Geotechnical Factors in Boston

The geotechnical factors associated with Boston's complex soils and bedrock need to be understood to design and build suitable structures.

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It is critical to understand the important geologic constraints of the various soil units and the geologic control for appropriate foundations, as well as the engineering properties of the overburden soils and bedrock and their influence on constructed facilities. The geotechnical factors associated with, and caused by, the complexity of the soils and bedrock underlying the Boston area include: bearing capacity (sometimes overestimated, leading to settlement), behavior of foundations in certain problematic soils, defining the rock/soil interface surface, the nature of the bedrock that can be soft as clay and groundwater conditions. (This discussion expands on a previous summary by E.G. Johnson and other workers at Haley & Aldrich, Inc., as well as Woodhouse [in Woodhouse & Barosh, 1991].)

Geologic Constraints

The Boston area is spared many of the geologic hazards of the type that plague other parts of the nation other than those imposed by occasional coastal storms accompanied by local flooding, and moderate earthquakes. Surface fault rupture, volcanism and land failure due to landslides, expansive soils or natural subsidence are unknown in the Boston area. However, when considering the geology of the Boston Basin and its influence on the development of Boston, it is seemingly paradoxical to discover that a large metropolitan area has flourished within a coastal basin where marine and glacial processes, in combination with significant past tectonic deformation, have produced one of the most geologically complex areas on the eastern seaboard.

The variable condition of the overburden and the complicated nature of the bedrock geology of the Boston Basin imposed limitations on urban construction that hindered the city's early development. Because of these conditions and the efforts necessary to overcome them, many early advances were made in the world-wide body of knowledge on subsurface exploration, soil mechanics and geotechnical engineering. Today, thanks to this pioneering work, the stratigraphy and engineering properties of the overburden and bedrock in the Boston area are, for the most part, understood and easily explored for any given site using state-of-the-art methods of subsurface exploration and geotechnical laboratory testing.

FIGURE 4-1. Colonial shoreline superimposed on a modern map.

Unique or unusual elements of the geologic setting and subsurface conditions of the Boston Basin have influenced foundation designs and construction practices. Unfortunately, there are still too many times when challenging geologic conditions are discovered late, or are "unseen" and discovered only during construction.

Fill & Organic Sediment. One of the more unusual features of the Boston area, and one that continually presents problems to foundation design in and surrounding Boston, has been the extensive areas reclaimed by filling since Colonial times (see Figures 1-21 & 4-1). The reclaimed lands include the extensive tidal marshes around Boston and along estuaries and rivers entering Boston Harbor (see Figure 1-17). Large tracts of salt marsh still remain outside the city where reclamation focused on draining the areas sufficiently to produce salt marsh hay.

The nature, quality and thickness of fill in and around Boston vary greatly from place to place. Fill encountered in Boston may consist of till excavated from original hills, local sand and gravel, trash, cinders and ash, miscellaneous rubble from construction demolition, silt and clay dredged from Boston Harbor or from excavations, imported sand and gravel, or any combination of these. The heterogeneity of fill in Boston is further complicated by the presence of old wharfs and bulkheads that have been buried by later extension and improvement of the Boston waterfront. Problems associated with potential obstructions in the fill — which may preclude or restrict the use of steel sheeting, tiebacks or piles — must be addressed in the design of foundations. A thorough and comprehensive chronology on the land filling within Boston is given by Seasholes (2003) and that book should be referenced for any site development being planned within the city of Boston.

Fill is often underlain by highly compressible deposits of peat, organic silt and other similarly compressible soil strata such as clay, especially in reclaimed salt marshes. These organic materials create problems associated with foundations, and foundation design must take such conditions into account. Also, the consequences of significant settlement due to the placement of additional fill over existing compressible deposits must be considered wherever site development occurs.

Upper Outwash. The upper outwash sand and gravel that is associated with the Lexington Substage is only sporadically found in the Boston area. The thickness based on the results of test borings can reach up to 6 meters (20 feet), as found in the Copley Square-Christian Science Church area. Geotechnical engineers have taken advantage of this shallow deposit to design buildings bearing on pressure-injected footings, especially in the area of the Christian Science Church complex (see Tables 4-1 & 4-2) with no known problems in the downtown area. However, foundation

TABLE 4-1.
Allowable Bearing Pressures for Foundation Materials in Massachusetts Earthquake Design Codes

Material Class	Description	Consistency in Place	Allowable Net Bearing Pressure (tons/ft^2)
1	Massive bedrock: granite, diorite, gabbro, basalt, gneiss.	Hard, sound rock minor jointing	100
	Quartzite, well-cemented conglomerate.	Hard, sound rock moderate jointing	60
2	Foliated Bedrock: slate, schist.	Medium hard rock, minor jointing	40
3	Sedimentary bedrock: cementation shale, siltstone, sandstone, limestone, dolomite, conglomerate	Soft rock, moderate jointing	20
4	Weakly cemented sedimentary bedrock: compaction shale or other similar rock in sound condition.	Very soft rock	10
5	Weathered bedrock: any of the above except shale.	Very soft rock, weathered and/or major jointing and fracturing	8
6	Slightly cemented sand and/or gravel, glacial till (basal or lodgement), hardpan.	Very dense	10
7	Gravel, widely graded sand and gravel, and granualr ablation till.	Very dense Dense Medium dense Loose Very loose	8 6 4 2 *
8	Sands and non-plastic silty sands with little or no gravel (except for Class 9 materials).	Dense Medium dense Loose Very loose	4 3 2 *
9	Fine sand, silty fine sand, and non-plastic inorganic silt.	Dense Medium dense Loose Very loose	3 2 *
10	Inorganic sandy or silty clay, clayey sand, clayey silt, clay, or varved clay; low to high plasticity.	Hard Stiff Medium Soft	4 2 *
11	Organic soils: peat, organic silt, organic clay.	—	*

Note: * Requires evaluation by registered professional engineer.

TABLE 4-2.
Recommended End Bearing & Side Friction for Piles

Depth Into Rock (m)	End Bearing (kPa) at 6.35 mm Downward Socket Movement			End Bearing (kPa) at 12.7 mm Downward Socket Movement		
	Bedrock Area					
	A	C	D	A	C	D
0 - 1.5	—	—	—	—	—	—
1.5 - 3	1,910	—	—	3,830	—	—
3 - 6	2,870	480	385	5,750	575	575
6 - 9	2,870	1,440 - 2,870	670	5,570	1,910 - 3,830	960
>9	—	2,870 - 4,310	1,440	—	3,830 - 5,750	1,910

Depth Into Rock (m)	Side Resistance (kPa) at 6.35 mm Downward Socket Movement			Side Resistance (kPa) at 12.7 mm Downward Socket Movement		
	Bedrock Area					
	A	C	D	A	C	D
Glacial Till	—	140	140	—	170	170
0 - 1.5	140	140	140	205	170	170
1.5 - 4.5	690	170	170	830	205	205
4.5 - 7.5	690	205	205	830	275	240
>7.5	—	205	205	—	275	240

Note: From Gorczyca *et al.* (1999)

construction problems arose earlier on some projects in this area, with older buildings presumably because the outwash's presence was not known and wood piles that were to bear on or in the marine clay unexpectedly "took up" in the outwash. The results were "brooming" of the tops of piles and displacement of some piles which, in the case of the Christian Science Church's new facade where the old pile caps were exposed, ended up as battered. Some of the piles under the church piles were actually pulled out of the pile cap, which was attributed to negative skin friction.

Marine Clay. The marine clay, known as the "Boston Blue Clay," can cause problems. In some areas of the Boston Basin, the clay deposit is extensive, thick and homogeneous, but elsewhere lenses of other material are present within the clay and the stratigraphic sequences are difficult to explain or predict. The local presence of till-like material that usually occurs as lenticular masses of limited thickness and lateral extent can be of concern. These layers may be easily misinterpreted in borings as being the lowermost till sequence that generally blankets the bedrock surface. Such a layer was encountered during the preliminary phase test borings for the Massachusetts Bay Transportation Authority (MBTA) Southwest Corridor Project east of Back Bay station in 1977. Design phase test borings showed this apparent till to be underlain by 6 meters (20 feet) of marine clay (Lambrechts, 2012). If

buildings and other structures were to be founded on or within these "apparent till" sequences that overlie soft clay, unacceptable structural settlement may occur. To avoid this problem, foundation borings need be advanced to a sufficient depth to investigate for any underlying, low-strength soils and to penetrate bedrock, thereby verifying the presence of a firm-bearing basal unit.

Another recurring problem is caused by the local interbedded, clean, permeable sand or gravelly sand layers. These layers range from less than a centimeter to over a meter (0.4 inches to several feet) in thickness, and are typically laterally discontinuous over relatively short distances. When undetected in exploratory borings, these layers are often the source of unexpected water inflow into excavations. Thicker layers, if not properly supported and dewatered, are easily undermined or otherwise disturbed. The sand layers pose similar problems in jetted pile operations, slurry wall construction and caisson installation. These layers within the clay also have been responsible for stability problems in tunneling projects and have occasionally caused erroneous interpretations of bearing values during pile driving.

Till. Significant variations in the stratigraphy and engineering characteristics of till require it to be thoroughly investigated for all moderate to heavy load-bearing foundations in the basin. The variable nature of till deposits must be addressed in foundation design since the bearing strength and engineering characteristics of the lodgement till can vary from place to place due to the composition and characteristics of the till matrix. For example, where till is almost entirely derived from less-resistant rock of the Boston Basin, degradation and incorporation of this rock produced a deposit with an appreciable amount of plastic silt and clay. Where the till is largely derived from granitic or other intrusive rock from areas adjacent to the Boston Basin, or even the granular Roxbury Conglomerate, the deposit tends to be silty or sandy with little or no plastic material. This condition was reported along the MBTA Southwest Corridor Project in Roxbury in the early 1980s where the bedrock changes from argillite to conglomerate and notably coarser till was excavated (Lambrechts, 1983). Variation in the matrix characteristics of till deposits may control the effort needed in excavation, the allowable bearing pressures, the potential for foundation settlement, the pile types needed and the dewatering requirements.

Occasional, localized pockets of sand, gravel or other permeable material within the till, especially those not revealed until construction begins, pose another concern in constructing foundations (see Figure 3-76). Such pockets have resulted in water inflow to excavations and installation problems for certain types of piles, caissons and slurry walls. The anomalous lenses are remnants of retreat deposits, likely glaciofluvial, between the two tills, or material deposited beneath glacial ice. These lenses are commonly not detected by exploratory boring due to sample spacing, poor recovery or misinterpretation of recovered soil.

Till-Bedrock Contact. Problems arise about the till-bedrock contact in locating the actual contact, determining the character of the contact and evaluating the water flow potential at the contact. Till usually contains boulders and dislodged wedges and slabs of bedrock. Where the lithology of a large boulder is identical to that of the bedrock, which is a common occurrence, boulders can be erroneously interpreted as the top of bedrock, when intersected in boreholes. To prevent this mistake, rock should be cored a minimum of 3 meters (10 feet) to differentiate between boulders and actual bedrock. False interpretation of the bedrock surface may result in piles and other deep foundation supports that require deeper and more costly penetration than originally expected.

The character of the contact between till and bedrock can vary considerably in the Boston area depending on the rock hardness and integrity. Where bedrock is hard and cut by only a few joints, the till rock contact tends to be distinct with little or no transitional material. Where loosened slabs rest on bedrock, identifying the true bedrock surface may be problematic. The contact appears gradational where bedrock is relatively soft, highly fractured or has undergone pre-glacial weathering. The latter transition may range

over 1 to 3 meters (3 to 10 feet) or greater. This zone is typically characterized by an increase of the size and frequency of rock fragments in the till with depth and may contain large blocks or slabs of the underlying bedrock at its base. Seams of till have been encountered in exploratory borings as much as 5 to 6.5 meters (15 to 20 feet) below what would otherwise be considered the top of competent, intact rock. It may be difficult to distinguish borehole occurrences of thin till layers that are beneath basal slabs and till surrounding boulders from fracture fillings in intact bedrock. This situation occurred in the excavation in Somerville for the MBTA Davis Square Red Line Station. Clay infillings of fractures may also be interpreted as gouge.

A unique geotechnical problem is associated with the till-bedrock contact on some construction projects. The "top of rock" shown in contract drawings for foundation design purposes often corresponds to an elevation where competent or intact bedrock was encountered in exploratory borings. However, the transition zone immediately above this elevation may be difficult to excavate using conventional drilling and sampling equipment. As a result, the position of the contact may become the subject of heated debate between engineers and contractors during construction when its elevation forms the basis for payment for rock excavation for basements, slurry walls, and shafts and tunnels because rock excavation is usually five to ten times more expensive, particularly when blasting is required.

Top of rock on some projects might be defined by the amount of effort required to excavate rock with heavy construction equipment. However, the character of till or bedrock at such excavations may be inadequate in terms of foundation support requirements. Therefore, the excavation requirements for each project must be clearly established in geotechnical reports and contract documents. The till-bedrock contact in some areas of the Boston Basin is a highly permeable zone that has caused water inflow problems in excavations. This inflow may result from:

- greater frequency of fractures in the upper meter (few feet) of bedrock due to pre-glacial weathering, freeze and thaw, residual strain release or strain from glacial loading or subsequent elastic rebound;
- altered permeability characteristics of the base of the till due to groundwater seepage for a number of centuries over the relatively impermeable rock surface, perhaps when the sea level was lower than the present;
- coarser matrix around the rock debris at the base of the till;
- coarser water-laid deposits from sub-glacial channels; and,
- permeable, pre-glacial residual weathered material at the top of the bedrock.

Whatever the cause, the recurrence of this problem at this till-bedrock contact zone on many construction projects over the years suggests that design schemes should incorporate the need for dewatering where this condition may exist.

Properties & Condition of Bedrock. The bedrock in the Boston Basin generally has adequate strength to support most engineered structures. The great length penetrated by the numerous bedrock tunnels in the Boston area and the many high-rise buildings founded on rock provide testimony to the overall structural integrity of the bedrock. However, faulting and soft-rock alteration have occasionally required design modification for tunnel support and for deep foundations supported on rock.

Soft-Rock Alteration. By 1914, geologists had noted that in certain areas of the Boston Basin (see Figures 3-48 & 3-49) the normally hard Cambridge Argillite had been altered to "whitish and more or less plastic clay" (Worcester, 1914). This alteration is due to kaolinization, produced by either deep-weathering or hydrothermal alteration (Kaye, 1967a). The effects produced by the kaolinization on the engineering properties of the argillite and other basin rocks depends on the degree of alteration. Slightly altered rock tends to retain its original color and brittle strength properties. As the degree of alteration increases, there is a corresponding but slight decrease in unit weight and considerable loss of strength. Where alteration is extreme, the rock is very soft and somewhat plastic, and

when dry, is very light in color, porous and chalk-like. The advanced stages of alteration produce a weak rock with strength and consolidation characteristics similar to that of over-consolidated clay as shown by triaxial compression tests.

Where the degree of alteration is advanced, conventional diamond rock coring equipment can sometimes deteriorate, break up or smear rock core, masking the structural characteristics and altering the engineering properties. Special drilling techniques are needed to obtain undisturbed samples of altered rock for either consolidation or triaxial testing. Pitcher-type sample barrels or saw-toothed carbide bits used in conjunction with conventional coring equipment have been successful in preserving the properties of the extremely altered bedrock. These sampling techniques usually show relict structures of the rock mass, such as jointing, foliation and bedding. The alteration appears to have been caused by leaching that has not disrupted the rock fabric or structural discontinuities, regardless of the degree of alteration. Design requirements for tunnels and deep foundations constructed on kaolinized rock vary with the degree of alteration. In rock-socket testing for the Central Artery/Tunnel Project at Fort Point Channel, deeply weathered argillite was determined to have only 10 percent as much end bearing capacity for drilled shafts as did hard argillite on the 27-acre site (see Table 4-2) of the Interstate 90/Interstate 93 Northbound Interchange Project (Gorczyca *et al.*, 1999). Tunnels encountering weak, extremely kaolinized zones have needed the use of steel support and concrete lining (Kaye, 1967a). The few high-rise buildings in Boston that are wholly or partially founded on altered bedrock have incorporated the use of belled piers, mat foundations and, in one instance, socketed piles to provide adequate support of these structures.

Faults. The bedrock of the Boston Basin has been broken by several episodes of faulting to form a mosaic of fault blocks. These faults include the principal high-angle, east-northeast trending faults and the myriad later complex series of intersecting transverse faults of smaller lateral displacements (Kaye, 1980b). The nature of the faults is displayed in the approximately 100 linear kilometers (60 miles) of bedrock water supply and drainage tunnels constructed in the basin that afforded an opportunity to observe the nature of faulting. Most fault displacement (where measurable) in these tunnels is limited, less than 3 meters (10 feet), and produced only minimal breccia and gouge. The majority of faults observed in these tunnels are characterized as localized zones of closed, tight fractures. Breccia and fault gouge (where present) is limited in thickness to a few decimeters or less. However, several of the large faults encountered in the tunnels are characterized by wide zones of highly fractured rock and structural steel support was needed in some of these zones, for distances up to about 100 meters (300 feet). However, modern tunnel design generally does not require such a heavy degree of ground support in similar rock. Most of the faults are characterized by minor fracturing and limited displacements and do not require substantial modification of support requirements for either tunnels or foundations. Due to the exhaustive compilation of boring logs, geologic maps and construction records over the past forty years, most of the large faults of the Boston Basin have been located. The potential for adverse bedrock conditions in these areas calls for consideration of their effects, as weak-rock zones, in designing construction projects. Additional large faults and many small faults are not yet mapped and, therefore, can lead to design modification during construction.

Artesian Conditions. Natural and man-made artesian conditions have caused failure in the Boston area. The natural ones occur along major buried valleys, four of which extend into the Boston Basin, and roughly correspond to present drainage courses and rivers flowing toward Boston Harbor (Upson & Spencer, 1964). Depths of these valleys extend up to 76.2 meters (250 feet) below mean sea level (MSL). However, glaciers have eroded some of these valleys to produce local enclosed basins (Kaye, 1982b). The unconsolidated deposits within these valleys tend to be locally complex, but the stratigraphy is generally similar to depositional sequences observed elsewhere in the Boston Basin. The valleys are filled typically by sequences of alternating marine clay

and outwash, underlain by basal till. Estuarine silt, peat and alluvium commonly form the surficial strata adjacent to present-day streams and their tributaries. Groundwater in the outwash layers is commonly under artesian conditions due to confinement by relatively impermeable clay or estuarine deposits. Because of a general inclination of flow gradients toward the ocean (Cotton & Delaney, 1975), it has been observed that upward gradients from a steep hydraulic gradient in a kame moraine in Plymouth, Massachusetts, produced artesian flow at the ocean surface with attendant erosion of ground.

Buried valleys outside of Boston generally have not been sites for large-scale underground construction. Excavations for subways and large buildings at the fringe of the Boston Basin, however, have locally penetrated outwash deposits overlain by impermeable clay or estuarine deposits, and have encountered unexpected artesian conditions. If artesian conditions are not properly considered in foundation design, they may lead to troublesome water inflow into excavations, disturbance of the subgrade by seepage or bottom heave and contribute to excessive uplift pressure beneath certain foundation types.

Woodhouse observed that artesian pressure from the unlined Dorchester water-supply tunnel drilled in the Cambridge Argillite south of the city caused considerable damage to the basements of homes that were about 30 meters (100 feet) above the tunnel and hundreds of feet away. The tunnel failed under hydrostatic pressure when the rock expanded, causing the joints to both open and close, which allowed the pressurized water to migrate along the joints to the surface (in some cases the surface is a hundred feet above the tunnel) and harm residences. Water inflow along a fault zone in the construction of the Deer Island Inter-Island Tunnel also caused major problems.

Slope Stability. Rock falls occurred in Acadia National Park, Maine, from a local earthquake in 2007, but rock falls and slides in highway cuts and excavations are rare in the region around Boston with or without earthquakes. Most rock falls are very small and occur in highway cuts containing joints that are adversely dipping into the cuts. The few rock bluffs present appear stable and granite quarries south of the city maintain very high rock faces.

In 1994, a toppling failure in a 9 to 12 meter (30 to 40 foot) high cut of fractured felsite of the Lynn volcanic rock (being quarried for use as "road metal") occurred in a quarry on Neal Street in Malden (McKown, 2012). The failure dumped more than 230 cubic meters (300 cubic yards) of rock and soil onto a residential property, destroying a car and a house. Fortunately, there were no injuries or loss of life. A contractor doing nearby construction work using a hoe ram to excavate the bedrock with resulting vibrations was held liable. However, the failure was more likely caused by water in joints in the bedrock from heavy rain over a period of a week.

The Massachusetts Turnpike (Interstate 90) from Boston to Auburn has numerous cuts in batholithic granite and metamorphic rock that have planes dipping adversely down toward the roadway (Barosh, 1976c & 1996a), chiefly in cuts along the southern side (outer edge of eastbound travel lanes). In the western portion, a parallelism of northerly-dipping foliation, shears, joints and small faults may result in open fractures along which the rock slides to the north into the roadway. Cuts along the turnpike in central Massachusetts intersect certain units of the Brimfield Group containing abundant sulfide minerals. The rock is hard, resistant gneiss that stands up well in high cuts when fresh, but it crumbles and blocks may fall owing to a general disintegration as the sulfides weather (Pease, 1975). Despite their name, most rock slopes are not composed of solid, homogeneous rock. They consist of intact rock separated by numerous joints, bedding planes, plants, shear zones and faults. These discontinuities can form blocks and wedges that slide or topple from the rock slope to the roadway or railway below when disturbed. There are three main mechanisms — planar, wedge and toppling — for slope failure (McKown, 2012).

Wedge failure has resulted in rock falls along highways. These failures occur where two intersecting joint planes form a wedge of rock that slides along the angle of intersection

and out of the rock face. In the 1960s, a large wedge failure deposited approximately 765 cubic meters (1,000 cubic yards) on the turnpike near Westfield, Massachusetts, killing a motorist. In the 1980s, the cut for the eastbound on-ramp to Massachusetts Turnpike in Auburn was closed for a week by a simple slide along northwest-dipping planes. In 1997, an extensive failure occurred along a 92 meter (300 foot) long portion of rock face in the same area, when rock slid on a continuous joint dipping toward the Massachusetts Turnpike, dumping more than 10,700 cubic meters (14,000 cubic yards) of rock on the heavily traveled on-ramp roadway. The rock slope failure closed the eastbound on-ramp of Exit 10 for ten days, and cost the Massachusetts Turnpike Authority approximately $230,000 to clean up and repair. The rock cut slopes failed due to hydrostatic pressure buildup (McKown, 2012).

The Massachusetts Turnpike undertook a multi-year rock cut slope scaling and stabilization program from 1992 to 1998 in order to stabilize rock cuts that by then had been exposed for about forty years. However, in spite of these efforts, another rock slope failure occurred in 1997, attesting to the unpredictability of rock slopes and the difficulty of prevention. Many small individual blocks at joint intersections above such planes come down during the winter and spring along the turnpike and other roads. These rock falls can be managed and largely prevented by simple periodic inspection and removal of rock blocks before they fall and bounce onto the roadway. Some commercial developments also have cut into hillsides undermining the toes of such dipping planes and making parking lots and buildings vulnerable to such rock falls (McKown, 2012).

Landslides are common in the marine clay, the Presumpscot Formation, in the Rockport area of the central Maine coast (Berry *et al.*, 1996). The Presumpscot is Maine's "Blue Clay" and is glaciomarine in origin, similar to Boston's marine clay. However, there are no problems around Boston with the clay because it is lower and does not form coastal bluffs susceptible to slides. However, if a high face of clay were exposed, a problem could develop. Failure of this clay did occur during construction at the Portsmouth, New Hampshire, Interstate 95 interchange, which was rectified by the installation of sand drains.

Standard Penetration Test for Soil Property Assessment

Foundation engineers need a system to estimate the density and friction angle of granular material and, in the case of cohesive soils, their consistency. Charles Gow, of Raymond Concrete, in 1902 and Harry Mohr (consulting engineer) in the 1920s, in concert with the drilling firm of Sprague and Henwood, developed equipment and techniques to sample soils. Mohr used the blows from a 63 kilogram (140 pound) hammer falling 76 centimeters (30 inches) onto a split spoon sampler (the *N*-value) to estimate the strength of the material. Karl Terzaghi, after meeting with Mohr, designated the technique as the Standard Penetration Test (SPT) in 1947 that is still used at present. In the case of granular soils, the presence of cobbles and boulders can inflate the SPT's *N*-value as the sampler meets resistance and give a falsely dense rating. The soil logger can mis-classify soils in the sampler if the person fails to distinguish the wash from the actual sample. In addition, driving the sampler through loose soils underlain by denser soils can compact the looser soils giving erroneous high blow counts. In deep holes, the weight of the drilling rods, especially in cohesive soils, decreases the *N*-value and the true measurement of the strength of the soil. Sampling of silts can prove troublesome depending on their moisture content. A dry silt of inherent low strength can have a high *N*-value. Silts with higher moisture contents can also give erroneous *N*-values as they dilate and the potential for liquefaction can be misinterpreted.

Soft rock in the Boston area is represented by the altered argillite and to a lesser degree the altered conglomerate. Kaolinized rock shows a consistency equal to that of clay based on the results of test borings but is confined within the unaltered bedrock. Examination of the altered rock in-situ finds the rock to be actually more competent. Sand and gravel can erroneously produce *N*-val-

TABLE 4-3.
Typical Engineering Properties of Foundation Material in Boston

Geologic Unit	General Description	Saturated Unit Weight kg/m^2 (lb/ft^3)	Natural Water Content (percent)	Atterberg Limits (percent) LL	PI	Undrained Shear Strength kg/m^2 (lb/ft^2)	Other	Allowable Bearing Pressure kg/m^2 (lb/ft^2)
I Miscellaneous Fill	Loose to very dense sand, gravelly sand or sandy gravel intermixed with varying amounts of silt, cobbles or boulders, & miscellaneous brick, rubble, trash or other foreign materials	1600-2000 (100-125)						
II Organics	Very soft to medium stiff, gray clayey organic silt or brown fibrous peat with trace amounts of shells, fine sand & wood	1440-1760 (90-110)	40-100			1465-3900 (300-800)	Organic content 5-25 percent	
III Outwash Deposits	Medium dense to dense, brown coarse to fine or medium to fine sand with varying amounts of gravel & silt	1760-2160 (110-135)						19,500-48,800 (4000-10,000)
IV Marine Clay	Stiff, yellow-gray silty clay.	1840-2160	25-35	40-55	15-30	3900-9760	Compression	14,650-39,000
	Medium stiff, gray silty clay, occasional layers of fine sand or silt.	1824-1920 (114-120)	30-40	40-55	15-30	2930-5860 (600-1200)	Ratio = 0.15-0.25	9760-19,500 (2000-4000)
	Soft to very soft, gray silty clay, occasional layers of fine sand or silt. (**Note:** This unit sometimes becomes stiffer at lower levels.)	1810-1890 (113-118)	30-50	40-55	15-30	1950-3900 (400-800)	Recompression Ratio = 0.02-0.04	4880-9760 (1000-2000)
IV-A Marine Deposits	Interbedded gray silty or sandy clay, silty fine sand & fine sandy silt.	Too variable						Variable
V Outwash	Medium to dense, stratified sands & gravels in discontinuous layers.							Variable
VI Glacial Till	Dense to very dense, heterogeneous mixture of sand, gravel, clay & silt with cobbles, & rock fragments	2000-2240 (125-140)	10-20	15-30	10-20	9760-39,000 (2000-8000)		39,000-98,000 (8000-20,000)
VI-A Moraine Deposits	Miscellaneous deposits of deformed glacial till, outwash & clays.	Too variable						Variable
VII Bedrock	Cambridge Argillite							78,000-195,000 (16,000-40,000)
	Roxbury Conglomerate							195,000-975,000 (40,000-200,000)

Note: From E.G. Johnson, in Woodhouse & Barosh (1991)

ues in excess of 50 due to cobbles or boulders, and the presence of iron and manganese oxides cement in, and otherwise a loose deposit can inflate the *N*-value. In-situ plate bearing tests made in the clay-rich Boston till found its high bearing capacity can be exaggerated.

Stiff soils by definition are clays with *N*-values between 15 and 50, corresponding to a consistency of stiff to hard. They represent mixtures of clay and silt of varying cohesiveness and plasticity. Pure clays are not found in the Boston area. The high N values would rule out any sensitivity and liquefaction

TABLE 4-4.
Engineering Properties of the Cambridge Argillite

Location & Rock Type	Unit Weight (kg/m^3; pcf)§§				Unconfined Compression f_c (N/m^2; psi)§§				Tangent Modulus E_{t50} (N/m^2; psi)§§			
	Low	Average	High	No. Tests	Low	Average	High	No. Tests	Low	Average	High	No. Tests
Dorchester Tunnel* Argillite	2,691 168.0	2,747 171.5	2,810 175.4	15	34,480 5,000	103,430 15,001	236,840 34,350	15	39,990 5,800	62,740 9,100	84,120 12,200	15
MBTA Red Line Extension** Argillite	2,538 158.4	2,747 171.5	2,844 177.5	52	41,990 6,090	131,420 19,060	255,120 37,000	50	20,690 3,000	47,580 6,900	63,430 9,200	18
MBTA Red Line Extension** Melaphyric Dike Rocks***	2,606 162.7	2,738 170.9	2,861 178.6	12	12,620 1,830	67,570 9,800	166,860 24,200	10	14,480 2,100	24,130 3,500	31,030 4,500	3
MBTA Red Line Extension** Tuff/Trachyte§	2,518 157.2	2,739 171.0	2,884 180.0	6	29,650 4,300	103,620 15,030	250,980 36,400	6	29,650 4,300	59,300 8,600	74,470 10,800	5

Notes: Table data from Hatheway & Paris (1979).

*Data taken from Haley & Aldrich (1977).

**Data taken from Bechtel (1978).

***Rock types include diabase, andesite, basalt & altered varieties of these rocks.

§ Rock previously identified as dark gray to black tuff; believed to be analogous to dark gray to black trachyte appearing as irregular sill-like intrusion in Porter Square exploration shaft.

§§ Metric units shown above English units in each data group.

potential. Clay with an *N*-value of 15 or less is defined as soft, but the soil classification system defines soft clay as having an *N*-value of 0 to 4. Greater than 4 and less than 15 are medium stiff. Low blow-count clays can be sensitive, as found, for example, in the reworked and redeposited clay in the Fort Point Channel and Charles River area. Their behavior in a major earthquake could produce significant damage to structures and the infrastructure.

Engineering Properties of Geologic Materials

Overburden. The variability in the surficial deposits forming the overburden in the Boston area affects foundation construction and its expected problems. Each type of deposit has its own characteristics for engineering purposes and the combination of deposits present beneath a site creates highly variable conditions across Boston. The chief variability comes from the amount and kind of marine clay, organic material and fill near the surface (see Tables 4-3, 4-4 & 4-5). Each type of soil is numbered as a key to the type of foundation (see Figures 4-2 & 4-3).

Fill. The previously discussed fill in and around Boston (exclusive of engineered fill) dates back to the seventeenth century along wharves and to major land fillings made in the nineteenth and twentieth centuries. Fill is generally considered to be deleterious and unsuitable for bearing structures. In rare cases, some of this old fill is compact and granular, which warrants a close look by the engineering community as a bearing stratum for lightly loaded buildings. The suitability of the fill has to be tested by borings.

Organic Sediment. The organic silt and clay, and peat deposits that were laid down throughout much of the lower lying areas surrounding the Shawmut Peninsula following glaciation vary greatly in overall thickness and content, but are generally from 1.5 to 7.5 meters (5 to 25 feet) thick. In those filled-in areas of the Back Bay, the layer has been compressed considerably due to the weight of the fill. Marsh gas that results from the decomposing organic matter is sometimes encountered in excavations.

TABLE 4-5.
Engineering Properties of Materials in Boston

Stratum	Consolidation Condition	Effective Friction Angle	Total Unit Weight (pcf)	Allowable Bearing Pressure (tsf)
Fill Sands distributed along entire project	Loose to medium dense	28-33	110-120	1.0
Sands & Gravels Glacial outwash deposits; sands, gravelly sands, silty sands	Medium dense to very dense	32-36	110-125	1.0-2.5
Marine Clay Silty clay	Over-consolidated	24	110-120	2.0-4.0
Till Glacially deposited mixture of sand, gravel, cobbles, boulders, silt & clay	Dense to very dense	36	125-140	3.0-5.0
Cambridge Argillite Bedrock (slightly indurated)	Medium hard to hard with locally weathered & broken layers	45	165-170	10-20

Note: Table data from Cullen *et al.* (1982).

Reworked Marine Clay. In some limited areas of Boston and Cambridge (such as around the Charles River and the Fort Point Channel), the marine clay was eroded after the Lexington glacial re-advance and redeposited with lower shear strength. The reworked post-glacial clay in Boston is sensitive in places. In the Fresh Pond-Alewife area of Cambridge

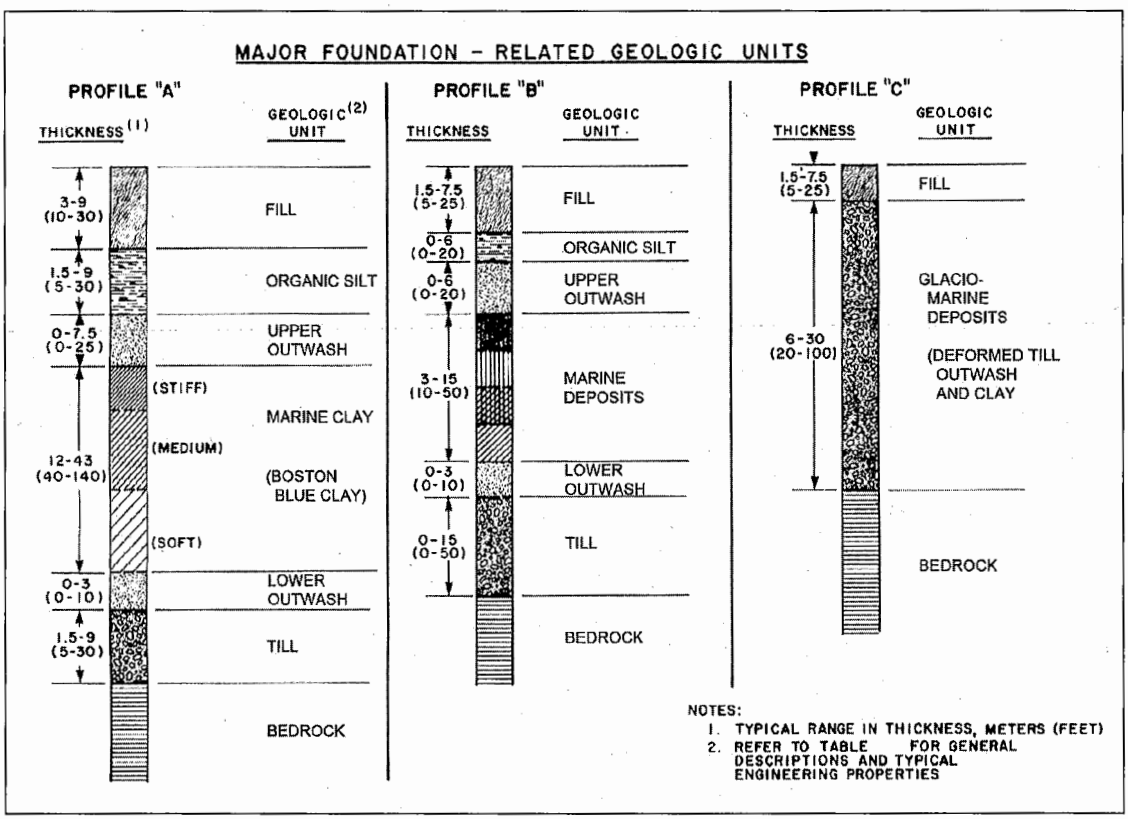

FIGURE 4-2. Geologic units encountered in typical deep foundations.

where the marine clay was eroded and redeposited in fresh water, it is very sensitive, with a higher natural water content than older, more-consolidated marine clay. Problems occur when the reworked clay is not recognized or its weaker condition is not fully taken into account.

Upper Outwash Sand & Gravel: Sand and gravel were deposited over the weathered surface of the marine clay in some areas following the last readvance of glacial ice. These well-stratified sands and gravels range in thickness from 3 to 7.5 meters (10 to 25 feet). They are medium compact to compact and are considered an important bearing stratum for the support of light- to medium-weight structures. These sand and gravel deposits have relatively high permeability, which is an important characteristic and construction consideration. They are generally found to underly organic soils and to direct-

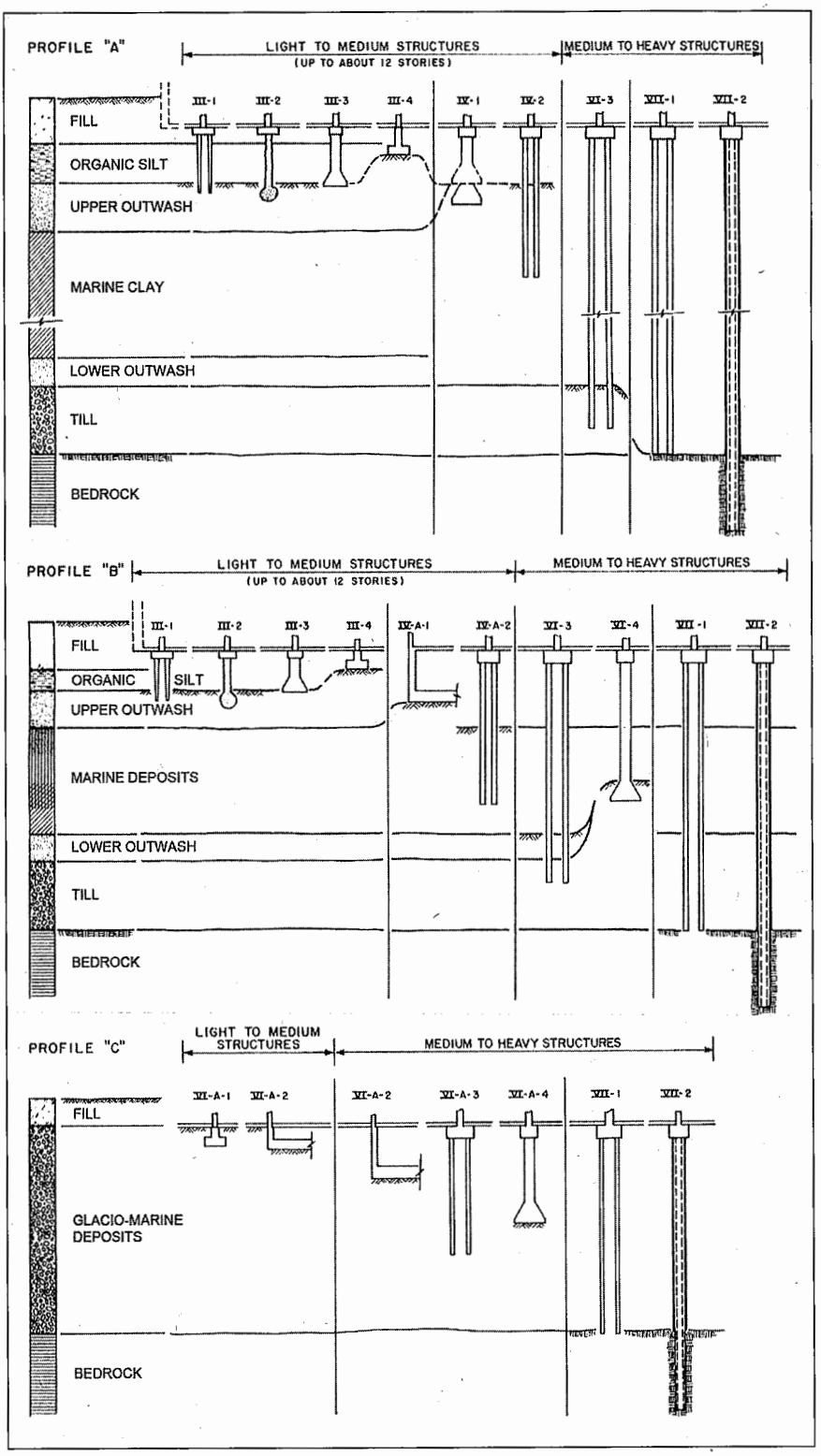

FIGURE 4-3. Typical foundation types used in Boston. (See Figure 4-2 for details on geologic columns.)

ly overly marine clay, particularly in the Copley Square/Christian Science Church area.

Marine Clay. The marine "Boston Blue Clay" varies in thickness and depth all around Boston, and its properties have been investigated thoroughly for foundation design. A weathered crust is present at the top of the clay as a result of desiccation, oxidation and capillary stress. The crust is yellowish or brownish in color, in contrast to the normal gray or olive-gray color of the lower clay. The presence of the stiffer crust plays an important role in the support of structures in the area. Extensive laboratory tests, such as those performed on clay from the Prudential site (Casagrande, 1958) and the Massachusetts Institute of Technology (MIT) (Ladd & Luscher, 1965), reveal that the stiff yellow clay has been pre-consolidated to four or more times the present overburden stress. The over-consolidation ratio decreases quite rapidly with depth so that the clay below about elevation –15 to –21 meters (–50 to –70 feet) MSL is considered to be normally consolidated to just slightly over-consolidated (with an overconsolidation ratio of 1.1 to 1.2). Discontinuous layers and lenses of sand and silt are common within the clay. Therefore, horizontal permeability is generally several times greater than the vertical. Typical ranges of undrained material shear strength and other engineering properties have been determined (see Table 4-3).

Lower Outwash Sand & Gravel. This glacial deposit consists of medium dense, stratified sand and gravel that form a discontinuous deposit over the lodgement till. It has similar characteristics as the upper outwash no matter its thickness.

Glaciomarine Clay. The till-like glaciomarine clay is highly variable and it is impractical to try to typify its engineering properties. The properties would vary from those of clay to very locally reaching almost those of till, which can be misleading. *N*-values generally lie between 25 to more than 100, but typically range around 45 to 60 as a rule of thumb to differentiate them from the denser till (Miller, 2012). The misidentification of this material as till prior to about 1990, when the results of the Central Artery/Tunnel Project borings allowed the proper identification of this deposit, resulted in over-estimating its bearing capacity.

Till. Till directly overlies the bedrock throughout much of the Boston area. For foundation purposes, the till is extremely variable as a result of the very complex processes of deposition with pockets and lenses of pervious sands and gravels, as well as zones of plastic silts and clays that are often encountered within the mass. The lower till may have a greater silt and clay content from the underlying argillite and have low plasticity, and its plastic nature has caused foundations problems when end-bearing piles failed to "take up" as expected in the till.

The SPT (*N*-value) is usually the only practical way to determine an indication of density of the in-situ material. *N*-values of over 80 blows per 30 centimeters (1 foot) are typical where there is more than 15 meters (50 feet) of overburden. Lesser values of *N*, from 40 to 80, are obtained in reworked till, glacial overthrust deposits and at shallower depths. It must be re-emphasized that the method by which *N*-values are determined is not always precise, and unusually high values for individual tests may reflect sampler impact on gravel, cobbles or boulders as noted above. Sample recovery may be poor, and visual examination and classification are often made on very limited quantities. Testing in place with pressure-meters may be appropriate for certain projects. Whenever possible, grain-size and hydrometer tests should be performed as well as finding Atterberg limits on cohesive portions. Typical grain-size distribution curves usually indicate a widely graded material with 10 to 25 percent finer than the Number 200 sieve.

Bedrock. Only the relatively shallow bedrock is of significance for foundation engineering purposes. The predominant upper bedrock underlying much of Boston and Cambridge is the Cambridge Argillite. A lesser amount of Roxbury Conglomerate is encountered locally, especially under the south and west portions of the city in Roxbury, and Brookline.

Argillite. The Cambridge Argillite has extremely variable engineering properties. The unweathered, unaltered rock may be quite

sound so that vertical cuts will remain stable with little or no support, and bearing intensities of up to 1,000 kilonewtons per square meter (60 tons per square foot) or more may be appropriate. The argillite provides satisfactory support for most engineering structures if it is not affected by soft-rock alteration or intense jointing or faulting. Steel piles under the John Hancock Tower were driven into the surface of this rock and the caissons under the Prudential Tower are socketed 3 meters (10 feet) deep into argillite. The unconfined compressive strength of unaltered argillite (Woodhouse & Barosh, 1991) ranges from 355 to 2,650 kilograms per square centimeter (5,050 to 37,690 pounds per square inch) with a mean value about 1,335 kilograms per square centimeter (19,000 pounds per square inch). On the other hand, highly altered zones may have properties similar to a medium or soft cohesive soil material. This potential variability, both vertically and laterally over short distances, requires a very thorough, well-planned exploration program if foundation support or other construction is planned on or within the rock (Kaye, 1982b). Such exploration planning begins with geologic research about the area and nearby sites to see if troublesome conditions were encountered.

Blasted fresh rock commonly shows only sparse joints. However, in shear zones, joints may be spaced as closely as 2 centimeters (0.8 inches) apart and form two or more sets. Cleavage has not been found to seriously affect strength of the argillite in tunnels or under buildings. Single bore tunnels, which in Boston have generally not exceeded 4 meters (13 feet) in diameter, have shown the argillite to be strong enough not to require steel supports, except where it has been altered, or where the tunnel parallels the strike of beds that are badly broken by joints, faults or dike-filled faults. This condition was true even for the Cambridge subway tunnel (Red Line NW) that is 6.5 meters (21 feet) in diameter. Soft-rock alteration probably is responsible for most of the bad tunneling ground in the Boston Basin. Unfortunately, these potentially problematic conditions are easily missed by exploratory borings since much of the alteration is restricted to relatively narrow zones or beds. In tunnels, the altered rock almost always requires steel support. The altered rock and sheared argillite have been commonly mislabeled as "shale" in geotechnical reports. Sheared argillite was found in the bedrock borings for the 500 Boylston Street project near the corner of Berkeley Street. The need for thorough understanding of the geology of the project/site alignment is imperative.

Cleavage and joints in deeply buried argillite have been found to be a source of water, which can be under considerable artesian pressure. The sockets drilled dry into argillite at the base of the caissons under the Prudential Tower rapidly filled with water that could not be controlled sufficiently to allow close inspection of the sockets (Ball, 1962). In building the new Charles River Dam between Boston and Charlestown in the 1974 to 1978 period, borings showed that hydrostatic pressures in fissured argillite were such as to be a threat to blow out the overlying till inside the construction cofferdam (U.S. Army Corps of Engineers, 1972). A ring of relief wells was drilled into bedrock, and these wells reduced uplift pressures to a satisfactory level (Kaye, 1982b). The hydrostatic head varied with the tides in Boston Harbor, an observation that had been made much earlier by Lathrop (1800) in a deep well for what was the then-new State House on the slope of Beacon Hill.

The effect of soft-rock alteration on the construction of surface engineered structures has, thus far, been relatively slight. In Boston, these rocks are generally eroded and deeply buried beneath thick clay and other overburden. Surface structures have relied in varying degrees on overlying materials for bearing. The slightly altered argillite and tuff beneath two large corner piers of Boston Company Building caused no apparent loss of rock strength (Johnson, 1973), although it seemed weaker at first. The use of water during the exploration coring process apparently softened the rock and made it appear more altered and weaker than it actually was. The gross bearing pressure on the altered rock here was 293,000 kilograms per square meter (30 tons per square foot).

Sandstone intervals in the argillite have held up well in several of Boston's tunnels. No support was required in approximately 460 meters (1,500 feet) of sandstone exposed in the west part of the City Tunnel (Tierney *et al.*, 1968). Two short sections of sandstone in the City Tunnel Extension required support, either because of excessive splitting parallel to the bedding or due to joints that were too closely spaced (Billings & Tierney, 1964).

Conglomerate. The Roxbury Conglomerate, in contrast with the argillite, is a very hard, durable stone that was commonly used in the late nineteenth century for building, such as for the Old South Church on Boylston Street in Boston, and for retaining wall construction (Kaye, 1976a). It is usually mottled brown in color, with embedded round to angular pebbles, and resembles a dense concrete with very large coarse aggregate. Surfaces on the conglomerate may be extremely uneven since they were not easily eroded by glaciation. Construction excavation or drilling in this massive rock may be very difficult, due to its hardness, sparse jointing or other discontinuities, as well as the presence of hard, embedded gravels.

The conglomerate is a strong rock when not weakened by soft-rock alteration. Only about 12 percent of the 900 meters (3,000 feet) of the Main Drainage Tunnel that were driven in conglomerate (Rahm, 1962) required steel supports. Most of the weak rock is badly altered and faulted and cut by a diabase dike. The Dorchester Tunnel pierced the entire width of the conglomerate cropping out in Brookline, Jamaica Plain and Roxbury (Richardson, 1977). About 13 percent of the 6,300 meters (20,700 feet) of conglomerate traversed by this tunnel required steel support because of soft-rock alteration, which seemed to be concentrated along a very wide fault zone. Another 2 percent of the tunnel required support of rock due to close jointing, faulting and the effects of a thick diabase dike and interbedded argillite (Kaye, 1979).

Volcanic Rock. About 880 meters (2,900 feet) of volcanic rock was pierced by the Dorchester Tunnel (Richardson, 1977), of which only 46 meters (150 feet) required steel support that was necessary due to a local concentration of jointing. No support was needed in the approximately 2,100 meters (7,000 feet) of andesite found in the City Tunnel (Tierney *et al.*, 1968), nor was any support required for the approximately 760 meters (2,500 feet) of this andesite rock in the City Tunnel Extension (Billings & Tierney, 1964).

Groundwater Level

The original groundwater level in the Beacon Hill area in colonial times was relatively high and responsible for the springs around its base. However, the lowering of the hill, reduction of recharge by man-made cover and other effects of construction have lowered its slope toward the original shore where the groundwater level may now be critical. The normal groundwater level in Boston's Back Bay area now is generally somewhat above mean sea level, as might be expected. The normal tidal range in the harbor is about 1.5 meters (5 feet) above and below mean tide and tidal fluctuations of groundwater levels below the city are seen only around the older part of the city along marginal waterfront areas, as measured in observation wells and some construction sites. However, it is usually not observed locally, due in large part to the stabilizing effect of the Charles River Basin being maintained at about 0.73 meters (2.4 feet) above MSL. Water rose and fell with the tide in the cellar of 80 Broad Street where the USGS previously had its offices — a location at the original shore, but now a few blocks from the harbor.

Variations and anomalies in the piezometric surface often are related to dewatering for construction projects or pumping from deep basements. However, leakage from or into storm sewers is another factor. The many subway tunnels and deep utilities conduits commonly form either barriers or drainage paths that interrupt or control normal groundwater flow and create local variations in its level that can cause serious problems, as in the Back Bay (see Figures 4-4 & 4-5).

Influence on Constructed Facilities. Groundwater levels are a key factor in any geotechnical assessment of conditions in the area. The determination of realistic present water levels, as well as past and potential future variations, is of major significance. In Boston, the conse-

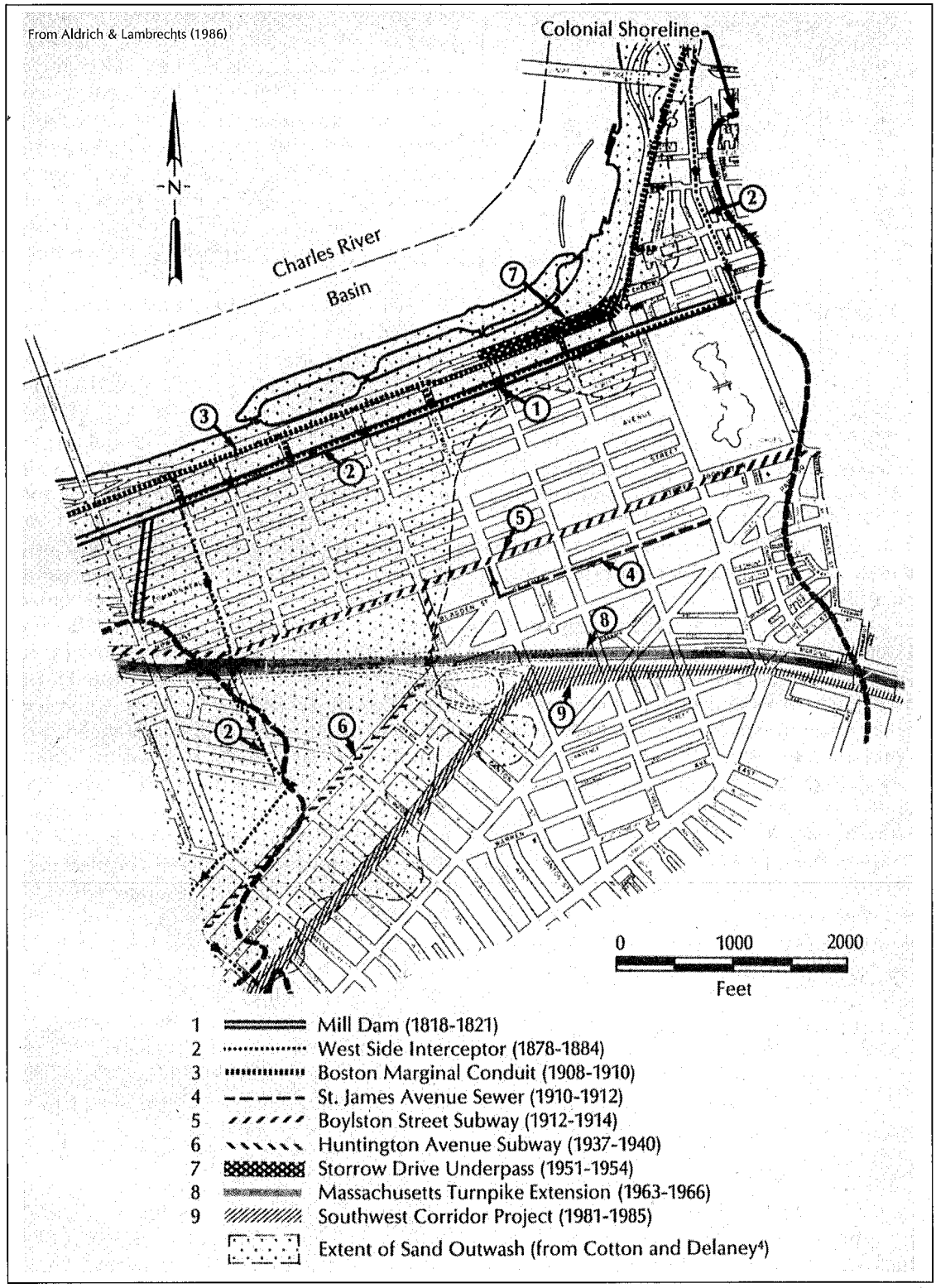

FIGURE 4-4. Map of the Back Bay showing locations of sewers, drains and major transportation routes.

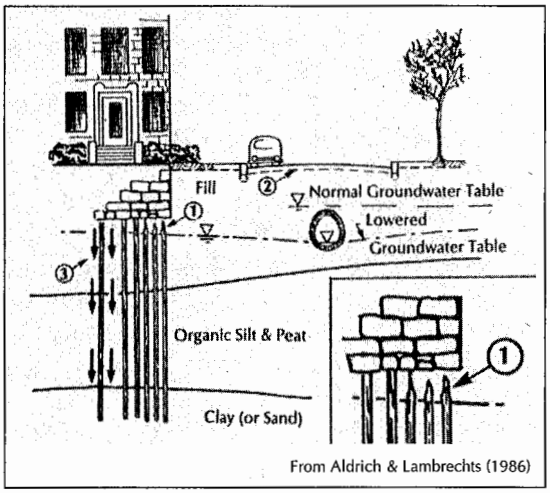

FIGURE 4-5. Diagram of Back Bay foundations showing the effects of lowered groundwater levels.

quences of lowering the water levels below normal, even temporarily, fall in two general categories:

- General subsidence of the land, including streets, utilities or buildings founded at shallow depths. Subsidence may occur if the water level is depressed in areas underlain by soft compressible layers, as in the filled areas of Back Bay. The settlement occurs very slowly and the magnitude reflects the relative thickness of the soft underlying material.
- Settlement of individual buildings supported on untreated wood piles. This type of settlement occurs if the pile butts (tops) are exposed to drying and decay.

The maintenance of "normal" water levels became a very important concern to city officials during the past century. After the filling of the Back Bay in the nineteenth century, most structures were supported on untreated wood piles, driven to bearing in the sand layer below the organic deposit or to/into in the marine clay (see Figure 4-5). Experience showed that the best-performing piles were Maine spruce, which was harvested in the summer months, and, later, Southern yellow pine (Worcester, 1914; Aldrich, 1970). The pile lengths were between 6 to 12 meters (20 to 40 feet) with tips usually 20 centimeters (8 inches) in diameter and the butts up to 30 centimeters (12 inches) in diameter. The clear distance between piles was as little as 30 centimeters (12 inches) to maximize friction, but center-to-center spacings of two to three diameters was commonly used. Since the groundwater level then was approximately 0.7 meters (2.3 feet) above MSL, piles were commonly cut off at elevation 5 Boston City Base (BCB). Experience has shown that if the wood piles are always thoroughly dry, or the piles are constantly submerged and not exposed to air, they will not be attacked by fungi and deteriorate. However, it is not considered practical to keep the piles dry when embedded in the ground so they must be kept submerged. However, with the subsequent effects of decreased surface infiltration as the areas were developed, dewatering for tunnels and by drainage systems, sewer and drain leakage and groundwater infiltration, and other local pumping activities, it was discovered that wood piles below many structures were no longer permanently submerged, and therefore became exposed to loss of saturation, exposure to air and subsequent decay that caused settlement (see Figure 4-5). The lowered water level also may result in consolidation of the material surrounding the piles if the lowering is great enough, and therefore the frictional forces that develop along the piles may create additional loads for which the piles were not designed. The settlement of this material is called negative skin friction and can pull a pile right out of the pile cap, which occurred under the Christian Science Church. Such was the case during the construction of the new entrance to the Mother Church in the late 1960s when the existing pile caps were uncovered and underpinned (Woodhouse & Barosh, 1991).

A notable example of the groundwater lowering problem occurred in 1929, when major cracks were discovered in the walls of the Boston Public Library at Copley Square that was constructed on wood piles in 1888 when the groundwater level was higher (Aldrich, 1979). Upon investigation by the city and its consulting engineers, it was discovered that the tops of wood piles were decaying. A major underpinning effort ensued and about 40 percent of the wood piles supporting the building

were affected. The efforts to restore the foundation system cost over $250,000 (in 1929 dollars). The affected pile heads were cut off and replaced with concrete. At the same time, the trustees of the nearby Trinity Church became alarmed that their church might be suffering a similar fate. City engineers discovered that deterioration of a few piles had occurred on one side of the church. These piles were stabilized with steel supports. Settlement of all the piles also had been caused by the very large structural load of the church itself. During the construction of the nearby John Hancock Tower in the 1970s, the Trinity Church and the Copley Plaza Hotel were damaged by the excessive lateral deflection of the steel sheeting and bracing of the Hancock's excavation support system along Clarendon Street and St. James Avenue. The wall warped inward because of insufficient internal support to counteract the load from the exterior overburden, which caused the ground to settle many inches under the transept wing of the church. Failure was avoided because of the inherent factor of safety in the wood pile foundation that used 4,500 piles. According to information made available by the church, the Hancock Tower now collects storm water, which is then used to recharge the area of the Trinity Church.

The apparent reason for the general lowering of the groundwater surface in the 1920s was traced back to earlier construction in 1912 of storm and sanitary sewer lines, with invert levels about 1.8 and 4 meters (6 and 13 feet) below the groundwater surface. Steps were taken to control the infiltration and restore normal levels by the construction of a permanent dam to partially block the sewer. Further remedial measures were carried out in 1955 at the northeast corner of Copley Square, where perforated metal pipes (designed to recharge the groundwater) were installed to intercept surface water flowing to the drain. The Boylston Street subway tunnel also was thought at the time to have contributed to the problem but this theory was never proven.

More recently, problems with foundation distress and rotted piles have occurred in the lower Beacon Hill area. Investigations have revealed that groundwater levels were as much as 1.8 meters (6 feet) below the water level in the Charles River. The lowered levels are attributed to leakage into sewers and lack of sufficient surface recharge. A comprehensive historical perspective on groundwater fluctuations in the Back Bay (see Figures 4-6, 4-7, 4-8 & 4-9), and the adverse effects of lowered levels, is provided by Aldrich and Lambrechts (1986). In the 1980s, seventeen wood-pile-founded homes on Brimmer Street that were built on filled land of the 1860s and 1870s at the base of Beacon Hill were initially condemned by the Boston Inspectional Services Department and some were temporarily vacated. In addition, more than a hundred homes in the area were placed under observation. Since 1929, over two hundred buildings in the lower Beacon Hill and Back Bay area have had their wood pile tops repaired. The residential repair usually consists of having the rotted wood pile tops cut off and replaced by steel and concrete at a cost that is currently on the order of $400,000 to $600,000 per building.

In response to these problems, the Boston Groundwater Trust was founded in 1986 in order to monitor groundwater levels in the affected areas. Existing wells and new monitoring wells were to be measured and the data compiled in yearly reports; however, funds were not then available. The trust was revived in 1997, and in 2002 the Massachusetts Legislature passed the Environmental Bond Bill to provide future funding. In 2005, city and state officials signed a memorandum of understanding to continue to monitor water levels, which resulted in a Groundwater Conservation Overlay District and the publication of well readings via the Boston Groundwater Trust web site (www.bostongroundwater.org).

During any construction excavation below the water table, it is now a requirement that an adequate cutoff system be installed to control drawdown beyond the site area. Adjacent areas must be monitored and, if necessary, remedial action be taken, such as modifying the pumping operation or installing a recharge system.

Geologic Control on the Selection of Appropriate Foundations

Early construction in Boston relied on granite

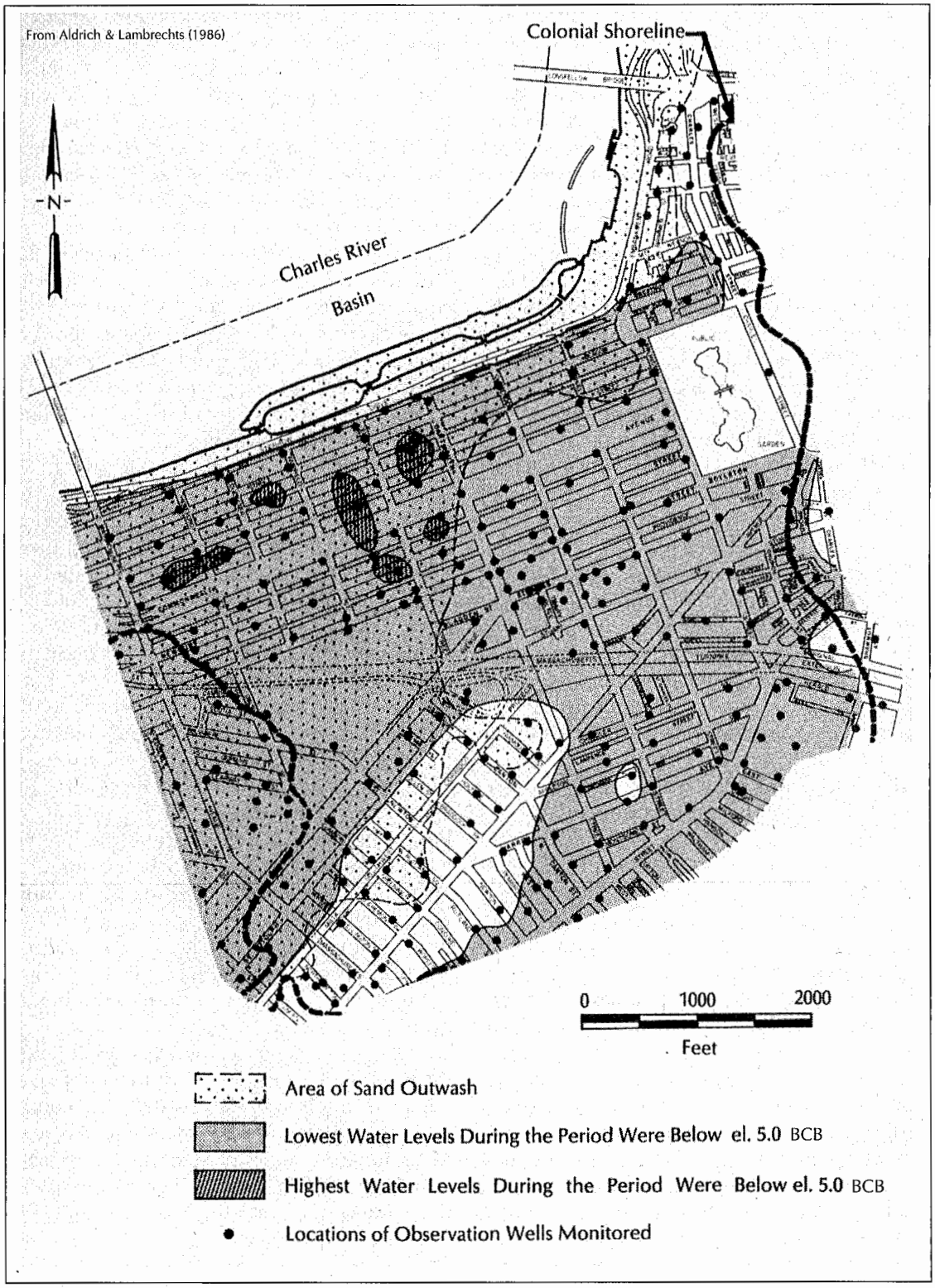

FIGURE 4-6. Areas of Back Bay having groundwater levels below elevation 5.0 BCB at some time from 1936 to 1940.

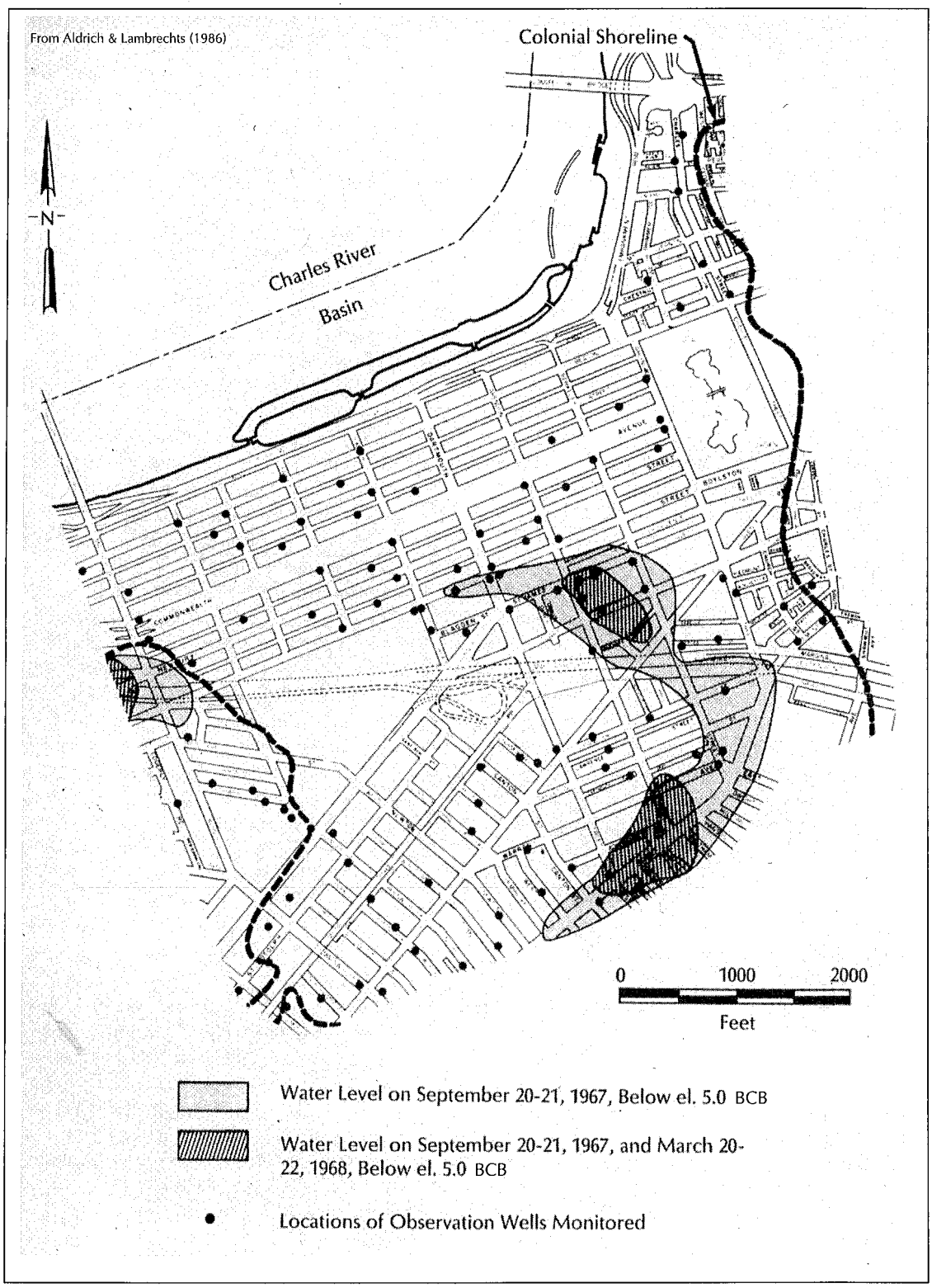

FIGURE 4-7. Areas of Back Bay having groundwater levels below elevation 5.0 BCB during measurements from 1967 to 1968.

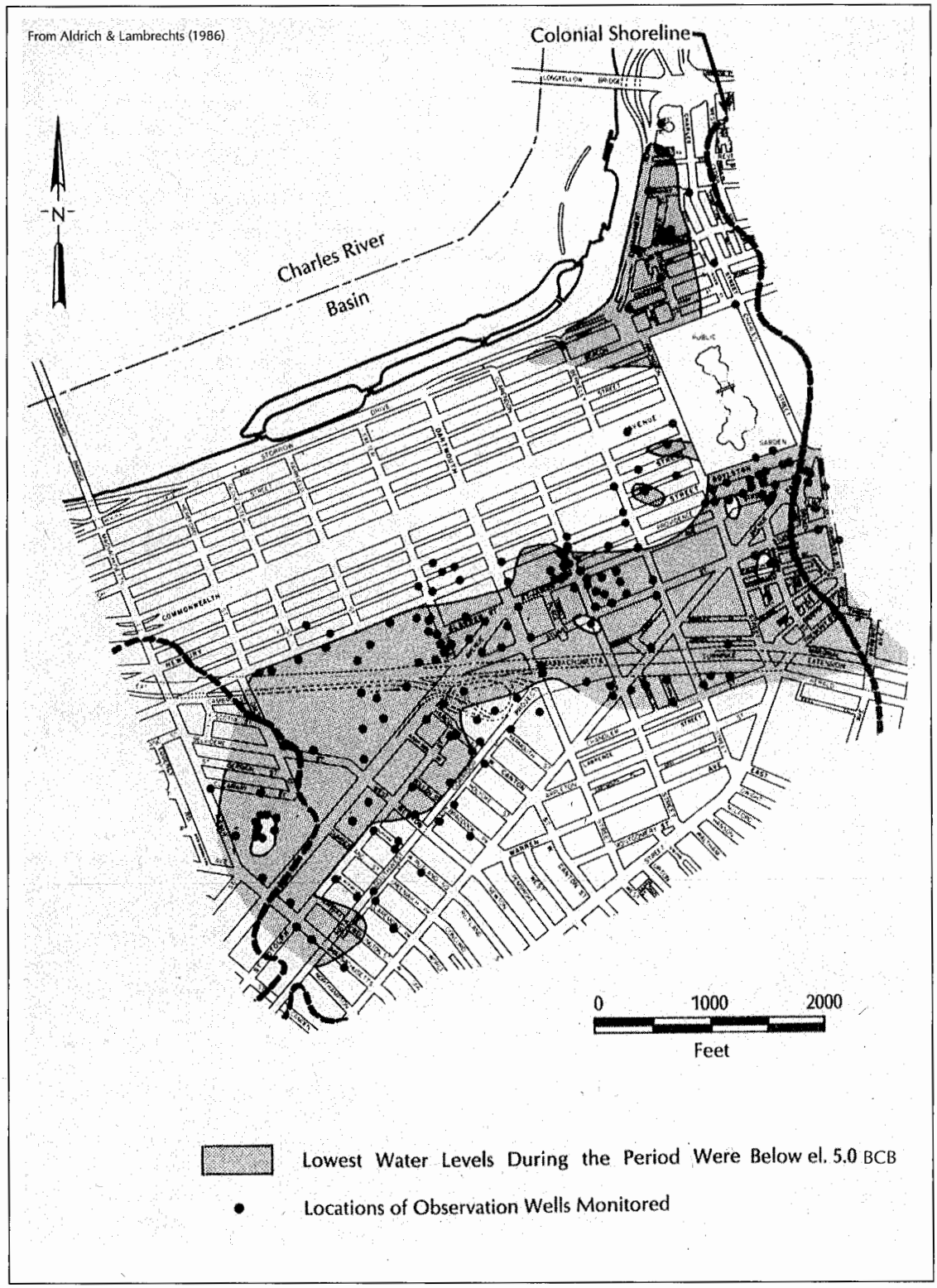

FIGURE 4-8. Areas of Back Bay having groundwater levels below elevation 5.0 BCB from 1970 to 1985.

block footings and walls (see Figure 4-3). The geologic layers were not given too much study except where soils were deemed soft enough to require wider footings or wood piles. The advent of large buildings in the twentieth century focused much more attention to the geology in designing for the mass of an edifice and later for seismic forces. The various geologic layers on which a foundation is to bear determine the selection of the appropriate type of foundation system to be used. Much of the current details and load limits, and to some extent selection, are determined by building code, and the code itself is based on long experience. Foundation designs in Boston must now comply with the International Building Code (IBC).

Three typical geologic sequences found in Boston were evaluated for suitable foundations for large buildings (see Figures 4-2 & 4-3). These sequences are arbitrarily designated as A, B and C. Foundation types considered appropriate for bearing within the separate geologic units are found in each of the sequences (illustrated and keyed into Figure 4-2). The types mentioned are representative of most, but not all, kinds of foundations used. The complex geology does not allow specific areal limits to be defined for the application of this simplified system of three typical geologic sequences and only general areas are indicated.

Geologic Sequence A. This sequence covers deposits below the filled-in Back Bay; areas of Cambridge, Charlestown and Somerville; East Boston; the Fenway; the Charles River, including Gravelly Point; and marginal waterfront areas that make up the most typical condition in Boston, although the upper outwash may be locally absent. The upper Lexington outwash is more commonly found from Copley Square to the Christian Science Church complex and south of Boston in the Quincy-Squantum area.

Geologic Sequence B. This sequence covers deposits in intermediate areas adjacent to the original Shawmut Peninsula that may be more heterogeneous and contain glaciomarine deposits such as along the route of the Central Artery O'Neil Tunnel B from South Station to North Station (following Atlantic and Commercial avenues), Haymarket Square and Post Office Square.

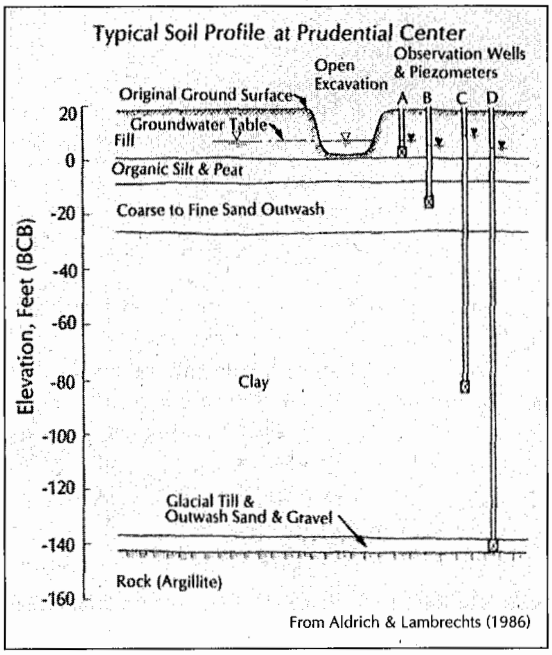

FIGURE 4-9. Profile of the surficial material below the Prudential Center showing the water monitoring wells and the static water level in Well A.

Geologic Sequence C. This sequence covers deposits within the limits of the original colonial shoreline of the Shawmut Peninsula that may be very complex, as in the area of Trimountain and its overthrust deposits, but has much less fill than in the other two sequences.

A description of the geologic units in these sequences together with their typical engineering properties were listed in Table 4-3.

Fill & Organic Sediment (I & II). The fill and the organic layers are not suitable for footings to support any significant structure.

Outwash Sand & Gravel (III & V). The upper outwash sand and gravel (Unit III) warrants consideration as a bearing stratum where it is sufficiently thick because of its density and composition. The lower outwash (Unit V) below the marine clay is too thin and often discontinuous and is not considered to be a specific bearing stratum. Light- to medium-weight structures may be supported on relatively short piles or caissons, in either Sequence A or B. Where this layer is relatively shallow, it may be feasible to use spread footings. For Sequence A, especially, estimates

must be made of the post-construction settlement of the underlying marine clay. Usually, the settlements of buildings up to ten to twelve stories, with one full basement level, will be nominal. Higher buildings may be possible if more than one basement level is provided since the stress relief from deeper excavations compensates for the greater building foundation loading.

Untreated wood piles were used predominantly throughout the early years of construction in the Back Bay and pile capacities of 62 to 89 kilonewtons (7 to 10 tons) were most common when driven to bear in the upper sand and gravel layer. Problems can develop if the pile butts are alternately exposed to drying and submergence, or are continually moist. Otherwise, the use of pressure-treated timber can overcome this problem. In earlier times, pile caps were made of dry laid granite blocks set on top of wood pile tops. For certain structures, mortared granite pile caps were placed on the wood piles, such as the one that was exposed and is today on display below the Trinity Church in Copley Square. However, in some areas, rock rubble consisting of pieces of Roxbury Conglomerate have also been found.

Pressure-injected footings (PIFs) having individual capacities up to 1,070 kilonewtons (120 tons) or more are feasible where the intended support stratum has a proper grain-size distribution with less than 15 percent fine soil material (clay and/or silt), and the layer thickness is at least 3 meters (10 feet). The PIFs are a unique pile type, made by advancing a heavy steel drive tube into the sand surface and then driving one or more batches of very dry, zero slump concrete mix 0.14 cubic meters (5 cubic feet) each out of the drive tube to form an expanded concrete base within the granular soil material, which densely compacts the granular soils and increases bearing capacity. A concrete shaft is then formed above the base to complete the unit.

Belled caissons may be installed to bear on the sand and gravel only if it is practical to make undercuts in the overlying organic layer. Otherwise, straight-shaft units would be required, which are less economical. During construction, it is generally required that the base be dewatered before concrete is placed. It is sometimes necessary to dewater the bearing sand in the vicinity by installing wells, provided there are no adverse effects in the adjacent area. However, dewatering the confined sand stratum can be problematic.

Spread footings are feasible where the depth to the top of the bearing layer is only a meter or so (a few feet), and dewatering is not a serious problem. The footings are usually sized for a bearing value of 240 to 480 kilonewtons per square meter (2.5 to 5.0 tons per square foot).

Marine Clay (IV). For Sequence A, some light to medium structures are founded directly on or within the marine clay. It is important to note, however, that estimates of potential settlement must be made. Belled caissons (IV-1) are perhaps the most common foundation type in the clay stratum (see Figure 4-3). Steel casings are advanced through the upper layers and sealed into the organic deposits or clay surface. The belled portions are undercut in the overlying organic soils by rotary machine to a diameter of 1.8 to 3 meters (6 to 10 feet) or more. In the early days, this belling was done by hand labor. Usually, there is little or no dewatering required and all excavation is in the dry. The caissons are typically designed for an end-bearing capacity of 190 to 380 kilonewtons per square meter (2 to 4 tons per square foot) in the upper stiff clay zone. If that zone is fully penetrated, caissons bearing on the softer clay below would have a reduced design bearing capacity. In all cases, the strength of the clay should be verified in the field by competent geotechnical personnel as each caisson is excavated.

Friction piles are also used to provide support in the upper clay. Wood piles, of capacity up to 196 kilonewtons (22 tons), are allowed by the Massachusetts Building Code. Other pile types have also been used, based on a typical design friction value of 24 kilonewtons per square meter (500 pounds per square foot) for the portion embedded in the clay. Any such installation should be verified by on-site pile-load tests.

For structures with very dry basements (three floors), thick concrete mat foundations can be used to bear on the top of the marine clay or upper outwash sand. Basement exca-

vation removes soil weight to counter the building weight, allowing such foundations to essentially "float" — thus, the term *floating foundation.* The behavior of the marine clay supporting the floating foundations under the MIT campus in the early part of the twentieth century gave rise to the science of soil mechanics under Karl Terzaghi.

Glaciomarine Sediment (VI-A). Light to medium structures are founded on or within glaciomarine sediment in Sequence B, especially when there is no sand layer or it is thin. Because conditions in this sequence may be very erratic, such as a combination of granular and cohesive units in discontinuous layers and lenses, each site must be carefully evaluated. Soil-bearing footings or mat foundations are usually most feasible. Occasionally, friction piles are used when the stratum is quite thick. There is one known case where PIFs have been used, but a very careful determination of the location and quality of granular deposits is required prior to PIF installation.

Footings or mats are feasible, especially where the design requires that basement excavations extend down to the glaciomarine unit. A reinforced mat and waterproofed wall system are usually required if the unit is below groundwater level. Friction piles may be considered when other foundation types are not practical in this marine deposit. A conservative design assumes that all the material is cohesive and allows for a frictional resistance of 24 kilonewtons per square meter (500 pounds per square foot) for the exposed pile surface.

Till (VI). Where suitable portions of till are close to the ground surface in geologic Sequence C, the most feasible foundation type, regardless of size of structure, would be soil-bearing footings or mats. Heavy structures extending below the groundwater surface would probably require a mat. In this case, a permanent under-drainage system may be used to relieve hydrostatic pressures if there would be no adverse effect to surrounding areas or buildings. In some places, where the upper portion of the deposit is weak, piles or belled caissons are used.

Medium to heavy structures, for which shallower foundations are not practical, may be supported on piles or caissons bearing in the glacial till in Sequences A or B. The depth to the till is usually 23 meters (75 feet) or more. In this case, it is advantageous to select the till as a bearing stratum because it has a high a load capacity.

Footings or mats are usually designed for soil bearing pressures of up to 960 kilonewtons per square meter (10 tons per square foot). Even higher values may be possible, taking into consideration that the code allows an increase of 5 percent per 0.3 meter (1 foot) of depth of penetration below the top of the till, up to two times the design value at the surface of the till.

Piles are usually designed for end bearing in the deep basal till at design values up to about 1,335 kilonewtons (150 tons). Concrete-filled steel pipe piles have been used extensively, but must take into account an allowance for corrosion if they pass through a layer of organic material. In the past several decades, pre-stressed concrete piles have been widely used. The Massachusetts Building Code had allowed capacities up to 872, 1,192 or 1,558 kilonewtons (98, 134 or 175 tons) for 30.5, 35.5 or 40.6 centimeter (12, 14 or 16 inch) square sections, respectively.

Belled caissons may be appropriate where a single caisson unit can be installed below any column. Multiple units are generally not economical because of the large cap required. They may be designed for end bearing in lodgement till at capacities of up to 960 kilonewtons per square meter (10 tons per square foot). Higher values may be used for deeper penetrations into the till. In some instances, straight-shaft caissons have been used, with support from side friction as well as end bearing.

Bedrock (VII). Normally, the bedrock is encountered 23 meters (75 feet) or more below the surface. Shallower foundations are usually more economical, unless the structure is quite heavy. The rock level is closer to the surface north of Beacon Hill in the area of the West End, Government Center and Charles River Park (where a thirty-six-story apartment structure is founded on spread footings directly on argillite).

Piles may be driven to end-bearing on, or a short depth into, the rock surface, where the

overlying units do not provide adequate driving resistance. In those areas where the argillite may be weathered, the piles may penetrate 2 to 3 meters (6 to 10 feet) into the rock. Close attention must be given to the selection of an appropriate design capacity for this case, which would be coupled with selected hammer energy and necessary driving resistance.

Drilled-in caissons, as described in Section 739 of the Massachusetts Building Code, were generally limited to unusual situations where very high column loads must be accommodated and other foundation types were not feasible. Drilled shaft designs usually call for a combination of end bearing and side friction in a rock socket. However, since the early 1990s there has been substantial increase in use of such high-capacity foundations. A permanent, heavy steel, open-end casing is advanced by driving and internal cleaning to the rock surface and seated. A socket is advanced into the rock using a churn drill or other methods to a depth of 3 to 7.5 meters (10 to 25 feet). A heavy cage with rebar, or less often a heavy steel H-section column, is lowered to the bottom and set with concrete. In previous times, it was desired to dewater the bottom so the bearing surface could be inspected by a remote video camera. However, today most often the completion is done under slurry with no visual inspection prior to placing the concrete. Total capacities of 11,600 to 14,700 kilonewtons (1,300 to 1,650 tons) per unit were developed for a major tower structure in the Back Bay (Ball, 1962). Since those times, loads nearly five times those capacities have been achieved with bearing capacity on the order of 267 to 890 kilonewtons (30 to 100 tons per square foot).

More recently, drilled shafts or piles have been advanced into the rock, using temporary casing or bentonite slurry to stabilize the hole. A steel core or reinforcement is installed and the hole backfilled with cement grout to develop load in friction as well as end-bearing capacity.

Exploration & Testing Practices

Subsurface Investigations. Many of the exploration practices now in effect had their beginnings in Boston because of the work of H.A. Mohr of Gow Caisson and Raymond Concrete Pile, and Arthur Casagrande. Local exploration practice, for the most part, consists of boring and sampling methods performed in accordance with ASTM International (formerly the American Society for Testing and Materials) standards. These methods are considered to be "direct" methods, wherein borings penetrate the overburden soils and rock, and physical samples are recovered for laboratory testing and determination of the stratigraphy and geotechnical properties. Other, "indirect," principally geophysical methods such as seismic refraction, resistivity and cross-hole seismic are less likely to be used in an urban area.

Most standard borings are made using 6.3 or 7.6 centimeter (2.5 or 3.0 inch) diameter steel casing to maintain the hole through unstable soils. Larger diameter casing is used when undisturbed piston samples are required. The casing is advanced by driving and the soil within is washed out with chopping bits, roller bits and cleanout tools to the desired sampling depth. When penetrating cohesive soils, such as the clay, casing is generally not required and the hole may be stabilized by drilling mud. For deep borings that are to penetrate bouldery glacial tills, rotary drilling techniques are generally used to advance flush joint casing, using a core barrel or tricone bit. An alternate procedure is the use of hollow-stem helical flight augers, mounted on large mobile truck rigs to advance the hole and provide for soil sampling after removing a closure plate at the bottom. Only limited success is achieved in pervious materials, which are under hydrostatic pressure. Offshore drilling platforms have been used successfully in exploring for offshore bedrock tunnels for cross-harbor tunnels and outfalls for power and sewage treatment plants.

Conventional sampling procedures (SPT) are usually employed, wherein disturbed samples are obtained by driving a 5 centimeter (2 inch) outside diameter split-spoon sampler at 1.5 meter (5 foot) intervals, or at changes in sediment type, using a 63 kilogram (140 pound) hammer, dropping 76 centimeters (30 inches). Continuous sampling is sometimes used when it is important to detect frequent changes in the stratigraphy. In the clay,

relatively undisturbed samples are recovered with a 7.6 centimeter (3 inch) inside diameter stationary piston tube sampler to retrieve Shelby tube samples. Sometimes 5 centimeter (2 inch) diameter tube samplers are used.

Core drilling in the rock is performed with either BX- or NX-size core barrels. In weathered or altered argillite, the best sample recovery is obtained by using a NX-size double tube barrel with a split inner liner.

Field permeability tests are performed below the casing in boreholes at selected depths. Observation wells or piezometers (or both) are often necessary to determine the long-term stabilized water levels. Seals are required where different piezometric levels may occur at various depths within the boring. Pressuremeter tests to determine properties in place are useful in measuring the stress-strain properties of the glacial till since undisturbed sampling of the till is not practical.

Boring Data Archives. A most valuable resource with regard to available subsurface information is the collection of boring data published by the Boston Society of Civil Engineers (1961, 1969, 1970 & 1971). The Society first collected about 3,900 boring records between 1923 and 1931 and listed them and their locations in the September 1931 issue of their journal and has updated them periodically. These volumes contain tabulations of the logs and locations in the Boston Peninsula as well as South Boston and Roxbury, along with some cross-sections. The data from many sources such as architects, engineers, contractors, public agencies and others were collected and processed by volunteers. A similar effort was undertaken to publish data for the Cambridge area. Although hundreds of new borings were made for the Central Artery/Tunnel Project in Boston, these have yet to be integrated with the previously collected data. Boring data are also contained in the reports of the Massachusetts Water Resources Authority (MWRA) that are kept in their library in their headquarters in Chelsea. Many others remain in private files and must be laboriously searched out. The Massachusetts Department of Transportation maintains extensive records of borings for bridge and highway work throughout the state. The recently revived Massachusetts Geological Survey is trying to record and preserve well boring data on a more regional manner in the state. Over a hundred thousand completion reports from well drillers across the state are on file with the Massachusetts Department of Conservation and Recreation.

Core & Sample Repositories. Core and sample repositories are maintained by both the Massachusetts Highway Department and the MWRA. The MWRA has over seventy thousand linear feet of core, some of which dates back to 1898, and five hundred thin sections, all of which is recorded in a digital database. These cores provide samples from the tunnels and other structures of the MWRA in the Boston area. The material is stored presently on Deer Island, where it is hoped the facility will be upgraded for their adequate preservation. Some MWRA core is also stored at the Quabbin Reservoir. The core maintained by the Massachusetts Highway Department (formerly in Wellesley) is chiefly from exploration for the interstate highways and is stored now at a facility located in Lawrence. Access to soil and rock core samples from the Central Artery/Tunnel Project is not known.