FROST INVESTIGATIONS AT DOW FIELD, BANGOR, ME.

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SYNOPSIS

An investigation was performed at Dow Field, Bangor, Maine, during the period February through August 1944 to determine the effect of frost action in the subgrade soil beneath three paved areas upon the load supporting capacity of the pavements. The investigation consisted of the following components:

(a) Detailed explorations and tests to determine pavement, base and subgrade conditions in the selected test areas.

(b) Observations of pavement heaving and ice lens formation in the subgrade during the freezing period.

(c) Performance of traffic tests using wheel loads from 10,000 to 40,000 lb. during the frost melting period.

(d) Field CBR tests upon the subgrade soils and field bearing tests upon the pavements during the frost melting and summer periods.

Three different pavement constructions were tested: (a) a 7-in. non-reinforced concrete pavement with about 15 in. of gravel base constructed partly upon a lean clay and partly upon a silty till subgrade, (b) a 4-in. plant mix bituminous concrete pavement with about an 18-in. gravel base constructed largely upon a lean clay with limited areas of silty till subgrade, (c) a 4-in. plant mix bituminous concrete pavement with a 3-ft. gravel base constructed upon a lean clay subgrade. In all test areas ground water was at or very close to the bottom of the gravel base.

The maximum frost penetration during the winter averaged 4 ft. Heaving of pavements ranged from about zero in the thick gravel base areas to 0.6 ft. in the areas of thinnest base. Ice lenses were observed to be thin and widely spaced in the subgrade immediately beneath the base becoming thicker and closely spaced at the maximum depth of frost penetration.

During the frost melting period, one pavement failed under traffic and two pavements showed definite indications of distress, traffic on one being stopped before failure was reached. On all test areas, daily traffic corresponding to the range of traffic expected under usage by planes was applied.

The investigation indicates conclusively the necessity for considering the effects of frost action in subgrades upon the plane supporting capacity of pavements constructed thereon.

The occurrence of frost action in the subgrade beneath an airfield pavement may result in the rapid failure of substantial portions of the paved area during the frost melting period. Failures of this nature cannot be tolerated at military installations. It is impractical to reduce plane wheel loads and limit air traffic during the critical period to prevent such failures. Thus, it is necessary to design and construct over subgrades susceptible to frost action airfield pavements which can support planes of the desired size regardless of seasonal conditions.

As a guide to design of flexible pavement bases, the Engineer Department has established the following design criteria where frost susceptible subgrades are encountered.

"20-23.[1] DESIGN OF BASE OVER SOILS AFFECTED BY FROST: The stability of some soils is greatly reduced by frost action. The detrimental effect of frost action occurs during the thawing periods when the moisture in the subgrade, accumulated in the form of ice segregation, is released, thereby softening the soil. The frost action of some soils also causes detrimental heave of pavement or treated surface. The degree to which soils will lose their

[1] Chapter XX, Engineering Manual, War Department, Office, Chief of Engineers, Washington, D. C.

stability and heave will depend upon the type of soil, variation of temperature during freezing and thawing, the permeability of the soil, and the drainage conditions. Observations have shown that a high ground water table is not necessary to cause a soil to lose stability due to frost, since soils affected by frost always retain, in the deeper (lower) zones, moisture which will rise during frost action.

"*a. Soils Affected by Frost.* The only practical method of determining the frost-action characteristics of a soil, without exhaustive laboratory tests, is by reference to the grain size distribution. Several investigators have established limiting grain size curves defining the boundary between frost-action and nonfrost-action soils for uniform and non-uniform grading. For design purposes, soil containing 3 per cent or more of grains smaller than 0.02 mm. in diameter should be considered as potentially capable of serious frost action. In all questionable cases, the percentage (by weight) of sizes below the critical diameter of 0.02 mm. should be determined. Very plastic soils heave only moderately due to the extremely low permeability and consequently limited volume of moisture which can be supplied, but the contained moisture may be sufficient to induce a reduction in bearing value during periods of thaw.

"*b. Base Requirements for Stability.* The most generally accepted method of insuring pavement or base course stability over a potential frost action soil is to provide a sufficient thickness of insulating material not affected by frost action (clean, well-graded gravelly or sandy soils, or fine grained soils treated by proven methods). The required thickness of base course material as determined by the California Method may not be sufficient to preclude frost penetration in the subgrade or lower zones of the base course, even though the base course material itself is relatively nonfrost heaving, hence it is often required that additional thickness of insulating material be provided to insure proper protection against frost action. Prior to the determination of the required thickness of nonfrost action soil, the range and average of frost penetration beneath pavements in the region should be be determined from local records. In general, to insure stability of flexible pavements and surface treated bases, the combined thickness of pavement and nonfrost action base material should be equal to the average depth of frost penetration except that the maximum required thickness (in locations where frost is a factor) is as follows:

Gross Weight of Plane Used for Design	Maximum Required Combined Thickness of Pavement and Nonfrost-Action Base Material
lb.	*in.*
120,000	40
74,000	30
30,000 or less	20

The above maximum thickness should be used whenever the underlying soil affected by frost is fine-grained (see Exhibit I of this Part). If the soil affected by frost is a coarse-grained soil (See Exhibit I of this Part) or the base is composed of insulating material such as slag, cinders, etc., the maximum required thicknesses may be less than the values shown above depending upon the character of the insulating, soils and drainage conditions, but in no case should the maximum required thickness be less than 75 per cent of the above values. Nonfrost-action material used to insure stability of underlying soil, subject to frost action, should be thoroughly drained (see Chapter XXI DESIGN OF DRAINAGE FACILITIES) and should conform with the requirements of paragraphs 20–20 and 20–21 above. In lieu of providing an insulation layer over a frost heaving subgrade it may be feasible and desirable to remove the frost-action subgrade material, particularly where it is of local occurrence."

As a guide to design of rigid pavement bases, the Engineer Department has established the following design criteria where frost susceptible subgrades are encountered.

"20–46.[2] DESIGN OF BASE COURSE THICKNESS ON SOILS AFFECTED BY FROST ACTION:

"*a. Reduction in Bearing.* The bearing value of a subgrade soil susceptible to frost action will be reduced during periods of thaw. Where climatic and moisture conditions are favorable, soils described in paragraph 20–23 above and cohesive soils will be adversely affected by frost action. Although cohesive soils heave only slightly, their bearing value will be greatly reduced. The design of a pavement will depend upon the reduced bearing value of such a soil, unless it is removed and a base course is constructed to a sufficient depth to provide suitable reinforcement. Accurate methods of evaluating the actual bearing value in soils affected by frost action are not known. Highway experience indicates that in areas which are subject to frost action the base course of nonfrost-heaving material under a 6 or 7-inch

[2] ibid. Chapter XX.

slab, as used for highway loads, should extend to a depth of at least 50 per cent of the average frost penetration, in order to provide suitable subgrade reinforcement. For the heavier wheel loads being used in airfield pavement design, the base course thickness required will be greater than that used in highway design. The base course thickness so determined will govern only when it exceeds the thickness determined as outlined in paragraph 20–45.

"*b. Frost Heaving.* Under conducive climatic and moisture conditions, certain subgrade soils (see paragraph 20–23 above) will heave due to frost action, causing the pavement to rise during a freezing period and settle during the spring or thawing period. If the movement is not uniform for the total area, cracking of the pavement will occur. To prevent cracking, a base course should be constructed to produce uniform movement. If the subgrade consists of both frost-heaving and nonfrost-heaving soils, highway experience has indicated that all of the frost-heaving soil should be removed to the full depth of the average frost penetration. The areas should be backfilled with a compacted select material to provide uniformity. It should be noted that cohesive soils may be used in the lower layers of thick base courses, if the soil has sufficient bearing value to provide a suitable modulus of soil reaction at the surface of the base course for pavement design."

Recognizing that the foregoing design criteria were preliminary, the Engineer Department is now carrying out a comprehensive frost investigation program to develop detailed design criteria for airport pavements. The tests described herein, which are a part of this investigation, were performed at Dow Field, Bangor, Maine in 1944. Observations were made of frost action, and laboratory and field tests were conducted. Tests to determine the effect of traffic on pavements upon subgrades affected by frost action consisted of applying daily traffic to six selected test areas during or immediately after the frost melting period. It was assumed that plane traffic on runways was represented by four daily coverages and plane traffic on taxiways by 40 daily coverages of the wheels of the traffic equipment. Wheel loads of 10,000, 20,000, 30,000 and 40,000 lb. were obtained using available rubber tired construction equipment. Two different pavements, portland cement concrete and bituminous concrete were tested. The base under all pavements was a bank run sand and gravel and ranged in thickness between 15 in. and 3 ft.

Plate bearing tests were performed at and adjacent to the six test areas during the frost melting period and repeated again at adjoining locations during the summer. The purpose of the bearing tests was to investigate the possibility of using this test as a measure of the reduction in supporting capacity of a pavement due to frost action in the subgrade.

A report is now being prepared which will include in detail the information summarized herein.

DEFINITION AND DESCRIPTION OF FROST ACTION

Frost action is defined as the physical phenomena by which layers or lenses of ice are built up within a soil mass at the boundary between frozen soil and unfrozen soil. Three conditions must simultaneously occur for these ice layers to form as follows:

(a) *Soil*—Frost action within a soil is a function of its void size. For convenience grain size has been found to be a satisfactory criteria to separate frost susceptible soils from those not susceptible. As defined by Casagrande[3] the frost action susceptible soils are those of uniform grading with greater than 10 per cent, and those non-uniformly graded with greater than 3 per cent finer than 0.02 mm. in size.

(b) *Water*—Frost action depends upon the availability of water either by virtue of an adjacent ground water table, a capillary supply, or as water within the soil voids.

(c) *Temperature*—Frost action within soils requires the maintenance of freezing temperature slightly below the surface of ice lens formation. The greatest accumulation of ice will occur when the penetration of the freezing temperature is slow; a rapid penetration may result in few or no ice lenses.

The process of frost action may be described as follows. The water in the void spaces becomes cooled below the normal freezing temperatures of water. This super cooled water has a high molecular attraction and travels to ice crystals which form in the larger

[3] Casagrande, Arthur—"Discussion on Frost Heaving," *Proceedings*, Highway Research Board, Vol. 11, p. 168 (1931).

voids. Upon contact with ice crystals the water solidifies. This process repeated forms an ice lens. A single lens will continue to grow in thickness, always against the direction of heat transfer, until the formation of a lens at a lower elevation cuts off the source of water, or until the temperature rises above freezing.

Frost action in a subgrade soil beneath a paved surface will result in heaving of the pavement during the winter. The heaving may be either uniform or non-uniform. Non-uniform heaving is characterized by a rough riding pavement with sometimes prominent cracks at the crests of bumps in bituminous concrete and cracks and differential displacements of concrete pavements. During the freezing period it is evident that the pavement has its greatest load supporting capacity by virtue of the frozen pavement, base and subgrade.

However, during and immediately after the spring thaw the pavement, within a very few days, reaches its least load supporting capacity. The reduction in load supporting capacity will depend to a large extent upon the rapidity of the thaw and the type of subgrade soil in which frost action occurred. A rapid thaw will result largely in melting from the surface downward. As the subgrade is thawed, the excess water from the ice lenses will be unable to drain except upward, downward drainage being prevented by the remaining frozen subgrade beneath. Lateral drainage may be prevented by shoulders still frozen. The excess water, depending upon its amount, may cause either a minor reduction or a large reduction in the shearing strength of the soil. Observations have been made of subgrades which were literally fluid during the melting period. A slow thaw with occasional periods of freezing may permit the excess water to flow from the soil without any or only very minor reduction in shearing strength.

Highway engineers in New England have learned by experience that frost susceptible soils must be covered by a pavement and base not susceptible to frost action and of sufficient thickness to prevent pavement damage by highway traffic during frost melting periods. This thickness for highway loads in northern New England is generally less than the average frost penetration, except in unusual cases. Recently constructed flexible type pavements with bases on frost action susceptible soils vary between 2 and 3 ft. total thickness in Maine and about 2 ft. in New Hampshire.

LOCATION OF TESTS

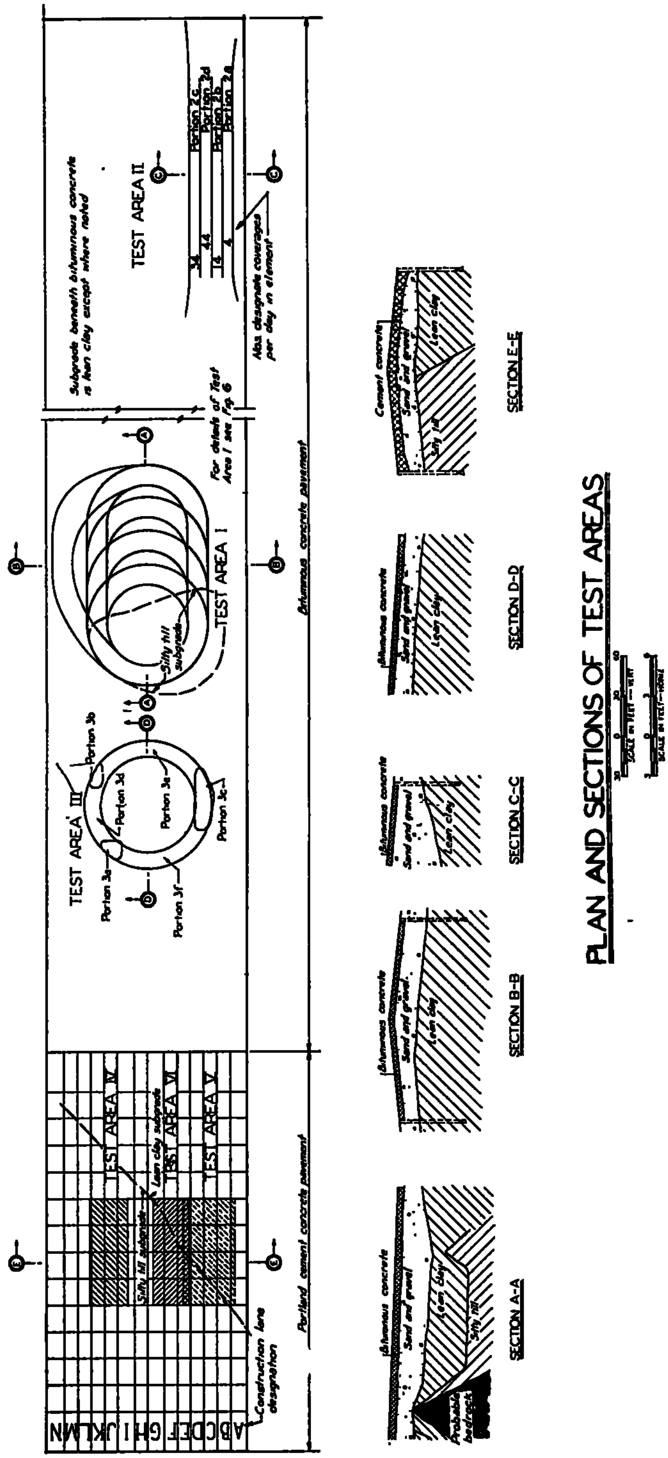
Dow Field at Bangor, Maine was selected for this investigation because it offered one of the best opportunities in northern New England to study the effect of frost action in subgrade soils beneath an airfield pavement. Conditions of subgrade, pavement and base, temperature, ground water, and past performance indicated that frost action could be expected to occur. At Dow Field the East-West runway was chosen for the tests. Figure 1 shows the relative locations of the several test areas together with sections showing pavement, base and subgrade details for each area.

WEATHER DATA

The temperature and precipitation data as recorded by the CAA Weather Station at Bangor, Maine are summarized on Figure 2 for the winter of 1943–44. The temperature data are plotted as a curve of time vs. degree days below freezing. Degree days below freezing have been computed by cumulating the difference between the average of the daily maximum and minimum and 32 deg. The maximum number of degree days below freezing, which is the ordinate between the top of the curve at the start of freezing and the bottom of the curve at the end of freezing is defined as the "freezing index." The freezing index is a measure of the severity of the winter. On Table 1 are summarized the computed freezing indices for the three months prior to start of freezing for the years 1938 through 1944. These values indicated that the winter of 1943–44 was a severe winter. The total precipitation recorded for Sept. Oct. and Nov. 1943 is 7.2 in. greater than the normal for this period, indicating that high ground water could be expected at the start of freezing weather.

SUBGRADE SOILS

Two different types of subgrade soils were encountered, referred to herein as "silty till" and "lean clay." The location of the two types of soil is shown on the plan and sections



PLAN AND SECTIONS OF TEST AREAS

Figure 1

on Figure 1. The silty till is the older of the two soils and was deposited as a heterogeneous mass during the period of glaciation. The lean clay was deposited during or after the retreat of the ice sheet in quiescent water. The top zone of both soils has since become weathered by alternate wetting and drying and frost action. The weathered zone was not fully removed during construction in the

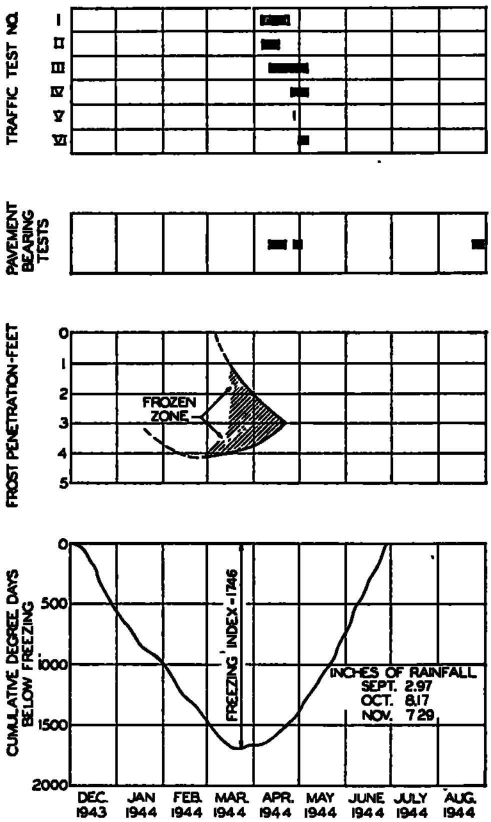


Figure 2. Testing Periods, Frost Penetration, and Temperature vs. Time

test areas. These two subgrade soil types are described in detail in the following subparagraphs:

(a) *Silty Till* consists of a well-graded, brown to gray, compact, slightly plastic, soil whose particle sizes are generally sub-angular and vary from about 6 in. to finer than 0.001 mm. When dry the soil is readily crumbled between the fingers. Evidences of weathering in the top zone of this soil are not readily discernable. Excavation of the unfrozen silty till by hand tools was difficult due to its high density and stone content.

(b) *Lean clay* consists of a lean to moderately plastic clay which is highly weathered at the top as marked by its mottled yellow and gray color and many minute fissures and pockets; it changes gradually in depth becoming moderately and occasionally highly plastic, soft, and of gray blue color at depths

TABLE 1
FREEZING INDICES FOR BANGOR, MAINE—1938 THROUGH 1944

Year	Freezing Index
1938–1939	1480
1939–1940	1550
1940–1941	1454
1941–1942	1180
1942–1943	1537
1943–1944	1746

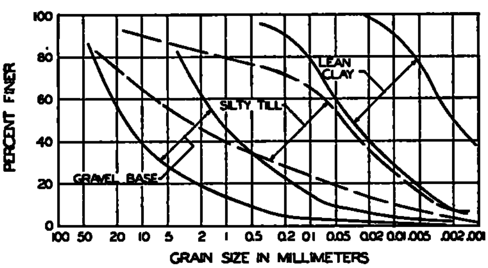


Figure 3. Range of Gradation of Gravel Base and Subgrade Soils

TABLE 2

	Lean Clay Range of 4 Samples	Silty Till One Sample
Liquid Limit	29–33	19
Plastic Limit	18–21	17
Plasticity Index	11–13	2
Shrinkage Limit (one sample)	17	
Specific Gravity		
Sizes retained on ¼-in.		2.60
Sizes passing ¼-in.	2.77	2.79

of 10 ft. or more. In some excavations a columnar structure in the weathered zone was noticed. Excavation of the unfrozen soil was easily made by hand tools. Upon drying the soil has sufficient cohesion so that it is not possible to crush a small cube between the fingers.

The range of gradation of the two subgrade soils is shown on Figure 3. The results of Atterberg limits and specific gravity determinations are summarized in Table 2.

The density and water content of the two subgrade soils as measured in place during the summer are as given in Table 3.

The California Bearing Ratio of both subgrade soils was determined by in-place field tests during the summer. These tests indicate that the CBR of the lean clay was between 5 and 16 with an average of 10 and the CBR of the silty till as indicated by two tests was 60 and 120. Remolded tests upon the lean clay are not applicable due to the destruction of soil structure. Remolded tests upon the silty till are considered representative and these indicate a value of 100 as average.

The CBR of both soils will be considerably less than the above values during and immediately after the frost melting period due to the excess water content of the soil. Tests in place on the lean clay immediately after the thaw averaged 4.

TABLE 3

Subgrade Soil	Average Unit Dry Weight lb. per cu. ft.	Average Water Content % Dry Weight
Lean Clay	102 to 107	21 to 24
Silty Till	125 to 128	11 to 12

FROST ACTION

During the winter observations in test pits indicated that ice lenses had developed in both the silty till and the lean clay subgrade. The ice lenses in the lean clay were in general thin and relatively widely spaced at the top of subgrade and became thick and closely spaced at the frost line as shown by the photograph, Figure 4. Continuous water content determinations including both soil and ice taken from the top of the base to a depth of 6 ft. indicate that the water content in the silty clay subgrade increased from a normal value of about 25 per cent at the top to a value of about 35 per cent at the frost line, below which the water content dropped immediately to about 24 per cent. This increase in water content is in agreement with the observed distribution and thickness of ice lenses. Figure 5 compares a set of continuous water contents in the lean clay and gravel base during the winter with a set during the summer at adjacent test pits. In the silty till, the water content at the top of the frozen subgrade averaged about 19 per cent and increased gradually to 25 per cent at the frost line, then dropped to 12 per cent. After the thaw the silty till had an average water content of about 12 per cent.

Observations of the heaving of the pavement surface are summarized on Table 4. Differential heaving was evident in the concrete pavements by displacement of one slab with respect to an adjacent slab. From the measured water contents during the frozen period and during the summer, the amount of pavement heave due to frost action may be

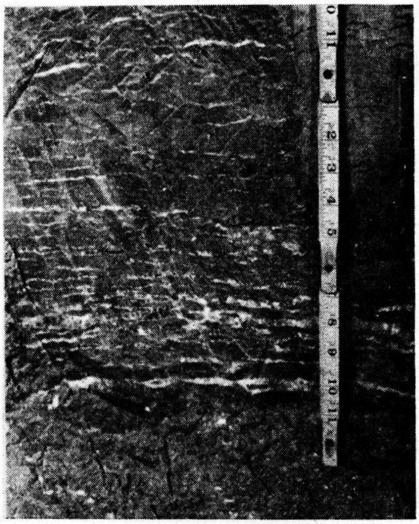


Figure 4. Ice Lens Formation in Lean Clay Subgrade

computed. In the area adjoining Test Area I where the frozen depth of the lean clay subgrade averaged approximately 2 ft., the average difference in water content between the frozen period and after the thaw is 3 per cent for the top 1 ft. and 11 per cent for the bottom 1 ft. These two values represent 3-in. heave of the pavement surface assuming that the 10 per cent expansion of water to ice is compensated by the compression of the clay between ice lenses. This computed heave compares favorably with the average measured heave in Test Area I of 2.5 in. In Test Area VI where the frozen depth of the silty till subgrade averaged approximately 2 ft., the average difference in water content between the frozen period and after the thaw is 8 per cent for the top 1 ft. and 13 per cent for the bottom 1 ft. These two values represent 5-in. heave of the pavement surface which value is equal to the average measured heave in Test Area VI.

A comparison between the measured heave of the pavement surface at several test pits and the cumulative total thickness of ice lenses measured in these pits is summarized on Table 5. These comparisons indicate fair agreement between the two sets of observations.

two-course, hot plant mix bituminous concrete with gravel and sand aggregates of total thickness averaging from 3.5 to 4.0 in. The binder course in Areas I and III had a heavy grade tar cement; the remainder of the bituminous concrete had an asphalt cement. Visual observations of the pavement indicated that in local areas the tar in the binder course was stripped from the gravel aggregate and the mixture crumbled easily.

The base course is a bank run, well-graded, sand and gravel with occasional cobbles up to 6-in. size. No sorting, blending or addition of

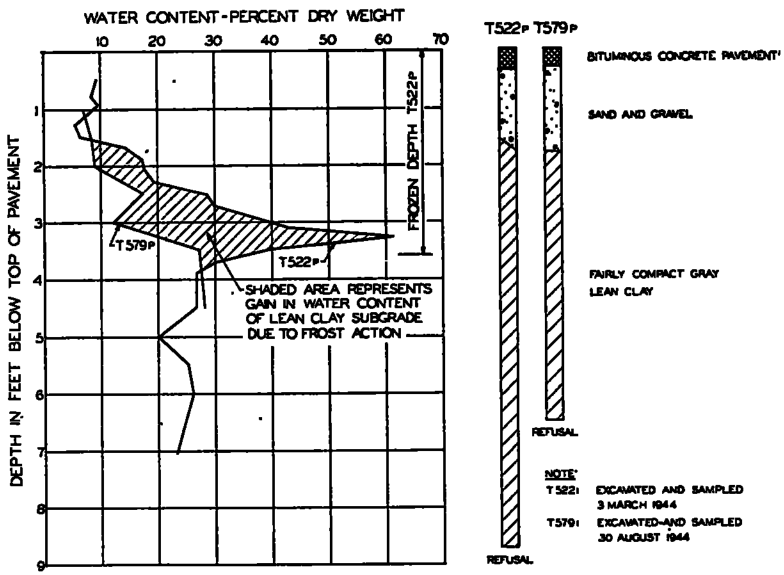


Figure 5. Typical Water Content vs. Depth Profile

It could not be satisfactorily determined whether or not ice lenses were formed in the gravel base. It is considered likely that a few thin lenses may have formed but neither the continuous water content determinations nor the visual observations confirm this belief.

Observations of ground water depth were first made in the spring when free water was encountered in the gravel base. Subsequent observations using wells in the subgrade indicate groundwater is approximately at the top of subgrade in all test areas.

TESTS ON FLEXIBLE PAVEMENTS

Three test areas, designated I, II, and III were located on flexible pavement, as shown on Figure 1. The pavement consisted of a

sizes was made to the run of bank material during construction. When dry, the mixture appears to have a fairly large percentage of silt. The gravel contains a small proportion of soft particles, generally of shale origin, and a larger portion of sizes which are flat or elongated. The gravel readily compacts to a hard, firm surface. The percentage passing the 200-mesh sieve varies from 4 to 13 per cent. Hydrometer tests indicate that 2 to 7 per cent are finer than 0.02 mm. in size. The range of gradation of the gravel base is shown on Figure 3. The tested unit dry weight of the compacted base in place ranged from 131 to 145 lb. per cu. ft., with a water content range of 5 to 10 per cent. Observations during the frost melting period in test pits indicated that free water was present in the gravel base. During the summer free water was encountered in only a few scattered pits.

The thickness of the gravel base in test areas I, II, and III averaged 17.5, 37, and 20 in. respectively and varied as indicated by the sections on Fig. 1.

Test areas I, II, and III were traffic tested using the following two pieces of construction equipment:

(a) 10,000-lb. wheel load equipment consisted of a 5-cu. yd. dump body Sterling truck with 11:00 by 24 tires so that each rear dual wheel unit applied a total load of 10,000 lb. to the pavement.

TABLE 4
SUMMARY OF PAVEMENT HEAVING OBSERVATIONS

Test Area	Predominate Subgrade Soil	Average Pavement and Base Thickness	Average Measured Heave of Pavement Surface	Range of Measured Heave of Pavement Surface
		in.	*in.*	*in.*
I	Lean Clay	21.5	2.5	0.0 to 4.0
II	"	40.7	1.5	0.0 to 3.0
III	"	23.8	2.5	2.0 to 3.5
IV	Silty Till	22.0	6.5	5.5 to 8.5
V	Lean Clay	22.9	5.0	4.0 to 5.5
VI	Silty Till	20.0	5.0	3.5 to 6.0

TABLE 5
COMPARISONS BETWEEN MEASURED TOTAL ICE LENS THICKNESS IN SUBGRADE AND MEASURED HEAVE OF PAVEMENT

Within or Adjoining Test Area	Cumulative Total Ice Lens Thickness at Test Pit	Total Heave of Pavement at Test Pit
	in.	*in.*
II	0.84	0
II	0.87	0.24
III	2.95	3.24
I	3.26	4.80
III	6.10	4.20

(b) 20,000-lb. wheel load equipment consisted of a 12-cu. yd. Gar Wood Scraper equipped with four single wheels each with 18:00 x 24 pneumatic tires which was loaded to produce 20,000 lb. on each rear wheel and 13,000 lb. on each front wheel. This scraper was towed by the Sterling truck.

Each of the equipments have wheels which apply less load to the pavement than the wheels most heavily loaded. Coverages are based upon the wheels most heavily loaded; thus for the 20,000-lb. equipment the combined coverages of rear truck wheels, the front scraper wheels and the rear scraper wheels would be approximately three times the coverages recorded and referred to herein. The effect of the wheels with lesser loads has been considered as minor; however, it is recognized that this assumption may be open to question.

Area I was subdivided into a number of separate elements lettered as shown on Figure 6 and traffic tested using the 20,000-lb. wheel load equipment. Certain elements received approximately 40 coverages each day simulating taxiway traffic while other elements received approximately 4 daily coverages

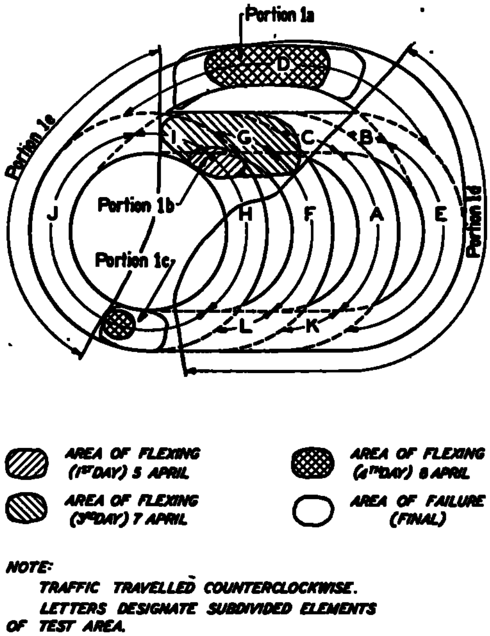


Figure 6. Plan of Test Area I Showing Subdivided Elements and Portions

simulating runway traffic. Traffic was spread uniformly over the full width of each test element. The remaining elements received by necessity an intermediate number of coverages. Traffic was started on April 5 and continued until April 15, 1944. The relation between the traffic test and the frost and temperature conditions is shown on Figure 2. A summary of the test is contained on Table 6 which gives the number of coverages each day on each element. On Figure 6 are outlined portions of the test area which failed or flexed under traffic and the portions which did not flex and were undamaged by the traffic. Each of these portions is given a separate designation. A portion includes several elements, thus the daily coverages of traffic may be different for each element within the portion.

Flexing was observed on the first day of testing and became steadily more pronounced as testing continued, as shown by Figure 6. Toward the end of the test in the portions being failed the flexing increased to 2 to 3 in. For the last several days of traffic it was the opinion of the observers that the equipment would break through the pavement at any moment. It was observed that the pavement did not flex as much at the start of each day's traffic and that the flexing became steadily worse during the day. It is considered possible that if traffic had been continued day and

TABLE 6
SUMMARY OF TRAFFIC TESTING OF AREA I
20,000-lb. WHEEL LOAD

Date April	No. of daily coverages on element											
	A	B	C	D	E	F	G	H	I	J	K	L
5	4		4			10	14	11	25	25	4	14
6	4		4			10	14	4	18	18	4	14
7	4	26	30		26	10	40		40	40	30	40
8				26	26					26	26	26
9				1	1					1	1	1
10				4	4					4	4	4
11				4	4					4	4	4
12				4	4					4	4	4
13				4	4					4	4	4
14				5	5					5	5	5
15				4	4	10	10	36	46	50	4	14
16				4	4	10	10	36	46	50	4	14
17				4	4	10	10	36	46	50	4	14
18				4	4	10	10	36	46	50	4	14
19				4	4	10	10	36	46	50	4	14
20				4	4	10	10	36	46	50	4	14
21						10	10	36	46	46		10
22						10	10	36	46	46		10

night without rest periods that the same damage to the pavement would have occurred at a relatively few total coverages.

Measurements indicate that the permanent deformation of the pavement surface in the traffic lanes except in failed portions ranged between 0 and 0.4 in. In the failed portions, the measured permanent deformations ranged from 1 to 5 in. These measurements are reasonably independent of heaving and subsidence of the pavement surface due to frost action in the subgrade.

The principal variables governing the analysis of Area I are summarized on Table 7 together with data for other test areas. Based upon a study of the data it is concluded that the failure or flexing of the three portions under the 20,000-lb. wheel load resulted primarily from a lesser pavement and base thickness in these areas than existed in the adjoining unfailed two portions. Observations and tests indicate that the degree of frost action and the other physical conditions with the exception of the subgrade soil were sufficiently similar throughout the test area to eliminate other possible causes for the distribution of failed and unfailed portions. The two subgrade soil types are considered in the analysis separately.

Figure 7. Test Area I, Portion 1a After Failure

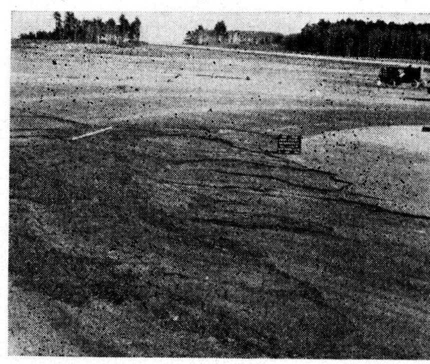


Figure 8. Test Area I, Portion 1c after Failure

In Area II traffic, using the same equipment as on Test Area I, was started on April 5, 1944. Traffic was routed in such a manner that each of four parallel elements as shown on Figure 1 received a different number of coverages each day. For ten consecutive days the traffic equipment travelled the test area making 4, 14, 44, and 34 coverages per day on the four elements respectively. Visual observations during the test together with measurements of permanent deformation of the pavement surface indicated that the pavement did not flex nor was it permanently deformed more than a maximum of 0.25 in. as measured along one cross-section located across the center of the area.

Traffic using the Sterling truck only was started over Area III on April 11. The truck was routed in such a manner that 50 coverages each day, with minor exceptions, were applied to the test area. Three portions located as shown on Figure 1 developed a minor amount of flexing which was noticed on the first day of traffic and continued without noticeable change in magnitude for the duration of the test. Two of these portions developed cracks. Observations of the flexing indicate that its maximum magnitude was approximately 0.3 in. Measurements indicate that permanent deformation in the area ranged between 0.1 and 0.7 in.

On Table 7 is a summary of the conditions affecting the analysis of Test Area III. The data obtained indicate that the physical conditions within the entire test area were quite uniform. Based upon the visual observations of the character of the flexing in the three portions which flexed, it is believed that a slightly greater wheel load than 10,000 lb. would have caused flexing of the entire test area.

The principal variables in the traffic tests on flexible pavement are summarized on Table 7. Particular attention is directed to the differences in pavement and base thickness within two of the three test areas. As shown by this summary, in certain portions of two of the three test areas the pavements either showed signs of distress or were failed by the traffic while in the remaining portions the pavement showed no signs of distress under the traffic imposed. A study of the summary Table 7 indicates the following conclusions as a result of the traffic tests conducted on flexible pavements for the particular conditions tested at Dow Field:

TABLE 7
SUMMARY OF TRAFFIC TESTS ON FLEXIBLE PAVEMENTS

Test Area	Portion	Thickness of Pavement and Base		Predominate Subgrade Soil	Heaving of Pavement		Wheel Load	No. of Daily Coverages		Type of Failure
		Average	Range		Average	Range		Average	Range	
		in.	*in.*		*in.*	*in.*	*lb.*			
I	1a (1)	18	14–22	Lean Clay	2.0	0.6–2.5	20,000	4	4–26	Map cracking, severe flexing and rutting
	1b (1)	18	14–22	Lean Clay	2.0	0.6–2.5	20,000	13	10–40	
								40	18–46	Map cracking, severe flexing and rutting
	1c (1)	21	20–22	Silty Till	3.5	2.5–4.0	20,000	29	4–50	Slight flexing and map cracking
	1d (1)	26	24–29	Lean Clay	2.0	1.0–3.0	20,000	7	4–26	None
	1e (1)	22	19–24	Silty Till	1.0	0–1.5	20,000	5	4–26	
								29	4–50	None
II	2a	33		Lean Clay	2.0	1.5–3.0	20,000	4	4	None
	2b	36		Lean Clay	1.5	1.0–2.5	20,000	14	14	None
	2c	39		Lean Clay	0.5	0–1.5	20,000	34	34	None
	2d	42		Lean Clay	1.0	0.5–2.0	20,000	44	44	None
III	3a, 3b, 3c	23	21–24	Lean Clay	2.5	2.0–3.5	10,000	50	50	Slight flexing and random cracking
	3d, 3e, 3f	23	21–24	Lean Clay	2.5	2.0–3.5	10,000	50	50	None

(1) For relation between portions and lettered elements, see Figure 6. For summary of daily traffic see Table 6.

(a) For traffic of 4 coverages per day, 24 in. of flexible pavement and base on lean clay subgrade, with 2 ft. of subgrade subjected to severe frost action, is adequate pavement and base to withstand a 20,000-lb. wheel load; and 18 in. is inadequate to withstand a 15,000 to 20,000-lb. wheel load during and immediately following the frost melting period.

(b) For traffic of 4 coverages per day, 21 in. of flexible pavement and base on a silty till subgrade, with 2 ft. of subgrade subjected to severe frost action, is adequate to withstand a 15,000 to 20,000-lb. wheel load.

(c) For traffic of 40 coverages per day, 24 in. of flexible pavement and base on lean clay subgrade, with 2 ft. of subgrade subjected to severe frost action, is inadequate to withstand a 7,000 to 10,000-lb. wheel load; and 37 in. of flexible pavement and base, with 1 ft. of subgrade subjected to severe frost action, is adequate to withstand a 20,000-lb. wheel load during and immediately after the frost melting period.

Plate bearing tests were performed upon the bituminous concrete pavement in the areas tested by traffic and at locations adjoining traffic tested areas. The purpose of performing these tests was to provide a measure of the loss of load supporting capacity of the pavement during the frost melting period compared to the summer period. On Figure 2 are shown the periods during which the tests were performed vs. temperature and frost conditions.

The load was applied to the plate by a 30-ton hydraulic jack and measured by a pressure gage. The deflection of the plate was measured by extensometers, with smallest division 0.001 in., mounted upon a 20-ft. I beam. The reaction for the jack consisted of four loaded 10-ton dump trucks each located over an extremity of a 4-way steel truss, the center of which was at the jack and plate assembly.

Two procedures were used for tests on the bituminous concrete pavement. The first procedure termed hereafter "static loading" consisted of applying to a 30-in. diameter bearing plate a 5-lb. per sq. in. seating load for a period of approximately ten minutes, unloading, zeroing the extensometers, then applying load to the plate in approximately five increments and observing the plate deformation until nearly complete deformation under each load was reached. The total load applied to the plate was 60,000 lb. When the final deformation under a load of 60,000 lb. had been reached, the load was removed in about three decrements. Seven static loading tests were performed during the frost melting period and seven tests during the summer. A summary of the static plate bearing tests on flexible pavement is contained on Table 8.

TABLE 8

SUMMARY OF STATIC LOADING PLATE BEARING TESTS ON FLEXIBLE PAVEMENTS, 30-IN. DIAMETER—PLATE

Test No. (1)	Located Adjoining or Within Test Area	Date of Test, 1944	Bituminous Concrete Pavement Thickness	Combined Thickness, Pavement and Base	Gross Load on Plate Which Produced Deformation in Inches of			Deflection in Inches Produced by Gross Load in Pounds of		
					0.05	0.10	0.20	10,000	20,000	40,000
			in.	in.						
20	I	12 April	4.6	21	2,000	3,000	6,000	0.40	0.80	1.41
42	I	28 August	3.7	22	13,000	25,000	45,000	0.03	0.08	0.17
21	I	13 April	4.1	21	8,000	15,000	25,000	0.06	0.15	0.34
39	I	26 August	4.1	21	19,000	30,000	48,000	0.025	0.06	0.15
22	II	15 April	3.4	47	17,000	26,000	50,000	0.04	0.08	0.15
34	II	23 August	3.4	47	20,000	39,000		0.025	0.05	0.105
23	II	17 April	3.4	39	15,000	36,000	65,000 (2)	0.035	0.06	0.12
54	II	3 October	3.4	39	22,500	44,000	60,000	0.015	0.045	0.09
24	II	17 April	3.6	33	12,000	23,000	42,000	0.04	0.085	0.19
44	II	23 September	3.6	33	20,000	40,000	67,000 (2)	0.025	0.05	0.10
26	I	20 April	3.4	22	8,000	16,000	40,000	0.06	0.12	0.20
35	I	24 August	3.4	22	25,000	48,000		0.015	0.04	0.08
32	III	1 May	4.0	26	14,000	27,000	48,000	0.03	0.07	0.16
37	III	25 August	4.0	26	8,000	17,000	39,000	0.06	0.115	0.205

(1) Tests are arranged in pairs with approximately 5 to 20-ft. distance between companion tests.
(2) Value extrapolated

The second method, termed "repeated loading," utilized the same equipment with the exception that a 24-in. diameter bearing plate was used instead of the 30-in. plate. After the seating load of approximately 5 lb. per sq. in. had been applied and released and the extensometers zeroed, a total load of 20,000 lb. was applied and maintained for 10 min., then reduced to zero and the plate permitted to rebound. The loading and unloading cycle was applied ten times. Three repeated loading tests were performed during the frost melting period and three tests were performed during the summer. For the summer tests a 19-in.

diameter plate was inadvertently used in place of the 24-in. plate. A summary of the repeated loading plate bearing tests on flexible pavement is contained on Table 9,

As summarized on Table 8 the static load tests performed upon the flexible pavement indicate the following conclusions for the particular conditions tested:

(a) Tests performed at adjoining locations in summer and during or immediately after the frost melting period had nearly the same load-deflection characteristics where the thickness of flexible pavement and base approached or equaled the frost penetration.

(b) As the thickness of pavement and base became less than the frost penetration, and severe frost action occurred in the subgrade, the load-deflection characteristics became increasingly different. With approximately 2 ft. of flexible pavement and base, about one-half the load was required during the frost melting period to produce the same plate deformation as produced during the summer. Stated in another way, the same load produced greater than twice the plate deformation during the frost melting period as resulted during the summer.

For the repeated load tests, as summarized on Table 9, the fact that a 24-in. diameter plate was used for the tests during the frost melting period and a 19-in. diameter plate during the summer complicates the analyses somewhat. However, it is believed that the effect of the two plate diameters in this instance is minor due to the substantial thickness of pavement and base over the subgrade. Neglecting the influence of plate diameter, the conclusions stated for the static loading tests in general are substantiated by the repeated loading tests. In addition the repeated tests indicate that the increment deformation under each load repetition was much greater during the frost melting period than during the summer.

TABLE 9

SUMMARY OF REPEATED LOADING PLATE BEARING TESTS OF FLEXIBLE PAVEMENTS

Test No. (1)	Located Adjoining or Within Test Area	Date of Test, 1944	Bituminous Concrete Pavement Thickness	Combined Thickness, Pavement and Base	Plate Diameter	Deflection in Inches Produced by 20,000 Pounds Gross Load at Load Repetition No.						
						1	2	3	6	7	9	10
			in.	*in.*	*in.*							
30	I	29 April	3.7	22	24	0.280	0.301	0.315	0.346	0.352	0.368	0.372
41	I	28 August	3.7	22	19	0.098	0.106	0.111	0.119	0.120	0.122	0.122
25	I	18 April	4.1	21	24	0.218	0.247	0.258	0.272	0.277	0.282	0.287
38	I	26 August	4.1	21	19	0.101	0.111	0.117	0.125	0.126	0.128	0.130
31	III	30 April	4.0	26	24	0.089	0.107	0.116	0.125	0.122	0.121	0.123
36 (2)	III	24 August	4.0	26	19	0.092	0.101	0.107	0.118	0.128	0.093	0.094

(1) Tests are arranged in pairs with approximately 5 to 20 ft. between companion tests.
(2) Test erratic.

TESTS ON RIGID PAVEMENTS

Three areas, located on portland cement concrete pavement, numbered IV through VI, Figure 1, were subjected to traffic tests. The deflection of selected corners of slabs located adjoining test areas was determined under one application of a wheel load. Four plate bearing tests were performed upon corners of slabs, two immediately after the frost melting period and two during the late summer. The relation of the periods of testing to the frost and temperature conditions are shown on Figure 2.

The pavement on which these tests were performed consisted of a 7-in. portland cement concrete pavement without mesh reinforcement. Slabs varied from 10 to 12 ft. in width by 20 to 30 ft. between dummy groove joints. Some lanes have 10-in. thickened edges as shown on Section EE, Figure 1. A longitudinal expansion joint is located on the runway centerline and a transverse doweled expansion joint is located through the middle of the area. The longitudinal construction joints are flush without keys.

Forty-eight 6-in. diameter diamond tool drilled cores were removed from the area and tested for compressive strength. Four slabs, each about 5 ft. square, were removed from concrete placed at the same time and using the same mix, one slab from the concrete adjoining the tested areas. From each of these slabs five beams were sawed each 6 by 6 by 30 in. and three 6-in. diameter diamond tool drilled cores were obtained and tested for flexural strength and compressive strength respectively. Analysis of these data indicates that the concrete pavement had an average compressive strength of 6000 lb. per sq. in. and an average flexural strength of 870 lb. per sq. in. at the time the traffic tests were performed.

The base beneath the portland cement concrete pavement consisted of a compact bank run, sand and gravel averaging 15 in. thick and of the same characteristics as that beneath the flexible pavement test areas. Tests to determine the subgrade modulus of the base, performed during the summer, indicate that the subgrade modulus averaged 300 lb. per sq. in. per in. deflection.

Both types of subgrade soil were encountered beneath the base, a lean clay and a silty till. The plan and Section EE on Figure 1 show the approximate line of demarkation between the two subgrade soil types in the three test areas on the portland cement concrete pavement.

For traffic testing, the equipment consisted of that employed on Areas I and II and in addition a Model A Turnapull equipped with an RU Scraper, capacity 23 cu. yd. The tractor was mounted on two 30.00 by 40 pneumatic tires and the scraper on 24.00 by 32 tires. The load was balanced so that each of the four wheels applied 40,000 or 30,000 lb. load to the pavement as desired.

In Area IV traffic using the 20,000-lb. wheel load equipment was started on April 25. The equipment moved across this area going in an easterly direction each time and turning around at random on the concrete pavement avoiding turning on areas being tested. Longitudinal pavement lane lettered, "I" on Figure 1, was given four coverages per day for 10 days and five on the eleventh day. Longitudinal lanes lettered "J" and "K" and part of "L" were given as many coverages per day as possible for 11 days, averaging 25 per day.

On the first day after one coverage water was pumped from the gravel base through the longitudinal joints between lanes J and K and between K and L. By the fourth day two slabs within lanes J, K and L had been cracked once, and vertical movements of all slabs were visible. At the end of the test on May 5 a total of five slabs within lanes J, K, and L had one or more cracks. Water was pumped continually through the longitudinal joints noted throughout the test. No slabs in lane I were cracked.

The deflection of four slab corners located adjoining the test area at the intersection of the transverse expansion joint and a longitudinal construction joint was measured. With the 20,000-lb. wheel load at the corner, the deflection ranged between 0.063 and 0.184 in. and averaged 0.12 in.

In Area V traffic using the 40,000-lb. wheel load equipment was started on April 27 and stopped on the same day. The equipment first made two coverages over the test area. These two coverages resulted in cracking of several slabs. The test area was then narrowed to two longitudinal construction lanes, lettered "B" and "C" on Figure 1, to minimize the area of pavement damaged. A total of 37 coverages was applied to a strip 2.3 ft. wide located on the north edge of lane B and a total of 18 coverages was applied to the remaining portion of lane B and three quarters of lane C. The test was then discontinued because of the large number of cracks which had developed in the pavement. Figure 9 illustrates the condition of the pavement after the test was stopped.

Measurements were made using the 40,000-lb. wheel load of the deflection of the corner of one slab. These tests indicate that the deflection of slab corners at the intersection of a longitudinal construction joint (no key) with a transverse doweled expansion joint ranged between 0.110 and 0.137 and averaged 0.12 in. without cracking under one application of the loaded wheel.

In Area VI traffic using the 30,000-lb. wheel load equipment was started on April 30, 1944 and continued for 5 days. As many coverages as possible per day were made on this area with an average per day of 13 on longitudinal lanes "E" and "G" and 26 lane "F." Traffic was stopped on May 4 after it became evident that continued traffic would result in the failure of a substantial portion of the test area. Figure 10 illustrates the pavement condition after traffic was stopped.

Measurements of the deflections of the corners of two slabs located adjoining the test area were made using the 30,000-lb. wheel load. These tests indicate that the deflection of slab corners at the intersection of a longitudinal construction joint (no key) with a transverse doweled expansion joint ranged between 0.085 and 0.185 in. and averaged 0.11 in. without cracking under one application of the loaded wheel.

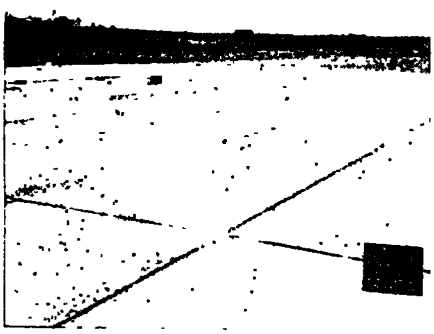


Figure 9. Cracked Slabs in Test Area V After Traffic Stopped

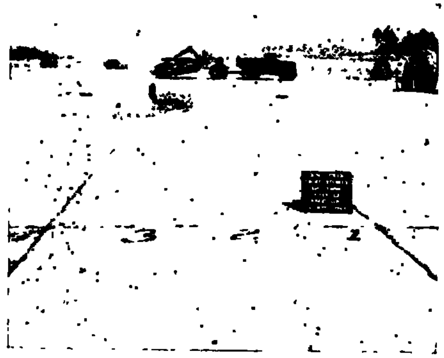


Figure 10. Cracked Slabs in Test Area VI After Traffic Stopped

The three traffic tests on the portland cement concrete were performed immediately following the frost melting period as shown by Figure 2. It is believed that the pavement damage would have been greater in each of the three test areas if it had been possible to start traffic before the frost started melting. Thus, the results of these tests do not indicate the minimum condition of pavement load supporting capacity during the frost melting period.

The traffic tests performed upon the portland cement concrete pavement indicate the following conclusions for the conditions tested:

(a) For 4 coverages per day, 7-in. uniform thickness of portland cement concrete without longitudinal joint keys on 15 in. of base on either lean clay or silty till subjected to severe frost action is just sufficient pavement to withstand a 20,000-lb. wheel load during the period immediately following the frost melting period.

(b) For 40 coverages per day, 7-in. uniform thickness portland cement concrete without longitudinal joint keys on 15 in. of base on either lean clay or silty till subgrade subjected to severe frost action is insufficient to withstand a 20,000-lb. wheel load during the period immediately following the frost melting period.

The deflection measurements performed using the loaded wheels of the traffic equipment indicate that deflections of corners of 7-in. uniform thickness portland cement concrete slabs averaged approximately 0.12 in. regardless of wheel load within the range of 20,000 to 40,000 lb. without cracking the corner.

Two plate bearing tests were performed during the frost melting period on the portland cement concrete pavement to determine the load required to fracture the corner of the slab at the intersection of a transverse expansion joint and a longitudinal construction joint. The 24-in. bearing plate was placed as near to the corner of the slab as possible and the three extensometers were placed, one at the point nearest the corner and the other two at 120 deg. either side. The method of loading the plate was the same as adopted for the static loading procedure described under tests on flexible pavements. For the first test the selected increments were too large to determine the exact load at which the corner ruptured. Approximately 10 sec. after the load was increased from 40,000 to 60,000 lb. a crack developed across the corner of the slab being loaded at an average deflection of approximately 0.25 in. The load was then carried up to 89,000 lb., the capacity of the set-up, and the slab adjacent (across the transverse expansion joint) cracked across the corner. The second test on the cement concrete during the frost melting period was performed similar to the first with the exception that the increments selected were smaller. The first fracture was observed in the slab adjacent to the one being loaded (across the transverse expansion joint) at a load of 48,000 lb. It is considered possible that this crack may have developed at the previous increment of 40,000 lb. Under the 48,000-lb. load, and a few minutes later, the slab being loaded started to crack at an average deflection of 0.2 in. The load was increased in increments to 72,000 lb. the capacity of the set up and the crack in the slab being loaded did not extend more than about two-thirds of the way across the corner.

Two plate bearing tests were performed during the summer on the cement concrete pavement adjacent to those tests performed during the frost melting period. These tests were performed similarly with the exception that a 19-in. diameter plate was inadvertently used. The load on the plate required to crack the corner of the tested slab was 75,000 lb. for the first test and 80,000 lb. for the second test performed during the summer. Cracks developed at average deflections of approximately 0.2 in.

It is considered that the effect of the difference in plate diameters used for the two sets of tests is minor due to the distributing qualities of the portland cement concrete pavement and base. All tests were performed during the middle of the day and slab corners are considered to be warped down.

The plate bearing tests on the slab corners of 7-in. uniform thickness portland cement concrete pavement on 16 in. of base for the conditions tested at Dow Field indicate the following conclusions:

(a) A total load of between 48,000 and 60,000 lb. was required during the frost melting period on a 24-in. diameter plate to break the slab corner at a longitudinal construction joint without key and a transverse expansion joint with dowels.

(b) A total load of between 75,000 and 80,000 lb. was required during the summer on a 19-in. diameter plate to break the slab corner at a longitudinal construction without key and a transverse expansion joint with dowels.

(c) Slab corners were cracked at a deflection of approximately 0.2 in. regardless of plate diameter or testing period.

GENERAL CONCLUSIONS

The subgrade soils at Dow Field during the winter of 1943–44 experienced severe frost action which resulted in heaving of pavement surface during the freezing period and a loss in shearing strength of the subgrade soil during and immediately after the frost melting period.

The results of the companion plate bearing tests performed upon the pavement surface of both rigid and flexible type pavements in the spring and in the summer furnish a qualitative indication of the loss in shearing strength of the subgrade soil. Likewise, California Bearing Ratio tests in place upon the subgrade immediately after the frost melting period and during the summer indicate qualitatively the reduction in shearing strength.

A comparison between the results of the traffic tests on both flexible and rigid pavements with the design criteria now used by the Engineer Department indicates that the design criteria, applied to Dow Field, is not sufficiently conservative. Additional tests now in progress are expected to indicate the effect of less severe frost action and greater pavement and base thickness upon pavement load supporting capacity.