Condition of Large Caissons During Construction

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The pier foundation consisted of a massive concrete caisson placed by open dredge method. The Brooklyn tower pier is embedded in glacial sand of the Wisconsin epoch and is founded 170 ft below sea level on silty clay of the Gardiner interval. Similar glacial sand is encountered at the Staten Island pier which founded at El. -105 ft on the transition of morainal deposit.

At any stage of sinking, the effective weight of the caissons should exceed the anticipated resistance which must be evaluated before the detailed planning. The present methods for determining such resistance, on the basis of soil tests or experience of individual engineers, are widely diversified.

During the sinking and buildup stages of both caissons, records were systematically compiled and tests were rationally analyzed. The bearing capacity of the silty clay at the cutting edge was determined to be 5.9 to 6.5 times shearing strength. The skin friction on the caisson is maximum at the very beginning of sinking and is reduced to a resistance equivalent to 45 percent of average overburden pressure. The lower range of skin friction would depend on the method and manner of applying water jets and compressed air.

•IMPORTANT FEATURES of New York City's Narrows Bridge have been described in an article published in the Engineering News Record (1). Among the many outstanding features of this longest suspension span are the tower foundations, which are of unprecedented dimension and depth of embedment in the ground. At the present stage of construction (August 1963), both pier foundations and steel towers have been completed and the spinning of the cables is in progress.

The pier foundation is actually of the most conventional type, consisting of concrete caisson with open wells for dredging. During the construction, the driving force for seating the caisson was gained by removing the supporting soils at the bottom. Therefore, at any stage of the sinking, the effective weight of a caisson should exceed the anticipated resistance, which consists of the skin friction on caisson surface as well as the bearing resistance of soils at the cutting edge. The theoretical methods for determining such resistance on the basis of soil tests are not often reliable and experiences of individual engineers are widely diversified. It is the purpose of this paper to review the actual mechanics of sinking a caisson.

SUBSOIL CONDITION

Geological Review (2)

The area is underlain by metamorphosed rocks of the pre-Cambrian age. Sediments of Cretaceous age are encountered above the basement rock. As the early continental glacier advanced toward the north shore of Long Island, outwash materials deposited great beds of gravel and clay (Jameco Stage) in the old Sound River Valley which was not too far from the Brooklyn pier. During the subsequent retreat (Sankaty Stage), the land was in a gradual uplift and the Sankaty sediments emerged about 50 ft above water.

With the return of the subsequent glacier, the older beds were overridden and folded. Gay Head outwash and Gardiner sediments were laid on the top of these folds. The area

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continued its progressive subsidence throughout the third glacial epoch. At the Tisbury Stage, the land had subsided about 250 ft. Beds of gravel and boulders were deposited on the older sediments and extended over the entire Sound River Valley. During the long erosion period of the Vineyard interval, the present drainage pattern of the Hudson River was established. Fresh water sediments were accumulated at a rather slow pace, permitting the growth of organisms.

At the return of the last glacier, the Wisconsin, initial advancement did not reach the bridge site. Because of the rapid melting of the ice, the outwash plain is predominantly of sand and gravel. During the intermission of glacial advancement, the sea

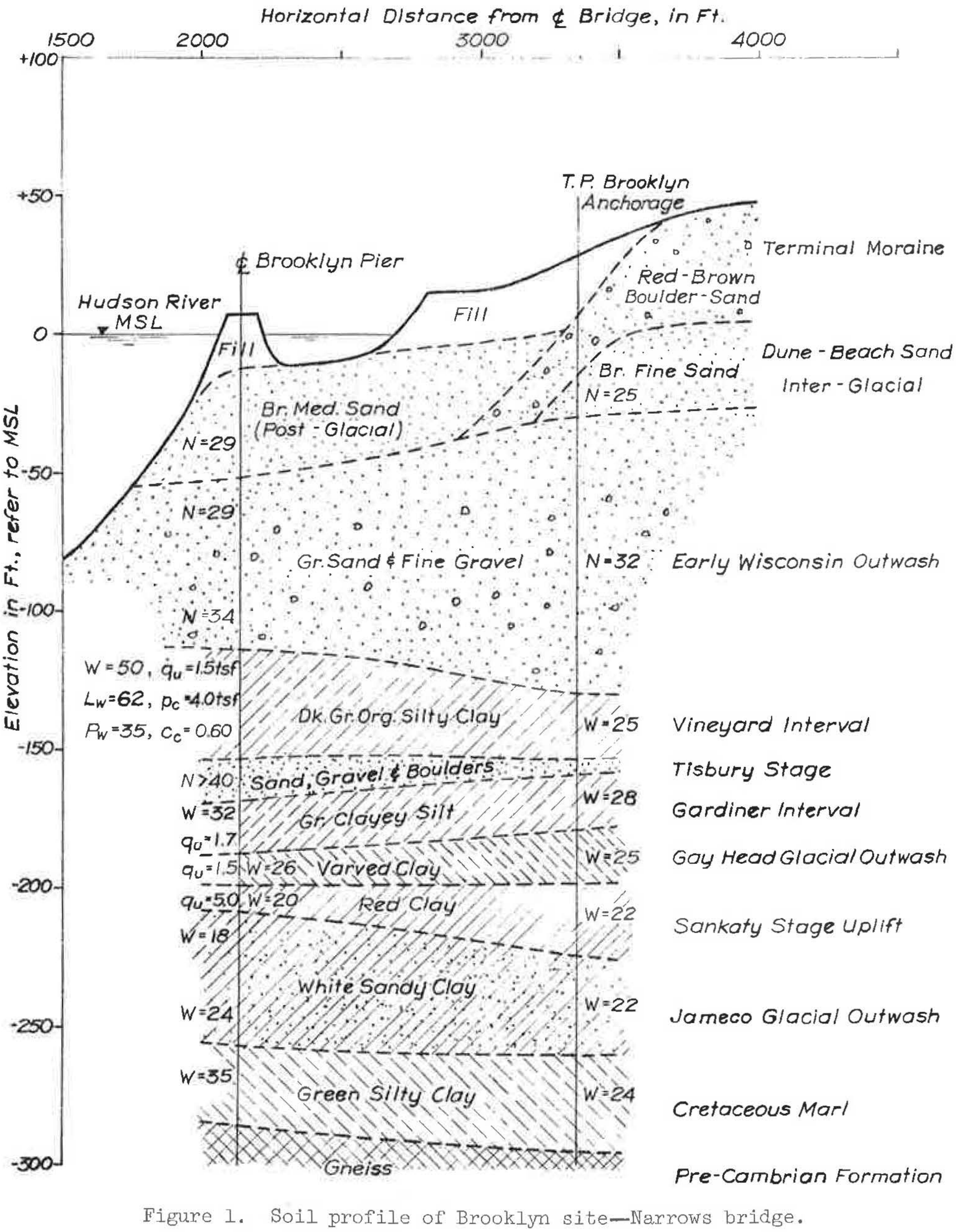

Figure 1. Soil profile of Brooklyn site—Narrows bridge.

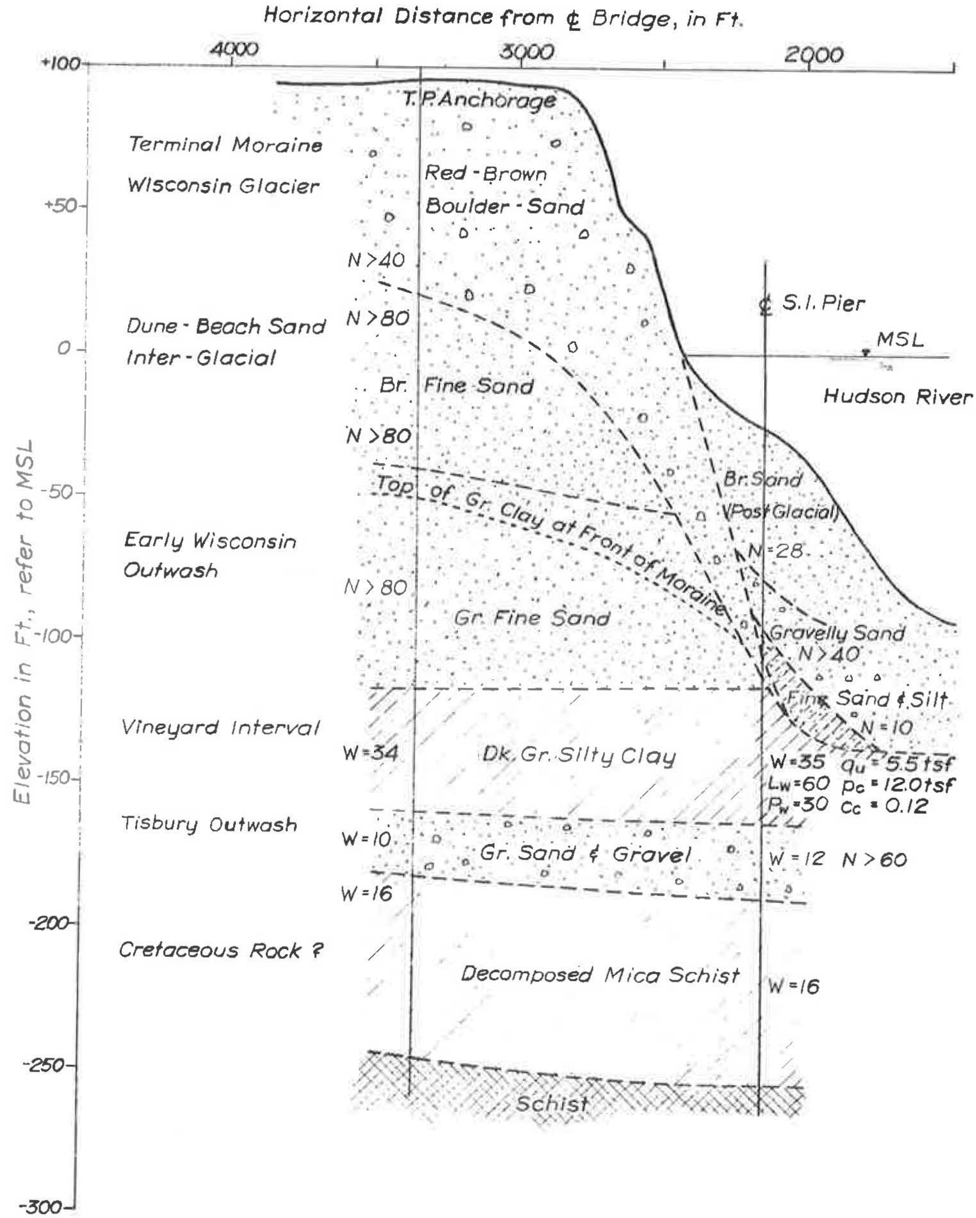

Figure 2. Soil profile at Staten Island site—Narrows bridge.

level was about 50 ft below the present level. The Hudson River cut through the outwash plain and sand dunes were formed along the beach. At the last movement of the Wisconsin glacier, the entire bridge site, except the Brooklyn pier, was covered by the ice. Terminal moraines formed the hilly topography along the present shoreline of the Narrows.

Since the retreat of the last glacier, there has been no significant sedimentation in the Hudson River, except the gradual fill-up of the deep channel following the rising of sea level. Soil profiles for the Brooklyn and Staten Island sites have been constructed and are shown in Figures 1 and 2, respectively.

Basement Rock

The basement rock at the Brooklyn pier is encountered at 290 ft below sea level and consists of gray gneiss. During the test pile operation, a NX-size diamond core was drilled 10 ft into the rock. Average drilling time was 7 min/ft and a solid piece 8.3 ft long was recovered. The uppermost several inches were slightly decayed. Overlying the gneiss is a layer about 1 ft thick of lime-cemented soft rock. At the Staten Island pier, weathered gneiss and schist were encountered in the borings. The rock surface is about 30 ft higher than that at Brooklyn pier.

Cretaceous Deposits

Green clay marl is encountered at the Brooklyn site. The average moisture content is 35 percent at the tower pier and 24 percent under the anchorage where the terminal moraine was located. The clay marl under the anchorage is likely to be precompressed by the weight of the glacier. At the Staten Island site, the formation of Cretaceous age is not clearly defined.

White Sandy Clay

It was a glacial filling of the Sound River Valley. The clay soil is possibly derived by the combined action of weathering and sedimentation. There are resemblances of residual soils. However, pockets of fine sand and patches of lignite are commonly pronounced in the deposit. The deposit is about 50 ft thick and is encountered at 210 ft below sea level. The uppermost part of the deposit is mottled with white, yellow and red, but the lower part is predominantly white. The unconfined compressive strength is estimated to be 4 to 6 tsf for the upper part of the deposit and 3 to 4 tsf for the lower one. Desiccation may have an important role in effecting the strength of this deposit.

Red Clay

It is a very sticky clay and was deposited during the gradual uplift of the coastal area. Although the red clay is only 5 to 10 ft thick, it represents the demarcation between the hard materials of older age and the subsequent deposits of moderate strength. The unconfined compressive strength ranges from 4 to 6 tsf and preconsolidation pressure is estimated from the consolidation tests to be 12 to 14 tsf. The compression index, C_c, is 0.10. The high strength of red clay is assumed to be caused by the internal consolidation pressure during desiccation.

Varved Clay

This deposit is encountered at 170 ft below sea level and extends 30 ft to the red clay layer. The sediments were brought down during the Gay Head and Gardiner Stages. The unconfined compressive strength ranges from 1.5 to 2.0 tsf. The preconsolidation pressure, according to the consolidation test, is estimated to be 4.5 to 5.0 tsf which is equivalent to the present overburden pressure. The compression index ranges from 0.20 to 0.30. It is a fairly compressible material.

Tisbury Sand

At the Tisbury Stage, glacial material appeared at the Staten Island site. The entire valley of the Sound River had been completely filled. The Tisbury formation is relatively thin, ranging from 5 to 10 ft at the Brooklyn site and 20 ft at the Staten Island site. Boulders are encountered in the deposit.

Organic Silty Clay

This deposit was brought down during the long recession (Vineyard Interval) of the continental glacier. The deposit is fairly thick, 40 ft at Brooklyn and 50 ft at Staten Island, as compared with the sediments of earlier glaciation. The Vineyard formation is a bay sediment of organic silt, having a liquid limit of 62 percent and plastic limit of 35 percent. At the Brooklyn pier, the natural water content is 50 percent, unconfined compressive strength ranges from 1.0 to 1.6 tsf, and preconsolidation pressure is estimated to be 4.0 tsf which is equivalent to the present overburden pressure. The compression index is about 0.60. It is a very compressible soil. The preconsolidation pressure of the same soil formation under the Brooklyn anchorage and the Staten Island pier and anchorage foundations ranges from 10 to 16 tsf. The natural moisture content is 25 to 35 percent and the unconfined compressive strength is 5.0 to 6.5 tsf. Compression index is only 0.10. It is likely that this portion of the Vineyard formation has been precompressed by the weight of the glacier. It is interesting to note that in boring B-3 at the southeast corner of the Staten Island pier, where the front of the terminal moraine is encountered, the unconfined compressive strength of the silty clay decreases from 6.4 to 5.5 and 4.9 tsf, whereas the depth of the clay soil increases from El. -145 to -156 and -163 ft, respectively. The natural water content of the corresponding soil sample varies from 33 to 36 and 38 percent, respectively.

The Vineyard silty clay is a sensitive soil. The ratio of unconfined compressive strength between undisturbed samples and the completely remolded ones ranges from 3.5 at Staten Island to 5.0 at the Brooklyn.

Wisconsin Sand

The outwash of early Wisconsin glacier consists of medium sand and fine gravel. The average size of D_{10}, 10 percent finer, is 0.17 mm (No. 80 sieve), and the average uniformity of grain size, ratio between D_{60}/D_{10}, is 3.7. It is a very clean sand containing only 2 to 4 percent finer than No. 200 sieve.

Postglacial Sand

The postglacial sediments consist of brown medium sand having a standard penetration resistance of 25 to 35 blows and uniformity of grain size of 4 to 6, similar to that of Wisconsin sand. The deposits at the Staten Island pier, however, are complicated by the presence of the terminal moraine and the old erosion channel of the Hudson River. Fine sand and soft silt were encountered in the old channel.

Fill

On the site of the Brooklyn pier was a 140-year-old fort, Fort Lafayette. This island was built up by sand fill and riprap to an elevation about 10 ft above sea level.

Discrepancy of Boring Information

Conventional wash borings were taken in 1954 and 1957 to determine the foundation condition and the strength of subsoil. As a common practice, the penetration resistance, in terms of hammer blows on soil samples, was used as a guide in determining the relative density of sand. In Figure 3, the number of hammer blows, equivalent to the N-value of standard penetration tests, varied from 85 to 320 for the soils at El. -160 to -240 ft in boring C-2. There was some belief that the sand layers were so compact that no pile could possibly be driven through. At the subsequent test pile operation, a 30-in. O. D. open-end pipe was easily driven through at a resistance ranging from 3 to 8 blows/in. which was not considered hard driving. When borings were taken at a great depth, the sampling rods became very flexible and their weight heavy in comparison with the driving hammer. The energy of the hammer impact was absorbed by the vibration of the rods and high blow counts were, therefore, recorded.

The presence of pore-water pressure at the bottom of a boring hole, however, would cause a significant decrease of blow count at the sampling. In a saturated fine sand, the pore water tended to flow into the casing pipe as soon as the overburden was removed. The fine sand could become partially unstable. Consequently, a low blow count was registered in the boring record. For the final borings taken during the caisson construction, drilling mud was used to fill up the casing to reduce the negative pressure at the bottom of the boring hole. The blow counts for the mud holes were significantly different from the early wash borings. In Figure 4, the field count of hammer blows was plotted against the relative density of the same sand tested in the laboratory. The N-value of the mud borings are in closer agreement with those found by others (3, 4).

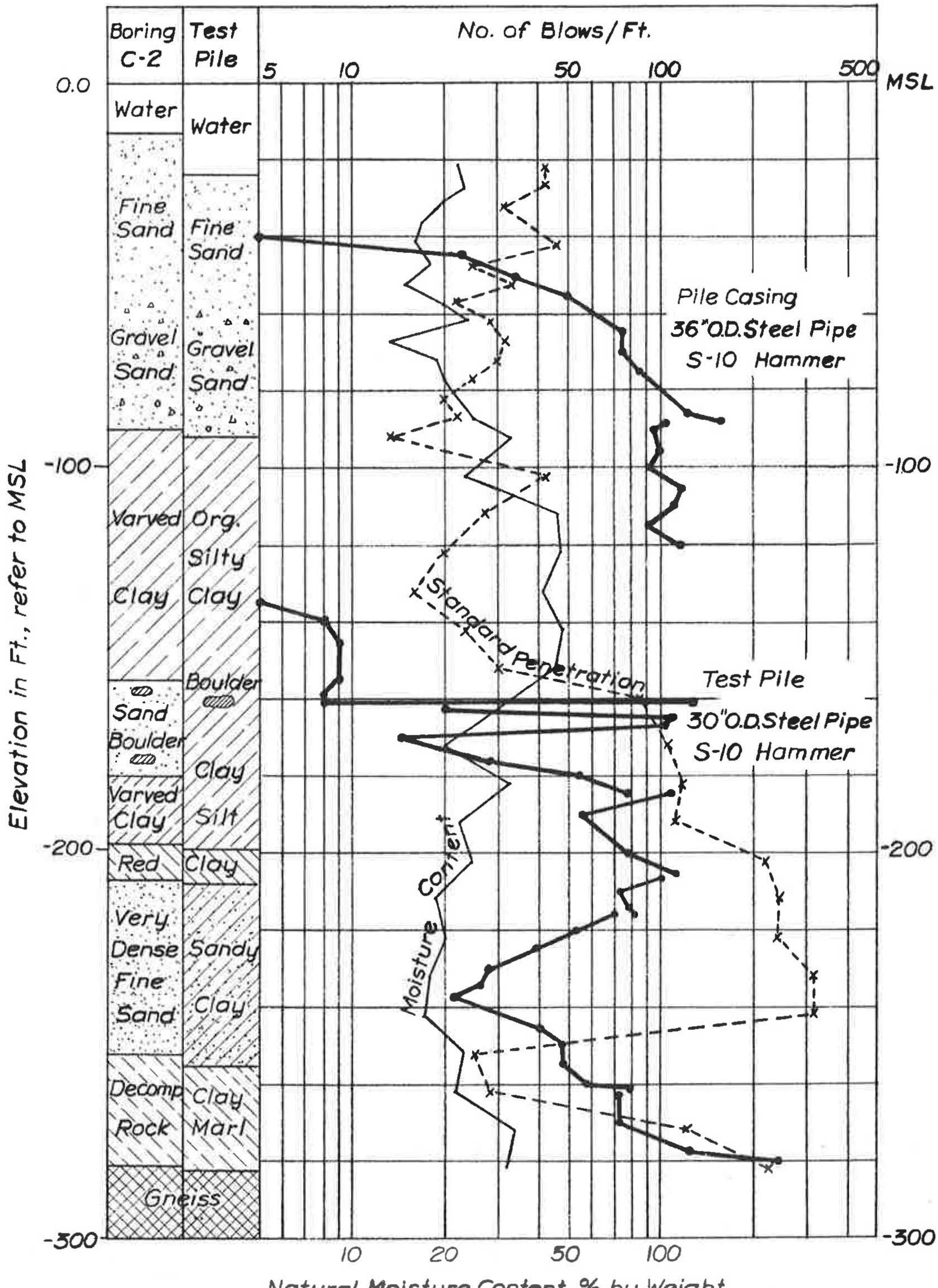

Figure 3. Comparison of pile driving and penetration resistance of boring.

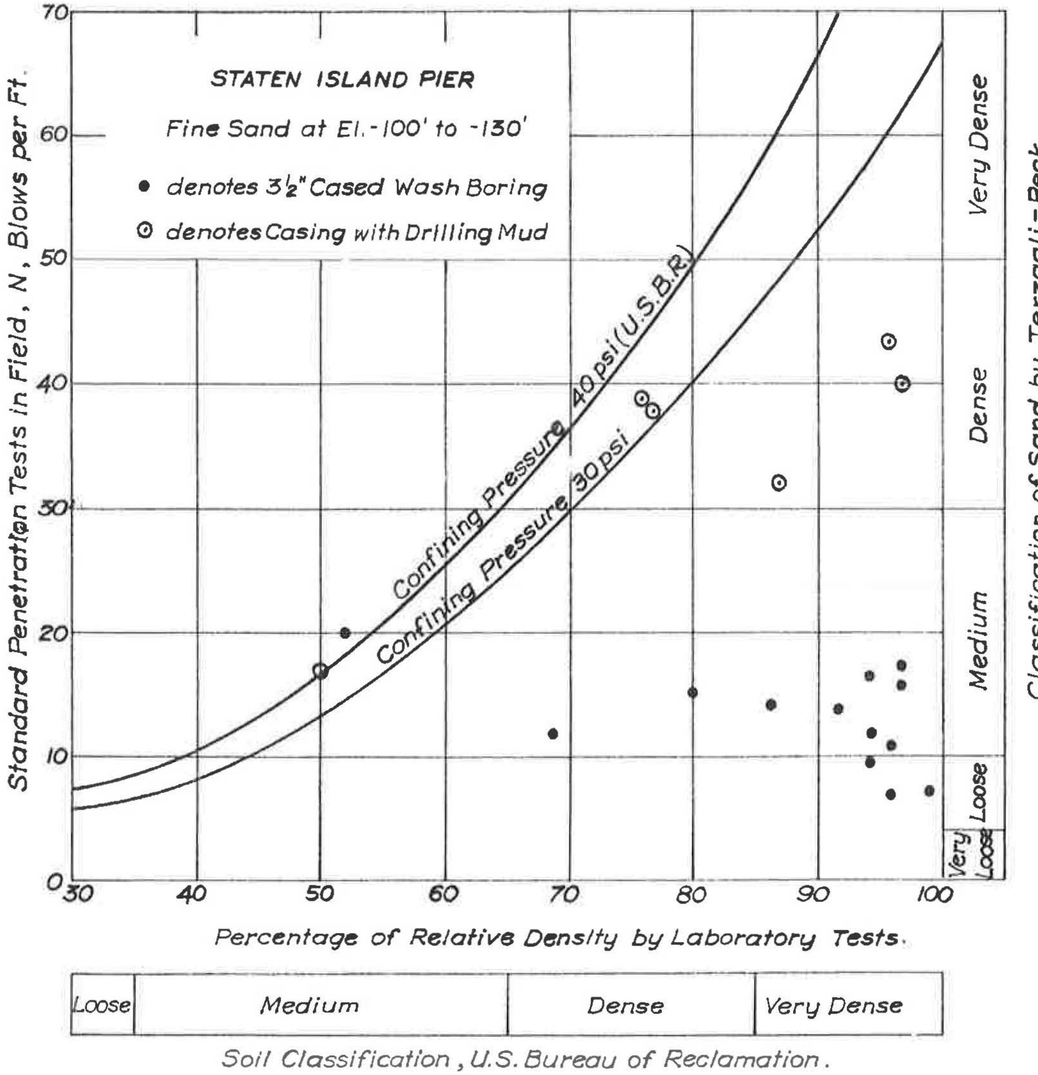

Figure 4. Relation between standard penetration and relative density.

CONSTRUCTION PROCEDURE

Sand Island

The average depth of water was 26 ft at the Staten Island pier and 20 ft at the north end of the Brooklyn pier. The sand island method was adopted for assembling the cutting edge of the caisson. The island consisted of a chain of cellular cofferdams erected around the caisson foundation. The cofferdams were filled with sand to El. +10 ft and the enclosed island to El. -12 ft. Sand fill was placed in water and no compaction was contemplated. Dewatering was done by open pumping, supplemented by a single-stage wellpoint system inside the cofferdam inclosure. A clay blanket was subsequently placed outside the cofferdam and the inclosed island was dewatered to El. -16 ft. The rate of pumping was about 5,000 gpm at the beginning and was reduced to 3,000 gpm after the installation of the clay blanket. The efficiency of the wellpoint system was not as great as described by Hoffman (5).

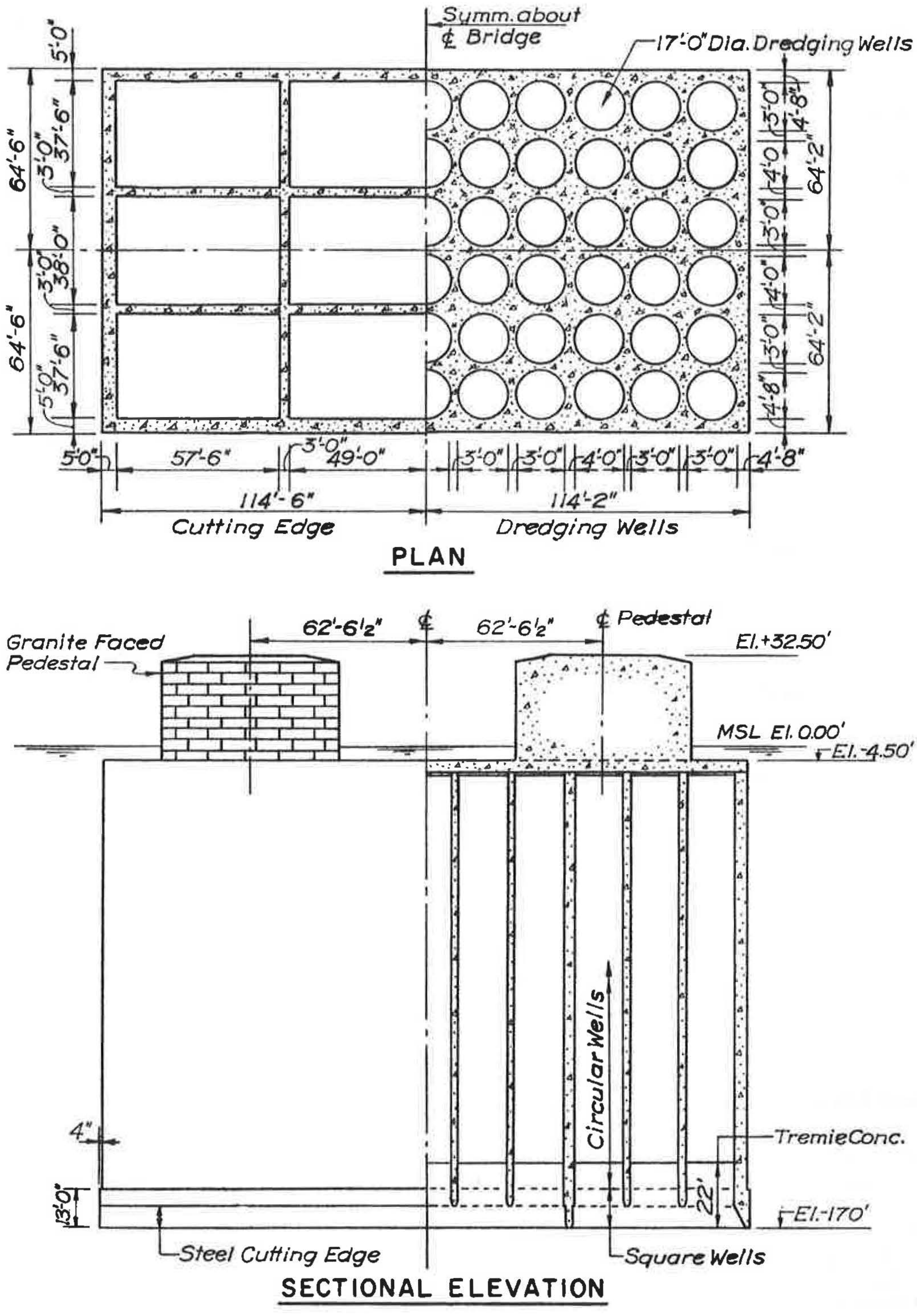

Figure 5. Detail of caisson structure—Brooklyn pier.

Cutting Edge

The steel cutting edge for both caissons consisted of a grid system of steel-plated boxes, 7 ft high and 3 ft wide for the interior walls and 5 ft wide for the peripheral walls. The purposes of the steel cutting edge were to provide a pointed edge during the sinking and to reinforce the caisson to withstand an excessive sagging stress at the early stage of construction. The steel cutting edge was assembled on timber ties. On its completion, the spaces between the walls were filled with sand. The entire assembling operation took about 5 wk. No significant settlement was observed during the period.

Caisson Buildup

The main body of the concrete caisson was progressively built up on the buried cutting edge. The overall dimension of the caisson was 129 by 229 ft, having 66 dredging wells, 17 ft in diameter (Fig. 5). Except for the first 13 ft and the final pour lift, the concrete caisson was poured in lifts of 10 ft and each lift was divided into four blocks with bulkheads at the narrowest parts of the caisson walls. Each stage of caisson buildup consisted of four lifts of concrete pour. The effective weight of the caisson was 37,000 tons at the end of the first stage buildup, and 42,000 tons were added at the completion of each subsequent stage of buildup. The first stage construction was completed in about 10 wk and the average buildup time was only 5 wk for the subsequent stages. During the caisson buildup, pouring sequence was properly controlled to avoid excessive eccentricity of the caisson weight. No significant differential settlement was observed at any stage of the concrete pours.

Open Dredge

The sinking of the caisson was accomplished by removing soil from the dredging wells. The digging was started from the center of the caisson and progressed toward the peripheral wells. The caisson tended to sag rather than to hog. The subsidence of the caisson was constantly measured by level with reference points on the cofferdam which were frequently adjusted with bench marks on shore. During the first stage of the sinking, the sagging stresses of the caisson were very critical. Vertical elevations of every intersection of dredge wells were constantly surveyed to check the stresses in the caisson walls.

REVIEW OF SINKING RECORD

Sinking of the caisson was a round-the-clock operation. Several sets of level readings were taken daily; the morning records were used in the analysis and are plotted on Figure 6. The sinking conditions of the Staten Island caisson were practically identical to those of the Brooklyn caisson; the sinking record of the latter is reviewed herein.

Rate of Sinking

The average daily rate of sinking was 2 ft for the first and second stages of sinking. The caisson could be driven as fast as the sand was removed from the dredging wells. The volume of daily excavation was about 2,500 cu yd in wet bucket measurement and the theoretical volume of caisson displacement was 2,200 cu yd. Cave-in of material was, therefore, insignificant. Where clay excavation was encountered below El. -105 ft, the efficiency of the dredging was considerably reduced because of the deep dredging and loss of material during lift-up. When lenses of fine sand and silt were encountered, additional material tended to flow into the dredging wells. Between El. -105 and -140 ft, the actual volume of caisson excavation was 1,400 cu yd/ft which was 27 percent more than the theoretical volume of caisson displacement. The average daily rate of sinking was about 1.2 ft (Fig. 7). Below El. -140 ft, the soil at the cutting edge became more silty and less able to withstand excessive pore-water pressure. Although positive waterhead was maintained inside the dredging wells to a height of 3 to 8 ft

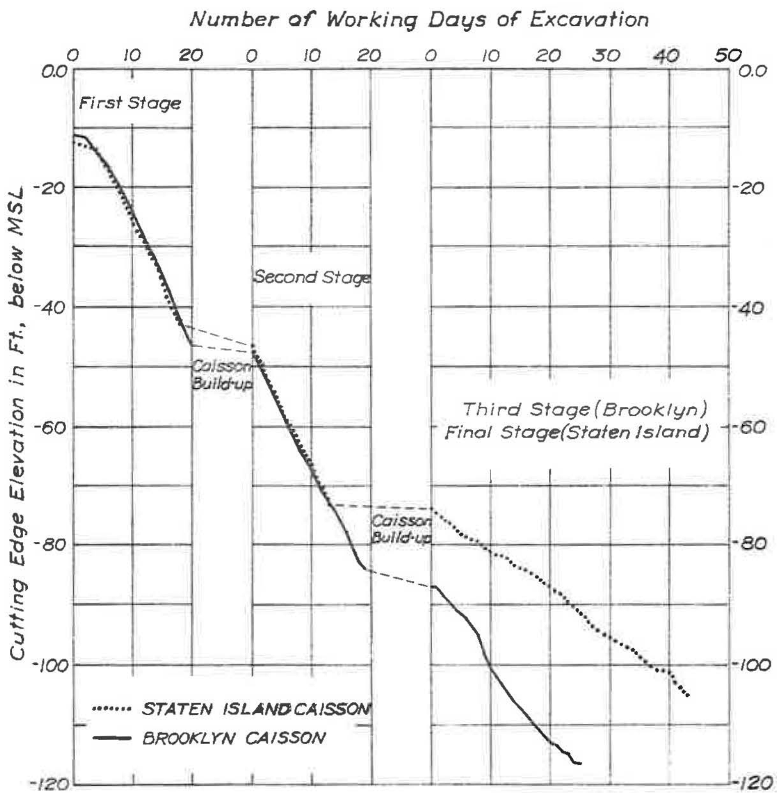

Figure 6. Daily sinking record of Brooklyn and Staten Island caissons.

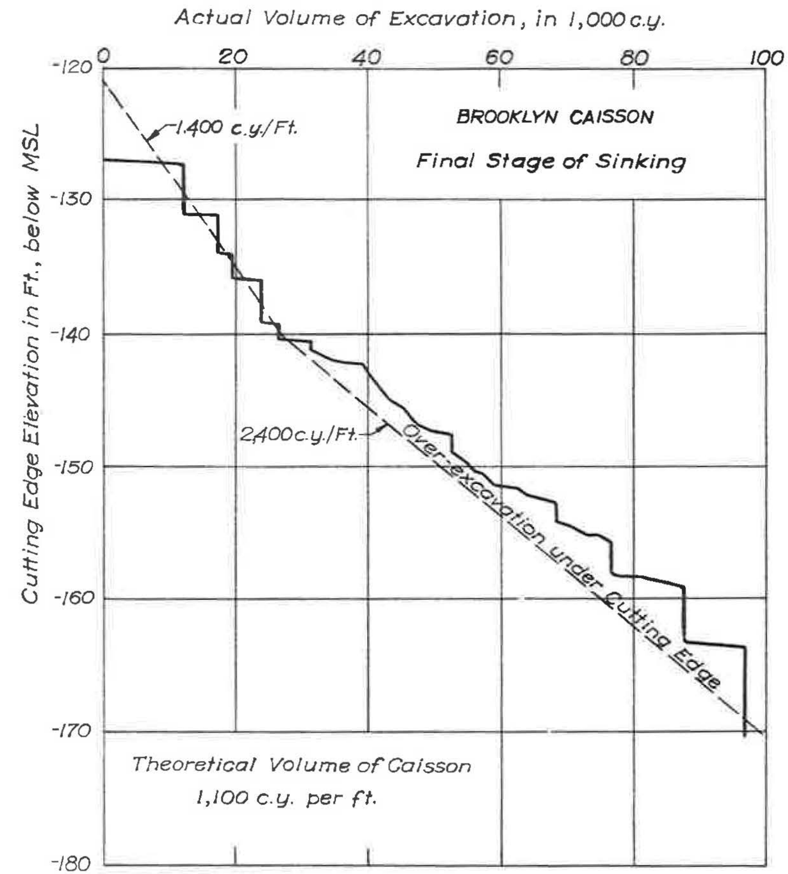

Figure 7. Mass diagram of sinking caisson.

above mean tide level, several cave-ins were noted. The magnitude of in-flows was also compounded by the undermining action below the cutting edge for reducing the sinking resistance. Between El. -140 and -170 ft, the actual volume of soil removed from the caisson was approximately 120 percent more than the theoretical caisson displacement. The average daily rate of sinking was 0.4 ft.

Resistance to Sinking

During the sinking process of a caisson, its downward movement is governed by the following factors: (a) effective weight of caisson, (b) bearing capacity of soil at cutting edge, (c) frictional resistance on caisson surface, (d) "freezing" action of soil, and (e) effectiveness of lubrication system. The first item represents the physical condition of caisson structure which can be readily determined. The last four items are closely related to the property of soils with which the caisson is in direct contact. At various sinking conditions, the contribution of each of the four factors is different and a diversified sinking resistance is therefore encountered (6, 7).

At the end of its third stage of sinking, the Brooklyn caisson had penetrated through 98 ft of sand and 15 ft of uniform clay in which the cutting edge was temporarily embedded. This was a favorable condition for studying the bearing capacity of the clay and the frictional resistance in the sand. This state of equilibrium is identified as Sequence 1 in the following discussion. At the subsequent caisson buildup, the relationship between the caisson settlement and the load increment due to the concrete pours was observed and is plotted in Figure 8. At the beginning of a new concrete pour, a certain amount of the weight was temporarily supported by the additional skin friction gained by "freezing." A new balance of soil resistance would be established until a rapid subsidence resulted. The states of equilibrium representing the conditions before and after the first rapid subsidence were identified as Sequences 2 and 3. Another rapid subsidence of the caisson was observed again at the later stage of buildup. The states of equilibrium representing the conditions before and after the second rapid subsidence were indicated as Sequences 4 and 5. During Sequences 1 through 5, there was no excavation in the caisson. The ultimate bearing capacity of the clay should not change appreciably except for a slight increase in confining pressure inside the dredging wells at Sequence 5.

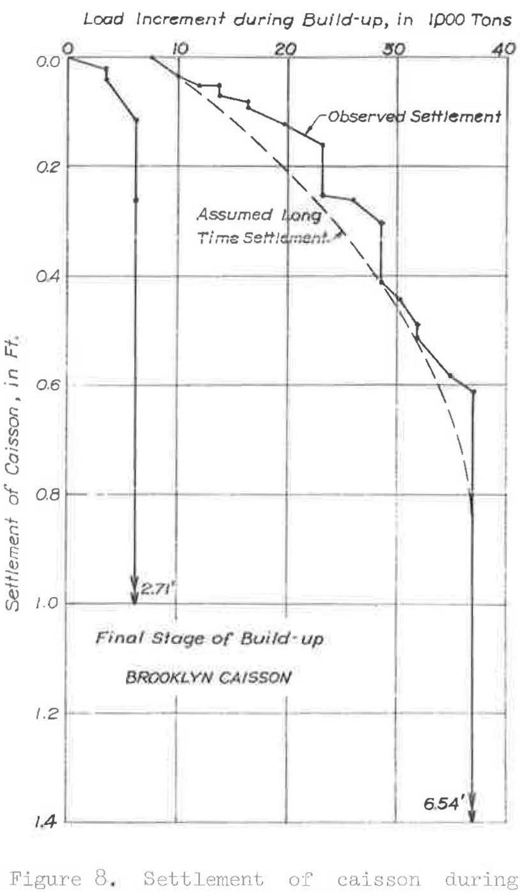

Figure 8. Settlement of caisson during final stage of buildup.

At the beginning of the final stage of sinking (Fig. 9), a rapid subsidence occurred when 14,000 cu yd of soil had been removed. The states of equilibrium were identified as Sequences 6 and 7 for the conditions at the beginning and the end of the rapid subsidence. The physical condition of these sequences is given in Table 1. The ultimate bearing capacity of a spread footing on clay can be expressed by:

$$qA = (cN_c + \gamma D_f)A \qquad (1)$$

in which c and γ are the shearing strength

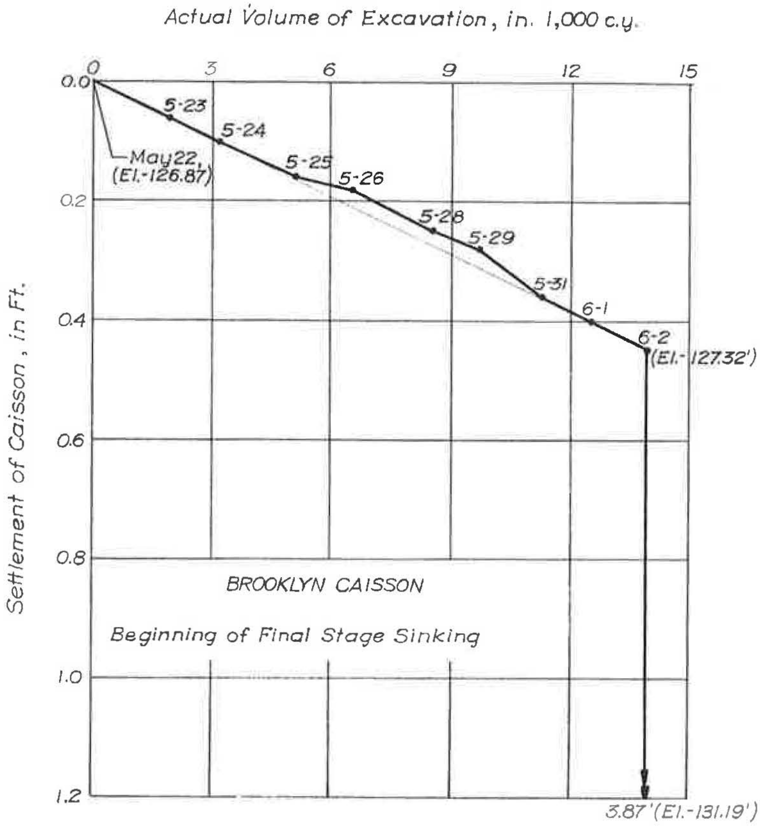

Figure 9. Settlement of caisson at beginning of sinking stage.

TABLE 1

CONDITION OF CAISSON DURING FINAL BUILDUP

Sequence	Cutting Edge Elev. (ft)	W (kips)	S (sq ft)	$\bar{P}_0$ (ksf)	A (sq ft)
1[a]	-116.64	155,220	77,460	3.79	2,010
2[b]	-116.90	168,580	77,640	3.80	2,100
3[b]	-119.61	166,210	79,570	3.88	3,010
4[c]	-120.27	229,170	80,040	3.91	3,230
5[c]	-126.81	223,450	84,710	4.12	6,640
6[d]	-127.35	227,640	85,020	4.13	4,170
7[d]	-131.19	224,210	87,800	4.26	5,470

[a]Condition on April 6 at end of third stage sinking.
[b]Taking place on April 18 at first rapid subsidence during the final buildup.
[c]Condition on May 17 at the second rapid subsidence.
[d]Initial subsidence on June 2 at beginning of final stage sinking.

and unit weight of clay soil, D_f is the depth of subcharge, A is the footing area, N_c is the bearing capacity factor of clay, and q is the unit bearing capacity. The depth of surcharge above the bottom of the cutting edge was 3 ft for Sequences 1, 2, and 7, 10 ft for Sequences 3 and 4, and 24 ft for Sequences 5 and 6. The value of γD_f is not significant in Eq. 1. Slight variations in the depth of surcharge have no appreciable effect on the analysis of the ultimate bearing capacity.

The skin friction on the caisson surface is developed when the caisson movement is sufficient to develop the shear strength of the surrounding soil mass. The intensity of the skin friction is proportional to the frictional coefficient of soil and the normal stress on the vertical surface of the caisson wall. The normal stress, in this case, is equivalent to the horizontal earth pressure. The total skin friction on the caisson surface can be expressed by the term $SK\bar{P}_o \tan\delta$, in which S is the embedded area of the caisson surface, δ is the angle of wall friction, $\bar{P}_o$ is the average overburden pressure, and K is the coefficient of earth pressure during the sinking of the caisson. As the effective weight of the caisson, W, is supported by the resistances of soil, condition of equilibrium is given by

$$W = qA + SK\bar{P}_o \tan\delta \tag{2}$$

In Table 1, the values of W, A, S and $\bar{P}_o$ were compiled for Sequences 1 through 7. The relation between q and $K \tan\delta$, in accordance with Eq. 2, is shown graphically in Figure 10.

At Sequences 1, 3, 5 and 7, the shearing strength of soil was sufficiently mobilized to resist the rapid subsidence of the caisson. The "freezing" action of the soil did not prevail. There was no lubrication system used before Sequence 7. Both Sequences 1 and 7

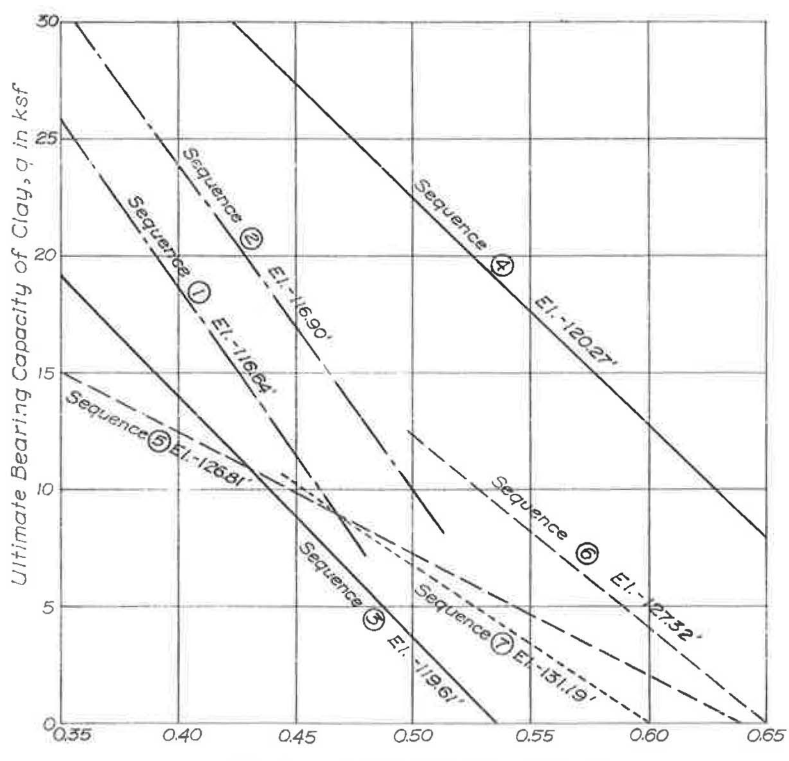

Figure 10. Graphic presentation on equilibrium of sinking caisson.

represented the condition of equilibrium immediately following a rapid subsidence. By equating the equilibrium condition for Sequences 1 and 7, it was possible to use the graphical method (Fig. 10) to determine that the unit bearing capacity would be 9.1 ksf. The corresponding bearing capacity factor of clay, N_c, would be 5.9 after adjusting the term γD_f and assuming the shearing strength of clay to be one-half of its unconfined compressive strength which had an average value of 1.5 tsf. A similar analysis would give the unit bearing capacity of 11.1 ksf and the N_c value of 6.5 for Sequences 3 and 5, another pair of identical equilibrium conditions. The theoretical values of N_c, according to Terzaghi and Peck (6), are 5.7 for continuous footing and 7.4 for individual ones. These are in close agreement with the values obtained here.

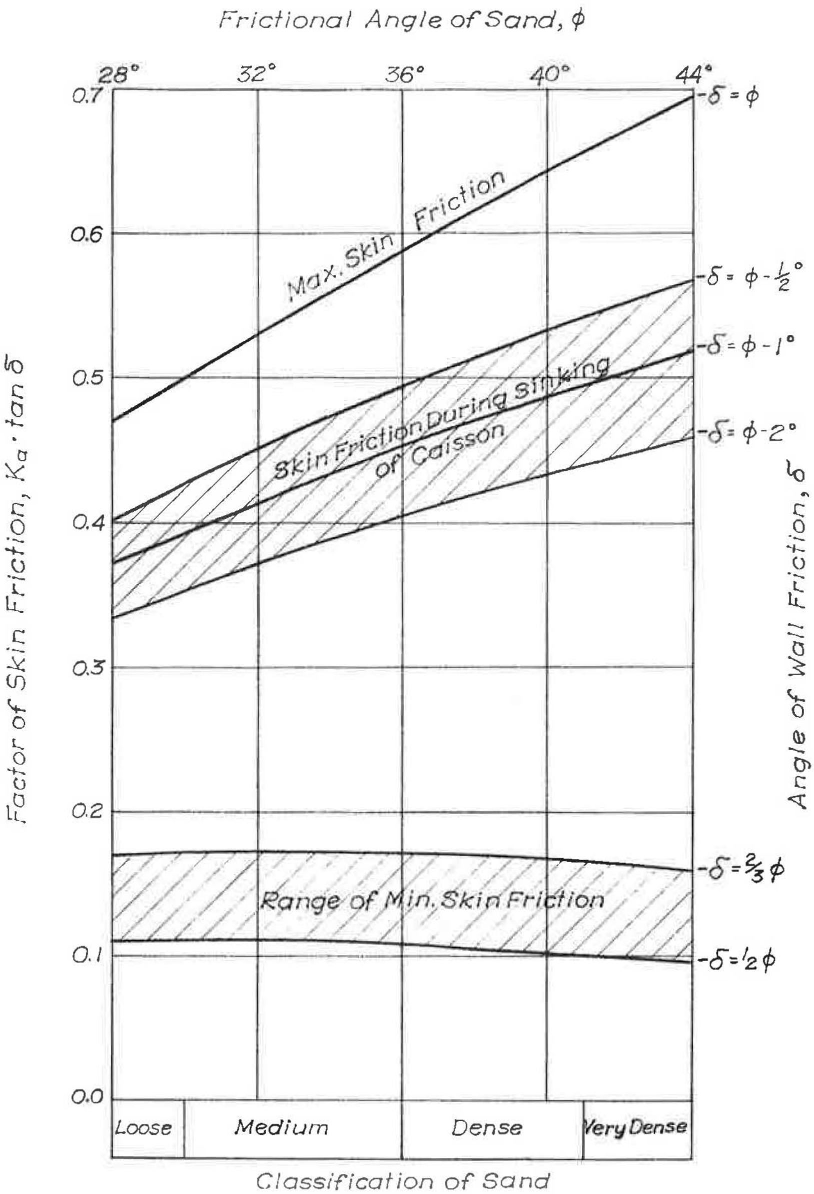

Figure 11. Variation of sinking resistance to change of wall friction.

The factor of skin friction, K $\tan\delta$, as given by the equation of equilibrium was 0.466 for Sequences 1 and 7 and 0.430 for Sequences 3 and 5. By assuming the angle of wall friction to be the same as the frictional angle of soil, estimated to be 36° for a 30-blow sand (4), the K-value would range from 0.59 to 0.64. During the sinking of a caisson, the normal stress on its vertical surface was much smaller than the overburden pressure. The sand tended to expand along its sliding surface. The increase of void ratio of sand at lower confining pressure is known in soil engineering as dilatancy. A dense sand could expand 2 or 3 percent by volume which is equivalent to a reduction of frictional angle of 1° or 2°.

In analyzing the relative movement at sinking a caisson, the surrounding soil can be considered to move upward along the caisson. The angle of wall friction would be a negative value in the general earth pressure equation derived by Coulomb (8). In Figure 11, the theoretical factor of skin friction has been computed, assuming that every part of the contacted soil mass is on the verge of failure and the normal stress on the vertical caisson surface is equivalent to the value given by Coulomb's active earth pressure. It is noted that the factor of skin friction would be reduced more than 20 percent from its maximum value when the angle of wall friction is only 1° less than the angle of soil friction. The dilatancy of sand would have a significant effect on the sinking of the caisson during Sequences 1, 3, 5 and 7.

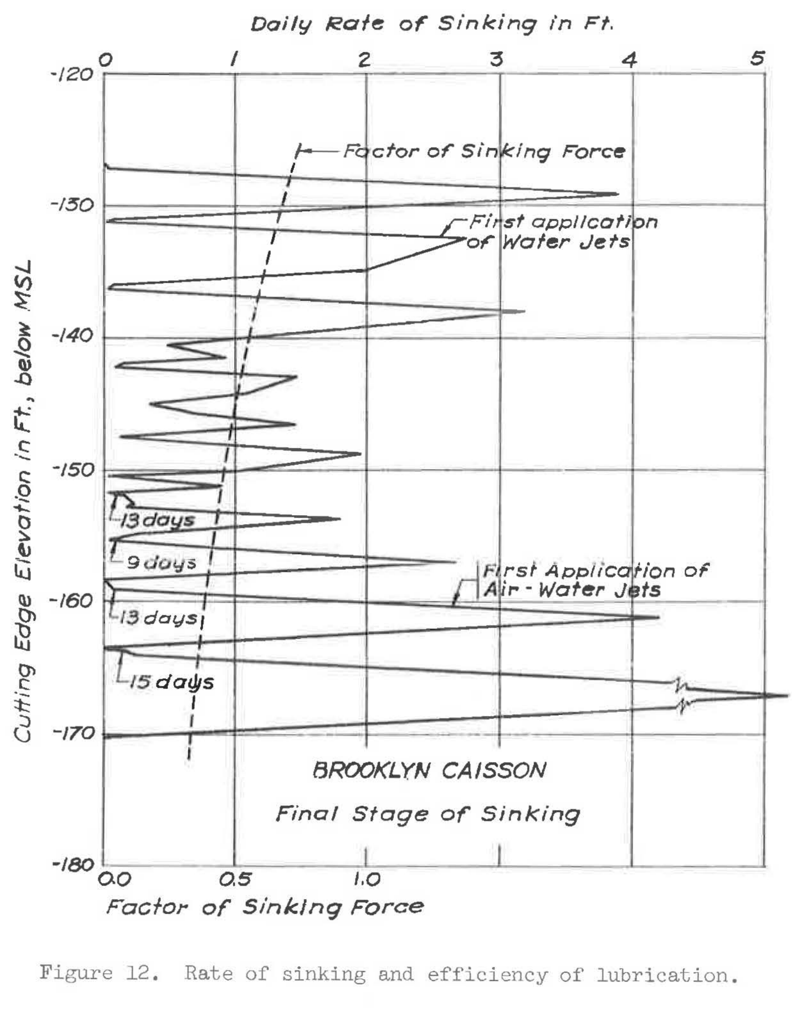

Figure 12. Rate of sinking and efficiency of lubrication.

Efficiency of Lubrication

During the sinking operation, the daily rate of subsidence was compared with the factor of sinking force, assumed to be the effective weight of the caisson per unit area of skin surface divided by the average overburden pressure. The relationship between the daily rate of sinking and the factor of sinking force was plotted on Figure 12. When the factor of sinking force was 0.65, the caisson would not move under its own weight. Water jets were introduced in the 2-in. pipe nozzles, spaced 12 to 15 ft apart and installed in the lower part of the peripheral walls. The excessive water tended to cause the expansion of sand and to reduce the skin friction. As the surface of the caisson represented the least resistance and the shortest passage for the water stream, the effect of the water jetting was confined to the surface of sliding. At cutting edge elevation of -158 ft, the bottom of the caisson was undermined to a depth more than 8 ft below the cutting edge and the factor of sinking force was 0.40. The caisson failed to subside under the influence of water jettings. A combination of air and water jetting was adopted for improving the sinking condition. The compressed air introduced into the same jetting system tended to expand and cause the breakdown of the original soil structure. In some instances, air bubbles were traced about 100 ft away from the nozzle where the compressed air was applied. Prolonged air jetting was avoided to prevent any secondary effect on the foundation. Due to the fast expansion of the compressed air, the breaking strength of the surrounding soil would be equivalent to the value of the consolidated quick shear tests which ranges from one-half to two-thirds of that of the slow shear tests. By assuming the angle of wall friction to be of the same value as that for the quick shear tests, the theoretical factor of skin friction ranges from 0.10 to 0.17 (Fig. 11). Handman (9) has observed that the actual factor of skin friction ranged from 0.12 to 0.14.

Effect of Freezing

"Freezing," in a strict sense, is not a proper engineering term. It is commonly used by field engineers to identify the condition when the resistance to driving a pile increases significantly after its installation. As observed on driving of piles (10), the increase of resistance is related to the adjustment of pore-water pressure and the thixotropic effect of fine-grained soils. The magnitude of freezing increases with increasing degree of disturbance at the pile installation and, to a lesser extent, with the duration after construction. No significant freezing effect has been observed for piles driven in soils coarser than fine sand. As the subsoil through which the caisson was installed consisted predominantly of medium sand and fine gravel, there would be no appreciable time lag of adjusting pore-water pressure. Moreover, the clay soil at the cutting edge was not disturbed and the freezing action would be negligible. However, as shown in Figure 10, the factor of skin friction for Sequences 2, 4 and 6 was appreciably higher than that for Sequences 1, 3 and 5 if the ultimate bearing capacity of clay was considered a constant. There were also indications that the skin friction increased at a rate of 1.09, 1.18 and 1.43 when the intermission between rapid subsidences was 12, 16 and 29 days, respectively (Table 1, Fig. 10).

When the movement of the caisson was temporarily ceased after the rapid subsidence at Sequences 1, 3 and 5, the sand along the caisson surface underwent a new process of reconsolidation. If sufficient time was available, the sand regained its original shearing strength. At the very beginning of the new subsidence, the angle of wall friction was of the same magnitude as the frictional angle of the soil mass. The frictional resistance against sinking of the caisson was maximum. According to Coulomb's active earth pressure, the theoretical factor of skin friction ranges from 0.60 to 0.65 for driving a caisson through dense sand. The actual sinking record of the Brooklyn caisson indicated a similar range of sinking force, i.e., from 0.65 to 0.70 at the beginning of sinking (Fig. 12). The slightly higher factor of sinking force was caused by the presence of a very small bearing support at the cutting edge.

CONCLUSIONS

1. The observed bearing capacity factor of normally loaded clay at the cutting edge is in close agreement with the theoretical value given by Terzaghi and Peck.

2. The skin friction on the caisson can be expressed by Coulomb's active earth pressure.

3. The effective weight of the caisson should always exceed the anticipated sinking resistance without the excessive use of a lubricating system. A factor of skin friction of 0.40 should be considered the practical minimum, if a built-in jetting system is installed to overcome any excessive skin friction.

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