Dewatering of Deep Excavation in Gravity Aquifer in Puerto Rico
Desagüe de Excavación Profunda en Acuífero Inconfinado en Puerto Rico

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Abstract

Design of dewatering for an excavation into a gravity aquifer is discussed. The excavation for a pumping station was located 1,500 feet from the ocean and was successfully dewatered using seven high capacity deep wells. Falling head permeability tests in boreholes were conducted followed by various pumping tests used to predict aquifer behavior. Aquifer parameters were inferred from borehole tests and were later compared with data from the pump tests. Permeability values using the Modified Theis Analysis were consistently higher than results using the Well Function. Borehole permeability values were consistently lower than pump test results and backfigured values. Drawdown and flow predictions are compared with results obtained during excavation. It is concluded that, while pumping tests are necessary for design of major excavations into gravity aquifers, the tests must include sufficient observation wells to detect local variations in behavior. These variations will affect the general behavior noted during the pump tests.

Resumen

El artículo presenta el diseño del sistema de desagüe para una excavación de 56 pies, con 38 pies bajo agua dentro de un acuífero inconfinado. La excavación se hizo a 1.500 pies del mar, para una estación de bombeo. Se utilizaron 7 bombas de alta capacidad. Los parámetros hidrogeológicos fueron inicialmente inferidos haciendo pruebas de permeabilidad con carga variable en los sondeos. Luego, fueron comparados con los resultados de pruebas de bombeo. Los valores de permeabilidad que se obtuvieron con el análisis Theis Modificado fueron consistentemente mayores de los que se obtuvieron usando la expresión aritmética W(u). Por otro lado, los resultados que se obtuvieron de las pruebas de permeabilidad en sondeos fueron menores a las de pruebas de bombeo. Las predicciones de abatimiento y flujo fueron comparadas con los resultados. Se concluye que, para acuíferos inconfinados, se requieren pruebas de bombeo para ejecutar diseños más confiables. Sin embargo, los resultados deben estar basados en un suficiente número de puntos de observación que permitan detectar variaciones locales que afectarán el comportamiento general que se obtiene de las pruebas de bombeo.

1 GEOLOGIC SETTING

A deep excavation was necessary for construction of a pumping station at a major treatment plant in the town of Arecibo, about 30 miles west from San Juan. The Atlantic Ocean lies 1,500 feet from the site. The required excavation depth was 56 feet, of which 38 feet were below water. Initial ground levels were about 20 feet above Mean Sea Level. Water levels were about 1.3 feet above Mean Sea Level; numerous morning and afternoon water level readings did not indicate variations due to sea levels.

Subsurface soils began with an upper layer of eolianite, 15 feet thick, followed by calcareous sands and silts with varying amounts of clay and cementation, past the bottom of the required excavation. The calcareous sand and silt deposit consisted of varying amounts of kaolinite and quartz sand (Briggs, 1966; Capacete, 1974) that is residual from volcanic and plutonic rocks; its initial deposition has been postulated in the Miocene Age. Its appearance varies from a calcareous sandy silty matrix with varying amounts of clay and thin fissures, to sandstone or cemented silt with fine sand and silt or clay; this deposit is misleadingly labeled as "Blanket

Sands" since it implies a deposit of granular material. Instead, from appearances based on extracted samples, the material is impervious. Many of the samples in the Blanket Sands present values for unconfined compressive strengths, indicating cohesion; the boring sample descriptions could suggest little flow. But, the presence of calcareous cemented material can now be seen as a clue to high water flows. Below the blanket sands, there is a layer known as the Camuy Limestone.

2 SUBSURFACE INVESTIGATION AND BOREHOLE TESTING

Test borings consisted of SPT borings with 5-foot sampling; many years later, the author would have recommended continuous sampling and a large diameter sampler. Nevertheless, even with the usual sampling program, the N-values and sample descriptions indicated the variability in subsoil conditions (Figure 1). N-values within the blanket sands had large variation, typically between 10 and 50 blows per foot (bpf), up to depths of 75 feet, then reaching near-refusal, which often decreased with greater penetration. Samples retrieved from the topmost portions of each borehole were described as cemented sandstone. These were followed by clay or clayey sand samples that eventually changed to cemented sands or silts, with numerous clay lenses or layers. As the Camuy Limestone was reached, N-values usually rose dramatically to refusal, although in some cases they dropped to values in the range between 30 to 50 bpf. The basement layer was characterized by high N-values and a reddish brown color. It varied from fine sandy silt and cemented sand fragments, to cemented sand fragments in a compact silt matrix, to slightly cohesive fine to medium silty sand and cemented sandstone with traces of shell fragments.

Borehole falling head permeability tests were conducted; the tests consisted of filling a 2.5-inch casing driven to the test depth then retracted 2 feet, as detailed in DM-7 (1982). Falling head tests yielded variable results, with permeabilities ranging from 10^{-1} to 10^{-4} ft/min; it was noted that some of these values tended to increase below elevation -66 feet, or 37 feet below the proposed excavation bottom (Figure 2). The lower N-values did not exactly correspond with higher k-values.

Grain size analyses on some split-spoon samples yielded D_{50} ~0.02mm (medium to coarse silt, MIT Classification). However, numerous boulders and coarser or finer layers were frequent throughout. Excavated material from several test pits indicated the presence of boulders in a clayey-silty matrix. Test pits within the site showed continuous flows through fissures, although the material had significant amounts of clay and silt. Surface water inflows could be seen to quickly disappear into the fissures in the clay-silt-boulder matrix. The fissures appeared to be geological features from high overconsolidation, or thin slits between weathered cemented material.

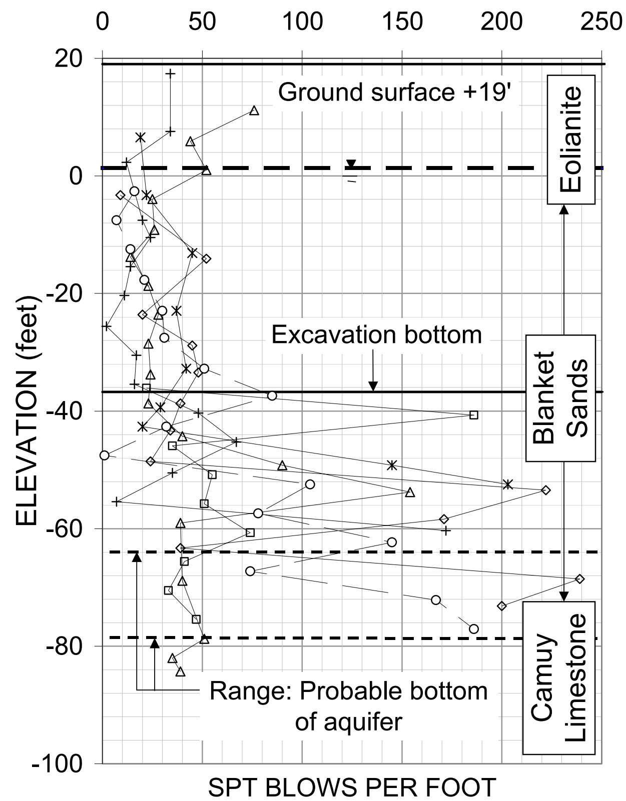

Figure 1 SPT N-values

3 PUMP TESTS

Due to uncertainties regarding permeability, full-scale pumping tests were deemed necessary (Powers, 1975). A 20-inch diameter test well was drilled with 10-inch casing and 5-inch gravel pack, strategically chosen so that it could possibly form part of any final solution; the well was drilled to a depth of 90 feet below the ground surface and 30 feet below the bottom of the excavation, where the borehole tests and visual observations indicated that tighter or denser material was to be found; this bottom would be, in essence, the bottom of the gravity aquifer. Unfortunately, installation procedures were such that the thickness of the gravel pack was uncertain, and possibly was inexistent at some depths.

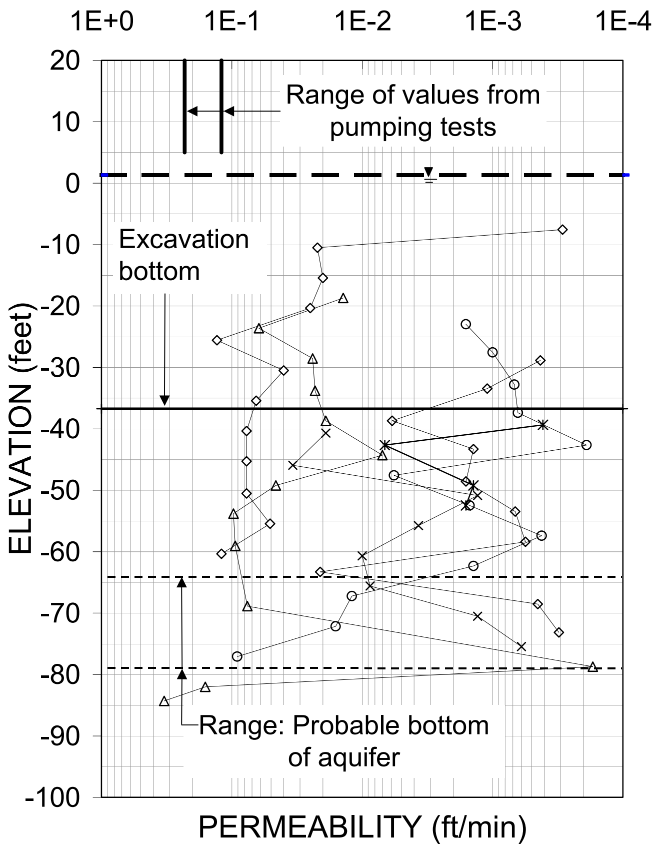

Figure 2 Falling head borehole permeabilities

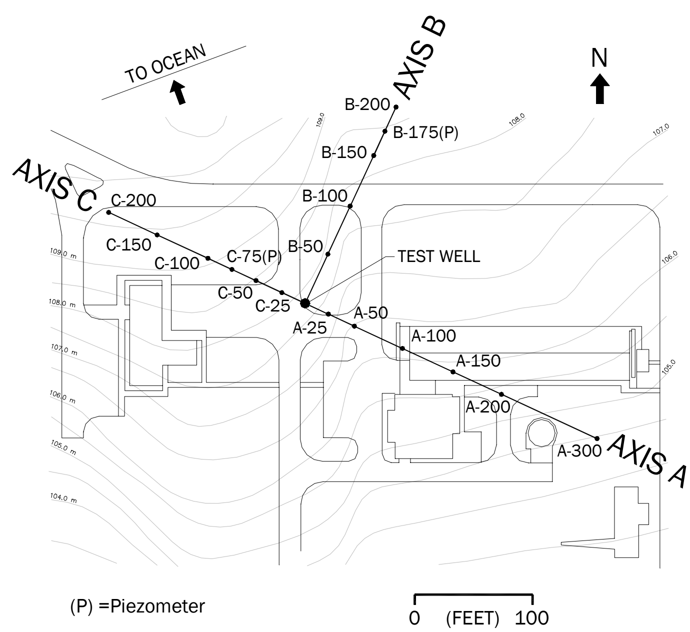

Figure 3 Test well and observation array

The parameters of most interest at this stage of design were: aquifer thickness or height of aquifer (H), permeability (k), radius of influence (R). Maximum flow from the least number of wells was a design goal, if the required drawdown could be attained without drying of the wells. Flows were calculated for the test well at approximately 500 gallons per minute (gpm). The methodology followed is described in Mansur and Kaufman (1962).

In this phase of work, it was assumed that the upper bound for permeability would be approximately $1x10^{-1}$ ft/min, but that it could reach as low as $1x10^{-2}$ ft/min, in spite of the relatively-low k-values obtained in the falling-head tests. Slotted screen, with open horizontal louvers and 0.25” horizontal shutter slots, was installed over the entire length of the well; the effective entry area for this type of screen was 5 percent of the screen length, a very low value. After installation, the well was developed by surging using bailers. A vertical turbine pump was installed in the well. To verify drawdown caused by pumping, 15 observation wells and 2 piezometers were installed as shown on Figure 3; installation depths varied between 20 and 45 feet, with the deeper wells nearer to the test well. The monitoring array consisted of three axes and followed a circular pattern around the test well; two of the axes were pointed towards the ocean. The three piezometers were installed in an attempt to determine the existence of artesian pressures, which could affect the dewatering analysis and the stability of the excavation.

4 PUMP TEST RESULTS

Three different pumping tests were conducted, with the first and third tests lasting three days over 48 hours, and a second test that was stopped in less than one day due to pump malfunction. Pumping rates were 480, 585 and 550 gpm, respectively; flow rates were measured by the orifice method (Anderson, 1971). Drawdown during all the pump tests was, not surprisingly, higher near the well. However, this was not always the case, and in several instances the water elevations in observation wells surged or varied and even increased past original levels for small periods; one of the observation wells dried up for a brief period of time. For the first pump test, water levels within the pumped well fluctuated at times, after apparently reaching steady-state, in spite of constant engine RPM. In the second 3-day test, the drawdown level in the well became stabilized but water levels in some, not all, of the observation wells continued to vary. Figure 4 shows an example of the drawdown levels that were achieved during pumping. Figure 5 presents drawdown observed at each axis during the first and third tests. The extrapolated radii of influence lie between 600 and 1,300 feet. Based on the sample descriptions and permeability tests, full penetration of the aquifer was assumed. Data were analyzed using both steady state and transient methods, using the Well Function W(u) and the Modified Theis Analysis. The Well Function, W(u) is used for unsteady flow in confined or unconfined aquifers (Driscoll, 1986). In these analyses, flow (Q) is related to drawdown (DD) and aquifer parameters T (transmissivity) and storage coefficient, s_c, are obtained. From the transmissivity, the permeability is obtained. The Modified Theis Analysis can be used for large times "t" and small distances from the pumped well. Table 1 presents an example of results obtained for drawdown tests, in this case for an assumed height of aquifer equal to 64 feet; analyses were also performed for aquifer heights of 73 and 80 feet, which would have resulted in smaller amounts of drawdown for the same pumping rate, and recovery data were also obtained. It should be noted that the permeability values obtained from the Modified Theis Analysis are roughly 50 percent higher than the values obtained from the Well Function Analysis. The calculations with different aquifer heights responded to uncertainties in the exact location of the bottom impervious layer.

Table 1 Test #3, Q=550 gpm, Drawdown

Method of Analysis	Location	Average k ft/min	s_c
Well Function W(u)	Axis A	1.2×10^{-1}	.008 (.0007 to .029)
	Axis B	1.8×10^{-1}	.009 (.0075 to .012)
	Axis C	1.2×10^{-1}	.007 (.0028 to .019)
Modified Theis Analysis	Axis A	1.9×10^{-1}	-
	Axis B	2.3×10^{-1}	-
	Axis C	3.6×10^{-1}	-

Note: Locations shown on Figure 3. Storage coefficient = s_c

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For artesian aquifers, storage coefficients range between 10^{-5} and 10^{-3}, while typical values for gravity aquifers are 0.01 to 0.30. Most of the calculated storage coefficients varied between 0.007 and 0.41, averaging 0.008. The values obtained are generally associated with gravity aquifers. Some of the results at particular observation wells, however, showed storage coefficients of 0.0040 to 0.0007. It was concluded that the site behaved as a gravity aquifer, but speculated that the lower values indicated areas or occasions that flow corresponded to temporary artesian conditions, and that the surging or variations noted in the test well and observation wells was related to situations where water became trapped and eventually flowed in response to large flow gradients, so that artesian conditions were created between less-pervious zones, or that portions of the aquifer became isolated. It was anticipated that trapped water pockets could be encountered in the excavation faces. Design was conducted for the gravity aquifer condition, which seemed to prevail based on the values for storage coefficient that were obtained, also indicated by review of the subsoil profile. The relatively-large number of wells was extremely helpful in the analysis; a large amount of data helped to confirm trends and to detect variations from expected behavior.

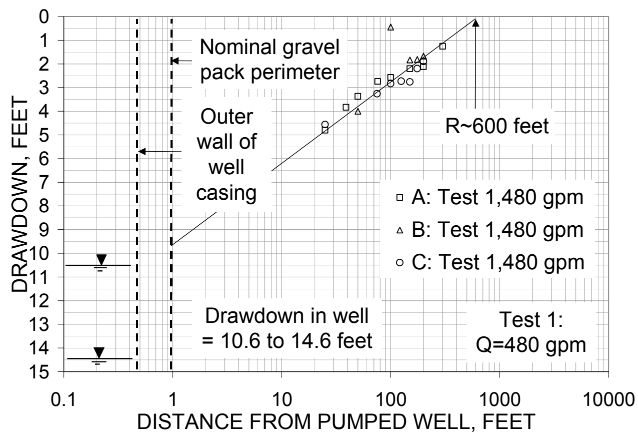

Figure 4 Drawdown during Pump Test 1

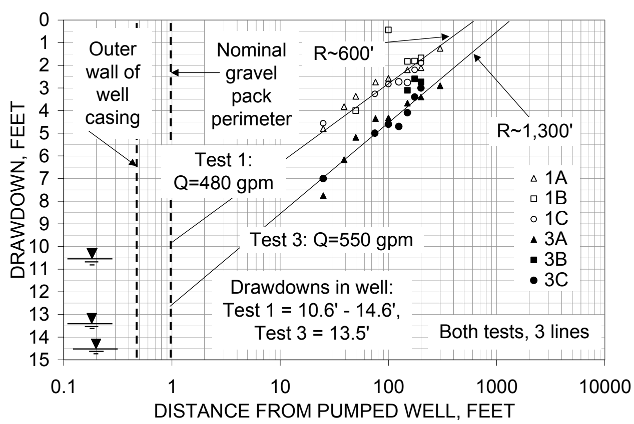

Figure 5 Drawdown along axes

5 DESIGN DETAILS

Design of a dewatering system requires determination of the radius of influence. In this case, if the radius was greater than 1,500 feet, the ocean would act as a line source, which would significantly increase the flows required for a given drawdown. The analyses indicated that the radius of influence R varied between 600 and

1,200 feet. Other design values obtained from the pumping tests and the borings were k = 1.0 x 10^{-1} ft/min to 2.0 x 10^{-1} ft/min and H= 64 to 80 feet. There were no important structures within the radius of influence; moreover, any settlements due to dewatering in the area would have been insignificant given the nature of the material being dewatered.

Wellpoints were impractical due to time and costs. Variations in permeability indicated that wellpoints would have to be encased in gravel filters to connect layers, adding to costs and uncertainties; the cemented nature of the material precluded jetting of the wellpoints, also adding to the costs. In fact, wellpoints would not have worked since flow in many areas was due to flow through large cavities or fissures that would not have been intercepted by the wellpoints.

For the excavation slopes, 1:1 slopes with intermediate 2-meter berms were selected, since the cemented nature of the material provided adequate stability. The most efficient dewatering scheme appeared to be a seven-well layout, with wells designed to pump 1,500 gpm each, which would adjust properly to the numerous uncertainties that existed.

Analyses for different scenarios indicated that the excavation could be effectively dewatered with 7 deep wells, each pumping 1,000 gpm to 1,500 gpm. The wells were sized at 12 inches diameter, with 4 inches of gravel packing on each side (retrospectively, too small). From the boring data, an aquifer height of 75 feet seemed reasonable and it was expected that the aquifer would be fully penetrated by using wells drilled to a depth 39 feet below the excavation bottom.

Since each well causes drawdown at other wells, a set of trial and error calculations with superposition was used to determine the most advantageous and efficient well locations (i.e., least number of wells) with large pumps that were then available (1976). The analyses first determined the pumping rate needed to dewater the center of the excavation for different scenarios, and then verified that the system in operation did not dry any of the wells, and that the slope faces were effectively dewatered. Several calculations were also performed assuming partial penetration to determine the effects of shorter wells or a deeper aquifer and varying heights of aquifer. The effect of a line source (the ocean) was considered, in case the radius of influence matched the distance to the ocean. A plan view of the excavation is shown in Figure 6.

The location and number of wells was selected by trial and error, trying to use the least number of wells to dewater the center, while at the same time maintaining the proposed slope faces dry. Several layouts were discussed with the contractor, and a seven-well array that allowed for construction of a ramp at 6% grade to remove spoils was selected; the entry ramp also reduced the excavation depth of the influent pipeline that was to be installed and connected to the pumping station, and spoils were trucked out using this ramp. Other nearby but shallower areas that required excavation were also considered in the design of the dewatering system and were simultaneously dewatered.

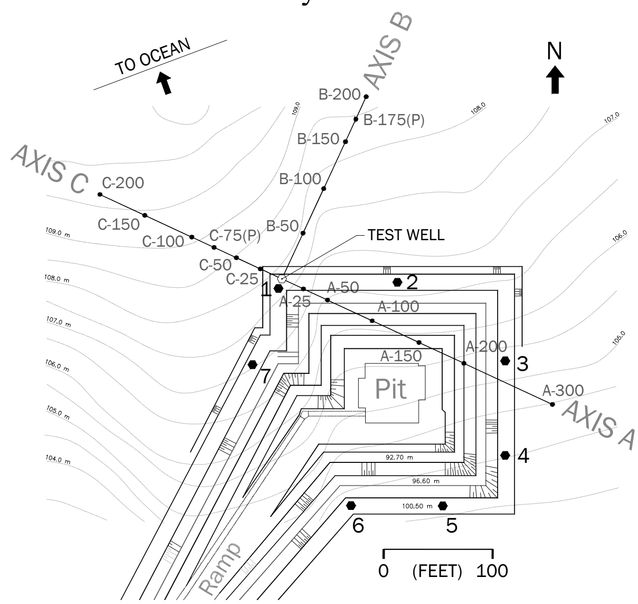

Figure 6 Excavation layout, plan view

Due to high flows required, screen selection was an important part of the well design. Corrosion and incrustation were potential problems since the wells might be in operation for up to 18 months (they were ultimately in operation for slightly more than a year); entrance velocities were to be kept less than 0.1 ft/sec at flows of 1,500 gpm (initial early-design flows). Slotted versus wirewound screens were compared, entrance areas for slotted screen were on the order of 5% of total area, versus 36% for continuous wire wound screens. To obtain low entrance velocities, wirewound screens were chosen for the 7 new wells; screen length was 30 feet on top of a 5-foot length of closed-end pipe that was first installed, which served as a sand trap at the bottom. One of the well contractors supplied louvered casing at some savings to the contractor, so that the wells included the inefficient casing over the wirewound screen. While a fully-screened well is unnecessary for a gravity aquifer, it did provide an additional margin if flows were segmented by layering.

The test well was deemed inefficient due to its size (10-inch diameter, which imposed use of small pumps) and uncertainties regarding the effective gravel screen thickness. Moreover, slotted screen had been used, which seriously limited the capacity of the test well. Therefore, the test well was not used further and pumping proceeded with the 7 new wells.

For the required pumping rates, pump engines were rated at 78 horsepower per pump, so electric power requirements were 42 kilowatts per pump. There were no nearby power sources so that the seven pumps were diesel-powered. Using a periodic maintenance schedule, it was intended that at least 6 of 7 pumps were always working. A preventive maintenance scheme was implemented, so that periodic maintenance involved 15 minutes of daily down-time for each engine.

6 INSTALLATION AND PERFORMANCE

The excavation was first taken to a depth that was one meter above water level. Three of the wells were installed by one contractor using percussion, and the other 4 were installed using rotary methods with a biodegradable bentonitic mix.

No mechanisms were installed to regulate flows at the pump head; it turned out that the motors would operate only at one specific speed, which tended to dry the wells. A valve was subsequently installed on each pump head to provide sufficient backpressure or head. Flow was measured using a plywood-box weir connected to the end of the discharge pipe for six of the wells (Anderson, 1971). Flow from the seventh well was not measured but appeared to be similar to the other wells.

An observation well was installed in the center of the proposed excavation area to observe drawdown. Initial flows were approximately 1,000 gpm per well but were not precisely measured during startup; moreover, not all wells were in operation continuously due to numerous operation problems, sand migration into the wells, and damaged impellers. In some of the wells, settlement of the surrounding gravel pack occurred with time, indicating its improper placement, which contributed to sand inflows. During operation, some of the wells had to be cleaned by air-surging until the gravel pack provided a proper filter. Pumping from some wells had to be stopped due to excessive wear on the impeller bowls; these wells were redeveloped by surging. Whenever pumps were stopped for engine maintenance, water would immediately seep from holes and fissures in the excavation face.

One week after the pumps were started, the water level within the excavation had dropped by 15 feet. Within 3 more weeks, water level dropped another 13 feet. Excavation bottom was reached in 70 days with a dry bottom. Once excavation bottom was reached, a thin lean-concrete working mat was poured.

Midway through the excavation, with 24 feet of drawdown at center and all wells in operation, roughly 14 feet above the required excavation bottom, water under pressure started coming out the sides of the excavation face. This lasted about 5 minutes until flows stopped. The excavation bottom was inundated with about 6 inches of water. Thus, the theory that perched or trapped bodies of water might be found was confirmed. It became evident that there existed small perched or trapped bodies of water, formed by discrete clay beds. Water tended to accumulate over these clay beds (as in a spoon). Beds were suddenly punctured by piping flows or by excavation against one of these zones. This hypothesis could explain observed variable and surging water levels in the observation wells during the pump tests and clarify the few occurrences of storage coefficients with values similar to artesian aquifers.

A photograph of the completed excavation is shown in Figure 7.

Figure 7 Completed excavation

7 BACKFIGURED PARAMETERS

Steady-state or final flows were approximately 450 gpm per well. The possibility exists that the wells had unequal flows, since the drawdown measured at each well were unequal; unfortunately, they were not recorded. The backfigured radius of influence was approximately 800 feet, using a permeability of 0.7×10^{-1} ft/min, or 1,300 feet with a permeability of 0.9×10^{-1} ft/min, about half the maximum design permeability. There was no evident influence from the ocean.

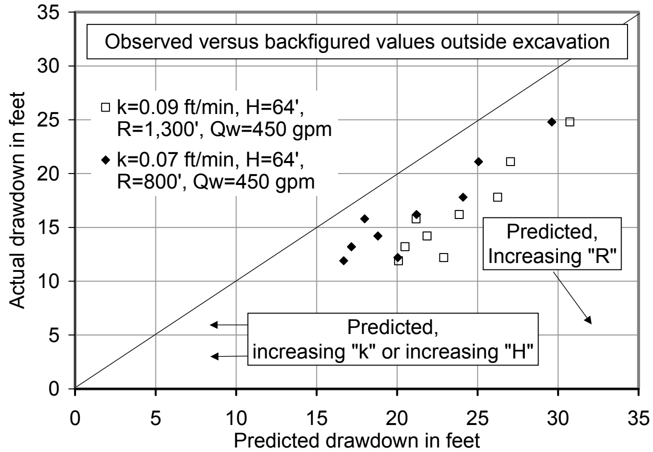

Figure 8 Predicted versus actual drawdown

While various sets of backfigured parameters can be made to match the amount of observed drawdown at the excavation center, the same backfigured values tend to overpredict the observed drawdown in the observation wells outside the excavation. In eight measured cases, there is no one set of parameters that can be made to match the measured outside drawdown while causing a dry excavation. Figure 8 compares predicted and actual values using one set of parameters and as measured at certain distances from the center of the excavation. Clearly, the data show a smaller radius of influence than is backfigured from all the data; when plotted on a logarithmic scale versus distance (Figure 9), the apparent radius of influence is about 900 feet. It would appear that this result is due to layering within the aquifer, such as caused by cementation or clay layers. In other words, layering in the deposit is reducing the radius of influence.

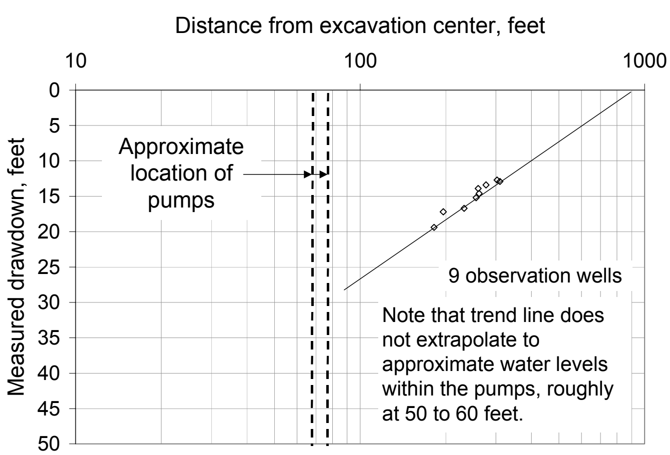

Figure 9 Measured drawdown

8 CONCLUSIONS

As the excavation was deepened, the character of the deposit became evident. It also became evident that larger diameter samples would have been very helpful in identification of the type of deposit; variations in cementation or numerous flow paths seen during excavation perhaps would have been apparent. The appearance of small-diameter soil samples normally obtained during sampling can be misleading, as was the case. Continuous sampling is thus recommended, as well as large-diameter sampling. The data confirm that borehole tests can indicate ranges in permeability but that the results are insufficient for design, except for small excavations, and must be tempered by experience. Moreover, falling head tests are affected by local permeability or spatial variations, whereas pumping tests can provide valuable clues to performance.

Full-scale pumping tests are necessary to properly design a major excavation. Even for small excavations, permeability and layering directly affect rate of flow, so that the investment in additional testing is usually justified. It is important to realize that dewatering emphasizes the inherent variations in any soil or rock deposit, and that close supervision by the designer during construction is necessary in order to respond to these structural discrepancies. Sufficient observation wells and piezometers are necessary to detect local variations in behavior.

A lesson learned was that close attention to pump and engineering specifications is essential. Insufficient attention was given to the gravel pack in the design stage, and a larger hole diameter should have been specified; given the wide variation in cementation and effective soil sizes, a larger gravel pack could have reduced infiltration and subsequent damage to impellers. Depending on the type of pumps that are selected, it may be necessary to install valves at each well head in order to control flows; alternately, easily-regulated pumps are required to avoid drying of the well. So, close attention must be given to well installation specifications, and generous gravel casings are recommended to reduce downtime and maintenance, which affects rate of dewatering and excavation.

In present-day terms (2003), a better design would perhaps include more wells and submersible pumps instead of vertical turbine pumps. A greater number of wells can account for local differences in the aquifer and can achieve the same drawdown with smaller individual flow rates.

In general, geotechnical investigations for foundation design have little relevance to dewatering design except for preliminary information, since data provided relative to dewatering or excavation are few. Construction designs are usually outside the scope of work contracted by the designer or owner in the design stage. More SPT borings or CPT soundings or other geophysical investigation may be required to design the excavation, than for foundation design. In this case, nine additional borings with falling-head tests were conducted, in addition to two borings previously drilled at this specific location and more than 20 borings for the foundation design of the entire treatment plant. A previous paper (Crumley, 1998) recommended one boring or sounding per 1,000 ft^2 for preliminary excavation or dewatering design, or one boring per 500 ft^2 for final excavation or dewatering design, taken over the structure footprint. For this project, boring density for dewatering and excavation design was one boring per 400 ft^2 considering the structure footprint only, or one boring per 7,000 ft^2 if the entire excavation outline is considered.

The excavation was successfully backfilled. Pumps were sequentially turned off as backfill was placed past the water level.

9 CLOSURE AND ACKNOWLEDGEMENTS

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