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STATUS REPORT NO. 2

PILE EXTRACTION TESTS

FAIRBANKS RESEARCH AREA

1 JUNE 1955

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ARCTIC CONSTRUCTION AND FROST EFFECTS LABORATORY
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BOSTON, MASSACHUSETTS

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FAIRBANKS RESEARCH AREA

1. Background. When piles are used in permafrost areas, certain special considerations apply which are not always obvious to engineers whose experience has been entirely in temperate zones. For example, it is not sufficient merely to drive piles to point bearing on permafrost. If the piles are not adequately anchored into the permafrost, winter freezing of the soil in the active layer may heave the piles and the overlying structure. When a pile does penetrate into permafrost, but the amount is inadequate, this uplift is usually cumulative since the building load is normally not sufficient to force the pile back to its original position, the portion of pile below the active zone remaining firmly gripped by the permafrost throughout the thaw. The result may be very severe structural distortion and damage, or excessive maintenance costs. Another complication which must be considered is the possibility of loss of supporting capacity and stability due to degradation of the permafrost. A third special consideration is the fact that frozen soils flow plastically under load at stresses which may be substantially below their rupture strength.

2. Site Conditions. The Fairbanks Research Area, in which the present series of pile tests is being performed, is located about 2-1/2 miles northeast of Fairbanks, Alaska. The terrain is characterized by a comparatively smooth gentle slope. The mean annual temperature at Fairbanks

is about 26°F. with extremes of plus 88°F. and minus 55°F. The total annual precipitation is about 12 inches, including an annual snowfall of about 4 feet. The natural soil underlying the Research Area to a depth in excess of 50 ft. is silt containing variable amounts of organic material, including occasional layers of peat. Under natural surface cover conditions, the maximum depth of seasonal thaw varies from 2 to 6 ft. The permafrost layer in the area is 100 to 150 ft. in thickness.

Subarea No. 4 located in the western part of the Research Area was selected as the location for the test pile installations. See Plate 1. Two test sites were laid out, namely: Site A which was cleared of trees and brush; and Site B from which the surface vegetation and the organic material immediately at the ground surface was stripped.

3. Installation of Piles. During the period from 15 April to 15 August 1952, 32 piles were installed to 8, 12 and 16 ft. nominal depths of embedment in permafrost in each of the two sites, that is a total of 64 piles. Twenty-four of the piles in each site were installed in 12-inch diameter drilled holes. Of the latter, the holes for Piles Nos. A-2, A-5, A-9, A-10, A-13, A-14, A-25, A-26, A-29 and A-30 (see Plate 1) were made by means of a truck-mounted earth auger without use of water. The remainder of the drilled holes were made with a core drill using water to wash the cuttings out of the hole. Eight piles in each of Site A and Site B were installed in steam-thawed holes. To form the pile holes, a steam pipe, 22 ft. long, with 80 psi steam pressure applied was started in a shallow hole and advanced to 22 ft. during a period of five to

twenty minutes. With the steam point at this depth, steaming was continued until the minimum diameter of the hole was twelve inches, an average time of about three hours. Records were kept of the quantity of steam used in jetting, rate of jet advance and the approximate dimensions of the thawed sections. The annular spaces around all piles were filled with silt water slurry. During the latter part of July 1952, wooden deck-type shade structures were erected over two groups of piles in Site B as shown in Plate 1, in order to simulate the shading effect of a building. Photograph No. 1 shows a typical concrete pile (in this case just after extraction). Photograph No. 2 shows a typical view of Area A with a concrete pile in the foreground and other types in the background.

4. General Observations.

a. Soil Data. - Three pile holes in each site were core-drilled for soil samples in the spring and summer of 1952. Other holes have been and will be core-drilled to obtain representative samples for routine soil tests of texture, density, moisture content and classification. Plate 2 shows soil data obtained in connection with building foundation studies at the research area. Plate 3 shows data from explorations in sites A and B.

Mr. C. W. Fulwider who was present at the time the piles were installed observed that the conditions in Site A and Site B are not entirely identical. He reports that Site A is slightly higher and better drained than Site B although the soils are somewhat finer grained.

Site B is lower and approaches rather closely to areas which contain standing water in summer, and the soils tend to be somewhat coarser than in Site A.

b. Ground Water Observations. - Ground water elevation observations were initiated in July 1952 in 13 wells bounding Sites A and B. Observations have been taken at two week intervals thereafter during the thawing periods. No analysis of these data is given in the present report.

c. Vertical Movement Observations. - A fixed vertical movement observation point was established on each pile. Observations referenced to a stable permanent bench mark were taken at weekly intervals for the first two months following installation and during subsequent periods of intense thaw or freeze. Between the latter periods, vertical movement observations have been taken monthly or at longer intervals.

d. Ground Temperatures. - Weekly ground temperature observations have been obtained by means of thermocouple assemblies installed on selected piles.

Around the 16 test piles installed in steam-jetted holes, thermocouples were installed and temperature observations were taken hourly during the period of jetting and daily thereafter for a period of two weeks. Since that time they have continued at weekly intervals. No analysis of these observations is presented in the present report.

5. Pile Extraction Tests.

a. Tests In 1953. - During October and November 1953 extraction tests were conducted on 11 piles installed to varying depths of embedment in drilled holes. These consisted of treated timber piles (tip down and butt down), reinforced concrete piles, steel pipe piles and steel I-beam piles. Several of the deeply embedded piles could not be extracted due to the difficulty of applying tensile forces to the piles without causing structural failure of the piles above ground surface. The extraction test procedure used on the first pile of each type, that is, wood, concrete, steel pipe and steel I-beam was as follows:

> Increments of 1000 lbs. were applied over a maximum period of 30 seconds and load held constant for five minutes or until rate of movement decreased to less than 0.002 inch per minute. When rate of movement was equal to or greater than 0.002 inch per minute thirty minutes after adding an increment, this was taken to indicate that failure by plastic flow had been reached. After plastic flow had occurred, increments of 4000 lbs. were added every five minutes until rupture between pile and frozen soil, or structural failure of pile. After initial tests on each type of pile, the increments used were selected so as to reach the estimated point of plastic flow in a minimum of six increments.

Photograph No. 3 shows the pulling arrangement.

b. Tests in 1954.-

(1) Short Time Tests. - Based on the results of extraction tests performed in fall of 1954 the initial loading increments for comparable tests carried out in F.Y. 1955 were selected as follows:

	Depth of Embedment in Frozen Soil		
	8 ft.	12 ft.	16 ft.
Increments (pounds)	5,000	8,000	10,000

The magnitudes of the above load increments were selected to permit at least six increments of load before reaching point of plastic flow. After applying each load increment, the pile movement was observed at one minute intervals for ten minutes. When a load was reached at which a pile continued to be pulled out of the ground at a rate greater than 0.001 inch per minute ten minutes after placing an increment, the load was held constant until rate of movement was less than 0.001 inch per minute. If movement thirty minutes after placing an increment exceeded 0.001 inch per minute, it was considered that a condition of plastic flow had been reached. After plastic flow, increments were held for five minutes with pile movement readings made every minute. Loading was continued until failure of pile member itself, rupture of bond between pile and frozen soil, or pile movement rate of 1/2-inch per minute.

(2) Long Term Test. - Since analysis of the results achieved up to the end of the 1954 short time loading tests had left some doubt as to the level of shear stress which could be tolerated for the life of the structure without excessive cumulative movement, it was decided to investigate the plastic flow characteristics of the pile failures in greater detail. The reason for this is clear when it is realized that a movement of only 0.01 inch per day accumulates to 3.65 inches if continued for a year and about three feet if continued for ten years. However, it is obvious that to complete tests of this nature in a reasonable time (that is in less than the lifetime of the structure) movements must be measured with an unusual degree of precision.

Loading of test Pile B-25 by means of dead weights and a lever arrangement was started late in 1954 and has continued through the winter. Stresses of 10, 15, 20 and 25 psi were successively applied. Each load increment was maintained for a period of thirty days or more; the movement of the pile was measured to 0.001 inch with 3 dial gauge extensometers placed at third points.

6. Test Results. Results of all pile tests performed to date are tabulated in Table 1. A condensed summary of pile extraction test data is given in Table 2. Typical load-deflection curves and load vs rate of deformation curves are given in Plates 4 and 5, respectively. A plot of vertical movement versus time for the long-term test on Pile B-25, for 15 and 20 psi stress increments, is shown on Plate 6. Plate No. 7 shows typical plots of vertical movement of piles versus time since the time of their original installation in 1952. Plate No. 8 is a layout of pile test Areas A and B with the total cumulative heave to 1 May 1953, 1 May 1954, and 1 May 1955, shown beside each pile. Plate No. 9 shows a generalized picture of assumed variation of load distribution along pile shaft.

In all computations of tangential shear stress developed in permafrost, an allowance has been made for skin friction in the active zone.

7. Special Test Difficulties.

a. Measurement of Pile Movement. - Measurement of pile movement to as close as 0.001 inch over long periods under field conditions of wide temperature extremes, etc., has presented an annoying problem which seems easier to solve than it actually is. Much effort has been spent trying to eliminate or minimize persistent up and downs, drifts, and other irregularities of the data.

The very first difficulty encountered was with the extensometer gauges used to measure the deflections. These froze up under their twenty-four hour a day exposure to the winter weather at the Research Area. A very effective solution for this, devised at the Research Area, consisted of placing a small incandescent lamp near the gauge as is shown in Photograph No. 4, the heat of which prevented any icing.

It was next found that the extensometers appeared to show the pile as sinking into the ground under the upward pull. This was found to be due to heaving of the extensometer beam supports, even though these had been driven to twice the depth of the active zone. New supports were then installed which eliminated the progressive heave effect.

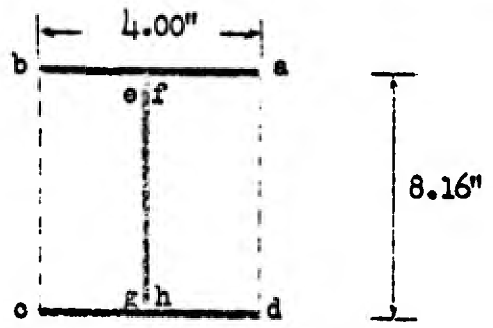
However, small fluctuations in readings continue to occur even at present. These are believed to be largely, if not wholly, fluctuations with temperature, even though substantial efforts have been made, in mounting the extensometers, to minimize the effect of temperature lengthening, shortening and twisting of the various supporting members. It is apparent that the mechanics of setting up and performing even the simplest field tests may require unexpected time and effort in frozen ground studies.

b. Application of Load to the Piles. - Difficulties in applying tensile forces to the piles have been substantial. In the original tests, for example, a slot was cut through the upper end of the pile and a pulling rod passed back through it. The result was often failure of the top of the pile as shown in Photographs Nos. 5 and 6, before bond failure in permafrost could be developed. In the concrete pile tests the concrete itself, of course, cracked very easily after which the reinforcing rods tended to stretch and spall off concrete. New gripping methods have now been devised. For example a steel collar and wedge device is being constructed for the wooden piles which avoids the weakening effect of cutting the slot through the pile. Little improvement is possible for the concrete piles since their strength is limited by the amount of reinforcing rods rather than by the method of gripping and pulling used. Of course no actual pile foundations would be designed for loadings which would cause such excessive stressing of the piles. However, in order to establish allowable design values of known factor of safety, it is necessary to understand what limiting values of bond stress are applicable. Since general values of bond stresses are now known, from the present tests, as indicated on Table No. 2, it will be possible to plan future pile tests in the program o as to keep depth of embedment and stresses in piles at practical levels.

c. Placing of Slurry. - It has been found in pile tests at the Research Area that placing of the slurry around the piles is a critical operation for cases where skin friction is depended upon for bearing capacity. Photograph No. 7, for example, shows a concrete pile after extraction. The light spot near the rule which does not show any discoloration from contact with soil suggests a void. It is considered advisable to place the slurry by a tremie type method in order to insure that the space between the pile and the drilled hole will be completely filled. If the slurry is simply dumped from the surface into the space between the pile and the wall of the hole, and if the space is narrow, extensive voids may occur along the sides of the pile. Preparation of the slurry by hand is a very tedious operation. It should be planned for a machine operation, even for a small job, if for no other reason than to avoid the skimping which may occur when the workers discover how slow and tedious the process is.

8. Discussion of Results. - The ultimate adfreeze bond strength in tests to date has varied from a low of 13.0 psi to a high of 40.7 psi; in drilled holes, averages thus far are 22.5 psi for concrete piles, 27.1 psi for steel pipe piles and 37.2* psi for steel I-beam piles. Driven pipe piles average 23.2 psi for piles pulled a short time after installation. A few tests on piles driven into the active zone only, have shown comparable skin friction values for the soil in the thawed state ranging from 1.3 to 5.7 psi.

* For steel I-beams, perimeter used for computing total shear area = abcd.



However, if the total surface perimeter of the I-section is used, abegcdhfa, the pile surface area is increased by almost one third, and the ultimate adfreeze bond strength values are reduced to nearly the same values as for the steel pipe piles, which seems reasonable. The perimeter distance bc is about two-thirds of the distance begc; however, reference to other data suggests that the shear strength through the soil should be of the order of double the adfreeze bond strength. Therefore failure should be expected along begc rather than bc. None of the I-beam piles have been extracted sufficiently to provide visual check of this point. It is possible that all I-beam pile results may have to be recomputed. However, all data in this report are based on abcd.

Of the 11 piles tested in the fall of 1953 in short term loading, 7 were frozen in so securely that they failed structurally before failure in bond between pile and soil occurred. Of the 20 piles tested in short term loading in the fall and winter of 1954, 2 piles failed structurally before failure in bond could be achieved, and 2 of them reached the maximum load capacity of the loading apparatus without reaching failure either in soil or in pile.

Reference to the plot of typical load-deflection curves obtained in the 1953 series of tests shown on Plate 4 indicates that the curves show progressively increasing deflection as the loads increase. Plate 5, which shows rate of deformation instead of cumulative deformation, indicates the importance of the rate of movement at the higher loads. In test of Pile B-16 (as shown on Plate 5) continuous plastic flow began between 20 and 30 kip loadings, then increased and became very much more rapid at loads of the order of 60,000 lbs. or more. It will be clear from this plate that the arbitrary figures of .002 and .001 inch per minute taken as indicating that plastic flow was occurring may give load values which are actually substantially above the actual stress at which plastic yield begins to occur. This progressive yield of piles in soil is not important in case of design for wind load uplift or similar short duration transient loading, but is a factor in design for long term loading of piles. It is also a factor in designing to obtain sufficient embedment to resist frost heave of the pile, since frost lifting may gradually cause the pile to move out of the ground under its steady force acting for many weeks and months, unless the pile is sufficiently embedded in permafrost so that the frost heave resisting stresses are kept below the level of plastic yield. Stresses at significant plastic yield as determined by the more or less arbitrary definition of 0.001 - 0.002 inch per minute movement after thirty minutes, averages about 80 per cent of the bond failure stress for the drilled and steam-thawed holes with individual values varying from 65 per cent to 100 per cent. Since these values were selected at an arbitrary time of 30 minutes after load had been applied and since as indicated on Plate 6 a transient stress condition existed in the pile, the load being <u>in process</u> of re-distribution along the pile length, the plastic yield values cannot be said to represent any well-defined physical concept.

Reference to Plate 6 showing the results of the long term loadings of Pile B-25 at 15 and 20 psi indicates that after a few days of gradual adjustment the pile became stable at both these stress increments. Initial data from loading of this pile at a stress of 25 psi indicate that the pile is also stable at the latter stress level. This suggests that the long term loading test on this steel I-beam pile may give results at least of about the same order as obtained in the previous short term loading tests. (As shown on Table 2 the steel I-beams in drilled holes tested to date have failed at an average of 37.2 psi with a range of 33.6 and 40.7 psi. As shown on Table 1 the stresses at start of nominal plastic yield in the I-beam short term tests completed to date in drilled holes have ranged from 25.2 psi to 32.2 psi). It may be that the short term loading test is actually less conservative than the long term test. It is possible that the short term test completed in a day or two gives insufficient time for redistribution of stresses along the length of the pile thus actually resulting in failure at lower stresses than in a long time test. It is expected that as further load increments are added to Pile B-25 a level will shortly be reached at which adjustment to a stationary condition will not occur. Reference to Plate 6 shows that the achievement of a steady, stable condition took distinctly longer at the 20 psi loading than at the 15 psi loading.

More than half of the 64 piles installed in the summer of 1952 have shown no progressive movement since installation although the data indicate apparent small, irregular movements of 1/2-inch or less; however, it is believed that most of these apparent movements are simply due to observational errors. About a third of the piles have shown an inch or more of cumulative heave two years after construction.

Plate 8, showing the cumulative heaves at the end of approximately one, two and three years since installation, indicates that negligible heave (less than 1/2 inch) has occurred in all but one of the piles in drilled holes in Area A, whereas the corresponding types of installations in Site B show significant heave for the majority of piles. A combination of factors may explain this. First, some of the holes in Area A were installed in holes drilled by augering without use of water; thus less heat was introduced in these particular holes. Second, the piles in Area A were first to be installed and were put in earlier in the year, when air temperatures were still fairly low and the ground was at its coldest. Thus there was less opportunity for disturbance of the thermal regime. Third, as previously pointed out, Area A is slightly different in topographic position from Area B and contains probably slightly finer-grained soil.

Plate 8 shows that the steam-thawed holes in Site A produced exceptional heaves during the first winter. Heave of wood Pile No. A-7 reached nearly five inches in the first winter and 6.6 inches after the second winter. In general, these piles required two full winters to become stabilized although a few showed only slight movement in the second winter. For some reason the piles placed in steam-thawed holes in Site B showed generally less heave than those in steam-thawed holes in Site A. Further study on the data of the amount of steam used may give some explanation of this. However, no reason is apparent at this date.

It presently appears that under the soil and permafrost conditions at the Fairbanks Research Area, and using the pile types and installation techniques here employed, reliance cannot be placed on <u>all</u> piles of a foundation to be stable against frost uplift until after one to three winters, depending on the installation method used, unless artificial refrigeration is employed to hasten attainment of solid freeze-back or unless the frost lifting action can be defeated by other means. Further study of the nature of permafrost, of other pile types, of refinement in installation techniques, of further test results from the pile test program and of the nature of the freeze-back process may yield new approaches. Except for piles which were installed by steam thawing, there is no apparent reason why some piles heaved and others did not under apparently similar conditions. It is considered likely that the heave is evidence of a very slow rate of freeze-back. This is presumably because the permafrost in the Fairbanks Area is close to 32°F. in temperature and has very little reserve of cold. Where the permafrost is much colder as at Point Barrow, Alaska, or Thule, Greenland, much more positive freeze-back should occur.

Inspection of the shear stress data on Table No. 1 shows no noticeable difference between either plastic yield or bond failure stresses for the shaded piles as compared with the unshaded piles.

Plate 9 illustrates a generalized concept of changes in load distribution which occur when load is applied to a pile in frozen ground, considering only skin friction. Initially the stresses may be carried entirely in the upper layers of ground as illustrated by Curve No. 1. However, when the stresses at the top of the ground become so high as to result in plastic flow in the material surrounding the pile, yield occurs, allowing the upper part of the pile to change in length and to distribute more of the resisting force progressively to lower levels in the ground as illustrated by Curves 2 and 3. Thus, if the load applied to the pile is small, or if the pile is sufficiently long, virtually no load may reach the bottom of the pile. This has been recognized for some time in the design of pile foundations in temperate regions; however, special physical properties of frozen ground make the analysis of pile bearing capacity in the arctic and subarctic a unique problem.

9. Conclusions.

a. Under the soil and permafrost conditions at the Fairbanks Research Area, and using the pile types and installation techniques here employed, all piles in an installation cannot be relied on to become stable against frost uplift in a reasonable period without artificial refrigeration, even when depth of embedment in permafrost is as much as 4 or 5 times the depth of the active zone.

b. Under the conditions of the tests for piles placed in drilled or steam-thawed holes, a design tangential shear stress in permafrost of the order of 15 psi seems to date tentatively safe against the possibility of continuous progressive deformation, providing solid freeze-back has been completed. This conclusion takes into account the results of the long-term test and the fact that plastic deformation as defined in the short-time tests represents a transient condition. It is not intended to apply to driven piles, on basis of data thus far available.

10. Recommendations. In order to establish pile design criteria which are applicable under various construction conditions, it is clearly necessary to obtain basic understanding of the way in which stresses are distributed from a pile to a body of frozen ground, both

through adfreeze bond and through end bearing. Also it is necessary to understand more fully the freeze-back process which occurs after installation of a pile in frozen ground and to investigate possible [illegible] methods for speeding this process and making it more positive. Therefore the following measures are recommended for continuation of the pile test program at the Research Area:

a. Install 9 additional test piles at the Research Area consisting of concrete, steel pipe and steel I-beam piles (3 depths of embedment, each type) especially instrumented with strain gauges and load pressure cells. Consider also possibility of three wood piles. The piles should be first subjected to increments of compressive loading to a high level of intensity (but short of failure). Following this the piles should be tested in tension. In b th types of loading the changes in stress distribution with load and with time should be measured.

b. A series of tests is recommended in which piles are loaded in end bearing only, without any skin friction, and the end bearing value determined as an individual separate component.

c. The bearing capacity of piles in combined skin friction and end bearing should be evaluated.

d. Installation and test of belled caisson or other special piles to resist frost heave uplift should be carried out.

e. The forces required to resist normal frost heave uplift should be determined.

f. The possibilities for eliminating frost heave uplift by means of sleeves or other methods, in the active zone, should be investigated.

g. Micro-measurements of the temperature condition around piles during placement and freeze-back should be made since present thermocouple methods are not sufficiently precise. It is suggested that soil resistance measurements offer a possible means of confirming the indications of the temperature readings, as to whether or not the material is in a frozen state.

h. The manner in which freezing occurs, i.e., the sequence of the crystallization process when piles freeze back, should be investigated.

Photo 1


PILE EXTRACTION
TESTS 1953
PILE NO A13
PRIOR TO TEST
LOOK NORTH 15 OCT 1953

Photo 2

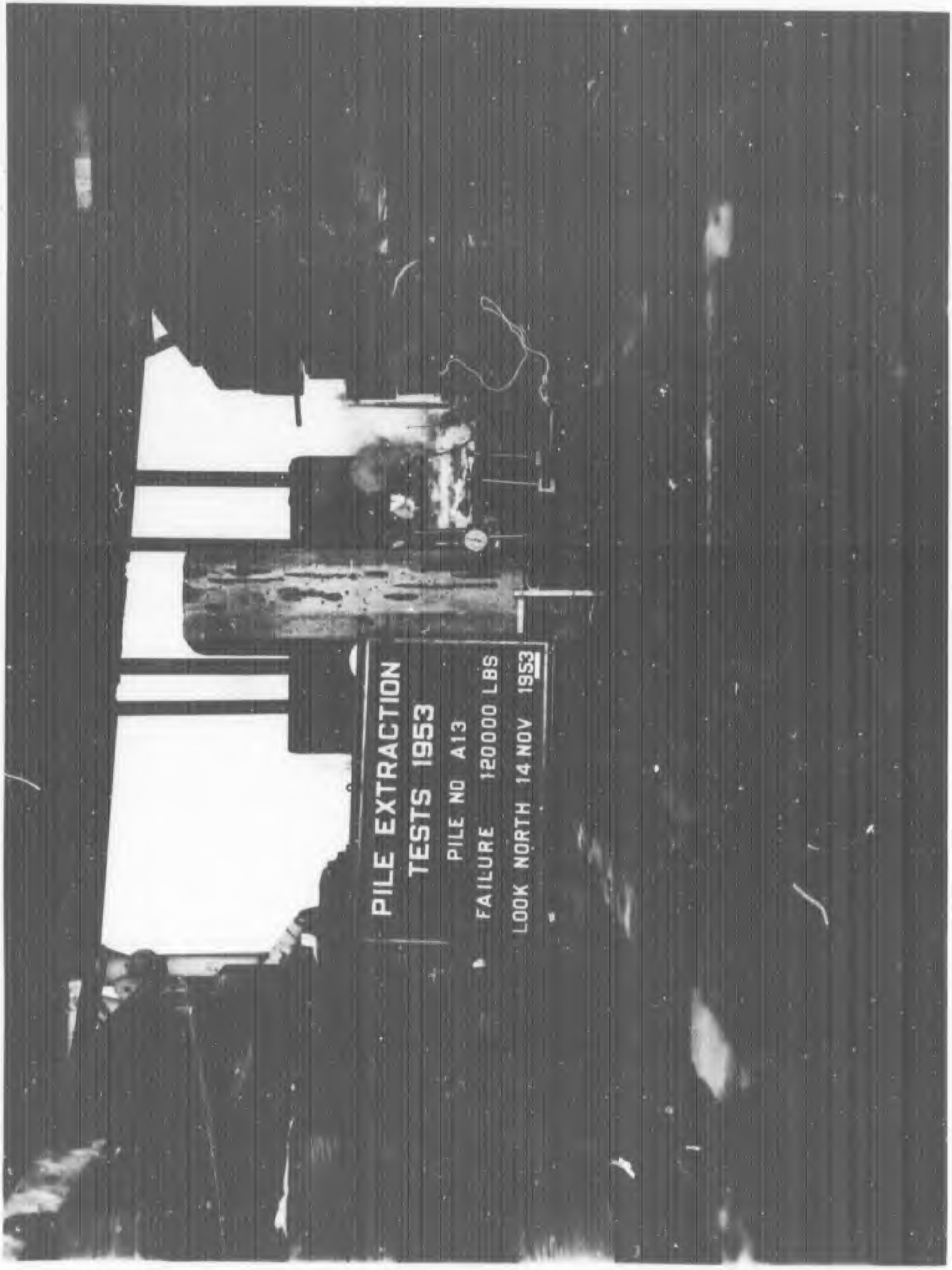

PILE EXTRACTION
TESTS 1953
PILE NO A13
FAILURE 120000 LBS
LOOK NORTH 14 NOV 1953

Photo 3

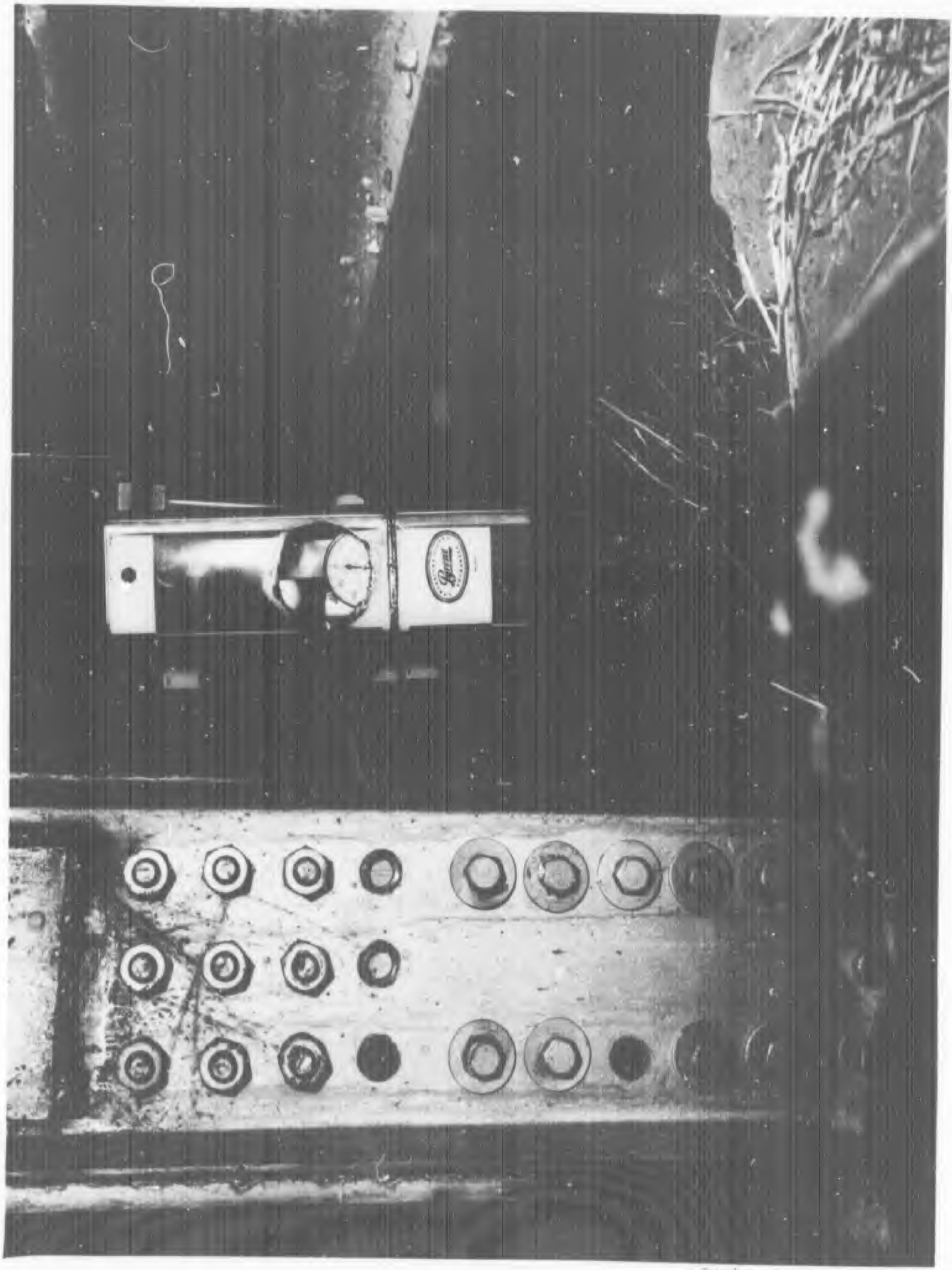


Photo 4

Photo 5

Photo 6

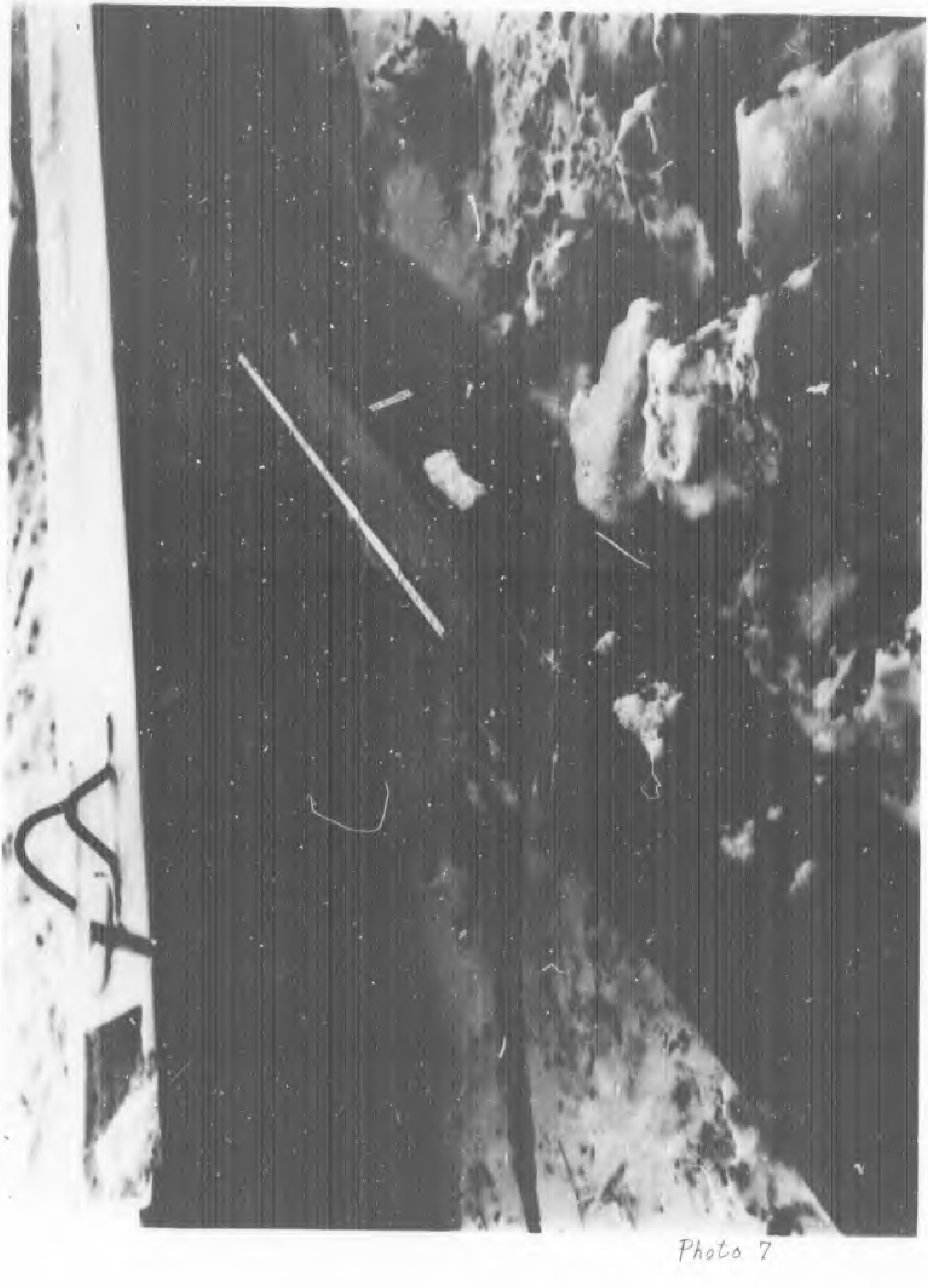


Photo 7

Type Pile	Method of Installation	Pile No.	Date of Extraction	Date of Installation	Heave: Approx. 1 June 1953 Ft.	Heave: Approx. 1 June 1954 Ft.	Embedments and Areas at: Total Embedment Ft.	Embedments and Areas at: Embedment In Unfrozen Soil Ft.
Wood (butt down)	Drilled Hole	A4	19-20 Oct. 53	14 Apr. 52	0.011	-	17.0	4.5
		B8	3 Dec. 53	5 May 52	0.010	-	15.0	6.9
	Steam-Thawed Hole							
	Driven	SA-33	4 Nov. 53	28 Oct. 53	-	-	3.0	3.0
		SB-33	7 Nov. 53	28 Oct. 53	-	-	3.7	3.7
Wood (tip down)	Drilled Hole	A5	20 Nov. 53	24 Apr. 52	0.012	-	13.0	3.2
	Steam-Thawed Hole							
	Driven	SA-34	5 Nov. 53	28 Oct. 53	-	-	2.4	2.4
Reinforced Concrete (precast)	Drilled Hole	A12	22-23 Oct. 53	13 May 52	0.009	-	17.0	4.5
		A13	14 Nov. 53	24 Apr. 52	0.012	-	13.0	3.2
		B16	5 Dec. 53	5 May 52	0.169	-	15.0	6.3
		B15	10 Nov. 54	3 May 52	0.059	0.072	16.9	3.6
		B14	10 Nov. 54	22 May 52		0.006	21.0	4.0
	Steam-Thawed Hole	B13	13 Nov. 54	12 June 52	0.130	0.162	20.8	3.6
	Driven							
Steel Pipe (Open end down)	Drilled Hole	A20	30 Oct. 53	5 May 52	0.020	-	17.0	4.4
		B19	26 Nov. 54	21 May 52	0.084	0.108	20.9	3.5
		B24	8 Dec. 53	2 May 52	0.267	-	15.0	6.0
		B23	4 Nov. 54	31 May 52	0.029	-	16.0	4.3
		B22	5 Nov. 54	26 May 52	0.006	0.027	22.0	4.2
		B17	22 Nov. 54	13 May 52	0.264	0.313	12.7	2.4
		B18	24 Nov. 54	12 May 52	0.247	0.276	16.7	3.2
	Steam-Thawed Hole	B20	6 Nov. 54	9 June 52	0.048	0.082	21.9	3.6
	Driven	SA36	6 Nov. 53	29 Oct. 53	-	-	2.1	2.1
		SB36	10 Nov. 53	29 Oct. 53	-	-	4.1	4.1
		B37	27 Nov. 54	15 Oct. 54	-	-	9.7	5.7
		B38	28 Oct. 54	13 Oct. 54	-	-	9.4	5.4
		B40	28 Oct. 54	13 Oct. 54	-	-	10.8	4.8
		B41	29 Oct. 54	14 Oct. 54	-	-	10.5	4.5
		B43	29 Oct. 54	14 Oct. 54	-	-	12.3	4.3
		B44	30 Oct. 54	15 Oct. 54	-	-	12.5	4.5
Steel Pipe (Closed end down)	Drilled Hole	A21	17 Nov. 53	23 Apr. 52	-0.088	-	13.0	3.4
	Steam-Thawed Hole							
	Driven							
Steel I-Beam	Drilled Hole	A28	2 Nov. 53	7 May 52	0.021	-	17.0	3.9
		A29	18 Nov. 53	23 Apr. 52	0.023	-0.036	13.0	3.6
		B30	4 Nov. 54	27 May 52	0.024	0.030	21.4	3.4
		B31	21 Oct. 54	28 May 52	0.003	0.004	17.0	2.8
		B32	21 Oct. 54	29 May 52	0.060	0.050	14.0	4.0
	Steam-Thawed Hole	B29	2 Nov. 54	12 June 52	0.213	0.244	21.3	3.6
		B28	2 Nov. 54	8 June 52	0.086	0.125	22.3	3.5
	Driven	SA-35	6 Nov. 53	28 Oct. 53	-	-	2.8	2.8
		SB-35	9 Nov. 53	29 Oct. 53	-	-	4.6	4.6

NOTE: (1) No plastic flow or failure at maximum capacity of loading apparatus in tests noted.

A

Date of Extraction			Plastic Flow			Ultimate			
								Unit Tangential Shear Stress, p	
Embedment In Frozen Soil Ft.	Surface Area In Unfrozen Soil Sq. In.	Surface Area In Frozen Soil Sq. In.	Upward Vertical Movement In.	Load Kips	Unit Tangential Shear Stress psi	Upward Vertical Movement In.	Load Kips	For Piles Which Failed in Bond	For Cases of Structural Pil Failure or Whi Reached Capaci of Assembly
12.5 8.1	1490 2940	4320 3560	Pile Failed Before Plastic Flow Pile Failed Before Plastic Flow			0.131 0.122	71 40		16.1 12.5
- -	1140 1365	- -	0.051 0.088	3.0 3.5	2.6 2.5	0.933 1.092	4.5 6.2	3.9 4.5	
9.8	1083	3165	Pile Failed Before Plastic Flow			0.069	68		21.2
-	940	-	0.100	3.6	3.8	0.899	5.4	5.7	
12.5 9.8 8.7 13.3 17.0	1940 1382 2722 1557 1729	5105 3940 3465 5846 7055	0.442 0.216 0.050 0.139 0.406	136 110 63 90 130	26.1 27.9 16.5 14.4 16.6	1.683 0.840 1.125 0.303 3.234	160 120 72 110 170	- 30.5 19.8 17.8 21.9	30.7
17.2	1555	7150	0.206	140	17.8	0.810	160		20.4
12.6 17.4 7.0 13.7 17.8 10.3 13.8	1430 1138 2630 1398 1365 780 1012	4050 5660 2300 4460 5280 3350 4420	Pile Failed Before Plastic Flow 0.145 0.041 0.098 0.344 0.094 0.140	 160 30 110 130 80 130	 27.6 13.0 23.8 21.8 23.2 28.7	0.072 0.393 0.091 0.347 0.263 0.180 0.171	133 Not reached 30 150 160 110 140	13.0 32.8 26.9 32.1 30.9	32.7 34.6 (
18.3	1180	5950	0.098	140	22.9	0.283	Not reached		33.2 (
- - 4.0 4.0 6.0 6.0 8.0 8.0	690 1346 1854 1755 1560 1463 1400 1463	- - 1300 1300 1950 1950 2600 2600	0.040 0.012 0.094 0.045 0.077 0.066 0.080 0.184	1.0 3.2 24 21 28 28 35 50	1.6 2.4 15.4 13.3 12.6 12.7 12.8 17.9	0.623 0.777 0.375 0.253 0.224 0.357 0.247 0.434	1.4 3.2 34 36 44 52 60 65	2.0 2.4 22.3 25.0 20.8 24.9 22.3 23.6	
9.6	1116	3020	0.086	88	29.3	0.236	88	29.1	
13.1 9.4 10.0 14.2 10.0	1470 1047 995 815 1164	4950 2735 5240 4132 2910	Pile Failed Before Plastic Flow Pile Failed Before Plastic Flow 0.297 0.080 0.261	 170 120 75	 32.2 26.7 25.2	0.080 0.051 0.322 0.751 2.558	86 62 180 170 100	40.7 33.6	22.5 24.3 34.0
17.7 16.8	1046 1019	5150 5470	0.194 0.165	130 140	24.9 25.3	0.729 0.785	200 200	38.4 36.2	
- -	1060 1740	- -	0.133 0.029	1.2 1.9	1.3 1.3	0.857 0.706	1.8 2.2	1.7 1.3	

Plastic Flow			Ultimate				Pile No.	Remarks
					Unit Tangential Shear Stress, psi			
Upward Vertical Movement In.	Load Kips	Unit Tangential Shear Stress psi	Upward Vertical Movement In.	Load Kips	For Piles Which Failed in Bond	For Cases of Structural Pile Failure or Which Reached Capacity of Assembly		
Pile Failed Before Plastic Flow Pile Failed Before Plastic Flow			0.131 0.122	71 40		16.1 12.5	A4 B8	
0.051 0.088	3.0 3.5	2.6 2.5	0.933 1.092	4.5 6.2	3.9 4.5		SA-33 SB-33	Active zone friction
Pile Failed Before Plastic Flow			0.069	68		21.2	A5	
0.100	3.6	3.8	0.899	5.4	5.7		SA-34	Active zone friction
0.442 0.216 0.050 0.139 0.196	136 110 63 90 130	26.1 27.9 16.5 14.4 16.6	1.683 0.840 1.125 0.303 3.234	160 120 72 110 170	- 30.5 19.8 17.8 21.9	30.7	A-12 A-13 B-16 B-15 B-14	
0.206	140	17.8	0.810	160		20.4	B-13	
Pile Failed Before Plastic Flow 0.145 0.041 0.098 0.144 0.094 0.140	 160 30 110 130 80 130	 27.6 13.0 23.8 21.8 23.2 28.7	0.072 0.193 0.091 0.347 0.263 0.180 0.171	133 Not reached 30 150 160 110 140	 13.0 32.8 26.9 32.1 30.9	32.7 34.6 (1)	A-20 B-19 B-24 B-23 B-22 B-17 B-18	 Shaded Shaded Shaded
0.098	140	22.9	0.283	Not reached		33.2 (1)	B-20	Shaded
0.040 0.012 0.094 0.045 0.077 0.066 0.080 0.184	1.0 3.2 24 21 28 28 35 50	1.6 2.4 15.4 13.3 12.6 12.7 12.8 17.9	0.623 0.777 0.375 0.253 0.224 0.357 0.247 0.134	1.4 3.2 36 36 44 52 60 65	2.0 2.4 22.3 25.0 20.8 24.9 22.3 23.6		SA-36 SB-36 B-37 B-38 B-40 B-41 B-43 B-44	Active zone friction
0.086	88	29.1	0.236	88	29.1		A-21	
Pile Failed Before Plastic Flow Pile Failed Before Plastic Flow 0.297 0.080 0.261	 170 120 75	 32.2 26.7 25.2	0.080 0.051 0.322 0.751 2.558	86 62 180 170 100	 40.7 33.6	22.5 24.1 34.0	A-28 A-29 B-30 B-31 B-32	Shaded
0.194 0.165	130 140	24.9 25.3	0.729 0.785	200 200	38.1 36.2		B-29 B-28	
0.133 0.029	1.2 1.9	1.3 1.1	0.857 0.706	1.8 2.2	1.7 1.3		SA-35 SB-35	

TABLE 1

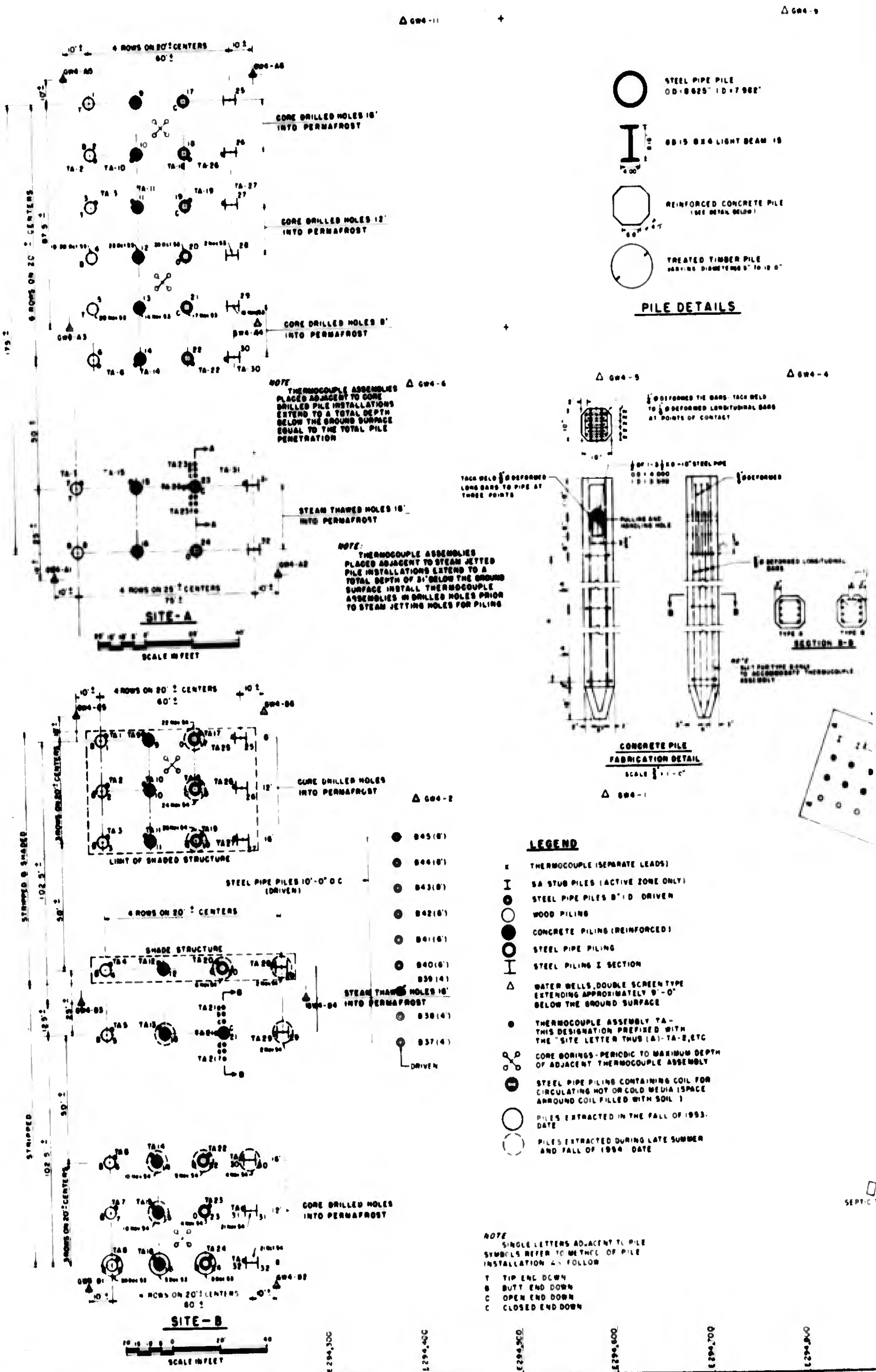
TABLE 2

CONDENSED SUMMARY OF PILE EXTRACTION TEST RESULTS
TO MAY 1955

Type Pile	Method of Pile Installation	No. of Piles Tested	Depth of Embedment in Permafrost Feet	Average Tangential Shear Stress at Failure of Bond Between Pile and Soil	Remarks
Wood (butt down except as noted)	Drilled Hole	3	8.1 to 12.5	These 3 piles failed structurally at 12.5 to 21.2 psi	Two butt down. One tip down.
	Steam-Thawed Hole	0	-	No tests completed to date	-
	Driven	1	0	5.7 (thawed soil skin friction)	Tip down. Driven into active zone only
Reinforced Concrete (precast)	Drilled Hole	4	8.7 to 17.0	22.5* Range 17.8 to 30.5	
	Steam-Thawed Hole	1	17.2	Pile failed structurally at 20.4 psi	Shaded Pile. Pile failed on one edge due to non-axial pull
	Driven	0	-	No tests performed to date	
Steel Pile (open at bottom)	Drilled Hole	4	7.0 to 17.8	27.1* (1) Range 13.0 to 32.8	3 shaded piles included in group tested.
	Steam-Thawed Hole	1	13.3	Pile did not fail in adfreeze bond at max. capacity loading apparatus, 32.2 psi	Shaded Pile.
	Driven	2 6	0 4.0 to 8.0	2.2 Range 2.0-2.4 23.2 Range 20.8 - 25.0	
Steel I-Beam	Drilled Hole	3 2	9.4 to 13.0 10.0 to 14.2	These 3 piles failed structurally at 22.5 to 34.0 psi 37.2* Range 33.6 to 40.7	
	Steam-Thawed Hole	2	17.7 to 18.8	37.3 Range 36.2 to 38.4	Shaded Piles
	Driven	2	0	1.5 Range 1.3 to 1.7	

* Indicates one or more piles omitted which failed structurally or which otherwise did not reach adfreeze bond strength failure

(1) Not included here is one steel pipe, closed end down, which failed in adfreeze bond at 29.1 psi



SITE-A
SCALE IN FEET
4 ROWS ON 20' ± CENTERS
60' ±
CORE DRILLED HOLES 16' INTO PERMAFROST
CORE DRILLED HOLES 12' INTO PERMAFROST
CORE DRILLED HOLES 8' INTO PERMAFROST
STEAM THAWED HOLES 16' INTO PERMAFROST
4 ROWS ON 25' ± CENTERS
75' ±
SITE-B
LIMIT OF SHADED STRUCTURE
SHADE STRUCTURE
STEEL PIPE PILES 10'-0" O C (DRIVEN)
STRIPPED & SHADED
STRIPPED
DRIVEN
STEEL PIPE PILE
REINFORCED CONCRETE PILE
TREATED TIMBER PILE
PILE DETAILS
CONCRETE PILE FABRICATION DETAIL
SECTION B-B
LEGEND
THERMOCOUPLE (SEPARATE LEADS)
SA STUB PILES (ACTIVE ZONE ONLY)
STEEL PIPE PILES 8" I D DRIVEN
WOOD PILING
CONCRETE PILING (REINFORCED)
STEEL PIPE PILING
STEEL PILING I SECTION
WATER WELLS, DOUBLE SCREEN TYPE EXTENDING APPROXIMATELY 9'-0" BELOW THE GROUND SURFACE
THERMOCOUPLE ASSEMBLY TA- THIS DESIGNATION PREFIXED WITH THE "SITE LETTER THUS (A)-TA-8, ETC
CORE BORINGS-PERIODIC TO MAXIMUM DEPTH OF ADJACENT THERMOCOUPLE ASSEMBLY
STEEL PIPE PILING CONTAINING COIL FOR CIRCULATING HOT OR COLD MEDIA (SPACE ARROUND COIL FILLED WITH SOIL)
PILES EXTRACTED IN THE FALL OF 1953. DATE
PILES EXTRACTED DURING LATE SUMMER AND FALL OF 1954 DATE
NOTE
SINGLE LETTERS ADJACENT TO PILE SYMBOLS REFER TO METHOD OF PILE INSTALLATION AS FOLLOW
T TIP END DOWN
B BUTT END DOWN
O OPEN END DOWN
C CLOSED END DOWN
NOTE
THERMOCOUPLE ASSEMBLIES PLACED ADJACENT TO CORE DRILLED PILE INSTALLATIONS EXTEND TO A TOTAL DEPTH BELOW THE GROUND SURFACE EQUAL TO THE TOTAL PILE PENETRATION
NOTE
THERMOCOUPLE ASSEMBLIES PLACED ADJACENT TO STEAM JETTED PILE INSTALLATIONS EXTEND TO A TOTAL DEPTH OF 31' BELOW THE GROUND SURFACE INSTALL THERMOCOUPLE ASSEMBLIES IN DRILLED HOLES PRIOR TO STEAM JETTING HOLES FOR PILING

A

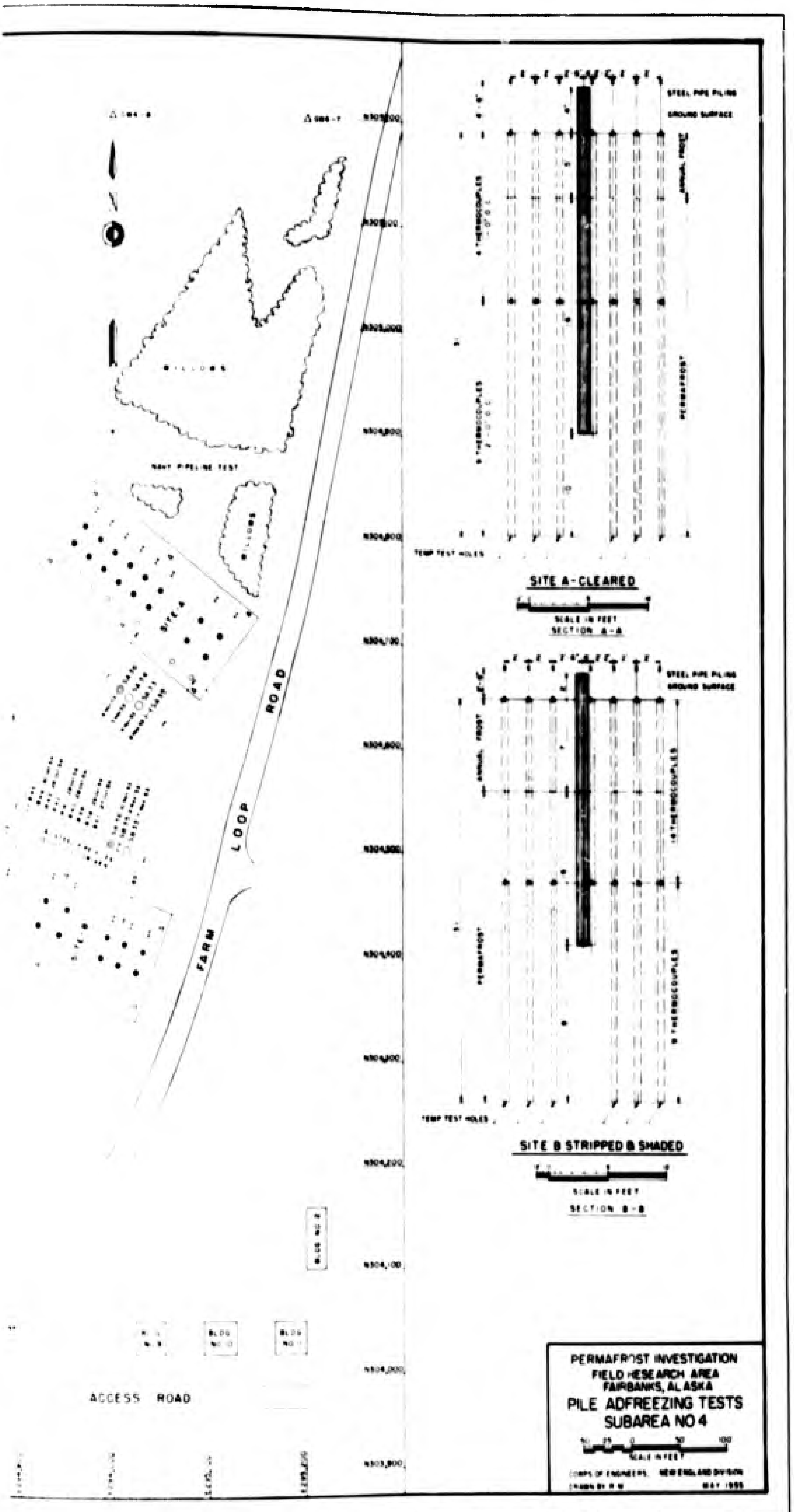
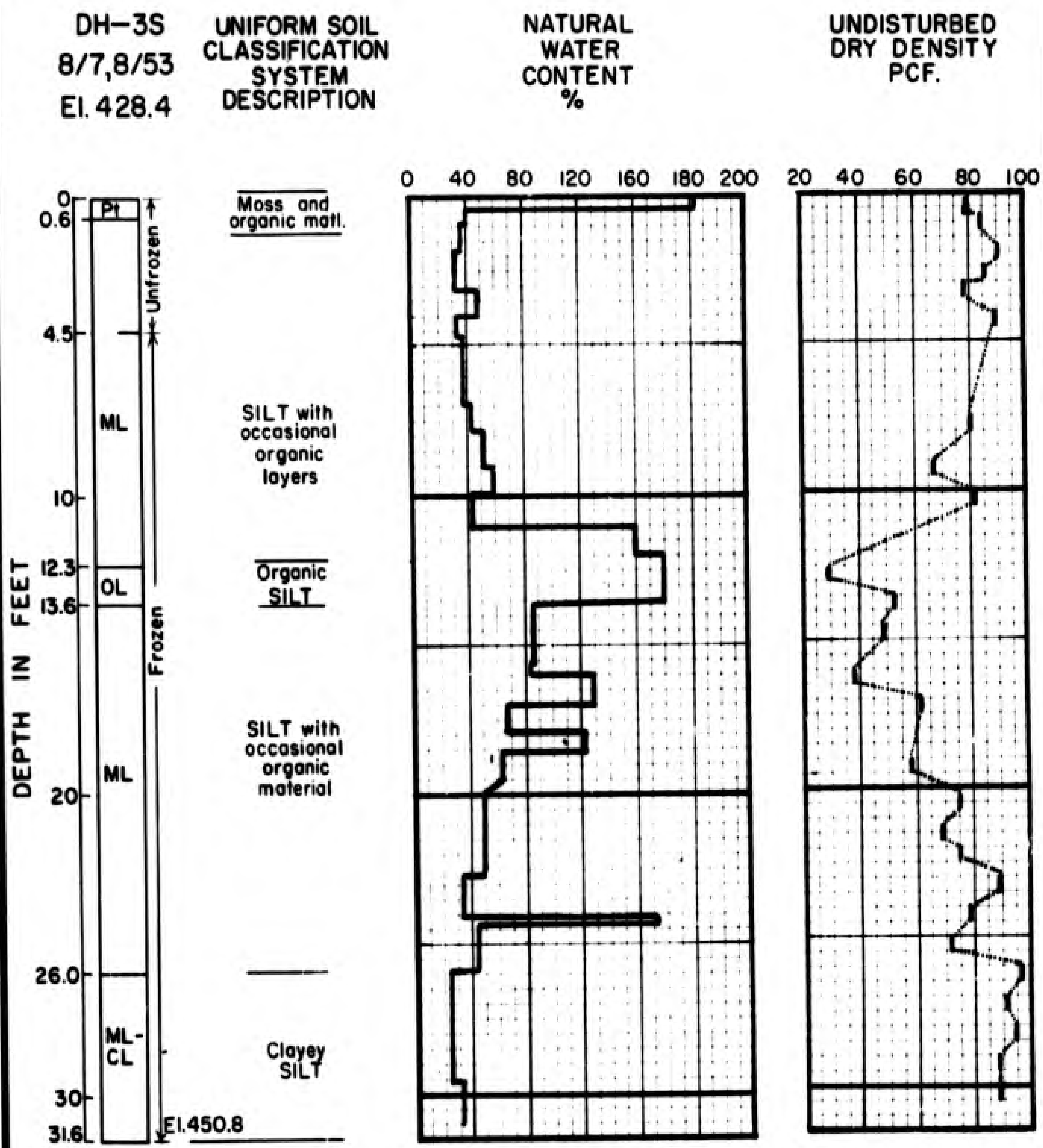

STEEL PIPE PILING
GROUND SURFACE
PERMAFROST
TEMP TEST HOLES
SITE A-CLEARED
SCALE IN FEET
SECTION A-A
SITE B STRIPPED & SHADED
SCALE IN FEET
SECTION B-B
NAVY PIPELINE TEST
WILLOWS
FARM
LOOP
ROAD
ACCESS ROAD
PERMAFROST INVESTIGATION
FIELD RESEARCH AREA
FAIRBANKS, ALASKA
PILE ADFREEZING TESTS
SUBAREA NO 4
SCALE IN FEET
CORPS OF ENGINEERS NEW ENGLAND DIVISION
MAY 1955

Plate 1 B



NOTES:

a. 2" Shelby tube samples 0-4.5'
b. 6" I.D. double tube core barrel samples 4.5'-31.6'
c. Sample recovery about 95% of total depth.
d. Organic content determined by loss in weight upon ignition of oven dried sample.

LABORATORY TEST RESULTS

SAMPLE DEPTH FT.	CLASS. SYMBOL	% ORGANIC	L.L.	P.I.
7.5 - 7.8	ML	4.3	38.3	7.5
12.5 - 12.9	OL	10.0	48.3	12.4
19.0 - 19.4	ML	4.1	33.8	9.3
26.7 - 27.0	ML-CL	2.2	28.4	8.1
31.0 - 31.6	ML-CL	3.5	27.3	4.6

SOIL DATA FOR BORING NO DH-3S

Plate ~~FIGURE~~ 2

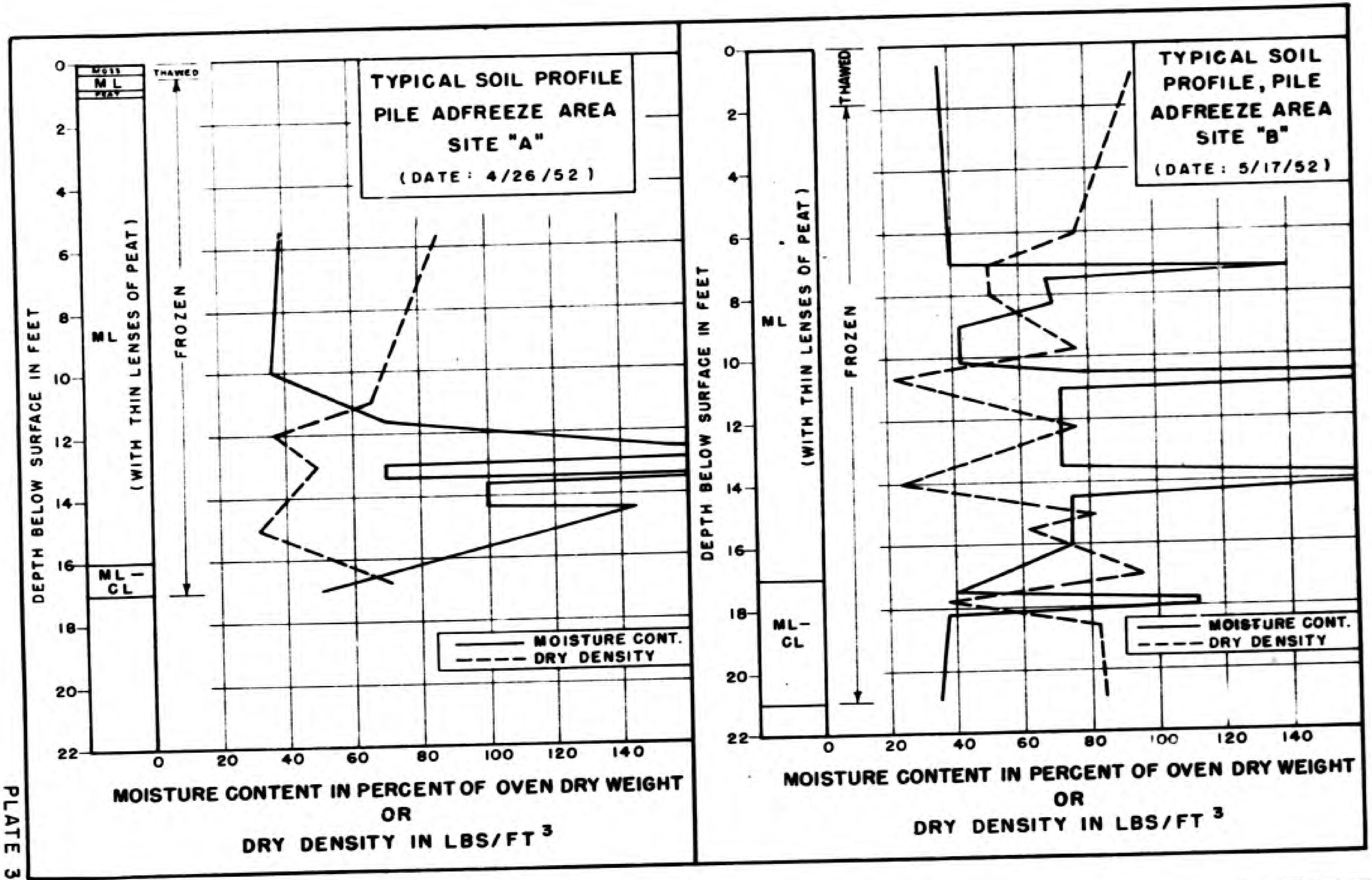
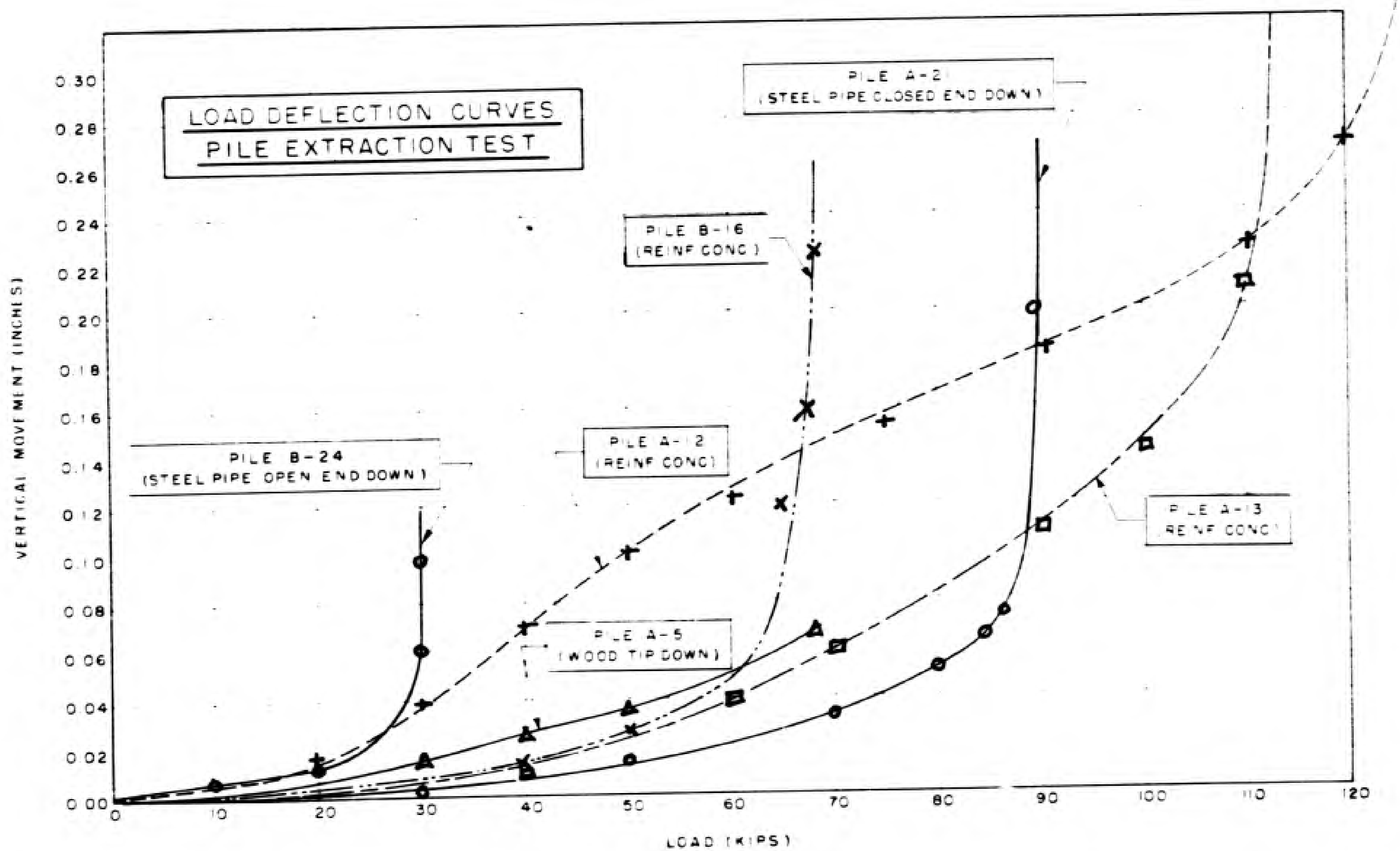
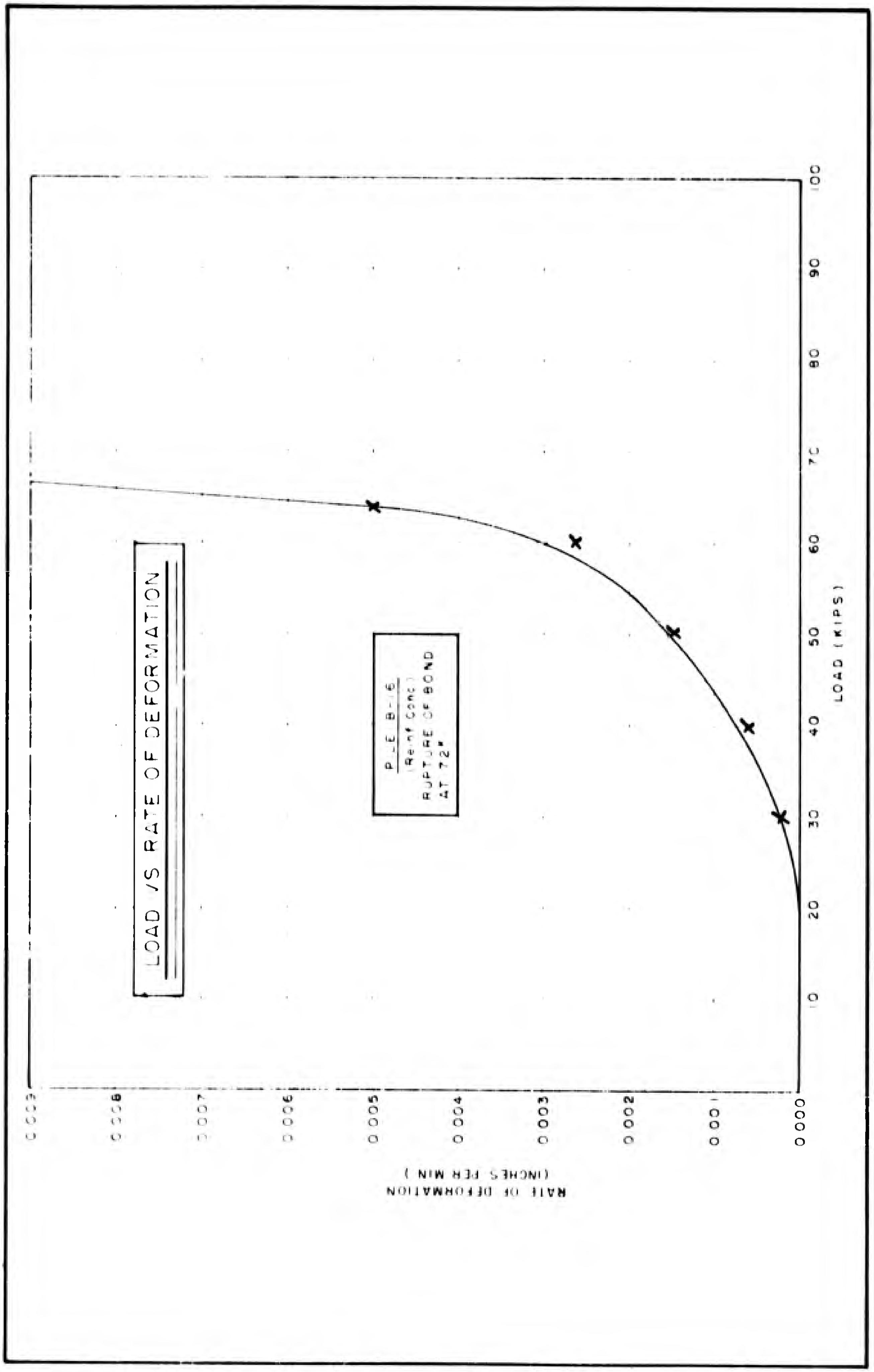
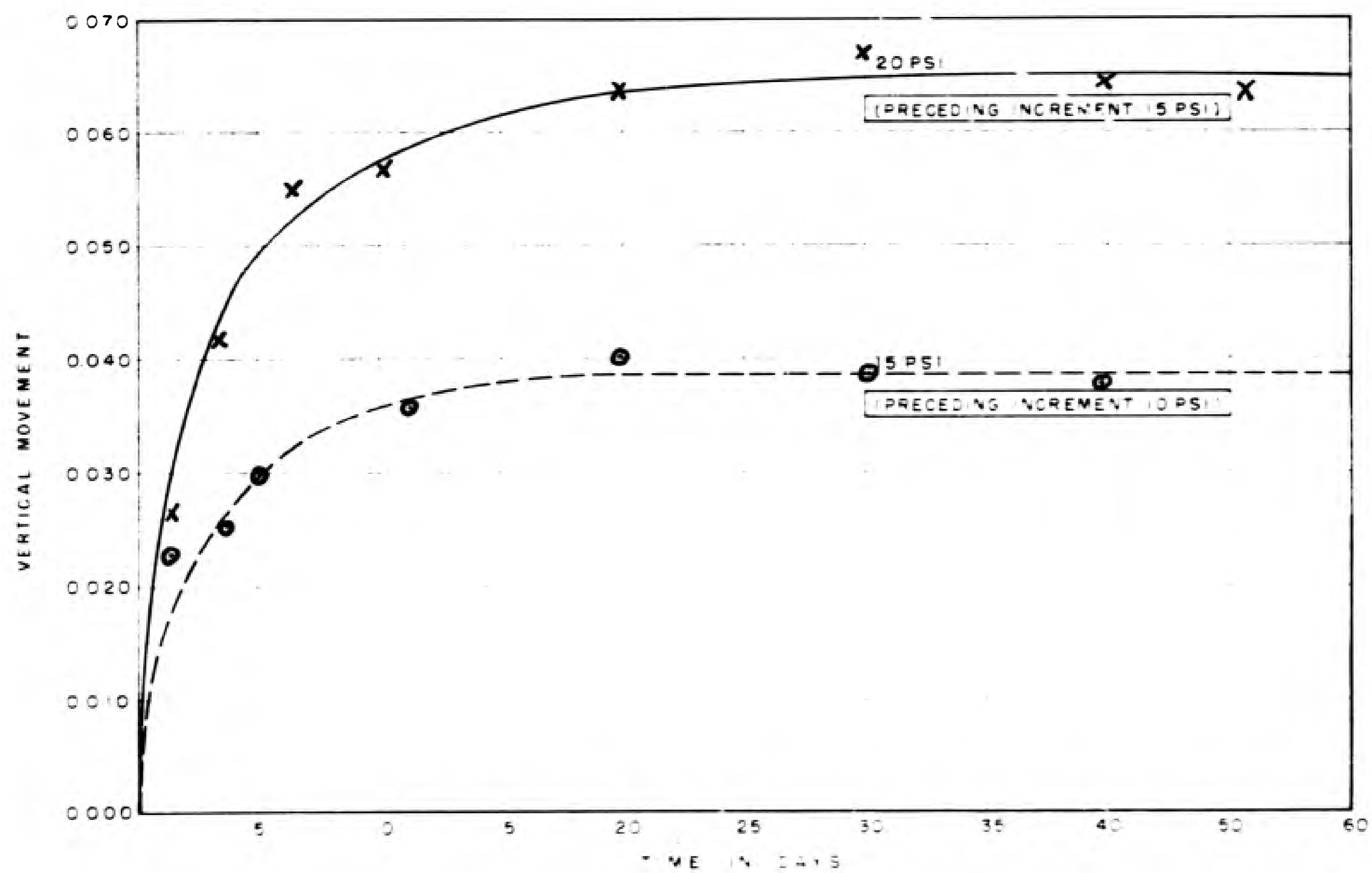

TYPICAL SOIL PROFILE
PILE ADFREEZE AREA
SITE "A"
(DATE: 4/26/52)
DEPTH BELOW SURFACE IN FEET
MOSS
ML
PEAT
ML
(WITH THIN LENSES OF PEAT)
ML-CL
THAWED
FROZEN
MOISTURE CONT.
DRY DENSITY
MOISTURE CONTENT IN PERCENT OF OVEN DRY WEIGHT
OR
DRY DENSITY IN LBS/FT 3
TYPICAL SOIL PROFILE, PILE
ADFREEZE AREA
SITE "B"
(DATE: 5/17/52)
DEPTH BELOW SURFACE IN FEET
ML
(WITH THIN LENSES OF PEAT)
ML-CL
THAWED
FROZEN
MOISTURE CONT.
DRY DENSITY
MOISTURE CONTENT IN PERCENT OF OVEN DRY WEIGHT
OR
DRY DENSITY IN LBS/FT 3

PLATE 3


LOAD DEFLECTION CURVES
PILE EXTRACTION TEST
PILE A-2 (STEEL PIPE CLOSED END DOWN)
PILE B-16 (REINF CONC)
PILE A-12 (REINF CONC)
PILE B-24 (STEEL PIPE OPEN END DOWN)
PILE A-13 REINF CONC
PILE A-5 (WOOD TIP DOWN)
VERTICAL MOVEMENT (INCHES)
LOAD (KIPS)
0.00
0.02
0.04
0.06
0.08
0.10
0.12
0.14
0.16
0.18
0.20
0.22
0.24
0.26
0.28
0.30
0
10
20
30
40
50
60
70
80
90
100
110
120



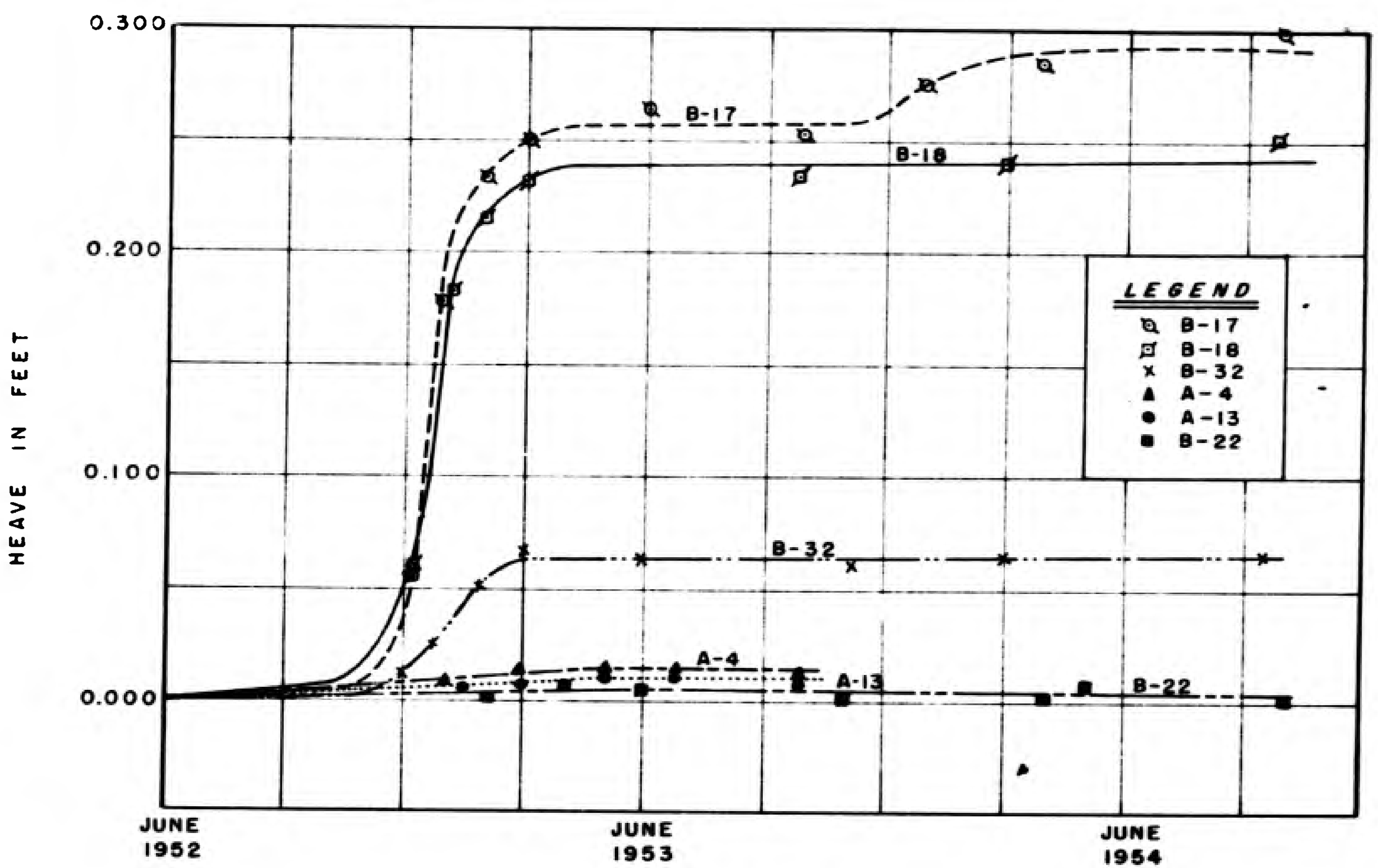
LOAD VS RATE OF DEFORMATION
(Reinf Conc)
RUPTURE OF BOND
AT 72"
LOAD (KIPS)
10
20
30
40
50
60
70
80
90
100
RATE OF DEFORMATION
(INCHES PER MIN)
0.000
0.001
0.002
0.003
0.004
0.005
0.006
0.007
0.008
0.009



PILE EXTRACTION TEST

PILE B25

LONG TERM LOADING



PILE EXTRACTION TEST

HEAVE VS TIME

FOR TYPICAL PILES

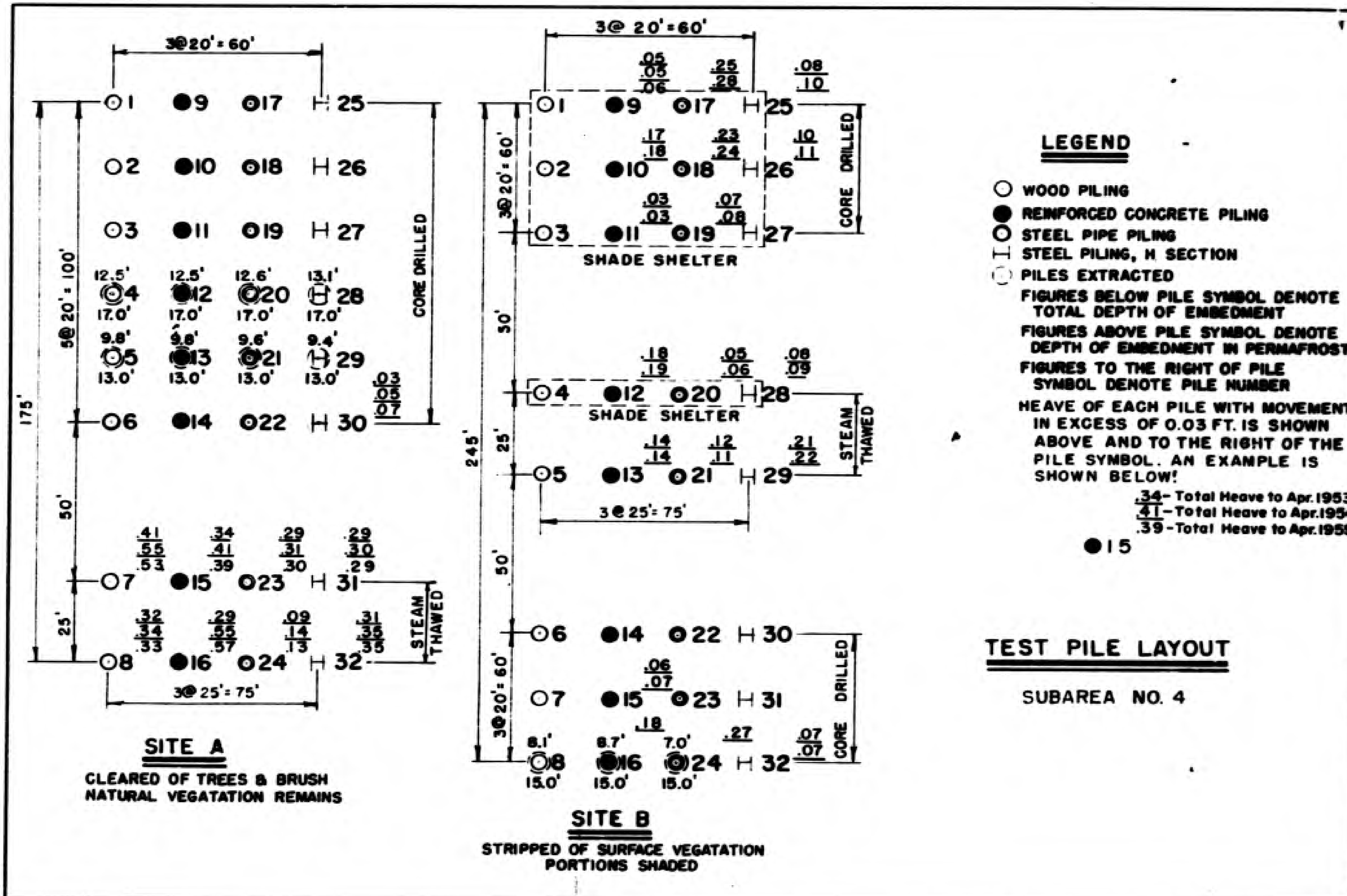
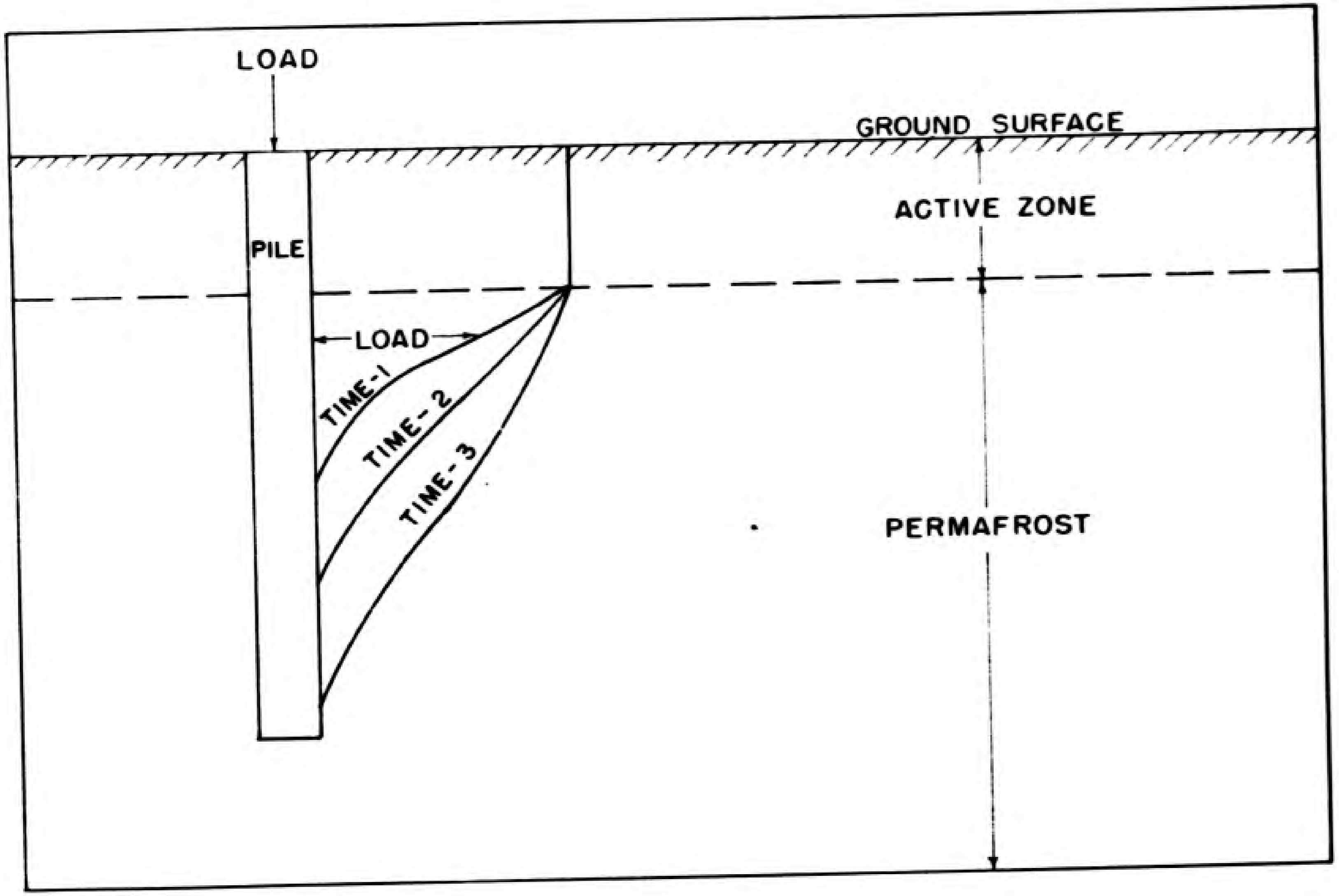

LEGEND
WOOD PILING
REINFORCED CONCRETE PILING
STEEL PIPE PILING
STEEL PILING, H SECTION
PILES EXTRACTED
FIGURES BELOW PILE SYMBOL DENOTE TOTAL DEPTH OF EMBEDMENT
FIGURES ABOVE PILE SYMBOL DENOTE DEPTH OF EMBEDMENT IN PERMAFROST
FIGURES TO THE RIGHT OF PILE SYMBOL DENOTE PILE NUMBER
HEAVE OF EACH PILE WITH MOVEMENT IN EXCESS OF 0.03 FT. IS SHOWN ABOVE AND TO THE RIGHT OF THE PILE SYMBOL. AN EXAMPLE IS SHOWN BELOW!
.34-Total Heave to Apr.1953
.41-Total Heave to Apr.1954
.39-Total Heave to Apr.1955
TEST PILE LAYOUT
SUBAREA NO. 4
SITE A
CLEARED OF TREES & BRUSH
NATURAL VEGATATION REMAINS
SITE B
STRIPPED OF SURFACE VEGATATION
PORTIONS SHADED
SHADE SHELTER
CORE DRILLED
STEAM THAWED
3@20'=60'
5@20'=100'
3@25'=75'
175'
245'

PLATE 8



ASSUMED CHANGE WITH TIME OF LOAD DISTRIBUTION ALONG PILE SHAFT