

Pumping Test Program for the Central Artery/Tunnel Project in Downtown Boston

A well conceived and implemented pumping test program for a large-scale excavation project offered greater protection to structures adjacent to the project area.

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Six full-scale pumping tests were conducted to obtain field data on the permeabilities of the soil and rock strata likely to require groundwater control during Interstate 93 (I-93) highway tunnel and ventilation building construction for the proposed Central Artery in Area Geotechnical Consultant (AGC) Area 04 (Central Area) of the Central Artery/Tunnel (CA/T) Project. The pumping tests were supplemented by extensive borehole permeability testing in the soil and rock during the AGC phase geotechnical investigations.

The results of the six pumping tests performed show that the rock is frequently more permeable than the overlying soils. Four of the tests were conducted in bedrock, with three tests indicating permeabilities in the range of 0.5 to 2×10^{-3} cm/sec and the fourth test indicating a permeability of about 5×10^{-5} cm/sec. These results are in the same range as indicated by the borehole permeability tests in rock. One pumping test was performed in glacial till and indicated a permeability of about 5×10^{-5} cm/sec, which is near the middle of the range measured by borehole permeability tests in glacial till. One pumping test was performed in glacial till and sand and gravel deposits and indicated a permeability of about 5×10^{-4} cm/sec. The radii of influence estimated from the four pumping tests in rock ranged from 350 to 2,200 feet for drawdowns in the pumping test wells that ranged from 42 to 81 feet.

Groundwater control in the deep excavations for the Central Area is expected to consist primarily of relieving piezometric pressure in the rock and glacial soils below the excavation. The pumping tests provided direct indications that during pressure relief operations high

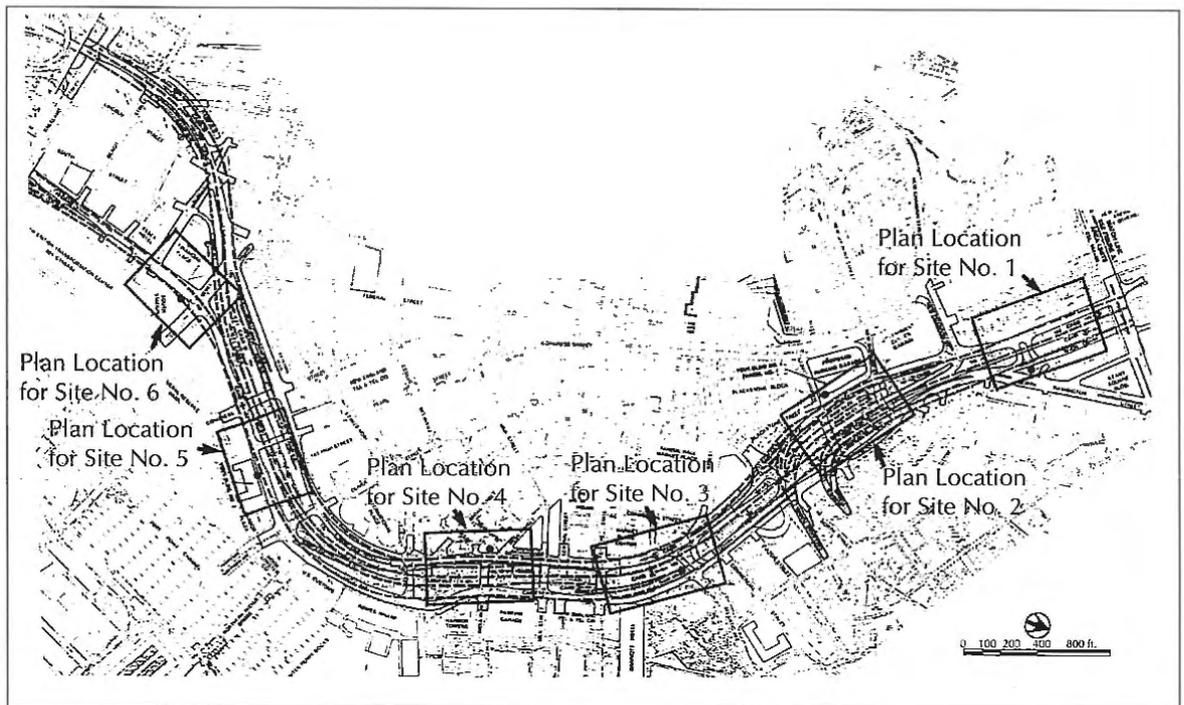


FIGURE 1. Location of the pumping test sites.

pumping rates and large zones of influence should be anticipated. Also, the borehole permeability tests indicated that large zones of permeable rock may be present below the proposed bottoms of the excavation support walls. The presence of such permeable zones could result in substantial drawdowns outside the excavation walls and, therefore, may necessitate the use of grout curtains and/or recharge wells to control drawdowns outside the excavation walls.

Introduction

The pumping test program was conducted in conjunction with a geotechnical investigation that was performed in the Central Area to determine the subsurface conditions and engineering properties of the subsoils and rock for use in the design of the proposed depressed Central Artery. The six different sites were selected to investigate a range of soil and rock conditions that are likely to require groundwater control during I-93 highway tunnel and ventilation building construction in the Central Area. Four of these tests (Tests 2, 3, 5 and 6) were performed in bedrock. Pumping Test 1 was performed in glacial till and a sand and

gravel stratum. Pumping Test 4 was performed in glacial till. The results of each test reflect the specific site and test stratum conditions within the zone of influence of the particular test location.

Figure 1 presents a key plan showing the locations of the pumping test sites in the Central Area. The installations at each of the pumping test sites consisted of a pumping test well, piezometers and observation wells. Installations of test wells and instrumentation at the pumping test sites were performed between July 8 and October 15, 1991. The pumping tests were conducted between September 3 and November 6, 1991.

Project Background

The CA/T Project in the Central Area involves the construction of a widened underground I-93, most of it to be constructed beneath the existing elevated Central Artery. The Central Area project site is located along the existing Central Artery from just north of Causeway Street to just south of Kneeland Street in downtown Boston. South of Congress Street the proposed artery alignment bifurcates into the northbound tunnel that will follow Atlantic

Avenue from Kneeland Street to Congress Street and the southbound tunnel that will follow the existing Dewey Square Tunnel. The proposed artery alignment passes over the Massachusetts Bay Transportation Authority (MBTA) Blue Line subway tunnel at State Street and underneath the MBTA Red Line station and tunnel at South Station on Summer Street. The project in the Central Area also includes two ventilation buildings (Nos. 3 and 4), with the former located on Atlantic Avenue adjacent to the Russia Wharf Building on Congress Street and the latter located at Blackstone and New Chardon streets adjacent to the Haymarket area.

The topography of the site is fairly level, with the ground surface elevations typically ranging from el. 108 to 116. Areas fronting the site are occupied by typical urban buildings including low- to high-rise residential, commercial and office structures, hotels, retail establishments and parking lots. Many of the structures adjacent to the site are relatively old brick buildings. Most of these buildings are concentrated in the northern part of the Central Area. The older structures in the Central Area are typically founded on granite block footings and timber piles, while foundations for the relatively new structures range from spread footings and mat foundations, to caissons and piles. Most of these foundations are susceptible to being affected by drawdowns outside the slurry walls. Timber piles are susceptible to dry-rotting when subjected to cycles of drying and wetting; while both shallow and deep foundations can experience settlements caused by the consolidation of the clay layer due to drainage from the base of the clay layer.

The process of depressing the Central Artery in the Central Area involves a major cut-and-cover construction effort. The depths of excavations required for the tunnels range from 40 to 110 feet. The widths of excavations range from about 55 to 235 feet. The depths of excavations for the ventilation buildings range from about 90 to 140 feet. Excavation support is to be provided generally by soldier pile and tremie concrete (SPTC) walls constructed using slurry techniques, although in many cases conventional slurry walls with reinforcement cages may be used. Excavation will be performed between cross-braced perimeter slurry walls,

with an additional center slurry wall in the wider sections.

The slurry walls, apart from providing temporary excavation support, are to be incorporated as part of the permanent tunnel structure. In addition, the existing elevated Central Artery, which is to remain in service during construction, will be supported on the slurry walls through a series of transfer beams and girders at the existing footing or pile cap level. The slurry walls generally penetrate at least 10 feet into the bedrock.

Subsurface Conditions

The subsurface conditions at each of the pumping test well locations are summarized in the schematic shown in Figure 2. The typical subsurface profile in the Central Area consists of a surface layer of urban fill, under which a layer of organic soil is generally encountered. The organic deposits are underlain by Boston Blue Clay, which is underlain by glacial till. Sand layers are not unusual above, below or within the clay stratum. The glacial till is underlain by bedrock, the upper layers of which typically consist of completely to severely weathered argillite. Following are descriptions of the strata in which the pumping tests were conducted.

Glacial Till. Glacial till encountered in the Central Area typically consists of a heterogeneous mixture of sand, silt, clay and gravel, with cobbles and boulders. Glacial till with a fines content (material passing the No. 200 sieve) of greater than about 30 percent is considered here to be cohesive till and is designated T₁ and T₂. Till consisting primarily of granular soil with less than 30 percent fines is described as granular till and is designated as T₃.

Standard penetration test (SPT) N-values for the till are typically greater than 50 blows/foot, reflecting a very dense soil. However, some zones with N-values less than 50 blows/foot are also present in the till, indicating looser materials, especially toward the top of the cohesive till stratum. The cohesive till with SPT values less than 50 blows/foot is designated as T₁, while the very dense cohesive till with SPT values greater than 50 blows/foot is designated as T₂. The granular till, T₃, generally has SPT values greater than 50 blows/foot.

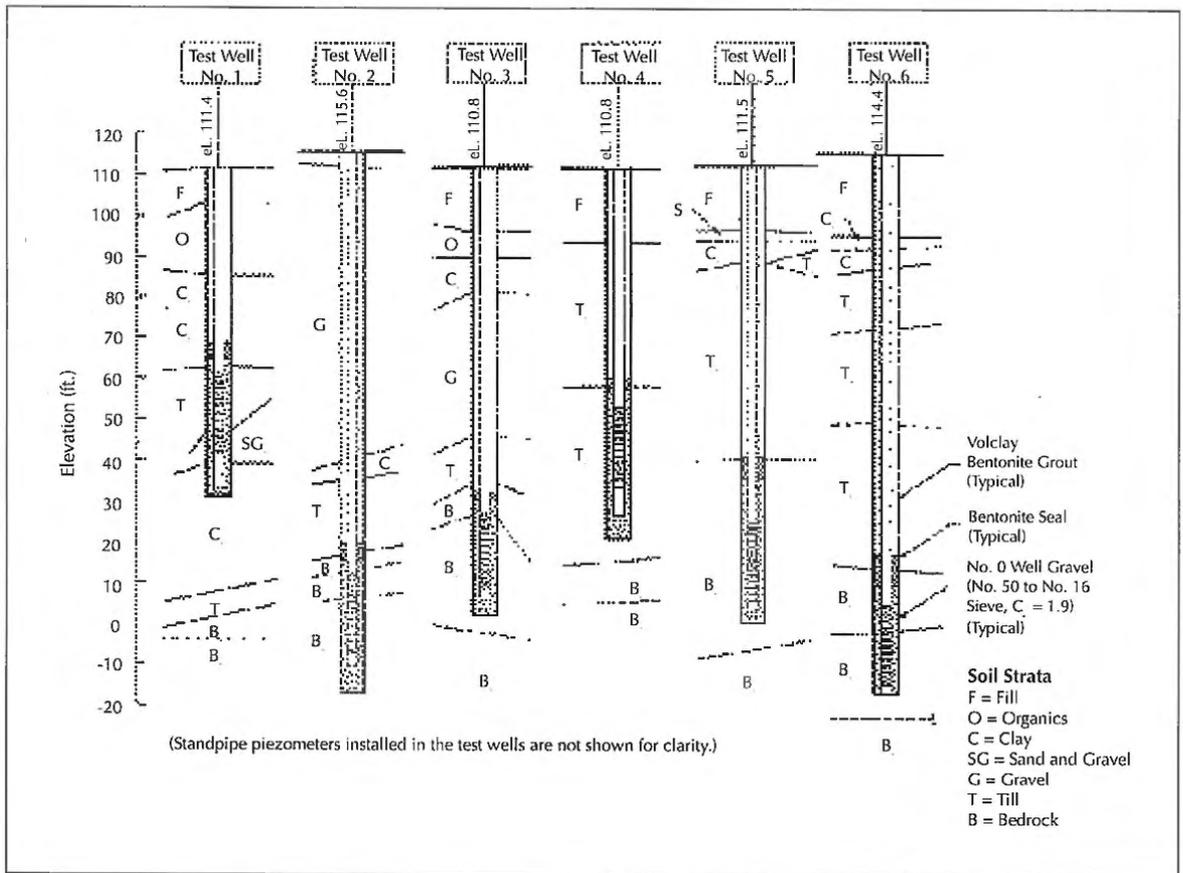


FIGURE 2. Subsurface conditions at the pumping test well sites.

Borehole permeability tests indicate that the permeability of the T₁ and T₂ till deposits ranged from 1×10^{-6} to 4×10^{-4} cm/sec and that of the T₃ till ranged from 1×10^{-4} to 3×10^{-3} cm/sec.

Sand & Gravel. Localized natural deposits of sand (S) and sand and gravel (SG) were encountered within the clay, glaciomarine and glacial till deposits in the northern part of the Central Area. The thickness of the localized granular strata were typically less than 10 feet. The gradation of the S and SG deposits ranged from fine sand with silt to widely graded sand and gravel.

SPT N-values in the S and SG deposits ranged from 8 to over 100 blows/foot, with most of the values greater than 30 blows/foot, indicating a dense to very dense condition. Sand and sand and gravel deposits with SPT values less than 50 blows/foot have been designated S₁ and SG₁, respectively, and deposits with SPT values greater than 50 blows/foot

have been designated S₂ and SG₂, respectively. Part of the pumping stratum in Pumping Test No. 1 was located in this deposit.

Bedrock. Top of bedrock in the Central Area was observed to vary between depths of 65 and 130 feet (el. 57 to el. -15) in the borings drilled along the proposed artery alignment. Bedrock was sampled by coring or, in some cases where the rock was sufficiently weathered, by split spoon. Depths of coring ranged from 5 to 103 feet. Rock recoveries in the coring runs were generally between 30 and 70 percent, although some were as high as 100 percent. The bedrock encountered in the Central Area is part of the Cambridge formation of the Boston Basin. The predominant rock type recovered in the borings was Cambridge argillite. Other rock types found include interbedded medium-grained sandstone and conglomerate, tuffaceous argillite and intrusive rocks such as basalt, andesite and diabase.

The argillite encountered in the borings is fine grained, very thinly bedded and is characterized by alternating dark and light grey beds of silt- and clay-sized particles. The rock is regionally folded, and locally the dip of the bedding is at a generally high angle (*i.e.*, 55 to 85 degrees) from the north end of the Central Area to just north of Summer Street, although it varies locally from horizontal to vertical. South of Summer Street the bedding is generally moderately dipping (*i.e.*, 35 to 55 degrees).

The top of the rock is typically moderately to extremely fractured, with close to very close joint spacing.

The argillite has been grouped into three categories based on weathering:

- B₁ rock, consisting of completely weathered or very severely weathered argillite;
- B₂ rock, consisting of severely or moderately weathered argillite; and,
- B₃ rock, consisting of moderately weathered, slightly weathered, and unweathered argillite.

Groundwater. The groundwater regime in the Central Area is typically distributed into two distinct aquifers: an unconfined shallow aquifer above the clay that acts as an aquitard, and a confined aquifer below the clay.

Groundwater levels measured in shallow observation wells in the fill, organic soils and shallow till ranged from el. 95.0 to el. 112.0, with most levels above el. 98.0. There was no clear pattern to the groundwater levels, nor was there a clear gradient toward the harbor. These data indicate that the groundwater elevations at any given point may be influenced more by local subsurface drainage, such as sumps, leaking sewers or storm drains, or drainage along pipeline bedding, than by the overall groundwater flow pattern.

The groundwater levels in the deeper observation wells and piezometers in clay, glaciomarine, till and bedrock strata ranged from el. 68.0 to el. 111.0, with most levels above el. 89.0 and below el. 102. Most of the piezometric levels in the clay, glaciomarine deposits, till and bedrock are below mean sea level, indicating that they may be affected by local subsurface drainage or pumping.

The tidal fluctuations in the groundwater level in the fill and shallow organic, clay, glaciomarine and till deposits ranged from 0 to 2.3 feet with most values below 0.6 feet, although values as high as 2.9 and 7.6 feet were measured at two locations. The tidal fluctuations in the deep wells and piezometers in the glaciomarine deposits, till and bedrock ranged from 0.1 to 2.0 feet with most values below 1.4 feet.

The groundwater elevations reported here are based on groundwater data collected mostly in 1991 and 1992.

Borehole Permeability Testing

In-situ permeability tests were performed in boreholes to estimate the permeability of the soils and bedrock during the drilling program that was conducted in the Central Area of the project. These tests included rising head and falling head tests in soil and rock, and packer pressure tests in rock.

The borehole permeability data indicate that the granular fill and the rock are considerably more permeable than most of the other soils, which act as low permeability layers between the two primary aquifers. The groundwater observation well and piezometer data support the observation that there are two primary aquifers: a shallow unconfined aquifer in the fill and a confined aquifer in the rock.

The borehole permeability test results indicate that, in general, the permeability of the rock ranges from 10^{-6} to 10^{-2} cm/sec. Permeability values as low as 2×10^{-7} cm/sec were measured. In general, the rock was found to be 1 to 3 orders of magnitude more permeable than the overlying till. However, in many areas the granular till was found to be as permeable as rock.

The bedrock permeability data indicate that, in many borings, the more permeable zones occur essentially at random within the rock, both horizontally and vertically. The borehole permeability data did not show a clear trend of decreasing rock mass permeability with depth within the depth of the borings (in excess of 100 feet below the top of the rock). The data also indicate that there does not appear to be a significant difference in the range of permeability between the completely

weathered argillite (B₁) and the less weathered rock (B₂ and B₃).

Pumping Tests: Installations

The installations at each of the pumping test sites consisted of a 12-inch diameter pumping test well screened and sealed in the desired test stratum, piezometers and observation wells. An array of piezometers was installed in the pumping stratum at progressively larger distances away from the test well in two directions approximately perpendicular to each other.

Pumping Test Well Installation. At each site, the piezometer boring closest to the pumping test well (typically 5 feet away) was drilled prior to start of the test well drilling. Split-spoon samples were collected at 5-foot intervals, and strata changes were recorded in this boring. The information obtained was used during the test well installation to select depths for well screen installation and to characterize the subsurface strata.

The pumping test wells consisted of 6-inch-diameter polyvinyl chloride (PVC) screen and riser pipe installed in a 12-inch-diameter drilled hole. Drilling for the test well was performed by a mud/air rotary drill rig using a direct mud rotary drilling technique. To prevent the collapse of the drill hole in the fill, the top 20 feet of the drill hole were cased with a 16-inch-diameter steel pipe. The remaining depth of the test well was drilled open hole with drilling fluid. A biodegradable drilling fluid was used in continuous circulation to remove the cuttings and keep the hole stabilized. The fluid is an organic drilling fluid that breaks down naturally. The purpose of using a biodegradable drilling fluid was to minimize the effects of the drilling fluid on the permeability of the formation to be tested.

In all the test wells, the screened sections were 6 inches in nominal diameter and 20 feet long. The screened sections were made up of 10-foot-long Schedule 80 PVC sections with an internal diameter of 5.68 inches and provided with 0.02-inch-wide slots. The riser pipe also consisted of Schedule 80 PVC pipe. The bottom of the screened sections were provided with PVC silt traps and plugs. The silt trap was a section of riser pipe typically 1.5 feet long (except for Sites 1 and 4 where it was 8 feet long). The

longer silt traps were used where the well screens were in soil to permit setting the pumps at a lower elevation.

Two standpipe piezometers were installed in the test well hole concurrently with the installation of the well screen and riser to permit measurement of the water level in the test well. The standpipes were taped to the outside of the test well screen and riser. The standpipes consisted of a 1.38-inch-inside-diameter slotted Schedule 40 PVC pipe 24 inches long attached to a 1.38-inch-inside-diameter Schedule 40 PVC riser pipe. The slots in the screen were 0.01 inch wide. The standpipe piezometers were also provided with a 1.5-foot-long silt trap and bottom plug. Two identical standpipes were provided in case one malfunctioned.

The test wells were completed by installing a filter pack around the well screen. The filter pack consisted of well gravel composed of particles ranging from No. 50 to No. 16 sieve size with a uniformity coefficient of 1.9. The filter pack generally extended 2 to 11 feet above the top of the well screen and 2 to 13 feet below the bottom of the well screen. The drilling fluid was flushed from the hole with clean water prior to installing the filter pack. Flushing was continued until the return water was usually clear. All well materials (filter pack, seals and riser backfill) were tremied into place during the installation.

A bentonite seal was installed above the filter pack, and the drill hole was grouted above the seal using a bentonite grout to prevent leakage from the upper layers. The bentonite seal varied in thickness from 2 to 7 feet. A 12-inch-diameter road box was installed at the surface to enclose the test well riser pipe and the two standpipes.

Piezometer & Observation Well Installations. Vibrating wire piezometers were used as the primary means of monitoring drawdown in the pumping test program because of their ability to provide a rapid response to changes in piezometric head without significant lag. An array of piezometers was installed in the pumping stratum at progressively larger distances away from the test well in two directions approximately perpendicular to each other. In addition, a few piezometers and observation wells were installed in the overlying clay, till

and fill layers to measure groundwater elevation or piezometric head changes in these overlying soils.

The piezometers installed for the pumping test program were assembled within a 12- to 14-inch-long, 1.25-inch-inside-diameter machine-slotted Schedule 40 PVC well screen. At several locations, two or three piezometers were installed in the same borehole with each piezometer located in a different stratum. The elevations of the piezometer tips ranged from el. -3.9 to el. 58.8.

All observation wells in the pumping test program were installed as shallow observation wells primarily in the fill, organics or upper portion of the clay deposit. Each observation well generally consisted of a 2-inch-inside-diameter machine-slotted Schedule 40 PVC well screen, connected with a threaded flush joint to a 2-inch solid-wall Schedule 40 PVC riser pipe extending to the ground surface.

In addition to the piezometers and observation wells installed as part of the pumping test program, a number of piezometers and observation wells installed earlier as part of the geotechnical investigation for the various design sections in the Central Area were also monitored during the pumping tests. (Detailed descriptions of the drilling and sampling procedures and of the piezometer and observation well installation procedures are provided elsewhere.¹⁻³)

Split-spoon samples were collected at 5-foot intervals, and strata changes were recorded in the piezometer boring closest to the pumping test well. The other borings drilled in the pumping test program for piezometer and observation well installation were generally advanced to the required depth of installation without sampling.

Test Well Development. Well development was performed to remove fines from the filter pack around the well screen and the surrounding pumping stratum. Test well development was performed using a two-step procedure. The first step included air-lift development and mechanical surging, while the second step utilized a combination of pumping and mechanical surging.

During the first step of the well development, air-lift surging and mechanical surging

were combined to achieve a gross cleaning of the fines in the filter pack and surrounding material. This step was used to preclude the clogging of the submersible pumps. Pumping during this step was performed by a suction pump and an eductor pipe. During air surging, a 1-inch-diameter air line was lowered inside a 2-inch-diameter PVC eductor pipe, and compressed air was injected into the well to lift the water to the surface. Surging was achieved by alternately injecting and cutting off the air supply to allow the water column to rise and fall. Mechanical surging was performed by operating a plunger up and down in the well screen and riser sections to raise and lower a close-fitting surge block causing water to flow in and out of the screen. Development was continued by alternately repeating these two techniques until the water being pumped through the eductor pipe was observed to be relatively clear. This first step in the development process generally lasted about 4 hours.

During the second step of well development, a submersible pump was lowered into the well and the well was alternately pumped and mechanically surged with surge blocks. This procedure was repeated until the pumped water was observed to be clear and free of sand, silt and bedrock fragments. The second step was generally continued for 4 to 8 hours except in the case of Sites 2, 3 and 4 where a longer period of up to 16 hours was required.

Pumping Tests: Procedures

A step drawdown test was performed prior to the primary pumping test at each of the pumping test sites. The purpose of the step test was to determine the initial pumping rate for the pumping test. The step tests were performed by increasing the pumping rate in steps, while monitoring the drawdown in the test well. The steps in the pumping rate were generally 25, 50, 75 and 100 percent of the target pumping rate. The target pumping rate was established on the basis of observations made during the well development. Each step typically lasted for 2 hours, after which the pumping rate was increased to the next higher step without allowing the well to rebound.

The drawdown observations made during the step test were used to select the initial

TABLE 1.
Summary of Aquifer Parameters

Test Site No.	Location	Pumping Zone		Estimated Effective Thickness of Pumping Stratum (feet)	Pumping Rate (gpm) & Duration (hours)	Elevation of Initial Water Level at Test Well Location (feet)	Drawdown in Test Well at End of Test (feet)
		Stratum*	Elevation of Sanded Zone				
1	N. Washington at Valenti Way	C ₂ T, SG ₂ , C ₄	65.7 to 31.4	30	5 (60) 10 (54)	101.9	34.5
2	Blackstone at Hanover	B ₁ , B ₂ , B ₃	16.1 to -16.4	40	25 (72)	98.0	42.0
3	Atlantic Avenue opposite Columbus Park	B ₂ , B ₃	28.8 to 0.8	40	50 (120)	97.1	81.0**
4	Surface Road at India	T ₂	57.8 to 20.8	40	0.7 (120)	88.4	31.5
5	Atlantic Avenue at Boston Edison Site	B ₁	35.4 to -1.1	50	2 (120)	89.4	43.8
6	Atlantic Avenue at Summer	B ₁ , B ₂	9.2 to -17.6	50	15 (10) 13 (17) 10 (46) 7.5 (47)	87.3	72.6**

Notes: * Soil and rock not sampled in test well. Stratum is assumed to be same as nearest well, typically 5 feet away. See Figure 2 for strata descriptions.
** The drawdown caused the water level to drop below the top of the well screen in the test well at the end of the test.

pumping rate so that the projected drawdown at the end of the pumping test would be large enough to be representative of the dewatering anticipated during the construction period but not so great as to cause the water level to drop below the top of the well screen. In Pumping Test No. 6, the pumping rate had to be decreased during the test because of excessive drawdowns. In Pumping Test No. 1, the pumping rate had to be increased during the test because of small drawdowns. Table 1 presents a summary of the aquifer parameters.

Pumping Test & Recovery Monitoring Procedures. The pumping tests were performed using a submersible electric pump at the bottom of the wells. The submersible pump used for the pumping tests was rated to provide a maximum flow rate of 50 gpm at 130-foot head. Electric power was supplied to the pump by a diesel-powered generator. A backup generator was also provided.

Flow was measured by an accumulating total flow meter, which measured the volume of water in cubic feet. The average flow rate was determined by dividing the volume of water

pumped by the elapsed time between successive readings. Flow meter readings were initially taken every 30 seconds. After a stable flow was achieved, the flow was measured every 30 minutes until the end of the test. The pumped water was discharged into a sedimentation basin before being discharged into storm sewers as required in the discharge permits obtained for the program by the project's contractor. Samples of the pumped water were collected for water quality analyses for each of the pumping tests to demonstrate compliance with the requirements of the discharge permits.

Piezometers and observation wells were monitored during the pumping test to measure drawdown. The water level also was monitored in the standpipes installed in the filter pack around the well screen. The flow velocities through the well screen in the test wells were estimated to be small enough so the head loss through the well screen could be considered negligible.

Water levels in the test well standpipes and in the observation wells were measured to an accuracy of 0.01 foot. Prior to conducting the

pumping test, water levels were measured under static (non-pumping) conditions in the pumping test well at 6-hour intervals for a 24-hour static data collection period. This data collection period started at least 6 hours after completion of the step test to allow the wells to recover.

Upon the start of the pumping test, water levels and piezometric levels were monitored at intervals ranging from 10 seconds to 4 hours, with the most frequent monitoring taking place within the first 5 hours according to the schedule shown in Table 2.

After pumping was stopped, groundwater recovery was monitored in the test well filter pack and in selected piezometers for a minimum of 24 hours for each of the pumping tests. The frequencies at which recovery readings were taken was the same as those described above in the schedule for the pumping test.

All pumping tests were continued for 120 hours, except for Pumping Test Nos. 1 and 2. Pumping Test No. 1, located in a parking lot at the corner of North Washington Street and Valenti Way, was performed for 114 hours and was terminated 6 hours before the scheduled time because of generator failure. Pumping Test No. 2, on Blackstone Street, was terminated after 72 hours because of restrictions on permissible working times in the Haymarket area.

Test Data

Details about the pumping test program including installation procedures, borehole permeability testing, subsurface profiles at the pumping test sites, data reduction, analysis and interpretation, as well as water quality sampling and testing are provided elsewhere.¹⁻³

Figures 3 and 4, which pertain to Pumping Test No. 3, are presented as examples of the data that were collected during each of the pumping tests. Figure 3 shows the drawdowns in the test well and the different piezometers and observation wells that were monitored during the pumping test as well as the flow rate versus the log of time. Figure 4 shows the residual drawdown in the piezometers and observation wells monitored during the recovery period versus log of time (with time, t , equal to 0

TABLE 2.
Monitoring Schedule

Time After Commencement of Test (minutes)	Reading Frequency
0-2	Every 10 seconds
2-5	Every 30 seconds
5-15	Every 1 minute
15-50	Every 5 minutes
50-100	Every 10 minutes
100-300	Every 30 minutes
300-2,880	Every 1 hour
2,880+	Every 4 hours

— being the time when pumping was stopped). The drawdown versus log of distance is plotted for Pumping Tests Nos. 1 to 3 in Figure 5 and for Pumping Test Nos. 4 to 6 in Figure 6.

Results of Analyses

Because of the limited pumping periods during the pumping tests, drawdowns were still increasing at the end of the pumping tests. Therefore, transient methods of analyses were used to analyze the test data. Transmissivity and storativity values were obtained using two different methods — namely, the Jacob Semilog Plot Method and the Hantush-Jacob Curve Matching Method for leaky aquifers. For comparison, the transmissivity also was calculated using the drawdown versus distance plot method, which assumes a steady-state condition at the end of the test.

Jacob Semilog Plot Method. The Jacob Semilog Plot Method consists of plotting the measured drawdowns in any well versus the elapsed time since the start of the pumping test on a semi-logarithmic scale and fitting a straight line through the data. The slope of the line as well as its intercept with the horizontal axis (*i.e.*, the time axis) are used to calculate the transmissivity and the storativity. The Jacob Semilog Plot Method offers an advantage when analyzing confined aquifers with impermeable and/or recharge boundaries, because

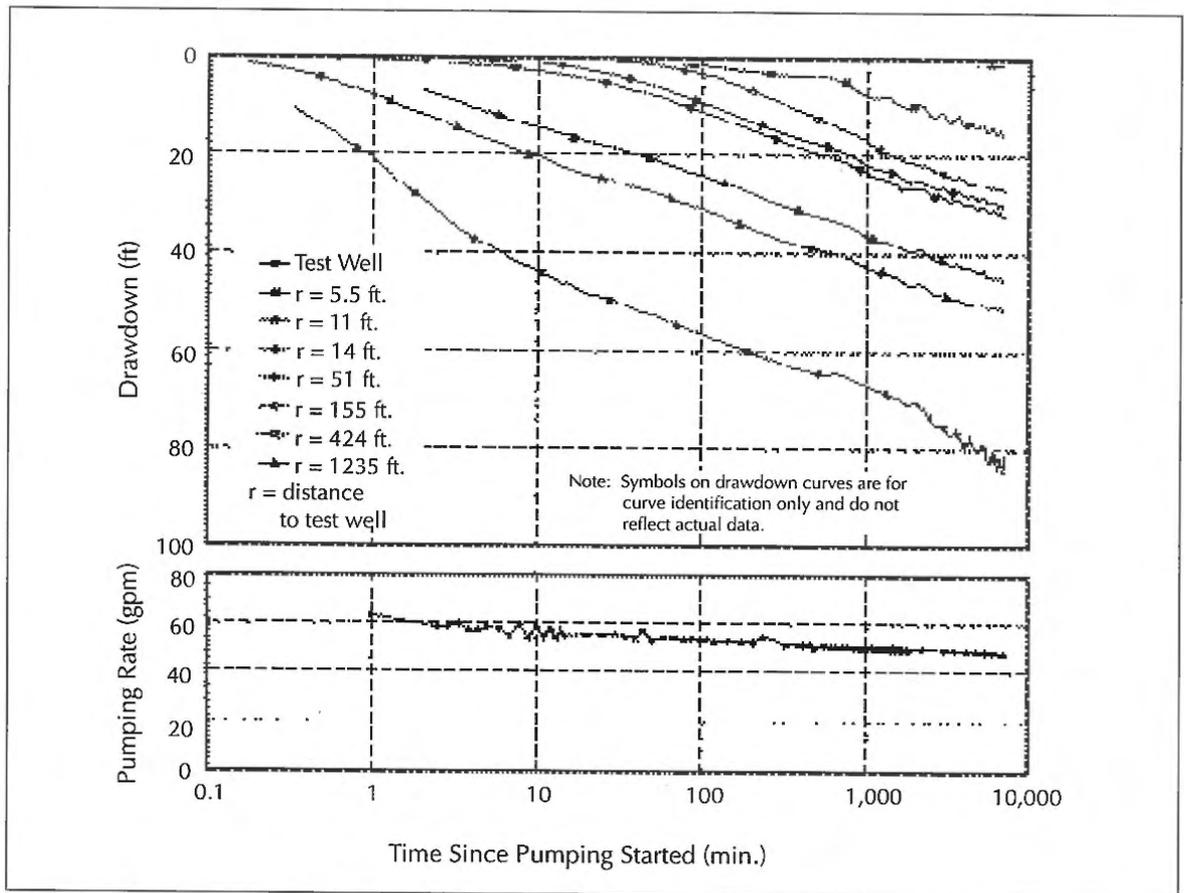


FIGURE 3. Drawdown in rock and the flow rate versus time for Pumping Test Site No. 3.

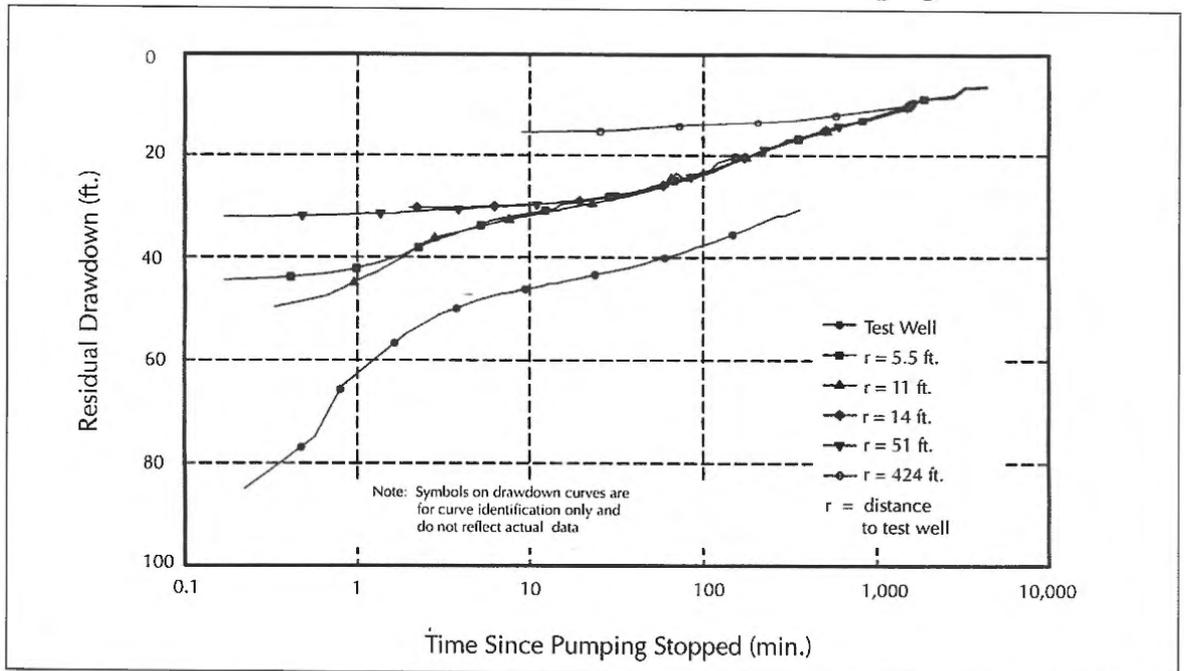


FIGURE 4. Recovery curves for instruments installed in rock for Pumping Test Site No. 3.

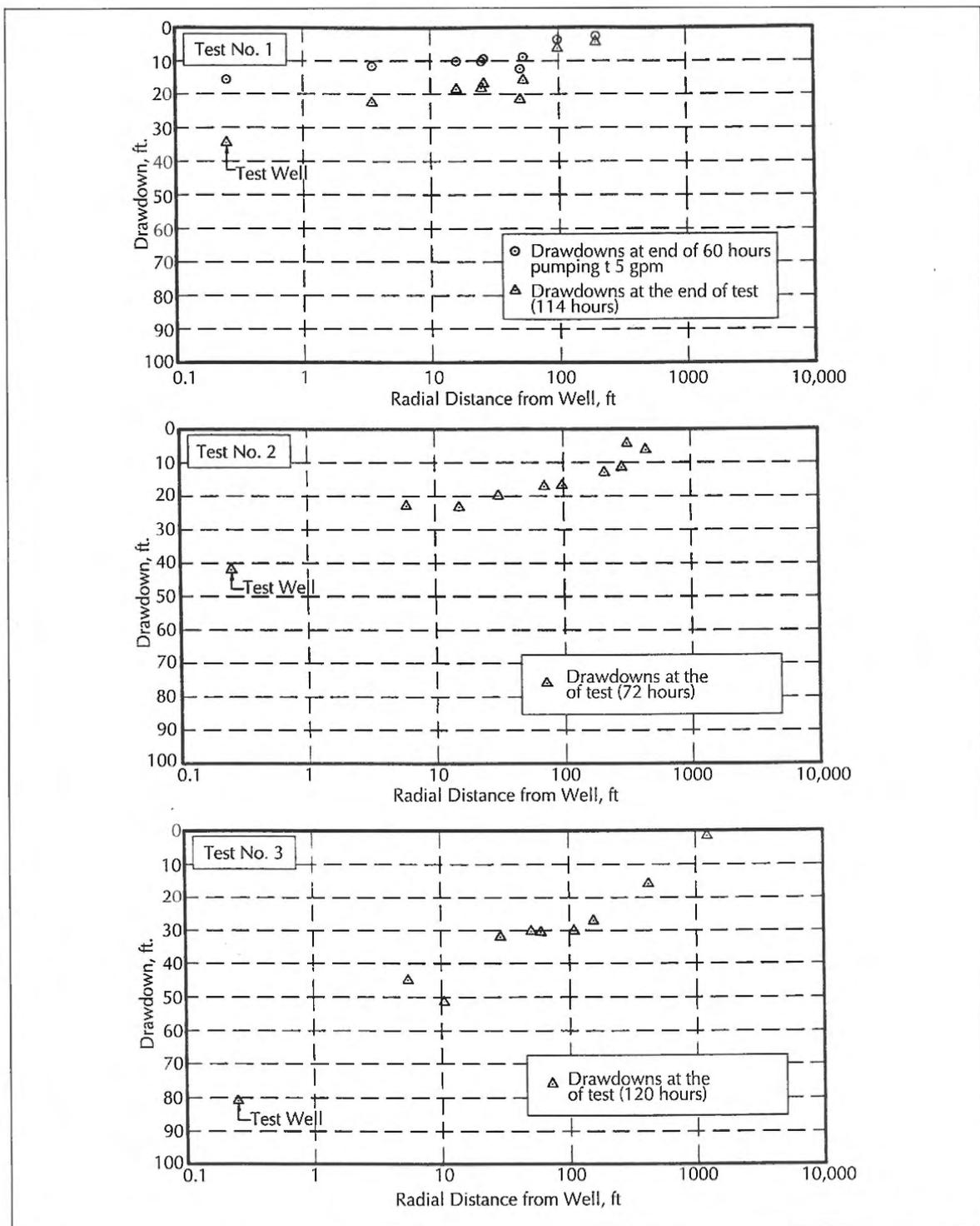


FIGURE 5. Drawdown versus distance (logarithmic scale) for Pumping Tests Nos. 1 to 3.

the presence of a boundary is reflected by a break in the drawdown versus time plot.⁴

The use of this method assumes that the aquifer is confined, saturated, homogeneous

and isotropic. The groundwater flow is assumed to take place radially in the horizontal direction. The method also assumes that the test well is screened over the entire thickness of

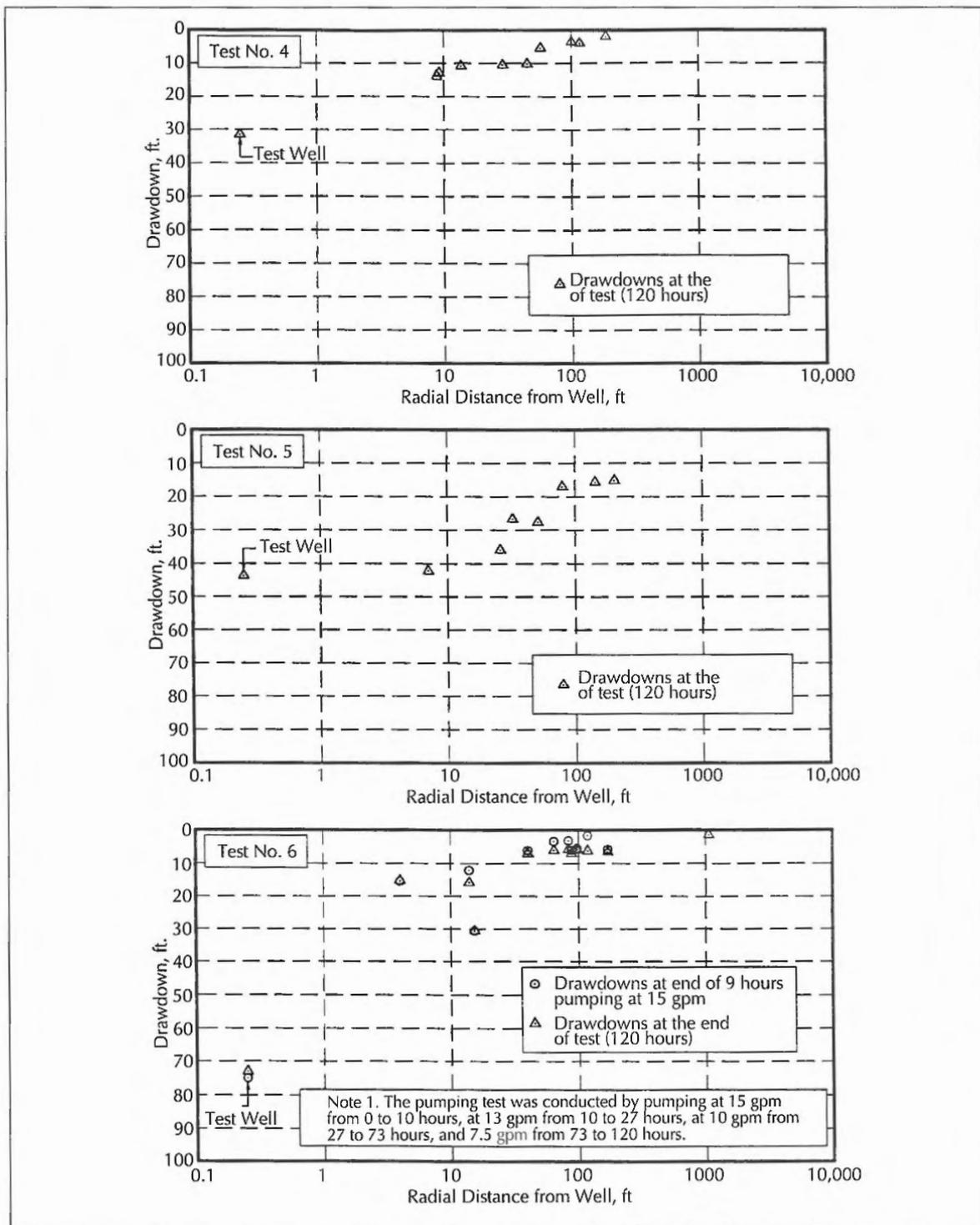


FIGURE 6. Drawdown versus distance (logarithmic scale) for Pumping Tests Nos. 4 to 6.

the aquifer. A number of other assumptions made for mathematical convenience in the Jacob Semilog Method are discussed by Dawson and Istok,⁵ and Driscoll.⁶

The Jacob Semilog Plot Method is applicable for values of $u = (1.87 r^2 S / T t)$ smaller than 0.05 — *i.e.*, for large values of t and small values of r (r is the distance from the center of a

pumping well in feet, S is the coefficient of storativity, T is the coefficient of transmissivity in gpd/ft and t is the time since pumping started in days).

Hantush-Jacob Curve Matching Method. The Hantush-Jacob Curve Matching Method is applicable to leaky confined aquifers. In a leaky aquifer, a part of the pumped water comes from water infiltrating (or leaking) from a less permeable overlying stratum called the aquitard. This leakage, caused by the drop in head in the pumped aquifer, consists of water from aquitard storage and/or the flow of water from an upper unconfined aquifer through the aquitard. The Hantush-Jacob Method assumes that all leakage occurs as a result of downward flow through the aquitard (*i.e.*, no water is released from aquitard storage).

In the Hantush-Jacob Curve Matching Method, the measured drawdowns in any well (other than the test well) are plotted versus the elapsed time since the start of the pumping test on a log-log scale. The obtained curve is then compared to a set of dimensionless plots of the theoretical response of an ideal homogeneous aquifer, called type curves. The shapes of the type curves vary with the degree of leakage. Once a match is obtained, a match point is selected — the coordinates of which are used to calculate the transmissivity and the storativity. The permeability of the aquitard can also be determined using this method of analysis.

The Hantush-Jacob Curve Matching Method assumes that the aquifer is bounded above by an aquitard and an unconfined aquifer and below by an impermeable stratum. The aquifer and the aquitard are assumed to be homogeneous and isotropic. The groundwater flow in the aquitard is assumed to be vertical, while the groundwater flow in the aquifer is assumed to take place radially in the horizontal direction. The method also assumes that the test well is screened over the entire thickness of the aquifer. A number of other assumptions made for mathematical convenience in the Hantush-Jacob method are discussed in detail by Dawson and Istok,⁵ and Driscoll.⁶

Distance Versus Drawdown Method. The Distance Versus Drawdown Method consists of

plotting the measured drawdowns on an arithmetic scale versus the distance from the test well to a logarithmic scale and fitting a straight line through the data. The equations developed for this method indicate this plot should be linear. The slope of the line, as well as its intercept with the drawdown axis, are used to calculate the transmissivity.

This method assumes that the aquifer is homogeneous and isotropic (having a constant thickness). Although the basic solution assumes that the well screen fully penetrates the aquifer, modified solutions for partially penetrating wells are available.

The equations and the assumptions associated with the drawdown versus distance method are discussed by Mansur and Kaufman.⁷

Table 3 summarizes the results and the calculated aquifer parameters for the six pumping tests performed in AGC Area 04. The radius of influence for each of the pumping tests was estimated by extrapolating the drawdown versus distance data plotted on a semilog scale. The table lists the ranges of values of transmissivities and storativities obtained by the three analysis methods used.

Data from all of the six pumping tests were analyzed assuming confined aquifers. This assumption is reasonable for the pumping tests performed in bedrock (Pumping Tests 2, 3, 5 and 6) because the subsurface conditions correspond to a pervious fractured bedrock formation overlain by a relatively impervious cover of glacial till, glaciomarine or clay deposits. These aquifers were also considered to be leaky confined aquifers because piezometers installed in the till and even in the clay overlying the bedrock (*i.e.*, installed in the aquitard) displayed substantial drawdowns. These drawdowns were observed to be higher at the bottom of the aquitard but were less than those in the pumped aquifer, clearly indicating downward flow (*i.e.*, leakage).

The data plots indicate that the fractures in the bedrock are sufficiently interconnected so that the assumption (inherent in the analytical method) that the aquifer is isotropic is a reasonable one.

The subsurface conditions for Pumping Test No. 1 show that the pumped stratum of sand

TABLE 3.
Summary of the Pumping Tests

Test Site No.	Location	Pumping Stratum	Estimated Radius of Influence (ft)	Aquifer Parameters				
				Jacob Semilog Plot Method		Hantush-Jacob Method		Drawdown Distance Method*
				Transmissivity (gal/day/ft)	Storativity (ft ² /ft ² /ft × 10 ⁻⁵)	Transmissivity (gal/day/ft)	Storativity (ft ² /ft ² /ft × 10 ⁻⁵)	Transmissivity (gal/day/ft)
1	N. Washington at Valenti Way	C ₂ T, SG ₂ , C ₄	350	194-314**	55-1,300 [§]	190-197 [§]	220-1,500 [§]	266
2	Blackstone at Hanover	B ₁ , B ₂ , B ₃	2,200	673-942	20-880	670-796	19-1,100	1,166
3	Atlantic Ave. opposite Columbus Park	B ₂ , B ₃	1,900	886-1,257***	30-1,300***	868-970***	44-1,300***	1,266
4	Surface Rd. at India	T ₂	300	28.9-44	5.7-220	—	—	35.8
5	Atlantic Ave. at Boston Edison	B ₁	350	34.1-46.3	1.6-70	29.4-35.8	5.8-49	44.3
6	Atlantic Ave. at Summer	B ₁ , B ₂	1,200	330-762 [§]	35-3,700 [§]	312-716 [§]	26-3,000 [§]	204-824

Notes: * Transmissivity calculated using drawdowns at the end of the pumping test.

** Calculations based on data from first 60 hours of pumping.

*** The analysis assumes that the aquifer is fully saturated and that the water level does not drop below the top of the well screen in the test well.

§ Calculations based on data from the first 9 hours of pumping.

and gravel and till is overlain by clay and underlain by a layer of hard clay that pinches out 100 to 300 feet west of the pumping well, which supports the assumption of a confined aquifer. The drawdown in piezometers installed in rock in the immediate vicinity of the pumping well at the end of this test was less than 4 feet.

In Pumping Test No. 4, the pumping was performed from a glacial till stratum overlain by glaciomarine and fill deposits and underlain by bedrock. The observation wells in the overlying glaciomarine and fill deposits did not undergo any drawdowns during the pumping test. Although the bedrock underlying the glacial till may be more pervious than the till, the data were analyzed as if the till were a confined aquifer. If the bedrock were a source of partial recharge for the glacial till, the assumption of a confined aquifer would result in a calculated permeability value greater than the actual permeability of the glacial till.

The effective thickness of the aquifer or the pumping stratum was estimated on the basis of the height of the sanded zone around the well screen and subsurface conditions at the test well location. The estimated aquifer thicknesses are shown in Table 3.

Conclusions

The pumping tests indicated that rock is frequently more permeable than the overlying soils. Of the four tests that were conducted in rock, three indicated permeabilities in the range of 0.5 to 2×10^{-3} cm/sec, while the fourth indicated a permeability of about 5×10^{-5} cm/sec. These results are in the same range as indicated by the borehole permeability tests in rock. One pumping test was performed in glacial till and indicated a permeability of about 5×10^{-5} cm/sec, which is near the middle of the range measured by the borehole permeability tests in glacial till. One pumping test was performed in glacial till and sand and gravel deposits and indicated a permeability of about 5×10^{-4} cm/sec.

The range of permeability values estimated from the pumping test for various strata were consistent with borehole permeability data and with permeability data values from grain size correlations as shown in Table 4.

The radii of influence estimated from the four pumping tests in rock ranged from 350 to 2,200 feet for drawdowns in the pumping wells ranging from 42.0 to 81.0 feet (as shown in Ta-

TABLE 4.
Summary of Permeability Values

Soil/Stratum Bedrock	Permeability (cm/sec)		
	Pumping Test	Borehole Permeability	Correlation With Grain Size
Bedrock	B ₂ & B ₃ Bedrock: 0.5×10^{-3} to 2.0×10^{-3} B ₁ Bedrock: 5×10^{-5}	1.0×10^{-6} to 1.0×10^{-2} with values as low as 2.0×10^{-7}	NA
Glacial Till	5.0×10^{-5}	T ₁ , T ₂ : 1.0×10^{-6} to 4.0×10^{-4} T ₃ : 1.0×10^{-4} to 3.0×10^{-3}	3.0×10^{-5} to 5.0×10^{-4}
Sand & Gravel	5.0×10^{-4}	1.8×10^{-5} to 3.4×10^{-4}	3.0×10^{-5} to 5×10^{-4}

Note: NA = Not available

ble 3). The extent of the zone of influence from pressure relief during construction may be affected by a number of factors including variations in rock mass permeability; the amount, duration and area of pressure relief; and the use of grout cutoff curtains. The zones of influence during construction are likely to be somewhat larger than those observed in the pumping test program unless grout cutoff curtains are used in more permeable zones of rock.

Groundwater control in the deep excavations for the mainline highway tunnels and the ventilation buildings will consist primarily of relieving piezometric pressure in the rock below the excavation. The proposed excavations for the mainline highway tunnels and the ventilation buildings extend to depths in excess of 100 feet below the piezometric water level in the rock underlying the bottom of the excavations. In much of the Central Area, excavation to the final subgrade elevation will leave a relatively thin layer of lower permeability soil remaining below the bottom of the excavation or will reach the rock. Pressure relief is necessary to avoid excess upward gradients in the less permeable soils and completely weathered rock, which could cause severe disturbance and softening, and to avoid mass uplift of the less permeable layer remaining below the bottom of the excavation. At the same time, it will be necessary to prevent excessive lowering of the piezometric pressure in the rock outside of the excavation limits that could lead to lower piezometric levels and cause consolidation and settlement of the overlying cohesive soils.

The relief wells will need to extend to a depth great enough to intercept potential water-bearing layers as necessary to control uplift and upward gradients. Therefore, the pressure relief wells may extend well below the bottom of the slurry walls and may lower the piezometric level outside the limits of CA/T construction. The pumping test program provided direct indications of potentially high pumping rates and large zones of influence. The significance of this observation is that grout curtains alone may not be effective in minimizing the drawdowns outside the limits of the excavations for the cut-and-cover tunnels during the pressure relief operations because of the presence of deep permeable zones of rock. Therefore, recharge wells are required to control drawdowns outside the excavation walls.

Observations from the pumping tests also indicate that lowering of the water pressure in the rock aquifer is not likely to directly affect the groundwater level in the fill due to the thickness of lower permeability soils (organic deposits, clay, glaciomarine and till) between the fill and rock.

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