Geotechnical Characteristics of the Boston Area

The wide range of geologic conditions influences the methods of testing a particular site as well as the type of foundation to be constructed.

EDMUND G. JOHNSON

Subsurface conditions, as created by both the natural geologic events of the past as well as the more recent activities of man, have played a contributing role in shaping the development of constructed facilities in the City of Boston since colonial times. The present-day urban planner, developer or architect who is contemplating initiating an underground construction project in this city should be sure to include on a design team members who have a firm understanding of these relationships. A thorough and well planned investigation of the site by qualified personnel is of extreme importance, since local subsurface conditions are often very complex and unpredictable.

A thorough assessment of the Boston metropolitan area's geotechnical characteristics should take into account these following factors:

Those conditions created by nature. These conditions include the depths, thicknesses, characteristics and properties of the natural soil overburden deposits and of the underlying bedrock.

Those conditions controlled by man. These conditions include location, depth, quality and information on the history of man-placed fill materials, such as those placed in the Back Bay, marginal waterfront areas and at other sites within the city.

Those conditions controlled jointly by nature and man. These conditions include the current, past and future (predicted) ranges of water levels. The water levels that result from natural runoff and infiltration often are modified by control structures (such as the Charles River Basin), sewer and tunnel routes, underdrainage systems, plus temporary construction dewatering or recharging activities.

Other such conditions include the presence of contaminants in the soil or water that would create potential environments that may be corrosive or destructive to underground construction and/or create human health hazards. These contaminants

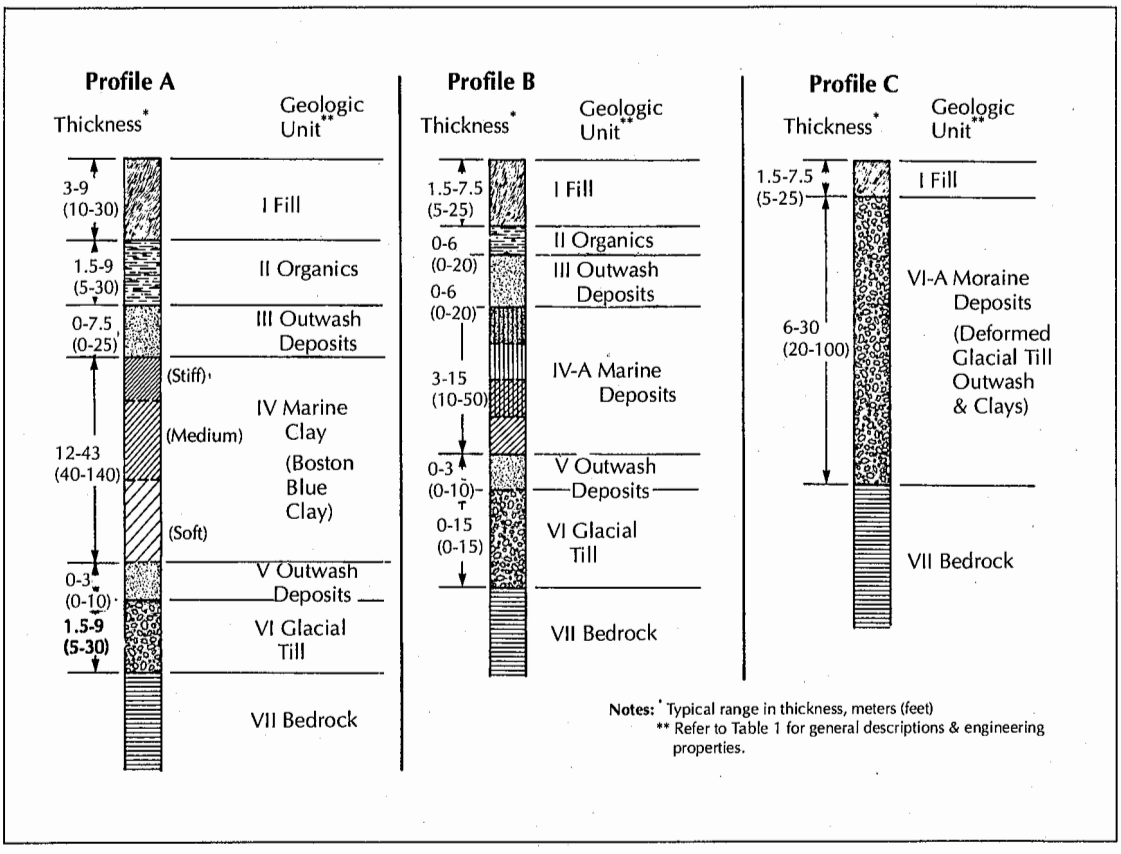

FIGURE 1. Geologic units encountered in typical major foundations.

can also be generated by organic soil deposits, decomposing landfills or uncontrolled hazardous waste sites.

Foundation Materials & Their Engineering Properties

Bedrock. Only the relatively shallow bedrock is of significance for foundation engineering purposes. The predominant upper bedrock that underlies much of Boston is argillite, referred to locally as the Cambridge Argillite.

In its fresh, unweathered condition, the argillite is typically a hard, blue-gray, finely-laminated rock. Local layers of tuff and sandstone are also typical of this formation, as well as numerous intrusive sills and dikes of diabase, diorite or basalt.

However, in many areas, the argillite is highly weathered or altered to the degree that the material can be readily crumbled between the fingers. The explanation of the processes by which the argillite was softened in these local areas is uncertain,[1] but is believed to reflect either hydrothermal alteration or extensive weathering. The altered argillite may vary from light gray to dark green in color. The distribution of the alteration is commonly quite erratic, and the alteration tends to follow the layering of the steeply folded strata. However, the intrusive rock units within the rock mass were apparently less affected by the alteration, and are predominantly moderately hard to hard. These harder intrusive layers may range in thickness from several centimeters to a meter or more (a few inches to many feet).

Where the entire rock mass has been altered, it generally retains a recognizable bedrock fabric such as foliation, bedding or jointing. The rock mass near the top of the altered zone can often be most accurately described as a soil. With increasing depth, the fabric of the rock becomes more evident.

The bedrock surface topography below Boston is quite irregular. A map showing the ap-

proximate contours of the bedrock surface has been developed from interpretation of data from boring logs and construction activities by Kaye.[2] Generally, the rock surface is at a depth of 23 to 53 m (75 to 175 ft) below the surface. The Back Bay borders the eastern edge of a deep bedrock valley that extends to known depths of at least 67 m (220 ft). On the other hand, the rock nearly crops out at the surface in a local area to the northwest of Beacon Hill.

The Cambridge Argillite, which predominates throughout the Boston Basin, has extremely variable engineering properties. The unweathered, unaltered rock may be quite sound. It is so sound that vertical cuts will remain stable with little or no support, and bearing intensities of up to 5,800 kiloNewtons per square meter (kN/m^2) (60 tsf) or more may be appropriate. On the other hand, highly altered zones may have properties similar to a medium or soft cohesive soil. Because of the potential variability, both vertically and laterally, within short distances, a very thorough, well-planned exploration program is warranted if foundation support or other construction is planned on or within the rock.

Conglomerates may be encountered locally in such places as the south and west of the city in portions of Roxbury and Brookline. In contrast to the argillite, it is a very hard, durable stone that was often used in the late 19th century for building and retaining wall construction.[3] It is usually a mottled brown in color, with embedded round to angular pebbles, and resembles a dense concrete material. Surfaces on the conglomerate may be extremely uneven, since these materials were not easily eroded by subsequent periods of glaciation. Construction excavation or drilling of this massive rock may be very difficult, due to its hardness and lack of natural jointing or fracture planes.

Overburden. The overburden in the Boston area is typified by three general sequences, or profiles, for the purposes of foundation construction (see Figure 1):

- Profile A is the most typical and is found below the filled-in Back Bay and marginal waterfront areas.
- Profile B is representative of intermediate areas adjacent to the original Boston Peninsula.
- Profile C is most complex and is found typically within the limits of the original colonial shoreline of the Boston Peninsula.

A description of the geologic units in these sequences, together with their typical engineering properties, is presented in Table 1.

Glacial Till (Unit VI): This unit directly overlies the bedrock throughout much of the Boston area. It is usually of the lodgement variety and it forms a very compact, unsorted, generally non-stratified mixture, of rock fragments and minerals of all sizes, ranging from clay and silt-size particles to cobbles and boulders. The rock fragments are often broken pieces of the underlying bedrock material. The till is extremely variable as a result of the very complex processes of deposition. Pockets and layers of pervious sands and gravels, as well as zones of plastic silts and clays, are often encountered within the mass.

The Standard Penetration Test (SPT) is often the only practical field test to determine an indication of the in-situ density. N-values of over 80 blows per 30 cm (1 ft) are typical where there is more than 15 m (50 ft) 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 emphasized that the method by which N-values are determined are not precise and unusually high values for individual tests may reflect the presence of gravel, cobbles or boulders encountered by the sampler.

Sample recovery is often poor, and visual examination and classification are often made on very limited quantities. In-situ testing with a pressuremeter device may be appropriate for certain projects. Whenever possible, grain-size and hydrometer tests should be performed as well as Atterberg Limits on cohesive portions. Typical grain-size distribution curves usually indicate a widely graded material with 10 to 25 percent or more of the grains finer than a number 200 sieve.

Outwash Deposits (Unit V): These glaciofluvial deposits consist of medium dense, stratified sands and gravels of a discontinuous nature that overlie the lodgement till.

TABLE 1
Typical Engineering Properties of Foundation Material in Boston

Geologic Unit	General Description	Saturated Unit Weight kg/m³ (lb/ft³)	Natural Water Content (percent)	Atterberg Limits (percent) LL	PI	Undrained Shear Strength kg/m² (lb/ft²)	Other	Allowable Bearing Pressure kg/m² (lb/ft²)
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, grey 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%	—
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)	—	—	—	—	—	19500-48800 (4000-10000)
IV. Marine Clay	Stiff, yellow-grey silty clay.	1840-2000 (115-125)	25-35	40-55	15-30	3900-9760 (800-2000)	Compression Ratio = 0.15-0.25	14650-39000 (3000-8000)
	Medium stiff, grey silty clay, occasional layers of fine sand or silt.	1824-1920 (114-120)	30-40	40-55	15-30	2930-5860 (600-1200)	Recompression Ratio = 0.02-0.04	9760-19500 (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)		4880-9760 (1000-2000)
IV-A. Marine Deposits	Interbedded grey silty or sandy clay, silty fine sand & fine sandy silt	Too variable	—	—	—	—	—	Variable
V. Outwash Deposits	Medium to dense, stratified sands & gravels in discontinuous layers.	—	—	—	—	—	—	Variable
VI. Glacial Till	Dense to very dense, heterogenous mixture of sand, gravel, clay & silt with cobbles & rock fragments.	2000-2240 (125-140)	10-20	15-30	10-20	9760-39000 (2000-8000)	—	39000-98000 (8000-20000)
VI-A. Moraine Deposits	Miscellaneous deposits of deformed glacial till, outwash & clays.	Too variable	—	—	—	—	—	Variable
VII. Bedrock	Cambridge Argillite.	—	—	—	—	—	—	78000-195000 (16000-40000)
	Roxbury Conglomerate.	—	—	—	—	—	—	195000-975000 (40000-200000)

Note: Metric units above English units in parentheses.

Marine Clays (Unit IV): These clay deposits are usually referred to locally as the Boston Blue Clay. Its properties have been investigated thoroughly for foundation design. There is generally very little clay directly below the original downtown Boston peninsula, but in the Back Bay, as well as along marginal waterfront areas, the clay is typically 15 to 38 m (50 to 125 ft) thick. Even greater thicknesses, up to 60 m (200 ft), have been found to the west of Massachusetts Avenue and in the City of Cambridge.

A weathered crust is present at the top of the clay. This crust is the result of desiccation, oxidation and capillary stress. It 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 test programs have been performed on the clay by researchers and practitioners over the past 40 years. Tests on samples from the site of the Prudential Center[4] and the Massachusetts Institute of Technology (MIT) in Cambridge,[5] reveal that the stiff yellow clay has been pre-consolidated (compacted) to four or more times the present overburden stress. The overconsolidation ratio decreases quite rapidly with depth, so that the clay below a depth of about 21 to 27 m (70 to 90 ft) is considered to be normally consolidated.

Discontinuous layers and lenses of sand and silt are often encountered within the clay. Thus, horizontal permeability is generally several times greater than the vertical. Typical ranges of undrained shear strength and other engineering properties are given in Table 1.

Marine Deposits (Unit IV-A): These deposits were formed in areas that were inundated with marine waters at locations close to the shoreline, creating a complex depositional environment. Quantities of silt and clay-sized particles, discharged by glacial meltwater streams into the sea, slowly settled out of suspension to form strata of clay. Simultaneously, sands and silts were deposited by meltwater streams and near-shore currents. As a result, a highly complex marine deposit of alternating and interfingering layers of fine sand, or silt and clay, developed in these areas. It is not practical to try to typify the engineering properties for this unit since its composition varies so widely.

Outwash Deposits (Unit III): Sand and gravel was deposited over the surface of the weathered clay in some areas, following another advance of glacial ice. These well-stratified sands and gravels range in thickness from 3 to 7.5 m (10 to 25 ft). They are medium to compact and are considered an important bearing stratum for supporting light to medium weight structures. Their relatively high permeability is also important.

Organic Deposits (Unit II): Organic silt and clay deposits were formed throughout much of the lower lying areas surrounding the Boston Peninsula following the ice age. The organic deposits vary greatly in overall thickness and content, but are generally from 1.5 to 7.5 m (5 to 25 ft) thick. In those filled-in areas of the Back Bay, this layer has been compressed considerably due to the weight of the fill. Marsh gas, resulting from the decomposing organic matter, is sometimes encountered in excavations.

Man-Placed Fills. Low-lying areas began to be filled starting in the late 18th century when colonial Boston outgrew the limited area of the original peninsula.[6] Previously, the entire Back Bay area was a mud flat and the Charles River was a tidal estuary. A mill dam was first constructed in 1820 along what is now Beacon Street, from Charles to Kenmore Square, to harness tidal power. Subsequently, railroad embankments were built across Back Bay. This construction, in effect, created stagnant water areas that eventually were filled in for development purposes. Between 1856 and 1890, the entire Back Bay between Charles Street and the Fenway was filled. The fill materials consisted of clean sands and gravels, brought by rail from a source in Needham, about 15 km (9 miles) to the west. Following the construction of a tidal dam across the Charles River in 1910 that controlled the water level in the Basin, embankment fill was placed along the river and Storrow Drive was completed in 1951. On the Cambridge side, the tidal marshes were filled in and the area was developed, including the present campus of MIT, following the construction of a granite seawall about 1890. The waterfront areas facing Boston Harbor were also filled in by stages. This process was quickly followed by pier and bulkhead construction. In general, the materials used for these fills were earth remnants from several high land areas within the original Boston Peninsula, such as Fort Hill, plus dredged materials and demolition rubble.

Groundwater Levels

General Conditions. As might be expected, the normal groundwater level in Boston and the Back Bay area is generally close to mean sea level. Although the normal tide range in the harbor is about 1.5 m (5 ft) above and below mean tide, similar fluctuations in groundwater levels below the city are usually not observed, except along marginal waterfront areas. A

stabilizing factor results from the Charles River Basin being maintained at about 0.73 m (2.4 ft) above mean sea level.

Variations and anomalies in the piezometric surface are often related to the dewatering implemented for construction projects or possibly pumping from deep basements. Leakage from or into storm sewers is another factor. The many subway tunnels and deep utilities in the area often form either barriers or drainage paths that interrupt or control normal groundwater flow.

There is evidence that the sea level was considerably lower in the past. Freshwater peat and tree stumps, as well as ancient Indian fish weirs have been found in excavations at levels from 5 to 7.5 m (15 to 25 ft) or more below the present mean sea level.

Influence on Constructed Facilities. Groundwater levels are a key factor in any geotechnical assessment of conditions in the Boston area. The determination of realistic present water levels, as well as past and potential future variations, are of major significance. In Boston, the consequences 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 — may occur if the water is depressed in areas underlain by soft compressible layers, such as the filled-in Back Bay. The settlement would occur very slowly and the magnitude would reflect the relative thickness of the soft underlying soils.
- Individual buildings supported on untreated wood piles may settle if the pile butts are exposed to drying and decay. As long as the piles are constantly submerged, and not exposed to air, they will not be attacked by fungi. Also, the lowered water level may result in the consolidation of the soils surrounding the piles, and thus the frictional forces that develop along the piles may create additional loads for which the piles were not designed.

The maintenance of "normal" water levels has been very important concern to city officials during the past century. After the filling of the Back Bay, most structures were supported on untreated wood piles, driven to bearing in the sand layer below the organic deposits or as friction piles in the clay. Since the groundwater level was at that time found at approximately 0.7 m (2.3 ft) above mean sea level, the piling was usually cut off at 0.6 to 0.9 m (2 to 3 ft) below this grade. However, with the subsequent effects of decreased surface infiltration and as the areas were developed (for example, dewatering for tunnels and drainage systems or other local pumping activities), it was discovered that the wood piles below many structures were no longer permanently submerged, and that these piles became exposed to drying and decay.

A notable example of the problem occurred in 1929, when major cracks were discovered in the walls of the Boston Public Library at Copley Square.[7] Upon investigation, it was discovered that the tops of wood piles were decaying. A major underpinning effort was required to restore the foundation system. The apparent reason for the lowering of the water table was traced back to the earlier construction of storm and sanitary sewer lines, with invert levels about 1.8 m (6 ft) below water table. Steps were taken to control the infiltration and restore the levels to normal.

More recently, problems with foundation distress and rotted piles have occurred in the lower Beacon Hill area. Investigations have revealed that the groundwater levels were as much as six feet below the water level in the Charles River. These lowered levels are attributed to leakage into sewers. Aldrich and Lambrechts provide an excellent historical perspective on groundwater fluctuations in the Back Bay and on the adverse effects of lowered levels.[8]

Due to the adverse effects of a drop in the groundwater level, it is a requirement that an adequate cutoff system be installed to control drawdown beyond the site during any construction excavation below the groundwater level. Adjacent areas must be monitored and, if necessary, remedial action must be taken such as modifying the pumping operation or installing a recharge system.

Exploration & Testing Practices

Subsurface Investigations. Local exploration practice, for the most part, consists of boring and sampling methods that are performed in accordance with American Society for Testing and Materials (ASTM) standards. These methods are considered to be "direct", and consist of borings that penetrate the overburden soils, the recovering of rock and physical samples for laboratory testing, and determining the stratigraphy and geotechnical properties. Other "indirect" and principally geophysical methods such as seismic refraction, resistivity and cross-hole seismicity tests are less likely to be used in the urban area. The exploration work is generally contracted to one of several qualified independent drilling firms, with the field work being monitored by representatives of the consultant for the construction project.

Most standard borings are made using a 6.3 or 7.6 cm (2.5 or 3.0 in) 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 and clean-out tools to the desired sampling depth. When penetrating cohesive soils, such as the clay, the 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 the flush-joint casing by 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 in order to advance the hole and provide for soil sampling after the removal of a closure plate at the bottom. This method achieves only limited success in pervious soils that are under hydrostatic pressure.

Conventional sampling procedures are usually employed, wherein disturbed samples are obtained by driving a 5 cm (2 in) outside diameter split-spoon sampler at 1.5 m (5 ft) intervals, or at change in soil type, using a 63 kg (140 lb) hammer dropping 76 cm (30 in). 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 cm (3 in) inside diameter stationary piston tube sampler or 5 cm (2 in) Shelby tube.

Core drilling in the rock is accomplished with either BX- or NX-size core barrels. In weathered or altered argillite, the best sample recovery method is the use of an NX-size double tube barrel with a split inner liner.

Field permeability tests are performed below the casing in boreholes at selected depths. The use of observation wells or piezometers, or both, are often necessary in order 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 the in-situ properties are useful in measuring the stress-strain properties of the glacial till, since undisturbed sampling of the till is not practical.

Available Boring Data. A most valuable resource with regard to the available subsurface information is the collection of boring data published by the Boston Society of Civil Engineers Section/ASCE.[9,10,11,12] These volumes contain the tabulations of the logs of several hundred borings that are located in the Boston Peninsula as well as in South Boston and Roxbury. The data were collected from many sources such as architects, engineers, contractors, public agencies and others. A similar effort was undertaken to publish data for the Cambridge area.[13]

Laboratory Testing. Laboratory testing is performed primarily in the private laboratories of geotechnical consultants in connection with specific projects. During the past 30 years, Boston area soils have been tested extensively at both Harvard University and MIT, either for particular projects or for general research. The Boston Blue Clay is considered one of the most thoroughly tested and researched soils in the world. Of particular note is the work done by Arthur and Leo Casagrande in the mid-1950s during design investigations for the Prudential Center in the Back Bay.[4] The prediction of the consolidation behavior of the clay by this work was critical for the project. During the 1960s there was considerable construction activity on the MIT campus, across the Charles River in

Cambridge. A program called Foundation Evaluation and Research-MIT (FERMIT) included extensive laboratory investigations performed on campus subsoils, particularly the blue clay. The published results from the work of this program are useful in understanding the behavior of the local soils.[5]

Foundation Types Used for Local Geologic Conditions

Selection of Appropriate Foundation Systems. Foundation designs in Boston must comply with the Massachusetts Building Code. The first state-wide Building Code was issued in 1975. Article 7, Structural and Foundation Loads and Stresses, was incorporated, nearly intact, from the then-existing City of Boston Code. Subsequent code revisions have been made, which allow for increased loads in piles and other changes as well as provisions for the design of foundations to withstand earthquakes. The seismic criteria were the first such criteria developed specifically for a jurisdiction in the eastern United States.

The soil and foundation seismic criteria in Section 716 of the code are innovative and comprehensive. The design philosophy recognizes that the probable maximum earthquake intensities for Massachusetts may be as large as those for California, but have much longer return periods. The criteria aim to minimize the loss of life in the event of major earthquakes, but without imposing excessive construction costs. The code emphasizes the ductility requirements for structures and it prescribes relatively modest lateral forces. Most "model" codes simply apply a "zone factor" to the lateral force requirements that were developed for California. The code also has specific provisions regarding potential liquefaction, earthquake-induced lateral earth pressures and the effects of local soil conditions.

The state Board of Building Regulations and Standards is responsible for the administration of the code. Technical assistance is provided by Loads, Geotechnical and Seismic Advisory Committees. These committees are composed of selected volunteer practicing professionals who periodically review code provisions and recommend revisions and additions. Changes are incorporated into the code following a public review process.

Foundation Types Used in Various Geologic Units. The foundation types considered appropriate for bearing within each of the geologic units found in each of the three typical geologic sequences are illustrated in Figure 2. These foundation types are keyed to the three sequences, or profiles, labeled A, B and C in the figure. Those shown are considered as representative of most, but not all, of the foundation types used in the Boston area.

Fill & Organic Deposits: The fill and the organic layers are not suitable for the support of any significant structures.

Outwash Deposits: Light to medium weight structures may be supported on short piles or caissons (cast-in-place shafts or piles) in profiles A or B. If the sand layer is relatively shallow, it may be feasible to use spread footings. For profile A, estimates must be made of the post-construction settlement of the underlying clay (Unit IV). Usually, the settlements of buildings with up to 10 to 12 stories and having one 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 additional foundation loading.

Untreated wood piles (III-1) were used predominantly for early construction in the Back Bay. Typical pile capacities of 62 to 89 kN (8 to 10 tons) were most common when driven to bear in the sand layer. However, problems can develop if the pile butts are exposed to drying. Therefore, they must always remain submerged below the groundwater level. Otherwise, the use of pressure-treated piles can overcome this problem.

Pressure-injected footings (PIFs) (III-2) that offer individual capacities up to 1070 kN (120 tons) or more are feasible where the layer has a proper grain-size distribution (less than 15 percent fine material) and the layer thickness is at least 3 m (10 ft). The PIFs are a unique pile type, and consist of advancing a heavy steel drive tube into the sand surface and then driving out one or more batches of very dry concrete mix, 0.14 m^3 (5 ft^3) each, to form an expanded base within the granular material. A concrete shaft is then formed above the base to complete the unit.

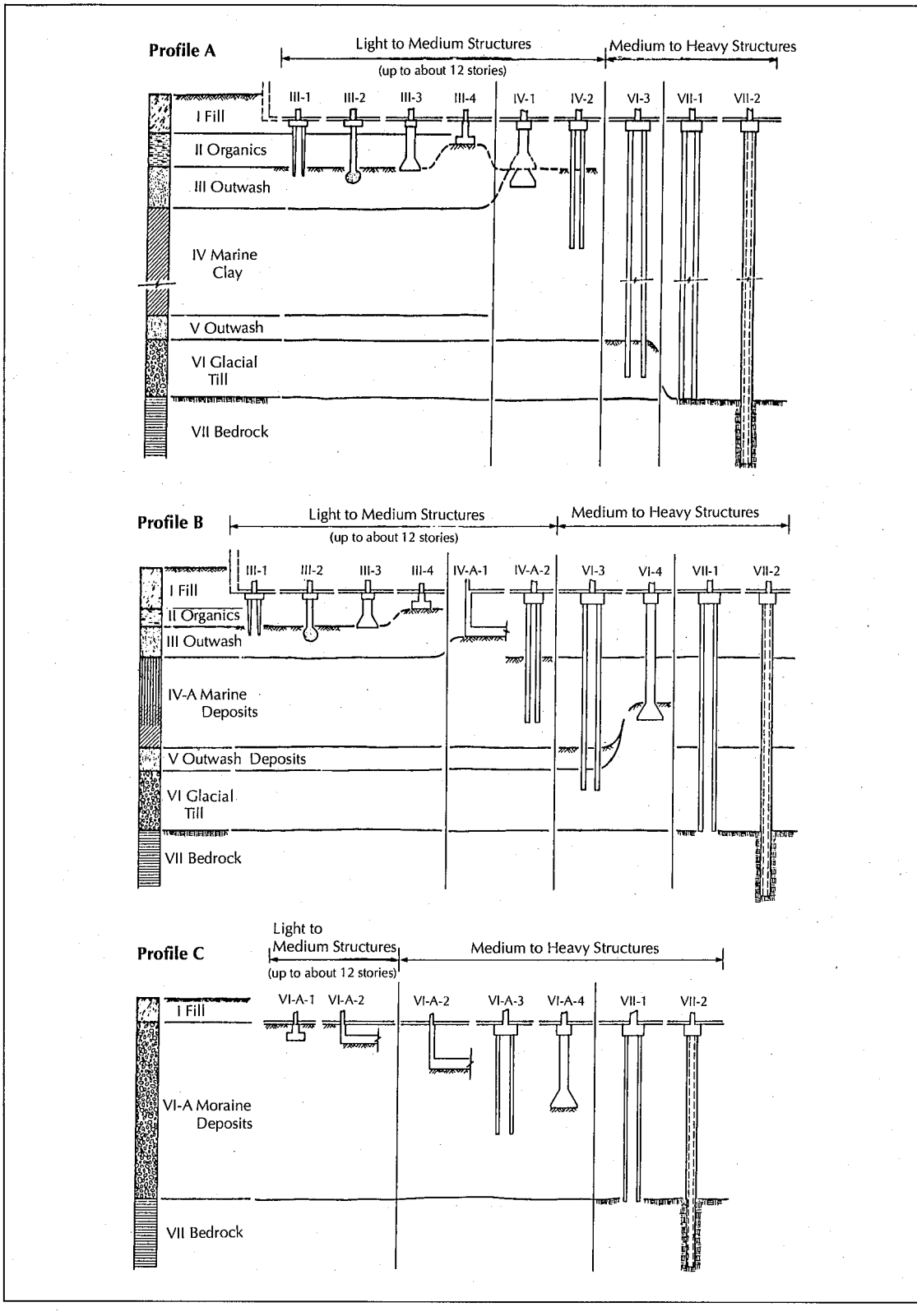

FIGURE 2. Typical foundation types used in Boston.

Belled caissons (III-3) may be installed to bear on the sand layer 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 the concrete is placed. It is sometimes necessary to dewater the sand bearing layer in the vicinity by installing wells, provided there will be no adverse effects from the dewatering in the adjacent area.

Spread footings (III-4) are feasible where the depth to the top of the bearing layer is only a meter or so (few feet) and dewatering does not pose a serious problem. The units are usually sized for a bearing value of 240 to 480 kN/m^2 (2.5 to 5.0 tsf).

Marine Clay: For profile A, some light to medium structures are founded directly on or within the Boston blue 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 Unit IV. Steel casings are advanced through the upper soils and sealed into the organic deposits or clay surface. The belled portions are then undercut by a rotary machine to a diameter of 1.8 to 3 m (6 to 10 ft) or more. In the early days, this was done by hand labor. Usually there is little or no dewatering required. The units are typically designed for an end-bearing capacity of 190 to 380 kN/m^2 (2 to 4 tsf) in the upper stiff clay zone. If that zone is fully penetrated, the caissons bearing on the softer clays below would have a reduced design capacity. In all cases, the strength of the clay should be verified in the field by competent geotechnical personnel.

Friction piles (IV-2) are also used to provide support in the upper clay. Wood piles with a capacity up to 196 kN (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 kN/m^2 (500 psf) for the portion embedded in the clay. Any such installation should be verified by on-site pile-load tests.

Marine Deposits: Light to medium structures are founded on or within profile B, especially when the overlying sand layer (Unit III) is absent or 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 the most feasible foundation system. Occasionally, friction piles are used. There is one known case where pressure-injected footings have been used. A very careful determination of the location and quality of granular deposits was required prior to installation.

Footings or mats (IVA-1) are feasible, especially where the design requires that basement excavations extend down to this unit. Where the foundation extends below the water level, a reinforced mat and waterproofed wall system are usually required.

Friction piles (IVA-2) may be considered when other foundation types are not practical. A conservative design would be to assume that all of the material is cohesive and allows a frictional resistance of 24 kN/m^2 (500 psf) for the exposed pile surface in the marine deposit.

Glacial Till: Where suitable portions of the moraine are close to the surface in profile C, the most feasible foundation type, regardless of the size of the structure, would be soil-bearing footings (VI-1) or mat (VI-2). Heavy structures extending below the water table would probably require a mat (VI-2). In these conditions, permanent underdrainage systems may have to be considered in order to relieve hydrostatic pressures. In some instances, where the upper portion of the deposit is weak, piles (VI-3) or belled caissons (VI-4) are used.

Medium to heavy structures, for which shallower foundations are not practical, may be supported on piles (VI-3) or caissons (VI-4) bearing in the glacial till in profiles A or B. The depth to the till is usually 23 m (75 ft) or more. In this case, it is advantageous to select a unit with as high a load capacity as possible.

Footings or mats (VI-1, VI-2) are usually designed for soil bearing pressures of up to 960 kN/m^2 (10 tsf). Even higher values may be possible, taking into consideration that the code allows an increase of five percent per foot of depth of penetration below the bearing soil, up to two times the design value at the surface.

Piles (VI-3) are usually designed for end-bearing in the glacial till at design values up to

about 1,335 kN (150 tons). Concrete-filled steel pipe piles have been used extensively as piles, but the design must take into account an allowance for corrosion if the piles must pass through a layer of organic material. In the past several years, prestressed concrete piles have been widely used. The current Massachusetts code allows capacities up to 872, 1,192 or 1,558 kN (98, 134 or 175 tons) for 30.5, 35.5 or 40.6 cm (12, 14 or 16 in) square sections, respectively.

Belled Caissons (VI-4) 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 the lodgement till at capacities of up to 960 kN/m^2(10 tsf). 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: Normally, the bedrock is located 23 m (75 ft) or more below the surface. Unless the structure is quite heavy, shallower foundations are usually more economical. The rock level is close to the surface west of Beacon Hill where a 36-story apartment structure is founded on spread footings that rest directly on the argillite.

Piles (VII-1) may be driven to end-bearing on 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 into the rock. Close attention must be given to the selection of an appropriate design capacity for this case.

Drilled-in caissons (DIC) (VII-2), as described in Section 739 of the code, are limited to unique situations where very high column loads must be accommodated and other foundation types are not feasible. DIC design usually calls for a combination of end-bearing and side friction in a rock socket. A permanent heavy steel open-end casing is advanced by driving and internal cleaning to the rock surface and then seated. A socket is advanced into the rock using a churn drill or other methods to a depth of 3 to 7.5 m (10 to 25 ft). A heavy steel H-section is lowered to the bottom and concreted. If the water is unable to be dewatered, it must be inspected by remote video camera prior to concreting. Total capacities of 11,600 to 14,700 kN (1300 to 1650 tons) per unit were developed for a major tower structure in the Back Bay.[14]

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 is backfilled with cement grout in order to develop the load in friction as well as in end-bearing.[15]

EDMUND G. JOHNSON *is a Principal of Haley & Aldrich, Inc. He joined the firm in 1959 and has wide experience in a variety of projects, with emphasis on the geotechnical aspects of major building projects in urban areas. He received his B.S. in civil engineering from Worcester Polytechnic Institute in 1951 and an M.S. from Princeton University in 1953. He is a member of BSCES and a Fellow of ASCE, and is a registered professional engineer in Massachusetts and New York.*

REFERENCES

1. Kaye, C.A., "Kaolinization of Bedrock of the Boston, Massachusetts, Area," U.S. Geol. Survey, Professional Paper 575-C, 1967, pp. C165-C172.

2. Kaye, C.A., "Bedrock Geology, Boston North, Boston South and Newton Quads," U.S. Geol. Survey, Misc. Field Studies Map, MF 1241, 1980, scale 1:24,000.

3. Kaye, C.A., "The Geology and Early History of the Boston Area of Massachusetts — A Bicentennial Approach," *U.S. Geol. Survey, Bulletin 1476*, 1976, 78 pp.

4. Casagrande, A.L., "Foundation Investigation for Prudential Center in Boston," unpublished report, Metcalf & Eddy, Engineers, 1958.

5. Ladd, C.C., & Luscher, U., "Engineering Properties of the Soils Underlying the MIT Campus," MIT Dept. of Civil Engineering, Research Report R65-58, Soil Mechanics Publication 185, 1965.

6. Aldrich, H.P., Jr., "Back Bay Boston, Part 1," *Journal of the Boston Society of Civil Engineers*, V. 57, 1970, pp. 1-33.

7. Aldrich, H.P., Jr., "Preserving the Foundations of Older Buildings: Technology & Conservation," Summer 1979.

8. Aldrich, H.P., Jr., & Lambrechts, J.R., "Back Bay Boston, Part II: Groundwater Levels," Boston Society of Civil Engineers, *Civil Engineering Practice*, V. 1, N. 2, 1986, pp. 31-64.

9. Boston Society of Civil Engineers, "Boring Data from Greater Boston," *Journal of the Boston Society of Civil Engineers*, 1961, 280 pp.

10. Boston Society of Civil Engineers, "Boring Data from Greater Boston: Sec. 1, Boston Peninsula," *Journal of the Boston Society of Civil Engineers*, V. 56, 1969, pp. 131-291.

11. Boston Society of Civil Engineers, "Boring Data from Greater Boston: Sec. 2, Roxbury," *Journal of the Boston Society of Civil Engineers*, V. 57, 1970, pp. 185-291.

12. Boston Society of Civil Engineers, "Boring Data from Greater Boston: Sec. 3, S. Boston," *Journal of the Boston Society of Civil Engineers*, V. 58, 1971, pp. 51-63.

13. Boston Society of Civil Engineers, "Boring Data from Greater Boston: Sec. 4, Cambridge," *Journal of the Boston Society of Civil Engineers Section/ASCE*, V. 70, Nos. 1 & 2, 1984.

14. Ball, Donald G., "Prudential Center Foundations," *Journal of the Boston Society of Civil Engineers*, V. 49, 1962, pp. 229-244.

15. Johnson, E.G., & Schoenwolf, D.A., "Foundation Considerations for the Expansion & Renovation of the Hynes Auditorium," Boston Society of Civil Engineers, *Civil Engineering Practice*, Vol. 2, No. 2, 1987, pp. 35-62.