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DEWATERING WELL ASSESSMENT FOR THE HIGHWAY DRAINAGE SYSTEM AT FOUR SITES IN THE EAST ST. LOUIS AREA, ILLINOIS (PHASE 1)

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Robert D. Olson, and Chester H. Neff*

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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 groundwater 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 based on data collected and reviewed by IDOT staff. During the period March 1, 1983, to February 29, 1984, a cooperative investigation was begun by IDOT and the State Water Survey to more adequately assess the condition of selected individual wells, review monitoring procedures, and determine hydraulically efficient operating procedures for the dewatering system by means of a groundwater flow model.

During the project, 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 well losses of 10 to 43% of total drawdown, specific capacities of 34 to 71 gpm/ft of drawdown, and head differences between the wells and their adjacent piezometers of 4.6 to 15 ft, four wells were recommended for treatment and one well was recommended for replacement.

A groundwater flow model was developed and indicated that at two adjacent dewatering sites (1-70 and 1-64) pumpage should be shifted toward the 1-70 wells and away from the wells at the SE end of 1-64. The modeling efforts also defined groups of wells that perform a specific role in the overall system. These were termed functional groups. Each group permits well rotation within the group without damaging the effectiveness of the system. Once the functional groups were defined, the performance of the well groups was evaluated by comparison with actual pumping configurations. Using the pumping distribution suggested by the functional groups resulted in operating from 1 to 3 fewer wells for each of the modeled periods.

Recommendations in the report for this recently completed investigation (Phase 1) include suggestions for further study to assess the condition of additional wells and to begin an attempt to understand the probable causes of well deterioration.

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 groundwater levels at acceptable elevations to prevent depressed sections of interstate and state highways from becoming inundated by groundwater. When the interchange of 170-55 and 164 was originally designed, groundwater levels were at lower elevations due to large withdrawals by the area's industry. Due to a combination of water conservation, production cutbacks, and conversion from groundwater to river water as a source, groundwater withdrawals by industry have decreased about 50% since 1970, and as a result, groundwater levels in many areas have recovered to early development levels. This exacerbates the Department'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 the Department showed that deterioration problems were continuing to develop. Chemical treatment of isolated wells was made by Department personnel as required. In 1982, six more wells were installed. In October 1982, the Department 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. The first phase of the work, begun in 1983, includes an assessment of the condition of selected wells, a review of the Department's monitoring program, a model study to outline efficient operating schemes, recommendations on wells to be treated, and recommendations for chemical treatment procedures.

Future work will measure the effectiveness of rehabilitation by chemical treatment, further investigate the potential causes of well deterioration, and assess the condition of additional 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 groundwater hydrology of the area is well documented (Bergstrom and Walker, 1956; Schicht, 1965; Collins and Richards, 1984; Ritchey, Schicht, and Weiss, 1984). Groundwater generally flows from the bluffs toward the river, except where diverted by pumpage or drainage systems.

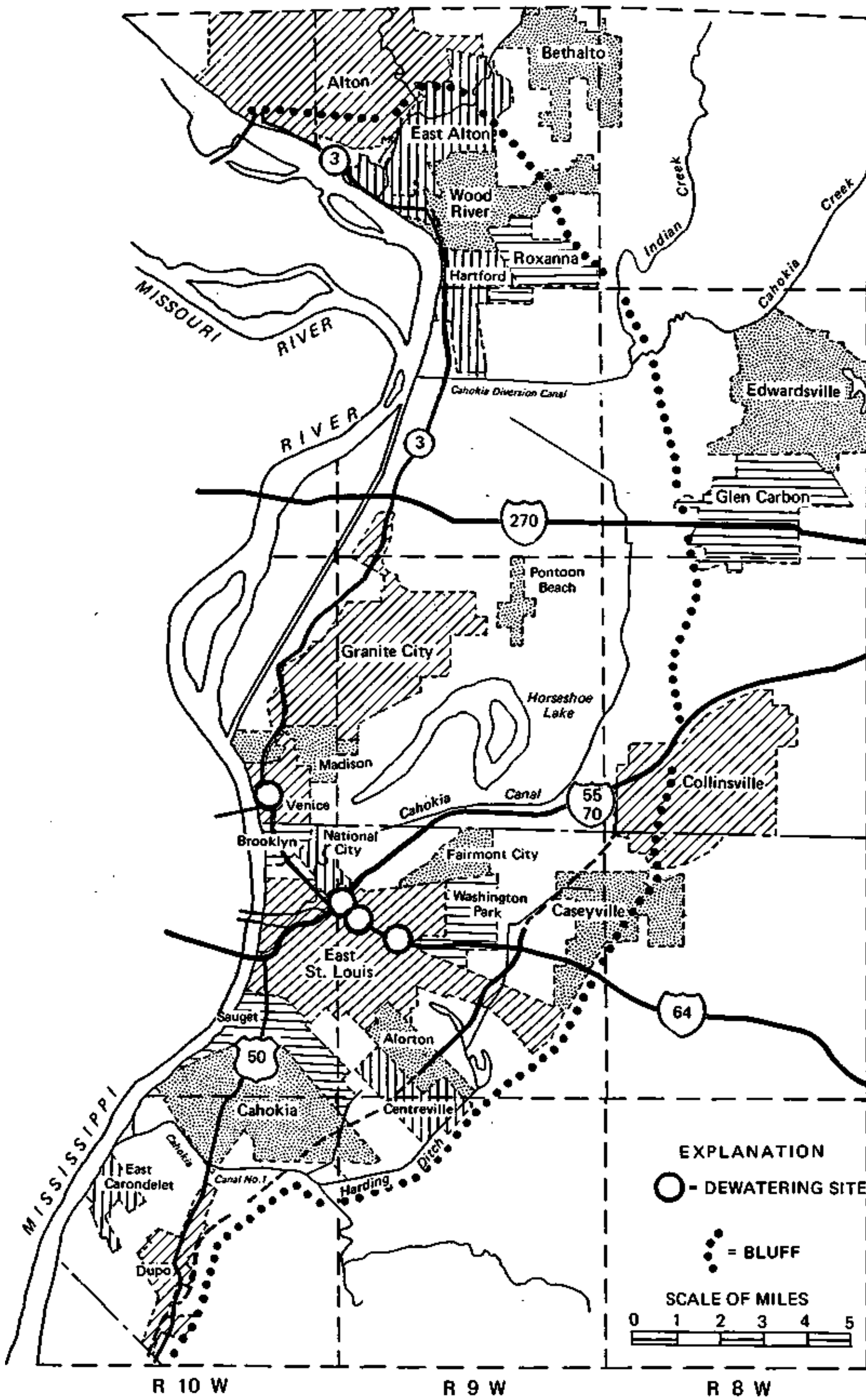


Figure 1. Location of the East St. Louis area

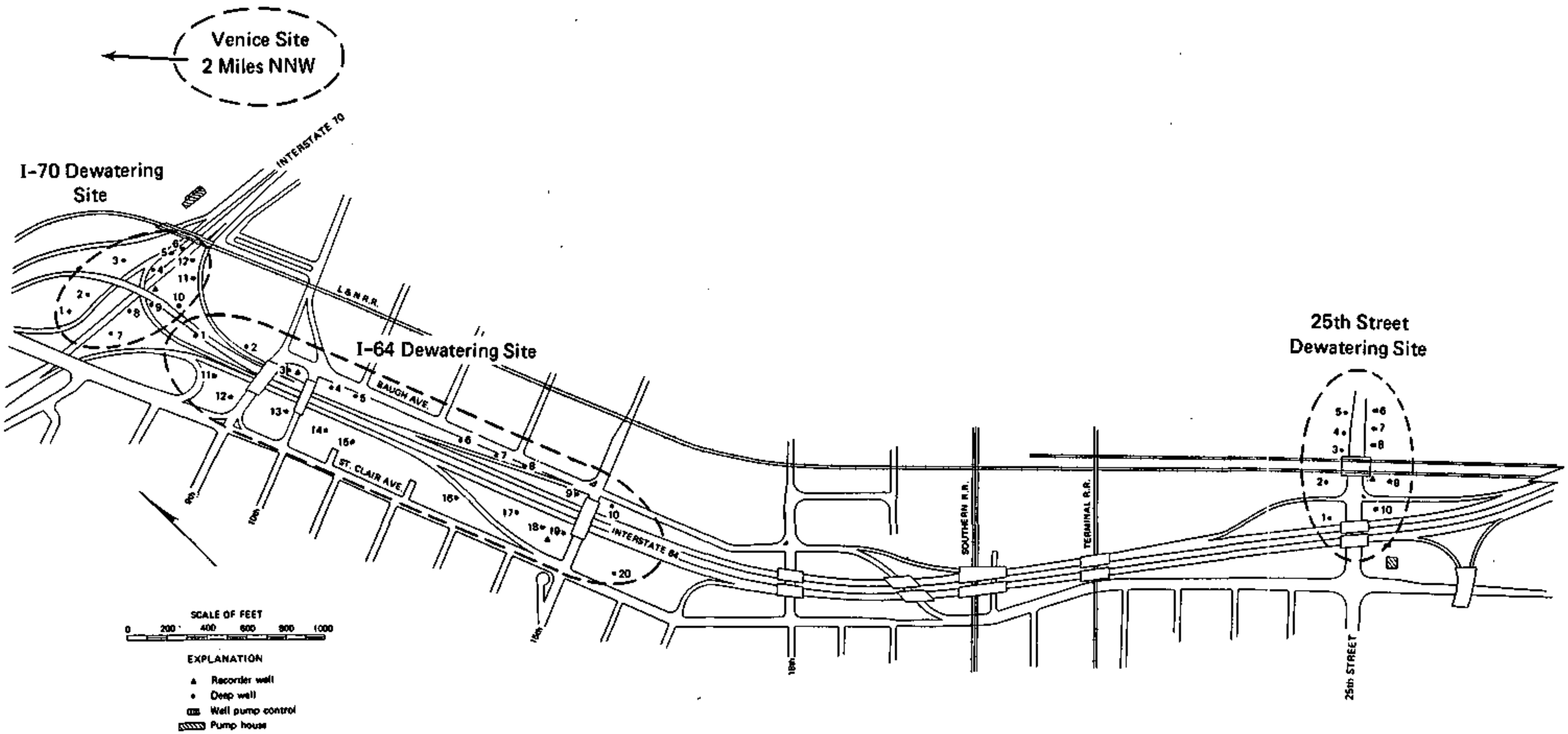


Figure 2. Location of dewatering wells at the 1-70 Tri-Level Bridge, 1-64, and 25th Street

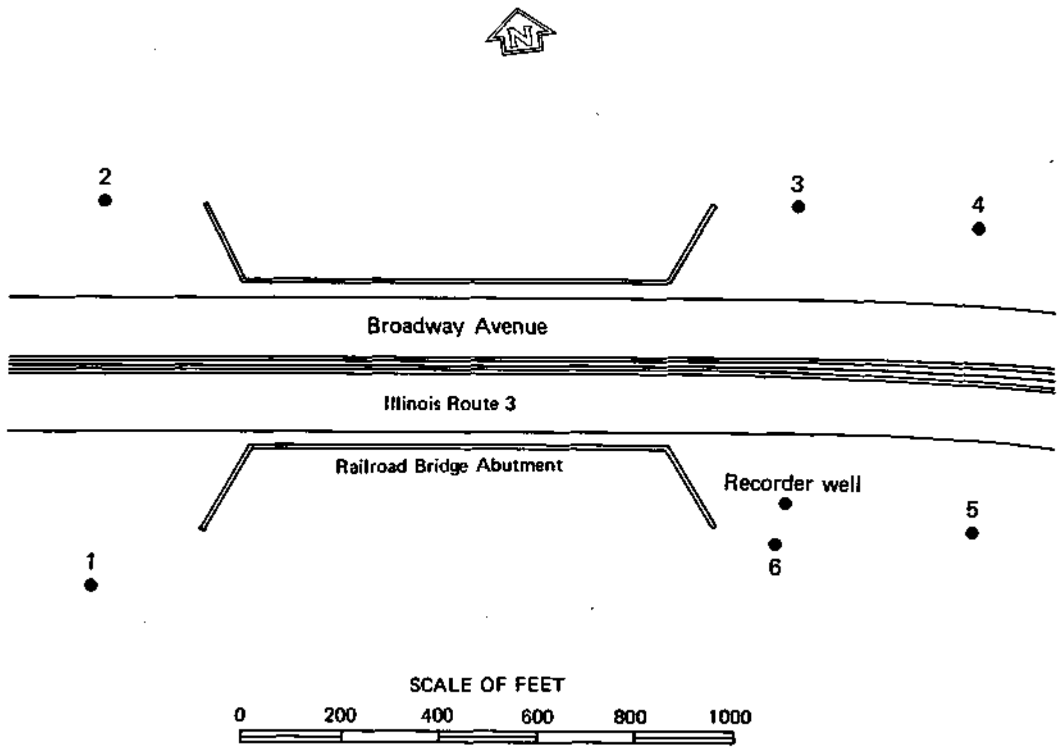


Figure 3. Location of dewatering wells at the Venice Subway (Illinois Route 3)

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 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. 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 Mr. Navin Rupani, Hydraulic Engineer, District 8, who reviewed and coordinated the investigation. Mr. Stan Gregowicz, Maintenance Division, provided valuable guidance during early project meetings and shared his knowledge of the history and operation of the dewatering systems. The Maintenance Division Pump Crew under the supervision of Mr. Gregowicz provided field support during the conduct of step drawdown tests on the selected wells. State Water Survey Groundwater Section staff who ably assisted the authors with field data collection included Robert Kohlhase, Michael Broten, Mark Sievers, Mark Hampton, David Cartwright, and Dinamuni Mendis. In addition, Michael Broten skillfully provided computer-generated graphs to review piezometer water levels and water level differences, and Michael Schock assisted in evaluating the chemical quality data.

Chemical analyses of water samples were done by the Analytical Chemistry Lab Unit under the direction of Jim Whitney. Computer data entry was supervised by Marvin Clevenger, editing was done by Gail Taylor, and the illustrations were prepared by John Brother, Jr., and William Motherway, Jr. The manuscript was typed by Pamela Lovett, Groundwater Section secretary.

HISTORICAL SUMMARY OF DEWATERING DEVELOPMENT

The eastbound lanes of Interstate 70 (1-70) 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 groundwater levels were near an elevation of 390 or about 6.5 feet above the planned highway (McClelland Engineers, 1971).

Horizontal Drain System

A horizontal French drain system was designed for controlling the groundwater levels along an 800-ft 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-gpm turbine pumps. The construction dewatering system was designed to maintain the groundwater 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-in. 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-in. perforations in the drain pipes. As a result, when the construction dewatering system was turned off and groundwater 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 groundwater table. Since it was very likely that the foundation sands had piped from beneath the pavement, the diagonal drains beneath the pavement were grouted to prevent any future loss of support beneath the pavement (McClelland Engineers, 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-in.-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 groundwater 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. Groundwater levels were lowered to elevation 375.5±, about 2 feet below the design groundwater 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 groundwater 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 1960's revealed some sinking of the asphalt shoulders and areas around the storm drainage inlets. Several breaks and/or blockages of the horizontal transite 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 transite 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 psi), a significant amount of scale was removed from inside the mild steel pipes, indicating serious corrosion. Nearly all transite 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 contents. A chemical analysis showed the extremely corrosive quality of the groundwater as evidenced by:

- Extremely high concentration of dissolved carbon dioxide, 160 to 240 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 investigation recommended that 304 stainless steel pipe should be used throughout any replacement system in order to withstand the possibility of severe corrosion caused by the chemical contents of groundwater and to prevent galvanic action between different metals (McClelland Engineers, 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 groundwater levels at desired elevations. This alternative as a permanent system, therefore, was given further study. A 1972 consultant's report (Layne-Western, 1972) showed that water levels at the 1-70 Tri-Level Bridge site could be maintained at desired elevations with 10 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-in. 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 groundwater 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-ft-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 groundwater 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 groundwater levels along 1-64, a series of 20 wells was added to the dewatering system. The wells were built in 1975 and are essentially identical to the ones constructed for the Tri-Level Bridge site.

About 6200 feet southeast of the Tri-Level Bridge, at the East St. Louis 25th Street interchange with 1-64, 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 groundwater levels. Ten wells were installed at this site to control groundwater levels. These wells also are identical in design to the 1-70 wells. The pumps installed in the wells along 1-64 and at 25th Street have nominal pumping capacities of 600 gpm. Two 8-inch observation wells, located near each end of the 1-64 depressed section, are used to monitor groundwater levels. An 8-inch observation well also is installed near the critical location at the 25th Street underpass. As at the 1-70 wells, each dewatering well for 1-64 and 25th Street has a piezometer located approximately 5 feet away for monitoring performance of each individual installation.

Approximately 2-1/4 miles north of the 1-70 Tri-Level Bridge, Illinois Highway 3 passes beneath the N and W, ICG, and Conrail railroad tracks. When the highway was constructed, groundwater 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 groundwater levels resulting from 3 to 4 months of continuous flood stage in the Mississippi River (about 2000 ft west). Subsequent investigation showed deterioration of the drainage system, and the consultants recommended installation of six wells to control groundwater 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:

1-70 (Tri-Level Bridge)	- 12 wells
1-64	- 20 wells
25th Street	- 10 wells
Venice (-Route 3)	- 6 wells

As shown in figure 4, the wells are of similar construction, with 16-inch-diameter stainless steel casing and screen, and 6-inch-diameter stainless steel column pipe. 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 groundwater 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 9.85 million gallons per day in 1983.

DEWATERING SYSTEM MONITORING

When originally constructed, the well installations at 1-70, 1-64, and 25th Street included pitot tube type flow rate meters. Reportedly, a combination of corrosion and chemical deposition caused premature failure of these

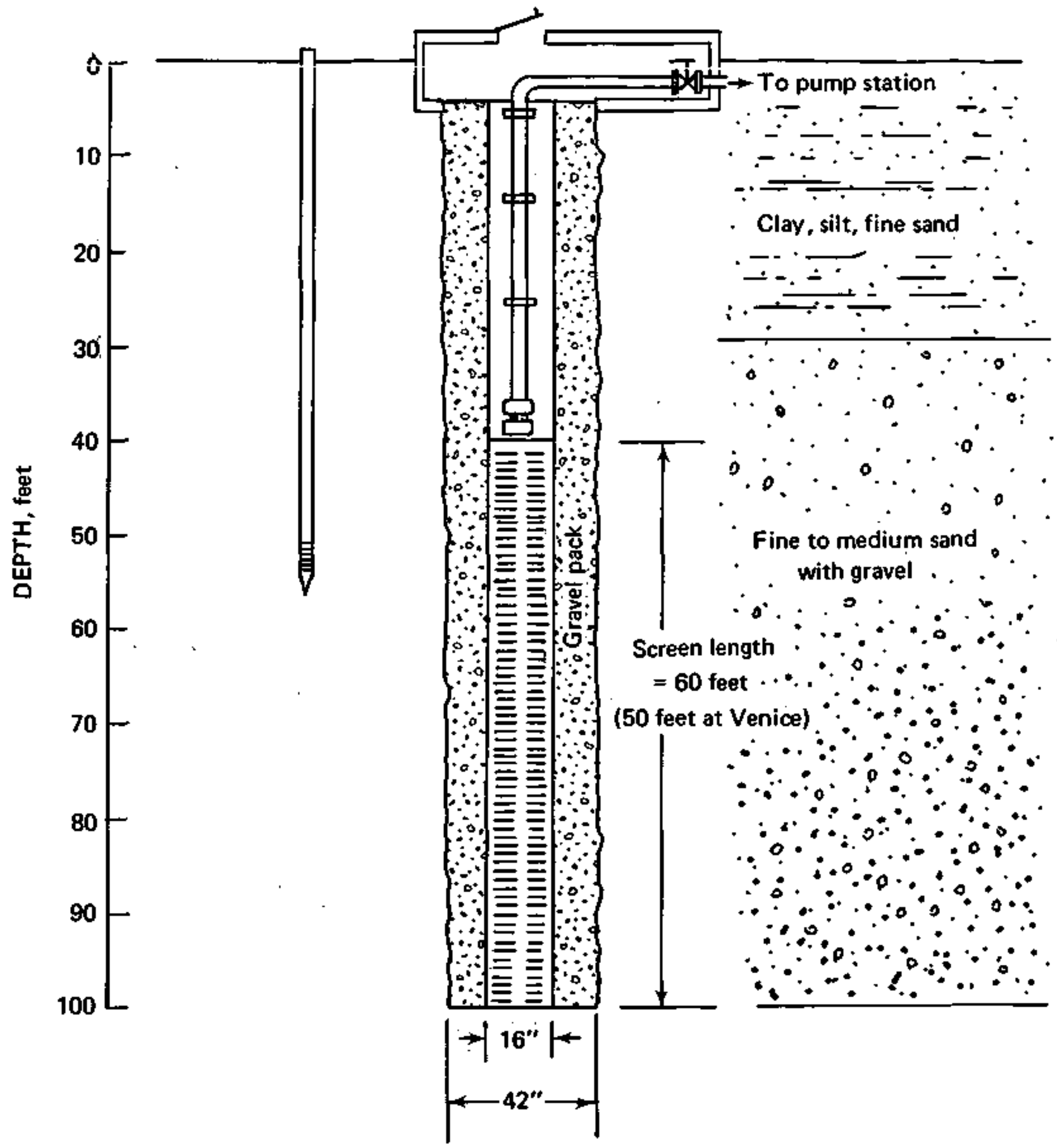


Figure 4. Typical features of a dewatering well

devices. Flow rates are now occasionally checked with a temporarily inserted pitot tube meter, but erratic results are reported by the field crew. The six new 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 operating experience with the new meters is too limited to document their expected reliability. 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 1/4-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. Close monitoring of the operation of these devices is necessary to assure collection of accurate flow rate data. Annual withdrawals currently are calculated on the basis of pumping time and estimated pumping rates.

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 of wells is operated. Since 1977, the percentage of days operated has ranged from 23 to 45?, averaging 38?, at 1-70; from 13 to 36%, averaging 21%, at 1-64; and from 12 to 47%, averaging 30?, at 25th Street. The operating pattern of the wells at Venice has not yet been established. No standard sequence of pumping rotation is followed because of maintenance and rehabilitation requirements. Bar charts showing the periods of operation are prepared by the Department for monitoring the accumulated hours of operation.

Water levels in the piezometer adjacent to each dewatering well are measured every 2-4 weeks. The pumping water level in each operating well also is measured. These water level data are reviewed by IDOT supervisors to monitor groundwater levels in relation to the pavement elevation and to assess the condition of individual dewatering wells. Water level differences of 3 to 5 feet between the pumping wells and the adjacent piezometers usually are considered normal by the Department. Greater differences are interpreted to indicate that well deterioration is occurring. 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 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 pumpage, 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.

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. Velocities may become sufficiently large that a change from laminar to turbulent flow may occur. Under these conditions the well-loss component of drawdown can become significant and can increase in a nonlinear manner with increases in pumping rate. Well losses often reflect a deterioration in the condition of a well, especially if they are observed to increase with time.

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

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

Jacob (1947) expresses these components as being proportional to pumping rate (Q) in the following manner:

$$s_0 = BQ + CQ^2 \tag{2}$$

where B is the formation loss constant at the well-aquifer interface per unit discharge and C is the well loss constant. 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

$$s_0 = BQ + CQ^n \tag{3}$$

In order 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

$$s_0/Q = B + CQ$$

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. 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 2nd 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. In order to facilitate a graphical procedure, equation 3 is rearranged as

$$(s_0/Q) - B = CQ^{n-1} \tag{5}$$

Taking logs of both sides of the equation leads to

$$\log C(S_0/Q) - B] = \log C + (n - 1) \log Q \tag{6}$$

A plot of $(s_0/Q) - B$ versus Q can be made on logarithmic graph paper from step test data. 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 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. 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. 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 1-minute frequency again, and so on until a full range of pumping rates within the capacity of the pump was tested.

Schematically, the time-water level relationship 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 water levels from the previous step and the final water level of the current step. For step 1 the nonpumping water level trend prior to the start of the test is extrapolated, and Δs_1 is measured from this datum. All data extrapolations should be performed on semilog graph paper for the most accurate results. For the

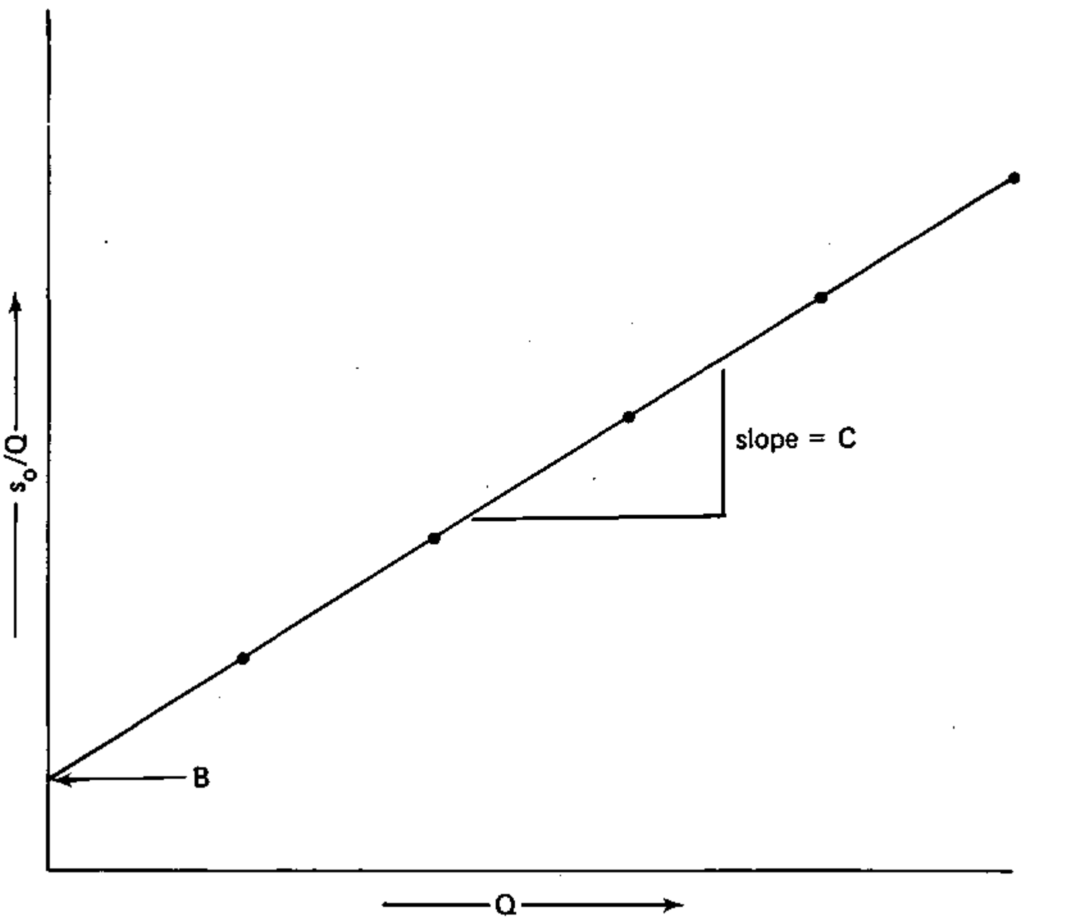


Figure 5. Graphical solution of Jacob's equation for well loss coefficient, C

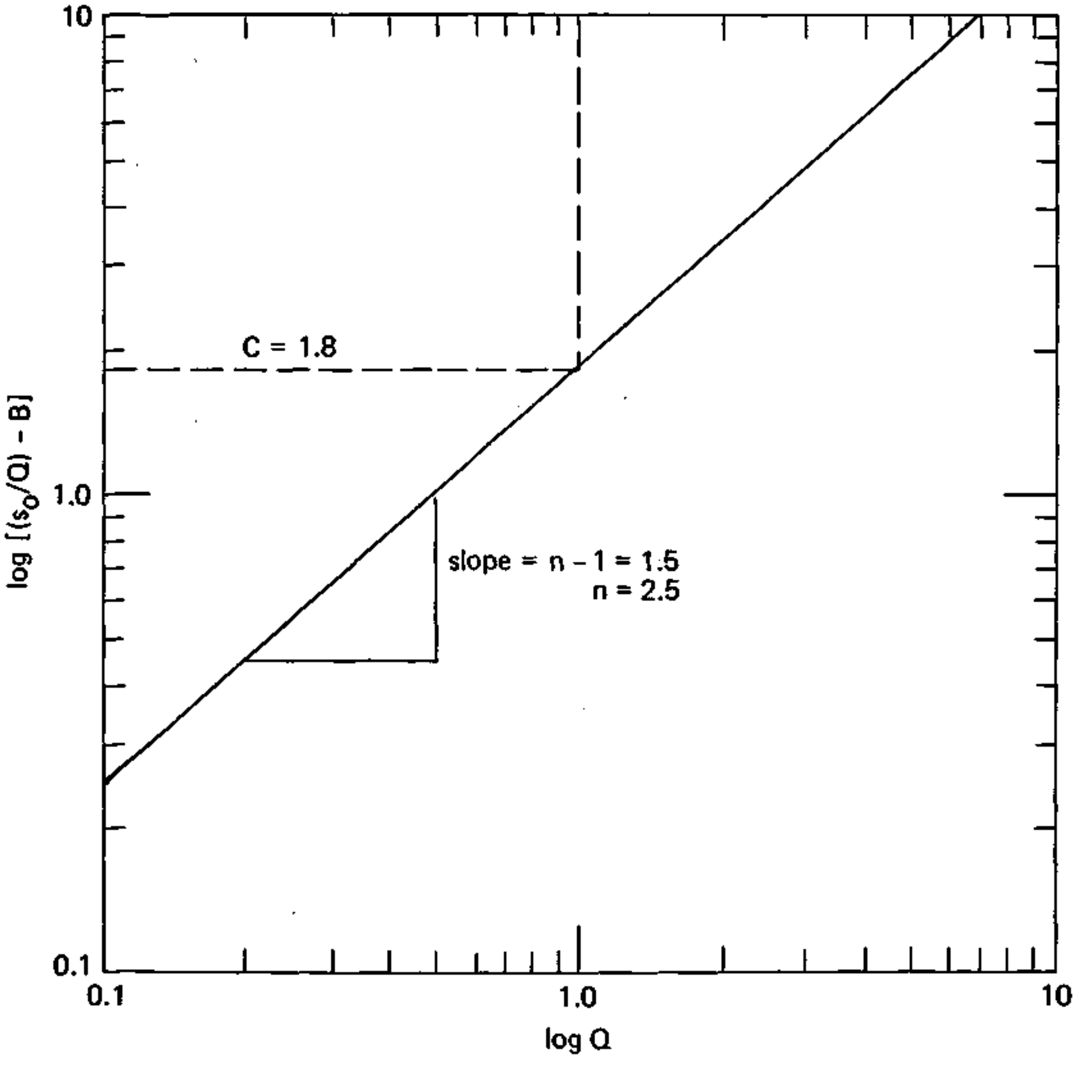


Figure 6. Graphical solution of Rorabaugh's equation for well loss coefficient (C) and exponent (n)

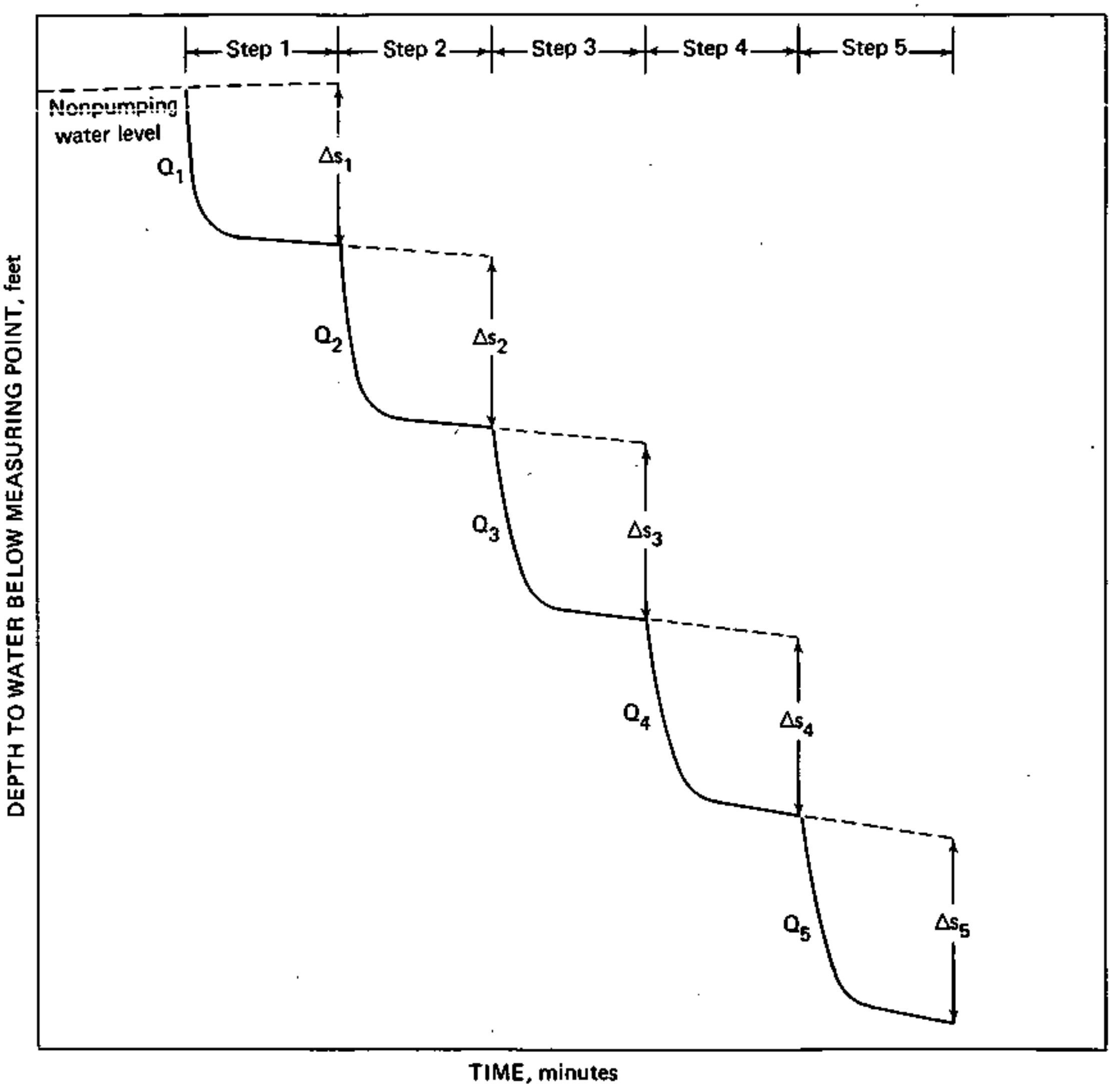


Figure 7. Time-water level relationship during a five-step drawdown test

purpose of plotting S_0/Q versus Q or $(s_0/Q) - B$ versus Q, values of observed drawdown s_0 are equal to the sum of Δs_i for the step of interest. Thus, for step 3, $s_0 = \Delta s_1 + \Delta S_2 + \Delta S_3$.

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 turbulent 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 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 wells selected for testing were determined during a meeting at the IDOT District Office on May 26, 1983. IDOT staff included Navin Rupani, Stan Gregowicz, and Dean Seib. ISWS staff were Ellis Sanderson, Adrian Visocky, and Robert Olson. The historical performance of the wells was briefly reviewed as documented by the data collected by IDOT. Those installations showing the greatest water level differences between the operating well and piezometer were judged to have experienced the greatest well deterioration. However, it also was desired to select wells for testing on the basis of relative location, access for testing, and other known physical characteristics. For example, 1-70 Well No. 7 was selected because it had been recased with 12-in. pipe and well screen inside the original 16-in. pipe and screen. The new Well No. 12 completed in 1980 at 1-70 (13th well) was selected because it would provide a comparison to the older wells at 1-70. The wells selected for testing at this meeting were as follows:

1-70	No. 2
	No. 3
	No. 7
	No. 12 (13th)
1-64	No. 8
	No. 15
25th Street	No. 2
	No. 8

Later, during a project progress meeting on September 26, 1983, with Navin Rupani, it was noted that the six new wells at the Illinois Route 3 underpass at Venice had just been accepted by the Department. To provide data for future comparisons of well performance, it was recommended that an extension of the project be planned to include step tests on each of the Venice wells. A proposal for supplemental work was prepared and accepted by the Department.

Field Testing Procedure

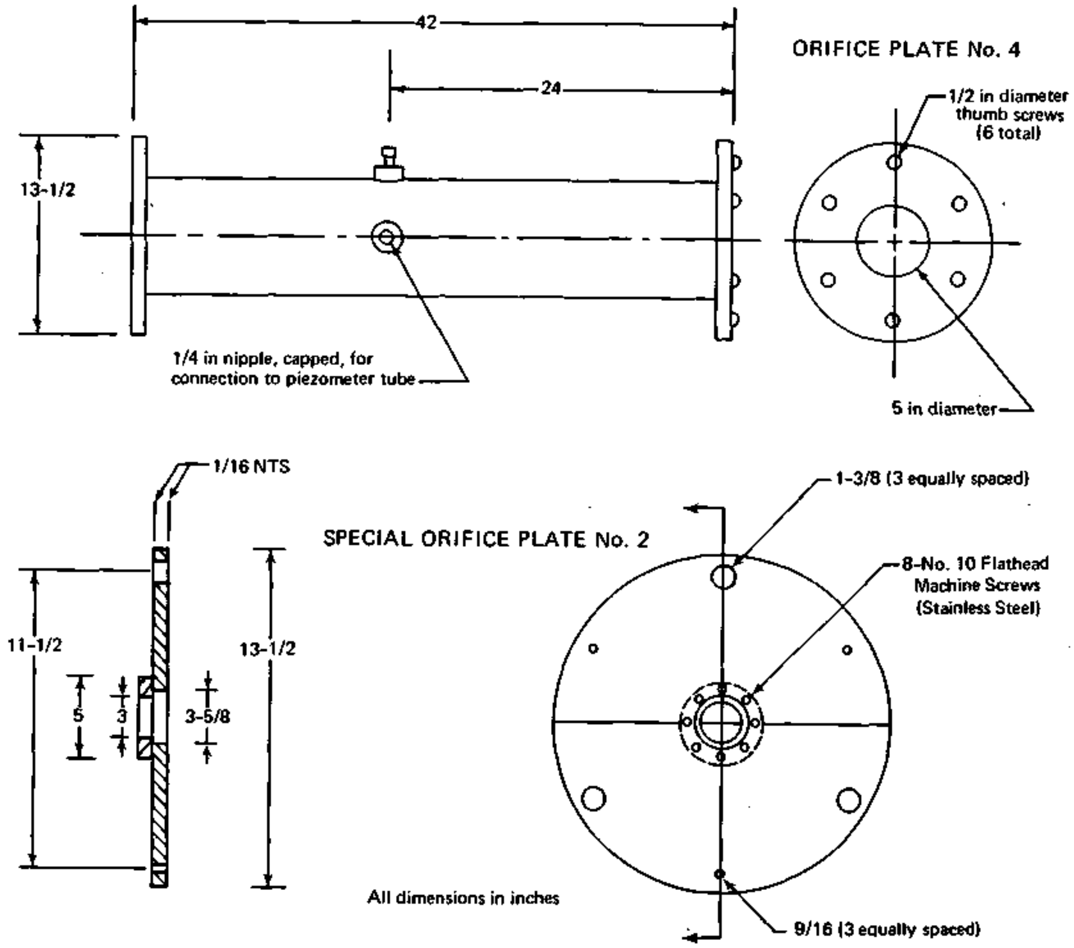
Field work was conducted by Water Survey staff with the assistance of the IDOT Maintenance Division Pump Crew under the general supervision of Stan Gregowicz. The IDOT Pump Crew made all necessary discharge pipe modifications and provided special piping adapters. This allowed the water from the pumped 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 storm water drains.

Orifice tubes are considered standard equipment for measuring flow rates. To permit the measurement of a wide range of flow rates for the desired step tests, a special orifice plate (see figure 8) was designed and constructed. The special orifice plate used to measure the lower range of flow rates Was fitted over an existing orifice plate which measures the higher range of flow rates. Both plates were calibrated in the University of Illinois Hydraulics Lab under discharge conditions similar to those expected in the field. The rating curves developed from the calibration procedures are shown in figure 9.

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 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 100 gpm and to include as many steps as possible 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 first two well tests were conducted during the week of June 14, 1983. Each of these tests was started at a pumping rate of 100 gpm and increased in increments of 100 gpm until full pump capacity was reached.



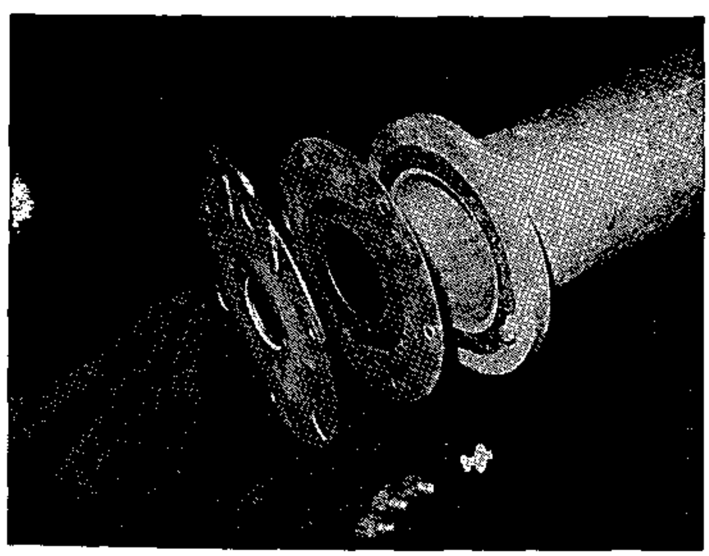


Figure 8. ISWS 8-inch diameter aluminum orifice tube and special orifice plate

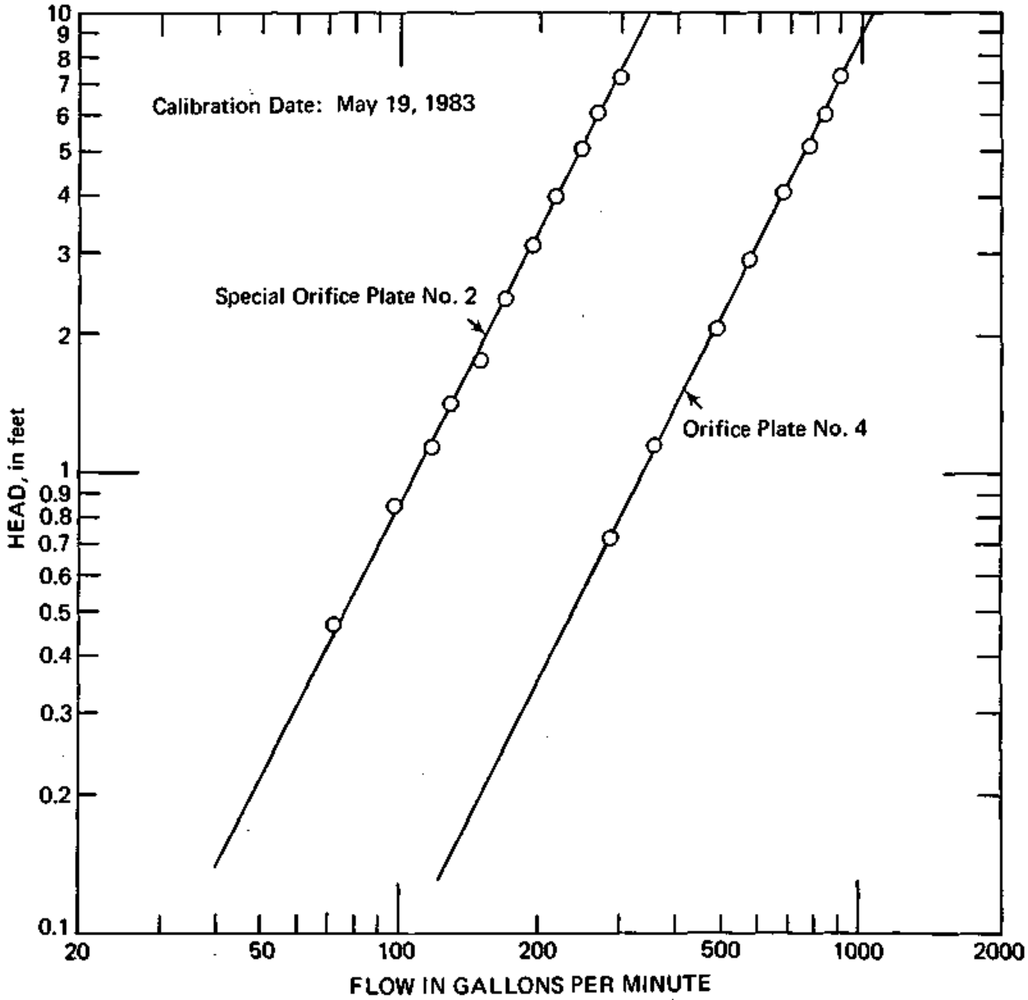


Figure 9. Rating curves for ISWS 8-inch orifice tube

Preliminary analysis of these test data suggested that more meaningful results might be obtained by initially running the early steps (300 gpm and less) in a sequence of decreasing rates and then increasing the rates through the full range of pump capacity. This procedure was followed for subsequent tests during the weeks of June 27 (three tests) and July 18 (three tests). During the week of July 18, 1-64 Well No. 8 was scheduled for testing; however, the maximum pumping rate was about 250 gpm, making it impossible to conduct a test with the desired range of discharge rates. Water level measurements showed that the reduced pumping rate was not due to the well condition. An immediate electrical check of the pump likewise revealed no apparent cause for the reduced pumping rate. Well No. 6 at 1-64 was then selected as a substitute and prepared for testing two days later; however, a maximum pumping rate of only 280 gpm was obtained. Again, water level measurements showed the problem was not with the well, and an immediate electrical check revealed no apparent cause for the reduced pumping rate. A step test at 50-gpm increments was conducted on Well No. 6, but the test results were not satisfactory due to the reduced pumping rate. Late in the summer a second substitute well, 1-64 Well No. 9, was selected. A test of this well was successfully conducted on October 5, 1983. The six wells at Venice were tested during the weeks of November 14 (two tests) and November 28 (four tests).

The water level and pumping rate measurements collected during each of the step tests are included in Appendix A. Near the end of each step test, a water sample was collected for chemical analysis. The results of the analyses are presented in Appendix B.

Results of Step Tests

During the analysis of data from each of the step tests it became evident that the graphical plots of s/Q versus Q were not amenable to analysis by the Rorabaugh method, so all analyses were made 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 December 1, 1983, test of Well No. 4 at Venice are analyzed.

Pumping at Well No. 4 commenced at 10:30 a.m. at an initial rate of about 200 gpm. After 30 minutes the discharge was reduced to 100 gpm. During each succeeding 30-minute step, pumpage was increased by 100 gpm so that steps 3 through 9 had discharge rates of about 200, 300, 400, 500, 600, 700, and 800 gpm, respectively. A water sample was collected during the 9th step at 2:50 p.m., and the test was concluded at 3:00 p.m.

Data from the pumped well are shown in a plot of s/Q versus Q (figure 10). 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 3.55 sec/ft^2 and 0.22 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

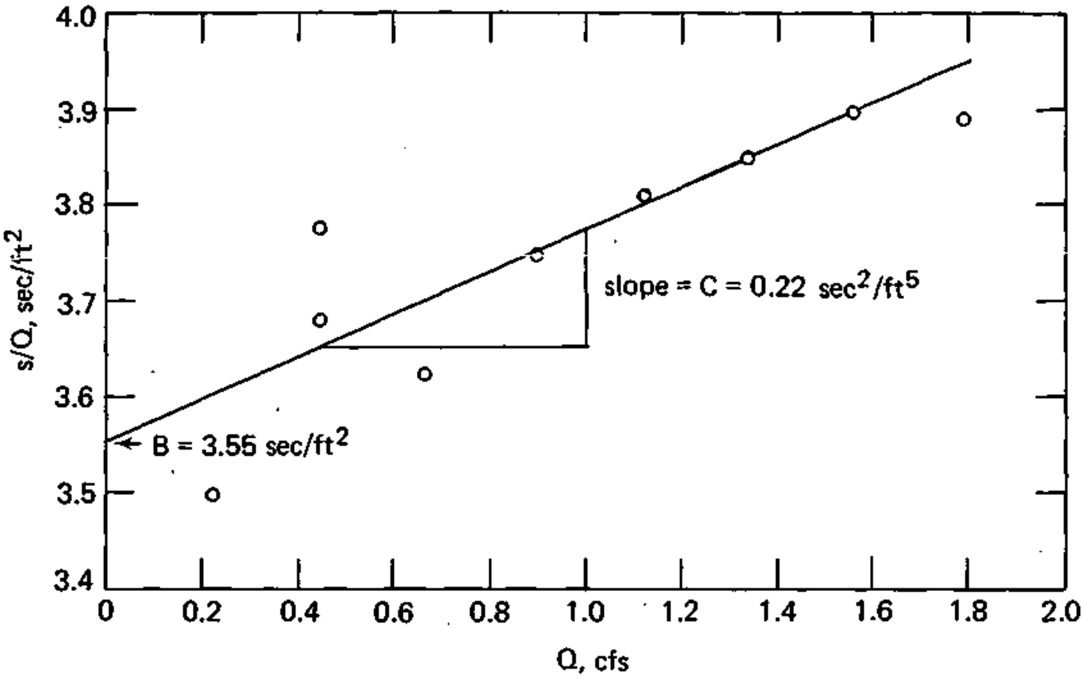


Figure 10. Graphical analysis for Venice Well No. 4 (December 1, 1983)

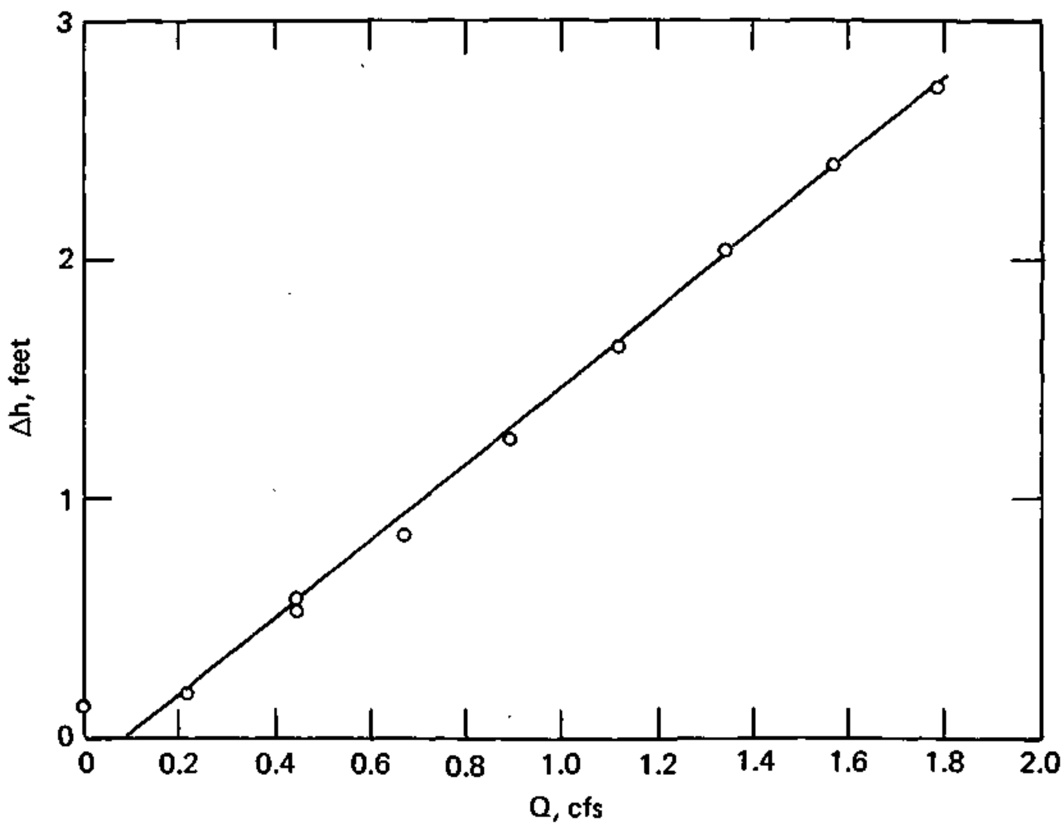


Figure 11. Plot of head difference vs. discharge, Venice Well No. 4 (December 1, 1983)

$$s = BQ + CQ2 \tag{7}$$

$$\begin{aligned} s &= 3.55(1.337) + 0.22(1.337)^2 \\ &= 4.75 + 0.39 \\ &= 5.14 \text{ feet} \end{aligned}$$

The total drawdown of 5.14 feet compares favorably with the observed drawdown, which was 5.15 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 0.39 feet or 7.6% of total drawdown, which is modestly low. Another indication that the well is in good hydraulic condition is the specific capacity. At 600 gpm the observed specific capacity was 116.5 gpm/ft, which compares favorably with values obtained at other sites and which also compares well with the theoretical specific capacity for the Venice area (estimated from hydraulic properties in the area).

Figure 11 shows water level differences (Ah) between Well No. 4 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 was corroborated by a plot of drawdowns at the piezometer versus pumpage, which showed a linear relationship.

The results of analyses performed on data gathered during the step drawdown testing of 14 IDOT wells in 1983 are summarized in table 1. As seen in the table, 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 losses were high only in Well No. 15 along 1-64, reaching 43.2% of the total drawdown at 600 gpm. The next highest percentage was 10.1% for Well No. 7 at 1-70. All of the other wells had calculated well losses less than 10%, the smallest well loss being 0.05 foot for Venice Well No. 2. Data from three wells—No. 2 and No. 3 at 1-70 and No. 3 at Venice—indicated conditions resembling development at the screen and showed no signs of turbulent flow. Hence, it is assumed that well losses in these wells were small at a discharge rate of 600 gpm.

Specific capacity values also were calculated for a 600 gpm pumping rate and ranged from 32.3 gpm/ft at 1-70 Well No. 7 to 157.1 gpm/ft at 1-70 Well No. 12. The overall average specific capacity was 88.7 gpm/ft, and the highest group average (116.5 gpm/ft) was seen at 25th Street. Averages for the other well groups were 90.1, 78.4, and 77.7 gpm/ft for Venice, 1-64, and 1-70, respectively. The 1-70 average without the value from Well No. 12 (a new well) was only 51 .2 gpm/ft.

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 1.1 to 15.0 feet at 600 gpm and averaged 4.9 feet. In general, Ah values varied inversely with specific capacity so that the 25th Street wells had the lowest values, followed next by the Venice wells. The relationship between Ah and well loss might be

Table 1. Results of Step Tests on IDOT Wells

Well	Date of test	Well loss @ 600 gpm (feet)	Drawdown @ 600 gpm (feet)	Well loss portion (%)	Specific capacity (gpm/ft)	Δh* @ 600 gpm (feet)	Remarks
I-70							
No. 2	7/19/83	**	11.9 est.	**	50.4 est.	7.9 est.	Q_{max} = 500 gpm
No. 3	6/28/83	**	8.53	**	70.9	5.65	
No. 7	6/30/83	1.88	18.55	10.1	32.3	15.0	Piezometer at 7.5 ft
No. 12	6/16/83	0.20	3.82	5.2	157.1	--	Piezometer plugged
I-64							
No. 9	10/5/83	0.37	6.22	5.9	96.5	2.3	
No. 15	6/29/83	4.29	9.94	43.2	60.4	4.6	
25th Street							
No. 2	7/20/83	0.54	5.69	9.5	105.4	1.1	
No. 8	6/15/83	0.11	4.70	2.3	127.6	1.5	
Venice							
No. 1	11/30/83	2.29	18.04 est.	12.7	34.1	10.9 est.	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	

* Head difference between pumped well and adjacent piezometer
** Graphical analysis indicated laminar flow conditions and possible well development, making a well-loss determination impossible

expected to be linear; 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 losses between the well and the piezometer. It is believed that this might be the case in 1-70 Wells 2, 3, and 7 and in Venice Wells 1, 3, and 6. It is very difficult to assess laminar well losses, however, because of the need to separate them from laminar formation losses.

Evaluation of Groundwater Quality

Fifteen wells were sampled for analysis by the State Water Survey analytical laboratory. The analytical results are reported in Appendix B. Analytical methods conform to procedures presented in the 15th Edition of Standard Methods for the Examination of Water and Wastewater (1980). 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 samples.

The range of concentrations and anticipated influence of each parameter is presented in table 2.

Table 2. Ranges of Concentration and Potential Influence of Common Dissolved Constituents

Parameter	Concentration, mg/l Min.	Concentration, mg/l Max.	Potential influence
Iron (Fe)	9.1	25.7	Major - incrustative
Calcium (Ca)	124	261	Major - incrustative
Magnesium (Mg)	38.7	61.2	Minor - incrustative
Sodium (Na)	17	166	Neutral
Strontium (Sr)	0.3	0.4	Minor - incrustative
Potassium (K)	5.2	8.5	Neutral
Silica (SiO_2)	22.9	35.8	Minor - incrustative
Nitrate (NO_3)	<0.6	0.8	Neutral
Chloride (Cl)	21	198	Moderate - corrosive
Sulfate (SO_4)	185	585	Major - corrosive
Alkalinity (as $CaCO_3$)	352	476	Major - incrustative
Hardness (as $CaCO_3$)	614	899	Major - incrustative
Total Dissolved Solids	659	1388	Major - corrosive
pH	6.9	7.2	Major - incrustative

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

An evaluation of the quality of groundwater may consider many factors which are not required for evaluating the IDOT wells. For the purposes of this discussion, only the corrosion and mineral deposition potential will be considered.

Corrosion Potential

The high mineralization, as characterized by chloride, sulfate, and total dissolved solids concentrations, is the most influential corrosive factor observed for this groundwater. The carbon dioxide concentration is also quite high and an important factor to consider if steel materials are employed in the well. Dissolved oxygen is not a factor, since high ferrous iron concentrations are present. Hydrogen sulfide, detected by odor in four of the fifteen wells, could be corrosive if significant concentrations were found. The alkalinity, pH, and hardness concentrations would tend to reduce the corrosivity of water produced by these wells. Presence of iron or sulfate-reducing bacteria was not determined. Although the problems typically associated with these bacteria have not been reported, some evidence of iron bacteria has been observed.

Of the indices used for predicting the corrosivity of water, only the Larson Index (Larson, 1955) indicates that water from these wells is corrosive. According to the Larson Index, the water ranges from 0.4 to 1.8, which predicts it to be very corrosive to cast iron and steel pipe. The high mineralization of the water would also encourage "galvanic" corrosion of dissimilar metals but would not be a problem if construction materials were selected properly (Moss, 1966).

Stainless steel is very resistant to corrosion when exposed to water of this quality and to soils with resistivities as reported by Saner in 1976. Therefore, cathodic protection of the well casing and screen would not provide sufficient improvement in corrosion resistance to justify the installation and operating costs required for this added protection. The potential corrosion problems associated with the IDOT wells may be associated with bacteria or well rehabilitation.

Bacteria can cause serious pitting corrosion of stainless steel. Wells should be protected from bacterial contamination and may require sterilization if bacteria are observed. Wells are frequently rehabilitated by injecting hydrochloric acid into the well. Since hydrochloric acid can initiate pitting corrosion of stainless steel, chemical corrosion inhibitors are required when employing hydrochloric acid. Sulfamic acid would be a satisfactory substitute when calcium carbonate is the primary incrustant.

A review of the past reports concerning corrosion of the IDOT wells indicates that adequate measures have been implemented to minimize corrosion. Stainless steel (type 304) casing and screens, bronze and nickel alloyed steel pump parts, and concrete collector piping are satisfactory materials to employ with the quality of water found in the IDOT wells.

Mineral Deposition Potential

To estimate the potential for minerals to deposit within the well, gravel pack, or aquifer, the "saturation indices" were calculated for several minerals by the computer program WATEQFC (Runnells and Lindberg, 1981). The saturation index (SI) for a solid is a measure of the magnitude of supersaturation (positive values) or undersaturation (negative values). An SI of zero represents a mineral in exact equilibrium with water.

The water chemistry data from four wells, analyzed using the WATEQFC program, were found to range from undersaturated to supersaturated for four common carbonate minerals (table 3).

Table 3. Saturation Indices for Selected Wells

	Saturation Index (S.I.)			
Well	$CaCO_3$	$FeCO_3$	$SrCO_3$	$Ca,Mg(CO_3)_2$
I-70 No. 2	+0.3	+1.2	-	+0.3
I-70 No. 7	+0.3	+1.0	-	+0.9
25th St. No.8	0.0	+0.7	+0.2	-0.4
I-70 No. 12 (13th)	0.0	+0.7	+0.1	-0.2

In addition, the Langelier Index (Larson and Buswell, 1942), the Ryznar. Index (Ryznar, 1944), and the Calcium Carbonate Precipitation Potential (Merrill and Sanks, 1977) were calculated for all fifteen wells sampled. These indices all predicted that calcium carbonate had the potential to form deposits.

Deposition of siderite ($FeC0_3$) and/or calcite ($CaCO_3$) was confirmed by chemical analysis of a deposit collected in November 1982 from the column pipe of the pump removed from 1-70 Well No. 12 (original). The constituents of the deposit were found to be 80.7% $FeCO_3.H_2O$ and 12.1$ $CaCO_3$.

The analytical evidence demonstrates that the deposition of minerals (commonly referred to as incrustation) is the major undesirable characteristic of the water produced by IDOT dewatering wells (Mogg, 1973). The quantity of mineral being deposited will be influenced by temperature, pressure, and chemical composition of the water (Cowan and Weintritt, 1976; Arceneaux, 1974). Since the temperature can be considered relatively constant, slight changes in pressure or chemical composition (i.e., calcium) would cause formation of a deposit in well screens and/or on media of the aquifer. Incrustation of well screens or gravel pack often occurs due to head losses when a pump is oversized as compared to well capacity. On the other hand, deposition may occur away from the well screen and reduce the transmissive properties of the aquifer.

An example demonstrates the potential magnitude of a problem which could not be solved with the chemical analyses of available water samples. Consider a well operating at 600 gpm for a six-month time period. For a 1 mg/1 decrease in hardness (as $CaC0_3$) concentration in the groundwater, 1300 lb of calcium would be precipitated during the pumping period. To redissolve each 1300 lb of precipitate would require 350 gal of a 37$ hydrochloric acid solution, assuming 100% efficiency for the rehabilitation procedures. The predictive indices do not provide adequate information on the rate of deposition and, consequently, on the total amount of material that may have been deposited. Additionally, they do not indicate the amount of carbonate minerals present in the aquifer materials, which bears upon rehabilitation efficiency. Water samples from wells indicate the water quality only after the water has been influenced by the pressure, temperature, and chemical factors previously discussed. Analyses of samples collected from an observation well would be informative if deposition was significant; however, a series of observation wells in the line of flow would be required to obtain reliable information.

Review of Flow Rate Meters

A review of flow meters which could be utilized at the 1-70, 1-64, and 25th Street sites was performed. The following six basic types were considered:

1) Positive displacement
2) Propeller/turbine
3) Magnetic doppler
4) Ultrasonic pulse time difference
5) Electro-magnetic
6) Pressure difference

The positive displacement and magnetic doppler types were eliminated from consideration due to the conditions required for their operation. The positive displacement types cannot be used for flows of more than about 150 gpm, and magnetic dopplers require that the flowing liquid contain reflective particles such as entrained air bubbles or solid particulates. These conditions are not met by the dewatering wells.

Each of the remaining four types of meters could be used on the 42 unmetered wells. Two of the classified types, the propeller/turbine and pressure difference, essentially rely on some mechanical means of sensing the physical motion of the water flowing through the pipes (i.e., motion against a propeller or a difference in water pressure). The resulting motion or force is then converted into an indication of flow through a direct, magnetic, or electrical linkage. The ultrasonic and electro-magnetic types rely on electrically generated sound waves or magnetic fields to infer water flow, which is then converted into analog or digital readouts of the flow rate. Each type of meter was evaluated by comparing cost, accuracy, corrosion resistance, and installation options. Table 4 provides a summary of this comparison.

Pressure Difference Meters

The pitot tube meters utilized at 1-70, 1-64, and 25th Street and the venturi tube meters in use at Venice are categorized as pressure difference meters. Through experience, it is known that the pitot tubes are unacceptable for use on a permanent basis. The venturi tube or orifice plate (another type of pressure difference meter) probably would provide reliable and reasonably accurate flow rates at the remaining well sites; however, this type of installation is considered permanent, and thus the complete metering device would need to be retrofitted into each well discharge line where flow rate measurements are desired. Labor and material costs for each installation would probably exceed $4,000. At this time, the ability to measure flow

Table 4. Characteristics of Types of Flow Rate Meters

	MECHANICAL		ELECTRONIC	
	Propeller/turbine	Pressure difference	Electro-magnetic	Ultrasonic pulse
Price of meter	\$500 to \$2,500 ea.	\$200 to \$4,000	\$2,500 to \$4,000	\$3,000 to \$5,000
Accuracy	±0.5% to +3%	±1.75% to high	±1% to ±2%	±1%
Min/max flow	220 to 1000	Not reported	0 to 1075	0 to 1250
Corrosion-chemical deposition resistance	Little (iron) to medium (epoxy coatings) to high (stainless steel)	Low for transfer and pitot tubes	High, inserted probes epoxy coated, or made of bronze, stainless steel	High, wetted transducers epoxy coated; some types do not come in contact with water
Type of installation	Permanent, limited portability on some models	Permanent	Permanent or portable	Permanent or portable

rates on a continuous basis at each well is considered convenient but not necessary to efficiently operate the system. A flow meter which would enable periodic measurements at each of the wells is considered sufficient.

Propeller/Turbine Meters

The propeller/turbine meters have been widely used throughout the water industry for many years. Their dependability and accuracy is well documented when manufacturer-recommended maintenance procedures are followed. Even though moving parts and, in some cases, the body of these meters are exposed to the water, corrosion-resistant materials such as plastics, stainless steel, or bronze can be specified or a protective coating can be applied to the sensitive materials.

In this type of meter there are several options that can be considered: 1) a segment of the well discharge line can be removed and a meter unit installed in its place; 2) a large hole can be cut into the discharge line, and the meter unit inserted and welded or clamped into place; or 3) the discharge line can be tapped and special fittings attached through which a small turbine type meter is inserted. The first two options require permanent installations for each well site with the same ramifications as mentioned for the pressure difference meters, although the costs could be about $1,000 less. Regularly scheduled maintenance involving moderate to large amounts of labor would be required to ensure that measurements continued to be reliable. The third option, while requiring plumbing work at each well site, would allow some portability of the meters so one meter could be moved periodically from well to well to obtain flow rate measurements. Thus, the initial purchase of meter units could be reduced.

Electro-Magnetic Meters

Electro-magnetic meters can be obtained which are designed specifically for either permanent installation or portable use. The probes or sensors used for this type of electronic instrument are inserted in the discharge line similarly to the manner of insertion of the inserted turbine meters. The probe must come into direct contact with the water, but probes can easily be sealed with a protective coating or constructed of a noncorrosive material. A device designed to be portable can be obtained for about $2,500 plus fittings, whereas a permanent type device costs $3,000 to $4,000 for each installation. A permanent device offers more capability in terms of totaling flow and providing a direct reading of flow rate. Accuracy of the electro-magnetic sensors is well within the acceptable range of ±1 to 2%. Durability of a portable device is somewhat limited and requires safeguards to prevent rough treatment.

Ultrasonic Pulse Time Difference Meters

The ultrasonic pulse time difference flow meter can be obtained for either permanent or portable use. Two types of units are available, one in which wettable transducers (the sensing components) are inserted inside the pipe and the other where the transducers are mounted on the outside of the discharge pipe. The dry type of transducer can be used only where the pipe is constructed of sonically conductive material such as the stainless steel discharge pipes on the dewatering wells. Use of external transducers eliminates the possibility of corrosion and augments the portability of the system. Accuracy with the ultrasonic units is about ±1% of the flow rate, which should be sufficient for monitoring purposes. Cost of a permanent wettable type of instrument is in excess of $3,000, and the portable or permanent dry types cost about $5,000. The dry type transducers are pipe-size specific and require an appropriate integrated circuit board in the accompanying display computer to transform the signals into flow rates. Therefore, a change in pipe size would require the purchase of additional transducers and boards. However, since all discharge lines on the dewatering wells are one size (6-inch), one type of transducer should be suitable for all of the sites. As with the electro-magnetic devices, the portable ultrasonic equipment would require safeguards to protect against rough handling.

Summary

The examination of meter types and options available to IDOT to monitor flow rates for their 42 unmetered wells indicates that a substantial capital outlay ($40,000 minimum) will be required to install permanent flow meters capable of providing reliable data for an indefinite period of time. Among the permanent meters considered, the inserted type (turbine, electromagnetic, or ultrasonic) or the external transducer ultrasonic type are the most attractive considering installation and maintenance. Moisture conditions in the well head pits may require a strictly mechanical flow measuring device (propeller/turbine) for permanent installation at some of the sites. If continuous flow measurements are not needed at each well, a portable unit should be given serious consideration. The most attractive portable meter is the ultrasonic pulse time difference flow meter with dry type transducers.

GROUNDWATER FLOW MODEL

The primary goal of the modeling study was to examine the efficiency of the present pumping configurations at the 1-70, 1-64, and 25th Street sites. The well fields have been designed to reduce water levels so they do not exceed an elevation four feet below the pavement crown elevation at any point within the system. These elevations are termed target elevations. The goal of the modeling effort was to evaluate the performance of various pumping configurations with respect to the target elevations and to suggest guidelines, if indicated, which would ensure a better distribution of drawdown at each site while minimizing pumpage.

Physical Setting and Data Available

The physical setting of the study area is described in the introductory section of this report. Briefly, the site consists of alluvium about 115 feet in thickness with the uppermost 20 feet being clay, silt, and fine sand and the lower 95 feet being medium- to coarse-grained sands with some gravel.

Within the modeled area, data were available from one aquifer test. The results of this test indicated the transmissivity (T) of the aquifer to be about 220,000 gpd/ft. Because the critical elevations are below the elevation of the top of the aquifer, the entire study area is assumed to be under water table conditions. The aquifer test indicated a specific yield (S_y) of about 0.1. These values are consistent with established values for similar geohydrologic settings in the American Bottoms (Schicht, 1965).

The locations of the wells were taken from a scale drawing of the sites (1" = ±175'). Individual well discharge data were not available, but each well is equipped with a 600 gpm pump for which rating curves were provided. Historic pumping schedules also were provided, and the total pumpage from the area was estimated at 9.85 mgd in 1983. Based on these factors it was initially assumed that each well was pumped at 600 gpm. An examination of water use data revealed three industrial sites with potentially significant withdrawals near the study area. These withdrawal points were included in the model.

At the outset of the modeling study insufficient data were available to define the aquifer boundary conditions and their effects on water levels. While extensive water level data are available for the pumping wells and the piezometers, these data do not provide sufficient information to detail the influence of the Mississippi River on water levels at the sites, the quantity of regional flow through the sites, or the head distribution at points external to the sites. Data were available which quantified these values regionally (Schicht, 1965; Collins and Richards, 1984; Ritchey et al., 1984), but not at the scale necessary to meet the demands of the model.

Model Selection and Description

Two types of models, a numerical finite difference model and one using analytical techniques, were considered through a comparison of their relative advantages and disadvantages. It was decided that an analytical model could be developed that was better suited to the needs of the problem.

An analytical model is a formula which computes exact results under a given set of assumptions and hydrologic conditions. The analytical model used during the modeling study approximated a nonpumping piezometric surface by simulating a regional uniform flow field. The drawdown attributable to a single well was computed using the Theis equation (Theis, 1935). The principle of superposition was employed to account for the effects of multiple wells. Well recovery was simulated by the addition of image wells. (An image well is a "fictitious" well used to mathematically duplicate certain boundary conditions.) Finally, a correction was applied to account for the increase in drawdown due to the effects of dewatering of the aquifer under water table conditions (Walton, 1962). The result was a model which assumed a homogeneous, isotropic aquifer, with transient flow to wells. The required input consists of well and observation point locations, observed or target water level elevations, individual well discharge rates and times, the magnitude and direction of the regional flow field, the location and magnitude of a reference water level elevation, and the geohydrologic flow parameters (T and S_y).

Model Calibration

Calibration of the model consisted of two phases. First, a sensitivity-analysis was performed on T and S_y to evaluate the accuracy of the values initially assumed and to evaluate the assumption of homogeneity. This phase revealed that these values were reasonable and that the assumption was satisfied.

The second phase, called a history match, attempted to match historical water level elevations by simulating known pumping conditions. The methodology used during this phase of the study involved simulating the pumping of the actual well configurations used during two consecutive pumping periods. The first pumping period was used to approximate the initial water level surface at the start of the second pumping period. At the start of the second time period, a second set of wells was activated, and recovery of the first set (used to create the initial water level surface) was modeled by simultaneously activating recharge image wells. The simulated water levels at the end of the second pumping period were then compared to observed water levels for the equivalent point in time. Water levels at three separate points in time were evaluated in this manner. Water level elevations were computed at piezometer locations rather than well locations to minimize well loss components of total drawdown at pumping wells.

During the history match, certain model parameters were adjusted until a satisfactory "fit" was obtained. The adjusted model parameters were the magnitude and direction of the regional flow system, the reference head, and the discharge rates of the wells. The success of the history match was measured by computing the error at each observation point (defined as the observed water level elevation minus the calculated elevation), the mean, the median, and the standard deviation of the error. The fit was judged satisfactory when the median was close to zero and the mean and standard deviation were minimized. The results of the model calibration are summarized in table 5.

Table 5. Summary of Model Calibration

	Discharge rates		Uniform flow					
Date	25th St. (gpm)	I-70 & I-64 (gpm)	Gradient (ft/mi)	Direction	Ref. head (ft msl)	Mean error (ft)	Median error (ft)	Std. dev.
8/28/78	600	525	3.2	WNW	398	.94	-0.02	1.22
11/5/79	600	500	2.1	WNW	405	.69	-0.03	0.85
1/28/80	600	500	1.4	WNW	404	.98	+0.27	1.24

The results of the model can be considered accurate to ±1.25 feet. This inaccuracy is probably due to variations in individual well discharge and possible well loss effects. The measured water level elevations add an additional source of potential error. Several data points were found during the history match which, upon examination, were found to be obviously in error and were consequently removed from the data set. Data points were deleted only if obvious justification existed; other erroneous data might remain.

Model Application

The model was used to assess the efficiency of various pumping configurations. This was interpreted to mean assessing the areal distribution of drawdown while minimizing total pumpage at the expense, if necessary, of , uniform usage of wells. Usage patterns were considered only to the extent of requiring some flexibility for the rotation of wells. This was necessary to allow for well and pump maintenance.

The general strategy used in this portion of the study was to define groups of wells which perform a specific role in the overall system. These are termed functional groups. Each group should permit well rotation without damaging the effectiveness of the system. This method emphasizes the relative importance of groups of wells and, in doing so, implies where well monitoring and well rehabilitation or replacement are most important. The methodology used to define these functional groups was to examine the behavior of the system under a set of somewhat severe assumed conditions, and then to refine the groups by simulating their behavior under more moderate known conditions. The performance of the well groups was evaluated by comparison with real pumping configurations.

Definition of Functional Well Groups

The assumed conditions used in the initial group definition phase represented a worst case approximation (severe conditions). Prepumping water levels were assumed to be relatively high (reference head = 405 ft msl) and the gradient flat (-1.4 ft/mi). These conditions combine to maximize the required drawdown. As is consistent with the results of the history match, the wells at 1-70 and 1-64 were assigned a discharge rate of 500 gpm, while those at 25th Street were allowed a rate of 600 gpm. All wells pumped continuously for a period of 90 days. Steady state conditions were not achieved during this time. Initial conditions were established by the same principle used during calibration.

The dewatering system was initially subdivided geographically into four sets: 12 wells at 1-70, the 10 wells on the NW end at 1-64 (wells 1-5 and 11-15), the 10 wells on the SE end at 1-64 (wells 6-10 and 16-20), and the 10 wells at 25th Street. Analysis proceeded on a site-by-site basis, varying the well configuration at the site in question and holding the configurations at other sites essentially constant. Water level elevations were computed for each configuration at the points near each well where target water level elevations are known. The number of wells required at each site was determined using distributed pumpage around the points of lowest elevation. The role of individual wells and the sensitivity of their distribution were examined by simulating unbalanced pumping distributions. The following tabulation summarizes the results of this phase of the study:

Site	Wells required under assumed severe conditions
1-70	7
1-64 (NW end)	3
1-64 (SE end)	1
25th Street	3

This phase of the study revealed four points where drawdown is critical; that is, if the target water level elevations are met at these points, the target elevations at all other points will also be satisfied. These critical points are at 1-70 Well Nos. 2 and 9, 1-64 Well No. 9, and 25th Street Well No. 9. The critical water level elevations associated with them are, respectively, 380.5, 379.0, 390.1, and 392.8 ft msl.

The results obtained during this phase are consistent with the assumed conditions. The required drawdowns at critical points for these conditions are 19.8 ft at 1-70 Well No. 2, 21.3 ft at 1-70 Well No. 9, 11.4 ft at 1-64 Well No. 9, and 10.2 ft at 25th Street Well No. 9. The heaviest pumpage, intuitively, should occur at 1-70, which the model results confirm. Of interest is that only a single well is required near the critical point at the SE end of 1-64. This is the result of the large pumpage at and near 1-70. Three wells are required at 25th Street where the required drawdown approximates that at 1-64, but its geographical separation diminishes the effect of the 1-70 pumpage.

The number of wells required at 1-70 is the result of two factors. First is the magnitude of the required drawdown. Second, the wells are not spatially distributed in accordance with the need for drawdown. The combination of these factors and the need for well rotation result in additional pumpage to satisfy all target elevations.

The wells at the NW end of 1-64 play a dual role. Target water level elevations in this area are the highest and would be met by pumpage at 1-70 without any pumpage in the immediate vicinity. The primary role of these wells is to add small but important quantities of drawdown at 1-70, permitting greater rotation flexibility among the 1-70 wells. A byproduct of this scheme is the addition of drawdown at the opposite end of 1-64. This reduces the requirement to a single well at the SE end of 1-64.

The conditions assumed for this phase of the study raised some interesting questions. First, the average number of wells presently operated at these three sites usually is 12, which represents "normal" conditions. The model indicates that 14 wells should be operated using "optimized" pumping configurations for the assumed worst case conditions. The brunt of the effect of these worst case conditions must be absorbed at 1-70, where target elevations are lowest and the effects of the reference head and the flat gradient are greatest. Furthermore, the criterion of well rotation flexibility required additional pumpage at or near 1-70.

The issue of the well discharge rates must also be raised. Well discharge is directly proportional to computed drawdown. An error in the estimate of an individual well discharge will result in corresponding error in the prediction of drawdown attributable to that well. The well discharges used during the modeling exercise were obtained from model calibrations and represent average withdrawal rates for blocks of wells. Individual well discharges could vary from these averages. Because of this variation, the number of wells required in actual operation could vary by ±1 well compared to that used by the model.

Refinement of Functional Well Groups

To refine the well groups suggested by the model during the initial definition, pumping configurations derived from the functional groups were used to simulate pumpage under the conditions used during model calibration. Experimentation with group composition led to an adjustment in the functional well groups to alleviate the problems associated with unbalanced pumpage distributions. Group composition is summarized in table 6 and is graphically displayed in figure 12. The number of wells indicated is required only during severe conditions.

Table 6. Recommended Functional Well Groups

Site	Functional well groups	Wells required (for severe conditions)
I-70	1,2,3,4,7,8,9,10	5
	5,6,11,12	2
I-64 (NW)	1,2,11,12	2
	3,4,5,13,14,15	1
I-64 (SE)	6,7,8,9,10 16,17,18,19,20	1
25th St.	1,5,6,10	1
	2,3,4,7,8,9	2

Evaluation of Functional Well Groups

The comparison of the use of these groups to actual pumping configurations used during calibration periods indicated that greater efficiency is possible through the use of these guidelines. Some examples of these comparisons are included in Appendix C. To summarize the results obtained using the pumping distribution suggested by the functional groups, a savings of three wells (13 vs. 16) was obtained for the period ending November 5, 1979, one fewer well (13 vs. 14) was required for the period ending January 28, 1980, and two fewer wells (8 vs. 10) were required for the period ending August 28, 1978. These results demonstrate the effectiveness of the functional well groups.

Results

The results of the model indicate that the greatest proportion of pumpage should occur near 1-70. As many as seven wells at the 1-70 site may be

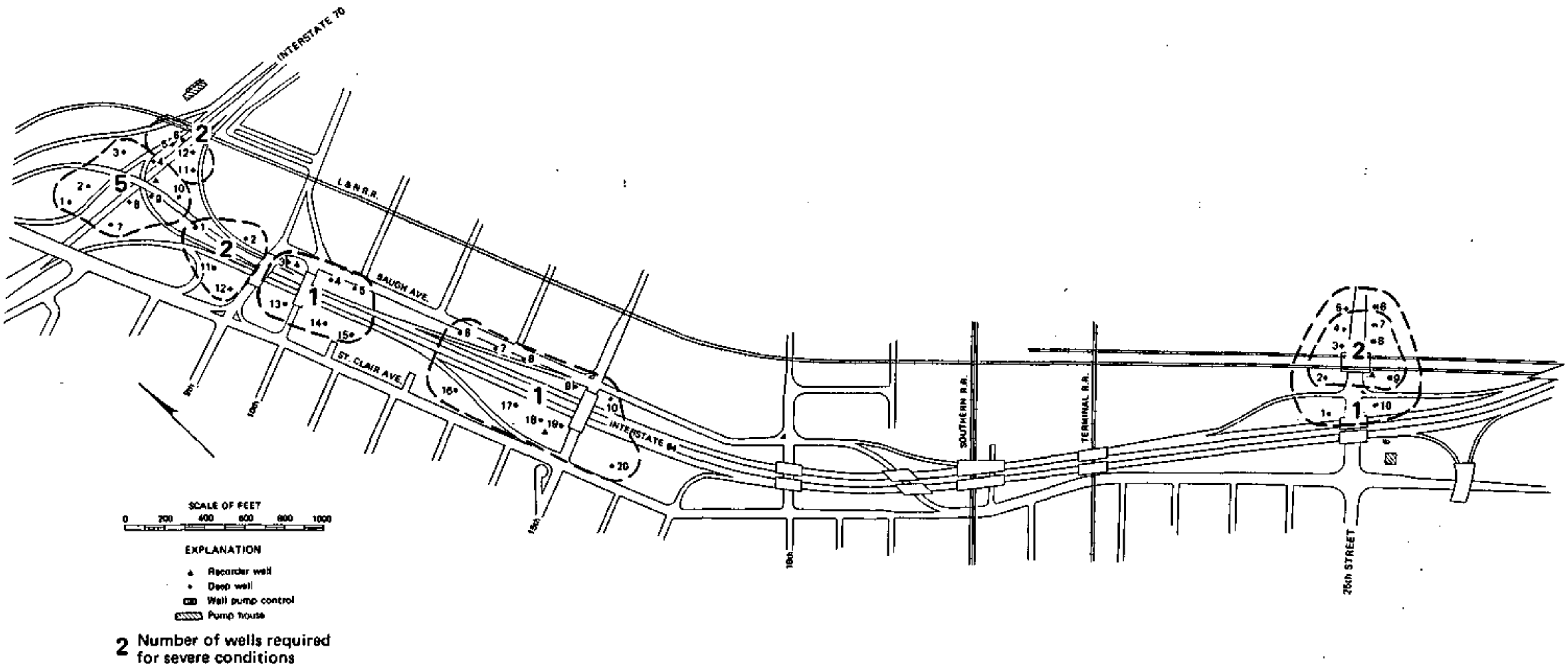


Figure 12. Functional well groups

required during severe conditions. Wells on the NW end at 1-64 add significant drawdown at 1-70 and at the critical point near the SE end at 1-64; their use allows greater scheduling flexibility at 1-70 and, together with 1-70 pumpage, reduces the need at the SE end of 1-64 to a single well. It appears that three wells are necessary at the 25th Street site. The use of the functional well groups, as outlined above, can result in greater pumping efficiency by creating drawdown where it is needed and, by minimizing it elsewhere, can reduce the overall number of pumps required.

CONCLUSIONS AND RECOMMENDATIONS

System Monitoring

The present technique of periodic water level measurement should be modified only slightly. We recommend that once each month (approximately every other time of measurement) the water levels in all wells and piezometers be measured. This will allow direct comparison of water level elevations between nonpumping wells and the adjacent piezometers to assure that errors have not been made in the field measurements. The historical records show that the piezometers are sometimes plugged. When this is the case, the dewatering well should be measured in lieu of the piezometer.

The present frequency of making measurements (1 or 2 times per month) is satisfactory. If taken with care and accuracy, a set of measurements once each month would be sufficient.

The present set of records and charts should be expanded to include a graph of water level differences (Ah) versus time for each well. These graphs would provide a visual record of well performance that would allow easy recognition of developing problems. Figure 13 shows the plot of Ah values for a well that is in good condition (25th Street Well No. 1) and a well that the step test shows to be in poor condition (1-64 Well No. 15). Similar graphs were made for all other wells and aided in the recommendations in the next section regarding the wells that should have step tests to properly assess their condition.

Examination of the recorder charts from the observation wells revealed little meaningful information due to the lack of documentation. An accurate taped measurement of water level and recording of date, well number, and water level on each chart is required, if useful data are to be collected. The recorders also show that maintenance must be improved if reliable recorder operation is to be achieved.

Metering flow rates on a continuous basis is not considered to be a high priority need for the dewatering system operation. However, the ability to check flow rates accurately while a well pump is in normal operation is important. The present technique of monitoring well condition by measuring water level differences has proven to be satisfactory. When these data suggest potential problems, a check of flow rates should be made to help verify and diagnose the problem.

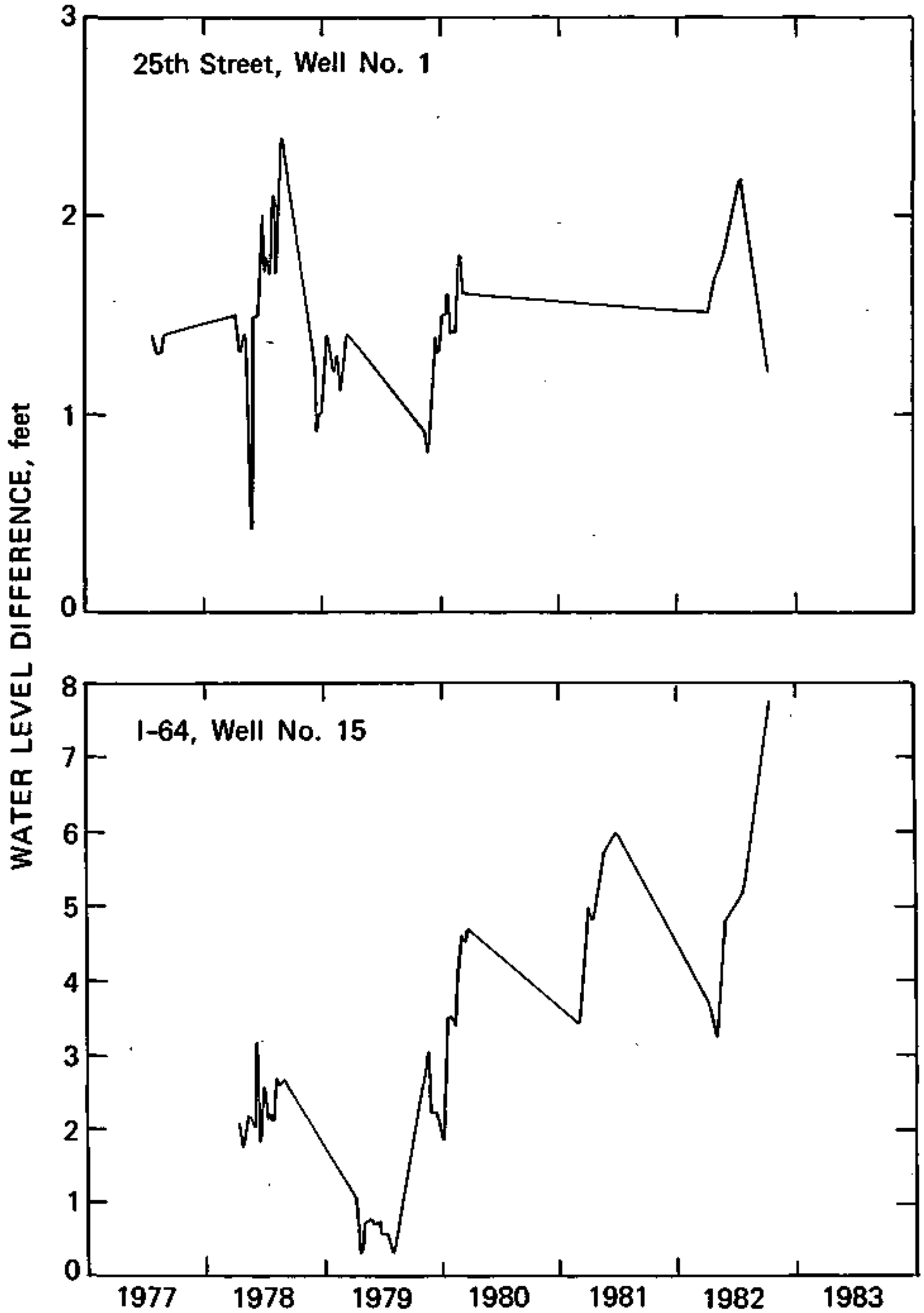


Figure 13. Water level differences between 25th Street Well No. 1 and piezometer and 1-64 Well No. 15 and piezometer

Purchase of a portable ultrasonic pulse time difference flow meter with dry type transducers is recommended. This type of meter has an accuracy of ±1% of the flow rate and is not subject to corrosion or chemical incrustation as it is externally mounted temporarily on the discharge pipe.

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 1-70 Well No. 12 (13th); 1-64 Well No. 9; 25th Street Well Nos. 2 and 8; and Venice Well Nos. 2, 4, and 5 are all in very good condition. The 30-minute specific capacities of these seven wells on the dates of testing were all greater than 95 gallons per minute per foot of drawdown, the differences in water levels between the wells and piezometers were less than 2.5 feet (unknown at 1-70 Well No. 12 because of a plugged piezometer), and drawdown due to well losses was less than 0.6 ft at a pumping rate of 600 gpm.

By comparison, 1-70 Well Nos. 2 and 3, 1-64 Well No. 15, and Venice Well Nos. 3 and 6 are in moderately good condition. Venice Well Nos. 3 and 6 are new and have not been subjected to many hours of operating times therefore, no deterioration at this early date would be expected. Rather, their present condition may reflect a lesser amount of well development work at the time of construction or a local change in aquifer conditions (thickness, hydraulic conductivity) at these well sites. Rehabilitation work on these two wells is not recommended at this time. However, their performance should be closely monitored during the next year.

The 1-70 Well Nos. 2 and 3 are in moderately good condition. They have specific capacities of about 50 and 70 gpm/ft and water level differences of about 4.9 and 7.9 feet at 600 gpm. The analysis for well loss coefficients was inconclusive. The specific capacity of these wells is much less than the new 1-70 Well No. 12 (13th), which has a specific capacity of 157 gpm/ft, but much greater than 1-70 Well No. 7. Wells No. 2 and No. 3 are at key sites with respect to the lowest point of the highway. To assure pumping capacity in this part of the site, we recommend rehabilitation work on both of these wells.

Well No. 15 at 1-64 is in only fair condition. The specific capacity is about 60 gpm/ft, the water level difference between the well and piezometer is about 4.6 feet at 600 gpm, and the drawdown due to well losses is about 4.3 feet (greatest among the wells tested). Since the well is not in a strategic location, rehabilitation work could be delayed. However, because of its high well loss, it is a good candidate to use to investigate rehabilitation techniques that will work best on the dewatering wells. Rehabilitation work on this well is recommended.

The step test on the No. 1 well at Venice showed that the well was in poor condition. The specific capacity of this well was about 34 gpm per foot of drawdown, the projected water level difference at a well discharge of 600 gpm was about 10.8 feet, and the drawdown due to well losses was about 2.3 ft. The well pump broke suction at a discharge rate slightly greater than 500 gpm. Well No. 1 was the original test well for this site. Data included in the technical report for the Venice site show the original specific capacity to be about 73 gpm per foot of drawdown after 30 minutes of pumping at 600 gpra. The present specific capacity is about one-half the original. It is evident that this well requires rehabilitation to attempt to restore its capacity.

Well No. 7 at 1-70 also was found to be in poor condition. Its specific capacity was about 33 gpm per foot of drawdown, the water level difference could not be determined (the piezometer was plugged), and the drawdown due to well losses was about 2.0 ft. Additional piezometers are available at this site because of the previous rehabilitation efforts. At the piezometer 7.5 feet distant, the water level difference at 600 gpm was about 15 feet. This suggests that the water level difference at the piezometer 5 feet from the pumped well also would also have been excessive. Normally, rehabilitation efforts would be recommended. However, since this well has experienced significant problems in the past and was modified with a 12-inch casing and screen inside the original 16-inch casing and screen, the prospects for successful rehabilitation are limited. Replacement of this installation with a new well (No. 14) is recommended within two to three years.

Well Rehabilitation

The evaluation of the groundwater quality at the dewatering wells indicates that the water is very unstable. A deposit of either calcite or siderite can occur easily in response to a small change in pressure or chemical composition. Interpretation of the step test data suggests that because of the linear relationships of Ah vs. well discharge, the chemical deposition is probably taking place within five feet of the well. A likely location is at the aquifer-gravel pack interface. The amount of incrustation and its relationship to well operation schedules cannot be determined from the investigation conducted thus far. Conclusions at this time regarding proper quantities and sequence of chemical treatments to assure maximum and lasting restoration of well capacity also would be speculative.

It is recommended that two contractors be hired during the summer or early fall of 1984 to utilize cooperatively determined treatment procedures on the four wells recommended for rehabilitation work. Two contractors who may be considered for this work are Layne-Western, Inc., St. Louis, Missouri, and Aylor Aqua Services, Inc., Dyersburg, Tennessee. Documentation of the procedures during the field work, followed by controlled step tests, is recommended to begin proper assessment of the rehabilitation techniques employed. Close monitoring of these wells with time will also be important to learn how long the restored capacity is maintained.

Rehabilitation work on other wells is not recommended until step tests have been conducted and analyzed to confirm their present condition. If other wells are found to be in need of treatment, this work can be planned for FY 86. Depending upon the results of the rehabilitation work on the first four wells, similar procedures may be recommended or other contractors may be utilized to demonstrate their rehabilitation techniques. Careful investigation and documentation of the various treatment methods together with post-treatment monitoring will result in a treatment technique known to provide good results under most conditions. This information will be invaluable in maintaining the dewatering system.

System Operational Management

Based on the results of the analytical model of the 1-70, 1-64, and 25th Street sites, the burden of pumpage should be shifted toward the 1-70 wells and away from the wells at the SE end of 1-64. The well groups, as outlined in the discussion of the model application and reiterated in table 7, can provide a basis for the distribution of pumpage.

Table 7. Recommended Distribution of Pumpage

Site	Well groups	Max. wells required	Max. utilization
I-70	1,2,3,4,7,8,9,10	5	63%
	5,6,11,12	2	50%
I-64 (NW)	1,2,11,12	2	50%
	3,4,5,13,14,15	1	17%
I-64 (SE)	6,7,8,9,10 16,17,18,19,20	1	10%
25th St.	1,5,6,10	1	25%
	2,3,4,7,8,9	2	33%

The success of a dewatering system is ultimately measured by its effectiveness in alleviating the conditions it was designed to prevent. In this respect, the present methods of scheduling well use are succeeding.

The number of wells recommended in table 7 represents the number required for severe conditions (that is, conditions of high recharge and high river stage), which typically occur in the spring. Groundwater levels recede in the summer, reducing the required drawdown and the number of wells required to produce that drawdown. For these conditions, it is recommended that well usage be uniformly reduced, reducing the total number of wells and preserving the general distribution of pumpage. An example of these least severe conditions is the simulated pumping period ending August 28, 1978, which is included in Appendix C.

With these general principles in mind, table 8 and figure 14 have been provided to aid in the analysis of specific well configurations. The data in table 8 consist of the drawdown produced by each well at each of the four critical points. These critical points are the target water level elevation locations for Well Nos. 2 and 9 at 1-70, Well No. 9 at 1-64, and Well No. 9 at 25th Street. Also included in table 8 are adjusted target drawdowns for three sets of reference heads and flow gradients. *Severe* conditions are represented by a high reference head and a flat gradient (405 ft msl and 1.5 ft/mile, respectively), mild conditions are represented by a low reference head and a steep gradient (398 ft msl and 3.5 ft/mile, respectively), and moderate conditions are represented by intermediate values of reference head and gradient (402 ft msl and 2.5 ft/mile, respectively). These heads and gradients can be used to evaluate a proposed well configuration by selecting a set of wells, summing their individual drawdowns at each critical point, and comparing the total to the target drawdown for the appropriate conditions. Generally, expectations would be for severe conditions during

Table 8. Computed Drawdowns for Critical Points

Well ID.	7002 (ft)	7009 (ft)	6409 (ft)	2509 (ft)
7001	2.1	1.8	0.7	0.3
7002	2.4	1.9	0.8	0.3
7003	1.9	2.1	0.8	0.3
7004	1.8	2.3	0.8	0.3
7005	1.7	2.1	0.8	0.3
7006	1.7	2.0	0.8	0.4
7007	2.5	2.0	0.8	0.3
7008	2.4	2.4	0.8	0.3
7009	2.0	2.5	0.8	0.3
7010	1.8	2.1	0.8	0.4
7011	1.6	1.9	0.8	0.4
7012	1.5	1.8	0.8	0.4
6401	1.7	1.8	0.9	0.4
6402	1.4	1.5	0.9	0.4
6403	1.3	1.3	1.0	0.4
6404	1.2	1.2	1.1	0.4
6405	1.1	1.2	1.1	0.4
6406	0.9	1.0	1.5	0.5
6407	0.9	0.9	1.6	0.5
6408	0.8	0.9	1.9	0.5
6409	0.8	0.8	2.5	0.6
6410	0.7	0.8	2.0	0.6
6411	1.5	1.6	0.9	0.4
6412	1.4	1.4	0.9	0.4
6413	1.2	1.3	1.0	0.4
6414	1.1	1.2	1.1	0.4
6415	1.1	1.1	1.2	0.4
6416	0.9	0.9	1.5	0.5
6417	0.8	0.8	2.0	0.5
6418	0.8	0.8	2.2	0.5
6419	0.8	0.8	2.4	0.5
6420	0.7	0.7	1.8	0.6
2501	0.4	0.4	0.7	2.4
2502	0.4	0.4	0.7	2.7
2503	0.4	0.4	0.7	2.6
2504	0.4	0.4	0.7	2.4
2505	0.4	0.4	0.7	2.2
2506	0.4	0.4	0.6	2.1
2507	0.4	0.4	0.6	2.3
2508	0.4	0.4	0.6	2.6
2509	0.4	0.4	0.6	2.6
2510	0.4	0.4	0.6	2.5

ADJUSTED TARGET DRAWDOWNS

Grad. (ft/mi)	Ref. Head (ft)	7002 (ft)	7009 (ft)	6409 (ft)	2509 (ft)	
1.5	405.	20.8	22.5	11.4	10.3	Severe
2.5	402.	16.7	18.5	7.7	7.2	Moderate
3.5	398.	11.1	12.8	2.8	3.2	Mild

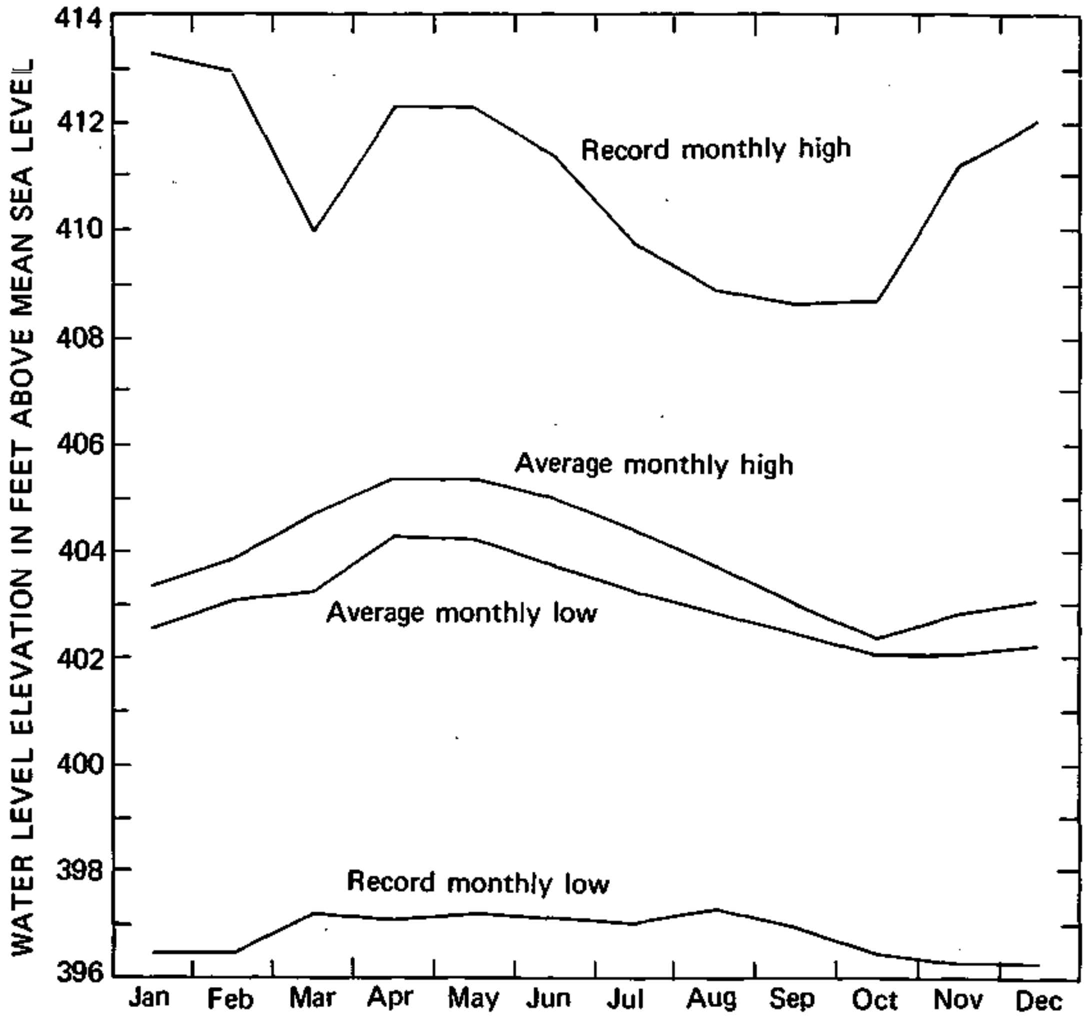


Figure 14. Monthly water level averages and extremes at ISWS Observation Well No. 2, January 1952 - December 1983

early spring months, moderate conditions in late spring and summer, and mild conditions in the fall and winter. Selection of the appropriate conditions, however, must also depend upon precipitation patterns and river stage. Figure 14 shows ranges of water levels for a State Water Survey observation well near the modeled area. These water level elevations can be used as a guide for selecting appropriate reference elevations.

With the distribution of pumpage shifted toward 1-70, accurate knowledge of the hydraulic performance of these wells becomes even more important. The functional well group with the highest percent utilization contains the eight wells from the center to the west end of the 1-70 site. Of these, this report has recommended three (Nos. 2, 3, and 7) for rehabilitation or replacement. The need for current knowledge of the condition of the others is critical. Well groups adjacent to the one described above are of somewhat lesser importance. Knowledge of their performance, while not as vital as knowledge of the above group, is very important when compared to the remaining groups.

In addition to knowledge of the hydraulic behavior of wells, individual well discharge rates should be monitored. Incomplete knowledge of the withdrawal rates was believed to be the largest source of error in the predictive capabilities of the model. Without this knowledge, precise evaluation of the system performance is impossible.

Future Investigations

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 based on present data is not warranted. Step tests are recommended to properly assess the condition of the following wells:

1-70 Well Nos.:
1, 4, 5, 8, 9, 10, 11

1-64 Well Nos.:
3, 10, 11, 13

25th Street Well No.:
6

These twelve wells could be tested during the next one or two summers, with priority given to testing the wells at the 1-70 site. In addition, step tests are recommended on replacement wells and following any rehabilitation efforts on existing wells. These step tests will permit proper assessment of the well improvement and new well condition.

Further investigation into the chemistry of the groundwater as it affects dewatering well performance and rehabilitation work is recommended. Samples collected from one well at various times of pumping and standby status may provide information on chemical reactions occurring in the gravel pack and aquifer as pumping continues. Samples collected from piezometers placed in line at distances from an operating well might also provide additional information on chemical reactions.

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A NOTE CONCERNING APPENDICES A, B, AND C

Three 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; and Appendix C provides selected results of the model calibration.

These three appendices have been printed under separate cover as Illinois State Water Survey Contract Report 341A. A limited number of copies is available and may be obtained by writing to the Illinois State Water Survey, Groundwater Section, P.O. Box 5050, Station A, Champaign, Illinois 61820, or by calling (217) 333-2210.