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REDUCTION OF FROST HEAVE BY SURCHARGE STRESS

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Cold Regions Research and Engineering Laboratory
Hanover, New Hampshire

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13. ABSTRACT

The results of a six-year field test program conducted near Fairbanks, Alaska, to investigate the reduction in frost heave obtained by applying a surcharge stress on the soil are presented. Seasonal heaves of 25-ft-square test sections with nominal surcharge loads of 2, 4, 6, and 8 psi were compared with heaves at adjacent unloaded sections. The test sections were on a silt soil in an area where permafrost existed at about a 7-ft depth. Results showed that only a small surcharge load was needed to cause significant reductions in heave. Data are included that indicate that heave reduction was achieved by minimizing groundwater migration. A method for correlating field and laboratory rate-of-heave data is suggested.

14. Key Words

Fairbanks, Alaska	Moisture migration
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PREFACE

Authority for this investigation was contained in FY 1960 Instructions and Outline, Military Construction Investigations, Engineering Criteria and Investigations and Studies, Investigation of Arctic Construction; Subproject 16, Surcharge Field Tests at Fairbanks, Alaska. At the time of this investigation the Military Construction Investigations program was conducted for the Engineering Division, Directorate of Military Construction, Office, Chief of Engineers, and was administered by the Civil Engineering Branch (Mr. T.B. Pringle, then Chief). The Military Construction Investigations program is currently conducted for the Office of Plans, Research and Systems (OPRS), Directorate of Military Construction, Office, Chief of Engineers.

This study was performed by Mr. G.W. Aitken, Research Civil Engineer, Construction Engineering Research Branch, under the direct supervision of Mr. E.F. Lobacz, Chief, Construction Engineering Research Branch, and the general supervision of Mr. K.A. Linell, Chief, Experimental Engineering Division, U.S. Army Cold Regions Research and Engineering Laboratory (USA CRREL).

This report was technically reviewed by Professor K.B. Woods, Goss Professor of Engineering, Purdue University; and Professor S.J. Poulos, Harvard University.

The author is indebted to Mr. F.F. Kitze, former Chief, Alaska Field Station (now the Alaskan Division), and his staff, without whose assistance this program could not have been accomplished. Mr. C.W. Fulwider, USA CRREL, contributed to the design of the field test installation. Messrs R.W. Huck, S.D. Murray, R.E. Brittain and C.J. Olsten, formerly of USA CRREL, made substantial contributions in the area of data reduction and preliminary analysis.

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REDUCTION OF FROST HEAVE BY SURCHARGE STRESS

by

George W. Aitken

INTRODUCTION

General

This investigation was conducted from August 1960 to March 1966 to determine the extent to which seasonal frost heaving can be reduced by application of a surcharge stress on the ground surface. The overall objectives of research in this area are to modify existing and/or to develop new design procedures which directly account for the beneficial effects of stresses imposed by base-course and pavement materials on frost-susceptible subgrade soils and to develop means for correlating field test results with laboratory data.

The objective of this investigation was to conduct a field test program at the Alaskan Division* of USA CRREL near Fairbanks, Alaska, to compare seasonal heaves of test sections with nominal surcharge loads of 2, 4, 6 and 8 psi with heaves observed at adjacent unloaded control sections.

Permafrost existed at a depth of about 7 ft beneath the test site and vertical movement caused by seasonal frost heaving normally amounted to about 0.5 ft. The groundwater table was near the surface throughout the summer and at the start of the freezing season, receding to the permafrost table early in the freezing season. The soil consists of a gray or brown silt of fairly uniform gradation, containing organic matter in layers and pockets.

Background

In 1946 a runway section was constructed at the Alaskan Division to determine the effect of various pavement types, insulators and base courses on the ground thermal regime (Corps of Engineers 1950). Base courses of various thicknesses from 2 to 12 ft were used, enabling preliminary computations relating frost heave to surcharge stress to be made. These analyses gave an indication of the effect of surcharge but were severely limited by large differences in frost penetration beneath the various sections.

Laboratory tests were conducted in 1953 at the Arctic Construction and Frost Effects Laboratory (ACFEL)† to investigate the effect of surcharge on frost heave (Arctic Construction and Frost

*Then the Alaska Field Station.
†ACFEL was merged with the U.S. Army Snow, Ice and Permafrost Research Establishment (USA SIPRE) in 1961 to form the U.S. Army Cold Regions Research and Engineering Laboratory (USA CRREL).

Effects Laboratory 1958). These tests showed the relationship between rate of heave and surcharge stress for several fine-grained soils and indicated general agreement with earlier results reported by Beskow (1947). Additional laboratory test results were presented by Hoekstra et al. (1965) indicating that a pressure of about 40 psi was required to completely restrain Fairbanks silt from heaving.

One of the procedures for selecting the thickness of pavements subjected to frost conditions (Linell et al. 1963, Department of the Army 1965), the limited-subgrade-frost-penetration method, takes into account the beneficial effect of the surcharge stress imposed by the pavement and the non-frost-susceptible base course. If suitable field test data were available, it would be desirable to refine this procedure to recognize soil and moisture conditions at individual sites. This could result in more efficient designs with possible savings of non-frost-susceptible base course material.

PLAN OF TEST

Description of Site

The site selected for the field test was an undisturbed area of the Alaskan Division, which is located about 3 miles northeast of Fairbanks (Fig. 1 and 2). Trees and brush had been removed from the area about 10 years earlier and the vegetative cover which had regrown was a relatively dense grass which reaches a height of about 2 ft in summer. The ground surface was essentially level, dipping gently west. A slight ground depression, extending through the site from east to west, was located approximately through the center of what was to become the test section (section 6) under a 6.6-psi load. This depression apparently served as a natural drainage channel because during construction the soil in this area was substantially wetter and softer than that in adjacent areas to the north and south.

The climate at Fairbanks is subarctic with a mean annual temperature of 26°F and extremes of −60 and +90°F. The mean annual precipitation is about 11 in. including an annual snowfall of about 60 in. Winds average about 2 mph with a prevailing northerly direction. They are southwesterly during June and July. Long-term climatological data are compared with those for the 1960-1965 period in Table I.

Table I. Climatological summary.*

		Record		*1960-1965*
Air temperature (°F)	(36 years of record)			
Mean annual		25.7		25.5
Recorded high	(25 Jul 1971)	93	(14 Jul 1960)	89
Recorded low	(14 Jan 1934)	−66	(29 Dec 1961)	−62
Precipitation (in.)	(36 years of record)			
Mean annual		11.3		11.6
Maximum annual	(1935)	17.48	(1962)	16.62
Maximum monthly	(Aug 1930)	6.88	(Jul 1962)	4.35
Snowfall (in.)	(36 years of record)			
Mean annual		61.1		71.1
Maximum annual		116.2	(1962)	90.3
Average length of freezing season (days)				193
Average length of thawing season (days)				171
Air freezing index (°days F) (1945-1965)		5770		5678
Air thawing index (°days F) (1946-1965)		3316		3276

*Data were obtained from the National Weather Service, Fairbanks International Airport, which is located about 8 miles from test site.

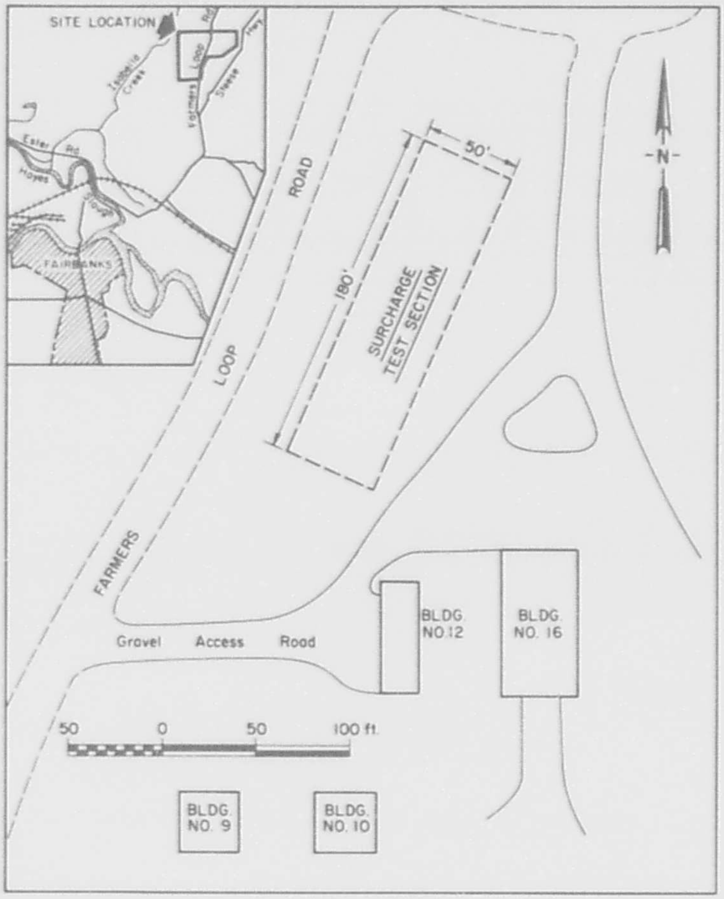


Figure 1. Site plan.

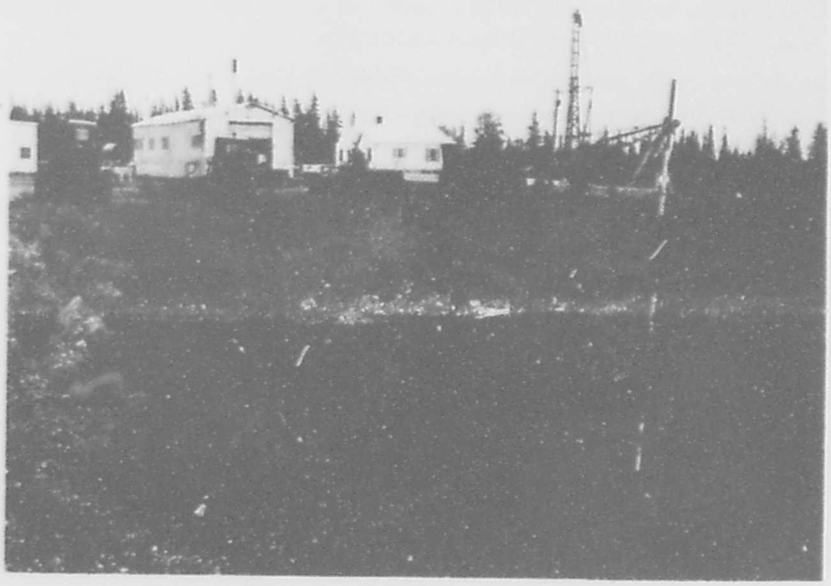
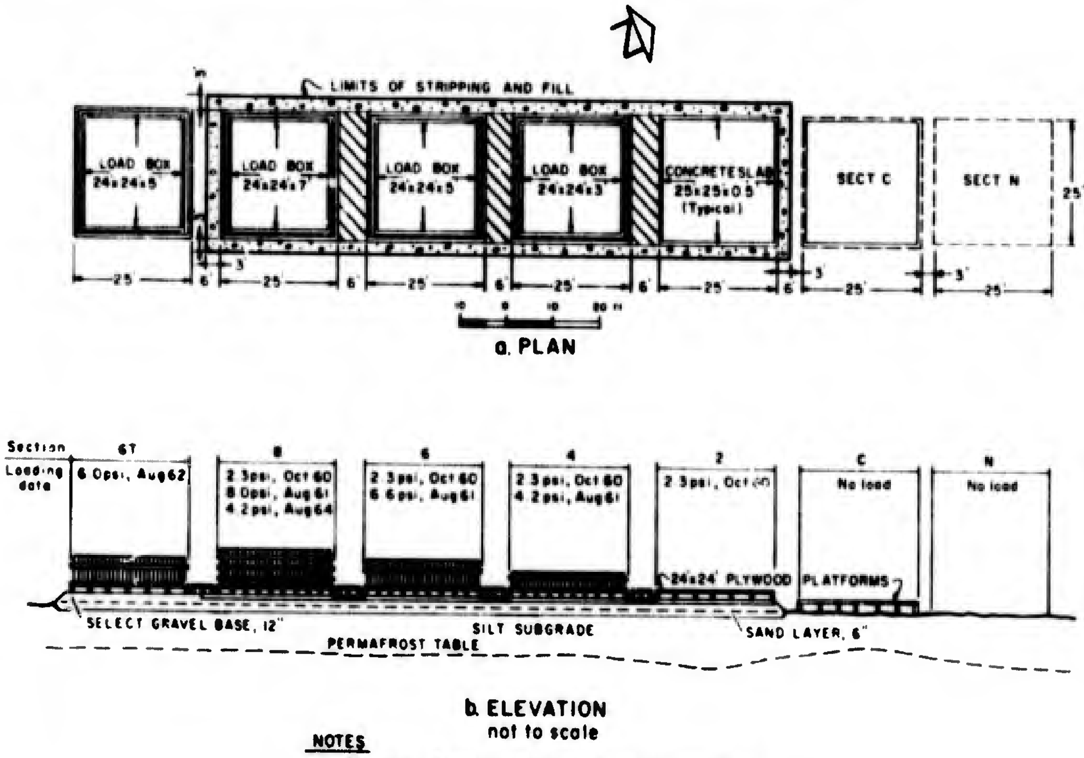


Figure 2. General view of test site, 5 July 1960.



NOTES

Concrete transition sections, shown as crosshatched on plan, were separated from main slabs by one inch expansion joints, and were covered by 6 x 24' plywood platforms

All slabs were reinforced with 6/6 welded wire mesh

Transition section, 6T, constructed 2 years after original sections

Figure 3. Plan and elevation of test installation.

Description of Test Sections and Subsurface Conditions

The test sections were designed to place surcharge stresses of approximately 0, 2, 4, 6 and 8 psi on the subgrade soil. A 25-ft-square area was used for each test because it was the largest area considered feasible to construct and instrument. It was hoped that this would be large enough to minimize edge effects at the center of each loaded area.

Construction of the test sections was started in late August 1960. Before the start of construction, a lane of steel-pierced plank was placed completely around the test area to protect the existing surface cover. No equipment, except drill rigs, was operated on the test area comprising the control and natural ground sections (sections C and N); the drill rigs were lifted onto these sections by crane to minimize surface disturbance. The plan and elevation of the completed test installation are shown in Figure 3.

The first step in construction was to remove 12 to 18 in. of surface material from the 118- by 25-ft area comprising sections 2, 4, 6 and 8, and then place a 6-in. layer of sand (filter) covered by a 12-in. layer of gravel to serve as the base course for the 6-in.-thick concrete slabs.

Placement of the base course was not completed until late September and concrete placement was not completed until late October because of delays caused by unusually heavy rainfall (precipitation for September 1960 was the highest for that month since 1925). Therefore, it was necessary to enclose the area in a heated shelter to prevent the base course from freezing and to provide suitable curing temperatures for the concrete. Four 25-ft-square, 6-in.-thick main slabs and three 6- by

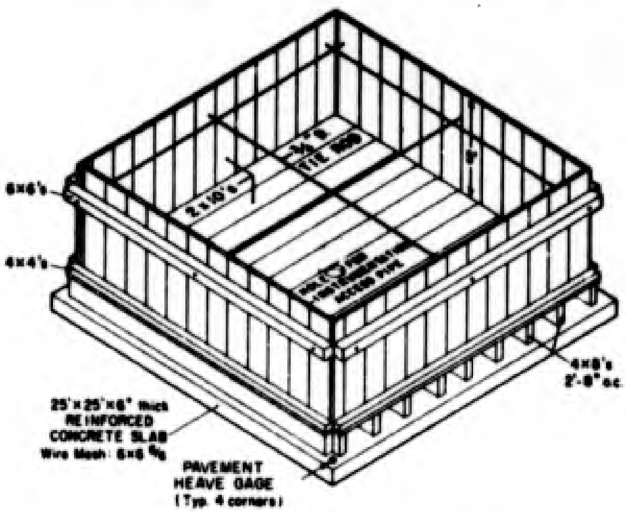


Figure 4. Load box, section 6.

25-ft transition slabs (also 6 in. thick) reinforced with welded wire mesh were poured. The main slabs were separated from the transition slabs by 1-in. premolded joint material.

Boxes to contain the gravel used for the loads on sections 4, 6 and 8 were constructed in April and May 1961. An isometric view of the section 6 load box is given in Figure 4 to illustrate typical load box construction. Plywood covers were placed on the load boxes to minimize the entrance of moisture..

Frost penetration would have varied significantly among the test sections if the load boxes had been placed directly on the concrete slabs. Therefore, to minimize nonuniform frost penetration, the load boxes were constructed on ducted bases to permit airflow beneath them. The 18-in.-thick sand-and-gravel base course and 6-in.-thick concrete provided the initial 2-psi increment of load.

The overall height of the section 8 load box was reduced to facilitate construction by substituting lead and steel weights for a portion of the gravel fill.

Figure 3 shows the two zero-surcharge sections (sections C and N) at the north end of the site and a 6-psi transition section (section 6T) at the south end. Section N was included to provide a natural-conditions reference section to permit observation of any effects of the changed snow cover and/or surface-exposing conditions introduced by the test installations. Section 6T was constructed to minimize edge effects observed on the south edge of section 8.

The transition sections between the loaded sections placed a load of 2-3 psi on the subgrade soil, since the gravel and concrete base extended over the entire loaded area. These sections and section C were covered by ducted wooden platforms to provide the same airflow and radiation characteristics as those of the loaded sections.

Placement of gravel in the load boxes for sections 4, 6 and 8 was started in early August 1961 and completed within 15 days. Performance of the sections during the loading period consisted of rapid primary consolidation of the subgrade accompanied by development of a slightly dished slab cross section.

Because the test sections were slightly elevated above the surrounding terrain, and oriented at right angles to the normal surface drainage, they acted as a barrier to surface water runoff in the spring, resulting in substantial ponding of water on the east (upslope) side.

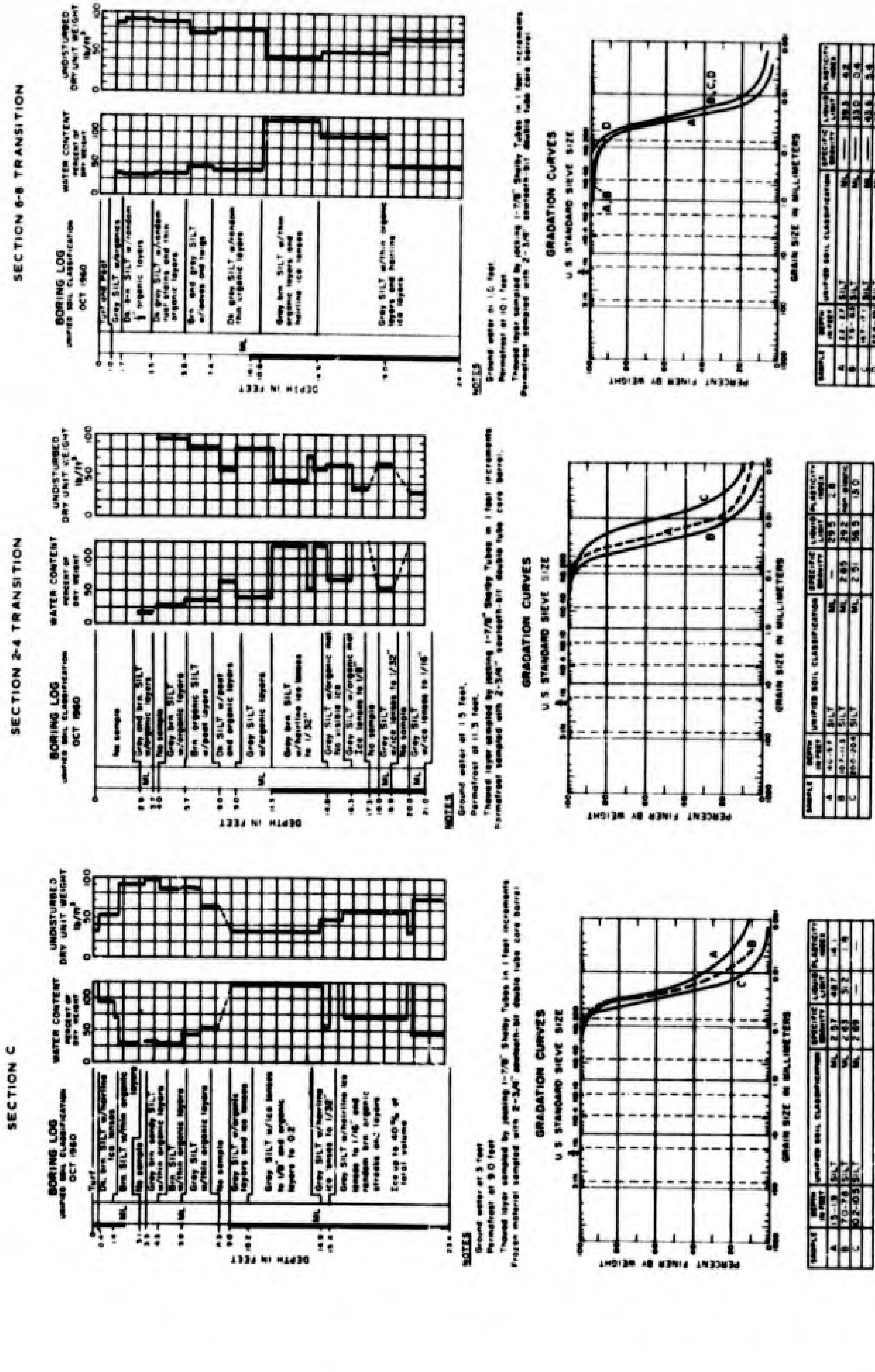


Figure 5. Boring log and soil data.

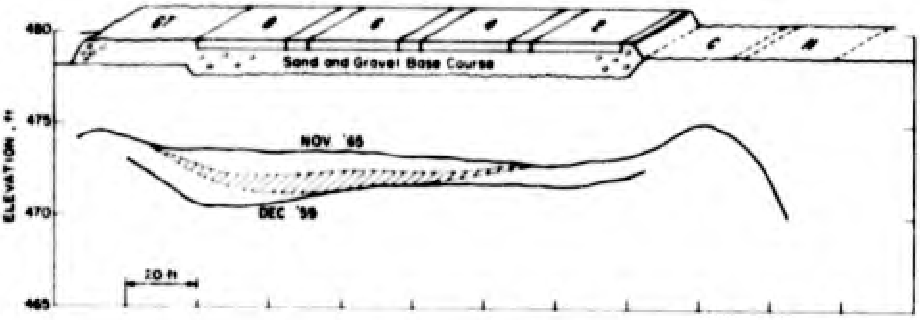


Figure 6. Depth to permafrost along centerline of test area.

Soil Data

The soil underlying the test site is silt with substantial amounts of organic matter in layers and pockets in various stages of decay.

In early fall 1960, continuous soil samples were obtained at three locations: the center of section C, the center of the transition section between sections 2 and 4, and the center of the transition section between sections 6 and 8. Gradation, density, moisture content and Atterberg limit tests were performed; the boring logs, typical gradation curves, and soil data are shown in Figure 5. The subgrade soil at the test site was classified as a silt [Unified Soil Classification System, ML, U.S. Army Engineer Waterways Experiment Station (1960)] in spite of some liquid limits greater than 50. The high liquid limits, and resulting increase in plasticity, were contributed by the partially decomposed organic material which is found in layers and pockets throughout the area. The liquid limit of the pure silt fraction was estimated at about 25.

A set of probings was made annually to record the depth to the permafrost table before the start of freeze-up. These data indicate that from 1959 through 1965 the elevation of the permafrost table was increased about 2 ft beneath the test sections (Fig. 6). This permafrost aggradation was probably the result of deep winter frost penetration facilitated by keeping the test sections free of snow and by the shading effect provided by the load boxes and platforms during the summer months. In the area indicated by the cross-hatched zone in Figure 6, the specimens were thawed when extracted from the sample tubes because of the close-to-thawing temperatures of the soil; however, based on ground temperature and probe data from other years, it is thought that the soil in this zone was frozen in November 1965. These data emphasize the marginal thermal stability of the perennially frozen soil.

Instrumentation

The location of instrumentation installed at the test site is shown in Figure 7. Vertical movement data were obtained using pavement-heave and vertical-movement gages (Fig. 8). The pavement-heave gages were installed in the spring of 1961 through 1-ft-square openings left in the concrete slabs for this purpose. The steel couplings for the gages were grouted in place with the grout bonded to the existing concrete by epoxy resin.

Reference rods and coupling extensions for the center pavement heave gages in sections 4, 6 and 8 were sleeved up through the load boxes and were accessible from the tops of the boxes. Observations were obtained on the heave gages using a machinist's depth gage having a resolution of 1/64 in. These observations established the difference in elevation between the inner reference

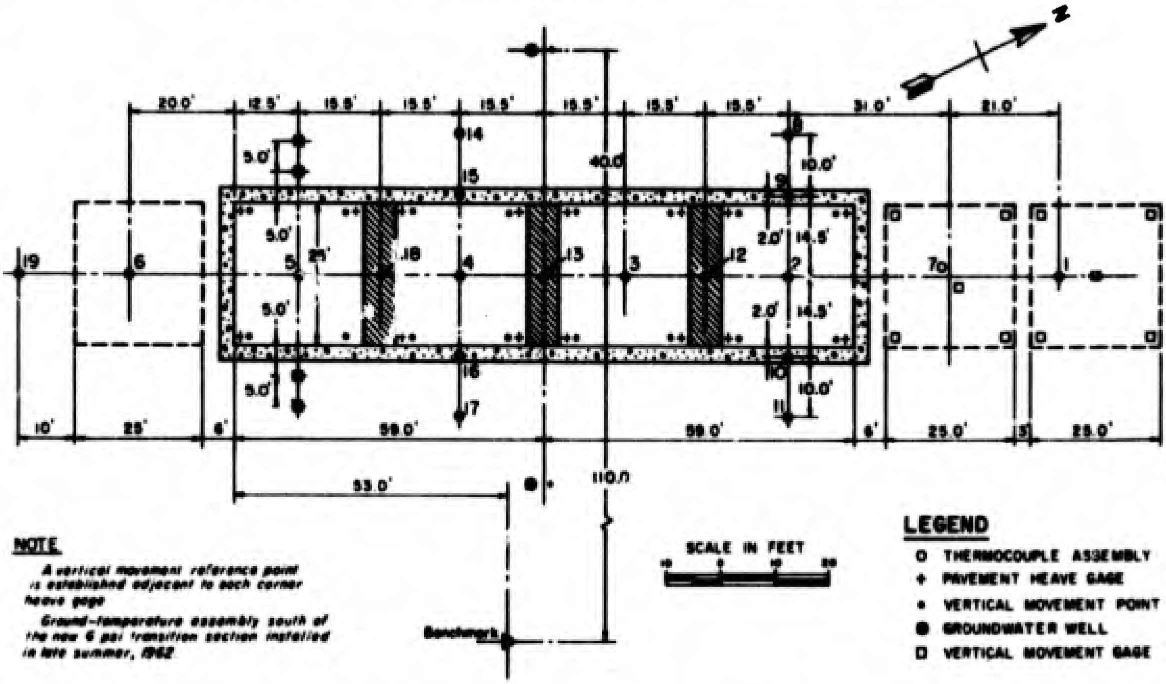


Figure 7. Instrumentation location.

Table II. Thermocouple spacing.

Depth below surface		
Assembly: 2-4, 12, 13 and 18 (ft)	*Assembly: 1, 5-11, 14-17 and 19 (ft)*	*Assembly: all (ft)*
0.0	0.0	5.5
0.5	0.25	6.0
1.0	0.5	7.0
1.5	0.75	8.0
2.0	1.0	9.0
2.25	1.5	10.0
2.5	2.0	12.0
3.0	2.5	14.0
3.5	3.0	16.0
4.0	4.0	18.0
4.5	4.5	20.0
5.0	5.0	22.0

rod, which served as a nonheaving benchmark, and the steel coupling embedded in the concrete. The vertical-movement gages in the platform-covered control section were observed by measuring the displacement between the gage reference rod and the steel angles attached to the platform over each reference rod. All other vertical-movement gages had steel H-section reference frames resting on the ground surface to produce a ground pressure of less than 0.5 psi.

Ground temperatures were obtained using copper-constantan thermocouple assemblies located as shown in Figure 7. Each assembly consisted of 24 thermocouples spaced as shown in Table II.

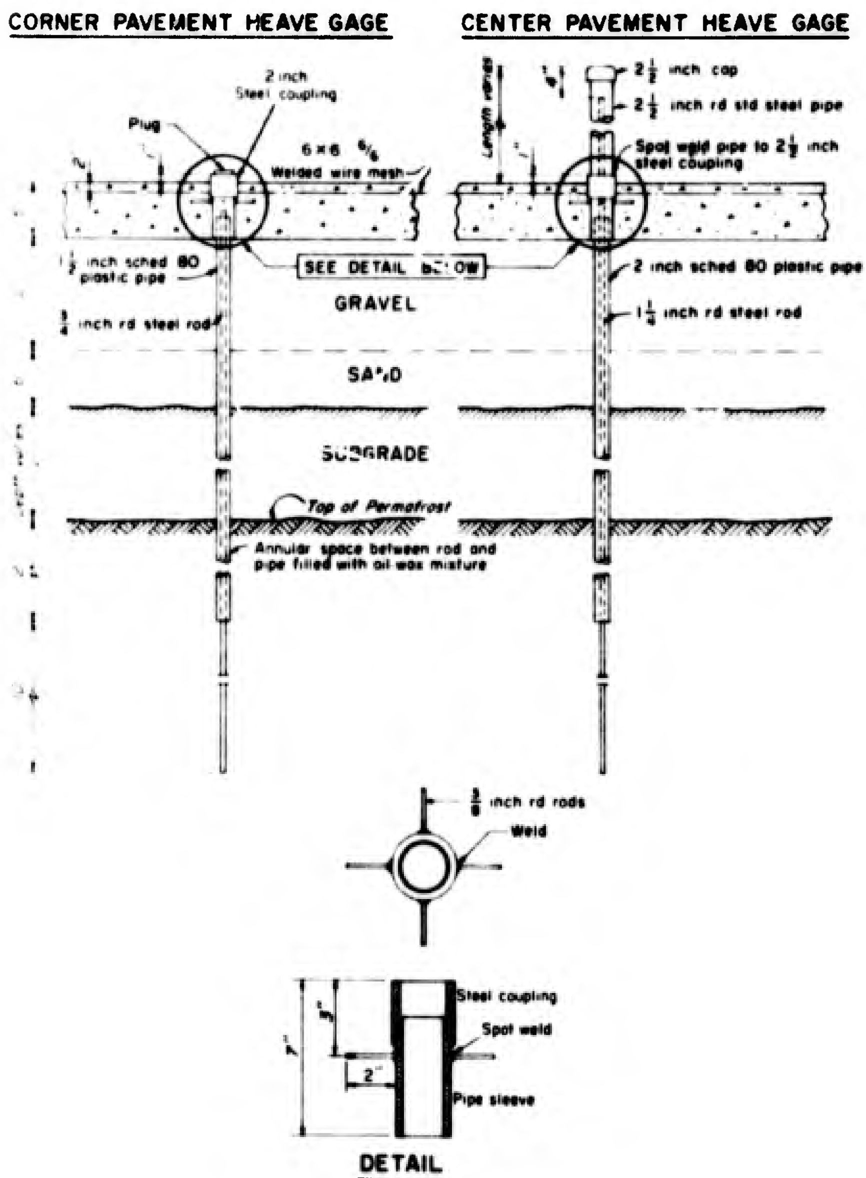


Figure 8. Heave gage details.

The 18-gage individual lead wires in each assembly were bundled together inside 3/4-in.-diam polyethylene tubing which was then filled with a mixture of 30% oil and 70% wax. With the exception of the five top sensors, the thermocouple junctions were inside the oil-wax-filled tubing. The five top sensors were brought out of the tubing on extension leads to permit accurate placement as the assembly was installed. The main portion of each assembly (below about 2 ft in the subgrade) was placed in a 4-in.-diam core-drilled hole which was backfilled with a soil-water slurry made from the cuttings removed from the hole. Typical slurry water contents ranged between 60 and 80%.

Tubing-encased leads from all assemblies were run to weathertight electrical junction boxes and connected to 24-point, copper-constantan panel boards. The assemblies were monitored using a portable precision millivolt potentiometer with an ice-bath reference junction. The temperature data were used to calculate frost penetration depths by plotting temperature-versus-depth (gradients) curves and using the depth at which the gradient crossed the 32°F isoline as the depth of frost penetration.

Considering the computational procedure, the accuracy of the measured temperatures (± 3/4°F), and the precision of the temperature sensor location (± 0.1 ft), it is estimated that the precision of the frost penetration data is about ± 0.25 ft. It is not possible to define the frost penetration curve over the entire freezing season using this method because towards the end of the season the soil in the area of the freezing interface is essentially isothermal.

The depth to groundwater table was measured at two groundwater wells (Fig. 7). Observations were obtained in these wells using a manually operated electric-water-level indicator.

The concrete slabs were checked for occurrence of any significant local deformation beneath the load boxes using a miniature cart on which a short section of level rod was mounted. This cart was towed through the load box ducts and slab elevations at desired locations were obtained by reading the rod with an engineer's level.

The observations taken at the test site are summarized in Table III.

Table III. Observational summary.

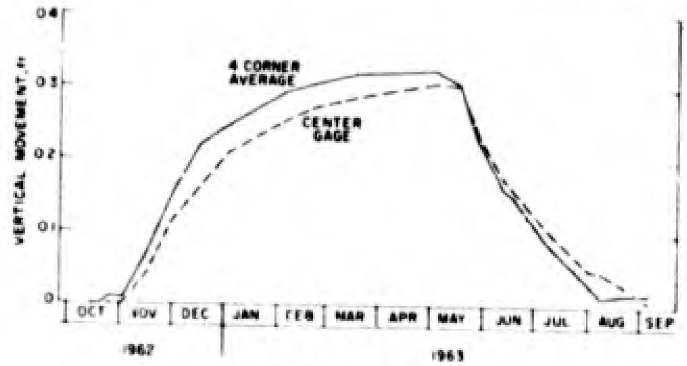
Observation	*Inclusive dates*	*Frequency*
Ground temperatures	Nov 60-Mar 66	Weekly
Level observation, sections 2, 4, 6, 8, N	Nov 60-Jun 61	Weekly
Pavement heave and vertical movement	Jul 61-Mar 66	Weekly except 3/wk during Oct-Mar 1962-63, 63-64, 64-65
Level observation, all sections	Jul 61-Mar 64	Monthly
Level observation, all sections	Apr 64-Mar 66	3/yr
Groundwater level	Jun 61-Nov 62	Biweekly June-Sept Weekly Sept-freeze-up
Slab deformation	Jun 63 and 64	Annually

DATA INTERPRETATION

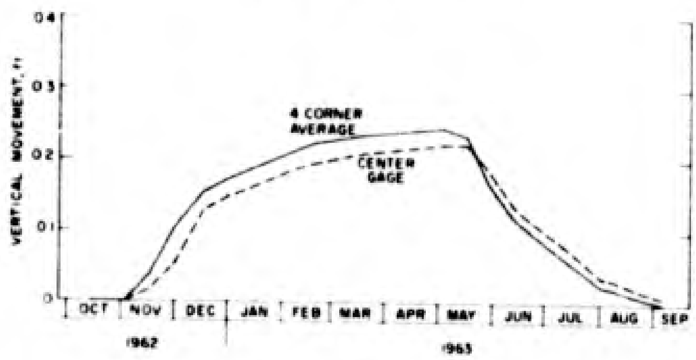
Heave Gage Data

The vertical movement data discussed here are those obtained from the center pavement heave gages in sections 2, 4, 6 and 8 and averages of data from the five vertical movement gages in sections C and N. The vertical-movement/time curves, based on both 4-corner-average and center gage data for the loaded sections (Fig. 9), show that center gage data give total seasonal vertical movements about 0.02 ft less than 4-corner-average data because of edge effects. Figure 10, showing seasonal frost penetration on a transverse plane through section 6, suggests that deeper subgrade frost penetration beneath the edges of the slab early in the freezing season when there is still an abundant supply of soil moisture available contributes significantly to edge effects.

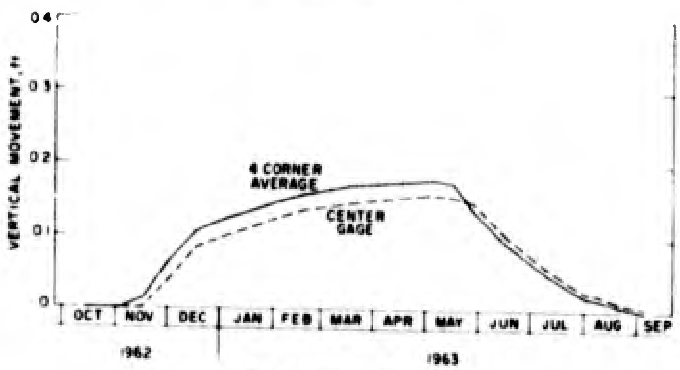
For the loaded sections, the center heave gage vertical movement data are used to minimize the influence of edge effects. For sections C and N, which are not influenced by edge effects, the data discussed are the average from all five gages.



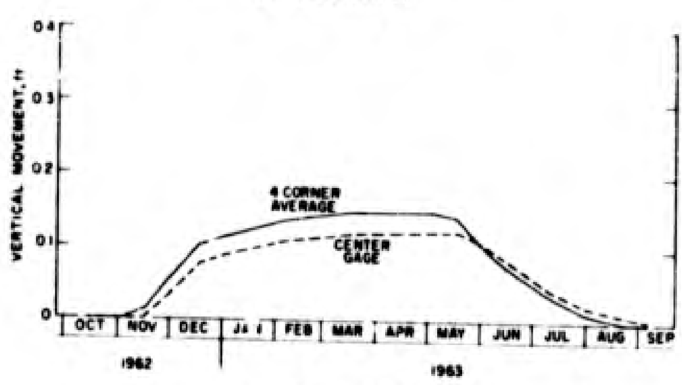
a. *Section 2.*



b. *Section 4.*



c. *Section 6.*



d. *Section 8.*

Figure 9. Vertical movement/time data comparison, sections 2, 4, 6 and 8.

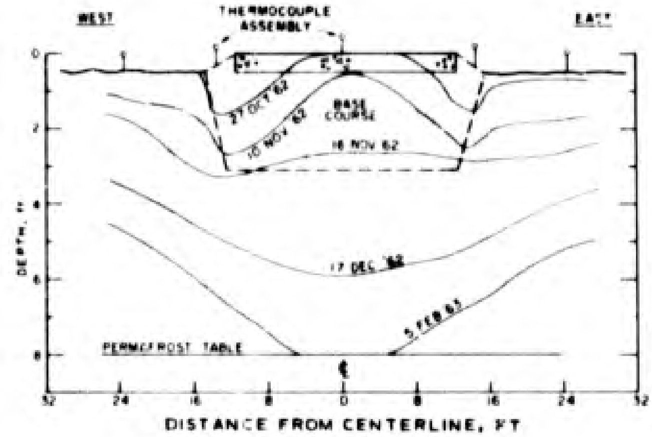


Figure 10. Transverse frost penetration, section 6.

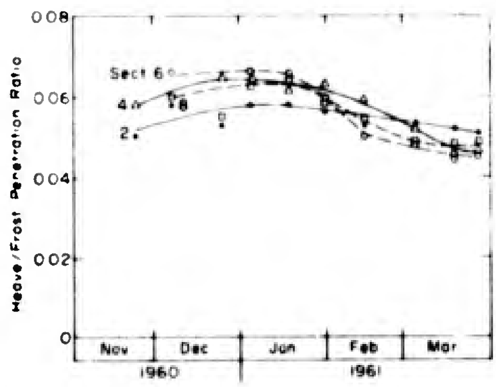


Figure 11. Heave/frost penetration ratio, 1960-61.

Uniformity of Subsurface Conditions

The uniformity of subsurface conditions at the site can be estimated using the data of 1960-61, before the load boxes were constructed, when sections 2, 4, 6 and 8 were all under a 2.3-psi surcharge stress. Figure 11 presents curves of the heave/frost penetration ratio vs time for November 1960 through March 1961. If subsurface conditions remained the same throughout the freezing season, it is suggested that the heave/penetration ratio should be the same for all sections. The data point spread reflects the normal range of error in the frost penetration measurements, while the changes in slopes of the curves suggest changes in the effective stress at the freezing front. This stress change is related to changes in pore water and overburden stresses at the freezing front.

Considering these factors and noting the similar shape of the curves for all sections, it appears that the subsurface conditions, although not static throughout the freezing season, were reasonably uniform from section to section.

DISCUSSION OF RESULTS

General

A continuous record of vertical movement for October 1960 through October 1965 is presented in Figure 12. Key factors relating to performance of the test sections during this period are listed below:

1. In July 1960, a 6-ft-wide by 2-ft-deep water ditch was excavated about 45 ft east of the test section centerline, extending the entire length of the test site, to ensure an adequate supply of water to completely saturate the soil, thus facilitating maximum frost heave. The ditch was maintained full of water from July through September 1960 and was then drained (because of unusually heavy rainfall) to assist in drying out the site during construction. Because extremely wet soil conditions were encountered during construction and because groundwater observations indicated that the piezometric level was at or only slightly below the ground surface at the end of subsequent thawing seasons, this ditch was not used after the 1960 summer.

2. As noted earlier, placement of the base course and concrete slabs was completed in the fall of 1960 but sections 4, 6 and 8 were not loaded until August 1961. Therefore, the sections were all under a 2.3-psi load for the 1960-1961 freezing season. All sections were under full design loads for the 1961-62 through 1963-64 freezing seasons.

3. A 6-psi-load transition section (section 6T, Fig. 3) was constructed before the start of the 1962-1963 freezing season to minimize boundary effects on the southerly side of section 8.

4. In August 1964, the load on section 8 was reduced to 4.2 psi to evaluate the effect of local moisture and density variations on performance during the 1964-1965 freezing season. Loads on the other sections were unchanged.

5. Observations were continued during the 1965-1966 freezing season to obtain data concerning the effect of surcharge on moisture migration, with the sections under the same loads as for the 1964-1965 season.

To allow easier evaluation of seasonal performance, the heave portions of the vertical movement curves (Fig. 12) were recomputed on a seasonal basis with subgrade frost penetration curves included on the same time scale. Examination of these data for the 1960-1961 freezing season (Fig. 13), when sections 2, 4, 6 and 8 were under a 2.3-psi load, shows greater heave of sections 2 and 8 than of sections 4 and 6. A major portion of this difference was attributed to deeper subgrade frost penetration beneath sections 2 and 8 early in the freezing season. It is suggested that the remainder was due to increased uplift caused by the one-third greater slab perimeter exposed to unsurcharged soil. The longitudinal heave curves (Fig. 14) show that in mid-February 1961 the outside ends of sections 2 and 8 had heaved 0.08 and 0.12 ft, respectively, more than the inside edges, whereas only minor differential heave was shown by sections 4 and 6. Most of the differentials had developed before 6 December 1960 in the period of least surcharge loading at the freezing plane and high moisture availability. This emphasizes the importance of an equal start of frost penetration beneath all test sections if heave performance is to be comparable.

The seasonal heave-time curves together with the subgrade-frost-penetration-time curves for 1961-1962 are given in Figure 15. Subgrade frost penetration for sections 2 and 8 was still ahead of that for sections 4 and 6 (section 2 only through mid-December), but not as much so as during the 1960-1961 season. The longitudinal heave-time profiles for this season (Fig. 16) show that section 2

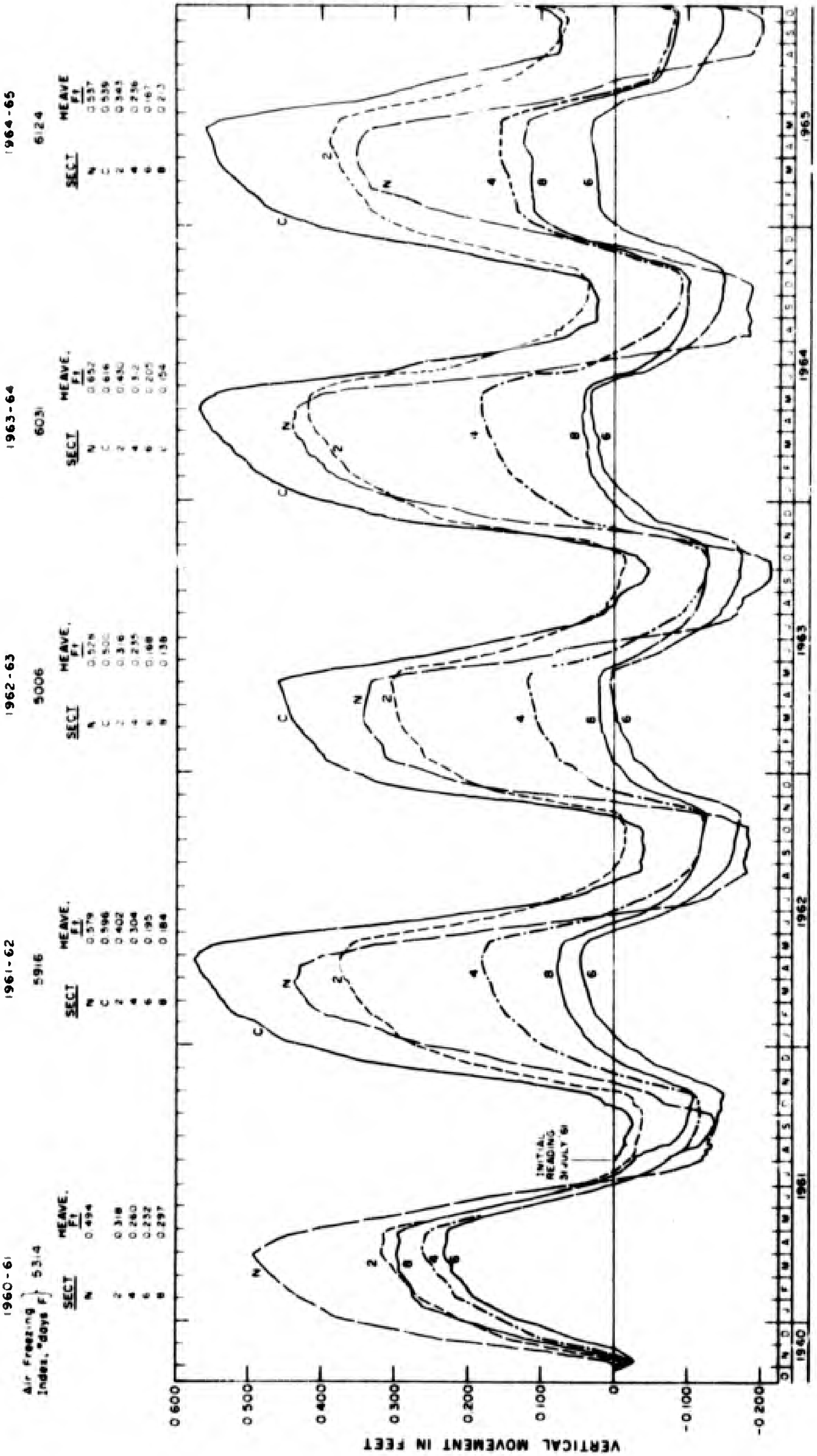


Figure 12. Vertical movement versus time, October 1960 through October 1965.

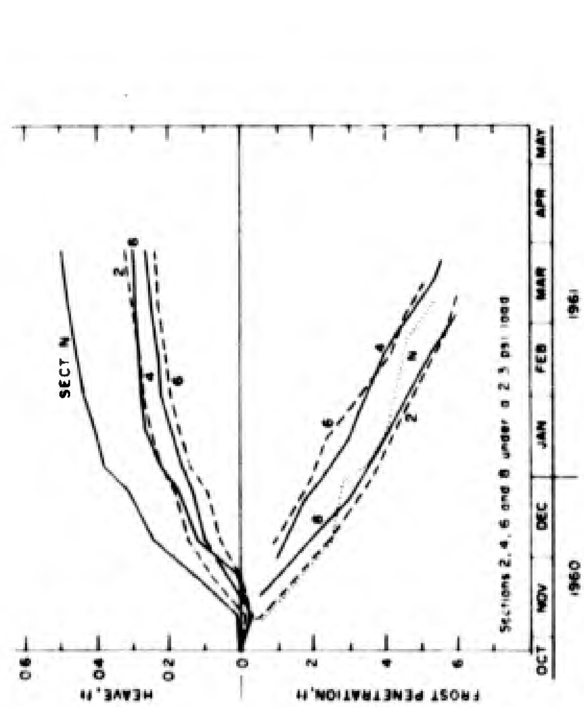


Figure 13. Field data, 1960-61.

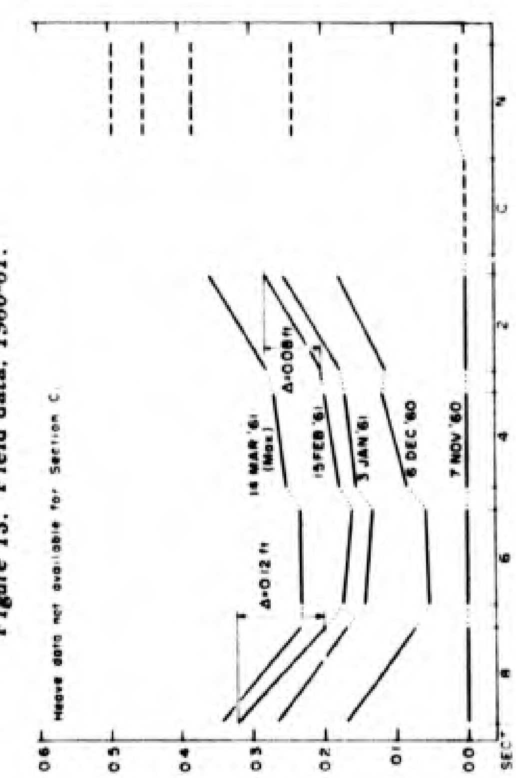


Figure 14. Longitudinal heave profiles, 1960-61.

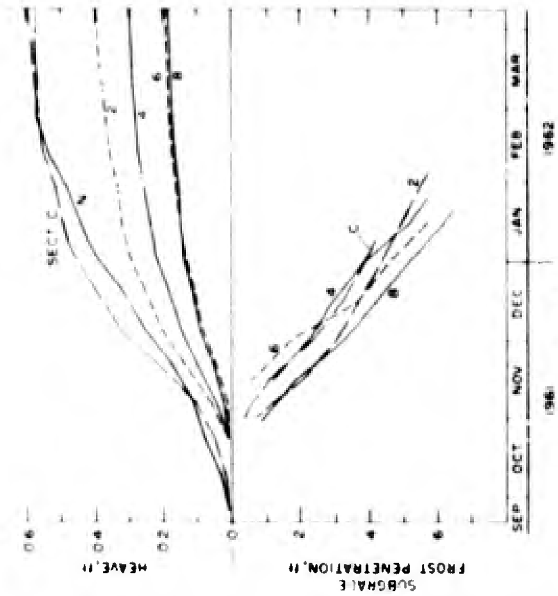


Figure 15. Field data, 1961-62.

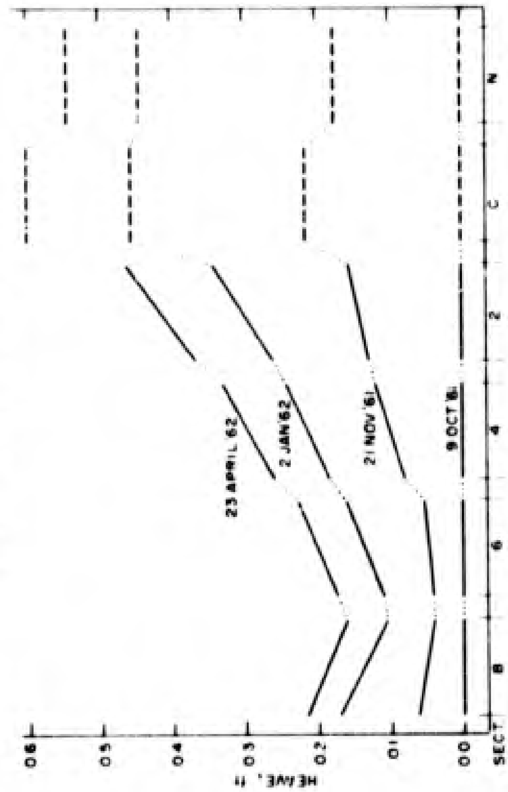
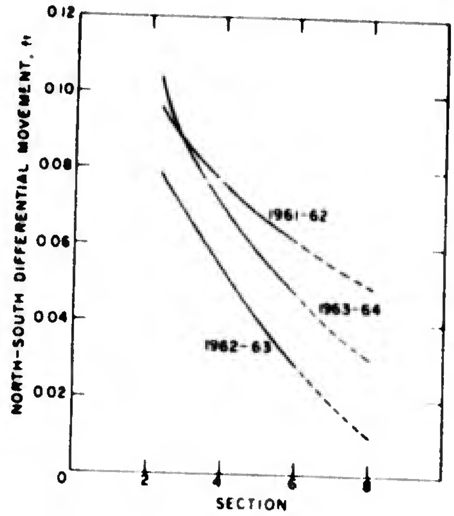


Figure 16. Longitudinal heave profiles, 1961-62.



SECTION 8 DATA

	NORTH EDGE MEAS (FT)	SOUTH EDGE MEAS (FT)	SOUTH EDGE COMPUTED* (FT)	CENTER MEAS (FT)	CENTER COMPUTED† (FT)
1961-62	0.160	0.217	0.114	0.184	0.137
1962-63	.145	.159	.135	.138	.140
1963-64	.169	.179	.143	.154	.156

*NORTH EDGE MINUS Δ FROM CURVE.
†AVG OF NORTH AND COMPUTED SOUTH.

Figure 17. North-south differential movement data.

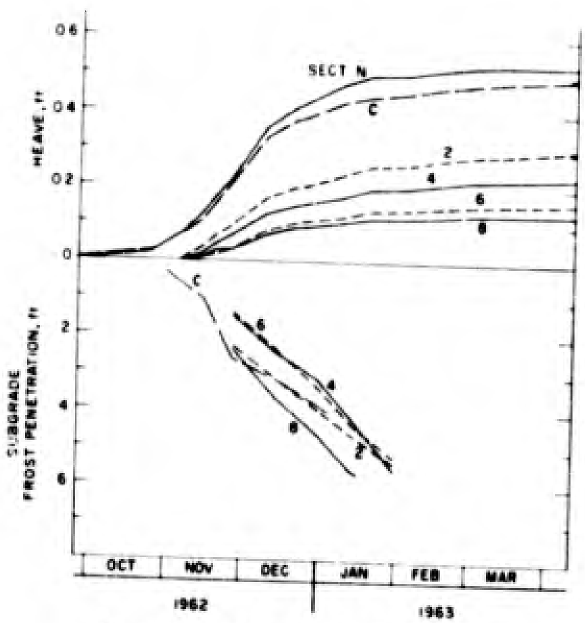


Figure 18. Field data, 1962-63.

exhibited a differential heave of about 0.1 ft on 23 April 1962. Sections 4 and 6 were also influenced by the lighter loads immediately north of them, resulting in north-south differential movements of 0.07 and 0.05 ft, respectively.

Examination of the 1961-1962 data indicates that a major parameter affecting the magnitude of differential heave exhibited by the loaded sections was magnitude of load on the adjacent section(s) and suggests that to directly compare heave data for sections 2, 4 and 6 with those for section 8 a 10-psi loaded section would be needed south of section 8. As it was physically impracticable to construct such a 10-psi section, a 6-psi section (section 6T, Fig. 3) was constructed in late summer 1962 to minimize the error introduced by the sudden 8- to 0-psi load drop at the south end of section 8. This section was not instrumented for vertical movement.

Section 8 data for 1961-1962 were corrected for edge uplift by extrapolating the differential movement from sections 2, 4 and 6 to obtain an estimated differential movement for section 8 (Fig. 17). This estimated differential heave and the measured heave at the north end of section 8 were used to compute the section 8 center gage heave. The influence of section 6T on section 8 performance was verified by performing a similar extrapolation using 1962-1963 and 1963-1964 data obtained after completion of section 6T. Measured and computed data are summarized in Figure 17.

The data for the 1962-1963 freezing season are given in Figures 18 and 19. Again, subgrade frost penetration under sections 2 and 8 was slightly deeper than that under sections 4 and 6 throughout the majority of the freezing season. The significant change in this year's data is the substantially reduced heave differential across section 8 in the north-south direction (Fig. 19) resulting from the addition of section 6T. The influence of section 6T is also shown in Figure 20, which indicates that the addition of this section reduced the uplift on the south end of section 8

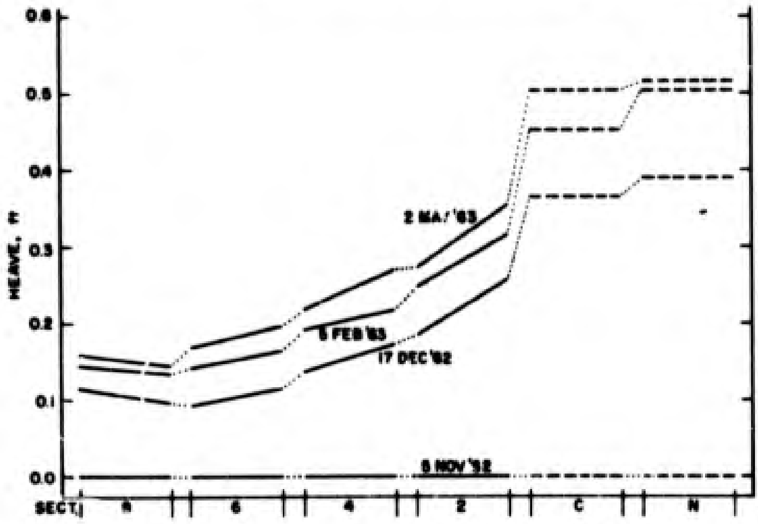


Figure 19. Longitudinal heave profiles, 1962-63.

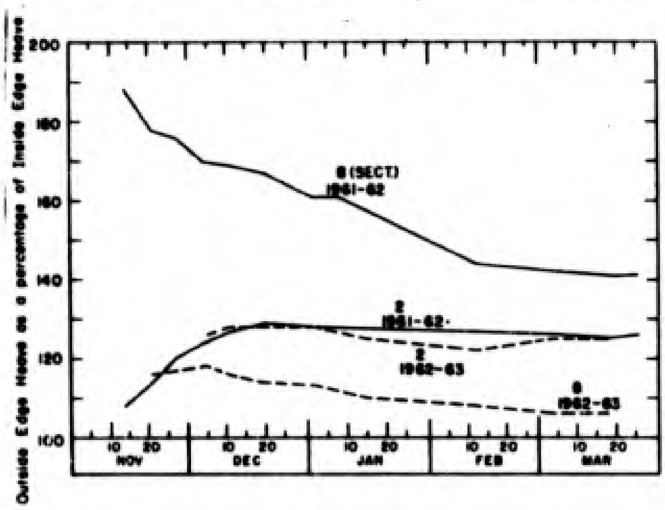


Figure 20. Effect of 6-psi transition, section 6T.

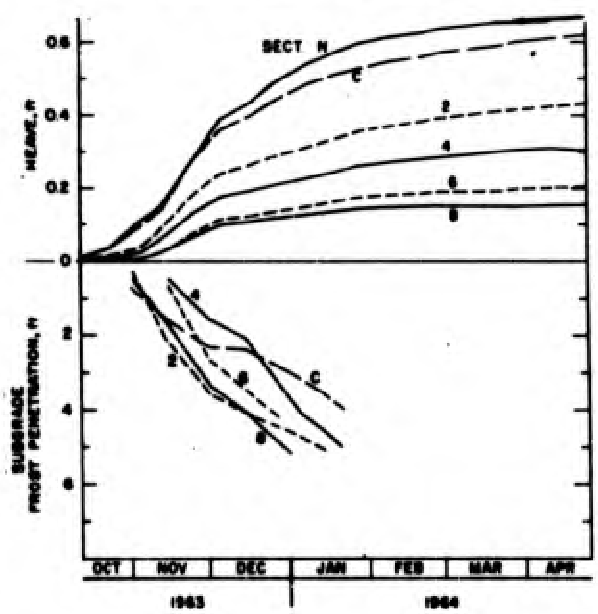


Figure 21. Field data, 1963-64.

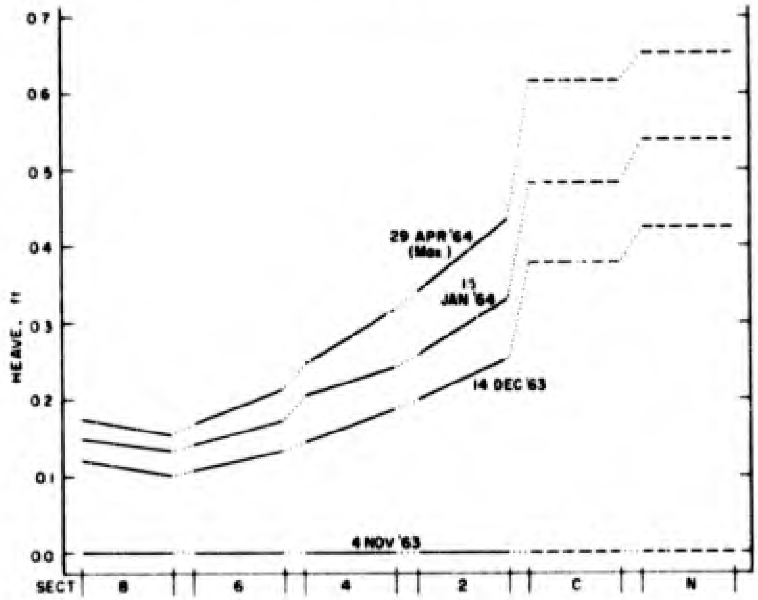


Figure 22. Longitudinal heave profiles, 1963-64.

from up to 1.9 times its north edge heave in 1961-1962 to only about 1.2 times in 1962-1963. Data from section 2, showing essentially no change in boundary effect from 1961-1962 to 1962-1963, are included for comparison.

The 1963-1964 freezing season was relatively severe, as reflected in the heave-time curves by heave of over 0.6 ft for sections C and N in Figure 21. In spite of the large cumulative vertical movement exhibited by all of the test sections during this year, edge uplift of section 8 was well controlled by section 6T, as shown in Figure 22. Also note that frost penetration under sections 2 and 8 was more than that under sections 4 and 6 during this freezing season.

Effect of Freezing Intensity

The effect of seasonal freezing index was considered to determine whether seasonal heave data directly reflect variations in the air freezing index. The cumulative seasonal movements of all sections, when expressed as a percentage of section C movement (Table IV), show that the reduction in heave accomplished by the applied surcharge stress was essentially independent of variations in freezing index during this period.

Table IV. Percentage of 0-psi heave (center gage data).
Data from Figure 12 with section C as base.

Season	Freezing index	Section C	2	4	6	8
		(% of 0-psi heave)				
1961-1962	5916	100	67	51	33	23
1962-1963	5006	100	63	47	33	28
1963-1964	6031	100	70	50	33	25
Average	5651	100	67	49	33	25

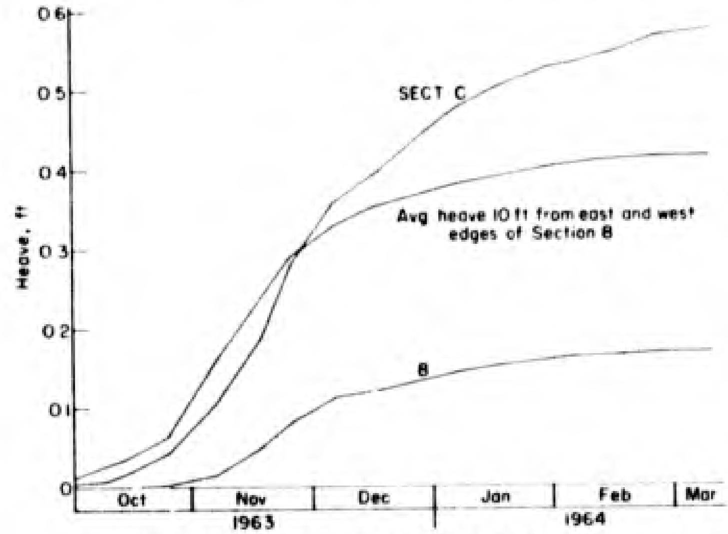


Figure 23. Area of influence of section 8.

Area of Influence of Surcharge Load

The area of influence of the surcharged sections can be estimated using the data in Figure 23. They show that 10 ft from the east and west edges of section 8 maximum heave of the unsurcharged soil was 70% as much as that at section C, while the heave of section 8 was 70% less than that of section C. This restraining effect on the unsurcharged soil adjacent to section 8 was not apparent until the end of November. At this time, frost penetration beneath section 8 was about 3 ft.

Effect of Surcharge Stress on Heave

The relationships between surcharge load, subgrade frost penetration and average heave are shown in Figure 24. The average heave of section C, after an initial linear phase, shows an increase per unit of frost penetration until about 4 ft of frost penetration has occurred. The heave of this section then tapers off, most probably because of exhaustion of the groundwater supply. The heave versus frost penetration data for section 2 are of similar form but with a substantial reduction in magnitude of heave for any particular depth of frost penetration. It is suggested that the relatively low heave of section 2 early in the freezing season, as compared with that of section 4, was caused by differences in the availability of groundwater.

The shape of the curve for section 4 is significantly different from the shapes of sections C and 2. Its linearity suggests that the heave of this section is not dependent to any great extent on groundwater moving to the section, but rather that it is the result of expansion during freezing of the soil moisture already available beneath the section at the start of the freezing season. The curves for sections 6 and 8 are very linear and have essentially equal slopes of about 0.02 ft of heave per ft of frost penetration, again suggesting the freezing of a small but uniformly distributed supply of soil moisture.

These data suggest that applying a surcharge load inhibits migration of groundwater to the freezing face beneath the load. This is in general agreement with the theoretical mechanism for heaving described by Chalmers and Jackson (1963, 1970). They show that the free energy available during the phase change of soil moisture must lift the frozen soil above the interface and move

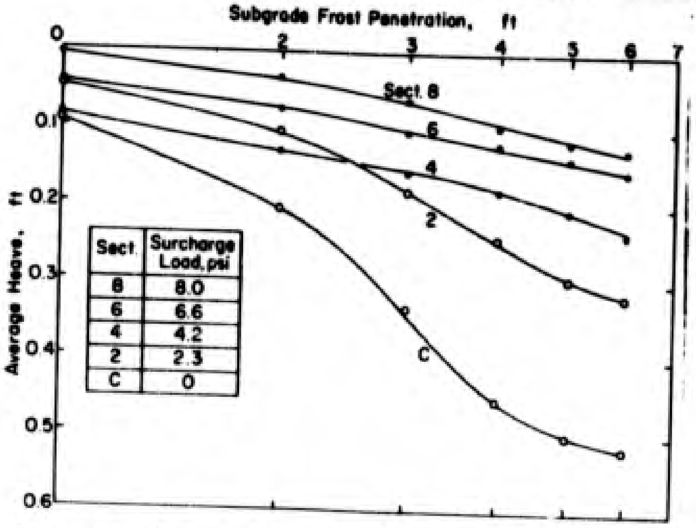


Figure 24. Subgrade frost penetration versus heave and surcharge load.

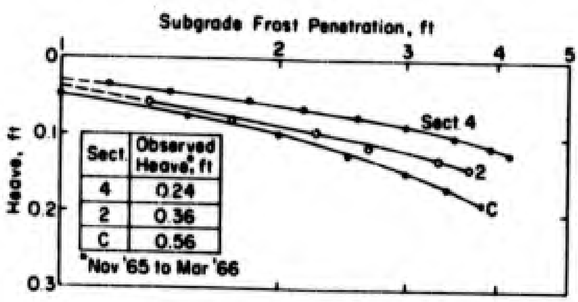


Figure 25. Heaves calculated from November 1965 soil data.

additional soil moisture to the interface. If a surcharge load is applied to the system, more of the available energy is needed to lift the increased load, with the result that less energy is available to move soil moisture. If a roughly uniform distribution of soil moisture is assumed at the start of the freezing cycle, expansion of this moisture as it freezes would produce linear heave-penetration curves like those for sections 6 and 8 in Figure 24.

Soil samples were obtained beneath some of the test sections in November 1965 to determine the soil moisture distribution just before a freezing season. Heaves calculated from the moisture content and density data obtained from these samples are plotted in Figure 25 for sections C, 2 and 4. These data corroborate the hypothesis that a linear heave-frost penetration relationship could be obtained at the test site if little or no moisture migration took place.

Effect of Load Reduction on Heave

The surcharge load on section 8 was reduced to 4.2 psi in August 1964 to determine whether its heave would then be close to that of section 4. The subgrade frost penetration during this season was similar to that of the preceding season (Fig. 21), but the different heaves are readily apparent in Figure 26.

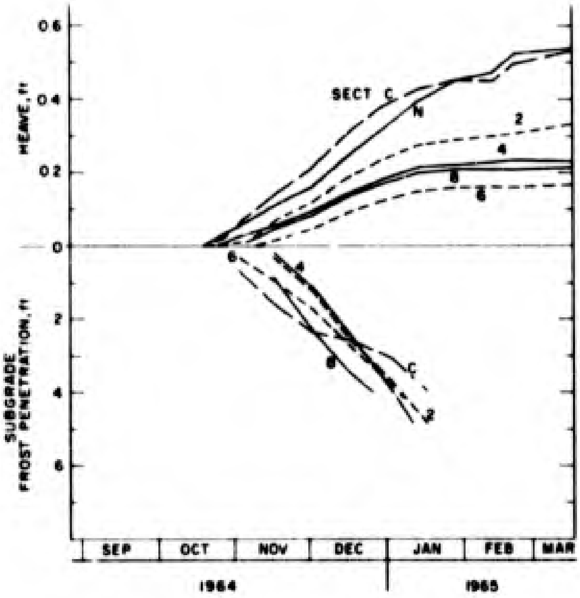


Figure 26. Field data, 1964-65.

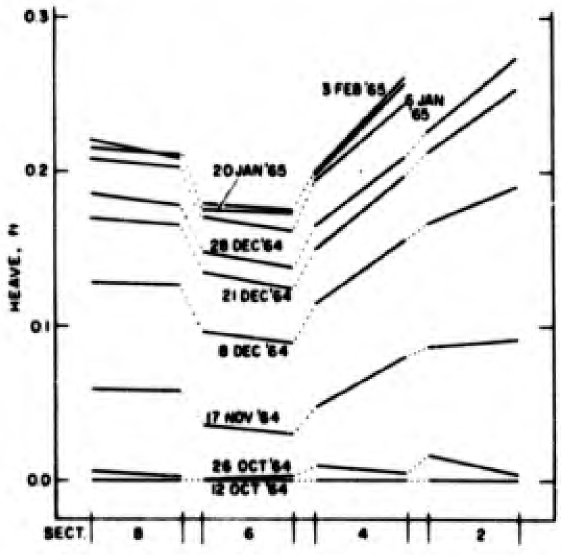


Figure 27. Longitudinal heave profiles, 1964-65.

The longitudinal heave profiles (Fig. 27) show only minor differential heave for section 8 because of the equal loads applied on its north and south boundaries. Section 4 was influenced by sections 2 and 6 during this freezing season as it had been previously; so to compare its performance with that of section 8 (4.2 psi) the section 4 data were adjusted assuming a 6-psi load instead of the existing 2.3-psi load immediately north of it. This adjustment was made by comparing the data from the two gages on the south (low) end of section 4 with those from the center gage of section 8 (4.2 psi). Using this method, total heaves of 0.20 and 0.21 ft were computed for sections 4 and 8 (4.2-psi-load), respectively. This agreement suggests that any variance in subsurface conditions at the test site did not contribute to the results. It also increases confidence in the methods used herein to correct the heave data for edge effects and the influence of adjacent sections under different loads.

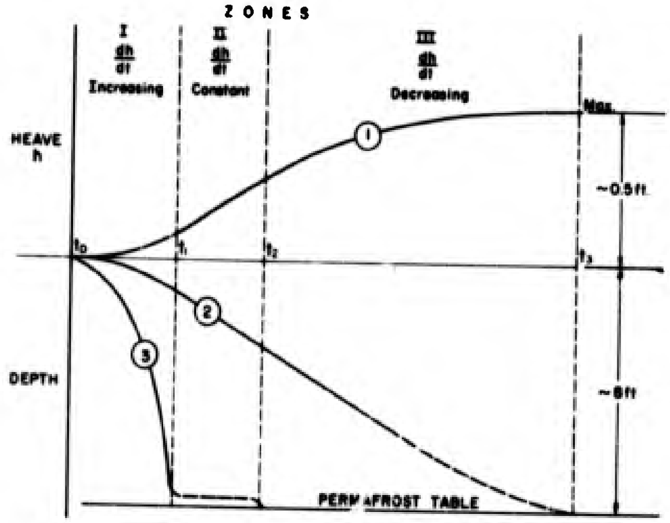


Figure 28. Idealized heave-versus-time relationship.

Comparison of Field Data With Laboratory Test Results

A substantial amount of laboratory test data is available for several soils showing the relationship between rate of heave and surcharge stress (ACFEL 1958, Aitken 1963, Beskow 1947). These laboratory data were obtained by freezing small specimens of soil unidirectionally from the top. The heat removal rate was varied during the tests to try to maintain a constant rate of heave, and the soil samples were supplied with an unlimited amount of water at their base.

The effective stress at the freezing interface during such tests is about equal to the surcharge load if side friction between the test specimen and its container is neglected. In order to compare these laboratory data with the field data reported herein, similar conditions, i.e. constant rate of heave and a known effective stress at the freezing interface, must exist.

An idealized heave-time curve (1) for the field test site, together with companion frost penetration curve (2) and piezometric surface curve (3), is given in Figure 28. The heave-time curve may be used to divide the freezing season into three time zones based on rate of heave, dh/dt. During the first zone period, the rate of heave increases while only a small amount of subgrade frost penetration takes place. The piezometric level is initially at the freezing interface; therefore soil pore water stress is initially negligible. The pore water stress increases throughout the time zone as the piezometric surface recedes, and greater suction is required to pull water to the freezing interface. Rate-of-heave data obtained during this time period are obviously not comparable with those from the laboratory tests.

During the zone II period, the rate of heave at the field test site is constant. This constant rate of heave was observed for about 30 days; about 40% of the seasonal heave occurred during this period. The stress at the freezing interface due to frozen overburden increased at a linear rate because of the linear rate of frost penetration. It is suggested that the pore water suction decreased at a rate sufficient to balance out the increase in overburden during this period because the frost line advanced closer to a groundwater supply near the permafrost surface. Groundwater well data indicated that such a water supply was available at the test site. Soil pore water pressures at the freezing interface can at best be only crudely estimated, but during this period they would obviously be negative to provide suction to pull soil moisture to the freezing surface. If they were about 1 or 2 psi negative, this would be sufficient to counteract the positive pressure caused by the frozen

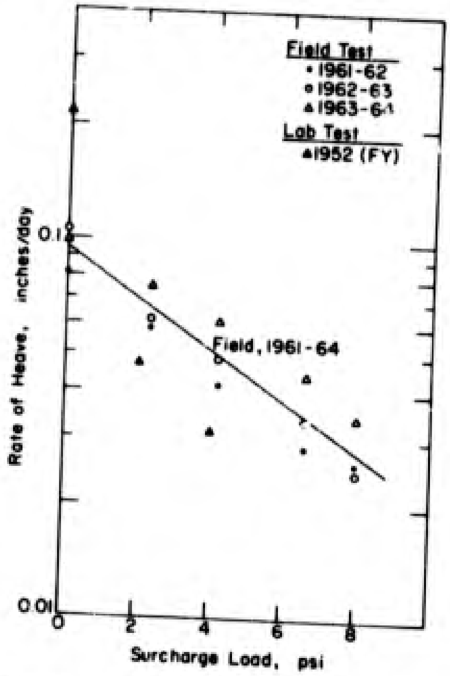


Figure 29. Rate of heave-versus-surcharge load.

overburden above the freezing interface. The effective stress at the interface would then be equal to the surcharge stress as it was in the laboratory tests.

The rates of heave observed during the zone II period are plotted versus surcharge load in Figure 29. Laboratory data from tests on Fairbanks silt (ACFEL 1958, vol. I, plate 3) are included for comparison. The laboratory results obtained at 2 and 4-psi surcharges appear to substantiate the hypothesis that comparison of field with laboratory results is possible. The high rate of heave of the laboratory specimens with no surcharge compared with that of section C is attributed to the overriding influence of the availability of unlimited free water in the laboratory tests.

CONCLUSIONS

Seasonal frost heave of a silt soil can be significantly reduced by relatively small surchage loads. The average reduction at the Fairbanks site was over 30% with a 2.3-psi surcharge and about 75% with an 8-psi surcharge. This could be accomplished because, at the Fairbanks test site, groundwater migration contributed substantially to heave. At section C, 70% of the observed heave was caused by water that was not within the freezing zone beneath the section at the start of the 1965-66 freezing season.

It is unlikely that heave could be completely eliminated by a surcharge load in such practical applications as highways or airfields because of the very high stresses needed. However, the data discussed herein suggest that a surcharge load might supplement drainage facilities to minimize heaving caused by moisture migration.

It appears feasible to compare results of field and laboratory tests on surcharged soils using a constant rate-of-heave approach. This would permit the effect of surcharge loads on different soils to be evaluated without additional field testing.

The area of influence of a surcharged section on reduction in heave was greater than anticipated. Significant reductions in frost heave were observed 10 ft away from the 25-ft-square section 8.

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