Lessons Learned: Dewatering of an Open Cut in a Confined Aquifer
Enseñanzas del Desagüe de una Excavación en un Acuífero Confinado

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Abstract
Construction of a deep pumping station required excavation into a confined aquifer. Soils consisted of alternating cohesive and granular layers. An extensive investigation was followed by various pumping tests used to predict aquifer behavior. The open cut excavation was dewatered by an initial level of wellpoints and was to be followed by another wellpoint level and surface drainage from trenches inside the final excavation berm. It was ultimately dewatered completely using a peripheral drainage system installed within each berm. Aquifer parameters were inferred from borehole tests and are compared with data from the pump tests. Drawdown and flow predictions are compared with results obtained during excavation. It is concluded that dewatering designs in confined aquifers must consider influences from different aquifers, and that layering can seriously affect predictions that are solely based on perceived or calculated parameters. Given time and cost constraints that normally preclude multiple well tests, it is necessary to consider various scenarios before excavation. Finally, it is once again confirmed that investigations for foundation design usually provide few data for use in a dewatering and excavation design, so that additional investigation and analyses are usually necessary to provide reasonable indications of parameters.

Resumen
Para la construcción de una estación de bombeo, fue necesario excavar en un acuífero confinado. El perfil del subsuelo consistía de capas alternas de suelos cohesivos y granulares. Luego de una extensa investigación, se realizaron pruebas de bombeo para predecir el comportamiento del acuífero. La excavación a cielo abierto fue inicialmente desaguada en una primera etapa con puntas coladeras; el diseño incluiría otro nivel de puntas y desagüe con trincheras en el fondo. El diseño fue modificado según los resultados. La excavación finalmente fue desaguada con un sistema de drenaje periferal en cada berma y al fondo usando zanjas rellenas de grava y pozos criollos. Al inicio se obtuvieron los parámetros del acuífero con sondeos y pruebas de pruebas de permeabilidad en sondeos con carga variable; se comparan estos resultados con los de las pruebas de bombeo. Se comparan las predicciones de flujo y abatimiento con los resultados obtenidos. Se concluye que obras de desagüe en acuíferos confinados deberán considerar influencias de otros acuíferos, y que la estratificación puede seriamente afectar las predicciones que se hacen a base de parámetros percibidos u obtenidos de pruebas en sondeos Dado las restricciones de costo y tiempo en la mayoría de los casos, es necesario considerar varios escenarios antes de proceder con la excavación. Finalmente, una vez más se confirma que las investigaciones realizadas para diseño de cimentaciones rara vez proveen datos utilizables en diseños de excavaciones y obras de desagüe, por lo que se requieren investigaciones y análisis adicionales para obtener los parámetros de diseño.

1 EXCAVATION AND SUBSURFACE DETAILS

At Guayama in south Puerto Rico, deep excavations were required for construction of various structures that were necessary at a waste treatment plant. Construction of a deep pump station required excavation to 53 feet that included 32 feet below water. The bottom of the pump station measured 60 by 90 feet; the top of the open excavation described herein ultimately measured 240 by 280 feet. While most of the land on the boundaries consisted of sugar cane fields, there was a nearby refinery on the western boundary of the excavation that could be affected by prolonged dewatering.

The site lies about 2,000 feet from the ocean. A subsoil profile presents an alluvial scenario, with 23 to 40 feet of intervening stiff clay and dense silty sand layers (about 20 to 40 feet thick), over 30 feet of highly weathered diorite bedrock (with

SPT N~100), over unweathered diorite. Water lies 15 feet below ground surface; this is also about 15 feet above mean sea level, which indicates significant recharge from mountains some 3 miles to the north.

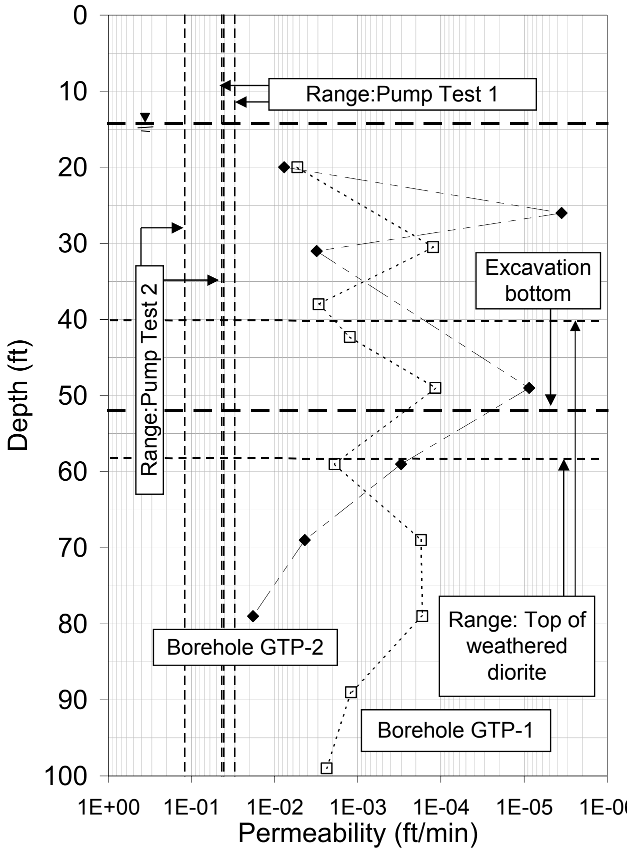

Figure 1 Borehole Falling Head Tests

For the entire wastewater plant, a rather thorough subsoil investigation had been performed for foundation design using Standard Penetration Tests (SPT) at 5-foot intervals. The presence of a heterogeneous stratigraphy became evident early in the design. The effect of layering on dewatering was recognized before excavation began, and it became a very important factor as initial results were evaluated. For the granular layers, grain size analyses indicated the following size ranges: gravel 28 to 40%, sand 52 to 53%, fines 20 to 7%. At D_{50}, the size range was on the order of 2 mm=0.35 inches, a coarse sand although, as noted, there existed significant variation in fines content. Up to 5 feet of artesian pressures were registered in some piezometers installed at the site. Falling head borehole permeability tests were conducted within the proposed excavation area, yielding the results shown on Figure 1. Numerous borings indicated that the sand and gravel deposits had variable thickness and were interbedded or interrupted by stiff clay layers. But, horizontal continuity of layers was not easily evident since continuous sampling was not performed.

2 PUMPING TESTS

Given the heterogeneous conditions encountered, and few data regarding subsurface flows at the site, two pumping tests were conducted on two 12-inch test wells, with 13 observation wells and 3 piezometers arranged in a convenient pattern (Figure 2); depths of instruments were on the order of 50 to 70 feet throughout the site, depending on location and distance. Since the initial profiles indicated that layering was not continuous, the design for the two test wells included 60' of continuous wire-wound screen, sufficient for a gravity or artesian situation. The first test lasted for 24 hours and pumped 300 gallons per minute (gpm). Drawdown at this well was 50 feet. The second well was pumped at 185 gpm for 9.5 hours and had 62 feet of drawdown. Both wells were 80 feet in length and 12 inches in diameter; flow rates were measured by the orifice method (Anderson, 1971). Neither of the wells could be pumped at higher rates without drying.

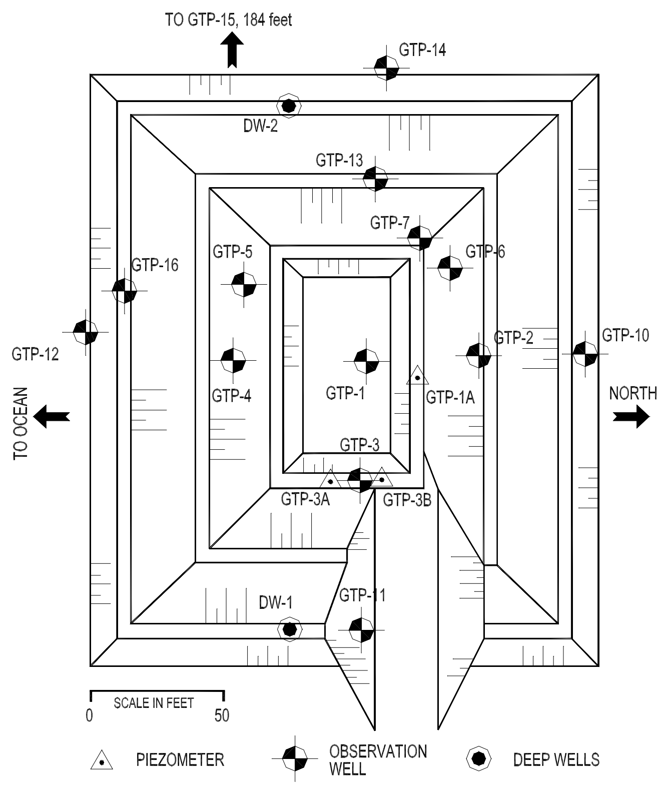

Figure 2 Test Layout

The time-drawdown relationship is greatly affected by the variation in aquifer thickness throughout the site, which at this site varied significantly, as shown on Figure 3. The best data available on spatial variation was the performance of the two pump test wells: two identical wells were installed, but the yield from one was about two-thirds of the other.

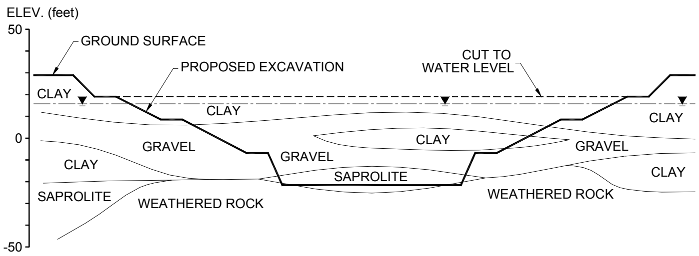

Figure 3 East-West Cross-Section at Excavation

Drawdown and recovery data were used with the Theis and Jacob methods. The time–drawdown plots indicated flattening of the curve over time, 80 to 300 minutes after start of pumping (Figure 4); this could indicate that the cone of depression has tapped into a recharge source, or that there exists leakage from another aquifer. Since there were no surface recharge sources within the radius of influence, it was concluded that the flattening of the drawdown curve was due to subsurface leakage, or intersection of the cone of depression with a zone of greater permeability, or a thicker aquifer. Figure 5 presents the response of two observation wells, one of which was located exactly midway the two pumping wells. Figure 6 shows drawdown against distance; the small radius of influence is apparent, although reliable data are few. Perhaps a better investment would have been multiple depth piezometers at the same locations.

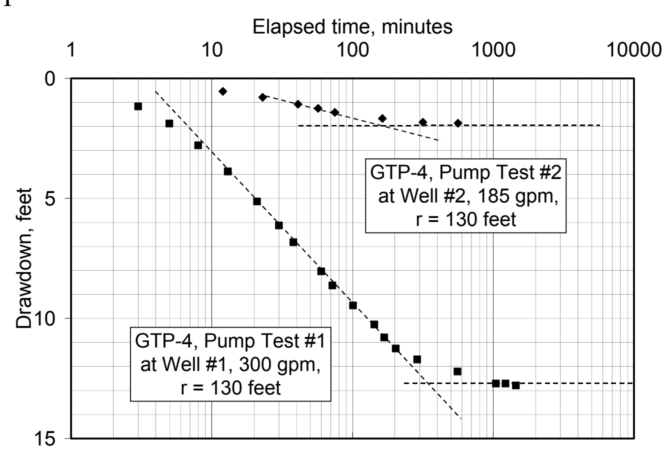

Figure 4 Time-drawdown curve

The results of the pump tests indicated transmissivities of 8,000 gpd/ft (gallons per day per foot) to 30,000 gpd/ft; higher values were noted in a north direction, towards the mountains. Only 4 of 14 observation wells or piezometers had significant response; one of the instruments had far greater drawdown than would have been expected from the other pump test. The average storage coefficient s_c was $7x10^{-4}$, but some values were recorded at 10^{-3}; these values are indicative of a confined aquifer. Storage coefficients were greater towards the hills than towards the ocean, indicating a “more-confined” condition. Permeabilities from each pumping test are presented in Table 1 and Table 2. The results indicate permeabilities that are significantly higher than those from falling head tests on borings within the same area. The responses in the instruments from each pump test clearly vary by two, with respect to permeability.

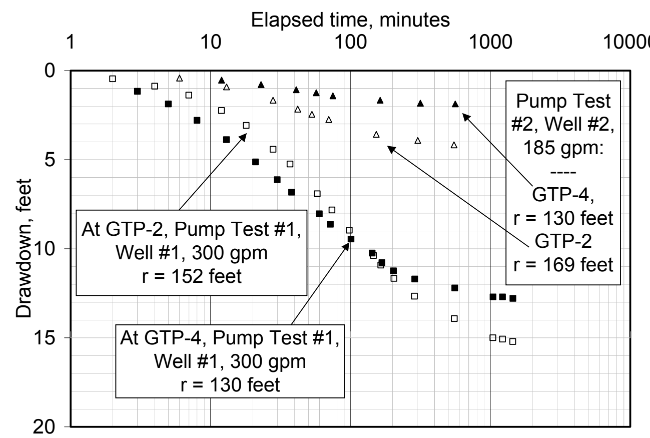

Figure 5 Leakage or Recharge

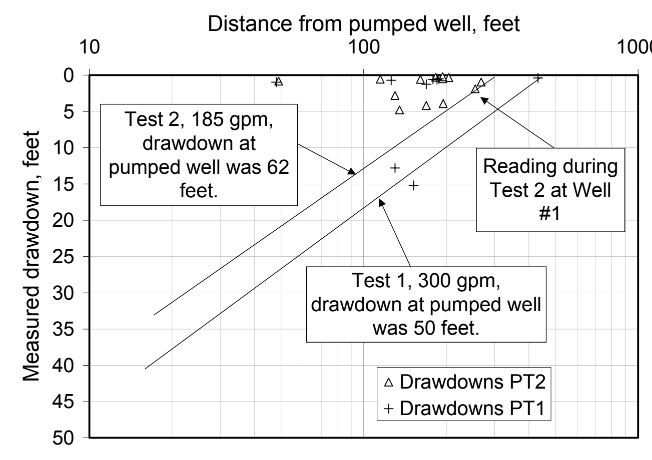

Figure 6 Distance against drawdown

Based on inspection of samples, it was decided to assume that saprolite and weathered rock were impervious, in spite of the fact that the two borehole falling head tests yielded permeabilities of 10^{-3} ft/min at the same depths. While these values could reflect faulty testing, they were probably due to deeper fractured rock zones.

Moreover, they would not affect dewatering above the assumed impervious layer. The relatively small size of the pumping station at the deepest excavation level and the lack of response from the piezometers during the pump test justified this assumption. Full penetration into the aquifer was assumed for the dewatering analyses. Since the water levels were much higher than mean sea level, it was apparent that flow would be similar to flow from a circular source, not from a line source such as the ocean.

Design values were as follows: k = 6.3 x 10^{-2} ft/min, to 1.9 x 10^{-1} ft/min, for alluvium; R = 500 to 1,000 feet.

Table 1 Results of Pump Test 1, Q=300 gpm

I	D	DD (t)	k1	k2	S
GTP -2 OW	152	12.7 (287)	3.0 x 10^{-2}	2.6 x 10^{-2}	7.0x10^{-4} (1.1x10^{-3})
GTP -4 OW	130	11.7 (288)	4.1 x 10^{-2}	3.5 x 10^{-2}	4.6x10^{-4} (7.2x10^{-4})
GTP-3 P	74	No DD	-	-	-
GTP -1 OW	126	No DD	-	-	-

Note: I=instrument name; OW= observation well; P= piezometer; D= distance to well; DD= drawdown in feet;(t)= time in minutes; k1= Theis permeability in ft/min, k2= Jacob permeability from recovery tests, ft/min; S= storage coefficient using Jacob and (Theis). All k based on H=30 feet. Well 1 was used as observation well in Pump Test 2.

Table 2 Results of Pump Test 2, Q=185 gpm

I	D	DD (t)	k1	k2	S
GTP -2 OW	169	3.9 (303)	5.1x 10^{-2}	-	7.0x10^{-4}
GTP -4 OW	130	2.6 (307)	9.7x 10^{-2}	8.3x 10^{-2}	1.5x10^{-3} (2.0x10^{-3})
GTP-3 P	195	3.7 (312)	4.3x 10^{-2}	-	1.3x 10^{-3} (1.7x10^{-3})
GTP -1 OW	138	4.6 (305)	5.5x 10^{-2}	5.3x 10^{-2}	7.0x 10^{-4} (9.9x10^{-4})
Well 1	255	1.8 (316)	1.2x 10^{-1}	-	7.0x 10^{-4} (9.9x10^{-4})

Note: See Table 1 for explanation of column headings.

3 DESIGN OF EXCAVATION AND DEWATERING SYSTEM

Dewatering analyses indicated that dewatering using deep wells was not feasible due to lack of horizontal continuity between some layers. It was concluded that dewatering by deep wells could be very ineffective. Well yields and drawdown were small and erratic due to layering, as indicated by the two wells that had already been installed, and by reevaluating the subsurface profiles. Given the variations in soil stratigraphy, in hydraulic conductivities and in aquifer behavior throughout the site, a two-stage wellpoint system was recommended to dewater the upper 22 to 24 feet of the excavation. The remaining 8 to 10 feet of the excavation would be dewatered based on the performance of the wellpoint system; sumps, drain trenches and/or possible use of the wells used for the pumping tests were considered feasible. Predicted steady-state flows to dewater the entire excavation were 1,000 gpm. It was also predicted that the upper stage could be turned off with time. But, the results of the initial wellpoint stage were not as expected, and it became necessary to modify the dewatering scheme.

The wellpoints were installed over a period of six weeks, with numerous difficulties encountered, due in part to the presence of boulders within the depth of the wellpoints. A sand and gravel filter was recommended at each wellpoint, installed using a holepuncher assembly, with backfill placed as the holepuncher was extracted. Once the wellpoint pumps were initially started, numerous problems with the pump engines arose. The wellpoints also became plugged and water levels did not significantly decrease within the excavation. Pumping rates for the wellpoint system were approximately 300 gpm. Most of the wellpoint tips were pulled and reinstalled with gravel backfill, and pumping was resumed. The new pumping rates were approximately 500 gpm, as measured by flows past a plywood weir, and the greatest amount of drawdown achieved inside the excavation was on the order of 6 feet. Vacuum gauge readings at the wellpoint pump confirmed an effective vacuum. Nonetheless, flow from the wellpoint system was not continuous, but in spurts. Tests showed that this behavior was due to little water at the south-southeast header pipe, since continuous flow was achieved only by pumping from the northwest-north header system.

A trench was opened at the northwest corner of the initial excavation level and layering was noted; stratification consisted of a well-defined gravel layer within clay layers. A sump pump was constructed, which consisted of an electrical dewatering pump installed within a stack of perforated steel drums, all encased within coarse gravel and boulders. Flows of 350 gpm were achieved from this single pump, as measured in the plywood weir. While the wellpoint system could be operated (at another 500 gpm), it turned out to be unnecessary: with the wellpoint system off, the sump pump was able to control all water inflows at the excavation at the first level. Water level readings were taken inside each of the previously-installed wellpoint tips, with wide variations in all the tips (6 to 10 feet difference, even between adjacent wells); the great variation in water levels confirms the obvious subsurface heterogeneity. It was concluded that a peripheral gravel drain system, installed in each berm, would adequately dewater the excavation without need for the wellpoint system. A system of sump pumps and peripheral trench drains within the excavation berms was quickly designed. Two peripheral trenches within the excavation berms and a lower sump effectively dewatered the excavation, pumping at a total rate of 850 gpm. The wellpoint system was not used further. Figure 7 shows one instance of a clay layer within gravel that impeded effective dewatering.


Figure 7 Clay layer within gravel

The excavation proceeded and dewatering was nearly immediate. Steep slopes were analysed and recommended; some sloughing of the excavation sides occurred towards the bottom of the excavation, due to fairly steep slopes and water flows from the excavation face.

Without any recharge system at the beginning of operations, pumping with the wellpoint system and sump pump had lowered water levels at the refinery, 250 feet from the excavation edge, by 3 feet in a boundary-line observation well. To recharge the topmost aquifer, a line source was installed at the west project boundary, which was the refinery's east property limit, and water from the sump pumps was reinjected into the aquifer. Piezometers within the lower aquifers were monitored to ensure that no significant drawdown existed at greater depths. The recharge system consisted of a two-foot-wide trench dug 20 feet into the underlying sand and gravel with eight 12"-diameter PVC pipe wells, slotted and open-ended, spaced 33 feet from each other on a line along the boundary with the refinery.

4 CONCLUSIONS

Backfigured values of permeability, based on actual pumping rates, were approximately 2 x 10^{-2} ft/min to 9 x 10^{-2} ft/min, for the entire excavation, assuming radii of influence of 400 to 800 feet. The borehole permeability tests appear to yield results that are one order of magnitude less than full-scale tests. Drawdown readings on nearby wells during excavation were not taken.

While the predicted flow (1,000 gpm) was similar to actual (850 gpm), the flow regime was not initially evident. The radius of influence was affected by leakage from another aquifer, although artesian conditions persisted during the excavation. In similar situations with various layers, it is recommended to install multiple-depth piezometers at several locations. This could allow better definition of high-yield layers that might remain undetected with a single instrument per location.

It was concluded that continuous sampling should have been used to design the dewatering system, to obtain a better profile for design. More borings and multiple-depth observation wells further from the excavation would have been helpful.

Once again, it is noted that significant testing was required to design the dewatering system. Geotechnical reports for foundation design that are made available for design of excavation and dewatering systems could provide little useful information. In this case, numerous additional borings at the excavation (15) and two pumping tests were conducted. The boring density was about one borehole per 6,000 square feet for the entire excavation, or 1 boring per 300 square feet if one considers only the structure area.

The total time to reach the bottom of the excavation, once the pump tests were completed and analysed, was 5 months.

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