Guidelines for Spring Highway Use Restrictions

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16. Abstract

This report desribes a survey of current practice as well as analysis performed to develop guidelines for agencies which need load restrictions during spring thaw periods. The results show for a range of locations in the U.S and Canada that the average load restriction applied is about 44 percent. The analysis performed in the study tends to confirm that level of load restriction. Further, it was found for the assumed conditions that a minimum load restriction level (if any load reduction is needed) is about 20 percent. Load restrictions greater than 60 percent are generally not warranted for the range of cases studied. An air temperature based criterion (Thawing Index) was developed which can be used to estimate when to apply and remove load restrictions.

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GUIDELINES FOR SPRING HIGHWAY USE RESTRICTIONS

by

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Prepared by the

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and the
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for the

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and in Cooperation with

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CHAPTER 1.0

INTRODUCTION

1.1 THE PROBLEM

In areas of the United States which are subject to moderate or severe seasonal freezing, pavement structures can be susceptible to weakening during the thawing period (normally during the spring but this can occur several times during the winter months). To preclude accelerated pavement deterioration two possibilities exist:

(a) Apply load restrictions during the thawing (or critical) period.

(b) Design, construct, or otherwise modify the pavement structure to prevent or reduce the thaw weakening phenomenon.

Due to budget constraints for many agencies faced with this problem, the only choice is Item (a) above.

A review of the literature quickly reveals that few rational procedures have been used to determine the magnitude of the load restrictions, when to apply them and when to remove them. Therefore a need exists to develop guidelines oriented toward local agencies to assist them in handling this serious problem.

1.2 BACKGROUND

Frost action in soils can cause several detrimental effects. The effect commonly addressed is that of frost heave. Less information is available on an equally serious problem, that of loss in structural capacity. This loss in strength occurs during the thaw period (usually late winter or early spring) when the moisture content increases in the pavement layers. This action is similar to the one due to the rise of the ground water table or infiltration of moisture through a porous pavement surfacing or shoulder. Whatever the cause, the presence of moisture levels in the subgrade above the amount assumed for pavement design will reduce the strength (or stiffness) of the various pavement layers. The same is true for most base and subbase materials.

The majority of currently used design methods is based on empirical studies of pavement behavior. The strength of the subgrade is usually estimated at the equilibrium conditions of moisture and density after soaking for several days (e.g., the CBR test). Empirical design methods based on the above classification procedures cannot account for adverse subgrade conditions caused by the thaw period or unusually high water tables, unless such conditions were generally prevalent when the original empirical studies, on which the methods are based, were conducted. This is because the methods are based on the average subgrade conditions exhibited by the subgrade throughout most of the pavement's life.

The damage to a pavement structure is directly related to the magnitude and frequency of the load applied. This was clearly demonstrated by the AASHO Road Test [1.1]. Subsequent studies of material behavior have demonstrated that the fatigue and permanent deformation characteristics of many materials depend on the magnitude and frequency of stress and strain levels induced [1.2]. A majority of the state DOT's use the AASHTO Interim Guide for Design of Pavement Structures [1.3] for designing their pavement thicknesses (or at least a portion of the AASHTO Guide). In designing a specific pavement using this method the traffic is converted to equivalent 18,000 lb. loads for a given design period and for known or assumed material properties. Any lowering of material strength or increase in the number of equivalent 18,000 lb. loads reduces the life of the pavement. Thus, the method of reducing loads when the strength of the pavement materials is reduced is a reasonable way to maintain the design life and general serviceability of the pavement. Hence, the need for load restrictions during critical pavement periods.

Local and state highway agencies have a wide variety of practices for imposing weight restrictions in advance of the "spring thaw." Truck weight enforcement programs adopted by the various agencies vary widely in terms of the weight limits applied, the forms the restrictions take and their implementation. The decision of closing or opening a facility is largely determined by experience and sometimes political pressures. There is very little definitive data to help in decision making, especially for secondary and lower category highways even though these types of highways form the bulk of county and city highway systems. The local governments generally have low to modest maintenance budgets and normally cannot afford to overlay the pavements after damage during the spring thaw. Therefore, a need exists to develop criteria for the restriction of truck weights during the spring thaw.

1.3 OBJECTIVES

The objective of the reported study was to develop guidelines for local governments to use in establishing weight restrictions on county and city pavements in advance of spring break-up. To achieve this objective the following was accomplished by the study team:

(a) conducted a literature search and summarized the findings,
(b) established contacts with various highway agencies and conducted in-person interviews,
(c) used the available data from the literature and interviews and analyzed them in order to develop load restriction magnitudes and timing,
(d) developed guidelines which can be used by local agencies to assess the need, magnitude, and time to apply and remove load restrictions, and
(e) developed a summary report and videotape presentation to be used for implementation of the study findings.

1.4 REPORT ORGANIZATION

The report is organized into six chapters and seven appendices. The six chapters are the following:

(a) Chapter 1.0 - Introduction
(b) Chapter 2.0 - Literature Review
(c) Chapter 3.0 - Survey of Current Practice
(d) Chapter 4.0 - Analysis
(e) Chapter 5.0 - Development of Guidelines
(f) Chapter 6.0 - Conclusions and Recommendations

CHAPTER 2.0

LITERATURE REVIEW

2.1 INTRODUCTION

In areas where the ground is subject to freezing and thawing, flexible pavements often experience extreme variations in bearing capacity. During the spring, periods of "thaw weakening" occur, greatly reducing the bearing capacity. Where pavements have not been adequately designed to substantially reduce or eliminate the loss of strength occurring during thaw, considerable damage may occur resulting in high maintenance costs. Many areas in the United States, Canada and Europe have experienced these problems and have resorted to imposing some form of load restrictions on particular classes of roads in critical locations to minimize the damaging effects.

This literature review deals with several subject areas related to the use of load restrictions. Among these are current practices regarding load restrictions in the United States, Canada and Europe. In addition, studies related to pavement response during spring thawing are reviewed, including methods for evaluating and predicting the pavement response. Since the spring bearing capacity reductions which occur are due to climatological effects, a review of the literature pertaining to the relationship of spring thaw weakening and climate is also included.

2.2 LOAD RESTRICTION PRACTICES

2.2.1 CURRENT U.S. AND CANADIAN PRACTICES

The NCHRP Report No. 26 [2.1] contains a summary of the states and Canadian provinces which, at that time, applied load restrictions on some classes of roads during spring thawing. The eighteen states and provinces which reported using load restrictions are listed in Table 2.1. In addition, Quimont [2.2] reported that load restrictions are used extensively in Quebec due to the severity of the freezing season.

Table 2.1. States and Provinces Applying Load Restrictions as of 1974 (NCHRP Report No. 26).

State or Province	Comments Regarding Use of Restrictions
Alaska	Older underdesigned roads
Alberta	Selected roads
British Columbia	Limit spring deflection to <.05 mm
Idaho	Experience dictates
Illinois	Local agencies restrict some secondary roads
Maine	Inadequate roads >20 years old
Michigan	Older roads
Minnesota	
Montana	
Nebraska	Only on incompleted stage constructed roads
New Hampshire	Feeder roads
North Dakota	Limited to classes of roads other than interstates and primary highways
Nova Scotia	Secondary roads, 75%± normal loads
Ontario	Weaker roads
Quebec	
Utah	
Wisconsin	Older inadequate roads
Wyoming	Occasionally

2.2.2 EUROPEAN PRACTICES

Several countries in Western Europe are in climatic zones where cyclic freeze-thawing occurs. At the 1974 Symposium on Frost Action in Roads, Finland [2.3] and France [2.4] reported the results of studies showing variations in load carrying capacity with season. France reported imposing load restrictions and reduced speed limits on certain classes of roads. In 1978 France implemented a program outlining procedures for imposing spring use restrictions [2.5]. Temperature, weather trend data and frost depth measurements are taken during freezing and thawing periods. In addition, deflection measurements are taken during thawing and compared to reference values. This is done on representative road sections in various locations and restrictions are imposed based on the data obtained.

Norway reported imposing load restrictions when thawing depths reach 4 to 8 in. [2.6]. The amount of the reduction is based on deflection measurements collected over several years throughout the country. The typical reduction is 20 percent of the maximum allowable load. The duration of the restriction is based on the total and "critical" frost depth, as shown in Table 2.2. Typical load restriction durations by geographic location are shown in Figure 2.1.

Several other Western European countries experience frost related problems including Sweden, Switzerland and West Germany. Kubler [2.7] reported that load restrictions were used in West Germany starting in 1954. While all of these countries report using various frost susceptibility measures in designing their roads, information was not found related specifically to the use of load restrictions.

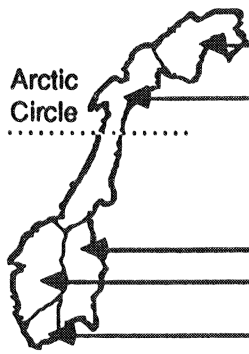
2.3 STUDIES OF SPRING BEARING CAPACITY

2.3.1 EARLY U.S. STUDIES

Most authors point to the pioneering work of Taber [2.8], which identified frost heaving phenomenon and related thaw weakening, as the first step of understanding the reduced bearing capacities of pavements in spring. The first formal investigation in the U.S. of thaw weakening was undertaken by a

Table 2.2 Time for Applying Load Restrictions Based on Thaw Depth, Norway (after Thomassen, 1982)

Total Frost Depth (ft.)	Critical Thaw Depth (ft.)	Time from critical thaw depth is reached Until load restriction can be lifted (weeks)	
		Spring Axle Load/Summer Axle Load	
		≃0.8	≃0.6
≥ 4.9	4.1	1.0 - 2.0	2.0 - 3.0
3.6 - 4.9	3.2	0.5 - 1.5	1.5 - 2.5
2.6 - 3.6	2.4	0.5 - 1.5	1.0 - 2.0
1.6 - 2.6	1.6	0 - 1.0	0.5 - 1.5
0.8 - 1.6	0.8	0 - 1.0	0 - 1.0



Region	Percentage of national roads with restrictions	Imposing Normal period	Lifting Normal Period	Normal Duration (weeks)
	17%	Apr 18	June 28	10
	73%	Apr 1 - Apr 6	May 18 - June 1	7
	54%	Mar 31 - Apr 9	May 21 - June 9	8
	36%	Mar 17 - Mar 28	Apr 21 - May 11	5
	11%	Apr 1 - Apr 3	May 25	8
Whole Country	51%	Mar 17 - Apr 18	Apr 21 - Apr 28	7 - 8

Figure 2.1. Typical Load Restriction Practices in Norway based on Geographic Location (after Thomassen, 1982).

committee formed at the 1948 Meeting of the Highway Research Board [2.9]. Regional and national maintenance engineers practicing in areas subject to cyclic freeze-thaw had been aware for years of the detrimental effects of heavy loads on roads during the spring and, as a result, prior to that time, load restrictions had been in use. However, the degree of thaw weakening had not been estimated quantitatively.

In 1947, field investigators in Minnesota using plate bearing tests showed a loss of strength of up to 60 percent during thawing. Typically the losses occurred nearly simultaneously with the beginning of thawing (Figure 2.2). Base and subgrade materials alike exhibited a loss of strength during thawing based on plate test results (Figure 2.3). Based on this information, nine states agreed to participate in an extensive field study of thaw weakening. These states included Indiana, Iowa, Michigan, New Hampshire, New York, North Dakota, Ohio, Oregon and Minnesota. Nebraska subsequently submitted data over the study period. Test sites were typically located in areas where load restrictions were currently in use with satisfactory results, i.e., little pavement deterioration occurred during thawing. Material profiles were identified at the test locations and samples of materials were examined in the laboratory to identify the dry density and moisture content of the bases and subgrades. In addition, air temperature, precipitation and ground temperature were measured in the vicinity of the test locations. Plate tests, performed at various times during spring thawing and throughout the year, were used to measure deflections. In some states, other deflection testing techniques were used including the North Dakota Cone Bearing Test, the Housel Penetrometer Test, and the Subgrade Resistance Test.

Results from the participating states were published throughout the study period [2.10, 2.11, 2.12, 2.13, 2.14]. Indiana reported that plate tests showed spring bearing values that were 52 percent to 95 percent of the previous fall values, with moisture contents in spring generally higher than those in fall. In addition a tabulation of the results by soil type was also presented [2.12]. Data from Oregon showed a definite trend in reduction of bearing capacity in the spring, although results showed a wide variation

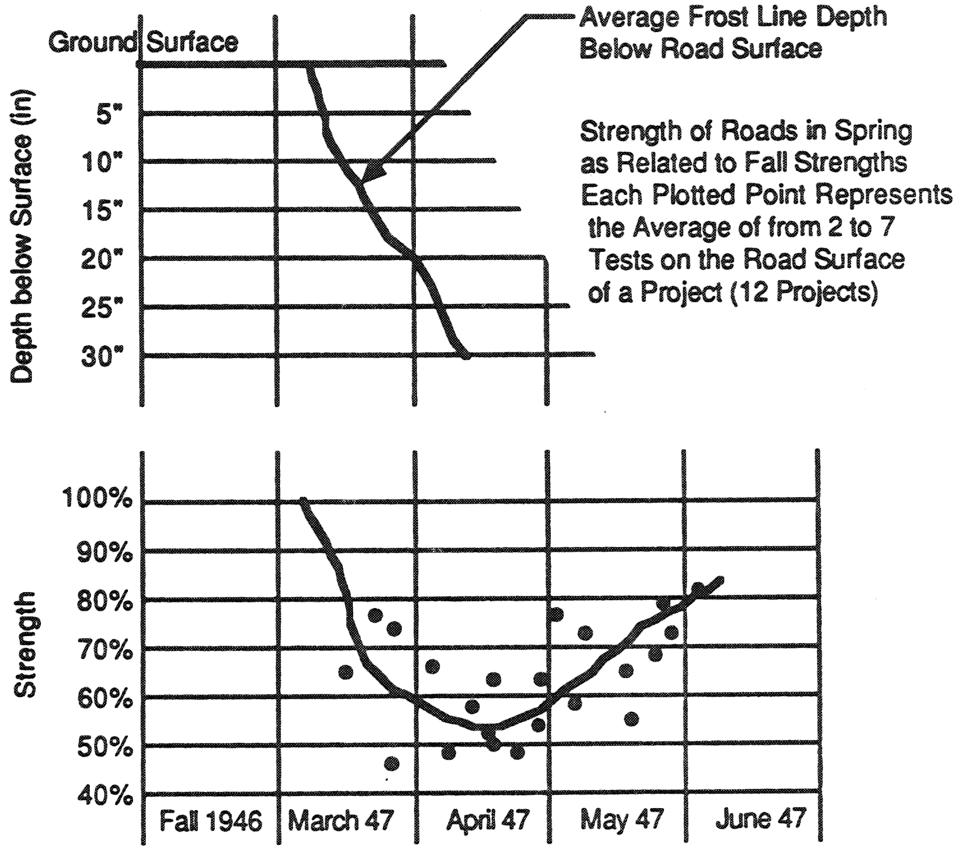


Figure 2.2. Percent Loss of Strength versus Time for Minnesota Plate Load Tests (after Motl, 1948).

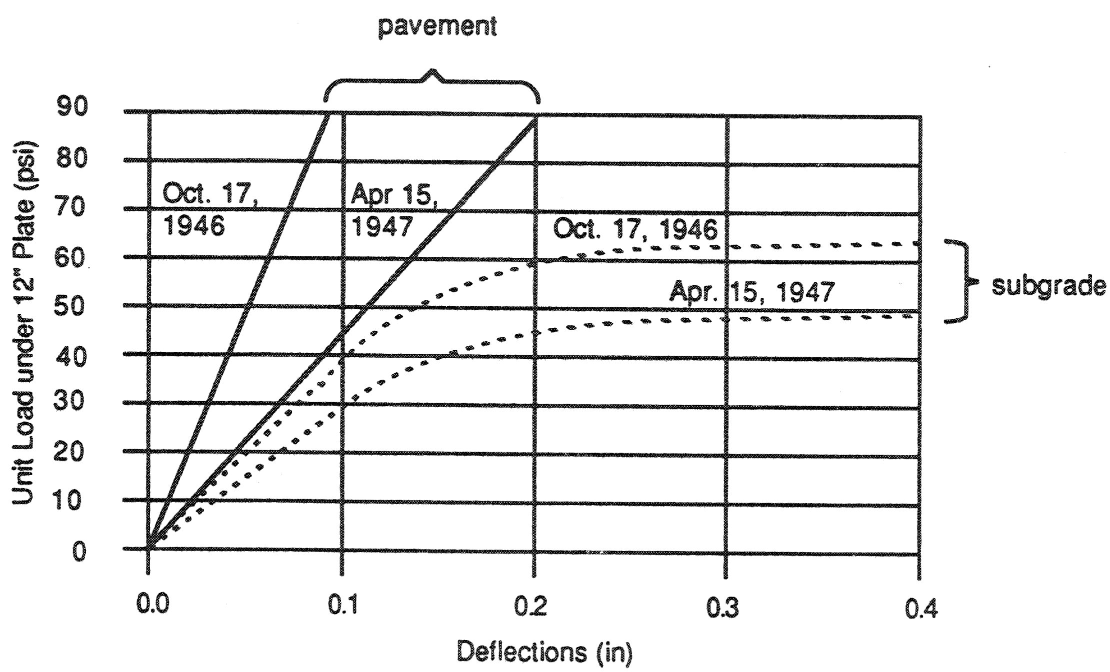


Figure 2.3. Load versus Deflection for Surface and Subgrade of Minnesota Pavement (after Motl, 1948)

[2.12, 2.13]. In general, the test period was quite mild with low frost penetrations, often less than one foot. The data therefore were inconclusive. Nebraska contributed data from approximately 160 sites using plate load tests performed in 1952-53 [2.12]. Strength losses in spring varied from 0 to 65 percent with an average value of 29 percent. A comparison of the loss and recovery of strength for major soil groups is shown in Figure 2.4. Tests were performed in North Dakota from 1948 to 1951 to estimate bearing values using the North Dakota Cone Device [2.11]. Average subgrade bearing values for all tests sites were also estimated for each year and were plotted against time. The results showed that the subgrade bearing value was reduced by 43, 55 and 25 percent (relative to fall values) for the years of 1949, 1950 and 1951. Plate bearing tests were performed in studies conducted in Iowa. The plates were located at the surface, top of the base course and top of the subgrade. Overall, spring bearing losses varied from 16 to 62 percent of the corresponding fall value.

Studies continued in Minnesota in 1948 and 1949 using plate tests. The results of 126 tests were recorded. The spring strength reduction ranged from 15 to 84 percent of the fall value with an average of 42 percent. Average strength values for all tests are plotted for the spring against time and shown with the comparable thawing depth in Figure 2.5.

In addition, correlations between moisture content and bearing and/or various meteorologic factors were considered in several of the studies. However, no conclusive findings were forthcoming.

2.3.2 EARLY BENKELMAN BEAM STUDIES

Preus and Tomes [2.15] performed early work using the Benkelman Beam for detecting seasonal changes in load carrying capacity. The approach taken was to use the Benkelman Beam to obtain a deflection profile by moving the wheel relative to the placement of the probe. Data was obtained on road sections in Minnesota using this technique. Maximum deflection, initial rate of deflection and flection were obtained (Figure 2.6). The results were plotted against bearing capacity estimates obtained from plate bearing tests and suggested that the critical parameter was flection when compared to autumn

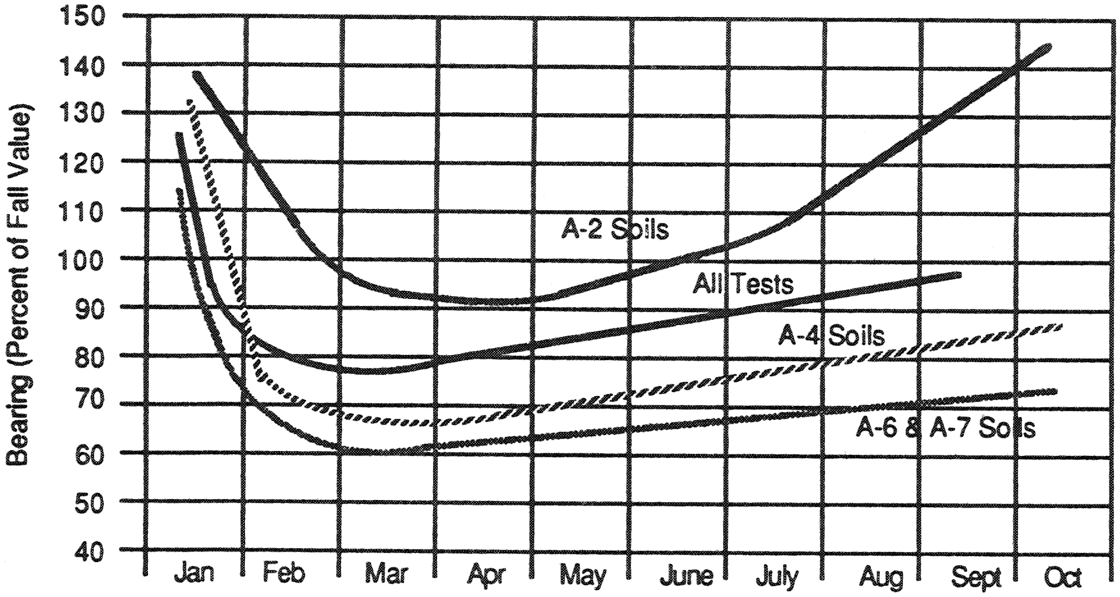


Figure 2.4. Percent Fall Bearing Value versus Time for Nebraska Soils (after Motl, 1955)

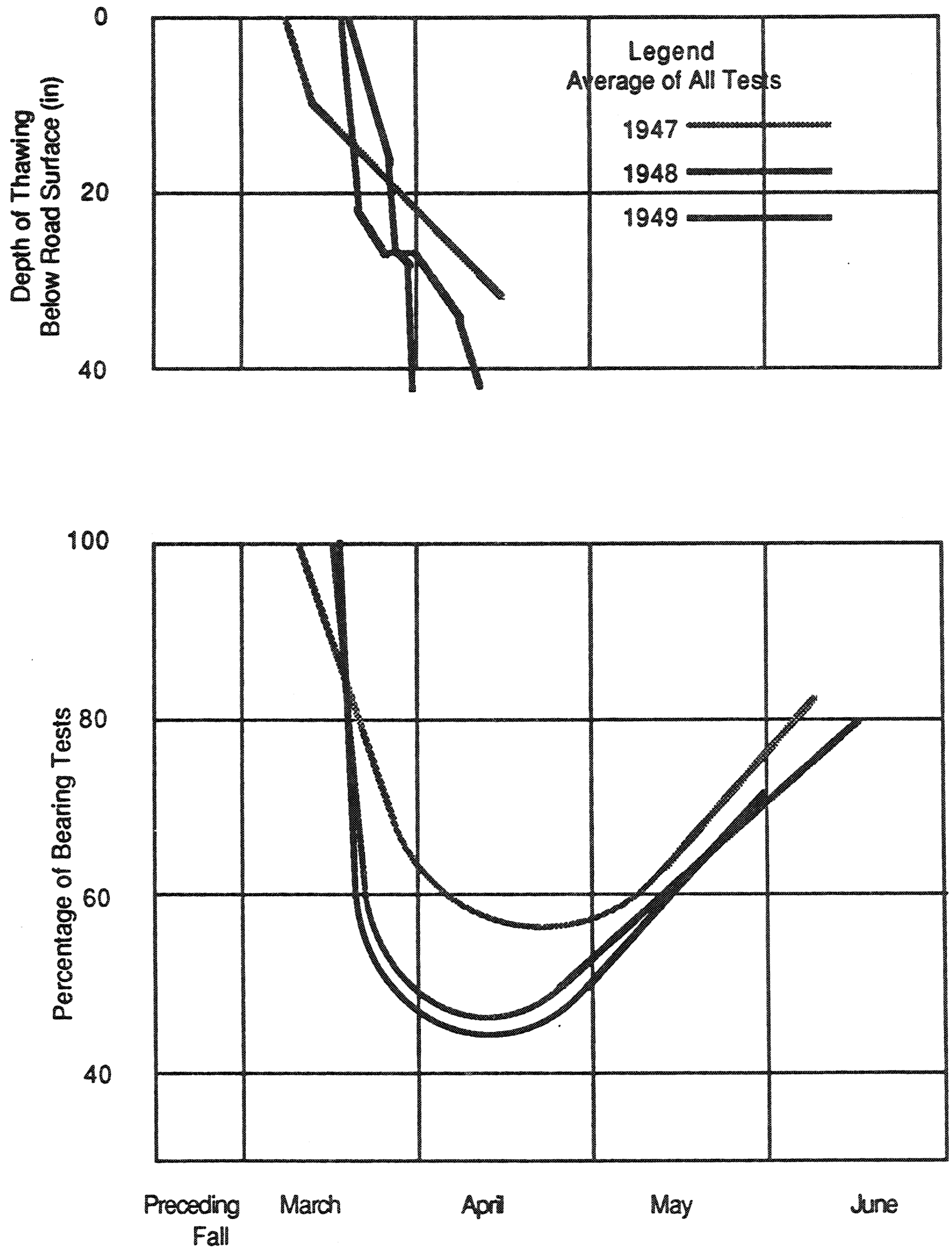


Figure 2.5. Percent Fall Bearing Value versus Time for Minnesota Soils (after Motl, 1951)

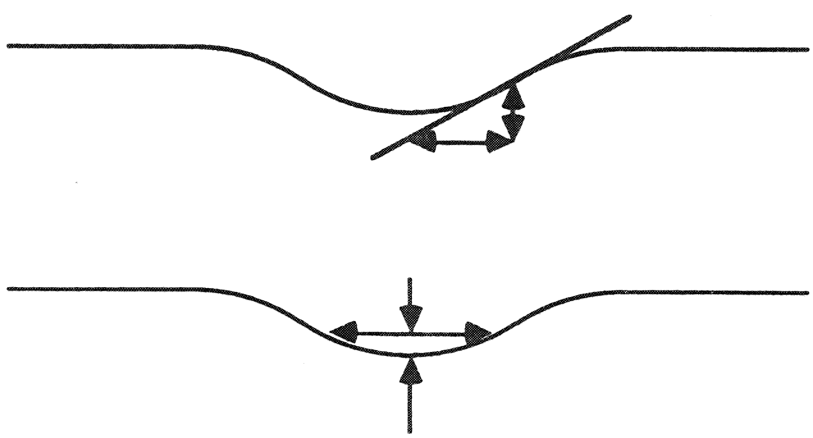


Figure 2.6. Measurements Obtained from Deflection Profiles (after Preus and Tomes, 1959)

reference values. Estimates of strength loss by plate bearing measurements, deflection measurements and rate of deflection showed reasonable agreement.

Armstrong and Csathy [2.16] suggested that older, flexible pavements in Canada are generally susceptible to damage as a result of thaw weakening. Benkelman Beam deflection data recorded throughout Canada suggested that spring load-carrying capacity was reduced by 40 percent in Alberta, 50 percent in Ontario and 30 to 60 percent in New Brunswick.

2.3.3 EARLY DYNAFLECT STUDIES

Early use of the Dynaflect to evaluate seasonal changes in the load carrying capacity of flexible pavements was performed by Scrivner et al. [2.12]. The measurements obtained and the typical deflection basin are shown in Figures 2.7 and 2.8. Using the measured deflections, a surface curvature index, SCI, can be obtained where:

$$SCI = w_1 - w_2$$

and

$$\frac{d^2w}{dx^2} = \frac{SCI}{500a^2}$$

where:

a = distance between w_1 and w_2

For all analysis in this study, "a" was assumed to be 12 inches.

Dynaflect measurements were taken on an average of once a week during spring thawing at 24 test sites located in Illinois and Minnesota. A comparison of the critical period, as defined by this study, and the actual restricted period is shown in Table 2.3. In general, the restricted period was conservative compared to the critical period obtained from deflection and SCI measurements. The maximum SCI and deflection measurements are shown in Table 2.4. It was felt that, based on this information, SCI was a somewhat better indicator for imposing load restrictions. Based on the wide range of temperature conditions at test sections in this study, the authors felt that the use of deflection and/or SCI measurements were most appropriate when the following conditions were met:

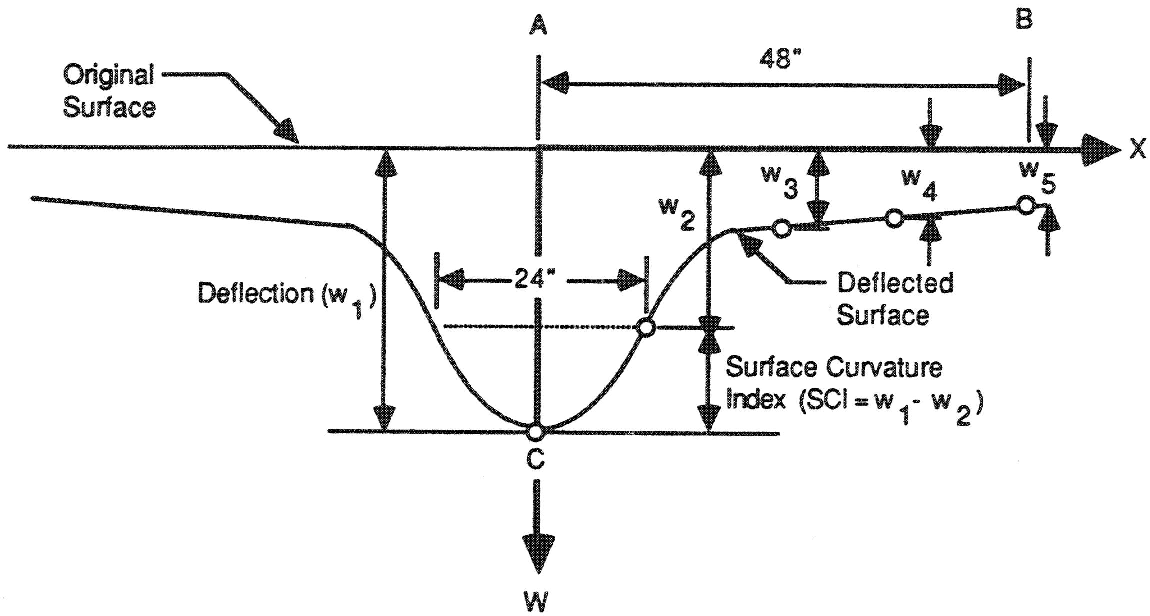


Figure 2.7. Typical Deflection Basin Constructed from Dynaflect Readings (after Scrivner et al., 1969)

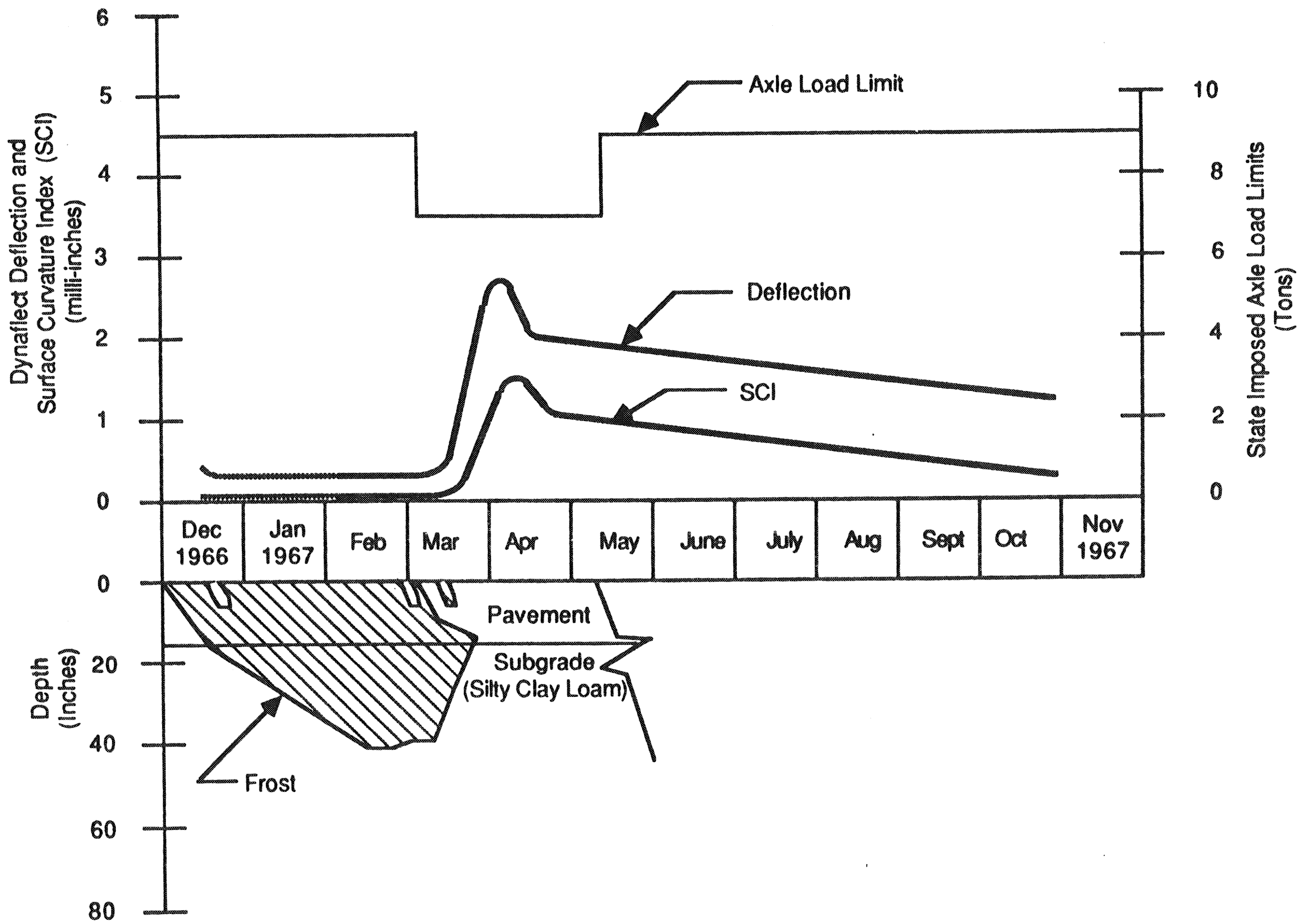


Figure 2.8. Typical Deflection, Surface Curvature, Frost Penetration and Axle Load Restriction Data versus Time

Table 2.3 Summary of Critical and Restricted Periods - 1967 (after Scrivner et al., 1969)

Location	Critical Period						Restriction Period					
	Begin Period of Rapid Strength Loss		End Period of Rapid Strength Recovery		Duration (days)		Restrictions Imposed		Restrictions Removed		Duration (days)	
	Ave. Date	Std. Dev. (days)	Ave. Date	Std. Dev. (days)	Avg.	Std. Dev.	Avg. Date	Std. Dev. (days)	Avg. Date	Std. Dev. (days)	Avg.	Std. Dev.
Northern Illinois	Mar. 3	2.3	Apr. 18	3.3	45.2	5.3	Feb. 14	0	Apr. 19	0	64	0
South-eastern Minnesota	Mar. 13	2.2	Apr. 23	3.3	40.5	2.9	Mar. 7	0	May 10	0	64	0
Eastcentral Minnesota	Mar. 19	1.0	May 1	3.0	43.0	3.1	Mar. 15	1.0	May 15	3.5	62	3.3

Table 2.4 Normal Deflections and Surface Curvature Index by Section (after Scrivner et al., 1969)

Ordered by Normal SCI

Section Location	SCI, $W_1 - W_2$			Restriction Imposed
	Norm.	Min.	Max.	
Eastcentral Minn	.09	.01	.09	No
Southeastern Minn	.20	.00	.32	Yes
Southeastern Minn	.28	.00	.51	No
Southeastern Minn	.28	.01	.55	Yes
Central Ill	.28	.09	.50	a
Eastcentral Minn	.29	.00	.45	No
Northern Ill	.31	.03	.35	No
Central Ill	.38	.02	.50	a
Southeastern Minn	.38	.00	1.22	Yes
Southeastern Minn	.45	.00	.76	Yes
Central Ill	.57	.06	1.15	a
Eastcentral Minn	.57	.01	.75	Yes
Eastcentral Minn	.59	.00	1.53	Yes
Southeastern Minn	.61	.00	1.27	Yes
Eastcentral Minn	.65	.00	.95	Yes
Eastcentral Minn	.78	.00	1.44	Yes
Central Ill	.79	.11	1.98	a
Northern Ill	.81	.02	1.38	Yes
Central Ill	.82	.07	1.78	a
Northern Ill	.82	.03	1.73	No
Northern Ill	.92	.02	1.80	Yes
Northern Ill	.94	.02	2.00	Yes
Central Ill	1.09	.11	2.27	a
Northern Ill	1.09	.03	2.26	Yes

Ordered by Normal Deflection

Section Location	Deflection, W_1			Restriction Imposed
	Norm.	Min.	Max.	
Eastcentral Minn	.60	.10	.62	No
Southeastern Minn	.81	.08	1.15	Yes
Northern Ill	.90	.22	.97	No
Eastcentral Minn	.90	.05	1.32	No
Southeastern Minn	1.07	.14	1.62	No
Southeastern Minn	1.14	.12	1.90	Yes
Central Ill	1.21	.74	1.72	a
Southeastern Minn	1.26	.12	1.80	Yes
Southeastern Minn	1.41	.13	3.05	Yes
Eastcentral Minn	1.64	.06	5.60	Yes
Central Ill	1.71	.67	2.70	a
Southeastern Minn	1.85	.23	3.12	Yes
Central Ill	1.86	.53	2.30	a
Eastcentral Minn	1.96	.09	2.87	Yes
Northern Ill	1.96	.33	3.30	Yes
Eastcentral Minn	2.19	.09	3.25	Yes
Northern Ill	2.21	.25	4.05	Yes
Central Ill	2.24	.97	4.10	a
Central Ill	2.34	1.02	4.16	a
Northern Ill	2.42	.56	4.20	No
Eastcentral Minn	2.49	.18	3.10	Yes
Northern Ill	2.52	.33	4.42	Yes
Central Ill	2.76	.81	4.60	a
Northern Ill	3.14	.43	5.20	Yes

[a] Located in the Springfield area where a restriction policy is not used.

(a) a single distinct freezing period existed, and

(b) the freezing index was greater than 200°F days.

The recommended equation for estimating the "safe" spring load, based on a normal SCI of 0.35 for an axle load of 18,000 lb is the following:

$$L_{safe}\ (\text{kips}) = \frac{6.3}{SCI_{max}}$$

Where the normal (summer) SCI is less than 0.35, the pavement should not require any load restriction.

In addition, Benkelman Beam, Curvature Meter and Plate Bearing measurements were obtained at different times throughout the year. The correlation of Dynaflect deflection and measurements from the Benkelman Beam and plate bearing test are shown in Figures 2.9 and 2.10.

2.3.4 FATIGUE BASED ANALYSIS OF THAW WEAKENING

Hardcastle and Lottman [2.18, 2.19] proposed an analytical method for obtaining spring load limits based on the cumulative damage ratio:

$$D = \sum_j \sum_i \frac{n_{ij}}{N_{ij}}$$

where:

n_{ij} = actual number of applications of the i^{th} load while the pavement is the j^{th} condition, and

N_{ij} = predicted number of applications to failure of the i^{th} load while the pavement is in the j^{th} condition.

The fatigue pa ameter used is the maximum tensile strain in the pavement (Figure 2.11). Comparisons of damage for load limit policies A and B can be made by:

$$\frac{D_A}{D_B} = \frac{\sum_j \sum_i \frac{n_{ij}}{N_{ij}} A}{\sum_j \sum_i \frac{n_{ij}}{N_{ij}} B}$$

Load levels for spring were obtained using this approach by collecting field samples of materials to measure elastic properties in the laboratory and

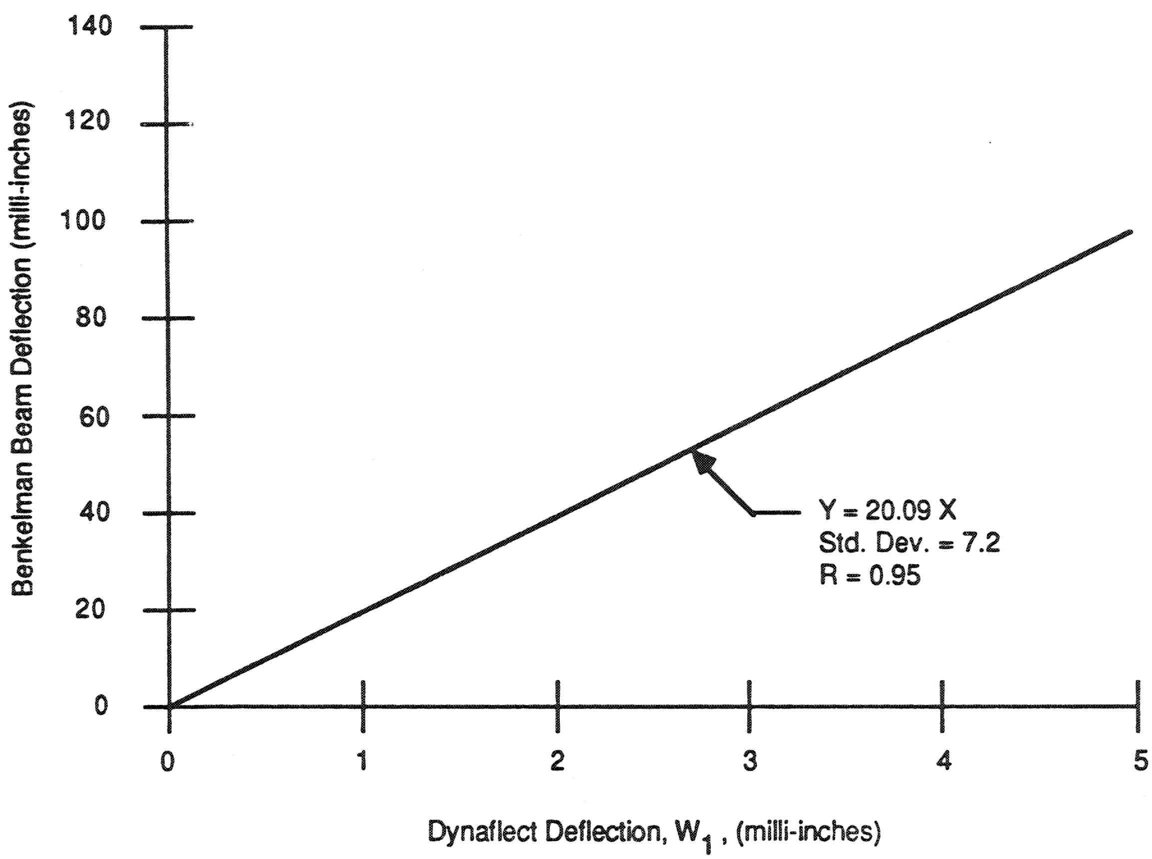


Figure 2.9. Benkelman Beam versus Dynaflect Deflections
(after Scrivner et al., 1969)

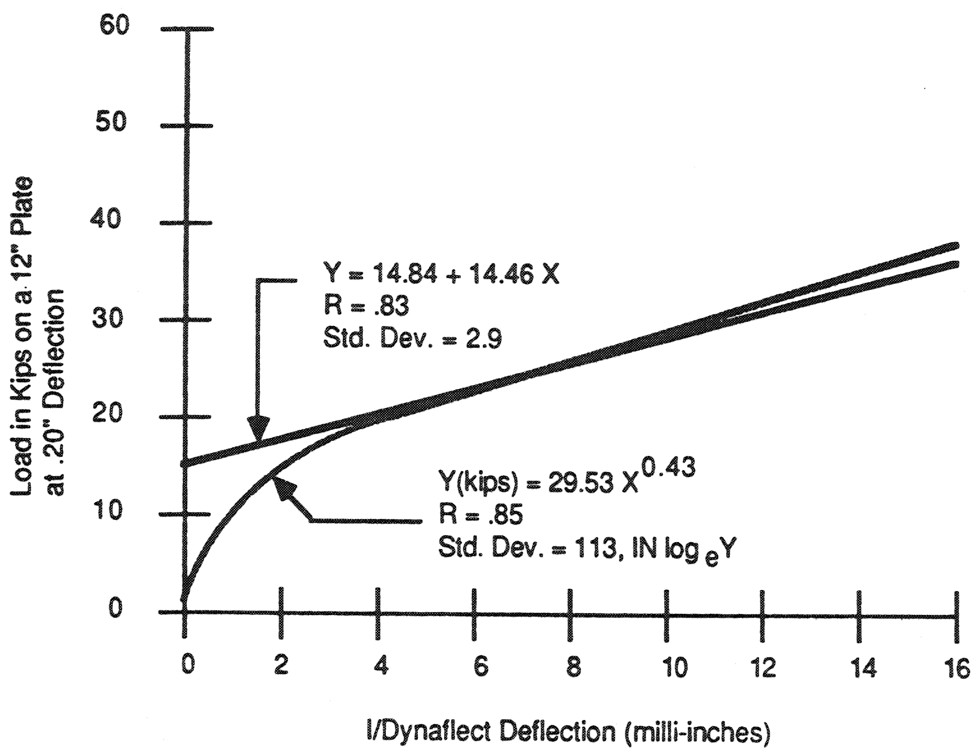


Figure 2.10. Plate Bearing versus the Reciprocal of Dynaflect Deflections (after Scrivner et al., 1969)

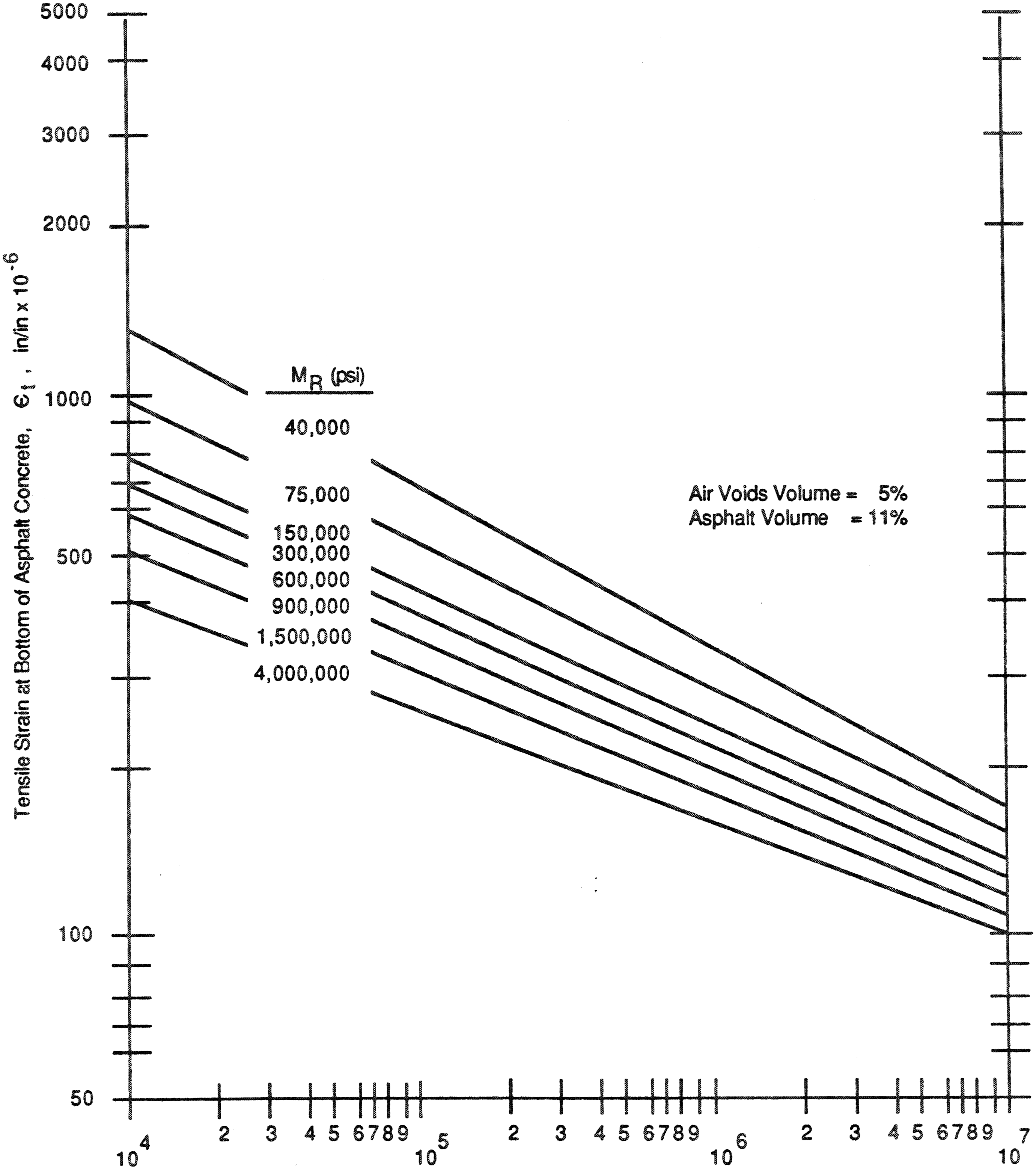


Figure 2.11. Fatigue Curves for Asphalt Mixes

performing a layered elastic analysis of the pavement system using computer program CHEV5L. From the results (stress strains and deflections) obtained, the spring load corresponding to the reference summer strain and deflection conditions could be determined. These results were compared with measured Benkelman Beam deflections with reasonable results. A comparison of spring loads obtained using fatigue consumption to load levels predicted by the NCHRP method [2.17] is shown in Table 2.5. The NCHRP method results in the greatest load reduction, approximately 50 percent. Using fatigue consumption further allows one to estimate the remaining service life of a flexible pavement for various choices of load level.

Connor [2.20] used a similar approach for estimating load reductions based on spring deflection measurements and equivalent fatigue life. He recommended comparing maximum spring deflections to acceptable pavement deflection levels based on asphalt concrete thickness and traffic index where summer reference deflections are unknown. The load level for an equivalent fatigue life can be obtained from Figure 2.12 knowing the maximum deflection in spring. Where summer deflections are known, this value can be used to enter the graph in Figure 2.12.

Stubstad and Connor [2.21] have developed an extensive pavement monitoring system using the Falling Weight Deflectometer (FWD) to be used in areas where severe winter weather conditions exist and thaw weakening affects a major portion of a road network. The FWD was selected in this study because material properties can be realistically backcalculated from the deflection basin data.

The configuration of loading and deflection measurements taken with the FWD are shown in Figure 2.13. The range of thaw depth conditions, layer thicknesses and modulus values assumed is shown in Table 2.6. From this, using the Chevron N-layer computer program a solution table was developed for about 350 cases or combinations of layer thicknesses, thaw depths, and resilient properties. For each case the resulting deflection basin, the horizontal tensile strain in the asphalt concrete and the vertical strain at the surface of the thawed base was obtained.

Table 2.5 Fatigue Life and Load Limit Comparisons (after Hardcastle and Lottman, 1978)

Origin of the Method	Spring-Thaw Load Limit Criterion	Maximum Spring-Thaw Axle Load, L_s (kips)	Critical Tensile Strain (ε_t)	Remaining Fatigue Life Repetitions	Relative Remaining Fatigue Life Percent
Hardcastle and Lottman, 1978	Equal tensile strains in asphalt treated base	11.5	80×10^{-6}	44×10^{6}	100
Idaho Transportation Department	Experience and judgment	14.0	106×10^{-6}	35.3×10^{6}	80
Hardcastle and Lottman, 1978	Equal surface deflection No Restriction	13.8 18.9	104×10^{-6} 157×10^{-6}	36×10^{6} 17.1×10^{6}	80 39
NCHRP Rpt. 76	Surface deflection correlated with experience and policy	9.6	Not computed	Not computed	>100

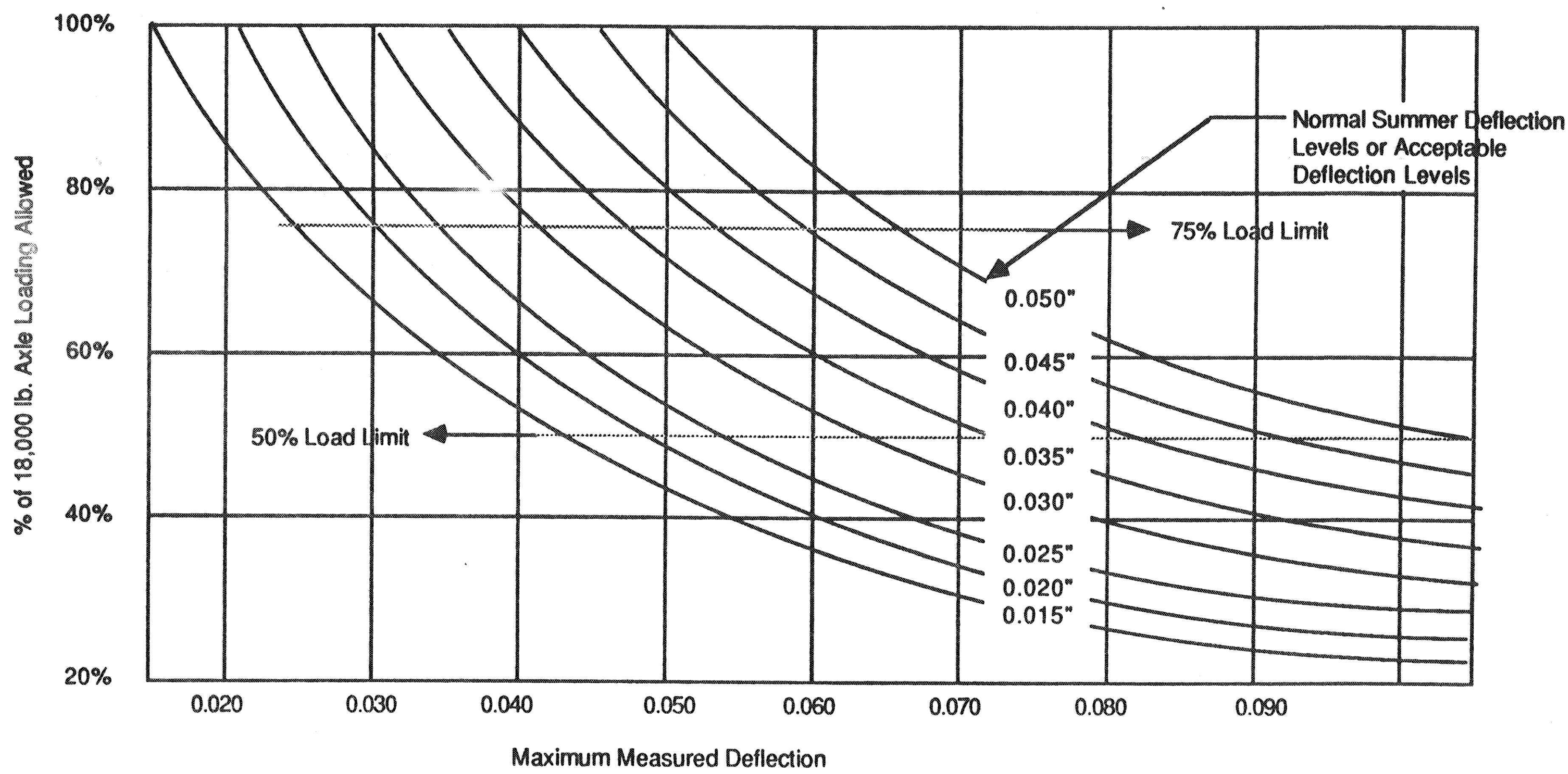


Figure 2.12. Load Limit Percentages from Measured Maximum Spring Deflections and Known or Assumed Acceptable Summer Deflection Levels (after Connor, 1980).

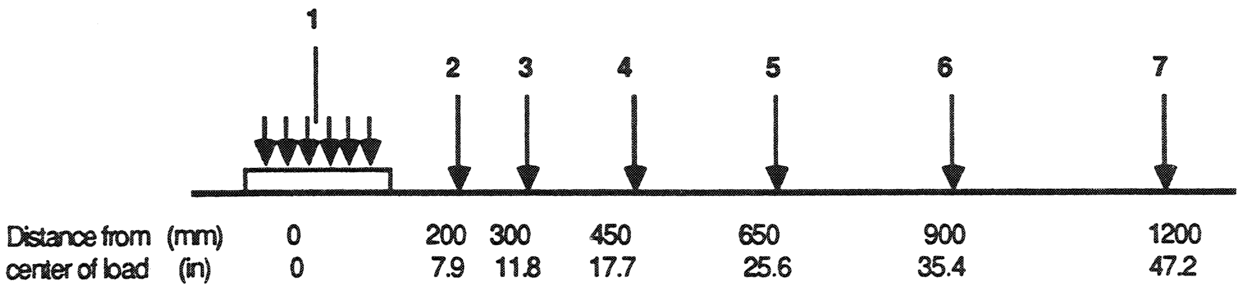


Figure 2.13. Falling Weight Deflectometer Load and Deflection Measurement Configuration (after Stabstad and Connor, 1982)

Table 2.6 Range of Pavement Structure Conditions Assumed to Represent Alaskan Roadway Conditions

Layer	Thickness (in.)	E-Value (psi)
Asphalt Concrete	3/4 to 3	430,000 to 870,000
Granular Base	12	3,500 to 65,000
Subbase/ Embankment	59	11,000 to 22,000
Subgrade	Semi-infinite	7,000 to 15,000
All Frozen Material		1,500,000

Note: The thaw depth below the asphalt was varied from 2 inches to 14 feet.

For monitoring, the FWD was used to obtain deflection basins at various stations along the road network. The data were input into the FROST program which compared the measured deflection basin at each station with the deflection basins in the solution table. The best fit of the data was obtained and the output gave the estimated depth of thaw, the adjusted center deflection for the "summer" (no frost) condition and the damage indicator. For this study the vertical strain at the top of the base course was assumed to be the damage indicator. The information obtained from analyzing the FWD data in the FROST program can be used to impose load restrictions and/or identify specific locations in need of repair.

Lary et al., [2.22] performed an extensive investigation of spring pavement bearing capacity in the State of Washington. The FWD was used to monitor pavement response at six locations during an eighteen month period. Field sampling and laboratory testing was performed for material identification. Most material properties, in particular the resilient modulus, was estimated using the measured deflection basins and backcalculation techniques in the program BISDEF. By assuming a nonlinear elastic stress distribution and the material properties obtained in BISDEF, the vertical strain at the top of the base and subgrade (ε_{vb} and ε_{vs}), the tensile strain at the bottom of the pavement (ε_t), and the surface deflections (δ) were evaluated. Using summer strain and deflection levels as reference values, load levels producing strains or deflections equivalent to the summer values were obtained. This was done for tire sizes ranging from 8 - 22.5 to 16.5 - 22.5. Assuming that any one of the four parameters (ε_{vs}, ε_{vb}, ε_t or δ) created a critical condition, the load level at which any one of these quantities exceeded the summer value was defined as critical. For the six sections analyzed, combining the most critical loading configuration and fatigue parameter, spring load limits of 33 to 45 percent of the equivalent summer loading configuration (i.e., a 55 to 67 percent load reduction) were obtained. Based on a review of all loading cases and their likelihood, a recommendation of a 60 percent <u>reduction</u> in loads during spring thaw weakening was recommended.

2.3.5 TEST ROAD STUDIES OF THAW WEAKENING

Studies on the loss of bearing capacity in spring have been performed on instrumented test roads. Some of these were reported in the Symposium on Bearing Capacity of Roads and Airfields in Trondheim, Norway, in 1982. These studies primarily focused on improved understanding of the mechanisms of frost heave and thaw weakening and their potential relationship. Kubo and Sugawara [2.23] investigated the bearing capacities of subgrades, subbases and bases using buried plates in the Bibi Test Road in Hokkaido, Japan. The results suggest a range of spring bearing capacities of 65 to 85 percent of normal values for all materials combined. This range of values is high compared to most results obtained from U.S. studies.

The Vormsund Test Road in Norway has been extensively studied for frost heave and bearing capacity during spring thaw by Nordal [2.24]. For this purpose several different test profile sections were established. For most sections, base and subbase materials were essentially the same. The subgrades were either silt or clay materials. Benkelman Beam deflection measurements were obtained during thawing and compared to summer values. No strong correlations of frost heaving and thaw weakening were found. Spring strength reductions were on the order of 30 percent for the silt material and 70 percent for the clay based on measurements obtained over a period of several years.

Dysli [2.25] studied thaw weakening on a full scale test road in Switzerland under carefully controlled environmental conditions. Loading, temperature and subgrade water level were maintained at specified levels in various tests. Subgrade and subbase densities, moisture contents and material stiffness properties were carefully measured. Soil temperature was measured at eight different depths. Vertical displacements were measured at nine depths with magnetic sensors. Water contents were monitored with nucleometers. A refrigeration system maintained temperature conditions and traffic loads were simulated with a dynamic jack acting on two circular plates. By varying environmental conditions, freeze-thaw cycles causing slight deformations up to punching failures could be reproduced. Dysli suggested that the results indicate that rate of thaw plays an important role along with the permeability of the subbase and subgrade. Where punching failures had occurred, an increase in pore pressures was observed prior to failure.

The results of a study performed by Esch [2.26] on 120 pavement sections in Alaska showed a significant correlation between the maximum seasonal deflection levels, obtained with a Benkelman Beam, and the percentage of 0.075 mm and 0.02 mm particles in the base and subbase, typically a quantity used as an indicator of frost susceptibility. The fines content was obtained at six depths in the pavements that were monitored in the study. Stress levels due to a standard dual wheel load were obtained assuming a homogeneous elastic material below the pavement with a Poisson's ratio of 0.25. For the resulting vertical stress levels with depth, the critical fines content was obtained, above which increased deflections in spring would occur. The critical fines content was 6 percent (passing 0.075 mm) for depth ranging from 0 to 6 in. The critical fines content increased for greater depth.

Johnson et al., [2.56] reported on the resilient modulus of a silt under various thicknesses of asphalt concrete (for frozen, thawed and fully recovered conditions). Both field and laboratory data was obtained to examine this process. Based on field deflection data, they found resilient moduli for this specific silt soil as low as 290 psi during the critical thaw period and as high as 14,500 psi when fully recovered (thus a loss in stiffness of 98 percent when compared to summer conditions). Further, the resilient modulus of the silt when frozen ranged from a low of 20,300 to 40,600 psi and a high exceeding 200,000 psi (the resilient modulus of the frozen silt being a function of temperature and water content).

2.4 THERMAL CONSIDERATIONS

2.4.1 INTRODUCTION

Soil properties, specifically structure, particle size, pore size and to a lesser extent surface chemistry, are largely responsible for the nature of the ice present in a frozen soil. In addition, and of equal significance, are the environmental factors controlling the degree, rate and history of freezing and thawing occurring in a particular season. Many studies of thaw weakening have focused on identifying climatic conditions and freezing depths, seeking relationships with the degree of thaw weakening.

While no evidence has been found to suggest that depth of frost penetration is an indicator of the severity of thaw weakening, the amount of frozen ground present suggests the potential for spring bearing strength loss. In addition, in order to study the pavement response in spring, the extent of frozen and thawed states must be known.

In 1929, at the Ninth Annual Meeting of the Highway Research Board [2.27], F.H. Eno outlined the importance of climate on

(a) drainage,

(b) subgrade and surface stability, and

(c) load restrictions.

The concept of duration of subfreezing temperatures as a critical index for frost related pavement problems was introduced by Bouyoucos and Petit. From this, Sourwine produced the first mapping of the critical index line for the United States in 1930. From the time of the work of Eno and Taber [2.8] until the 1950's, numerous studies were performed investigating the relationship of several climatic factors related to thaw weakening. However, no conclusive correlations were forthcoming. It was suggested by Crawford and Boyd [2.27] and later echoed by Kubler [2.7, 2.28] that rate of accumulation of the freezing and or thawing index is significant in the severity of thaw weakening. Kubler's conclusions were based on an extensive study of climatological data collected in West Germany from 1952 to 1957.

2.4.2 DEVELOPMENT OF A ONE-DIMENSIONAL MODEL FOR GROUND FREEZING

2.4.2.1 SINGLE LAYER MODELS

In 1860, Neumann presented the first solution for the one dimensional advance of a freezing front due to a step increase in surface temperature in a homogeneous soil. This solution can be found in Carlslaw and Jaeger [2.29]. The solution is of the form:

$$X = \alpha t^{1/2}$$

where:

X = depth of freezing,

t = duration of the freezing period, and

α = constant which is a function of several soil and temperature parameters.

An approximate solution for this problem was proposed by Stefan in 1890, assuming a linear temperature distribution in the zone above the freezing front, and neglecting the temperature profile in the unfrozen zone. This solution becomes:

$$X = \left[\frac{2k_f T_s t}{L}\right]^{1/2}$$

where:

k_f = thermal conductivity of frozen soil,

T_s = applied constant temperature,

t = duration of freezing period, and

L = soil latent heat of fusion.

While the Stefan equation was considerably easier to solve, the resulting calculated freezing depths were typically greater than measured values.

Aldrich and Paynter [2.30] obtained a solution, which closely approximated the Neumann solution upon which it is based, by introducing dimensionless parameters α , μ and λ and making some slight approximations in the transcendental equation in the Neumann solution so that it could be solved digitally. The value necessary for the solution is presented in a nomograph form. This solution is called the Modified Berggren solution and is expressed as:

$$X = \left[\frac{48\ k_{avg}\ n\ FI}{L}\right]^{1/2}$$

where:

$$k_{avg} = \frac{k_u + k_f}{2} \text{ in Btu/ft°F hr,}$$

n = surface temperature coefficient, and

FI = air freezing index, (°F-days).

All other terms have been defined previously.

2.4.2.2 MULTILAYER MODELS

This solution was expanded by Aldrich [2.31] to include any number of layers of different materials. The equation for the depth of freezing for multilayer modified Berggren becomes:

$$X = \lambda \left[\frac{48\ n\ FI}{(L/K)_{eff}}\right]^{1/2}$$

where:

$(\frac{L}{k})_{eff}$ = ratio of the effective thermal properties for an n-layer system

$$(\frac{L}{k})_{eff} = \frac{2}{X^2}\left[\frac{d_1}{k_1}\left(\frac{L_1 d_1}{2} + L_2 d_2 + \ldots + L_n d_n\right) + \frac{d_2}{k_2}\left(\frac{L_2 d_2}{2} + L_3 d_3 + \ldots\ L d_n\right) + \frac{d_n}{k_n}\left(\frac{L_n d_n}{2}\right)\right]$$

In addition, the value of λ is determined by using weighted values of C and L to evaluate the fusion parameter μ,
where:

$$C_{wt} = \frac{C_1 d_1 + C_2 d_2 + \ldots + C_n d_n}{X}$$

$$L_{wt} = \frac{L_1 d_1 + L_2 d_2 + \ldots + L_n d_n}{X}$$

where:

C = volumetric heat capacity.

A multilayer Stefan solution was proposed by Kersten and Carlson [2.32] which follows the same assumptions as the single-layer Stefan solution. The solution proceeds by requiring that heat flow be balanced at the layer interfaces. This approach yields the following equations:

for Layer 1: $F_1 = \dfrac{L_1 h_1^2}{48\ k_1}$

where:

F_1 = the number of °F-days required to freeze layer 1

for Layer n: $$F_n = \frac{L_n h_n}{24}\left(\frac{h_{n-1}}{k_{n-1}} + \frac{h_n}{2k_n}\right)$$

The Stefan and Berggren solutions are by far the most widely used methods for estimating depth of freezing or thawing. Several similar approaches have been proposed throughout the early to mid 1900's. The reader is referred to an excellent literature summary by Moulton [2.33] for a thorough treatment of this topic.

2.4.3 EVALUATION OF THERMAL PROPERTIES

Three thermal properties, conductivity, volumetric specific heat and latent heat, are required to evaluate the equations outlined above or to perform any ground heat transfer analysis where freezing occurs. Latent heat and specific heat can be measured using calorimetric techniques. Thermal conductivity can only be evaluated indirectly by measuring temperature differences resulting from controlled heat flow in the medium where boundary conditions conform to some known analytic solution.

For engineering purposes, these properties are rarely measured. For soils, they are primarily functions of the dry density (γ_d) and the moisture content (w). Typically, estimates for ground thermal properties are made using the following equations:

(a) Latent heat:

$$L = (144\ \text{Btu/lb})\gamma_d w \qquad (\text{Btu/ft}^3)$$

(b) Volumetric specific heat:

Unfrozen soil

$$C_u = \gamma_d \left(0.17 + 1.0\frac{w}{100}\right) \qquad (\text{Btu/ft}^3)$$

Frozen soil

$$C_f = \gamma_d \left(0.17 + 0.5\frac{w}{100}\right) \qquad (\text{Btu/ft}^3)$$

The equations for thermal conductivity of soils most frequently used were developed by Kersten (2.34). They are the following:

(c) Thermal conductivity:

Unfrozen soil:

Fine-grained: $k_u = (0.9 \log_{10}w - 0.2)10^{0.01\gamma d}$ (w)

Coarse-grained: $k_u = (0.7 \log_{10}w + 0.4)10^{0.01\gamma d}$ (w)

Frozen soil:

Fine-grained: $k_f = 0.01(10)^{0.022\gamma d} + 0.085(10)^{0.008\gamma d}(w)$

Coarse-grained: $k_f = 0.076(10)^{0.013\gamma d} + 0.032(10)^{0.0146\gamma d}(w)$

2.4.4 EVALUATION OF THE "n" FACTOR

Lunardini [2.35] discusses the necessity of observing the precise definition of the n factor used in the Stefan and Berggren solutions:

$$n = \frac{\text{Surface FI}}{\text{Air FI}}$$

It should be obtained from temperatures measured above the ground surface level (typically four feet) and on a particular surface type and not "back-calculated" from a particular heat transfer solution such as Modified Berggren. The n-factor, as it appears in the Stefan and Berggren equations, is intended to be representative only of surface effects.

Kersten and Johnson [2.36] suggest an n-factor for freezing of 0.8 for Minnesota pavements. This, however, is based on comparing measured and predicted freezing depths. Argue and Denyes [2.37] reported the comparison of air freezing index and surface freezing index based on the measured values of the frost depth compared to calculated values using a Modified Berggren approach, which is not in strict adherence with the definition. The results, shown in Figure 2.14 for cleared asphalt surfaces, show decreasing n with decreasing FI. Using an n-factor from Figure 2.14 and the specific layer properties, the Modified Berggren equation predicted frost depths within a standard error of seven inches when compared to the measured depths.

An extensive study of climatological factors related to frost action was performed in Pennsylvania from 1969 to 1976 and reported by Hoffman et al. [2.38]. Fourteen sites throughout the state were instrumented with thermocouples to collect ground temperature data. Surface and air temperatures were compared at all sites to estimate n-factors. The average value of n for

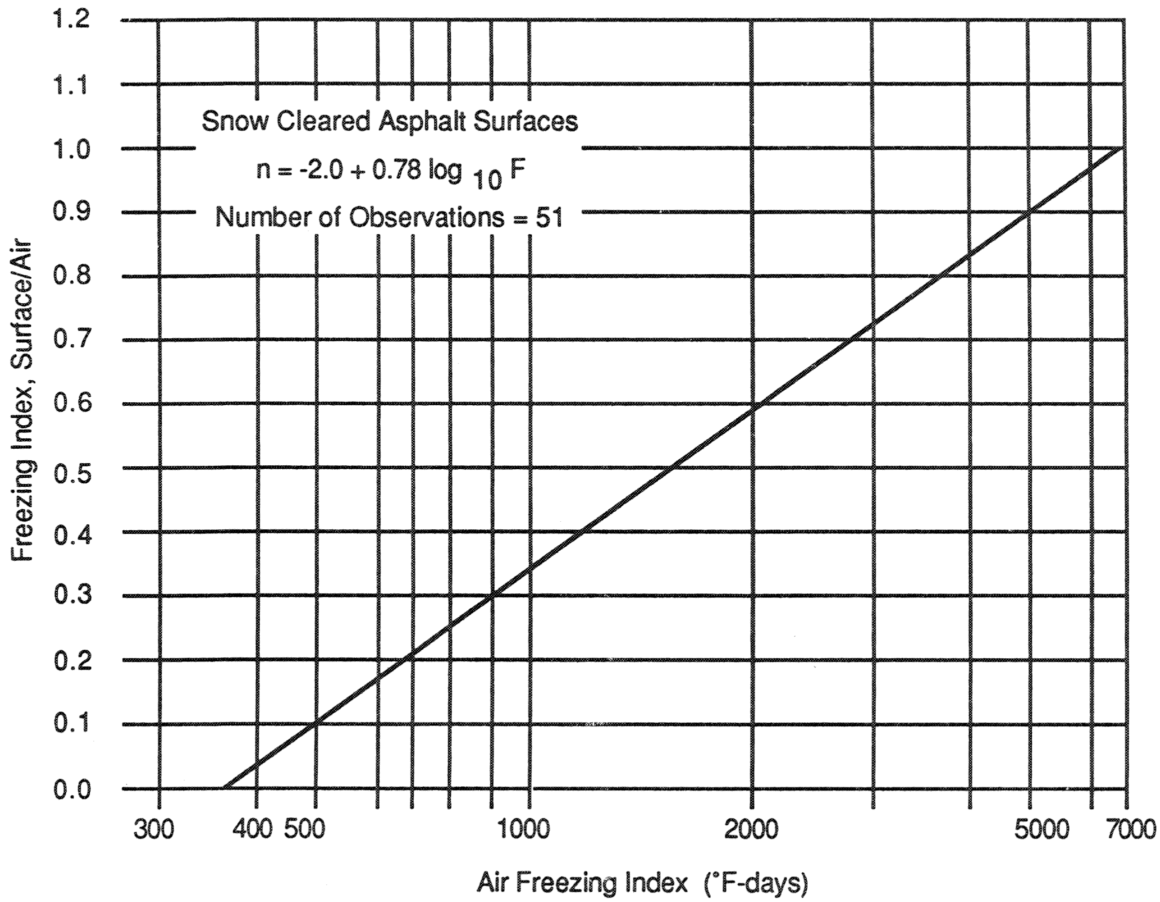


Figure 2.14. Freezing Index Surface/Air Correction Factor versus Air Freezing Index (after Argue and Denyes, 1974)

the eight years of data collection at all sites ranged from 0.25 to 0.51. The n-factor was found to increase with increasing air freezing index for the Pennsylvania data. The regression line obtained for the data was:

$$n = 0.6106 - \frac{68.0596}{AFI}$$

where:

AFI = air freezing index

Surface and air temperatures were recorded during freezing seasons in New Jersey from 1975 to 1977 at three different locations (report by Berg [2.39]). The freezing season duration, air freezing index and n-factors are shown in Table 2.7.

2.4.5 MEASUREMENT AND PREDICTION OF FROST DEPTH

Early estimates of frost penetration beneath pavements were made by Kersten and Johnson [2.36] using the layered Stefan solution. Estimates based on this technique were compared to field measurements made at nine sites near Minneapolis in 1953-54. At each location studied, the soil was sampled to a depth of eight feet and moisture contents were determined every six inches. Dry densities for the samples were evaluated using approximate methods. Air temperatures were measured in the region of the test sites as U.S. Weather Service temperature data was also collected. The depth of freezing was determined from borings done every two to three weeks.

From 1964 to 1971, 38 airports throughout Canada were instrumented with Gandahl type f ost depth indicators (Argue and Denyes, [2.37]). These were installed beneath pavement surfaces kept clear of snow. Temperatures were measured at all locations and after the start of freezing the air freezing index was tabulated. The data obtained for measured freezing depth and air freezing index is shown in Figure 2.15.

Several of the quantities used in the Modified Berggren and Stefan equations are difficult to estimate precisely, in particular, n-factors, thermal conductivity and latent heat during freezing. The sensitivity of the

Table 2.7 Freezing Indices and "n" Factors for Three New Jersey Locations (after Berg, 1979)

Location	Air		Portland Cement Concrete			Asphaltic Concrete		
	Season (days)	Index (°F-days)	Season (days)	Index (°F-days)	n-factor	Season (days)	Index (°F-days)	n-factor
Bordentown	46	316	41	93	0.29	44	181	0.57
Bedminster	51	446	52	388	0.87	52	400	0.90
Rockaway	84	783	72	295	0.38	70	304	0.39

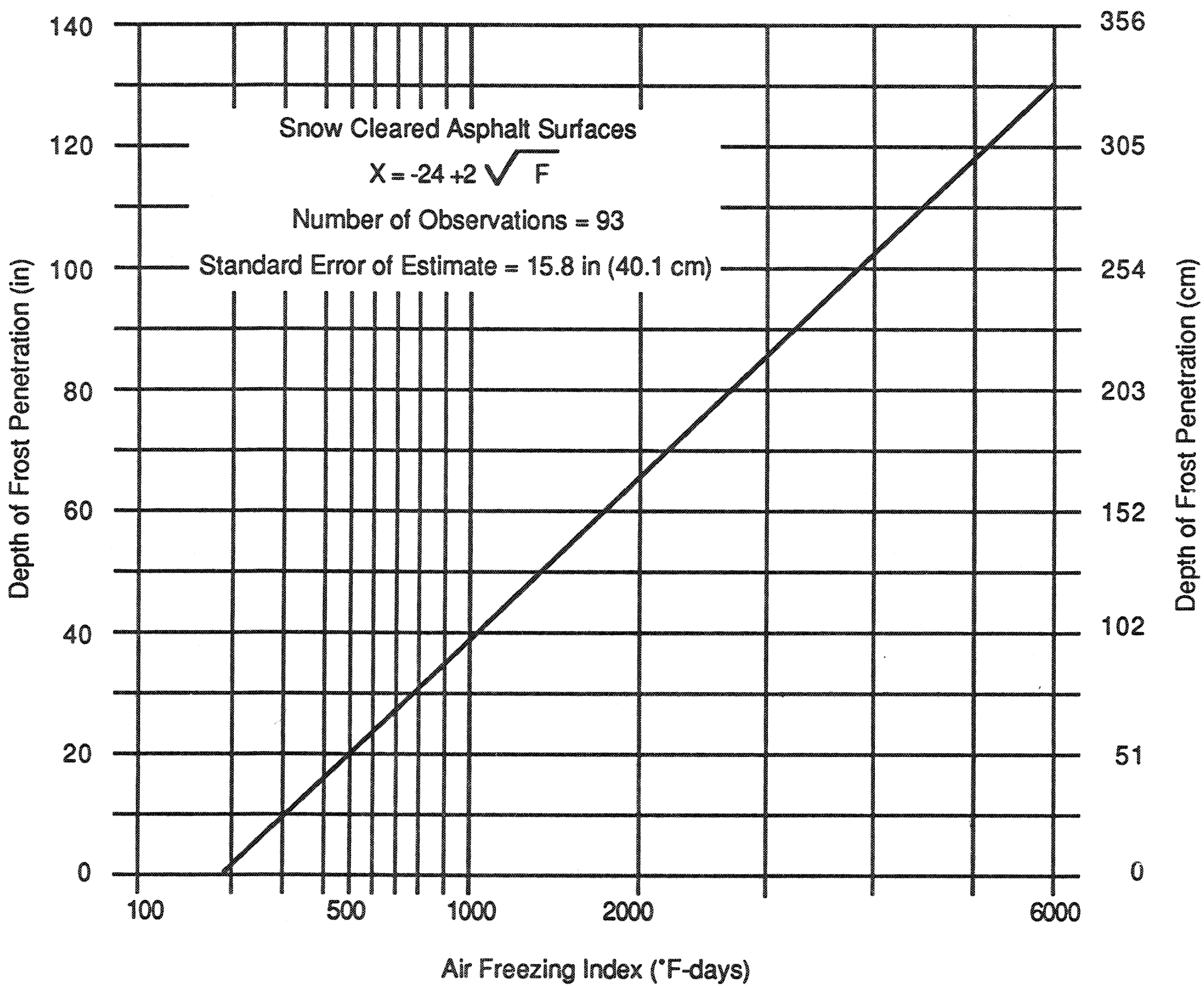


Figure 2.15. Depth of Frost Penetration versus Air Freezing Index, Canadian Pavements (after Argue and Denyes, 1974)

frost depth determined from Modified Berggren to these quantities was studied by Berg and McGaw [2.40] and Berg [2.39] for New Jersey soils.

In their work, Berg and McGaw [2.40] measured freezing depths at 30 sites in New Jersey and Modified Berggren estimates were compared to measured values. Typically, the measured values of frost penetration exceeded the predicted frost depth by a large amount. To investigate the sensitivity of the analytical solution to changes in some thermal properties which are difficult to identify, variations in water content in frozen and unfrozen soil were considered in estimations of thermal conductivity and latent heat of fusion. Using the results of Lovell [2.41] an estimate was made of the amount of unfrozen water present in the frozen soil by soil type. In addition, some adjustments in the Kersten thermal conductivity values for granular soils were made to account for the percentage of fines in the soil. In general, improved results were obtained when including these effects; however, in all cases, the freezing index and n-factor were subject to some uncertainty as well so that no strong conclusions could be made. Also, in some instances, improved results were found when using the ground temperature immediately before freezing instead of the mean annual temperature.

In a study of Pennsylvania pavements (Hoffman [2.38]), an extensive material characterization and thermal instrumentation was performed. Moisture content, dry densities, gradation analyses and Atterberg limits were estimated at several levels in a pavement profile. In addition, temperatures in the ground were measured at several elevations with thermocouples. The sites were monitored on a monthly basis during the freezing period. In addition, surface heave and deflection measurements were taken. Air temperature and precipitation data was collected from the local weather service station.

This data was used for several purposes. Estimates of depths of freezing were compared to predictions using the Corps of Engineers frost depth measurement procedure. Their findings suggest that frost depth at these sites was a function of the air freezing index (Figure 2.16). An excellent comparison was found using this very simple technique with the Pennsylvania data. In addition, an extensive study of the Modified Berggren equation was

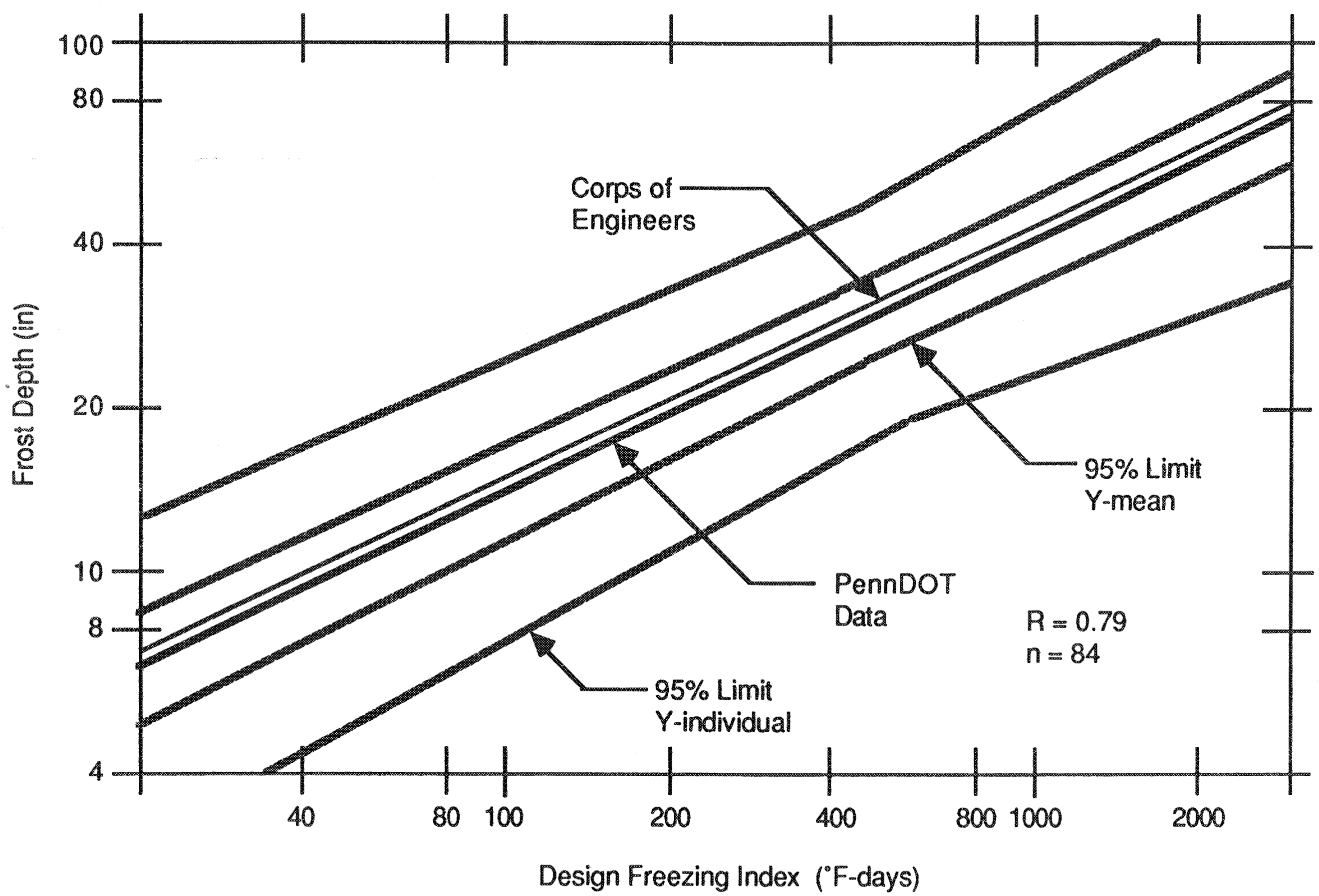


Figure 2.16. Comparison of Pennsylvania Data to Corps of Engineers Method of Frost Depth Prediction (after Hoffman et al.., 1979)

performed. The actual average ground temperature at the beginning of the freezing season was used in the analysis. In addition, thermal properties based on measured moisture contents were used. Air and pavement freezing indices obtained from measurements were used in separate analyses using the Modified Berggren equation. The best comparison between measured and predicted frost depth penetration was obtained using the air freezing index, the unfrozen moisture content and calculated thermal conductivity. The results for both analyses are tabulated in Tables 2.8 and 2.9.

Chisholm and Phang [2.42] measured frost penetration at 62 locations in Ontario, Canada, between 1970 and 1975 using frost tubes. Using air temperature data collected at nearby weather stations a correlation equation was established from a regression of the penetration depth, P, and the air freezing index, F, in °C-days where:

$$P = -0.328 + 0.0578\,[F]^{1/2}$$

Many of the studies mentioned considered the possibility of a relationship between freezing index or freezing depth and maximum spring thaw deflections. There is, as yet, no strong evidence to suggest that these variables are correlated.

2.4.6 MEASUREMENT AND PREDICTION OF THAW DEPTH AND THAW WEAKENING

2.4.6.1 PREDICTIONS OF THAW DEPTH

The analytical techniques for evaluating thawing depth are the same as those used for freezing depth. A major source of uncertainty is the surface coefficient, or n-factor, which for a given location and surface type is most definitely different for freezing and thawing. Several references noted in earlier sections have focused on the estimation of freezing depths and corresponding n-factors. Little research was found on the associated thawing problem.

Early investigations of thawing were performed in Minnesota by Korfhage (2.43). Six field sites were instrumented with copper-constantan thermocouple strings to observe the advancement of the thaw plane. Field measurements of thawing were compared with estimates using the Stefan equation with

Table 2.8 Measured and Predicted Frost Depths in Pennsylvania Using Air Freezing Index (after Hoffman et al., 1979)

Site Location	Actual Depth (in.)	Frozen Moisture Content				Unfrozen Moisture Content				Corps of Engineers
		Stefan		Exact Modified		Stefan		Exact Modified		
		(a)	(b)	(a)	(b)	(a)	(b)	(a)	(b)	
Butler	51	45.0	42.0	39.6	36.7	46.8	42.8	41.0	37.3	33
Center Point	24	30.9	33.2	22.9	25.0	32.0	33.4	23.6	24.9	24
Clarion	37	47.9	45.3	41.7	39.1	52.1	48.0	45.1	41.0	36
Fulling Mill Rd.	14	31.9	31.8	22.0	21.9	33.1	31.6	22.6	21.4	22
Lairdsville	48	47.2	54.1	43.8	50.1	48.7	55.5	45.1	51.3	38
Lantz Corners	41	57.7	50.0	53.1	45.4	59.3	51.0	54.5	46.2	44
Meadville	30	37.3	42.9	31.8	37.0	37.8	43.0	32.2	37.0	37
Perkiomenville	26	42.6	52.1	31.8	39.8	46.9	54.0	34.4	40.4	30
Roseglen	36	50.7	46.5	43.8	40.3	54.8	50.7	46.7	43.4	31
Somerset	34	40.7	44.0	35.0	37.9	41.4	44.0	35.5	37.8	32
State College	38	49.9	46.6	44.1	41.2	51.1	46.3	45.0	40.8	39
Washington	24	38.3	40.7	31.3	33.8	39.0	40.9	31.8	33.8	31
Wellsboro	45	82.8	77.2	70.3	65.0	84.2	77.2	71.1	64.4	41
Wilkes-Barre	44	53.9	59.9	47.9	53.5	56.3	60.6	49.9	53.7	40

Notes: (a) Inches of frost penetration predicted using graphical thermal conductivities from fiield data.

(b) Inches of frost penetration predicted using calculated thermal conductivities by Kersten's equations.

Table 2.9 Measured and Predicted Frost Depths in Pennsylvania Using Pavement Freezing Index (after Hoffman et al., 1979)

Site Location	Actual Depth (in.)	Frozen Moisture Content				Unfrozen Moisture Content				Corps of Engineers
		Stefan		Exact Modified		Stefan		Exact Modified		
		(a)	(b)	(a)	(b)	(a)	(b)	(a)	(b)	
Butler	51	40.9	38.2	35.5	32.9	42.1	38.8	36.4	33.2	29
Center Point	24	25.7	26.9	16.7	18.4	26.9	27.4	17.2	18.4	17
Clarion	37	41.8	38.8	35.8	32.5	42.9	39.4	36.5	32.9	26
Fulling Mill Rd.	14	23.6	23.3	14.7	14.4	24.7	23.4	15.1	14.1	16
Lairdsville	48	35.4	40.1	32.6	36.9	36.4	41.1	33.5	37.8	28
Lantz Corners	41	41.4	37.0	37.7	33.1	42.3	37.6	38.5	33.7	32
Meadville	30	29.0	32.5	24.6	27.8	29.4	32.6	24.9	17.9	28
Perkiomenville	26	29.8	33.5	20.0	23.3	32.1	35.1	21.2	23.9	19
Roseglen	37	27.0	27.6	22.1	22.7	29.3	28.8	23.7	23.3	17
Somerset	34	31.1	34.0	25.8	28.4	31.8	34.1	26.2	28.4	24
State College	38	35.4	33.9	30.4	29.3	36.5	34.0	31.2	29.1	27
Washington	24	26.2	27.3	19.1	20.4	27.1	27.7	19.5	20.5	19
Wellsboro	45	68.4	56.0	56.5	46.3	69.7	55.9	57.0	46.7	28
Wilkes-Barre	44	31.7	35.2	27.4	30.6	34.4	36.9	29.4	31.8	25

Notes: (a) Inches of frost penetration predicted using graphical thermal conductivities from field data.

(b) Inches of frost penetration predicted using calculated thermal conductivities by Kersten's Equations.

thermal conductivity values calculated using their Kersten equations. From this comparison Korfhage estimated surface n-factors and base temperatures used in computing degree-days of thaw. He concluded that a base temperature of 32°F for fine grained soils and 29°F for coarse grained soils should be used in the Stefan equation. In addition a surface correction factor, varying from 1.7 to 2.7 for fine-grained soils and 1.2 to 2.0 for coarse-grained soils was suggested by the results.

Argue and Denyes [2.37] collected thaw penetration data on cleared gravel runways from permafrost areas in Northern Canada. The data summary is shown in Figure 2.17. In addition, thaw depths were established in several locations by soundings. Based on the combined data set, an upper limit for the thaw depth as a function of thawing index (Figure 2.18) was established as:

$$x = 1.85 [I]^{1/2}$$

where:

I = thawing index

2.4.6.2 THAW DEPTH AND THAW WEAKENING

Relationships between thaw depth and maximum spring deflections have been suggested by Connor [2.20]. Based on Benkelman Beam deflection data collected in Alaska, it was found that most road sections reached about one half the peak spring deflection level when the thaw depth reached about one foot. Peak deflections generally occurred when the thaw depth reached two to four feet below the pavement layer. In addition, it was noted that peak deflections often occurred very soon after average daily temperatures rose above 32°F. For five of seven sections studied, average daily air temperatures had been above 32°F for only four days and the average air thawing index was 31°F days.

In a study of Washington pavements, Lary et al. [2.22], found that the pavements studied reached a critical condition when the thawing index was approximately 30°F-days.

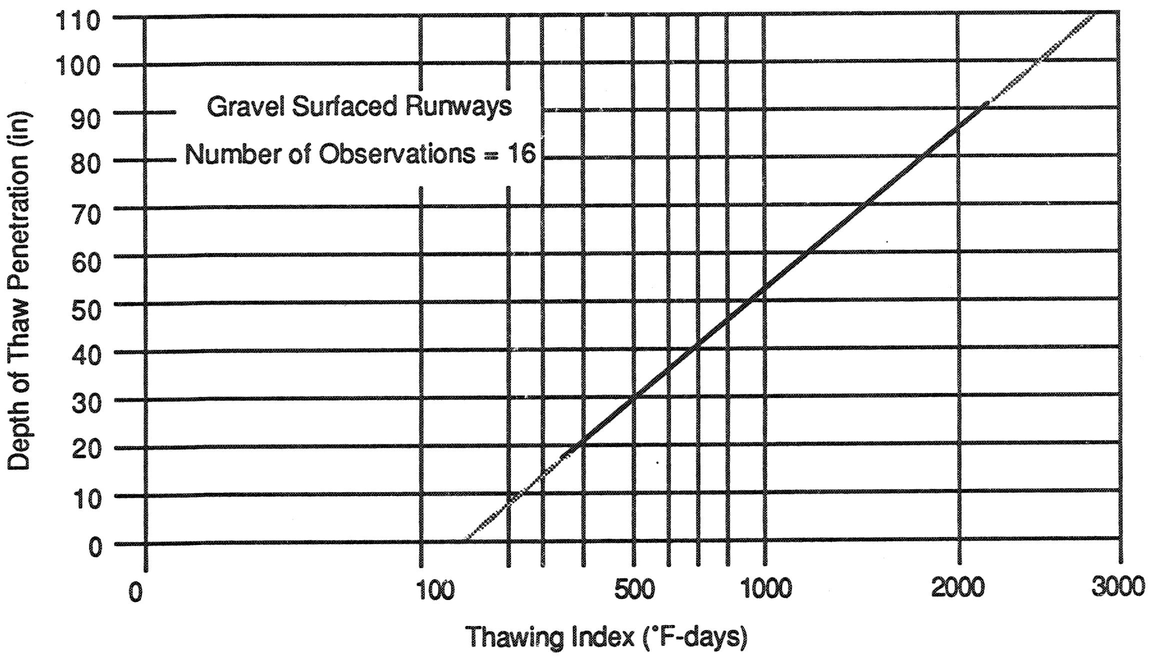


Figure 2.17. Maximum Thaw Penetration in Gravel-Surfaced Runways on Permafrost in Northern Canada (after Argue and Denyes, 1974)

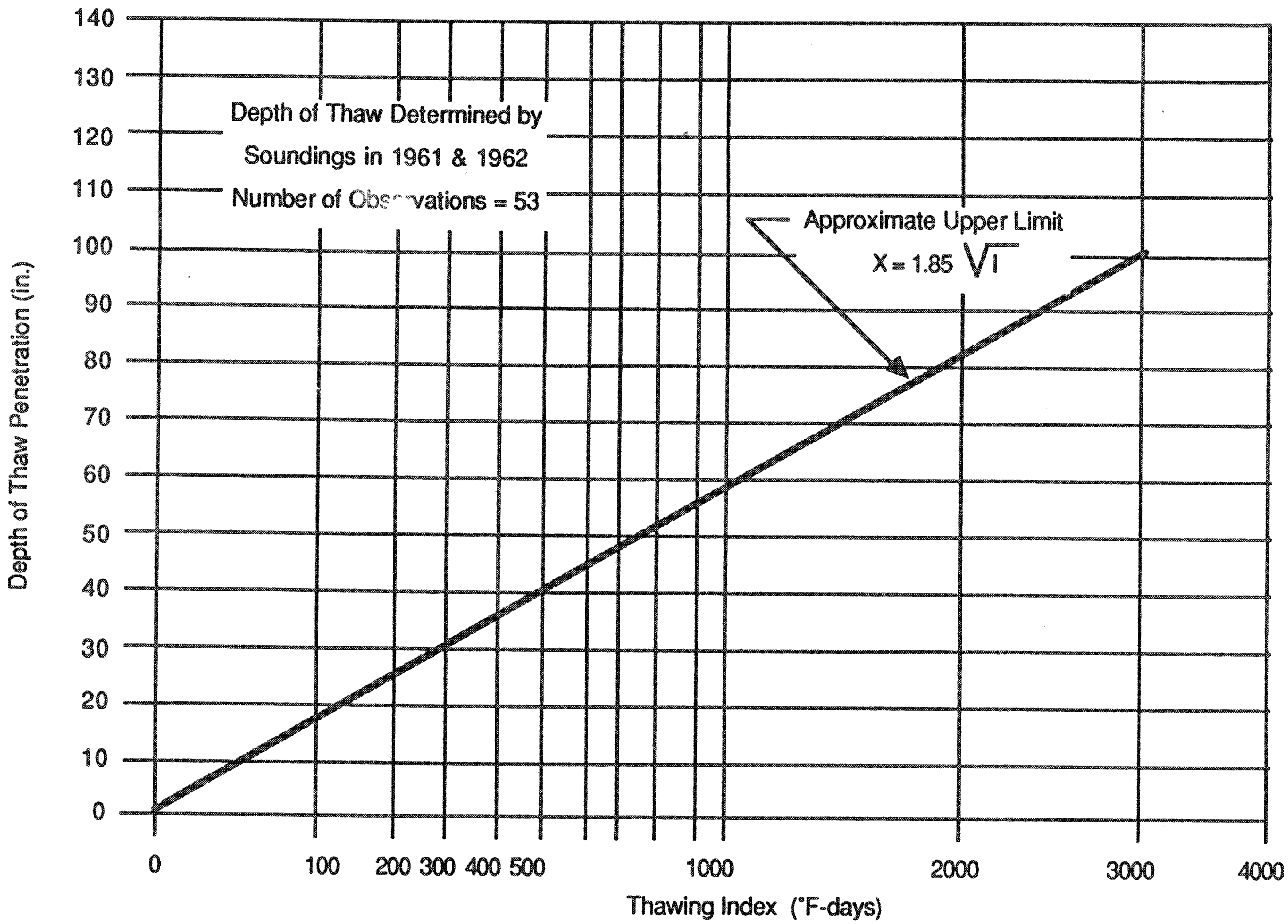


Figure 2.18. Maximum Thaw Penetration in Undisturbed Permafrost Areas in Northern Canada (after Argue and Denyes, 1974)

2.4.7 NUMERICAL METHODS FOR GROUND THERMAL ANALYSIS

Several heat transfer models have been used to evaluate frost penetration. Dempsey and Thompson [2.44] used a one-dimensional forward finite difference model for multilayer pavement thermal response. The surface energy balance equation considered the effects of short and long wave radiation, convection and air temperature. Comparisons of measured and predicted temperatures were made only at shallow depths (3 to 6 in.) in composite laboratory specimens. These results showed good agreement. The authors state that these results suggest that the surface modelling is adequate and accurate estimates of subsurface thermal properties would produce good comparisons at any depth.

Thomas and Tart [2.45] proposed using a two-dimensional finite element simulation of heat flow in soils to predict freezing and thawing. In contrast to Dempsey and Thompson, little emphasis was placed on surface effects and greater emphasis was placed on modelling the phase change effects. This was accomplished by using temperature dependent heat capacity functions to model latent heat. Large increases in specific heat (equal to latent heat) were specified over a temperature range at the freezing point of the material. The program used was DOT (Determination of Temperature) developed at Berkeley by Polinka and Wilson [2.46]. Several more sophisticated multi-dimensional finite element heat transfer programs are available that offer several model options (modes of heat generation and dissipation).

Chisholm and Phang [2.42] used a finite difference heat transfer model with stepwise insertion of weather data to predict frost depth and ground temperature conditions. The surface energy balance was obtained by considering solar radiation, cloud cover, air temperature, wind speed, atmospheric pressure, albedo and surface aerodynamic roughness. Using this approach a surface temperature can be obtained for input into the flow equations in the ground.

Two sites were specially instrumented to compare model predictions with field temperature conditions. Observed and predicted frost lines through the freezing and thawing season are shown in Figure 2.19. The freezing depths

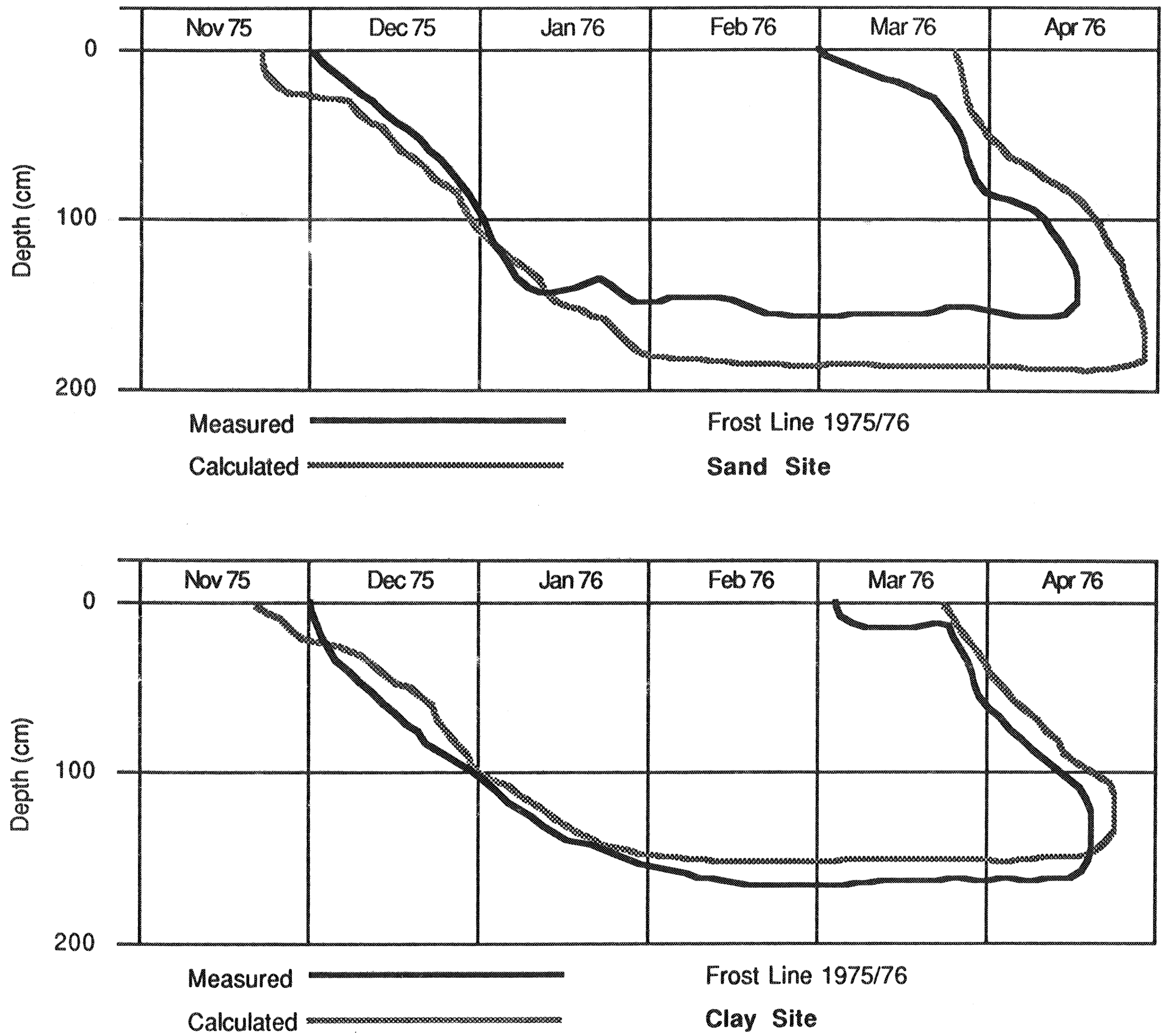


Figure 2.19. Observed and Predicted Frost Lines, Ottawa, Canada (after Chrisholm and Phang, 1983)

show reasonable agreement. Thaw lines, however, are not well predicted by the model.

Goering and Zarling [2.47] have developed a two dimensional, finite element, ground heat transfer model that runs on an IBM-PC or XT. A sinusoidally varying annual surface temperature function can be used. In addition, convective heat transfer at the ground surface can be modelled as well as radiant heat in the form of a heat flux. The latent heat of fusion is modelled using the Dirac delta function in the formulation of the global heat capacity matrix.

2.4.8 MODELLING GROUND SURFACE EFFECTS

In addition to heat being transferred at the ground surface by conduction, convection and radiation play an important role in the surface energy balance. Convective heat transfer at the ground surface is primarily due to air movement across the interface. The radiant heat is a combination of atmospheric short and long wave radiation and long wave radiation emitted from the earth's surface. The energy balance can be written as:

$$Q_{COND} + Q_{CONV} + Q_{RSN} + Q_{RLN} = 0$$

The various sources of heat interacting at the ground surface are shown schematically in Figure 2.20.

2.4.8.1 SHORT WAVE RADIATION

Theoretically, direct, clear sky, short wave radiation is a function of latitude and solar declination. The available daily direct short wave radiation on a horizontal surface for a transparent atmosphere is given by (Lunardini [2.35]):

$$Q_{RS} = 60 \times 24S \left(\frac{180 - H_{SR}}{180} \cos H_{SR} + \frac{\sin H_{SR}}{\pi}\right) \cos\delta \cos\phi$$

where:

Q_{RS} = direct short wave radiation heat flux, in langleys/day

S = solar constant, in langleys/minute

H_{SR} = hour angle at sunrise

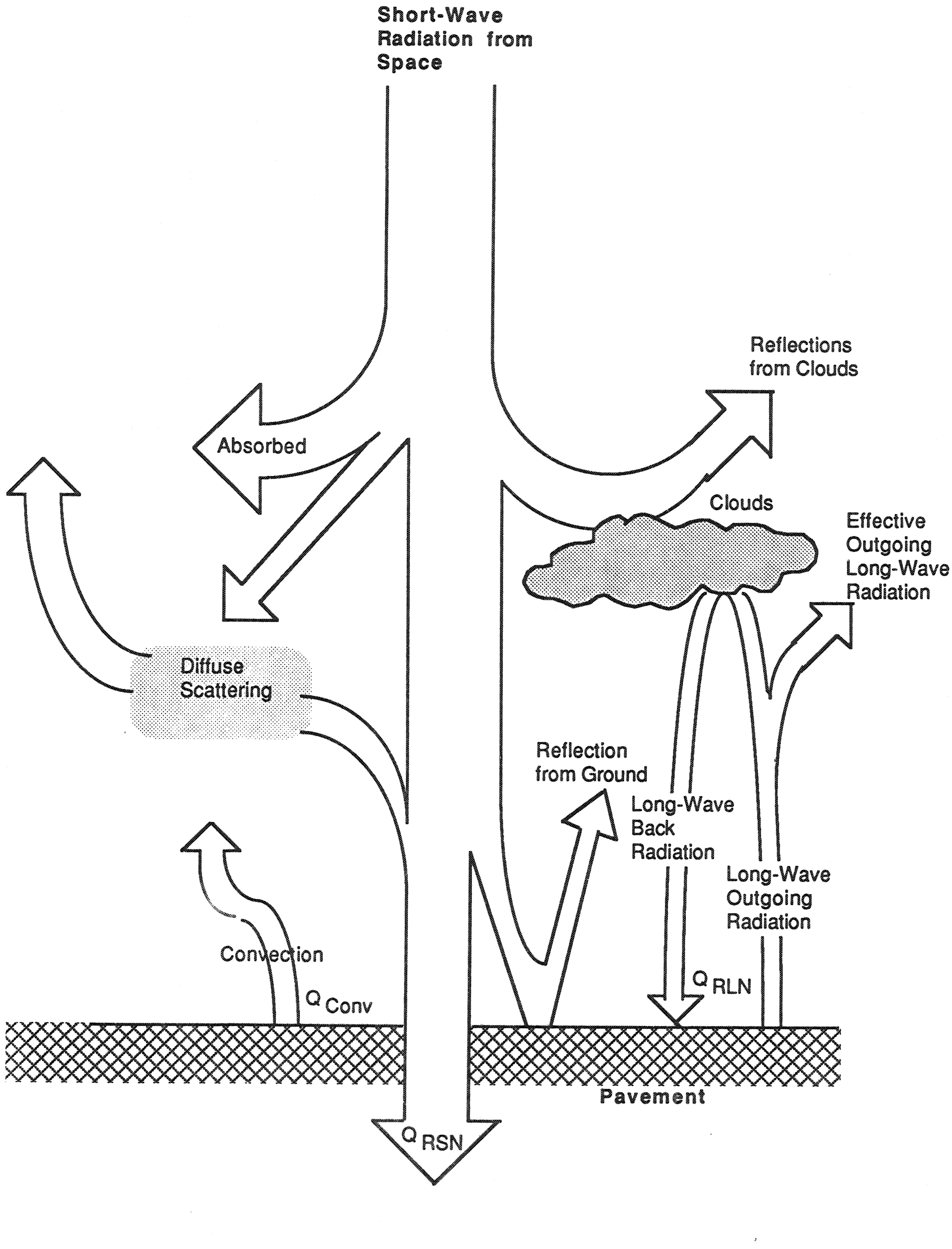


Figure 2.20. Heat Transfer between Pavement Surface and Air (after Dempsey and Thompson, 1970).

δ = solar declination angle, and

ϕ = latitude.

Actual atmospheric conditions which include particulate matter, water vapor and clouds cause scattering, reflection and absorption of the short wave radiation emitted from the sun. Also, the distance between the sun and the earth varies throughout the year, altering the intensity of radiation. These factors are accounted for with empirically derived constants and function incorporated in the equation given above.

The effect of the distance between the sun and the earth on the value of the solar constant used in the equation is accounted for by the following:

$$S = S_m \left(\frac{r_m}{r}\right)^2 \text{ ,and}$$

$$\frac{r}{r_m} = 1 - 1.6733 \times 10^{-2} \cos(0.9856D)$$

where:

S_m = 1.99 langleys/min., the mean solar constant,

r_m = the mean earth/sun distance, and

D = days elapsed since December 31.

Two constants, A and B, are introduced to account for wave attenuation due to scattering and absorption and dust attenuation. The precise form of the expressions for estimating A and B varies with researchers. The B value reflects surface albedo and attenuation characteristics which are primarily due to the preciptable water vapor and the optical air mass. The cosntant A primarily accounts for the particulate matter present in the atmosphere.

In addition, corrections are made for the amount of cloud cover present, which significantly affects the amount of short wave radiation reaching the earth's surface. Several empirical expressions have been derived (see [2.35]). Some of the differences in expressions are a result of the period over which the cloud cover is being estimated, daily or monthly. The following equation includes the considerations noted above, solar distance, attenuation and cloud cover in its formulation:

$$Q_{RS} = (1 - 0.67C_s^2)\ 2865.6\ AB \left(\frac{r_m}{r}\right)^2 \cos\delta \cos\phi \left(\frac{180 - H_{SR}}{180} \cos H_{SR} + \frac{\sin H_{SR}}{\pi}\right)$$

where:

C_s = the average daily cloud cover during daylight hours.

This expression estimates the net incoming direct and diffuse daily short wave radiation. Surface albedo or reflexivity results in some of the incoming heat being reflected back to the atmosphere. Therefore, the net heat flux transmitted to the ground at the surface becomes:

$$Q_{RSN} = (1 - \alpha_s)\, Q_{RSC}$$

where:

α_s = surface short wave reflexivity

2.4.8.2 LONG WAVE RADIATION

Long wave radiation is emitted by the earth's surface and the atmosphere. Long wave radiation from the earth can be expressed as:

$$Q_{RLE} = \sigma\, \varepsilon_e T_e^4$$

where:

σ = 1.714 x 10^{-9} Btu/hr ft^2 $^oR^4$, the Stefan-Boltzmann constant,

ε_e = long wave emissivity of the surface, and

T_e = surface temperature, in ^oR

Similarly, long wave atmospheric radiation can be written as:

$$Q_{RLA} = \sigma\, \varepsilon_a T_a^4$$

where:

ε_a = long wave emissivity of the atmosphere, which is a function of water vapor pressure and temperature, and

T_a = air temperature at reference level, in ^oR

An empirical relationship obtained by Swinbank gives:

$$\varepsilon_a = 0.398 \times 10^{-5} T_c^{2.148}$$

where:

T_c = air temperature, in ^oK

The net clear sky outgoing long wave radiation becomes:

$$Q_{RLO} = \sigma\, \varepsilon_e T_e^4 - \alpha_e \sigma\, \varepsilon_a T_a^4$$

For long wave radiation, the absorptivity is approximately equal to the emissivity or

$$\varepsilon_e = \alpha_e$$

An additional simplifying assumption is that

$$T_s \simeq T_a$$

Incorporating these assumptions, the resulting equation for net clear sky outgoing long wave radiation becomes:

$$Q_{RLO} = \sigma \varepsilon_e T_a^4 (1 - \varepsilon_a)$$

The net outgoing long wave radiation will be reduced by cloud cover. A simple approximate empirical relation based on results of several researchers is proposed in Lunardini [2.35] as:

$$Q_{RLN} = (1 - 0.8C_e) Q_{RLO}$$

where:

C_e = the net 24 hour cloud cover.

2.4.8.3 CONVECTIVE SURFACE HEAT TRANSFER

The convective portion of the surface heat balance is due primarily to air movement and can be calculated from the following equation:

$$Q_{CONV} = h(T_s - T_a)$$

where:

h = convective heat transfer coefficient, in Btu/hr ft^2 ^oF

T_s = surface temperature, in ^oF

T_a = air temperature, in ^oF

The convective process is very complex, particularly at times when temperature conditions cause air stratification which affects the natural convection process (Miller [2.48]). Convection coefficients describing convective heat transfer primarily due to the movement of fluid across a surface of particular roughness characteristics are applicable when solar and long wave radiation are at moderate levels, creating neutrally stable air conditions. The convective coefficient for forced convection is affected by windspeed, surface roughness and orientation of the surface to the direction of air flow on the ground surface.

Duffie and Beckman [2.49] report the results of various researchers for estimating convective coefficients on horizontal plates. McAdams reports a convective coefficient of:

$$h = 5.7 + 3.8V$$

where:

h = convective coefficient, in W/m^2 ^{o}C

V = wind speed, in m/s

Vehrencamp [2.50] developed an empirical formula for a convective coefficient from data obtained on a dry packed lake bed. The coefficient is given by:

$$h = 122.93[0.00144T_m{}^{0.3}V^{0.7} + 0.00097(T_s - T_a)^{0.3}]$$

where:

h = convective coefficient, in Btu/hr ft^2 ^{o}F

T_s = surface temperature, in ^{o}C

T_a = air temperature, in ^{o}C

T_m = 273.0 + $(T_s + T_a)/2$, in ^{o}K

2.5 LOADING CONFIGURATIONS ON FLEXIBLE PAVEMENTS

2.5.1 INTRODUCTION

Typically in the U.S., legal load levels for roads have been established by states and the federal government. These load levels are designated by maximum allowable axle loads and maximum allowable load per inch width of tire. The majority of states imposes an 18 or 20 kip axle load limit. Allowable tire loads range from 450 to 800 lbs per inch width. A wide variety of tire sizes, typically ranging from 8 to 18 inch widths, a variety of tire configurations, single or dual, and multiple axle arrangements, create an extensive number of potential loading cases to be considered in a pavement analysis.

2.5.2 SINGLE AND DUAL TIRES

Mahoney [2.53] studied the response of flexible pavement to five single tire widths and 10 inch dual tires. The pavements studied ranged from 2 to 9 in. of asphalt concrete over an aggregate base. The analysis was performed using layered elastic theory programs. The analysis compared the damaging effects of the various tire sizes and configurations using the horizontal tensile strain as the fatigue criteria. Example results are shown in Table 2.10. The results are normalized with respect to the 10 inch dual tire configuration with an 18,000 lb. axle load. For the pavement cases considered, the single tires presented more damaging effects than dual tires for the same axle load. In a survey conducted in conjunction with this research, it was found that over 90 percent of the trucks had dual tires, with an average inflation pressure of 95 psi.

2.5.3 SINGLE AND MULTIPLE AXLES

The surface courses of pavement structures in Alaska are typically less than 3 inches. Base and subbase courses combined are typically about 12 inches thick. Johnson [2.54] studied the effects of multiple axle configurations on these relatively thin pavements. Falling Weight Deflectometer measurements on four pavement sections were taken. The resilient moduli for the four layers in the pavement structure were evaluated using reverse iterative techniques. The tensile strains for multiple axle loadings could then be evaluated for each pavement type. It was found that for average strength Alaskan pavements, multiple axles had damage factors twice as large as single axle configurations with the same load. The comparative damage factor was calculated as:

$$CDF = N_r/N_L$$

where:

N_r = number of 18 kip single axle dual wheel loads to failure on a standard pavement, and

N_L = number of multiple axle dual wheel loads with total axle group load TL on a given pavement.

Table 2.10 Traffic Equivalence Factors for Asphalt Concrete Pavement (after Mahoney, 1984)

Axle Load (lbs)	Dual Tire Width		Single Tire Width									
	10"		10"		12"		14"		16"		18"	
	sn=2	sn=6	sn=2	sn=6	sn=2	sn=6	sn=2	sn=6	sn=2	sn=6	sn=2	sn=6
10,000	0.35	0.17	1.24	0.30	0.89	0.27	0.64	0.24	0.50	0.22	0.40	0.20
18,000	1.00	1.00	3.52	1.76	2.45	1.58	1.82	1.41	1.41	1.28	1.21	1.15
20,000	1.21	1.37	4.25	2.42	2.96	2.16	2.19	1.94	1.69	1.75	1.35	1.58
30,000	2.47	4.64	8.71	8.17	6.06	4.49	4.49	6.56	3.48	5.92	2.77	5.35

Notes: sn=2 represents 2 to 4 inches of asphalt concrete over aggregate base.

sn=6 represents 9 inches or more of asphalt concrete over aggregate base.

From the results of this study, the following is obtained:

$$CDF_m = 3.5 \times 10^{-10} \left(\frac{TL}{n}\right)^{2.22} n$$

where:

n = number of equally loaded dual wheel axles.

Similar results were found in a study by Haven and Southgate [2.55] comparing trailers with tandem axles and three axles. In addition, it was found that the most damaging effects occurred when weight on the front steering axle was increased.

2.6 LITERATURE REVIEW SUMMARY

The literature reviewed showed a number of studies which attempted to quantify the loss in pavement strength during the spring thaw. A number of the field studies showed clearly the loss in bearing capacity during this period. Further, these same studies revealed that the primary loss in pavement strength occurs in the subgrade and unstabilized base courses. Laboratory studies have been conducted to simulate the freeze-thaw process and obtain the magnitude of strength loss for various subgrade soils. These laboratory studies compared reasonably well with field studies using deflection equipment.

Field and theoretical studies had determined the depth of freezing and the duration of the freezing period. These studies and models (or variations thereof) can be used to determine the rate of advance of the thawing front. This in turn can be used to estimate the length of the critical period starting from the onset of thawing to complete thaw.

The literature reviewed, however, is short on methods used to deal with the problem of spring thaw. There are few studies on methods used to determine the magnitude of spring load restriction. Of the studies that exist, none had been fully adopted by any local or state agency. Little literature existed on methods used to determine the length of the critical thaw period. Nothing was found also on enforcement of load restrictions where these have been applied.

CHAPTER 3.0
SURVEY OF CURRENT PRACTICE

3.1 INTRODUCTION

This chapter summarizes the results of contacts and visits with selected agencies throughout the United States and Canada. The purpose of the contacts was to assess the following:

(a) types of pavement failures associated with spring thaw,

(b) types of facilities requiring weight restriction during the spring thaw period,

(c) the intended purpose of weight restriction and how such policies were developed and implemented,

(d) cost benefit analysis of weight limit enforcement on a specific facility (if available data existed), and

(e) legal aspects of truck weights limits.

3.2 SURVEY INTERVIEW TECHNIQUES

To collect the needed information, three survey techniques were used and will be individually described.

3.2.1 INITIAL INFORMATION REQUEST

In November 1984, the request form given in Figure 3.1 was sent to 38 state agencies and Canadian provinces. This initial survey was used to identify those agencies which were then involved with load restrictions.

3.2.2 INTERVIEWS

Selected agencies with considerable experience with spring load restrictions were visited to obtain first hand their experiences with spring load restrictions. The form given in Appendix E was used by the project staff to collect the needed data.

INFORMATION REQUEST

1. ARE LOAD RESTRICTIONS PLACED ON ANY ROADS IN YOUR STATE DURING SPRING THAWING?

   Yes _______ No _______

2. HOW ARE LOAD RESTRICTIONS DETERMINED?

   ANALYSIS _______ EXPERIENCE _______ OTHER (describe briefly)

3. DOES THE STATE HAVE GUIDELINES OR LEGISLATIONS WHICH ADDRESS THIS ISSUE?

   Yes _______ No _______ (If yes, please enclose copy)

4. ARE THERE SPECIFIC DISTRICTS OR COUNTIES WITHIN YOUR STATE WHERE LOAD RESRICTIONS ARE IMPOSED?

   Yes _______ No _______ (If yes, can you identify these and possibly list a contact in these locations?)

ADDITIONAL COMMENTS, INFORMATION, PERTINENT REFERENCE MATERIAL REGARDING THIS SUBJECT WOULD BE GREATLY APPRECIATED.

Thank you very much for your time and assistance.

Return address:

Dr. Joe P. Mahoney
Department of Civil Engineering
121 More Hall, FX-10
University of Washington
Seattle, WA 98195

Figure 3.1 Initial Information Request Form

3.2.3 FOLLOW-UP REQUESTS

Some agencies were sent the interview form given in Appendix E to obtain information on their experiences with spring load restrictions (i.e. an on-site interview was not conducted). The results of the surveys are given in the following sections.

3.3 INITIAL INFORMATION REQUEST TO STATE DOT'S

Table 3.1 summarizes the results of the initial information request. The major findings include the following:

(a) Sixteen of the 33 states and four of the five Canadian provinces responding indicated they did impose load restrictions.

(b) Four of the states and three of the Canadian provinces indicated that their load restrictions were based on analysis. The remaining agencies established their load restriction policies on experience.

(c) Thirteen of the states and four of the Canadian provinces indicated their agency had guidelines and/or legislation establishing load restrictions.

Based on the results of this preliminary information request, the following state DOT's were selected for follow-up contact:

VISITS	FOLLOW-UP INFORMATION REQUEST
Iowa DOT	Alaska DOT/PF
Minnesota DOT	Idaho DOT
New Hampshire DOT	Maine DOT
Oregon DOT	Montana DOT
Washington DOT	North Dakota DOT
	Nova Scotia DOT
	South Dakota DOT

Table 3.1 Summary of Information Request to State and Province DOT's Regarding Current Load Restriction Practices

State or Province	Load Restrictions During Spring			How Load Restrictions are Determined		Does State have Guidelines or Legislation Establishing Spring Load Restrictions	
	Yes	No	No Reply	Analysis	Experience	Yes	No
Alaska	X			X		X	
California		X					
Colorado		X					
Connecticut		X					
Delaware		X					
Idaho	X			X		X	
Illinois	X				X	X	
Indiana	X				X	X	
Iowa	X				X	X	
Kansas		X					
Maine	X				X		X
Maryland		X					
Massachusetts		X					
Michigan	X			X	X	X	
Minnesota	X			X	X	X	
Missouri		X					
Montana	X				X	X	
Nebraska		X					
New Hampshire	X				X	X	
New Jersey		X					
New Mexico		X					
New York		X					
North Dakota	X				X	X	
Ohio		X					
Oregon	X				X	X	
Pennsylvania			X				
Rhode Island		X					
South Dakota	X				X	X	
Texas		X					
Vermont	X				X		X
Washington	X			X	X	X	
Wisconsin			X				
Wyoming	X				X		X
Alberta	X			X	X	X	
New Brunswick	X			X		X	
Nova Scotia	X			X		X	
Ontario			X				
Saskatchewan	X				X	X	

3.4 RESULTS OF INTERVIEWS AND FOLLOW-UP REQUESTS

Detailed information on load restrictions was solicited from the agencies identified above. Personal interviews were conducted in five states with a total of twelve agencies (Table 3.2). Follow-up questionnaires were obtained from six states and one Canadian Province (Table 3.3). This section describes the results of this effort.

Each agency was asked questions dealing with:

(a) development of load restrictions,
(b) types of highways receiving load restrictions,
(c) design information for roads receiving load restrictions,
(d) criteria for imposing load restrictions, and
(e) enforcement methods.

The detailed interview form is given in Appendix E. Responses to each of the above topic areas are summarized in Tables 3.4 through 3.8.

3.4.1 DEVELOPMENT OF GUIDELINES

Specific questions dealing with (a) types of pavement failure associated with spring thaw, (b) extent of the problems, and (c) procedures used for determining locations for load restrictions were asked of all agencies (state, county, and city). The results given in Table 3.4 indicate:

(a) The predominant types of pavement failure included alligator cracking, rutting, frost boils, and potholes.

(b) The extent of the problem varied from very little to agency-wide, and predominantly on low volume roads.

(c) The locations for load restrictions were based on past experience and/or surface deflection. For some of the smaller agencies, the restrictions were placed on all roads.

Table 3.2 Agencies Interviewed

State	Agency	Contact
Iowa	Department of Transportation Ames, IA	Charles L. Huisman
Minnesota	Department of Transportation	George Cochoran
	City of Maple Grove Maple Grove, MN	Gerald E. Butcher
	Wright County Buffalo, MN	W. Fingalson
	Anoka County Anoka, MN	Paul Roode
New Hampshire	Dept. of Public Works and Hwy Lebanon, NH	Dick Heath
	CRREL Hanover, NH	T. Johnson
Oregon	Department of Transportation Salem, OR	John Sheldrake
	Benton County Corvallis, OR	James Blair
Washington	Department of Transportation Olympia, WA	N. Jackson
	Benton County Prosser, WA	J. McAuliff

Table 3.3 Follow-up Requests

State	Agency	Contact
Alaska	Department of Transportation and Public Facilities Fairbanks, AK	Dave Esch
Idaho	Department of Transportation Boise, ID	James W. Hill
Maine	Department of Transportation Augusta, ME	Richard Schofield
Montana	Department of Highways Helena, MT	Richard Wegner
North Dakota	Highway Department Bismark, ND	Stanley Haas
Nova Scotia	Department of Transportation Halifax, NS	D.C. Pugsley
South Dokata	Department of Transportation Pierre, SD	James R. Anton

Table 3.4 Development of Guidelines for Spring Load Restrictions

Location	Types of Pavement Failure Associated with Spring Thaw	Extent of Problem	How are Locations for Load Restrictions Determined?
Alaska DOT	Alligator cracking, rutting, frost boils	Statewide	FWD, visual observations, measurements of thaw depth, experience
Idaho DOT	Foundation, deep base,surface	15% of system	Experience
Iowa DOT	Spring breakup	Low volume roads	Selected by district engineers
Bremer County, Iowa	Pavement breakup, rutting	Up to 50% on aggregate surfaced, up to 10% on paved	Visual observation of heaving and/or pumping
Maine DOT	Alligator cracking	Low volume roads statewide	Selected by district engineers
Minnesota DOT	Rutting, alligator cracking	Limited	Experience of maintenance engineer and deflection measurements with road rater and FWD
Anoka County, Minnesota	Alligator cracking, potholes	Not too extensive due to restrictions	Construction history and design, and Benkelman beam deflections
Maple Grove, Minnesota	Frost boils, alligator cracking	City wide	Uniform load restriction policy for all streets

Table 3.4 Development of Guidelines for Spring Load Restrictions (Cont.)

Location	Types of Pavement Failure Associated with Spring Thaw	Extent of Problem	How are Locations for Load Restrictions Determined ?
Wright County, Minnesota	Rutting, alligator cracking	Variable from year to year	Road Rater deflections
Montana DOT	Frost boils	Statewide on minimum structure roads	Judgment of maintenance personnel
New Hampshire DOT, Div 2	Alligator cracking, rutting, frost heave	Modest	Judgment of maintenance personnel based on whether heavy hauling is occurring
North Dakota DOT	Surface breaks, potholes	Varies yearly depending on frost penetration	Experience
Nova Scotia DOT	Varies depending on structure and loads	Not extensive	Benkelman beam testing
Oregon DOT	Heave, cracking, pavement breakup	Central, eastern part of state	Experience and visual observation
Benton County, Oregon	Alligator cracking and breakup	All road construction types	Experience
South Dakota DOT	Potholes, edge failure, alligator cracking	Highways with thin mats typically restricted statewide	Experience

Table 3.4 Development of Guidelines for Spring Load Restrictions (Cont.)

Location	Types of Pavement Failure Associated with Spring Thaw	Extent of Problem	How are Locations for Load Restrictions Determined
Washington State DOT	Alligator cracking, pavement breakup	Central and Eastern Washington on a few low volume roads	Judgment of maintenance personnel
Benton County, Washington	Pavement breakup, frost heave, base failure	Moderate	Observation of road conditions

Table 3.5 Description of Highways to Which Load Restrictions are Applied

Location	Functional Class	ADT % Trucks	Soil Types	Surface Types	Typical Cross Section
Alaska DOT	All	All	Frost susceptible	ACP	1½ - 2" ACP 4" - 6" Base Select Varies Subgrade
Idaho DOT	All	-	-	-	-
Iowa DOT	Secondary, arterials and collectors	<1000 <10%	Clay and silt	ACP or BST	1" - 5" Surface 6" - 8" Base Subgrade
Bremer County, Iowa	All	<200, 5% >200, 5-10%	Heavy black clay	ACP or PCC	3" - 6" Surface 1" - 8" Base Subgrade (clay) 1" = 3" Aggregate Subgrade (clay)
Maine DOT	Collector, local, light duty secondary	50 - 2,500 % Variable	Clay and till	Seal and maintenance mixes	-

Table 3.5 Description of Highways to Which Load Restriction are Applied (Cont.)

Location	Functional Class	ADT % Trucks	Soil Types	Surface Types	Typical Cross Section
Minnesota DOT	All	All	All but granular	Bituminous w/aggregate base	3" Surface 6" - 12" Base Subgrade
Anoka County, Minnesota	All	300 - 30,000 5 - 10%	SM, CH	BST, ACP	1" - 2" BST 6" Base Subgrade 4" ACP 3 - 5" Black Base 4 - 5" Aggregate Subgrade
Maple Grove, Minnesota	All city streets	400 - 1000 8 - 10%	Clay (A-6), some gravel	ACP and gravel	3" - 6" ACP 6 - 12" Base Subgrade 8 - 12" Gravel Subgrade
Wright County, Minnesota	All	200 - 16,000	A-6, range from gravel to clay	ACP primarily	2 - 2½" ACP 3 - 4" Base Granular Subgrade 3" ACP 15 - 18" Base Clay Subgrade
Montana DOT	Primary and secondary	-	-	-	-
New Hampshire DOT Div. 2	Secondary	20 - 1000	Glacial till	BST and ACP	4" Surface 6 - 8" Base Subgrade

Table 3.5 Description of Highways to Which Load Restrictions are Applied (Cont.)

Location	Functional Class	ADT % Trucks	Soil Types	Surface Types	Typical Cross Section
North Dakota DOT	All except Interstate	-	-	-	-
Nova Scotia Dot	-	-	GM, SM	ACP	2½" ACP 10" Base Subgrade
Oregon DOT	Secondary	<500 5 - 10%	Clays and pavement w/no base rock	Oil mats, thin ACP	2 - 4" ACP < 10" Base Subgrade
Benton County, Oregon	Collector, minor arterial	200 - 4000 5 - 10%	Clay	Macadam, ACP	-
South Dakota DOT	All except Interstate	Variable	All except rock	Thin mats primarily	-

Table 3.5 Description of Highways to Which Load Restrictions are Applied (Cont.)

Location	Functional Class	ADT % Trucks	Soil Types	Surface Types	Typical Cross Section
Washington State DOT	Secondary	< 1000	SM	Thin bituminous	1.5 - 2" Bituminous 6" Base Subgrade
Benton County, Washington	All	-	-	-	-

Table 3.6 Design Information for Roads Restricted During Spring Thawing

Location	Use of Frost Protection in Thickness Design	Are Load Restrictions Used in Lieu of Frost Protection?	Thickness Design Method Used	Age of Pavement Restricted	Drainage Conditions
Alaska DOT	More than 50% but not full	Sometimes	Alaska procedure	-	Fair
Idaho DOT	Frost protection not included in design	-	AASHTO, Hveem	5-10 years	Fair to poor
Iowa DOT	Less than 50% frost protection	-	AASHTO	Pre-WWII	Good to poor
Bremer County, Iowa	Less than 50% frost protection	-	Experience, nominal thickness	Up to 20 years	Good to excellent
Maine DOT	More than 50% but less than full protection	-	AASHTO, MDOT	10 to 20 years	Poor
Minnesota DOT	Variable	Used on old roads which have not been replaced	Minn DOT (flexible pavements)	-	Good to poor
Anoka County, Minnesota	No	-	Minn DOT	15 to 20 years	Good
Maple Grove, Minnesota	No	Yes	Hveem, Minn DOT	7 years ±	Fair

Table 3.6 Design Information for Roads Restricted During Spring Thawing (Cont.)

Location	Use of Frost Protection in Thickness Design	Are Load Restrictions Used in Lieu of Frost Protection?	Thickness Design Method Used	Age of Pavement Restricted	Drainage Conditions
Wright County, Minnesota	No	-	Minn DOT, Asphalt Inst. MS-1	15 to 20 years	Fair to poor
Montana DOT	No	-	AASHTO	-	-
New Hampshire DOT Div. 2	No	-	None used for secondary roads	Very old	Poor
North Dakota DOT	No	Yes	Stage construction	20 years	Good
Nova Scotia DOT	No	-	RIAC	10 years	Poor
Oregon DOT	More than 50% but not full protection	-	Hveem	20 years +	Poor
Benton County, Oregon	No	-	Hveem	-	Fair
South Dakota DOT	No	Yes	AASHTO	-	Good to Poor
Washington State DOT	Depth $\geq$ 50% of frost depth	-	WSDOT	15 years +	Fair
Benton County, Washington	Full protection	-	Standard section	10 to 15 years	Fair

Table 3.7 Load Restriction Criteria

Location	Normal Load Limits Single Axle, Tandem Axle	Spring Load Limits	How are Spring Loads Limits Established?	Basis for Initiation of Load Restriction	Basis for Removal of Load Restriction	Is Deflection Measuring Equipment Used to Establish Load Restrictions?
Alaska DOT	20K, 34K	50 to 75% of normal	Experience, studies	One foot thaw and increasing deflection	Regain strength, political pressure	Yes (FWD)
Idaho DOT	20K 34-37.8K	14K - 20K 28K - 37.8K	Experience	Judgment	Judgment	No
Iowa DOT	20K, 34K	-	Studies	Judgment	Judgment	No
Bremer County, Iowa	20K, 34K	10K/Axle	Experience	Presence of water or signs of distress	When unpaved roads dry	No
Maine DOT	22K, 34K	Gross weight 23K	Experience	Soft weather in winter and spring	Clear frost guage and visual inspection of roads	No
Minnesota DOT	20K, 34K	10K - 14K 18.9 - 26.4K	Experience, studies	Thaw depth, weather forecast	Experience, deflection measurements	Yes (FWD)
Anoka County, Minnesota	20K, 34K	10K - 14K 18.9 - 26.4K	Experience, testing	Increasing Benkelman beam deflection	Allowable loads increase w/time, Benkelman beam deflection	Yes (Benkelman beam)

Table 3.7 Load Restriction Criteria (Cont.)

Location	Normal Load Limits Single Axle, Tandem Axle	Spring Load Limits	How are Spring Load Limits Established?	Basis for Initialtion of Load Restriction	Basis for Removal of Load Restriction	Is Deflection Measuring Equipment Used to Establish Load Restrictions?
Maple Grove, Minnesota	18K, 34K	10K, 20K	Follows state guidelines	State restriction periods or when moisture appears in pavement cracks and joints	State guides or visual observation of pavement drying	No
Wright County, Minnesota	18K, 34K	10K - 14K	Studies by Minn DOT	Observations of pumping	Examination of frost tubes, practice of surrounding counties	No
Montana DOT	20K, 34K		Experience	When subgrade begins to lose strength	When subgrade has stabilized	No
New Hampshire DOT Div. 2	20K, 34K	300 lb/in width of tire	Experience	"Mud Season"	Observe moisture conditions	No
North Dakota DOT	20K, 34K	12K, 24K	Experience	Experience	Experience	No
Nova Scotia DOT	9,000 KG, 17,000 KG	6,500 KG, 12,000 KG	Experience	Benkelman Beam deflection measurements	Benkelman Beam deflection measurements	Yes (Benkelman Beam)

Table 3.7 Load Restriction Criteria (Cont.)

Location	Normal Load Limits Single Axle, Tandem Axle	Spring Load Limits	How are Spring Load Limits Established?	Basis for Initiation of Load Restriction	Basis for Removal of Load Restriction	Is Deflection Measuring Equipment Used to Establish Load Restrictions?
Oregon DOT	20K, 34K	8 - 10 tons gross	Experience	When breakup begins	Not well defined	No
Benton County, Oregon	-	-	-	-	-	-
South Dakota DOT	20K, 34K	12K - 14K 24K - 28K	Experience	When thawing begins - not before 2/15	When roadbed is dry and solid, not later than 5/1	No
Washington State DOT	20K, 34K	Based on tire size	Experience, research	Judgment	Judgment	No
Benton County, Washington	-	Based on tire size	Experience	Observation	Observation	No

Table 3.8 Enforcement Methods for Spring Load Restrictions

Location	How Are Load Restrictions Enforced?	How Are Vehicle Operations Notified?	Can Overweight Permits be Obtained?	What Enforcement Methods Are Used?	Are Fines Levied?
Alaska DOT	Fixed scale installation	Newspapers, road signing	Yes	Scale crossing	Yes $0.05/lb
Idaho DOT	Portable scale	Mail	No	All trucks stopped at scale	Yes cost per 1,000 lb
Iowa DOT	Fixed portable scale and patrol	Detour and embargo maps	Yes	Patrol	Yes cost per 1,000 lb
Bremer County, Iowa	Fixed scale, patrol	Detour and embargo maps	Yes	Patrol	Yes cost per 1,000 lb
Maine DOT	-	Roads posted	Yes	-	Yes
Minnesota DOT	Fixed and portable scale, relevant evidence law	News, mail, road signing	Yes	All trucks stopped at scale	Yes lb
Anoka County, Minnesota	Portable scale	Post roads	Yes	Observation of vehicles	Yes cost per 1,000 lb
Maple Grove, Minnesota	Portable scale	Newspapers, road signing	Yes	Patrol	Yes cost per 1,000 lb
Wright County, Minnesota	Portable scale	Newspapers, radio, post roads	Yes school busses only	Selective sample	Yes cost per 1,000 lb

Table 3.8 Enforcement Methods for Spring Load Restrictions (Cont.)

Location	How are Load Restrictions Enforced?	How are Vehicle Operators Notified?	Can Overweight Permits be Obtained?	What Enforcement Methods are Used?	Are Fines Levied?
Montana DOT	Fixed, portable scale	Newspapers, radio, news, roads posted	No	All trucks checked at random locations	Yes
New Hampshire DOT Div. 2	Portable scale	News releases, post roads	Yes	Selective sample	-
North Dakota DOT	-	-	-	-	-
Nova Scotia DOT	Fixed, portable scale	News, notices	No	Stop trucks at scale	Yes cost per 1,000 lb
Oregon DOT	Fixed, portable scale	Roads signs, media	Yes	Selective sample	No
Benton County, Oregon	Portable scales	Road signs, newspapers, notices	No	Stop all trucks	Yes cost per 1,000 lb
South Dakota DOT	Fixed and portable scales	Road signs, notices mailed	No	Stop all trucks	Yes cost per 1,000 lb
Washington State DOT	Portable scales	Road signs, newspapers	Yes	Selective sample	Yes
Benton County, Washington	-	Post roads	-	-	Yes cost per 1,000 lb

3.4.2 HIGHWAYS RECEIVING LOAD RESTRICTIONS

This question was concerned with defining the types of highways receiving load restrictions. Specifically, it addressed:

(a) What functional class of highway receives load restrictions?

(b) What are typical values for ADT and percent trucks for these highways?

(c) What soil types are found beneath these highways?

(d) What surface types receive load restrictions?

(e) What are typical cross sections for the roadways receiving load restrictions?

The responses to these questions given in Table 3.5 generally indicate the following:

(a) Load restrictions by state agencies were applied to both primary and secondary roads but mostly secondary. Few states have applied them to Interstate facilities. Local agencies generally applied load restrictions to all types of facilities.

(b) Of those states responding, load restrictions were generally applied to roads with ADT less than 2500 and 10 percent trucks or less. Local city and county agencies applied restrictions to roads with ADT's up to 30,000 and up to 10 percent trucks.

(c) Primarily, load restrictions were applied to pavements which had moisture susceptible silt or clay subgrades. If the agencies had granular subgrades, load restrictions were not usually required.

(d) Load restrictions (if used) were normally applied to aggregate and/or asphalt surfaced roads. Most portland cement concrete pavements reportedly had adequate structure to withstand the critical thaw period.

(e) The pavement cross sections to which load restrictions were applied generally ranged as follows:

	Range	Normal
Asphalt surface, in	1-½ - 5	2 - 4
Aggregate base, in	4 - 18	6 - 12

Thicker pavements apparently have sufficient strength to overcome the thaw weakening period.

3.4.3 DESIGN INFORMATION FOR ROADS RECEIVING LOAD RESTRICTIONS

This question dealt with design information such as:

(a) Is frost protection considered in thickness design?

(b) Are load restrictions used in lieu of full frost protection?

(c) What is the age of pavements receiving load restrictions?

(d) What are the typical drainage conditions of pavements receiving load restrictions?

Responses to these questions are given in Table 3.6. The results indicate:

(a) Some of the state agencies surveyed design pavements for partial frost protection while others did not consider frost protection in design at all. Most local agencies did not consider frost protection in their design procedure.

(b) Several of the agencies interviewed used load restrictions in lieu of designing for full frost protection.

(c) A variety of thickness design procedures were used to determine layer thickness. The most common was the AASHTO method. Others included the Hveem method, experience and/or precedent.

(d) The age of pavements receiving load restrictions tended to be 10 to 20 years or older. In some cases they tended to be farm-to-market kinds of roads constructed just after World War II.

(e) Drainage conditions for pavements receiving load restrictions varied from poor to good. There appeared to be little relation between surface drainage and the need for load restrictions.

3.4.4 LOAD RESTRICTION CRITERIA

This question dealt with:

(a) the current load limits (normal vs spring),

(b) methods used to establish load limits,

(c) the basis for initiating and/or removing of the load restriction, and

(d) whether deflection measuring equipment have been used to establish load restrictions.

Table 3.7 is used to summarize the results. The significant findings include:

(a) For most agencies normal load limits were 18,000 to 20,000 lbs on a single axle and 34,000 lbs on tandem axles.

(b) Spring load restrictions generally ranged from 10,000 to 14,000 lbs for single axles and 18,000 to 28,000 lbs for tandem axles.

(c) Percentage reductions were 30 to 50 percent for single axles and 18 to 47 percent for tandem axles.

(d) Most load limits had been established from experience. Only a few agencies such as Alaska [3.1], Minnesota [3.2] and Washington DOT [3.3] had conducted extensive studies. Much of this information has already been discussed in the literature review (Chapter 2.0).

(e) The basis for starting the load restriction varied from experience (presence of water coming through cracks/joints or pumping) to the use of deflection measurements. By far the majority of the agencies relied on the judgment (or experience) of field personnel.

(f) Load restrictions were removed based on the judgment of field personnel, deflection measurements, or when sufficient political pressure mounted. Most agencies, however, relied on judgment or past experiences.

(g) Only three of the agencies interviewed used deflection measurements to establish load limits.

3.4.5 ENFORCEMENT METHODS

The next question dealt with enforcement methods for spring load restrictions. Specifically, it requested information to questions such as:

(a) how load restrictions are enforced,
(b) how vehicle operators are notified,
(c) are overweight permits available,
(d) what enforcement methods are used, and
(e) are fines levied, and if so, what are they?

Table 3.8 summarizes the responses to these questions. In general, the following impressions are noted:

(a) Both fixed and portable weigh scales were used. Some agencies relied only on patrols.

(b) Methods used to notify vehicle operators of the load restrictions included:

  (i) newspapers and news releases,
  (ii) road signs,
  (iii) detour and embargo maps,
  (iv) radio and television.

(c) Most of the agencies used overweight permits. Some agencies had exceptions to the load limits (e.g., school buses and/or emergency situations).

(d) Enforcement methods used included patrol (by police) or weighing trucks (all or a selective sample).

(e) Fines were levied by almost all agencies. The fine was normally assessed as a cost per 1000 lb.

3.4.6 LEGAL ASPECTS

The last question dealt with legal aspects of load restrictions. Specifically, the requested information related to:

(a) the availability of local regulations addressing load restrictions,
(b) enforcement problems with the use of load restrictions, and
(c) legal problems associated with load restrictions.

Table 3.9 summarizes the results of this question. The significant findings are discussed below:

(a) All agencies had regulations allowing them to initiate and enforce load restrictions.

(b) The major problems with enforcement included:
   (i) lack of personnel to adequately enforce the load restriction,
   (ii) political pressure to allow truck operations, and
   (iii) evasive tactics of truckers.

(c) Most agencies had not experienced legal action as a result of enforcing load limits.

3.5 EVALUATION OF SURVEY RESULTS

The survey of agencies with load restrictions provided significant information in several areas including:

(a) types of load restrictions currently used,
(b) basis for load limits,
(c) criteria used to initiate and remove load restrictions,
(d) unique capabilities of local agencies, and
(e) requirements and problems associated with enforcement.

Each of these issues are discussed in the following sections.

Table 3.9 Legal Aspects of Load Restrictions

Agency	Local Regulations	Problems with Enforcement	Legal Problems with Restrictions
Alaska DOT	Yes	Lack of personnel	None
Idaho DOT	Yes	None	None
Iowa DOT	Yes	Lack of personnel, political pressure	None
Iowa (Bremer County)	Yes	None	None
Maine DOT	Yes	Lack of personnel	None
Minnesota DOT	Yes	None	None
Minnesota (City of Maple Grove)	Yes	Illegal loads moved during the night	Yes (on specific violation)
Minnesota (Wright County)	Yes	Political pressure	None
Minnesota (Anoka County)	Yes	Agricultural loads	None
Montana DOT	Yes	Complaints from truckers	None

Table 3.9 Legal Aspects of Load Restrictions (Cont.)

Agency	Local Regulations	Problems with Enforcement	Legal Problems with Restrictions
New Hampshire DOT	Yes	Lack of compliance by truckers	-
North Dakota DOT	-	-	-
Nova Scotia DOT	Yes	None	Yes (occasional court case)
Oregon DOT	Yes	Complaints from operators	None
Oregon (Benton County)	Yes	Lack of communication with truckers, reduction of penalties by court	None
South Dakota DOT	Yes	Difficulties in weighing with portable scales	Yes (evidence using portable) scales not accepted)
Washington State DOT	Yes	None	None
Washington (Benton County)	Yes	Lack of personnel	None

3.5.1 TYPES OF LOAD RESTRICTIONS

Most agencies interviewed restricted loads on a per axle basis. Limits differed between single and tandem axles, but not with tire size (conventional vs. flotational). The load reductions were a maximum of 60 percent for single axles and 60 percent for tandem axles.

3.5.2 BASIS FOR LOAD LIMITS

Current limits were established primarily on the basis of prior experience. Only the Alaska, Minnesota and Washington DOT's reported that they used research studies to establish or verify their load limits. There appears to be a definite need to develop a more rational approach to establish load limits.

3.5.3 CRITERIA USED TO INITIATE AND REMOVE LOAD LIMITS

Most agencies surveyed indicated that they initiated limits based on judgment. This could range from evidence of water at the surface (indicating a saturated base) or signs of cracking (which is too late). Other agencies simply relied on an established date. Few agencies used deflection or weather data to establish a starting date for load limits. Clearly, there is a need for an improved method of establishing this date.

Removal of load limits was also generally based on experience. Use of deflection measurements could greatly aid in this process and should be encouraged.

3.5.4 CAPABILITIES OF LOCAL AGENCIES TO MEASURE DEFLECTIONS

Most local agencies currently do not have the equipment or personnel to measure surface deflections. Unless this changes, it would be impractical to recommend use of deflections to establish the initiation and removal of the load limits.

Personnel used to establish these critical periods have often been from the maintenance department and would have to be trained in the use and interpretation of deflection data.

3.5.5 REQUIREMENTS AND PROBLEMS WITH ENFORCEMENT

Enforcement is usually accomplished by the county sheriff or city police. Special training is not usually required to enforce load limits.

The major problem to be overcome with enforcement is to develop a proper data base to resist political pressures to waive the limits. If the amount of damage done to the roads during the critical spring period and the associated cost of early wearout could be shown, the political problems of load limits could be minimized. The development of a visual aids package to assist local engineers in this effort would be of great value. Such a package has been developed as part of this study.

CHAPTER 4.0
ANALYSIS

4.1 INTRODUCTION

At the onset of this study methods were sought in existing practice for evaluating load restrictions and timing. A survey of current practice and interviews suggested that only organizations having access to deflection testing equipment (typically Benkelman Beam or FWD) were doing investigations more rigorous than relying on experience and observation of distress. It was decided, therefore, to undertake an extensive analysis program to try to establish some guidelines for spring load restrictions.

This study addresses two distinct issues which will be treated separately in the analysis. They are:

(a) What magnitude of load restrictions should be imposed during the critical spring thawing period?

(b) When should load restrictions be imposed and removed?

To evaluate the load restriction magnitude, several cases of structure and load were evaluated in a pavement structural analysis. The results of the this analysis suggest when (with respect to the position of the thawing front) the pavement structure is experiencing strains or deflections in excess of those experienced in the summer reference case. The thermal analysis suggests the actual time that the thawing has proceeded to the "critical" levels as suggested by the structural analysis.

4.2 LOAD LIMITS

4.2.1 APPROACH

4.2.1.1 INTRODUCTION

The development of guidelines for the magnitude of load restrictions during spring thawing requires the following:

(a) method of analysis,
(b) pavement structure composition,

(c) loads to be analyzed, and

(d) a basis for identifying a "critical" spring condition.

4.2.1.2 ANALYTICAL PROCEDURE

Layered elastic theory has been widely applied to analyze pavement response to load. Several analysis programs exist for mainframe and micro computers. The program selected for this study was ELSYM5. This program was developed at the University of California, Berkeley, and can be used to analyze up to ten identical loads in a five layer system. It computes stresses, strains and displacements at specified points. The program assumes the material behavior is linear elastic.

It has been widely recognized that base course and subgrade materials (both coarse and fine) exhibit nonlinear elastic behavior. Since test cases are "hypothetical," representing a range of structural conditions that might be found anywhere in the frost areas of the U.S., it was not possible to identify any meaningful nonlinear relationships. In addition, in reviewing data from previous frost studies performed for the Washington State DOT [3.3], it was found that the behavior of the materials was not highly non-linear in the ranges studied. Therefore, it is felt that a linear elastic analysis is capable of providing adequate results.

4.2.1.3 LOADING CASES

Currently, most jurisdictions, whether national, state or local, restrict loads on classes of roads according to axle loads. Based on information obtained in the interviews and a review of current practice throughout the U.S., a maximum single axle load of 20,000 lb. and a tandem axle load of 34,000 lb. were selected as reference load levels.

The ELSYM5 program models the applied loads as wheel loads with a circular configuration. It was decided by the study team that the loading was most accurately represented by selecting the maximum load and corresponding tire pressure recommended by the Tire and Rim Association for a particular tire size. Load reductions would be modelled by <u>maintaining the contact</u>

pressure (tire pressure) and reducing the load, thereby reducing the contact area.

Several loading cases were evaluated including:

Single Axles		Tandem Axles	
Dual	10 - 22.5 tires	Dual	10 - 22.5 tires
Single	16.5 - 22.5 tires		

The loads and pressures for each of these cases are shown in Table 4.1. All loading cases were analyzed for 20 and 100 percent of the maximum load to obtain load-deflection and load-strain plots.

4.2.1.4 STRUCTURE CROSS SECTION

The structure cross sections used in the study were selected to represent as well as possible the types of road construction and subgrade materials existing in the geographic region and jurisdictions of interest. Therefore, the data obtained in the interviews (such as Table 3.5 in Chapter 3.0) were weighted heavily in the selection of the structure cross section cases.

Surface courses were assumed to be either asphalt concrete (AC) or bituminous surface treatment (BST) with thicknesses ranging from two to four inches. The base course was assumed to be unstabilized aggregate varying from six to twelve inches thick. No subbases were considered. Subgrades of both coarse and fine materials were investigated. The specific cases analyzed are shown in Table 4.2.

4.2.1.4 (a) MATERIAL PROPERTIES

Several different cases of environmental conditions occur in a pavement structure annually which have an effect on the pavement structure's stiffness properties and therefore, its response. If it is desirable to restrict loads during spring when overall structural stiffness is reduced so that the strains and deflections experienced are comparable to those during the "full strength" summer case, then the stiffness properties of the summer case and various stages of spring thawing need to be modelled.

Table 4.1 Loading Cases

Case	Size (Nomial)	Tire Pressure (psi)	Tire Load (lbs)
Single Axle (Max. Load = 20,000 lb)			
a) Single Tires	16.5 - 22.5	90	9900
b) Dual Tires	10 - 22.5	100	5000
Tandem Axle (Max. Load = 34,000 lb Axle Spacing = 48")			
a) Dual Tires	10 - 22.5	100	4250

Notes: a) All tire/axle combinations will be analyzed for 20% and 100% of the maximum allowable load. The maximum allowable load was computed using 600 lb/in width of tire of the maximum axle load, whichever is the limiting criteria

b) Tire pressures used for all cases were the maximum recommended by the Tire and Rim Association. The contract area was adjusted to give the correct load.

Table 4.2 Summer Pavement Structure

Type	Material	Thickness (in.)	Resilient Modulus (psi)
Surface	BST or ACP	2	300,000
	ACP	4	300,000
Base	Gravel	6	Base M_R = 1.5 subgrade M_R
		12	
Subgrade	Fine-grained	212	7,500
	Coarse-grained	212	40,000

For the reference condition a range of resilient properties were selected to represent the surface course, base course and subgrade. The analysis performed assumed that for the condition of a base course underlain by a weaker material, the base course resilient modulus was a function of the underlying material. The following relationship was used:

$$Mr_{base} = 1.5\ Mr_{subgrade}$$

This type of relationship was originally used by Henkelom and Klomp [4.5], has been subsequently used by the Shell Oil Company [4.6] and by the Asphalt Institute [4.1] in their respective pavement design methods. The commonly used range for the modular ratio is about 1.0 to 4.0 (for this study a value of 1.5 was selected, which is in the lower end of the range).

A range of subgrade resilient moduli were selected from results of field and laboratory data and are shown in Tables 4.2 through 4.5. The values represent typical moduli for soils ranging from silty-clay to gravel [4.1, 4.7, 3.3].

The asphalt concrete and bituminous surface treatment resilient moduli are highly dependent on temperature. The resilient modulus selected for the summer case was 300,000 psi and was based on a reference temperature of 75°F [4.6]. Based on the same reference, the surface course resilient modulus during the spring thaw (temperature of 40°F) was chosen to be 1,200,000 psi.

During the early thawing period, the base course resilient modulus can be reduced substantially due to moisture conditions and undrained loading. The base course assumed during this period was either 25 or 50 percent of the reference (summer) condition. This decision was based in part on work reported by Lary, et al. [3.3], and Shook, et al. [4.1].

When thawing occurred in the subgrade, the $Mr_{subgrade}$ was assumed to be 5 to 50 percent of the reference (summer) condition. For cases where the subgrade material was frozen, the resilient modulus was assumed to be 50,000 psi.

Table 4.3 Spring Thaw Pavement Structure (Complete Thaw)

Type	Material	Thickness (in.)	Resilient Modulus (psi)
Surface	BST or ACP	2	1,200,000
	ACP	4	1,200,000
Base	Gravel	6 12	Base M_R = 1.5 Subgrade M_R
Subgrade	Fine-grain	-	15,20,25% of Summer Subgrade M_R
	Coarse-grain	-	25,30,50% of Summer Subgrade M_R

Table 4.4 Spring Thaw Pavement Structure
(Thaw to Bottom of Base)

Type	Material		Thickness (in.)	Resilient Modulus (psi)
Surface	BST or ACP		2	1,200,000
	ACP		4	1,200,000
Base	Gravel		6	25,50% of Summer Base M_R
			12	
Subgrade	Frozen		Depth of freeze minus surface, base and thawed subgrade	50,000
	Unfrozen	Fine-grain	212	7,500
		Coarse-grain	212	40,000

Table 4.5 Spring Thaw Pavement Structure (Thaw to 4 in. Below Base)

Type	Material	Thickness (in.)	Resilient Modulus (psi)
Surface	BST or ACP	2	1,200,000
	ACP	4	1,200,000
Base	Gravel	6 12	Base M_R=1.5 subgrade M_R
Subgrade	Thawed fine-grain	4	5,15% of summer subgrade M_R
	Thawed coarse-grain	4	25,50% of Summer Subgrade M_R
	Frozen fine-grain and coarse-grain	Depth of freeze minus surface, base and thawed subgrade	50,000

The cases which were analyzed during thawing included the following:

(a) thaw to the bottom of the base course,
(b) thaw four inches into the subgrade, and
(c) thawing complete.

4.2.1.5 PARAMETERS CALCULATED

When a pavement fatigue analysis is performed, two strain parameters are used. These parameters are the tensile strain at the bottom of the surface course (ε_t) and the vertical strain at the top of the subgrade (ε_{vs}). Another parameter typically considered as well is the maximum pavement surface deflection. In addition to these widely used damage indicators some researchers (Stubstad and Conner [2.21] and Lary, et al. [2.22]) have found that the vertical strain at the top of the base course (ε_{vb}) was also an indicator of distress due to a weakened condition. As a result, for this study, all of these parameters were considered as potential indicators of excessive load. Therefore, an increase in any one of these parameters above the reference level (summer condition) constituted a required reduction in the load level sufficient to maintain these parameters at levels comparable to the reference (or summer) conditions. The locations of these parameters are shown in Figure 4.1.

Once the ELSYM5 deflections and strains were calculated, the determination of the spring load which caused the same damage as the maximum legal allowable load during the summer could be computed. This can be illustrated using a plot such as the one shown in Figure 4.2. The plot was constructed as follows:

(a) Surface deflection (δ), ε_t, ε_{vb}, and ε_{vs} were plotted for two loads used in the spring analysis (hence spring thaw material properties), and load-deflection and load-strain lines were drawn through these points. The load levels used in the analysis were 20 and 100 percent of the legal maximum.

(b) This was done for different structural profiles and material combinations.

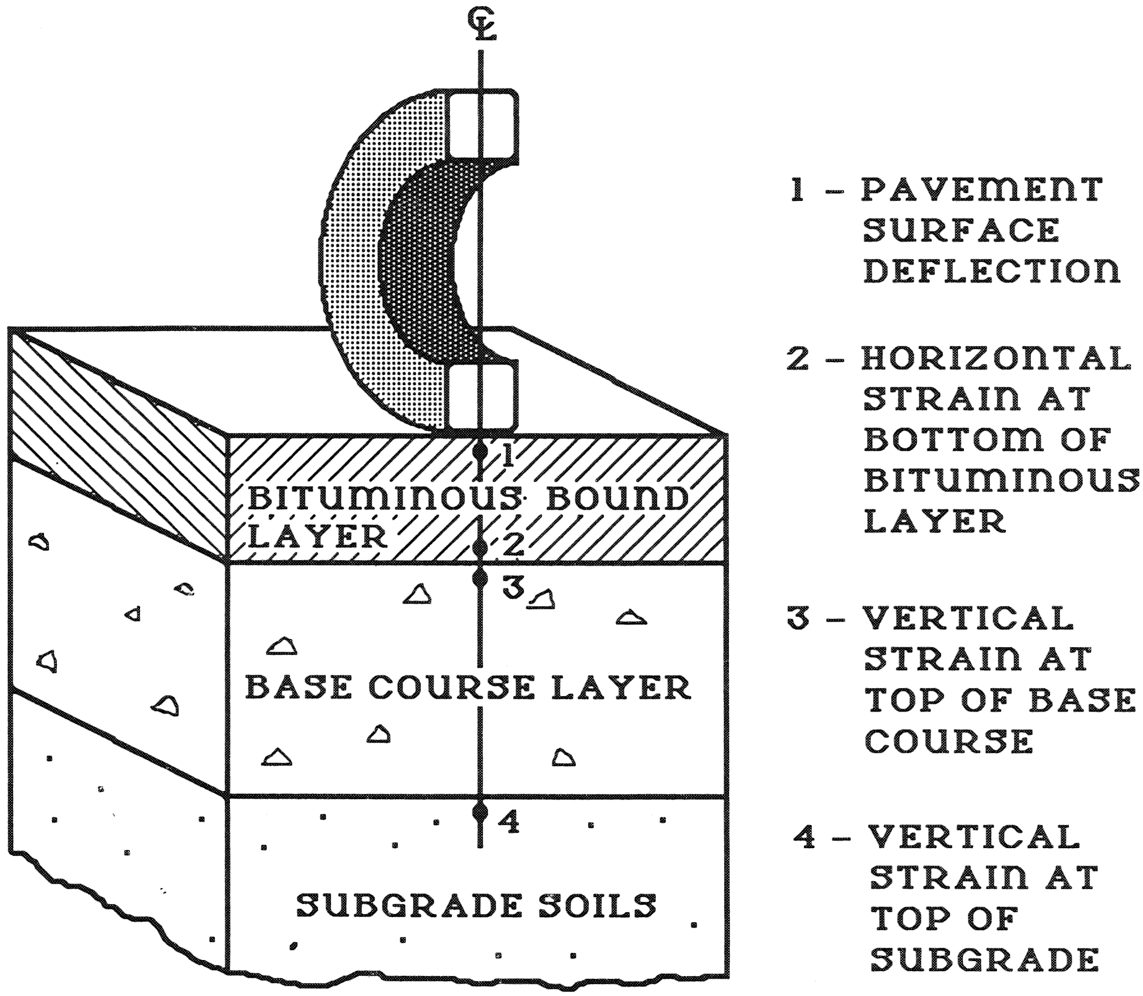


Figure 4.1 Pavement Response Locations Used in Evaluating Load Restrictions

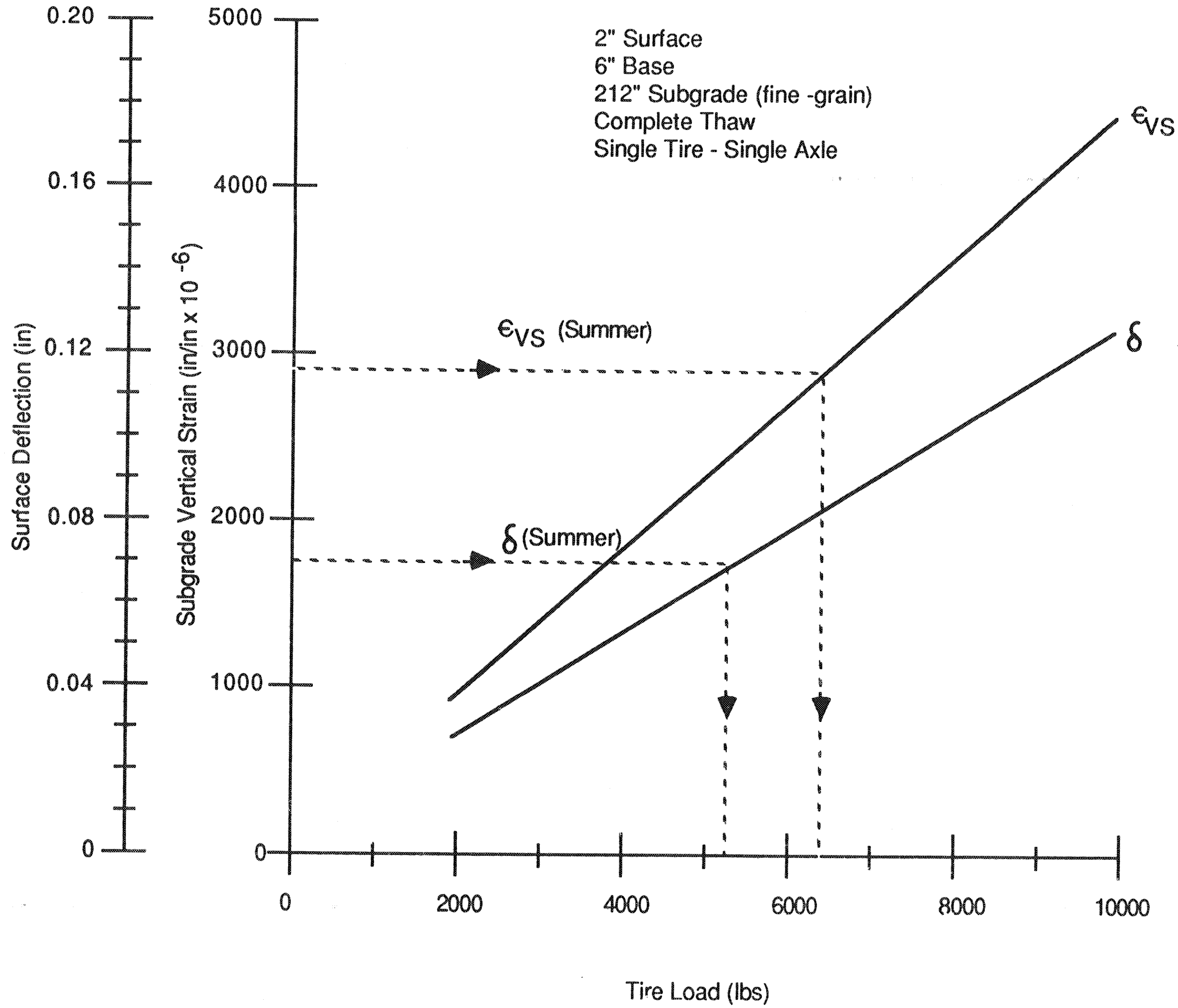


Figure 4.2 Graphical Illustration of the Determination of Allowable Load During Spring Thaw Period

The next step was to determine the spring load which would result in the same deflections and strains as the summer case. This was accomplished by entering the plot on the vertical axis with the summer deflection, or any summer strain value. A horizontal line was then drawn across to intersect the appropriate load-deflection or load-strain line. At the intersection a vertical line was drawn down to intersect the horizontal or tire load axis. These values were the tire loads which would result in the same deflection and strains as obtained during the summer under the maximum allowable loading. From these values, the percentage reduction in summer load required to maintain the same strains and deflections were computed.

4.2.1.6 SENSITIVITY ANALYSIS

A sensitivity analysis was carried out to test how the magnitude of load reduction varied with some variation in the input parameters. To do this first the pavement surface modular ratio (M_r spring/M_r summer) was varied from 1.25 to 3.75. The second item tested was the magnitude of the subgrade strength reduction during the spring thaw. The percentage reduction in resilient modulus was varied from 70, 80 and 85 percent for fine-grained soils, and 50, 70 and 75 percent for coarse-grained soils.

The results of the sensitivity analysis showed that:

(a) Load reduction during spring thaw is more sensitive to changes in subgrade than pavement surface modulus.

(b) The subgrade strength reduction of 75 percent for fine grained soils resulted in a reasonable values for spring load reductions when compared to current practice. The corresponding values for coarse grained soils was found to be 50 percent.

4.2.2 STRUCTURAL ANALYSIS RESULTS

The summary of the results of the structural analysis are shown in Tables 4.6 through 4.12 for all cases considered. The thawing cases include: complete thaw, partial thaw to the bottom of the base course, and partial thaw four inches into the subgrade (i.e., four inches below the bottom of the

Table 4.6 Percent Load Reduction for Complete Thaw - Fine-grained Soils - Single Axle - 75 Percent Reduction in Subgrade Resilient Modulus

Pavement Structural Section		Load Reduction (Percent)							
		Single Tire[a] - Pavement Response Criteria				Dual Tire[b] - Pavement Response Criteria			
Surface Thickness (in.)	Base Thickness (in.)	Pavement Surface Maximum Deflection	Bituminous Tensile Strain	Base Vertical Strain	Subgrade Vertical Strain	Pavement Surface Maximum Deflection	Bituminous Tensile Strain	Base Vertical Strain	Subgrade Vertical Strain
2	6	46	NR	13	31	53	NR	9	45
	12	47	NR	16	50	54	NR	12	55
4	6	21	NR	NR	NR	27	NR	NR	5
	12	23	NR	NR	22	29	NR	NR	3

Notes: (a) Single tire
Tire size: 16.5 - 22.5
Maximum legal tire load: 9,900 lb.
Tire pressure: 90 psi

(b) Dual tires
Tire size: 10 - 22.5
Maximum legal load per tire = 5,000 lb.
Tire pressure: 100 psi

(c) NR = No Reduction

Table 4.7 Percent Load Reduction for Complete Thaw - Coarse-grained Soils - Single Axle - 50 Percent Reduction in Subgrade Resilient Modulus

Pavement Structural Section		Load Reduction (Percent)							
		Single Tire[a] - Pavement Response Criteria				Dual Tire[b] - Pavement Response Criteria			
Surface Thick-ness (in.)	Base Thick-ness (in.)	Pavement Surface Maximum Deflection	Bituminous Tensile Strain	Base Vertical Strain	Subgrade Vertical Strain	Pavement Surface Maximum Deflection	Bituminous Tensile Strain	Base Vertical Strain	Subgrade Vertical Strain
2	6	32	67	37	31	40	NR	7	41
	12	33	69	38	39	38	NR	8	42
4	6	10	NR	NR	NR	23	NR	NR	14
	12	11	NR	NR	19	24	NR	NR	25

Notes: (a) Single tire
Tire size: 16.5 - 22.5
Maximum legal tire load: 9,900 lb.
Tire pressure: 90 psi

(b) Dual tires
Tire size: 10 - 22.5
Maximum legal load per tire = 5,000 lb.
Tire pressure: 100 psi

(c) NR = No Reduction

Table 4.8 Percent Load Reduction for Complete Thaw - Dual Tire-Tandem Axle[(c)]

Pavement Structural Section		Load Reduction (Percent)							
		Fine-grained Soil[(a)] Pavement Response Criteria				Coarse-grained Soil[(b)] Pavement Response Criteria			
Surface Thick-ness (in.)	Base Thick-ness (in.)	Pavement Surface Maximum Deflection	Bituminous Tensile Strain	Base Vertical Strain	Subgrade Vertical Strain	Pavement Surface Maximum Deflection	Bituminous Tensile Strain	Base Vertical Strain	Subgrade Vertical Strain
2	6	63	NR	51	45	46	NR	NR	41
	12	52	NR	53	56	39	NR	1	42
4	6	29	NR	5	13	11	NR	NR	13
	12	33	NR	7	39	22	NR	NR	24

Notes: (a) Fine-grained soil 75 percent reduction in resilient modulus (relative to summer condition)

(b) Coarse-grained soil 50 percent reduction in resilient modulus (relative to summer condition)

(c) Dual tire tandem axle tire size: 10 - 22.5 maximum legal load per tire: 4,250 lb. Tire pressure: 100 psi

(d) NR = No Reduction

Table 4.9 Percent Load Reduction for Thaw to Bottom of Base Course-fine-grained Soil - Single Axle - 75 Percent Reduction in Base Course Resilient Modulus

Pavement Structural Section		Load Reduction (Percent)							
		Single Tire[a] - Pavement Response Criteria				Dual Tire[b] - Pavement Response Criteria			
Surface Thickness (in.)	Base Thickness (in.)	Pavement Surface Maximum Deflection	Bituminous Tensile Strain	Base Vertical Strain	Subgrade Vertical Strain	Pavement Surface Maximum Deflection	Bituminous Tensile Strain	Base Vertical Strain	Subgrade Vertical Strain
2	6	NR	NR	37	NR	NE	NE	NE	NE
	12	NR	NR	24	NR	NE	NE	NE	NE
4	6	NR	NR	17	NR	NE	NE	NE	NE
	12	NR	NR	NR	NR	NE	NE	NE	NE

Notes: (a) Single tire
Tire size: 16.5 - 22.5
Maximum legal tire load: 9,900 lb.
Tire pressure: 90 psi

(b) Dual tires
Tire size: 10 - 22.5
Maximum legal load per tire = 5,000 lb.
Tire pressure: 100 psi

(c) NR = No Reduction

(d) Not Evaluated

Table 4.10 Percent Load Reduction for Thaw to Bottom of Base Course - Coarse-grained Soil - Single Axle - 50 Percent Reduction in Base Course Resilient Modulus

Pavement Structural Section		Load Reduction (Percent)							
		Single Tire[a] - Pavement Response Criteria				Dual Tire[b] - Pavement Response Criteria			
Surface Thick-ness (in.)	Base Thick-ness (in.)	Pavement Surface Maximum Deflection	Bituminous Tensile Strain	Base Vertical Strain	Subgrade Vertical Strain	Pavement Surface Maximum Deflection	Bituminous Tensile Strain	Base Vertical Strain	Subgrade Vertical Strain
2	6	1	57	39	NR	8	NR	9	NR
	12	18	66	38	NR	24	NR	8	NR
4	6	NR	NR	NR	NR	NR	NR	NR	NR
	12	NR	NR	NR	NR	3	NR	NR	NR

Notes: (a) Single tire
Tire size: 16.5 - 22.5
Maximum legal tire load: 9,900 lb.
Tire pressure: 90 psi

(b) Dual tires
Tire size: 10 - 22.5
Maximum legal load per tire = 5,000 lb.
Tire pressure: 100 psi

(c) NR = No Reduction

Table 4.11 Percent Load Reduction for Partial Thaw (4 in. below bottom of base) - Single Tire - Single Axle

Pavement Structural Section		Load Reduction (Percent)							
		Fine-grained Soil[a] Pavement Response Criteria				Coarse-grained Soil[b] Pavement Response Criteria			
Surface Thickness (in.)	Base Thickness (in.)	Pavement Surface Maximum Deflection	Bituminous Tensile Strain	Base Vertical Strain	Subgrade Vertical Strain	Pavement Surface Maximum Deflection	Bituminous Tensile Strain	Base Vertical Strain	Subgrade Vertical Strain
2	6	13	NR	45	55	34	62	38	31
	12	36	NR	39	64	41	66	38	39
4	6	NR	NR	25	32	23	NR	NR	NR
	12	5	NR	12	42	30	NR	NR	17

Notes:
(a) Fine-grained soil
85 percent reduction in resilient modulus (relative to summer condition)

(b) Coarse-grained soil
50 percent reduction in resilient modulus (relative to summer condition)

(c) Single tire
Tire size: 16.5 - 22.5
Maximum legal load: 9,900 lb.
Tire pressure 90 psi

(d) NR = No Reduction

Table 4.12 Percent Load Reduction for Partial Thaw (4 in. below bottom of base) - Dual Tire - Single Axle

Pavement Structural Section		Load Reduction (Percent)							
		Fine-grained Soil[a] Pavement Response Criteria				Coarse-grained Soil[b] Pavement Response Criteria			
Surface Thickness (in.)	Base Thickness (in.)	Pavement Surface Maximum Deflection	Bituminous Tensile Strain	Base Vertical Strain	Subgrade Vertical Strain	Pavement Surface Maximum Deflection	Bituminous Tensile Strain	Base Vertical Strain	Subgrade Vertical Strain
2	6	1	NR	42	57	NR	NR	8	28
	12	31	NR	37	66	8	NR	8	39
4	6	NR	NR	33	41	NR	NR	NR	7
	12	NR	NR	21	25	NR	NR	NR	23

Notes:
(a) Fine-grained soil
85 percent reduction in resilient modulus (relative to summer condition)

(b) Coarse-grained soil
50 percent reduction in resilient modulus (relative to summer condition)

(c) Dual Tires
Tire size: 10-22.5
Maximum legal load per tire = 5,000 lb
Tire pressure = 100 psi
Pavement response taken under inside tire duals

(d) NR = No Reduction

base). The results are also shown by the three tire and axle configurations used: single tire-single axle, dual tire-single axle and dual tire-tandem axle.

4.2.2.1 DISCUSSION OF RESULTS

4.2.2.1 (a) MAGNITUDE OF LOAD REDUCTION

As shown in Tables 4.6 through 4.12, the magnitude of load restriction varies with both pavement structure and load response parameter (deflection and strain). The calculated load reductions (for those cases which require a reduction) ranged from a low of 1 percent to a high of 69 percent. For all cases, the surface deflection and vertical subgrade strain provided the most consistent load reduction values (for the assumed conditions). The tensile strain (bottom of surface course) and vertical strain at the top of the subgrade criteria resulted in the largest reductions in load. An average load reduction of 34 percent results for the complete thaw and partial subgrade thaw cases for fine and coarse-grain soils for the subgrade vertical strain criterion (includes both two and four inch thick surface courses). For the same conditions but for two inch thick surface courses only, the average load reduction increases to 45 percent. The corresponding value for four inch thick surface courses is 21 percent. An average load reduction of 39 percent results for the complete thaw and partial subgrade thaw cases for fine-grained soil and both thickness levels of surface course (based on the subgrade vertical strain criterion as before). For the same conditions but for two and four inch thick surface courses, the average load reduction is 52 and and 25 percent, respectively.

Thus, for fine-grained soils (which are the kinds of soils which generally necessitate the need for load restrictions), a load reduction of about 50 percent is needed for thin surfaced bituminous pavements. The benefit of thicker surface courses (or stabilized pavement layers in general) is illustrated for the four inch thick surface course. For the fine-grain subgrade case, a load reduction of about 25 percent is needed (or one-half the load reduction amount needed for the two inch thick surface course).

4.2.2.1 (b) TIRE CONFIGURATION

From the data in Tables 4.6 though 4.12, there are no significant differences in reductions for single and dual tires. For both fine and coarse-grained soils in the complete thaw case, the dual tire configuration results in slightly higher reductions than the single tire. The dual tandem configuration results in about the same range of load reductions; although, the deflections and strain levels are lower than the single and dual tire single axle cases. The maximum strain values for the dual tandem configuration generally occurred between the dual tires.

4.2.2.1 (c) CONSEQUENCE OF MAINTAINING LOADS

An evaluation of the consequences of maintaining the maximum summer loads during the spring was performed. This was done by examining criteria generally accepted as indicators of pavement distress. These are the maximum tensile strain at the bottom of the bituminous bound layer (fatigue cracking) and the vertical strain at the top of the subgrade (rutting). The Asphalt Institute criteria, as used in MS-1 [4.1], have been used to determine the number of load applications to failure for any given strain. The results are shown in Tables 4.13 through 4.20 for prediction of loads to failure for complete thaw, thaw to bottom of base and thaw four inches below the bottom of the base.

The predicted loads to failure for the load cases evaluated are relatively low for the fine-grained subgrade cases (both summer and spring conditions). This is in part due to the cross sections selected for evaluation but primarily the material properties (the principal material property being resilient modulus). The negative percent change in the loads to failure (summer to spring) is consistently high for the two inch thick surface course cases. For the four inch thick surface course, occasionally the spring condition (with the higher stiffness surface course) results in a positive change in the estimated loads to failure (i.e., longer pavement life).

Table 4.13 Change in Pavement Life - Single Tire - Single Axle - Tensile Strain Bottom of Bituminous Bound Layer - Complete Thaw[a][b]

Pavement Structural Section		Fine-grained Soil					Coarse-grained Soil				
		Summer		Spring[c]		Percent Change Loads to Failure	Summer		Spring[c]		Percent Change Loads to Failure
Surface Thickness (in.)	Base Thickness (in.)	Strain (in/in $\times 10^{-6}$)	Loads to Failure	Strain (in/in $\times 10^{-6}$	Loads to Failure		Strain (in/in $\times 10^{-6}$	Loads to Failure	Strain (in/in $\times 10^{-6}$	Loads to Failure	
2	6	950	10,800	902	3,900	-64%	190	2.1×10^6	312	128,600	-94%
	12	899	12,900	870	4,400	-66%	182	2.5×10^6	296	152,900	-94%
4	6	655	36,600	372	72,100	+97%	243	956,100	193	624,600	-34%
	12	629	41,800	365	76,700	+84%	232	1.1×10^6	186	705,900	-37%

Notes: (a) Equation for estimating number of loads to cause up to 10% cracking in the wheel path: $\log N_f = 15.947 - 3.291 \log \left(\frac{\varepsilon_t}{10^{-6}}\right) - 0.854 \log \left(\frac{M_R}{10^3}\right)$

(b) Single tire - single axle: Load = 9,900 lb.
Tire pressure = 90 psi

(c) Spring case for complete thaw
(i) Fine-grain: 75% reduction in subgrade resilient modulus
(ii) Coarse-grain: 50% reduction in subgrade resilient modulus

Table 4.14 Change in Pavement Life - Single Tire - Single Axle - Subgrade Vertical Strain Criterion - Complete Thaw(a)(b)

Pavement Structural Section		Fine-grained Soil					Coarse-grained Soil				
		Summer		Spring(c)		Percent Change Loads to Failure	Summer		Spring(c)		Percent Change Loads to Failure
Surface Thickness (in.)	Base Thickness (in.)	Strain (in/in $\times 10^{-6}$)	Loads to Failure	Strain (in/in. $\times 10^{-6}$)	Loads to Failure		Strain (in/in $\times 10^{-6}$)	Loads to Failure	Strain (in/in $\times 10^{-6}$)	Loads to Failure	
2	6	3,120	230	4,482	45	-80%	755	1.3×10^{5}	1,060	2.9×10^{4}	-78%
	12	1,670	3,810	3,330	172	-95%	368	3.4×10^{6}	592	0.4×10^{6}	-88%
4	6	1,570	5,020	1,480	6,540	+30%	500	8.5×10^{5}	497	0.9×10^{6}	+6%
	12	1,000	37,960	1,290	12,120	-68%	270	1.3×10^{7}	334	5.2×10^{6}	-60%

Notes: (a) Equation for estimating number of loads to cause a 0.75 in. rut: $N_f = 1.077 \times 10^{18} \left(\frac{1}{\varepsilon_{vs}}\right)^{4.4843}$

(b) Single tire - single axle: Load = 9,900 lb
Tire pressure = 90 psi

(c) Spring case for complete thaw
(i) Fine-grain: 75% reduction in subgrade resilient modulus
(ii) Coarse-grain: 50% reduction in subgrade resilient modulus

Table 4.15 Change in Pavement Life - Single Tire - Single Axle - Tensile Strain Bottom of Bituminous Bound Layer - Thaw to Bottom of Base Course(a)(b)

Pavement Structural Section		Fine-grained Soil					Coarse-grained Soil				
		Summer		Spring(c)			Summer		Spring(c)		
Surface Thick-ness (in.)	Base Thick-ness (in.)	Strain (in/in X10-6)	Loads to Failure	Strain (in/in X10-6)	Loads to Failure	Percent Change Loads to Failure	Strain (in/in X10-6)	Loads to Failure	Strain (in/in X10-6)	Loads to Failure	Percent Change Loads to Failure
2	6	950	10,800	641	12,020	+ 11%	190	$2.1X10^6$	274	197,130	-91%
	12	899	12,900	742	7,430	- 42%	182	$2.5X10^6$	286	171,190	-93%
4	6	655	36,600	270	206,900	+465%	243	956,000	170	948,360	- 1%
	12	629	41,800	301	144,680	+246%	232	$1.1X10^6$	176	846,050	-23%

Notes: (a) Equation for estimating number of loads to cause up to 10% cracking in the wheelpath:

$$\log N_f = 15.947 - 3.291 \log \left(\frac{\varepsilon_t}{10^{-6}}\right) - 0.854 \log \left(\frac{M_R}{10^3}\right)$$

(b) Single tire - single axle: Load = 9,900 lb
Tire pressure = 90 psi

(c) Spring case for thaw to bottom of base
(i) Fine-grain: 75% reduction in base resilient modulus
(ii) Coarse-grain: 50% reduction in base resilient modulus

Table 4.16 Change in Pavement Life - Single Tire - Single Axle - Tensile Strain Bottom of Bituminous Bound Layer - Thaw 4 in. Below Bottom of Base(a)(b)

Pavement Structural Section		Fine-grained Soil					Coarse-grained Soil				
		Summer		Spring(c)			Summer		Spring(c)		
Surface Thickness (in.)	Base Thickness (in.)	Strain (in/in X10-6)	Loads to Failure	Strain (in/in X10-6)	Loads to Failure	Percent Change Loads to Failure	Strain (in/in X10-6)	Loads to Failure	Strain in/in X10-6)	Loads to Failure	Percent Change Loads to Failure
2	6	950	10,800	824	5,260	- 51%	190	$2.1X10^6$	291	161,700	-93%
	12	899	12,900	890	4,080	- 68%	182	$2.5X10^6$	288	167,300	-93%
4	6	655	36,600	317	122,000	+233%	243	956,100	178	815,170	-15%
	12	629	41,800	343	94,130	+125%	232	$1.1X10^6$	179	800,280	-27%

Notes: (a) Equation for estimating number of loads to cause up to 10% cracking in the wheel path:

$$\log N_f = 15.947 = 3.291 \log \left(\frac{\epsilon_t}{10^{-6}}\right) - 0.854 \log \left(\frac{M_R}{10^3}\right)$$

(b) Single tire - single axle: Load = 9,900 lb.
Tire pressure = 90 psi

(c) Spring case for complete thaw
(i) Fine-grain: 85% reduction in subgrade resilient modulus
(ii) Coarse-grain: 50% reduction in subgrade resilient modulus

Table 4.17 Change in Pavement Life - Single Tire - Single Axle - Subgrade Vertical Strain Criterion - Thaw 4 in. Below Bottom of Base (a)(b)

Pavement Structural Section		Fine-grained Soil					Coarse-grained Soil				
		Summer		Spring (c)			Summer		Spring (c)		
Surface Thickness (in.)	Base Thickness (in.)	Strain (in/in $X10^{-6}$)	Loads to Failure	Strain (in/in $X10^{-6}$)	Loads to Failure	Percent Change Loads to Failure	Strain (in/in $X10^{-6}$)	Loads to Failure	Strain in/in $X10^{-6}$)	Loads to Failure	Percent Change Loads to Failure
2	6	3,120	230	6,532	8	-97%	755	$1.3X10^5$	1,066	28,500	-78%
	12	1,670	3,810	4,534	43	-99%	368	$3.4X10^6$	587	413,820	-88%
4	6	1,570	5,020	2,323	870	-83%	500	$8.5X10^5$	498	865,030	+ 2%
	12	1,000	37,960	1,773	2,910	-92%	270	$1.3X10^7$	325	$5.9X10^6$	-55%

Notes: (a) Equation for estimating number of loads to cause a 0.75 in. rut: $N_f = 1.077 \times 10^{18} \left(\frac{1}{\varepsilon_{vs}}\right)^{4.4843}$

(b) Single tire - single axle: Load = 9,900 lb.
Tire pressure = 90 psi

(c) Spring case for complete thaw
(i) Fine-grain: 85% reduction in subgrade resilient modulus
(ii) Coarse-grain: 50% reduction in subgrade resilient modulus

Table 4.18 Change in Pavement Life - Dual Tire - Single Axle - Subgrade Vertical Strain Criterion - Complete Thaw(a)(b)(d)

Pavement Structural Section		Fine-grained Soil					Coarse-grained Soil				
		Summer		Spring(c)			Summer		Spring(c)		
Surface Thickness (in.)	Base Thickness (in.)	Strain (in/in X10^{-6})	Loads to Failure	Strain (in/in X10^{-6})	Loads to Failure	Percent Change Loads to Failure	Strain (in/in X10^{-6})	Loads to Failure	Strain in/in X10^{-6})	Loads to Failure	Percent Change Loads to Failure
2	6	2,101	1,360	3,766	99	-93%	438	$1.5X10^6$	742	144,700	-90%
	12	1,360	9,560	3,015	270	-97%	284	$1.1X10^7$	489	938,700	-91%
4	6	1,295	11,910	1,357	9,660	-19%	352	$4.1X10^6$	409	$2.1X10^6$	-49%
	12	1,190	17,400	1,230	15,000	-14%	224	$3.1X10^7$	300	$8.4X10^6$	-73%

Notes: (a) **Equation for estimating number of loads to cause a 0.75 in.** rut: $N_f = 1.077X10^{18} \left(\frac{1}{\varepsilon_{vs}}\right)^{4.4843}$

(b) Dual tire - single axle: Load = 5,000 lb
Tire pressure = 100 psi

(c) Spring case for complete thaw
(i) Fine-grain 75% reduction in subgrade resilient modulus
(ii) Coarse-grain: 50% reduction in subgrade resilient modulus

(d) Strain response between dual tires

Table 4.19 Change in Pavement Life - Dual Tire - Single Axle - Subgrade Vertical Stain Criterion - Thaw 4 in. Below Bottom of Base(a)(b)(d)

Pavement Structural Section		Fine-grained Soil					Coarse-grained Soil				
		Summer		Spring(c)			Summer		Spring(c)		
Surface Thickness (in.)	Base Thickness (in.)	Strain (in/in X10-6)	Loads to Failure	Strain (in/in X10-6)	Loads to Failure	Percent Change Loads to Failure	Strain (in/in X10-6)	Loads to Failure	Strain in/in X10-6)	Loads to Failure	Percent Change Loads to Failure
2	6	2,105	1,350	4,983	28	-98%	513	$7.6X10^5$	707	179,710	-76%
	12	1,218	15,680	3,800	95	-99%	260	$1.6X10^7$	426	$1.7X10^6$	-89%
4	6	1,167	18,990	1,996	1,710	-91%	344	$4.5X10^6$	368	$3.4X10^6$	-24%
	12	1,147	20,520	1,518	5,840	-72%	202	$4.9X10^7$	262	$1.5X10^7$	-69%

Notes: (a) Equation for estimating number of loads to cause a 0.75 in. rut: $N_f = 1.077X10^{18} \left(\frac{1}{\epsilon_{vs}}\right)^{4.4843}$

(b) Dual tire - single axle: Load = 5,000 lb
Tire pressure = 100 psi

(c) Spring case for complete thaw
(i) Fine-grain: 85% reduction in subgrade resilient modulus
(ii) Coarse-grain: 50% reduction in subgrade resilient modulus

(d) Strain response beneath inside tire of dual set

Table 4.20 Change in Pavement Life - Dual Tire - Tandem Axle - Subgrade Vertical Strain Criterion - Complete Thaw(a)(b)(d)

Pavement Structural Section		Fine-grained Soil					Coarse-grained Soil				
		Summer		Spring(c)			Summer		Spring(c)		
Surface Thickness (in.)	Base Thickness (in.)	Strain (in/in $X10^{-6}$)	Loads to Failure	Strain (in/in $X10^{-6}$)	Loads to Failure	Percent Change Loads to Failure	Strain (in/in $X10^{-6}$)	Loads to Failure	Strain in/in $X10^{-6}$)	Loads to Failure	Percent Change Loads to Failure
2	6	1,780	2,860	3,227	200	-93%	370	$3.3X10^6$	629	303,550	-91%
	12	1,150	20,280	2,581	540	-97%	240	$2.3X10^7$	412	$2.0X10^6$	-91%
4	6	1,058	29,480	1,213	15,970	-46%	297	$8.8X10^6$	341	$4.7X10^6$	-47%
	12	670	228,690	1,056	29,730	-87%	190	$6.5X10^7$	250	$1.9X10^7$	-71%

Notes: (a) Equation for estimating number of loads to cause a 0.75 in. rut: $N_f = 1.077X10^{18} \left(\frac{1}{\epsilon_{vs}}\right)^{4.4843}$

(b) Dual tire - tandem axle: Load = 4,250 lb.
Tire pressure = 100 psi

(c) Spring case for complete thaw
(i) Fine-grain: 75% reduction in subgrade resilient modulus
(ii) Coarse-grain: 50% reduction in subgrade resilient modulus

(d) Strain response between one set of dual tires (with exception of 4/12 fine-grain case where strain response directly under inside tire)

4.2.3 STRUCTURAL ANALYSIS SUMMARY

The following summary statements are warranted:

(a) The range of magnitudes for spring load reductions depends on the subgrade soil type and the thickness of the pavement surface and base layers.

(b) The allowable loads during the spring thaw period were based on the assumption that critical pavement response parameters (such as deflection and strain) should not exceed those estimated for summer conditions. The load reduction needed for fine-grain subgrades was about 50 percent and approximately one-half that amount for coarse-grained subgrades.

(c) The maximum pavement surface deflection and the vertical strain on top of the subgrade were load response parameters that consistently necessitated load reductions over the range of cases considered.

4.3 TIMING LOAD LIMITS

4.3.1 APPROACH

In order to perform a realistic ground thermal analysis for climate conditions where ground freezing occurs, the following capabilities must be present in a heat transfer model:

(a) the ability to include latent heat effects,
(b) the ability to analyze a transient problem, and
(c) the ability to include energy fluxes (i.e., energy changes) at the surface due to radiant and convective heat transfer.

The finite element program TDHC, developed at the University of Alaska-Fairbanks (Goering and Zarling [2.47]), was selected for the thermal analysis in this study. The program is capable of performing a transient, two-dimensional heat transfer analysis. Latent heat is modelled using a Dirac Delta function in the heat capacitance matrix. Surface temperatures may be input as a sinusoidally varying function. Convective heat transfer can be included for a sinusoidally varying fluid temperature. Radiant heat can be included as a surface heat flux.

4.3.2 THERMAL DATA REQUIRED FOR INPUT

In order to identify the surface temperature function, the mean annual and monthly average temperatures were obtained for 60 locations in frost areas in the United States (excluding Alaska) from Cinquemani et al. [4.2]. Harmonic temperature functions for all locations were obtained by equating the area under the discontinuous monthly temperature function to the area under a sine curve (Figure 4.3). Once the equivalent sine curve was defined, the amplitude of temperature variation, the phase lag with respect to January 1, the freezing and thawing indices and the duration of the freezing and thawing periods were obtained (Table 4.21). The results from all 60 locations were combined into seven cases of freezing conditions ranging from 400 to 2000°F-days, as shown in Table 4.22.

Fixed temperature boundary conditions were required to perform the analysis. In order to identify a fixed temperature at some depth in the ground, the geothermal gradient as well as the depth where surface temperature oscillation effects become negligible were required. Many values have been reported in the literature for the geothermal gradient (see Lunardini, [2.35]) ranging from 0.00309 to 0.031°F/ft. A value of 0.02°F/ft. was selected for this study.

The depth at which the ground temperature oscillates less than one percent of the surface temperature oscillation in a homogeneous material can be found from the following:

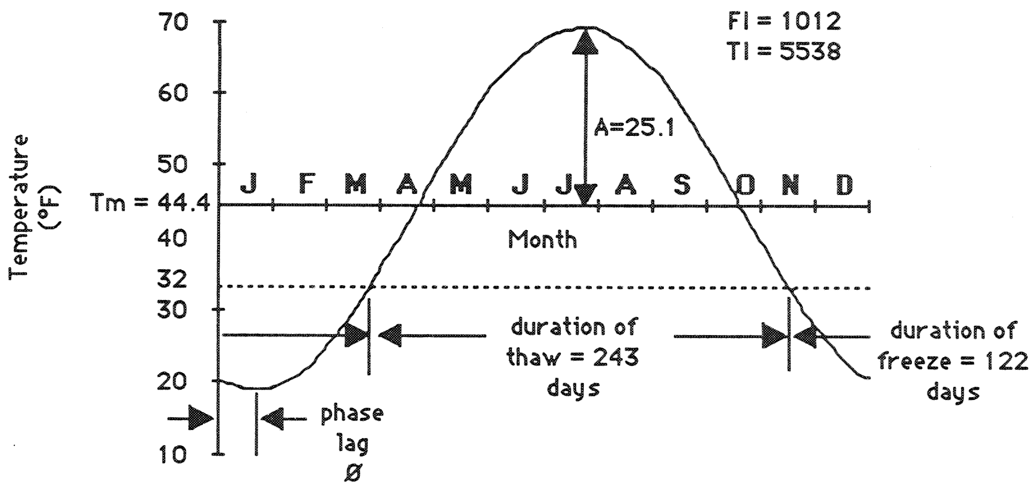
$$\frac{T - T_m}{T_a} = e^{-2\pi \left(\frac{x}{2\sqrt{\pi p \alpha}}\right)}$$

where:

T = ground temperature,

T_m = mean annual surface temperature,

T_a = amplitude of temperature variation,



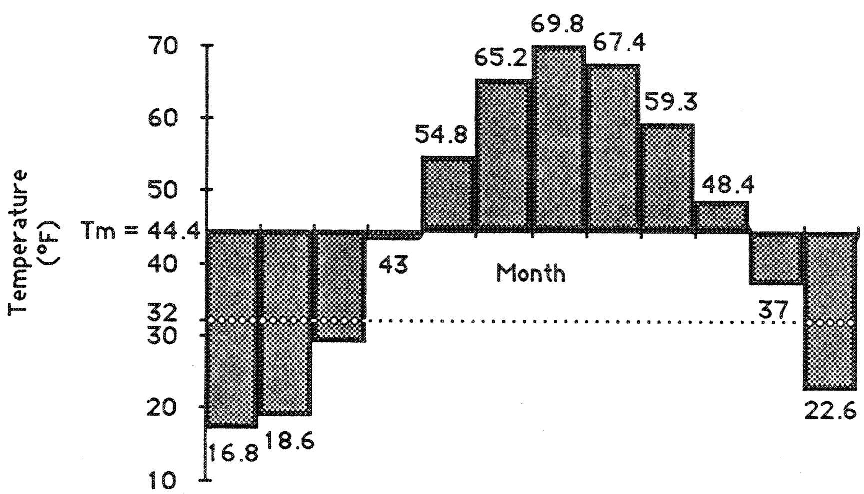


Figure 4.3 Area Under Discontinuous Temperature Function Equated to Area Sinusoidal Temperature Function for Burlington, Vermont

Table 4.21 Temperature Function Data

Location	Freezing Index (°F-days)	Duration of Freeze (days)	Mean Annual Temp. (°F)	Amplitude of Temp. Variation (°F)	Phase Lag (days)	Thawing Index (°F-days)	Start of Thaw
Hartford CONN	335	85	49.1	23.0	18	6576	01-Mar
Burlington IA	368	85	50.8	25.3	13	7230	25-Feb
DesMoines IA	657	102	49.0	26.7	14	6862	05-Mar
Mason City IA	1232	127	44.8	27.7	13	5904	17-Mar
Sioux City IA	785	108	48.4	27.5	13	6771	07-Mar
Pocatello ID	533	101	46.7	22.7	19	5899	10-Mar
Chicago ILL	339	84	50.6	24.7	17	7128	28-Feb
Moline ILL	486	94	49.8	25.7	14	6983	01-Mar
Fort Wayne IND	312	83	49.9	23.6	16	6845	27-Feb
South Bend IND	390	89	49.1	23.7	17	6631	02-Mar
Caribou ME	1830	151	38.3	25.5	19	4312	04-Apr
Alpena MI	674	110	45.0	22.3	22	5419	17-Mar
Detroit MI	1127	131	42.1	23.3	20	4813	26-Mar
Flint MI	308	82	49.9	23.6	21	6841	02-Mar
Grand Rapids MI	559	102	46.8	23.1	18	5961	09-Mar
Sault Ste. Ma MI	533	99	47.8	24.0	18	6300	08-Mar
Traverse City MI	1482	143	40.0	24.0	19	4402	31-Mar
Duluth MN	835	116	44.8	23.7	22	5507	20-Mar
Int'l Falls MN	1999	154	38.6	26.8	15	4408	01-Apr
Minn St. Paul MN	2686	165	36.5	29.8	11	4328	03-Apr
Rochester MN	1419	132	44.1	28.6	13	5835	19-Mar
Billings MT	1388	132	43.6	27.7	14	5622	20-Mar
Cut Bank MT	676	108	46.3	23.8	20	5895	14-Mar
Dillon MT	1267	138	40.5	22.6	18	4370	27-Mar
Glasgow MT	924	124	42.6	22.0	20	4793	22-Mar
Great Falls MT	1767	143	41.5	28.5	12	5234	24-Mar
Helena MT	730	133	44.9	22.8	9	5439	16-Mar
Lewiston MT	956	124	43.2	23.0	17	5044	19-Mar
Miles City MT	1022	129	41.9	22.1	20	4636	25-Mar
Missoula MT	1153	124	45.3	27.5	15	6008	17-Mar
Bismarck ND	738	115	43.7	21.4	14	5008	12-Mar

Table 4.21 Temperature Function Data (Cont.)

Location	Freezing Index (°F-days)	Duration of Freeze (days)	Mean Annual Temp (°F)	Amplitude of Temp Variation (°F)	Phase Lag (days)	Thawing Index (°F-days)	Start of Thaw
Fargo ND	1907	145	41.4	29.7	11	5338	24-Mar
Minot ND	2103	149	40.8	30.7	12	5315	27-Mar
Grand Island NEB	2007	149	40.1	28.9	12	4964	27-Mar
North Omaha NEB	514	95	50.1	26.4	15	7121	03-Mar
North Platte NEB	579	99	49.4	26.3	13	6930	03-Mar
Scottsbluff NEB	582	100	48.6	25.5	18	6641	08-Mar
Concord NH	527	98	48.2	24.4	21	6440	10-Mar
Albany NY	723	111	45.6	23.5	18	5687	14-Mar
Binghamton NY	562	101	47.6	24.1	16	6256	07-Mar
Buffalo NY	622	106	46.0	22.9	19	5732	12-Mar
Massena NY	508	99	47.1	22.9	21	6020	11-Mar
Rochester NY	1096	127	43.2	24.4	17	5184	21-Mar
Syracuse NY	455	95	47.9	23.2	19	6259	07-Mar
Toledo OH	433	93	48.1	23.2	19	6310	06-Mar
Youngstown OH	356	87	49.3	23.5	17	6671	01-Mar
Burns ORE	308	84	48.7	22.3	18	6403	29-Feb
Erie PA	411	93	47.1	21.8	21	5922	08-Mar
Huron SD	1366	129	44.8	29.0	12	6038	17-Mar
Pierre SD	1295	125	46.2	30.0	14	6478	17-Mar
Rapid City SD	760	110	46.6	25.1	21	6089	16-Mar
Sioux Falls SD	1216	125	45.4	28.3	13	6107	16-Mar
Burlington VT	1012	122	44.4	25.1	19	5538	20-Mar
Eau Claire WIS	1529	136	43.1	28.4	13	5580	21-Mar
Green Bay WIS	1196	128	43.7	26.0	15	5466	19-Mar
La Crosse WIS	1026	119	46.4	27.6	12	6282	12-Mar
Madison WIS	1045	122	44.9	26.0	14	5754	15-Mar
Milwaukee WIS	789	113	45.7	24.3	17	5790	14-Mar
Casper WYO	850	116	45.4	24.6	23	5741	21-Mar
Cheyenne WYO	615	106	45.9	22.7	25	5689	18-Mar
Sheridan WYO	818	115	45.0	23.8	19	5563	17-Mar

Table 4.22 Freezing Index Cases for Thermal Analysis

Freezing Index (°F-days)	Mean Annual Temperature (°F)	Amplitude (°F)	Phase Lag (days)	Duration of Freeze (days)	Duration of Thaw (days)	Start of Thaw
400	49.0	23.8	18	90	275	03-Mar
500	48.2	24.0	18	96	269	06-Mar
750	45.6	24.1	18	112	253	14-Mar
1000	44.1	24.6	18	124	241	20-Mar
1250	44.0	26.9	15	130	235	20-Mar
1500	42.5	27.1	15	136	229	23-Mar
2000	40.0	28.5	13	150	215	28-Mar

x = depth,

p = period of oscillation, and

α = thermal diffusivity.

When the quantity $x/2\sqrt{\pi p \alpha}$ is greater than 0.8, the amplitude of the temperature envelope is less than one percent of the surface fluctuation. For the materials assumed in this study, the depth where fluctuations were less than one percent ranged from 35 to 40 feet. Therefore, temperatures were fixed at a depth of 50 feet for the ground thermal modelling. At 50 feet the temperature was fixed based on the mean annual temperature for the freezing index case of interest and the geothermal gradient.

The short wave radiation heat flux during spring at the ground surface was estimated using the data provided be Cinquemani et al. [4.2]. The data are measured monthly values of average daily incoming direct and diffuse solar radiation. Therefore, scattering, cloud cover and solar distance are reflected in the values. The data for all 60 locations for the months of March, April and May are shown in Table 4.23.

Correlations of locations (primarily latitude) or freezing index and solar radiation could not be verified by the data. The primary dependent variable for solar radiation was solar declination or time of the year. Therefore, average values for March, April and May were calculated from all 60 locations. The net short wave radiant heat flux absorbed at the pavement surface was calculated as $(1 - \alpha_s)$ times the monthly value obtained above. A value of 0.1 for α_s, the surface reflectivity, was used (Scott [4.3]). The values of net short wave radiation used for the thermal analysis are given in Table 4.24.

No data was found for values of long wave radiation over the area of interest. Therefore, the long wave radiation was estimated following the procedure outlined in Chapter 2.0 for the months of March, April and May. The mean monthly temperature was calculated for the seven freezing index cases for March, April and May and are shown in Table 4.24. The values for average monthly sunshine for seventeen locations in the geographic areas of interest were obtained from U.S. Weather Service data found in Ruffner and Bair [4.4].

Table 4.23 Solar Radiation Data for March, April and May

Location	Incoming Short Wave Radiation (BTU/day)		
	March	April	May
Hartford CONN	477.5	714.7	978.5
Burlington IA	579.2	858.6	1165.1
DesMoines IA	580.7	860.7	1180.5
Mason City IA	553.7	836.2	1168.0
Sioux City IA	568.6	841.6	1170.4
Pocatello ID	539.2	882.0	1371.4
Chicago ILL	507.0	759.5	1106.9
Moline ILL	535.1	812.0	1118.6
Fort Wayne IND	455.2	697.6	982.0
South Bend IND	415.7	659.6	992.5
Caribou ME	419.3	724.0	1133.1
Alpena MI	362.1	616.6	1028.2
Detroit MI	417.4	680.4	1000.2
Flint MI	383.1	636.4	956.8
Grand Rapids MI	369.6	648.3	1014.4
Sault Ste. Mar MI	324.8	603.3	1028.6
Traverse City MI	310.8	567.5	1001.0
Duluth MN	388.6	672.8	1034.5
Int'l Falls MN	355.7	662.5	1045.9
Minn-St. Paul MN	464.0	763.9	1103.5
Rochester MN	477.0	752.8	1081.9
Billings MT	486.0	763.2	1189.5
Cut Bank MT	402.2	687.8	1128.0
Dillon MT	526.5	846.2	1279.2
Glasgow MT	388.0	671.3	1104.9
Great Falls MT	420.5	720.2	1170.4
Helena MT	419.4	708.8	1145.5
Lewistown MT	420.0	692.2	1128.4
Miles City MT	457.0	745.3	1185.0
Missoula MT	311.8	574.2	981.5
Bismarck ND	466.8	775.7	1168.1
Fargo ND	414.9	705.7	1097.9
Minot ND	383.7	655.9	1044.3
Grand Island NEB	661.3	917.0	1265.2
North Omaha NEB	634.0	892.1	1225.0
North Platte NEB	692.4	958.3	1333.0
Scottsbluff NEB	675.7	950.5	1307.4
Concord NH	459.5	686.1	973.6
Albany NY	456.5	688.4	985.9
Binghamton NY	385.8	575.8	851.2
Buffalo NY	348.9	546.4	888.5
Massena NY	391.2	620.1	977.5
Rochester NY	364.3	559.5	903.4

Table 4.23 Solar Radiation Data for March, April and May (Cont.)

Location	Incoming Short Wave Radiation (BTU/day)		
	March	April	May
Syracuse NY	385.1	571.3	890.4
Toledo OH	434.8	680.4	996.7
Youngstown OH	385.1	586.5	890.1
Burns OR	490.0	792.0	1187.1
Erie PA	345.6	576.8	920.4
Huron SD	488.2	744.7	1113.7
Pierre SD	530.0	795.1	1206.5
Rapid City SD	542.3	826.5	1228.8
Sioux Falls SD	532.6	802.1	1152.2
Burlington VT	385.3	606.8	940.2
Eau Claire WIS	451.7	746.4	1090.2
Green Bay WIS	451.2	724.9	1104.2
LaCrosse WIS	481.3	764.7	1100.8
Madison WIS	515.2	804.0	1136.0
Milwaukee WIS	479.4	736.5	1088.8
Casper WYO	683.2	1013.5	1441.1
Cheyenne WYO	765.8	1067.8	1433.1
Sheridan WYO	517.5	788.2	1204.8

Table 4.24 Radiation and Weather Data for TDHC Analysis

	March	April	May
Net short wave radiation flux (BTU/hr)	27.0	40.5	54.0
Average monthly Temperature (°F)	30.8	43.7	55.4
Average monthly cloud cover (%)	44	44	40
Net long wave radiation flux (BTU/hr)	18.4	17.9	18.4
Net radiant heat flux at ground surface (BTU/hr)	9.0	22.5	36.0

The data are shown in Table 4.25. The average monthly values used for the estimate of long wave radiation are shown in Table 4.24. The resulting values of hourly average long wave radiation by month and the net radiant heat flux at the ground surface due to all radiant effects are given in Table 4.24.

It was decided that the empirical formula of Vehrencamp was most suited to estimating the convection coefficient for a pavement surface. The value obtained using the average spring temperatures above and an average windspeed of 11.7 miles per hour (Ruffner and Bair, [4.4]) was equal to 3.2 Btu/hr ft^2 °F.

4.3.3 PAVEMENT STRUCTURE SECTIONS

The sections used in the thermal analysis were selected from those analyzed in the structural analysis. It was felt that typically the majority of pavements experiencing thaw weakening are underlain with fine grained materials. Therefore, this type of subgrade was emphasized in the analysis. Sections included two and four inches of asphalt concrete, six and twelve inches of base and fine and coarse-grain subgrade for freezing conditions ranging from 400 to 2000°F-days. A total of four basic sections were analyzed and are shown in Figure 4.4. All structural sections and freezing index cases analyzed with TDHC are given in Table 4.26.

4.3.4 MATERIAL THERMAL PROPERTIES

The thermal properties required for the analysis are the frozen and unfrozen thermal conductivity, the frozen and unfrozen volumetric specific heat and the latent heat. These properties are functions of the dry density of the material, γ_d, and the moisture content, w, as outlined in Chapter 2.0. The dry density and moisture content used in the study for all materials including asphalt, aggregate base, and subgrades are shown in Table 4.27. Also included in this table are the thermal properties.

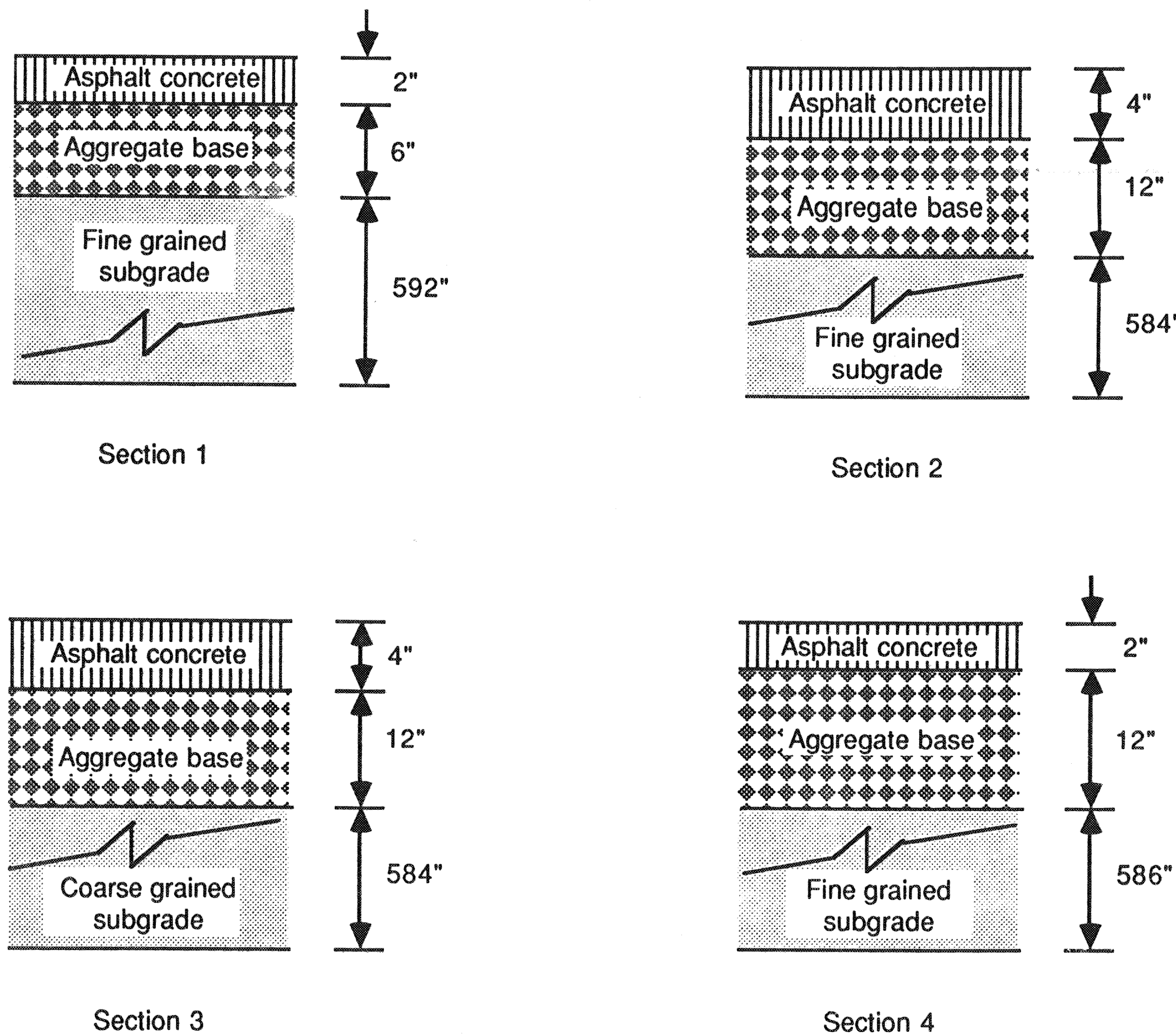


Figure 4.4 Pavement Structures for Thermal Analysis

Table 4.25 Percent Monthly Sunshine for March, April and May

Location	Percent Sunshine		
	March	April	May
Boise ID	60	65	70
Chicago ILL	62	49	62
Des Moines IA	53	55	60
Detroit MI	50	55	60
Sault Ste. Marie MI	53	55	58
Minn. St. Paul MN	55	56	60
Havre MT	70	66	72
Missoula MT	48	51	54
Williston MT	60	57	61
N. Platte ND	59	62	61
Lincoln NEB	56	60	62
Buffalo NY	45	51	57
Bismarck ND	60	57	62
Fargo ND	56	56	57
Rapid City SD	61	57	55
Burlington VT	50	50	55
Green Bay WIS	52	51	57

Table 4.26 Pavement Structures and Freezing Index Cases for TDHC Analysis

Freezing Index Case (°F-days)	Pavement Structural Sections			
	2 in. AC/BST 6 in. Base Fine Subgrade	4 in. AC 12 in. Base Fine Subgrade	4 in. AC 12 in. Base Coarse Subgrade	2 in. AC/BST 12 in. Base Fine Subgrade
400	X	X	X	
500	X	X	X	X
750	X	X	X	
1000	X	X	X	X
1250	X	X	X	
1500	X	X	X	
2000	X	X	X	

Table 4.27 Material Thermal Properties

Material	Dry Density γ_d (lb/ft³)	Moisture Content, w (%)	Frozen Thermal Conductivity, k_f (BTU/lb ft °F)	Unfrozen Thermal Conductivity, k_u (BTU/lb ft °F)	Frozen Volumetric Specific Heat, C_f (BTU/ft³)	Unfrozen Volumetric Specific Heat, C_u (BTU/ft³)	Latent Heat, L (BTU/ft³)
Asphalt Concrete	138	0	0.86	0.86	28.0	28.0	0
Aggregate Base	130	4	1.15	1.36	24.7	27.3	749
Fine-grained Subgrade	95	15	0.71	0.64	23.3	30.4	2052
Coarse-grained Subgrade	120	10	1.74	1.45	26.4	32.4	1728

4.3.5 ANALYTICAL METHOD

To perform the thermal finite element analysis, a generalized finite element grid was generated (Figure 4.5) using triangular elements. The four structure section grids are shown in Figures 4.6 to 4.9. In order to accurately model the ground thermal response to surface temperature oscillations, the procedure discussed below was followed for all cases analyzed.

Each freezing index case and profile was initialized by performing a TDHC analysis which began when the surface temperature (T_s) was equal to the mean annual surface temperature T_m on day (365/4 + ϕ) from January 1. The initial ground temperature profile for this day equals T_m for all nodes except 81 and 82 which are T_m plus one-degree Fahrenheit. The analysis runs for one year using a time step of two days. The temperature profile obtained when T_s equals T_m minus the amplitude of temperature variation (T_a) on day January 1 plus ϕ is input into a subsequent analysis where time steps are reduced to one day through the remaining freezing season and the duration of the thawing period.

In order to include the effects of radiation and convection at the surface in the spring months, the radiant heat flux (Btu/hr ft^2) and the convection coefficient (Btu/hr ft^2 °F) are input as step functions each month until thawing is complete. Each month the problem is initialized with the final temperature profile from the preceding month and the appropriate convective and heat flux values for that month. An example of the stepwise input for a freezing index of 1000°F-days and a fine-grained subgrade with a four inch surface and twelve inch base is shown in Appendix C.

4.3.6 RESULTS

Based on results from previous studies and observations of thawing it was determined that some indication of a) when thawing reached the bottom of the base course; b) when thawing proceeded a small amount into the subgrade (four inches was selected); and c) when thawing was complete should be estimated. These cases are shown in Figure 4.10. The date given in days after January 1 for these times as well as the day when the air temperature went above 32°F for all structure and freezing cases are shown in Table 4.28. In

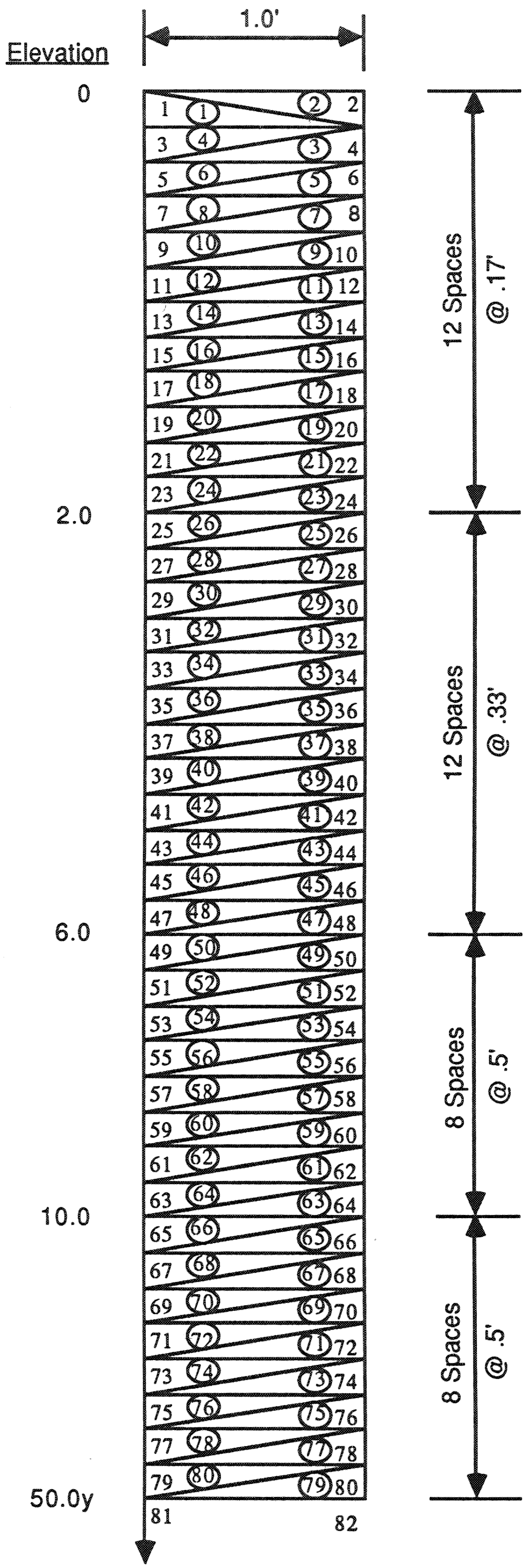


Figure 4.5 Generalized Finite Element Grid

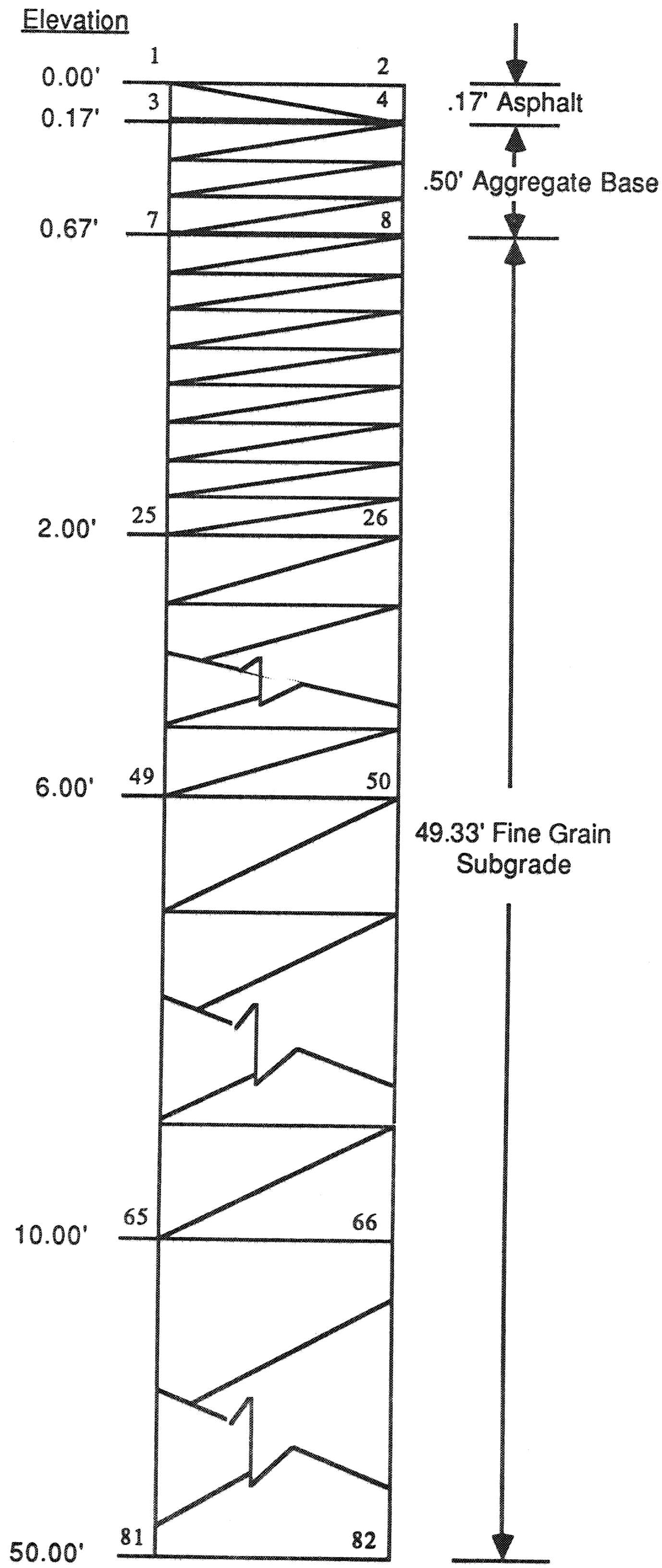


Figure 4.6 Finite Element Mesh for Section 1

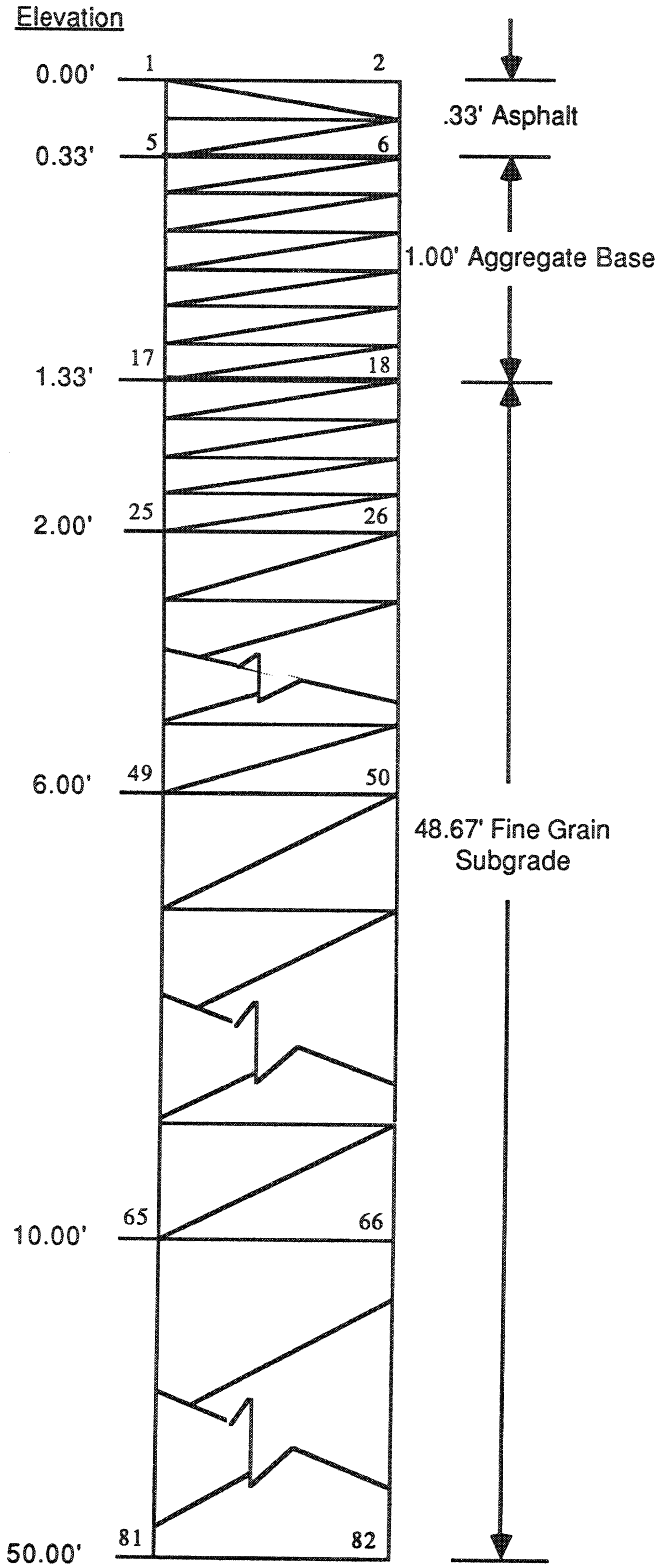


Figure 4.7 Finite Element Mesh for Section 2

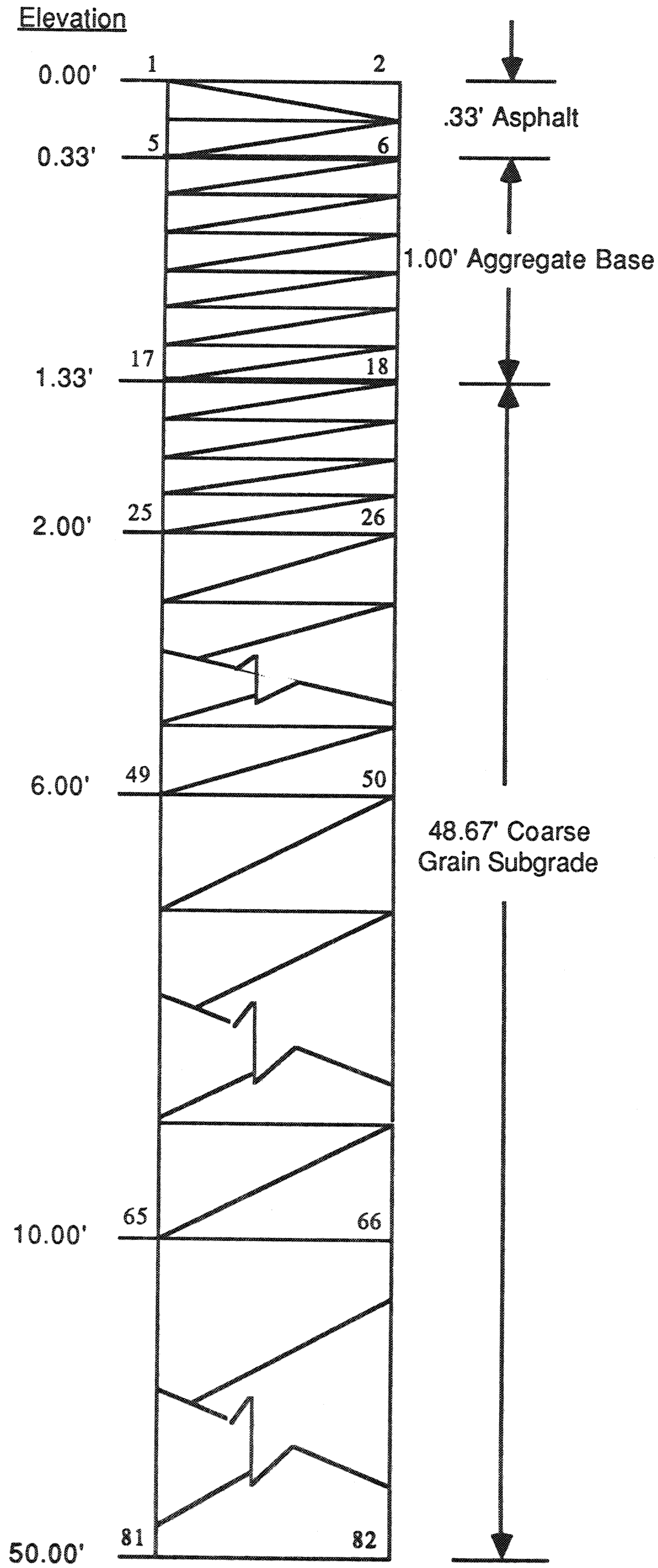


Figure 4.8 Finite Element Mesh for Section 3

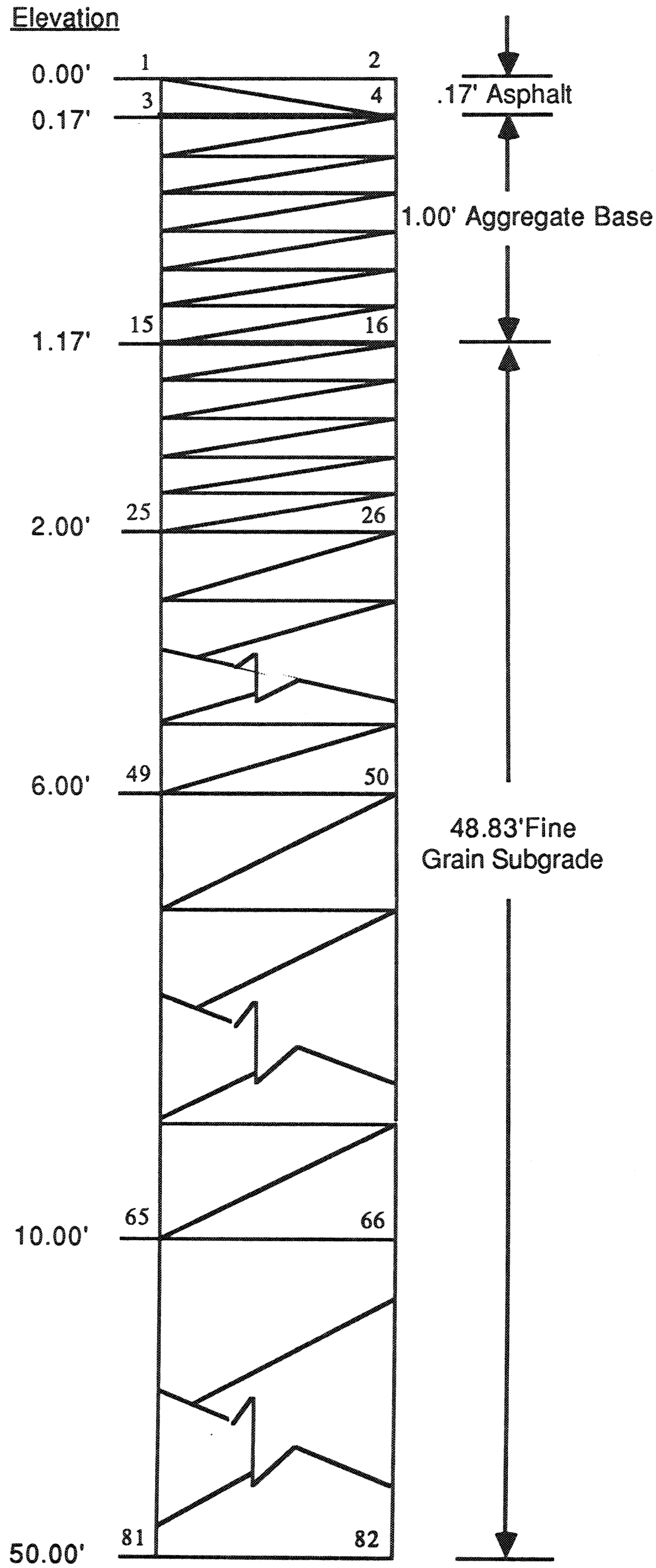


Figure 4.9 Finite Element Mesh for Section 4

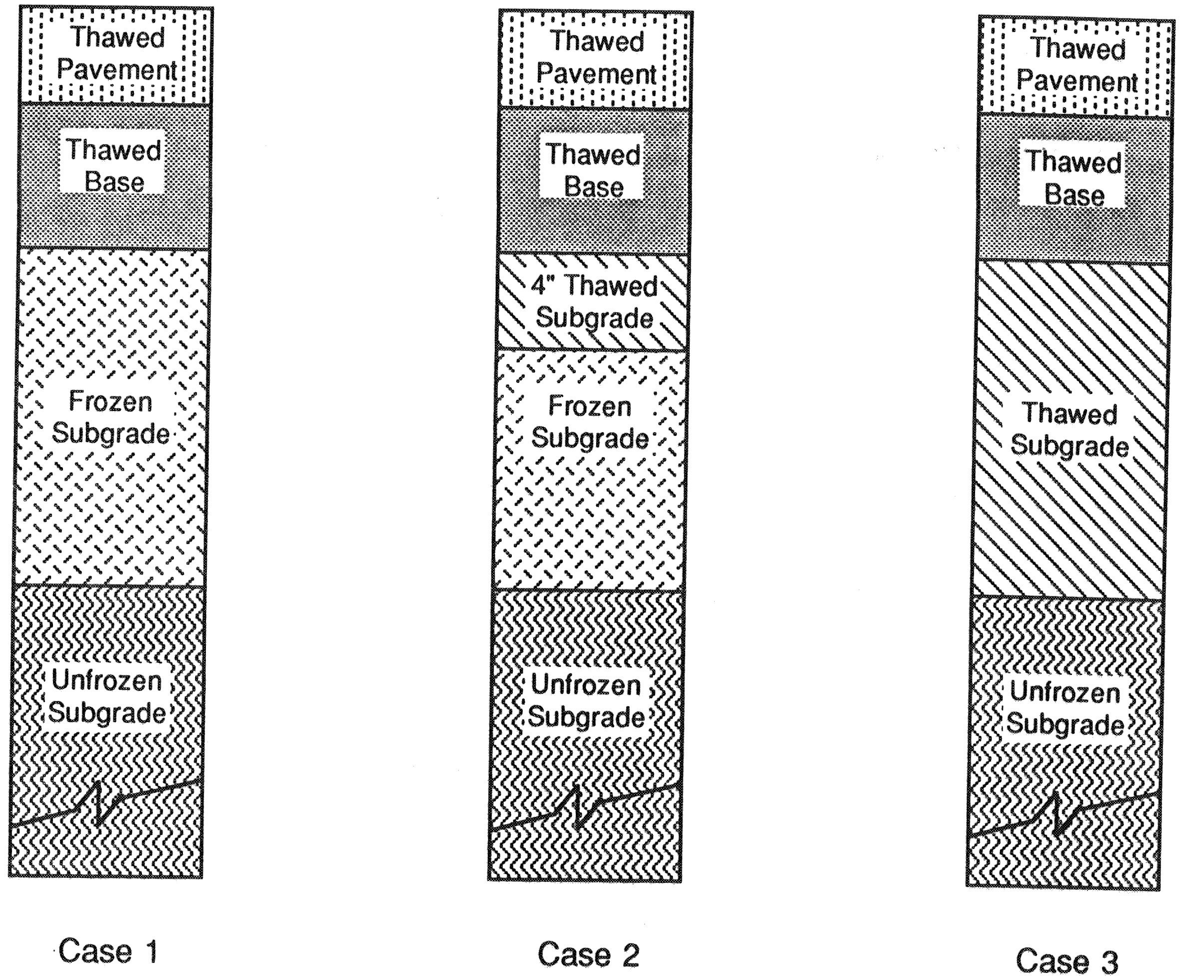


Figure 4.10 Thawing Cases Evaluated from Results of TDHC

Table 4.28 Advancement of the Thawing Plane Referenced to an Air Temperature = 32°F

Freezing Index (°F-days)	Day Air Temp = 32°F [b]	Day Base Thawed [b]	Day Thaw is 4 in. Below Base [b]	Day of Complete Thaw [b]	Duration of Thaw (days)
2/6/592 fine [a]					
400	63	61	61	76	13
500	66	65	70	91	25
750	74	68	80	101	27
1000	80	78	80	102	22
1250	80	74	81	113	33
1500	84	79	87	120	36
2000	88	89	95	128	40
4/12/584 fine					
400	63	62	66	73	10
500	66	70	80	87	21
750	74	76	78	96	22
1000	80	77	82	102	22
1250	80	79	89	104	24
1500	84	85	87	113	29
2000	88	92	95	122	34
4/12/584 coarse					
400	63	61	-	61	-
500	66	68	80	87	21
750	74	75	82	94	20
1000	80	80	84	98	18
1250	80	81	87	99	19
1500	84	82	86	107	23
2000	88	92	93	124	36

Table 4.28 Advancement of the Thawing Plane Referenced to an Air Temperature = 32°F (Cont.)

Freezing Index (°F-day)	Day Air Temp = 32°F [b]	Day Base Thawed [b]	Day Thaw is 4 in. Below Base [b]	Day of Complete Thaw [b]	Duration of Thaw (days)
2/12/586 fine					
500	66	62	70	82	18
1000	80	78	86	106	26

Notes: (a) Pavement section:
2" Asphalt concrete pavement
6" Aggregate Base
592" Fine-grained subgrade

(b) The values shown in these columns represent the calendar day number (e.g. day = 1 is January 1 and day = 63 is March 4 for calendar year 1985) addition, the duration of thawing for the three thawing cases is shown in the table.

The thawing index is a measure of the temperature input and duration required to cause thawing. Based on a traditional reference temperature of 32°F, the thawing index for the three cases of thawing for all structural sections and freezing cases was calculated (Table 4.29).

Due to the net incoming heat flux at the ground surface during spring, the surface temperature (T_s) is greater than the air temperature. The surface temperature for all cases when the air temperature is 32°F is shown in Table 4.30 as well as the air temperature and day when the surface temperature reaches 32°F and thawing actually begins.

The results obtained suggest relatively consistent air temperatures between 29 and 30°F when thawing actually begins with the exception of the lower freezing index cases of 400 and 500°F-days. The anomalies are due to the fact that temperatures are very close to 32°F when the first heat flux step is introduced. Since the air temperatures when pavement thawing actually begins are typically between 29 and 30°F, the data were reanalyzed based on these reference thawing temperatures.

Tables 4.31 and 4.32 show the day when air temperatures reach 29 and 30°F respectively, the day when thawing has progressed to the bottom of the base, four inches into the subgrade and through the originally frozen material. The duration of thawing based on these reference temperatures is also given. Thawing indices for all levels of thawing noted above were calculated based on 29 and 30°F. These are shown in Tables 4.33 and 4.34.

Plots of the thawing index as a function of freezing index for each structural section for 29, 30 and 32°F based thawing indices are included in Appendix D. The fine-grained subgrade cases generally suggest a good correlation of these variables with R squared values greater than 0.9. The coarse grained subgrade results were not as satisfactory with R squared values much lower. The linear equations representing the least squares fit of the data and the R squared values for all cases are shown in Table 4.35. In addition, the results for all fine-grained sections were combined. The results for 29, 30 and 32°F based thawing indices are shown in Figures 4.11 to 4.13.

Table 4.29 Thawing Indices for Three Thawing Cases Based on 32°F

Freezing Index (°F-days)	Thawing Index for Base (32°F datum) (°F-days)	Thawing Index for 4 in. into Subgrade (32°F datum) (°F-days)	Thawing Index For Total Thaw (32°F datum) (°F-days)
2/6/592 fine			
400	-	-	30
500	-	4	113
750	-	10	153
1000	-	2	117
1250	-	2	280
1500	-	5	341
2000	1	20	430
4/12/584 fine			
400	-	2	18
500	4	34	79
750	2	5	103
1000	-	2	117
1250	-	28	144
1500	5	5	228
2000	9	20	318
4/12/584 coarse			
400	-		
500	1	36	80
750	1	15	84
1000	-	7	81
1250	2	18	100
1500	-	3	148
2000	11	11	354
2 /12/586 fine			
500	-	4	46
1000	-	12	160

Note: (a) Pavement section:
2" Asphalt concrete pavement
6" Aggregate Base
592" Fine grained subgrade

Table 4.30 Surface and Air Temperatures

Freezing Index (°F-days)	Air Temperature When Surface Temp = 32°F (°F)	Surface Temperature When Air Temp = 32°F (°F)
2/6/592 fine[a]		
400	31.4	-
500	30.5	34.0
750	29.3	34.0
1000	29.4	34.5
1250	28.7	34.4
1500	29.4	34.3
2000	29.4	34.0
4/12/584 fine		
400	31.4	34.1
500	30.5	34.0
750	29.5	34.0
1000	29.4	34.5
1250	29.4	34.4
1500	29.4	34.3
2000	29.4	34.0
4/12/584 coarse		
400		
500	31.5	34.0
750	29.6	33.8
1000	29.4	34.4
1250	28.3	35.1
1500	29.4	34.4
2000	29.8	33.9

Notes: (a) Pavement section:
2" Asphalt concrete pavement
6" Aggregate Base
592" Fine-grained subgrade

Table 4.31 Advancement of the Thawing Plane Referenced to an Air Temperature = 29°F

Freezing Index (°F-days)	Day Air Temp = 29°F[b]	Day Base Thawed[b]	Day Thaw is 4 in. Below Base[b]	Day of Complete Thaw[b]	Duration of Thaw (days)
2/6/592 fine[a]					
400	52	61	61	76	24
500	56	65	70	91	35
750	65	68	80	101	36
1000	71	78	80	102	31
1250	72	74	81	113	41
1500	76	79	87	120	44
2000	81	89	95	128	47
4/12/584 fine					
400	52	62	66	73	21
500	56	70	80	87	31
750	65	76	78	96	31
1000	71	77	82	102	31
1250	72	79	89	104	32
1500	76	85	87	113	37
2000	81	92	95	122	41
4/12/584 coarse					
400	52	61	-	61	9
500	56	68	80	87	31
750	65	75	82	94	29
1000	71	80	84	98	27
1250	72	81	87	99	27
1500	76	82	86	107	31
2000	81	92	93	124	43

Table 4.31 Advancement of the Thawing Plane Referenced to an Air Temperature = 29°F (Cont.)

Freezing Index (°F-days)	Day Air Temp = 29°F [b]	Day Base Thawed [b]	Day Thaw is 4 in. Below Base [b]	Day of Complete Thaw [b]	Duration of Thaw (days)
2/12/586 fine					
500	56	62	70	82	26
1000	71	78	86	106	35

Notes: (a) Pavement section:
2" Asphalt concrete pavement
6" Aggregate Base
592" Fine-grained subgrade

(b) The values shown in these columns represent the calendar day number (e.g. day = 1 is January 1 and day = 63 is March 4 for calendar year 1985).

Table 4.32 Advancement of the Thawing Plane Referenced to an Air Temperature = 30°F

Freezing Index (°F-days)	Day Air Temp = 30°F [b]	Day Base Thawed [b]	Day Thaw is 4 in. Below Base [b]	Day of Complete Thaw [b]	Duration of Thaw (days)
2/6/592 fine [a]					
400	56	61	61	76	20
500	60	65	70	91	31
750	68	68	80	101	33
1000	74	78	80	102	28
1250	75	74	81	113	38
1500	79	79	87	120	41
2000	83	89	95	128	45
4/12/584 fine					
400	56	62	66	73	17
500	60	70	80	87	27
750	68	76	78	96	28
1000	74	77	82	102	28
1250	75	79	89	104	29
1500	79	85	87	113	34
2000	83	92	95	122	39
4/12/584 coarse					
400	56	61	-	61	5
500	60	68	80	87	27
750	68	75	82	94	26
1000	74	80	84	98	24
1250	75	81	87	99	24
1500	79	82	86	107	28
2000	83	92	93	124	41

Table 4.32 Advancement of the Thawing Plane Referenced to an Air Temperature = 32°F (Cont.)

Freezing Index (°F-days)	Day Air Temp = 32°F (b)	Day Base Thawed (b)	Day Thaw is 4 in. Below Base (b)	Day of Complete Thaw (b)	Duration of Thaw (days)
2/12/586 fine					
500	60	62	70	82	22
1000	74	78	86	106	32

Notes: (a) Pavement section :
2" Asphalt concete pavement
6" Aggregate Base
592" Fine-grained subgrade

(b) The values shown in the columns represent the calendar day number (e.g. day = 1 is January 1 and day = 63 is March 4 for calendar year 1985)

Table 4.33 Thawing Indices for Three Thawing Cases Based on 29°F

Freezing Index (°F-days)	Thawing Index for Base (29°F datum) (°F-days)	Thawing Index for 4 in. into Subgrade (29°F datum) (°F-days)	Thawing Index for Total Thaw (29°F datum) (°F-days)
2/6/592 fine[(a)]			
400	14	14	88
500	16	34	207
750	6	50	258
1000	13	23	214
1250	4	27	401
1500	4	33	467
2000	20	58	561
4/12/584 fine			
400	16	31	68
500	33	95	160
750	26	35	193
1000	11	27	202
1250	16	73	232
1500	25	34	409
2000	38	58	461
4/12/584 coarse			
400	14	14	14
500	25	95	161
750	22	58	163
1000	21	40	156
1250	24	58	176
1500	13	30	237
2000	40	43	479
2/12/586 fine			
500	7	43	112
1000	13	48	256

Notes: (a) Pavement section:
2" Asphalt concrete pavement
6" Aggregate Base
592" Fine-grained subgrade

Table 4.34 Thawing Indices for Three Thawing Cases Based on 30°F

Freezing Index (°F-days)	Thawing Index for Base (30°F datum) (°F-days)	Thawing Index for 4 in. into Subgrade (30°F datum) (°F-days)	Thawing Index for Total Thaw (30°F datum) (°F-days)
2/6/592 fine[a]			
400	5	5	65
500	6	20	172
750	0	34	220
1000	6	12	173
1250		18	359
1500	0	21	423
2000	12	43	520
4/12/584 fine			
400	7	17	48
500	20	72	129
750	15	22	160
1000	4	16	172
1250	7	55	211
1500	15	22	367
2000	25	43	422
4/12/584 coarse			
400	5	5	5
500	13	72	130
750	12	40	134
1000	12	27	129
1250	14	42	148
1500	6	19	205
2000	25	30	434
2/12/586 fine			
500	2	27	86
1000	6	33	221

Notes: (a) Pavement section
2" Asphalt concrete pavement
6" Aggregrate Base
592" Fine-grained subgrade

Table 4.35 Regression Analysis for Thawing Index as a Function of Freezing Index

Case	Regression Equation	Correlation Coefficient (R)	R^2
Section 1; 29°F	TI = 18.350 + 0.280 FI	.956	.915
Section 2; 29°F	TI = 0.794 + 0.232 FI	.952	.906
Section 3; 29°F	TI = -35.332 + 0.221 FI	.895	.801
Section 1, 2, 4; 29°F	TI = 4.154 + 0.259 FI	.930	.865
Section 1; 30°F	TI = -10.051 + 0.271 FI	.956	.914
Section 2; 30°F	TI = -21.178 + 0.224 FI	.961	.924
Section 3; 30°F	TI = -48.793 + 0.206 FI	.899	.808
Section 1, 2, 4; 30°F	TI = -20.398 + 0.250 FI	.936	.877
Section 1; 32°F	TI = -46.439 + 0.242 FI	.961	.923
Section 2; 32°F	TI = -35.974 + 0.170 FI	.976	.952
Section 3; 32°F	TI = -59.660 + 0.172 FI	.864	.747
Section 1, 2, 4; 32°F	TI = -44.449 + 0.208 FI	.921	.848

Table 4.35 Regression Analysis for Duration of Thawing as a Function of Freezing Index (Cont.)

Case	Regression Equation	Correlation Coefficient (R)	R^2
Section 1; 29°F	D = -43.598 + 27.141 log FI	.872	.760
Section 2; 29°F	D = -32.341 + 21.704 log FI	.890	.792
Section 3; 29°F	D = -60.133 + 29.780 log FI	.752	.565
Section 1, 2, 4; 29°F	D = -39.771 + 24.985 log FI	.834	.696
Section 1; 30°F	D = -54.133 + 29.634 log FI	.892	.795
Section 2; 30°F	D = -42.876 + 24.198 log FI	.908	.824
Section 3; 30°F	D = -70.668 + 32.273 log FI	.774	.599
Section 1, 2, 4; 30°F	D = -50.496 + 27.541 log FI	.858	.736
Section 1; 32°F	D = -67.846 + 32.333 log FI	.896	.802
Section 2; 32°F	D = -56.589 + 26.897 log FI	.916	.840
Section 3; 32°F	D = -35.788 + 19.381 log FI	.628	.394
Section 1, 2, 4; 32°F	D = -63.760 + 30.088 log FI	.883	.779

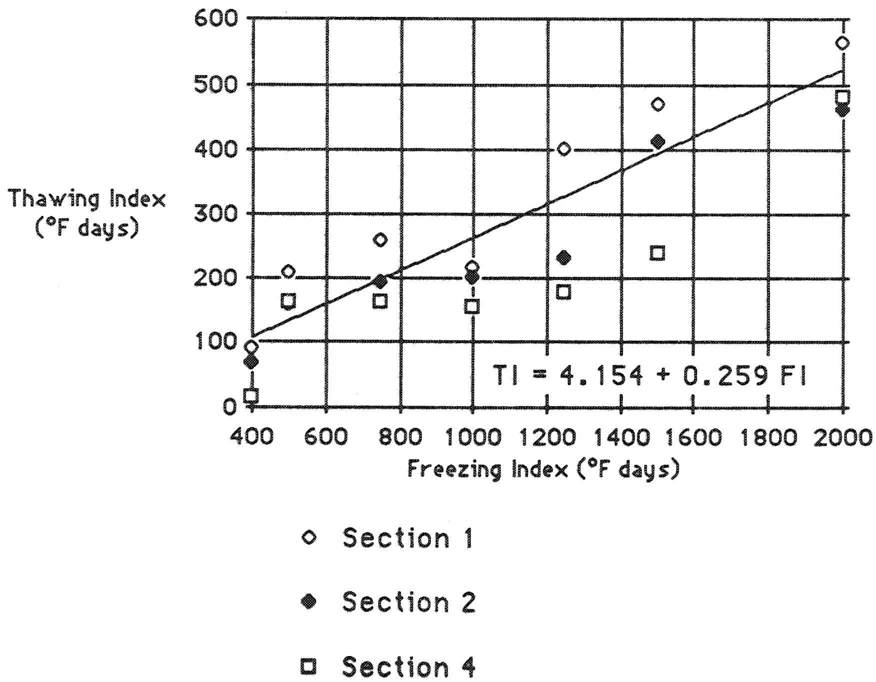


Figure 4.11 Thawing Index (based on 29°F) versus Freezing Index for all Fine Grain Subgrade Cases

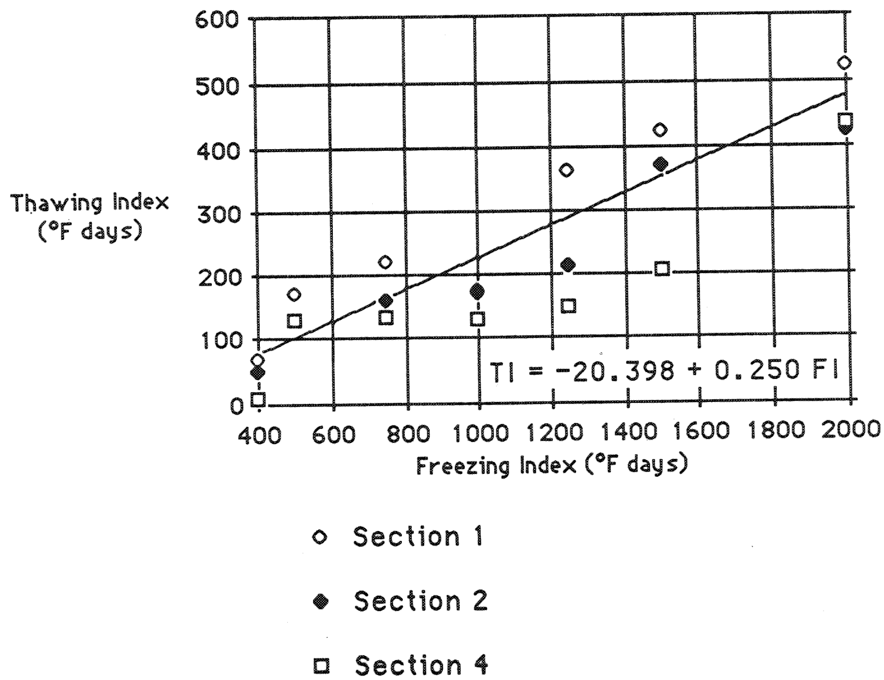


Figure 4.12 Thawing Index (based on 30°F) versus Freezing Index for all Fine Grain Subgrade Cases

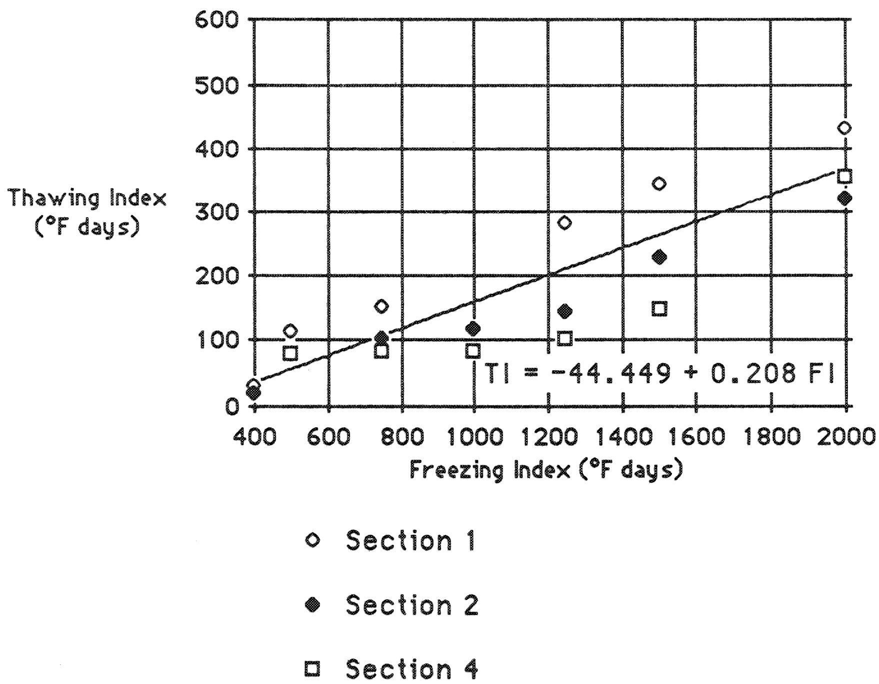


Figure 4.13 Thawing Index (based on 32°F) versus Freezing Index for all Fine Grain Subgrade Cases

In addition, the correlation of freezing index and duration of thaw based on 29, 30 and 32°F was considered. In general, the results were not as significant as the relationship of freezing and thawing index. The best fit was found relating duration of thaw to the logarithm of the freezing index. The resulting equations and R squared values for all cases are shown in Table 4.36. Plots of all sections are included in Appendix D. Figures 4.14 through 4.16 show the results of all fine grained sections combined for 29, 30, and 32°F based thaw durations. Here again, the coarse-grained results were less consistent than the fine-grained results. A possible explanation for poor results from the coarse-grained section may be that the low latent heat and high thermal conductivity result in thawing that is sufficiently rapid to cause the finite element program to be unstable for time steps of one day.

In addition, the TDHC analyses generated freezing depths for each profile and freezing index case analyzed. The results are shown in Table 4.37. Also shown in the table are freezing depths computed using the Multilayered Modified Berggren analysis using a surface "n" factor of 1.0 which is comparable to the TDHC input. In general, the Modified Berggren results yield greater freezing depths than the TDHC analysis. These results are shown graphically in Figure 4.17. While Modified Berggren depths are typically greater, a good correlation exists for all sections between the depth predicted using both analysis techniques (Figure 4.17). The regression equations for the relationship of TDHC freezing depth and Modified Berggren freezing depth are given in Table 4.37 for each pavement section individually and all cases combined.

Table 4.36 Regression Analysis for TDHC Depth of Freezing as a Function of Modified Berggren Depth of Freezing

Case	Regression Equation	Correlation Coefficient (R)	R^2
Section 1	DF = 0.555 + 0.750 MB	.996	.993
Section 2	DF = 0.393 + 0.808 MB	.993	.986
Section 3	DF = 0.102 + 0.812 MB	.993	.986
Section 1,2,3,4	DF = 0.436 + 0.773 MB	.989	.978

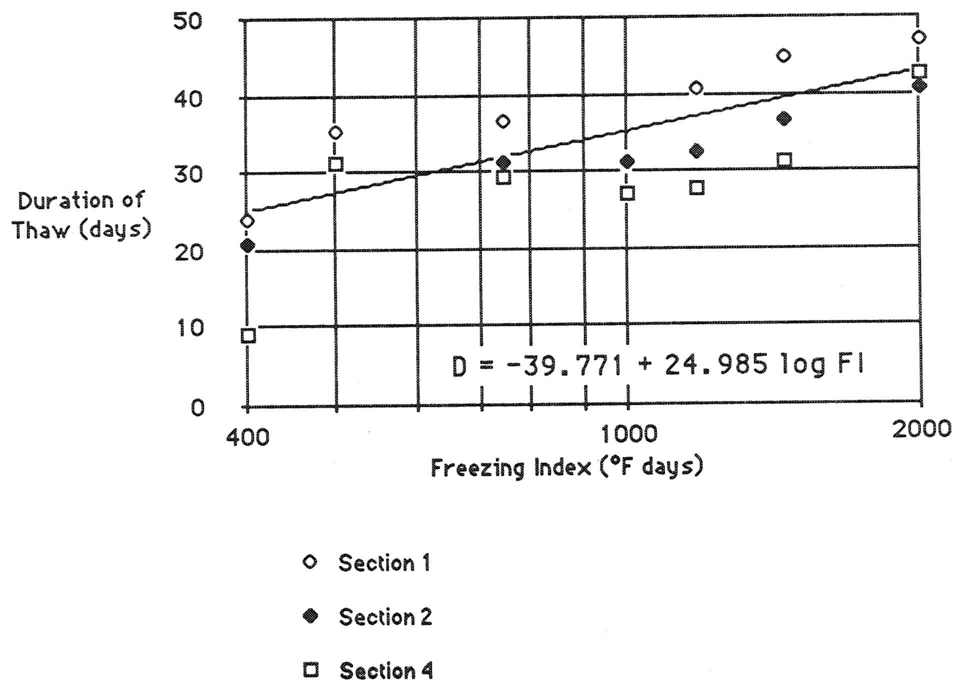


Figure 4.14 Duration of Thaw (based on 29°F) versus log Freezing Index for all Fine Grain Subgrade Cases

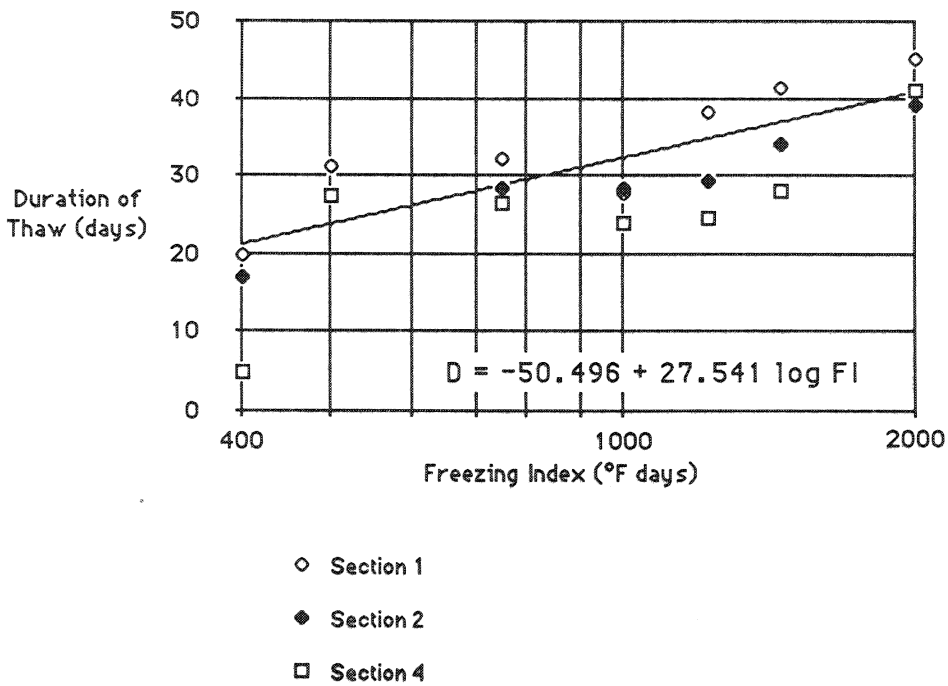


Figure 4.15 Duration of Thaw (based on 30°F) versus log Freezing Index for all Fine Grain Subgrade Cases

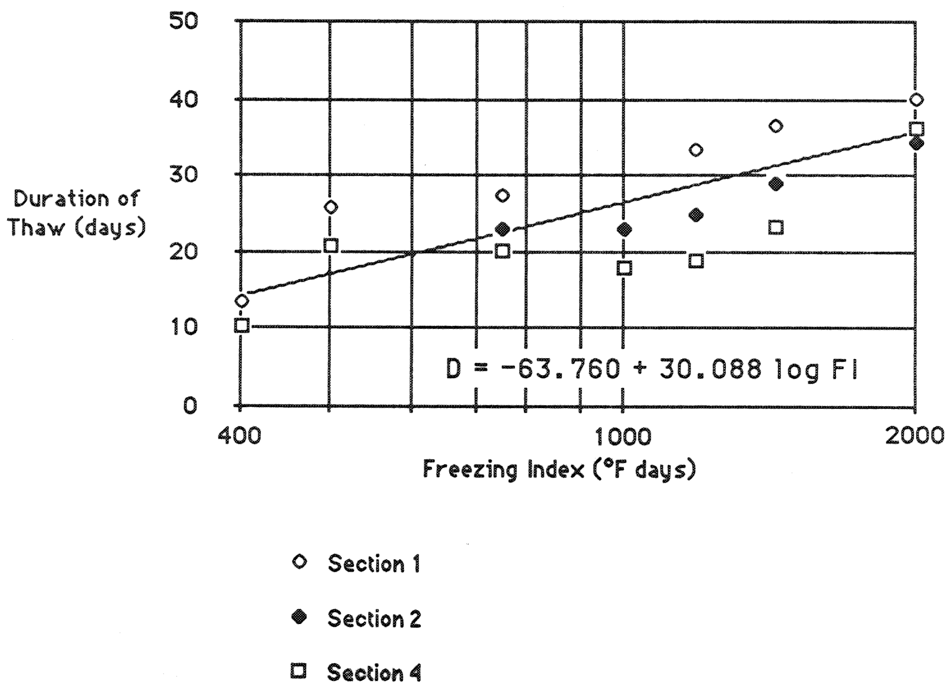


Figure 4.16 Duration of Thaw (based on 32°F) versus log Freezing Index for all Fine Grain Subgrade Cases

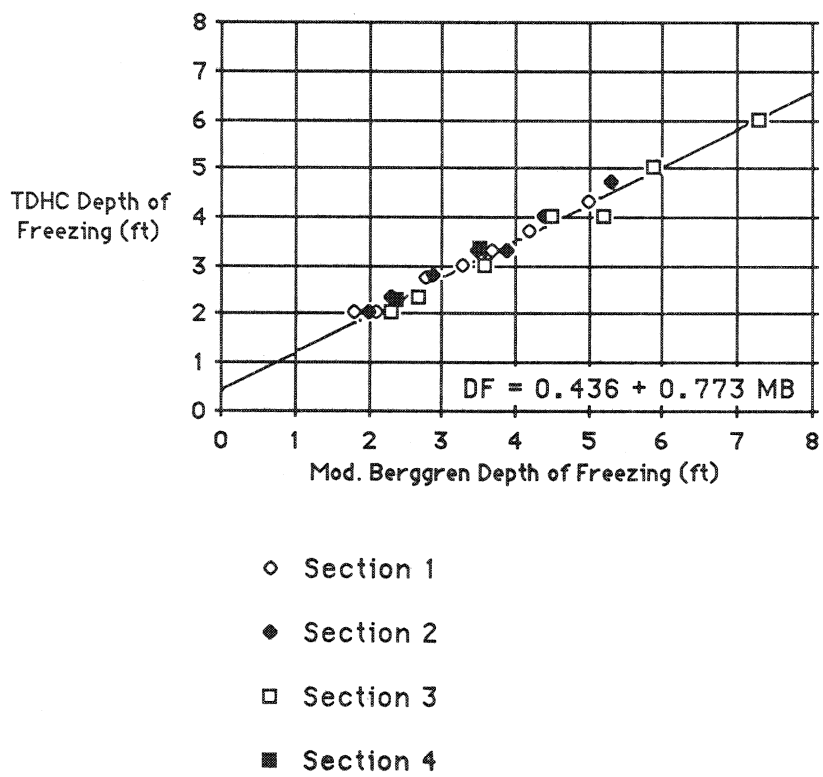


Figure 4.17 TDHC Depth of Freeze versus Modified Berggren Depth of Freeze for all Cases

Table 4.37 Freezing Depths Estimated from TDHC and Multilayered Modified Berggren

Freezing Index (°F-days)	Depth of Freeze TDHC (ft)	Mod. Berg Depth of Freeze (ft)
2/6/592 fine[a]		
400	2.0	1.8
500	2.0	2.1
750	2.7	2.8
1000	3.0	3.3
1250	3.3	3.7
1500	3.7	4.2
2000	4.3	5.0
4/12/584 fine		
400	2.0	2.0
500	2.3	2.3
750	2.8	2.9
1000	3.3	3.5
1250	3.3	3.9
1500	4.0	4.4
2000	4.7	5.3
4/12/584 coarse		
400	2.0	2.3
500	2.3	2.7
750	3.0	3.6
1000	4.0	4.5
1250	4.0	5.2
1500	5.0	5.9
2000	6.0	7.3
2/12/586 fine		
500	2.3	2.3
1000	3.3	3.5

Notes: (a) Pavement profile:
2" Asphalt concrete pavement
6" Aggregate Base
592" Fine-grained subgrade

CHAPTER 5.0
DEVELOPMENT OF GUIDELINES

5.1 INTRODUCTION

Based on the literature review and analysis conducted in this study, the following guidelines will be presented in this chapter:

(a) where to apply load restrictions,

(b) the magnitude of the load restrictions, and

(c) when to apply and remove load restrictions.

The guidelines are general in scope and not intended to be "absolute" being as the nature of the problem is site specific.

5.2 GUIDELINES FOR WHERE TO APPLY LOAD RESTRICTIONS

The analysis presented in Chapter 4.0 (specifically Tables 4.6 through 4.12) was based on the assumption that pavement response (deflection and strain) during the spring thaw should be limited to those estimated for summer conditions. The way to achieve equal pavement response is to reduce allowable axle loads (or individual tire loads). Further, many agencies have the capability to measure pavement surface deflections with equipment such as the Benkelman Beam, Dynaflect, or Falling Weight Deflectometer. Thus for both the fine and coarse-grain subgrade cases, the percent increase in surface deflection was calculated for summer to complete spring thaw for both single tire - single axle and dual tires - single axle conditions. These deflection increases were matched with the associated load reduction percentages with a summary shown in Table 5.1 and plotted in Figure 5.1.

An examination of Figure 5.1 reveals that pavement sections which have surface deflections 45 to 50 percent higher during the spring thaw than summer values are candidates for load restrictions. Clearly, this is not an absolute criterion for selecting pavement sections to receive load restrictions. Site specific conditions could significantly alter the deflection increase threshold. For example, a relatively "thin" or "weak" pavement section may have relatively high summer deflections. Thus spring thaw deflections may need to increase much less than the threshold level of

Table 5.1 Surface Deflection Increases (from Summer to Complete Spring Thaw Case) and Associated Load Reductions

Pavement Structural Section		Single Tire - Single Axle		Dual Tires - Single Axle	
Surface Thickness (in)	Base Thickness (in)	Surface Deflection Increase (a) (Percent)	Load Reduction (b) (Percent)	Surface Deflection Increase (a) (Percent)	Load Reduction (b) (Percent)
Fine-grained Subgrade					
2	6	84	31	114	45
2	12	86	50	119	55
4	6	25	-	38	5
4	12	29	22	41	3
Coarse-grained Subgrade					
2	6	44	31	67	41
2	12	46	39	68	42
4	6	10	-	29	14
4	12	12	19	31	25

Notes: (a) Increase in pavement surface deflection from summer to complete spring thaw.

(b) Load reductions from Tables 4.6 and 4.7 for the subgrade vertical strain response criterion.

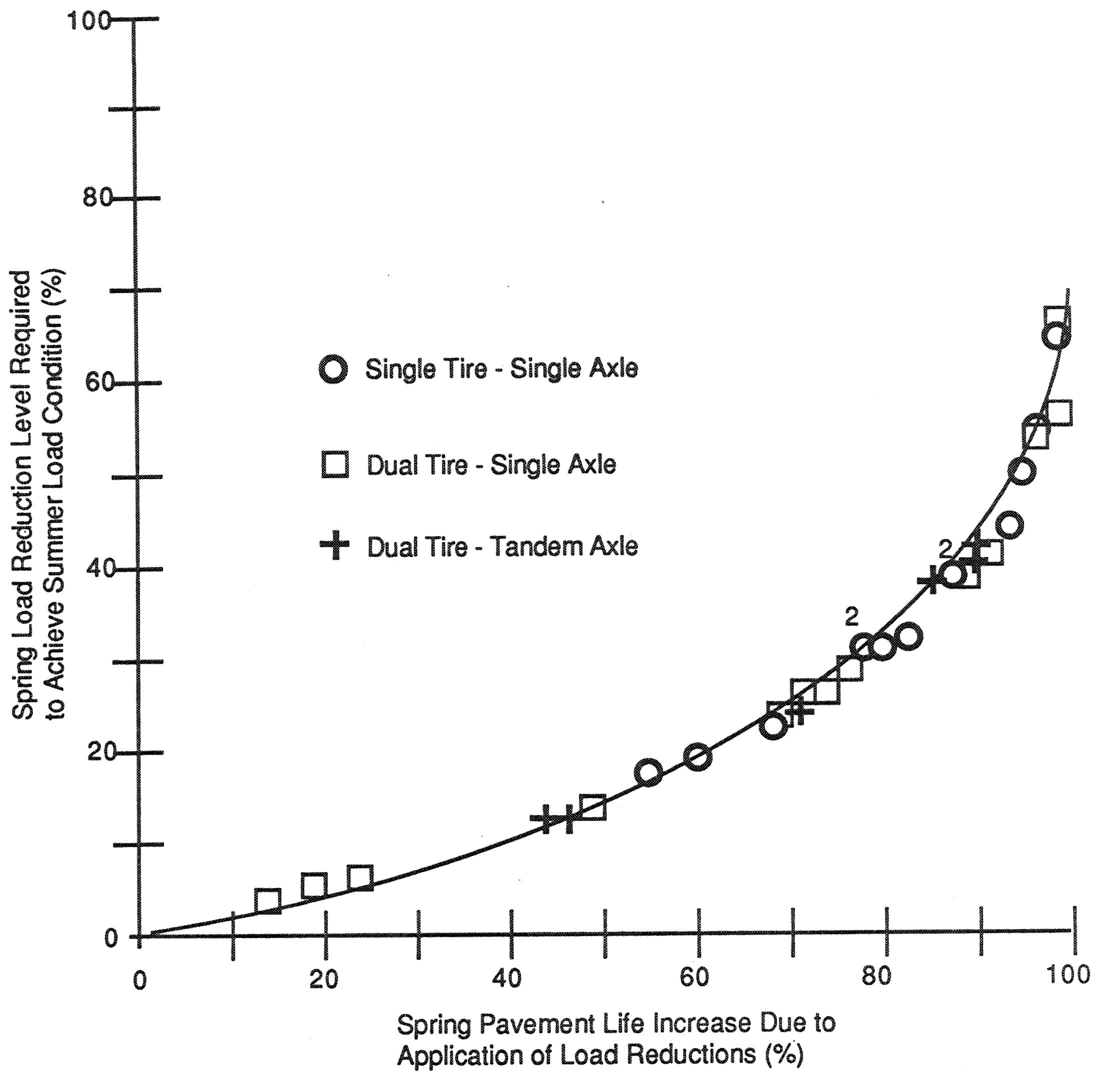


Figure 5.2 Increase in Pavement Life Due to Application of Load Reductions (Based on Rutting Failure Criterion).

45 to 50 percent to necessitate load reductions. Surface deflection increases of less than 45 percent result in load reductions of about 25 to 30 percent or less which is in agreement with the work by Connor [2.20] as originally described with Figure 2.12.

Other criteria which should be considered in selecting pavements for load restrictions include:

(a) surface thickness,
(b) pavements on fine-grained subgrades, and
(c) local experience relating to observed moisture and pavement distress.

If the surface thickness of a pavement is about two inches or less and in an area where the FI is greater than 400°F-days (i.e., modest depth of freezing), then this suggests that load restrictions should be considered.

Pavements on fine-grained subgrades such as silts and clays (Unified Soil classifications ML, MH, CL and CH) are candidates for load restrictions. Again, the depth of ground freezing is important.

The observed site specific drainage is significant in assessing the need for load restrictions. Items such as poor drainage from side ditches, available ground water, high winter precipitation, and snow removal policies should be considered. For example, pavement in cold but dry locations probably will not need any type of restrictions.

Another criterion to use for selecting load restriction locations involves observation of pavement distress such as fatigue (alligator) cracking and rutting. If these distress types primarily occur during the spring thaw, load restrictions are needed if options such as strengthening the overall pavement structure are not possible (or appropriate).

Overall, local experience relating to the conditions associated with the performance an individual agency's road network is important. Clearly, various nondestructive pavement response measures such as surface deflection can help define the potential pavement weakening during the thaw period; however, the experience of agency personnel should be used to the fullest extent possible.

5.3 GUIDELINES FOR LOAD RESTRICTION MAGNITUDE

From Chapter 3.0 (specifically Table 3.7), the range of load reductions used by the summarized agencies range from about 20 to 60 percent. An average load reduction for seven locations (individual state areas) is approximately 44 percent (standard deviation of about 8 percent). This suggests that reducing the load on individual axles (or tires) by about 40 to 50 percent reduces the associated pavement response to levels that preclude or reduce the resulting pavement distress to acceptable levels.

To further examine the amount of load reduction needed, Figures 5.2 and 5.3 were developed. Figures 5.2 is a plot of load reduction (percent) versus the increase in pavement life due to the application of load restrictions (percent). The load reduction percentages were obtained from Tables 4.6, 4.7, 4.8, 4.11, and 4.12 in Chapter 4.0 (for the vertical strain at the top of the subgrade cases only). The increase in pavement life was obtained from Tables 4.14, 4.17, 4.18, 4.19, and 4.20. To determine the increase in pavement life from these tables, the negative change in pavement life (based on the rutting failure criterion) is eliminated due to load reductions, thus increasing the potential pavement life. All three tire-axle configurations were used. This curve contains data points for both the two and four inch thick surface courses and both fine and coarse-grain subgrades for the rutting failure criterion (a wide range of conditions). Undoubtedly, different failure criteria would tend to shift the curve.

The results based on Figure 5.2 show that as the load reduction percentage is increased the associated pavement life is increased (as one would expect). An increasing slope is noted for load reductions greater than about 20 percent. The following potential pavement life increases result as a function of load reduction (starting with a load reduction of 20 percent):

Load Reduction (%)	Pavement Life Increases (%)
20	62
30	78
40	88
50	95

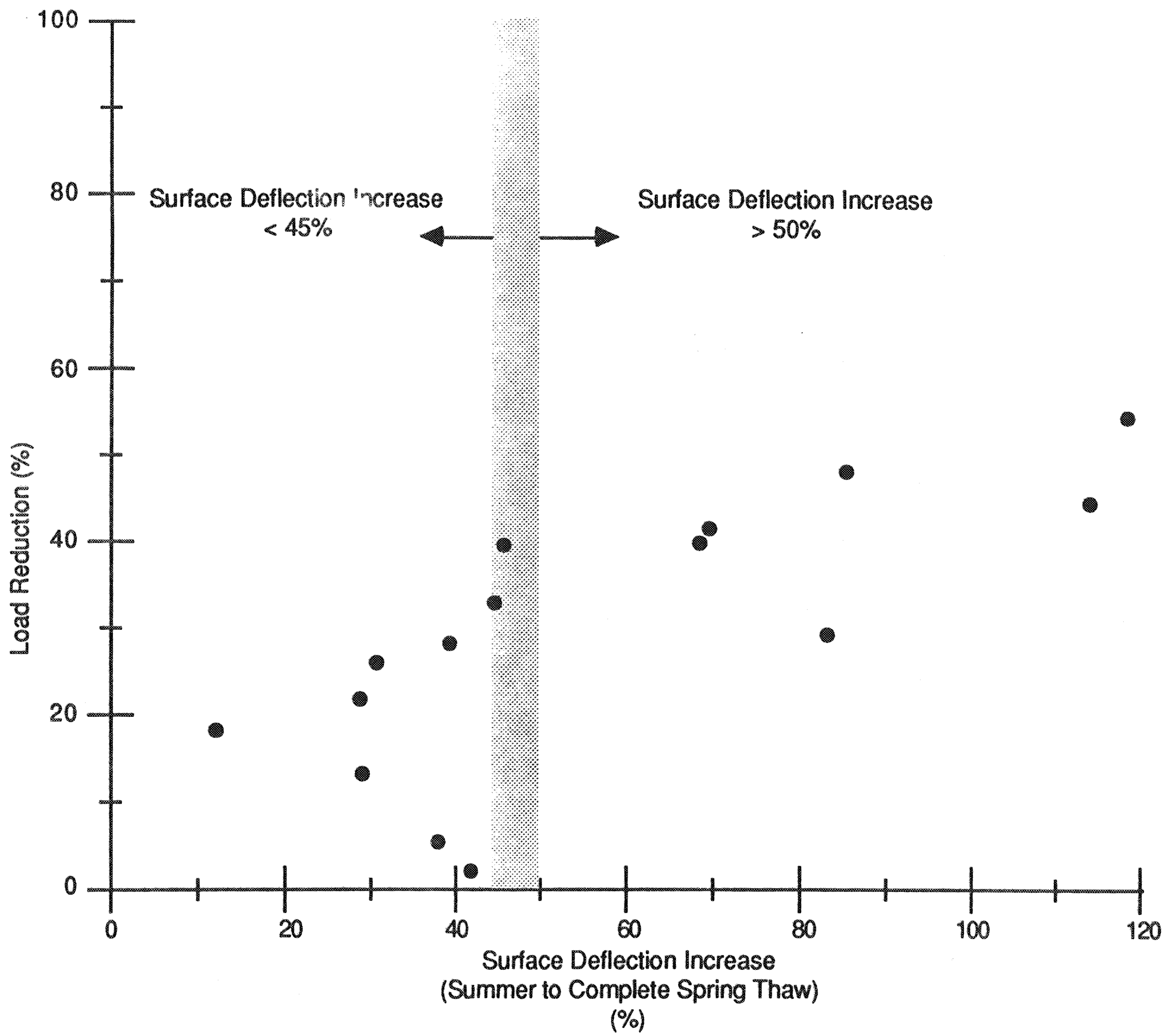


Figure 5.1. Development of Surface Deflection for Locating Pavements Requiring Load Restrictions.

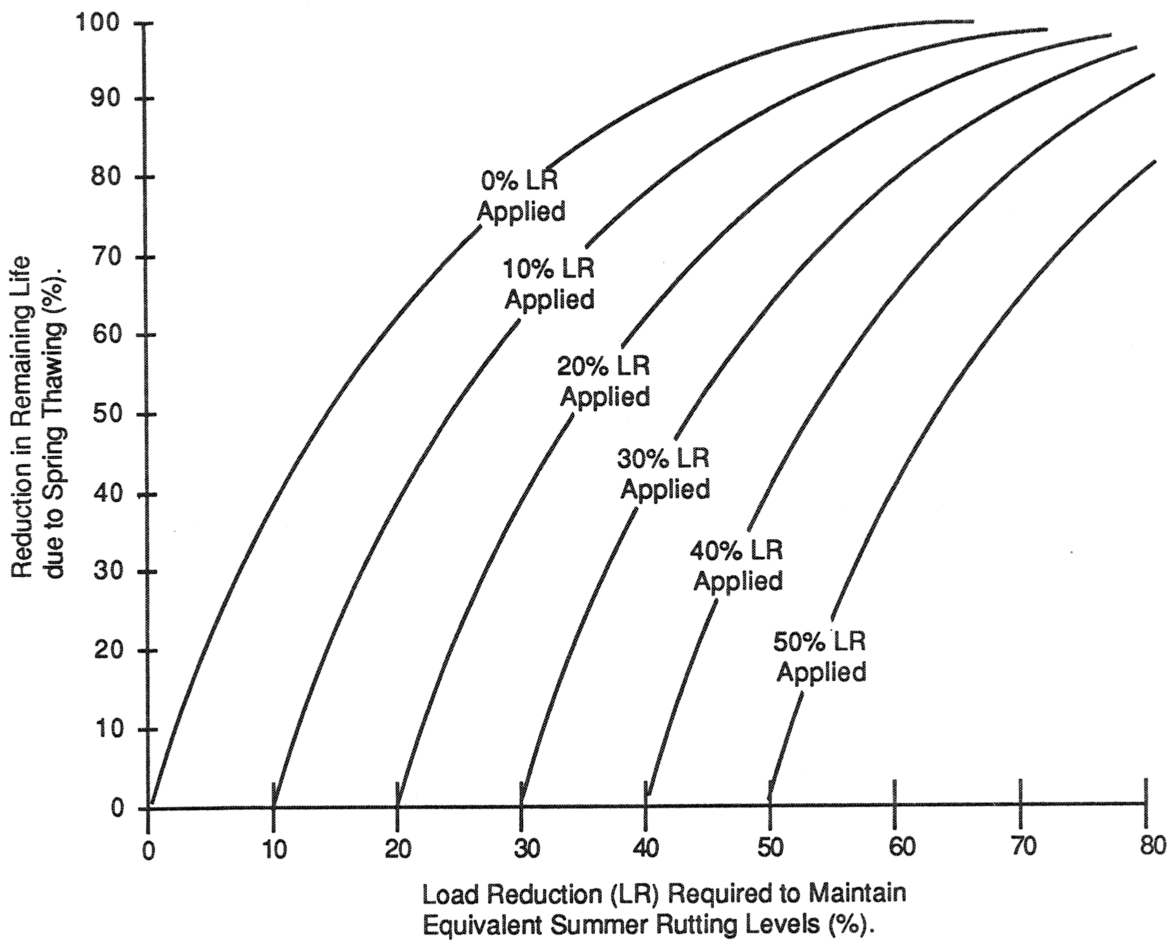


Figure 5.3. Reduction in Remaining Life Due to Difference between the Load Reduction Applied and the Load Reduction Required.

Thus, if the 44 percent load reduction level is used (average of the seven state areas previously noted), this results in a potential improvement in pavement life of about 90 percent. The basic (and very conservative) assumption is that all the pavement damage (hence load reduction benefit) can occur during the thaw weakened period. For some pavements, this may actually occur but generally would not be the case for most. What this curve allows is for an agency to select the amount of benefit desired and restrict loads accordingly.

Clearly, the needed level of load reduction is not as simple as an examination of Figure 5.2 suggests. For example, many thin or generally weak pavement structures need high levels of load reduction during the spring thaw period to prevent significant pavement damage (i.e., small or even modest levels of load reduction will not preclude significant pavement damage). To further assist agencies, Figure 5.3 was developed. This figure is a plot of the load reduction required to maintain equivalent summer rutting levels (similar to Figure 5.2) versus reduction in remaining life due to spring thawing. The family of curves shown are for various levels of actually applied load reduction (0 to 50 percent). For example, if a pavement section actually needed (or required) a 40 percent load reduction to prevent pavement damage from exceeding that accumulated during the summer but only a 30 percent load reduction was actually applied, then the reduction in remaining life would be about 40 percent. Again, if the required load reduction is 40 percent but only a 20 percent load reduction was applied, then the reduction in remaining life would be slightly more than 60 percent. (Figure 5.3 was developed for the same tire-axle cases as used in Figure 5.2 and the rutting failure criterion. The differences in remaining life between the actually applied and required load reductions were based on the relative values of the equivalent summer vertical subgrade strain (which results in the required load reduction) and that strain resulting from the actually applied load reduction.)

If load restrictions are to be used, it appears that a minimum load reduction of 20 percent is needed. Load reductions greater than 60 percent would appear to be excessive (given the assumptions used in preceding analysis). Further, general national practice is to use load reductions ranging from 40 to 50 percent. The analysis performed in this study tends to confirm this range of load reduction.

5.4 GUIDELINES FOR WHEN TO APPLY AND REMOVE LOAD RESTRICTIONS

5.4.1 WHEN TO APPLY LOAD RESTRICTIONS

A primarily activity in the study was to develop guidelines on when to apply and remove load restrictions (assuming that load restrictions are needed). These guidelines are based on easy to obtain air temperature data from local weather stations or site specific high-low recording thermometers. It is assumed that most agencies do not have the capability to use deflection measuring equipment during the start of the critical period to assess when to apply load restrictions.

A review of the thermal analysis information presented in Chapter 4.0 results in a two possible times for applying load restrictions. Both are based on Thawing Index (TI) calculated by use of a 29°F datum (not the normally used 32°F). (A discussion on how to calculate the thawing index and an example is included in Appendix F). These two criteria follow:

5.4.1.1 SHOULD LEVEL

The "should" load restriction level occurs after accumulating a TI = 25°F-days following the start of the thawing period. This is used to estimate thaw to the bottom of the base course.

5.4.1.2 MUST LEVEL

The "must" load restriction level occurs after accumulating a TI = 50°F-days following the start of the thawing period. This is used to estimate thaw to approximately four inches below the bottom of the base course.

5.4.1.3 SHOULD AND MUST LEVELS FOR THIN PAVEMENT SECTIONS

The "should" level for thin pavements (such as two inches of asphalt concrete with a six inch aggregate base) could be as low as a TI = 10°F-days. The corresponding "must" level TI = 40°F-days.

5.4.1.4 DISCUSSION

The above criteria are best suited for use during the "normal" start of the spring thaw period (generally late February to April). A different condition exists for mid-winter thawing cases. First, the sun angle is lower for a mid-winter thaw than used in the analysis suggesting a higher base temperature (such as 31°F) for calculating TI. Second, for most areas, the percent cloud cover is higher during mid-winter.

The temperature based TI criteria are best applied for fine-grained soils. The analysis presented in Chapter 4.0 showed more consistent results for this soil type than coarse-grained.

5.4.2 WHEN TO REMOVE LOAD RESTRICTIONS

Based on the literature review (Chapter 2.0), interviews (Chapter 3.0), and the structural and thermal analyses (Chapter 4.0), the duration of the load restriction period should approximate the time required to achieve complete thawing.

Two different approaches were developed in the study to predict the duration of load restrictions both of which are based on regression equations with Freezing Index (FI) as the independent variable.

The first equation was developed for the fine-grained subgrade cases (which tend to be the most critical) and can be used to estimate the load restriction duration as a function of FI. This equation is:

$$\text{Duration (days)} = 22.62 + 0.011\ (\text{FI})$$

where:

Duration = duration for complete thaw based on a start date when the air temperature is 29°F or above, (days),

FI = freezing index (°F-days)

An approximate solution to the above equation is:

$$\text{Duration} \simeq 25 + 0.01\ (\text{FI})$$

A brief comparison of the two solutions as a function of FI is shown in Table 5.2.

The two above equations are based on fine-grained soils at a moisture content of 15 percent and a range of FI from 400 to 2000 ^{o}F-days. Predicted durations outside of this data range may result in poor estimates. Further, for locations with relatively low FI (400 to 500 ^{o}F-days), the predicted durations are probably conservative (i.e., longer than actual).

Another approach to use in estimating the time required for complete thawing to occur (hence duration of load restrictions) is based on a TI criterion. The TI (again based on a 29^{o}F air temperature datum) is estimated from a regression equation which has the independent variable of FI. The resulting equations have higher correlation coefficients than those for estimating duration as a function of FI. The equation selected for potential use is (based on fine-grain cases and 15 percent moisture content):

$$TI = 4.154 + 0.259 (FI)$$

An approximate solution is:

$$TI \simeq 0.3 (FI)$$

A comparison of these two equations is provided in Table 5.3. An example of estimating when to place load restrictions and how long to maintain them using temperature data obtained from a high-low thermometer throughout the freezing and thawing period is shown in Appendix G.

Table 5.2 Comparison of Equations Used to Predict Duration for Complete Thaw

Freezing Index (°F-days)	Complete Thaw-Duration Based on Original Regression Equation (a) (days)	Complete Thaw-Duration Based on Approximate Regression Equation (b) (days)
400	27	29
500	28	30
750	31	32
1000	34	35
1250	36	38
1500	39	40
2000	45	45

Notes: (a) Duration (days) = 22.62 + 0.011 (FI)

(b) Duration (days) ≃ 25 + 0.01 (FI)

Table 5.3 Comparison of Predictions Used for Determining the Duration of the Load Restriction Period Based on Thawing Index

Freezing Index (°F-days)	Prediction of Thawing Index (29°F datum) Based on Original Regression Equation (a) (°F-days)	Prediction of Thawing Index (29°F datum) Based on Approximate Regression Equation (b) (°F-days)
400	108	120
500	134	150
750	198	225
1000	263	300
1250	328	375
1500	393	450
2000	522	600

Notes: (a) TI = 4.154 + 0.259 (TI)

(b) TI ≈ 0.3 (FI)

CHAPTER 6.0
CONCLUSIONS AND RECOMMENDATIONS

6.1 CONCLUSIONS

The following conclusions are warranted:

(a) The use of load restrictions to reduce or preclude pavement damage during spring thaw periods is widely used in the U.S. and Europe. Load restrictions are primarily applied to low volume road networks.

(b) Investigations in the U.S. of the thaw weakening process in pavement structures increased during the late 1940's.

(c) Extensive examinations have been conducted in recent years in states such as Alaska, Minnesota and Washington.

(d) Surveys conducted in this study reveal the following:

(i) Load restrictions are applied mostly to pavements which have subgrades composed of moisture susceptible silts and clays.

(ii) Load restrictions are applied mostly to aggregate and/or asphalt surfaced pavements. These pavements are usually older (about 20 years).

(iii) The maximum legal loads are generally reduced from about 40 to 50 percent for single axles and 30 to 50 percent for tandem axles.

(iv) Judgment by field personnel is primarily used to assess where, when, how much and how long to apply load restrictions.

(e) For determining where to apply load restrictions, the following is often considered:

(i) comparison of summer and spring pavement surface deflection data,

(ii) surface thickness,

(iii) moisture conditions,

(iv) subgrade type,

(v) local experience.

(f) A temperature based criterion appears to be a straightforward and easy way to determine when and for how long to apply load restrictions.

(g) The average load restriction applied by the agencies interviewed (based on seven individual state areas) is about 44 percent. Further, an analysis based on characterizing a pavement structure as a layered elastic system suggests that a minimum load restriction level (if any load reduction is needed) is 20 percent. Load reductions greater than 60 percent are not justifiable for the wide range of cases studied. Current national practice and the analysis performed in this study suggests that for those pavements needing load restrictions, load reductions ranging from 40 to 50 percent should accommodate a wide range of pavement conditions.

6.2 RECOMMENDATIONS

The following recommendations are provided on where, how much, when and how long to apply load restrictions:

(a) Where to apply load restrictions. If pavement surface deflections are available to an agency, spring thaw deflections greater than 45 to 50 percent of summer deflections suggest a need for load restriction. Further, considerations such as depth of freezing (generally areas with Freezing Indices of 400°F-days or more), pavement surface thickness, moisture condition, type of subgrade, and local experience should be considered. Subgrades with Unified Soil Classifications of ML, MH, CL and CH will result in the largest pavement weakening.

(b) Load restriction magnitudes can be based on guidance provided in Figures and 5.2 and 5.3 (Chapter 5.0). A minimum load reduction level should be 20 percent. Load reductions greater than 60 percent generally are not warranted based on potential pavement damage. A load reduction range of 40 to 50 percent should accommodate a wide range of pavement conditions.

(c) When to apply load restrictions. Load restrictions should be applied after accumulating a Thawing Index (TI) of about 25°F-days (based on an air temperature datum of 29°F) and must be applied at a TI of about 50°F-days (again based on an air temperature datum of 29°F). Corresponding TI levels are less for thin pavements (e.g. two inches of asphalt concrete and six inches of aggregate base).

(d) When to remove load restrictions. Two approaches are recommended both of which are based on air temperatures. The duration of the load restriction period can be directly estimated by:

$$\text{Duration (days)} = 25 + 0.01\ (\text{FI})$$

Further, the duration can be estimated by use of TI and the following relationship:

$$\text{TI} \approx 0.3\ (\text{FI})$$

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APPENDIX A

DATA SUMMARY FOR SUMMER CONDITIONS

A.1 Summer Conditions - Single Tire - Single Axle

Pavement Structural Section		Resilient Modulus (psi)			Pavement Response (a)(b)			
Surface Thickness (in.)	Base Thickness (in.)	Surface	Base	Subgrade	δ (in.)	ε_t (in/in X 10^{-6})	ε_{vb} (in/in X 10^{-6})	ε_{vs} (in/in X 10^{-6})
Fine-grained Subgrade								
2	6	300,000	11,250	7,500	0.0700	+950	-4370	-3120
2	12	300,000	11,250	7,500	0.0657	+899	-4400	-1670
4	6	300,000	11,250	7,500	0.0455	+655	-2200	-1570
4	12	300,000	11,250	7,500	0.0433	+629	-2250	-1000
Coarse-grained Subgrade								
2	6	300,000	60,000	40,000	0.0161	+190	-1050	- 755
2	12	300,000	60,000	40,000	0.0149	+182	-1050	- 368
4	6	300,000	60,000	40,000	0.0126	+243	- 809	- 500
4	12	300,000	60,000	40,000	0.0119	+232	- 814	- 270

Notes: (a) + tension
- compression

(b) (i) Surface deflection (δ)
(ii) Horizontal strain bottom of surface coarse (ε_t)
(iii) Vertical strain top of base (ε_{vb})
(iv) Vertical strain top of subgrade (ε_{vs})

Table A.2 Summer Conditions - Dual Tires - Single Axle - Pavement Response Between Tires

Pavement Structural Section		Resilient Modulus (psi)			Pavement Response(a)(b)				
Surface Thickness (in.)	Base Thickness (in.)	Surface	Base	Subgrade	Location (c)	δ (in.)	ε_t (in/in X 10^{-6})	ε_{vb} (in/in X 10^{-6})	ε_{vs} (in/in X 10^{-6})
Fine-grained Subgrade									
2	6	300,000	11,250	7,500	BDT	0.0547	-426	-1774	-2101
					BIT	0.0560	+706	-3494	-2105
2	12	300,000	11,250	7,500	BDT	0.0508	-455	-1778	-1360
					BIT	0.0524	+682	-3507	-1218
4	6	300,000	11,250	7,500	BDT	0.0399	+ 75	-1337	-1295
					BIT	0.0390	+399	-1685	-1167
4	12	300,000	11,250	7,500	BDT	0.0379	+ 54	-1362	-1190
					BIT	0.0370	+382	-1702	-1147
Coarse-grained Subgrade									
2	6	300,000	60,000	40,000	BDT	0.0110	-284	- 171	- 438
					BIT	0.0131	+231	-1050	- 513
2	12	300,000	60,000	40,000	BDT	0.0101	-290	- 168	- 284
					BIT	0.0123	+225	-1051	- 260
4	6	300,000	60,000	40,000	BDT	0.0096	- 76	- 336	- 352
					BIT	0.0104	+173	- 656	- 344
4	12	300,000	60,000	40,000	BDT	0.0089	- 84	- 335	- 224
					BIT	0.0098	+167	- 657	- 202

Notes: (a) + tension
- compression

(b) (i) Surface deflection (δ)
(ii) Horizontal strain bottom of surface coarse (ε_t)
(iii) Vertical strain top of base (ε_{vb})
(iv) Vertical strain top of subgrade (ε_{vs})

(c) (i) BDT = between dual tires
BIT = beneath inside tire of dual set

Table A.3 Summer Condition - Dual Tires - Tandem Axle

Pavement Structural Section: Surface Thickness (in.)	Pavement Structural Section: Base Thickness (in.)	Resilient Modulus (psi): Surface	Resilient Modulus (psi): Base	Resilient Modulus (psi): Subgrade	Pavement Response (a)(b): Location (c)	δ (in.)	ε_t (in/in X 10^{-6})	ε_{vb} (in/in X 10^{-6})	ε_{vs} (in/in X 10^{-6})
Fine-grained Subgrade									
2	6	300,000	11,250	7,500	BA	0.0226	+ 15	+ 34	+ 100
					BW	0.0382	- 404	-1468	-1780
					DT	0.0525	+ 683	-3200	-1812
2	12	300,000	11,250	7,500	BA	0.0210	+ 6	+ 30	- 45
					BW	0.0475	- 430	-1470	-1150
					DT	0.0497	+ 661	-3218	-1030
4	6	300,000	11,250	7,500	BA	0.0206	+ 70	- 22	- 48
					BW	0.0507	+ 52	-1125	-1058
					DT	0.0376	+ 368	-1490	- 984
4	12	300,000	11,250	7,500	BA	0.0222	+ 60	- 9	- 159
					BW	0.0366	+ 30	-1150	- 744
					DT	0.0362	+ 353	-1510	- 670
Coarse-grained Subgrade									
2	6	300,000	60,000	40,000	BA	0.0042	- 6	+ 4	+ 234
					BW	0.0090	- 258	- 123	- 370
					DT	0.0124	+ 237	-1014	- 450
2	12	300,000	60,000	40,000	BA	0.0042	- 11	+ 6	- 2
					BW	0.0095	- 260	- 120	- 240
					DT	0.0118	+ 232	-1014	- 220
4	6	300,000	60,000	40,000	BA	0.0041	- 5	+ 8	+ 14
					BW	0.0102	- 77	- 276	- 297
					DT	0.0100	+ 165	- 600	- 295
4	12	300,000	60,000	40,000	BA	0.0043	0	+ 10	- 14
					BW	0.0086	- 80	- 276	- 190
					DT	0.0095	+ 165	- 600	- 170

Notes: (a) + tension
- compression

(b) (i) Surface deflection (δ)
(ii) Horizontal strain bottom surface coarse (ε_t)
(iii) Vertical strain top of base (ε_{vb})
(iv) Vertical strain top of subgrade (ε_{vs})

(c) (i) BA = between axles on centerline of dual tires
(ii) BW = between dual wheels centered on one axle
(iii) DT = centered directly under inside wheel

APPENDIX B

DATA SUMMARY FOR SPRING THAW CONDITIONS

Table B.1 Spring Thaw Condition - Single Tire - Single Axle - Complete Thaw - Pavement Structure 2/6/212[a]

Subgrade Type	Reduction in Subgrade Resilient Modulus	Resilient Modulus (psi) Surface Course	Base	Subgrade Thawed	Subgrade Unfrozen	Pavement Response Percent of Full Load	δ (in.) (b)	ε_t (in/in X 10^{-6}) (b)	ε_{vb} (in/in X 10^{-6}) (b)	ε_{vs} (in/in X 10^{-6}) (b)
Fine-grain	85%	1,200,000	1,700	1,120	7,500	20	0.0326	+ 341	-1600	-1200
						100	0.1690	+1030	-6060	-5790
	80%	1,200,000	2,250	1,500	7,500	20	0.0285	+ 326	-1480	-1074
						100	0.1440	+ 956	-5414	-5020
	75%	1,200,000	2,800	1,880	7,500	20	0.0260	+ 315	-1390	- 979
						100	0.1290	+ 902	-4945	-4482
Coarse-grain	75%	1,200,000	15,000	10,000	40,000	20	0.0088	+ 210	- 783	- 416
						100	0.0383	+ 462	-2190	-1714
	70%	1,200,000	18,000	12,000	40,000	20	0.0079	+ 198	- 729	- 373
						100	0.0336	+ 420	-1974	-1518
	50%	1,200,000	30,000	20,000	40,000	20	0.0057	+ 166	- 590	- 269
						100	0.0232	+ 312	-1440	-1060

Notes: (a) Surface course = 2 in.
Base = 6 in.
Thawed subgrade = 40 in.
Unfrozen subgrade = 212 in.

(b) (i) Surface deflection (δ)
(ii) Horizontal strain bottom of surface course (ε_t)
(iii) Vertical strain top of base (ε_{vb})
(iv) Vertical strain top of subgrade (ε_{vs})

Table B.2 Spring Thaw Condition - Single Tire - Single Axle - Complete Thaw-Pavement Structure 2/12/34/212[(a)]

Subgrade Type	Reduction in Subgrade Resilient Modulus	Resilient Modulus (psi)				Pavement Response				
		Surface Course	Base	Subgrade Thawed	Subgrade Unfrozen	Percent of Full Load	δ (in.) (b)	ε_t (in/in X 10^{-6}) (b)	ε_{vb} (in/in X 10^{-6}) (b)	ε_{vs} (in/in X 10^{-6}) (b)
Fine-grain	85%	1,200,000	1,700	1,120	7,500	20	0.0319	+ 335	-1676	- 860
						100	0.1620	+1000	-6346	-4577
	80%	1,200,000	2,250	1,500	7,500	20	0.0278	+ 320	-1540	- 742
						100	0.1380	+ 927	-5650	-3834
	75%	1,200,000	2,800	1,880	7,500	20	0.0252	+ 308	-1443	- 659
						100	0.1220	+ 870	-5150	-3330
Coarse-grain	75%	1,200,000	15,000	10,000	40,000	20	0.0083	+ 204	- 795	- 223
						100	0.0358	+ 440	-2235	-1040
	70%	1,200,000	18,000	12,000	40,000	20	0.0074	+ 193	- 740	- 195
						100	0.0314	+ 400	-2000	- 898
	50%	1,200,000	30,000	20,000	40,000	20	0.0054	+ 163	- 595	- 131
						100	0.0217	+ 296	-1457	- 592

Notes: (a) Surface course = 2 in.
Base = 12 in.
Thawed subgrade = 34 in.
Unfrozen subgrade = 212 in.

(b) (i) Surface deflection (δ)
(ii) Horizontal strain bottom of surface course (ε_t)
(iii) Vertical strain top of base (ε_{vb})
(iv) Vertical strain top of subgrade (ε_{vs})

Table B.3 Spring Thaw Condition - Single Tire - Single Axle - Complete Thaw - Pavement Structure 4/6/38/212[a]

Subgrade Type	Reduction in Subgrade Resilient Modulus	Resilient Modulus (psi) Surface Course	Base	Subgrade Thawed	Subgrade Unfrozen	Pavement Response Percent of Full Load	δ (in.) (b)	ε_t (in/in X 10^{-6}) (b)	ε_{vb} (in/in X 10^{-6}) (b)	ε_{vs} (in/in X 10^{-6}) (b)
Fine-grain	85%	1,200,000	1,700	1,120	7,500	20	0.0159	+106	- 483	- 329
						100	0.0678	+403	-1990	-1790
	80%	1,200,000	2,250	1,500	7,500	20	0.0137	+104	- 435	- 316
						100	0.0613	+385	-1750	-1614
	75%	1,200,000	2,800	1,880	7,500	20	0.0125	+102	- 405	- 286
						100	0.0571	+372	-1640	-1480
Coarse-grain	75%	1,200,000	15,000	10,000	40,000	20	0.0042	+ 82	- 256	- 151
						100	0.0206	+244	- 892	- 717
	70%	1,200,000	18,000	12,000	40,000	20	0.0039	+ 79	- 243	- 140
						100	0.0186	+231	- 829	- 653
	50%	1,200,000	30,000	20,000	40,000	20	0.0031	+ 70	- 209	- 112
						100	0.0139	+193	- 665	- 497

Notes: (a) Surface course = 4 in.
Base = 6 in.
Thawed subgrade = 38 in.
Unfrozen subgrade = 212 in.

(b) (i) Surface deflection (δ)
(ii) Horizontal strain bottom of surface course (ε_t)
(iii) Vertical strain top of base (ε_{vb})
(iv) Vertical strain top of subgrade (ε_{vs})

Table B.4 Spring Thaw Condition - Single Tire - Single Axle - Complete Thaw - Pavement Structure 4/12/32/212(a)

Subgrade Type	Reduction in Subgrade Resilient Modulus	Resilient Modulus (psi)				Pavement Response				
		Surface Course	Base	Subgrade Thawed	Subgrade Unfrozen	Percent of Full Load	δ (in.) (b)	ε_t (in/in X 10^{-6}) (b)	ε_{vb} (in/in X 10^{-6}) (b)	ε_{vs} (in/in X 10^{-6}) (b)
Fine-grain	85%	1,200,000	1,700	1,120	7,500	20	0.0159	+106	- 510	- 324
						100	0.0669	+398	-1990	-1619
	80%	1,200,000	2,250	1,500	7,500	20	0.0137	+103	- 452	- 272
						100	0.0602	+379	-1829	-1427
	75%	1,200,000	2,800	1,880	7,500	20	0.0124	+101	- 420	- 239
						100	0.0558	+365	-1700	-1290
Coarse-grain	75%	1,200,000	15,000	10,000	40,000	20	0.0041	+ 80	- 263	- 99
						100	0.0197	+237	- 920	- 526
	70%	1,200,000	18,000	12,000	40,000	20	0.0038	+ 77	- 250	- 90
						100	0.0178	+223	- 854	- 468
	50%	1,200,000	30,000	20,000	40,000	20	0.0030	+ 69	- 213	- 68
						100	0.0133	+186	- 682	- 334

Notes: (a) Surface course = 4 in.
Base = 12 in.
Thawed subgrade = 32 in.
Unfrozen subgrade = 212 in.

(b) (i) Surface deflection (δ)
(ii) Horizontal strain bottom of surface course (ε_t)
(iii) Vertical strain top of base (ε_{vb})
(iv) Vertical strain top of subgrade (ε_{vs})

Table B.5 Spring Thaw Condition - Dual Tires - Single Axle - Complete Thaw Thaw - Pavement Structure 2/6/40/212[(a)] - Between Wheels

Subgrade Type	Reduction in Subgrade Resilient Modulus	Resilient Modulus (psi)				Pavement Response Between Wheels				
		Surface Course	Base	Subgrade Thawed	Subgrade Unfrozen	Percent of Full Load	δ (in.) (b)	ε_t (in/in X 10^{-6}) (b)	ε_{vb} (in/in X 10^{-6}) (b)	ε_{vs} (in/in X 10^{-6}) (b)
Fine-grain	85%	1,200,000	1,700	1,120	7,500	20	0.0316	+ 50	- 924	-1023
						100	0.1560	+ 332	-4592	-4999
	80%	1,200,000	2,250	1,500	7,500	20	0.0267	+ 37	- 801	- 875
						100	0.1320	+ 270	-3989	-4271
	75%	1,200,000	2,800	1,880	7,500	20	0.0237	+ 28	- 714	- 773
						100	0.1170	+ 229	-3556	-3766
Coarse-grain	75%	1,200,000	15,000	10,000	40,000	20	0.0064	- 25	- 233	- 264
						100	0.0316	- 42	-1205	-1279
	70%	1,200,000	18,000	12,000	40,000	20	0.0056	- 28	- 199	- 230
						100	0.0274	- 59	-1042	-1114
	50%	1,200,000	30,000	20,000	40,000	20	0.0037	- 33	- 122	- 153
						100	0.0184	- 93	- 665	- 742

Notes: (a) Surface course = 2 in.
Base = 6 in.
Thawed subgrade = 40 in.
Unfrozen subgrade = 212 in.

(b) (i) Surface deflection (δ)
(ii) Horizontal strain bottom of surface course (ε_t)
(iii) Vertical strain top of base (ε_{vb})
(iv) Vertical strain top of subgrade (ε_{vs})

Table B.6 Spring Thaw Condition - Dual Tires - Single Axle - Complete Thaw - Pavement Structure 2/12/34/212[a] - Between Wheels

Subgrade Type	Reduction in Subgrade Resilient Modulus	Resilient Modulus (psi)				Pavement Response Between Wheels				
		Surface Course	Base	Subgrade Thawed	Subgrade Unfrozen	Percent of Full Load	δ (in.) (b)	ε_t (in/in X 10^{-6}) (b)	ε_{vb} (in/in X 10^{-6}) (b)	ε_{vs} (in/in X 10^{-6}) (b)
Fine-grain	85%	1,200,000	1,700	1,120	7,500	20	0.0304	+ 45	- 971	- 858
						100	0.1500	+ 308	-4814	-4211
	80%	1,200,000	2,250	1,500	7,500	20	0.0255	+ 32	- 839	- 713
						100	0.1260	+ 246	-4166	-3497
	75%	1,200,000	2,800	1,880	7,500	20	0.0225	+ 24	- 745	- 616
						100	0.1110	+ 204	-3700	-3015
Coarse-grain	75%	1,200,000	15,000	10,000	40,000	20	0.0060	- 28	- 237	- 182
						100	0.0294	- 57	-1227	- 883
	70%	1,200,000	18,000	12,000	40,000	20	0.0052	- 31	- 203	- 156
						100	0.0255	- 73	-1057	- 758
	50%	1,200,000	30,000	20,000	40,000	20	0.0024	- 35	- 123	- 101
						100	0.0170	-103	- 670	- 489

Notes: (a) Surface course = 2 in.
Base = 12 in.
Thawed subgrade = 34 in.
Unfrozen subgrade = 212 in.

(b) (i) Surface deflection (δ)
(ii) Horizontal strain bottom of surface course (ε_t)
(iii) Vertical strain top of base (ε_{vb})
(iv) Vertical strain top of subgrade (ε_{vs})

Table B.7 Spring Thaw Condition - Dual Tires - Single Axle - Complete Thaw - Pavement Structure 4/6/38/212[(a)] - Between Wheels

Subgrade Type	Reduction in Subgrade Resilient Modulus	Resilient Modulus (psi)				Pavement Response Between Wheels				
		Surface Course	Base	Subgrade Thawed	Subgrade Unfrozen	Percent of Full Load	δ (in.) (b)	ε_t (in/in X 10^{-6}) (b)	ε_{vb} (in/in X 10^{-6}) (b)	ε_{vs} (in/in X 10^{-6}) (b)
Fine-grain	85%	1,200,000	1,700	1,120	7,500	20	0.0132	+ 37	- 324	- 338
						100	0.0657	+ 210	-1616	-1664
	80%	1,200,000	2,250	1,500	7,500	20	0.0119	+ 34	- 292	- 302
						100	0.0591	+ 193	-1460	-1488
	75%	1,200,000	2,800	1,880	7,500	20	0.0111	+ 31	- 269	- 276
						100	0.0549	+ 181	-1343	-1357
Coarse-grain	75%	1,200,000	15,000	10,000	40,000	20	0.0038	+ 9	- 125	- 125
						100	0.0187	+ 73	- 632	- 613
	70%	1,200,000	18,000	12,000	40,000	20	0.0034	+ 7	- 113	- 113
						100	0.0168	+ 63	- 575	- 553
	50%	1,200,000	30,000	20,000	40,000	20	0.0025	+ 2	- 84	- 84
						100	0.0124	+ 37	- 431	- 409

Notes: (a) Surface course = 4 in.
Base = 6 in.
Thawed subgrade = 38 in.
Unfrozen subgrade = 212 in.

(b) (i) Surface deflection (δ)
(ii) Horizontal strain bottom of surface course (ε_t)
(iii) Vertical strain top of base (ε_{vb})
(iv) Vertical strain top of subgrade (ε_{vs})

Table B.8 Spring Thaw Condition - Dual Tires - Single Axle - Complete Thaw - Pavement Structure 4/12/32/212[a] - Between Wheels

Subgrade Type	Reduction in Subgrade Resilient Modulus	Resilient Modulus (psi)				Pavement Response Between Wheels				
		Surface Course	Base	Subgrade Thawed	Subgrade Unfrozen	Percent of Full Load	δ (in.) (b)	ε_t (in/in X 10^{-6}) (b)	ε_{vb} (in/in X 10^{-6}) (b)	ε_{vs} (in/in X 10^{-6}) (b)
Fine-grain	85%	1,200,000	1,700	1,120	7,500	20	0.0130	+ 36	- 339	- 316
						100	0.0648	+ 205	-1690	-1562
	80%	1,200,000	2,250	1,500	7,500	20	0.0117	+ 33	- 305	- 277
						100	0.0581	+ 188	-1524	-1370
	75%	1,200,000	2,800	1,880	7,500	20	0.0108	+ 30	- 280	- 249
						100	0.0536	+ 175	-1400	-1230
Coarse-grain	75%	1,200,000	15,000	10,000	40,000	20	0.0036	+ 8	- 130	- 98
						100	0.0179	+ 66	- 653	- 482
	70%	1,200,000	18,000	12,000	40,000	20	0.0032	+ 6	- 117	- 87
						100	0.0160	+ 56	- 593	- 427
	50%	1,200,000	30,000	20,000	40,000	20	0.0024	+ 1	- 87	- 61
						100	0.0117	+ 31	- 441	- 300

Notes: (a) Surface course = 4 in.
Base = 12 in.
Thawed subgrade = 32 in.
Unfrozen subgrade = 212 in.

(b) (i) Surface deflection (δ)
(ii) Horizontal strain bottom of surface course (ε_t)
(iii) Vertical strain top of base (ε_{vb})
(iv) Vertical strain top of subgrade (ε_{vs})

Table B.9 Spring Thaw Condition - Dual Tires - Single Axle - Complete Thaw - Pavement Structure 2/6/40/212(a) - Beneath Tire

Subgrade Type	Reduction in Subgrade Resilient Modulus	Resilient Modulus (psi)				Pavement Response Beneath Inside Tire of Duals				
		Surface Coarse	Base	Subgrade Thawed	Subgrade Unfrozen	Percent of Full Load	δ (in.) (b)	ε_t (in/in X 10^{-6}) (b)	ε_{vb} (in/in X 10^{-6}) (b)	ε_{vs} (in/in X 10^{-6}) (b)
Fine-grain	85%	1,200,000	1,700	1,120	7,500	20	0.0314	+188	-1129	- 896
						100	0.1426	+641	-4788	-4585
	80%	1,200,000	2,250	1,500	7,500	20	0.0261	+179	-1014	- 765
						100	0.1212	+586	-4229	-3924
	75%	1,200,000	2,800	1,880	7,500	20	0.0232	+173	- 935	- 677
						100	0.1083	+549	-3826	-3461
Coarse-grain	75%	1,200,000	15,000	10,000	40,000	20	0.0066	+123	- 501	- 255
						100	0.0301	+290	-1645	-1200
	70%	1,200,000	18,000	12,000	40,000	20	0.0058	+118	- 476	- 226
						100	0.0264	+269	-1489	-1054
	50%	1,200,000	30,000	20,000	40,000	20	0.0041	+104	- 382	- 158
						100	0.0182	+215	-1115	- 721

Notes: (a) Surface course = 2 in.
Base = 6 in.
Thawed subgrade = 40 in.
Unfrozen subgrade = 212 in.

(b) (i) Surface deflection (δ)
(ii) Horizontal strain bottom of surface course (ε_t)
(iii) Vertical strain top of base (ε_{vb})
(iv) Vertical strain top of subgrade (ε_{vs})

Table B.10 Spring Thaw Condition - Dual Tires - Single Axle - Complete Thaw - Pavement Structure 2/12/34/212[(a)] - Beneath Tire

Subgrade Type	Reduction in Subgrade Resilient Modulus	Resilient Modulus (psi)				Pavement Response Beneath Inside Tire of Duals				
		Surface Coarse	Base	Subgrade Thawed	Subgrade Unfrozen	Percent of Full Load	δ (in.) (b)	ε_t (in/in X 10^{-6}) (b)	ε_{vb} (in/in X 10^{-6}) (b)	ε_{vs} (in/in X 10^{-6}) (b)
Fine-grain	85%	1,200,000	1,700	1,120	7,500	20	0.0306	+186	-1169	-1757
						100	0.1369	+619	-4978	-3785
	80%	1,200,000	2,250	1,500	7,500	20	0.0254	+176	-1048	- 619
						100	0.1159	+565	-4378	-3150
	75%	1,200,000	2,800	1,880	7,500	20	0.0223	+170	- 964	- 530
						100	0.1027	+526	-3950	-2718
Coarse-grain	75%	1,200,000	15,000	10,000	40,000	20	0.0062	+120	- 506	- 158
						100	0.0281	+278	-1666	- 785
	70%	1,200,000	18,000	12,000	40,000	20	0.0055	+116	- 472	- 137
						100	0.0246	+258	-1505	- 674
	50%	1,200,000	30,000	20,000	40,000	20	0.0039	+102	- 384	- 89
						100	0.0170	+207	-1122	- 436

Notes: (a) Surface course = 2 in.
Base = 12 in.
Thawed subgrade = 34 in.
Unfrozen subgrade = 212 in.

(b) (i) Surface deflection (δ)
(ii) Horizontal strain bottom of surface course (ε_t)
(iii) Vertical strain top of base (ε_{vb})
(iv) Vertical strain top of subgrade (ε_{vs})

Table B.11 Spring Thaw Condition - Dual Tires - Single Axle - Complete Thaw - Pavement Structure 4/6/38/212[(a)] Beneath Tire

Subgrade Type	Reduction in Subgrade Resilient Modulus	Resilient Modulus (psi) Surface Coarse	Base	Subgrade Thawed	Subgrade Unfrozen	Pavement Response Beneath Inside Tire of Duals Percent of Full Load	δ (in.) (b)	ϵ_t (in/in X 10^{-6}) (b)	ϵ_{vb} (in/in X 10^{-6}) (b)	ϵ_{vs} (in/in X 10^{-6}) (b)
Fine-grain	85%	1,200,000	1,700	1,120	7,500	20	0.0172	+ 68	- 415	- 375
						100	0.0611	+265	-1615	-1486
	80%	1,200,000	2,250	1,500	7,500	20	0.0148	+ 64	- 368	- 318
						100	0.0544	+252	-1458	-1324
	75%	1,200,000	2,800	1,880	7,500	20	0.0133	+ 62	- 339	- 280
						100	0.0505	+244	-1348	-1211
Coarse-grain	75%	1,200,000	15,000	10,000	40,000	20	0.0039	+ 42	- 170	- 110
						100	0.0175	+152	- 701	- 564
	70%	1,200,000	18,000	12,000	40,000	20	0.0035	+ 41	- 159	- 99
						100	0.0158	+143	- 647	- 510
	50%	1,200,000	30,000	20,000	40,000	20	0.0026	+ 36	- 132	- 74
						100	0.0118	+118	- 512	- 377

Notes: (a) Surface course = 4 in.
Base = 6 in.
Thawed subgrade = 38 in.
Unfrozen subgrade = 212 in.

(b) (i) Surface deflection (δ)
(ii) Horizontal strain bottom of surface course (ϵ_t)
(iii) Vertical strain top of base (ϵ_{vb})
(iv) Vertical strain top of subgrade (ϵ_{vs})

Table B.12 Spring Thaw Condition - Dual Tires - Single Axle - Complete Thaw - Pavement Structure 4/12/32/212 (a) - Beneath Tire

Subgrade Type	Reduction in Subgrade Resilient Modulus	Resilient Modulus (psi)				Pavement Response Beneath Inside Tire of Duals				
		Surface Coarse	Base	Subgrade Thawed	Subgrade Unfrozen	Percent of Full Load	δ (in.) (b)	ϵ_t (in/in X 10^{-6}) (b)	ϵ_{vb} (in/in X 10^{-6}) (b)	ϵ_{vs} (in/in X 10^{-6}) (b)
Fine-grain	85%	1,200,000	1,700	1,120	7,500	20	0.0170	+ 68	- 429	- 382
						100	0.0607	+261	-1680	-1378
	80%	1,200,000	2,250	1,500	7,500	20	0.0145	+ 64	- 377	- 312
						100	0.0539	+248	-1519	-1202
	75%	1,200,000	2,800	1,880	7,500	20	0.0130	+ 61	- 341	- 269
						100	0.0500	+239	-1402	-1080
Coarse-grain	75%	1,200,000	15,000	10,000	40,000	20	0.0038	+ 42	- 174	- 87
						100	0.0168	+146	- 718	- 434
	70%	1,200,000	18,000	12,000	40,000	20	0.0034	+ 40	- 162	- 76
						100	0.0151	+137	- 662	- 385
	50%	1,200,000	30,000	20,000	40,000	20	0.0025	+ 36	- 135	- 53
						100	0.0130	+113	- 521	- 270

Notes: (a) Surface course = 4 in.
Base = 12 in.
Thawed subgrade = 32 in.
Unfrozen subgrade = 212 in.

(b) (i) Surface deflection (δ)
(ii) Horizontal strain bottom of surface course (ϵ_t)
(iii) Vertical strain top of base (ϵ_{vb})
(iv) Vertical strain top of subgrade (ϵ_{vs})

Table B.13 Spring Thaw Condition - Dual Tires - Tandem Axle - Complete Thaw - Pavement Structure 2/6/40/212(a)

Subgrade Type	Reduction in Subgrade Resilient Modulus	Resilient Modulus (psi)				Pavement Response					
		Surface Course	Base	Subgrade Thawed	Subgrade Unfrozen	Percent of full Load	Location (c)	δ in. (b)	ϵ_t (in/inX10^{-6}) (b)	ϵ_{vb} (in/inX10^{-6}) (b)	ϵ_{vs} (in/inX10^{-6}) (b)
Fine-grain	75	1,200,000	2,800	1,800	7,500	20	BA	0.0119	+ 30	- 80	- 89
							BW	0.0210	+ 26	- 598	- 660
							DT	0.0211	+154	- 801	- 577
						100	BA	0.0598	+152	- 408	- 463
							BW	0.1040	+189	-2986	-3227
							DT	0.0960	+502	-3301	-2963
	85	1,200,000	1,700	1,120	7,500	20	BA	0.0193	+ 48	- 180	- 198
							BW	0.0296	+ 48	- 783	- 847
							DT	0.0287	+168	- 976	- 749
						100	BA	0.0882	+222	- 916	-1005
							BW	0.1380	+282	-3894	-4151
							DT	0.1250	+585	-4137	-3788
Coarse-grain	50	1,200,000	30,000	20,000	40,000	20	BA	0.0012	+ 2	+ 3	+ 7
							BW	0.0034	- 29	- 103	- 129
							DT	0.0037	+ 94	- 338	- 133
						100	BA	0.0059	- 9	+ 12	+ 34
							BW	0.0166	- 91	- 556	- 629
							DT	0.0166	+206	-1013	- 619

Notes:
(a) Surface course = 2 in.
Base = 6 in.
Thawed subgrade = 40 in.
Unfrozen subgrade = 212 in.

(b) (i) Surface deflection (δ)
(ii) Horizontal strain bottom of surface course (ϵ_t)
(iii) Vertical strain top of base (ϵ_{vb})
(iv) Vertical strain top of subgrade (ϵ_{vs})

(c) BA = between axles on centerline of dual tires
BW = between dual wheels centered on one axle
DT = centered directly under inside wheel

Table B.14 Spring Thaw Condition - Dual Tires - Tandem Axle - Complete Thaw - Pavement Structure 2/12/34/212[a]

Subgrade Type	Reduction in Subgrade Resilient Modulus	Resilient Modulus (psi)				Pavement Response					
		Surface Course	Base	Subgrade Thawed	Subgrade Unfrozen	Percent of full Load	Location (c)	δ in. (b)	ϵ_t (in/inX10^{-6}) (b)	ϵ_{vb} (in/inX10^{-6}) (b)	ϵ_{vs} (in/inX10^{-6}) (b)
Fine-grain	75	1,200,000	2,800	1,800	7,500	20	BA	0.0116	+ 28	- 68	- 160
							BW	0.0201	+ 21	- 628	- 526
							DT	0.0205	+151	- 828	- 459
						100	BA	0.0582	+168	- 348	- 810
							BW	0.0996	+167	-3128	-2581
							DT	0.0921	+482	-3423	-2305
	85	1,200,000	1,700	1,120	7,500	20	BA	0.0172	+ 41	- 165	- 290
							BW	0.0270	+ 40	- 823	- 712
							DT	0.0281	+165	-1009	- 645
						100	BA	0.0862	+207	- 838	-1460
							BW	0.1330	+261	-4085	-3502
							DT	0.1214	+566	-4304	-3108
Coarse-grain	50	1,200,000	30,000	20,000	40,000	20	BA	0.0012	+ 1	+ 2	- 4
							BW	0.0031	- 31	- 104	- 84
							DT	0.0036	+ 92	- 339	- 74
						100	BA	0.0060	+ 7	+ 12	- 22
							BW	0.0155	-100	- 560	- 412
							DT	0.0156	+199	-1019	- 367

Notes: (a) Surface course = 2 in.
Base = 12 in.
Thawed subgrade = 34 in.
Unfrozen subgrade = 212 in.

(b) (i) Surface deflection (δ)
(ii) Horizontal strain bottom of surface course (ϵ_t)
(iii) Vertical strain top of base (ϵ_{vb})
(iv) Vertical strain top of subgrade (ϵ_{vs})

(c) BA = between axles on centerline of dual tires
BW = between dual wheels centered on one axle
DT = centered directly under inside wheel

Table B.15 Spring Thaw Condition - Dual Tires - Tandem Axle - Complete Thaw - Pavement Structure 4/6/38/212[a]

Subgrade Type	Reduction in Subgrade Resilient Modulus	Resilient Modulus (psi)				Pavement Response					
		Surface Course	Base	Subgrade Thawed	Subgrade Unfrozen	Percent of full Load	Location (c)	δ in. (b)	ε_t (in/inX10^{-6}) (b)	ε_{vb} (in/inX10^{-6}) (b)	ε_{vs} (in/inX10^{-6}) (b)
Fine-grain	75	1,200,000	2,800	1,800	7,500	20	BA	0.0103	+ 30	- 133	- 147
							BW	0.0111	+ 30	- 238	- 245
							DT	0.0133	+ 58	- 301	- 260
						100	BA	0.0516	+148	- 666	- 735
							BW	0.0553	+169	-1188	-1213
							DT	0.0514	+230	-1207	-1072
	85	1,200,000	1,700	1,120	7,500	20	BA	0.0135	+ 36	- 204	- 215
							BW	0.0136	+ 36	- 300	- 304
							DT	0.0173	+ 65	- 389	- 347
						100	BA	0.0672	+179	-1023	-1075
							BW	0.0676	+200	-1505	-1503
							DT	0.0638	+254	-1521	-1348
Coarse-grain	50	1,200,000	30,000	20,000	40,000	20	BA	0.0014	+ 5	- 4	- 6
							BW	0.0023	+ 2	- 71	- 69
							DT	0.0025	+ 32	- 114	- 62
						100	BA	0.0068	+ 27	- 23	- 33
							BW	0.0115	+ 28	- 363	- 341
							DT	0.0111	+108	- 450	- 316

Notes: (a) Surface course = 4 in.
Base = 6 in.
Thawed subgrade = 38 in.
Unfrozen subgrade = 212 in.

(b) (i) Surface deflection (δ)
(ii) Horizontal strain bottom of surface course (ε_t)
(iii) Vertical strain top of base (ε_{vb})
(iv) Vertical strain top of subgrade (ε_{vs})

(c) BA = between axles on centerline of dual tires
BW = between dual wheels centered on one axle
DT = centered directly under inside wheel

Table B.16 Spring Thaw Condition - Dual Tires - Tandem Axle - Complete Thaw - Pavement Structure 4/12/32/212[(a)]

Subgrade Type	Reduction in Subgrade Resilient Modulus	Resilient Modulus (psi)				Pavement Response					
		Surface Course	Base	Subgrade Thawed	Subgrade Unfrozen	Percent of full Load	Location (c)	δ in. (b)	ε_t $(in/in X 10^{-6})$ (b)	ε_{vb} $(in/in X 10^{-6})$ (b)	ε_{vs} $(in/in X 10^{-6})$ (b)
Fine-grain	75	1,200,000	2,800	1,800	7,500	20	BA	0.0102	+ 29	- 131	- 172
							BW	0.0109	+ 29	- 248	- 227
							DT	0.0131	+ 57	- 308	- 257
						100	BA	0.0508	+142	- 654	- 804
							BW	0.0543	+163	-1238	-1165
							DT	0.0510	+225	-1255	-1056
	85	1,200,000	1,700	1,120	7,500	20	BA	0.0133	+ 35	- 203	- 250
							BW	0.0135	+ 35	- 313	- 291
							DT	0.0171	+ 64	- 397	- 356
						100	BA	0.0667	+175	-1016	-1251
							BW	0.0671	+195	-1563	-1444
							DT	0.0638	+250	-1575	-1276
Coarse-grain	50	1,200,000	30,000	20,000	40,000	20	BA	0.0013	+ 5	- 3	- 14
							BW	0.0022	+ 1	- 73	- 51
							DT	0.0024	+ 31	- 116	- 44
						100	BA	0.0066	+ 23	- 17	- 69
							BW	0.0110	+ 23	- 372	- 250
							DT	0.0106	+104	- 458	- 225

Notes: (a) Surface course = 4 in.
Base = 12 in.
Thawed subgrade = 32 in.
Unfrozen subgrade = 212 in.

(b) (i) Surface deflection (δ)
(ii) Horizontal strain bottom of surface course (ε_t)
(iii) Vertical strain top of base (ε_{vb})
(iv) Vertical strain top of subgrade (ε_{vs})

(c) BA = between axles on centerline of dual tires
BW = between dual wheels centered on one axle
DT = centered directly under inside wheel

Table B.17 Spring Thaw Condition - Single Tire - Single Axle - Thaw to Bottom of Base - Pavement Structure 2/6/40/212[a] - Base M_R @ 25%

Case No.	Thawed Base M_R as a Percent of Summer Base M_R	Resilient Modulus (psi)				Pavement Response				
		Surface Course	Base	Subgrade Frozen	Subgrade Unfrozen	Percent of Full Load	δ (in.) (b)	ε_t (in/in X 10^{-6}) (b)	ε_{vb} (in/in X 10^{-6}) (b)	ε_{vs} (in/in X 10^{-6}) (b)
1	25	1,200,000	2,810	50,000	7,500	20 100	0.0119 0.0506	+259 +641	-1797 -6561	- 51 - 224
2	25	1,200,000	3,750	50,000	10,000	20 100	0.0104 0.0434	+247 +592	-1568 -5556	- 61 - 262
3	25	1,200,000	9,380	50,000	25,000	20 100	0.0068 0.0269	+209 +443	-1028 -3204	- 97 - 391
4	25	1,200,000	15,000	50,000	40,000	20 100	0.0056 0.0214	+189 +373	- 831 -2377	- 115 - 454

(a) Surface course = 2 in.
Base = 6 in.
Frozen subgrade = 40 in.
Unfrozen subgrade = 212 in.

(b) (i) Surface deflection (δ)
(ii) Horizontal strain bottom of surface course (ε_t)
(iii) Vertical strain top of base (ε_{vb})
(iv) Vertical strain top of subgrade (ε_{vs})

Table B.18 Spring Thaw Condition - Single Tire - Single Axle - Thaw to Bottom of Base - Pavement Structure 2/12/34/212[a] - Base M_R @ 25%

Case No.	Thawed Base M_R as a Percent of Summer Base M_R	Resilient Modulus (psi)				Pavement Response				
		Surface Course	Base	Subgrade Frozen	Subgrade Unfrozen	Percent of Full Load	δ (in.) (b)	ε_t (in/in X 10^{-6}) (b)	ε_{vb} (in/in X 10^{-6}) (b)	ε_{vs} (in/in X 10^{-6}) (b)
1	25	1,200,000	2,810	50,000	7,500	20 100	0.0160 0.0687	+285 +742	-1575 -5646	- 29 - 144
2	25	1,200,000	3,750	50,000	10,000	20 100	0.0138 0.0582	+270 +683	-1407 -4874	- 37 - 170
3	25	1,200,000	9,380	50,000	25,000	20 100	0.0084 0.0340	+222 +499	- 974 -2983	- 56 - 248
4	25	1,200,000	15,000	50,000	40,000	20 100	0.0065 0.0257	+197 +410	- 804 -2274	- 63 - 279

(a) Surface course = 2 in.
Base = 12 in.
Frozen subgrade = 34 in.
Unfrozen subgrade = 212 in.

(b) (i) Surface deflection (δ)
(ii) Horizontal strain bottom of surface course (ε_t)
(iii) Vertical strain top of base (ε_{vb})
(iv) Vertical strain top of subgrade (ε_{vs})

Table B.19 Spring Thaw Condition - Single Tire - Single Axle - Thaw to Bottom of Base - Pavement Structure 4/6/38/212(a) - Base M_R @ 25%

Case No.	Thawed Base M_R as a Percent of Summer Base M_R	Resilient Modulus (psi)				Pavement Response				
		Surface Course	Base	Subgrade Frozen	Subgrade Unfrozen	Percent of Full Load	δ (in.) (b)	ϵ_t (in/in X 10^{-6}) (b)	ϵ_{vb} (in/in X 10^{-6}) (b)	ϵ_{vs} (in/in X 10^{-6}) (b)
1	25	1,200,000	2,810	50,000	7,500	20 100	0.0053 0.0257	+ 90 +270	- 598 -2629	- 12 - 70
2	25	1,200,000	3,750	50,000	10,000	20 100	0.0048 0.0221	+ 87 +258	- 535 -2254	- 18 - 90
3	25	1,200,000	9,380	50,000	25,000	20 100	0.0033 0.0142	+ 78 +217	- 336 -1373	- 37 - 162
4	25	1,200,000	15,000	50,000	40,000	20 100	0.0028 0.0118	+ 73 +199	- 300 -1059	- 47 - 203

(a) Surface course = 4 in.
Base = 6 in.
Frozen subgrade = 38 in.
Unfrozen subgrade = 212 in.

(b) (i) Surface deflection (δ)
(ii) Horizontal strain bottom of surface course (ϵ_t)
(iii) Vertical strain top of base (ϵ_{vb})
(iv) Vertical strain top of subgrade (ϵ_{vs})

Table B.20 Spring Thaw Condition - Single Tire - Single Axle - Thaw to Bottom of Base - Pavement Structure 4/12/32/212(a) - Base M_R @ 25%

Case No.	Thawed Base M_R as a Percent of Summer Base M_R	Resilient Modulus (psi)				Pavement Response				
		Surface Course	Base	Subgrade Frozen	Subgrade Unfrozen	Percent of Full Load	δ (in.) (b)	ϵ_t (in/in X 10^{-6}) (b)	ϵ_{vb} (in/in X 10^{-6}) (b)	ϵ_{vs} (in/in X 10^{-6}) (b)
1	25	1,200,000	2,810	50,000	7,500	20 100	0.0067 0.0329	+ 95 +301	- 480 -2148	- 3 - 43
2	25	1,200,000	3,750	50,000	10,000	20 100	0.0058 0.0281	+ 92 +286	- 436 -1872	- 8 - 59
3	25	1,200,000	9,380	50,000	25,000	20 100	0.0038 0.0173	+ 83 +239	- 326 -1219	- 23 - 113
4	25	1,200,000	15,000	50,000	40,000	20 100	0.0031 0.0137	+ 77 +214	- 278 - 977	- 29 - 140

(a) Surface course = 4 in.
Base = 12 in.
Frozen subgrade = 32 in.
Unfrozen subgrade = 212 in.

(b) (i) Surface deflection (δ)
(ii) Horizontal strain bottom of surface course (ϵ_t)
(iii) Vertical strain top of base (ϵ_{vb})
(iv) Vertical strain top of subgrade (ϵ_{vs})

Table B.21 Spring Thaw Condition - Single Tire - Single Axle - Thaw to Bottom of Base - Pavement Structure 2/6/40/212(a) Base M_R @ 50%

Case No.	Thawed Base M_R as a Percent of Summer Base M_R	Resilient Modulus (psi)				Pavement Response				
		Surface Course	Base	Subgrade Frozen	Subgrade Unfrozen	Percent of Full Load	δ (in.) (b)	ε_t (in/in X 10^{-6}) (b)	ε_{vb} (in/in X 10^{-6}) (b)	ε_{vs} (in/in X 10^{-6}) (b)
1	50	1,200,000	5,625	50,000	7,500	20 100	0.0090 0.0380	+230 +523	-1299 -4373	- 74 - 311
2	50	1,200,000	7,500	50,000	10,000	20 100	0.0079 0.0326	+218 +477	-1138 -3675	- 86 - 352
3	50	1,200,000	18,750	50,000	25,000	20 100	0.0053 0.0205	+179 +339	- 750 -2051	- 121 - 476
4	50	1,200,000	30,000	50,000	40,000	20 100	0.0044 0.0163	+157 +274	- 603 -1468	- 135 - 518

Notes: (a) Surface course = 2 in.
Base = 6 in.
Frozen subgrade = 40 in.
Unfrozen subgrade = 212 in.

(b) (i) Surface deflection (δ)
(ii) Horizontal strain bottom of surface course (ε_t)
(iii) Vertical strain top of base (ε_{vb})
(iv) Vertical strain top of subgrade (ε_{vs})

Table B.22 Spring Thaw Condition - Single Tire - Single Axle - Thaw to Bottom of Base - Pavement Structure 2/12/32/212[a] - Base M_R @ 50%

Case No.	Thawed Base M_R as a Percent of Summer Base M_R	Resilient Modulus (psi)				Pavement Response				
		Surface Course	Base	Subgrade Frozen	Subgrade Unfrozen	Percent of Full Load	δ (in.) (b)	ε_t (in/in X 10^{-6}) (b)	ε_{vb} (in/in X 10^{-6}) (b)	ε_{vs} (in/in X 10^{-6}) (b)
1	50	1,200,000	5,625	50,000	7,500	20 100	0.0117 0.0494	+ 248 + 600	-1197 -3941	- 43 - 196
2	50	1,200,000	7,500	50,000	10,000	20 100	0.0100 0.0416	+ 233 + 543	-1066 -3375	- 49 - 221
3	50	1,200,000	18,750	50,000	25,000	20 100	0.0061 0.0238	+ 185 + 368	- 732 -1984	- 64 - 282
4	50	1,200,000	30,000	50,000	40,000	20 100	0.0047 0.0178	+ 160 + 286	- 597 -1466	- 67 - 295

Notes: (a) Surface course = 2 in.
Base = 12 in.
Frozen subgrade = 32 in.
Unfrozen subgrade = 212 in.

(b) (i) Surface deflection (δ)
(ii) Horizontal strain bottom of surface course (ε_t)
(iii) Vertical strain top of base (ε_{vb})
(iv) Vertical strain top of subgrade (ε_{vs})

Table B.23 Spring Thaw Condition - Single Tire - Single Axle - Thaw to Bottom of Base - Pavement Structure 4/6/38/212(a) - Base M_R @ 50%

Case No.	Thawed Base M_R as a Percent of Summer Base M_R	Resilient Modulus (psi)				Pavement Response				
		Surface Course	Base	Subgrade Frozen	Subgrade Unfrozen	Percent of Full Load	δ (in.) (b)	ε_t (in/in X 10^{-6}) (b)	ε_{vb} (in/in X 10^{-6}) (b)	ε_{vs} (in/in X 10^{-6}) (b)
1	50	1,200,000	5,625	50,000	7,500	20 100	0.0045 0.0211	+ 83 +240	- 454 -1810	- 24 - 114
2	50	1,200,000	7,500	50,000	10,000	20 100	0.0040 0.0183	+ 80 +228	- 402 -1549	- 30 - 138
3	50	1,200,000	18,750	50,000	25,000	20 100	0.0028 0.0120	+ 71 +190	- 272 - 935	- 50 - 216
4	50	1,200,000	30,000	50,000	40,000	20 100	0.0024 0.0100	+ 65 +170	- 223 - 718	- 59 - 252

Notes: (a) Surface course = 4 in.
Base = 6 in.
Frozen Subgrade = 38 in.
Unfrozen subgrade = 212 in.

(b) (i) Surface deflection (δ)
(ii) Horizontal strain bottom of surface course (ε_t)
(iii) Vertical strain top of base (ε_{vb})
(iv) Vertical strain top of subgrade (ε_{vs})

Table B.24 Spring Thaw Condition - Single Tire - Single Axle - Thaw to Bottom of Base - Pavement Structure 4/12/32/212[(a)] - Base M_R @ 50%

Case No.	Thawed Base M_R as a Percent of Summer Base M_R	Resilient Modulus (psi)				Pavement Response				
		Surface Course	Base	Subgrade Frozen	Subgrade Unfrozen	Percent of Full Load	δ (in.) (b)	ε_t (in/in X 10^{-6}) (b)	ε_{vb} (in/in X 10^{-6}) (b)	ε_{vs} (in/in X 10^{-6}) (b)
1	50	1,200,000	5,625	50,000	7,500	20 100	0.0054 0.0257	+ 89 +265	- 383 -1545	- 12 - 73
2	50	1,200,000	7,500	50,000	10,000	20 100	0.0047 0.0219	+ 85 +251	- 350 -1350	- 17 - 91
3	50	1,200,000	18,750	50,000	25,000	20 100	0.0031 0.0135	+ 73 +202	- 258 - 878	- 31 145
4	50	1,200,000	30,000	50,000	40,000	20 100	0.0025 0.0107	+ 67 +176	- 218 - 698	- 36 - 165

Notes: (a) Surface course 4 in.
Base = 12 in.
Frozen Subgrade = 32 in.
Unfrozen subgrade = 212 in.

(b) (i) Surface deflection (δ)
(ii) Horizontal strain bottom of surface course (ε_t)
(iii) Vertical strain top of base (ε_{vb})
(iv) Vertical strain top of subgrade (ε_{vs})

Table B.25 Spring Thaw Condition - Dual Tires - Single Axle - Thaw to Bottom of Base - Pavement Structure 2/6/40/212[a] - Base M_R @ 50% - Between Wheels

Case No.	Thawed Base M_R as a Percent of Summer Base M_R	Resilient Modulus (psi)				Pavement Response Between Wheels				
		Surface Course	Base	Subgrade Frozen	Subgrade Unfrozen	Percent of Full Load	δ (in.) (b)	ε_t (in/in X 10^{-6}) (b)	ε_{vb} (in/in X 10^{-6}) (b)	ε_{vs} (in/in X 10^{-6}) (b)
1	50	1,200,000	5,625	50,000	7,500	20 100	0.0062 0.0306	+ 28 + 52	- 586 -2912	- 47 - 226
2	50	1,200,000	7,500	50,000	10,000	20 100	0.0052 0.0258	+ 32 + 72	- 469 -2332	- 52 - 253
3	50	1,200,000	18,750	50,000	25,000	20 100	0.0031 0.0154	+ 39 +111	- 206 -1074	- 68 - 324
4	50	1,200,000	30,000	50,000	40,000	20 100	0.0024 0.0120	+ 38 +118	- 126 - 686	- 71 - 344

Notes: (a) Surface course = 2 in.
Base = 6 in.
Frozen subgrade = 40 in.
Unfrozen subgrade = 212 in.

(b) (i) Surface deflection (δ)
(ii) Horizontal strain bottom of surface course (ε_t)
(iii) Vertical strain top of base (ε_{vb})
(iv) Vertical strain top of subgrade (ε_{vs})

Table B.26 Spring Thaw Condition - Dual Tires - Single Axle - Thaw to Bottom of Base - Pavement Structure 2/12/34/212[a] - Base M_R @ 50% - Between Wheels

Case No.	Thawed Base M_R as a Percent of Summer Base M_R	Resilient Modulus (psi)				Pavement Response Between Wheels				
		Surface Course	Base	Subgrade Frozen	Subgrade Unfrozen	Percent of Full Load	δ (in.) (b)	ε_t (in/in X 10^{-6}) (b)	ε_{vb} (in/in X 10^{-6}) (b)	ε_{vs} (in/in X 10^{-6}) (b)
1	50	1,200,000	5,625	50,000	7,500	20 100	0.0084 0.0410	- 20 0	- 524 -2621	- 34 - 164
2	50	1,200,000	7,500	50,000	10,000	20 100	0.0069 0.0339	- 20 - 30	- 426 -2140	- 38 - 184
3	50	1,200,000	18,750	50,000	25,000	20 100	0.0037 0.0183	- 40 - 90	- 199 -1040	- 47 - 230
4	50	1,200,000	30,000	50,000	40,000	20 100	0.0027 0.0133	- 40 -110	- 125 - 677	- 49 - 278

Notes: (a) Surface course = 2 in.
Base = 12 in.
Frozen subgrade = 34 in.
Unfrozen subgrade = 212 in.

(b) (i) Surface deflection (δ)
(ii) Horizontal strain bottom of surface course (ε_t)
(iii) Vertical strain top of base (ε_{vb})
(iv) Vertical strain top of subgrade (ε_{vs})

Table B.27 Spring Thaw Condition - Dual Tires - Single Axle - Thaw to Bottom of Base - Pavement Structure 4/6/38/212(a) - Base M_R @ 50% - Between Wheels

Case No.	Thawed Base M_R as a Percent of Summer Base M_R	Resilient Modulus (psi)				Pavement Response Between Wheels				
		Surface Course	Base	Subgrade Frozen	Subgrade Unfrozen	Percent of Full Load	δ (in.) (b)	ε_t (in/in X 10^{-6}) (b)	ε_{vb} (in/in X 10^{-6}) (b)	ε_{vs} (in/in X 10^{-6}) (b)
1	50	1,200,000	5,625	50,000	7,500	20 100	0.0039 0.0193	+ 7 +62	- 293 -1452	- 19 - 95
2	50	1,200,000	7,500	50,000	10,000	20 100	0.0033 0.0165	+ 5 +52	- 243 -1213	- 24 - 123
3	50	1,200,000	18,750	50,000	25,000	20 100	0.0021 0.0105	0 +27	- 131 - 654	- 36 - 176
4	50	1,200,000	30,000	50,000	40,000	20 100	0.0017 0.0085	+ 2 +17	- 92 - 469	- 41 - 201

Notes: (a) Surface course = 4 in.
Base = 6 in.
Frozen subgrade = 38 in.
Unfrozen subgrade = 212 in.

(b) (i) Surface deflection (δ)
(ii) Horizontal strain bottom of surface course (ε_t)
(iii) Vertical strain top of base (ε_{vb})
(iv) Vertical strain top of subgrade (ε_{vs})

Table B.28 Spring Thaw Condition - Dual Tires - Single Axle - Thaw to Bottom of Base - Pavement Structure 4/12/32/212[a] - Base M_R @ 50% - Between Wheels

Case No.	Thawed Base M_R as a Percent of Summer Base M_R	Resilient Modulus (psi)				Pavement Response Between Wheels				
		Surface Course	Base	Subgrade Frozen	Subgrade Unfrozen	Percent of Full Load	δ (in.) (b)	ε_t (in/in X 10^{-6}) (b)	ε_{vb} (in/in X 10^{-6}) (b)	ε_{vs} (in/in X 10^{-6}) (b)
1	50	1,200,000	5,625	50,000	7,500	20 100	0.0048 0.0237	+10 +80	- 247 -1234	- 13 - 66
2	50	1,200,000	7,500	50,000	10,000	20 100	0.0040 0.0200	+10 +70	- 210 -1050	- 17 - 82
3	50	1,200,000	18,750	50,000	25,000	20 100	0.0024 0.0119	0 +40	- 122 - 613	- 27 - 130
4	50	1,200,000	30,000	50,000	40,000	20 100	0.0019 0.0092	0 +20	- 90 - 454	- 30 - 146

Notes: (a) Surface course = 4 in.
Base = 12 in.
Frozen subgrade = 32 in.
Unfrozen subgrade = 212 in.

(b) (i) Surface deflection (δ)
(ii) Horizontal strain bottom of surface course (ε_t)
(iii) Vertical strain top of base (ε_{vb})
(iv) Vertical strain top of subgrade (ε_{vs})

Table B.29 Spring Thaw Condition - Dual Tires - Single Axle - Thaw to Bottom of Base - Pavement Structure 2/6/40/212[(a)] - Base M_R @ 50% - Beneath Tire

Case No.	Thawed Base M_R as a Percent of Summer Base M_R	Resilient Modulus (psi)				Pavement Response Beneath Inside Tire of Duals				
		Surface Course	Base	Subgrade Frozen	Subgrade Unfrozen	Percent of Full Load	δ (in.) (b)	ε_t (in/in X 10^{-6}) (b)	ε_{vb} (in/in X 10^{-6}) (b)	ε_{vs} (in/in X 10^{-6}) (b)
1	50%	1,200,000	5,625	50,000	7,500	20 100	0.0060 0.0288	+131 +304	- 836 -3182	- 43 - 204
2	50%	1,200,000	7,500	50,000	10,000	20 100	0.0053 0.0247	+125 +283	- 727 -2665	- 50 - 232
3	50%	1,200,000	18,750	50,000	25,000	20 100	0.0035 0.0155	+109 +225	- 476 -1519	- 70 - 312
4	50%	1,200,000	30,000	50,000	40,000	20 100	0.0029 0.0125	+100 +195	- 385 -1137	- 76 - 340

Notes: (a) Surface course = 2 in.
Base = 6 in.
Frozen subgrade = 40 in.
Unfrozen subgrade = 212 in.

(b) (i) Surface deflection (δ)
(ii) Horizontal strain bottom of surface course (ε_t)
(iii) Vertical strain top of base (ε_{vb})
(iv) Vertical strain top of subgrade (ε_{vs})

Table B.30 Spring Thaw Condition - Dual Tires - Single Axle - Thaw to Bottom of Base - Pavement Structure 2/12/34/212[a] - Base M_R @ 50% - Beneath Tire

Case No.	Thawed Base M_R as a Percent of Summer Base M_R	Resilient Modulus (psi)				Pavement Response Beneath Inside Tire of Duals				
		Surface Course	Base	Subgrade Frozen	Subgrade Unfrozen	Percent of Full Load	δ (in.) (b)	ε_t (in/in X 10^{-6}) (b)	ε_{vb} (in/in X 10^{-6}) (b)	ε_{vs} (in/in X 10^{-6}) (b)
1	50%	1,200,000	5,625	50,000	7,500	20 100	0.0077 0.0382	+140 +345	- 769 -2933	- 24 - 142
2	50%	1,200,000	7,500	50,000	10,000	20 100	0.0066 0.0319	+133 +317	- 683 -2497	- 29 - 161
3	50%	1,200,000	18,750	50,000	25,000	20 100	0.0040 0.0181	+112 +238	- 468 -1484	- 41 - 204
4	50%	1,200,000	30,000	50,000	40,000	20 100	0.0031 0.0136	+101 +200	- 385 -1126	- 43 - 211

Notes: (a) Surface course = 2 in.
Base = 12 in.
Frozen subgrade = 34 in.
Unfrozen subgrade = 212 in.

(b) (i) Surface deflection (δ)
(ii) Horizontal strain bottom of surface course (ε_t)
(iii) Vertical strain top of base (ε_{vb})
(iv) Vertical strain top of subgrade (ε_{vs})

Table B.31 Spring Thaw Condition - Dual Tires - Single Axle - Thaw to Bottom of Base - Pavement Structure 4/6/38/212(a) - Base M_R @ 50% - Beneath Tire

Case No.	Thawed Base M_R as a Percent of Summer Base M_R	Resilient Modulus (psi)				Pavement Response Beneath Inside Tire of Duals				
		Surface Course	Base	Subgrade Frozen	Subgrade Unfrozen	Percent of Full Load	δ (in.) (b)	ε_t (in/in X 10^{-6}) (b)	ε_{vb} (in/in X 10^{-6}) (b)	ε_{vs} (in/in X 10^{-6}) (b)
1	50%	1,200,000	5,625	50,000	7,500	20 100	0.0034 0.0180	+ 42 +142	- 309 -1463	- 13 - 86
2	50%	1,200,000	7,500	50,000	10,000	20 100	0.0030 0.0156	+ 41 +134	- 267 -1234	- 18 - 113
3	50%	1,200,000	18,750	50,000	25,000	20 100	0.0021 0.0101	+ 36 +111	- 173 - 715	- 32 - 160
4	50%	1,200,000	30,000	50,000	40,000	20 100	0.0018 0.0083	+ 34 +100	- 140 - 544	- 38 - 188

Notes: (a) Surface course = 4 in.
Base = 6 in.
Frozen subgrade = 38 in.
Unfrozen subgrade = 212 in.

(b) (i) Surface deflection (δ)
(ii) Horizontal strain bottom of surface course (ε_t)
(iii) Vertical strain top of base (ε_{vb})
(iv) Vertical strain top of subgrade (ε_{vs})

Table B.32 Spring Thaw Condition - Dual Tires - Single Axle - Thaw to Bottom of Base - Pavement Structure 4/12/32/212[(a)] - Base M_R @ 50% - Beneath Tire

Case No.	Thawed Base M_R as a Percent of Summer Base M_R	Resilient Modulus (psi)				Pavement Response Beneath Inside Tire of Duals				
		Surface Course	Base	Subgrade Frozen	Subgrade Unfrozen	Percent of Full Load	δ (in.) (b)	ϵ_t (in/in X 10^{-6}) (b)	ϵ_{vb} (in/in X 10^{-6}) (b)	ϵ_{vs} (in/in X 10^{-6}) (b)
1	50%	1,200,000	5,625	50,000	7,500	20 100	0.0043 0.0225	+ 45 +163	- 275 -1275	- 7 - 61
2	50%	1,200,000	7,500	50,000	10,000	20 100	0.0037 0.0191	+ 43 +152	- 241 -1098	- 10 - 76
3	50%	1,200,000	18,750	50,000	25,000	20 100	0.0024 0.0115	+ 37 +119	- 165 - 680	- 21 - 119
4	50%	1,200,000	30,000	50,000	40,000	20 100	0.0019 0.0090	+ 34 +104	- 137 - 532	- 25 - 133

Notes: (a) Surface course = 4 in.
Base = 12 in.
Frozen subgrade = 32 in.
Unfrozen subgrade = 212 in.

(b) (i) Surface deflection (δ)
(ii) Horizontal strain bottom of surface course (ϵ_t)
(iii) Vertical strain top of base (ϵ_{vb})
(iv) Vertical strain top of subgrade (ϵ_{vs})

Table B.33 Spring Thaw Condition - Dual Tires - Tandem Axle - Thaw to Bottom of Base - Pavement Structure 2/6/40/212(a) - Base M_R @ 50% - Beneath Tire

Case No.	Thawed Base M_R as a Percent of Summer Base M_R	Resilient Modulus (psi)				Pavement Response Beneath Inside Tire of Duals				
		Surface Course	Base	Subgrade Frozen	Subgrade Unfrozen	Percent of Full Load	δ (in.) (b)	ε_t (in/in X 10^{-6}) (b)	ε_{vb} (in/in X 10^{-6}) (b)	ε_{vs} (in/in X 10^{-6}) (b)
1	50%	1,200,000	5,625	50,000	7,500	20 100	0.0057 0.0281	+ 117 + 284	- 720 -2791	- 35 - 170
2	50%	1,200,000	7,500	50,000	10,000	20 100	0.0050 0.0240	+ 113 + 266	- 631 -2349	- 42 - 196
3	50%	1,200,000	18,750	50,000	25,000	20 100	0.0033 0.0148	+ 98 + 215	- 419 -1363	- 59 - 268
4	50%	1,200,000	30,000	50,000	40,000	20 100	0.0027 0.0118	+ 90 + 188	- 277 - 757	- 68 - 305

Notes: (a) Surface course = 2 in.
Base = 6 in.
Frozen subgrade = 40 in.
Unfrozen subgrade = 212 in.

(b) (i) Surface deflection (δ)
(ii) Horizontal strain bottom of surface course (ε_t)
(iii) Vertical strain top of base (ε_{vb})
(iv) Vertical strain top of subgrade (ε_{vs})

Table B.34 Spring Thaw Condition - Dual Tires - Tandem Axle - Thaw to Bottom of Base - Pavement Structure 2/12/34/212(a) - Base M_R @ 50% - Beneath Tire

Case No.	Thawed Base M_R as a Percent of Summer Base M_R	Resilient Modulus (psi)				Pavement Response Beneath Inside Tire of Duals				
		Surface Course	Base	Subgrade Frozen	Subgrade Unfrozen	Percent of Full Load	δ (in.) (b)	ε_t (in/in X 10^{-6}) (b)	ε_{vb} (in/in X 10^{-6}) (b)	ε_{vs} (in/in X 10^{-6}) (b)
1	50%	1,200,000	5,625	50,000	7,500	20 100	0.0072 0.0362	+ 124 + 319	- 661 -2574	- 18 - 114
2	50%	1,200,000	7,500	50,000	10,000	20 100	0.0061 0.0302	+ 119 + 295	- 590 -2200	- 23 - 133
3	50%	1,200,000	18,750	50,000	25,000	20 100	0.0037 0.0170	+ 101 + 225	- 411 -1333	- 34 - 172
4	50%	1,200,000	30,000	50,000	40,000	20 100	0.0029 0.0127	+ 91 + 192	- 277 - 756	- 35 - 173

Notes: (a) Surface course = 2 in.
Base = 12 in.
Frozen subgrade = 34 in.
Unfrozen subgrade = 212 in.

(b) (i) Surface deflection (δ)
(ii) Horizontal strain bottom of surface course (ε_t)
(iii) Vertical strain top of base (ε_{vb})
(iv) Vertical strain top of subgrade (ε_{vs})

Table B.35 Spring Thaw Condition - Dual Tires - Tandem Axle - Thaw to Bottom of Base - Pavement Structure 4/6/38/212(a) - Base M_R @ 50% - Beneath Tire

Case No.	Thawed Base M_R as a Percent of Summer Base M_R	Resilient Modulus (psi)				Pavement Response Beneath Inside Tire of Duals				
		Surface Course	Base	Subgrade Frozen	Subgrade Unfrozen	Percent of Full Load	δ (in.) (b)	ε_t (in/in X 10^{-6}) (b)	ε_{vb} (in/in X 10^{-6}) (b)	ε_{vs} (in/in X 10^{-6}) (b)
1	50%	1,200,000	5,625	50,000	7,500	20 100	0.0035 0.0186	+ 36 + 129	- 261 -1251	- 10 - 67
2	50%	1,200,000	7,500	50,000	10,000	20 100	0.0031 0.0159	+ 35 + 122	- 226 -1058	- 14 - 85
3	50%	1,200,000	18,750	50,000	25,000	20 100	0.0021 0.0100	+ 31 + 101	- 147 - 623	- 26 - 136
4	50%	1,200,000	30,000	50,000	40,000	20 100	0.0017 0.0081	+ 29 + 92	- 110 - 460	- 37 - 180

Notes: (a) Surface course = 4 in.
Base = 6 in.
Frozen subgrade = 38 in.
Unfrozen subgrade = 212 in.

(b) (i) Surface deflection (δ)
(ii) Horizontal strain bottom of surface course (ε_t)
(iii) Vertical strain top of base (ε_{vb})
(iv) Vertical strain top of subgrade (ε_{vs})

Table B.36 Spring Thaw Condition - Dual Tires - Tandem Axle - Thaw to Bottom of Base - Pavement Structure 4/12/32/212 (a) - Base M_R @ 50% - Beneath Tire

Case No.	Thawed Base M_R as a Percent of Summer Base M_R	Resilient Modulus (psi)				Pavement Response Beneath Inside Tire of Duals				
		Surface Course	Base	Subgrade Frozen	Subgrade Unfrozen	Percent of Full Load	δ (in.) (b)	ε_t (in/in X 10^{-6}) (b)	ε_{vb} (in/in X 10^{-6}) (b)	ε_{vs} (in/in X 10^{-6}) (b)
1	50%	1,200,000	5,625	50,000	7,500	20 100	0.0043 0.0226	+ 39 + 148	- 235 - 1087	- 4 - 43
2	50%	1,200,000	7,500	50,000	10,000	20 100	0.0037 0.0190	+ 37 + 138	- 205 - 940	- 8 - 60
3	50%	1,200,000	18,750	50,000	25,000	20 100	0.0023 0.0112	+ 32 + 109	- 140 - 593	- 17 - 99
4	50%	1,200,000	30,000	50,000	40,000	20 100	0.0018 0.0086	+ 30 + 96	- 106 - 380	- 23 - 119

Notes: (a) Surface course = 4 in.
Base = 12 in.
Frozen subgrade = 32 in.
Unfrozen subgrade = 212 in.

(b) (i) Surface deflection (δ)
(ii) Horizontal strain bottom of surface course (ε_t)
(iii) Vertical strain top of base (ε_{vb})
(iv) Vertical strain top of subgrade (ε_{vs})

Table B.37 Spring Thaw Condition - Dual Tires - Tandem Axle - Thaw to Bottom of Base - Pavement Structure 2/6/40/212(a) - Base M_R @ 25% - Beneath Tire

Case No.	Thawed Base M_R as a Percent of Summer Base M_R	Resilient Modulus (psi)				Pavement Response Beneath Inside Tire of Duals				
		Surface Course	Base	Subgrade Frozen	Subgrade Unfrozen	Percent of Full Load	δ (in.) (b)	ε_t (in/in X 10^{-6}) (b)	ε_{vb} (in/in X 10^{-6}) (b)	ε_{vs} (in/in X 10^{-6}) (b)
1	25%	1,200,000	2,810	50,000	7,500	20 100	0.0073 0.0369	+ 129 + 334	- 958 -4207	- 22 - 121
2	25%	1,200,000	3,750	50,000	10,000	20 100	0.0064 0.0312	+ 124 + 313	- 868 -3566	- 28 - 145
3	25%	1,200,000	9,380	50,000	25,000	20 100	0.0041 0.0187	+ 109 + 254	- 570 -2057	- 48 - 222
4	25%	1,200,000	15,000	50,000	40,000	20 100	0.0033 0.0147	+ 102 + 228	- 462 -1556	- 57 - 258

Notes: (a) Surface course = 2 in.
Base = 6 in.
Frozen subgrade = 40 in.
Unfrozen subgrade = 212 in.

(b) (i) Surface deflection (δ)
(ii) Horizontal strain bottom of surface course (ε_t)
(iii) Vertical strain top of base (ε_{vb})
(iv) Vertical strain top of subgrade (ε_{vs})

Table B.38 Spring Thaw Condition - Dual Tires - Tandem Axle - Thaw to Bottom of Base - Pavement Structure 2/12/34/212(a) - Base M_R @ 25% - Beneath Tire

Case No.	Thawed Base M_R as a Percent of Summer Base M_R	Resilient Modulus (psi)				Pavement Response Beneath Inside Tire of Duals				
		Surface Course	Base	Subgrade Frozen	Subgrade Unfrozen	Percent of Full Load	δ (in.) (b)	ε_t (in/in X 10^{-6}) (b)	ε_{vb} (in/in X 10^{-6}) (b)	ε_{vs} (in/in X 10^{-6}) (b)
1	25%	1,200,000	2,810	50,000	7,500	20 100	0.0095 0.0499	+ 136 + 383	- 866 -3723	- 10 - 80
2	25%	1,200,000	3,750	50,000	10,000	20 100	0.0082 0.0419	+ 132 + 356	- 776 -3209	- 15 - 100
3	25%	1,200,000	9,380	50,000	25,000	20 100	0.0050 0.0236	+ 115 + 278	- 540 -1951	- 28 - 153
4	25%	1,200,000	15,000	50,000	40,000	20 100	0.0039 0.0176	+ 105 + 243	- 449 -1509	- 34 - 171

Notes: (a) Surface course = 2 in.
Base = 12 in.
Frozen subgrade = 34 in.
Unfrozen subgrade = 212 in.

(b) (i) Surface deflection (δ)
(ii) Horizontal strain bottom of surface course (ε_t)
(iii) Vertical strain top of base (ε_{vb})
(iv) Vertical strain top of subgrade (ε_{vs})

Table B.39 Spring Thaw Condition - Dual Tires - Tandem Axle - Thaw to Bottom of Base - Pavement Structure 4/6/38/212(a) - Base M_R @ 25% - Beneath Tire

Case No.	Thawed Base M_R as a Percent of Summer Base M_R	Resilient Modulus (psi)				Pavement Response Beneath Inside Tire of Duals				
		Surface Course	Base	Subgrade Frozen	Subgrade Unfrozen	Percent of Full Load	δ (in.) (b)	ε_t (in/in X 10^{-6}) (b)	ε_{vb} (in/in X 10^{-6}) (b)	ε_{vs} (in/in X 10^{-6}) (b)
1	25%	1,200,000	2,810	50,000	7,500	20 100	0.0043 0.0225	+ 39 + 150	- 380 -1856	- 3 - 35
2	25%	1,200,000	3,750	50,000	10,000	20 100	0.0037 0.0191	+ 38 + 141	- 325 -1585	- 7 - 52
3	25%	1,200,000	9,380	50,000	25,000	20 100	0.0024 0.0117	+ 34 + 116	- 203 - 930	- 19 - 104
4	25%	1,200,000	15,000	50,000	40,000	20 100	0.0020 0.0093	+ 32 + 106	- 163 - 709	- 24 - 128

Notes: (a) Surface course = 4 in.
Base = 12 in.
Frozen subgrade = 38 in.
Unfrozen subgrade = 212 in.

(b) (i) Surface deflection (δ)
(ii) Horizontal strain bottom of surface course (ε_t)
(iii) Vertical strain top of base (ε_{vb})
(iv) Vertical strain top of subgrade (ε_{vs})

Table B.40 Spring Thaw Condition - Dual Tires - Tandem Axle - Thaw to Bottom of Base - Pavement Structure 4/12/32/212(a) - Base M_R @ 25% - Beneath Tire

Case No.	Thawed Base M_R as a Percent of Summer Base M_R	Resilient Modulus (psi)				Pavement Response Beneath Inside Tire of Duals				
		Surface Course	Base	Subgrade Frozen	Subgrade Unfrozen	Percent of Full Load	δ (in.) (b)	ε_t (in/in X 10^{-6}) (b)	ε_{vb} (in/in X 10^{-6}) (b)	ε_{vs} (in/in X 10^{-6}) (b)
1	25%	1,200,000	2,810	50,000	7,500	20 100	0.0057 0.0284	+ 43 + 175	- 340 -1534	- 1 - 18
2	25%	1,200,000	3,750	50,000	10,000	20 100	0.0048 0.0239	+ 41 + 163	- 291 -1336	- 3 - 34
3	25%	1,200,000	9,380	50,000	25,000	20 100	0.0028 0.0140	+ 36 + 130	- 187 - 841	- 13 - 78
4	25%	1,200,000	15,000	50,000	40,000	20 100	0.0022 0.0108	+ 33 + 115	- 153 - 664	- 17 - 95

Notes: (a) Surface course = 4 in.
Base = 12 in.
Frozen subgrade = 32 in.
Unfrozen subgrade = 212 in.

(b) (i) Surface deflection (δ)
(ii) Horizontal strain bottom of surface course (ε_t)
(iii) Vertical strain top of base (ε_{vb})
(iv) Vertical strain top of subgrade (ε_{vs})

Table B.41 Spring Thaw Condition - Single Tire - Single Axle - Thaw 4 in. into Subgrade - Pavement Structure 2/6/4/36[a]

Subgrade Type	Reduction in Subgrade Resilient Modulus	Resilient Modulus (psi)				Pavement Response				
		Surface Course	Base	Subgrade Thawed	Subgrade Frozen	Percent of Full Load	δ (in.) (b)	ε_t (in/in X 10^{-6}) (b)	ε_{vb} (in/in X 10^{-6}) (b)	ε_{vs} (in/in X 10^{-6}) (b)
Fine-grain	95	1,200,000	560	375	50,000	20 100	0.0275 0.1330	+ 356 +1040	- 2940 -13,140	- 2381 -11,340
	85	1,200,000	1,690	1,120	50,000	20 100	0.0184 0.0801	+ 306 + 824	- 1958 - 7449	- 1525 - 6532
Coarse-grain	75	1,200,000	15,000	10,000	50,000	20 100	0.0073 0.0313	+198 +412	- 800 - 2260	- 436 - 1760
	50	1,200,000	30,000	20,000	50,000	20 100	0.0055 0.0239	+161 +291	- 593 - 1457	- 274 - 1066

Notes: (a) Surface course = 2 in.
Base = 6 in.
Thawed subgrade = 4 in.
Frozen subgrade = 36 in.

(b) (i) Surface deflection (δ)
(ii) Horizontal strain bottom of surface course (ε_t)
(iii) Vertical strain top of base (ε_{vb})
(iv) Vertical strain top of subgrade (ε_{vs})

Table B.42 Spring Thaw Condition - Single Tire - Single Axle - Thaw - 4 in. into Subgrade - Pavement Structure 2/12/4/30[a]

Subgrade Type	Reduction in Subgrade Resilient Modulus	Resilient Modulus (psi)				Pavement Response				
		Surface Coarse	Base	Subgrade Thawed	Subgrade Frozen	Percent of Full Load	δ (in.) (b)	ε_t (in/in X 10^{-6}) (b)	ε_{vb} (in/in X 10^{-6}) (b)	ε_{vs} (in/in X 10^{-6}) (b)
Fine-grain	95	1,200,000	560	375	50,000	20 100	0.0332 0.1740	+ 368 +1120	- 2565 -11,610	-1525 -8388
	85	1,200,000	1,690	1,120	50,000	20 100	0.0219 0.1010	+ 322 + 890	- 1798 - 6888	- 953 -4534
Coarse-grain	75	1,200,000	15,000	10,000	50,000	20 100	0.0080 0.0342	+ 200 + 420	- 800 - 2250	- 243 -1030
	50	1,200,000	30,000	20,000	50,000	20 100	0.0057 0.0246	+ 160 + 288	- 596 - 1462	- 134 - 587

Notes: (a) Surface course = 2 in.
Base = 12 in.
Thawed subgrade = 4 in.
Frozen subgrade = 30 in.

(b) (i) Surface deflection (δ)
(ii) Horizontal strain bottom of surface course (ε_t)
(iii) Vertical strain top of base (ε_{vb})
(iv) Vertical strain top of subgrade (ε_{vs})

Table B.43 Spring Thaw Condition - Single Tire - Single Axle - Thaw 4 in. into Subgrade - Pavement Structure 4/6/4/34[(a)]

Subgrade Type	Reduction in Subgrade Resilient Modulus	Resilient Modulus (psi)				Pavement Response				
		Surface Coarse	Base	Subgrade Thawed	Subgrade Frozen	Percent of Full Load	δ (in.) (b)	ε_t (in/in X 10^{-6}) (b)	ε_{vb} (in/in X 10^{-6}) (b)	ε_{vs} (in/in X 10^{-6}) (b)
Fine-grain	95	1,200,000	560	375	50,000	20 100	0.0107 0.0559	+104 +376	-1019 -5129	- 715 -3976
	85	1,200,000	1,690	1,120	50,000	20 100	0.0070 0.0367	+ 98 +317	- 599 -2917	- 429 -2323
Coarse-grain	75	1,200,000	15,000	10,000	50,000	20 100	0.0040 0.0192	+ 77 +214	- 279 - 978	- 170 - 740
	50	1,200,000	30,000	20,000	50,000	20 100	0.0033 0.0164	+ 67 +178	- 216 - 690	- 117 - 498

Notes: (a) Surface course = 4 in.
Base = 6 in.
Thawed subgrade = 4 in.
Frozen subgrade = 34 in.

(b) (i) Surface deflection (δ)
(ii) Horizontal strain bottom of surface course (ε_t)
(iii) Vertical strain top of base (ε_{vb})
(iv) Vertical strain top of subgrade (ε_{vs})

Table B.44 Spring Thaw Condition - Single Tire - Single Axle - Thaw 4 in. into Subgrade - Pavement Structure 4/12/4/28 (a)

Subgrade Type	Reduction in Subgrade Resilient Modulus	Resilient Modulus (psi)				Pavement Response				
		Surface Coarse	Base	Subgrade Thawed	Subgrade Frozen	Percent of Full Load	δ (in.) (b)	ε_t (in/in X 10^{-6}) (b)	ε_{vb} (in/in X 10^{-6}) (b)	ε_{vs} (in/in X 10^{-6}) (b)
Fine-grain	95	1,200,000	560	375	50,000	20 100	0.0144 0.0693	+107 +407	- 935 -4145	- 566 -2995
	85	1,200,000	1,690	1,120	50,000	20 100	0.0086 0.0454	+100 +343	- 541 -2554	- 295 -1773
Coarse-grain	75	1,200,000	15,000	10,000	50,000	20 100	0.0043 0.0209	+ 78 +221	- 271 - 953	- 104 - 507
	50	1,200,000	30,000	20,000	50,000	20 100	0.0034 0.0169	+ 67 +179	- 216 - 691	- 68 - 325

Notes: (a) Surface course = 4 in.
Base = 12 in.
Thawed subgrade = 4 in.
Frozen subgrade = 28 in.

(b) (i) Surface deflection (δ)
(ii) Horizontal strain bottom of surface course (ε_t)
(iii) Vertical strain top of base (ε_{vb})
(iv) Vertical strain top of subgrade (ε_{vs})

Table B.45 Spring Thaw Condition - Dual Tires - Single Axle - Thaw 4 in. into Subgrade - Pavement Structure 2/6/4/36(a) - Beneath Tire

Subgrade Type	Reduction in Subgrade Resilient Modulus	Resilient Modulus (psi)				Pavement Response Beneath Inside Tire of Duals				
		Surface Course	Base	Subgrade Thawed	Subgrade Frozen	Percent of Full Load	δ (in.) (b)	ϵ_t (in/in X 10^{-6}) (b)	ϵ_{vb} (in/in X 10^{-6}) (b)	ϵ_{vs} (in/in X 10^{-6}) (b)
Fine-grain	95	1,200,000	560	375	50,000	20 100	0.0197 0.1064	+185 +616	- 2250 -11,200	-1692 -9489
	85	1,200,000	1,690	1,120	50,000	20 100	0.0110 0.0568	+164 +466	- 1293 - 5888	- 947 -4983
Coarse-grain	75	1,200,000	15,000	10,000	50,000	20 100	0.0040 0.0177	+118 +259	- 512 - 1685	- 263 -1179
	50	1,200,000	30,000	20,000	50,000	20 100	0.0030 0.0127	+102 +205	- 385 - 1124	- 159 - 707

Notes: (a) Surface course = 2 in.
Base = 6 in.
Thawed subgrade = 4 in.
Unfrozen subgrade = 36 in.

(b) (i) Surface deflection (δ)
(ii) Horizontal strain bottom of surface course (ϵ_t)
(iii) Vertical strain top of base (ϵ_{vb})
(iv) Vertical strain top of subgrade (ϵ_{vs})

Table B.46 Spring Thaw Condition - Dual Tires - Single Axle - Thaw 4 in. into Subgrade - Pavement Structure 2/12/4/30(a) - Beneath Tire

Subgrade Type	Reduction in Subgrade Resilient Modulus	Resilient Modulus (psi)				Pavement Response Beneath Inside Tire of Duals				
		Surface Course	Base	Subgrade Thawed	Subgrade Frozen	Percent of Full Load	δ (in.) (b)	ε_t (in/in X 10^{-6}) (b)	ε_{vb} (in/in X 10^{-6}) (b)	ε_{vs} (in/in X 10^{-6}) (b)
Fine-grain	95	1,200,000	560	375	50,000	20 100	0.0274 0.1436	+194 +701	-2087 -9771	-1325 -7403
	85	1,200,000	1,690	1,120	50,000	20 100	0.0142 0.0761	+170 +519	-1225 -5484	- 658 -3800
Coarse-grain	75	1,200,000	15,000	10,000	50,000	20 100	0.0044 0.0201	+119 +265	- 509 -1678	- 152 - 759
	50	1,200,000	30,000	20,000	50,000	20 100	0.0031 0.0133	+102 +204	- 384 -1126	- 87 - 426

Notes: (a) Surface course = 2 in.
Base = 12 in.
Thawed subgrade = 4 in.
Frozen subgrade = 30 in.

(b) (i) Surface deflection (δ)
(ii) Horizontal strain bottom of surface course (ε_t)
(iii) Vertical strain top of base (ε_{vb})
(iv) Vertical strain top of subgrade (ε_{vs})

Table B.47 Spring Thaw Condition - Dual Tires - Single Axle - Thaw 4 in. into Subgrade - Pavement Structure 4/6/4/34(a) - Beneath Tire

Subgrade Type	Reduction in Subgrade Resilient Modulus	Resilient Modulus (psi)				Pavement Response Beneath Inside Tire of Duals				
		Surface Course	Base	Subgrade Thawed	Subgrade Frozen	Percent of Full Load	δ (in.) (b)	ε_t (in/in X 10^{-6}) (b)	ε_{vb} (in/in X 10^{-6}) (b)	ε_{vs} (in/in X 10^{-6}) (b)
Fine-grain	95	1,200,000	560	375	50,000	20 100	0.0101 0.0407	+ 59 +249	-1110 -4330	- 788 -3344
	85	1,200,000	1,690	1,120	50,000	20 100	0.0052 0.0250	+ 50 +204	- 543 -2502	- 386 -1996
Coarse-grain	75	1,200,000	15,000	10,000	50,000	20 100	0.0021 0.0100	+ 38 +126	- 179 - 768	- 109 - 568
	50	1,200,000	30,000	20,000	50,000	20 100	0.0017 0.0079	+ 34 +106	- 136 - 529	- 74 - 368

Notes: (a) Surface course = 4 in.
Base = 6 in.
Thawed subgrade = 4 in.
Frozen subgrade = 34 in.

(b) (i) Surface deflection (δ)
(ii) Horizontal strain bottom of surface course (ε_t)
(iii) Vertical strain top of base (ε_{vb})
(iv) Vertical strain top of subgrade (ε_{vs})

Table B.48 Spring Thaw Condition - Dual Tires - Single Axle - Thaw 4 in. into Subgrade - Pavement Structure 4/12/4/28(a) - Beneath Tire

Subgrade Type	Reduction in Subgrade Resilient Modulus	Resilient Modulus (psi)				Pavement Response Beneath Inside Tire of Duals				
		Surface Course	Base	Subgrade Thawed	Subgrade Frozen	Percent of Full Load	δ (in.) (b)	ε_t (in/in X 10^{-6}) (b)	ε_{vb} (in/in X 10^{-6}) (b)	ε_{vs} (in/in X 10^{-6}) (b)
Fine-grain	95	1,200,000	560	375	50,000	20 100	0.0145 0.0525	+ 66 +267	- 982 -3531	- 679 -2542
	85	1,200,000	1,690	1,120	50,000	20 100	0.0072 0.0316	+ 53 +224	- 504 -2126	- 324 -1518
Coarse-grain	75	1,200,000	15,000	10,000	50,000	20 100	0.0024 0.0114	+ 39 +132	- 177 - 749	- 76 - 423
	50	1,200,000	30,000	20,000	50,000	20 100	0.0018 0.0084	+ 35 +107	- 136 - 529	- 49 - 262

Notes: (a) Surface course = 4 in.
Base = 12 in.
Thawed subgrade = 4 in.
Frozen subgrade = 28 in.

(b) (i) Surface deflection (δ)
(ii) Horizontal strain bottom of surface course (ε_t)
(iii) Vertical strain top of base (ε_{vb})
(iv) Vertical strain top of subgrade (ε_{vs})

APPENDIX C

TEMPERATURE INPUT
DATA FOR TDHC ANALYSIS

Table C.1 Temperature Input Data of TDHC Analysis

Phase I

ME DATA
1.825,109.25,1.1,50
NODE DATA

Node	X(FT)	Y(FT)	Temp
1.000	0.000	0.000	44.1
2.000	1.000	0.000	44.1
3.000	0.000	0.167	44.1
4.000	1.000	0.167	44.1
5.000	0.000	0.333	44.1
6.000	1.000	0.333	44.1
7.000	0.000	0.500	44.1
8.000	1.000	0.500	44.1
9.000	0.000	0.667	44.1
10.000	1.000	0.667	44.1
11.000	0.000	0.833	44.1
12.000	1.000	0.833	44.1
13.000	0.000	1.000	44.1
14.000	1.000	1.000	44.1
15.000	0.000	1.167	44.1
16.000	1.000	1.167	44.1
17.000	0.000	1.333	44.1
18.000	1.000	1.333	44.1
19.000	0.000	1.500	44.1
20.000	1.000	1.500	44.1
21.000	0.000	1.667	44.1
22.000	1.000	1.667	44.1
23.000	0.000	1.833	44.1
24.000	1.000	1.833	44.1
25.000	0.000	2.000	44.1
26.000	1.000	2.000	44.1
27.000	0.000	2.333	44.1
28.000	1.000	2.333	44.1
29.000	0.000	2.667	44.1
30.000	1.000	2.667	44.1
31.000	0.000	3.000	44.1
32.000	1.000	3.000	44.1
33.000	0.000	3.333	44.1
34.000	1.000	3.333	44.1
35.000	0.000	3.667	44.1
36.000	1.000	3.667	44.1
37.000	0.000	4.000	44.1
38.000	1.000	4.000	44.1
39.000	0.000	4.333	44.1
40.000	1.000	4.333	44.1
41.000	0.000	4.667	44.1
42.000	1.000	4.667	44.1
43.000	0.000	5.000	44.1
44.000	1.000	5.000	44.1

Table C.1 Temperature Input Data of TDHC Analysis (Cont.)

Phase I (Cont.)

ME DATA

1.825,109.25,1.1,50

NODE DATA

Node	X(FT)	Y(FT)	Temp
45.000	0.000	5.333	44.1
46.000	1.000	5.333	44.1
47.000	0.000	5.667	44.1
48.000	1.000	5.667	44.1
49.000	0.000	6.000	44.1
50.000	1.000	6.000	44.1
51.000	0.000	6.500	44.1
52.000	1.000	6.500	44.1
53.000	0.000	7.000	44.1
54.000	1.000	7.000	44.1
55.000	0.000	7.500	44.1
56.000	1.000	7.500	44.1
57.000	0.000	8.000	44.1
58.000	1.000	8.000	44.1
59.000	0.000	8.500	44.1
60.000	1.000	8.500	44.1
61.000	0.000	9.000	44.1
62.000	1.000	9.000	44.1
63.000	0.000	9.500	44.1
64.000	1.000	9.500	44.1
65.000	0.000	10.000	44.1
66.000	1.000	10.000	44.1
67.000	0.000	15.000	44.1
68.000	1.000	15.000	44.1
69.000	0.000	20.000	44.1
70.000	1.000	20.000	44.1
71.000	0.000	25.000	44.1
72.000	1.000	25.000	44.1
73.000	0.000	30.000	44.1
74.000	1.000	30.000	44.1
75.000	0.000	35.000	44.1
76.000	1.000	35.000	44.1
77.000	0.000	40.000	44.1
78.000	1.000	40.000	44.1
79.000	0.000	45.000	44.1
80.000	1.000	45.000	44.1
81.000	0.000	50.000	45.1
82.000	1.000	50.000	45.1

FIXED NODE TEMPERATURES

2

81,45.1

82,45.1

HARMONIC NODE TEMPERATURES

2

1,44.1,24.6,18

2,44.1,24.6,18

END

Table C.1 Temperature Input Data of TDHC Analysis (Cont.)

Phase II

TIME DATA

1,18,.33,4

NODE DATA

Node	X(FT)	Y(FT)	Temp.
1.000	0.000	0.000	19.50
2.000	1.000	0.000	19.50
3.000	0.000	0.167	20.300
4.000	1.000	0.167	20.300
5.000	0.000	0.333	21.100
6.000	1.000	0.333	21.100
7.000	0.000	0.500	21.700
8.000	1.000	0.500	21.700
9.000	0.000	0.667	22.300
10.000	1.000	0.667	22.300
11.000	0.000	0.833	22.900
12.000	1.000	0.833	22.900
13.000	0.000	1.000	23.500
14.000	1.000	1.000	23.500
15.000	0.000	1.167	24.100
16.000	1.000	1.167	24.100
17.000	0.000	1.333	24.680
18.000	1.000	1.333	24.680
19.000	0.000	1.500	25.640
20.000	1.000	1.500	25.640
21.000	0.000	1.667	26.600
22.000	1.000	1.667	26.600
23.000	0.000	1.833	27.540
24.000	1.000	1.833	27.540
25.000	0.000	2.000	28.480
26.000	1.000	2.000	28.480
27.000	0.000	2.333	30.340
28.000	1.000	2.333	30.340
29.000	0.000	2.667	32.120
30.000	1.000	2.667	32.120
31.000	0.000	3.000	32.960
32.000	1.000	3.000	32.960
33.000	0.000	3.333	33.780
34.000	1.000	3.333	33.780
35.000	0.000	3.667	34.570
36.000	1.000	3.667	34.570
37.000	0.000	4.000	35.340
38.000	1.000	4.000	35.340
39.000	0.000	4.333	36.100
40.000	1.000	4.333	36.100
41.000	0.000	4.667	36.830
42.000	1.000	4.667	36.830
43.000	0.000	5.000	37.540
44.000	1.000	5.000	37.540

Table C.1 Temperature Input Data of TDHC Analysis (Cont.)

<u>Phase II (Cont.)</u>

Time Data

1,18,.33,4

NODE DATA

Node	X(FT)	Y(FT)	Temp.
45.000	0.000	5.333	38.230
46.000	1.000	5.333	38.230
47.000	0.000	5.667	38.900
48.000	1.000	5.667	38.900
49.000	0.000	6.000	39.550
50.000	1.000	6.000	39.550
51.000	0.000	6.500	40.490
52.000	1.000	6.500	40.490
53.000	0.000	7.000	41.380
54.000	1.000	7.000	41.380
55.000	0.000	7.500	42.230
56.000	1.000	7.500	42.230
57.000	0.000	8.000	43.030
58.000	1.000	8.000	43.030
59.000	0.000	8.500	43.780
60.000	1.000	8.500	43.780
61.000	0.000	9.000	44.480
62.000	1.000	9.000	44.480
63.000	0.000	9.500	45.140
64.000	1.000	9.500	45.140
65.000	0.000	10.000	45.750
66.000	1.000	10.000	45.750
67.000	0.000	15.000	49.600
68.000	1.000	15.000	49.600
69.000	0.000	20.000	50.470
70.000	1.000	20.000	50.470
71.000	0.000	25.000	49.910
72.000	1.000	25.000	49.910
73.000	0.000	30.000	48.940
74.000	1.000	30.000	48.940
75.000	0.000	35.000	47.950
76.000	1.000	35.000	47.950
77.000	0.000	40.000	47.010
78.000	1.000	40.000	47.010
79.000	0.000	45.000	46.060
80.000	1.000	45.000	46.060
81.000	0.000	50.000	45.1
82.000	1.000	50.000	45.1

FIXED NODE TEMPERATURES

2

81,45.1

82,45.1

HARMONIC NODE TEMPERATURES

2

1,44.1,24.6,18

2,44.1,24.6,18

END

Table C.1 Temperature Input Data of TDHC Analysis (Cont.)

Phase III

TIME DATA

1,62,.25,2

NODE DATA

Node	X(FT)	Y(FT)	Temp.
1.000	0.000	0.000	26.23
2.000	1.000	0.000	26.23
3.000	0.000	0.167	26.420
4.000	1.000	0.167	26.420
5.000	0.000	0.333	26.640
6.000	1.000	0.333	26.640
7.000	0.000	0.500	26.800
8.000	1.000	0.500	26.800
9.000	0.000	0.667	26.970
10.000	1.000	0.667	26.970
11.000	0.000	0.833	27.150
12.000	1.000	0.833	27.150
13.000	0.000	1.000	27.330
14.000	1.000	1.000	27.330
15.000	0.000	1.167	27.520
16.000	1.000	1.167	27.520
17.000	0.000	1.333	27.710
18.000	1.000	1.333	27.710
19.000	0.000	1.500	28.030
20.000	1.000	1.500	28.030
21.000	0.000	1.667	28.360
22.000	1.000	1.667	28.360
23.000	0.000	1.833	28.690
24.000	1.000	1.833	28.690
25.000	0.000	2.000	29.040
26.000	1.000	2.000	29.040
27.000	0.000	2.333	29.740
28.000	1.000	2.333	29.740
29.000	0.000	2.667	30.460
30.000	1.000	2.667	30.460
31.000	0.000	3.000	31.180
32.000	1.000	3.000	31.180
33.000	0.000	3.333	31.920
34.000	1.000	3.333	31.920
35.000	0.000	3.667	32.520
36.000	1.000	3.667	32.520
37.000	0.000	4.000	33.110
38.000	1.000	4.000	33.110
39.000	0.000	4.333	33.700
40.000	1.000	4.333	33.700
41.000	0.000	4.667	34.290
42.000	1.000	4.667	34.290
43.000	0.000	5.000	34.870
44.000	1.000	5.000	34.870

Table C.1 Temperature Input Data of TDHC Analysis (Cont.)

Phase III (Cont.)
TIME DATA
1,62,
NODE DATA

Node	X(FT)	Y(FT)	Temp.
45.000	0.000	5.333	35.440
46.000	1.000	5.333	35.440
47.000	0.000	5.667	36.010
48.000	1.000	5.667	36.010
49.000	0.000	6.000	36.570
50.000	1.000	6.000	36.570
51.000	0.000	6.500	37.390
52.000	1.000	6.500	37.390
53.000	0.000	7.000	38.180
54.000	1.000	7.000	38.180
55.000	0.000	7.500	38.950
56.000	1.000	7.500	38.950
57.000	0.000	8.000	39.690
58.000	1.000	8.000	39.690
59.000	0.000	8.500	40.410
60.000	1.000	8.500	40.410
61.000	0.000	9.000	41.100
62.000	1.000	9.000	41.100
63.000	0.000	9.500	41.760
64.000	1.000	9.500	41.760
65.000	0.000	10.000	42.380
66.000	1.000	10.000	42.380
67.000	0.000	15.000	47.020
68.000	1.000	15.000	47.020
69.000	0.000	20.000	49.080
70.000	1.000	20.000	49.080
71.000	0.000	25.000	49.390
72.000	1.000	25.000	49.390
73.000	0.000	30.000	48.840
74.000	1.000	30.000	48.840
75.000	0.000	35.000	47.960
76.000	1.000	35.000	47.960
77.000	0.000	40.000	47.010
78.000	1.000	40.000	47.010
79.000	0.000	45.000	46.060

Table C.1 Temperature Input Data of TDHC Analysis (Cont.)

Phase III (Cont.)

Time Data

1,62,.25,2

NODE DATA

Node	X(FT)	Y(FT)	Temp.
80.000	1.000	45.000	46.060
81.000	0.000	50.000	45.10
82.000	1.000	50.000	45.10

FIXED NODE TEMPERATURES
2
81,45.1
82,45.1
CONVECTION SURFACES WITH HARMONIC TEMPERATURES
1
1,2,3.2,44.1,24.6,18
HEAT FLUX AT SURFACES
1
1,2,9.0
END

Table C.1 Temperature Input Data of TDHC Analysis (Cont.)

Phase IV

TIME DATA
1,90,.31,2
NODE DATA

Node	X(FT)	Y(FT)	Temp.
1.000	0.000	0.000	37.93
2.000	1.000	0.000	37.93
3.000	0.000	0.167	37.340
4.000	1.000	0.167	37.340
5.000	0.000	0.333	36.740
6.000	1.000	0.333	36.740
7.000	0.000	0.500	36.380
8.000	1.000	0.500	36.380
9.000	0.000	0.667	36.020
10.000	1.000	0.667	36.020
11.000	0.000	0.833	35.680
12.000	1.000	0.833	35.680
13.000	0.000	1.000	35.340
14.000	1.000	1.000	35.340
15.000	0.000	1.167	35.000
16.000	1.000	1.167	35.000
17.000	0.000	1.333	34.670
18.000	1.000	1.333	34.670
19.000	0.000	1.500	33.980
20.000	1.000	1.500	33.980
21.000	0.000	1.667	33.280
22.000	1.000	1.667	33.280
23.000	0.000	1.833	32.600
24.000	1.000	1.833	32.600
25.000	0.000	2.000	32.040
26.000	1.000	2.000	32.040
27.000	0.000	2.333	31.970
28.000	1.000	2.333	31.970
29.000	0.000	2.667	31.940
30.000	1.000	2.667	31.940
31.000	0.000	3.000	31.920
32.000	1.000	3.000	31.920
33.000	0.000	3.333	32.180
34.000	1.000	3.333	32.180
35.000	0.000	3.667	32.640
36.000	1.000	3.667	32.640
37.000	0.000	4.000	33.100
38.000	1.000	4.000	33.100
39.000	0.000	4.333	33.560
40.000	1.000	4.333	33.560
41.000	0.000	4.667	34.020
42.000	1.000	4.667	34.020
43.000	0.000	5.000	34.480
44.000	1.000	5.000	34.480

Table C.1 Temperature Input Data of TDHC Analysis (Cont.)

Phase IV (Cont.)
TIME DATA
1,90,.31,2
NODE DATA

Node	X(FT)	Y(FT)	Temp.
45.000	0.000	5.333	34.950
46.000	1.000	5.333	34.950
47.000	0.000	5.667	35.410
48.000	1.000	5.667	35.410
49.000	0.000	6.000	35.860
50.000	1.000	6.000	35.860
51.000	0.000	6.500	36.540
52.000	1.000	6.500	36.540
53.000	0.000	7.000	37.220
54.000	1.000	7.000	37.220
55.000	0.000	7.500	37.880
56.000	1.000	7.500	37.880
57.000	0.000	8.000	38.520
58.000	1.000	8.000	38.520
59.000	0.000	8.500	39.160
60.000	1.000	8.500	39.160
61.000	0.000	9.000	39.770
62.000	1.000	9.000	39.770
63.000	0.000	9.500	40.370
64.000	1.000	9.500	40.370
65.000	0.000	10.000	40.960
66.000	1.000	10.000	40.960
67.000	0.000	15.000	45.600
68.000	1.000	15.000	45.600
69.000	0.000	20.000	48.090
70.000	1.000	20.000	48.090
71.000	0.000	25.000	48.880
72.000	1.000	25.000	48.880
73.000	0.000	30.000	48.640
74.000	1.000	30.000	48.640
75.000	0.000	35.000	47.910
76.000	1.000	35.000	47.910
77.000	0.000	40.000	47.000
78.000	1.000	40.000	47.000
79.000	0.000	45.000	46.060

Table C.1 Temperature Input Data of TDHC Analysis (Cont.)

Phase IV (Cont.)

```
TIME DATA
1,90,.31,2
NODE DATA
    Node      X(FT)     Y(FT)    Temp.
  80.000    1.000   45.000   46.060
  81.000    0.000   50.000   45.10
  82.000    1.000   50.000   45.10
  FIXED NODE TEMPERATURES
  2
  81,45.1
  82,45.1
  CONVECTION SURFACES WITH HARMONIC TEMPERATURES
  1
  1,2,3.2,44.1,24.6,18
  HEAT FLUX AT SURFACES
  1
  1,2,22.5
  END
```

APPENDIX D

PLOTS OF MODELS
FOR PREDICTING THAWING INDEX OR THAWING DURATION
FROM FREEZING INDEX

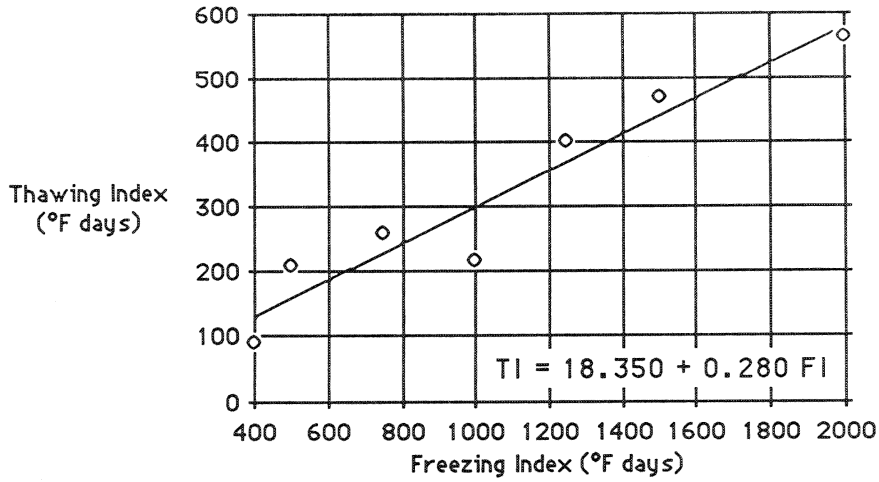

Figure D.1 Thawing Index (based on $29^{0}F$) versus Freezing Index for Section 1.

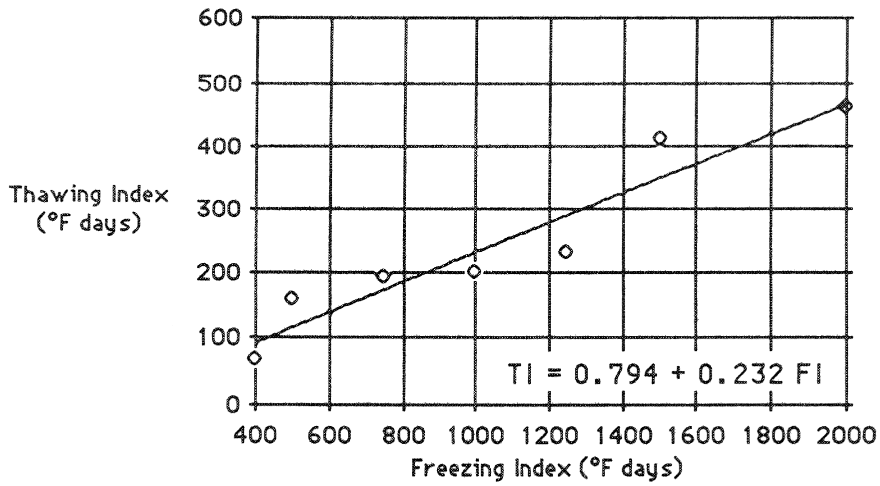

Figure D.2 Thawing Index (based on 29^{o}F) versus Freezing Index for Section 2.

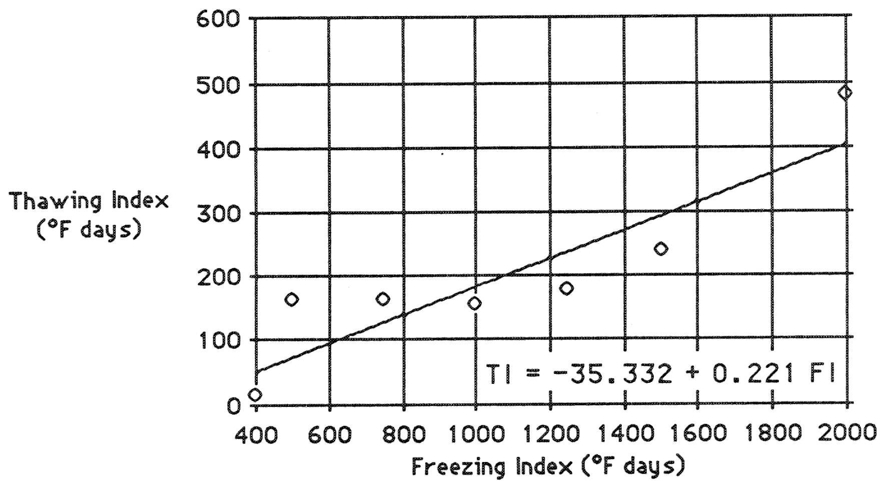

Figure D.3 Thawing Index (based on 29^{o}F) versus Freezing Index for Section 3.

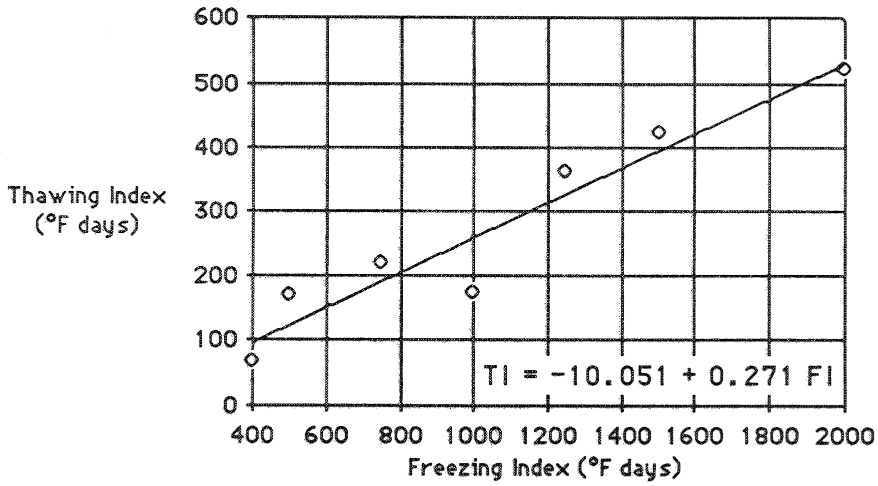

Figure D.4 Thawing Index (based on 30^{0}F) versus Freezing Index for Section 1.

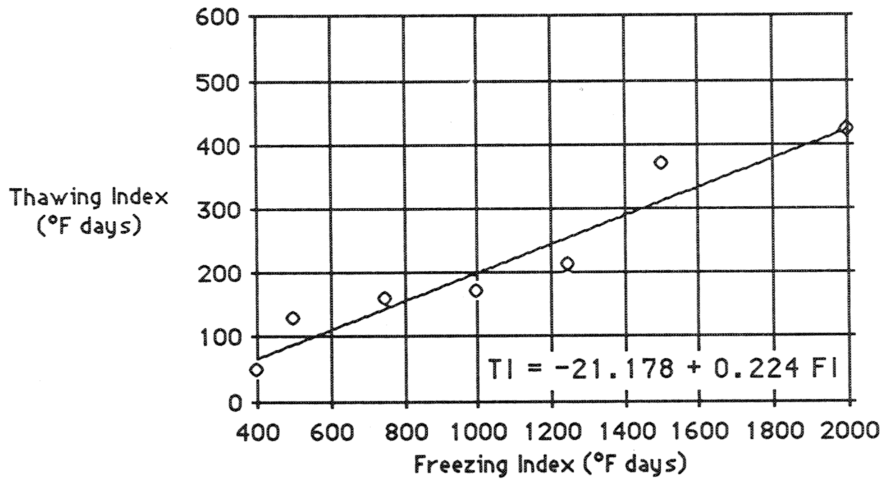

Figure D.5 Thawing Index (based on 30^{0}F) versus Freezing Index for Section 2.

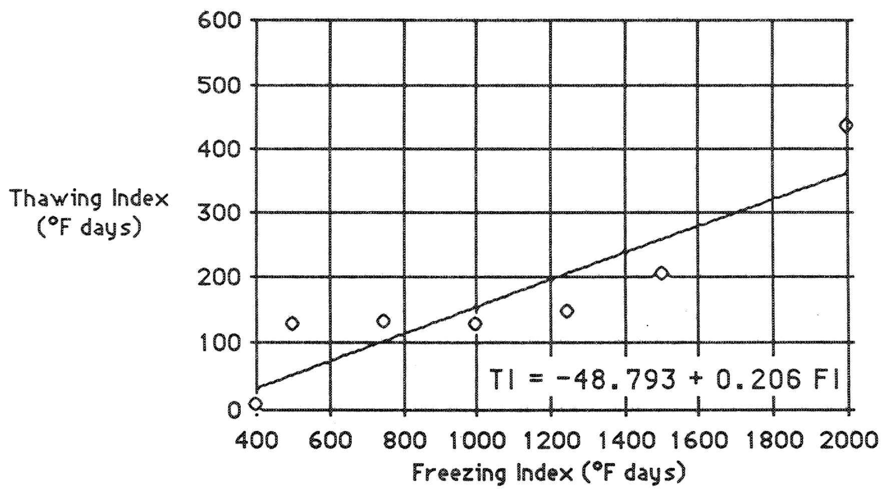

Figure D.6 Thawing Index (based on $30^{0}F$) versus Freezing Index for Section 3.

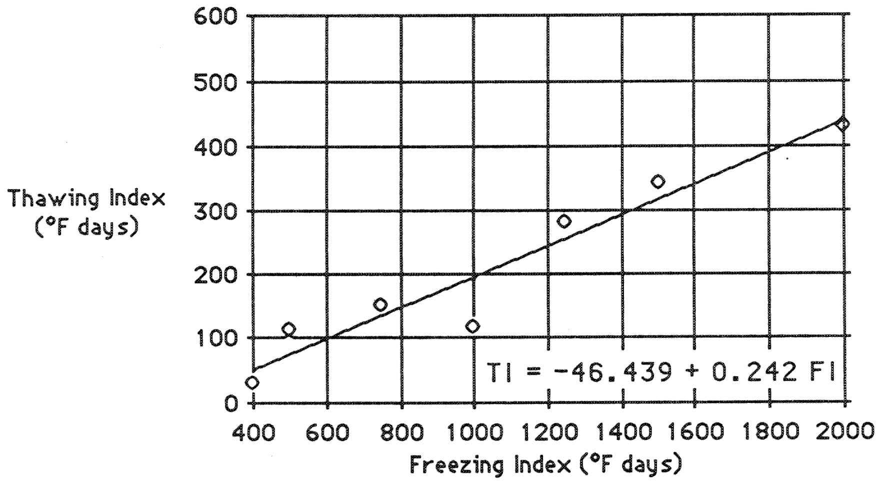

Figure D.7 Thawing Index (based on 32^{O}F) versus Freezing Index for Section 1.

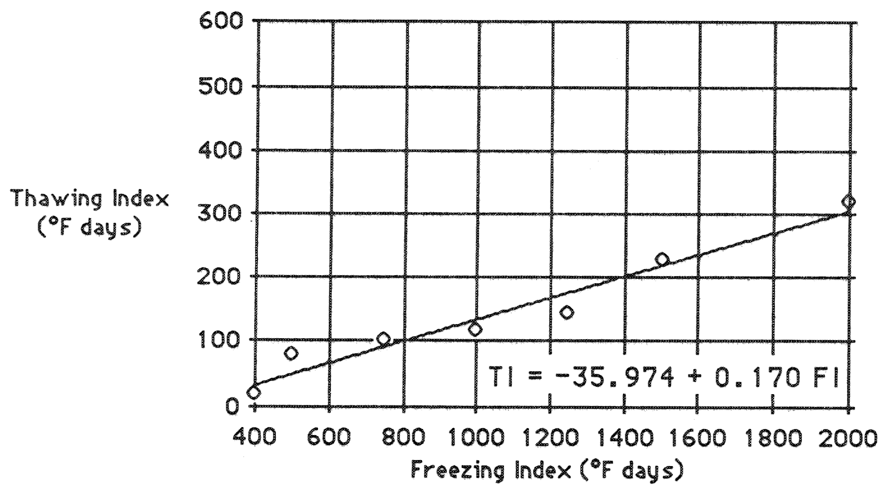

Figure D.8 Thawing Index (based on 32^{o}F) versus Freezing Index for Section 2.

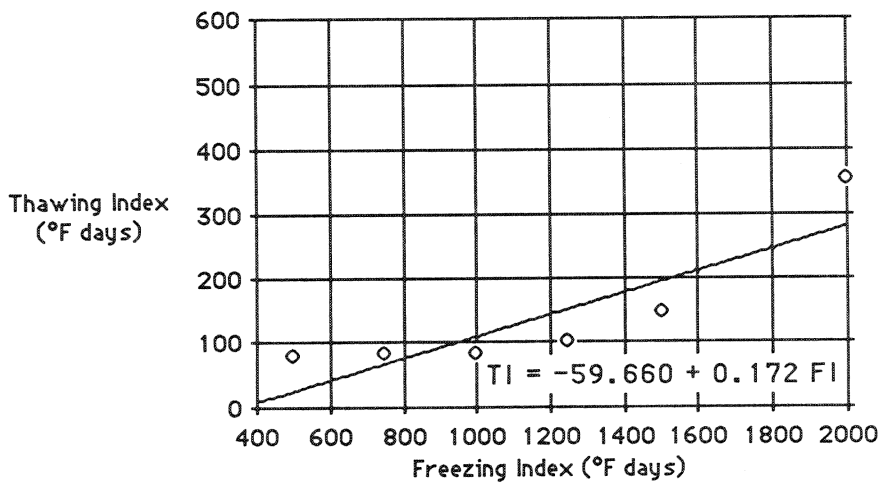

Figure D.9 Thawing Index (based on $32^{0}F$) versus Freezing Index for Section 3.

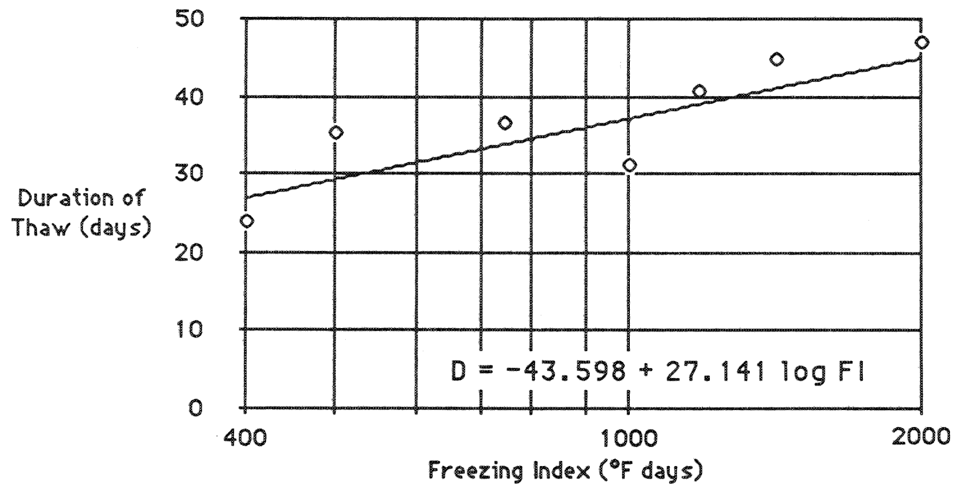

Figure D.10 Duration of Thaw (based on 29^{0}F) versus log Freezing Index for Section 1.

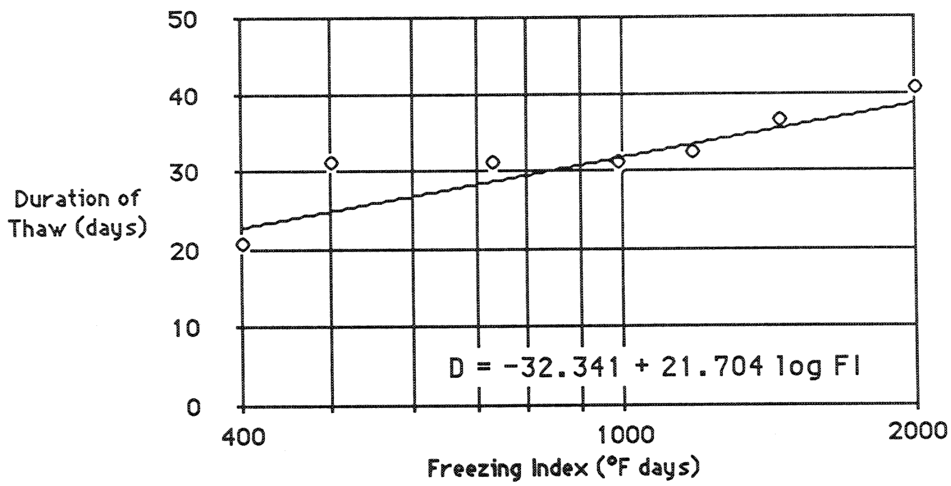

Figure D.11 Duration of Thaw (based on 29^{o}F) versus log Freezing Index for Section 2.

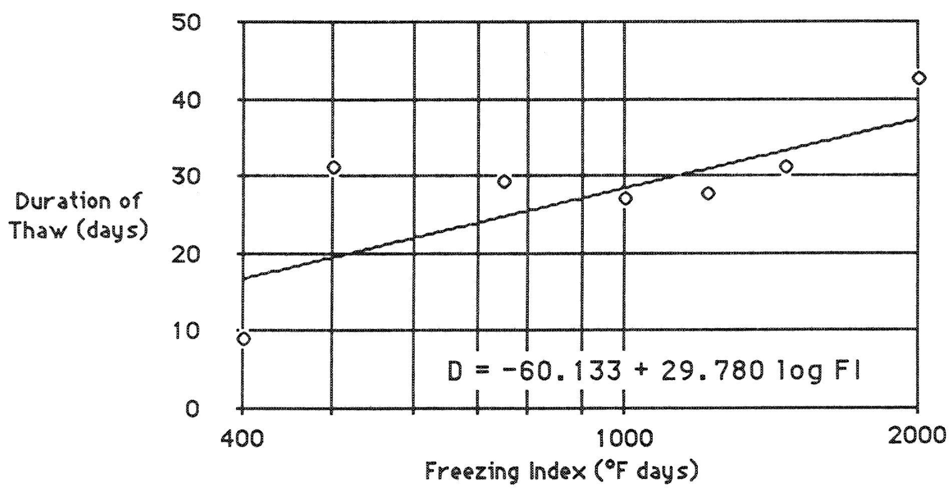

Figure D.12 Duration of Thaw (based on 29^{o}F) versus log Freezing Index for Section 3.

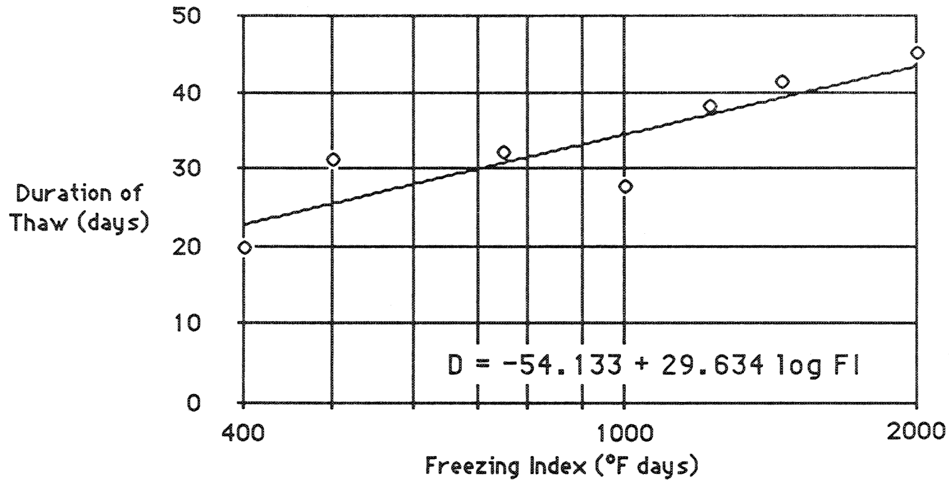

Figure D.13 Duration of Thaw (baed on 30^{O}F) versus log Freezing Index for Section 1.

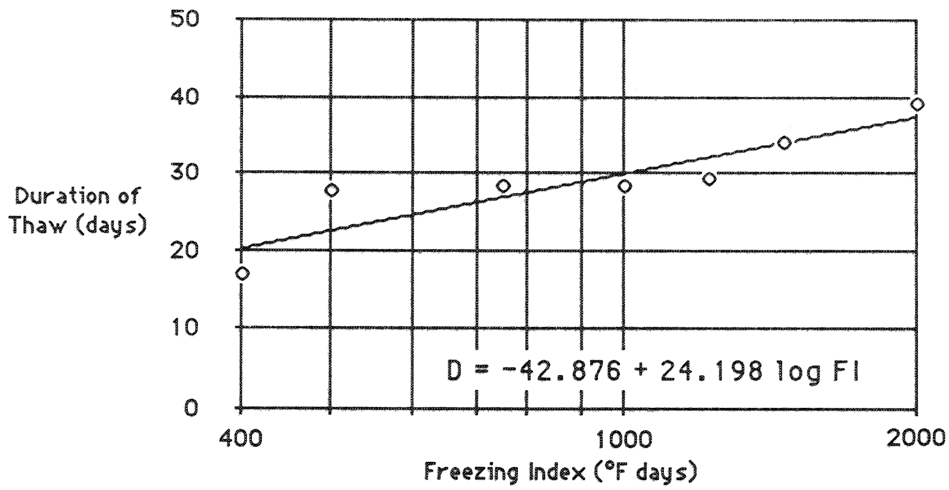

Figure D.14 Duration of Thaw (based on 30^{O}F) versus log Freezing Index for Section 2.

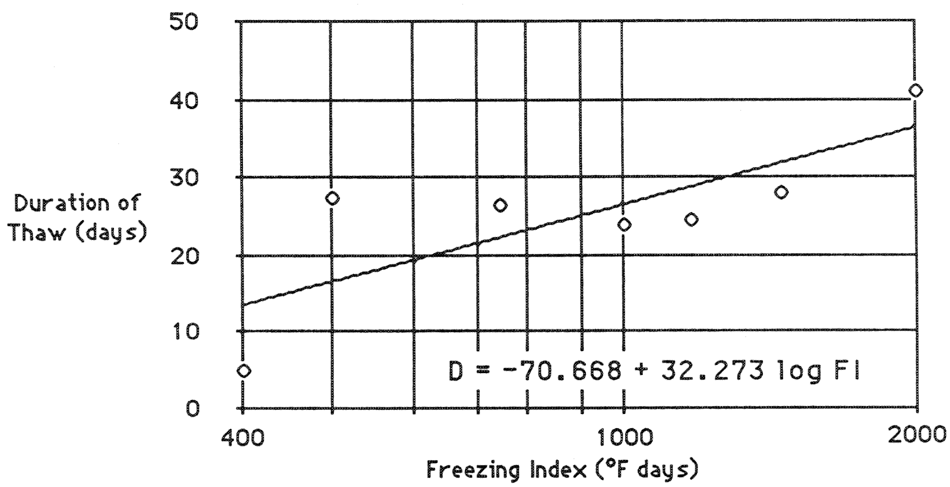

Figure D.15 Duration of Thaw (based on 30^{o}F) versus log Freezing Index for Section 3.

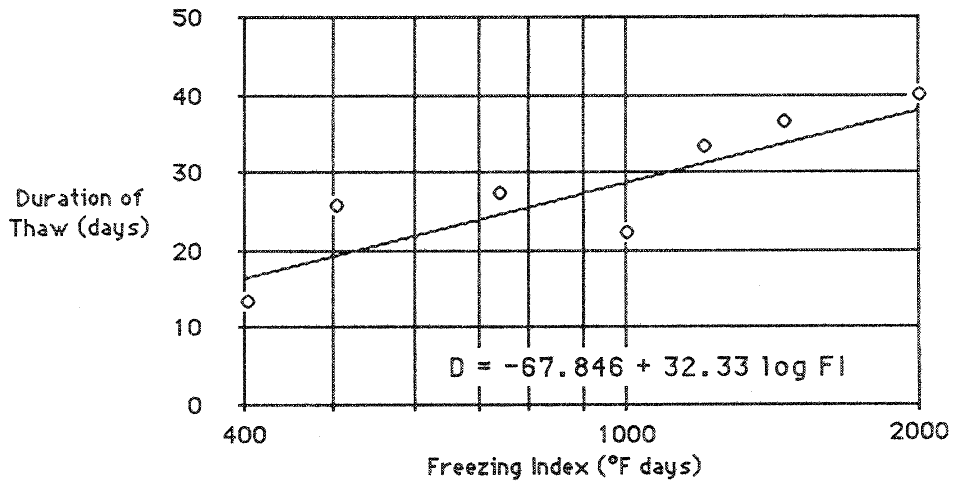

Figure D.16 Duration of Thaw (based on $32^{o}F$) versus log Freezing Index for Section 1.

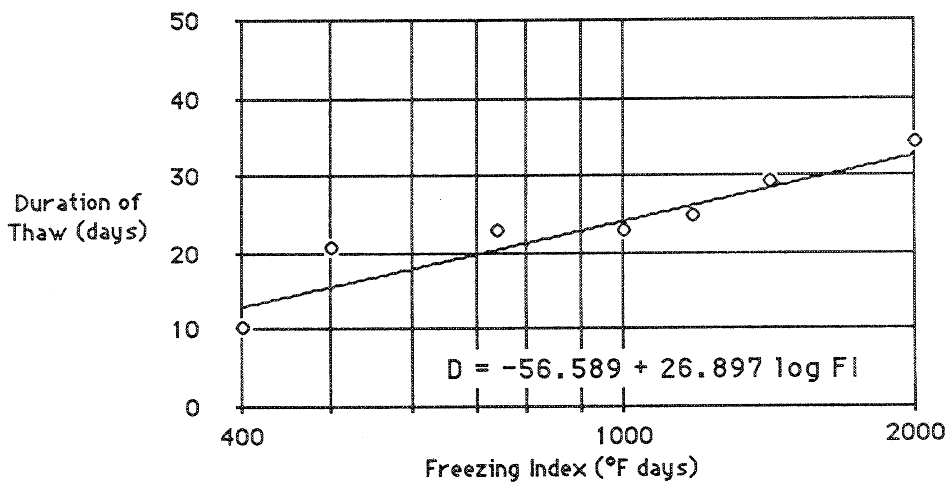

Figure D.17 Duration of Thaw (based on 32°F) versus log Freezing Index for Section 2.

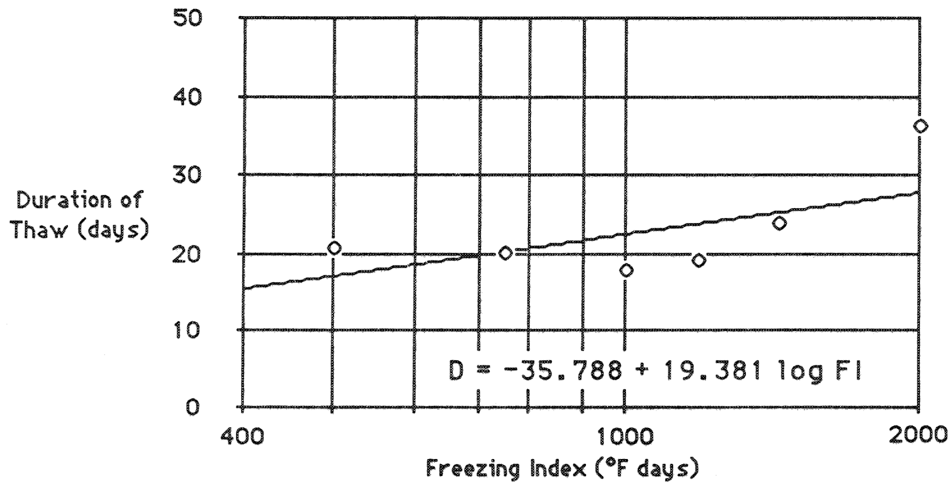

Figure D.18 Duration of Thaw (based on 32^{O}F) versus log Freezing Index for Section 3.

APPENDIX E

INTERVIEW FORM

INTERVIEW FORM
for
FHWA Project on "Guidelines for Spring Highway Use Restrictions"

* *

Agency: ______________________ Date: ____________

Completed By: ______________________ Phone: (   ) ____________

Address: ______________________________

____

____

* *

INSTRUCTIONS: Please complete the following form prior to the interview. If your agency has supporting data, reports or legislation, please enclose them with your response.

* *

I. DEVELOPMENT OF LOAD RESTRICTIONS

I.1 What types of pavement failure are associated with spring thaw?

I.2 How extensive are these problems? (e.g., agency-wide)

I.3 a) When were the first Spring Load Restrictions initiated by your agency?

b) How are specific locations for load restrictions determined?

c) Would guidelines addressing where to impose load restrictions be of use to your agency?

I.4 a) What studies, if any, were conducted or decision processes used prior to instituting the restriction measures?

b) If studies were conducted, they were performed by:

____ Federal Agency ____ State Agency

____ Local ____ Other (________________)

c) Have any follow-up studies been carried out to access the effectiveness of the Spring Load Restriction program?

II. CRITERIA FOR IMPOSING LOAD RESTRICTIONS (Information requested applies to those areas of your jurisdiction for which load restrictions apply):

II.1 To what classes of highways are Load Restriction applied to?

a) Functional Class(es)________________________________

b) ADT & % Trucks ________________________________

c) Soil Type(s)________________________________

d) Surfacing Type(s)________________________________

e) Typical Cross-Section(s)________________________________
(sketch)

f) Othe ________________________________

II.2 Environmental Factors:

a) Annual Precipitation

i) Rainfall Amount (in.)______________

ii) Snow Amount (in.)_ ______________

iii) Typical start date of freezing weather________________

iv) Typical start date of thawing weather________________

b) Freezing Index (°F-day)________________________

c) Depth of frost penetration (ft.)____________________

d) Basis for frost determination (if instruments were used, please state what instruments)________________________________

e) Source of weather data (if used)________________________________

II.3 Design Information:

a) Is frost protection used in thickness design in all susceptible areas? ____ Yes ____ No

If yes, i) ____ Full Protection (Total Pavement = Frost Depth)

ii) ____ More than 50% but less than Full Protection

iii) ____ Less than 50%

If no, are load restrictions used in lieu of design for full frost protection? ________________________________

b) What thickness design method is used?

i) ____ Standard Section

ii) ____ Hveem Method

iii) ____ AASHTO

iv) ____ Other ________________________________

c) Average age of pavements which receive load restrictions__________

d) Drainage conditions in pavements with load restrictions

i) ____ Good

ii) ____ Fair

iii) ____ Poor

e) Source of Water ________________________________

III. ENFORCEMENT OF LIMITS

III.1 Criteria for Enforcement:

a) Weight limit on trucks (normal or other than spring thaw):

i) Gross weight limit ____________________

ii) Single axle weight limit ____________________

iii) Tandem axle weight limit ____________________

b) Weight limit on trucks (spring thaw):

i) Gross weight limit ____________________

ii) Single axle weight limit ____________________

iii) Tandem axle weight limit ____________________

c) How are the weight limits set?

____ Local experience

____ Past studies (please reference specific study)

____ General (state or national) guidelines

____ Bridge Formula

d) Enforcement Period

i) Basis for initiation of load restriction

ii) Basis for removing load restriction

iii) Do you use deflection measuring equipment to initiate or remove load restrictions?

____ Yes ____ No

If yes, what type of deflection equipment is used?

____

III.2 ENFORCEMENT:

a) What agency is responsible for enforcement of Spring Load Restrictions?

Name ____

Address ____

Phone No. ____

b) How are Load Restrictions enforced?

____ Fixed scale installations

____ Portable scale

____ Other

c) How are truck operators notified?

d) Are there exceptions to overweight trucks (e.g., school buses, movement of vital commodities, etc.) ___ Yes ___ No

e) Which agency issues overweight permits* ______________________

Cost______________

f) What percent increase of personnel (if any) is required for the enforcement effort?

g) What level of training is required of enforcement personnel?

h) What enforcement methods are used?

____ Stop all trucks ____ Other (please describe)

____ Selective sample

---

* Obtain copies of forms

i) What is the total annual additional cost of spring load enforcement?

Significant: _____ Yes _____ No

j) How are fines levied on overweight trucks?

____ Cost/1,000 lbs.

____ Other

III.3 Has any cost-benefit analysis of weight limit enforcement been carried out on any facility? ____ Yes ____ No

(If yes, please provide reference or relavent information)

IV. LEGAL ASPECTS

IV.1 Are there existing state or local regulations which address load restrictions? ____ Yes ____ No

(If yes, please provide a copy)

IV.2 What problems (if any) are associated with the enforcement of load restrictions?

IV.3 Have there been any legal problems with load restrictions (e.g. court cases, etc.)?

APPENDIX F

CALCULATION OF THE THAWING INDEX BASED ON A 29°F DATUM

APPENDIX F

CALCULATION OF THE THAWING INDEX BASED ON A 29°F DATUM

The surface thawing index for this pavement problem is a measure of the magnitude and duration of the temperature differential when thawing begins. It is measured in degree-days. The thawing index can be evaluated using the following equation:

$$TI_{29} = \Sigma\,(\bar{T} - 29°F)$$

where:

$\bar{T} = \frac{1}{2}(T_H + T_L)$ in °F,
T_H = maximum daily temperature (°F), and
T_L = minimum daily temperature (°F).

Estimate the thawing index given the temperature data shown in Figure F-1.

Steps

1. The values in columns 2 and 3 are obtained from reported local daily high-low temperatures <u>or</u> from a high-low thermometer located near the road section to be restricted.

2. The average daily temperature is equal to

   $\frac{1}{2}$ (column 2 + column 3)

   For 3/11:

   $\bar{T} = \frac{1}{2}\,(33 + 27) = \frac{1}{2}\,(60) = \underline{30\ °F}$

3. The thawing degree-days per day is equal to

   Daily $TI_{29} = \bar{T}$ (from column 4) - 29°F

Col. 1 Day (date)	Col. 2 Measured Daily Temperature (° F) High (T_H)	Col. 3 Low (T_L)	Col. 4 Average Daily Temperature, (°F) (T)	Col. 5 Daily Thawing Index based on 29° F datum (° F - days)	Col. 6 Accumulated Thawing Index based on 29° F datum (°F days)
3/ 1	30	20	25	-4	--
3/ 2	28	17	22	-7	--
3/ 3	31	23	27	-2	--
3/ 4	27	19	23	-6	--
3/ 5	33	25	29	0	--
3/ 6	34	24	29	0	--
3/ 7	36	28	32	3	3
3/ 8	35	28	32	3	6
3/ 9	31	25	28	-1	5
3/10	27	21	24	-5	0
3/11	33	27	30	1	1
3/12	37	27	32	3	4
3/13	39	30	34	5	9
3/14	32	26	30	1	10
3/15	41	29	35	6	16
3/16	40	30	35	6	22
3/17	40	32	36	7	29
3/18	43	33	38	9	38
3/19	40	30	35	6	44
3/20	36	28	32	3	47

Note: The values in columns 2 and 3 are obtained from reported local daily high-low temperatures or from a high-low recording thermometer located near the road section to be restricted.

Figure F-1. Form for Calculating Thawing Index.

For 3/11:

$$\text{Daily TI}_{29} = (30 - 29) = \underline{1°\text{F-day}}$$

4. The accumulated degree days is equal to the sum of the daily thawing indexes from the start of thawing up to the day of interest. The work performed in this study suggests that for thawing periods starting in late February to April, thawing below an asphalt or bituminous pavement begins when air temperatures go above 29°F. Therefore, the thawing period will start when values of the average daily temperature (column 4) go above 29°F for several days. For this example, the thawing period begins on 3/7. From this date, the calculation of thawing index begins.

$$TI_{29} = \Sigma \text{(column 5 after the start of thawing)}$$

On 3/11:

$$\begin{aligned} TI_{29} &= 3 \text{ (from 3/7)} + 3 \text{ (from 3/8)} - 1 \text{ (from 3/9)} \\ &\quad - 5 \text{ (from 3/10)} + 1 \text{ (from 3/11)} \\ &= \underline{1°\text{F-day}} \end{aligned}$$

APPENDIX G

EXAMPLE OF DATA COLLECTION AND ESTIMATION OF START AND DURATION FOR IMPOSING LOAD RESTRICTIONS

APPENDIX G

EXAMPLE OF DATA COLLECTION AND ESTIMATION OF START AND DURATION FOR IMPOSING LOAD RESTRICTIONS

Location: Coldspot, U.S.A.
Pavement section typically restricted during spring thawing
2½ inches asphalt
6-8 inches base
Silty subgrade

High and low daily temperatures are collected through freezing and thawing period to calculate freezing index, based on 32°F, and thawing index based on 29°F.

Calculating the Freezing Index

The freezing index is a measure of the magnitude and duration of the temperature differential during the freezing period. The freezing index is calculated using the following equation:

$$FI = \Sigma (32 - \bar{T})$$

where:

$\bar{T} = \frac{1}{2} (T_H + T_L)$ in °F,
T_H = maximum daily temperature (°F), and
T_L = minimum daily temperature (°F).

The following temperature data was collected for Coldspot to identify the freezing period and the freezing index (see Figure G-1).

Steps:

1. When $\bar{T}$ becomes less than or equal to 32°F for several days, the freezing season begins. The freezing season for this year begins on November 7.

2. The average daily temperature is equal to

   $\bar{T} = \frac{1}{2}$ (column 2 + column 3)

Col. 1	Col. 2	Col. 3	Col. 4	Col. 5	Col. 6
	Measured Daily Temperature (° F)		Average Daily Temperature, (°F)	Daily Freezing Index based on 32° F datum (° F - days)	Accumulated Freezing Index based on 32° F datum (°F days)
Day (date)	High (T_H)	Low (T_L)	(T)		
10/15	45	28	36	--	--
10/16	50	25	38	--	--
10/17	67	41	54	--	--
10/18	43	31	37	--	--
10/19	38	28	33	--	--
10/20	41	30	36	--	--
10/21	37	31	34	--	--
10/22	38	29	34	--	--
10/23	46	31	38	--	--
10/24	48	35	42	--	--
10/25	36	29	32	--	--
10/26	32	27	30	2	2
10/27	40	27	34	-2	0
10/28	40	20	30	2	2
10/29	41	23	32	0	2
10/30	53	29	41	-9	--
10/31	45	34	40	--	--
11/ 1	35	26	30	2	2
11/ 2	40	26	33	-1	1
11/ 3	53	36	44	-12	--
11/ 4	44	29	36	--	--
11/ 5	40	24	32	--	--
11/ 6	52	34	43	--	--
11/ 7	35	23	29	3	3
11/ 8	36	12	24	8	11
11/ 9	38	24	31	1	12
11/10	28	21	24	8	20
11/11	33	16	24	8	28
11/12	33	16	24	8	36
11/13	35	30	32	0	36
11/14	32	26	29	3	39
11/15	30	11	20	12	51
11/16	36	6	21	11	62
11/17	32	23	28	4	66

Note: The values in columns 2 and 3 are obtained from reported local daily high-low temperatures or from a high-low recording thermometer located near the road section to be restricted.

Figure G-1. Form for Calculating Freezing Index.

Col. 1 Day (date)	Col. 2 Measured Daily Temperature (° F) High (T_H)	Col. 3 Low (T_L)	Col. 4 Average Daily Temperature, (°F) (T)	Col. 5 Daily Freezing Index based on 32° F datum (° F - days)	Col. 6 Accumulated Freezing Index based on 32° F datum (°F days)
11/18	32	22	27	5	71
11/19	48	22	35	-3	68
11/20	43	28	36	-4	64
11/21	36	26	31	1	65
11/22	53	28	40	-8	57
11/23	39	27	33	-1	56
11/24	29	15	22	10	66
11/25	34	12	23	9	75
11/26	37	12	24	8	83
11/27	33	20	27	5	88
11/28	32	26	29	3	91
11/29	33	24	28	4	95
11/30	36	24	30	2	97
12/ 1	30	17	24	8	105
12/ 2	20	11	16	16	121
12/ 3	23	11	17	15	136
12/ 4	27	21	24	8	144
12/ 5	33	25	29	3	147
12/ 6	37	31	34	-2	145
12/ 7	37	33	35	-3	142
12/ 8	34	20	27	5	147
12/ 9	23	15	19	13	160
12/10	15	1	8	24	184
12/11	11	-5	3	29	213
12/12	29	6	18	14	227
12/13	18	8	13	19	246
12/14	14	5	10	22	268
12/15	20	9	14	18	286
12/16	21	-4	8	24	310
12/17	34	3	18	14	324
12/18	18	-6	6	26	350
12/19	5	-6	0	32	382
12/20	9	-8	0	32	414
12/21	13	-6	4	28	442

Note: The values in columns 2 and 3 are obtained from reported local daily high-low temperatures or from a high-low recording thermometer located near the road section to be restricted.

Figure G-1 (cont.). Form for Calculating Freezing Index.

Col. 1 Day (date)	Col. 2 Measured Daily Temperature (° F) High (T_H)	Col. 3 Low (T_L)	Col. 4 Average Daily Temperature, (°F) (T)	Col. 5 Daily Freezing Index based on 32° F datum (° F - days)	Col. 6 Accumulated Freezing Index based on 32° F datum (°F days)
12/22	18	8	13	19	461
12/23	22	12	17	15	476
12/24	12	-4	4	28	504
12/25	13	-2	6	26	530
12/26	13	4	8	24	554
12/27	23	-5	9	23	577
12/28	35	22	28	4	581
12/29	31	17	24	8	589
12/30	22	15	18	14	603
12/31	27	18	22	10	613
1/ 1	26	5	16	16	629
1/ 2	6	-12	-3	35	664
1/ 3	-3	-12	-8	40	704
1/ 4	5	-8	-2	34	738
1/ 5	13	-6	4	28	766
1/ 6	21	-1	10	22	788
1/ 7	8	-2	3	29	817
1/ 8	12	-5	4	28	845
1/ 9	10	0	5	27	872
1/10	9	-9	0	32	904
1/11	11	1	6	26	930
1/12	19	-2	8	24	954
1/13	24	5	14	18	972
1/14	27	17	22	10	982
1/15	23	13	18	14	996
1/16	19	6	12	20	1016
1/17	28	4	16	16	1032
1/18	44	19	32	0	1032
1/19	46	11	28	4	1036
1/20	30	12	21	11	1047
1/21	44	12	28	4	1051
1/22	31	20	26	6	1057
1/23	39	15	27	5	1062
1/24	39	9	24	8	1070

Note: The values in columns 2 and 3 are obtained from reported local daily high-low temperatures or from a high-low recording thermometer located near the road section to be restricted.

Figure G-1 (cont.). Form for Calculating Freezing Index.

Col. 1 Day (date)	Col. 2 Measured Daily Temperature (° F) High (T_H)	Col. 3 Low (T_L)	Col. 4 Average Daily Temperature, (°F) (T)	Col. 5 Daily Freezing Index based on 32° F datum (° F - days)	Col. 6 Accumulated Freezing Index based on 32° F datum (°F days)
1/25	45	25	35	-3	1067
1/26	34	12	23	9	1076
1/27	20	4	12	20	1096
1/28	18	2	10	22	1118
1/29	15	7	11	21	1139
1/30	31	-11	10	22	1161
1/31	27	8	18	14	1175
2/ 1	26	6	16	16	1191
2/ 2	7	-16	-5	37	1228
2/ 3	5	-21	-8	40	1268
2/ 4	6	-12	-3	35	1303
2/ 5	24	-2	11	21	1324
2/ 6	20	8	14	18	1342
2/ 7	23	7	15	17	1359
2/ 8	16	-2	7	25	1384
2/ 9	17	-9	4	28	1412
2/10	8	-11	-2	34	1446
2/11	-1	-20	-10	42	1488
2/12	11	-32	-10	42	1530
2/13	21	-13	4	28	1558
2/14	24	-5	14	18	1576
2/15	46	26	36	-4	1572
2/16	53	34	44	-12	1560
2/17	61	37	49	-17	1543
2/18	44	37	40	-8	1535
2/19	44	29	36	-4	1531
2/20	36	26	31	1	1532
2/21	44	21	32	0	1532
2/22	36	31	34	-2	1530
2/23	38	32	35	-3	1527
2/24	33	25	29	3	1530
2/25	27	20	24	8	1538
2/26	26	17	22	10	1548
2/27	36	18	27	5	1553

Note: The values in columns 2 and 3 are obtained from reported local daily high-low temperatures or from a high-low recording thermometer located near the road section to be restricted.

Figure G-1 (cont.). Form for Calculating Freezing Index.

Col. 1 Day (date)	Col. 2 Measured Daily Temperature (° F) High (T_H)	Col. 3 Low (T_L)	Col. 4 Average Daily Temperature, (°F) (T)	Col. 5 Daily Freezing Index based on 32° F datum (° F - days)	Col. 6 Accumulated Freezing Index based on 32° F datum (°F days)
2/28	31	26	28	4	1557
3/ 1	32	21	26	6	1563
3/ 2	21	11	16	16	1579
3/ 3	29	-5	12	20	1599
3/ 4	27	9	18	14	1613
3/ 5	24	3	14	18	1631
3/ 6	22	9	16	16	1647
3/ 7	35	14	24	8	1655
3/ 8	39	19	29	3	1658
3/ 9	39	17	28	4	1662
3/10	30	20	25	7	1669
3/11	38	18	28	4	1673
3/12	44	23	34	-2	1671
3/13	24	7	16	16	1687
3/14	48	5	26	6	1693
3/15	41	16	28	4	1697
3/16	34	5	20	12	1709
3/17	23	12	18	14	1723
3/18	20	13	16	16	1739
3/19	24	15	20	12	1751
3/20	30	23	26	6	1757

Note: The values in columns 2 and 3 are obtained from reported local daily high-low temperatures or from a high-low recording thermometer located near the road section to be restricted.

Figure G-1 (cont.). Form for Calculating Freezing Index.

For 11/25:

$\bar{T} = \frac{1}{2}(34 + 12) = \underline{23°F}$

3. The freezing degree-days per day (column 5) is equal to

Daily FI = $32 - \bar{T}$ (from column 4)

For 11/25:

Daily FI = $(32 - 23) = \underline{9°F\text{-days}}$

4. The freezing index is the accumulation of daily freezing degree days from the start of freezing

FI = $\Sigma(32 - \bar{T})$ from the start of freezing

For 11/25:

$$FI = (3 + 8 + 1 + 8 + 8 + 8 + 0 + 3 + 12 + 11 + 4 + 5 - 3 - 4 + 1 - 8 - 1 + 10 + 9)$$
$$= \underline{75°F\text{-days}}$$

5. The freezing season ends for pavements when the average daily air temperatures (column 4) in spring go above 29°F for several days causing thawing of the pavement to begin. The thawing season for Coldspot for this year begins on March 21 (refer to Figure G-2). The freezing index for the entire freezing season from November 7 to March 20 is

$$FI = \Sigma(32 - \bar{T})$$
$$FI = (3 + 8 + 1 + 8 + \ldots + 16 \text{ (March 18)} + 12 \text{ (March 19)} + 6 \text{ (March 20)})$$
$$FI = \underline{1757°F\text{-days}}$$

Estimating the Time to Place Load Restrictions

The pavement consists of 2½ inches of AC on 6 to 8 inches of base. This would be classified as a thin pavement. The "should" level for placing load restrictions for thin pavements is

Col. 1 Day (date)	Col. 2 Measured Daily Temperature (° F) High (T_H)	Col. 3 Low (T_L)	Col. 4 Average Daily Temperature, (°F) (T)	Col. 5 Daily Thawing Index based on 29° F datum (° F - days)	Col. 6 Accumulated Thawing Index based on 29° F datum (°F days)
3/21	43	22	32	3	3
3/22	47	16	32	3	6
3/23	40	23	32	3	9
3/24	44	20	32	3	12
3/25	51	18	34	5	17
3/26	40	29	34	5	22
3/27	49	26	38	9	31
3/28	61	34	48	19	50
3/29	57	34	46	17	67
3/30	39	33	36	7	74
3/31	51	32	42	13	87
4/ 1	42	36	39	10	97
4/ 2	59	27	43	14	111
4/ 3	52	33	42	13	124
4/ 4	34	21	28	-1	123
4/ 5	33	19	26	-3	120
4/ 6	53	16	34	5	125
4/ 7	51	38	44	15	140
4/ 8	50	32	41	12	152
4/ 9	58	26	42	13	165
4/10	69	40	54	25	190
4/11	52	32	42	13	203
4/12	51	30	40	11	214
4/13	54	38	46	17	231
4/14	39	25	32	3	234
4/15	55	17	36	7	241
4/16	69	43	56	27	268
4/17	70	28	49	20	288
4/18	41	23	32	3	291
4/19	43	26	34	5	296
4/20	32	18	25	-4	292
4/21	45	17	31	2	294
4/22	45	32	38	9	303
4/23	42	30	36	7	310
4/24	37	29	33	4	314
4/25	43	28	36	7	321

Note: The values in columns 2 and 3 are obtained from reported local daily high-low temperatures or from a high-low recording thermometer located near the road section to be restricted.

Figure G-2. Form for Calculating Thawing Index.

Col. 1 Day (date)	Col. 2 Measured Daily Temperature (° F) High (T_H)	Col. 3 Low (T_L)	Col. 4 Average Daily Temperature, (°F) (T)	Col. 5 Daily Thawing Index based on 29° F datum (° F - days)	Col. 6 Accumulated Thawing Index based on 29° F datum (°F days)
4/26	59	30	44	15	336
4/27	58	30	44	15	351
4/28	45	31	38	9	360
4/29	58	29	44	15	375
4/30	42	31	36	7	382
5/ 1	51	28	40	11	393
5/ 2	58	26	42	13	406
5/ 3	54	39	46	17	423
5/ 4	56	42	49	20	443
5/ 5	44	30	37	8	451
5/ 6	54	24	39	10	461
5/ 7	64	27	46	17	478
5/ 8	61	41	51	22	500
5/ 9	53	27	40	11	511
5/10	38	25	32	3	514
5/11	49	30	40	11	525
5/12	56	36	46	17	542
5/13	60	34	47	18	560
5/14	63	30	46	17	577
5/15	68	32	50	21	598
5/16	50	30	40	11	609
5/17	47	27	37	8	617
5/18	60	24	42	13	630
5/19	69	29	49	20	650
5/20	79	40	60	31	681
5/21	81	48	64	35	716
5/22	81	46	64	35	751
5/23	76	56	66	37	788
5/24	69	53	61	32	820
5/25	64	47	56	27	847
5/26	49	40	44	15	862
5/27	58	40	49	20	882
5/28	69	36	52	23	905
5/29	77	50	64	35	940
5/30	54	34	44	15	955
5/31	66	32	49	20	975

Note: The values in columns 2 and 3 are obtained from reported local daily high-low temperatures or from a high-low recording thermometer located near the road section to be restricted.

Figure G-2 (cont.). Form for Calculating Thawing Index.

TI_{29} should restrict = 10°F-days

The thawing season starts on March 21.

$$TI_{29} = 3\ (\text{March } 21) + 3\ (\text{March } 22) + 3\ (\text{March } 23)$$
$$3\ (\text{March } 24)$$
$$= 12°\text{F-days}$$

The load restrictions <u>should</u> be placed by March 25.

(Note: Example of calculating thawing index is in Appendix F.)

The "must" level for restricting a thin pavement is

TI_{29} must restrict = 40°F-days

$$TI_{29} = 3\ (3/21) + 3\ (3/22) + 3\ (3/23) + 3\ (3/24) + 5\ (3/25)$$
$$+ 5\ (3/26) + 9\ (3/27) + 19\ (3/28)$$
$$= 50°\text{F-days}$$

The load restrictions <u>must</u> be placed by March 29.

<u>Estimating the Duration for Load Restrictions</u>

The duration may be estimated in days or in thawing degree-days (this method is preferred). It is preferable to estimate the duration of the thawing period using the thawing index based on 29°F.

To estimate the number of thawing degree days required for the restricted period the exact equation is:

$$TI_{29} = 4.154 + 0.259\ (FI)$$
$$TI_{29} = 4.154 + 0.259\ (1757°\text{F-days})$$
$$= 459°\text{F-days}$$

On May 5, the TI_{29} (column 6) is 451°F-days

On May 6, the TI_{29} is 461°F-days

Therefore, the load restrictions should be removed by May 7.

The simpler approximate equation for the thawing degree-days required for the restricted period which may be used in place of the above equation is:

$$TI_{29} = 0.3\ (FI)$$

$$TI_{29} = 0.3\ (1757°F\text{-days})$$

$$= 527°F\text{-days}$$

On May 11, the TI_{29} is 525°F-days.

Therefore, the load restrictions should be removed by May 12.

Alternatively, the duration of the thawing period may be estimated in days.

The exact equation for estimating duration in days is

$$D = 22.62 + 0.011\ (FI)$$

For this freezing season in Coldspot,

$$FI = 1757°F\text{-days}$$

$$D = 22.62 + 0.0111\ (1757°F\text{-days})$$

= <u>42 days</u> from the start of thawing (March 21) = <u>May 2</u>

A simpler approximate equation for estimating duration in days which may be used instead of the preceding equation is

$$D = 25 + 0.01\ (FI)$$

$$D = 25 + 0.01\ (1757°F\text{-days})$$

$$= 25 + 17.57$$

= <u>42 days</u> = <u>May 2</u>