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Hydrogeological Characterization in the Development of Underground Structures, Los Angeles Basin, California

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ABSTRACT

Underground construction offers economic benefit to urban developments where land values demand maximizing the potential of available resources, and the vertical nature of urban development requires thorough characterization of the hydrogeological as well as the geotechnical properties of sites. This unique set of challenges, if ignored, can result in engineering complications and economic disadvantages for urban development projects. Urban hydrogeology has often been studied in its relationship to water-resources management and large-scale trends, rather than the site-specific testing and analysis required for dewatering during building construction. Aquifer pumping tests were performed at two sites in the Hollywood Basin in Los Angeles, California, where there are ongoing subsurface construction and dewatering operations. Step drawdown and constant-rate pumping tests were performed at each site, and data were collected from both pumping and observation wells screened in sand and gravel aquifer units. Time-drawdown curves were analyzed via well-known analytical solutions for drawdown in confined and leaky aquifers. While one site responded to pumping in accordance to traditional analytical models, the other exhibited evidence of secondary recharge to the aquifer from local underground construction features. As a result of these findings, construction and dewatering plans at each site were altered in ways that deviated significantly from preconceived estimates. These case studies demonstrate the need for rigorous aquifer testing and analysis at urban construction sites undergoing dewatering, and they show the pitfalls that can be avoided through the application of such methods.

INTRODUCTION

Conducting aquifer pumping tests within an urban setting presents a series of challenges related to features unique to the urban environment. Groundwater flow in the near subsurface is affected by underground buildings and utilities, and while recharge from variably distributed anthropogenic sources can augment natural hydrogeological conditions, quantification of the sources of this recharge is problematic (Lerner, 2002). Similarly, the impact of groundwater flow on subsurface structures is also highly variable, and it can negatively affect development projects through infiltration damage and higher-than-anticipated dewatering costs. Despite the importance of urban hydrogeology to construction and development activities, most published work has focused on water-resource issues and large-scale trends (e.g., Howard and Israfilov, 2002) rather than localized impacts of infrastructure on groundwater flow systems, and vice versa. In practice, the impact of groundwater on underground structures is often studied within the framework of a geotechnical investigation, where short-duration rising or falling head tests are often performed in lieu of formal pumping tests for budgetary reasons. The resurgence of urbanism and green building practices in recent decades will contribute to increase infill and new development of multi-level residential and commercial properties that include subsurface structures. Given this trend, the need to understand the engineering and economic pitfalls from poor hydrogeological site characterization becomes critical.

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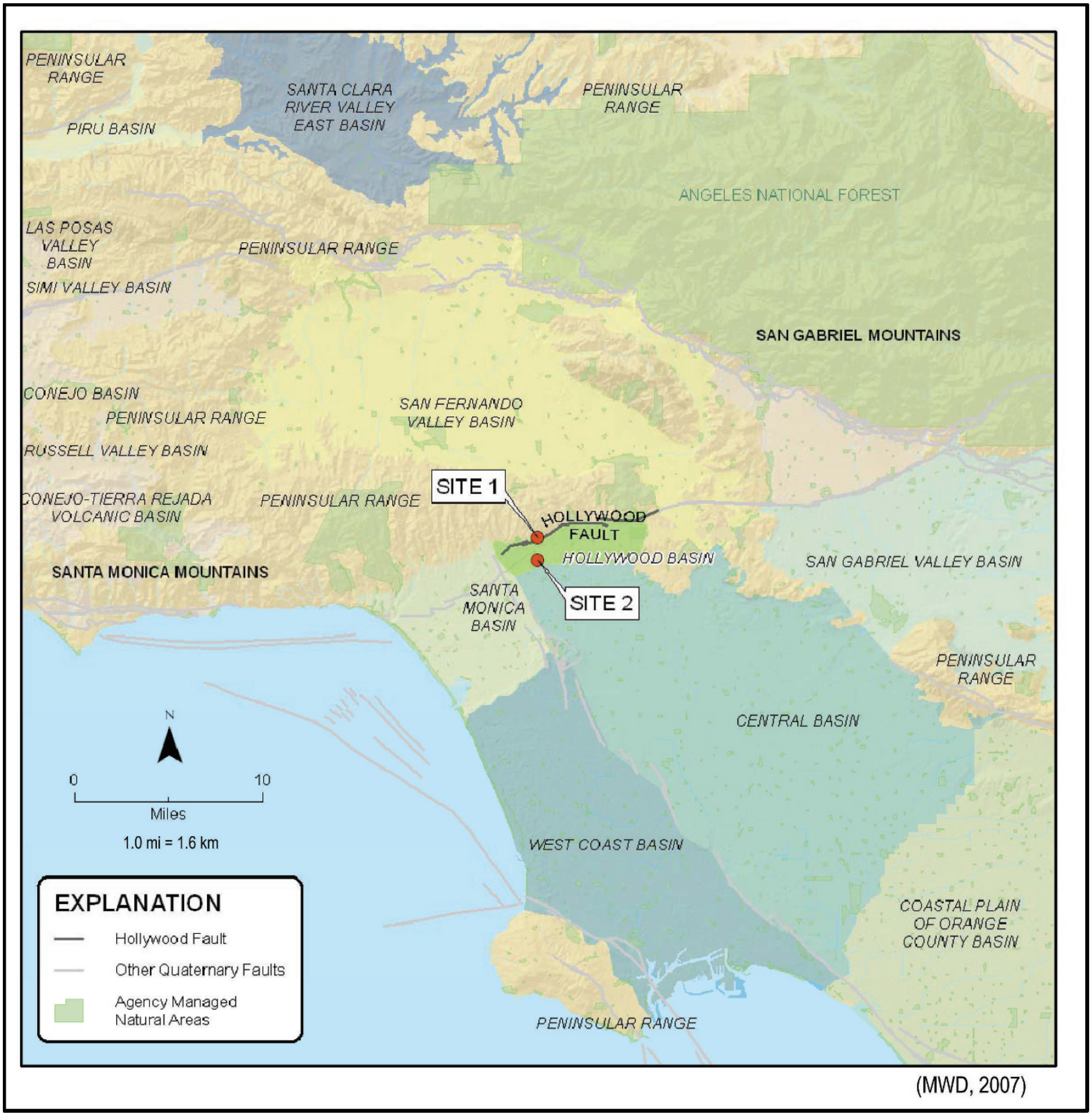

Figure 1. Regional geomorphic map.

Aquifer pumping test results collected within an urban setting can be analyzed via traditional methods, but incorporation of knowledge of the unique features of the local area is critical to making meaningful interpretations. This paper presents aquifer pumping test data and interpretations from two sites located within the Hollywood Basin, California (Figure 1), and contrasts the performance of each series of tests. Aquifer pumping tests were performed at Site 1, located in the City of West Hollywood, and Site 2, located nearby in the City of Los Angeles, to obtain estimates of hydraulic properties in order to develop dewatering requirements for site development. At both sites, developers wished to construct structures with underground parking levels. As such, post-construction dewatering requirements strongly influenced development design and economics.

Table 1. *Hydrogeologic formations of the Hollywood Basin.*

Feature	Description
Aquifers(s)	Alluvium Lakewood Formation (Exposition and Gage Aquifers) San Pedro Formation (Jefferson, Lynwood, Silverado, and Sunnyside Aquifers)
Depth of groundwater basin	Up to 660 ft
Thickness of water-bearing units	Alluvium (up to 60 ft [201 m]) Lakewood (up to 175 ft [53 m]) San Pedro Formation (up to 100 ft [30 m])

The land-use conditions at both sites are similar in that each is located in a developed commercial area. Site 1 is an approximately 3 acre parcel occupied by one- to two-story slab-on-grade buildings with shallow foundations. Development in the vicinity of Site 1 is restricted to shallow structures, and there is currently little to no pumping of groundwater. Site 2 is located approximately 0.7 mi (1.1 km) southeast (down gradient) of Site 1, and it occupies a 1.6 acre (0.7 hectare) parcel that is in the process of being developed as a facility with underground levels. It is located adjacent to a structure with three levels of underground parking extending to about 50 ft below site grade. The majority of structures in the immediate vicinity of Site 2 have one to three underground levels, and recent development includes ongoing local dewatering operations.

Data quality obtained from the aquifer testing at both sites is considered high; accurate, high-resolution data were collected from the pumping well and multiple observation wells, and the response to pumping stresses is clearly defined in both cases. At Site 1, pumping test results were interpreted within the framework of traditional theoretical aquifer models, whereas at Site 2, the influence of building construction on test results required knowledge of site-specific subterranean features to develop viable interpretations. The insight gained from both sites and presented herein should add valuable experience to the hydrogeologist's body of knowledge in the performance and interpretation of pumping test results in developed urban settings.

REGIONAL SETTING

The two sites are located within the Hollywood Basin underlying the northeastern portion of the Coastal Plain of the Los Angeles Basin (California DWR, 2004), as shown in the regional groundwater basin map (Figure 1).

Regional groundwater conditions in the Hollywood Basin are documented in the *Groundwater Assessment Study* published by the Metropolitan Water District of Southern California (MWD, 2007). The sub-basin is bounded to the north by the Santa Monica Mountains and the Hollywood fault, to the east by the Elysian Hills, to the west by the Newport-Inglewood Uplift, and to the south by the La Brea High (an area of shallow bedrock). A summary of the major hydrogeologic formations within the Hollywood Basin is listed in Table 1.

The DWR reports that alluvium covers much of the Hollywood Basin, and aquifer thicknesses range from 5 to 60 ft (2 to 18 m). Groundwater within the alluvium exists under semi-perched unconfined conditions, and limited groundwater is produced from this zone. The majority of potable groundwater is produced from the deep aquifers of the San Pedro Formation and shallower aquifers of the Lakewood Formation. The Gage Aquifer of the Lakewood Formation is the major water-bearing member of the Hollywood Basin (California DWR, 1961).

Site 1—Hydrogeological Characteristics

Site 1 is a triangular-shaped parcel of land of approximately 3 acres (Figure 2). The geological framework at Site 1 was characterized by five geotechnical soil borings and four cone penetrometer (CPT) borings; at each CPT location, a groundwater well was installed to allow for the performance of the aquifer pumping tests.

The maximum depth explored was 125 ft (38 m) below ground surface (bgs). Sediment types encountered in boreholes are consistent with alluvial deposits of gravel, sand, silt, and clay. The hydrogeologic framework for Site 1 is illustrated in Figure 3. The upper alluvial deposits to depths of approximately 35 to 40 ft (11 to 12 m) bgs consist predominantly of fine-grained sediments interlaced with sand stringers. Groundwater appears to be perched within these discontinuous sand deposits. Lateral continuity of coarse sediments occurs at depths below 40 ft (12 m) bgs, where deposits may represent upper units of the Exposition Aquifer. A probable geological boundary occurs near the 40 ft (12 m) bgs level and is coincident with an increase in sediment compaction, as indicated by the increased blow counts observed on the standard penetrometer test records on boring logs

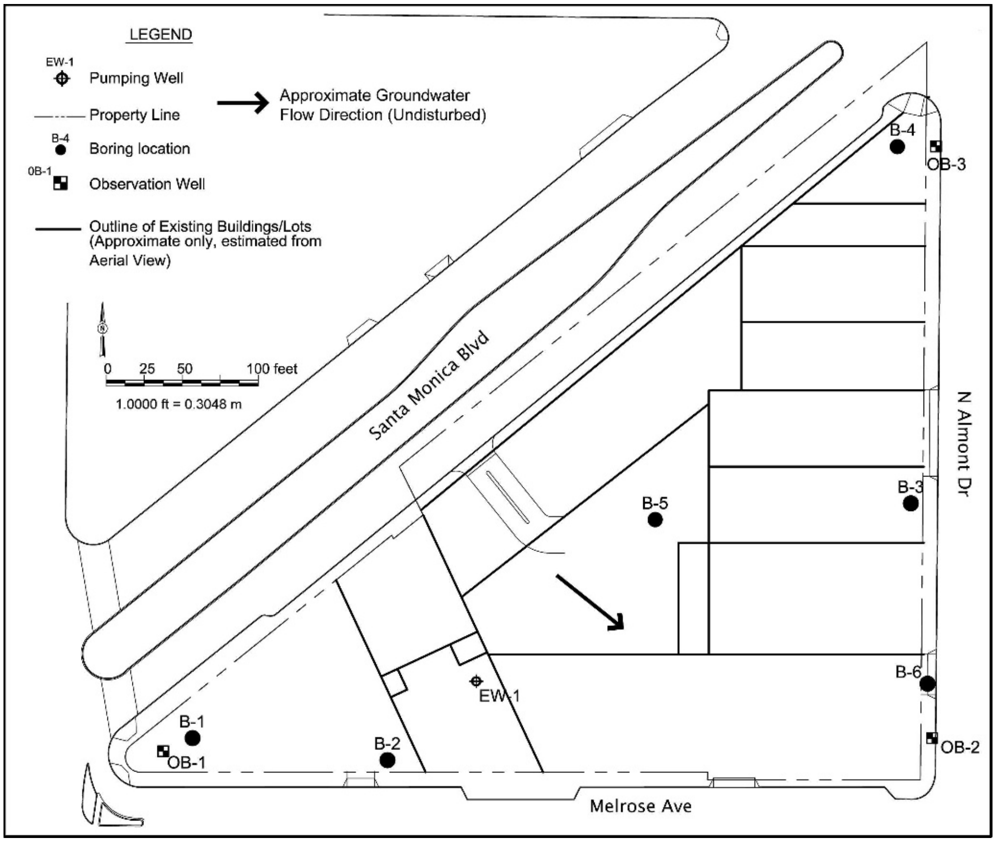

Figure 2. Map of Site 1.

and N60 count (a measure of penetration resistance) derived from CPT logs.

The alluvium and underlying Exposition Aquifer water-bearing units appear to extend to the maximum depths of exploration at 125 ft (38 m) bgs. As indicated in Figure 3, the upper sand unit within the Lakewood Formation extends from about 40 to 70 ft (12 to 21 m) bgs. A continuous layer of fine-grained material exists below the alluvium, extending from about 70 to 80 ft (21 to 24 m) bgs, and it acts as a confining unit. The thicker sand unit below the confining unit likely represents the upper Exposition Aquifer of the Lakewood Formation. At depths below 110 ft (34 m) bgs, sand aquifer and clay units appear to be interstratified.

To address the lack of data regarding the local hydrogeology of Site 1, four wells (one 6-in.-diameter [15 cm] pumping well, EW-1, and three 2-in.-diameter [5 cm] observation wells) were installed to obtain site-specific hydrological data. Depth-to-water measurements were made to determine the site-specific groundwater elevations, gradient, and flow direction. The groundwater gradient was determined to range between 0.002300 and 0.00260 ft/ft (0.00070 to 0.00079 m/m) with flow directed toward the southeast.

Site 2—Hydrogeological Characteristics

The geological framework at Site 2 was characterized by nine geotechnical soil borings and four CPT borings; four wells were installed by the dewatering contractor for the purpose of performing aquifer pumping tests (Figure 4).

The maximum depth explored was 100 ft (30 m) bgs. The hydrogeologic framework for Site 2 is illustrated in Figure 5. As shown in Figure 5, the geologic materials that underlie the site consist of gravel, sand, silt, and clay deposits associated with a fluvial (river-floodplain) depositional environment. Additionally, portions of the site had been previously excavated to aid construction of the adjacent parking structure, consisting of three underground levels south of the site, and the central plant utility building located west of the site (Figure 4). Farther west, there is a main facility building complex that contains dewatering systems. Immediately east of the site, there is a large shopping complex with three underground parking levels, and active dewatering is known to occur there also.

As presented in Figure 5, fine-grained silt and clay sediments predominate throughout much of the site to explored depths. The silt and clay units are

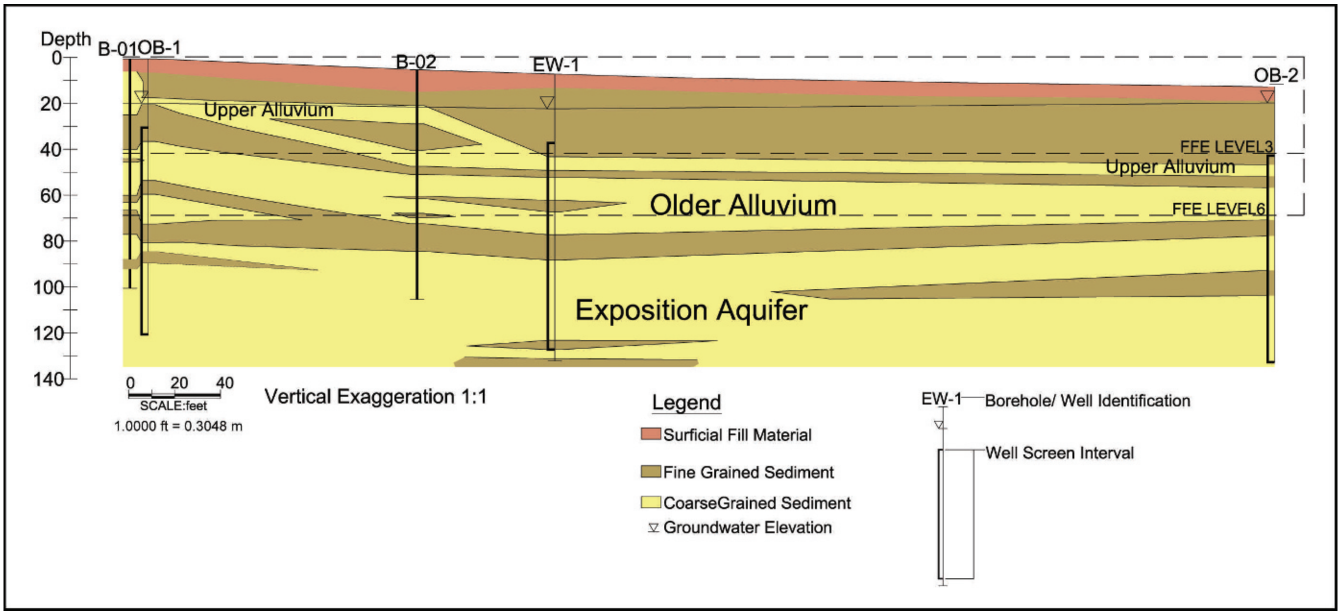

Figure 3. Site 1 hydrogeological framework. EW-1 is the pumping well. Note proposed parking structure depths (dashed lines).

considered low-permeability materials that will not readily produce or transmit groundwater. However, within the depth range of interest (from ground surface to about 50 ft [15 m] bgs) for the proposed subterranean structure, two intervals of coarse-grained sand and gravel units are encountered site-wide, which comprise the main water-bearing aquifer units, and are correlated with the alluvium as shown on Table 1.

The upper aquifer unit occurs at depths of approximately 15 to 20 ft (5 to 6 m) bgs. The thickness of this unit averages approximately 5 ft (2 m) across the site. The lower aquifer occurs between 35 and 40 ft (11 to 12 m) bgs, and it also varies in thickness, with an average thickness of about 8 ft (2 m) across the site. Wells TW-1 through TW-4 were screened across this lower aquifer unit.

Thin, isolated sand stringers exist throughout the upper 50 ft (15 m) of section. Typically, these sand stringers do not have a large areal extent, so water production from them is expected to be limited. Thicker, more contiguous sand and gravel units occur at depths greater than 50 ft (15 m) bgs, but these are not likely to contribute water to the construction project unless connected to the overlying aquifer units by a vertical conduit, such as an improperly abandoned well casing or relic construction artifacts (for example, an abandoned oil well was located on-site).

Construction activities in the vicinity of the central plant and parking structure (Figure 4) had disturbed and modified the geologic materials near these structures. These anthropogenic features include beds of drain rock associated with building foundation base rock, drain rock placed to stabilize the previous excavation due to flooding, compacted fill associated with the backfill of previous excavations, and impermeable barriers associated with building walls and sheet piles. The quantity of drain rock placed at the site during previous excavation efforts is unknown, but drain rock placement was reported anecdotally by the dewatering contractor. Groundwater seepage was observed in the subsurface structures of Site 2, specifically through the face of the lower level of the parking structure, approximately 30 to 40 ft (9 to 12 m) bgs (Figure 5).

In the vicinity of Site 2, local groundwater flow data for the shallow aquifer units were available through the State of California GeoTracker database. Three sites within 0.25 mi (0.40 km) of Site 2 indicate that local groundwater to depths of 50 ft (15 m) bgs flows under a gradient of 0.0010 to 0.0060 ft/ft (0.0003 to 0.0018 m/m) in a southerly direction, with local variations to the southeast and southwest.

AQUIFER TEST METHODS

At both sites, aquifer testing procedures were performed in accordance with ASTM guidelines (ASTM Standard D4043-96 [2000] and D5786-95 [2000]) and consisted of a step drawdown test followed by a continuous constant-rate pumping test. The step drawdown test was performed to establish pumping rates for the continuous constant-rate pumping tests, and to evaluate well efficiency. At Site 1, pumping was performed from well EW-1, and the applied stress to the aquifer system was monitored in observation wells OB-1, OB-2, and OB-3. At Site 2, pumping was

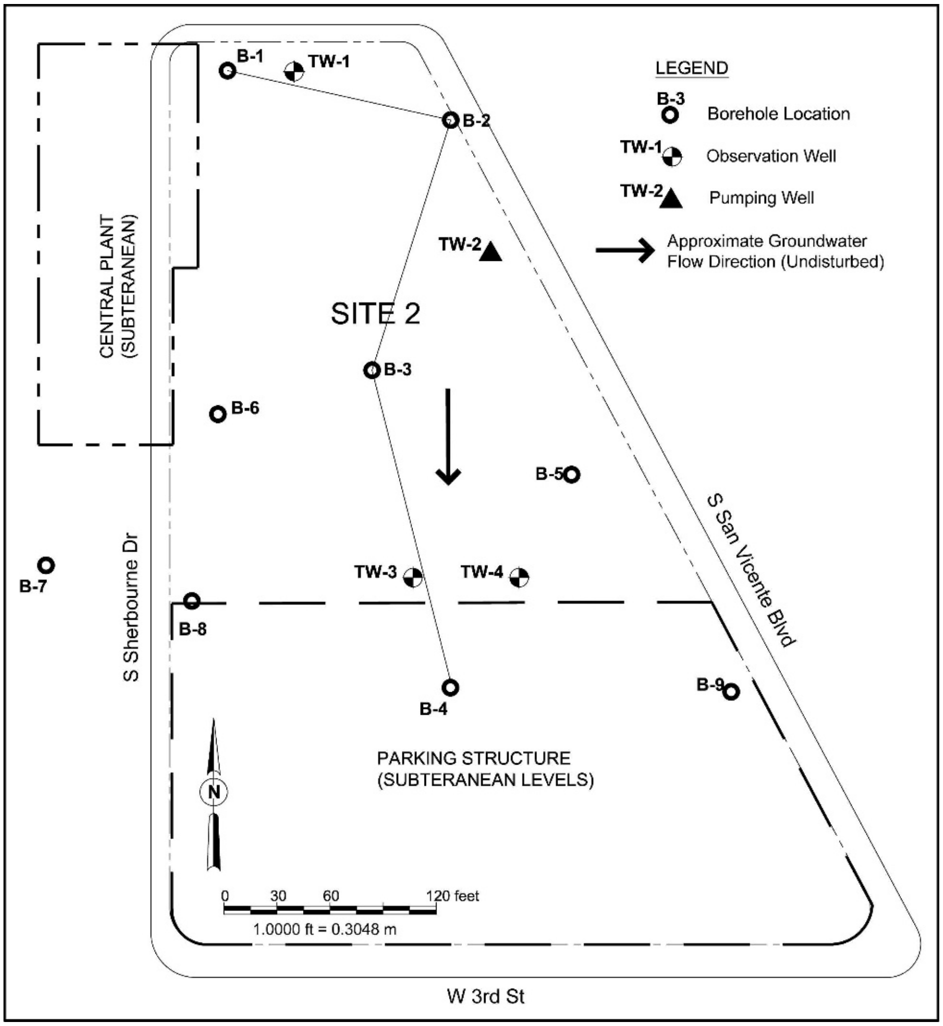

Figure 4. Map of Site 2.

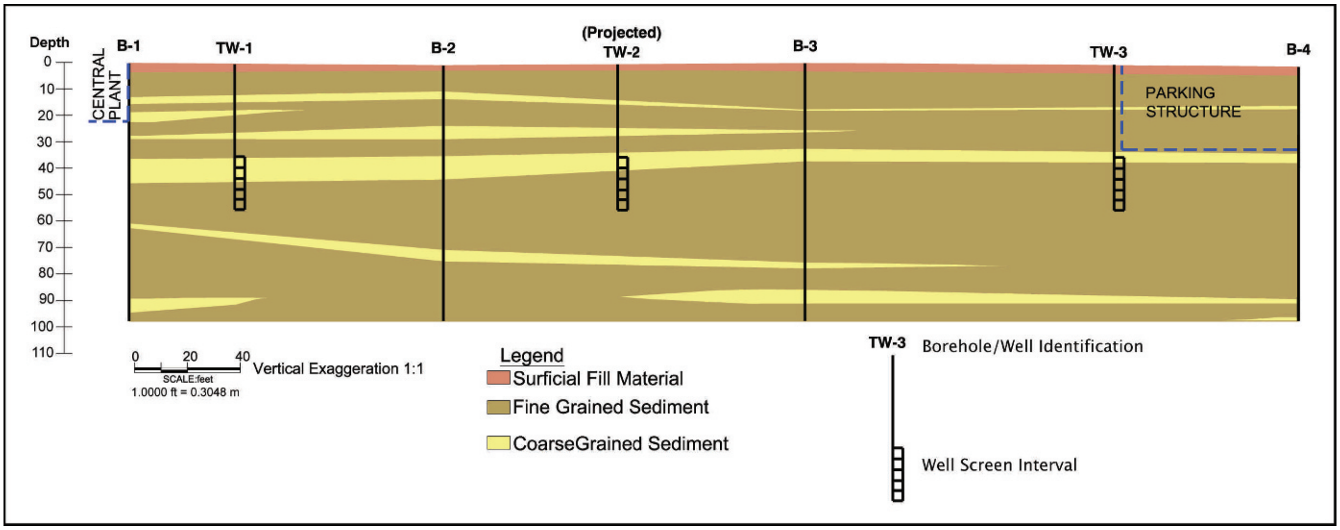

Figure 5. Site 2 hydrogeological framework. TW-2 is the pumping well.

performed from well TW-2, and the applied stress to the aquifer system was monitored in observation wells TW-1, TW-3, and TW-4. At both sites, the quality of the aquifer pump test data is considered good to excellent, in that high-resolution data were collected, where responses to pumping and recovery phases were clearly evident at the pumping and monitoring wells. Drawdown within the pumping and observation wells was measured using pressure transducers and checked manually with an electronic water-level meter. The drawdown data were logged at 5 minute intervals from pumping and observation wells.

PUMPING TEST ANALYSIS

The objective of the pumping test data analysis was to quantify local hydrogeologic parameters such as aquifer transmissivity (T), lateral hydraulic conductivity (K), and aquifer storativity (S) to aid in the design of construction and post-construction dewatering systems at each site. To estimate the hydraulic properties of the subsurface materials, the observed field data (plotted as time-drawdown curves) were matched to theoretical curves derived from mathematical equations that represent specific groundwater-extraction conditions within an ideal aquifer. Time-drawdown plots were created from each of the pumping and observation wells used in the constant-rate aquifer pumping test. In accordance with ASTM guidelines (ASTM Standard D6034–96 [2004]), data reduction and analysis were achieved using the commercial AQTESOLV® software package and evaluated with the aid of professional judgment. Aquifer properties were obtained by curve matching of time-drawdown plots using the best fit from either the Theis solution (1935) or Hantush-Jacob solution for leaky aquifers without aquifer storage (Hantush, 1960).

Although aquifers rarely exhibit idealized conditions, we conceptualized the geology at both sites as a system of coarse-grained aquifer units confined by leaky fine-grained units that are infinite in areal extent. The hydraulic parameters derived from curve fitting of theoretical equations to field data incorporate the limitations inherent in the theoretical assumptions due to well construction/efficiency and aquifer variability. Where an acceptable fit between theoretical and observed drawdown may exist, actual hydraulic properties of the aquifer likely vary through time and space. Nevertheless, we use our site conceptual model to constrain and guide our subsequent analyses.

Site 1—Pumping Test Procedures and Results

The step drawdown test performed at Site 1 consisted of stressing well EW-1 at four pumping rates (steps): step 1 at 31 gallons per minute (gpm) (169 m^3/d) for 59 minutes, step 2 at 42 gpm (229 m^3/d) for 62 minutes, step 3 at 55 gpm (300 m^3/d) for 87 minutes, and step 4 at 65 gpm (354 m^3/d) for 73 minutes. The time-drawdown curve is presented in Figure 6. The sustainability of applied pumping rates indicates a relatively productive aquifer zone(s) beneath the site. The aquifer system was able to sustain relatively high pumping rates, in excess of 50 gpm (272 m^3/d) without drawing down water levels in well EW-1 more than 12 ft (4 m) below the height of the static water column. The step-drawdown test was terminated due to sand and silt being taken up by the pump and entering the discharge water filter system, not because of excessive drawdown in the pumping well. As shown in Figure 6, water levels were observed to rebound quickly when pumping was interrupted or ceased.

The corresponding curve matches for the step drawdown test are shown in Figure 7. Crosses represent the measured drawdown in pumping well EW-1, and squares represent the observed drawdown in well OB-1. Time-drawdown curves for the observation wells OB-2 and OB-3 exhibited similar response, so the graph in Figure 7 is representative of the site-wide response to pumping. The immediate and parallel drawdown response observed in well OB-1 to pumping well EW-1 suggests that the aquifer is highly transmissive. An analysis of well loss parameters using the Jacob-Rorabaugh method (Rorabaugh, 1953) returned a well loss coefficient, C, of 0.7 min^2/ft^5 (266.2 min^2/m^5), and non-linear exponent, P, of 0, indicating that the pumping well was in good communication with the surrounding aquifer units, and that well damage or skin effects were minimal. Based on this analysis, the uptake of sand by the pump was most likely due to transport by high groundwater velocities in the vicinity of the well screen during the final step of the pumping test. In the case of a full-scale dewatering system at Site 1, selection of larger-diameter well bores for dewatering wells may mitigate this process.

Following the completion of the step drawdown test, the aquifer was allowed to recover for 15 hours in order for groundwater elevations to return to static pre-stressed conditions. The results from the step drawdown test indicated that a sustainable pumping rate of 55 gpm (300 m^3/d) was achievable for the constant-rate pumping test. While pumping at 65 gpm did not cause excessive drawdown (only 12 ft [4 m] within the well), the induced flow at this elevated pumping rate was likely the cause of turbulent, high-velocity flow in the vicinity of the well screen, resulting in sediment being drawn into the pump and filtration system. As such, 52 gpm (283 m^3/d) was

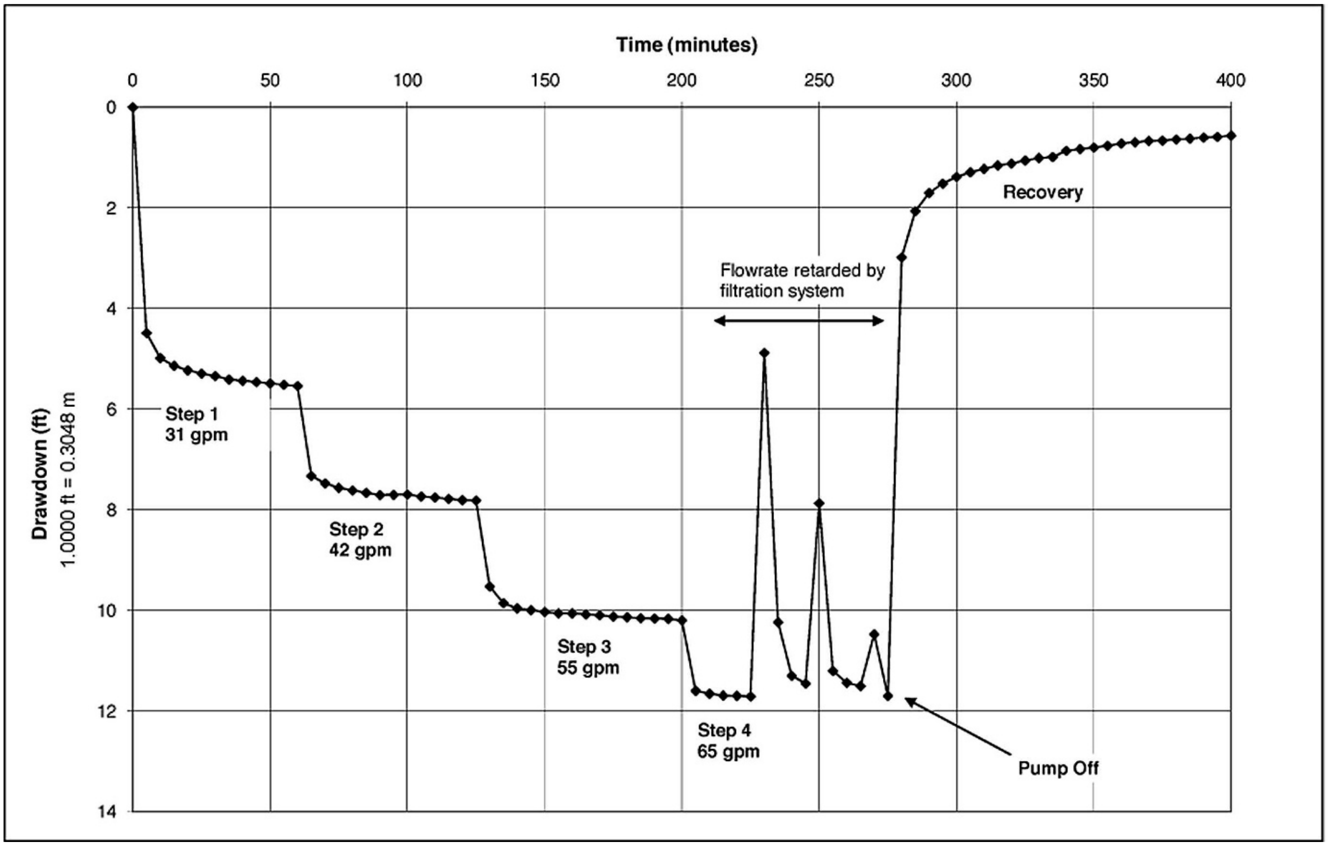

Figure 6. Site 1 step drawdown test observations at pumping well.

chosen as the conservative pumping rate for the constant-rate test.

The constant-rate pumping test involved extracting groundwater from well EW-1 while observing the effects of the applied stress to the surrounding aquifer system in both pumping and observation wells. The duration of the pumping phase was 1928 minutes, followed by recording the recovery phase for 1560 minutes. The constant-rate pumping test time-drawdown curve for observation well OB-1 is presented in Figure 8.

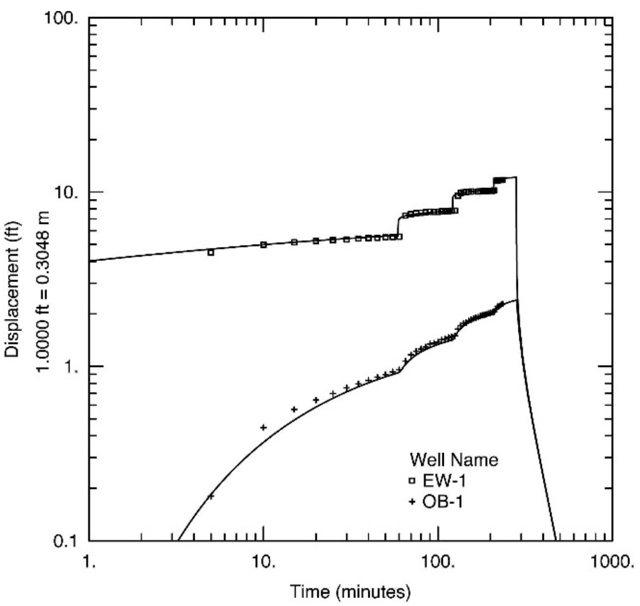

Figure 7. Site 1 step drawdown test curve match at observation well OB-1 and pumping well.

The theoretical curves from the Hantush leaky aquifer solution provide a better match to observed data than the Theis solution, which is consistent with our conceptual model of a leaky confined aquifer system. This result was anticipated due to the interbedded nature of the high- and low-permeability units, as well as the screen interval of wells that are open to multiple aquifer zones. The Hantush leaky aquifer solution curve match for observation well OB-1 is presented in Figure 9. The hydraulic properties determined from the step drawdown and continuous rate pumping tests are summarized in Table 2.

Curve matching for both the step drawdown and constant-rate tests via the Hantush solution for a confined aquifer returned a relatively narrow range of K values between 8.7 and 9.3 ft/d (2.7 and 2.8 m/d). The storativity (S) value for the pumping well EW-1 ranged from 3.4 to 6.1 $\times$ 10^{-4}, while the S values for the observation wells in the constant-rate pumping test

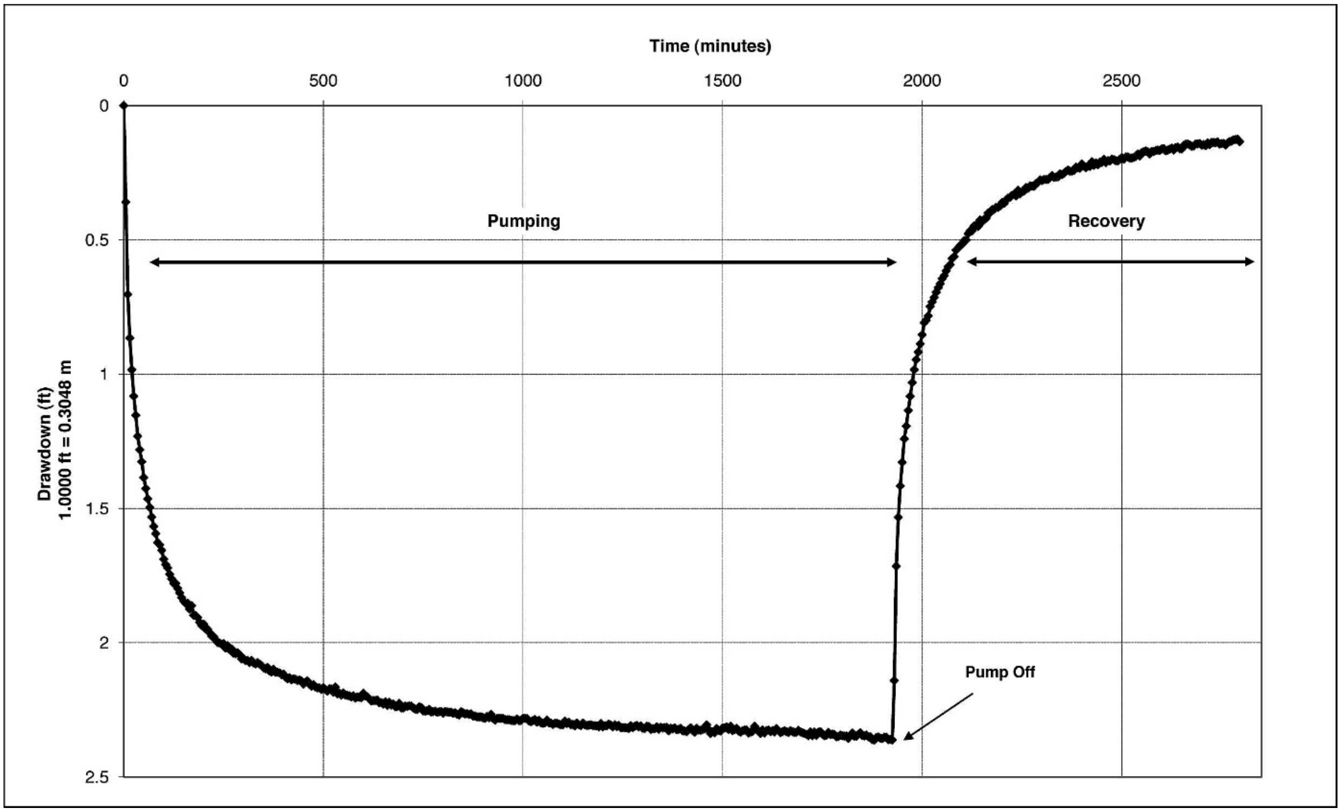

Figure 8. Site 1 drawdown observations at observation well OB-1 during the constant-rate pumping test.

ranged from 10^{-6} to 10^{-7}. The S values are indicative of a confined aquifer system (Lohman, 1972). The lower storativity values determined from observation well data suggest that the applied stress was transmitted through thicker aquifer sequences of the compacted Exposition Aquifer, while the higher S value determined from pumping well EW-1 is indicative of contribution from the less-consolidated upper alluvium and older alluvium water-bearing zones. This conclusion is based on the nature of S, which acts inversely proportional to the bulk modulus (or compressibility) of aquifer materials (Lohman, 1972). The leakance factor (r/B) for each well EW-1, OB-1, OB-2, and OB-3 was estimated at 0.02, 0.003, 0.001, and 0.001, respectively. The leakance observed in well EW-1 was an order of magnitude higher than that of observation wells and supportive of a contribution derived from all encountered water-bearing zones. Collectively, the aquifer pumping test results support the site conceptual model, where water-bearing zones exist as highly transmissive confined aquifers that exhibit some degree of leakance.

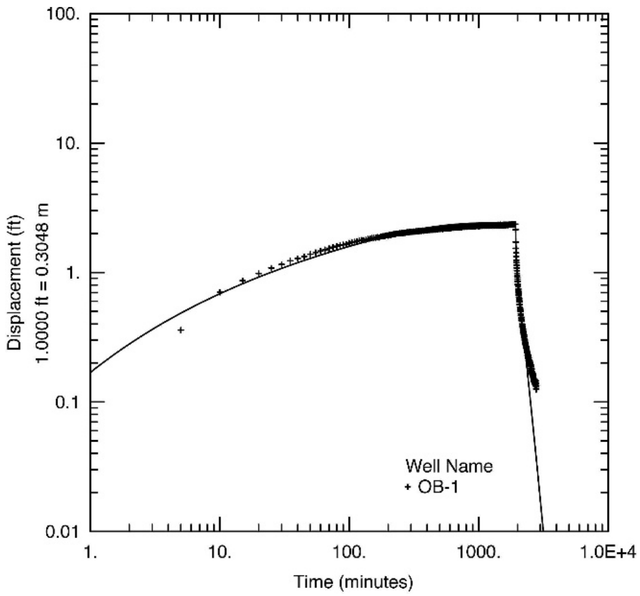

Figure 9. Site 1 constant-rate pumping test curve match at observation well OB-1.

Site 2—Pumping Test Procedures and Results

Previous subterranean construction had occurred at Site 2, including a parking structure adjacent to wells TW-3 and TW-4, and the central plant west of well TW-1. As such, well TW-2 was selected for use as the pumping well for aquifer testing at this site. The

Table 2. *Summary of Site 1 step drawdown and constant-rate pumping test results. A cumulative aquifer thickness of 70 ft (21 m) was used throughout.*

Well	Step	Constant-Rate Pump Test			
	EW-1	EW-1	OB-1	OB-2	OB-3
T (ft^2/d)[1]	640	653	652	609	653
K (ft/d)[2]	9.14	9.33	9.31	8.70	9.33
S [–]	3.40×10^{-4}	6.10×10^{-4}	7.30×10^{-7}	7.69×10^{-7}	1.25×10^{-7}

[1]1.0000 ft^2/d = 0.0929 m^2/d.
[2]1.0000 ft/d = 0.3048 m/d.

location of TW-2 provided the greatest potential to apply pumping stress to the aquifer system, while minimizing the interference from anthropogenic features.

The step drawdown test performed at Site 2 consisted of stressing well TW-2 in four steps: step 1 at 5.8 gpm (31.6 m^3/d) for 30 minutes, step 2 at 8.8 gpm (48.0 m^3/d) for 90 minutes, step 3 at 14.0 gpm (76.3 m^3/d) for 90 minutes, and step 4 at 17.8 gpm (97 m^3/d) for 60 minutes. At 45 minutes into step 4, the water level in the well began to steadily increase for a period of 15 minutes, at which time the pump was turned off. The increase in water level did not appear to be linked to changes in pumping rate or equipment error and was attributed to the pumping stress impinging a subterranean recharge zone. However, the nature of this apparent transient gain in groundwater elevation may warrant further investigation. Common causes of transient well response such as nearby river-stage fluctuations are not applicable to the test at Site 2, so future evaluation of poro-elastic effects related to deformation in this data set may be worthwhile. The step-drawdown and recovery curves are presented in Figure 10, and curve match via the Theis solution is presented in Figure 11.

As indicated in the previous section, each well (including observation wells and the pumping well) was screened within and fully penetrated the lower confined sand zone at a depth of 35 to 40 ft (11 to 12 m) bgs (with an average thickness of approximately 8 ft [2 m]). As such, analytical methods associated with the confined aquifer are suitable for data analysis on the completed aquifer tests.

Analysis of the step-drawdown test indicates a local T in the vicinity of well TW-2 of 354 ft^2/d (39 m^2/d). Assuming an aquifer thickness of 8 ft (2 m), then a local K of 44 ft/d (13 m/d) is estimated. The S value of 1.98×10^{-6} is indicative of a confined aquifer system. An analysis of well loss parameters using the Jacob-Rorabaugh method (Rorabaugh, 1953) returned a well loss coefficient, C, of 0.25 min^2/ft^5 (95.06 min^2/m^5), and non-linear exponent, P, of 3, which indicate non-ideal pumping conditions and that potential well inefficiency or skin effects may have impacted flow into the pumping well.

Following the completion of the step drawdown test, the aquifer was allowed to recover for 24 hours so that groundwater elevations could to return to static pre-stressed conditions. The results from the step drawdown test showed that a sustainable pumping rate of 14.0 gpm (76.3 m^3/d) was achievable; however, in order to be conservative, the constant-rate pumping test was performed at a reduced discharge rate of 13.2 gpm (71.9 m^3/d).

The constant-rate pumping test involved extracting groundwater from well TW-2 while observing the effects of the applied stress to the surrounding aquifer system in the pumping and observation wells. The duration of the pumping phase was conducted for 1400 minutes, followed by recording the recovery phase for 720 minutes. The time-drawdown curve for the constant-rate pumping test for well TW-2 is presented in Figure 12.

Constant-rate pumping well drawdown and curve matching for well TW-1 are presented in Figures 13 and 14, respectively. Similar drawdown and curve matching was observed for wells TW-3 and TW-4. As with Site 1, the theoretical curves for the Hantush leaky aquifer solution provided a better fit to observed data than the Theis solution. Again, this result was anticipated due to the interbedded geology as well as the anthropogenic features unique to Site 2.

The aquifer step drawdown and continuous rate pumping test results for Site 2 are summarized in Table 3.

A notable feature observed in the pumping phase of the time-drawdown curves for wells TW-1, TW-3, and TW-4 is the flattening to negative drawdown observed at late times in these wells. This curve flattening is an indication of secondary recharge to the wells via an external source. Within the context of natural groundwater systems, the external source is typically ascribed to leakage from a secondary reservoir such as a river or dam, but in this case, it is likely associated with storage within drain rock placed beneath nearby structures during previous construction activities in the vicinity of the site.

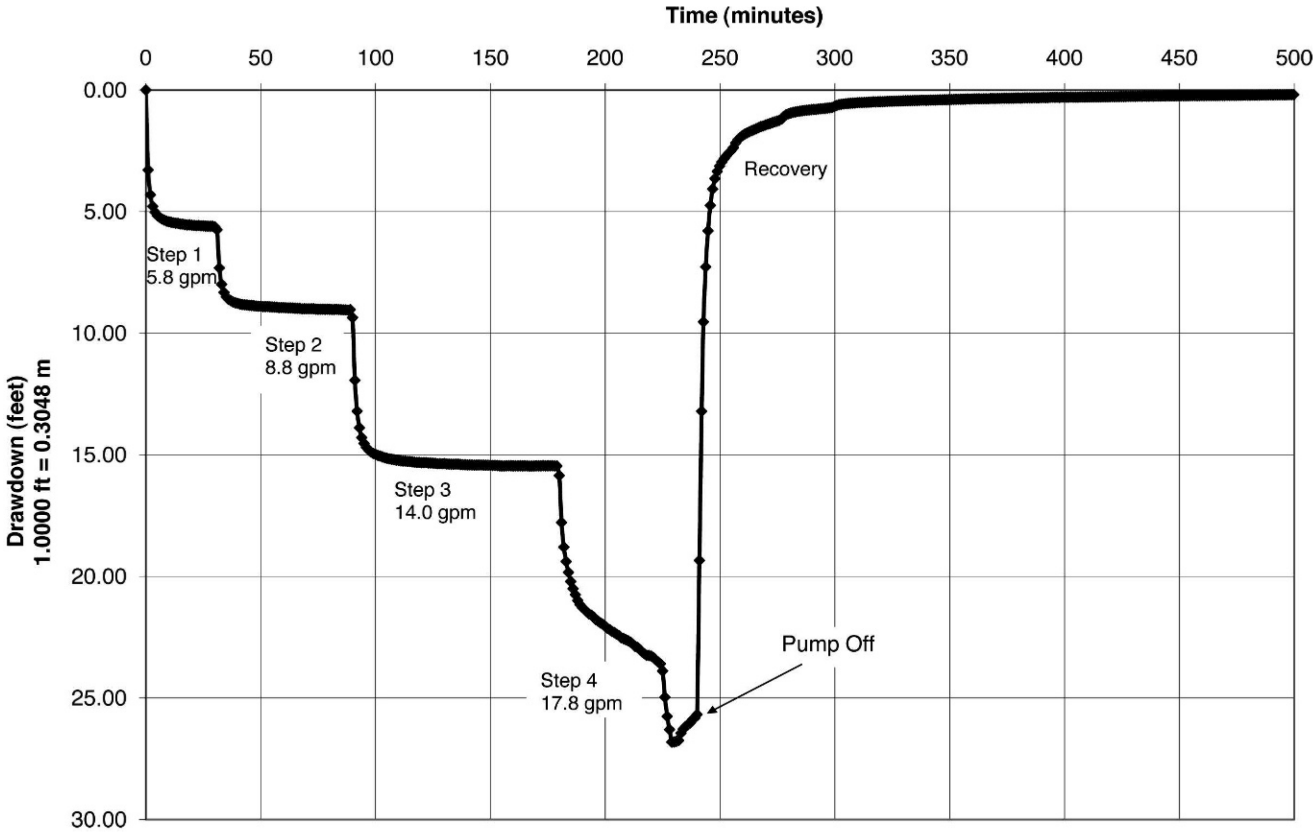

Figure 10. Site 2 step drawdown test observations at pumping well.

The storativity values are consistent with values obtained from confined aquifers. Slightly higher storativity values were obtained near the drain rock wells TW-3 and TW-4. This is likely due to the leaky connection to the aquifer units provided by the drain rocks around these wells. Although the assumed aquifer thickness is relatively thin at only 8 ft (2 m), the aquifer transmissivity estimates from the pumping tests indicate that the aquifer system will be relatively productive.

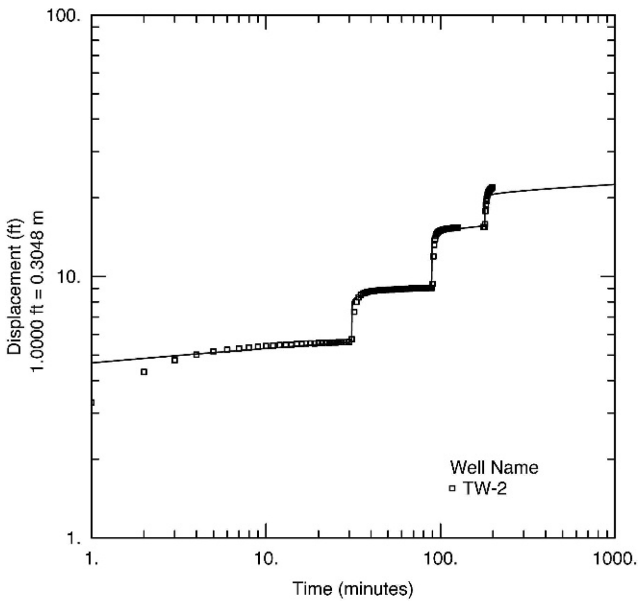

Figure 11. Site 2 step drawdown test curve match at pumping well.

As mentioned already, the shape of the drawdown curves for observation wells TW-1, TW-3, and TW-4 indicates that secondary recharge sources are likely located to the south and west of the site, respectively. Using the Hantush-Jacob solution for leaky aquifers, the transmissivity values ranged from 820 ft^2/d (76 m^2/d) to 940 ft^2/d (87 m^2/d) for the lower aquifer. Hydraulic conductivity values ranged from 103 ft/d (31 m/d) to 117 ft/d (36 m/d) and are consistent with aquifer material consisting predominantly of a mixture of sand and/or gravel grain sizes (Freeze and Cherry, 1979) and known Site 2 geology.

The leakage factor (r/B) for well TW-1 was 0.1, and for wells TW-3 and TW-4, the leakage factor was 0.2. However, when compared to Site 1, in addition to the leakance from low-permeability silt and silty clay material, the majority of observed leakance at Site 2 is likely due to the method of well construction and the presence of drain rock surrounding well casings. As the observation wells at Site 2 had been installed by a previous contractor, well-construction details at

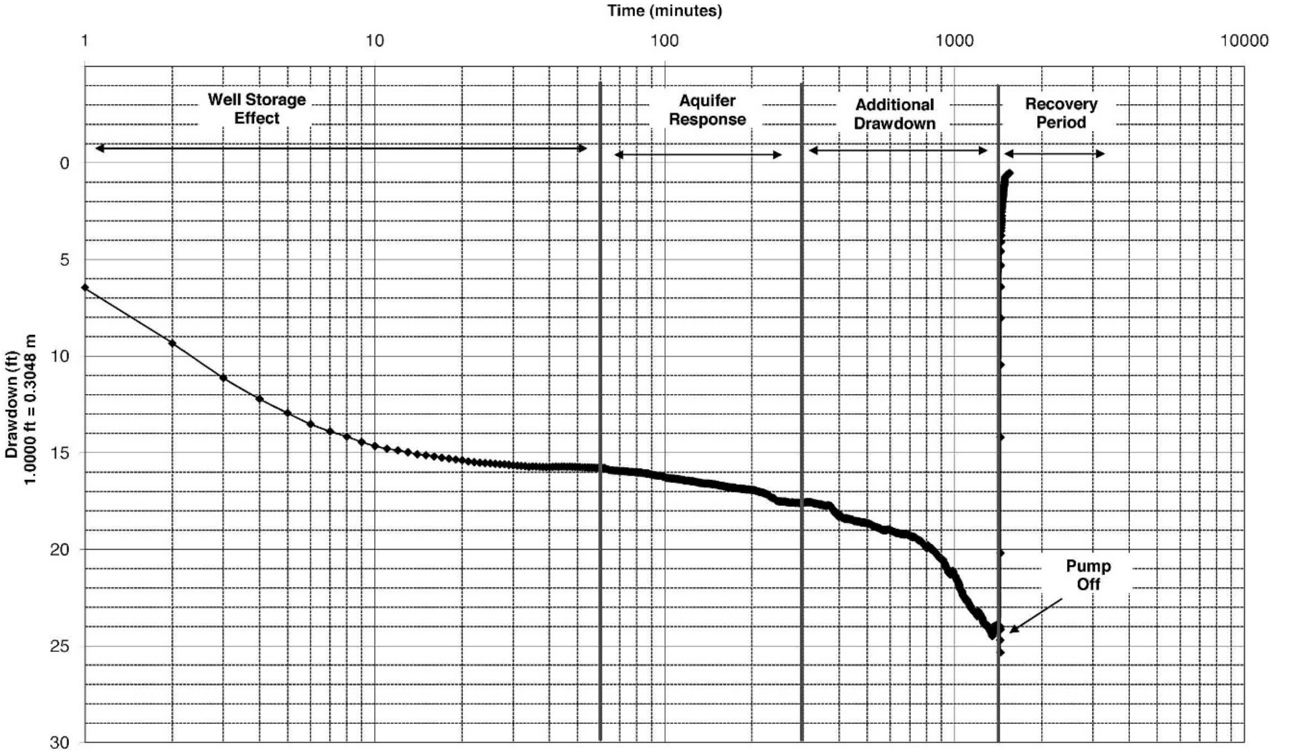

Figure 12. Site 2 drawdown observations at pumping well TW-2 during the constant-rate pumping test. Note the drawdown curve shows periods of well-storage effects, aquifer response, and additional drawdown due to secondary recharge.

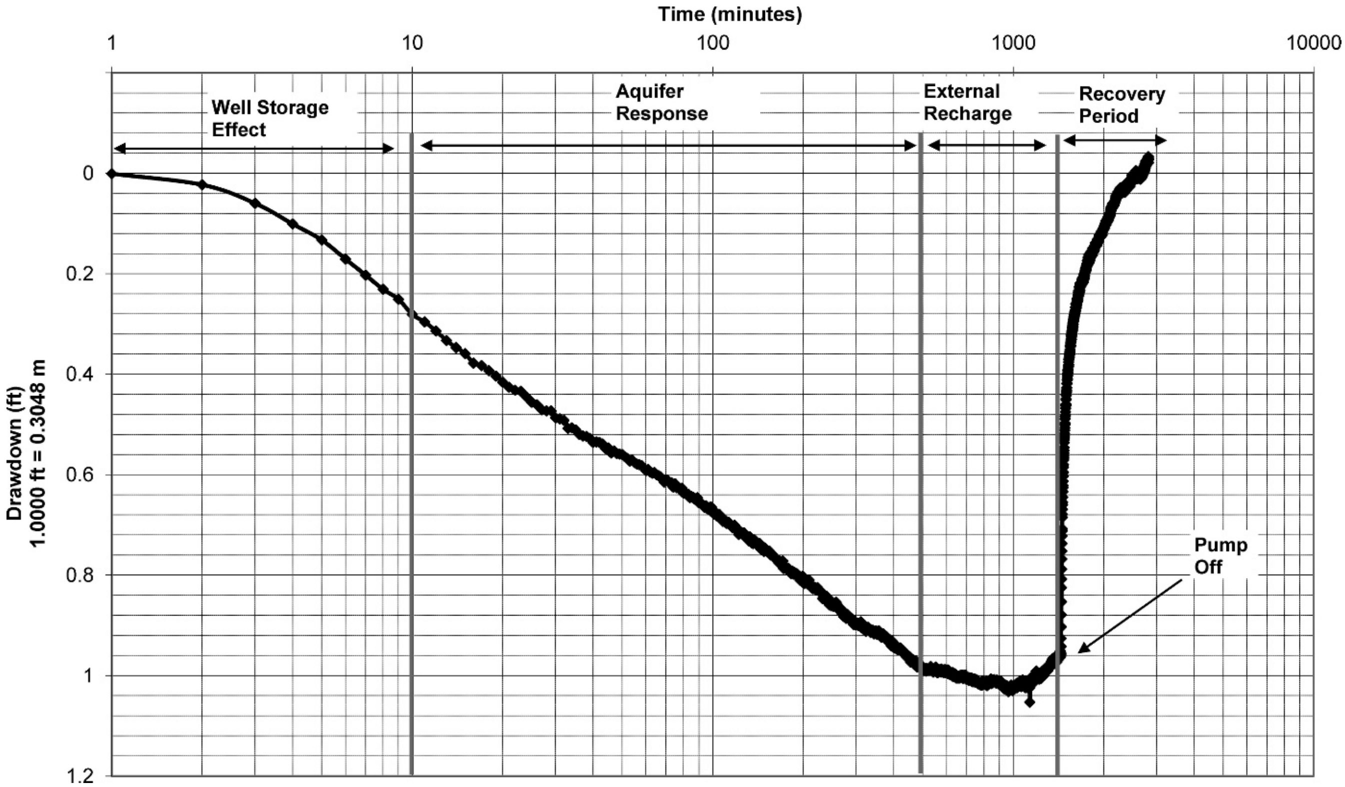

Figure 13. Site 2 drawdown observations at observation well TW-1 during the constant-rate pumping test.

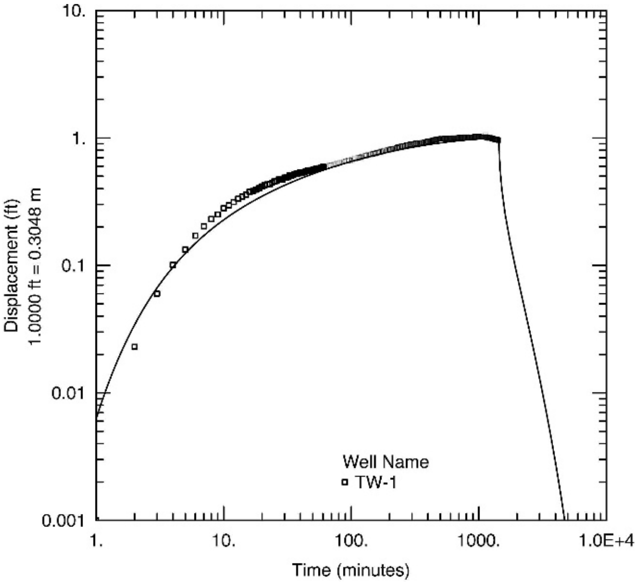

Figure 14. Site 2 constant-rate pumping test curve match at observation well TW-1. TW-2 is the pumping well.

these locations are poorly understood. The large degree of subsurface compaction and disturbance due to underground construction at Site 2 compared to Site 1 may also contribute to greater leakance there.

DISCUSSION AND CONCLUSIONS

The results from aquifer testing at two sites within interbedded alluvial settings and at relatively close geographical proximity to each other show differing responses to an applied stress.

At Site 1, the response to pumping site-wide was relatively uniform and agreed with traditional analytical time-drawdown models. However, use of well-known analytical scenarios was not sufficient to explain discrepancies in the time-drawdown response from wells at Site 2; here, the data show clear evidence of secondary recharge. Along with an understanding of local hydrogeology, knowledge of the site development history was critical to inferring the source of this observed secondary recharge as occurring in the vicinity of TW-3. The base rock and/or drain rock placed during the construction of surrounding buildings act as a reservoir for this additional observed recharge capacity. Other anthropogenic sources such as leaky pipes or sewer lines can be ruled out as sources of secondary recharge because these are generally limited to the near surface, and would fall within the upper confining unit beneath Site 2 (Figure 5).

Interpretations from Sites 1 and 2 demonstrate significant differences in subsurface conditions at two sites within the same groundwater basin and urban agglomeration due to highly localized subsurface features. K values at Site 2 are almost an order of magnitude greater than at Site 1. Although both sites contained interbedded sand and fine-grained materials, the elevated K values at Site 2 can be attributed to the relatively high permeability and uniformity of drain rock placed near the observation wells. Leakage factors at Site 2 were also approximately two orders of magnitude greater than at Site 1, indicating varying aquifer–confining unit interactions, as well as the level of subsurface disturbance at each site.

The insight gained through aquifer testing of the local hydrogeology at each site was also instrumental to construction and development planning in these areas. At Site 1, six subterranean parking levels were initially envisioned. Had this come to fruition, the result would have been building foundation encroachment into the older alluvium, with construction dewatering wells penetrating into the upper portion of the Exposition Aquifer. Such a construction scenario would have required construction of a dewatering system with capacity in excess of 1,000,000 gal/d for the 3.0 acre (1.2 hectare) site, with associated taxing of municipal storm-water capacity. The economic viability of such a development was therefore questionable. In light of aquifer test results, development plans were revised to include only three subterranean levels, which did not require excavation into the Exposition Aquifer and thereby significantly reduced construction dewatering and post-construction drainage requirements.

At Site 2, groundwater seepage through the north face of the parking structure was occurring at the

Table 3. *Step drawdown and continuous rate pump test results for Site 2. A cumulative aquifer thickness of 8 ft was used throughout. Step drawdown test curve matching was found using Theis solution. Constant-rate pump test curve matching was found using Hantush solution.*

	Step	Constant-Rate Pump Test			
Well	TW-2	TW-2	TW-1	TW-3	TW-4
T (ft^2/d)[1]	346	998	940	830	820
K (ft/d)[2]	43.2	125	117	104	103
S [–]	2×10^{-6}	0.0006	0.0002	0.0006	0.0007

[1]1.0000 $ft^2/d = 0.0929\ m^2/d$.
[2]1.0000 ft/d = 0.3048 m/d.

structure's deepest level. To alleviate this problem, a critical factor in the proposed development at Site 2 was to recognize all sources of recharge to groundwater. The aquifer pumping test was successful in accounting for secondary sources of recharge, and these data were used to design the permanent dewatering system for the new building. Prior to the performance of the pumping test, a dewatering contractor with strong local knowledge recommended that a stabilized dewatering rate of 200 gpm (1,090 m^3/d) would be sufficient. In light of the results obtained from aquifer testing, it was determined that a dewatering system with a capacity of 450 gpm (2,453 m^3/d) would be required to dewater the 1.6 acre (0.7 hectare) site. In this instance, rigorous field and analytical methods were critical to developing the correct dewatering scheme, contrary to initial rule-of-thumb estimates.

These two examples emphasize the importance of performing aquifer testing in an urban setting. Unlike rural and undeveloped areas, where excess capacity from water wells may be throttled down or simply discharged to a nearby drain or canal, retrofitting a constructed building to account for excess groundwater flow is an expensive and challenging undertaking. Results from Site 1 show that deep building excavation into highly permeable regional aquifer units should be avoided. While we possess the engineering capability to dewater such a construction, costs to the developer, as well as future risk and stress on local resources (limited municipal discharge limits) combine to restrict the viability of such a project. Work performed at Site 2 demonstrates that lack of site-specific information can lead to underestimation of dewatering costs, leading to structural damage and associated unforeseen costs in the future. As urban areas continue to be in-filled and built-up vertically, hydrogeological investigation is a critical path item for development projects. Aquifer testing and analytical methods such as those presented here must be taken into consideration during the planning stage of development.

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