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INTERFERENCE IMPACTS CAUSED BY GROUNDWATER WITHDRAWALS FROM PUBLIC WATER SUPPLY WELLS IN THE CONSOLIDATED SEDIMENTARY ROCK AQUIFERS OF CENTRAL MARYLAND

by

Patrick A. Hammond

2022

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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)
Discharge Rate		
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)
Transmissivity		
gallon per day per foot (gal/d-ft)	0.0124	square meter per day (m^2/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)

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

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KEY RESULTS

The few published investigations of potential impacts of withdrawals in fractured rock aquifers are related to nearly horizontal, bedding plane, controlled groundwater flow or one exceptionally high yielding well field in a semi-confined limestone aquifer. There have been less than 100 known domestic wells, of nearly 200,000 completed in the State, that have been impacted by groundwater withdrawals in the fractured rock aquifers of Maryland. More than 90% of those impacts can be attributed to withdrawals by Poolesville and Taneytown municipal wells, and dewatering of the Mettiki Coalmine, all in consolidated sedimentary rock formations, and dewatering of limestone quarries throughout the state. This investigation presents case studies of the results of testing and monitoring at the Taneytown well 14 and Poolesville wells 9 and 10 sites. Also, presented is a case study at the Cloverhill III subdivision. Although no impacts are known to have occurred at that site, it is the first project that included mandatory monitoring during several aquifer tests and for which a numerical model was developed. The results at that site were led to the development of the aquifer test and monitoring protocols presently used by the Maryland Department of the Environment (MDE) in consolidated sedimentary rock aquifers. A companion study of the impacts caused by dewatering of the Mettiki Coalmine has previously been completed. Added to the ¼ fracture length (¼ L) model that was developed in a study of impacts of withdrawals in the crystalline rock aquifers in Maryland was a model for an anisotropic aquifer. Such an aquifer is usually described as having a high transmissivity along a major axis and a low transmissivity along a minor axis, while the shape of drawdown contours is determined by the ratio of the two transmissivities. Modelling simulations indicate that transmissivity values derived from the aquifer tests were reliable, but that the storage constants were underestimated by at least an ½ order of magnitude, probably due to lags in drawdowns in observation wells. The combination of the ¼ L method and anisotropic aquifer model produces improved results over common radial flow solutions, but the expected error with their application is about 25% or more.

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. The aquifers vary from high yielding (wells commonly producing more than 500 gpm) in confined and unconfined, unconsolidated sandstone layers on the eastern shore and southern Maryland to relatively, low yielding aquifers (wells generally producing less than 100 gpm) in the fractured rock areas 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 complex, where about 5 million people live. Most of the metropolitan area is served by surface water from the Potomac River and the Baltimore City reservoir system. Some of the fastest growing suburban areas, however, are in the Piedmont and Blue Ridge areas, and are supplied by wells in fractured rock aquifers.

There have been about 100 known domestic wells, of nearly 200,000 completed in the state, that have been impacted by groundwater withdrawals in the fractured rock aquifers of Maryland. More than 90% of those impacts can be attributed to withdrawals by Poolesville and Taneytown municipal wells, and dewatering of the Mettiki Coalmine, all in consolidated sedimentary rock formations, and dewatering of limestone quarries throughout the state. In all those cases, the impacts to those domestic wells were successfully mitigated, mostly by drilling replacement wells, providing public water to affected homes, or by adjusting the withdrawals of the large users. Most of the projects included estimates of impacts made prior to withdrawals and post-audits to determine the reliabilities of those predictions.

Location of Study Area

In addition to case studies of impacts associated with Taneytown well 14 in Carroll County and Poolesville wells 9 and 10 in Montgomery County, the results of aquifer testing at Cloverhill III located in Frederick County is presented, Figure 1. Although no impacts are known to have occurred at that site, it was the initial testing from which the present methods for evaluating impacts in consolidated sedimentary rocks were developed.

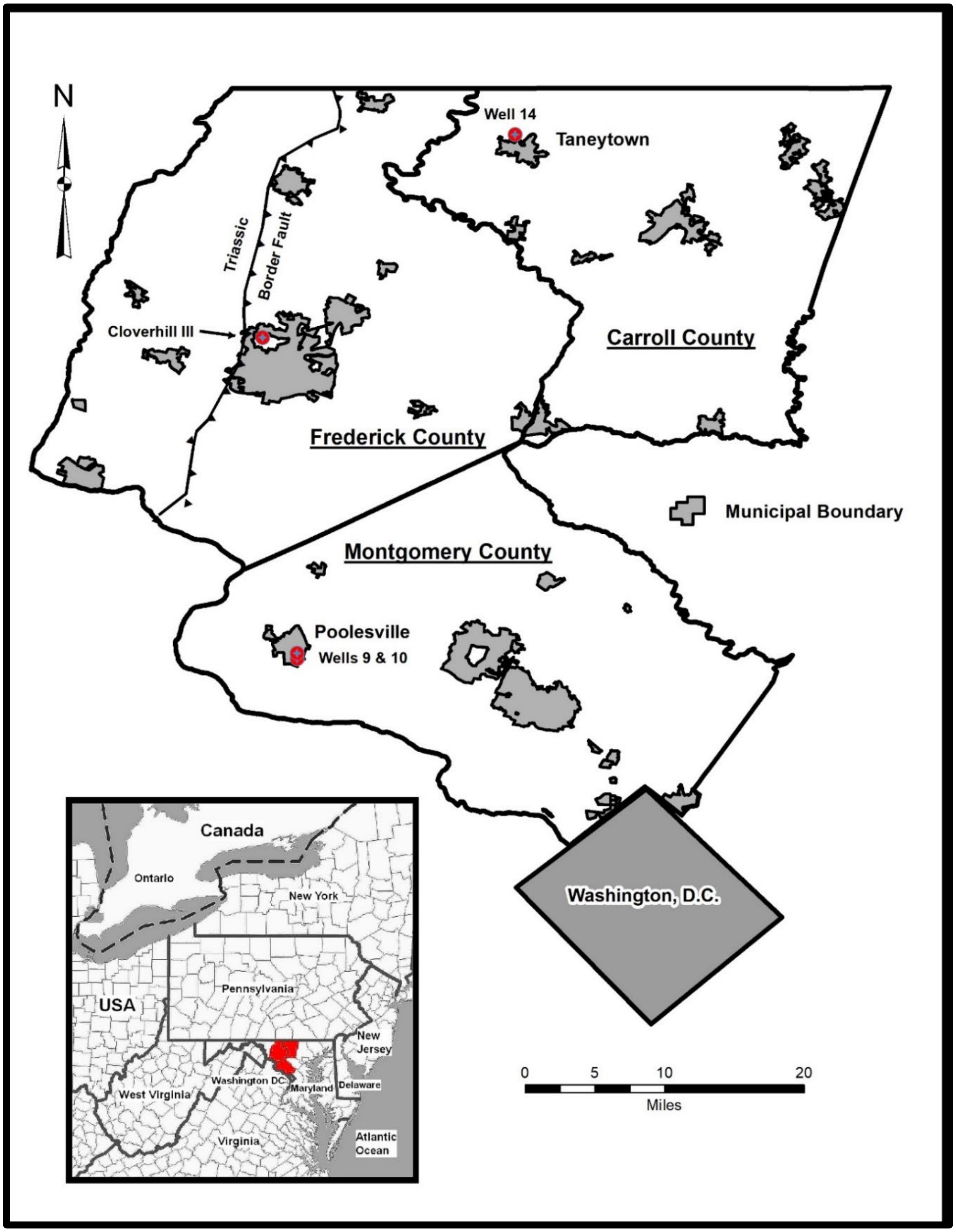


Figure 1. Location map of study area.

The History of Water Appropriation or Use Regulations in Maryland

The Water Appropriations Act of 1933 created regulatory authority over the appropriation of surface and ground waters for any use (with significant exemptions, especially for subdivisions, and municipal and agricultural users). The Well Drillers Law was passed in 1945 and addressed the issue of licensing well drillers. It also required permits before and completion reports after drilling of any water well, providing a wealth of data on the ground waters of the state. The permitting system for well drillers and water appropriations was one of the earliest such programs in the nation. The 1933 law was largely ignored until about 1957, when the “Regulated Riparian” system for surface water adopted. At that time, the “American Rule” or Reasonable Use Doctrine governed groundwater use, which states that a landowner has the right only to a reasonable and beneficial use of the waters upon his land. The reasonable use theory does not prevent the proper, non-wasteful consumption of such waters for the development of land for mining, allowing the underground waters of neighboring properties to be interfered with or diverted. In 1988, the water use regulations were modified based on the Restatement (second) of Torts, Section 858, which requires replacement of impacted water supplies, with some restrictions. They also require consideration of the aggregate and cumulative changes of new and future appropriations, and their contributions to future degradation of the state’s waters, which are provisions used to protect the hydrologic balance of the state’s water resources.

Previous Studies on the Physical Properties of Fluid Flow in Fractured Rock Aquifers

The two main factors controlling flow in groundwater aquifers are permeability and porosity. Early studies considered two-dimensional, infinite acting radial flow (IARF) analytical solutions for application in unconsolidated sedimentary rock aquifers, Theis (1935). With the advent of hydraulic fracturing of petroleum reservoirs, it was soon seen that these models were inadequate for analyses of test data from fractured rock wells.

Warren and Root (1963) introduced the concept of “double porosity” naturally fractured formations composed of low permeability matrix rock combined with natural fractures. Primary porosity is intergranular and controlled by deposition and lithification. Secondary porosity is controlled by fracturing, jointing and/or solution by circulating water. The fractures are generally vertical and formed by tensional failure during mechanical deformation. It was noted that the build-up curve associated with this type of porous system is like that obtained from a layered single porosity reservoir. Odeh (1965) and Carlson (1999), however, have indicated that responses during testing may be scale dependent, indicating that that fractures within a “tight-gas” sandstone reservoir are not necessarily interconnected over a well’s drainage area. In such a fracture network intersects a well, then the preferentially oriented high permeability extends only a short distance away from the wellbore. Beyond that distance, flow takes place through the lower permeability matrix that connects the fracture network to its neighboring networks. Given sufficient time, the pressure response of a formation with disjoined fracture networks assumes the character associated with radial flow.

A second body of work involved the development of deterministic models to describe the flow characteristics of individual fractures in single porosity systems. Gringarten (1982); however, indicated that these discrete fracture models best apply to small-scale geotechnical projects, while the continuum approach is appropriate for groundwater and petroleum engineering investigations.

Although considerable research has been done on contaminant transport in porous media in unconsolidated rock formations, fewer studies have described transport in complex and variable fractured-rock terranes. An early work by Vecchioli (1967) indicated drawdowns during a pumping test in the fractured Brunswick Shale in New Jersey exhibited directional rather than isotropic hydraulic behavior, and that maximum and minimum directions of anisotropy were related to the structural strike and dip of the formation, respectively. Measurements of well yields by Vecchiola et al. (1985) indicated that ground water flow in the Brunswick Shale occurs mainly in discrete zones controlled by bedding. Spayd (1985) investigated the movement of groundwater contaminated with volatile organic compounds through the Brunswick Shale in Fairlawn, New Jersey. The direction of groundwater flow was predominantly parallel to strike resulting in two overlapping plumes, each over 4,000 feet in length. Carlton et al. (1998) found that the layered sedimentary rocks in the Newark Basin commonly contain water-bearing partings along bedding planes in fissile layers separated by massive layers with virtually no such partings. Joint sets perpendicular to bedding planes can transmit water across the massive layers separating fissile zones. Many fractures visible in outcrop indicated that individual fractures are not extensive; rather, they joined other fractures in an interconnected network.

Dimmen et al. (2020) proposed a system where flow in fractured rock structures and networks is primarily controlled by structure type (e.g., joint and deformation band), geometry (e.g., length and orientation), connectivity (i.e., number of connections in a network), kinematics (e.g., dilation and compaction), and interactions (e.g., relays and intersections) within the network. Additionally, host rock properties and depositional architecture represent important controls on flow and may interfere to create hybrid networks, which are networks of combined structural and stratal conduits for flow. Depositional architecture as such represents a fundamental control on fluid flow in consolidated sedimentary rocks in conjunction with structural controls on flow.

Hammond (2018) observed that leakage and IARF conditions were the dominant flow regimes during most of the hydraulic tests in that study. The responses reflected single-porosity, homogeneous, anisotropic aquifers produced by layering in weathered transition zones in crystalline rocks or sandstone/limestone units in consolidated sedimentary formations. Conversely, the modified dual-porosity Dougherty-Babu (1984) model provided the best fit to the time drawdown data collected during the step-tests. One possible explanation for this difference was that the rock matrix consists of numerous blocks, which are large relative to the volume tested during the step tests, but small compared to the reservoir size. A second possibility is that the fracture networks are discontinuous, where a well intersects a local fracture system that only extends a short distance from the wellbore beyond which a radial flow regime develops.

Methods of Investigation

All significant case studies in Maryland were reviewed where impacts are known to have occurred and testing produced significant drawdowns in nearby wells. These included analyses of aquifer tests that were performed, description of methods used to predict impacts, presentation of long-term monitoring data, comparisons of actual to predicted drawdowns, and methods used to mitigate the impacts.

The step and aquifer test data were analyzed using the computer assisted automated curve fitting AQTESOLV program, Duffield (2007), and the methods developed by Hammond and Field (2014) and Hammond (2018) for interpretation of those analyses.

Relatively simple analytical techniques were used to predict the impacts of well interference. An effective well radius (= ¼ fracture length), instead of the actual well radius, was used to calculate drawdowns in pumping wells. The heterogeneity of an aquifer was estimated by assigning higher storage and transmissivity values to the weathered zone relative to the bedrock portion of an aquifer. While this improves the estimates of drawdowns relative to the commonly used Theis (1935) and Cooper-Jacob (1946) methods, there can be substantial errors (about 50-100+%) involved when using this technique.

For interference studies in consolidated sedimentary rocks a model was developed to address the anisotropic nature of those type of aquifers. Such an aquifer is usually described as having a high transmissivity along a major axis and a low transmissivity along a minor axis, while the shape of drawdown contours is determined by the ratio of the two transmissivities. It was found that the methods developed by Papadopoulos (1965) were useful in describing the groundwater flow in consolidated sedimentary rock aquifers during pumping tests and for the prediction of impacts due to withdrawals.

These methods were developed in lieu of complex groundwater flow models, since reliable numerical analyses often require more data than are commonly available at most sites, and they are usually very costly and time-consuming. There are, however, a few studies where numerical analyses were used to define groundwater flow in fractured rock aquifers, such as van Tonder et al. (2001a), Rushton & Chen (1976) and Teideman & Hsieh (2001). None of these studies presented long-term test or monitoring data to confirm the reliability of the flow models. Over the past 25+/- years the Water Supply Program, first with the Maryland Department of Natural Resources (MDNR) and then the Maryland Department of the Environment (MDE) have collected long-term test or monitoring data from several dozen projects, mostly in Poolesville, Taneytown, Myersville, and Middletown, that will be used to verify the accuracy of the predictions of impacts made using the present MDE methods and techniques.

MDE now requires that an inventory be completed to identify nearby water supplies and determine which ones should be monitored during aquifer tests. The radial distances from a proposed production well to which inventories must be completed are based on case studies conducted by the State over the past 30 years. Those distances are: 1500 feet, crystalline rocks; 2000 feet, carbonate rocks; and 3000 feet, consolidated sedimentary rocks. The have been no impacts in crystalline rock aquifers outside of 1200 feet, although significant drawdowns (up to 26 ft) have been observed at distances up to 1760 feet during aquifer tests. There have been no impacts in carbonate rock formations (not including any associated with quarry activity), although a drawdown of three feet was observed at 3000 feet during one long-term aquifer test. Finally, impacts have occurred at distances more than 5000 feet from a pumping well in consolidated sedimentary rocks, as will be demonstrated in the present study.

Acknowledgements

This study fulfills one of the objectives of a cooperative regional study, Fleming et al., 2012, (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.

General Hydrogeology and Geology of the Study Area

All the public water supply wells in the present study are in the Mesozoic Lowland (ML) Hydrogeomorphic Region (HGMR), Brakebill et al. (1998), that is present in central and northeastern Frederick County, northwestern Carroll County, and western Montgomery County, Figure 2. 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 MDE Water Supply Program, the New Oxford Formation name is retained throughout this report.

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), and in some places, may be even higher, due to secondary solution of calcite cementing materials.

The ML is bounded on the west by the Triassic Border Fault, which separated the ML from the crystalline rocks of the Blue Ridge HGMR (BR). The Frederick and Wakefield valleys and small portions of the Piedmont Upland border the ML to the east and are underlain by rocks of the Piedmont Carbonate HGMR (PCA) and the Piedmont Crystalline HGMR (PCR).

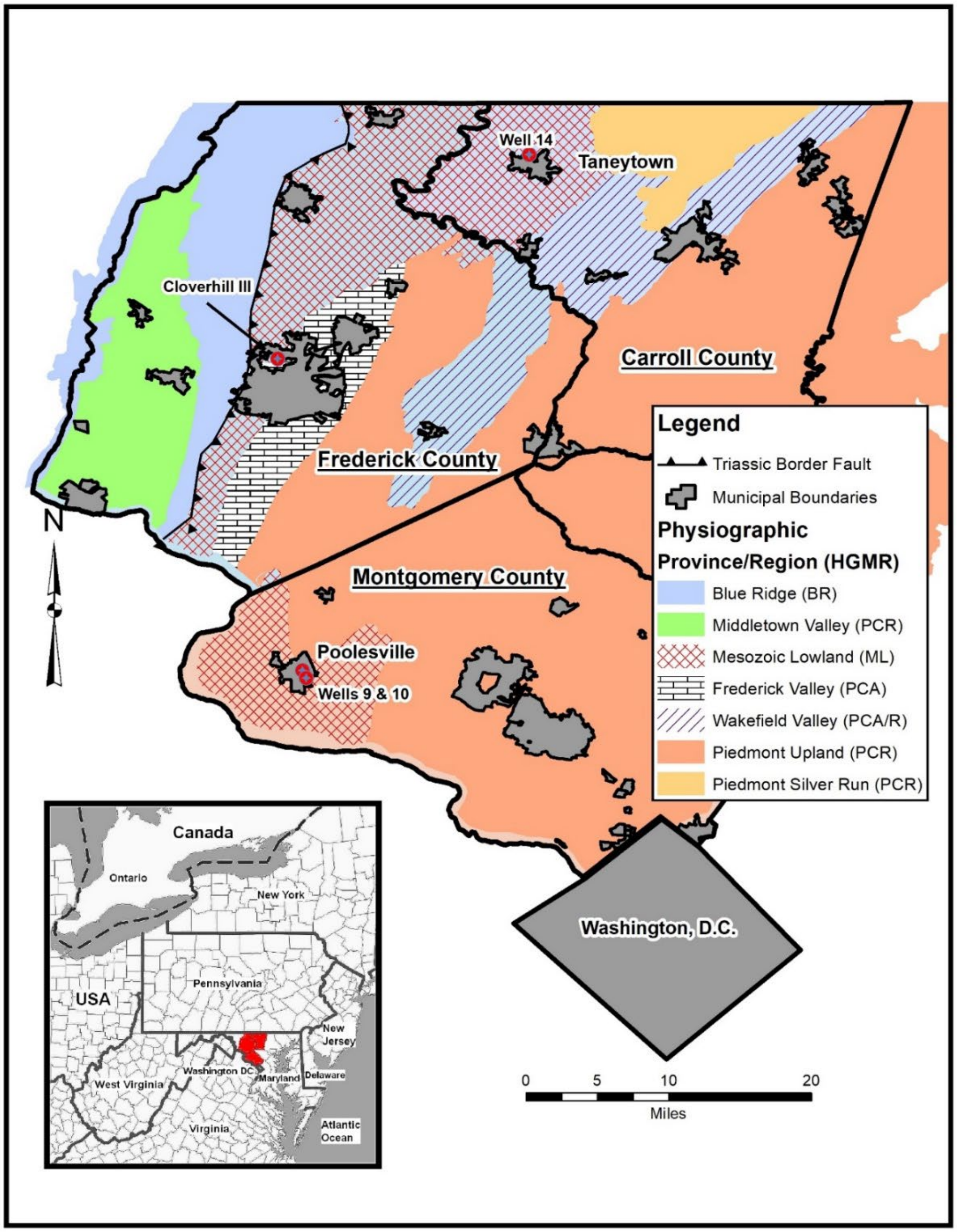


Figure 2. Physiographic and hydrogeomorphic (HGMR) regions of the study area. The HGMRs are the Blue Ridge (BR), Mesozoic Lowland (ML), Piedmont Carbonate (PCA) and Piedmont Crystalline (PCR) regions.

Consolidated Sedimentary Rock Aquifer Case Studies

Cloverhill III Case Study

The Coverhill III Subdivision was supplied water during the period 1988 to 2016 from two wells in the New Oxford Formation under Water Appropriation and Use permit FR1986G026 in the amounts of 74,300 gpd avg / 124,100 gpd max to serve 328 single family homes. Separate aquifer tests (50 to 96 hours in length) were conducted for three different wells at the site with a variable number of four on-site and eight off-site observation wells monitored during the tests (Maximum of four on-site/four off-site in any test), Figure 3. This was first known project where the State required extensive monitoring of offsite wells during aquifer testing, although previous offsite monitoring did occur on a voluntary basis or when impacts were noted during drilling and completion operations. It is also the first known application of fracture flow models and the development of a numerical model for the evaluation of potential interference impacts.

During the 96-h, variable rate step/aquifer test of well TW-2 (FR-81-3692), the drawdowns were 70 feet in the pumping well, 61 feet in TW-1 (FR-81-3693) located 209 feet SSW of TW-2 and 19 feet in the Church Well located 1800 feet SW of TW-2, Table 1. No response was noted in the remaining four observation wells. During the 49.5-h, variable rate step-aquifer test of well TW-3 (FR-81-4199), the drawdowns were 61 feet in the pumping well and 42 feet in the Farm Well, located about 100 feet NW of TW-3. A spring adjacent to TW-3 went dry during that test. No response was noted in the remaining five observation wells. During the 72-h, variable-rate aquifer test of the production well (FR-81-5372), the drawdown was 240 feet in the production well, with no response in the eight observation wells.

The ground water flow at the site could be controlled by increased permeability along tension fractures caused by folding or along bedding plane parting. In the case of the fractures caused by folding, the orientation of the prominent lineament features, Figure 4, or fractures would be nearly vertical along a NE-SW axis. Based on the location of the outcrop for the basal conglomerate of the New Oxford Formation, USGS geologic map of Frederick County (Jonas and Stose, 1938), and the lithologic log for well FR-81-4199, the bedding plane of the New Oxford Formation dips at about 20 degrees to the NW.

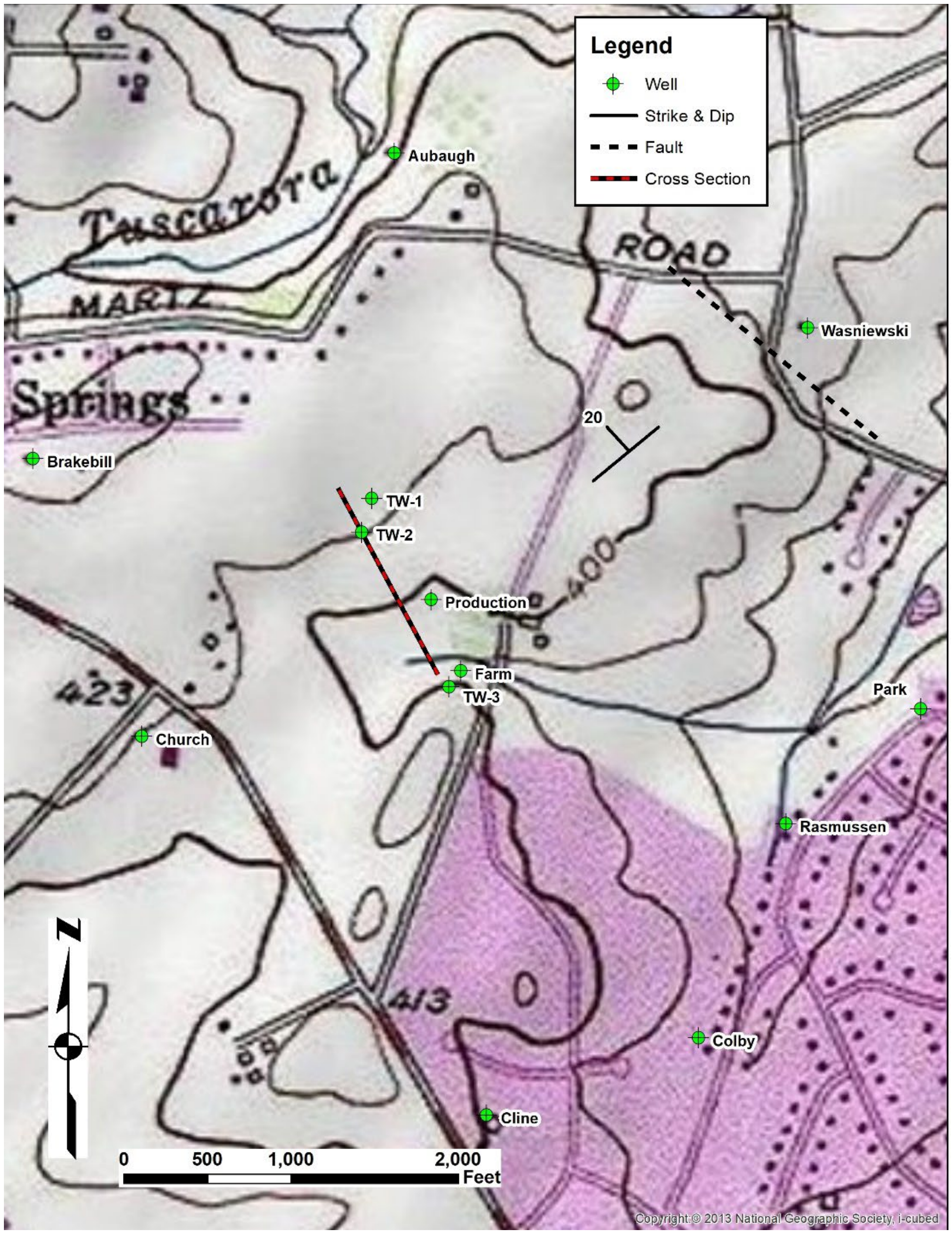


Figure 3. Location Map. Cloverhill III test site

Table 1. The drawdowns observed in the pumping and monitoring wells during the aquifer tests of Cloverhill III wells TW-2, TW-3, and the production well.

TW-2				TW-3				Production			
Well	Distance	SWL	Drawdown	Well	Distance	SWL	Drawdown	Well	Distance	SWL	Drawdown
Name	feet	feet	feet	Name	feet	feet	feet	Name	feet	feet	feet
TW-2	0	40.5	70	TW-3	0	6.6	61	Production	0	N/R	240
TW-1	209	33.8	61	TW-1	1220		0	TW-1	690	N/R	0
Church	1800	31	19	Church	1860		0	Church	1915	N/R	0
Brakebill	2000	51	0	Sring	adjacent		dry	TW-2	580	N/R	N/R
Aubaugh	2275	N/R	0	Farm	100	11.0	42	TW-3	530	N/R	0
Park	3500	34	0	Park	2825		0	Wasniewski	2780	53.9	0
Cline	3565	N/R	0	Cline	2570		0	Rasmussen	2500	19	0
				TW-2	1060		0	Colby	3070	25.9	0

The fracture trace map, Figure 4, indicates that there is no fracture that directly connects the Church Well to TW-2. There is a prominent fracture near the Church Well that extends about 3000 feet to the NE and intersects several fractures that pass near TW-2. These interconnecting fractures might be a conduit for ground water flow that explains the 19-foot drawdown noted in the Church Well when pumping TW-2; however, the current production well (FR-81-5372) is located near the same prominent fracture as the Church Well and no response was noted in the Church Well during the test of FR-81-5372, although 240 feet of drawdown was noted in the pumping well during that test. The fracture trace map indicates that TW-1 and TW-2 may be connected by a fracture and there appears to be no fracture connecting the Farm Well to TW-3. The fracture map also indicates that the production well (FR-81-5372) is located near the intersection of two fractures that could connect that well to TW-1, TW-2, and TW-3, in addition to the Church Well; however, no response was noted in any of these wells during the pumping test of well FR-81-5372, although, again, 240 feet of drawdown was observed in the pumping well. Based on the response of observation wells noted during the tests of TW-2 and TW-3 and the lack of response noted during the test of the current production well, it appears that the fractures indicated on the fracture trace map are not the primary control of groundwater flow direction at the Cloverhill III site.

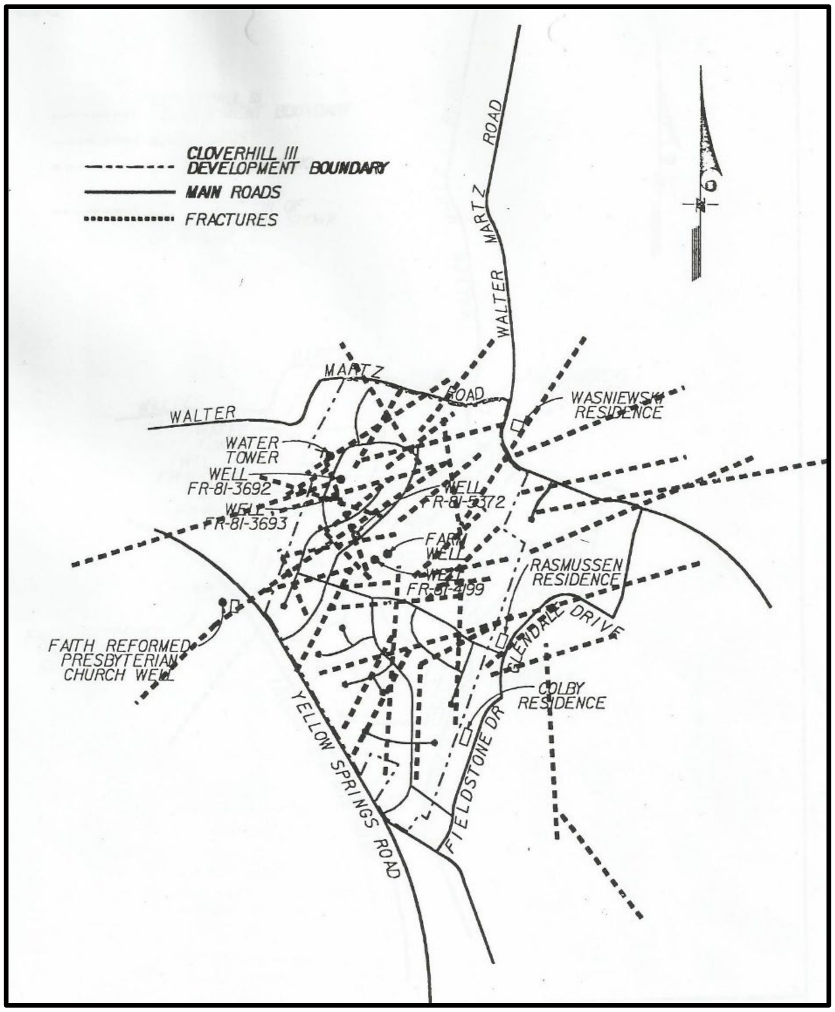


Figure 4. Fracture trace map Cloverhill III test site.

The cross-section diagram, Figure 5, indicates that bedding plane parting control of ground water flow could explain the water level response of all the observation wells, except that of TW-3 (FR-81-4199) during the test of the production well, FR-81-5372. The response noted in the pumping well during that test indicates that the major water-bearing zone in that well is located between about 50 and 190 feet, which corresponds to the major water-bearing zone (50-150 feet) noted in the lithologic log for TW-3 (FR-81-4199). The cross section constructed by Greenhorne & O'Mara (1992) applied a dip of 15° to the northwest; however, the Frederick County Geologic Map, Jonas, and Stose (1938), indicates that the beds dip at 20°. At that angle, it is possible that the effectively confined water-bearing zone in the production well would not intersect any of the observation wells. If bedding plane parting controls ground water flow at the site, a second well that might have shown a response during the pumping test is the Wasniewski well located approximately in a strike direction from FR-81-5372. In this case the Wasniewski well is so shallow (about 50 feet), and at such a great distance (2780 feet) that a bedding plane in the production well might not intersect that well. In addition, the Frederick County Geologic Map indicates that a NW-

SE trending fault may hydraulically separate the Wasniewski well from well FR-81-5372, near Walter Martz Road. A bedding plane producing zone dipping at 20° could provide an explanation as to why no drawdowns in observation wells occurred during the testing of FR-81-5372.

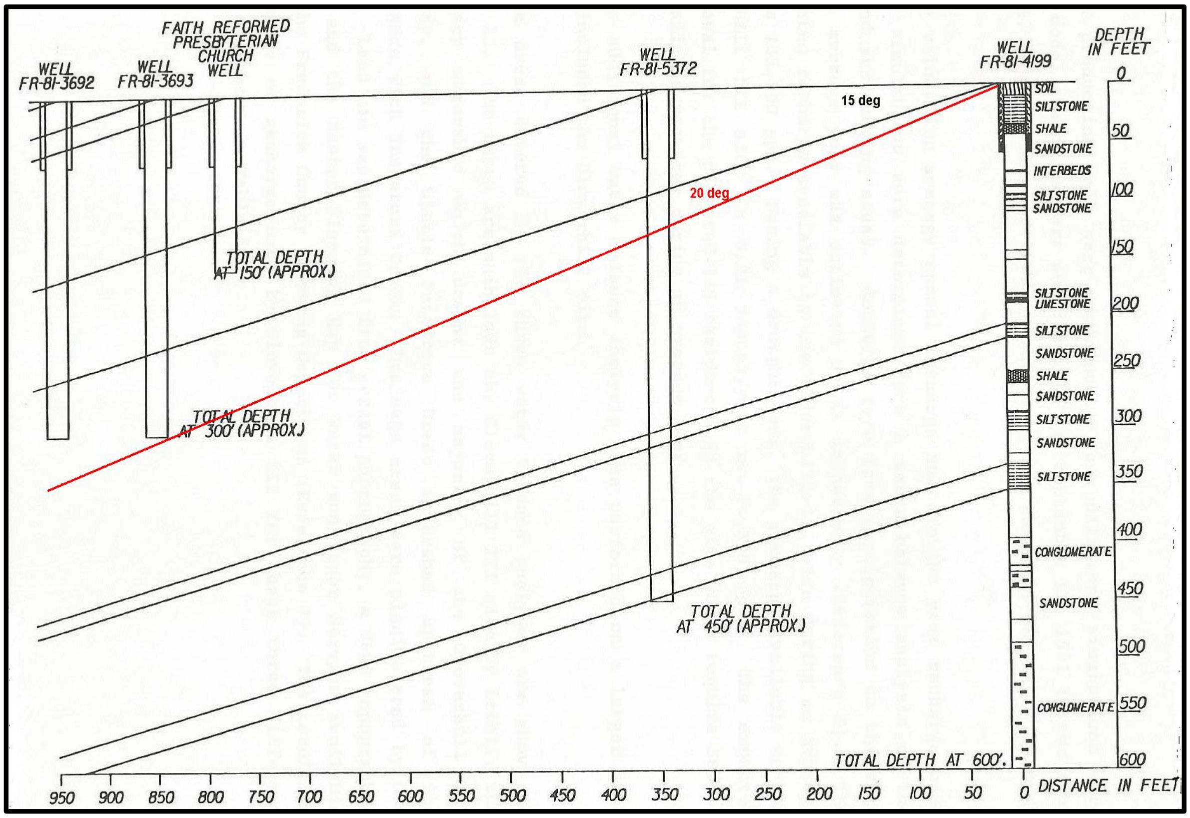


Figure 5. Cross-section diagram of the Cloverhill III test site.

Bedding plane parting control of ground water flow provides the best, but not conclusive, explanation of the observed data. All the available evidence (i.e., that of the orientation of bedding plane and prominent fracture traces and observation well responses to pumping) indicate that there is a preferred SW-NE direction for ground water flow at the site under the stress of pumping from the wells.

Certain other geologic features may have provided additional protection to other users of the resource. In addition to being located perpendicular to the estimated SW-NE ground water flow trend, the domestic wells supplying Cloverhill II were located down gradient of the three potential Cloverhill III production wells. The NW-SE trending fault may protect homes northeast of Walter Martz Road. All but about 4 or 5 homes within the Indian Springs Subdivision, NW of the Cloverhill III wells, are located outside of the ground water drainage basin. The nearest known Indian Springs home is located about 800 feet from any Cloverhill III production well. The Indian Springs wells suppling those homes are located perpendicular to the SW-NE ground water flow trend, were too shallow to be affected by bedding plane flow and there are no apparent fractures that could connect Indian Springs wells to Cloverhill III wells.

Aquifer Test Analyses

Table 2 contains the results of analyses using the AQTESOLV automatic curve fitting program for the step and aquifer tests of TW-2 (FR-81-3693) and TW-3 (FR-81-4199), and the aquifer test of the production well (FR-81-5372). The solutions provided the best fit to the drawdown data in each case are highlighted in yellow.

The step/aquifer test of TW-2 was conducted during 11/3-11/6/1986, starting at 16.5 gpm, then continuing through six, 60-min steps at increasing rates ending at 88.5 gpm. The test then continued at a constant rate of 60 gpm until the end of the 98-hr test. The drawdown was initially transcribed from the consultant's graphs using the GraphGrabber digitizer to compare with the actual recorded data to determine the effectiveness of the digitizing program. The best fit to the step-test or early drawdown data was by the Dougherty-Babu (1984) dual porosity solution, producing a T value of 539 gpd/ft and a Well Efficiency (W.E.) of 32%, Figure 6. No correction for dewatering was needed, indicating that the aquifer existed under confined conditions. The Hantush-Jacob (1955) solution for a leaky aquifer, with aquitard storage, provided a good fit to the late-time constant rate test, producing a T of 316 gpd/ft. This indicated that there was a transition from a confined aquifer to a leaky aquifer at late time, with a decline in the aquifer permeability. The Hantush-Jacob solution provided the best fit to the early time data from the observation wells, producing T values of 574 gpd/ft and 528 gpd/ft and S values of 6.3E-5 and 3.4E-5 for well TW-1 and the Church well, respectively, Figures 7 and 8. The Hantush-Jacob solution provided a good fit to the drawdown data from TW-1 during the late-time constant rate test, producing a T of 349 gpd/ft. There was a lag in the response to pumping in the Church Well of 300 minutes and the drawdown was limited to 10 feet; consequently, there was no equivalent late time data. These results are consistent with the observation by Hammond (2018) that dual porosity conditions are scale dependent and only occur in the immediate vicinity of the pumping well, while single porosity conditions are dominant as a trough of depression expands outward into a fractured rock aquifer.

Table 2. Summary of the aquifer test results at the Cloverhill III Subdivision.

Test-Obs Well	T	S	r	s	t	Model	Source	Step-Test				RSS	Var	S.D.	Mean
	gpd/ft		ft	ft	min			B	C	P	W.E.	ft^2	ft^2	ft	ft
TW-2 Well (TW-2 Test) (11/3-11/6/86)	539	-	0	56	0-360	D-B	Raw Data	0.35	0.07	2.1	32%	9.8	0.20	0.45	-0.043
	444	-	0	70	0-5760	D-B	Raw Data	0.31	0.116	2	24%	602	7.08	2.66	0.212
	1276	-	0	56	0-360	H-J	Raw Data	0.24	0.015	2.8	28%	28.9	0.58	0.76	-0.185
	482	-	0	13	0-120	D-B	digitizer	0.35	0.022	3	10%	13	0.71	0.84	-0.094
	770	-	0	13	0-120	H-J	digitizer	0.002	0.027	3	0.05%	13	0.08	0.087	0.066
	154		0	70	1000-5880	H-J A/S	Raw Data	-	-	-	-	5.35	0.23	0.482	0.001
	316	-	0	70	1000-5880	H-J A/S	digitizer	-	-	-	-	2.0	0.11	0.325	-0.0007
TW-1 (TW-2 Test)	681	4.5E-05	209	39	0-360	D-B	Raw Data	-	-	-	-	14.6	0.30	0.55	0.091
	574	6.3E-05	209	39	0-360	H-J	Raw Data	-	-	-	-	8.6	0.17	0.41	0.063
	349	1.0E-04	209	61	385-5760	H-J	Raw Data	-	-	-	-	2.0	0.06	0.24	2.5E-05
	416	7.9E-05	209	61	60-5760	H-J	Raw Data	-	-	-	-	21	0.27	0.52	0.104
Church Well (TW-2 Test)	528	3.4E-05	1600	19	300-5000	H-J	Raw Data	-	-	-	-	1.9	0.10	0.32	0.034
	760	3.6E-05	1600	19	300-5000	Theis	Raw Data	-	-	-	-	3.8	0.19	0.44	-0.004
TW-3 Well (TW-3 Test) (2/11-2/13/87)	1206	-	0	58	0-300	D-B	Raw Data	0.4	0.0003	3	86%	4.9	0.12	0.340	0.0195
	758	-	0	58	0-300	D-B	digitizer	0.327	0	2.9	100%	20.7	0.56	0.750	0.1004
	2539	-	0	58	500-2970	H-J	Raw Data	-	-	-	-	122	2.98	1.730	0.001
	2563	-	0	58	500-2970	D-B	Raw Data	-	-	-	-	122	3.21	1.790	0.0005
	2066	-	0	61	0-2970	D-B	Raw Data	0.567	0.0038	2.4	77%	336	3.62	1.90	-0.089
Farm Well (TW-3 Test)	968	2E-03	100	40	930-2190	H-J	Raw Data					0.33	0.08	0.29	2E-06
	1767	5E-04	100	40	930-2190	Theis	Raw Data	-	-	-	-	0.47	0.09	0.31	-2E-05
Production Well (Production Well Test) (12/17-12/20/91)	1254	-	0	37	70-800	Theis	digitizer	-	-	-	-	14.5	0.54	0.73	0.0015
	1420	-	0	37	70-800	SVF-F	digitizer	Bilinear Flow				13.2	0.53	0.73	0.0049
	825	-	0	37	70-800	Barker	digitizer	-	-	-	-	10.1	0.39	0.62	0.0076

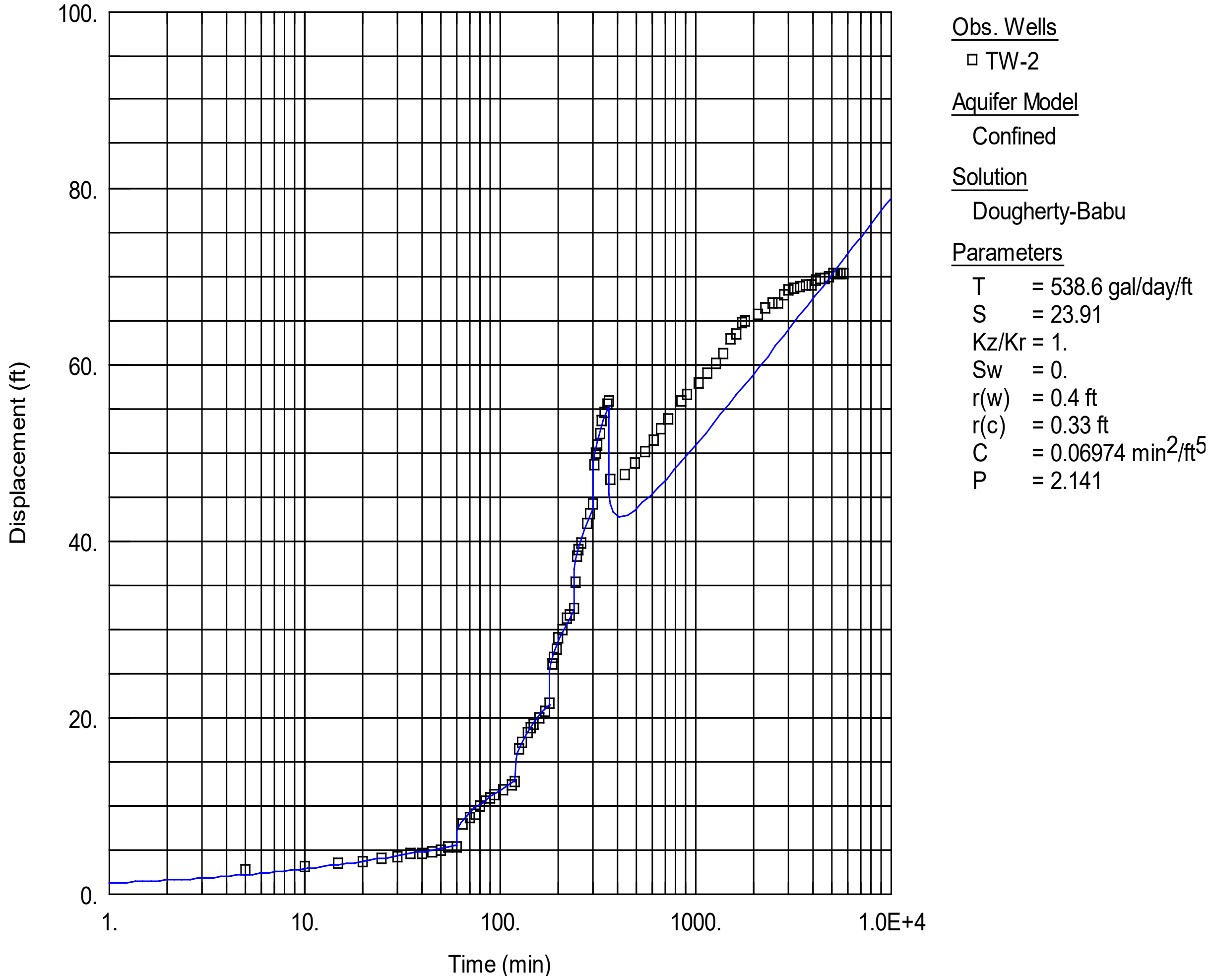


Figure 6. Cloverhill III TW-2 well – Semi-log plot of drawdowns from variable rate (16.5 to 88.5 gpm) step test, Dougherty-Babu dual porosity solution, followed by a 60-gpm constant rate aquifer test until the end of the 98-h test.

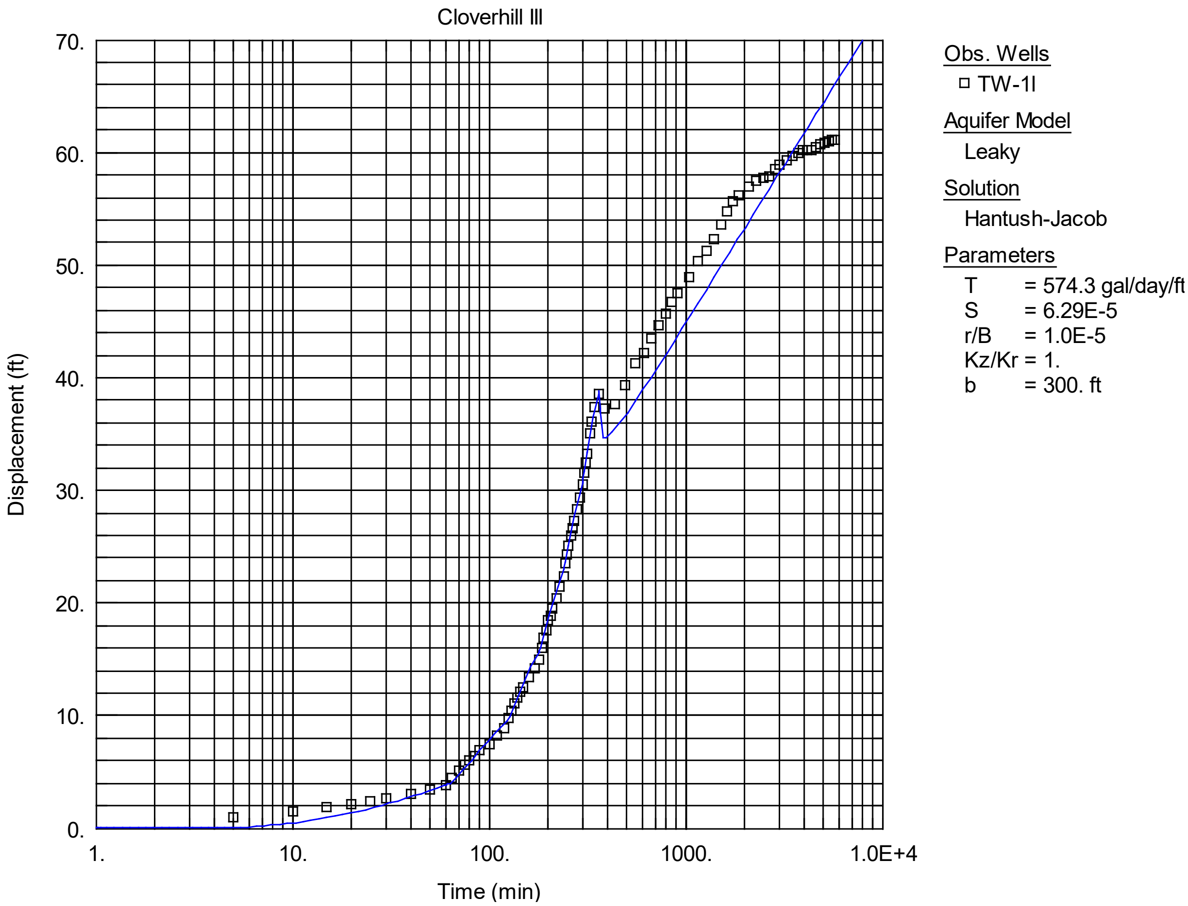


Figure 7. Cloverhill III TW-2 test, observation well TW-11 – Semi-log plot of drawdowns from variable rate (16.5 to 88.5 gpm) step test, followed by a 60-gpm constant rate aquifer test, Hantush-Jacob solution for early-time data.

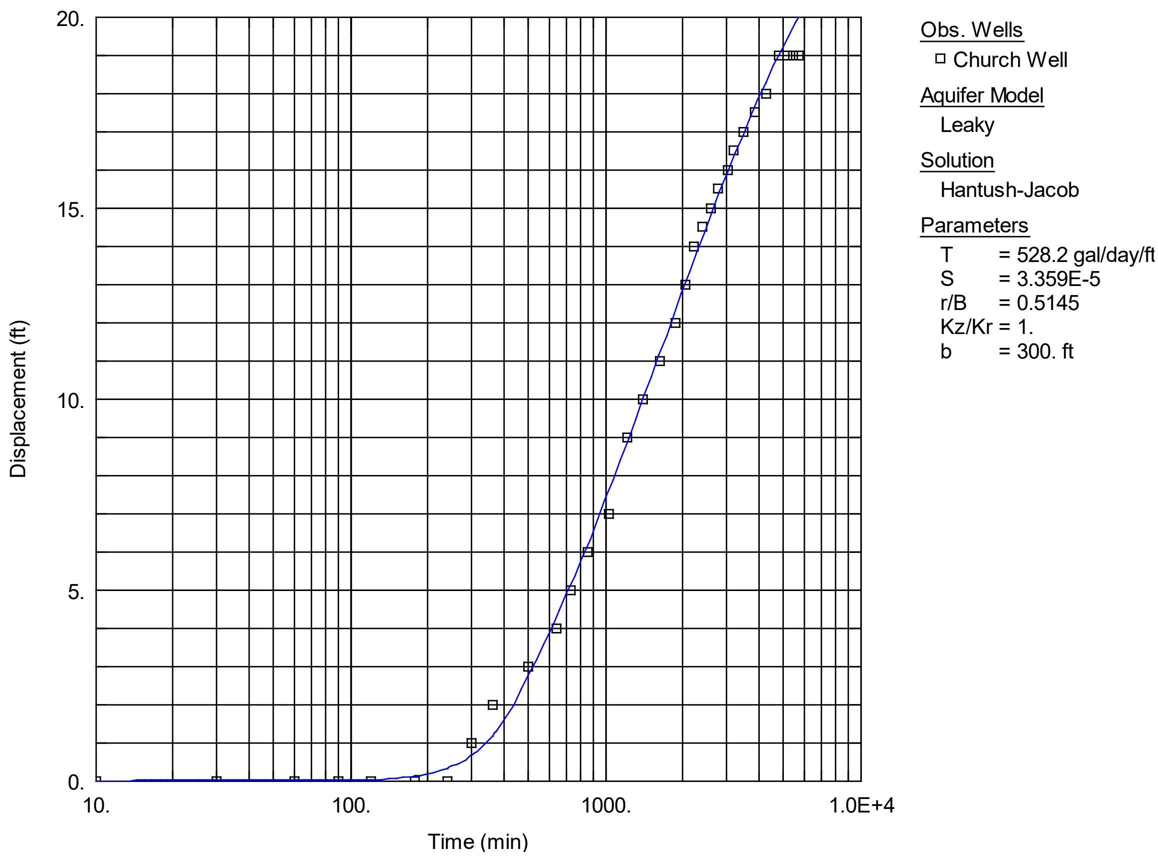


Figure 8. Cloverhill III TW-2 test, church observation well – Semi-log plot of drawdowns from variable rate (16.5 to 88.5 gpm) step test, followed by a 60-gpm constant rate aquifer test, Hantush-Jacob solution for early-time data.

The step/aquifer test of TW-3 was conducted on 2/11-2/13/1987, starting at 34 gpm, then continuing through six, 60-min steps at increasing rates ending at 200 gpm. The test then continued at a constant rate of 120 gpm until the end of the 49.5-hr test. The best fit to the step-test or early drawdown data was the Dougherty-Babu dual porosity solution, producing a T value of 1206 gpd/ft and a W.E. of 86%, Figure 9. No correction for dewatering was needed, indicating that the aquifer existed under confined conditions. The Hantush-Jacob and Dougherty-Babu solutions both provided good fits to the late-time constant rate drawdown data, producing T values of greater than 2500 gpd/ft. These may not be true T values since there was a substantial recovery of nearly 30 feet prior to the start of the constant rate test. The recovering water levels may have flattened the drawdown curve, producing a higher than actual T value. The Hantush-Jacob and Theis (1935) solutions both provided good fits to the drawdown time data from the Farm (observation) Well, producing T values of 968 gpd/ft and 1767 gpd/ft and S values of 0.0015 and 0.0005, respectively. Only a few mid-time points were available since the early and late time data were erratic.

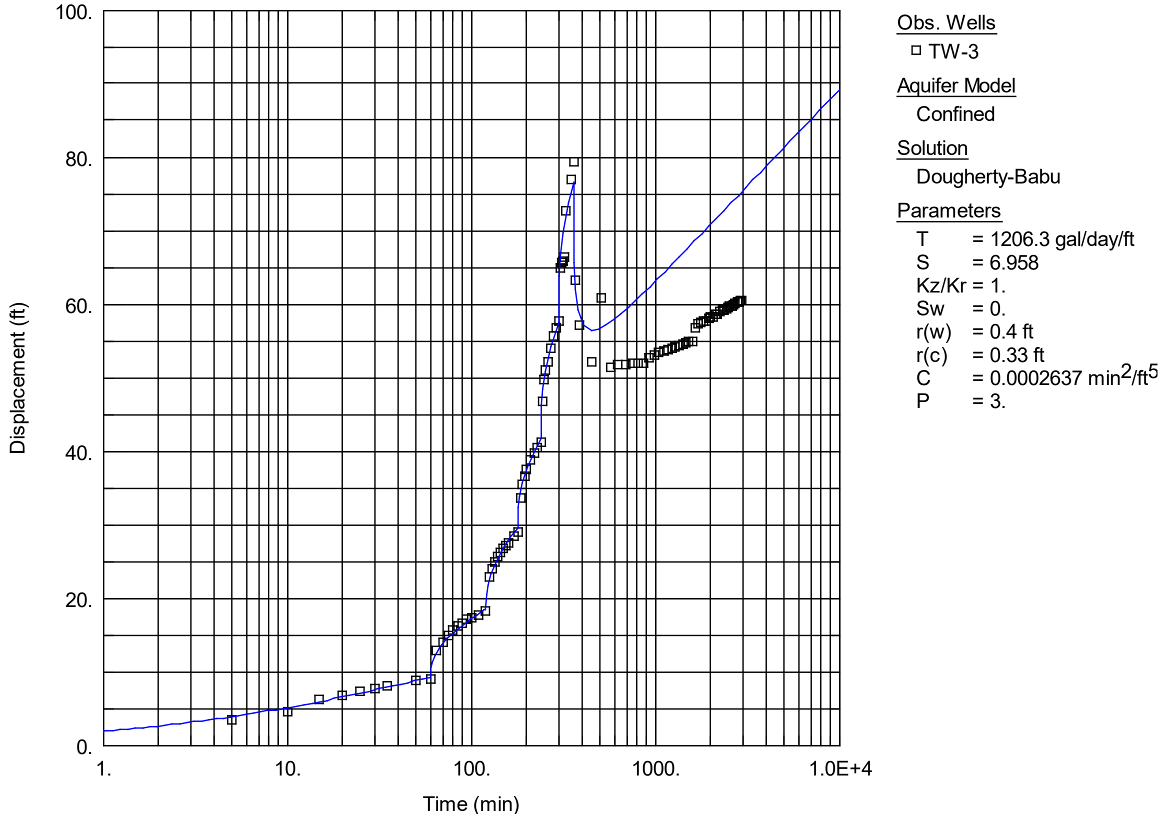


Figure 9. Cloverhill III TW-3 well – Semi-log plot of drawdowns from variable rate (34 to 200 gpm) step test, Dougherty-Babu dual porosity solution, followed by a 120-gpm constant rate aquifer test until the end of the 49.5-h test.

An aquifer test of the production well was conducted during 12/17-12/20/1991. The water level data had to be transferred by a digitizer from the consultant's graph. The test started at 87 gpm, Figure 10. After 7 minutes, there was a recovery in the water level that the derivative indicates was a recharge event; however, the water started drawing down again after 35 minutes, indicating that the fluctuation could also have been due to an unrecorded adjustment in the pumping rate. Nonetheless the data for that period is unsuitable for analysis. At 930 min and 37 ft of drawdown on the consultant's graph, Figure 11, there was a break in the drawdown curve, followed by a rapid decline in the water level after 1360 min to a drawdown of 80 ft. The declines noted between 930 and 1360 min may have been due to dewatering of a reservoir unit, most likely a consolidated sandstone. After that point, the water level fluctuated and never stabilized, and the pumping rate continuously declined to 45 gpm as the flow regulating valve was in a fully open position; consequently, those data were also not suitable for analysis. There was no observed drawdown in any of the five onsite observation wells or the three offsite residential wells.

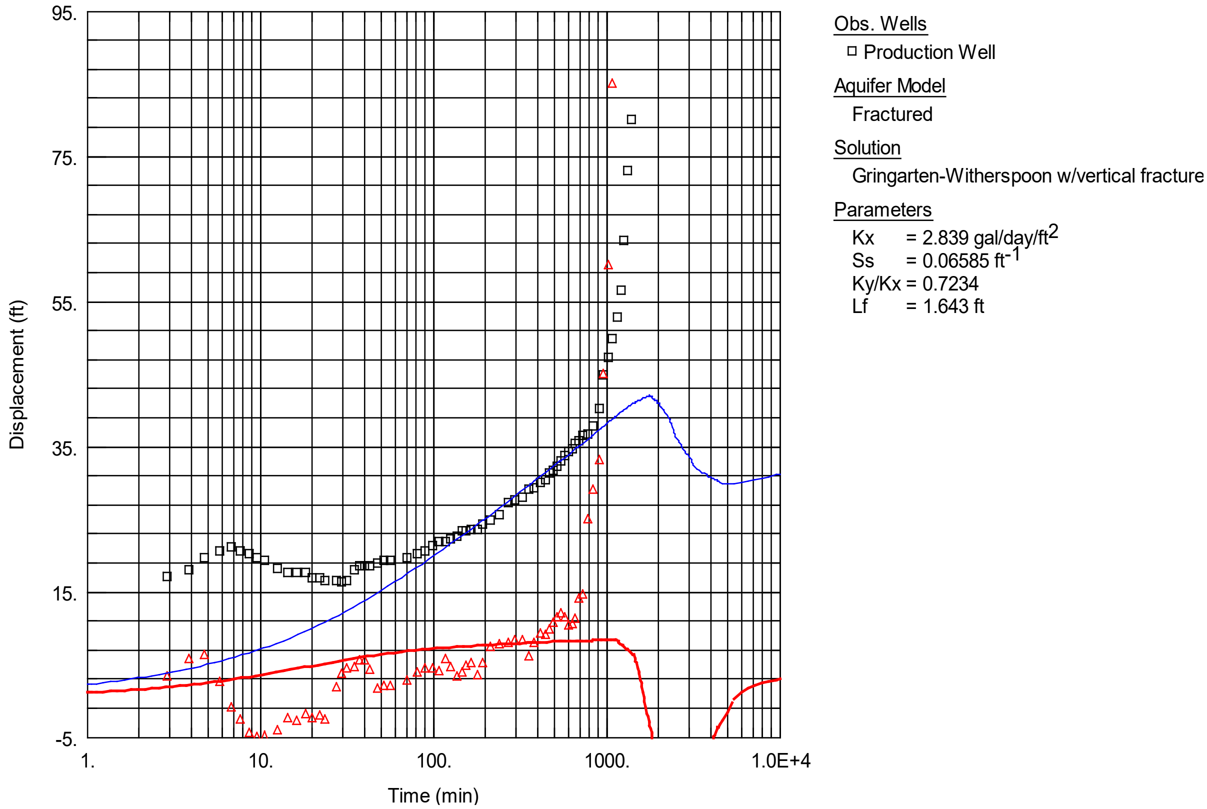


Figure 10. Cloverhill III production well – Semi-log plot of drawdowns from a variable rate test, starting and continuing at 87 gpm for the first 20 hours, then at a steadily declining rate until reaching 45 gpm at the end of the 72-h test, Gringarten and Witherspoon solution for a single vertical fracture, for an intermediate period (70-800 min).

During the intermediate period of 70 to 800 min, a diagnostic plot, Figure 10, indicated that bilinear flow had occurred, which is the typical signature of a single vertical fracture with finite conductivity. The Gringarten and Witherspoon solution produced a T value of 1420 gpd/ft. That value is like that from the testing of TW-3 and several times greater than that of the TW-2 test.

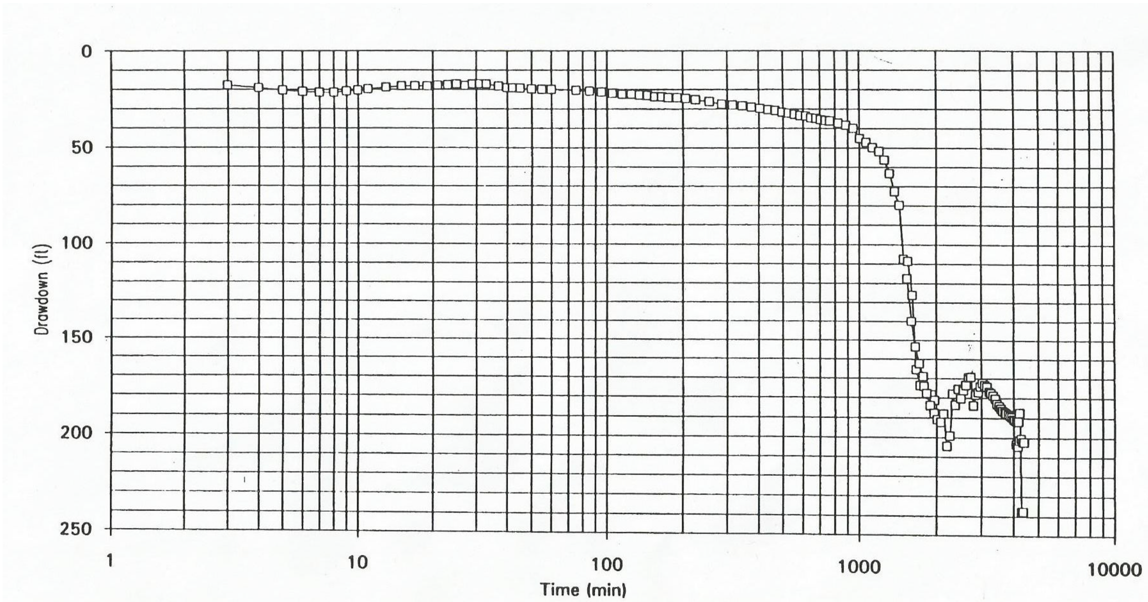


Figure 11. Cloverhill III production well. Greenhorne & O'Mara semi-log plot of the drawdown data from a 72-h, variable rate (87 to 45 gpm) aquifer test.

Well Interference Impacts of Groundwater Withdrawals

To determine the interference or impacts of withdrawals from the Cloverhill III water supply wells, a conceptual model needs to be developed, elements of which are depicted in the following equations and Figure 12. Contours developed from the transmissivities derived from the step/aquifer tests indicated that the aquifer was anisotropic. An anisotropic aquifer is usually described as having a high transmissivity along a major axis and a low transmissivity along a minor axis, while the shape of drawdown contours is determined by the ratio of the two transmissivities. The drawdown (s_x) at any point is determined by the following equation:

$s_x = 264\ Q / (T_{\xi\xi} * T_{\eta\eta})^{1/2} * \log(0.3\ t / S)\ (T_{\xi\xi} * T_{\eta\eta}) / (T_{\xi\xi}\eta^2 + T_{\eta\eta}\xi^2)$

where:

s_x = drawdown (ft) at given distance

Q = pumping rate (gpm)

$T_{\xi\xi}$ = Transmissivity in the major direction of anisotropy (gpd/ft)

$T_{\eta\eta}$ = Transmissivity in the minor direction of anisotropy (gpd/ft)

$(T_{\xi\xi} * T_{\eta\eta})^{1/2} = T_E$ (effective Transmissivity)

ξ = distance from pumping well (ft) along major axis

η = distance from pumping well (ft) along minor axis

t = time (d)

S = Composite Storage Coefficient (dimensionless)

Aquifer Constants (T&S) from: On-site aquifer tests

In this case, contours were determined by interpolation between the transmissivities of TW-1, TW-2, and the Church well (500 gpd/ft), and the production well, TW-3 and the Farm well (1200-1500 gpd/ft). The result indicates that the aquifer is also anisotropic along each of the axes. In the anisotropic model, the average transmissivity, which is equal to the square root of the sum of the two transmissivities, is used in calculations of drawdown in a pumping well. To determine the average transmissivity in the potential trough of depression, the areas within the three contours on the map were multiplied by each transmissivity, the sum of the results were divided by the total area to produce an average transmissivity of 580 gpd/ft. The ellipse on the map indicates that the anisotropic ratio is about 2 to 1. With such a ratio, a T of 820 gpd/ft along the major axis and a T of 410 gpd/ft would produce an average T of 580 gpd/ft.

Scenario 1: Anisotropic Model from test of TW-2

t = 4 days Q = 59.5 gpm S = 0.00006
TW-2 EOT: s = 70ft (TW-2) 61 ft (TW-1) 19 ft (Church)

$s_x = (264*59.5 / (580)) * \log (((0.3*4 / 0.00006) ((336{,}200) / (820\eta^2 + 410\xi^2)))$

$s_x = (27.1) * \log ((20{,}000) (336{,}200) / (820\eta^2 + 410\xi^2))$

$s_x = (27.1) * \log 16{,}400{,}000/\xi^2 \quad \eta = 0$
$s_{.25} = 228$ ft $s_{100} = 87$ ft $s_{209} = 70$ ft $s_{300} = 61$ ft $s_{500} = 49$ ft $s_{1000} = 33$ ft $s_{1800} = 19$ ft $s_{3000} = 7$ ft

$s_x = (27.1) * \log 8{,}200{,}000/\eta^2 \quad \xi = 0$
$s_{.25} = 220$ ft $s_{100} = 79$ ft $s_{209} = 62$ ft $s_{300} = 53$ ft $s_{500} = 41$ ft $s_{1000} = 25$ ft $s_{1800} = 11$ ft $s_{3000} = 0$ ft

This simulation matches the drawdown in the Church well (19 ft at r = 1800 ft along the major axis), is close to the drawdown in TW-1 (61 ft at r = 209 ft along the minor axis), but greatly over-estimated the drawdown (70 ft) in the pumping well when well bore radius is used in the calculations.

Next, the potential effects of an extended well-fracture during the test of TW-2 on drawdowns in TW-2 (70 ft), TW-1 (61 ft) and the Church well (19 ft) need to be considered. This is accomplished by using the ¼ fracture length model developed in the companion study on the impacts of withdrawals from crystalline rock aquifers. The method consists of adjusting the ¼ fracture length and distance to the fracture until a simulation provides reasonable fits to the drawdown data in the pumping and observation wells. Below are presented various scenarios to demonstrate the interference effects of withdrawals from the Cloverhill III water supply wells.

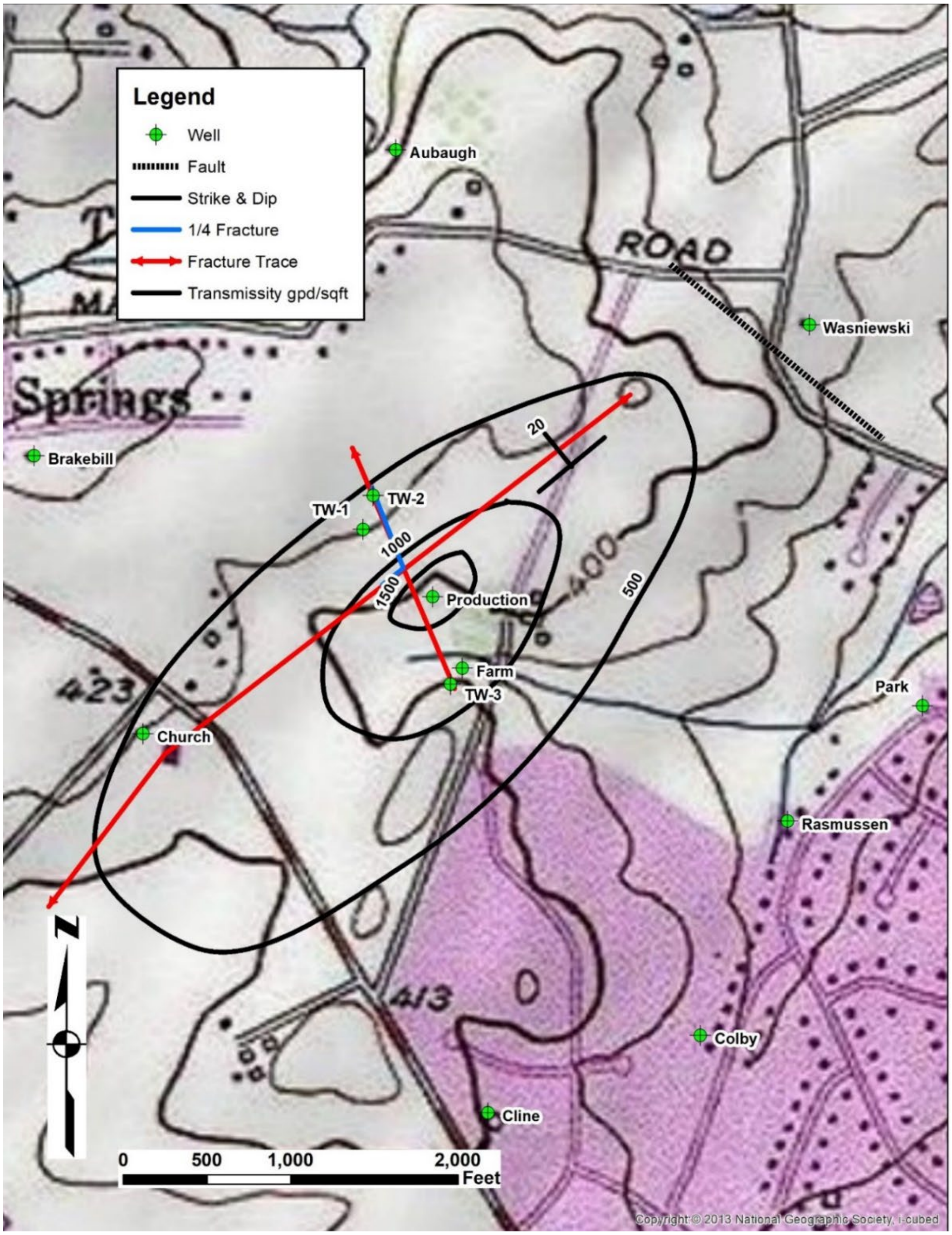


Figure 12. Topograhic map depicting anisotropic aquifer, determined from aquifer testing at the Cloverhill site, a vertical fracture, two primary fracture traces, and regional strike (NE-SW) and dip (20° NW).

Scenario 1A: Anisotropic aquifer plus ¼ L Model:

t = 4 days Q = 59.5 gpm S = 0.00008
TW-2 EOT: s = 70ft (TW-2) 61 ft (TW-1) 19 ft (Church)

$s_x = (264*59.5 / (580)) * \log (((0.3*4 / 0.00008) ((336{,}200) / (820\eta^2 + 410\xi^2)))$

$s_x = (27.1) * \log ((15{,}000) (336{,}200) / (820\eta^2 + 410\xi^2))$

x = ¼ L + d:
TW2: ¼ L, x = 180; TW-1, d = 140, x = 320; Church, d = 1685, x = 1865

$s_x = (27.1) * \log 12{,}300{,}000/\xi^2 \quad \eta = 0$
$s_{180} = 70$ ft $s_{320} = 57$ ft $s_{1865} = 15$ ft $s_{3000} = 4$ ft

$s_x = (27.1) * \log 6{,}150{,}000/\eta^2 \quad \xi = 0$
$s_{180} = 62$ ft $s_{320} = 48$ ft $s_{1865} = 7$ ft $s_{3000} = 0$ ft

TW-2 and the Church well are along the major axis and TW-1 is about 30° off the major axis. The estimated errors are 11% in TW-1, 26% in the Church well, and 0% in TW-2.

Scenario 2: 90-d Production Well Model

In this case, the T values were recalculated to reflect the higher transmissivity in the vicinity of the production well.

t = 90 days Q = 78 gpm (maximum reported use - 115,000 gpd in June 1999)
$s_x = ((264*78 / (1060)) * \log ((0.3*90 / 0.00008) (1{,}250{,}000) / (1500\eta^2 + 750\xi^2))$

$s_x = (19.4) * \log ((337{,}500) (1{,}250{,}000) / (1500\eta^2 + 750\xi^2))$

$s_x = (19.4) * \log 562{,}500{,}000/\xi^2 \quad \eta = 0$
$s_{100} = 93$ ft $s_{200} = 81$ ft $s_{300} = 74$ ft $s_{500} = 65$ ft $s_{1000} = 54$ ft $s_{2000} = 42$ ft $s_{3000} = 35$ ft $s_{5000} = 26$ ft

$s_x = (19.4) * \log 281{,}250{,}000/\eta^2 \quad \xi = 0$
$s_{100} = 87$ ft $s_{200} = 75$ ft $s_{300} = 68$ ft $s_{500} = 59$ ft $s_{1000} = 48$ ft $s_{2000} = 36$ ft $s_{3000} = 29$ ft $s_{5000} = 20$ ft

This analytical model indicates that significant drawdowns could occur at distances of at least one mile along the strike major axis. Drawdowns along the minor axis should be less than calculated due to the substantial bedding plane dip (20° NW) in the area.

The next scenario (3) simulates the drawdowns that would occur at the maximum monthly allocation of 124,000 gpd over a 30-d period. Those results are compared to a Greenhorne and O'Mara (1992) numerical model constructed to meet the criteria required by the Water Supply Program for that permit application. The results of that simulation are depicted in Figure 13. Again, the T values were adjusted to reflect the higher T calculated from the production well aquifer test.

Scenario 3: 30 d Production Well Model

Water Supply Program analytical simulation.
t = 30 days Q = 86 gpm (124,000 gpd – maximum monthly allocation)
$s_x = ((264*86 / (1060)) * \log ((0.3*30 / 0.00008) (1,250,000) / (1500\eta^2 + 750\xi^2))$

$s_x = (21.4) * \log ((112,500) (1,250,000) / (1500\eta^2 + 750\xi^2))$
Water Supply Program analytical simulation.
$s_x = (21.4) * \log 187,500,000/\xi^2 \quad \eta = 0$
s_{300} = 71 ft s_{1000} = 49 ft s_{1800} = 38 ft s_{2200} = 34 ft s_{2700} = 30
Greenhorne & O'Mara numerical simulation
s_{300} = 200 ft s_{1000} = 100 ft s_{1800} = 50 ft s_{2200} = 25 ft s_{2700} = 10 ft

Water Supply Program analytical simulation.
$s_x = (21.4) * \log 93,750,000/\eta^2 \quad \xi = 0$
s_{135} = 79 ft s_{260} = 67 ft s_{600} = 52 ft s_{1000} = 42 ft s_{2000} = 29 ft s_{3000} = 22 ft s_{5000} = 12 ft
Greenhorne & O'Mara numerical simulation
s_{135} = 200 ft s_{260} = 50 ft s_{600} = 10 ft s_{1000} = 0 ft

The substantial differences between the drawdowns in the present study and those of the consultant can be largely explained by the anisotropic ratios used in each simulation. The ratio used in the present study was 2:1, based on the T values measured during each test. The ratio used by the consultant was 50:1, the basis for which was not explained; but it may have been designed to simulate a long fracture and/or produce the lack of drawdown in the observation wells during the aquifer testing of the production well. It is also not clear how the transmissivity (1822 gpd/ft) was calculated by the consultant. It was indicated that the value was calculated from the recovery data of TW3 (FR-81-4199); however, there was no drawdown or recovery measured in the well by an automatic data recorder during that test. Finally, the consultant assumed that steady-state conditions existed, so no storage constant was required for the model; however, transient flow conditions would exist due to seasonal variations in groundwater levels and related changes in groundwater storage.

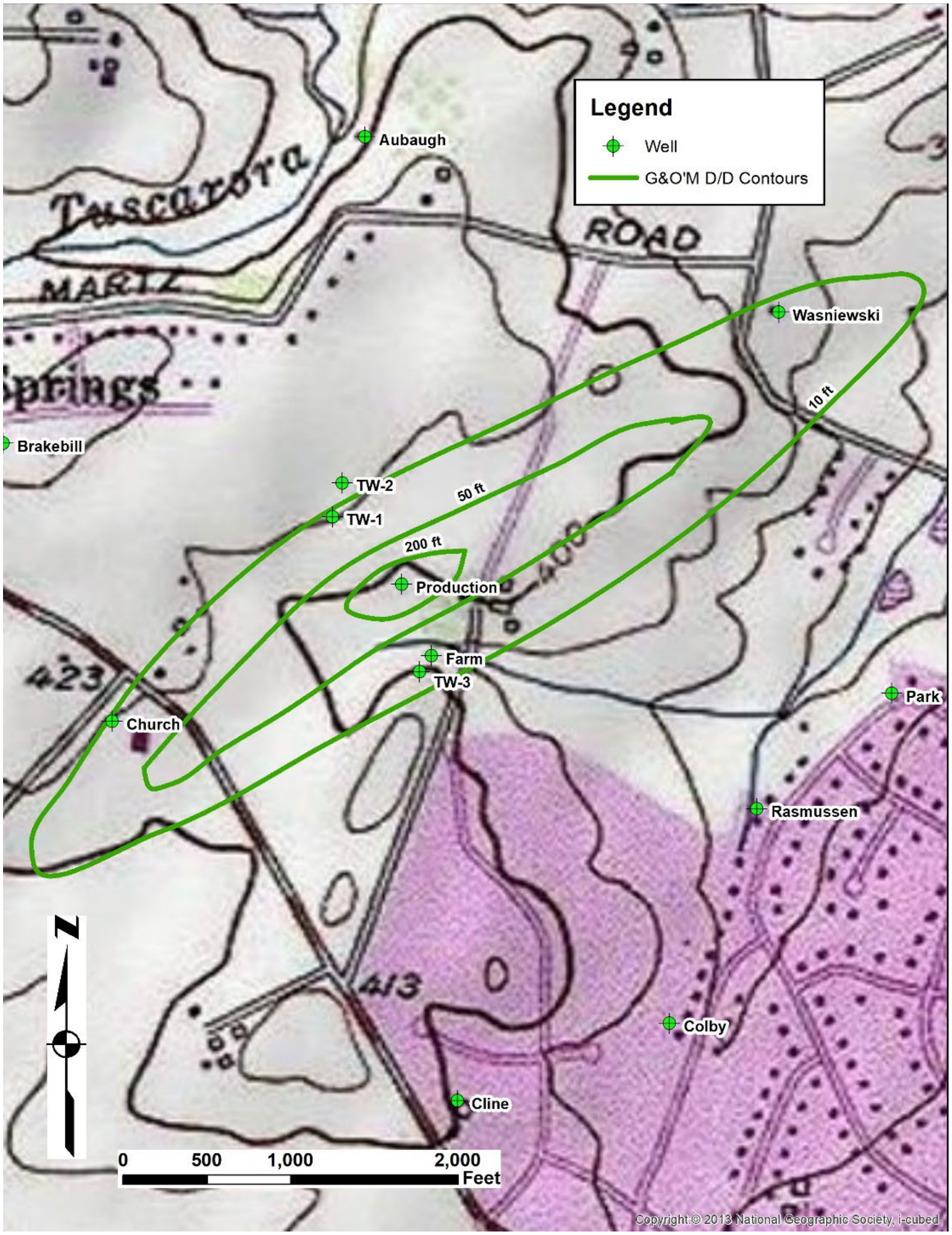


Figure 13. Greenhorne & O'Mara (1992) drawdowns from a numerical model while pumping the production well at maximum monthly use of 124,000 gpd for 30 days.

The consultant measured water levels during November 1991 in wells TW-1, -2, and -3. The consultant's analytical and numerical models were used to simulate the water levels that occurred at the end of November 1991. In the present study, the measured water levels were converted to drawdowns by adjusting the drawdowns using the ratio of actual average use to the allocated average use and a simulation (3a) was conducted while pumping the production wells at the average withdrawal rate during November 1991 (38,300 gpd).

Scenario 3a: 30d Production Well Model, Nov. 1991 W/Ls (D/Ds)

Water Supply Program Simulation
Drawdown adjusted by ratio: 0.515 or 38,300 gpd / 74,300 gpd
TW1-W/L=48 ft (s =14 ft), TW2-W/L=51 ft (s=13 ft) & TW3-W/L=31 ft (s=24 ft)
Radius: TW1 = 353 ft, TW2 = 441 ft, and TW3 = 397 ft

t = 30 days Q = 26.6 gpm (38,300 gpd or 30d use prior to Nov. 1991)
$s_x = ((264*26.6 / (1060)) * \log ((0.3*30 / 0.00008) (1,250,000) / (1500\eta^2 + 750\xi^2))$

$s_x = (6.6) * \log ((112,500) (1,250,000) / (1500\eta^2 + 750\xi^2))$

$s_x = (21.4) * \log 187,500,000/\xi^2 \quad \eta = 0$
TW1, s_{353} = 21 ft; TW3, s_{397} = 20 ft; TW2, s_{441} = 20 ft

$s_x = (6.6) * \log 93,750,000/\eta^2 \quad \xi = 0$
TW1, s_{353} = 19 ft; TW3, s_{397} = 19 ft; TW2, s_{441} = 18 ft
All observation wells located along the minor axis.

Greenhorne & O'Mara Analytical model
TW1, s_{353} = 34 ft; TW3, s_{397} = 33 ft; TW2, s_{441} = 32 ft

Greenhorne & O'Mara Numerical model
s_{353} = 34 ft s_{397} = 33 ft s_{441} = 32 ft
TW1, s_{353} = ~2 ft; TW3, s_{397} = 8 ft; TW2, s_{441} = ~1 ft

Table 3. Summary of the results of the simulations of the Greenhorne & O'Mara analytical and numerical models and the MDE simulation comparing the estimated to actual drawdowns in monitoring wells observed in November 1991 at the Cloverhill III Subdivision.

			Greenhorne & O'Mara										
Obs Well	r	Rate	Analytical Model (TW-1 aquifer contants)					Modflow (TW-3 T?)			MDE study		
Name	ft	gpd	Est W/L	Obs W/L	Est. D/D	error	Actual D/D	Est. D/D	error	Actual D/D	Est D/D	error	Actual D/D
			ft	ft	ft	%	ft	ft	%	ft	ft	%	ft
TW-1	353	38,300	68	48	34	-58.8	14	~2	600	14	19	-26.3	14
TW-2	441	38,300	72	51	32	-59.4	13	~1	1200	13	18	-27.8	13
TW-3	397	38,300	40	31	33	-27.3	24	8	200	24	18	33.3	24

Table 3 presents the calculated drawdown results of the different simulations relative to the actual drawdowns measured in November 1991. The best fit to the observed drawdowns was achieved using the MDE analytical model developed in the present study. It is noted that the simulated drawdowns in TW1 and TW2 exceeded the actual ones, while the simulated drawdown in TW3 was less that the actual drawdown. This could be because of the relatively steep dip of bedding planes in the aquifer. The potential permeable unit in downdip wells TW1 and TW2 would be substantially deeper than in the up dip well TW3. In the semi-confined aquifer, the effects of leakage would be less in the downdip wells than in the up dip well, producing less effective drawdown.

Taneytown Well 14 Case Study

A 72-hr aquifer test of Taneytown's Well 14 was conducted from July 23 to July 31, 1990. The annual average allocation under Water Appropriation Permit CL78G079 was increased by 227,000 gpd, based on an estimated yield of that well. Subsequently, MDE received a report that a resident along Fringer Road (Mr. Munderloh) had turbidity and low yield problems with his well during the test.

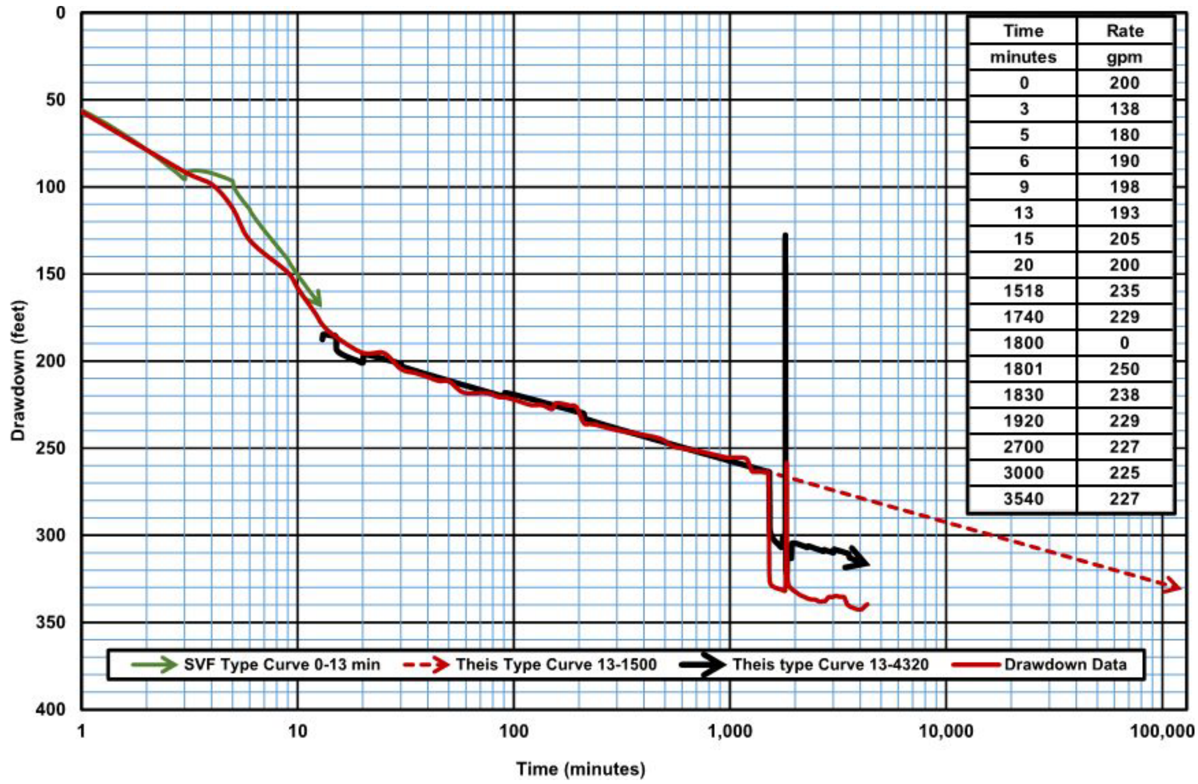
In October 1994, the Water Supply Program received reports residents along Fringer Road had been without (or intermittently without) water or had muddy water for several months, and that those same residents indicated that there were no problems with their wells before well 14 was placed in service. The town indicated that well 14 had been intermittently used at about 233 gpm and about 10 days after pumping began the town started to receive complaints from the Fringer Road residents. Records for the periods July 11 - August 10, 1994, and September – October 1994 indicate that the average use from well 14 was about 127,000 gpd (88 gpm), with a maximum use of 214,000 gpd (149 gpm).

Well 14 was then retested while monitoring two residential wells (about 2500-3000 feet from well 14) along Fringer Road. While pumping well 14 for 72 hours at nearly the same rate (230 gpm) as that used during the first test (200-230 gpm), the drawdowns (13 ft and 47 ft) observed in the two Fringer Road domestic wells were much greater than those measured in the observation wells (15 ft or less and located within 325 feet of well 14) during the 1990 test.

On August 29, 1997, the Water Supply Program received a report that the Fringer Road residents were again having problems with their wells. The pumping rate of well 14 was reduced to a level that restored the supply to the domestic wells and no further complaints were made by the residents along Fringer Road. This, however, did not provide a permanent solution to the problem, so all available hydrogeological data were reviewed to develop a set of special permit conditions that would ensure an adequate water supply for the Fringer Road residents.

Aquifer Test Data

During the 1990 72-h test, water levels were measured in well 14, piezometer P-2 (in the annulus of well 14), well 6 (85 ft NE of well 14), and well 4 (325 ft NE of well 14). Near the end of the second day, while pumping at 200 gpm, the drawdowns were 338 ft in well 14, 15 ft in well 6, 5 ft in well 4 and 40 ft in P-2. An increase to about 230 gpm near the end of the second day produced an immediate drawdown of about 60 feet in well 14, Figure 14, but no significant changes of the water levels in the observation wells. This may have been due to a delayed reaction in a leaky aquifer due to the change in the pumping rate.



Time	Rate
minutes	gpm
0	200
3	138
5	180
6	190
9	198
13	193
15	205
20	200
1518	235
1740	229
1800	0
1801	250
1830	238
1920	229
2700	227
3000	225
3540	227

Figure 14. Taneytown well 14 – Semi-log plot of drawdowns from a 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.

The second 72-h test of well 14 was conducted during the period March 6-9, 1995, at a pumping rate of 230 gpm, while monitoring water levels in well 14, well 6, and the Fringer Road Kirkpatrick (2500 ft from well 14) and Welty (3200 ft from well 14) house wells.

On a semi-log, time-drawdown graph, the response in well 6 was like that noted during the first test, except that the total drawdown of 8.5 feet was only about ½ of that recorded during the first test, although the pumping rates were similar. The consultant did not include the water level data for well 14 in its report. The responses in the Kirkpatrick and Welty wells were significantly different than those noted in wells 14, 4, and 6, in that there was no flattening of the curves for the two domestic wells during the test. The response in those two wells was one with a steadily increasing rate of decline, producing radial-linear flow curve on semi-log graphs, Figures 15 and 16. At the end of the second test, the total drawdown in the Kirkpatrick well was 47 ft and 13 ft in the Welty well.

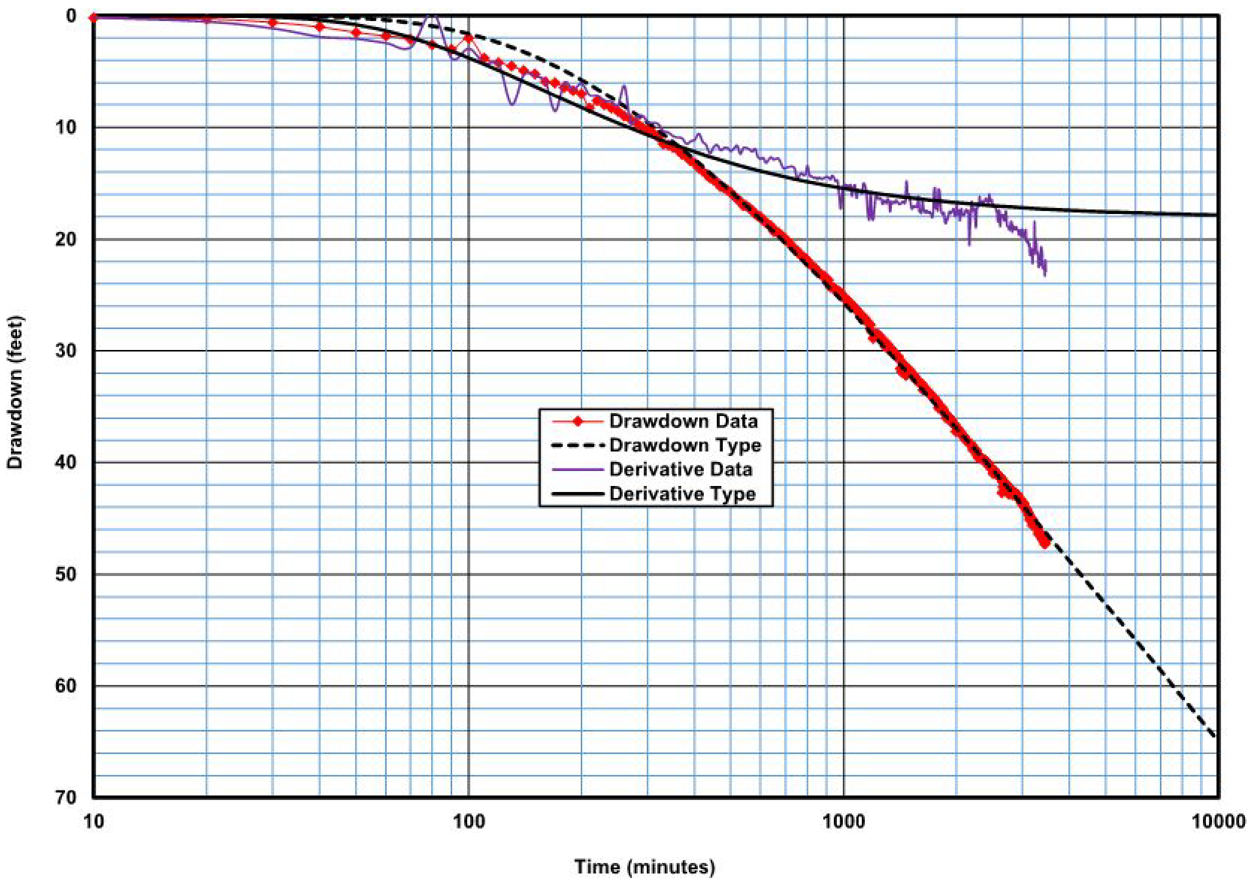


Figure 15. Semi-log plot of drawdowns in the Kirkpatrick observation well from a 72-h, variable rate aquifer test of Taneytown well 14, Barker (General Radial Flow) solution.

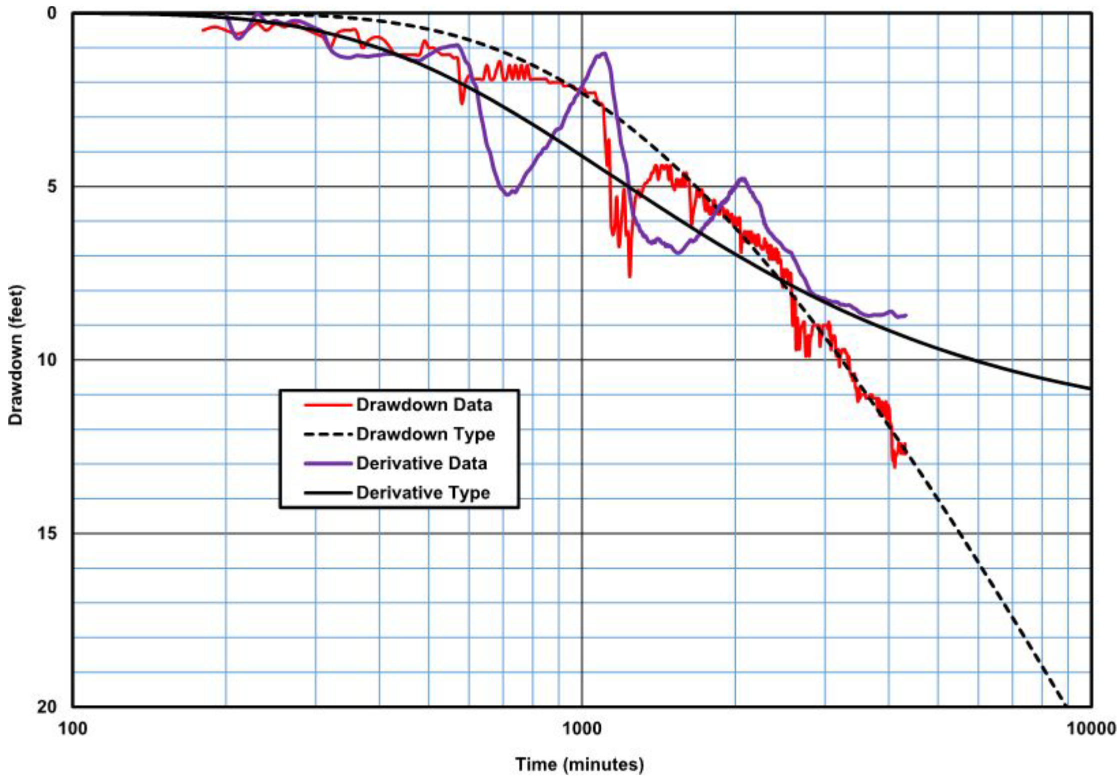


Figure 16. Semi-log plot of drawdowns in the Welty observation well from a 72-h, variable rate aquifer test of Taneytown well 14, Gringarten-Witherspoon SVF-F solution.

Aquifer Constants

During previous investigations, a wide range of aquifer constants were calculated from the data collected during the tests of well 14.-Transmissivities (T) of 1650 to 2640 gpd/ft were derived from the data taken from well 14, the two Fringer Road observation wells, and, during the early portion (13-50 minutes) of the first test, from the piezometer P-2. From the data taken from the early-time data of well 6 during the first test, the T values was 6500 gpd/ft, and 18,000 to 60,000 gpd/ft from late-time data in both wells 4 and 6. The calculated T values could be biased by lower permeability zones between a deep, horizontal (bedding plane), rate-controlling fracture system and the shallow observation wells. These lower permeability zones may have caused an attenuation of the responses to the pumping of well 14 in the shallow observation wells, like that which would occur in a semi-confined, or leaky, coastal plain aquifer. The drawdown in the shallow observation wells would, then, be less than would occur if the lower permeability zones were not present. If the vertical anisotropic nature of such an aquifer were not accounted for, then the calculated T values would much higher than the actual aquifer transmissivity.

Table 4 presents the results of AQTESOLV analyses from the well 14 tests and nearby tests of Taneytown's wells 9, 10, 10R, 11 and 13. Highlighted in red are the best solutions from each test. The data indicate that the aquifer is anisotropic with a long axis or maximum transmissivity from well 14 in the direction of the Kirkpatrick house well, Figure 17.

Table 4. Summary of the results of the step and aquifer testing of Taneytown well 14.

Test-Obs Well	Notes	T	S	r	s	t	Model	DERIV	RSS	Var	S.D.	Mean
WELL 14 TESTS												
Well 14 (1990 Test @ 200/230 gpm)	Uncorrected	1039	N/A	-	340	0-4320	Moench3	Leaky	12,160	118	10.9	-0.499
	Uncorrected	872	N/A	-	264	0-1500	Moench3	Leaky	1225	26.60	5.16	-0.033
	Corrected (b=270 ft)	1813	N/A	-	135	0-1500	Moench3	Leaky	253	5.62	2.37	-0.039
Well 14 (1990 Step-test)	Steps 2-4 (b=270 ft)	1985	N/A	-	104	67-1460	Dough-Babu	-	801	8.01	2.83	0.196
	Step 1 (No correction needed)	2354	N/A	-	19	0-65	Barker	-	8.1	0.31	0.56	0.086
		2186	N/A	-	19	0-65	Dough-Babu	-	2.5	0.11	0.33	4E-05
		2264	N/A	-	19	0-65	Moench3	-	6.9	0.29	0.54	0.019
Kirkpatrick Well (1994 Test @ 230 gpm)		5514	3E-05	2450	47	0-3460	Barker	Not Diagnostic	31.3	0.091	0.03	0.0015
		503	6E-03	2450	47	0-3460	Moench3	Not Diagnostic	1519	4.44	2.11	0.442
		1440	1E-05	2450	47	0-3460	SVF-F	Not Diagnostic	97.5	0.285	0.534	0.035
Welty Well (1994 Test @ 230 gpm)		37,400	7E-04	3300	13	1500-4320	Barker	Radial	45.0	0.165	0.401	0.0007
		979	3E-02	3300	13	1500-4320	Moench3	Radial	54.8	0.196	0.443	0.010
		2481	7E-05	3300	13	1500-4320	SVF-F	Radial	46.6	0.167	0.409	0.0025
Well 13 (Step-test)(steps 1-3)	Corrected (b=164)	2357			12	0-80	Moench3	Leaky/erratic	58	1.32	1.15	0.102
Well 9 (single-well, variable rate)	Uncorrected	1295			245	0-1440	Barker	Leaky/erratic	1628	60.3	7.77	0.708
	Uncorrected	602			245	0-1440	Moench3	Leaky/erratic	1477	59.1	7.69	0.711
Well 11 (single-well, variable rate)	Uncorrected	154			373	0-1440	Moench3	Leaky/erratic	2E+04	337	18.34	1.645
Well 10 (Steps 1-7)		1399			136	0-420	Dough-Babu	-	1551	9.88	3.14	-1.58
Well 10 (Steps 1-5)		1338			92	0-300	Dough-Babu	-	142	1.12	1.06	0.026
Well 10 (Steps 1-4)		1145			69	0-240	Dough-Babu	-	11.0	0.104	0.322	0.011
Well 10 (Steps 1-4)		530			69	0-240	Moench3	-	92.3	0.871	0.933	-0.169
Well 10 (Steps 1-4)		925			69	0-240	SVF-F	-	81.9	0.773	0.879	-0.109
Well 10R (Steps 1-4)		1778			66	0-330	SVF-F	-	3648	11.2	3.34	0.751
Well 10R (Steps 1-4)		2163			66	0-330	Dough-Babu	-	707	2.16	1.47	-0.137
Well 10R (Steps 1-4)		804			66	0-330	Moench2	-	720	2.20	1.48	-0.289
Well 10R (Steps 1-4)		1391			66	0-330	Barker	-	2453	7.46	2.73	-0.476
Well 10R (72hr, early time)		901			92	0-270	Barker	Bilinear	415	1.55	1.25	-0.119
Well 10R (72hr, late time)		336			180	271-4353	Barker	Bilinear	3E+04	7.38	2.72	-0.231
Well 10R (72hr)		845			180	0-4353	Moench3	Bilinear	5E+04	11.0	3.32	-0.328

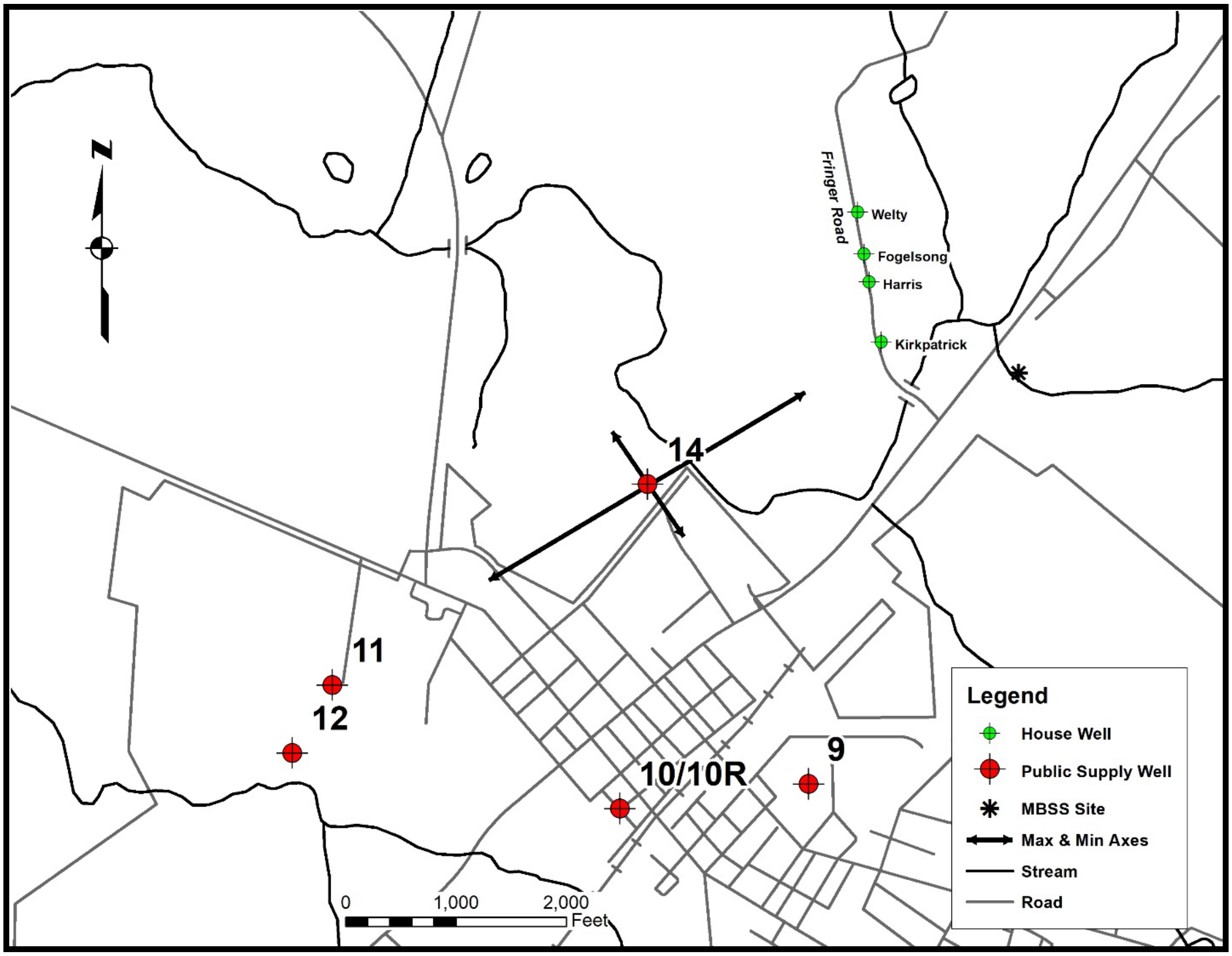


Figure 17. Results of the well 14 aquifer test indicating the maximum (long axis) and minimum (short axis) transmissivity directions of an anisotropic aquifer.

Time-Distance-Drawdown Calculations

The drawdown (s_x) at any point near Taneytown well 14 is determined by the following equation:

$s_x = 264\ Q / (T_{\xi\xi} * T_{\eta\eta})^{1/2} * \log (0.3\ t / S)\ (T_{\xi\xi} * T_{\eta\eta})/(T_{\xi\xi}\eta^2 + T_{\eta\eta}\xi^2)$

s_x = drawdown (ft) at given distance
Q = pumping rate (gpm)
$T_{\xi\xi}$ = Transmissivity in the major direction of anisotropy (gpd/ft)
$T_{\eta\eta}$ = Transmissivity in the minor direction of anisotropy (gpd/ft)
$(T_{\xi\xi} * T_{\eta\eta})^{1/2} = T_E$ (effective Transmissivity)
ξ = distance from pumping well (ft) along major axis
η = distance from pumping well (ft) along minor axis
t = time (d)
S = Composite Storage Coefficient (dimensionless)
Aquifer Constants (T&S) from:
On-site aquifer tests
Observed drawdowns:
Well 14 3460 min of 72-h test: s = 338ft (Well 14), 47ft (Kirkpatrick), and 11 ft (Welty)
Kirkpatrick well – X = 2500 ft; Welty well – X = 3000 ft, Y = 1650 ft

Through an iterative process, various scenarios were constructed to best derive the drawdowns observed during the aquifer test of well 14.

First Scenario: Anisotropic Model

The first scenario assigned the approximate T value (5000 gpd/ft) for the Kirkpatrick well as the maximum T, a first approximation of 750 gpd/ft as a minimum T and a storage coefficient of 0.00005. This produced an error of -42% in the estimated drawdown (33 ft) in the Kirkpatrick well relative to the measured drawdown (47 ft). The estimated drawdown in the Welty well is assumed to be between 28 ft (X-axis) and 19 ft (Y-axis). The resulting error then lies between -61% (X-axis) and -42% (X-axis).

t = 2.4 days Q = 230 gpm S = 0.00005
$s_x = (264*230 / 1936) * \log (((0.3*2.4 / 0.00005)\ ((3{,}750{,}000) / (5000\eta^2 + 750\xi^2)))$

$s_x = (31.4) * \log ((14{,}400)\ (3{,}750{,}000) / (5000\eta^2 + 750\xi^2))$

$s_x = (31.4) * \log 72{,}000{,}000/\xi^2 \quad \eta = 0$
s_{100} = 121 ft s_{200} = 102 ft s_{500} = 77 ft s_{1000} = 58 ft s_{2000} = 39 ft $\underline{s_{2500} = 33\text{ ft}\ s_{3000} = 28\text{ ft}}$
$s_x = (31.4) * \log 10{,}800{,}600/\eta^2 \quad \xi = 0$
s_{100} = 95 ft s_{200} = 76 ft s_{500} = 51 ft $\underline{s_{1650} = 19\text{ ft}}$ s_{2000} = 14 ft s_{3000} = 2.5 ft s_{5000} = 0 ft

Final Scenario: Anisotropic Model

The final scenario for the anisotropic model assigned the approximate T value (6000 gpd/ft) for the Kirkpatrick well as the maximum T, a first approximation of 600 gpd/ft as a minimum T and a storage coefficient of 0.00004. The 10 ft, 20 ft and 50 ft drawdowns from this scenario were plotted on Figure 18. This produces an estimated drawdown of 40 ft in the Kirkpatrick well against the measured value of 47 feet for an error of -17.5%. The estimated drawdown is 15 ft in the Welty well against the measured value of 11 feet produced an error of 27%.

$t = 2.4$ days $Q = 230$ gpm $S = 0.00004$
$s_x = (264*230 / 1897) * \log (((0.3*2.4 / 0.00004) ((3{,}600{,}000) / (6000\eta^2 + 600\xi^2)))$

$s_x = (32.0) * \log ((18{,}000) (3{,}600{,}000) / (6000\eta^2 + 600\xi^2))$

$s_x = (32.0) * \log 108{,}000{,}000/\xi^2 \quad \eta = 0$
$s_{100} = 129$ ft $s_{1700} = 50$ ft $s_{2500} = 40$ $s_{3000} = 34.5$ ft $s_{5000} = 20$ ft $s_{7250} = 10$ ft

$s_x = (32.0) * \log 10{,}800{,}000/\eta^2 \quad \xi = 0$
$s_{100} = 97$ ft $s_{540} = 50$ ft $s_{1000} = 37$ ft $s_{1600} = 20$ ft $s_{2000} = 14$ ft $s_{2300} = 10$ ft $s_{3000} = 2.5$ ft

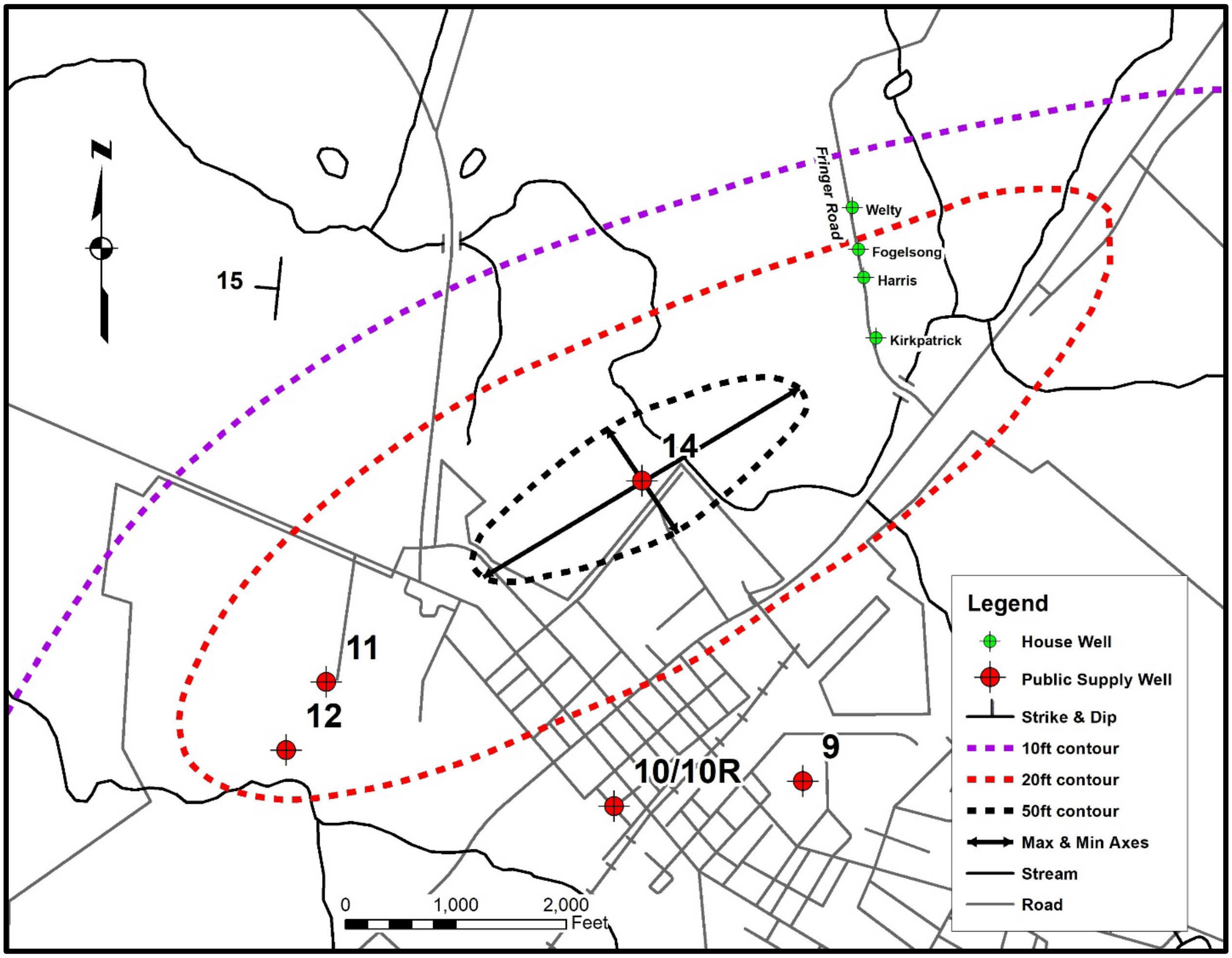


Figure 18. Anisotropic model derived from the results of the well 14 aquifer test. Shown are the maximum (long axis) and minimum (short axis) transmissivity directions, the calculated drawdown contours, and the strike and dip of the New Oxford Formation.

Scenario: Anisotropic model plus ¼ L model:

Next, using the hydraulic constants from the final anisotropic model, a ¼ L model was devised which indicated that the estimated fracture length was insignificant (i.e., less than one foot) and radial flow can be assumed to occur at the observation wells.

s = 338ft (Well 14)

t = 2.4 days Q = 230 gpm S = 0.00004

$s_x = (264*230 / 1897) * \log (((0.3*2.4 / 0.00004) ((3{,}600{,}000) / (6000\eta^2 + 600\xi^2)))$

$s_x = (32.0) * \log ((18{,}000)\ (3{,}600{,}000) / (6000\eta^2 + 600\xi^2))$
$s_x = (32.0) * \log 108{,}000{,}000/\xi^2 \quad \eta = 0$
$s_1 = 257$ ft

$s_x = (32.0) * \log 10{,}800{,}000/\eta^2 \quad \xi = 0$
$s_1 = 225$ ft

¼ L in Well 14 <1ft and is insignificant. Radial flow assumed at observation wells.

Scenario: Maximum permitted use in 1998 because of the impact analysis

The result is an estimated drawdown of 29 feet in the Kirkpatrick well and 22-27 feet in the Welty well.

t = 90 days Q = 73 gpm
$s_x = (264*73 / 1897) * \log (((0.3*90 / 0.00004)\ ((3{,}600{,}000) / (6000\eta^2 + 600\xi^2)))$

$s_x = (10.2) * \log ((675{,}000)\ (3{,}600{,}000) / (6000\eta^2 + 600\xi^2))$

$s_x = (10.2) * \log 4{,}050{,}000{,}000/\xi^2 \quad \eta = 0$
$s_{100} = 57$ ft $s_{1700} = 32$ ft $s_{2500} = 29$ $s_{3000} = 27$ ft $s_{5000} = 22$ ft $s_{7250} = 19$ ft

$s_x = (10.2) * \log 405{,}000{,}000/\eta^2 \quad \xi = 0$
$s_{100} = 47$ ft $s_{540} = 32$ ft $s_{1000} = 27$ ft $s_{1650} = 22$ ft $s_{2000} = 20$ ft $s_{2300} = 19$ ft $s_{3000} = 17$ ft

Scenario: 61d at maximum reported use

A simulation at the maximum reported production (61 d @ 95 gpm: May-Jun 2004).

t = 61 days Q = 95 gpm
$s_x = (264*95 / 1897) * \log (((0.3*61 / 0.00004)\ ((3{,}600{,}000) / (6000\eta^2 + 600\xi^2)))$

$s_x = (13.2) * \log ((457{,}500)\ (3{,}600{,}000) / (6000\eta^2 + 600\xi^2))$

$s_x = (13.2) * \log 2{,}745{,}000{,}000/\xi^2 \quad \eta = 0$
$s_{100} = 72$ ft $s_{1700} = 39$ ft $s_{2500} = 35$ (K) $s_{2570} = 35$ ft (H) $s_{2980} = 33$ ft (F) $s_{7250} = 23$ ft

The measured drawdowns were 31 ft in the Kirkpatrick (K) well (11% error), 31 ft in the Harris (H) well (11% error) and 21 ft in the Fogelsong (F) well (-36% error). It is also noted that the drawdowns are less near the pumping well than during the 72h tests, but greater as distance from well 14 increases. This is consistent with the GeoServices, Ltd. observation that the interference problems propagate westward with time along Fringer Road from the Kirkpatrick well to Welty well.

Scenario: 90d at maximum permitted use

The permit revised in 2006 to increase use to 90,000 gpd avg / 197,000 gpd max, with provisions that 12 house wells be replaced immediately (with permission of the owner) and 9 other house wells not requiring immediate water supply replacement. Periodic water level measurements were required in 5 other house wells, while the depths of 4 other wells had to be verified. The estimated drawdowns at the maximum approved rate (137 gpm) follow:

t = 90 days Q = 137 gpm

$s_x = (264*137 / 1897) * \log (((0.3*90 / 0.00004) ((3{,}600{,}000) / (6000\eta^2 + 600\xi^2)))$

$s_x = (19.1) * \log ((675{,}000) (3{,}600{,}000) / (6000\eta^2 + 600\xi^2))$

$s_x = (19.1) * \log 4{,}050{,}000{,}000/\xi^2 \quad \eta = 0$

$s_{100} = 107$ ft $s_{1700} = 60$ ft $s_{2500} = 54$ $s_{3000} = 51$ ft $s_{5000} = 42$ ft $s_{7250} = 36$ ft

$s_x = (19.1) * \log 405{,}000{,}000/\eta^2 \quad \xi = 0$

$s_{100} = 88$ ft $s_{540} = 60$ ft $s_{1000} = 50$ ft $s_{1600} = 42$ ft $s_{2000} = 38$ ft $s_{2300} = 36$ ft $s_{3000} = 32$ ft

Another factor to consider is that the strike of the formation is northerly with a 15° dip to the west, Figure 19. This places the Fringer Road house wells up dip of well 14. The main water bearing zones in well 14 are 470 and 595 ft deep. The house wells are about 1600 ft or more up dip of well 14, which places the interval of the Well 14 water bearing zones at 40 to 167 ft along the Fringer Road impact area, at the approximate depths of the house wells. The long axis of the anisotropic aquifer also stretches in the direction of well 11 which has a much lower calculated transmissivity (154 gpd/ft) than might be expected. This, however, is not inconsistent, since well 11 is about 2800 ft downdip of well 14 and the equivalent to the Well 14 producing interval is 1280 to 1345 ft deep in well 11. That is about 1000 ft below the 290-315 ft water bearing zone in that well. The aquifer supplying well 11 is a much shallower lower transmissivity unit than that of well 14. The structural and stratigraphic orientation of the aquifer supplying well 14 might also explain why the drawdown of wells (wells 4 and 6) in the immediate vicinity had much less drawdown than the house wells along Fringer Road. It also may provide a reason that no impacts were reported for private wells further updip along Francis Scott Key Highway (FSK HWY), downdip along Harney Road or in the immediate vicinity of well 14 along Pumphouse Road.

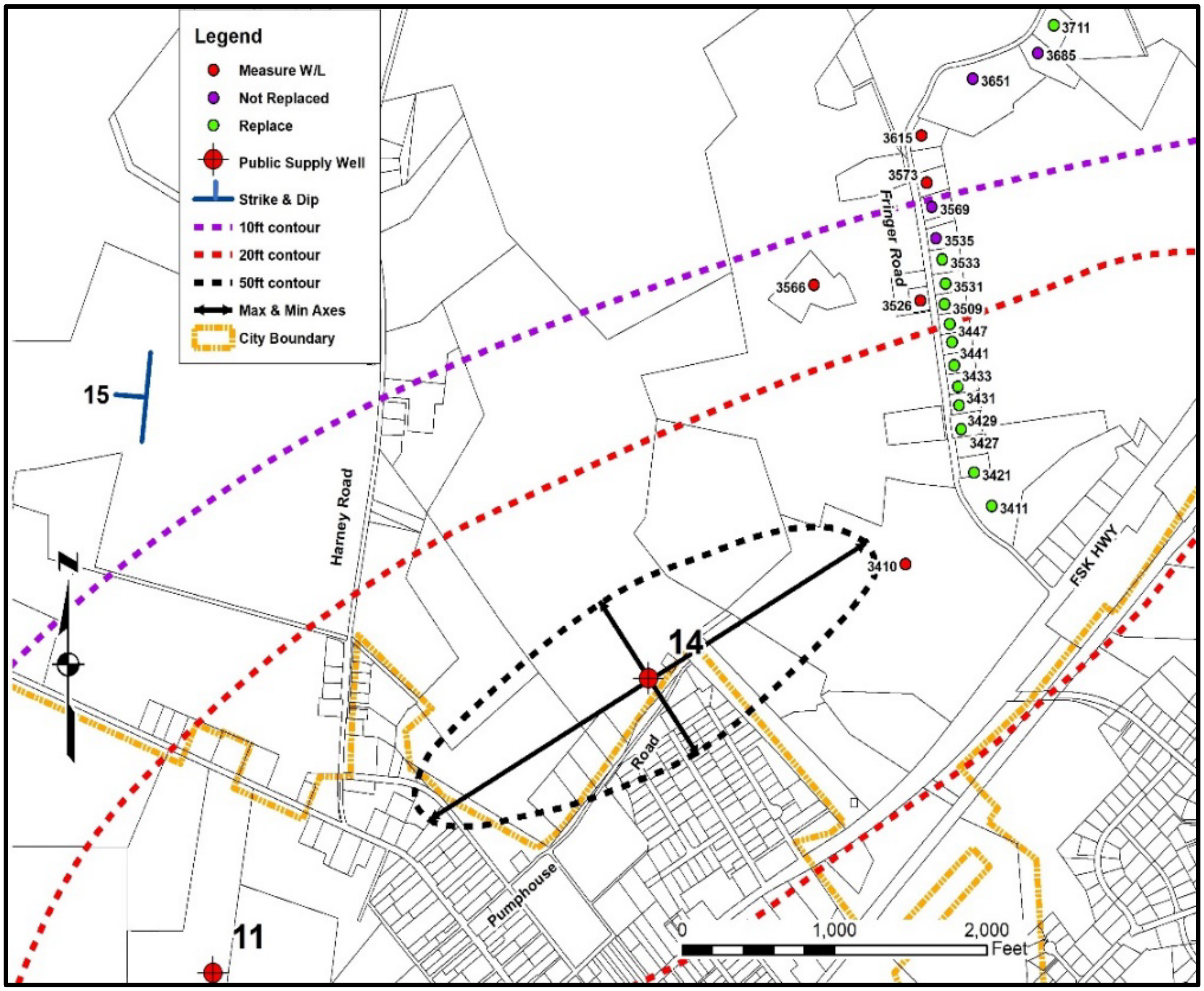


Figure 19. Anisotropic model derived from the results of the well 14 aquifer test. Shown are the maximum (long axis) and minimum (short axis) transmissivity directions, the calculated drawdown contours, and the strike and dip of the New Oxford Formation, as well as the house wells to monitored or replaced.

Table 5. Characteristics of the domestic water supplies potentially impacted due to withdrawals from Taneytown well 14.

Address	Owner	Old Well					New well			Water level monitoring	
Fringer Road	1994(2011+)	depth (ft)	SWL (ft)	yield (gpm)	turbidity	Lost yield or low pressure	Tag No.	depth	yield	Period	Frequency
3410	Munderloh (Stapleton)	~72	N/R	N/R	Y	Y	N/A	N/A	N/A	Permission not granted	N/A
3411	Smith	100	27	30	N	N/R	CL-95-0558	240	8	N/A	N/A
3421	Kirkpatrick	100	20	20	Y	Y	CL-95-0545	200	30	N/A	N/A
3427	Grimes (Mulleneaux)	106	40	10	N	N/R	Permission not granted			N/A	N/A
3429	Harris	90	30	7	Y	Y	Permission not granted			N/A	N/A
3431	Eckard (Harris)	86	23	10	Y	N/R	Permission not granted			N/A	N/A
3433	Liller	90	40	5	N	N/R	Permission not granted			N/A	N/A
3441	Fogelsong	82	25	4.5	Filter	Y	Permission not granted			N/A	N/A
3447	Guthrie(Curtis)	˜80	25	8	Y	N	CL-95-0544	200	20	N/A	N/A
3509	Gross	98	29	3	Y	Y	CL-95-0550	240	8	N/A	N/A
3526	Overholtzer	170	38	23	N	N/R	N/A	N/A	N/A	Permission not granted	N/A
3531	Welty	76	17	4	Y	N	CL-95-0549	300	10	N/A	N/A
3533	Lehigh	95/78?	16	5	Y	N/R	CL-95-0548	200	30	N/A	N/A
3535	Shipley(Warner)	N/R	N/R	N/R	N/R	N/R	N/A	N/A	N/A	Jan 2007-present	monthly
3566	Riley (Bowers)	173	45	9.5	N/R	N/R	N/A	N/A	N/A	Permission not granted	N/A
3569	Storey(Reno)	N/R	N/R	N/R	N/R	N/R	N/A	N/A	N/A	Jan 2007-present	monthly
3573	Nussbaum(Oliver)	135	20	30	N/R	N/R	N/A	N/A	N/A	Jan 2007-present	monthly
3603	Green(Geiman)	72	13	7	N/R	N/R	N/A	N/A	N/A	N/A	N/A
3615	Ohler(Carl)	130	27	5	N/R	N/R	N/A	N/A	N/A	N/A	N/A
3630	Osborne	220	75	8	N/R	N/R	N/A	N/A	N/A	Jan 2007-present	monthly
3651	Garrett	N/R	N/R	N/R	N/R	N/R	N/A	N/A	N/A	N/A	N/A
3685	Dintermon(Wilhide)	N/R	N/R	N/R	N/R	N/R	N/A	N/A	N/A	N/A	N/A
3711	Haines (Ruby)	75	N/R	N/R	N	Y	CL-95-0546	200	6	N/A	N/A
Pumphouse Road	1994	depth (ft)	SWL (ft)	yield (gpm)	turbidity	Lost yield or low pressure	Tag No.	depth	yield	Period	Frequency
61	Uttenreither	125	52	12	N/R	N/R	N/A	N/A	N/A	N/A	N/A
63	Morningstar	125	40	15	N/R	N/R	N/A	N/A	N/A	N/A	N/A
67	Eanes	150	29	12	N/R	N/R	N/A	N/A	N/A	N/A	N/A
71	Carico	N/R	N/R	N/R	N/R	N/R	N/A	N/A	N/A	N/A	N/A

Ultimately, only 7 house wells were replaced, Table 5, as 5 homeowners would not give permission to drill new wells. A new permit was issued in July 2006 and, since then, there have been no further reports of impacts to house wells along Fringer Road. This is likely due to the overall reduced use over time from well 14. After the peak withdrawals and maximum drawdowns in the house wells in 2004, water use then declined by about 50% through 2018, Figure 20. As noted by Hammond (2018), this may have been due to either compression of or calcite cementation in the reservoir unit supplying well 14 due to lowering of the pumping water level in the well.

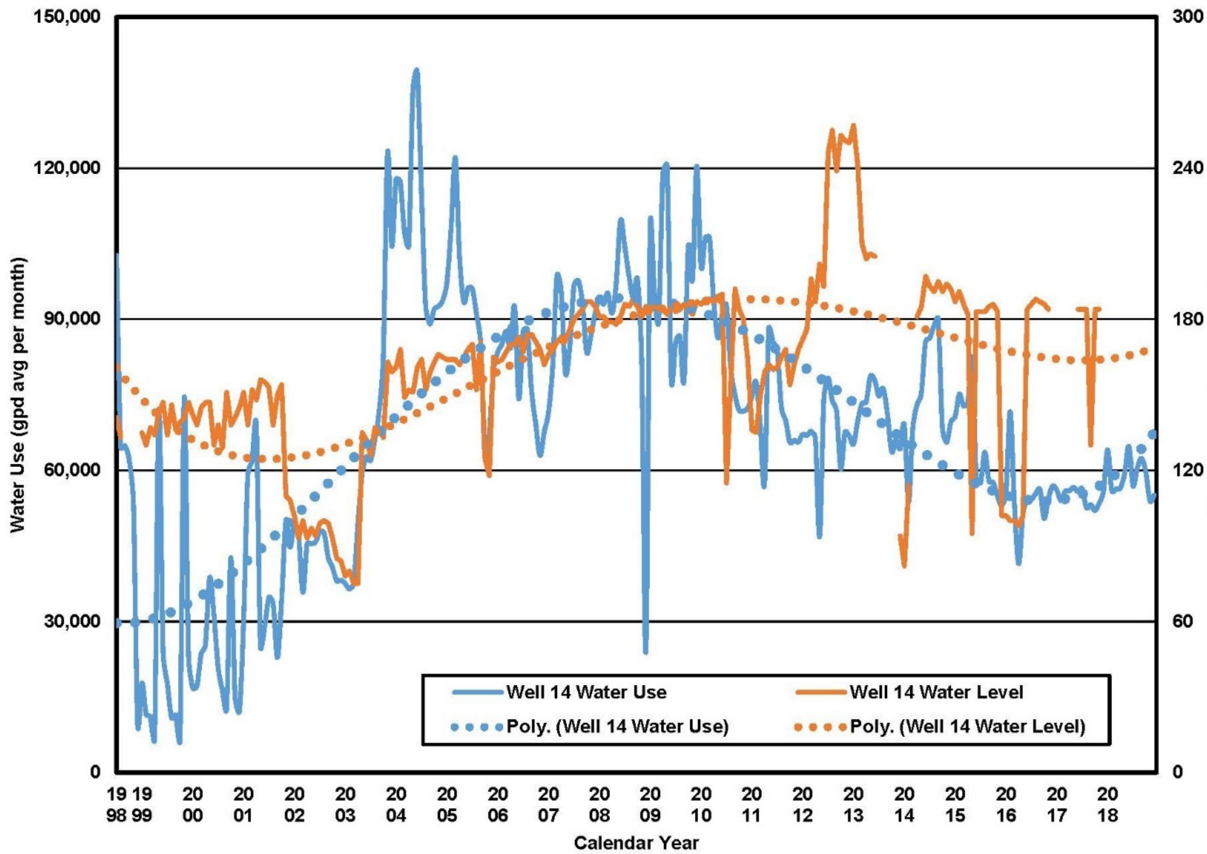


Figure 20. Water use and water level data collected from Taneytown well 14.

Poolesville Wells 9 and 10 Case Study

Otten (1981) provides an early history of development of the municipal groundwater supply for the Town of Poolesville, in western Montgomery County. Prior to 1969, about 50-60 homes or stores in the town were supplied by individual wells or springs. A few were dug wells, while the remaining ones were drilled to depths between 75 and 150 feet. By 1963, several of the wells became contaminated, primarily by effluent from domestic wastewater disposal systems. In 1969, the first two wells were completed for a municipal water supply system to serve the town. One additional well each was drilled in 1972 and 1977. Since the Otten study was published, well 1 was abandoned and ten additional wells were completed to supply a population of about 5500 people.

Potential impacts to private water supply wells due to the Poolesville municipal withdrawals were first reported in 1973, after completion of the third town well. These were about a dozen domestic and commercial wells in the immediate vicinity of the three municipal wells. The State Water Commission directed the town government to hook up any private homeowner to the public water supply who was within 2000 feet of a municipal well and requested to be added to the system. Follow-on monitoring of groundwater levels by Otten in 1978/79 indicated that troughs of depression extended to as much as about ½ mile from the three town wells. Single-well aquifer tests had been conducted for the first three wells to estimate their reliable yields. Apparently due to the potential impacts caused by the first three wells, multi-well tests were conducted for wells 4 and 5. Those tests indicated that impacts may occur, so nearby homeowners were given the option of hooking into the public supply. Multi-well tests were also conducted for wells 6, 7 and 8 in 1986, 1992, and 1994, respectively. While those tests indicated that it was possible that impacts might occur; if any did happen, none were reported to the State agency.

In 1999, a multi-well test was conducted on a proposed municipal well at the Bachelor's Purchase property. The test was secured after 65 hours, due to a statewide drought emergency. The nearest domestic well (Wilkins) went dry 57 hours into the test with 23 feet of drawdowns; but the water level recovered enough two hours after the end of the pumping phase of the test to supply the homeowner's needs. Drawdowns in nine other observation wells along Hughes Road varied from 2 to 12 feet. The town abandoned the proposed well due to a relatively low yield and the potential for impacting a significant number of nearby wells. The extensive amount of data collected during the test was not wasted, as it was used during the follow-on multi-well tests and impacts related to town wells 9 and 10.

Figure 21 is a map showing the location of the Town of Poolesville's public water supply wells. Figure 22 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. Step-drawdown and aquifer test and water level monitoring data are presented for town wells 9 (Powell) and 10 (Cahoon).

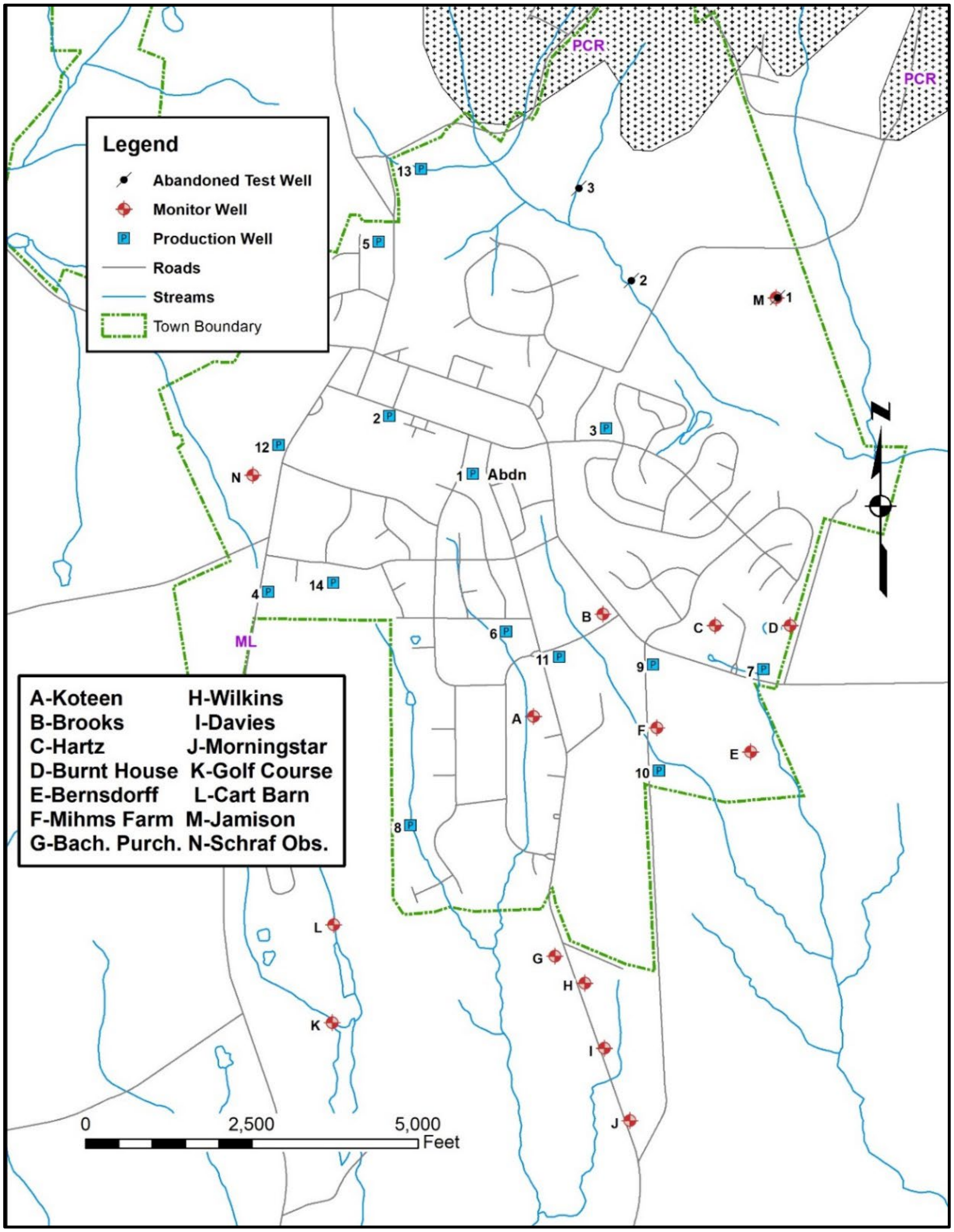


Figure 20. Location map. Poolesville public water supply, monitoring, and abandoned test wells. The wells are in the New Oxford Formation of the Mesozoic Lowland (ML) HGMR. To the north of the town are rocks in the Piedmont Crystalline (PCR) HGMR.

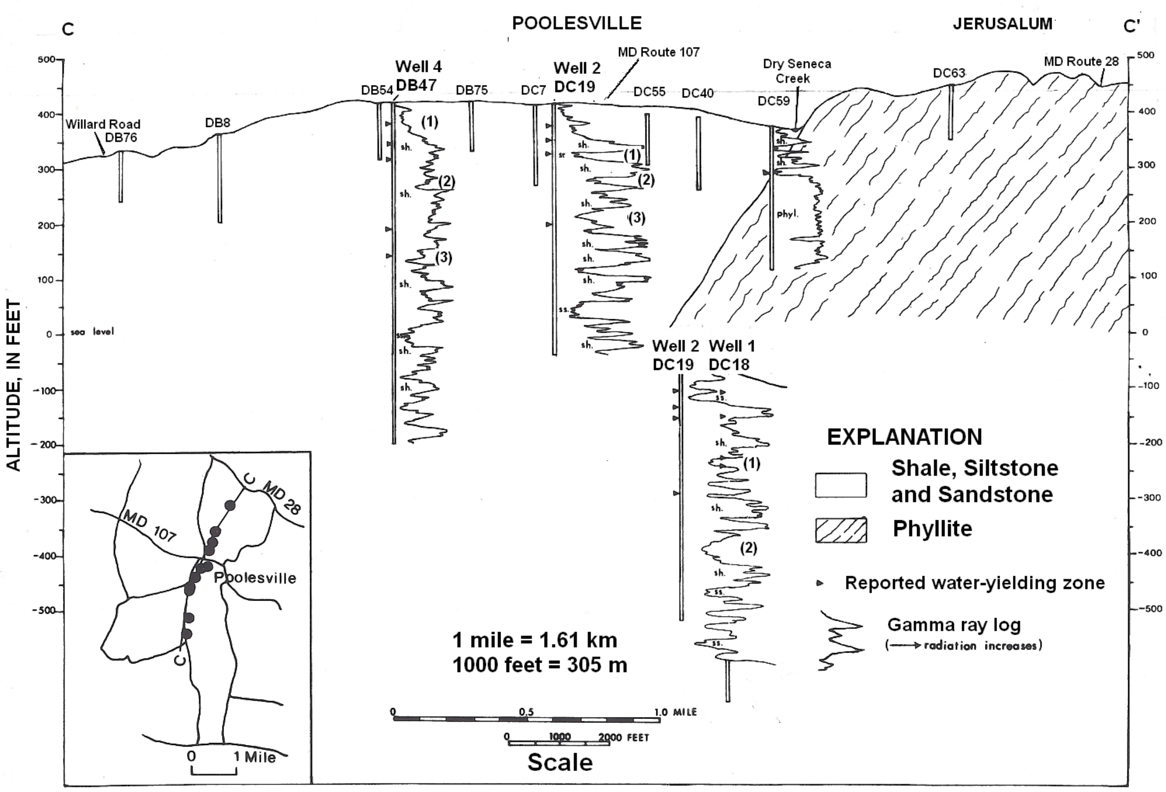


Figure 21. Geologic cross sectionof the Poolesville area (modified from Otton, 1981)

Well 9 (Powell) Tests

Multi-rate and aquifer tests were performed on town 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), Figure 23. 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, Figure 24, 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).

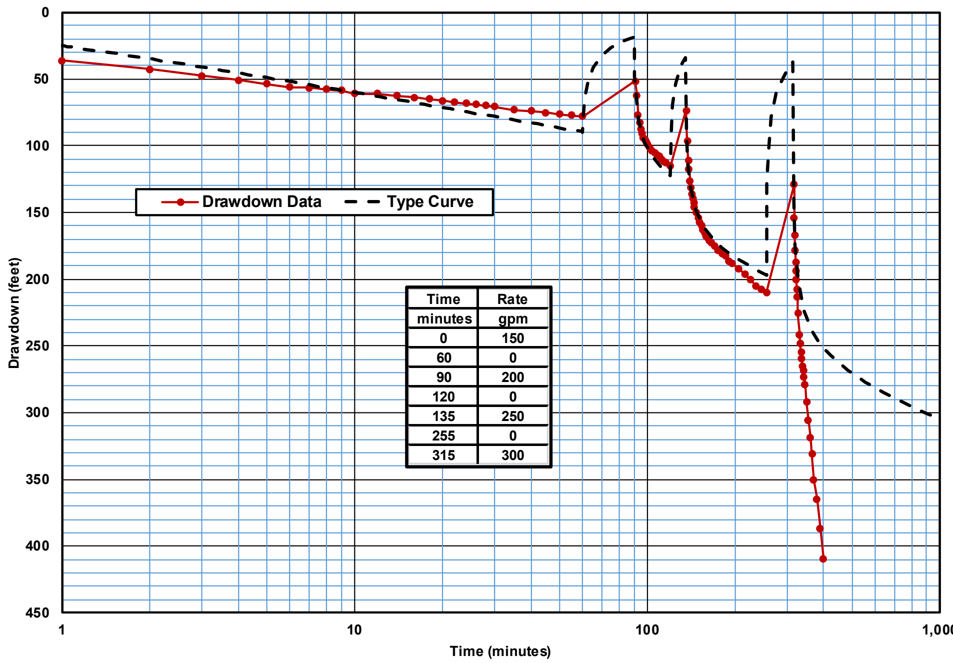


Figure 23. Poolesville well 9 (Powell)– Semi-log plot of drawdowns from steps 1−3 of a multi-rate test, Hantush-Jacob leaky aquifer solution.

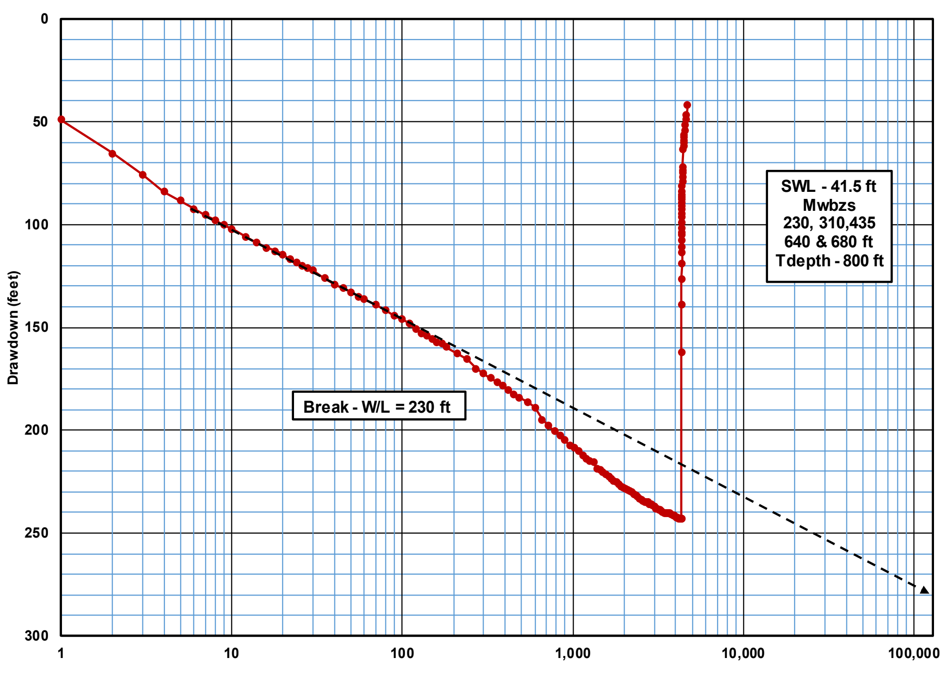
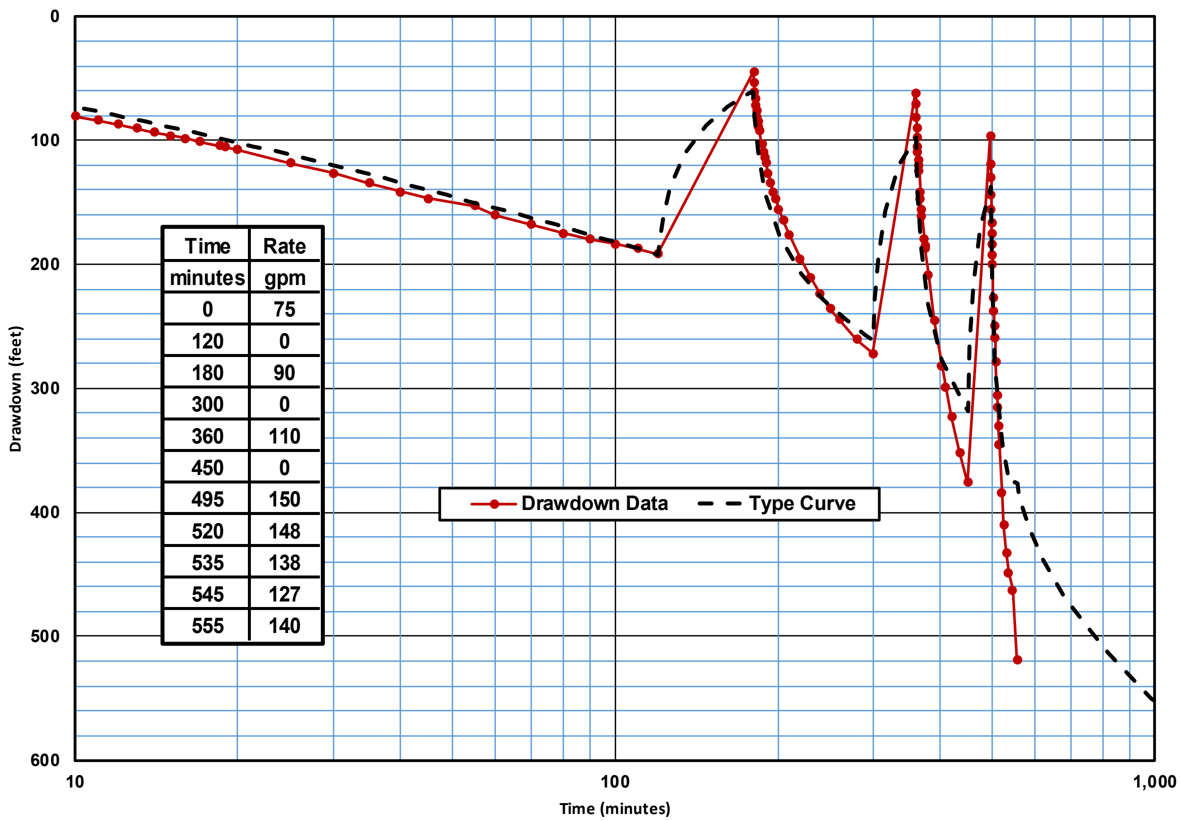


Figure 24. Poolesville well 9 (Powell) – Semi-log plot of drawdowns from a 72-h, 225-gpm aquifer test, 0-600 min Hantush-Jacob solution.

Well 10 (Cahoon) Tests

A multi-rate test and a 72-h, 80 gpm (303 L/min) aquifer test were performed on town well 10 in May 2001, under average climatic conditions. The Hantush-Jacob leaky aquifer solution best fit the drawdown data from the step test; 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, Figure 25. 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), Figure 26. 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.



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

Figure 25. Poolesville well 10 (Cahoon) – Semi-log plot of drawdowns from a multi-rate test, Hantush-Jacob leaky aquifer solution.

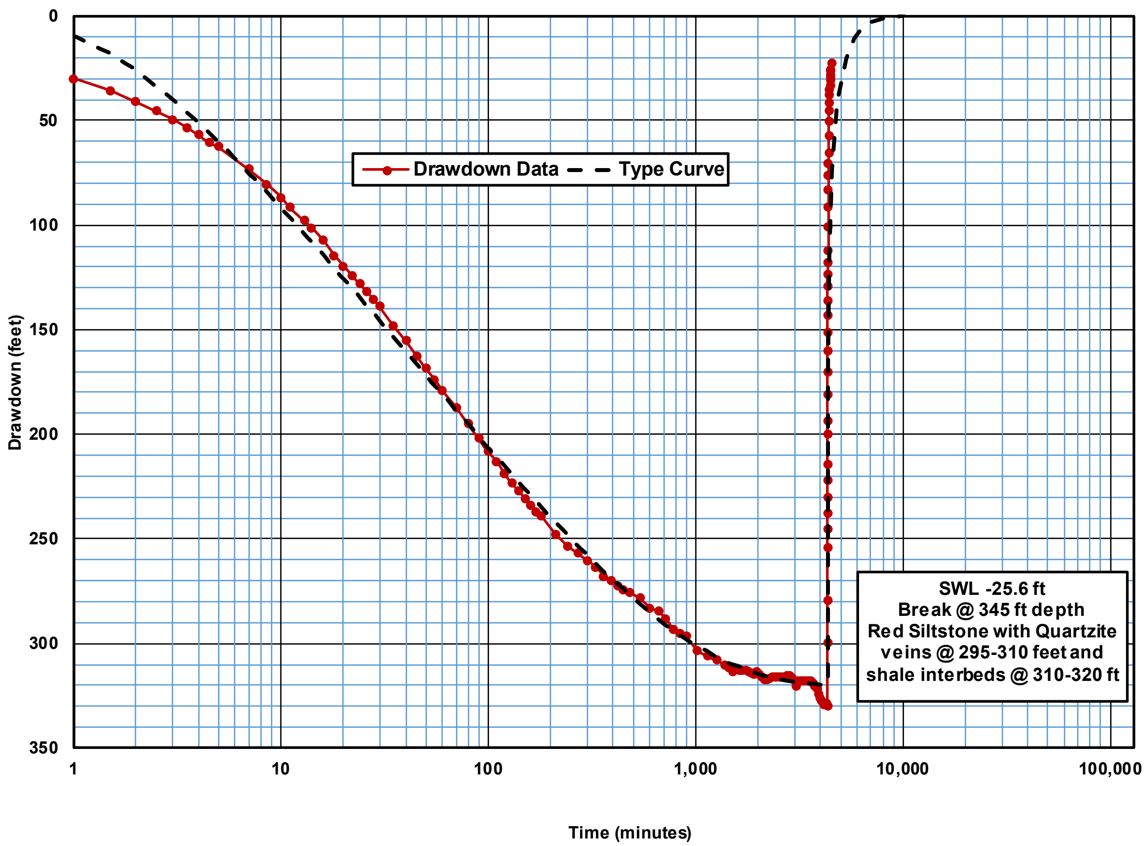


Figure 26. Poolesville well 10 (Cahoon) – Semi-log plot of drawdowns from 0−3600 min of a 72-h, 80-gpm aquifer test, Hantush-Jacob leaky aquifer solution.

Following completion of the permit process and to address potential impacts, the following house wells were replaced by the town: 19010 Fisher Avenue (Davis), 19101 Fisher Avenue (Hartz), 19200 Fisher Avenue (Powell), 16815 Budd Road (Mihm), 16615 Budd Road (Cahoon), and 19315 Fisher Avenue (Northern). Public water was supplied to the Mihm nursery at no cost up to an average of 8,000 gpd. In addition, monthly water level monitoring was required for four house wells and old well 10 on the Bachelor's Purchase property to determine if impacts due to withdrawals would occur in the Sugarland Forest community, about one mile south of municipal wells 9 and 10.

The drawdowns observed during the aquifer test of the Powell well (well 9) were simulated using the analytical techniques and iterative methods developed in this investigation and the companion crystalline rock study. These following calculations present those results:

1/4 Fracture Length Calculation

$s_x = 264\ Q/T \log 0.3\ T\ t / x^2 S$

s_x = drawdown (ft)   Q = pumping rate (gpm)
x = distance to fracture (d) + 1/4 L
r = radial distance to pumping well
L = fracture length   t = time (days)
T = Transmissivity (gpd/ft)
S = Storage Coefficient (dimensionless)
Aquifer Constants (T&S) from:
On-site test

Q= 225 gpm   t = 3.0 d (72 hr)

Ground Water Model

Powell Pumping Well

T = 2000 gpd/ft   S = 0.00007   s = 243 ft

$s_x = 264\ (225)/2000 \log (0.3)\ (2000)\ (3.0) / x^2\ 0.00007 = 29.7 \log 25{,}714{,}286 / x^2\ x(s_0) = 5071$ ft

$s_1 = 220$ ft, L (1/4) = 1 ft, L = 4 ft, radial flow at r = 10 ft

Match indicates fracture length very short and insignificant.

Observation Well (Mihm's Nursery irrigation well)

T = 2000 gpd/ft, S = 0.00007, s = 50 ft, r = 950 ft

$s_x = 264\ (225)/2000 \log (0.3)\ (2000)\ (3.0) / x^2\ 0.00007 = 29.7 \log 25{,}714{,}286 / x^2\ x(s_0) = 5071$ ft

$s_{730} = 50$ ft, d = x -L (1/4) = 7300-1 = 729 ft, r = x + L = 730+4 = 734 vs 950 actual
$s_{950} = 43$ ft, error = 14%

Other Observation Wells

Owner	Observed D/D		Projected. D/D	
Cahoon Production well	r = 1600 feet	s = 32 ft	s_{1600} = 30 ft	error = 6%
Hartz domestic well	r = 1100 feet	s = 32 ft	s_{1100} = 39 ft	error = -22%
Hughes Road well #10	r = 4625 feet	s = 3 ft	s_{4625} = 2.4 ft	error = 20%

To determine the potential impacts of withdrawals from wells 9 and 10, the drawdowns over a 90-d period with no recharge were simulated while pumping well 9 at the maximum permitted use by the following calculations:

T = 2000 gpd/ft; S = 0.00007, t = 90 days & Q =129 gpm from well 9

$s_x = 17.0 \log 771{,}428{,}570 / x^2$

Owner	Projected. D/D
Mihm nursery well	s_{950} = 50 ft
Hartz domestic well	s_{1100} = 48 ft
Cahoon Production Well	s_{1600} = 42 ft
Hughes Road well #10	s_{4625} = 26 ft

In August 2007, MDE received complaints that five house wells in the Sugarland Forest community had problems associated with low water pressure, turbidity or the well went dry. An investigation was conducted by MDE to determine if those problems were due to withdrawals from Poolesville's wells 9 and 10 and the nearby Poolesville golf course irrigation well. The results of that investigation are included in the following discussion.

A considerable amount of data is available concerning the Hughes/Budd Road domestic wells and most of the important information has been complied in Table 6. Most of the wells were monitored with automatic water level recorders during the 65-h, 50/60-gpm test of the proposed Bachelor's Purchase municipal water supply well, in August 1999. The test was secured after 65 hours due to the imposition of statewide water restrictions during the 1999 drought. At the end of the test, there were drawdowns of 2 to 23 feet in the domestic wells, at distances of 300 to 3000 feet from the test well. In addition, the Wilkens well went dry during the last few hours of that test and immediately, but only partially, recovered after pumping of the test well ceased. During the May-June 2001 individual tests of wells 9 (225 gpm) and 10 (100 gpm), drawdowns of about 3 feet were measured in the Bachelor's Purchase well, at an average distance of about 4000 feet from the pumping wells. In addition, Mr. Wilkens provided water level measurements that indicated there was about 3 feet of drawdown in his well, located about 5000 feet from the Rabanales well (well 11), during a 200-gpm, 72-h test of that well, in October 2001.

Table 6. Information obtained from domestic water supplies along Budd/Hughes Road during the 1999 aquifer test of the Bachelor's Purchase well.

								1999 Evaluation			
Name	Address	Depth**		Yield	Pump	Completion	Permit	well failed/	MDE risk	Present	Remarks
		record	report	gpm	Type	Date	Number	decreased*	assessment	Problems	
Kohlhoss	19400 Willis Lane	125		30	Sub	6/11/1974	MO-73-0489		Mod/low	None	
Wilkens	16101 Hughes Rd	155	148	10	Jet	8/28/1959	MO-03-5552	NO/NO	High	Dry	Replace/Repair
											(Check Pump 1st)
Davies	16015 Hughes Rd		80		Jet	1960's		NO/NO	High	Yield	Replace/Repair
Padayachee	16001 Hughes Rd		120	16	Jet	1949		YES/YES	Mod/Hi	Yield	Replace/Repair
(Dr. Smith)										Turbidity	
Hsu	15821 Hughes Rd	160	160	6	Sub	1/26/1967	MO-67-W149	NO/NO	Mod/low	None	
(Bacon)											
MacGregor	15811 Hughes Rd		70		Sub				High	Yield	Replace/Repair
(Hall)											
Morningstar	15801 Hughes Rd	143	95	3-8	Sub	7/24/1967	MO-67-0304	NO/NO	D=95 Hi	None	
									D=143 Mod		
Taylor	15711 Hughes Rd	90	95	4	Sub	7/12/1965	MO-65-W618	NO/NO	Mod/Hi	Yield	Replace/Repair
Bandy	15800 Budd Road	175	75	10	Sub	6/11/1977	MO-73-1549	NO/NO	Mod/low	None	
N.Smith/Reed	16020 Budd Road		99		Sub	1965		NO/NO	Mod/Hi	Yield	Replace/Repair
Parrish	16030 Budd Road	100	200		Sub	11/4/1964	MO-65-W235	NO/NO	D=200 low	Yield	Replace/Repair
(Lawson)											(Check Depth 1st)
R. Jameson***	16410 Budd Road	97?	80		Jet	1940?	MGS DC-11?	NO/NO	Unk	Yield	Replace/Repair
Hansen/Gilmore	14921 Sugarland Rd	****								None	

* - From consultant's water supply inventory prior to 1999 aquifer test of the Bachelor's Purchase well
** - Record-depth on record at County Health Department; Report-depth reported by homeowner or consultant
*** - Well located 1000+/- ft from well 10, 2000+/- ft from well 9, and 6000 ft from the golf course well
**** - Well located 2 1/2 miles southeast of the golf course well and 2 1/2 miles south of wells 9 and 10

Figure 27 is a chart of water levels in representative monitoring wells (Bachelor's Purchase, Davies, Morningstar, and USGS 50W 4C at Leesburg) during the period from late 2002 to late 2007. Except for an initial 45-day period, wells 9 and 10 were relatively under-utilized until 2007. Table 7 provides water use data from wells 9 and 10 and the golf course irrigation well, and water level data from the Bachelor's Purchase, Morningstar and USGS 50W-4C wells, during five periods when the municipal wells and one period when the golf course were pumped at rates that produced measurable drawdowns in the monitoring wells. The data indicate that the drawdowns in the house wells initially lagged, but then approached those of the deeper Bachelor Purchase well, responses that were likely due to leaky aquifer effects.

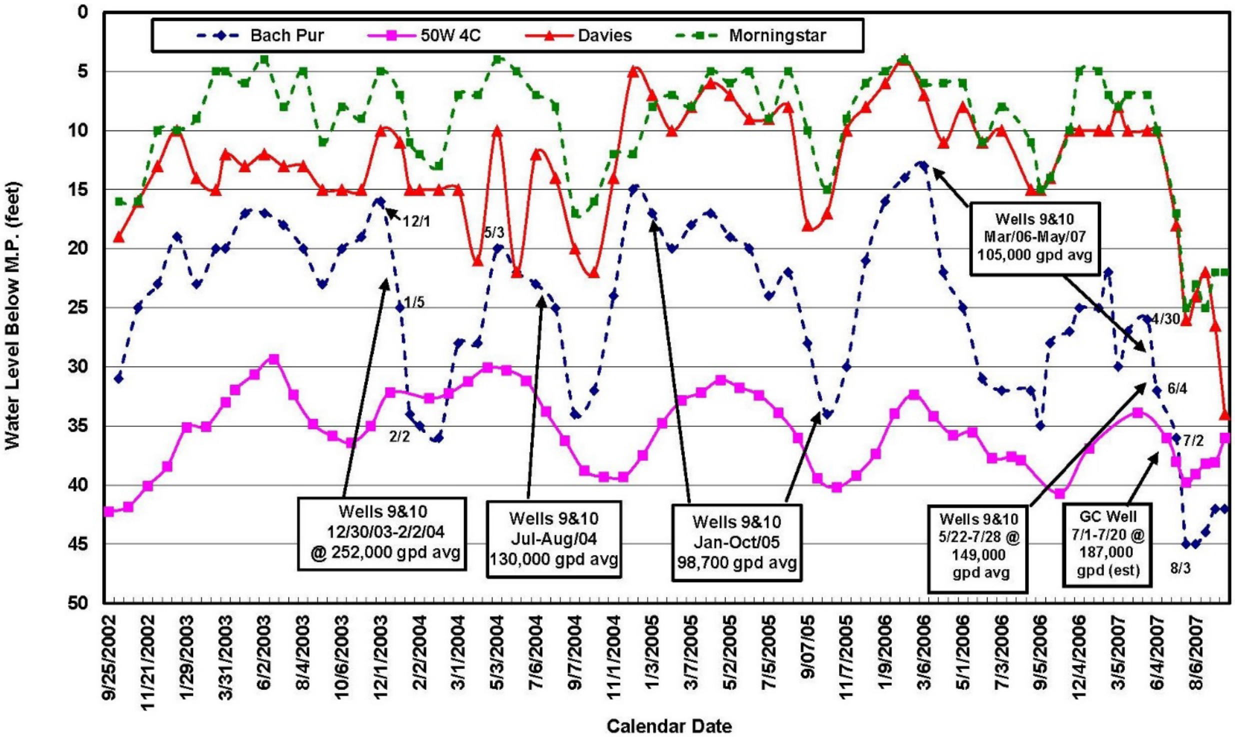


Figure 22. Water level data from the Bachelor's Purchase well, the Davies and Morningstar house wells, and the USGS monitoring well 50W 4C in Leesburg from 9/25/2002 to 8/27/2007. The average withdrawals for various periods from Poolesville wells 9 and 10 and the golf course well are indicated in the text boxes.

Table 7. Selected water use data from Poolesville wells 9 and 10, and the Poolesville golf course irrigation well, and water levels from the Bachelor's Purchase well, the Morningstar house well, and USGS monitoring well 50W 4C.

Period	dates	Δ water level USGS 50W 4C feet	Δ water level Bachelor's Purchase feet	Drawdown Bachelor's Purchase feet	Δ water level Morningstar feet	Drawdown Morningstar feet	Pumping rate Wells 9 & 10 gpd	Pumping rate GC well gpd
1	12/30/03-2/2/04	0	19	19	7	7	252,000	
2	Jul-Aug/04	7	9	2	9	2	130,000	
3	Jan-Oct/05	5	17	12	7.5	2.5	99,000	
4	Mar/06-May/07	3	16	13	6	3	105,000	
5	5/22-7/28/07	5	16	11	15	10	149,000	
	7/1-7/20/07							130,000
4 & 5	Mar/06-7/28/07	7.5	32	24.5	21	13.5	116,000	

During the last period starting July 1, 2007, drawdowns in the house wells were also affected by withdrawals from the Poolesville Golf Course irrigation well. Based on information provided by the golf course superintendent, Poolesville's town manager, and several citizens, it appears that the golf course used the irrigation well for about three weeks, starting on the first of July, due to a low level in the golf course's primary surface water supply. Pumping ceased when the generator used to operate the well malfunctioned. There was no meter on the well at that time; however, one was installed later which indicated that the irrigation well produced an average of 84.5 gpm during the period 9/14/2007 to 10/2/2007. The maximum reported use was 130,000 gpd during September 2015 or an average of 90 gpm, so it is assumed that the well was pumped at that rate during the 20 days in July 2007. A 72-h, 130 gpm aquifer test was conducted in December 2001, during a severe drought, while pumping the golf course irrigation well and monitoring the golf course cart barn well. The following is a simulation of the drawdown observed during that test in the monitoring well as well as predicted drawdowns in Poolesville's well 8, and the Bachelor's Purchase and Morningstar wells:

Constants are T = 2500 gpd/ft; S = 0.00015
For T-D-D calculation t = 3 days & Q = 130 gpm

$s_x = 13.7 \log 2250 / x^2 S$

Owner	Projected D/D	
Cart Barn	$s_{1900} = 9.1$ ft	s = 9.2 ft, error = 1%
Poolesville well 8	$s_{3185} = 3$	
Morningstar Hughes Road	$s_{4700} = 0$ ft	
Bachelor's Purchase (old #10)	$s_{3480} = 2$ ft	

There was 1% error in the predicted drawdown in the cart barn monitoring well and negligible predicted drawdowns in the other three wells.

Prior to bringing wells 9and 10 online, there were substantial recoveries of 12 feet from the end of the drought (9/25/2002) to the following wet period (6/2/2003) in the Bachelor's Purchase, Morningstar and USGS 50W 4C wells. The recovery was less (7 feet) in the Davies well; however, the water level measurements in that well were erratic and may have been affected by domestic water use from the well.

Poolesville's wells 9 and 10 were placed in service on December 30, 2003, and then pumped 24-h/d for 35 days at an average of 252,000 gpd, while conducting a groundwater under the direct influence (GWUDI) of surface water test. During that period, the water levels in the domestic wells declined by 4-7 feet and by 19 feet in the Bachelor's Purchase well. Withdrawals from the municipal wells then ceased and it took at least three months for the water levels to recover in the domestic wells, while the Bachelor's Purchase well only recovered about 75% during the same period. These slow responses were probably due to depleted storage, which required substantial recharge to effect full water level recoveries. The following are simulations of the drawdowns observed during that 35-d period made using an iterative process by varying the T and S values until reasonable fits (highlighted in red) to the observed drawdown data were achieved:

Estimated T = 2000 gpd/ft
For T-D-D calculation t = 35 days (12/30/03 to 02/02/04) & Q =175 gpm are assumed
Distances weighted based on a yield ratio of ratio 2:1 (Well 9: Well 10)
<u>S = 0.00007</u>
$s_x = 23.1 \log 300{,}000{,}000 / x^2$

Bachelor's Purchase municipal obs well s_{4156} = 29 ft, actual 19 ft, error = -34%
Hughes Road Morningstar house well s_{6332} = 20 ft, actual 7 ft, error = -65%

<u>S = 0.00035</u>
$s_x = 23.1 \log 60{,}000{,}000 / x^2$

Bachelor's Purchase municipal obs well s_{4156} = 12 ft, actual 19 ft, error = 58%
Hughes Road Morningstar house well s_{6332} = 4 ft, actual 7 ft, error = 75%

<u>S = 0.0002</u>
$s_x = 23.1 \log 105{,}000{,}000 / x^2$

Bachelor's Purchase municipal obs well s_{4156} = 18.1 ft, actual 19 ft error = 5%
Hughes Road Morningstar house well s_{6332} = 9.7 ft, actual 7 ft error = -28%

Estimated T = 1500 gpd/ft
For T-D-D calculation t = 35 days & Q =175 gpm are assumed
S = 0.00007
s_x = 30.8 log 225,000,000 / x^2

Bachelor's Purchase municipal obs well s_{4156} = 34 ft, actual 19 ft error = -44%
Hughes Road Morningstar house well s_{6332} = 23 ft, actual 7 ft error = -70%

S = 0.0002
s_x = 30.8 log 78,750,000 / x^2

Bachelor's Purchase municipal obs well s_{4156} = 20 ft, actual 19 ft error = -5%
Hughes Road Morningstar house well s_{6332} = 9.0 ft, actual 7 ft error = -22%

S = 0.00022
s_x = 30.8 log 71,590,909 / x^2 - Best Fit

Bachelor's Purchase municipal obs well s_{4156} = 19 ft, actual 19 ft, error = 0%
Hughes Road Morningstar house well s_{6332} = 7.8 ft, actual 7 ft, error = -10%

When the hydraulic constants derived from the aquifer test of well 9 (T =2000 gpd/ft and S = 0.0007) were applied, the drawdowns in the Bachelor's Purchase and Morningstar wells were over-estimated by 34% and 65%, respectively. The best fit to the drawdown data was achieved when it was assumed that the T = 1500 gpd/ft and the S = 0.00022, producing 0% error in the Bachelor's Purchase well and 10% error in the Morningstar well. Şen (1992) indicated that fracture length can affect the storage constant by as much as an order of magnitude. In this case, it is likely that the actual S value is 3 times greater than the calculated value. Changes in transmissivity may be possible due to dewatering of semi-confined, permeable reservoir units which would tend to lead to a decrease in T or by inflow from areas with different hydraulic properties, as the trough of depression expands during long-term withdrawals. To see if this were the case; data from nearly all the aquifer tests conducted in the Town of Poolesville were analyzed, Table 8. The isolines of T values on Figure 28 provide a reason for the difference in the T values between the aquifer test and the long-term effects of pumping. During the aquifer test, water was withdrawn from an area of higher T near the pumping well. With long-term use, additional water is then supplied from further areas with lower T values, reducing the overall average T of the aquifer in the capture zone of the pumping wells.

Table 8a. The results of multi-well aquifer tests of Poolesville public water supply wells 6, 9 & 10.

MULTI-WELL TESTS														
Test-Obs Well	Depth	Mwbz	T	S	r	s	t	Model	DERIV	Data Points	RSS	Var	S.D.	Mean
Well 6-225 gpm	500	180&300	771	-	0	139.2	2460	Moench3	IARF / Leaky?	84	0.001	2E-05	0.004	-2E-04
			1039	-				Barker			0.013	2E-04	0.012	-7E-04
Koteen	275	N/R	1149	1E-05	1800	69.5	2460	Moench3	IARF	84	26.7	0.33	0.58	0.063
			1437	1E-05				Barker			23.3	0.29	0.54	0.018
Brooks	N/R	N/R	506	1E-05	1400	82.2	2000	Moench3	W-B Stor Frac D/W Erratic	58	145	2.69	1.64	-0.05
			248	4E-06				Barker			135	2.46	1.57	0.009
Well 9 (Powell)-225 gpm	800	230-435	954	-	0	189	600	Moench2P	Dual Porosity	50	57.2	1.24	1.12	0.056
	800	230-435	375	-	0			Moench3			210	4.56	2.14	0.63
Well 10 (Cahoon)	762	560-600	2117	5E-05	1300	32.2	4320	Moench3	Leaky	4320	101	0.023	0.15	0.012
Hartz	>120	N/R	1246	3E-05	1250	30.0	3001	Moench3	Leaky	2713	7.4	0.003	0.052	0.003
Bernsdorff	145	N/R	2160	1E-04	1850	11.1	2688	Moench3	Leaky?	1361	4.95	0.005	0.068	-0.001
			3250	2E-04				SVF			22.4	0.021	0.146	-0.017
Mihm AG	N/R	N/R	1737	7E-05	550	25.6	376	Moench3	W-B Stor Linear	376	42.9	0.115	0.34	-0.051
								Barker			27.8	0.076	0.276	0.027
								SVF			22.8	0.063	0.251	0.022
Well 10 (Cahoon)-80 gpm	762	560-600	154	-	0	319.4	3720	Moench3	Leaky Const HD	101	726	7.48	2.74	0.167
Well 9 (Powell)	800	230-435	2833	4E-05	1300	4.0	420	Moench3		420	0.4	7E-04	0.026	0.003
Hartz	>120	N/R	1985	9E-06	2250	5.7	500	Moench3	IARF Frac D/W	100	0.176	0.002	0.043	0.013
			2699	1E-05				Barker			0.066	7E-04	0.026	0.004
Bernsdorff	145	N/R	1308	4E-05	1400	12.8	3405	Moench3	Leaky	346	1.75	0.005	0.071	-0.009

Table 8b. The results of multi-well aquifer tests of Poolesville public water supply wells 7 & 11, and the Bachelor's Purchase and Golf Course wells.

MULTI-WELL TESTS														
Test-Obs Well	Depth	Mwbz	T	S	r	s	t	Model	DERIV	Data Points	RSS	Var	S.D.	Mean
Bachelors Purchase Well-50 gpm	>240	220-240	560	-	0	80.2	1100	Moench3	Leaky	29	6.16	0.246	0.496	-0.034
Wilkins	148	N/R	446	1E-04	700	12.6	1530	Moench3	W-B Stor Leaky Closed	765	23.3	0.031	0.175	0.026
			497	1E-04				Moench2			19.6	0.026	0.16	0.019
			594	1E-04				Papa-Coop			21.7	0.028	0.169	0.024
Golf Course Well-130 gpm	400	170-322	447	-	0	123	4320	Moench3	IARF Frac D/W	112	316	2.92	1.71	0.124
			2155			68	60	Barker		26	89	3.88	1.97	0.157
Cart Barn Well	N/R	N/R	2014	1.E-04	1900	9.2	2184	Moench3	W-B Stor Leaky Frac D/W	170	1.75	0.01	0.102	0.027
			2836	1.E-04				Moench2			7.92	0.023	0.156	0.02
			3746	1.E-04				Papa-Coop			1.54	0.009	0.095	0.013
Well 7-50 gpm	700	430	135	-	0	230	2040	Moench3	Leaky	74	3.65	0.051	0	0.029
Well 7 (corrected drawdown)	B = 289 ft		272	-	0	164	2040	Moench3	Leaky	74	1.21	0.017	0	-2E-04
Burnt House	N/R	N/R	292	3E-04	525	12.7	4320	Moench3	IARF	15	1.27	0.116	0.34	0.038
			882	5E-04				Barker			1.92	0.148	0.384	-0.043
			1043	4E-04				SVF			1.94	0.177	0.42	-0.042
Bernsdorff	145	N/R	202	4E-05	1100	4.6	4320	Moench3	Linear	38	0.057	0.002	0.041	-0.001
			1302	5E-04				Barker			0.094	0.003	0.052	0.007
			1296	5E-04				SVF			0.097	0.003	0.053	0.015
Well 11 (Rabanales) Well	1200	600-900	282	-	0	390	400	Moench3	Leaky	54	1260	25.2	5.02	-0.35
Well 9 (Powell)	800	230-435	1691	2E-05	1320	37.5	4274	Moench3	Erratic	9	9.06	1.81	1.35	0.034
Well 10 (Cahoon)	762	560-600	510	4E-07	2010	15	4284	Moench3	Leaky?	8	10.35	2.59	1.61	0.031
Hartz	>120	N/R	1580	2E-05	2400	12.9	3501	Moench3	Leaky	8	0.04	0.01	0.101	-0.005
Bernsdorff	145	N/R	728	6E-05	3000	4.5	3490	Moench3	Leaky?	7	0.115	0.038	0.196	0.037

Table 8c. The results of single well aquifer tests of various Poolesville public water supply wells.

Test-Obs Well	Depth	Mwbz	T	S	r	s	t	Model	DERIV	Data Points	RSS	Var	S.D.	Mean
Well 2 - 100 gpm	600	224	1250	-	0	85	642	Barker	2poro Frac D/W	25	189	8.21	2.87	0.274
			1218	-	0			Moench2P			150	7.14	2.67	0.218
			293	-	0			Moench3			152	7.25	2.69	0.288
Well 3 - 100 gpm	285	203	2974	-	0	51	4400	Barker	2P?	61	247	4.18	2.04	0.047
			2116	-	0			Moench2P	2P?		144	2.53	1.59	-0.007
			811	-	0			Moench3	2P?		480	8.42	2.9	0.415
Well 4 - 50 gpm	600	228	471	-	0	67	0-3120	Moench3	Leaky	34	27.7	0.92	0.96	0.074
			311	-	0	116	3120-10800	Barker	Leaky?	36	72	2.12	1.46	8E-04
Well 5 - 120 gpm	300	345	1050	-	0	108	900	Moench3	Leaky	48	199	4.51	2.13	0.234
Well 8 - 80 gpm	500	217	341	-	0	170	4320	Moench3	Leaky	29	122	4.88	2.21	0.231
Well 13 (Elgin) - 100 gpm	500	155	489	-	0	104	300	Moench3	Leaky	42	149	3.91	1.98	-0.368
Well 14 (Jamison) - 50 gpm	N/R	N/R	205	-	0	132	1000	Moench3	Leaky	50	52	1.12	1.06	0.116
Well 12 (Schraf) - 175 gpm	466	1408&233	198	-	0	96	200	Moench	W-B Stor? Linear?	200	897	4.57	2.14	-0.003
			1354	-	0	103	270	SVF		270	665	2.5	1.58	-0.017
			1367	-	0	103	270	Barker			638	2.38	1.54	-0.025
Schraf Monitor Well	~700	N/R	32	1E-08	1100	26	126	Moench		126	2.8	0.023	0.151	0.003
			1313	1E-08		26	126	SVF			27	0.22	0.47	0.113

Table 8d. The results of step tests of various Poolesville public water supply wells and the Poolesville Golf Course irrigation well

STEP TESTS (1st Step)														
Test-Obs Well	Depth	Mwbz	T	S	r	s	t	Model	DERIV	Data Points	RSS	Var	S.D.	Mean
Well 11 (Rabanales) - 150 gpm	1200	600-900	452	-	0	229	150	Moench3	Leaky	35	406	13.1	3.62	-0.414
Well 10 (Cahoon)I - 75 gpm	762	560-600	177	-	0	192	120	Barker	IARF	37	399	11.7	3.43	0.836
			7	-	0			Moench3			332	10.1	3.17	0.641
			169	-	0			SVF			85	2.58	1.61	0.234
Well 9 (Powell) - 150 gpm	800	230-435	2101	-	0	78	60	Barker	2Poro	26	46	1.94	1.39	0.1
			831	-	0			Moench3			35	1.57	1.25	0.517
			2201	-	0			Moench2P			61	2.78	1.67	0.152
Well 8 - 60 gpm	500	217	1705	-	0	88	60	Barker	IARF	7	1849	462	21.5	5.75
Well 14 (Jamison) - 25 gpm	N/R	N/R	395	-	0	36	120	Barker	IARF	20	1.72	0.095	0.309	0.028
			12	-	0			Moench3			11.5	0.719	0.848	0.215
Golf Course - 80 gpm	400	170-322	1326	-	0	48	70	Barker	Erratic	18	6.16	0.385	0.62	8E-04
			601	-	0			Moench3			6.91	0.494	0.703	-0.015
			1027	-	0			Moench2P			5.82	0.416	0.645	2E-04
Well 13 (Elgin) - 25 gpm	500	155	579	-	0	21	60	Moench3	Leaky	25	4.32	0.206	0.454	-0.052
Well 12 (Schraf) - 100 gpm	~700	N/R	1302	-	0	45	120	Barker	Leaky?	25	9.62	0.418	0.649	-0.013
			749	-	0			Moench3			8.14	0.388	0.622	-0.009

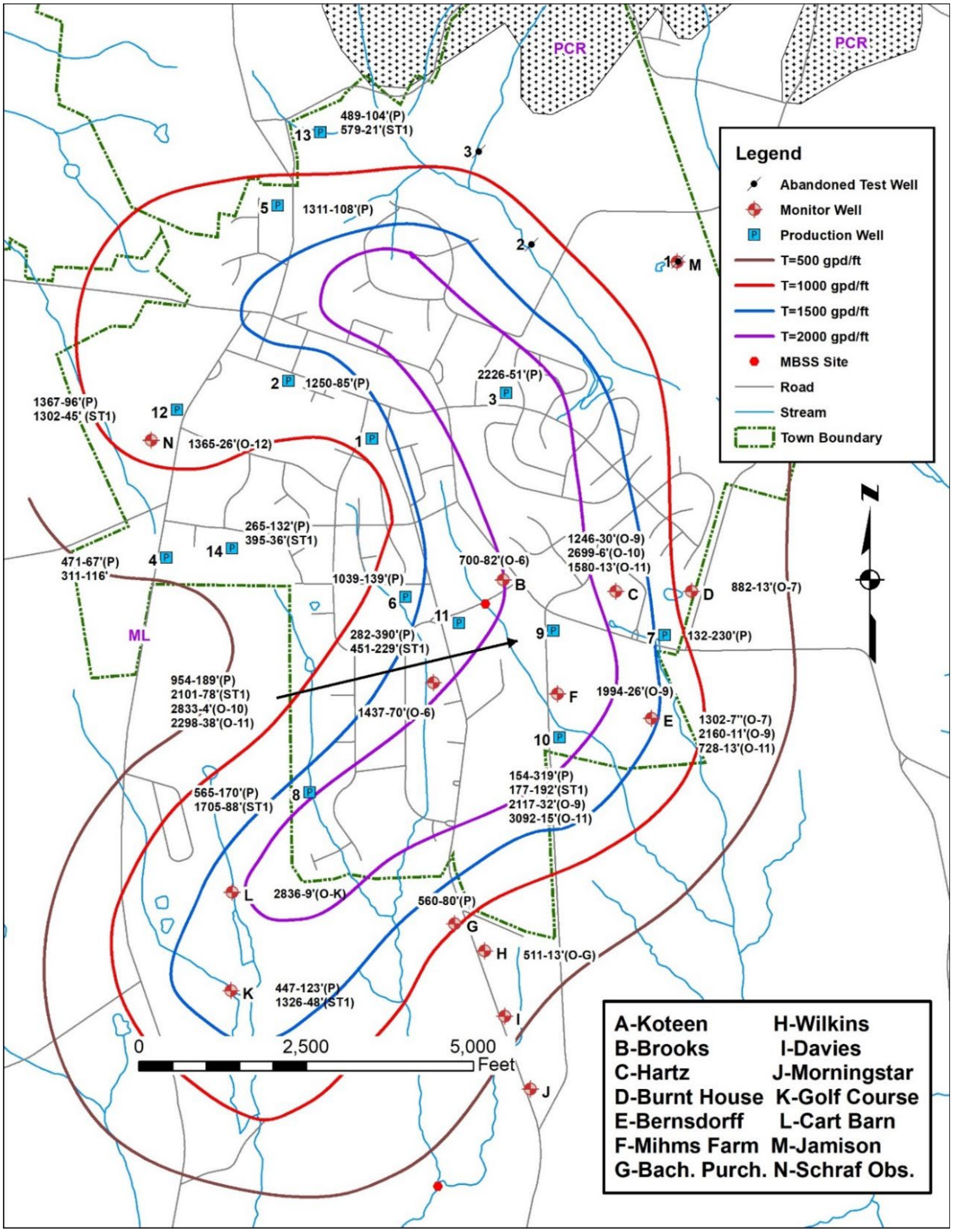


Figure 28. Map of transmissivity isolines derived from various aquifer tests in the Town of Poolesville.

During the period 4/30 to 8/3/2007, wells 9 and 10 were pumped at 160,000 gpd for 4 months, while the golf course irrigation well was pumped at an estimated 130,000 gpd for 20 days near the end of the period. In that case, the drawdown in the Bachelor's purchase well was only about 1 foot more than in the Morningstar well. Although both observation wells are approximately equidistant from the golf course well, pumping of the irrigation well had a much greater influence on the Morningstar well. This suggests that an anisotropic aquifer exits in the vicinity of the golf course and Sugarland Forest community. Due to heterogeneity, anisotropy, and leakage effects, a numerical model would best demonstrate the long-term effects caused by withdrawals from wells 9 and 10; however, the golf course well was only pumped for about 20 days, so a simple, analytical model might be used to estimate the impacts due the use from the irrigation well. The model applied the previously derived hydraulic constants (T = 2500 gpd/ft and S = 0.0015) from the 72-h, 130-gpm test which produced 1% error relative to the measured drawdowns in the cart barn monitoring well and negligible predicted drawdowns in the Morningstar and Bachelor's Purchase wells, and Poolesville well 8:

During the period 4/30 to 8/3/2007, the golf course irrigation well was pumped at an estimated 130,000 gpd for 20 days near the end of the period. The analytical solution by iterative methods for the drawdowns observed in nearby wells follows:

Initial Constants are T = 2500 gpd/ft; S = 0.00015
For T-D-D calculation t = 20 days & Q = 90 gpm are assumed

$s_x = 9.5 \log 15000 / x^2 S$

Owner	Projected D/D	
Cart Barn	s_{1900} = 13.7 ft, no water level measurements	
Poolesville well 8	s_{3185} = 9.4 ft, no water level measurements	
Morningstar Hughes Road	s_{4700} = 6.2 ft	s = 8 ft, error = 29%
Bachelor's Purchase (old #10)	s_{3480} = 8.7 ft	s = 10 ft, error = 15%

Constants are T = 2500 gpd/ft; S = 0.0001 (aquifer test)
For T-D-D calculation t = 20 days & Q = 90 gpm are assumed

$s_x = 9.5 \log 15000 / x^2 S$

Owner	Projected D/D	
Cart Barn	s_{1900} = 15 ft	
Poolesville well 8	s_{3185} = 11 ft	
Morningstar Hughes Road	s_{4700} = 7.9 ft	s = 8 ft, error = 1%
Bachelor's Purchase (old #10)	s_{3480} = 10.4 ft	s = 10 ft, error = -4%

In this case, the aquifer constants from the aquifer test produced the best results, indicating that relatively small changes in the storage constant can produce significant errors in predicted drawdowns.

Next, a simulation (2a) was performed for the period 5/22/2007 to 7/28/2007 (68 days) while pumping wells 9 and 10 at an average of 103.5 gpm, Table 9. The calculated drawdowns were combined with those from a simulation (2b) for the period 7/1/2007 to 7/20/2007 while pumping the golf course well at an average of 90 gpm. The results overestimated the drawdown in the Bachelor's Purchase well by 30% and by 6% in the Morningstar (Hughes Road) well. Finally, a simulation (2c) was performed for the period March 2006 to 7/28/2007 (478 days) while pumping wells 9 and 10 at an average of 76.6 gpm. The calculated drawdowns were combined with those from the simulation (2b) for the period 7/1/2007 to 7/20/2007 while pumping the golf course well at an average of 90 gpm. The results overestimated the drawdown in the Bachelor's Purchase well by 28% and by 50% in the Morningstar well. In this case, it is possible that recharge may have caused the less than predicted drawdowns.

The Poolesville Source Water Assessment Plan (SWAP), Yoxtheimer (2006), includes a map of water levels measured on 4-5 May 2005 in the town wells and nearby private wells, while Town wells 3-9 were in service, Figure 29. When the water level contours are compared with the T value isolines, Figure 30, there is a fair correlation between the two features. For example, the 300 ft water level contour generally follows the 1500 gpd/ft transmissivity isoline. Where there are differences may be explained as follows. The water level map does not include well 12, which was not completed at that time, well 2 or the golf course irrigation well, all which occur in areas with relatively high T values. The is no well control for water level measurements southwest of town well 8. It is then possible that the water level contour lines could be extended in that direction to more closely match the T value isolines. Finally, there is a limited amount of aquifer test data to the northeast of town well 3, which could lead to an adjustment of the T isoline values in that direction. The SWAP and the T value map in the present study were developed independently and the relatively good match between the water level contours and T value isolines provides a validation of the aquifer test analyses and the T values map.

Table 9. Analytical simulations of drawdowns caused by withdrawals from Poolesville wells 9 & 10 and the Poolesville Golf Course irrigation well.

Simulation 2A for period 5/22/2007 to 7/28/2007 (wells 9 and 10 pumping)
Withdrawal point - weighted distance between wells 9 and 10
Drawdown - weighted use between wells 9 and 10

T	S	Q	t	x	x = 1/4L	Drawdowns	
gpd/ft		gpm	day	ft		Well	ft
1500	2.2E-04	103.5	68	0.25	L=1ft	Wells 9 & 10	170
1500	2.2E-04	103.5	68	4156		B.P. Well	
1500	2.2E-04	103.5	68	6332		Morningstar Well	
S_x =	264Q/T X	Log $0.3Tt/x^2S$	=	S_x	error %		
S_7 =	18.2	9.35	=	170	0	Wells 9 & 10	170
S_{4156} =	18.2	0.91	=	17	-	B.P. Well	-
S_{6332} =	18.2	0.54	=	10	-	Morningstar Well	-

Simulation 2B for period 7/1/2007 to 7/20/2007 (Golf course well pumping)

T	S	Q	t	x	x = 1/4L	Drawdowns	
gpd/ft		gpm	day	ft		Well	ft
2500	1.0E-04	90	20	1	-	GC Well	85(est)
2500	1.0E-04	90	20	3480	-	B.P. Well	-
2500	1.0E-04	90	20	4700	-	Morningstar Well	-
S_x =	264Q/T X	Log $0.3Tt/x^2S$	=	S_x	error %		
S_1 =	9.5	8.18	=	78	9	GC Well	85(est)
S_{3480} =	9.5	1.09	=	10	-	B.P. Well	-
S_{4700} =	9.5	0.83	=	8	-	Morningstar Well	-

Simulation 2C for period Mar/06 to 7/28/2007 (wells 9 and 10 pumping)
Withdrawal point - weighted distance between wells 9 and 10
Drawdown - weighted use between wells 9 and 10

T	S	Q	t	x	x = 1/4L	Drawdowns	
gpd/ft		gpm	day	ft		Well	ft
1500	2.2E-04	76.7	478	0.25	L=1ft	Wells 9 & 10	170
1500	2.2E-04	76.7	478	4156		B.P. Well	
1500	2.2E-04	76.7	478	6332		Morningstar Well	
S_x =	264Q/T X	Log $0.3Tt/x^2S$	=	S_x	error %		
S_7 =	13.5	10.19	=	138		Wells 9 & 10	170
S_{4156} =	13.5	1.75	=	24	-	B.P. Well	-
S_{6332} =	13.5	1.39	=	19	-	Morningstar Well	-

Total drawdown Simulations 2A and 2B		
B.P. (Bachelors Purchase) well	27 feet simulated vs 19 feet actual	
Morningstar residential well	18 feet simulated vs 17 feet actual	
Total drawdown Simulations 2B and 2C		
B.P. (Bachelors Purchase) well	34 feet simulated vs 24.5 feet actual	
Morningstar residential well	27 feet simulated vs 13.5 feet actual	
Estimated simulated error (period beginning May 2007)		
B.P. (Bachelors Purchase) well	-30%	
Morningstar residential well	-6%	
Estimated simulated error (period beginning March 2006)		
B.P. (Bachelors Purchase) well	-28%	Possible effects of recharge
Morningstar residential well	-50%	

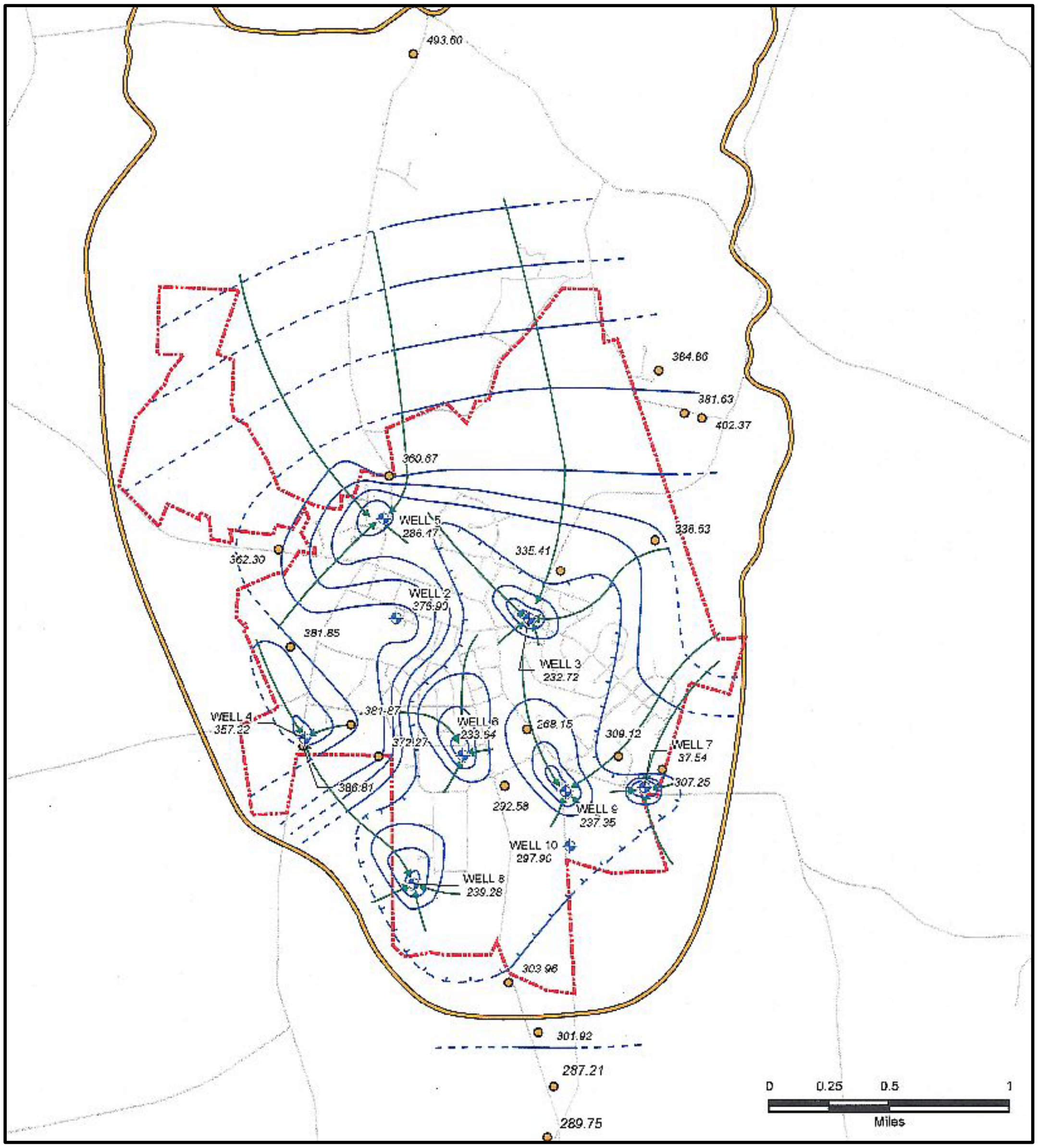


Figure 23. Poolesville wellhead protection area with groundwater level contours and well locations.

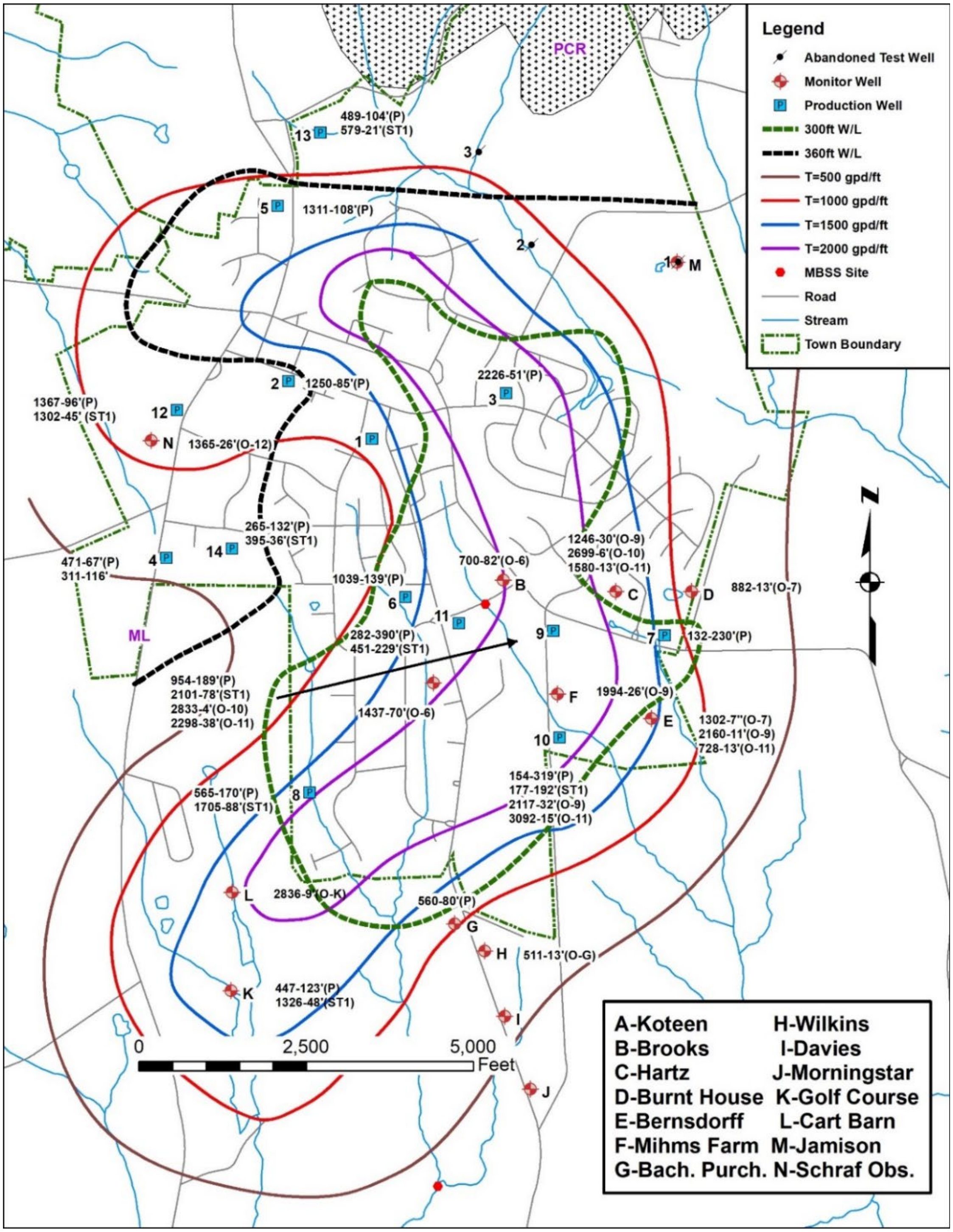


Figure 30. Poolesville study area with transmissivity isolines from various aquifer tests and representative groundwater level contours in the wellhead protection area.

Summary

More than 90% of the approximately 100 known interference impacts to domestic wells in the fractured rock aquifers of Maryland can be attributed to withdrawals by Poolesville and Taneytown municipal wells, and dewatering of the Mettiki Coalmine, all in consolidated sedimentary rock formations and dewatering of quarries in the state. The impacts to those domestic wells were successfully mitigated, mostly by drilling replacement wells, providing public water to affected homes, or by adjusting the withdrawals of the large users. Most of the projects included estimates of impacts made prior to withdrawals and post-audits to determine the reliabilities of those predictions. In addition to case studies of impacts associated with Taneytown well 14 and Poolesville wells 9 and 10, the results of aquifer testing and limited follow-on monitoring at Cloverhill III subdivision in Frederick County, all in the consolidated sedimentary rock New Oxford Formation, were presented in this investigation.

Cloverhill III Subdivision Case Study

The Coverhill III Subdivision was supplied with water from two wells in the New Oxford Formation during the period from 1988 to 2016. Separate aquifer tests (50 to 96 hours in length) were conducted in three different wells at the site with various combinations of four on-site and eight off-site observation wells monitored during the tests. This was first project where the State required extensive monitoring of offsite wells during aquifer testing. It is also the first known application of fracture flow models and where a numerical model was developed for evaluation of potential interference impacts.

During the 96-h variable rate step/aquifer test of well TW-2, the drawdowns were 70 ft in the pumping well, 61 ft in TW-1 located 209 ft SSW of TW-2 and 19 ft in the Church Well located 1800 ft SW of TW-2. No response was noted in the remaining four observation wells. During the 49.5 variable rate step-aquifer test of well TW-3, an adjacent spring went dry during the test and no response was noted in the five observation wells. During the 72-hr variable-rate test of the production well, there was no response in any of eight observation wells.

The ground water flow at the site could be controlled by increased permeability along tension fractures caused by folding or along bedding plane parting. The dip of any fractures caused by folding would be nearly vertical along a NE-SW axis. A geologic map of the area indicates that bedding planes dip about 20 degrees to the NW.

A fracture trace map indicates that there is a prominent fracture near the Church Well that extends about 3000 feet to the NE and intersects several fractures that pass near TW-2. This might explain the 19-foot drawdown noted in the Church Well when pumping TW-2. The fracture map also indicates that the production well is located near the intersection of two fractures that could connect that well to TW-1, TW-2, TW-3 and the Church Well; however, no response was noted in any of these wells during the pumping test of the production well. Based on these responses, it appears that the fractures traced on the map did not control the direction of ground water flow at the Cloverhill III site.

A cross-sectional diagram indicates that bedding plane parting control of ground water flow could explain the water level response of all the observation wells, except that of TW-3 during the test of the production well. At the 20° bedding plane dip angle of the producing zone, it is possible that an effectively confined water-bearing zone in the production well would not intersect any of the observation wells. In addition, a NW-SE trending fault may hydraulically separate domestic wells near Walter Martz Road from the production well.

Bedding plane parting control of groundwater flow provides the best, but not conclusive, explanation of the observed data. All the available evidence, that of orientation of bedding plane and prominent fracture traces and observation well response to pumping, indicate that there is a preferred SW-NE direction for ground water flow at the site.

Certain other geologic features may have provided additional protection to other users of the resource. Some domestic wells were located perpendicular to the dominant SW-NE ground water flow trend and down gradient of the production wells. Others were too shallow to be affected by the bedding plane flow. The remaining ones were outside of the ground water drainage basin.

Taneytown Well 14 Case Study

A 72-hour aquifer test of Taneytown's well 14 was conducted in 1990. A permit was issued based on the estimated yield of that well (158 gpm). During the test, a resident along Fringer Road reported turbidity and low yield problems with his well.

In 1994, the Water Supply Program received reports residents along Fringer Road had been without water or had muddy water for several months, where previously there had been no reported problems. Well 14 was then retested while monitoring two residential wells (about 2500-3000 feet from well 14) along Fringer Road. During the 72-h, 230-gpm test, drawdowns (13 and 47 ft) observed in the two Fringer Road domestic wells were much greater than those in the observation wells (15 ft or less and within 325 feet of well 14) during the first test.

In 1997, Fringer Road residents were again having problems with their wells. The pumping rate of well 14 was reduced which restored the supply to the domestic wells; however, this was not a permanent solution to the problem, so a set of special permit conditions were developed to will ensure an adequate water supply for the Fringer Road residents.

In 2006, the city requested an increase in the appropriation, which required the replacement of certain domestic wells with deeper wells. The request was approved, the domestic wells were replaced, and the Water Supply Program has received no further complaints of impacts along Fringer Road.

Poolesville Wells 9 & 10 Case Study

Multi-rate and aquifer tests were performed on Poolesville wells 9 and 10 in 2001, under average climatic conditions. The estimated predicted drawdowns were 50 ft within 1000 ft of the town wells and 26 ft at 0.9 mi from the test wells at the requested withdrawal. Six domestic wells ¼ mile of wells 9 and 10 were replaced by the town and public water was supplied to a nursery. Monthly water level monitoring was required for four domestic wells and an inactive town well to determine if impacts would occur in the Sugarland Forest community, about one mile south of town wells 9 and 10.

In August 2007, MDE received complaints that five house wells in the Sugarland Forest community had problems associated with low water pressure, turbidity or the well went dry. MDE investigated to see if the impacts were due to withdrawals from Poolesville's wells 9 and 10 and the nearby Poolesville Golf Course irrigation well.

Water levels were measured in representative monitoring wells during the period from late 2002 to late 2007. Except for an initial 45-day period, wells 9 and 10 were relatively under-utilized until 2007, when drawdowns in the house wells were also affected by withdrawals from the golf course irrigation well. While there was no meter on the golf course well at that time; one was installed later that indicated that the maximum reported use was 130,000 gpd during September 2015 or an average of 90 gpm, so it is assumed that the well was pumped at the rate during the 20 days in July 2007. A 72-h, 130 gpm aquifer test had been conducted on the golf course irrigation well and the golf course cart barn monitoring well. A simulation of the drawdowns observed during that test produced an error of 1% when compared to the measured drawdown in the cart barn well and minimal drawdowns in Poolesville's well 8, and the Bachelor's Purchase and Morningstar wells.

Poolesville's wells 9 and 10 were placed in service on December 30, 2003 and were pumped 24-hr/d for 35 days at an average of 252,000 gpd. During that period, the water levels in the domestic wells declined by 4-7 feet and by 19 feet in the Bachelor's Purchase well. Withdrawals from the municipal wells then ceased and it took at least three months for the water levels to recover in the domestic wells, while the Bachelor's Purchase well only recovered about 75% during the same time. These slow responses were probably due to depleted storage, which required substantial recharge to effect full water level recoveries. A simulation using a T of 1500 gpd/ft and an S of 0.00022 for the 35-d period produced errors of 0% in the Bachelor's Purchase well and -10% in the Morningstar house well. The hydraulic constants (T = 2000 gpd/ft and S – 0007) derived from the aquifer test produced errors of 34% and 65% in the two monitoring wells. This is evidence that fracture length can affect the storage constant by as much as an order of magnitude or, in this case, the actual S value is likely 3 times greater than the value calculated from the aquifer test data. The changes in transmissivity may be due to either dewatering of semi-confined, permeable reservoir units which would tend to lead to a decrease in T or by inflow from areas with different hydraulic properties, due to the expansion of the trough of depression because of long-term withdrawals. An isoline map of T values provides an explanation for the difference in the T values between the aquifer test and the long-term effects of pumping. During the aquifer test, water was withdrawn from an area of higher T near the pumping well. With long-term use, additional water is then supplied from farther areas with lower T values, reducing the overall average T of the aquifer in the capture zone of the town's pumping wells.

During the period 4/30 to 8/3/2007, wells 9 and 10 were pumped at 160,000 gpd for 4 months, while the golf course irrigation well was pumped at an estimated 130,000 gpd for 20 days near the end of that period. An analytical solution using the hydraulic constants of T = 2500 gpd/ft and S = 0.00015 produced errors of 15 % and 29% in the Bachelor's Purchase and Morningstar wells, respectively, while a simulation using the constants (T = 2500 gpd/ft and S = 0.0001) derived from the aquifer test produced errors of 1% and -4%, indicating that relatively small changes in the storage constant can produce significant errors in predicted drawdowns.

A simulation was performed for the period 5/22/2007 to 7/28/2007 (68 days) while pumping wells 9 and 10 at an average of 103.5 gpm. The calculated drawdowns were combined with those from a simulation for the period 7/1/2007 to 7/20/2007 while pumping the golf course well at an average of 90 gpm. The results overestimated the drawdown in the Bachelor's Purchase well by 30% and by 6% in the Morningstar well. Finally, a simulation was performed for the period March 2006 to 7/28/2007 (478 days) while pumping wells 9 and 10 at an average of 76.6 gpm. The calculated drawdowns were combined with those from a simulation for the period 7/1/2007 to 7/20/2007 while pumping the golf course well at an average of 90 gpm. The results overestimated the drawdown in the Bachelor's Purchase well by 28% and by 50% in the Morningstar (Hughes Road) well. In that case, it is possible that recharge may have caused the less than predicted drawdowns.

The Poolesville SWAP includes a map of water levels measured on 4-5 May 2005 in the Town wells and nearby private wells, while Town wells 3-9 were in service. When the water level contours are compared with the T value isolines, there is a fair correlation between the two features. Where there are differences may be explained as follows. The water level map does not include well 12, which was not completed at that time, well 2 or the golf course irrigation well, all which occur in areas with relatively high T values. There was no well control for water level measurements southwest of Town well 8 and there is limited aquifer test data to the northeast of town well 3. The relatively good match between the water level contours and T value isolines provides a validation of the aquifer tests analyses.

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