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Water Balance, Well Pumping, and Streamflow Analysis of the Upper Parker River

June 2008

Submitted to:
Parker River Clean Water Association
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Byfield, MA 01922-0798

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Horsley Witten Group, Inc.

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APPENDICES

Water Balance, Well Pumping, and Streamflow Analysis of the Upper Parker River

1.0 INTRODUCTION

The Horsley Witten Group, Inc. (HW) is pleased to have assisted The Parker River Clean Water Association (PRCWA) and the Massachusetts Department of Fish and Game Riverways Program (Riverways) with this assessment of low flow conditions in the upper Parker River. The PRCWA goal is to protect and preserve the Parker River and Plum Island Sound Watersheds for present and future generations, and to educate the general public and local officials on watershed topics. The River Instream Flow Stewards (RIFLS) program, initiated in 2003 by the Riverways Program, aims to educate the public about the importance of adequate stream flows for river health by providing high quality, river-specific stream flow data to aid in stream flow analysis and water management, and to restore rivers suffering from unnaturally low flows. This project was funded by a RIFLS Restoration Grant, the goal of which is to assist RIFLS groups to use their stream flow data and take the next steps toward restoring more natural flow regimes and healthier river ecosystems.

The Parker River is located on the north shore of Massachusetts, north of the Ipswich River and south of the Merrimack River, and it discharges to the Plum Island Estuary. The upper Parker River is considered (for the purposes of this study) to be the river upstream of Interstate Route 95. The upper Parker River Watershed is located primarily in the towns of Boxford, Georgetown, Groveland, West Newbury, and Newbury (Figure 2.1). Glacial geologic processes have left the upper Parker River Watershed with a mixture of sand/gravel and till for surficial land cover, with lesser amounts of wetland soils, floodplain alluvium, and exposed bedrock. This area receives approximately 42 inches per year of annual precipitation (Gay and Delaney, 1980). Land use is a mixture of rural and suburban development with limited pockets of denser residential and commercial development in the downtown Georgetown area. Most watershed residents receive municipal water supply from wells located within the watershed while more rural residents have private supply wells. Wastewater is almost exclusively handled by private onsite septic systems. Stormwater management is primarily informal country drainage with limited areas of catch basins, pipes, and river discharges in the more densely populated areas.

Previously, the Parker River Low Flow Study (Gomez & Sullivan, 2003) raised concerns about low flow conditions after conducting statistical analyses of the historical flow records from the USGS gage at Route 95, and also documented occurrences of dry river beds in the reach of the Parker River near the Georgetown well fields. The dominant causes of low flow conditions, however, are potentially varied and were not adequately identified in that study. It has been difficult to make significant strides toward flow restoration without a better understanding of the likely sources of flow stress. This new study was intended to help evaluate potential causes of the flow stress in the upper Parker River and will move the restoration effort forward by developing recommendations to increase river flow. The study includes average annual water balance analyses intended to evaluate the overall health or stress level of the system as well as shorter term, or seasonal, analyses intended to isolate the potential impacts of water supply withdrawals relative to other hydrologic factors.

Unlike the previous Gomez and Sullivan study, which looked at changes in the river's flow record over time, this current study focuses on current conditions. 2006 is the target year for most of the analyses because it is the year for which the most complete and current data are available.

The study approach is three-pronged. The first step was to evaluate the annual "water budget" in the upper Parker River Watershed. This water budget approach calculates the long-term groundwater recharge in the watershed under predeveloped "natural" conditions compared to current conditions to assess whether the watershed is experiencing a net loss or gain of water relative to predeveloped conditions. A key underlying assumption in this model is that baseflow in the stream is equivalent to recharge in the watershed. This approach also allowed us to evaluate scenarios in which water withdrawals or other human variables are altered.

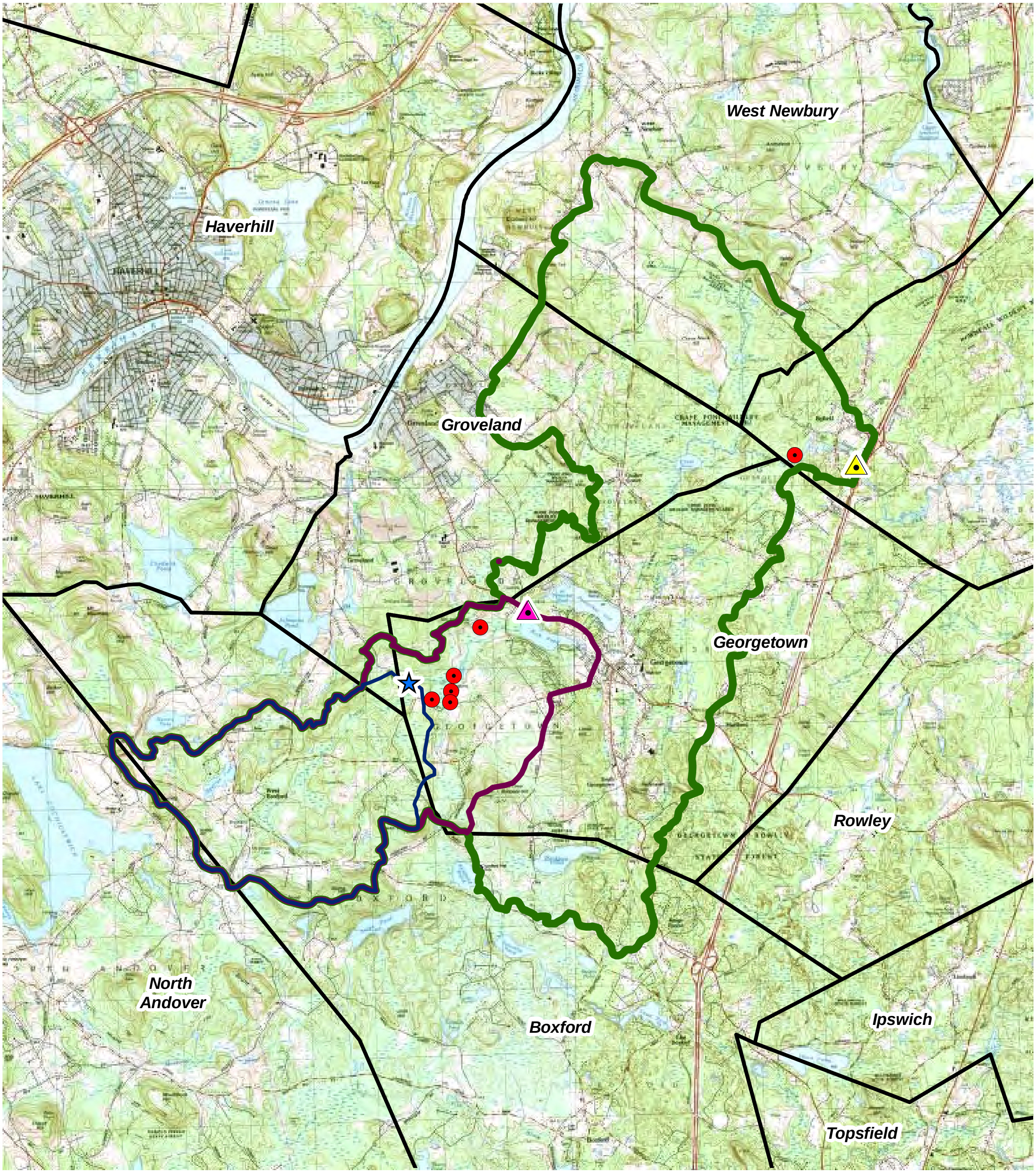
The second prong of our approach was to use the USGS stream depletion model to evaluate the time-varying streamflow depletion caused by pumping at the larger public supply wells. This model looks at seasonal and other short-term fluctuations in flow and pumping, as opposed to the long-term water balance approach.

The third prong is a comparison of the existing Parker River flow regime to estimates of potential natural or unimpacted flow regimes. These analyses provide another means to view the overall severity and the seasonal nature of hydrologic impacts in the watershed.

2.0 WATER BUDGET ANALYSIS

The water budget method presented here is a planning level tool designed to evaluate the annual hydrologic impacts associated with water supply withdrawals, wastewater discharges and stormwater runoff associated with land uses. The water budget method is a spreadsheet and GIS-based model that uses a mass balance approach in which groundwater recharge is considered equivalent to stream baseflow on an annual basis. It estimates stream baseflow changes resulting from water withdrawal, water transfer, wastewater discharges and stormwater runoff associated with different land uses. Baseflow is the flow that sustains the stream between precipitation and runoff events. Model input factors include land use type and associated water use and wastewater flows, recharge rates associated with varying surficial geology, and recharge rates for wetlands, impervious cover, and water and sewer service areas. This water budget tool calculates both pre-development (natural) and post-development (current) recharge. It also provides a tool to evaluate potential future land use scenarios and associated water, sewer and stormwater infrastructure changes.

The upper Parker River Watershed is comprised of three "nested" subwatersheds (Figure 2.1). The largest subwatershed is the land draining to the Byfield USGS gage station on the Parker River. The middle subwatershed is comprised of the area that drains to a RIFLS monitoring site at Route 97. The smallest of the subwatersheds is the mostly undeveloped land that drains to the Parker River crossing at Uptack Road.



Legend

- Uptack Road Study Point
- Uptack Road Study Point Subwatershed
- Route 97 RIFLS Site / New USGS Gauge
- RIFLS Route 97 Gauge Subwatershed
- USGS Byfield Gauge Station
- USGS Byfield Gauge Subwatershed
- Groundwater Wells
- Town Boundaries

N

7,000 Feet

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Modeled Subwatersheds
Parker River Clean
Watershed Association

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USGS_Locus 8.5x11

Fig. 2.1

The water budget model was run for each nested subwatershed, incorporating factors such as groundwater recharge, permitted water withdrawals, permitted groundwater discharges, and the existing land uses in order to estimate the predicted baseflow for each segment of the river draining a given subwatershed. These estimates for the developed conditions were then compared to pre-development streamflow estimates for each subwatershed. These comparisons give a measure of the relative impacts that development has on the water balance in each subwatershed in order to help prioritize actions to address these development impacts. Additionally, this planning tool was used to evaluate the hydrologic impacts of potential restoration alternatives.

The management goal for the watershed is to mimic the natural pre-development recharge as closely as possible to restore the natural hydrologic regime in the river, and to protect water resources and dependant habitat. The more groundwater recharge that occurs on a long-term average basis, the greater the baseflow contribution to streamflow, and the healthier the resulting in-stream habitat. By concentrating on groundwater recharge on a long-term average basis, the temporal fluctuations in streamflow can be ignored. This helps to keep the model relatively simple and less data intensive.

2.1 Structure of the Water Budget Model

One key characteristic of this water budget tool is that it focuses on average annual conditions as a planning-level assessment of the overall hydrologic balance of subject watersheds. Because water discharged to a gaining stream from groundwater is the primary source of the "baseflow" that occurs between precipitation-runoff events, average annual groundwater recharge within a watershed can be considered as a proxy for average annual baseflow discharge. Mathematically, the groundwater recharge-based water budget approach is expressed as follows:

$$\mathrm{BF} = \text{Water Inputs} - \text{Water Outputs} = (GW_{\mathrm{nat}} + \mathrm{WW}_{\mathrm{GWDP}} + \mathrm{WW}_{\mathrm{septic}}) - (WS_{\mathrm{WMA}} + WS_{\mathrm{prvt}} + SW_{\mathrm{EIA}})$$

Where:

- BF = Average annual baseflow in a stream;
- GW_{nat} = Natural groundwater recharge;
- WW_{GWDP} = Groundwater Discharge Permit inflows;
- WW_{septic} = Private septic system inflows;
- WS_{WMA} = Water Management Act permitted groundwater withdrawals;
- WS_{prvt} = Private groundwater withdrawals; and
- SW_{EIA} = Stormwater runoff from effective impervious areas.

Groundwater Discharge Permit (GWDP) and Water Management Act (WMA) permit data from the upper Parker River Watershed for the years 2002-2007 were available through the Massachusetts Department of Environmental Protection (MADEP). However, this data was variable in the actual time periods covered. To compensate for these information gaps, this water budget focuses on the year for which the most complete dataset is available – 2006. To further ensure data consistency in this model, input variables that were derived from these datasets also focus on the year 2006 (e.g., per capita water usage assumptions).

2.2 Water Budget Inputs

2.2.1 Natural Recharge

Groundwater recharge rates were selected based upon representative USGS studies, surficial geology, long-term precipitation data, and professional judgment. These rates were then run through the water budget model for existing conditions and the model-simulated baseflow results compared to actual measured baseflow at the Byfield USGS gage station. Because anthropogenic flow impacts are captured in other aspects of the water budget model, the recharge estimates generated in this way (on a per unit land area basis for each surficial geology cover) should be reasonably unaffected by anthropogenic alterations to the observed flow regime. Baseflow was estimated to be approximately 16.6 cfs, which is the annual average of monthly minimum flows at the station, using the last ten years of data (1998-2007) (Appendix A). The most recent data were selected to best match current land and water use characteristics. Ten years was selected as a reasonably long enough time span to be statistically significant with regard to natural climatic variability.

Land areas for each surficial geological formation were calculated in GIS. Initial recharge rates based on available USGS information and best professional judgment were evaluated for each surficial geological area and then compared to measured baseflow estimates from USGS stream gage data to arrive at representative values.

According to MassGIS, surficial geology in the sub-watershed is divided into the following five main categories (Figure 2.2) with recharge rates used in the water budget shown:

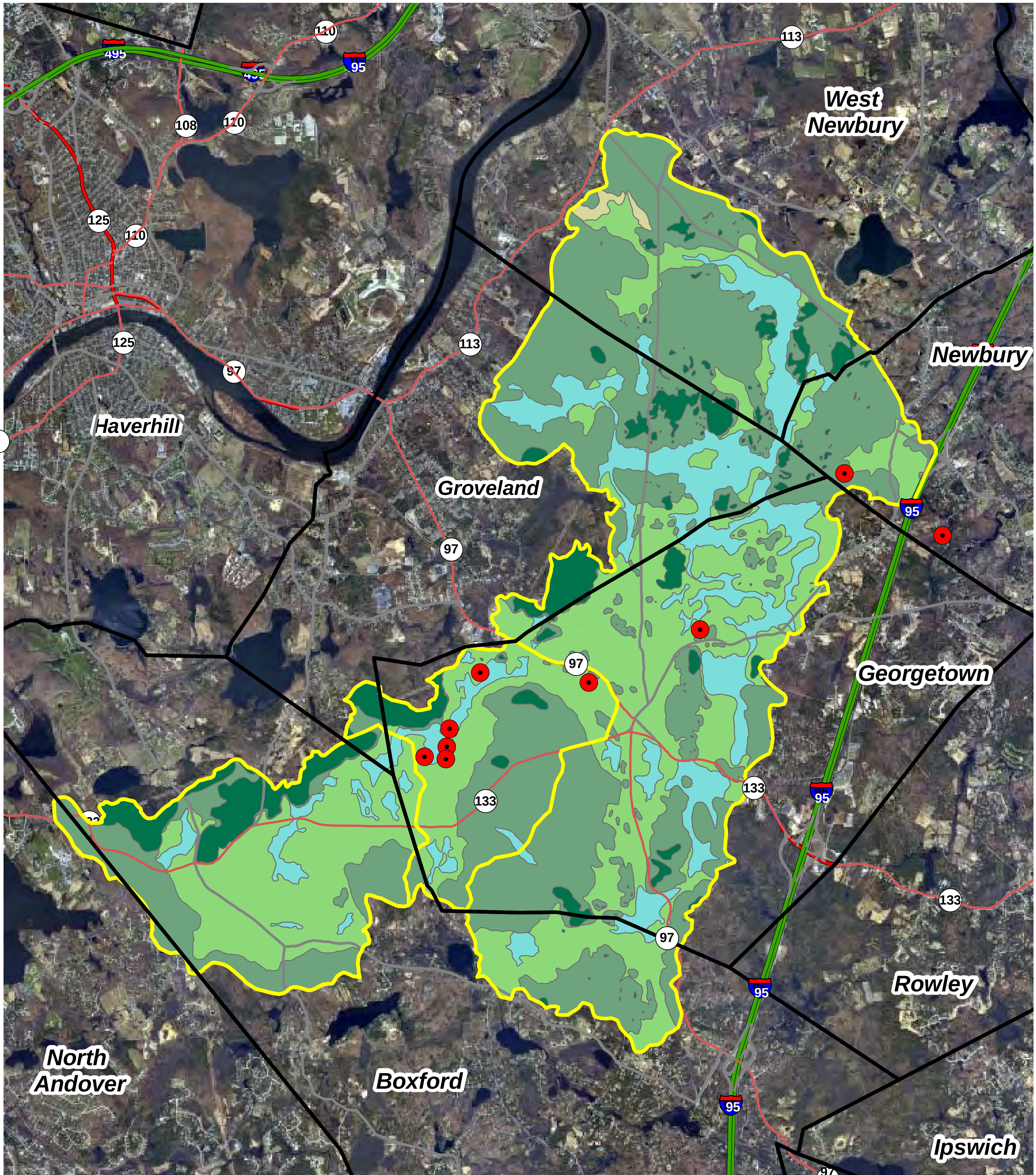
- Stratified sand and gravel – 25 inches per year;
- Shallow or exposed bedrock – 2 inches per year;
- Floodplain alluvium – 5 inches per year;
- Swamp deposits – 5 inches per year; and
- Till – 10 inches per year.

Because wetlands are generally groundwater discharge areas where annual evapotranspiration equals or exceeds direct precipitation, the presence of wetlands supersedes the underlying surficial geology such that all wetland areas have a simulated recharge rate of 0 in/yr, regardless of the underlying surficial geology. Wetland areas are shown in Figure 2.3.

2.2.2 Impervious Surfaces

Impervious surfaces were identified throughout the watershed using a MassGIS raster data layer produced in 2007 that displays all of the impervious areas throughout the state. Impervious surfaces include rooftops, roads, parking lots, and incidental impermeable surfaces such as sidewalks, patios, pools, etc.

However, some of the impervious area is small and disconnected from other impervious areas, such that it drains to grassed or vegetated areas and is able to infiltrate into the ground before it is channelized and/or discharged via a stormwater system. The subset of the impervious area that



Legend

Watersheds
Town Boundaries
Swamp Deposits
Till
Shallow/Exposed Bedrock
Floodplain Alluvium
Glacial Stratified Deposits, Coarse
Drinking Water Wells

N

Feet
6,000

**1:5,000 Color Ortho Imagery, MassGIS 2005*
Surficial Geology (1:24,000), MassGIS 2007

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Surficial Geology
Parker River Clean
Watershed Association

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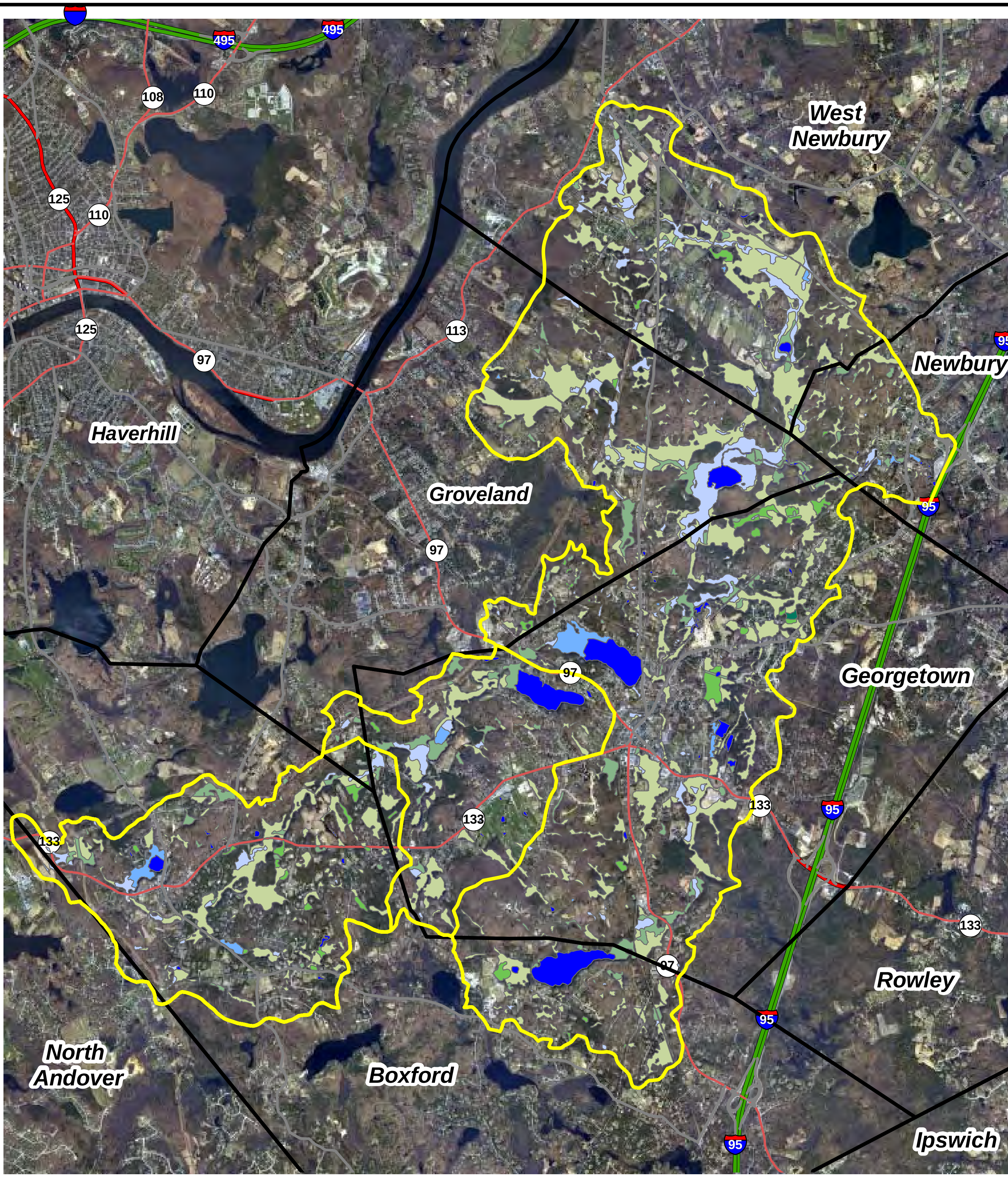
Fig. 2.2



Legend

Watersheds
Town Boundaries
Open Water
Deep Marsh
Shallow Marsh, Meadow or Fen
Shrub Swamp
Wooded Swamp Coniferous
Wooded Swamp Deciduous
Wooded Swamp Mixed Trees

*1:5,000 Color Ortho Imagery, MassGIS 2005
DEP Wetlands (1:12,000), MassGIS 2007

N

Feet
5,500

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DEP Wetlands
Parker River Clean
Watershed Association

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R:\8007 Parker River Clean Water Association\GIS\8.5x11 Wetlands

Fig. 2.3

is directly connected to centralized stormwater systems that directly discharge to surface waters is commonly called effective impervious area (EIA). It is this EIA that results in higher runoff volumes and peak flow rates as well as reduced recharge or baseflow. Runoff from small fragmented impervious areas that is not connected into organized drainage systems does not appreciably change the recharge versus runoff characteristics of the underlying surficial geology. The recharge loss from EIA in the water budget tool was determined based on a delineated area that is currently serviced by a storm sewer system. Storm sewers are present only for a small area in Georgetown, which is shown in Figure 2.4. All impervious area within the delineated storm sewer area was considered EIA. Stormwater from these EIAs is not available for aquifer recharge and thus is not included in the water budget recharge calculation.

2.2.3 Septic System Inputs

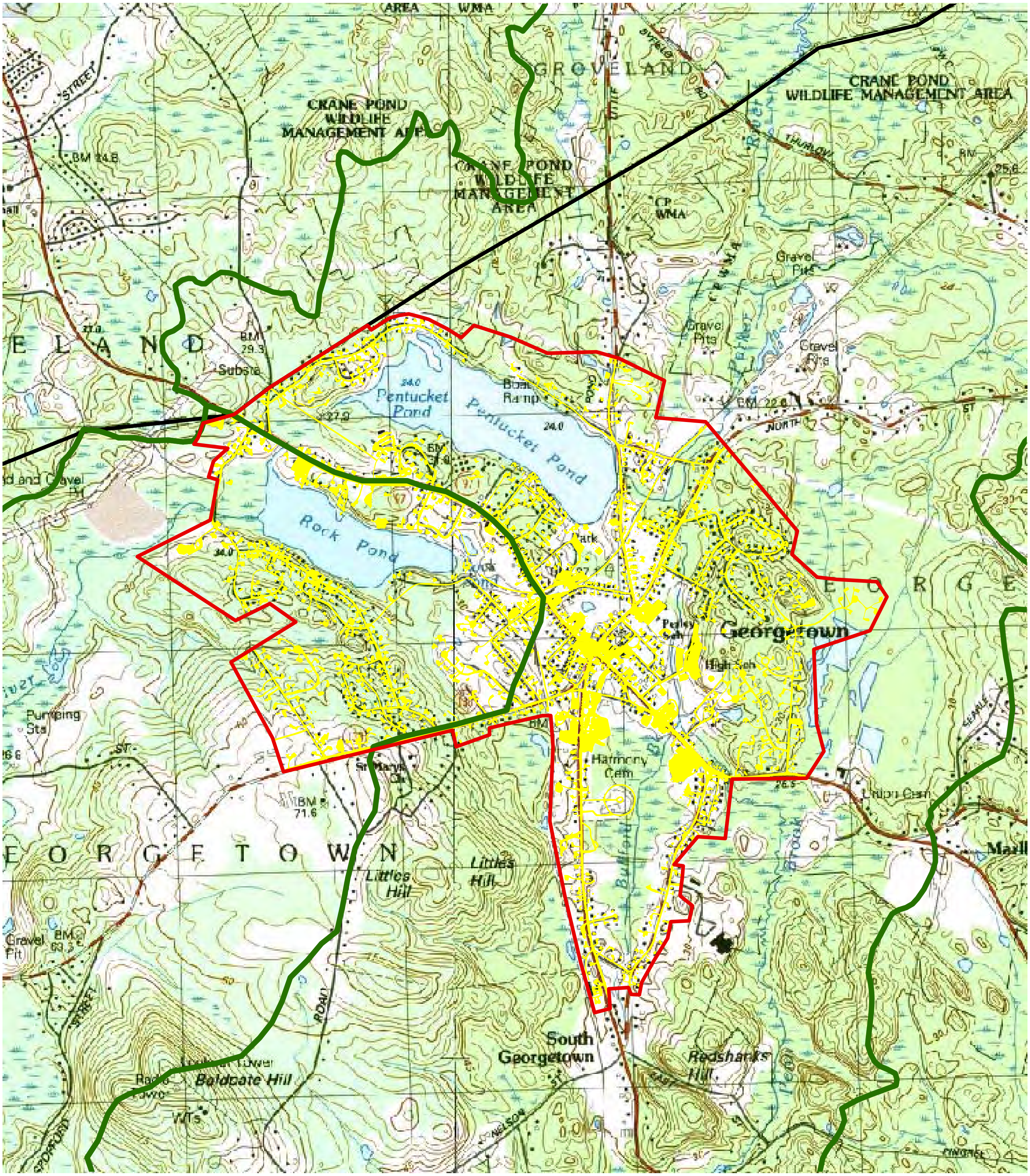
The only town in the watershed with public sewer is Groveland. The Town provided a map of the sewer main, which was used to identify the parcels serviced by public sewer. A 50-foot buffer was applied to the public sewer line map in GIS, and then merged with parcel data (provided by the Merrimac Valley Planning Commission) to capture all parcels that intersected the buffer. These areas were considered to be serviced by the public sewer system. The resulting service area map was then verified by the Town and is shown in Figure 2.5. The remaining parcels within the watershed were assumed to be serviced by private septic systems. Septic system inputs were estimated for both on-site septic systems and known on-site/small decentralized wastewater treatment plants for all areas that were not determined to be connected to public sewer systems.

Public sewer pipe networks frequently contain cracks that allow both groundwater inflow to the pipes and wastewater infiltration from the pipes into surrounding soils. This is commonly referred to as "inflow and infiltration" or "I & I." I & I volume assumptions are typically based on miles of sewer pipes. Given that there is less than a half a mile of sewer pipe in our study area, the impacts of I & I were not included in our analyses.

Using GIS, MassGIS land use data (1999; Figure 2.6) was applied to the areas served by private wastewater (septic systems) within each subwatershed. The land use categories that were included in the septic flow calculation include:

- Residential – Multi-family;
- Residential – 0.25 to 0.5 acre lots;
- Residential – Larger than 0.5 acre lots;
- Commercial, and
- Industrial.

The remaining MassGIS land use categories were either not located within the watershed or were assumed to have no significant septic flow contribution.



Legend

Watersheds

Boundary of Area Serviced by Storm Sewer

Effective Impervious Area

Town Boundaries

**Based on parcel and catch basin data supplied by Merrimack Valley Planning Commission*

N

Feet

2,000

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Effective Impervious Area
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Watershed Association

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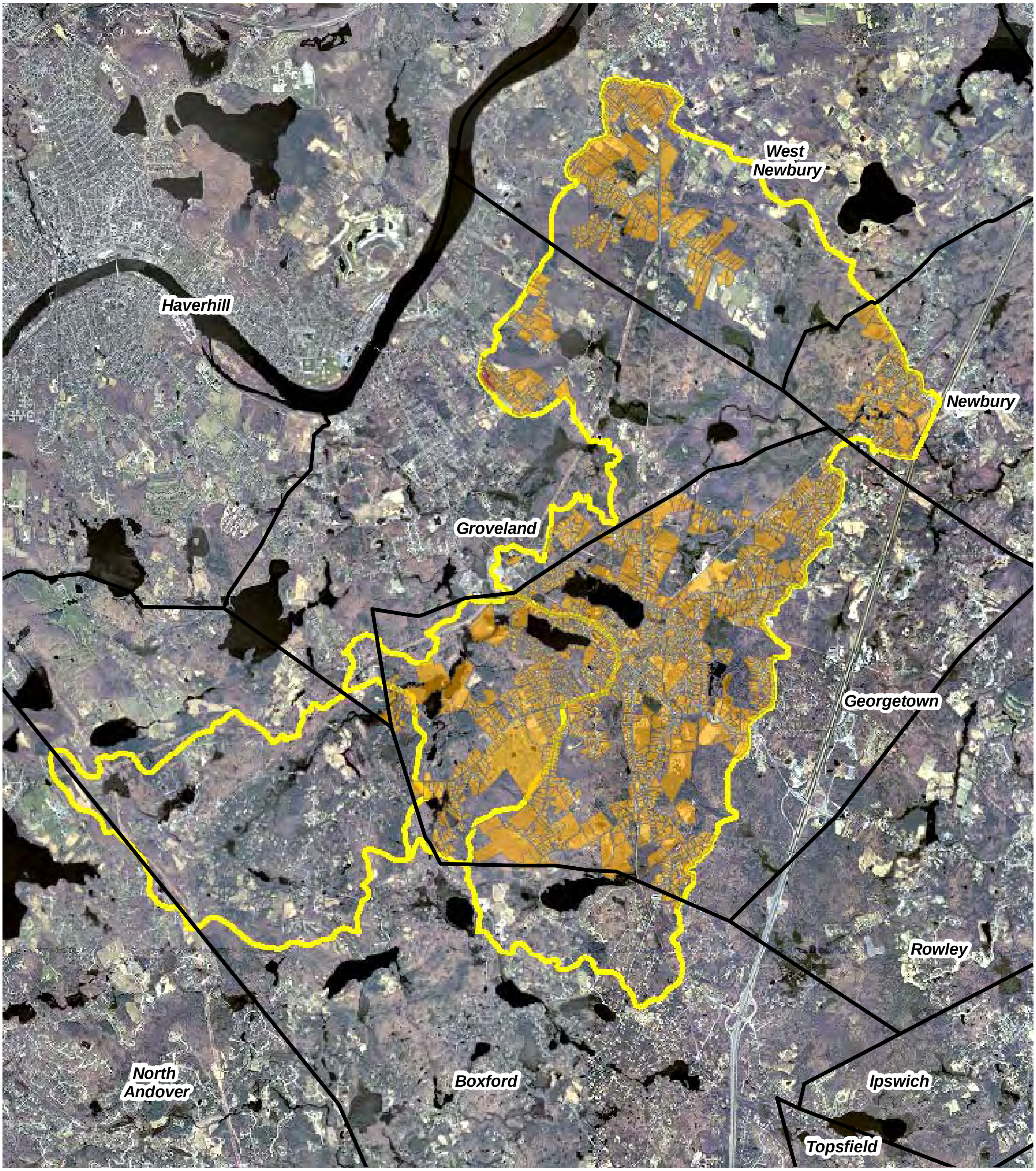
EIA 8.5x11

Fig. 2.4



Legend

*Based on parcel data supplied by Merrimack Valley Planning Commission
**1:5,000 Color Ortho Imagery, MassGIS 2005

- Watersheds
- Town Boundaries
- Sewer Service Areas
- Water Service Areas
- Both Water & Sewer Service Areas

N

Feet
6,000

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Water and Sewer Service Areas
Parker River Clean
Watershed Association

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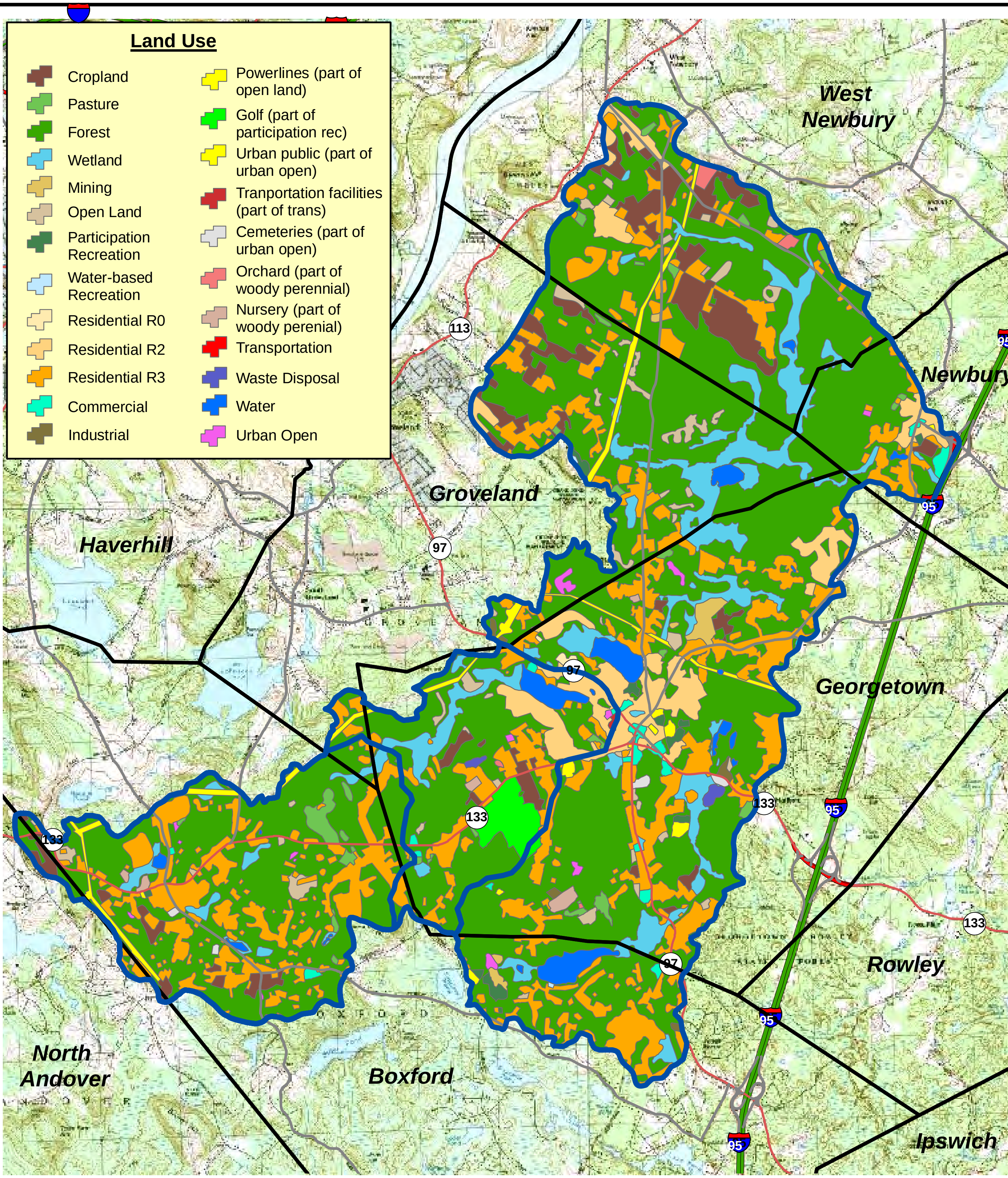
Fig. 2.5



*Land Use, MassGIS 2002

Legend

Watersheds

Town Boundaries

N

Feet

5,500

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Land Use
Parker River Clean Watershed Association

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Fig. 2.6

Residential

The three residential land use categories include: "Multi-family"; "¼ - ½ acre lots"; and "Larger than ½ acre lots." The acreage associated with each residential land use was first divided by the average lot size for each category, which was determined based on actual parcel data from the Merrimac Valley Planning Commission: 0.37 acres for "Multi-family lots"; 0.5 acres for "¼ – ½ acre lots"; and 1.8 acres for "Larger than ½ acre lots." This provides the estimated number of lots for each residential area. Next, the average occupancy rate for the watershed (2.57 people per household) was applied to each number of lots to determine the number of people per residential area (Census, 2000).

Finally, an average wastewater flow in gallons per capita per day (gpcd) was applied to each. The wastewater flow (gpcd) was calculated as the average water use (64 gpcd), based on Georgetown water use data for 2006, subtracted by 15 percent. On average, fifteen percent of household water use is estimated to be lost via outdoor water use (e.g., lawn watering) and therefore would not contribute to the wastewater effluent (USGS, 1982). The resulting assumption for wastewater flow per person is 54 gpcd.

Commercial

Within the MassGIS land use definitions, Commercial areas are defined as "general urban; shopping center." For the purposes of the study these areas were divided into three components: office, retail, and restaurant. The percentages of the total commercial area which each component comprises were estimated using US Census data as follow (Census, 2005):

- Office space: 60%;
- Retail space: 30%; and
- Restaurant space: 10%

According to 310 CMR 15.203 (Title 5), the wastewater design flows for each of these components are as follows:

- Office building: 75 gallons per day (gpd) per 1,000 gross square feet;
- Retail store: 50 gpd per 1,000 gross square feet; and
- Restaurant: 35 gpd per seat

These design flows were then divided by a factor of two, since Title 5 design flow calculations are generally about double actual flows (310 CMR § 15.203 (6)). Then, gross square footage was calculated for the entire commercial area within the subwatersheds. In order to calculate gross square footage, twenty percent of the commercial area was then assumed to be the building footprint (Cappiella and Brown, 2001). Based on site reconnaissance, it was determined that the average number of floors per commercial building is one (1) floor.

The percentage of each commercial component (office, retail and restaurant) was then applied to the resulting commercial gross square footage to provide a gross square footage value for office space, retail space, and restaurant space. The values for office space and retail space were each divided by 1,000 square feet (Title 5) and then multiplied by 50% of the wastewater design flow (37.5 gpd and 25 gpd respectively) to determine a total wastewater flow for each component.

According to Title 5, restaurant wastewater flow is based on number of seats. An average of 29 seats per 1,000 gross square feet of restaurant space was used for the calculation (NRBL, 2008). The restaurant space gross square footage was multiplied by 0.029 (29 seats per 1,000 square feet) and multiplied by 50% of the wastewater design flow (17.5 gpd).

Industrial

Industrial wastewater flow was calculated in a similar fashion to commercial wastewater flow. All industrial area was assumed to have the same flow per 1,000 gross square feet as office space. The water withdrawal volumes and wastewater discharge volumes were calculated using the same equations as office space, described above. Based on site reconnaissance, it was determined that the average number of floors per industrial building is the same as commercial buildings (1). The value for industrial space was then divided by 1,000 square feet (Title 5) and multiplied by 50% of the wastewater design flow (37.5 gpd) to determine a total industrial wastewater flow.

2.2.4 Groundwater Discharge Permit Inputs

Groundwater Discharge Permits (GWDP) are generally required by DEP for all groundwater discharges that are greater than 10,000 gpd. A set of GWDP data collected from DEP was utilized to determine the wastewater flow associated with all of these discharges. This compiled dataset included the total annual discharge from these facilities for a given year, based on Daily Monitoring Reports provided to DEP for the year 2006. This dataset was incorporated in the GIS model to determine the discharge flow associated with GWDPs in each subwatershed.

Two GWDPs were identified within the study area. GWDP data were used for the Little’s Hill Condominiums and Georgetown Housing Authority, which discharge an average of 4,287 and 6,213 gpd, respectively. In addition, the Georgetown Club and the High School/Elementary Schools are each known to have large Title 5 systems that do not quite meet the threshold to require a GWDP with design flows of 9,905 and 15,000 gpd (this latter flow being grandfathered prior to the current Title 5 regulations), respectively. Flows from these systems were estimated in the water budget by assuming that actual flows are 50% of the permitted Title 5 flows.

2.2.5 Private Drinking Water Well Withdrawals

GIS was used to estimate areas serviced by public drinking water systems. Line data indicating the public water mains were collected from the individual communities within the watershed. As with the sewer service areas, a 50-foot buffer was applied to the public water lines in GIS, and then merged with parcel data (MVPC, 2008) to capture all parcels that intersected the buffer. These areas were considered to be serviced by a public water system. The public water service areas were hand-checked to ensure that undeveloped lots were not included, and those maps for Georgetown, Newbury (Byfield) and Groveland were verified by the applicable water departments. The parcels in the watershed that are serviced by public water are shown in Figure

2.5. The remaining parcels within the watershed were assumed to be serviced by private drinking water wells.

Using GIS, MassGIS land use data (1999) were applied to the areas served by private water sources within each subwatershed. The land use categories that were included in the withdrawal calculation include:

- Residential – Multi-family;
- Residential – 0.25 to 0.5 acre lots;
- Residential – Larger than 0.5 acre lots;
- Commercial; and
- Industrial

The remaining MassGIS land use categories were either not located within the watershed or were assumed to have no significant private drinking water well withdrawals.

Residential

The three residential land use categories include: "Multi-family"; "¼ - ½ acre lots"; and "Larger than ½ acre lots." The acreage associated with each residential land use was first divided by the average lot size for each category, which was determined based on parcel data from the Merrimac Valley Planning Commission: 0.37 acres for "Multi-family lots"; 0.5 acres for "¼ – ½ acre lots"; and 1.8 acres for "Larger than ½ acre lots." This provides the estimated number of lots for each residential area. Next, the average occupancy rate for the watershed (2.57 people per household) was applied to each number of lots to determine the number of people per residential area (Census, 2000).

Finally, an average water use estimate (64 gpcd) was applied to the number of people within each residential area to determine the total estimated private drinking water withdrawal volume.

Commercial and Industrial

Private water withdrawal estimates for commercial and industrial land uses are determined using the same method as described under the septic system section, except that total drinking water withdrawal volumes are estimated by multiplying the wastewater design flow by 60% (USGS, 1982).

2.2.6 Water Management Act Withdrawals

Water withdrawals above 100,000 gpd require a permit under the MA Water Management Act (WMA). WMA permit and registration data collected from DEP were used to determine the major water withdrawals within each subwatershed. The permit information and Annual Statistical Reports as required under the Water Management Act, provided by DEP, were used in combination with the MassGIS Public Water Supply data layer to determine WMA permitted and registered public water supply and non-public water supply withdrawals, such as industrial withdrawals and golf irrigation well withdrawals, within each subwatershed (see Figure 2.2 for well locations). In the upper Parker River Watershed, there are three public water supply wells in active use in Georgetown with average withdrawals (based on the year 2006) of 450,526, 189,764, and 16,698 gpd; and one in Byfield (Newbury) with an average withdrawal of 130,632 gpd.

Georgetown Club irrigation withdrawals are currently included in the model even though they do not have a WMA permit. Based upon the DEP Golf Course Water Use Policy (June, 2000), any existing golf course with 35 acres of irrigated turf or greater is presumed to use enough water to require a WMA permit. Our rough estimation of irrigated turf for the Georgetown Club, using GIS measurements of the 2005 aerial photography, is approximately 80 acres, or more than two times the area presumed to require a WMA permit. Using the methodology described in the Golf Course Water Use Policy, 80 acres of irrigated turf results in approximately 28.9 million gallons per year (MGY) of irrigation use, assuming an 18-week irrigation season. The Georgetown Club presented the Town with a water use estimate of approximately 15 MGY. Because it was unclear where, or if, those volumes reported by the Club were metered, and because the reported numbers seemed low, the estimated water use described above was used in the model.

2.3 Water Budget Results

Table 2-1 shows a summary of relevant outputs from the water budget method applied to the subwatersheds. Overall, the analysis shows an 8% water deficit throughout the entire modeled upper Parker River Watershed. The RIFLS Route 97 Subwatershed shows the greatest water deficit at 19%, while the mostly undeveloped Uptack Road Subwatershed results show neither an increase nor a decrease in groundwater recharge.

Table 2-1. Annual Water Budget Results for Three Nested Subwatersheds in the Upper Parker River Watershed

	Subwatersheds		
	USGS Gage	**RIFLS Route 97**	**Uptack Road**
Total Area (acres)	13,715	4,188	2,578
Water Inputs (MGY)			
Estimated effluent from Groundwater Discharge Permit data	8.4	1.8	0.0
Estimated effluent from septic systems	163	47	22
Estimated natural recharge	4,074	1,454	954
Total Inputs	**4,246.6**	**1,502.5**	**975.9**
Water Outputs (MGY)			
Estimated withdrawal volume from private wells	76	32	25
Estimated withdrawal volume from WMA Permit data	316.4	268.7	0.0
Estimated Losses to I&I	0	0	0
Total Outputs	**392**	**301**	**25**
Existing Net Recharge			
Existing Baseflow Estimate (MGY)	3,854.6	1,201.5	951.3
Existing Baseflow Estimate (cfs)	16.4	5.1	4.0
Existing Net Recharge			
Pre-Development Baseflow Estimate (MGY)	4,196	1,484	954
Pre-Development Baseflow Estimate (cfs)	17.8	6.3	4.1
Percent Change in Net Recharge	**-8%**	**-19%**	**0%**

To further evaluate the subwatersheds and pinpoint the most affected area, HW looked at the water budgets for the drainage areas between gage stations. For example, HW took the difference of the values from Table 2-1 for the USGS Gage and RIFLS Route 97 Subwatersheds to determine the change in net recharge for only the land between those gages. The same was done for the area between the RIFLS Route 97 gage and the Uptack Road crossing. The results are summarized in Table 2-2. The most important result from Table 2-2 is that the drainage area between the RIFLS Route 97 gage and the Uptack Road crossing has a 53% water deficit. This is not surprising since the majority of the public water supply groundwater wells are located within this area. The Georgetown Club irrigation withdrawals are also located in this same section of the watershed.

Table 2-2. Annual Water Budget Results for Drainage Areas Between Gage Stations

	Drainage Area Between Route 97 Gage and USGS Gage	Drainage Area Between Uptack Rd and Route 97 Gage
Total Area (acres)	9527.2	1610.0
Water Inputs (MGY)		
Estimated effluent from Groundwater Discharge Permit data	6.6	1.8
Estimated effluent from septic systems	117.7	25.2
Estimated natural recharge	2619.8	499.5
Total Inputs	**2,744.1**	**526.6**
Water Outputs (MGY)		
Estimated withdrawal volume from private wells	43.3	7.6
Estimated withdrawal volume from WMA Permit data	47.7	268.7
Estimated Losses to I&I	0.0	0.0
Total Outputs	**91**	**276.3**
Existing Net Recharge		
Existing Baseflow Estimate (MGY)	2653.1	250.3
Existing Baseflow Estimate (cfs)	11.3	1.1
Existing Net Recharge		
Pre-Development Baseflow Estimate (MGY)	2712.8	529.1
Pre-Development Baseflow Estimate (cfs)	11.5	2.2
Percent Change in Net Recharge	**-2%**	**-53%**

2.4 Alternatives Analysis

In order to investigate potential ways to reduce the water deficit, and thus increase the baseflow in the RIFLS Route 97 Subwatershed, several alternatives were analyzed based on discussions with Town and State officials, as well as a regional planning agency.

2.4.1 Georgetown Seasonal Water Use Ratio

The first alternative considered was reducing the public water use ratio in the Town of Georgetown. The current summer: winter water use ratio is 2:1 (estimate from the Georgetown Water Department). A State-wide water conservation goal is to reduce this ratio to 1.2:1. By maintaining the amount of water used during the 2006 winter months (79.9 MGY), HW calculated the future summer water use based on the proposed ratio (95.9 MGY). This alternative decreases Georgetown's public water use by 27%, reducing it from 239.8 MGY to

175.9 MGY. The resulting effect on the water budget is that the water deficit for the drainage area between the RIFLS Route 97 gage and the Uptack Road crossing decreased from 53% to 41% - a significant change of 12%.

2.4.2 Georgetown Potential Sewer Area

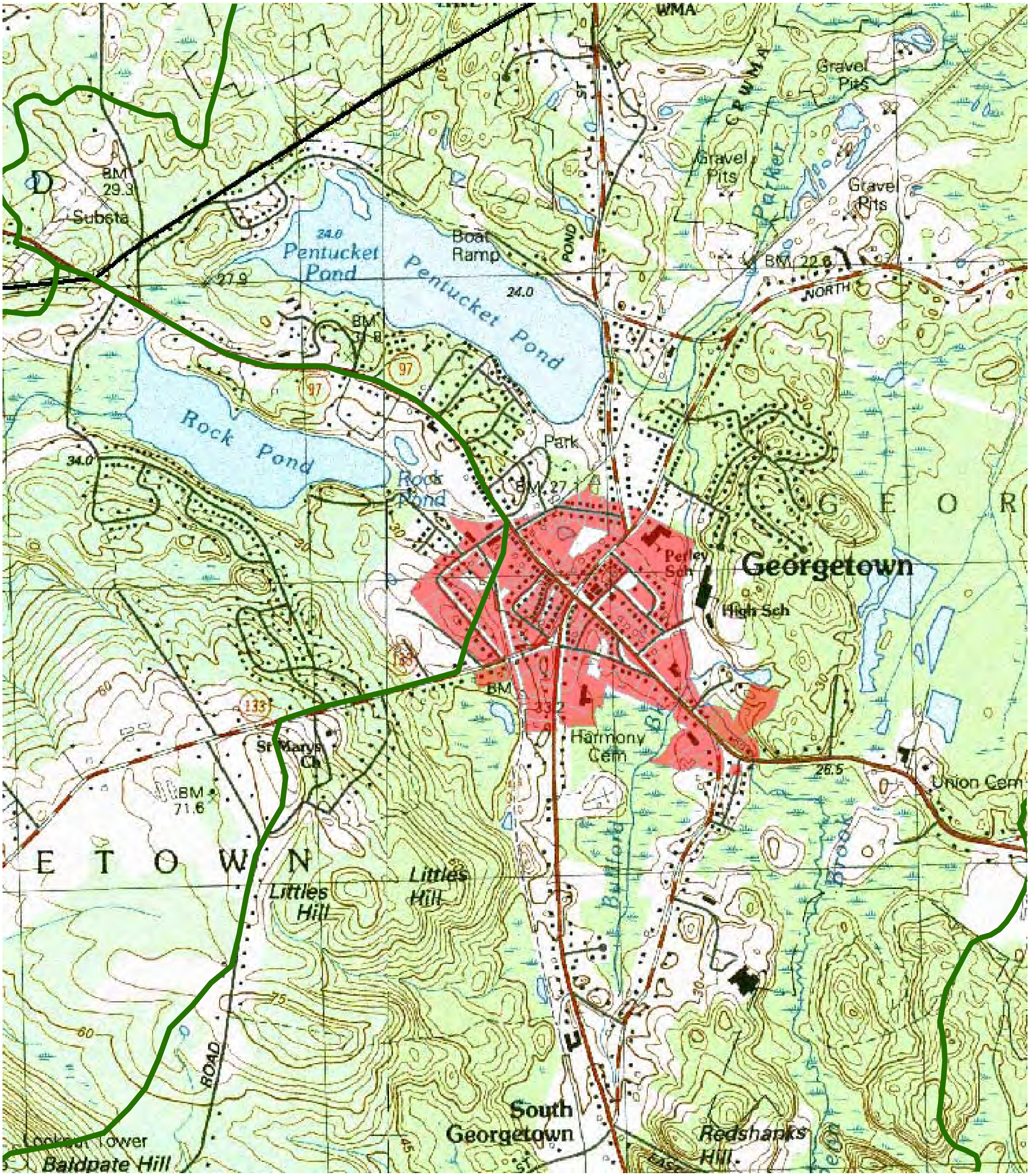
The second alternative considered was adding public sewer service for an area of downtown Georgetown, as determined by Town representatives, and discharging the treated wastewater in an appropriate location upgradient of the RIFLS Route 97 gage. Figure 2.7 shows the area of the potential sewer in Georgetown and Figure 2.8 shows possible discharge locations for the treated wastewater based on soils, wetlands, open space, and existing drinking wells. These potential locations are not based on any field testing, do not consider property ownership concerns, and may or may not be feasible.

Adding sewered area affects the water budget by decreasing the volume of recharge from septic systems in the proposed sewer service area and increasing the volume of GWDP recharge at an upstream discharge location, effectively moving 13 MGY of water back upstream to support baseflow in the most stressed river reach. The resulting effect on the water budget is that the water deficit for the drainage area between the RIFLS Route 97 gage and the Uptack Road crossing has decreased from 53% (existing conditions) to 50% - a change of 3%.

The potential sewer area described above provides only a minimal offset to the water deficit that currently exists. A greater offset could be accomplished if a larger area of Georgetown were sewered. One option is that a future sewer area encompass the same area as the current storm sewer system. Figure 2.9 illustrates a potential sewer area that includes not only the downtown area, but also the developed areas around Pentucket and Rock Ponds. This alternative increases the upstream transfer of treated wastewater from 13 MGY to 47 MGY. The resulting effect on the water budget is that the water deficit for the drainage area between the RIFLS Route 97 gage and the Uptack Road crossing has decreased from 53% (existing conditions) to 47% - a more significant change of 6%.

2.5 Water Budget Discussion

The water budget analysis for the upper Parker River Watershed indicates that a large water deficit exists, particularly in the drainage area between the RIFLS Route 97 gage and the Uptack Road crossing (53%), where significant water supply and irrigation withdrawals occur with minimal return flow. The most significant improvement to this deficit can be achieved by modifying water use patterns, particularly in respect to the summer/winter use ratio. Reducing this ratio from 2:1 to 1.2:1 reduces the water deficit from 53% to 41%. In addition, adding public sewer to the most densely populated portions of Georgetown can also reduce the deficit. The smaller potential sewer area initially proposed by the Town has a relatively minor impact on the water budget. However, if a larger sewer area (corresponding to the current area with managed stormwater infrastructure) is implemented along with the water use reduction, the deficit can be decreased to 35%. Inclusion of stormwater in the volume transferred upstream for GWDP discharge would further reduce the deficit by an unquantified, but likely significant,



Legend

**Based on parcel data supplied by Merrimack Valley Planning Commission*

Watersheds

Potential Sewer Area - Option 1

Town Boundaries

N

Feet

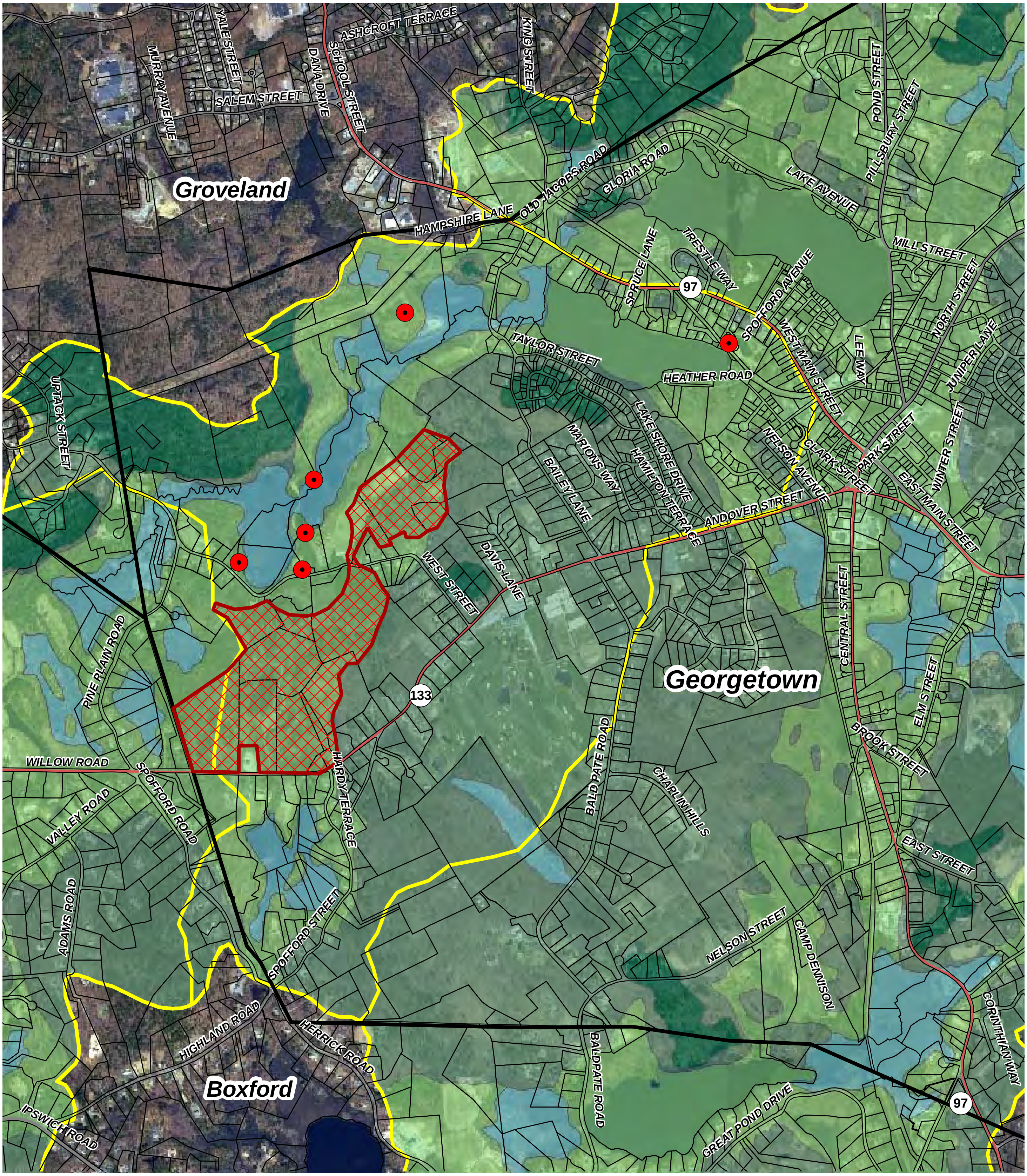
1,500

Horsley Witten Group
phone: 508-833-6600
www.horsleywitten.com

Potential Sewer Area - Option 1
Parker River Clean
Watershed Association

6/20/08 mw
R:\8007 Parker River Clean Water Association\GIS\
Proposed Sewer 8.5x11

Fig. 2.7



Legend

- Watersheds
- Town Boundaries
- Swamp Deposits
- Till
- Shallow/Exposed Bedrock
- Floodplain Alluvium
- Glacial Stratified Deposits, Coarse
- Drinking Water Wells
- Potential Area for WWTP Discharge

**Based on parcel data supplied by Merrimack Valley Planning Commission*
**1:5,000 Color Ortho Imagery, MassGIS 2005*
Surficial Geology (1:24,000), MassGIS 2007

N

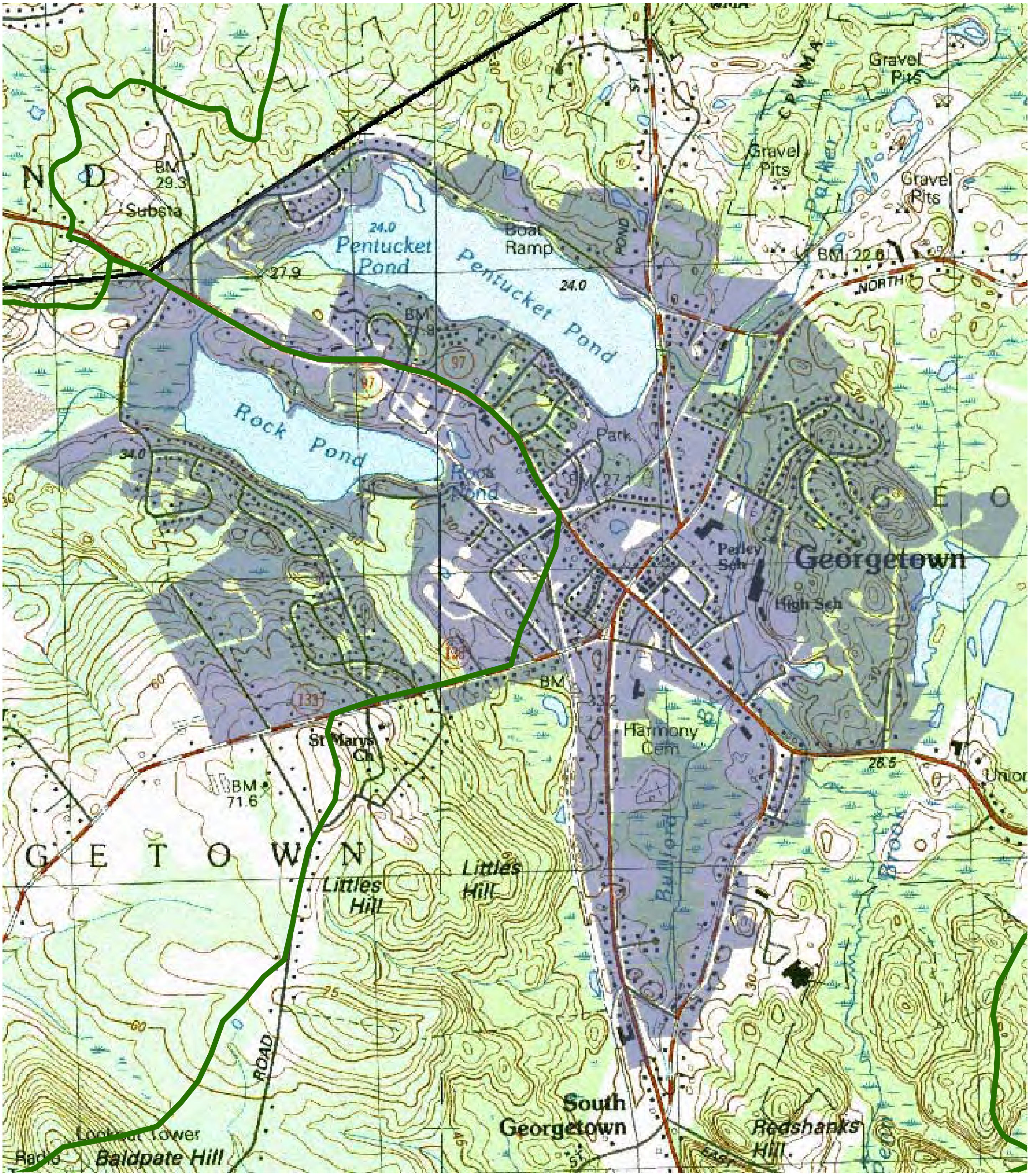
2,000 Feet

Horsley Witten Group
phone: 508-833-6600
www.horsleywitten.com

Potential WWTP Discharge
Parker River Clean
Watershed Association

6/20/08 mw
R:\8007 Parker River Clean Water Association\GIS\
8.5x11 SurfGeo

Fig. 2.8



Legend

*Based on parcel data supplied by Merrimack Valley Planning Commission

Watersheds

Potential Sewer Area - Option 2

Town Boundaries

N

Feet

1,500

Horsley Witten Group
phone: 508-833-6600
www.horsleywitten.com

Potential Sewer Area - Option 2
Parker River Clean
Watershed Association

6/20/08 mw
R:\8007 Parker River Clean Water Association\GIS\
Recom Sewer 8.5x11

Fig. 2.9

amount. Another possibility would be to consider wastewater re-use at the Georgetown Club for irrigation. That option would not only add the increased recharge described above, but would also eliminate the Club's irrigation withdrawals, bringing the central watershed area closer to balance.

3.0 STREAM DEPLETION ANALYSIS

In order to focus on potential seasonal hydrologic impacts to the upper Parker River that may occur specifically from public water supply withdrawals, and on a shorter time frame than can be captured by the annual water budget analysis, HW performed a Stream Depletion analysis using the USGS model STRMDEPL (Barlow, 2000). In contrast to the annual water budget approach in Task 2, which is more of a long-term impact analysis, the STRMDEPL analysis evaluates impacts to the stream flow that occur within seasonal timeframes, or even shorter. This model, also used by the USGS for a study of the Ipswich River (Zarriello and Ries, 2000), provides an evaluation of the impact of the Georgetown wells on Parker River flows caused by time-varying pumping of the wells. The STRMDEPL model uses well pumping rates, distance from the well to the stream, and aquifer characteristics as input factors. It takes into account two components: 1) groundwater discharge that is captured by the well during pumping, and 2) induced infiltration in the stream bed caused by the pumping of the well when the aquifer is drawn down below the stream bed.

3.1 Stream Depletion Methodology

Methodology for the STRMDEPL model was adopted from that documented by Barlow (2000) and used for the Ipswich River (Zariello and Ries, 2000). Time-variable impacts on the river from current pumping conditions were evaluated and estimated for the three active Georgetown wells - the William Marshall Well, the Commissioners' Well and the Ronald Marshall (Duffy) Well – and for the Byfield Forrest Street well. Aquifer and well characteristics were determined for the analyzed wells using available hydrogeologic information from the well permitting and development documents (Appendix B), and total monthly well withdrawal data, provided by the Georgetown and Byfield Water Departments (Appendix B). Total monthly well withdrawals were used to estimate daily pumping rates for each of the wells by evenly dividing each month's total withdrawal into the number of days in that month.

In addition to the daily pumping rates, the aquifer and well characteristics that were used as input variables to the model include:

- Aquifer transmissivity (T);
- Specific yield (Sy);
- Distance between well and river (center of channel for this study) (XWELL);
- Diffusivity of the aquifer (=T/Sy) (DIFFUS);
- Presence of streambank materials (IBANK);
- Streambank leakage (SLEAK);
- Number of pumping days prior to start of analysis (INTIME);
- Pumping rate prior to start of analysis (WWINT); and
- Number of pumping days in analysis (NPD).

3.2 Stream Depletion Base Model Runs

The STRMDEPL model was first run considering the base assumptions previously used by the USGS in their Ipswich River work. The input variables used for the stream depletion analysis of each subject well are provided in Table 3-1.

Table 3-1. Parker River Stream Depletion Input Variables – Base Conditions Run

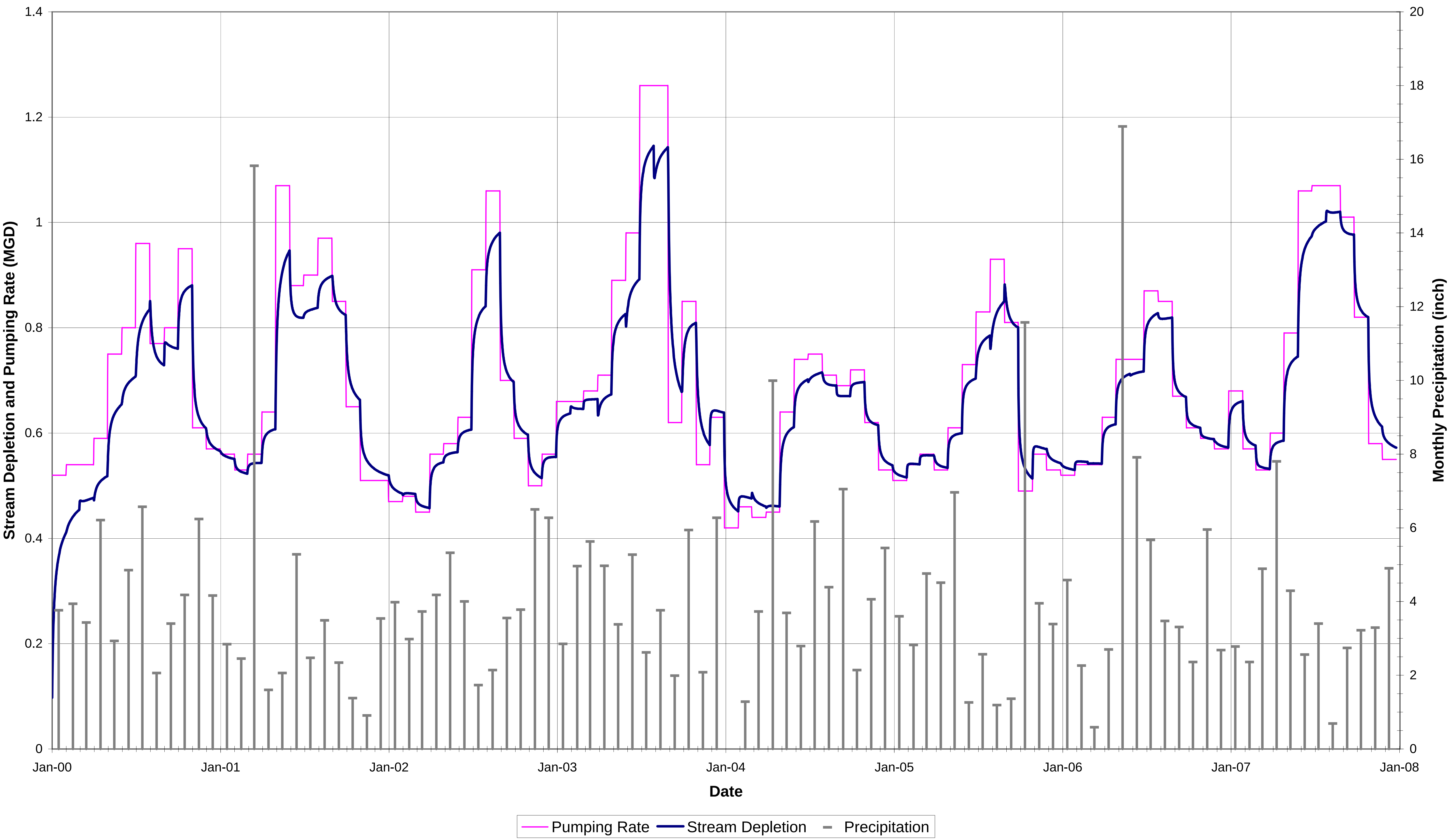
Input Variable	Marshall Well	Commissioner's Well	Duffy Well	Forrest St Well
T (ft^2/d)	8,000	3,000	14,500	428
Sy	0.02	0.24	0.03	0.0007
XWELL (ft.)	500.0	600.0	250.0	125
DIFFUS (ft^2/s)	0.463	0.463	0.463	0.463
IBANK (0/1)	0	0	0	0
SLEAK (ft.)	0.0	0.0	0.0	0.0
INTIME (days)	10000	10000	10000	10000
QWINIT (cfs)	0.000	0.000	0.000	0.000
NPD (days)	2922	2922	2922	2191
Period of Pumping/Depletion	2000-2007	2000-2007	2000-2007	2002-2007*

*Pumping data for the Byfield Forrest Street well was available for a shorter time frame than the four Georgetown wells

Graphical outputs of the estimated time varying stream depletion resulting from pumping of each subject well under the base conditions are shown in Appendix C (Figures C1 – C4). As shown in the figures, the estimated stream depletion quickly approaches the monthly pumping rate of each well, and the average daily stream depletion over the entire period of record is close to the average daily pumping rate. Figure 3.1 shows the cumulative river depletion from the pumping of all four subject wells.

These base model results suggest that pumping withdrawals from these wells very quickly and directly reduce stream flow in the River. Given the close proximity of the wells to the river, this is not a great surprise but certain assumptions of the base model likely tend to overestimate the magnitude of the impact.

Figure 3.1 Cumulative Georgetown Well Stream Depletion



Daily well withdrawals are based on total monthly withdrawals for 2000-2007 (minimum = 0.00 MGD; maximum = 0.89 MGD; average = 0.25 MGD)

3.2 Stream Depletion Sensitivity Analysis

The base model assumptions (namely that both the stream and the well fully penetrate the aquifer and that no low permeability streambed materials exist to slow stream aquifer communication) tend to simulate a maximum interaction between the well and the stream in the model and, therefore, show greater impacts. In recognition of this fact, other sensitivity runs were conducted to evaluate the significance of partial stream penetration and the presence of low conductivity streambed materials.

Following the methodology outlined by Barlow (2000), a representative streambank leakance term was estimated based on a description of the stream bottom in the vicinity of the Georgetown wells provided by the Georgetown water superintendent and best professional judgment. While the Parker River streambed in general is described to be relatively sandy, the impounded area in the vicinity of the supply wells has some finer grained wetland sediments. The streambed leakance is, therefore, likely moderate. The thickness of lower permeability streambed materials was assumed to be 1 foot, the hydraulic conductivity of the low permeability streambed materials was assumed to be 1 foot per day, and the surrounding aquifer properties used were those reported in the respective well permitting reports.

Barlow (2000) defines streambank leakance for use in the STRMDEPL model as aquifer conductivity multiplied by the streambed thickness, divided by streambed conductivity. The resulting leakance terms calculated for the Marshall, Commissioner's, Duffy, and Forrest Street wells, respectively, are 50, 22, 26, and 8 feet. The use of these leakance terms in sensitivity model runs had minimal impact on the resulting estimated stream depletions and those model output graphs are, therefore, not shown.

A sensitivity analysis of the impact of partial stream penetration showed a more significant impact on the modeled stream depletion results. The STRMDEPL model assumes that both the well and the stream completely penetrate the aquifer, allowing for complete communication between the two. The more common real-world situation is that the stream barely penetrates the top of the aquifer and the well is screened across a small portion of the aquifer at greater depth. The result of this geometric relationship is that the hydraulic connection between the well and the stream is reduced.

Barlow (2000) describes how partial penetration is accounted for in STRMDEPL by increasing the actual distance from the well to the stream with an effective distance that is proportional to the effect of partial penetration. Spalding and Khaleel (1991) give guidance on calculating an effective distance to account for partial penetration. Figure 3 from Spalding and Khaleel is a graph that allows the user to estimate effective distances based upon stream size and aquifer thickness. Using aquifer properties from the supply well permitting documents and estimates of stream properties, effective distances were calculated for the subject wells. Effective distances are shown in Table 3-2.

Table 3-2 Effective Distances from Wells to River

Well	Aquifer Thickness (ft) *	Estimated Stream Depth (ft)	Actual Distance (ft)	Partial Penetration Factor	Effective Distance (ft)
Marshall	59	4	500	4X	2,000
Commissioner's	27	4	600	4X	2,400
Duffy	55	4	250	2.3X	575
Forrest St	20	4	125	2.2X	275

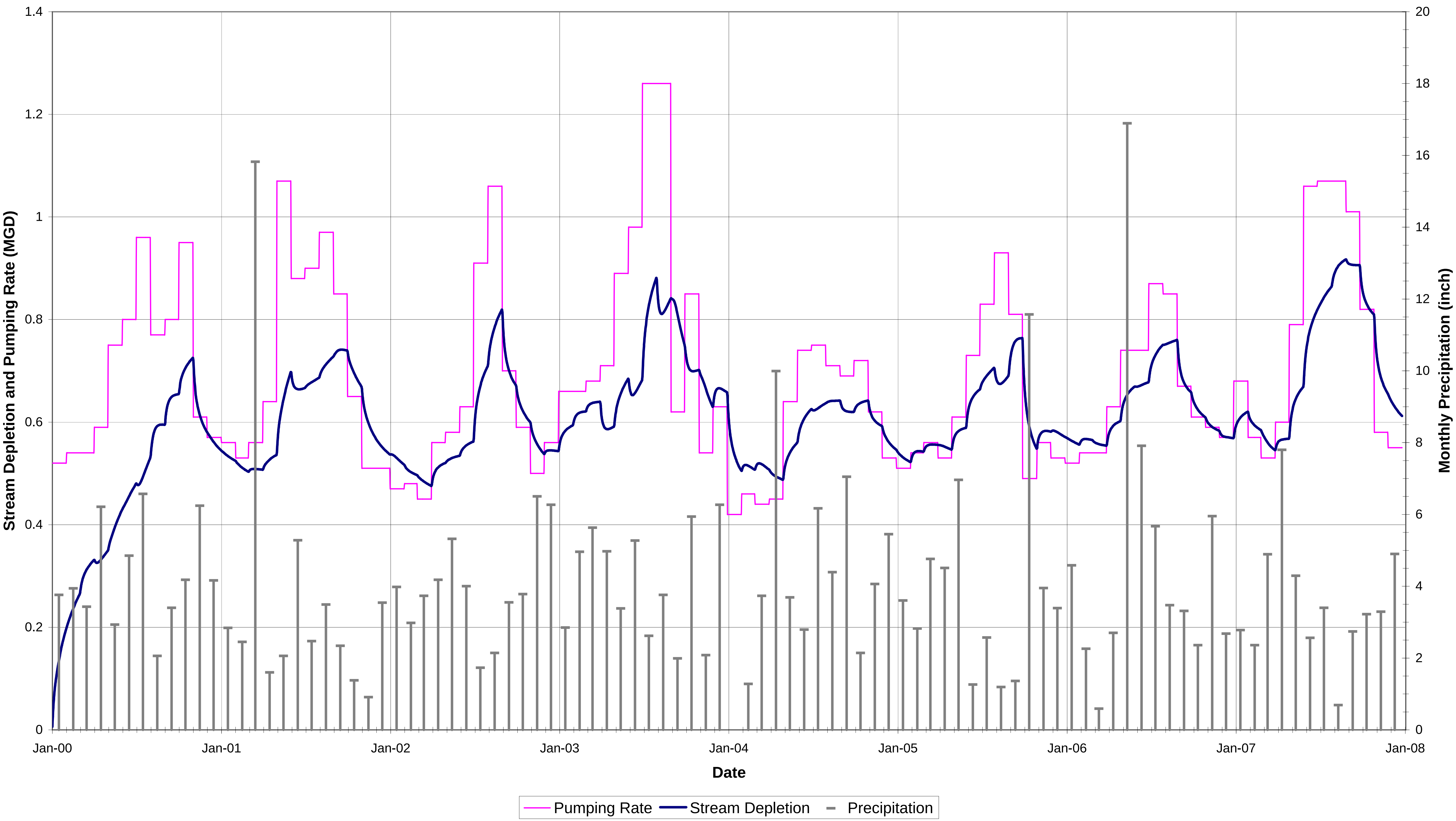
* From well permitting and development documents except estimated overburden thickness for Forrest Street bedrock well

STRMDEPL model output graphs for individual wells including the effects of both partial penetration and low permeability streambed materials are shown in Appendix C (Figures C5-C8). Figure 3.2 shows the cumulative river depletion from the pumping of all four subject wells including the effects of both partial penetration and low permeability streambed materials. The effect of partial penetration is to reduce the simulated stream depletion to approximately half to two thirds of the pumping rate. A lag time is also introduced where peak stream depletions occur weeks after peak pumping periods. Because the modeled effects of low permeability streambed materials is minimal, the STRMDEPL graphs accounting for partial penetration alone are nearly identical to the graphs including the combined effects of partial penetration and low permeability streambed materials. Only the graphs showing the combined effects are included here. These sensitivity scenario model results that consider partial penetration factors are considered more reasonable than the initial base model runs.

3.3 Stream Depletion Alternatives Analysis

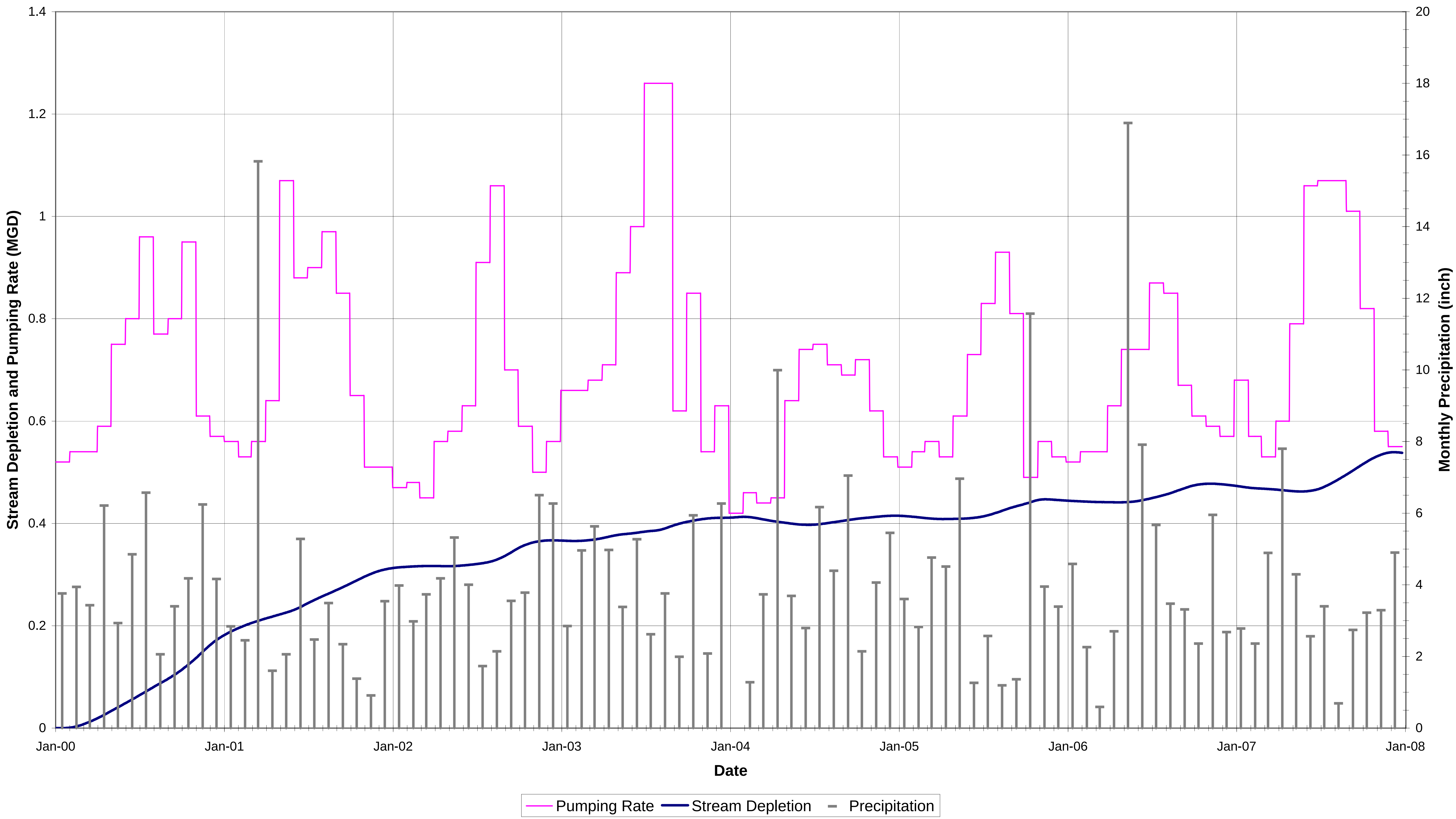
To further investigate the importance of proximity of the wells to the river, another set of STRMDEPL models were created and run moving each well a hypothetical 2,000 feet from the river. It is understood that there may not be good locations to move the wells 2,000 feet or greater away from the river and that such a venture would be costly, but this is a hypothetical analysis for informational purposes only. Except for distance, all other model input factors for each well were held the same, including correcting for partial penetration with effective distance and the presence of low permeability streambed materials. Table 3-3 lists the hypothetical distance factors for each well. Sand and Gravel deposits mapped in MassGIS are located up to 5,000 feet from the river in some areas of the watershed but only approximately 1,000 feet in the area of the Georgetown wells.

Figure 3.2 Cumulative Georgetown Well Stream Depletion
Corrected for Partial Stream Penetrations and Semipervious Streambank



Daily well withdrawals are based on total monthly withdrawals for 2000-2007 (minimum = 0.00 MGD; maximum = 0.89 MGD; average = 0.25 MGD)

Figure 3.3 Hypothetical (2,000 ft from river) Cumulative Georgetown Well Stream Depletion Corrected for Partial Stream Penetrations and Semipervious Streambank



Daily well withdrawals are based on total monthly withdrawals for 2000-2007 (minimum = 0.00 MGD; maximum = 0.89 MGD; average = 0.25 MGD)

Table 3-3 Effective Distances from Hypothetical Wells to River

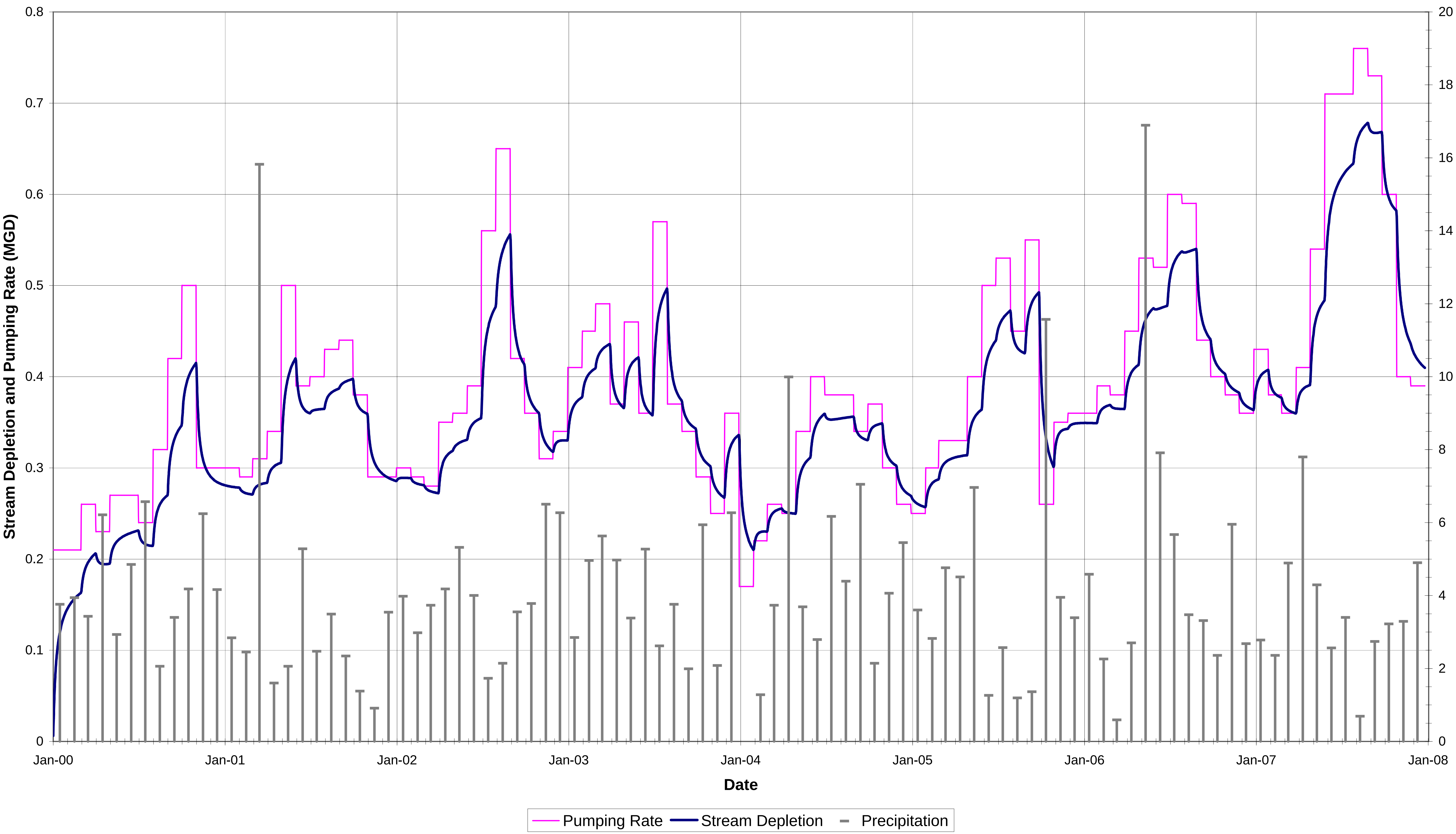
Well	Actual Distance (ft)	Hypothetical Distance (ft)	Partial Penetration Factor	Hypothetical Effective Distance (ft)
Marshall	500	2,000	4X	8,000
Commissioner's	600	2,000	4X	8,000
Duffy	250	2,000	2.3X	4,600
Forrest St	125	2,000	2.2X	4,400

Stream depletion graphs for each individual hypothetical well are shown in Appendix C (Figures C9-C12), and a cumulative impact graph for all four hypothetical wells is shown in Figure 3.3. Streamflow depletion from the hypothetical wells is significantly reduced from those estimated for the actual wells in the watershed. A very significant lag time is also evident where streamflow depletions respond over a period of years to broad trends in pumping rather than individual pumping periods. This lag time is evident both as pumping increases (and stream depletion remains relatively lower) and as pumping decreases (and stream depletion remains relatively higher). The effect of this significant lag time is to distribute stream depletion impacts evenly throughout the year, rather than concentrating them during summer peak pumping periods when the river is already naturally stressed.

The STRMDEPL model was also used to evaluate the hypothetical affect of summer water conservation on estimated stream depletion. The same hypothetical reduction of summer pumping described above in the section 2.4 water budget alternatives analysis was used here for this analysis. Namely, actual monthly summer pumping volumes were reduced proportionally to represent a hypothetical change in the summer/winter water use ratio from 2:1 (current) to 1.2:1 (recommended). This STRMDEPL model alternative analysis was only run for the Georgetown Duffy Well as a demonstration. Figure 3.4 shows the current conditions pumping and STRMDEPL model results for the Duffy Well and Figure 3.5 is the same except for its incorporation of summer water conservation.

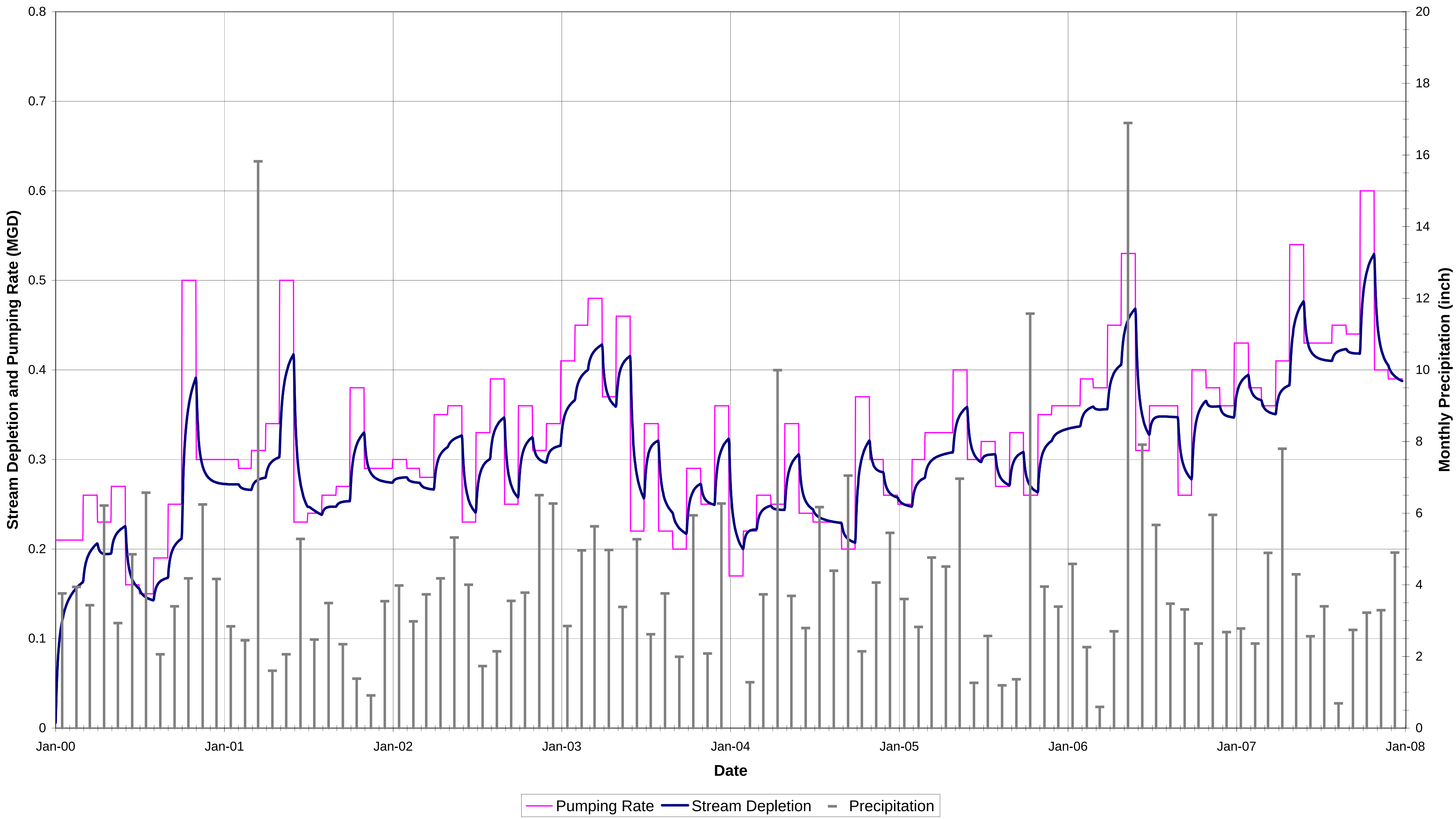
The affect of summer water conservation is to reduce the overall pumping volumes during the summer months. The estimated stream depletion is shown to be reduced proportionally. In other words, if the pumping in a given month is reduced by 40% (relative to current conditions), the estimated stream depletion is also reduced by approximately 40% (relative to the stream depletion estimated for current conditions). This does NOT mean that a direct volumetric relationship holds true (e.g. 1 MGD of pumping reduction equivalent to 1 MGD of stream depletion reduction). The relationship of pumping to stream depletion is complex and not one to one; that is the point behind using the STRMDEPL model. This proportional relationship shown for the Duffy well is likely to hold true for the other wells as well.

Figure 3.4 Duffy Well Stream Depletion
Corrected for Partial Stream Penetrations and Semipervious Streambank



Daily well withdrawals are based on total monthly withdrawals for 2000-2007 (minimum = 0.00 MGD; maximum = 0.89 MGD; average = 0.25 MGD)

Figure 3.5 Duffy Well Stream Depletion Modified to Incorporate Summer Conservation Measures Corrected for Partial Stream Penetrations and Semipervious Streambank



Daily well withdrawals are based on total monthly withdrawals for 2000-2007 (minimum = 0.00 MGD; maximum = 0.89 MGD; average = 0.25 MGD)

4.0 COMPARISON TO INDEX STREAMFLOWS

In order to assess what the Parker River flow regime might be like under pre-development conditions, current Parker River streamflow data were compared with index streamflows using three approaches:

- Annual Target Hydrograph Approach;
- Aquatic Baseflow (ABF) Methodology; and
- Indicators of Hydrologic Alteration (IHA) method.

These approaches are recommended by the Massachusetts Water Resources Commission (MA WRC) in their guidance document, *Draft 2008 Index Streamflows for Massachusetts*, and are further discussed in Sections 4.2 through 4.4.

In addition, the Massachusetts Riverways Program evaluated Parker River streamflow utilizing the QPPQ Transform™ approach, developed by Hydologic Services, Inc. The model was created to generate natural daily streamflow at ungaged locations in the northeastern United States using data from similar gaged sites. A discussion of the QPPQ Transform™ analysis for the Parker River is provided in Section 4.5. The results of the QPPQ Transform™ analysis were then analyzed with the Parker River annual target hydrographs, ABF standards, and IHA statistics and presented in Section 4.6. Finally, a summary of the findings from all of the analyses is presented in Section 4.7.

4.1 Index Stream Selection

The most similar index stream was chosen among the set of 61 index streams developed by MA WRC (2008). Index streams are those gaged streams considered by the USGS to have minimal anthropogenic impacts. Consistent with the MA WRC's methodology for index stream selection, the following drainage area characteristics were included in the determination:

- Drainage area (square miles);
- Mean basin slope (percent);
- Basin area of stratified drift per total stream length (square miles per mile);
- Region (east or west), as defined by Ries and Friesz, 2000; and
- Distance from the Parker River to account for climatic differences.

In order to quantitatively determine the most similar index gage, a method, which was previously used in the Firm Yield Estimator developed by the Massachusetts Department of Environmental Protection (MA DEP), was modified and applied. The Firm Yield Estimator chooses the best index gage by minimizing the sum of the distance between the subject gage and the index gage (in kilometers) plus the relative percent differences in drainage area, mean basin elevation, and average annual precipitation. For this study, the best index gage was chosen as the minimum sum of the distance between the subject gage and the index gage (in miles) plus the relative percent differences in drainage area, mean basin slope and basin area of stratified drift per total stream length. The resulting best fit index gage was the Indian Head River in Hanover, MA (USGS gage #1105730; See Table 4-1).

Table 4-1. Comparison of Parker River to Index Streams

USGS Gage Number	Location	Distance from Parker River at Byfield Gage (mi)	Drainage Area (mi2)	Mean Basin Slope (%)	Stratified Drift per Stream Length (mi2/mi)	Region (0 or 1)	Best Fit (lowest is better)
1101000	**Parker River at Byfield**	**-**	**21.3**	**2.11**	**0.16**	**0**	**-**
1105730	Indian Head River, Hanover	45.46	30.3	2.44	0.2509	0	160.17
1097300	Nashoba Brook Acton, MA	28.67	12.8	4.67	0.2135	0	223.34
1111300	Nipmuc River near Harrisville, RI	65.42	16	5.27	0.1148	0	268.32
1073000	Oyster River Durham, NH	27.34	12.1	4.37	0.013	0	269.52

4.2 Annual Target Hydrograph Approach

The annual target hydrograph approach was used to determine whether natural flows at the Byfield Parker River gage at Route 95 had been significantly impacted. Median of mean monthly flows between 1960 and 2004 at the Parker River gage and the index gage, Indian Head River, were compared with the 25th and 75th percentile index gage flows in units of cubic feet per second per square mile (cfsm). These percentile flows are published in the *Draft 2008 Index Streamflows for Massachusetts*. The total number and percentage of months that flows at the Parker River gage and Indian Head River gage fell below, between, and above the quartile flows was calculated and compared for the period 1960-2004 (Table 4-2).

Table 4-2. Annual Target Hydrograph Comparison

	Percent of months Below Index Gage 75th percentile Flow in cfsm	Percent of months Between Index Gage 25th and 75th percentile Flow in cfsm	Percent of months Above Index Gage 25th percentile Flow in cfsm
Parker River (Non-Index Gage)	39%	45%	16%
Indian Head River (Index Gage)	21%	61%	18%
Expected Normal	25%	50%	25%

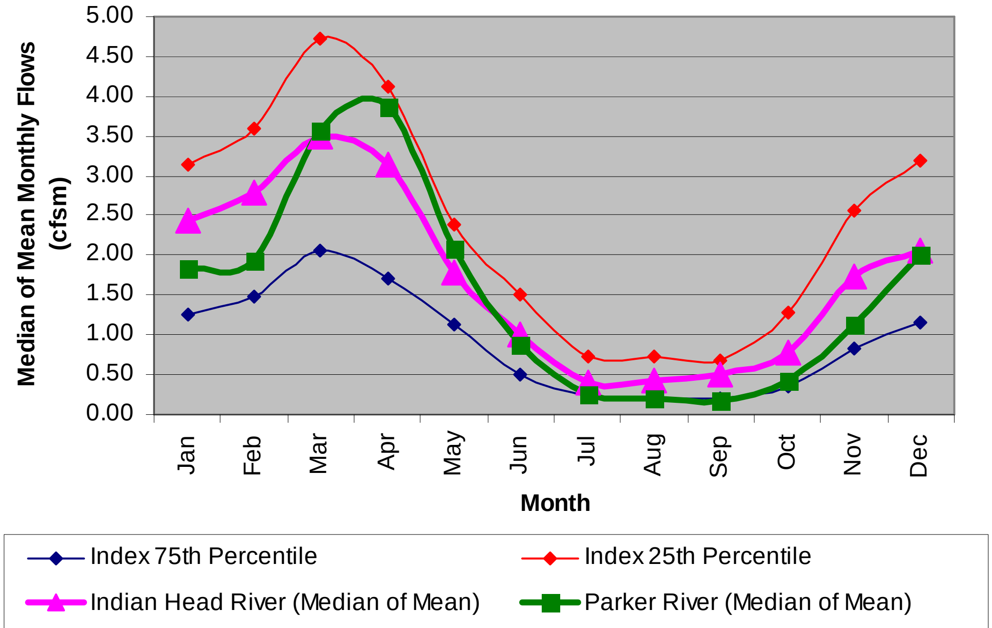
The flow distributions of the two gages varied significantly. The distribution exhibited at the index gage was similar to the expected distribution, and also comparable to the Nashoba Brook distribution provided on page 22 of the *Draft 2008 Index Streamflows for Massachusetts* and reproduced below in Table 4-3. Nashoba Brook (USGS gage #1097300) is the second most similar index gage to the Parker River gage.

Table 4-3. Nashoba Brook Target Hydrograph

	Percent of months Below Index Gage 75th percentile Flow in cfsm	**Percent of months Between Index Gage 25th and 75th percentile Flow in cfsm**	**Percent of months Above Index Gage 25th percentile Flow in cfsm**
Nashoba Brook (Index Gage)	25%	59%	16%

Although the percent of months with high flows above the index gage 25th percentile for the Parker River gage was lower than the expected normal, it was similar to both the Indian Head River and Nashoba Brook for this period of record. The percent of months with Parker River low flows below the index gage 75th percentile, however, is much higher than the expected normal and both index gages. This statistic indicates that low flow conditions in the Parker River are currently significantly more frequent than would be expected under pre-development conditions. Figure 4-1 on the following page displays the quartile flows throughout the year as well as the median of mean monthly flows for the Parker River and Indian Head River. Winter and spring flows for the Parker are generally higher than for the Index Stream, but are lower in the summer and fall, indicating seasonal stress for the Parker. It is also evident from the graph that the median of mean Parker River flows recede to and sometimes fall below the 75th percentile during the summer months. These are the months when natural flows are typically already at their lowest; therefore, the significantly low flows experienced continuously during the summer at the Parker River threaten its critical habitats.

Figure 4-1. Annual Target Hydrograph Comparison



4.3 ABF Method

The Aquatic Baseflow (ABF) method developed by the US Fish & Wildlife Service (USFWS) can be used to establish seasonal flow standards for subject streams based upon the median of monthly mean flows of index streams. The median average monthly flows (in cfsm) for the index stream, in this case the Indian Head River in Hanover, MA, are used along with the subject stream drainage area to provide monthly flow standards (in cfs) for the subject stream (See Table 4-4). The ABF method assumes that the most critical flows to be maintained are in August when metabolic stress to aquatic organisms is at its highest due to increased water temperatures, diminished living space, low dissolved oxygen, and low or diminished food supply (MA WRC, 2008).

Table 4-4. ABF Flows Compared with Actual Flows

Month	Indian Head River Median of Mean Monthly Streamflow* (cfsm)	Parker River at Byfield Drainage Area (mi^2)	Parker River Recommended Flow (cfs)	Parker River Actual Monthly Means (cfs; 1960-2004)	Difference in Flow (%)
January	1.96	21.3	41.8	41.1	-2%
February	1.96	21.3	41.8	49.1	17%
March	2.81	21.3	59.8	82.9	39%
April	2.81	21.3	59.8	87.9	47%
May	2.81	21.3	59.8	48.7	-19%
June	0.42	21.3	8.9	29.8	233%
July	0.42	21.3	8.9	9.2	3%
August	0.42	21.3	8.9	5.9	-34%
September	0.42	21.3	8.9	6.1	-32%
October	1.96	21.3	41.8	18.1	-57%
November	1.96	21.3	41.8	30.1	-28%
December	1.96	21.3	41.8	44.6	7%

*Summer (June - September) input streamflows are equal to the August median of mean monthly streamflow (1960-2004); Fall/Winter (October - February) input streamflows are equal to the average median of mean monthly streamflow for those months (1960-2004); Spring (March - May) streamflows are equal to the average median of mean monthly streamflows for those months (1960-2004).

Upon comparison of ABF recommended flows with average actual flows, it appears that although throughout much of the year the actual Parker River flow volume exceeds the recommended standards, flows in August (and September) are over 30% lower. This could indicate that the stream flow regime cannot support natural critical habitats.

4.4 IHA Method

The Indicators of Hydrologic Alteration (IHA) method was utilized to compare flow statistics observed at the Parker River gage with the index stream, Indian Head River in Hanover, MA. The IHA method was chosen because, compared to providing a single flow or seasonal value, determining a group of flow statistics can provide a more comprehensive picture of the river's flow regime, along with its capacity to adequately sustain natural hydrology, biology, geomorphology, water quality and connectivity characteristics (MA WRC, 2008). The IHA method is also recommended within MA WRC's *Draft 2008 Index Streamflows for Massachusetts* as an approach to compare subject streamflows with index streamflows, which are assumed to characterize natural flow regimes.

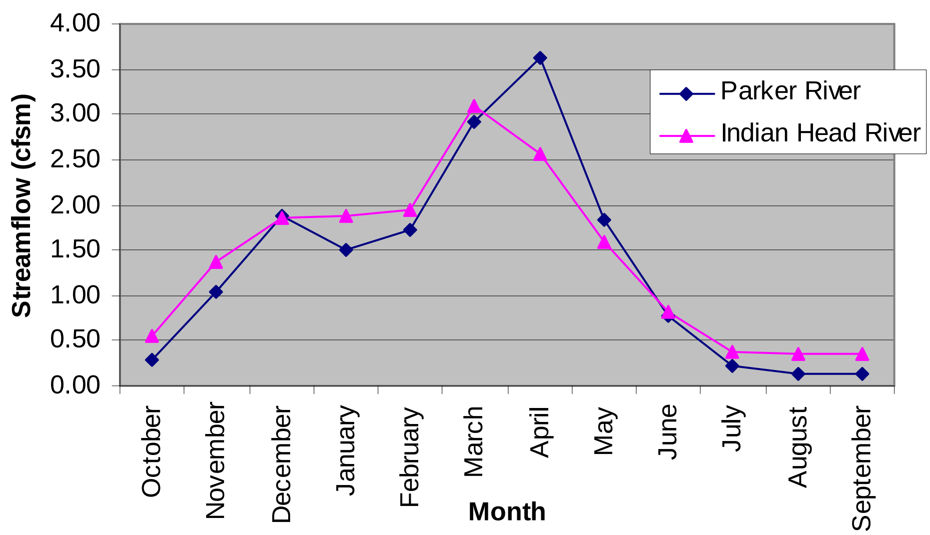
The IHA program developed by The Nature Conservancy (TNC) was used to calculate streamflow statistics at the Parker River gage using USGS daily streamflow data. MA WRC also used TNC's IHA program to develop IHA statistics for the index streams. Table 4-5 provides IHA statistical comparisons between the Parker River at Byfield gage and the Indian Head River gage.

A total of five parameter groups of IHA statistics, developed by Richter, et al. (1997), were analyzed. Each group signifies a different set of flow characteristics (MA WRC, 2008):

- Group 1 statistics describe monthly means;
- Group 2 statistics describe minimums and maximums;
- Group 3 statistics describe timing of seasonal flows;
- Group 4 statistics describe occurrence and duration of low flow events; and
- Group 5 statistics describe frequency and rates of flow rises and falls.

Average monthly flow conditions (group 1 statistics) illustrate seasonal fluctuations in the flow regime. Although both of the rivers demonstrated similar seasonal trends (see Figure 4-2), some significant disparities between the Parker River gage and index gage monthly mean flow values were evident. The analysis of group 1 statistics demonstrates that summer flows (July through September), which were the lowest flows at both rivers, were visually lower at the subject gage (Parker River) than the index gage. Conversely, flows in April, which were among the highest observed flows, were much higher at the subject gage than at the index gage.

Figure 4-2. Monthly mean flows observed at subject gage and index gage



Analyses of extreme flows (group 2 statistics) suggest that extreme minimum flows at the subject gage are significantly lower than those observed at the index stream. Extreme maximum flows seen at the subject gage were all slightly lower than those at the index gage as well, with the exception of the subject gage 30-day maximum which was slightly higher than the Indian Head River 30-day maximum. The fact that low flows for the Parker River are much lower than for the index gage, while high flows are only marginally higher suggests that water withdrawal-derived baseflow alterations may be more important than impervious cover related stormwater

Table 4-5. IHA Statistical Comparison between Massachusetts Index Gage (Indian Head River - 01105730) and Subject Gage (Parker River at Byfield, MA - 01101000), 1960-2004

	Means		Deviation Factor	
	Index Indian Head River	Non-Index Parker River		
Parameter Group #1	**cfsm**	**cfsm**	**Magnitude**	**Percent**
October	0.56	0.30	-0.27	-47%
November	1.37	1.03	-0.34	-25%
December	1.85	1.88	0.03	2%
January	1.88	1.50	-0.38	-20%
February	1.95	1.71	-0.23	-12%
March	3.10	2.91	-0.19	-6%
April	2.57	3.62	1.04	40%
May	1.58	1.83	0.25	16%
June	0.83	0.77	-0.05	-6%
July	0.37	0.21	-0.16	-43%
August	0.36	0.14	-0.22	-61%
September	0.36	0.12	-0.24	-66%
Parameter Group #2	**cfsm**	**cfsm**	**Magnitude**	**Percent**
1-day minimum	0.12	0.02	-0.10	-81%
3-day minimum	0.13	0.03	-0.10	-75%
7-day minimum	0.15	0.04	-0.11	-74%
30-day minimum	0.23	0.05	-0.17	-76%
90-day minimum	0.44	0.22	-0.22	-49%
1-day maximum	17.39	10.19	-7.20	-41%
3-day maximum	13.88	9.84	-4.04	-29%
7-day maximum	9.38	8.74	-0.64	-7%
30-day maximum	5.52	5.65	0.13	2%
90-day maximum	4.16	3.96	-0.20	-5%
Number of zero days	0.00	0.00	0.00	0%
Base flow index	0.08	0.02	-0.05	-69%
Parameter Group #3	**Julian Day**	**Julian Day**	**Magnitude**	**Percent**
Date of minimum	243	263	20.00	8%
Date of maximum	63	84	21.00	33%
Parameter Group #4	**Days**	**Days**	**Magnitude**	**Percent**
Low pulse count	6	3	-3.00	-50%
Low pulse duration	8	15.5	7.50	94%
High pulse count	13	5	-8.00	-62%
High pulse duration	3	12	9.00	300%
Low Pulse Threshold	0.53	0.3	-0.23	-43%
High Pulse Threshold	2.55	2.39	-0.16	-6%
Parameter Group #5	**cfsm**	**cfsm**	**Magnitude**	**Percent**
Rise rate	0.26	0.113	-0.15	-57%
Fall rate	-0.13	-0.094	0.04	-29%
Number of reversals	101	68	-33.00	-33%

Notes: Deviation Magnitude is the difference between the Index value and the Non-Index value for any statistic. A negative value indicates the Non-Index value is less than the Index value. Percent Deviation is calculated as the Deviation Magnitude divided by the Index value.

alterations, for example, which would tend to produce flashy high flows as well as diminished low flows.

The timing of the annual maximum and minimum flows are relatively similar between the subject gage and the index gage, with minimum flows in late August or September and maximum flows in March (Table 4-6).

Table 4-6. Timing of Seasonal Flows (dates of annual minimum, maximum)

	Parker River	Indian Head River
Date of Minimum	Julian Day 263 (Sept. 19)	Julian Day 243 (Aug. 30)
Date of Maximum	Julian Day 84 (Mar. 24)	Julian Day 63 (Mar. 3)

Group 4 statistics describe the occurrence and duration of low flow and high flow events. Although the occurrence of low flow and high flow events was lower at the subject gage than the index gage, the duration of low flow and high flow events were significantly (90-300%) higher at the subject gage versus the index gage. Longer durations are again indicative of factors other than flashy stormwater runoff contributing to the observed hydrologic alterations.

The rise and fall rates (group 5 statistics) experienced at the subject gage are lower than those experienced at the index gage. Slower hydrologic responses are yet another indicator that stormwater alterations in the watershed are not the most significant hydrologic alteration.

4.5 QPPQ Approach

Riverways staff estimated natural stream flow for the Parker River at Route 97 in Georgetown and at the USGS stream gauge site in Byfield using the DEP's Firm Yield Estimator methodology for surface water inflows (Water Management Program, 1996), which was developed based on previous USGS firm yield work (USGS, 2006). Drainage area, mean basin elevation, mean channel slope and geographic coordinates were derived from MassGIS data layers and the USGS National Hydrography Dataset. Average annual precipitation and snowfall were obtained from the National Oceanic and Atmospheric Administration. Maximum soil retention was calculated from surficial geology and landuse datalayers from MassGIS using NRCS TR-55 methods (NRCS, 1986). Flow duration curves for the Parker River at Route 97 and Byfield were developed using regression equations based on these seven parameters (Water Management Program, 1996). The timing of streamflow at the USGS Index Gauge site on the Indian Head River in Hanover from 1960 to 2004 was used to transform the flow duration curve into a time series of stream flows for the same period of record using the QPPQ transformation (Fennessey, 1994). The Indian Head River was chosen as the most appropriate USGS Index gauge based on its similarity in drainage area, mean basin elevation, average annual precipitation and distance from the Parker River (Massachusetts Department of Conservation and Recreation, Office of Water Resources, May 2008)).

4.6 Comparison of QPPQ Flows with Annual Target Hydrographs, ABF Standards, and IHA Statistics

The estimated Parker River natural daily flow data (in cfs) that was generated by the QPPQ Transform™ model was compared with Parker River actual flow data using the same approaches as were used for the index stream comparison:

- Annual Target Hydrograph Approach;
- Aquatic Baseflow (ABF) Methodology; and
- Indicators of Hydrologic Alteration (IHA) method.

4.6.1 Annual Target Hydrograph

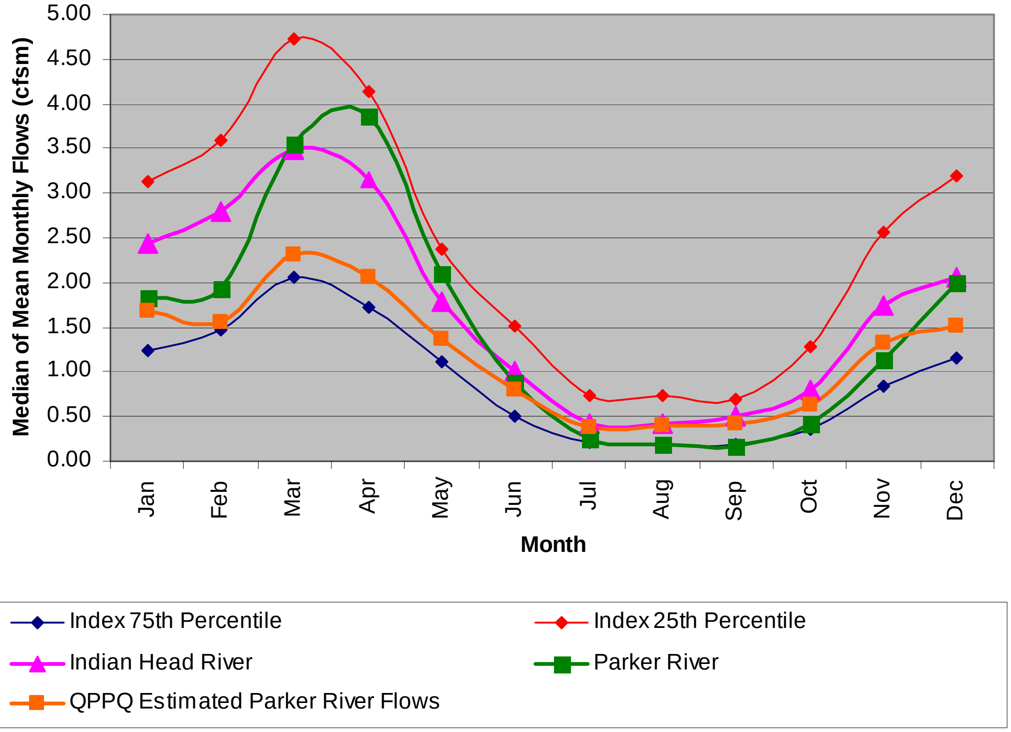
The annual target hydrograph for the QPPQ estimated Parker River flows showed a distribution of flow that included a lower percentage of low flow (below the 75th percentile) than the actual Parker River flow distribution (Table 4-7). However, the model also provided a distribution that had a significantly low percentage of higher flows (above the 25th percentile). The expected normal distribution would include 25% higher flows, and the index stream exhibited 18% higher flows. The QPPQ estimated natural flows for Parker River produced a percentage of higher flows equal to half the index stream at just 9%.

Table 4-7. Annual Target Hydrograph with QPPQ Estimated Parker River Flow Distribution

	Percent of months Below Index Gage 75th percentile Flow in cfsm	Percent of months Between Index Gage 25th and 75th percentile Flow in cfsm	Percent of months Above Index Gage 25th percentile Flow in cfsm
Parker River (Non-Index Gage)	39%	45%	16%
Indian Head River (Index Gage)	21%	61%	18%
QPPQ Estimated Parker River Flows	33%	58%	9%
Expected Normal	25%	50%	25%

The graph of median of mean monthly flows for the actual Parker River flows, the Indian Head River flows, and the QPPQ estimated Parker River flows shows that the curve of the QPPQ estimated Parker River flows is much shallower than either the Parker River actual flow curve or the Indian Head River Curve (Figure 4-3). Modeled flows are generally lower than actual Parker River flows in the winter and spring and higher in the summer and fall.

Figure 4-3. Annual Target Hydrograph Comparison with QPPQ Estimated Parker River Flows



4.6.2 ABF Method

The median average monthly QPPQ estimated Parker River flows (in cfsm) were used to provide monthly flow standards (in cfs) for the subject stream (See Table 4-8). Similar to the ABF comparison with the index stream in Section 4-3, Parker River actual flows are significantly less than the ABF recommended flows generated using the QPPQ estimated flows for the month of August, which is estimated by the US Fish and Wildlife Service to be the most critical month for aquatic habitat sustainability (MA WRC, 2008). The overall negative flow differential estimated by comparison to the QPPQ natural flow estimate, however, is significantly less than that estimated by index stream comparison (See Table 4-4).

Table 4-8. QPPQ Estimated ABF Flows Compared with Actual Flows

Month	QPPQ Estimated Parker River Median of Mean Monthly Streamflow* (cfsm)	Parker River at Byfield Drainage Area (mi^2)	Parker River Recommended Flow (cfs)	Parker River Actual Monthly Means (cfs; 1960-2004)	Difference in Flow (%)
January	1.33	21.3	28.4	41.1	45%
February	1.33	21.3	28.4	49.1	73%
March	1.91	21.3	40.7	82.9	104%
April	1.91	21.3	40.7	87.9	116%
May	1.91	21.3	40.7	48.7	20%
June	0.39	21.3	8.3	29.8	257%
July	0.39	21.3	8.3	9.2	10%
August	0.39	21.3	8.3	5.9	-29%
September	0.39	21.3	8.3	6.1	-27%
October	1.33	21.3	28.4	18.1	-36%
November	1.33	21.3	28.4	30.1	6%
December	1.33	21.3	28.4	44.6	57%

*Summer (June - September) input streamflows are equal to the August median of mean monthly streamflow (1960-2004); Fall/Winter (October - February) input streamflows are equal to the average median of mean monthly streamflow for those months (1960-2004); Spring (March - May) streamflows are equal to the average median of mean monthly streamflows for those months (1960-2004).

4.6.3 IHA Method

Parker River actual flows were compared with QPPQ estimated Parker River flows using the IHA approach to determine whether actual flow conditions were similar to expected pre-development conditions. Table 4-9 shows IHA statistical comparison between Parker River and Parker River QPPQ estimated flows. In addition, QPPQ estimated Parker River flows were compared to the index stream flows to determine how similar the modeled natural flows are to the natural flows at the index stream. Table 4-10 shows the IHA statistical comparison between Parker River QPPQ estimated flows and the Indian Head River.

Table 4-9. IHA Statistical Comparison between QPPQ estimated flows for Parker River under Natural Conditions and Subject Gage (Parker River at Byfield, MA - 01101000), 1960-2004

	Means		Deviation Factor	
	Parker River - Estimated Natural Conditions	Parker River - Actual		
Parameter Group #1	**cfsm**	**cfsm**	**Magnitude**	**Percent**
October	0.513	0.30	-0.22	-42%
November	1.088	1.03	-0.06	-5%
December	1.418	1.88	0.46	32%
January	1.446	1.50	0.06	4%
February	1.499	1.71	0.22	14%
March	2.161	2.91	0.75	35%
April	1.91	3.62	1.71	89%
May	1.237	1.83	0.59	48%
June	0.686	0.77	0.09	13%
July	0.366	0.21	-0.16	-42%
August	0.361	0.14	-0.22	-61%
September	0.35	0.12	-0.23	-65%
Parameter Group #2	**cfsm**	**cfsm**	**Magnitude**	**Percent**
1-day minimum	0.119	0.02	-0.10	-81%
3-day minimum	0.1245	0.03	-0.09	-75%
7-day minimum	0.147	0.04	-0.11	-74%
30-day minimum	0.2183	0.05	-0.16	-75%
90-day minimum	0.4287	0.22	-0.21	-48%
1-day maximum	5.35	10.19	4.84	90%
3-day maximum	4.936	9.84	4.91	99%
7-day maximum	4.181	8.74	4.56	109%
30-day maximum	3.022	5.65	2.62	87%
90-day maximum	2.425	3.96	1.54	63%
Number of zero days	0	0.00	0.00	0%
Base flow index	0.1191	0.02	-0.10	-80%
Parameter Group #3	**Julian Day**	**Julian Day**	**Magnitude**	**Percent**
Date of minimum	241.5	263	21.50	9%
Date of maximum	60.5	84	23.50	39%
Parameter Group #4	**Days**	**Days**	**Magnitude**	**Percent**
Low pulse count	6	3	-3.00	-50%
Low pulse duration	8.25	15.5	7.25	88%
High pulse count	13.5	5	-8.50	-63%
High pulse duration	3.25	12	8.75	269%
Low Pulse Threshold	0.5	0.3	-0.20	-40%
High Pulse Threshold	1.9	2.39	0.49	26%
Parameter Group #5	**cfsm**	**cfsm**	**Magnitude**	**Percent**
Rise rate	0.1873	0.113	-0.07	-40%
Fall rate	-0.0945	-0.094	0.00	-1%
Number of reversals	102	68	-34.00	-33%

Notes: Deviation Magnitude is the difference between the Index value and the Non-Index value for any statistic. A negative value indicates the Non-Index value is less than the Index value. Percent Deviation is calculated as the Deviation Magnitude divi

Table 4-10. IHA Statistical Comparison between Massachusetts Index Gage (Indian Head River - 01105730) and QPPQ estimated flows for Parker River under Natural Conditions, 1960-2004

	Means		Deviation Factor	
	Indian Head River	Parker River - Estimated Natural Conditions		
Parameter Group #1	**cfsm**	**cfsm**	**Magnitude**	**Percent**
October	0.56	0.513	-0.05	-9%
November	1.37	1.088	-0.28	-21%
December	1.85	1.418	-0.43	-23%
January	1.88	1.446	-0.44	-23%
February	1.95	1.499	-0.45	-23%
March	3.10	2.161	-0.94	-30%
April	2.57	1.91	-0.66	-26%
May	1.58	1.237	-0.35	-22%
June	0.83	0.686	-0.14	-17%
July	0.37	0.366	0.00	-1%
August	0.36	0.361	0.00	-1%
September	0.36	0.35	-0.01	-4%
Parameter Group #2	**cfsm**	**cfsm**	**Magnitude**	**Percent**
1-day minimum	0.12	0.119	0.00	0%
3-day minimum	0.13	0.1245	0.00	-2%
7-day minimum	0.15	0.147	0.00	0%
30-day minimum	0.23	0.2183	-0.01	-3%
90-day minimum	0.44	0.4287	-0.01	-3%
1-day maximum	17.39	5.35	-12.04	-69%
3-day maximum	13.88	4.936	-8.94	-64%
7-day maximum	9.38	4.181	-5.20	-55%
30-day maximum	5.52	3.022	-2.49	-45%
90-day maximum	4.16	2.425	-1.73	-42%
Number of zero days	0.00	0	0.00	0%
Base flow index	0.08	0.1191	0.04	54%
Parameter Group #3	**Julian Day**	**Julian Day**	**Magnitude**	**Percent**
Date of minimum	243	241.5	-1.50	-1%
Date of maximum	63	60.5	-2.50	-4%
Parameter Group #4	**Days**	**Days**	**Magnitude**	**Percent**
Low pulse count	6	6	0.00	0%
Low pulse duration	8	8.25	0.25	3%
High pulse count	13	13.5	0.50	4%
High pulse duration	3	3.25	0.25	8%
Low Pulse Threshold	0.53	0.5	-0.03	-6%
High Pulse Threshold	2.55	1.9	-0.65	-25%
Parameter Group #5	**cfsm**	**cfsm**	**Magnitude**	**Percent**
Rise rate	0.26	0.1873	-0.08	-29%
Fall rate	-0.13	-0.0945	0.04	-28%
Number of reversals	101	102	1.00	1%

Notes: Deviation Magnitude is the difference between the Index value and the Non-Index value for any statistic. A negative value indicates the Non-Index value is less than the Index value. Percent Deviation is calculated as the Deviation Magnitude divi

Upon comparison of actual Parker River average monthly flow conditions with modeled Parker River average monthly flow conditions (group 1 statistics), it appears that the results mimic the comparison of the Parker River with the index gage. Summer flows, between July and September, were significantly lower for actual conditions than for expected pre-development conditions. Similar to the comparison with the index gage, actual flows in the Parker River in April, which were among the highest observed flows under actual and modeled conditions, were much higher than modeled natural flows.

When the QPPQ estimated Parker River flows were compared with the index gage flows, the average monthly flows were all less for the modeled flows than the index gage flows. The percent difference between them (1% to 30% lower for modeled flows versus the index gage), however, was much less than percent difference that resulted from the comparison of Parker River actual flows with modeled flows (65% lower to 89% higher for actual flows versus modeled flows).

The comparison of extreme flows (group 2 statistics) between actual conditions and expected pre-development conditions was different than the comparison of actual flows and the index gage. Although extreme minimum flows at the Parker River were significantly lower than expected pre-development conditions, extreme maximum flows were all higher than the modeled flows.

The timing of the annual maximum and minimum flows were similar between the actual Parker River flows and modeled Parker River flows; however, the maximum and minimum flow days were almost identical between the modeled Parker River flows and the Indian Head River flows (Table 4-11).

Table 4-11. Timing of Seasonal Flows (dates of annual minimum, maximum)

	Parker River	**QPPQ Estimated Parker River**	**Indian Head River**
Date of Minimum	Julian Day 263 (Sept. 19)	Julian Day 241.5 (Aug. 28-29)	Julian Day 243 (Aug. 30)
Date of Maximum	Julian Day 84 (Mar. 24)	Julian Day 60.5 (Feb. 29 - Mar. 1)	Julian Day 63 (Mar. 3)

Comparison of Group 4 statistics, which describe the occurrence and duration of low flow and high flow events, between the QPPQ estimated Parker River flows and actual Parker River flows, resulted similarly to the comparison of actual Parker River flows and the Indian Head River flows. Although the actual occurrence of low flow and high flow events was lower than that under modeled conditions, the duration of low flow and high flow events was significantly (88-269%) higher for actual conditions. When compared with the index gage, the modeled natural occurrence and duration of low flow and high flow events at the Parker River using the QPPQ method were very similar to those observed at the index gage (See Table 4-10). The rise rate (group 5 statistics) of the Parker River was 40% lower than the modeled natural rise rate; however, the fall rate was almost identical.

4.7 Index Stream Discussion

The Index Stream evaluation of the Parker River flow regime indicates that historically, flows within the Parker River are much lower than both index streamflow (which is indicative of pre-development conditions) and modeled Parker River natural streamflow. The annual target hydrograph suggested that the frequency of low flows in the Parker River (39%) was higher than the expected normal (25%), the QPPQ estimated natural Parker River flows (33%), the Indian Head River index stream (21%), and an alternative index stream, Nashoba Brook (25%).

More specifically, August and September streamflow, which are the lowest natural flow months, are consistently and significantly low in the Parker River. Two sets of Parker River monthly flow standards were calculated using the ABF approach applied to both the index stream flow data and the QPPQ estimated Parker River flow data. Although average monthly Parker River flows fell within the ABF monthly flow standards for many of the months throughout the year, average actual flows in August and September greatly exceeded both sets of standards for those months. This is significant since the US Fish and Wildlife Service has determined that the most critical flows to be maintained are in August (MA WRC, 2008).

The extreme flow duration statistics and rise and fall rate statistics indicate that the flow of the Parker River is most likely more impacted by withdrawals within the watershed than excessive stormwater runoff from impervious surfaces. When compared to both the index stream and the QPPQ estimated pre-development conditions, Parker River extreme high and extreme low flows are less frequent. However, the average duration of the extreme high and extreme low events is significantly longer. This can be interpreted to mean that it takes the Parker River longer to recuperate from extreme events, most likely because of reduced baseflow. This is further supported by the rise and fall rates which are lower for the Parker River than for the index stream.

While excessive stormwater runoff from increased impervious surfaces can reduce baseflow, they also result in more flashy conditions during storm events with increased rise and fall rates and shorter extreme flow durations.

The QPPQ estimated pre-development conditions were determined to be similar to the index stream when comparing their IHA flow statistics; however, the annual target hydrograph shows that the QPPQ curve is shallower than the index stream curve, indicating that the model may devalue seasonal fluctuations. Therefore, although flows generated by the model predict higher natural flows in August similar to the index stream, it also predicts much lower flows than the index stream for the spring (See Table 4-10).

5.0 DISCUSSION

All of the analyses conducted during this study support the findings from previous investigations (Gomez and Sullivan, 2003) that the upper Parker River is experiencing noticeable hydrologic stress under current conditions compared to estimates of natural or pre-developed conditions. This hydraulic stress is evident at all time frames, from average annual to seasonal, and is particularly apparent for the area of the watershed downstream of Uptack Road and upstream of Route 97 where most of the major withdrawal points are concentrated and the volume of return flow through septic systems is limited. The transport of water from this area of the watershed for use in developed areas downstream constitutes a net loss of water for the most highly stressed portion of the watershed.

In contrast, the watershed area upstream of Uptack Road is in near hydrologic balance from a water budget perspective due to the relatively low level of development and the use of both on-site private wells and septic systems that effectively "keep water local." The portion of the watershed downstream of Route 97 and upstream of Route 95 feels the negative effects of diminished flow that originate upstream, but is in itself only slightly out of balance.

Part of the situation leading to the observed hydrologic stress is no doubt natural. The upper Parker River Watershed is relatively small in size and has a relatively small proportion of thick sand and gravel deposits capable of storing water for later release to support streamflow. From that regard, the capacity of the watershed to support anthropogenic water demands is somewhat limited. However, the pattern of increasing development in the watershed, commonly referred to as suburban sprawl, is particularly stressful for water resources, and certainly not unique to this watershed.

The seasonal impact analyses conducted here using the USGS STRMDEPL model showed that summer stresses on the river are particularly acute owing to the extremely close proximity of the wells to the river and the disproportional increase in summer water demand owing to lawn irrigation. An alternatives analysis conducted using the annual water budget model showed that reducing the summer to winter water use ratio from 2:1 down to 1.2:1 would have the greatest net hydrologic benefit to the river of all the remedial alternatives evaluated. A STRMDEPL model alternative analysis run of that same summer conservation scenario showed that estimated stream depletion is reduced proportionally to the estimated reduction in monthly pumping (e.g. a 40% reduction in pumping from current conditions for a given months results in a 40% reduction of estimated streamflow depletion relative to the depletion estimated for current conditions). A hypothetical evaluation that examined estimated stream depletions occurring from wells located 2,000 feet from the river showed greatly reduced streamflow impacts compared to the actual geometry and a significant lag time where impacts are broadly spread out over periods of years rather than concentrated during peak pumping months.

The Index Stream evaluation of the Parker River flow regime indicates that historically, flows within the Parker River are much lower than both index streamflow (which is indicative of pre-development conditions) and modeled Parker River natural streamflow. More specifically, August and September streamflow, which are the lowest natural flow months, are consistently and significantly low in the Parker River. This is significant since the US Fish and Wildlife Service has determined that the most critical flows to be maintained are in August (MA WRC, 2008). The extreme flow duration statistics and rise and fall rate statistics indicate that the flow of the Parker River is most likely more impacted by withdrawals within the watershed than excessive stormwater runoff from impervious surfaces.

The beaver population in the Upper Parker River watershed has rebounded in recent decades and there has been significant discussion about the potential hydrologic impacts of beavers and their dams. When beavers build dams and impound the river, water levels rise behind the dam until the spillway elevation of the dam is attained. The inundated area also increases correspondingly, depending upon the topography behind the dam; with deeper water in areas with steep banks and broad shallow wetlands in flatter areas. While the water behind the dam is rising, the surface water elevation increases relative to the surrounding ground water level; reversing the normal gradient of flow from the aquifer to the river. In this fashion, the impoundment provides a temporary increase of groundwater recharge for the area immediately around the impoundment. After the impounded water has reached a new equilibrium elevation equivalent to the top of the spillway, the groundwater also equilibrates at a new higher elevation and the normal flow from the groundwater to the river is resumed.

The flooded area behind the dam may potentially create greater acreage of wetlands than had existed prior to the presence of the dam. However, the flooded area may also sometimes convert previous wetland areas to lands under water with the new wetland areas on the fringe of the ponded area being smaller, larger, or equivalent to the old wetland area; depending upon topography and other site-specific factors. Because of the high evapotranspiration rates from wetlands' emergent vegetation, they tend to be areas that provide no or minimal aquifer recharge. All of the precipitation that falls on them is either lost to evapotranspiration or flows away downstream. Therefore, from a water budget perspective, increased wetland areas would tend to reduce the overall amount of aquifer recharge and its corresponding support of base flow to the river. However, it is unclear if significant new wetland areas are generally created as a result of beaver dam impoundment, and whether those impounded areas support a significant increase in emergent vegetation with high rates of evapotranspiration.

For this study, beaver dams would affect the water budget analysis only in terms of the amount of wetland area in the calculations. Current MassGIS wetlands coverage was used for the existing conditions, pre-development conditions, and alternatives analyses so that any water budget impact from beaver dams would be counted in the "before" and "after" analyses. Therefore the calculated water budget results reflect anthropogenic impacts that are over and above any beaver dam impact that may, or may not, exist. The MassGIS wetlands coverage was visually compared against the current aerial photographic coverage to ensure that no major inaccuracies were present. Beaver dams would not affect the Stream Depletion analyses and could only affect the Index Stream analyses in terms of their impact on observed Parker River flows relative to flows from the Index Stream. This potential impact, if any, cannot be quantified here but is not considered likely to be significant.

It should be remembered that beavers have long been part of the natural environment in New England. While their population has rebounded in recent decades relative to the recent past, it is obviously much less than it was in pre-colonial times. The affect of beaver dams are part of the natural hydrologic environment. This study focused on potential anthropogenic alterations to stream flow.

6.0 RECOMMENDATIONS

The major factors contributing to observed hydrologic stress in the upper Parker River (in no particular order) are:

- Small watershed size and limited groundwater storage capacity naturally limits the capacity to support anthropogenic water demands;
- Extremely close proximity of water supply wells to the river allows the river to "feel" the impacts from pumping very quickly;
- A concentration of supply wells in one river reach which amplifies the local-scale impacts;
- The net export of water from the area of the watershed where supply wells are concentrated to more developed areas downstream which creates additional local-scale stresses; and
- Disproportional summer water use for lawn irrigation which amplifies seasonal stresses at times when the river is already naturally stressed.

Of the remedial alternatives evaluated during this study, a reduction of the summer/winter water use ratio to 1.2:1 produced the most significant water budget benefits to the most stressed portion of the watershed (12% improvement). In addition, that benefit would occur during the summer months when the river is most stressed. The water budget model cannot produce results for time frames of less than a year, but the summer season benefit would likely be higher than the estimated 12% average annual benefit. The option to sewer a very small area of the Georgetown downtown and transport that treated effluent back upstream for groundwater discharge produced a very minimal (3%) benefit that would probably not be worth doing. Sewering a larger area of the Georgetown downtown that mimics the current stormwater management area produces a more significant annual water budget benefit (6% improvement) to the most stressed portion of the watershed. The combination of summer water conservation and sewer improvements to the downtown would result in a cumulative estimated water budget improvement of approximately 18%. Although not specifically quantified in our analyses, transferring stormwater from the downtown area upstream for groundwater discharge along with the treated sewage could significantly increase the annual water budget benefits. The water budget benefits of sewering and/or stormwater management would likely be distributed relatively evenly over the course of the year.

When cost and public perception are factored, water conservation appears as a more easily attainable first step towards bringing the water budget closer to balance. Overall water conservation and, particularly, summer water use reduction can be accomplished relatively inexpensively provided that public support is available. Public sewering, water reuse, and other similar large scale public works project are generally expensive and may face permitting and/or political obstacles.

A reduction of the summer/winter water use ratio requires a reduction of outdoor water use and a corollary change in resident water use habits. A detailed discussion of a potential methodology to accomplish such a reduction in outdoor water use is beyond the scope of this study, but techniques range from outright bans on automatic sprinkler systems to educational programs on water conservation. Land conservation programs can limit potential future water demands while Low Impact Development bylaws and techniques can reduce the potential water use impacts from new development. Stormwater management retrofits that incorporate best management practices maximizing infiltration of stormwater can help to offset some consumptive water use

Another alternative not specifically addressed in this study would be to irrigate the Georgetown Club with treated effluent transferred from the downtown sewered area. This option would not only benefit from the upstream transfer of water from the sewered downtown, but would also eliminate a significant water withdrawal source (golf course irrigation) from the most highly stressed portion of the watershed. Because this alternative was not specifically run through the model, the water budget benefit is not quantified here.

The extreme proximity of the public supply wells to the stream creates a situation where short-term impacts to the river occur very quickly and may not always be able to be overcome with improvements to the overall annual water budget. Alternative water supply location feasibility was not part of this study, and it appears unlikely that good supply alternatives exist significantly further from the river. In addition, the expense and permitting difficulties of developing new water supply sources are significant. However, in the event that a new supply source becomes needed, priority should be given to locations at maximum distance from the river.

During planning discussions with Town representatives, State officials, and local planning agencies, the possibility was voiced of augmenting water inputs to the upper Parker River Watershed through flood skimming from the adjacent Merrimack River. This alternative would be considered an interbasin transfer which is normally fraught with permitting, engineering, and financial hurdles. In this case, however, circumstances may be more favorable. First, flow in the Merrimack River is many orders of magnitude greater than in the Parker River so that, particularly if transferred during high spring flow periods, a quantity of water could be transferred from the Merrimack River to the Parker River that would be nearly imperceptible to the Merrimack but constitute a significant volume for the Parker. Second, the Merrimack River is only approximately 3 miles from the upper Parker River Watershed boundary in Boxford, and mapped sand and gravel deposits potentially capable of accepting groundwater discharge of transferred water are within 2,000 feet of the boundary. Lake Cochichewick in North Andover is tributary to the Merrimack River and is located only approximately 2,000 feet from the boundary of the upper Parker River watershed. The feasibility of this option is, however, obviously uncertain and much more detailed analyses would be required to better assess its feasibility.

Seasonal skimming of the Parker River itself during periods of high flows is another possibility. To provide much utility, the skimmed water would need to be stored in a manner that would allow the skimmed water to be available many months after it was stored. This would require either significant man-made storage or the ability to transport the skimmed water for aquifer recharge to an area with suitably permeable sand and gravel deposits upgradient of the river reach with the greatest deficit. Both options would have cost and permitting hurdles not evaluated here.

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APPENDIX A

Streamflow Data for Baseflow Estimation

Appendix A - Annual Average of Mean Monthly Minimum Flows at the USGS Byfield Gage over Ten-year Period (1998-2007)

Month	Monthly Minimum Flows (cfs)										10-yr Mean (cfs)
	1998	1999	2000	2001	2002	2003	2004	2005	2006	2007	
January	32.0	9.7	14.0	21.0	1.6	14.0	14.0	66.0	55.0	34.0	26.1
February	38.0	34.0	16.0	27.0	7.7	7.1	9.7	42.0	38.0	9.1	22.9
March	72.0	52.0	36.0	27.0	13.0	43.0	12.0	38.0	19.0	9.0	32.1
April	31.0	14.0	39.0	38.0	18.0	49.0	80.0	42.0	18.0	67.0	39.6
May	24.0	10.0	34.0	7.6	24.0	23.0	30.0	28.0	17.0	27.0	22.5
June	29.0	0.2	23.0	10.0	15.0	17.0	8.1	15.0	49.0	5.0	17.1
July	1.7	0.2	4.5	0.9	1.0	0.7	4.9	3.0	8.1	1.5	2.6
August	0.8	0.0	3.5	0.4	0.1	1.0	10.0	0.2	0.9	0.1	1.7
September	0.4	0.0	2.9	0.1	0.1	0.1	12.0	0.2	2.8	0.1	1.9
October	0.9	2.6	3.9	0.1	0.5	1.6	14.0	0.7	2.3	0.3	2.7
November	7.4	10.0	11.0	0.2	2.1	15.0	13.0	31.0	28.0	2.3	12.0
December	8.1	13.0	13.0	0.2	14.0	19.0	46.0	38.0	21.0	7.4	18.0
										Annual Average:	**16.6**

APPENDIX B

Summary of Aquifer Characteristics

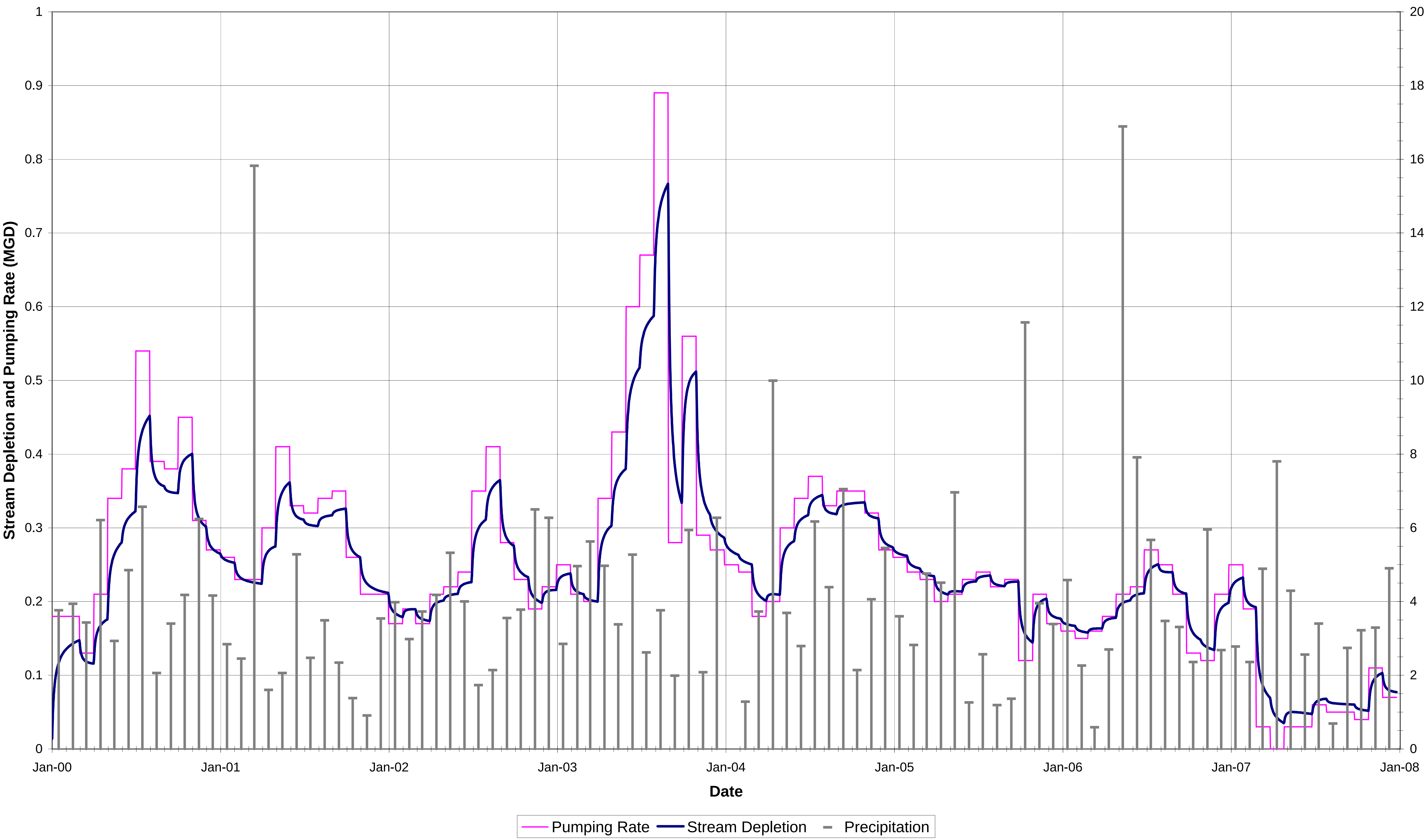
Appendix B. Summary of Aquifer Characterstics

Well ID	Well Name	Distance to River (ft)	Aquifer Transmissivity (gpd/ft)	Aquifer Transmissivity (ft^2/d)	Aquifer Thickness (ft)	Aquifer Storage Coefficient	Aquifer Specific Yield	Actual Pumping Rate - Peak Day (MGD)	Actual Pumping Rate - Avg Day over 5 yrs (MGD)	DEP Zone II Approved Pump Rate (MGD)	Pump Capacity (gpm)	Source
05G	Ronald Marshall (Duffy)	250	108,443	14,498	50 - 60	2.65E-02		1.37	0.32	1.51	1,000	Source Final Report - Duffy's Landing Well Site 30-83 - Prolonged Pumping Test - October 1994 - Haley and Ward, Inc. & Annual Statistical Reports
2-92	Near Duffy Well	200	101,200	13,529	59							
31-83		370	68,100	9,104	21	1.75E-02						
4U-92		50	112,900	15,094	10	5.04E-03						
4L-92		5	109,000	14,572	50	5.44E-03						
5-92		70	119,800	16,016	61	5.39E-03						
23E-66		200	124,300	16,618		1.85E-03						
Well to South			123,800	16,551		3.00E-03						
04G	Commissioners'	600	22,000	2,941	27		0.24	0.44	0.23	0.58	350	New Source Approval Site Examination Request - January 29, 1992 - Haley and Ward, Inc.& Annual Statistical Reports
179	Near Commissioners;	600			27							Public Water Supply Resources of the Parker River Basin - May 29, 1973 - Metcalf & Eddy, Inc.
03G	W.M. Marshall (Marshall)	500	59,000	7,888	59		0.02	0.96	0.27	1.01	700	New Source Approval Site Examination Request - January 29, 1992 - Haley and Ward, Inc.& Annual Statistical Reports
186	Near Marshall Well	500	47,000.00	6,283	62	0.1						Public Water Supply Resources of the Parker River Basin - May 29, 1973 - Metcalf & Eddy, Inc.
04G	Forrest Street	125	3,200	428.3		6.90E-04			0.13	0.36	340	Final Report on the Well Construction, Pumping Testing and Zone II & III Delineations for the Forrest Street Well Site - May 1994 - D.L. Maher Co.& Annual Statistical Reports

APPENDIX C

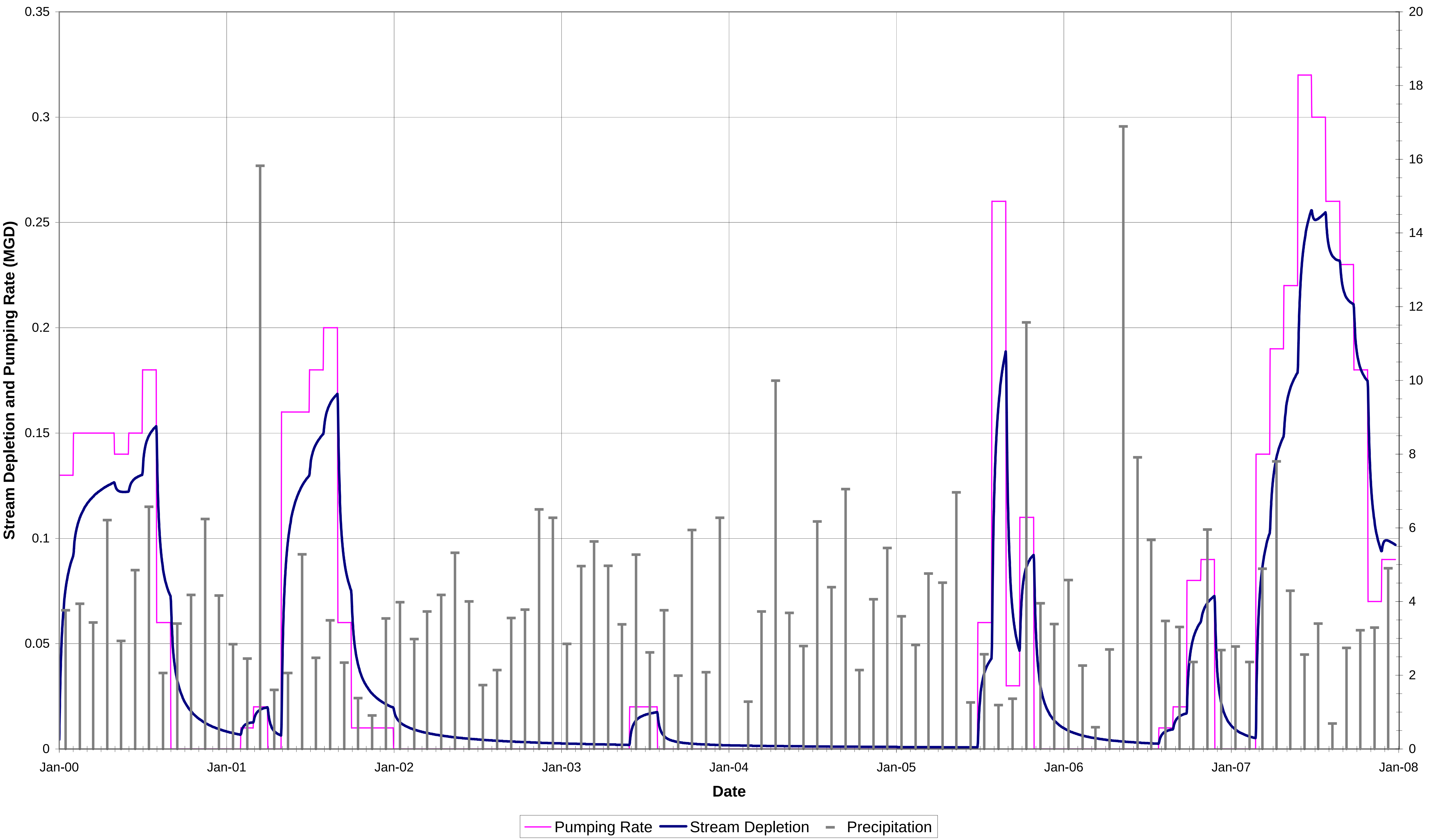
Stream Depletion Graphs for Individual Wells and Alternatives Analyses

Figure C.1 WM Marshall Well Stream Depletion



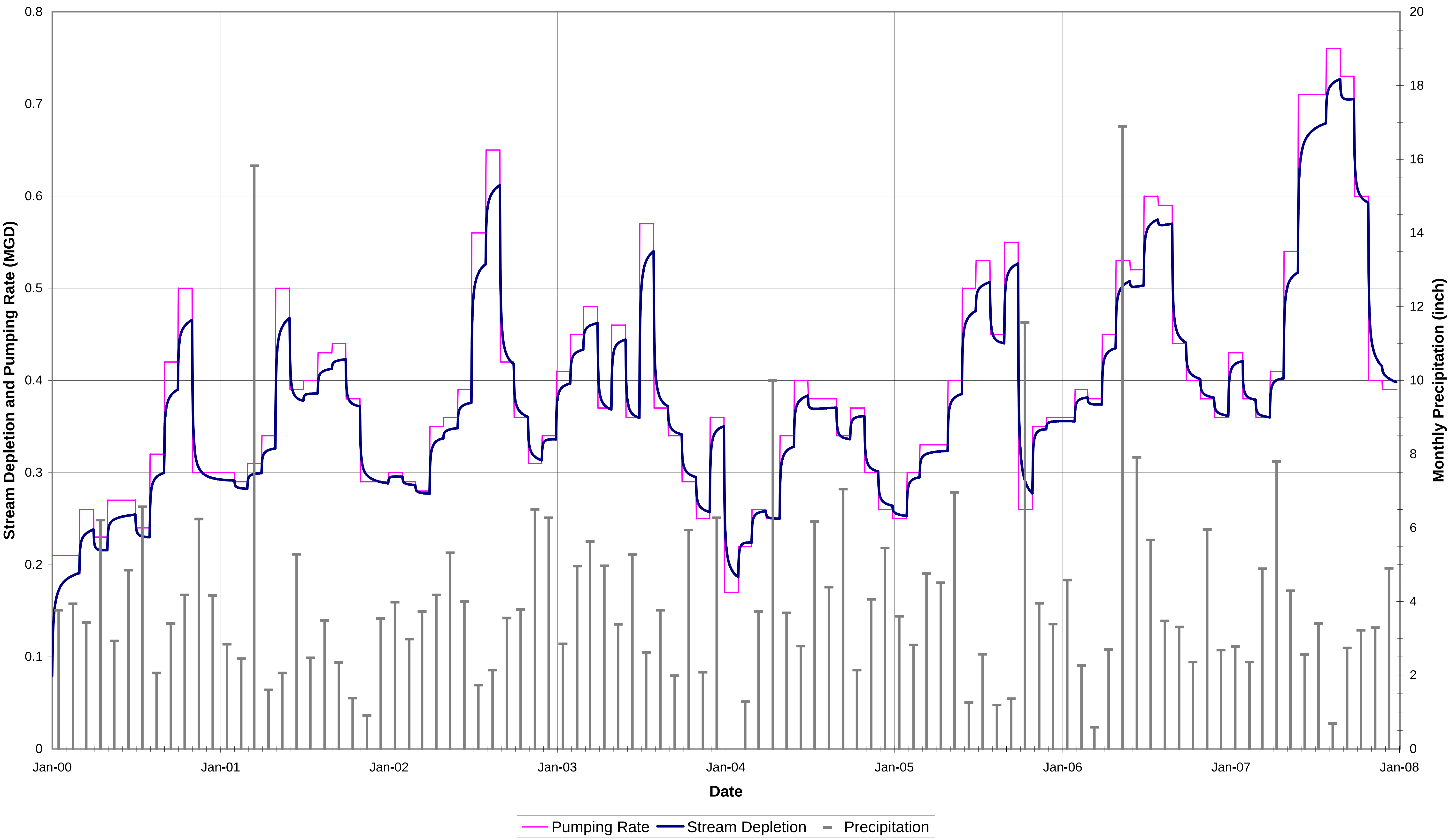
Daily well withdrawals are based on total monthly withdrawals for 2000-2007 (minimum = 0.00 MGD; maximum = 0.89 MGD; average = 0.25 MGD)

Figure C.2 Commissioner Well Stream Depletion



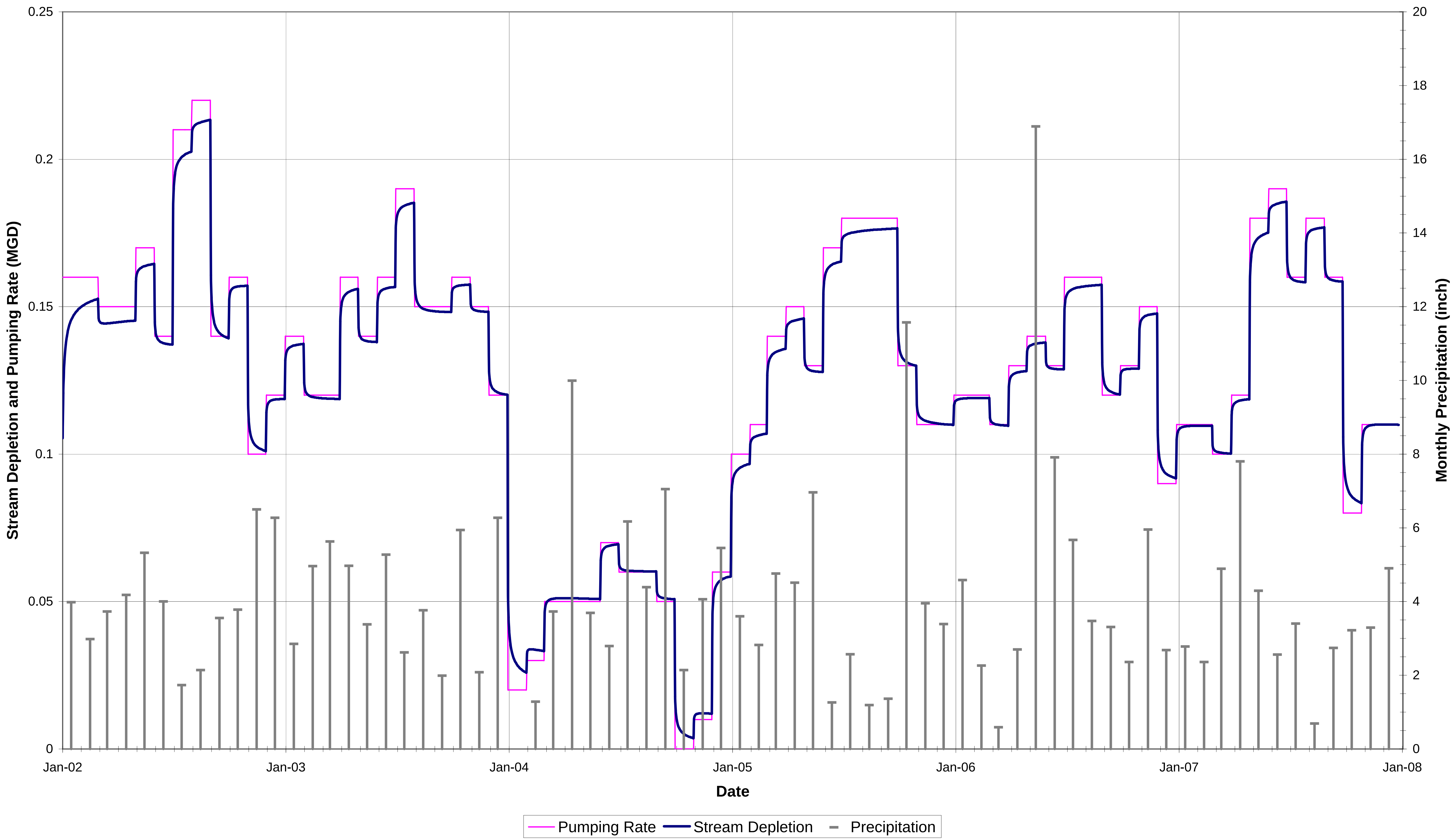
Daily well withdrawals are based on total monthly withdrawals for 2000-2007 (minimum = 0.00 MGD; maximum = 0.89 MGD; average = 0.25 MGD)

Figure C.3 Duffy Well Stream Depletion



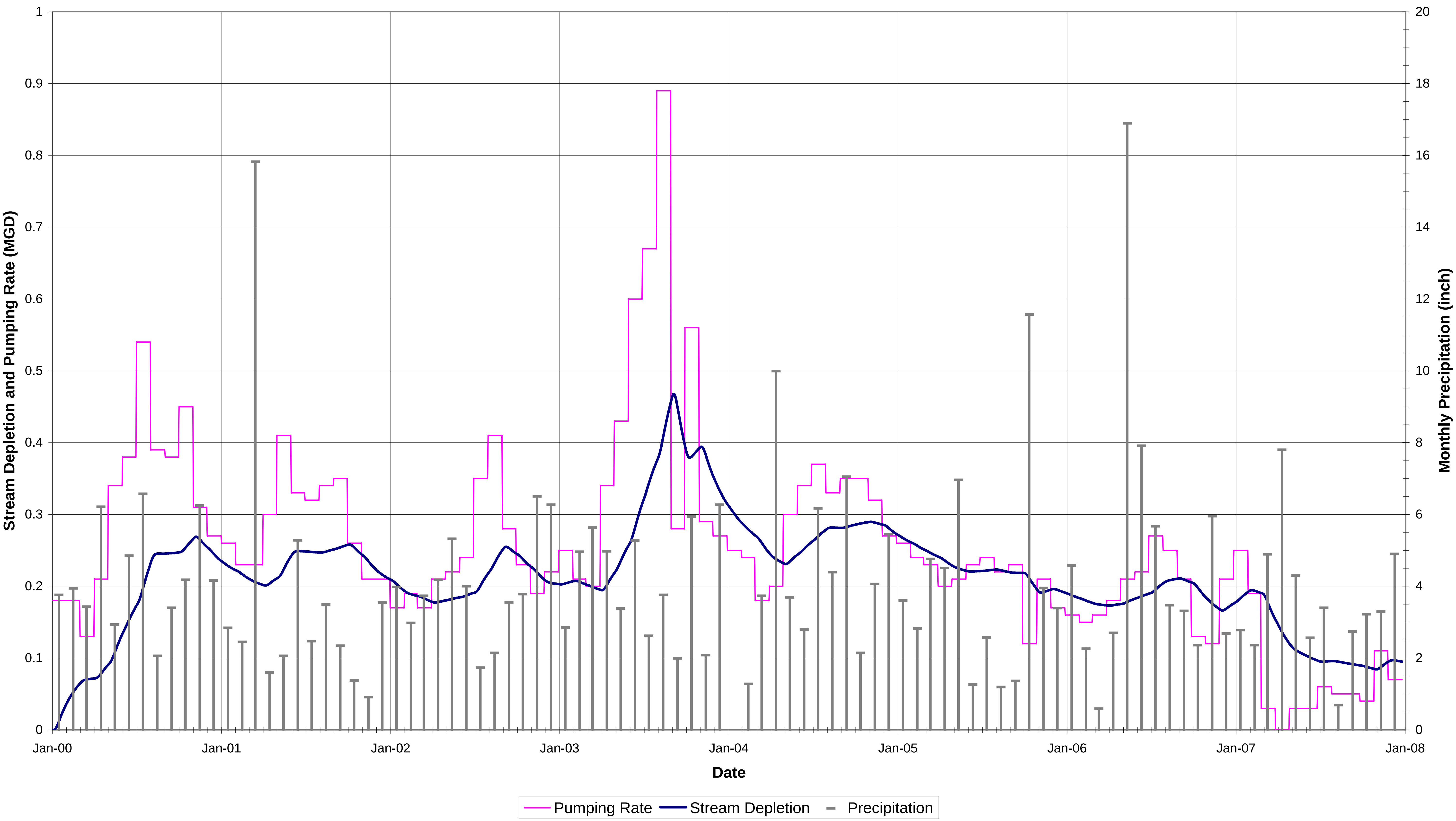
Daily well withdrawals are based on total monthly withdrawals for 2000-2007 (minimum = 0.00 MGD; maximum = 0.89 MGD; average = 0.25 MGD)

Figure C.4 Forrest Street Well Stream Depletion



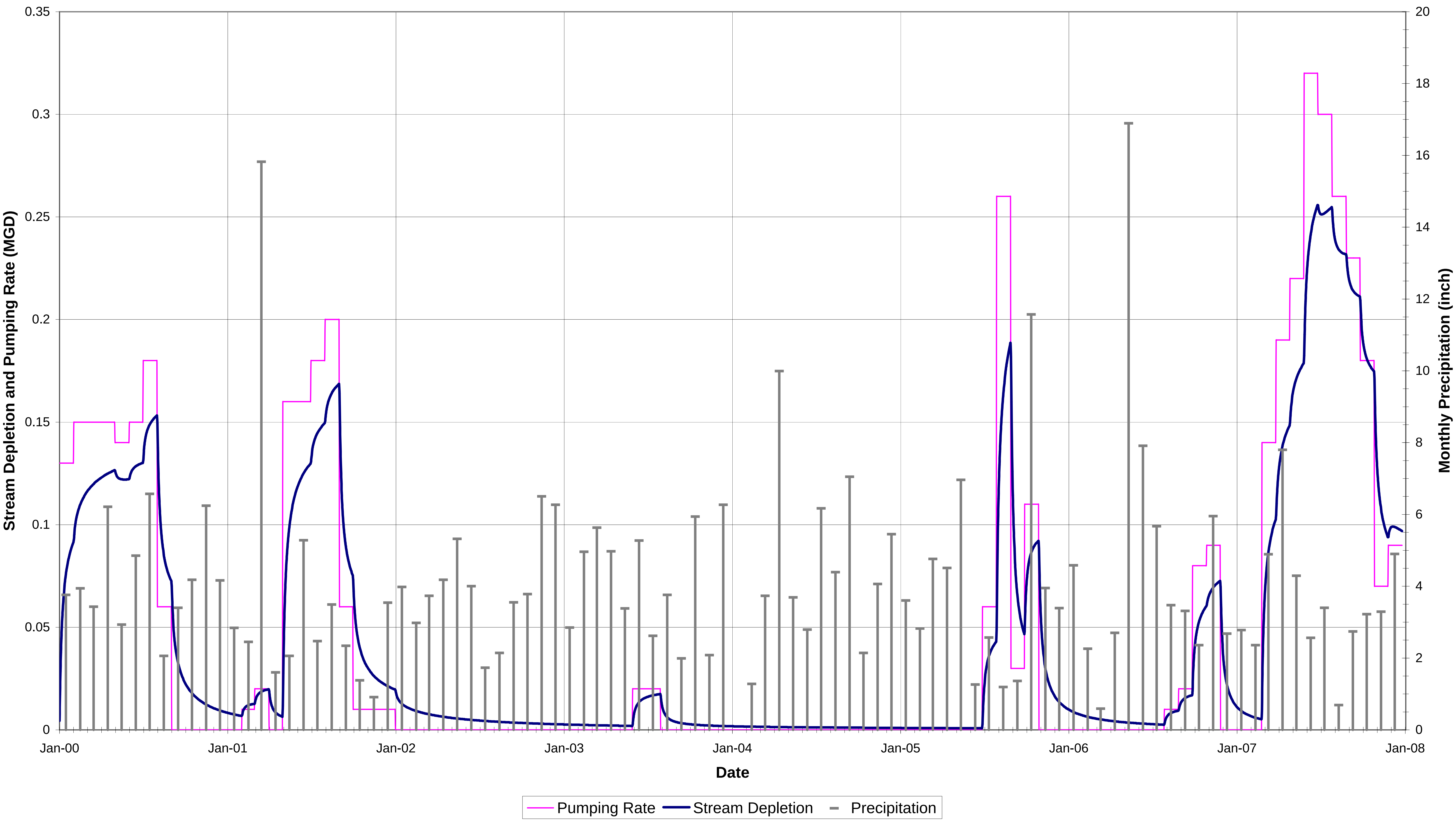
Daily well withdrawals are based on total monthly withdrawals for 2000-2007 (minimum = 0.00 MGD; maximum = 0.89 MGD; average = 0.25 MGD)

Figure C.5 WM Marshall Well Stream Depletion
Corrected for Partial Stream Penetrations and Semipervious Streambank



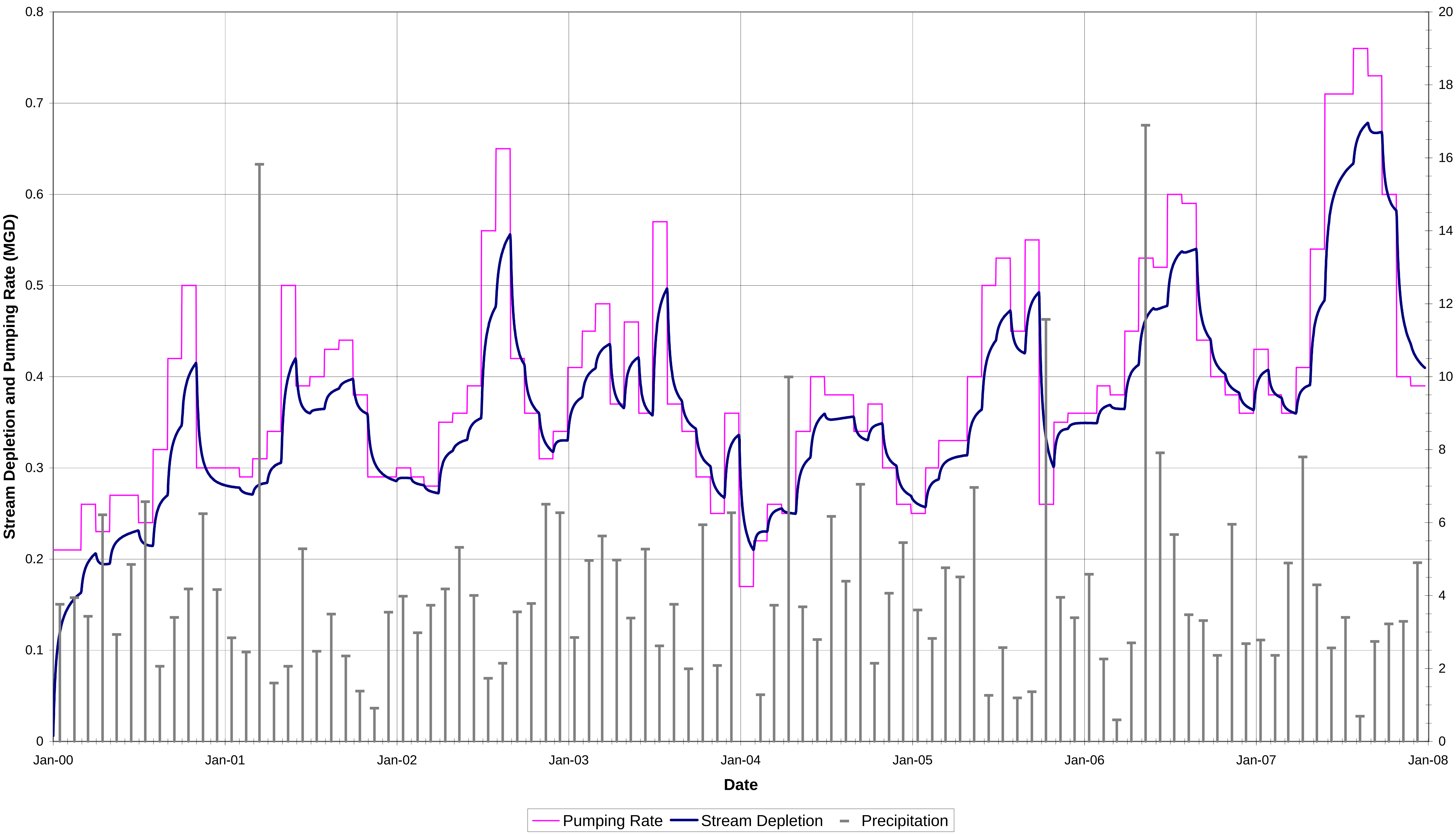
Daily well withdrawals are based on total monthly withdrawals for 2000-2007 (minimum = 0.00 MGD; maximum = 0.89 MGD; average = 0.25 MGD)

Figure C.6 Commissioner Well Stream Depletion
Corrected for Partial Stream Penetrations and Semipervious Streambank



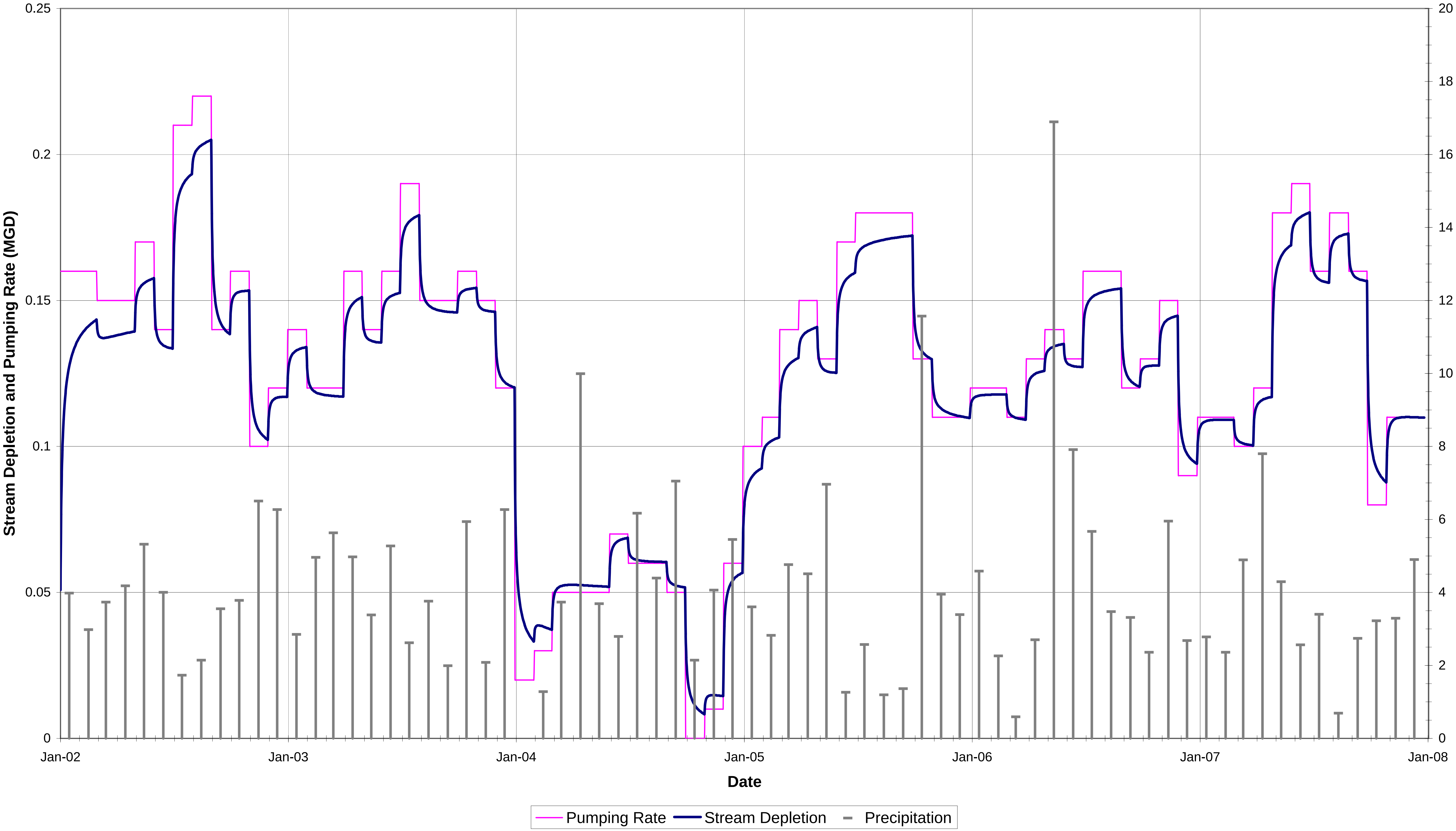
Daily well withdrawals are based on total monthly withdrawals for 2000-2007 (minimum = 0.00 MGD; maximum = 0.89 MGD; average = 0.25 MGD)

Figure C.7 Duffy Well Stream Depletion
Corrected for Partial Stream Penetrations and Semipervious Streambank



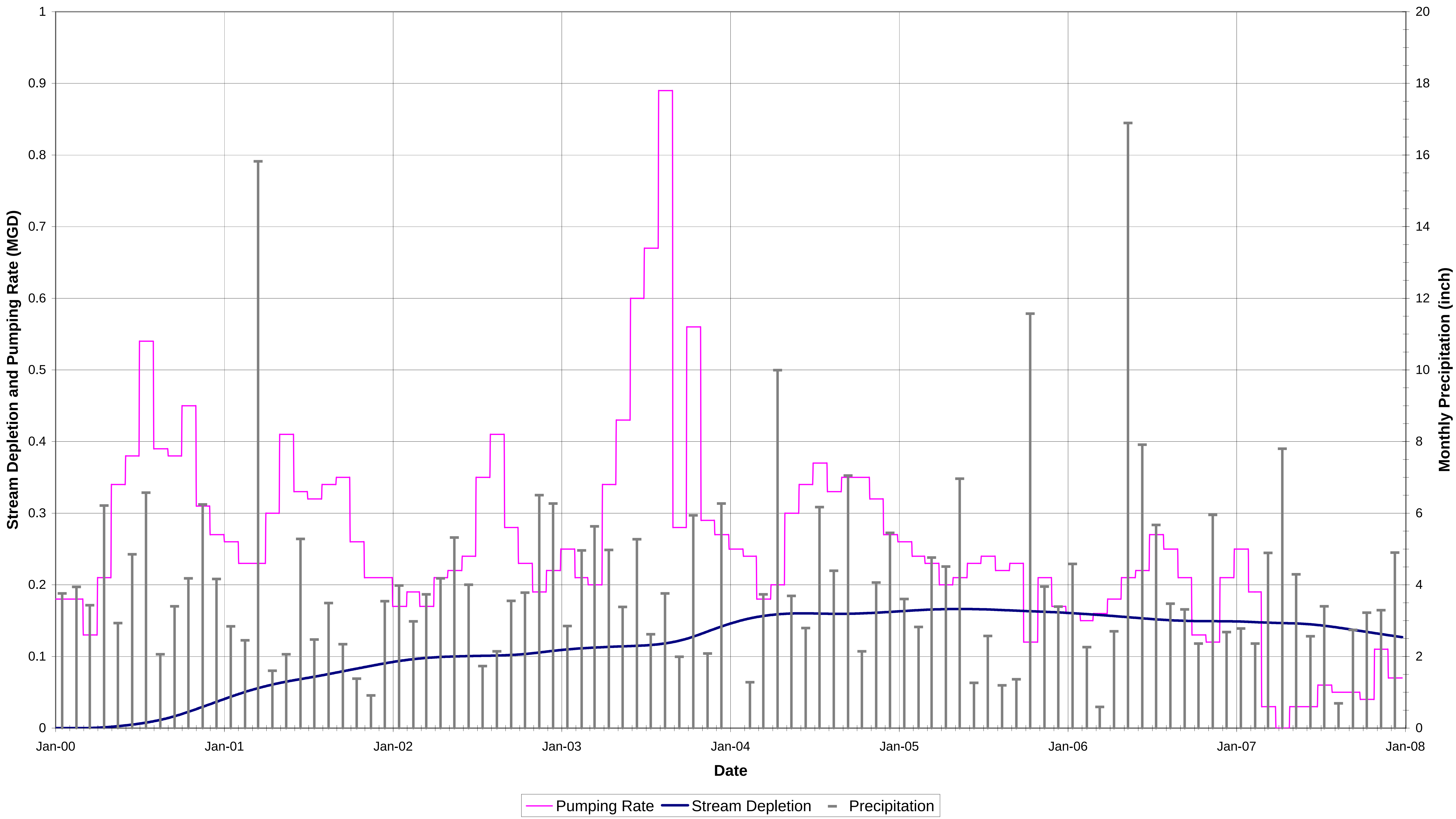
Daily well withdrawals are based on total monthly withdrawals for 2000-2007 (minimum = 0.00 MGD; maximum = 0.89 MGD; average = 0.25 MGD)

Figure C.8 Forrest Street Well Stream Depletion
Corrected for Partial Stream Penetrations and Semipervious Streambank



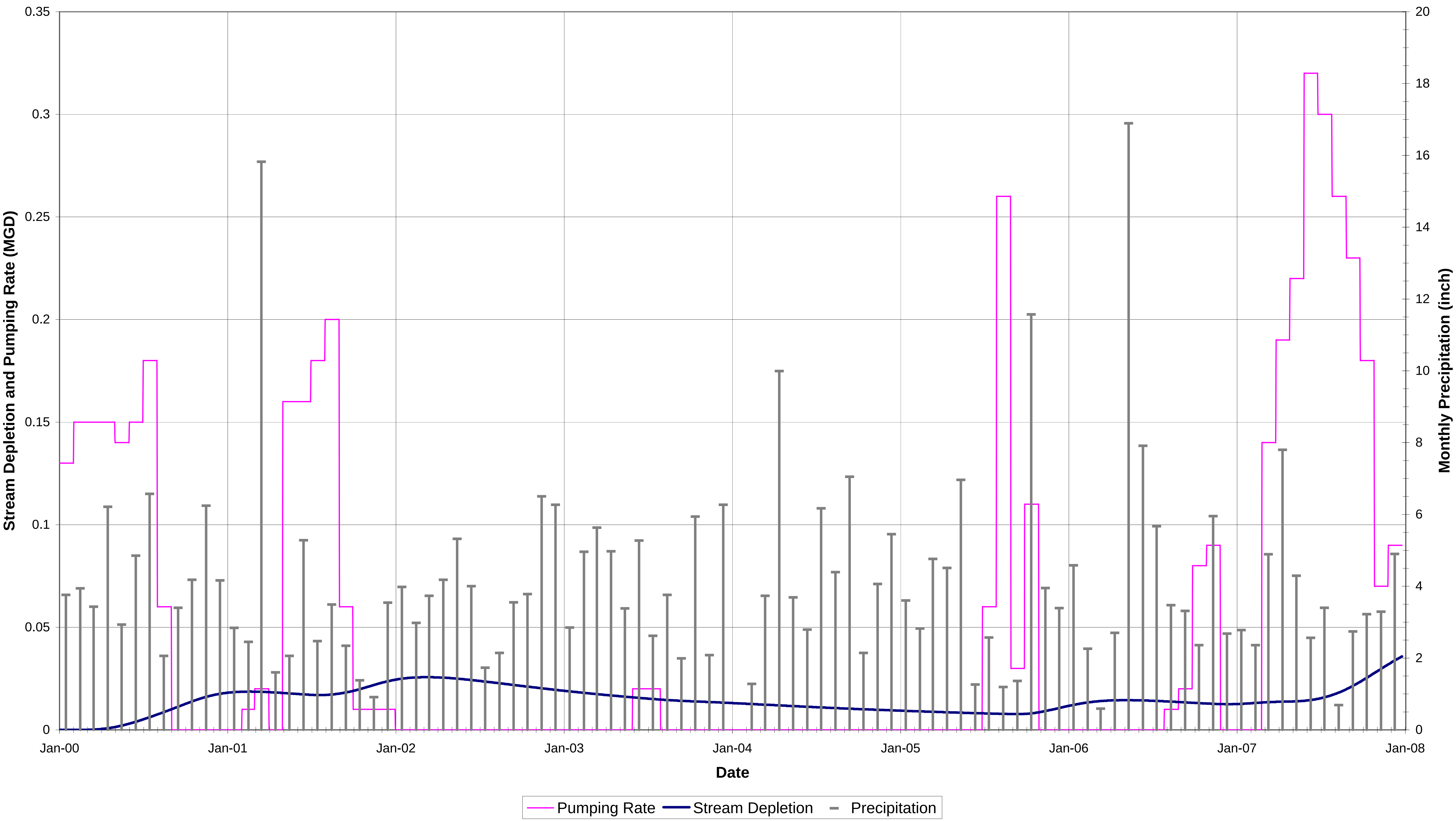
Daily well withdrawals are based on total monthly withdrawals for 2000-2007 (minimum = 0.00 MGD; maximum = 0.89 MGD; average = 0.25 MGD)

Figure C.9 Hypothetical (2,000 ft from river) WM Marshall Well Stream Depletion Corrected for Partial Stream Penetrations and Semipervious Streambank



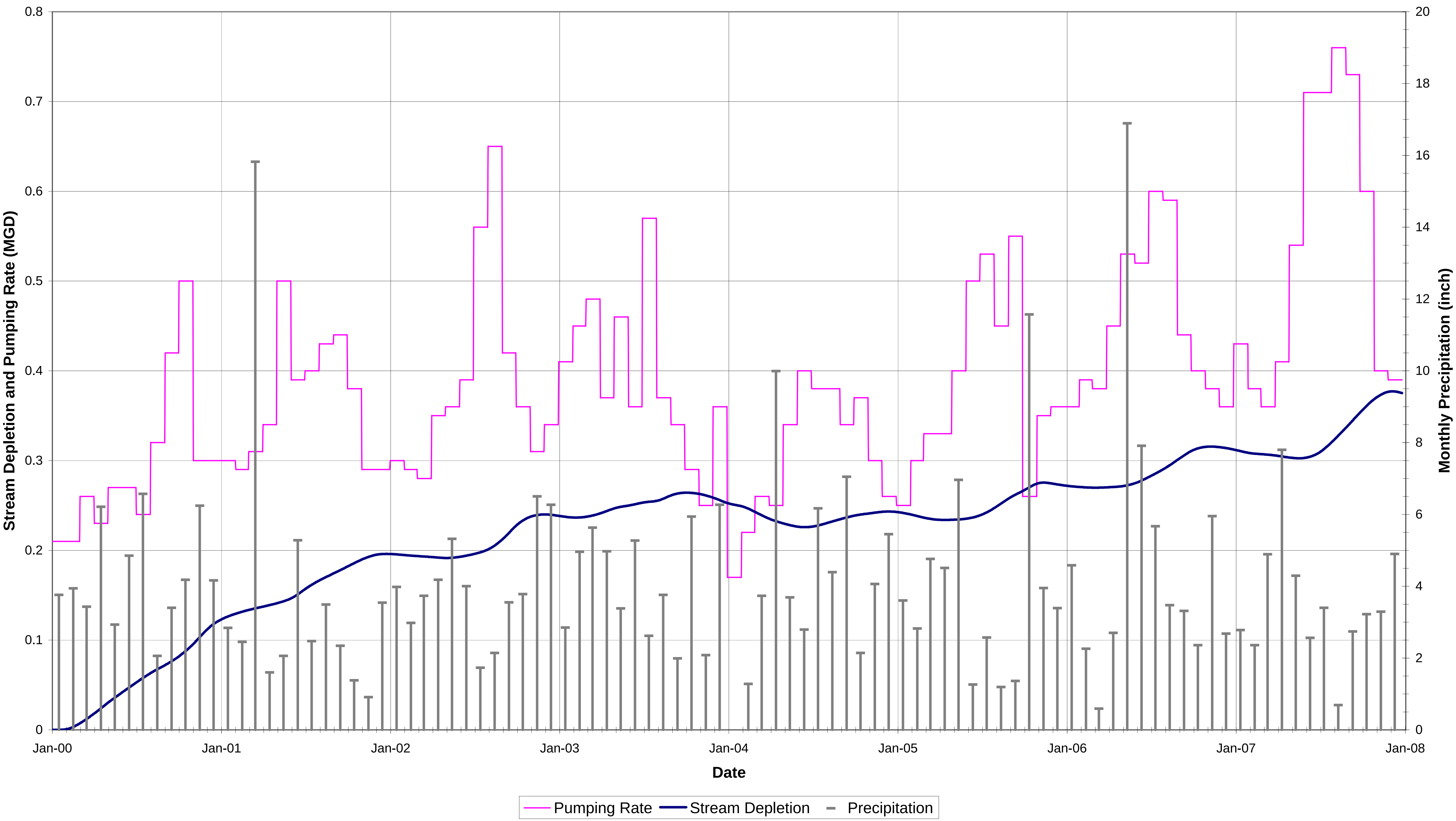
Daily well withdrawals are based on total monthly withdrawals for 2000-2007 (minimum = 0.00 MGD; maximum = 0.89 MGD; average = 0.25 MGD)

Figure C.10 Hypothetical (2,000 ft from river) Commissioner Well Stream Depletion
Corrected for Partial Stream Penetrations and Semipervious Streambank



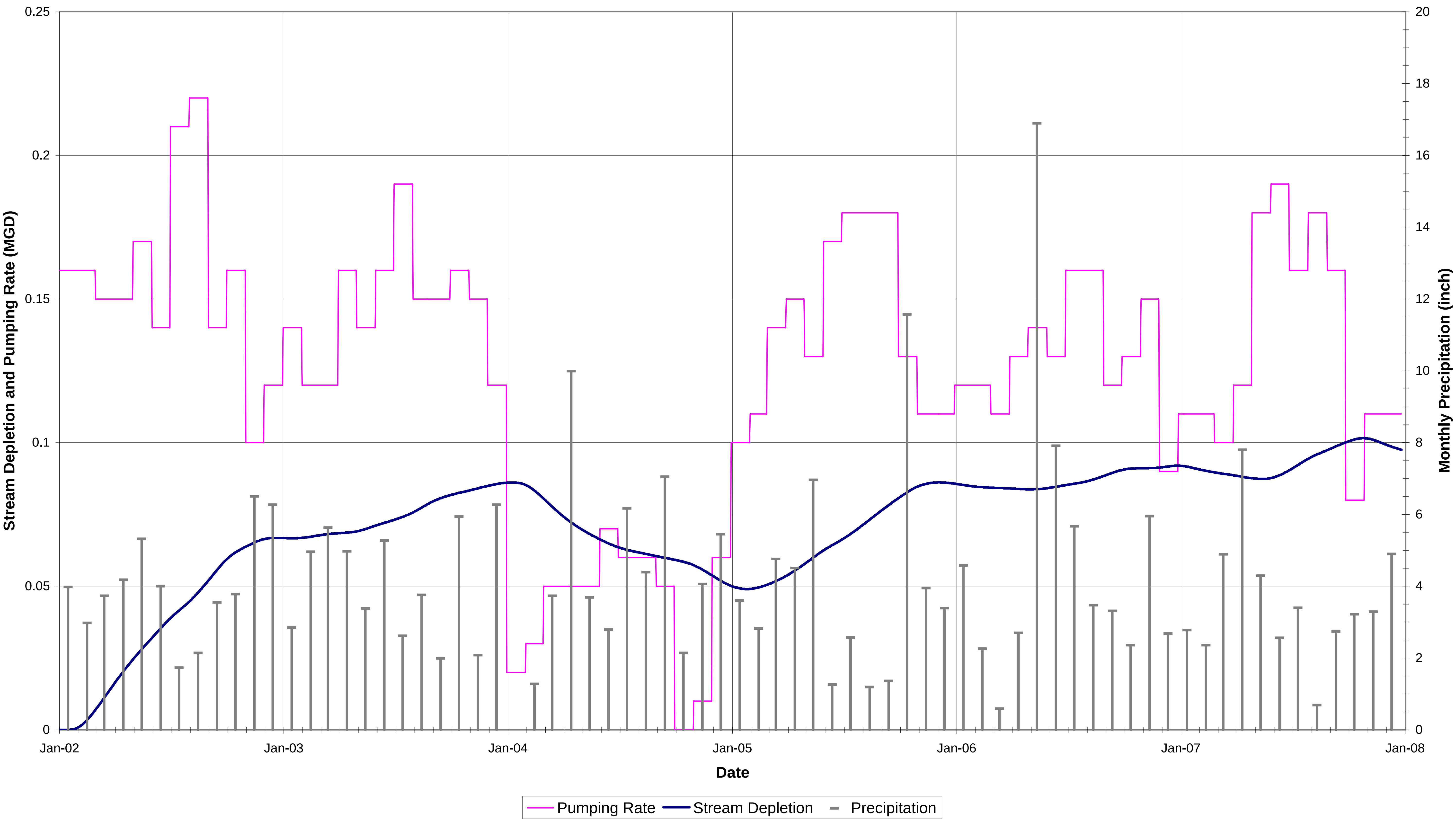
Daily well withdrawals are based on total monthly withdrawals for 2000-2007 (minimum = 0.00 MGD; maximum = 0.89 MGD; average = 0.25 MGD)

Figure C.11 Hypothetical (2,000 ft from river) Duffy Well Stream Depletion
Corrected for Partial Stream Penetrations and Semipervious Streambank



Daily well withdrawals are based on total monthly withdrawals for 2000-2007 (minimum = 0.00 MGD; maximum = 0.89 MGD; average = 0.25 MGD)

Figure C.12 Hypothetical (2,000 ft from river) Forrest Street Well Stream Depletion Corrected for Partial Stream Penetrations and Semipervious Streambank



Daily well withdrawals are based on total monthly withdrawals for 2000-2007 (minimum = 0.00 MGD; maximum = 0.89 MGD; average = 0.25 MGD)