1.21	1.60	1.71	1.60	1.38
1992	1.73	1.90	1.71	2.02	1.27	1.18	1.15	1.21	1.59	1.71	1.60	1.38
1993	1.73	1.83	1.71	2.02	1.27	1.18	1.14	1.20	1.59	1.70	1.59	1.38
1994	1.73	1.82	1.71	2.02	1.26	1.18	1.14	1.20	1.59	1.70	1.59	1.37
1995	1.72	1.82	1.71	2.01	1.26	1.18	1.14	1.20	1.58	1.70	1.59	1.37
1996	1.72	1.88	1.70	2.01	1.26	1.18	1.14	1.20	1.58	1.70	1.58	1.37
1997	1.72	1.81	1.70	2.01	1.26	1.17	1.13	1.20	1.58	1.69	1.58	1.37
1998	1.72	1.81	1.70	2.00	1.25	1.17	1.13	1.19	1.58	1.69	1.58	1.36
1999	1.71	1.81	1.69	2.00	1.25	1.17	1.13	1.19	1.57	1.69	1.58	1.36
2000	1.71	1.87	1.69	1.99	1.25	1.17	1.13	1.19	1.57	1.68	1.57	1.36
2001	1.75	1.84	1.73	2.04	1.28	1.19	1.15	1.22	1.61	1.72	1.61	1.39
2002	1.79	1.89	1.77	2.08	1.31	1.22	1.18	1.24	1.64	1.76	1.64	1.42
2003	1.82	1.93	1.80	2.13	1.33	1.25	1.20	1.27	1.68	1.80	1.68	1.45
2004	1.86	2.04	1.84	2.18	1.36	1.27	1.23	1.30	1.71	1.84	1.71	1.48
2005	1.90	2.01	1.88	2.22	1.39	1.00	1.28	1.32	1.91	2.17	1.85	1.81
2006	2.20	2.40	1.71	2.15	1.58	1.81	1.42	1.53	1.84	1.90	1.95	1.52
2007	1.71	1.65	1.96	2.76	1.32	1.15	1.14	1.20	1.60	1.64	1.55	1.29
2008	2.02	2.30	2.20	2.02	1.44	1.14	1.33	1.40	1.85			

TABLE 3 - WILLIMANTIC RIVER WASTEWATER FROM UCONN AND/OR MANSFIELD TRAINING SCHOOL SYSTEM (GAL) - BOLD VALUES ARE KNOWN VALUES

Year	January	Gal pcd	February	Gal pcd	March	Gal pcd	April	Gal pcd	May	Gal pcd	June	Gal pcd	July	Gal pcd	August	Gal pcd	September	Gal pcd	October	Gal pcd	November	Gal pcd	December	Gal pcd
1958	22,656,338	53.3	23,919,213	62.3	22,401,294	52.7	26,450,448	64.3	16,577,808	39.0	15,467,136	37.6	14,962,534	35.2	15,770,171	37.1	20,814,816	50.6	22,316,280	52.5	20,855,952	50.7	18,023,053	42.4
1959	23,421,353	53.3	24,726,870	62.3	23,157,698	52.7	27,343,575	64.3	17,137,575	39.0	15,989,400	37.6	15,467,760	35.2	16,302,668	37.1	21,517,650	50.6	23,069,813	52.5	21,560,175	50.7	18,631,620	42.4
1960	24,186,367	53.3	26,446,475	62.3	23,914,101	52.7	28,236,702	64.3	17,697,342	39.0	16,511,664	37.6	15,972,986	35.2	16,835,164	37.1	22,220,484	50.6	23,823,345	52.5	22,264,398	50.7	19,240,187	42.4
1961	25,071,339	53.3	26,468,828	62.3	24,789,110	52.7	29,269,874	64.3	18,344,882	39.0	17,115,821	37.6	16,557,432	35.2	17,451,157	37.1	23,033,525	50.6	24,695,034	52.5	23,079,046	50.7	19,944,180	42.4
1962	25,956,311	53.3	27,403,128	62.3	25,664,120	52.7	30,303,047	64.3	18,992,423	39.0	17,719,978	37.6	17,141,879	35.2	18,067,151	37.1	23,846,566	50.6	25,566,723	52.5	23,893,693	50.7	20,648,172	42.4
1963	26,841,283	53.3	28,337,429	62.3	26,539,130	52.7	31,336,219	64.3	19,639,963	39.0	18,324,134	37.6	17,726,326	35.2	18,683,144	37.1	24,659,606	50.6	26,438,412	52.5	24,708,341	50.7	21,352,165	42.4
1964	27,726,255	53.3	30,317,149	62.3	27,414,139	52.7	32,369,392	64.3	20,287,504	39.0	18,928,291	37.6	18,310,772	35.2	19,299,138	37.1	25,472,647	50.6	27,310,101	52.5	25,522,988	50.7	22,056,158	42.4
1965	28,611,227	53.3	30,206,030	62.3	28,289,149	52.7	33,402,564	64.3	20,935,044	39.0	19,532,448	37.6	18,895,219	35.2	19,915,132	37.1	26,285,688	50.6	28,181,790	52.5	26,337,636	50.7	22,760,150	42.4
1966	29,496,199	53.3	31,140,331	62.3	29,164,159	52.7	34,435,736	64.3	21,582,584	39.0	20,136,605	37.6	19,479,666	35.2	20,531,125	37.1	27,098,729	50.6	29,053,479	52.5	27,152,284	50.7	23,464,143	42.4
1967	30,381,171	53.3	32,074,632	62.3	30,039,169	52.7	35,468,909	64.3	22,230,125	39.0	20,740,762	37.6	20,064,113	35.2	21,147,119	37.1	27,911,770	50.6	29,925,168	52.5	27,966,931	50.7	24,168,136	42.4
1968	31,266,142	53.3	34,187,823	62.3	30,914,178	52.7	36,502,081	64.3	22,877,665	39.0	21,344,918	37.6	20,648,559	35.2	21,763,112	37.1	28,724,810	50.6	30,796,857	52.5	28,781,579	50.7	24,872,128	42.4
1969	32,151,114	53.3	33,943,233	62.3	31,789,188	52.7	37,535,254	64.3	23,525,206	39.0	21,949,075	37.6	21,233,006	35.2	22,379,106	37.1	29,537,851	50.6	31,668,546	52.5	29,596,226	50.7	25,576,121	42.4
1970	33,036,086	53.3	34,877,534	62.3	32,664,198	52.7	38,568,426	64.3	24,172,746	39.0	22,553,232	37.6	21,817,453	35.2	22,995,099	37.1	30,350,892	50.6	32,540,235	52.5	30,410,874	50.7	26,280,114	42.4
1971	33,141,833	53.3	34,989,175	62.3	32,768,755	52.7	38,691,882	64.3	24,250,122	39.0	22,625,424	37.6	21,887,290	35.2	23,068,706	37.1	30,448,044	50.6	32,644,395	52.5	30,508,218	50.7	26,364,235	42.4
1972	33,247,581	53.3	36,354,417	62.3	32,873,311	52.7	38,815,338	64.3	24,327,498	39.0	22,697,616	37.6	21,957,126	35.2	23,142,312	37.1	30,545,196	50.6	32,748,555	52.5	30,605,562	50.7	26,448,357	42.4
1973	33,353,328	53.3	35,212,458	62.3	32,977,868	52.7	38,938,794	64.3	24,404,874	39.0	22,769,808	37.6	22,026,963	35.2	23,215,919	37.1	30,642,348	50.6	32,852,715	52.5	30,702,906	50.7	26,532,478	42.4
1974	33,459,075	53.3	35,324,100	62.3	33,082,425	52.7	39,062,250	64.3	24,482,250	39.0	22,842,000	37.6	22,096,800	35.2	23,289,525	37.1	30,739,500	50.6	32,956,875	52.5	30,800,250	50.7	26,616,600	42.4
1975	33,564,822	53.3	35,435,742	62.3	33,186,982	52.7	39,185,706	64.3	24,559,626	39.0	22,914,192	37.6	22,166,637	35.2	23,363,131	37.1	30,836,652	50.6	33,061,035	52.5	30,897,594	50.7	26,700,722	42.4
1976	33,670,569	53.3	36,816,933	62.3	33,291,539	52.7	39,309,162	64.3	24,637,002	39.0	22,986,384	37.6	22,236,474	35.2	23,436,738	37.1	30,933,804	50.6	33,165,195	52.5	30,994,938	50.7	26,784,843	42.4
1977	33,776,317	53.3	35,659,025	62.3	33,396,095	52.7	39,432,618	64.3	24,714,378	39.0	23,058,576	37.6	22,306,310	35.2	23,510,344	37.1	31,030,956	50.6	33,269,355	52.5	31,092,282	50.7	26,868,965	42.4
1978	33,882,064	53.3	35,770,666	62.3	33,500,652	52.7	39,556,074	64.3	24,791,754	39.0	23,130,768	37.6	22,376,147	35.2	23,583,951	37.1	31,128,108	50.6	33,373,515	52.5	31,189,626	50.7	26,953,086	42.4
1979	33,987,811	53.3	35,882,308	62.3	33,605,209	52.7	39,679,530	64.3	24,869,130	39.0	23,202,960	37.6	22,445,984	35.2	23,657,557	37.1	31,225,260	50.6	33,477,675	52.5	31,286,970	50.7	27,037,208	42.4
1980	34,093,558	53.3	37,279,448	62.3	33,709,766	52.7	39,802,986	64.3	24,946,506	39.0	23,275,152	37.6	22,515,821	35.2	23,731,163	37.1	31,322,412	50.6	33,581,835	52.5	31,384,314	50.7	27,121,330	42.4
1981	34,171,051	53.3	36,075,762	62.3	33,786,386	52.7	39,893,456	64.3	25,003,208	39.0	23,328,055	37.6	22,566,998	35.2	23,785,103	37.1	31,393,606	50.6	33,658,165	52.5	31,455,649	50.7	27,182,975	42.4
1982	34,248,544	53.3	36,157,574	62.3	33,863,007	52.7	39,983,926	64.3	25,059,910	39.0	23,380,958	37.6	22,618,175	35.2	23,839,043	37.1	31,464,800	50.6	33,734,495	52.5	31,526,984	50.7	27,244,620	42.4
1983	34,326,037	53.3	36,239,387	62.3	33,939,627	52.7	40,074,396	64.3	25,116,612	39.0	23,433,862	37.6	22,669,353	35.2	23,892,982	37.1	31,535,995	50.6	33,810,824	52.5	31,598,319	50.7	27,306,266	42.4
1984	34,403,530	53.3	37,618,385	62.3	34,016,248	52.7	40,164,866	64.3	25,173,314	39.0	23,486,765	37.6	22,720,530	35.2	23,946,922	37.1	31,607,189	50.6	33,887,154	52.5	31,669,654	50.7	27,367,911	42.4
1985	34,481,023	53.3	36,403,011	62.3	34,092,868	52.7	40,255,337	64.3	25,230,017	39.0	23,539,668	37.6	22,771,707	35.2	24,000,862	37.1	31,678,383	50.6	33,963,484	52.5	31,740,989	50.7	27,429,556	42.4
1986	34,558,515	53.3	36,484,824	62.3	34,169,489	52.7	40,345,807	64.3	25,286,719	39.0	23,592,571	37.6	22,822,884	35.2	24,054,802	37.1	31,749,577	50.6	34,039,814	52.5	31,812,323	50.7	27,491,202	42.4
1987	34,636,008	53.3	36,566,636	62.3	34,246,110	52.7	40,436,277	64.3	25,343,421	39.0	23,645,474	37.6	22,874,062	35.2	24,108,741	37.1	31,820,771	50.6	34,116,143	52.5	31,883,658	50.7	27,552,847	42.4
1988	34,713,501	53.3	37,957,322	62.3	34,322,730	52.7	40,526,747	64.3	25,400,123	39.0	23,698,378	37.6	22,925,239	35.2	24,162,681	37.1	31,891,966	50.6	34,192,473	52.5	31,954,993	50.7	27,614,492	42.4
1989	34,790,994	53.3	36,730,261	62.3	34,399,351	52.7	40,617,217	64.3	25,456,825	39.0	23,751,281	37.6	22,976,416	35.2	24,216,621	37.1	31,963,160	50.6	34,268,803	52.5	32,026,328	50.7	27,676,138	42.4
1990	34,868,487	53.3	36,812,073	62.3	34,475,971	52.7	40,707,687	64.3	25,513,527	39.0	23,804,184	37.6	23,027,594	35.2	24,270,560	37.1	32,034,354	50.6	34,345,133	52.5	32,097,663	50.7	27,737,783	42.4
1991	34,805,204	53.3	36,745,263	62.3	34,413,400	52.7	40,633,806	64.3	25,467,222	39.0	23,760,982	37.6	22,985,801	35.2	24,226,511	37.1	31,976,215	50.6	34,282,799	52.5	32,039,409	50.7	27,687,442	42.4
1992	34,741,921	53.3	37,988,397	62.3	34,350,830	52.7	40,559,926	64.3	25,420,918	39.0	23,717,779	37.6	22,944,008	35.2	24,182,463	37.1	31,918,075	50.6	34,220,466	52.5	31,981,154	50.7	27,637,100	42.4
1993	34,678,638	53.3	36,611,642	62.3	34,288,259	52.7	40,486,045	64.3	25,374,613	39.0	23,674,577	37.6	22,902,215	35.2	24,138,414	37.1	31,859,936	50.6	34,158,133	52.5	31,922,900	50.7	27,586,759	42.4
1994	34,615,355	53.3	36,544,831	62.3	34,225,688	52.7	40,412,164	64.3	25,328,308	39.0	23,631,374	37.6	22,860,422	35.2	24,094,365	37.1	31,801,796	50.6	34,095,800	52.5	31,864,646	50.7	27,536,417	42.4
1995	34,552,071	53.3	36,478,021	62.3	34,163,118	52.7	40,338,284	64.3	25,282,004	39.0	23,588,172	37.6	22,818,629	35.2	24,050,316	37.1	31,743,657	50.6	34,033,466	52.5	31,806,392	50.7	27,486,076	42.4
1996	34,488,788	53.3	37,711,610	62.3	34,100,547	52.7	40,264,403	64.3	25,235,699	39.0	23,544,970	37.6	22,776,836	35.2	24,006,267	37.1	31,685,518	50.6	33,971,133	52.5	31,748,137	50.7	27,435,734	42.4
1997	34,425,505	53.3	36,344,400	62.3	34,037,976	52.7	40,190,522	64.3	25,189,394	39.0	23,501,767	37.6	22,735,043	35.2	23,962,218	37.1	31,627,378	50.6	33,908,800	52.5	31,689,883	50.7	27,385,393	42.4
1998	34,362,222	53.3	36,277,589	62.3	33,975,405	52.7	40,116,641	64.3	25,143,089	39.0	23,458,565	37.6	22,693,250	35.2	23,918,170	37.1	31,569,239	50.6	33,846,467	52.5	31,631,629	50.7	27,335,051	42.4
1999	34,298,939	53.3	36,210,779	62.3	33,912,835	52.7	40,042,761	64.3	25,096,785	39.0	23,415,362	37.6	22,651,457	35.2	23,874,121	37.1	31,511,099	50.6	33,784,133	52.5	31,573,374	50.7	27,284,710	42.4
2000	34,235,656	53.3	37,434,824	62.3	33,850,264	52.7	39,968,880	64.3	25,050,480	39.0	23,372,160	37.6	22,609,664	35.2	23,830,072	37.1	31,452,960	50.6	33,721,800	52.5	31,515,120	50.7	27,234,368	42.4
2001	35,008,519	53.3	36,959,911	62.3	34,614,427	52.7	40,871,170	64.3	25,615,990	39.0	23,899,782	37.6	23,120,073	35.2	24,368,031	37.1	32,163,005	50.6	34,483,063	52.5	32,226,568	50.7	27,849,179	42.4
2002	35,781,383	53.3	37,775,854	62.3	35,378,590	52.7	41,773,460	64.3	26,181,500	39.0	24,427,404	37.6	23,630,482	35.2	24,905,991	37.1	32,873,049	50.6	35,244,326	52.5	32,938,016	50.7	28,463,989	42.4
2003	36,554,246	53.3	38,591,797	62.3	36,142,754	52.7	42,675,749	64.3	26,747,009	39.0	24,955,026	37.6	24,140,890	35.2	25,443,950	37.1	33,583,094	50.6	36,005,589	52.5	33,649,463	50.7	29,078,800	42.4
2004	37,327,109	53.3	40,815,160	62.3	36,906,917	52.7	43,578,039	64.3	27,312,519	39.0	25,482,648	37.6	24,651,299	35.2	25,981,909	37.1	34,293,138	50.6	36,766,853	52.5	34,360,911	50.7	29,693,610	42.4
2005	38,099,973	53.3	40,223,684	62.3	37,671,080	52.7	44,480,329	64.3	27,878,029	39.0	**19,991,000**	35.9	**25,599,000**	35.8	26,519,868	37.1	**38,228,000**	55.3	**43,573,000**	61.0	**37,155,000**	53.7	**36,236,000**	50.7
2006	**44,170,000**	60.6	**48,019,000**	72.9	**34,324,000**	47.1	**42,987,000**	60.9	**31,563,000**	43.3	**36,207,000**	51.3	**28,497,000**	39.1	**30,560,000**	41.9	**36,801,000**	52.1	**38,145,000**	52.3	**39,003,000**	55.3	**30,411,000**	41.7
2007	**34,249,000**	46.0	**32,980,000**	49.1	39,199,406	52.7	**55,289,000**	76.8	**26,496,000**	35.6	**23,138,000**	32.1	**22,771,000**	30.6	**23,983,000**	32.2	**31,990,000**	44.4	**32,873,000**	44.2	**31,061,000**	43.2	**25,797,000**	34.7
2008	40,418,563	53.3	**46,141,000**	65.0	**44,175,000**	58.3	**40,399,000**	55.1	**28,928,000**	38.1	**22,837,000**	31.1	26,692,934	35.2	28,133,746	37.1	37,133,316	50.6						
Average of Knowns		53.3		62.3		52.7		64.3		39.0		37.6		35.2		37.1		50.6		52.5		50.7		42.4

TABLE 5 - COMPARISON OF NATURAL DATASET VS. MANSFIELD DEPOT GAUGE DATA

Date	Original USGS at Coventry Discharge, cfs	"Natural" wellfield dataset No watershed ratio < 13 cfs Discharge, cfs	"Natural" wellfield dataset Watershed ratio always applied Discharge, cfs	Original USGS Mansfield Depot Discharge, cfs	Mansfield Depot Gauge Adjusted for 50% of withdrawal Discharge, cfs	Comparison Rank
8/31/1999	**7.5**	7.4	6.0	*10*	*10.5*	Poor
9/5/2007	**7.5**	8.2	6.7	**5.9**	7.1	Excellent
9/6/2007	**7.8**	8.5	6.9	**5.8**	7.0	Excellent
9/3/2007	**7.9**	8.6	7.0	**5.9**	7.1	Excellent
9/2/2007	**8**	8.7	7.1	**6.6**	7.8	Good
8/30/1999	**8.2**	8.1	6.5	*10*	*10.5*	Poor
9/7/2007	**8.3**	9.0	7.3	**6.2**	7.4	Excellent
9/4/2007	**8.4**	9.1	7.4	**5.8**	7.0	Excellent
8/8/1999	**8.6**	8.5	6.9	*13*	*13.5*	Poor
9/8/2007	**8.6**	9.3	7.6	**6.8**	8.0	Excellent
9/9/2007	**8.7**	9.4	7.6	**6.4**	7.6	Excellent
8/7/1999	**8.8**	8.7	7.0	*13*	*13.5*	Poor
9/10/2007	**8.9**	9.6	7.8	**6.4**	7.6	Excellent
9/11/1995	**9.1**	9.3	7.5	*8*	*8.9*	Excellent
8/10/1999	**9.1**	9.0	7.3	*15*	*15.5*	Poor
9/7/1995	**9.3**	9.5	7.7	*7.8*	*8.7*	Good
9/1/2007	**9.3**	10.0	8.1	**7.3**	8.5	Excellent
9/12/1995	**9.4**	9.6	7.8	*8.1*	*9.0*	Good
9/8/1995	**9.5**	9.7	7.9	*7.9*	*8.8*	Good
9/4/1995	**9.6**	9.8	7.9	*8.5*	*9.4*	Excellent
9/5/1995	**9.6**	9.8	7.9	*8*	*8.9*	Good
9/6/1995	**9.6**	9.8	7.9	*7.9*	*8.8*	Good
9/9/1995	**9.6**	9.8	7.9	*8.7*	*9.6*	Excellent
9/16/1995	**9.6**	9.8	7.9	*9.8*	*10.7*	Good
8/20/1999	**9.7**	9.6	7.7	*12*	*12.5*	Poor
8/29/1999	**9.8**	9.7	7.8	*12*	*12.5*	Poor
8/31/1995	**9.9**	9.3	7.6	*9.8*	*10.1*	Good
9/3/1995	**9.9**	10.1	8.2	*9.2*	*10.1*	Excellent
9/1/1995	**10**	10.2	8.3	*11*	*11.9*	Fair
9/2/1995	**10**	10.2	8.3	*10*	*10.9*	Good
9/10/1995	**10**	10.2	8.3	*8.7*	*9.6*	Good
9/13/1995	**10**	10.2	8.3	*8.5*	*9.4*	Good
8/9/1999	**10**	9.9	8.0	*14*	*14.5*	Poor
8/11/1999	**10**	9.9	8.0	*15*	*15.5*	Poor
8/25/1999	**10**	9.9	8.0	*12*	*12.5*	Poor
8/28/2007	**10**	10.8	8.8	**8.4**	9.4	Good
8/29/2007	**10**	10.8	8.8	**8.6**	9.6	Good
8/30/2007	**10**	10.8	8.8	**8.1**	9.1	Excellent
8/31/2007	**10**	10.8	8.8	**8.1**	9.1	Excellent
8/26/1995	**11**	10.4	8.5	*12*	*12.3*	Fair
8/29/1995	**11**	10.4	8.5	*11*	*11.3*	Good
8/30/1995	**11**	10.4	8.5	*12*	*12.3*	Fair
9/14/1995	**11**	11.2	9.1	*11*	*11.9*	Good
9/15/1995	**11**	11.2	9.1	*10*	*10.9*	Excellent
8/13/1999	**11**	10.9	8.8	*13*	*13.5*	Poor
8/21/1999	**11**	10.9	8.8	*12*	*12.5*	Fair
8/22/1999	**11**	10.9	8.8	*14*	*14.5*	Poor
8/26/1999	**11**	10.9	8.8	*12*	*12.5*	Fair
8/27/2007	**11**	11.8	9.6	**8.6**	9.6	Excellent
9/24/2007	**11**	11.7	9.5	**7.7**	8.9	Good
9/25/2007	**11**	11.7	9.5	**7.1**	8.3	Fair
8/28/1995	**12**	11.4	9.3	*12*	*12.3*	Good
8/6/1999	**12**	11.9	9.6	*13*	*13.5*	Fair
8/12/1999	**12**	11.9	9.6	*14*	*14.5*	Poor
8/19/1999	**12**	11.9	9.6	*14*	*14.5*	Poor
8/23/1999	**12**	11.9	9.6	*14*	*14.5*	Poor
8/24/1999	**12**	11.9	9.6	*13*	*13.5*	Fair
8/27/1999	**12**	11.9	9.6	*12*	*12.5*	Good
8/28/1999	**12**	11.9	9.6	*13*	*13.5*	Fair
8/19/2002	**12**	12.6	10.2	*11*	*11.9*	Good
9/25/2005	**12**	11.9	9.6	*11*	*11.9*	Excellent
8/25/2007	**12**	12.8	10.4	**9.7**	10.7	Excellent
8/26/2007	**12**	12.8	10.4	**9.4**	10.4	Excellent
9/18/2007	**12**	12.7	10.3	**9.4**	10.6	Excellent
9/19/2007	**12**	12.7	10.3	**9.1**	10.3	Excellent
9/20/2007	**12**	12.7	10.3	**9.1**	10.3	Excellent
9/21/2007	**12**	12.7	10.3	**9.4**	10.6	Excellent
9/22/2007	**12**	12.7	10.3	**9.4**	10.6	Excellent
9/23/2007	**12**	12.7	10.3	**8.6**	9.8	Excellent
9/26/2007	**12**	12.7	10.3	**7.6**	8.8	Fair

Approved USGS data is in **bold,** Provisional USGS data is in *italics.* Approved data range is 10/1/2005 through 9/30/2008.
Comparison ranks are rated as Excellent (< 0.5 cfs difference), Good (0.5 to 1 cfs difference), Fair (1 to 2 cfs difference), and Poor (> 2 cfs difference).
Provisional data comparisons are in tan and approved data comparisons are in turquoise.

APPENDIX C
UCONN DRAFT DROUGHT RESPONSE PLAN (AUGUST 2008)

DROUGHT RESPONSE PLAN

DRAFT 08-22-08

UCONN
Water Supply Emergency Contingency Plan

1. TRIGGER LEVELS:

Stage IA – Water Conservation Alert:

- Projected Available Supply[1] is forecast to be greater than or equal to the Projected Water Usage[2] for an extended period yet flow in the Fenton River is at 3.0 cfs or less.

Stage IB – Water Supply / Drought Advisory:

- Projected Available Supply[1] is forecast to be equal to or less than Projected Water Usage[2], or
- Continuous pumping at maximum available supply results in an overall decrease in tank storage, as expressed by water levels in the High Head Reservoir.

Stage II – Water Supply / Drought Watch:

- Projected Available Supply[1] is forecast to be significantly less than Projected Water Usage[2] for an extended period, or
- Three consecutive days of continuous pumping at maximum available supply results in an overall decrease in tank storage, as expressed by water levels in the High Head Reservoir.

Stage III – Water Supply / Drought Warning:

- If the High Head Reservoir fails to recover to two-thirds full (10' level) for three consecutive days.

Stage IV – Water Supply / Drought Emergency:

- If the High Head Reservoir fails to recover to 40% full (6' level) for four consecutive days.

---

[1] Projected Available Supply is the expected capacity of the system's sources operating concurrently, and adjusting for any losses due to well maintenance or repair; transmission or pumping limitations due to depressed groundwater levels at the Willimantic wells; anticipated reductions in Fenton well withdrawal based on flow recession equations developed in the Study Report; or other supply-reducing events.

[2] Projected Water Usage is the expected production for the particular time of year for which the assessment is made, and includes any reductions or increases in demand due to historical variation or known significant changes.

2. RESPONSE:

Stage IA – Water Conservation Alert:

- Implement Demand-Side Water Conservation Plan for voluntary conservation.
- Contact the Departments of Public Health and Environmental Protection and other state and local agencies, as outlined in the plan, concerning the initiation of an Alert.
- Maintain compliance with Fenton River Study flow management recommendations, including cessation of Fenton Well Field withdrawals when flow is less than 3 cfs, as measured at USGS gaging station 01121330.
- Evaluate the operative status of system components and availability of supply.
- Monitor daily production, storage and consumption to quantify any demand reductions.

Stage IB – Water Supply / Drought Advisory:

- Re-issue Demand-Side Water Conservation Plan for voluntary conservation.
- Contact the Departments of Public Health and Environmental Protection and other state and local agencies, as outlined in the plan, concerning the initiation of an Advisory.
- Maintain compliance with Fenton River Study flow management recommendations, including phased scaling back of Fenton Well Field withdrawals when flow is less than 6 cfs, as measured at USGS gaging station 01121330.
- Investigate any material deviation from normal consumption, production or storage patterns.
- Evaluate the operative status of system components and availability of supply. Evaluate and identify operating adjustments, emergency equipment, or other actions necessary to temporarily increase available supply.
- Contact DPH and DEP regarding the possible activation of Fenton Well D and/or issuance of temporary or emergency authorization allowing rebalancing of registered diversion rates to allow increased withdrawals from Willimantic Wells 1 and/or 3.
- Review Water Supply Plan Emergency Contingency Plan and update if necessary.
- Monitor daily on-campus, metered consumption, storage and metered production to ensure consumption and production reductions are met (10% from previous non-advisory average).
- Ensure all operating adjustments are made to increase available supply, with the exception of activating Fenton wells that are off-line or restricted due to low-flow conditions.

Stage II – Water Supply / Drought Watch:

- Re-issue Demand-Side Water Conservation request for voluntary conservation.
- Issue Demand-Side Water Conservation notice for water use restrictions.
- Contact the Departments of Public Health and Environmental Protection and other state and local agencies, as outlined in the plan, concerning the initiation of a Watch.
- Maintain compliance with Fenton River Study flow management recommendations, including phased scaling back of Fenton Well Field withdrawals when flow is less than 6 cfs, as measured at USGS gaging station 01121330.

Stage II – Water Supply / Drought Watch (continued):

- Continue investigation of any material deviation from normal production, consumption and storage patterns.
- Evaluate the operative status of system components and availability of supply. As required, schedule necessary in-house emergency equipment; order additional equipment or services from outside vendors following University purchasing procedures.
- Contact DPH/DEP regarding activation of Fenton Well D in accordance with recommended abbreviated pumping plan and/or issuance of temporary or emergency authorization allowing rebalancing of registered diversion rates to allow increased withdrawals from Willimantic Wells 1 and/or 3.
- Review Mandatory Conservation measures and update if necessary.
- Monitor daily on-campus, metered consumption and metered production to ensure consumption and production reductions are met (15% from previous non-advisory average).

Stage III – Water Supply / Drought Warning:

- Re-issue Demand-Side Water Conservation request for voluntary conservation.
- Re-issue Demand-Side Water Conservation Plan for mandatory conservation.
- Contact the Department of Public Health and other state and local agencies, as outlined in the plan, concerning the initiation of a Warning.
- Evaluate the operative status of system components, availability of supply, and the effect of demand reduction measures taken to date. Evaluate and prioritize reactivation of any Fenton wells off-line or throttled due to flow-imposed limits, including Fenton Well D.
- Eliminate all unnecessary outdoor water usage and routinely monitor and enforce compliance with mandatory conservation measures.
- Activate Fenton Well D in accordance with recommended abbreviated pumping plan, if feasible.
- During increasing severity of stage, and upon notification and consultation with appropriate state agencies, initiate limited Willimantic Well 1 and 3 well use to maintain level in the High Head Reservoir.
- Review High Priority User list and update if necessary.
- Schedule necessary purchase of supplemental water, either bottled or by tanker, for critical areas.
- Monitor daily on-campus, metered consumption, metered storage, and metered production to ensure consumption and production reductions are met (20% from previous non-advisory average).

Stage IV – Water Supply / Drought Emergency:

- Re-issue Demand-Side Water Conservation request for voluntary conservation.
- Re-issue Demand-Side Water Conservation notice for water use restrictions.
- Contact the Department of Public Health and other state and local agencies, as outlined in the plan, concerning the initiation of an Emergency.

Stage IV – Water Supply / Drought Emergency (continued):

- Eliminate all outside water usage, and enforce all mandatory conservation restrictions, as necessary.
- Evaluate the operative status of all system components, availability of supply, and the effect of demand reduction measures taken to date. Make necessary operating adjustments to meet needs of high priority users.
- During increasing severity of stage, and upon notification and consultation with appropriate state agencies, increase production at the Fenton Well Field to maintain level in the High Head Reservoir.
- Order supplemental water supplies for high priority users.
- Monitor daily on-campus, metered consumption and metered production to ensure consumption and production reductions are met (25% from previous non-advisory average).

3. CONSERVATION MEASURES:

Demand-Side Water Conservation Plan – Voluntary Measures

Water Conservation Measures	Departmental Responsibilities

Water Conservation Alert; Water Supply Advisory; Water Supply Watch: <u>Voluntary Measures</u>
• Reduce use
  o shorter showers
  o condense washing of loads (dishes and laundry)
• Be more conscious of use
  o Not letting water run to warm up or cool down
  o Not letting faucets run while brushing teeth, shaving, etc
• Eliminate non-essential consumption of water (lawn watering, garden watering at night only, car washing).
• Raise air conditioning thermostats for centrally-chilled buildings to 75 degrees
• Report leaks immediately
  o Facilities Operations (6-3113)
	OEP – draft WC request for voluntary conservation measures
	
	Univ Relations – review and approve draft WC request
	
	VP/COO – issue WC request as UConn Announcement
	
	FacOps/NEWUS – issue WC request to off-campus users
	
	FacOps/NEWUS – respond to reported leaks as high priority repairs
	
	FacOps/NEWUS – report relevant water demand changes to UConn WC communications team (A&O, OEP, Univ Relations, Town of Mansfield)
	
	The following communications responsibilities applicable to each step in this Plan:
	
	A&O - notify DPH, DEP, DOC, Town of Mansfield, and town members of Water/Sewer Advisory Board
	
	Univ Relations – notify legislators and governor, as needed

Demand-Side Water Conservation Plan – Mandatory Measures

Water Supply Warning; Water Supply Emergency: Mandatory Water <u>Conservation Measures</u> • No routine maintenance flushing of hydrants, pipes and sewer lines • No fleet vehicle washing • 50% reduction in irrigation of athletic fields, landscaping and research facilities, unless separate irrigation ponds or wells are used • Curtail running of lasers, autoclaves and other research lab devices that consume water for cooling (once-through cooling) • No use of UC water for construction site dust control or rinsing activities • No use of water for street sweeping • No pool filling • Thermostats set at 78 degrees for centrally-cooled buildings	*VP/COO – Issue department-head directives applicable to UConn operations (COO direct reports and Athletics) *EVP/Provost - Issue directives applicable to academic/research activities (Deans and Directors) *FacOps/NEWUS – Issue directives applicable to non-University and off-campus water system users FacOps provide updated list of CUP and centrally-cooled buildings FacOps – report relevant water production and demand changes to UConn Water Team --- *When warranted, these parties are also responsible for notifying same water system users about repeal of mandatory measures.

4. RECOVERY FROM EMERGENCY:

The method of recovery from a water emergency will vary according to the stage and responsible trigger. In general, once the emergency condition is rectified, the emergency can be considered over and normal system operating conditions can resume. Several non-drought emergencies may not result in formal activation of the stage response plan due to the short-term duration of the emergency. Therefore, the recovery can be quite rapid, compared to recovery from a drought.

The steps to be taken to step down from longer term and drought related emergencies are as follows:

Stage IV – Water Supply / Drought Emergency:

When the water level in the High Head Reservoir is maintained above 6 feet, 40% full, for three consecutive days with an overall trend showing an increase in tank storage, and continued recovery can be sustained without use of the Fenton wells, well use may be curtailed as flow management recommendations dictate. When water level in the High Head Reservoir can be maintained above 10 feet, two-thirds full, with an overall trend showing an increase in tank storage, Stage III can be re-implemented and mandatory restrictions eased.

Stage III – Water Supply / Drought Warning:

When response measures have resulted in the water level in the High Head Reservoir being maintained above 10 feet, two-thirds full, for three consecutive days with an overall trend showing an increase in tank storage, and continued recovery can be sustained without use of the Fenton wells, well use may be curtailed as flow management recommendations so dictate. Production from all sources is to be reviewed and if projected available supply is greater than the projected water usage, Stage II can be re-implemented and mandatory restrictions further eased.

Stage II – Water Supply / Drought Watch:

When response measures have resulted in the water level in the High Head Reservoir returning to normal, and when Projected Available Supply is greater than the Projected Water Usage, Stage I can be re-implemented and voluntary conservation maintained.

Stage IB – Water Supply / Drought Advisory:

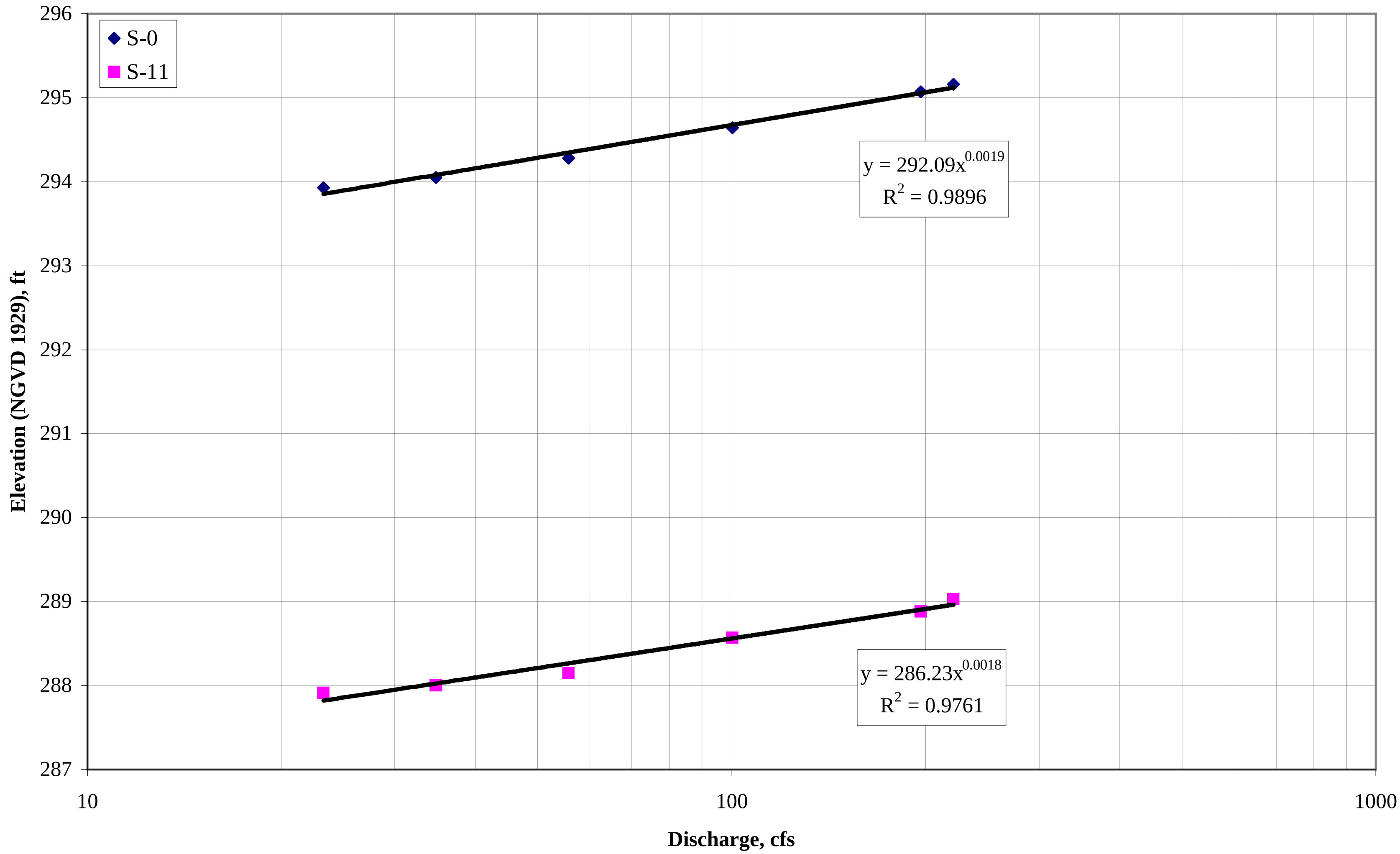
When response measures have resulted in the water level in the High Head Reservoir returning to normal for five consecutive days, and when Projected Available Supply is greater than the Projected Water Usage, the advisory can be lifted.

Stage IA – Water Conservation Alert:

When response measures have resulted in the water level in the High Head Reservoir returning to normal for five consecutive days, and when Projected Available Supply is greater than the Projected Water Usage, and when the flow in the Fenton River is greater than 3.0 cfs, the Alert can be lifted.

APPENDIX D
2008 AND 2009 MONITORING DATA

Staff Gauge Rating Curves - 2008 & 2009 Hydrogeologic Study
Willimantic River Wellfield
University of Connecticut



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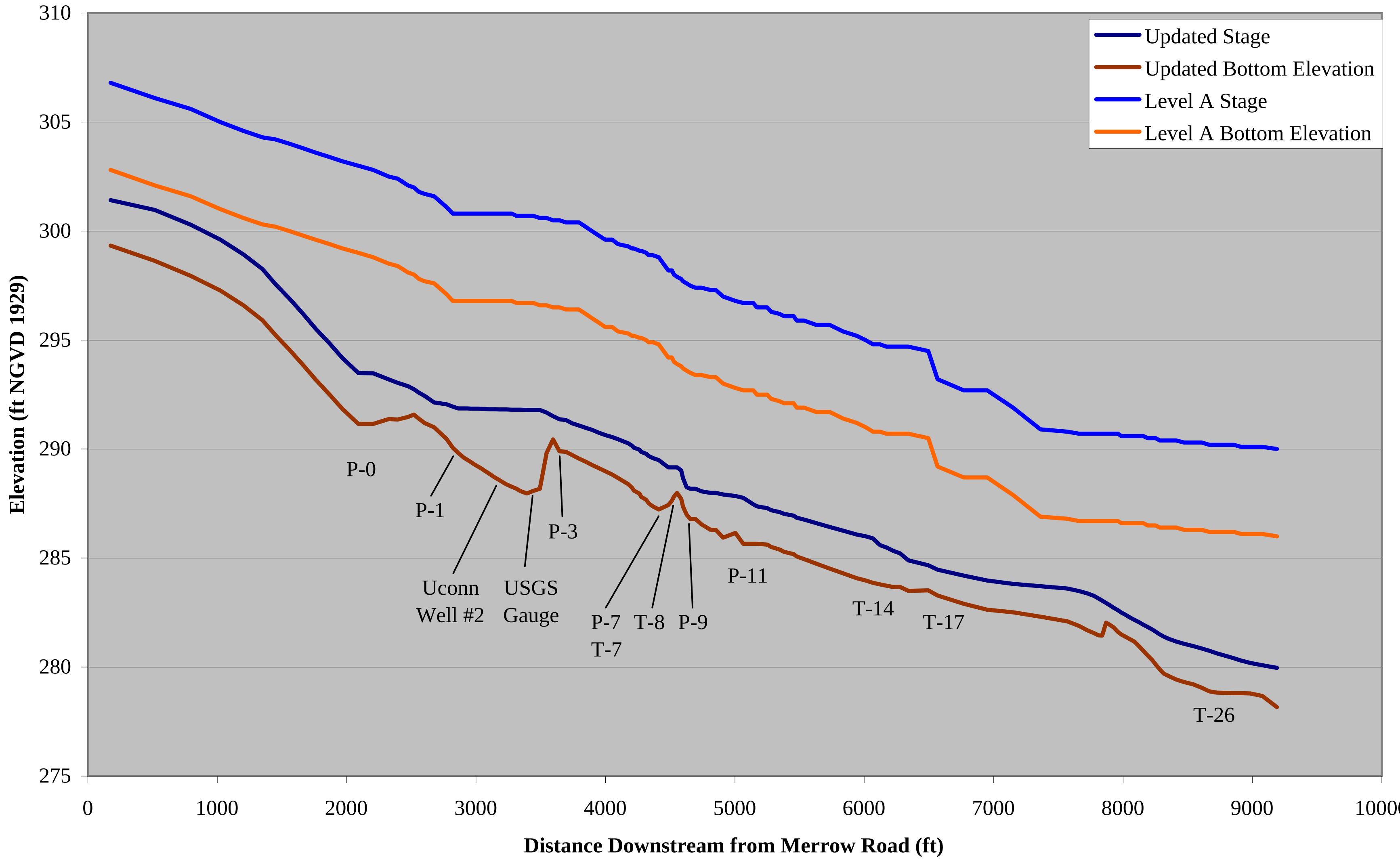
Comparison of Modeled River Elevations



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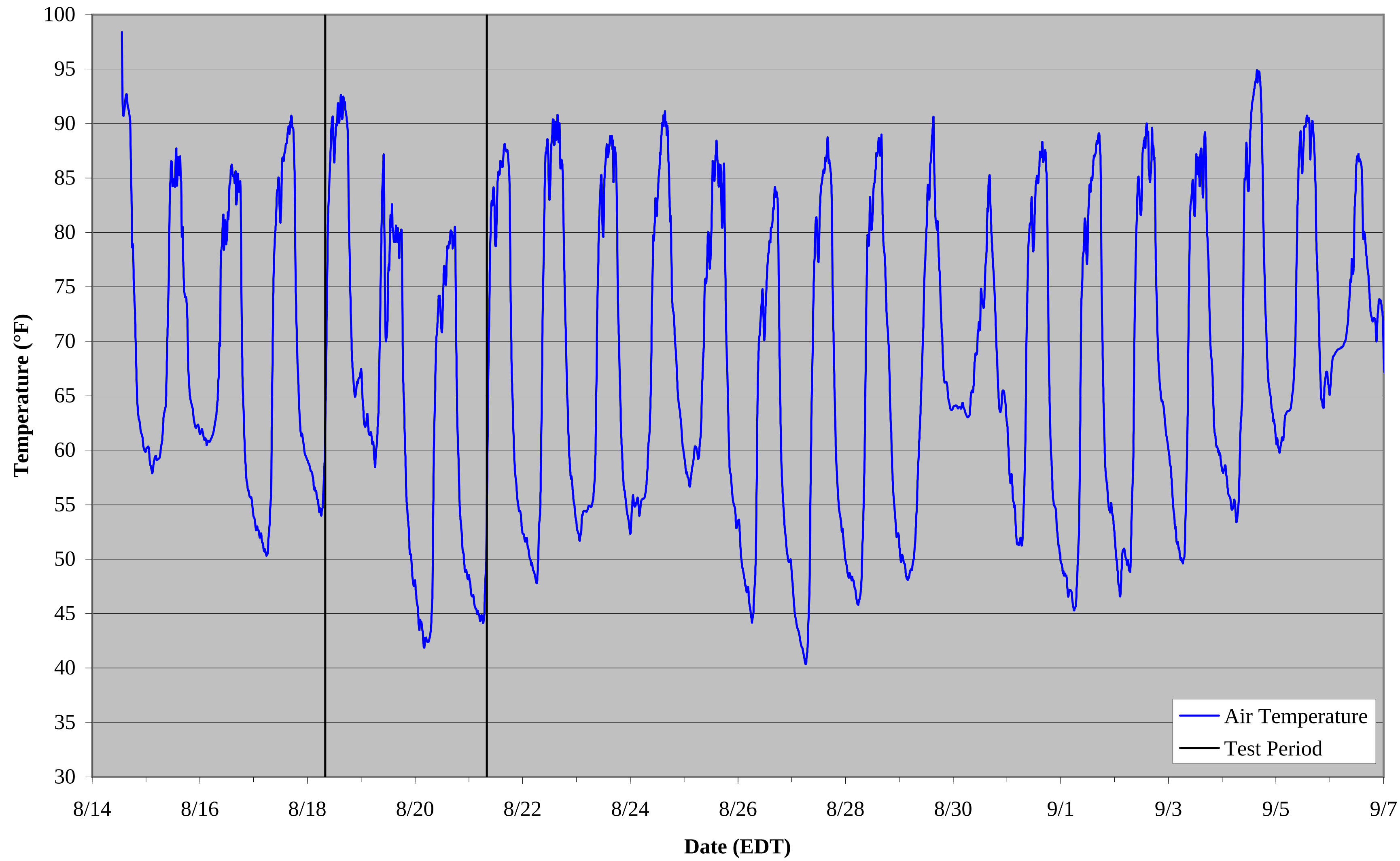
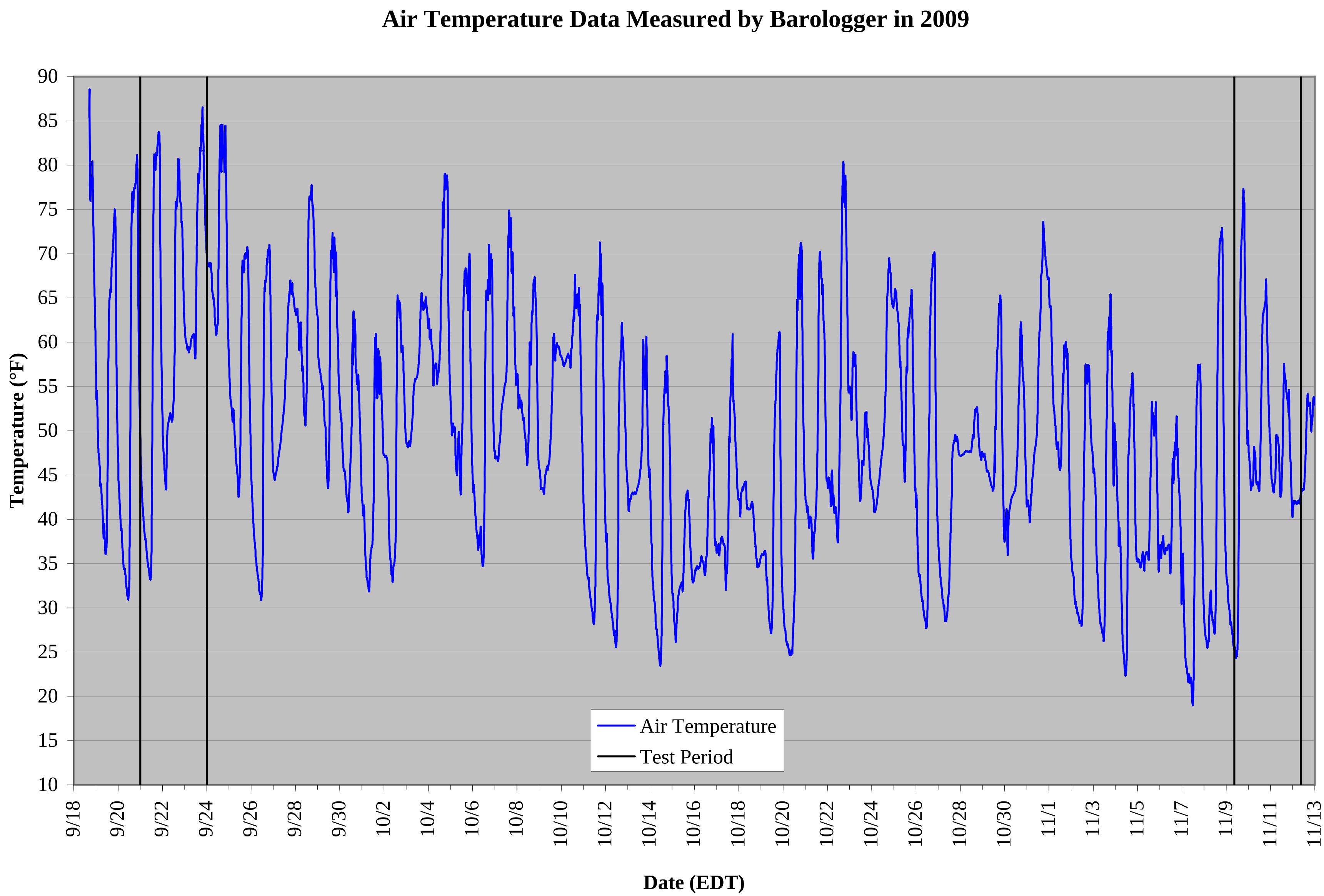
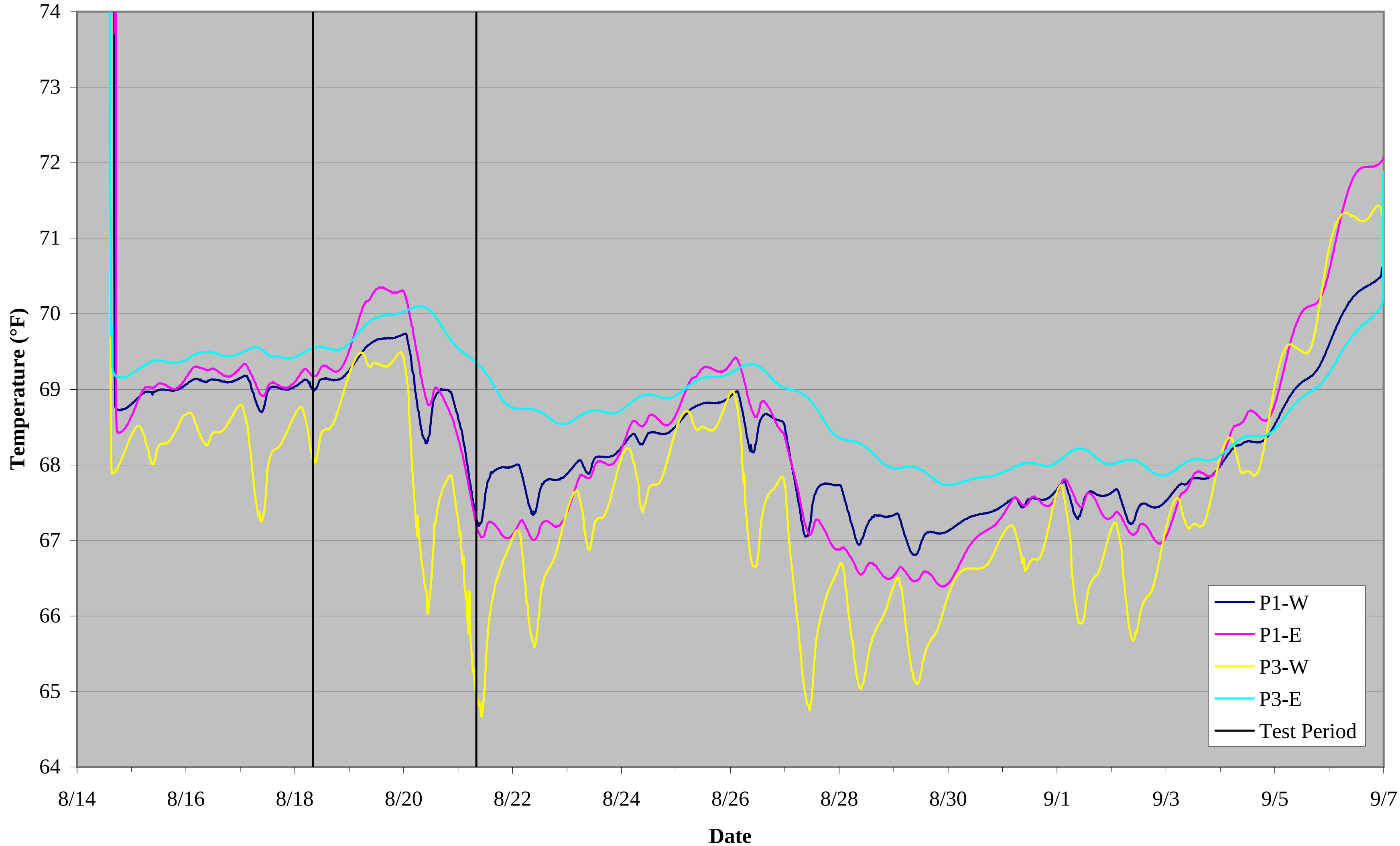
Air Temperature Data Measured by Barologger in 2008

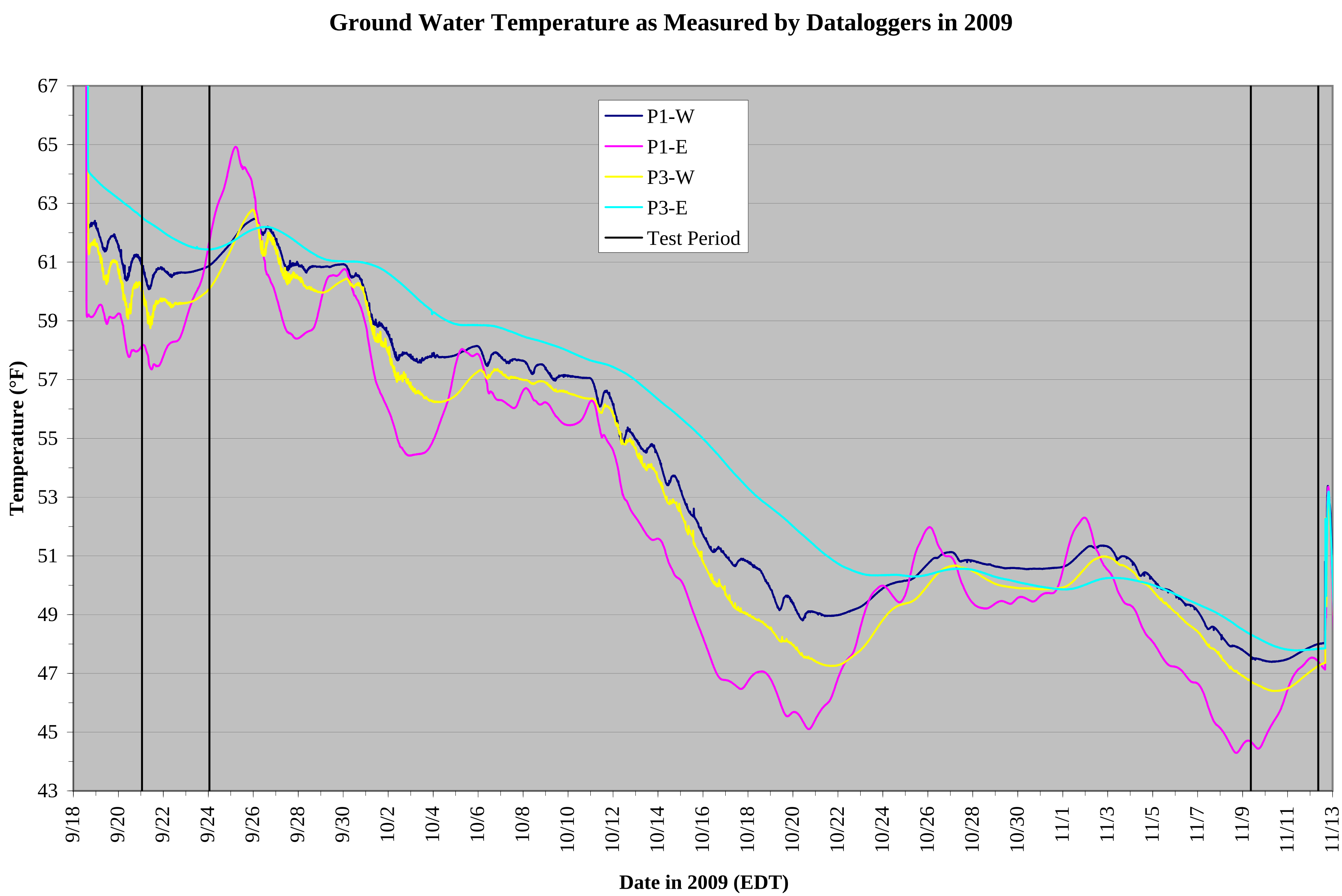


MILONE & MACBROOM®


Air Temperature Data Measured by Barologger in 2009
Temperature (°F)
90
85
80
75
70
65
60
55
50
45
40
35
30
25
20
15
10
9/18
9/20
9/22
9/24
9/26
9/28
9/30
10/2
10/4
10/6
10/8
10/10
10/12
10/14
10/16
10/18
10/20
10/22
10/24
10/26
10/28
10/30
11/1
11/3
11/5
11/7
11/9
11/11
11/13
Date (EDT)
Air Temperature
Test Period

Ground Water Temperature as Measured by Dataloggers in 2008





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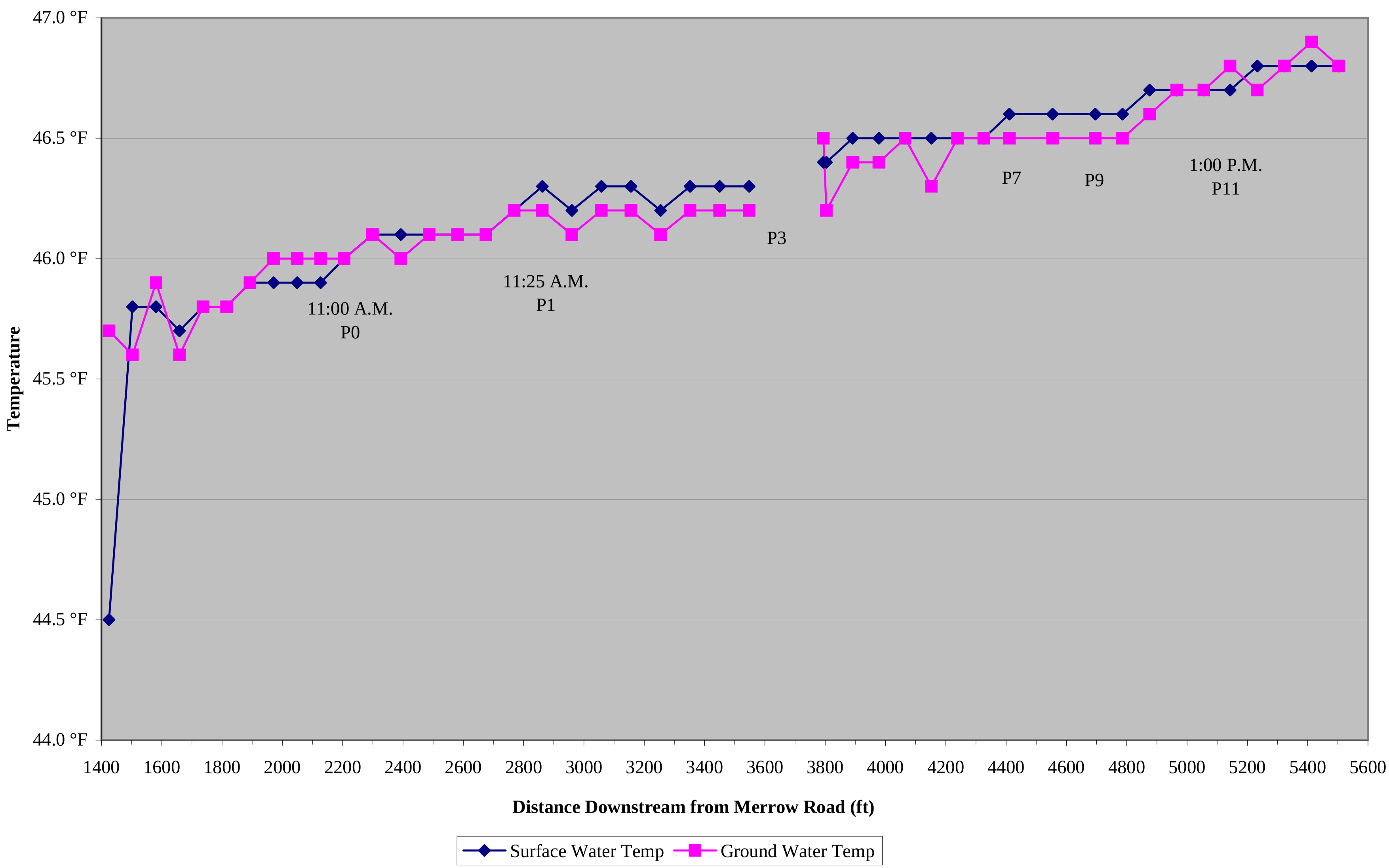
11/12/2009 Temperature Profile along Willimantic River Thalweg



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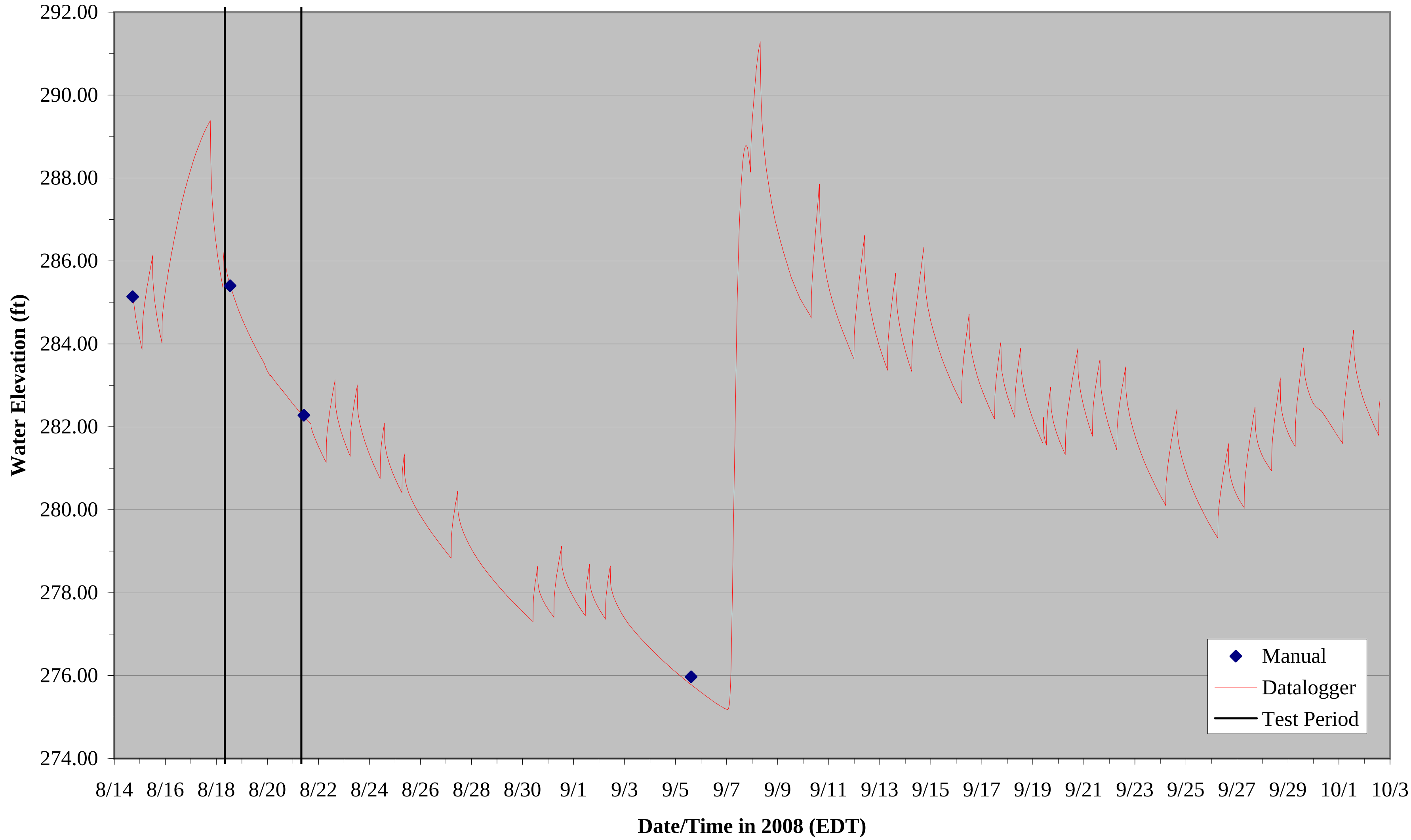
Ground Water Elevation at Ms-65 in 2008



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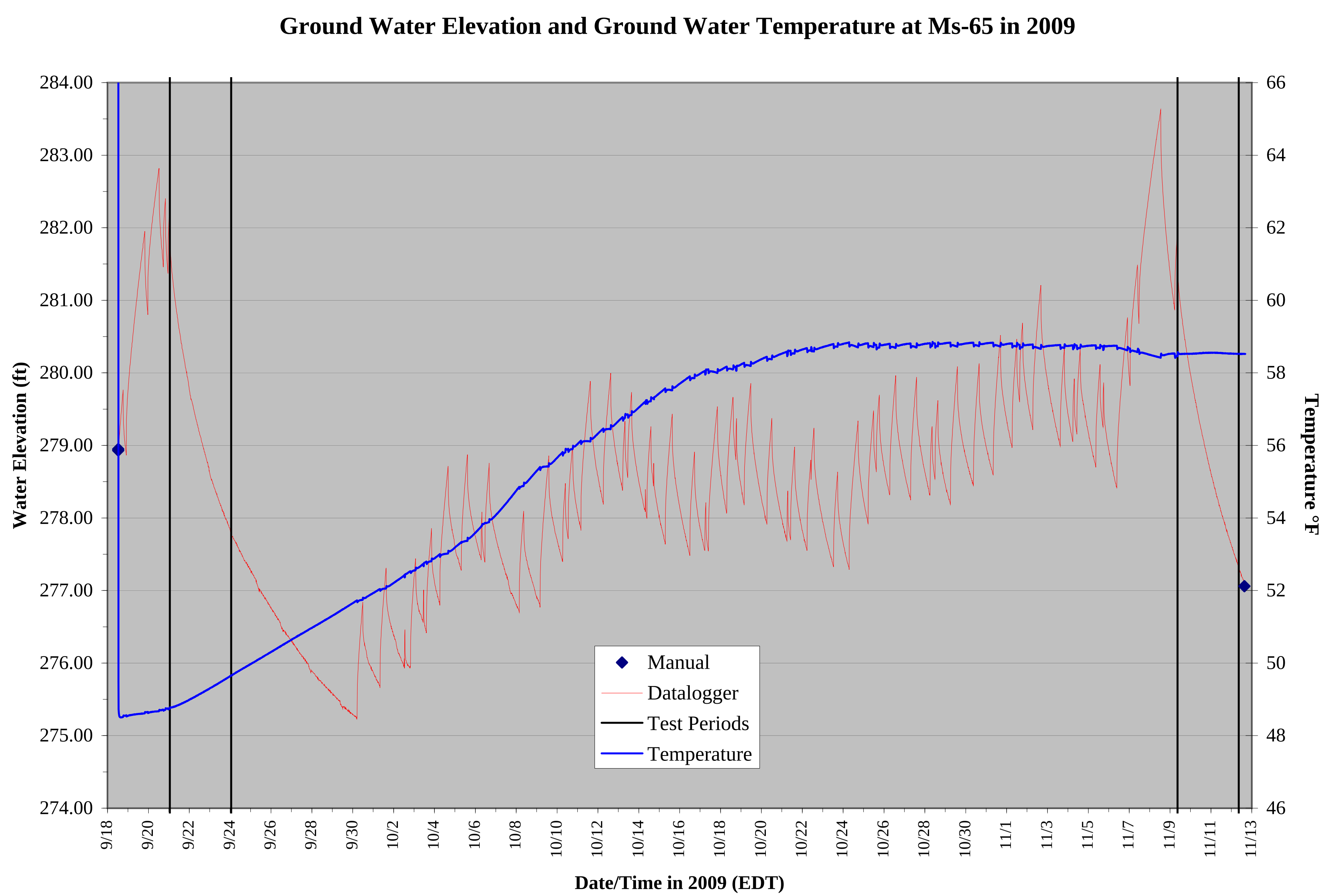
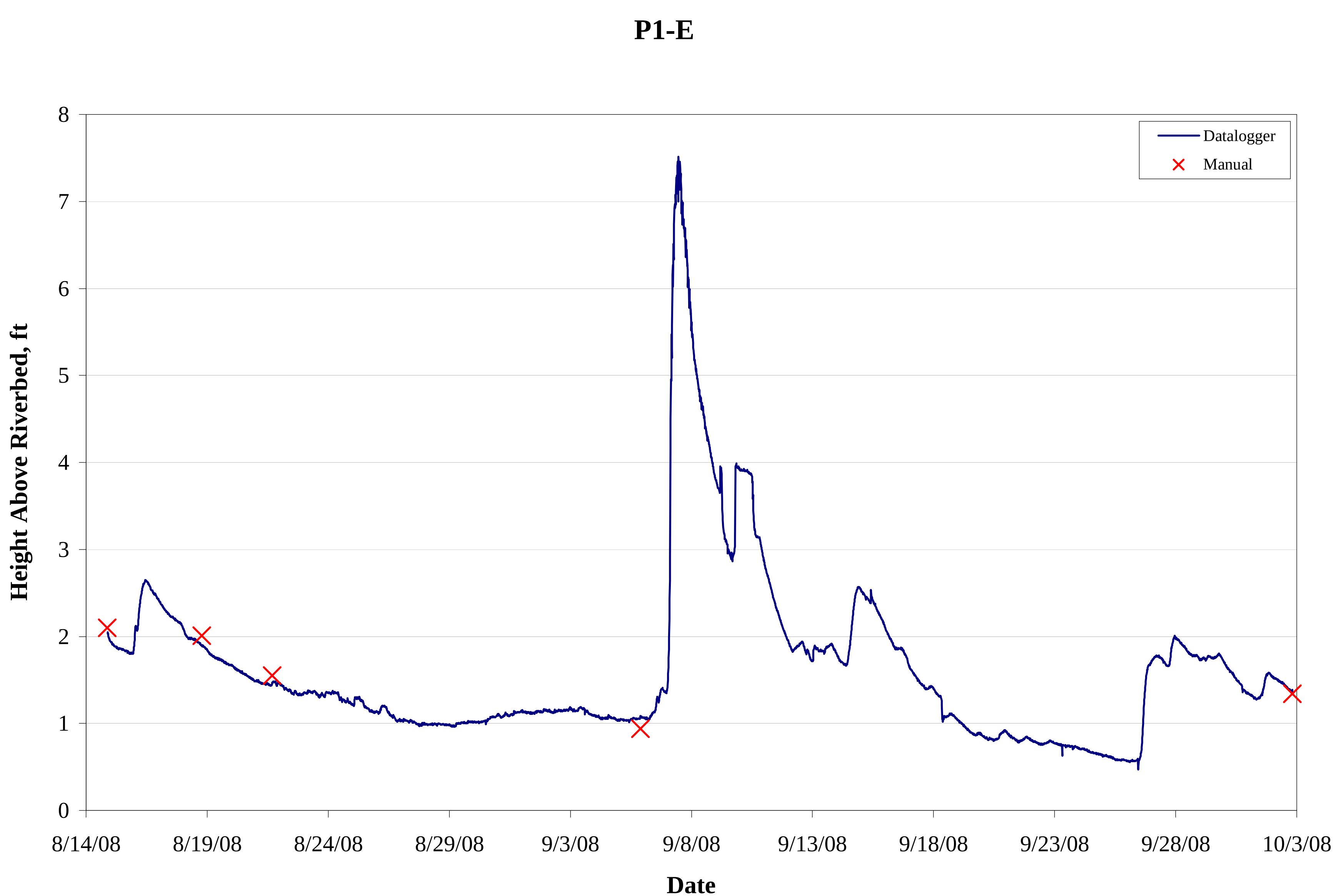


MILONE & MACBROOM®



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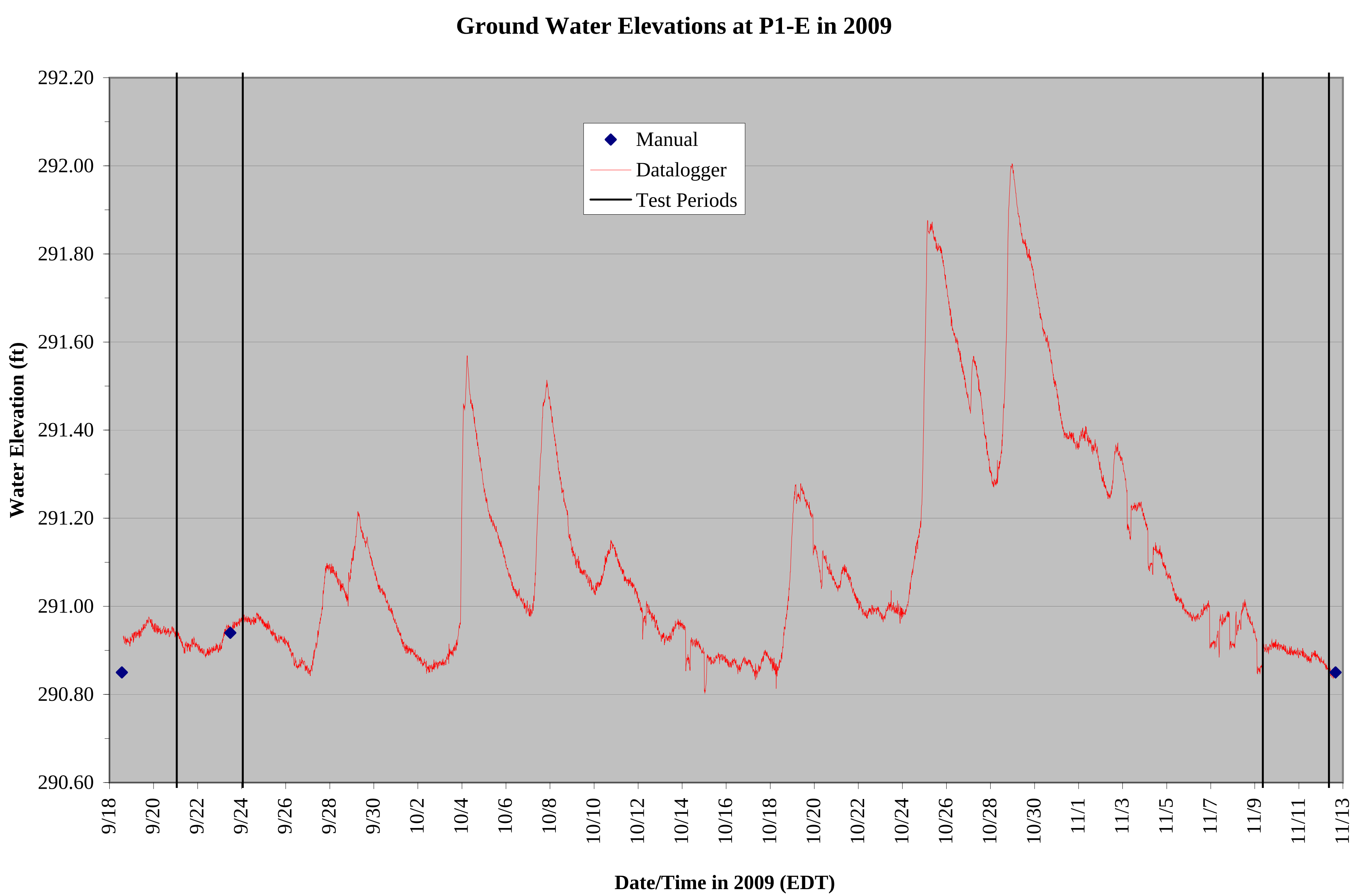

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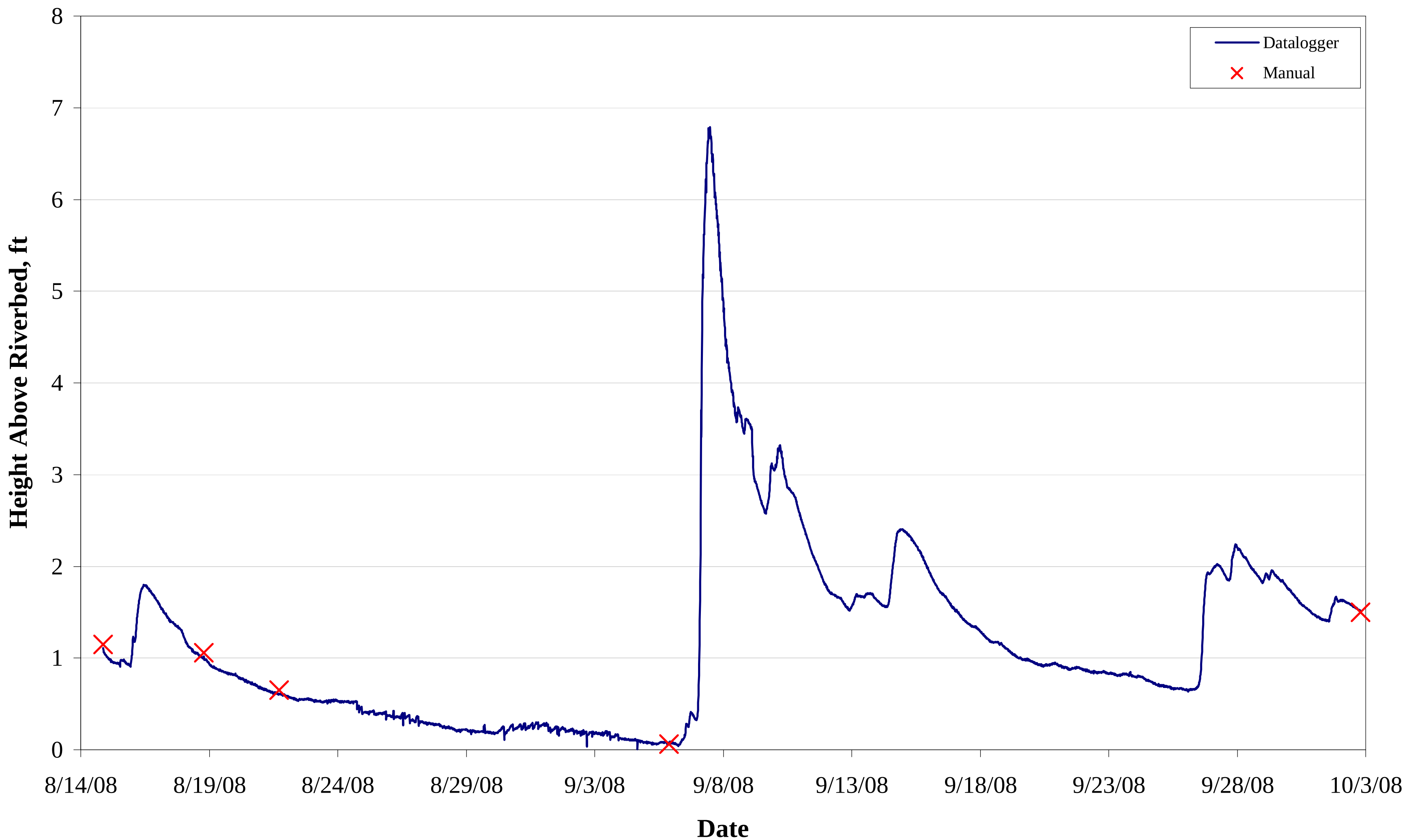
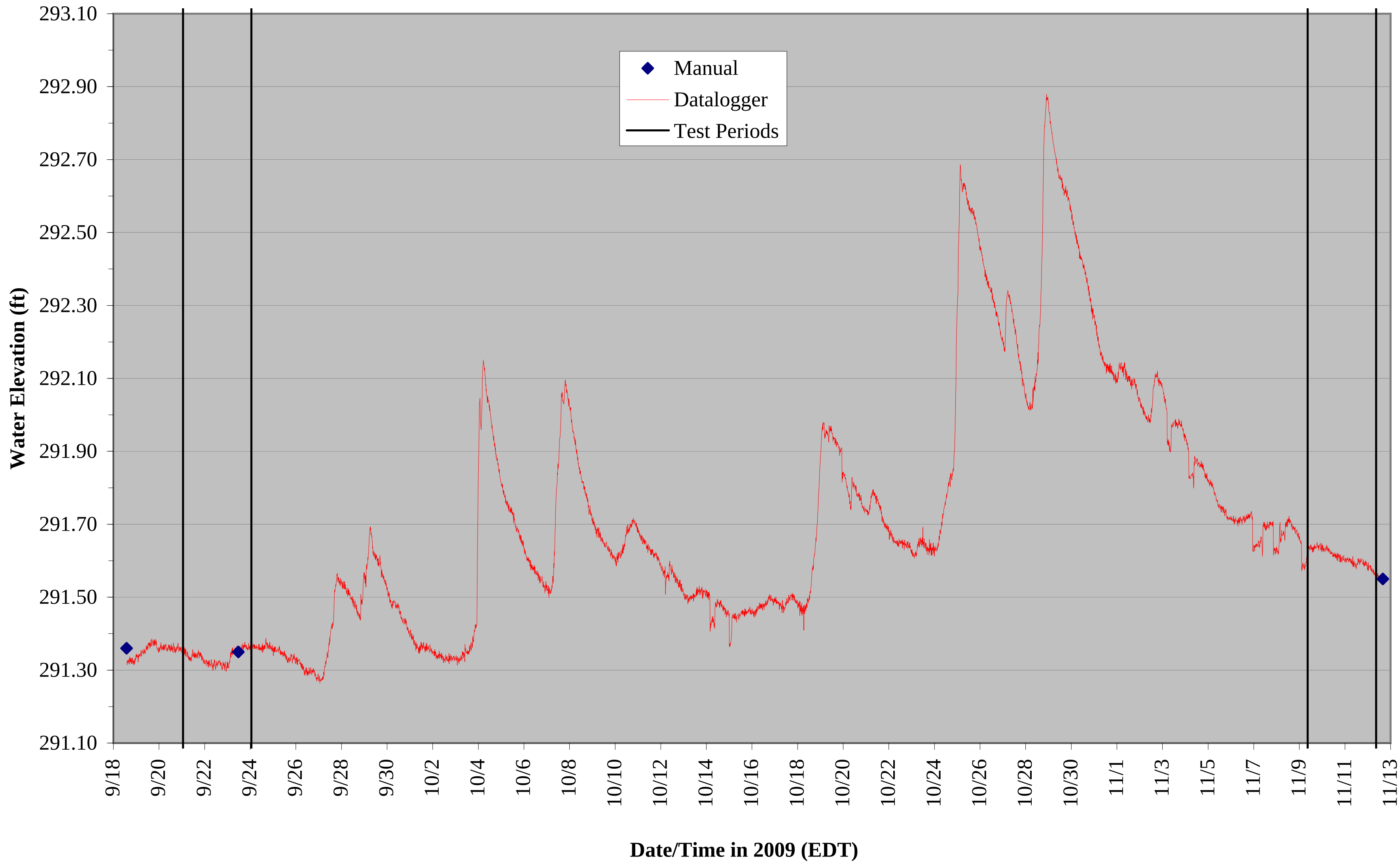
P1-W

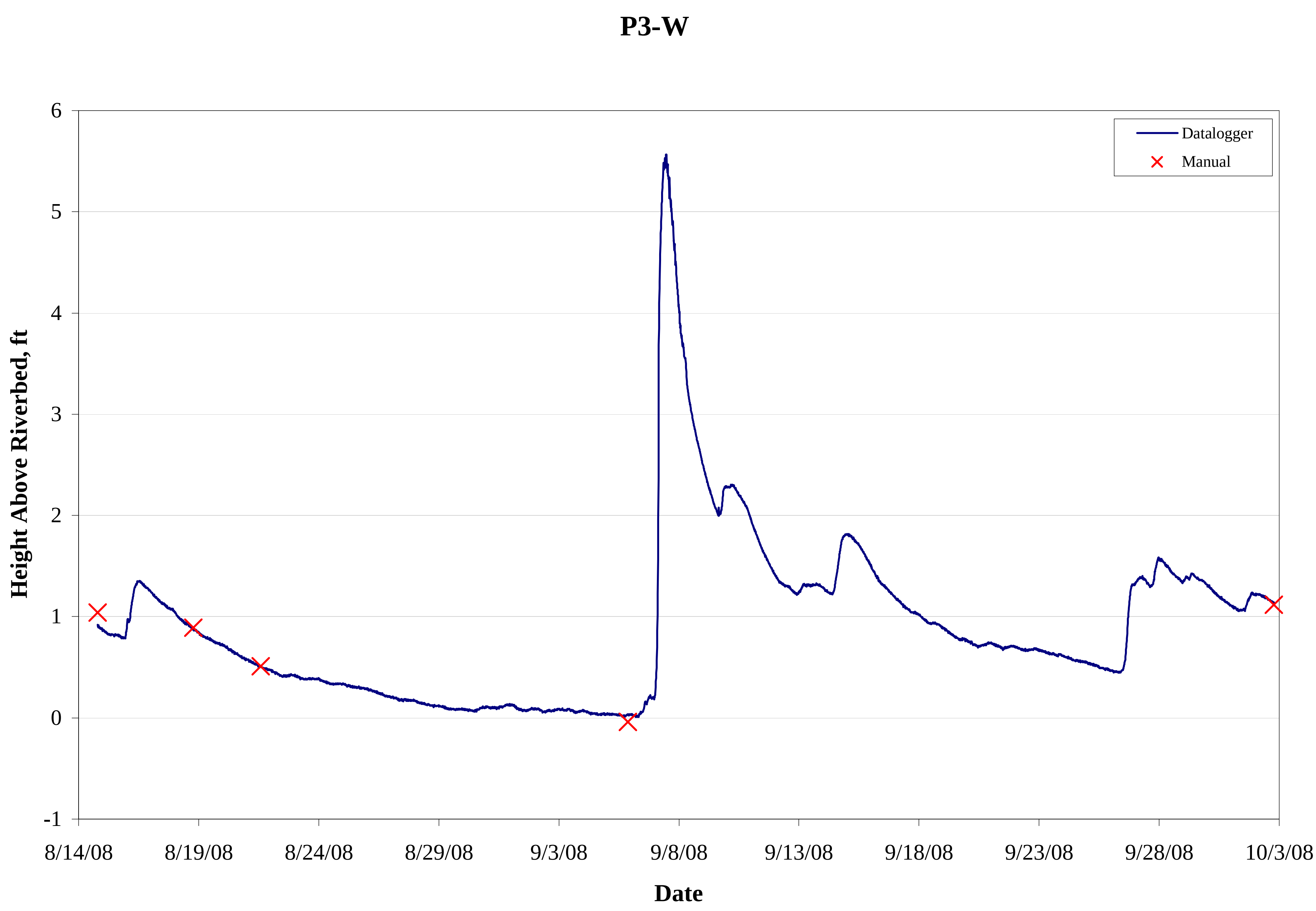


MILONE & MACBROOM®

Ground Water Elevations at P1-W in 2009

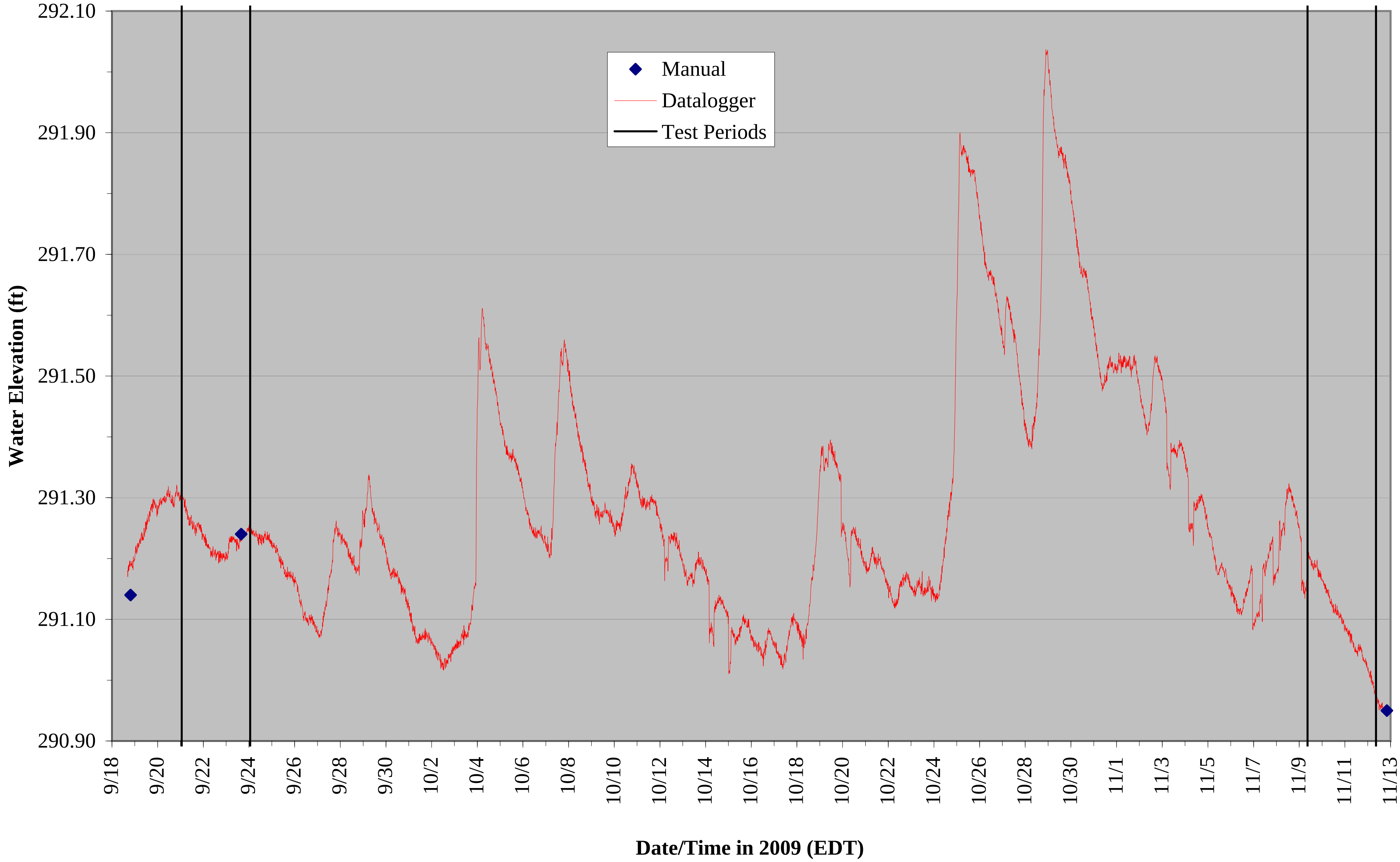


MILONE & MACBROOM®



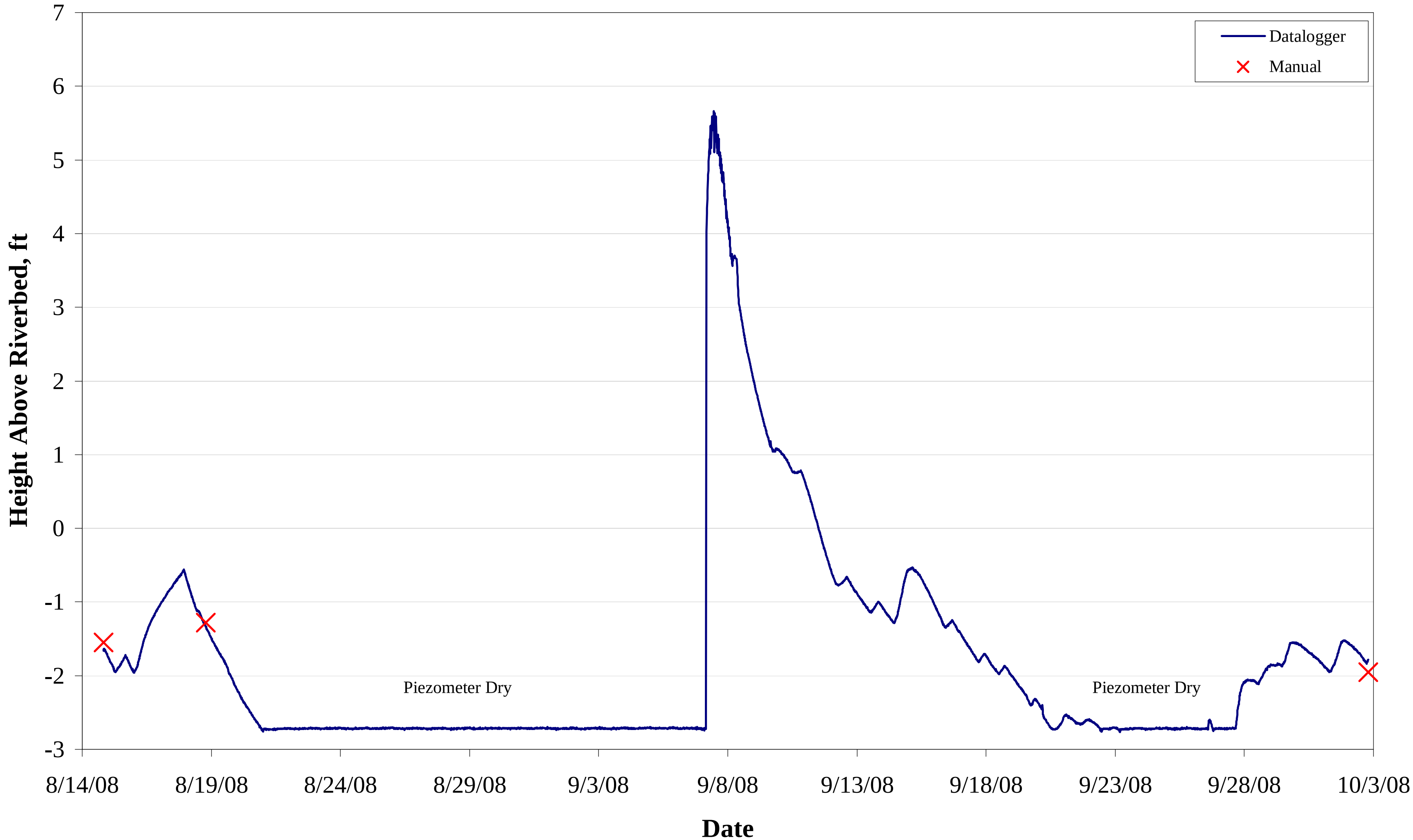
MILONE & MACBROOM®

Ground Water Elevations at P3-W in 2009



MILONE & MACBROOM®

P3-E



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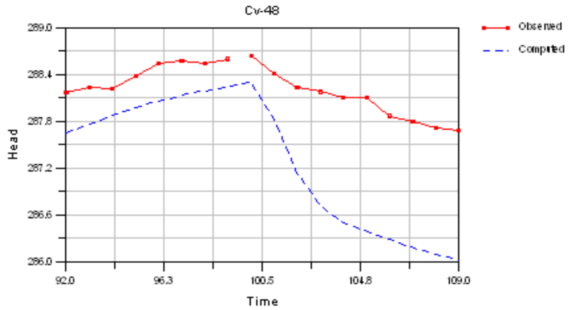
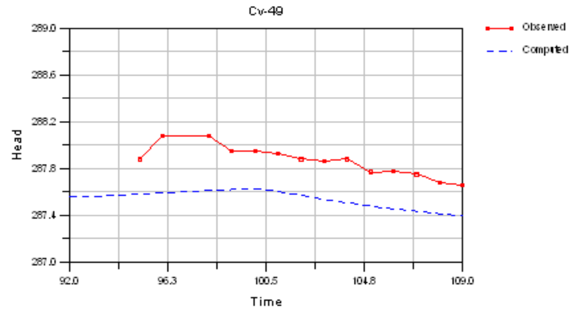
APPENDIX E
MODEL DRAWDOWN CURVES

1999 Model


Cv-48
289.0
288.4
287.8
287.2
286.6
286.0
Head
92.0
96.3
100.5
104.8
109.0
Time
Observed
Computed


Cv-49
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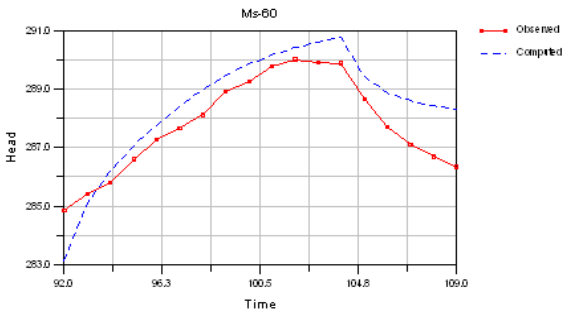
288.6
288.2
287.8
287.4
287.0
Head
92.0
96.3
100.5
104.8
109.0
Time
Observed
Computed


Ms-60
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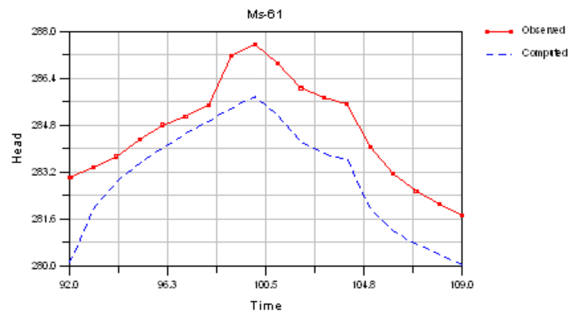
287.0
285.0
283.0
Head
92.0
96.3
100.5
104.8
109.0
Time
Observed
Computed


Ms-61
288.0
286.4
284.8
283.2
281.6
280.0
Head
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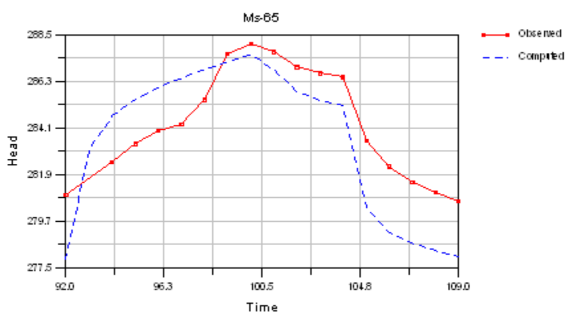
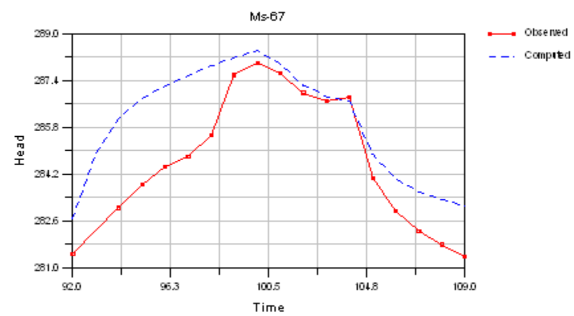
96.3
100.5
104.8
109.0
Time
Observed
Computed


Ms-65
288.5
286.3
284.1
281.9
279.7
277.5
Head
92.0
96.3
100.5
104.8
109.0
Time
Observed
Computed


Ms-67
289.0
287.4
285.8
284.2
282.6
281.0
Head
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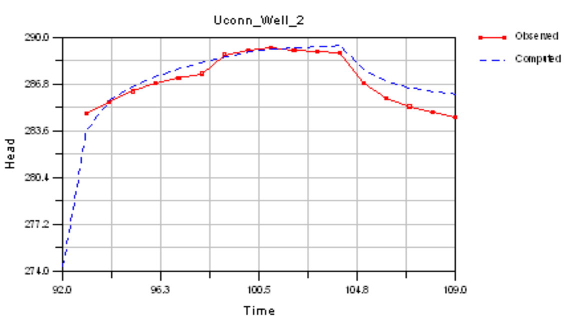
96.3
100.5
104.8
109.0
Time
Observed
Computed


Uconn_Well_2
290.0
286.8
283.6
280.4
277.2
274.0
Head
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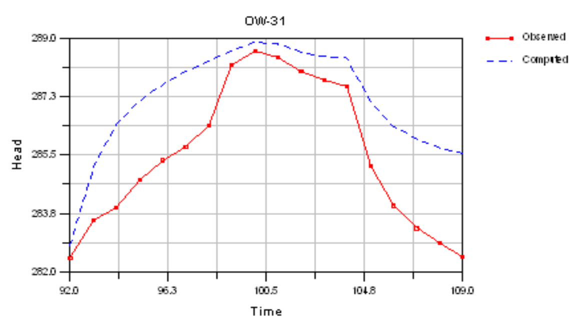
96.3
100.5
104.8
109.0
Time
Observed
Computed


OW-31
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283.8
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Head
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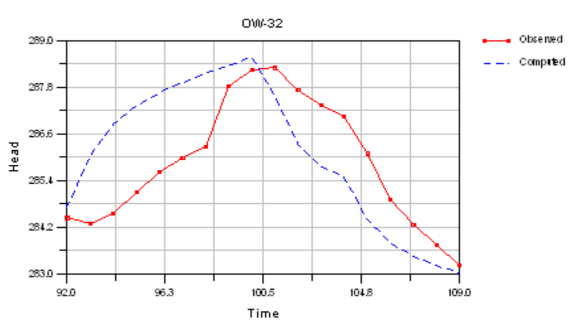
109.0
Time
Observed
Computed


OW-32
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Head
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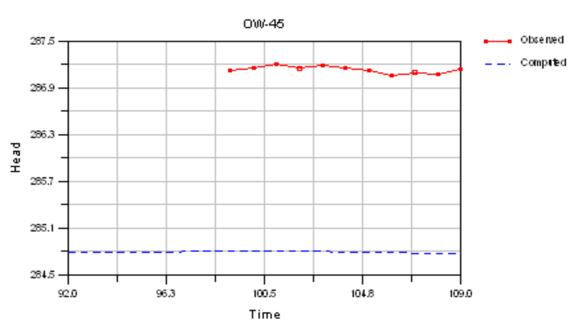
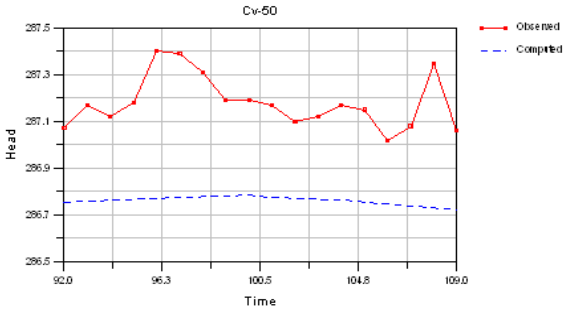
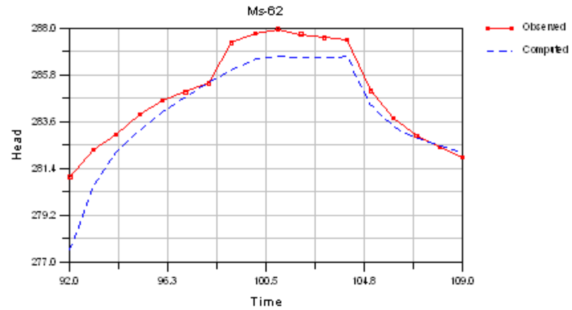
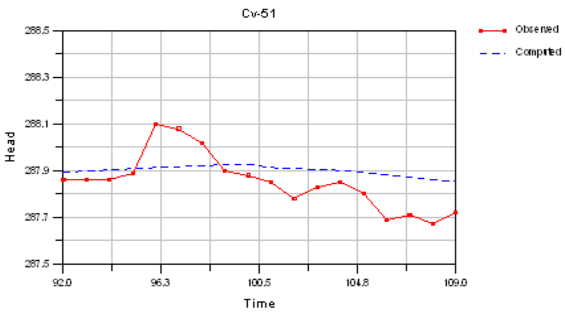
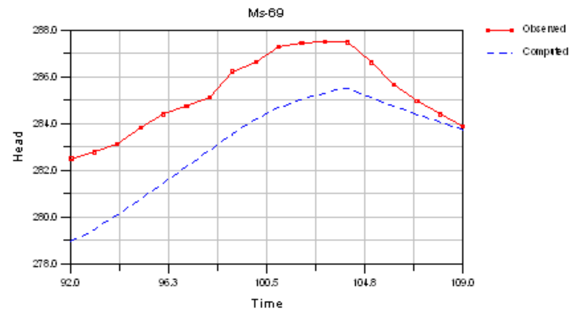
109.0
Time
Observed
Computed

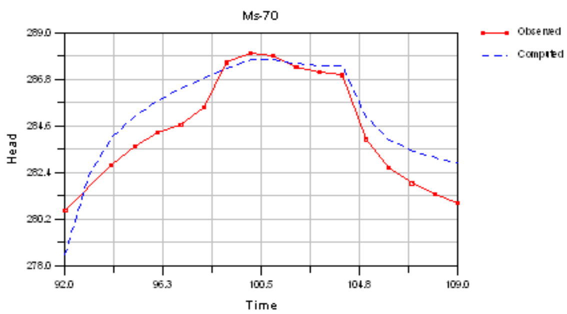

OW-45
287.5
286.9
286.3
285.7
285.1
284.5
Head
92.0
96.3
100.5
104.8
109.0
Time
Observed
Computed

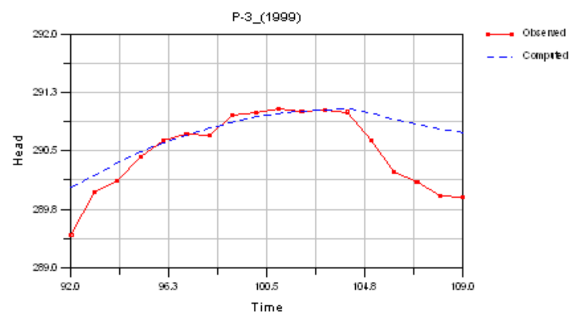

Cv-50
287.5
287.3
287.1
286.9
286.7
286.5
Head
92.0
96.3
100.5
104.8
109.0
Time
Observed
Computed

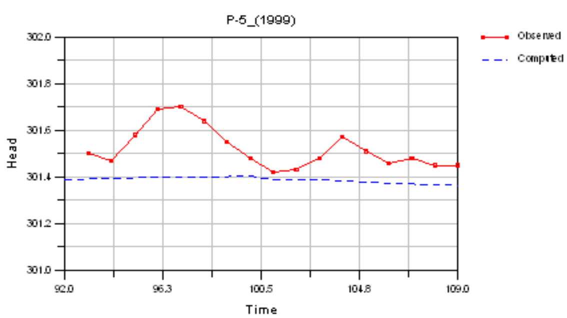

Ms-62
288.0
285.8
283.6
281.4
279.2
277.0
Head
92.0
96.3
100.5
104.8
109.0
Time
Observed
Computed

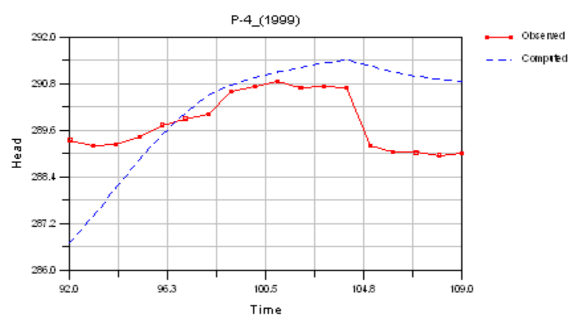

Cv-51
288.5
288.3
288.1
287.9
287.7
287.5
Head
92.0
96.3
100.5
104.8
109.0
Time
Observed
Computed

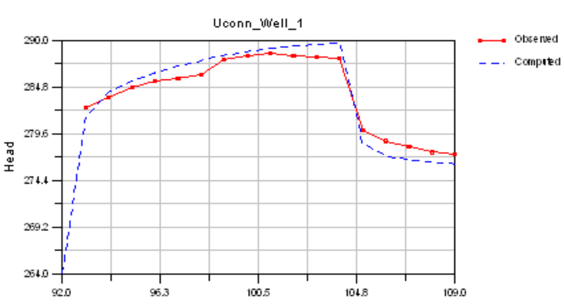

Ms-69
288.0
286.0
284.0
282.0
280.0
278.0
Head
92.0
96.3
100.5
104.8
109.0
Time
Observed
Computed

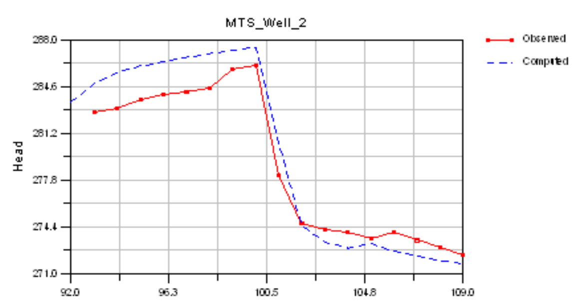

Ms-70
289.0
286.8
284.6
282.4
280.2
278.0
Head
92.0
96.3
100.5
104.8
109.0
Time
Observed
Computed

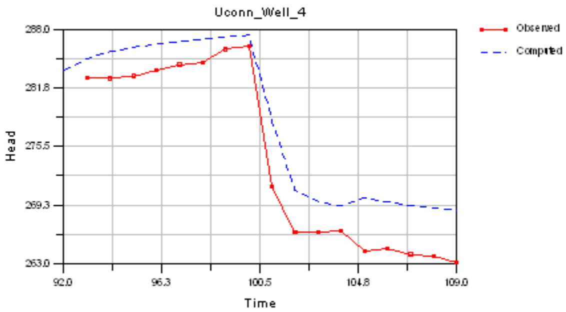

P-3_(1999)
292.0
291.3
290.5
289.8
289.0
Head
92.0
96.3
100.5
104.8
109.0
Time
Observed
Computed


P-5_(1999)
302.0
301.8
301.6
301.4
301.2
301.0
Head
92.0
96.3
100.5
104.8
109.0
Time
Observed
Computed


P-4_(1999)
292.0
290.8
289.6
288.4
287.2
286.0
Head
92.0
96.3
100.5
104.8
109.0
Time
Observed
Computed


Uconn_Well_1
290.0
284.8
279.6
274.4
269.2
264.0
Head
92.0
96.3
100.5
104.8
109.0
Time
Observed
Computed

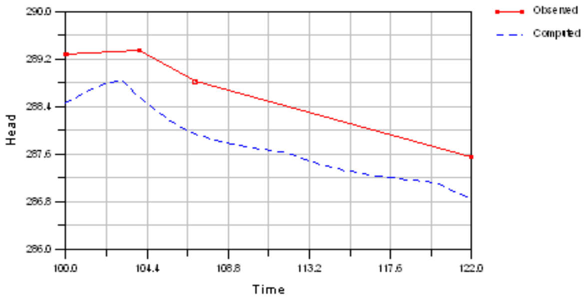

MTS_Well_2
288.0
284.6
281.2
277.8
274.4
271.0
Head
92.0
96.3
100.5
104.8
109.0
Time
Observed
Computed



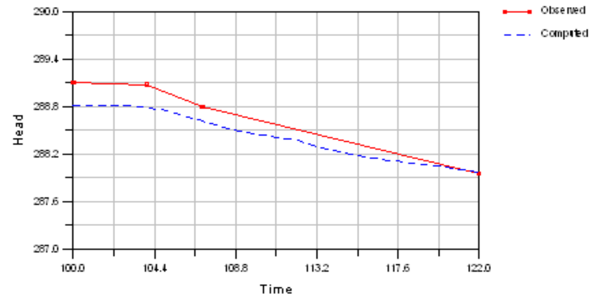
Uconn_Well_4
Observed
Computed
Head
Time
288.0
281.8
275.5
269.3
263.0
92.0
96.3
100.5
104.8
109.0

2008 Model

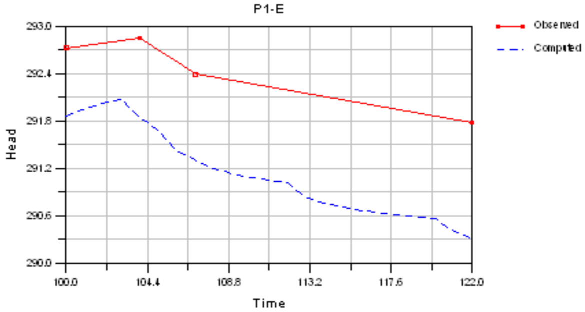
Cv-48



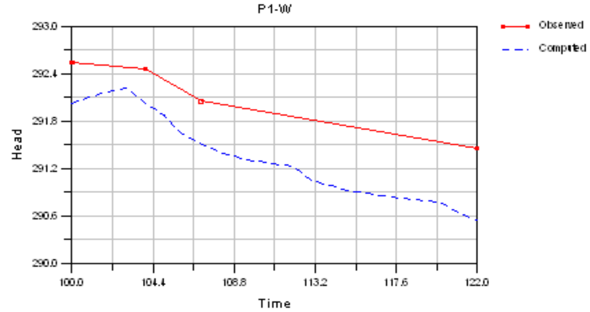
Cv-49



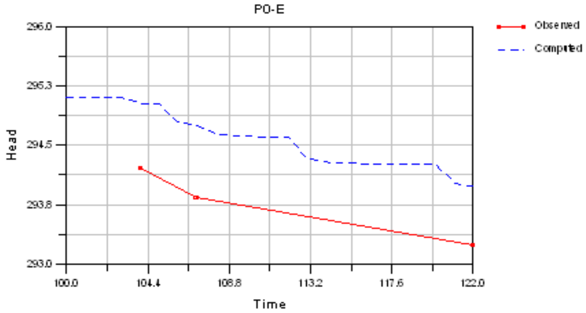
P1-E



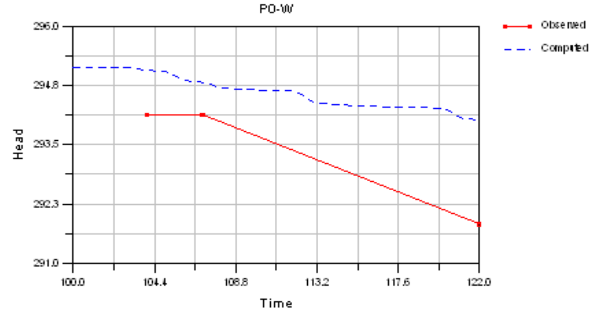
P1-W



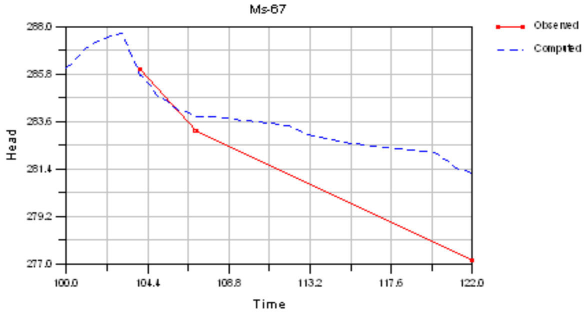
P0-E



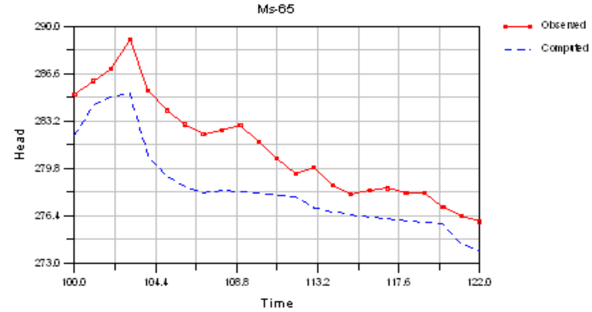
P0-W



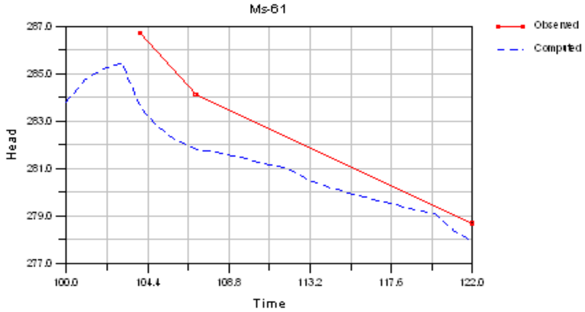
Ms-67



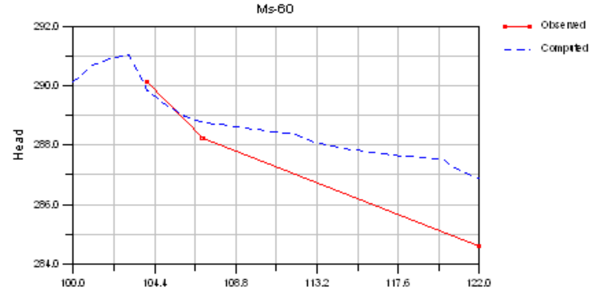
Ms-65

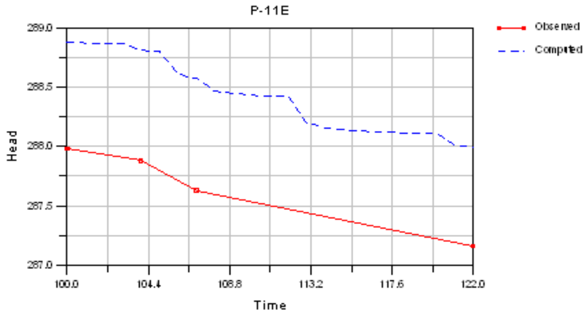


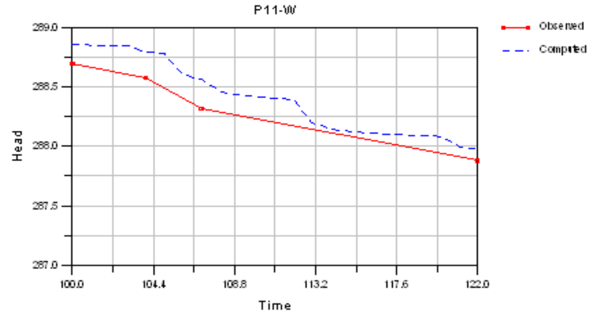
Ms-61

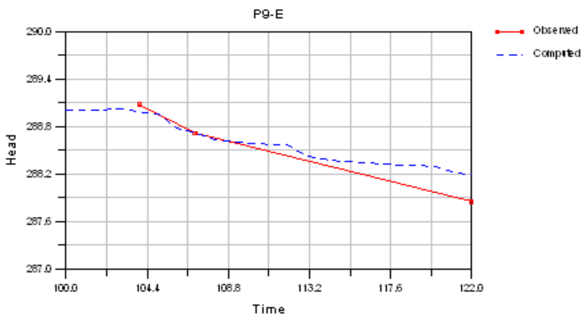


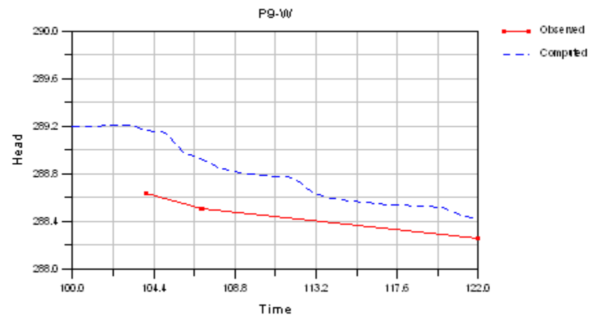
Ms-60

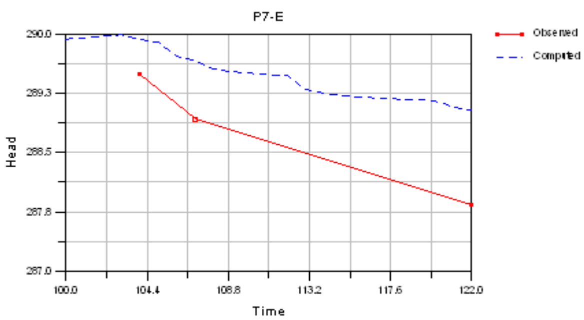


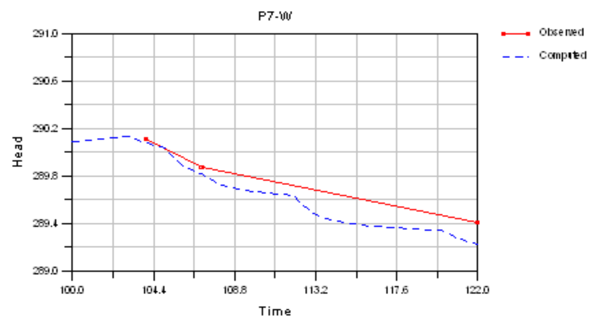

P-11E
Head
289.0
288.5
288.0
287.5
287.0
Time
100.0
104.4
108.8
113.2
117.6
122.0
Observed
Computed

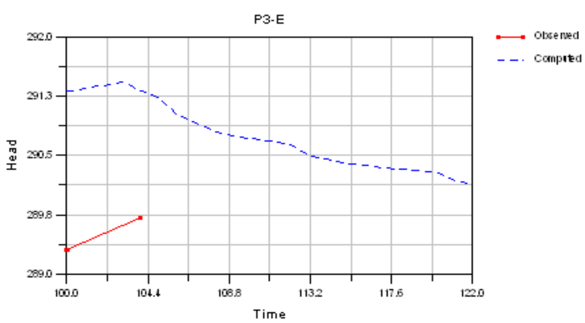

P11-W
Head
289.0
288.5
288.0
287.5
287.0
Time
100.0
104.4
108.8
113.2
117.6
122.0
Observed
Computed

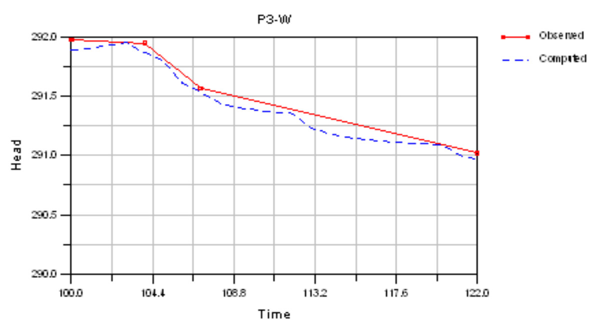

P9-E
Head
290.0
289.4
288.8
288.2
287.6
287.0
Time
100.0
104.4
108.8
113.2
117.6
122.0
Observed
Computed


P9-W
Head
290.0
289.6
289.2
288.8
288.4
288.0
Time
100.0
104.4
108.8
113.2
117.6
122.0
Observed
Computed

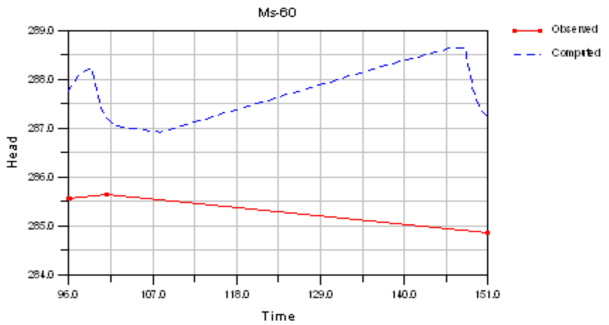

P7-E
Head
290.0
289.3
288.5
287.8
287.0
Time
100.0
104.4
108.8
113.2
117.6
122.0
Observed
Computed

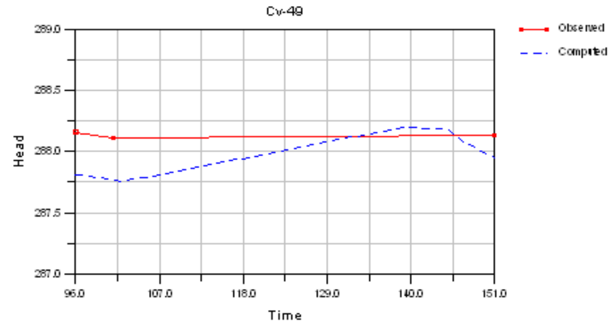

P7-W
Head
291.0
290.6
290.2
289.8
289.4
289.0
Time
100.0
104.4
108.8
113.2
117.6
122.0
Observed
Computed

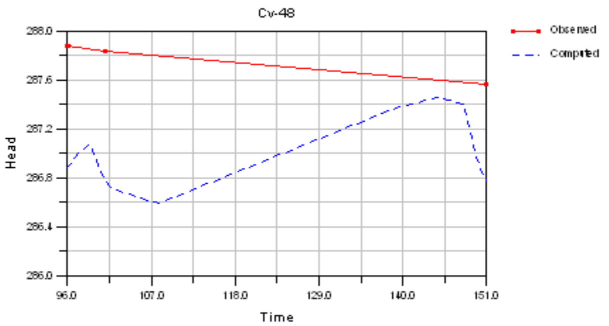

P3-E
Head
292.0
291.3
290.5
289.8
289.0
Time
100.0
104.4
108.8
113.2
117.6
122.0
Observed
Computed

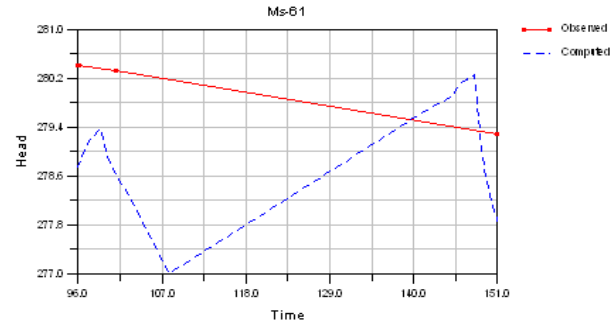

P3-W
Head
292.0
291.5
291.0
290.5
290.0
Time
100.0
104.4
108.8
113.2
117.6
122.0
Observed
Computed

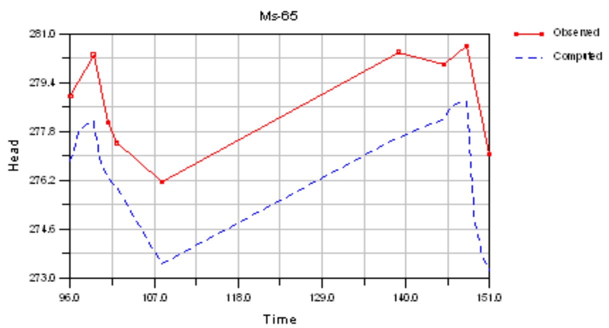
2009 Model

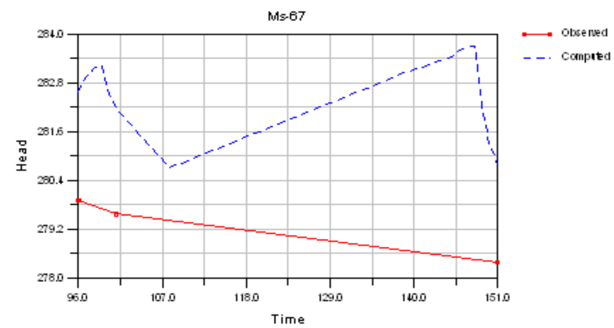

Ms-60
289.0
288.0
287.0
286.0
285.0
284.0
96.0
107.0
118.0
129.0
140.0
151.0
Head
Time
Observed
Computed

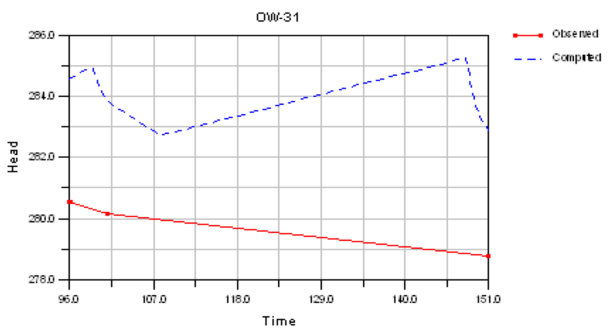

Cv-49
289.0
288.5
288.0
287.5
287.0
96.0
107.0
118.0
129.0
140.0
151.0
Head
Time
Observed
Computed

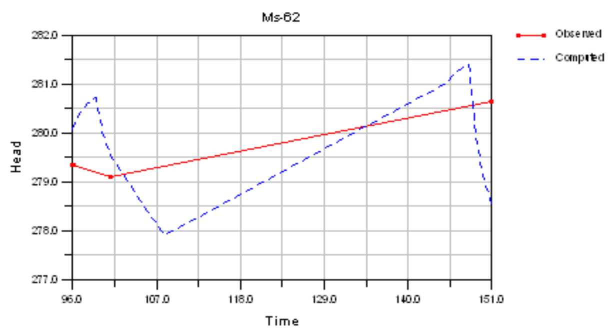

Cv-48
288.0
287.6
287.2
286.8
286.4
286.0
96.0
107.0
118.0
129.0
140.0
151.0
Head
Time
Observed
Computed

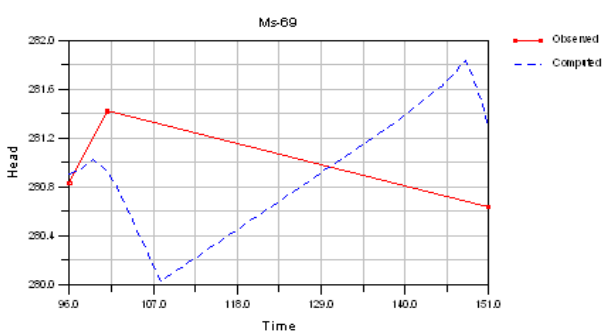

Ms-61
281.0
280.2
279.4
278.6
277.8
277.0
96.0
107.0
118.0
129.0
140.0
151.0
Head
Time
Observed
Computed

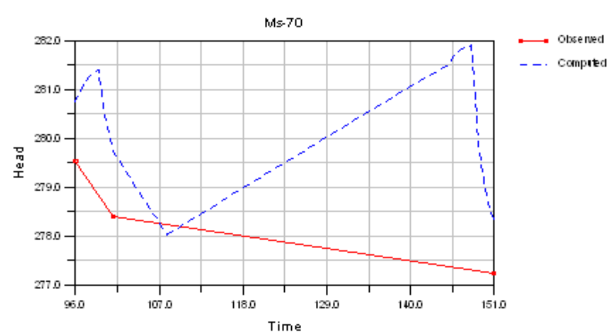

Ms-65
281.0
279.4
277.8
276.2
274.6
273.0
96.0
107.0
118.0
129.0
140.0
151.0
Head
Time
Observed
Computed

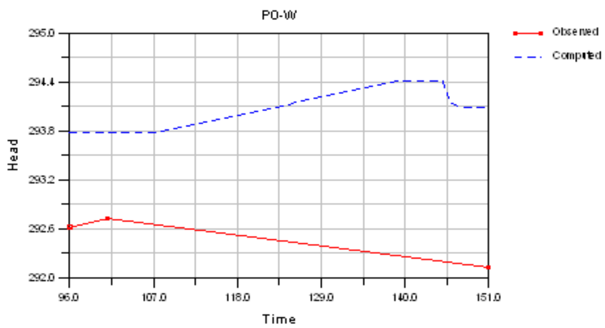

Ms-67
284.0
282.8
281.6
280.4
279.2
278.0
96.0
107.0
118.0
129.0
140.0
151.0
Head
Time
Observed
Computed

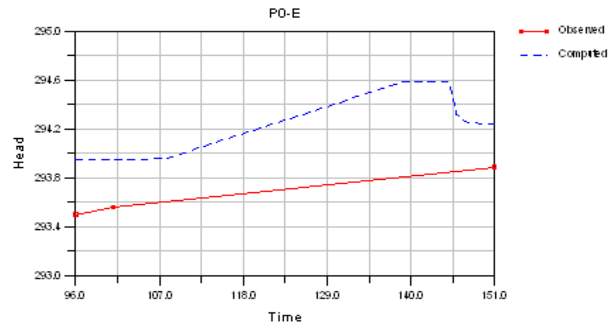

OW-31
286.0
284.0
282.0
280.0
278.0
96.0
107.0
118.0
129.0
140.0
151.0
Head
Time
Observed
Computed

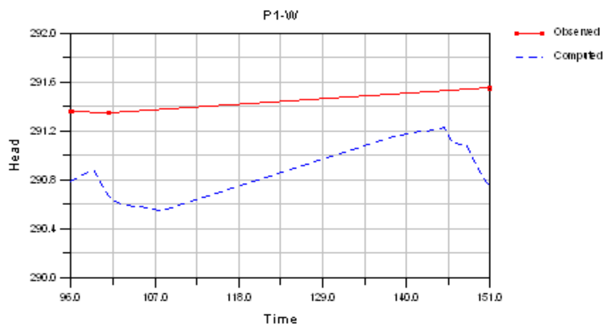

Ms-62
282.0
281.0
280.0
279.0
278.0
277.0
96.0
107.0
118.0
129.0
140.0
151.0
Head
Time
Observed
Computed

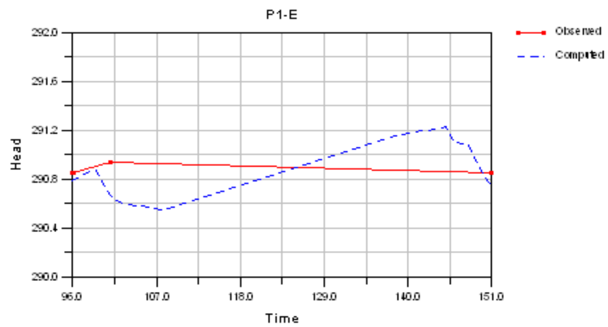

Ms-69
282.0
281.6
281.2
280.8
280.4
280.0
96.0
107.0
118.0
129.0
140.0
151.0
Head
Time
Observed
Computed

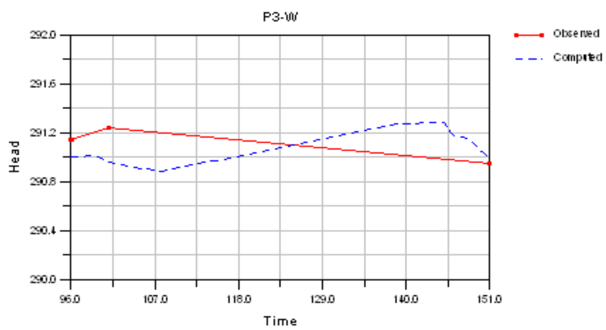

Ms-70
282.0
281.0
280.0
279.0
278.0
277.0
96.0
107.0
118.0
129.0
140.0
151.0
Head
Time
Observed
Computed

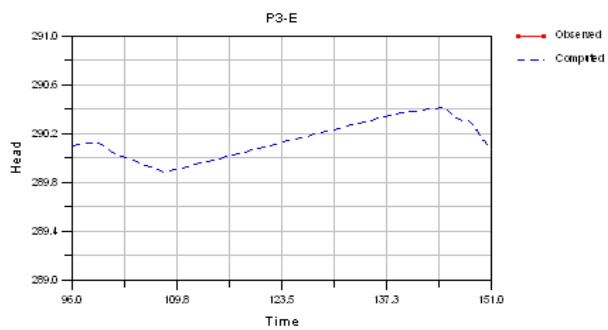

P0-W
Observed
Computed
Head
Time
295.0
294.4
293.8
293.2
292.6
292.0
96.0
107.0
118.0
129.0
140.0
151.0

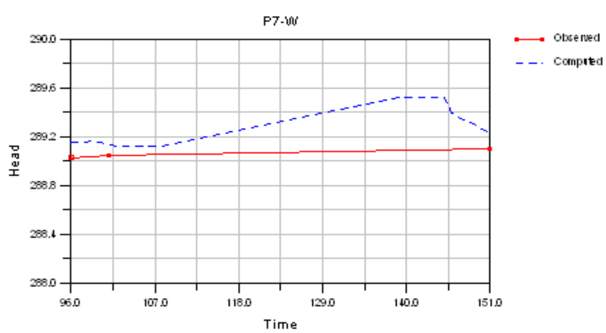

P0-E
Observed
Computed
Head
Time
295.0
294.6
294.2
293.8
293.4
293.0
96.0
107.0
118.0
129.0
140.0
151.0

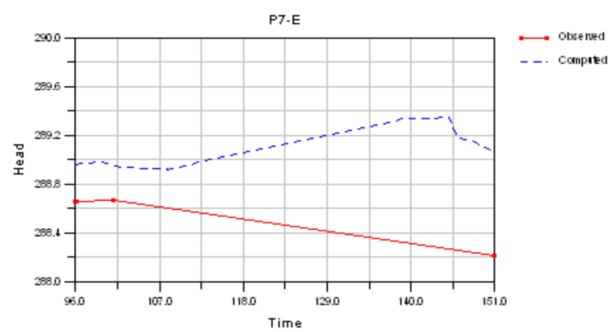

P1-W
Observed
Computed
Head
Time
292.0
291.6
291.2
290.8
290.4
290.0
96.0
107.0
118.0
129.0
140.0
151.0

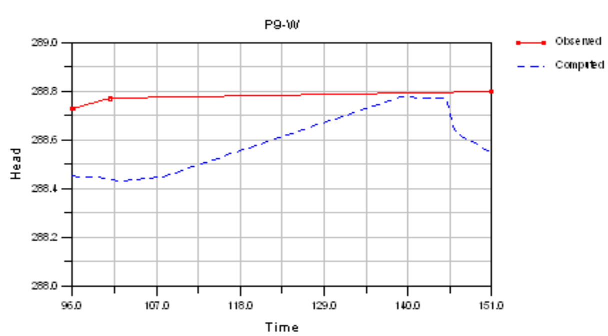

P1-E
Observed
Computed
Head
Time
292.0
291.6
291.2
290.8
290.4
290.0
96.0
107.0
118.0
129.0
140.0
151.0

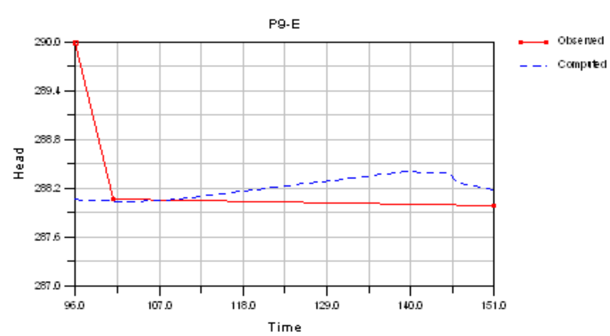

P3-W
Observed
Computed
Head
Time
292.0
291.6
291.2
290.8
290.4
290.0
96.0
107.0
118.0
129.0
140.0
151.0

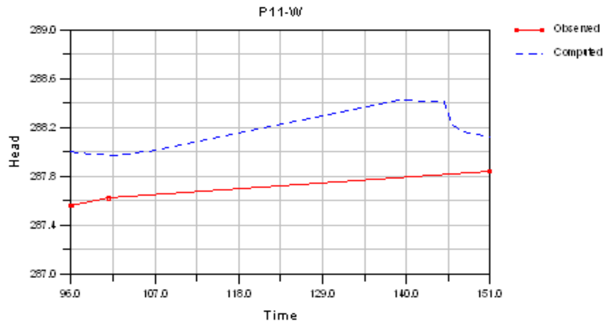

P3-E
Observed
Computed
Head
Time
291.0
290.6
290.2
289.8
289.4
289.0
96.0
109.8
123.5
137.3
151.0


P7-W
Observed
Computed
Head
Time
290.0
289.6
289.2
288.8
288.4
288.0
96.0
107.0
118.0
129.0
140.0
151.0

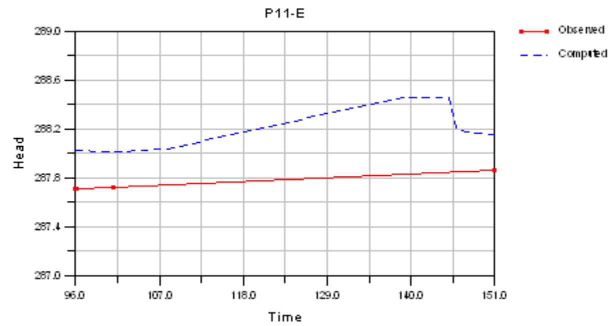

P7-E
Observed
Computed
Head
Time
290.0
289.6
289.2
288.8
288.4
288.0
96.0
107.0
118.0
129.0
140.0
151.0


P9-W
Observed
Computed
Head
Time
289.0
288.8
288.6
288.4
288.2
288.0
96.0
107.0
118.0
129.0
140.0
151.0


P9-E
Observed
Computed
Head
Time
290.0
289.4
288.8
288.2
287.6
287.0
96.0
107.0
118.0
129.0
140.0
151.0



P11-W
Observed
Computed
Head
Time
289.0
288.6
288.2
287.8
287.4
287.0
96.0
107.0
118.0
129.0
140.0
151.0



P11-E
Observed
Computed
Head
Time
289.0
288.6
288.2
287.8
287.4
287.0
96.0
107.0
118.0
129.0
140.0
151.0

APPENDIX F
CD OF NUMERICAL MODEL FILES

APPENDIX G
DISCUSSION OF IMPOUNDMENTS IN WILLIMANTIC RIVER HEADWATERS

Appendix G
Discussion of Impoundments in Willimantic River Headwaters

Table 13 of the USGS bulletin *Water Resources Inventory of Connecticut, Shetucket River Basin* (1967) lists lakes, ponds, and reservoirs in the Shetucket River basin. Impoundments with site numbers beginning 1191 and 1192 are located upstream of the Willimantic River Wellfield. Of the 12 listed impoundments, six are flood control reservoirs and two are public water supply reservoirs with limited opportunity for making streamflow releases. The remaining four have significant storage.

Name	Location	Area (acres)	Total Storage (MG)	Usable Storage (MG)	Safe Draft Rate*		
					Driest Year (cfs)	Median Year (cfs)	Wettest Year (cfs)
State Line Pond	Stafford	79.9	81.5	81.5	1.3	2.7	10.0
Crystal Lake	Stafford and Ellington	201	1,287	314	1.2	3.9	8.3
New City Pond	Stafford	36.6	68	68	0.5	1.0	3.8
Staffordville Reservoir	Stafford	165	565	565	4.0	8.8	24.0

*Based on flows measured at reference gages during 1930-1960

The USGS estimated the "safe draft" rates for these impoundments using the storage volumes provided and the watershed sizes, with the criteria that all impoundments would refill within one year. Safe draft rates are independent of reservoir spillage; it is assumed that drawdown of the impoundment would occur and spillage would cease. Three sets of safe drafts were reported corresponding to a dry year from the reference period (1930-1960), the median year, and the wettest year. Given the methodology and the information provided, it is not possible for the reader to determine how low each impoundment would drop for each scenario (dry, median, and wet) before refilling.

The reported safe draft rates are significant. Neglecting New City Pond (since it is upstream of Staffordville Reservoir), a total of 6.7 cfs was reported as the driest year safe draft for State Line Pond, Crystal Lake, and Staffordville Reservoir combined. If the driest year safe drafts were directed to the downstream watercourses instead of consumed elsewhere (as if used for public water supply), the instream flow of the Willimantic River would likewise be increased by 6.7 cfs, neglecting any losses associated with less frequent spillage.

While the safe draft rates provided by USGS are excellent for making rule-of-thumb estimates of how various impoundments could perform as water supplies, the drafts likely allow for wide variation in the impoundment water levels. If an impoundment were to be used to augment instream flow to the Willimantic River, it would be prudent and likely necessary to minimize the variation in impoundment levels in order to be fair to stakeholders that utilize impoundments for recreation and industrial purposes. Consider the following:

- The Town of Stafford owns the Staffordville Reservoir dam while a manufacturing company controls the dam outlet works to manage the impoundment level for fire protection. When the dam underwent emergency repairs in 2008, significant drawdown was not possible because the manufacturing company required a minimum head. Furthermore, the Town of Stafford provides a small beach for residents, and numerous homeowners along the perimeter of the lake use the impoundment for recreation. It is believed that the dam may have additional repair needs.

- Crystal Lake Clear Water Preservation, LLC owns the Crystal Lake dam. The Town of Ellington provides a small beach for residents, the State of Connecticut provides a boat launch, and numerous homeowners along the perimeter of the lake use the impoundment for recreation. The lake suffers from milfoil problems. A milfoil control program is reportedly underway.

It is likely not necessary to provide continuous instream flow releases in the Willimantic River. Pulsed or short-term releases may be appropriate for managing among the common, critical, extreme, and rare thresholds. Pulsed and short-term releases would allow for protection of stakeholder interests in keeping water levels as high as possible.

Based on the surface area and neglecting side slopes, precipitation, and inflow from tributaries, one foot of drawdown in Staffordville Reservoir can yield a flow of 2.8 cfs for approximately 30 days. This is a significant flow relative to the gaps between the critical, extreme, and rare thresholds. Furthermore, a period of 30 days is substantial and could carry the wellfield through a month of uninterrupted production if low natural flows occurred in the river.