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RELIABLE DROUGHT YIELDS OF PUBLIC SUPPLY WELLS IN THE FRACTURED ROCK AREAS OF CENTRAL MARYLAND

by

Patrick A. Hammond

2021

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CONVERSION FACTORS AND SYMBOLS

Multiply	By	To obtain
Length		
inch (in)	2.54	centimeter (cm)
foot (ft)	0.3048	meter (m)
mile (mi)	1.609	kilometer (km)
Area		
square foot (ft^2)	0.0929	square meter (m^2)
square mile (mi^2)	2.59	square kilometer (km^2)
Volume		
gallon (gal)	3.785	liter (l)
gallon (gal)	3.785×10^{-3}	cubic meter (m^3)
Discharge Rate		
gallon per minute (gpm)	0.063	liter per second (L/s)
gallon per minute (gpm)	3.785	liter per minute (l/min)
Production Rate		
gallon per day (gpd)	3.785×10^{-3}	cubic meter per day (m^3/d)
Annual average use gallons per day		
gallons per day average (gpd avg)		
Use during the month of maximum		
use gallons per day maximum (gpd max)		
Transmissivity		
gallon per day per foot (gal/d-ft)	0.0124	square meter per day (m^2/d)
square foot per day (ft^2/d)	0.0929	square meter per day (m^2/d)

Use of notation: As close as possible, the original scientific or mathematical notations of any papers discussed have been retained, in case a reader wishes to review those studies.

RELIABLE DROUGHT YIELDS OF PUBLIC SUPPLY WELLS IN THE FRACTURED ROCK AREAS OF CENTRAL MARYLAND

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KEY RESULTS

In Maryland, there have been relatively few investigations on estimating reliable, long-term drought yields of fractured rock wells. In this study, "reliable drought yield" is defined as the amount of water that a well can supply to meet peak summertime demand during a severe drought without having to impose water restrictions. During two droughts that occurred between 1998 and 2002, many municipal water suppliers in the Piedmont/Blue Ridge (fractured rock) areas of central Maryland had to institute water restrictions related to declining well yields. Estimates of the yields of those wells, made prior to placing them into service, were commonly based on extrapolating drawdowns, measured during short-term pumping tests to shallowest, primary or major, water-bearing fractures and often did not consider drought conditions. Because the extrapolations were often made from apparent pseudo-equilibrium phases of drawdown curves, this frequently resulted in substantially over-estimated well yields. The methods used to estimate reliable drought yields in this study consist of (1) extrapolating drawdown data from infinite acting radial flow (IARF) periods, which includes pseudo-radial flow in fracture-controlled aquifers, or (2) by fitting type curves of other conceptual models to the data, using combinations of diagnostic plots, and inverse analysis and derivative analysis techniques. The positions of transition zones in crystalline rocks or thin-bedded consolidated sandstone/limestone layers (reservoir rocks) were used to determine available drawdowns in the wells. Aquifer dewatering effects were detected by type-curve matching of step-test data or by breaks (sharp changes in slopes) in the drawdown curves constructed from aquifer pumping tests. Operational water use and water-level data collected by water system personnel were then used to confirm the predicted yields. Those data were also compared to changes in regional groundwater levels to determine seasonal variations in well yields. Estimates of reliable drought yields are critical for effective design of production wells in fractured rock aquifers in central Maryland. Additionally, long-term monitoring should be conducted to verify those estimates and provide evidence of any decline in yield due to changes in aquifer properties or mechanical malfunctions of the well.

INTRODUCTION

The State of Maryland is in the Mid-Atlantic region of the eastern United States and has a wide range of geology and aquifer types. Well yields typically vary from those that are relatively high (commonly more than 500 gallons per minute [gpm]) in confined and unconfined, unconsolidated sand aquifers on the eastern shore and in southern Maryland to relatively those that are low (generally less than 100 gpm) in the fractured rock aquifers of the Piedmont, Blue Ridge, Valley and Ridge, and Appalachian Plateau provinces of central and western Maryland. The state includes much of the major Washington-Baltimore metropolitan region, where more than 5 million people live. Most of the metropolitan area is served by surface water from the Potomac River and associated reservoirs, and the Baltimore City reservoir system and, in an emergency, diversion from the Susquehanna River. Some of the fastest growing suburban areas, however, are in the Piedmont and Blue Ridge areas, and are supplied by small reservoirs and/or wells in fractured rock aquifers.

There was a prolonged drought in the State during the period 1998–2002, culminating in one of the three worst droughts on record: the others occurring in 1930–1932 and 1962–1969. The major surface water suppliers in the metropolitan area had little difficulty meeting customer demand, due to their substantial reservoir storage facilities. Many of the small to medium size towns or cities (populations less than 10,000 people), however, were required to institute voluntary or mandatory water restrictions, primarily due to declining well yields. The declining well yields were commonly attributed to the drought; however, this was largely speculation with no supporting data. An earlier study, Hammond (2004) by the Maryland Department of the Environment indicated that initial predictions of the reliable yields of public supply wells in the fractured rock areas of the State had been substantially overestimated. After the 1998-2002 drought, a testing and evaluation program was instituted by the State to develop better methods for estimating reliable drought well yields in fractured rock wells. This report presents some of the findings from that program.

PURPOSE AND SCOPE

The purpose of this report is to present methods for improving the estimates of reliable drought yields of wells in the fractured rock aquifers of Maryland through case histories, examples, and analysis of field data. A total of more than 200 step-drawdown and aquifer tests along with well pumpage data, regional groundwater levels, geologic features, and climate data (precipitation) were analyzed for this study. Well-construction records (Appendix A2), lithologic (drillers') logs and geologic/geophysical logs were also used in the analysis of reliable drought yields. The study focuses on the municipal or community water supplies of Emmitsburg, Thurmont, Point of Rocks, Myersville, Middletown, Fountaindale, Poolesville, Mount Airy, Westminster, and Taneytown, plus irrigation wells at the Musket Ridge Golf Club. These water systems were chosen because they illustrate characteristic responses during step-drawdown and aquifer tests and have, in most cases, long-term test and/or operational data to help verify the estimated reliable yields of the wells.

LOCATION OF STUDY AREA

The locations of the study sites discussed in this report are shown in Figure 1. The study area is in central Maryland, and includes Carroll, Frederick and Montgomery Counties, a portion of which is part of the Baltimore-Washington Metropolitan region.

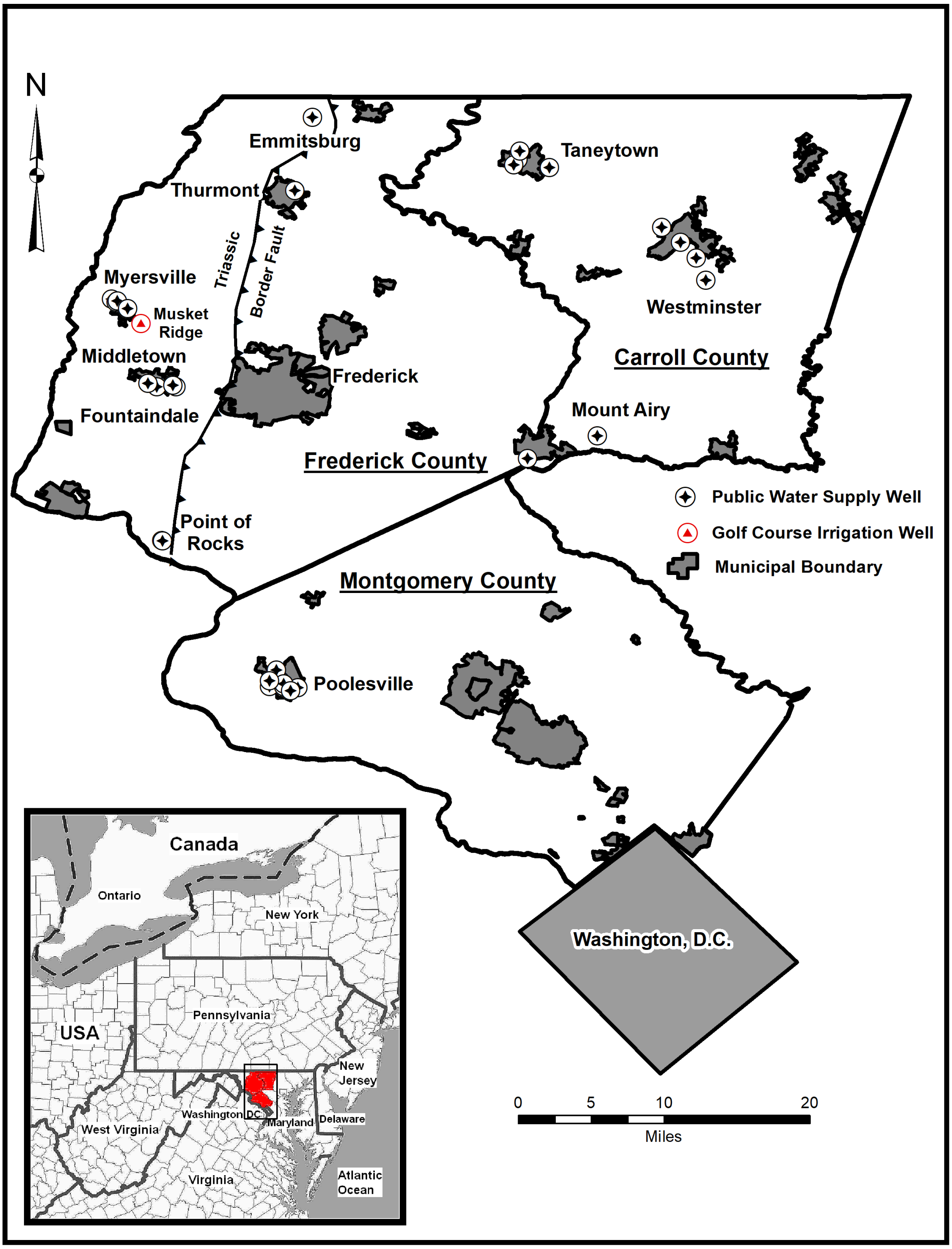

Figure 1. Location of the study area.

METHODS OF INVESTIGATION

A common method for analyzing aquifer test data in the past was to use graphical type-curve matching techniques developed for various analytical models. The one most frequently applied is the two-dimensional solution describing infinite acting radial flow (IARF). Other models solve for linear followed by pseudo-radial flow, pseudo-equilibrium, delayed yield and dual porosity responses, and leaky aquifers. These graphical methods are prone to errors in individual judgment, because different flow models can provide relatively good visual fits to the same set of data.

In addition to the graphical type-curve matching techniques, this study utilized specialized diagnostic plots, inverse analyses using a computer-assisted automatic curve fitting program, and derivative analysis and deconvolution solutions to analyze step-drawdown, other variable-rate, and constant-rate aquifer tests. The analysis of aquifer-test data also included a determination of the presence of internal or external boundaries, and the effects of aquifer dewatering. Once a solution was derived, the drawdown data were extrapolated forward to produce an estimated yield, for a target operating water level in a well. The result is then verified by comparing the estimate to withdrawal data. The well yield can then be adjusted by relating it to regional groundwater levels and rainfall data to provide information concerning seasonal variations in yield. The technique for estimating well yields was compared to the results of two published synthetic numerical models.

PREVIOUS STUDIES

In a literature review, Hammond (2018) found that most studies of fractured rock aquifers have focused on contaminant transport, derivation of hydraulic constants from aquifer-test data or estimates of safe aquifer yields on a watershed or regional basis. The Hammond (2018) study was conducted, in part, because there were few studies available on how to predict reliable yields of individual wells in fractured rock aquifers. Of those that attempted to predict reliable yields, the estimates were often inaccurate when compared to actual production data.

For carbonate aquifers under water-table or semi-water-table conditions, Parizek and Siddiqui (1970) indicated that well yields are determined more by the position of the water-table with respect to producing zones than the proportion of the saturated rock penetrated by the well bore. They also suggested that a permeability profile be determined along with available drawdown and inclusion of water bearing zones. Their study included the results of an extensive exploration program to develop groundwater supplies for Pennsylvania State University and the Borough of State College. UN-26 is the only one of 11 university production wells in that study whose yield as a function of pumping level was well known. UN-24 and UN-17 were each producing less than ½ of design capacity and UN-16 was abandoned. From those wells, the total production was an average of 61 percent of the estimated yield. There were no production data for the remaining seven wells. The authors indicated that the lower than estimated yields were due to the difficulty in predicting yields, drought effects, and well interference. The authors indicated that it is difficult to predict long-term yields in anisotropic aquifers, where extensive dewatering of aquifer materials is involved, permeability is highly anisotropic, and the effects of drought and well interference are present.

Daniel (1990) proposed techniques for maximizing sustained yields of 11 new and 2 existing wells completed in Piedmont crystalline rocks for the Town of Cary, North Carolina public water supply. After the results of initial 24-hour aquifer tests were used to estimate yields of individual wells, the well field was placed on a 5-day on and 2-day off operating schedule, starting at the estimated yields of the wells. After several weeks, the water levels in the wells reached low-level cutouts, causing the well pumps to shut off and start to cycle. The flow rates were then reduced, but several weeks later the cycling began again. The wells with the most problems were those in which the main water bearing zones (wbzs) were many feet above the pump intake. The well field was then changed to 18-hour on and 6-hour off daily operating schedule. While this appears to have prevented water levels from reaching low-level cutouts, certain wbzs were still dewatered. A monitoring program was initiated that included the measuring and recording of daily production, pumping rates, number of hours pumped, water levels, and other operational data. The total daily production during the period May 1983 to Dec 1984 was an average of 596,000 gallons per day (gpd) or 60 percent of the anticipated yield of the well field. The production from six wells (382,000 gpd total) was 52 percent of the estimates made from the 24-hour pumping test data (727,000 gpd total) of those wells.

Misstear and Beeson (2000) used operational data to establish reliable yields in a shallow, unconfined, fissure flow (UK Chalk) aquifer. Their calculations were based on extrapolation of drawdowns using the Cooper-Jacob (1946) straight-line method and assuming 200 days of continuous pumping, during a dry summer under average demand conditions. The drawdowns at 100 minutes from step-test data were chosen to determine well losses and were then added, to produce a total drawdown at the depth of the first water strike. No yields were directly predicted, because the results were achieved by fitting yield curves to existing operational data. Available drawdown appears to have been based on the level of main inflow zones, since the authors indicated that where the locations of inflow zones in a fissured aquifer were unknown, only 50% of the aquifer should be dewatered. Water level data from only one observation well, located near a high-capacity production well, were used to make the prediction of drought yield; however, the water level change in that well (108 feet or 33 m), along with an especially large decline after 1988, indicate that the water levels may have been affected by withdrawals from the nearby production well.

Van Tonder, Botha, Chiang, Kunstmann and Xu (2001a) used derivative analysis and semi-analytical techniques to develop their Flow Characteristic (FC) method for estimating yields of wells in the layered, sedimentary rock (multi-porous) Karoo aquifer in South Africa. That paper presents a semi-analytical method and two numerical methods. The semi-analytical method, called the Method of Derivative Fitting (MDF) involves characterizing the various flow periods in fractured rock aquifers with numerical approximations of the first logarithmic derivative of drawdown (the derivative of drawdown with respect to the logarithm of time) to identify the conceptual model needed in such analyses. Semi-analytical methods were used to estimate the influence of boundaries and to quantify the errors in estimates of aquifer parameters. These approximations and the MDF comprise the Flow Characteristics method (FCM) developed by the authors. The authors indicated that the MDF method is applied only to layered sedimentary rocks (multi-porous aquifer). The conceptual models involved three types of flow: radial flow (Theis, 1935), flow in vertical and horizontal fractures (Gringarten and Ramey, 1975), and flow in a dual porosity medium (Moench, 1985).

Using data from a test of a well, UO5, at an extensively monitored research test site at the University of Free State, Bloemfontein, Free State, South Africa, the authors conducted a numerical analysis to verify their model, where the available drawdown was related to the depth of a single, discrete horizontal fracture. Van Tonder, Botha, and van Bosch (2001b) proposed that natural fractures may close over time, due to deformation of an aquifer as water levels in a well decline. In that study, a step-test of well UP16 was conducted to identify the effects of non-linear deformation of the aquifer. Those authors indicated that the calculated yield from the second test agreed with that obtained during the earlier study (Van Tonder et al., 2001a). However, it is noted that each test was conducted on a different well (UO5 and UP16). When even near each other, it is common for two wells in fractured rock aquifers to have very different yields. In this case, however, because the authors indicated that both wells were located on the same primary fracture, it is possible that the yields of the wells could be similar. It is expected, however, that there would be severe interference between the two wells if they were placed in service. Although these were two comprehensive studies, no operational data were presented to confirm the authors' estimates of long-term well yields.

Piscopo and Summa (2007) conducted a one-year, constant drawdown test of a carbonate rock well; however, the drawdown, 39.4 feet (12 m), was only 18 percent of the potentially available drawdown. The drawdown was limited to that level to produce an estimated well efficiency of 75 percent, based on the results of a step-test of the well analyzed using the Jacob (1947) method. Their estimated yield, 14.9 gpm (0.94×10^{-3} m^3/s), was only a fraction of the production, 32−79 gpm ($2{-}5 \times 10^{-3}$ m^3/s), from the well during the year-long test. The higher than predicted yields were subsequently verified by a numerical model developed by Baiocchi, Lotti, Piecentini and Piscopo (2014).

The results of the Van Tonder et al. (2001a, b) and Piscopo and Summa (2007) testing are described in greater detail in the section on examples of synthetic models.

ACKNOWLEDGEMENTS

This study fulfills one of the objectives of a cooperative regional study (USGS Publication SIR 2012-5160) of the fractured rock areas of Maryland that involved the Maryland Department of the Environment, the Maryland Geological Survey, the U.S. Geological Survey and the Monitoring and Non-Tidal Assessment (MANTA) division of the Maryland Department of Natural Resources. Reviewers of the report, Dr. Malcolm S. Field and Dr. Emelia Furlong, and technical editorial advisors, David Andreasen and Andrew Staley provided many useful comments and suggestions that greatly improved the content and organization of the manuscript. The views expressed in this report do not necessarily reflect the views or policies the Maryland Department of the Environment.

GENERAL HYDROGEOLOGY AND GEOLOGY OF CENTRAL MARYLAND

The study area is in central Maryland and consists of parts of the major Blue Ridge and Piedmont physiographic provinces. The Piedmont province in this area has been further subdivided into the Western Piedmont and Mesozoic Lowland provinces. Bachman, Lindsey, Brakebill and Powers (1998) combined physiographic provinces with generalized lithology to define eleven hydrogeomorphic regions (HGMRs) for the Chesapeake Bay watershed. Figure 2 shows the four HGMRs that are within the study area; the Piedmont Crystalline (PCR), the Piedmont Carbonate (PCA), the Mesozoic Lowland (ML), and the Blue Ridge (BR) regions. The rock types generally consist of carbonates and consolidated sedimentary rocks in the central lowland areas (Frederick and Wakefield valleys, and the Gettysburg and Culpeper basins), and metamorphosed volcanic, volcaniclastic, and epiclastic rocks in the eastern portion of the study area and the Blue Ridge Mountains to the west. All discussions of geologic formations are based on the Geologic Map of Maryland (Cleaves et al.,1968), with some minor changes, William Junkin (personal communication, 2019).

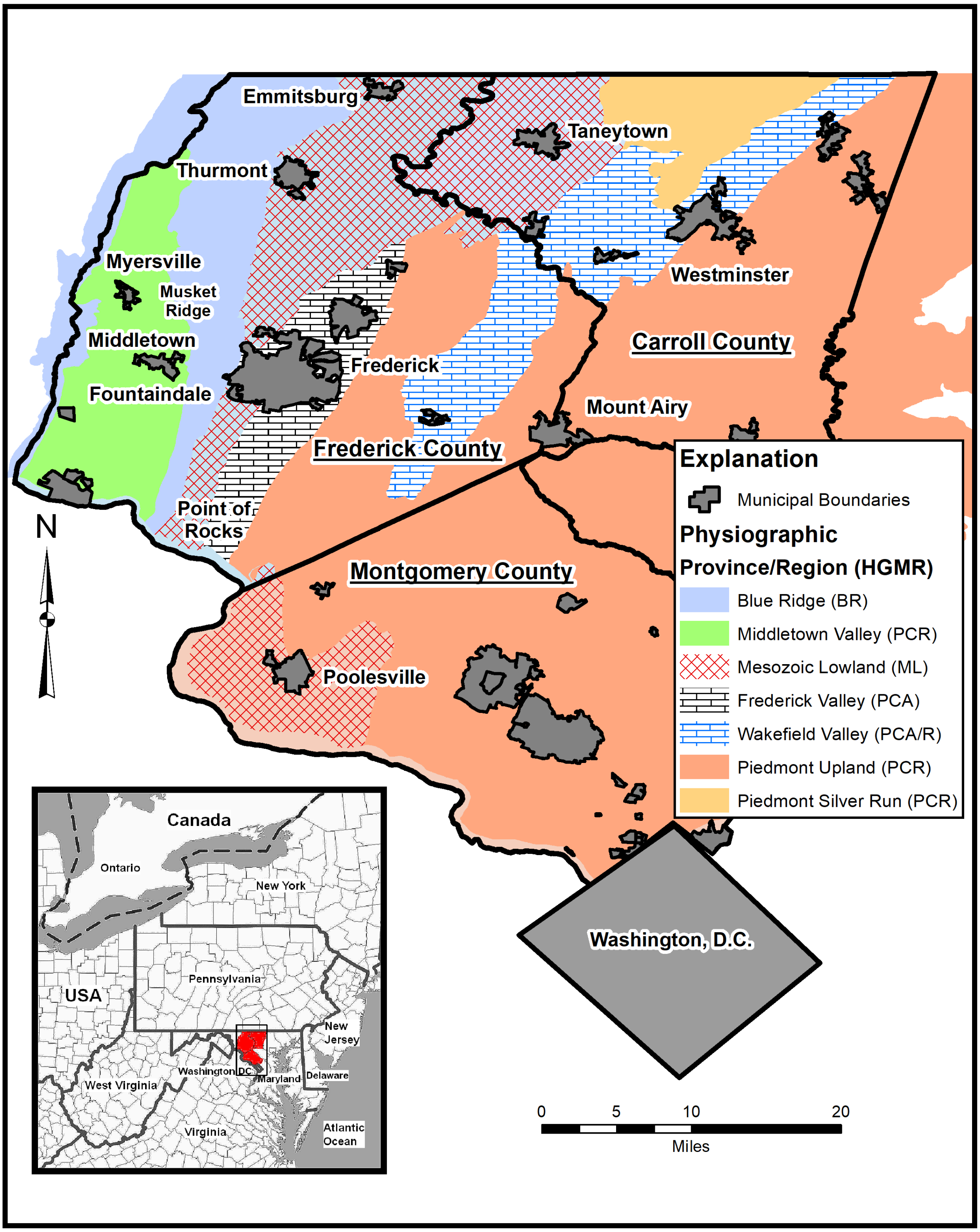

Figure 2. Physiographic and hydrogeomorphic (HGMR) regions of the study area. The HGMRs are the Blue Ridge (BR), Mesozoic Lowland (ML), Piedmont Carbonate (PCA) and Piedmont Crystalline (PCR) regions.

The Piedmont Crystalline HGMR (PCR) occurs in the eastern portion of the study area and the Middletown Valley to the west. It is primarily underlain by Precambrian and Cambrian metamorphic and igneous rocks, including, in some areas, metacarbonates and metaquartzites. Two prominent formations underlie most of the eastern area. The Marburg Schist is a bluish gray to silvery green, fine-grained schist that underlies parts of Carroll, Frederick, and Montgomery counties. The Ijamsville Formation consists of blue, green, or purple phyllite and phyllitic slate and interbedded metasiltstone and metagraywacke rock units and underlies an area of approximately 100 square miles (260 square kilometers) in Frederick and Carroll counties. Intermingled with the Ijamsville Formation are other metavolcanic and carbonate rocks, located primarily in the Wakefield Valley. Westminster and Mount Airy are in the eastern PCR region. The Middletown Valley is flanked by South Mountain to the west and Catoctin Mountain to the east. It is underlain by Precambrian granitic gneisses and metavolcanic rocks, intruded by metadiabase dikes. Myersville, Middletown, Fountaindale and the Musket Ridge Golf Club are in the Middletown Valley.

Figure 3 is a conceptual profile of a Piedmont crystalline rock aquifer, taken from Harned and Daniel (1992). They indicated that the regolith consists of saprolite, soil and alluvium, and contains most of the groundwater stored in crystalline-rock aquifers. Maximum porosity occurs near the contact or transition zone between the regolith and un-weathered bedrock. The regolith exists under water-able conditions and is hydraulically coupled by leakage to locally confined bedrock. Evidence of this connection is that water levels in wells completed in bedrock follow topography and their yields vary seasonally in response to changing climatic conditions. Well drillers frequently report "sand" material and boulders in the transition zone, and that it is often where the first groundwater occurs in a well. The higher hydraulic conductivity in the transition zone relative to upper portion of the regolith is due to a lesser degree of weathering. Chemical weathering of the bedrock causes certain minerals to expand, especially feldspars, producing extensive fracturing of the bedrock, but not the formation of clays that clog the fractures. Fractures are more numerous and have the largest openings near the top of bedrock, typically less than depths of 100 ft (30 m). As depth increases, lithostatic pressure tends to inhibit the formation of fractures, such that the permeability decreases and total porosity typically is less than one percent, Daniel (1990). The optimum well depth in the Piedmont crystalline rocks of central Maryland is about 300 ft (91 m), though some wells may be 500 or more feet (152 m) deep (Richardson, 1980).

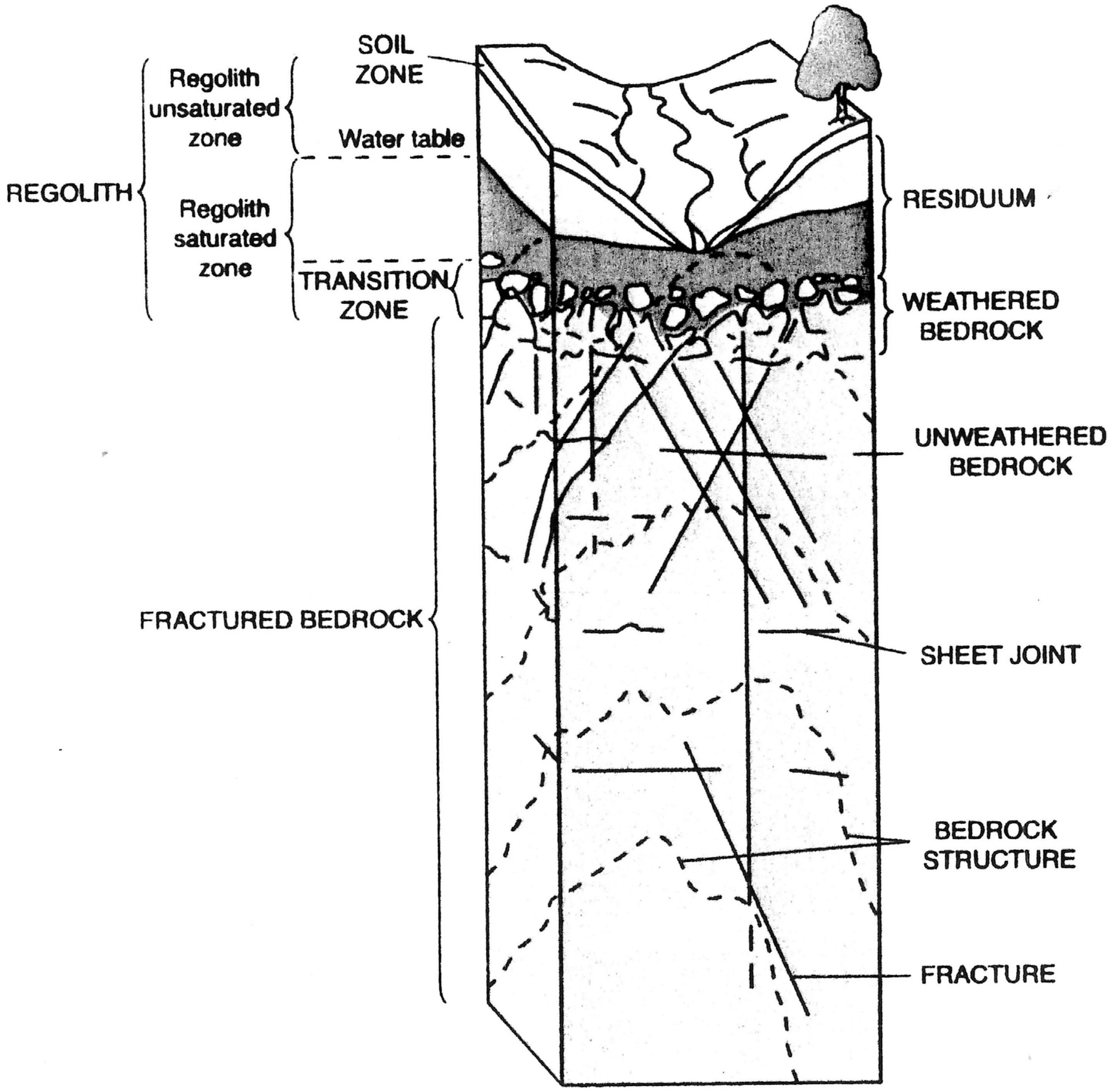

Figure 3. Conceptual profile of a Piedmont crystalline rock aquifer (from Harned and Daniel, 1992).

The Piedmont Carbonate HGMR (PCA) is represented in the Monocacy River Basin in the central part of Frederick County. The rocks underlying Frederick Valley form a syncline, bounded on the west by the high-angle reverse Triassic Border Fault and in the east by the Piedmont Upland. It is typically a light colored thin-bedded dolomite with some gray limestone, which weathers to a red clay. Most of the floor of the Frederick Valley is underlain by the Frederick Limestone, which is a thin-bedded, dark colored clayey limestone and weathers to slab-like medium-colored layers. The Grove Limestone overlies the Frederick Limestone in a narrow strip in the central part of the Frederick Valley and is a massive pure limestone, with a fine-grained dolomite in the lower part and a basal highly quartzose limestone. The Tomstown Dolomite and Frederick Limestone are exposed in a narrow belt along the foothills of Catoctin Mountain and adjacent to the Triassic border fault.

The Mesozoic Lowland (ML) HGMR is present in central and northeastern Frederick County, northwestern Carroll County, and western Montgomery County. This HGMR is characterized by its underlying geology of Triassic sedimentary rocks and Jurassic intrusions. The Triassic rocks north of the City of Frederick are part of the Newark-Gettysburg basin, which extends from the New York City area to Frederick. The Triassic rocks south of Frederick are part of the Culpepper basin, which extends from Frederick to near Charlottesville, Va. The Triassic rocks in the study area are comprised primarily of the Gettysburg Formation and the underlying New Oxford Formation (Cleaves et al. 1968; Nutter,1975; Otton, 1981; and Duigon and Dine, 1987). In the Culpepper basin, the correlative unit to the New Oxford Formation is the Poolesville Member of the Manassas Formation (Brezinski, 2004). To be consistent with the nomenclature used by the Maryland Department of the Environment Water Supply Program, the New Oxford Formation name is retained.

The Gettysburg Formation generally consists of a soft, reddish-brown shale containing interbedded siltstones, sandstones, and quartz and limestone conglomerates. It is exposed in the western part of the ML portion of the study area, in the vicinity of the towns of Emmitsburg and Thurmont

The New Oxford Formation consists of an interbedded sequence of sandstones, siltstones, shales, and conglomerates. The sandstone beds are lenticular, are not regionally extensive, and appear to be more competent and have denser fracture networks than the shale units. The residuum in Triassic-rock aquifers can be thin and may not extend below the zone of saturation. The mean porosity of Triassic sandstones and conglomerates is 6 percent (Otton, 1981, Table 5), and in some places, may be even higher, due to secondary solution of calcite cementing materials.

The Blue Ridge HGMR (BR) is in the western part of Frederick County and is bounded on the east by the Triassic Border Fault. The rocks of the Blue Ridge HGMR are principally Late Precambrian metavolcanic rocks that make up the Catoctin Formation and include metabasalt, metarhyolite, and greenstone schist rocks. There are, also, minor quartzite and phyllite, units in the HGMR. Small areas on the up-thrown side of the Triassic Border Fault are underlain by the Tomstown Dolomite and Frederick Limestone, but those units are part of the Piedmont Carbonate HGMR (PCA).

ANALYTICAL TECHNIQUES USED TO ESTIMATE WELL YIELDS

Hammond (2018) noted that previously published studies used different methods to predict reliable well yields. Only one study used derivative analysis techniques. Of the ones that included operational/production data, none of those data confirmed the estimated yields in the respective studies. No study provided evidence demonstrating the effects of aquifer dewatering. With one exception, the depth to discrete water-bearing fractures were used to determine available drawdown. In Maryland, extrapolations of drawdown data were often made from pseudo-equilibrium phases, something that derivative analyses could help prevent. Also, while the step-test was used primarily to determine the optimum rate for an aquifer test, initial rates were commonly too high, requiring rate adjustments, which makes it difficult to determine what flow regimes are present.

Analytical Models

Prior to the early 1990s and before the development of the methods in this study, it was common for the Maryland Department of the Environment Water-Supply Program in groundwater appropriation permitting to conduct an aquifer test in a manner to cause the water level in a pumping well to stabilize at a constant level. Frequently, the well was initially pumped at a relatively high rate to draw the water level down to, or very near, the shallowest, primary or major, water-bearing fracture. This was followed by reductions in the pumping rate, until stabilization or apparent equilibrium of the water level occurred. In other cases, either an apparent equilibrium was achieved without changing the pumping rate or a steady rate of drawdown occurred, and those data were extrapolated to the shallowest, major, water-bearing fracture. As part of this study, a review of the literature was conducted to determine which analytical methods could best be used to determine reliable yields of fractured rock wells.

The model used to analyze pumping test data in the early 1990s was the two-dimensional solution developed by Theis (1935), or the infinite acting radial flow (IARF) model, which assumes confined conditions in a uniform, homogeneous, isotropic aquifer, where flow is radial and there is no vertical flow component. Renard, Glenz and Mejias (2009) indicated that the constant derivative associated with the late-time solution Cooper –Jacob (1946) straight line solution supports the IARF assumption of the Theis model. Boulton (1954, 1963), Neuman (1975) and Streltsova (1976) developed analytical models that defined delayed yield and pseudo-equilibrium responses noted during tests. In those cases, the type curves are S-shaped, and are very similar to those derived for double porosity aquifers, such as the ones developed by Moench (1984), and Dougherty and Babu (1984). However, Odeh (1965) and Carlson (1999) have indicated that double porosity responses may be scale dependent and only reflect conditions near a well bore. Other commonly applied models are those describing linear/pseudo-radial flow for single vertical and horizontal fractures (SVF and SHF), Gringarten, Ramey and Raghavan (1975), Gringarten and Ramey (1974), and Gringarten and Witherspoon (1972); generalized radial flow in single- and double-porosity fractured aquifers, Barker (1988); and leaky aquifers, Hantush (1960), Hantush and Jacob (1955) and Moench (1985). Many of these models converge with Theis curves at late time (IARF), so that the Cooper-Jacob (1946) straight-line method can often be used on a semi-log plot to evaluate time-drawdown data. In that case, it is essential to identify what portion of a curve represents an IARF period. In those cases where an IARF segment is not present, then an appropriate single- or double-porosity model should be used to analyze drawdown data from aquifer tests.

Derivative Analysis

The primary tool for the derivative analysis method is a simultaneous plot of drawdown and the logarithmic derivative of drawdown as a function of time. This can be a useful method for identifying an appropriate conceptual model to use when analyzing aquifer test data. Bourdet, Ayoub and Pirard (1989) developed an algorithm for the petroleum industry that calculates the first derivative of the pressure change with respect to the natural logarithm of the change of time, using the following expression to calculate the derivative:

$$(dP/dX)_i = \left[(\Delta P_1/\Delta X_1)\Delta X_2 + (\Delta P_2/\Delta X_2)\Delta X_1\right]/(\Delta X_1 + \Delta X_2) \tag{1}$$

Where subscript 1 = point(s) before the point of interest *i*; subscript 2 = point(s) after the point of interest *i*; and X = natural logarithm of the time function, t^*. Drawdown during an aquifer test is equivalent to pressure buildup during a drill stem test of an oil or gas well. Because drawdown *s* during an aquifer test is related to pressure, Equation (1) may be applied to the drawdown *s* with time *t* to estimate the logarithmic derivative of the drawdown (van Tonder et al., 2001a).

Different flow mechanisms or behaviors can be combined, which requires that different phases of an aquifer test be analyzed separately (Hammond and Field, 2014). Renard, et al. (2009) provides a synthesis of typical drawdown behaviors (drawdown and log derivative plots) in response to constant pumping rates. Like the analytical models, to accurately estimate well yields using the derivative technique it is essential to properly identify an IARF period, which occurs when the derivative stabilizes at a constant level, after which the Cooper-Jacob solution can be applied. If IARF is not present, then the results of a derivative analysis can be used to determine which other conceptual model should be used to evaluate the drawdown data from a test.

Step-Drawdown Test Methods

Conventional well hydraulics theory assumes that laminar flow conditions exist in an aquifer during pumping and that drawdown is directly proportional to pumping rate. If turbulent flow occurs, a linear relationship no longer exists, and part of the drawdown is related to the pumping rate raised to some power greater than 1.

Jacob (1947) indicated that where turbulent flow exists, drawdown (*s)* equals the sum of a first order (laminar) component and a second order (turbulent) component. This was expressed by the equation: $s = BQ + CQ^n$ where n = 2, and $B = 2.30/4\pi T \log\left(2.25Tt/r^2S\right)$. For laminar flow in a perfectly efficient well, $s = BQ$, which is called aquifer loss, and CQ^2 is the well loss (head loss due to inefficiency), where aquifer losses increase with time and well losses increase with pumping rate. The additional, non-linear well losses were attributed to turbulent flow occurring just outside the well, through the well screen, and casing. This model applies to confined aquifers and unconfined aquifers where drawdown is small relative to aquifer thickness.

Bouwer (1978) indicated that well losses increase by Q to a power of 2 to 3.5 and when drawdown increases rapidly; at high pumping rates drawdown may consist mostly of well losses. Wells with sufficient screen or slot area and radius can minimize well losses. The example he gave was that doubling a well radius could reduce entry losses by 75 to 87.5 percent. Gringarten, et al. (1975) indicated that the effective well radius was equal to about ¼ of the length of a fracture. The typical fractured rock municipal/irrigation well in Maryland has a 6−8 in (0.15−0.2 m) diameter, while fractures tend to be a few feet in length to more than several hundred feet in length, which could produce effective well radii one or more orders of magnitude greater than actual radii. Bierschenk (1963) noted that formation loss BQ varies inversely with transmissivity T; if T is low, as is common in fractured rock aquifers, BQ theoretically is great and well loss is small relative to the total drawdown in a well, and well efficiency is less affected than in an aquifer with high transmissivity.

Mogg (1969) indicated that reductions in specific capacity as discharge rate increases (well loss) may be due to some factor other than turbulent flow. Pumping fractured rock wells in unconfined aquifers where there is a major reduction in saturated thickness can introduce additional head losses. Mogg suggested that, if the specific capacity of the last step of a test is within 10 percent of the first step, then significant turbulent losses were not present. The Theis method can be applied to these types of aquifers, when the drawdown is small relative to saturated thickness b. If drawdown is not negligible (less than 20 percent), however, the observed drawdown s_{obs} can be corrected using the Jacob (1944) formula:

$$s_{corr} = s_{obs} - (s_{obs} \times s_{obs})/2b \qquad (2)$$

Automatic Curve Fitting Techniques

The common graphical method of fitting type curves to drawdown data from aquifer tests on a log-log plot is prone to errors in individual judgment. The primary reason is that type curves representing different flow mechanisms often have such similar shapes that each can provide relatively good visual fits to the same set of data.

Estimating aquifer parameters and extrapolating drawdown data by automatic curve matching techniques can remove much of the subjective nature involved with graphical methods. Automatic curve matching uses a nonlinear least squares procedure that seeks to minimize the sum of the squared residuals, RSS, between the measured data and the theoretical (type curve) drawdowns for the analytical model under consideration.

The Saleem (1970) study included the first known use of automatic curve matching techniques to actual pumping tests. A significant number of other studies on the subject were published over the next 20 years, and many were included as references in the Sayed (1990) and Johns, Semprini and Roberts (1992) investigations. Most consisted of programs designed to match test data to one or a few analytical models. The commercially available AQTESOLV®[1] program (Aquifer Test Solver) (Hydrosolve, Inc., 2007), includes 35 different analytical models, most of which can be applied to fractured rock aquifers. The AQTESOLV® program uses four diagnostic or specialized flow plots to aid in the identification of different flow regimes. These are for radial flow (s vs t), linear flow (s vs $t^{1/2}$), bilinear flow (s vs $t^{1/4}$), and spherical flow (s vs $t^{-1/2}$). Renard et al. (2009) also provides a synthesis of the behaviors on typical drawdown and log derivative plots in response to constant pumping rates. Some of the flow mechanisms potentially detected in this study on these plots are:

1. Log-log radial flow plot, wellbore storage effects (unit slope-early time) or a closed aquifer (unit slope-late time);
2. Semi-log radial flow plot, infinite acting radial flow or IARF (constant slope at late time); or constant head boundary (zero slope at late time);
3. Log-log linear flow plot, an infinite or high conductivity fracture (unit slope at early time) and;
4. Log-log bilinear flow plot, a finite or low conductivity fracture (unit slope at early time).

While the derivative analysis method is now considered the best method for identifying an appropriate conceptual model to use when analyzing aquifer test data, it requires many calculations that are best handled by computer-generated algorithms. It is a common practice to plot both drawdown and derivative data on the same plot. Some of the flow mechanisms that can be detected using derivative analysis are:

1. IARF (constant positive drawdown);
2. Vertical fractures (one-half unit slope followed by IARF);
3. Well-bore storage effects (initial unit slope followed by a drop-off to form a peak on the plot);
4. Water table and dual porosity aquifers (dip in the drawdown at mid-time);
5. Leaky aquifers (initial positive drawdown, followed by an approach to zero drawdown at late time);
6. Recharge boundaries (constant zero drawdown), and;
7. Impermeable barriers (two levels with constant positive drawdowns).

Duffield (2007) discusses in detail methods for the performance and interpretation of derivative analyses when using the AQTESOLV® program.

The Cooper-Jacob (1946) method can be used to simulate variable-rate tests by using the deconvolution algorithm developed by Birsoy and Summers (1980). The technique applies the superposition principle, which treats the variable rate as a sequence of steps in which the discharge is held constant in each step. For the typical step-test, this can lead to many mathematical calculations

[1] The use of brand names in this report is for identification purposes only and does not constitute endorsement by the Maryland Department of the Environment

that can be programmed on a spreadsheet but are more easily performed using an automatic curve fitting program. The AQTESOLV® program modifies the Dougherty-Babu (1984) solution by a term for nonlinear well loss in a confined aquifer and adds terms for linear and nonlinear well losses to the Theis (1935) and Hantush-Jacob (1955) solutions for pumping in a confined aquifer and a leaky confined aquifer, respectively.

After application of inverse analysis techniques, various models could still provide similar fits to sets of drawdown data. A correct conceptual model may be found using knowledge of the hydrogeology of the site, careful interpretation of the diagnostic plots and derivatives, and review of the calculated residuals from the curve fitting process.

Extrapolation Method

Over the past approximately 30 years, reliable yield of a fractured rock well in Maryland has been defined, through practice by the Maryland Water Supply Program, as that yield which can be sustained over approximately 90 days ("extrapolation period") of severe drought when demand is at its highest, recharge is negligible, and groundwater availability is at its lowest.

Support for the extrapolation period is provided by Schultz, Tipton, and Palmer (2005), who defined summertime (or third quarter, 92 days) water availability as the sum of beginning-of-summer groundwater storage and summer recharge. Within the area of the present central Maryland study, Schultz et al. (2005) indicated that summertime groundwater availability is lowest in the upper Monocacy sub-basin (Mesozoic lowland- primarily consolidated sedimentary rocks), dropping to below 0.10 in. (2.6 mm) during the drought period of the mid-1960s and in 2001. It is also low in the Catoctin sub-basin (Middletown Valley - mostly crystalline rocks) in the Blue Ridge Province, dropping to below 0.15 in. (3.8 mm) in both 1965 and 2002.

Fractured rock aquifer storage capacity may be considered approximately equivalent to that of a surface water reservoir. Since reservoir geometries are easy to define and there are abundant stream flow data available, determinations of reliable yields of surface water supplies, using flow mass or reservoir analysis techniques, are relatively accurate. Such reservoir simulations are analogous to numerical groundwater models. Malcolm Pirnie (2004) developed a STELLA© (flow mass) model for the City of Frederick which indicated that the reliable yield of its surface water supply was 19 percent higher with water restrictions in place than when they were not in force. The difference is that with restrictions in place only average demand must be met, while without restrictions maximum seasonal demand must be supplied. If these results were applied to a groundwater supply system, this would indicate that a well may be able to supply the average use during a drought but could still go dry due to the substantial additional drawdown that would occur while having to meet peak summertime demand.

To estimate reliable yields in this study, drawdowns s_t are first extrapolated to a prescribed time, *t*, on a semi-log graph, using the Cooper-Jacob straight-line method or an appropriate type curve, if an IARF period cannot be identified. A *t* of 129,600 minutes (90 days) was chosen for this study. The specific capacity, Q_{obs}/s_t at that point is calculated, using the continuous pumping test rate, Q_{obs} . The result is then applied to an available drawdown s_A to a permeable (but not necessarily water-bearing) zone or during the 2004 preliminary study, a discrete, water-bearing fracture. The calculated reliable yield Q_R is then,

$$Q_R = (Q_{obs}/s_t) \times s_A \tag{3}$$

If well losses occur, the yield can be adjusted by subtracting the losses from s_A.

Drought yields were derived by comparing the static water level in the pumping well to seasonal water levels in nearby U.S. Geological Survey monitoring wells and adjusting the well yield proportionally. Details concerning that method are provided below in the section on seasonal variations in well yields.

APPLICATION OF TECHNIQUES

When analyzing the results of an aquifer test, the time-drawdown data is imported into the AQTESOLV program. Diagnostic flow plots are then used to find evidence of, generally in fractured rock aquifers, radial or linear flow. A derivative plot is added and inspected for a characteristic flow regime (e.g. IARF, SVF, dual porosity, leaky aquifer). Visual curve matching is then used to interactively match test solutions and provide a preliminary estimate of aquifer properties. After an appropriate active type curve is selected, the automatic curve matching procedure is initiated. By an iterative method, the program adjusts hydraulic property values to produce the best statistical match between the solution and the test data. The result can then be extrapolated to the end of any desired time period, at which point the specific capacity can be calculated and applied to the drawdown at a target operating level in the well to produce an estimated well yield.

The AQTESOLV program contains many case studies that use examples of how the techniques are applied. Three of those cases could apply to fractured rock aquifers and, with some variations, most of the sites in the present study: The Moench (1984) double porosity/IARF model at the Nevada Test Site; a leaky aquifer model at the Grand Junction, Colorado, site, Lohman (1972); a forward solution applied to an aquifer with a single vertical fracture and late-time IARF response, and the step-drawdown test in an unconsolidated sandstone aquifer in Saudi Arabia using the Theis (1985) solution. The application of other models will be demonstrated in the following results section. In addition, methods will be demonstrated on how to use step-test data to determine available drawdown and the optimum pumping rate for an aquifer test.

Once a well is placed in service, the estimated yield should be best verified by the collection and analysis of the following daily operational well data: Pumpage (water use), hours pumped and, as a minimum, the water level at the end of the pumping drawdown cycle.

SUMMARY OF SYNTHETIC ANALYSIS EXAMPLES

To verify the accuracy of the methods used in the present study, the results were compared to those achieved by the best available numerical models used to estimate well yields in fractured rock aquifers.

Van Tonder, et al. (2001a) used derivative analysis and semi-analytical techniques to develop semi-analytical and numerical methods for estimating yields of wells in the layered, sedimentary rock (multi-porous) Karoo aquifer, South Africa. To prove their methods, they constructed a calibrated 3D numerical model to verify the estimated sustainable yield of a fractured rock well (UO5). The geometry of the model was based on the hydraulic characteristics of the Karoo aquifer at the Campus Test Site, from data collected during an extensive field investigation.

Using the techniques developed in the present study, drawdowns from the Karoo pumping test were extrapolated to various peak, seasonal and annual pumping periods. The calculated specific capacity at the end of each pumping period was then applied to the available drawdown to the single, discrete horizontal fracture in UO5 (19.7 ft or 6 m), producing yields like those of van Tonder et al (2001a), when seasonal variations in water demand are considered.

Piscopo and Summa (2007) analyzed the drawdown data from a 120 gpm (7.6×10^{-3} m^3/s) aquifer pumping test by applying the straight-line Cooper-Jacob method to both early-time and late-time segments. They attributed the difference between the two transmissivities that were derived to aquifer heterogeneity. Based on the results of a step-test, they indicated the drawdown should be limited to 39.4 ft (12 m). Their data were re-analyzed in the present study using derivative and type curve methods, indicating that the observed drawdown was caused by a leaky aquifer response. Further, analysis of the step-test data indicated that the available drawdown was at least 140 ft (43 m), and the estimated yield was a peak of 83 gpm (5.3×10^{-3} m^3/s) and an average of 59 gpm (3.7×10^{-3} m^3/s), or about four times higher than Piscopo and Summa (2007) estimate. The well discharge varied from a maximum of 79 gpm (5.0×10^{-3} m^3/s) in April−May to a minimum of 32 gpm (2.0×10^{-3} m^3/s) in December during the period 2004 to 2006, while maintaining a constant drawdown of 39−49 ft (12-15 m). The higher than predicted yields were subsequently verified by Baiocchi et al. (2014), who constructed a 3D model consisting of a confining unit overlying a carbonate aquifer. During one simulation, a pumping rate of 87 gpm (5.5×10^{-3} m^3/s) captured the entire residual outflow during the 2005 depletion period after 214 days, with a drawdown of 154 ft (47 m), providing evidence agreeing with the results from the present study.

The results of the Van Tonder et al. (2001a, b), Piscopo and Summa (2007) and Baiocchi et al. (2014) investigations are described in greater detail in the section (Appendix A8) on examples of synthetic models.

EXAMPLES OF ESTIMATING RELIABLE DROUGHT YIELDS IN CENTRAL MARYLAND

Three examples of step-drawdown and aquifer-test analysis to estimate reliable drought yields are provided to illustrate the types of analyses preformed in this study. Hammond (2018) analyzed the step-drawdown and aquifer test data to predict the reliable yields of the following fractured rock wells: Town of Emmitsburg well 3, Poolesville well 7 and Taneytown well 13. Follow-on monitoring data were collected by water systems personnel that were used to verify those estimates. These tests were the prototype examples used to develop MDE's current test and evaluation program for groundwater appropriation or use permits.

Town of Emmitsburg Well 3

The Town of Emmitsburg's well 3 (well permit FR-65-0432) was completed in the Catoctin Metabasalt, a crystalline rock. A 9−d (November 13−22, 1996) pumping test of that well was conducted, with follow-up monitoring of water levels and daily production. Figure 4 is a semi-log plot of drawdown vs time for that test, which was started at 143 gpm (541 L/min), at an initial or static water level (SWL) of 8.5 ft (2.6 m) below top of casing (BTOC) or the measuring point. At first glance the drawdown observed during the first eight hours of the test appears to be due to a delayed yield response in an unconfined or water table aquifer.

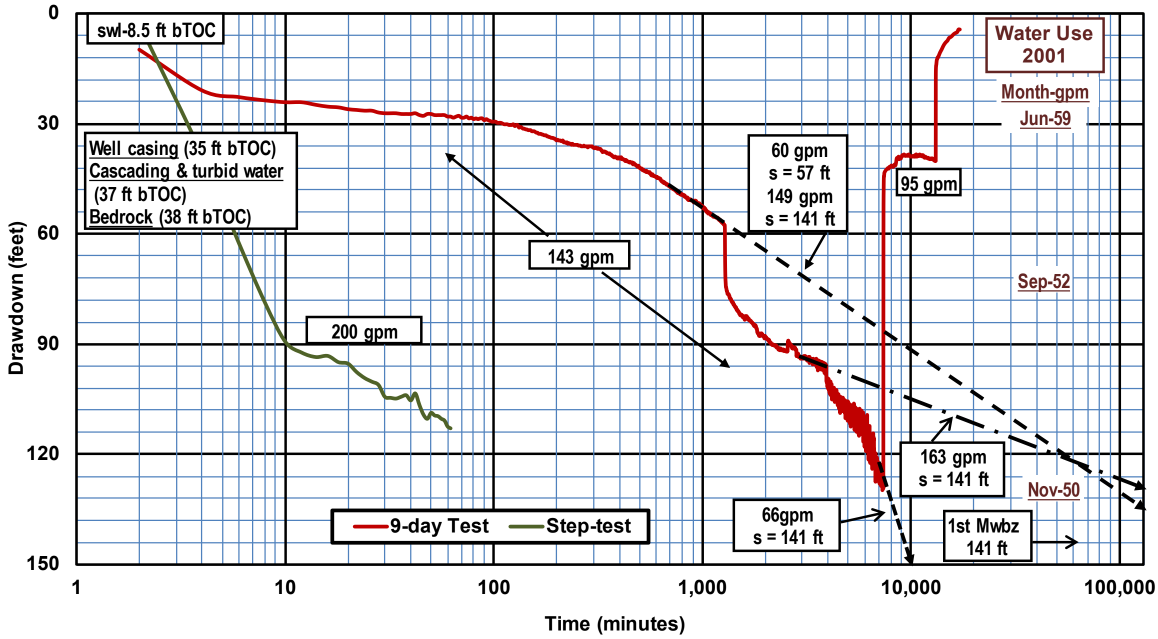

Figure 4. Semi-log plot of drawdown data from a step-drawdown test and 9-d pumping test of Emmitsburg well 3, with estimated yields from drawdowns extrapolated to 90 days and drought production.

A derivative analysis (fig. 5) indicated that the early drawdown was affected by a constant head or intermittent recharge boundary. This was supported by reports of high turbidity and cascading water at the level where the transition zone could occur. The source of the turbid water was probably from an unlined, raw-water reservoir, with a limited storage capacity, located 66 ft (20 m) from the well. Operators had reported that the pond level dropped as water was pumped from well 3. Most likely the transition zone acts as a pathway between the pond and the wellbore. This is supported by operator reports that the well clears up a few hours after pumping is started, which could be related to dewatering of the weathered zone and breaking of the hydraulic connection between the pond and the well.

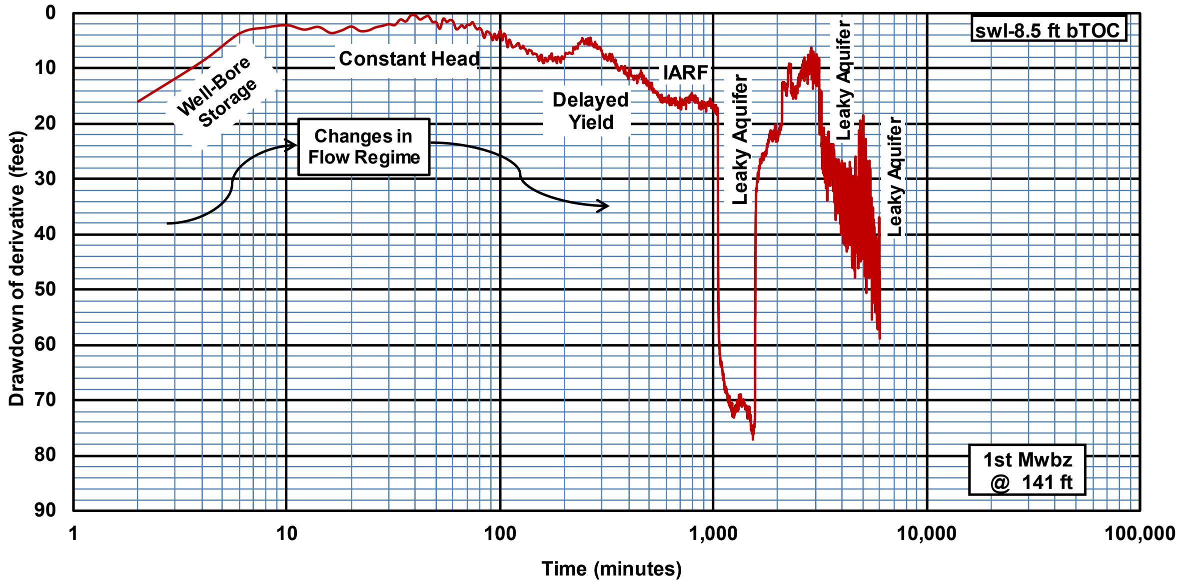

Figure 5. Semi-log plot of logarithmic derivative of drawdown from first five days of the 9-d aquifer test of Emmitsburg well 3.

The next phase of the test was a water table response, which was then followed by an IARF segment which lasted for the next 13 hours. At the end of the IARF period and at a depth of about 15 ft (4.6 m) below the casing there was a sharp break in the drawdown data. The change in slope appears to have occurred at or near the weathered zone/bedrock interface and was probably due to a sharp reduction in permeability. (Note: This break was not observed during the 200-gpm (757 L/min) step-drawdown test, due to rapid dewatering of the aquifer.) It was followed by a pseudo-equilibrium phase, which is attributed to leakage and lasted for about two days. Had the test ended at this point, the conventional wisdom would have been that it was a high yielding well. The test, however, was continued for several more days, during which the drawdown increased rapidly through two other leaky aquifer phases, until the water level approached the main water-bearing zone, at which time the test was secured. After 25 hours of recovery, the test was re-started at 95 gpm (360 L/min), the water level stabilized, and the test was secured after nine days of pumping.

The available geologic and pumping test data indicate that the casing for well 3 was set above the transition zone, a common method used by drillers to potentially increase well yields when the well was completed in 1965 (but is not now allowed, to prevent surface water contamination). This provided a short-circuit pathway between the weathered zone and bedrock portions of the aquifer. Upon dewatering of the weathered zone, the short-circuit pathway was bypassed, groundwater flow was then by leakage from the weathered zone to the discrete, main, water-bearing fracture and from there to the wellbore.

This test was conducted under very wet conditions. Based on preliminary studies, it was estimated that the well could produce about one-half of the final test yield during a severe drought, or 50 gpm (189 L/min). On the right side of figure 4 are the production rates and water levels measured in the well during the drought of 2001−2002, which confirmed that the well could only produce 50 gpm (189 L/min) during the worst part of the drought (November 2001), with the water level near the main, water-bearing fracture.

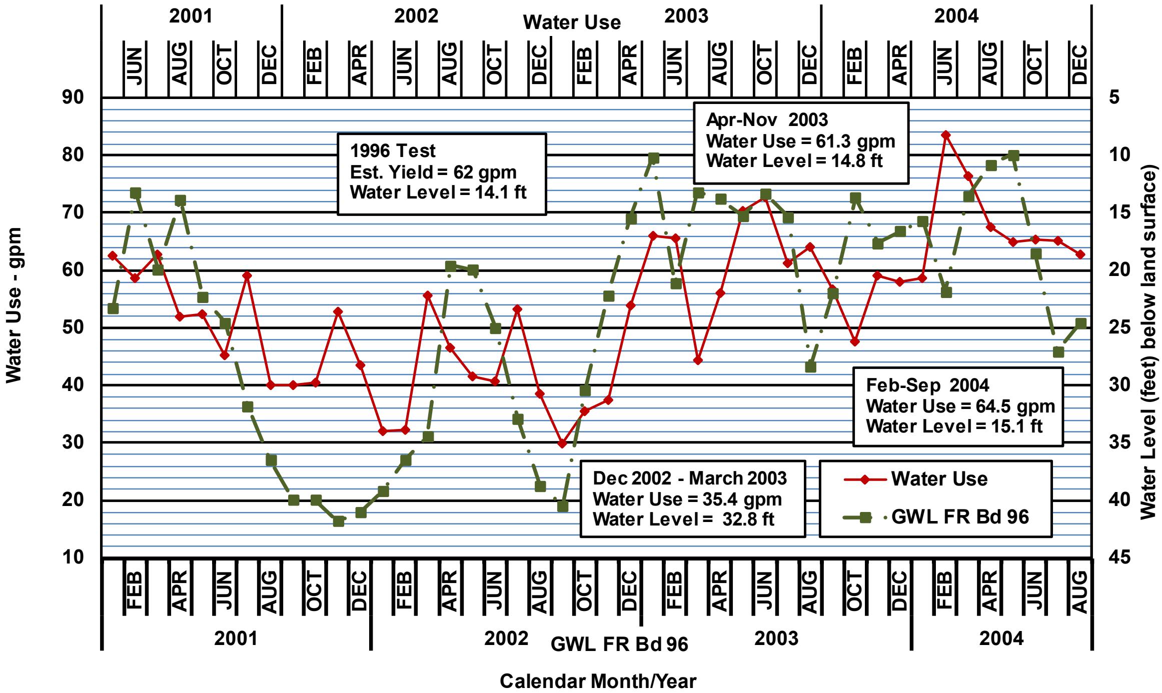

Figure 6. Emmitsburg wells 3 and 5 water-use data and USGS FR Bd 96 water levels (offset by ~4 months).

Figure 6 is a graph of monthly production from well 3 and regional water levels plotted against time, for the drought of 2001−2002 and the following, very wet period of 2003−2004. The regional water levels were measured in a nearby monitoring well (FR Bd 96) of the U.S. Geological Survey. That well has the largest range of water levels of any USGS monitoring well in the fractured rock areas of Maryland, which is believed to be due to the lack of aquifer storage in the vicinity of that well and a strong hydraulic connection to the shallow aquifer. Town well 5 was placed in service during early 2003. Due to the relatively low estimated yield (<20 gpm or 75 L/min) and proximity (165 ft or 50 m) to well 3, it was expected that well 5 would have added little to the reliable yield of the system. Both wells were run continuously (24 h/d), at the request of the author, to eliminate the need to estimate the effects of a variable pumping period. These data indicated that the maximum monthly amount of water produced was about three times the minimum monthly amount. In addition, a good correlation existed between the regional water levels and well production, where the regional water levels were offset by four months. This indicates that there is a delayed effect of drought or high recharge events on the sustained yield of the well, probably due to changes in aquifer storage and leakage effects.

The test data in figure 4 were used to make estimates of the well's yield by extending a straight-line from various parts of the drawdown curve to 129,600 minutes (90 days). The specific capacity, Q/s, at that point (1.06 gpm/ft or 13.2 Lpm/m) was determined and then applied to values of available drawdown. From the IARF period, the estimated yield was 60 gpm (229 L/min), when the extrapolated Q/s is multiplied by the available drawdown (57 ft) to the breakpoint (weathered zone/bedrock interface) in the data, or 149 gpm (566 L/min) using the available drawdown (141 ft) to the first, major, water-bearing zone. Extrapolations from the later parts of the curve produced estimates of 66 to163 gpm (250 to 617 L/min). The production data indicated that the estimated value of 60 gpm (229 L/min) was closest to the final reliable yield, under the very wet conditions that existed during the test.

Towards the end of the aquifer test, rapid fluctuations in the water level began to occur. Kawecki (1995) suggested that small, practically immeasurable changes in discharge can cause significant changes in drawdowns, if well losses are high. Another possible explanation is based on similar responses noted during a recent test in Poolesville. In that case, manual measurements with an electric probe indicated that the water levels were stable. It is possible that pump vibrations on the PVC measuring tube may have affected the pressure recorded by the transducer of the automatic data logger in use.

The step-drawdown test data collected from Emmitsburg well 3 were generally unsuitable for analysis, since there were three separate tests, consisting of only two steps each, which were conducted over a period of several weeks. Half of the steps were influenced by complete dewatering of the transition zone, with water levels ending in bedrock, such as the 200 gpm (757 L/min) step shown on the semi-log plot (Figure 4). The other three steps, at 55 gpm and 101 gpm (208 L/min and 382 L/min) (test 1), and 150 gpm (568 L/min) (test 2), if corrected for dewatering of the water table, produced a difference in specific capacities of 12 percent, with the last rate being higher than the well's reliable yield.

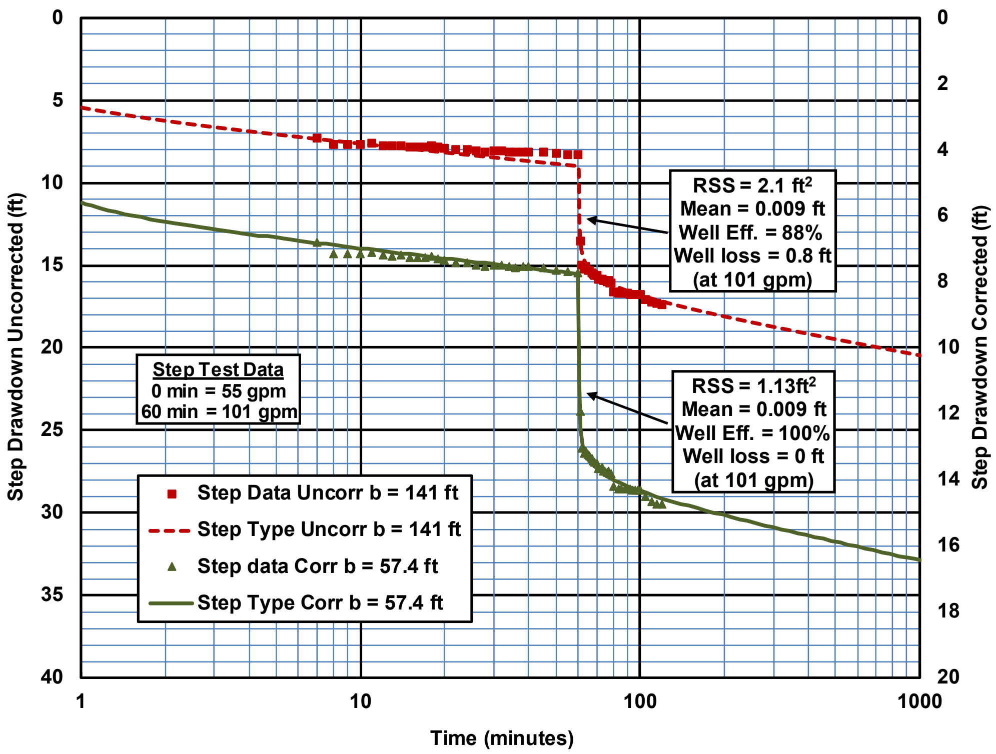

Figure 7. Semi-log plot of drawdown from the first step-drawdown test of Emmitsburg well 3. Data is both uncorrected and corrected for aquifer thickness *b* of 57 feet.

Figure 7 is a semi-log plot of the first two steps from test 1, with drawdowns both uncorrected and corrected for an aquifer thickness of 57 ft (17.5 m). The AQTESOLV® software for analysis of aquifer tests uses a modified version of the deconvolution methods described in Birsoy and Summers (1980) that allows automated fitting of type curves to step-test data. The automated curve matching technique seeks to minimize the residual sum of squares (RSS) and obtain a residual mean close to zero. Application of the modified Dougherty-Babu (1984) solution to the data from the uncorrected step-test data produced fairly good results, with a RSS of 2.1 ft^2 and a mean of 0.0009 ft; and, at the last rate of 101 gpm (382 L/min), a well efficiency of 88 percent and well loss of 0.8 ft. When S.I. units were used the results were: RSS, 0.437 m^2; mean, 0.016 m; well loss of 0.2 m; and a well efficiency of 95 per cent. These results indicate that there was not a direct proportional relationship between the U.S. English and S.I. units. The best fit to the data, however, was achieved by correcting the data for the effects of dewatering, using a saturated thickness of 57.4 ft (17.5 m). This produced a RSS of 1.13 ft^2 (0.106 m^2) and a mean of 0.009 ft (0.002 m); and, at 101 gpm (382 L/min), a well efficiency of 100 percent and well loss of 0.0 ft (0.0 m). In this case, the U.S. English and S.I. units are directly proportional, within the limits of rounding errors. These data indicate that the differences between the results using U.S. English and S.I. units in the uncorrected solutions were because of aquifer dewatering.

Town of Poolesville Well 7

In 1999, the operational pumping water level in Poolesville's well 7 (well permit MO-88-2384) was very shallow and its production was less than the best estimated yield of about 50 gpm (189 L/min). That prediction was based on extrapolating drawdown data from an apparent early-time IARF segment to the only water-bearing zone in the well (431 ft or 131 m BTOC) (fig. 8). A higher yield (67.4 gpm or 255 L/min) was possible, based on extrapolation from late-time data, but it was unclear if that was an IARF response. This last estimate was essentially the same interpretation that Piscopo and Summa made for the pumping test included in their 2007 study (Piscopo and Summa, 2007). It was suggested that the town pump well 7 continuously and see where the water level stabilized. If the water level stabilized much below 100 ft (30.5 m), previous crystalline rock tests suggested that little increase in yield should be expected by installing a higher-capacity pump. After the change to continuous pumping, the water level stabilized near 141 ft (43 m) at 41.2 gpm (156 L/min). A pump with a capacity of 85−90 gpm (322−341 L/min) was then installed, probably because there was still nearly 300 feet (91 m) of remaining available drawdown to the first major water-bearing zone. Changing the pump, however, only increased the operating water level in the well; but, most importantly, led to a net decrease in yield to 28 gpm (106 L/min).

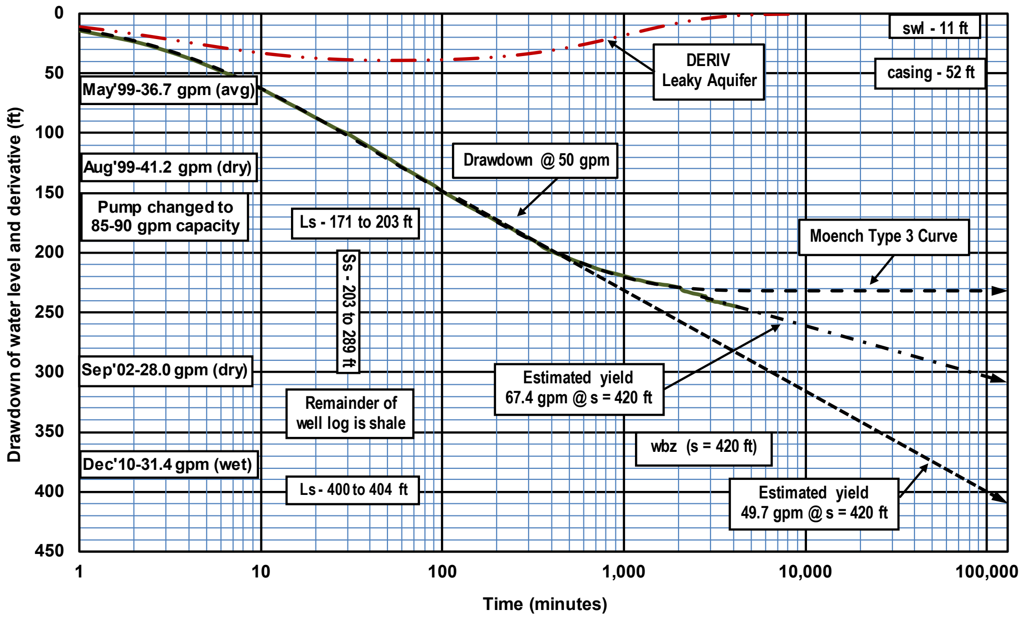

Figure 8. Semi-log plot of drawdown and its logarithmic derivative from the 2011 72-h, 50 gpm test of Poolesville well 7 showing the best fit of the Moench leaky aquifer solution to the data (with lithologic description).

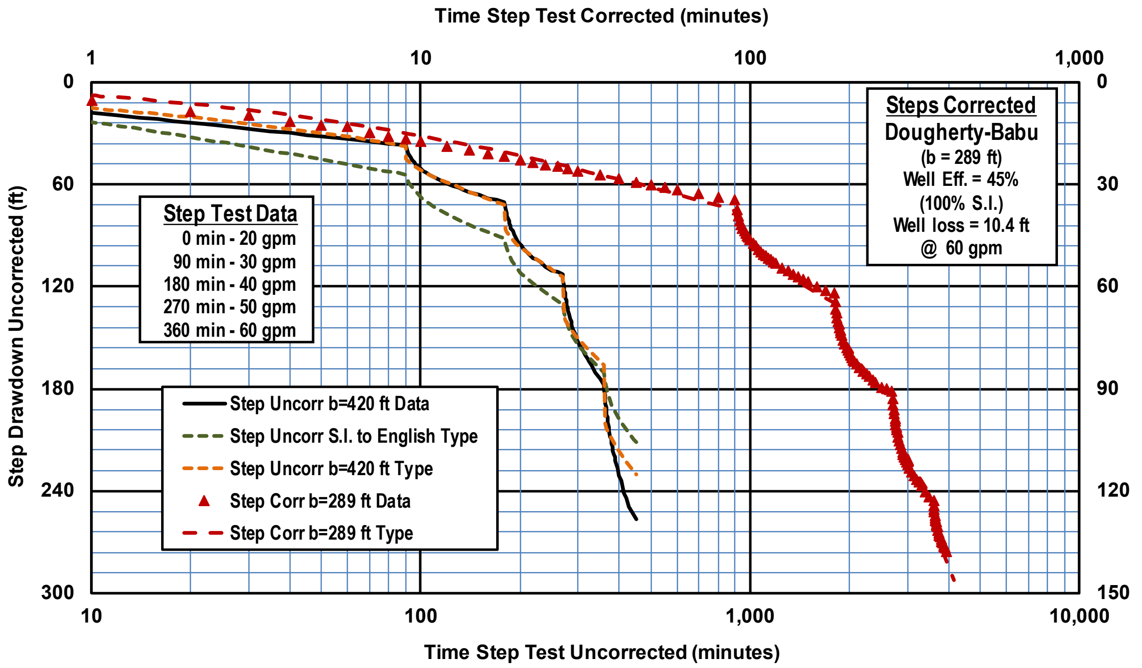

Figure 9. Semi-log plot of drawdown from step-drawdown test of Poolesville well 7. Data is both uncorrected and corrected for aquifer thickness *b* of 289. Includes S.I. type curve converted to English units for uncorrected data.

The results of a step-test performed on well 7 are shown in Figure 9. Application of the AQTESOLV® modified Dougherty-Babu double porosity solution for step-drawdown tests initially produced a good visual match, but the calculated statistical results were poor; RSS = 8,871 ft^2 (821 m^2) and mean = −0.55 ft (−0.17 m), with a calculated well efficiency of 11 percent at the final rate of 60 gpm (227 L/min). In the Hammond (2018) study, S.I. units were used in the AQTESOLV® simulation, which also produced both poor visual and statistical results, but had a calculated well efficiency of 99 percent at the final rate. By iterative methods, the drawdown data were corrected for dewatering effects, with a good match achieved by using a saturated thickness of 289 ft (88 m); RSS = 554 ft^2 and mean = 0.20 ft^2, with a well efficiency of 45 percent at 60 gpm. Upon careful review of the geologic log for the well, virtually all the rock penetrated by the well was shale, except for a limestone/sandstone sequence between 182 ft and 300 ft (55 m and 91.4 m) and the corrected aquifer thickness was near the base of that limestone/sandstone unit. There was also a thin bedded limestone near the single water-bearing zone at 431 ft (131 m). After correcting for the effects of aquifer dewatering, the calculated well efficiency was 45 percent (100 percent S.I.), with a well loss was 10.4 ft (0.0 m), and 56 percent at the estimated reliable yield of 45 gpm (170 L/min). By comparison, an arithmetic plot of s_w/Q versus Q (fig. 10) produced a good fit to the Jacob (1947) model according to the following expression:

$$s_w = 0.6673\,Q + 0.0524\,Q^2 \tag{4}$$

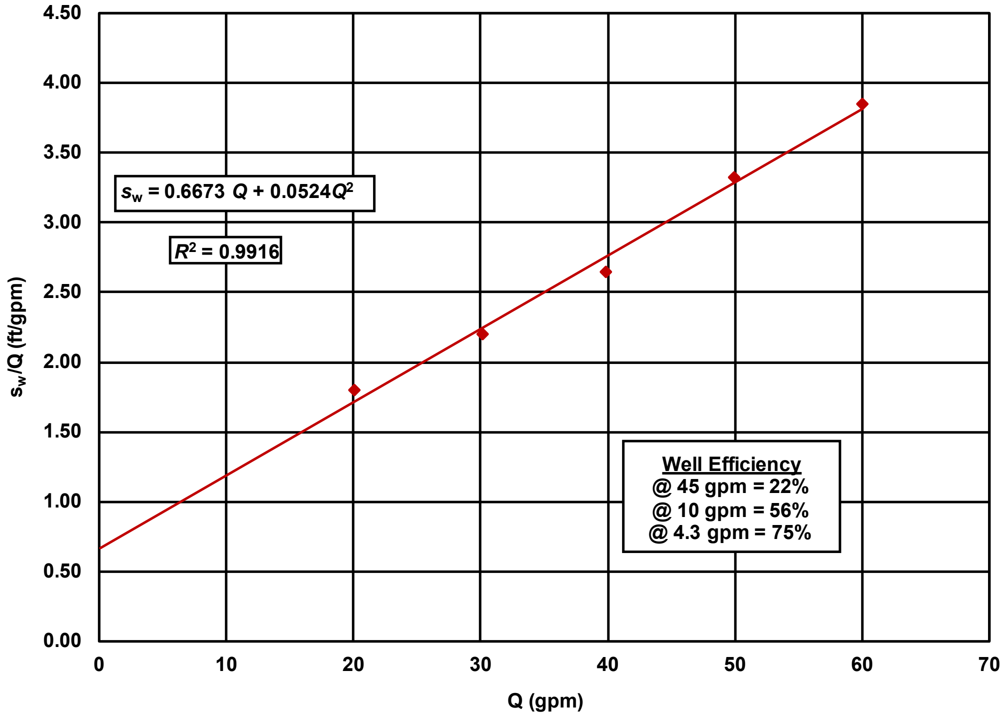

Figure 10. Arithmetic plot (Jacob Method) of specific drawdown (s_w/Q) and discharge Q for the step-drawdown test of Poolesville well 7. Aquifer-test data shown in Figure 9.

At the estimated reliable yield of 45 gpm (170 L/min) for well 7, the calculated well efficiency using the Jacob equation is 22 percent. To obtain the relatively higher efficiency of 56 percent calculated from the AQTESOLV® solution for the reliable yield of 45 gpm (170 L/min) would require a pumping rate of 10 gpm (38 L/min). To achieve the 75 percent efficiency recommended by Piscopo and Summa (2007) the well would have to be pumped at a rate 78 percent less than the demonstrated reliable yield. These results indicate that an apparently inefficient well using the Jacob equation was a more efficient well that was affected by aquifer dewatering during the step-test.

Derivative and type curve analyses of the pumping test data for well 7 were performed. The results are shown in Figure 8 and indicate that there was no clear IARF period present during the 72-hour test and that the Moench leaky aquifer conceptual model best fit the data until the drawdown reached 230 ft (70 m) at 2,040 minutes. At that point there was a break in the slope, possibly due to dewatering of a permeable unit. These observations, the production data, and the results from the step-test indicate that a permeable unit within the 182–300 ft (55–91 m) limestone/sandstone interval controls the available drawdown, not the water-bearing fracture at 431 ft (131 m). These results further demonstrate that the reliable yield of the well was less than the test rate of 50 gpm (189 L/min), but more than the maximum production rate of 42 gpm (159 L/min), suggesting a reliable yield of about 45 gpm.

A flow control valve was installed in well 7 to help maintain relatively constant operating water levels in the well. During a follow-on test, the well was pumped continuously for 60 days ending in December 2009, producing a sustained yield of 31.4 gpm (119 L/min) at a water level of 377 ft (115 m) under wet climatic conditions. This was substantially less than the 40 gpm (151 L/min) that was produced during the 1999 drought with a water level of 141 ft (43 m). These data indicate well 7 may have been damaged due to dewatering of the intermediate level limestone/sandstone unit. Two possible explanations for the reduced yield, fracture compression and mineralization, are described in detail in the section “Discussion of Results”.

City of Taneytown Well 13

The City of Taneytown's well 13 (well permit CL 81-1687) was also completed in the Triassic New Oxford Formation. That well contains five water-bearing zones between 325 ft and 580 ft (99 m and 177 m). The observed static water level of 84 ft (25.6 m) was about 50 ft (15 m) below regional water levels, which was probably due to interference with the City's nearby well 10.

A 49-h, variable-rate, average of 412 gpm (1,559 L/min), pumping test of well 13 was conducted in September 1985 and was effectively a long-term step-test. Figure 11 is a semi-log plot of the drawdown data from that test. When the Dougherty-Babu model is applied to all seven steps in the Hammond (2018) study using S.I. units, there is a relatively poor fit to the data (RSS = 601 m^2; mean = 0.23 m). The data appeared to diverge after the first three steps, upon dewatering of a 34 ft (10 m) thick, gray, arkosic sandstone unit, the base, 164 ft (50 m), of which is located near the level where rapid changes in operating water levels had been noted. Similar gray sandstone and white sandstone units are penetrated by the City's higher capacity wells but are absent in its lower capacity wells. The best fit (RSS = 9.3 m^2 and mean = −0.035 m) to the data from the three, early steps were obtained using a saturated thickness of 152 ft (50 m) produced a well efficiency of 100 percent and a well loss = 0.36 m. When English units are used, the application of the Dougherty-Babu solution to all seven steps produced a good visual fit, but a poor statistical fit to the drawdown data, (RSS = 5984 ft^2 and mean = 0.14 m) until the data diverged from the type curve after the fourth step. The best fit (RSS = 103 ft^2 and mean = −0.005 ft) occurred when the model was applied to the first four steps using a saturated aquifer thickness of 152 ft (46.3 m). This produced an efficiency of 79 percent and a well loss of 2.1 ft. The saturated aquifer thickness is about 100 ft (31 m) deeper than the observed break in drawdown data and between the depths of the gray and the white sandstone layers in the well. The estimated saturated thickness may represent a composite effect caused by groundwater flow from those two units. Another explanation is that the aquifer is confined above the first, gray sandstone, but has some limited leakage, so that dewatering of that layer caused a change in the flow mechanism and the sharp decline in the water level. These results suggest that the drawdown to the first, gray sandstone unit primarily controls the well's yield, even though it contains no discrete, water-bearing fractures.

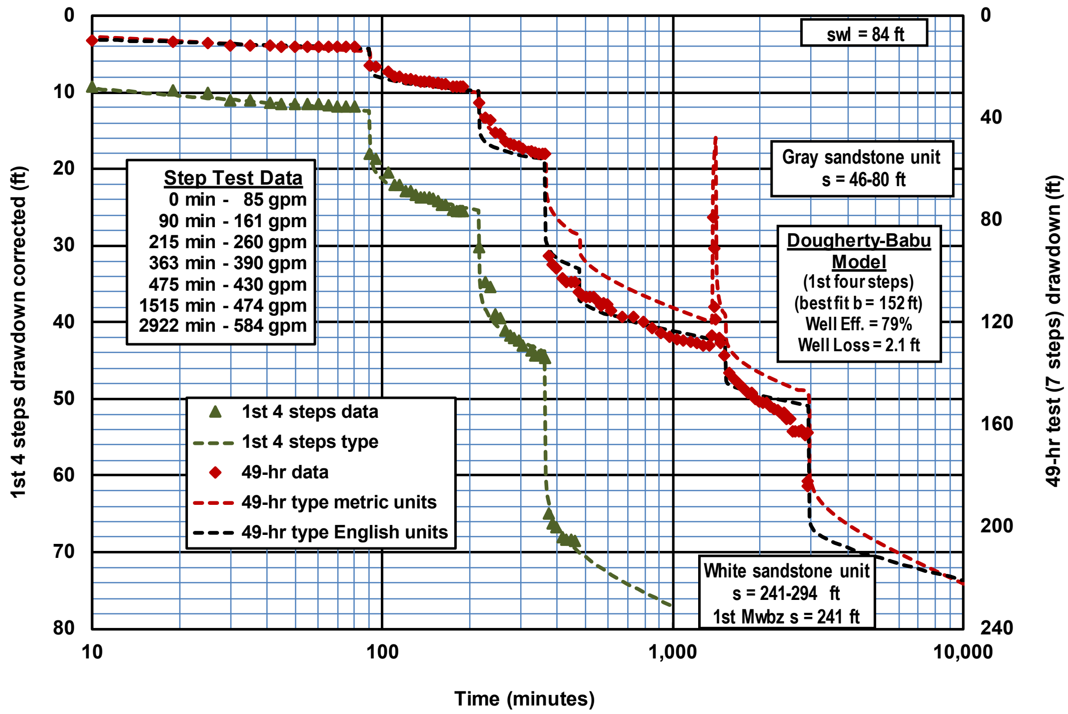

Figure 11. Semi-log plot of drawdown from 49-h, variable-rate test of Taneytown well 13. Data is both uncorrected and corrected for aquifer thickness (*b)* of 152 feet, positions of reservoir rock units, and first major water-bearing zone.

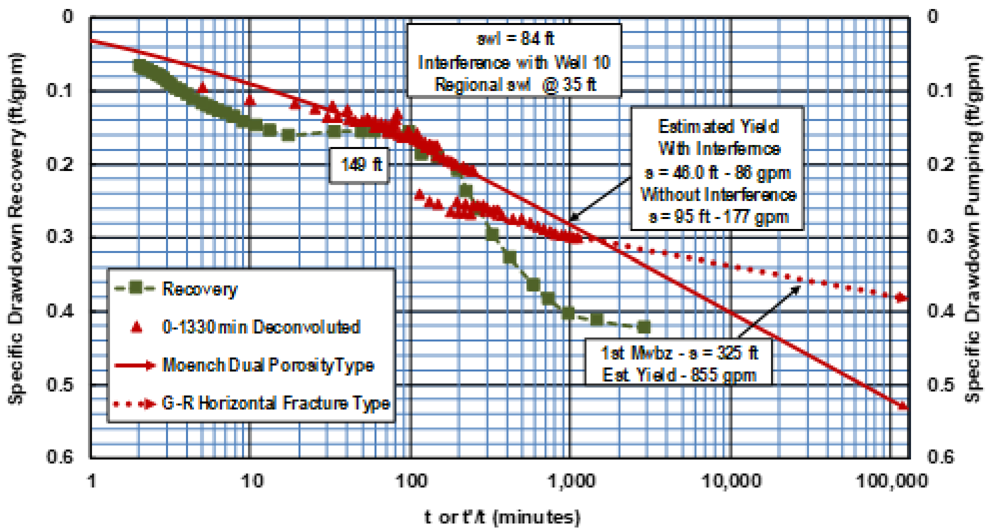

Figure 12. Semi-log plot of specific drawdown and recovery data from a 49-h, variable-rate test of Taneytown well 13. Yield derived by extrapolating deconvoluted data using the Moench double porosity model.

The Cooper and Jacob (1946) method for variable-rate tests was initially used to evaluate the data, by using the specific drawdown, *s/Q*, to produce a curve from which water levels could be extrapolated. Figure 16 is a semi-log graph of *s/Q* versus time of the data collected during the test of well 13. There was evidence of a breakpoint in the drawdown and recovery data at 149 ft (45 m) or a drawdown of 65 ft (20 m) and in the middle of the upper gray sandstone unit. Due to the variable pumping rate, the early-time specific drawdown data were erratic. Those data were adjusted by using the deconvolution techniques published by Birsoy and Summers (1980). The best fit to the resulting curve was the Moench dual porosity model. When the extrapolated *s/Q* at 129,600 minutes is applied to the drawdown to the break point in the drawdown and recovery data (65 ft or 20 m), the estimated yield is 121 gpm (458 L/min), but this included interference with well 10. To eliminate the effects of interference, regional water levels were used to estimate an available drawdown of 114 ft (35 m). This produced an estimated individual yield for well 13 of 213 gpm (806 L/min). When the data collected after the breakpoint were extrapolated to the first, water-bearing zone at 325 ft (99 m), by fitting a power curve to the data, the estimated yield is 855 gpm (3,236 L/min). Based on the last full step, the consultant's estimated yield was 474 gpm (1,794 L/min).

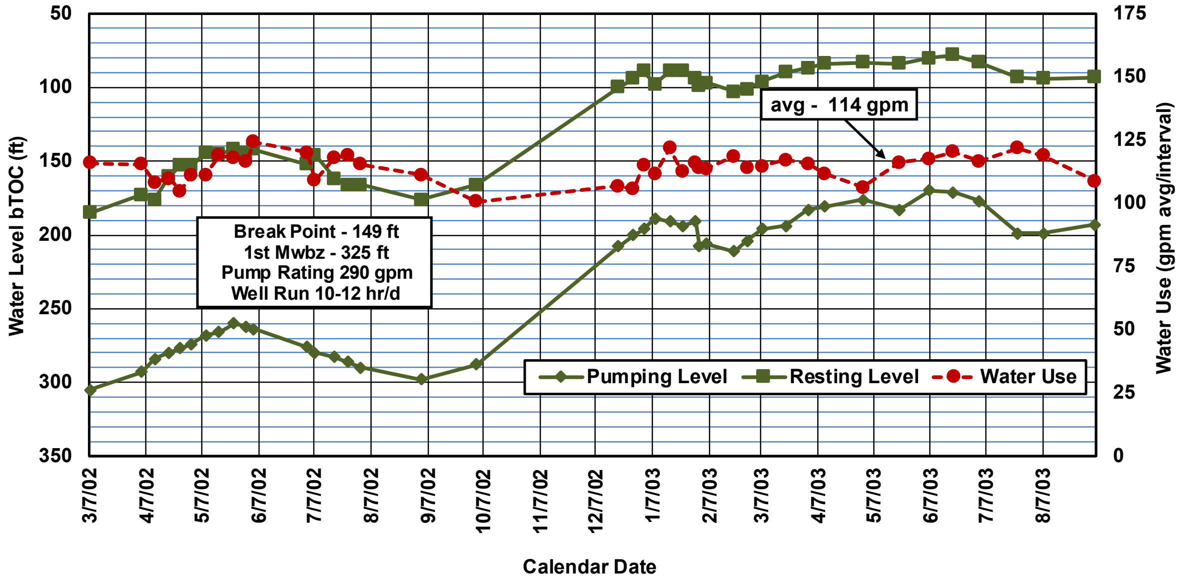

Figure 13. Water-use and water-level data for the period March 2002 to August 2003 for Taneytown well 13.

Figure 13 is a plot over time of the resting and pumping water levels and the average monthly production data collected from well 13 between March 2002 and August 2003. The well was pumped at a relatively constant average rate during that period of 114 gpm (623 m^3/d). The pumping water level in the well was about 280−300 ft (85−91 m) during the drought, which was near the first water-bearing zone. After the drought, the pumping water level rapidly recovered about 80−115 ft (24−35 m) to 180−200 ft (55−61 m), a depth near the break in drawdown data that occurred during the pumping test. The operator indicated that the well was pumped for about 10−12 hours per day during the drought, due to the low water levels. Because of contamination and other problems, it has not been feasible to perform a test to determine if well 13 could have produced more water during the drought. However, based on the limited remaining available drawdown during the drought, it is expected that water levels in well 13 would have reached the pump intake set at 345 ft (105 m), with only a modest increase in the pumping rate. Since well 10 was out of service and effectively abandoned, the reliable yield of well 13 is about one-half of the author's estimated yield without interference and about ¼ of what the city thought was available.

Based on the conceptual model developed in this report, two, initial, recovery wells were completed in the upper, gray, sandstone unit. Withdrawals from those wells reduced tetrachloroethylene (PCE) levels of raw water samples taken from well 13 from an average of 12.1 μg/L (2003−2004) to an average of 4.7 μg/L (2005−2008), the latter which was below the Maximum Contaminant Level (MCL) of 5 μg/L. The remainder of the PCE was removed by a granulated activated charcoal treatment system. Potential, follow-on recovery wells in the lower gray sandstone unit or the discrete, water-bearing zones have not been drilled to date. Well 13 has been out of service since 2009, due to radiological contamination (adjusted gross alpha radiation). Daily dewatering of the upper gray sandstone probably formed an air gap that provided a radiation shield, resulting in lower levels in 2002 (7.1 pCi/L) than those in 2008 (15.7 pCi/L) (MCL = 15 pCi/L) when the sandstone unit was saturated. This suggests that the upper gray sandstone may be a source of the radiological contamination.

SEASONAL VARIATION IN WELL YIELDS

Understanding seasonal variation in well yields is critical for ensuring that public water supplies can supply water demand during a severe drought. As rainfall is the major source of recharge in fractured rock aquifers, there should be a good correlation between variations in rainfall, groundwater levels and well yields. However, a perfect correlation should not be expected because of evapotranspiration, leakage, and changes in groundwater storage. Little information is available on the seasonal variation of well yields in fractured rock aquifers likely due to the difficulty and expense in obtaining synoptic measurements of rainfall, groundwater levels, and well yield. Water system operational data (water levels, pumpage and hours pumped per day) collected during drought and wet periods can provide a suitable substitute for such data. To illustrate seasonal variations in well yields, two case studies at Emmitsburg and Poolesville are discussed. A review of selected published analyses of seasonal variations in well yields is also included

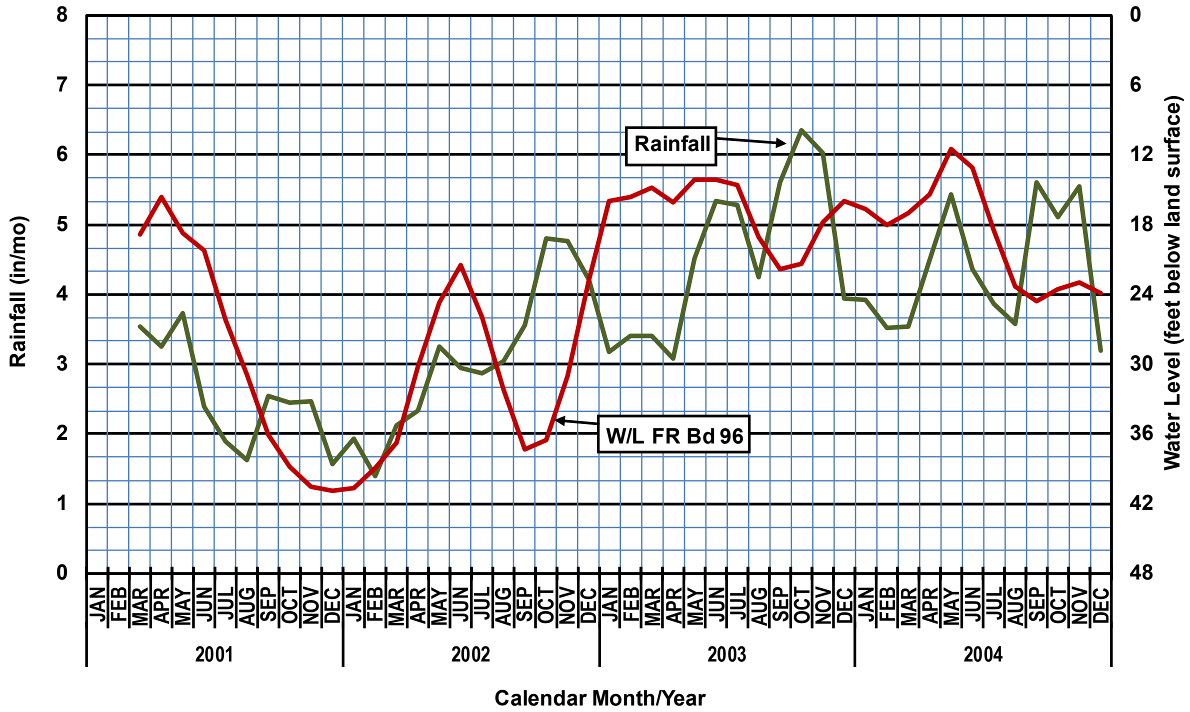

Figure 14. NOAA Emmitsburg 2SE cooperative weather station rainfall data and USGS FR Bd 96 groundwater levels (three-month moving averages).

Figure 14 shows the relationship between rainfall measured at the NOAA Emmitsburg 2SE cooperative weather station and groundwater levels in a nearby USGS monitoring well, FR Bd 96. To smooth out the effects of evapotranspiration, changes in storage, and possible leakage, three-month moving averages were calculated for the data. As a result, there was a good correlation between rainfall and water levels in the monitoring well. Emmitsburg well 3 was pumped continuously during the same monitoring period. As discussed in the previous section, when comparing the regional groundwater levels and well discharge, (fig. 6), a good correlation could only be obtained by shifting the water levels forward by four months. The lagging well yields were most likely due to aquifer leakage and changes in groundwater storage but may also include some water recharged by the nearby surface water reservoir. Based on the averages during the seasonal wet and dry periods, the data indicate that the maximum yield was about twice the minimum, while the highest individual monthly use was about three times the minimum monthly use.

In the case of Poolesville, detailed operational data have been collected by the town that included the period 1996−2003. Shown in Figure 15 are three-monthly moving averages of rainfall data collected by the town and that taken at the NOAA Dulles Airport weather station. It can be seen there is a good visual correlation between the two sets of data and that the Poolesville average rainfall is 13 percent higher than the Dulles Airport rainfall.

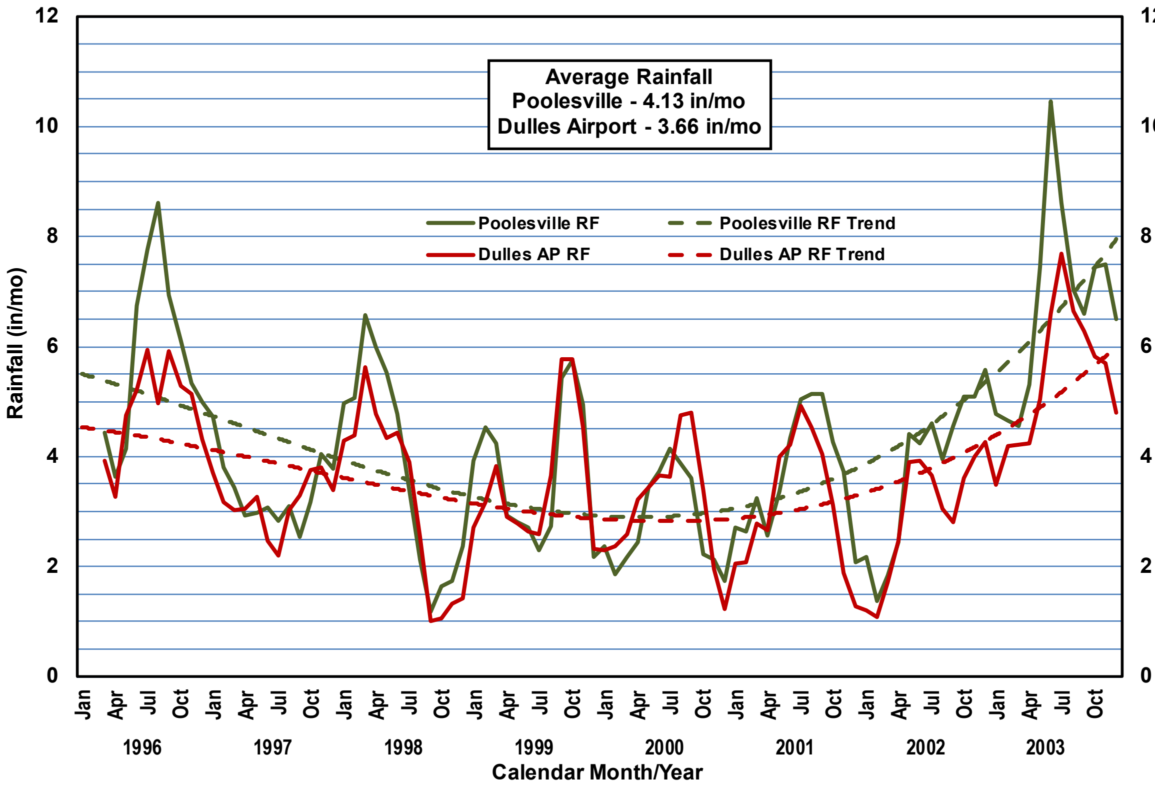

Figure 15. Poolesville and Dulles Airport rainfall data (three-month moving averages).

Figure 16 is a plot of groundwater levels in USGS well 50W 4C, located about 7 miles west-southwest of Poolesville, and rainfall data at Dulles Airport, located about 10 miles south of the town. The data shown are calculated using the three-month moving averages for each station. The trend lines indicate that initial groundwater levels lagged rainfall by several months. In late 2000, groundwater levels then started to follow closely the rainfall pattern until the period ended in 2003. Changing aquifer hydraulic characteristics could provide an explanation for this response. Initially, the main source of recharge to the well may have been due to leakage from an area with high groundwater storage, resulting in the lag in the response of the groundwater levels. As the drought proceeded, dewatering may have occurred to such an extent that there was then a more direct connection between the deep, semi-confined portion of the aquifer and the shallower water table.

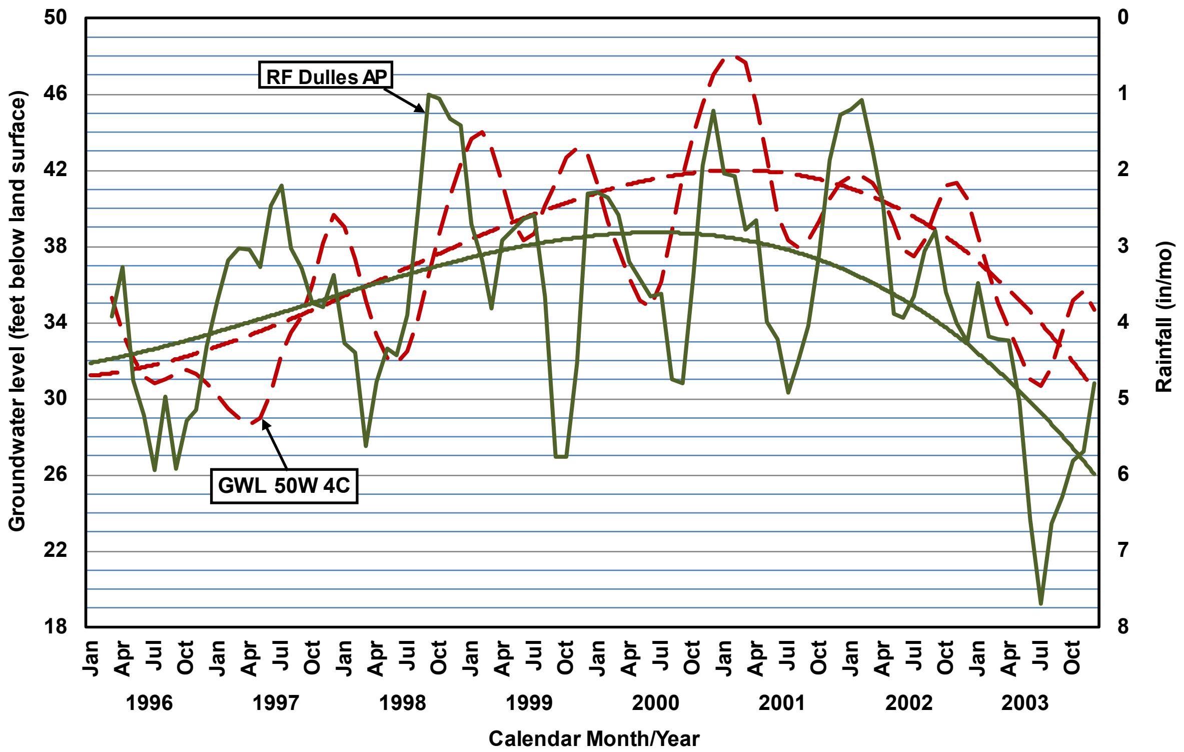

Figure 16. Dulles Airport rainfall data and USGS 50W 4C groundwater levels (three-month moving averages).

Figure 17 is a graph of the production from wells 2−7 plotted against regional water levels in USGS well 50W 4C. Initially, the average production led the groundwater levels by about two months, so the water use data were shifted forward to produce matching trends. This would suggest that the aquifer storage capacity at the reference well site is greater than that near the municipal well field. After mid-1999, the production from the wells increased relative to groundwater levels. This change was related to the replacement of the pump in well 2, which increased its yield from 20 gpm to 100 gpm (76 L/min to 379 L/min). Using the groundwater levels as a proxy, the maximum (wet) unit production rate from the system was about twice as much as the minimum (drought) rate prior to the changing the pump in well 2.

Poolesville's water system is operated intermittently to meet customer demand. Most of the time, the wells in use are pumped for the same specific number of hours each day. During the period from 1997 to 2000, wells 2−7 were pumped in tandem; while, wells 3−5, 7 and 8 were primarily used from 2001 through 2003. During those periods, regional groundwater levels varied from near record lows in December 2000 and record highs in April 1997. Since the wells were operated intermittently, the town's monthly production data were normalized by dividing the average of the daily use by the average number of hours pumped each day.

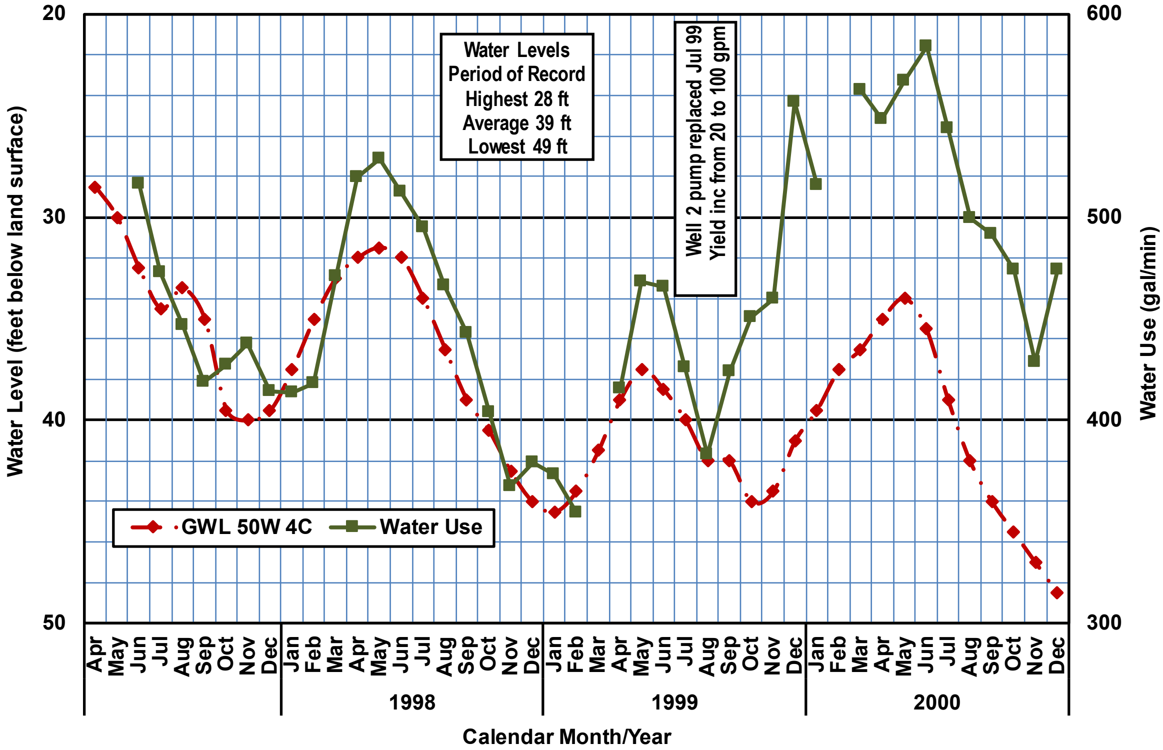

Figure 17. Poolesville production data wells 2−7 and USGS 50W 4C groundwater levels (three-month moving averages [1997−2000]). Water use shifted forward two months.

After 2000, wells 2 and 6 were primarily out of service, while well 8 was then added to the system. Figure 18 is a graph of the production from wells 3−5, 7 and 8 plotted against regional water levels, with the production data shifted forward one month to produce matching trends. Until 2003, the sets of data tracked well, after which the system production declined relative to regional groundwater levels. A review of water level data from the common municipal wells in service during the two wet periods (2003 and 1997) revealed that the wells in 2003 were being underutilized. In March 2003, the water levels were between 79 and 289 ft (24 and 88 m) above those in April 1997 in wells 3, 4 and 5. An exception was well 7, which had water levels of 207 ft (63 m) in March 2003 versus 105 ft (32 m) in April 1997. This anomaly is probably related to a higher capacity pump that was installed in well 7 between the two periods. This is additional evidence that well 7 may have been damaged because of dewatering and compaction of the reservoir rock unit in the wellbore, producing a decreased yield and an increase in operating water levels.

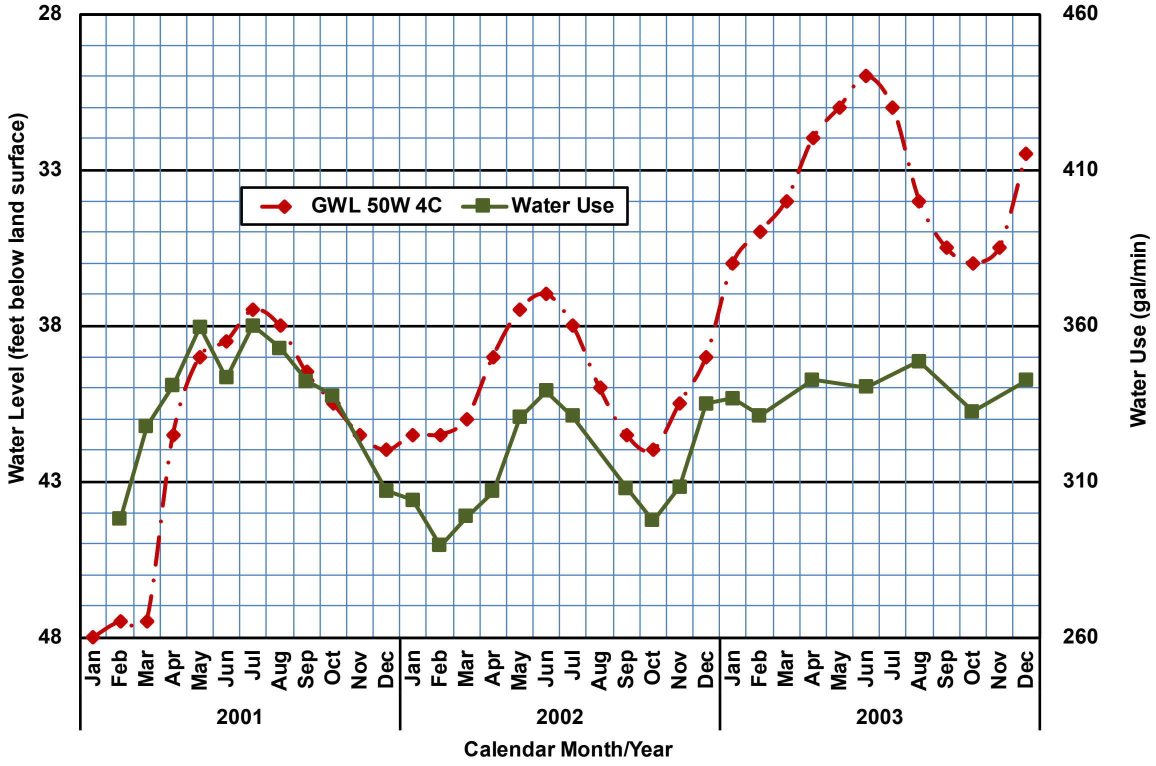

Figure 18. Poolesville production data wells 3−5, 7 and 8 and USGS FR Bd 96 groundwater levels (three-month moving averages [2001−2003]). Water use shifted forward one month.

From records maintained by Poolesville water system personnel, Otton (1981) analyzed pumping rates for wells 1−4 during two, two-month periods in 1978 (Otton, 1981, Figure 13). Otton attributed the 36-percent decline in the total yield of the wells (from 267 gpm or 1,011 L/min in Jan−Feb to 172 gpm or 651 L/min in October − November) to decreases in transmissivity caused by the lowering of water levels in the aquifer near the well field. Groundwater levels in USGS monitoring well (MO Cc 14) indicated that regional levels may have declined about 6.9 ft (2.1 m) during the same period, or 26 percent of the full range of the water levels. The water levels in the monitoring well were above the long-term average for that well due to wet climatic conditions. The town personnel also collected water level data in each municipal well, which are maintained in State record files. These data indicate that major water-bearing zones were dewatered in wells 1, 3 and 4 during the test period. No water-bearing zones were dewatered in well 2 and the yield of that well only declined by 2 percent. Although well 1 has been out of service since 1997, operational data collected by the Town in the 1970s indicate that wells 1 and 2 may have also interfered with each other. The dewatering of major water-bearing zones and well interference indicate that the data in the Otton (1981) cannot be used to reliably determine seasonal variations in well yields.

Finally, it is noted that the Poolesville water system was operated to meet water demand producing a maximum yield of 582 gal/min and a minimum yield of 290 gal/min or a ratio of 2.0:1.

SUMMARY OF OTHER INVESTIGATIONS ON SEASONAL VARIATION IN WELL YIELDS

The following is a summary of seasonal variations in fractured rock well yields from previously published investigations. Detailed discussions of the results of these studies are included in Appendix A8.

Parizek and Siddiqui (1971) noted sharp declines of the water levels and decreased productivity in two wells at Pennsylvania State University, which they indicated were due to interference with other wells in the area and the prolonged effects of drought. There was not enough information provided to separate the effects of drought from well interference.

Daniel (1990) provided daily average production rates for 12 wells during the period September 1983 through October 1984, while constantly pumping the Cary, North Carolina municipal wells 18 hours per day. In the present study, seasonal variation in yields was estimated by multiplying the unit change in production by the differences between the minimum and maximum ground water levels, and two reference levels in a USGS monitoring well. The result was a maximum (wet) yield that was 2.9 times the minimum (drought) yield.

Misstear and Beeson (2000) applied their methodology to a well field in the UK Chalk aquifer. Their operational approach produced an estimated well field yield of 3,300 gpm (18,000 m^3/d) in 1996. Misstear and Beeson indicated that regional water levels in area observation wells were 1.6 ft (0.5 m) lower in 1990 than in 1996. They then applied that value to their drought bounding curve for 1996, indicating that the yield would have been reduced to 2,200 gpm (12,000 m^3/d) in 1990, or a ratio of 1.5 to 1 between the two years. Since that is a ratio is based on annual average yields and neither 1990 nor 1996 were severe drought or wet years, it is expected that the seasonal ratio would be higher. Using ratios of water levels in a nearby observation well and a National Hydrological Monitoring Program well and production from the well field, it is estimated that the maximum seasonal yield of the well field is 1.9 times the minimum yield. It, however, is also noted that the nearby observation well may have been affected by well interference

Piscopo and Summa (2007) collected water level, discharge, and precipitation data for 13 months after the well in their study was placed in service. The average discharge varied from 30 to 52 gpm, while precipitation varied from annualized values of 22.8 in (578 mm) to 42.3 in (1074 mm), and there was a lag of about four months between changes in precipitation relative to changes in well yields. Using precipitation data for the Tanagro River watershed in which the well was located, it was determined in the present study that the long-term average precipitation for the basin, is 47 in/yr (1194 mm/yr) and was 32.5 in/y (826 mm/y) in 2004. These values are like average and severe drought conditions in Maryland. The well continued to be pumped at a constant water level until 2007 and Biaocchi et al. (2014) developed a numerical model showing a seasonal variation between 32 and 79 gpm (2 and 5×10^{-3} m^3/s) in the well's yield, or a ratio of 2.5 to 1.

METHOD FOR ADJUSTING WELL YIELD ESTIMATES FOR SEASONAL VARIATIONS

The studies described above indicate that the maximum yield of a well can be two to three times the minimum yield, depending on climatic conditions. Much of the water use data presented in this report were collected from wells that were pumped continuously (Emmitsburg well 3), at a constant water level (Piscopo and Summa well) or on a constant cycle (Cary, NC), resulting in seasonal ratios of 2.5-3.0:1. Most municipal water supplies, however, are usually operated intermittently to meet customer demand. The Poolesville and the UK well field systems were operated in such a manner, producing ratios between maximum and minimum yields of 2:1 and 1.9:1, respectively. This means that the wells would be underutilized during most periods, leaving more water in aquifer storage to meet drought demands, than if they were run continuously.

Typically, aquifer tests are often conducted under non-drought conditions. In this study, an analytical method for correcting aquifer-test data for climatic effects was developed. The method uses empirical aquifer test data and compares the observed water level in a nearby observation well during the test to the maximum and minimum seasonal water levels in the same observation well. The ratios of the maximum and minimum seasonal well yields presented in this report are then assigned to the maximum (2-3) and minimum (1) regional water levels. By using the ratios between the estimated aquifer-test yield and regional water levels and maximum and minimum yields, the estimated drought yield of the well can be determined. Until additional studies are completed, it is suggested that the following equations be used to determine the reliable drought yield of a well:

$Q_{f\text{obs}} = {}^{A\text{x}B}/_{C} + Q_{f\text{min}}$, where $Q_{f\text{obs}}$ is a factor for the estimated yield extrapolated to time, *t,* on the test date: $A = h_{\text{obs}} - h_{\text{min}}$, $B = Q_{f\,\text{max}} - Q_{f\,\text{min}}$ and $C = h_{\text{max}} - h_{\text{min}}$; where

$Q_{f\text{min}} = 1$ and $Q_{f\text{max}} = 2–3$; where h_{max}, h_{min} and h_{obs} are the maximum and minimum regional water levels, and the observed water level on the date of the aquifer test in a nearby reference monitoring well. The minimum (drought) yield is then: $Q_{\text{min}} = {}^{Q_t}/_{Q_{f\text{obs}}}$, where Q_t = the extrapolated yield at time, *t,* of the well on the test date.

SUMMARY OF ADDITIONAL FIELD INVESTIGATIONS IN MARYLAND

Wells in Crystalline Rock Aquifers

Town of Myersville, Maryland

Water Treatment Plant (WTP) Well

At the Myersville Water Treatment Plant (WTP) well site two aquifer pumping tests and a long-term operational test were conducted. The first was a 1987, 35 gpm (132 L/min) 54-h test. The SVF model best fit the pumping well drawdown data in the interval 0−300 minutes, producing an estimated yield of 18 gpm (68 L/min). After 300 minutes (43 m), a sharp decline in the water level to more than 200 feet was likely caused by sediment clogging the pump intake. The test was secured due to impacts to a nearby domestic well. A geophysical survey confirmed a complex discrete fracture system connected the WTP well to nearby domestic wells. The second 1989, 15 gpm (57 L/min) test was secured when water levels reached apparent equilibrium during the last 10 hours of the test, after 40 feet of drawdown.

Due to the potential impacts to an additional domestic well and the relatively low estimated yield of the WTP well, a long-term test of the WTP well was conducted for 51 days at 16-19 gpm (61-72 L/min) in 1994. The early response (60 ft or 18m) was like the 30-h test drawdown, while the late-time drawdown, 200 feet (61 m), was like the that the end of the 54-h, 16 gpm (61 L/min) test, confirming the estimated yield from the early SVF segment of the 54-h test.

Canada Hill Wells 1 and 2

Three aquifer pumping tests were performed on wells 1 and 2 at the Canada Hill Subdivision. The first was a 72-h test of well 1, starting at 50 gpm (189 L/min) and ending at 25 gpm (95 L/min). There were breaks in the drawdown curve at 70 minutes, caused by a change in the pumping rate, and at 180 minutes, due to a change in formation properties. The Theis solution produced an estimated yield of 10 gpm (38 L/min). The second was a 24-h test of well 2 at 30 gpm (114 L/min), producing an apparent late-time equilibrium. If it had been true equilibrium, the estimated yield would be 30 gpm (114 L/min). If the water level stabilization was temporary, the estimated yield would be 8 gpm (30 L/min). The third was a 24-hour combined aquifer test, while pumping well 1 at 21 gpm (79 L/min) and well 2 at 24 gpm (91 L/min). Those rates were maintained throughout the test; however, the troughs of depression overlapped, suggesting that interference could be an issue under normal operating conditions.

Due to potential impacts to nearby domestic wells and questions about reliable well yields, a follow-on 24-d long-term test was conducted in 1994. The test of well 1 started at 20 gpm (76 L/min), but due to excessive drawdowns, the rate was successively decreased until it reached 10 gpm (38 L/min) at day 10 and remained at that rate until the end of the test. The results confirmed the estimated yield of 10 gpm (38 L/min). The test of well 2 started at 25 gpm (95 L/min), but the pumping rate declined over time until the 18th day when the water level nearly stabilized at 13 gpm for the remainder of the test, or a rate in between the two possible estimated yields of 8 and 30 gpm. The total yield of the two wells is about ½ of the combined rates during the 24-h simultaneous test of the wells, which could be partly explained by interference between the two wells. Also, when correcting for seasonal climatic variations and accounting for interference, it is expected that the total yield of the wells could be a little as 11−14 gpm (45-57 L/min) under severe drought conditions.

Deer Woods Well

A 24-h, 25 gpm (95 L/min) aquifer test was performed on the Deer Woods property well. The time drawdown data best fits a leaky aquifer model, and it was estimated that well could produce 25 gpm (95 L/min) under the average climatic conditions. A 24-d operational aquifer test of the Deer Woods well was conducted, starting at 20 gpm (76 L/min), and declining to a final rate of 12 gpm (45 L/min). Dewatering of the weathered zone could account for some or all the difference between the estimated and demonstrated yields. Another factor to consider is that the leaky aquifer model may not have accurately described the conditions that existed when the bedrock portions of the crystalline rock aquifer are being dewatered. The reported uses during the droughts of 1999 and 2001−2002 were about one-half of the uses reported during the two wettest years of 2003 and 2018.

Town of Middletown

Well 16

After an initial multi-rate test, in lieu of the standard step-drawdown test, and a 72-h, 16 gpm (61 L/min) aquifer test were performed on well 16, the single fracture in the well at 160 ft was isolated and the well was then hydraulically fractured. After that follow-on multi-rate and 72-h, 38 gpm (144 L/min) aquifer tests were performed. The Hantush-Jacob leaky aquifer solution only provided a good fit to the post-fracturing multi-rate test data, probably due to the lack of the recording of recovery data during the first test. The Hantush-Jacob model provided a good fit to the first aquifer test, while the Neuman-Witherspoon two aquifer model provided a good fit to the post-fracturing aquifer test. There was little change of transmissivity between the multi-rate tests, but the post-hydraulic aquifer test indicated that there was a significant increase in transmissivity. This suggested that the stimulation had little influence on the well bore hydraulics, but the flow regime changed from a leaky aquifer to one were there was a direct connection between the weathered zone and the primary water-bearing fracture. The estimated yield increased by at least 23 per cent to 31 gpm after hydraulic fracturing. There, however, is no production data to verify the long-term well yield or to determine if the fracture seals over time, since no proppants were used to keep it open.

Well 15

A multi-rate test was performed on well 15, in lieu of the standard step-drawdown test. The Hantush-Jacob leaky aquifer solution provided the best fit to the multi-rate test data and produced a well efficiency of 87%. The Hantush-Jacob leaky aquifer model produced a reasonable fit to the early-time (0−1500 minutes) drawdown data of the follow-on 72.5-h, 50 gpm (189 L/min) aquifer test. No evidence of a distinct reservoir unit was found. In other cases where a leaky aquifer response occurred, without any clear breaks in the drawdown curve and the drawdown exceeds 100 ft (30 m), and there was more than 100 ft of drawdown to the primary water-bearing zone, those test rates provided good estimates of reliable well yields. A follow-on 60-d, 50 gpm (189 L/min) operational test was then performed, which produced an estimated yield of 100 gpm (379 L/min) if the drawdown was extrapolated to the 370−375 ft (113-114 m) main fracture zone. After the well was placed in service in late 2010, the available pumpage data indicate that the reliable yield of the well was closer to 50 gpm (189 L/min) than the 100 gpm (379 L/min) extrapolation.

Fountaindale Community

A step-drawdown test of Fountaindale well A started at 39 gpm (148 L/min) and reached 190 gpm (719 L/min) during the final step. The Dougherty-Babu solution provided a good fit to all the data, indicating that there was no aquifer dewatering during the test and suggesting that the available drawdown was greater than the 82 ft (25 m) that occurred at the end of the test.

The follow-on 103-h, 173 gpm (655 L/min) aquifer test also ended with a drawdown of 82 ft (25 m). Both tests ended with the water level between the casing depth and the major wbz. The drawdown data required that the curve be divided into two segments. The Hantush wedge-shaped aquifer and Hantush-Jacob leaky aquifer solutions both provided good matches to the early portion of the curve. The Barker General Radial Flow solution fit the late-time data and produced an extrapolated yield of 121 gpm (458 L/min).

The 73-h test of well B was essentially a long-term step-drawdown test starting at 181 gpm (685 L/min) and ending at 310 gpm (1173 L/min). No model could fit the entire set of drawdown data; however, the Dougherty-Babu solution provided a good fit to the first four steps. There was a sharp break in the drawdown curve below 140 ft (43 m) that may have been due to dewatering of a large void at 118−130 ft (36-40m). With an extrapolated drawdown of 276 ft (84 m) and an available drawdown of 140 feet (43 m), the estimated yield of well B is 139 gpm (526 L/min).

The two wells are located about 1000 feet apart, which could lead to significant well interference, but that could only be determined by long-term monitoring. The pumpage data is limited, because it includes withdrawals from six other low yielding wells in the well field and variable uses from wells A and B; but the available water use data indicate that the initial combined estimated yield of the two wells was reasonable and that the effects of well interference were minimal.

Musket Ridge Golf Club

Although not a public water supply, the test and production data for the Musket Ridge Golf Club's wells were analyzed because of the high reported yields. The data and analysis of the well 17 tests are presented to demonstrate a useful feature of step-drawdown tests.

During the step-drawdown test of well 17, the drawdown data started to deviate from the Dougherty-Babu double porosity solution at a drawdown of 65 ft (20 m). During the following 72-h, 200/150 gpm (757/568 L/min) aquifer test, breaks occurred after drawdowns of 63 ft (19 m) and 84 ft (26 m), which occurred within about 10 ft (3 m) of the two major wbzs in the well. After the last break, rate adjustments were made to maintain the drawdown above about 80 ft (24 m). The Moench double porosity solution provided the best fit to the drawdown data before the first break in the curve (0-55 min). In that case, a 90-d extrapolation of 151 ft (54 m) produced estimated yields of 85 gpm (269 L/min) when the calculated specific capacity is applied to the drawdown to the first wbz and 111 gpm when applied to the drawdown to the second wbz.

If the average of 147 gpm (556 L/min) for the first five steps of the step-drawdown test was used during the 72-h tests instead of the 200 gpm initial rate, the drawdown may not have reached the first break point until much later in the test. This would have produced a longer period of steady-state drawdown, leading to a better estimate of the well yield. In this case, it is estimated that a pumping rate of 130 gpm may have prevented the sharp, late-time declines of water levels that occurred during the aquifer test.

Point of Rocks Community Wells N and M

There were multiple aquifer tests of wells N and M, at Point of Rocks. During the first test of well N the drawdown ended at only 6 ft (1.8 m), while pumping at 100 gpm (379 L/min), so the well was tested again to determine if a higher yield could be obtained. During the second test at 125 gpm (473 L/min), the water level stabilized after 1000 minutes, then there was a break in the water level data after 8.2 ft (2.5 m) of drawdown, at a pumping water level of 36 ft (11 m). In 1999, data from the first test of well N were used to estimate the reliable yield of the well; however, the extrapolation was made using an approximation of a drawdown curve made by visual inspection, producing an estimated yield of 22 gpm (83 L/min) at the drawdown of 8.2 ft (2.5 m). When the same data were extrapolated to the water-bearing zone (drawdown = 86 ft or 26 m), the estimated yield was 232 gpm (878 L/min). The initial test data for well M produced an estimated yield of 119 gpm (450 L/min), when water levels were extrapolated to the water-bearing zone (drawdown = 282 ft or 86 m) in that well. A follow-on 20-d aquifer test was conducted on the two wells to verify the sustainability of the water supply. The initial total test rate was 94 gpm (356 L/min) and after 2-3 weeks of pumping there was clear evidence of declining yields. The final total test rate of about 31 gpm (117 L/min) was only about the same as the lower estimated yield for well N. All the test data and a fracture trace analysis indicated that the two wells were directly, hydraulically connected by a single, prominent, vertical fracture, so it was expected that the total yield of the two wells would probably only equal that of the best well (N). Due primarily to the low total yield from the long-term test, the wells were never placed in service.

Town of Mount Airy

Well 6A (Gillis Falls)

Mount Airy well 6A (Gillis Falls) was the only one of the 28 wells drilled for the project that had a thick (29−94 ft or 9−29 m), heavily fractured, highly weathered zone and a high blown yield (200 gpm or 757 L/min). Four aquifer pumping tests were conducted: the first for 72 hours at variable rates; the second for 72 hours at 164 gpm (621 L/min); a following 90-d test, with a continuously declining rate; and a final 96-h, 200 gpm (757 L/min) test. At the drawdown to the base of the weathered zone, the estimated yields were 138 gpm (522 L/min), 104 gpm (394 L/min), and 236 gpm (893 L/min) from the first, second and final tests, respectively. During the 90-day operational test the initial pumping rate was about 230 gpm (871 L/min), with an operating water level of about 75 ft (23 m). The rate declined to about 110 gpm (416 L/min) after 12 days, then to about 60 gpm (227 L/min) after 53 days until the end of the test, at operating water levels between 70 ft and 75 ft (21 m and 23 m). The Gringarten-Witherspoon SVF solution for the second test provided the best fit to the drawdown data from any of the tests. Of the three tests, the estimated yield from the second one came closest to the final yield observed during the 90-d operational test. The overall results of the testing of well 6A indicate that there was high, but limited, aquifer storage near the well that sustained the relatively high yields during the short tests, that was then depleted during the 90-d test, resulting in the final low yield of the well.

Well 11 (South Main Street)

After a step-drawdown test of well 11 (South Main Street) was completed, two aquifer pumping tests were conducted. The first was started at 165 gpm (625 L/min), was stopped, and restarted at 135 gpm (511 L/min), and then secured after 21 hours. During the second test, well 11 was continuously pumped at 75 gpm (284 L/min) test for the full 72 hours.

The step-drawdown test was started at 150 gpm (568 L/min), producing a flat curve, which was followed by a rapid and accelerated drawdown during the second step at 200 gpm, reaching a drawdown of nearly 400 ft (122 m). The sharp break in the drawdown curve occurred at the approximate depth of first wbz (80 ft or 24 m), suggesting that the response was an effect of dewatering of that fracture.

The first aquifer test was started at 165 gpm (625 L/min). After 100 minutes, there was a steep decline in the water level and then an excessive drawdown after 400 minutes. The test was secured and was re-started at 135 gpm, a rate which was also unsustainable.

The second aquifer test was started at 75 gpm (284 L/min), which was maintained throughout the entire 72 hours. No analytical model provided a good fit to the drawdown data. By visual inspection, the estimated yield was 27 gpm (102 L/min) at an available drawdown to the casing depth and 44 gpm (167 L/min) at the drawdown to the wbz.

After the well was placed in service, the maximum reported use was 26 gpm (98 L/min) avg. / 32 gpm (121 L/min) max. in 2013. Water level measurements indicate the water level was generally below the first major wbz.

City of Westminster

Well 6 (South Center Street)

For Westminster's well 6 (South Center Street), the Dougherty-Babu double porosity solution provided a good fit to the step-drawdown test data and the Neuman-Witherspoon two aquifer model best matched the drawdowns from the follow-on 72-h, 82.5 gpm (312 L/min) aquifer test. During neither test was there any evidence of dewatering of a permeable zone, suggesting that the available drawdown was equal to or exceeded the final drawdown of 85 ft (26 m) during the long-term test. The well efficiency was only 23%, which may have been due to turbulent flow because of a screen installed in the well. Using the drawdown to top of the screen of 90 ft (27 m), the estimated yield is 87 gpm (329 L/min). The maximum reported use was 84.5 gpm, or 320 L/min, max.) in 1995. Since then, the maximum reported use was 71.5 gpm, (271 L/min) max. in 2014. The well produced 63 gpm (238 L/min) during the non-drought period 2009-2016 with a water level of 115 ft (25 m); but also 63 gpm (238 L/min) with a water level of 106 ft in (32 m) during the severe drought period of Nov−Dec 2001. These data indicate that there may have been a decrease in the well yield over time of about 25%, which was likely due to lowering the water level below the top of the screen.

Well 7 (Carfaro)

Numerous sinkholes developed during the drilling and testing of the Westminster's well 7 (Carfaro) and the well had a high turbidity problem. What essentially was a 10.9-d step-drawdown test was performed. The Dougherty-Babu double porosity solution fits the data from the first four steps, but there is a substantial deviation from the curve when drawdown reached 26 ft (8 m) at the start of the 437 gpm (1654 L/min) step. That water level was near the casing depth suggesting dewatering of the weathered transition zone had occurred. Using the Dougherty-Babu solution, the estimated yield is 189 gpm (715 L/min).

Reported water use reached a peak of 191 gpm (723 L/min) max. in 2004. The reported use has since declined, with a maximum use of 109 gpm (413 L/min) avg. during the past 10 years. The decline might be related to a limited recharge area.

Well 8 (Vo-tech)

The lithology in well 8 (Vo-tech) indicates that it is heavily weathered to a depth of 55 ft (17 m). During the first of two aquifer tests, the well was pumped for 96 hours at 400 gpm (1,514 L/min), producing an initial IARF segment. Dewatering of shallow water-bearing zones followed, which probably lead to high turbidity and clogging of the pump intake screen. The shallow fractures were then cased off and a second, 14-d, 205 gpm (776 L/min) test was conducted in January 1988. The response during the second test appeared to be one caused by a delayed yield effect. During the first test the shallow, water-bearing zones were directly connected by a short-circuit through the well-bore to the primary fracture system; while, during the second test, water from the, then isolated, shallow zones had to leak into the bedrock fracture system and then flow to the well-bore. The drawdown extrapolated from the IARF segment during the first test and applied to an available drawdown of 37.4 ft (11 m) produced an estimated yield of 211 gpm (800 L/min).

Application of the Dougherty-Babu double porosity solution to the data from the first step-drawdown test produced a poor visual fit, but good statistical results. Better visual and statistical fits were achieved by correcting the data for the effects of aquifer dewatering, using a saturated thickness of 43 ft (13 m). Application of the Dougherty-Babu solution to the data from the second step-drawdown test produced good visual and statistical results; however, the well efficiency was only 15 percent. The best fit to those data was achieved by correcting it for the effects of dewatering, using a saturated thickness of 36 ft (11 m) and producing a well efficiency of 87 percent. These results suggest that casing off the upper, water-bearing zones may have increased the well efficiency by reducing the turbidity and clogging of the pump intake.

Production and water level records were collected for the period 1997–2002. These data indicate that the initial well production of 224 gpm (1244 m^3/d) matched the estimated yield; however, over time, production declined to about 70 gpm to 110 gpm. Regional water levels were generally near drought levels during the period from mid-1998 through 2002. The most likely explanation for these results is that a substantial amount of groundwater was initially taken out of storage, producing the high initial yield. The yield was subsequently reduced, due to limited potential recharge in the capture zone of the well and the prolonged effects of the drought.

Well 11 (Roops Mill)

A step-drawdown test and 74-h, 110 gpm (416 L/min) aquifer test were conducted on Westminster's well 11 (Roops Mill). The Dougherty-Babu solution provides a reasonably good fit to the drawdown data, with some deviation from the type curve during the last few feet of drawdown, when the upper part of a potential limestone reservoir unit may have been dewatered.

During the first 500 minutes of the aquifer test the Hantush wedge aquifer solution matched the drawdown data. After 1500 minutes the Moench dual porosity model provided the best fit to the data and extrapolating from those data produced an estimated yield of 135 gpm (511 L/min), using the drawdown to the top of the limestone unit.

The maximum reported use was 94 gpm (356 L/min) in 2011, while the average water level was 56 ft (17 m) or the possible equivalent of a maximum of 121 gpm (458 L/min) if the water level had been drawdown down to the top of the limestone unit at 70 ft (21m). The use has since declined in 2018 to 64 gpm (242 L/min) max., with an average water level of 67 ft (20 m), suggesting that dewatering of the limestone unit may have decreased the yield of the well.

Wells in Consolidated Sedimentary Rock Aquifers

Town of Poolesville

Well 1

During a 1969 24-h, variable rate test of well 1 there were successive, stepwise reductions in the pumping rate, probably to maintain the water level above the pump intake, until reaching a final rate of 53 gpm and producing an apparent pseudo-equilibrium phase. When an extrapolation from the first 18 minutes of the drawdown data was applied to the 155-ft (47 m) water-bearing zone within a sandstone unit, a much lower estimated yield of 29 gpm (110 L/min) was produced. Otton (1981) reported that well 1 produced an average yield of 24 gpm (91 L/min) under dry conditions and a maximum use of 39 gpm (148 L/min) under wet conditions. The 1996 production was 26 gpm (98 L/min) during a very wet year, indicating the well's yield may have declined due to dewatering of the

155-ft (47 m) wbz and/or the sandstone unit. The well was abandoned in 1999 due to a high turbidity problem that could not be repaired.

Well 2

Well 2 was only producing about 20 gpm (76 L/min) in 1999, although the aquifer test data indicated the well should be able to produce at least 100 gpm (379 L/min). The pump was replaced, and the yield immediately increased to 93 gpm (352 L/min), during a 32-day test. During a subsequent analysis, the Hantush leaky aquifer solution provided the best fit to the drawdown data during the early portion (0−642 minutes) of a 1969 24-h variable rate test, starting at 108 gpm, producing estimated yields of 44, 95 and 137 gpm (167, 360 and 519 L/min) when the drawdowns was extrapolated to each of three individual sandstone units in the well. During a 2009 extended pumping test the water level dropped below the second unit and still exceeded 100 gpm (379 L/min), suggesting that the third or deepest sandstone unit may be the primary reservoir rock unit in the well.

Well 4

The first (24-h) of two aquifer tests of well 4 was conducted at 50 gpm (189 L/min). A straight line was produced on the drawdown curve during the first 1000 minutes, followed by a flattening of the curve until the end of the test. The second (7-d) test was conducted at 48 gpm (182 L/min). A pseudo-equilibrium phase occurred during the first two days of that test, followed by a sharp break in the drawdown curve at 95 ft (29 m), and then by a straight-line response for the remainder of the test. The sharp break occurs at the base of a massive sandstone unit, suggesting that the response may have been due to dewatering of that zone.

The water level may have rapidly passed through a transition zone during the first test, isolating flow from the shallow zone, producing no change in slope, until the water level stabilized near the end of the test, which was probably due to leakage. During the second test, the early-time phase was probably due to a leaky aquifer effect. A sharp decline occurred once the dewatering of the sandstone unit was complete, followed by the late-time, straight-line response.

Extrapolations from the straight-line portion of the second test to the second sandstone unit produced an estimated yield of 31 gpm (117 L/min), while extrapolations to the third sandstone unit produced a yield of 55 gpm (208 L/min). The maximum production from the well was an average of 33 gpm (125 L/min) in Jan 2001−Feb 2001. Data collected by Otton (1981) indicated that well 4 produced an average of 45 gpm (170 L/min) during 1978. Subsequently, the yield may have declined due to dewatering of the sandstone units.

Well 5

A 24-h test of well 5, under average climatic conditions, was started at 129 gpm (488 L/min) and at 50 minutes the rate was adjusted to 120 gpm (454 L/min), due to a reported airline leak. The Hantush-Jacob leaky aquifer solution provided a good fit to the drawdown data, indicating that the well could produce 120 gpm, at the final drawdown of 133.5 ft (41 m). The IARF solution also fit the late-time drawdown data, producing an estimated yield of 94 gpm (356 L/min).

The well was operated 24 h/d during the period October 2009 − December 2009, under average climatic conditions, producing 97 gpm (367 L/min) with a water level of 132 ft (40 m). These data indicated that there was no change in yield over time, if the estimate from the IARF segment is valid.

If the estimate from the leaky aquifer solution is correct, the the yield may have declined by about 19 percent.

Well 6

Initial estimates of the sustained yield of well 6 were 225−261 gpm (852−988 L/min), based on two aquifer tests conducted in 1984 and 1985; however, the well was producing less than a daily average of 100 gpm (379 L/min) in 1999. A review of production data indicated that the greatest declines in yield occurred after the first major water-bearing zone in the well was dewatered. An estimated yield of 168 gpm (636 L/min) was based on extrapolating drawdown from an IARF segment of the 1985 test. The town installed a valve to control and maintain the water level in the well above the 1st major water-bearing zone; however, this did not increase the yield of the well. Production data indicate that the well initially could produce a maximum yield of 185 gpm (700 L/min) in 1991, but that it had declined by 2006 to a maximum of 113 gpm (428 L/min). The reduced yield appears to have been due to dewatering of the upper water-bearing zone.

Well 8

The Hantush-Jacob solution provided a good fit to the drawdown data from a step-drawdown test of well 8, producing a well efficiency of 100%, and a good fit to the follow-on 72-h, 80-gpm aquifer test. This was evidence that was no dewatering of a reservoir rock unit. The test results indicate that the well could sustain 80 gpm (303 L/min). The maximum effective production occurred during a wet period in 1997, when the well pumped at the 24 h/d equivalent of 71 gpm (269 L/min). By comparison, the well produced a 24 h/d equivalent of 61 gpm (231 L/min) under dry conditions in 2001. In January 2019, well 8 produced a 24 h/d equivalent of 57 gpm (216 L/min), under wet conditions. These data indicate that there may have been a 20% decrease in the well's yield over time, although there was no evidence of dewatering of a reservoir unit.

Well 9

Step-drawdown and 72-h, 225-gpm aquifer tests were performed on well 9 (Powell well) in June 2001, under average climatic conditions. The Hantush-Jacob leaky aquifer solution provided the best fit to the first three steps with a total drawdown ~210 ft (64 m), at which point the drawdown data deviates from the model. The Hantush-Jacob solution also provided a good fit to the early portion of the aquifer test, with a break in the drawdown curve at 189 ft (58 m), which was probably due to dewatering of the first major wbz at a drawdown of 189 ft (58 m). An extrapolation from the early portion of the aquifer test curve produces an estimated yield of 152 gpm (575 L/min).

Well 10

A step-drawdown test and a 72-hr, 80-gpm (303 L/min) aquifer test were performed on well 10 (Cahoon well), under average climatic conditions. The Hantush-Jacob leaky aquifer solution provided the best fit to the step-drawdown data; however, the data deviated from the model between 216 ft (66 m) and 288 ft (88 m), possibly due to aquifer dewatering effects. The Hantush-Jacob solution also provided a good fit to the aquifer test data until a break in the drawdown data occurs at a drawdown of 320 ft (98 m). There are no obvious reservoir units, so the break in data probably reflects a change in the bulk aquifer permeability. The relative stability of the water level prior 3600 minutes suggests the yield of well 10 should be slightly less than the test rate of 80 gpm (303 L/min).

Production from wells 9 and 10

Due to their proximity (1300 ft apart), potential interference between wells 9 and 10 was expected, so the production from those two wells had to be analyzed in aggregate. Near the end of a long-term operational well 10 was shut down. Well 9 continued to be pumped at an average of 126 gpm (477 L/min) for about 5 days, after which the rate was then reduced, with recovery of the water level to 90 ft (27 m) after 17 days. The water level in well 10 recovered to 136 ft (41 m) by the end of that period. This was evidence of interference since the water level in well 10 should have recovered to near its original SWL of 26 ft (8 m). Other evidence of interference is that domestic wells in the Sugarland Forest community, about one mile south of well 9, were impacted by withdrawals from that well in 2007.

Well 10 has essentially been out of service since 2003. A 72-hour operational test of well 9 was conducted in November 2015 that produced an average of 108 gpm (409 L/min), with a water level of 152 ft (46 m). It appears that the yield may have declined somewhat, but it is not clear why this happened. Since there was still substantial available drawdown, this may suggest that a reduced pump capacity caused the decline in the yield of well 9.

Well 12

Step-drawdown and 48-hour variable-rate aquifer tests were conducted on well 12 (Schraf well). The Dougherty-Babu solution fit the first two steps of the step-drawdown test, diverging from the model after about 120 ft (37 m) of drawdown. The aquifer test was started at 175 gpm (662 L/min) with the drawdown reaching a break in the curve after 180 minutes and 96 ft (29 m) of drawdown, at about the same depth as the base of the casing and/or the first minor wbz. When an extrapolated drawdown of 180 ft (55 m) is applied to the 96-ft (29 m) break, the estimated yield is 94 gpm (356 L/min).

A long-term test (60 days) of well 12 was conducted during which the well produced 43 gpm (163 L/min), running 24 h/d. The well later produced a 24 h/d equivalent of 93 gpm (352 L/min). The substantial difference in yields was probably due to interference with well 2, located 1850 ft (564 m) from well 12, since well 2 was operated 24 h/d during the long-term test and shutdown during the later pumping period.

City of Taneytown

Wells 10 and 10R

When well 10 was completed in 1967, the driller reported that there was a green sand at 80−120 ft (24−37 m) and a white sand at 415−438 ft (126−134 m). The green sand was isolated by a casing and the SWL was 39 ft (12 m). The well was taken out of service in the early 1990s, due to declining yields related to interference with well 13 and this was confirmed by a SWL of 149 ft (45 m) in 1999. The Dougherty-Babu solution provided the best fit to the first 4 steps of a step-drawdown test; however, there was a break in the drawdown curve at 105 ft (32 m) at the base of a green sandstone, indicating that it was a possible reservoir unit. The follow-on aquifer test was started at 350 gpm (1325 L/min) and ended at 165 gpm (625 L/min). A leaky aquifer model best fit the drawdown data prior to a late-time water level equilibration, resulting in an estimated yield of about 71 gpm; however, there are no operational data to confirm that estimate.

In 2010, well 10R was drilled 10 ft (3 m) from and as a replacement for well 10. It encountered three grey or green sandstones, which correspond to the possible reservoir units noted in wells 10 and 13, and the casing isolated the upper sandstone unit. The Dougherty-Babu double porosity solution provided a good fit to the step-drawdown test data, with no breaks in the drawdown curve. The SWL was 29 ft (9 m), or 120 ft (37 m) above the 1999 SWL in well 10, because well 13 was no longer interfering since it was out of service. The Moench double porosity solution best fit the drawdown data from the aquifer test, which produced an estimated yield of 183 gpm (693 L/min). That estimated yield is more than 100 gpm (379 L/min) greater than that of well 10, giving further evidence of interference with well 13.

To date, there has been no reported use from well 10R and there are no available records of water use from well 10. The data from testing of these two wells were analyzed and presented to support the concept of reservoir units providing the source of water to wells in fracture rock aquifers, in addition to serving as a good example of well interference.

Well 14

A step-drawdown test of well 14 was started at 23.5 gpm (89 L/min) and ended at 95 gpm (360 L/min). The Dougherty-Babu solution matched the entire set of drawdown data, but with a poor statistical fit. The best fit to the data was achieved when a window of 0–400 minutes was applied, to account for disruption of the test due to the shutdown of the well pump, and a correction was made for dewatering of a 270-ft (82 m) aquifer.

A follow-on 72-h variable-rate test averaged about 200 gpm (757 L/min) for the first 1500 minutes and then the rate was changed to 235 gpm (889 L/min) for the remainder of the test. The SVF solution fit the early (0-13 min) drawdown data. The Theis radial flow confined aquifer solution provided the best fit during the period of 13-1500 min; but, after 1500 min, the drawdown starting at 264 ft (80 m) was about twice what would have been predicted by the Theis model, suggesting dewatering of a deep reservoir unit. This corresponds to the corrected aquifer thickness (270 ft or 82 m) in the step-drawdown test solution. An estimate based on an available drawdown of 264 ft (81 m) and a 90-d extrapolated drawdown of 331 ft (101 m) from the 13-1500 period produces an estimated yield of 160 gpm (606 L/min).

Initial use from the well was limited by impacts to nearby domestic wells. The maximum reported use was 96.5 gpm (365 L/min) max. in 2004 with pumping water levels of about 165 ft (50 m). Production was 54 gpm (204 L/min) max., with operating levels of about 185 ft (56 m) during January – June 2012 and 250 ft (76 m) during July – December 2012. The yield may have declined by as much as 50 percent between 2004 and 2017, since the use was 39 gpm (148 L/min) max., with an operating water level of about 184 ft (56 m) during July – December 2017. As with well 13 and the Poolesville wells, dewatering of the aquifer may have reduced the yield of well 14.

Well 17

A step-drawdown test and 72–h, 250 gpm (946 L/min) aquifer test were performed on well 17. Initially, there was a very poor fit to the step drawdown data using the Hantush-Jacob solution. The well has four green sandstone units in the well above 600 ft (183 m). The best results were achieved when the data were then corrected for an aquifer thickness of 400 ft (122 m) and a 0–90 minutes window (1st step and recovery) was used in the analysis, suggesting that dewatering of the uppermost green sandstone unit may have occurred. One or more of the deeper sandstones may also serve as reservoir units. There was no obvious dewatering of a reservoir unit during the 72-h test, as the drawdown reached about 620 ft (189 m); however, the results of the step-drawdown test indicate that dewatering during the aquifer test may have been so rapid that any deflection in the drawdown curve could not be detected. The data from the early part (0−100 min) of the test best fit the Hantush wedge aquifer model and the SVF solution provided the best fit to the later data (300−4320 min). Extrapolating the late-time drawdown, when corrected for an aquifer thickness of 400 ft (122 m) produced an estimated yield of 120 gpm (454 L/min).

Well 17 was placed in service to only meet peak demand. The well has been pumped at a very low maximum rate of 24 gpm (91 L/min). The water levels have fluctuated between 60 ft (18 m), at high regional water levels, and 289 ft (88 m), at low regional water levels. These data indicate that the well yield may be substantially less than could be predicted from the pumping test data. One possible reason is that the available drawdown is much less than the 400 ft used to make the estimated yield. If the uncorrected drawdown at the end of the first step (84 ft or 26 m) were used, the calculated yield could be as low as 25 gpm (95 L/min).

Town of Thurmont

Wells 8 and 8A

The aquifer test of well 8 started at 325 gpm (1,230 L/min) and a sharp break in the drawdown curve occurred near the end of the first day, producing an excessive decline in the water level. The water system operator indicated that the test had to be secured due to a clogged pump intake. The test was-secured and re-started at 162 gpm (613 L/min), with the water level stabilizing until the end of the test. The derivative was approaching a constant value at the end of the first day, so the Cooper-Jacob straight-line (IARF) solution was used to determine the estimated well yield. The results were estimated yields of 126 gpm (477 L/min), when the drawdowns were extrapolated to the break in the drawdown curve (drawdown = 38 ft or 12 m), and 320 gpm (1,211 L/min), when the data were extrapolated to the first major water-bearing zone (drawdown = 93 ft or 28 m). Well 8A was a replacement well drilled 15 ft (4.6 m) from well 8 and had a screen installed, which helped reduce, but did not eliminate a turbidity problem. During the 24-h test of well 8A, there was no sharp decline in water level, either because the test ended too soon, or the screen was operating efficiently. Production data indicated that the maximum yield from well 8A was a daily average of 158 gpm (598 L/min) during the drought, with pumping water levels within 10 feet of a basal limestone conglomerate, or the possible reservoir unit. This was close to the minimum estimated yield 152 (575 L/min) derived by extrapolating to the drawdown (40 ft or 12.3 m) that occurred at the end of the test of well 8A.

The Dougherty-Babu model provided a good fit to the drawdown data from a step-drawdown test of well 8 with no evidence aquifer dewatering. The results of the step-drawdown test and the presence of 66 ft (20 m) of clay in the upper portion of the saturated zone indicate that the aquifer is confined in the vicinity of wells 8 and 8A. No fractures were identified in the clay layer and the alluvial mountain wash was unsaturated. This suggests that the sharp decline in water level during the test of well 8 was neither due to fracture zone nor weathered zone dewatering but was most likely due to sediment clogging the pump intake filter.

DISCUSSION

Reductions in well yield (specific capacity) as discharge rate during step-drawdown tests increases in fractured rock wells may be attributed to factors not related to turbulent flow. Pumping of unconfined, fractured rock wells and wells completed in semi-confined, consolidated sedimentary rock aquifers, where there is a significant reduction in saturated thickness due to dewatering, can lead to other head losses which reduce specific capacity. Application of automatic curve-fitting techniques achieved the best results when dewatering did not occur during step-drawdown tests or the drawdown test data were corrected for the effects of dewatering, thus producing high well efficiencies and low well losses, at reliable yields. When making corrections for aquifer dewatering, the best results were achieved when the depths near the bases of the transition zone in crystalline rocks and those of thin-bedded, permeable units in consolidated sedimentary rocks were used as the aquifer thickness. These transition zones or permeable units act as high porosity reservoir rocks, which may not allow direct flow of groundwater to a well, because the permeability of the units and the areas exposed to wellbores are small. A continuum of pervasive (non-discrete) macro- and micro-fractures in the matrix of the deeper portion of the aquifer may extend over a large area, which can allow efficient flow from the reservoir rocks to hydraulically connected, discrete, water-bearing fractures intersecting the wellbore. In tests where high turbidity levels were recorded, well efficiencies were relatively low, due to possible clogging of pump intake screens.

Summaries of the results of the analyses of the field aquifer tests (multi-rate, step, and aquifer-tests) are shown in Appendixes A3 and A4. Initially, during the period from the 1970s through the early 2000s, the results of step-drawdown tests often led to aquifer test rates that were too high, causing unstable water levels to occur before the aquifer test could be completed. This led to the initiation of the use of multi-rate tests in lieu of the standard step-drawdown test procedure. The main advantage of the multi-rate test is that, because of the recovery periods, later steps are relatively unaffected by drawdowns occurring during earlier steps, so that it is easier to determine what should be the optimum rate for an aquifer test. The one disadvantage is that only a limited portion of the aquifer may be tested by that method. Conversely, the standard step test can better test more of the aquifer and identify break points or limits of available drawdown, where the drawdown deviates from a model solution type curve. As a result of this investigation, a new method was developed that combines the best features of these two types of tests. This consists of conducting a step-drawdown test designed to produce the maximum controlled drawdown during a test. The data are then analyzed to determine which model best fits the drawdown data and if there is a deviation from the type curve that can be used to determine the available drawdown. The average pumping rate during the test at the point of deviation is then calculated. Due to the logarithmic relationship between drawdown and time, a factor equal to about 80 to 90 percent of the average rate should then be used as the optimum rate for the follow-on aquifer test. This method requires that analysis of the step-drawdown-test data be conducted in the field or shortly after return to the office, but before the start of the aquifer test. This is now more feasible than in the past due to the ready availability of computer assisted analytical programs.

IARF and pseudo-equilibrium responses commonly occur during fractured rock aquifer pumping tests. Where leaky aquifer effects occurred, water levels were initially stable, but often were declining at the end of the tests, due to boundary effects related to dewatering of permeable zones. Evidence indicates that two crystalline rock wellbores (Emmitsburg well 3 and Westminster well 8-Votech) were open to the weathered transition zone, producing short-circuits that connected the weathered zone and bedrock portions of the aquifers. When a second casing was set to bedrock in the Vo-tech well, the drawdown changed from an IARF to a leaky aquifer response. This effect was not observed during the consolidated sedimentary rock tests, possibly because the weathered zones in those wells were unsaturated. In many cases, when good quality data were available, the modified Dougherty-

Babu double porosity solution provided the best fit to step-test data, even in the cases where leaky conditions existed. This was probably due to the relatively short duration of the tests, such that the drawdowns occurred before the start of leakage effects.

Leakage and IARF conditions were the dominant flow regimes during most of the hydraulic aquifer tests conducted in the central Maryland study area. The responses reflect single-porosity, homogeneous, anisotropic aquifers produced by layering in weathered transition zones in crystalline rocks or sandstone/limestone units in consolidated sedimentary rock formations. In addition, there were several examples (e.g., Myersville Water Treatment Plant well and the Point of Rocks wells) where flow was controlled by single-porosity, vertical fractures. Conversely, the modified double-porosity Dougherty-Babu model provided the best fit to the time-drawdown data collected during most of the step-drawdown tests, especially in the crystalline rock aquifers. One possible explanation for this difference is that the rock matrix usually consists of numerous blocks, which are large relative to the volume tested during the step tests, but small compared to the reservoir size as troughs of depression expand during the longer tests. A second possibility is that the fracture networks are discontinuous. In that case, a well may intersect a local fracture system that only extends a short distance from the wellbore beyond which a radial flow regime can develop, given sufficient pumping time.

Derivative analyses were useful in identifying IARF segments on drawdown curves (e.g., Emmitsburg well 3, Poolesville well 6 and Thurmont wells 8/8A), from which drawdown data could be extrapolated. If IARF segments were not present, then type-curve and derivative methods were used to determine the appropriate conceptual model for making predictions of well yields (e.g.: Poolesville well 7, leaky aquifer; Myersville WTP well, single vertical fracture; and Fountaindale well B, double porosity aquifer, effectively a long-term step test). The best results in this study were obtained by applying extrapolated specific capacities to available drawdowns determined by the depths to the reservoir rocks, rather than those to discrete, water-bearing fractures.

These methods may apply to crystalline rocks with unsaturated weathered zones, and consolidated siltstone and shale formations, but additional study is required, since changes in porosity/permeability in those types of aquifers may be subtle and hard to detect with standard, single-well, pumping test procedures.

There was evidence that, after being placed into service, the yields of multiple consolidated sedimentary rock wells (e.g., Poolesville and Taneytown municipal wells) in the present study declined about 40−50% each after being placed into service. It is likely that natural fractures in the wells or aquifer may have closed over time, due to deformation as water levels in the wells declined. Drawdowns may have exceeded elastic limits of the aquifers in the vicinity of the wells, causing permanent damage to each well. Until additional research is conducted, this suggests that water levels should not drop below reservoir units, instead of using the commonly held precaution of limiting drawdowns to the depths of discrete, water-bearing fractures. A similar problem could not be detected during the crystalline rock well investigations, possibly due to the adjustment of the weathered zone to seasonal variations in water levels, and the low permeability and porosity of the bedrock portions of those aquifers. Also, in the case of the Poolesville well field, the maximum yield was obtained by operating the wells continuously rather than in a cyclic manner. Oversized pumps in wells 6 and 7 required cycling of the pumps in those wells. This led to dewatering of reservoir units and long-term reductions in yields.

When wells were pumped continuously (Emmitsburg well 3), on a uniform cycle (Cary, NC wells) or at a constant water level (Piscopo and Summa well), the maximum:minimum seasonal water use ratio was 2.5:1 to 3:1. If wells were used to meet seasonal water demands, the ratio was 1.9:1 (UK chalk well field) and 2.0:1 (Poolesville well field). The difference is likely to underutilization of wells to meet non-growing season demand allowing more water to remain in storage to meet summertime demand. These ratios, and the proportional relationships between maximum and minimum and observed water levels during aquifer tests in nearby USGS observation wells were used to estimate seasonal variations in well yields.

SUMMARY AND CONCLUSIONS

Derivative analysis techniques were useful for locating IARF segments on a semi-log, time-drawdown curve or, if an IARF segment was not present, determining the correct conceptual model from which drawdown data could be extrapolated. The best results were obtained by applying extrapolated specific capacities to available drawdown at permeable zones. In the central Maryland study area, these permeable zones are commonly located in the transition zone between weathered rock and bedrock in crystalline rock aquifers, or thin-bedded sandstones or limestones in consolidated sedimentary rocks. These units may not yield water directly to the wellbore, but rather act as reservoirs that are hydraulically connected to primary water-bearing fractures. The discharge rate for a long-term pumping test must be chosen carefully, because if too low, the drawdown does not reach the permeable units, and if too high, dewatering is too rapid for the zone to be identified. Reservoir rocks may be better identified by extending the standard three-rate step-test to five or more steps, with progressively increasing discharge rates designed to produce the maximum possible controlled drawdown.

For the cases in this study, well efficiencies were high and well losses were minimal when step-test data were corrected for the effects of dewatering. This was probably due to the absence of screens and gravel packs (as is typical for most fractured rock, open-borehole wells), which can increase entrance velocities. Also, open fractures may extend some distance from each of the wells, increasing the effective radii of the boreholes. To check for evidence of the effects of aquifer dewatering, step-test data should be analyzed using deconvolution techniques and type-curve methods, in lieu of arithmetic plots of specific drawdown versus discharge. Should significant well losses occur, they can be subtracted from available drawdowns to reservoir rock units when estimating reliable well yields. The best features of step and multi-rate tests can be combined to derive the optimum pumping rate for the associated aquifer test by maximizing the drawdown during a step test, analyzing the data to identify a deviation from the type curve, and then taking about 80−90 percent of the average rate at that point as the pumping rate for the aquifer test.

Operational data from this and other studies indicate that when a well is pumped continuously, the maximum yields during wet periods may be two and one-half to three times greater than minimum drought yields. When pumped intermittently to meet demand, the ratio is probably closer to two to one. At present, the only practical method for correcting aquifer test data for climatic effects is to use this empirical data and compare the seasonal water levels in a nearby monitoring well to the water level that occurred during the aquifer test.

Step tests and short-term, single-well, aquifer pumping tests are commonly used to determine well capacities for water system design. These determinations have typically relied on relatively costly programs that may include geophysical well logging, multi-well and packer/interval testing, water balance calculations and numerical analyses. Where financial resources are limited, the methods used in this study provide an alternative to those programs, and ultimately can help in planning and support of the more costly methods. Once a well is on-line, cost-effective, long-term, operational data can be collected by water systems operators to firmly establish its yield.

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APPENDIXES

A1. List of terms

Organizations

MDE	Maryland Department of the Environment
DNR	Maryland Department of Natural Resources
WRA	Water Resources Administration
WMA	Water Management Administration
WSA	Water and Science Administration
WSP	Water Supply Program

General

English units	U.S. English units of measurement
S.I. units	International units of measurement (Système international)
Duration	length of test
Rates	range of pumping rates
Window	time period of analysis
RSS	sum of the squared residuals
Mean	average of the residuals
T	transmissivity
t-ratio	estimated T/standard error
HGMR	Hydrogeomorphic Region
Formation	geologic formation
wbz	water-bearing zone

Aquifer pumping tests

Theis	radial flow solution for a confined aquifer
IARF	infinite acting radial flow (straight-line) late-time solution for a confined aquifer
SVF	Gringarten and Witherspoon solution for a single vertical fracture in a confined aquifer
H-J	Hantush-Jacob/Hantush leaky aquifer solution for a leaky confined aquifer without aquitard storage
H/Aqst	Hantush solution leaky aquifer solution for a leaky confined aquifer with aquitard storage
N-W	Neuman-Witherspoon solution for a leaky confined two-aquifer system
M 2por	Moench solution for a double-porosity confined aquifer
M-3	Moench Leaky aquifer solution (case 3) overlying constant head/underlying no-flow boundary
GRF	Barker generalized radial flow solution for a single- or double-porosity fractured aquifer

A1. Continued.

	Step-drawdown tests
D-B	Dougherty-Babu solution in a double-porosity confined aquifer
H-J	Hantush-Jacob solution in a leaky confined aquifer
B	linear (aquifer) head loss coefficient
C	non-linear (well) head loss coefficient
P	nonlinear well exponent
W.E.%	well efficiency percent
Multi-rate test	modification of step-drawdown test with recovery between steps

Well construction and water use

SWL	static water level
BTOC	below top of casing
BMP	below measuring point
Diam	diameter of main casing
Tdepth total	depth of well
Available D/D	prescribed lowest operating level
S_{90}	90-d extrapolated drawdown
Production	average daily or monthly water withdrawals
Well Permit	number of State well drilling permit application and completion report
Water Use Permit	State Water Appropriation and Use Permit (groundwater)

Appendix A2. Well construction information.

[in, inch; ft, feet; HGMR, hydrogeomorphic regions; NR, not reported; >, greater than]

Town/Site	Name No.	Well Permit	Water Use Permit	Diam. (in)	Well Depth (ft)	Casing Depth (ft)	Pump Depth (ft)	Main Water-bearing Fracture Depths (ft)	Reservoir Rock Depths (ft)	HGMR	Formation
Emmitsburg	3	FR-65-0432	FR1976G114	8	203	43	167	112,184	66 (bottom)	BR	Catoctin Metabasalt
Myersville	WTP	FR-81-4337	FR1987G004	6	410	41	350	350	140 (bottom)	BR	Catoctin Metabasalt
	CH 1	FR-88-3098	FR1988G035	6	450	63	210	242	107 (bottom)	BR	Catoctin Metabasalt
	CH 2	FR-88-3400	FR1988G035	10	300	51	172	150	unknown	BR	Catoctin Metabasalt
	DW	FR-88-3046	FR1987G204	6	425	63	>167	160	unknown	BR	Catoctin Metabasalt
Musket Ridge GC	17	FR-94-1822	FR1998G022	8	240	37	NR	57-62,77-82,90,105	37-57 (bottom)	BR	Catoctin Metabasalt
Middletown	15	FR-94-1544	FR1974G025	8	500	60	>150	94-96, 370-375	unknown	PCR	Catoctin Metabasalt
	16	FR-94-3317	FR1974G225	8	500	55	>150	160	>90	PCR	Catoctin Metabasalt
Fountaindale	A	FR-88-4859	FR1966G012	8	500	60	>92	105-110	92+ (bottom)	PCR	Catoctin Metabasalt
	B	FR-88-4860	FR1966G012	8	500	44	>200	415-420	118-130	PCR	Catoctin Metabasalt
Point of Rocks	M	FR-69-0376	FR1968G001	6	345	30	300	320	66 (bottom)	PCR	Tomstown Dolomite
	N	FR-94-0328	FR1968G001	6	134	38	100	112	Unknown	PCR	Tomstown Dolomite
Westminster	6	CL-81-0249	CL1977G336	8	623	116	> 115	65-96	90	PCR	Sams Creek
	7	CL-81-4631	CL1977G636	8	328	33	131	159-62, 211?	33 (bottom)	PCR	Wissahickon Schist
	8	CL-81-2458	CL1977G736	8	175	69	100-120	125	39 (bottom)	PCR	Ijamsville-Marburg
	11	CL-94-3169	CL2000G025	8	250	73	NR	89-100	70-93	PCR	Sams Creek
Mount Airy	6A	No tag	CL2007G011W	12	507	94	NA	29-94	29-94	PCR	Ijamsville-Marburg
	11	CL-95-1415	CL2009G001	8	800	60	NR	80, 350	60 (bottom)	PCR	Ijamsville-Marburg
Poolesville	1	MO-70-0014	N/A (Abdn)	7	597	63	167	86, 155	Unknown	ML	New Oxford
	2	MO-70-0046	MO1970G007	6	600	63	210	224	184 (bottom)	ML	New Oxford
	4	MO-73-1584	MO1970G007	8	600	63	280	141,228,240,276	95 (bottom)	ML	New Oxford
	5	MO-73-2905	MO1970G107	8	500	104	400	345	>145	ML	New Oxford
	6	MO-81-0765	MO1970G007	8	500	105	370	180, 290	Unknown	ML	New Oxford
	7	MO-88-2384	MO1970G207	8	700	63	400	430	171-233	ML	New Oxford
	8	MO-93-0007	MO1970G007	8	500	66	375	217	>200	ML	New Oxford
	9	MO-94-1881	MO1970G207	8	800	105	310	310,435,640,680	230	ML	New Oxford
	10	MO-94-1848	MO1970G207	8	762	75	325	550, 600	320	ML	New Oxford
	12	MO-94-3610	MO2004G006	8	466	60	200	140, 233	118	ML	New Oxford
Taneytown	10	CL-67-0338	N/A (Abdn)	12	500	131	NR	415	80-120,415-438	ML	New Oxford
	10R	CL-95-1690	CL2010G002	12	789	138	NR	280	80-111, 429-441	ML	New Oxford
	13	CL-81-1687	CL1978G379	8	581	164	345	325-495	131-164, 325-377	ML	New Oxford
	14	CL-88-1060	CL1978G179	8	615	61	NR	470, 595	270	ML	New Oxford
	17	CL-95-1336	CL2007G003	8	1040	60	NR	815-35, 1010-20	600	ML	New Oxford
Thurmont	8	FR-88-3686	FR1993G036	8	174	131	131	135, 144, 151	125-174	ML	Gettysburg Shale
	8A	FR-94-0911	FR1993G036	12	161	131	98	131-161	125-161	ML	Gettysburg Shale

Appendix A3. Summary of aquifer-test results.

Town/Site	Name/ No.	Fig.	Date	Rate	Duration	Window	Flow Model	SWL	Available Drawdown	s_{90}	Estimated Yield	Climatic Conditions	Production	Statistics		
														RSS	Mean	Transmissivity
Name		No.	mo-yr	gpm	hr	min		ft	ft	ft	gpm		gpm	ft^2	ft	gpd/ft - t-ratio
Emmitsburg	3		Nov-96	143/95	216	0-1278	IARF	8.5	57	134	60	Wet	35-65	Visual inspection - straight line		
Myersville	WTP		Oct-87	35	54	0-300	SVF	7.6	140	270	18	Dry	9-17	34.2	0.1	145 (Kx-4.6x10^{-4})
	CH 1		Sep-92	50-25	72	0-180	IARF	22	85	205	10	Dry	14-23	210	-0.02	161-52.7
	CH 2		Apr-93	30	24	0-120	IARF	6	130	490	8-30	Wet		35	0.02	68-17
	DW		Sep-92	25	24	0-1440	Leaky	25	160	112	25+	Avg	10-21	27	1.22	122-29
Musket Ridge GC	17		Feb-00	200/150	72	0-50	IARF	8	63	178	71	Avg	NR	Visual inspection - straight line		
Middletown	15		Feb-02	50	72.5	0-1500	Leaky	31	Unk	130	50	Avg	45-56	1133	1.53	304-48
	16		Oct-03	38	216	0-12960	2 Por	5.8	>90	103	>33	Dry	NR	131	0.28	477 (K-96)
Fountaindale	A		Mar-93	173	103	1300-6175	GRF	8.7	82	117	121	Avg	132	101	0.001	1781 (K-118)
	B		Mar-96	275	73	0-335	D-B	14.7	140	276	139	Wet	111	88	-0.28	899-13
Point of Rocks	M		Mar-99	50	168	6000-10080	IARF	38	Unk	118	27	Wet	31	Visual inspection - straight line		
	N		Apr-97	100	72	80-4320	SVF	28	8.2	30		Avg		Visual inspection - curved line		
Westminster	6		Sep-83	82.5	72	0-4230	Leaky	0	90	92	81	Dry	84.5	123	-0.03	560-63
	7		Aug-86	254	262	0-5300	D-B	6	26	35	189	Dry	105-191	46	0.19	26,080-12
	8		Aug-87	400	96	400-1500	IARF	4	37.4	71	211	Dry	225-80	Visual inspection - straight line		
	11		Jul-01	110	74	1500-4440	2 Por	9	58.5	53	121	Dry	88-94	4.5	0.008	1771 (K-480)
Mount Airy	6A		Apr-08	164	72	0-4320	SVF	11	53	131	66	Wet	60	125	0.66	1372 (Kx-6x10^{-4})
	11		Jul-09	75	72	1000-4320	SVF	29	38	76	>37.5	Dry	34	6.5	-0.00006	773 (Kx-66)
Poolesville	1		Sep-69	53	24	0-18	IARF	29	155	416	20	Avg	24-39	1539	7.81	424 (Kx-10 x10^{-8})
	2		Sep-69	95	24	0-642	Leaky	38	162	118	130	Avg	105	610	0.68	789-111
	4		Jun-77	48	168	6000-10080	IARF	28	112	166	32	Avg	33	1419	-0.16	490-91
	5		Mar-80	120	24	0-1440	Leaky	26	>108	108	>100	Wet	100	78	1.13	1311-65
	6		Jun-85	225	72	100-2300	IARF	16	164	220	168	Avg	185	Visual inspection - straight line		
	7		May-92	50	72	0-2040	Leaky	3	<230	<230	<50 (45)	Avg	42	41	0.12	134-22
	8		Apr-94	80	74	0-4320	Leaky	18	165	165	80	Wet	61-71	268	-0.005	540-35
	9		Jun-01	225	72	0-600	Leaky	42	189	280	152	Avg	181	154	0.086	1243-469
	10		May-01	80	72	0-3600	Leaky	26	<317	<317	<80(70-75)	Avg		225	0.401	171-167
	12		Oct-05	175-130	48	6-200	IARF	25	96	180	94	Dry	55	Visual inspection - straight line		
Taneytown	10		Dec-99	350-165	100	0-840	Leaky	149	70	430	69	Avg	NR	509	0.011	583-84
	10R		Oct-10	275	72	0-4352	2 Por	29	120	330	100	Dry	NR	15,920	0.018	703 (K-6965)
	13		May-85	85-584	49	0-1330	2 Por	84	46	95	177	Avg	125	Deconvoluted-specific capacity		
	14		Jul-90	200+/-	72	210-1000	SVF	15.5	123-265	329	75-161	Dry	75-97	26	0.024	1648 (Kx-1807)
	17		Jun-09	250	72	300-4320	SVF	41	620	830	187	Avg	24	8.2x10^4	0.007	439 (Kx-6828)
Thurmont	8		Sep-94	325	230	0-720	IARF	41	38	100	126	Dry	158	Visual inspection - straight line		
	8A		Jun-98	325	24	0-1440	IARF	42	40-93	89	152	Avg	158	Visual inspection - straight line		

Appendix A4. Summary of step-drawdown test results.

Test Design Factors							Statistics																		
Site Name	Well No./Name	Fig. No.	Steps Number	Steps Analyzed	Duration min	Window Min	Rates gal/min	Model	RSS ft^2	Mean ft	T gpd/ft	t-ratio	B min/ft^2	C min^2/ft^5	P	Multi Rate	W.E. % English	W.E. % S.I.	Last Step gpm	Last Step ft^3/min	Well Loss ft	Max D/D ft (corr)	Well Loss / Max D/D %	Est. Yield gpm	W.E. %
Emmitsburg	3	To be provided upon completion of reviews	2	2	60		55-101	D-B	2.1	-9E-04	9,672	141	0.71	0.0006	2.9	No	88	95	101	11.8	0.8	17.4	4.6	60	95
Corrected (b = 57 ft)			2	2	60			D-B	1.13	0.009	14,830	113	0.76	0	2.0	No	100	100	101	13.5	0.0	14.7	0.0	60	100
Westminster	8 (1st)		7	7	365		94-467	D-B	4.6	-0.06	31,570	49	0.17	0.03	1.5	No	40	65	467	62.4	14.8	35.5	41.7	211	52
Corrected (b = 43 ft)			7	4	365	0-240	94-253	D-B	0.67	-0.007	24,160	43	0.19	0.0066	1.5	No	84	79	253	33.8	1.3	13.7	9.5	211	84
	8 (2nd)		7	7	360		30-270	D-B	41	-0.009	12,080	25	0.37	0.059	2.0	No	15	68	270	36.1	76.9	104.7	73.4	211	18
Corrected (b = 36 ft)			7	4	360	0-240		D-B	9.8	-0.06	9,805	18	0.462	0.046	1.5	No	71	85	120	16.0	2.9	16.5	17.6	211	65
Poolesville	7		5	5	450		20-60	D-B	8771	-0.55	88	24	1.18	0.142	3.0	No	11	99	60	8.0	72.7	256.6	28.3	45	11
Corrected (b = 289 ft)			5	5	450		20-60	D-B	554	0.20	172	37	1.43	0.46	1.5	No	45	100	60	8.0	10.4	142.6	7.3	45	56
Taneytown	13		7	7	2925		85-584	D-B	5984	-0.45	6128	12	0.35	0.01	2.1	No	22	100	584	78.1	94.3	184	51.3	177	52
Corrected (b = 152 ft)			7	4	2925		85-390	D-B	103	-0.005	7456	78	0.53	$5x10^{-5}$	3.0	No	79	81	260	34.8	5.1	152	3.4	177	95
Corrected (b = 164 ft)			7	3	2925	0-355	85-260	D-B	100	-0.116	7240	33	0.54	0.0005	2.2	No	94	100	260	34.8	1.2	45.1	2.7	177	96
Thurmont	8		4	4	270		200-450	D-B	3.7	-0.003	1141	6	0.04	0.002	1.6	No	60	56	450	60.2	1.4	31.9	4.4	126	79
Piscopo & Summa			Many		810	0-350	73-130	D-B	1780	0.93	501	45	-0.41	0.121	1.5	No	-1.#I0	100	B is negative (English) deviates from type curve at 140 ft (167 ft - 51 m S.I.)						
Karoo			4	4	120		9.7-55.5	D-B	6.2	0.013	106	62	0.31	0.010	2.6	No	58	62	55.5	7.4	1.8	21.1	8.5	11	94
Musket Ridge GC	17	To be provided upon completion of reviews	7	7	285	0-215	63-208	D-B	21	0.004	6039	25	0.77	0.001	3	No	41	N/A	190	25.4	16.38	65	25.2	70	90
Fountaindale	A		5	5	1680		39-190	D-B	144	-0.034	4060	24	0.73	0.006	2.5	No	46	N/A	190	25.4	19.50	81.1	24.0	126	64
	B		6	4	4545	0-355	181-295	D-B	366	-0.03	1419	23	0.46	0.065	1.6	No	48	N/A	295	39.4	23.20	140.3	16.5	111	58
Middletown	15		5	5	400		75-50	H-J	2672	-0.17	429	52	1.29	0.002	3	Yes	87	N/A	75	10.0	2.00	99.7	2.0	50	94
Pre-Frack	16		6	6	414		60-13.5	H-J	4443	-0.238	248	41	0.84	0.080	2.1	Yes	50	N/A	60	8.0	4.16	56.8	7.3	21	80
Post-Frack	16		3	3	558		45-25	H-J	725	-0.13	281	53	1.46	0.064	2.0	Yes	79	N/A	45	6.0	2.32	88.1	2.6	26	87
Mount Airy	11		4	2	360	0-128	150-200	D-B	336	-0.21	8944	21	0.78	0.0015	3.0	Yes	56	N/A	200	26.7	28.66	69.7	41.1	38	67
Westminster	6		4	4	390		19.5-81.8	D-B	26	-0.003	3673	15	1.11	0.953	1.6	No	23	N/A	81.8	10.9	43.77	72.0	60.8	63	24
	7		5	4	15,750	0-5490	105-387	D-B	48	-0.003	25,770	31	0.2	0	1.9	No	100	N/A	387	51.7	0.00	47.8	0.0	189	100
	11		2	2	270		130-260	D-B	286	-0.493	7,798	8	0.57	0.006	2.2	Yes	76	N/A	260	34.8	14.78	56.5	26.2	126	76
Poolesville	8		3	3	270		60-80	H-J	650	0.6	618	43	5.60	0	2.0	No	100	N/A	80	10.7	0.00	142	0.0	80	60
	9		4	3	400	0-255	150-250	H-J	3355	-1.23	1039	32	0.94	0.0008	3.0	Yes	74	N/A	250	33.4	29.81	157	19.0	152	74
	10		4	2	555	0-300	75-90	H-J	6321	0.326	157	25	0.46	0.157	1.5	Yes	48	N/A	140	18.7	12.71	519	2.4	75	48
	12		4	2	300	10-240	100-175	D-B	265	0.51	1624	35	1.34	0.062	1.5	No	80	N/A	175	23.4	7.01	241.3	2.9	94	86
Taneytown	10		7	7	420		43-102.5	D-B	136	0.1	1242	90	0.64	0.09	1.9	No	29	N/A	155	20.7	8.23	69.1	11.9	71	46
	10		7	4	420	0-240	43-102.5	D-B	9.4	0.009	1384	309	0.76	0.02	2.3	No	56	N/A	102.5	13.7	8.23	69.1	11.9	71	67
	10R		7	7	540		100-600	D-B	3750	-0.17	2013	187	0.30	0.006	2.0	No	78	N/A	600	80.2	38.6	154	25.1	183	67
	14		4	4	1460		24-95	D-B	7770	0.65	839	12	1.13	0.68	1.7	Yes	24	N/A	95	12.7	51.2	141	36.3	160	16
	14		4	4	1460	0-400	24-95	D-B	311	0.04	1096	31	1.65	0.13	2.2	No	37	NA	95	12.7	34.9	141	24.8	160	24
Corrected (b = 270 ft)	14		4	4	1460		24-95	D-B	5806	0.37	1850	14	1.91	0.38	1.5	Yes	58	N/A	95	12.7	17.2	104	16.5	160	52
Corrected (b = 270 ft)	14		4	4	1460	0-400	24-95	D-B	92	-0.028	1878	93	1.90	0.37	1.5	No	57	N/A	95	12.7	16.7	104	16.1	160	53
	17		5	5	383		100-290	H-J	$4x10^5$	-7.62	75	71	0.018	0.003	3.0	Yes	3	98	290	38.8	175	547	32.0	187	1
corr (b = 600 ft)	17		5	5	383		100-290	H-J	$5x10^4$	-1.48	225	26	0.25	0.001	3.0	Yes	59	100	290	38.8	58.40	297.6	19.6	187	29
corr (b = 400 ft)	17		5	3	270		100-250	H-J	9612	-0.073	36	13	0.577	0.0063	2.1	Yes	84	100	250	33.4	9.80	95.9	10.2	187	73
corr (b = 400 ft)	17		5	1	270	0-90	100-250	H-J	513	-0.056	72	24	1.39	0	1.5	Yes	100	100	100	13.4	0.00	66.2	0.0	187	100

Appendix A5. Fredrick City STELLA model.

This evaluation consists of an evaluation estimated seasonal water use for the City of Frederick based on the water use input parameters for the City's STELLA (mass flow) model, Malcolm Pirnie (2004). The initial model input for the maximum monthly to annual average demand ratio was 1.1:1.0, but this appeared to be too low, because it reflected data which were averaged over several years, during which water use had been suppressed by restrictions and wet to average summers.

MDE studies of municipal water supplies in Maryland had indicated that the typical maximum monthly to annual average municipal demand ratio falls within the range of 1.3-1.5 to 1, during a moderately severe drought, with no restrictions in place. Monthly ratios developed from those studies were provided to the city, to run a separate scenario to show the effects of unrestricted water use during a severe drought. This reduced the estimated sustained yield of the system by 16 percent from 9.63 mgd avg to 8.13 mgd avg. This indicated that water restrictions increased the annual average water supply availability, because water from storage was not required to meet the normal peak monthly summertime demand.

Reference

Malcolm Pirnie Inc., 2004. Water Supply Benefits of Fort Detrick Interconnection. Frederick, Maryland, U.S.A.

Appendix A6. Municipal and community seasonal water use.

Based on a USGS study, previously DNR and now MDE applied 80 gpdpc for water use at residential subdivisions. One-third of that water was for outdoor use, indicating that it was a drought year rate. The equivalent maximum monthly use was 1.67 times the average, or 134 gpdpc. For municipal water systems per capita use is variable, but typically is about 100 gpd(avg)pc (80% residential and 20% non-residential uses). In the early 1990's, it was proposed that the 1.67 ratio be applied to municipal permits. Work completed in southern and central Maryland indicated that, while such a ratio was appropriate for subdivisions served by community supplies or individual wells, municipal systems typically had max:avg ratios of 1.3-1.5 to 1.

To determine water allocations, it has been proposed that municipal demand be based solely on past water use data, even when such data may have been biased by suppressed water demand, due to water restrictions or excessive leaks, both which can produce low maximum month:annual average water use ratios.

A review of the water use data contained in the following tables indicates that systems which have or have had adequate water supplies can fit into four different categories. The first is community water supplies for residential subdivisions, located primarily in Frederick County. These systems have ratios that can exceed 2 to 1 during build-out, which then settle in at about 1.5 to 1 once most of the lawns have been established. The second category is municipal systems that supply water mostly to residential customers. Water use from eight of those systems (Hagerstown before 1987, Aberdeen, Taneytown, Walkersville, Middletown, Fountaindale, Poolesville, and Rockville) has been tracked over the years and those municipalities have ratios of 1.31 to 1.56 to 1 (unweighted average of 1.43 to 1). Third are generally larger municipalities, such as Hagerstown (after 1987), Frederick and Westminster, which have ratios of 1.2-1.3 to 1, due to the high percentage (about 30 to 50%) of non-residential uses that they supply. Finally, 76% of the water supplied by WSSC serves residential customers, but the max:avg ratio is a relatively low 1.25 to 1. A review of census data indicates that the typical population density of the communities served by WSSC system is about 7 people per acre or about twice the density of typical municipalities in the outer suburban areas.

Appendix A6. Continued.

[mo; month; max, maximum; avg, average; est, Estate; Mgd, million gallons per day; red numbers are years with high (1.3-1.5:1) max/avg ratios]

	Cloverhill III	Cloverhill III	Ratio	Knolls/ Windsor	Knolls/ Windsor	Ratio	Sam Hill Est	Sam Hill Est	Ratio	Waterside	Waterside	Ratio	Copperfield	Copperfield	Ratio
	Avg	Max Mo	Max/Avg	Avg	Max Mo	Max/Avg	Avg	Max Mo	Max/Avg	Avg	Max Mo	Max/Avg	Avg	Max Mo	Max/Avg
1979															
1980															
1981															
1982															
1983															
1984															
1985															
1986															
1987															
1988	0.007	0.018	2.57												
1989	0.016	0.029	1.81												
1990	0.025	0.043	1.72							0.009	0.011	1.28			
1991	0.035	0.068	1.94							0.014	0.029	2.04	0.005	0.011	2.20
1992	0.035	0.062	1.77							0.013	0.017	1.31	0.006	0.009	1.50
1993	0.045	0.093	2.07							0.017	0.027	1.60	0.010	0.019	1.90
1994	0.045	0.070	1.56	0.013	0.022	1.69				0.020	0.027	1.35	0.013	0.021	1.62
1995	0.059	0.114	1.93	0.023	0.053	2.30				0.026	0.033	1.30	0.016	0.025	1.56
1996	0.051	0.055	1.08	0.029	0.039	1.34				0.020	0.022	1.10	0.016	0.019	1.19
1997	0.064	0.101	1.58	0.038	0.070	1.84	0.020	0.033	1.65	0.028	0.045	1.60	0.022	0.029	1.32
1998	0.070	0.099	1.41	0.058	0.115	1.98	0.038	0.056	1.47	0.034	0.047	1.39			
1999	0.074	0.111	1.50	0.056	0.083	1.48	0.047	0.074	1.57	0.035	0.052	1.48	0.026	0.037	1.42
2000	0.072	0.083	1.15	0.052	0.063	1.21	0.074	0.159	2.15	0.037	0.043	1.15	0.024	0.027	1.13
2001	0.081	0.101	1.25	0.060	0.077	1.28	0.065	0.086	1.32	0.047	0.060	1.28	0.026	0.031	1.19
2002	0.065	0.070	1.08	0.052	0.057	1.10	0.063	0.074	1.17	0.050	0.059	1.18	0.027	0.031	1.15
2003	0.067	0.076	1.13	0.056	0.065	1.16	0.067	0.079	1.18	0.051	0.061	1.18	0.030	0.033	1.10
2004	0.068	0.073	1.07	0.055	0.059	1.07	0.077	0.094	1.22	0.057	0.065	1.13	0.030	0.034	1.13
2005	0.071	0.083	1.17	0.071	0.092	1.30	0.073	0.083	1.14	0.063	0.076	1.21	0.028	0.033	1.18
2006	0.069	0.093	1.35	0.058	0.078	1.34	0.073	0.096	1.32	0.064	0.085	1.34	0.028	0.034	1.21
2007	0.071	0.093	1.31	0.071	0.110	1.55	0.079	0.093	1.18	0.062	0.081	1.29	0.032	0.037	1.16

Appendix A6. Continued.

	Middletown Avg	Middletown Max Mo	Ratio Max/Avg	Fountaindale Avg	Fountaindale Max Mo	Ratio Max/Avg	Taneytown Avg	Taneytown Max Mo	Ratio Max/Avg	Hampstead Avg	Hampstead Max Mo	Ratio Max/Avg	Poolesville Avg	Poolesville Max Mo	Ratio Max/Avg
1979	0.135	0.161	1.19	0.106	0.130	1.23	0.32	0.34	1.06	0.11	0.12	1.13	0.249	0.274	1.10
1980	0.184	0.273	**1.48**	0.117	0.164	**1.40**	0.33	0.36	1.10	0.12	0.14	1.16	0.237	0.254	1.07
1981	0.222	0.322	**1.45**				0.28	0.32	1.15	0.13	0.16	1.19	0.255	0.289	1.13
1982				0.112	0.126	1.13	0.29	0.32	1.11	0.14	0.18	1.22	0.256	0.286	1.12
1983	0.131	0.180	**1.37**				0.26	0.28	1.10	0.15	0.17	1.19			
1984	0.135	0.166	1.23	0.133	0.146	1.10	0.28	0.30	1.07	0.16	0.18	1.16	0.276	0.310	1.12
1985	0.125	0.155	1.24	0.133	0.142	1.07	0.29	0.33	1.11	0.18	0.20	1.14	0.282	0.306	1.09
1986	0.131	0.167	1.27	0.146	0.161	1.10	0.30	0.34	1.13	0.18	0.21	1.19	0.294	0.378	1.29
1987	0.152	0.194	1.28	0.164	0.178	1.09	0.30	0.34	1.16	0.18	0.20	1.12	0.283	0.332	1.17
1988	0.152	0.194	1.28	0.157	0.188	1.20	0.32	0.36	1.12	0.21	0.24	1.17	0.306	0.394	1.29
1989	0.148	0.164	1.11	0.141	0.162	1.15	0.39	0.49	1.26	0.22	0.24	1.11	0.337	0.376	1.12
1990	0.146	0.176	1.21	0.147	0.164	1.12	0.35	0.40	1.15	0.23	0.25	1.08	0.351	0.406	1.16
1991	0.151	0.178	1.18	0.162	0.190	1.17	0.38	0.43	1.13	0.28	0.34	1.21	0.433	0.540	1.25
1992	0.147	0.170	1.16	0.157	0.165	1.05	0.37	0.43	1.17	0.26	0.28	1.07	0.438	0.487	1.11
1993	0.160	0.201	1.26	0.159	0.197	1.24	0.39	0.42	1.09	0.30	0.33	1.10	0.421	0.538	1.28
1994	0.192	0.247	1.29	0.143	0.165	1.15	0.44	0.53	1.20	0.35	0.39	1.11	0.445	0.531	1.19
1995				0.157	0.182	1.16	0.39	0.43	1.11	0.36	0.40	1.11	0.424	0.496	1.17
1996	0.184	0.200	1.09	0.166	0.212	1.28	0.41	0.45	1.11	0.37	0.38	1.04	0.420	0.498	1.19
1997	0.183	0.219	1.20	0.207	0.242	1.17	0.40	0.48	1.19	0.37	0.42	1.15	0.420	0.493	1.17
1998	0.188	0.267	**1.42**	0.189	0.209	1.11	0.39	0.42	1.08	0.37	0.39	1.06	0.453	0.544	1.20
1999	0.226	0.287	1.27	0.189	0.214	1.13	0.41	0.46	1.11	0.37	0.40	1.09	0.419	0.539	1.29
2000	0.230	0.261	1.13	0.163	0.213	**1.31**	0.42	0.47	1.12	0.38	0.41	1.08	0.477	0.603	1.26
2001	0.241	0.278	1.15	0.183	0.267	**1.46**	0.40	0.44	1.12				0.430	0.540	1.26
2002	0.222	0.264	1.19	0.286	0.363	1.27	0.38	0.41	1.07	0.39	0.42	1.05	0.395	0.419	1.06
2003	0.239	0.279	1.17	0.306	0.340	1.11	0.40	0.42	1.06	0.41	0.44	1.06	0.378	0.434	1.15
2004	0.282	0.321	1.14	0.273	0.327	1.20	0.48	0.53	1.10	0.42	0.45	1.06	0.418	0.467	1.12
2005	0.324	0.376	1.16	0.237	0.292	1.23	0.47	0.52	1.10	0.44	0.48	1.07	0.418	0.466	1.11
2006	0.328	0.406	1.24	0.204	0.240	1.18	0.48	0.58	1.23	0.43	0.46	1.07	0.410	0.512	1.25
2007	0.314	0.383	1.22	0.208	0.246	1.18	0.51	0.61	1.20	0.46	0.50	1.08	0.466	0.643	**1.38**

Appendix A6. Continued.

	Walkersville	Walkersville	Ratio	Rockville	Rockville	Ratio	Aberdeen	Aberdeen	Ratio	Hagerstow n	Hagerstow n	Ratio	WSSC (avg)		WSSC Total	WSSC Total	Ratio
	Avg	Max Mo	Max/Avg	Avg	Max Mo	Max/Avg	Avg	Max Mo	Max/Avg	Avg	Max Mo	Max/Avg	Potomac	Patuxent	avg	Max Mo	Max/Avg
1979	0.306	0.412	1.35	4.67	5.16	1.10	0.967	1.265	1.31	5.40	8.43	1.56	93.30	45.10	138.40	147.40	1.07
1980	0.443	0.558	1.26	4.78	5.76	1.21	0.983	1.294	1.32	6.54	8.63	1.32	104.96	41.95	146.91	170.18	1.16
1981	0.370	0.431	1.16	5.01	6.12	1.22	1.003	1.242	1.24	6.37	8.67	1.36	117.50	26.05	143.55	162.37	1.13
1982	0.335	0.422	1.26	4.94	6.02	1.22	0.967	1.212	1.25	6.02	8.37	1.39	N/A	30.27	N/A		
1983	0.382	0.561	1.47	5.03	6.99	1.39				6.19	9.07	1.47	112.19	35.93	148.12	176.44	1.19
1984	0.412	0.462	1.12	5.24	6.44	1.23	1.163	1.273	1.09	6.53	8.42	1.29	109.82	35.57	145.39	158.42	1.09
1985	0.445	0.528	1.19	5.09	5.99	1.18	1.140	1.230	1.08	6.99	9.17	1.31	113.85	39.65	153.50	170.86	1.11
1986	0.554	0.680	1.23	5.15	6.98	1.36	1.218	1.394	1.14	7.95	10.44	1.31	130.40	33.11	163.51	202.38	1.24
1987	0.561	0.684	1.22	5.34	6.93	1.30	1.132	1.360	1.20	9.56	11.99	1.25	132.28	35.94	168.22	204.62	1.22
1988	0.615	0.724	1.18	5.05	6.31	1.25	1.068	1.300	1.22	10.53	12.33	1.17	142.57	34.01	176.58	219.29	1.24
1989	0.639	0.693	1.08	5.23	6.29	1.20	1.256	1.350	1.07	10.00	10.88	1.09	133.41	38.37	171.78	192.96	1.12
1990	0.648	0.738	1.14	5.31	6.17	1.16	1.289	1.451	1.13	9.89	10.69	1.08	140.09	34.30	174.39	197.85	1.13
1991	0.668	0.825	1.24	5.01	6.17	1.23	1.285	1.469	1.14	9.98	11.37	1.14	140.08	40.13	180.21	209.62	1.16
1992	0.669	0.722	1.08	5.14	6.18	1.20	1.281	1.404	1.10	9.88	10.38	1.05	123.65	48.53	172.18	194.17	1.13
1993	0.705	0.812	1.15	5.15	7.16	1.39	1.291	1.438	1.11	9.95	11.25	1.13	116.62	57.98	174.60	218.92	1.25
1994	0.760	0.844	1.11	5.33	6.46	1.21	1.332	1.463	1.10	10.07	11.10	1.10	125.57	56.18	181.75	200.96	1.11
1995	0.753	0.929	1.23	5.06	6.10	1.21	1.214	1.360	1.12	10.29	11.64	1.13	128.67	45.03	173.70	207.73	1.20
1996	0.712	0.762	1.07	4.24	4.84	1.14	1.121	1.23	1.10	10.28	11.15	1.08	124.55	45.58	170.13	178.87	1.05
1997	0.737	0.854	1.16	4.97	6.98	1.40	1.192	1.307	1.10	9.79	10.96	1.12	122.98	48.81	171.79	205.05	1.19
1998	0.755	0.875	1.16	5.36	6.47	1.21	1.311	1.594	1.22	9.05	10.20	1.13	116.96	52.86	169.82	205.80	1.21
1999				4.95	5.80	1.17	1.438	1.767	1.23	10.04	11.76	1.17	140.14	36.41	176.55	218.63	1.24
2000	0.716	0.763	1.07	4.61	5.33	1.16	1.421	1.585	1.12	10.53	11.18	1.06	120.43	49.73	170.16	181.48	1.07
2001	0.746	0.887	1.19	4.66	5.28	1.13	1.338	1.496	1.12	10.55	11.79	1.12	130.32	43.07	173.39	190.44	1.10
2002	0.576	0.708	1.23	4.93	6.36	1.29	1.163	1.617	1.39	10.68	11.53	1.08	139.10	28.16	167.26	194.93	1.17
2003	0.552	0.586	1.06	5.03	5.60	1.11	1.148	1.336	1.16	10.68	11.54	1.08	126.58	38.57	165.15	175.33	1.06
2004	0.589	0.635	1.08	4.11	5.39	1.31	1.249	1.391	1.11	10.54	11.32	1.07	130.21	37.37	167.58	178.27	1.06
2005	0.573	0.607	1.06	4.69	5.90	1.26	1.198	1.409	1.18	11.08	12.07	1.09	124.92	44.81	169.73	196.43	1.16
2006	0.570	0.714	1.25	4.98	6.67	1.34	1.196	1.400	1.17	10.79	12.37	1.15	114.97	53.97	168.94	207.01	1.23
2007	0.567	0.668	1.18	5.16	6.49	1.26	1.239	1.594	1.29	10.94	12.15	1.11	133.99	39.24	173.23	202.78	1.17

Appendix A7. Synthetic model examples.

The results of methods used in the present study were compared to those achieved using the calibrated 3D numerical model developed by van Tonder et al. (2001a) to verify the estimated sustainable yield of a fractured rock well (UO5). The geometry of the model was based on the hydraulic characteristics of the Karoo aquifer at the Campus Test Site, which were derived from data collected during an extensive field investigation. The horizontal extent of the model, 1640.5 ft × 1640.5 ft (500 m × 500 m), was chosen to be large enough to avoid boundary effects. The horizontal dimension of the fracture was 394 ft × 394 ft (120 m × 120 m), on which the pumping well and three observations (UO6, UO15 and UO16) were placed. A recharge rate of 0.59 in/y (15 mm/y) and a pumping period of one year were applied, producing a sustainable yield of 6.3 gpm (0.4 L/s). The result of the one-year numerical simulation was like the 5.7 gpm (0.36 L/s) estimated by the FC-method, except in the case of the FC-method, the pumping period was two years. An estimated yield of 7.9 gpm (0.5 L/s) was derived from an analysis of a follow-on step-test of UO16 (van Tonder et al., 2001b).

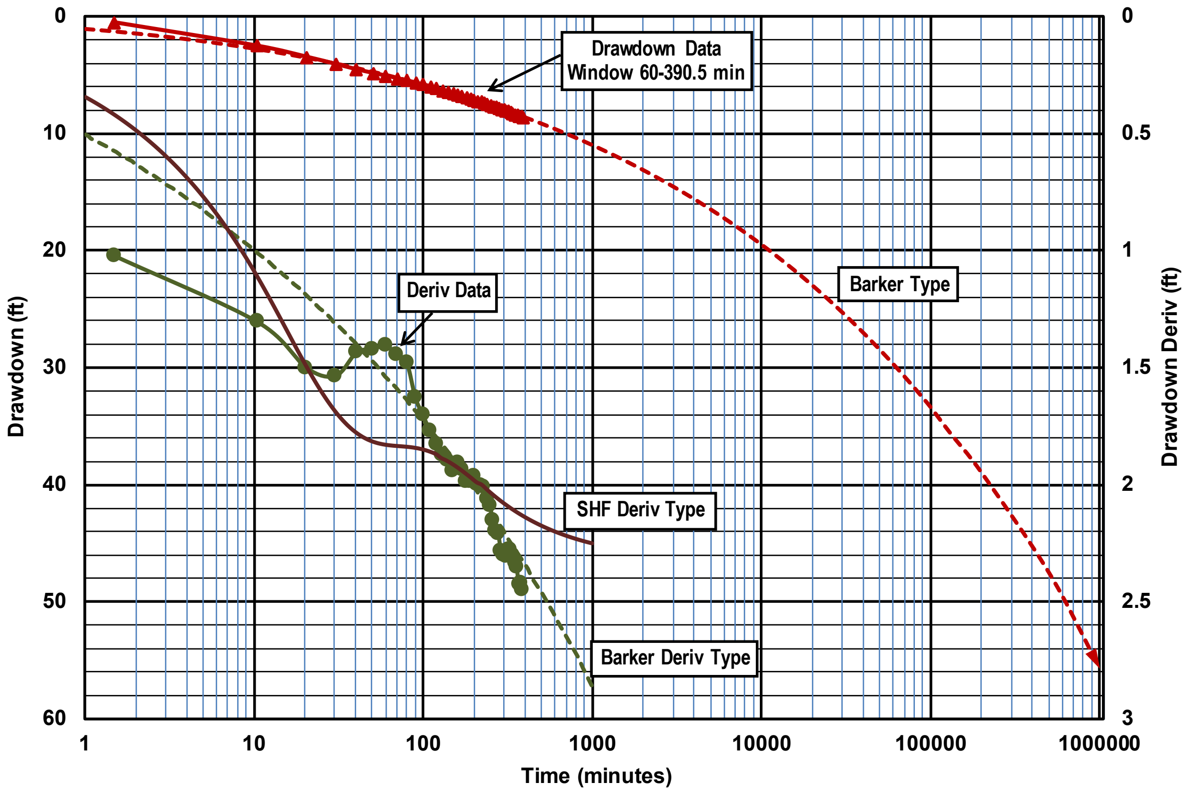

Figure A1. **Karoo aquifer well UO5 – Semi-log plot of drawdown and its logarithmic derivative for a 390.5-min, 20 gpm pumping test, with Barker general radial flow (GRF) drawdown type curve, and GRF and single horizontal fracture (SHF) derivative type curves.**

Using the Barker (1988) generalized, single and double porosity, radial flow type curve, drawdowns from the Karoo pumping test were extrapolated in this study (fig. A1) to 90 days, 200 days, 1 year, and 2 years, reflecting pumping periods used in this study, by Misstear and Beeson (2000) and by van Tonder et al. (2001a) for the Karoo model and FC-method, respectively. The calculated specific capacity at the end of each pumping period was applied to the available drawdown of 19.7 ft (6 m), producing yields of 10.9 gpm, 9.2 gpm, 8.1 gpm, and 7.0 gpm (0.69 L/s, 0.58 L/s, 0.51 L/s, and 0.44 L/s), respectively. Typical ratios for municipal water demand in Maryland are 1.4:1 for maximum monthly use and 1.15: 1 for summertime seasonal water demands relative to annual average use. These ratios are based on the results of studies completed by MDE (Appendix A6: Municipal Water Use), which are consistent with those contained in various water supply engineering studies such as the American Water Works Association's Manual M1 (AWWA 2000; Billings and Jones 2008). When these ratios were applied to the seasonal periods, the estimated equivalent yearly average yields are both 7.9 gpm (0.5 L/s), while the long-term average yields are 8.1 gpm and 7.0 gpm (0.51 L/s and 0.44 L/s), demonstrating that similar results were achieved when seasonal variations in water demand were considered.

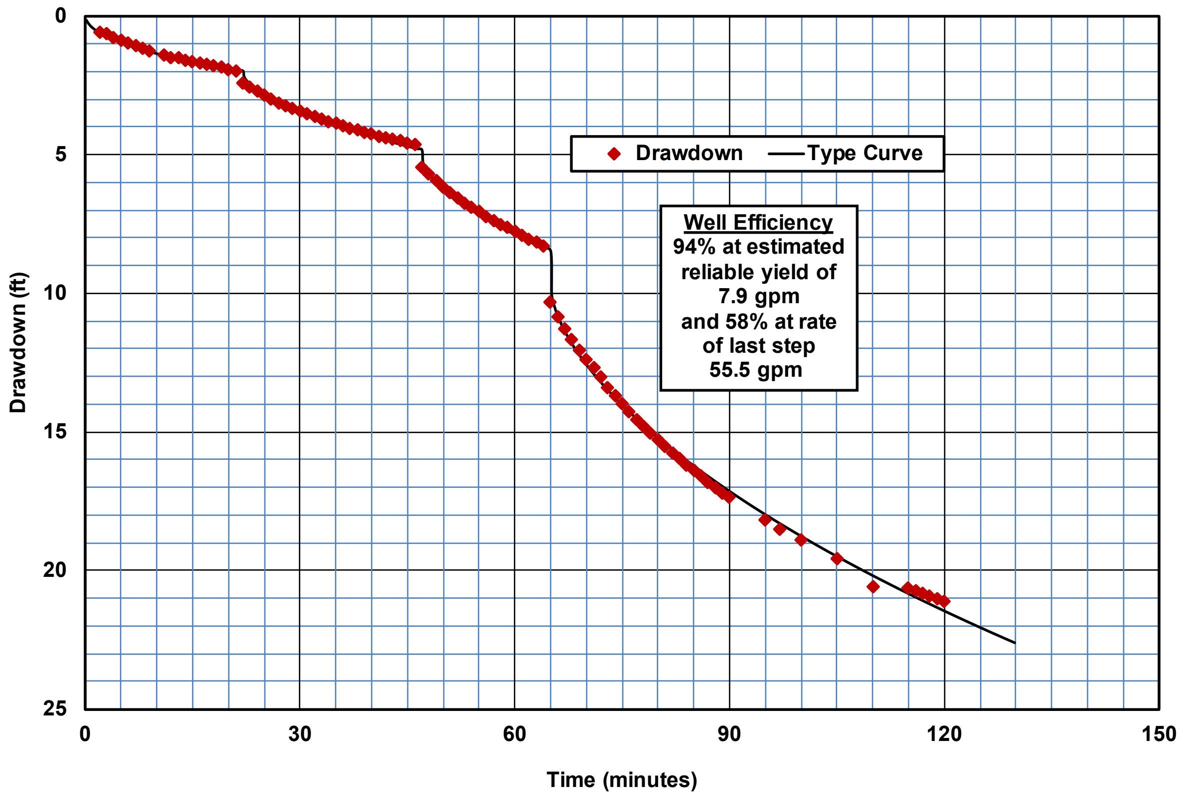

Figure A2. Karoo aquifer well UP16 – Semi-log plot of drawdown from step-drawdown test, Dougherty-Babu double porosity solution.

The data from the step-test of UP16 were analyzed in the present study (fig. A2). It was found that the Dougherty-Babu double porosity model provided the best fit to the data and the resulting type curve was a nearly identical match to the that of the van Tonder et al. (2001b) study (shown in their Figure 9). At the reliable estimated yield of 7.9 gpm (0.5 L/s) the calculated well efficiency was 94% and at the final step of 55.5 gpm (3.5 L/s) it was 58%. These tests indicate that the Karoo aquifer at the site is confined, and leakage and/or dewatering of permeable zones did not occur during the test. There were no operational data presented in the South African studies that could confirm the validity of the numerical model.

Piscopo and Summa (2007) analyzed the drawdown data from a 120 gpm (7.6×10^{-3} m^3/s), 240-min aquifer pumping test (their Figure 4) by applying the straight-line Cooper-Jacob method to both early-time and late-time segments. They indicated that the well was completed in a confined aquifer and attributed the difference between the two transmissivities that were derived to aquifer heterogeneity. Their data were re-analyzed in the present study using derivative and type curve methods. The results are shown in Figure 6, indicating that the observed drawdown was caused by a leaky aquifer response and that there was no evidence of an external boundary.

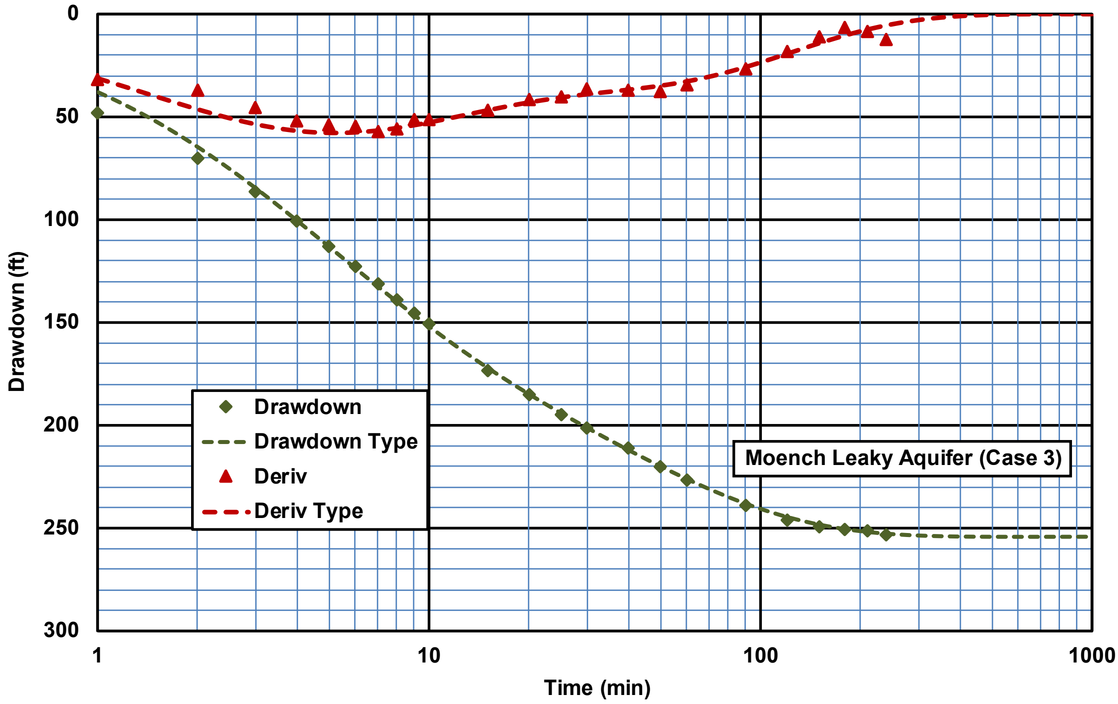

Figure A3. Piscopo and Summa well – Semi-log plot of drawdown and its logarithmic derivative from a 240-min, 120 gpm, Moench leaky aquifer solution.

From the results of the step-test, Piscopo and Summa (2007) calculated that for a drawdown of about 39.4 ft (12 m) the well's efficiency was greater than 75% and flow should essentially be laminar. With the drawdown of 12 m at that efficiency, their estimated sustained yield was 14.9 gpm (0.94×10^{-3} m^3/s). The discharge and drawdown graphs (fig. 6 in their paper) show that the discharge decreased from 50.7 to 34.9 gpm (3.2×10^{-3} to 2.2×10^{-3} m^3/s) and the average was 42.0 gpm (2.65×10^{-3} m^3/s) during a 104-day period (28 July to 8 November 2004) at a constant drawdown of 36.1 ft (11 m in fig. 6, but 12 m in their text). The step-drawdown test data contained in Summa (2010) were analyzed in the present study using the Dougherty-Babu model. On the resulting graph (fig. A4), a deviation from the type curve can be seen starting at a drawdown of about 140 ft (43 m) when U.S. English units are used and 167 ft (51 m) when The International System of Units (S.I.) are used, a response that was most likely due to dewatering of a permeable zone. It is not entirely clear as to why this is the case, but it may simply be an artifact related to one or both of the following factors: 1. the nonlinear coefficient (C) was derived empirically and has peculiar units (T^2/L^5), and, more likely, 2. Step-drawdown test equations do not include a factor for nonlinear losses due to aquifer dewatering effects. In this case, the best fit was obtained using the actual well bore radius of 0.08 m (0.26 ft), while simulations at larger well radii, to approximate effective fracture lengths, produced increasingly poor results. The calculated well efficiency prior to dewatering of that zone is 100% when S.I. units were used and cannot be calculated when U.S. English units are used because *BQ* has a negative value. This is compared to the 55% efficiency using the Summa (2010) method, using S.I. units. These results suggest that the available drawdown in the well is three to four times greater than indicated in the Piscopo and Summa (2007) paper.

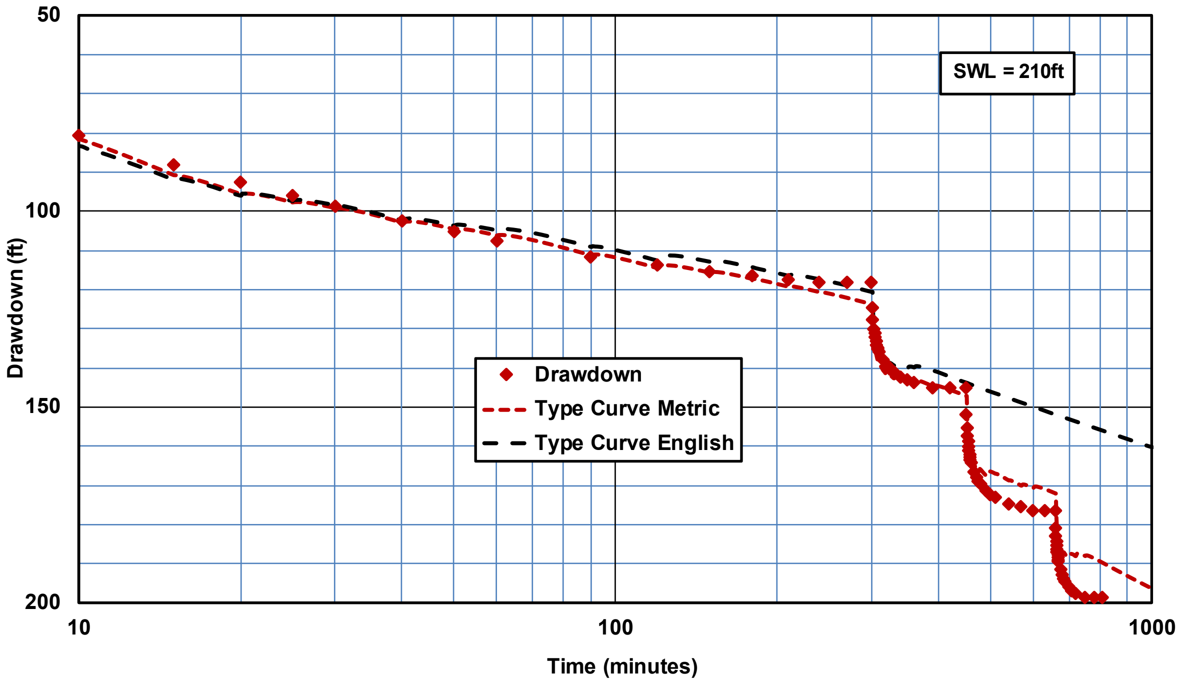

Figure A4. Piscopo and Summa well – Semi-log plot of drawdown and its logarithmic derivative from a variable rate, constant (pump) speed step test, and showing deviations from type curves at drawdowns of 140 ft (43 m S.I. units) and 167 ft (51 m S.I. units).

Besides the 104-day period, there were several other periods of about 2−4 weeks where the pumping rates and drawdowns remained constant. Three of these had similar drawdowns and discharge rates. They were: March 2004 (Q = 53.9 gpm, s = 45.9 ft); August 2004 (Q = 47.6 gpm, s = 36.1 ft) and April 2005 (Q = 68.2 gpm, s = 55.8 ft). The average specific capacity and drawdown for those periods were 1.27 gal-ft/min and 45.9 ft, respectively. In June 2004, the discharge rate was 82.4 gpm, and the drawdown was 95.1 ft, producing a specific capacity of 0.87 gal-ft/min. These data are used in the present study to determine a revised well yield. The ratio of the products of the drawdowns at the two operating levels, 95.1 and 45.9 ft, and the estimated available drawdown, 140 ft, or (140 ft / 95.1 ft) / (95.1 ft / 45.9 ft) were multiplied by the difference, 0.40 gal-ft/min in specific capacities at s = 45.9 ft and s = 95.1 ft. The result was then subtracted from the specific capacity at s = 95.1 ft (0.87 gal-ft/min), producing an estimated specific capacity of 0.59 gal-ft/min at s = 140 ft. When that result was multiplied by the estimated available drawdown, 140 ft, the maximum estimated reliable yield was then 83 gpm. When the typical ratio of maximum to average municipal water demand of 1.4 to 1 was applied, the revised estimated average yield was 59 gpm (estimated $s \cong 52.5$ ft). While this is a rough approximation, it is four times greater than the estimated yield in the Piscopo and Summa (2007) paper of 14.9 gpm.

The well continued to be pumped under controlled conditions until February 2007 (Baiocchi et al., 2014). The well discharge varied from a maximum of 79 gpm in April−May to a minimum of 32 gpm in December during the period 2004 to 2006, while maintaining a constant drawdown of 39−49 feet. Baiocchi et al. (2014) constructed a 3D model consisting of a confining unit overlying a carbonate aquifer. During one simulation, a pumping rate of 87 gpm captured all the residual outflow during the 2005 depletion period after 214 days, with a drawdown of 154 feet. This is evidence of the maximum reliable yield of the well and agrees with the previous results of the analysis in the present study (83 gpm at 140 ft of drawdown).

The Misstear and Beeson (2000) estimated well yield calculations were based on the Cooper-Jacob method, which assumed that the well was efficient and did not make separate allowances for the effects of well loss. The extrapolations (in their Table 1) for a single well were made by adding the drawdowns at 100 minutes from step-drawdown test data to additional drawdowns for 200 days of pumping, using a T value of 16,100 gal/d-ft (200 m^2/d). This produced a total drawdown and a pumping water level of 43.7 ft (13.3 m) relative to the local datum. Two-hundred days of continuous pumping were used to produce estimates for the operation of most of the production wells in their study during a dry summer, to meet the average water demand.

A T of 16,100 gal/d-ft (200 m^2/d) was derived from a 10-day pumping test conducted in 1981, while a T = 6,442 gal/d-ft (80 m^2/d) was derived from short-term recovery test in 1997. Misstear and Beeson (2000) indicated that it is unclear why there was such a difference, but they thought that it may be that the higher transmissivity value obtained from the longer test was more representative of regional aquifer conditions. Another explanation is that the decrease in transmissivity could be evidence that the aquifer may have been dewatered between those two periods.

Analytical models describe relatively simple geometries, and derivative analysis techniques can help identify specific flow regimes. More detailed characterizations of the groundwater flow systems are only possible using data-intensive, calibrated numerical models, commonly used to identify water supplies at risk for potential contamination. Many governing authorities in the United States require some estimate of well yields for water abstraction or use permits; however, due to the large number of wells that are tested, it is presently unlikely that they will require completion of expensive, calibrated groundwater flow models to obtain such permits. They may accept uncalibrated simulation models (such as the FC-method used in the Karoo study), with the attendant errors in estimating hydraulic parameters and recharge values. The methods developed in this study are practical, cost effective and have produced reliable results; thus, they are more likely to be accepted by governmental agencies for use in water-use-permitting activities.

Appendix A8. Seasonal variations in well yields: Results from previously published investigations.

Parizek and Siddiqui (1971) noted sharp declines of the water levels and decreased productivity in wells UN-17 and UN-24 (Parizek and Siddiqui, 1971, Figure 3), which they indicated were due to interference with other wells in the area (Parizek and Siddiqui, 1971, Figure 2) and the prolonged effects of drought. Production data from nearby wells or during wet periods were not given, so there was not enough information to separate the effects of drought from well interference.

Daniel (1990) provided daily average production rates for 12 wells during the period September 1983 through October 1984 (Daniel, 1990, Figure 18), while pumping Cary, North Carolina municipal wells constantly for 18 hours per day. The lowest production was 42,000 gal/d (159 m^3/d) per well in September 1983, while the highest use was 53,900 gal/d (204 m^3/d) per well in March 1984, or a difference of 11,900 gal/d (45 m^3/d) per well. In the present study, regional water levels in the Cary area were based on those taken in the USGS monitoring well OR-069 (NC-126), which is a regolith well. That well's water levels were 40 ft (12 m) in September 1983 and 36.4 ft (11.1 m) in March 1984, or a difference of 3.6 ft (1.1 m). The production change was 10,800 gal/d (40.9 m^3/d) per well per 3.3 ft (1.0 m) change in the water level of the monitoring well. The range of water levels in the monitoring well was 11.8 ft (3.6 m). The seasonal variation in yields was estimated by multiplying the unit change in production by the differences between the minimum and maximum ground water levels and the two reference levels. The result was a low estimated (dry) yield of 20,340 gal/d (77 m^3/d) per well and a high yield of 59,200 gal/d (224 m^3/d) per well, or a maximum (wet) yield that was 2.9 times the minimum (drought) yield.

Piscopo and Summa (2007) collected water level, discharge, and precipitation data for 13 months after the well in their study was placed in service (Piscopo and Summa, 2007, Figure 6). During the first five months of pumping, the discharge and drawdowns were highly variable. After that period, the drawdown was a constant 36 ft (11 m) for the next five months, with the discharge declining from 51 gpm to 30 gpm (3.2×10^{-3} m^3/s to 1.9×10^{-3} m^3/s). They related the decline in yield to summertime drought conditions. During the following three months, after a period of high fall rains, the discharge increased to 52 gpm (3.3×10^{-3} m^3/s). There was a lag of about four months between changes in precipitation relative to changes in well yields. During the last month of the monitoring period, the drawdown increased to 56 ft (17 m) and the yield increased to 68 gpm (4.3×10^{-3} m^3/s).

During the growing season of 2004 (April−September), the precipitation at the site was 11.4 in (289 mm) or an annualized 22.8 in (578 mm). During the following non-growing season (October 2004 – March 2005), the precipitation was 21.1 in (537 mm) or an annualized 42.3 in (1074 mm). Rossi and Silvagni (1980) determined that the average precipitation for the Tanagro River at the Polla station was 50.2 in/y (1275 mm/y) (1925−70). Polemio and Casarano (2008) indicated that the precipitation trend for the Campania region, in which the river and well are located, was −0.09 in/y (–2.39 mm/y) during the period 1924−2001. Applying that trend to the Tanagro River data would indicate that the long-term average precipitation for the basin would have been 47 in (1194 mm) by 2004, when the measured annual value was 32.5 in/y (826 mm/y), which are like average and severe drought conditions in Maryland.

When the late period yield is adjusted from a drawdown of 56 ft (17m) to 36 ft (11m), the estimated well yield would be 44 gpm (2.8×10^{-3} m^3/s). If the low summertime discharge of 30 gpm (1.9×10^{-3} m^3/s) is the minimum achievable yield and the ratios of the precipitation to discharge are representative of the seasonal variation in yield, then the maximum achievable yield would be 65 gpm (4.1×10^{-3} m^3/s) if the drawdown was limited to 36 ft (11m). In that case, the maximum estimated yield is 2.2 times the minimum estimated yield. Biaocchi et al. (2014) produced similar results showing a seasonal variation between 32 and 79 gpm (2 and 5×10^{-3} m^3/s), or a ratio of 1:2.5, in the yield of the well in Piscopo and Summa (2007). Those results are limited by not having any available data that could show the seasonal change in regional groundwater levels.

Misstear and Beeson (2000) applied their methodology to a well field in the UK Chalk aquifer. That field consisted of six wells that were pumped continuously from various combinations of the wells. The available drawdowns were uncertain and arbitrarily set. The non-pumping levels of the sources could not be measured, since at no time were all wells shutdown, so they estimated the non-pumping levels to be 7.5 ft (2.3 m) above datum. Their operational approach produced an estimated well field yield of 3,300 gpm (18,000 m^3/d) in 1996 (Misstear and Beeson, 2000, Figure 5). Misstear and Beeson indicated that regional water levels in area observation wells were 1.6 ft (0.5 m) lower in 1990 than in 1996. Misstear and Beeson then applied that value to their drought bounding curve for 1996, indicating that the yield would have been reduced to 2,200 gpm (12 tcmd) in 1990; however, no production data were provided for that year. None of the observation wells were identified and no water level data from those wells were presented Misstear and Beeson (2000).

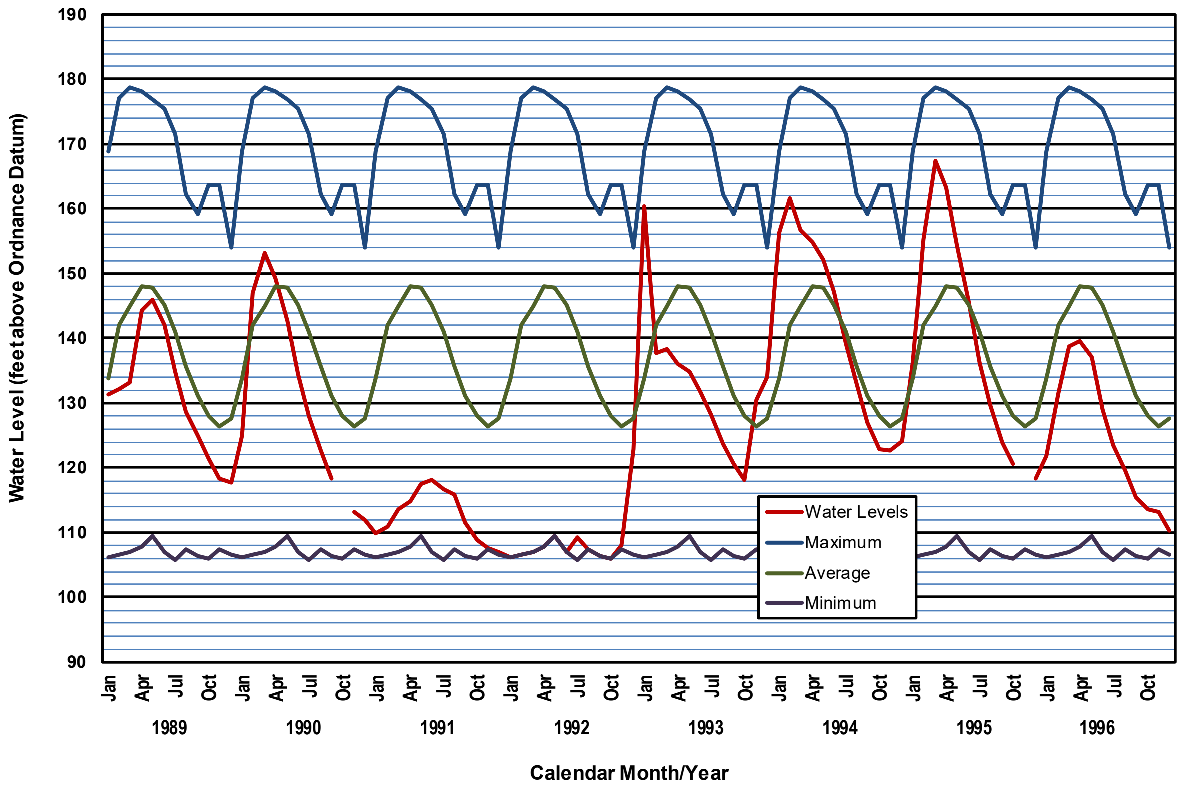

Figure A5. UK Chalk well, Redlands Hall, TL 44/12, groundwater levels.

A review of ground water levels available in the National Hydrological Monitoring Program — Monthly Summaries for the UK was conducted in the present study. Three observation wells were found in the Chalk aquifer that had comparable water level fluctuations as the unnamed observation well shown in Misstear and Beeson (2000, Figure 3). These were TF 81/2 (Washpit Farm), TL 11/9 (The Holt) and TL 44/12 (Redlands Hall). The water level data from TL 44/12 (fig. A5) correlated best with those of the observation well in Misstear and Beeson (2000) for the period 1989−1996. TL 44/12 is in East Anglia and, although the location of the production well was not published, Beeson indicated that it was operated by the Anglian Water Company (Beeson, personal communication, 2019), which primarily serves East Anglia. During 1989−1996, the range of water levels in TL 44/12 (56 ft or 17 m) was like that of the unnamed observation well (62 ft or 19 m). The average water levels in TL 44/12 were 109 ft (33 m) in 1992 and 124 ft (38 m in) 1996, while the long-term range was 73 ft (22 m). At 670 gpm per one ft (12 tcmd per one m) change in regional water levels and from a base of 3,300 gpm (18 tcmd) in 1996, the estimated range of yields for the well field varies from 0 to 36,700 gpm (0 to 200 tcmd). These results seem implausible, especially when compared to those of the present report and other available studies. Possible reasons for the discrepancy in the Misstear and Beeson (2000) drought estimate are that the available drawdown and static water levels had to be assumed, the drought bounding curve for the well field is relatively flat, the production data are relatively scattered and 1992, not 1990, was the worst drought during the study period.

It is noted that Misstear and Beeson (2000) did not use data from the individual well shown in Figure 2 of their paper to determine a drought yield, although there were operational data available from that well for both 1992 and 1996. The drought bounding curve for that well was developed for 1992, producing a yield of 517 gpm (2.82 tcmd). When that curve is shifted to the 1996 data, the estimated yield is 578 gpm (3.15 tcmd). The low water levels in the unnamed observation well were 49 ft (15 m) in 1996 and 39 ft (12 m) in 1992, producing an equivalent production of 6.2 gpm per one ft (0.11 tcmd per one m) change in regional water level. Based on the data for TL44/12 (Redlands Hall), the range of water levels in the unnamed observation well would be 73 ft (22.4 m). Using a base of 517 gpm (2.82 tcmd), the maximum estimated yield of the well would be 968 gpm (5.28 tcmd), or 1.9 times the minimum yield, which is like the results of from the Poolesville case study discussed in this report.

Figure 3 in Misstear and Beeson (2000) is a hydrograph of an observation well near the public supply well that the authors analyzed in their Figure 2. That well was constructed and tested in 1981. There was moderate decline of about five meters between late 1981 and 1986, with a typical seasonal variation in water levels in the range of about 26 ft (8 m). There was a sharp decline of about 33 ft (10 m) at that point until early 1988, followed by a recovery of about 26 ft (8 m) until middle 1988. There was then a sharp decline of 59 ft (18 m) until early 1989. At that time a multi-year drought started that lasted until late 1992, based on the hydrograph for observation well TL44/12 (Redlands Hall). From that point to the end of the record in late 1997, the variation in water level after the drought were like that of TL44/12 and TF81/2A, except the range, 49 ft (15 m), was much greater than before the drought. The early moderate decline followed by the 59 ft (18 m) decline, which occurred after the production well was completed and before the drought started, are indications of well interference. The greater variation in water levels after the drought is also an indication of aquifer dewatering due to a reduced transmissivity. These are factors that should be considered when estimating reliable drought yields of the UK wells.

Appendixes B.-K. Field Investigations

B.-H. Wells in Crystalline Rock Aquifers

B. Town of Myersville, Maryland

B1. Water Treatment Plant Well

Two aquifer tests were conducted on the Town of Myersville Water Treatment Plant (WTP) well (well permit FR-81-4337) in 1987 and 1989, at 35 gpm (132 L/min) and 15 gpm (57 L/min) for 54 hours and 30 hours, respectively. The first test had to be terminated early, when a drawdown of 195 ft (59 m) caused a nearby domestic well to go dry. This was like the 240 ft (73 m) of drawdown in the WTP well at the end of the test. The second test was concluded when water levels reached apparent equilibrium during the last 10 hours of the test Figure B1a is a time-drawdown graph of the two tests.

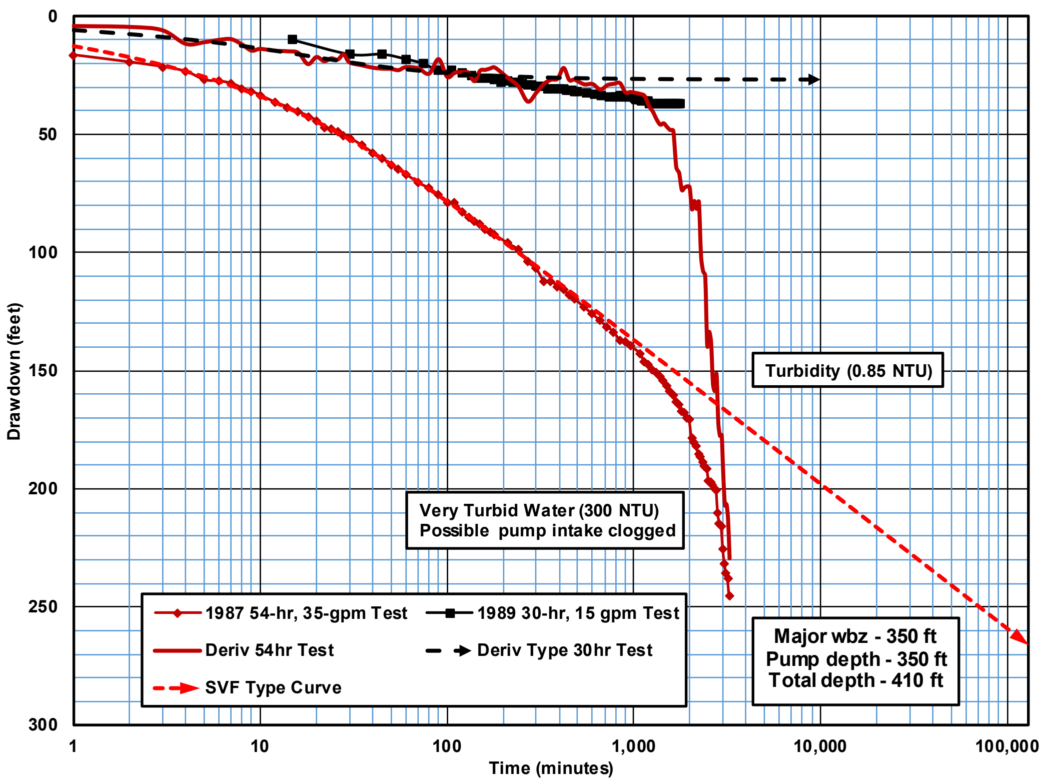

Figure B1a. Myersville Water Treatment Plant well – Semi-log plot of drawdowns from 54-h and 30-h aquifer tests, using the Gringarten-Witherspoon SVF solution.

The Gringarten-Witherspoon SVF analytical model best fit the drawdown data in the interval 0–300 minutes for the 54-hr test, when a clear break in the data occurs at 140 ft (43 m) of drawdown. The extrapolated 90-day drawdown is 265 ft (81 m) for an estimated yield of 18 gpm (68 L/min). The sharp decline in the water level after 300 minutes was likely due to sediment clogging the pump intake filter, as indicates by the high turbidity measurements.

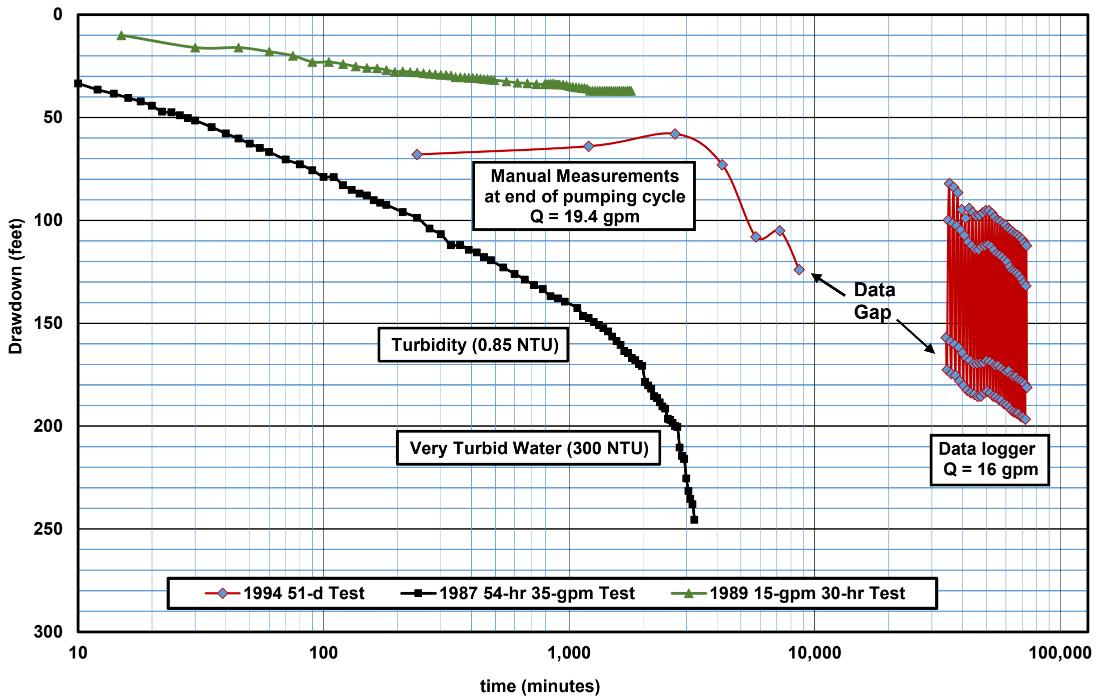

Figure B1b. Myersville Water Treatment Plant well – Semi-log plot of a composite of drawdowns from the 54-d, 54-h and 30-h aquifer tests.

Due to the potential impacts to a remaining domestic well and the relatively low estimated yield (18 gpm), a long-term test of the WTP well was conducted for 51 days in July-August 1994. Figure B1b compares the results of the long-term test to the earlier shorter tests in 1987 and 1989. Due to an oversized pump, it had to be cycled to achieve the target pumping rate. Like the 1989 30-hour test, the initial the water level decline was a relatively stable drawdown of about 60 feet for the first 3−4 days at about 19 gpm (72 L/min). After that point, and like the 54-hour test, the water level declined steadily to a drawdown at the end of the test of 200 feet (61 m), at 16 gpm (61 L/min). This indicates that the reliable yield may be less than 16 gpm (61 L/min) since the test was not conducted under drought conditions.

The drought calculation method was used to determine the drought year yield of the Water Treatment Plant well. The groundwater monitoring well data used for this analysis was from USGS well WA Dj 2 about 3.3 miles southwest of Myersville. $A = h_{obs} - h_{min} = 17$ ft, assuming a two-month lag of yield in response to changes in groundwater levels during the 1994 test. Since the well is used to meet demand then: $Q_{f\min} = 1$ and $Q_{f\max} = 2$ $\quad Q_{fobs} = {}^{17\text{ x}2}/_{49} + 1 = 1.69$

$$Q_{\min} = {}^{16\text{ gpm}}/_{1.69} = 9.5\text{ gpm}$$

In this case, the minimum yield occurred in February 2002, when water demand was at seasonal low levels, due primarily to the lack of outdoor water use. Based on studies conducted by MDE, the demand in February of a drought year is about 75 per cent of the average use and 54 per cent of the peak monthly use. This would indicate that the demand during the month of maximum use (July) would be 17.6 gpm. As the water level was about 8 feet deeper in July 2001 than during the 51-d test, an adjustment needs to be made to the test rate of 16 gpm. The range of the calculated seasonal variation in yields is 9.5 to 19.0 gpm. The difference between the maximum water level (61 ft) and the July 2001 water level (38 ft) of 23 ft is 47 per cent of the range (49 ft) and 47 percent more than the minimum yield (9.5 gpm) or 14.0 gpm. The result is 12.5 percent less than the estimated yield and 20.5 percent less than the estimated demand.

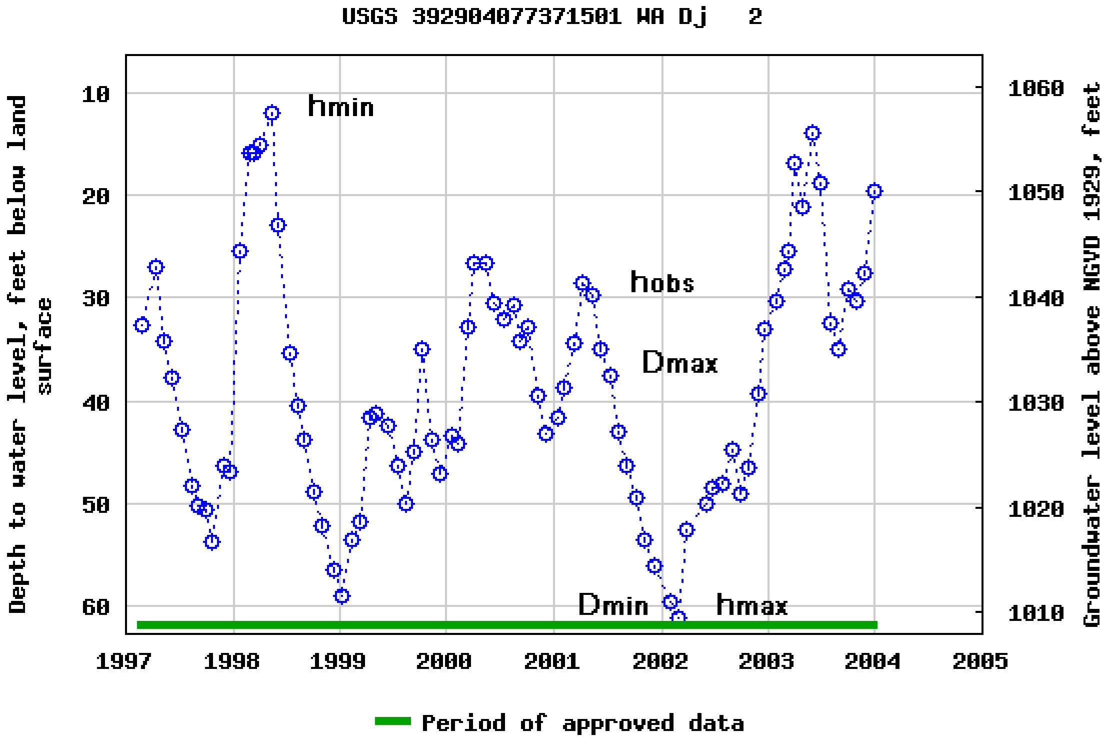

Figure B1c. USGS observation well WA Dj 2. Water levels (ft) for the period 1997-2004. Dmax (summertime demand), Dmin (wintertime demand), hmin & hmax (minimum and maximum water levels during drought period 1998-2002) and hobs is the water level in during the 51-d test of the WTP well (July-August 1994)

The reported water use during February 2002 was 11,636 gpd or 8.1 gpm. The maximum use was 12,997 gpd during August 2001 or 9.0 gpm; however, water use restrictions were in place, which could have reduced water demand and withdrawals from the well. During the moderately severe drought of 2007, the average withdrawals from the well were 13,401 gpd (9.3 gpm) and 18,733 gpd (13.0 gpm) during the month of maximum use (June).

These results are consistent with the MDE policy that fractured rock public wells need to be able to supply water demand during a 1-in-10 (moderately severe) drought without restrictions and water demand during an extreme drought with water restrictions in place.

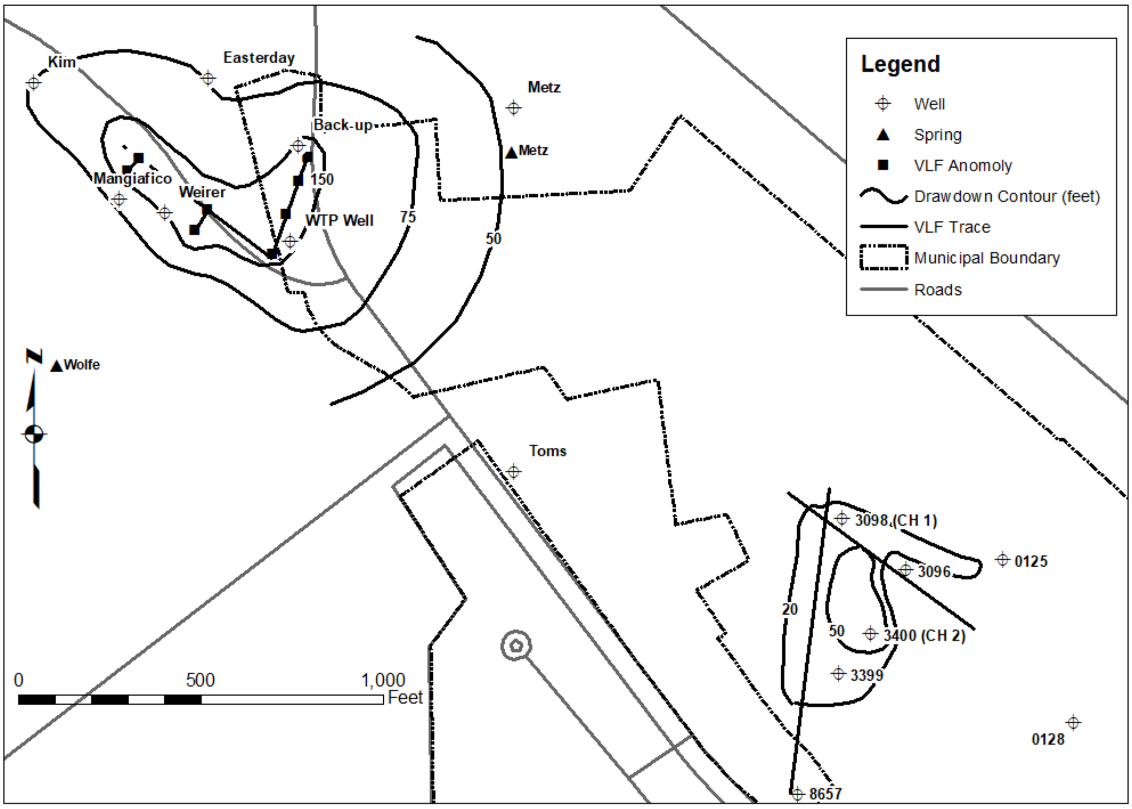

Figure B1d. Map showing drawdowns from 51-d, 19-16 gpm test of the Water Treatment Plant well and the 24-h, 30 gpm aquifer test of Canada Hill well 2.

B2. Canada Hill Wells 1 and 2

An early 76-hour test performed at the Canada Hill site in December 1988, pumping FR-88-0128 at an initial rate of 10 gpm (38 L/min), followed by 6 step decreases to a final rate of 4.76 gpm (18 L/min), after which the well was abandoned. Three aquifer tests were then conducted on the Canada Hill wells 1 (FR-88-3098) and 2 (FR-88-3400). The first test was a 72-hour test performed in September of 1992, pumping well 1 at 25 gpm (95 L/min). The third test was a 24-hour test performed in April of 1993, pumping well 2 at 30 gpm (114 L/min). The fourth test was a 24-hour combined aquifer test performed in May 1993, while pumping well 2 at 24 gpm (91 L/min) and well 1 at 21 gpm (79 L/min).

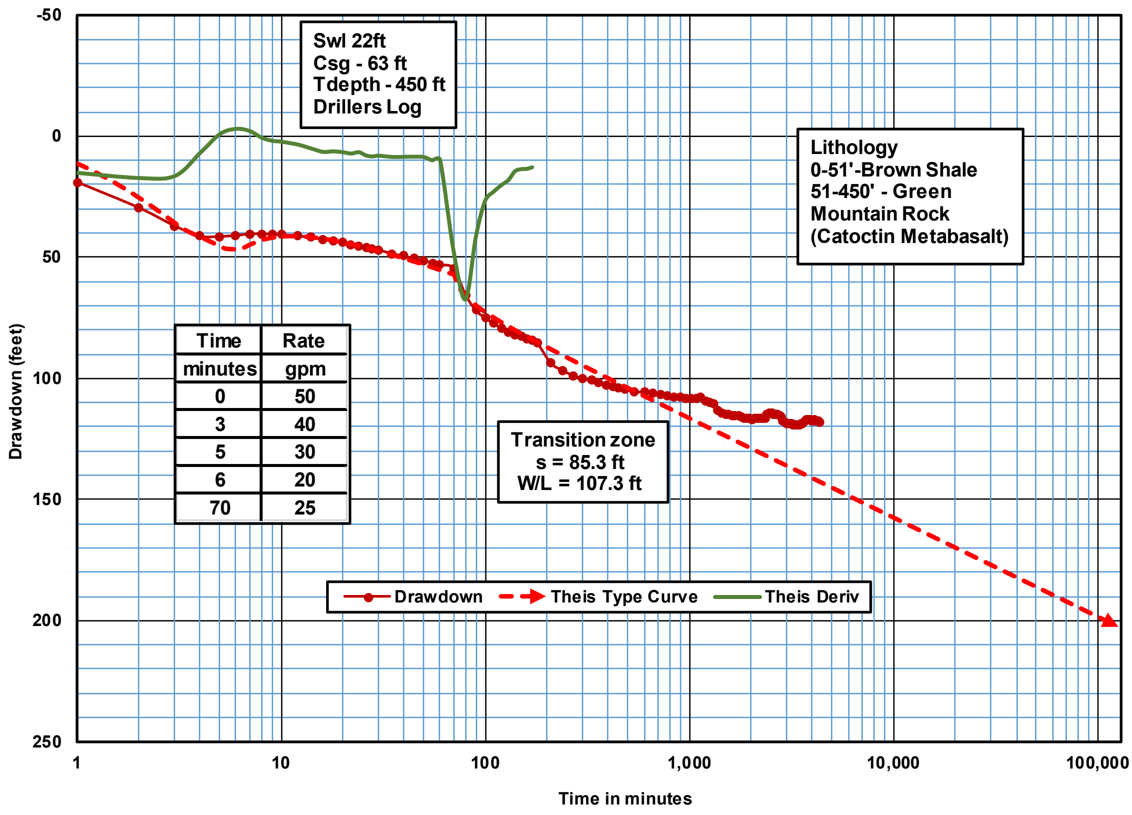

Figure B2a. Myersville Canada Hill well 1 – Semi-log plot of drawdowns from a 72-h, variable rate aquifer test, drawdowns from 0−180 min, Theis IARF type curve and derivative.

Figure B2a is a time-drawdown graph of the results of the graph of the 72-hour test of well 1. The initial rate was 50 gpm (189 L/min), which was decreased in steps to 20 gpm (76 L/min) after 6 minutes. It continued at that rate until 70 minutes, after which an increase to 25 gpm produced a sharp break in the curve. That rate continued for the remainder of the test, but there was another sharp break in the curve at 180 minutes ((s = 85.3 ft (26 m) at a W/L = 107 ft (33 m)). An AQTESOLV® Theis IARF solution provided a good fit to only the first 180 minutes of the test. This indicates that the first break in the data was due to the change in the pumping rate, while the second reflected a change in the formation hydraulic properties. When the IARF solution is extrapolated to 90 days, the drawdown is 205 ft (62 m). When the ratio of the drawdown at the second break to the 90-day extrapolation is applied to the average rate (23.7 gpm or 90 L/min), the estimated reliable yield is 10 gpm (38 L/min).

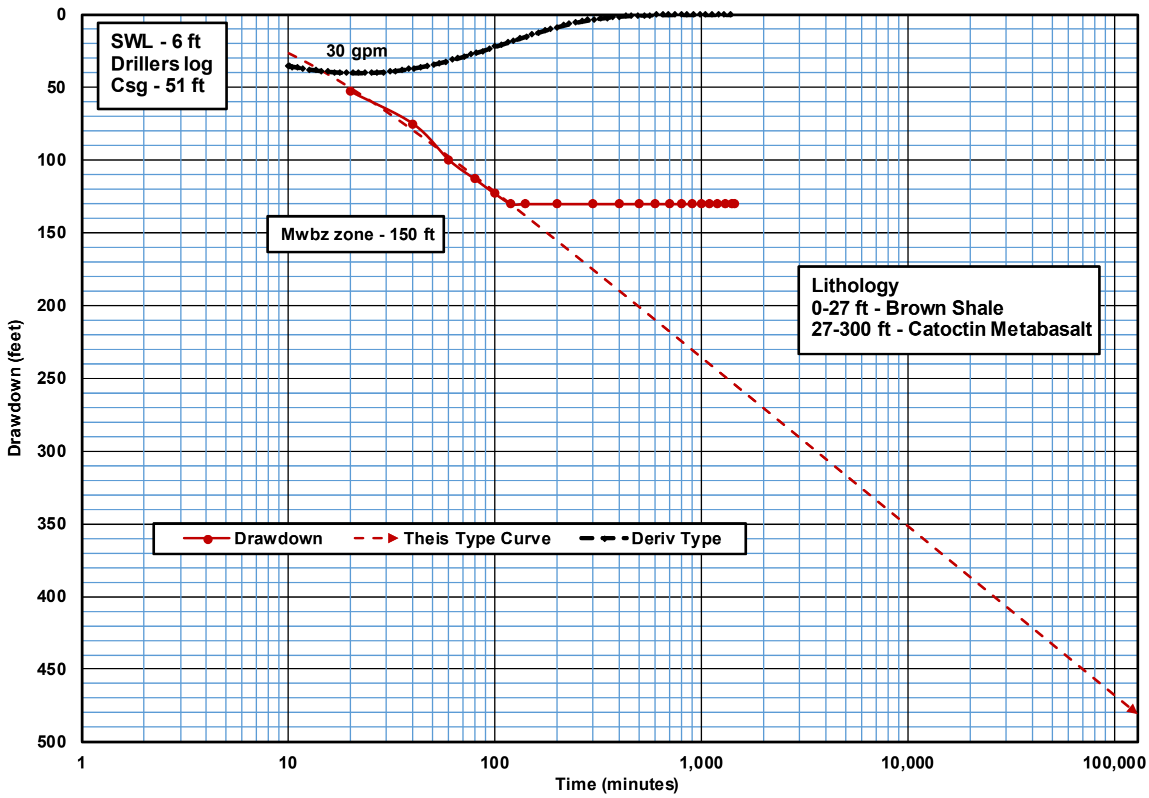

Figure B2b. Myersville Canada Hill well 2 – Semi-log plot of drawdowns from a 24-h, 30 gpm aquifer test, 0−120 min Theis IARF solution and derivative type curve 0-1440 min.

During the 30 gpm, 24-hout test of Well 2 (fig. B2b), apparent equilibrium was reached after 120 minutes at a water level of 136 ft (41 m) or 130 ft (40 m) of drawdown. Although the data fits a leaky aquifer model, the derivative indicates that there is an early time IARF segment, after which it reaches zero drawdown, which is the typical response of a recharge boundary. There, however, is no direct evidence of a recharge source, such as a lake or stream; but March and April 1992 had high rainfall amounts of 6.8 in (17 cm) and 7.3 in (19 cm), respectively, which in has caused water levels to temporarily stabilize for extended periods during long-term tests. A second possibility is that dewatering of a water-bearing zone at 150 ft (46 m) may have caused the water level in the well to temporarily stabilize, Hammond and Field (2014). If the water level had stabilized in true equilibrium, then the estimated yield would be 30 gpm (114 L/min) The IARF model provides the best fit to the first 120 minutes of the data. The 90-day extrapolated drawdown is 490 ft (149 m). When the specific capacity is applied to an available drawdown of 130 ft (40 m), the estimated yield is 8 gpm (30 L/min).

Figure B1d shows the drawdowns in the pumping and observation wells at the end of the 24-h, 30 gpm aquifer test of Canada Hill well 2 and the results of a geophysical survey using very low frequency (VLF) techniques. When the drawdowns in the pumping and observation wells during the 24-hour and 72-hour tests were mapped, the orientations of the resulting contours are like those of the two fractures determined from VLF anomalies; except, the NW-trending fracture appears to be parallel to but offset by about 150 feet from the equivalent trough of depression. Several possible explanations for this anomaly are that the fracture is not vertical, well or fracture locations are not accurately plotted or that secondary fractures may have affected the drawdown data during the pumping test. Also, Canada Hill well 1 and observation well 8957 are on or near the same fracture trace, but the drawdown at the end of the 72-h test of Canada Hill well 1 in well 8957 was only 7 feet when the drawdown in Canada Hill well 1was 118 feet. The indicates that the fracture is hydraulically closed between the two wells. During the 24-h test of Canada Hill 1, the drawdown at the end of the test was 24 feet in Canada Hill well 2 and 130 feet in the pumping well. This suggests that there would be interference between the two wells, but the degree of interference would be less than at the Water Treatment Plant site.

Due to the potential impacts to nearby domestic wells and to verify the reliable yields of the Canada Hills wells, a long-term test (24 days) of those two wells was conducted in 1994, (figs. B2c and B2d)

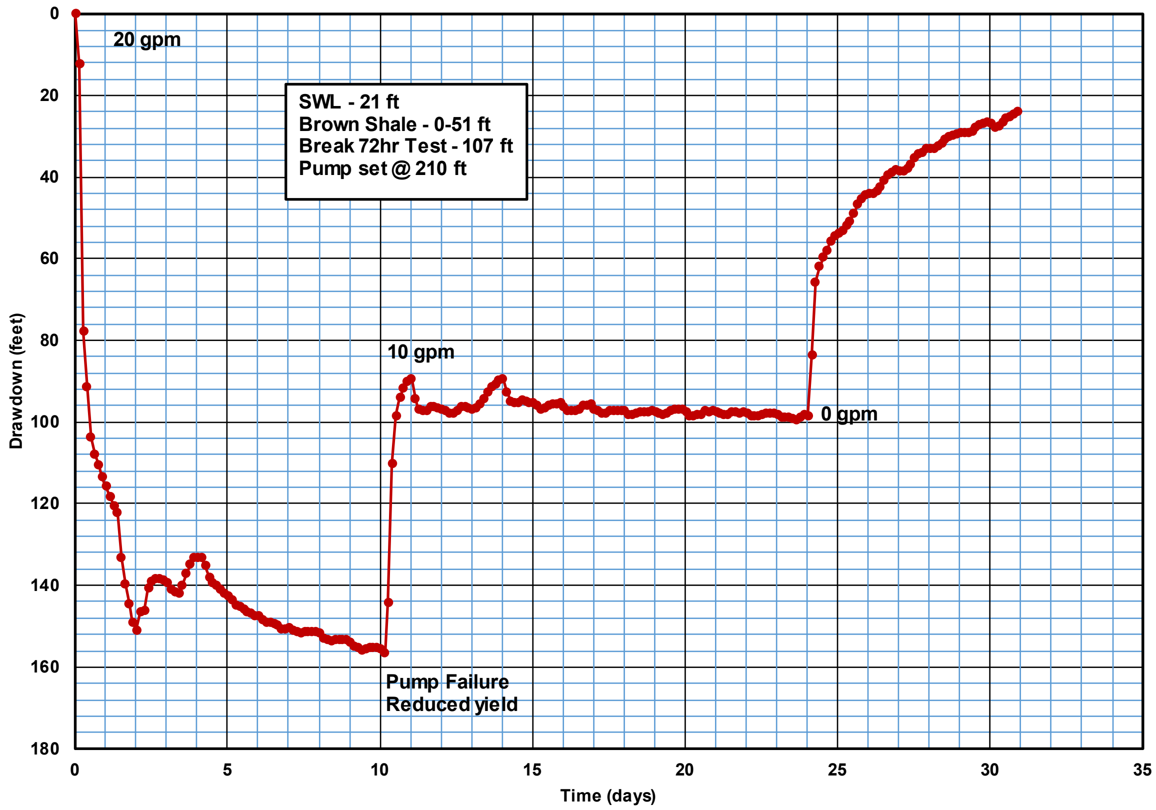

Figure B2c. Myersville Canada Hill well 1 – Arithmetic plot of drawdowns from a 24-d, variable rate aquifer test.

The test of well 1 started at 20 gpm (76 L/min) and the drawdown was excessive, reaching 150 ft (46 m) after the first day. It appears some unrecorded rate adjustments caused the water level to recover over the next several days, after which there was a steady drawdown until the pump malfunctioned at 10 days; but could still produce 10 gpm (38 L/min), so the test was continued at that rate for the remainder of the 24-day test. The final water level stabilized at a drawdown very near the breakpoint that occurred during the 1992 72-hour test; consequently, these results appear to confirm the estimated reliable yield of 10 gpm (38 L/min).

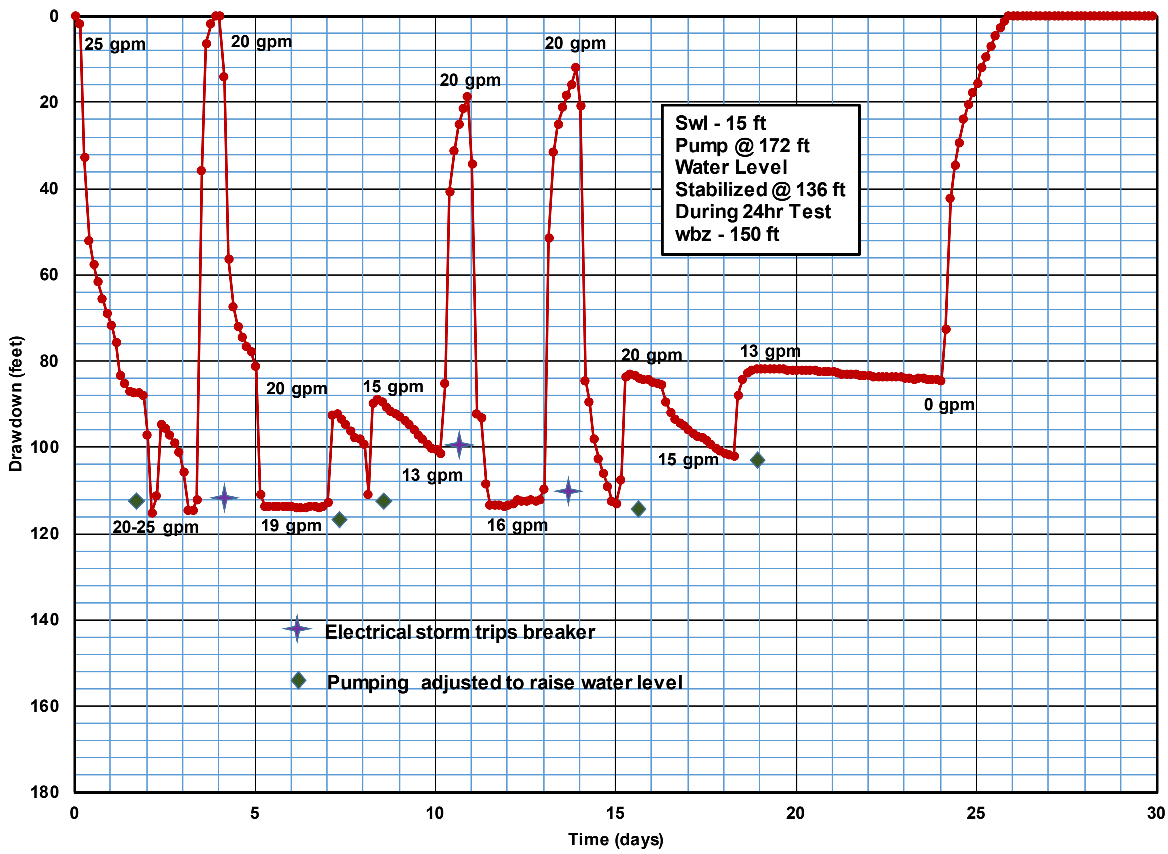

Figure B2d. Myersville Canada Hill well 2 – Arithmetic plot of drawdowns from a 24-d, variable rate aquifer test.

The 24-day test of well 2 started at 25 gpm (95 L/min) with a sharp drop in the water level occurring during the second day. The well was then shut down briefly and restarted at 25 gpm (95 L/min). After about three days, an electrical storm caused the generator breaker to trip and the well shut down again, resulting in a full recovery, and it was then restarted at 20 gpm (76 L/min). Cycling of the well continued for the next 16 days, during which time the water level stabilized only twice, at 19 gpm (72 L/min) (days 5-7) and 16 gpm (61 L/min) (days 11-13). Both times the water levels were the same as that which occurred during the 1993, 24-h test at the higher rate of 30 gpm (114 L/min). On the 18th day, the water level was allowed to recover about 30 feet, and it nearly stabilized at 13 gpm for the remainder of the test. Although this was an erratic test, the data suggests that the yield of the well during the tests was no more than about the final rate of 13 gpm (49 L/min) or the average rate of 15 gpm (57 L/min). This is about one-half of the estimated rate based on the stabilized water level during the 1993 test. The 24-day test was conducted during slightly above average climatic conditions, based on the water levels in WA Dj 2 and a rainfall of 4.75 in (12 cm) during June 1994. Based on the method developed in this study for adjusting for seasonal variations, the yield should have been about 20−25 gpm (76-95 L/min) relative to the 1993 30 gpm (114 L/min) test rate. Conversely, the extrapolated rate of 8 gpm (30 L/min) from the early-time data of the 1993 test, is about one-half of the final rate of the 1994 test.

The total yield of the two wells during the 24-d test was about one-half the combined rates during the 1993 24-hour simultaneous test of the wells. Interference could explain that result and the reason that the yield of FR-88-3400 was less than 20−25 gpm (76-95 L/min). Also, when correcting for climatic variations, the drought yields of the wells would be proportional to the drought yield of the

WTP well, since the long-term tests were conducted concurrently. Consequently, it is expected that the total yield of the wells could be as little as 11−14 gpm (42-53 L/min) under severe drought conditions (February 2002) and twice those amounts during very wet periods. Water use (pumpage) reports were unreliable until late 2006. The reported use during the moderately severe drought of 2007 was an average of 26.6 gpm in January-April under high water table conditions and 15.5 gpm in August-November under low water table conditions.

B3. Deer Woods Well

A 24-h, 25 gpm (95 L/min) aquifer test was conducted on the Deer Woods well (FR-88-4026) in September 1992 (fig. B3a).

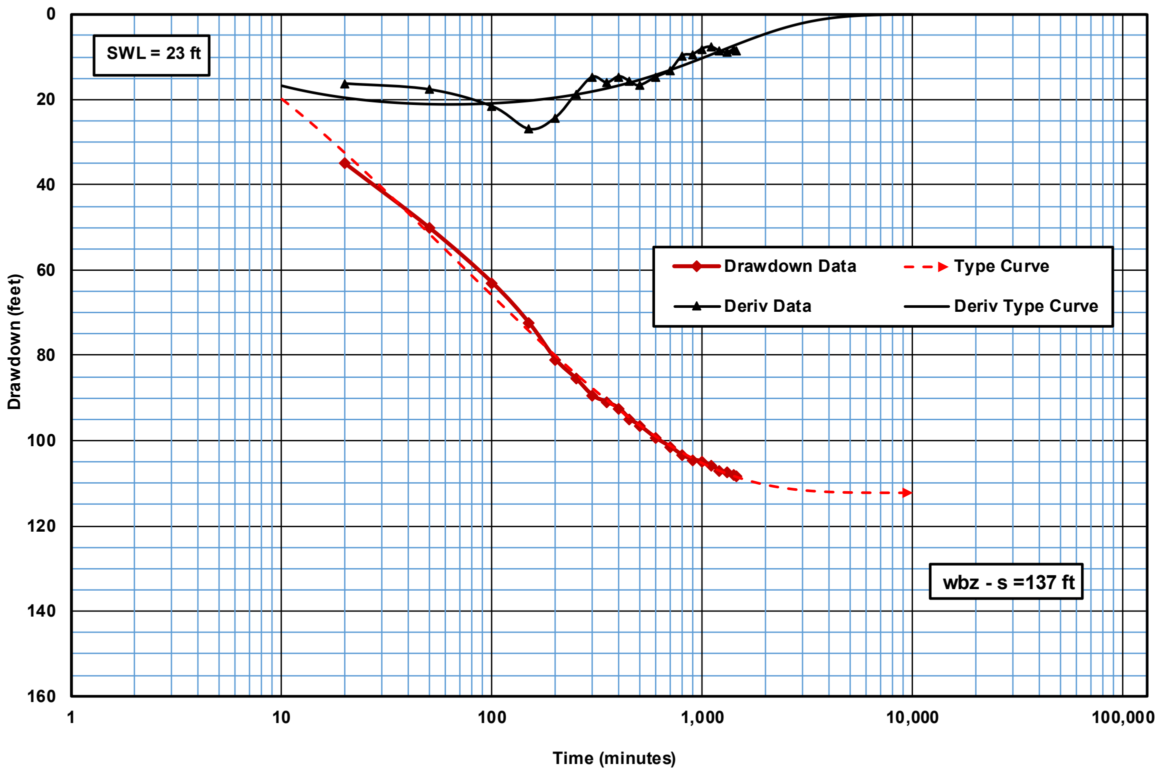

Figure B3a. Myersville Deer Woods well – Semi-log plot of drawdowns from a 24-h, 25 gpm aquifer test, Hantush-Jacob leaky aquifer solution

The time drawdown data best fits a leaky aquifer model (fig. B3a), which indicates that the water level would stabilize about 20 ft above the main water-bearing zone (wbz) at 160 ft (49m) and suggests the well should have been able to produce 25 gpm (95 L/min) under the average climatic conditions that existed during the test.

A 24-day operational aquifer test of the Deer Woods well was conducted starting on June 6, 1994, at 20 gpm (76 L/min) (fig. B3b). For the first two days, the average rate was 16 gpm (61 L/min); however, the water level was about 7 ft (2 m) below the wbz, so the rate was reduced to 13 gpm (49 L/min). At day 5, the water level had dropped an additional 20 ft (6 m); but it was unclear why this was, but it could be dewatering of the fracture or an error in measurement (the recorded value had been over-written). This could not be resolved since there was no record of a rate adjustment to account for the recovery of the water level by the next reading. The water level was stable for the next 6 days at 12 gpm (45 L/min), at which time the pump was shut off to raise the water level above the fracture to see if this would help increase the yield. While the water level did rise above the fracture, the pumping remained unchanged at 12 gpm (45 L/min). From these data, it appeared that the well could produce 12 gpm (45 L/min).

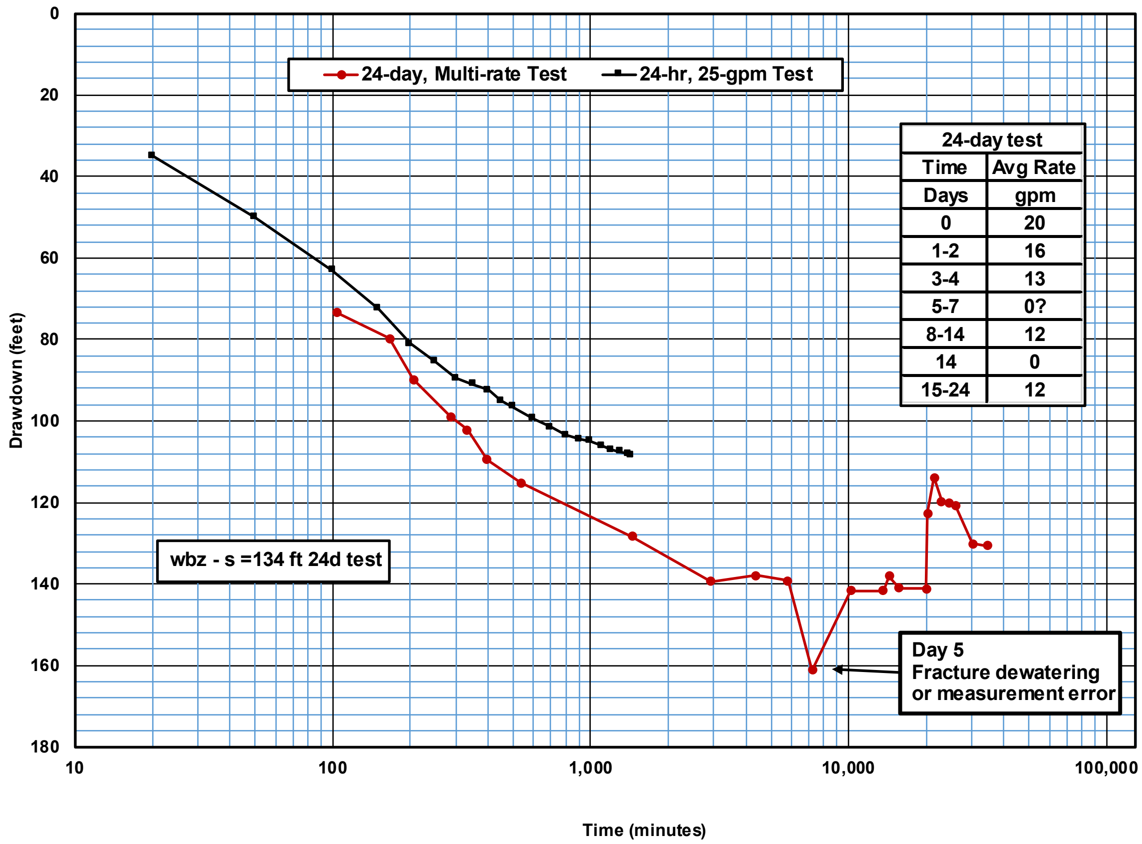

Figure B3b. Myersville Deer Woods well – Semi-log plot of a composite of drawdowns from the 24-d and 24-h aquifer tests.

It is not clear why the yield during the 24-day test was one-half of the 24-h test. Regional water levels were about average during both tests; however, the static water, 26.4 ft (8 m) below the top of the casing (BTOC), during the 24-day test was more than 3 ft (1 m) deeper than the 24-hr test, indicating that conditions were dryer during the long-term test. Based on the completion report, the base of the weathered zone was 52 ft (16 m), suggesting that it was saturated, but then dewatered during both tests. It is a likely reservoir rock, but the thickness is unknown, so a difference of 3 ft (1m) in static water levels (SWLs) could account for some, if not all, of the difference in yields. Another factor to consider is that the leaky aquifer model, although resulting in a good fit to the data, may not have accurately described the conditions that exist when the bedrock portions of the crystalline rock aquifer are being dewatered.

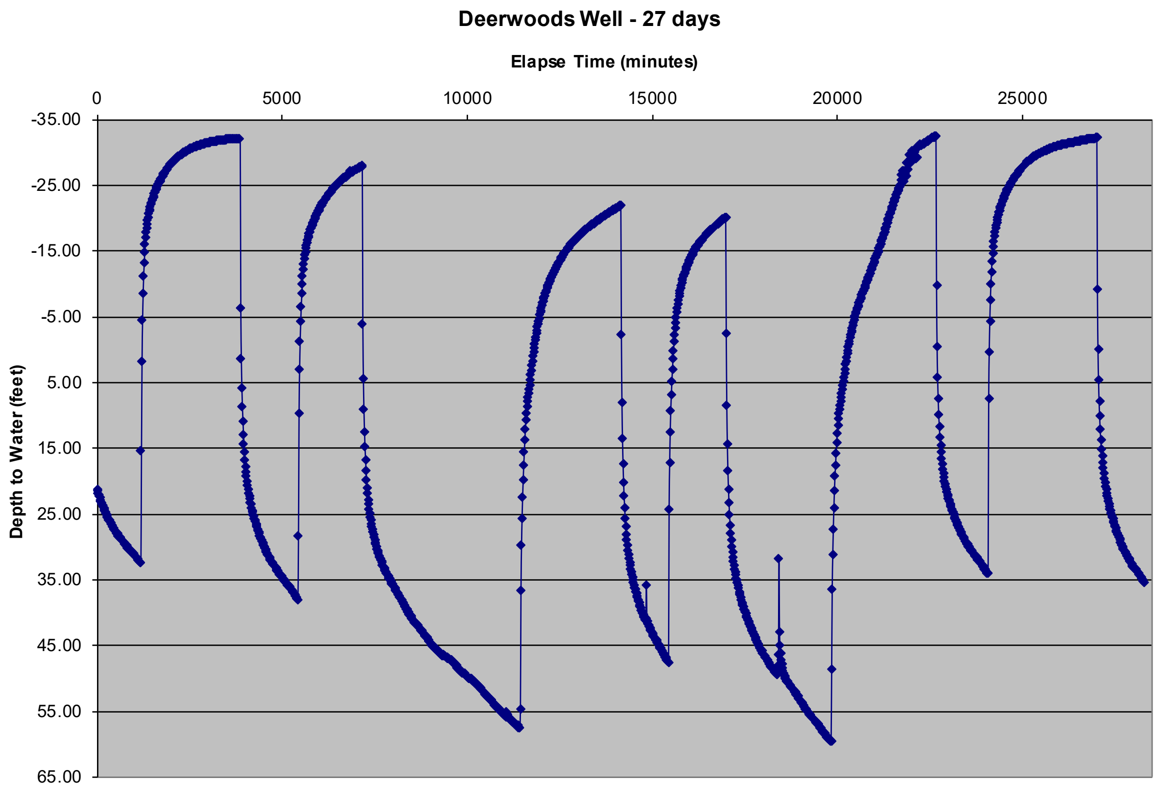

Figure B3c. Myersville Deer Woods well – Arithmetic plot of water level data collected during the period December 23, 2008, to January 11, 2009. The measuring point is unknown

Between December 23, 2008, and January 11, 2009, water levels were measured in the Deer Woods well (fig. B3c). While the absolute depths cannot be determined, the data do show that there was at least 90 ft (27 m) of drawdown while operating the well on a one- to three-day cycle. The average monthly pumpage was 10,906 gpd (41 m^3/d) or 7.6 gpm (29 L/min). Since there was still about 50 ft (15 m) of drawdown available to the main wbz, the reliable yield might have been close to 12 gpm (45 L/min). The average and maximum reported withdrawal rates during the drought of 1999 and 2001−2002 ranged from 7.3 gpm (28 L/min) to 11 gpm (42 L/mim) and 10 gpm (38 L/min) to 16 gpm (61 L/min), respectively. Conversely, during the two wettest years, 2003 and 2018, the average and maximum reported withdrawal rates ranged from 14 gpm (53 L/min) to 16 gpm (61 L/min) and 18 gpm (68 L/min) to 21 gpm (79 L/min), respectively. These yields were about twice as high during the wet years than during the drought years.

C. Town of Middletown

C1. Well 16

Aquifer tests were performed on Town of Middleton's well 16 (FR-94-3317). An initial multi-rate test with recovery between steps, in lieu of the standard step-drawdown test, and 72-h constant-rate test at 16 gpm (61 L/min) were performed on well 16 in January 2003 (figs. C1a and C1b). The only fracture in the well at 160 feet was isolated by packers and the well was then hydraulically fractured, followed by another multi-rate test and 72-h constant-rate test at 38 gpm (144 L/min) in October 2003. (figs. C1a and C1b).

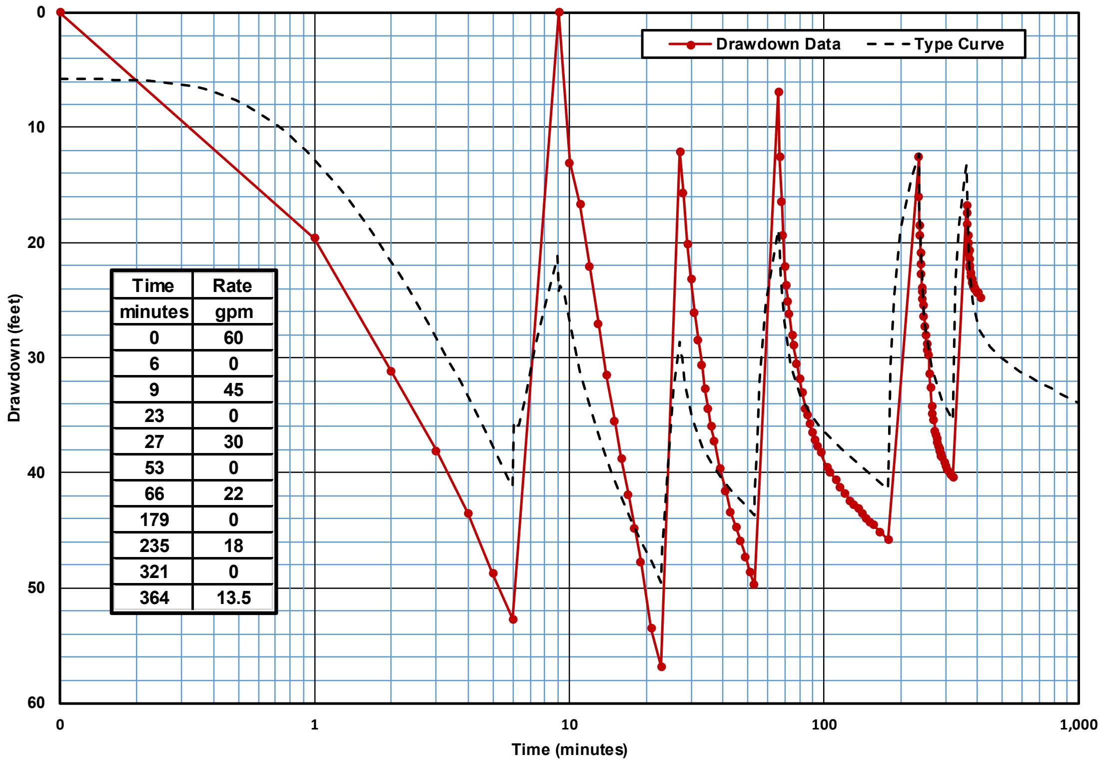

Figure C1a. Middletown well 16 – Semi-log plot of drawdowns from a multi-rate test conducted prior to hydraulic fracturing, Hantush-Jacob leaky aquifer solution.

The Hantush-Jacob leaky aquifer solution provided the best fit to the data from the two multi-rate tests (figs. C1a and C1c); however, the fit to the post-fracturing data was much better than the pre-fracturing data. This may have been due to the lack of the recording of recovery data in the first test, which had to be estimated in the analysis, while complete records were taken during the recovery periods of the second test.

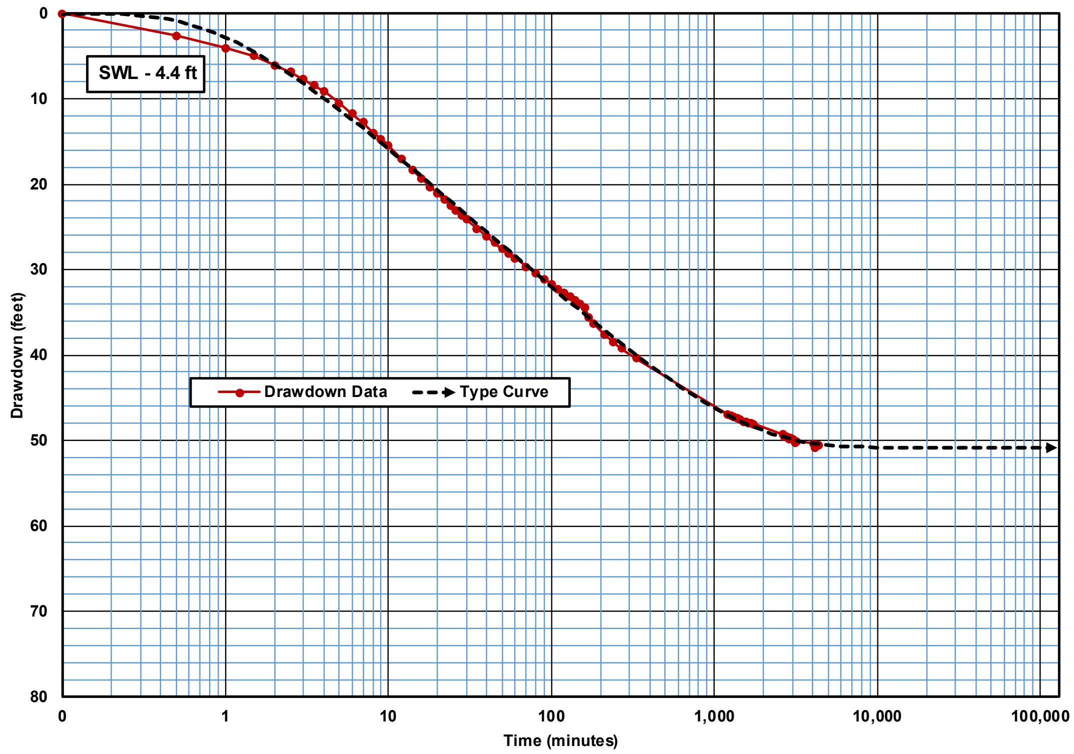

Figure C1b. Middletown well 16 – Semi-log plot of drawdowns from a 72-h, 16 gpm aquifer test conducted prior to hydraulic fracturing, Hantush-Jacob leaky aquifer solution.

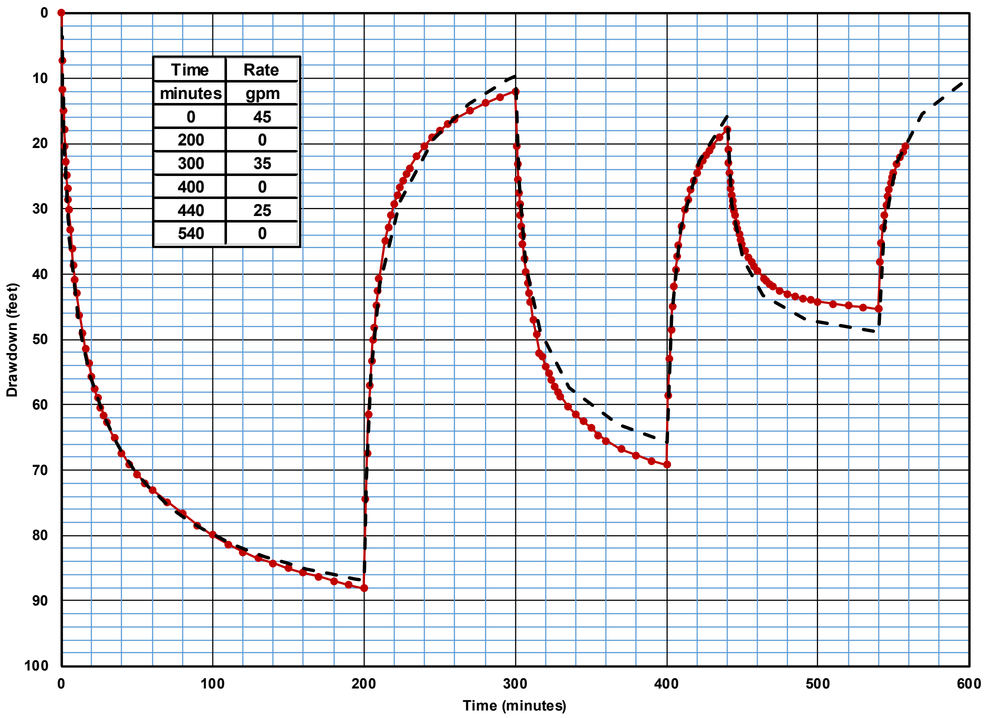

Figure C1c. Middletown well 16 – Semi-log plot of drawdowns from a multi-rate test conducted after hydraulic fracturing, Hantush-Jacob leaky aquifer solution.

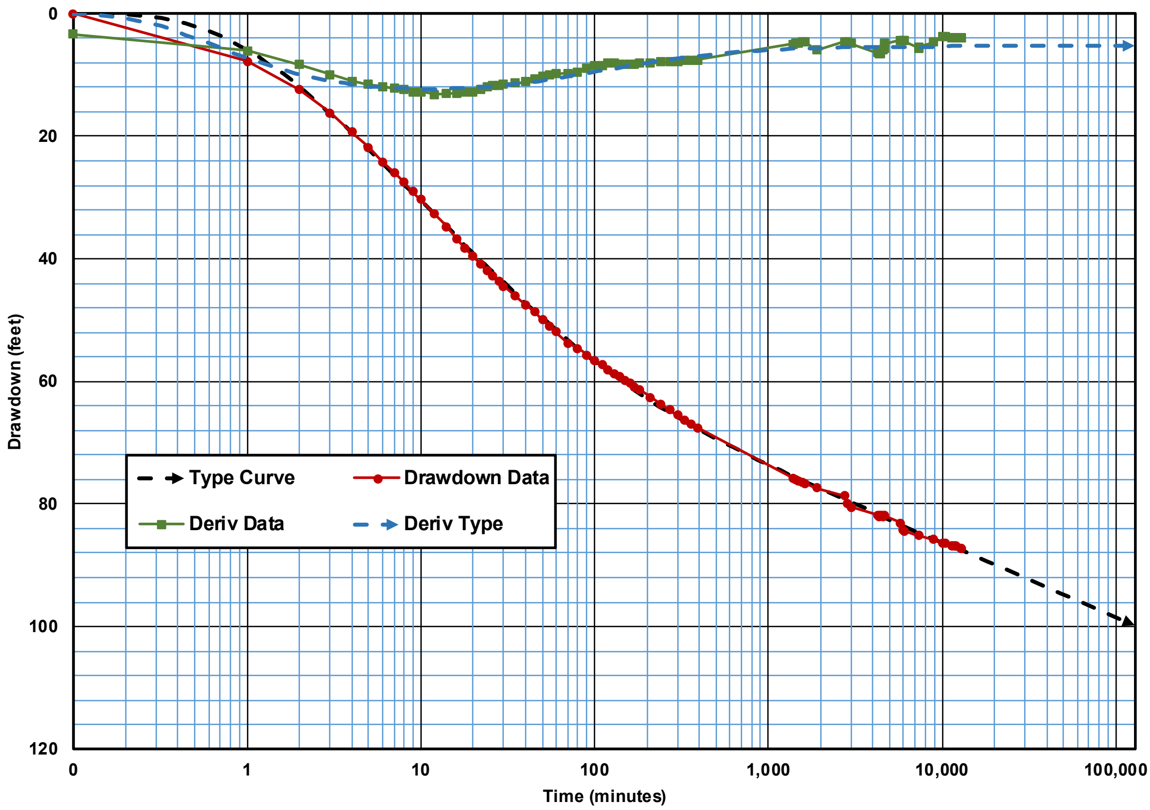

Figure C1d. Middletown well 16 – Semi-log plot of drawdowns from a 9-d, 38 gpm aquifer test conducted after hydraulic fracturing, Neuman-Witherspoon two aquifer solution.

The well efficiency increased from 50 to 79 percent (a 58-percent increase), after hydraulic fracturing of the well. Due to the poor quality of the data from the first multi-rate test, as seen by visual inspection and statistical analysis, a comparison of the results of the calculated specific capacities from the two aquifer tests was made. The specific capacity at 100 minutes increased from 0.51 gpm/ft (6.27 Lpm/m) during the first test to 0.67 gpm/ft (8.34 Lpm/m) during the second test (a 33-percent increase over the first test). At 1500 minutes, the specific capacity increased from 0.34 gpm/ft (4.16 Lpm/m) during the first test to 0.50 gpm/ft (6.18 Lpm/m) during the second test (a 49-percent increase over the first test). The multi-rate and constant-rate test data indicate that hydraulic fracturing may have increased the yield by one-third to one-half more than the initial rate. There was little difference between the transmissivity calculated from the pre-fracturing multi-rate test (302 gpd/ft or 3.1 m^2/d) and post-fracturing multi-rate test (281 gpd/ft or 3.5 m^2/d) while there was a significant increase in the transmissivity calculated from the pre-fracturing constant-rate test (281 gpd/ft or 3.5 m^2/d), using the Hantush-Jacob leaky aquifer model (fig. C1b), and post fracturing constant-rate test (314 gpd/ft or 5.9 m^2/d [aquifer 1] and 514 gpd/ft or 6.4 m^2/d [aquifer 2]) (a 12-83 percent increase over the first aquifer test), using the Neuman-Witherspoon two aquifer model. The similarity of transmissivity calculated from the multi-rate tests combined with the increase in transmissivity from the constant-rate tests after hydraulic fracturing suggest that the fracturing had little influence on the well bore hydraulics; however, the flow regime changed from a leaky aquifer to one were there was a direct connection between the weathered zone and the primary water-bearing fracture. The 90-day extrapolated drawdown from the second test is 100 ft at 38 gpm (125 L/min). This seems reasonable, since there are no obvious breaks in the drawdown curves, suggesting that the available drawdown is greater than the maximum observed drawdown of 90 feet. The Hantush-Jacob model fit the entire set of data for the 72-h, 16 gpm pre-facturing aquifer test, with the water level stabilizing at 51 ft of drawdown. When an extrapolation is made to 100 ft (30 m) of drawdown, the estimated yield is 31 gpm (117 L/min), indicating that hydraulic fracturing may have increased the yield by about 23 percent, or less than one-half the increase in yield determined by the calculated specific well capacities or transmissivities. It is possible that the aquifer hydraulic conductivity decreases with depth, but this cannot usually be detected during a standard aquifer test, which typically only produces a bulk aquifer transmissivity. It is then possible that the change in well yield could be greater, sincet the estimated reliable yield from the first test could be lesser than 31 gpm (1171 L/min). There, however, is no production data to verify the long-term well yield or to determine if the fracture seals over time, since no proppants were used to keep it open.

C2. Well 15

A multi-rate test was performed in February 2002 on well 15 (FR-94-1544) (fig. C2a), in lieu of a standard step-drawdown test. The Hantush-Jacob leaky aquifer solution provided the best fit to the multi-rate test data and produced a well efficiency of 87 percent.

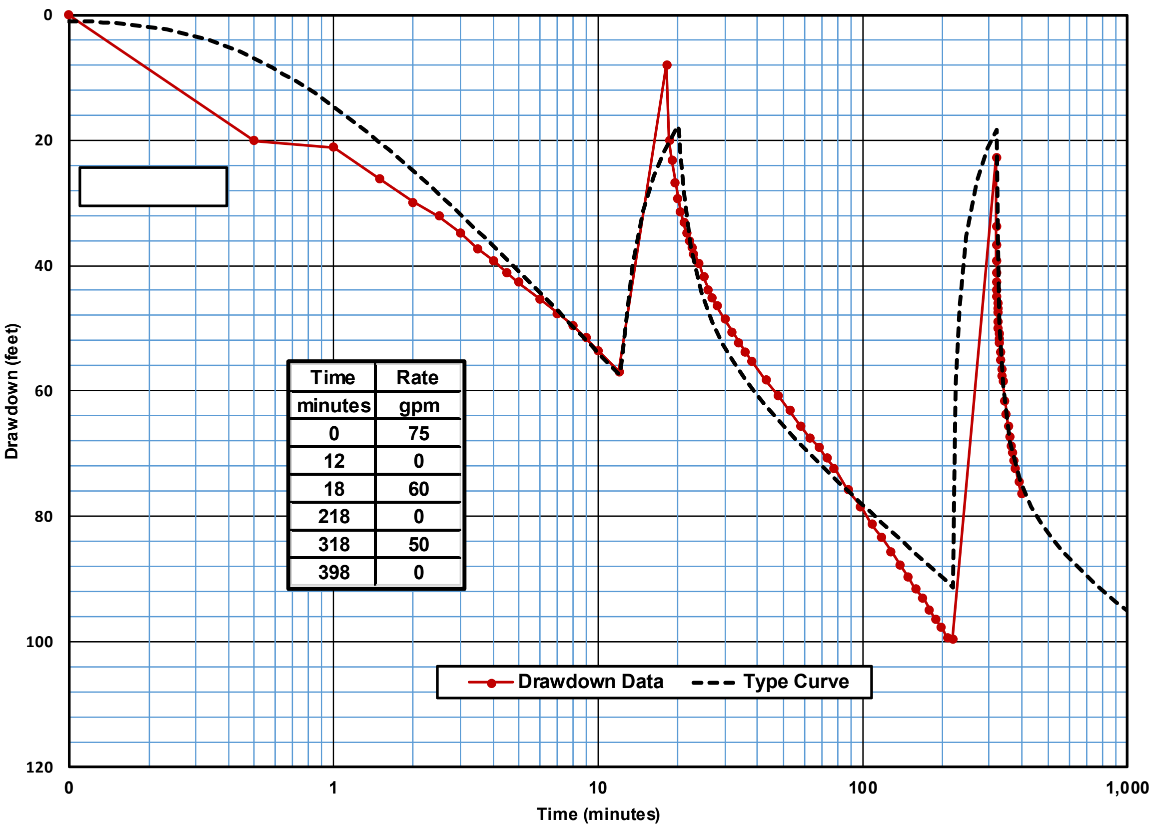

Figure C2a. Middletown well 15 – Semi-log plot of drawdowns from a multi-rate test, Hantush-Jacob leaky aquifer solution.

When the Hantush-Jacob leaky aquifer model is applied to the drawdown data of the follow-on 72.5-hour, 50 gpm (189 L/min) aquifer test (fig. C2b), the solution produced a reasonable fit; with an extrapolated 90-d drawdown of 120 ft (40m). The maximum drawdown observed during the multi-rate test was 100 ft (30 m) and 116 ft (35 m) during the 72.5-hr test. There was no evidence of dewatering, as would be shown by deviation or break in the drawdown curve, of the transition zone near the base of the well casing (60 ft [18 m]; equivalent to drawdown of 29 ft [9m]), or the first water bearing zone at 94 feet (29 m) (equivalent to drawdown of 63 feet or [19 m]), which indicates there was no distinct reservoir unit and the fracture at 370–375 feet (113-114 m) is the primary water bearing zone. While this makes it impossible to determine the available drawdown; however, in other cases, such as the Poolesville well 7 aquifer test, where a leaky aquifer response occurred, without any clear early breaks in the drawdown curve, the drawdown exceeds 100 ft (30 m), and there remains more than 100 ft (30m) of drawdown to the primary water-bearing zone, then the test rates provided good estimate of reliable well yields.

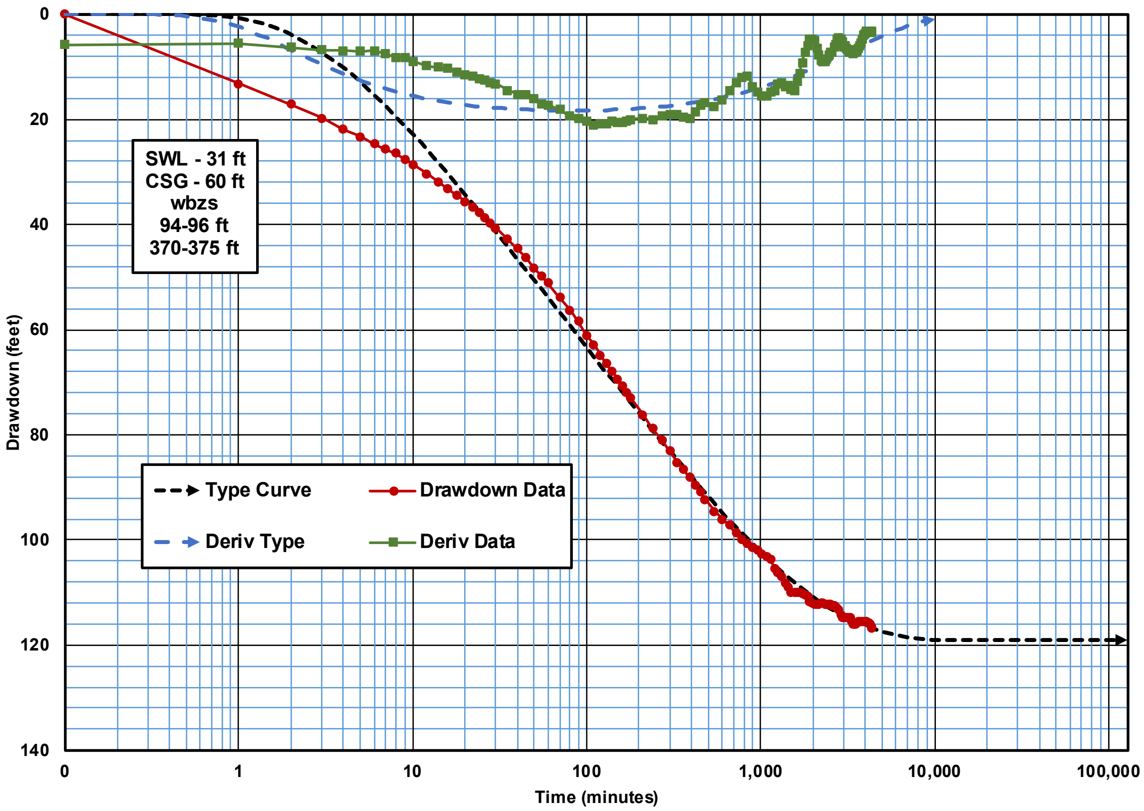

Figure C2b. Middletown well 15 –semi-log plot of drawdowns from a 72.5-h, 50 gpm aquifer test, Hantush-Jacob leaky aquifer solution.

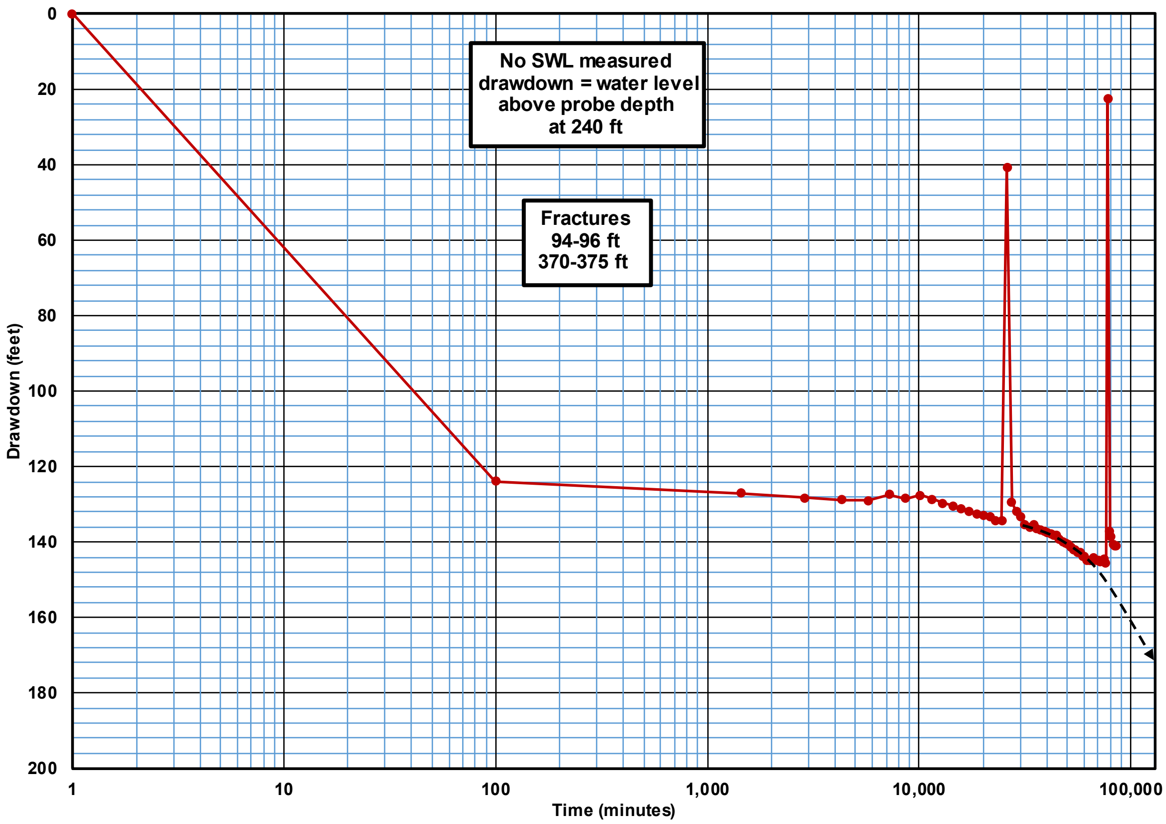

Figure C2c. Middletown well 15 – Semi-log plot of drawdowns from a 60-d, 50 gpm aquifer test.

A follow-on 60-d, 50 gpm (189 L/min) test was performed from November 2004 to January 2005 (fig. C2c), which resulted in a drawdown at the end of the test of about 140 ft (43 m), but this included two periods of about 100 feet of recovery that were probably due to pump shutdowns. It should be noted that there was a moderate rate of drawdown after 10,000 minutes that continued to 60,000 minutes, after which the last recovery period started. When the late-time drawdown is extrapolated to 90 days, the drawdown at that point is about 170 ft (52 m) and 50 ft (15 m) deeper than the extrapolated value from the Hantush-Jacob early-time simulation. It is further noted that if the 90-day extrapolation specific capacity is applied to the drawdown to the 370–375 ft (113-114 m) fracture zone, then the calculated yield would be about 100 gpm (379 L/min). As noted by Hammond (2004), using the drawdown to a major water-bearing fracture was the common method used prior to that study and was the primary reason for the substantially over-estimated well yields.

After the well was placed in service in late 2010, the reported withdrawal rate from Middletown's monthly operating reports (MORs) was relatively steady at 52–56 gpm (197-212 L/min) until mid-2015. After that time there is a gap in data until mid-2018, when the reported withdrawal rate was 45 gpm (170 L/min). It is unclear whether the yield declined or that use was reduced as the town took more water from the town's spring supply, due to the high flow from those springs during the wet year of 2018. The available withdrawal data indicate that the reliable yield of the well was closer to 50 gpm (189 L/min) estimate from the 72-h aquifer test than the 100 gpm (379 L/min) extrapolation from the 60-d operational test.

D. Fountaindale Community Wells A and B

Fountaindale wells A (FR-88-4859) and B (FR-88-4860) were completed in December 1995. In March 1996, step-drawdown and constant-rate aquifer tests were performed on well A, and a combined step-drawdown and long-term pumping test was conducted on well B.

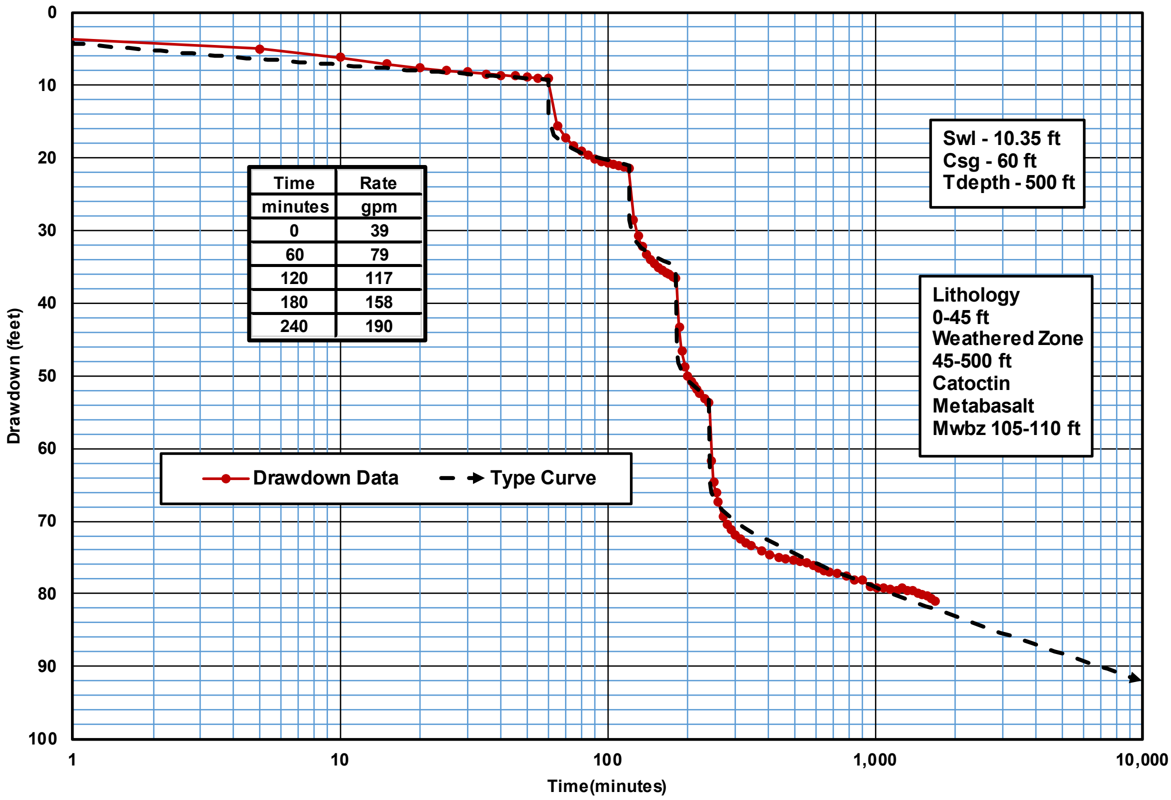

Figure D1a. Fountaindale well A – Semi-log plot of drawdowns from a step-drawdown test, Dougherty-Babu double porosity solution.

The step-test (fig. D1a) for well A started at 39 gpm (148 L/min) and reached 190 gpm (719 L/min) during the final step. The Dougherty-Babu solution provided a good fit to all the data, indicating that there was no dewatering during the test suggesting that the available drawdown was equal to or greater than the drawdown of 82 ft (25 m) that occurred at the end of the test.

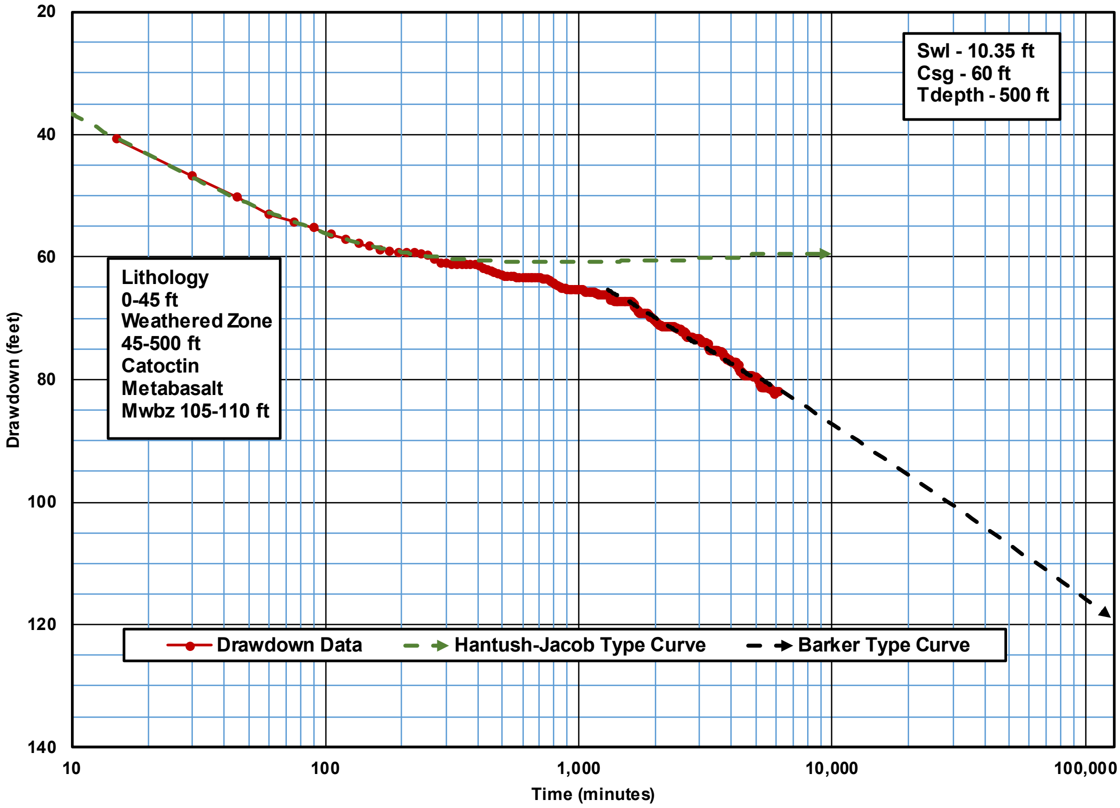

Figure D1b. Fountaindale well A – Semi-log plot of drawdowns from a 103-h, 180 gpm aquifer test: 0−250 min, Hantush wedge-shaped aquifer and Hantush leaky aquifer solutions; 1300−6175 min, Barker General Radial Flow solution.

The 103-h constant-rate test at 173 gpm (655 L/min) also ended with a drawdown of 82 ft (25 m) (fig. D1b). Both tests ended with the water level between the base of the casing at 60 ft (18 m) (equivalent to drawdown of 50 ft [15 m]) and the major water bearing zone at 105 ft (32 m) (equivalent to drawdown of 95 ft [29 m]). No single model provided a good fit to the entire set of drawdown data; however, applying different models to separate portions of the curve did provide acceptable solutions. The Hantush wedge-shaped aquifer and Hantush-Jacob leaky aquifer solutions provided good matches to the early portion of the curve (0-250 min.). The Barker General Radial Flow solution fit the late-time data (1,300-6,175 min.), from which a 90-day extrapolated drawdown of 119 ft (36 m) was derived. When the calculated specific capacity is multiplied by the estimated available drawdown of 82 ft (25 m), the predicted minimum reliable yield is 124 gpm (469 L/min).

The 73-hour aquifer test of well B was essentially a long-term step-drawdown test starting at 181 gpm (685 L/min) and ending at 310 gpm (1173 L/min) (fig. D1c). No model could fit the entire set of drawdown data; however, the Dougherty-Babu solution provided a good fit to the first four steps. There was a sharp break in the drawdown curve below 140 ft (43 m), at the start of the final long-term phase of the test. That accelerated drawdown may have been due to dewatering of a large void at 118−130 ft (36-40m) noted by the site consulting geologist. While there was no evidence of dewatering during the early portion of the test, the initial measured drawdown was below the weathered zone, suggesting that a shallow reservoir unit might have been too rapidly dewatered to be able to identify it as a reservoir unit. For comparison, a similar type of response was observed during the step and aquifer tests of Emmitsburg well 3. The 90-d extrapolated drawdown from the first four steps is 276 ft (84 m). If the specific capacity (0.99 gpm/ft) at that point is multiplied by the available drawdown of 140 feet (43 m), the estimated yield of well B is 139 gpm (526 L/min).

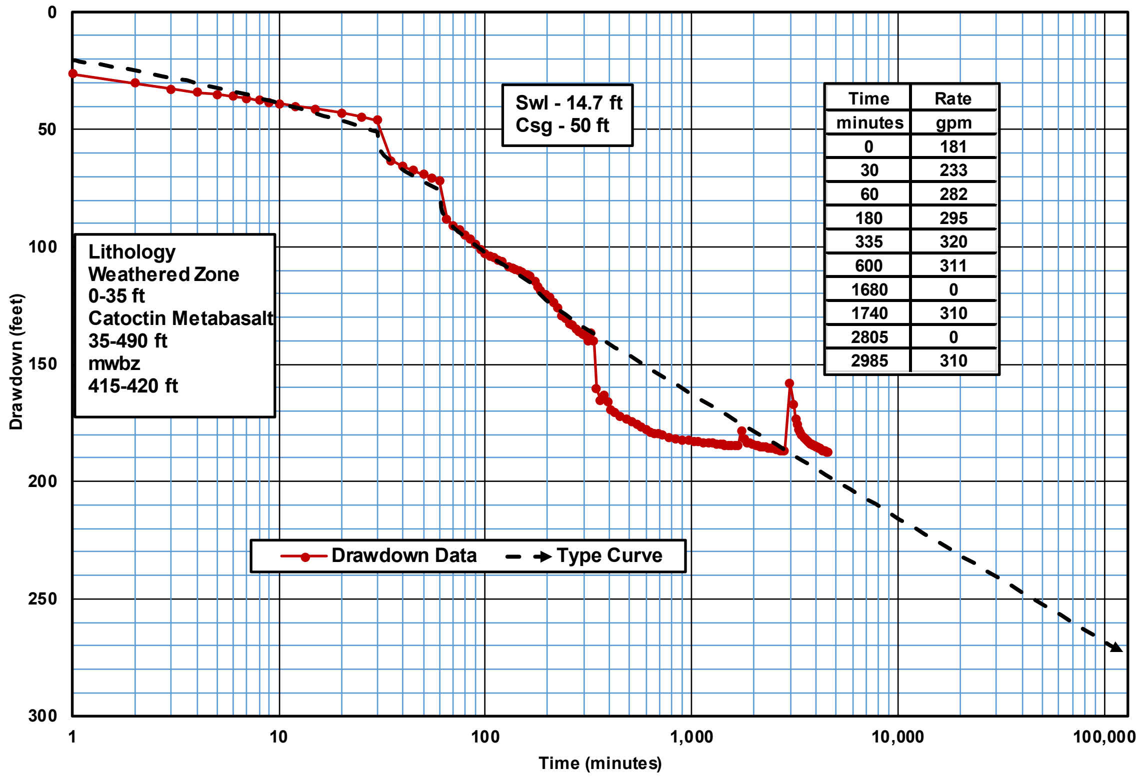

Figure D1c. Fountaindale well B – Semi-log plot of drawdowns from a 73-hour step-drawdown test, drawdowns from the first four steps fit Dougherty-Babu double porosity solution.

As the Myersville water treatment plant test indicate that interference could occur at distances equal to or greater than 1000 ft and Fountaindale wells A and B are located about 1000 feet apart, the potential for significant well interference must be considered. Since water levels in observation wells were not measured during either test, the best way to determine interference is by long-term monitoring of withdrawals from the wells. Pumpage reports indicate that the maximum amount of system use was 314 gpm (1189 L/min) in October 2012. This may not indicate the reliable yields of wells A and B, since there are eight other, but low yielding, wells in the system. Monthly operating reports (MORs) indicate that well A was out of service in 2014–2015, during which well B could produce the equivalent of 111 gpm (420 L/min), if operated 24 h/d, and the remaining wells could produce the equivalent of 69 gpm (261 L/min), if operated 24 h/d. It is then possible that well A could have produced the equivalent of 134 gpm (507 L/min), if operating 24 h/d in October 2012. While this analysis is limited by the available data, it appears the initial combined yield of the two wells was reasonable and the effects of interference were minimal.

E. Musket Ridge Golf Club Well 17

A step-drawdown test was performed on the Musket Ridge Golf Club well 17 (FR-94-1822) in February 2000. The drawdown data started to deviate from the Dougherty-Babu solution after the fifth step, at a drawdown of 65 ft (20 m) (fig. E1a). During the follow-on 72-h variable rate aquifer test, performed at 200/150 gpm (757/568 L/min), there was a clear break in the drawdown curve after 55 minutes, with a drawdown of 63 ft (19 m) (fig. E1b). A much greater change in slope occurred at 170 minutes, at 84 ft (26 m) of drawdown. Both breaks in the curve occur within about 10 ft (3 m) of the two major water bearing zones identified by the consulting geologist. After the second break several rate adjustments were made to maintain the drawdown above about 80 ft (24 m), to maintain the water level above the deeper fracture.

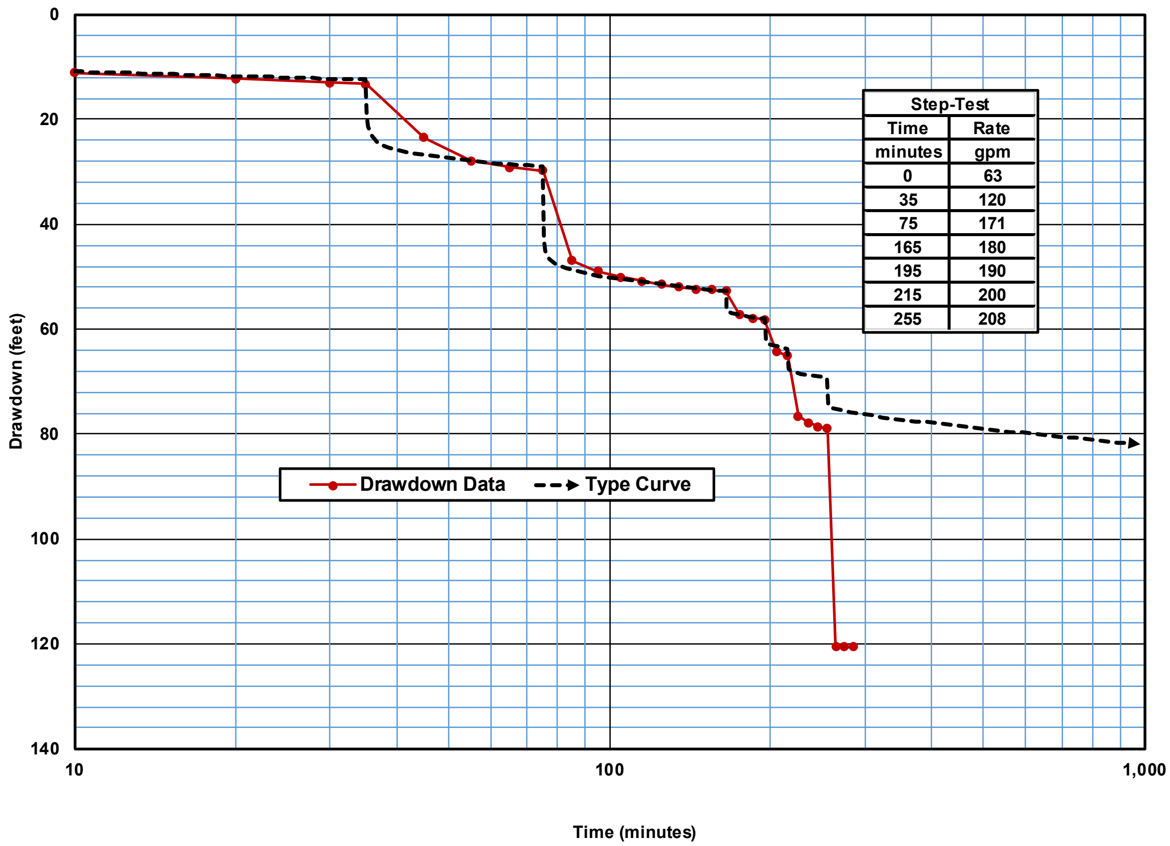

Figure E1a. Musket Ridge GC well 17 – Semi-log plot of drawdowns from a step-drawdown test, Dougherty-Babu double porosity solution fit to the first five steps.

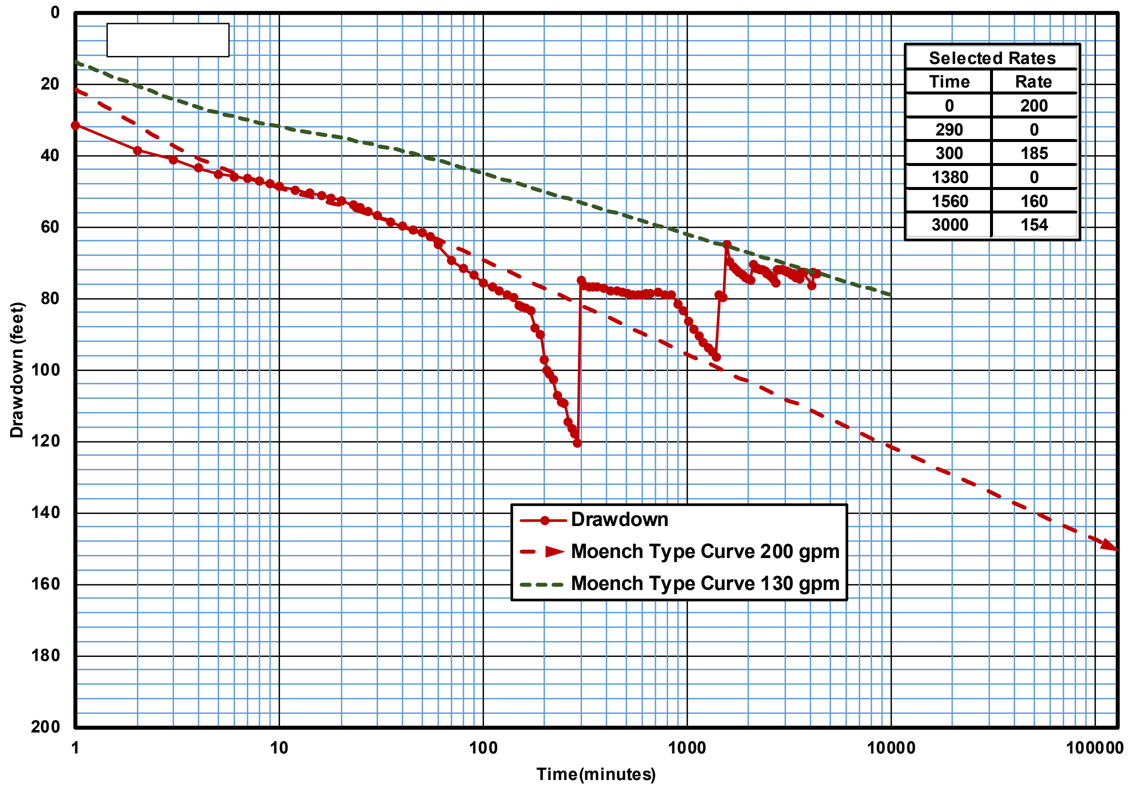

Figure E1b. Musket Ridge GC well 17 – Semi-log plot of drawdowns from a 72-h variable-rate aquifer test, Moench Double Porosity solution (0-55 min) and with estimated drawdowns if the test was conducted at 130 gpm.

In this case, an extrapolation from the Moench double porosity solution from the early portion of the test produced a 90-d extrapolated drawdown of 151 ft (46 m), which produces an estimated yield of 85 gpm (321 L/min) at the drawdown to the first wbz (64 ft) or 111 gpm (365 L/min) at the drawdown to the second wbz. If the average of 147 gpm (556 L/min) for the first five steps of the step test were used during the 72-h test, the drawdown may not have reached the first break point until much later in the test. If dewatering of the fractures did not occur or happened late in the test, then the longer period of steady-state drawdown would have helped produce a better estimate of the well yield. Using 130 gpm (492 L/min) or 88 percent of the average step rate prior to any deviation from a type curve may have ensured that erratic late time declines in water levels did not occur during the follow-on aquifer pumping test.

There are no monitoring data available to confirm the estimated reliable yield of the well, but the case study is offered as an example of how step test data can be used to best determine optimum rates for aquifer tests.

F. Point of Rocks Community Wells N and M

Figure F1a shows the locations of proposed production wells N and M, at Point of Rocks, which were completed in the Tomstown Dolomite. Several aquifer tests were completed on Point of Rocks wells N and M between April 1997 and December 1999. Figure F1b is a semi-log plot of drawdowns taken during the initial tests of those wells. The first test of well N, conducted in April 1997 at 100 gpm (379 L/min), indicated it was probably a good well; however, due to the limited drawdown, a second test was conducted in February 1999 to determine if well N could produce more water. During that test there was a break in the water-level data after about 1900 minutes of pumping, at a pumping water level of 36 ft (11 m) below measuring point.

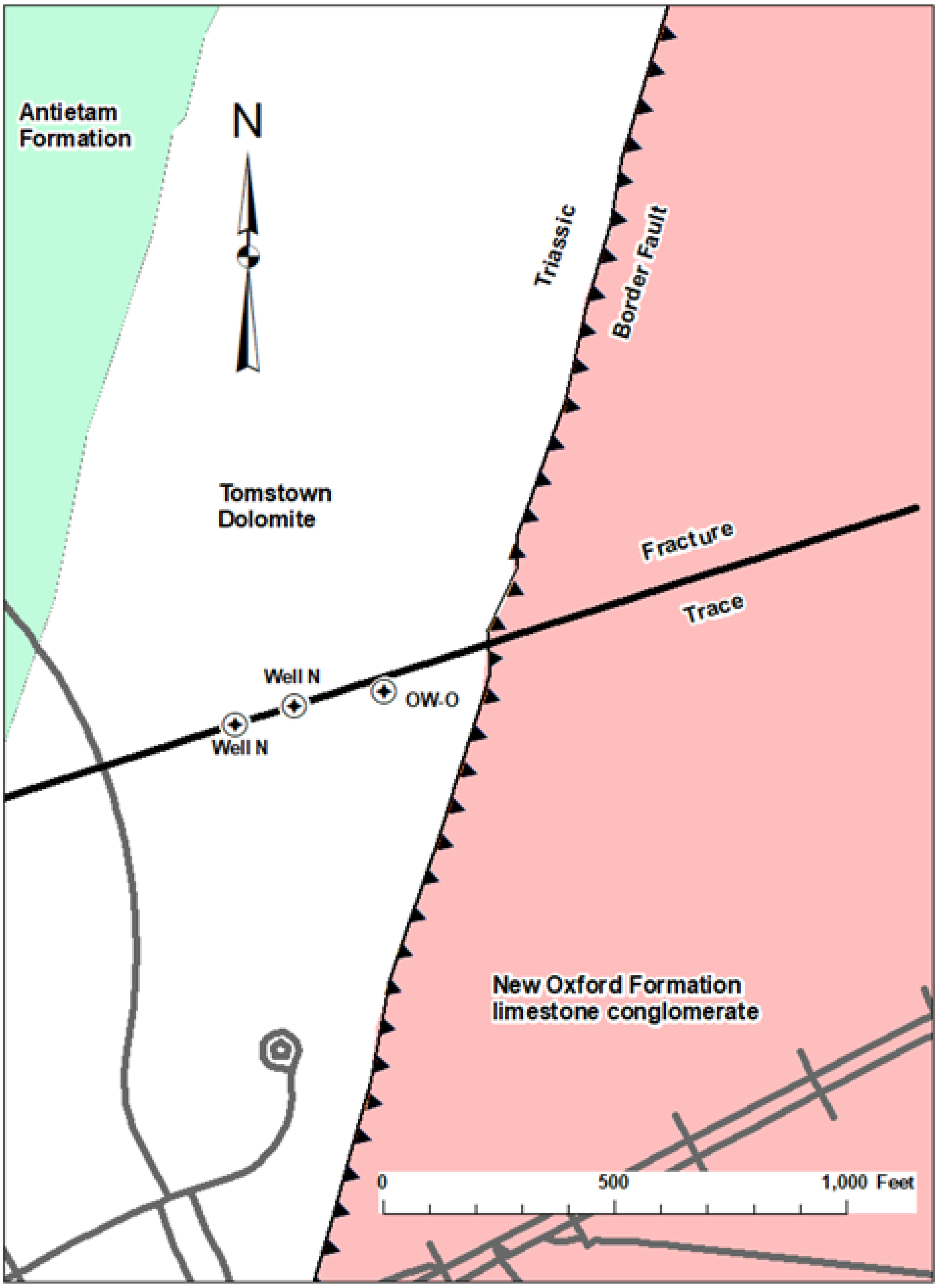

Figure F1a. Geologic map. Point of Rocks test site.

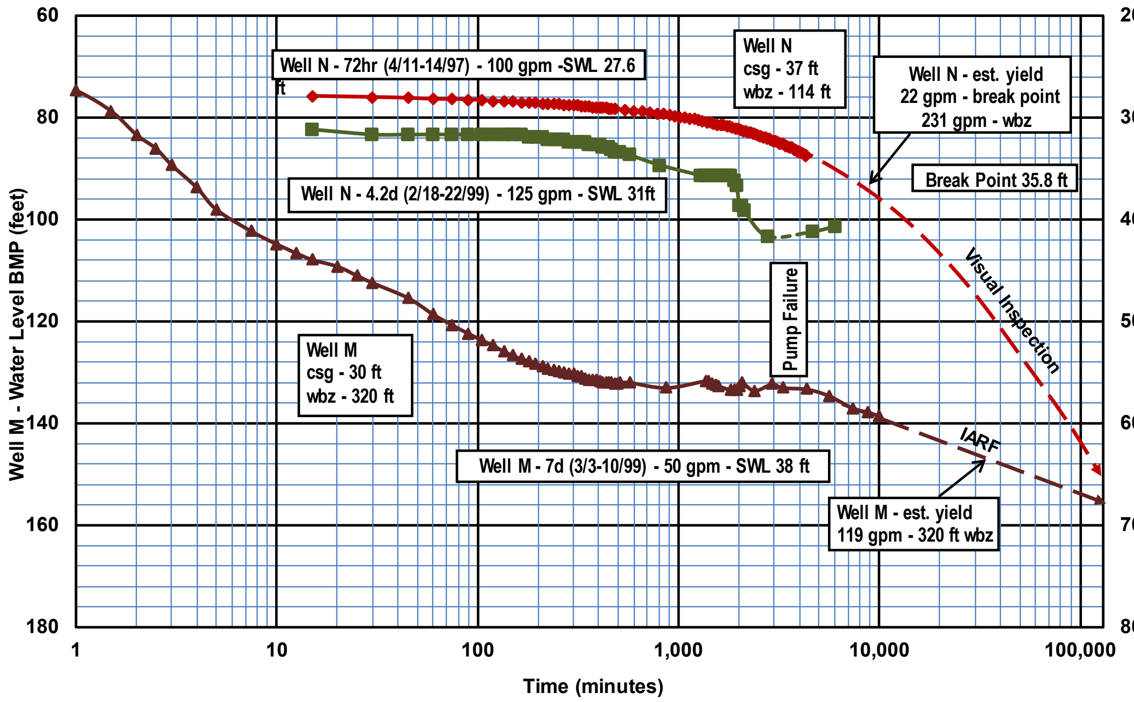

Figure F1b. Point of Rocks wells N and M – Initial aquifer tests, semi-log plot of drawdowns (BMP – below measuring point).

During a 1999 analysis, before the techniques in this study were developed, data from the 1997 test were used to estimate the reliable yield of well N; however, since the water level did not stabilize as a straight line on either radial or linear flow diagnostic plots, the extrapolation was made using a visual approximation of a drawdown curve that would fit between those two plots. The extrapolation was extended to 129,600 minutes and the resulting specific capacity (0.37 gpm/ft) was applied to the drawdown of 8 ft (2.4 m) that would have occurred at the break observed at 36 ft (11 m) during the second test. This produced an estimated yield for well N of 22 gpm (83 L/min). When the same data were extrapolated to the water-bearing zone at 114 ft below measuring point (equivalent to drawdown of 86 ft [26 m]), the estimated yield is 232 gpm (878 L/min).

The drawdown data from the March 1999 test of well M produced an estimated yield of 119 gpm (450 L/min), when water levels were extrapolated from an IARF segment to the water-bearing zone at 320 ft below measuring point (equivalent to drawdown of 282 ft [86 m]) in that well. The drawdown in well M (5 ft [1.5 m]) was nearly identical to that in well N (6 ft [1.8 m]) at the end of the first test of well N, indicating that the two wells were directly, hydraulically connected by a single, prominent, vertical fracture. It is likely that such strong interference would result in the total yield of the two wells being no more than the yield of the best well. The drawdown in well O was 1.3 feet at the end of the test and the well was only 25 feet from the fracture trace, indicating that there was a highly anisotropic aquifer in the vicinity of the test wells.

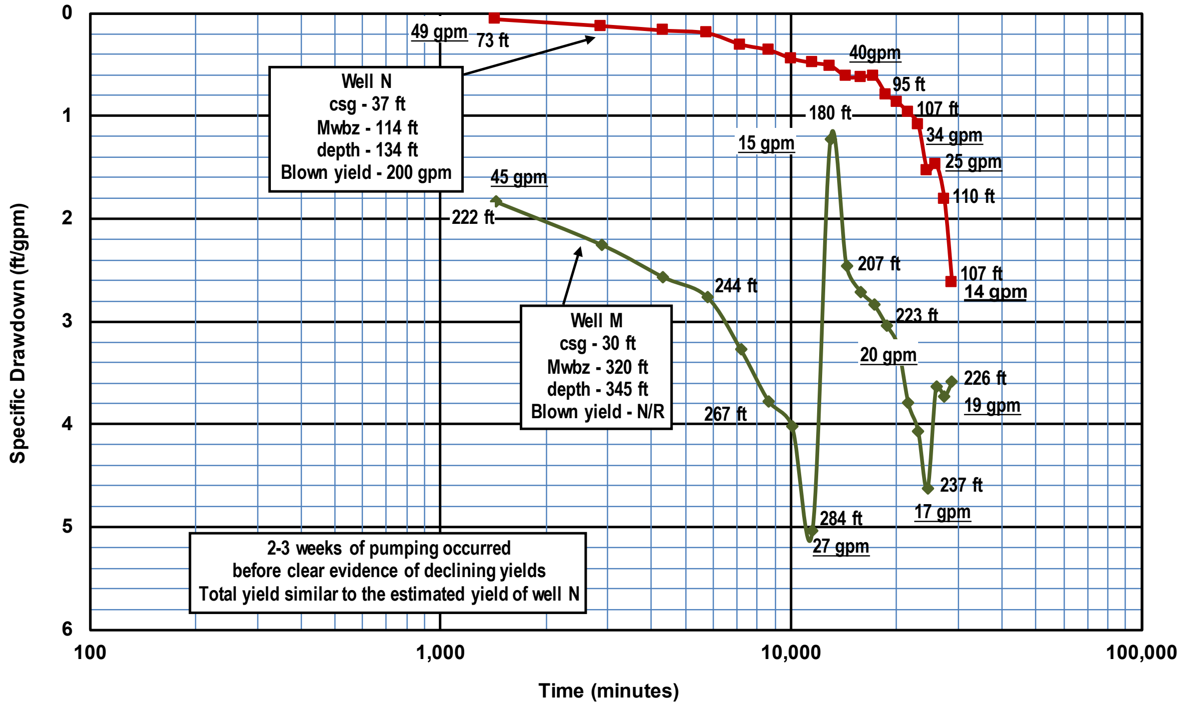

Figure F1c. Point of Rocks wells N and M – Semi-log plot of drawdowns from a **long-term pumping test.**

The water purveyor wanted a longer-term test (20 days in Nov-Dec 1999) of wells N and M to verify the sustainability of the water supply. The data from that test are presented in Figure F1c. The initial total test rate was 94 gpm (356 L/min) and, after 2-3 weeks of pumping, there was clear evidence of declining yields. The final total test rate of about 31 gpm (117 L/min) was only about the same as the estimated yield for well N, when extrapolating the drawdowns to the shallow break noted during the second individual test of that well. The water levels, however, in both wells at the end of the long-term test were not stable, suggesting an even lower sustained yield. Since the final total yield was only a fraction of the previously estimated yield of well M, this suggests that there was little contribution of flow to that well from the bedrock portion of the aquifer. The Triassic Border Fault, (Cleaves et al., 1968) is located about 400 ft (122 m) east of well N. The potential presence of a sealing fault could act as a no flow boundary. This could provide an explanation for the sharp declines in water levels noted in the production data from well N and M during the long-term test. Such a boundary was not detected during the short-term pumping tests since the typical doubling of the late-time slope for an impervious barrier was not observed during any of those tests.

A step-drawdown test was conducted using well N, but the data were not available for the present investigation. The consultant indicated that the specific capacity increased with each increase in the pumping rate, so it is possible that any data would not have been usable for determining well losses or efficiency.

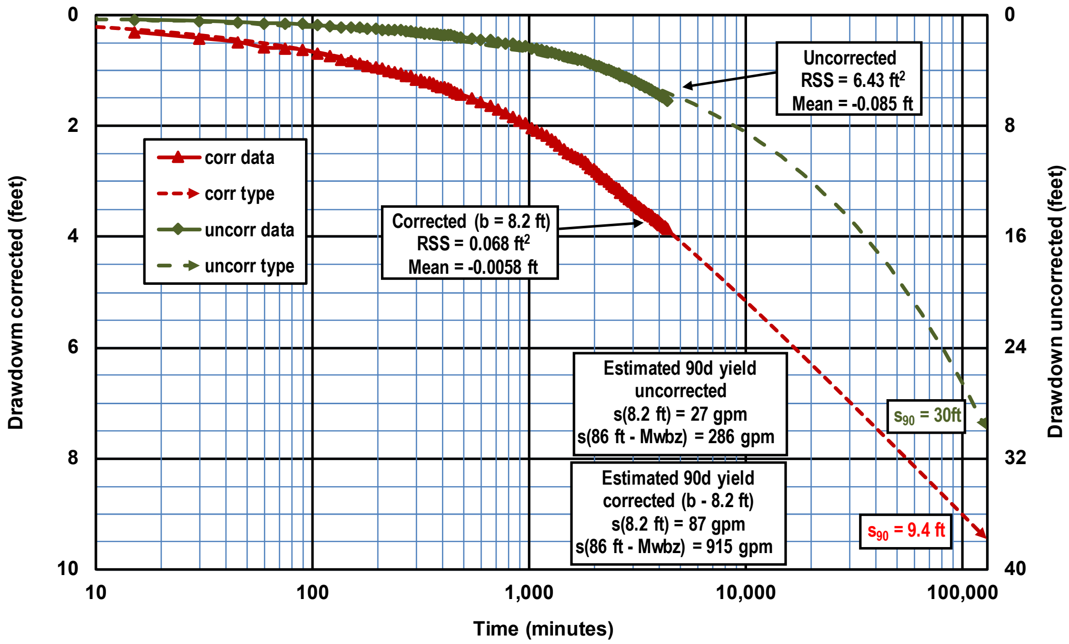

Figure F1d. Point of Rocks well N – Semi-log plot, drawdown data and SVF type curves for a 72-h, 100 gpm test; data is both uncorrected and corrected for an aquifer thickness *b* of 8.2 feet.

Figure F1d is a semi-log plot of the results of the 72-hour test of well N. The uncorrected data indicate that the drawdowns did not stabilize during the testing in an IARF response, but the extrapolated drawdown (30 ft or 9 m) was very similar to that performed by visual inspection (37 ft or 11 m) during the 1999 evaluation of that test and produces an estimated yield of 27 gpm (102 L/min). The Gringarten-Witherspoon solution for a single vertical fracture (SVF) provided the best fit to the drawdown data, when corrected for the effects of dewatering using an aquifer thickness *b* of 8.2 feet, as determined by the break in the drawdown curve that occurred during the follow-on 4.2-day test. An early linear flow period was followed by an IARF (straight-line) segment, which is the typical response for flow from a vertical fracture. This would also confirm the previous observation that wells N and M were hydraulically connected by a prominent vertical fracture. Extrapolating from the late-time, corrected data on the semi-log plot produced an estimated yield of 87 gpm (330 L/min). The extrapolation of the uncorrected data produced a yield (27 gpm or 102 L/min), which was close to the combined long-term test of wells N and M (31 gpm or 117 L/min, but declining). These results provide further evidence that aquifer dewatering must be considered when evaluating the yield of a fractured rock well. The wells were eventually abandoned prior to being placed into service, so there are no operational data that could be used to verify the ultimate reliable yield of the wells.

G. City of Westminster

G1. Well 6 (South Center Street)

Westminster's South Center Street well 6 (CL-81-0249) was completed in May 1983, and step-drawdown and aquifer tests were conducted in September 1983. The Dougherty-Babu solution provided a good fit to the step-drawdown test data (fig. G1a) and the Neuman-Witherspoon two aquifer model best simulated the follow-on 72-hour, 82.5 gpm (312 L/min) aquifer test (fig. G1b). During both tests there was no evidence of dewatering of a permeable zone, suggesting that the available drawdown would equal or exceed the final drawdown of 85 ft (26 m) (SWL = 0 ft or 0 m) during the long-term test. While there was a good fit to the step-drawdown data and no clear evidence of dewatering, there were excessive well losses, producing a low well efficiency of 23 percent. This may be due to increased turbulent flow because of a screen installed in the well. The top of the screen was set at 90 ft (27 m), which would limit the drawdown to that level and with a 90-d extrapolated drawdown of 92 ft (28 m), the estimated reliable yield is 81 gpm (307 L/min).

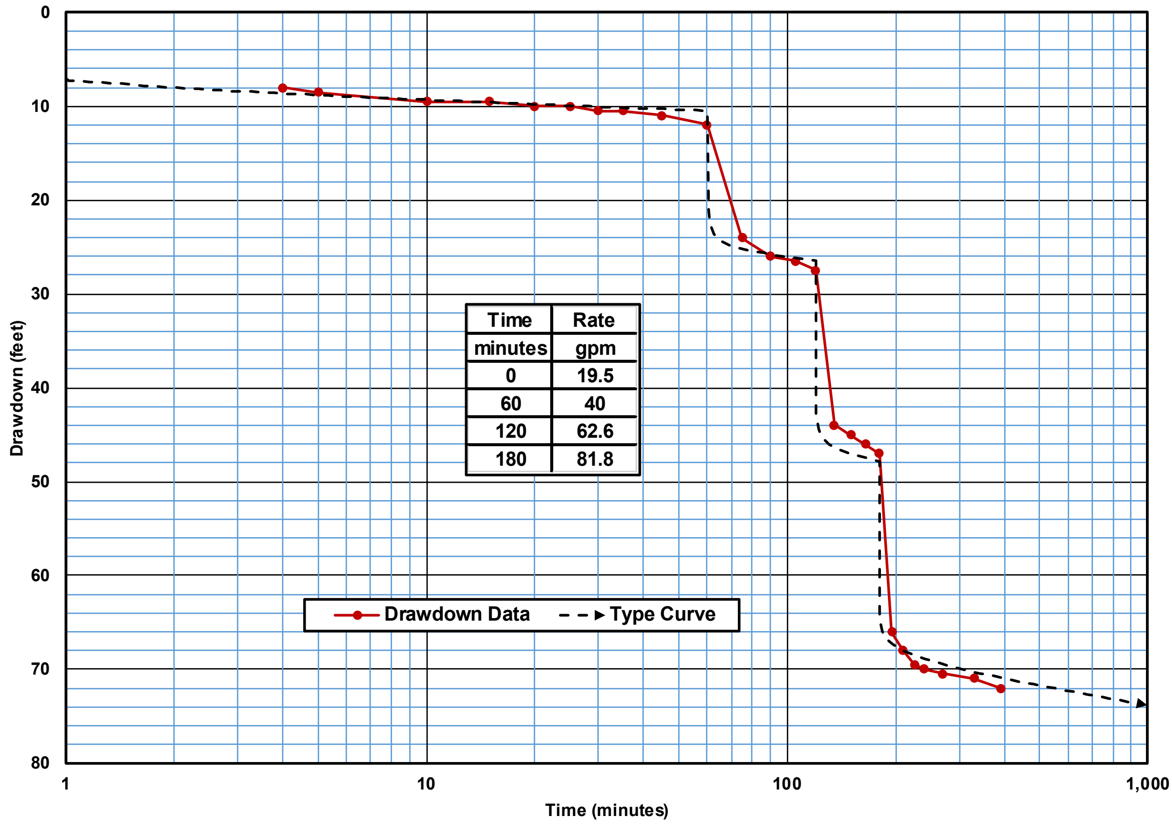

Figure G1a. Westminster well 6 (South Center Street) – Semi-log plot of drawdowns from a step drawdown test, Dougherty-Babu double porosity solution.

In 1995, the maximum reported use was 106,217 gpd (402 m^3/d) avg. (74 gpm, or 280L/min, avg.) and 121,622 gpd (460 m^3/d) max. (84.5 gpm, or 320 L/min, max.). The maximum use was like the estimated yield of 81 gpm. Since then, the maximum reported use was 99,420 gpd (376 m^3/d) avg.

(69 gpm, or 261 L/min, avg.) and 102,955 gpd (390 m^3/d) max. (71.5 gpm, or 271 L/min, max.) in 2014. The well produced 63 gpm (238 L/min) with a water level of 106 ft in (32 m) Nov−Dec 2001 of the 2001-2002 drought, which was like the 63 gpm (238 L/min) and a water level of 115 ft (35 m) during the non-drought period 2009−2016. Conversely, the pumping rate was also 63 gpm (238 L/min) in 1998, but the water level was 81 ft (25 m). These data indicate that there may have been a decrease of about 25% between 1995 and 2009-2016 in the well yield over time due to lowering the water level below the top of the screen.

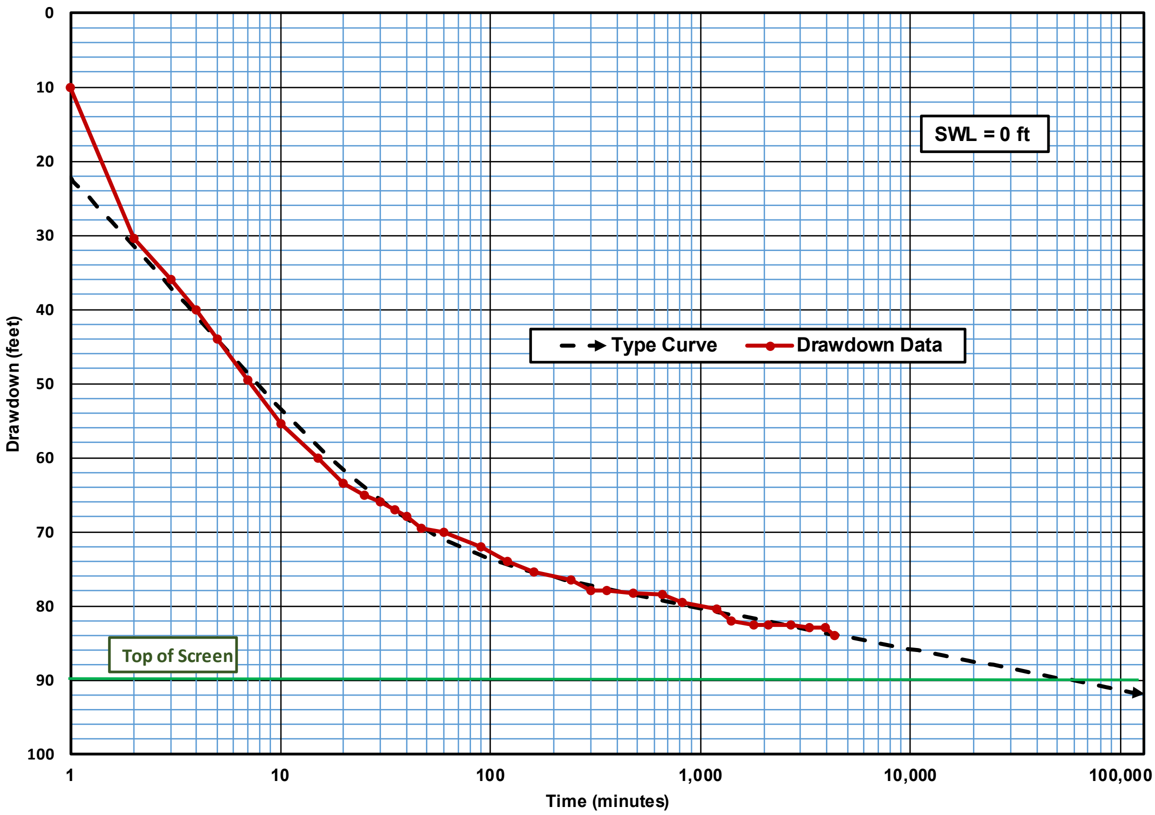

Figure G1b. Westminster well 6 (South Center Street) – Semi-log plot of drawdowns from a 72-h 82.5 gpm aquifer test, Neuman-Witherspoon two aquifer solution.

G2. Well 7 (Carfaro)

During drilling of the Westminster's Carfaro well 7 (CL-81-2458), numerous sinkholes developed and the well had a high turbidity problem. A 10.9-d step test with 6 rate steps was performed in 1986 (fig. G2a). The Dougherty-Babu solution fits the data from the first four steps, but there is a substantial deviation from the curve when drawdown reached 26 ft (8 m) at about 5,300 minutes, after the start of the 437 gpm (1654 L/min) step. This water level was near the casing depth, suggesting that dewatering of the weathered transition zone had occurred. In addition, the high turbidity could have caused clogging of the pump intake screen, producing the excessive drawdown. In either case, this would indicate that there may be a limited amount of available drawdown. Using the s_{90} = 35 ft (11 m) from the Dougherty- Babu solution, the estimated yield would be 189 gpm (715 L/min).

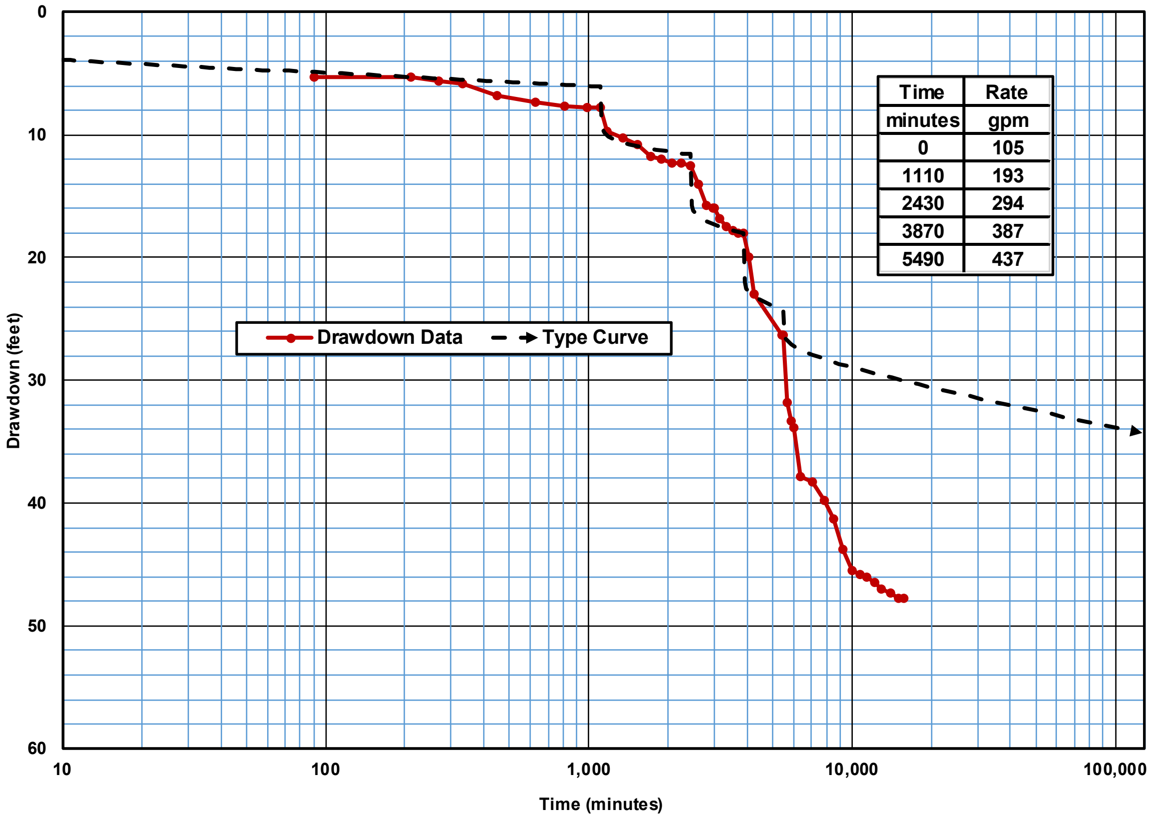

Figure G2a. Westminster well 7 (Carfaro) - Semi-log plot of drawdowns from the first four steps of a 10.9-d step-drawdown) test, Dougherty-Babu double porosity solution.

Although the testing was conducted in 1986, the well was not placed in full service until late 2001. The pumpage reached a peak of 249,365 gpd (944 m^3/d) avg. (173 gpm, or 655 L/min, avg.) and 274,657 gpd (1040 m^3/d) max (191 gpm, or 723 L/min, max.) in 2004. The reported use has since declined, with a maximum use of 157,495 gpd (596 m^3/d) avg (109 gpm, or 413 L/min, avg.) during the past 10 years. Water levels in late 2001 were an average of 30.5 ft (9 m), while pumping at an average of 93 gpm (352 L/min). During the non-drought period of 2009–2016, the average water level was 45 ft (14 m), while pumping at an average of 105 gpm (397 L/min), or a 27-percent decline in calculated specific capacity relative to 2001. The decline might be even higher since the comparison was between drought and non-drought periods.

G3. Well 8 (Vo-tech)

Westminster's well 8 (Vo-tech well) was completed in the crystalline rock Ijamsville Formation. The consultant described the lithology in the well as being heavily weathered to a depth of 55 ft (17 m). Figure G3a is a semi-log plot that shows the results of two aquifer pumping tests. During the first test (August 1987), the well was pumped for 96 hours at 400 gpm (1,514 L/min), producing a curvilinear drawdown response, which was originally interpreted by the present author as being due to either or both linear/pseudo-radial flow and dewatering of the weathered zone. During that test, several shallow water-bearing zones were dewatered between 30.8 ft and 54 ft (9 m and 16 m) BTOC, which probably produced the high turbidity noted by the consultant. The shallow fractures were then cased off and the second, 14-day, 205 gpm (776 L/min) test was conducted in January 1988. The response during that test appeared to be one caused by a delayed yield effect. A plausible explanation for the two different responses is that during the first test the shallow, water-bearing zones were directly connected by a short-circuit through the well-bore to the primary fracture system; while during the second test, water from the then isolated shallow zones had to leak into the bedrock fracture system and then flow to the well-bore.

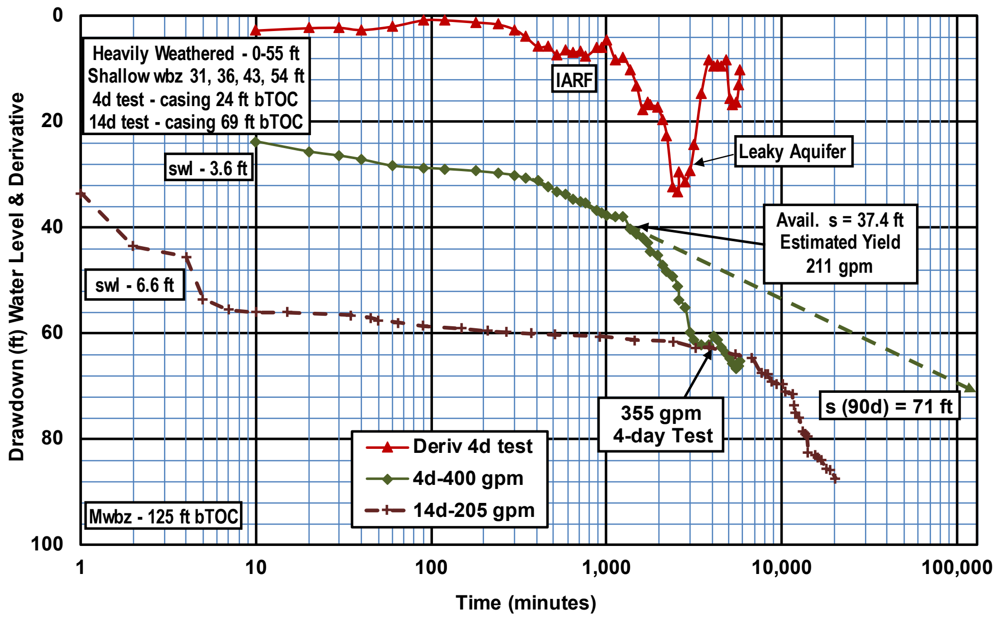

Figure G3a. Westminster well 8 (Vo-tech) – Semi-log plot of drawdown and its logarithmic derivative for 4-d and 14-d pumping tests, with estimated yield based on 90-d extrapolation from IARF segment.

The derivative analysis of the first test indicated that an IARF segment began after about four hours of pumping and lasted for about 13 hours. This was followed by a leaky aquifer response at a drawdown of 37.4 ft (11 m). The end of the test was affected by aeration in the flowmeter reportedly due to clogging of the pump intake screen, which required a reduction in the pumping rate to 355 gpm (1,344 L/min). Based on the second test, it appeared that the well could supply a reliable yield of 200 gpm (757 L/min). In preparing the present report, the drawdown was extrapolated from the IARF segment to 129,600 minutes and the calculated specific capacity at that point was applied to an available drawdown of 37.4 ft (11 m), producing an estimated yield of 211 gpm (800 L/min). The derivative data calculated from the 14-day test were somewhat erratic, but tended towards 0 m, typical of a constant head boundary. The Neuman-Witherspoon (1969) leaky aquifer model provided the best fit to the early data (first 6,800 minutes). The drawdown during the remainder of the test was probably due to dewatering of the bedrock portion of the aquifer. Under other circumstances, it could also represent the effects of a no-flow boundary, but there was no typical doubling of the slope of the drawdown curve for that type of response.

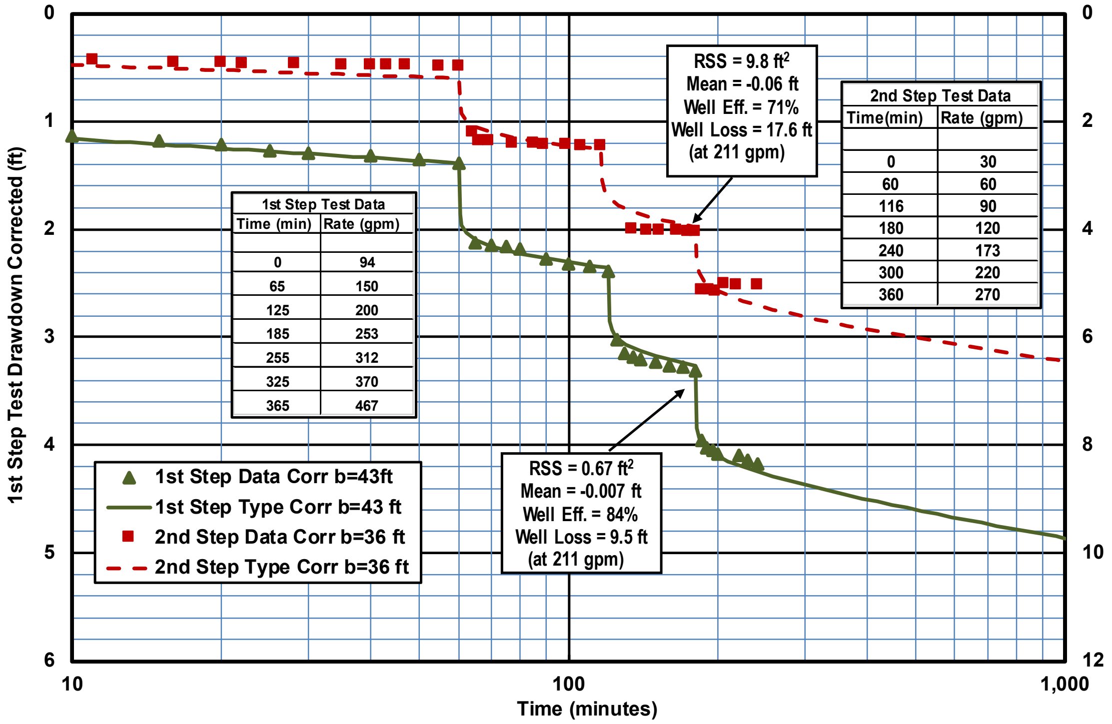

Figure G3b. Westminster well 8 (Vo-tech) – Semi-log plot of drawdowns from the 1st step-drawdown test, data is both uncorrected and corrected for aquifer thickness *b* of 43 ft, and the 2nd step-drawdown test, data is both uncorrected and corrected for aquifer thickness *b* of 36 ft, Dougherty-Babu double porosity solution.

Step-drawdown tests were conducted prior to each of the two aquifer tests of well 8, both at seven different rates. Application of the modified Dougherty-Babu (1984) solution to the data from the first test produced a poor visual fit, but good statistical results, with a RSS of 4.6 ft^2 and a mean of -0.06 ft. At the estimated reliable yield of 211 gpm (800 L/min), the well efficiency was 40 percent, and the well loss was 14.8 ft. A better visual and statistical fit (RSS of 0.67 ft^2 and a mean of -0.007 ft) to the data was achieved by correcting them for the effects of dewatering using a saturated thickness of 43 ft (13 m) (fig. G3b). This produced a well efficiency of 84 percent at 211 gpm (800 L/min) and well loss of 1.3 ft. Application of the Dougherty-Babu solution to the data from the second step-drawdown test produced good visual and statistical results, with a RSS of 41 ft^2 and mean of -0.009 ft; however, the calculated well efficiency was 15 percent with a well loss of 77 ft. The best fit to those data was achieved by correcting it for the effects of dewatering and using a saturated thickness of 36 ft (11 m). This resulted in a RSS of 9.8 ft^2 and mean of −0.06 ft, producing at 211 gpm (800 L/min) with a well efficiency of 71 percent and well loss of 2.9 ft. The results were similar after correction for dewatering effects whether English or S.I. units were used; however, there was no match on simulations conducted using the uncorrected drawdown data. These results suggest that clogging of the pump intake and dewatering of the aquifer both had an effect of the well efficiency, although the efficiency was still relatively high and well losses were small relative to the available drawdown 37.4 ft (11 m). Casing off the upper, water-bearing zones may have increased the well efficiency by reducing the turbidity and clogging of the pump intake.

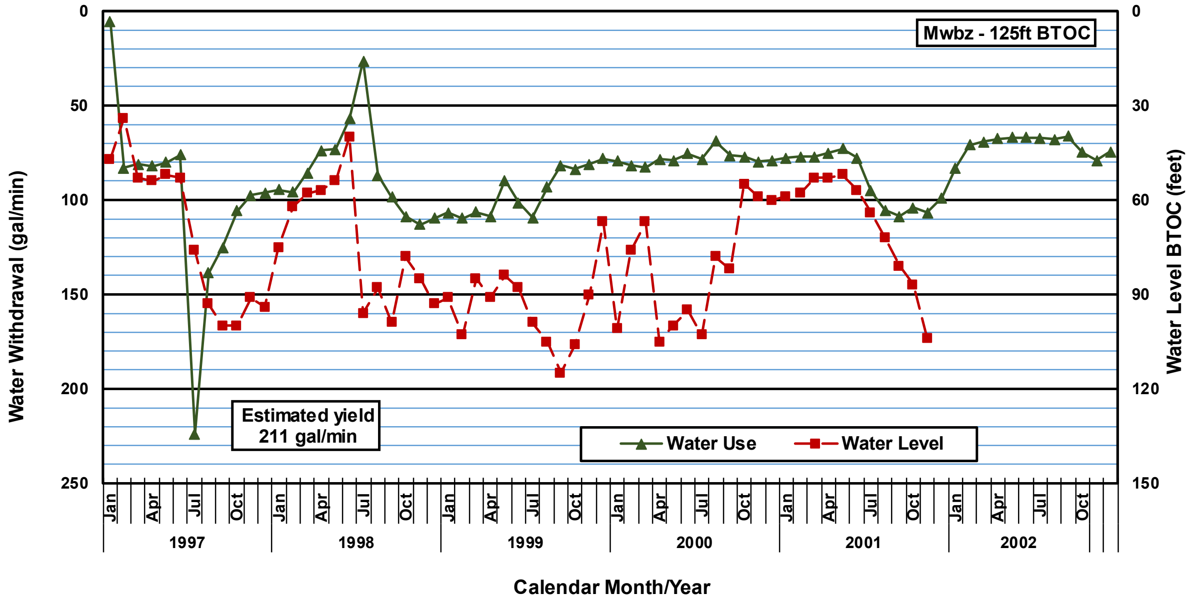

Figure G3c. Westminster well 8 (Vo-tech) – Water-withdrawal and water-level data.

After the well was placed in service, detailed production and water level records were collected for the period 1997−2002, as shown in Figure G3c. These data indicate that the initial reliable well production of 224 gpm (1244 m^3/d) matched the estimated yield; however, over time, production declined to about 70 gpm to 110 gpm (400 m^3/d to 600 m^3/d). Levels in nearby monitoring wells indicate that regional water levels were generally below average for the period from mid-1998 through 2002. The most likely explanation for the declining production is that a substantial amount of groundwater was initially taken out of storage, producing the high initial yield. The yield subsequently declined due to limited potential recharge in the capture zone of the well (topographic drainage area of 148 acres or 60 hectares), and the prolonged effects of the drought.

G4. Well 11 (Roops Mill)

A multi-rate test and a 74-hour, 110 gpm (416 L/min) test of the Westminster Roops Mill well 11 (CL-94-369) were completed in July 2001 (figs. G4a and G4b). The first of two rates of the multi-rate test was 130 gpm (492 L/min), followed by a recovery period. The recovery times and water levels were not recorded, so it was assumed that the period was 50 minutes or ½ of the initial 100-minute step. The second step rate was 260 gpm (984 L/min). Notwithstanding the limited number of steps, the Dougherty-Babu solution provides a reasonably good fit to the drawdown data, with some evidence of dewatering during the last few feet of drawdown when the upper part of a potential limestone reservoir unit may have been dewatered (70−105 ft, or 21-32 m, BTOC).

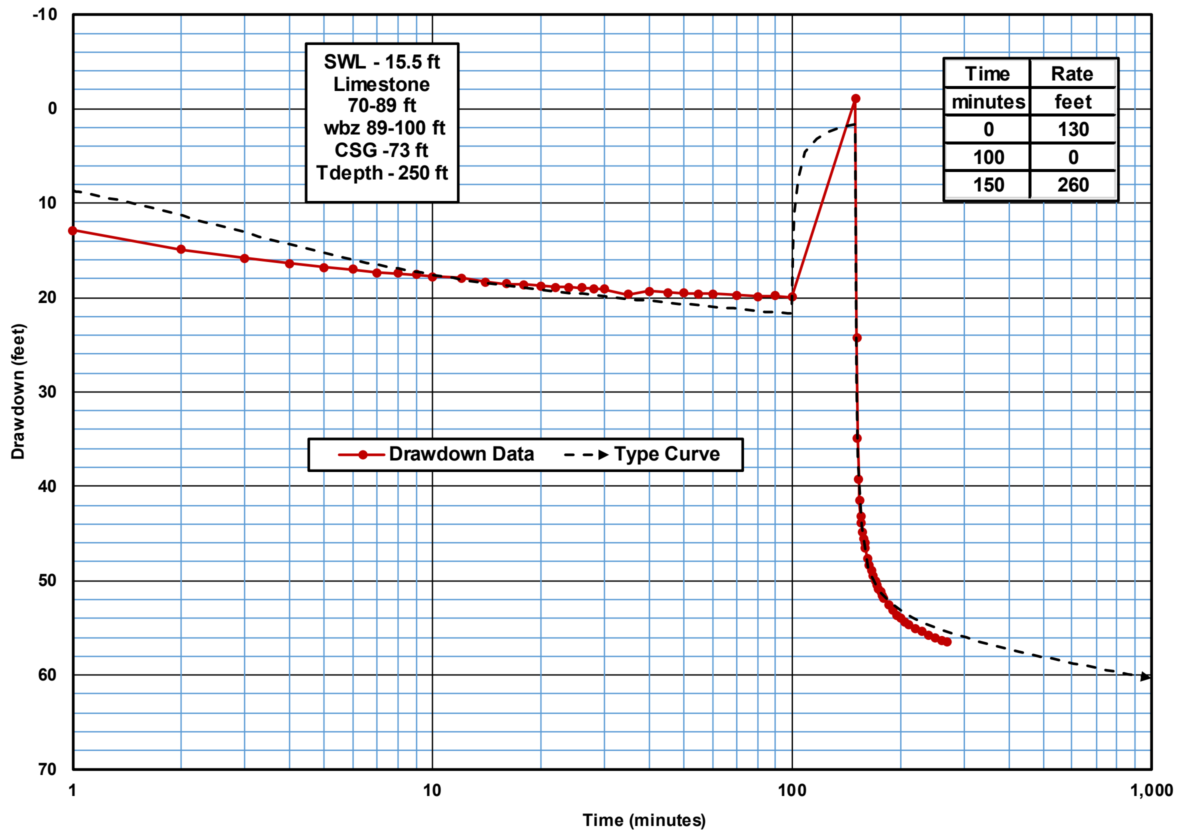

Figure G4a. Westminster well 11 (Roops Mill) – Semi-log plot of drawdowns from a multi-rate test, Dougherty-Babu double porosity solution.

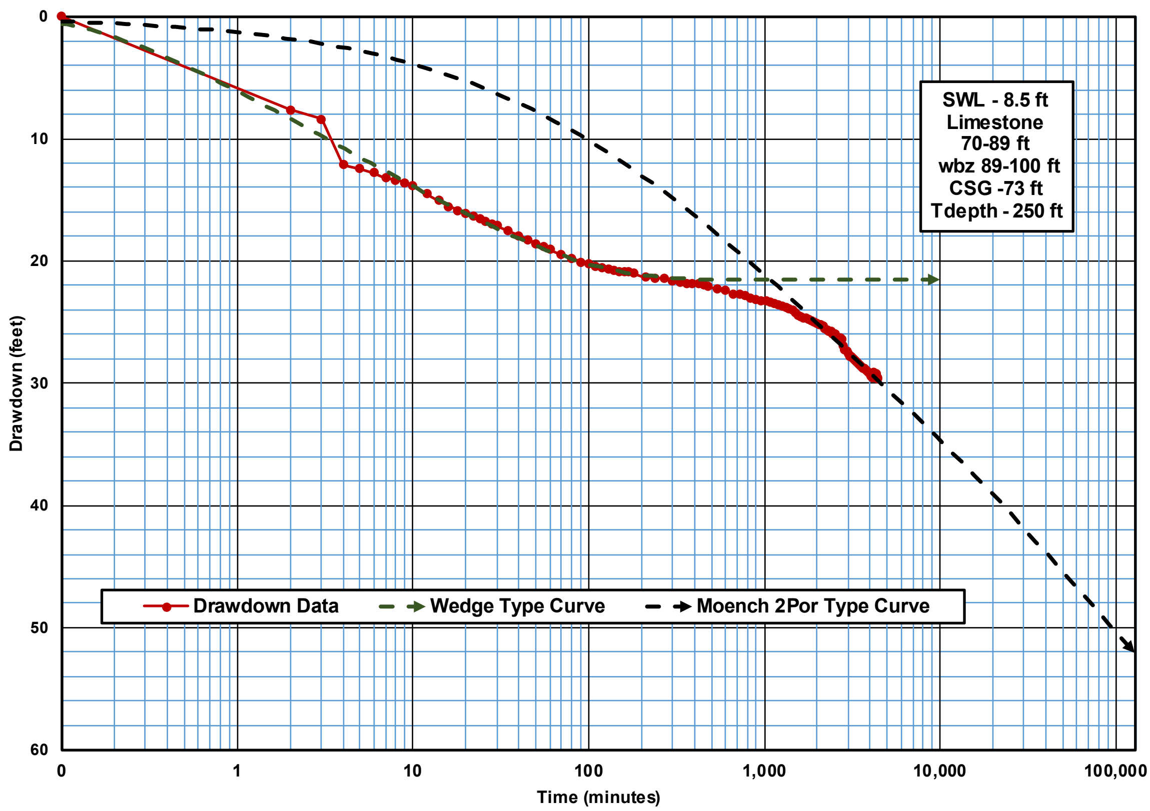

Figure G4b. Westminster well 11 (Roops Mill) – Semi-log plot of drawdowns from a 74-h, 110 gpm aquifer test, Hantush wedge aquifer and Moench double porosity solutions.

There appear to be two segments on the drawdown curve for the 74-hour test. The first occurs during the first 500 minutes, where the Hantush wedge aquifer solution provides a good match to the drawdown data. The second occurs after 1,500 minutes, where the Moench dual porosity model provides the best fit to the data. An extrapolation of the second solution to 90 days results in a drawdown of 52 ft (15 m). Using the drawdown (61.5 ft or 19 m) to the top of the limestone unit, the estimated reliable yield of the well is 130 gpm (492 L/min).

The maximum reported use was 126,927 gpd (480 m^3/d) avg. (88 gpm, or 333 L/min, avg.) and 134,835 gpd (510 m^3/d) max. (94 gpm, or 356 L/min) in 2011, while the average water level was 56 ft (17 m). If the water level had been drawn down to the top of the limestone unit at 70 ft (21m), it is estimated the well could have pumped a maximum of 121 gpm (458 L/min) or about 93% of the estimated yield. Since 2011 use steadily declined to 69 gpm (261 L/min) avg / 72 gpm (273 L/min) by 2017, with the yearly average water levels (70 – 86 ft [21-26 m]) equal to or deeper than the top of the limestone unit. In 2018, the use was even less (54 gpm [204 L/min] avg. / 64 gpm [242 L/min] max.), after the average water level (67 ft [20 m]) was raised above the limestone unit. These water withdrawal data suggest that dewatering of the limestone unit may have caused either compaction or calcite cementation to close fractures and may have permanently decreased the yield of the well.

H. Town of Mount Airy

H1. Well 6A (Gillis Falls)

Mount Airy (Gillis Falls) well 6A was completed in 2007 and was the only one of the 28 wells drilled for the project that had a thick (29−94 ft or 9−29 m), heavily fractured, highly weathered zone, as well as a 200 gpm (757 L/min) blown yield. No step-drawdown test data could be found for this well. Three aquifer pumping tests were conducted; the first was for 72 hours at variable rates in November 2008 and the second ran for 72 hours at 164 gpm (621 L/min) in April 2008. After that test, a 90-d test with a continuously declining rate was performed from May to June 2008. After the long-term test, the third aquifer test was a 96-h, 200 gpm (757 L/min) test performed in October 2008.

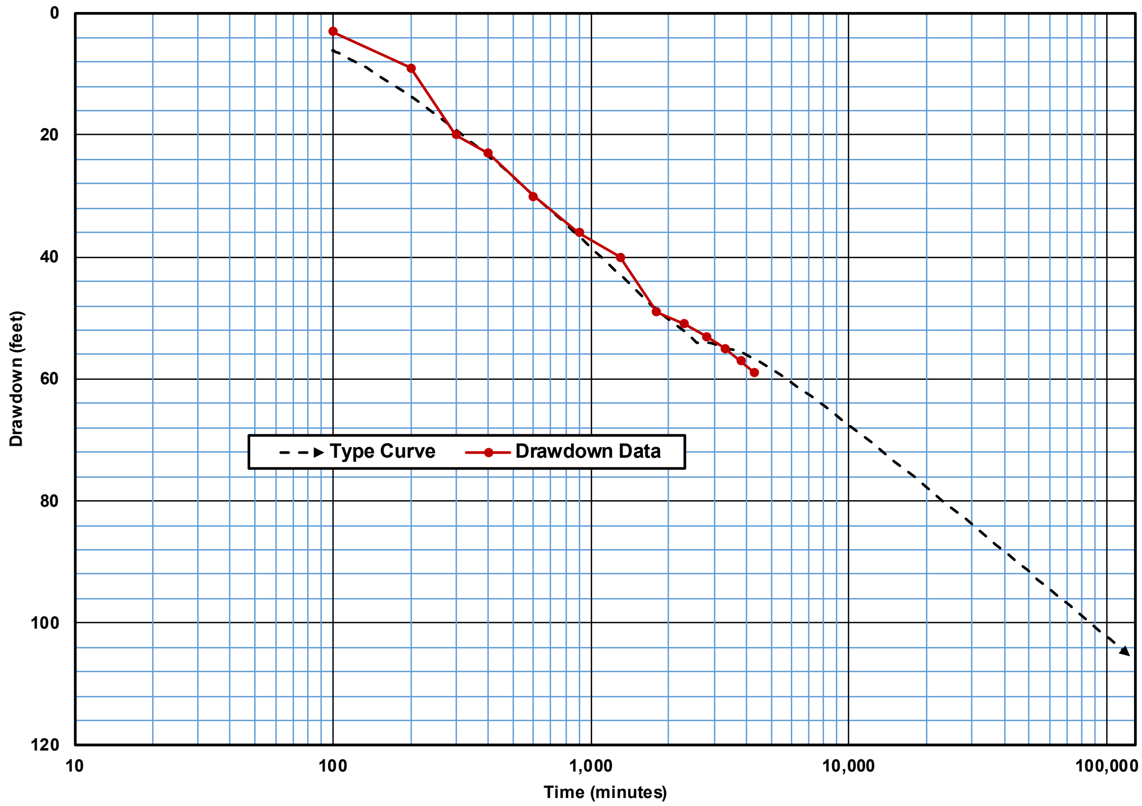

Figure H1a. Mount Airy well 6A (Gillis Falls) - Semi-log plot of drawdowns from a 72-h, variable rate aquifer test, November 2007, Hantush-Jacob leaky aquifer solution.

No tabulated data from the first aquifer test exist in Maryland State files, so numerical data had to be extracted from paper graphs. There is some question as to the initial pumping rate, so it was assumed that it was the same as the rate (210 gpm or 795 L/min) recorded on the second day. The final rate was 175 gpm (663 L/min). Given the quality of the data, the Hantush-Jacob solution provided a good fit to the drawdown data (fig. H1a). The 90-day extrapolated drawdown is 112 ft (34 m). Using the base of the weathered zone (s = 79 ft or 24 m), this would produce an estimated yield of 138 gpm (522 L/min) at the estimated average test rate of about 196 gpm (742 L/min).

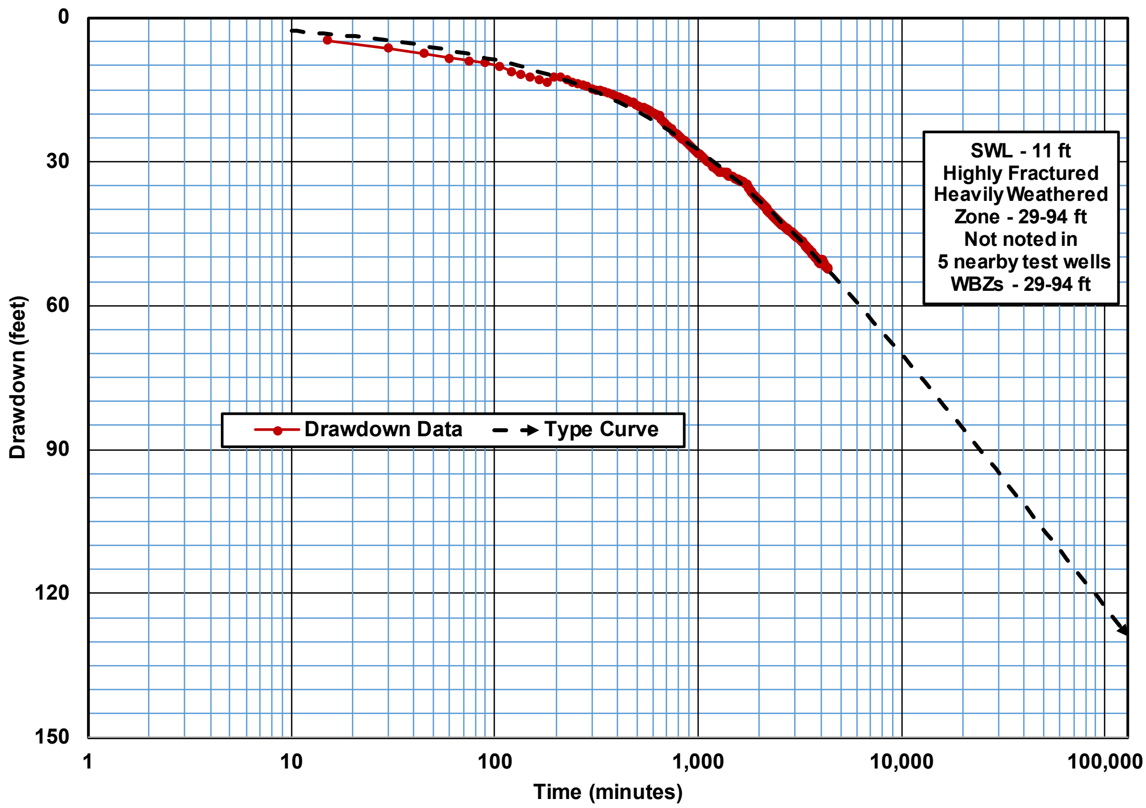

Figure H1b. Mount Airy well 6A (Gillis Falls) - Semi-log plot of drawdowns from a 72-h, 164 gpm aquifer test, April 2008, Gringarten-Witherspoon SVF solution.

The second test produced a reliable set of data. The Gringarten-Witherspoon SVF solution provided a good fit to the drawdown curve, from which an extrapolated 90-day drawdown of 131 ft (40 m) was made (fig. H1b). There was no obvious break in the data, so the drawdown at the end of the test (53 ft or 16 m) would be the minimum available drawdown, producing an estimated yield of 66 gpm (250 L/min). At the drawdown to the base of the weathered zone (83 ft or 25 m), the estimated yield would be 104 gpm (394 L/min).

During the 90-d operational test (Figure H1c) the initial pumping rate was about 230 gpm (871 L/min), with an operating water level of about 75 ft (23 m) (60 ft or 18 m of drawdown). The rate declined to about 110 gpm (416 L/min) after 12 days, then to about 60 gpm (227 L/min) after 53 days until the end of the test, at operating water levels between 70 ft and 75 ft (21 m and 23 m).

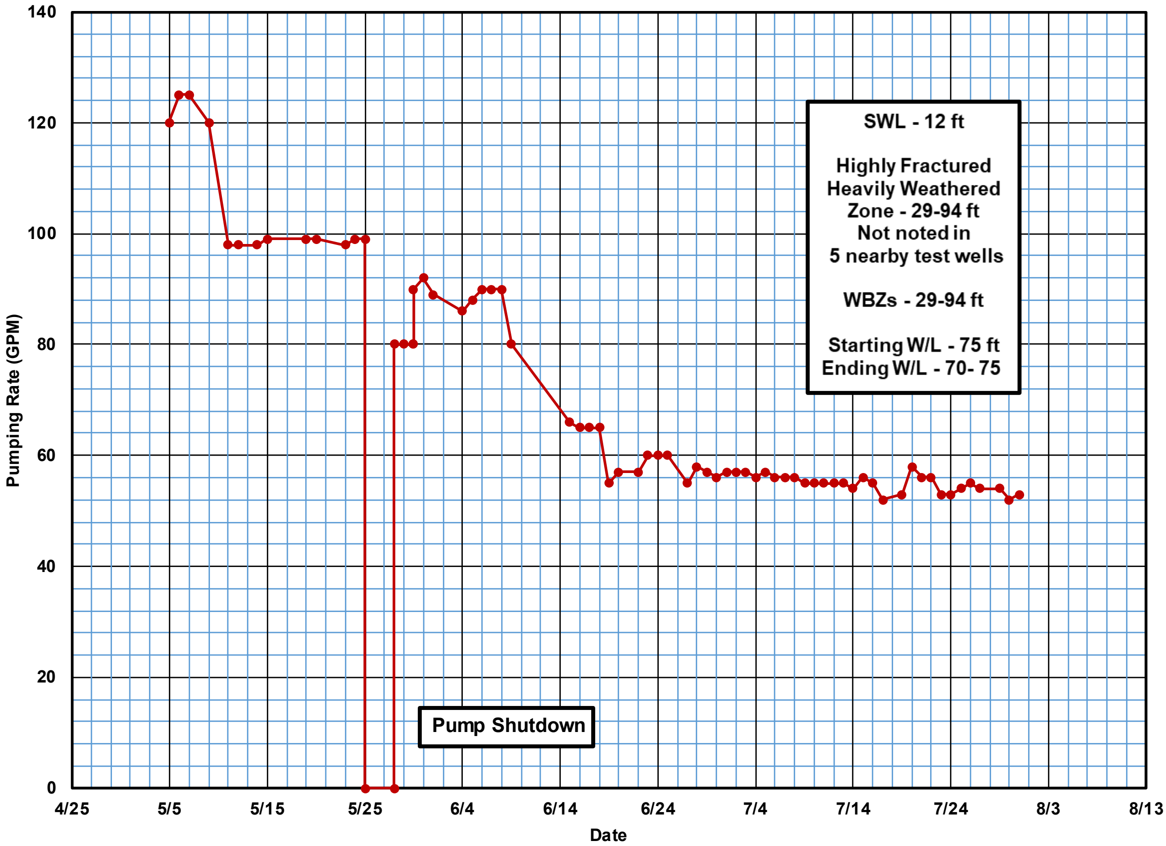

Figure H1c. Mount Airy well 6A (Gillis Falls)– Arithmetic plot of pumping rates over time during a 90-day operational test, May 2008 − July 2008.

The third aquifer test had a much deeper SWL (22 ft or 7 m) than the other tests, possibly reflecting dry conditions. There was no reasonable solution for the early time drawdown data. The derivative for that period is near zero, which is typical of a recharge boundary, although there is no evidence that one exists. The Moench leaky aquifer model fit the late-time data (fig. H1d), producing a 90-d extrapolated drawdown of 61 ft (19 m). If the specific capacity is applied to the minimum (42 ft or 13 m) and maximum (72 ft or 22 m)) drawdowns from the previous tests, and adjusted for the deeper SWL, the estimated yields would be 138 gpm (522 L/min) and 236 gpm (893 L/min), respectively.

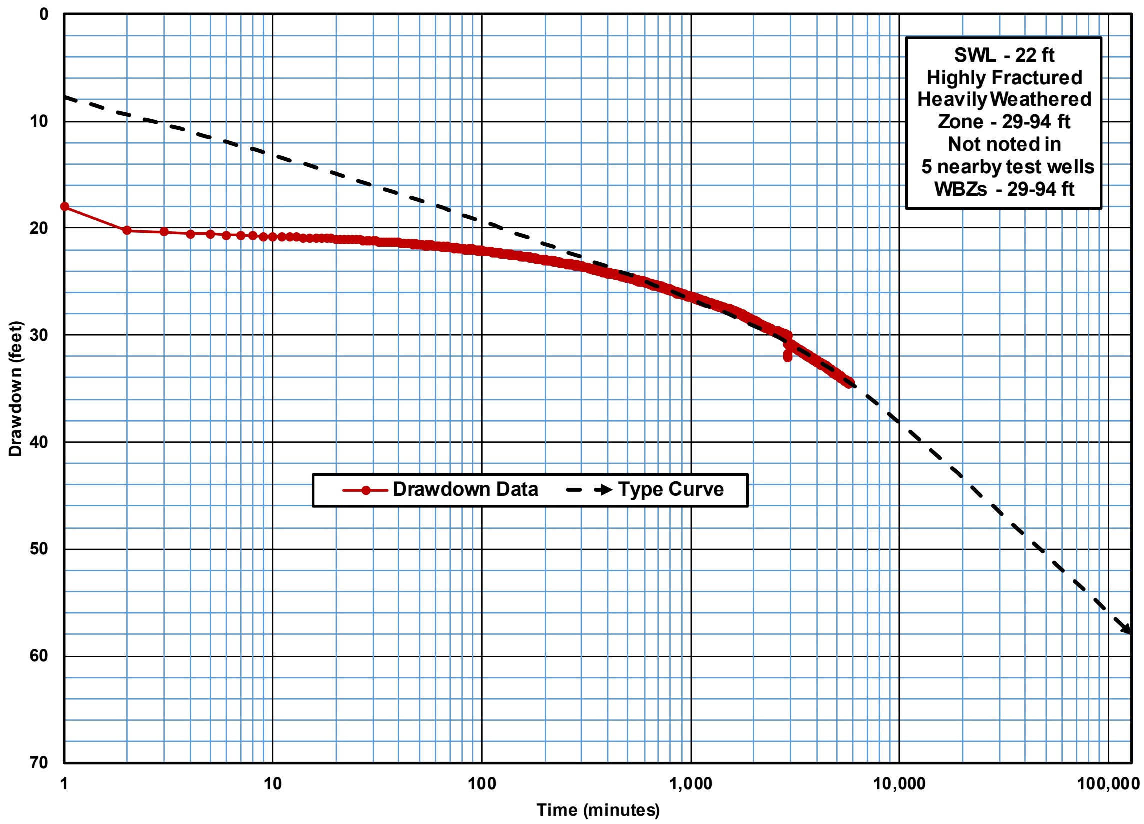

Figure H1d. Mount Airy well 6A (Gillis Falls)- Semi-log plot of drawdowns from a 96-h, 200 gpm aquifer test, October 2008, Moench (type 2) leaky aquifer solution.

This was an interesting project, because the three aquifer pumping tests each produced different results. Of the three, the estimated yield from the second one came closest to the final yield during the 90-d operational test. It is again noted that, of the 28 wells drilled for the project, the highly fractured and weathered zone only occurs in well 6A. This may indicate that there was high, but limited, aquifer storage near the well that sustained the yield during the short tests, and then was depleted during the 90-day test, resulting in the final low yield of the well.

H2. Well 11 (South Main Street)

After the Gillis Falls project was abandoned, the Town of Mount Airy completed well 11 (CL-95-1415) in 2009. A step-drawdown test was initially conducted in June 2009. This was followed by two aquifer pumping tests. The first started at 165 gpm (625 L/min), then was stopped, and restarted at 135 gpm (511 L/min), and then secured after 21 hours. For the second test, the well was continuously pumped at 75 gpm (284 L/min) for the full 72 hours.

The step-drawdown test was started at 150 gpm (568 L/min), producing a flat curve with 50−60 ft (15-18 m) of drawdown for the first 90 minutes (fig. H2a). After 30 minutes of recovery, the second step was started at 200 gpm (757 L/min). This immediately produced a rapid drawdown for about one hour, followed by an accelerated rate of drawdown until pumping was stopped when the drawdown nearly reached 400 ft (122 m). After the recovery, the test was re-started at 250 gpm (946 L/min), producing an immediately unsustainable drawdown at which point the rate was changed to 175 gpm (662 L/min), which continued until the end of the test. These data indicate that there was a sharp break in the drawdown data when the pumping rate was increased from 150 gpm (568 L/min) to 200 gpm (757 L/min) at about the depth of the first wbz, suggesting that the response was an effect of dewatering of that fracture.

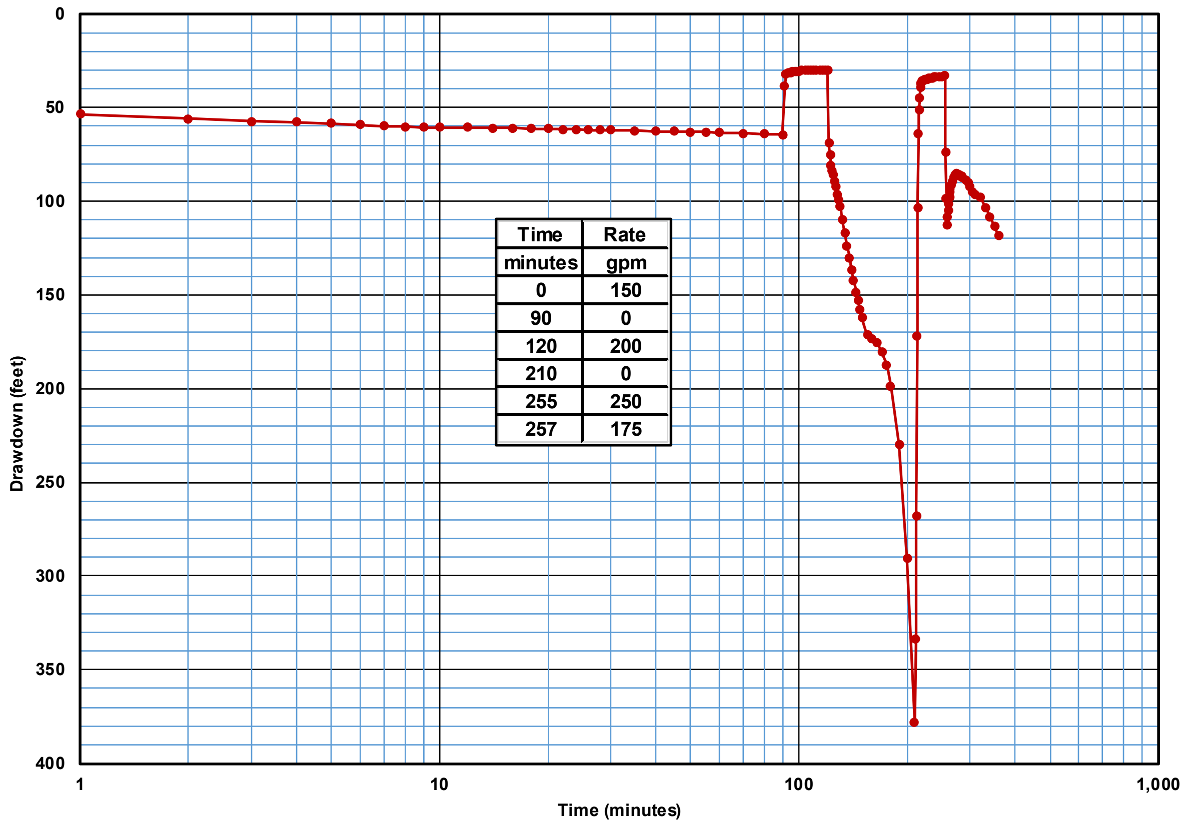

Figure H2a. Mount Airy well 11 (South Main Street) – Semi-log plot of drawdowns from a step-drawdown test.

The 21-h aquifer test was started at 165 gpm (625 L/min) because the step test indicated that the long-term test rate should not be much more than 150 gpm (568 L/min) (fig. H2b). Initially, a relatively flat curve developed, followed by a steeper decline after 100 minutes when water levels reached the depth of the first wbz. At about 400 minutes an excessive drawdown started to occur, culminating at 600 minutes with a drawdown of 310 ft (94 m). The test was then secured and restarted at 135 gpm (511 L/min). That was also an unsustainable rate, and the test was finally secured at 1,260 minutes.

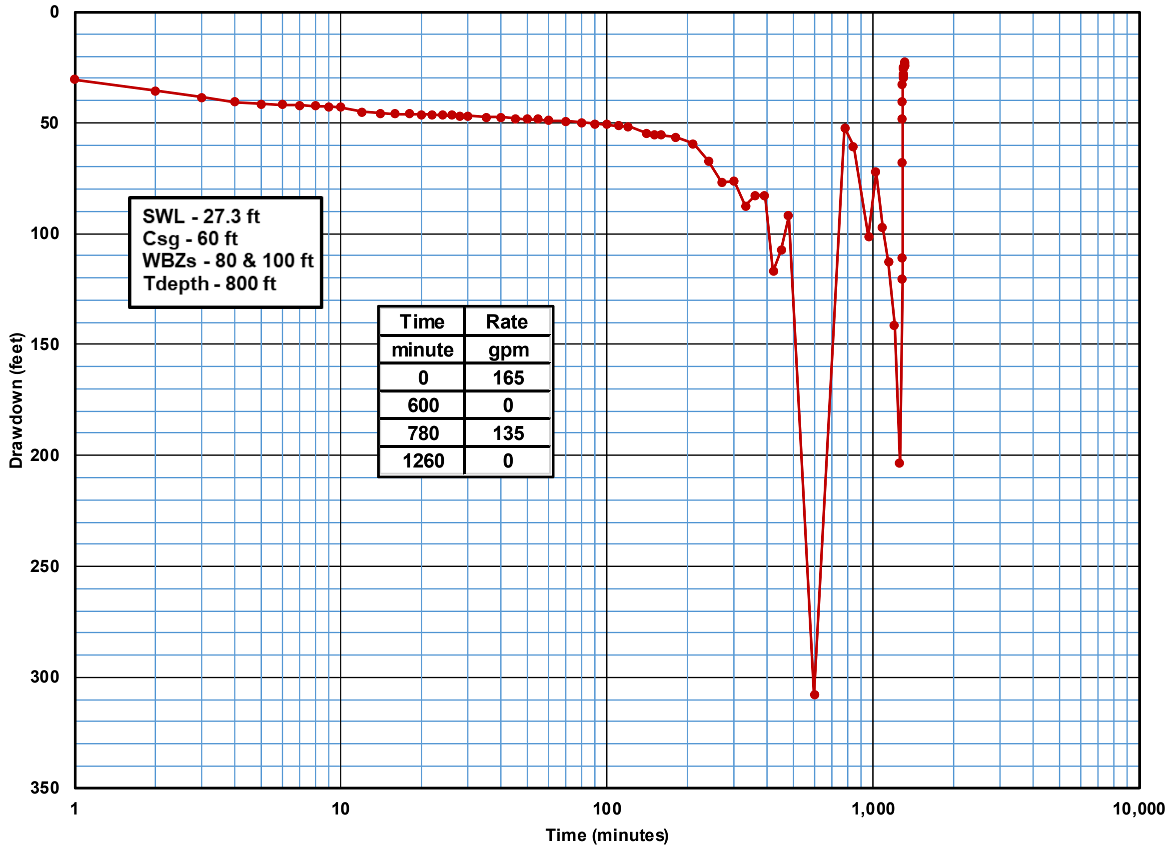

Figure H2b. Mount Airy well 11 (South Main Street) – Semi-log plot of drawdowns from a 21-h, variable rate aquifer test.

The second aquifer test was started at a rate of 75 gpm (284 L/min), which was maintained throughout the entire 72 hours (fig. H2c). It produced a steadily increasing, moderate decline in the water level reaching a drawdown of 38 ft (12m), or a point several feet below where the casing was set. No model provided a good fit to the data, so by visual inspection, a drawdown curve was extrapolated to 90 days, resulting in an estimated yield of 27 gpm (102 L/min), at an available drawdown to the casing depth and 44 gpm (167 L/min) when the drawdown to the wbz was used in the calculations.

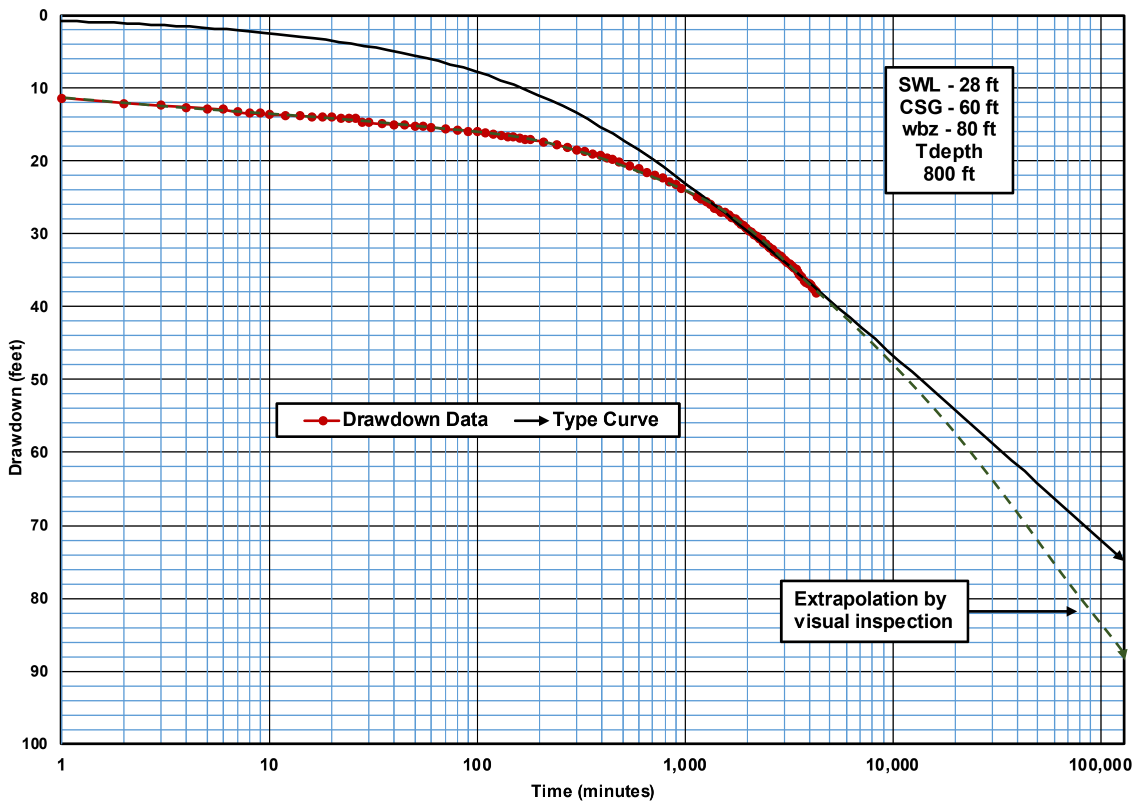

Figure H2c. Mount Airy well 11 (South Main Street) – Semi-log plot of drawdowns from a 72-h, 75 gpm aquifer test, Gringaten-Witherspoon SVF solution, and an additional extrapolation by visual inspection.

There was no AQTESOLV® analytical solution that could match the entire data set. The best fit occurred when the Gringarten-Witherspoon SVF solution was fit to the late-time data (1,000−4,320 minutes). This produced a 90-d extrapolated drawdown of 76 ft (23 m). When applied using the drawdown to the casing depth (32 ft or 9 m) as the limiting factor, the estimated yield is 32 gpm (121 L/min). If the depth of the wbz were used, the estimated yield is 51 gpm (193 L/min). This suggests that the reliable yield is at least 37.5 gpm (142 L/min) as there was no break in the drawdown curve upon exceeding the casing depth (s = 38 ft or 12 m) at the end of the test.

After the well was placed in service, the maximum reported use in any year was 26 gpm (98 L/min) avg. / 32 gpm (121 L/min) max., in 2013. The peak monthly use was 34 gpm (129 L/min) in March 2016. Water level measurements are required as a water appropriation and use permit condition. Appendix A6 provides the monthly water use and water levels for well 11. Except for 2010 and, possibly, October and November 2017, the Town appears to report the water level above the probe set at 200 ft (61 m). These data indicate that the water level is generally below the first major wbz at 80 ft (24 m), with no evident decrease in yield between 2011 and 2017. The well was last reported in operational use in May 2018. The production data indicate that the 90-d extrapolation from the 75 gpm (284 L/min) test applied to the drawdown at the end of the 72-hour test provided the best estimated yield, suggesting that the weathered transition zone is a reservoir rock unit in the well.

Table 1. Pumpage (black) and water levels (red) in Mount Airy Well 11.

Page 1 of 1
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Pumpage Report Summary By Permit

MARYLAND

Permit: CL2009G001 — Total Pumpage Reports: 0

Year	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Oct	Nov	Dec	Annual Total Usage	Annual Average	Annual % Overage	High Month	Monthly % Overage
2010									35	33	33	33					
2010	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.00		0.00
2011	34	34		26		22				74	82	85					
2011	0	0	0	0	0	0	0	0	0	1,319,000	1,329,000	1,424,000	4,072,000	11,156	0.00	(12) 45,935	0.00
2012	1,348,000	1,154,000	1,216,000	286,000	1,105,000	811,000	1,101,000	1,084,000	917,000	845,000	991,000	963,000	11,821,000	32,298	0.00	(1) 43,484	0.00
2013	92	86	82	75	93	61	88	100	116	122	116	111					
2013	1,074,000	1,191,000	1,359,000	1,359,000	1,366,000	1,266,000	958,000	1,308,000	955,000	919,000	919,000	985,000	13,659,000	37,422	0.00	(4) 45,300	0.00
2014	90	74	68	62	57	62	63	51	65	59	111	75					
2014	1,386,000	1,087,000	1,301,000	1,044,000	1,216,000	1,054,000	0	246,000	483,000	501,000	991,000	213,000	9,522,000	26,088	0.00	(10) 44,710	0.00
2015	52	50	47	73	104	98	77	85	94	101	100	94					
2015	0	0	0	1,396,000	1,515,000	1,179,000	1,267,000	1,300,000	959,000	902,000	866,000	926,000	10,310,000	28,247	0.00	(5) 48,871	0.00
2016	1,162,000	1,170,000	1,534,000	1,321,000	273,000	1,022,000	1,257,000	953,000	786,000	800,000	706,000	667,000	11,651,000	31,833	0.00	(3) 49,484	0.00
2017										28	28	68					
2017	663,000	617,000	135,000	4,000	312,000	1,239,000	941,000	961,000	804,000	353,000	396,000	960,000	7,385,000	20,233	0.00	(6) 41,300	0.00
2018	699,000	625,000	892,000	889,000	460,000	0	0	0	0	0	0	0	3,565,000	9,767	0.00	(4) 29,633	0.00

Water level reports in pumpage files appear to be level above probe set at 200 feet, except for 2010 and, possibly, Oct & Nov 2017

I.-K. Wells in Consolidated Sedimentary Rock Aquifers

I. Town of Poolesville

Figure Ia is a map showing the location of the Town of Poolesville's public water supply wells. Figure Ib is a cross-section through the Poolesville area constructed from geophysical (gamma ray) and geologic logs, adapted from Otton (1981). The town's municipal wells were all completed in consolidated sedimentary rocks of the Triassic New Oxford Formation. The depths to water-bearing zones recorded in the available drillers/geologic logs do not all correlate with the depths of the water-bearing zones shown in the Otton (1981) cross-section. Aquifer test data are presented for wells 1, 2, 4, 5 and 6. Step-drawdown and aquifer test data are presented for wells 8, 9, 10 and 12. Yields were estimated and compared to production data collected by the town.

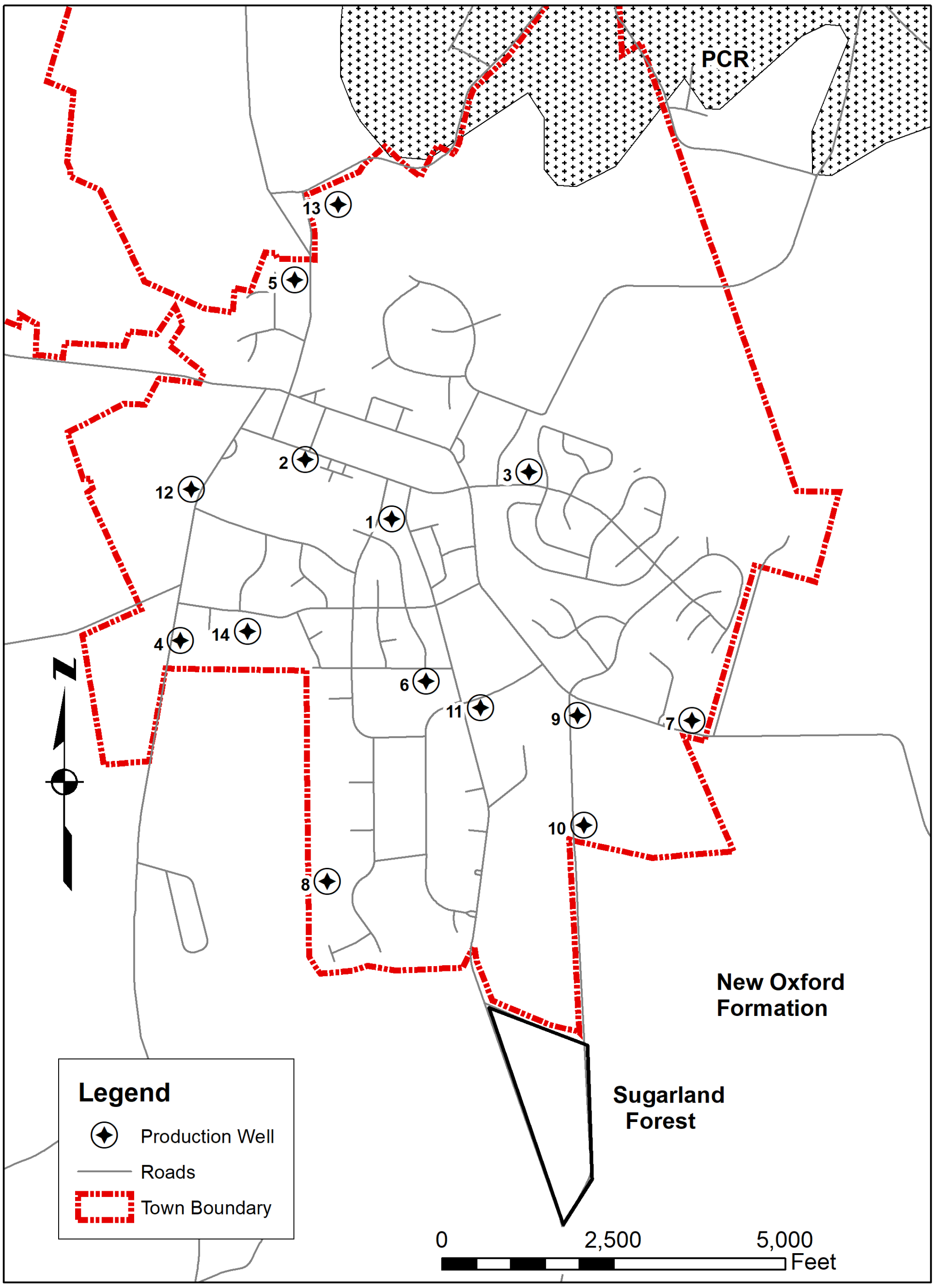


Figure Ia. Map showing the locations of the Town of Poolesville's production wells.

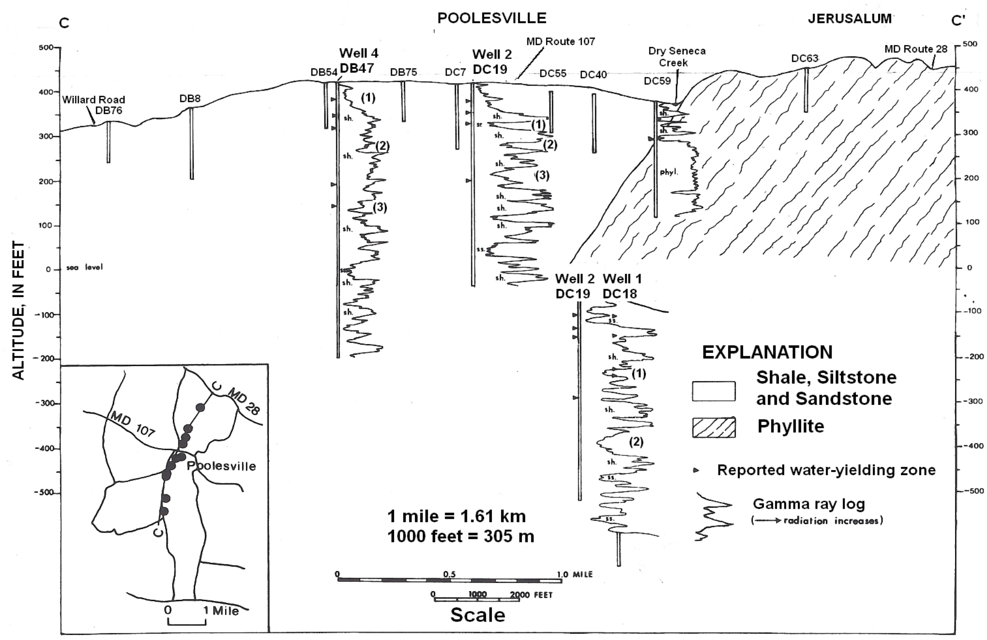


Figure Ib. Geologic cross-section of the Poolesville area (modified from Otton, 1981).

I1. Well 1

An apparent pseudo-equilibrium phase occurred during the test of well 1 (MO-70-0014) (fig. I1a) which was probably caused by successive, stepwise reductions in the well's pumping rate to maintain the water level above the pump intake. It appears that the final rate of 53 gpm (201 L/min) was thought to be the reliable yield of the well. The 90-d extrapolation (414 ft or 126 m) from the first 18 minutes of data as applied to the 155-ft (47 m) water-bearing zone within sandstone unit (1), produced a much lower estimated yield of 29 gpm (110 L/min). This was a somewhat higher yield relative to the 1996 production of 26 gpm (98 L/min), although the initial test was conducted under dry conditions and 1996 was a very wet year. Otton (1981) reported that well 1 produced an average of 27 gpm (102 L/min) in 1978, with water levels between 140 and 220 ft (43 and 67 m), and an average of about 200 ft (61 m). Regional groundwater levels (41 ft, or 12.6 m, in 50W-4C) in 1978 were below average values. In addition, WSA files indicate that well 1 produced an average of 29 gpm (110 L/min) (range 20–38 gpm, or 76-144 L/min, per month) during the period of May 1976 to December 1977, with water levels below 150 ft (46 m). During that period, the average yield was 24 gpm (91 L/min) under dry conditions (June 1977 – December 1977) and 32 gpm (121 L/min) under average conditions (October 1976 – May 1977). The maximum reported use was 39 gpm (148 L/min) in April 1978 under wet conditions. Based on these data, it would be expected that the yield in the wet year of 1996 would have been much higher than the equivalent of 26 gpm (98 L/min), indicating that the well's yield may have declined due to dewatering of the 155-ft (47 m) wbz and/or the sandstone unit.

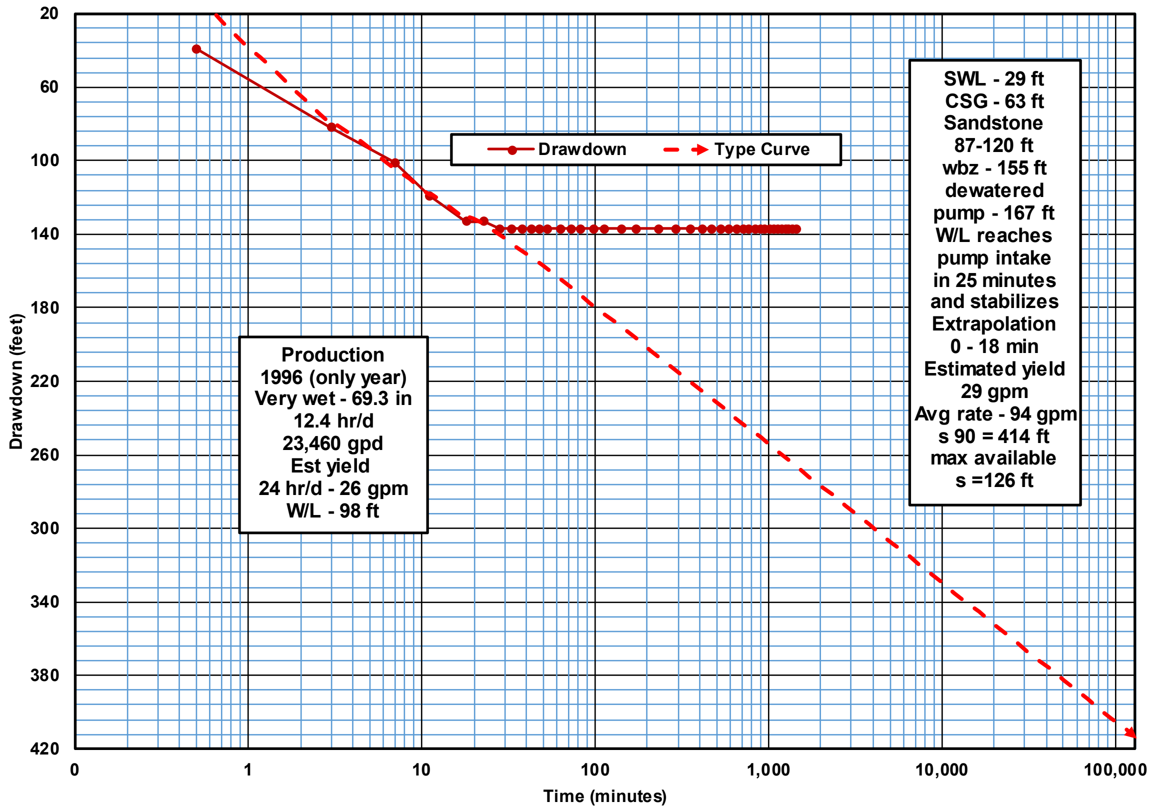


Figure I1a. Poolesville well 1 – Semi-log plot of drawdowns from a 24-h, variable rate aquifer test, Gringarten-Witherspoon SVF solution.

Wells 1 and 2 were pumped 24 h/d during 1978, with water levels approaching 200 ft (61 m). The production in well 2 remained steady while there was a substantial decline in the yield of well 1. The total production from the two wells was about the same as the demonstrated yield of well 2 (see discussion below). These factors indicate that there was severe interference between the two wells. Ultimately, well 1 was abandoned due to a turbidity problem that was not cost-effective to correct, considering its low yield and possible interference with well 2.

I2. Well 2

In 1999, well 2 (MO-70-0046) was only producing about 20 gpm (76 L/min). It was suggested to the Town that the aquifer test data indicated the well should be able to produce about 100 gpm (379 L/min) and the pump should be changed. The pump was replaced at that time and the yield immediately increased. This was confirmed with a 93 gpm (352 L/min), 32-d test conducted in June 2001, which had an operating water level of 130 ft (40 m). During subsequent operational tests, well 2 produced an average of 97 gpm pumping 21.4 h/day, with a water level of 164 ft (50 m) in October 2009 and 105 gpm in November 2009 pumping 23.4 hours/day, with an operating water level of 147 ft (45 m). The water systems supervisor eventually indicated that he thought several stages in the pump had burnt out, causing the earlier reduced yield.

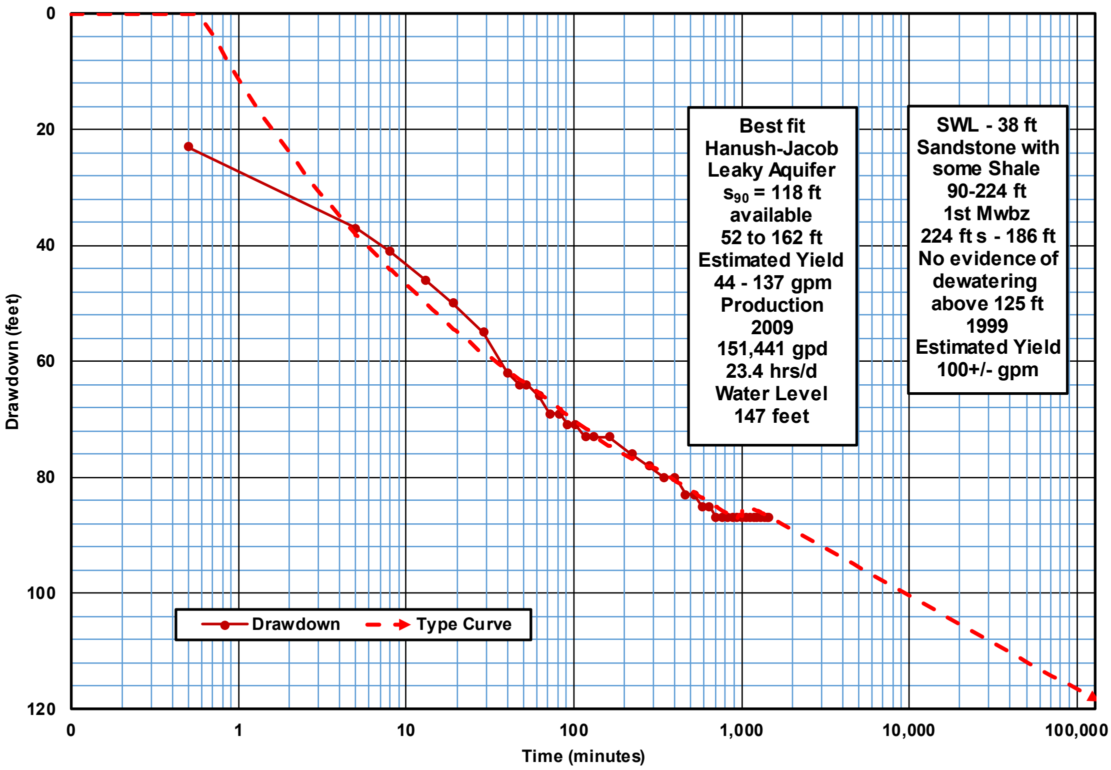


Figure I2a. Poolesville well 2 – Semi-log plot of drawdowns from a 24-h, variable rate aquifer test, Hantush leaky aquifer solution.

The yield was estimated from data taken during a 1969 test, using the methods developed in the Hammond (2018) study (fig. I2a). The 24-h test was started at 108 gpm and several rate adjustments were made. The water level stabilized before the end of the test at 125 ft (38 m), between sandstone units 1 (90 ft or 27 m) and 2 (150 ft or 46 m) and far above sandstone unit 3 (200 ft or 61 m), (fig. Ia), and the major wbz (224 ft or 68 m). The best fit to the drawdown data prior to water level stabilization (0−642 minutes) was achieved using the Hantush, with aquifer storage, leaky aquifer solution. The 90-d extrapolated drawdown is 119 ft (36 m). When applied to the drawdowns at the three units: 1at 52 ft (16 m), 2 at 112 ft (34 m) and 3 at 162 ft (49m), the estimated yields are 44, 95 and 137 gpm (167, 360 and 519 L/min), respectively. Because the operating levels during the 2009 extended test dropped below unit 2 at times and still exceeded 100 gpm (379 L/min), and unit 3 is thicker and appears to be the more permeable sandstone on the geophysical log for the well, this would suggest that unit 3 may be the primary reservoir rock for well 2.

I3. Well 4

Figure I3a shows the results of two aquifer tests conducted on well 4 (MO-73-1584). The first (24 h) test was conducted in June 1977 at 50 gpm (189 L/min). A straight line was produced on the drawdown curve during the first 1,000 minutes, followed by a flattening of the curve until the end of the test. A second (7-d) test was conducted in August 1977 at 48 gpm (182 L/min). The longer test was required to evaluate the potential impacts to nearby domestic wells noted during the first test. A pseudo-equilibrium phase occurred during the first two days of that test, followed by a sharp break in the drawdown data at a water level of 95 ft (29 m) or drawdown of 67 ft (20 m), and then by a straight-line response for the remainder of the test. The sharp break occurs at the base of the massive sandstone unit (1) in Figure Ia, suggesting that the response may have been due to dewatering of that zone.

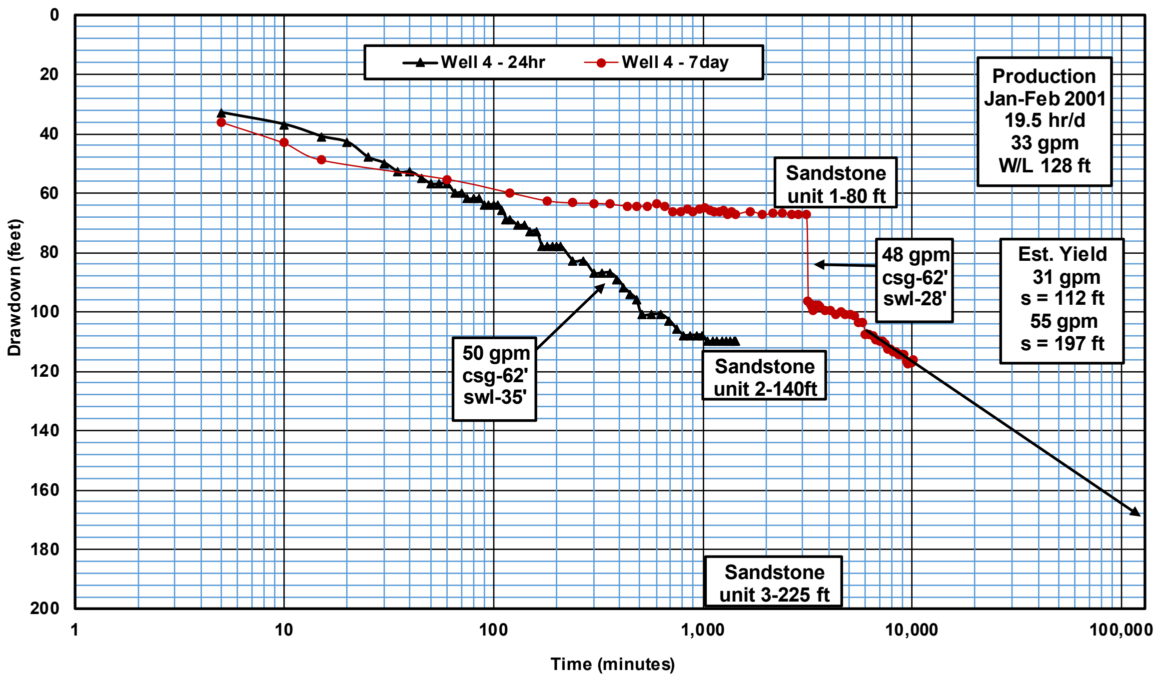


Figure I3a. Poolesville well 4 – Semi-log plot of drawdowns from 24-h, 50 gpm and 7-d, 48 gpm aquifer tests.

The static water level was slightly lower and the pumping rate slightly higher during the first test of well 4 than during the second test. This may have allowed the water level to rapidly pass through the transition zone during the first test, isolating flow from the shallow zone, and producing no change in slope. The water level then stabilized near the end of the test, probably due to leakage. During the second test, the early-time phase probably was due to a leaky aquifer effect. The sharp decline occurred once the dewatering of the sandstone unit was complete, followed by a late-time, straight-line response.

The tests of well 4 indicated that slight changes in the pumping rate or climatic changes can result in entirely different responses during tests in fractured rock aquifers. This suggests that the shallow, massive sandstone and the bedrock portion of an aquifer are hydraulically connected, but that the shallow zone can be rapidly and effectively isolated if the pumping rate is high enough.

In the case of well 4, extrapolations from the straight-line portions of the final test to the second sandstone unit produced an estimated yield of 31 gpm (117 L/min), while the projections to the third sandstone unit produced a yield of 55 gpm (208 L/min). The maximum production from the well was an average of 33 gpm (125 L/min) in Jan 2001–Feb 2001, while pumping the well 19.5 h/d (37 gpm, or 140L/min, at 24 h/d) with a water level just above the second sandstone unit at 128 ft (39 m). Initial operational data collected by Otton (1981) indicated that well 4 produced an average of 45 gpm (170 L/min) during 1978, when regional groundwater levels (41 feet or 12.6 m in 50W-4C) were below average values. The operating water levels during that year varied between 130 and 250 ft (40 and 76 m), with an average of about 200 ft (61 m). Subsequently, the yield may have declined due to dewatering of sandstone unit 2.

I4. Well 5

A 24-h test of well 5 (MO-73-2905) was conducted on March 12, 1980 under average climatic conditions (fig. I4a). The test was started at 129 gpm (488 L/min) and at 50 minutes the rate was adjusted to 120 gpm (454 L/min) due to a reported airline leak. From that point an apparent IARF segment continued until the last 9 hours of the test, when the water level then stabilized until the end of the test at 133.5 ft (41 m). It is not clear what caused the late-time response, but it might have been due to dewatering of sandstone units at depths of 115−126 ft (35−38 m) and 127−130 ft (39−40 m). Extrapolating from the IARF segment produces a s_{90} of 138 ft (43 m), which when applied to the stabilized drawdown of 108 ft (33 m) produces an estimated yield of 94 gpm (356 L/min). The Hantush-Jacob leaky aquifer solution also provides a good fit to the drawdown data, that would indicate that the well should be able to produce 120 gpm (454 L/min).

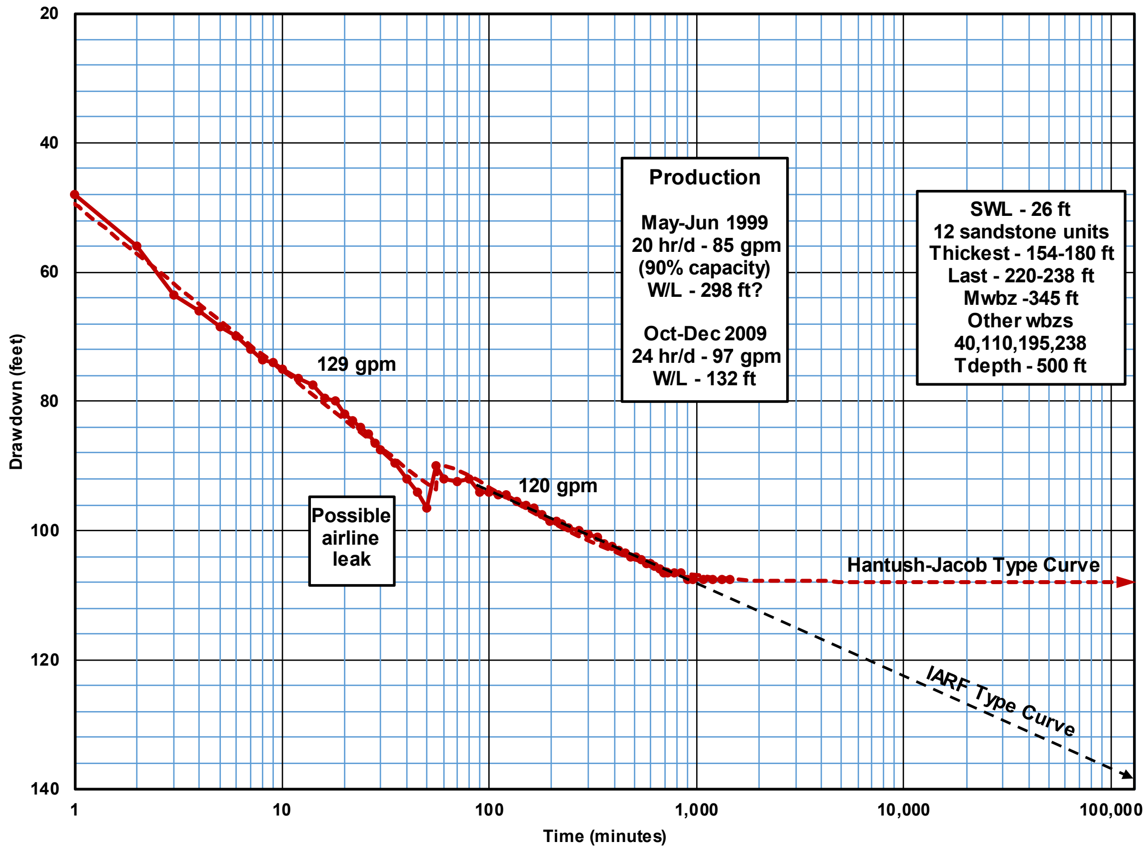


Figure I4a. Poolesville well 5 – Semi-log plot of drawdowns from a 24-h, variable rate aquifer test, with estimated yields based on 90-d extrapolations from Hantush-Jacob and IARF solutions.

In May 1999 − June 1999, the well was pumped 20 h/d under above average climatic conditions, producing an average of 85 gpm (322 L/min) (estimated to be 94 gpm, or 356 L/min, at 100-percent capacity) with an operating water level of 298 ft (91 m). Subsequent repairs to the well indicated that this was probably an unreliable water level measurement due to a faulty airline. This was later confirmed when the well was operated 24 h/d during the period October 2009 − December 2009, under average climatic conditions, producing 97 gpm (367 L/min) with a water level of 132 ft (40 m).

Although a reservoir unit can not be clearly defined, it is likely below 132 ft (40 m), since there has been no apparent decline in yield over time, if the estimate from the IARF segment is valid. If the estimate from the leaky aquifer solution is correct, the the yield may have declined by about 19 per cent.

I5. Well 6

Well 6 (MO-81-0765) was once considered to be Poolesville's best well and was completed in 1984. Based on aquifer tests in 1984 and 1985, consultants estimated the well's sustained yield to be 225−261 gpm (852−988 L/min). Data from the second test are shown in Figure I5a. In 1999, the well was producing less than a daily average of 100 gpm (379 L/min). Upon review of available production data, the greatest declines in yield occurred after the first major water-bearing zone in the well was dewatered. The driller had identified the lithology of that zone as fractured red rock, a term used for New Oxford Formation sandstones. The author's 1999 estimated yield of 168 gpm (636 L/min) was based on extrapolating drawdown from an IARF segment, a flow regime that was confirmed by a recent derivative analysis. The estimated yield was lower than previous predictions, but more than the well was producing; so, it was suggested that the town install a smaller pump and a valve to control and maintain the water level in the well above the first major water-bearing zone.

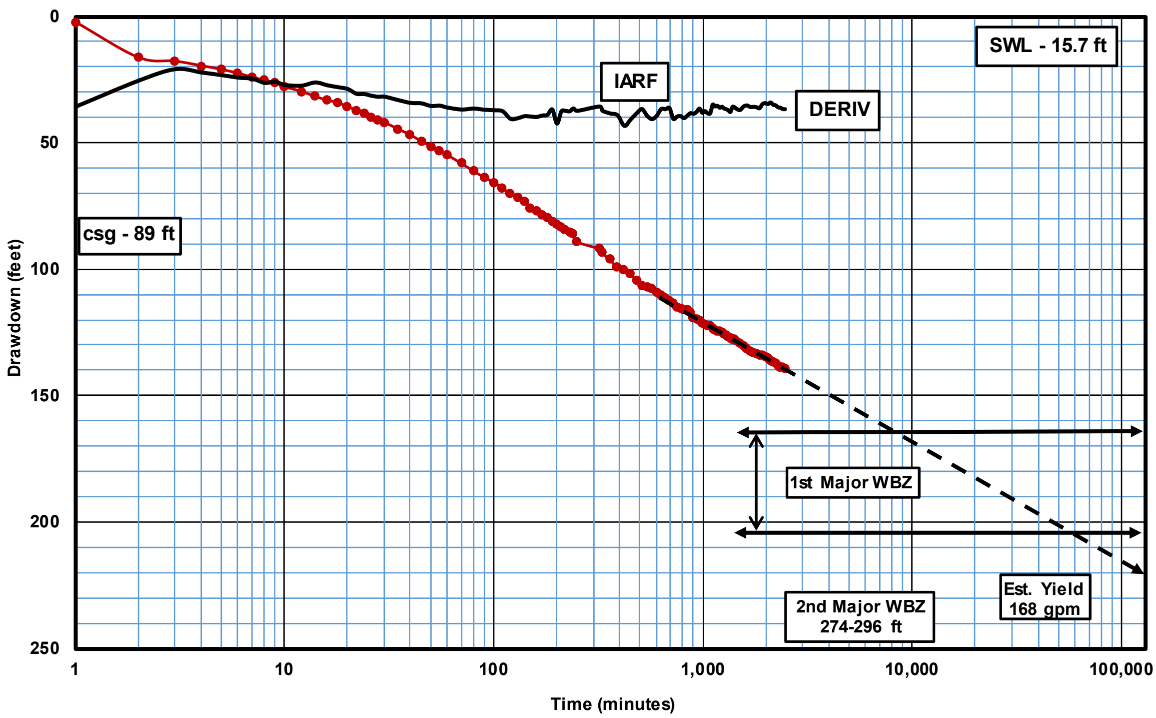


Figure I5a. Poolesville well 6 – Semi-log plot of the water levels and the adjusted logarithmic derivatives from a 72-h, 225 gpm pumping test, with estimated yield based on 90-d extrapolation from an IARF segment.

The town did install a flow control valve in well 6, but the pump was not changed. Water use and water level data (fig. I5b) indicate that the well initially could produce a maximum daily yield of 185 gpm (700 L/min), but that declined over time to a maximum of about 113 gpm (428 L/min). The reduced yield appears to have been due to dewatering of the upper water-bearing zone.

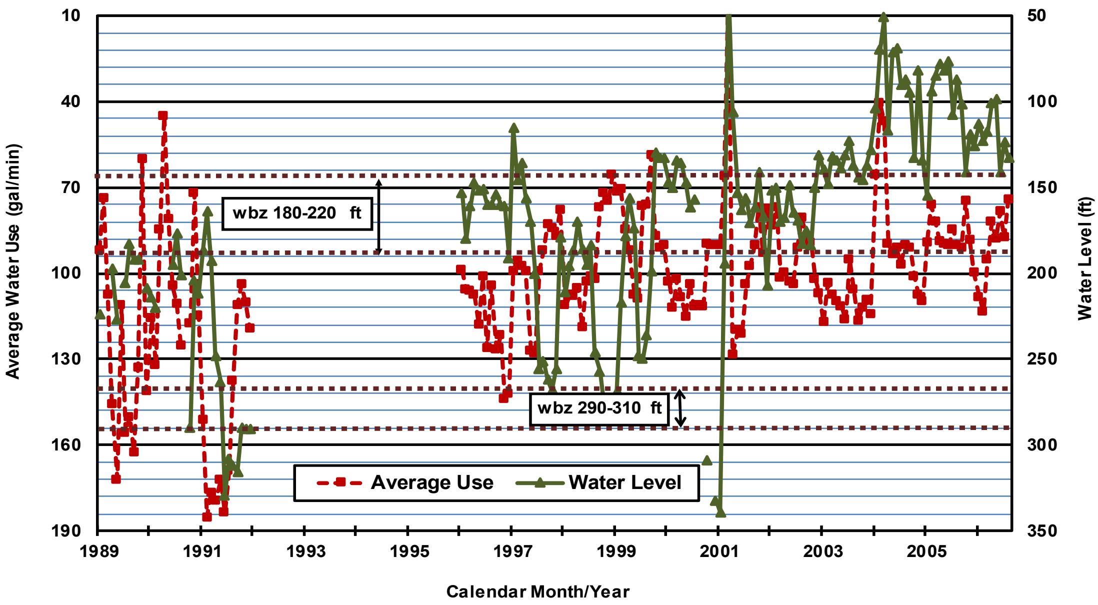


Figure I5b. Poolesville well 6 – Water use and water level data.

I6. Well 8

Well 8 (MO-93-0007) was completed in February 1994. The driller reported that the static water level (SWL) was 160 ft (49 m) during construction. It is not clear if this was an anomaly or recording error; however, the SWL was 17.5 ft (5 m) one week after the well was drilled.

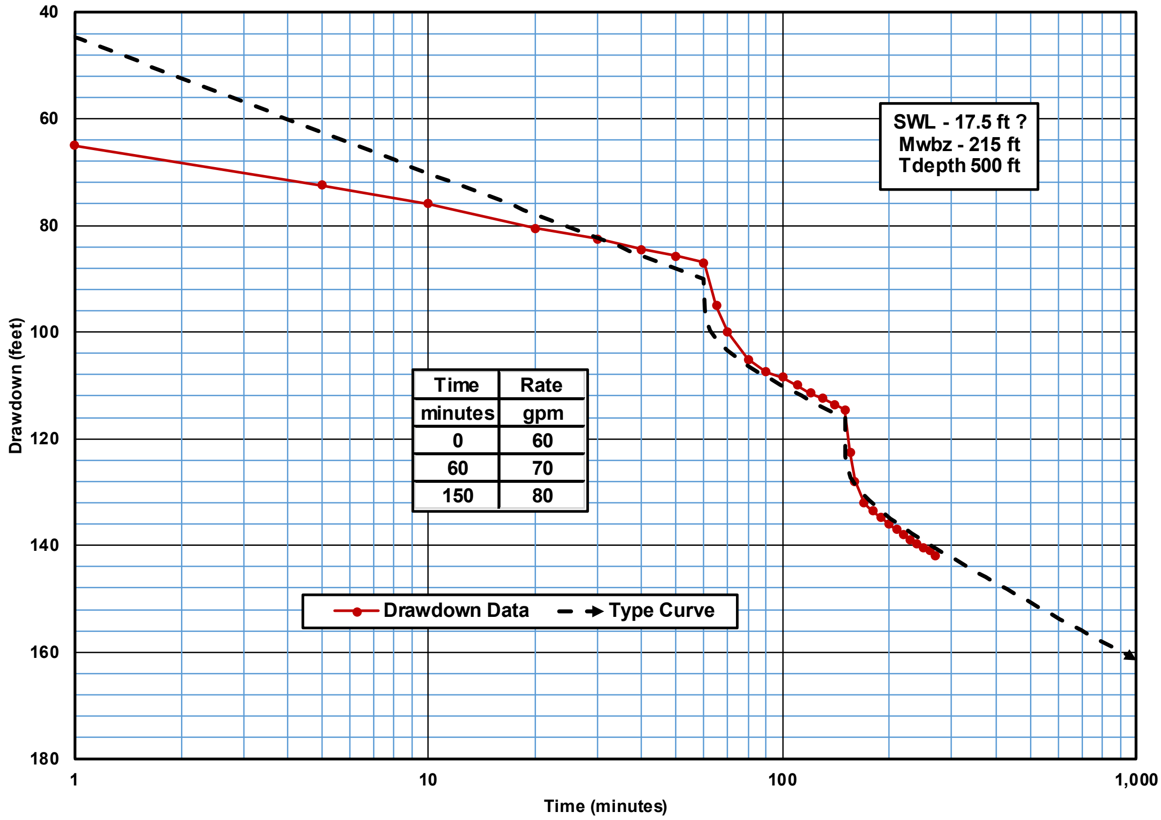


Figure I6a. Poolesville well 8 – Semi-log plot of drawdowns from a step-drawdown test, Hantush-Jacob leaky aquifer solution.

A step-drawdown test for well 8 was conducted on April 14, 1994 (fig. I6a), followed by a 72-h, 80 gpm (303 L/min) test on April 18, 1994 (I6b). No static water levels were reported; however, the regional water levels were about four ft (one m) higher than in February 1994, suggesting that the SWL was 13−14 ft (4 m) BMP and reflected wet climatic conditions. The step-test consisted of 60, 70 and 80 gpm (227, 265 and 303 L/min) steps and the Hantush-Jacob solution provided a good fit to the drawdown data, producing a well efficiency of 100 percent and indicating that there was no evidence of dewatering of a reservoir rock unit. The 72-h aquifer test data also fit the Hantush-Jacob model, again with no evidence of dewatering effects. This indicates that the well could sustain 80 gpm (303 L/min) if it were maintained above the extrapolated drawdown of 170 ft (52 m) or a water level of about 185 ft (56 m).

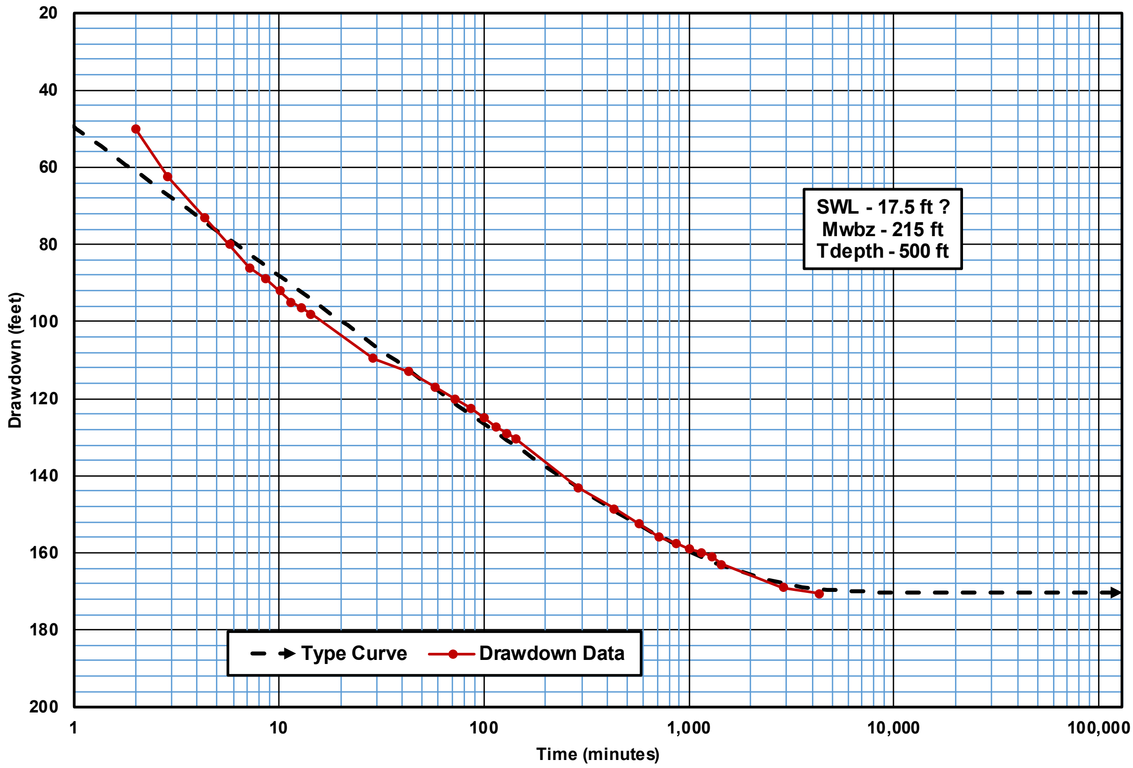


Figure I6b. Poolesville well 8– Semi-log plot of drawdowns from 72-h, 80 gpm aquifer test, Hantush-Jacob leaky aquifer solution.

The driller described the lithology as being red clay or red slate in the interval 1−345 ft (0.3−105 m), followed by two thin limestone units and more red slate to the final depth of 500 ft (152 m). There was a minor wbz at 160−163 ft (49-50 m) and a major one at 215−217 feet (65−66 m), with estimated flows during drilling of 15 gpm (57 L/min) and 55 gpm (208 L/min), respectively. There was no deflection of the drawdown curve during dewatering of the shallow fracture; however, it still would be prudent to keep the water level above 160 feet during well operations.

From available records, the maximum effective production was during the wet period of November 1997 − December 1997, when the well pumped an average of 60 gpm (227 L/min) at 17.9 h/d, or the equivalent of about 71 gpm (269 L/min) at 24 h/d, at a water level of 150 ft (46 m), which was 89 percent of the predicted yield. By comparison, 53.6 gpm (203 L/min) were pumped at 19.5 h/d (the equivalent of 61 gpm, or 231 L/min, at 24 h/d) under dry conditions in January 2001 − February 2001, with a water level of 146 ft (45 m). More recently, in January 2019, well 8 produced an average of 34.8 gpm (132 L/min), while operating 11.7 h/d (equivalent to 57 gpm, or 216 L/min, at 24 h/d), with a water level of 67 ft (20 m), under wet conditions. There may have been a 20-percent decrease in the well's yield over time, but that should be confirmed by additional long-term testing of the well.

I7. Well 9 (Powell)

A multi-rate test (I7a) and aquifer test (fig. I7a) were performed on well 9 (MO-94-1881) in June 2001, under average climatic conditions. The Hantush-Jacob leaky aquifer solution best fit the first three steps of the multi-rate test (0−255 minutes, total drawdown ≈ 210 ft, or 64 m). The data deviates from the model near the end of the third step. The Hantush-Jacob solution also provided a good fit to the early portion (0−600 minutes) of the aquifer test (fig. I7b), with a break in the drawdown data at s = 189 ft (58 m). With a SWL of 41.5 ft (13 m), the break corresponds to and was probably due to dewatering of the first major wbz at 230 ft (70 m). A 90-d extrapolation of the early portion of the aquifer test curve produces a drawdown of 280 feet (85 m). When the specific capacity at the point is applied to the break in the drawdown data at 189 ft (58 m), the estimated yield is 152 gpm (575 L/min).

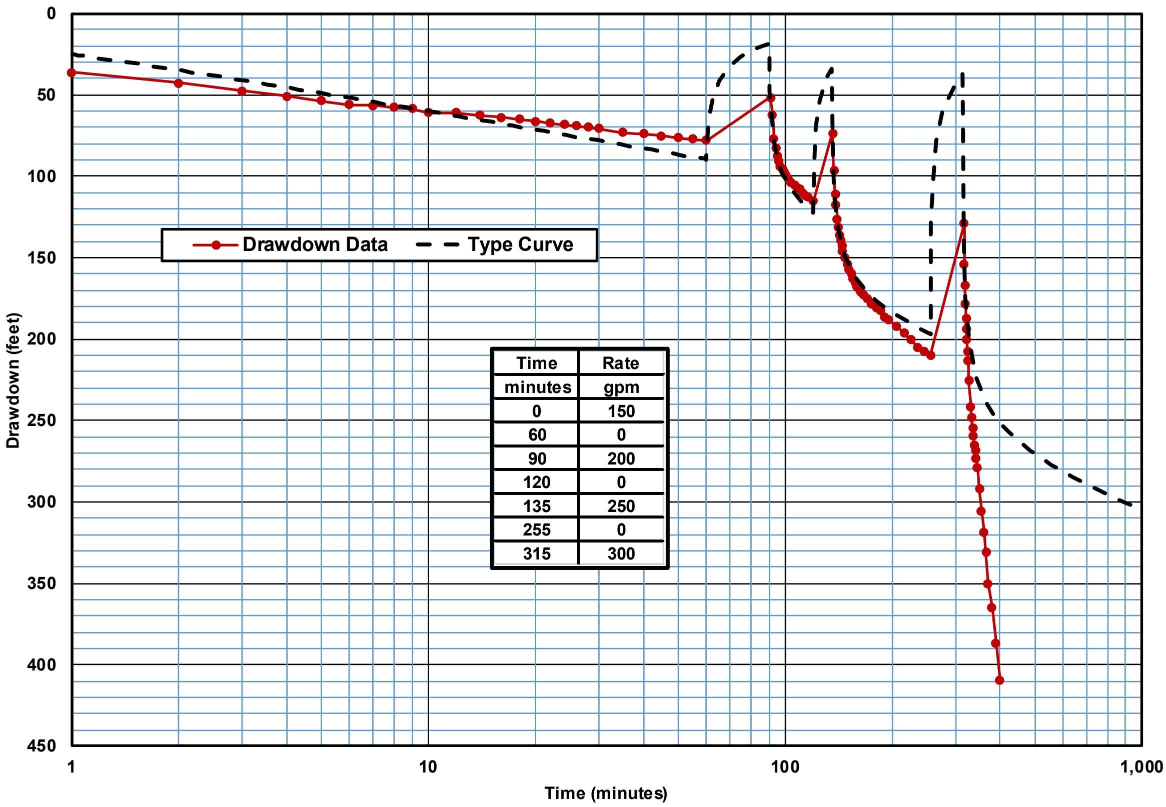


Figure I7a. Poolesville well 9 (Powell)– Semi-log plot of drawdowns from steps 1−3 of a multi-rate test, Hantush-Jacob leaky aquifer solution.

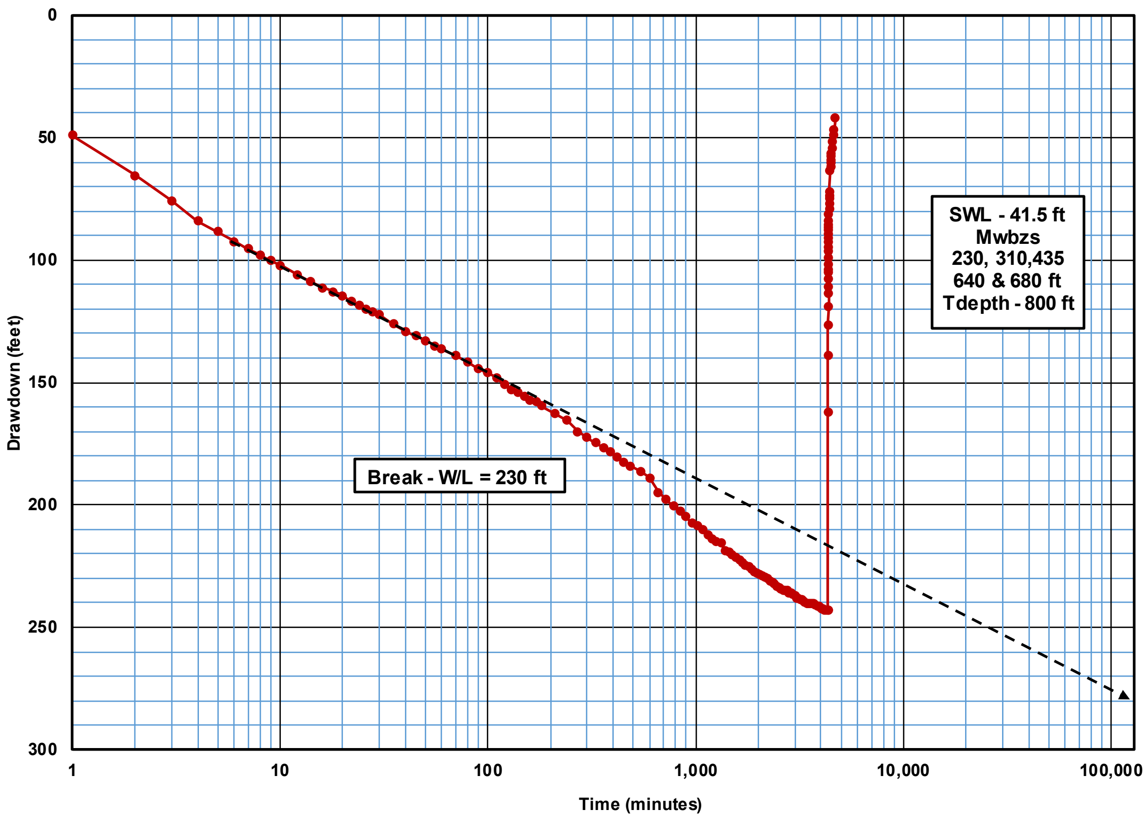
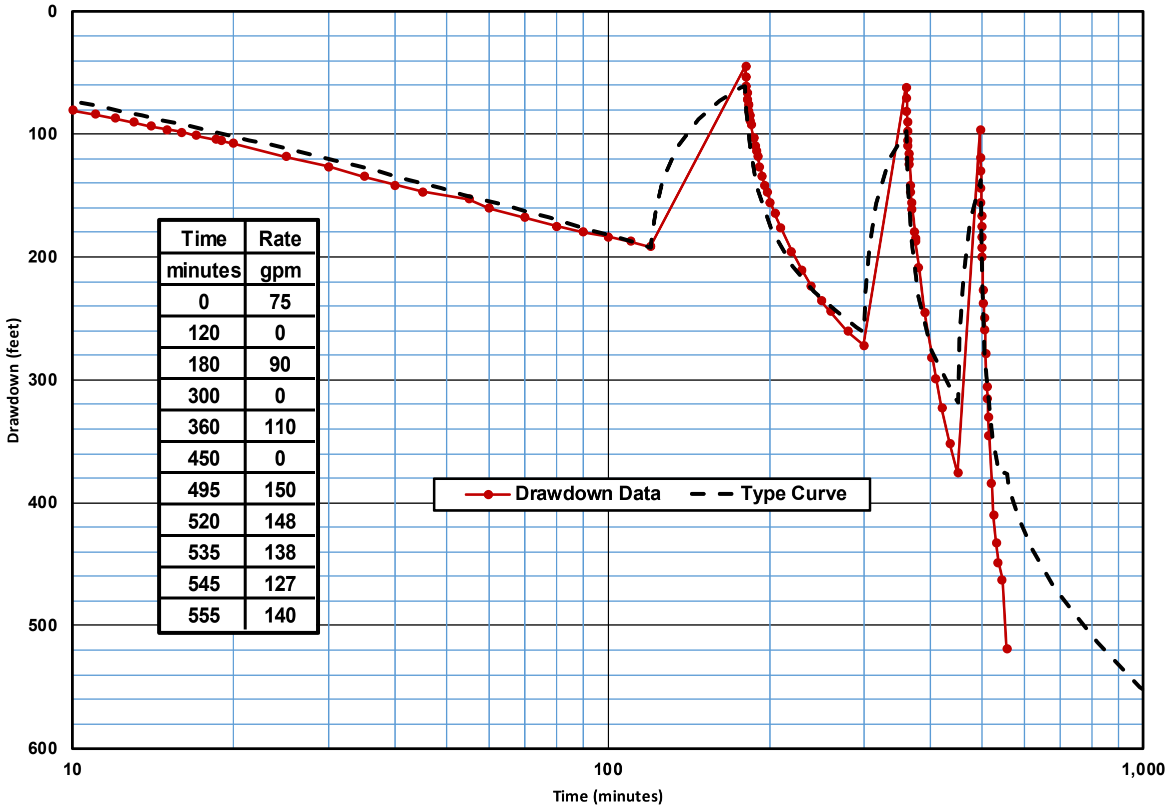


Figure I7b. Poolesville well 9 (Powell) – Semi-log plot of drawdowns from 72-h, 225 gpm aquifer test, 0-600 min Hantush-Jacob solution

I8. Well 10 (Cahoon)

A multi-rate test (fig. I8a) and a 72-h, 80 gpm (303 L/min) aquifer test (fig. I8b) were performed on Cahoon well 10 in May 2001, under average climatic conditions. The Hantush-Jacob leaky aquifer solution best fit the drawdown data; however, the data deviated from the model between 216 ft (66 m), during the second and third steps, and 288 ft (88 m), during the fourth step, possibly due to aquifer dewatering effects. The Hantush-Jacob solution also provided a good fit to the aquifer test data until 3,600 minutes, when a break in the drawdown data occurs at s = 320 ft (98 m) or a water level of 345 ft (105 m). The geologic section is described as consisting mostly of siltstones interbedded a few shale beds. There are no obvious reservoir units, therefore the break in data probably reflects a change in the bulk aquifer permeability. Much like the response noted during the test of Poolesville well 7, the relative stability of the water level prior 3,600 minutes suggests the yield of well 10 should be slightly less than the test rate of 80 gpm (303 L/min), possibly 70−75 gpm (265−284 L/min).



Time	Rate
minutes	gpm
0	75
120	0
180	90
300	0
360	110
450	0
495	150
520	148
535	138
545	127
555	140

Figure I8a. Poolesville Cahoon well 10 – Semi-log plot of drawdowns from a multi-rate test, Hantush-Jacob leaky aquifer solution.

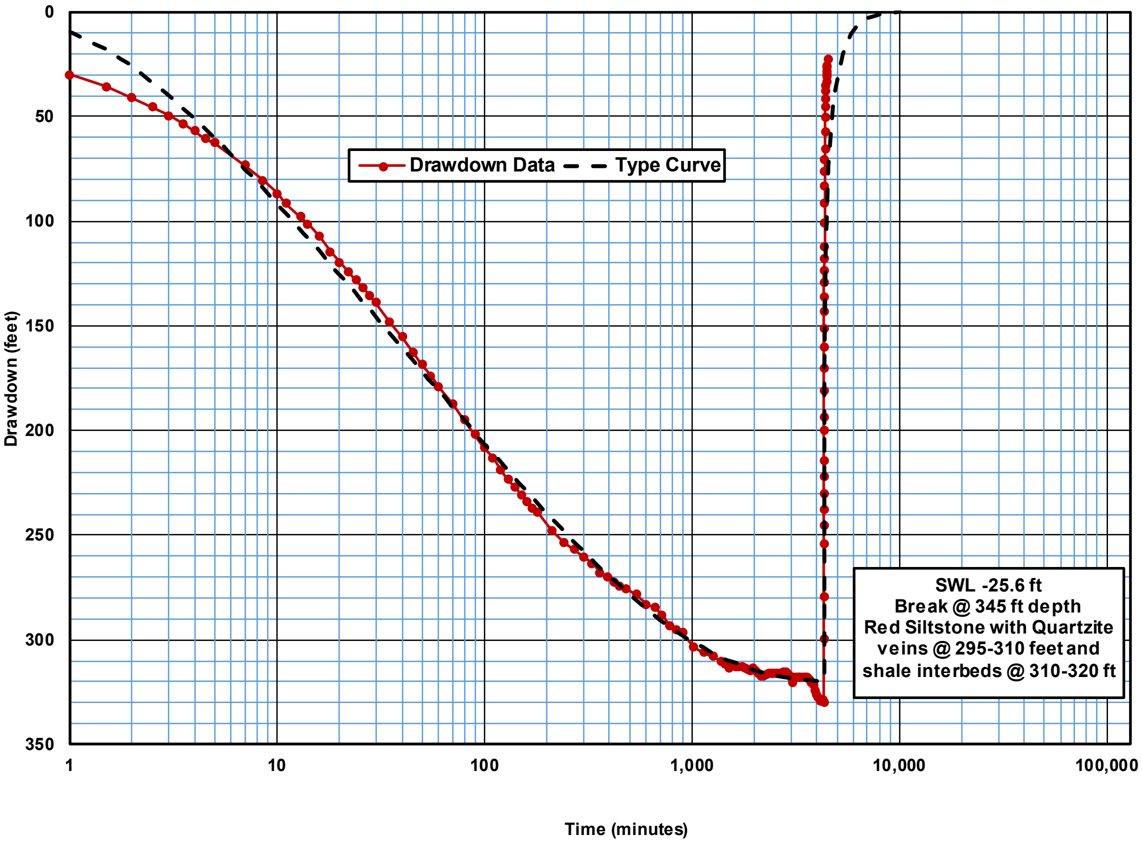


Figure I8b. Poolesville Cahoon well 10 – Semi-log plot of drawdowns from 0–3600 min of a 72-h, 80 gpm aquifer test, Hantush-Jacob leaky aquifer solution.

I9. Effects of well interference between Powell Well 9 and Cahoon Well 10 on reliable yield estimates

Due to the relatively proximity to each other (1,300 feet), potential interference between wells 9 and 10 was expected. Therefore, the production from the two wells must be analyzed in aggregate. A long-term operational test was conducted from December 30, 2003, to February 29, 2004, with both wells operating near full capacity for the first 45 days and then with well 10 shut down for the last 17 days. During the first period, the production rate averages were 172,274 gpd (652 m^3/d) or 120 gpm (454 L/min) from well 9 with an operating water level of 152 ft (46 m), and 87,971 gpd (333 m^3/d) or 61 gpm (231 L/min) from well 10, with a water level of 275 ft (84 m). The test was run under average climatic conditions and the production was 82 percent of the total of the individual predicted well yields.

Clear evidence of well interference occurred when well 10 was shut down near the end of the test. Well 9 continued to be pumped at an average of 126 gpm (477 L/min) for about 5 days, after which time the rate was reduced, leading to a recovery of water level in that well to 90 ft (27 m) by the end of the 17-day period. The water level in well 10 recovered immediately from 275 ft (84 m) to 150 ft (46 m) during the first day, but then the water level slowly recovered to 136 ft (41 m) at the end of the period. If there was no interference between the wells, the water level in well 10 should have recovered to something near its original SWL of 26 ft (8 m).

Further evidence of interference occurred in 2007, when domestic wells in the Sugarland Forest community, located up to one mile south of wells 9 and 10, were impacted, by lowered or lost yield, and turbidity problems, due to pumping from the two public supply wells. The impacts were remediated by drilling 300-foot replacement domestic wells.

Well 10 has essentially been out of service since 2003. Recent production, in November 2015, from well 9 has been 63.5 gpm (240 L/min) operating 13.7 h/d, with a water level of 110 ft (34 m), under average water table levels, but possibly under lagging low recharge conditions. A 72-hour operational test was conducted that month that produced an average of 108 gpm (409 L/min), with a water level of 152 ft (46 m) or 90 percent of the 2003−2004 operational test. It appears that the yield may have declined somewhat, but it is not clear why this happened. Since there was still substantial available drawdown, this may suggest that a reduced pump capacity caused the decline in the yield of well 9. Whether this is the case should be resolved by additional testing and evaluation.

I10. Well 12 (Schraf)

A step-drawdown test (fig. I10a) and 48-h variable-rate test (fig. I10b) were conducted on (Schraf) well 12 (MO-94-3610) in October 2005. The Dougherty-Babu solution fit the first two steps of the step-drawdown test, but the data diverged from the model after about 120 ft (37 m) of drawdown. The 48-h aquifer test started at 175 gpm (662 L/min), producing a steady-state drawdown until after 180 minutes and 96 ft (29 m) of drawdown, when the data diverged from the trendline. The break in the drawdown curve occurred at about the same depth as the base of the casing at 118 ft (36 m), or 93 ft (28 m) of drawdown, which is also the depth of the first minor wbzs. The sharp decline that followed may have been due to dewatering of the remaining shallow wbzs. Extrapolating the early-time data to 90 days produces a drawdown of 180 ft (55 m). When that result is applied to the 96-ft (29 m) break in the drawdown curve, the estimated yield is 94 gpm (356 L/min).

A long-term test (60 days) of well 12 was conducted October 2009 − December 2009. The well produced 43 gpm (163 L/min), running 24 h/d, with a water level of 103 ft (31 m). The well later produced 55 gpm (208 L/min) in December 2018, while operating 11.3 h/d (about 93 gpm, or 352 L/min, at 24 h/d), with a water level of 112 ft (34 m). The substantial difference in yield probably was due to interference with well 2, located 1,850 ft (564 m) from well 12, since well 2 was also operated 24 h/d during the long-term test and was shut down during the second period.

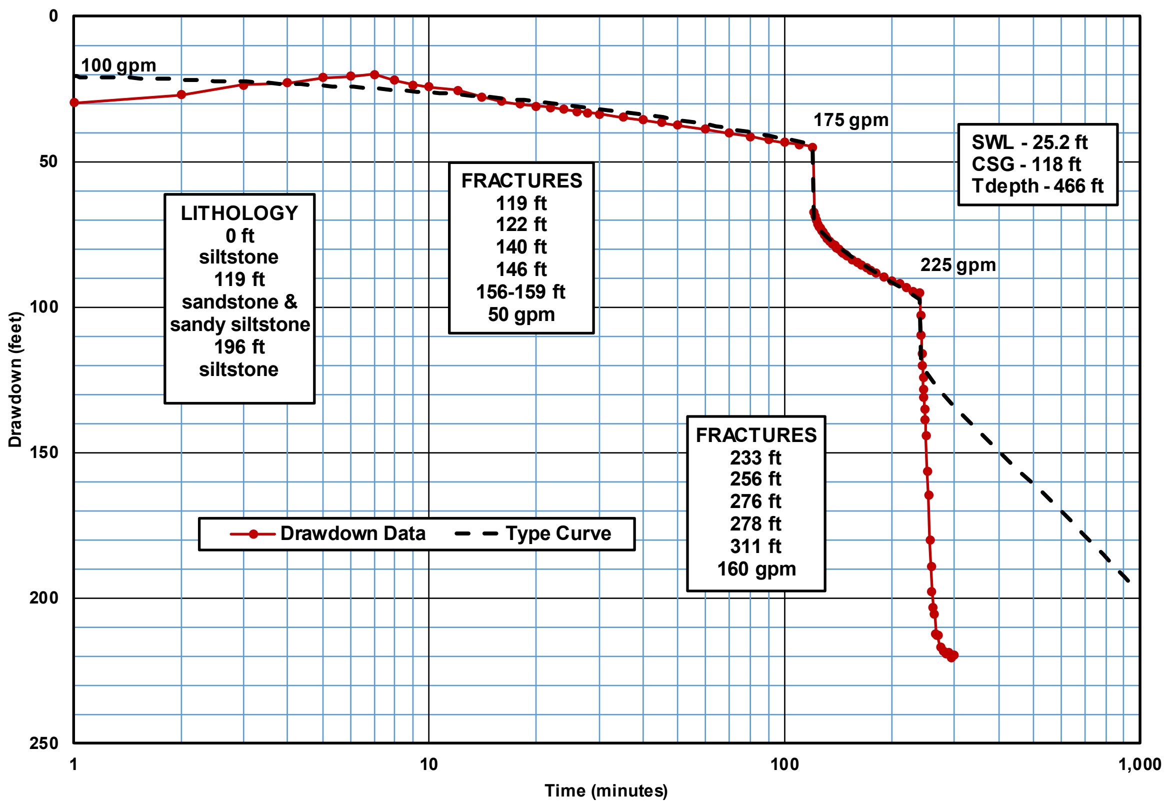


Figure I10a. Poolesville Schraf well 12 – Semi-log plot of drawdowns from a step-drawdown test, Dougherty-Babu double porosity solution.

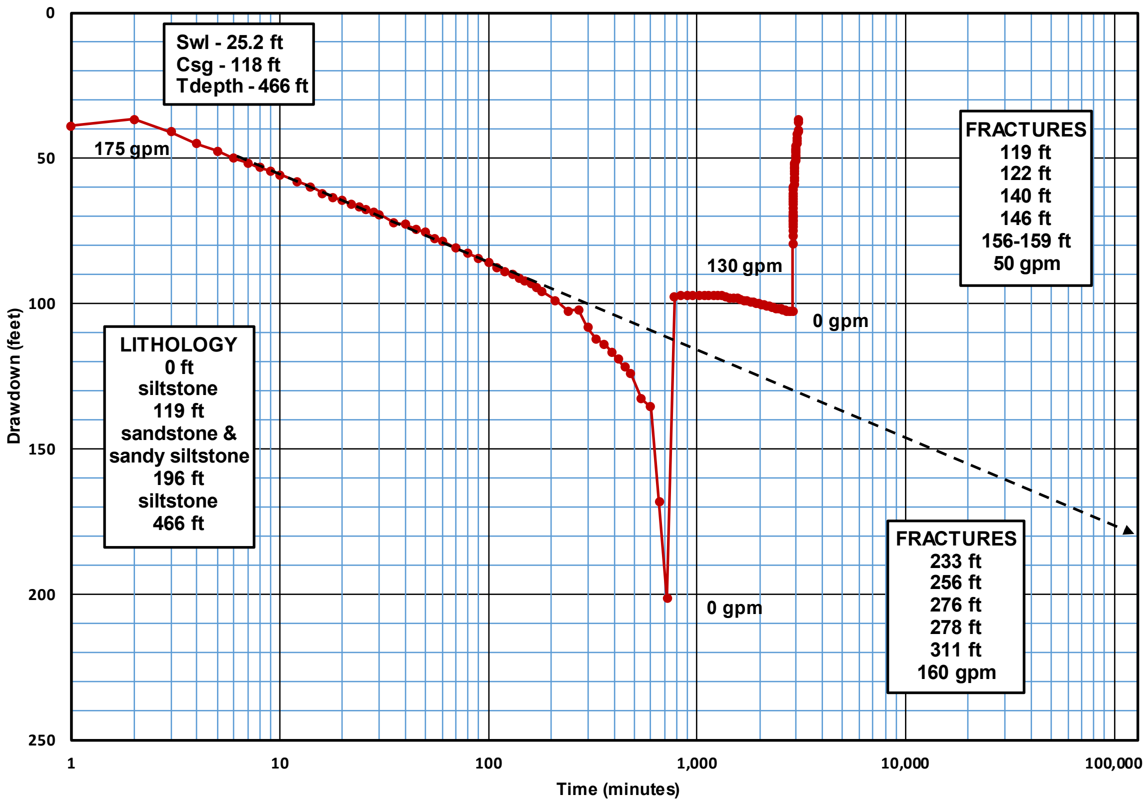


Figure I10b. Poolesville Schraf well 12 – Semi-log plot of drawdowns from 48-h, variable rate aquifer test, with estimated yield based on 90-d extrapolation from an IARF solution, 2-180 min.

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A review of daily data during the period 1996 to 2001 indicated that the Poolesville system could produce daily averages of about 555 gpm (2,101 L/min) under very wet conditions and 434 gpm (1,643 L/min) during a severe drought. The 22 percent decline is substantially less than the over 50-percent decline that was noted in Emmitsburg wells 3 and 5 during the same drought. The smaller decline in the Poolesville yields reflect that the wells were only operated to meet demand; while Emmitsburg's wells 3 and 5 were pumped continuously, leaving less water in aquifer storage to supply those two wells at the start of the drought. In addition, there was a lag in the changes of Poolesville's well yields relative to recharge/drought events of about 4−6 weeks, which was confirmed by a dye trace study involving Poolesville well 2. In contrast, the lag in response to recharge/drought event was about 4 months in the Emmitsburg wells. This may be due to different hydraulic characteristics, especially between the low permeability of the bedrock portion of the crystalline rock aquifer in Emmitsburg and the higher permeability of the consolidated sedimentary rock aquifer in Poolesville and/or the difference in leakage constants.

There was evidence that, after being placed into service, the yields of multiple consolidated sedimentary rock wells in Poolesville declined about 40−50% each. Lowering of water levels in the wells may have caused calcite precipitation to clog fractures, but only one of the wells, Poolesville well 7, is known to have penetrated a limestone layer. It is possible, however, that calcite cementation was present, but not noted, in the sandstone units of the other wells. Another, and more likely, possibility is that natural fractures in the wells or aquifer may have closed over time, due to deformation as water levels in the wells declined. In some cases (e.g., Poolesville well 6, as well as Taneytown well 13), reservoir rock units were dewatered that caused reduced yields; but, in only one known case (Poolesville well 7) was a discrete, water-bearing fracture drained, but the reservoir unit in that well was also dewatered. In a few cases (e.g., Poolesville wells 5, 8 and 9) where yields declined, it did not appear that reservoir rocks were dewatered. These data suggest that drawdowns may have exceeded elastic limits of the aquifers in the vicinity of the wells, causing permanent damage to each well. Until additional research is conducted, this suggests that water levels should not drop below reservoir units, instead of using the commonly held precaution of limiting drawdowns to the depths of discrete, water-bearing fractures. A similar problem could not be detected during the crystalline rock well investigations, possibly due to the adjustment of the weathered zone to seasonal variations in water levels, and the low permeability and porosity of the bedrock portions of those aquifers.

Another factor to consider is that some of the Poolesville wells with reduced yields have or had pumps installed that were substantially oversized. Kawecki (2001) mathematically compared continuous to cyclic pumping from a well using the Theis analytical model and demonstrated that continuous pumping of a well at the maximum sustainable rate produced considerably more water. In the present study, Poolesville well 7 initially had a well pump that was designed to be pumped continuously at or near the maximum sustainable rate. When a higher capacity pump was installed, the daily production declined substantially. Dahl flow control valves were installed in that well and well 6 by the Town's water system operators. While this allowed the wells to be pumped continuously without excessive drawdowns, the production from those two wells has remained at reduced levels. In 1999, Poolesville's practice was to limit operation of the wells to 16 hr/d, apparently with the thought that resting the wells might increase their yields. A review of daily records, however, indicated the maximum daily production occurred when the wells were pumped 24 h/d, while only 77% of the maximum yield was produced when the wells were pumped 16 h/d.

J. City of Taneytown

Figure Ja shows the locations of the City of Taneytown's public water supply wells and three domestic wells. In addition to well 13 discussed in this report, other important tests of interest in Taneytown include those conducted on wells 10 (abandoned), 10R, 14 and 17. Step tests and follow-on long-term aquifer tests were performed on all four wells. Some production data are available; however, each well has complicating factors which make verification of reliable yields a challenge, including interference (well 10), impacts to domestic wells (well 14), and no use (recent installation) or under-utilization (wells 10R and 17).

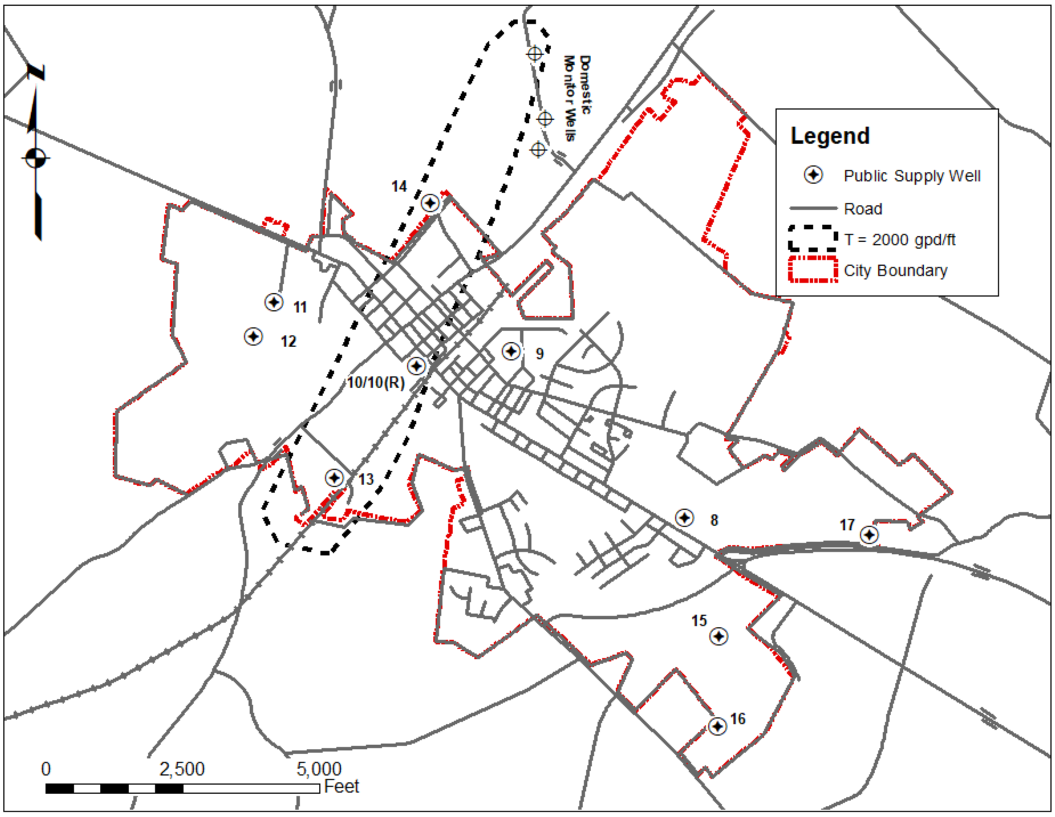

Figure Ja. Map showing the locations of the City of Taneytown's public supply wells and three domestic wells.

J1. Wells 10 and 10R

Well 10 (CL-67-0338) was completed in 1967 and the driller reported that there was a green sand at 80−120 ft (24−37 m) and a white sand at 415−438 ft (126−134 m). The green sand was isolated by a casing that was set from 0 to 131 ft (0 to 40 m) in the well. No explanation could be found for this, but it was likely related to a borehole stability problem. The yield during a 24-h test by the driller was 298 gpm (1128 L/min), with a drawdown to 400 ft (122 m) from a SWL of 39 ft (12 m). There were wbzs at 100 ft (30 m), 215 ft (66 m) and 415 ft (126 m). The well was online from the 1960s to the early 1990s and was then taken out of service due to declining yields related to interference with well 13. In 1999, a step-drawdown test and 100-h aquifer test were conducted on well 10 (figs. J1a and J1b).

At the time of the step-drawdown test the SWL was 149 ft (45 m) or about 100 ft (30 m) below regional water levels, providing clear evidence of the interference with well 13. The Dougherty-Babu solution best fit the first 4 steps of the 7-step test; however, the data diverges from the type curve at a drawdown of 105 ft or 32 m (253 ft or 77 m BTOC) when all seven steps were analyzed. This corresponds to a depth near the base of a green sandstone noted in the geologist's report at 209−240 ft (64−73 m) in well 10R, located 10 ft (3 m) from well 10. This sandstone is a possible reservoir unit limiting drawdown to that break point.

The long-term test started at 350 gpm (1,325 L/min) and ended at 165 gpm (625 L/min). The Hantush-Jacob leaky aquifer model best fit the drawdown data prior to the late-time equilibration (0−840 minutes), with a 90-d extrapolated drawdown of about 430 feet (131 m); which, when the specific capacity is applied to the estimated available drawdown of 128 feet produces a yield of about 71 gpm, at the average test rate of 237 gpm. There were no operational or water use data to confirm that estimate. The T value derived from the step-drawdown test was 1,145 gpd/ft.

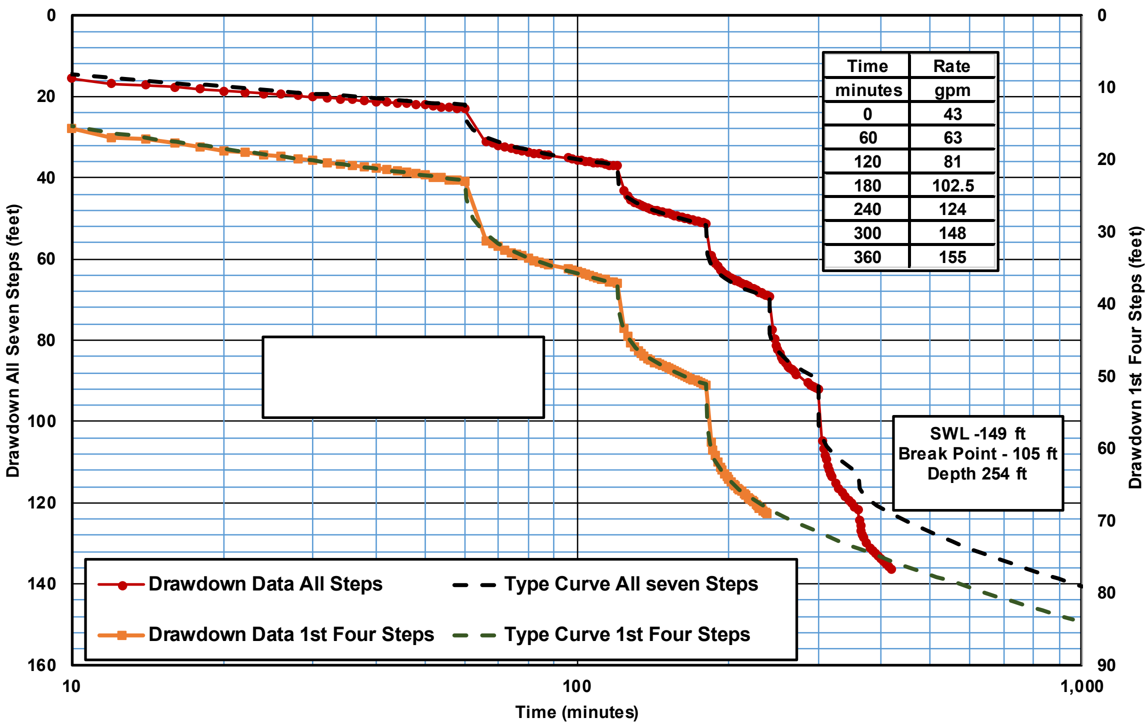

Figure J1a. Taneytown well 10 – Semi-log plot of drawdowns from all steps and the first four steps of a step-drawdown test, Dougherty-Babu solution.

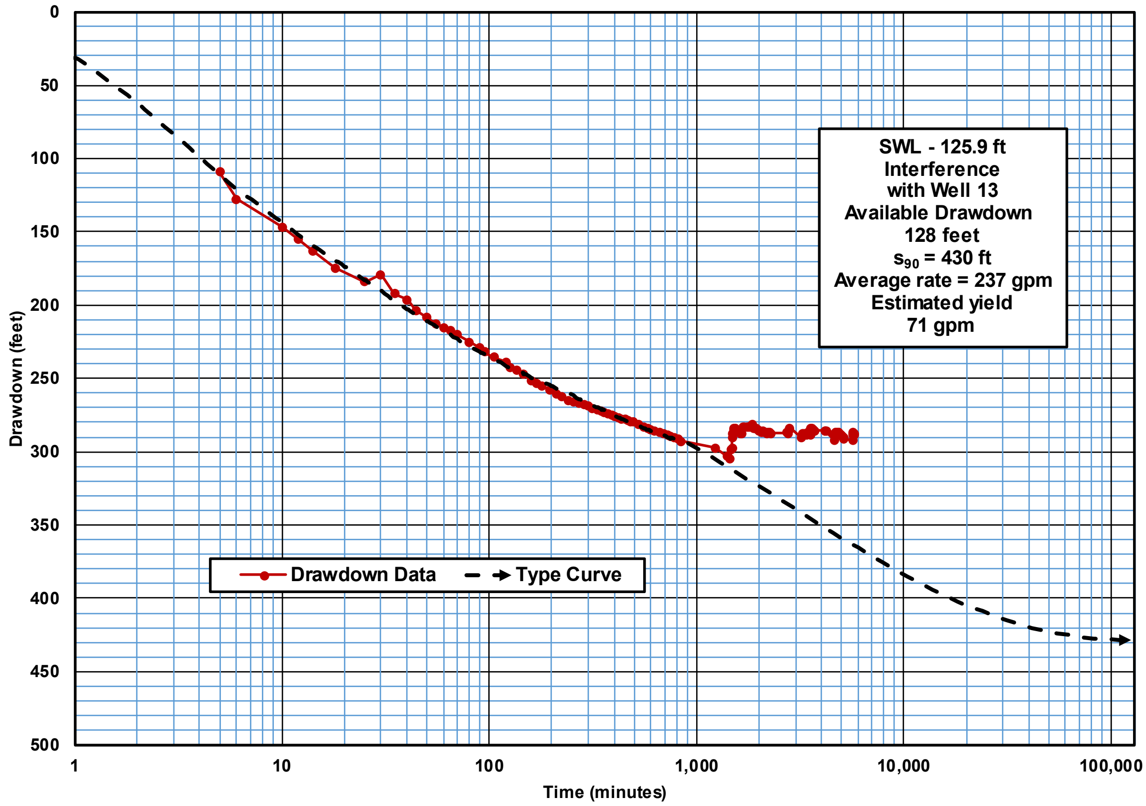

Figure J1b. Taneytown well 10 – Semi-log plot of drawdowns from a 100-h, variable rate aquifer test, 0−840 minutes Hantush-Jacob leaky aquifer solution.

In 2010, well 10R (CL-95-1690) was drilled as a replacement for well 10 (10 ft or 3 m away), primarily because there was a hole is the well 10 riser pipe that could not be repaired. It encountered a coarse–grained, arkosic, grey sandstone between 80 and 111 ft (24 and 34 m), a green sandstone at 209−240 ft (64−73 m) and another grey sandstone at 429−441 ft (131−134 m), all of which correspond to the possible reservoir units noted in wells 10 and 13. Also, like wells 10 and 13, the casing was set from 0−138 feet (0−42 m) such that the upper sandstone was isolated. That construction reduced the blown yield from 200+ gpm to 4−5 gpm. Several minor wbzs were encountered during drilling, until reaching the primary wbz at 280−283 feet (85−86 m).

Both a step-drawdown test and a 72-h pumping test were conducted in October 2010 (figs. J1c and J1d). The AQTESOLV® solution for the Dougherty-Babu dual porosity aquifer model provided a good fit to the step-drawdown test data, with no breaks in the drawdown curve. This indicates that no dewatering of the aquifer occurred above the maximum drawdown of 154 ft (47 m), which was well above the green sandstone unit noted in the geologist's log. The SWL was 29 ft (9 m) and had recovered 120 ft (37 m) from the 1999 SWL, since well 13 was no longer interfering because it was out of service due to radiological contamination.

The Moench dual porosity solution best fit the drawdown data from the 72-h aquifer test, and when extrapolated to 90 days (325 ft or 99 m) produced an estimated yield of 183 gpm (693 L/min) when applied to 223 ft (68 m) of available drawdown. This estimated yield is more than 100 gpm (379 L/min) greater than that of well 10, which is further evidence that the yield of well 10 was reduced due to interference with well 13.

The T value derived from the step-drawdown test of well 10R was 2239 gpd/ft or nearly twice that of the step-drawdown test of well 10. This result supports dewatering of a reservoir unit by interference during the 1999 test of well 10 and the higher estimated yield of well 10R.

To date, there has been no reported pumpage data from well 10R and there are no available records of pumpage from well 10 either. The data from testing of these two wells were analyzed and presented to support the concept of reservoir units providing the source of water to wells in fracture rock aquifers, in addition to serving as a good example of well interference.

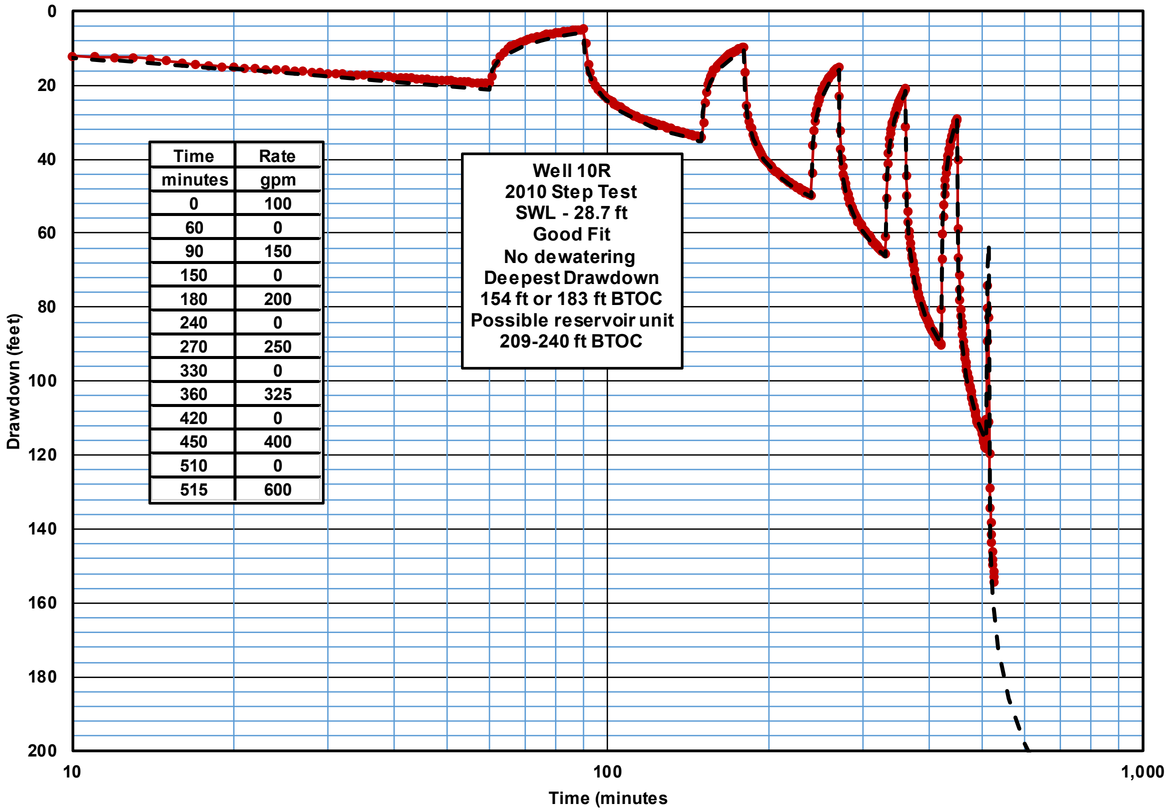

Figure J1c. Taneytown well 10R – Semi-log plot of drawdowns from a step-drawdown test, Dougherty-Babu double porosity solution.

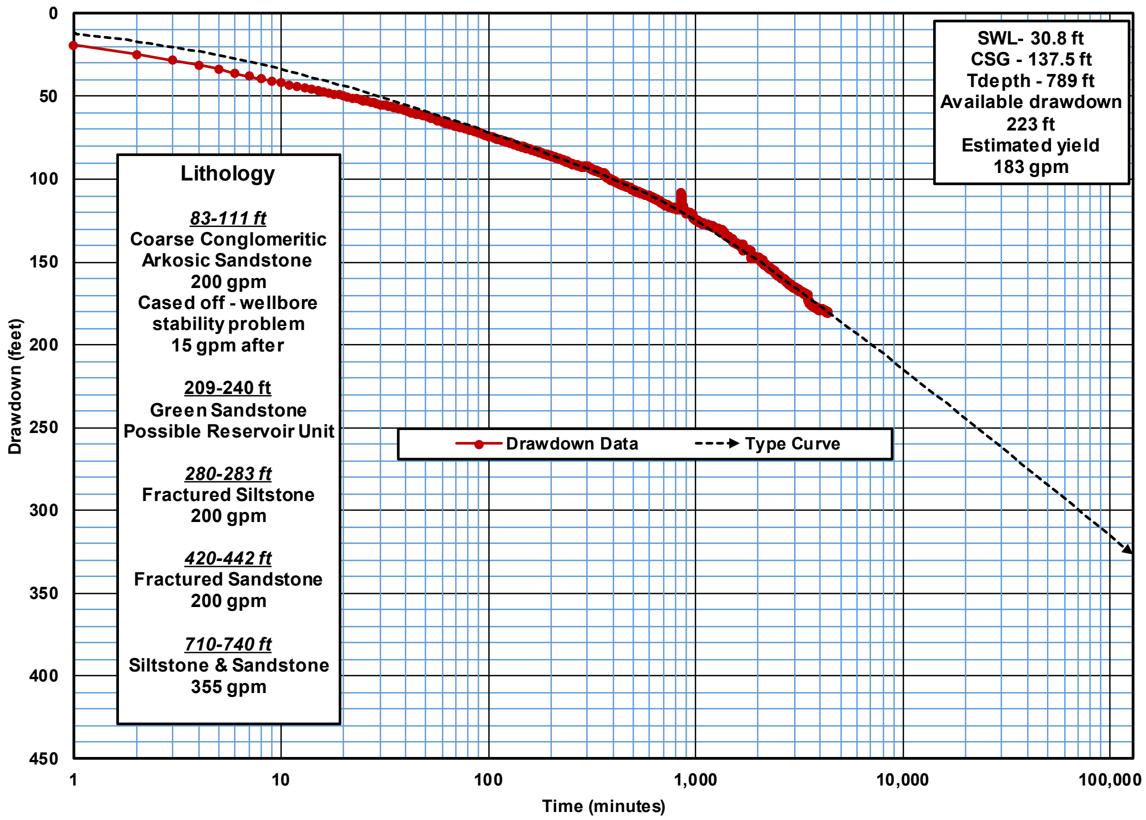

Figure J1d. Taneytown well 10R – Semi-log plot of drawdowns from 72-h, 275 gpm aquifer test, Moench double porosity solution.

J2. Well 14

Well 14 (CL-88-1060) was completed in 1990 to a total depth of 615 ft (187 m). The driller reported a light brown sandstone in the interval 68−183 ft (21−56 m), which the consultant described primarily as a shale. A step-test was conducted, starting at 23.5 gpm (89 L/min), and ending at 95 gpm (360 L/min). The Dougherty-Babu solution matched the drawdown data, but the RSS was high (7,770 $ft^{2)}$ with a well efficiency of 24 percent. When a window of 0−400 minutes was applied to account for disruption of the test due to the shutdown of the pump at 400 minutes, the RSS was 311 ft^2 and the well efficiency was 37 percent. Through an iterative process, the best fit to the data was achieved when a correction was made for dewatering of a 270−ft (82 m) thick aquifer, producing a RSS of 91 ft^2 and a well efficiency of 57 percent.

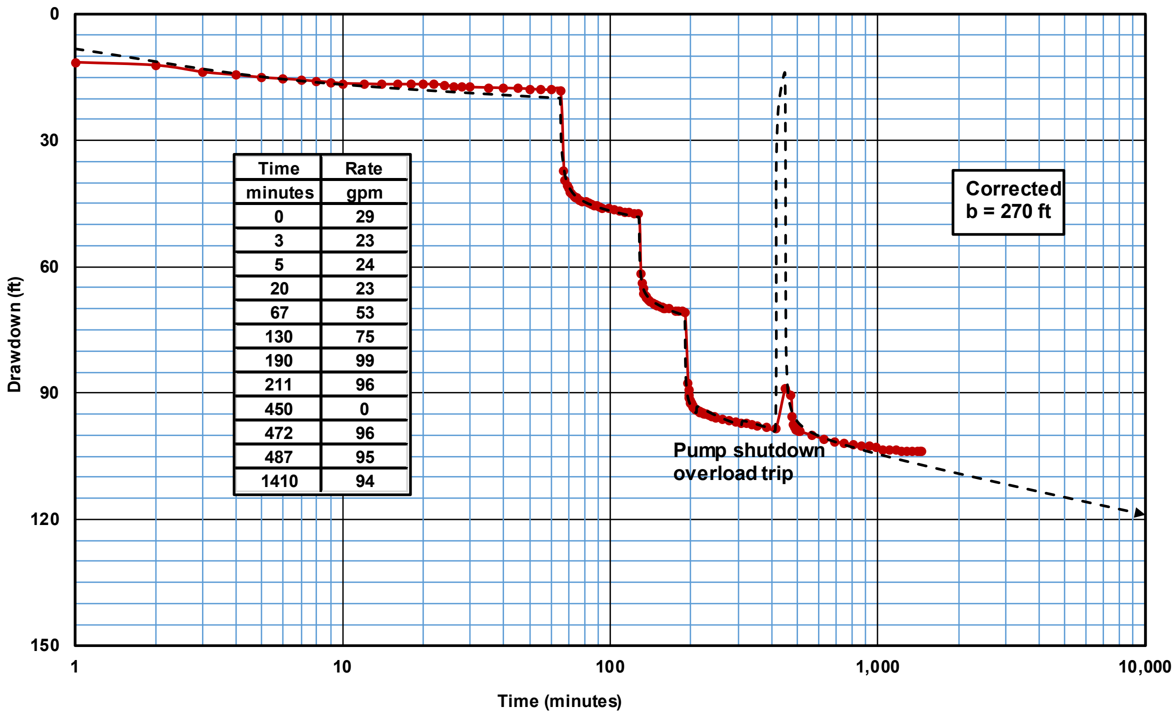

Figure J2a. Taneytown well 14 – Semi-log plot of drawdowns from a step-drawdown test, Dougherty-Babu solution, drawdowns corrected for dewatering with *b* = 270 ft.

A follow-on 72-hour variable-rate test, averaging about 225 gpm (852 L/min), was performed that produced about 350 ft (107 m) of drawdown by the end of the test (J2b). There are three segments to the drawdown curve. The first one is from 0−13 minutes and the data best fits the Gringaten-Witherspoon model for a single vertical fracture, with a final drawdown of 180 ft (55 m). The data from the second segment (13-1500 minutes) best fits the Theis radial flow model, suggesting that there is a deep reservoir unit under confining conditions. During the last segment from 1500 minutes until the end of the test, there was an initial rapid drawdown starting at s = 264 ft (80 m) that was about twice what would have been predicted by the Theis model, suggesting dewatering of the deep reservoir unit. This corresponds to the corrected aquifer thickness (270 ft or 82 m) in the step test solution.

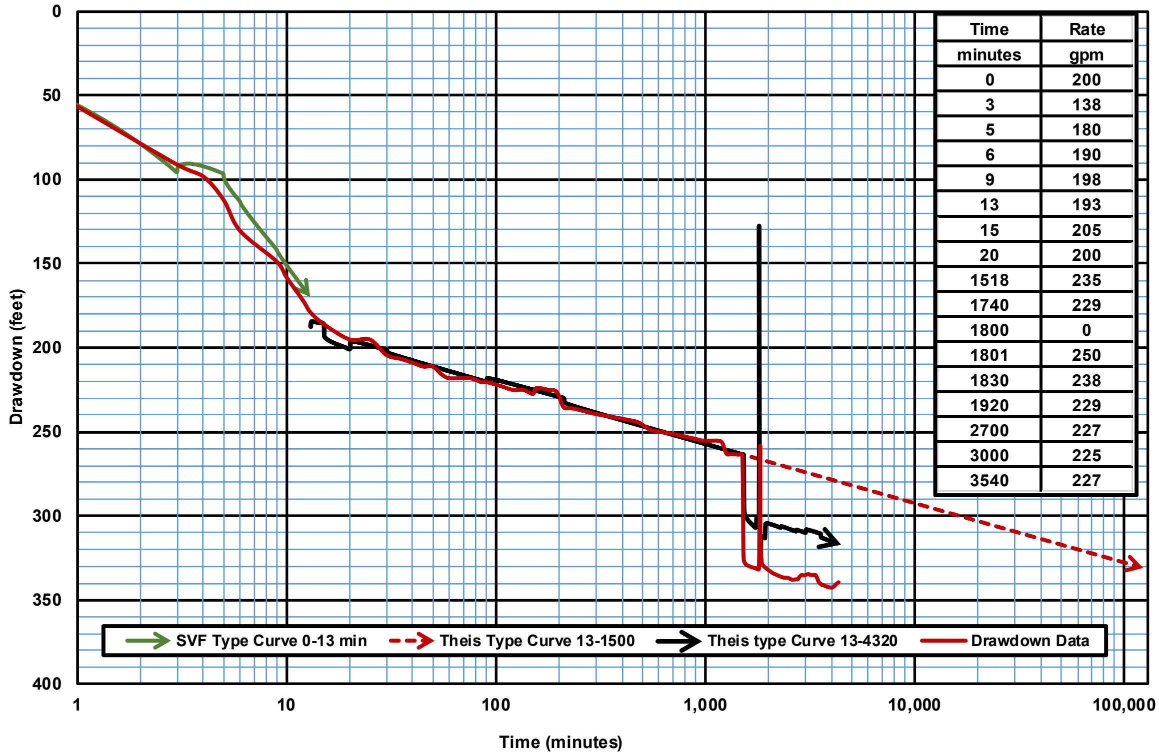

Figure J2b. Taneytown well 14 – Semi-log plot of drawdowns from 72-h, variable rate aquifer test, Gringarten-Witherspoon SVF solution (0−13 min) and Theis solution (13-4320 min), with 90-d extrapolation from 1500 min.

From the geologic log and the results of the step-drawdown and aquifer tests, it appears that there may be a shallow, relatively low permeability unit, composed of shales, siltstones and possibly sandstones overlying a higher permeability sandstone unit. Drillers/geologist logs indicate 1there is a light brown sandstone at 281 ft (86 m) or s = 266 ft (81 m) that may be that reservoir unit. The logs also indicate that there are deep (below 390 ft or 119 m) water-bearing fractures, primarily in siltstones and shales. If the estimate is based on the drawdown to the deep reservoir unit (s = 264 ft or 80 m) and a 90-d drawdown of 331 ft (101 m), the estimated yield would be 160 gpm (606 L/min), based on the average test rate of 200 gpm (757 L/min), prior to 1518 minutes.

In 1993, the initial permitted use for well 14 was 227,000 gpd (859 m^3/d) avg. or 158 gpm (598 L/min) avg. but was later adjusted to 70,000 gpd (265 m^3/d) avg. / 105,000 gpd (397 m^3/d) max. or 49 gpm (185 L/min) avg. / 73 gpm (276 L/min) max. in 1998. That change was based on a concern about the long-term yield of the well and its potential impacts to domestic wells along Fringer Road, about ½ to ¾ mi (0.8 to 1.2 km) north-northeast of well 14. At that time, the Water Management Administration (WMA) estimated yields were 83 gpm (314 L/min), 119 gpm (450 L/min) and 142 gpm (538 L/min), based on different 60-day extrapolations and on an available drawdown of 227 ft (69 m). Due to the possible impacts to domestic wells, located 2200 to 3300 feet north of well 14 (fig. Ja), the city was required to measure water levels in some of the domestic wells and well 14.

The data collection started in July 1998 and included monthly measurements of the pumping and recovered water levels in well 14 that has continued to date. During the drought period of 1998 to 2002, the well produced 15 gpm (57 L/min) avg. to 47 gpm (178 L/min) avg. and 30 gpm (114 L/min) max. to 75 gpm (284 L/min) max., while operating with a pumping water level of about 140 ft (43 m).

During the following non-drought period of 2003 to 2010, the well produced 46 gpm (174 L/min) avg. to 75 gpm (284 L/min) avg. and 65 gpm (246 L/min) max. to 97 gpm (367 L/min) max., with a pumping water level of about 170−180 ft (52−55 m). The maximum reported pumpage in any year, 75 gpm (284 L/min) avg. / 96.5 gpm (365 L/min) max. occurred in 2004, when the pumping water levels were about 165 ft (50 m). In 2012, production was 47 gpm (178 L/min) avg. and 54 gpm (204 L/min) max., with operating levels of about 185 ft (56 m) during January 2012 – June 2012 and 250 ft (76 m) during July 2012 – December 2012.

The yield may have declined by as much as 50 percent between 2004 and 2017, since the use in 2017 was 38 gpm (144 L/min) avg. and 39 gpm (148 L/min) max., with an operating water level of about 184 ft (56 m) during July – December 2017. As with well 13 and Poolesville's wells 6 and 7, dewatering of the aquifer may have reduced the yield of well 14. It is likely that the reliable well yield was never close to the average test rate of 225 gpm (852 L/min).

J3. Well 17

In June 2009, a step-test (fig. J3a) and 72−hour, 250 gpm (946 L/min) aquifer test (fig. J3b) were conducted in well 17 (CL-95-1336). Initially, there was a very poor fit to the step drawdown data using the Hantush-Jacob solution, producing a well efficiency of 3 percent; although when S.I. units were used, the computer generated well efficiency was 98 percent. The well has four green sandstone units in the well above 600 ft (183 m), two of which are above 400 ft (122 m). When the drawdown data were corrected for dewatering using an aquifer thickness of 600 ft (183 m), there was still a poor fit to the drawdown data. When the data were then corrected for an aquifer thickness of 400 ft (122 m), there was a better fit to the data. When the data were then analyzed using a 0−90 minutes window (1st step and recovery), the best results were achieved, producing a well efficiency of 100 percent. This suggest that dewatering of the uppermost green sandstone unit may have influenced the test, but that one or more of the deeper sandstones may also serve as reservoir units.

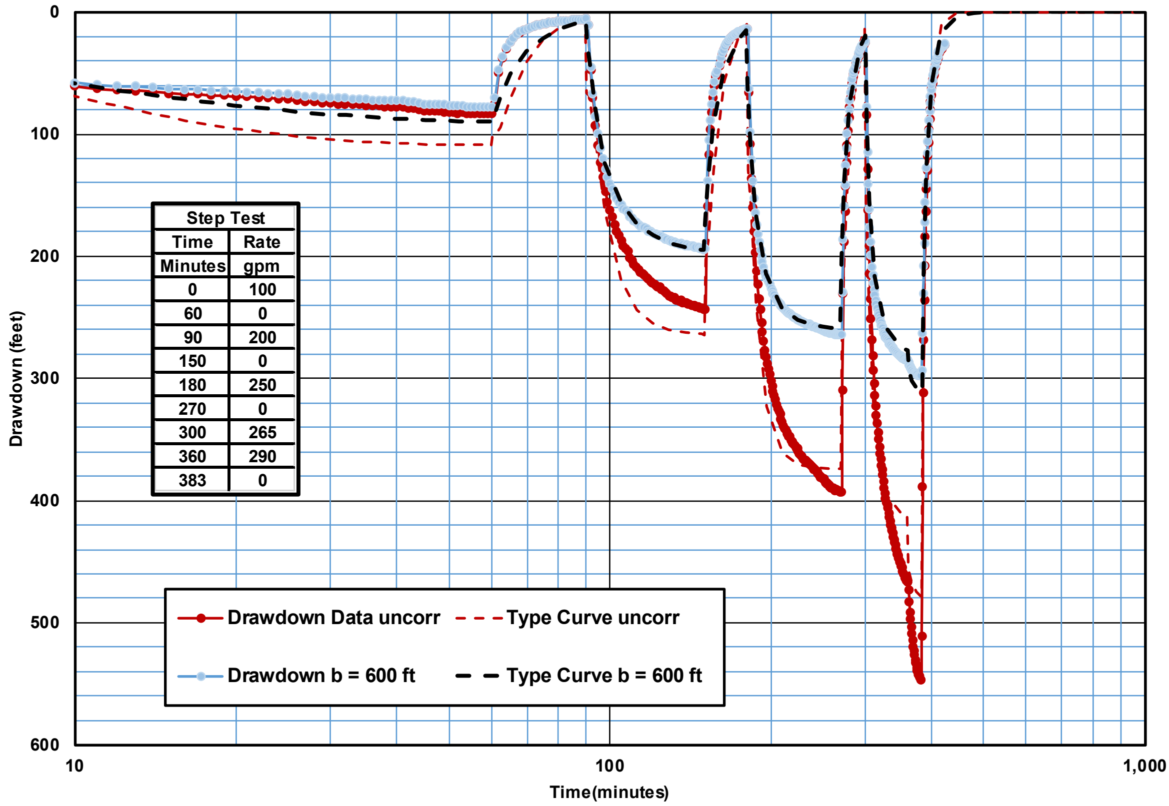

Figure J3a. Taneytown well 17 – Semi-log plot of drawdowns from a step-drawdown test corrected *b* = 600 feet, Hantush-Jacob leaky aquifer solution.

During the 72-hour test, there was no obvious dewatering of a reservoir unit as the drawdown reached about 620 ft (189 m); however, the results of the step test indicate that dewatering during the aquifer test may have been so rapid that any deflection in the drawdown curve could not be detected. Like well 14, the data from the early part (0–100 minutes) of the test best fit the Hantush wedge aquifer model (fig. J3b), producing a T of 297 gpd/ft (3.7 m^2/d) and the SVF solution provided the best fit to the later data (300–4320 minutes) and resulted in a T of 455 gpd/ft (5.7 m^2/d). Extrapolating the late-time drawdown to 90 days produces a drawdown of 830 ft (253 m). When the specific capacity at that point is applied to the maximum drawdown (620 ft or 189 m) observed during the 72-hour test, the estimated yield is 187 gpm (708 L/min). If applied to the corrected aquifer thickness of 400 ft (122 m), the estimated yield is then 120 gpm (454 L/min).

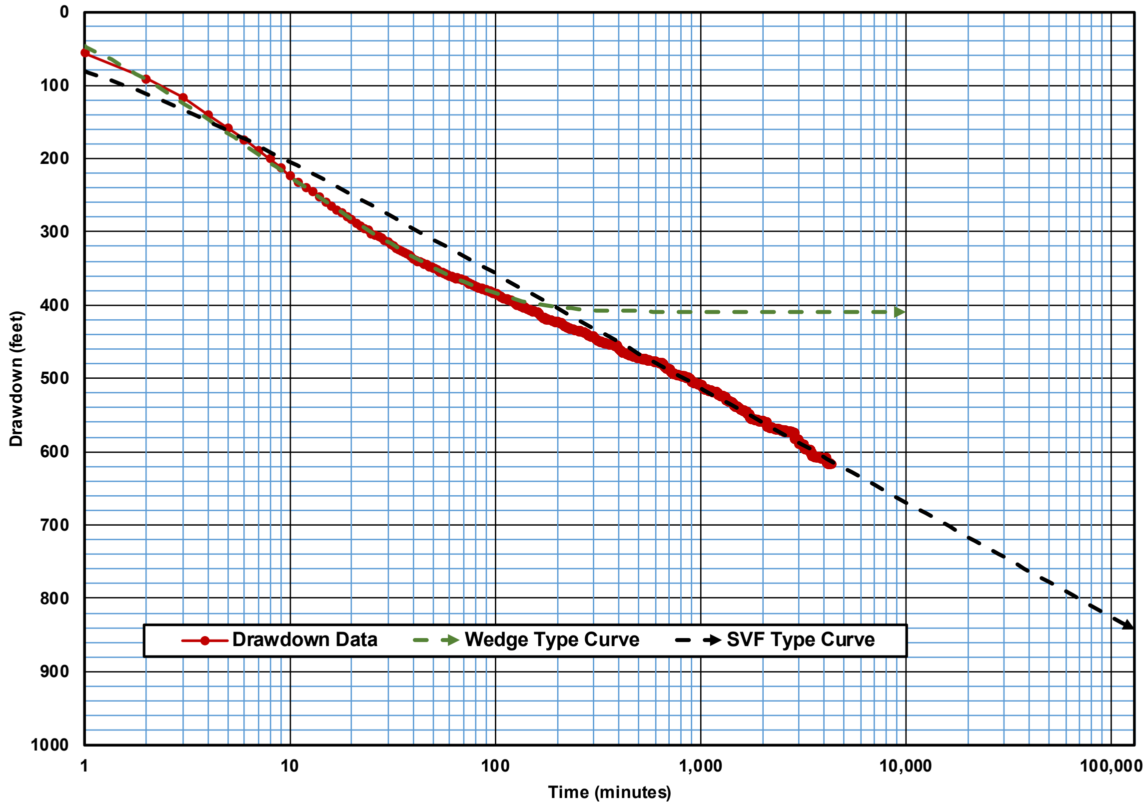

Figure J3b. Taneytown well 17 – Semi-log plot of drawdowns from 72-h, 250 gpm aquifer test, Hantush wedge shaped aquifer solution (0−100 minutes) and Gringarten-Witherspoon SVF solution (300−4320 minutes).

Well 17 was placed in service during 2015. A permit was issued for 19,100 gpd (72 m^3/d) avg. / 225,000 gpd (852 m^3/d) max. (156 gpm, or 591 L/min, max.), as a supplemental permit to meet peak demand until the yields and degree of potential impacts with that well were determined, and the system water demand was established. As a permit condition, monthly water levels in well 17 had to be recorded and reported to Maryland Department of Environment (MDE). The well has been pumped at relatively low and nearly constant rates of about 32,000 gpd (121 m^3/d) avg. and 35,000 gpd (132 m^3/d) max. (24 gpm, or 91 L/min, max.). The water levels have fluctuated between 60 ft (18 m) during periods of high regional water levels, and 289 ft (88 m) during periods of low regional water levels. These data indicate that the well yield may be substantially less than could be predicted from the pumping test data. One possible reason is that the available drawdown is much less than the 620 ft (189 m) observed at the end of the 72-h test. If the uncorrected drawdown at the end of the first step (84 ft or 26 m) were used, the calculated yield could be as low as 25 gpm (95 L/min). Additional testing should be considered, with emphasis on using standard step test procedures to compare the results to those of the previous multi-rate test.

K. Town of Thurmont Wells 8 and 8a

Thurmont well 8 (FR-88-3686) was completed in the Gettysburg Shale in 1993 and well 8A (FR-94-0911) was a replacement for well 8, completed in 1998. Figure Ka shows data collected during a test of well 8 conducted in September 1994, and a follow-on 1998 test of well 8A.

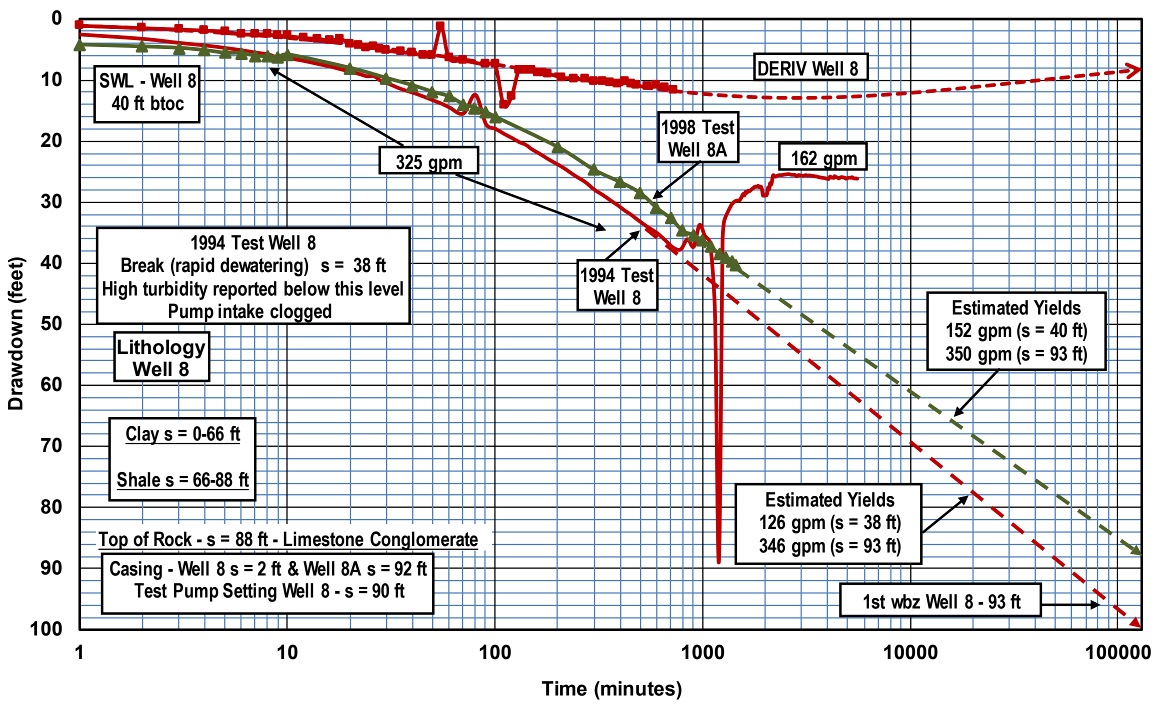

Figure Ka. Thurmont wells 8 and 8A – Semi-log plot of drawdown from aquifer tests of wells 8 (1994) and 8A (1998) and the logarithmic derivative for well 8, both wells pumped at 325 gpm, with estimated yields based on 90-day extrapolations from IARF solutions.

The test of well 8 started at 325 gpm (1,230 L/min) and a sharp break in the drawdown data occurred near the end of the first day. The consultant thought that the response was due to dewatering of an unidentified fracture. Pumping was secured after the first day and then re-started at 162 gpm (613 L/min), with the water level stabilizing until the end of the test. The derivative was approaching a constant value, so the author of the present study used the Cooper-Jacob straight-line solution to determine the estimated well yield. The result was an estimated yield of 126 gpm (477 L/min), when the drawdowns were extrapolated to the break in the data (s = 38 ft or 12 m), and 320 gpm (1,211 L/min), when the data were extrapolated to the first major water-bearing zone (s = 93 ft or 28 m). The Town's consultant developed a numerical model, from which a sustained yield of 226 gpm (854 L/min) was derived.

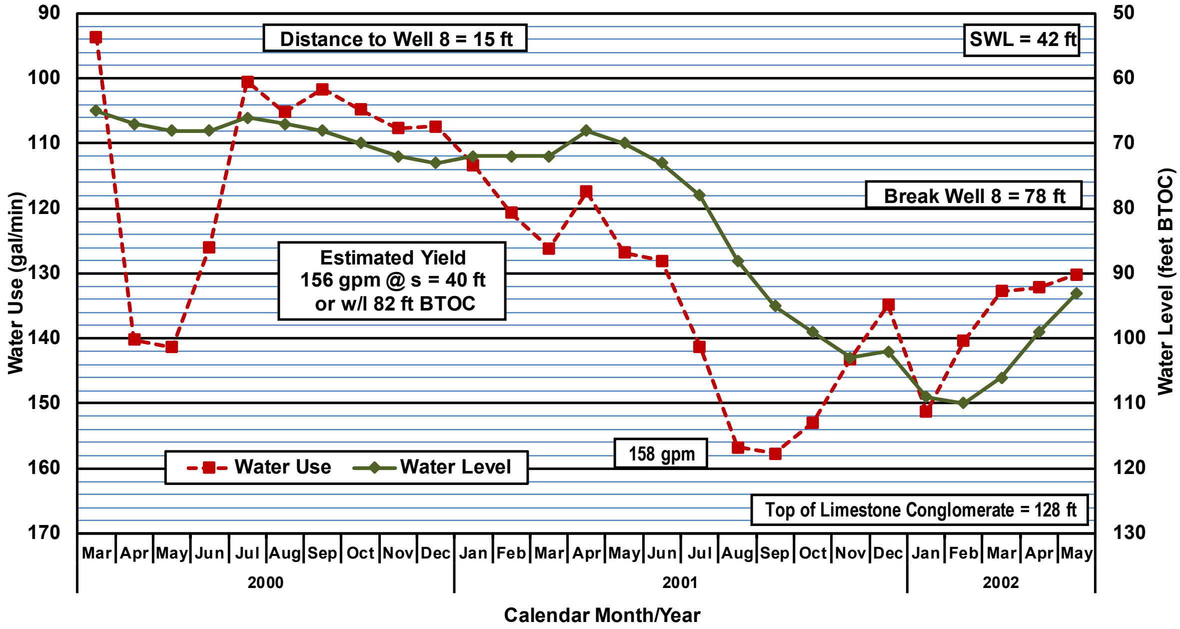

Figure Kb. Thurmont well 8A – Water use and water level data

Figure Kb shows production and water level data for Well 8A, collected during the drought of 2001−2002. This was a replacement well drilled 15 ft (4.6 m) from well 8, which was replaced because of high turbidity. In 2012, the operator reported that mud was discharged from well 8 during the 1994 test. This probably caused the pump filter to become clogged, which was the likely reason for the sharp decline in water level noted during that test. A screen was installed in well 8A, which helped reduce, but did not eliminate the turbidity problem. During the 24-hour test of well 8A, there was no sharp decline in water level though the test may have ended too soon for that effect to occur. Another explanation is that the screen was operating efficiently. The production data indicated that the maximum reliable yield from well 8A was a daily average of 158 gpm (598 L/min) during the drought, with the pumping water level within 10 feet of a basal limestone conglomerate. This was close to the minimum estimated yield 152 (575 L/min) derived by extrapolating to the drawdown (40 ft or 12.3 m) that occurred at the end of the 1998 test of well 8A.

The bedrock in well 8 was described by the driller as a "calico" rock, which is a term describing a limestone conglomerate at the base of the Gettysburg Shale. That formation outcrops in a small area along the Triassic Border Fault (Cleaves et al., 1968), which is located about 2,460 ft (750 m) north of wells 8 and 8A. In addition, located about 650 ft (198 m) west of the two wells is the contact with the massive Frederick Limestone, which is relatively impermeable, except where solution features are present. The presence of a no-flow barrier would provide an explanation for the sharp declines in water levels noted in the production data from well 8A and during the test of well 8, except that the typical doubling of the late-time slope for an impervious barrier was not observed during the test of well 8. In addition, using hydraulic constants estimated from the test of well 8A, time-drawdown calculations indicate that the trough of depression did not reach the contact with the Frederick Limestone during the test of well 8. Those calculations, however, indicate that during normal operations the trough of depression could have reached the potential barriers and could have affected the long-term production from well 8A.

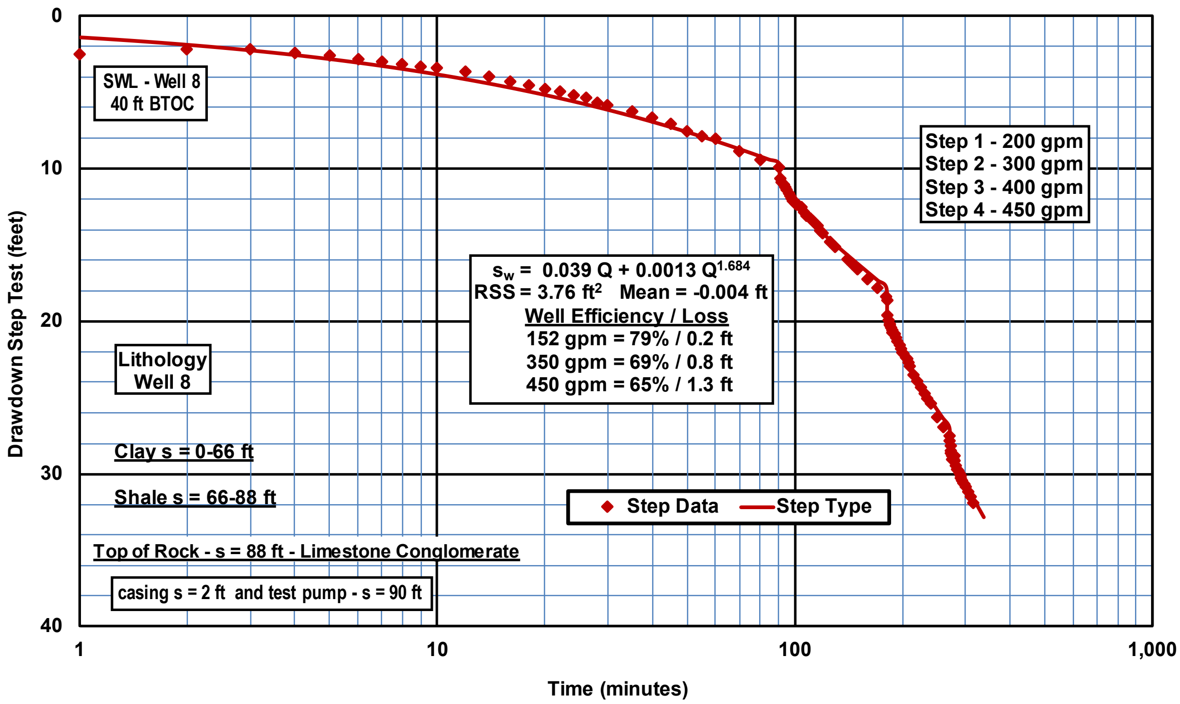

Figure Kc. Thurmont well 8 – Semi-log plot of drawdown from step-drawdown test, Dougherty-Babu double porosity solution.

The results of a step-test conducted on well 8 are shown in Figure Kc. A good fit to the Dougherty-Babu model was obtained without having to correct for any dewatering effects. The results of the step-test and the presence of 66 ft (20 m) of clay in the upper portion of the saturated zone indicate that the aquifer is confined in the vicinity of well 8. No fractures were identified nor would be expected to occur in the clay layer, and the alluvial mountain wash was unsaturated. This suggests that the sharp decline in water level during the test of well 8 was neither due to fracture zone nor weathered zone dewatering. The most plausible explanation appears to be that the elevated turbidity noted when the water level dropped below 78 ft (24 m) may have clogged the pump intake screen, reducing the capacity of the well and effectively limited the available drawdown to that level. The presence of a barrier may, also, have affected the long-term yield of the well. The step-drawdown test further indicated that, at the reliable estimated yields of 126 gpm or 320 gpm (477 L/min and 1211 L/min), the well efficiencies were 79 and 68 percent, respectively, while the well losses were negligible (0.2 ft and 0.8 ft., respectively). No step-test was conducted on well 8A, but the greater early drawdown during the pumping test of that well relative to that of well 8 suggests that its efficiency may be less and the well losses greater than well 8; but they were still, probably, minimal.