Predictions & Observations of Groundwater Conditions During a Deep Excavation in Boston

The uncertainty associated with predicting groundwater flow through bedrock in a deep excavation requires using a flexible design approach and a monitoring program.

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Several groundwater-related issues were investigated and documented in connection with the design and construction of a seven-level underground parking garage at Post Office Square in downtown Boston. Identifying and interpreting local hydrogeological conditions were critical to assessing the feasibility of a permanent underdrain system and its impacts on adjacent streets and buildings.

During the design phase, an extensive subsurface exploration program was undertaken to characterize subsurface hydrogeologic conditions. The program included installing observation wells and piezometers, permeability testing and a pump test program. Analytical and numerical dewatering simulation models were used to predict changes in groundwater conditions as the excavation progressed. Due to the lack of local experience associated with the unprecedented excavation depth, there were many uncertainties associated with the selection of hydrogeologic properties, model assumptions and boundary conditions.

During the construction phase, an extensive groundwater instrumentation monitoring program was implemented to measure groundwater response in the vicinity of the excavation.

Project Background

Post Office Square Garage consists of a seven-level below-grade parking structure with a public park located on the garage roof at

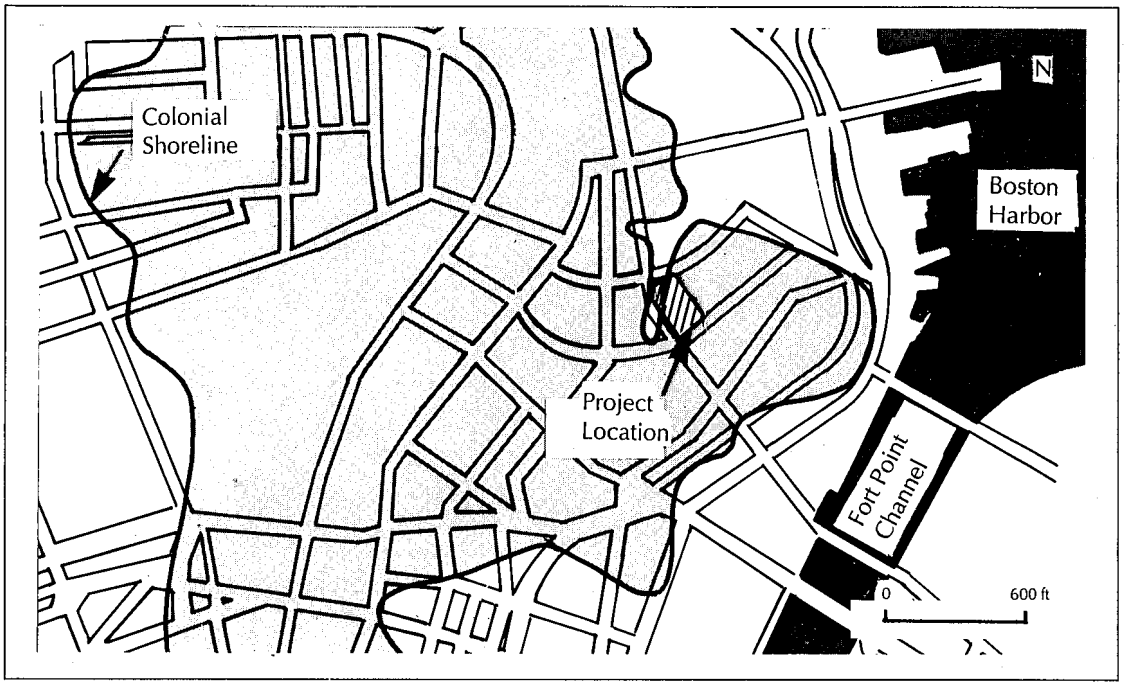

FIGURE 1. Project location.

ground surface. The project is located within Boston's downtown Financial District as shown in Figure 1, and is bounded by Milk, Congress, Franklin and Pearl Streets.

The project was built using the top-down construction method as described by Whitman *et al.*[1] and Schoenwolf *et al.*[2] The perimeter wall consisted of a cast-in-place concrete diaphragm wall (slurry wall) extending to a depth of 85 to 90 feet. A cross section through the garage is shown in Figure 2.

The subsurface soil and rock conditions (also indicated on Figure 2) from the ground surface downward are primarily composed of five strata.

There is a surficial layer of miscellaneous fill consisting of a fine sand to sandy gravel with varying amounts of brick, concrete and granite blocks. This fill varies in thickness from two to 13 feet.

Below the fill is a gray silty clay deposit, ranging in thickness from 35 to 50 feet with lenses and layers of sand and silt. The undrained shear strength of the clay varied from 1,000 to 1,500 pounds per square foot.

A sand layer, ranging in thickness from one to nine feet, was encountered at most locations around the site. This deposit can be described as medium dense to very dense, fine to coarse sand with varying amounts of fine to coarse gravel.

The glacial till underlying the site generally consisted of a hard, clayey to gravelly silt to a very dense silty coarse to fine sand, ranging in thickness from five to 38 feet. (Portions of this stratum have since been reclassified by Humphrey as glaciomarine deposits.[3])

Bedrock consisted of a very soft to moderately hard, completely to very slightly weathered argillite and sandstone.

The garage is surrounded by city streets. The nearest buildings range from 50 to 75 feet from the site perimeter. Foundations of these structures vary from spread footings supported in the upper clay stratum to deep caissons and piles installed into the glacial till. Numerous utilities are located below the adjacent streets in the fill stratum.

Two options were considered for the design of the lowest level garage floor slab:

- A structural mat capable of resisting full hydrostatic pressures; or,
- A slab-on-grade with hydrostatic pressures relieved by a permanent underdrain system below the slab.

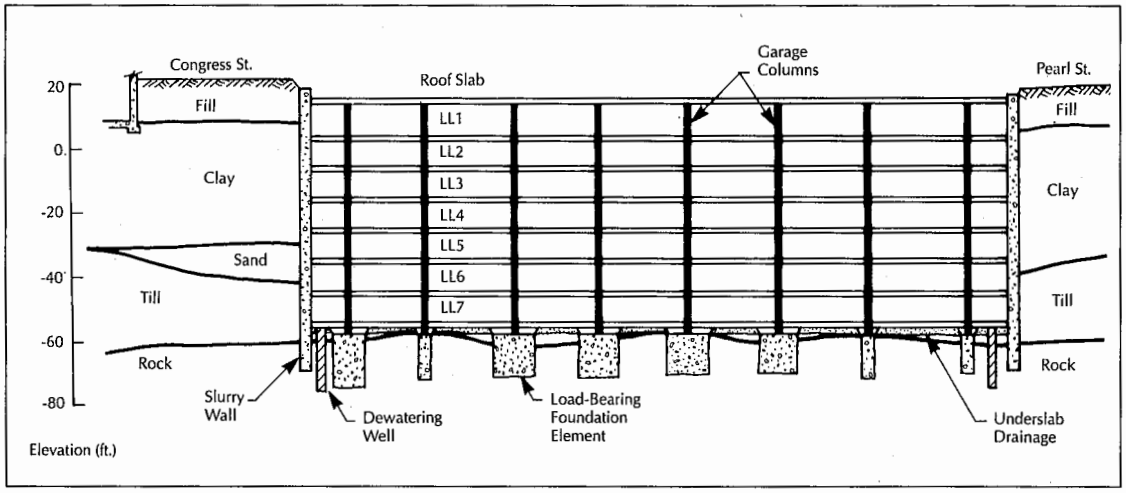

FIGURE 2. East-West cross section through the project.

In order to determine the feasibility of the latter option, major hydrogeologic engineering issues were investigated:

- What quantity of groundwater flow will enter the underdrain system, and how might this quantity (seepage) be minimized?
- What are the long-term effects of the underdrain system on the surrounding groundwater and piezometric levels? What would be the aereal extent of the potential drawdown?
- Due to reductions in piezometric levels, what would be the resulting settlement of the clay stratum and what would be the potential effects on adjacent buildings and utilities?

To evaluate these issues, in-situ permeability testing and a pump test program were conducted to estimate hydrogeologic parameters for analytical and numerical dewatering simulation models.

Subsurface Explorations

Two phases of subsurface explorations were undertaken for the project. The first phase consisted of 15 borings that were drilled within the site limits. These borings were primarily conducted for foundation design and included installing six shallow observation wells and eight piezometers. The groundwater instrumentation installed during this phase was for preliminary design purposes and was later destroyed when the old above-ground garage occupying the site was demolished. During this phase of the subsurface explorations, 14 falling head permeability tests were performed in the clay, sand, glacial till and bedrock strata. Four water pressure (packer) tests were also performed in the bedrock.

Supplementary explorations were undertaken primarily to install instrumentation for groundwater monitoring during construction. These instruments were installed approximately one year before the anticipated start of construction to establish preconstruction groundwater and piezometric levels. The instrumentation included seven observation wells and 34 piezometers that were installed within an approximate two-block area surrounding the site as shown in Figure 3. During this exploration phase, 35 additional variable head permeability tests (rising and falling head) were performed in the sand and glacial till strata and bedrock. However, these tests were primarily conducted within the bedrock in an attempt to better define permeability variations with depth below the top of rock.

Pneumatic and standpipe piezometers were installed in the completed boreholes. Pneumatic piezometers were primarily installed within the clay stratum and consisted of a diaphragm-acting porous stone. Standpipe pie-

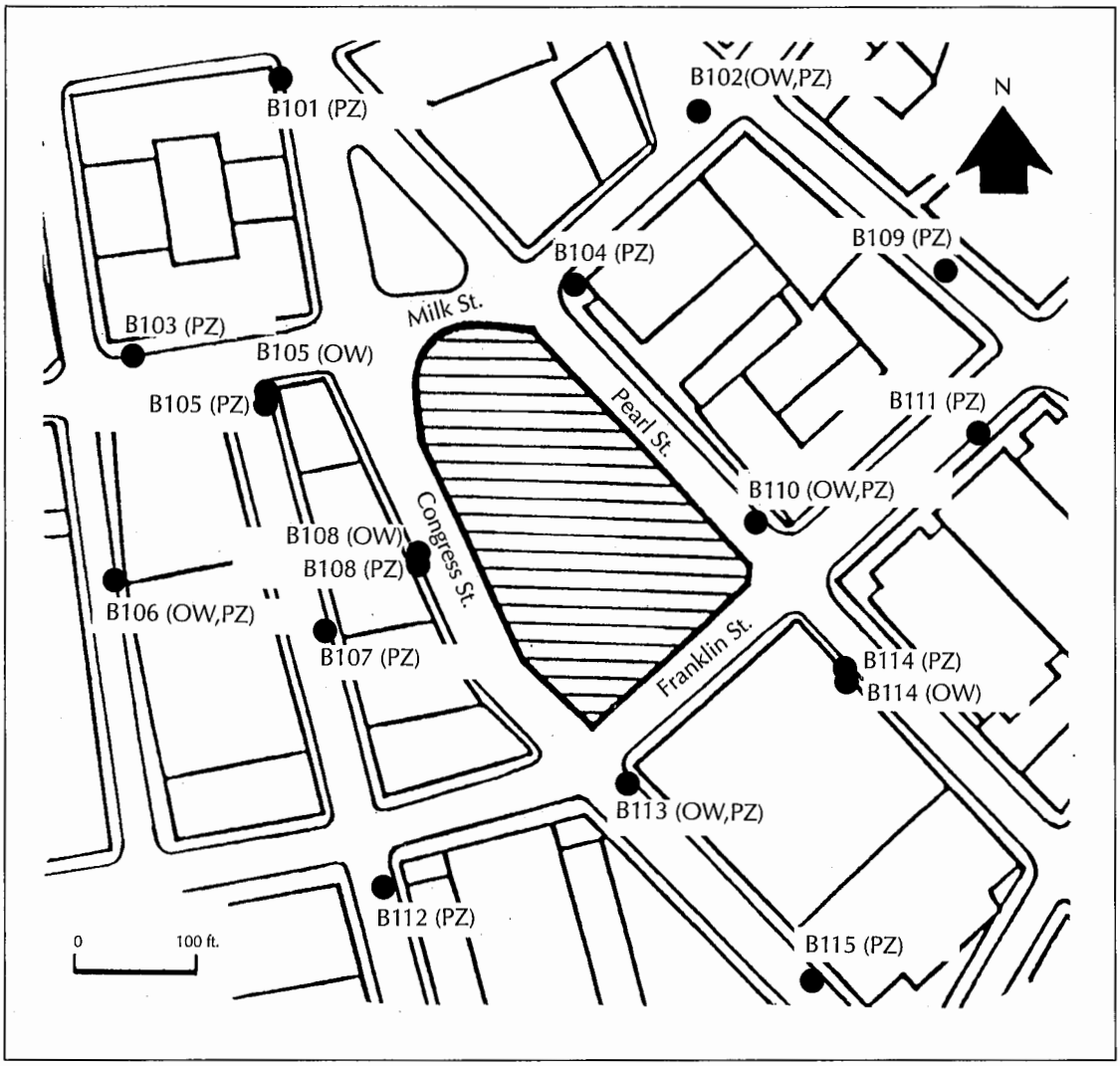

FIGURE 3. Location of the groundwater instrumentation.

zometers were typically installed in the more pervious glacial till and bedrock. The standpipe piezometer tips were 12 inches long, 1-½ inches in diameter, and had a screen consisting of a 40-micron vyon filter.

Figure 4 presents preconstruction groundwater and piezometric levels from both exploration phases. The figure indicates an apparent hydraulic gradient within the clay stratum. Observation wells installed in the fill and piezometers installed in the upper clay layer (at or above el. 0) indicated hydrostatic conditions with a corresponding groundwater level of approximately el. 10 (typical of this area). However, at or below el. -30, depressed piezometric levels were recorded in the lower clay layer, glacial till and bedrock, with an average piezometric level of approximately el. -8.

It is uncertain what conditions may have caused this variation. There was no apparent deep (bedrock) pumping near the project area.

Results from both phases of in-situ permeability tests are summarized in Figure 5. The results have been normalized to reflect the range of permeabilities as a function of depth below the top of bedrock. In general, the bedrock permeability ranged from 10^{-3} to 10^{-5} centimeters per second (cm/sec), with little to no trend with depth. The permeability of the gla-

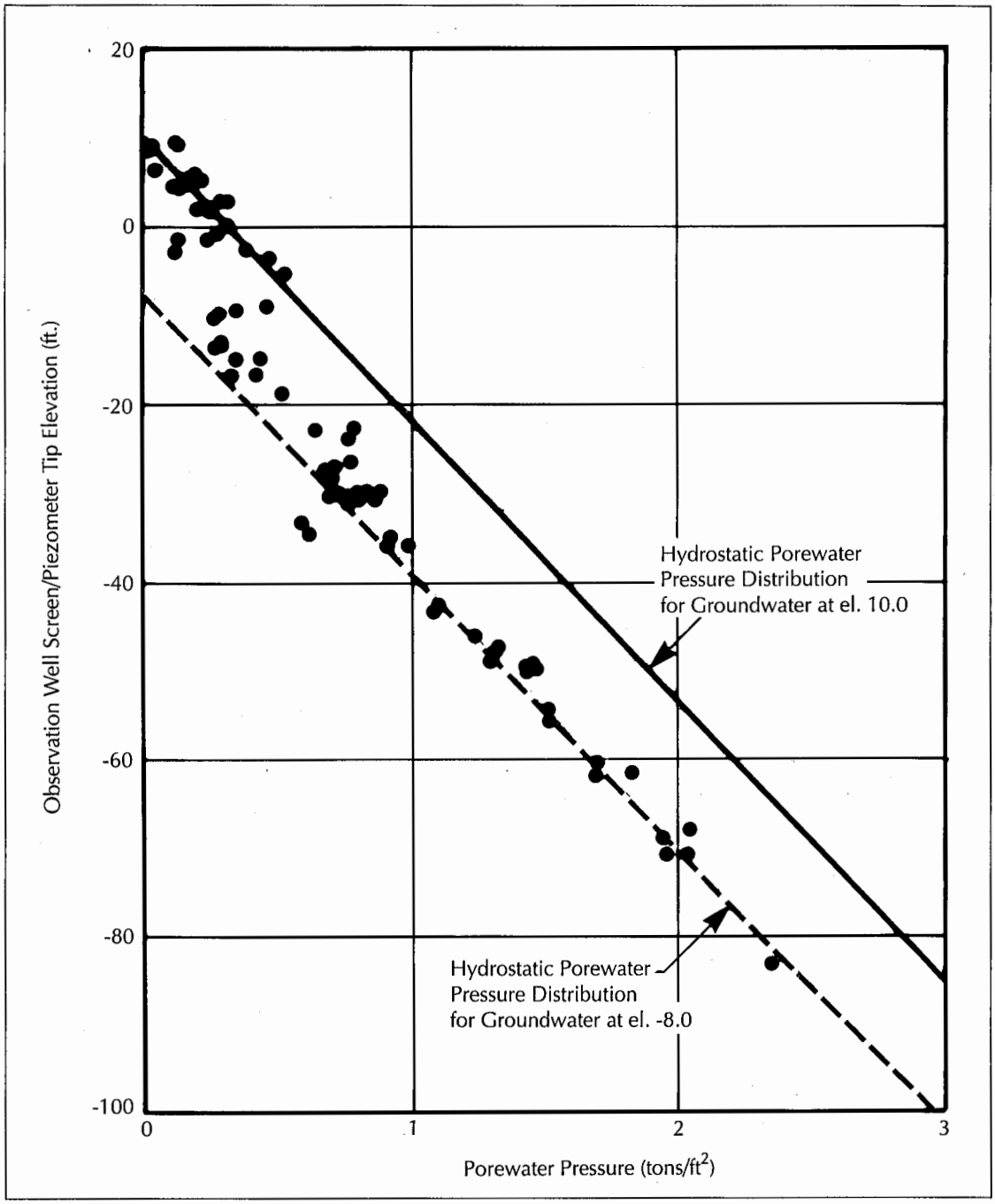

FIGURE 4. Preconstruction groundwater and piezometric levels.

cial till typically appeared to be less than the bedrock, varying from 10^{-4} to 10^{-6} cm/sec.

Also of interest were six tests within the bedrock in which both falling and rising head permeability tests were conducted. Four of the six tests indicated higher permeability values for the rising head tests. A comparison between the tests revealed that the results were within 0.05 cm/sec of each other.

Finite Element Analyses

Parametric analyses were performed on a mi-

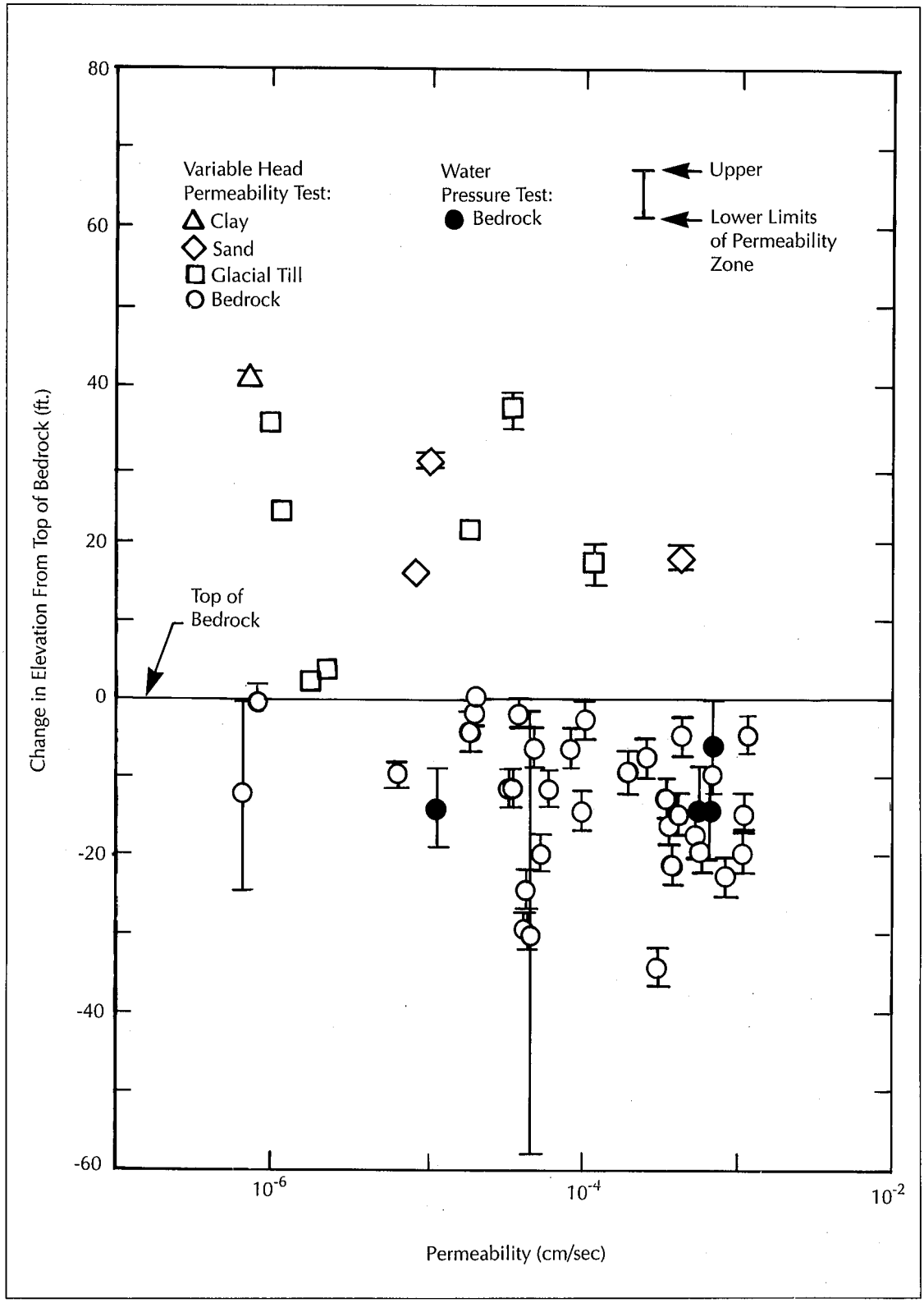

FIGURE 5. Summary of in-situ permeability tests.

cro-computer using finite element analysis software. The results of the finite element analyses were presented in flownet form using a graphical presentation software package.

The analyses were conducted to estimate the quantity of flow into the excavation and the potential drop in piezometric levels at the bottom of the compressible clay stratum. Table 1 summarizes the assumptions associated with the analyses.

From the results of the analyses, several conclusions were drawn. Theoretical estimates of groundwater flow into a permanent underdrain system depend to a high degree on the in-situ permeability of the bedrock and the assumed boundary conditions for the particular analysis. For the average value of bedrock permeability (see Table 1), the estimated total flow into the underdrain system was approximately 20 gallons per minute (gpm). For the highest assumed bedrock permeability, the total estimated flow was approximately 250 gpm.

Increasing the slurry wall penetration below the bottom of the excavation minimally reduces flow into the excavation. The greatest reduction of flow with increasing slurry wall depth occurred for the case of lowest bedrock permeability. For a constant total head boundary at the bottom of the bedrock stratum, the depth of slurry wall cutoff had the least influence on the flow quantity.

Less piezometric head loss and higher seepage quantities were calculated when assuming a constant total head at the bottom bedrock boundary as compared to an impervious boundary.

The decrease in piezometric head at the base of the clay stratum varied considerably, based on the assumed bedrock permeability and analysis boundary conditions. For the analysis that considered the highest rock permeability and an impervious bedrock bottom boundary, a total piezometric head loss was estimated within 50 feet of the excavation (zero piezometric head at the bottom of the clay). Alternatively, minimal piezometric head loss was estimated at the bottom of the clay when the lowest bedrock permeability and a constant head boundary was assumed.

TABLE 1
Summary of the Finite Element Analyses

Excavation Support/Cutoff System Details:

- Ground Surface: el. 20
- Bottom of Excavation: el. -55
- Excavation Support System:
  Reinforced Concrete Diaphragm Wall (Slurry Wall):
  Thickness: 3.0 ft.
  Depth of Embedment Below Bottom of Excavation Varied From 5 to 30 ft.

Soil Profile & Permeabilities

Subsurface Stratum	Thickness of Stratum (ft.)	Permeability (cm/sec)
Fill	15	1×10^{-5}
Clay	40	7×10^{-7}
Sand	10	1×10^{-5}
Glacial Till	15	2×10^{-6}
Bedrock	>135	Low Est.: 7×10^{-7}
		Ave. Est. 5×10^{-5}
		High Est.: 7×10^{-4}

Piezometric Boundary Conditions:

- Hydrostatic Groundwater Level: el. 10
- Bottom Bedrock Boundary: el. -195 With Two Conditions Analyzed:
  1) Impervious;
  2) Constant Head Boundary With Hydrostatic Level at el. 9

Pump Test Program

Three deep bedrock dewatering wells were installed and tested as part of an on-site pump test program. The wells were installed through approximately 100 feet of overburden soils and screened approximately 40 feet into bedrock. The well screens consisted of six-inch diameter wire wound screen. Observation wells and piezometers (as shown in Figure 3) were monitored during the pump test. Specific details of the pump test program are presented in Table 2. Results of the pump test and subsequent analytical and numerical simulation models indicated the following:

- Assuming a saturated aquifer thickness

TABLE 2
Summary of the Pump Test Program

Test Well No.	Bedrock Screen Range (el.)	Duration of Pumping (Days)	Pumping Rate (gpm)	Static Piezometric Level Prior to Tests (el.)	Drawdown at End of Pumping (el.)
TW-1	-75 to -115	5	5.9	-8.3	-51.0
TW-2	-76 to -118	4	3.4	-8.5	-84.0
TW-3	-78 to -118	5	0.9	-7.1	-94.8

approximately equal to the 40-foot well screen length, the bedrock permeability values ranged from 1×10^{-4} to 9×10^{-4} cm/sec.

- Although several factors may reduce the actual long-term flow into the excavation, it was determined that the underdrain system should be designed to accommodate permanent flows of up to 100 gpm.
- The major source of flow into the excavation would be through the bedrock. The slurry wall cutoff will serve to decrease the flow from the overburden soils into the excavation. However, the wall provides little effect in decreasing the flow from the bedrock.

Ground Settlement Due to Dewatering

As piezometric head losses develop within the clay stratum, consolidation of the clay occurs, resulting in ground surface settlement. These settlements are typically proportional to the clay stratum thickness and the magnitude of head loss within the clay stratum, which theoretically decreases with distance from the excavation.

Long-term ground surface settlements due to dewatering were calculated based on the piezometric head losses estimated from the finite element analyses. The head losses were calculated based on assuming an impervious bottom boundary in the bedrock and the low and average bedrock permeabilities as indicated in Table 1. The stress history of the clay deposit, as estimated from consolidation tests, indicated that the increased effective stress in the clay caused by dewatering would not exceed preconsolidation values. The estimated settlements ranged from 1-⅜ inches for locations nearest the site, where the clay stratum was observed to be the thickest; to ¼ inch at locations greater than 75 feet from the site, in areas where the clay layer is thinner.

The estimated long-term settlements were calculated assuming hydrostatic conditions through the clay layer. However, as indicated in Figure 4, depressed piezometric levels were measured at the bottom of the clay layer. It was not certain how long these levels were depressed. If depressed piezometric levels were assumed in the settlement analysis, settlements were estimated to be ¼ to ½ inch less than those calculated assuming hydrostatic conditions.

Groundwater Control Considerations

Using the results of the hydrogeologic studies and settlement analyses, an approach to groundwater control was adopted. It was generally concluded that significant uncertainty exists in quantifying the bedrock permeability. Although the bedrock was modeled as a homogeneous porous media, seepage quantities into the excavation could locally vary significantly depending on bedrock joint spacing, width and orientation. In order to assess this uncertainty during construction, an extensive groundwater instrumentation monitoring program was undertaken. Piezometric levels were to be monitored both inside and outside the excavation limits.

Settlement predictions of adjacent structures, due solely to consolidation of the clay,

were estimated to be tolerable. For several adjacent buildings located within 50 feet of the excavation, "worst case" foundation settlements of ¾ inch or less were estimated.

For the permanent underdrain system alternative, consideration had to be given to:

- The quantity of flow the present and future owners would be willing to provide for and maintain; and,
- The amount of discharge from the system that the associated public agency would determine to be acceptable.

Since the estimated flow quantity was relatively uncertain, the substructure design included provisions for installing either a permanent underdrained slab or a structural slab that could resist full hydrostatic uplift pressures. Economically, the permanent underdrain system was the preferred alternative. However, if during construction the quantity of flow into the excavation was deemed to be excessive, a pressure slab could be constructed with minimal delay.

Studies indicated that extending the bottom of the slurry wall deeper into the bedrock stratum did not significantly reduce groundwater flow into the excavation. Therefore, the slurry wall was typically installed at least two feet into rock. For contingency purposes, four-inch diameter PVC pipes were installed in slurry wall panels at ten-foot spacing to allow access for post-grouting below the slurry wall, if necessary. A grouting program was to be considered if isolated areas of seepage were identified below the bottom of the wall. It was envisioned that grouting would potentially decrease seepage quantities into the excavation.

Construction Dewatering Program

To maintain stable subgrades during the excavation sequence, a temporary dewatering program was implemented. The program consisted of ten dewatering wells in addition to the three existing test wells. Locations of the wells are shown in Figure 6.

Seven of the ten wells were advanced using a 17-inch diameter roller bit. The completed wells consisted of an eight-inch diameter riser pipe with screen (0.040-inch louvered) installed through the sand, glacial till and bedrock. After installing the well screen, the annular space around each well screen was backfilled with a filter-compatible clean processed gravel.

The remaining three dewatering wells (W2, W6 and W9) were installed using a three-foot wide by eight-foot long clamshell bucket excavated under a biodegradable polymer slurry ("trench" wells). The riser pipe and screen were 24 inches in diameter.

In an attempt to limit the amount of piezometric drawdown outside the project limits, the rotary-drilled wells were typically installed ten feet into rock and generally no greater than two feet below the bottom of the slurry wall (el. -70). Wells W2, W6 and W9 were installed to the lowest excavation level, approximately el.-55.

Pumping from the wells was conducted on a continuous basis that began during the excavation of the first subgrade level. The number of active wells and the pumping level within each well varied as the depth of the excavation advanced. To monitor the effectiveness of the pumping, nine vibrating wire piezometers were installed at selected locations — two in sand, one in glacial till and six in bedrock. The locations of these piezometers are also shown in Figure 6.

Results of the Construction Monitoring Program

As indicated in Figure 7, construction dewatering was initiated in August 1989 and continued until the end of June 1990. Initially, one well was used to dewater the upper excavation levels. As the excavation deepened, the number of wells in operation gradually increased to nine. Total pumping rates for all wells in operation during construction varied from three to 16 gpm. Groundwater collected during construction dewatering was pumped into a settling tank prior to discharge. Minimal fines or sediments were observed in the settling tank during dewatering operations.

In July 1990 the permanent underdrain system was installed. This system consisted of a series of six-inch diameter perforated PVC pipes embedded in a 12-inch thick layer of ¾-inch stone. Filter fabric was laid on the exposed subgrade prior to stone placement. Total flows into the underdrain system and meas-

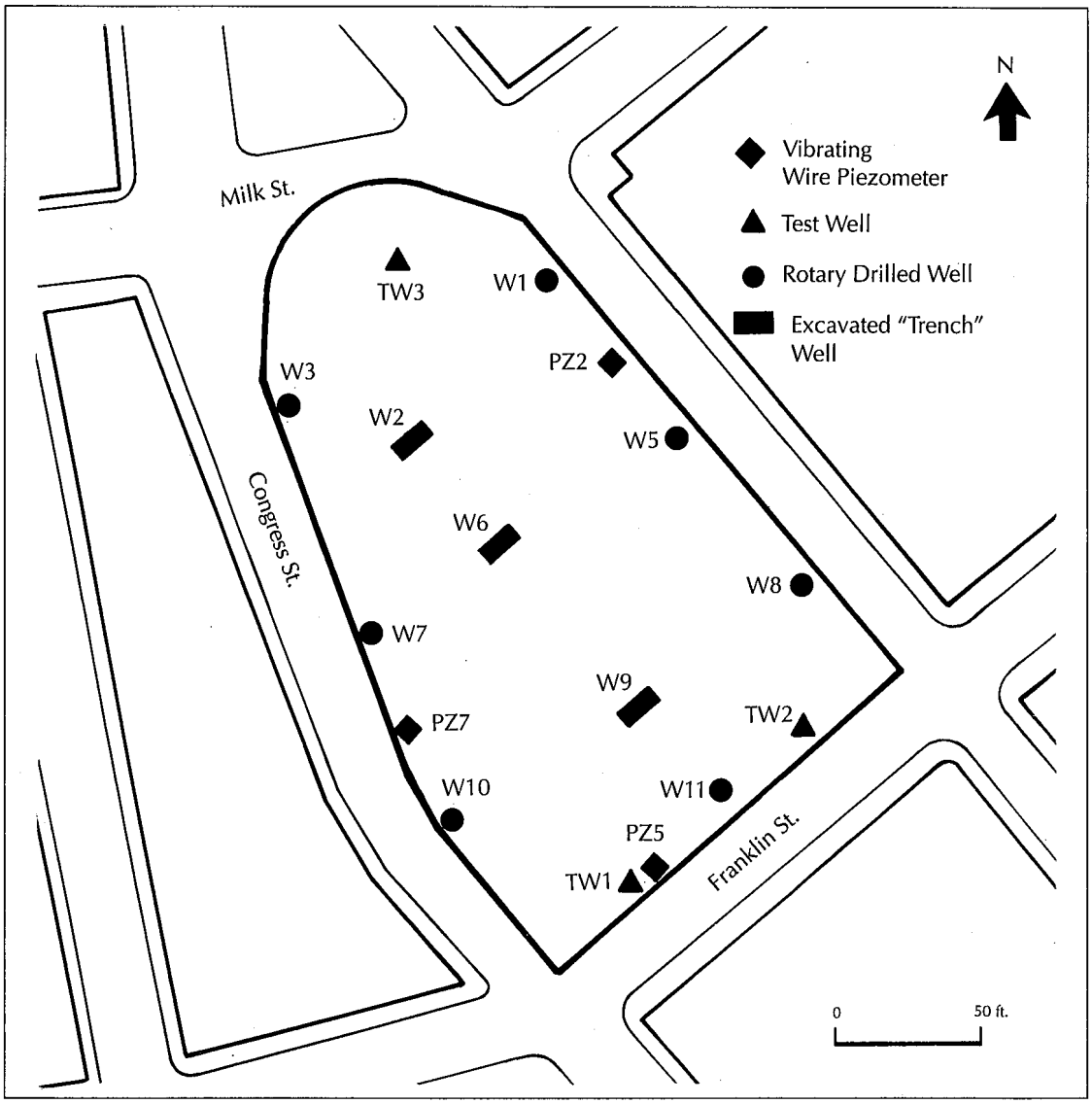

FIGURE 6. Location of the construction dewatering wells and piezometers.

ured in the sump pit ranged from eight to 12 gpm.

The results of the groundwater instrumentation monitoring outside the site are presented in Table 3. The finite element analyses performed during design had indicated that the greatest drawdown would occur in the bedrock. This drawdown was confirmed at B110-C, where a maximum drawdown of 37 feet was observed (see Figure 8). It appears that the magnitude of drawdown indicated by the bedrock piezometers may have been affected by the location of the tips since the deeper piezometers showed the greatest amount of drawdown.

Maximum drawdown in the glacial till and clay during construction was seven and five feet, respectively. Drawdown in the clay was typically minimal and appeared to depend more on the distance of the piezometer tip from the bottom of the clay layer than the distance of the piezometer from the excavation (see Table 3). Virtually no changes in groundwater levels were observed in the fill during construction.

Figures 8 and 9 present typical piezometric response to construction dewatering at various

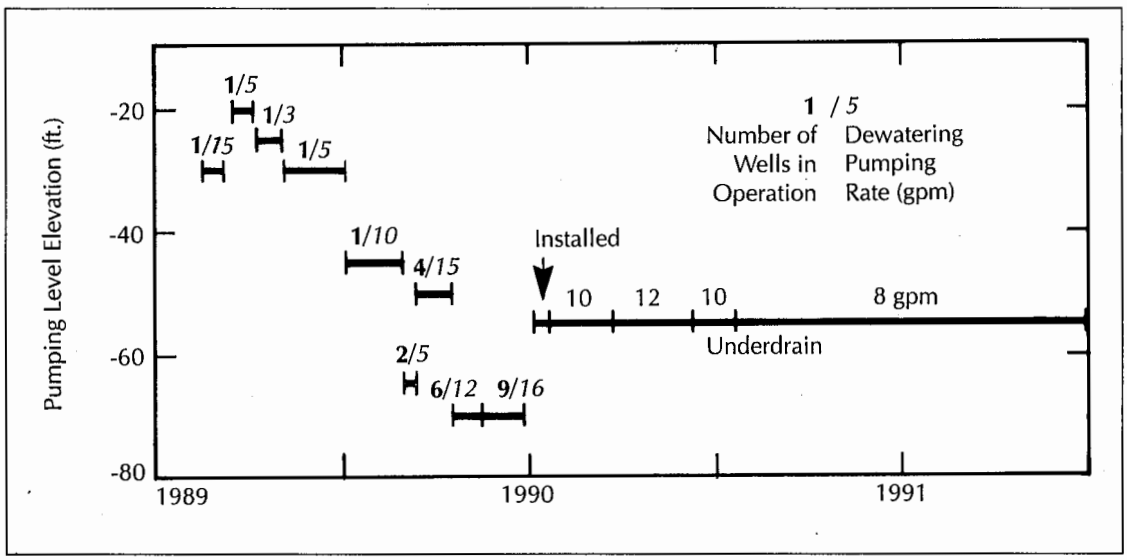

FIGURE 7. Time line of construction dewatering.

TABLE 3
Summary of Groundwater Instrumentation for the Post Office Square Garage Project

Piezometer/ Observation Well No.	Tip Location (el.)	Subsurface Stratum	Approximate Distance From Excavation (ft.)	Pre-Construction Level 10/88 (el.)	Maximum Level 6/90 (el.)	Level 12/91 (el.)	Maximum Drawdown 6/90 (ft.)	Drawdown 12/91 (ft.)
B110-OW	5.5	Fill	30	11	11	11	—	—
B113-OW	4.3	Fill	55	9	9	9	—	—
B102-OW	4.4	Fill	225	8	8	8	—	—
B106-OW	6.1	Fill	280	8	8	8	—	—
B110-PZ-A	0.3	Clay	30	11	10	11	1	—
B110-PZ-B	-18.7	Clay	30	-3	-10	-8	7	5
B108-PZ-A	-16.5	Clay	45	-3	-6	-3	3	—
B110-PZ-B	-31.0	Clay	45	-6	-17	-11	11	5
B104-PZ-A	-27.4	Clay	50	-5	-9	-6	4	1
B113-PZ-A	-10.2	Clay	55	-2	-2	-2	—	—
B114-PZ-A	5.9	Clay	115	12	10	(Destroyed)	2	—
B105-PZ-A	-1.4	Clay	130	6	6	6	—	—
B105-PZ-B	-16.4	Clay	130	-5	-8	-5	3	—
B107-PZ-A	-30.9	Clay	140	-5	-9	-5	4	—
B112-PZ-A	-9.5	Clay	215	-1	-3	-1	2	—
B112-PZ-B	-30.5	Clay	215	-8	-13	-10	5	2
B111-PZ-A	9.7	Clay	220	14	12	14	2	—
B102-PZ-A	-9.1	Clay	225	5	2	2	3	3
B103-PZ-A	-13.6	Clay	240	-4	-7	-7	3	3
B101-PZ-A	-30.9	Clay	250	-3	-5	-3	2	—
B115-PZ-A	-2.7	Clay	275	2	2	2	—	—
B113-PZ-B	-30.2	Sand	55	-6	-9	-6	3	—
B114-PZ-B	-23.6	Glacial Till	115	1	-4	(Destroyed)	5	—
B105-PZ-C	-48.9	Glacial Till	130	-8	-10	-9	2	1
B107-PZ-B	-43.4	Glacial Till	140	-8	-15	-12	7	4
B112-PZ-C	-47.5	Glacial Till	215	-8	-12	-9	4	1
B111-PZ-B	2.2	Glacial Till	220	9	(Destroyed)		—	—
B102-PZ-B	-28.1	Glacial Till	225	-5	-12	-12	7	7
B103-PZ-B	-36.1	Glacial Till	240	-7	-7	-7	—	—
B101-PZ-B	-55.4	Glacial Till	250	-7	-7	-7	—	—
B115-PZ-B	-13.2	Glacial Till	275	-5	-5	-5	—	—
B109-PZ-A	3.3	Glacial Till	280	12	12	12	—	—
B109-PZ-B	-3.2	Glacial Till	280	11	11	11	—	—
B106-PZ-A	-34.4	Glacial Till	280	-15	-15	-15	—	—
B110-PZ-C	-83.2	Bedrock	30	-8	-45	-40	37	32
B108-PZ-C	-71.0	Bedrock	45	-8	-36	-26	28	18
B104-PZ-B	-70.9	Bedrock	50	-5	-30	-25	25	20
B113-PZ-C	-62.2	Bedrock	55	-8	-16	-12	8	4

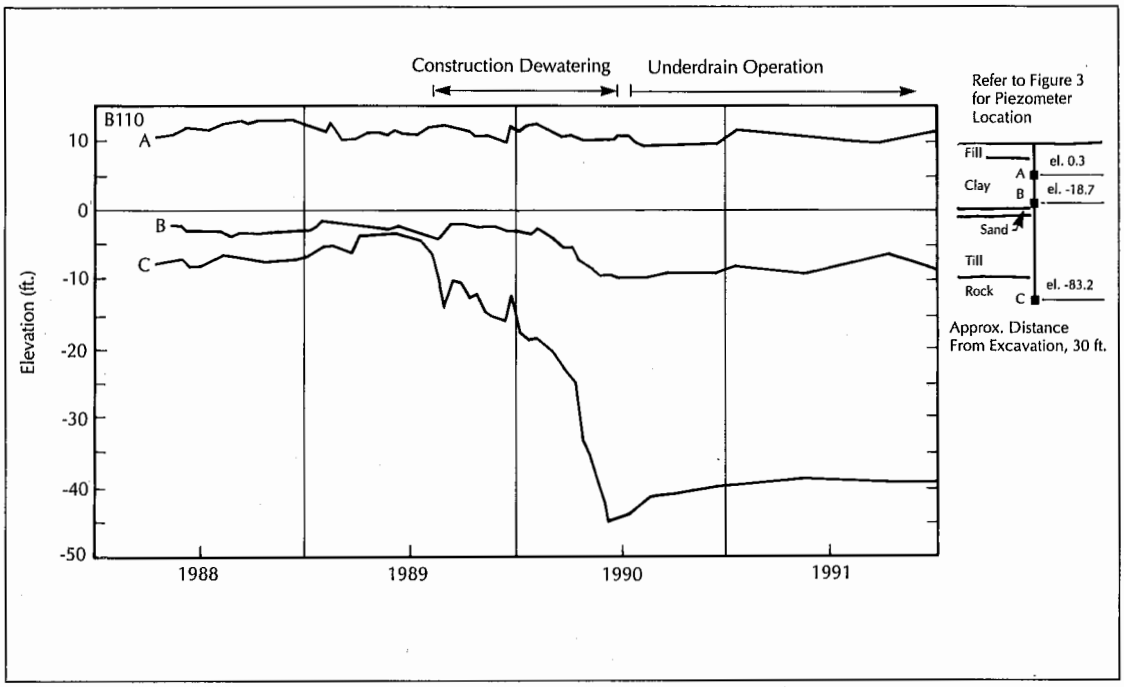

FIGURE 8. Piezometric head versus time for B110.

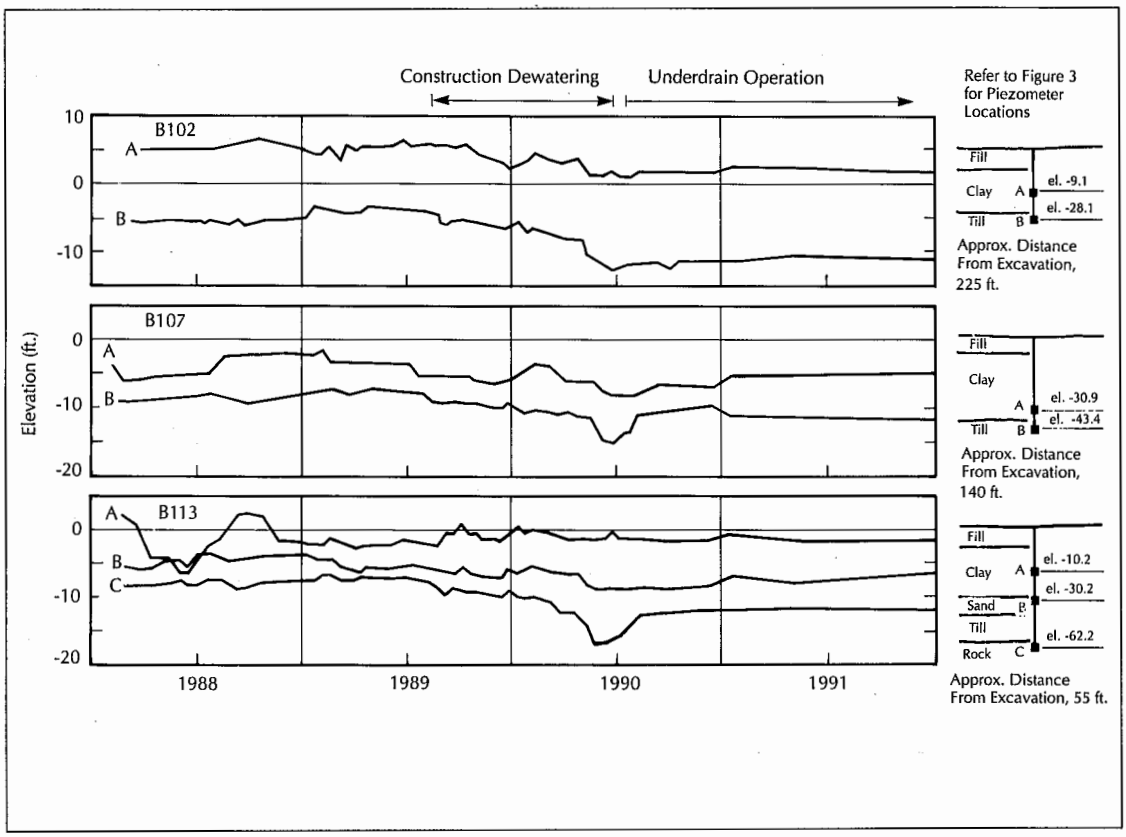

FIGURE 9. Piezometric head versus time for B102, B107 and B113.

instrumentation locations. The piezometers shown were selected to illustrate the effects of the varying subsurface conditions surrounding the site.

Piezometers located in boreholes B102 and B107 represented the subsurface conditions northeast and west of the site where the clay stratum directly overlies glacial till. As indicated in Figure 9, piezometers located in the clay and glacial till responded to construction dewatering and data approximately paralleled each other.

Piezometers located in boreholes B110 and B113 represented subsurface conditions at the southern end of the site where a sand layer existed between the clay and glacial till strata. The data collected at these piezometers (see Figures 8 and 9) show a significant response to construction dewatering in the bedrock. However, due to the relatively pervious sand layer and the less pervious glacial till below, proportionally less head loss was observed within the clay. It is likely that the sand layer served to provide recharge to the bottom of the clay layer and, thus, reduced drawdown in this stratum.

Only three of nine piezometers installed within the site provided continuous data throughout construction as shown in Figure 10. The other piezometers were destroyed or temporarily obstructed or disabled during excavation operations.

The data presented in Figure 10 indicate the relatively rapid response of piezometric levels within the site with increased dewatering effort. However, during some periods of constant construction dewatering, drops in piezometric levels within the site appeared to have been related to increased excavation depth. This behavior was generally observed during the last three levels of excavation in mid-1990 when the level of dewatering within the site remained constant at approximately el. -70. These data would suggest that a relatively effective cutoff was provided by the slurry wall.

Figure 10 also indicates piezometric levels of up to eight feet greater than the existing subgrade level during May/June 1990 at piezometer PZ2. Although the elevated piezometric level at this location was of concern, no subgrade instability was observed and the piezometric head dropped during the next stage of excavation. Upon completion of the excavation, all three piezometer locations indicated piezometric levels above the subgrade level, reflecting seepage gradients below the underdrain system.

To relate the effects of groundwater drawdown with adjacent ground surface settlements, nine surface reference points were selected from points that were installed to monitor settlements near the site. The selected points are shown in Figure 11 and were installed prior to construction in September 1988 and were monitored until mid-1990. The nine points were selected since they were located near the corners of the slurry wall and were assumed to be essentially independent of slurry wall-related movements. The points were also located at least 100 feet from the excavation.

Measurements taken as of July 1990 indicated negligible movements at points 24, 42 and 62. Settlements at points 1, 2, 3, 22, 41 and 54 were approximately ¼ inch. These recorded settlements were generally within preconstruction estimates.

The maximum settlement for an adjacent building was ½ inch for a structure located approximately 50 feet from the excavation. Settlement of this structure was believed to be primarily related to lateral movements of the slurry wall.

Conclusions

Significant uncertainty exists in estimating the effects of groundwater control during the construction of a deep excavation. This uncertainty is typically related to the inability to accurately determine subsurface soil and rock permeabilities. For the Post Office Square Garage project, this uncertainty was compounded since the bottom of the excavation cutoff was embedded in bedrock.

Unlike the determination of flow through porous media, bedrock permeabilities may be significantly influenced by joint spacing, width and orientation. In addition, boundary conditions for analytical and numerical modeling are difficult to determine due to the potentially extensive depth of bedrock aquifer.

In general, the effects of groundwater control on excavations is directly related to specific

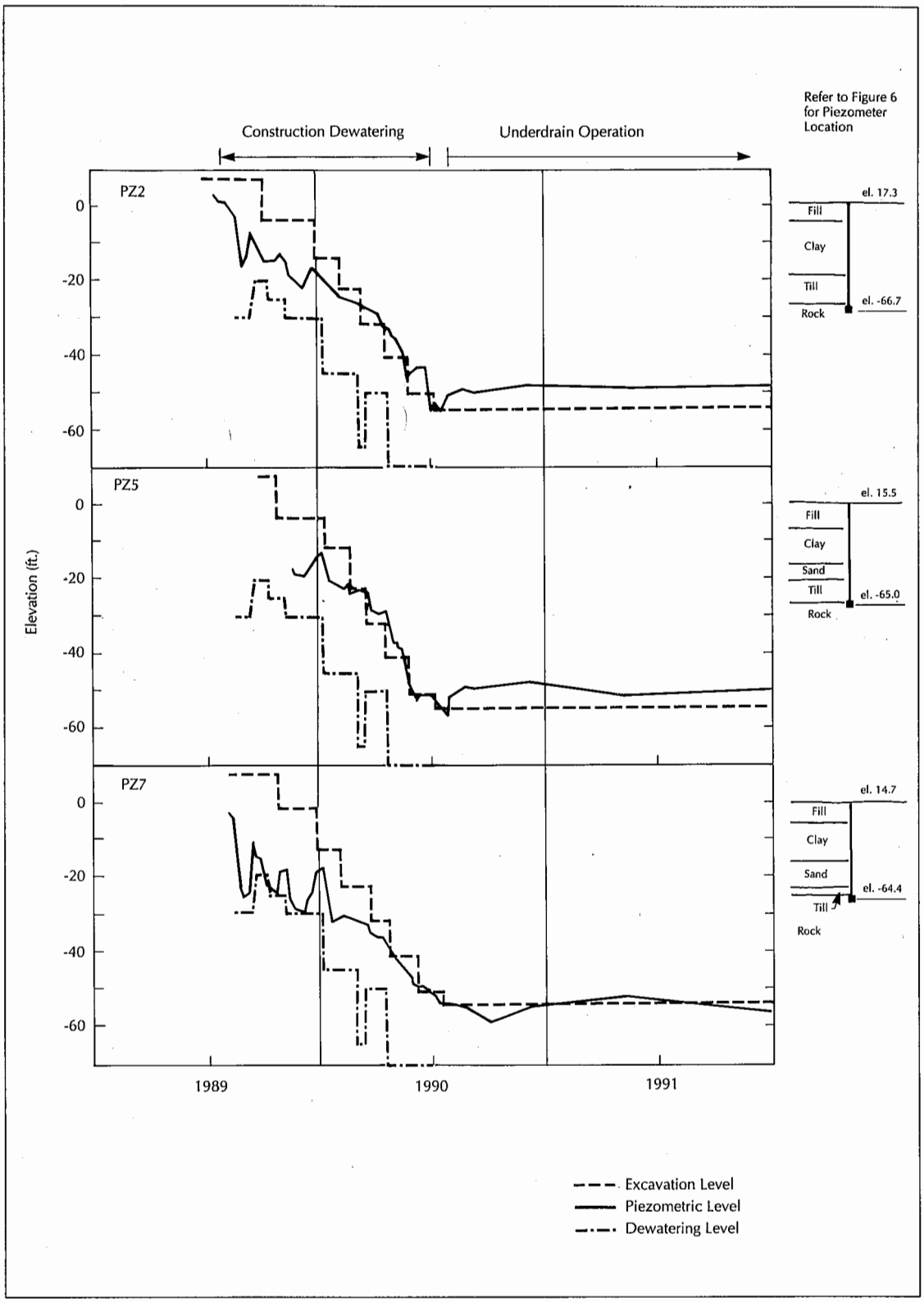

FIGURE 10. Excavation, piezometric and dewatering levels versus time within the project site.

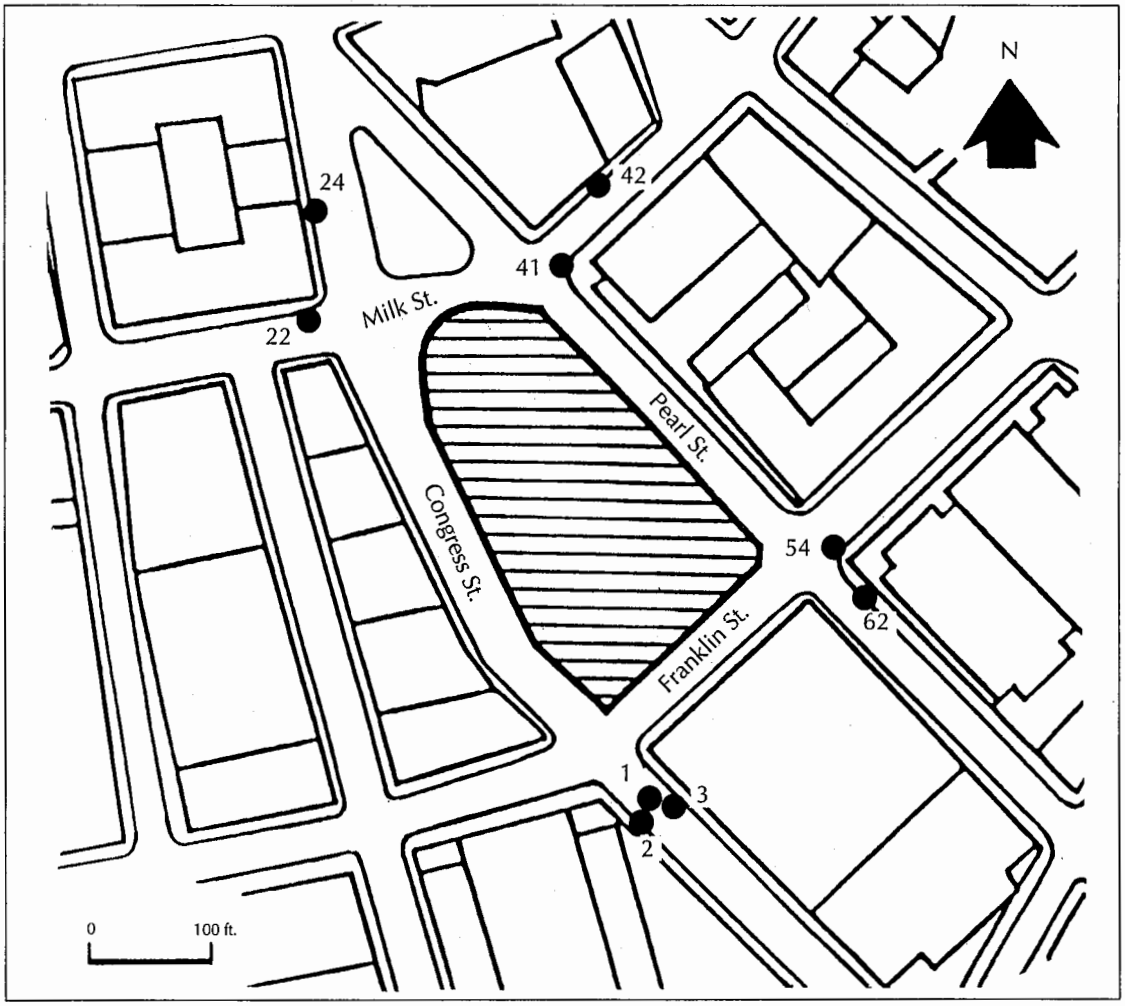

FIGURE 11. Selected surface reference points.

site details and should be investigated on a case-by-case basis for each excavation. Such details may include:

- The relationship between the depth of cut off and subsurface conditions;
- Areas of potential groundwater recharge;
- Adjacent foundation types; and,
- Existing structures that may act as conduits of groundwater.

As indicated by this project, a monitoring program should be developed to evaluate design assumptions for situations that contain significant uncertainty. Contingencies can then be developed that can be effectively implemented if the conditions that are encountered during construction vary significantly from those that have been assumed.

NOTES — *All elevations referred to here are referenced to Boston City Base Datum (BCB). Finite element analysis software used for the project was FLOWNET; the graphical presentation software was FLOWPLOT.*

ACKNOWLEDGMENTS — *This article was prepared with the assistance of a grant from the John R. Freeman Fellowship Fund of the Boston Society of Civil Engineers. The authors are grateful to the Freeman Fund Committee for its assistance and support. The organizations that participated in the project were: Ellenzweig Associates, Inc., Architects; LeMessurier Consultants, Inc., Structural*

Engineer; Haley & Aldrich, Inc., Geotechnical Engineer; Parsons Brinckerhoff Quade & Douglas, Inc., Civil Engineer; J.F. White Contracting Co., General Contractor. The authors would like to thank the project owners, The Friends of Post Office Square, specifically James Becker, for permission to publish this article. The authors also gratefully acknowledge their colleagues at Haley & Aldrich who assisted in the Post Office Square Project: John Kastrinos, pump test program; Cristian de la Huerta, groundwater modeling; Chris Kittredge, instrumentation monitoring. Thanks also to Edmund Johnson for his thorough review of this article and to Acey Welch for preparation of the illustrations.

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