Illinois State Water Survey Division
GROUND-WATER SECTION

SWS Contract Report 479

DEWATERING WELL ASSESSMENT FOR THE HIGHWAY DRAINAGE SYSTEM AT FOUR SITES IN THE EAST ST. LOUIS AREA, ILLINOIS (PHASE 3)

by Robert D. Olson, Ellis W. Sanderson,
Sarah H. Smothers, and Michael R. Schock

Prepared for the
Illinois Department of Transportation

Champaign, Illinois
January 1990

Illinois Department of Energy and Natural Resources

DEWATERING WELL ASSESSMENT FOR THE HIGHWAY DRAINAGE SYSTEM

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

(PHASE 3)

by Robert D. Olson, Ellis W. Sanderson,
Sarah H. Smothers, and Michael R. Schock

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

Prepared for the
Illinois Department of Transportation

January 1990

CONTENTS

DEWATERING WELL ASSESSMENT FOR THE HIGHWAY DRAINAGE SYSTEM AT FOUR SITES IN THE EAST ST. LOUIS AREA, ILLINOIS (PHASE 3)

by Robert D. Olson, Ellis W. Sanderson,
Sarah H. Smothers, and Michael R. Schock

ABSTRACT

In the East St. Louis vicinity, the Illinois Department of Transportation (IDOT) owns 48 wells that are used to maintain the elevation of the ground-water table below the highway surface in areas where the highway is depressed below the original land surface. The dewatering systems are located at four sites in the alluvial valley of the Mississippi River in an area known as the American Bottoms. At the dewatering sites the alluvial deposits are about 90-115 feet thick and consist of fine sand, silt, and clay in the upper 10-30 feet underlain by medium to coarse sand about 70-100 feet thick.

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, a cooperative investigation has been conducted by IDOT and the State Water Survey to more adequately assess the condition of selected individual wells, and to begin an attempt to understand the probable causes of well deterioration. Phase 3 work has established the condition of six dewatering wells, monitored and evaluated the rehabilitative treatment of seven dewatering wells, and concluded the detailed investigation of probable ground-water chemistry changes as the water moves toward a pumping well.

During Phase 1, fourteen wells were field-tested by conducting step tests to determine the response of the wells at various rates of pumping. Most of the tested wells were in relatively good condition. Based upon the analysis of the step-test data, four wells were recommended for treatment and one well for replacement. During Phase 2, twelve additional wells were field-tested with step tests. Most of these wells were in relatively good condition. Based upon well losses of 10 to 29% of total drawdown, specific capacities of 33 to 44 gallons per minute per foot (gpm/ft) of drawdown, and head differences between the wells and their adjacent piezometers of 10 to 13 feet, four wells were recommended for treatment.

During Phase 3, six additional wells were step-tested and found to be in relatively good condition with specific capacities ranging from 62.8 to 136.4 gpm/ft of drawdown and well losses at 15% or less. Head differences between the wells and piezometers were also acceptable, about 5.2 feet or less. Therefore none of these wells appears to require treatment at this time, although one probably would be improved with treatment and several others should be monitored closely.

Rehabilitation of seven dewatering wells was monitored and evaluated for improvement in the condition of the wells. Results of the rehabilitation work were encouraging, with an average improvement in the

specific capacity of almost 100%. Well loss drawdown decreased from an average of 12.5% of total drawdown to 6.4%, and average head differences between the wells and piezometers were reduced from 9.6 to 2.7 feet. It appears that much of the blockage causing the deteriorated condition of the wells occurs within a few feet of the well. The data from the treatment monitoring also suggest that one or two of the last polyphosphate applications might be eliminated in the future without much reduction in the total improvement of the wells. However, because of the small sample of treated wells, this probably should be confirmed following treatment and evaluation of additional wells prior to implementation.

The ground-water chemistry study was continued in this phase with the addition of two new sampling points in the pumped well and four new monitoring wells. Although some of the problems encountered in Phase 2 were resolved, the inability to model the flow field in sufficient detail and difficulties with the chemical sampling kept the study results and conclusions from progressing as far as originally intended.

Results of the geochemical modeling indicate that periodic scaling and incrustation of the pump, well screen, gravel pack, and aquifer material next to the well are inevitable. There are two plausible reasons for this. The first reason is the delicate equilibrium relationships between the dissolved constituents (Fe(II), calcium, magnesium, etc.), pH, inorganic carbonate concentration, and carbon dioxide gas partial pressure, which can easily be disturbed by turbulence or pressure drops such as those caused by pumpage and water moving through the well screen. Such disturbance can then allow mineral deposition. The second reason is the oxidizing and reducing conditions in the gravel pack and aquifer near the wells, which alternate with the pumping/nonpumping episodes of the wells and ultimately result in the precipitation of ferric oxyhydroxide. Measurements of dissolved iron and redox potential (Eh) also indicate the likely oxidation of iron if the water gets mixed with air in the well at the air/water interface or during pumping. When well rehabilitation is necessary, inhibited acids and iron sequestering agents should be considered for use.

Recommendations for further study for this recently completed investigation (Phase 3) include suggestions to assess the condition of additional wells and to continue to monitor and study the well rehabilitation work.

INTRODUCTION

Background

The Illinois Department of Transportation (IDOT) operates 48 high-capacity water wells at four 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 I-70/55 and I-64 was originally designed, ground-water levels were at lower elevations because of large withdrawals by the area's industry. Due to a combination of water conservation, production cutbacks, and conversion from ground water to river water as a source, ground-water withdrawals by industry have decreased about 50% since 1970, and as a result, ground-water levels in many areas have recovered to early development levels. This exacerbates IDOT's need to dewater the areas of depressed highways.

Scope of Study

The Illinois Department of Transportation first installed 12 dewatering wells in 1973, followed by an additional 30 in 1975. By 1977, the initial 12 wells were showing signs of loss of capacity. As a result, all 42 wells in use at that time were chemically treated to restore capacity. Although good results were obtained on most of the wells, routine monitoring by IDOT showed that deterioration problems were continuing to develop. Chemical treatment of isolated wells was made by IDOT personnel as required. In 1982, six more wells were installed. In October 1982, IDOT asked the Illinois State Water Survey (ISWS) to begin an investigative study of the dewatering wells to learn more about their condition, to determine efficient monitoring and operating procedures, and to determine suitable methods of rehabilitation. Two phases of work were done in 1983 and 1984 that included an assessment of the condition of 26 selected wells, a review of IDOT's monitoring program, a model study to outline efficient operating schemes, recommendations on wells to be treated, recommendations for chemical rehabilitative treatment procedures, testing of a portable flowmeter, and an initial study of the chemistry of the ground water as it moves toward an operating well. The third phase of the work, begun in 1985, includes an assessment of the condition of six selected wells; demonstration of a non-invasive, portable flowmeter; continued study of the chemistry of the ground water as it moves toward an operating well; and documentation of the rehabilitation treatments along with follow-up condition assessments performed on each of seven wells.

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 (see figure 1). The geology of the area consists of alluvial deposits overlying limestone and dolomite of the Mississippian and Pennsylvanian Ages. 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). Ground water generally flows from the bluffs toward the river, except where diverted by pumpage or drainage systems.

Detailed locations of the four dewatering sites operated by IDOT are shown in figures 2 and 3. The geology at these sites is consistent with regionally mapped conditions. The land surface lies at about 410-415 feet above mean sea level (ft msl). The alluvial deposits are about 90-115 ft thick, meaning the bedrock surface lies at approximately 300-320 ft msl.

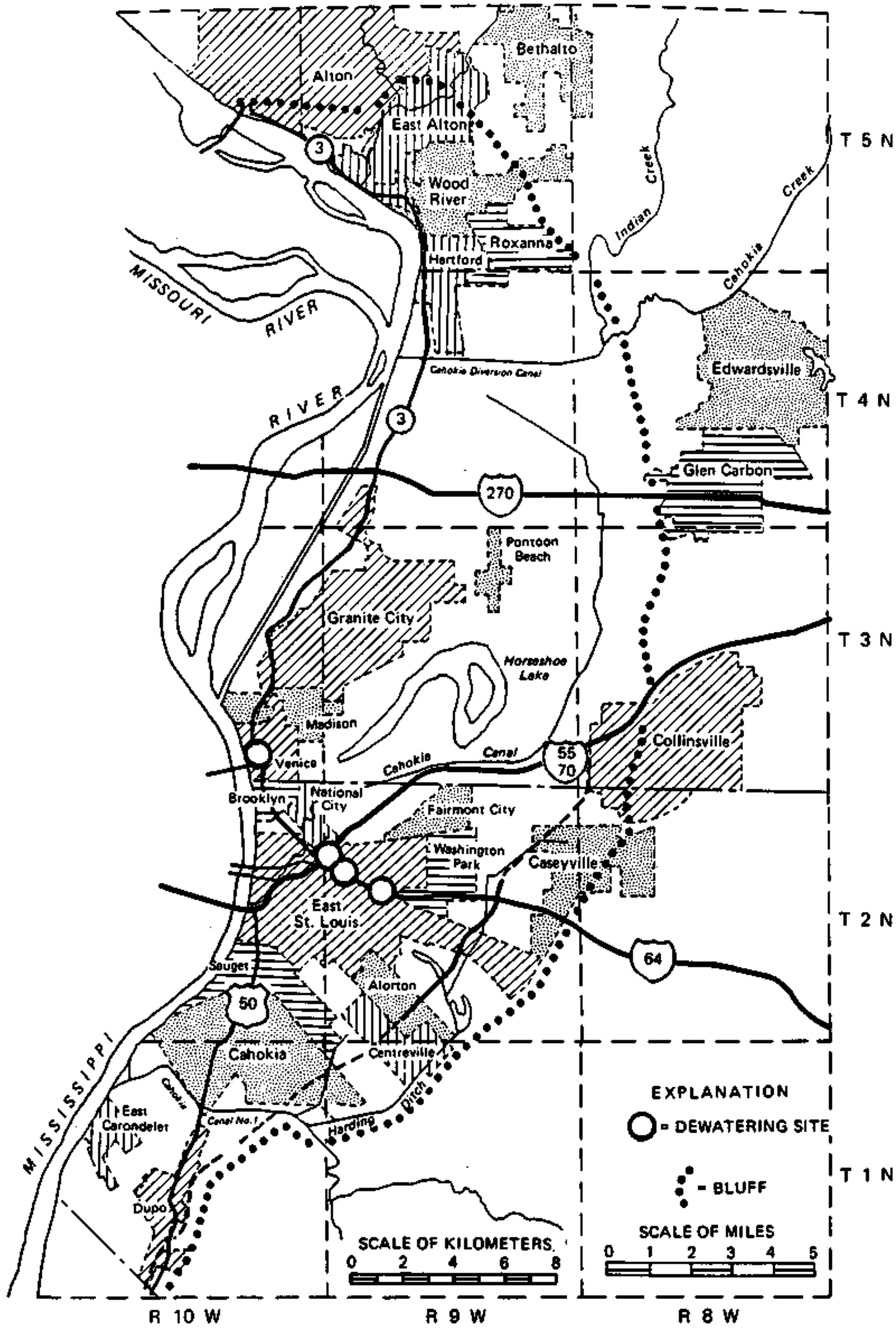


Figure 1. Location of the East St. Louis area

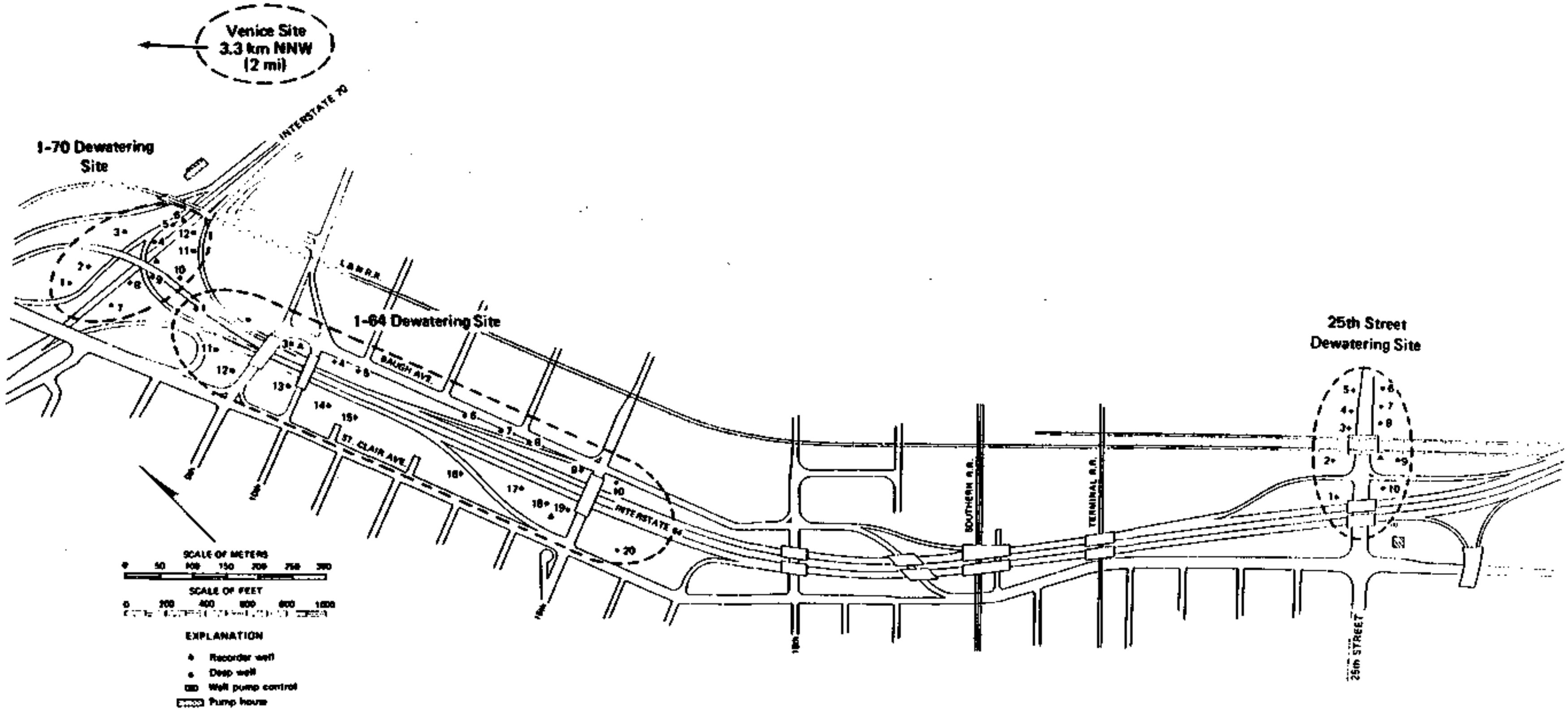


Figure 2. Locations of dewatering wells at the I-70 Tri-Level Bridge, I-64, and 25th Street

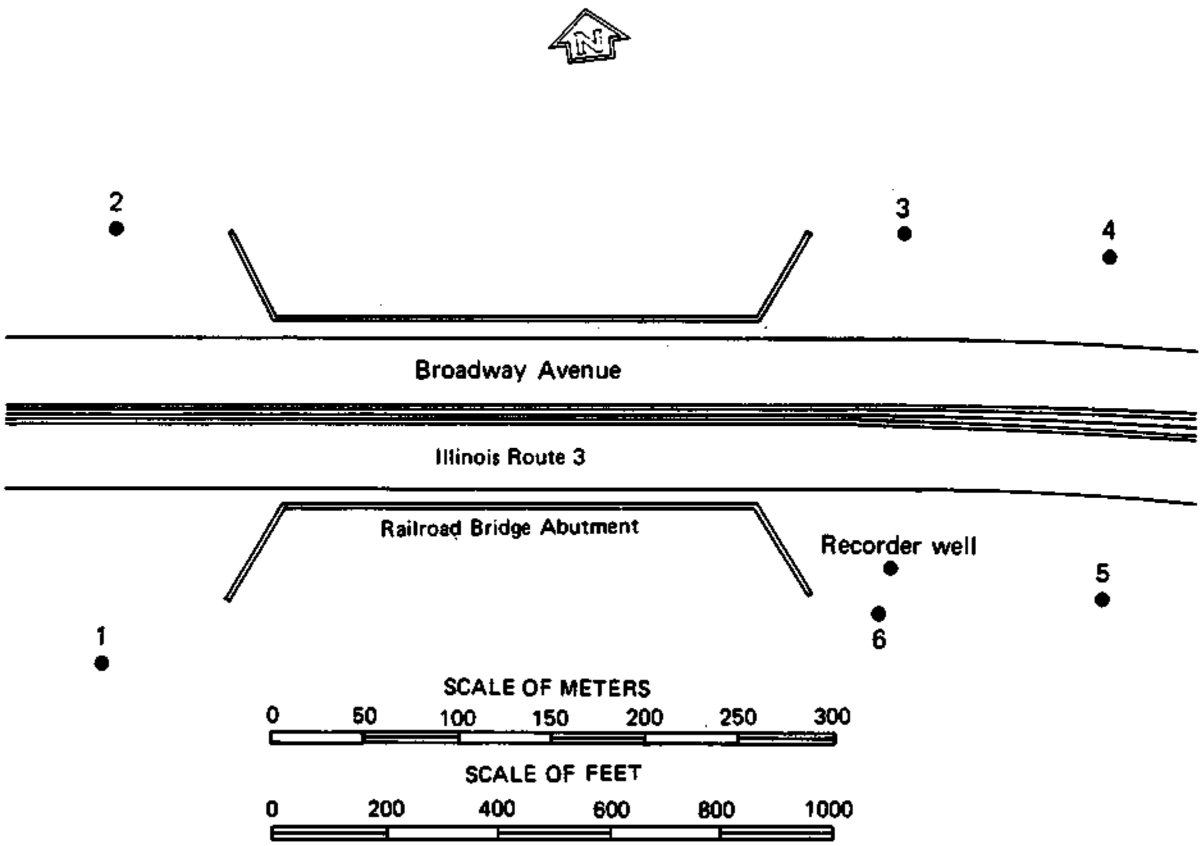


Figure 3. Locations of dewatering wells at the Venice Subway (Illinois Route 3)

The alluvium becomes progressively coarser with depth. The uppermost 10 to 30 feet consists of extremely fine sand, silt, and clay, underlain by the aquifer, which is about 70-100 feet thick. The elevation of the top of the aquifer is about 390-395 ft msl.

Acknowledgments

This phase of the assessment of the condition of the highway dewatering well systems in the American Bottoms was funded by the Illinois Department of Transportation, John D. Kramer, Secretary. Special thanks are due Frank Opfer, Hydraulic Engineer, District 8, who reviewed and coordinated the investigation. Robert Nebblesink, Supervisor of District 8's drilling crew, coordinated the construction of the four sampling wells used in the chemistry study. The Maintenance Division Pump Crew under the supervision of Stan Gregowicz provided field support during the conduct of step-drawdown tests on the selected wells. State Water Survey Ground-Water and Aquatic Chemistry Section staff who ably assisted the authors in collecting field data and water samples included Robert Kohlhase, Adrian Visocky, Paul Jahn, Stuart Cravens, Jeffrey Stollhans, Ed Garske, Kent Smothers, Len Patrick, Mark Hampton, Steve Hanson, David Cartwright, Mark Sievers, and Steve Wilson.

Analytical work was done by Ed Garske, Sarah Smothers, Kent Smothers, and Osia Smith, supported by additional laboratory effort by the Analytical Chemistry Lab Unit under the direction of Mark Peden. Manuscript editing was done by Gail Taylor, and the illustrations were prepared by John Brother, Jr., and Linda Riggin. Word processing was done by Pamela Lovett, Ground-Water Section secretary.

HISTORICAL SUMMARY OF DEWATERING DEVELOPMENT

The eastbound lanes of Interstate 70 (I-70) below the Tri-Level Bridge between St. Clair and Bowman Avenues in East St. Louis dip down to elevation 383.5, or approximately 32 feet below natural ground surface. At the time of highway design in 1958, the ground-water levels were near an elevation of 390, or about 6.5 feet above the planned highway (McClelland Engineers, Inc., 1971).

Horizontal Drain System

A horizontal French drain system was designed for controlling the ground-water levels along an 800-foot reach of depressed highway. For highway construction, the excavation area was dewatered by pumping from seven wells 100 feet deep and 16 inches in diameter. The wells were equipped with 1800-gallon-per-minute (gpm) turbine pumps. The construction dewatering system was designed to maintain the ground-water level at the site near elevation 370.

The French drain system failed shortly after the construction dewatering system was turned off in the fall of 1962. The failure was attributed to the fact that the filter sand around the perforated diagonal drains and collector pipes was too fine for the 1/4-inch holes in the drain pipes. A sieve analysis on the filter sand showed that 98.5% of the filter sand was finer than the 1/4-inch perforations in the drain pipes. As a result, when the construction dewatering system was turned off and ground-water levels rose above the drains, filter sand migrated through the holes into the drain pipes. After the filter sand migrated into the drain, the very fine "sugar" sand used as the pavement foundation was free to move downward to the drains, resulting in development of potholes above the drains. Further migration of sand into the French drainage system was halted by operating the construction dewatering system to lower the ground-water table. Since it was very likely that the foundation sands had piped from beneath the pavement, the diagonal drains beneath the pavement were cement-grouted to prevent any future loss of support beneath the pavement (McClelland Engineers, Inc., 1971).

Horizontal and Vertical Well Drainage System

A new drainage system was designed and installed in early 1963. It consisted of 20 vertical wells and 10-inch- to 12-inch-diameter horizontal drain pipes. The 20 wells (10 wells on each side of the highway) were spaced about 75 feet apart. They were 6 inches in diameter, about 50 feet deep, and equipped with 32 feet of stainless steel well screen (Doerr) with 0.010-inch slots. The horizontal drains were sized for a flow of about 1 gpm per foot of drain, were perforated with 3/8-inch-diameter holes on 3-inch centers, and were surrounded with 6 inches of gravel and sand filter. A total of six 2-inch-diameter piezometers were installed for ground-water level measurements.

Tests immediately after the installation indicated that the new system was performing satisfactorily with a discharge of about 1200 to 2000 gpm compared to a computed design flow of 4500 gpm. Ground-water levels were lowered to elevation 375.5±, about 2 feet below the design ground-water elevation of 377.5, or about 8 feet below the top of the concrete pavement.

The system performed efficiently until March 1965, when a gradual rise in ground-water levels was detected. By July 1967 a rise of 1 foot had occurred, and from July 1967 to April 1969 an additional 4-foot rise was observed. No additional rise was observed between August 1969 and August 1970.

Visual inspection during the late 1960s revealed some sinking of the asphalt shoulders and areas around the storm drainage inlets. Several breaks and/or blockages of the horizontal transit drain pipes were noted on both sides of the pavement, and a break in the steel tee in Well 17 was also observed. Depressions in the earth slopes immediately adjacent to the curb and gutter section were noticed. Loss of foundation sands through the transit pipe breaks appeared to be the cause of these depressions. One manhole had settled a total of 15 inches. The attempt to correct this condition was suspended with the detection of a shift in the bottom of this manhole.

A thorough field investigation was begun to correct the damages to the underground system or to replace it if necessary. During the cleaning process of the collector pipes (using a hydrojet at the rate of 100 gpm under a pressure of about 800 pounds per square inch), a significant amount of scale was removed from inside the mild steel pipes, indicating serious corrosion. Nearly all the transit drain pipes also showed signs of stress. Some drains were broken and filled with sand. Attempts to clean or restore the drain pipes were abandoned in favor of a complete replacement of the system.

The field investigation also showed that the tees in the manholes, the collector pipes, and the aluminum rods on the check valves were badly corroded. Sinks, potholes, and general settlement of the shoulders indicated a distressed condition requiring immediate attention. Television inspection of the vertical wells showed no damage to the stainless steel well screens.

Excessive corrosion of the mild steel tees, well risers, and collector pipes was one of the major causes or contributors to the overall failure of the drainage system. The investigations concluded that the corrosion was caused primarily by galvanic action between the stainless steel (cathode) and mild steel (anode) components of the drainage system, with anaerobic bacteria and carbonic acid attack from the carbon dioxide (CO_2) dissolved in the well water. Galvanic action was magnified by the lack of oxygen and the high chloride content of the water. A chemical analysis showed the extremely corrosive quality of the ground water, as evidenced by:

- Extremely high concentration of dissolved carbon dioxide, 160 to 240 parts per million (ppm)
- Complete lack of oxygen, 0 ppm
- High chloride, 54 to 128 ppm; sulfates, 294 to 515 ppm; and iron concentration, 13 ppm
- Biological activity

The field investigators recommended that 304 stainless steel pipes should be used throughout any replacement system to withstand the possibility of severe corrosion caused by the chemical contents of ground water and to prevent galvanic action between different metals (McClelland Engineers, Inc., 1971).

Individual Deep Well Systems

Experience during highway construction in 1961-1962 and during the 1963 drainage system replacement showed that individual deep wells were effective in temporarily maintaining ground-water levels at desired elevations. This alternative as a permanent system, therefore, was given further study. A 1972 consultant's report (Layne-Western Company, Inc.,

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. An additional two wells were included to permit well rotation and maintenance. These 12 wells were constructed in 1973 and the new system placed in service in April 1974. The wells are 16-inch gravel-packed (42-inch borehole) wells averaging about 96 feet deep and are equipped with 60 feet of Layne stainless steel well screen. The pumps are 600-gpm capacity with 6-inch-diameter stainless steel (flanged coupling) column pipe.

A recorder well was included in the well dewatering system to monitor ground-water levels near the critical elevation of the highway. The well is 8 inches in diameter and is constructed of stainless steel casing and screen. 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 dewatering well to depths corresponding to the upper third point of each dewatering well screen. The purpose of these piezometers is to provide information on ground-water levels and to monitor the performance of individual wells by measuring water-level differences between the wells and the piezometers.

The western terminal of Interstate 64 joins Interstate 70 at the Tri-Level Bridge site. A 2200-foot stretch of this highway also is depressed below 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. The wells were built in 1975 and are essentially identical to those constructed for the Tri-Level Bridge site.

About 6,200 feet southeast of the Tri-Level Bridge, at the East St. Louis 25th Street interchange with I-64, the street was designed to pass below the highway and adjacent railroad tracks. As a result, the 25th Street pavement would be about 3.5 feet below the ground-water surface. Ten wells were installed at this site to control ground-water levels. These wells also are identical in design to the I-70 wells. The pumps installed in the wells along I-64 and at 25th Street have nominal pumping capacities of 600 gpm. Two 8-inch observation wells, located near each end of the I-64 depressed section, are used to monitor ground-water levels. An 8-inch observation well also is 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 for monitoring the performance of each individual installation.

Approximately *2k* miles north of the I-70 Tri-Level Bridge, Illinois Highway 3 passes beneath the N and W, ICG, 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 and are 16 inches in diameter with 50 feet of well screen. They 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.

Thus at present the highway dewatering operation in the American Bottoms consists of 48 individual dewatering wells fully penetrating the water-bearing sand and gravel aquifer. The wells are distributed at four sites as follows:

I-70 (Tri-Level Bridge)	-	12 wells
I-64	-	20 wells
25th Street	-	10 wells
Venice (Route 3)	-	6 wells

The wells are of similar construction, with 16-inch-diameter stainless steel casing and screen, and 6-inch-diameter stainless steel column pipe (figure 4). Each well is equipped with a 600-gpm submersible pump with bronze impellers, bowls, and jacket motors. The early experience with severe corrosion problems showed that corrosion-resistant materials are required to maximize service life. A total of five 8-inch recorder wells are available to monitor ground-water elevations near critical locations at the four sites. Each of the 48 wells has a 2-inch-diameter piezometer for monitoring individual well performance.

Usually, about one-third of the wells are in operation simultaneously. Total pumpage was estimated to be about 10.8 million gallons per day in 1984.

DEWATERING SYSTEM MONITORING

When originally constructed, the well installations at I-70, I-64, and 25th Street included flow-rate meters of the pitot-tube type. Reportedly, a combination of corrosion and chemical deposition caused premature failure of these devices. Flow rates were occasionally checked with a temporarily inserted pitot-tube meter, but erratic results were reported by the field crew. The six installations at Venice include a venturi tube coupled to a bellows-type differential pressure indicator to measure the flow rate. Flow measurements from the venturi tube are reported to be accurate to within ±1% of full pipe flow rate, and the differential pressure indicators to within ±0.75% of the deflection. The bronze-lined venturi tubes will probably remain unaffected over time by the quality of water pumped from these wells; however, the water comes in direct contact with the bellows in the differential pressure indicators via two ¼-inch water lines from the venturi tubes. The same corrosion and chemical deposition affecting the pitot tubes could, over time, cause obstructions in the water lines and/or water chambers or direct failure of the bellows. Some operational problems with these instruments have already been noted.

Operational records show that wells are pumped for periods of about two to nine months and then left off for longer periods while another set

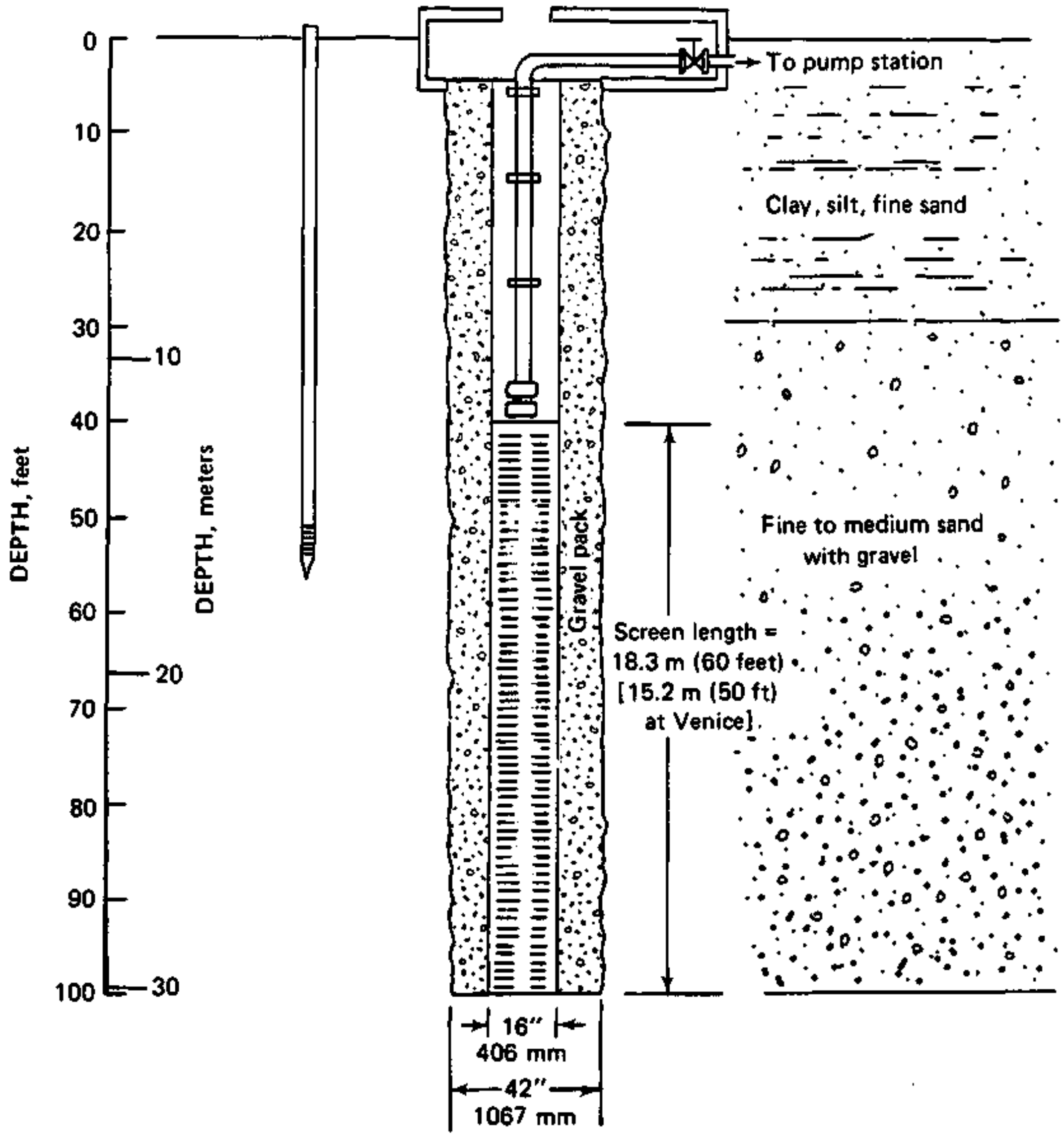


Figure 4. Typical features of a dewatering well

of wells is operated. No standard sequence of pumping rotation is followed because of maintenance and rehabilitation requirements. Bar charts showing the periods of operation are prepared by IDOT for monitoring the accumulated hours of operation. Annual withdrawals currently are calculated on the basis of pumping time and estimated or measured pumping rates.

Water levels in the piezometer adjacent to each dewatering well are measured every 2 to 4 months. The pumping water level in each operating well also is measured. These water-level data are reviewed by IDOT supervisors to monitor ground-water levels in relation to the pavement elevation and to assess the condition of individual dewatering wells. Water-level differences (referred to as Ah) of 3 to 5 feet between the pumping wells and the adjacent piezometers usually are considered normal by IDOT. Greater differences are interpreted to indicate that well deterioration is occurring. Plots of the Ah data have been done for each dewatering well and appear in Appendix E. Piezometer water levels also are superposed on drawings of longitudinal sections of the highway for visual comparison. This technique suggests probable errors in field measurements or a plugged piezometer when the water-level elevation for a given piezometer is not consistent with water levels in adjacent piezometers.

Finally, each dewatering well site includes an observation well equipped with a Leupold-Stevens water-level recorder. The recorder charts are changed monthly and are intended to provide a continuous record of water levels near the critical location at each dewatering site.

INVESTIGATIVE METHODS AND PROCEDURES

Well Loss

When a well is pumped, water is removed from the aquifer surrounding the well, and the water levels are lowered. The distance that the water level is lowered, whether within the well or in the surrounding aquifer, is referred to as drawdown, which under ideal conditions is a function of pumping rate, time, and the aquifer's hydraulic properties. However, other geohydrologic and hydraulic factors also can affect the observed drawdown, especially within the pumped well. Aquifer boundaries, changes in aquifer thickness or hydraulic properties, interference from nearby wells, partial-penetration conditions, and well losses all can affect observed drawdowns. Well losses usually are associated only with the pumped well and are the only non-ideal condition addressed in this report.

The observed drawdown in a pumped well is usually greater than that in the aquifer formation outside the borehole because of the well losses caused by the water moving from the fully penetrated aquifer into the well. The amount of well loss depends on the materials used and the job done in constructing the well. A limited amount of well loss is to be expected as 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 unacceptable well losses. In addition, well losses often reflect a deterioration in the condition of an existing well, especially if they are observed to increase with time.

Well losses are related to pumping rate and ideally are not a function of time. These losses are 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. In some cases, well loss occurs entirely under conditions of laminar flow; however, velocities may become sufficiently large that a change from laminar to turbulent flow occurs. Under these conditions the well-loss component of drawdown can rapidly become excessive, increasing in a nonlinear manner with linear increases in pumping rate.

Thus, under near-ideal conditions, the observed drawdown (s_0) in a pumping well is made up of two components: the formation loss (s_a) resulting from laminar (and sometimes turbulent) flow head loss within the aquifer, and well loss (s_w) resulting from the turbulent (and sometimes laminar) flow of water into and inside the well, as shown in equation 1.

$$s_0 - s_a + s_w \qquad (1)$$

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

$$s - BQ + CQ^2 \qquad (2)$$

where s is the drawdown, B is the formation loss coefficient at the well-aquifer interface per unit discharge, and C is the well loss coefficient. 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

$$s = BQ + CQ^n \qquad (3)$$

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). This analysis requires a controlled pumping test, called a step drawdown test, 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 can be employed after first modifying equation 2 as

$$s/Q - B + CQ \qquad (4)$$

After this modification, a plot of s_0/Q versus Q can be prepared on arithmetic graph paper from data collected during a step drawdown test, with the observed drawdown, s_0, substituted for s. The slope of a line fitted to these data is equal to C, while the y-intercept is equal to B, as shown in figure 5. If the data do not fall on a straight line but, instead, curve concavely upward, then Rorabaugh's method usually is suggested. The curvature of the plotted data indicates that the second-order relationship between Q and s_0 is not valid.

If Rorabaugh's equation is used, then the 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 leads to

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

A plot of $(s_0/Q) - B$ versus Q can be made on logarithmic graph paper from step test data, again using s_0 for s. Values of B are tested until the data fall on a straight line (figure 6). 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, the graphical procedure is facilitated if Q is plotted as cubic feet per second and $(s_0/Q) - B$ is plotted as seconds per foot squared. It is also convenient (although not mandatory) to use these same units in the Jacob method.

Step Test Procedure

The primary objective of a step drawdown test (or step test) is determination of 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 well is pumped successively at a number of selected pumping rates. Equally spaced pumping rates are selected to facilitate the data analysis. Each pumping period at a given 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.

During each step pumpage is held constant. At a minimum, water-level measurements are made every minute for the first six minutes, every two minutes for the next ten minutes, and then every four to five minutes thereafter until the end of the step. In this investigation, water levels were measured for 30 minutes per step. At the end of each 30-minute interval, the pumping rate was immediately changed, the water level measurements reverted to the one-minute frequency again, and so on 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 7. Drawdowns for each step (shown as Δs_i) are measured as the distance between the extrapolated

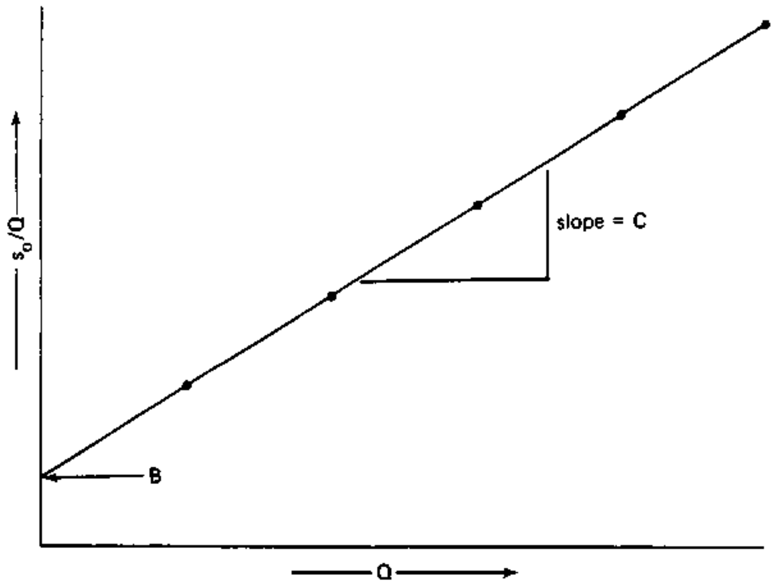


Figure 5. Graphical solution of Jacob's equation
for well loss coefficient, C

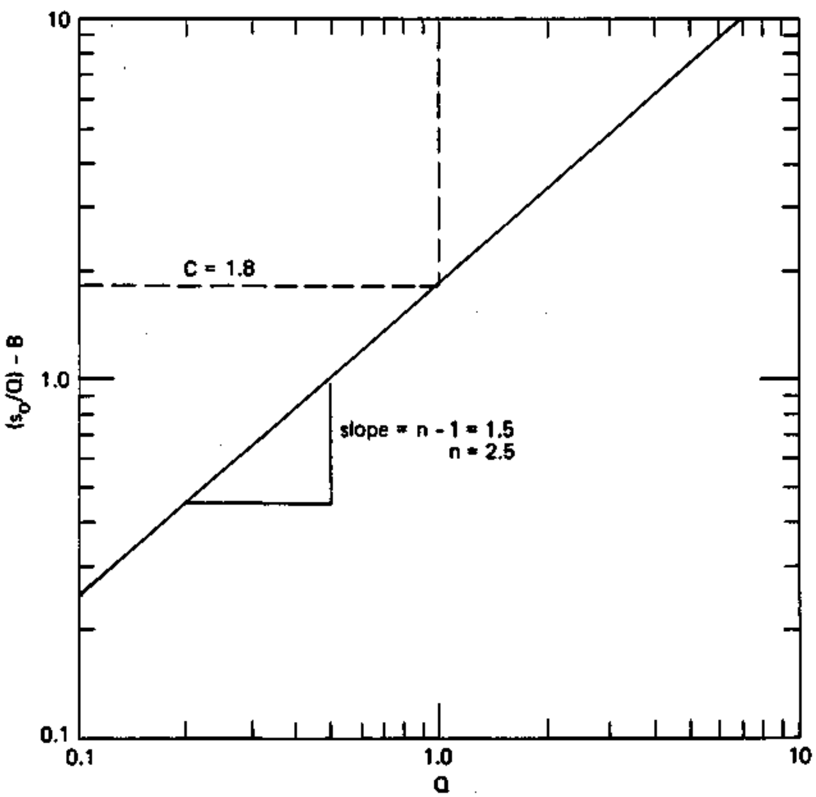


Figure 6. Graphical solution of Rorabaugh's equation
for well loss coefficient (C) and exponent (n)

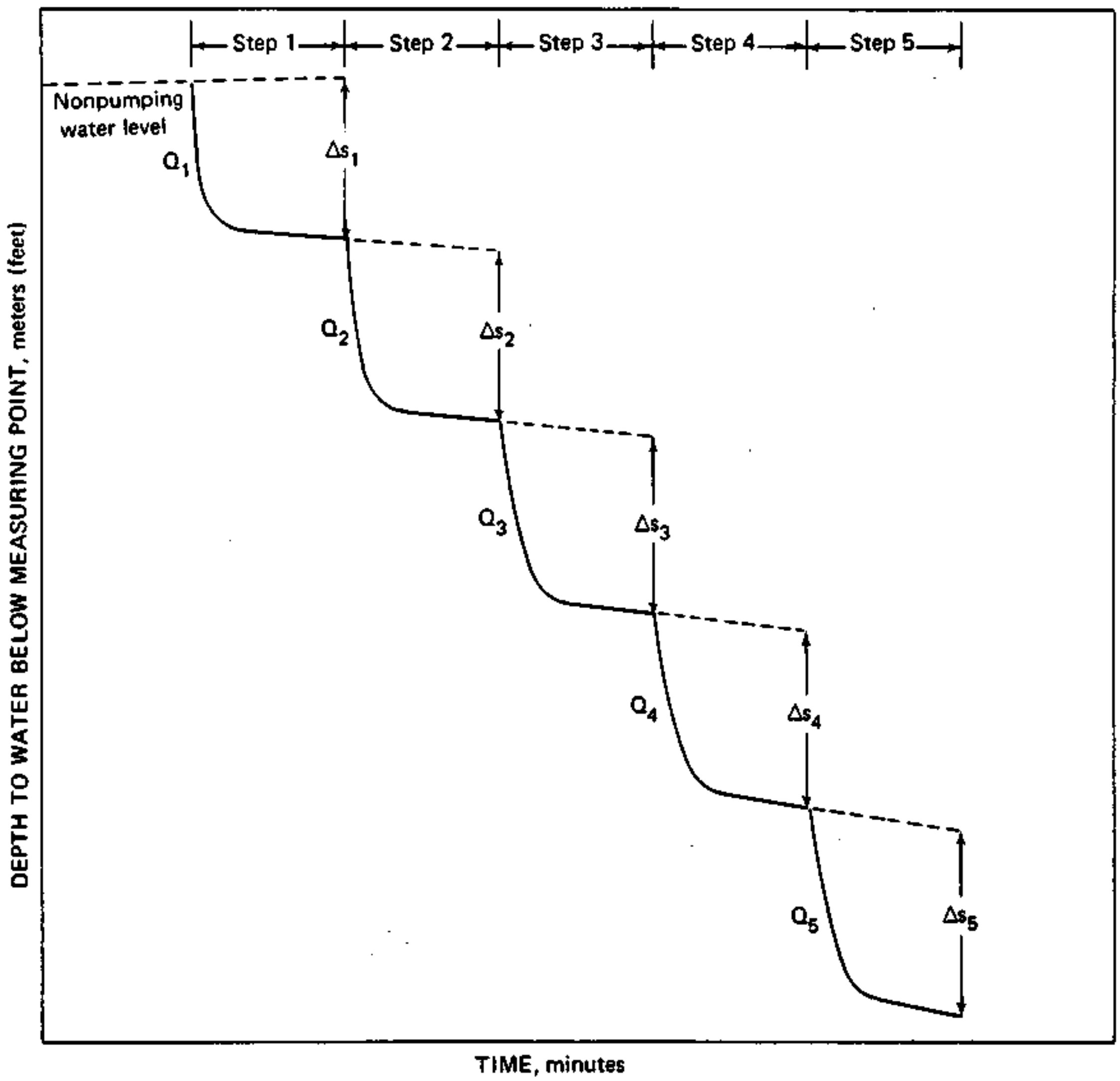


Figure 7. Relationship between time and water level during a five-step drawdown test

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 As_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_0/Q versus Q or $(s_0/Q) - B$ versus Q, values of observed drawdown s_o are equal to the sum of As_i for the step of interest. Thus, for step 3, $s_o = \Delta s_1 + \Delta s_2 + \Delta s_3$.

Piezometers

Piezometers--small-diameter wells with a short length of screen--are used to measure water levels at a point in space within an aquifer and are often used in clustered sets to measure variations in water levels (head) with depth. In the case of well loss studies, piezometers can be employed to measure head losses across a well screen or across a gravel pack or well bore.

All 48 of the IDOT dewatering wells 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. An indication of turbulent well losses in a pumped well can be found in such an arrangement by comparing the difference in head between water levels in the well and those in the adjacent piezometer over a sufficiently large range of pumping rates. If turbulent losses exist within that range, the difference in heads should be nonlinear with increasing pumping rate. It can also sometimes 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, then this relationship will also be nonlinear. Additionally, the piezometers can be used as mechanisms to continually monitor head differences between the wells and the piezometers to detect deterioration at any well. This has been IDOT's primary use of data from the piezometers.

FIELD RESULTS

Well Selection

The six wells selected for testing were determined by examining graphs of the monitoring data collected by IDOT as well as the well location and its relative importance to the dewatering system. Those installations showing the greatest water level differences between the operating well and piezometer were judged most likely to have deteriorated, thus requiring verification of their condition. The wells selected for testing in 1985 were:

I-70	No. 6
I-64	No. 2
	No. 4
	No. 12
25th St.	No. 3
	No. 10

Field Testing Procedure

Field work was conducted by Water Survey staff with the assistance of the IDOT Maintenance Division pump crew under the supervision of Stan Gregowicz. The IDOT pump crew made all necessary discharge pipe modifications and provided special piping adapters. This allowed the water from the pumped wells to be discharged through a flexible hose and orifice tube provided by the Water Survey. Discharge water from the orifice tube was directed to nearby stormwater drains.

Orifice tubes are considered standard equipment for measuring flow rates. The orifice plate used to measure the range of flow rates was calibrated in the University of Illinois Hydraulics Lab under discharge conditions similar to those expected in the field. The rating curve developed from the calibration procedure is shown in figure 8.

Prior to the start of each test, the nonpumping water levels in the well and the piezometer were measured with a steel tape. Standard electric droplines, or pressure transducers coupled to a field computer for analog-to-digital conversion and data storage (McDAS), were used to determine depths to water during the step tests.

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 steps as possible at 300 gpm or greater for each well. In addition, since routine monitoring by IDOT personnel is based upon the difference in water levels between the operating well and the piezometer, water level declines (drawdowns) during the step tests were observed in both the pumped well and the piezometer. This routine provided data for comparison with the historical monitoring data available from IDOT.

The six well tests were conducted during the period of July 15 through September 9, 1985. Some of the tests were conducted by using an initial 300-gpm step followed by steps with pumping rates increasing at 50-gpm increments, while the initial step for the others was set near the maximum well pumping rate with subsequent steps using pumping rates that decreased at 50-gpm increments. The water level and pumping rate measurements collected during each of the step tests are included in Appendix A. Near the end of the step tests, water samples were collected for chemical analysis. The results of the analyses are included in Appendix B.

Results of Step Tests

Data from each of the step tests were analyzed by using the Jacob method (the exponent of Q for the well loss component of drawdown is 2). To illustrate this technique in detail, an example follows wherein data from the September 6, 1985, test of 25th Street Well No. 3 are analyzed.

Pumping at Well No. 3 commenced at 10:15 a.m. at an initial rate of 800 gpm. During each succeeding 30-minute step, pumpage was decreased by 50 gpm so that steps 2 through 6 had discharge rates of 750, 700, 650,

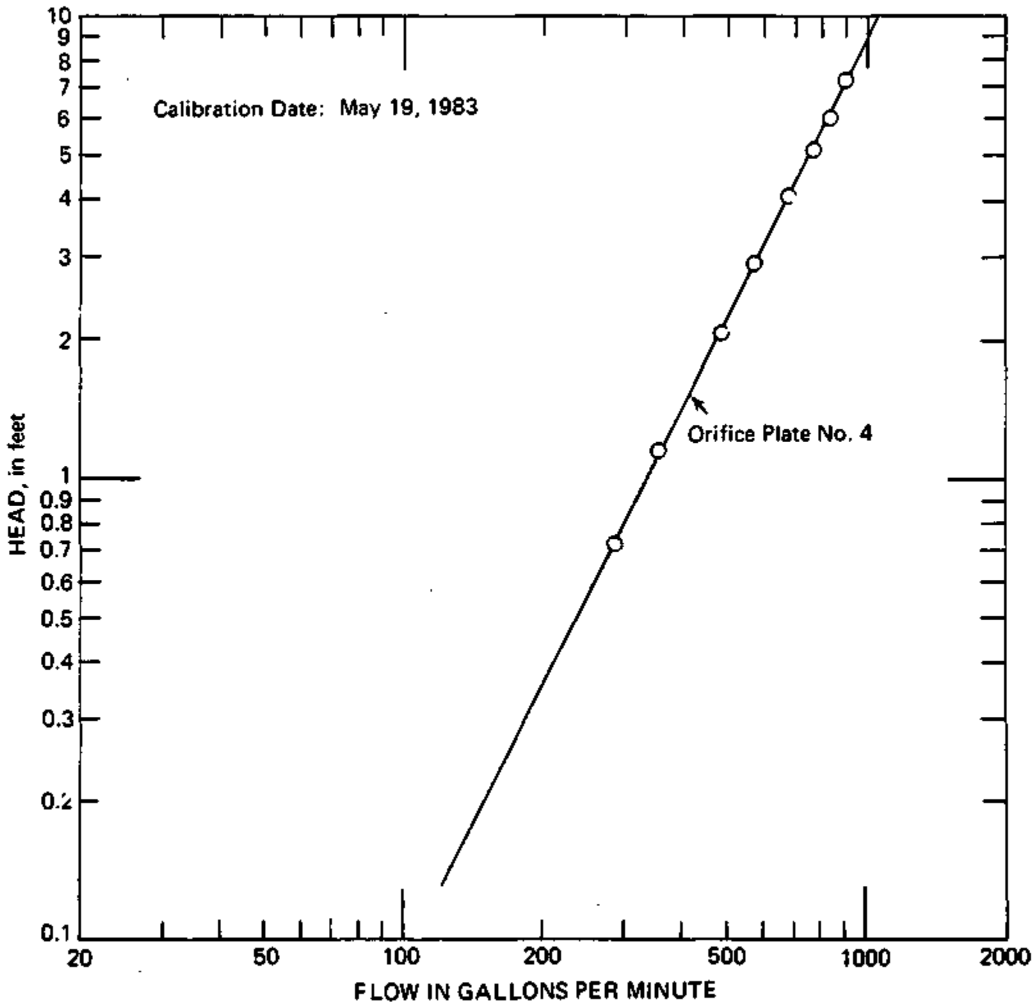


Figure 8. Rating curve for ISWS 8-inch orifice tube with plate no. 4

600, and 550 gpm, respectively. A water sample was collected during step 5 at 12:38 p.m., and the test was concluded at 1:15 p.m.

Data from the pumped well are shown in a plot of s_0/Q versus Q (figure 9a). To facilitate the procedure, the discharge rate was plotted in units of cubic feet per second. As described earlier, the two components of drawdown can be determined by solving for the coefficients B and C, where B is the aquifer loss coefficient and C is the well loss coefficient. From the analysis, the coefficients B and C were determined to be 3.64 seconds per square foot (sec/ft^2) and 0.017 sec^2/ft^5, respectively. Applying these coefficients to equation 2 at a discharge rate of 600 gpm (1.337 cfs), for example, we have

$$s - BQ + CQ^2$$

$$- 3.64(1.337) + 0.017(1.337)^2$$

$$- 4.87 + 0.03$$

$$- 4.90 \text{ feet}$$

The total drawdown of 4.90 feet compares favorably with the observed drawdown, which was 5.00 feet, suggesting a good correlation between theoretical and observed results.

The analysis indicates that at 600 gpm, the portion of drawdown caused by turbulent well losses at the well screen and inside the well was 0.03 feet or 0.6% of the total drawdown, which is low. Another indication that the well is in good hydraulic condition is the specific capacity. At 600 gpm, the observed specific capacity was 120.0 gpm/foot, which compares favorably with values obtained at other sites and which also compares well with the theoretical specific capacity for the I-70 area (estimated from hydraulic properties in the area, Schicht, 1965).

Figure 9b shows water-level differences (Ah) between 25th Street Well No. 3 and its nearby piezometer during the test. The relationship appears to be linear, suggesting that turbulent losses in the vicinity of the well are small. This is corroborated by a plot of drawdowns at the piezometer versus pumpage in figure 9c, which shows a linear relationship.

The results of analyses performed on data gathered during the step-drawdown testing of six IDOT wells in 1985 are summarized in table 1. (A summary of the results from all of the step tests conducted for the three phases of work is presented in table 2.) As seen in table 1, turbulent well losses in most cases were a relatively small portion of the total drawdowns at 600 gpm. The 600-gpm discharge rate was selected as a standard since it is the design rate for the IDOT wells. Well loss was relatively high in I-64 Well No. 4, reaching 15% of the total drawdown at 600 gpm. However, total drawdown at 600 gpm was low (4.40 feet, lowest of the six wells tested) and specific capacity high (136.4 gpm/foot, highest of the six wells), suggesting that the amount of well loss is acceptable and that the well is in good condition. All of the other wells had calculated well losses of 4.3% or less, the smallest well loss being 0.6% for 25th Street Well No. 3. Data from 25th Street Well No. 10 indicated conditions resembling development at the screen, which does not allow

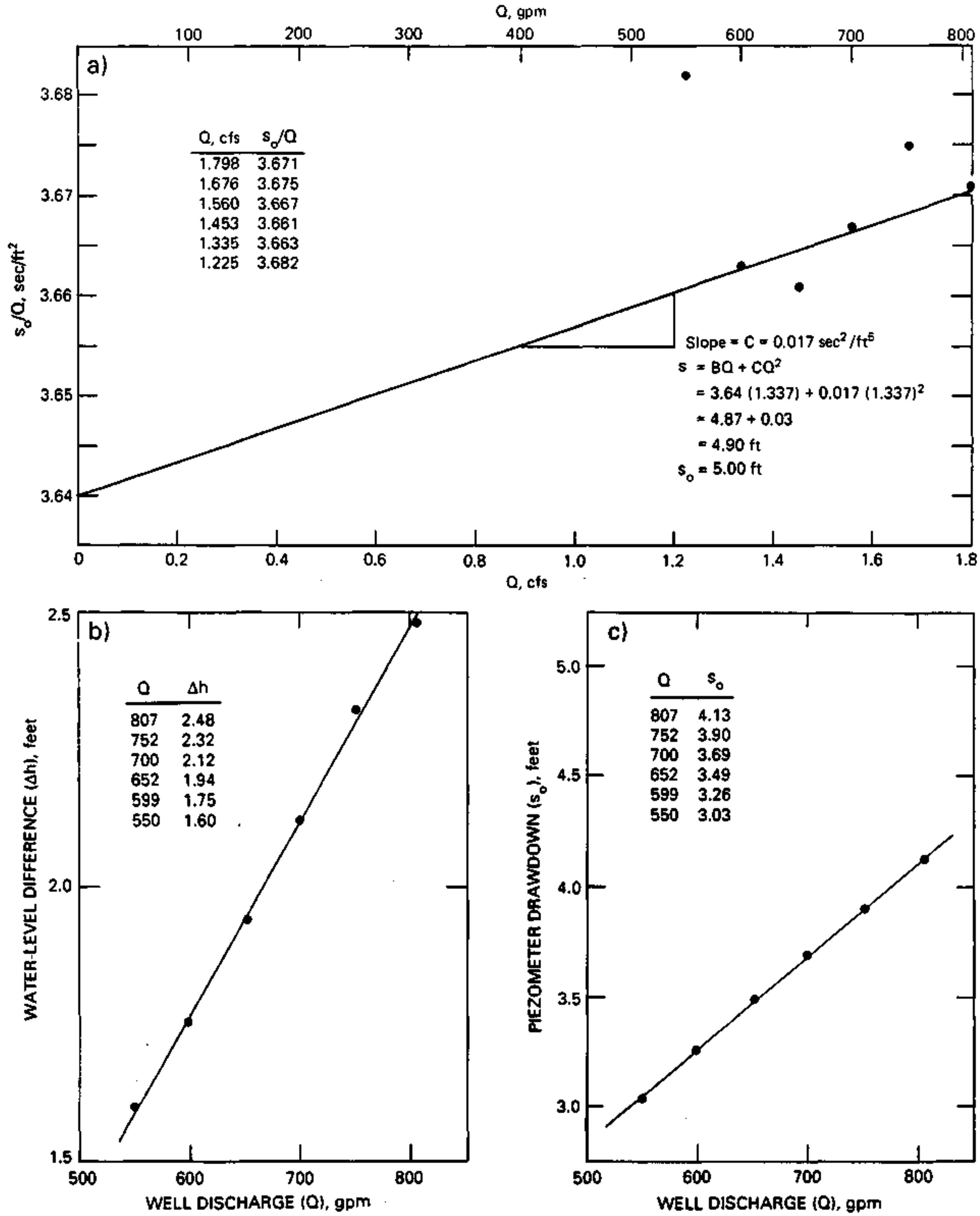


Figure 9. Results from step test of 25th Street Well No. 3, September 6, 1985: a) Determination of well loss coefficient, C; b) water-level difference vs. well discharge; c) piezometer drawdown vs. well discharge

Table 1. Results of Step Tests on IDOT Wells, 1985 (Phase 3)

Well	Date of test	Well loss @ 600 gpm (ft)	Drawdown @ 600 gpm (ft)	Well loss portion (%)	Specific capacity (gpm/ft)	Δh* @ 600 gpm (ft)	Remarks
I-70							
No. 6	7/19/85	.23	5.39	4.3	111.3	--	Piezometer plugged, Q_{max} = 625 gpm
I-64							
No. 2	7/25/85	.09	5.32 e	1.7	112.8	5.22 e	Q_{max} = 550 gpm
No. 4	7/15/85	.66	4.40	15.0	136.4	--	Piezometer plugged
No. 12	7/18/85	.17	6.22 e	2.8	96.5 e	1.62 e	Q_{max} = 590 gpm
25th St.							
No. 3	9/6/85	.03	4.89	0.6	122.7	1.75	
No. 10	7/26/85	**	9.56	**	62.8	3.59	

e-Estimate
*-Head difference between pumped well and adjacent piezometer
**-Coefficient undeterminable. Possible well development. Turbulent well loss negligible over the pumping rates tested.

Table 2. Results of Step Tests on IDOT Wells (Phases 1, 2, and 3)

Well	Date of test	Well loss @ 600 gpm (ft)	Drawdown @ 600 gpm (ft)	Well loss portion (%)	Specific capacity (gpm/ft)	Δh* @ 600 gpm (ft)	Remarks
I-70							
No. 1	8/15/84	**	18.1 e	**	33.1 e	12.8 e	Q_{max} = 328 gpm
No. 2	7/19/83	**	11.9 e	**	50.4 e	7.9 e	Q_{max} = 500 gpm
No. 3	6/28/83	**	8.53	**	70.9	5.65	
No. 4	8/16/84	.07	9.33	0.8	64.3	--	Piezometer plugged
No. 5	7/10/84	.89	6.53	13.6	91.9	2.11	Q_{max} = 740 gpm
No. 6	7/19/85	.23	5.39	4.3	111.3	--	Piezometer plugged, Q_{max} = 625 gpm
No. 7	6/30/83	1.88	18.55	10.1	32.3	15.0	Piezometer at 7.5 ft
No. 8	8/1/84	2.68	13.54	19.8	44.3	9.94	Q_{max} = 625 gpm
No. 9	6/28/84	**	9.46	**	63.4	5.94	Q_{max} = 630 gpm
No. 10	7/11/84	5.97 e	16.93 e	35.3	35.4 e	--	Piezometer plugged, Q_{max} = 480 gpm
No. 11	8/2/84	1.58 e	15.55 e	10.2	38.6 e	13.35 e	Q_{max} = 555 gpm
No. 12	6/16/83	0.20	3.82	5.2	157.1	--	Piezometer plugged
I-64							
No. 2	7/25/85	.09	5.32 e	1.7	112.8 e	5.22 e	Q_{max} = 550 gpm
No. 3	6/26/84	.52	10.73 e	4.8	55.9 e	--	Piezometer plugged, Q_{max} = 525 gpm
No. 4	7/15/85	.66	4.40	15.0	136.4	--	Piezometer plugged
No. 9	10/5/83	0.37	6.22	5.9	96.5	2.3	
No. 10	7/11/84	**	7.46	**	80.4	2.73	Q_{max} = 605 gpm
No. 11	8/14/84	**	7.22 e	**	83.1 e	3.2 e	Q_{max} = 520 gpm
No. 12	7/18/85	.17	6.22 e	2.8	96.4 e	1.62 e	Q_{max} = 590 gpm
No. 13	7/12/84	**	6.44	**	93.2	2.65	Q_{max} = 600 gpm
No. 15	6/29/83	0.73	9.94	7.3	60.4	4.6	

Table 2. Concluded

Well	Date of test	Well loss @ 600 gpm (ft)	Drawdown @ 600 gpm (ft)	Well loss portion (%)	Specific capacity (gpm/ft)	Δh* @ 600 gpm (ft)	Remarks
25th St.							
No. 2	7/20/83	0.54	5.69	9.5	105.4	1.1	
No. 3	9/6/85	.03	4.89	0.6	122.7	1.75	
No. 6	6/27/84	.14	9.44	1.5	63.6	--	Piezometer plugged, Q_{max} = 775 gpm
No. 8	6/15/83	0.11	4.70	2.3	127.6	1.5	
No. 10	7/26/85	**	9.56	**	62.8	3.59	
Venice							
No. 1	11/30/83	2.29	18.33 e	12.5	32.7 e	10.9 e	Q_{max} = 500 gpm
No. 2	11/17/83	0.05	4.70	1.0	127.6	1.2	
No. 3	11/28/83	**	9.20	**	65.2	4.2	
No. 4	12/1/83	0.39	5.15	7.6	116.5	2.3	
No. 5	11/15/83	0.16	4.98	3.2	120.5	1.9	
No. 6	11/29/83	0.16	7.82	2.0	76.7	6.1	

e-Estimate
*-Head difference between pumped well and adjacent piezometer
**-Coefficient undeterminable. Possible well development. Turbulent well loss negligible over the pumping rates tested.

calculation of well loss. Because of the well's relatively low specific capacity of 62.8 gpm/ft for this site and a Ah of 3.59 feet, it is suspected that some deterioration has taken place, which may be a partial cause for this response.

The specific capacity values in the table, calculated for a 600-gpm pumping rate, ranged from 62.8 gpm/ft at 25th Street Well No. 10 to 136.4 gpm/ft at I-64 Well No. 4. The overall average specific capacity was 107.1 gpm/ft. On the basis of 32 tests that have now been conducted during Phases 1 through 3 (table 2), the highest group average of 96.4 gpm/ft was seen at 25th Street. Averages for the other well groups were 90.6, 89.8, and 66.1 gpm/ft for I-64, Venice, and I-70, respectively. The I-70 average without the value from Well No. 12 (a new well), Well No. 5, and Well No. 6 was only 48.5 gpm/ft. The averages are based on samples of various sizes, but can be used for general comparative purposes.

Since head differences between water levels in the wells and in their adjacent piezometers (Ah) form the basis for current monitoring practices by IDOT, these values also were determined during the step tests and are included in table 1. Values ranged from 1.62 to 5.22 feet at 600 gpm and averaged 3.0 feet. In general, Ah values have varied inversely with specific capacity (see figure 10). A direct, linear relationship between Ah and well loss might be expected; however, the relationship is not a strong one. Part of the reason for this is that the distribution of the Ah values is skewed toward the lower end of their range of values. Another possibility is that the Ah value includes some laminar well losses between the well and the piezometer.

Well Rehabilitation

Seven of the dewatering wells recommended for treatment in Phases 1 and 2 (I-70 Nos. 1, 2, 8, 10, and 11; I-64 No. 15; and Venice No. 1) were rehabilitated by Aylor Aqua Services, Inc. The well treatments were completed during the period from July 18 to September 5, 1985. A daily chronology of activity at each well is included in Appendix D. An outline of the typical, or ideal, well treatment schedule is presented in table 3.

Considerable adjustments and aberrations from the typical treatment procedure were observed from well to well and from day to day. These are apparent in the actual treatment chronologies for each well. The chronology for I-70 No. 1, the first well treated, is more detailed in its description of the well treatment equipment and general sequence of events. Later well treatment chronologies note any significant changes made in equipment and arrangement.

It should be noted that the contractor sought and obtained approval for switching the order of the third and fourth treatment steps as detailed in IDOT's contract specifications. This placed the first application of polyphosphates ahead of the acidization rather than after it as originally specified. The contractor indicated that this change might improve the effectiveness of the acidization step.

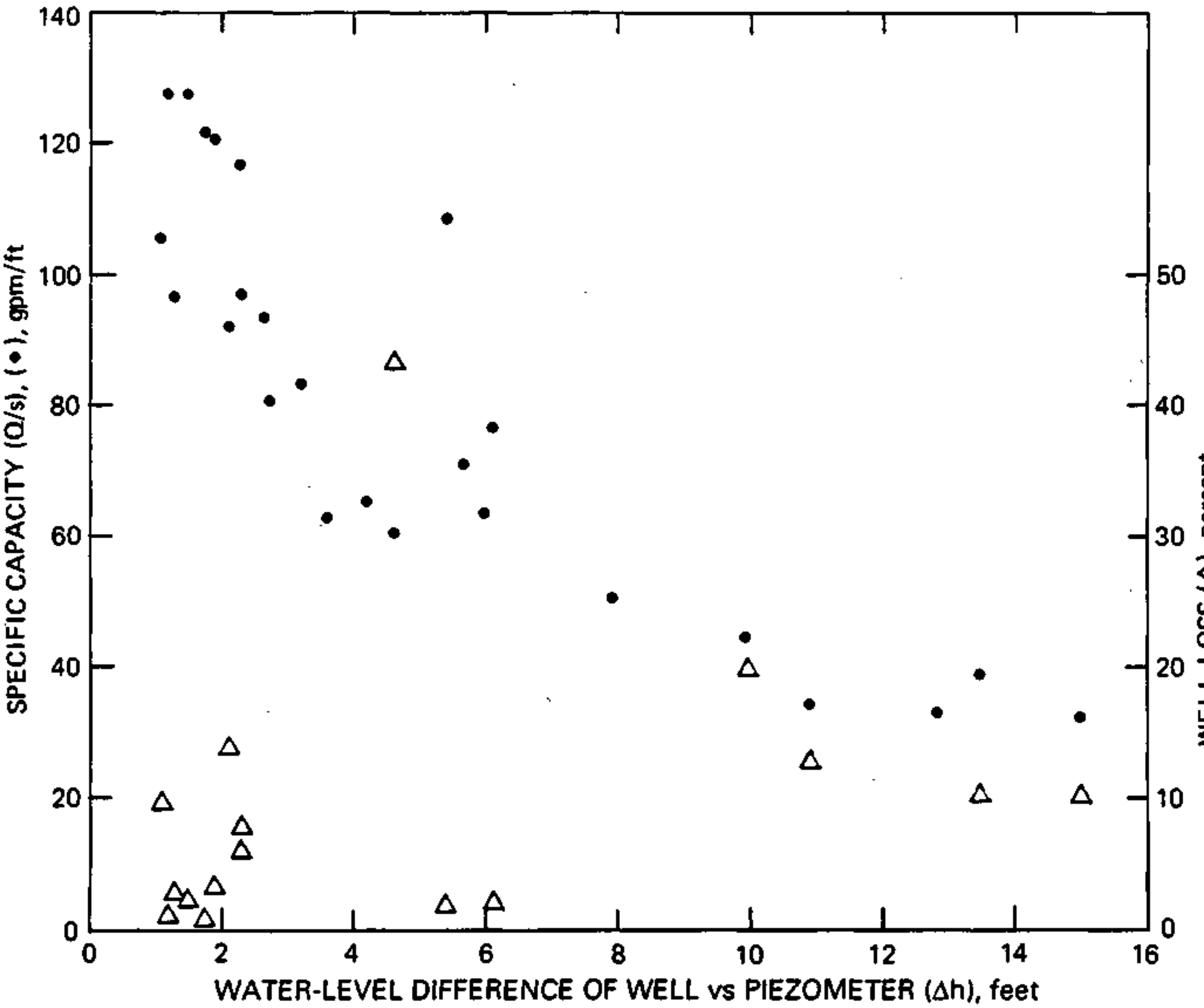


Figure 10. Water-level difference vs. specific capacity and well loss from Phases 1, 2, and 3 step tests

Table 3. Outline of Typical Well Rehabilitation

Day 1

1. Pre-treatment specific capacity test (contractor orifice tube open to free discharge used for flow measurements).

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

2. Polyphosphate application, 400 lbs., and displacement with 16,000 gallons water containing at least 500 ppm chlorine.

    a. Initial chlorination of well with water containing 500 ppm or more chlorine injected at approximately 500 gpm.
    b. Injection of polyphosphate solution at a rate of approximately 500 gpm in two 2,000-gallon batches, each batch containing 200 lbs. polyphosphate, at least 500 ppm chlorine, and 1-2 cups unknown agent.
    c. Injection of 16,000 gallons water chlorinated to at least 500 ppm in 2,000-gallon batches (injection rate varied widely from a few gpm to greater than 1,200 gpm).
    d. Time allowance for chemicals to react, 30 or more minutes.

3. Pump to waste and check specific capacity.

    a. Same procedure as step 1 above.
    b. Pumping continued 60 or more minutes to clear well of chemicals.

Day 2

1. Acidization with 990 gallons 20° Baume inhibited muriatic (hydrochloric) acid, and displacement with 2,500 gallons water (not chlorinated).

    a. Siphoning of acid from 18 55-gallon drums into wells at approximately 30 gpm.
    b. Allowance for acid to react, 60 or more minutes.
    c. Injection of 2,500 gallons water (rate varied widely from a few gpm to greater than 1,000 gpm).
    d. Allowance for reaction, 30 or more minutes.

2. Pump to waste and check specific capacity.

    a. Same procedure as day 1, step 1 above.
    b. Buffer solution prepared with 400 lbs. soda ash and injected into discharge stream to neutralize well discharge.
    c. Pumping continued 60 or more minutes to clear well of acid.

Table 3. Concluded

Day 3

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

   Same procedure as day 1, step 2 above, except three batch injections of 200 lbs each in part b, and injection of 30,000 gallons in part c.

2. Pump to waste and check specific capacity.

   a. Same procedure as day 1, step 1 above.
   b. Pumping continued 60 or more minutes to clear well of chemicals.

Day 4

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

   Same procedure as day 1, step 2 above, except three batch injections of 200 lbs each in part b, and injection of 54,000 gallons in part c.

2. Pump to waste and check specific capacity.

   a. Same procedure as day 1, step 1 above.
   b. Pumping continued 60 or more minutes to clear well of chemicals.

Day 5

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

   Same procedure as day 1, step 2 above.

2. Pump to waste and final specific capacity test.

   a. Same procedure as day 1, step 1 above.
   b. Pumping continued 60 or more minutes to clear well of chemicals.

Another change of note is that the pumping rates actually used in the field often apparently were less than that called for in the specifications. This most probably was due to limitations in the contractor's equipment and the plumbing arrangements connecting his equipment to the well head. It was not possible to determine what effect, if any, this had on treatment effectiveness. Better verification of contractor equipment capabilities is needed; or, if the results obtained so far are acceptable, treatment specifications for future work should be modified to conform with what actually is being used in the field.

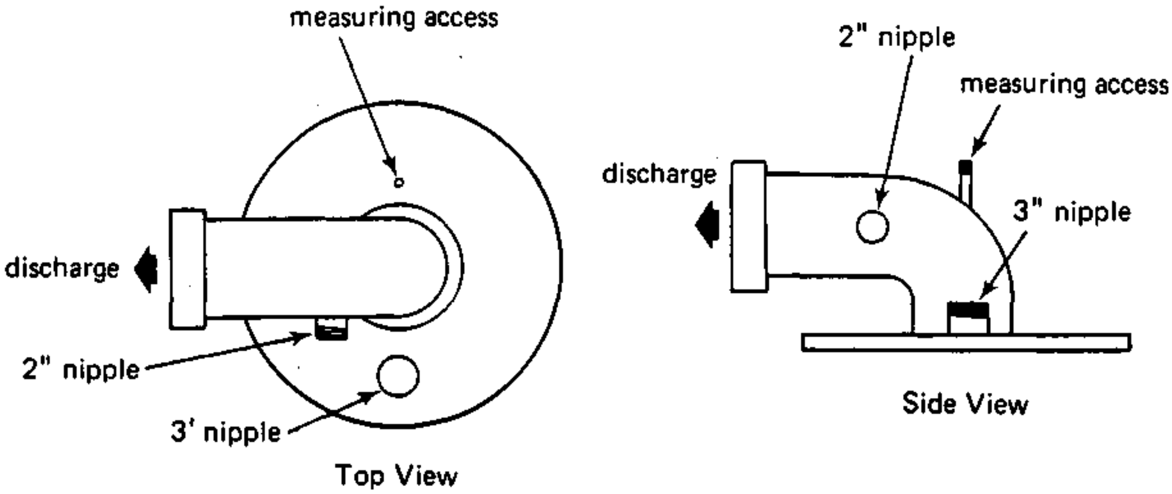
Figure 11 depicts the typical assembly for injecting chemical solutions (except for the acid) into the wells, and also the discharge apparatus used for pumping solutions to waste and conducting pumping tests. Figure 12 illustrates the other typical assembly that was used for acidization of the wells. Modifications of the well heads to facilitate treatment were made by the contractor prior to activity at each well.

Monsanto ACL 90 Plus was the chlorine source throughout the contract work. Monsanto Sodium Hexametaphosphate and Sodium Tripolyphosphate were used in some polyphosphate applications, although most of the polyphosphate applied was an undisclosed custom blend prepared for the contractor by Monsanto. Copies of Monsanto's technical brochures for the chemicals used in the treatment work are available from the Ground-Water Section of the Water Survey on request. The source of muriatic acid was Chemtec, Inc., in St. Louis, but no applicable technical publications are available.

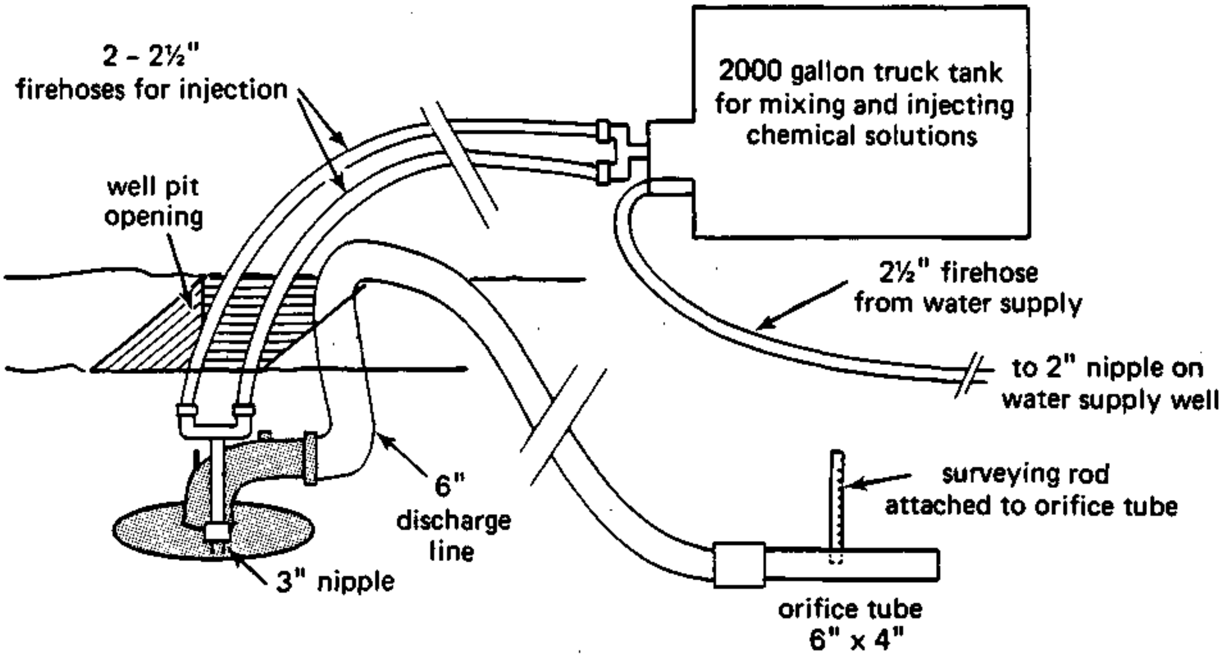
Theoretical horizontal displacement of the polyphosphate solution and muriatic acid from the treated well by the quantities of displacement water used are presented in table 4. It has been assumed for these calculations that displacement occurs through a 65-foot vertical section of homogenous sand and gravel aquifer with a porosity of 30%. (A section 5 feet larger than the length of screen was chosen to allow for dispersion.) The amount of chemical displacement was recommended by Aylor Aqua Services, Inc. (the contractor) and accepted by IDOT after review by the Water Survey.

The 2,500 gallons of water used to displace the acid essentially pushed the acid out through the gravel pack (about 21 inches) to an area just beyond the gravel pack/aquifer interface. All batches of the phosphate are displaced about 4 to 9 feet into the aquifer. Samples of water collected from two monitoring wells around I-70 Well No. 2 and the piezometers at distances of 5 to 15 feet from the center of I-70 Wells 1, 2, 8, 10, and 11 and I-64 No. 15 verified the presence of treatment chemicals at various stages of the treatment.

Table 5 summarizes the pumping test data collected by the contractor during the treatment of each well. It contains an estimate of specific capacity prior to the start of treatment and following each step in the treatment process (phosphate or acid injection episode). The average specific capacity for all of the wells at each step in the treatment process is given at the end of the table along with an analysis of the improvement between steps. Diminished returns in improvement are noted for each successive step in the treatment process. Nearly one-half of the



WELL COVERPLATE MODIFICATIONS



CHEMICAL INJECTION ASSEMBLY

Figure 11. Injection assembly and discharge apparatus

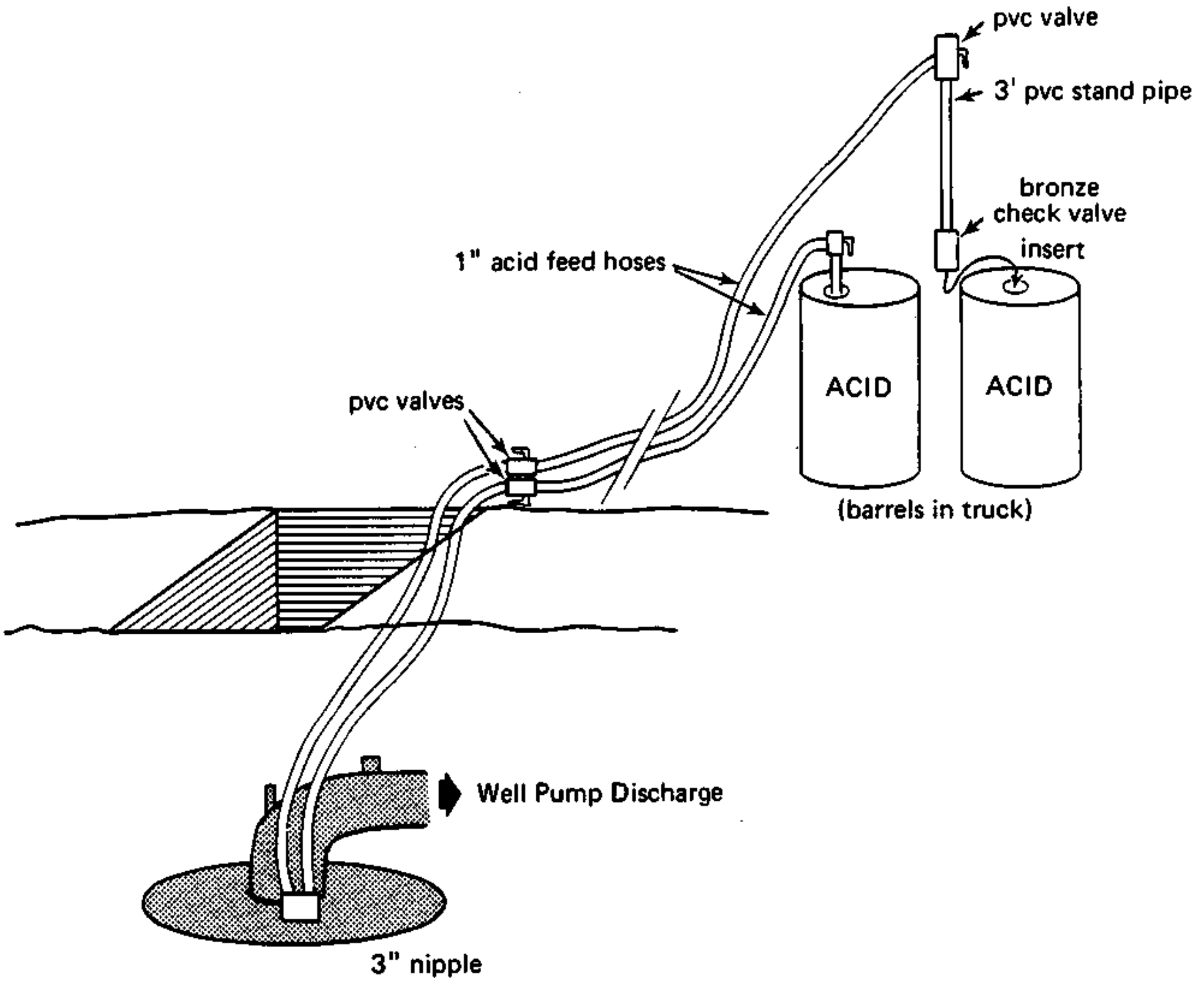


Figure 12. Acidization assembly

Table 4. Horizontal Displacement of Treatment Solutions

Quantity of displacement water (gallons)	Theoretical horizontal displacement from well (feet)
2,500	2.3
16,000	5.9
30,000	8.1
54,000	10.4

total improvement occurs with the first phosphate treatment, and 94% of the improvement occurs by the end of the second phosphate treatment (with an acid treatment sandwiched in between). These results are not particularly surprising and may in fact support the work completed in Phase 2 and the chemistry investigation presented in a later section of this report, which suggest that most of the plugging occurs within about 5 feet of the wells. Since the early stages of the treatment process concentrate on this zone, it is logical to expect significant improvement from them.

The Water Survey conducted step tests on each of the wells following treatment. Table 6 summarizes the results of these tests. A comparison with the pre-treatment step test data in table 2 indicates marked improvement for all of the parameters listed. The average specific capacity was improved from 42.1 to 83.2 gpm/ft, or almost 100% (41.1 gpm/ft). The well loss portion of drawdown dropped from an average of 12.5 to 6.4%, and the average Ah dropped from 9.56 to 2.70 ft for the four wells with pre- and post-treatment data available.

Finally, table 7 contains a comparison of pre- and post-treatment specific capacity data for each of the wells as collected by the contractor and the Water Survey (ISWS). For some of the wells, data from the contractor and the ISWS differ significantly. It can be seen that for all but one well, the post-treatment specific capacity reported by the ISWS is higher than that reported by the contractor. This may indicate that additional development occurred in the wells after the contractor conducted his final tests but before the ISWS performed the step tests. Another likely cause of the deviation is the fact that some of the ISWS pre-treatment step tests were conducted well in advance of the treatments (usually a period of one to two years or more). Nonetheless, the data concur in indicating that I-70 Well No. 11 had the most improvement, while results for I-64 No. 15 were least favorable. Substantial improvement (40% or greater) is noted at all but I-70 No. 2 and I-64 No. 15, but it is also recognized that both of these wells had the highest specific capacity prior to treatment. The average increase in specific capacity based on the contractor's data is 29.4 gpm/ft or 64%.

Table 5. Individual Well Treatment Steps

I-70 No. 1

	Pretreatment	1st PP treatment	Acid treatment	2nd PP treatment	3rd PP treatment	4th PP treatment
	7/18 AM	7/19 PM	7/24 AM	7/25 AM	7/26 AM	7/27
SWL	32.92	33.22	33.03	33.22	33.22*	32.98**
PWL	40.98	39.49	38.52	38.54	38.27	37.95**
s	8.06	6.27	5.49	5.32	5.05	4.97**
Piez.	25.25	27.0	27.5	29.0	28.25	29.5**
Q	318	329	332	340	336	343**
Q/s	39.5	52.5	60.5	63.9	66.5	69.0**

I-70 No. 2

	Pretreatment	1st PP treatment	Acid treatment	2nd PP treatment	3rd PP treatment	4th PP treatment
	8/6 PM	8/7 PM	8/7 PM	8/8 PM	8/9 PM	8/12 PM
SWL	33.00	33.03	33.00	33.15	33.13	33.00
PWL	39.78	39.48	39.28	39.71	39.42	39.52
s	6.78	6.45	6.28	6.56	6.29	6.52
Piez.	9.25	11.0	8.5	13.5	13.0	11.0
Q	377	412	361	457	448	412
Q/s	55.6	63.9	57.5	69.7	71.2	63.2

I-70 No. 8

	Pretreatment	1st PP treatment	Acid treatment	2nd PP treatment	3rd PP treatment	4th PP treatment
	8/20 PM	8/21 AM	8/21 PM	8/22 PM	8/27 PM	8/28 PM
SWL	5.53	7.69	7.68	7.85	8.76	9.50
PWL	21.55	21.40	18.48	17.55	18.50	18.91
s	16.02	13.71	10.80	9.70	9.74	9.41
Piez.	28.25	30.0	37.5	40.25	41.5	40.25
Q	647	668	754	783	795	783
Q/s	40.4	48.7	69.8	80.7	81.6	83.2

I-70 No. 10

	Pretreatment	1st PP treatment	Acid treatment	2nd PP treatment	3rd PP treatment	4th PP treatment
	8/14 AM	8/14 PM	8/15 PM	8/16 PM	8/19 PM	8/20 PM
SWL	29.66	29.40	29.38	29.20	29.04	29.24
PWL	40.60	36.53	36.41	35.15	35.30	35.46
s	10.94	7.13	7.03	5.95	6.26	6.22
Piez.	15.25	13.5	13.5	14.0	16.0	18.5
Q	484	457	457	465	495	530
Q/s	44.2	64.1	65.0	78.2	79.1	85.2

Table 5. Continued

	Pretreatment	1st PP treatment	Acid treatment	2nd PP treatment	3rd PP treatment	4th PP treatment
I-70 No. 11						
	7/25 PM	7/26 PM	7/27 PM	7/30 AM	7/31 AM	8/6 AM
SWL	21.29	21.45	21.30	22.06	21.87	21.25
PWL	35.30	29.76	30.16	29.94	29.35	28.80
s	14.01	8.31	8.86	7.88	7.48	7.55
Piez.	10.0	17.0	25.0	26.5	28.0	28.25
Q	393	510	608	626	644	647
Q/s	28.1	61.4	68.6	79.4	86.1	85.7
I-64 No. 15						
	7/29 PM	**7/30** AM	**7/31** AM	**8/1** PM	8/5 AM	8/5 PM
SWL	11.41	11.29	11.42	11.48	11.30	11.31
PWL	19.89	18.54	19.15	18.80	18.66	18.79
s	8.48	7.25	7.73	7.32	7.36	7.48
Piez.	23.0	21.5	22.25	22.25	22.5	23
Q	584	566	575	575	578	584
Q/s	68.9	78.1	74.4	78.6	78.5	78.1
Venice No. 1						
	8/29 AM	**8/29** PM	**8/30** PM	**9/3** PM	9/4 PM	9/5 PM
SWL	18.33	17.95	18.14	18.18	18.13	18.37
PWL	35.03	32.62	32.74	31.25	31.06	31.85
s	16.70	14.67	14.60	13.07	12.93	13.48
Piez.	35	38.25	39.75	45	45	44
Q	726	762	778	828	828	820
Q/s	43.5	51.9	53.3	63.4	64.0	60.8

Table 5. Concluded

	Pretreatment	1st PP treatment	Acid treatment	2nd PP treatment	3rd PP treatment	4th PP treatment
Averages						
Q/s	45.7	60.1	64.2	73.4	75.3	75.0
Δ(Q/s)	14.4	4.1	9.2	1.9	-0.3	(Note)
% increase over original specific capacity	31.5	9.0	20.1	4.6	-0.9	
% of total improvement	49.0	13.9	31.3	7.1	-1.4	

Note: Total Δ(Q/s) - 29.4 (64.3% improvement over original Q/s)

* Reading collected at beginning of treatment
** Data reported by contractor

SWL - Static water level below temporary measuring point (ft)
PWL - Pumping water level below temporary measuring point (ft)
s - Drawdown (ft)
Piez. - Orifice tube piezometer reading (in.)
Q - Gallons per minute (gpm)
Q/s - Specific capacity (gpm/ft)
PP - Phosphate

Table 6. Results of ISWS Step Tests on IDOT Wells Following Treatment

Well	Date of test	Well loss @ 600 gpm (ft)	Drawdown @ 600 gpm (ft)	Well loss portion (%)	Specific capacity (gpm/ft)	Δh* @ 600 gpm (ft)	Remarks
I-70							
No. 1	8/14/85	**	8.89 e	**	67.5 e	3.3 e	Q_{max} = 390 gpm
No. 2	8/15/85	**	8.32 e	**	72.1 e	--	Piezometer plugged, Q_{max} = 410 gpm
No. 8	12/5/85	0.07	6.83	1.0	87.8	2.21	Q_{max} = 750 gpm
No. 10	9/4/85	0.66	6.61 e	10.0	90.8 e	--	Piezometer plugged, Q_{max} = 490 gpm
No. 11	9/5/85	**	5.63	**	106.6	--	Piezometer plugged
I-64							
No. 15	8/13/85	0.71	7.24	9.8	82.9	2.97	Q_{max} = 615 gpm
Venice							
No. 1	12/4/85	0.39	7.89	4.9	74.5	2.33	Q_{max} = 870 gpm
Averages							
Post-treatment				6.4	83.2	2.70	
Pre-treatment				12.5	42.1	9.56	

e-Estimate
*-Head difference between pumped well and adjacent piezometer
**-Coefficient undeterminable. Possible well development. Turbulent well loss negligible over the pumping rates tested.

Table 7. Results of Chemical Treatment

Site	Well		Pre-treatment Date	Pre-treatment Q/s (gpm/ft)	Post-treatment Date	Post-treatment Q/s (gpm/ft)	% Change
I-70							
	No. 1	ISWS	8/15/84	33.1	8/14/85	67.5	+104
		AASI	7/18/85	39.5	7/27/85	69.0	+ 75
	No. 2	ISWS	7/19/83	50.4	8/15/85	72.1	+43
		AASI	8/6/85	55.6	8/12/85	63.2	+14
	No. 8	ISWS	8/1/84	44.3	12/5/85	87.8	+ 98
		AASI	8/20/85	40.4	8/28/85	83.2	+106
	No. 10	ISWS	7/11/84	35.4	9/4/85	90.8	+156
		AASI	8/14/85	44.2	8/20/85	85.2	+93
	No. 11	ISWS	8/2/84	38.6	9/5/85	106.6	+176
		AASI	7/25/85	28.1	8/6/85	85.7	+205
I-64							
	No. 15	ISWS	6/29/83	60.4	8/13/85	82.9	+ 37
		AASI	7/29/85	68.9	8/5/85	78.1	+13
Venice							
	No. 1	ISWS	11/30/83	32.7	12/4/85	74.5	+128
		AASI	8/29/85	43.5	9/5/85	60.8	+40
Average		ISWS		42.1		83.2	+98
		AASI		45.7		75.0	+64

Q/s - Specific capacity
ISWS - Illinois State Water Survey
AASI - Aylor Aqua Services, Inc.

Evaluation of Ground-Water Quality

Ten wells were sampled for analysis by the ISWS Analytical Laboratory. The results are reported in Appendix B. Analytical methods conform to procedures presented in the 16th edition of Standard Methods for the Examination of Water And Wastewater (1985). Samples were preserved with acid for determining iron, calcium, and magnesium concentrations. The sample temperature was determined at each well site, and pH was determined in the laboratory immediately after transit of the samples. The ranges of concentrations and anticipated influence of each parameter are presented in table 8.

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

Table 8. Ranges of Concentrations and Potential Influence of Common Dissolved Constituents

Parameter	Concentration, mg/1 Min.	Concentration, mg/1 Max.	Potential influence
Iron (Fe)	5.6	17.9	Major - incrustative
Calcium (Ca)	182.0	254.0	Major - incrustative
Magnesium (Mg)	42.4	62.4	Minor - incrustative
Sodium (Na)	33.1	179.0	Neutral
Silica (SiO_2)	20.0	35.6	Minor - incrustative
Nitrate (NO_3)	<0.6	2.9	Neutral
Chloride (C1)	30.0	140.0	Moderate - corrosive
Sulfate (SO_4)	310.0	710.0	Major - corrosive
Alkalinity (as $CaCO_3$)	396.0	488.0	Major - incrustative
Hardness (as $CaCO_3$)	628.0	890.0	Major - incrustative
Total Dissolved Solids	1011.0	1580.0	Major - corrosive
pH	6.9	7.1	Major - incrustative

Demonstration of Portable Flow-Rate Meter

The Phase 1 report (Sanderson et al., 1984) recommended that IDOT acquire a portable flowmeter so that the collection of individual well discharge data could be added to the current monitoring program for the 48 dewatering wells. Currently such data are almost totally unavailable for the dewatering well system because of problems with the mechanically operated flowmeters originally installed. The study further recommended that primary consideration be given to an ultrasonic-pulse time-difference flowmeter with dry type (non-invasive) transducers. This type of flowmeter was tested during Phase 2 by using a demonstration unit supplied by the distributor.

Results of the testing were presented in the Phase 2 report (Sanderson et al., 1987). The testing showed that the instrument has some limitations in its use and absolute accuracy. However, the trained operator through careful use should be able to obtain relatively consistent pumping rate data that can serve for comparative purposes.

Since the tests in Phase 2 indicated some use characteristics not originally anticipated, it was decided that IDOT staff should also have an opportunity to see the flowmeter demonstrated before purchase. Such a field demonstration was arranged during a step test on I-70 Well No. 11. During this demonstration, the performance of the meter was similar to that found during the testing previously conducted during Phase 2. IDOT subsequently agreed with the conclusions about meter performance presented in Phase 2 and confirmed that it should be purchased. Possession of the flowmeter is to remain with the Water Survey for future testing and calibration purposes and for eventual training of one or two IDOT operators.

GROUND-WATER CHEMISTRY INVESTIGATION

Background

Water chemistry and step-test data results for Phase 1 suggested that the ground water at the dewatering sites has a significant potential for deposition of minerals on the aquifer and gravel pack material, which would induce deterioration in dewatering well performance.

In Phase 2, the ground-water chemistry was investigated through the use of careful sampling techniques, precise and accurate research analyses, and computerized geochemical models to determine the chemical mechanism causing the deterioration. The major part of the chemical evaluation for this phase was the collection of samples from the production well (I-70 No. 3) and an array of 12 monitoring wells installed and cased with PVC. Samples were taken after various periods of production well pumping and stagnation. The intent was 1) to obtain an idea of the constituents in the water that might govern chemical reactions that could cause well plugging; and 2) to determine the accuracy and reproducibility of sampling and chemical analysis. This would help determine if it would be possible to infer the location and mechanism of chemical reactions that might be able to cause loss of well capacity (presumably through precipitation of minerals).

The monitoring wells were installed in pairs (~30 feet and -95 feet deep) at two distances (~6 feet and -21 feet) and in three directions (north, west, and east) from I-70 No. 3 (see figure 13, which includes the locations of four additional wells drilled for Phase 3). The monitoring wells were equipped with 2 feet of well screen.

To determine the length of time it takes water to pass from the outer monitoring wells to the inner ones, a brief tracer study was conducted. The tracer study indicated that the water flow field about I-70 Well No. 3 was highly distorted and not as expected. The water levels and the flow

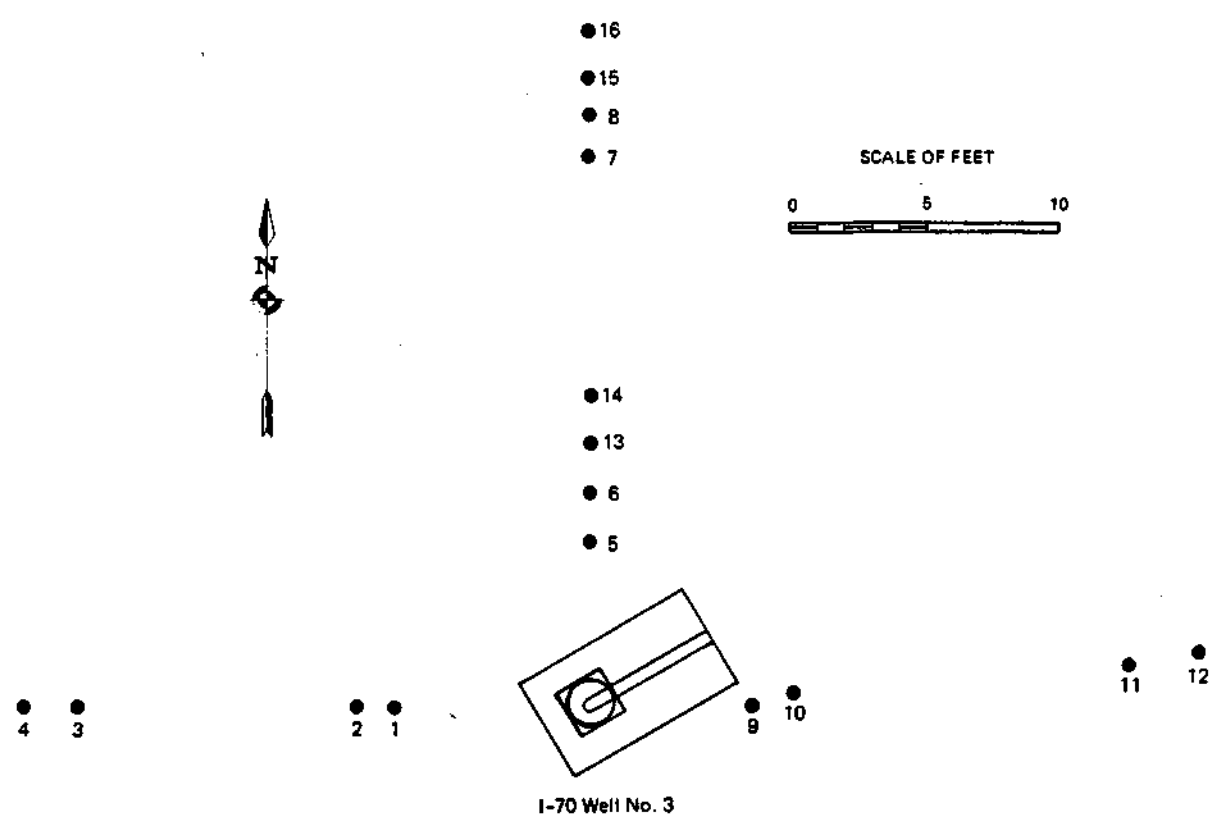


Figure 13. Locations of monitoring wells at I-70 Well No. 3 (Wells 1 through 12 were used for Phase 2 of the study; wells 13 through 16 were added for Phase 3)

paths were highly sensitive to the operation of the other wells in the field. In addition, the water levels in the shallow monitoring wells were much lower than expected. These factors, coupled with the inhomogeneity of the water quality about the production well, made it impossible to conduct sampling with pumping and flow conditions as controllable as originally intended.

New analytical methods were developed or implemented for several important chemical constituents, including ferrous iron, inorganic carbon, sulfate, nitrate, nitrite, ammonia, silica, and total phosphate. The high level of dissolved iron, and the high amount of silica relative to phosphate, resulted in interference with many of the generally accepted "standard" procedures. Most of the new analytical methods developed had precision and accuracy exceeding those normally expected from standard procedures; therefore, not much additional delineation of trends is possible by improving analytical procedures and taking replicate field samples. Several procedures do need additional effort to overcome certain interferences that remain unresolved.

Some computer analysis of the data in terms of mineral saturation/precipitation equilibria was begun, and the precision of field sampling and the precision and accuracy of the analytical methods were analyzed. Significant conclusions from Phase 2 were:

1) There is great variability in the chemical composition of major water constituents between monitoring wells on one side of the production well and those on another side. There also is a great difference in chemical composition between the deep monitoring wells and their shallow counterparts.

2) The shallow wells apparently represent a much more oxidizing environment, because they are in or near the saturated/unsaturated zone boundary. This favors ferric iron precipitation and, therefore, plugging of the aquifer material. Iron levels were found to be very low in these wells.

3) Drawdown from the other nearby production wells is so great that samples can be taken from just two or three of the shallow monitoring wells and then only when Well No. 3 has been off. Thus deeper replacement monitoring wells must be added to accurately sample vertical variability of water composition.

4) The water flow paths to the production well are highly distorted by the other wells in the field.

5) Reliable samples cannot be collected with the high-capacity pump currently installed in the production well.

6) The design of the production well (I-70 No. 3) and pump may be a contributor to incrustation, by raising the water pH through degassing of CO_2 and introduction of oxygen, facilitating the precipitation of various carbonates and iron hydroxide.

The inhomogeneity of the water quality may reflect both vertical and lateral differences in aquifer material composition and chemical environment. Because of the distortion of the flow paths, the most efficient and effective data collection should be obtained by concentrating on the one direction (N-S) that is most direct, and adding more wells to cover the indicated vertical variability.

Some sensitivity to ground-water recharge might be indicated in the chemical data; thus a study that would give a better indication of possible seasonable effects appeared desirable.

The early computer modeling results provided an indication that some potentially important chemical relationships involving deposition potential could be defined. A wider variety of geochemical models was planned for use during Phase 3 to further define these relationships following expansion of the ISWS VAX computer system floating-point computation capability.

The chemical differences between the deep and shallow monitoring wells indicated that there could be considerable potential for incrustation in the upper part of the saturated zone at the saturated/ unsaturated zone interface because of iron oxidation resulting from aeration. Whether or not the low iron concentrations in the zone represent post-deposition conditions or a lack of the iron mobility, as takes place under more reducing conditions at greater depths, could not be determined. The water analyses and modeling do indicate that there is a great potential for deposition of a variety of carbonate minerals, with only subtle changes in pH or redox conditions. These changes could conceivably be induced by the production well pump itself. If the deposition occurs away from the production well, the elucidation of probable deposition reactions would give a good clue as to the most appropriate rehabilitation techniques, and as to whether or not the problem will keep recurring.

Reliable samples could not be obtained from the production well for many constituents critical to the geochemical modeling. It was recommended that future work concentrate on devising a representative sampling system, and on attempting to isolate the chemical changes in the water that might be induced by the pump and how that would relate to the depositional tendency of the water as estimated by the samples from the closest monitoring wells.

Well Head Modification and Sampling Well Construction

As part of the continuation of the ground-water chemistry study in Phase 3, an access port was installed in the well coverplate of I-70 Well No. 3 so that a positive displacement bladder pump, such as is used for the sampling of monitoring wells, could be used to extract samples under conditions of less disturbance. Samples were collected with the bladder pump set below the well pump at depths of 45 and 90 feet, which locate it within the upper and lower sections of the well screen. Experience with production well samples in Phase 2 showed that it was otherwise impossible to obtain samples without volatile gas (O_2 or CO_2) exchange and the consequent alteration of pH, iron, and carbonate chemistry. All of these factors are intimately involved with the tendency for incrustation to take place in a well.

An additional four monitoring wells were drilled on the north side of the production well (I-70 No. 3) to try to better intercept water along the presumed flow path, and to obtain samples from the zone that would alternate between saturation and unsaturation, depending on recharge volume and pump operation pattern (figure 14). This zone is of particular interest because of the periodic change in redox conditions, which could drastically affect iron solubility. The presence of significant residual dissolved oxygen could result in the deposition of insoluble ferric oxyhydroxide instead of the more soluble ferrous carbonate with which the water is apparently in a state of equilibrium or even metastable supersaturation (the water is oversaturated, but precipitation is slow to occur). In Phase 2, adequate samples could not usually be obtained from the shallowest wells, which were often dry. The two monitoring wells in each of the northern sets (close and far clusters) that were added were drilled to depths of approximately 41 and 70 feet.

The monitoring wells were constructed with the use of a hollow-stem drilling rig operated by IDOT District No. 8. For each monitoring well, the hole was augered to the desired depth, the hollow stem was washed clean to the bottom, and a 2-inch-diameter PVC casing with a 2-foot length of screen was inserted to the bottom of the hole. The augers were then pulled from the hole. The sandy formation materials collapsed around the casing to near the water table elevation, usually about 25 to 30 feet below land surface. A bentonite plug 1½ to 2 feet long was then formed by using 50 pounds of bentonite pellets. The remainder of the hole was filled with drill cuttings, and a lockable well protector was placed at the top.

Chemical Sampling and Geochemical Modeling Results

In Phase 3, samples were collected only from the northern row of monitoring wells and the production well at two depths (45 and 90 feet). Samples were taken approximately monthly for five months. The results from the chemical analyses of the samples are presented in Appendix C. Vertical concentration differences were significant for many parameters. For instance, chloride and sodium concentrations increased greatly with depth. Conversely, calcium, inorganic carbonate (and alkalinity), and sulfate concentrations, along with the redox potential (Eh), were significantly higher in the shallow wells. Similar vertical concentration differences were seen in the production well samples extracted by the bladder pump. Concentrations of the monitored parameters did not display obvious seasonability and were most likely linked to recharge and drainage events, or to the use of other dewatering wells in the well field.

Calculations of saturation states of potential precipitating solids by the computerized geochemical model WATEQFC on the ISWS VAX mainframe computer were performed as described in the previous Phase 2 report (Sanderson et al., 1987). The geochemical model also allowed the computation of ion balances that could be used to evaluate the

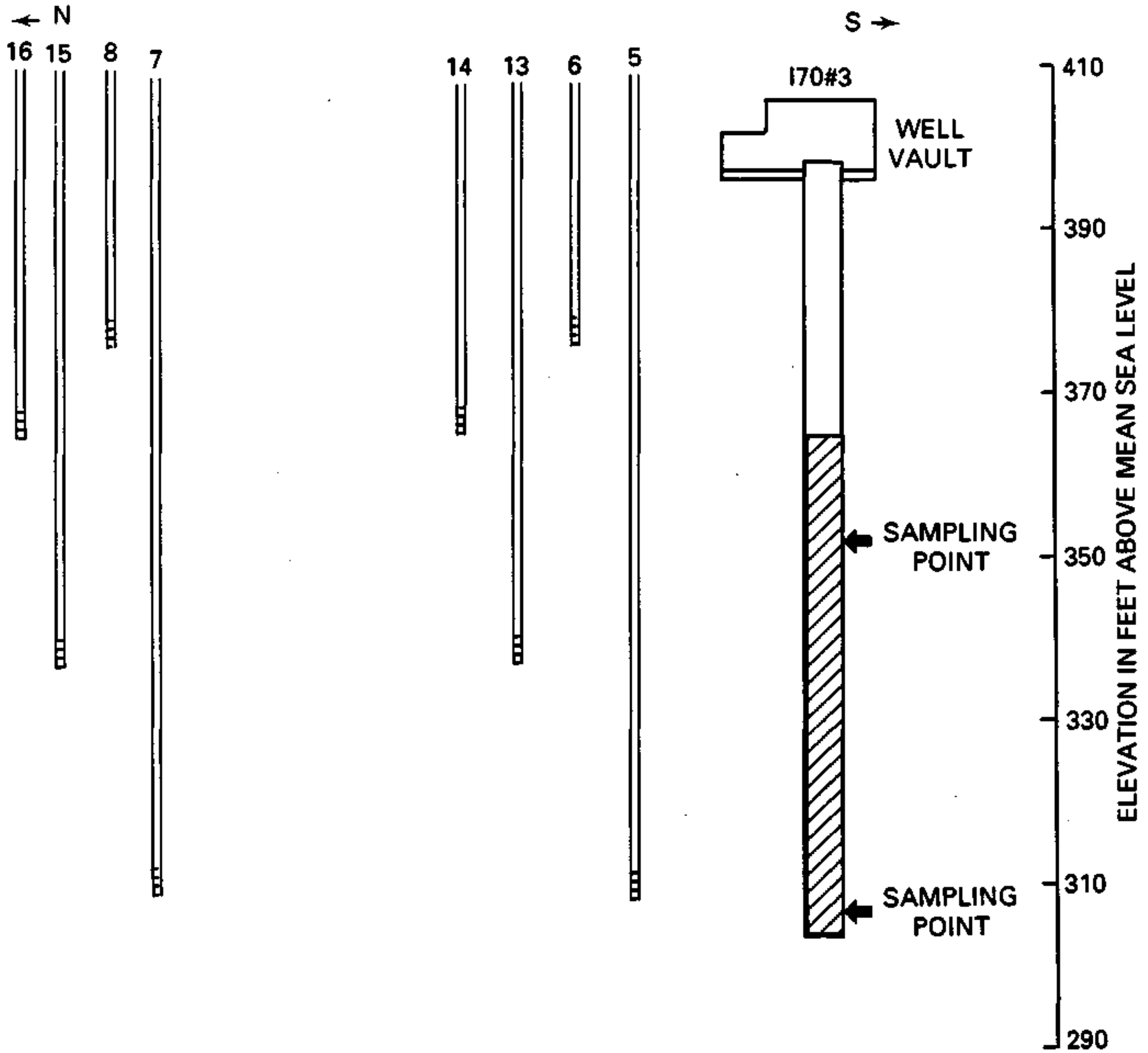


Figure 14. Relative locations of north set of monitoring wells at I-70 Well No. 3

completeness and accuracy of the determinations of the chemical constituents in each sample. The ion balances routinely checked within ±3% for almost all samples. The significance of several of the computed indices was described in the Phase 2 report.

The saturation indices indicated some variation over time, but the waters were all essentially oversaturated with calcite ($CaCO_3$) and ferrous carbonate (siderite, $FeCO_3$), and were near saturation with dolomite ($CaMg(CO_3)_2$) and amorphous silica (SiO_2). Plots of two of the saturation indices ($SI_{siderite}$, $SI_{calcite}$) are shown in figures 15 and 16.

The hydrologic modeling was not detailed enough to accurately determine if the sampling wells were allowing access to points directly along the flow path to the production well. The flow path also undoubtedly changes with the number and location of other proximate wells pumping at a given time. Therefore actual precipitation of solids along the flow path could not be detected through the water analyses. Aquifer solid samples taken at the time the initial monitoring wells were drilled could not be adequately preserved against changes brought about by normal atmospheric contact, and adequate analytical instrumentation (X-ray diffraction, polarizing light microscopy, luminescence petrographic microscopy) for solids characterization was not available to the Water Survey when needed. Therefore, there was no physical evidence to confirm or deny precipitation and consequent loss of permeability in the saturated zone along the flow path into the production well.

Information from step tests indicated that the decrease in permeability that restricted the flow into the production well probably was located within about 5 feet of the well, or within the gravel pack itself. These areas were inaccessible to chemical sampling. Differences between the water chemistry of production well samples and the chemistry of samples from the innermost monitoring wells could not be used because: 1) there was spatial variability in water chemistry at the same depth, but at different points around the well; 2) the volume contribution of water represented by the innermost monitoring well of the appropriate depth could not be ascertained by the hydrologic modeling, and could not be consistently sampled; 3) there was vertical variability in water composition; and 4) the production well screen was long, which introduced water from a range of depths, representing a suite of ground-water chemical compositions.

Summary of Chemical Observations

In spite of the difficulties involved in the flow modeling, chemical sampling, and monitoring efforts, several important conclusions can be drawn from the data and geochemical modeling calculations. One is that continued periodic scaling and incrustation of the pump, well screen, and aquifer or gravel pack in the near proximity to the well are inevitable. Several factors contribute to this. First, the high degree of saturation of carbonate minerals of calcium, calcium+magnesium, strontium, and ferrous iron in the ground water in the vicinity of the well means that disturbance of the delicate equilibrium relationships among the dissolved constituents (the Fe(II), calcium, magnesium, etc.), pH, inorganic

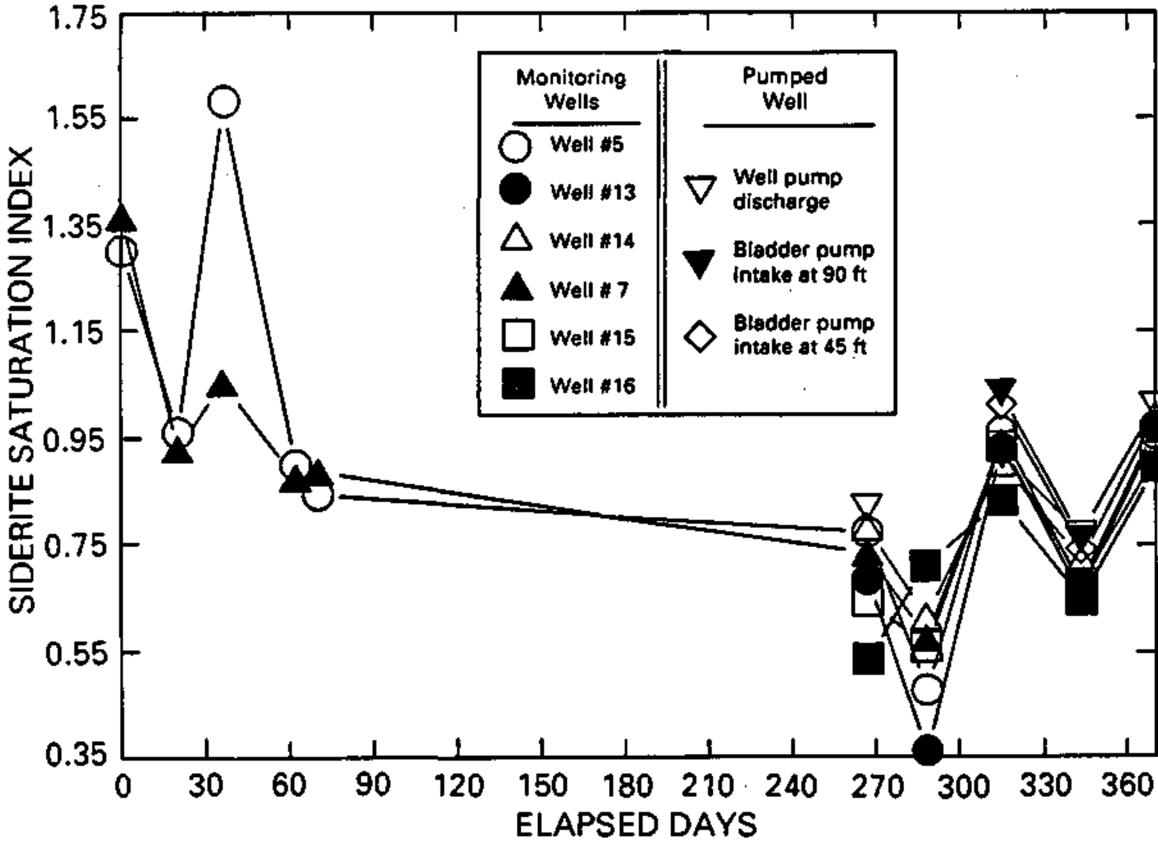


Figure 15. Plot of the saturation index for ferrous carbonate (siderite) for the north line of monitoring wells and I-70 Well No. 3, showing the probable control of iron concentration by this mineral or an impure form (The well locations are given in figures 13 and 14; the elapsed days count from November 6, 1984)

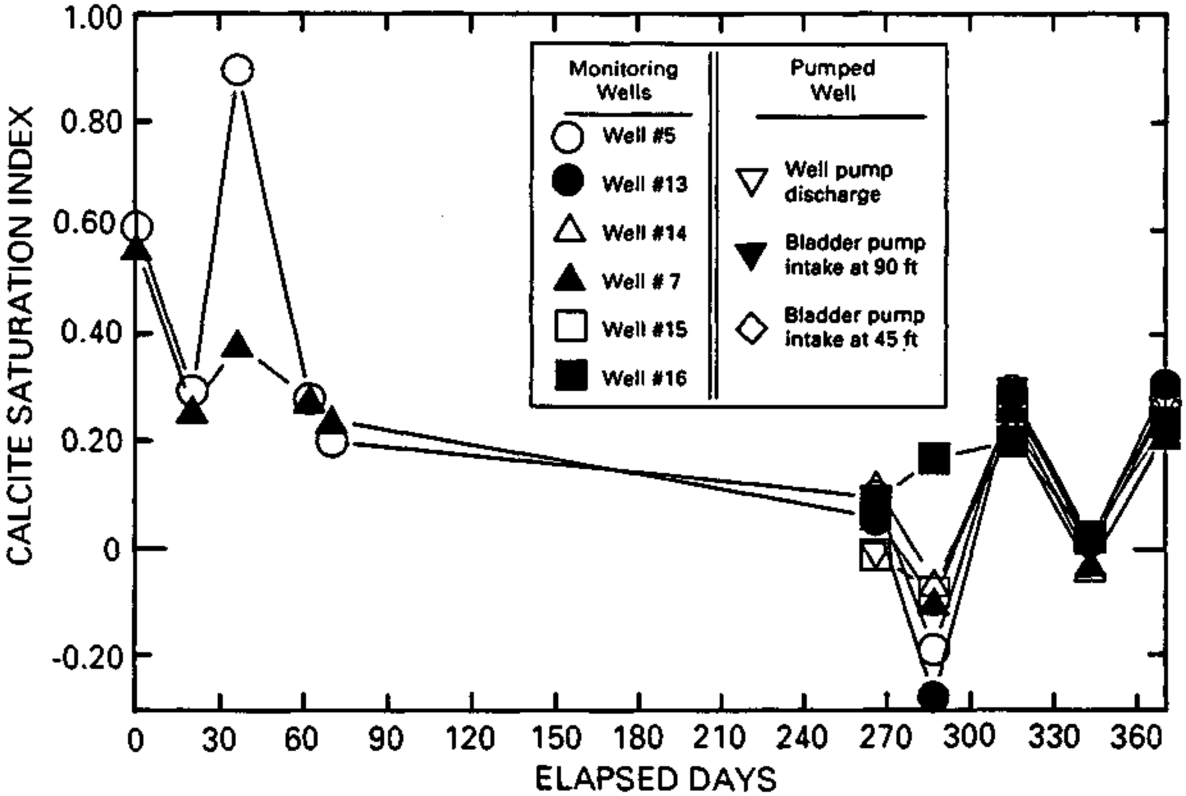


Figure 16. Plot of the saturation index for calcium carbonate (calcite) for the north line of monitoring wells and I-70 Well No. 3 (The well locations are given in figures 13 and 14; the elapsed days count from November 6, 1984)

carbonate concentration, and carbon dioxide gas partial pressure can easily induce precipitation. Degassing of carbon dioxide from pressure drops caused by passage through well screens or disturbance by pump-induced water turbulence, or exchange with the atmosphere above the water level in the production well, would induce an increase in water pH. This, in turn, would allow mineral deposition at the points of disturbance. Specifically, precipitation would be likely at the pump, in the gravel pack and aquifer materials next to the well, and on the screen starting at the top and progressing downward as the flow path through the upper screen becomes restricted.

A second cause of permeability decrease can be the alternating oxidizing and reducing conditions in the aquifer and gravel pack near the well, where the condition varies between saturated and unsaturated (with water) when the water level draws down during pumping and recovers when pumping is shut off (or when other nearby wells are shut off). Ferrous iron is readily oxidized to the relatively less soluble ferric form (Fe(III)) by dissolved oxygen, and ferric oxyhydroxide precipitate forms under the pH conditions of these waters. Such reactions are probably only partially and slowly reversible, so over time the well will be forced to extract water from a smaller vertical portion of the gravel pack or aquifer. The measurements of dissolved iron and Eh indicate favorable conditions for oxidation of iron within the well at the air/water interface, and during mixing with air possibly brought into contact with the water during pumping. The precipitates formed by this mechanism can be an additional source of material clogging the well screen and pump components.

Whether or not any alternative designs of well screen or the use of continuous pumping by more wells at reduced rates would lengthen the time to failure is not known. The high ferrous iron concentration would indicate that well rehabilitation programs should concentrate on the use of inhibited acids (to protect the stainless steel well screens and casings) or sequestering agents for iron, and not on the use of oxidizing agents normally used for disinfection, such as hypochlorite or related chemicals. Adequate compensation for the high buffering capacity of the water and the sequestering capacity used by iron, calcium, and magnesium must be made when required chemical dosages are calculated.

CONCLUSIONS AND RECOMMENDATIONS

Condition of Wells

The detailed step tests have provided data that allow a comparison of the condition of the wells. As presented earlier in the field results, the data show that I-70 Well No. 6, I-64 Wells 2, 4, and 12, and 25th Street Well No. 3 are all in good condition. The 30-minute specific capacities of these five wells on the dates of testing were all greater than 96 gallons per minute per foot (gpm/ft) of drawdown. The differences in water levels between the wells and piezometers were less than about 5.2 feet (estimated at I-64 Wells 2 and 12 because of the pumping rate adjustment to 600 gpm). The well loss portion of drawdown for these wells is very low with the exception of I-64 No. 4 (15%). Because of their favorable condition, rehabilitation work is not recommended on any of these wells at this time. However, their performance should be closely monitored, particularly I-64 Wells 2 and 4 where at least one of the step test parameters approaches a marginal value.

The remaining well (25th Street Well No. 10) is in moderately good condition with a specific capacity of 62.8 gpm/ft and a water level difference between the well and piezometer of 3.59 ft. Because of laminar flow conditions or possible well development, well loss could not be determined at this well. Although the condition of this well is moderately good, the specific capacity is about 1/3 less than the average for wells at this site (96.4 gpm/ft) and likely would improve with treatment.

Well Rehabilitation

Seven dewatering wells (I-70 Nos. 1, 2, 8, 10, and 11; I-64 No. 15; and Venice No. 1) were rehabilitated by Aylor Aqua Services, Inc., while being monitored by ISWS staff. The treatment was successful in improving the condition of the wells by increasing specific capacity, lowering well loss, and decreasing the head differences between the wells and piezometers (Ah). Specific capacity was increased by an average of almost 100% (64% according to the contractor's data) with substantial improvement (40% or greater) noted for all but two wells. However, the two wells exhibiting the least improvement also had the highest pre-treatment specific capacity. The well loss component of drawdown was reduced on the average from 12.5% of total drawdown to 6.4%, and the well/piezometer head differences were reduced on the average from about 9.6 to 2.7 feet.

Apparently much of the blockage causing the well deterioration occurs within the first few feet of the well, as most (94%) of the improvement occurred after the first two polyphosphate treatments, which concentrate on this zone. It appears that the third and fourth phosphate treatments resulted in little immediate improvement to the wells (in fact, the data suggest a decrease after the fourth treatment), although improvement evidently continues for some time after treatment as indicated by the follow-up step test data. Although some additional treatment work probably should be done for confirmation, the fourth and perhaps even the third applications of polyphosphate are candidates for omission in the future.

In light of the success of the well treatment, adjustments in the treatment process appear unnecessary at this time, other than the possible elimination of the last application or two of polyphosphate. Close monitoring of the treated wells over time will be important to learn how long the restored capacity remains.

Flowmeter

IDOT staff were given a field demonstration of the portable, non-invasive ultrasonic flowmeter, PNUF, described in the Phase 2 report, prior to a final decision concerning its purchase. IDOT subsequently agreed with the conclusions presented in Phase 2 about the meter and agreed that it should be purchased for further evaluation and calibration by the ISWS. Following the period of evaluation, IDOT staff will be trained in its use.

Ground-Water Chemistry

As part of the continuation of the ground-water chemistry study in Phase 3, adjustments were made to try to circumvent the sampling problems experienced in Phase 2. An access port was installed in the well coverplate of I-70 Well No. 3 so that a positive displacement bladder pump, such as is used for the sampling of monitoring wells, could be used to extract samples under conditions of less disturbance.

Previous study had indicated that the flow path of the water is least distorted as it approaches Well No. 3 from the north. Therefore an additional four monitoring wells were drilled on the north side of the production well to try to better intercept water along the flow path and to obtain samples from the zone that would alternate between saturation and unsaturation.

In Phase 3, only the northern sets of monitoring wells and two depth locations (45 and 90 feet) in the production well were sampled. Samples were taken approximately monthly for five months. Vertical concentration differences were significant for many parameters.

The hydrologic modeling was not detailed enough to accurately show whether the sampling wells were allowing access to points directly along specific flow paths to the production well. The flow paths also undoubtedly change with the number and location of other proximate wells pumping at a given time. Therefore, actual precipitation of solids along the flow paths could not be detected through the water analyses. Because the aquifer material samples collected at the time the monitoring wells were drilled could not be properly collected and analyzed for solids characterization, there was no physical evidence to confirm or deny precipitation and consequent loss of permeability in the saturated zone along the flow path into the production well.

In spite of the difficulties involved in the flow modeling and chemical sampling efforts, several important conclusions can be drawn from the data and geochemical modeling calculations:

1) Continued periodic scaling and incrustation of the pump, well screen, and aquifer or gravel pack in the near proximity to the well are inevitable because of the mechanisms highlighted in numbers 2, 3, and 4 below.

2) The high degree of saturation of carbonate minerals of calcium, calcium+magnesium, strontium, and ferrous iron in the ground water in the vicinity of the well means that disturbance of the delicate equilibrium relationships among the dissolved constituents (the Fe(II), calcium, magnesium, etc.), pH,

inorganic carbonate concentration, and carbon dioxide gas partial pressure can easily induce precipitation. The equilibrium can be disrupted by turbulence caused by pumping or head loss across the well screen and gravel pack.

3) The alternating oxidizing and reducing conditions in the aquifer and gravel pack near the well, where the condition varies between saturated and unsaturated (with water) when the water level draws down during pumping and recovers when it is shut off (or when other nearby wells are shut off), are favorable for the precipitation of ferric oxyhydroxide.

4) The measurements of dissolved iron and the redox potential (Eh) indicate favorable conditions for the oxidation of iron within the well at the air/water interface, and during pumping when air is possibly brought into contact with the water.

5) The high ferrous iron concentration would indicate that well rehabilitation programs should concentrate on the use of inhibited acids (to protect the stainless steel well screens and casings) or sequestering agents for iron, and not on the use of oxidizing agents normally used for disinfection, such as hypochlorite or related chemicals. Adequate compensation for the high buffering capacity of the water and the sequestering capacity used by iron, calcium, and magnesium must be made when required chemical dosages are calculated.

Future Investigations

A program of continued investigation of the condition of additional dewatering wells is recommended. Comparison of the water level differences recorded during the controlled step tests with the historical water level differences reported during routine monitoring indicates that other wells may also need rehabilitation. However, a blanket inference of that need is not warranted on the basis of present data. On the basis of this comparison, step tests are recommended to properly assess the condition of the following wells: I-70 Wells 3 and 12; I-64 Well No. 1; and 25th Street Well No. 9.

Thirty-two dewatering wells have been step-tested as part of Phases 1, 2 and 3. Any of the previously tested and/or rehabilitated dewatering wells suspected of developing a deterioration problem can easily be retested as part of this program. Consideration should also be given to step-testing any replacement dewatering well(s) constructed. Such testing will help evaluate the effectiveness of the contractor's development of the well and the "as new" condition of the well for future reference.

The close monitoring of future rehabilitative treatments is also recommended to help verify the results of the treatment work completed in Phase 3. The analysis of the results from a number of well rehabilitations will be very helpful in determining if any adjustments in the treatment process are warranted.

BIBLIOGRAPHY

Arceneaux, W. 1974. Operation and maintenance of wells. Journal, American Water Works Association, March.

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.

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.

Cowan, J. C, and D. J. Weintritt. 1976. Water-formed scale deposits. Gulf Publishing Company.

Downes, M. T. 1978. An automated determination of low reactive phosphorus concentrations in natural waters in the presence of arsenic, silicon, and mercuric chloride. Water Research, 12:743.

Driscoll, F. G. 1986. Groundwater and wells. Johnson Division, St. Paul, Minnesota, 1089 p.

Emmons, J. T. 1979. Groundwater levels and pumpage in the East St. Louis area, Illinois, 1972-1977. Illinois State Water Survey Circular 134.

Freeze, R. A., and J. A. Cherry. 1979. Groundwater. Prentice-Hall, Englewood Cliffs, New Jersey, 604 p.

Garske, E. E., and M. R. Schock. 1986. An inexpensive flow-through cell and measurement device for monitoring selected ground water chemical parameters. Ground Water Monitoring Review, 6(3):79-84.

Jacob, C. E. 1947. Drawdown tests to determine effective radius of artesian well. Transactions American Society of Civil Engineers, v. 72.

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

Johnson, Depp, and Quisenberry. 1980. Venice subway dewatering. Consultant's report to the Division of Highways.

Kohlhase, R. C. 1987. Ground-water levels and pumpage in the East St. Louis area, Illinois, 1981-1985. Illinois State Water Survey Circular 168.

Larson, T. E. 1955. Report on loss of carrying capacity of water mains. Journal, American Water Works Association, November.

Larson, T. E., and A. M. Buswell. 1942. Calcium carbonate saturation index and alkalinity interpretations. Journal, American Water Works Association, November.

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.

McClelland Engineers, Inc. 1971. Highway I-70 drainage system at Tri-Level Bridge, East St. Louis, Illinois. Consultant's report to the Division of Highways.

Merrill, D. T., and R. L. Sanks. 1977. Corrosion control by deposition of $CaC0_3$ films, Part 2. Journal, American Water Works Association, December.

Mogg, J. L. 1973. Corrosion and incrustation guidelines for water wells. Water Well Journal, March.

Moss, R., Jr. 1966. Evaluation of materials for water well casings and screens. Proceedings, National Association of Corrosion Engineers Western Region Meeting, October.

Palmer, C. D., and J. A. Cherry. 1984/1985. Geochemical evolution of ground-water in sequences of sedimentary rocks. Journal Hydrol., 75:27.

Parkhurst, D. L., et al. 1980. PHREEQUE - A computer program for geochemical calculations. U.S. Geological Survey, WRI 80-96, 210 p.

Ritchey, J. D., R. J. Schicht, and L. S. Weiss. 1984. Groundwater level analysis by computer modeling--American Bottoms groundwater study. Illinois State Water Survey Contract Report 352A-E.

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.

Runnells, D. D., and R. D. Lindberg. 1981. Hydrogeochemical exploration for uranium ore deposits: use of the computer model WATEQFC. Journal of Geochemical Exploration, vol. 15, no. 37.

Rupani, N. 1976. Highway I-55 & 70 and I-64 deep drainage system at Tri-Level Bridge in East St. Louis, Illinois. Division of Highways file report.

Ryznar, J. W. 1944. A new index for determining amount of calcium carbonate scale formed by a water. Journal, American Water Works Association, April.

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.

Sanders on, 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.

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.

Skougstad, M. W., et al. 1978. Methods for analysis of inorganic substances in water and fluvial sediments. U.S. Geological Survey Open-File Report 78-679, 1005 p.

Standard methods for the examination of water and wastewater. 1985. 16th Edition, APHA-AWWA-WPCF, 1268 p.

Stauffer, R. E. 1983. Determination of arsenic and phosphorous compounds in ground water with reduced molybdenum blue. Analytical Chemistry, 55:1205-1210.

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. EPA 600/4-79-020, 460 p.

Walton, W. C. 1962. Selected analytical methods for well and aquifer evaluation. Illinois State Water Survey Bulletin 49.

A NOTE CONCERNING APPENDICES A, B, C, D, AND E

Five appendices to this report have been compiled. Appendix A provides the water level and pumping rate measurements collected during each of the step tests; Appendix B gives the results of chemical analyses on water samples collected near the end of each step test; Appendix C contains the results of chemical analyses performed on samples collected from the monitoring wells at I-70 Well No. 3; Appendix D provides the field note chronology for the dewatering well rehabilitations; and Appendix E gives the graphs of water level differences (Ah) between the dewatering wells and adjacent piezometers as measured by IDOT staff.

These appendices have been printed under separate cover as Illinois State Water Survey Contract Report 479A. A limited number of copies is available. Copies may be obtained by writing to the Illinois State Water Survey, Ground-Water Section, 2204 Griffith Drive, Champaign, Illinois 61820-7495 or by calling (217) 333-8888.