

59-F-69

H.W.Y. # 17

BIG PIC RIVER

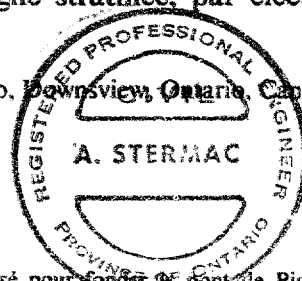
PILE LOAD TESTS.

PILE LOAD TESTS AT BIG PIC RIVER
And
The Effect of Electro Osmosis
On Pile Bearing Capacities, Hwy. #17
Marathon, District #18
W.O. 59-F-69

Capacity of Friction Piles in Varved Clay Increased by Electro-Osmosis

Augmentation de la force portante de pieux flottants, dans une argile stratifiée, par électro-osmose

by L. G. SODERMAN Principal Foundation Engineer, Department of Highways, Ontario, Downsview, Ontario, Canada
and
V. MILLIGAN Assistant Chief Engineer, Geocon Ltd, Rexdale, Ontario, Canada



Summaries

The problem of founding the Big Pic River bridge on over 300 feet of soft varved clay and loose silt deposits is described in the paper. Details of the initial site investigation and pile driving and loading tests, together with adjacent piezometric observations during driving of the test piles are presented. It was found that, due to the presence of excess hydrostatic head within coarse silt layers at depth, the capacity of long friction piles was markedly less than that of short piles within the soft varved clay stratum; consequently, it was decided to found the structure on short steel 'H' section friction piles within the upper clay and to apply electro-osmotic treatment in the area of the bridge pier and abutment pile groups.

Further tests were carried out to determine the increase in pile capacity and the variation in piezometric levels in the general area both during and after treatment. Boring and sampling, together with additional laboratory testing, was also continued during the construction period to assess the changes in soil properties.

The overall effect of the electro-osmosis was to markedly increase the pile capacity, as determined from load tests. Detailed observations of the structure are being continued.

Introduction

This paper describes the solution adopted for the substructure support at the Big Pic River Bridge, which is a three-span, through truss, cantilever structure, 600 feet in length. It is one of many along the route of the Trans-Canada Highway, which in part, skirts the north shore of Lake Superior.

Valleys in this area, of irregular volcanic and derived metamorphic Precambrian bedrock, are overlain by considerable thicknesses of stratified silts and clays deposited in post glacial Lake Algonquin. These soil conditions give rise to difficult foundation problems for embankments and structures, particularly at river crossings.

Geological description

The subsoil stratigraphy at the site was defined by means of detailed sampled borings carried to a maximum depth of 300 feet. Bedrock surface was not determined. The upper strata were sampled using a Swedish Foil Sampler (KJELLMAN, KALLSTENIUS and WAGER, 1950) to a depth of 70 feet. Below 70 feet depth, sampling was carried out using thin walled and open drive samplers.

The upper stratum, 15 feet in thickness, consists of a

Sommaire

Le problème qui s'est posé pour fonder le pont de Big Pic River au-dessus de trois cents pieds d'argile stratifiée et de dépôts de silts peu denses est décrit dans ce texte. Les détails de la première reconnaissance du site, du battage des pieux et des essais de chargement, ainsi que des observations piezométriques adjacentes pendant le battage des pieux sont présentés.

A cause de la présence d'une charge hydrostatique excessive dans les couches profondes de silts grossiers on a trouvé que la force portante des pieux longs était nettement plus faible que celles des pieux courts dans l'argile stratifiée molle. En conséquence, on a décidé de fonder la structure sur pieux métalliques flottants de section 'H' dans la couche supérieure de l'argile, et d'appliquer un traitement par l'électro-osmose à l'emplacement du pont et de l'ensemble des culées.

D'autres essais ont été faits pour déterminer l'augmentation de la force portante et la variation des niveaux d'eau dans le site général, tous deux pendant et après le traitement. Sondage, échantillonnage et essais additionnels en laboratoire, étaient encore continués pendant la période de construction pour être certains des changements des propriétés du sol.

L'effet général de l'électro-osmose a été d'augmenter d'une manière remarquable la force portante des pieux. Des observations détaillées de la structure seront continuées.

compact, fluvial silty sand. This is underlain by about 60 feet of medium to stiff, varved silty clay. The varves are composed of dark grey, brittle clay laminae, approximately 1 inch thick, and light grey, clayey silt laminae, typically 1/2 inch in thickness. The particle size distribution, determined from tests on individual laminae, is shown in Figure 1. This distribution falls within similar type curves reported by COOLING, 1959. The variations in Atterberg Limits, water content and undrained triaxial and in-situ vane shear strength with depth, are shown in Fig. 2.

The varved clay stratum grades into a grey, stratified coarse silt which becomes a silty fine sand with increasing depth. Between depths of 67 and 170 feet, the standard penetration resistance or "N" values ranged from 20 to 10 blows per foot, gradually decreasing with depth. Artesian conditions were observed on first encountering the silt stratum at 67 feet depth, elevation 546. This condition became more pronounced with depth, as reflected in the decrease in "N" values. The maximum artesian head rose to 20 feet above existing ground level at a depth of 250 feet. At this depth and below, the "N" values were sensibly zero due to piping in boreholes.

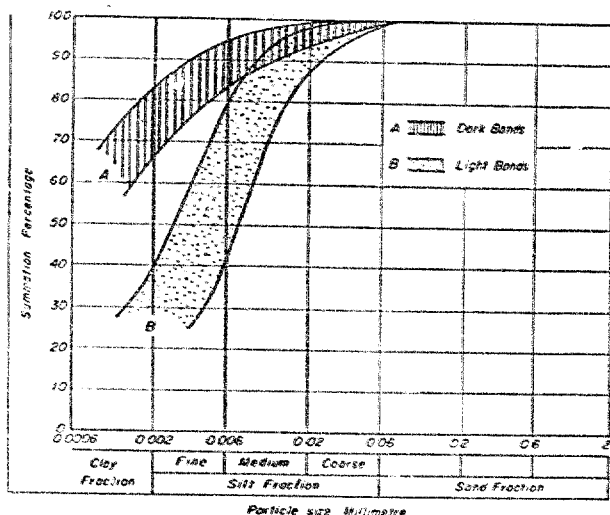


Fig. 1 Grain size distribution curves for varved clay.
Courbes granulométriques de l'argile stratifiée.

Initial Pile Tests

Due to the low strength and high compressibility of the deposits, a friction pile foundation was adopted. Initially, twelve 12-inch \times 53 lb. "H" piles, varying in embedded

length from 55 feet to 166 feet, were driven. Open tube Casagrande type piezometers, (CASAGRANDE, 1949) installed to record pore water pressure due to pile driving, were positioned at distances of 5 to 18 feet from a test pile location, with tip elevations as shown in Fig. 3.

Piles were driven with a 2-ton drop hammer, developing 32,000 ft. lbs. energy per blow at a frequency of 15 blows per minute. The driving resistance increased linearly to a value of 20 blows per foot at a depth of 166 feet.

The increase in pore water pressure above existing hydrostatic head was measured during driving of the test piles and has been plotted as a function of horizontal distance away from a pile in Fig. 3. At a distance of 16 feet away from the test pile, no excess pore water pressure was recorded. The maximum increase in pore water pressure at any elevation within the varved clay stratum due to the cumulative effect of several piles within the radius of 16 feet from a piezometer, did not exceed 9 pounds per square inch. In no instance, did driving of a pile affect the pore water pressure when the pile tip was below the piezometer tip elevation. Dissipation of 90 per cent of the excess pore water pressure occurred within 3 days after driving. These findings agree in part with observations by BJERRUM and JOHANNSEN, 1960.

Typical results of static load tests on piles of varying lengths, are summarized in Fig. 4. These tests show that static pile capacity generally decreased with an increase in embedded length. This is believed due to artesian effects at depth. Piles tested up to 400 days after driving, showed no significant increase in capacity above that measured 5 days after driving.

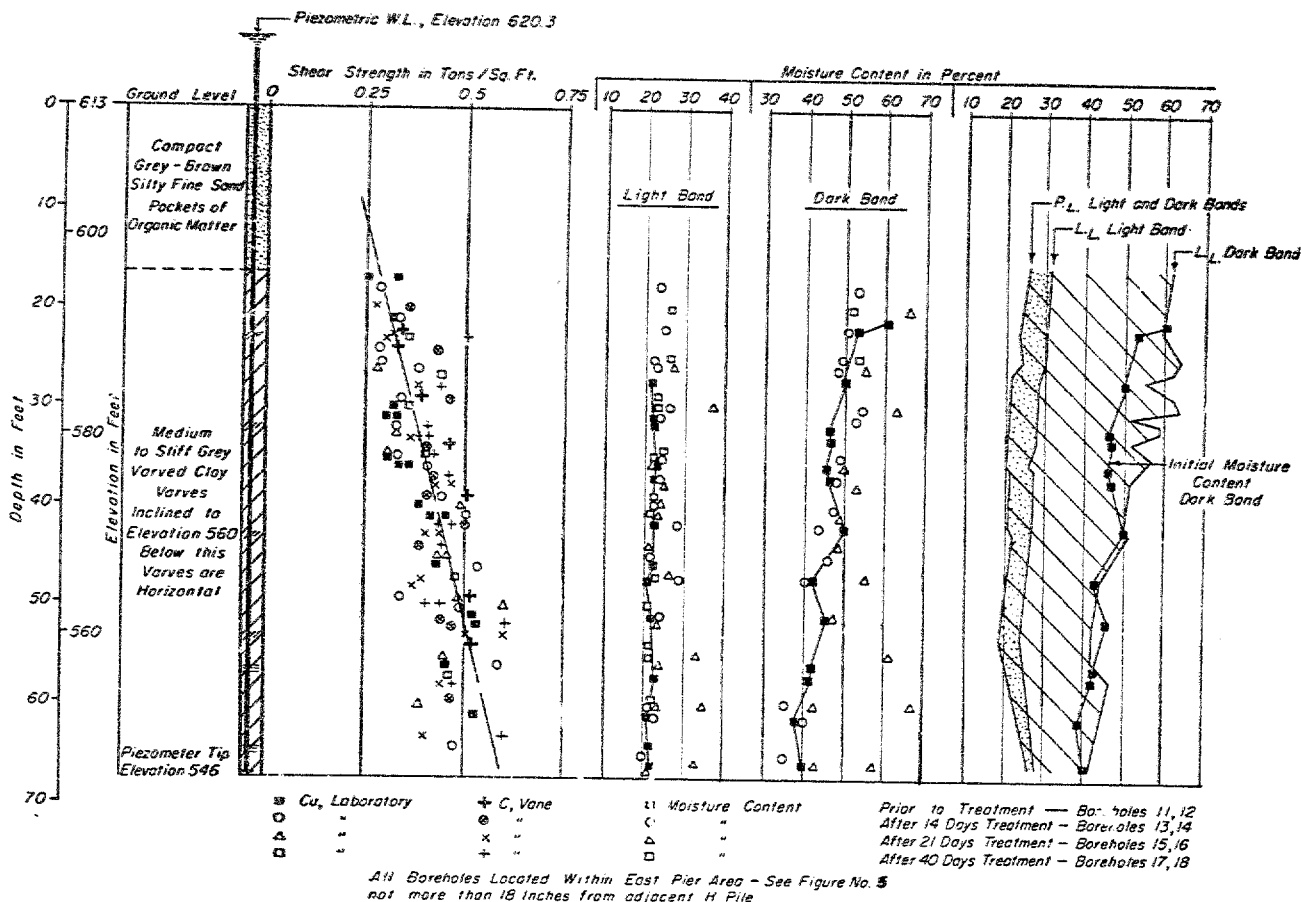


Fig. 2 Variation in water content, Atterberg Limits, and shear strength with depth.

Variation de la teneur en eau, des limites d'Atterberg, et de la résistance au cisaillement en fonction de la profondeur.

OVER

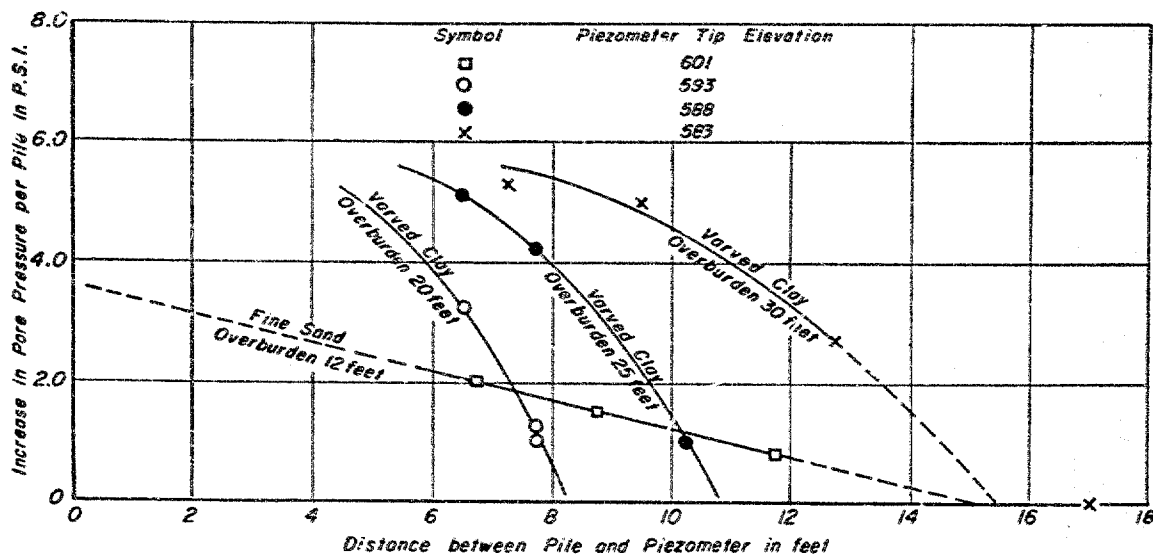


Fig. 3 Summary of pore water pressures during pile driving.
Résumé de la pression interstitielle pendant le battage des pieux.

An analysis of the elastic compression showed that the pile capacities resulted from resistance within the upper strata. To increase the capacity of friction piles founded in the upper strata, the positive terminal of an electric arc welding machine with a maximum output of 374 amperes at 115 volts was connected to a test pile and the negative terminal to an adjacent pile. After 3 hours of treatment, the pile was retested and the capacity was found to have increased from 30 tons to 60 tons. Laboratory tests were carried out by Dr. L. Casagrande on undisturbed samples of the varved clay confirming that electro-osmosis would be effective in this soil type. It was therefore decided to found the structure on H^{25} piles, 12 x 12 inches in section at 53 lb. per foot, 55 feet long. To increase pile capacity and possibly reduce the

high compressibility of the varved clay stratum electro-osmotic treatment was applied.

Electro-Osmotic Treatment

The general arrangement of perimeter cathodes, 70 feet in length, at the east pier and abutment is shown in Fig. 5. Prior to pouring the reinforced concrete pile caps, each pile was individually wired as an anode. The test piles were boxed out for future testing. A 70-120 volt, 1 000-600 ampere direct current diesel generator was used as a power source for electro-osmotic treatment. Initially, it was attempted to treat the pile group as a whole. However, apparent shielding effects took place and in order to apply sufficient amperage to each pile, it was found necessary to disconnect the exterior piles. The treatment was continued working from the interior of the group outwards. Typical current measurements on an individual pile are given in Fig. 6. The total period of treatment for each group was 1960 hours.

Control Tests

The results of static load tests carried out during treatment are summarized in Fig. 6. It was found possible to continue treatment of the pile group during pile testing by placing insulation between the test pile head and the loading jack. This prevented the test pile from becoming cathodic. The typical ultimate capacity obtained, as for test pile E-16, was in excess of 100 tons.

Sampled borings were carried out at distances of between 12 and 18 inches away from specific piles before, during and immediately after treatment. The results of detailed laboratory tests on thin walled tube samples are summarised in Fig. 2. It may be observed that no definite trend in decrease of moisture content or increase of shear strength was measured within 12 to 18 inches from a pile. The measured shear strength gave good agreement with values obtained prior to pile driving. It is also significant to note that strength values from both in-situ vane tests and laboratory compression tests on undisturbed samples, are of the same order. No visible distortion of laminae in the undisturbed samples could be detected.

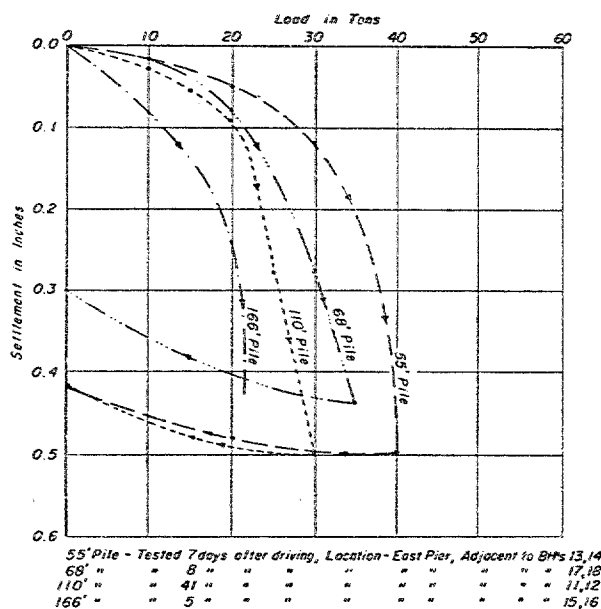


Fig. 4 Typical results of initial static load pile tests.
Résultats typiques des premiers essais de chargement statique de pieux.

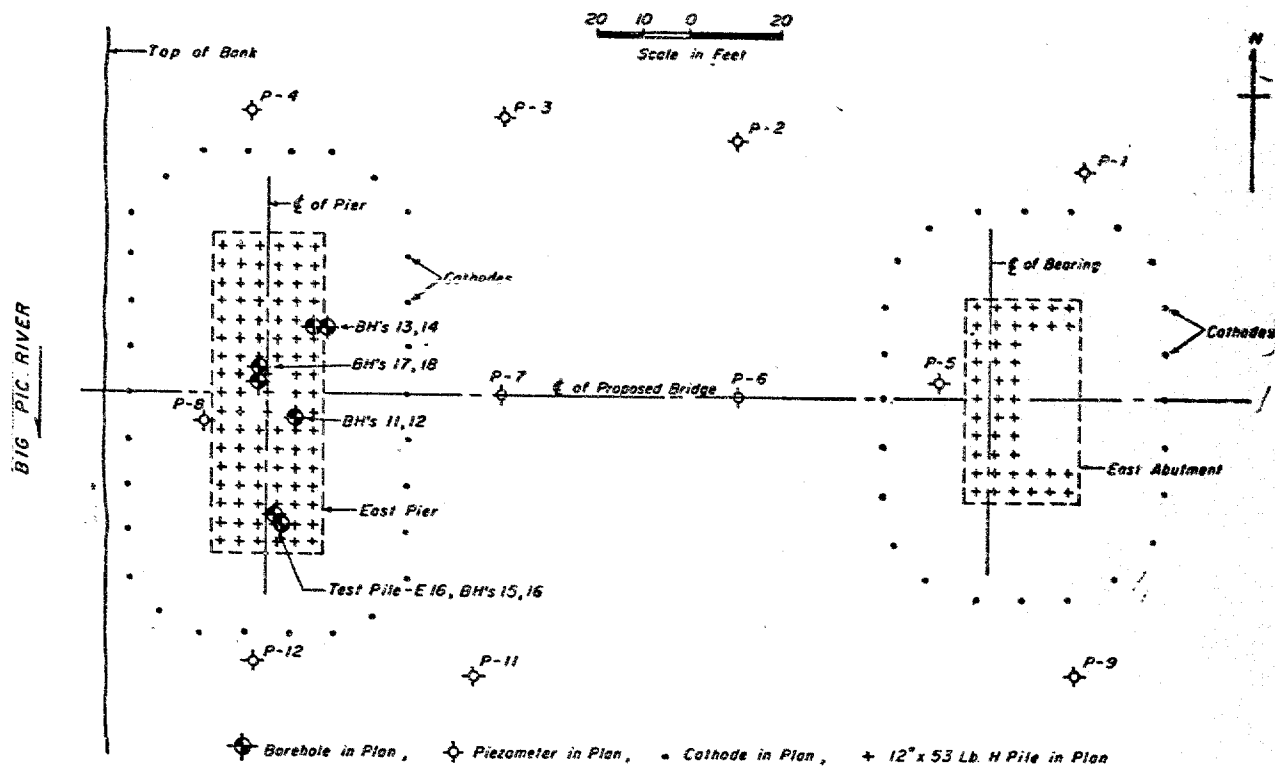


Fig. 5 Plan of East Pier and East Abutment.
Plan de la pile Est et de la culée Est.

Pore Pressure Measurement

In order to record changes in pore water pressure during and after electro-osmotic treatment, piezometers were installed both within and outside the pile group. The location of the piezometers is shown in Fig. 5. The piezometer tips were placed above the pile tips and thus above elevation 546 where artesian pressures were first observed. Prior to treatment normal ground water level was established at elevation 611 or 2 feet below general ground level.

The piezometer installed within the east pier pile group, adjacent to borehole 18 and with tip elevation 560, indicated a depressed piezometric level of 30 feet below normal water level at the end of treatment. Recovery of the piezometer to normal piezometric level was complete 90 days after treatment was stopped.

Piezometer P4, tip elevation 570 and located outside the perimeter cathodes, showed an increase in piezometric level of 7 feet above normal ground water level at the end of treatment. Recovery of the piezometer took place within 100 days after stopping treatment. This effect is also typical of the response measured in piezometers P1, P7, P9 and P12.

The variation in water levels in the remaining piezometers P2, P6 and P11 located outside the cathodes was insignificant.

Piezometer readings taken up to 400 days after completion of treatment showed no change from the normal piezometric level at elevation 611, as recorded prior to treatment. One piezometer with tip elevation below elevation 546 still indicated the artesian effect previously observed at this level.

Settlement

Settlement observations during treatment, showed that the top of the pile cap at the east pier settled 1.2 inches, while ground level below the underside of the pile cap settled 3 inches. An irregular pattern of cracking of the ground surface developed between the east pier and east abutment during treatment.

Settlement observations are being continued.

Conclusions

The effect of electro-osmotic treatment was to markedly increase pile capacity when the steel piles were used as anodes. The results of control testing indicate that the treatment affected only that soil within a distance of 1 diameter from each pile. It is further inferred that remoulding of the varved clay due to pile driving was confined to the soil within 1 diameter of the pile. Horizontal stratification of the varved clay served to accelerate the dissipation of pore pressures set up by pile driving and by electro-osmotic treatment. The permanence of the treatment with respect to pile capacity is being determined by further long term testing.

Acknowledgment

This paper is presented with the approval of Mr. W. A. Clarke, Chief Engineer, Department of Highways, Ontario. The authors are indebted to Mr. A. Prior and Dr. F. A. De Lory for their careful supervision of control testing, to Dr. L. Casagrande for his invaluable advice and to Mr. J. Alexander for his assistance in the preparation of this paper.

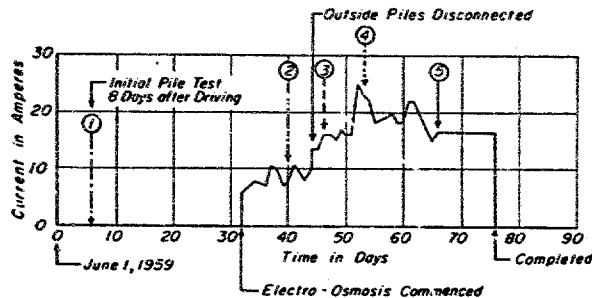
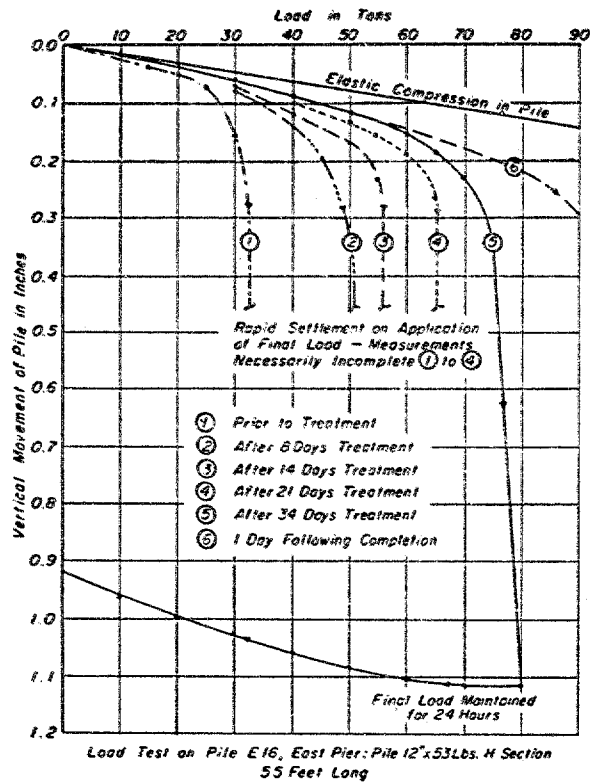


Fig. 6 Typical results of control static load pile tests.
Résultats typiques de contrôle par essais de chargement statique sur pieux.

References

- [1] BJERRUM, L. and JOHANNSEN, I. (1960). Pore Pressures resulting from driving piles in soft clay. *Pore Pressure and Suction in Soils, Conference*, London, 1960.
- [2] CASAGRANDE, A. (1949). Soil Mechanics in the Design and Construction of the Logan Airport. *Journal Boston Society of Civil Engineers* 1941-1953.
- [3] COOLING, L. F. (1959). Soil Engineering at Steep Rock Iron Mines, Ontario, Canada. *Discussion to paper by R. F. Leggc., Proceedings Institution of Civil Engineers*, vol. II, 1958.
- [4] KJELLMAN, W. KALLSTENIUS, T., and WAGER, O. (1950). Soil Sampler with Metal Foils. *Royal Swedish Geotechnical Institute, Proceedings*, No. 1.

INCREASE OF BEARING CAPACITY
OF FRICTION PILES BY ELECTRO-OSMOSIS

by

L. Casagrande¹⁾

L.G. Soderman²⁾

R.W. Loughney³⁾

Presented at the ASCE Convention
in Boston, Mass.
October 11, 1960

- 1) Consulting Engineer, Gordon McKay Visiting
Professor of the Practice of Foundation
Engineering, Harvard University
- 2) Principal Foundation Engineer, Ontario
Department of Highways, Toronto, Canada
- 3) Vice-President, Wellpoint Dewatering
Corporation, N.Y.

SYNOPSIS

The first part of this paper reviews the most important findings of past investigations concerning the strength increase of fine-grained soils achieved by means of electro-osmotic treatment, and with particular attention to possible applications for the increase of bearing capacity of friction piles. In the second part is described a full-scale application of this method in connection with the foundations for a bridge in Canada. The third part discusses the development of a novel type of friction pile which would lend itself well to application of electro-osmosis.

I. COMMENTS ON THE ELECTRO-OSMOTIC STRENGTH INCREASE OF FINE-GRAINED SOILS

When applying direct current to a pair of electrodes driven into a reasonably homogeneous mass of clay or silt, the soil surrounding the anode and frequently also surrounding the cathode will gradually increase in strength. As illustrated in Fig. 1, this strength increase progresses with time from the electrodes radially outward. The rate at which a noticeable strength increase progresses depends on the electrical potential gradient and the duration of electrical treatment. The increase in strength of these zones is due to the following phenomena:

1. Water is transported from one electrode to the other, with few exceptions the flow being from the anode toward the cathode.
2. Base exchange takes place in the soil along the path of the electric current, i.e., low valence ions, such as sodium, attached loosely by molecular forces to the surface of clay particles, are replaced by ions of higher valence such as aluminum or iron. (1) ^{*)}
3. In the pores of the soil surrounding the anode, metal derivatives are deposited (as a result of the gradual decomposition of the metallic anode) which act as cementing agents between soil particles. (1), (2)

^{*)} Numerals in parentheses refer to items in the list of references.

4. Remolded clay soils treated by electro-osmosis gradually develop a type of structure which resembles that of an undisturbed, sensitive clay. (3)
5. When free calcium is present in the soil it leads to the formation of calcium carbonate deposits in the soil surrounding the cathode.

These phenomena take place simultaneously. Usually the first three of the phenomena listed above are dominant in clay soils. However, even if the water content is not reduced, such as may happen in stratified soils with high permeability parallel to stratification (e.g. in some varved clays) which have a tendency of partially or fully replenishing pore water which is being removed by electro-osmosis, base exchange and cementation by metallic deposits will still be effective and cause an increase in strength.

Since the time, thirty years ago, when these basic observations were made by the senior author, a number of investigators have conducted laboratory and field tests in order to study the feasibility of increasing the bearing capacity of friction piles by electro-osmosis. The results of several of these investigations were published, (4) to (9), while other important work was compiled in the form of reports and has not yet been made available to the profession. In the majority of these investigations the anodic piles consisted of aluminum because of the early discovery that the aluminates

deposited in the soil adjacent to the anode are more effective in cementing the soil particles than the derivatives of iron anodes. (2)

In the photograph in Fig. 2 is shown the effect of electro-osmosis on a 3-inch diameter steel pipe, serving as anode, driven to a depth of approximately twelve feet into a man-made fill of soft, very sandy clay, mixed with calcium carbonate wastes.

About 10 ft to the right of the three-inch steel pipe was driven a 1.5-inch diameter steel pipe of equal length to serve as cathode. DC current with a potential of approximately 70 volts was applied for a period of 10 days. At the beginning 16.4 amps passed through these two electrodes and the current dropped gradually to less than 1.5 amps after 10 days. Attempts to increase the conductivity by adding sodium chloride or other salts to the soil near the anode were effective only for a duration of several hours after which the resistivity of this system increased again to the value established prior to the addition of salts.

The anodic pipe was load-tested after it had been in place for a period of 6 hours, but prior to application of electro-osmosis. The ultimate bearing capacity was 177 pounds which included the weight of the 3-inch pipe. At this load the settlement progressed steadily and had reached 3.3 inches when the load test was discontinued.

After 10 days of electrical treatment the load test was repeated and failure occurred rather suddenly after the load had been increased to 3470 pounds and the total settlement had reached 1.2 inches.

After completion of this load test the anodic pipe was pulled. As can be seen in Fig. 2 a cemented clay cylinder of about 10 inches in diameter had formed around the pipe. The upper portion of this cylindrical body of soil was removed by means of hammer blows and subjected to laboratory testing.

II. PRACTICAL APPLICATION FOR INCREASING THE BEARING CAPACITY OF FRICTION PILES

Despite the fact that all investigations which have come to the authors' attention proved the feasibility of substantially increasing the bearing capacity of friction piles by means of electro-osmosis at reasonable cost, this method encountered barely more than theoretical interest amongst practicing engineers. This is primarily due to the fact that the substantial increase in bearing capacity of friction piles resulting from electro-osmosis does not eliminate the problem of settlements due to consolidation of compressible strata beneath the pile points.

The first application of this method was completed about one year ago in connection with the foundations of a bridge over the Big Pic River near Marathon, Ontario, for the Trans-Canada Highway. (10) As illustrated in Fig. 3, the foundation soils at this location consist of a few feet

of sand and gravel fill followed by a 10 to 20 foot stratum of fluvial deposits of silty clay and fine sand which are underlain by 40 to 60 feet of varved clay of soft to medium strength. The varved clay, consisting of alternating layers of clay and silt, is underlain by a very thick deposit of rock flour which at the depth of approximately 250 feet changes gradually into silty fine sand. The borings were stopped at a depth of 300 feet without reaching bedrock. In the silt and fine sand strata artesian pressure was encountered.

Because of the excessive depth to bedrock and the sensitive character of the varved clay, the original design called for the bridge footings to be founded on 110-foot long, 12-inch, 53-pound steel H-piles driven as friction piles through the clay into the underlying silt stratum, and using a design load of 40 tons per pile. Several load tests on such piles gave ultimate bearing capacities ranging as low as 20 tons. Repetitions of load tests after the piles had been in place for a period of over one year did not show any increase in bearing capacity.

In order to avoid a radical change in the design of this bridge at this late stage, the Ontario Department of Highways decided to attempt to increase the frictional resistance of the piles by applying electro-osmosis. A preliminary field test was arranged utilizing two 56-foot long test piles as electrodes. With a potential of

115 volts the bearing capacity of the anodic pile, which prior to treatment had carried an ultimate load of barely 30 tons, showed an increase to approximately 60 tons after three hours of treatment. Subsequent laboratory tests indicated that with longer duration of treatment even better results could be anticipated. On the basis of these favorable results it was decided to use 56-foot long piles which would not extend into the silt stratum beneath the varved clay.

In Fig. 4 is shown the electrical installation for the West Pier, including the arrangement of the cathodes relative to the H-piles, to be utilized as anodes. The average distance between the electrodes was slightly over 23 feet. The layout for the electrical treatment at the East Pier was similar to that at the West Pier as can be seen in the photograph in Fig. 5. In order to prevent clogging of the cathodic pipes with calcium carbonate, a combination of steel pipes and plastic pipes was used as shown in the photograph in Fig. 6. Numerous small holes were drilled into the plastic pipe to allow the water (carried by electro-osmosis toward the cathode) to penetrate the plastic pipe and to discharge on the surface. Three diesel generators with an output of 70 to 120 volts and 1000 to 600 amps per unit were used. The average current consumption per H-pile for a potential of 100 volts amounted to 15 amps.

A number of H-piles in the foundations for the piers and one abutment had been boxed out in order to enable performance of load tests during and after electrical treatment. In this manner it was possible to follow the progress of the increase in bearing capacity with the duration of treatment. The results for a typical test pile are plotted in Fig. 7 showing an increase in ultimate bearing capacity from 30 tons before electrical treatment to 100 tons after a treatment lasting four weeks. On the basis of such tests it was possible to decide on the necessary duration of electro-osmotic treatment. While the H-piles closest to the cathodes were treated for approximately two weeks, the piles located farthest from the cathodes had to be treated for a period of approximately 6 weeks.

The total cost of this electro-osmotic treatment, performed by the Wellpoint Dewatering Corporation of New York, was approximately \$55,000, i.e. a fraction of the savings which had accrued due to the use of much shorter piles than the original design had called for.

III. DEVELOPMENT OF A NEW TYPE OF FRICTION PILE

In an attempt to develop a friction pile which would combine the qualifications of causing minimum disturbance to sensitive soils during driving with a maximum benefit from electro-osmotic treatment, the senior author developed a skeleton-type pile. As shown in the photograph in Fig. 8, it consists of a series of rods or pipes which are

welded to spacers made of short sections of pipe. The appreciable reduction in displacement of soil, and therefore in disturbance caused by the driving of such a skeleton pile as compared to conventional piles, is illustrated by the model tests shown in the photograph in Fig. 9.

A semi-cylinder, which is closed in front with a lucite wall, was filled with bentonite clay of very soft consistency. Embedded in this clay were thin layers of brown, fine sand, spaced approximately 2 inches. On the left side of the photograph is shown the effect of pushing into the clay a full-displacement pile. It consists of a thin-wall aluminum tube which was cut in half lengthwise and filled with plaster-of-paris. The other half of this aluminum tube was pushedⁱⁿ as an "open-ended pipe pile", and is shown in the center of the photograph. On the right side a half-section of a skeleton pile was pushed in. During pushing of the full-displacement and open-end piles the surface of the clay heaved and cracked, and below the bottom ends of these piles the clay mass deformed to a depth of several pile diameters. Inside the open pipe the clay had risen to less than one-half of the length of the pile, when the friction between the inside of the tube and the clay plug had increased to a magnitude which caused the pile to be driven further as a full displacement pile. In contrast, the pushing of the skeleton pile into the clay (which was done after the installation of the other two piles) created hardly noticeable disturbance of the clay,

and the surface of the clay plug inside the skeleton was practically at the same elevation as the clay on the outside. This skeleton pile was pushed to the same depth as the other two piles. Nevertheless, below its tip no additional deflection of the sand layers was created.

All three piles were 1.5 inches in diameter. The skeleton pile consisted of six 1/8-inch diameter rods. It was cut lengthwise so that two half-rods were in contact with the lucite plate. These rods were held in place by four thin steel collars 1/4 inches high. The force necessary to push the skeleton pile down was one-fifth of the force required for the open-ended pipe, and one-sixth of the force required to push the solid pipe down to the same depth.

For the sake of interest, buckling tests were made on single 1/8-inch diameter rods (of which the skeleton pile was built) and on the entire skeleton pile. It was found that the buckling strength, tested without confinement in clay, was for the skeleton pile 8.4 times greater than the combined buckling strength of six individual rods of equal length.

While the surface area of such a skeleton pile is only a fraction of a conventional pile of similar diameter, it lends itself well to an increase in bearing capacity by electro-osmotic treatment. As shown in Fig. 10, the cylindrical bodies of cemented soil surrounding each rod or pipe of this pile will gradually meet and form one unit which

contribute to the bearing capacity of the pile. Model tests have shown that already before these hardened cylinders (indicated by dashed line in Fig. 10) grow into one unit, the larger circumferential surface area indicated by a full line in Fig. 10 will govern its bearing capacity.

The effect of electro-osmosis on such a skeleton pile in a model test in Boston Blue Clay is shown in Fig. 11. The 1.5-inch diameter skeleton pile was pushed into a uniform mass of remolded clay having an average water content of 43.8 per cent. The liquid limit of this clay was 47.6 and the plastic limit 23.3. The clay was tilled into a plastic bucket. The skeleton pile in the center of the clay mass served as anode and two 1/8-inch diameter rods were arranged on two opposite sides of the anode, next to the wall of the bucket, to serve as cathodes. A potential of 6 volts was supplied by a storage battery for a duration of approximately 2 weeks. A number of load tests were made before and during treatment and the results, expressed in unit frictional resistance, are recorded in the diagram in Fig. 12.

After the last load test was completed, electro-osmosis was applied for several additional hours. Then an attempt was made to pull the skeleton pile from the body of clay. This resulted in the full content of the bucket to be pulled up with the pile. Although it may be startling to see a weight of 40 pounds of clay hanging on a few thin rods, this result is not unexpected if compared with the bearing

capacities recorded in Fig. 12. The same block opened up along a vertical plane is shown in Fig. 13. It can be seen that the clay has developed a structure with distinct conchoidal stratification and fissuring. Along the outer zone the water content of the clay was still near the liquid limit, but it developed substantial strength and a structure which was quite sensitive to remolding.

IV. CONCLUSIONS

From laboratory and field investigations, including one full-scale application, it may be concluded that (1) the bearing capacity of friction piles can be increased greatly and economically by the use of electro-osmosis, and (2) the ideal type of friction pile for this purpose would be a skeleton pile of the type discussed above and shown in Fig. 8.

REFERENCES

- (1) K. Endell, Beitrag zur Chemischen Erforschung und Behandlung von Tonböden, Bautechnik, Vol. 13, Berlin, 1935.
- (2) L. Casagrande, Die Elektrochemische Bodenverfestigung, Bautechnik, Vol. 17, Berlin, 1935.
- (3) L. Casagrande, Structures Produced in Clays by Electric Potentials and Their Relation to Natural Structures, Nature, Vol. 160, p. 470, 1947.
- (4) L. Erlenbach, Anwendung der Elektrochemischen Verfestigung auf Schwimmende Pfahlgründungen, Bautechnik, Vol. 14, pp. 257-259, Berlin, 1936.
- (5) L. Casagrande, Grossversuch zur Erhöhung der Tragfähigkeit von Schwebenden Pfahlgründungen durch Elektrochemische Behandlung, Bautechnik, Vol. 17, pp. 228-230, Berlin, 1939.
- (6) E. Kumutat, Über die Elektrochemische Bodenverfestigung nach dem Verfahren von L. Casagrande, Angewandte Chemie, Vol. 53, No. 15-16, 1940.
- (7) L. Bendel, Ergebnisse von Belastungsproben bei Pfahlfundationen, Schweiz. Techn. Zeitschrift, p. 41, Zürich, 1941.
- (8) M.G. Spangler and H.L. King, Electrical Hardening of Clays Adjacent to Aluminum Friction Piles, Proc. Highway Research Board, Vol. 29, pp. 589-599, 1949.
- (9) A.B. Bridgwater, Electro-Kinetic Phenomena in Soils, Civil Engineering and Public Works Review, Vol. 45, 1950.

REFERENCES (Cont.)

- (10) L.G. Soderman and V. Milligan, Capacity of Friction Piles in Varved Clay Increased by Electro-Osmosis. Paper submitted to the Fifth International Conference on Soil Mechanics and Foundation Engineering, Paris, 1961.

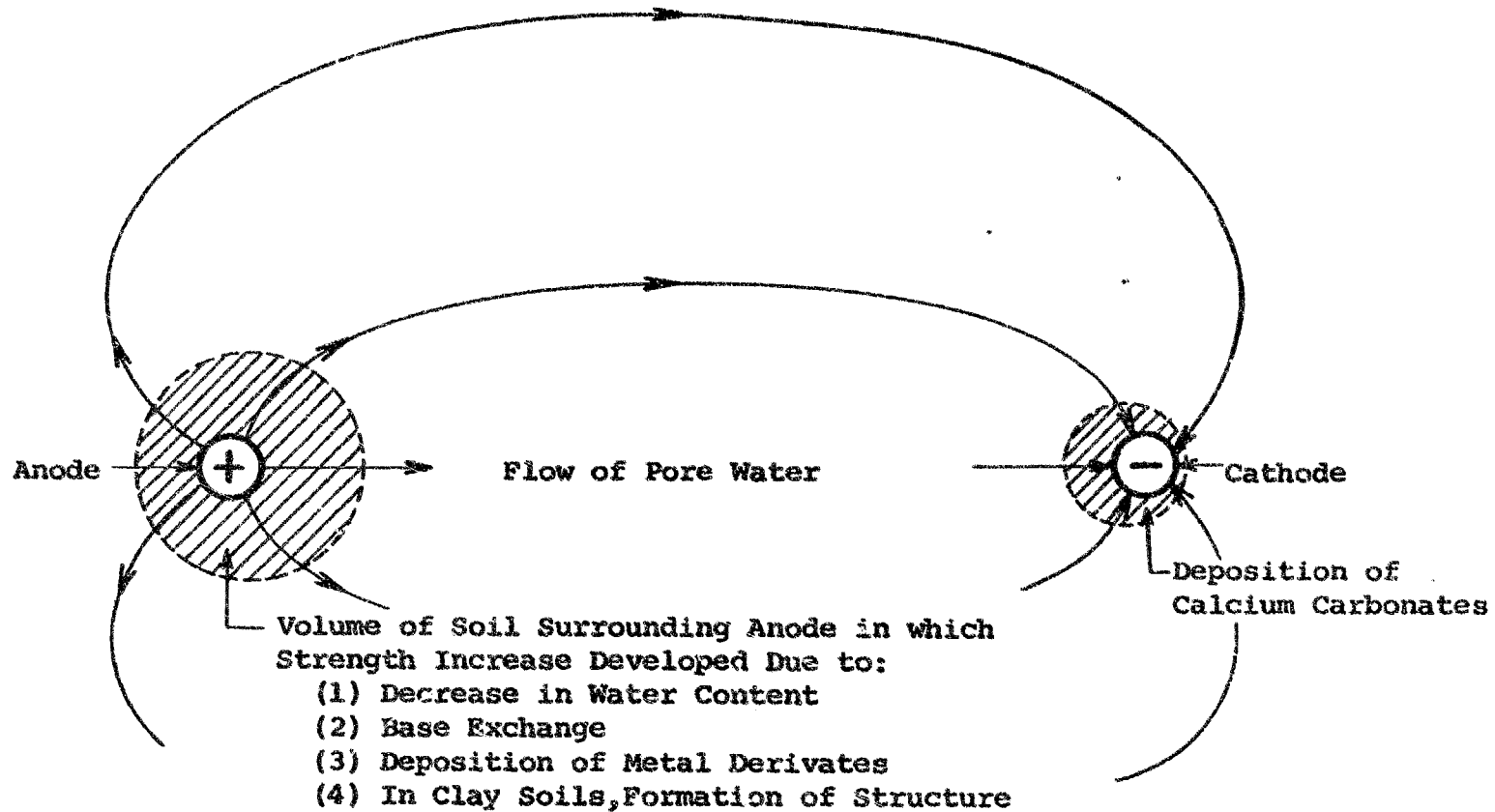


FIG.1.- PROGRESS OF STRENGTH INCREASE OF SOIL SURROUNDING ELECTRODES



FIG.2.- CEMENTED CLAY CYLINDER FORMED AROUND ANODIC PIPE
AFTER TEN DAYS OF TREATMENT

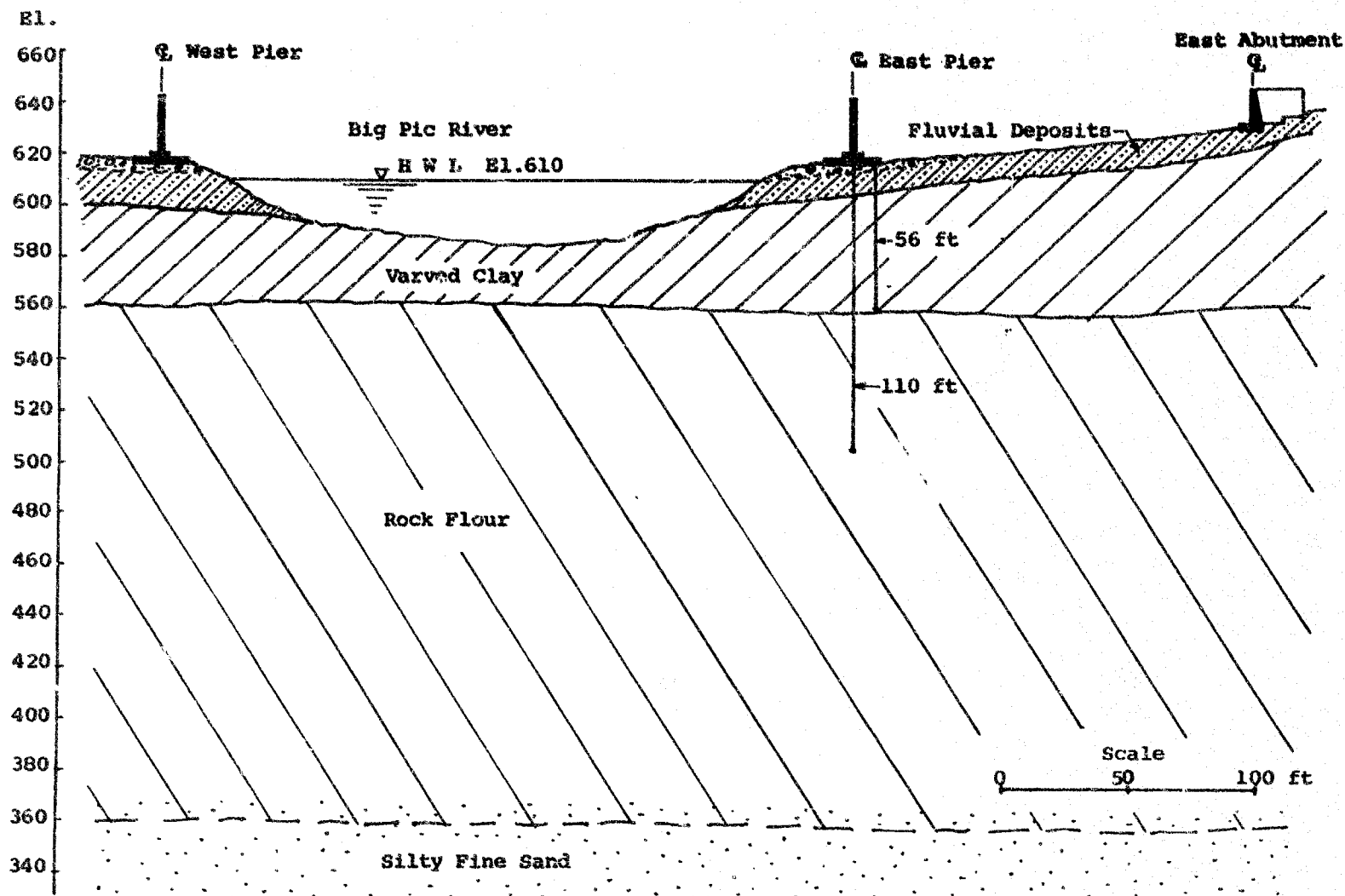


FIG.3.- SUBSOIL PROFILE AT BIG PIC RIVER BRIDGE, MARATHON, ONTARIO

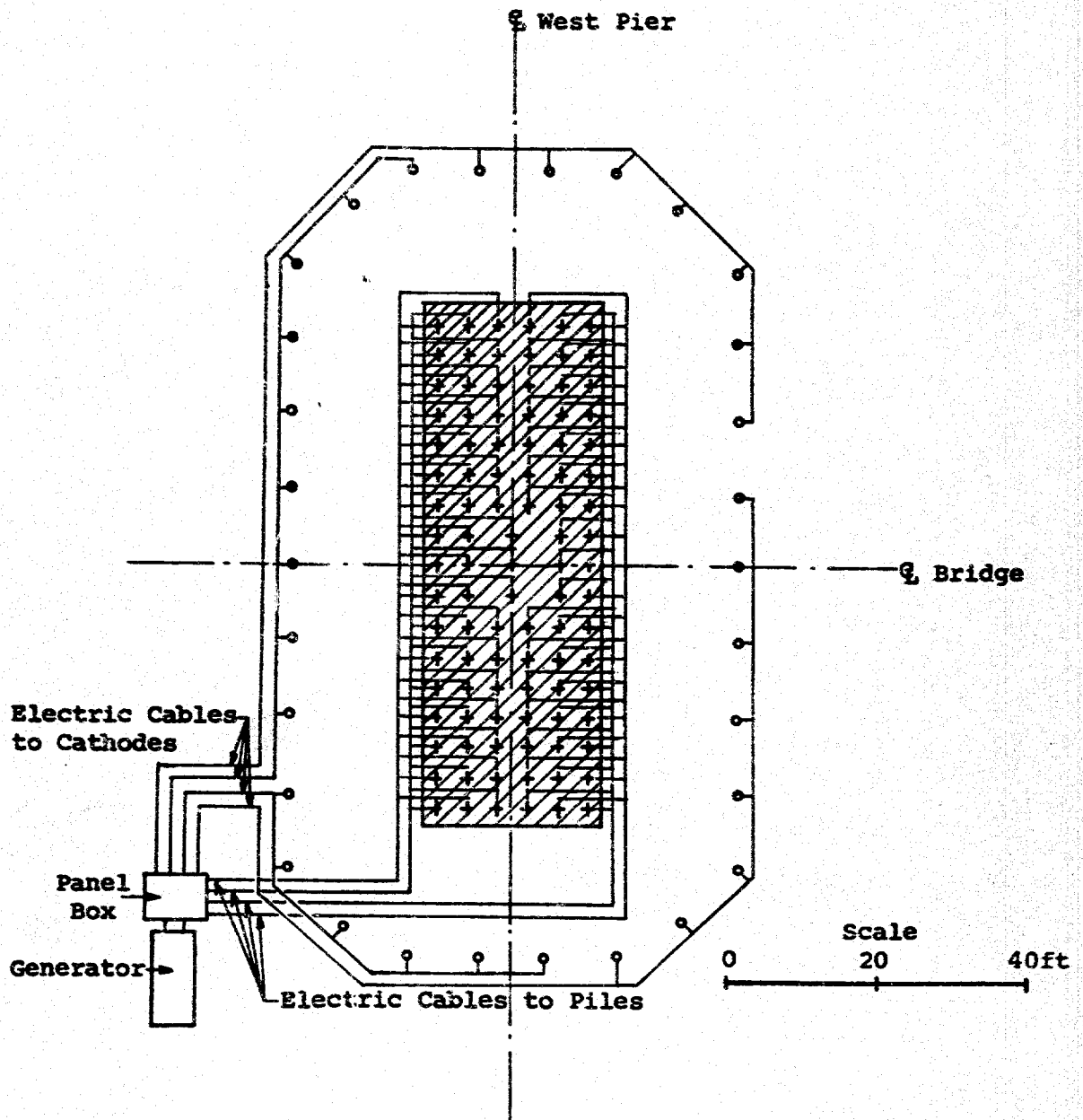


FIG.4.- ELECTRICAL INSTALLATION AT WEST PIER OF BIG PIC RIVER BRIDGE,
MARATHON, ONTARIO

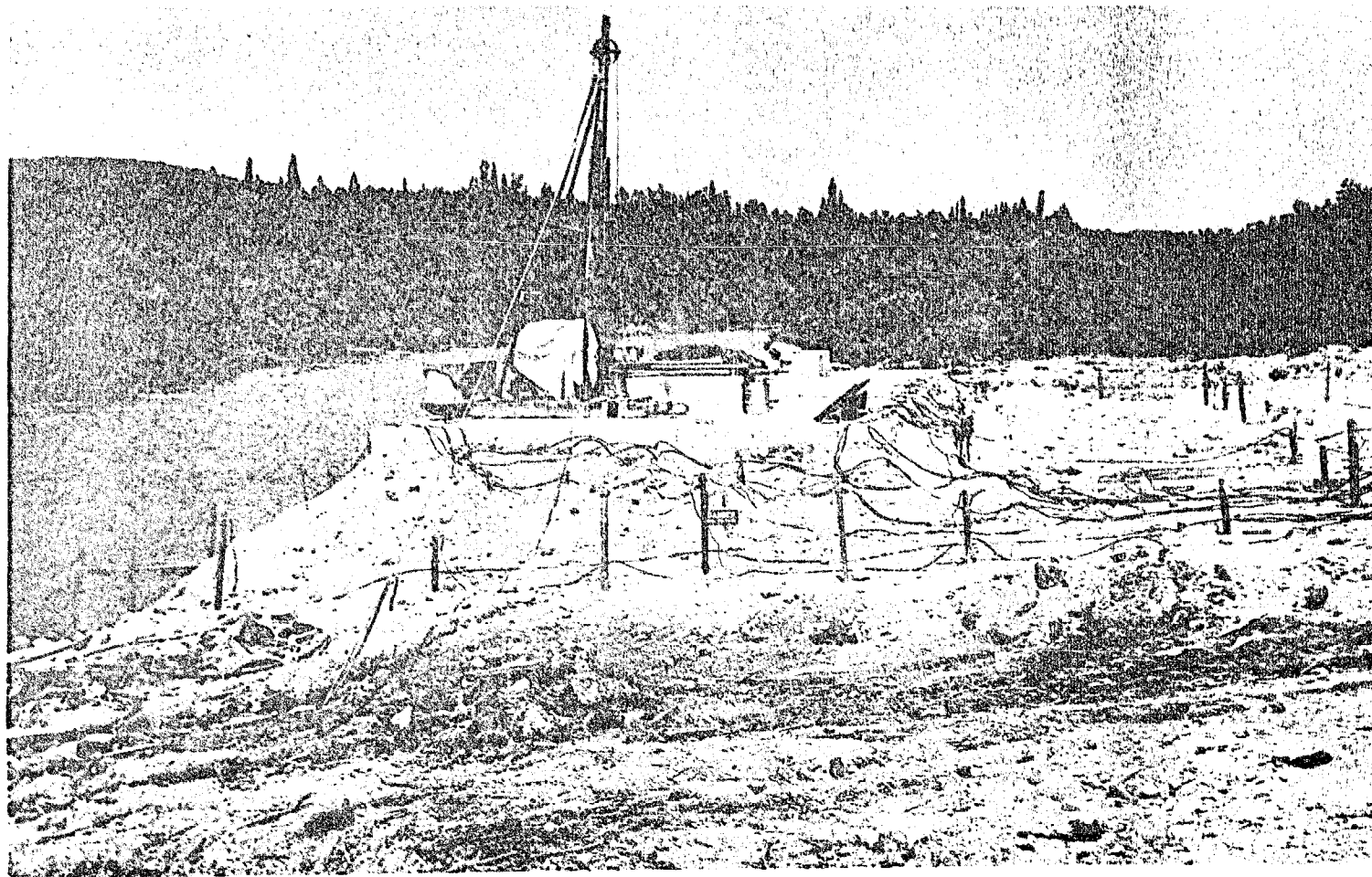


FIG.5.- VIEW OF ELECTRICAL INSTALLATION AT EAST PIER, BIG PIC RIVER BRIDGE,
MARATHON, ONTARIO

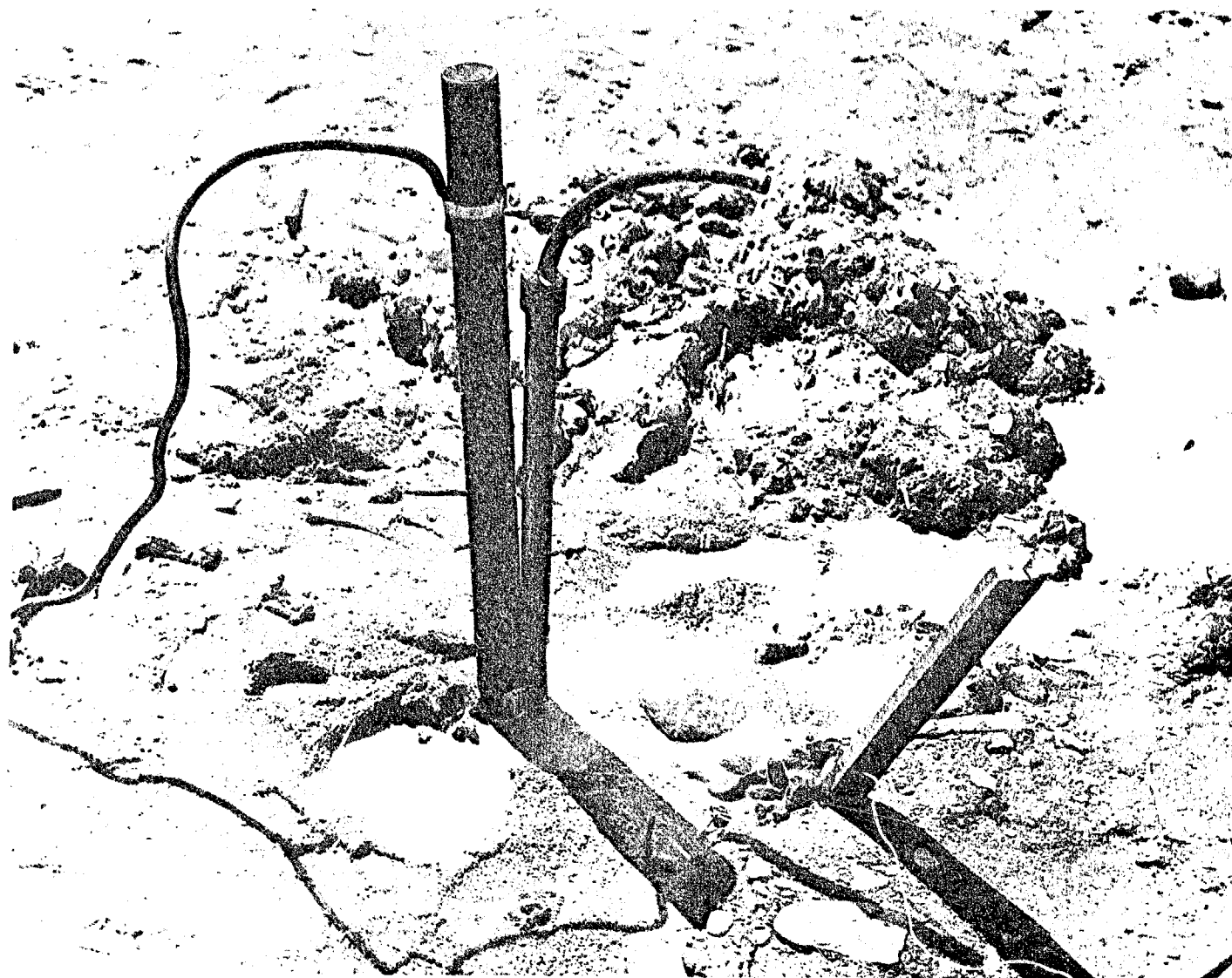


FIG.6.- DRAINAGE FROM CATHODIC PIPE AT BIG PIC RIVER BRIDGE, MARATHON, ONTARIO

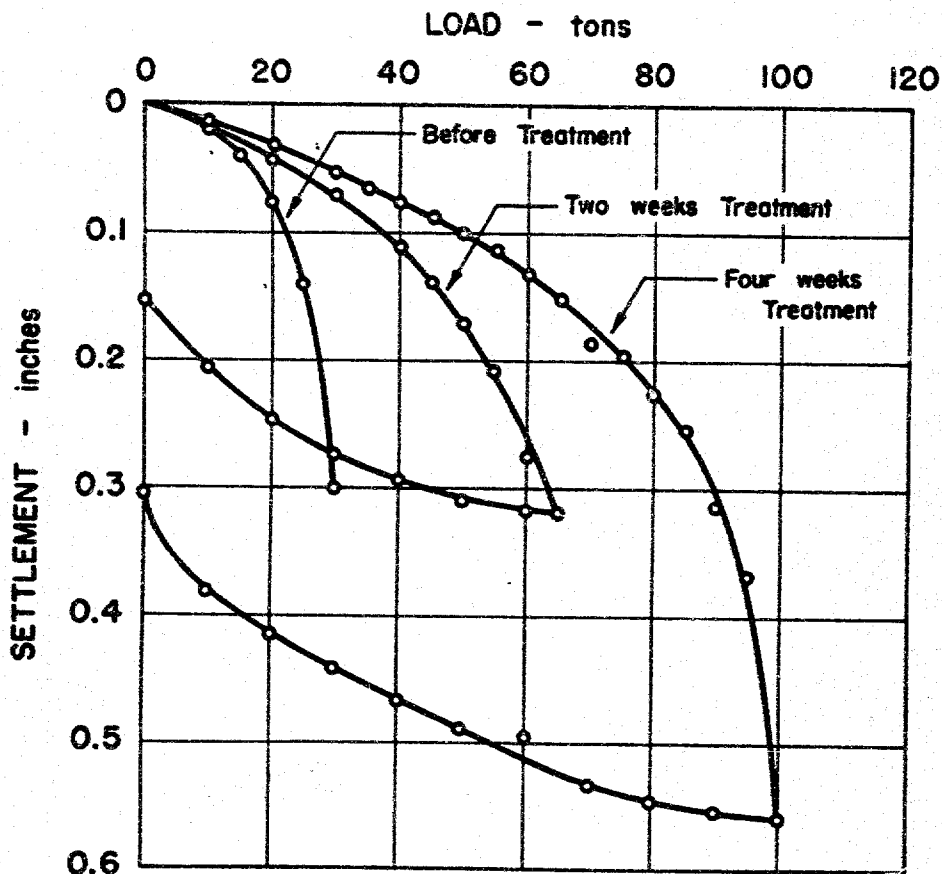


FIG.7.- LOAD TESTS ON TYPICAL PILE BEFORE AND DURING TREATMENT.
BIG PIC RIVER BRIDGE, MARATHON, ONTARIO



FIG.8.- MODEL OF SKELETON-TYPE PILE

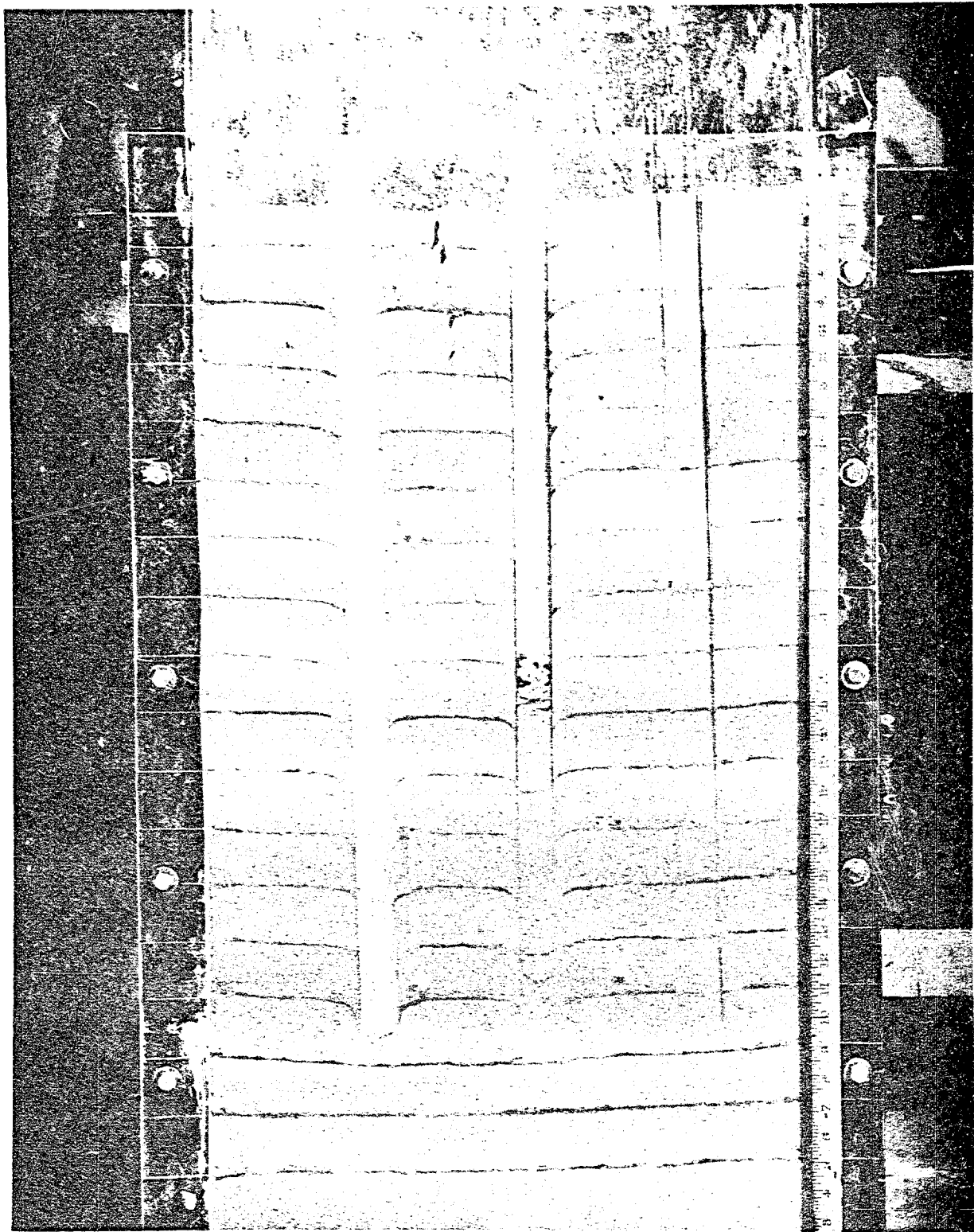


FIG.9.- DISPLACEMENT TESTS OF CONVENTIONAL PILES
AND SKELETON-TYPE PILE

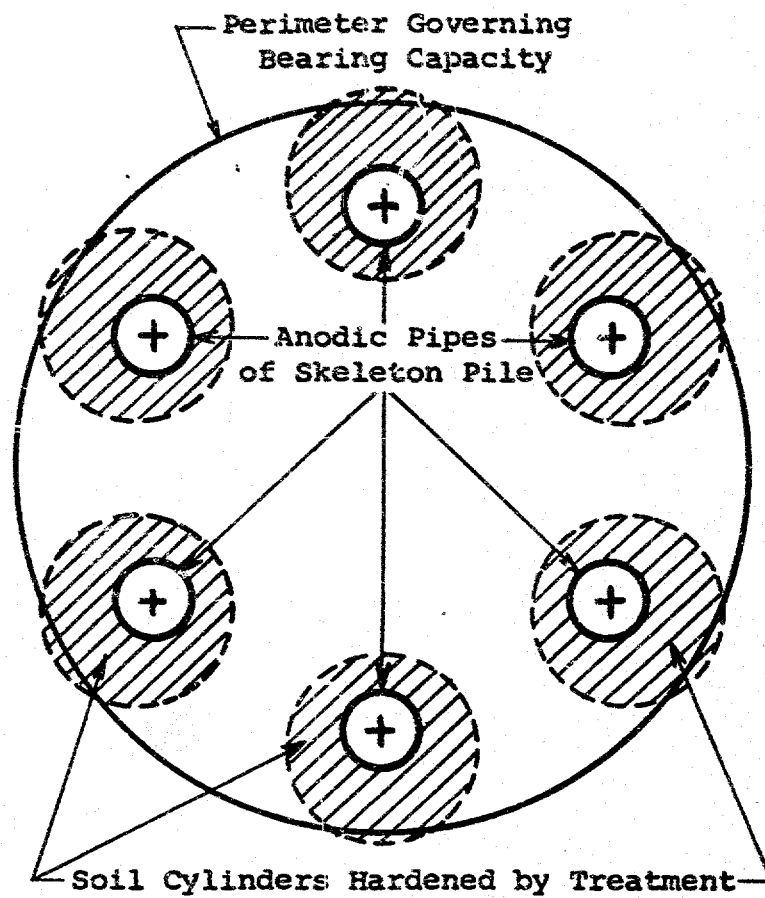


FIG.10.- STRENGTH INCREASE OF SOIL SURROUNDING SKELETON PILE
DURING ELECTRICAL TREATMENT

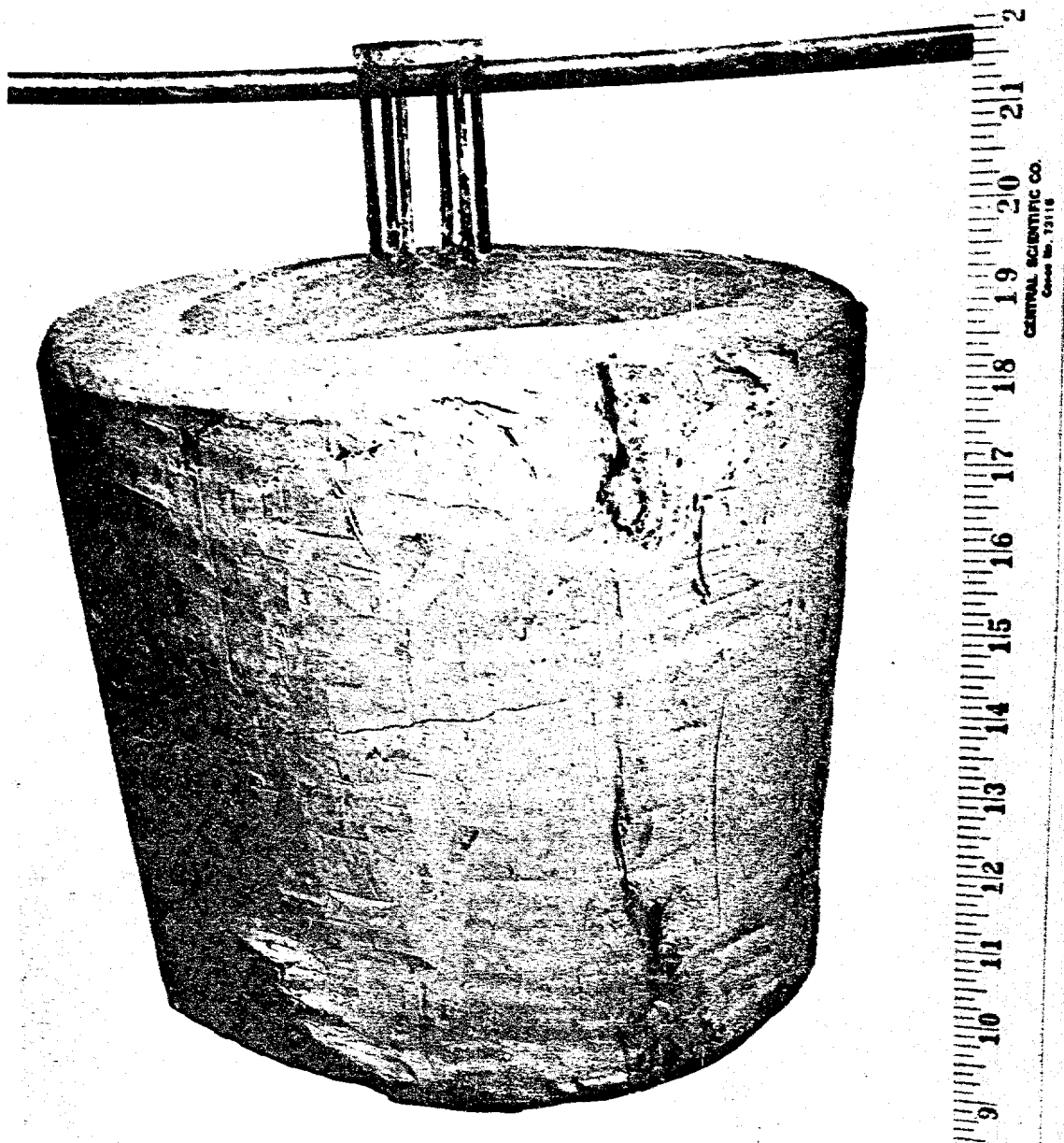


FIG.11.- EFFECT OF ELECTRO-OSMOSIS ON SKELETON PILE MODEL
IN BOSTON BLUE CLAY

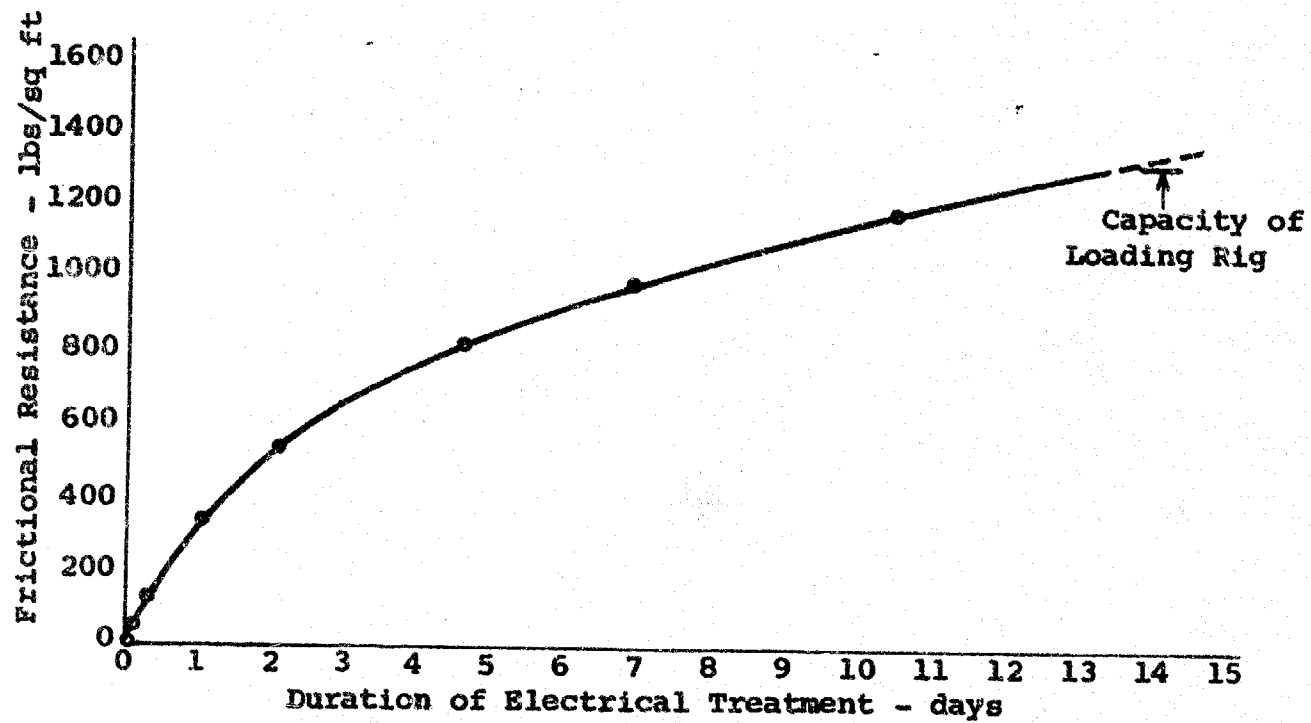


FIG.12.- INCREASE OF UNIT FRICTIONAL RESISTANCE OF MODEL PILE
WITH DURATION OF TREATMENT

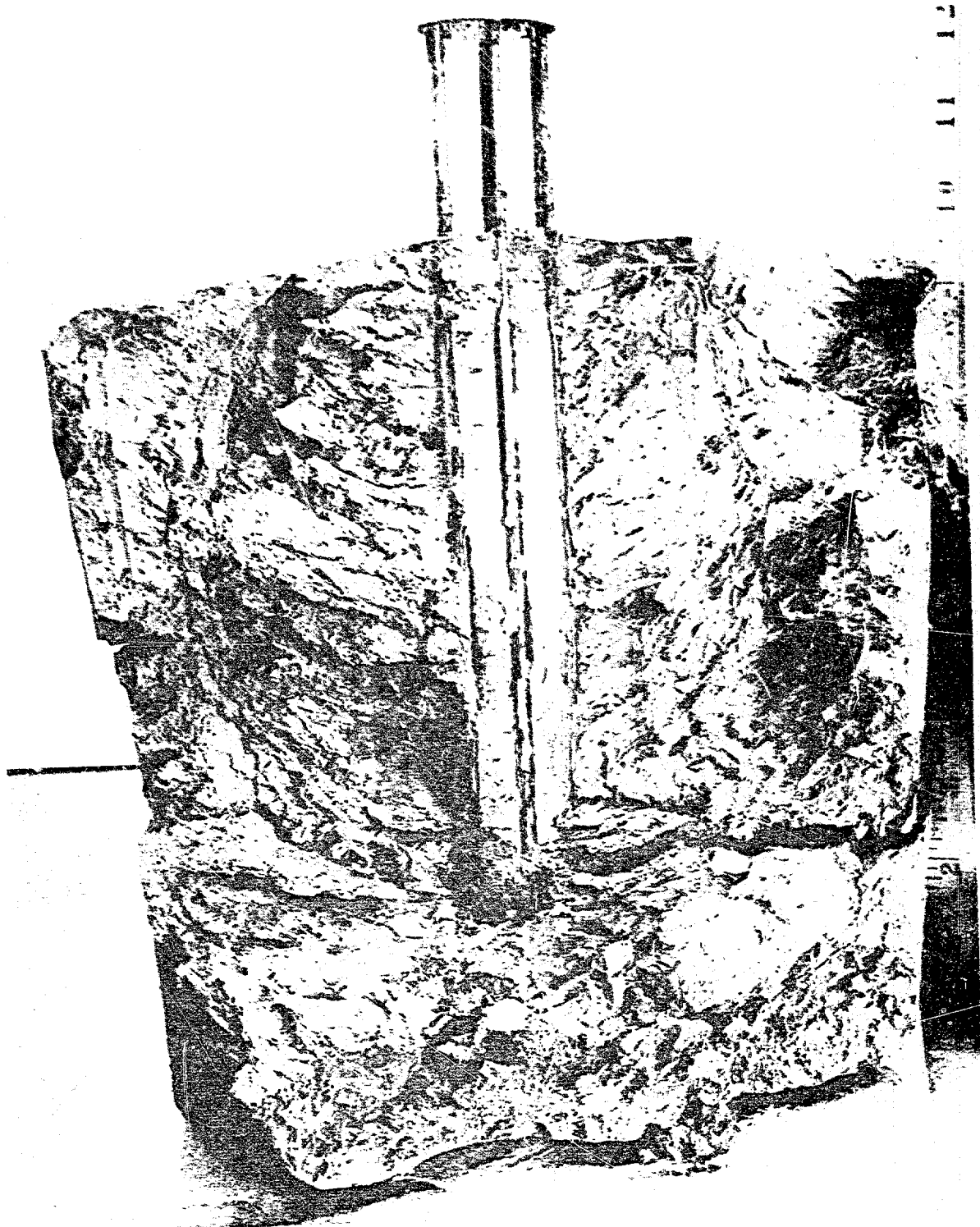


FIG.13.- DEVELOPMENT OF SOIL STRUCTURE BY ELECTRICAL TREATMENT