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INTERIM REPORT ON LOAD TESTS OF PILES IN PERMAFROST

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In temperate climates, pile foundations have been used for centuries to transfer loads of structures to suitable earth and rock foundation materials, where the surface soils are too compressible or too unstable to carry the loads directly. In permafrost areas, piles can be similarly used to transmit loads through the unstable active zone to underlying high bearing capacity permanently frozen ground. However, certain special considerations apply to use of piles in arctic areas, 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. This uplift may be several inches each winter. When the piles penetrate into permafrost, but in an inadequate amount, 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 the pile below the active zone remaining firmly gripped by the permafrost throughout the thaw period. The result may be very severe structural distortion and damage or excessive maintenance cost. 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 strengths.

In order to develop and improve engineering design criteria for construction of pile foundations in areas of frozen ground, the Arctic Construction and Frost Effects Laboratory is carrying out a program of pile tests at its Fairbanks Research Area on Farm Loop Road near Fairbanks, Alaska. Mr. F. F. Kitze, Civil Engineer, is in charge of the work at the Research Area. He is assisted by Mr. S. C. Burnham, Soils Engineer. The research program includes study of the manner in which load supporting capacity is developed in piles supported in permafrost and measurement of skin friction and end bearing values under various combinations of conditions.

The test pile layout at the Research Area is shown on Figure 1. The area is divided into two sites, namely: Site A, which was cleared only of trees and brush; and Site B, from which all vegetation and the surface layer of organic material were stripped. During the period from 15 April to 15 August 1952, 32 piles were installed to 8, 12 and 16-foot nominal depths of embedment 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. The remaining eight piles in each site were installed in steam-thawed holes. The annular spaces around all the piles were filled with silt-water slurry. During the fall of 1952, wooden deck-type shade structures were erected over two groups of piles in Site B as shown on Figure 1, in order to simulate the shading effect of a building.

Photograph #1 shows a typical concrete pile. Photograph #2 is a typical view of Area A, with a concrete pile in the foreground and other types in the background. As shown on Figure 2, the natural soil underlying the pile test area to a depth in excess of 50 ft. is silt, with a variable content of organic material, including occasional layers of peat. Under natural surface conditions, the maximum depth of seasonal thaw varies from 2 to 6 ft. The permafrost layer at the site is of the order of 100 to 150 ft. in thickness. The mean annual temperature at Fairbanks, here taken as typical for the site, is about 26°F., with extremes of $+88^{\circ}$F., and -55°F. The total annual precipitation is about 12 inches, including an average annual snowfall of about 4 ft.

A pile develops supporting capacity from two sources: (1) from end bearing on the point of the pile and (2) from skin friction over the surface of the pile embedded in soil. The supporting ability of a given pile may be obtained largely from either one or the other of these factors, or from a combination of both. In most of the pile tests performed thus far at the Fairbanks Research Area, the piles have been pulled upward out of the ground in order to measure the skin friction component, separately from the end bearing component.

The pile tests were started in the fall of 1953 when extraction tests were conducted on 11 piles. The test procedure was to apply loads in increments, which were held for periods which varied from 5 to 30 minutes while rate of vertical movement was observed. The more or less arbitrary assumption was made that plastic yield was occurring when the rate of movement continued at the end of 30 minutes at 0.001 or more inches per minute (0.002 inches per minute in initial tests). Each test was completed in one to two days. Photograph #3 shows the pulling arrangement.

Seven of the initial group of eleven piles were frozen in so securely that they failed structurally before failure in bond between pile and soil could occur. Typical load-deflection curves obtained in this 1953 series of tests are shown on Figure 3. It will be seen that the curves are not straight but show progressively increasing deflection as the loads increase. Further, the rate of deformation becomes very important at the higher loads. In test of pile B-16, as shown on Figure 4, continuous plastic flow began between 20 and 30 kip loadings, increased, and became very much more rapid at loads of the order of 60,000 lbs. or more. This progressive yield of the pile in the 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 over 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.

Tests were run on an additional 21 piles in the fall and winter of 1954. All but one of these tests were run using short-term loading procedures, modified somewhat from those used at the start of the preceding year. Two of these piles failed structurally before tangential shear failure could be achieved, and two of them reached the maximum load capacity of the loading apparatus without reaching either failure in soil or in pile.

As the final test of the 1954 series, it was decided to investigate the plastic flow characteristics of the pile failures in greater detail, since analysis of the results achieved up to that time had left some doubt as to the level of shear stress which could be tolerated for the life of a structure without excessive accumulative movement. For example, a movement of only 0.01" per day accumulates to 3.65 inches if continued for a year and about 3 ft. if continued for 10 years. It is obvious from this that in order to complete tests of this nature in a reasonable time, that is in less than the lifetime of a structure, that movements must be measured with a degree of precision which is exceptional for this type of field test.

Loading of test ~~xx~~ 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 and ~~25~~ 20 psi were successively applied. Each load increment was maintained for a period of 30 days or more; the movement of the pile was measured to 1/1000 inch with dial gage extensometers. A plot of the load-deflection data from this test for 15 and 20 psi load increments is shown on Figure 5. It is seen that after a few days of gradual adjustment, the pile became stable at both these stress increments. It is expected, however, that as further load increments are added a level will be reached at which adjustment to a stationary condition will not occur.

Difficulties have been experienced in applying tensile forces to the piles. In the original tests, for example, a slot was cut through the upper end of the pile and a pulling bar passed back through it. The result was often failure of the top of the pile as shown on Photos Nos. 4 & 5, before bond failure 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 has been constructed for the wooden piles which avoids the weakening effect of cutting a 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 foundation 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, it will be possible to plan future tests in the program so as to keep depths of embedment and stresses in piles at practical levels.

It has also been found at the Research Area that placing of the slurry around the piles is a critical operation if skin friction is to be depended upon for bearing capacity. 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.

More than half of the 64 piles installed in the summer of 1952, have shown no progressive movement since installation. However, about a third showed an inch or more of cumulative heave two years after construction, particularly those installed in steam-thawed holes. Wood pile A7 heaved nearly 5 inches in the first winter. Although there was very marked general reduction in heaving in the second winter, some in steam-thawed holes still heaved 2 or 3 inches in this second winter season. However, by the third winter even the piles installed in steam-thawed holes had become stabilized. 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. **Figure 6 shows typical heave versus time plots.**

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 Pt. Barrow, Alaska, or Thule, Greenland, much more positive freeze-back should occur.

It is usually impractical to wait for periods up to a half a year or more for piles to become solidly frozen in, before construction can proceed. It presently appears that, under the soil and permafrost conditions of the Fairbanks Research Area, and using the pile types and installation techniques here employed, reliance cannot be placed on all piles of a foundation to be stable until after one or two winters, depending on the installation method used, unless artificial refrigeration is employed to hasten attainment of solid freeze-back. However, further study of the nature of permafrost, of other pile types, of refinements in installation techniques and of the nature of the freeze-back process may yield new approaches.

Figure 7 illustrates a concept of the changes in load distribution with time, 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 design of pile foundations in temperate regions; however, the special physical properties of frozen ground make the analysis of pile bearing capacity in the Arctic and Subarctic a unique problem.

The ultimate adfreeze bond strength has varied from a low of 13.0 psi to a high of 31.3 psi in tests completed to date; in drilled holes, averages thus far are 22.5 psi for concrete piles, 30.7 psi for steel pipe piles, and 28.4 psi for steel I-beam piles. Driven pipe piles have averaged 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 of the order of 1-1/2 to 5-1/2 psi for the soil in the thawed state. Stresses at significant plastic yield, as determined by our more or less arbitrary definition of 0.001 inch per minute movement after 30 minutes, averaged about 80% of the bond failure stress for the drilled and steam-thawed holes, with individual values varying from 65% to 100%. *

In order to establish pile design criteria which are applicable under various possible 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. It is therefore planned as part of the continuation of this program to install additional test piles at the Research Area, especially instrumented with strain gages and with load pressure cells. 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 economical methods for speeding this process and making it more positive. Various other investigations are planned to more fully develop the possibilities of pile foundations in areas of frozen ground, including study of special pile types.

* "Subsequent to the presentation of this paper, additional tests have been completed. In some of these tests, slurried piles have shown ultimate skin friction values lower than 5 psi, under certain adverse freeze-back conditions. The values given in this paper should not be interpreted in any way as design working stresses."