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BRIDGE FOUNDATIONS IN PERMAFROST AREAS

Moose and Spinach Creeks
Fairbanks, Alaska

Frederick E. Crory

July 1975

PREPARED FOR
ALASKA DEPARTMENT OF HIGHWAYS
IN COOPERATION WITH
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BY

CORPS OF ENGINEERS, U.S. ARMY
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Under a joint research project between the Alaska Department of Highways and the U.S. Army Cold Regions Research and Engineering Laboratory, cooperative field observations and tests were conducted during and following construction of the Moose and Spinach Creek bridges, Fairbanks, Alaska. Site investigations and bridge foundation designs of the Alaska Department of Highways, bridge pile installation data, and ground temperature conditions for a one-year period are presented. Two test piles and three anchor piles were installed in close proximity to the Moose Creek bridge and load settlement tests were performed. The capacity of a sand-water slurried test pile was less than 10 tons, while that of an adjacent driven pile was about 45 tons. Greater capacities could have been easily achieved by driving the piles

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20. Abstract (cont'd)

to bedrock rather than a specified elevation. To prevent frost heaving of the shallow piles at Spinach Creek an anti-heaving soil-oil-wax mixture was employed to a depth of 10 feet.

PREFACE

The study of piles in permafrost for bridge foundations reported here resulted from a 1964 memorandum of understanding for a cooperative research and investigational program between the Alaska Department of Highways and the U.S. Army Cold Regions Research and Engineering Laboratory. CRREL's contribution to the study was made possible under former Projects 19 and 53 (1964-1966) and Task 23 of Military Construction Investigations. The Military Construction Investigations are a part of the Engineering Criteria and Investigations and Studies for cold regions, conducted by CRREL for the Office, Chief of Engineers, Directorate of Military Construction, Engineering Division, Advanced Technology Branch. The program is administered by the Civil Engineering Section of the latter office.

The Alaska Department of Highways conducts the investigation under authority and funds granted by the Bureau of Public Roads (now Federal Highway Administration) for Research Project FR-15 (P12270), under the direction of W. Neimi, Regional Engineer, Bureau of Public Roads, Juneau, Alaska. This study would not be possible without the generous support and cooperation of the Bureau of Public Roads. No funds have been exchanged between CRREL and the Alaska Department of Highways in this study.

This report was prepared, in accordance with the memorandum of understanding, by F.E. Crory, Chief of the Foundations and Materials Research Branch, Experimental Engineering Division, USA CRREL. Bruce Campbell was Commissioner of Highways during the final portion of this investigation, H. Mahon was Planning Director and the late H. Golub was Chief Bridge Engineer.

In view of the large number of personnel (present and former) engaged in various aspects of this study, only a general acknowledgment to CRREL's Alaska Field Station (now Alaskan Projects Office) and the Alaska Department of Highways Road Materials Laboratory and Fairbanks District Office is possible.

Acknowledgment must, however, be made of the substantial support rendered by J. Tiemesson, Resident Engineer, and L. Voss and T. McFetridge, Engineering Aides, who performed the observations at the pile tests during the night shifts. The contributions of E. Lutzen and G. Utermohle, Foundation Geologists, R. Sherman, Chief Geologist, and L. Trent, Laboratory Technician of the Road Materials Laboratory are gratefully acknowledged. The contributions of W. Tizzard of CRREL during the pile installation and of the late D. Townsend, who assisted the author in the pile test program, are also gratefully acknowledged.

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Multiply	*By*	*To obtain*
°F	5/9 (°F-32)	°C
miles	1.609	km
ft	.3048	m
in.	25.4	mm
tons (short)	907.185	kg
lb	0.45359237	kg
psi	0.070307	kg/cm^2
	6894.757	Pa

BRIDGE FOUNDATIONS IN PERMAFROST AREAS
Moose and Spinach Creeks, Fairbanks, Alaska

by

Frederick E. Crory

INTRODUCTION

The installation and performance of piles in permafrost for bridge foundations are being studied in a joint research program between the Alaska Department of Highways and the U.S. Army Cold Regions Research and Engineering Laboratory. The purpose of this study is to develop and improve design criteria for piles in permafrost by: 1) verifying and extending existing data and criteria on installation methods, load capacity and frost heave control techniques for bridge foundation piles in frozen ground; and 2) observing long-term performance of bridge foundation piles with respect to ground temperature and vertical displacement. The study is to be accomplished in conjunction with actual construction projects selected to encompass a variety of soil conditions and permafrost temperatures.

This report presents the site investigations and bridge foundation designs of the Alaska Department of Highways, bridge pile installation data, and ground temperature conditions for the 15-month period following construction of the Moose and Spinach Creek bridges. Data on the installation of two test piles and three anchor piles in the proximity of the Moose Creek bridge and the results of load settlement tests are included. A companion report (Crory 1968) covers the investigation and construction of the nearby Goldstream Creek bridge. The performance of these three bridges over a ten-year period following construction will be covered in a separate report.

Location

The Moose and Spinach Creek bridges are located on the new Defense Access Road, Route R-AD-3 (1), to Murphy Dome Air Force Station about 1.8 and 8.1 miles, respectively, from the easterly junction with the Sheep Creek Road (Fig. 1). The junction of these two highways is approximately 7.0 statute (air) miles northwest of Fairbanks, Alaska. The Defense Access Road was constructed through virgin territory about 1 mile north of and parallel to Goldstream Creek. The Alaska Railroad from Fairbanks to Anchorage runs midway between this creek and the new access road. The road, unsurfaced except for a 6-in. wearing surface of select material, crosses a series of silt-capped ridges, the valleys of which drain southward to Goldstream Creek. Construction of the Sheep Creek Road and the Defense Access Road was started in 1964. Bridges at Goldstream, Moose and Spinach Creeks were completed in late August 1965.

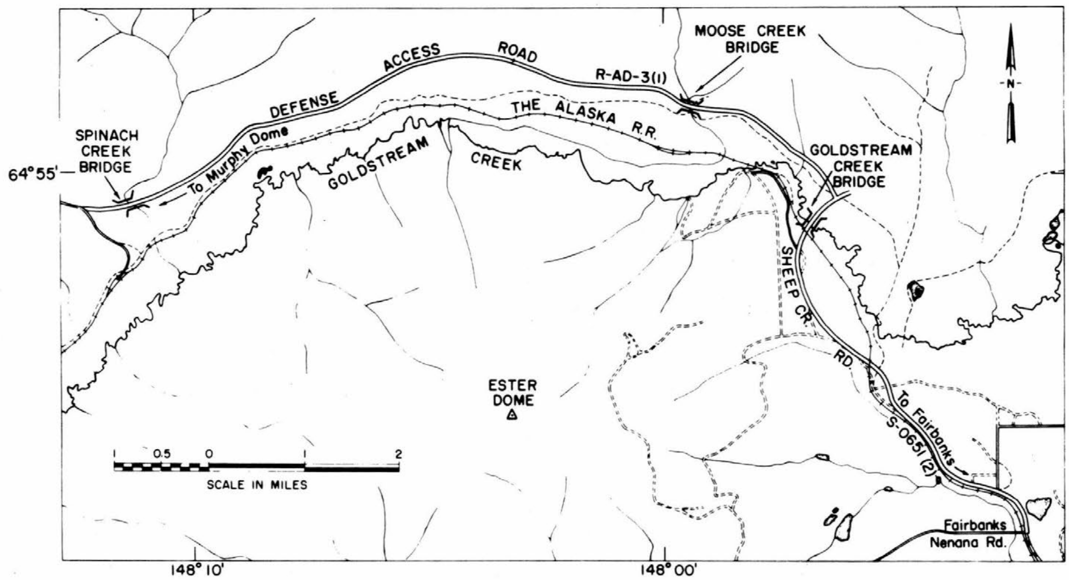


Figure 1. Location map.

Climate (U.S. Weather Bureau, 1964)

The climate in the Fairbanks area is typical of interior Alaska, well sheltered from maritime influences by mountain ranges on all sides. The Tanana Valley area has a definite continental climate, conditioned in a large measure by the ready response of the land mass to variations in solar heat received throughout the year. During June and July the sun is above the horizon from 18 to 21 hours each day and daily average maximum temperatures reach the lower 70's. Extreme highs of 90°F or more have occurred in May, June and July. During the period from November to March, when sunshine periods range from 10 to less than 4 hours, the lowest temperatures normally fall below zero quite regularly and extremes near or below −60°F have occurred in three mid-winter months. The average last day of freezing temperatures in the spring is 21 May; the average first occurrence of freezing temperatures in the fall is 30 August.

The amount of cloudiness is quite low, particularly from February through April. Prevailing winds are from the southwest during June and July, and from the north and northwest during the remainder of the year. Wind speeds are light during winter months. Ice fog, which frequently and persistently occurs over the city and Ft. Wainwright during periods of extremely low temperatures, is rare in the area surrounding the city. Precipitation follows a fairly regular pattern with total annual precipitation about 12 in., which must be classified as relatively light. This includes a mean annual snowfall of about 40 in. Maximum precipitation, in the form of rain, occurs in August. Normally precipitation decreases from September through December; however, snowfall increases in late December and reaches a maximum in January. April has the least precipitation and the greatest percentage of possible sunshine.

The persistent snow cover during winter months is a major factor contributing to extremely low air temperatures. The white snow surface acts as an insulator which minimizes heat loss from the ground to the air, and the white snow surface prevents the absorption of solar energy from the limited sunshine available during the winter months. During December and January, maximum air temperatures are usually below zero.

Fairbanks International Airport has a mean annual air temperature of about 26°F. The average air freezing and thawing indexes for the years 1946-1972 were 5790 and 3390 degree-days Fahrenheit, respectively.

MOOSE CREEK BRIDGE

General geology and site conditions (Utermohle 1964[2])

While the Moose and Spinach Creek sites are similar in many respects, each bridge will be discussed separately to avoid confusion. The two creeks provide the major drainage from the hills along and to the north of the Defense Access Road. Moose Creek is in rolling country, the entire area being underlain by Birch Creek schist which near the surface exhibits various degrees of weathering. A large hill with exposed bedrock is only about 150 yards northeast of the bridge site. The hill was used as a source of rock borrow during construction. While the hills are generally covered with a loess mantle the low areas such as Moose Creek are normally an organic silt. Just upstream there is a wide, swampy area. The site was originally covered with scrub spruce, alders, birch and occasional stands of tall spruce.

For a more detailed description of the geology of the area, see Geologic Maps of the Fairbanks Area, D-2 Quadrangle, Alaska (Péwé 1958).

While the normal summer flow of the creek is very small, a considerable flow is experienced during the spring runoff period. During the preliminary site investigations during the summer and fall of 1963, Moose Creek was found to be flowing in a shallow, well drained channel, approximately 6 to 10 ft wide. In mid-December 1963, however, the stream developed an icing extending out of its channel and measuring approximately 7 ft thick (Fig. 2). By May 1964 the icing extended more than ½ mile downstream of the bridge site, with the ice almost touching the ties of the railroad bridge. The railroad bridge (Fig. 1) is 8 to 10 ft above the stream bottom. Similar icings at this and other railroad bridges in this area had been previously reported (Péwé and Paige 1963).

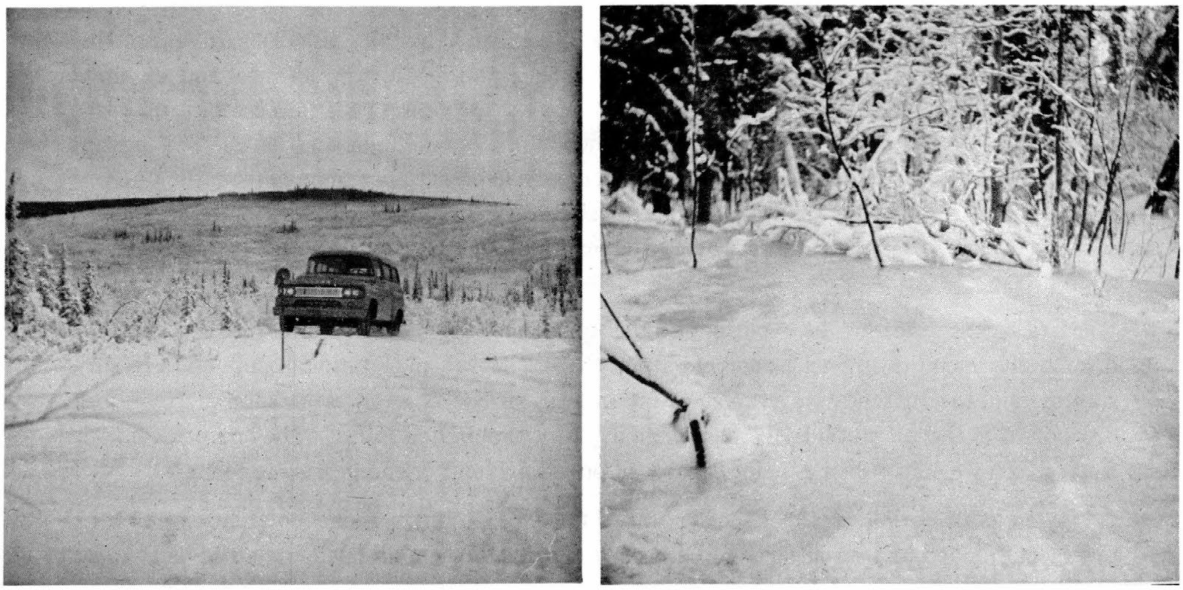


Figure 2. Icings at Moose Creek prior to construction.

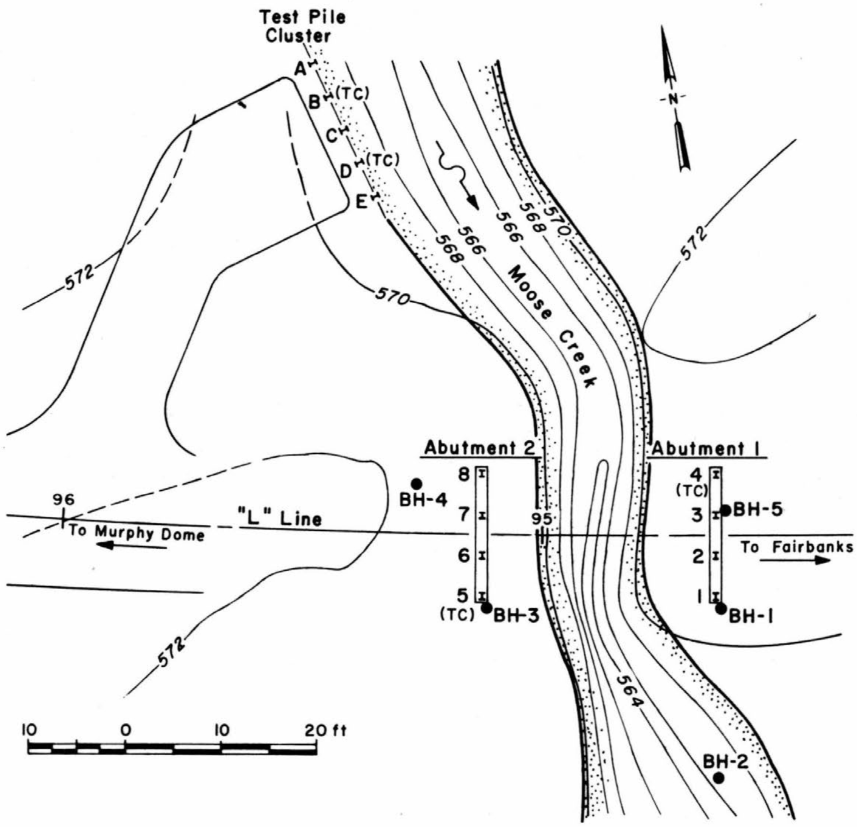


Figure 3. Foundation plan and test pile cluster, Moose Creek. BH-1–4 are test borings. BH-5 is standard penetration test.

Soil investigations (Utermohle 1964[2])

Soil explorations for the bridge foundations were conducted by the Alaska Department of Highways, Road Materials Laboratory, in November and December 1963, using an auger and a hydraulic feed rotary core drill, with water as the drilling fluid. The locations of the four borings and one standard penetration test at Moose Creek are shown on the foundation plan (Fig. 3). Soil logs and the results of the penetration test are given in Figure 4. Representative samples from the four explorations were tested at the Road Materials Laboratory at Fairbanks to determine natural water content, dry unit weight, specific gravity, Atterberg limits and grain size distribution. Results are given in Table I.

Based on the results of the explorations, the soils at the Moose Creek site may be described as organic silts from the original ground elevations (570-572 ft) to an elevation of about 555 ft; silty gravelly sand to silty sandy gravel to an elevation of about 535 ft; and weathered micaceous schist bedrock from that elevation to the maximum depth of exploration (50 ft). The bedrock contained zones which were described as being capable of providing point or end bearing to piling.

Frozen ground at Moose Creek, at the time of the explorations, was encountered 3 to 8.5 ft below the natural ground surface. The frozen portion of the organic silt layer contained an abundance of small ice lenses or crystals $^1/_8$ to 1 in. in length, to a depth of about 15 ft. The sand and gravel

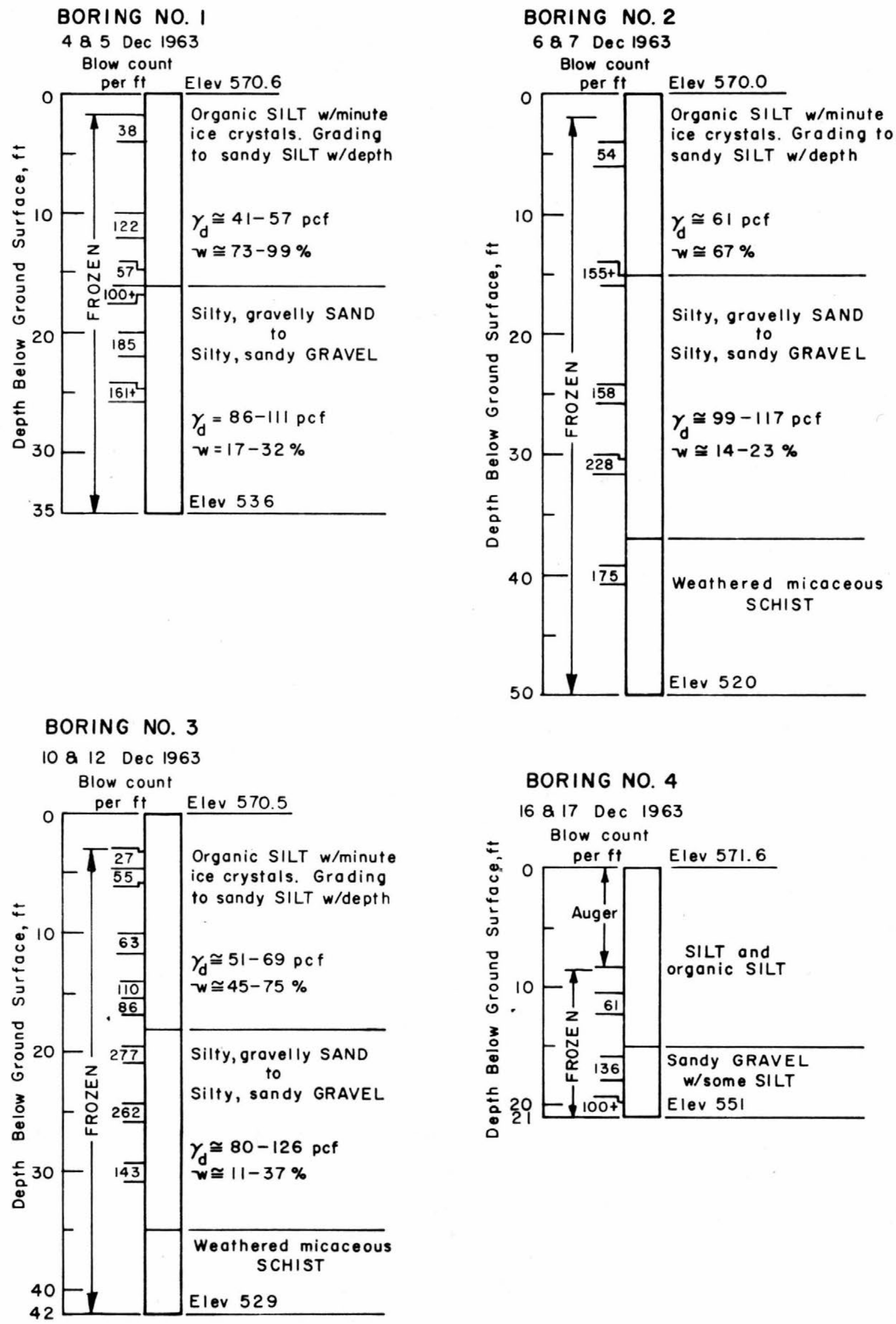


Figure 4. Boring logs, Moose Creek.

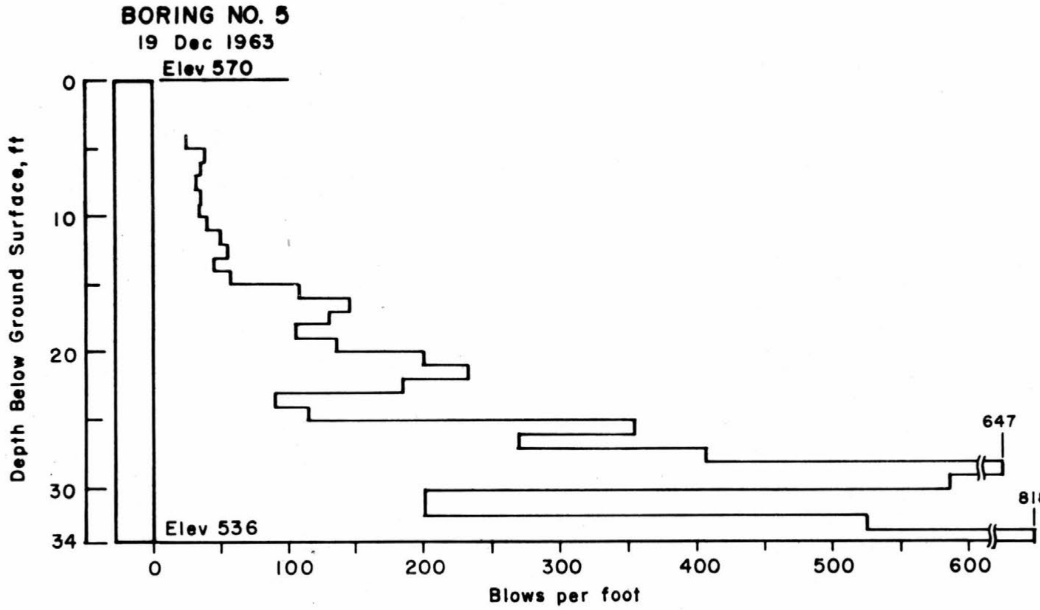


Figure 4 (cont'd). Boring logs, Moose Creek.

stratum and the schist bedrock were generally interstitially frozen with occasional ice crystals being noted. A thermocouple assembly was installed in boring No. 2 and temperatures a month after installation indicated that freezeback had still not occurred. The slow rate of freezeback was thought to be due to the warm drill water (about 40°F) and to the marginal permafrost conditions.

Foundation design

The following foundation recommendations were given in the Foundation Geologist's report (Utermohle 1964[2]), based on the soil and permafrost conditions encountered at the site:

1. "H" piles, driven to practical refusal, should be used. Due to anticipated heavy driving in the sand and gravel stratum, a heavy section (10BP 57 or larger) with a reinforced tip is recommended.

2. A minimum pile tip elevation of 543 ft is specified. Estimated pile length is 37 ft (to bedrock, about elevation 533 ft).

3. Once the driving of a pile begins, it should be continued until the desired depth or bearing is obtained. Preboring or augering of piles may be necessary in order to obtain the specified tip elevation. If piles are pre-bored, it is recommended that the annulus around the pile be filled with sand at optimum moisture. Thawing, jetting or boring with water is not to be permitted.

4. Piles should be exposed ("daylighted") a minimum of $^1/_{10}$ the thickness of the abutment fill. The fill in the area of the abutments should be non-frost-susceptible and of A-1-a (HRB) classification.

5. It is anticipated that the jacking action of the seasonal frost on the upper sections of the piles will be counteracted by the adfreeze of the permafrost on the lower sections.

6. At least one thermocouple near each abutment is recommended to observe temperature variations after construction.

Table I. Analysis of foundation investigations, Project No. RAD-3(1), Moose Creek, Bridge 358.

B/F	*Depth /feet*	*F'd no.*	*Lab no.*	*Sieve analysis - percent passing* 2"	1½"	1"	¾	½	⅜	#4	#10	#40	#200	*LL*	*PI*	*Class (HRB)*	*Unif soil class*	*Nat wet*	*Density dry*	*Nat mois*	*Spec grav*
									BORING NO. 1, STATION 94+63, 15′ LT.												
38	2.0-4.0	1	1974									100	84.4	37	NP	A-4(8)	ML-OL	98.1	56.6	73.2	†
122	10.0-12.0	2	1975							100	95	76	44.8	31	NP	A-4(2)	ML-OL	80.5	40.5	98.7	†
57-6″	15.0-16.0	3	1976					100	97	94	88	66	40.4	24	NP	A-4(1)	ML			31.9	
100-.2′	16.0-16.2	4	Insufficient material for testing																		
185	20.0-22.0	5	1977			100	97	95	91	80	67	46	33.5	29	NP	A-2-4	SM			20.1	
161-6″	25.0-26.0	6	1978				100	90	84	62	43	37	25.7	24	NP	A-2-4	SM			16.8	
									BORING NO. 2, STATION 94+63, 45′ LT.												
54	4.0-6.0	1	1979								100	96	81.0	41	NP	A-5(8)	MH-OH	102.0	61.1	66.6	†
155-6″	15.0-16.0	2	1980			100	98	89	83	65	47	33	18.0	24	NP	A-1-b	SC			23.1	
158	25.0-26.5	3	1981	100	98	97	93	88	80	69	54	34	20.9				SM-SC	Gradation only			
288	30.5-31.5	4	1982			100	97	85	79	58	45	35	24.3	35	NP	A-1-b	SC			14.1	
									BORING NO. 3, STATION 95+12, 15′ LT.												
27	3.5-4.5	1	2001									100	94.0	32	NP	A-4(8)	ML-OL	102.1	68.3	47.2	2.47
55	5.0-7.0	2	2002								100	98	82.0	34	NP	A-4(8)	ML-OL	88.6	50.7	74.7	2.36
63	10.0-12.0	3	2003									100	90.2	32	NP	A-4(8)	ML-OL	92.2	64.6	65.4	2.21
196	15.0-16.5	4	2004		100	97	96	90	85	69	50	30	18.6							17.2	
80	16.5-17.5	5	2005				100	97	93	93	92	86	73.4							36.7	
277	20.0-22.0	6	2006			100	98	92	84	70	55	36	19.2							18.7	
413	25.0-26.0	7	2007		100	95	93	82	73	58	46	30	15.1							10.5	
143	30.5-31.5	8	2008			100	99	87	80	70	59	41	25.8	16	NP	A-2-4	SM	122.2	105.8	15.3	2.55
									BORING NO. 4, STATION 95+26, 10′ RT.												
Auger	1.5-2.0	1	B-1789								100	93	85	34	5.8	A-4(8)			53*	72.2	2.68
Auger	4.0-4.5	2	B-1787								100	98	92	26	1.5	A-4(8)			80*	37.1	2.72
Auger	5.0-5.5	3	B-1788									100	97	25	NP	A-4(8)			79*	37.9	2.63

All tests by State of Alaska, Road Materials Laboratory.
* Theoretical, based on Nat moisture and Spec gravity.
† Organic noted.

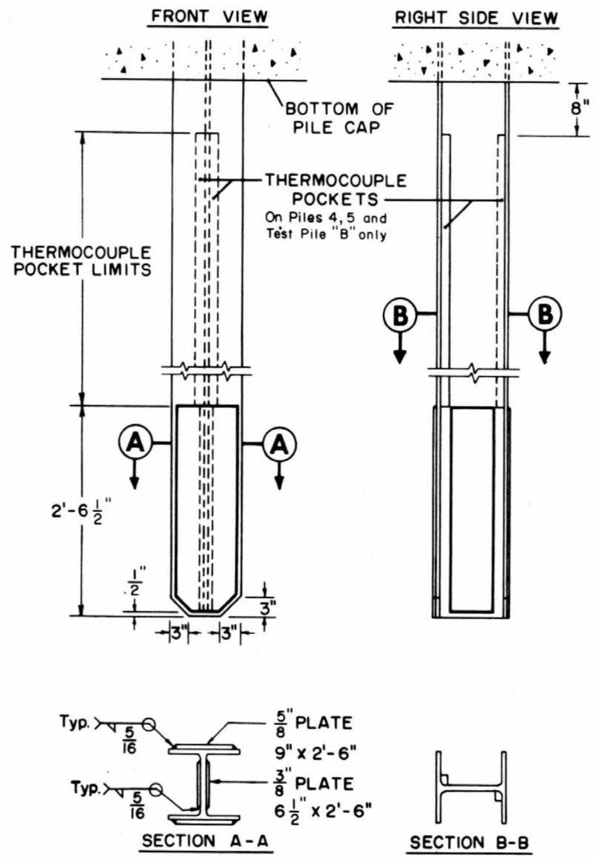


Figure 5. Pile shoe detail.

The final design incorporated all of the above recommendations; specified and estimated tip elevations were 543 ft and 533 ft, respectively.

Piles

The four piles for each abutment of this single span bridge were 10BP57 sections, 8 ft 5 in. on centers, with all pile webs at right angles to the longitudinal axis of the bridge (Fig. 3). All piles were driven vertically and were equipped with shoes. The shoes were fabricated from $^3/_8$- and $^5/_8$-in. plates that extended 2 ft $6^1/_2$ in. up from the tip on the outside of both flanges and both sides of the web, as shown in Figure 5. One pile in each abutment was equipped with two pockets for housing thermocouple assemblies. The pockets consisted of $1 \times 1 \times {}^3/_{16}$ in. angle irons welded to the flange and web, on diagonally opposite sides, and extended from the top of the shoe to 8 in. below the concrete pile caps. Piles equipped with thermocouples are on a NE-SW diagonal to the bridge centerline as shown in Figure 3.

Pile installation

The eight bridge foundation piles were driven with a Delmag D-12 diesel hammer operated in hanging leads supported by a truck-mounted crane. The diesel hammer had an overall weight of 5512 lb, including a 2750-lb piston and a 754-lb anvil. Energy output per blow, as reported by the manufacturer, is 22,500 ft-lb with a maximum explosive pressure on the pile of 93,700 lb. The hammer was operated at rates of 46 to 58 blows per minute, with the major portion of the driving between 53 and 55. No cushion blocks were used as the pile fitted directly into the anvil at the base of the diesel hammer.

Foundation piles were specified to be driven to at least the design tip elevation (543 ft) and to a bearing value of not less than the design load of 45 tons. The calculated dead load was 17.3 tons/pile. For construction control on the project, the safe bearing value of each pile was to be determined using the modified Engineering News Formula for single acting steam or air hammers and diesel hammers, with unrestricted rebound of ram:

$$P = \frac{2WH}{S + 0.1}$$

where P = safe bearing value, lb

W = weight of ram, lb

H = height of fall of ram, ft*

S = average penetration per blow for last 10 to 20 blows of steam, air or diesel hammer, in.

The above formula was considered applicable only when the hammer had true fall, the head of the pile was square and in good condition, and penetration was at a reasonably rapid and uniform rate.

Driving was started on 13 May 1965 at abutment 2 (west) and concluded at abutment 1 on 17 May. The driving records for piles 1, 2, 3 and 4 of abutment 1 are shown in Figure 6. All four piles in abutment 1 had somewhat similar penetration records. Piles 1 and 2 indicate a distinct increase in blow count at elevations 553 to 555, where the soil conditions change from organic silt to sand or gravel. Piles 3 and 4 showed a more uniform increase in blow count with depth, with pile 4 indicating a substantially softer soil layer between elevations 545 and 547 ft. The latter pile was equipped with the two thermocouple pockets. The average blow count for piles in abutment 1 increased 3.6 blows/ft for each foot of penetration, culminating at the specified tip elevation (543 ft) at an average of 97.5 blows/ft. From observations during the last foot of penetration, and the height of the hammer ram during the same period, the bearing capacities of the piles in abutment 1 were computed using the above formula. Theoretical bearing capacities were 85, 102, 73 and 69 tons for piles 1-4, respectively.

Driving records for piles 5, 6, 7 and 8 of abutment 2 are shown in Figure 6. Blow counts for piles 5 and 6 increased from 2 and 11 blows/ft at elevation 563 to 50 and 60 blows/ft at elevations 552 to 555. Both piles then exhibited a decrease in blow count with additional penetration, requiring only 28 to 32 blows/ft at elevation 548. In the last 5 ft of driving, the blow counts of both piles increased rapidly, reaching 97 to 100 blows at elevation 543. Pile 5 was equipped with the thermocouple pockets, like pile 4. There appeared to be good agreement between the driving record of pile 5 and the previously conducted penetration test at borings 3 and 4.

The driving records of piles 7 and 8 differed widely. With the exception of the somewhat harder driving between 556 and 558 ft, pile 7 exhibited a very slow increase in blow count with depth until reaching elevation 550 ft. From 28 blows/ft at elevation 550 the rate then rapidly increased to about 80 blows/ft in the last 3 ft of driving. While pile 8 had a similar or only slightly higher blow count at elevations 556-558, the blow counts continued to increase with depth, reaching 90 blows/ft at elevation 550. From elevation 550 to 543 the blow count varied from 74 to 100 blows/ft.

* For a diesel hammer with unrestricted rebound of ram, this value is the average height of fall for the blows used to determine average penetration per blow.

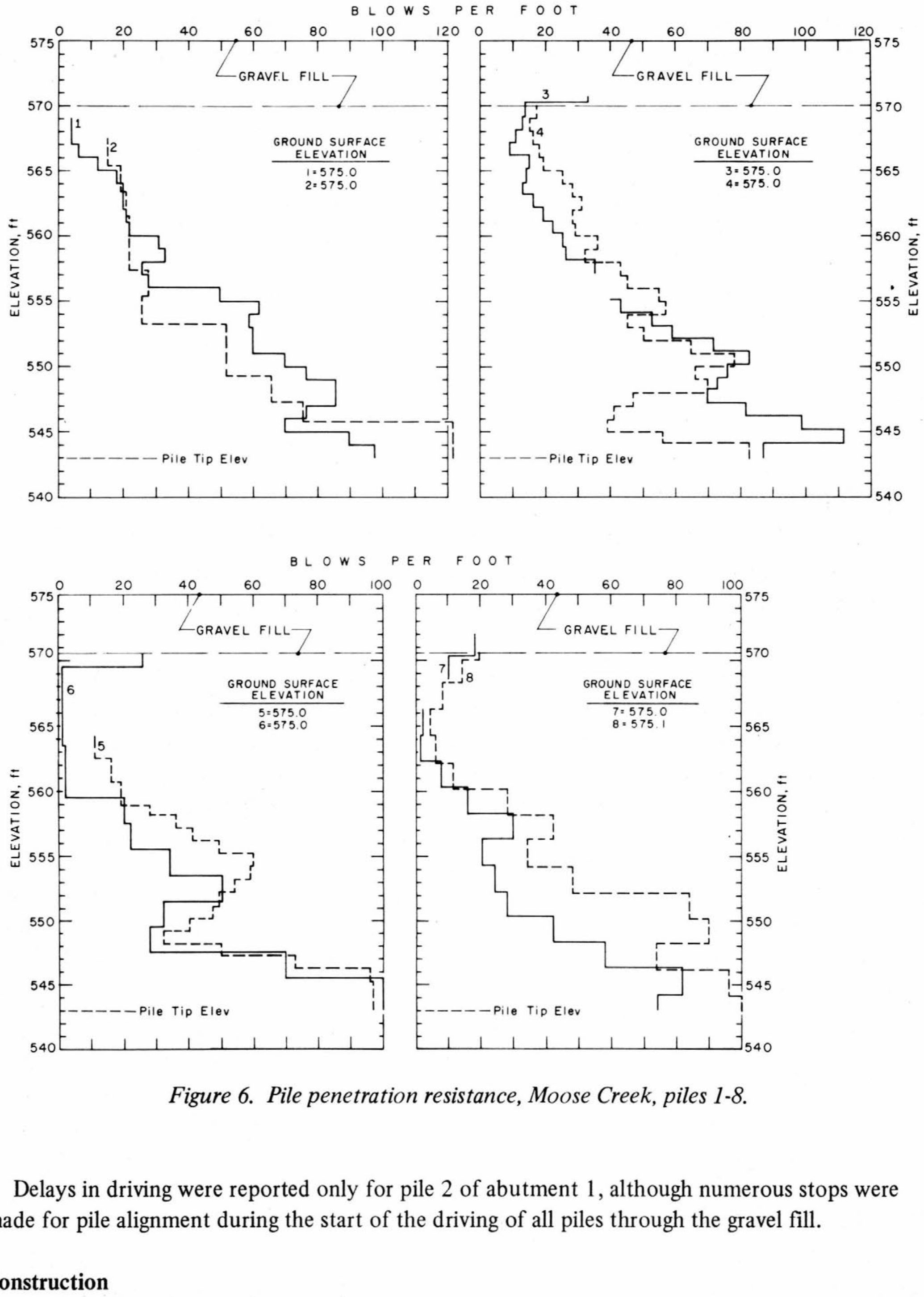


Figure 6. Pile penetration resistance, Moose Creek, piles 1-8.

Delays in driving were reported only for pile 2 of abutment 1, although numerous stops were made for pile alignment during the start of the driving of all piles through the gravel fill.

Construction

The 50-ft-long single span bridge over Moose Creek (Fig. 7), was constructed in July and August 1965. The abutments were designed and constructed to provide maximum protection against damage by frost heaving. A 1-ft space was provided between the approach fill and the bases of the 2 ft 6 in. wide by 2 ft high pile caps, to prevent potential upward frost thrust on the caps, as shown in Figure 8. The exposure of the piles at the abutments is called "daylighting." To prevent the

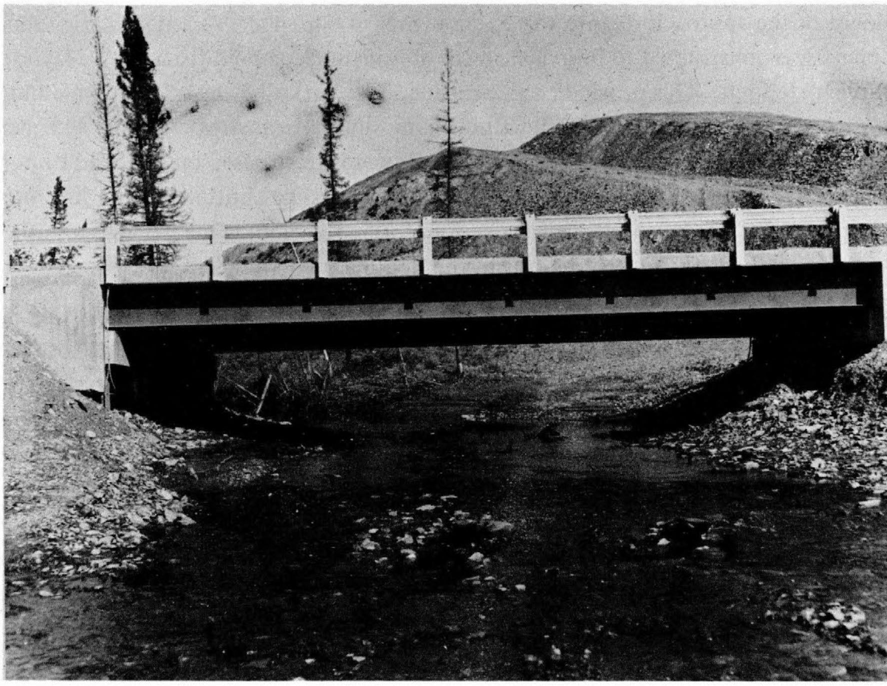


Figure 7. Completed bridge at Moose Creek.

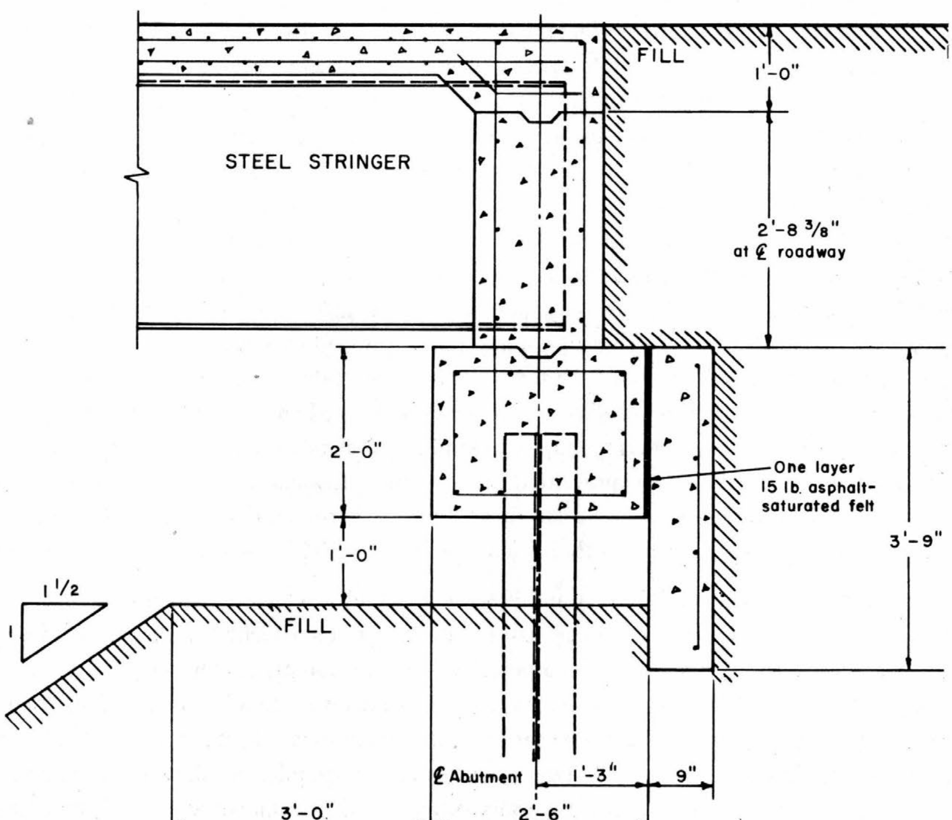


Figure 8. Cross section of abutment.

encroachment of the approach fill into this exposed area, a 9 in. wide by 3 ft 9 in. high reinforced concrete plank was constructed to bear against the abutment, separated from it by a layer of 15-lb asphalt-impregnated felt. This provided a shear plane. Theoretically, the concrete retaining plank was free to move up or down with the fill without imposing vertical stresses on the abutment. To further retain the approach fill, cantilevered wingwalls were constructed on each end of both abutments. These 1-ft-thick wingwalls are slightly over 7 ft high at the abutment centerline and taper, from the bottom, to a 3-ft height at their ends. Reinforcing steel joins the wingwalls to the abutment pile caps and wall between stringers, but not to the floating retaining plank.

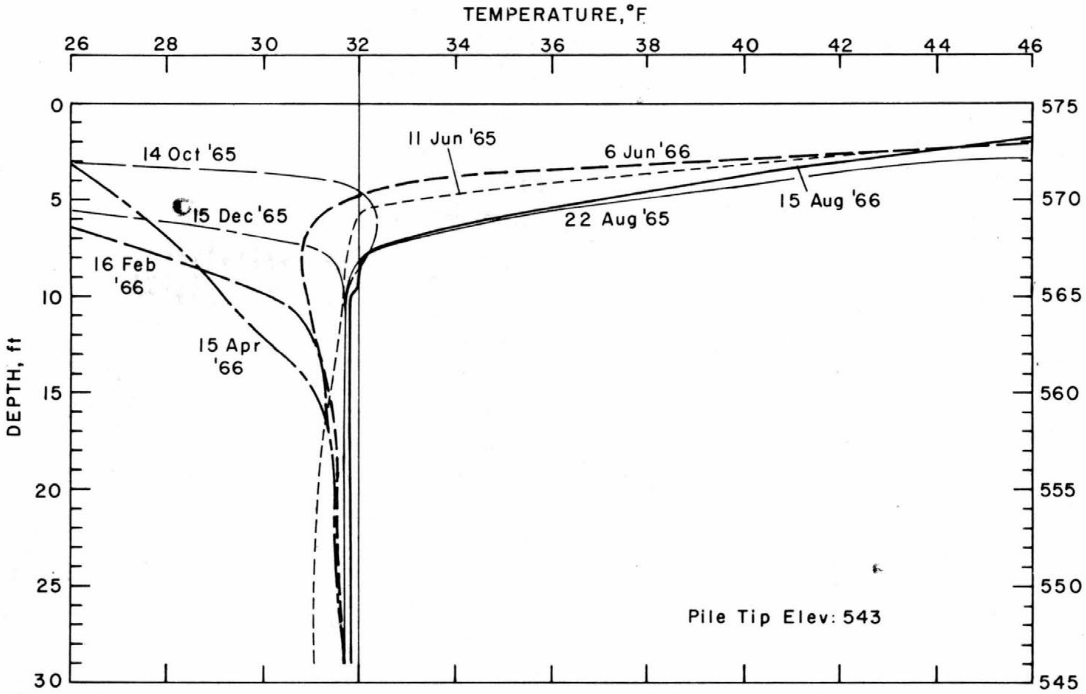
The superstructure of the bridge consists of five composite welded steel girders having webs 30 in. by $^5/_{16}$ in. thick and flanges made from 10-in.-wide steel plate, $^1/_2$ and $^3/_4$ in. thick, for the top and bottom flanges respectively. The girders are joined at mid-span by 15WF 33.9 diaphragms. The ends of the girders are embedded 1 ft into the concrete abutment endwalls. The 29 ft 7 in. wide reinforced concrete deck of the bridge is supported on galvanized corrugated decking and joined to the girders with stud type shear connectors. The concrete deck is sloped 4%, the superelevation being on the south side, and the bridge deck is about 7 in. thick between girders and about 9 in. thick above the girders. Elevation of the centerline of the bridge deck at the beginning and end of the bridge was 580.93 ft. The bridge was completed with the installation of guard rails on 6WF 15.5 posts, bolted to the edges of the deck and wingwalls. The completed bridge was opened to traffic in late August 1965.

Ground temperature observations

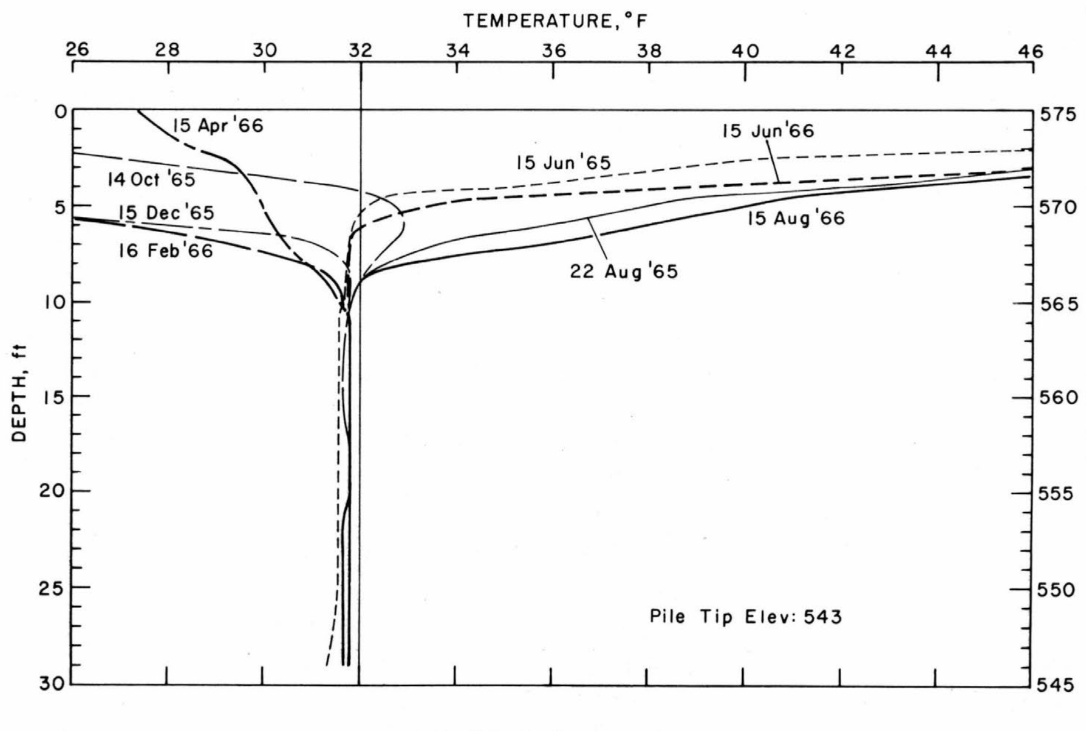
Two thermocouple assemblies were installed on the bridge piles (piles 4 and 5, see Fig. 3) to study ground temperatures during construction and as an integral part of the long-term performance observations at this bridge. All thermocouple assemblies were constructed from 18-gage copper-constantan wire and enclosed in polyethylene tubing. Each assembly consisted of two parts. Part A had thermocouple junctions at 2-ft intervals, starting at or near the ground (gravel fill) surface, to a depth of 12 ft and then at 4-ft intervals to 24 ft. Part B had thermocouples at odd 2-ft intervals, beginning at or near the 1-ft depth, to a depth of 11 ft, and at 4-ft intervals from 14- to 28-ft depths. Subdivision of the assembly into two parts permitted use of two smaller pockets rather than one large one,* and provided a margin of safety to insure installation of at least one part in the event one pocket was damaged during construction. The pile shoe design provided sufficient protection of the pockets against damage during driving. No rotation of any piles equipped with diametrically opposite thermocouple pockets was observed. None of the thermocouples were damaged by construction, although it was necessary to remove and reinstall some of the assemblies when the contractor cut the piles to grade. It was intended that the thermocouple assemblies be installed within an hour of the driving, but fabrication and shipment delays prevented this schedule. Installation of the assemblies was also hampered by delays in removing ice that had formed in the thermocouple pockets. The bridge temperature assemblies were permanently installed on 27 May 1965, some 10 to 13 days after driving.

The thermocouple wires from parts A and B of each assembly were connected to a 24-point copper-constantan panel board and mounted in electrical junction boxes, attached to the guardrail posts. Ground temperatures were observed using a portable millivolt potentiometer, with an ice-bath reference junction. Due to the inherent errors in the thermocouple circuits, ice-bath, instrument or observer, the practical accuracy of any one set of field observations is assumed to be ± $^3/_4$°F. Ground temperatures with depth for each of the two instrumented bridge piles are shown in Figure 9. Temperatures at various time intervals, normally every other month, are shown in each of the figures from soon after the installation period through mid-August 1966.

* Symmetrical pile sections are desired. A single large pocket could produce twisting or drifting of the pile during driving.



a. Pile 4, abutment 1.



b. Pile 5, abutment 2.

Figure 9. Ground temperatures, Moose Creek bridge.

The annual variations of ground temperature at both bridge piles are interesting. Pile 4 on the north or shady side of the bridge had about 9 ft of thawing by the end of the second summer after construction, as shown in Figure 9a. The length of pile embedment in permafrost was about 23 ft. The temperature in the middle and bottom sections of the pile length in permafrost was only 31.4° to 31.8°F, although initial observations during the first summer of construction reflected ground temperatures about 0.5°F colder. By January the active layer appeared to have been completely refrozen at this location, with low temperatures penetrating to depths of 17 ft or more. Icing of the stream during the winter of 1965-66 was not severe, reaching a height of only 3 ft above normal stream flow.

Ground temperatures at pile 5, on the southwest or sunny side of the bridge, were different from those at pile 4. While the depth of thaw and ground temperatures during the warm months, shown in Figure 9b, were similar to those at pile 4, the winter temperatures were quite different. At this pile frost seems to be slow to penetrate to the permafrost table, and complete freezeback is achieved only in late winter. There appeared to be no deep cooling of the permafrost as experienced at pile 4. Both instrumented piles were about the same distance from the stream, and the greatest influence on the piles will be the thaw bulb beneath the stream, as discussed and illustrated by Crory (1968). The ground temperature changes in Figure 9 do not reflect a one-dimensional (vertical) heat flow, as the plots would suggest. Unfortunately no thermocouple assemblies were installed within the streambed. The ground temperatures observed during this first year after completion of the bridge do not necessarily reflect the normal long-term regime of the abutments at this site.

Installation of test piles

Two test piles and three anchor piles were installed at Moose Creek on 7.5-ft centers, parallel to and adjacent to the stream (Fig. 3). The location of the test pile group was selected to approximate conditions of the actual bridge abutment piles. Incremental load tests were performed on the two test piles to verify design assumptions and for direct comparison with the theoretical capacities determined by the Engineering News Formula.

The three anchor piles (A, C and E) and test pile D were plain 10BP 57 sections, while test pile B was a similar section with the standard shoe, previously shown in Figure 5. The three anchor piles and test pile B were driven to the same elevation as the bridge piling (elevation 543 ft), with the same hammer used in the production driving. Test pile D, however, was placed, to the same elevation, in an 18-in. augered hole and backfilled with a sand-water slurry. The sand for this slurry was angular and well graded, meeting the specification for fine concrete aggregate. The sand was placed fully saturated and rodded, with the pile being periodically rapped with a sledgehammer to further consolidate the sand backfill. Thermocouple assemblies were installed in angle iron pockets on test pile B and placed in the sand slurry backfill adjacent to test pile D (in polyethylene tubing filled with oil-wax).

Driving records for the four driven piles are shown in Figure 10. Installation of the test pile cluster began on 17 May 1965 and was completed on 18 May 1965. Anchor pile A started with a blow count of 14 blows/ft and stayed fairly uniform until elevation 557. From that elevation the blow count increased greatly from 16 to 74 blows/ft at elevation 543. The driving record for test pile B was fairly uniform until elevation 559 but from there it was quite erratic, going as high as 115 blows/ft at elevation 546 and ending with 89 blows/ft at elevation 543. Anchor pile C started the easiest, with the blow count running 10 blows/ft or less for the first 5 ft. Between elevations 558 and 550 ft the blow count increased rapidly, and from there to elevation 543 it continued to increase, although more slowly, to a final count of 92 blows/ft. The driving of anchor pile E was quite similar to that of anchor pile C and finished with a blow count of 84 blows/ft.

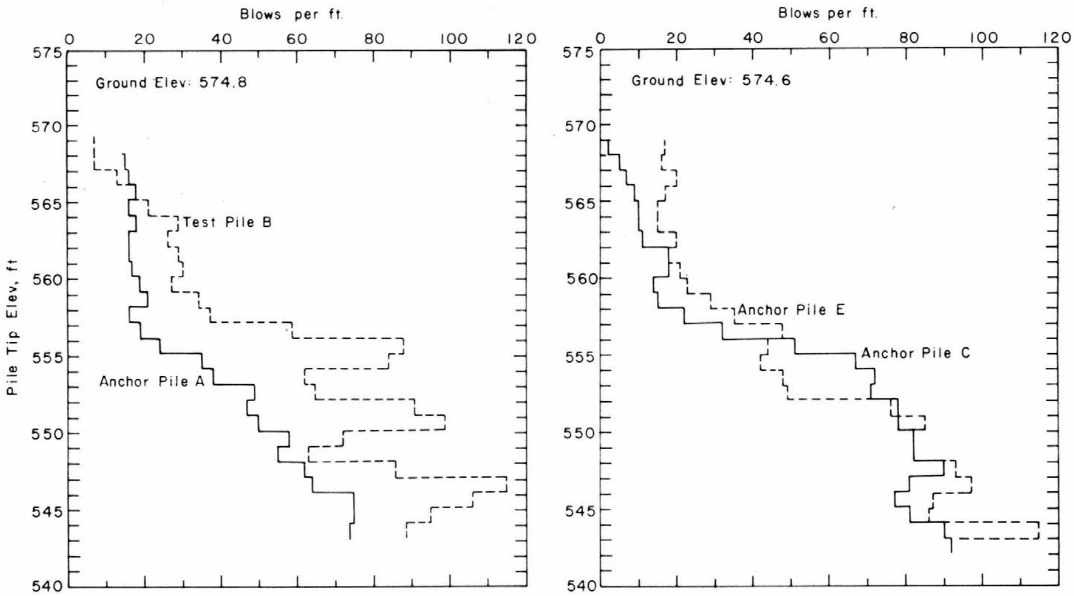


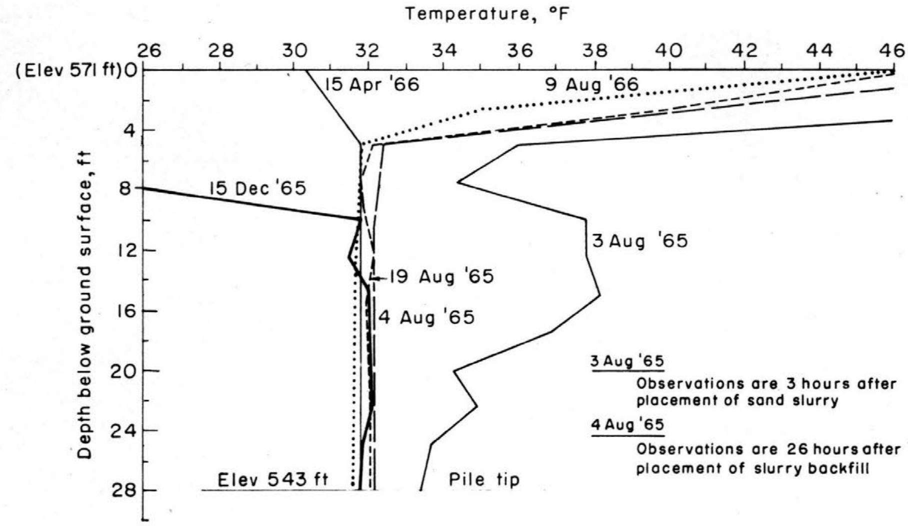
Figure 10. Pile penetration resistance, Moose Creek, test piles A, B, C and E.

Ground temperatures at the test piles were monitored to obtain information on the rate of freezeback (pile D) and the temperatures existing during the actual pile test period, and to determine if the thermal regime at the test piles was similar to that at the bridge abutments. Figure 11a shows temperatures in the sand slurry adjacent to test pile D for a year after installation, reflecting the change in temperatures during both the freezeback and pile testing periods. The observations indicated that the slurry lost virtually all of its sensible heat within 24 hours of placement. However, 16 days after placement (19 Aug 1965) the slurry temperatures still hovered at the freezing point. Accordingly the testing of piles at Moose Creek was rescheduled for 1966. Ground temperatures within the permafrost on 9 August 1966 were about 31.7°F, very similar to those of the bridge abutments. Ground temperatures at driven test pile B (Fig. 11b) indicated a consistent permafrost temperature of 31.4°F, with the annual fluctuation of near-surface temperature and frost penetration being similar to that of pile 5 of abutment 2.

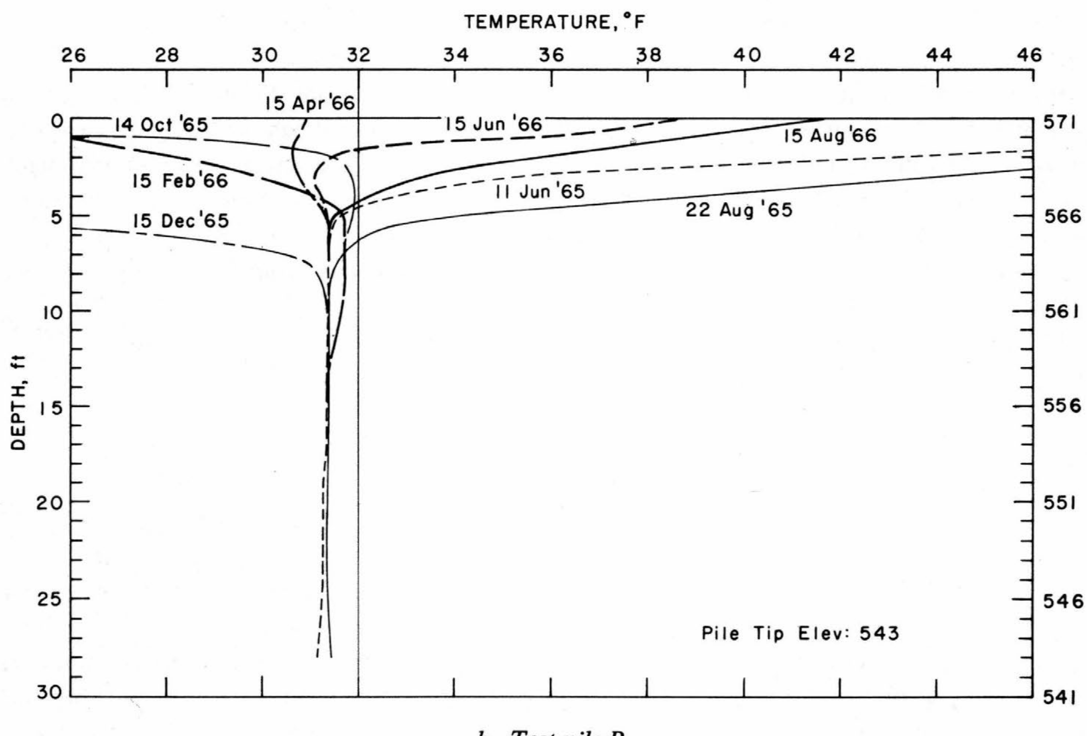
Pile load tests

To avoid the need for a load platform or box for testing piles B and D, a reaction beam connected to anchor piles was used. This same reaction system had been used for pile tests at Bethel, Alaska, and at the CRREL low-temperature permafrost test area at Kotzebue, Alaska. The reaction beam was made up of two 24 × 105.9-lb I beams, 20 ft long, connected by five bolted diaphragms. The ends of the reaction beams were restrained by 3-ft-long stub sections welded to both sides of each anchor pile. By placing a jack on the test pile and bearing against a short cross-beam, at right angles to and beneath the reaction beam, the desired reaction was achieved. Loads were applied by means of a 200-ton hydraulic jack. Photographs of the anchor pile reaction system and the instrumentation for the load settlement tests are shown in Figure 12.

To record the motion of the test piles and the center anchor pile under imposed loads, deflection gages were mounted on, or referenced to, a pair of instrumentation beams. One end of each instrumentation beam was supported on a 12-in. channel, temporarily welded to the top of the test pile



a. Test pile D.



b. Test pile B.

Figure 11. Ground temperatures, Moose Creek.

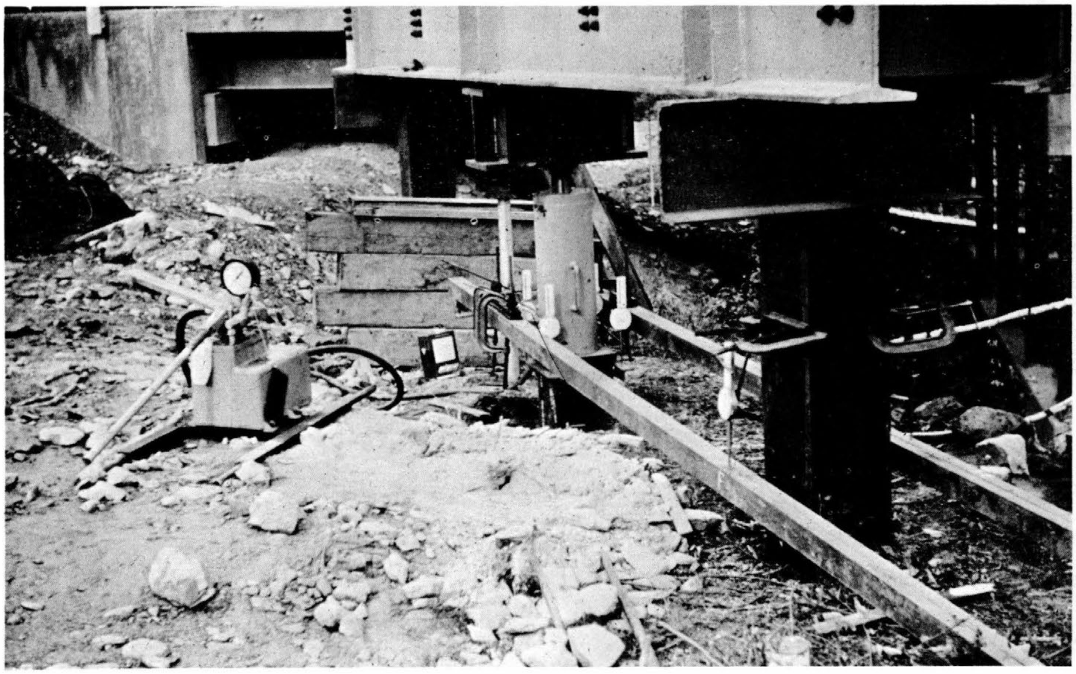


Figure 12. Pile testing equipment and instrumentation.

not under load. The other end was supported on a horizontal cross-bar welded to a 2-in. pipe located midway between the pile under test and the end anchor pile. The support pipes were installed in small-diameter dry-bored holes and driven the last 4 to 5 ft. The support pipes extended 23 to 24 ft below the ground surface or about 17 to 18 ft into permafrost.

Four deflection gages, mounted on brackets, were clamped to the instrumentation beams and referenced to equidistant points marked on the ends of angle irons welded to each side of each test pile. The two angle irons form a horizontal reference plane 1 in. below the pile head. The reference points were purposely placed at equal distances from each pile flange edge to accurately reflect any bending of the pile and provide a true average deflection of the pile under load.

Two deflection gages were mounted at the center of both flanges of anchor pile C, in both of the tests, to measure the upward deflection of this pile during each test. The supporting brackets for these two gages were mounted on the anchor pile rather than the reference beam to reduce the weight and attenuating deflection at or near the center of the instrumentation beams. Only the pressure of the reference plunger of these two deflection gates bore on the beams. All deflection gages used in these two tests had a resolution of 0.001 in. Overall travel of the dials was 4 in. The upward movement of anchor piles A and E during the two tests was not monitored, being assumed to be the same as that of anchor pile C.

After the reaction and instrumentation beams had been positioned, the entire test apparatus was covered with canvas, supported on 2 X 4 timber "poles" from the ground to the reaction beam. With the canvas completely shielding the instrumentation beams and reaction system from rain and sunlight, the deflection gages were mounted and initial or zero readings taken. A thermometer was also installed beneath the enclosure, midway between the center anchor pile and the jack ram, to monitor air temperatures. Air temperature readings were taken at hourly intervals, when the deflection gages were being observed.

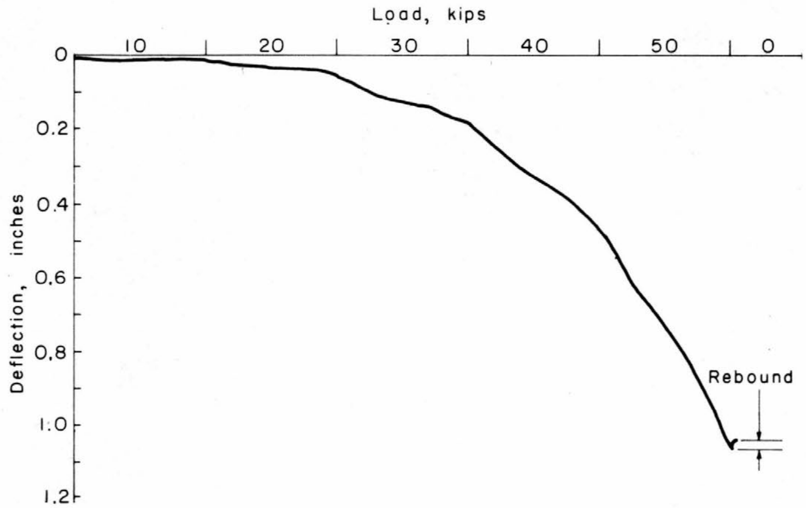


Figure 13. Load-settlement data, test pile D.

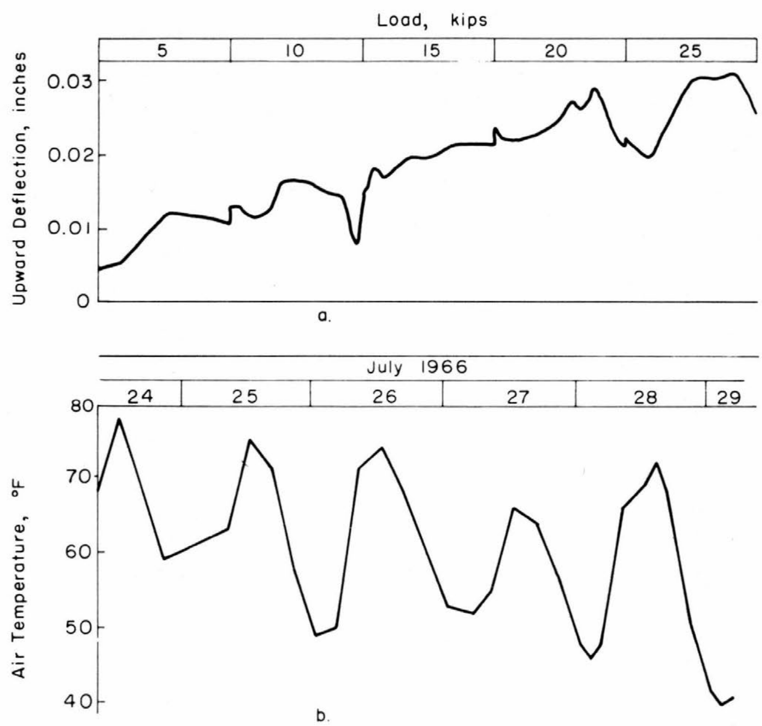


Figure 14. Load-settlement and air temperature data, anchor pile C, July 1966.

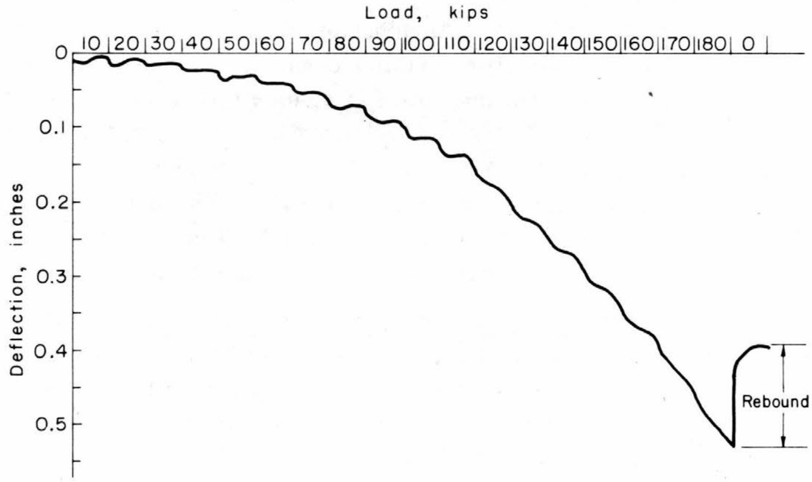


Figure 15. Load-settlement data, test pile B.

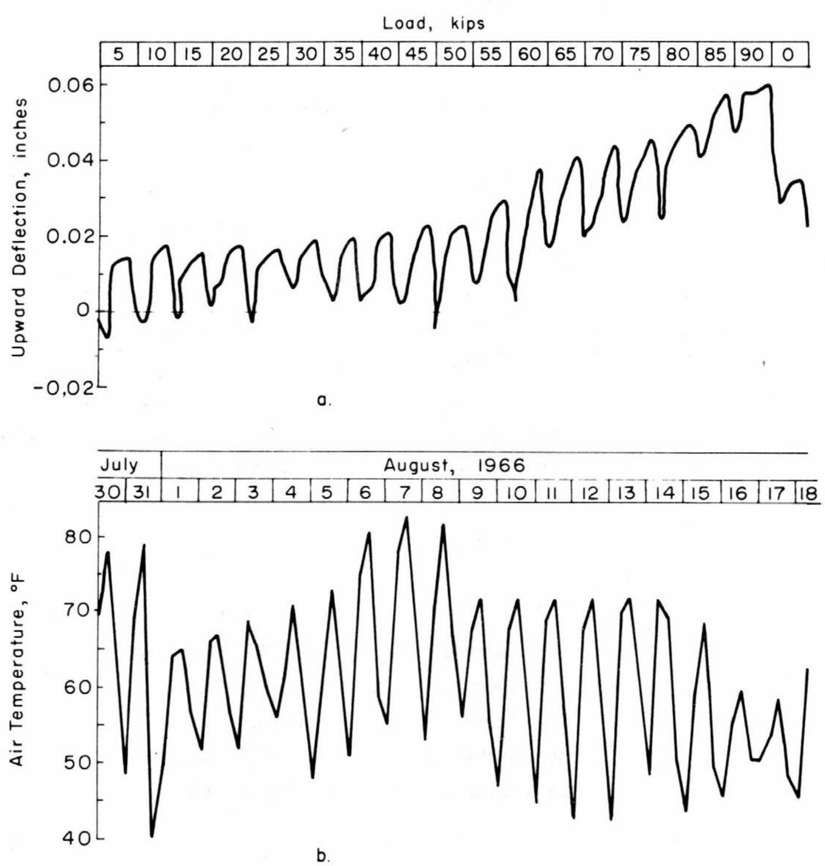


Figure 16. Load-settlement and air temperature data, anchor pile C, August 1966.

Pile testing at Moose Creek got underway on 24 July 1966 with testing of pile D, the pile placed in the augered hole backfilled with slurry. An initial load of 5 tons was applied at 0900 hours on the morning of the 24th, with subsequent five-ton load increments being added and maintained constant at 24-hour intervals until 29 July. The final load applied to test pile D was 25 tons, the pile having deflected 1.062 in. At 0900 hr on 29 July the load was reduced in one smooth decrement to zero and the rebound of the pile observed. A plot of settlement vs time and load for test pile D is shown in Figure 13. The upward deflection of anchor pile C was also recorded and plots of upward deflection vs load and time and air temperature vs time are shown in Figure 14. Because the system was symmetrical, the load on anchor pile C was assumed to be exactly half that imposed on the test pile.

The test on pile B began on 29 July at 1635 hr. Observations were made until 0900 hr 30 July with no load, to check the stability and temperature sensitivity of the instrumentation system. The test was continued with the addition of 5-ton load increments, added at 0900 hr every morning, until 17 August when the maximum load of 90 tons had been maintained constant for 24 hours. The average deflection at the end of the 24 hours under the maximum load was 0.534 in. At 0900 hr on the 17th the load was reduced in one smooth decrement to zero with rebound observations being continued for the next 24 hours. A plot of deflection vs load and time for test pile B is shown in Figure 15. The upward deflection of anchor pile C was again recorded and plots of deflection vs load and temperature vs time are shown in Figure 16.

SPINACH CREEK BRIDGE

General geology and site conditions (Utermohle 1964b)

The site for the Spinach Creek bridge is quite similar to that at Moose Creek, lying in the same rolling terrain west of Fairbanks. During the summer and fall months the original creek flowed in a U-shaped channel about 3 ft deep and 4 ft wide with the water rarely more than several inches deep. As at Moose Creek the area at Spinach Creek is generally underlain by Birch Creek schist displaying various degrees of weathering. Most of the surface area is covered by an organic silty soil which is derived from the surrounding loess-covered slopes. The ground cover at Spinach Creek was also similar to that at Moose Creek, consisting of scrub spruce, alders, birch and occasional stands of tall spruce. An examination of the vegetation in the immediate area of the bridge site indicated that there was no appreciable scour. No icing at, above, or below the proposed bridge site was observed during the winters of 1963-64 and 1964-65.

Soil investigations (Utermohle 1964b)

The soils investigation at the bridge site was conducted by the Alaska Department of Highways, Road Materials Laboratory. A plan of the bridge site, with original ground contours and location of drill holes, is shown in Figure 17. The explorations showed an organic silt layer approximately 10 ft thick underlain by a zone of severely weathered schist. The weathered schist zone takes the form of silty sand and gravel and was found to be permanently frozen.

Below the severely weathered schist was a zone of more competent and somewhat less weathered schist. The geologist described this zone as being about 70% slightly compact, highly weathered schist with a minor amount of loose or soft graphite schist and approximately 30% hard quartz and quartz schist layers or lenses. The standard penetrometer could not penetrate this zone which continued to 45 ft, the depth of the deepest hole drilled. The penetration tests in holes No. 5 and 7, and the logs of the six other drill holes, are presented in Figure 18. Laboratory analyses of soil from augered holes 7 and 8 are given in Table II.

Table II. Analysis of foundation investigation, Project no. S 0651(2), Spinach Creek, Bridge 359.

Depth /feet	F'd no.	Lab no.	Mechanical & sieve analyses - percent pass 1	$^3/_4$	$^1/_2$	$^3/_8$	#4	#10	#40	#200	0.02	.005	LL	PI	Class (HRB)	Spec grav	Nat mois
BORING NUMBER 7, STATION 429+27, ON CENTERLINE																	
0.5-1.0	1	2013							100	88.0	29.0	11.0	56	NP	A-5(11)	2.39	117.9
1.0-1.5	2	2014							100	94.0	42.5	16.9	43	NP	A-5(9)	2.65	68.4
2.0-2.5	3	2015							100	92.0	39.8	15.3	43	NP	A-5(9)	2.51	72.5
3.5-4.0	4	2016						100	99	88.0	32.8	12.1	33	NP	A-4(8)	2.63	62.9
5.6-6.0	5	2017			100	98	89	73	50	31.2							29.0
7.0-7.5	6	2018	100	98	97	90	74	50	30	12.7							30.6
10.5-11.0	7	2019		100	96	89	73	57	40	22.9							9.6
BORING NUMBER 8, STATION 429+87, ON CENTERLINE																	
1.0-1.5	1	2020						100	99	73.0	30.5	12.2	32	NP	A-4(8)	2.53	50.6
2.0-2.5	2	2021					100	98	94	78.4	30.4	11.3	32	NP	A-4(8)	2.65	40.2
4.5-5.0	3	2022						100	95	77.0	29.5	12.1	33	NP	A-4(8)	2.58	73.5
6.0-6.5	4	2023		100	94	88	70	58	47	31.7	14.5	6.3	25	NP	A-244	2.63	24.2
7.5-8.0	5	2024	100	99	94	89	78	65	47	28.0							18.7
9.5-10.0	6	2025		100	98	93	81	64	42	25.0							19.2
12.0-12.5	7	2026	100	99	95	90	75	58	41	22.4							13.1

All tests by State of Alaska, Road Materials Laboratory, College Alaska.
Note: Borings Number 7 and 8 made with auger.

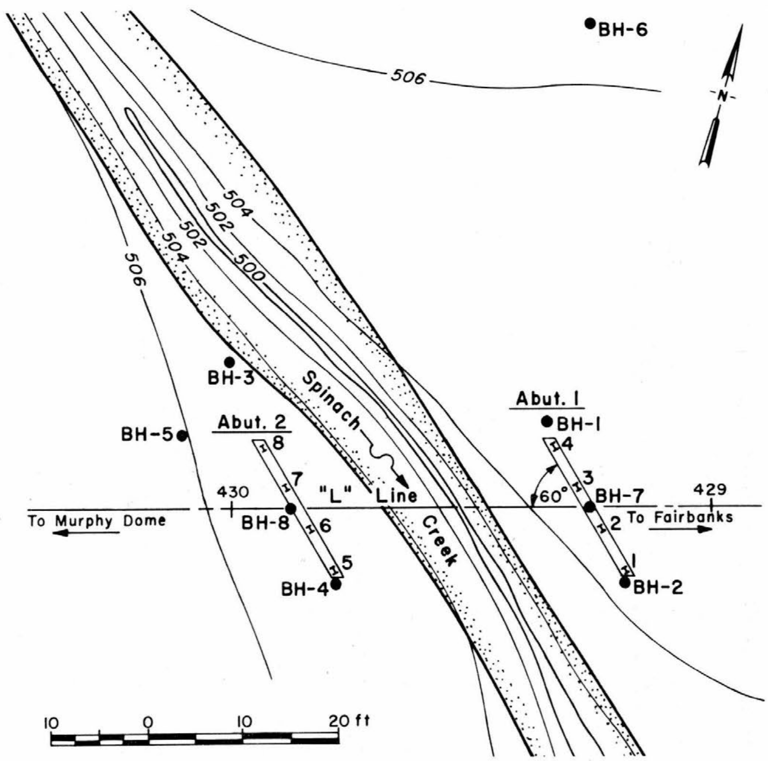


Figure 17. Foundation plan, Spinach Creek.

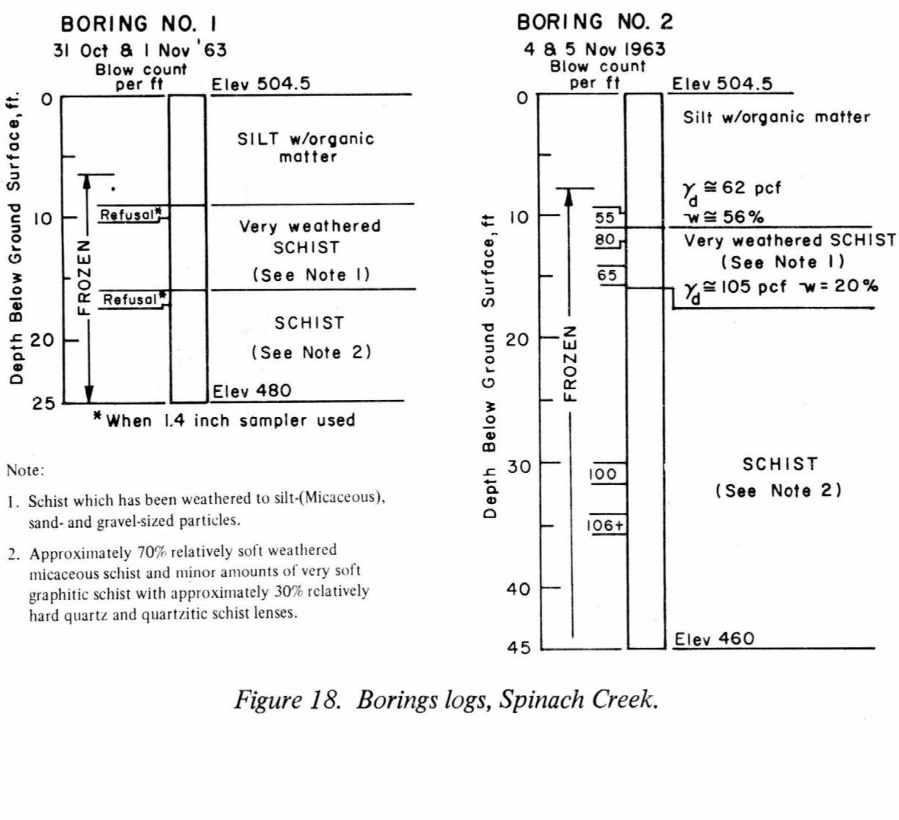


Figure 18. Borings logs, Spinach Creek.

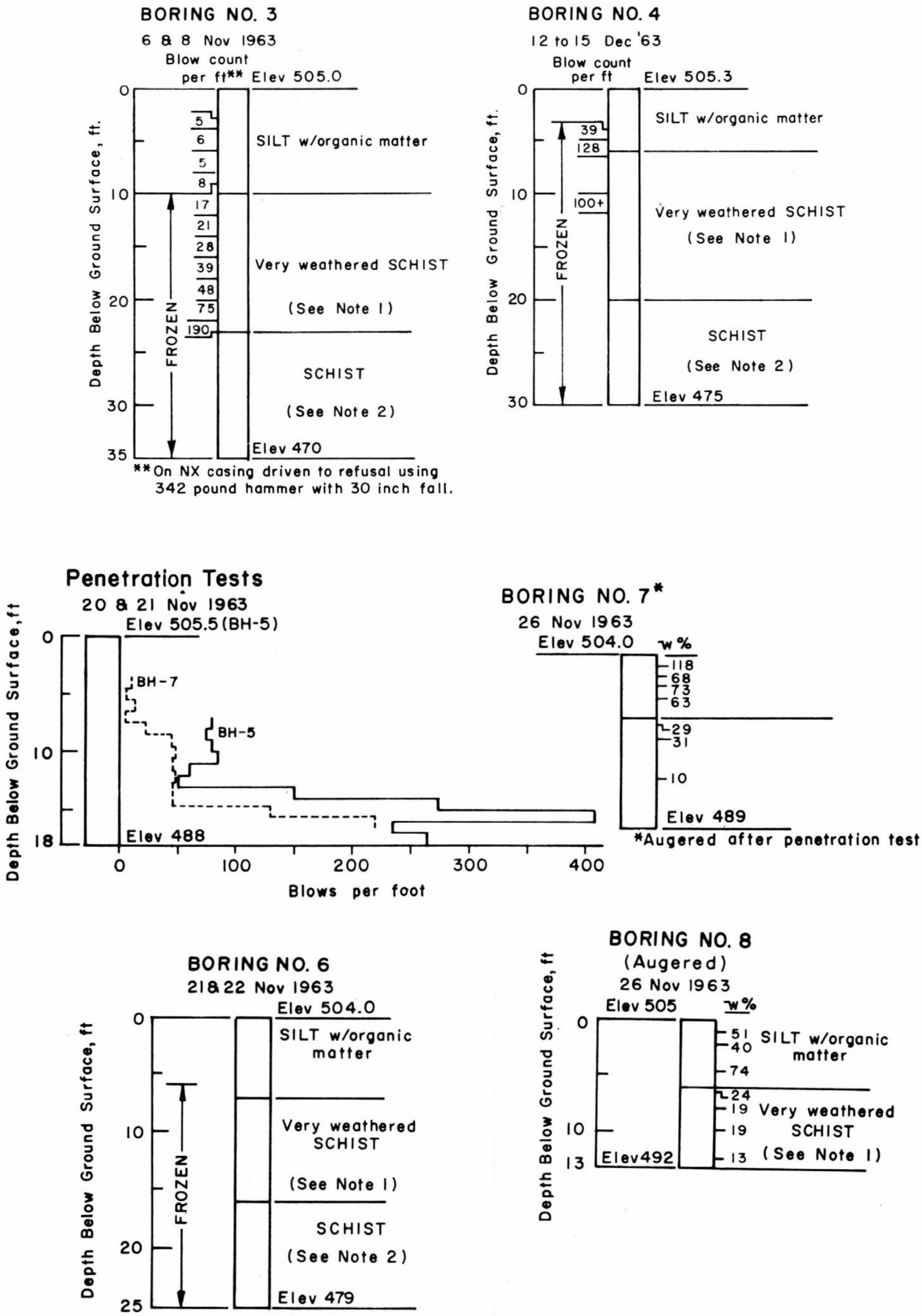


Figure 18 (cont'd).

Foundation design

The following recommendations were made by the geologist (Utermohle 1964b) after studying the conditions of soil, rock and permafrost at the Spinach Creek site.

1. An "H" pile foundation is recommended. Piles should be supported by end bearing or point bearing on the schist strata.

2. It is recommended that the "H" piles have reinforced tips and be driven to refusal, short of damage to the pile. Estimated pile length is 20 ft; however, pile lengths could vary from approximately 15 to 25 ft. Because of permafrost, once the driving of a pile begins, driving should be continued until the desired depth and bearing is obtained.

3. The zone around each pile, from elevation 500 ft to the gravel fill surface, should be filled with an oil-wax-soil mixture. The purpose of this mixture is to eliminate or decrease the jacking action of seasonal frost, and also reduce the negative skin friction on the piles should settlement occur from consolidation of the organic silt under the gravel or from subsequent melting of the permafrost.

4. Piles should be exposed ("daylighted") a minimum of $^1/_{10}$ the thickness of the abutment fill.

5. Fill in the area of the abutments should be non-frost-susceptible and of A-1-a (HRB) classification.

6. It is also recommended that at least one thermocouple be installed near each abutment in order to observe temperature variations after construction.

Pile installation

Pile driving for the Spinach Creek bridge began on 25 May 1965 with pile 4 in abutment 1 and ended 27 May 1965 with pile 8 in abutment 2. The same equipment was used at this site as at Moose Creek. Pilings were again 10BP 57 sections, with shoes. All piles at Spinach Creek were started in 10-ft-deep pre-augered and cased holes, which were later backfilled with the non-frost-susceptible oil-wax-soil mixture and, with one exception, driven to tip elevation 485. The modified Engineering News Formula was used to compute the bearing capacity of each pile, and compared in the field with the design load of 45 tons per pile. As at Moose Creek, two diagonally opposite piles (1 and 8) were equipped with thermocouple assemblies to monitor soil temperatures at various depths.

The driving records for all eight piles are found in Figure 19. Pile 1, containing the double thermocouple pockets, was driven with little difficulty. The blow count showed a general increase from 22 blows/ft at elevation 496 ft to 84 blows/ft at elevation 486 ft. From 486 ft to the final elevation of 485 ft the blow count dropped, ending at 76 blows/ft with a theoretical bearing capacity of 58 tons. Piles 2 and 3 had similar records, showing gradual increases in blow count, except for one interval. In both records there is a sharp drop in blow count between elevations 494 and 492 ft. This probably indicates a small localized thaw or weathered zone, since the boring records show no distinct discontinuity at that elevation. The bearing capacities of piles 2 and 3 were 69 and 78 tons, respectively. Pile 4 showed a slight increase in blow count until the last foot where the count went to 110 blows/ft, yielding a bearing capacity of 92 tons.

Pile 5 in abutment 2 showed a general increase in blow count starting at 40 blows/ft and ending at 90 blows/ft at tip elevation 485, with a bearing capacity of 82 tons. Pile 6 showed a fairly slight increase, starting at 52 blows/ft at tip elevation 496 and stopping at 72 blows/ft at elevation 485 for a bearing capacity of 62 tons. Pile 7 showed a sharp increase in blow count starting with 22 blows/ft at elevation 496 and increasing to 94 blows/ft at tip elevation 485. This produced a bearing capacity of 86 tons. Pile 8, which contained the second thermocouple assembly, had a sharp increase in blow

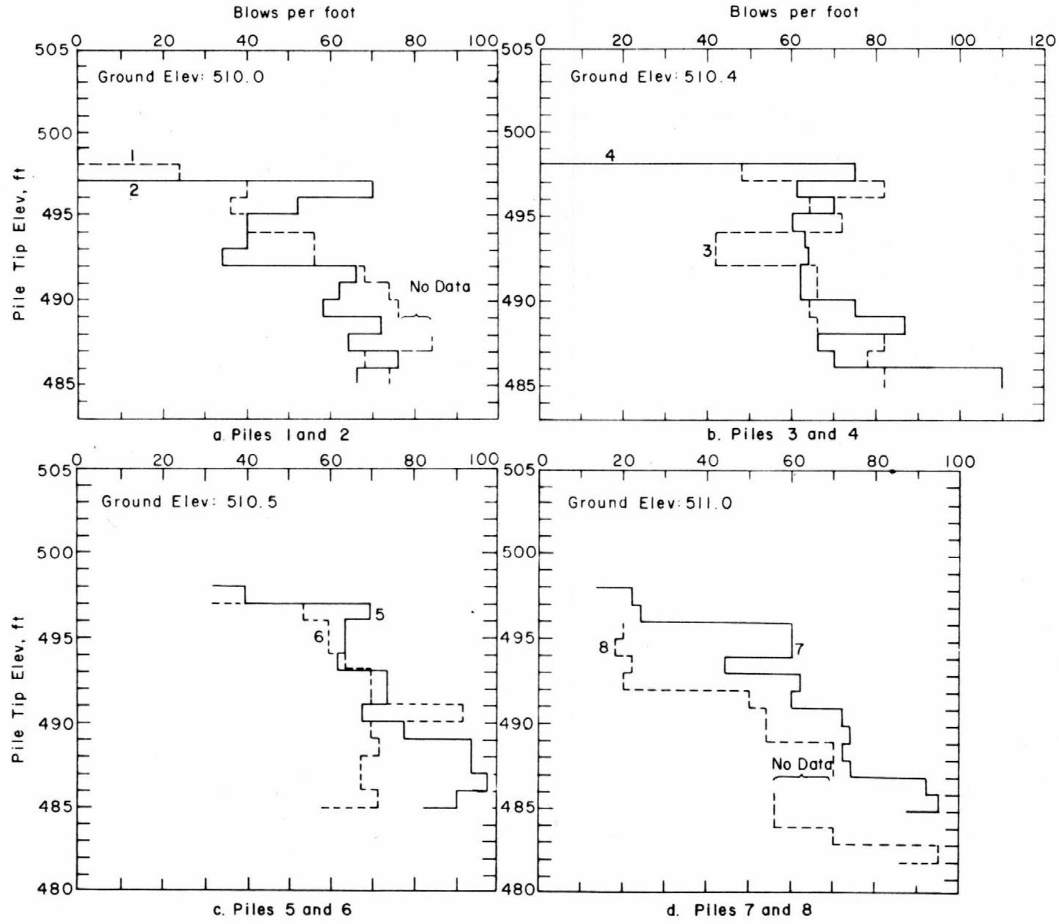


Figure 19. Pile penetration resistance, Spinach Creek, piles 1-8.

count from 20 to 50 blows/ft, at elevation 491. At elevation 485, however, the blow count was still only 56 blows/ft which gave a bearing capacity of 35 tons. Because the calculated bearing capacity of this pile was less than the design capacity of 45 tons, the contractor was required to drive this pile another 2 ft, to elevation 482 ft, where the blow count was 96 blows/ft and the bearing capacity was calculated to be 89 tons.

Thus, with the exception of pile 8 at elevation 482 ft, all piles for this bridge were driven to elevation 485 ft, where the theoretical capacity of each pile exceeded the design load of 45 tons. The construction criterion was to achieve a theoretical capacity by the pile driving formula at or below the estimated pile tip elevation, and was not in accordance with the geologist's recommendation to drive the piles to refusal.

Anti-heave backfill

The bridge at Spinach Creek required special anti-heave backfill to counteract frost action. The usual practice is to make the length of pile embedment in the permafrost layer 2 to 3 times the length of piling in the active zone. This ratio of lengths of embedment is usually adequate to resist the heave forces (frost jacking) imposed on the pile by the freezing of the active zone. However, it

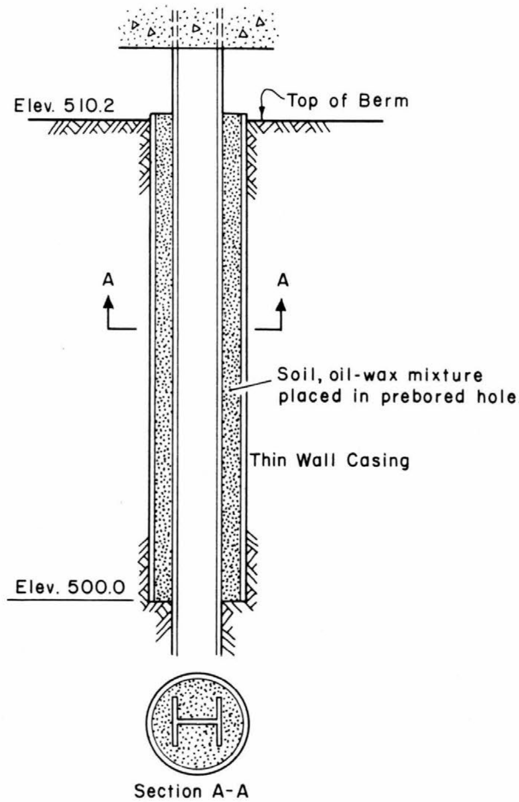


Figure 20. Anti-heave backfill for piles, Spinach Creek.

was considered impractical to drive the piling deep enough in the schist to achieve the desired length of embedment in permafrost to resist frost action. The heaving forces on piles in the Fairbanks area have been previously investigated, the maximum heave thrust on an 8-in. pipe pile being 55,000 lb (Crory and Reed 1965).

Since the length of pile embedded in permafrost overlying the schist was considered too short to resist frost heave a special anti-heave backfill was specified for the Spinach Creek bridge. Piles were driven in holes pre-augered and cased from the ground surface to elevation 500. This elevation was considered the top of the permafrost zone. After the piles were driven, the holes were backfilled with a special oil-wax-soil mixture which would protect the pile from the uplifting action of frost, as shown in Figure 20.

The oil in the backfill is similar or equal to Esso's Mentor oil 28 and the wax similar or equal to Esso's Mikrovan wax 1650. Soil used for the mixture was a dry sand. The oil and wax were combined and mixed thoroughly at a temperature of approximately 175°F in a ratio of 70% oil and 30% wax, by weight. After the oil and wax were carefully combined the soil was added until the oil-wax consisted of not less than 20% nor more than 25% of the dry unit weight of the soil.

The oil-wax-soil backfill was mixed in a small concrete mixer at the bridge site. After cooling, the mixture was placed in 2-in. lifts and compacted, with a 2 × 4, into the annular space between the pile and casing. When the oil-wax mixture is cool it has the consistency of vaseline, with a low

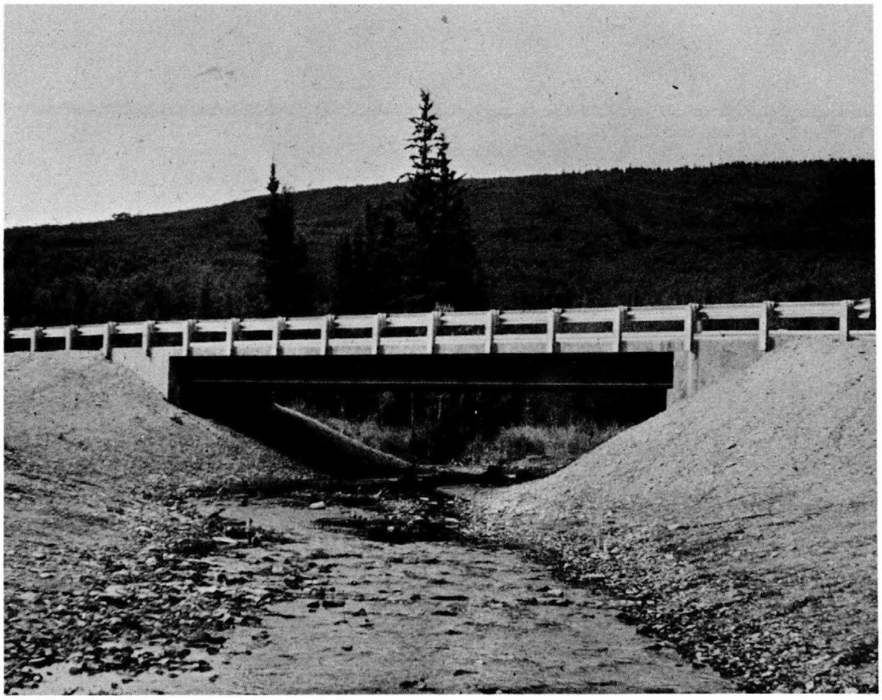


Figure 21. Completed bridge at Spinach Creek.

coefficient of expansion over a wide range of temperatures. This low coefficient of expansion, coupled with the oil film the mixture imparts to the piling, protects the piles from frost heaving. The oil-wax-soil mixture had previously been tested by CRREL in both the field and laboratory.

Construction

Spinach Creek bridge (Fig. 21) was built in July and August of 1965. It is very similar to the bridge at Moose Creek, the main differences being grade, girder size, and the anti-heave backfill (discussed above). The 60 ft $8^3/_4$ in. long bridge has a level deck, at a centerline elevation of 516.8 ft, with a $1^1/_2$% transverse grade. In contrast to Moose Creek, the Spinach Creek bridge is skewed 60° to the centerline, conforming to the original stream channel. General abutment details are the same at Spinach as Moose Creek, including the cap size, " daylighting," and retaining plank details.

The superstructure of the Spinach Creek bridge consisted of five composite welded steel girders. The top flange was 10 × $^1/_2$-in. plate while the web was 30 × $^5/_{16}$-in. plate. The bottom flange was 12 × $^5/_8$-in. plate for 13 ft on each end, while the middle 33 ft was 12 × 1-in. plate. Stud type shear connectors (5 in.) were used on the top flange to make the deck and girders act as a composite section. The girders are embedded 1 ft in the abutment end walls and are joined by 15 U 33.9 diaphragms at 19 ft 6 in. from both ends. The deck is about 7 in. thick between girders and 9 in. thick over them. Spinach Creek bridge was completed and opened to traffic in August of 1965.

Since the bridge pilings at Spinach Creek were designed for end-bearing on the schist bedrock, no test piles were installed or tested at this location.

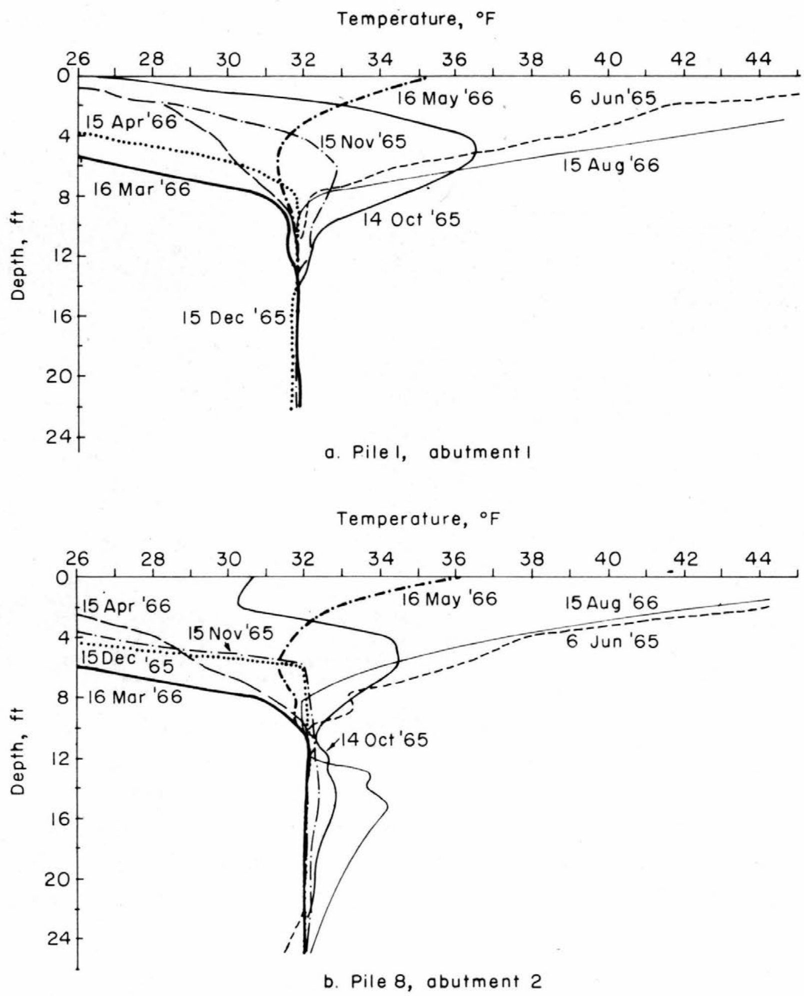


Figure 22. Ground temperatures, Spinach Creek.

Ground temperature observations

Thermocouple assemblies were installed on diagonally opposite bridge piles (pile 1 of abutment 1 and pile 8 of abutment 2), on opposite corners from those at Moose Creek. The thermocouple assemblies, in two parts, were fabricated and installed in the same manner as previously described for Moose Creek. Ground temperatures with depth at the two abutments at the Spinach Creek bridge are shown in Figure 22 for selected observations during the first 15 months. Pile 1, on the southeast corner of the bridge, experienced a degradation of permafrost from about 8 ft to 13 ft, from June till late fall (Fig. 22a). During the winter of 1965-66 frost penetrated to a depth of only about 10 ft. There were apparently only minor differences in ground temperatures in the late summer of 1966, as compared to 1965.

Ground temperatures at pile 8, on the northwest (shady) corner of the bridge, experienced completely different temperatures during the same period (Fig. 22b). The temperature data indicate that a layer of frost persists at about the 10-ft depth until late in the summer, while there is a subterranean flow of water which causes the 34°F or warmer temperatures at the 15-ft depth. Based on the thermal gradient between the 15- and 25-ft depths in mid-August 1966, there is no permafrost above the 25-ft depth at this pile.

DISCUSSION

The small single span bridges at Moose and Spinach Creeks provide a unique opportunity to study the performance of shallow pile foundations, in contrast to the deeper piling at Goldstream Creek (Crory 1968). The relatively shallow depth to bedrock at Moose and Spinach Creeks offered simple foundation designs, based on end bearing piles. When it was determined by borings that competent bedrock was available below the weathered bedrock, consideration was given to the possible uplifting of the piles by frost action. At the Spinach Creek bridge, where the overburden was only about 10 ft thick, the piles were protected by anti-heave backfill in the active layer. Moose Creek, having a thicker overburden and greater depth of frozen soils, had no frost heave protection around the upper portions of the piles. Since both bridges had similar abutment designs (except that Spinach Creek bridge is skewed), the only other major difference between the two bridges is the occurrence of icings at Moose Creek.

The ground temperatures reported herein reflect a very delicate frozen condition at both bridge sites. The permafrost which existed prior to construction has been greatly disturbed, even to the point of thawing, as demonstrated by the complete absence of permafrost at pile 8, abutment 2 of the Spinach Creek bridge. The delicate permafrost is also reflected in the freezeback time and bearing capacity of the slurried test pile at Moose Creek. The slow rate of freezeback of slurried piles in relatively warm permafrost has been previously investigated and reported (Crory 1963, 1967).

Of particular concern, however, was the very low bearing capacity of the slurried test pile when loaded one year after installation. The inability of this test pile to hold more than 10 tons without experiencing a continuing plastic settlement with time was completely unexpected and in direct conflict with other test pile data, using similar sand backfilled piles in permafrost (Crory 1963). The very low adfreeze bond of slurried test pile D is attributed to the near thawing temperature of the permafrost (and slurry). The low capacity of this pile reflects no contribution from end bearing, the pile tip being above the bedrock. It was indeed fortunate that all production piles could be driven to the specified tip elevation, without resorting to the pre-boring and sand backfilling method given in the geologist's recommendations (see *Foundation design*).

Test pile B, driven to the same depth as slurried test pile D, supported about four times the load of test pile D, at the same deflection. An analysis of the load test data on driven pile B indicates that such piles are capable of supporting 50 tons, without large continuing plastic deflection. Thus the piles have a factor of safety of 3, with respect to the dead load of the bridge. The capacity of test pile B and the foundation piles reflects the adfreeze bond strength of the permafrost at the time of the test. Should the permafrost get warmer, or even thaw out, the bearing capacity will decrease if the piles do not bear directly on competent bedrock.

The theoretical safe working loads determined by the modified Engineering News Formula, averaging about 90 tons, are approximately five times greater than that determined by the test data from pile B. Since the test pile and production piles could have capacities which are completely different because of the condition or penetration into the weathered or solid bedrock, no real comparison should be made between piles. The blow counts during pile driving were used for construction control rather than determination of safe working loads, since there is probably little correlation between a dynamic load and the long term creep strength of piles in permafrost. The installation of the 10 BP57 piles at these two bridge sites, by conventional driving, confirmed the earlier observations of pile driving in permafrost at Bethel (Crory 1973) and at the Alaska Field Station at Fairbanks (Crory 1963).

CONCLUSIONS

Both the delicate temperature and shallow depth of permanently frozen soils at these two bridge sites point out the need for more detailed information on the pre-construction and long-term thermal regime at stream crossings. Since it was realized that such delicate conditions existed at these sites both bridge designs were based on point bearing piles. The construction criteria, however, do not necessarily provide adequate controls to achieve these design assumptions. The use of the Engineering News Formula does not provide meaningful data on the true bearing capacity of such piles, nor does the use of a specified minimum depth of embedment. This had been particularly evident at Spinach Creek where the piles, meeting the required blow count, were driven to the specified depth. Thus the piles could well have been driven to only the weathered, but then frozen, bedrock. The geologist's recommendation of driving the piles to refusal appears to have been overlooked.

The use of augered and slurried piles for bridge foundations is highly questionable in light of the testing of pile D at Moose Creek. While such pile installations could be and have been effectively used in colder permanently frozen ground, their use in foundations so close to water in warm permafrost areas should be carefully scrutinized. The use of augered and slurried piles which are drilled into solid bedrock, however, could be considered a viable design technique, with the supporting capacity based on point bearing rather than adfreeze bond.

While shallow pile foundations on point bearing piles could be designed to resist downward loads, careful attention must also be given to uplifting of the piling and/or abutments from frost action. Without sufficient embedment, or anti-heave protection, shallow piles could be heaved by frost, imparting serious stresses on the bridge superstructure. The performance observations of these bridges, reflecting both heaving and settlement, will be contained in a companion report.

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