Design & Construction of Deep Stone Columns in Marine Clay at Spectacle Island

Stone columns can be an effective ground modification technique. However, their implementation depends to a high degree on specific site characteristics.

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Spectacle Island is located in Boston Harbor, north of Long Island and southwest of Deer Island. It originally consisted of two drumlins connected by a sandbar along the west side of the island. A plan of the island as it existed in 1857 is shown in Figure 1. From the seventeenth century until 1921, the island served a variety of uses, including farming and logging, and it was also the site of hotels, a quarantine hospital and a horse-rendering plant. In 1921, the city of Boston began bringing municipal refuse and various other debris to the island for incineration and disposal. Although the incinerator ceased operation in 1930, materials continued to be dumped on the island until 1960. The island has been unused since 1960. The outline of the island as it existed in 1989 is shown in Figure 1, where the extent to which the island was enlarged by offshore dumping of the refuse can be seen. The thickness of the refuse on the island varies up to about 100 feet near the center of the island. The thickness of the refuse is significantly less to the north and south. On the east side of the island, the refuse has been eroded by wave action, creating a bluff as much as 60 feet in height above the harbor. In addition, leachate from the refuse has flowed into the harbor.

Construction of the Central Artery/Tunnel (CA/T) Project is expected to generate 14 million cubic yards of excavated soils, much of which cannot be reused in the tunnel construction. According to current plans, approximately 3.6 million cubic yards of this soil (referred to as CA/T fill) will be placed at Spectacle Island. After this soil is placed over the existing refuse at the island, a landfill cap will be constructed over all of the fill areas to reduce infiltration and leachate generation. The island will then become a park with a public

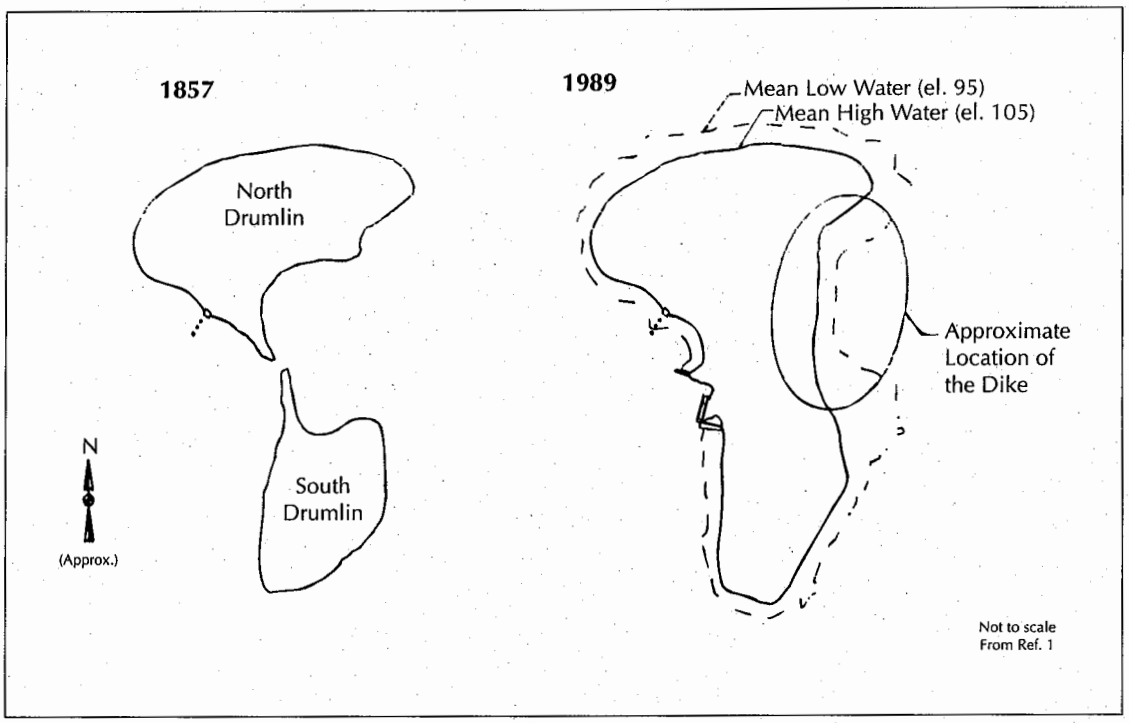

FIGURE 1. Spectacle Island plan.

marina for use as a public recreation facility. During the course of design (which followed a subsurface exploration and laboratory testing program), it was discovered that the underlying marine clay soils could not support the proposed landfill containment dike on the east side of the island. After an analysis of several schemes to improve slope stability, it was decided to install 65-foot-long stone columns using the dry bottom-feed technique.

Description of the Containment Dike

A major construction item in the Spectacle Island disposal facility is a dike that has been constructed just off the eastern shore of a portion of the island (see Figures 1 and 2). The purposes of the dike are to:

- Establish a new shoreline for the east side of the island so that fill can be placed in front of and over the refuse bluff (in order to cover it without excavating any existing refuse);
- Control future erosion of the east side of the island; and,
- Contain and collect leachate flowing from the refuse.

The dike is about 1,700 feet long with a 20-foot-wide crest at elevation 128.5 feet (based on Project Datum, which is 100 feet below the 1929 National Geodetic Vertical Datum). The lowest preconstruction elevation of the harbor bottom under the proposed dike was about elevation 92± before excavation for dike construction. The outside slope of the dike is protected against erosion by rip-rap shore protection consisting of 5-ton stones underlain by graded filters.

A plan showing the cofferdam and dike alignments is shown in Figure 2. A cross section of the proposed dike is shown in Figure 3. The outside slope is 2:1 (H:V), and the backslope is 1.5:1. Proposed grades upslope from the dike are about 3:1. The maximum elevation of the island will be about elevation 270 on the north drumlin.

The dike was built in the dry by constructing a steel sheet pile cofferdam and dewatering the construction area. About 20 feet of existing soft sediments and refuse in the dike area were removed, and the dike was constructed of compacted till fill obtained from the south drumlin

of the island. Compacted till fill was placed behind the dike up to elevation 105, which is the mean high water (MHW) elevation. Placement of CA/T fill below MHW was not allowed under the project permits.

The project criteria required the static factor of safety to be at least 1.3 for both short- and long-term conditions. Stability under seismic loading conditions was evaluated by estimating earthquake-induced deformations.

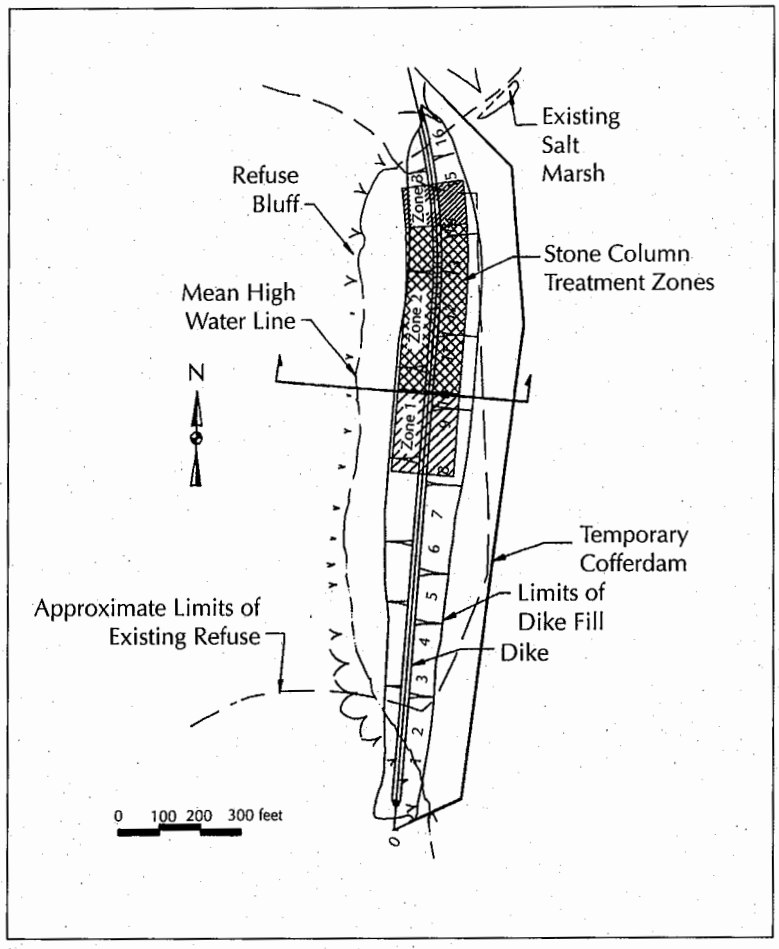

FIGURE 2. Dike plan showing the treatment areas.

Subsurface Conditions

The natural soils in the north and south drumlins consist of dense glacial till. Before work on the island began, the top of the till was about elevation 170 on the north drumlin and elevation 160 on the south drumlin.

Offshore and in the saddle between the drumlins is a deposit of marine clay (commonly referred to as Boston Blue Clay). The portion of the deposit between the drumlins is referred to herein as the clay valley. This clay is very stiff in the upper 6 to 12 feet, becoming medium stiff to stiff with depth. The upper part of the clay deposit is highly overconsolidated due to desiccation during a period when it was exposed above sea level. Generally, the clay is overconsolidated throughout its entire depth, except beneath the thicker zones of refuse on the island, where the lower portion of clay in the deepest part of the clay valley appears to be normally consolidated or even slightly underconsolidated.

The top of the marine clay is generally at about elevation 85 to 95 on the island, but it slopes downward to about elevation 75 in the area of the cofferdam. Under the dike, the top of the clay is at approximately elevation 78 to 80, and the bottom of the clay is at about elevation 10 in the deepest portion of the clay valley (near dike station 10+00). The clay pinches out toward the ends of the dike. The clay generally has medium to high plasticity and is stratified with lenses and layers of silt and fine sand. The clay also contains occasional coarse material — which ranges from coarse sand to coarse gravel — distributed randomly throughout the deposit.

For this project, the clay stratum was subdivided into two zones: the highly desiccated upper zone of very stiff to hard clay, C_1 (standard penetration test [SPT] N-values greater than or equal to 15 blows/foot) and a lower zone of medium stiff to stiff clay, C_2 (SPT N-values less than 15 blows/foot).

Ocean sediments consisting of soft organic silt and sandy silt are present over the till and

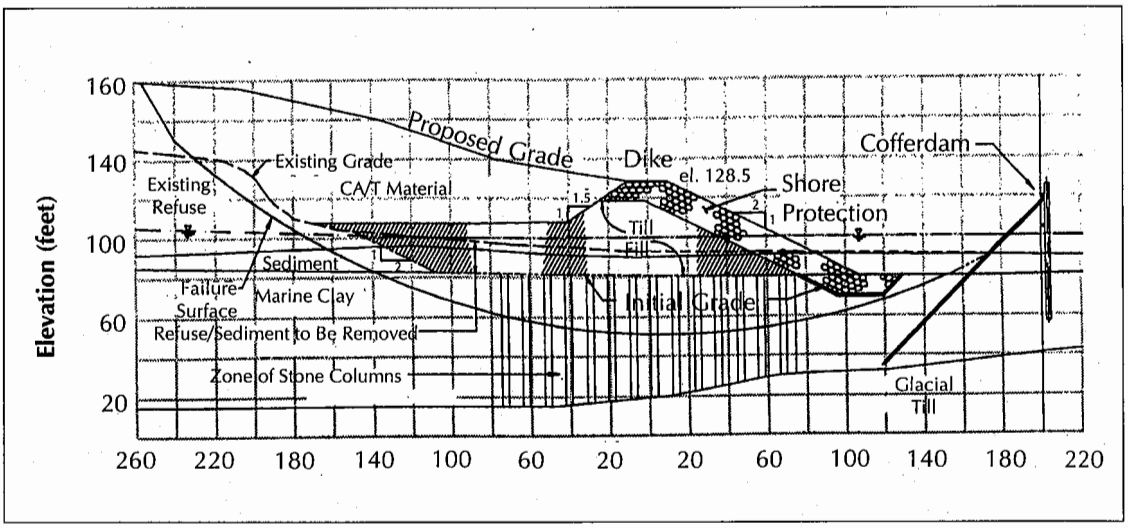

FIGURE 3. Dike cross section.

clay offshore and over the clay between the drumlins on the island.

The refuse, located in the center and south portions of the north drumlin and over the clay valley between the drumlins, consists of a variable mixture of municipal refuse, sand and gravel, cinders from the burned refuse, wood and other debris. The maximum thickness is estimated to be 100 feet. In much of the central part of the island, the bottom of the refuse is at about elevation 90.

For the slope stability analyses, the existing soils of most concern were the marine clay and the refuse. The ocean sediments were removed from areas where they would have significantly affected the stability, and the shear strength of the till was so high that it did not affect the most critical potential failure surfaces.

The SPT blow counts (N-values) in the clay ranged from zero (weight of rods) to 78 blows/foot, with most values between 4 and 20 blows/foot. Stress history profiles from consolidation test data for the offshore clay are plotted in Figure 4. These profiles indicate that the offshore clay is heavily overconsolidated at higher elevations. The average overconsolidation ratio (OCR) in the vicinity of the dike decreases with depth from greater than 20 to about 2 near the bottom of the deposit.

Undrained shear strength estimates (with no plasticity correction) from 40 field vane shear tests ranged from 0.8 to 6.4 kips per square foot (ksf). Most values ranged between 1.0 and 2.0 ksf. Testing for remolded (residual) strength of the clay was undertaken except when the peak shear strength exceeded the capacity of the vane shear device. Sensitivity estimates ranged from about 2 to 13.

Undrained shear strength values for the offshore clay that were measured in 62 unconsolidated undrained triaxial compression (UUC) tests ranged from about 0.5 to about 3.4 ksf, with most values above 1.2 ksf. Eleven isotropically consolidated undrained triaxial compression tests with pore pressure measurements (CIUC') were also run on samples of the offshore clay. Undrained shear strengths measured in these tests ranged from 0.8 to 1.9 ksf.

For comparison of the various strength data for the offshore clay, a plot of undrained shear strength versus elevation (see Figure 4) was made using the SHANSEP approach.[3] Based on this approach, the peak undrained shear strength, S_u, of typical clays is related to the effective overburden stress, $\bar{\sigma}_{vo}$, and the OCR as:

$$S_u/\bar{\sigma}_{vo} = S(OCR)^m$$

where:

S and m are parameters for a particular soil deposit

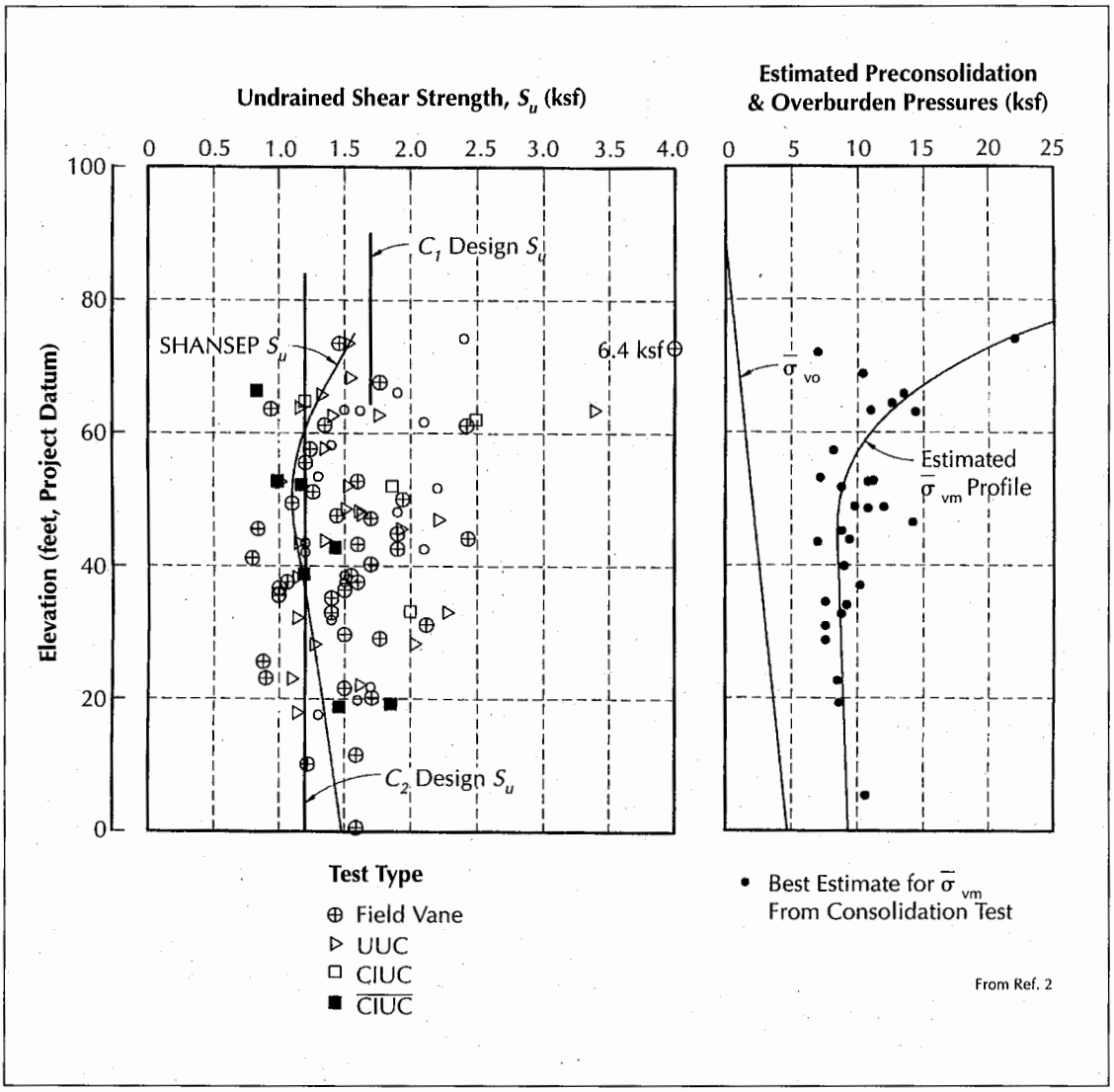

FIGURE 4. Soil property profiles.

The values of parameters S and m were estimated from 15 direct simple shear (DSS) tests performed in two earlier investigations on samples consolidated to OCR values ranging from 1 to 8. The DSS test results indicated values of S equal to 0.2 and m equal to 0.7. These values for S and m fall in the range of those estimated for Boston Blue Clay elsewhere on the CA/T Project.

For the onshore clay, 12 UUC tests were conducted on samples of the C2 clay. Measured undrained shear strengths ranged from 0.7 to 2.4 ksf.

Properties for the existing refuse were estimated from available literature on the subject.[4] Based on values reported in the literature and descriptions of the materials encountered in the borings and test pits, it was estimated that a peak angle of internal friction of 30 degrees could be mobilized in the refuse at an axial strain of about 20 percent. At lower shear strains, less shear strength would be mobilized. Because the clay is expected to achieve its peak shear strength at an axial strain of 5 to 10 percent, a strain-compatible shear strength was used for the refuse (friction angle of 16 degrees) in the analyses where potential failure surfaces would pass through both the clay and the refuse. To check against the possibility that the

TABLE 1.
Soil Properties

Soil	Total Unit Weight (pcf)	Undrained Shear Strength (psf)	Effective Friction Angle (degrees)
In-situ Glacial Till	135	—	40
Stiff Clay (C_2)	117	1,200	—
Very Stiff Clay (C_1)	117	1,600	—
Clay Under Refuse	117	1,400	—
Sediments	105	120	—
Compacted Glacial Till	135	—	35
Refuse	70	—	16*
Compacted CA/T Fill	115	1,500	—

Note: * Estimated strain-compatible value for potential failure surfaces that included both refuse and clay.
From Ref. 2

strength or stiffness of the refuse was lower than expected, analyses were also run using zero strength for the refuse to show that a factor of safety of at least 1.0 was obtained.

The undrained shear strength of the CA/T fill was assumed to be 1,500 psf, which was the strength needed to maintain a factor of safety of at least 1.3 for the proposed 3:1 fill slopes. Strength testing on a limited number of samples taken from source areas of the CA/T fill indicated that this strength could usually be achieved if the soil was compacted to at least 92 percent of the maximum dry density based on American Association of State Highway and Transportation Officials (AASHTO) Test Method T-180 (Modified AASHTO Compaction). The construction specifications require that the soil be compacted to 92 percent.[5] The shear strength of the compacted soil was to be confirmed by testing on compacted samples during construction. Table 1 summarizes the soil properties used in the slope stability analyses.

Design Alternatives

Slope stability analyses were performed using a computer program. The Modified Bishop Method was used to locate the critical failure surface and estimate the minimum factor of safety for the existing condition and the end of construction condition. The existing condition was evaluated to confirm the estimated soil and refuse parameters. Spencer's Method was then used to further evaluate the factor of safety of the most critical failure surface. A manual check of the most critical failure surface using the Modified Bishop Method was performed to verify the computer results.

Using the soil properties listed in Table 1, the factor of safety under static loading conditions was found to be about 1.0, which was lower than the required 1.3. The critical potential failure surfaces passed through the refuse well behind the dike and extended out into the harbor beyond the toe of the dike. They passed through the clay deep beneath the dike near the clay-till interface.

To evaluate the dike and fill under seismic loading, earthquake-induced slope movements in the dike area were estimated using a deformation analysis.[6] There was concern that large, earthquake-induced displacements could create shear planes within the clay. The reduction in shear strength that would occur along these planes, if they formed, could then result in post-earthquake instability. Using a weighted average shear modulus for the composite stratum of clay and stone columns, permanent displacements were estimated to be less than 6 inches, even under the maximum design earthquake, with a peak bedrock acceleration of 0.30 g. These displacements were judged to be too small to cause reduction in shear strength of the

clay and, therefore, the displacements are acceptable.

The following design alternatives were evaluated to improve the static stability of the dike and the fill behind the dike:

- Wick drains/staged construction;
- Counterberm/flatter foreslope;
- Flatter proposed fill slope behind the dike; and,
- Stone columns.

Improvements to the dike itself, such as internal reinforcement, were not considered and would not have been useful because the most critical surfaces did not pass through the dike.

Because of the large thickness of refuse behind the dike, the improvement of the soils beneath the refuse was not practical. Therefore, soil improvement was limited to only the foundation soils in the immediate vicinity of the dike. The strength increase that was provided by wick drains in this limited area was not sufficient to satisfactorily raise the factor of safety. A counterberm in front of the dike would have had to extend about 150 feet out from the toe of the dike to provide the necessary improvement. This approach would have required filling several more acres of the harbor than allowed in the project's permits, which allowed the filling of no more than about eight acres below MHW (elevation 105). This area was determined during the preliminary design phase to be the minimum necessary to construct the dike and fill in front of the refuse bluff without having to cut down the bluff.

Making the slopes behind the dike less steep would have reduced the storage capacity of the island significantly and would have made the project difficult to justify on economic grounds. Also, this option would have required the removal of some of the existing refuse, which was not considered acceptable.

Stone Columns

General Description. Stone columns are vertical cylinders of compacted stone that have been successfully used on numerous projects to improve the bearing capacity of soils, to reduce settlements and to increase the time rate of settlement of clays. The columns increase the shear strength of the stratum not only by introducing high-strength stone, but also by providing drainage to permit the clay to consolidate rapidly, thereby gaining shear strength. By using stone columns, the modified soil acts as a composite material that is stiffer and has a higher shear strength than the unmodified soils.

Installation Procedures. Typically, stone columns are installed to depths of up to 40 feet using the wet method, where a jet of water from the bottom of the probe (vibroflot) creates a hole for the probe and carries the cuttings to the ground surface. This method generates a large amount of muddy wastewater. For this project, the generation of this wastewater would be a serious drawback, considering the confined area of work within the cofferdam and the need to clarify the water before it was released to the harbor. However, a few recent projects have used dry, bottom-feed stone columns to depths of 60 feet into glacial clays. For the recent Blue Water Bridge Project in Port Huron, Michigan, for the Federal Highway Administration (FHWA), stone columns with an average effective diameter of 3.25 feet were installed using the dry bottom-feed method to a depth of 60 feet in clays having undrained shear strengths between 0.8 and 1 ksf.[7] Based on this recent work and consultation with several stone column contractors, it was determined that dry, bottom-feed stone columns would provide a technically feasible, cost-effective technique to provide a stable foundation for the containment dike.

The dry, bottom-feed stone columns are typically installed by vibrating a probe to the required depth without jetting and with little, if any, pre-augering. At Spectacle Island, pre-augering was expected to be needed in order to penetrate the upper stiff clay. The stone to be used in the columns is loaded pneumatically into a housing (follower tube), and the probe and follower tube advance into the soil under their own weight and the weight of the stone. When the planned depth of the bottom of the stone column is reached, the probe is raised several feet, which allows some of the stone to drop out. The probe is then raised and lowered while vibrating to compact the stone and to force it out to the sides to enlarge the stone

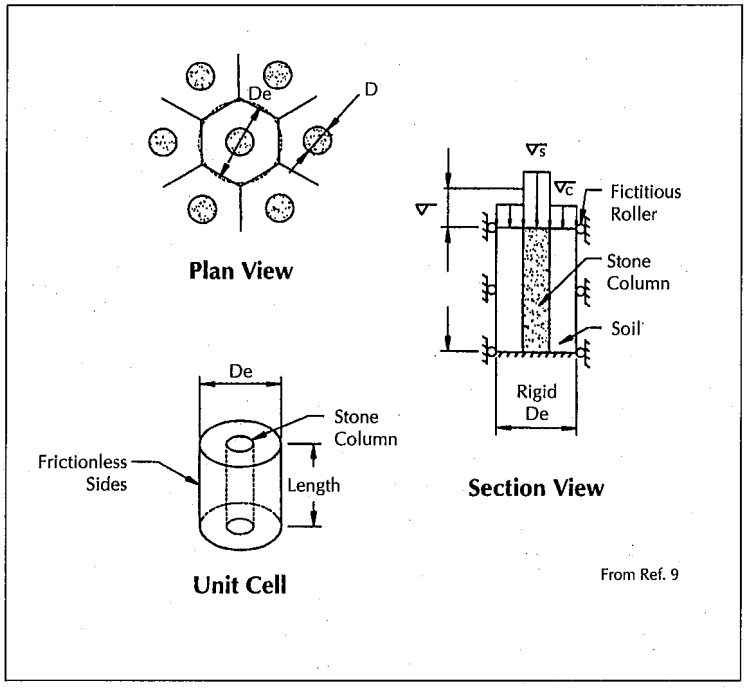

FIGURE 5. Unit cell.

column. This process is repeated as the probe is gradually withdrawn from the hole. The size of the probe and follower tube is about 22 inches in diameter. The planned diameter of the stone columns at Spectacle Island was 3.5 feet.

One of the advantages of using the bottom-feed method is that the stone is placed in the column in measured doses. This method allows for a fairly accurate estimate of the final constructed diameter for each charge of stone. The loose and compacted unit weights of the stone are determined in the laboratory prior to use on site. Since the volume of the receiver tank is known and the lift thickness can be measured by observing the probe penetrations between charges, the column diameter can be easily calculated for each charge.

This method of stone column installation can significantly disturb the clay because the stone column is created without removing any soil. If the clay is sensitive, this can seriously reduce its shear strength, which must be considered in the design of the columns.

Design Procedures. The design is based on the average strength parameters method, where the stone column treated area is modeled as a composite c-ϕ material.[8] The composite parameters used for design — γ_{comp}, ϕ_{comp} and c_{comp} — will vary with:

- The area ratio, a_s;
- The stress concentration ratio, n;
- The strength of the soil and the stone column material;
- The vertical stresses due to the weight of the soil and stone column and the load placed on the completed stone column; and,
- The expected reduction in soil strength due to disturbance during stone column installation.

Figure 5 illustrates the unit cell concept and the stress concentration ratio concept. The area ratio, a_s, is defined as the fraction of soil tributary area of the stone column replaced by the stone:

$$a_s = \frac{A_s}{A} = \frac{D^2}{De^2}$$

where :

A_s = The area of the stone column
A = The total area of the unit cell
D = The diameter of the stone column
De = The equivalent diameter of the unit cell

Figure 5 also illustrates another important concept — *stress concentration ratio.* Stress concentration occurs on the stone columns because the columns are stiffer than the surrounding clay and because the overlying embankment will arch loads from the clay onto the columns. This stress concentration effect only applies to loads applied after the stone columns are complete and not to the weight of the stone column. The stress concentration ratio, n, is defined as the ratio of stress applied to the stone column, σ_s, to the stress applied to the clay soil, σ_c:

TABLE 2.
Stone Column Locations

Zone	Location	Design Area Ratio	Column Spacing	Actual Area Ratio*
1	Sta. 8+00 to 10+00	0.12	9 ft 7 in	0.128
2	Sta. 10+00 to 14+00	0.17	8 ft 1 in	0.182
3	Sta. 14+00 to 15+00	0.10	10 ft 6 in	0.104

Note: * Calculation based on estimated actual stone column diameters.

$$n = \frac{\sigma_s}{\sigma_c}$$

The composite shear strength of a stone column-reinforced soil is:

$$a_s\left(\frac{n\Delta\bar{\sigma}_v}{1+(n-1)a_s} + \bar{\sigma}_{vo}\right)\tan\phi_s + R_f(S_u)(1-a_s)$$

where:

$\Delta\sigma_v$ = The increase in effective vertical stress due to the weight of the fill added above the completed stone columns, but not considering any stress concentration effect.

σ_{vo} = Vertical effective stress in the stone column at average depth of potential failure surface due to the weight of the stone column.

ϕ_s = Angle of internal friction of the stone column material.

S_u = Undisturbed peak undrained shear strength of the clay.

R_f = Reduction factor to account for disturbance to the clay during installation of the columns.

Final Design. For the analyses, a stress concentration ratio (n) of 3 was used, based on the Japanese method of design for sand columns.[8] That ratio was applied only to the applied external loads — *i.e.*, the weight of the overlying embankment. The weight of the stone column itself was not multiplied by n. In the design procedure in the FHWA stone column design manual, smaller values of n are recommended, but the ratio is applied to all vertical loads (internal and external).[9]

During design, it was assumed that the stone column installation would significantly disturb the marine clay, thus reducing the shear strength. To account for this, a reduction factor (R_f) was included on the undrained shear strength of the clay between the columns — assuming the strength of the clay between the columns to be 30 percent of the peak strength for the undisturbed clay (*i.e.*, R_f equal to 0.3). This reduction was selected because the average sensitivity of the clay (which was determined by field vane tests) was about 3. Although the clay will eventually regain shear strength, the rate at which the clay would recover could not be predicted, so any potential strength recovery was not included in the analysis.

Based on the above assumptions, the composite soil parameters for various area ratios were estimated. The factor of safety and the required area ratio were estimated for every 100 feet along the dike, as well as at closer intervals where required to determine the limits of the treatment zones.

The final design called for a 150-foot-wide, treated area centered on the dike baseline (extending from dike station 8+00 to station 15+00). This area straddled the clay-filled valley. Three different area ratios were used, depending on the amount of treatment required to achieve the required factor of safety. Figure 2 shows the locations of the different treatment zones. Table 2 summarizes the column spacings in those zones, assuming a column diameter of 3.5 feet and a triangular arrangement of columns.

The most efficient location for stone columns is on the active side of the failure surface. However, at this site, placing stone columns at this

point would have required penetrating over 65 feet of existing refuse, creating significant logistical and safety difficulties. In addition, it was felt that the stone columns should be uniformly distributed across the width of the dike in order to provide for more uniform settlements of the dike.

Construction at Spectacle Island

General Procedures. After the dike area was dewatered and the soft sediments removed, the contractor constructed a working platform consisting of a separation geotextile and an 18-inch layer of 0.75-inch crushed stone. The crushed stone was the same material used in the stone columns. After the completion of the stone columns, the working platform was graded at a slope downward to the east, and the stone columns were hydraulically connected to a drainage geonet placed atop the working platform.

The contract documents allowed the contractor to pre-auger to a depth of as much as 20 feet below the top of the clay in order to penetrate the very stiff upper clay layer. The stone columns were then constructed in the previously described manner for dry, bottom-feed stone columns. The contractor initially mobilized two bottom-feed rigs and two caisson rigs to install the stone columns.

The contractor encountered difficulties in penetrating the medium stiff clays below the pre-augering depths. The vibrator probe could penetrate the clay; however, the time required to reach the top of the till was much longer than anticipated, in some cases requiring 80 minutes. Various techniques were tried to improve the performance of the penetrator, including jetting water out the sides of the probe to lubricate the probe and follower tube. This method improved penetration slightly but created a large quantity of muddy slurry. Since the improvement was minor and the excess water was difficult to handle, this method of installation was discontinued after a few test columns were installed.

Eventually, it was decided to pre-auger using a 3.0-foot-diameter auger for the full depth of the stone columns and then use top-feed procedures to construct the stone columns. This method had the added advantage of permitting examination of the soil that was being excavated from the bottom of the hole, thus increasing confidence that the stone column extended completely through the clay and into the till. A 42-inch-diameter casing about 4 feet long was set at the top of each column to keep water and spoil from falling into the hole.

To speed stone column construction, an additional caisson rig was mobilized to the site (for a total of three). Also, the bottom-feed probes and follower tubes were replaced with conventional probes, which are more convenient to use. With this system, two probes could keep up with three auger rigs.

With the top-feed method, the stone was dumped into the hole using a loader with a 2.5 cubic yard bucket. The bucket capacity was verified at the start of the job. However, using this method, a significant amount of stone remains at the ground surface instead of being placed down the hole. Since this volume must be estimated, the calculation of the stone column diameter is not as accurate as for the bottom-feed method. To reduce this uncertainty, a mobile hopper was constructed so that all the stone from the bucket was directed into the hole. In comparing the mean calculated diameters, it was found that the average with the hopper was about 0.1 foot smaller than the average without the hopper, and that the standard deviation was reduced from 0.15 to 0.10 foot.

Some difficulties were encountered with the initial use of the bottom-feed system due to the weather and the supply of crushed stone. The stone specified was similar to ASTM No. 57 stone and could have no more than 5 percent fines. For acceptable performance of the stone column, a larger amount of fines was permissible, and some stone samples submitted contained as much as 10 percent fines. These fines caused two difficulties for the bottom-feed rigs. One problem was that the fines tended to accumulate in the apparatus and clog the hoses that the stones travel through, thus slowing down production and requiring frequent work stoppages to clear the blockages. Also, since the work was being performed from December to March, the moisture in the fines frequently froze, causing the machines to clog. Since the project was on a very tight schedule, it was not possible to delay stone column construction until spring. In addition, some of the 0.75-inch

stone became contaminated with larger sizes of stone, which were being used for other purposes on the site. These larger stones caused blockages in the bottom-feed equipment. With the top-feed procedures, none of these stone-related issues caused any problems, except for some occasional frozen stone.

Quality control of the columns included observing the thickness of each lift, observing that the vibrator completely penetrated each lift, calculating the average diameter of the stone columns for each lift and monitoring the amperage draw of the vibrator.[10] The lift thickness and the amperage draw are selected based on trial stone columns constructed at the beginning of the project by the contractor using the proposed equipment.

The Constructed Product. After the top-feed procedures were agreed on, three test columns were installed in the zone with the highest area ratio (Zone 2). About one week later, a soil boring was drilled in the middle of these test columns. Fixed-piston samples were obtained for laboratory testing, and CIUC' triaxial tests were performed on three of the samples from this boring.

The purpose of this testing was to verify the design assumptions regarding the amount of clay disturbance. During design, it was assumed that:

- The bottom-feed method would be used;
- Only the upper clay would be pre-augered; and,
- The clay would be heavily disturbed during installation of the stone columns.

As stated above, the estimate of the strength loss was based on the results of the vane shear testing. We assumed that the disturbance would reduce the undrained shear strength to about one third of the undisturbed shear strength. However, because the columns were pre-augered for their entire length, the clay was not disturbed enough to significantly reduce the shear strength. The CIUC' tests performed after installation of these test columns indicated the undrained shear strengths to be 1.7, 1.7 and 0.9 ksf at elevations 21, 26 and 41, respectively.

The contractor used two shifts per day to install the stone columns. A typical shift could produce between 500 and 700 feet of stone column per rig. This rate varied greatly depending on the weather, logistical constraints from working on a crowded island and the depth of the stone columns. In total, 1,621 stone columns (totaling 80,800 feet) were installed from January to March 1994.

The stone column diameter is estimated for each charge of stone by placing a known volume of stone in the hole, measuring the thickness of the charge of stone after compaction with the vibrator and then calculating the corresponding diameter. One-foot intervals are marked on the probe to aid in estimating the depth of hole before and after each charge. Profiles from three typical stone columns from Zone 1 are shown in Figure 6. The stone columns were required to have an average diameter of at least 3.5 feet. As Figure 6 shows, even though the average diameter was at least 3.5 feet, the diameters can vary significantly along the length of the column. Table 2 compares the design and actual area ratios. The actual area ratios were calculated using the average diameter of each stone column.

In areas where a stone column was not constructed to the design diameter, the local area ratio (including the small-sized stone column and all of the adjacent stone columns) was calculated. In all cases on this project where there was an undersized stone column, the local area ratios were found to be adequate. Most of the undersized stone columns were test columns that were installed before procedures had been firmly established. If a local area ratio had been found to be inadequate, the contractor would have been required to replace the defective stone column.

The dike core was constructed of compacted till fill from April to June 1994. The shore protection was completed to the top of the dike in early October 1994. Once adequate shore protection was installed up to the high tide level, the work area was flooded in mid-July 1994, and demolition of the cofferdam commenced.

To monitor the behavior of the island and dike as placement of fill on the island proceeded, geotechnical instrumentation was installed. Pore water pressures in the marine clay were measured using vibrating wire piezometers (VWPZs), and horizontal and vertical dis-

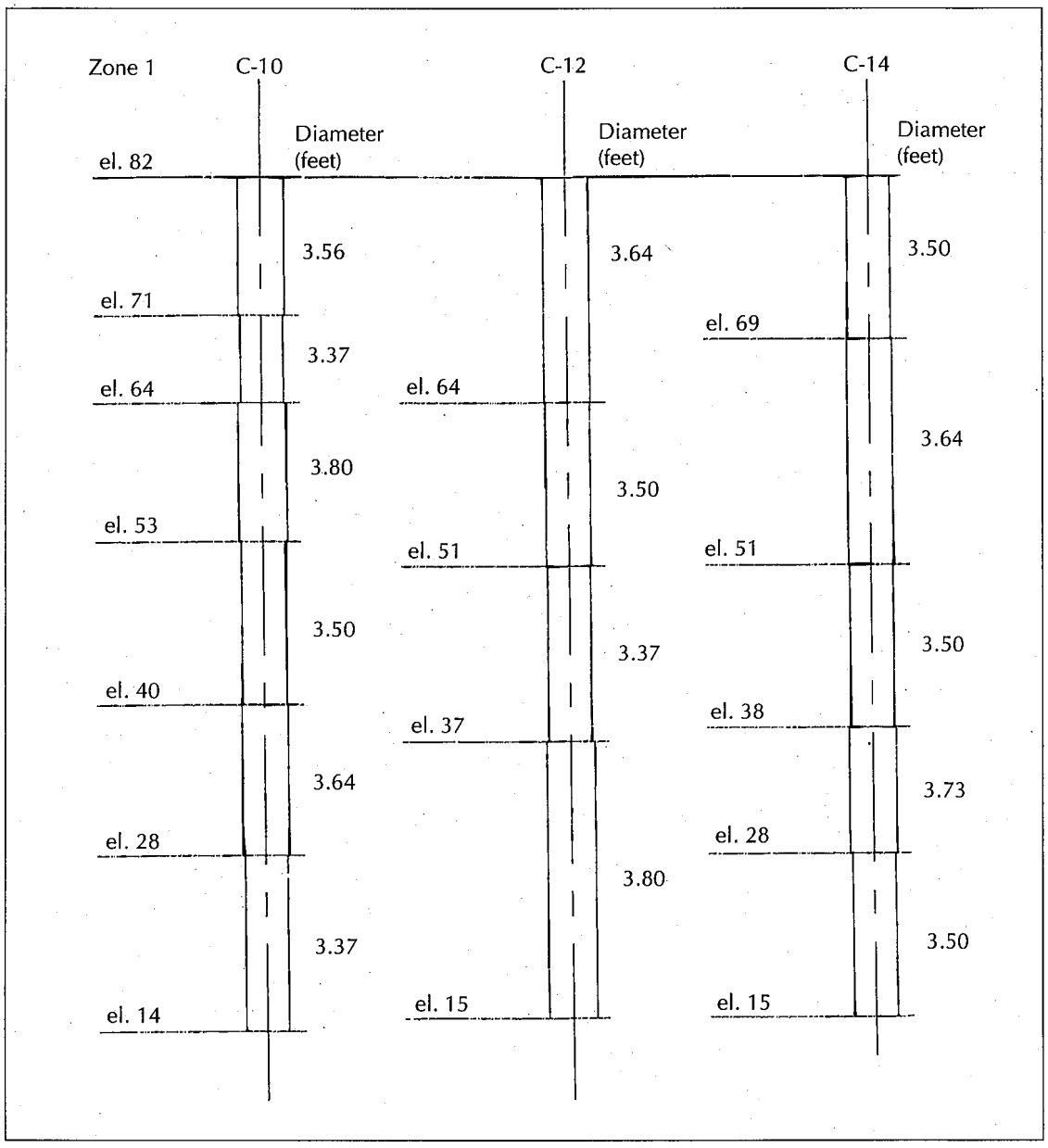

FIGURE 6. Cross section of typical stone columns.

placements within the soil were measured using inclinometer/probe extensometers (IPEs). These instruments were installed at the locations listed in Table 3. In the stone column-treated areas, the instruments were installed midway between the stone columns as close as could be estimated.

Plots of piezometric level versus time for piezometers installed in the clay below the dike are shown in Figure 7. Measurements of pore pressures in the stone column-treated areas show that placement of about 40 feet of fill (embankment and CA/T fill) caused insignificant increase in pore pressures (5 percent or less of the weight of the added fill). By comparison, MS209 (located outside the treated area) showed the pore pressure as a percent of estimated surcharge to be 77 percent on November 16, 1994, with a steady decrease to 55 percent as of September 8, 1995.

TABLE 3.
Dike Instrumentation

Monitoring Station	Location	IPE Number	Initial Plate Magnet Elevation	VPWZ Number	Tip Elevation
MS202	Sta. 14+32 o/s 22'L	101	90.29	—	—
MS204	Sta. 12+20 o/s 20'L	103	82.48	105 106	49.45 70.45
MS207	Sta. 10+35 o/s 10'L	105	81.19	110 111 112	25.70 45.45 65.65
MS207A*		505	84.92	—	—
MS209	Sta. 7+80 o/s 22'L	107	87.89	116	63.20
MS209A*		507	93.36	—	—

Note: * IPE105 and IPE107 were damaged in July 1994 and replaced with IPE505 and IPE507, respectively, in April 1995 at offset locations.

Plots of piezometric level versus time for piezometers installed behind the dike (outside the stone column-treated area) are shown in Figure 8. These instruments are located in areas where CA/T fill was placed behind the dike to a level at, or slightly above, the dike crest elevation. They show pore pressure increases similar to those measured at MS209. However, the pore pressure increases occurred much more gradually because the rate of fill place-

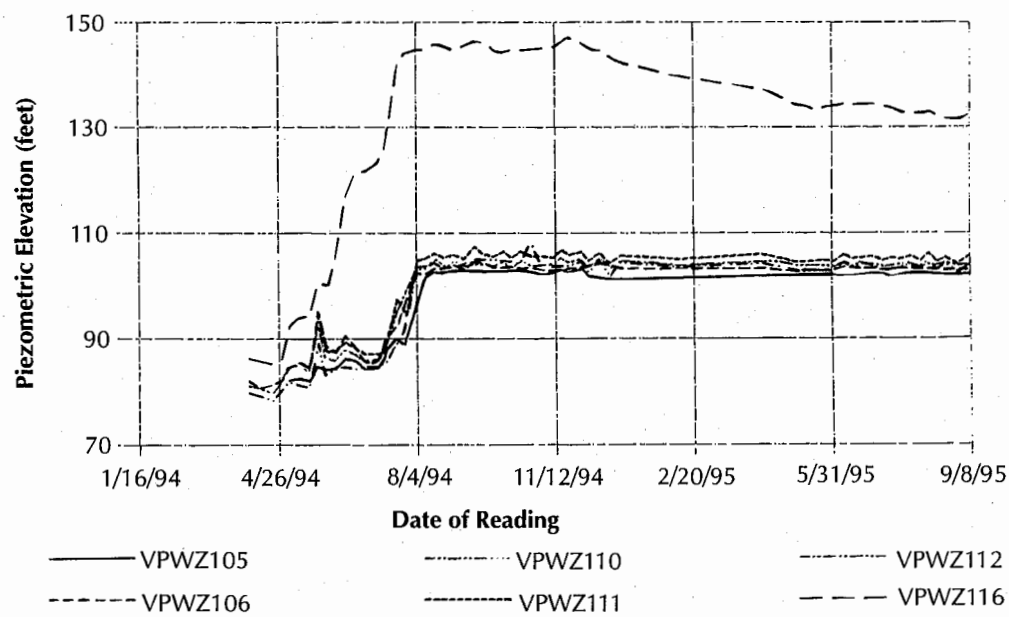

FIGURE 7. Measured piezometric levels in the clay below the dike.

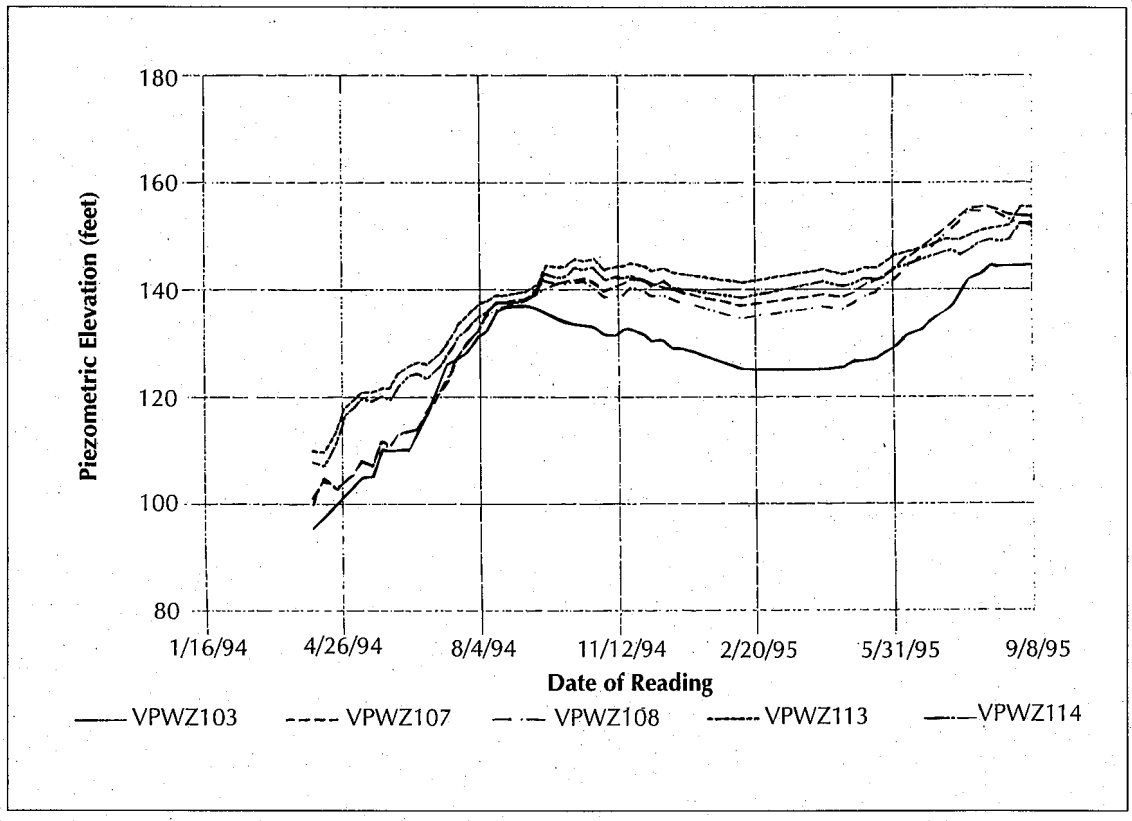

FIGURE 8. Measured piezometric readings behind the dike outside the stone column treated area.

ment behind the dike was much lower than the rate of dike construction.

It can be seen on Figure 7 that the piezometric level in the treated zone rose up to slightly above mean sea level (elevation 100) in late-July 1994 when the cofferdam was flooded.

To estimate the time rate of settlement, a coefficient of consolidation, c_v, of $1.7(10)^{-3}$ cm^2/sec was used for the marine clay, based on the average value from the consolidation tests. Without drainage provided by the stone columns and assuming a single drainage path, it was estimated that it would require about 70 years to dissipate 90 percent of the excess pore pressure for a clay thickness of 68 feet and 9 years for a clay thickness of 24 feet. Using stone columns with an area ratio of 0.15 (the average area ratio in the treated area), it was estimated that 50 percent of the excess pore pressure would dissipate in 8 days and that 90 percent would dissipate in 35 days.

The crest of the dike was designed to be crowned, so that it would be approximately level when all settlement is complete. To aid in estimating the needed height of the crown for the dike, the consolidation settlement of the dike was estimated at each station along the baseline. The equilibrium method was used to estimate the settlement ratio and then the consolidation settlement of the stone column treated area.[9] In this method, it is assumed that the stone column and the surrounding soil compress the same as each other. The measured settlements were compared with the predicted settlements and the settlement ratios used in the design.[9] These results are summarized in Table 4.

For the design, settlement ratios of 0.75 and 0.80 were assumed at MS204 and MS202, respectively. The settlement ratios as measured by these two instruments are reasonably close to the estimates, considering the difficulties in estimating the actual vertical stress increase in the clay. At MS207 and MS209, the IPEs were

TABLE 4.
Settlement

Station	Clay Thickness (feet)	Estimated Settlement in Untreated Soils (feet)	Measured Settlement (feet)	Settlement Ratio
MS204	41	0.92	0.67	0.725
MS202	32	0.50	0.36	0.720

damaged before significant settlement measurements could be obtained.

Lateral movements under the dike were measured using IPE101, IPE103, IPE105 and IPE107. Lateral movements behind the dike were also monitored using IPE102, IPE104 and IPE106. Movements in the clay under the dike were typically less than 2 inches. However, IPE103 measured a deflection of about 5 inches between elevation 43 (top of till) and elevation 70, with a nearly uniform shear strain of 1.5 to 2 percent. IPE105 showed a strain of about 1.0 percent between elevation 50 and 57, but the total deflection over this depth range was only about 1 inch. The IPEs behind the dike all showed less than 1 inch of movement in the clay.

Conclusion

Stone columns can be a cost-effective and technically feasible ground modification technique. As in all engineering projects, the advantages and disadvantages of different techniques must be weighed and trade-offs compared to find the best solution for the unique characteristics of a given project.

It is important to recognize that when comparing a technique that has been successfully used in other projects, even small differences can make a significant difference in the suitability of the technique at a second site. For example, the undrained shear strength of the clay at the Michigan site was not much lower than at Spectacle Island. However, the small increase over the large depth made a significant difference in the performance of the installation equipment, requiring that each stone column be pre-augered for the full depth. The project was successful because the contractor was given enough freedom to adjust techniques and tools to contend with actual site conditions and because the original design engineers were retained and actively involved during construction to advise on changes recommended by the contractor.

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