Contract Report 2000-09

Dewatering Well Assessment for the Highway Drainage System at Four Sites in the East St. Louis Area, Illinois

(FY99 - Phase 16)

by

Mark A. Anliker and Robert D. Olson

**Prepared for the
Illinois Department of Transportation
Division of Highways**

June 2000

Illinois State Water Survey
Groundwater Section
Champaign, Illinois

A Division of the Illinois Department of Natural Resources

DEWATERING WELL ASSESSMENT FOR THE

HIGHWAY DRAINAGE SYSTEM

AT FOUR SITES IN THE EAST ST. LOUIS AREA, ILLINOIS

FY 99 (Phase 16)

by Mark A. Anliker and Robert D. Olson

Illinois State Water Survey
2204 Griffith Drive
Champaign, IL 61820-7495

Prepared for the
Illinois Department of Transportation
Division of Highways

June 2000

CONTENTS

CONTENTS (Concluded)

DEWATERING WELL ASSESSMENT FOR THE HIGHWAY DRAINAGE SYSTEM AT FOUR SITES IN THE EAST ST. LOUIS AREA, ILLINOIS, FY 99 (PHASE 16)

by Mark A. Anliker and Robert D. Olson

Abstract

In the East St. Louis vicinity, the Illinois Department of Transportation Division of Highways (IDOT) owns 55 high-capacity wells that are used to maintain the elevation of the ground-water table below the highway surface in areas in which the highways were constructed below the original land surface. The dewatering systems are located at five sites in the alluvial valley of the Mississippi River in an area known as the American Bottoms. The alluvial deposits at the dewatering sites are about 90 to 115 feet thick and consist of fine sand, silt, and clay in the upper 10 to 30 feet, underlain by about 70 to 100 feet of medium to coarse sand.

The condition and efficiency of a number of the dewatering wells became suspect in 1982 on the basis of data collected and reviewed by IDOT staff. Since 1983, IDOT and the Illinois State Water Survey have conducted a cooperative investigation to more adequately assess the operation and condition of the wells, to attempt to understand the probable causes of well deterioration, and to evaluate rehabilitation procedures used on the wells.

Work scheduled for FY 99 (Phase 16) included ten step tests, monitoring of the rehabilitation of seven wells, and checking eight dewatering wells for sand pumpage. Three of the step tests were conducted to assess the present condition of wells to determine their need for chemical treatment in the future or to monitor the results of previous chemical treatments. One of these three wells was in acceptable to good condition, with a specific capacity of about 95 gallons per minute (gpm) per foot of drawdown (gpm/ft). The second well was in fair condition, with a specific capacity of about 56 gpm/ft. The third well was in poor condition, with an average specific capacity of about 39 gpm/ft, and treatment for it was recommended.

Posttreatment step tests were used to help document the rehabilitation of seven dewatering wells during FY 99 (Phase 16): I-70 Wells 3A and 10; I-64 Well 8; and Venice Wells 2, 3, 4, and 6A. Chemical treatments used to restore the capacity of these seven wells were moderately successful. The improvement in specific capacity per well averaged 111 percent based on specific-capacity data from pre- and posttreatment step tests. The specific capacity of I-70 Well 3A was restored to about 73 percent and I-70 Well 10 to about 86 percent of the average specific capacity of wells in good condition at the I-70 site. The specific capacity of I-64 Well 8 was restored to about 112 percent of the average specific capacity of wells in good condition at the I-64 site. The specific capacities of Venice Wells 2, 3, 4, and 6A were restored to 92 percent, 96 percent, 101 percent, and 110 percent, respectively, of the average specific capacity of wells in good condition at the Venice site.

The sand pumpage investigation, conducted during eight step tests, revealed that Missouri Avenue Well 1, I-64 Well 8, and Venice Well 3 were pumping sand. These conditions may pose a threat to the long-term operation of these wells, especially Missouri Avenue Well 1. Smaller amounts of sand were found following the step test for I-64 Well 8 and Venice Well 3.

Introduction

Background

The Illinois Department of Transportation Division of Highways (IDOT) operates 55 high-capacity water wells at five sites in the East St. Louis area. The wells are used to control and maintain ground-water levels at acceptable elevations to prevent depressed sections of interstate and state highways from becoming inundated by ground water. When the interchange of Interstate I-55/I-70 and I-64 was originally designed, ground-water levels were at lower elevations because of large withdrawals by the area's industries. Due to a combination of water conservation, production cutbacks, and conversion from ground water to river water as a source, industrial ground-water withdrawals have decreased at least 50 percent since 1970. As a result, ground-water levels in many areas have recovered to early development levels, which exacerbates IDOT's need to keep ground-water levels below the areas of depressed highways.

In October 1982, IDOT asked the Illinois State Water Survey (ISWS) to begin an investigative study to learn more about the condition of the dewatering wells, to determine efficient monitoring and operating procedures, and to determine suitable methods of well rehabilitation.

Previous Reports

Several ISWS publications document the dewatering well assessment activities since the ISWS has been involved. Phases 1-11, which document project activities corresponding to fiscal years (FYs) 1984-1994, respectively, are contained in the reports listed below. Sanderson and Olson (1999) provide a brief (approximately one paragraph) description of the scope of work for each of these phases on previous years' studies. A historical summary of dewatering development, including discussion of earlier dewatering systems that failed, also is provided.

Listing of Previous Years Dewatering Well Assessment Reports by Year

Phase 1 - Sanderson et al., 1984
Phase 2 - Sanderson et al., 1987
Phase 3 - Olson et al., 1990
Phase 4 - Wilson et al., 1990
Phase 5 - Wilson et al., 1991
Phase 6 - Olson et al., 1992
Phase 7 - Sanderson et al., 1993
Phase 8 - Sanderson and Olson, 1993
Phase 9 - Olson and Sanderson, 1997
Phase 10 - Sanderson and Olson, 1998
Phase 11 - Sanderson and Olson, 1999

Scope of Study

The scope of this study is to present a summary of the field activities, the data collected, and an analysis of these data for the FY 99 phase of this ongoing study.

Physical Setting of Study Area

The study area is located in the alluvial valley of the Mississippi River in East St. Louis, Illinois, in an area known as the American Bottoms (figure 1). The geology of the area consists of alluvial deposits overlying limestone and dolomite of the Mississippian and Pennsylvanian Age. The alluvium varies in thickness from zero to more than 170 feet, averaging about 120 feet. The region is bounded on the west by the Mississippi River and on the east by upland bluffs. The regional ground-water hydrology of the area is well documented (Bergstrom and Walker, 1956; Schicht, 1965; Collins and Richards, 1986; Ritchey et al., 1984; Kohlhase, 1987; Schicht and Buck, 1995). Except where it is diverted by pumpage or drainage systems, ground water generally flows from the bluffs toward the river.

Detailed location maps of the five dewatering sites operated by IDOT are shown in figures 2-4. The geology at these sites is consistent with regionally mapped conditions. The land surface lies at about 410 to 415 feet above mean sea level (ft-msl). Alluvial deposits are about 90 to 115 feet thick, which corresponds to a bedrock surface at approximately 300 to 320 ft-msl. The alluvium becomes progressively more coarse with depth. The uppermost 10 to 30 feet of the alluvium consists of extremely fine sand, silt, and clay, underlain by the aquifer, which is about 70 to 100 feet thick. The elevation of the top of the aquifer is about 390 to 395 ft-msl.

Individual Well Systems

I-70 System

Experience during highway construction in 1961-1962 and during the 1963 drainage system replacement showed that individual dewatering wells were effective in temporarily maintaining ground-water levels at desired elevations. This alternative was, therefore, given further study as a permanent system. A consultant's report (Layne-Western Company, 1972) showed that water levels at the I-70 Tri-Level Bridge site could be maintained at desired elevations with ten deep wells equipped with 600 gpm pumps. Two additional wells were included to permit well rotation and maintenance. These 12 wells were constructed in 1973, and the new system was placed in service in April 1974 (I-70 site). The 16-inch gravel-packed (42-inch borehole) wells had an average depth of about 96 feet, and they were equipped with 60 feet of Layne stainless steel well screen. Pumps with 600-gpm capacity and 6-inch-diameter stainless steel (flanged coupling) column pipe were set in the wells.

A recorder well, 8 inches in diameter and constructed of stainless steel casing and screen, was included in the well dewatering system to monitor ground-water levels near the critical (i.e., lowest) elevation of the highway. A Leupold-Stevens Type F recorder is in use. Additionally, 2-inch-diameter piezometers with 3-foot-long screens were placed about 5 feet from each

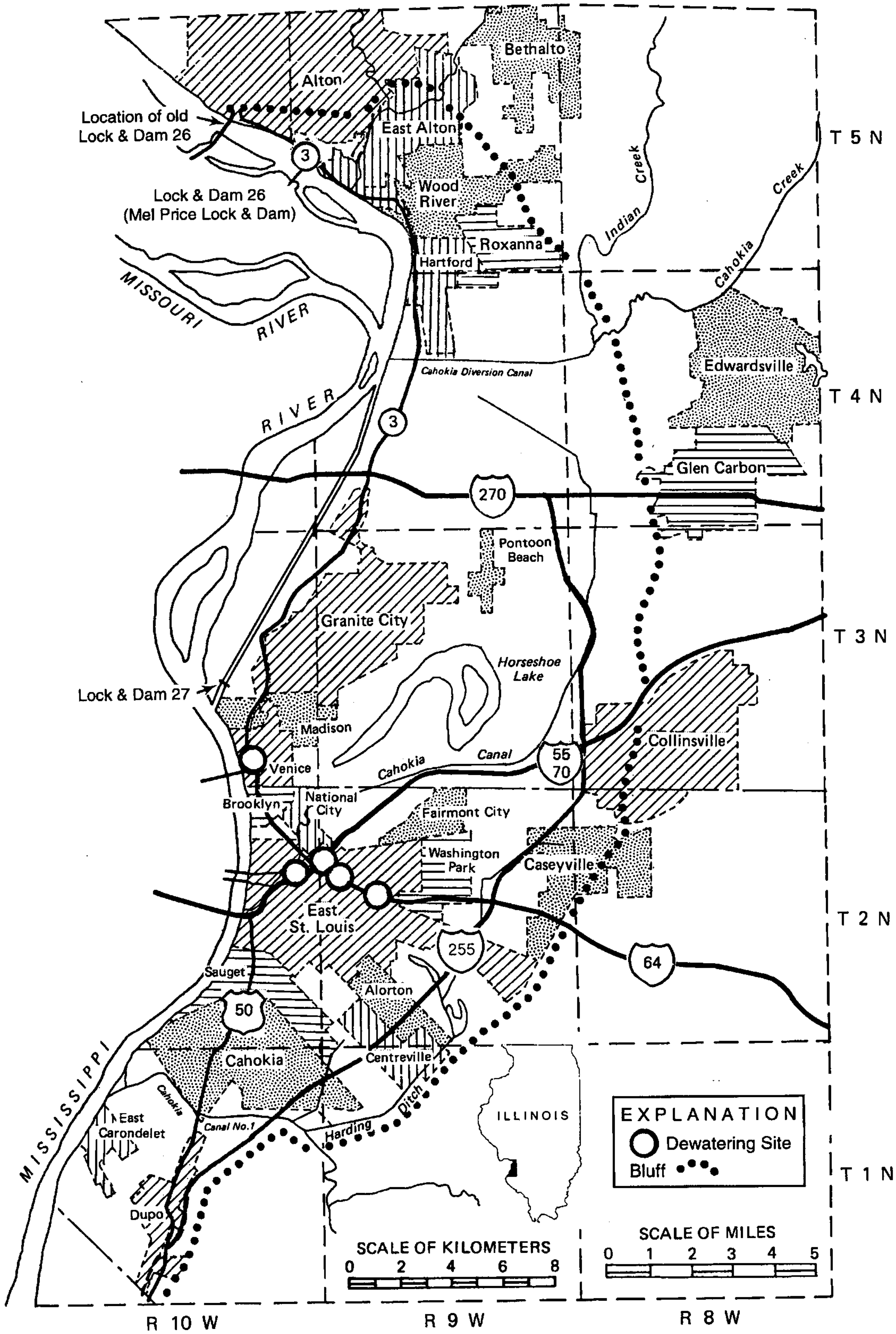


Figure 1. Location of the East St. Louis area

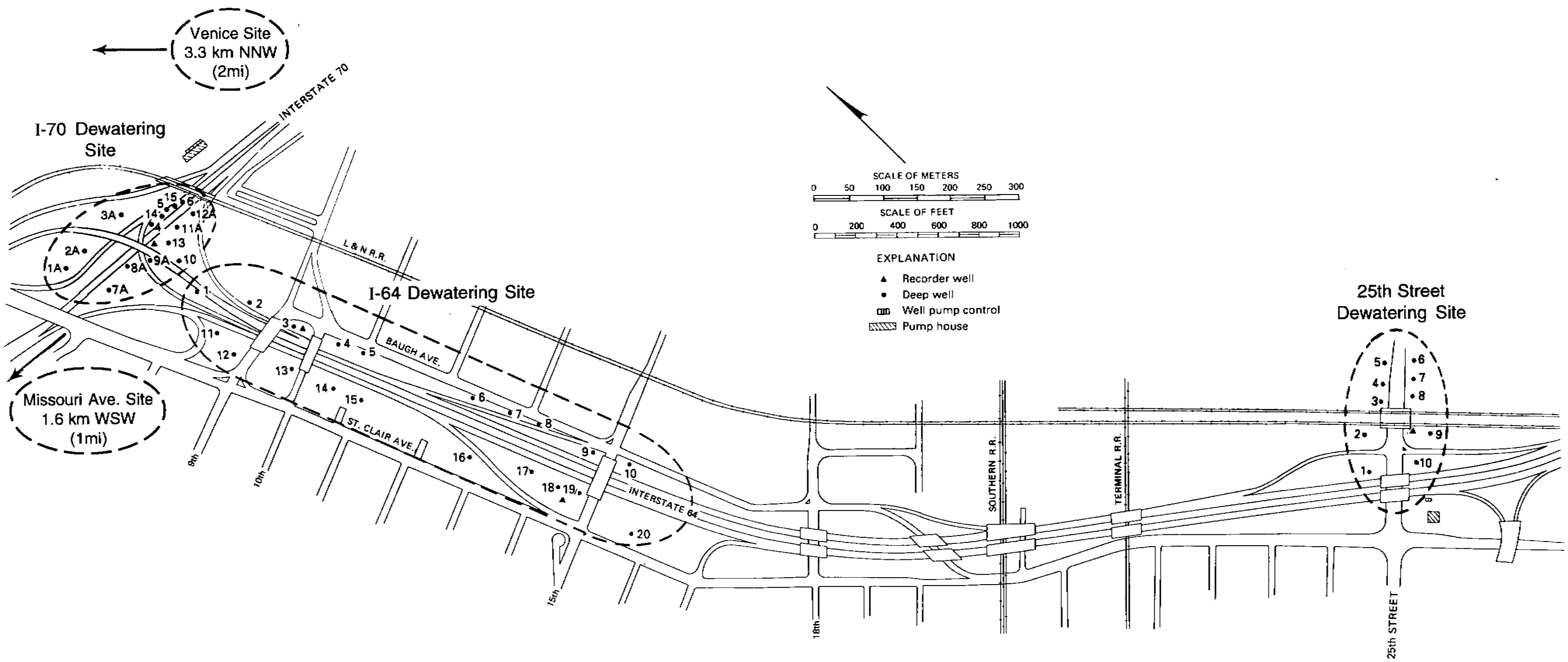


Figure 2. Locations of dewatering wells at the I-70 Tri-Level Bridge, I-64, and 25th Street

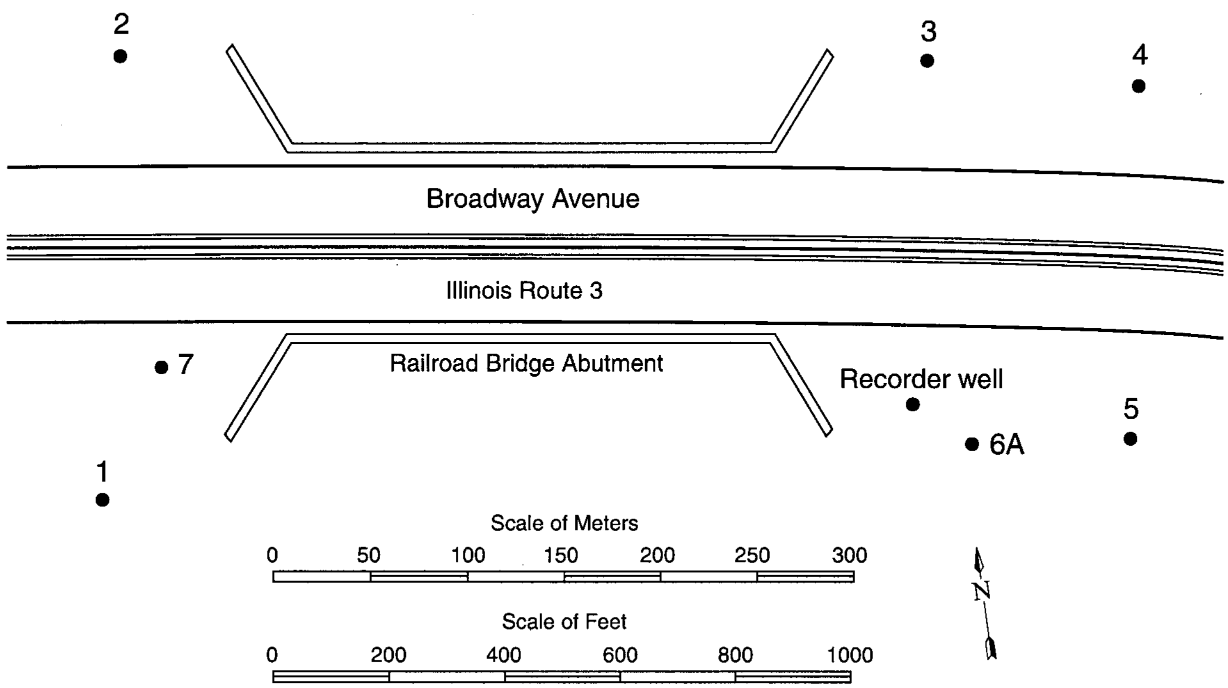


Figure 3. Locations of dewatering wells at the Venice Subway (Illinois Route 3)

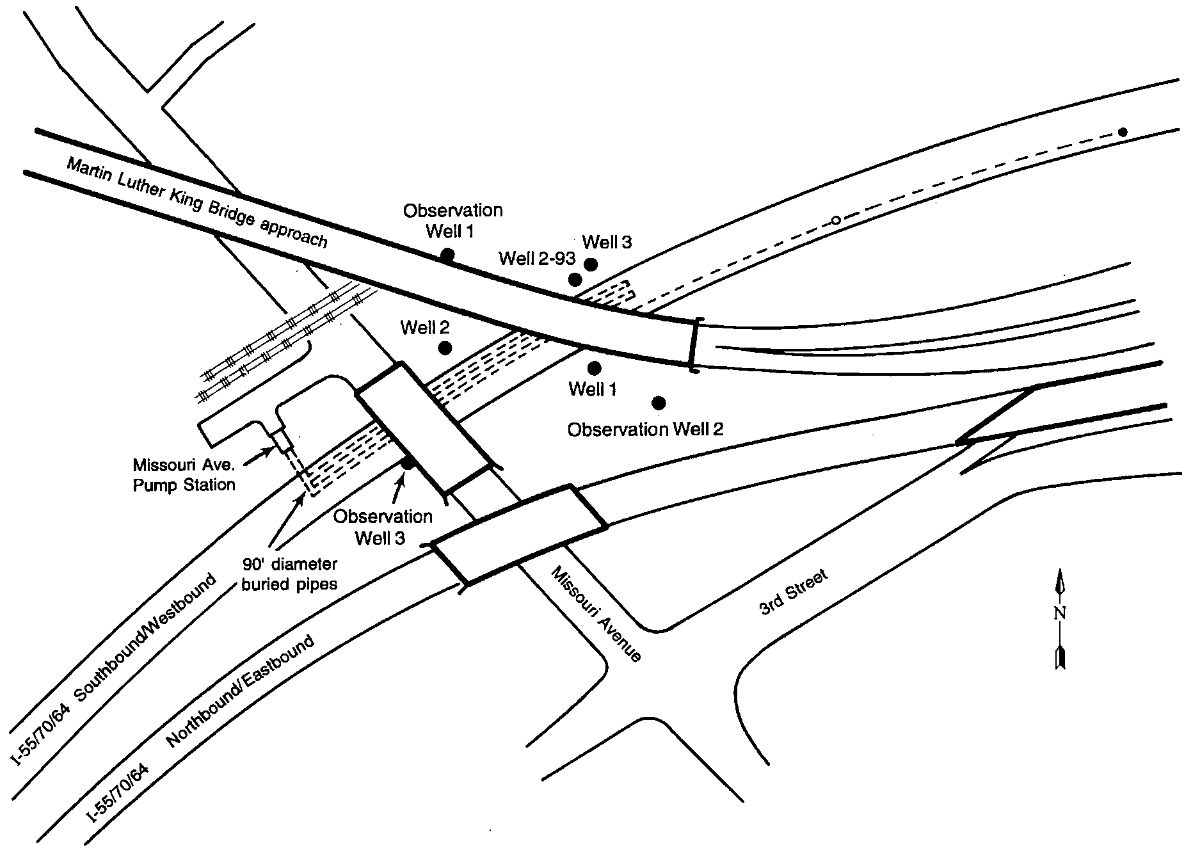


Figure 4. Locations of dewatering wells at Missouri Avenue

dewatering well to depths corresponding to the upper third point of each dewatering well screen. These piezometers provide information on ground-water levels and monitor the performance of individual wells by measuring water-level differences between the wells and the piezometers.

In the late 1970s, the exit ramp from the I-64 westbound lanes onto the I-55/I-70 northbound lanes was relocated, necessitating the abandonment of I-70 Well 12. Replacement Well 12A was then constructed at a nearby location using components similar to those in the original wells. The well screen in I-70 Well 7 reportedly failed in the 1970s, and an attempt was made to rehabilitate the well by inserting a new screen inside the old screen. The well's pumping capacity remained unsatisfactory following this modification, so the well was used only on an emergency basis until it was replaced in 1986. The replacement well (Well 7A) was constructed using components similar to those used in the original wells, with the exception of a continuous-slot well screen designed on the basis of the sieve data from the nearest original test boring (Wilson et al., 1990).

In late 1986, loss of gravel pack was discovered at I-70 Well 9, and subsequent investigation revealed pumpage of fine sand, apparently from the upper 5 to 10 feet of well screen. In 1987, sand pumpage also was discovered at I-70 Wells 2 and 8 and at Venice Well 6. Replacement wells were constructed in the spring of 1989 for I-70 Well 8 (now Well 8A) and I-70 Well 9 (now Well 9A). Continuous-slot well screens also were used in these wells as in I-70 Well 7A (Olson et al., 1992).

In 1990 (FY 91), two more wells were added at the I-70 site to provide greater flexibility in operation, maintenance, treatment, and repair of the other wells at the site. These wells (I-70 Wells 13 and 14) were located on either side of the eastbound lanes of I-55/I-70 near the lowest point of the highway. The wells were similar in construction to the replacement wells (Wells 7A, 8A, and 9A) drilled in 1987 and 1989.

In 1991 and 1992 (FY 92), four replacement wells and one new well were added to the I-70 site. Because of various sand pumpage, settlement, and potential operational problems, replacement wells were constructed for Wells 1, 2, 3, and 11 (new Wells 1A, 2A, 3A, and 11A). The new well (Well 15) was placed between Wells 5 and 6. The wells were similar in construction to the new wells drilled in 1987, 1989, and 1990.

I-64 System

The western terminal of I-64 joins I-70 at the Tri-Level Bridge site. A 2,200-foot stretch of this highway also is constructed below the original land surface as it approaches the Tri-Level Bridge site. To maintain ground-water levels along I-64, a series of 20 wells was added to the dewatering system (I-64 site). The wells were built in 1975 and are essentially identical to the original wells constructed for the Tri-Level Bridge site.

25th Street System

About 6,200 feet southeast of the Tri-Level Bridge, at the interchange with I-64, 25th Street in East St. Louis was designed to pass below the interstate highway and adjacent railroad tracks. As a result, the 25th Street pavement is about 3.5 feet below ground-water levels. Ten wells were installed in 1975 to control ground-water levels at the 25th Street site. These wells are identical in design to the original I-70 wells. Pumps installed in the wells along I-64 and at 25th Street have a nominal pumping capacity of 600 gpm. Two 8-inch-diameter observation wells, located near each end of the depressed section of I-64, are used to monitor ground-water levels. An 8-inch-diameter observation well also was installed near the critical location at the 25th Street underpass. As at the I-70 wells, each dewatering well for I-64 and 25th Street has a piezometer located approximately 5 feet away to monitor performance at the installation.

Venice System

Approximately 2¼ miles north of the I-70 Tri-Level Bridge, Illinois Highway 3 passes beneath the Norfolk and Western, Illinois Central Gulf, and Conrail railroad tracks. When the highway was constructed, ground-water levels were controlled with a horizontal drain system placed 3 feet below the pavement. Problems with the pavement and drainage system were noted in May 1979 and were attributed to the above-normal ground-water levels resulting from 3 to 4 months of continuous flood stage in the Mississippi River (about 2,000 feet west). Subsequent investigation showed deterioration of the drainage system, and the consultants recommended installation of six wells to control ground-water levels at the site (Johnson, Depp, and Quisenberry, 1980). The wells were installed in 1982. They are 16 inches in diameter with 50 feet of well screen, range in depth from 78 to 89 feet below grade, and are equipped with submersible turbine pumps with nominal capacities of 600 gpm. One recorder well for the site and piezometers at each dewatering well were constructed to monitor system performance.

Problems were encountered with Venice Well 6 after chemical treatment in FY 88 (Phase 5). The well pumped sand formation and gravel-pack particles, indicating a possible split or weld failure of the well screen or well casing. Replacement Well 6A was drilled, and a new Well 7 was added at the Venice site in FY 91 (Phase 8). District highway staff considered the additional well desirable because of operational problems maintaining appropriate ground-water levels in 1984 when the Mississippi River was at high stages for several months. The wells are similar in construction to the original wells at this site.

Missouri Avenue System

During the spring and summer of 1993, the Mississippi River was at flood elevations for an extended period. Just east of the Martin Luther King Bridge near downtown East St. Louis beneath the southbound/westbound lanes of I-55/I-64/I-70, two large-diameter, stormwater detention structures were found to be subject to failure due to excessive infiltration of ground water and piping of foundation material into the structures. The IDOT engineers contracted, on an emergency basis, for the construction of four high-capacity dewatering wells to drawdown the high ground-water levels at the stormwater structures to help minimize the chance for their failure. Three wells are now equipped with 1,200 to 1,500 gpm well pumps and are in regular use. The fourth well (Well 2-93) is capped and remains available as an alternate for nearby Well 3.

Summary

The highway dewatering operation in the American Bottoms consists of 55 individual dewatering wells finished in the water-bearing sand-and-gravel aquifer. The wells are distributed at five sites as follows:

I-70 (Tri-Level Bridge)	-	15 wells
I-64	-	20 wells
25th Street	-	10 wells
Venice (Route 3)	-	7 wells
Missouri Avenue	-	3 wells

The wells are of similar construction, generally with 16-inch-diameter stainless steel casings and screens (figure 5). The IDOT's early experience with severe corrosion problems showed that corrosion-resistant materials are required to maximize service life. Except for the Missouri Avenue site, each well is equipped with a 600-gpm submersible pump with bronze impellers, bowls, jacket motors, and a 6-inch-diameter stainless steel column pipe. Five 8-inch-diameter recorder wells are available to monitor ground-water elevations near critical locations at these four sites. Most of the 52 wells have a 2-inch-diameter piezometer nearby to help monitor individual well performance. The wells at Missouri Avenue are equipped with 1,200 to 1,500 gpm pumps with Niresist© impellers and bowls, stainless steel jacket motors, and 6- to 8-inch-diameter stainless steel column pipes. Three 2-inch-diameter piezometers are measured periodically to monitor ground-water elevations at the site.

Usually, about one-third of the wells operate simultaneously. Total pumpage was estimated to be about 23 million gallons per day (mgd) in 1993, about twice the average estimated amount because of the 1993 Mississippi River flood conditions.

Acknowledgments

This phase of the assessment of the condition of the highway dewatering well systems in the American Bottoms was funded by IDOT, Kirk Brown, Secretary. Barry Roberts, Pump Technician, District 8, reviewed and coordinated the investigation. He and maintenance personnel provided field support during step-drawdown tests on the selected wells. Bryan Coulson of the ISWS assisted the authors with field data collection.

Analytical work by the ISWS Analytical Chemistry and Technology Unit was performed under the direction of Kent Smothers, with Brian Kaiser, Lauren Sievers, and Daniel Webb performing the lab analyses. Eva Kingston and Agnes Dillon edited the manuscript, Linda Hascall prepared the illustrations, and Pamela Lovett provided word processing support. Steve Burch and Ken Hlinka reviewed the manuscript for technical content.

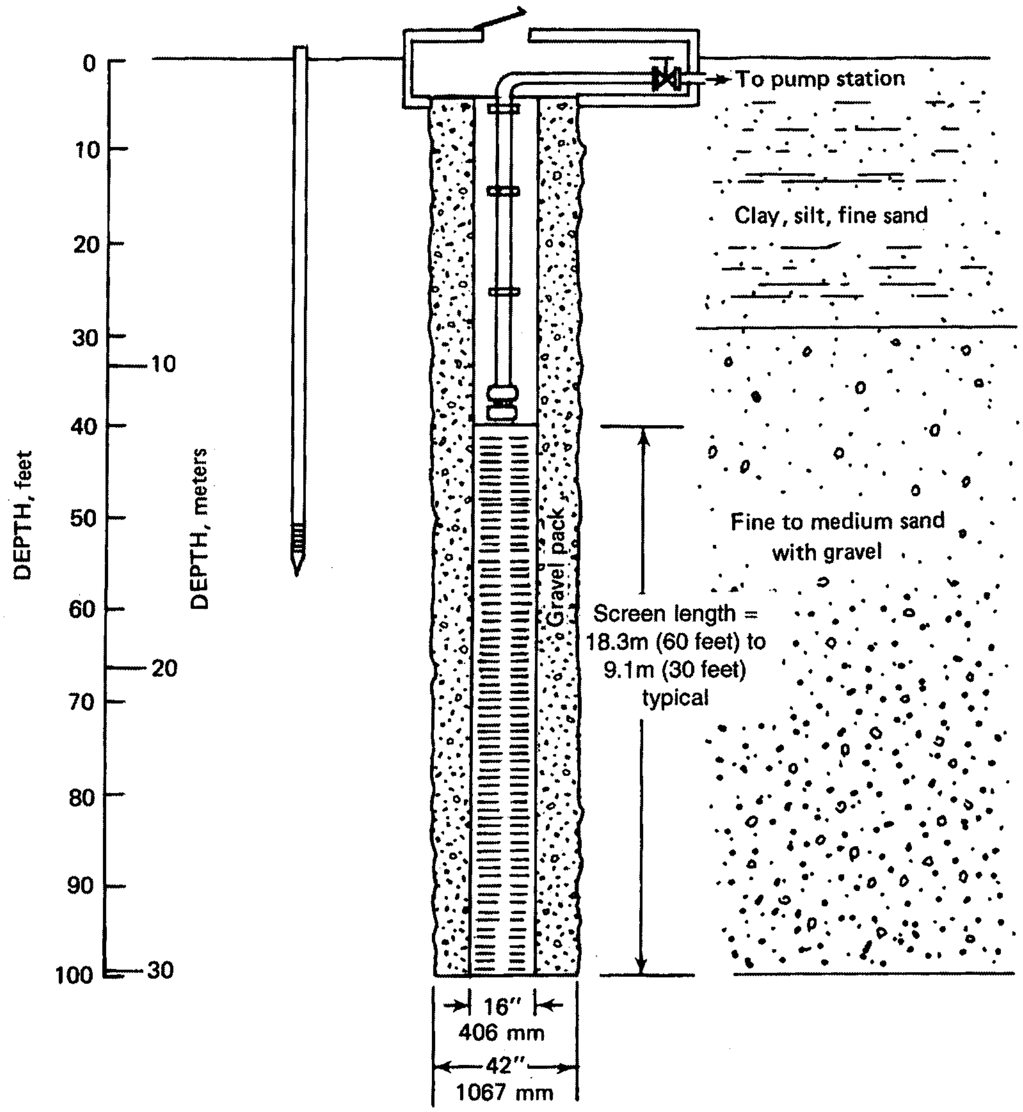


Figure 5. Typical features of a dewatering well

Any opinions, findings, and conclusions or recommendations expressed in this report are those of the authors and do not necessarily reflect those of the sponsor or the Illinois State Water Survey.

Dewatering System Monitoring

When originally constructed, the wells at I-70, I-64, and 25th Street included pitot-tube flow-rate meters. A combination of corrosion and chemical deposition caused premature failure of these devices. Flow rates were occasionally checked with a pitot-tube meter temporarily inserted, but the field crew reported erratic results. The six installations at Venice in 1982 included a venturi tube coupled to a bellows-type differential pressure indicator to measure the flow rate. However, the water quality and environment in the well pits also adversely impacted the operation of these instruments. Accurate flow measurements became impossible within a few years, and the field crew reported at least one direct failure of the venturi tube. These meters were subsequently disconnected.

As part of the scope of work in FY 85-FY 87 (Phases 2-4), a noninvasive, portable ultrasonic flowmeter was tested, calibrated, and used to check flow rates for specific capacity calculations of 21 dewatering wells. Although this meter was found to be limited in some cases, it was turned over to IDOT for use in their routine monitoring program.

Operational records have shown that wells are pumped for periods of about two to nine months, then idled for longer periods while another set of wells is operated. No standard sequence of pumping rotation is followed because of maintenance and rehabilitation requirements. Annual withdrawals currently are calculated on the basis of pumping time and estimated pumping rates.

Until November 1989, IDOT highway maintenance personnel periodically measured water levels at each dewatering well to monitor the overall performance of the dewatering system. Due to internal reorganization of the highway maintenance staff in District 8, ISWS staff began monitoring ground-water levels at the dewatering sites at the end of February 1990. Until the mid-1990s, water levels were measured every two months in each dewatering well and in the adjacent piezometer of each pumping well. After this time, the frequency of the water level measurements was reduced to a quarterly basis. Data collected during FY 99 (Phase 16) have been tabulated and are listed in appendix A.

Each dewatering well site (except Missouri Avenue) also includes at least one observation well (two at the I-64 site) equipped with a Leupold-Stevens Type F water-level recorder. Recorder charts are changed monthly and provide a continuous record of water levels near the critical location at each dewatering site. Because of the District 8 reorganization, the ISWS also assumed responsibility for the monthly servicing of the recorders beginning at the end of November 1989.

Each time measurements are collected, the ISWS forwards a report to IDOT of the ground-water level data, including any recommendations. This information is used to compare ground-water elevations to pavement elevations and evaluate if any adjustments in pumpage are necessary. The data also are useful for assessing the condition of individual dewatering wells. Water-level differences of 3 to 5 feet between the pumping wells and the adjacent piezometers are considered normal by IDOT. Greater differences are interpreted to indicate that well deterioration is occurring.

Investigative Methods and Procedures

Well Loss

When a well is pumped, water is removed from storage within the aquifer, causing water levels to decline over time in the vicinity of the well. This effect, referred to as drawdown, is most pronounced at the pumped well and gradually diminishes at increasing distances away from the well. Drawdown is the distance that the water level declines from its nonpumping stage. Under ideal conditions, drawdown is a function of pumping rate, time, and the aquifer's hydraulic properties. Aquifer boundaries, spatial variation in aquifer thickness or hydraulic properties, interference from nearby wells, and partial-penetration conditions all can affect observed drawdowns at both pumping and observation wells. However, well loss or additional drawdown inside the pumped well due to turbulent flow of water into and inside the well is a measure of the hydraulic efficiency of the pumping well only, reflecting the unique flow geometry of the borehole, well screen, and pump placement.

Because of well loss, the observed drawdown in a pumped well is usually greater than that in the aquifer formation outside the borehole. In addition to considerations of flow geometry, as noted above, the amount of well loss also can depend on the materials used (screen openings, gravel-pack size distribution, drilling fluids, etc.) and the care taken in constructing and developing the well. Some well loss is natural because of the physical blocking of the aquifer interstices caused by the well screen and the disturbance of aquifer material around the borehole during construction. However, an improperly designed well and/or ineffective well construction and development techniques can result in excessive well losses. In addition, well losses often reflect a deterioration in the condition of an existing well, especially if well losses increase over time.

Specific capacity, the quotient of pumping rate divided by the drawdown observed after a given time period, is often used in the field as an indicator of well performance. However, specific capacity combined with an analysis of well loss provides a more complete picture of the condition of the well that allows for normalization and comparison at various pumping rates.

Well loss is a function of pumping rate, but theoretically not of time. It is associated with changes in flow velocity in the immediate vicinity of the well, resistance to flow through the well screen, and changes in flow path and velocity inside the well, all of which cause the flow to change from laminar to turbulent. Head losses under turbulent conditions are nonlinear; that is, drawdowns increase more rapidly with increases in pumping rate than under laminar conditions, as discussed below.

Although it is possible to have turbulent flow within the aquifer and laminar flow within a pumping well, under near-ideal conditions the observed drawdown (s_o) in a pumping well is made up of two components: the formation loss (s_a), resulting from laminar flow head loss within the aquifer, and well loss (s_w), resulting from the turbulent flow of water into and inside the well, as shown in equation 1.

$$s_o = s_a + s_w \tag{1}$$

Jacob (1947) devised a technique for separating well losses from formation losses, assuming that all formation losses are laminar and all well losses are turbulent. These components of theoretical drawdown, s, in the pumped well are then expressed as being proportional to pumping rate, Q, in the following manner:

$$s = BQ + CQ^2 \tag{2}$$

where B is the formation-loss coefficient per unit discharge, and C is the well-loss coefficient. For convenience, s is expressed in feet and Q in cubic feet per second (ft^3/sec). Thus, the well loss coefficient C has units sec^2/ft^5.

Rorabaugh (1953) suggested that the well-loss component be expressed as CQ^n, where n is a constant greater than 1. He thus expressed the drawdown as:

$$s = BQ + CQ^n \tag{3}$$

To evaluate the well-loss component of the total drawdown, one must know the well-loss coefficient (if using equation 2) or both the coefficient and the exponent (if using equation 3). These analyses require a controlled pumping test, called a step drawdown test (described below), in which total drawdown is systematically measured while pumping rates are varied in a stepwise manner.

Methodology for Determining Well Loss

If Jacob's equation is used to express drawdown, then the coefficients B and C must be determined. A graphical procedure (Bierschenk, 1964) can be used after first modifying equation 2 as:

$$s/Q = B + CQ \tag{4}$$

Substituting the observed drawdown, s_o, for s, a plot of s_o/Q versus Q can be prepared on arithmetic graph paper from data collected during a step drawdown test. The slope of a line fitted to these data is equal to C, and the y-intercept is equal to B, as shown in figure 6. If the data do not fall within a straight line but curve concavely upward, the curvature of the plotted

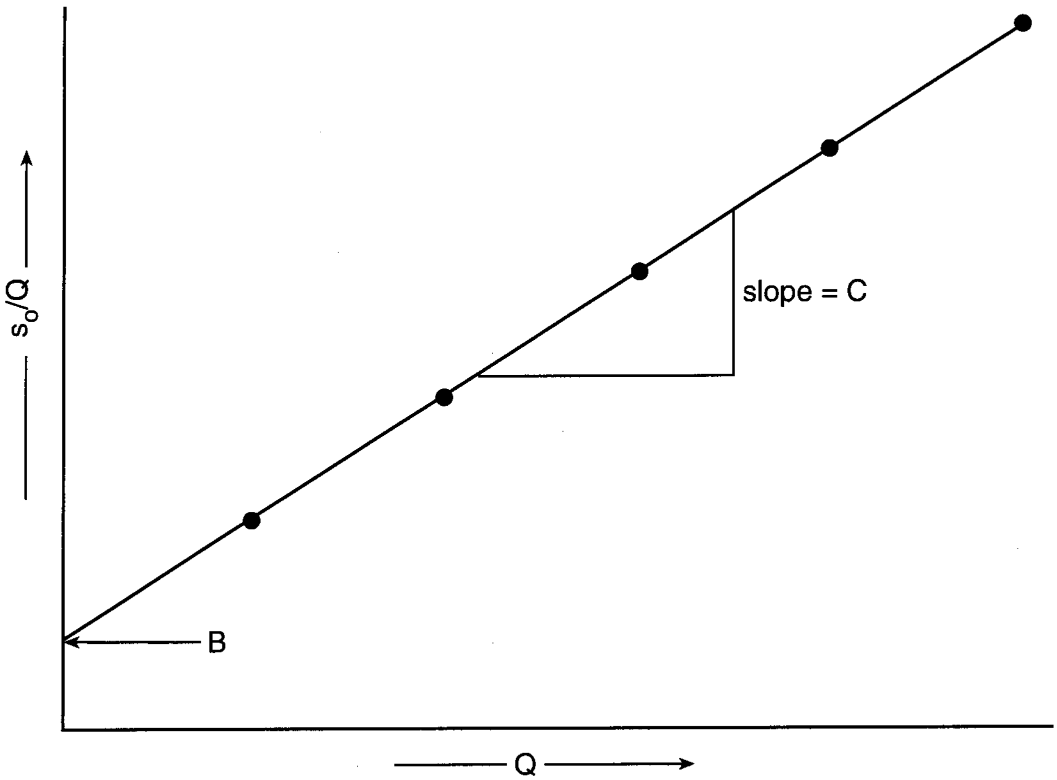


Figure 6. Graphical solution of Jacob's equation for well loss coefficient, C

data indicates that the second-order relationship between Q and s_o is invalid, and that the Rorabaugh method of analysis usually is appropriate.

Occasionally the data plot of s_o/Q versus Q may yield a straight-line fit with essentially zero slope or with a negative slope, or the data may be too scattered to allow a reasonable fit to be made at all. In these instances, the well-loss parameters are immeasurable. Possible explanations for this are: 1) turbulent well loss was negligible for the range of pumping rates used during the test; 2) inadequate data collection or test methods were used during the test; 3) the hydraulic condition of the well was unstable, as is the case during well development; or 4) the contribution of water from the aquifer was not uniform along the entire length of well screen over the range of pumping rates, as might occur due to the pump setting in relation to the screen or to vertical heterogeneity of the aquifer materials.

If Rorabaugh's equation is used, then coefficients B and C as well as the exponent n must be determined. To facilitate a graphical procedure, equation 3 is rearranged as:

$$(s/Q) - B = CQ^{n-1} \tag{5}$$

Taking logs of both sides of the equation,

$$\log [(s/Q) - B] = \log C + (n - 1) \log Q \tag{6}$$

A plot of $(s_o/Q) - B$ versus Q can be made on logarithmic graph paper from step-test data by replacing s with s_o. Values of B are determined by trial and error until the data form a straight line (figure 7). The slope of the line equals n - 1, from which n can be found. The value of C is determined from the y-intercept at Q = 1. In the example shown, plotting the data is facilitated if Q is plotted as ft^3/sec, and $(s_o/Q) - B$ is plotted as seconds per foot squared (sec/ft^2). It also is convenient to use these same units in the Jacob method.

Step-Test Procedure

The primary objective of a step drawdown test (or step test) is to determine the well-loss coefficient (and exponent, if Rorabaugh's method is used). With this information, the turbulent well-loss portion of drawdown for any pumping rate of interest can be estimated. During the test, the discharge rate is successively increased or decreased from the previous rate, in approximately equal increments, in order to facilitate the data analysis. Each pumping rate is called a step, and all steps are of equal time duration. Generally, the pumping rates increase from step to step, but the test also can be conducted by decreasing pumping rates. Conducting the steps at decreasing rates has been found to be the most efficient procedure at the dewatering well sites.

During each step, pumpage is held constant. If data are collected manually, water-level measurements are made every minute for the first six minutes, every two minutes for the next ten minutes, then every four to five minutes thereafter until the end of the step. For the step tests in this study, an Omnidata datalogger, an InSitu Hermit datalogger, or an electric dropline was used to collect the data. Generally, the dataloggers were programmed to collect water-level data at

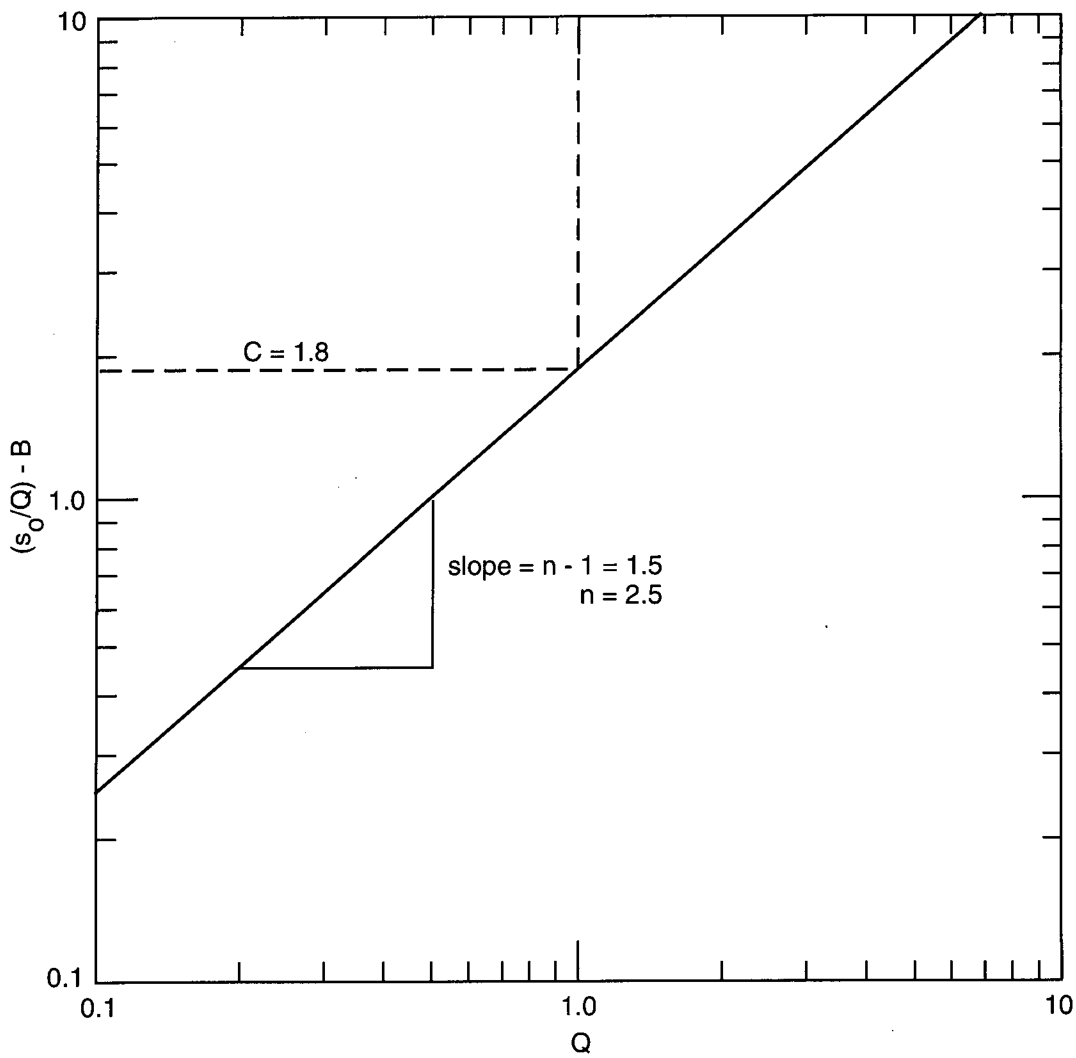


Figure 7. Graphical solution of Rorabaugh's equation for well loss coefficient (C) and exponent(n)

least once each minute during the step test. Water levels were measured for 30 minutes per step for this investigation. At the end of each 30-minute interval, the pumping rate was immediately changed, and water levels were monitored for another 30-minute interval until a wide range of pumping rates within the capacity of the pump was tested.

Schematically, the relationship between time and water level resembles that shown for a five-step test in figure 8. Incremental drawdowns for each step (shown as Δs_i) are measured as the distance between the extrapolated water levels from the previous step and the final water level of the current step. For step 1, the nonpumping water-level trend prior to the start of the test is extrapolated, and Δs_1 is measured from this datum. All data extrapolations should be performed on semilog graph paper for the most accurate results. For the purpose of plotting s_o/Q versus Q or (s_o/Q) - B versus Q, values of observed drawdown s_o are equal to the sum of Δs_i for the step of interest. Thus, for step 3, $s_o = \Delta s_1 + \Delta s_2 + \Delta s_3$.

Piezometers

Piezometers are small-diameter wells with a short length of screen; they are used to measure water levels (head) at a point in space within an aquifer. They often are used in clustered sets to measure variations in water levels with depth. For well-loss studies, piezometers can be used to measure head losses across a well screen, gravel pack, or well bore. As previously described, 52 of the IDOT dewatering wells (except at Missouri Avenue) have piezometers drilled approximately 5 feet from the center line of each well and finished at a depth corresponding to approximately the upper third point of the screen in the pumping well. Historical monitoring of the difference in head (Δh) between water levels in the well and in the adjacent piezometer has been used to help detect and track well deterioration problems.

Measuring piezometer water levels continuously during each step test also allows an indication of turbulent well losses in the pumped well to be found by plotting the Δh data over a large range of pumping rates. If turbulent losses exist within that range, the head differences should be nonlinear with increasing pumping rate. In addition, it sometimes can be useful to simply plot depth to water (or drawdown) in the piezometer versus pumping rate. If turbulence extends outward from the well to the piezometer, this relationship will be nonlinear.

Field Results

Well Selection for Step Tests

Ten wells were step-tested in FY 99 (Phase 16). Three wells were selected for step tests to assess their condition while posttreatment step tests were conducted on the other seven wells that had been chemically treated to restore production capacity.

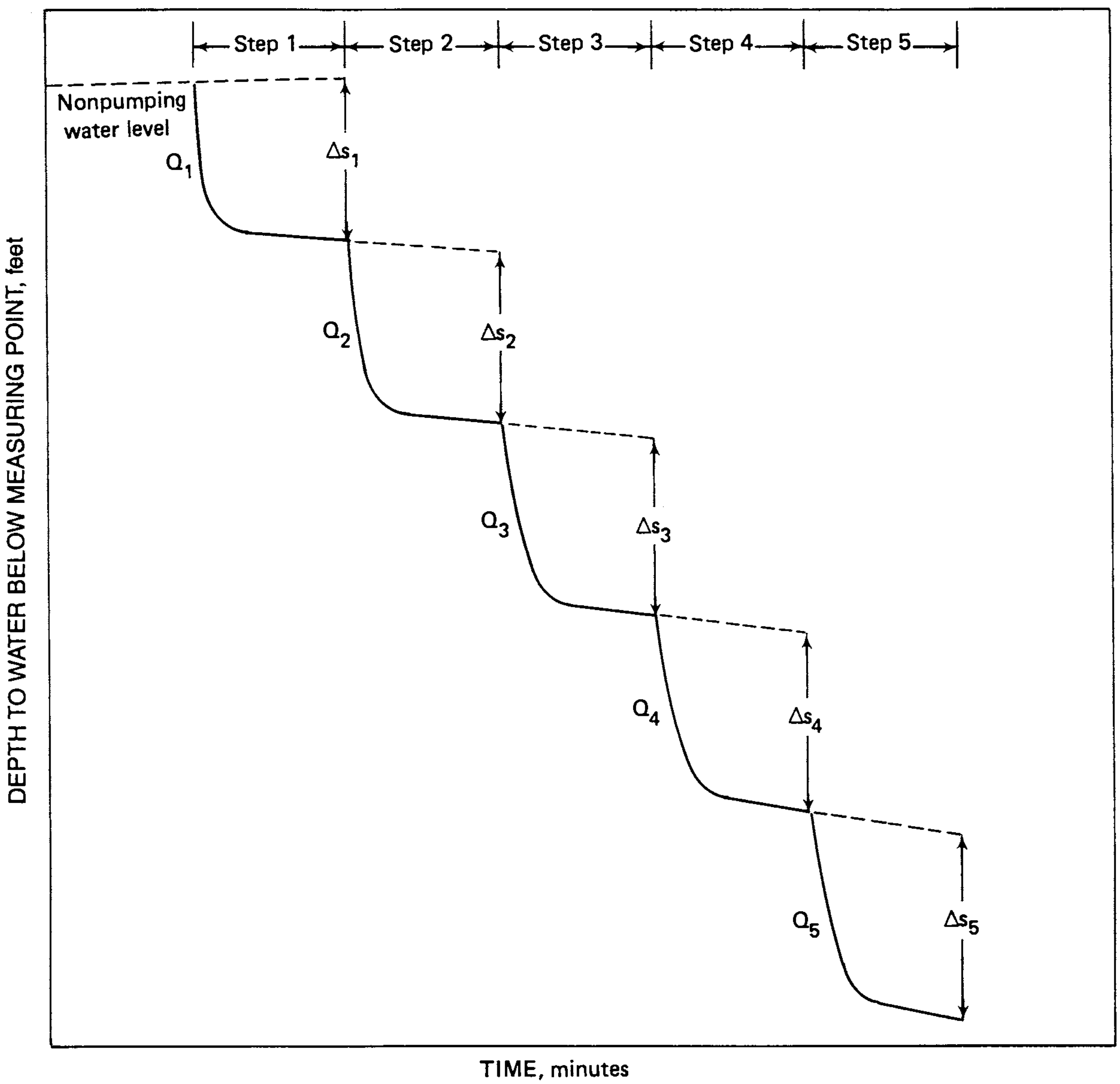


Figure 8. Relationship between time and water level during a five-step drawdown test

The three wells selected for condition assessment step tests were:

Missouri Avenue	Wells 1, 2, and 3

The seven wells treated then tested in posttreatment step tests were:

I-70	Wells 3A and 10A
I-64	Well 8
Venice	Wells 2, 3, 4, and 6A

Step Tests

Field Testing Procedure

The ISWS staff conducted field work with the assistance of the IDOT Bureau of Maintenance pump crew under the supervision of Barry Roberts. The IDOT crew made all necessary wellhead pipe modifications and provided special piping adapters that allowed connection of the ISWS flexible hose and orifice tube to measure the flow rate. Discharge from the orifice tube was directed to nearby stormwater drains.

Orifice tubes are standard equipment for accurately measuring flow rates. The orifice tube and orifice plate used to measure the range of flow rates were previously calibrated at the University of Illinois Hydraulics Lab under discharge conditions similar to those expected in the field.

The objective of each step test on the selected wells was to control the flow rate at increments of 50 gpm and to include as many 30-minute steps as possible at 300 gpm or greater for each well. Early experience with the step tests showed that, at rates of less than about 300 gpm, well-loss coefficients rarely could be determined from the collected data. Also, such a low pumping rate often results from a very low specific capacity, indicating a well already in poor condition. When there is a maximum pumping rate less than about 300 gpm during a step test for a dewatering well, the drawdown in water levels is observed for a period of 30 to 60 minutes to obtain an approximate specific capacity for later comparison; this then is called a drawdown test instead of a step test.

Prior to the start of each test, the water levels in the well and piezometer were measured with a steel tape or electric dropline. Pressure transmitters coupled to one of the previously mentioned dataloggers were placed in the pumped well and the adjacent piezometer (if available) to measure water levels during the step tests.

During the step tests, the discharge from each well also was checked for the presence of sand (unless the site accessibility or site condition precluded set-up of the testing equipment) by directing the open flow from the orifice tube into a 1,000-gallon portable tank. The tank acts as a sedimentation basin, allowing sand grains to be caught, collected at the end of the step test as the tank is drained, and delivered to the geotechnical laboratory for grain-size analysis. Water samples were collected at the time of each test and analyzed for chemical/mineral content and nuisance bacteria. The results from the water sample analyses are described in the following sections and are presented in appendix B.

Results of Step Tests

The step-test data were analyzed by using the Jacob method, as described earlier in this report. Table 1 summarizes results of the analyses of data from the ten step tests conducted for the FY 99 investigation. The Δh values reported in table 1 have been observed or estimated for the standardized rate of 600 gpm. However, comparisons of Δh values are valid only among step tests on the same well because of the varying distances of the piezometers from individual dewatering wells. All step tests conducted for FY 99 were run with steps at decreasing rates so the observed specific capacities included in table 1 were calculated based on the total observed drawdown at the end of the first step when the highest pumping rate was used. Thus, observed specific capacity values were calculated after 30 minutes of pumping but were not standardized to the 600 gpm rate.

Step tests were scheduled to assess the condition of the three dewatering wells at the Missouri Avenue site during FY 99. (For results of the posttreatment step tests conducted on I-70 Wells 3A and 10; I-64 Well 8; and Venice Wells 2, 3, 4, and 6A, see the section, Chemical Treatment Results.)

A step test on Missouri Avenue Well 1 had been conducted on November 13, 1997. The observed specific capacity was about 50 gpm/ft, the well loss was about 11 percent, and the Δh value could not be determined because the piezometer was plugged. The step test conducted on Well 1 for FY 99 on October 15, 1998, showed an observed specific capacity of about 39 gpm/ft. The Δh value could not be determined because of the plugged piezometer, and the well loss could not be determined from the collected data. Well 1 appears to be in poor condition, with an observed specific capacity about 45 percent of the average observed specific capacity of wells at the Missouri Avenue site in good condition (table 2).

A step test on Missouri Avenue Well 2 had been conducted on November 4, 1997. The observed specific capacity was about 117 gpm/ft. The well loss could not be determined via a graphical analysis of the collected data, and the value of Δh could not be determined because there is no piezometer. The step test conducted on Well 2 for FY 99 on December 2, 1998, indicated an observed specific capacity of about 95 gpm/ft. Again, the well loss and Δh value could not be determined. Well 2 appears to be in good condition, with an observed specific capacity about 109 percent of the average observed specific capacity of wells at the Missouri Avenue site in good condition (table 2).

A step test on Missouri Avenue Well 3 had been conducted on November 4, 1997. The observed specific capacity was about 67 gpm/ft, the well loss was about 7 percent, and the Δh value was about 3.5 feet. The step test conducted on Well 3 for FY 99 on December 1, 1998, indicated that the observed specific capacity had decreased to about 56 gpm/ft. The well loss could not be determined via a graphical analysis of the collected data, and the Δh value could not

Table 1. Results of State Water Survey Step Tests on IDOT Wells, FY 99 (Phase 16)

Well	*Date of step test*	*Well loss at 600 gpm (ft)*	*Drawdown at 600 gpm (ft)*	*Well loss portion (%)*	*Observed specific capacity (gpm/ft)*	*Δh* at 600 gpm (ft)*	*Observed Q_{max}, gpm*	*Remarks*
I-70								
No. 3A	10/8/98	**	8.55	**	69.6	1.76	570	T
No. 10	10/7/98	**	7.34	**	82.1	P	545	T
I-64								
No. 8	10/14/98	**	5.09	**	117.9	P	520	T
Venice								
No. 2	10/1/98	**	6.78	**	88.2	2.44	770	T
No. 3	11/17/98	**	6.71	**	91.1	3.44	760	T
No. 4	11/24/98	**	6.25	**	97.1	6.63	720	T
No. 6A	9/30/98	**	5.70	**	105.3	2.01	900	T
MO Ave.								
No. 1	10/15/98	**	15.09	**	39.3	N	470	CA
No. 2	12/2/98	**	6.55 e	**	95.0	N	1,425	CA
No. 3	12/1/98	**	14.6 e	**	55.8	10.5 e	1,375	CA

Notes:

* Head difference between pumped well and adjacent piezometer.
** Coefficient immeasurable. Turbulent well loss negligible over the pumping rates tested.
e = Estimate based on interpolated values adjusted to 600 gpm.
P = Piezometer plugged or partially plugged.
T = Posttreatment step test.
N = No piezometer.
CA = Condition assessment step test.

Table 2. Average Observed Specific Capacity of Dewatering Wells Based on Step Test Data from 196 Tests since FY 84

Well category	*I-70*	*I-64*	*25th St.*	*Venice*	*MO Ave.*	*All sites*
All wells:						
Number of step tests	87	24	32	37	14	194
Average observed specific capacity, gpm/ft	74	92	82	76	75	78
Wells in good condition or posttreatment:						
Number of step tests	46	17	16	22	7	108
Average observed specific capacity, gpm/ft	96	105	119	96	87	101
Wells in poor condition or pretreatment:						
Number of step tests	41	7	16	15	7	86
Average observed specific capacity, gpm/ft	49	61	46	47	62	50

be determined because of the plugged piezometer. Well 3 appears to be in poor condition, with an observed specific capacity about 64 percent of the average observed specific capacity of wells at the Missouri Avenue site in good condition (table 2).

Chemical treatment was recommended for the two wells in poor condition, Missouri Avenue Wells 1 and 2.

Since FY 84 (Phases 1-16), a total of 194 step tests (including six drawdown tests) have been completed at the five dewatering sites in the East St. Louis area. The observed specific capacity data are summarized in table 2. The average observed specific capacity for all 194 step tests is about 78 gpm/ft. By excluding the results from 82 pretreatment step tests and other step tests that show wells in poor condition, the average observed specific capacity of 104 step tests is about 101 gpm/ft. The highest observed specific capacities for all step tests conducted are generally found at the I-64 site, at which 24 step tests have been completed. Observed specific capacities for all step tests at the I-64 site averaged about 92 gpm/ft, or 105 gpm/ft if the seven pretreatment step tests are excluded. Without the pretreatment step tests and other step tests on wells in poor condition, the 25th Street wells would have produced the highest specific capacities on average. The average observed specific capacity for wells in good condition or posttreatment is 119 gpm/ft at the 25th Street site.

Well Rehabilitation

Chemical Treatment Procedure

The specifications for the well rehabilitation work initially were developed in FY 86 by IDOT and the ISWS based on chemical treatment practices in common use. Revisions to the specifications have been made periodically, based on results and experience from chemical treatment of the dewatering wells since 1986. Similar treatment procedures were used for all wells treated in FY 99, although adjustments occurred as specific conditions were encountered from day to day and from well to well. Table 3 summarizes the treatment procedure as required by IDOT specifications. The actual procedure used by the contractor, Layne-Western Company, Inc., varied in some instances, and the significant changes are noted in table 3.

Figure 9 shows schematically the typical injection assembly/discharge apparatus used by the contractor for injecting solutions and acid into the wells, to pump spent solutions to waste, and to conduct drawdown pumping tests during the treatment. The well rehabilitation work was observed and documented by ISWS staff.

Chemical Treatment Results

The wells were selected for chemical treatment on the basis of data from the most recent ISWS step tests and available Δh information (see section, Piezometers). Under a FY 99 IDOT contract, Layne-Western Company, Inc., chemically treated the seven dewatering wells between July 27 and September 23, 1998.

The condition of I-70 Well 3A had been checked during a step test on December 11, 1997. The observed specific capacity of the well was only about 32 gpm/ft, and the Δh was about 10 feet. The well was chemically treated in July 1998. The results of the posttreatment step test conducted for FY 99 on October 8, 1998, showed the observed specific capacity to be about 70 gpm/ft, and the Δh to be about 1.8 feet. Well 3A now appears to be in fair condition, with an observed specific capacity about 73 percent of the average specific capacity of wells at the I-70 site in good condition.

The condition of I-70 Well 10 had been checked during a step test on August 1, 1995. The observed specific capacity of the well was only about 58 gpm/ft. The well was chemically treated in August 1998. The results of the posttreatment step test conducted on Well 10 for FY 99 on October 7, 1998, showed the observed specific capacity to be about 82 gpm/ft. Well 10 now appears to be in good condition, with an observed specific capacity about 85 percent of the average specific capacity of wells at the I-70 site in good condition.

The condition of I-64 Well 8 had been checked during a step test on April 15, 1996. The observed specific capacity of the well was only about 58 gpm/ft, and the well loss was about 20 percent. The well was chemically treated in August 1998. The results of the posttreatment step test conducted on Well 8 for FY 99 on October 14, 1998, showed the observed specific capacity to be about 118 gpm/ft. Well 8 now appears to be in good condition, with an observed specific capacity about 112 percent of the average specific capacity of wells at the I-64 site in good condition.

The condition of Venice Well 2 had been checked during a step test on November 12, 1997. The observed specific capacity of the well was only about 22 gpm/ft, and the Δh was about 23 feet. These results showed that Well 2 had deteriorated significantly since its first treatment in 1990. The well was chemically treated in August 1998. The results of the posttreatment step test conducted on Well 2 for FY 99 on October 1, 1998, showed the observed

Table 3. Outline of Typical Well Rehabilitation

<u>Day 1</u>

1. Pretreatment specific capacity test (contractor orifice tube, open to free discharge, used for flow measurements).

    a. Measurement of SWL (static water level) following 30 or more minutes of well inactivity.
    b. Measurement of PWL (pumping water level) and orifice piezometer tube following 60 or more minutes of pumping.

2. Polyphosphate application, 400 pounds, and displacement with 16,000 gallons of water containing at least 500 ppm (mg/L) chlorine.

    a. Initial chlorination of well with 2,500 gallons of water containing 500 ppm or more of chlorine injected at a minimum rate of 750 gpm (actual rate: 1,300 to 2,100 gpm).
    b. Injection of polyphosphate solution at a minimum rate of 2,000 gpm (actual rate: 1,500 to 2,100 gpm) in two 1,800-gallon batches, each batch containing 200 pounds of polyphosphate.
    c. Displacement injection of 16,000 gallons of water chlorinated to at least 500 mg/L in 2,000-gallon batches at a minimum rate of 1,500 gpm (actual rate: 800 to 2,900 gpm).
    d. Time allowance for chemicals to react, 1 to 2 hours.
    e. Repeatedly surge and backflush well to loosen encrustants with multiple cycles (actual 9 to 19) of pumping well at high rates (actual: 700 to 2,300 gpm) to fill 2,000 gallon holding tank and pumping the contents of the tank back into the well at high rates (actual rate: 960 to 3,600 gpm).

3. Pump to waste and check specific capacity.

    a. Pump continuously for 6 or more hours to clear well of chemicals (actual time, when known: 15.5 to 19.75 hours).
    b. Same procedure for specific capacity check as Day 1, step 1.

<u>Day 2</u>

1. Acidization with 1,000 gallons 20° Baume-inhibited muriatic (hydrochloric) acid and displacement with 4,000 to 5,000 gallons of water (not chlorinated).

    a. Pump 1,000 gallons of bulk-inhibited acid into well within 1 hour, 17 gpm minimum (actual rate: 23 to 130 gpm).
    b. Allowance time for acid to react, 1 hour.
    c. Injection of 4,000 to 5,000 gallons of water at 1,000 to 2,000 gpm (actual rate: 1,500 to 3,000 gpm).
    d. Allowance for reaction, 2 to 3 hours.
    e. Repeatedly surge and backflush well to loosen encrustants with multiple cycles (actual 9 to 14) of pumping well at high rates (actual rates: 222 to 1,100 gpm) to fill a 2,000 gallon holding tank, then pumping the contents of the tank back into the well at high rates (actual rate: 1,000 to 2,700 gpm).

Table 3. Continued

2. Pump to waste and check specific capacity.

   a. Pump continuously for 3 or more hours (actual time: 17 hours) to clear well of acid.
   b. Same procedure for specific capacity check as Day 1, step 1.

<u>Day 3</u>

1. Polyphosphate application, 600 pounds, and displacement with 30,000 gallons of water containing at least 500 ppm chlorine.

   Same procedure as Day 1, step 2, except three batch injections of 1,800 gallons (5,400 gallons total) with 200 pounds of phosphate each in part b, and injection of 30,000 gallons in part c.

   Noted actual pumping rates and surging cycles for indicated steps of procedure.

   a. Initial chlorination: 1,800 to 2,500 gpm.
   b. Polyphosphate solution injections: 1,300 to 3,000 gpm.
   c. Displacements: 1,500 to 3,000 gpm.
   d. No change.
   e. Surging/backflushing actual cycles: 18 to 25; well to tank pumping rate: 800 to 1,400 gpm; tank to well pumping rate: 1,800 to 2,900 gpm).

2. Pump to waste and check specific capacity.

   a. Pump continuously for 6 or more hours to clear well of chemicals (actual time: 17.5 to 65.5 hours).
   b. Same procedure for specific capacity check as Day 1, step 1.

<u>Day 4</u> (Optional)

1. Polyphosphate application, 600 pounds, and displacement with 54,000 gallons of water containing at least 500 ppm chlorine.

   Same procedure as Day 1, step 2, except three batch injections of 1,800 gallons (5,400 gallons total) with 200 pounds phosphate each in part b, and injection of 54,000 gallons in part c.

   Noted actual pumping rates and surging cycles for indicated steps of procedure.

   a. Initial chlorination: 1,412 gpm.
   b. Polyphosphate solution injections: 2,300 to 2,700 gpm.
   c. Displacements: 1,100 to 2,600 gpm.
   d. No change.
   e. Surging/backflushing actual cycles: 25; well to tank pumping rate: 1,300 to 1,500 gpm; tank to well pumping rate: 2,400 to 3,00 gpm.

Table 3. Concluded

2. Pump to waste and check specific capacity.

   a. Pump continuously for 6 or more hours to clear well of chemicals (actual time: 14 hours).
   b. Same procedure for specific capacity check as Day 1, step 1.

<u>Day 5</u> (Optional)

1. Polyphosphate application, 400 pounds, and displacement with 16,000 gallons of water containing at least 500 ppm chlorine.

   Same procedure as Day 1, step 2.

2. Pump to waste and final specific capacity test.

   a. Pump continuously for 6 or more hours to clear well of chemicals.
   b. Same procedure for specific capacity check as Day 1, step 1.

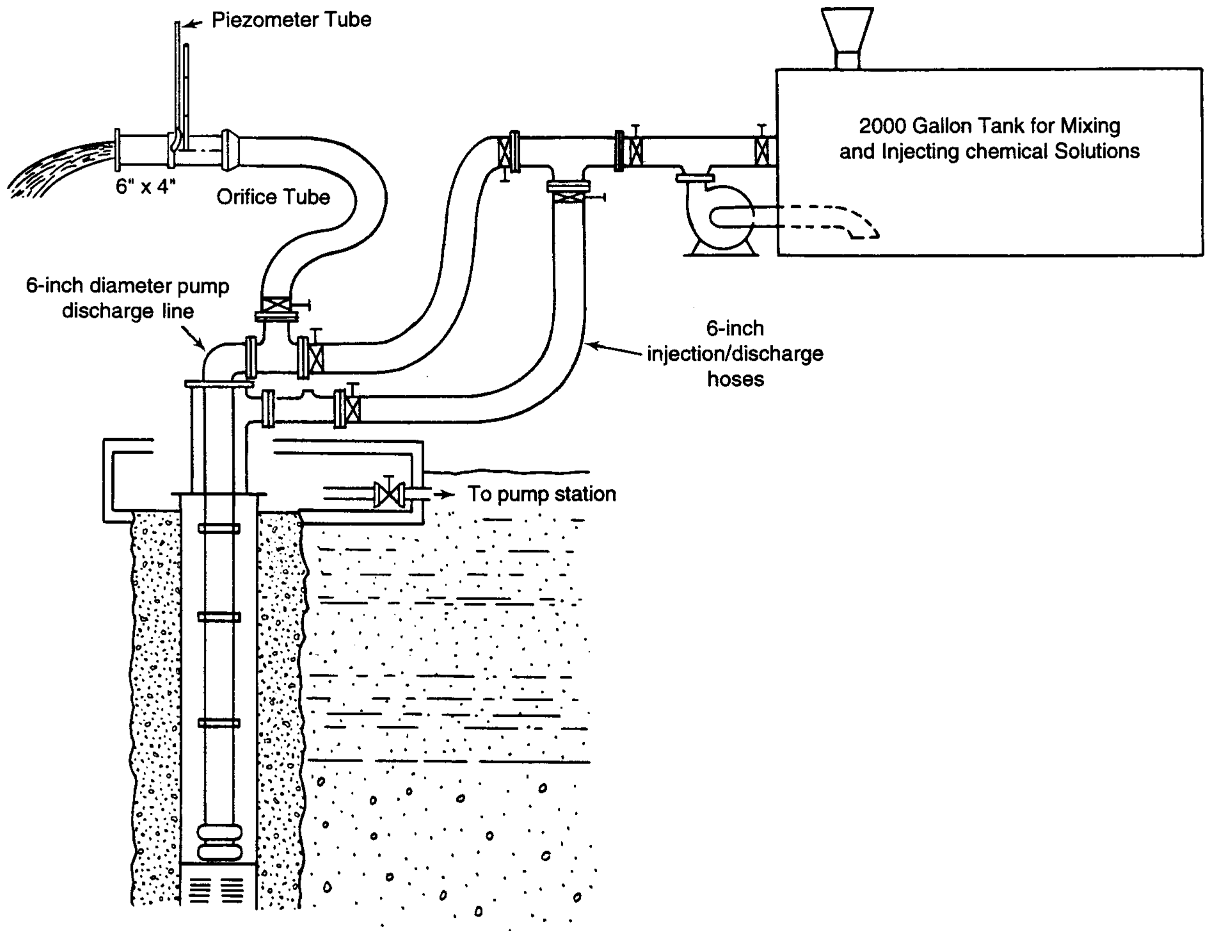


Figure 9. Schematic diagram of equipment used in well rehabilitation

specific capacity to be about 88 gpm/ft, and the Δh to be 2.4 feet. Well 2 now appears to be in good condition, with an observed specific capacity about 92 percent of the average specific capacity of wells at the Venice site in good condition.

The condition of Venice Well 3 had been checked during a step test on July 1, 1994. The observed specific capacity of the well was only about 66 gpm/ft, and the Δh was about 5 feet. Well 3 was chemically treated in September 1998. The results of the posttreatment step test conducted on Well 3 for FY 99 on November 17, 1998, showed the observed specific capacity to be about 91 gpm/ft, and the Δh to be about 3.4 feet. Well 3 now appears to be in good condition, with an observed specific capacity about 95 percent of the average specific capacity of wells at the Venice site in good condition.

The condition of Venice Well 4 had been checked during a step test on May 11, 1994. The observed specific capacity of the well was only about 45 gpm/ft. These results showed that Well 4 had deteriorated significantly since its prior step test in 1991, when the specific capacity was about 102 gpm/ft. The well was chemically treated in September 1998. The results of the posttreatment step test conducted on Well 4 for FY 99 on November 24, 1998, showed the observed specific capacity to be about 97 gpm/ft, and the Δh to be about 6.6 feet. Well 4 now appears to be in good condition, with an observed specific capacity about 101 percent of the average specific capacity of wells at the Venice site in good condition.

The condition of Venice Well 6A had been checked during a step test on November 13, 1996. The observed specific capacity of the well was only about 63 gpm/ft. Well 6A was chemically treated in September 1998. The results of the posttreatment step test conducted on Well 6A for FY 99 on September 30, 1998, showed the observed specific capacity to be about 105 gpm/ft, and the Δh to be about 2.0 feet. Well 6A appears to be in good condition, with an observed specific capacity about 109 percent of the average specific capacity of wells at the Venice site in good condition.

As indicated in table 3, the chemical treatment procedure required that the contractor conduct 60-minute drawdown tests to measure the specific capacity after each successive treatment step. Table 4 summarizes drawdown pumping test data collected as part of the field documentation during the chemical treatment of each dewatering well. Table 4 shows the measured specific capacity before treatment and after each step in the treatment process (polyphosphate or acid injection episode). The average specific capacity for all wells prior to treatment and at each of the first three steps in the treatment process is given at the end of table 4 with an analysis of the improvement between steps. In general, the percentage of improvement in specific capacity diminishes with each successive step of the treatment. This trend also has been noted in the results of the chemical treatment in some prior years. In FY 99 about 70 percent of the total improvement occurred with the first polyphosphate treatment, and about 14 percent occurred during the second polyphosphate treatment (following acidization). This trend of reduced improvement for successive treatment steps agrees well with the results of the treatment for the preceding years that this general well treatment procedure has been followed (one polyphosphate treatment, followed by a muriatic acid treatment, followed by up to three polyphosphate treatments) (Sanderson and Olson, 1999).

Table 4. Drawdown Test Data Collected by Contractor during Well Rehabilitation

	Pretreatment	*1st PPP treatment*	*Acid treatment*	*2nd PPP treatment*	*3rd PPP treatment*	*4th PPP treatment*
I-70 Well 3A						
Date (1998)	7/28	7/29	7/31	8/4	8/5	None
SWL	34.25	36.02	34.15	33.96	35.60	
PWL	53.75	52.23	50.05	49.30	51.90	
s	19.50	16.21	15.90	15.34	16.3	
Q	972	944	990	1007	1026	
Q/s	49.8	58.2	62.3	65.6	62.9	
I-70 Well 10						
Date (1998)	8/7	8/10	8/11	8/11	8/12	None
SWL	41.18	41.24	41.27	41.37	41.71	
PWL	52.40	49.95	52.64	52.7	52.73	
s	11.20	8.71	11.37	11.3	11.02	
Q	682	653	849	867	865	
Q/s	60.9	75.0	74.7	76.7	78.5	
I-64 Well 8						
Date (1998)	8/17	8/18	8/19	8/20	8/21	8/21
SWL	24.21	24.95	25.04	24.48	24.82	24.92
PWL	59.55	32.55	35.28	28.48	30.29	31.85
s	35.34	7.60	10.24	4.00	5.47	6.93
Q	1015	647	1019	457	653	828
Q/s	28.7	85.1	99.5	114.2	119.4	119.5
Venice Well 2						
Date (1998)	8/25	8/26	8/27	8/28	8/28	8/31
SWL	21.60	21.68	21.90	22.00	21.80	22.91
PWL	53.75	34.27	33.35	32.51	31.77	32.15
s	32.15	12.59	11.45	10.51	9.97	9.24
Q	889	858	880	871	856	835
Q/s	27.6	68.2	76.8	82.9	85.9	90.4
Venice Well 3						
Date (1998)	9/9	9/11	9/14	9/15	9/16	9/17
SWL	18.80	19.18	9.50	19.51	19.60	20.05
PWL	35.33	26.82	27.38	26.90	26.32	26.32
s	16.53	7.64	7.88	7.39	6.72	6.27
Q	535	543	603	607	584	557
Q/s	32.4	71.1	76.5	82.1	86.9	88.8

Table 4. Concluded

	Pretreatment	1st PPP treatment	Acid treatment	2nd PPP treatment	3rd PPP treatment	4th PPP treatment
Venice Well 4						
Date (1998)	9/1	9/3	9/4	9/8	9/9	None
SWL	19.60	19.29	18.98	19.28	19.37	
PWL	54.12	28.52	27.70	25.87	26.54	
s	34.52	9.23	8.72	6.59	7.17	
Q	490	528	659	590	641	
Q/s	14.2	57.2	75.6	89.5	89.4	
Venice Well 6A						
Date (1998)	9/17	9/21	9/22	9/23	None	None
SWL	16.75	16.98	16.97	17.00		
PWL	28.05	23.65	23.28	22.78		
s	11.30	6.67	6.31	5.78		
Q	550	622	665	625		
Q/s	48.7	93.2	105.4	108.8		
Averages						
Q/s	37.5	72.6	81.5	88.5		
ΔQ/s		35.1	9.0	7.0		
% increase over original Q/s		128.2	37.2	29.3		
% of total improvement		70.1	16.0	13.9		

Notes:

Total average ΔQ/s = 51.1 gpm/ft (194.7 percent improvement over initial Q/s)

SWL = Static (nonpumping) water level, feet
PWL = Pumping water level, feet
s = Drawdown (PWL-SWL), feet
Q = Pumping rate, gpm
Q/s = Specific capacity, gpm/ft
PPP = Polyphosphate

Table 5 summarizes the results of the posttreatment step tests conducted during FY 99 and summarizes results for comparison with the contractor's drawdown tests conducted during the well treatment.

Table 5. Results of Chemical Treatment, FY 99 (Phase 16)

			Pretreatment		*Posttreatment*		
Site	*Well*	*Test*	*Date*	*Q/s (gpm/ft)*	*Date*	*Q/s (gpm/ft)*	*% Change*
I-70	3A	ISWS	12/11/97	31.8	10/08/98	69.7	119
		LWC	07/28/98	49.8	08/05/98	62.9	26
I-70	10	ISWS	08/01/95	57.9	10/07/98	82.2	42
		LWC	08/07/98	60.9	08/12/98	78.5	29
I-64	8	ISWS	04/15/96	57.9	10/14/98	117.6	103
		LWC	08/17/98	28.7	08/21/98	119.5	316
Venice	2	ISWS	11/12/97	22.3	10/01/98	88.4	296
		LWC	08/25/98	27.6	08/31/98	90.4	228
Venice	4	ISWS	05/11/94	44.7	11/24/98	96.8	116
		LWC	09/01/98	14.2	09/09/98	89.4	530
Venice	3	ISWS	07/01/94	65.8	11/17/98	91.5	39
		LWC	09/09/98	32.4	09/17/98	88.8	174
Venice	6A	ISWS	11/13/96	63.4	09/30/98	104.8	65
		LWC	09/17/98	48.7	09/23/98	108.8	123
Average		ISWS		49.1		93.0	111
		LWC		37.5		91.2	204

Notes:
Q/s = Specific capacity, gpm/ft
ISWS = Illinois State Water Survey
LWC = Layne-Western Company, Inc.

Sand Pumpage Investigation

Field Procedure

Prior occurrences of sand pumpage from the dewatering wells resulted in the standard practice of checking for the presence of sand in the discharge during each step test, unless precluded by site conditions and available equipment. To continue to address these concerns, the possibility of sand pumpage was investigated during eight of the ten step tests conducted on ten wells in FY 99 (Phase 16). The other two wells, Missouri Avenue Wells 2 and 3, are located where the site conditions are not appropriate for the settling tank to be used.

During each step test when site conditions allowed, water was discharged from the orifice tube into a portable 1,000-gallon tank (figure 10). Siphon tubes were used, as necessary, to help control the discharge from the tank. The tank acts as a sedimentation basin that, under ideal conditions, should allow sand with grain diameters of about 0.1 millimeter (mm) and larger to settle out at the design pumping rates of the wells (600 to 800 gpm). Usually 80 to 90 percent or more of the aquifer material in the screened interval of the wells exceeds the 0.1 mm grain size.

Sand Sample Collection

Samples were collected following the step tests, whenever enough sediment remained in the tank to allow analysis of the grain size distribution. The samples were prepared and sieved at the Geotechnical Laboratory of the Illinois State Geological Survey (ISGS). In all, three of the eight step tests in which the portable tank was used generated a sample large enough for collection. Appendix C contains the data for these sample analyses. A discussion of the results for each well follows.

I-70 Well 3A:

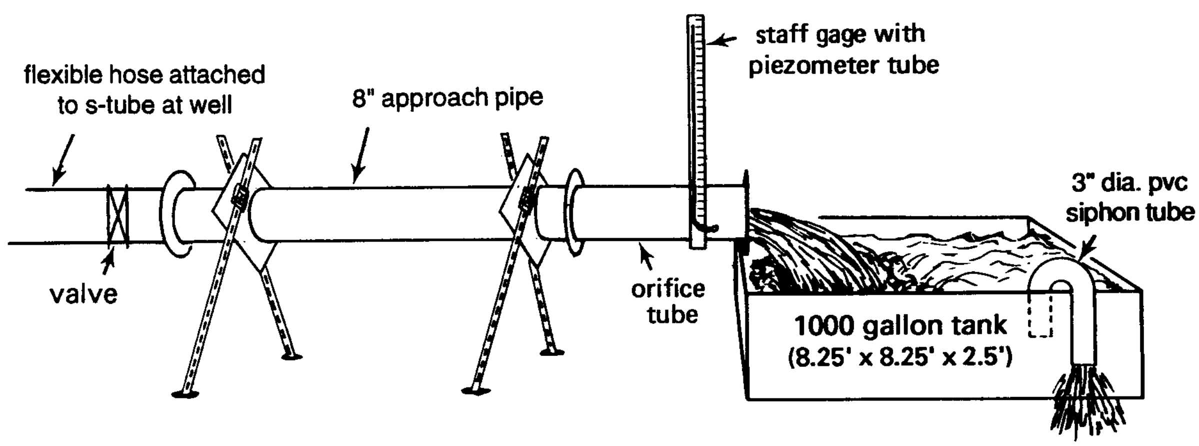
No sand was observed in the tank following the FY 99 posttreatment step test on October 8, 1998.

I-70 Well 10:

No sand was observed in the tank following the FY 99 posttreatment step test on October 7, 1998.

I-64 Well 8:

After the FY 99 posttreatment step test on this dewatering well, conducted on October 14, 1998, 18.97 grams of very fine sand, silt, and apparent gravel-pack material were observed in the portable tank. The sample collected was probably about 90 percent of the material in the settling tank; 7.27 grams of material remained following the iron extraction (acidification) process conducted prior to sieving. Approximately 50 percent (by weight) of the sample appeared to be gravel-pack material. The sieving data are included in appendix C.



SIDE VIEW

Figure 10. Sand pumpage test setup

Venice Well 2:

No material was observed in the portable sedimentation tank following the FY 99 posttreatment step test on October 1, 1998.

Venice Well 3:

After the FY 99 posttreatment step test on this dewatering well, conducted on November 17, 1998, 17.11 grams of sand-and-gravel material were collected from the portable tank. Following the iron extraction (acidification) process conducted prior to sieving, 11.86 grams of material remained. More than half of this material (about 54 percent) was approximately pea-sized, angular gravel retained on a 4.00 mm screen. The relatively large-sized material does not appear to be gravel-pack material and suggests a possible small breach in the well screen. The sieving data for the material collected from this well are included in appendix C.

Venice Well 4:

No material was observed in the portable sedimentation tank following the FY 99 posttreatment step test on November 24, 1998.

Venice Well 6A:

No material was observed in the portable sedimentation tank following the FY 99 posttreatment step test on September 30, 1998.

Missouri Avenue Well 1:

After the FY 99 posttreatment step test on this dewatering well, conducted on October 15, 1998, 170.52 grams of fine sand and iron material were collected from the portable tank. Following the iron extraction (acidification) process conducted prior to sieving, 152.89 grams of fine sand remained. The sieve data for the material collected from this well are included in appendix C.

Missouri Avenue Wells 2 and 3:

Site conditions preclude using the portable sedimentation tank during step tests for these wells.

Sand Pumpage Summary

During the FY 99 step tests in which the portable water tank could be used, three wells produced measurable amounts of sand-and-gravel material. Of these three, Missouri Well 1 produced the largest quantity (about 170 grams), compared to about 17 grams and 19 grams pumped by Venice Well 3 and I-64 Well 8, respectively. It appears that sand is being pumped from Missouri Avenue Well 1 on a continuing basis during routine operation. Sanderson and Olson (1999) describe the construction of this well and mention that Merimac gravel pack (grain size unknown) from Winter Brothers Gravel Co., St. Louis, Missouri, was used to fill the annulus between the borehole and casing-well screen assembly.

Evaluation of Ground-Water Quality

The ISWS Office of Analytical and Water Treatment Services analyzed water samples collected during all ten step tests. Appendix B reports the results. Analytical methods used conform to the latest procedures certified by the U.S. Environmental Protection Agency (1979). The sample temperature was determined in the field at each well site, and the pH of samples was determined in the laboratory. Table 6 presents the range of concentrations and potential influence of the major water quality parameters analyzed.

Although the ground-water samples vary in water chemistry, generally the ground water can be described as highly mineralized, very hard, and alkaline, with unusually high concentrations of soluble iron. The water quality is consistent with that of previously analyzed samples from the dewatering wells.

Nuisance Bacteria Sampling

Nuisance bacteria (e.g., iron bacteria, sulfate-reducing bacteria) that inhabit wells, gravel packs, and the aquifer matrix often produce well-plugging biofilms, as well as a favorable environment for chemical deposition and corrosion processes. To explore the possibility that such nuisance bacteria might be present in the dewatering wells, the Biological Activity Reaction Test (BART), developed by Droycon Bioconcepts, Inc., Regina, Saskatchewan, Canada, was run on water samples collected from the well discharge at the time of the step tests. The BART tests

Table 6. Range of Concentrations and Potential Influence of Common Dissolved Constituents, FY 99 (Phase 16)

	Concentration, mg/L		
Parameter	*Minimum*	*Maximum*	*Potential influence*
Iron (Fe)	7.81	17.71	Major - incrustative
Manganese (Mn)	0.32	1.23	Major - incrustative
Calcium (Ca)	166	268	Major - incrustative
Magnesium (Mg)	42.9	67.8	Minor - incrustative
Sodium (Na)	35.7	135	Neutral
Silica (SiO_2)	26.5	38.2	Minor - incrustative
Nitrate (NO_3)	< 0.09	0.30	Neutral
Chloride (Cl)	47.1	85	Moderate - corrosive
Sulfate (SO_4)	220	506	Major - corrosive
Alkalinity (as $CaCO_3$)	373	571	Major - incrustative
Hardness (as $CaCO_3$)	593	937	Major - incrustative
Total dissolved solids	856	1355	Major - corrosive
pH	6.9	7.2	Major - incrustative

are customized to detect three general classes of nuisance bacteria commonly associated with problems in wells: iron-related bacteria (IRB), slime-forming bacteria (SLYM), and sulfate-reducing bacteria (SRB). The BART system was used during FY 90 to identify the presence of nuisance bacteria in the I-255 Detention Pond relief wells and in conjunction with 14 step-tested dewatering wells during FY 91 (Sanderson et al., 1993), 16 step-tested dewatering wells during FY 92 (Olson and Sanderson, 1997), 12 step-tested dewatering wells during FY 93 (Sanderson and Olson, 1998), and 11 step-tested dewatering wells for FY 94 (Sanderson and Olson, 1999).

The testing protocol requires placing a water sample in a vial for examination over a period of days, and documenting any reactions that may occur. The bacterial population or activity in the water sample is inversely related to the length of time before reactions occur. Reaction types and patterns of occurrence depend on the dominant bacterial groups present in the water sample (Cullimore, 1990). Thus, the type and size of the bacterial community can be inferred from this reaction signature. Multiple sets of samples collected at time intervals of pumping are recommended for detailed analysis of the bacterial activity (Mansuy et al., 1990).

The BART samples were collected during the ten step tests conducted during FY 99, all using the same procedure. Because the purpose was to simply determine whether nuisance bacteria are present in the wells, only one sample set, consisting of IRB, SLYM, and SRB samples was collected for each step-tested well. Samples were collected from the orifice tube discharge, usually in sequence with the other water samples being collected for analysis of the dissolved constituents.

The results for most of the BART samples indicated high to moderate amounts of nuisance bacteria activity in the discharge water from all the wells. Generally, the IRB tests appeared to show more moderate aggressivity. The SLYM and SRB tests showed predominantly very aggressive biological activity (table 7).

The BART samples were collected near the end of the step tests, after many well casing and screen volumes of water were pumped, so it is assumed that the water sampled is being derived totally from the aquifer. Therefore, the rapid bacterial activity usually observed suggests that there is substantial biomass development within the well casing and screen that is slowly sloughing off during the step test pumping on most of the wells, or a significant population of the bacteria is present in the aquifer, or both.

When taking into consideration that the tops of the dewatering wells, except those at the Missouri Avenue site, are located in pits that can be readily subjected to contamination from pit seepage or spill water, the high degree of nuisance bacteria activity is not surprising. Although nuisance bacteria can be present in ground water, most of these types of bacteria are ubiquitous in the surface environment. The use of sanitary wellheads and using precautions such as disinfection after performing maintenance activities on the wells are good preventative measures for keeping the wells free of bacterially induced problems.

Table 7. Biological Aggressivity, FY 99 (Phase 16)

Site	*Well no.*	*Type of step test*	*Aggressiveness* *Iron-related bacteria (IRB)*	*Slime-forming bacteria (SLYM)*	*Sulfate reducing bacteria (SRB)*
I-70	3A	Posttreatment	3	2	2
	10	Posttreatment	3	2	2
Site average			3	2	2
I-64	8	Posttreatment	2	2	2
Venice	2	Posttreatment	4	3	2
	3	Posttreatment	2.5	3	2
	4	Posttreatment	--	2	2
	6A	Posttreatment	3	2	2
Site average			3.2	2.5	2
MO Ave.	1	Pretreatment	3	2	2
	2	Pretreatment	2.5	2	--
	3	Pretreatment	2.5	2	2
Site average			2.7	2	2
Overall average			2.8	2.2	2.0

Notes:
1 = extremely aggressive
2 = very aggressive
3 = moderately aggressive
4 = background flora
5 = negative
-- = missing data

Conclusions and Recommendations

Condition Assessments of Wells

Results of the step tests conducted to assess the condition of Missouri Avenue Wells 1, 2, and 3 show that Missouri Avenue Well 1 is in poor condition with an observed specific capacity well below the average of wells in good condition at all other sites. Missouri Avenue Well 2 is in good condition, with an observed specific capacity well above the average observed specific capacity of wells in good condition at all other sites. Missouri Avenue Well 3 also appears to be in poor condition, with observed specific capacity about one-half of the average observed specific capacity of wells in good condition at all other sites. Therefore, chemical treatment was recommended to improve the condition of Missouri Avenue Wells 1 and 3. Underwater video inspection of these wells for excessive buildup of incrusting minerals also should be considered.

Well Rehabilitations

Results of the evaluation of well rehabilitation activities range from fair to good. Evaluation of posttreatment data show specific capacities ranging from 73 to 112 percent compared to the respective site averages for wells in good condition at each site. Based on data collected by the contractor during well treatment, increases in specific capacity for individual wells range from 26 to 530 percent and averaged 203 percent. Similarly, based on pre- and posttreatment step tests conducted by the ISWS, increases in specific capacity for individual wells ranged from 39 to 296 percent and averaged 111 percent.

The change in chemical treatment specifications made in FY 90 to provide for optional polyphosphate treatment steps after the second application reduced the total number of polyphosphate treatments applied to the seven wells chemically treated during FY 99. On the basis of the field observations made at the time of the treatment, the optional third polyphosphate treatment step was omitted for Venice Well 6A, and the optional fourth polyphosphate treatment step was dropped at I-70 Wells 3A and 10 and Venice Wells 4 and 6A.

Sand Pumpage Investigations

Discharge from eight dewatering wells was examined for sand pumpage during eight of the ten step tests conducted for FY 99. For the two step tests on Missouri Avenue Wells 2 and 3, the discharge could not be checked because of site conditions. Sediment collected after three of the step tests was visually inspected for the presence of sand-and-gravel pack and sieved for the grain-size distribution. The three wells that yielded sand-and/or-gravel pack material were Missouri Avenue Well 1, Venice Well 3, and I-64 Well 8. Following iron extraction, the material samples weighed about 153 grams, 12 grams, and 7 grams, respectively, for these three wells.

Results of the tests for sand pumpage from the dewatering wells for this and prior years have yielded interesting information. The chemical treatment of some wells to restore production capacity may influence the tendency for a dewatering well to pump sand. In some instances, it appears that the treatment may cause sufficient disturbance of the gravel pack and native aquifer material to allow the well to either pump sand for some period of time after treatment or pump sand of a somewhat coarser grain size than is pumped in routine operation.

The most significant sand pumpage appears to be occurring at Missouri Avenue Well 1, and it may be occurring on a continuing basis in routine operation. As indicated earlier, the grain size of the gravel pack selected for use in this well is unknown. It is recommended that testing for the presence of sand in the discharge be continued during future step tests. This will continue to allow a qualitative assessment of the sand pumpage problem. Some of the wells may produce sand occasionally because of well development, as might occur immediately after an idle well is restarted. This can be verified as more wells are repeatedly checked during the step tests.

Nuisance Bacteria Sampling

The BART samples were collected during step tests on ten dewatering wells in FY 99, all using the same procedure. Although relatively high levels of nuisance bacteria were identified in the dewatering wells, the data clearly show that even wells in good condition contain the bacteria. Chemical treatments used to rehabilitate the wells apparently do not eliminate the nuisance bacteria from the wells. The prevalence of bacteria in the wells sampled might mean that they are indigenous to the ground water, or that they are being regularly introduced into the wells from some other source. In either case, the problems associated with their presence will need to be managed on a continual basis. It is recommended that more background data be collected using the BART sets as additional dewatering wells are step tested. Although the use of BART sets for more detailed analysis of some of the wells probably is not warranted now, it may be considered in the future.

Future Investigations

A program of continued investigation of the condition of the dewatering wells is recommended. Measuring the difference between water levels in a well and the adjacent piezometer will continue to be an important first step in determining whether or not a well is a candidate for future step tests or treatment. In addition, a well pumping sand may indicate a potentially major problem with the well. A sand pumpage investigation is recommended as a standard part of each step test.

Bibliography

Bergstrom, R. E., and T. R. Walker. 1956. *Ground-Water Geology of the East St. Louis Area, Illinois*. Illinois State Geological Survey Report of Investigation 191.

Bierschenk, W. H. 1964. *Determining Well Efficiency by Multiple Step-Drawdown Tests*. Publication 64, International Association of Scientific Hydrology, pp. 493-507.

Borch, M. A., S. A. Smith, and L. N. Noble. 1993. *Evaluation and Restoration of Water Supply Wells*. AWWA Research Foundation and American Water Works Association, Denver, CO.

Collins, M. A., and S. Richards. 1986. *Groundwater Levels and Pumpage in the East St. Louis Area, 1978-1980*. Illinois State Water Survey Circular 165.

Cullimore, D. R. 1990. *An Evaluation of the Risk of Microbial Clogging and Corrosion in Boreholes*. Water Wells Monitoring, Maintenance, Rehabilitation: Proceedings of the International Groundwater Engineering Conference, September 1990. E.& F.N. SPON, University Press, Cambridge, Great Britain, pp. 25-34.

Driscoll, F. G. 1986. *Groundwater and Wells*. Johnson Division, St. Paul, Minnesota.

Emmons, J. T. 1979. *Groundwater Levels and Pumpage in the East St. Louis Area, Illinois, 1972-1977*. Illinois State Water Survey Circular 134.

Freese, R. A., and J. A. Cherry. 1979. *Groundwater*. Prentice-Hall, Englewood Cliffs, NJ.

Jacob, C. E. 1947. Drawdown Tests to Determine Effective Radius of Artesian Well. *Transactions, American Society of Civil Engineers*, v. 72, pp. 1047-1070.

John Mathes and Associates, Inc. 1981. *Venice Subway FAUS Report 8807 Section 27T, Madison County, Illinois, Job No. P-98-041-79*. Consultant's engineering investigation report to the Division of Highways, Columbia, IL.

Johnson, Depp, and Quisenberry. 1980. *Technical Report Venice Subway Dewatering*. Job P-98-041-79. Consultant's technical report to the Division of Highways, Owensboro, KY.

Kohlhase, R. C. 1987. *Ground-Water Levels and Pumpage in the East St. Louis Area, Illinois, 1981-1985*. Illinois State Water Survey Circular 168.

Layne-Western Company, Inc. 1972. *Ground Water Drainage System F.A.I. Route 70 Tri-Level Location, East St. Louis, Illinois*. Consultant's report to the Division of Highways, St. Louis, MO.

Mansuy, N., C. Nuzman, and D. R. Cullimore. 1990. *Well Problem Identification and Its Importance in Well Rehabilitation*. Water Wells Monitoring, Maintenance, Rehabilitation: Proceedings of the International Groundwater Engineering Conference, September 1990. E.& F.N. SPON, University Press, Cambridge, Great Britain, pp. 87-99.

Mogg, J. L. 1973. Corrosion and Incrustation Guidelines for Water Wells. *Water Well Journal*, **27**(3), March.

Moss, R., Jr. 1966. *Evaluation of Materials for Water Well Casings and Screens*. Proceedings, National Association of Corrosion Engineers Western Region Meeting, October.

Olson, R. D., and E. W. Sanderson. 1997. *Dewatering Well Assessment for the Highway Drainage System at Four Sites in the East St. Louis Area, Illinois (FY 92 - Phase 9)*. Illinois State Water Survey Contract Report 613.

Olson, R. D., E. W. Sanderson, S. H. Smothers, and M. R. Schock. 1990. *Dewatering Well Assessment for the Highway Drainage System at Four Sites in the East St. Louis Area, Illinois (Phase 3)*. Illinois State Water Survey Contract Report 479.

Olson, R. D., S. D. Wilson, and E. W. Sanderson. 1992. *Dewatering Well Assessment for the Highway Drainage System at Four Sites in the East St. Louis Area, Illinois (FY 89 - Phase 6)*. Illinois State Water Survey Contract Report 540.

Ritchey, J. D., R. J. Schicht, and L. S. Weiss. 1984. *Ground Water Levels and Pumpage for the Study Ground Water Level Analysis by Computer Modeling: American Bottoms Ground Water Study*. Illinois State Water Survey Contract Report 352A.

Ritchey, J. D., R. J. Schicht, and L. S. Weiss. 1984. *The American Bottoms Digital Ground Water Flow Model for the Study Ground Water Level Analysis by Computer Modeling: American Bottoms Ground Water Study*. Illinois State Water Survey Contract Report 352B.

Ritchey, J. D., R. J. Schicht, and L. S. Weiss. 1984. *Existing and Future Ground Water Levels for the Study Ground Water Level Analysis by Computer Modeling: American Bottoms Ground Water Study*. Illinois State Water Survey Contract Report 352C.

Ritchey, J. D., R. J. Schicht, and L. S. Weiss. 1984. *Evaluation of Alternative Measures for the Study Ground Water Level Analysis by Computer Modeling: American Bottoms Ground Water Study*. Illinois State Water Survey Contract Report 352D.

Ritchey, J. D., R. J. Schicht, and L. S. Weiss. 1984. *Digital Flow Model Description and User's Guide for the Study Ground Water Level Analysis by Computer Modeling: American Bottoms Ground Water Study*. Illinois State Water Survey Contract Report 352E.

Rorabaugh, M. I. 1953. *Graphical and Theoretical Analysis of Step Drawdown Test of Artesian Well*. Proceedings, American Society of Civil Engineers, v. 79, Separate No. 362, pp. 362-1 to 362-23.

Rupani, N. 1976. *Highway I-55 & 70 and I-64 Deep Drainage System at Tri-Level Bridge in East St. Louis, Illinois*. Illinois Department of Transportation, Division of Highways file report.

Ryznar, J. W. 1944. A New Index for Determining Amount of Calcium Carbonate Scale Formed by a Water. *Journal of the American Water Works Association*, **36**(4):472-483.

Sanderson, E. W., and R. D. Olson. 1993. *Dewatering Well Assessment for the Highway Drainage System at Four Sites in the East St. Louis Area, Illinois (FY 91 - Phase 8)*. Illinois State Water Survey Contract Report 566.

Sanderson, E. W., and R. D. Olson. 1998. *Dewatering Well Assessment for the Highway Drainage System at Four Sites in the East St. Louis Area, Illinois, FY 93 (Phase 10)*. Illinois State Water Survey Contract Report 624.

Sanderson, E. W., and R. D. Olson. 1999. *Dewatering Well Assessment for the Highway Drainage System at Five Sites in the East St. Louis Area, Illinois, (FY 94 - Phase 11)*. Illinois State Water Survey Contract Report 641.

Sanderson, E. W., R. D. Olson, and S. D. Wilson. 1993. *Dewatering Well Assessment for the Highway Drainage System at Four Sites in the East St. Louis Area, Illinois (FY 90 - Phase 7)*. Illinois State Water Survey Contract Report 546.

Sanderson, E. W., and M. R. Schock. 1986. *Dewatering Well Assessment for the Highway Drainage System at Four Sites in the East St. Louis Area, Illinois*. Proceedings, Third Canadian Hydrogeological Conference, April 20-23, pp. 227-237.

Sanderson, E. W., M. R. Schock, and R. D. Olson. 1987. *Dewatering Well Assessment for the Highway Drainage System at Four Sites in the East St. Louis Area, Illinois (Phase 2)*. Illinois State Water Survey Contract Report 424.

Sanderson, E. W., A. P. Visocky, M. A. Collins, R. D. Olson, and C. H. Neff. 1984. *Dewatering Well Assessment for the Highway Drainage System at Four Sites in the East St. Louis Area (Phase 1)*. Illinois State Water Survey Contract Report 341.

Saner, J. L. 1976. *Corrosion Study of East St. Louis Dewatering Wells*. Bureau of Materials and Physical Research Report.

Schicht, R. J. 1965. *Ground-Water Development in the East St. Louis Area, Illinois*. Illinois State Water Survey Report of Investigation 51.

Schicht, R. J., and A. G. Buck. 1995. *Ground-Water Levels and Pumpage in the Metro-East Area, Illinois*. Illinois State Water Survey Circular 180.

Smith, H. F. 1954. *Gravel Packing Water Wells*. Illinois State Water Survey Circular 44.

Standard Methods for the Examination of Water and Wastewater. 1985. 16th Edition, APHA-AWWA-WPCF, Washington, D.C.

Suiden, M. T. 1989. Physical Principles of Environmental Engineering Processes. University of Illinois CE 340 course notes.

Theis, C. V. 1935. The Relation Between the Lowering of Piezometric Surface and the Rate and Duration of Discharge of a Well Using Ground-water Storage. *Transactions, American Geophysical Union*, 16th Annual Meeting, pt. 2.

U.S. Environmental Protection Agency. 1979. *Methods for Chemical Analysis of Water and Wastes*. Publication No. EPA 600/4-79-020, Cincinnati, OH.

U.S. Environmental Protection Agency. 1988. *Methods for the Determination of Organic Compounds in Drinking Water*. Publication No. EPA-600/4-88/039, Cincinnati, OH.

Viessman, W., and M. J. Hammer. 1985. *Water Supply and Pollution Control*. Harper and Row, New York, NY.

Walton, W. C. 1962. *Selected Analytical Methods for Well and Aquifer Evaluation*. Illinois State Water Survey Bulletin 49.

Wilson, S. D., R. D. Olson, and E. W. Sanderson. 1991. *Dewatering Well Assessment for the Highway Drainage System at Four Sites in the East St. Louis Area, Illinois (Phase 5)*. Illinois State Water Survey Contract Report 509.

Wilson, S. D., E. W. Sanderson, and R. D. Olson. 1990. *Dewatering Well Assessment for the Highway Drainage System at Four Sites in the East St. Louis Area, Illinois (Phase 4)*. Illinois State Water Survey Contract Report 480.

Appendix A

Dewatering Well Ground-Water Levels and Operation
FY 99 (Phase 16)

Appendix A. Dewatering Well Ground-Water Levels and Operation, FY 99 (Phase 16)

I-70 Site

Well/ Piez.		MP elev.	Temp MP	*September 30, 1998* GW elev.	Pump Δh	*December 23, 1998* GW elev.	Pump Δh	*March 29, 1999* GW elev.	Pump Δh	*June 30, 1999* GW elev.	Pump Δh
W	1A	407.7	414.8	379.1	Off	381.8	Off	383.6	Off	384.05	Off
P	1A	*								36.64	
W	2A	408.2	413.9	376.7	Off	380.1	Off	381.0	Off	382.28	Off
P	2A	*									
W	3A	402.6	407.5	364.3	Off	377.8	Off	379.7	Off	379.17	Off
P	3A	*									
W	4	389.1	396.6	360.7	On	361.5	On	362.0	On	358.38	On
P	4			Piezometer destroyed by new concrete footing for road sign.							
W	5	385.9	391.1	374.0	Off	376.9	Off	379.0	Off	376.94	Off
P	5	391.1				Plugged		Plugged		Plugged	
W	6	386.6	391.7	376.3	Off	378.6	Off	380.4	Off	379.73	Off
P	6	391.9									
W	7A	*		13.77	Off	10.90	Off	8.80	Off	8.27	Off
P	7A	*									
W	8A	*		21.41	On	18.78	On	16.63	On	16.92	On
P	8A	*		16.52		12.97		10.28		9.63	
W	9A		407.8	365.9	On	369.8	On	371.8	On	370.58	On
P	9A	407.5		369.1	3.2	372.4	2.6	374.9	3.1	374.89	4.31
W	10	401.5	410.2	371.1	Off	374.1	Off	376.4	Off	376.25	Off
P	10	409.8		Plugged		Plugged		Plugged		Plugged	
W	11A	*		44.00	On	44.79	Off	45.22	On	45.30	On
P	11A	*		33.87		30.75		27.86		27.55	
W	12A		395.8	377.0	Off	379.3	Off	381.2	Off	380.80	Off
P	12A	395.8					1.1				

Appendix A. (Continued)

I-70 Site (Concluded)

			September 30, 1998		*December 23, 1998*		*March 29, 1999*		*June 30, 1999*	
Well/ Piez.	*MP elev.*	*Temp MP*	*GW elev.*	*Pump Δh*	*GW elev.*	*Pump Δh*	*GW elev.*	*Pump Δh*	*GW elev.*	*Pump Δh*
W 13	397.0	407.0	372.2	Off	375.3	Off	377.8	Off	377.58	Off
P 13	407.2									
W 14	382.5	391.0	372.9	Off	376.2	Off	378.3	Off	368.86	On
P 14	390.8								373.93	5.07
W 15			22.67	On	22.53	On	22.61	On	29.36	On
P 15			20.12		18.59		17.39		18.93	
RW	390.6		372.1				377.9		377.60	

Appendix A. (Continued)

I-64 Site (Westbound)

Well/ Piez.	*MP elev.*	*Temp MP*	*September 30, 1998* *GW elev.*	*Pump Δh*	*December 23, 1998* *GW elev.*	*Pump Δh*	*March 30, 1999* *GW elev.*	*Pump Δh*	*June 30, 1999* *GW elev.*	*Pump Δh*
W 1	399.7	407.6	378.3	Off	380.4	Off	382.3	Off	382.54	Off
P 1	406.6									
W 2	397.1	402.1	382.2	Off	383.5	Off	385.1	Off	385.33	Off
P 2	401.5									
W 3	394.6	402.1	383.9	Off	384.9	Off	386.4	Off	386.71	Off
P 3	400.0									
W 4	394.0	400.2	384.9	Off	385.7	Off	387.1	Off	387.45	Off
P 4	399.4									
W 5	396.5	401.1	385.9	Off	386.4	Off	387.7	Off	388.12	Off
P 5	400.2									
W 6	394.3	400.2	386.6	Off	386.8	Off	388.1	Off	388.60	Off
P 6	399.9									
W 7	392.2	398.0	386.6	Off	386.6	Off	387.9	Off	388.4	Off
P 7	397.6									
W 8	396.7	405.5	385.8	Off	385.7	Off	386.8	Off	387.4	Off
P 8	404.9		Plugged				Plugged		Plugged	
W 9	391.4	397.4	378.1	On?	375.3	On	375.7	On	375.3	On
P 9	397.0		382.0	3.9	381.8	6.5	382.4	6.7	382.5	7.2
W 10	395.4	404.7	387.0	Off	386.8	Off	387.8	Off	388.47	Off
P 10	404.6									
RW 1	403.0		384.4		385.4 (1/6/99)		386.7		387.07	

Appendix A. (Continued)

I-64 Site (Eastbound)

			September 30, 1998		*December 23, 1998*		*March 30, 1999*		*June 30, 1999*	
Well/ Piez.	*MP elev.*	*Temp MP*	*GW elev.*	*Pump Δh*	*GW elev.*	*Pump Δh*	*GW elev.*	*Pump Δh*	*GW elev.*	*Pump Δh*
W 11	397.0	402.8	381.9	Off	383.3	Off	384.9	Off	385.36	Off
P 11	402.5									
W 12	395.2	401.6	383.4	Off	384.4	Off	386.0	Off	386.41	Off
P 12	401.5									
W 13	394.3	399.1	384.7	Off	385.5	Off	386.9	Off	387.41	Off
P 13	399.1									
W 14	396.0	400.5	385.7	Off	386.2	Off	387.6	Off	388.10	Off
P 14	399.7									
W 15	395.1	400.5	386.6	Off	386.9	Off	388.1	Off	388.66	Off
P 15	399.7									
W 16	393.7	399.8	386.8	Off	386.9	Off	388.1	Off	388.74	Off
P 16	398.8									
W 17	392.1	398.0	386.5	Off	386.5	Off	387.6	Off	388.28	Off
P 17	397.8									
W 18	391.3	396.6	385.6	Off	385.4	Off	386.5	Off	387.21	Off
P 18	396.4									
W 19	391.8	397.0	377.6	On	375.6	On	373.8	On	371.17	On
P 19	397.0		382.2	4.7	382.3	6.7	383.4	9.6	383.02	7.85
W 20	395.4	405.3	388.2	Off	387.9	Off	389.0	Off	389.64	Off
P 20	404.7									
RW 2	398.2		384.8		384.7 (1/6/00)		385.7		386.35	

Appendix A. (Continued)

25th Street Site

Well/ Piez.	MP elev.	Temp MP	October 1, 1998		December 22, 1998		March 30, 1999		July 1, 1999	
			GW elev.	*Pump Δh*	*GW elev.*	*Pump Δh*	*GW elev.*	*Pump Δh*	*GW elev.*	*Pump Δh*
W 1	399.7	407.4	391.8	Off	391.5	Off	392.9	Off	393.60	Off
P 1	407.3									
W 2	394.6	402.8	379.2	On	377.5	On	376.8	On	373.71	On
P 2	401.9		381.2	2.0	380.2	2.7	380.3	3.5	378.03	4.32
W 3	390.4	400.3	390.1	Off	390.0	Off	391.5	Off	391.37	Off
P 3	400.2									
W 4	392.4	401.6	378.3	On	379.9	On	382.8	On	385.34	On
P 4	401.5		Plugged		Plugged		Plugged		Plugged	
W 5	396.2	404.2	384.6	On	383.6	On	384.7	On	384.90	On
P 5	403.8		Plugged		Plugged		Plugged		Plugged	
W 6	396.5	405.4	390.8	Off	390.7	**	392.1	**	392.97	**
P 6	404.5									
W 7	392.6	402.9	372.0	On	371.3	On	371.3	On	372.0	On
P 7	402.0		Plugged		Plugged		Plugged		Plugged	
W 8	390.8	401.0	377.9	On	377.2		378.3	On	380.39	On
P 8	400.5		386.5	8.6	386.5	9.3	387.7	9.4	388.60	8.21
W 9	409.4	414.5	385.2	On	384.8	On	385.7	On	386.03	On
P 9	414.7		391.0	5.8	390.8	6.0	392.1	6.4	392.88	6.85
W 10	398.6	407.5	392.3	Off	392.1	Off	393.5	Off	394.23	Off
P 10	406.1									
RW	401.4		391.1		390.9		392.2		393.03	

Appendix A. (Continued)

Venice Site

			September 30, 1998		*December 22, 1998*		*March 29, 1999*		*July 1, 1999*	
Well/ Piez.	*MP elev.*	*Temp MP*	*GW elev.*	*Pump Δh*	*GW elev.*	*Pump Δh*	*GW elev.*	*Pump Δh*	*GW elev.*	*Pump Δh*
W 1	405.6	411.6	384.4	On	386.4	On	387.3	On	390.10	On
P 1	411.2		Plugged		Plugged		Plugged		Plugged	
W 2	405.6	411.0	390.9	Off	386.7	On	387.6	On	390.61	On
P 2	410.3		396.2	0.8	388.7	2.0	389.8	2.2	392.47	1.86
W 3	402.6	408.6	387.5	On	392.7	Off	394.5	Off	392.62	On
P 3	408.4		389.8	2.3					Plugged	
W 4	403.1	408.1	391.4	Off	393.2	Off	394.7	Off	395.59	Off
P 4	407.2									
W 5	401.1	407.4	381.2	On	380.5	On	393.9	On	384.13	On
P 5	407.2		Plugged		Plugged		Plugged		Plugged	1.0
W 6A	400.8	408.4	391.3	Off	390.0	On	390.3	On	393.50	On
P 6A	408.6		Damaged/plugged		Damaged/plugged		Damaged/plugged		Damaged/plugged	
W 7	399.3	407.5	375.8	On	385.6	Off	387.0	Off	381.86	On
P 7	409.1		Plugged		Plugged		Plugged		Plugged	
RW	407.3		390.8		390.4		392.9		394.58	

Appendix A. (Concluded)

Missouri Avenue Site

Well/ Piez.		MP elev.	Temp MP	September 30, 1998		December 23, 1998		March 29, 1999		June 30, 1999	
				GW elev.	*Pump Δh*	*GW elev.*	*Pump Δh*	*GW elev.*	*Pump Δh*	*GW elev.*	*Pump Δh*
W	1	408.72		371.9	On	377.4	On	379.5	On	361.75	On
W	2	317.63		381.6	On	382.1	On	383.9	On	386.92	On
W	3	415.44		374.1	On	374.9	On	375.2	On	372.66	On
P	2-93			381.2	7.1	382.5	7.6	384.4	9.2	386.71	14.06
OW	1	416.75		385.5		Piez. damaged		Piez. damaged		391.40	
OW	2	418.67		Plugged		Plugged				Piez. damaged	
OW	3	402.49		387.7		389.0		390.0		392.73	

Notes:
* Measuring point elevations not available; depths to water recorded
** Pump removed from well
GW elev. = ground-water elevation
MP elev. = measuring point elevation
OW = observation well
P or piez. = piezometer
Pump = pump operation status
RW = recorder well
Temp MP = elevation of temporary measuring point
W = well
? Status uncertain/not verified
Δh = difference in ground-water elevation between well and piezometer

Appendix B

Chemical Quality of Ground Water from Dewatering Wells
FY 99 (Phase 16)

Appendix B. Chemical Quality Data, FY 99 (Phase 16)

Well	*Date*	*Lab No.*	*Iron*	*Manganese*	*Calcium*	*Magnesium*	*Sodium*	*Silica*	*Nitrate*	*Chloride*	*Sulfate*	*Alkalinity*	*Hardness*	*TDS*
I-70 Site														
3A	10/8/1998	231012	14.22	0.89	197	46.7	50.6	32.3	<0.09	85.0	312	373	684	1010
10	10/7/1998	231011	12.20	0.57	195	45.9	135	35.7	<0.09	75.2	393	483	675	1226
I-64 Site														
8	10/14/1998	231014	13.86	0.66	264	67.8	39.7	35.0	<0.09	84.4	506	413	937	1355
Venice Site														
2	10/1/1998	230990	17.71	0.70	205	42.9	36.1	36.6	<0.09	71.1	221	463	688	924
3	11/17/1998	231073	15.09	0.46	193	48.5	42.6	33.3	<0.09	71.3	244	423	681	937
4	11/24/1998	231098	16.85	0.54	196	49.3	36.3	35.8	<0.09	67.0	220	447	692	869
6A	9/30/1998	230991	8.75	0.32	166	43.7	35.7	38.2	<0.09	47.1	226	401	593	856
Missouri Avenue Site														
1	10/16/1998	231013	7.81	1.15	224	44.1	74.6	26.5	0.30	83.7	327	457	740	1104
2	12/2/1998	231100	12.62	1.23	268	55.2	76.7	31.0	<0.09	73.9	339	571	896	1190
3	12/1/1998	231099	12.00	0.88	208	44.2	71.0	31.7	<0.09	57.2	298	457	701	981

Notes:

TDS - Total dissolved solids

All chemical concentration data units are in mg/L

* - Reported as calcium carbonate ($CaCO_3$)

Appendix B. Chemical Quality Data (Continued)

Fluoride	*Aluminum*	*Arsenic*	*Barium*	*Beryllium*	*Boron*	*Cadmium*	*Chromium*	*Copper*	*Lead*	*Mercury*	*Nickel*	*Potassium*	*Selenium*	*Silver*	*Zinc*	*Well*
0.2	0.09	<0.11	0.09		0.48	<0.017	<0.007	<0.01	<0.066		<0.031	8.6	<0.18		<0.02	3A
0.3	0.03	<0.11	0.06		0.83	<0.017	<0.007	<0.01	<0.066		<0.031	10.9	<0.18		<0.02	10
0.2	0.03	<0.11	0.02		0.56	<0.017	<0.007	<0.01	<0.066		<0.031	7.4	<0.18		<0.02	8
0.2	0.08	<0.11	0.15		1.46	<0.017	<0.007	<0.01	<0.066		<0.031	7.2	<0.18		<0.02	2
0.2	<0.02	<0.11	0.09		0.88	<0.017	<0.007	<0.01	<0.066		<0.031	5.3	<0.18		<0.02	3
0.3	0.03	<0.11	0.12		0.67	<0.017	<0.007	<0.01	<0.066		<0.031	9.4	<0.18		<0.02	4
0.3	0.06	<0.11	0.05		0.60	<0.017	<0.007	<0.01	<0.066		<0.031	6.9	<0.18		<0.02	6A
0.3	0.04	<0.11	0.11		0.96	<0.017	<0.007	<0.01	<0.066		<0.031	10.9	<0.18		<0.02	1
0.3	<0.02	<0.11	0.13		1.16	<0.017	<0.007	<0.01	<0.066		<0.031	18.3	<0.18		<0.02	2
0.3	0.05	<0.11	0.11		1.04	<0.017	0.010	<0.01	<0.066		<0.031	17.8	<0.18		<0.02	3

Appendix C

Sieve Data for Material Pumped from Dewatering Wells
FY 99 (Phase 16)

ISGS GEOTECHNICAL LABORATORY
PARTICLE SIZE CALCULATION WORKSHEET

Sample Name = I-64

Location = Well No. 8

Job/Lab # = J1030/11934

Total sand weight = 18.97

Sand split weight after FE extraction= 7.27

Screen Tare Wt	Screen Gross Wt	Screen (mm)	Weight Retained	Cum. Weight	Cum. Percent	Percent Finer
105.93	106.63	2.00	0.70	0.70	9.63	90.37
98.59	101.18	1.00	2.59	3.29	45.25	54.75
90.47	91.30	0.500	0.83	4.12	56.67	43.33
89.12	89.59	0.355	0.47	4.59	63.14	36.86
85.66	86.22	0.250	0.56	5.15	70.84	29.16
82.78	83.99	0.125	1.21	6.36	87.48	12.52
80.67	81.42	0.063	0.75	7.11	97.80	2.20
71.70	71.89	Pan	0.19	7.30	100.41	-0.41

Grams Retained=	7.30
Grams loss/gain=	0.03
Percent loss/gain=	0.41

NOTE: All weights are measured in grams

Tested on 3" sieve set

ISGS GEOTECHNICAL LABORATORY
PARTICLE SIZE CALCULATION WORKSHEET

Sample Name = MO Ave

Location = Well No. 1

Job/Lab # = J1030/11932

Total sand weight = 170.52

Sand split weight after FE extraction= 152.89

Screen Tare Wt	Screen Gross Wt	Screen (mm)	Weight Retained	Cum. Weight	Cum. Percent	Percent Finer
444.23	444.66	1.00	0.43	0.43	0.28	99.72
306.88	315.78	0.500	8.90	9.33	6.10	93.90
294.44	318.74	0.355	24.30	33.63	22.00	78.00
280.35	336.31	0.250	55.96	89.59	58.60	41.40
262.80	307.58	0.180	44.78	134.37	87.89	12.11
313.78	329.11	0.125	15.33	149.70	97.91	2.09
256.72	259.42	0.090	2.70	152.40	99.68	0.32
246.12	246.53	0.063	0.41	152.81	99.95	0.05
364.00	364.31	Pan	0.31	153.12	100.15	-0.15

Grams Retained=	153.12
Grams loss/gain=	0.23
Percent loss/gain=	0.15

NOTE: All weights are measured in grams

Specific gravity (average of 5 tests) = 2.63

ISGS GEOTECHNICAL LABORATORY
PARTICLE SIZE CALCULATION WORKSHEET

Sample Name = Venice Well

Location = Well No. 3

Job/Lab # = J1030/11933

Total sand weight = 17.11

Sand split weight after FE extraction= 11.86

Screen Tare Wt	Screen Gross Wt	Screen (mm)	Weight Retained	Cum. Weight	Cum. Percent	Percent Finer
115.42	121.87	4.00	6.45	6.45	54.38	45.62
105.93	107.62	2.00	1.69	8.14	68.63	31.37
98.59	99.49	1.00	0.90	9.04	76.22	23.78
90.47	91.76	0.500	1.29	10.33	87.10	12.90
89.12	89.84	0.355	0.72	11.05	93.17	6.83
85.66	86.18	0.250	0.52	11.57	97.55	2.45
82.78	83.11	0.125	0.33	11.90	100.34	-0.34
80.67	80.75	0.063	0.08	11.98	101.01	-1.01
71.70	71.76	Pan	0.06	12.04	101.52	-1.52

Grams Retained=	12.04
Grams loss/gain=	0.18
Percent loss/gain=	1.52

NOTE: All weights are measured in grams

Tested on 3" sieve set

Illinois State

WATER

Survey (1895)

ILLINOIS

DEPARTMENT OF NATURAL RESOURCES

ILLINOIS