Verification of Surface Vibratory Compaction of Sand Deposit

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A site in Rhode Island underlain by loose to dense sand was to accommodate the construction of a submarine assembly building (SAB). The SAB was a one-story, 90-ft-high, steel frame industrial building with column spacings of approximately 75 by 205 ft. Maximum design column loads were approximately 1,600 kips. Inside the building were computer-controlled welding machine fixtures used to assemble the submarine bodies with precision tolerances. The fixtures were about 55 ft in diameter. Differential settlement tolerances of the welding machine fixtures were very small, whereas settlement tolerances of the building were consistent with flexible steel frame construction. Standard penetration test N-values in the sand deposit were generally higher than 15 blows/ft. However, within a depth of 0 to 10 ft below proposed footing grades, some areas exhibited N-values as low as 4 blows/ft. After evaluating several foundation schemes, spread footings (designed for 3 TSF) were selected to support the building. The footings would bear on a natural sand subgrade densified to a depth of about 6 ft by multiple passes of heavy vibratory rollers. The feasibility of subgrade densification was verified by a field test section using standard penetration tests, cone penetration tests, and plate load tests. Production densification was verified by cone penetration testing. The engineering evaluation, the field test section, and production verification test results are described.

During the late 1970s and 1980s, the nation strengthened its military forces. This included increasing the number of submarines in our arsenal. To accomplish this submarine assembly buildings (SABs) and related facilities were constructed.

The SAB considered in this paper is a one-story, 90-ft-high, steel frame industrial building occupying about 283,000 ft^2. The building is about 615 by 460 ft in plan dimensions and composed of three modules, each 205 by 460 ft in plan and housing 12 single-cylinder assembly fixtures. A building footprint is shown in Figure 1.

Column pairs supporting the structure are spaced on a 75- × 205-ft grid. Maximum combined load per column pair is approximately 1,600 kips. Allowable differential settlements are about 1½ in. because of the relatively long span between column supports. Allowable total settlements are about twice the allowable differential settlement. Each cylinder assembly fixture occupies a 55-ft-diameter circular area, with the following nonsimultaneous loadings: vertical loads, 500 kips; lateral load, 200 kips; radial tensions, 410 kips; moments, 2,600 kip-ft; torsion, 1,800 kip-ft; and moving vertical load, 800 kips.

The specified allowable differential settlement is less than ¼ in. across the fixture diameter. Details of the various loading conditions are described by Cuoco et al. (*1*).

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Investigations were conducted to determine subsurface conditions and develop recommendations for foundation support for the structure located on a site in Rhode Island. This paper describes the investigations, evaluations, foundation recommendations, and verification of site densification that allowed the use of spread footing foundations for the SAB structure. The evaluations described herein were accomplished in 1978. The intent is to present a case study showing the selection of foundation type and verification of subgrade densification that was necessary for the success of the selected foundation.

SITE CONDITIONS

Site Description

Several one-story warehouses occupied portions of the building site area. The site sloped gently towards the northeast from el 22.0 ft (local datum), to about el 14.0 ft.

Pertinent Geology

Overburden

Soil deposits were encountered (as described later) from the ground surface to depths averaging about 60 ft below original grades. These deposits are of the Pleistocene Age and consist mainly of fine sands and silt. The fine sands are likely ice-contact deposits (kames and kame terrace deposits) that were deposited between or beneath stagnant ice masses during the later stages of retreat of the Wisconsin glacier. The silt is probably the result of periods of quiescence following summer flooding.

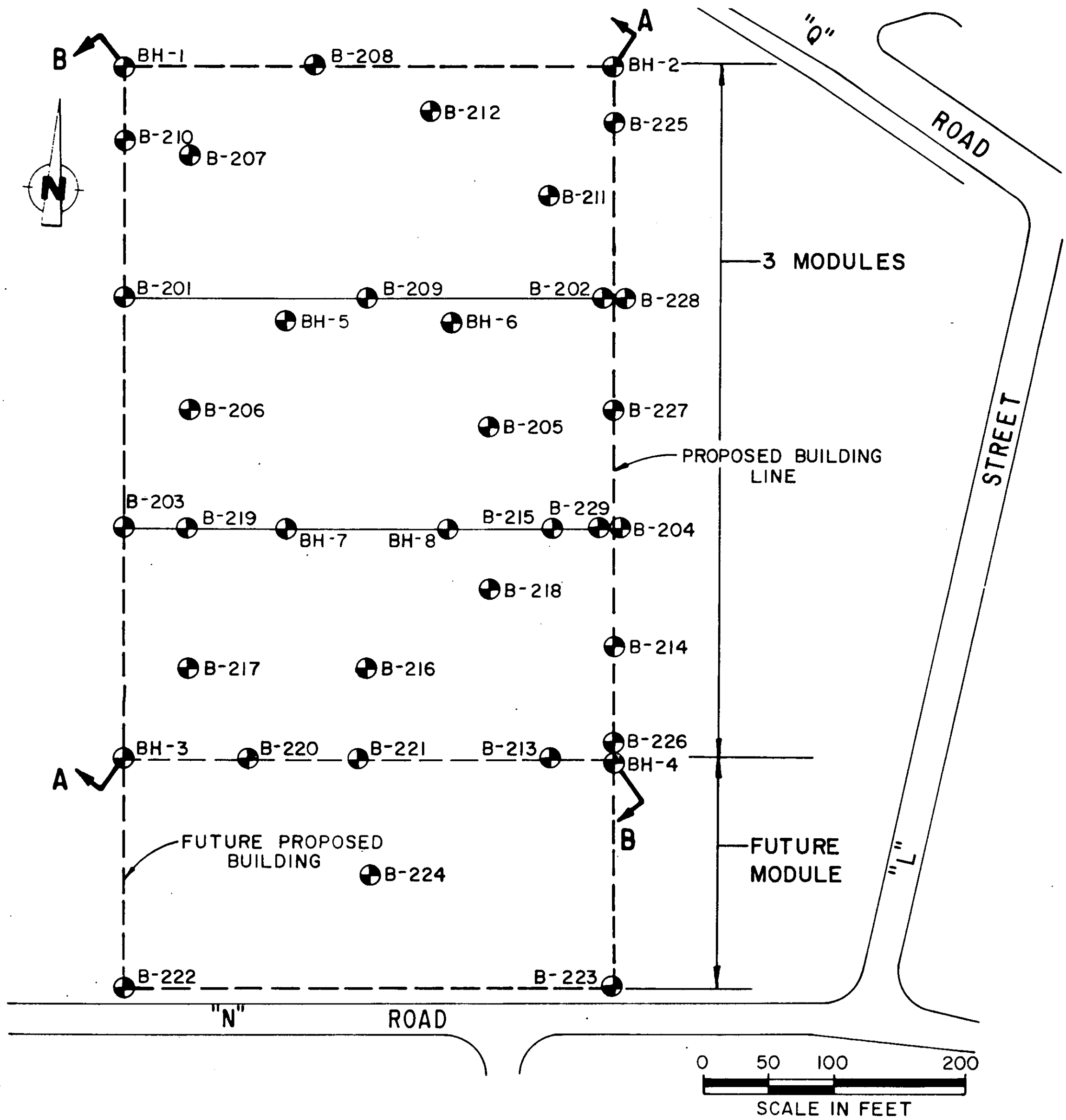
Glacial till underlying the silt stratum is a result of the action of the advancing and retreating glacial ice as it picked up and then redeposited soil and rock particles. The till is a dense heterogenous mixture of boulder, cobbles, gravel, sand, silt, and clay.

Bedrock

Underlying the Pleistocene deposits are metamorphosed quartz-feldspar sandstone, conglomerate, and phyllite.

Subsurface Investigation

Subsurface conditions in the building area were investigated during the design phase with 34 borings. Observation wells



LEGEND:

B-207 SOIL BORING & NUMBER

NOTES:

1.) OBSERVATION WELLS WERE INSTALLED AT BH-1, BH-2, BH-3, BH-4, BH-6 AND BH-7.

2.) SEE FIGURES 2 AND 3 FOR GENERALIZED PROFILES ALONG SECTION A AND B.

FIGURE 1 Plan of SAB and boring locations.

were installed in six borings. Additional borings were drilled in possible future expansion areas. The locations of the borings and observation wells are shown in Figure 1.

Bedrock was cored in selected borings using BX- or NX-size double-tube core barrels, generally to depths of at least 10 ft below the rock surface. These data were necessary for the evaluation of deep foundations.

Generalized subsurface conditions at the site are shown in Figures 2 and 3. The descriptions of soil strata given below are based on visual soil classifications and laboratory test data.

Fill

The fill is a fine to coarse sand of variable density. It contains varying percentages of silt and gravel and is classified SP to SM. The boundary between the fill and the underlying natural sand was difficult to identify in the boreholes because the fill and sand were similar in grain size and color. This stratum varied in thickness from about 2 ft to about 4½ ft.

Gray to Brown Fine Sand, Trace to Some Silt (SP to SM)

This stratum of highly variable density, as indicated by variations in *N*-values, is believed to be glacially derived as discussed earlier. Its high density at varying depths may be due to the compacting effect of rapidly flowing streams. Its contrasting low density may be due to the variable energy involved in this type of deposition, because the position of stream channels, the amount of flow, and flow velocities were all variable with time. Its thickness varied from about 40 to 60 ft. The *N*-values in this layer varied in the extreme from 4 to greater than 100. Thin silt layers occasionally were encountered within the sand stratum. Grain-size distributions of soil samples obtained at various depths are shown in Figure 4.

Gray to Dark Gray Nonplastic Silt (ML)

This stratum underlying the fine sand stratum is also believed to be glacially derived. Its thickness varied from 0

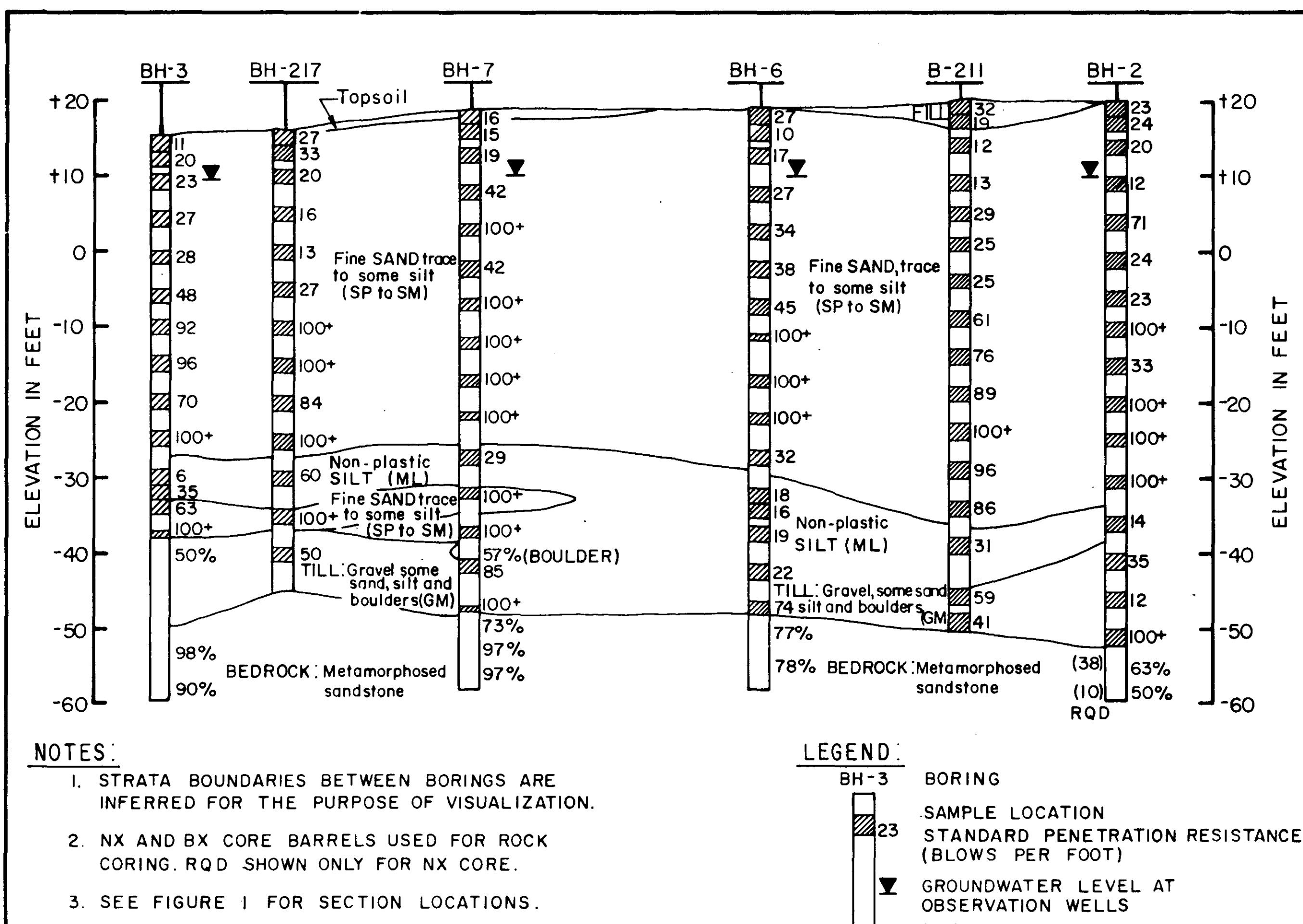


FIGURE 2 Generalized Subsurface Section A-A.

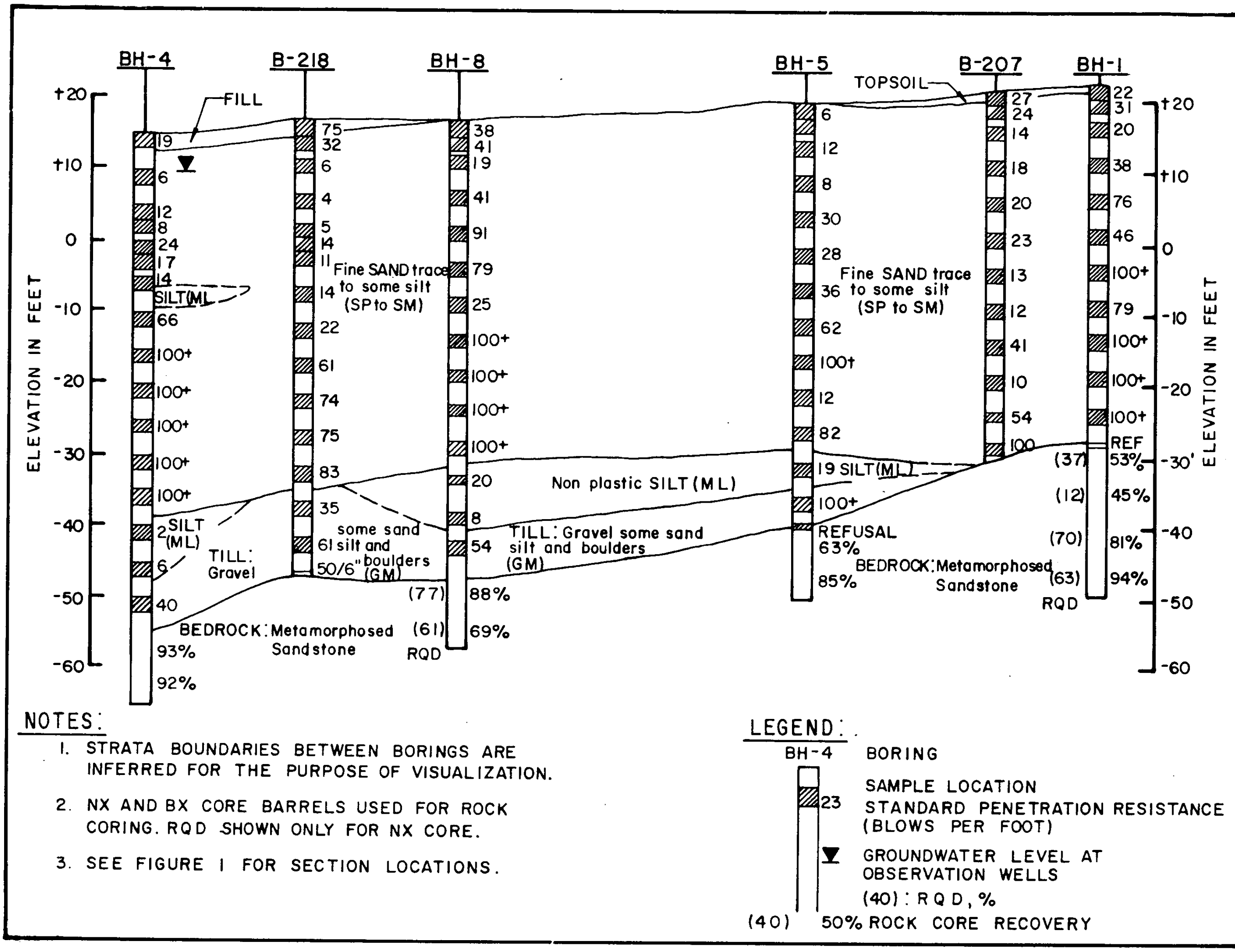
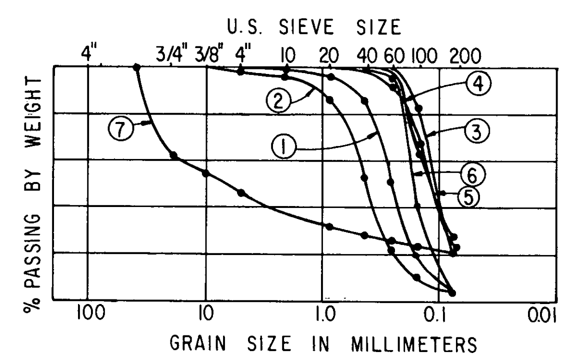


FIGURE 3 **Generalized Subsurface Section B-B.**



CURVE#	DEPTH (FT)
1	5-7
2	10-12
3	15-17
4	24-26
5	30-32
6	50-52
7	65-67

FIGURE 4 **Grain size distributions.**

to 14 ft. Occasionally, this stratum was interlayered with fine sand.

Six "undisturbed" samples of the silt (ML) were obtained (using a 3-in.-diameter Osterberg piston sampler) for laboratory compressibility testing because of concern that this stratum could affect shallow foundations.

Gray to Dark Gray Gravel, Some Coarse to Fine Sand, Silt, and Boulders (GM)

This stratum underlies the silt or the fine sand layer and is believed to be a glacial till. Its thickness varies from 0 to 16 ft. The N-values in this layer vary in the extreme from 12 to more than 100 with most values in excess of 35.

Bedrock

The bedrock consists mainly of interlayered quartz-feldspar sandstone, conglomerate, and phyllite. The top of bedrock within the site, as indicated by the borings, varies from el −28.0 to el −55.0.

Groundwater

The groundwater level, based on six observations wells, varied from el 9.0 to 11.7.

SELECTION OF SUITABLE FOUNDATIONS

The selection of the most suitable foundation system is based on the tolerance of the structure and equipment fixtures to settlement and on the compressibility of the subsoils. The structural engineer indicated that the steel-framed structure, with bay dimensions of 75½ by 205 ft, could tolerate total settlements of about 2 to 3 in. and differential settlements of about 1½ in. between column pairs.

Cylinder-assembly fixtures had to be treated as a "machine tool," according to its manufacturer. No out-of-plane settlements were tolerable, and only small differential settlements (about ¼ in.) between adjacent parts and between the column posts and the adjacent circular area were acceptable. Rotations of the column posts were unacceptable.

Spread Footings

The most significant stratum affecting the choice of foundation was the fine sand (SP to SM). This stratum is extremely variable, exhibiting both low (4) and high (100+) *N*-values. The variable *N*-values are indicative of variable density and compressibility. Untreated, this stratum was judged unsuitable to support the heavy column and equipment loads. Treatment was deemed necessary to reduce the lateral variability in soil properties and decrease the compressibility within a depth of about 50 percent of the footing widths. Since the extent of the areas exhibiting low *N*-values could not be accurately defined, the treatment was required over the entire area supporting foundations.

D'Appolonia et al. (*2*), Forssblad (*3*), Moorhouse and Baker (*4*), and others indicated that compaction at the ground surface with heavy vibratory rollers could be effective in densifying loose sand deposits to limited depths. The densification reduces the compressibility and the variability of the deposit. Surface compaction had been found to be effective to a maximum depth of about 7 ft. Thus, excavating the column areas to about el 12 ft and then compacting with about eight passes of a vibratory roller having a minimum weight of 12 tons was expected to produce a densified granular mat about 7 ft thick. This mat would support spread footings designed for a net bearing value of 3 t/ft^2.

The compressibility of the silt stratum underlying the SP to SM sand was evaluated on the basis of both *N*-values and the results of the laboratory consolidation tests (Figure 5) on relatively undisturbed samples. The two methods of evaluation gave similar indications of compressibility. Settlement analyses indicated that if the column loads were supported well above the top of the silt in a relatively competent stratum (at least 15 ft above the top of the silt), settlements resulting from one-dimensional compression of the silt were likely to range between about ¼ and ½ in.

Estimated postconstruction settlements with the above densification procedure ranged from about ¾ to 2½ in. (including settlement of the silt stratum). The maximum footing size was approximately 16 ft square. Settlement estimates were based on the settlement coefficient (SN/P) versus footing width presented by Schultze and Sherif (*5*) and variations of *N*-values versus elevation as shown in Figure 6 for the southeast quadrant of the site.

Schultze and Sherif (*5*) summarized the commonly used methods of estimating settlements on sands at that time (1970).

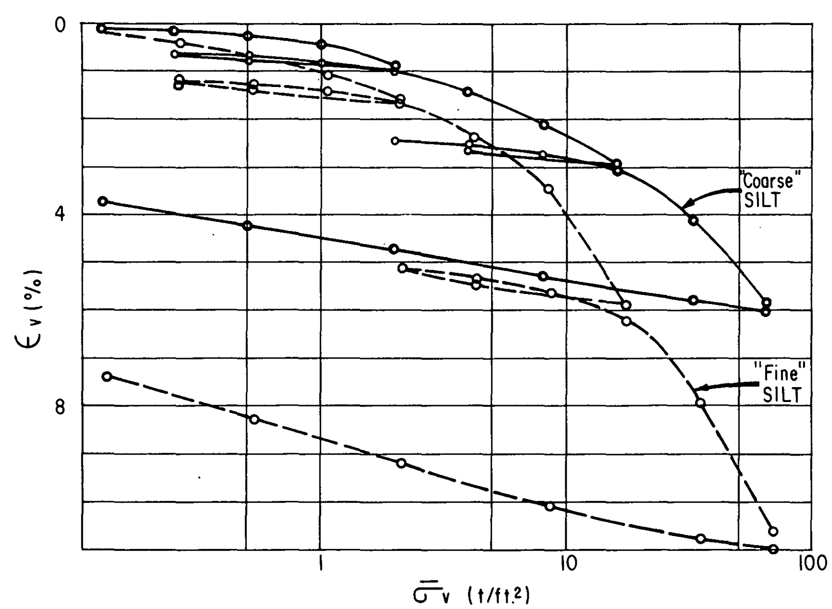


FIGURE 5 Consolidation tests on silt.

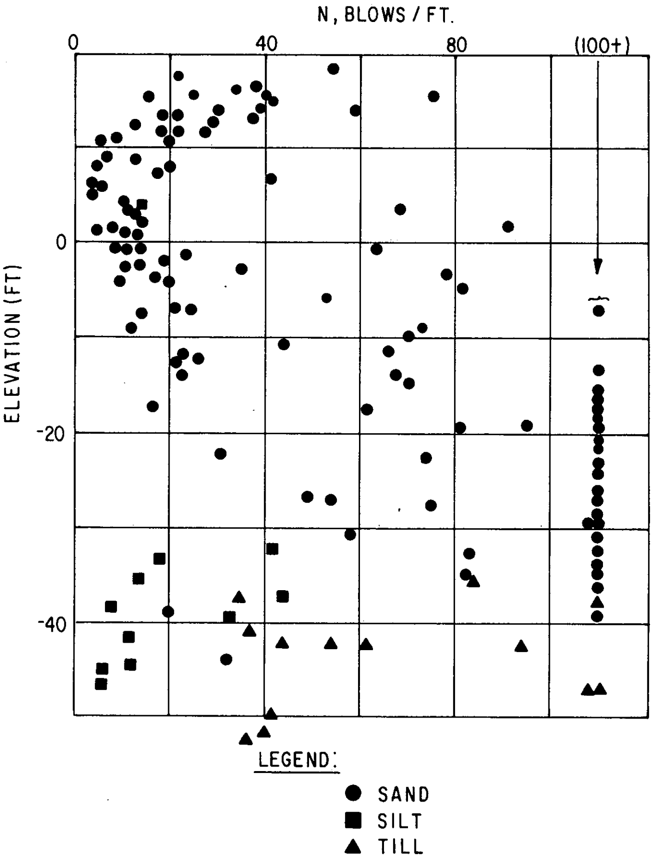


FIGURE 6 *N*-values versus elevation—southeast quadrant.

The summary indicated that a three-fold difference in estimated settlements could result depending on the method used. Therefore, the choice in method is highly subjective. The author used Schmertmann's (*6*) method, which was the most conservative one considered by Schultze and Sherif (*5*) for a 16-ft-wide footing. The point of this discussion is that for most practical projects wide variation in available methodology and wider variations in field data are the norm. Engineering judgment is required in selecting the approach to the project. Judgment is usually developed by experience on actual projects or by reviewing published case studies containing sufficient detail.

The cylinder assembly fixtures, if supported in the same manner as described above for the structure, were expected to experience settlements of about ¾ to 1¾ in. with differential settlements of up to about ¾ in. The amount of out-of-plane settlement and the amount of fixture column rotation could not be estimated reliably. The estimated settlements could have been reduced somewhat by reducing the bearing pressures. Piers drilled into bedrock were selected to support the fixtures.

Deep Foundations

Deep foundations, including various types of piles and drilled piers, were considered as alternatives to shallow foundations as described earlier. Since this paper is concerned with verification of support for shallow foundations, the discussion of deep foundations is omitted.

Required Construction Monitoring

A limited field test section to verify the feasibility of surface densification and a construction monitoring program were deemed necessary because of uncertainties in potential densification results as discussed later. The field test section was to verify that densification could be accomplished to an acceptable depth. The production construction subgrade monitoring program was to verify that densification was being achieved as a routine aspect of construction. Both of these programs were no more elaborate in concept than a conventional pile load test program and inspection of pile driving operations for a pile-supported foundation.

FIELD TEST PROGRAM

The field test program was deemed necessary because successful foundation performance depended on (*a*) elimination of loose and highly compressible zones from immediately beneath the footings and (*b*) a relatively uniform and dense granular mat to help minimize differential settlements. Also, the success of surface compaction depends on specific conditions at the site (e.g., groundwater, grain size of the soil, localized soil layering, characteristics of compactor, and other unknown factors) that can only be evaluated by a field test.

The verification consisted of the evaluation of the depth of densification, the relative amount of densification, and the relative uniformity of the densification. Experience indicated that *N*-values could be anomalous. Therefore several types of testing were used for the feasibility evaluation.

A 100- × 20-ft test section, as shown in Figure 7a, was chosen within the southeastern quadrant of the proposed building area. The entire test section was excavated to a depth of 2½ ft, resulting in an average test section elevation of 13 ± or about 1 ft above groundwater level.

The southeast quadrant was selected because the results of the boring program carried out during the geotechnical investigation seemed to indicate that the loosest near-surface soil conditions existed in this area. This "indication" was recognized as a possible misrepresentation of actual conditions, since the geology of the area suggested no differences from one portion of the site to the other. Nevertheless, this portion of this site was selected because it was potentially the most critical area.

Description of Verification Tests

As shown in Figure 7b, the test section was divided into three major areas, identified as A, B, and C, and a fourth minor area, Aux. Three plate bearing tests (one in each of Areas A, B, and C) were performed before densification, and three tests were performed after densification. Three test borings were drilled before densification in the test section (one each at Areas A, B, and C) and three after densification. A fourth boring to a depth of 7½ ft was added in the field when the

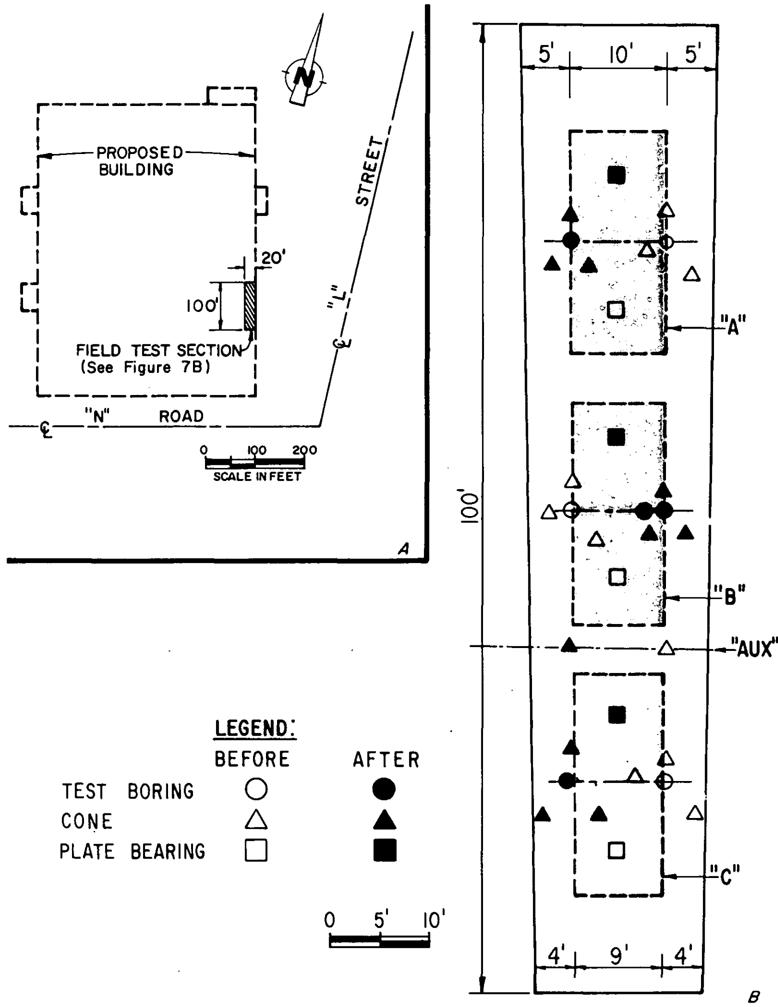


FIGURE 7 Test section—(*a*) location, (*b*) plan.

near surface *N*-values in B-2 appeared low, possibly because of drilling techniques. Three cone penetration tests were performed in each of the Test Areas A, B, and C before densification. The cone penetration tests were located approximately 2 to 3 ft from the test boring. After densification, three additional cone penetration tests were performed in each of the three Test Areas A, B, and C. The cone penetration tests were located around each boring. The auxiliary area was used for performing only cone penetration tests, one before densification and a second after densification. The locations of all these tests are shown in Figure 7b, and brief descriptions of the test procedures follow.

Test Borings

The test borings were drilled using rotary drilling techniques with recirculating bentonite drilling mud and an Acker AD-11 truck-mounted rig. Standard penetration tests (SPTs) were performed in each of the borings in accordance with ASTM D 1586-67. The SPT was conducted continuously for the first 10-ft depth interval. At greater depths, the tests were conducted at 5-ft intervals. The maximum boring depth was 20 ft. SPT results are shown in Figure 8.

Cone Penetration Tests

The cone penetration tests were conducted using an electrical static cone penetrometer with a cone having 60-degree tip and 10-cm^2 base area. The cone was pushed hydraulically by the Acker AD-11 drilling rig at a uniform rate in the range of about 2 to 2.5 cm/sec. The cone point resistance was monitored by a waterproof, full-bridged strain gauge load cell (capacity 6,000 lb) housed in a stainless steel casing. Standard EW drill rods were used for pressing the cone into the ground. In some cases, where the maximum rig reaction was reached at a depth less than 15 ft, additional penetration was accomplished by mud drilling to clean out the hole before advancing the cone. In this way friction on the rods was eliminated. The maximum depth of the cone penetration tests was 20 ft. The strain signals were transmitted to a small electronics package containing a bridge balancing circuit and power supply through a shielded electrical cable. The signals were recorded continuously as cone resistance on one channel of a two-channel strip chart recorder. Depending on the actual cone resistance recorded, the range of scale settings on the strip chart recorder was appropriately adjusted to produce data portrayed suitable for interpretation. The results of the cone penetration tests are shown in Figure 9.

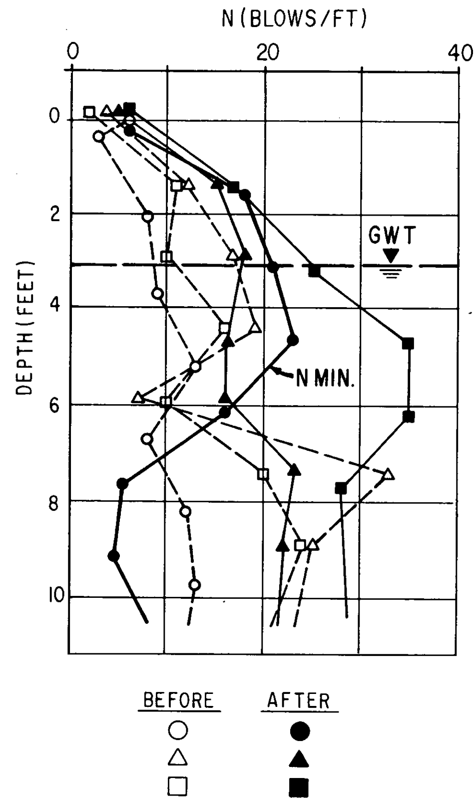


FIGURE 8 **Test section results—SPT.**

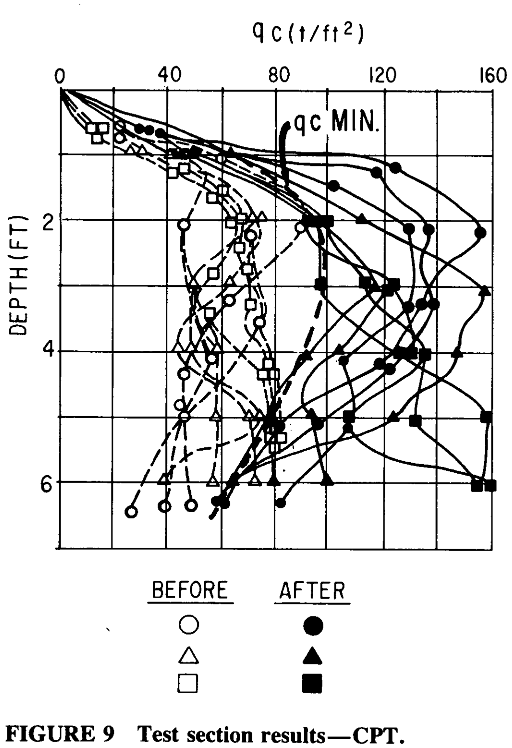


FIGURE 9 **Test section results—CPT.**

Plate Bearing Tests

The plate bearing tests were conducted in 30- × 30-in. hand-dug pits. A 24- × 24- × 1-in.-thick steel plate was placed at elevation 11± under 12- × 12- × 1-in.-thick and 18- × 18- × 1-in. thick steel plates to minimize deflection of the plate under load. The tests were performed in general accordance with the procedures given in ASTM D 1194-72. Each test was carried to a maximum load of 7.2 k/ft^2 (1.2 × design bearing value of 6.0 k/ft^2), applied in five equal load increments. Each load increment was maintained until the rate of settlement was small, usually less than 0.001 in., in 10 min. After reaching and holding the maximum load, the unloading was accomplished in three equal decrements. The load was applied through a calibrated 20-ton hydraulic jack. Reaction was provided by two metal boxes, each filled with 20 kips of lead and mounted on a girder weighing 5 kips as reaction frame. Settlement (deformation) measurements were made using four dial gauges mounted at the four plate corners. The gauges had a least count of 0.001 in. and measurements were documented at 1-min to 5-min intervals. In all other details, the tests and documentation thereof were performed in accordance with ASTM procedures. The results of the tests are shown in Figure 10.

Compaction of Test Subgrade

Upon completion of the three types of "before densification" tests, the entire test subgrade was cleaned, leveled, and graded with a D7E Caterpillar to an average elevation of 12.8. The test area was then compacted with eight passes of a heavy Ingersoll-Rand SP-60 DD drum drive self-propelled vibratory compactor with a 100-in. drum width and 60-in. drum diameter, delivering an applied force of 83,100 lb per impact and having a static weight of 24,130 lb (at drum).

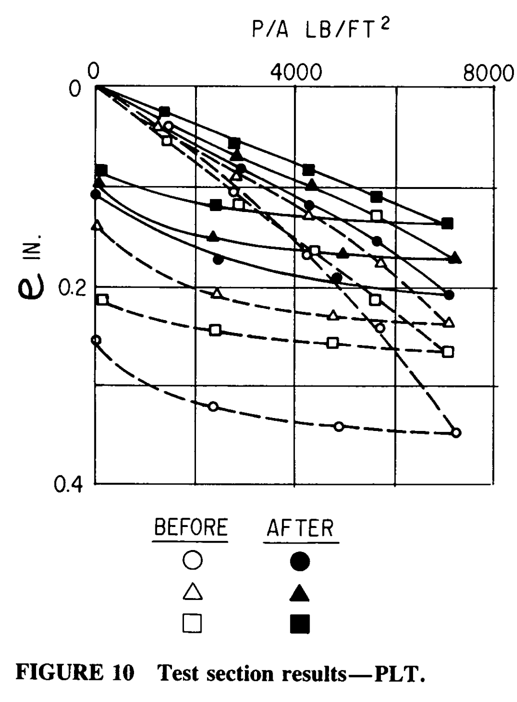


FIGURE 10 **Test section results—PLT.**

TABLE 1 Before and After Compaction Soil Parameters

	Before (Average)	After (Average)	After/Before Ratio
qc (t/ft^2)	55±	110±	2
N	10±	22±	2
qc/N	5.5 (4-6)	5±	1
E (t/ft^2)	224	369	1.65
E/qc	4	3.3±	0.8
E/N	22	17	0.8

The compactor was operated at a frequency of 25 Hz and at a uniform travel speed of 2 mph, except for the last two passes when, due to trafficability difficulty, the speed was increased to 3 mph. Upon increasing the travel speed, no further difficulties were encountered. Spot level readings taken before and after compaction indicated that the subgrade settled about 3½ in. because of compaction activities. This suggests about a 5 percent average increase in density over a 7-ft thickness. The three series of "after densification" tests were then commenced and completed over the next few days.

Test Results

The "before" and "after" test results from the test borings, cone penetration tests, and the plate bearing tests are presented in Figures 8, 9, and 10, respectively. These tests show that (with the exception of the borings in Section B) a substantial increase in soil resistance to penetration occurred after densification to a depth of about 6 to 7 ft. In most cases the penetration resistance almost doubled after densification was achieved. Although the borings in Area B did not indicate that compaction had been achieved, both the plate load tests and the cone penetration tests indicated substantial densification.

The plate load test data shown in Figure 10 are a direct indication of the compressibility of the approximately 4-ft-thick zone immediately below the 2-ft-square test plates. The before and after tests indicate that the average compressibility within this zone was reduced by about 60 percent by the densification. The estimated settlements at the design loads (after compaction) were about 0.15 in. The actual foundations were to be much larger than 2 ft square and the stressed zone beneath them would extend well below the densified depth.

Evaluation of the before and after plate load test data, the densification suggested by spot survey, the cone penetration and boring data, and the proposed structure and fixture foundations indicated that the settlement estimates stated earlier were reasonable and prudent. From these series of tests results and analyses, it was concluded that the subsoils to a depth of about 6 ft (corresponding to about elevation 6) were densified to the extent anticipated. The compaction effort had little, if any, effect in the upper 6-in. to 1-ft depth zone and had a maximum beneficial effect between the depths of about 2 to 4 ft. The vibratory compactor used in this study was not capable of densifying the loose zones occasionally encountered below about elevation 5.5 (corresponding to a depth of 7 ft) mostly in Test Area B. Such loose zones were not encountered in other test areas. However, this type of variability is typical of that encountered in the original subsurface investigation. One purpose of the field densification program was to eliminate this variability from the shallow zone immediately below the footings. The remaining deeper loose zones were considered in previous evaluations in estimating settlement and differential settlement.

The ratio of cone resistance to *N*-value *qc*/*N* is useful in converting from one test to the other and is believed to be a function of soil type. Alperstein and Leifer (*7*) summarized *qc*/*N* presented by several investigators and reported for fine sand (SP-SM) that *qc*/*N* varied from 3 to 6. The data in Figures 8 and 9 indicate that *qc*/*N* for this site was about 5 before and after compaction, well within the 3 to 6 correlation.

Table 1 gives a comparison of before and after average soil properties. Clearly, the penetration resistances (*qc* and *N*) doubled and Young's modulus increased by 65 percent. Possibly, the higher penetration resistance increase compared to modulus increase results from an increase in lateral stresses as well as densification (*8*). The ratio *qc*/*N* remained essentially constant at a value consistent with that reported in the literature. Significantly, the before and after ratio of *E*/*qc* was significantly higher than assumed during design (*6*). Therefore, the earlier settlement estimates were considered prudent, because a conservative modulus was originally used. Also, the relationship between *E* and *N* is consistent with data reported by Wrench and Nowatski (*9*) some 7 years after the information presented herein was developed.

CONSTRUCTION VERIFICATION PROGRAM

The construction verification program was performed during December 1978 and January 1979 using the same equipment and procedures as with the test section.

Frequency and Depth of Testing

The frequency of testing was one cone penetration sounding (or one boring) at each major column location and one at the center of each fixture.

Ninety cone penetration tests and 10 borings with standard penetration tests were performed as part of the construction monitoring program. The cone penetration tests and standard penetration tests were performed after eight passes of the SP-60DD vibratory roller. Generally, the depth of cone penetration after densification was terminated at 7.0 ft. The SPT

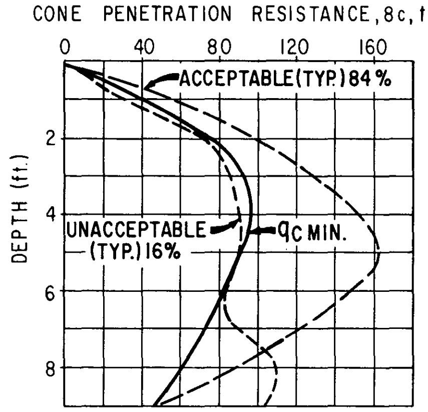


FIGURE 11 Typical production test results.

was conducted continuously to maximum depths of 6.5 to 8.0 ft.

Test Results and Criteria for Acceptability

The criteria for acceptability of the field densification procedure were based on the "minimum envelope" results from the field test section. The minimum envelope for the cone penetration tests performed after densification *gc*(MIN) is shown in Figure 9; the minimum envelope for the standard penetration tests performed after densification *N*(MIN) is shown in Figure 8. In most cases the observed penetration resistance during production exceeded the minimum envelopes and was considered acceptable. Approximately 84 percent of the production test results were considered acceptable. The remaining 16 percent were considered originally to be either marginal or unacceptable. Figure 11 shows typical "acceptable" and "unacceptable" results.

Where tests typified on Figure 11 failed to meet the established criteria (marginal or unacceptable), the actual differences between the observed resistances and the minimum envelope were usually quite small. Therefore rerolling (minimum of eight additional passes with the SP-60DD) of the surface was the required corrective action.

The production tests show that a substantial increase in soil resistance to penetration occurred after densification to a depth of about 6 to 7 ft and that densification was achieved to the extent anticipated.

CONCLUSIONS

The following conclusions are drawn from the data and discussion:

1. Eight passes of the SP-60DD vibratory roller were successful in densifying the sand subgrade to a depth of approximately 7 ft.
2. A test section demonstrated the potential success of the vibratory densification method and showed the applicability of three verification techniques—cone penetration sounding, standard penetration test, and plate load tests.
3. Production verification using cone penetration soundings and SPT occasionally was successful. Verification criteria were based on the test section results.
4. Postconstruction settlements were not measured. However, the facility has been used for many years without complaints by the owner. This suggests that the foundation treatments are successful.

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