The Use of Slurry Caissons for High-Rise Buildings

The application of new foundation construction techniques requires a thorough program of geotechnical study.

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A deep foundation design utilizing rock-socketed concrete caissons installed with a bentonite slurry technique was developed for a high-rise office building in downtown Boston, Massachusetts. Since little precedence for this design existed in the Boston area, a detailed rock engineering and geological study was undertaken. The program included several in-situ rock testing methods and a complementary program of laboratory testing. This effort yielded new technical information on the Boston Peninsula bedrock and experience with a new foundation procedure.

The Site

The project, known as 101 Federal Street, consisted of a 31-story office building with two underground parking levels. It is located on an approximately 20,000-square-foot parcel of land between Devonshire Street (west of site) and Federal Street (east) in downtown Boston as shown in Figures 1 and 2. Construction was begun in 1985 and was completed in 1987.

Two existing structures abut the site: 75 Federal Street to the north, and a City of Boston Parking Garage to the south. Seventy-five Federal Street is a 21-story office building that was constructed in 1929 on a caisson foundation bearing in glacial till and is part of the new development. The garage consists of five levels supported by a shallow foundation bearing in fill above a clay layer.

The combination of heavy column loads and a relatively small building footprint would have required a very dense array of high-capacity driven piles. The driving of these piles, even with preaugering, would have potentially disturbed the 75 Federal Street foundations. As a result, a high-capacity foundation type that was an alternative to driven piles was sought and developed. A drilled caisson system was selected. This system had the advantages of non-displacement and low vibration features as well as competitive cost and an accelerated construction schedule. The main disadvantage to this system was that the installation technique did not allow for direct visual observation of the caisson bottom. However, a monitoring program was specifically designed to over-

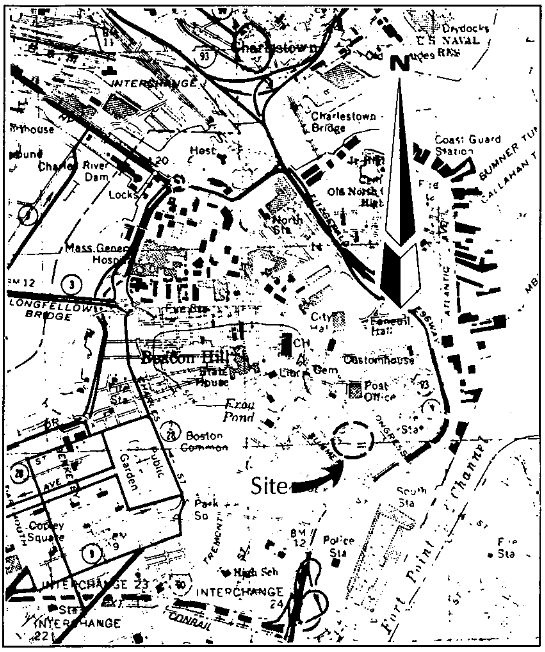


FIGURE 1. Location of the site in downtown Boston.

come this limitation.

Subsurface Exploration Program

Prior to initiating extensive work on the design, a preliminary subsurface exploration program that included seven borings was performed during December 1984. The locations of these and subsequent borings are shown on Figure 2. These borings provided detailed information on overburden soil strata and limited information on the underlying bedrock formation.

The preliminary program included Standard Penetration Testing (STP) in each borehole and shallow rock coring at three of the seven locations. These borings indicated that (in order of increasing depth) approximately 10 to 15 feet of fill, 40 to 55 feet of medium to stiff silty clay, and 8 to 14 feet of very dense glacial till were present above a severely weathered bedrock surface 60 to 80 feet below the ground surface.

To evaluate the bedrock conditions in detail for the feasibility of utilizing the bedrock for high capacity rock-socket caissons, a second phase of explorations was taken during June and July 1985. This phase included seven "rock core" borings.

The rock core borings ranged in depth from 112 to 125 feet below ground surface. Fill and silty clay were penetrated to glacial till using a hollow stem auger. At selected locations, split spoon samples were obtained in the glacial till and in an underlying transition zone of badly decomposed bedrock. When a depth was achieved where it was deemed feasible to recover rock core, bedrock was cored continuously in 1- to 5-foot sections using an NX-size split tube core barrel sampler. Between 27 and 45 feet of bedrock were cored at each location.

Upon the completion of the rock coring phase, three kinds of in-situ borehole tests were performed:

- Permeability tests were performed in the bedrock of four of the boreholes using inflatable packers.
- Goodman Jack tests were performed in the rock in four of the boreholes.
- Upon completion of the above field tests, 2.5-inch-inside-diameter PVC pipe was installed to the bottom of four of the holes for a cross-hole velocity survey.

Rock cores were logged in detail and eight specimens of intact rock were selected for laboratory compression testing. In December 1985, six additional borings were drilled to provide specific data at heavily loaded column locations.

Bedrock Geology

United States Geological Survey (USGS) maps by Clifford A. Kaye, as well as discussions with the author of the maps, indicated that the general site area is underlain by a complex assortment of sedimentary geologic units.[1] This bedrock, which underlies virtually all of downtown Boston's high-rise business district, is composed of soft and altered, interlayered tufaceous sediment and argillite, sandstone, and quartzite.

The geologic map also indicated a secondary bed of conglomerate passing through the rock formation. This conglomerate has an east-west strike and extends from the vicinity of the Boston Common east to the waterfront. On the map, this conglomerate is shown to lie ap-

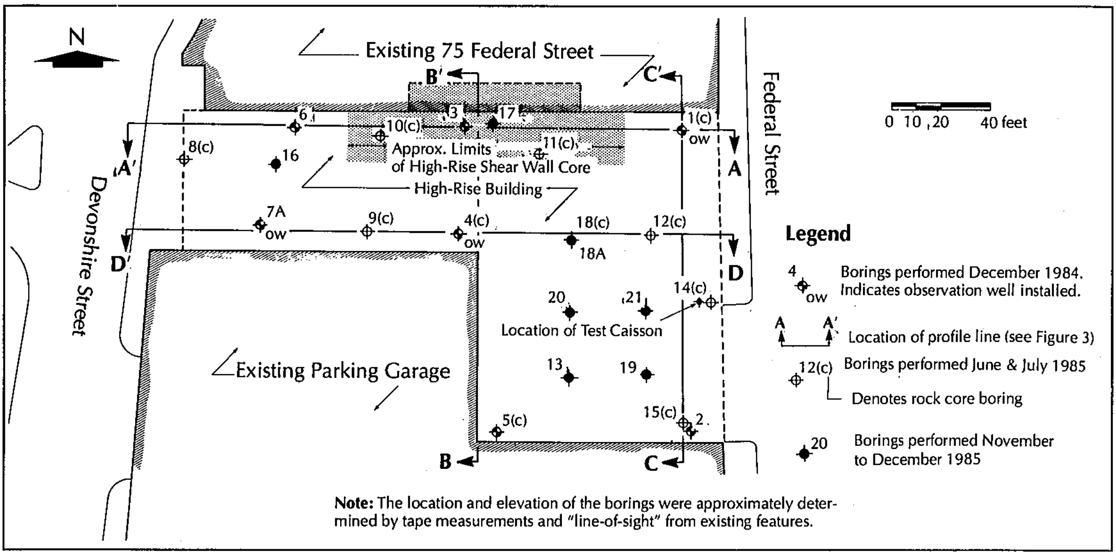


FIGURE 2. Exploration and test caisson location plan.

proximately just north of the 101 and 75 Federal Street block. Results of the subsurface exploration program indicated that the conglomerate bed actually passes through the north portion of the 101 Federal Street site.

These subsurface conditions are portrayed on the subsurface sections shown on Figures 3 through 5 based on interpretation of the boring data. In general, the bedrock consists of the following three basic units:

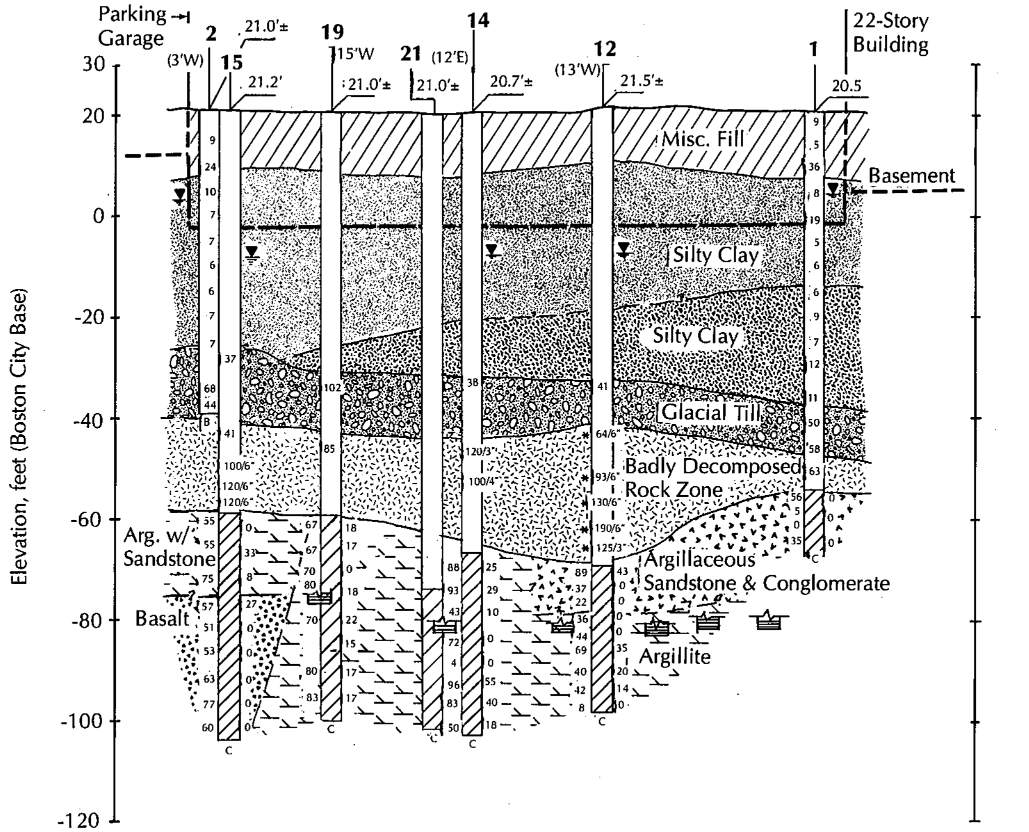
- *Badly Decomposed Argillite* — underlies glacial till in an approximately 5- to 20-foot thickness; kaolinized argillite severely to completely weathered to hard clay-like consistency; overlies more competent sandstone and conglomerate, and argillite.
- *Sandstone and Conglomerate* — predominant in the northern portion of site; soft to hard, moderately to severely weathered, argillaceous sandstone and conglomerate.
- *Argillite* — predominant in the south portion of the site; very soft argillite; moderate to severe weathering; underlies sandstone and conglomerate in northern portion of the site.

In-Situ Bedrock Testing Program

A program of in-situ tests were conducted to characterize the bedrock as a mass. This program consisted of permeability tests, measurements of stress-strain modulus of the rock in the specific vicinity of the borehole with a Goodman Jack, and cross-hole velocity tests with seismic methods.

Packer Permeability Tests. Permeability tests were performed in order to evaluate the variability of the rock mass by measuring its capacity to accept water under pressure. Zones in the NX-size rock core holes at four locations were isolated using inflatable packers and then were hydraulically pressurized. The test results are summarized in Figure 6. A general trend of decreasing permeability with depth below the top of the badly decomposed bedrock surface is evident. This trend suggests that the joints and fractures of the rock are tighter and less continuous with depth. These data support the notion that the effects of weathering on bedrock are usually more severe near the top of the previously exposed bedrock surface and attenuate with depth.

Since the weathering process weakens the structure of intact rock, the engineering properties of the rock as related to foundation support are expected to be more favorable with depth. The permeability data suggest that, in general, a "tighter," more competent rock is encountered at depths of more than 20 feet below the bedrock surface.



Soil Descriptions:

Miscellaneous Fill — Loose to very dense granular fill including brick with occasional zones of silty clay (fill).

Silty Clay — Medium stiff to stiff gray silty clay to clay and silt with some fine sand layers or little sand.

Silty Clay — Medium stiff to very stiff gray silty clay to clay and silt.

Glacial Till — Hard gray silt and clay with little fine to loose sand and gravel (cohesive till) or very dense gray gravel with little fine to coarse sand and clayey silt.

Badly Decomposed Rock Zone — Kaolinized argillite, severe to complete weathering.

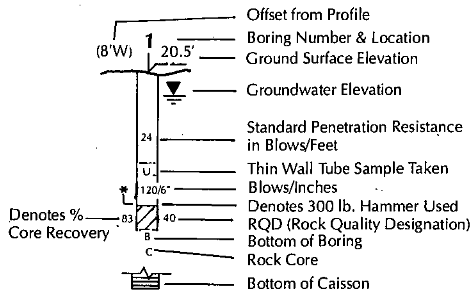
Argillite — Green, very soft argillite, moderate to severe weathering

Argillite with Sandstone — Soft, moderately to severely weathered argillite with interbedded calcareous sandstone.

Argillaceous Sandstone and Conglomerate — Soft to hard moderately to severely weathered calcareous, argillaceous sandstone and conglomerate.

Basalt — Volcanic intrusion of dark hard slightly porphyritic basalt with sparse white feldspar phenocrysts.

Legend:



Notes: The stratification lines are based on interpolations between widely spaced borings and thus represent the approximate boundaries between soil types. Actual transitions may vary from those shown. Water level readings have been made in the drill holes. Fluctuations in the level of groundwater may occur due to rainfall, temperature and other features.

FIGURE 3. Subsurface section C-C′.

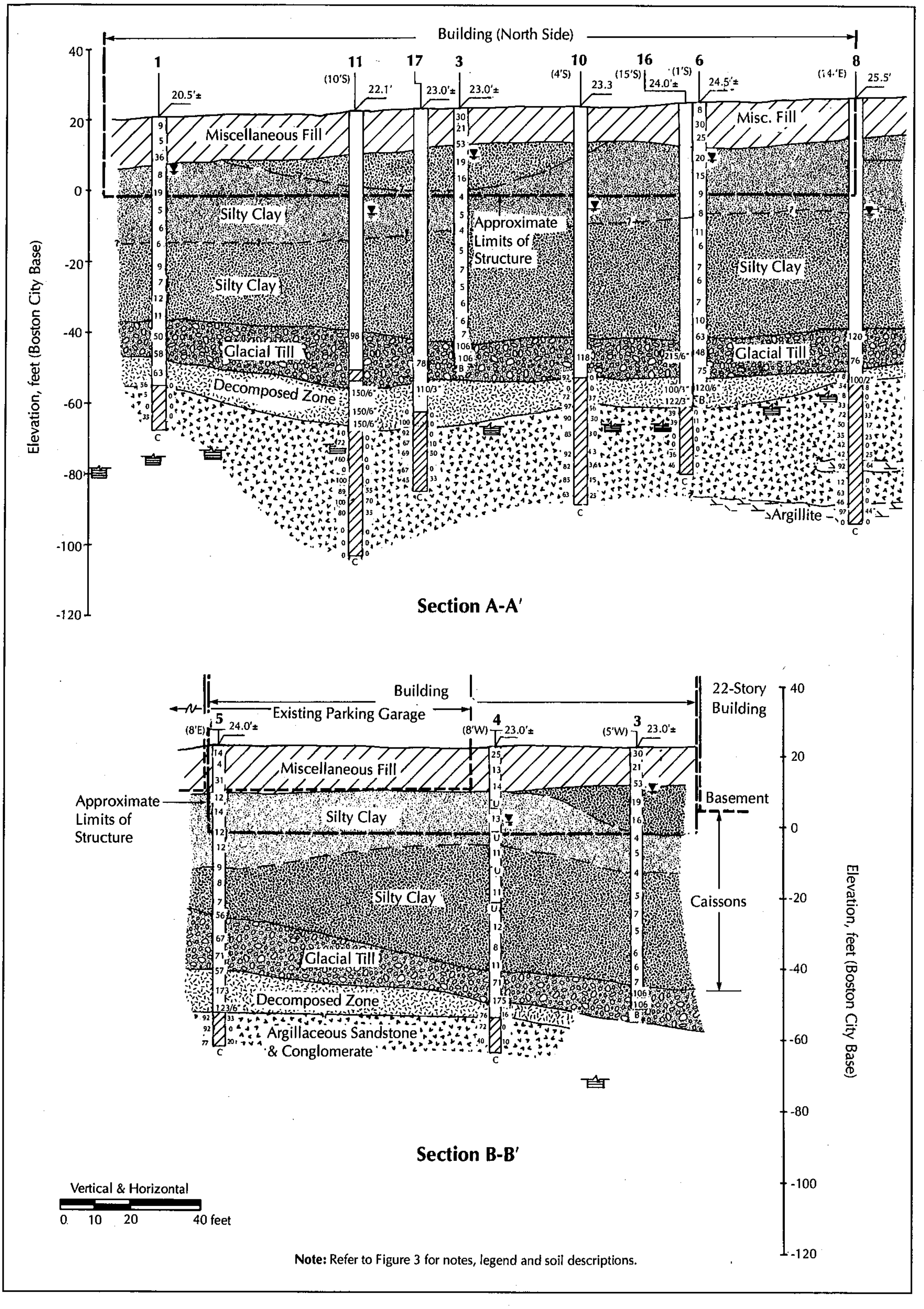


FIGURE 4. Subsurface sections A-A′ and B-B′.

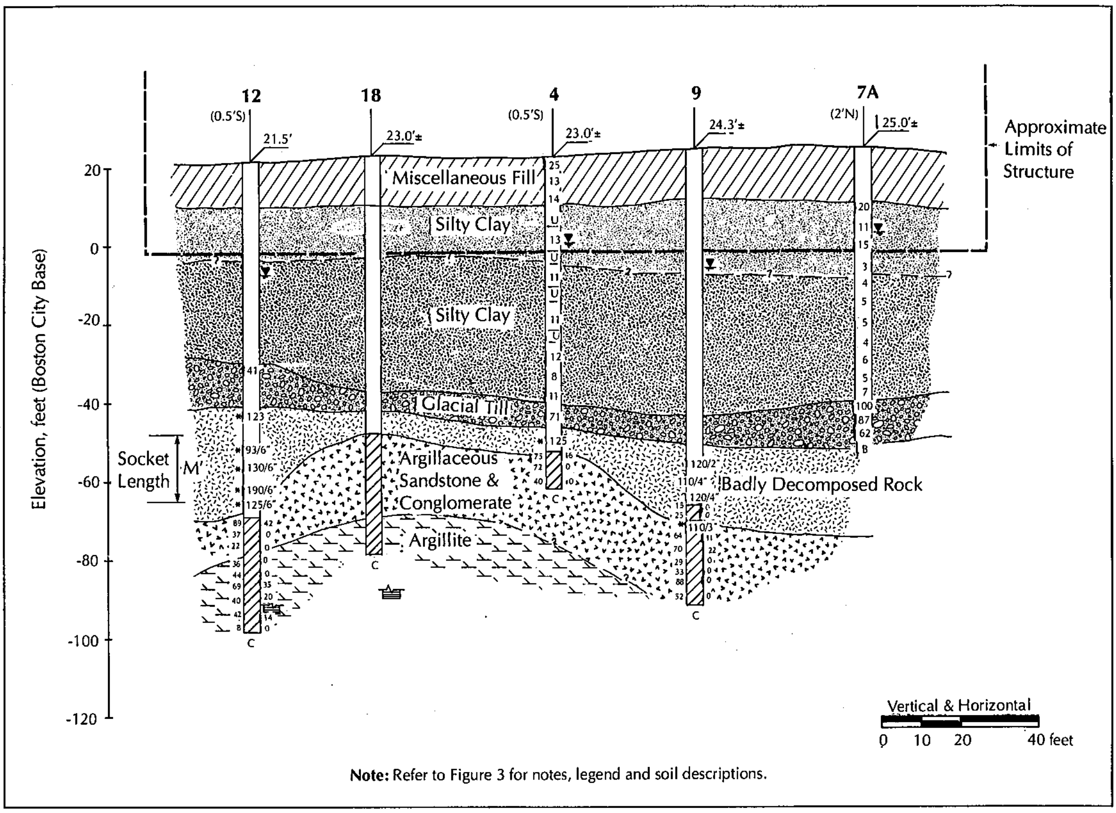


FIGURE 5. Subsurface section D-D′.

Goodman Jack Tests. Borehole deformability tests using the Goodman Jack were performed in order to measure the stress-strain properties of the bedrock in the borehole. This cylindrical jack is inserted into an NX borehole and hydraulic pressure is applied that activates jacks inside the unit that, in turn, horizontally expand metal platens against the sides of the borehole. Linear variable displacement transducer (LVDT) gages are used to record deformation in inches and a stress-strain modulus is developed from the resulting data.

Figure 7 presents a summary of the Goodman Jack modulus data versus depth below the bedrock surface. The plot depicts data from two perpendicular test orientations in the borehole. Due to irregularities in the borehole dimensions resulting from the rock's sensitivity to erosion by water and abrasion during coring, only a limited number of the tests performed were successful.

The limited Goodman Jack data indicate that the modulus values generally increase with depth below the bedrock surface. The modulus values range between 200,000 and 800,000 pounds per square inch (psi). In view of the difficulty in preparing the borehole for the test, some of the reported values may be unreliably low due to borehole disturbance and thus are considered to be a lower bound for stress-strain modulus in the areas of testing.

Seismic Cross-Hole Velocity Tests. Seismic cross-hole velocity measurements were performed below the bedrock surface in four boreholes with a seismograph coupled to multiple three-component seismometer packages. Seismograms were obtained of the seismic waves that were propagated by the detonation of blasting caps in a borehole and then measuring the arrival time with geophones in separate boreholes.

Several pertinent conclusions derived from the seismic study were:

- Cross-hole measurements at the site indicated that the sandstone and con-

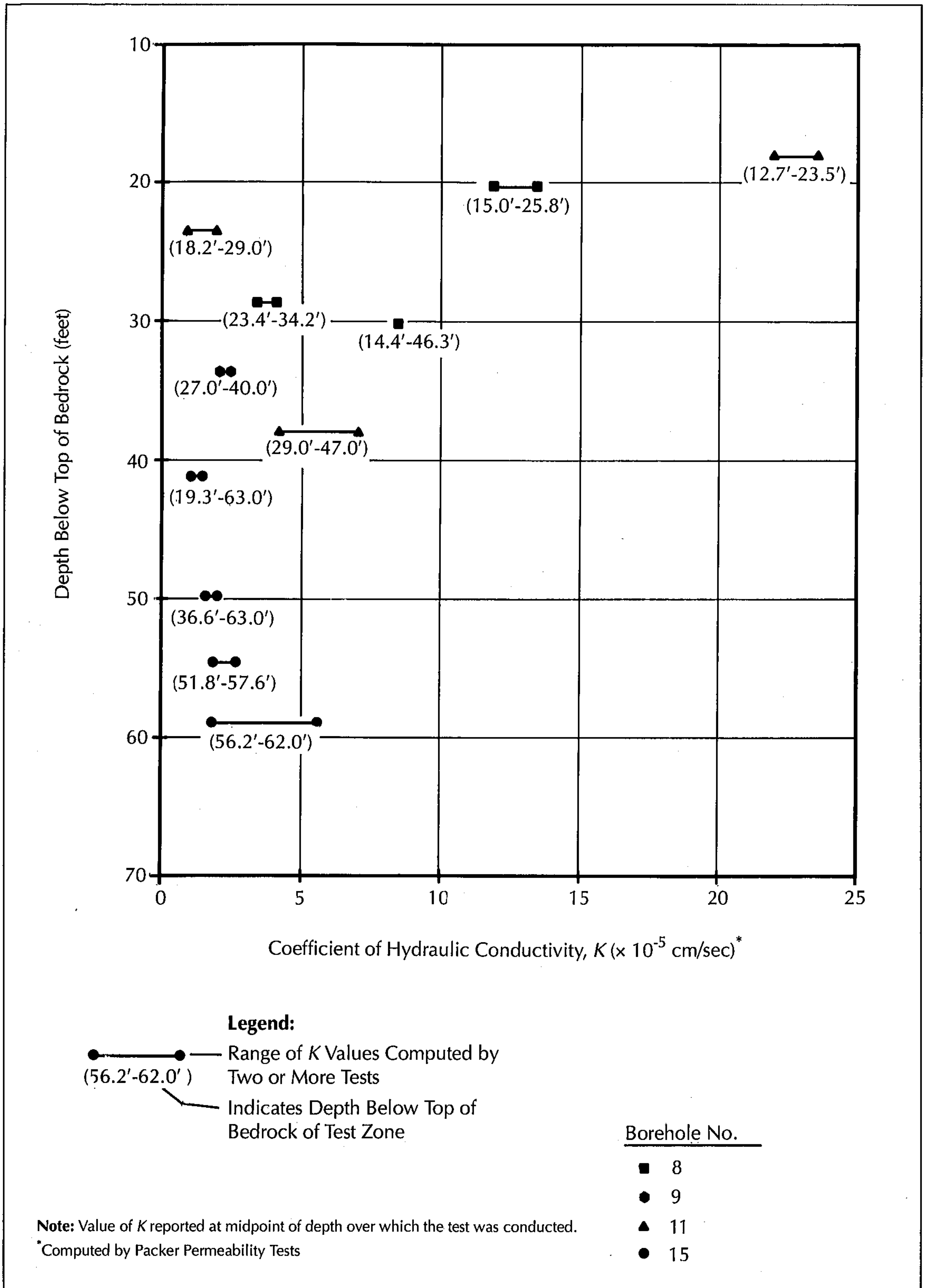


FIGURE 6. Summary of packer permeability test results.

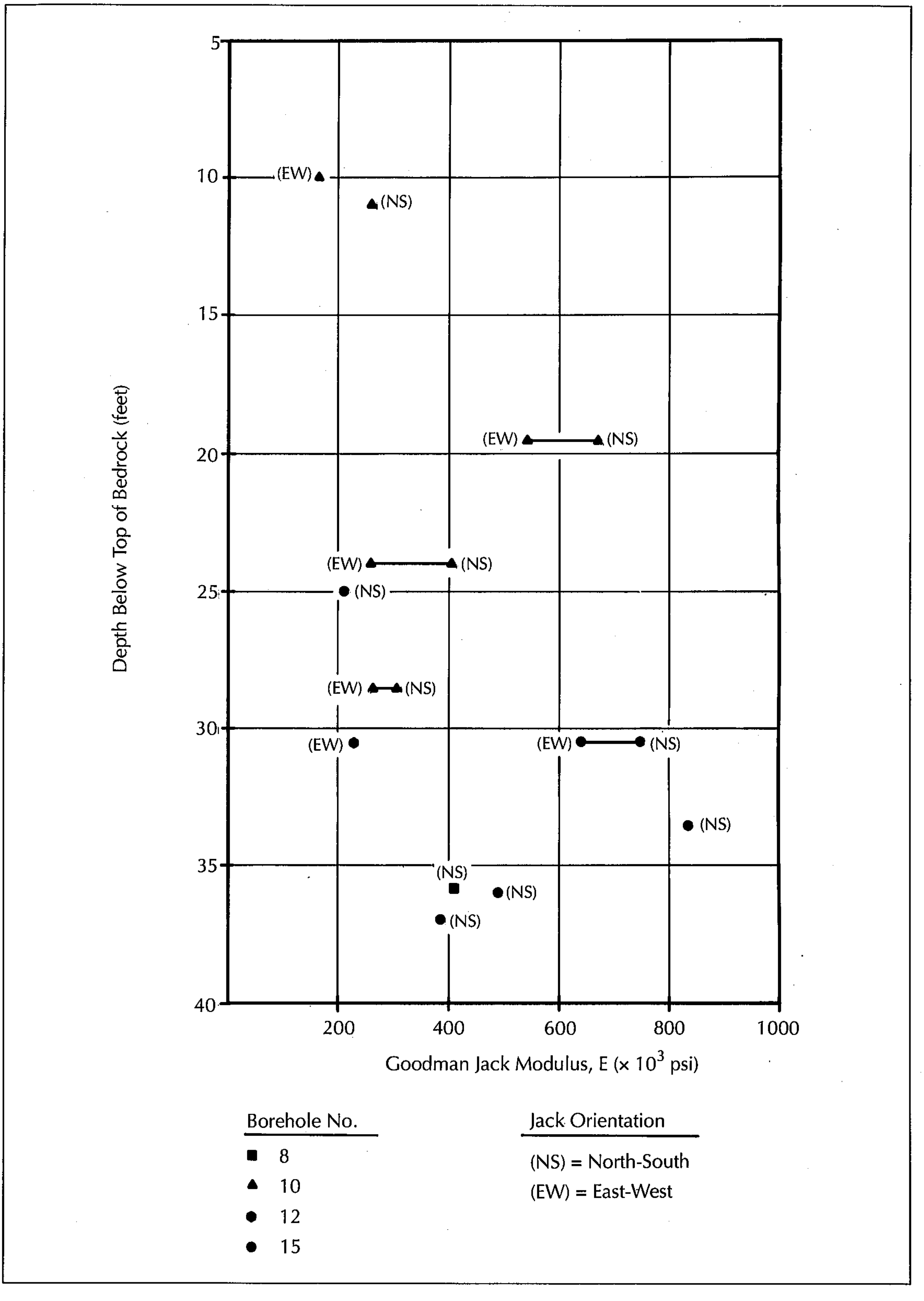


FIGURE 7. Modulus data summary.

glomerate rock zone in the northern portion of the site exhibits higher velocities than the argillite rock located toward the south end of the site and below the sandstone and conglomerate. The higher velocities correspond to higher stress-strain moduli in the sandstone and conglomerate. The measurements corroborate the results of geologic reconnaissance and the other in-situ testing methods of investigation.

- Computed modulus values typically ranged between 900,000 and 1,500,000 psi for the bedrock mass.
- Moduli determined by seismic methods exceed Goodman Jack moduli by a factor of two or more. However, Goodman Jack data, as noted above, were expected to yield lower than actual values for the bedrock mass.

Laboratory Test Results

Eight unconfined compression tests were performed on samples of intact rock specimens that were selected from the rock cores. Sample unconfined compressive strengths varied from 500 to 4,300 psi. The lower strength values occurred at the east end of the site where rock classification, geophysical measurements, and geological reconnaissance all indicated the presence of a very soft zone of argillite that was considerably weaker than the sandstone/conglomerate.

Two compression tests of samples selected from very soft kaolinized seams in the argillite from the east end of the site exhibited strengths of 500 and 600 psi. Conversely, the compressive strengths in the sandstone and conglomerate bed ranged from a low of 1,600 to a high of 4,300 psi. The highest measured strength of 4,300 psi was from an argillite sample recovered from an isolated zone within the sandstone and conglomerate bed at the west end of the site.

Values of the stress-strain modulus for the samples from the sandstone and conglomerate bed ranged approximately from 500,000 to 700,000 psi. The stress-strain moduli from the tests performed on the weak samples of kaolinized argillite were on the order of 100,000 to 150,000 psi.

Load Test

The purpose of the load test was to transmit a load to a test rock socket in order to confirm the provisional design friction and end bearing values.

A test caisson was installed in September 1985 at the east end of the site in an area that was considered representative of the weaker bedrock to be used for foundation support at the site. The purpose of this test caisson was to observe the performance of the "worst case" behavior of foundation loading in the soft, severely-weathered argillite.

The test caisson location is shown in Figure 2 and the test caisson profile is depicted in Figure 8. The test caisson was 24 inches in diameter, with a 90.5-foot-long shaft and a 36-inch-long rock socket. The upper 87.5 feet of the shaft were isolated from friction in the overburden by a 24-inch steel shell in a 27-inch hole filled with bentonite slurry. Reinforcing bars instrumented with strain gages were placed in the socket in an attempt to measure the distribution of the load to the socket. Unfortunately, the strain gages did not provide the desired load distribution data.

The caisson load tests were completed on October 2, 1985. The test was taken to a maximum load of 460 tons in an effort to reach twice the maximum provisional values of socket friction and end bearing of 100 psi and 30 tons per square foot (tsf) judged to be allowable for the more competent sandstone and conglomerate found north of the test caisson. It was understood that, on this basis, a test load of 460 tons would likely cause a failure in terms of allowable settlement of the test unit.

Load test results summarized in Figure 9 indicated that in terms of acceptable deformation criteria as set forth in the Massachusetts State Building Code (0.5 inch total settlement after half the allowable load has been removed), 100 psi and 30 tsf design parameters were not acceptable for the design in the soft, severely-weathered argillite.[2]

Following the analysis of the load test data, revised rock socket caisson design parameters were developed for the weaker argillite zone. An allowable stress of 65 psi in socket friction and 25 tsf in end bearing (equivalent to a 155-

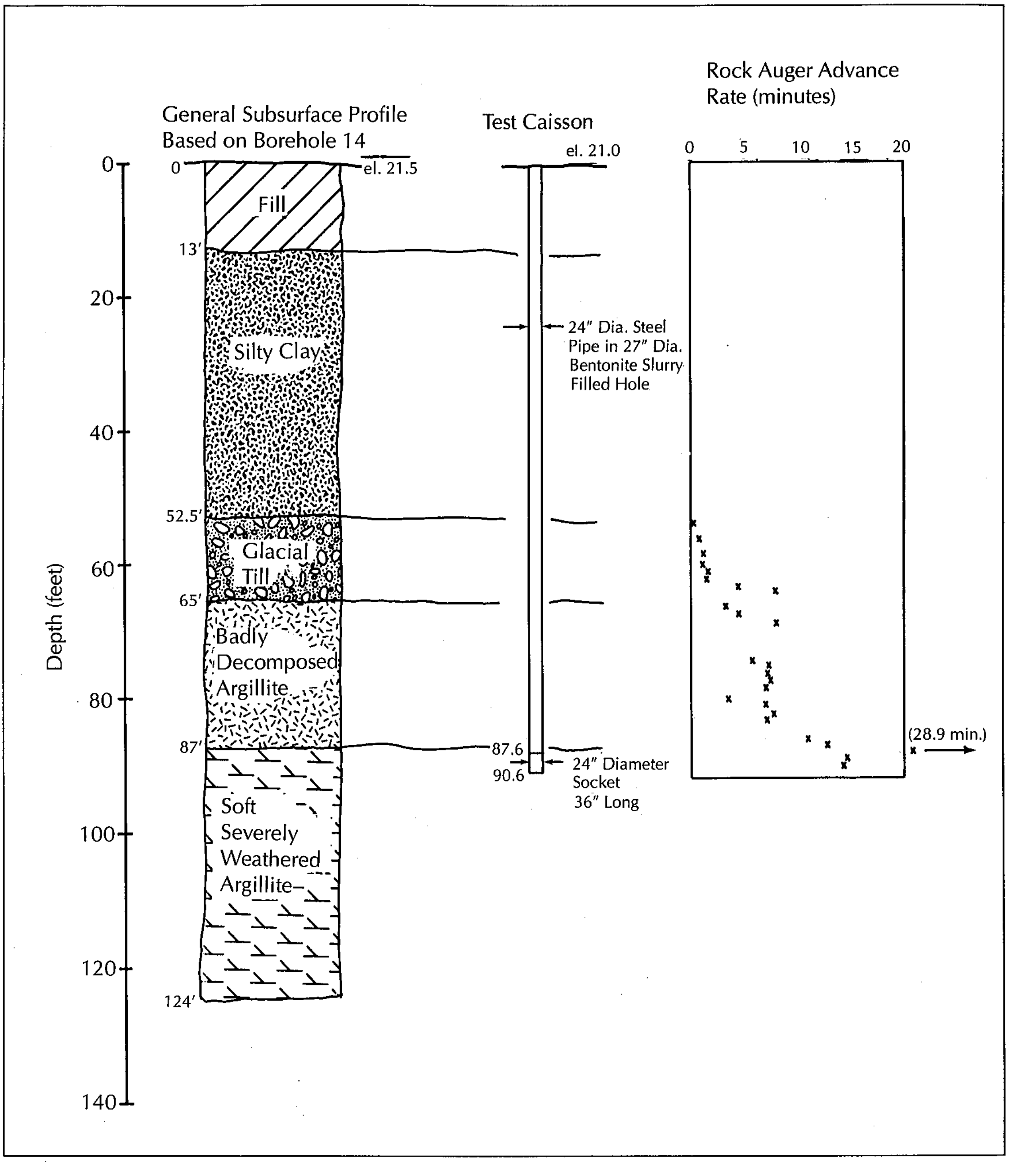


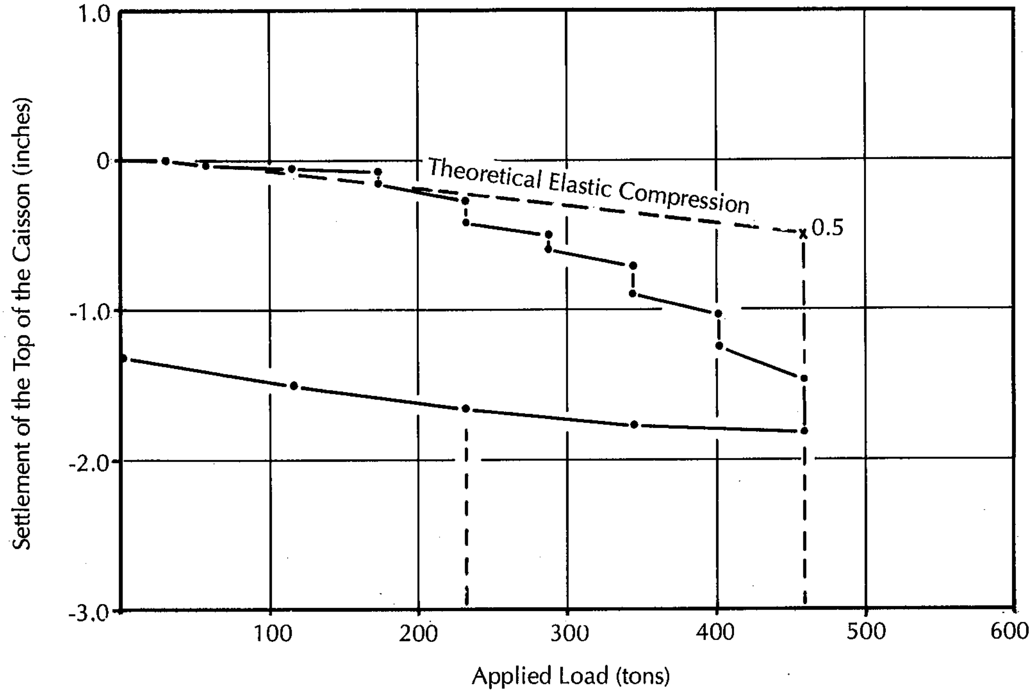
FIGURE 8. Test caisson profile.

ton design load on the test caisson) were adopted for the design in the soft, severely-weathered argillite and 100 psi and 30 tsf (equivalent to a 220-ton design load on the test caisson) were maintained for the more competent sandstone and conglomerate. Frictional capacity above the top of the socket was neglected for the design.

The use of these values assumed that all production caissons would penetrate the badly decomposed zone of argillite overlying the bearing stratum.

Rock Socket Design

Load transfer in a rock socket is a function of the ratio of the shaft material modulus to the



Applied Load (tons)	Settlement (inches)	Load Duration (hours)
57.5	0.04	2.0
115.0	0.10	4.0
172.5	0.16	2.0
230.0	0.40	24.0
287.5	0.58	2.0
345.0	0.88	4.0
402.5	1.23	2.0
460.0	1.80	24.0
345.0	1.73	2.0
230.0	1.62	3.0
115.0	1.50	2.0
0.0	1.30	11.5

FIGURE 9. Caisson load test — load versus settlement.

modulus of the bearing stratum in the vicinity of the socket. Theoretical studies have been carried out which show the dissipation of load along the socket as a function of geometry and ratio of the moduli.[3,4,5] Figure 10 shows this relation for the simple case of a uniform bearing medium.

Case histories in the literature suggest relationships for friction versus the bond strength of rock sockets. Figure 11 presents a correlation between the measured bond and unconfined compressive strength for six cases of varying rock types and strengths.

Rock socket design parameters for allowable friction and end bearing were developed on the basis of the field and laboratory data recovered from the site, a review of case histories[6,7] and theoretical studies in the literature,[3,4,5] and on

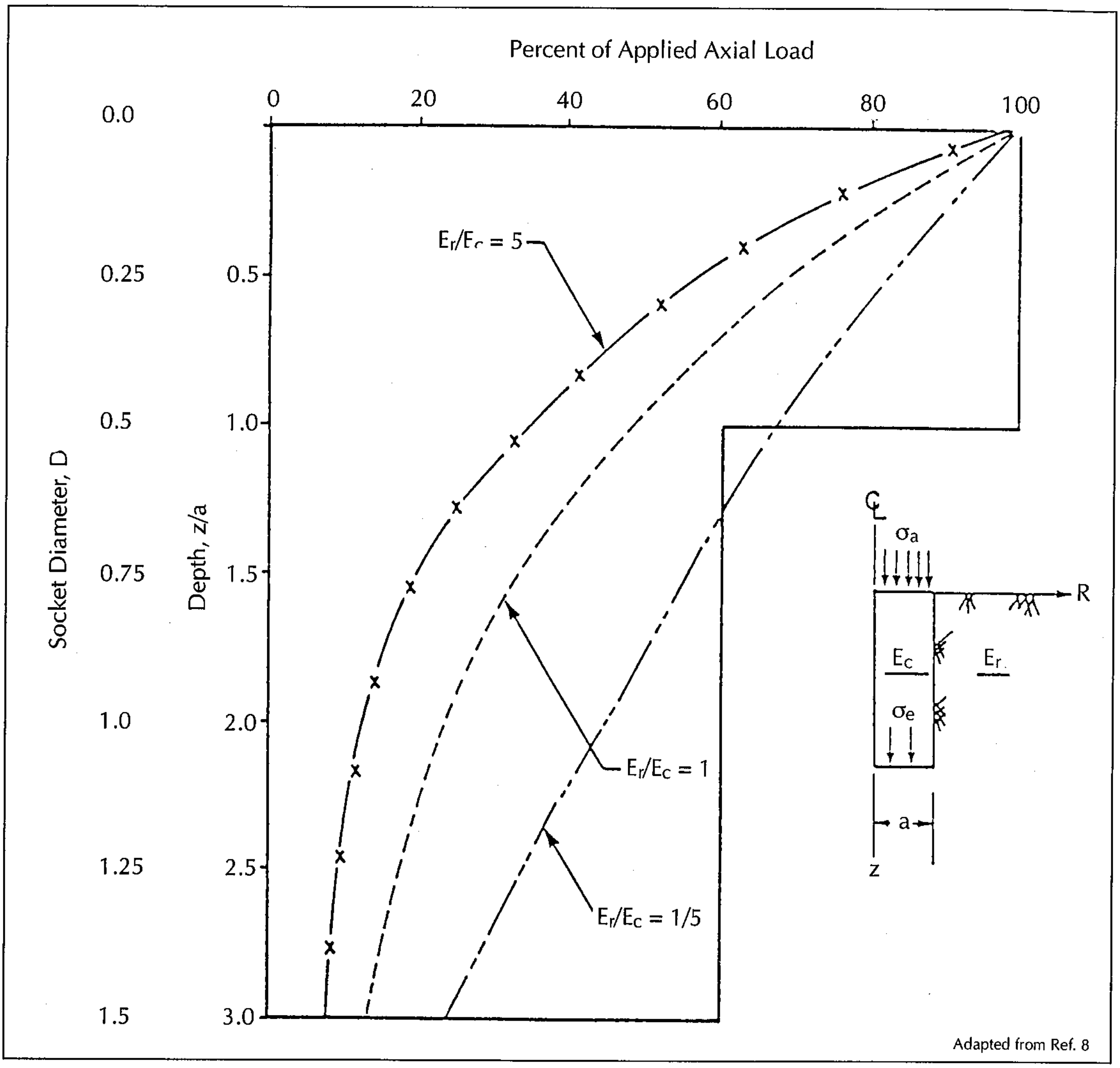


FIGURE 10. Load transfer curves for varying moduli ratios.

the results of the load test. Dominant factors considered in the design were rock strength and the roughness of the socket surface.

The caissons were designed by the foundation contractor using the engineering parameters determined from the series of tests performed at the site and in the laboratory. Caisson diameters varied from 48 to 87 inches and sockets varied from 5 to 26 feet in depth. Adopted design values for friction in pounds per square inch and end bearing in tons per square foot are shown in Table 1.

Peer Review

While developing the final design, peer review by a third-party engineer that had expertise in drilled shaft/rock socket design and construc-

Table 1
Design Values for Friction and End Bearing

Rock Type	Friction (psi)	End Bearing (tsf)
Sandstone & Conglomerate	100	30
Argillite	65	25
Transition Zones	65	25

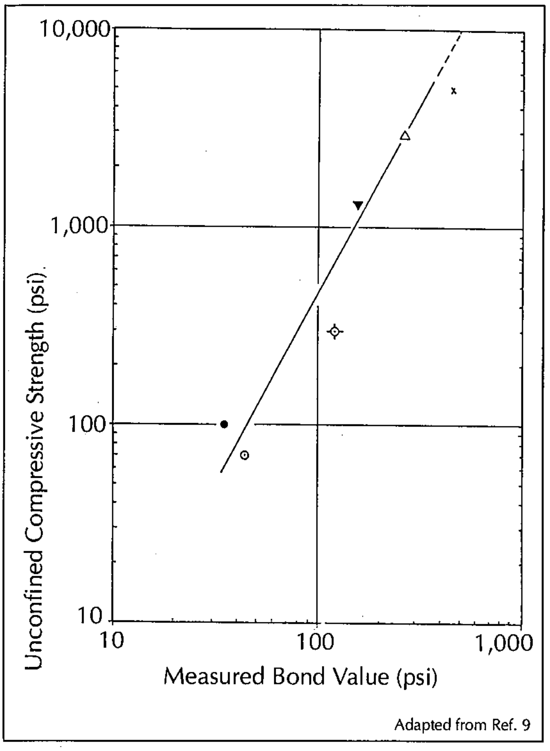


FIGURE 11. Bond versus unconfined compression strength.

tion was implemented. The peer review engineer met with the project team, visited the project site, reviewed reports and test data, examined the rock cores, and reviewed the load test results. The peer reviewer concluded that the design values selected for the project were reasonable.

Construction

Construction of the 101 Federal Street rock socket caisson foundations started in mid January of 1986 and finished in late March of 1986. A total of 41 straight shaft rock socket caissons were installed during the ten-week working period. Figure 12 notes the location and relative size of the caissons. The caissons varied in diameter from 48 inches to 87 inches, with rock sockets ranging from 5 feet to 26 feet in length. Twenty-one caissons were constructed in a central core area for the new structure, while the remaining 20 caissons were placed below the structure columns.

The excavation for the basements was retained by a load-bearing concrete diaphragm wall cast in a bentonite slurry trench. The wall was designed to support column loads along the south perimeter wall by transferring the loads to the glacial till stratum.

The caissons were constructed under a design/build agreement between the owner and the contractor. The owner retained the services of the project geotechnical engineer during this phase of the project to provide performance specifications, monitor construction activities, and document caisson installation. As part of this process, continual updates of subsurface information were performed that yielded a greater confidence level in the construction method.

Construction Techniques

The rock socket caissons for the 101 Federal Street structure were constructed using bentonite slurry. Slurry walls have been used extensively in the Boston area for construction of subway excavations and to a lesser extent for high-rise building foundations. However, prior to the 101 Federal Street project, the slurry technique had not been used in the Boston area for drilled caisson construction.

The caissons were excavated by both truck-mounted and track-mounted rigs equipped with soil and rock augers fixed to the end of a kelly bar. The excavation was advanced by the auger, with the cuttings being collected at the surface and removed offsite. The auger size varied, depending on the design diameter of the caisson. A tooth on the outside of the rock auger scored the socket sides and roughened the rock surface.

For each caisson, a steel casing was installed (conforming to the caisson diameter) through the upper fill deposit and approximately 5 feet into the upper clay crust to provide added stability and reduce the potential for the sloughing of the fill into the excavation.

The caissons were then excavated without slurry to the top of the glacial till deposit. At the top of the till, slurry was introduced to maintain the stability of the hole. The slurry was continuously circulated and cleaned using a desander throughout the excavation. For the caissons located along the perimeter of the existing 75 Federal Street structure, slurry was employed from the top of the caisson to the

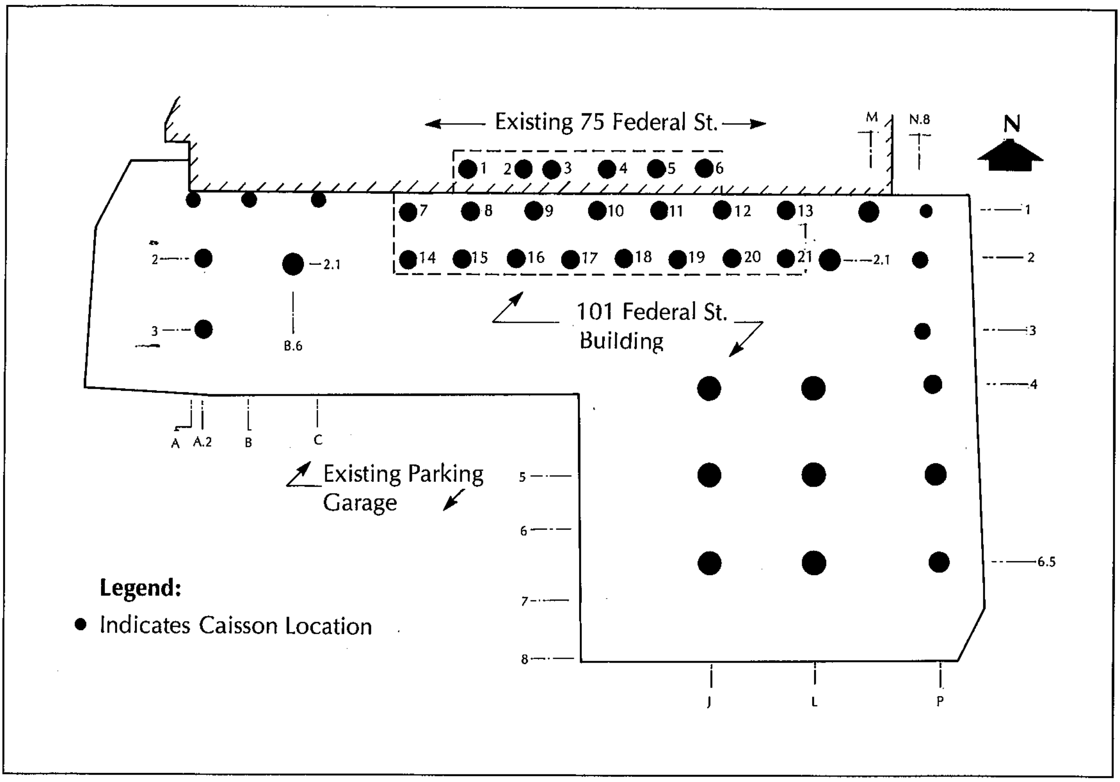


FIGURE 12. Caisson location plan.

bottom.

After the completion of the caisson excavation to the required depth, the caisson bottom was cleaned of loose material and debris using an air lift. The cleaning was performed until the air lift return indicated that a clean bottom existed.

Within two hours after completion of the caisson excavation and the desanding, the placement of concrete within the shaft was started. The concrete was placed from the bottom of the socket up using a tremie pipe. A minimum of 8 feet of concrete was maintained above the bottom of the tremie pipe to prevent the reentry of slurry into the tremie pipe.

Construction Monitoring

Throughout the caisson installation process, a detailed monitoring program was followed to ensure that the rock socket was of adequate depth; that the socket bottom was dense, sound, and properly cleaned; and that the integrity of the caisson shaft was maintained during and after construction.

During the excavation of the caissons, the geotechnical engineer was present at all times to observe and log the operation. Samples of the auger cuttings from the glacial till and various zones of the underlying rock were collected and compared to boring logs and rock cores. To aid in the interpretation, the rock core samples were kept at the site during construction for easy observation. The rate of penetration of the caisson auger was also used to indicate when a change in strata was encountered.

Observations gathered from the monitoring of the construction were used so that the rock socket was either maintained at the design depth or increased. In most cases, no changes in the rock socket depth were necessary. However, where the top of the weathered rock was found to be deeper than expected, or where weak or loose zones were encountered, the socket depth was increased. Superimposed on Figures 3, 4 and 5 are the final bottom elevations at selected caisson locations.

The bottom of each caisson was periodically probed using a weighted tape during excavation. After completion of the excavation, the bottom was cleaned by placing the slurry desander pipe at the base of the caisson socket and circulating the slurry for a period of two to four hours. The slurry discharge was visually monitored and checked for contamination. After completion of the desanding, the bottom of the socket was probed using a "neutral buoyancy" rod specially constructed for the project out of 10-foot sections of aircraft-grade aluminum tubing with O-ring joints and a weighted bottom. The rod was used to detect debris which had not been removed by the desander and the air lift.

Throughout the caisson excavation process, the quality of the slurry was monitored. Tests were performed daily on the density and viscosity of the slurry in order to ensure that the proper slurry quality was attained and that the stability of the open shafts would be maintained.

During the placement of the caisson concrete with the tremie pipe, the volume of concrete introduced into the excavation was measured. This volume was compared to the theoretical volume of the excavation in order to identify possible areas where the caisson shaft had become larger or narrower than believed. Good agreement was observed at each caisson throughout this process.

Construction Engineering

During the construction of the caissons, additional subsurface information was obtained in order to aid in the construction of the six heavily-loaded caissons in the southern wing of the proposed building. These caissons were over 7 feet in diameter and carried as much as 6,200 kips. Because of the critical nature of these foundations, a boring was completed at each of the six locations. These borings provided specific data to help establish the caisson socket depth and bottom elevation.

As part of the monitoring effort, a series of subsurface profiles were developed based on the boring and caisson installation data. Upon the completion of each caisson excavation, the profiles were updated by plotting the field monitoring data. These data were used to evaluate the bottom elevations of subsequent caisson elevations.

Summary

This case study documents the successful use of rock-socketed caissons installed with bentonite slurry. In order to properly implement this foundation technique, a thorough study of the bedrock at the site was performed. The various exploration techniques employed to determine the rock condition proved to be of tremendous value in allowing the design team to select reasonable engineering properties for the caisson design while maintaining a high degree of confidence.

The caisson installation technique was monitored closely throughout construction and was found to meet the design criteria with requiring only minor adjustments. The schedule and cost benefits accrued by using this technique also added greatly to the overall success of the project.

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