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LOAD RESTRICTION CRITERIA FOR ALASKAN ROADS
PHASE 1: GENERAL CONSIDERATIONS AND FIELD STUDIES

Volume 1

by

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16. Abstract

This report presents the results of a study to evaluate the effects of removing spring load restrictions on the Steese/Elliot and Haines highways. Specifically, the research addressed the actual time in which FWD pavement response is most critical and estimated the potential damage and strengthening requirements to these routes if no springtime load restrictions are applied. Also, it investigated the influence of tire pressure on pavement damage and the possibility of reducing tire pressure while maintaining springtime 100 percent legal load limits.

The research involved extensive field work and analyses using FWD data on the Steese/Elliot and Haines highways, roughness and rut measurements on the Steese/Elliot, and ground temperature measurements at specific pavement sites in the Central and Northern regions. In addition, multilayer elastic analyses were conducted to determine the impact of reducing tire pressure on pavement behavior and damage for different thaw conditions.

FWD results for Steese/Elliot and Haines highways were used to estimate pavement remaining life and establish criteria for "weak" pavements if no springtime load restrictions are applied. These results indicate that the loss of pavement strength is most critical during thaw initiation in the base and least critical when the thaw reaches a depth of 3.5 feet approximately. Ground temperature measurement is, therefore, a better indicator than the FWD for estimating timing and duration of the load restriction period. Criteria for using ground temperature data to estimate the load restriction period were developed. Rutting and roughness measurements on the Steese/Elliot highway indicate that road damage associated with frost heaving and foundation instability due to permafrost thaw seems to be more significant than load related damage.

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ABSTRACT

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CHAPTER ONE

INTRODUCTION

1.1 BACKGROUND

Restricting allowable axle weights is essentially the only practical way of minimizing excessive pavement damage that would otherwise occur during spring-thaw conditions. It has been shown by numerous researchers that a given axle or wheel load applied to a thaw-weakened pavement structure can cause orders of magnitude more damage than the same load applied to the thawed pavement (Rutherford 1988; Coetzee and Connor 1994). In particular, Alaskan researchers have performed significant work relating pavement load and deflection response to damage (Esch 1972; Esch et al. 1980; Connor 1980; Stubstad and Connor 1982; Connor 1984; Coetzee and Connor 1994). The pavement damage factors research performed by Coetzee and Connor (1994) illustrates that maximum pavement damage does not necessarily coincide with maximum deflection, and that damage occurs at different times during the thaw period for different pavement materials (AC, base, subgrade). As expected, damage is closely related to thaw depth. This may provide an "early warning" indicator so that load restriction notices may be possible well in advance of actual pavement thawing conditions. Current policy involves monitoring deflections, and notice periods typically allow fully loaded axles on the pavement during the early thaw period — the time at which surface damage potential is highest.

Of particular importance is the evaluation of "no load restriction" policy and reduced tire pressure on pavement damage of selected routes in Alaska. These routes, chosen for the investigation of ability to carry legal loads year round without restrictions, are the Haines and the Steese/Elliot highways. This work was conducted through periodic monitoring of pavement surface deflections using the Falling Weight Deflectometer (FWD) and mechanistic evaluations of corresponding pavement damage. Periodic deflection measurements were conducted in the early spring of 1993 and 1994 and were compared to deflections obtained for a summer reference condition. In addition, roughness and rut measurements were conducted on the Steese/Elliot highway to evaluate the potential pavement damage associated with lifting spring load restrictions during 1994 and 1995. Possible damage control through reduced tire pressure was also investigated via structural analysis of representative pavement sections. And, ground temperature data obtained at different sites in the Central and Northern regions were analyzed to investigate ground thaw initiation and propagation.

1.2 OBJECTIVES

This research evaluated the effects of removing spring load restrictions on the Haines and Steese/Elliot highways. Specifically, the following were addressed:

1. Evaluate the true critical time during which springtime load limits should be applied on these routes and the time period in which FWD measurements reveal most critical stresses and strains in the pavement.
2. Assess the potential damage to these routes if no springtime load restrictions are applied; also determine what parts of these routes require strengthening to eliminate the need for springtime restrictions below the 100 per cent legal load limit.

3. Investigate the influence of tire pressure reduction on springtime pavement damage and the possibility of reducing tire pressure while maintaining full legal load limits during spring.

1.3 ORGANIZATION OF REPORT

This report is organized as follows:

1. Chapter One - *Introduction*

Background information on damage associated with spring-thaw weakening of pavements and current load restriction policy in Alaska are presented and the objectives of the research study are summarized.

2. Chapter Two - *A Review of Load Restriction Practices and Criteria*

This chapter reviews available restriction criteria. Specifically, applications of load restrictions by different agencies are summarized and the limitations of current restriction criteria are discussed.

3. Chapter Three - *FWD Measurements and Pavement Damage Assessment*

FWD data and backcalculated layer moduli are presented for the Haines and Steese/Elliot highways. These data, obtained periodically during spring and summer, were used in the determination of damage factors resulting from spring-thaw weakening. The strength of pavement sections along these routes is evaluated and strengthening requirements to eliminate the need for springtime restrictions below the 100 per cent legal limit are determined. In addition, roughness and rut measurement data for the Steese/Elliot highway are compared before and after spring-thaw in order to assess the influence of removing load restrictions on pavement damage.

4. Chapter Four - *Ground Temperature Measurement and Analysis*

Ground temperature data for selected sites in the Northern and Central regions are analyzed to estimate spring-thaw initiation and propagation. A probabilistic approach is proposed to estimate thaw initiation in the pavement base assuming normal distribution of ground temperature data. Thaw propagation models are developed using best fit regression equations.

5. Chapter Five - *Effect of Tire Pressure on Pavement Damage*

The influence of tire pressure on pavement damage under spring-thaw weakening conditions is investigated. Specifically, the possibility of reducing tire pressure while maintaining full legal load limits during spring is addressed.

6. Chapter Six - *Summary, Conclusions and Recommendations*

Summary and conclusions of the research findings are presented particularly in relation to the objectives of the research project. Recommendations for spring load restriction guidelines are proposed for Alaskan highways. Results obtained under this investigation are used to identify future research needs.

CHAPTER TWO

A REVIEW OF LOAD RESTRICTION PRACTICES AND CRITERIA

2.1 INTRODUCTION

Observations of field pavements indicate that seasonal changes in structural strength are dependent upon temperature, pavement materials, and drainage conditions. The problem becomes more significant in areas susceptible to spring-thaw conditions where accelerated pavement distress such as alligator cracking, block cracking, rutting, and potholes may occur shortly after pavement thawing. On primary roads, the anticipated changes in load carrying capacity of the pavement structure are usually accommodated during the design phase. Secondary roads, subjected to lower traffic, are designed with thinner sections and without consideration to thaw weakening during spring break-up. In Alaska, the majority of the roads fall in this category, and spring load restrictions are imposed to minimize pavement damage and reduce maintenance costs (Connor 1980; Stubstad and Connor 1982; Coetzee and Connor 1994).

Loss of load carrying capacity of pavements affected by frost has been estimated using pavement deflection data. Deflection measurements using plate load tests indicate that possible strength reduction during spring relative to fall could be as much as 85 percent depending on climate, drainage, and subgrade conditions (Motl 1950; Motl 1955; Meskal 1959). Other studies using Benkleman beam measurements show similar trends (Preus and Tomes 1959; Armstrong and Csathy 1963). In Alaska, studies to develop seasonal load limits started as early as 1951 (Culley 1976), and recommendations for maximum allowable axle loads were developed as a function of total thickness of base and wearing course. Additional monitoring of Alaskan highways led to the application of load restrictions during the spring thaw period, starting when FWD deflections begin to increase and continuing until they pass their maximum value (Connor 1980). In a follow-up study, Stubstad and Connor (1982) state that "small deflections over a very weak base and a frozen subgrade are more damaging than large deflections over stronger base and thawed subgrade, since the condition for dramatic or total failure in the base is indicated by, e.g., the excessively high vertical base strains under load." Further research by Coetzee and Connor (1994) clearly illustrates that maximum pavement damage does not necessarily coincide with maximum deflection, and that damage occurs at different times during the thaw period for different pavement components.

State-of-the-art application of seasonal load restrictions as suggested by a number of state agencies and published research indicates that restriction criteria may be categorized as follows:

1. Experience and visual observation of pavement distress during spring break-up such as cracking, heaving, and pumping.
2. Surface deflection measurements including Benkleman beam and FWD. Axle loads in this case are restricted so that spring deflections do not exceed a summer "reference" condition.
3. Pavement response parameters such as tensile strains in the asphalt concrete, vertical strains or stresses in the unbound base and subbase, and subgrade vertical strains. These parameters are backcalculated from surface deflections or computed using mechanistic analysis with estimated layer properties. Load

restrictions are imposed so that these parameters do not exceed acceptable summer values.

4. Application of cumulative damage concepts (i.e., Miner's hypothesis) such that the damage induced by one application of the restricted axle load would not exceed the damage associated with a corresponding application of the standard axle load for a summer reference condition.
5. Reduction of the spring axle loads so that spring and summer pavement serviceability (i.e., PSI) are essentially the same.
6. Determination of the depth and duration of thaw below the pavement surface using field measurements or analysis in order to assess the appropriate period for imposition of restrictions.

This chapter reviews available load restriction criteria. Specifically, applications of load restrictions by different agencies will be summarized and the limitations of current restriction criteria will be discussed.

2.2 CURRENT PRACTICE

Rutherford et al. (1985) conducted a survey on the application of load restrictions in Canada and the U.S. A summary of the findings (Tables 2.1-2.3) includes the following:

1. Sixteen of the thirty-three states and four of the five Canadian provinces surveyed indicated they did impose load restrictions.
2. Four of the states and three of the Canadian provinces indicated that their load restrictions were based on analysis of pavement deflections. The remaining established their load restrictions based on experience. The types of pavement

failure observed during spring-thaw include alligator cracking, rutting, frost heave, and pot-holes.

3. Thirteen of the states and four of the Canadian provinces had guidelines and/or legislation for load restrictions.
4. The range of load restrictions for Spring load limits varied from 40 percent to 75 percent of normal axle loads.
5. A total of three of the states and Canadian provinces indicated that they used pavement deflection measurements as the basis for removal of load restrictions, whereas fourteen of the surveyed states and provinces used field observations and experience.

Haas (1992) conducted a survey on seasonal load restriction applications in Michigan. Most counties surveyed indicated that timing of load restrictions was based on thaw initiation data from frost tubes and other visual signs of thawing, mainly water seeping out through cracks in thin surfaced roads. Load restrictions were lifted when the structural strength of the pavement was restored as indicated by drying of the pavement surface and thawing to a depth of at least 4 feet.

The Washington Department of Transportation (1994) published results on a policy plan for weight restrictions and road closures. Survey results from 39 counties and 271 cities show that fifty-nine percent of the respondents were affected by temporary road weight restrictions or closures during the past two years. Results also indicate that advanced notification of weight restrictions was generally accomplished by posting signs on road sections less than 24 hours before the effective date. The study concluded that in spite of numerous ways to determine when to implement and remove load restrictions,

Table 2.1 Summary of Current State and Province Load Restriction Practices (Rutherford et al. 1988)

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		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	
Minnesota	x			x	x	x	
Missouri		x		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		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	

Table 2.2 Guidelines for Spring Load Restrictions (Rutherford et al. 1985)

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 2.2 Guidelines for Spring Load Restrictions (Rutherford et al. 1985) (Cont'd)

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	All road construction types	Experience
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
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 2.3 Applications of Load Restriction Criteria (Rutherford et al. 1985)

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 gauge 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)
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

Table 2.3 Applications of Load Restriction Criteria (Rutherford et al. 1985) (Cont'd)

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?
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 Defection measurements	Yes (Benkelman Beam)
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

the primary difficulty is in identifying the road segments that should be restricted and the corresponding weight restriction limit. Results of a survey of current practice in the U.S. and Canada are presented and summarized as follows:

1. Load restrictions are applied mostly to aggregate and/or asphalt surfaced pavements with subgrades consisting of moisture susceptible silts and clays.
2. The maximum legal loads are generally reduced by 40 to 50 percent for single axles and 30 to 50 percent for tandem axles.
3. In general, judgment of field personnel is used to determine the criteria for application of load restrictions.
4. Route selection for load restriction application depends on a number of factors such as spring and summer deflections, surface thickness, moisture conditions, and subgrade type.
5. Ground temperature data are also used to assess when and for how long to apply load restrictions.

In Alaska, spring-thaw load restrictions are limited, at present, to 75 percent of the legal axle load. The timing of load restrictions is set by the Regional Maintenance Engineer based on ground temperature data from selected sites, field observations of moisture seepage through open cracks in the road surface, judgment, and experience. Attempts are generally made to impose load restrictions as soon as the pavement base starts to thaw. Load restrictions are lifted when the observed pavement surface becomes relatively dry and the thaw depth reaches about 4 feet below the pavement surface. According to research findings by Connor (1980), Stubstad and Connor (1982), and

Coetzee and Connor (1994), the decisions of the Regional Maintenance Engineer are often influenced by FWD testing and pavement damage analysis.

2.3 LOAD RESTRICTION CRITERIA

2.3.1 Measured Surface Deflections

The loss of pavement strength during spring-thaw was first determined from surface deflection measurements using plate load tests. The first study on the reduction of pavement bearing capacity as a result of frost heaving and thaw weakening was conducted by Taber (1929). This was followed by field and laboratory studies, performed between 1948 and 1955, in a number of states that applied load restrictions. Results of these studies (Motl 1948, 1951, 1955,) indicated that the loss of bearing capacity in the spring ranges from zero to 65 percent. Preus and Tomes (1959) assessed the spring-thaw weakening effect of pavement sections in Minnesota from surface deflection profiles obtained using the Benkelman Beam. Other Benkelman Beam data collected in Canada showed the loss of pavement carrying capacity during spring ranges from 30 and 60 percent (Armstrong and Csathy 1963). In a study of the Vormsund Test Road in Norway, Nordal (1982) compared spring deflections with summer values using the Benkelman Beam. Results indicate a 30 percent increase in deflections for sections with silt subgrade and a 70 percent increase for sections with clay subgrade. After conducting a study on a test road in Switzerland, Dysili (1982) concluded that increased surface deformations during thaw-weakening may result from shear failure in the base due to excess pore pressure and low base permeability.

In addition to being used for the identification of weak pavement sections during spring-thaw, surface deflection criteria have been utilized for load restriction applications. Scrivner et al. (1969) proposed an empirical criterion for allowable spring load in terms of a Surface Curvature Index, SCI, and maximum deflection, δ_m. Load restrictions, in this case, should be imposed if the freezing index is greater than 200 ^{o}F-days, the summer SCI is greater than 0.35 milli-inches, and the summer Benkelman Beam deflection is greater than 0.023 inches. SCI is determined from Dynaflect deflection data such that:

$$SCI = w_1 - w_2 \tag{2.1}$$

where w_1 is the deflection at sensor 1 under the load, and w_2 is the deflection at sensor 2, 12 inches from the load.

The allowable spring load L_s is then expressed as

$$L_s = 6.3/SCI_m \tag{2.2}$$

or as

$$L_s = 346/\delta_m \tag{2.3}$$

where L_s is the allowable spring load in kips, SCI_m is the maximum spring Surface Curvature Index, and δ_m is maximum Benkelman Beam deflection in spring in milli-inches under an applied 18 kip load.

Connor (1980) presented a comprehensive summary of a number of Alaskan studies that have been conducted on spring load restrictions. These studies started as early as 1951, when a joint investigation by the Alaska Road Commission and the Bureau of Public Roads was conducted to determine load restriction requirements for Alaskan highways during spring-thaw. Results were determined from observed pavement distress under applied truck loads. These observations were used to develop recommendations for maximum allowable axle load as a function of total base and surface thickness of pavements. In the spring and summer of 1972, the State Materials Laboratory performed a deflection study on a number of pavements in Fairbanks using the Benkelman Beam. According to Connor (1980), the study indicated that "peak spring deflections can occur very quickly after average daily air temperatures rise above 32 ^{o}F." A follow-up investigation in 1976, using Benkelman Beam data on selected roads in Anchorage, showed that in most cases load restrictions on roads originally selected by maintenance personnel were not necessary. In other words, according to Connor (1980) "methods using intuition often prove wrong." Esch et al. (1980) reported the results of a study conducted on typical pavement sites in Fairbanks and Anchorage which compared thaw depth with surface deflections. It was concluded that: 1) thaw depth data would be useful in estimating the beginning and end of the load restriction period; 2) there is no substitute for using surface deflections to determine spring load restriction level; and 3) maximum spring deflections seem to occur when thaw depths are between 3.5 ft and 5 ft.

Connor (1980) proposed a method for determining required load limits for thaw-weakened pavement sections using surface deflection measurements and a design relationship for surface deflections and axle load repetitions developed in Ontario, Canada (Haggstrom 1974). The axle load limit for the thaw-weakened section, W_2, is determined from the following equation:

$$W_2 = (\text{DTN (lower)/DTN (desired)})^{0.209} (W_1 + 1) - 1 \qquad (2.4)$$

where DTN (lower) is the Design Traffic Number for the higher deflection level of the thaw-weakened section, DTN (desired) is the Design Traffic Number for the normal or "reference" pavement condition, W_1 is the unrestricted axle load in kips, and W_2 is the restricted axle load in kips. Connor (1980) presented this procedure in nomographic form (Figure 2.1). Based on a number of contacts with maintenance engineers in the Alaska Department of Transportation and Public Facilities (AKDOT&PF), it appears that the application of this method to determine the timing and level of spring load restrictions is limited. In Alaska, 75 percent spring load restriction level is used. Maintenance engineers at AKDOT&PF rely more on ground temperature data for application and removal of load restrictions, particularly since the observed maximum FWD deflections do not coincide with the critical spring-thaw period.

Minnesota Department of Transportation (1985) proposed using an "area parameter" calculated from FWD surface deflection bowl. The area parameter, A, is defined as,

$$A = 1 / (2D_o) \sum_{i=1}^{n} (D_{i-1} + D_i)(R_i - R_{i-1}) \qquad (2.5)$$

where A is the area parameter, D_i is the deflection under the ith sensor, R_i is the radius to the ith sensor, D_0 is the deflection under the center of the load, and n is the number of load sensors, generally equal to 7.

The area parameter can be used as an indicator of the stiffness of the pavement. The area parameter increases with increasing stiffness of the pavement system and should vary from 11 (minimum) to 36 inches (maximum) according to available theoretical solutions (Hoffman 1980). FWD testing reported by Minnesota Department of Transportation (1985) indicates the area parameter is at its lowest value for thaw depths between 6 inches and 12 inches from the bottom of the surface course. In addition, the calculated tensile strains in the asphalt concrete surface reach their peak values prior to

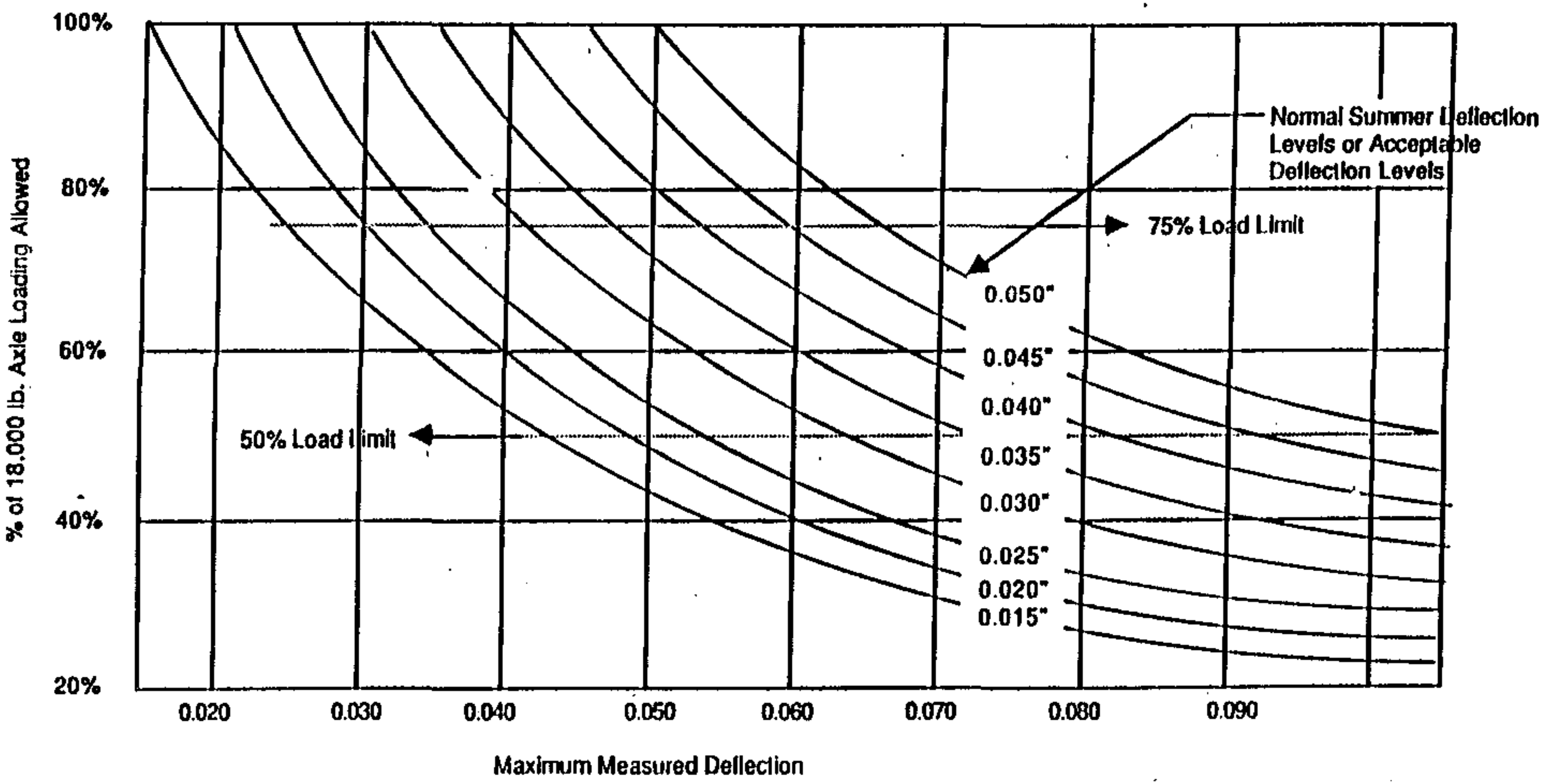

Figure 2.1 Percent Allowable Spring Axle Loads Versus Maximum Measured Deflections (Connor 1980)

maximum surface deflections. Recommendations based on this study suggest that the beginning of the load restriction period should be determined by means other than deflection testing because of the quick deterioration of the pavement strength at the onset of thaw. Deflection testing may be used, however, to estimate the end of the load restriction period.

2.3.2 Pavement Response Parameters

Load restriction applications during spring-thaw can be determined by limiting the critical response parameters of the pavement structure to acceptable summer-fall normal values. In this case, the maximum axle load allowed on the thaw-weakened pavement is restricted so that the critical response parameters do not exceed the normal "reference" values. Stubstad and Connor (1982) indicated that in partially thawed pavements total deflections alone may not be adequate to establish load restrictions. In pavements with weak base overlying a frozen subgrade, high strains in the granular base can occur before maximum spring deflections are attained. This could lead to failure initiation in the granular base and eventually propagate to the asphalt concrete surface. Stubstad and Connor (1982) developed an improved method for determining the allowable axle load on Alaskan roads during spring-thaw. This method uses FWD surface deflection measurements which are coded under a computer program called FROST. The program compares the measured deflection basin under a 9,000 lb applied load to a series of theoretical deflection basins for 350 combinations representing typical Alaskan pavements and summarized in a "solution table." A "best fit" deflection basin is selected, and corresponding estimates are made for thaw depth, vertical strain on top of the granular base, and a "corrected" center deflection assuming no frozen material exists in the pavement structure. Knowing the "corrected" peak deflection and the normal summer deflection, the appropriate level of load restriction is then determined from the nomograph in Figure 2.1. Stubstad and Connor (1982) suggest using the vertical strain at the top of the granular base as "the most indicative measure of load-damage potential for springtime, thawing conditions."

Lary et al. (1984) investigated spring-thaw weakening of pavements in the state of Washington using both field FWD measurements and laboratory studies on pavement materials. A number of pavement response parameters were evaluated including maximum surface deflection, asphalt tensile strain, vertical base strain, and vertical subgrade strain. The load level for which any of these parameters exceeds the summer reference value was defined as critical. Both surface deflection and vertical base strain were found to be the most critical parameters necessitating spring load restriction. A load reduction of 60% was recommended during spring-thaw weakening.

Rutherford et al. (1985) analyzed typical pavement sections using representative material properties, different combinations of wheel loads, and a range of thaw depth values. Multilayer elastic assumptions were considered in calculating surface deflections, vertical base strain, and vertical subgrade strain, using the computer program ELSYM5 (Ahlborn 1972). The applied wheel load was restricted so that the critical spring response parameters did not exceed the summer reference values. Results of the analyses showed that the tensile strain at the bottom of the asphalt layer and the vertical strain on top of the subgrade resulted in the largest reductions in the applied load. The following guidelines for where to apply load restrictions and the magnitude of the allowable loads were recommended:

1. Pavement sections which have maximum spring surface deflections that are 45 to 50 percent higher than the summer reference values are candidates for load restrictions.
2. Load restrictions need to be considered in areas where the Freezing Index (FI) is greater than 400 ^{O}F-days for pavements with surface thicknesses less than or equal to 2 inches.
3. Load restrictions become more important for pavements with fine-grained subgrades and poor drainage conditions. Additional considerations include local experience and observed pavement distress (fatigue cracking, rutting).
4. Load reductions with a magnitude of between 40 and 50 percent are generally required to accommodate a range of pavement conditions.

Similar analyses using ELSYM5 and assumed pavement properties were conducted by Rutherford (1988). It was shown that for the selected pavement sections, allowable loads for the two inch uncracked asphalt concrete pavements were always governed by fatigue. The allowable loads did not exceed 36 percent, based on equal summer fatigue performance (i.e., equal repetitions to failure of asphalt concrete surface). In this case, fatigue performance was most critical during early subgrade thawing and remained essentially unchanged throughout the spring-thaw period. For the four inch uncracked asphalt concrete pavements, the subgrade vertical strains became most critical during early subgrade thawing. Load restrictions in the range of 19 to 100 percent were required to maintain vertical strain values equal to the summer reference values. Based on the results of analyses for applying load restrictions according to critical pavement response parameters, Rutherford

(1988) assessed the use of surface deflections for determining spring load reductions in the following words:

> "One of the most valuable conclusions to be drawn from these results is that generally there is no direct correlation between deflection relative to summer and the need for load restrictions during spring thawing. Pavements with the best base and subgrade materials experiencing the least reductions in stiffness during spring often resulted in the greatest increases in deflection relative to summer. The absolute values of deflection during spring and summer for such pavement are quite low and although these pavements will experience accelerated rutting or fatigue during spring they generally would not experience any permanent observable damage."

2.3.3. Pavement Damage

It is generally accepted that pavement damage associated with traffic loads depends on both the magnitude and number of repetitions of the applied loads. Miner's hypothesis (Miner 1945) has been used in pavement analysis to determine load associated damage as follows:

$$CD = \sum_{p,k} (n_{pk} / N_{pk}) \tag{2.6}$$

where, CD is the cumulative pavement damage; n_{pk} is the number of applications of the pth load while the pavement is in the kth condition; and N_{pk} is the number of applications of the pth load that would produce pavement damage, as determined by a given performance criterion, while the pavement is in the kth condition. The pavement is expected to "fail" when CD becomes equal to one.

Hardcastle et al. (1983) applied this concept to determine spring load restrictions by equating spring and summer cumulative damage associated with fatigue of the asphalt concrete surface. In this case, multilayer elastic theory was used to predict the asphalt tensile strain which, in turn, was used as a damage indicator to select allowable spring loading. Results indicate that the pavement considered required 39 percent load reduction in comparison with 27 percent when equal surface deflection criterion was used. Using the criterion proposed by Scrivner et al. (1969), a conservative estimate of load reduction equal to 49 percent was obtained.

Connor (1980) used in part the concept of cumulative damage in determining spring load restrictions according to either Equation 2.4 or the nomograph in Figure 2.1. Coetzee and Connor (1994) suggested spring load restriction levels for Alaskan pavements such that the pavement damage induced by one application of the restricted load during the critical spring-thaw period does not exceed the damage caused by the standard axle load during a reference summer condition. In this case, Miner's hypothesis can be used to define a damage factor, RD, associated with a given axle load, P, as follows:

$$RD = N_{fo}/N_{fs} \tag{2.7}$$

where N_{fo} is the number of applications of a standard axle load required to "fail" the pavement during a summer reference condition and N_{fs} is the number of applications of axle load, P, required to "fail" the pavement during spring-thaw conditions. Values of N_{fo} and N_{fs} can be determined by using appropriate pavement distress models and corresponding response parameters for pavement summer reference condition and spring condition, respectively. Spring loads are restricted in this case so that the load damage factor is one.

The number of load repetitions required to induce failure in the pavement structure can be determined using limiting criteria for the critical pavement response parameters (NCHRP (1-26) 1990; Anderson et al. 1990; Gillespie et al. 1993). For flexible pavements, these criteria are used to assess two primary modes of distress. Flexure strains on the underside of the asphalt concrete surface promote fatigue which eventually leads to cracking and breakup of the pavement structure. Similarly, vertical strains on top of the subgrade result in the accumulation of permanent strains or rutting of the pavement structure. During spring-thaw, excessive moisture conditions in the granular base may cause a reduction in its shear strength and stiffness thereby resulting in "weakening" conditions that could accelerate fatigue and rutting of the pavement surface (Stubstad and Connor 1982). According to Ullidtz (1987), pavement damage associated with granular base or subbase "failure" is controlled by the maximum vertical stress on top of these layers.

The limiting criteria for asphalt concrete surface, base/subbase, and subgrade as used in this study are summarized below.

Asphalt Concrete Surface

The Asphalt Institute fatigue equation (Shook et al. 1982) is used to determine the number of repetitions required to cause fatigue failure in the asphalt concrete surface. This equation takes the form

$$N_f = 18.4\ C\ (4.32 \times 10^{-3}\ \varepsilon^{-3.29}\ (E_d)^{-0.854}) \tag{2.8}$$

where

$$C = 10^M \tag{2.9}$$

and

$$M = 4.84\ (V_b/(V_b + V_v) - 0.69) \tag{2.10}$$

where, N_f is the number of load repetitions to failure; ε is the tensile strain (in/in); E_d is the dynamic modulus (psi); V_v is the volume of air voids (%) in mix; and V_b is the volume of asphalt cement (%) in mix. In many cases, C in Equation (2.8) is set equal to 1 (Shook et al. 1982) and the resulting fatigue relation can be written as

$$N_f = 0.07745\ \varepsilon^{-3.29}\ (E_d)^{-0.854} \tag{2.11}$$

For typical Alaskan mixes (i.e., 3 percent air voids, 5.8 percent asphalt (by volume), and mix density equal to 146 lb/cu. ft.), the corresponding value of C in Equation (2.9) is equal to 4.22, and Equation (2.8) can be rewritten as

$$N_f = 0.336\ \varepsilon^{-3.29}\ (E_d)^{-0.854} \tag{2.12}$$

E_d can be estimated from the following Asphalt Institute relationship

$$E_d = F(P_{200}, f, V_v, \eta_{70^\circ F}, t_p, V_b) \tag{2.13}$$

where, P_{200} is the percent minus 200 sieve; f is the frequency of loading, Hz; $\eta_{70^\circ F}$ is the original absolute viscosity of the bitumen measured at 70 $^\circ$F, in 10^6 poises; t_p is the temperature, $^\circ$F; V_v is the percent of air voids in the mix; and V_b is the percent volume of binder in the mix.

Granular Base/Subbase

The failure of granular layers in pavement structures under repeated applications of traffic loads is governed, according to Ullidtz (1987), by the maximum vertical stress or strain on top of the layer. Ullidtz (1987) suggests using a stress rather than a strain criterion since stresses of the layer under consideration are less dependent on the Poisson's ratio. In this case, the number of stress applications to failure, N_f, is related to the vertical stress, σ_v (psi) by the following equation (Ullidtz 1987):

$$\sigma_v = 17.4(N_f/10^6)^{(-0.307)}(E_b/23{,}200)^n \tag{2.14}$$

where, E_b is the modulus (psi) of the granular layer; n is 1.16 when E_b is less than 23,200 psi, otherwise n is equal to 1. This equation is derived from AASHO regional design factor (Yoder and Witczak 1975), R equal to 1.75. For typical Alaskan conditions, R is assumed equal to 3 and Equation (2.14) reduces to

$$\sigma_v = 1024.9\ (N)^{-0.307}(E_b/23{,}200)^n \tag{2.15}$$

Subgrade

The Asphalt Institute rutting criterion (Shook et al. 1982) is used to determine the number of allowable repetitions for a given subgrade vertical strain required to cause 13 mm rut depth. This criterion is applicable to fine-grained subgrades and is expressed as

$$\varepsilon_v = 482\ (N_f/10^6)^{-0.223} \tag{2.16}$$

where ε_v is the maximum vertical strain on top of the subgrade in micro-in/in and N_f is the allowable repetitions for 13 mm rut depth. For the coarse-grained subgrade, the limiting criterion for vertical stress presented in Equation (2.15) can be used.

2.3.4 Pavement Serviceability Considerations

AASHO Road Test data presented by Painter (1965) on the variation of pavement serviceability with load applications show a significant loss in the Serviceability Index (PSI) during spring-thaw. The estimated loss of pavement serviceability for two typical pavement sections (5 in. AC/3 in. base/12 in. subbase and 3 in. AC/6 in. base/8 in.

subbase) associated with spring-thawing varies between 10 and 20 percent. Mamlouk (1984) suggested using pavement serviceability as a criterion for spring load restrictions. Typical pavement sections were analyzed using VESYS-3-A computer program (Kenis 1977) where elastic, plastic, and viscous properties of the pavement could be modeled. The program LOADLMT, developed by Mamlouk (1984), is used with VESYS-3-A to compare pavement PSI during spring and summer loading. Spring load limits determined by reducing the axle load until spring and summer PSI's were within acceptable tolerance.

2.3.5. Ground Temperature Criteria

Spring-thaw weakening of pavement structures starts with base thawing and ends generally when complete thawing of the pavement structure has occurred. Ground temperature can therefore be used to indicate when to apply and remove load restrictions. A comprehensive study based on ground thermal analysis using the finite element method of typical pavement sections of varying thicknesses and properties was performed by Rutherford et al. (1985). Results of this study were used to establish criteria for applying and removing load restrictions. These criteria utilize the Thawing Index (TI) calculated from a 29 ^{o}F datum and high-low daily air temperature. A summary of the established criteria is as follows:

1. Load restrictions "should" be applied when the Thawing Index (TI) reaches 25 ^{o}F-days. This corresponds to complete thaw of the base course.
2. Load restrictions "must" be applied when TI reaches 50 ^{o}F-days. This corresponds to approximately 4 inches of thaw below the bottom of the base course.

3. For thin pavements consisting of 2 inches of asphalt concrete surface and 6 inches of base, TI is 10 °F-days for the "should" level, and 40 °F-days for the "must" level.

4. The duration of thaw, t_c (in days), is a function of the Freezing Index (FI) and can be approximated by

$$t_c = 22.62 + 0.011\ (FI) \tag{2.8}$$

5. The TI associated with complete pavement thawing is estimated from

$$TI = 4.154 + 0.259\ (FI) \tag{2.9}$$

Rutherford (1988), conducting a follow-up study using finite element solutions for ground thermal analysis for a variety of pavement and climatic conditions, concluded that base thawing was complete in 1 to 4 days after the start of thaw for thin pavements (2 inch AC, 6 inch base) and 4 to 9 days for thick pavements (4 inch AC, and 12 inch base). On the other hand, 4 inches of thawing into the subgrade was complete 6 to 10 days after the start of thaw for thin pavements and 9 to 16 days for thick pavements. The duration for complete thaw of the pavement structure, t_c (days), could be estimated from

$$t_c = 0.02\ (FI) + 9.5 \tag{2.10}$$

Results of the thermal analyses performed by Rutherford (1988) were also used to establish estimates for the initiation of pavement thawing. This is illustrated in Figure 2.2. In this case, if the average daily temperature is equal to or exceeds the temperature shown in the plot, then pavement thawing is more likely to have started.

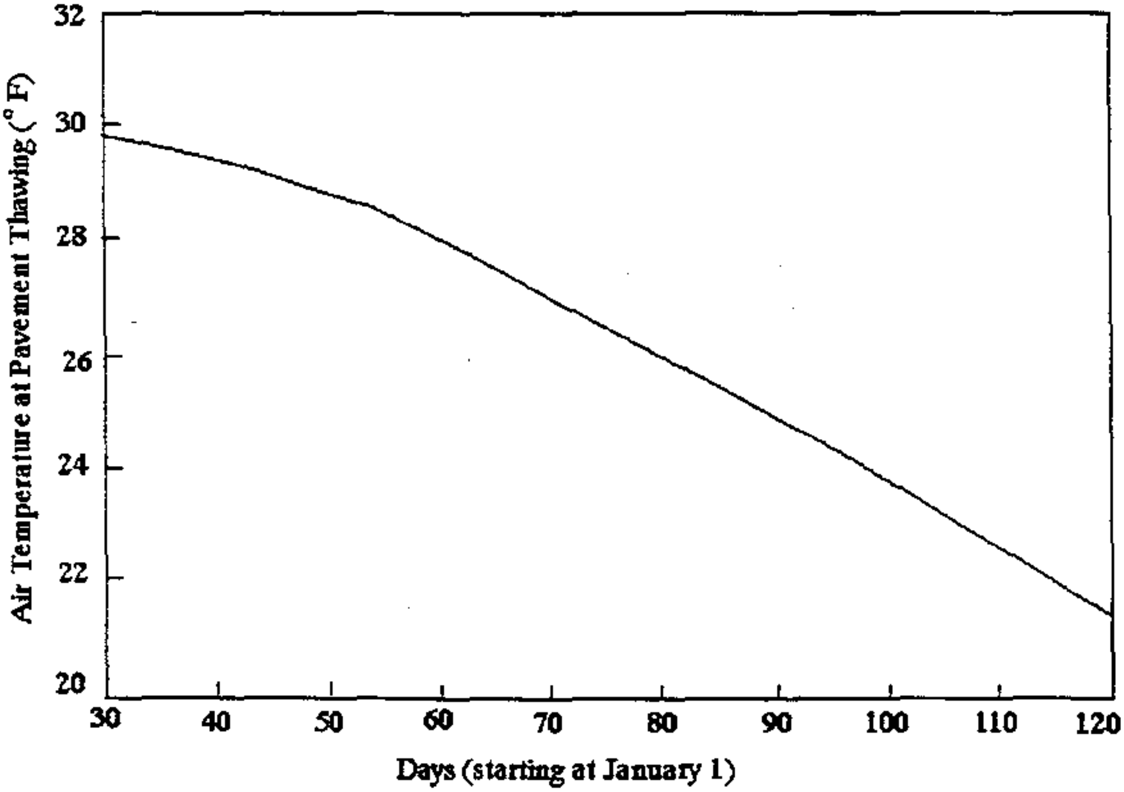

Figure 2.2 Air Temperature at Pavement Thawing Variation with Days Starting January 1 (Rutherford 1988)

2.4 LIMITATIONS OF LOAD RESTRICTION CRITERIA

The load restriction criteria presented in this Chapter could be used to provide guidelines for estimating allowable load levels during the critical period of spring-thaw weakening of the pavement structure. However, since these guidelines have a number of limitations, engineering judgment and experience remains a major factor in applying spring load restrictions. These limitations include the following:

1. Spring surface deflections may not reflect the critical period of pavement thaw-weakening since maximum deflections do not necessarily coincide with maximum base strains and stresses. In many cases, maximum deflections occur after complete thawing of the pavement section. Limiting spring loads using maximum deflections may still cause unacceptably large vertical strains and stresses in the base during early thawing.
2. As a "damage indicator," load restriction levels determined by limiting the critical base strain would be a significant improvement.
3. Surface deflections may not provide an accurate estimate for the beginning of the load restriction period because of the quick deterioration of pavement strength following initiation of pavement thawing. Surface deflections, however, could be used in determining the end of the load restriction period.
4. There seems to be no direct correlation between surface deflection relative to summer and the need for spring load restrictions. Pavements with the best base and subgrade materials may end up with the maximum load restrictions. Although the absolute values of surface deflection in the spring and summer will

be small in this case, the ratio of spring to summer deflection could be large thereby leading to larger load restrictions.

5. Spring load reductions obtained from equating spring and summer critical pavement response parameters associated with a given distress mode may be unconservative. In this case, although the response parameters are equal, the strength of pavement materials during spring-thaw could be much smaller than the summer reference condition.
6. Load restrictions determined by equating the pavement damage induced by one application of the restricted load during spring-thaw to that induced by a standard axle load application for the summer reference condition could result in larger load restrictions for pavements with better quality materials. In this case, although the absolute damage values in spring and summer could be small, the damage ratio could be large.
7. Estimates of spring load levels using pavement deflections, stresses and strains, or load damage factor criteria do not account for possible increase in spring load applications as a result of imposing load restrictions.
8. Load restrictions using equal spring and summer pavement PSI require the ability to predict pavement rutting and fatigue cracking under a variety of loading and climatic conditions. The application of this method in Alaska, for example, will be more complicated since a loss of pavement serviceability may result from unstable foundations due to frost heaving and permafrost thaw.
9. The Thawing Index approach for predicting the beginning and duration of pavement thaw is based on thermal analysis of typical pavement sections and

climatic conditions. Although this approach provides estimates for average conditions, it does not account for local variations such as the existence of permafrost and extremes in solar radiation due to latitude factors that can significantly alter ground temperature profiles.

2.5 SUMMARY

In this Chapter, the application of seasonal load restrictions by different agencies in the U.S. and Canada was summarized. The current practices emphasize engineering experience and judgment for imposing or lifting of load restrictions. These decisions are supported in many cases by site specific ground temperature data or frost tube data. Surface deflection measurements are rarely used in the load restriction procedures.

A number of load restriction criteria based on published research were also presented and discussed. The major limitations of these criteria focus on the ability to predict when to apply and remove load restrictions and the required load restriction level associated with a given acceptable pavement damage.

The next chapter of this report addresses: 1) the potential pavement damage that could occur to the Steese/Elliot and Haines highways in Alaska if no load restrictions are applied, and 2) the strengthening of "weak" pavement sections required if spring load restrictions are not imposed. The analysis incorporates FWD deflection data and limiting criteria using critical pavement response parameters, pavement roughness, and rut measurements.

CHAPTER THREE

FWD MEASUREMENTS AND PAVEMENT DAMAGE ASSESSMENT

3.1 OBJECTIVES

Backcalculation procedures using pavement surface deflections to determine the reduction of structural strength during spring-thaw and the level of load restrictions that need to be applied have been suggested by a number of investigators(Connor 1980; Lary et al. 1984; Coetzee and Connor 1994). These procedures are generally applied to backcalculate layer moduli, pavement response, and remaining pavement life for a given loading condition. Of particular importance in many cases is the determination of pavement damage that would result if no load restrictions are applied. In this Chapter, results of field studies using FWD measurements on both the Haines and Steese/Elliot highways were used to assess potential pavement damage associated with truck loading during spring-thaw. The field studies also included monitoring rutting and roughness of the Steese/Elliot highway during spring/summer of 1994 and 1995 when load restrictions were lifted and truck traffic using 100 percent legal axle weights was allowed. The following objectives were addressed:

1. Determine the extent of pavement weakening during spring-thaw in relation to applied wheel loads.
2. Identify the weak pavement sections along the Steese/Elliot and Haines highways and design alternative sections that will resist the applied loads with no load restrictions.

3. Determine the extent of measured pavement distress on the Steese/Elliot highway associated with lifting the load restrictions during spring/summer 1994 and 1995.
4. Determine whether FWD surface deflections can be used as criteria for load restrictions.
5. Determine the influence of thaw depth and surface temperature on loss of pavement support.

3.2 FIELD CONDITIONS AND FWD TESTING

FWD tests were conducted during spring/summer of 1993 at one week intervals covering essentially the critical spring-thaw weakening period for both the Steese/Elliot (Station 9.1 to Station 37.3, in CDS miles) and the Haines highways (Location 1 to 436, starting at the ferry terminal and proceeding north). These tests were performed at 0.2 mile intervals along the Steese/Elliot using 9 and 14 kip loads and at 0.1 mile intervals along the Haines highway using 7, 9, and 14 kip loads. Additional FWD tests were also performed at selected locations on the Steese/Elliot highway during spring of 1994 for 5, 7, 9. and 14 kip loads. Tables 3.1 and 3.2 summarize the dates of FWD tests. Pavement thickness data are presented in Tables 3.3 and 3.4.

Table 3.1 FWD Testing Dates for the Steese/Elliot and Haines Highways (Spring/Summer 1993)

Steese/Elliot	Haines
April 12, 1993	March 21, 1993
April 19, 1993	March 28, 1993
April 26, 1993	April 6, 1993
May 3, 1993	April 13, 1993
May 10, 1993	April 20, 1993
May 17, 1993	April 27, 1993
September 27, 1993*	August 16, 1993*

* FWD tests conducted only for 9 kip load (i.e. Summer "reference" condition)

Table 3.2 FWD Testing Dates for the Steese/Elliot Highway (Spring 1994)

Testing Dates
April 21, 1994
April 27, 1994
May 4, 1994
May 10, 1994
May 17, 1994
May 24, 1994

Table 3.3 Pavement Thickness for Steese/Elliot Highway

Station (CDS Miles)	Thickness of Asphalt Concrete (inches)	Thickness of Base (inches)
9.1 - 11.1	3	10.5
11.1 - 17.8	5.5	24
17.8 - 34.3	6	24
34.4 - 36.2	8	24
36.2 - 37.7	6	24

Table 3.4 Pavement Thickness for the Haines Highway

Location	Thickness of Asphalt Concrete Layer (inches)	Thickness of Base (inches)
1 - 50	4	12
50 - 36	3	12

3.3 THAW WEAKENING AND PAVEMENT LIFE PREDICTION

The loss of pavement strength during spring-thaw is caused mainly by the increase of unfrozen moisture in the base as a result of pavement thawing (Taber 1929; Preus and Tomes 1959; Dysili 1982). This is reflected in the reduction of base and subgrade moduli and the corresponding increase in pavement deflections, stresses and strains (Stubstad and Connor 1982; Coetzee and Connor 1994). Increased moisture results in excess pore pressures under moving loads that reduce the effective confining pressure and cause loss of shear strength in addition to pumping and channeling (Dempsey 1982; Raad 1982). The corresponding loss of support under the asphalt concrete layer causes increased fatigue cracking and potholing. These conditions are much more significant in dense-graded bases than in open-graded bases (Raad et al. 1992). Dense graded bases, which have more fines and are less permeable, tend to retain pavement moisture for longer periods. Open-graded bases, on the other hand, have better drainage properties and are less susceptible to pore pressure generation under moving loads.

The presence of a stiff layer of frozen material underlying the thawing front will result in excessive vertical strains and, possibly, stresses on top of the "weakened" thawed zone (Stubstad and Connor 1982), and tensile strains in the asphalt concrete layer (Rutherford 1988; Coetzee and Connor 1994). Moreover, the loss of asphalt layer stiffness associated with the gradual increase in average pavement surface temperature will increase the stresses transmitted to the "weakened" base thereby causing additional pavement damage.

These thaw-weakening mechanisms were identified in the analyses performed by using appropriate limiting criteria and pavement response parameters as determined by backcalculation. In this case, the layer moduli for the asphalt concrete surface, the unbound base, and subgrade were backcalculated using the computer program ELMOD (Ullidtz and Stubstad 1985; Ullidtz 1987). ELMOD was also used to determine the critical tensile strain in the asphalt surface and vertical stress in the granular base. Of particular interest is the location of the depth to the stiff layer during pavement thawing. ElMOD determines an “equivalent” depth using Odemark’s transformation (Ullidtz 1987). FWD applied loads were normalized to 5, 7, 9, and 14 kips for the Steese/Elliot and 7, 9, and 14 kips for the Haines, as applicable. Contact surface area was adjusted to maintain a contact surface pressure of 110 psi for all the cases considered. The pavement remaining life repetitions were estimated from appropriate limiting criteria for the asphalt surface and the granular base according to Equations 2.11 and 2.15, respectively. The remaining life repetitions were also used to determine damage factors, or damage ratios, from Equation 2.7 which defines relative damage with respect to a standard wheel load and pavement condition. For a given testing period, relative damage was determined in terms of equivalent 9 kip applications. In other words, the "damaging" effect of a given wheel load application was expressed in terms of the number of 9 kip applications required to induce equivalent pavement damage. In addition, relative damage was also determined for spring-thaw condition in comparison with "dry" reference condition during summer.

3.4 RESULTS OF FWD ANALYSES

FWD backcalculation analyses were performed and results were presented to illustrate the following (Appendices A and B):

1. FWD surface deflections.
2. Pavement surface temperature.
3. Remaining life repetitions.
4. Pavement damage relative to 9 kip loading for a given testing period.
5. Pavement damage relative to 9 kip loading during "dry" summer reference period.
6. Backcalculated thaw depth.

3.4.1 Remaining Life and Damage Factors

The resulting damage to pavements during spring-thaw could be assessed by determining the remaining life of the pavement structure associated with excessive damage to its most critical component and also, by evaluating the relative damage caused by a given load during spring in comparison with damage resulting from the application of a standard 9 kip load for a summer reference condition.

Remaining life analyses for both the Steese/Elliot and the Haines highways indicate that for most loading cases and field testing conditions, damage to the granular base layer was most critical. The remaining life associated with damage to the base was shorter in general than the fatigue life of the asphalt concrete surface. In other words, loss of stiffness and strength of the granular base under repeated pavement loads will occur first and will eventually cause accelerated fatigue and rutting of the asphalt surface. The variation of remaining life for both the Steese/Elliot and the Haines highway is shown for different periods of spring testing using a standard 9 kip load (Figures 3.1 and 3.2). Results indicate a significant increase in remaining life as thaw progresses and the pavement becomes drier. The variability in remaining life of the Steese/Elliot is much more pronounced than that of the Haines highway. For example, the remaining life under

9 kip loading condition for the Steese/Elliot highway could increase from about 1,000 repetitions under spring-thaw weakening conditions to about 10^7 repetitions for the relatively "dry" summer reference condition as shown for pavement sections between Stations 15 and 17 (Figure 3.1). The Haines highway exhibits, in general, longer remaining life during spring thaw with much less variability than the Steese/Elliot (Figure 3.2).

Results of the analyses also indicate that excessive values of damage factors could occur during the thawing period (Figures 3.3 - 3.8). For example, pavement damage factors for the 9 kip load application during spring-thaw relative to the 9 kip application in the "dry" summer condition could reach values as high as 400 for the Steese/Elliot and 50 for the Haines highways. These values decrease gradually with the downward progression of the thawing front and drainage of the pavement section. As expected, reducing pavement loads will result in a decrease of damage factors (Figures 3.5 and 3.6). For example, a decrease in the applied load from 9 kips to 7 kips will reduce the damage factor by approximately one-half. For many sections on the Steese/ Elliot and the Haines highways, this decrease will still result in damage factors that are significantly larger than one. Reducing the damage factor to a value of 1 for these sections could, therefore, result in unreasonable load restriction policy since the resulting restricted spring load value could be very small.

A typical variation of average remaining life during spring thaw with damage factor for the 9 kip load data is illustrated in Figures 3.9 and 3.10. As expected, the remaining life decreases as the damage factor increases. These relationships also show larger values

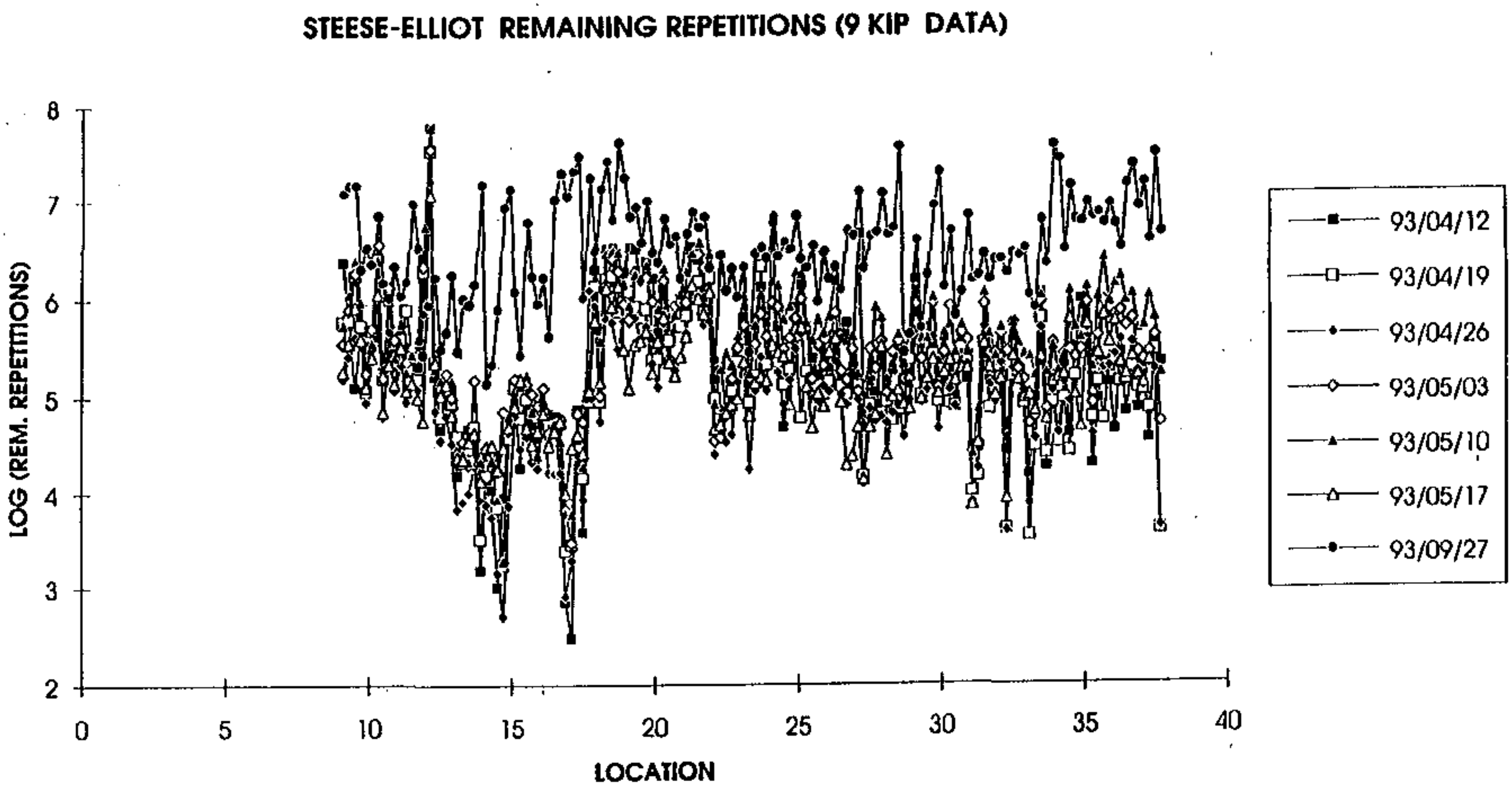

Figure 3.1 Variation of Remaining Repetitions along the Steese/Elliot Highway

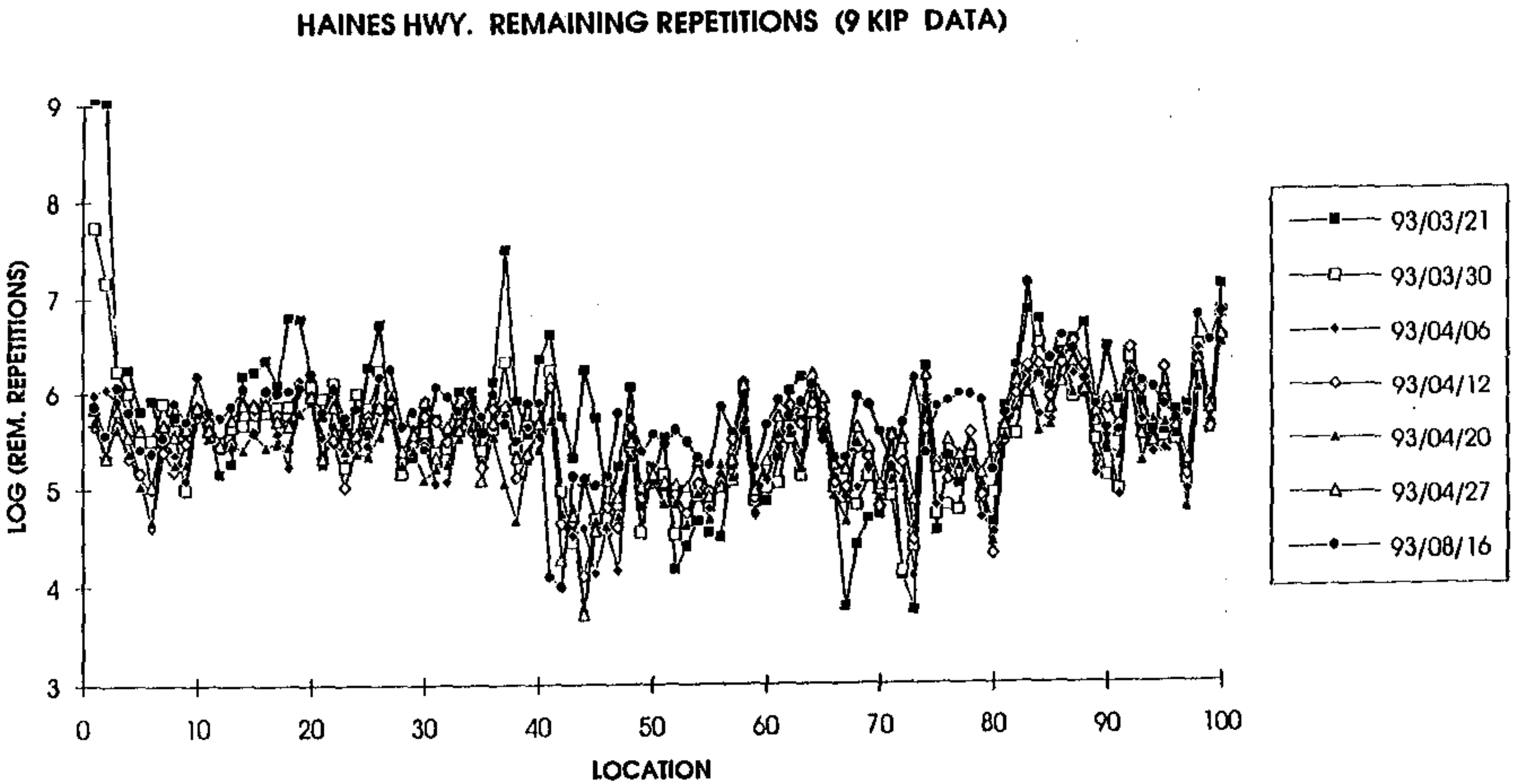

Figure 3.2 Variation of Remaining Repetions along the Haines Highway

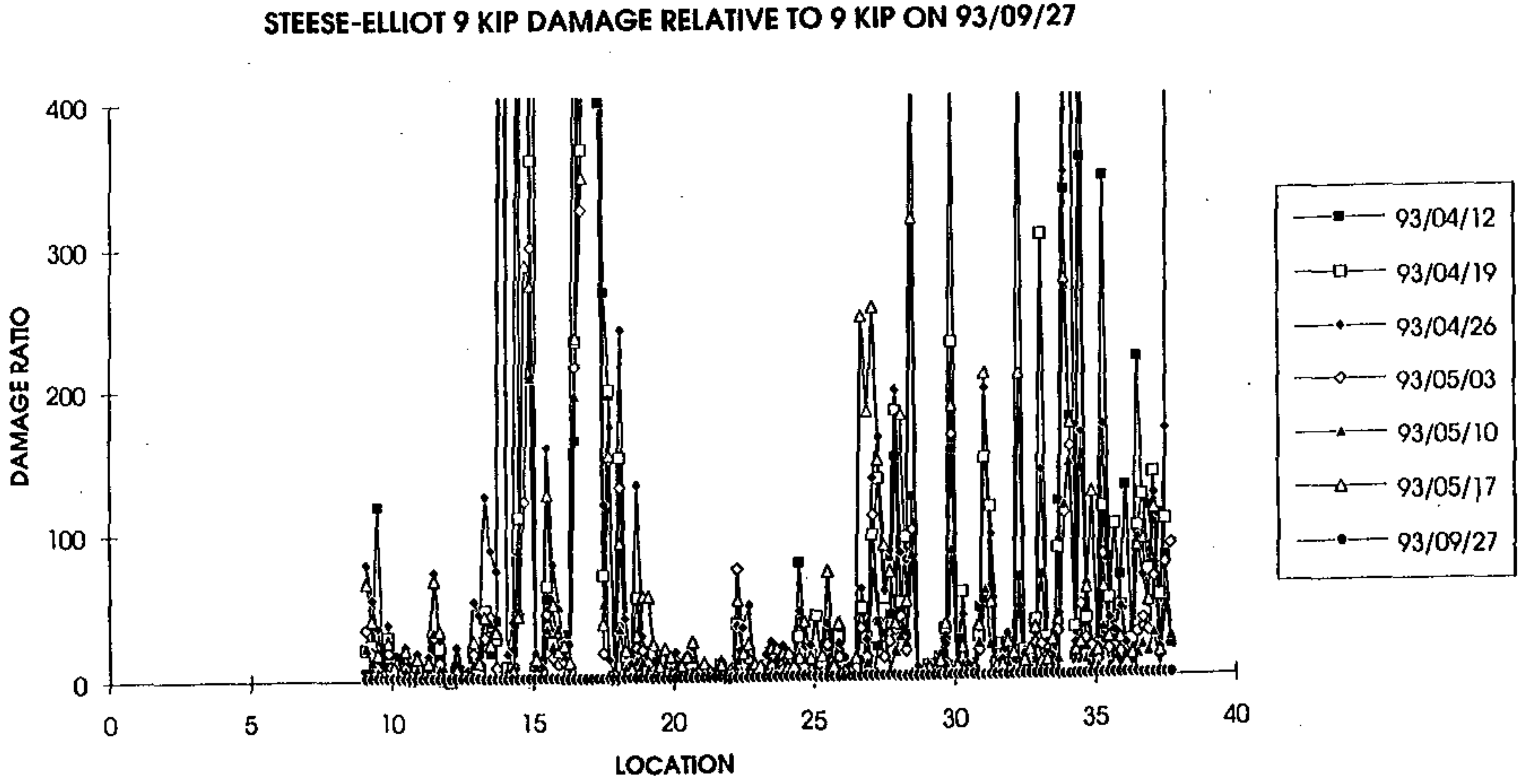

Figure3.3 Variation of 9 kip Damage Relative to 9 kip Reference Summer Condition for the Steese/Elliot Highway

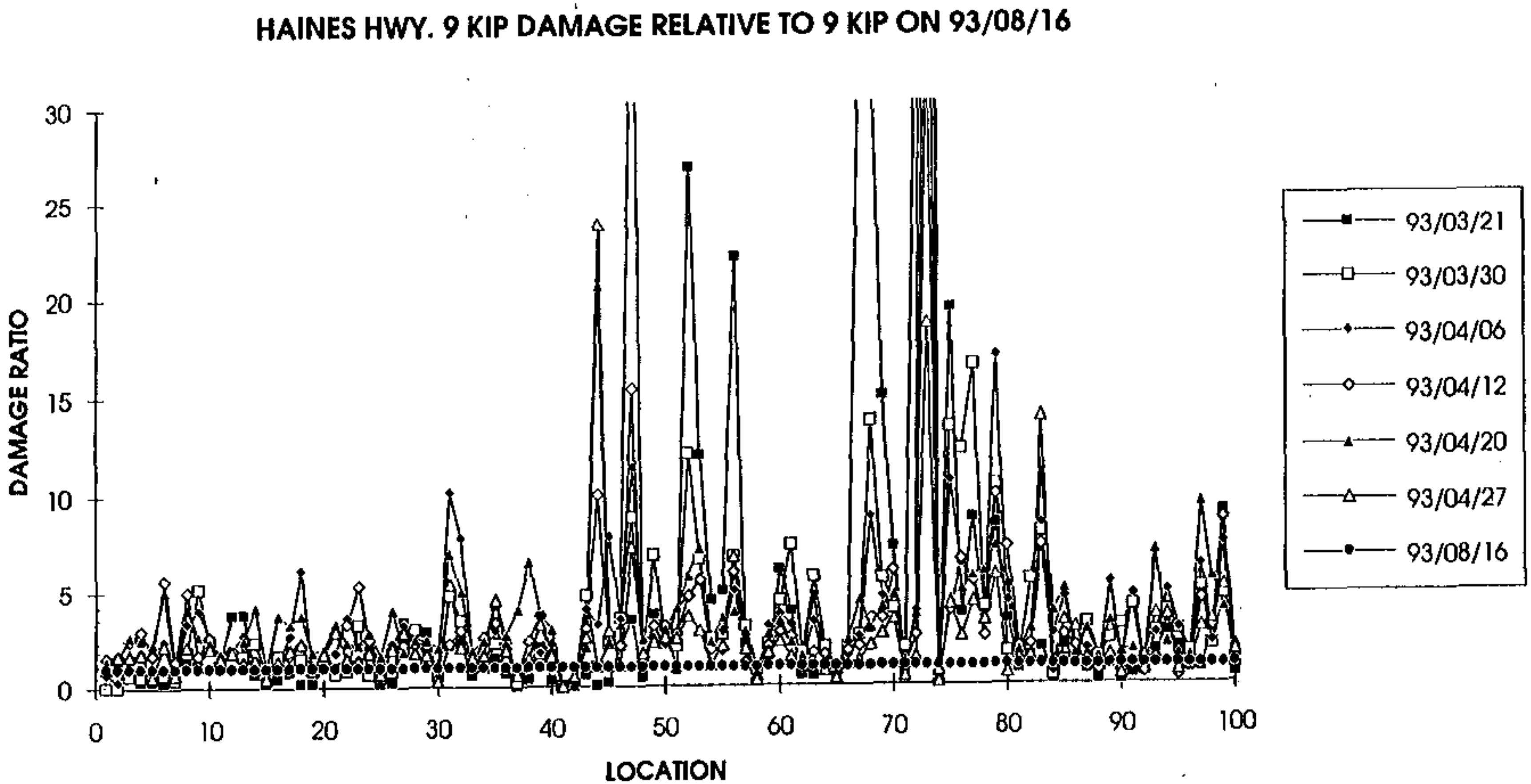

Figure 3.4 Variation of 9 kip Damage Relative to 9 kip Reference Summer Condition for the Haines Highway

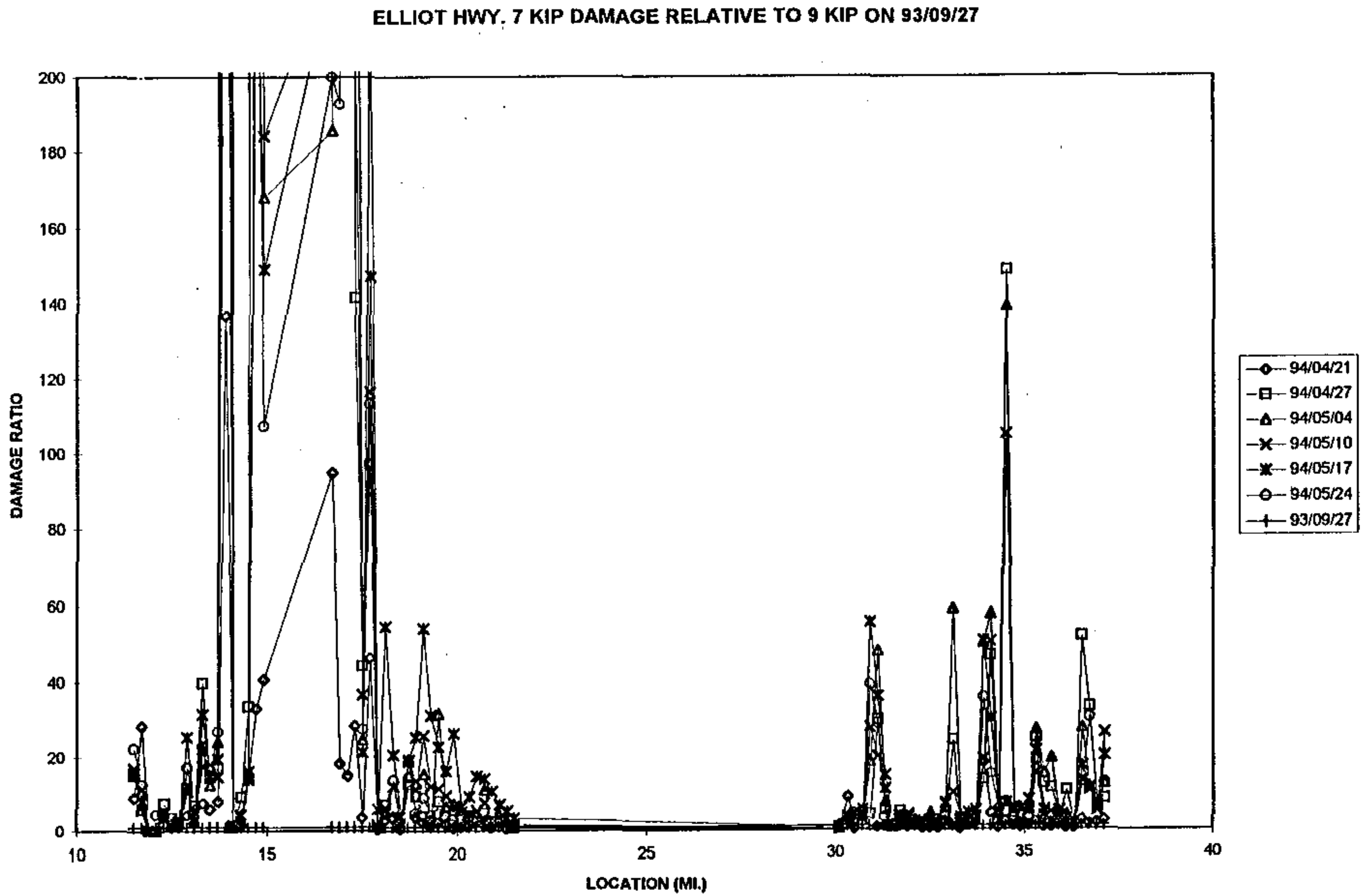

Figure 3.5 Variation of 7 kip Damage Relative to 9 kip Reference Summer

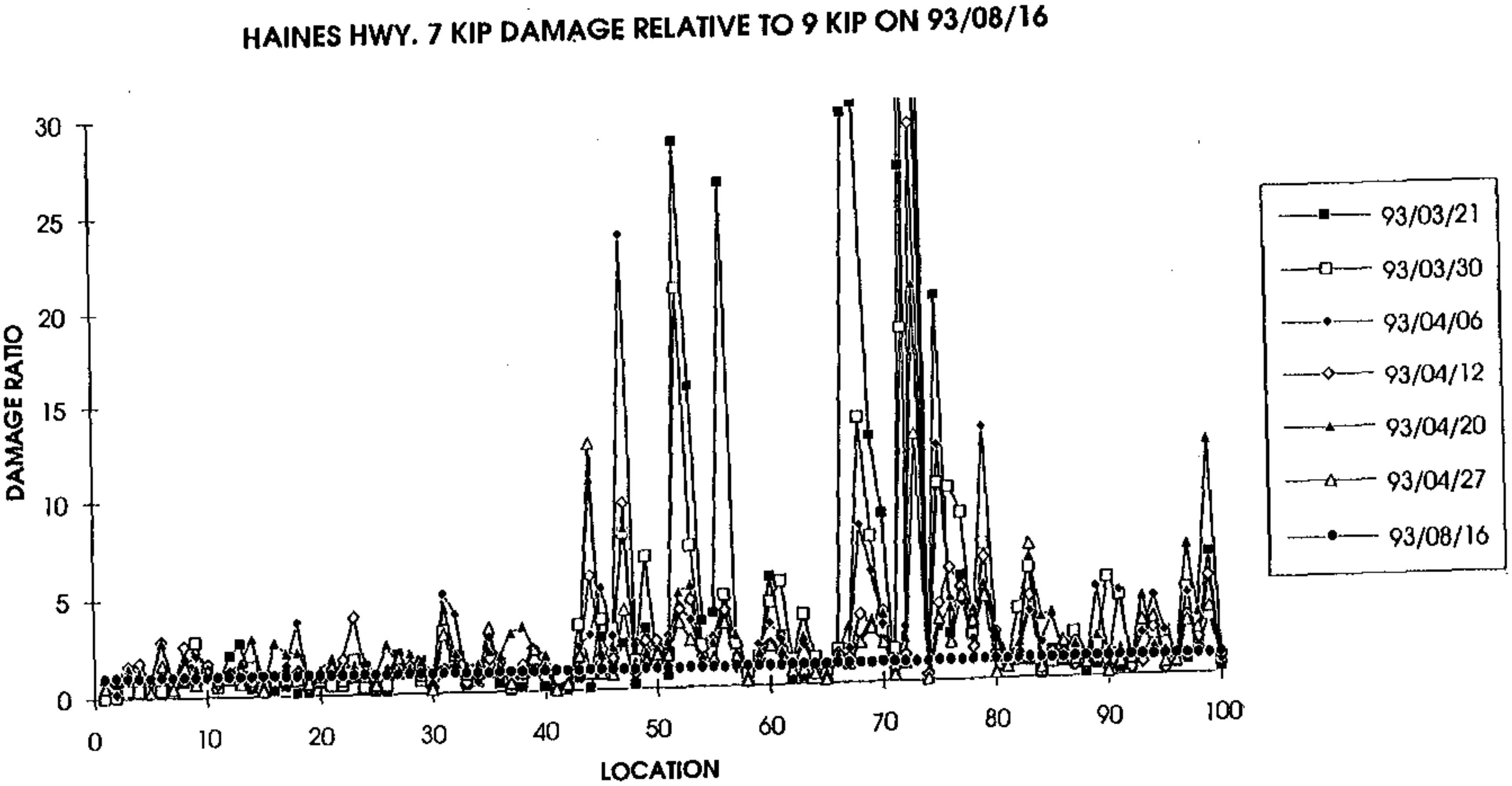

Figure 3.6 Variation of 7 kip Damage Relative to 9 kip Reference Summer Condition for the Haines Highway

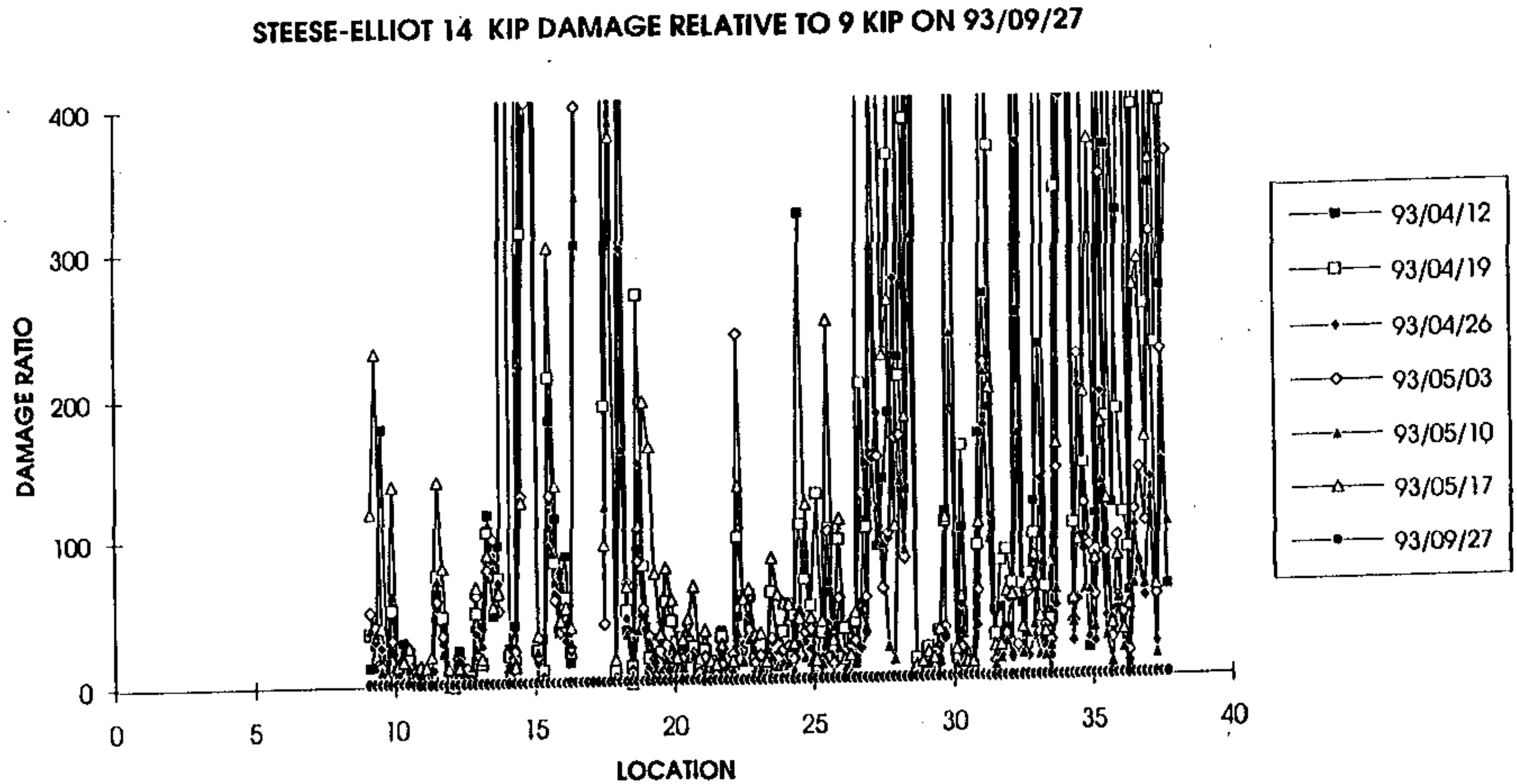

Figure 3.7 Variation of 14 kip Damage Relative to 9 kip Reference Summer Condition for the Steese/Elliot Highway

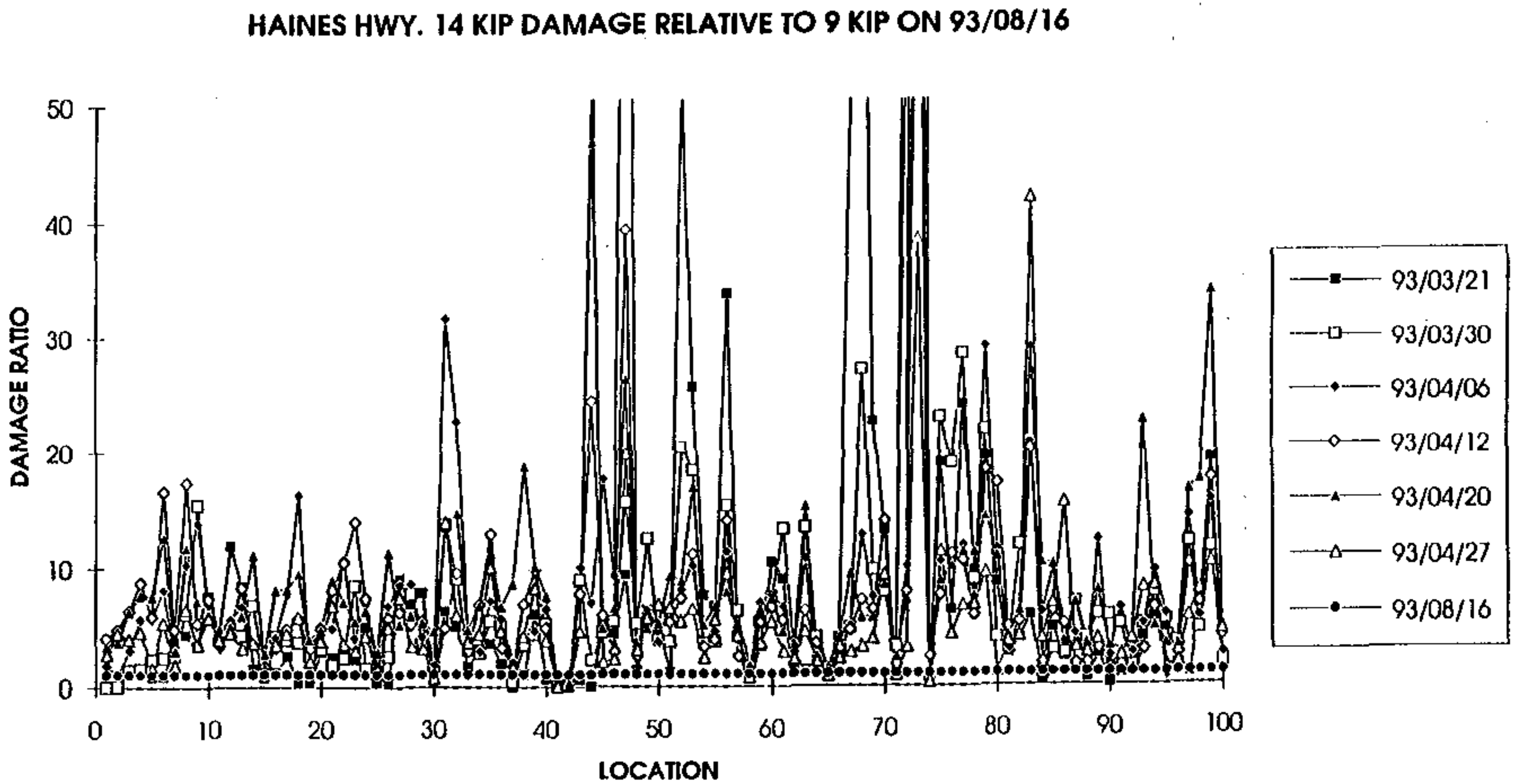

Figure 3.8 Variation of 14 kip Damage Relative to 9 kip Reference Summer Condition for the Haines Highway

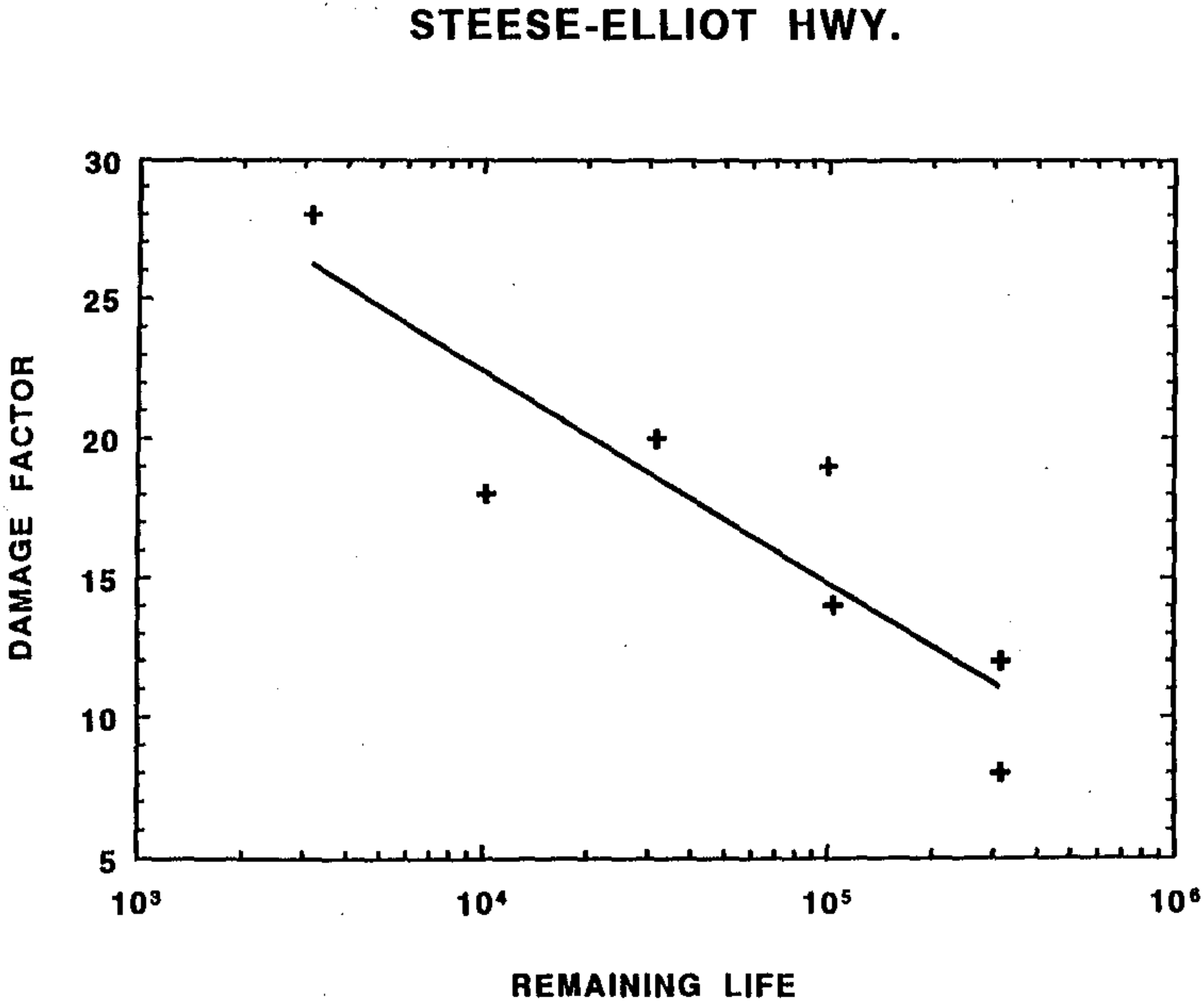

Figure 3.9 Influence of Damage Factor on Remaining Life Repetitions for the Steese/Elliot Highway

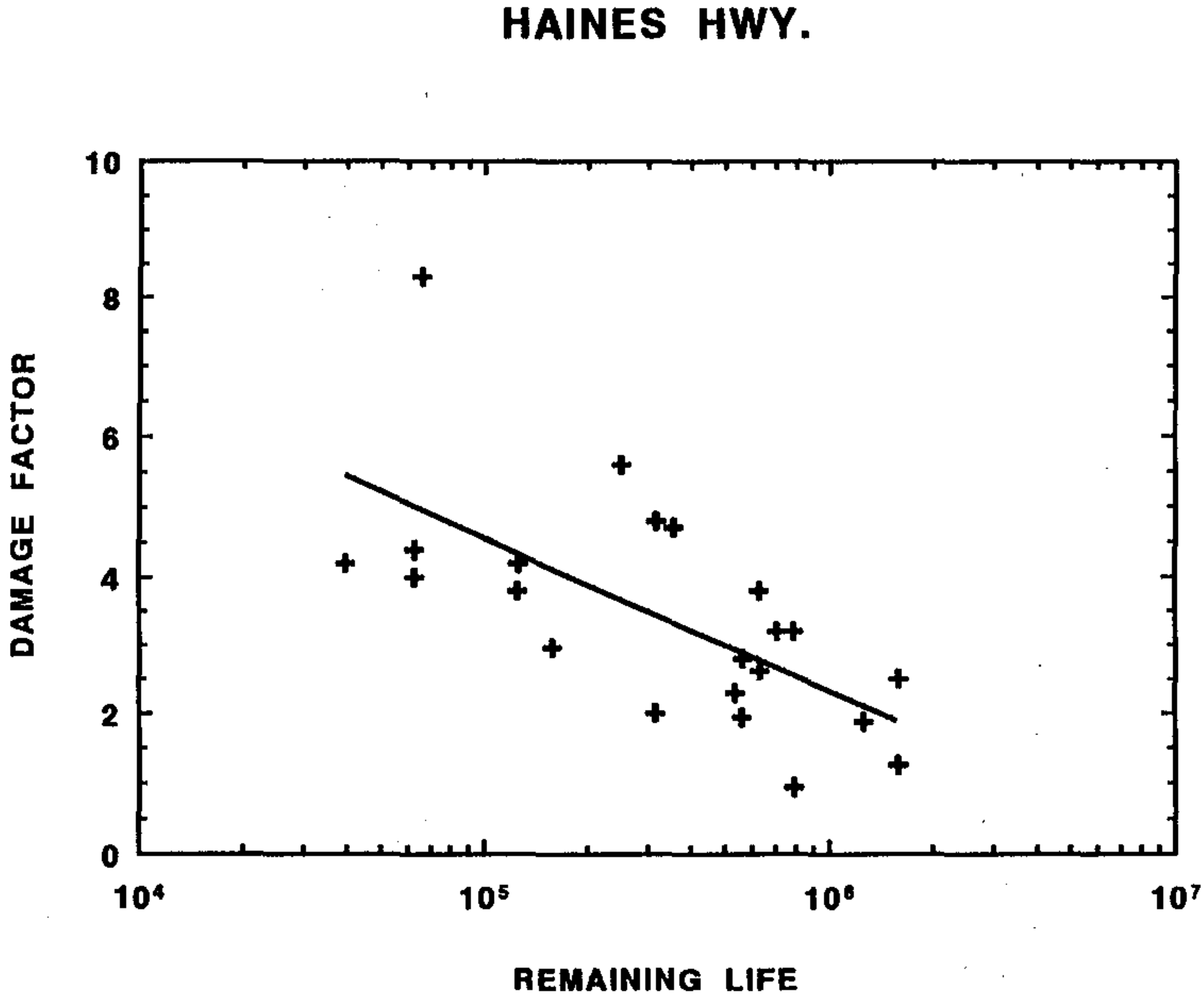

Figure 3.10 Influence of Damage Factor with Remaining Life Repetitions for the Haines Highway

of damage factors for the Steese/Elliot in comparison with the Haines, thereby indicating more susceptibility to thaw weakening relative to the "reference" summer condition.

3.4.2 Identification of Weak Pavement Sections

One of the main objectives of this study was the determination of the extent of pavement weakening during-spring thaw and identification of the corresponding weak pavement sections along the Steese/Elliot and Haines highways. Of particular interest in this case is the reduction of service life that could result from a "no load restriction" policy. The variation in remaining life, as illustrated in Figures 3.1 and 3.2, could result mainly from variations in section properties, pavement surface temperature, and thaw depth. These factors were considered in the analysis for identifying weak pavement sections. Specifically, the following steps were used:

1. Divide the highway into segments that exhibit essentially "similar" section properties, thaw depth, and surface temperature, as judged from the variation of pavement remaining life along the length of the highway.
2. Determine an "average" remaining life during the spring thaw testing period for each highway segment. This will identify an overall "average" predicted performance but does not necessarily reflect the local variations associated with the "weakest" or most critical sections within each segment.
3. Determine a "critical" remaining life by considering the 90th percentile ranking of remaining life repetitions during spring thaw within each segment. This corresponds to the remaining life value below which the "worst" 10 percent sections would perform.

4. Estimate "average" and "critical" years of service life from backcalculated remaining life repetitions. In this case, design life and traffic data were provided by the Alaska Department of Transportation and Public Facilities (AKDOT&PF). Existing pavement sections had been designed to last an average of 10 years during which the total number of 18 kip equivalent axle load (EAL) repetitions were expected to be about half a million. Average yearly EAL applications corresponding to different pavement conditions are summarized in Table 3.5. Miner's cumulative damage hypothesis (Miner 1945) was used to estimate the years of service for each segment from EAL data and remaining life repetitions for the different pavement conditions. In this case, both "average" and "critical" remaining life repetitions determined from spring-thaw FWD data were used for the "weak" spring-thaw period in Table 3.5. The analyses also assumed that the cumulative damage that occurs during "stiff" pavement conditions in Fall and Winter is negligible when compared to damage that occurs during the rest of the year. An average pavement remaining life of 10^6 repetitions was used for late spring and summer conditions. A summary of results for both "average" and "critical" remaining lives is presented in Tables 3.6 - 3.9.

Table 3.5 Pavement Conditions and Corresponding Design EAL Repetitions

Pavement Condition	Duration	EAL Repetitions/Year
Weak (Spring-Thaw)	2 months	9,250
Moderate (Late Spring / Summer)	4 months	18,250
Stiff (Fall / Winter)	6 months	22,500

Table 3.6 "Average" Remaining Life of Steese/Elliot Highway (9 kip Load)

Station (CDS Miles)	Spring-Thaw Remaining Life (Repetitions)	Remaining Service Life (Years)
9.1 - 11.5	3.16×10^5	21
11.5 - 14.9	1.02×10^4	1.06
14.9 - 16.7	3.18×10^4	3.2
16.7 - 17.5	3.16×10^3	0.33
17.5 - 21.5	3.16×10^5	21
21.5 - 30.1	1.04×10^5	9.4
30.1 - 37.1	1.00×10^5	9.0

Table 3.7 "Average" Remaining Life for the Haines Highway (9 kip Load)

Location (0.1 Miles From Ferry Terminal)	Spring-Thaw Remaining Life (Repetitions)	Remaining Service Life (Years)
0 - 40	5.37×10^5	28
40 - 55	6.30×10^4	6.1
55 - 65	2.51×10^5	18
65 - 80	1.26×10^5	11
80 - 90	9.54×10^5	36
90 - 100	3.55×10^5	23
100 - 125	1.58×10^6	41
125 - 140	5.62×10^5	28
140 - 150	7.9×10^5	33
150 - 160	6.3×10^5	30
160 - 165	1.58×10^6	41
165 - 250	3.16×10^5	21
250 - 270	7.08×10^5	32
270 - 285	1.26×10^6	39
285 - 300	5.62×10^5	29
300 - 320	2.51×10^5	18
320 - 330	3.16×10^5	21
330 - 340	6.31×10^4	6.1
340 - 370	7.9×10^5	33
370 - 400	6.6×10^4	6.3
400 - 420	1.25×10^5	11
420 - 436	3.98×10^4	4.0

Table 3.8 "Critical" Remaining Life and Corresponding FWD Center Deflections and Thaw Depth for the Steese/Elliot Highway (9 kip Load)

Station (CDS Miles)	Spring-Thaw Remaining Life (Repetitions)	Remaining Service Life (Years)	FWD Center Deflections (10^{-3} in/in)	Backcalculated Thaw Depth (inches)
9.1 - 11.5	1.25 x 105	11	18	66
11.5 - 14.9	1.58 x 103	0.17	37	34
14.9 - 16.7	1.27 x 104	1.3	24	30
16.7 - 17.5	5.01 x 102	0.05	32	29
17.5 - 21.5	1.25 x 105	11	17	70
21.5 - 30.1	3.98 x 104	3.9	23	50

Table 3.9 "Critical" Remaining Life and Corresponding FWD Center Deflections and Thaw Depth for the Haines Highway (9 kip Load)

Location (0.1 Miles From Ferry Terminal)	Spring-Thaw Remaining Life (Repetitions)	Remaining Service Life (Years)	FWD Center Deflections (10^{-3} in/in)	Backcalculated Thaw Depth (inches)
0 - 40	1.26×10^5	11	24	34
40 - 55	7.94×10^3	0.85	29	30
55 - 65	1.62×10^5	13	23	39
65 - 80	1.58×10^4	1.7	22	31
80 - 90	2.19×10^5	16	13	103
90 - 100	1.00×10^5	9.0	15	49
100 - 125	1.58×10^5	13	14	50
125 - 140	1.00×10^5	9.0	16	64
140 - 150	3.31×10^5	21	15	60
150 - 160	7.94×10^4	7.4	15	57
160 - 165	3.16×10^5	21	13	42
165 - 250	1.26×10^5	10	14	54
250 - 270	1.00×10^5	9.0	13	36
270 - 285	2.51 x 105	18	12	47
285 - 300	1.58×10^4	1.7	18	34
300 - 320	1.58×10^4	1.7	22	32
320 - 330	5.01×10^4	4.9	14	34
330 - 340	5.01×10^3	0.54	26	35
340 - 370	1.58×10^4	1.7	23	32
370 - 400	5.01×10^3	0.54	27	35
400 - 420	1.58×10^4	1.7	23	33
420 - 436	1.00×10^4	1.0	25	30

"Average" Remaining Service Life

The "average" remaining service life provides the design engineer with an estimate of the overall all performance of the pavement if spring load restrictions are not applied. If the "average" remaining service life is greater than the design life, then probably no strengthening requirements will be necessary. If, on the other hand, the "average" remaining service life is smaller than the design life, then strengthening of weak pavement sections may be required. Prioritization and selection of the weak pavement sections depend on maintenance management strategies selected by AKDOT&PF. In this case, "critical" remaining life data would provide more information about the performance of the most critical sections within each highway segment thereby facilitating the selection of appropriate maintenance strategies and methods by maintenance engineers.

Results of analyses indicate a significant variation in "average" remaining service life along the Steese/Elliot highway (Table 3.6). Predicted values could be larger or smaller than the 10 year design life and range from less than one year to about 20 years. If no load restrictions are applied, most of the predicted damage will be concentrated between Stations 11.5 and 17.5. The corresponding "average" remaining service life ranges from 0.3 to 1 year. The Haines highway, on the other hand, exhibits larger "average" remaining life values ranging from 4 to 40 years (Table 3.7). Highway segments with remaining life values smaller than the 10 year design life include locations 40 - 55, 330 - 340, 370 -400, and 420 - 436. The remaining life at these locations varies between 4 and 6 years.

"Critical" Remaining Service Life

The "critical" remaining service life provides additional information about localized damage that could occur within a given highway segment. A summary of predicted "critical" life values is illustrated for the Steese/Elliot and Haines highways in Tables 3.8 and 3.9 respectively. These correspond to the remaining life value below which the "worst" 10 percent sections would perform. As expected, "critical" remaining service life may reach values much lower than the design value of the pavement structure. This is observed for some cases where the "average" remaining service life is much greater than the design life. For example, the segment of the Haines highway between locations 285 and 300 has an "average" remaining life equal to 29 years whereas its "critical" life is 1.7 years. This means that although the average remaining life is about 3 times the design life, 10 percent of the highway segment will have a service life of 1.7 years or less and will therefore perform below design standards.

Both the "average" and "critical" remaining lives could be used to establish criteria for selecting pavement sections that need strengthening if no load restrictions are imposed. For example, highway segments that exhibit a 15 percent or more reduction in "average" remaining life in comparison with the anticipated design life are candidates for strengthening if no restrictions are applied. Strengthening *should be* applied if the "critical" remaining life is less than 30 percent of the design life and *need to be* applied if the "critical" remaining life is greater. If these criteria are used to identify "weak" pavements of the Steese/Elliot and Haines highways, and if it is decided to strengthen sections that exhibit a remaining life less or equal to 3 years, then the estimated mileage required to be maintained or rehabilitated for the Steese/Elliot and the Haines highways will be approximately 3 miles (i.e. 11 percent) and 2 miles (i.e. 5 percent), respectively. It should be emphasized that these results are tentative and based on assumed criteria using backcalculated FWD data. These results are strongly influenced by the following:

1. Limiting criterion for base failure. If the stress limiting criterion used to define base failure is conservative (i.e. predicted remaining life repetitions is small) then the analyses will predict more highway segments that are "weak" and need strengthening.
2. Traffic analysis. If actual EALs are smaller than the design values used in the analyses, then the predicted remaining life will be less than actually experienced by the pavement.
3. Criteria for selecting candidate "weak" sections that need strengthening. The proposed criteria need to be verified in relation to maintenance requirements, costs and field performance.

3.5 STRENGTHENING WEAK PAVEMENT SECTIONS

An attempt is made to design the "weak" pavement sections so that the service life remains unaffected if no load restrictions are applied during spring-thaw. Pavement layer properties were estimated from FWD backcalculated moduli. Critical thawing conditions were assumed and the corresponding design thaw depth did not exceed 20 inches for the design period. The design periods and traffic data used are summarized in Table 3.5. Average thaw depth values were assumed to be 10 inches and 20 inches for the spring-thaw and late spring/summer periods, respectively. The pavement section was designed for 0.5 x 10^6 EALs which corresponds to a 10 year service life for the given average traffic conditions presented in Table 3.5. Limiting criteria for the asphalt concrete surface and base/subbase are defined by Equations 2.11 and 2.15, respectively. Proposed section thickness are as follows:

4 inches hot-mix asphalt surface

6 inches open-graded base, preferably bituminous-treated

24 inches non-frost susceptible granular subbase

The proposed pavement should be properly drained and should include materials with adequate strength and durability to resist applied traffic and extreme climatic conditions.

Table 3.10 Material Properties and Pavement Thickness Requirements for Weak Sections

Layer	AC Surface (Spring-Thaw)	Base (Spring-Thaw	Subbase (Spring Thaw)	AC Surface (Spring/ Summer)	Base (Spring/ Summer)	Subbase (Spring/ Summer)
Thickness (inches)	4	6	24	4	6	30
Elastic Modulus (psi)	1.5×10^6	45,000	15,000	5.0×10^6	45,000	25,000
Poisson's Ratio	0.20	0.30	0.35	0.30	0.30	0.35

3.6 SURFACE DEFLECTIONS AND PAVEMENT DAMAGE

Along the Steese/Elliot and Haines highways, the variation in surface deflections for the 9 kip load condition is illustrated in Figures 3.11 and 3.12. Deflections seem to increase gradually with progressive thawing in the pavement as indicated by the larger deflection readings for late spring in comparison with early spring testing periods. The most damaging period during spring-thaw, however, does not correlate in general with maximum deflections (Figures 3.3 - 3.4, and Figures 3.11 - 3.12). Pavement damage seems to increase when thawing starts in the granular base and decreases as the thawing front progresses in the pavement. This is observed by comparing Figures 3.3 - 3.4 with Figures 3.13 - 3.14. It is expected that the most damaging period during spring-thaw occurs when the thawing front starts penetrating the granular base. In this case, the base stresses are maximum, while the base is in its weakest state. However, although the stresses are high, the deflections need not be maximum. In fact, surface deflections are relatively low. As thawing proceeds in the base and eventually the subgrade, surface deflections become more relevant in assessing pavement damage. A summary of thaw depth and surface deflections illustrating such behavior for the most critical sections (90th percentile data) is presented in Tables 3.8 and 3.9.

The use of FWD center deflections to determine maximum pavement damage is therefore not recommended. However, the FWD surface deflection bowl can still be used to calculate critical stresses in the base and corresponding periods of maximum pavement damage. The FWD can also be used to determine the periods associated with complete pavement thaw and strength increase.

3.7 PAVEMENT SURFACE TEMPERATURE EFFECTS

Significant variations in pavement surface temperatures were observed for a given testing period, for both the Steese/Elliot and Haines highways (Figures 3.15 and 3.16). It is clear that the surface temperature varies depending on the location and time of the day and does not seem to correlate to thaw depth. This agrees with other findings indicating that a more representative parameter could be the accumulation of average daily air temperature above freezing (i.e. Thawing Index) (Rutherford et al. 1985). The Thawing

Index does not reflect the influence of local conditions on the rate of pavement thawing. The variation of backcalculated thaw depth for different locations and a given testing date (Figures 3.13 and 3.14) illustrates the significance of site location on thaw propagation.

3.8 STEESE/ELLIOT RUTTING AND ROUGHNESS

Load restrictions on the Steese/Elliot highway were lifted starting spring 1994 in order to assess pavement damage through field rutting and roughness measurements. Extensive field tests were conducted during 1994 and 1995 to monitor rutting and roughness and to determine how much damage would be induced if no load restrictions are applied. As part of this study, a field survey was conducted to estimate the location of pavements with unstable foundation problems resulting from permafrost thaw. Results of this survey are summarized in Table 3.11 and show that an estimated total of 4.6 miles experiences problems with unstable foundations.

Table 3.11 Locations of Pavement Sections with Unstable Foundations along the Steese/Elliot Highway

Starting Station (CDS Miles)	Ending Station (CDS Miles)
11.8	12.5
13.0	13.1
18.0	18.2
19.2	19.3
19.6	20.3
21.2	21.6
21.8	21.9
22.3	22.7
22.9	24.0
24.2	24.3
25.6	26.0
30.7	31.1

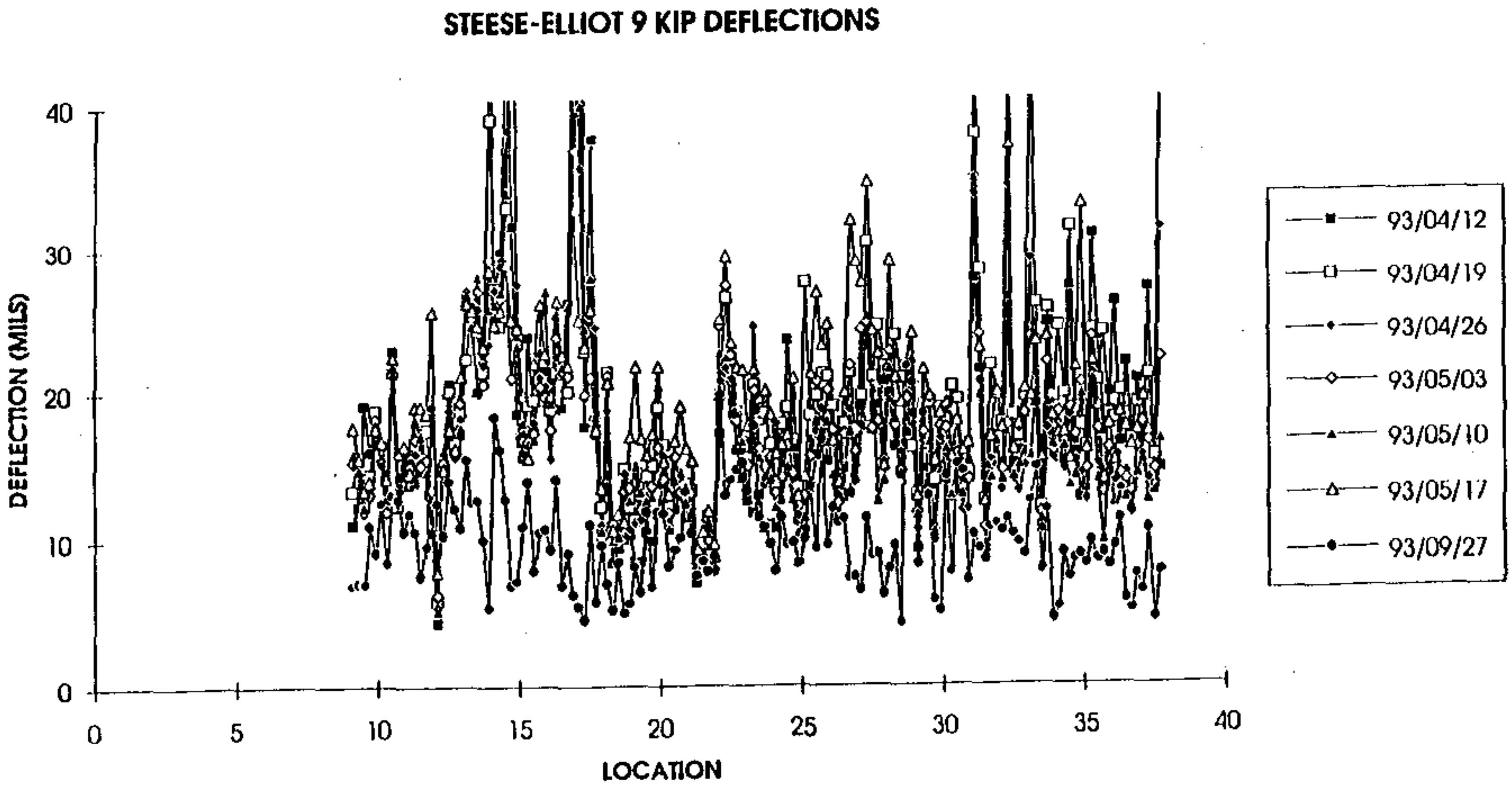

Figure 3.11 FWD Deflections for the Steese/Elliot Highway (9 kip Load)

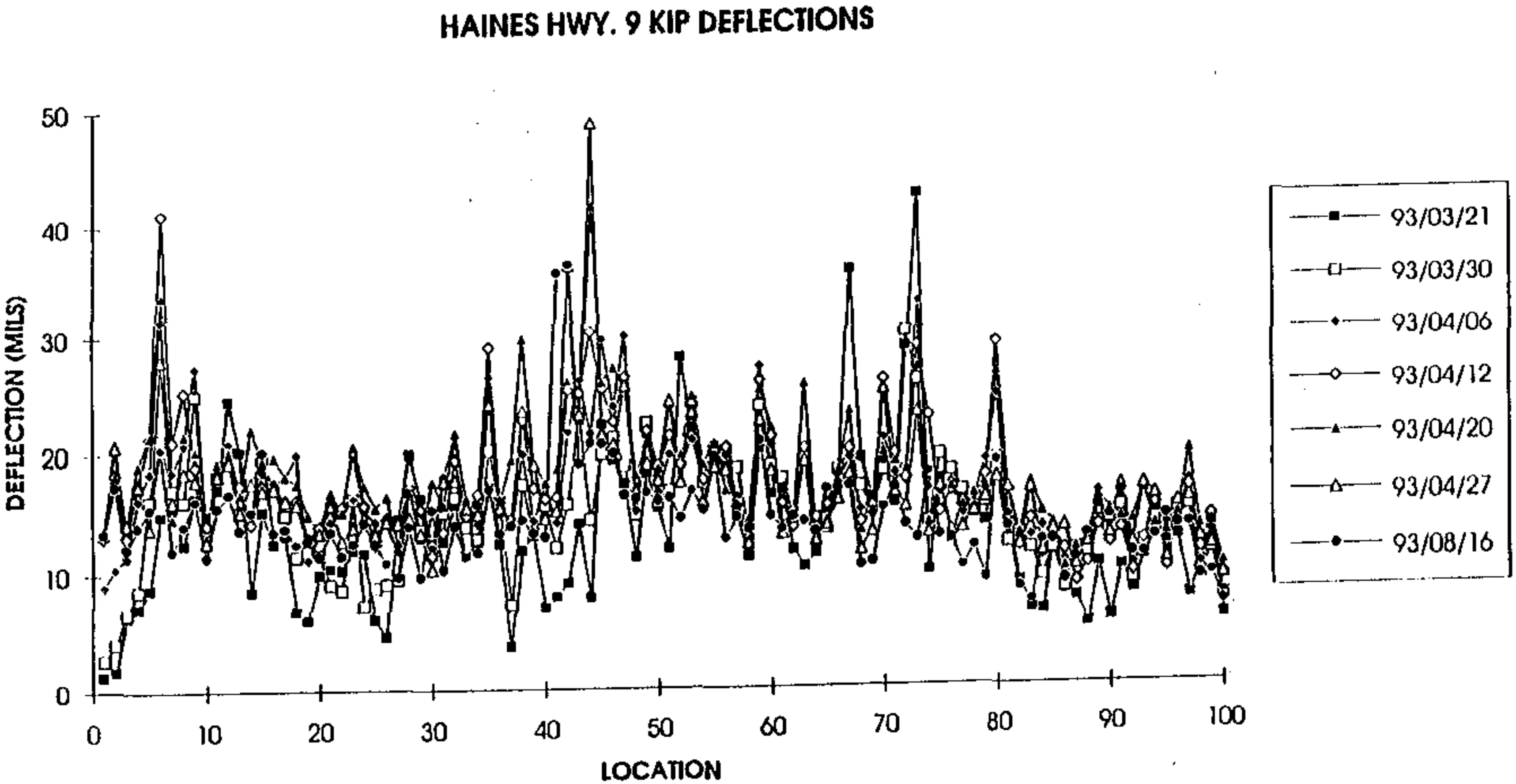

Figure 3.12 FWD Deflections for the Haines Highway (9 kip Load)

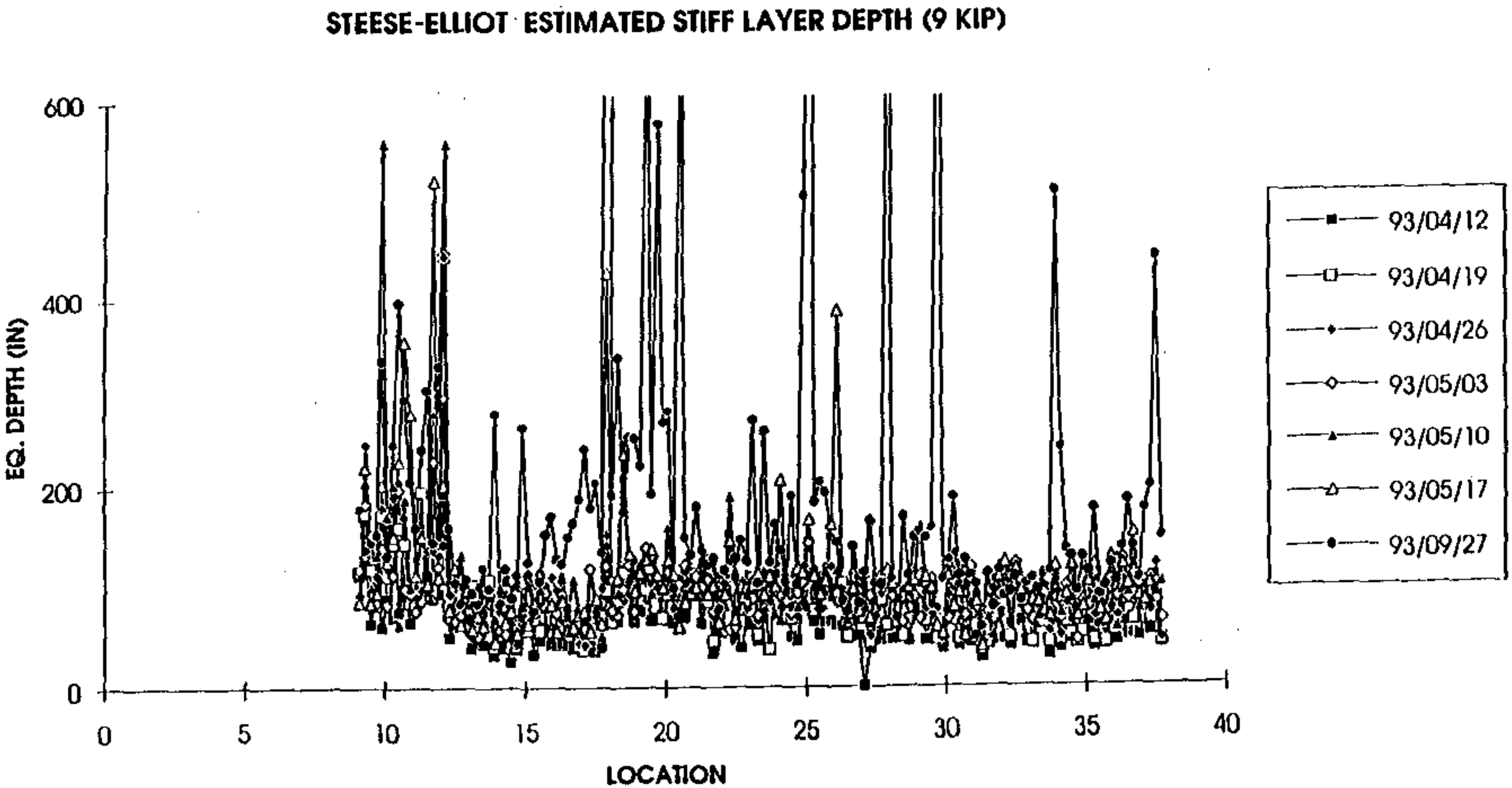

Figure 3.13 Variation of Backcalculated Thaw Depth for the Steese/Elliot Highway (9 kip Load)

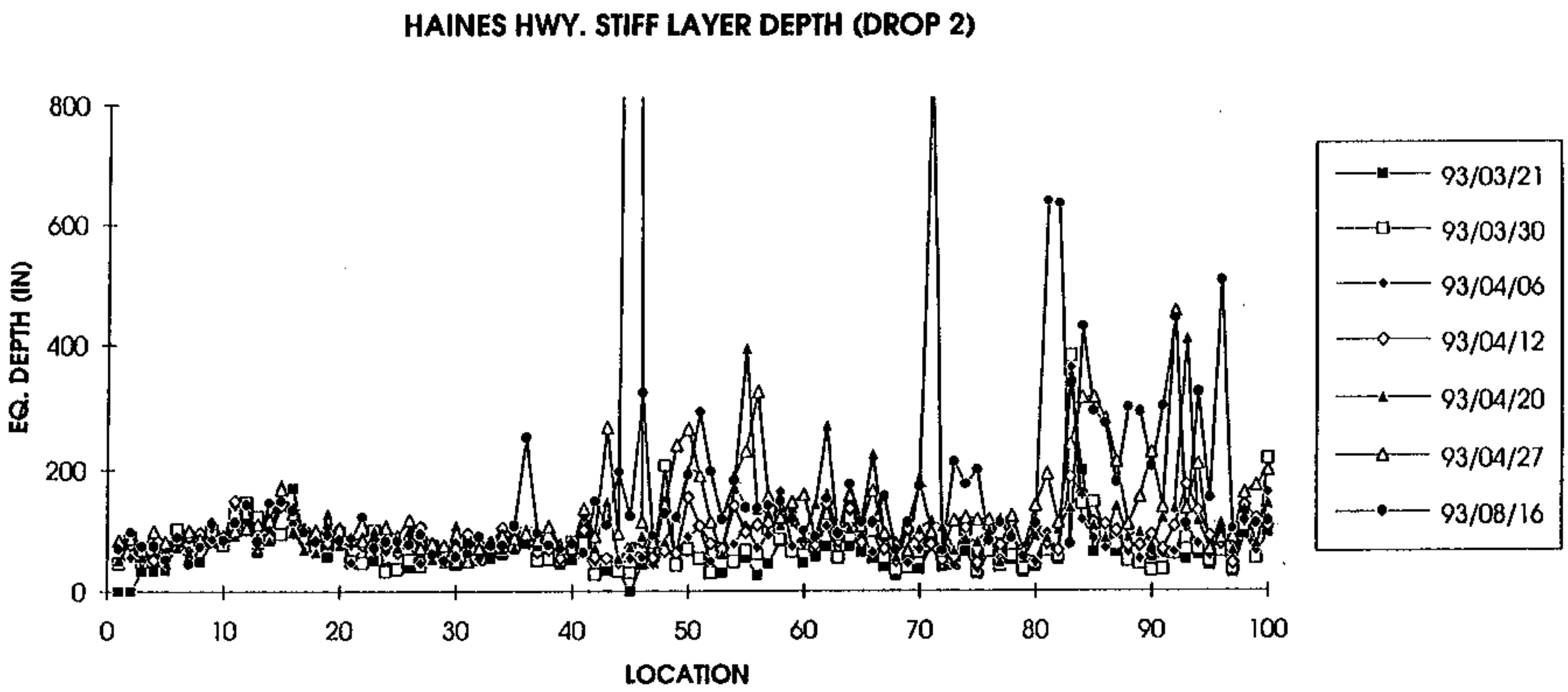

Figure 3.14 Variation of Backcalculated Thaw Depth for the Haines Highway (9 kip Load)

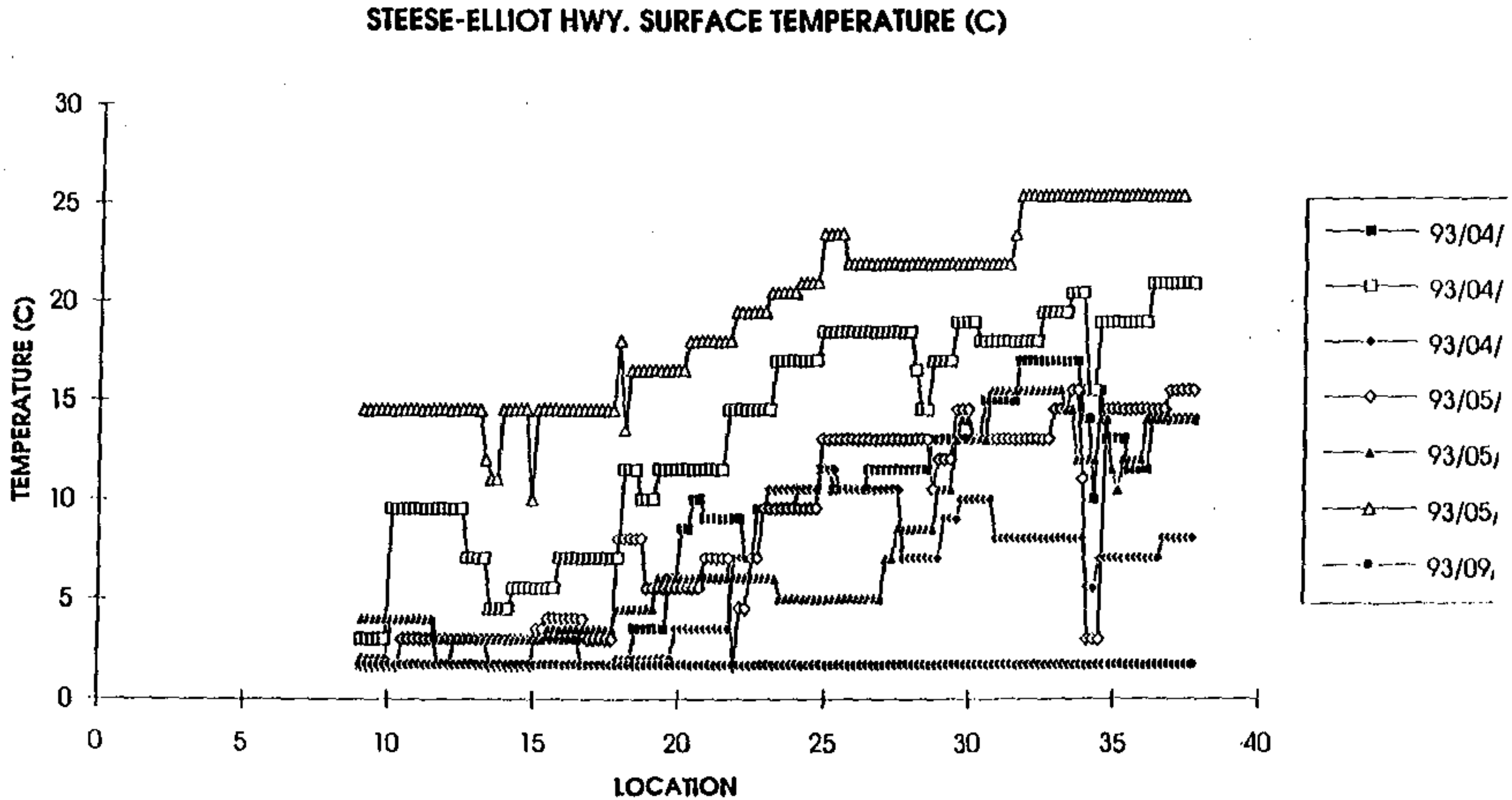

Figure 3.15 Steese/ Elliot Highway Pavement Surface Temperature

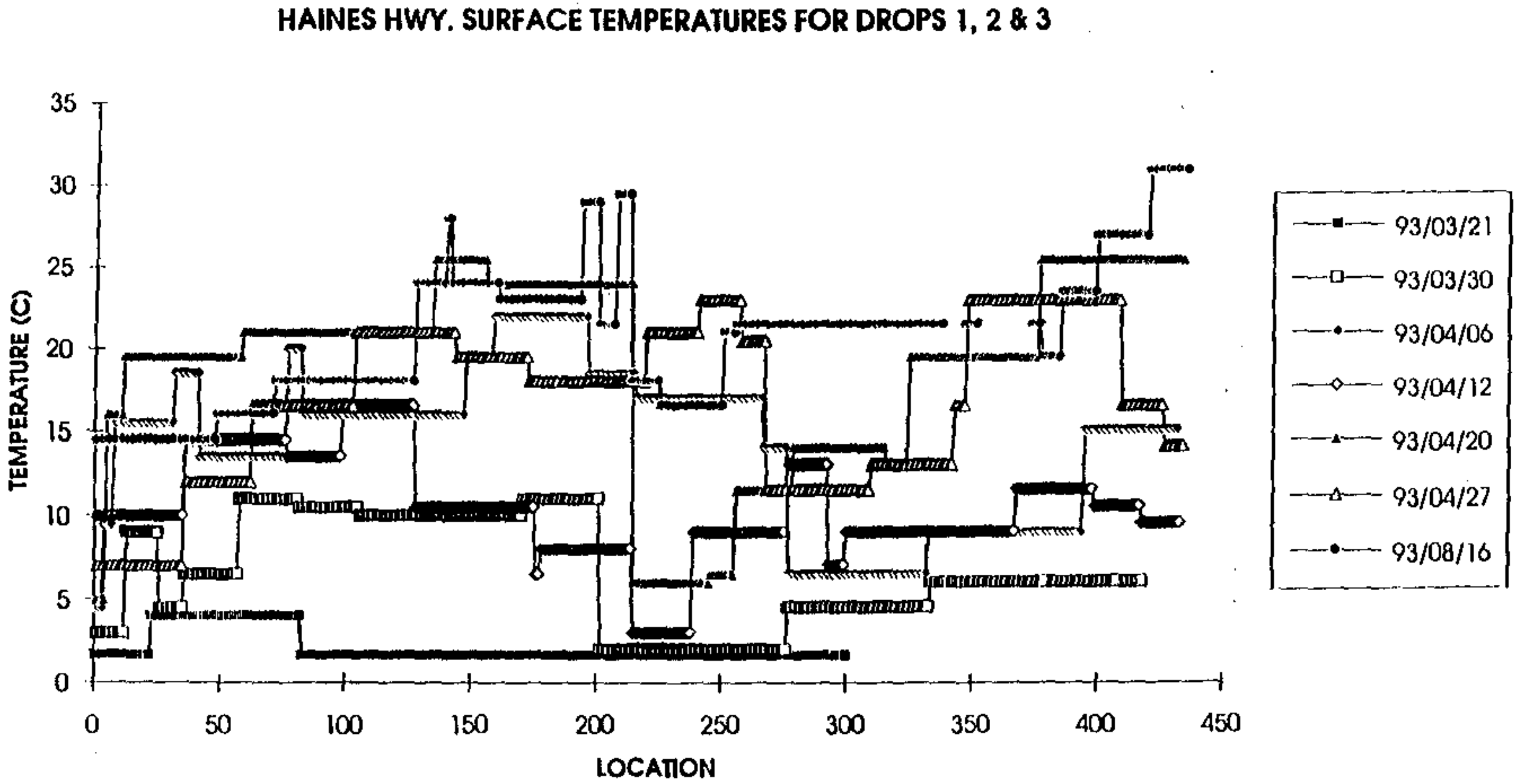

Figure 3.16 Haines Highway Pavement Surface Temperature

Three categories of tests were performed: 1) Rut measurements were conducted periodically (between April and September 1994) by AKDOT&PF at selected points along the Northbound lane of the Steese/Elliot; 2) Rut studies for both the Northbound and Southbound lanes were also conducted by the University of Alaska Fairbanks Transportation Research Center (UAF/TRC) between September 1994 and September 1995; and 3) Profilometer rut and roughness data were collected by AKDOT&PF between February and August 1995. A summary of these studies is presented in Table 3.12. All field data are summarized in Appendix C.

Table 3.12 Field Rutting and Roughness Measurement Schedule

Agency	Date	Direction/Lane	Measurement	Equipment
AKDOT&PF	4/28/94 5/05/94 5/12/94 5/19/94 5/26/94 9/28/94	Northbound	Rutting	Straight-Edge (rut measured at selected marked locations)
UAF/TRC	9/08/94 6/01/95 9/01/95	Northbound and Southbound	Rutting	Straight-Edge (max. rut in wheel path)
AKDOT&PF	2/21/95 5/11/95 8/31/95	Northbound Northbound Northbound and Southbound	Rutting, IRI	Profilometer

3.8.1 Rut Measurements at Selected Locations

Rutting of the Steese/Elliot highway was periodically monitored by both AKDOT&PF and UAF/TRC during the "no load restriction" period. Measurements were performed using a simple straight-edge device. AKDOT&PF conducted measurements at selected locations along the Northbound lane. UAF/TRC measured maximum ruts at 0.25

mile intervals along the wheel path for both the Northbound and Southbound lanes. The variation in measured rut depth with location is presented in Figures 3.17 - 3.21. Results are summarized as follows:

1. AKDOT&PF data (Figures 3.17 and 3.18) indicate that observed pavement rutting during spring break-up, between 4/28/94 and 5/26/94, is insignificant. In fact, there were more sections that showed a "decrease" in rutting than sections exhibiting increased rutting. This is probably the result of heaving in the pavement embankment during fall and winter which could influence rut measurements during early spring. The data also indicate that very little occurred during the period 5/26/94 to 9/28/94 except for some weak sections. These are essentially located at stations 19.9, 30.5, and 34.5 and experienced total rutting between 3 mm (0.12 inches) and 15 mm (0.6 inches).
2. In the UAF/TRC study (Figures 3.19 and 3.20) the most critical rut depth in the wheel path was reported for all selected locations. Larger rut values were therefore reported in comparison with AKDOT&PF measurements. Increases in rutting magnitude for the period 9/8/94 to 6/1/95 remained generally localized and ranged, in general, from 3mm (0.12 inches) to 50 mm (2 inches). In comparison, more sections exhibited increased rutting between 6/1/95 and 9/1/95. This indicates that although pavement damage associated with spring-thaw "weakening" does not necessarily appear at the end of spring break-up, it "carries over" and induces more rutting during summer. Such behavior seems to be most critical at weak pavement sections. A total of 26 percent of the surveyed locations along the Northbound lane, between 9/8/94 and 9/1/95, exhibited increased

rutting in excess of 3 mm (0.12 inches) with an overall average equal to 24 mm (0.9 inches). On the other hand, 18 percent of the measurements for the Southbound lane (Figure 3.20) experienced ruts that exceed 3 mm (0.12 inches) with an average of 11 mm (0.4 inches). It should be emphasized that these data are biased towards the most critical pavement sections since only the maximum ruts at the selected locations were reported.

3.8.2 Profilometer Rut and Roughness Study

Rut and roughness measurements were also performed using an AKDOT&PF profilometer. Data were collected periodically between 2/21/95 and 8/31/95. The profilometer provides continuous reading of rutting and roughness along the road. The data are averaged each 0.1 mile and, therefore, are more indicative of the overall behavior of the pavement than with simple "straight-edge" rut measurements. Graphical representation of the data is presented in Figures 3.21 - 3.24. Results indicate the following:

1. Profilometer data collected on 2/21/95 showed more roughness and rutting when compared with the measurements of 5/11/95 (Figure 3.21). An estimated 45 percent of the pavement exhibited more rutting on 2/21/95. Similar observations could also be made relative to road roughness, expressed in terms of the International Roughness Index (IRI, m/km) (Figure 3.22). Rutting ranged from -8 mm (-0.3 inches) to about 18 mm (0.7 inches) in February and from about -4 mm (-0.16 inches) to 20 mm (0.8 inches) in May. The February readings showed evidence of heaving as indicated by the "negative" rut measurements. Increased

road roughness in February could also be a result of uncleared ice and snow at some locations.

2. More rutting seems to occur during the period from May to August than during spring break-up. The observed increase in rutting is in the range of 5 mm (0.2 inches) to 8 mm (0.3 inches). It is interesting to note that for the same period, the IRI decreased slightly and ranged between 1 and 6.

3. The influence of increased loads on pavement damage was assessed by comparing pavement rutting and roughness, of the Northbound and Southbound lanes. Traffic data obtained from the Fox weigh station for the period 1992 - 1994 were analyzed and the Northbound and Southbound EALs were determined (Table 3.13). Pavement rutting and roughness data were also analyzed and expressed as "average" values for a given highway segment (Table 3.14). "Critical" values for the 90th percentile level (i.e. the most critical 10 percent of data for a given segment) were also determined (Table 3.15).

Table 3.13 Equivalent 18 kip Single Axle Loads for Northbound and Southbound Traffic on the Steese/Elliot Highway

Year	Northbound EALs	Southbound EALs
1992	22,708	6,172
1993	25,450	5,071
1994	26,670	8,714
TOTAL	74,828	19,957

Results of analysis indicate that although the Southbound traffic, in terms of EALs, is about 73 percent less than the Northbound traffic, the corresponding change in rutting and IRI is much smaller. In this case, the average decrease in rutting and IRI for Southbound pavements in comparison with Northbound pavements is about 14 percent (ranges between -35 and 40 percent) and 3 percent (ranges between -11 and 19 percent), respectively (Table 3.14).

Table 3.14 Average Rut and Roughness for the Northbound and Southbound Lanes of the Steese/Elliot Highway (8/31/95 Data)

Station (CDS Miles)	Northbound Rutting (mm)	Northbound IRI (m/km)	Southbound Rutting (mm)	Southbound IRI (m/km)
9.1 - 11.5	4.3	3.0	5.8	3.3
11.5 - 14.9	5.0	1.8	5.5	2.0
14.9 - 16.7	6.8	1.6	6.3	1.7
16.7 - 17.5	8.8	1.8	7.1	1.8
17.5 - 21.5	6.5	2.1	6.5	2.3
21.5 - 23.5	9.1	3.0	6.9	2.6
23.5 - 24.5	6.8	1.7	6.2	1.5
24.5 - 26.0	8.2	2.5	7.8	2.7
26.0 - 27.5	6.8	1.3	4.8	0.9
27.5 - 30.0	7.1	1.6	5.1	1.3
30.0 - 31.0	5.8	2.0	3.5	1.9
31.0 - 35.0	6.2	1.1	4.1	1.0
35.0 - 37.0	8.4	1.6	5.4	1.3

Table 3.15 Critical Rut and Roughness (90th percentile) for the Northbound and Southbound Lanes of the Steese/Elliot Highway (8/31/95 Data)

Station (CDS Miles)	Northbound Rutting (mm)	Northbound IRI (m/km)	Southbound Rutting (mm)	Southbound IRI (m/km)
9.1 - 11.5	9.1	4.1	10.7	4.7
11.5 - 14.9	8.4	2.5	7.2	2.6
14.9 - 16.7	12.4	2.2	12.0	2.1
16.7 - 17.5	16.1	2.4	9.5	2.0
17.5 - 21.5	11.4	3.2	10.1	3.6
21.5 - 23.5	16.3	5.0	11.0	4.2
23.5 - 24.5	9.4	3.0	11.6	2.5
24.5 - 26.0	13.2	3.9	13.0	3.8
26.0 - 27.5	10.5	2.1	6.4	1.1
27.5 - 30.0	11.4	2.6	7.2	2.0
30.0 - 31.0	9.4	3.0	5.2	3.0
31.0 - 35.0	8.8	1.3	5.4	1.1
35.0 - 37.0	11.3	2.1	6.9	1.6

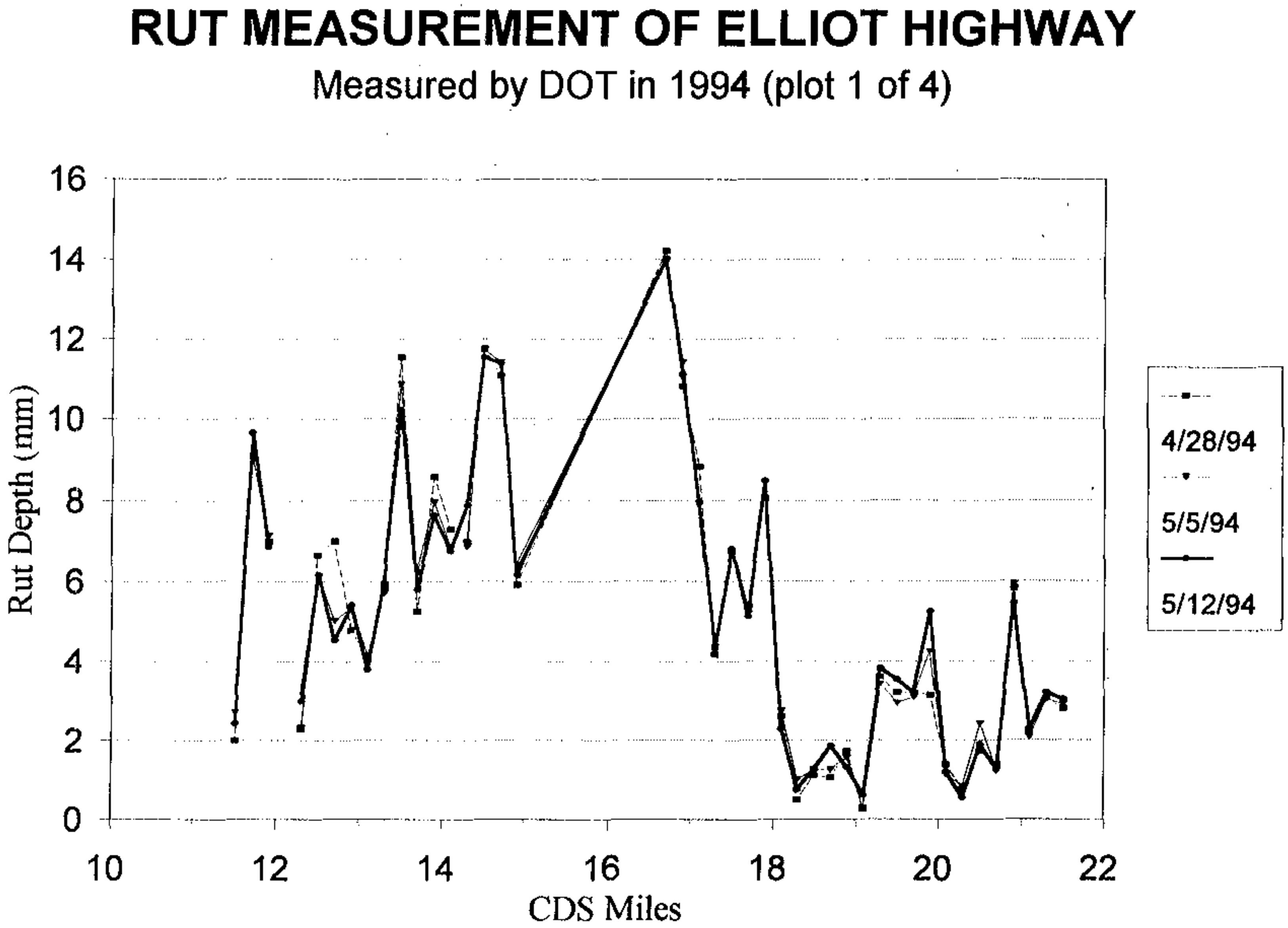

Figure 3.17 AKDOT&PF Rut Measurements for the Steese/Elliot (4/28/94 to 5/12/94 Northbound)

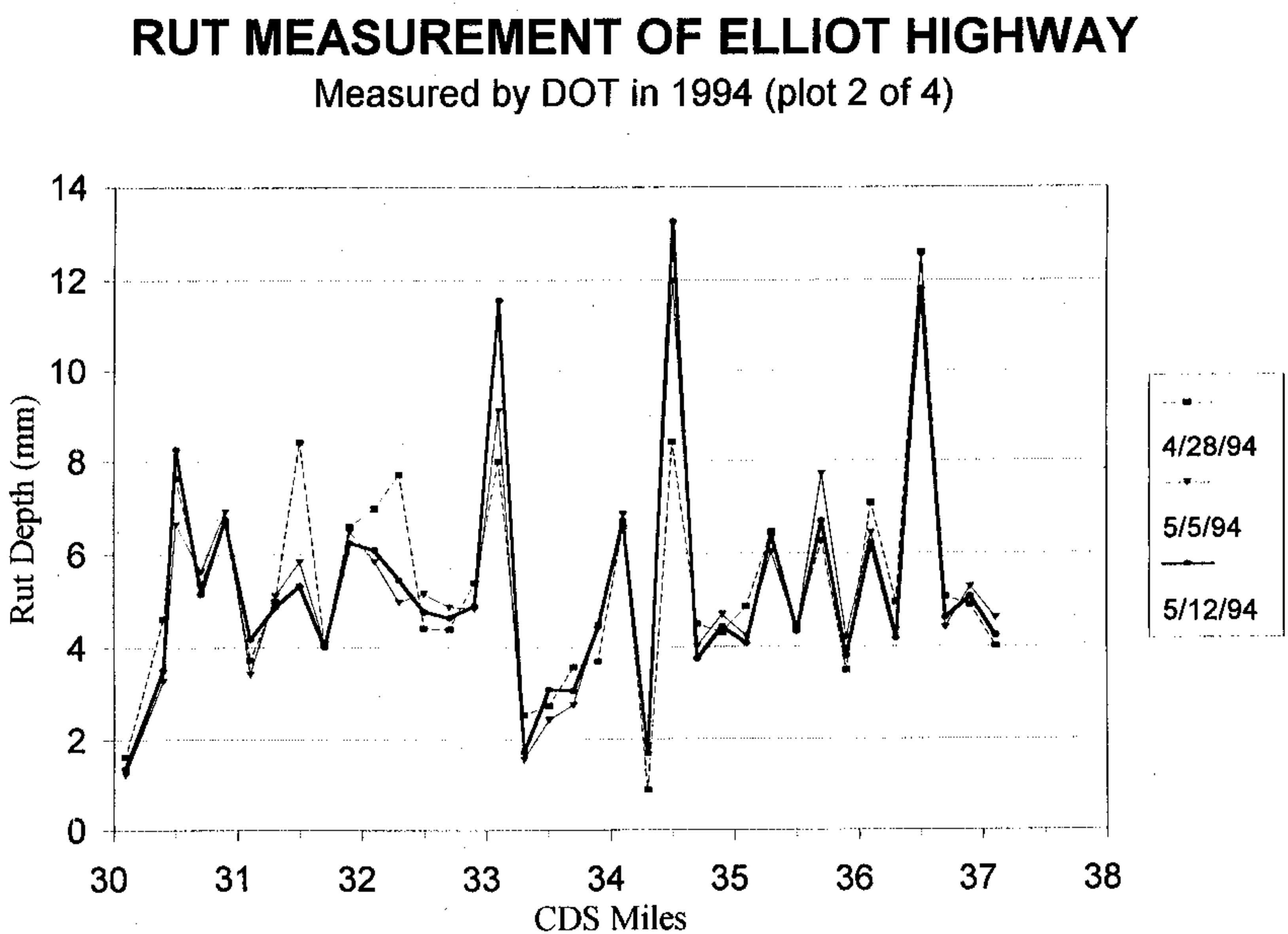

Figure 3.17 (Con't) AKDOT&PF Rut Measurements for the Steese/Elliot

(4/28/94 to 5/12/94 Northbound)

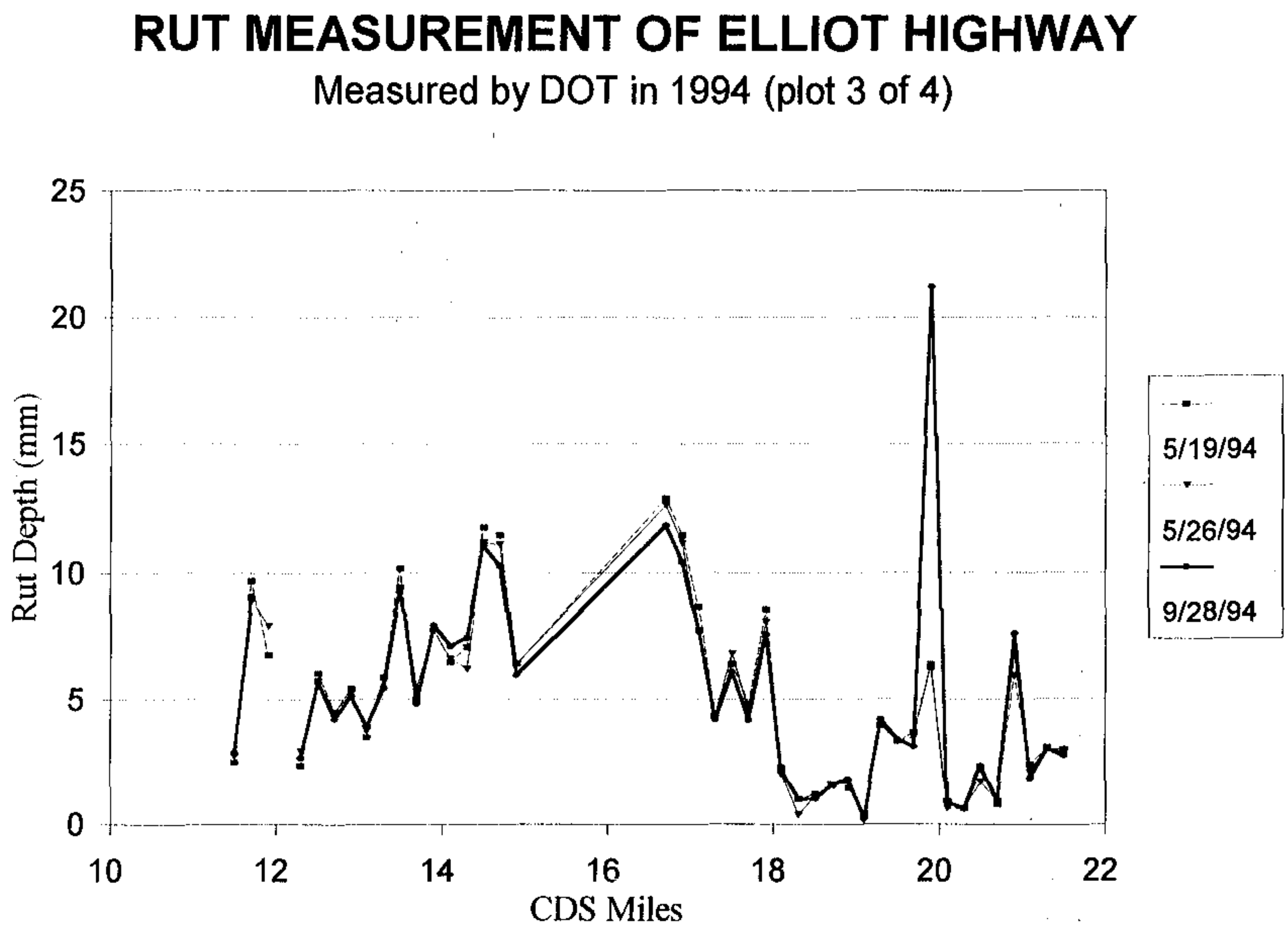

Figure 3.18 AKDOT&PF Rut Measurements for the Steese/Elliot (5/19/94 to 9/28/94 Northbound)

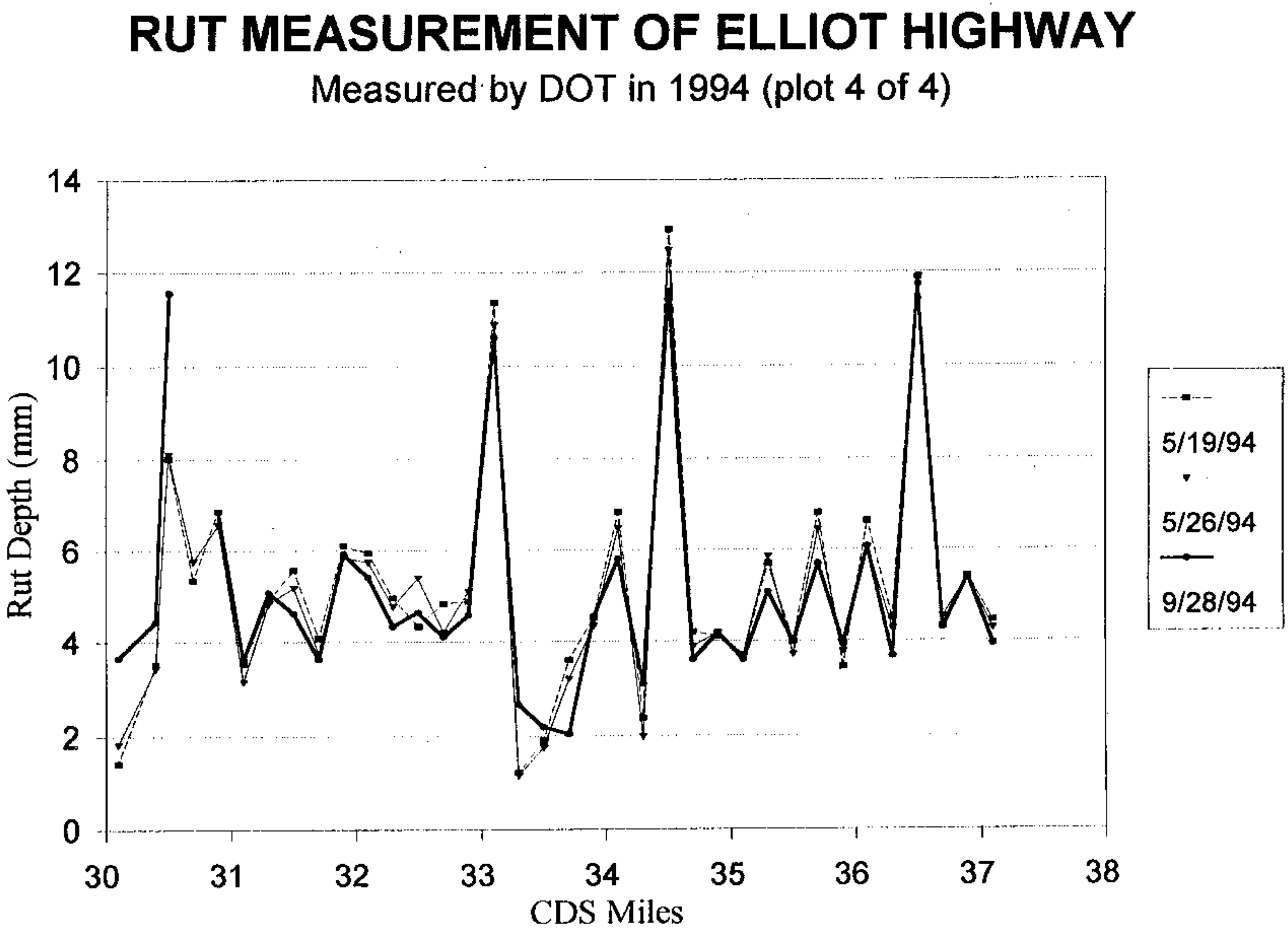

Figure 3.18 (Con't) AKDOT&PF Rut Measurements for the Steese/Elliot

(5/19/94 to 9/28/94 Northbound)

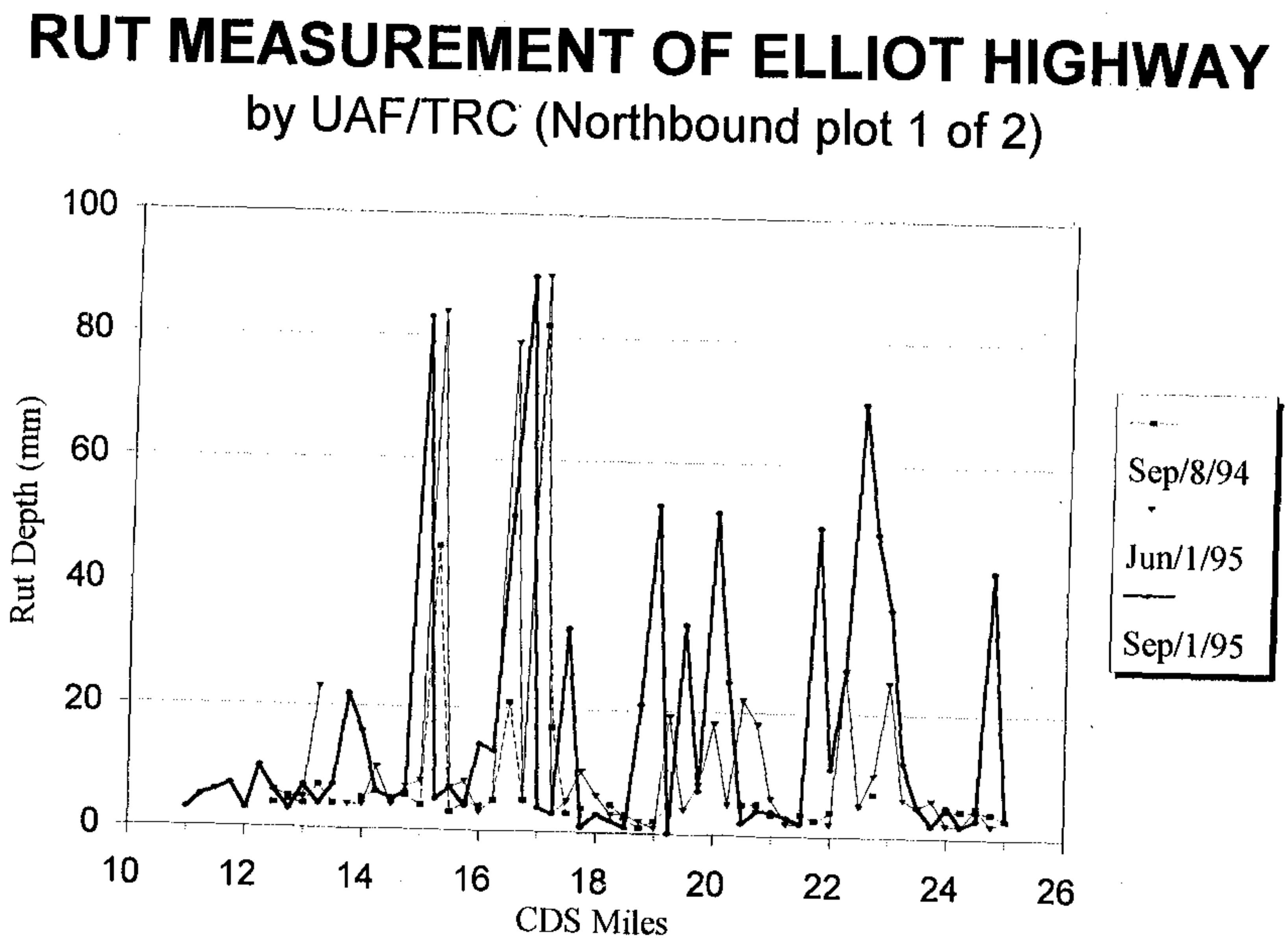

Figure 3.19 UAF/TRC Rut Measurements for the Steese/Elliot (Northbound)

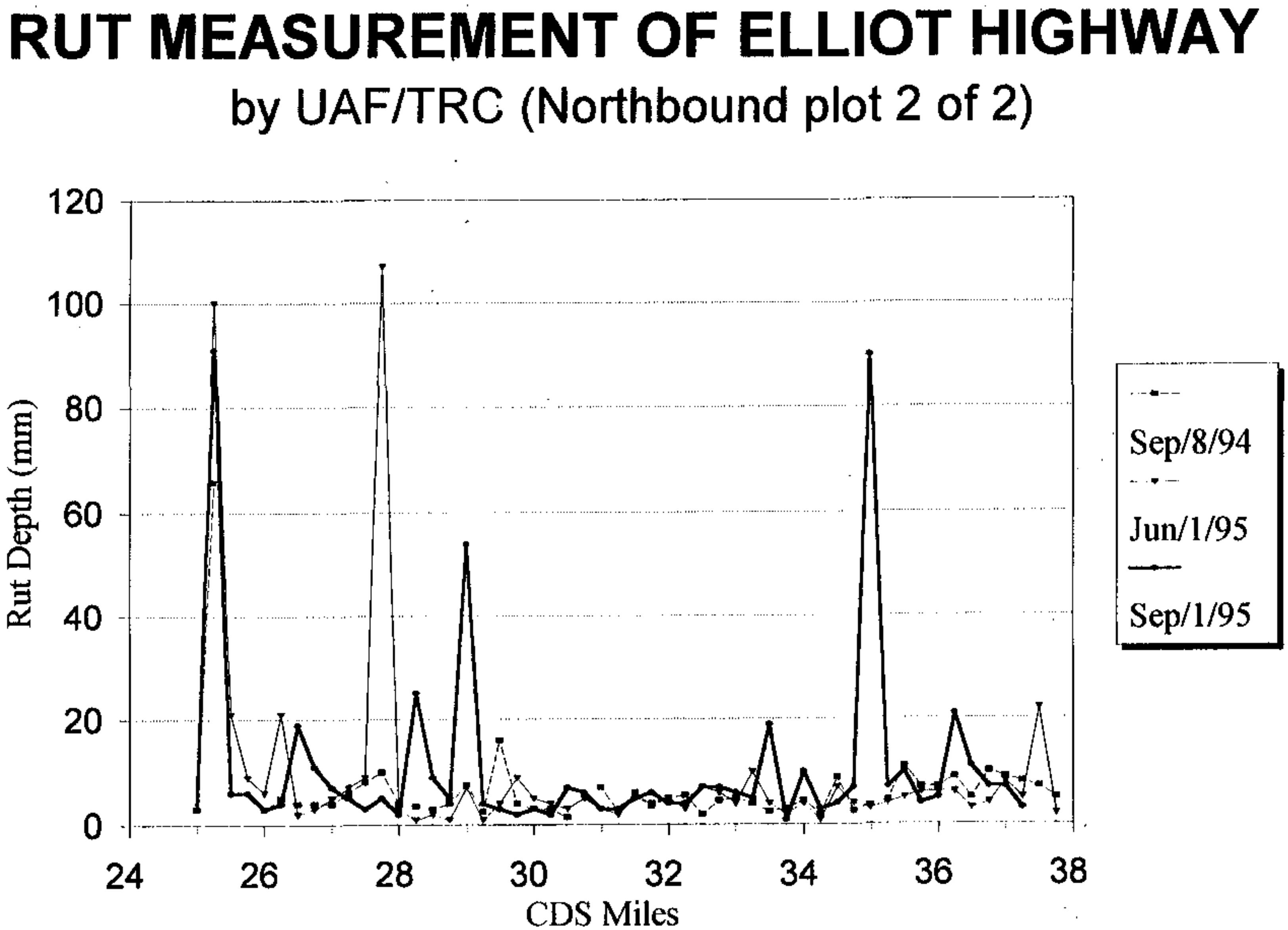

Figure 3.19 (Con't) UAF/TRC Rut Measurements for the Steese/Elliot (Northbound)

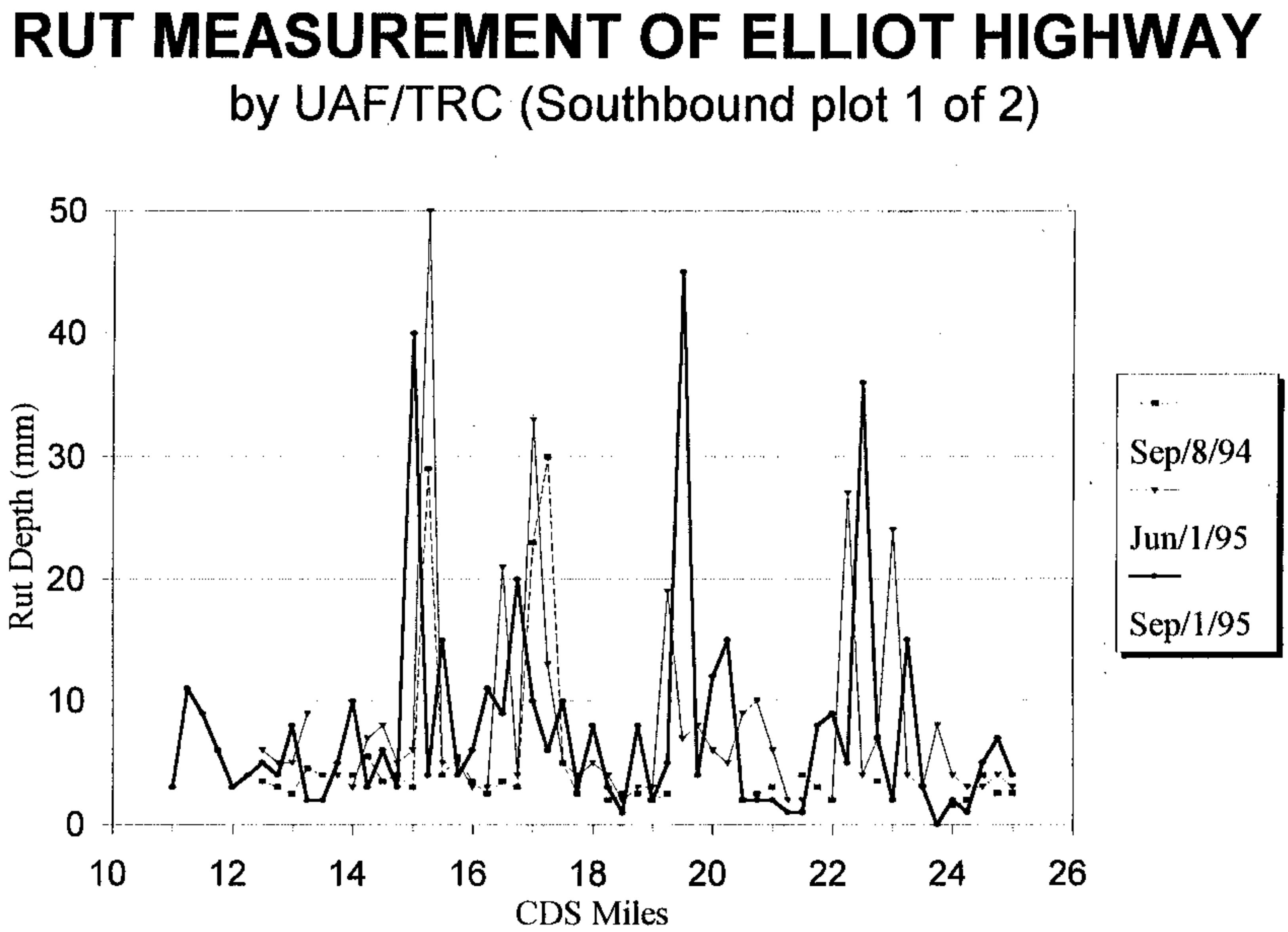

Figure 3.20 UAF/TRC Rut Measurements for the Steese/Elliot (Southbound)

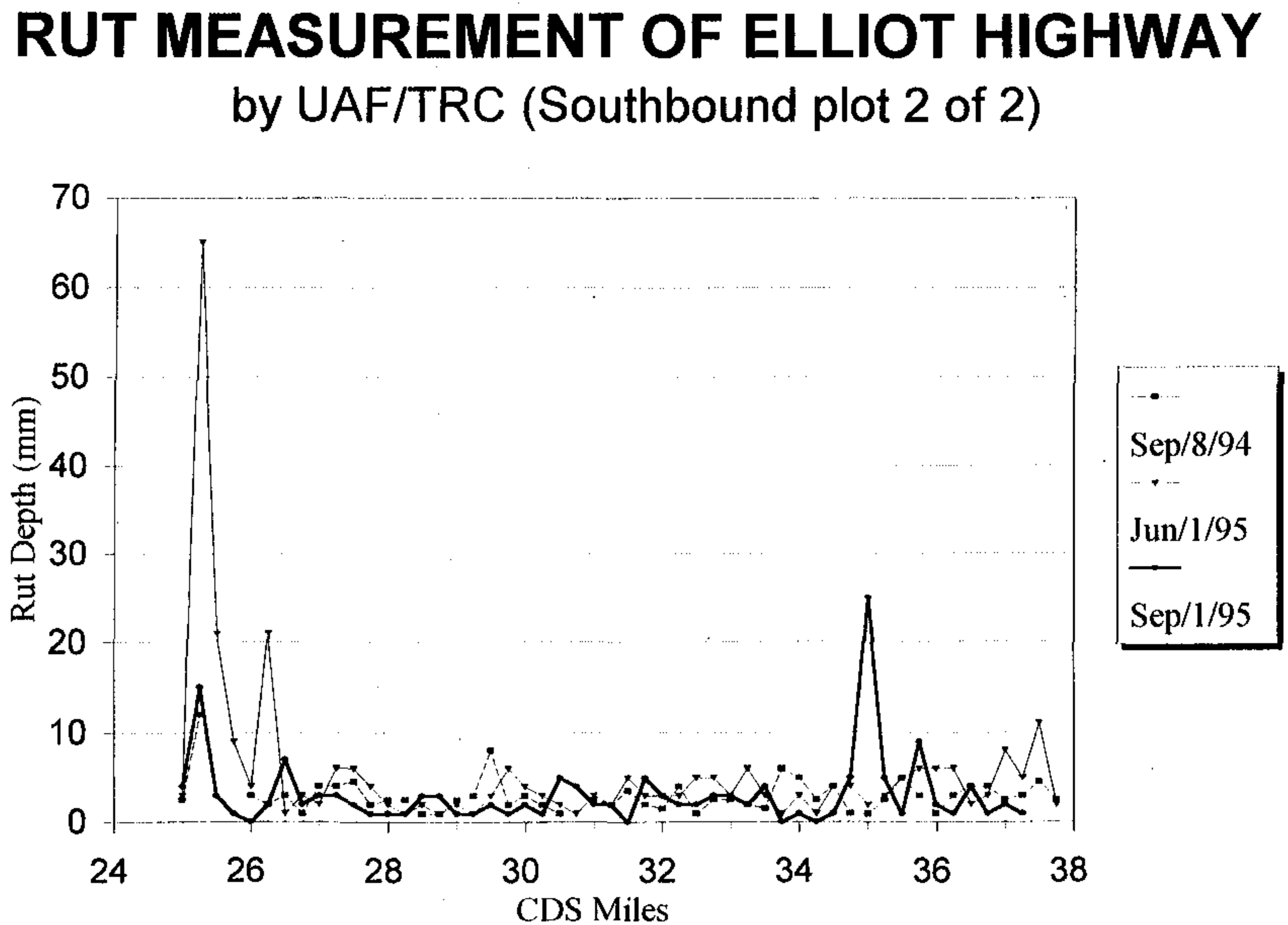

Figure 3.20 (Con't) UAF/TRC Rut Measurements for the Steese/Elliot (Southbound)

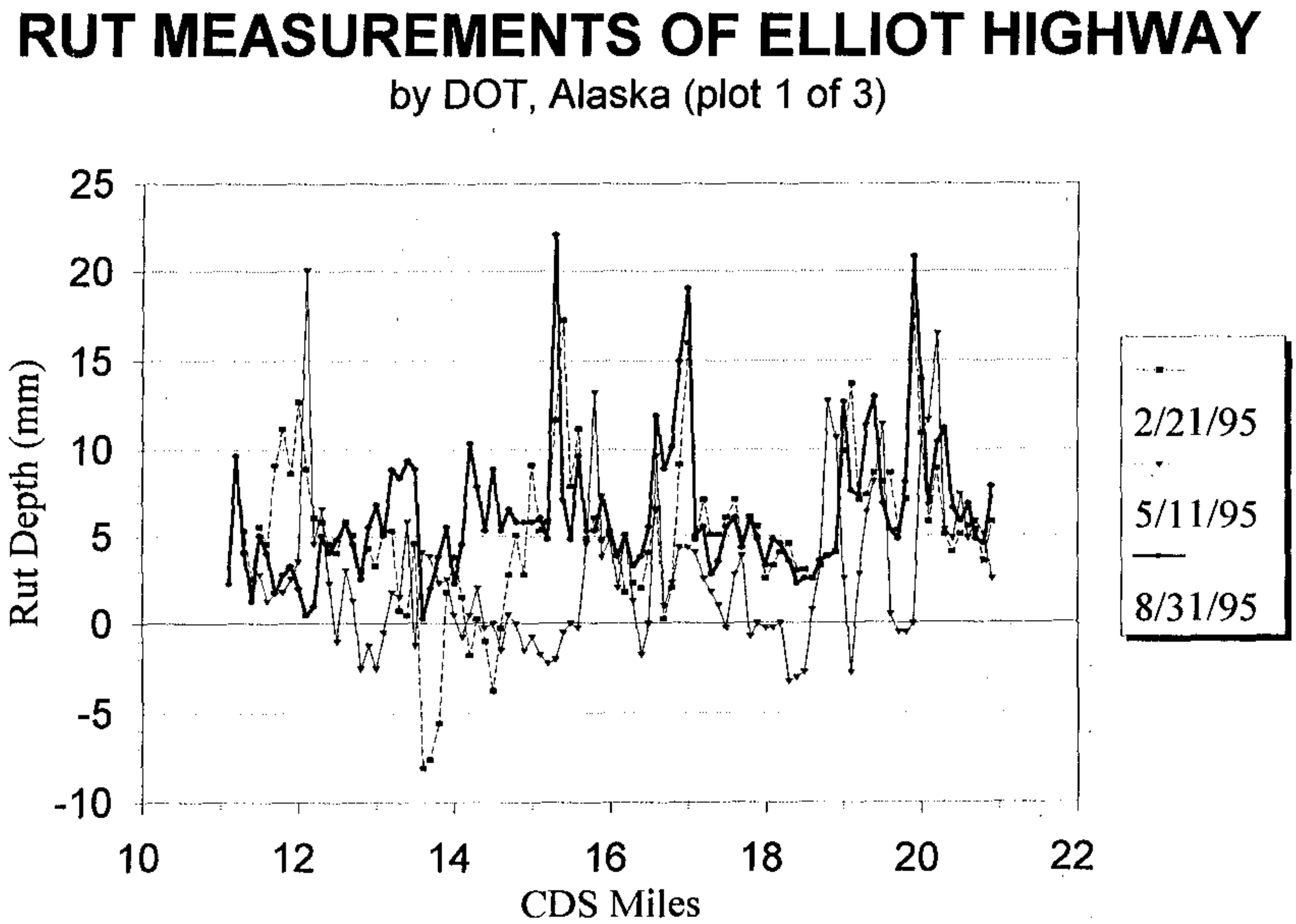

Figure 3.21 Profilometer Rutting Measurement for the Steese/Elliot (Northbound)

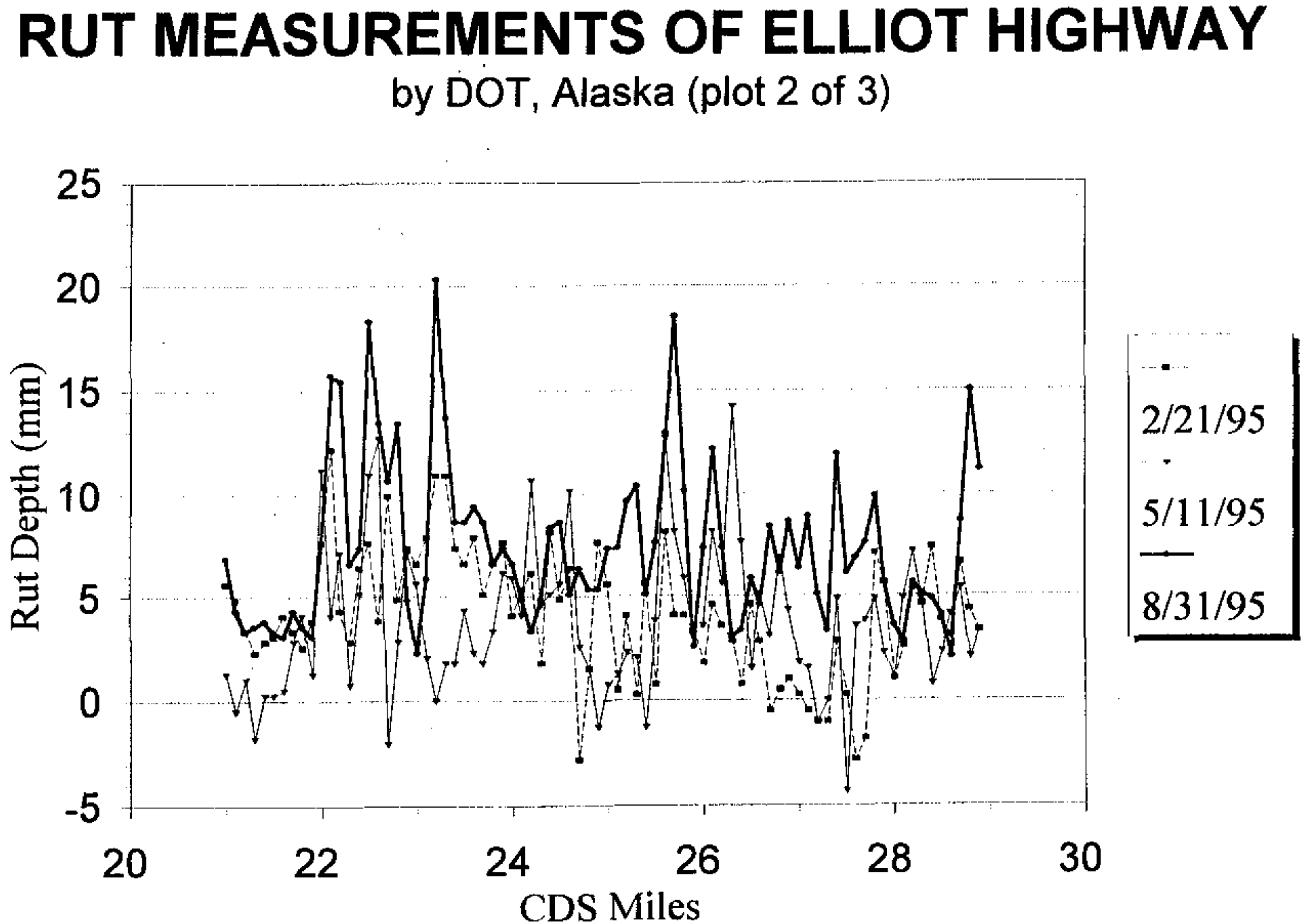

Figure 3.21 (Con't) Profilometer Rutting Measurement for the Steese/Elliot (Northbound)

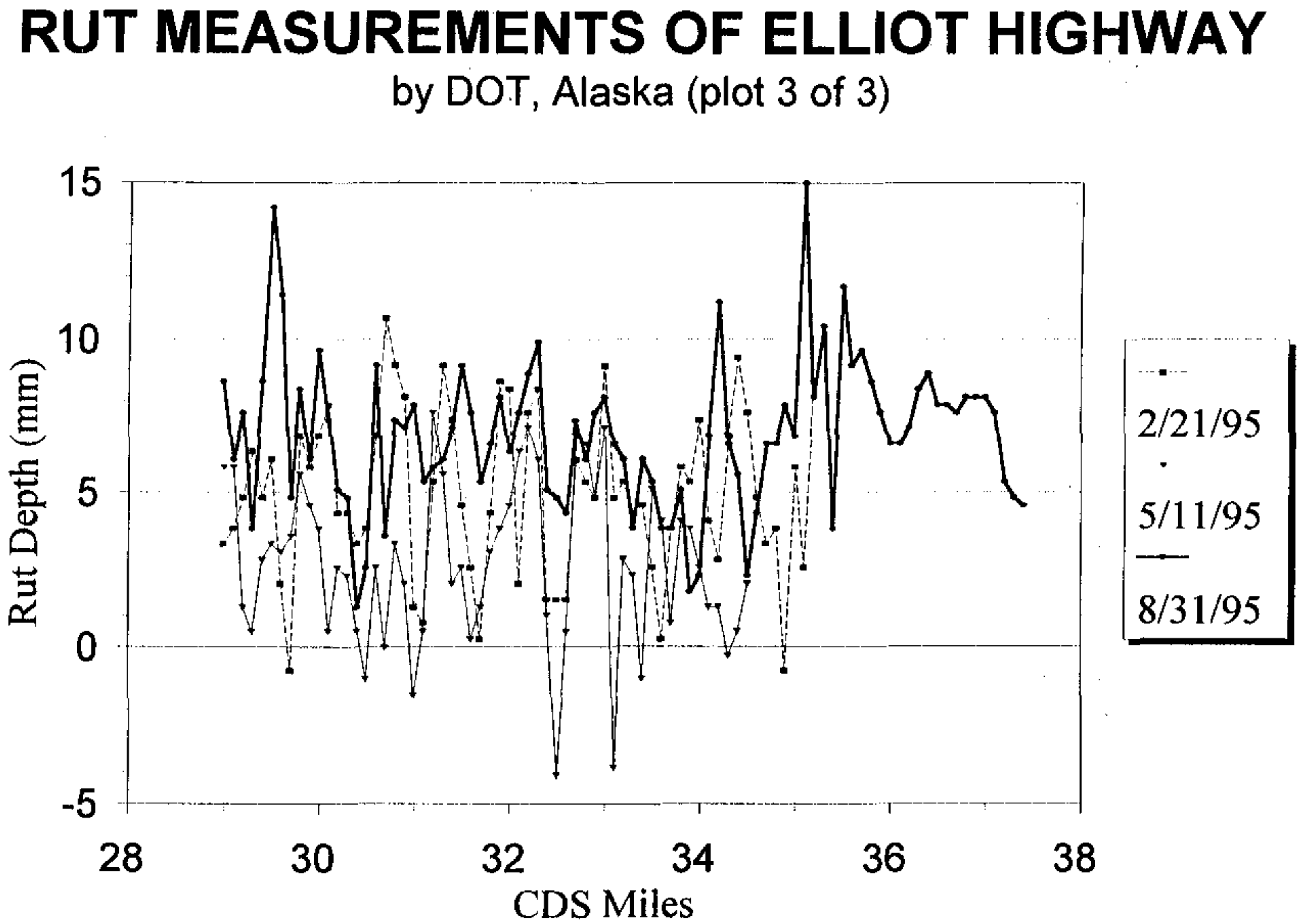

Figure 3.21 (Con't) Profilometer Rutting Measurement for the Steese/Elliot (Northbound)

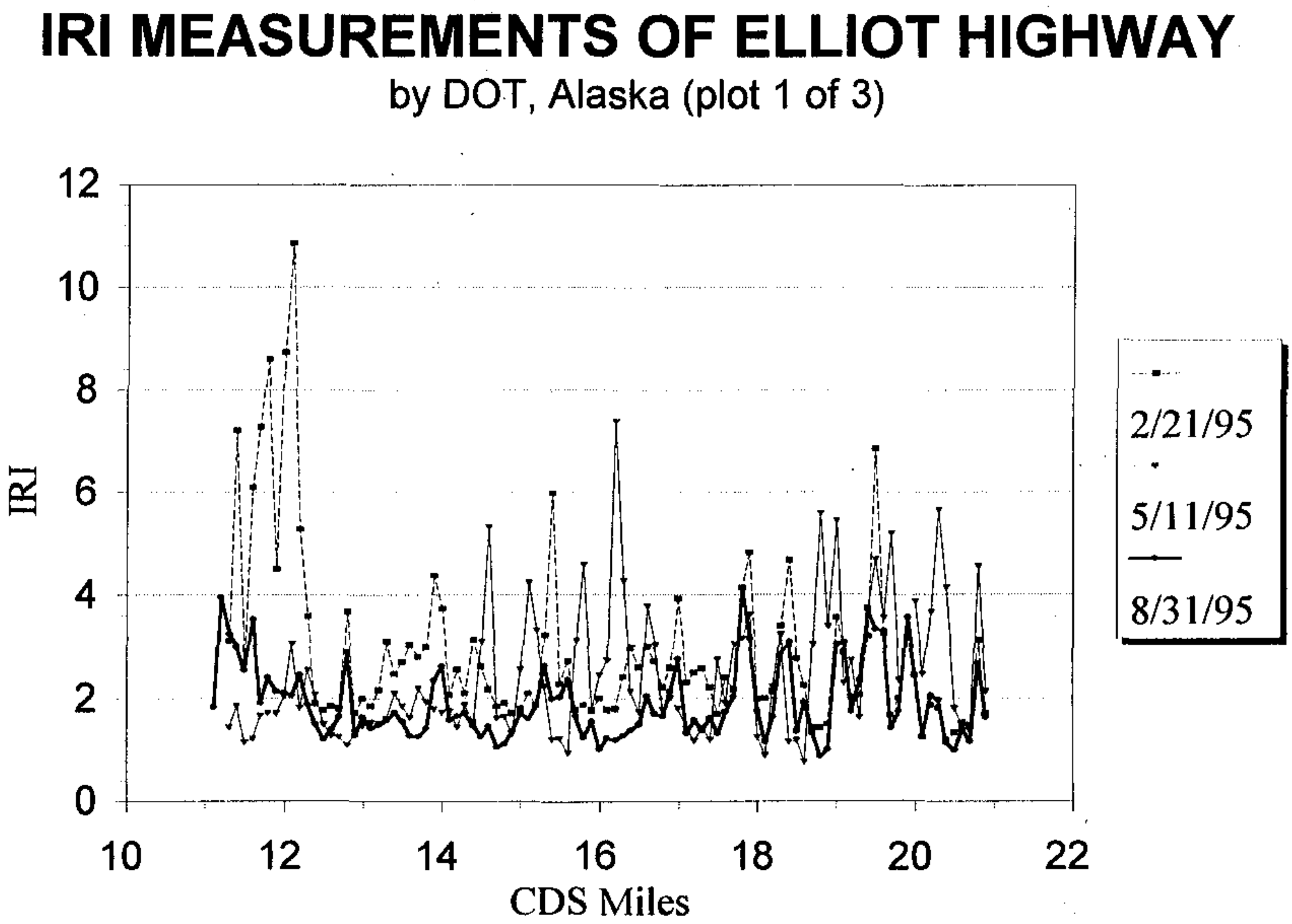

Figure 3.22 Profilometer IRI Roughness for the Steese Elliot (Northbound)

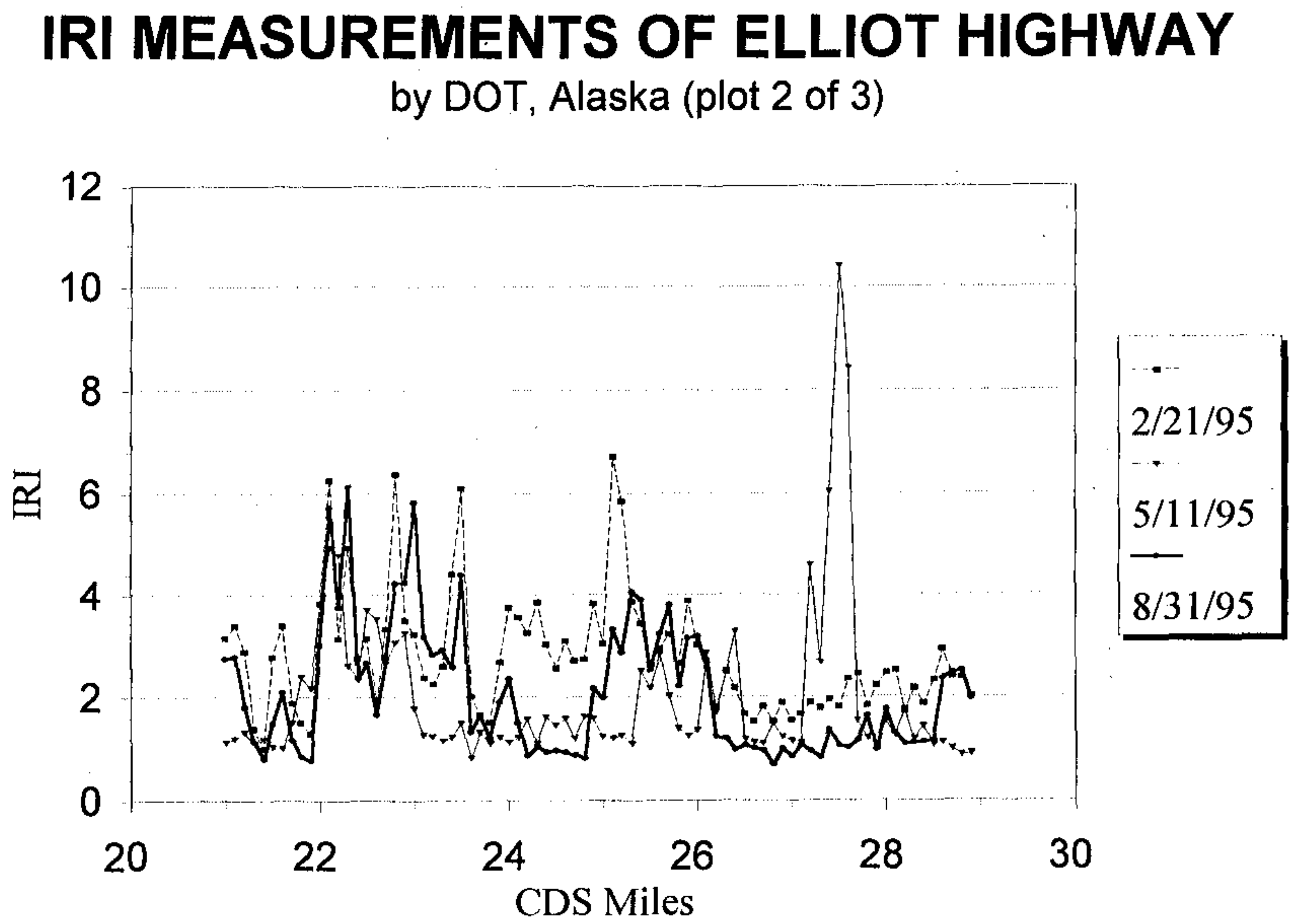

Figure 3.22 (Con't) Profilometer IRI Roughness for the Steese Elliot (Northbound)

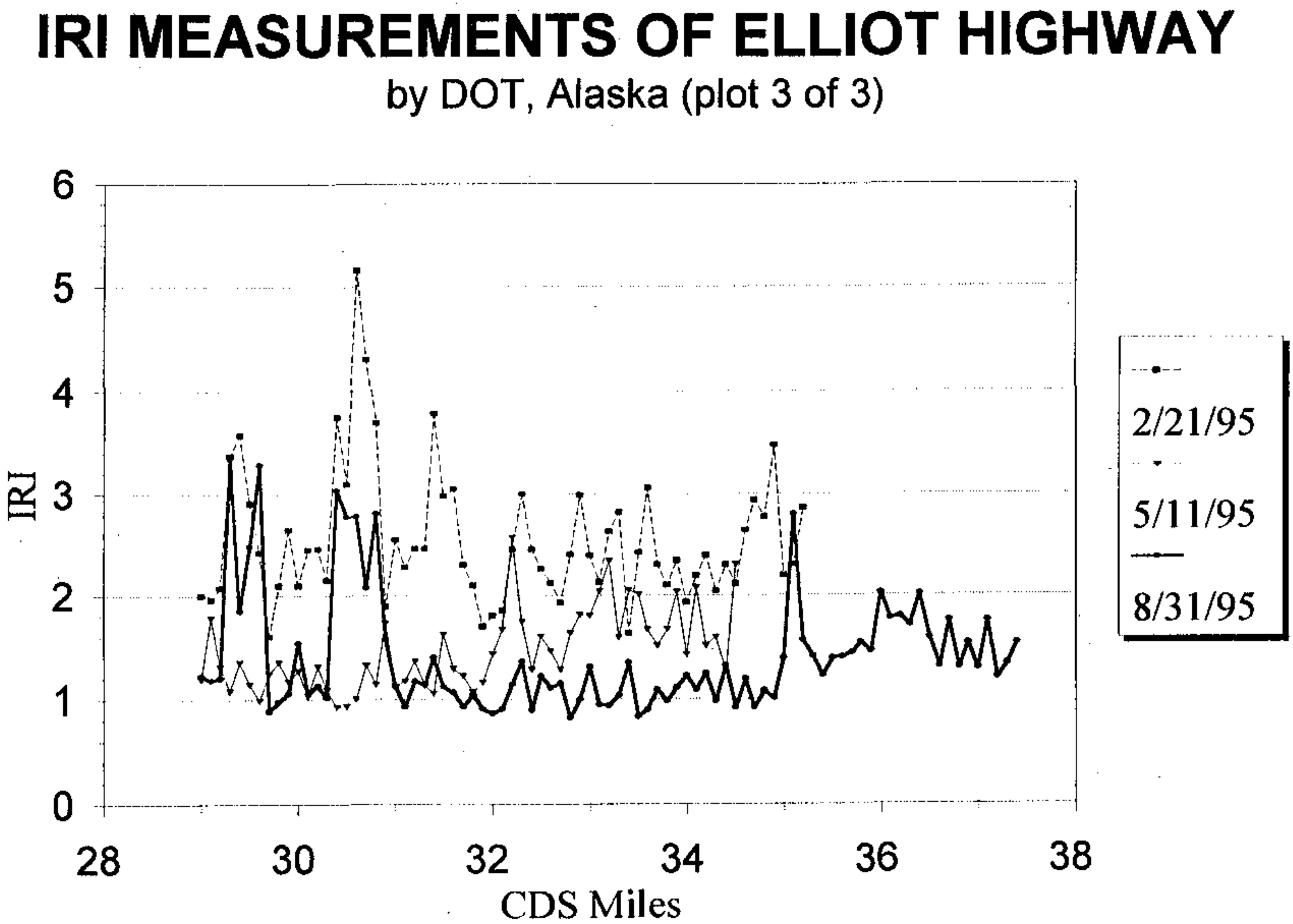

Figure 3.22 (Con't) Profilometer IRI Roughness for the Steese Elliot (Northbound)

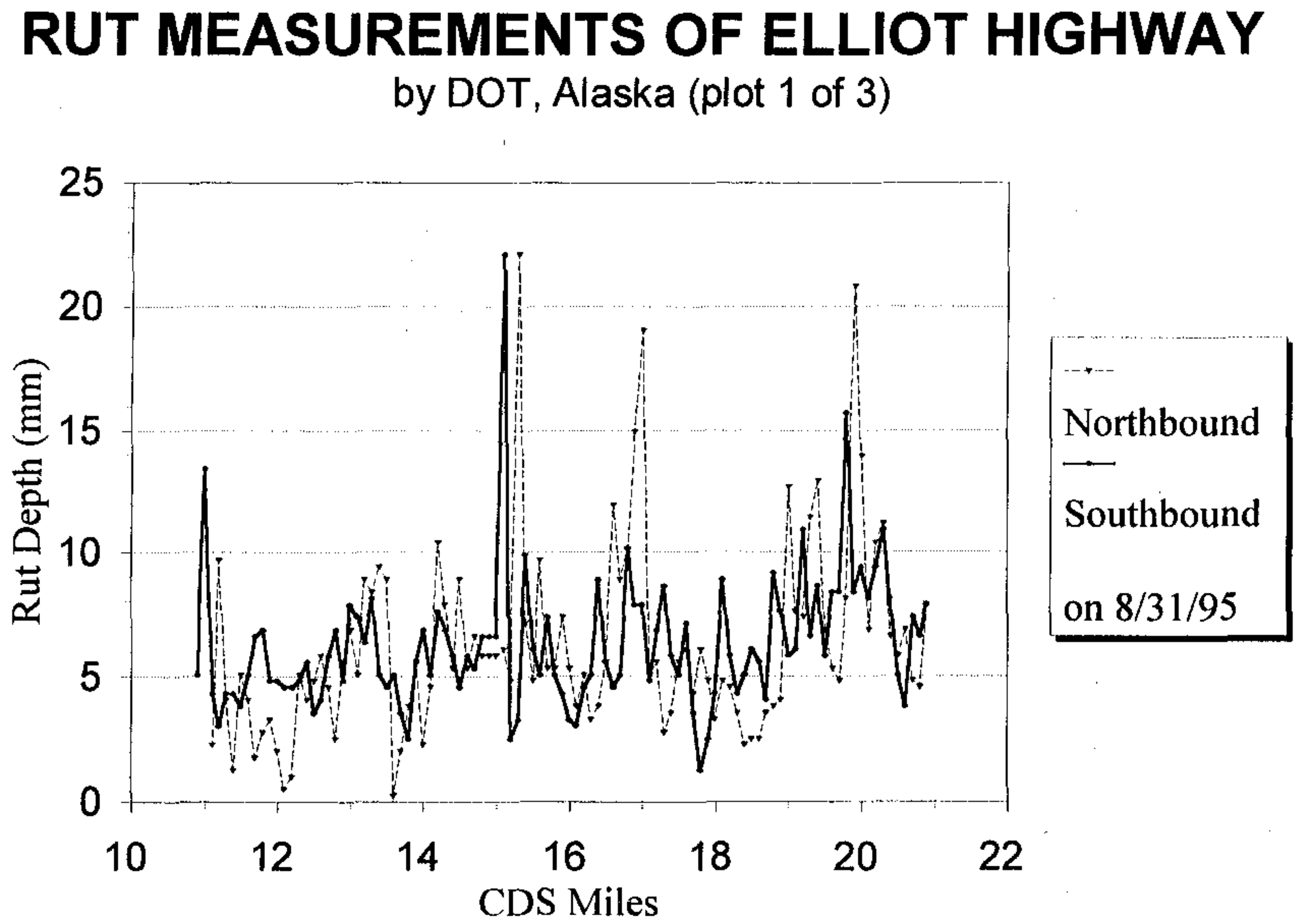

Figure 3.23 Northbound/Southbound Profilometer Rutting for the Steese/Elliot

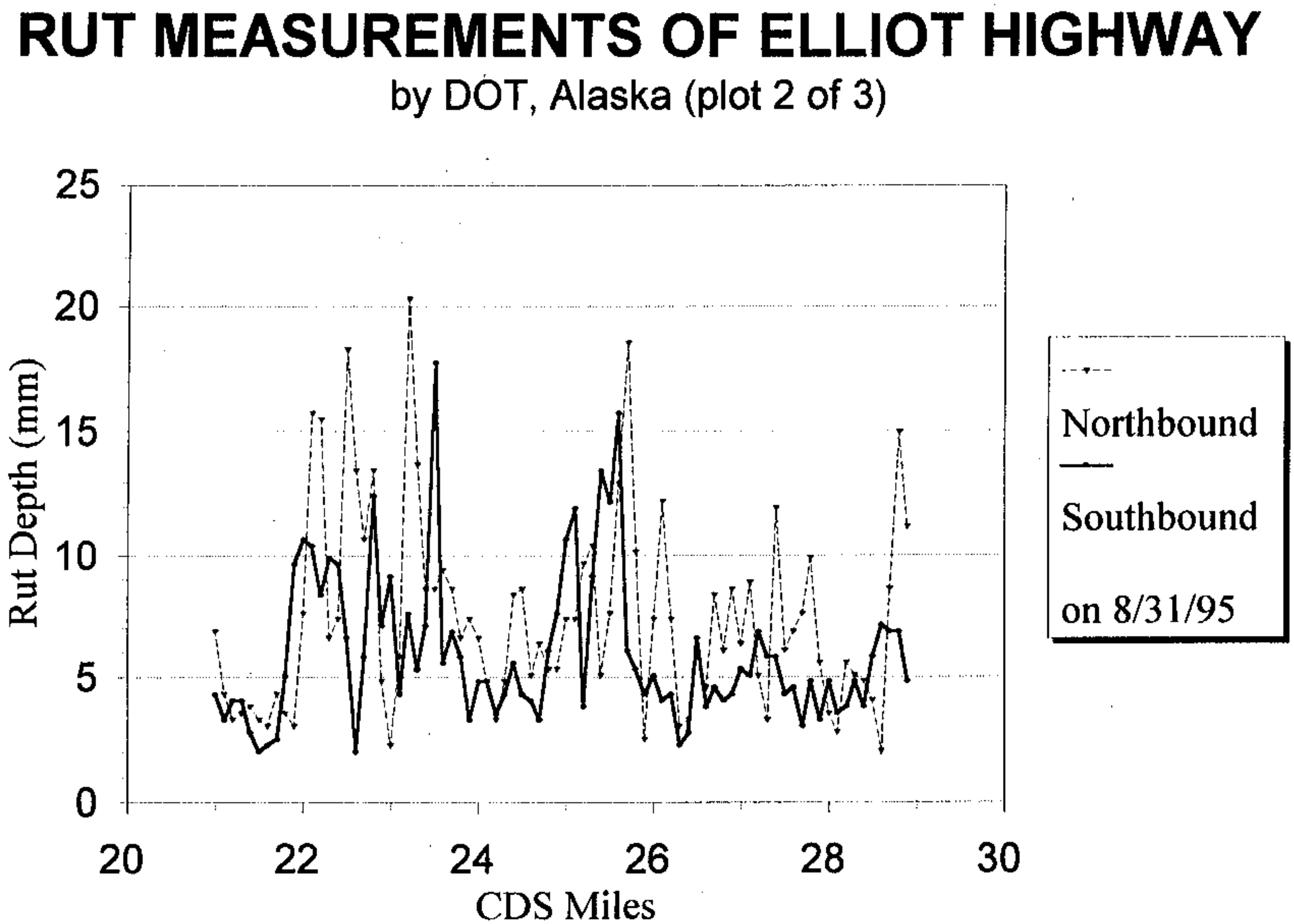

Figure 3.23 (Con't) Northbound/Southbound Profilometer Rutting for the Steese/Elliot

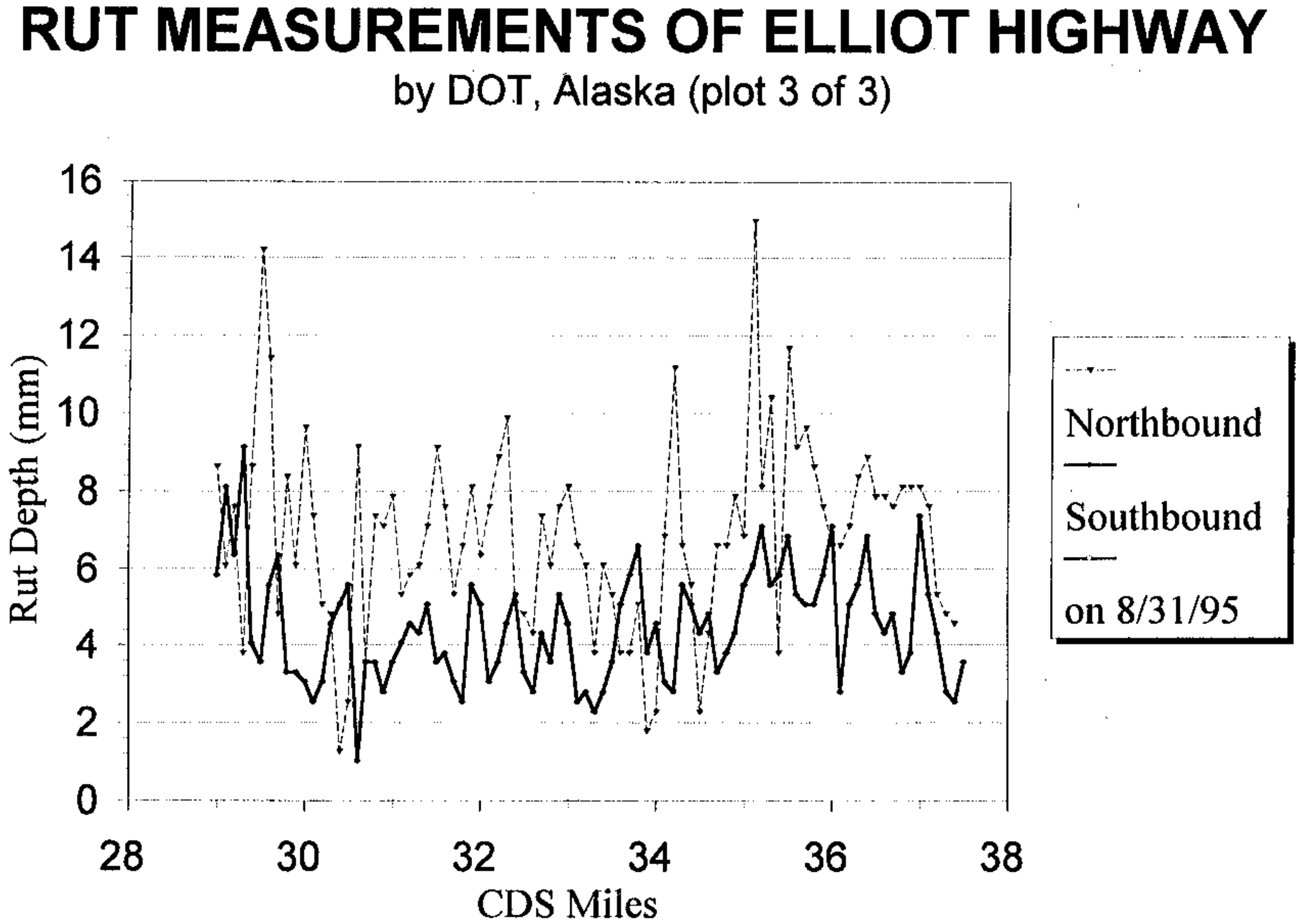

Figure 3.23 (Con't) Northbound/Southbound Profilometer Rutting for the Steese/Elliot

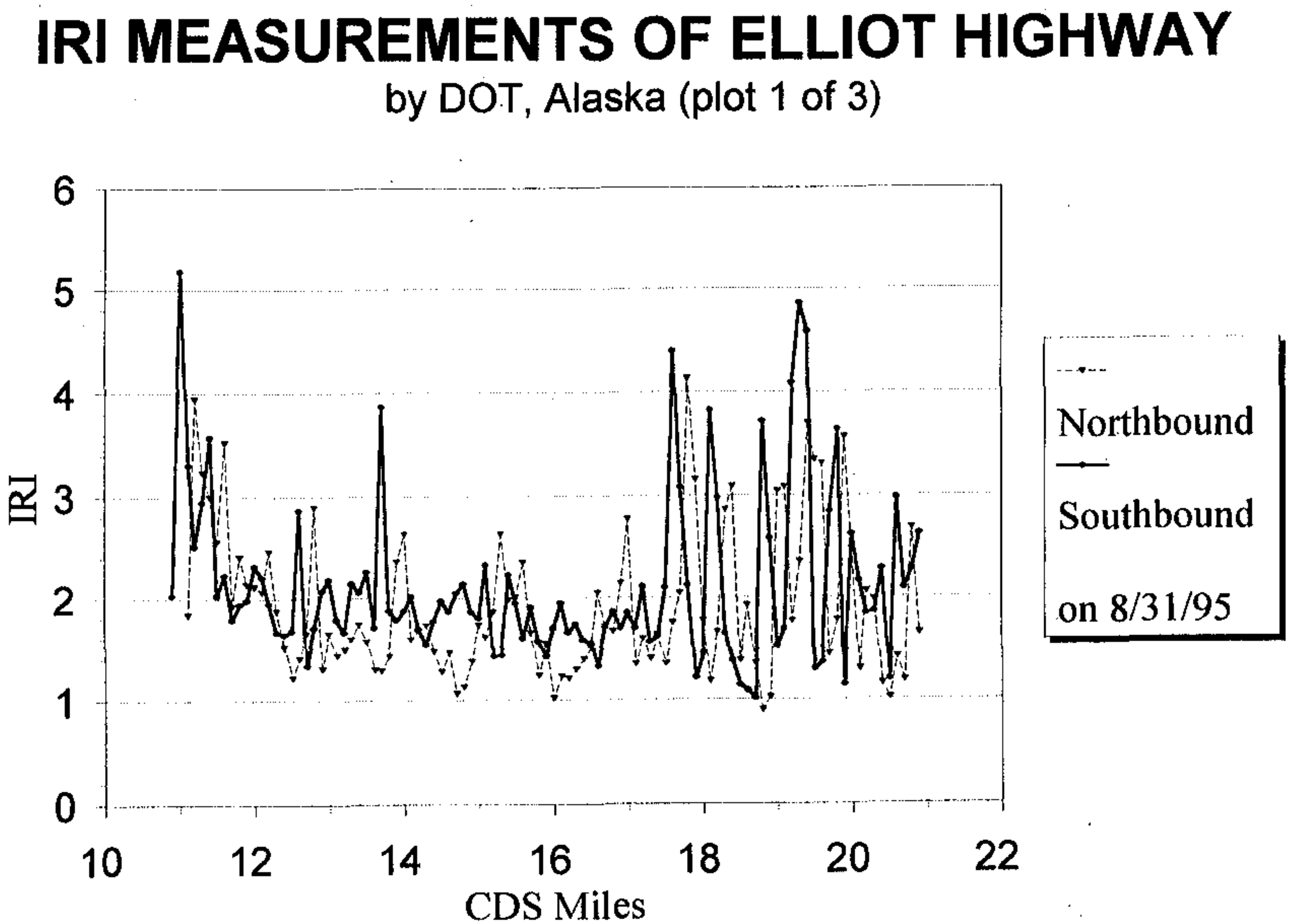

Figure 3.24 Northbound/Southbound Profilometer IRI for the Steese/Elliot

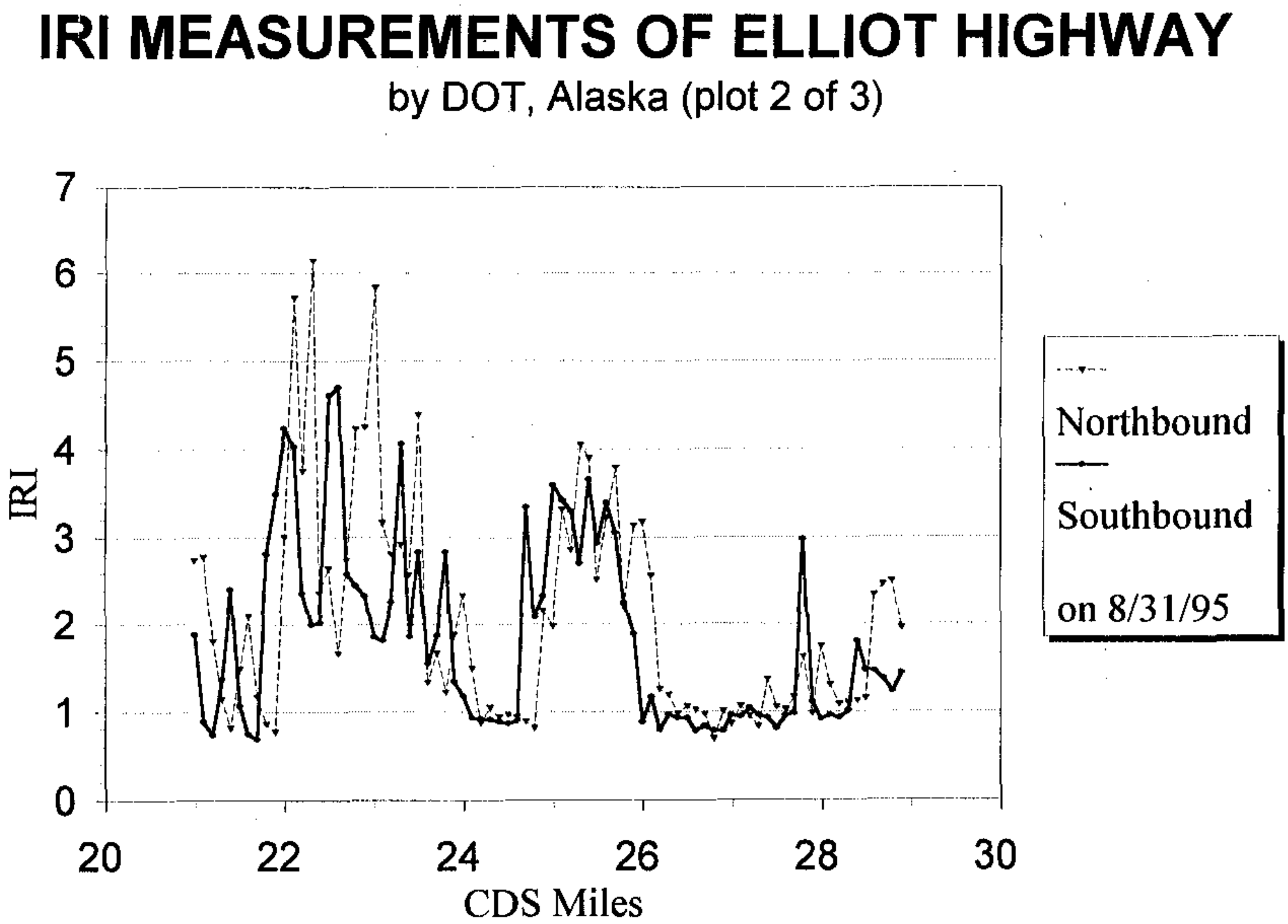

Figure 3.24 (Con't) Northbound/Southbound Profilometer IRI for the Steese/Elliot

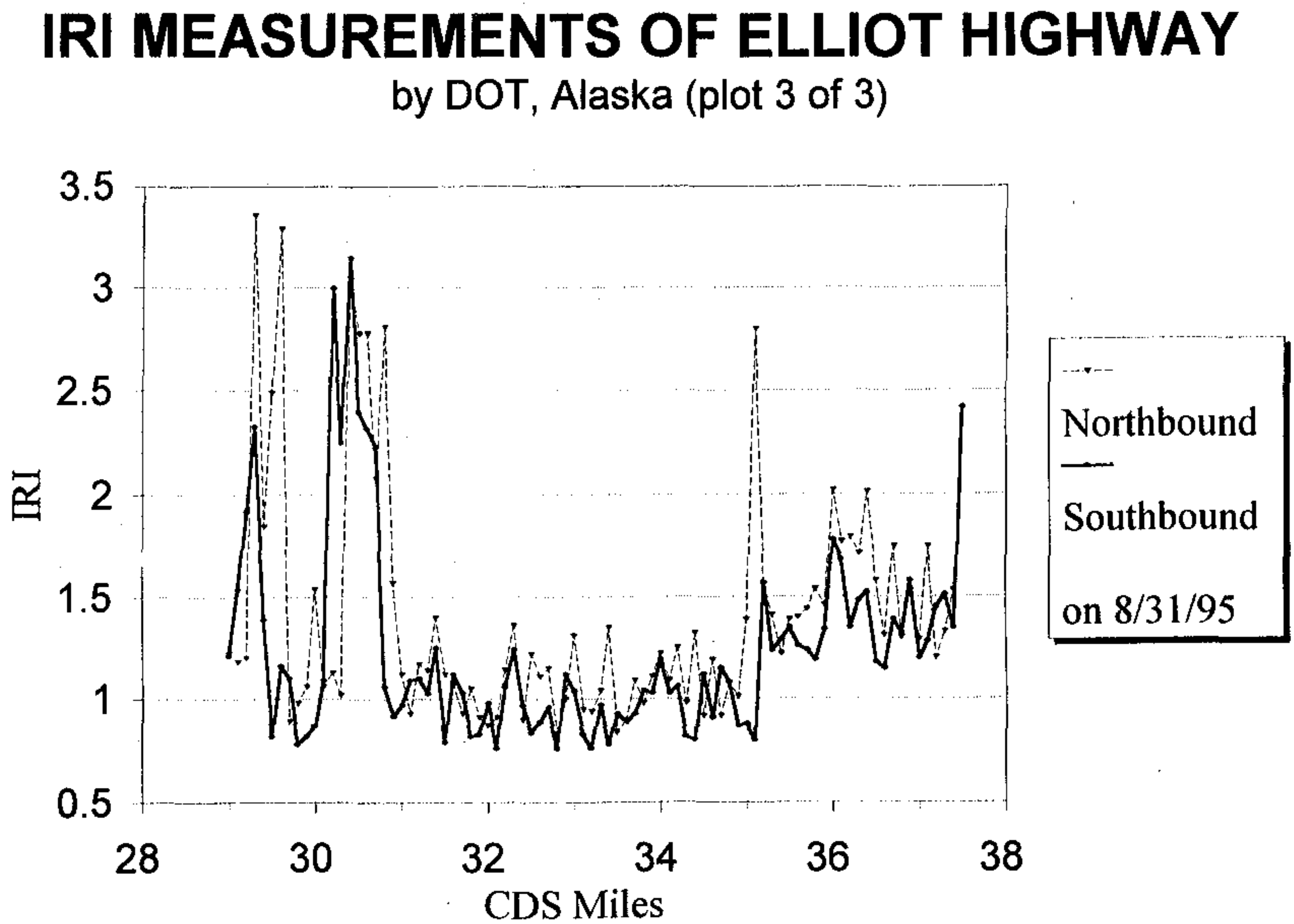

Figure 3.24 (Con't) Northbound/Southbound Profilometer IRI for the Steese/Elliot

For "critical" sections, the average decrease in rutting and IRI is 20 percent (ranges between -23 and 45 percent) and 10 percent (ranges between -15 and 48 percent), respectively (Table 3.15). These values, particularly the IRI results, are much lower than expected and may indicate that damage associated with frost heave and foundation instabilities is more significant than load related damage.

3.9 SUMMARY

Results of field studies using FWD measurements on both the Haines and Steese/Elliot highways were used to assess the potential of pavement damage associated with truck loading during spring-thaw. These studies also included monitoring pavement rutting and roughness of the Steese/Elliot during spring/summer of 1994 when load restrictions were lifted. A summary of the findings is as follows:

1. The extent of pavement weakening was determined by using FWD deflections profiles and backcalculation analyses. In almost all cases, "weakening" of the granular base was the most critical factor in pavement damage during spring-thaw. A criterion for selecting weak pavement sections was tentatively proposed. This criterion utilizes predictions of spring-thaw remaining life repetitions for both "average" and most "critical" pavement conditions and corresponding estimates for "average" and "critical" remaining pavement service life. Accordingly, highway segments that exhibit a 15 percent or more reduction in "average" remaining life in comparison with the anticipated design life are candidates for strengthening if no restrictions are applied. Strengthening *should be* applied if the "critical" remaining life is less than 30 percent of the design life

and *need to be* applied if the "critical" remaining life is greater. If these criteria are used to identify "weak" pavements of the Steese/Elliot and Haines highways, and if it is decided to strengthen sections that exhibit a remaining life less or equal to 3 years, then the estimated mileage required to be maintained or rehabilitated for the Steese/Elliot and the Haines highways will be approximately 3 miles (i.e. 11 percent) and 2 miles (i.e. 5 percent), respectively.

2. A proposed design alternative for the pavement section that will resist the applied loads with no load restrictions is as follows:

   4 inches hot-mix asphalt surface

   6 inches open-graded base, preferably bituminous-treated

   24 inches non-frost susceptible granular subbase

   The proposed pavement should be properly drained and should include materials with adequate strength and durability to resist applied traffic and extreme climatic conditions.

3. FWD center deflections do not correlate, in general, to spring-thaw pavement damage, particularly for small thaw depths, and should not be used as a criterion for load restrictions. However, the FWD surface deflection bowl can be used to backcalculate which pavement response parameters, and the corresponding remaining life repetitions. This can be used in assessing when and how much load restrictions are needed. It could also be used to determine when to remove load restrictions.

4. Significant variations in pavement surface temperatures were observed during a given testing period for both the Steese/Elliot and Haines highways. It is clear that the surface temperature varies depending on the location and time of the day and does not seem to correlate with thaw depth.

5. Extensive field tests were conducted during 1994 and 1995 on the Steese/Elliot highway to monitor rutting and roughness and to determine how much damage could be induced if no load restrictions are applied. Results indicate that pavement damage is not general but, rather, occurs at localized sections. Comparison of Northbound and Southbound traffic in terms of EALs indicates that although the Southbound traffic is about 73 percent smaller than the Northbound traffic, the average decrease in rutting and IRI is estimated to be 14 and 3 percent respectively. For "critical" sections, the average decrease in rutting and IRI is 20 and 10 percent respectively. These values, particularly the IRI results, are much lower than expected and may indicate that damage associated with frost heave and foundation instabilities is more significant than load related damage.

CHAPTER FOUR

GROUND TEMPERATURE MEASUREMENT AND ANALYSIS

4.1 OBJECTIVES

As indicated in Chapter Two of this report, ground temperature is used by most state agencies to determine the beginning and end of the spring load restriction period. In general, load restrictions are imposed as soon as pavement thawing begins. Restrictions are removed when the thaw depth becomes sufficient for the pavement to regain its strength. In most cases, visual observations of pavement distress and moisture conditions associated with pavement thawing are used to establish the timing and duration of the load restriction period. These decisions, although supported in many cases by ground temperature and frost tubes data, are based for the most part on engineering judgment.

The Thawing Index approach (Rutherford et al. 1985) for predicting the beginning and duration of pavement thaw is based on analytical solutions for typical pavement sections and climatic conditions. Although this approach provides estimates for average conditions, it does not account for local variations, e.g. the existence of permafrost and extremes in solar radiation due to latitude, that could significantly alter ground temperature profiles. Ground temperature data in this case could provide improved assessment of the thermal regime for pavements during spring-thaw.

In this Chapter, ground temperature data for different sites in Alaska's Central and Northern Regions were collected and analyzed in order:

1. Develop appropriate models for predicting thaw initiation typical of pavements in those regions.

2. Develop appropriate methods for assessing thaw progression below the pavement surface.
3. Discuss the significance of thaw initiation and prediction models used in developing the current load restriction policy in Alaska.
4. Determine whether night-time refreezing occurs and assess the possibility of removing restrictions during the night.

4.2 FIELD TEST PROGRAM AND DATA COLLECTION

Ground temperature data at twelve pavement sites in the Central Region and six sites in the Northern Region were analyzed. These sites are listed in Tables 4.1. Ground temperature data for all sites in the Central Region were collected in 1993, whereas temperature data for the Northern Region sites were collected between 1988 and 1993. The variation of temperature with time and depth below ground surface, and also the progression of thaw depth with time, for selected sites, are illustrated in Figures 4.1- 4.8. Similar data for all the sites considered in this study are presented in Appendices D and E.

4.3 RESULTS

4.3.1 Ground Temperature Variation

Typical variation of ground temperature with depth and time is illustrated in Figures 4.1 - 4.2. The progression of thaw is not influenced by surface temperature fluctuations but seems to follow a trend similar to the average surface temperature, as shown in Figures 4.3 - 4.8. In this case, surface temperature is measured 3 inches below the surface using the top thermistor of the string of thermistors installed at a given site.

Table 4.1 Location of Sites

Central Region		Northern Region	
1.	Anchor Point	1.	Cantwell
2.	Bertha Creek	2.	Ester
3.	Chulitna	3.	Hilltop
4.	Glenn Highway	4.	Steese
5.	Glenn MP 53	5.	Tok
6.	Glenn Bragaw	6.	Peger Road
7.	New Seward		-
8.	Palmer/ Wassila		-
9.	Potter		-
10.	Rabbit Creek		-
11.	Summit Lake		-
12.	Tudor		-

The ground temperature data used in this study were all recorded periodically between approximately 9:00 a.m. and 1:00 p.m. Although no night-time temperature measurements were available for this study, limited data obtained by the AKDOT&PF Maintenance Engineer in the Central Region from sites recently equipped with automatic data loggers indicate that complete night-time refreezing of the base course was not common and could not therefore be considered an appropriate criterion for removing load restrictions during the night (Shook 1995). The only refreezing data for the Northern Region were obtained from day-time temperature measurements at the Peger road site. In this case refreezing occurred at one-day intervals between March 1 and April 2 during 1988. Refreezing data show that for thaw depths in the range of 3 to 18

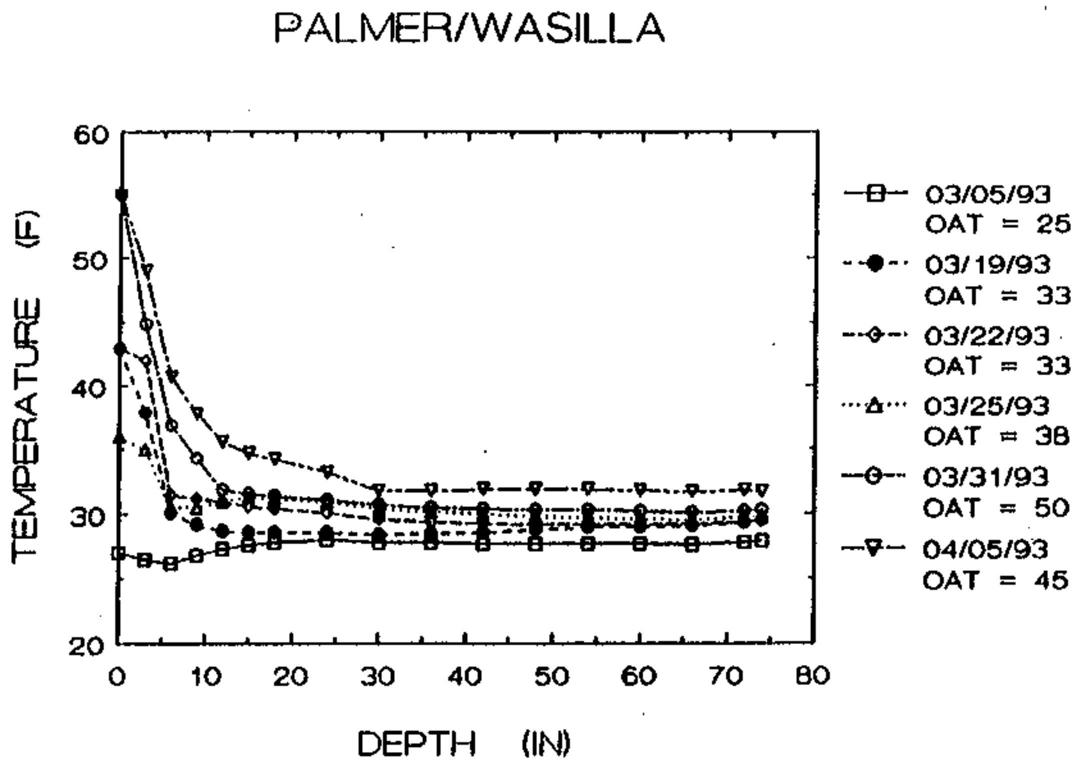

Figure 4.1 Ground Temperature Variation for the Palmer/Wasilla Site

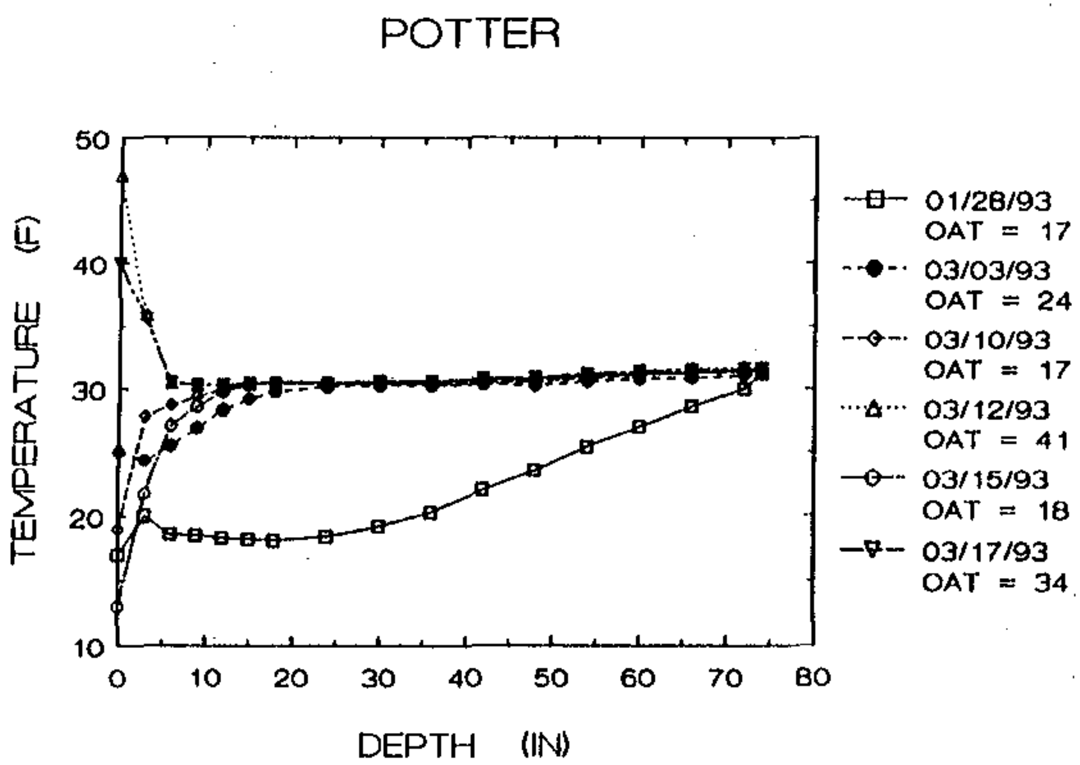

Figure 4.2 Ground Temperature Variation for the Potter Site

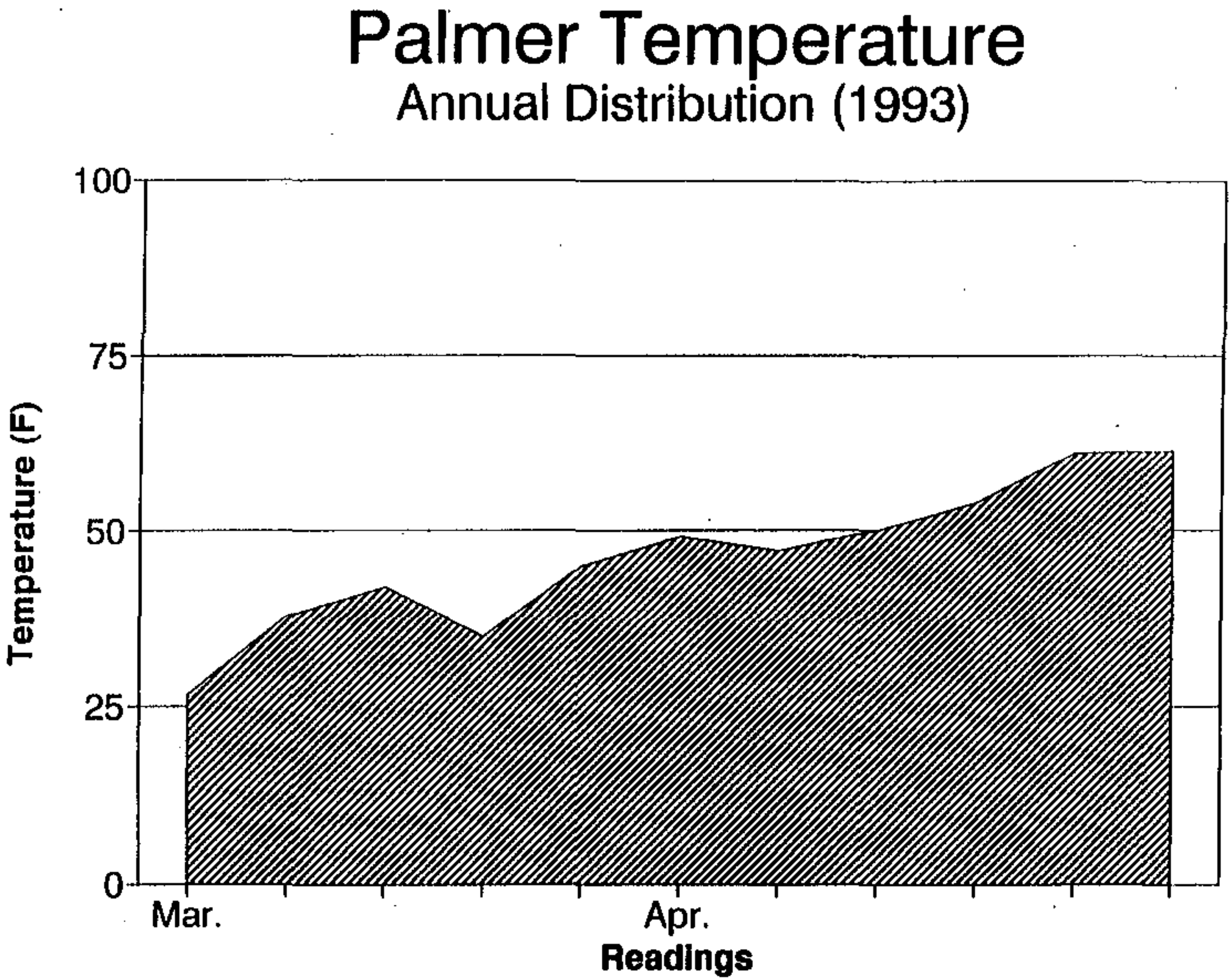

Figure 4.3 Pavement Surface Temperature Versus Frequency of Readings (Palmer Site - 1993 Data)

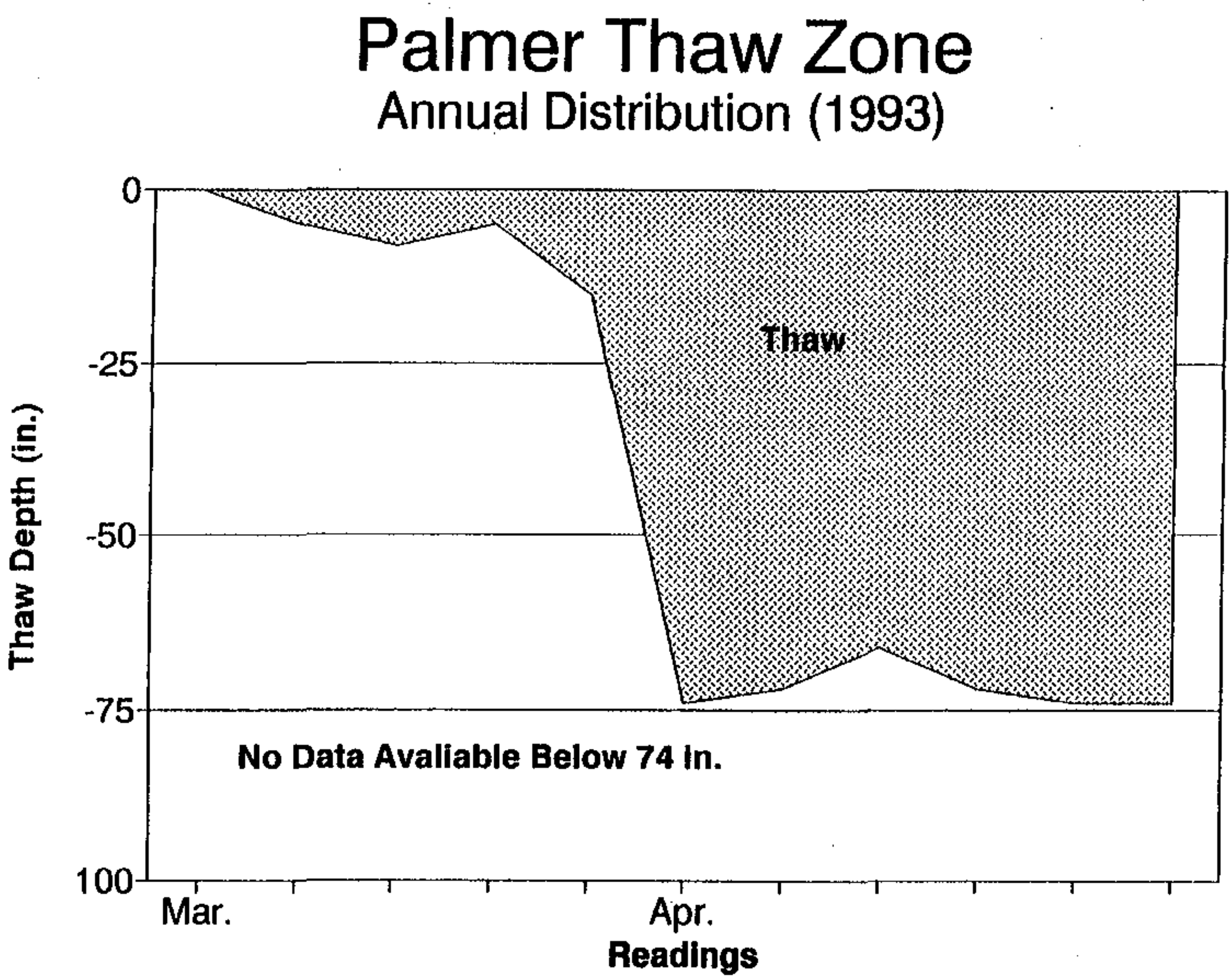

Figure 4.4 Thaw Depth Variation Versus Frequency of Readings
(Palmer Site - 1993 Data)

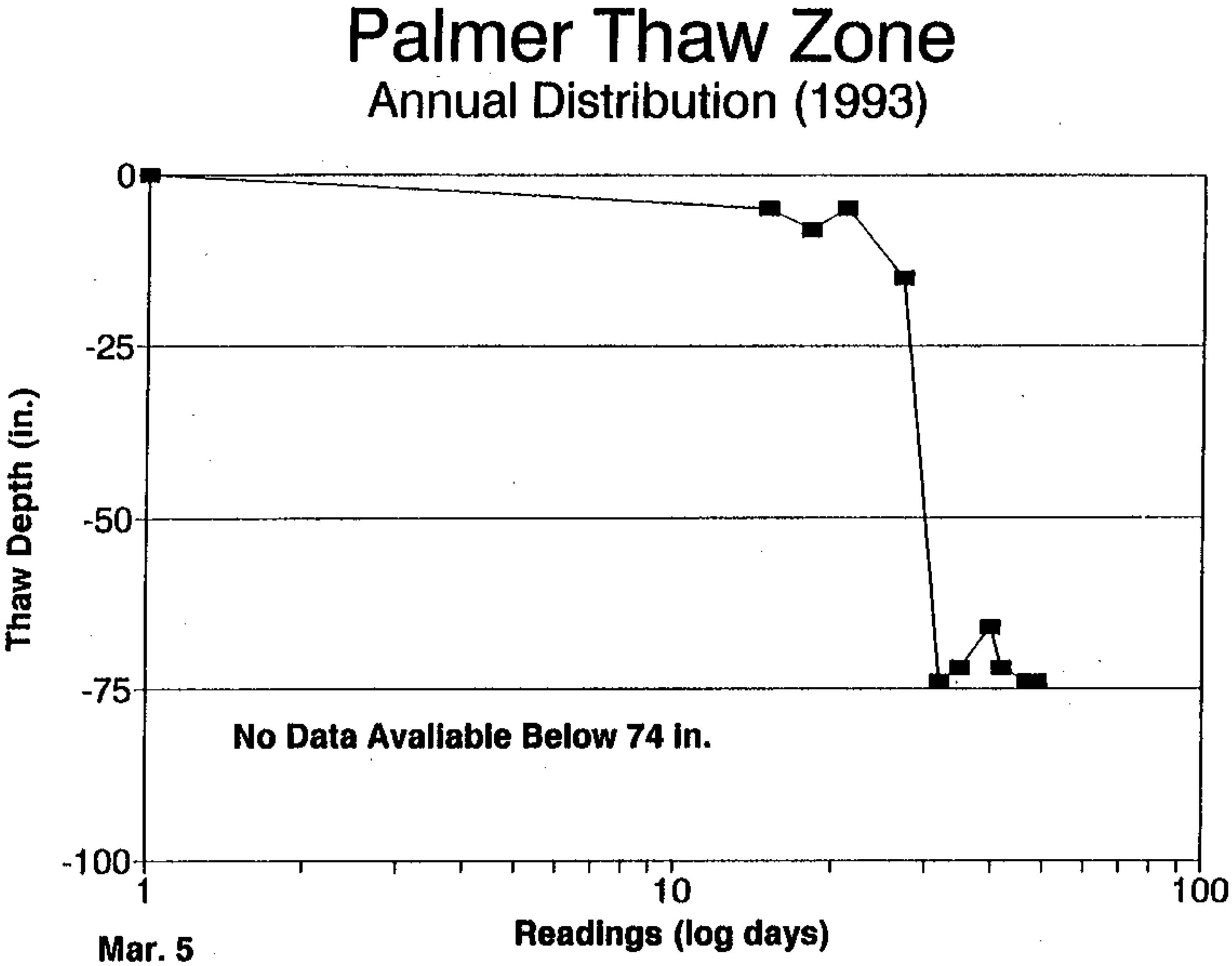

Figure 4.5 Thaw Depth Propagation (Palmer Site - 1993 Data)

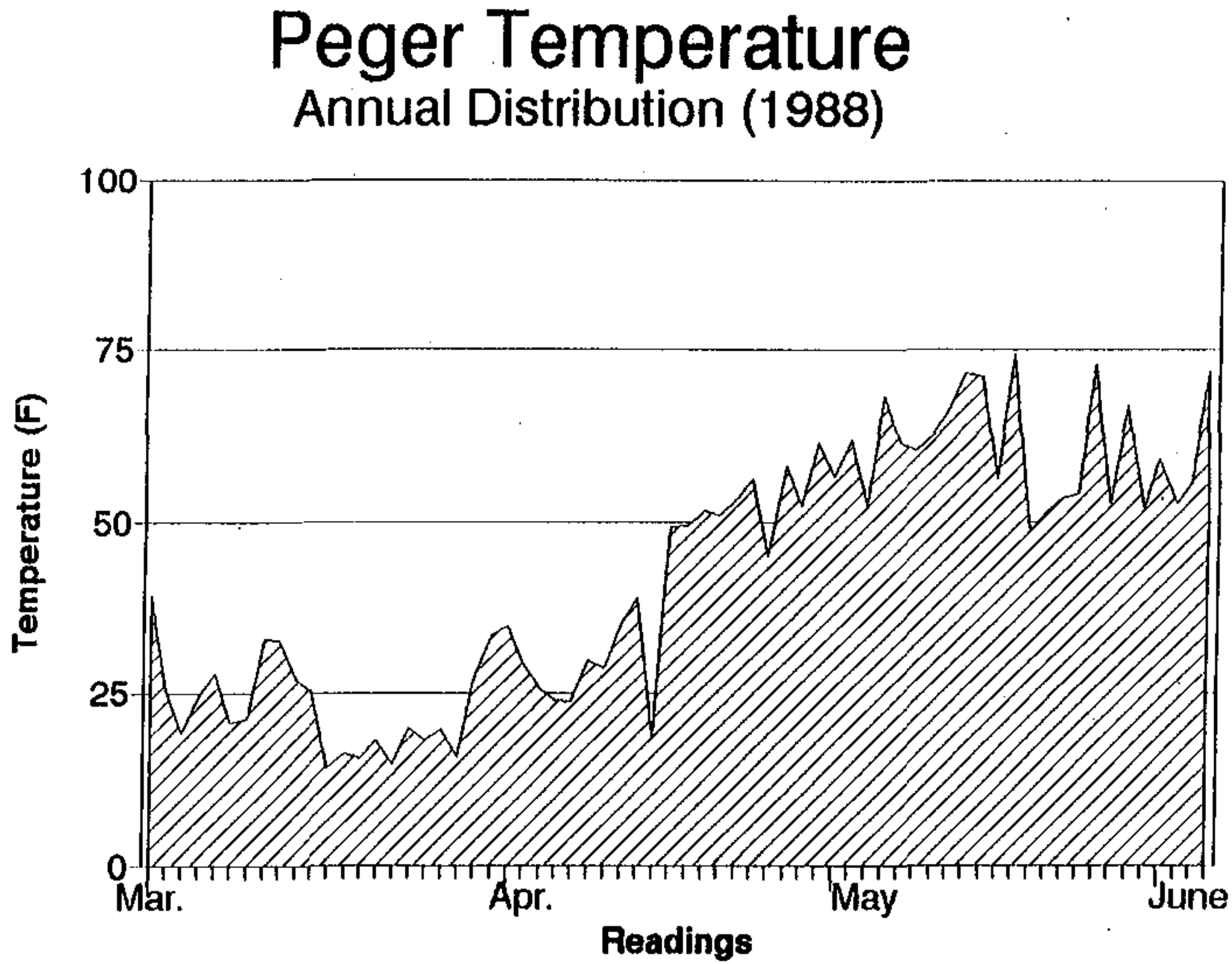

Figure 4.6 Pavement Surface Temperature Versus Frequency of Readings (Peger Site - 1988 Data)

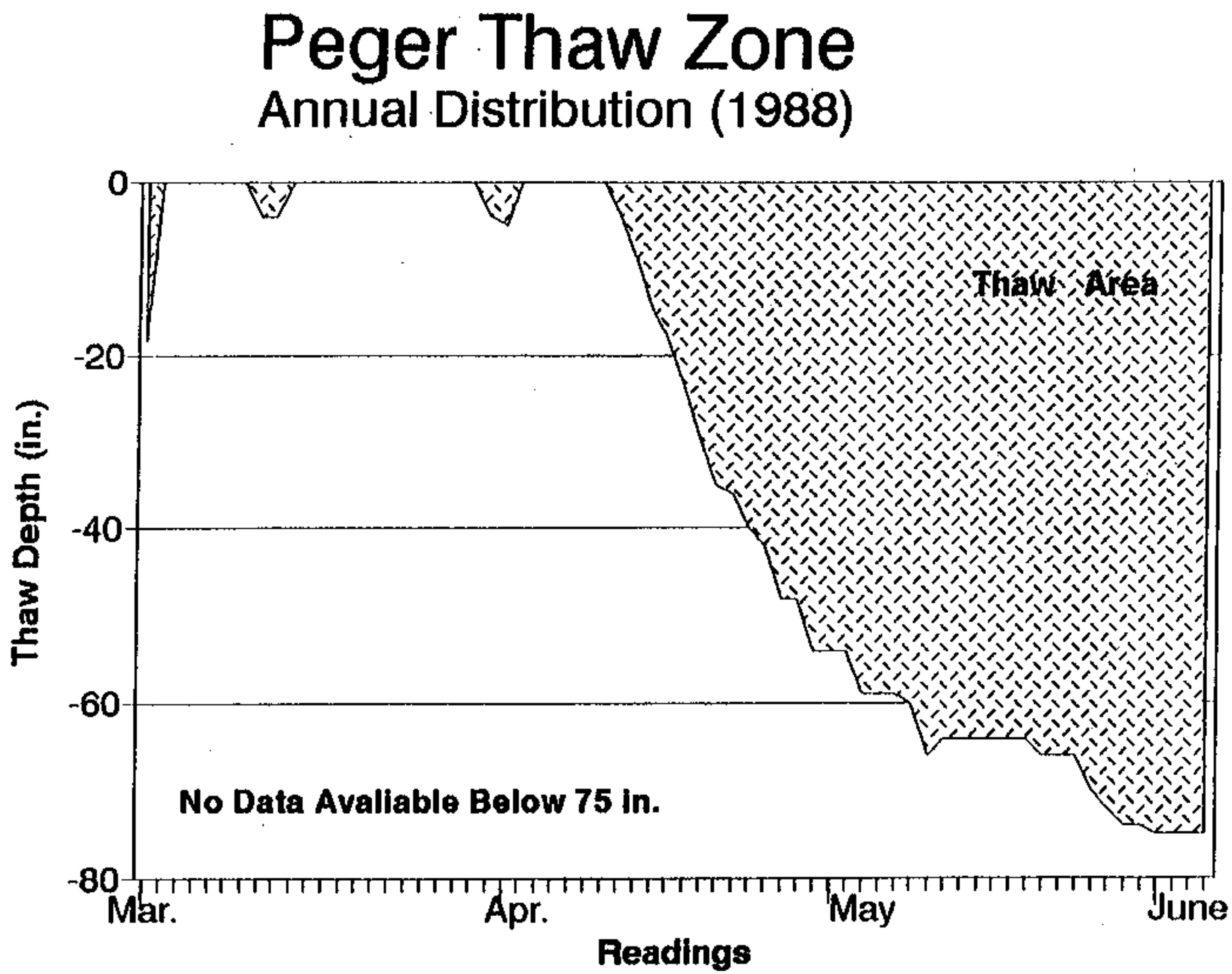

Figure 4.7 Thaw Depth Variation Versus Frequency of Readings (Peger Site - 1988 Data)

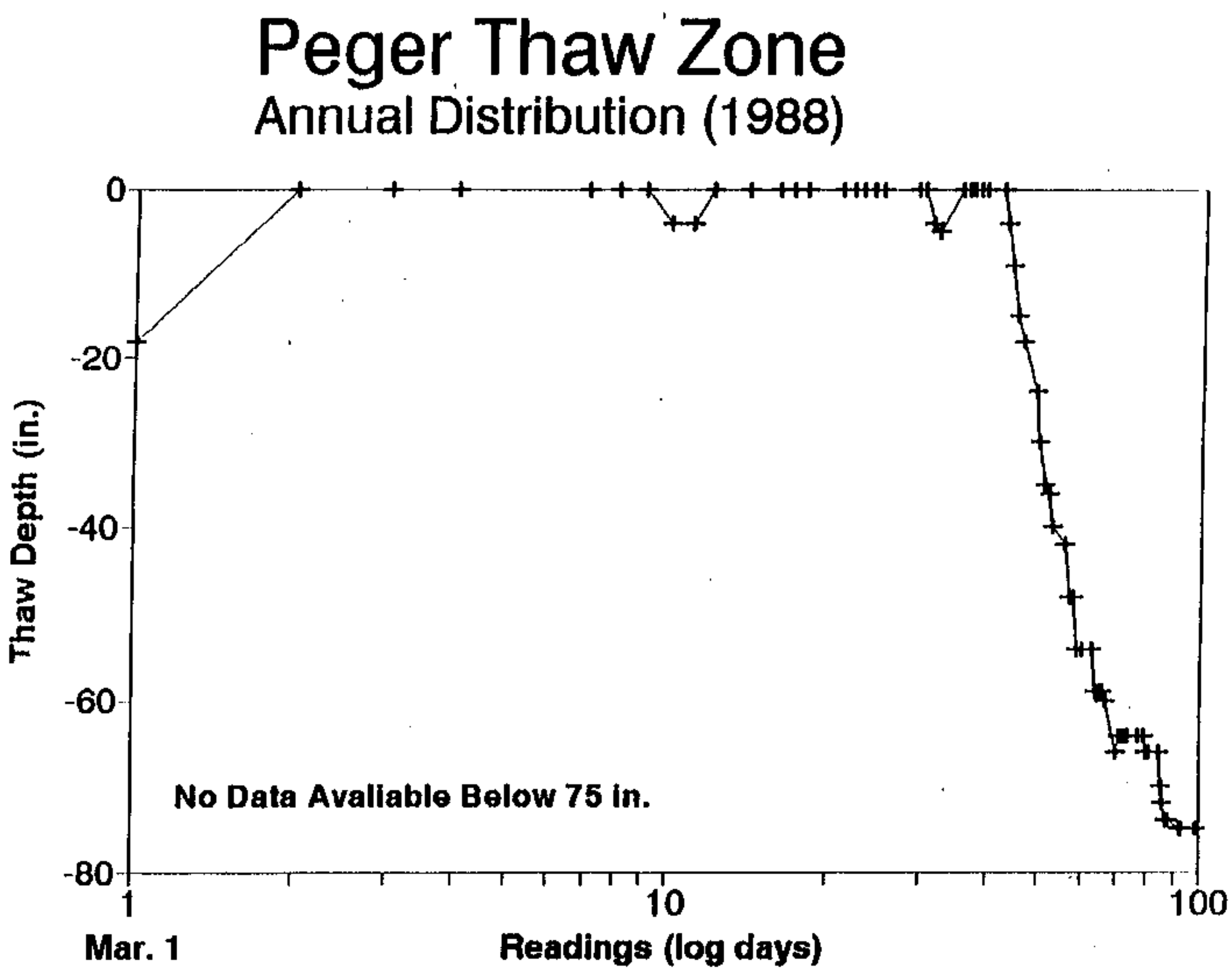

Figure 4.8 Thaw Depth Propagation (Peger Site - 1988 Data)

inches, the refreezing rate, expressed in terms of pavement temperature change below freezing, is approximately 40 ^{O}F-hr/inch. For example, assuming a 2 inch base thaw and initial pavement temperature equal to 40 ^{O}F, the pavement temperature needs to drop to 24 ^{O}F, in a period of 10 hours, in order for complete refreezing of the base to occur. Additional night-time temperature data need to be collected and analyzed in order to assess the frequency of ground refreezing and obtain better estimates of refreezing rates.

4.3.2 Thaw Initiation and Propagation

Thaw initiation and propagation in pavements depend essentially on air temperature, pavement materials, and layer thicknesses, in addition to site specific conditions such as absorbed solar radiation, long wave surface radiation, and surface convection (Goering and Zarling 1985; Rutherford 1988). Although analytical techniques could be used to estimate ground temperature distribution (Goering and Zarling 1985; Rutherford et al. 1985; Nixon 1986; Hromadka 1987), these techniques do not adequately account for variations in material properties and site location. In this study, ground temperature measurements were used to develop predictive models for thaw initiation and propagation below the pavement surface.

For thaw initiation in the base (measured at 3 inches below pavement surface) the distribution of temperature data was determined for the selected sites in the Central and Northern regions (Table 4.1). The frequency distribution for thaw initiation data is shown in Figures 4.9 and 4.10. The mean and standard deviation for these distributions, in terms of the number of days (starting March 1) required for thaw initiation, were determined to be:

Region	Mean	Standard Deviation
Central	29.09	14.72
Northern	40.35	16.00

The mean and the standard deviation were used to predict the probability of thaw initiation assuming a normal frequency distribution for thaw initiation data. The estimated thaw initiation date corresponding to a given probability of thaw initiation was estimated using normal distribution tables (Yoder and Witczak 1975). Results are summarized in Table 4.2. For example, the date corresponding to thaw initiation of 30 percent of the pavement temperature sites is March 20 in the Central Region and March 31 in the Northern Regions. It should be emphasized that the results in Table 4.2 depend on the mean and the standard deviation of the occurrence dates for thaw initiation. Improved predictions therefore necessitate continuous collection and upgrade of the data base for all instrumented sites.

Table 4.2 Probability of Thaw Initiation

Probability of Occurrence (Percent)	Thaw Initiation Occurs before the Date below (Central Region)	Thaw Initiation Occurs before the Date below (Northern Region)
10	March 10	March 19
20	March 17	March 26
30	March 20	March 31
40	March 25	April 4
50	March 29	April 9
60	April 2	April 14
70	April 6	April 18
80	April 10	April 23
90	April 17	April 30

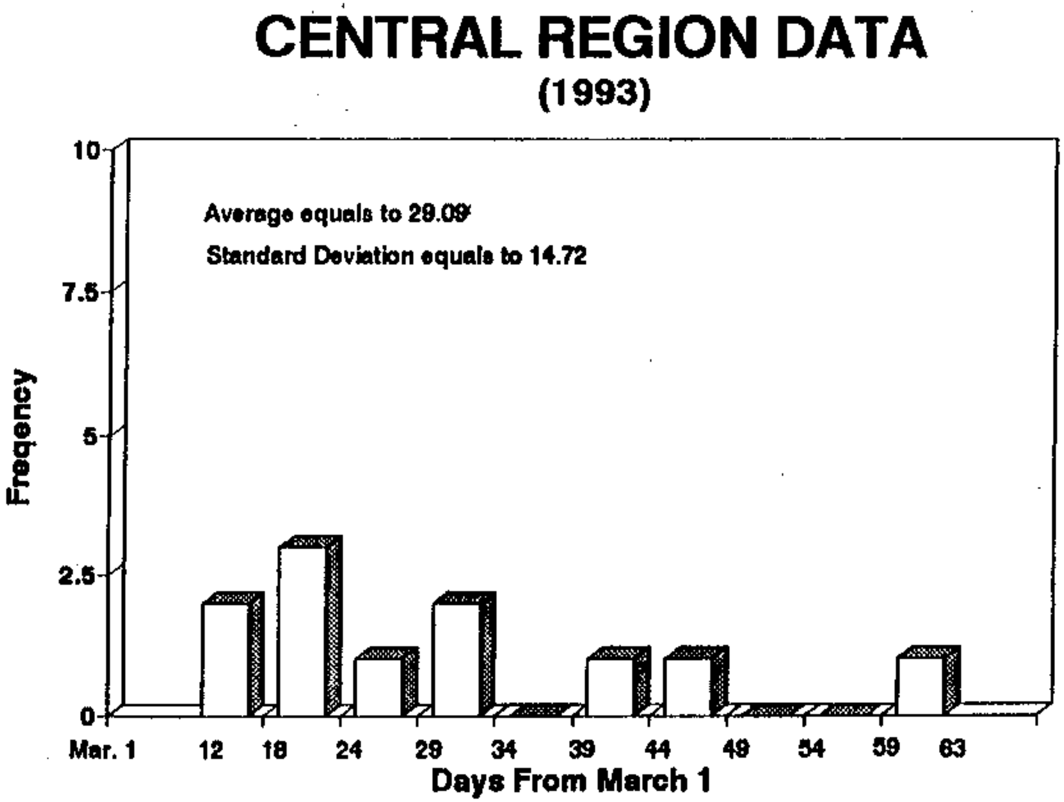

Figure 4.9 Frequency Distribution of Thaw Initiation Date (Central Region)

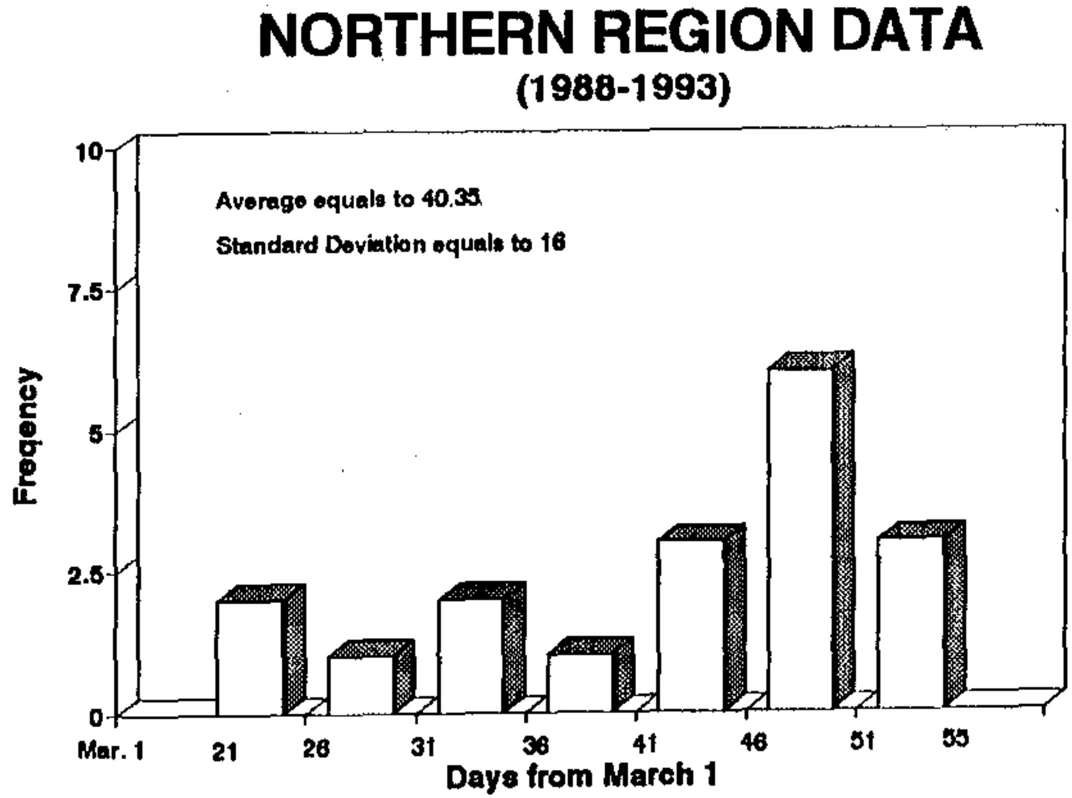

Figure 4.10 Frequency Distribution of Thaw Initiation Date (Northern Region)

Ground temperature data were also used to estimate the depth of thaw progression below the pavement surface for a given duration of thaw. Linear regression of the data is illustrated in Figures 4.11 and 4.12 and yield the following relations:

For the Central Region,

$$\text{Log } Z_t = 0.4224 + 0.8755 \text{ Log } T_d \qquad (R^2 = 0.78) \qquad (4.1)$$

or

$$Z_t = 2.645\ (T_d)^{0.8755} \qquad (4.2)$$

For the Northern Region,

$$\text{Log } Z_t = 0.4638 + 0.7564 \text{ Log } T_d \qquad (R^2 = 0.64) \qquad (4.3)$$

$$Z_t = 2.909\ (T_d)^{0.7564} \qquad (4.4)$$

where

Z_t = Depth of thaw (inches)

T_d = Duration of thaw (days)

For example, using the above equations, the time required for thawing to propagate 3 feet below the pavement surface following thaw initiation would be equal to 20 and 28 days for the Central and Northern regions respectively.

4.4 LOAD RESTRICTION CONSIDERATIONS

The proposed thaw initiation and propagation models could be used for assessing appropriate dates for application and removal of load restrictions. The following criteria for "normal" and "critical" site conditions are suggested:

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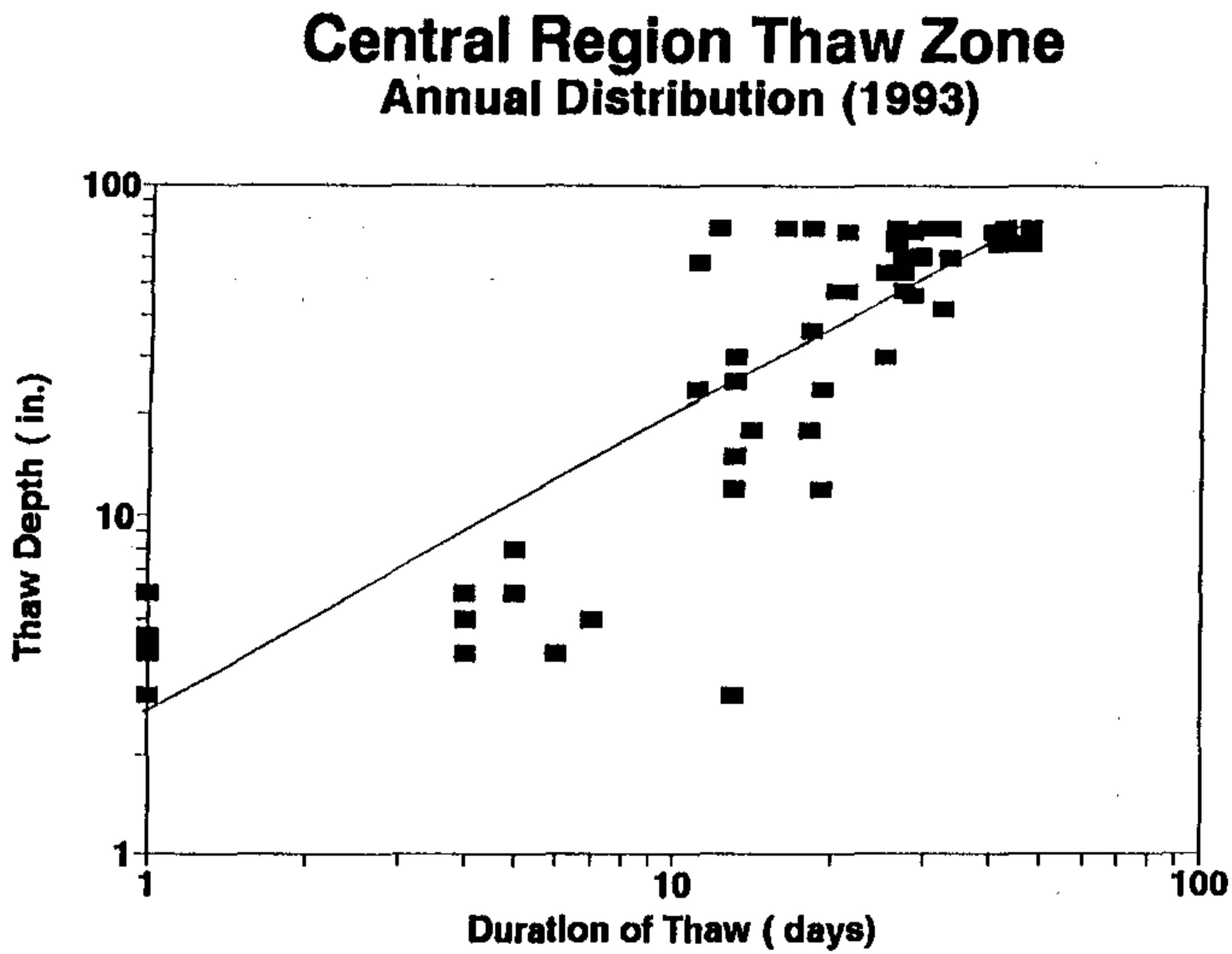


Figure 4.11 Variation of Thaw Depth with Time (Central Region)

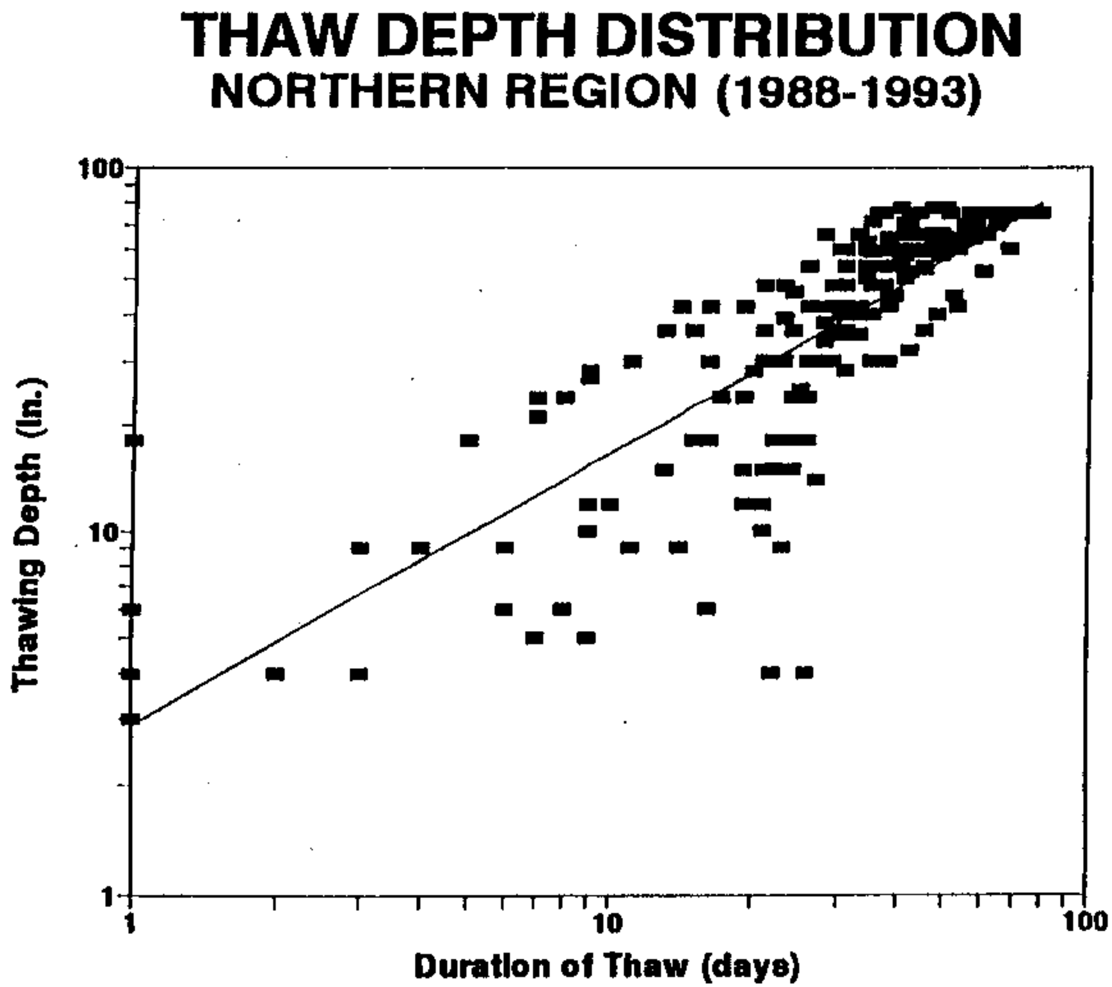


Figure 4.12 Variation of Thaw Depth with Time (Northern Region)

4.4.1 Criteria for "Critical" and "Normal" Site Conditions

Critical Conditions

For site conditions judged by the Maintenance Engineer as having a significant number of weak pavement sections, as determined by excessive cracking, rutting, patching, and drainage problems, the suggested date for imposing load restrictions corresponds to a 30 percent probability of thaw initiation. In other words, 30 percent of the pavement sections would have started to thaw by the time load restrictions are imposed.

According to current load restriction practice summarized in Chapter Two, most state agencies remove load restrictions when the pavement becomes "dry" based on observations, engineering judgment, and in some cases deflection measurements. In Alaska, this seems to occur when thaw depth is between 3 feet and 4 feet (Shook 1995). Assuming that load restrictions could be removed when the thaw depth is about 3.5 feet, the corresponding thaw duration period could be determined from equations 4.1 - 4.4. In order to determine the date for removing restrictions, the duration of thaw should be started from a reference date corresponding to an "acceptable" probability level for thaw initiation. In this case, the use of the date corresponding to a 70 percent probability level is suggested. This means that at least 70 percent of the pavement sections would have thawed to a depth of 3.5 feet or greater by the time load restrictions are removed.

Normal Conditions

Fo. site conditions judged by the Maintenance Engineer as "average" or "normal" and where the number of weak pavement sections is minimal as determined, for example, by observed pavement distress, performance history, and structural strength, a 50 percent probability level is suggested for estimating thaw initiation and the corresponding date for imposing load restrictions. It is also suggested that restrictions be removed when at least 50 percent of the pavement sections experience a minimum of 3.5 feet of thawing.

4.4.2 Applications of Proposed Criteria

The proposed criteria are applied to estimate the timing and removal of load restrictions for both the Central and Northern Regions as follows:

Central Region

For "critical" site conditions, restrictions would start on March 20 and be removed on April 29; for "normal" site conditions, the dates for application and removal of restrictions would be March 29 and April 21, respectively.

Northern Region

For "critical" site conditions, restrictions would be applied on March 31 and removed May 12; for "normal" site conditions, the application and removal dates for load restriction would be April 9 and May 8, respectively.

Limitations

1. It should be emphasized that the above dates are determined based on the limited ground temperature data that were available for this study. More temperature data will be needed to improve and upgrade the proposed predictive models.
2. The probability levels associated with the suggested criteria could be changed based on engineering judgment and experience.

3. The proposed criteria assume that the thaw initiation dates have a normal frequency distribution. This assumption needs to be verified as more data become available.
4. The proposed criteria are not site specific. In other words, data from a number of sites are used to estimate load restriction periods. Improved criteria could be developed if load restrictions for a given road segment are based on temperature data from representative pavement sites along the road under consideration.

4.5 SUMMARY

Ground temperature data from a number of pavement sites in the Central and Northern regions were analyzed. The progression of thaw did not seem to be influenced by surface temperature fluctuations but followed a trend similar to the average surface temperature. No conclusion could be made concerning the significance of night-time refreezing on load restriction since night-time ground temperature data were not available for this study. According to the Maintenance Engineer in the Central Region, ground refreezing is scarce and insignificant. Limited data using day-time ground temperature data from Northern Region sites indicate that refreezing could occur and the refreezing rate is estimated to be 40 ^{o}F-hr/in.

Ground temperature data were also used to suggest probabilistic criteria for timing and removal of load restrictions. These criteria were based on thaw initiation and progression below the pavement surface for both "critical" sites and "normal" sites. The limitations of these criteria were discussed particularly in relation to upgrading the proposed models as more ground temperature data become available.

CHAPTER FIVE

EFFECT OF TIRE PRESSURE ON PAVEMENT DAMAGE

5.1 INTRODUCTION

Tire inflation pressure has been recognized by a number of investigators to be a major factor of influence on pavement response and performance (Papagianakis and Haas 1986; Haas and Papagianakis 1986; Hudson and Seeds 1988; Hansen et al. 1989; Seebaly 1992; Grau 1993; Smith 1993). Accelerated highway pavement damage during the last 50 years has been attributed primarily to increased tire inflation pressure in addition to other factors such as increased truck traffic and axle loads (Haas and Papagianakis 1986; Eisenmen and Hilmer 1987). Hudson and Seeds (1988) proposed a system for estimating changes in flexible pavement design as a result of increased truck loading and tire pressure. Work by Hansen et al. (1989) shows that increased tire inflation pressure significantly increases the horizontal tensile strains at the bottom of the pavement layer thereby reducing fatigue life. In addition, increased inflation pressure also increases compressive strains in the asphalt layer which could result in excessive rutting (Papagianakis and Haas 1986). Experimental studies on field pavement sections were performed by Grau (1993) and Smith (1993) to evaluate the effects of tire pressure on pavement damage. These results indicate that high tire pressure increases pavement distress and is more damaging than low tire pressure. For example, the ratio of low tire pressure (40 psi) traffic to high tire pressure (100 psi) traffic associated with a given observed pavement distress ranges from 1.5 to 21. Seebaly (1992) reported that the tire inflation pressure did not significantly change the calculated and measured strains for pavements with 6 inches and 10 inches asphalt concrete thicknesses.

In this Chapter, the influence of tire pressure on pavement damage for pavements under spring-thaw weakening conditions was investigated. Specifically, the possibility of reducing tire pressure while maintaining full legal load limits during spring was addressed. Multilayer elastic analyses were conducted on typical pavement sections using the computer program ELSYM5 (Ahlborn 1972) to evaluate the influence of wheel load magnitude, tire pressure, and thaw depth on potential damage to the pavement structure.

5.2 ANALYSIS OF PAVEMENT SECTIONS

Pavement sections in Alaska generally consist of two inches of asphalt concrete surface and 40 inches of a granular base/subbase layer with base thicknesses in the range of 6 to 12 inches. These sections overlay natural subgrade conditions. The pavement section selected for these analyses had a 2 inch asphalt concrete surface layer and a 40 inch granular section representing average base/subbase conditions. The natural subgrade was assumed to be either fine-grained or coarse-grained. For spring-thaw conditions, the pavement section extends to the depth of the thaw line below which rigid boundary conditions were assumed. For summer conditions, the natural subgrade was considered to be semi-infinite. In this study, thaw depth values are measured from the pavement surface. Multilayer elastic analyses were performed on the selected sections using the computer program ELSYM5 (Ahlborn 1972). A summary of the properties of the pavement materials used in the analysis is presented in Table 5.1. Pavement response was determined for a range of wheel load magnitude, tire pressure, and thaw depth values (Table 5.2). Pavement response parameters considered include surface deflections, flexure strains in the asphalt concrete, vertical stresses and strains on top of the base/subbase and subgrade. The potential damage to the pavement was assessed by examining the performance of the pavement surface, granular base, and subgrade using the appropriate response parameter and limiting criterion.

Table 5.1 Properties of Selected Pavement Sections

Layer	Thickness (inches)	Elastic Modulus (Spring) (psi)	Elastic Modulus (Summer) (psi)	Poisson's Ratio (Spring)	Poisson's Ratio (Summer)
Asphalt Surface	2.0	1.5×10^6	0.50×10^6	0.20	0.30
Base/ Subbase	40	15,000	30,000	0.40	0.35
Subgrade	-	7,500	15,000	0.45	0.40

Note:
1) Subgrade is assumed to be semi-infinite for summer loading conditions
2) Subgrade thickness is limited by thaw depth when underlaid by a rigid boundary for spring loading conditions.

Table 5.2 Pavement Variables Considered

Wheel Load (lbs)	Tire Pressure (psi)	Thaw Depth (inches)
6,000, 9,000, 13,500	60, 70, 80, 90, 100, 110	4, 10, 20, 40, 60, 80

5.2.1 Limiting Criteria

Fatigue failure in the asphalt concrete surface was analyzed using the Asphalt Institute fatigue relationship as defined by Equation 2.12 (Shook et al. 1982). The dynamic modulus, E_d, of the asphalt concrete surface was estimated from Equation 2.13

using a loading frequency of 10 Hz and spring and summer temperatures equal to 40 ^{O}F and 70 ^{O}F, respectively.

The base/subbase failure criterion was assumed to depend on the maximum vertical stress and layer modulus according to Equation 2.14 (Ullidtz 1987). The same criteria were used for the coarse-grained subgrade. For fine-grained subgrade the Asphalt Institute defined by Equation 2.16 was used (Shook et al. 1982).

5.2.2 Assessment of Pavement Damage

Pavement damage during spring-thaw is evaluated by comparing the damage induced by one application of a given wheel load and tire pressure during spring conditions to that induced by a standard 9,000 lb. wheel load during a summer "reference" condition. In this case, a damage factor, RD, is defined as the ratio of the number of repetitions of the standard 9,000 lb. wheel load required to cause pavement failure in the summer "reference" condition to the number of spring load applications of a given wheel load that will cause failure of the thaw-weakened pavement. In other words, one repetition of the given wheel load during spring will cause the same amount of damage to the pavement as "RD" applications of the standard wheel load during the summer "reference" condition.

In this study, RD values for the asphalt concrete surface, the base/subbase layer, and the subgrade are calculated using the appropriate response parameter and limiting criterion. Pavement response for the summer "reference" condition is determined for a standard 9,000 lb. wheel load with 110 psi tire pressure and is summarized in Table 5.3.

5.3 RESULTS

Results of analyses illustrate 1) the effect of tire pressure on pavement response (Figures 5.1 - 5.6), 2) number of load repetitions to failure associated with the critical response parameters and limiting criteria for the pavement structure (Figures 5.7 - 5.10), and 3) damage during spring loading versus summer normal conditions (Figures 5.11 - 5.14). In this Chapter, only the results for the 9,000 lb. wheel load are shown. Loading conditions corresponding to the 6,000 lb. and 13,500 lb. wheel loads are presented in Appendix E. Results of the analyses are summarized as follows:

1. Surface deflections increase with increasing thaw depth (Figure 5.1). Maximum deflections seem to occur at thaw depths equal to 60 inches and beyond which the increase in surface deflections is very small. Surface deflections are also influenced by tire pressure. The maximum deflections increase by about 17 percent with a corresponding tire pressure change from 60 psi to 110 psi.
2. Tensile strains on the underside of the pavement asphalt layer increase with increasing thaw depth (Figure 5.2). Maximum values are attained after about 20 inches of thawing. It is interesting to note that in this case although these strains seem to level off, the surface deflections continue to increase until the thaw depth reaches about 60 inches. The tensile strains also increase with increasing tire pressure. For example, an increase in tire pressure from 60 psi to 110 psi for the 9,000 lb. wheel load will increase the tensile strains from 300 micro-in/in to about 450 micro-in/in. The corresponding fatigue life will decrease from an estimated 1.0×10^6 repetitions to about 2.0×10^5 repetitions (Figure 5.7).
3. Vertical strains and stresses on top of the granular layer are largest at the onset of pavement thawing. These are most critical for higher tire pressure. The vertical strains and stresses decrease with increasing thaw depth and seem to stabilize at thaw depths of 20 inches (Figures 5.3 - 5.4).

4. Vertical strains and stresses on top of the subgrade remain essentially unaffected by changes in tire pressure. Both vertical strains and stresses seem to decrease with increased depth of thawing in the subgrade (Figures 5.5 - 5.6).
5. Repetitions to failure determined for the range of thaw depth values considered indicate that the granular layer exhibits the lowest repetitions to failure (Figure 5.8) in comparison with the asphalt concrete surface (Figure 5.7) or the subgrade (Figures 5.9 - 5.10) and is, therefore, the most critical. The subgrade, on the other hand, is the least critical since it requires the largest number of repetitions to failure.
6. Damage factors for the asphalt concrete layer increase with increasing tire pressure and wheel load magnitude (Figure 5.11). The range of variation of the damage factors for the 9,000 lb. wheel load is between 0.5 and 2 depending on tire pressure. These values correspond to a thaw depth of 20 inches, which is most critical for tensile strains in the asphalt surface.
7. Damage factors for the base/subbase are at maximum at the onset of base thawing and decrease as thawing progresses until the thaw depth reaches 20 inches (Figure 5.12). For thaw depths greater than 20 inches and a given tire pressure, the damage factor remains essentially constant. The variation of the damage factor during the first 20 inches of thaw ranges between 1 and 11 when the applied load is 9,000 lb. Higher tire pressure and wheel loads will result in larger damage factors. A reduction of tire pressure from 110 psi to 60 psi reduces the maximum damage factor from 11 to 4.

8. Damage factors are greatest for the subgrade (Figures 5.13 - 5.14). These factors increase significantly with increase in wheel load magnitude but are essentially unaffected by tire pressure. Larger values of damage factors are observed for shallower thaw depths in the subgrade. An increase in wheel load magnitude from 9,000 lbs to 13,500 lbs increases the damage factors from about 30 to 130.

9. Identification of the critical layer that is most susceptible to failure during spring-thaw weakening should be based on the number of load repetitions associated with excessive distress or failure according to appropriate limiting criteria. The use of damage factor to indicate the most critical pavement layer could be misleading, since a larger damage factor does not necessarily imply a smaller number of load repetitions to failure. For example, although the subgrade has the largest damage factor in comparison with the base/subbase or the asphalt concrete surface, it can resist more load repetitions under spring-thaw weakening conditions (Figures 5.9 - 5.10) and is, therefore, the least critical. In this case, limiting the damage factor of the most critical layer in the pavement structure could be useful in determining spring load restrictions (Coetzee and Connor 1994).

10. Pavement damage during spring-thaw weakening can be minimized by reducing the tire pressure. For a standard wheel load magnitude of 9,000 lb., maximum damage to the granular layer (critical pavement layer in this case) is reduced by a factor of 3, approximately, when the tire pressure is reduced from 110 psi to 60 psi (Figure 5.11). This occurs when thawing is less than 10 inches into the base/subbase layer. For larger thaw depths, however, the influence of tire

pressure becomes less significant and the damage factor of the granular layer varies between 1.5 and 1 for tire pressures of 110 psi and 60 psi, respectively.

11. In Alaska, spring load restrictions are limited, at present, to 75 percent of the legal axle load. Of particular interest in this case is whether it is possible to use tire pressure reduction as an alternative to axle load restriction. A comparison of pavement damage factors associated with axle load restriction and tire pressure reduction is presented in Tables 5.4 and 5.5. It is estimated, based on these results, that reducing the tire pressure for a standard 9,000 lb. wheel load from 110 psi to 77 psi is equivalent in terms of pavement damage to applying 75 percent spring load restriction limit while keeping the tire pressure equal to 110 psi.

Table 5.3 Summary of Pavement Response Parameters and Corresponding Repetitions to Failure for the Summer Reference Condition

Pavement Layer	Response Parameter Value	Repetitions to Failure
<u>Asphalt Concrete</u>		
Radial Strain	$- 4.54 \times 10^{-4}$ in/in	4.54×10^{5}
<u>Base/Subbase</u>		
Vertical Stress	77.2 psi	1.85×10^{4}
Vertical Strain	20.8×10^{-4} in/in	-
<u>Subgrade</u>		
(Coarse-Grained) Vertical Stress	1.75 psi	3.53×10^{8}
(Fine-Grained) Vertical Strain	1.13×10^{-4} in/in	6.64×10^{8}

Note: Tensile stresses and strains are negative; compressive stresses and strains are positive.

Table 5.4 Influence of Wheel Load Magnitude on Critical Damage Factor for 4 inches Thawing into the Base/Subbase

Wheel Load (lbs) (110 psi Tire Pressure)	Restricted Load (% of Standard 9,000 lbs)	Damage Factor (Base/Subbase)
6,000	66.7	5
6,750	75	6.5
9,000	100	11
13,500	150	23

Table 5.5 Influence of Tire Pressure on Critical Damage Factor for 4 inches Thawing into the Base/Subbase

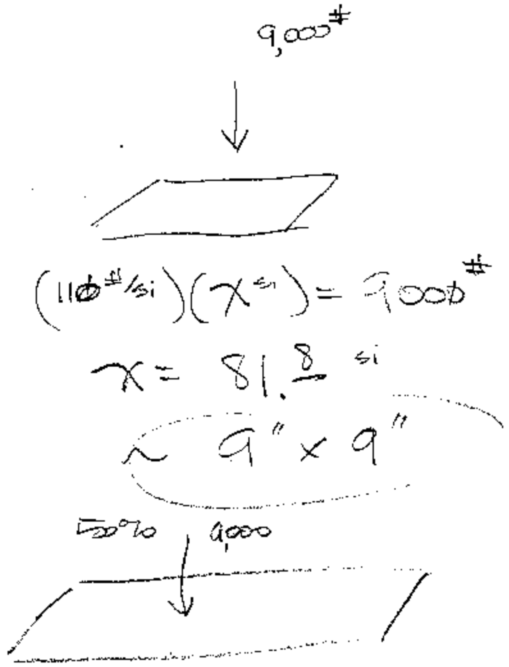
Tire Pressure (psi)	Tire Pressure Reduction (% of 110 psi)	Damage Factor (Base/Subbase)
70	64	5.5
77	70	6.5
80	73	7.0
90	82	8.5
110	100	11

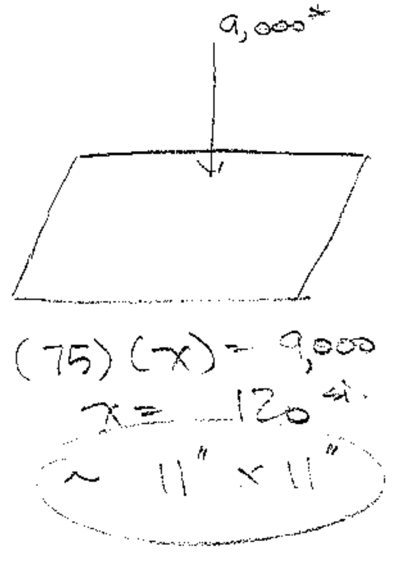
5.4 SIGNIFICANCE OF THE RESULTS

The results presented in this Chapter show that for thin pavements (2 inch asphalt surface) maximum damage during spring-thaw occurs when thawing initiates in the base. Results also show that damage in the granular base represents the most significant contribution to overall pavement damage. Fatigue of the asphalt concrete surface is also critical but to a lesser degree. The subgrade has the least critical damaging effect on the pavement sections analyzed. The results also show that maximum base damage does not coincide with maximum surface deflection. This indicates that the application of spring load restrictions should be based on the vertical stress on top of the granular base rather than maximum surface deflections. The critical damaging period occurs during the first 20 inches of thawing. No further reduction in damage is predicted for greater thaw depths. Reducing tire pressure could have a significant effect on limiting pavement damage. For example, a reduction of tire pressure from 110 psi to 60 psi for a 9,000 lb. wheel load would reduce maximum damage by a factor of 3.

Results also indicate that reducing the tire pressure for a standard 9,000 lb. wheel load from 110 psi to 77 psi is equivalent in terms of pavement damage to applying 75 percent spring load restriction limit but keeping the tire pressure equal to 110 psi.

It should be emphasized that the results obtained are limited to selective pavement geometries and material properties using multilayer elastic analyses. The analyses use the simplifying assumption that the contact pressure between the tire and the pavement surface is equal to the tire inflation pressure. Additional studies should be conducted to investigate the influence of inflation pressure on tire contact pressure and corresponding pavement damage. Field verification of analytical predictions is required before final recommendations on tire pressure can be established.





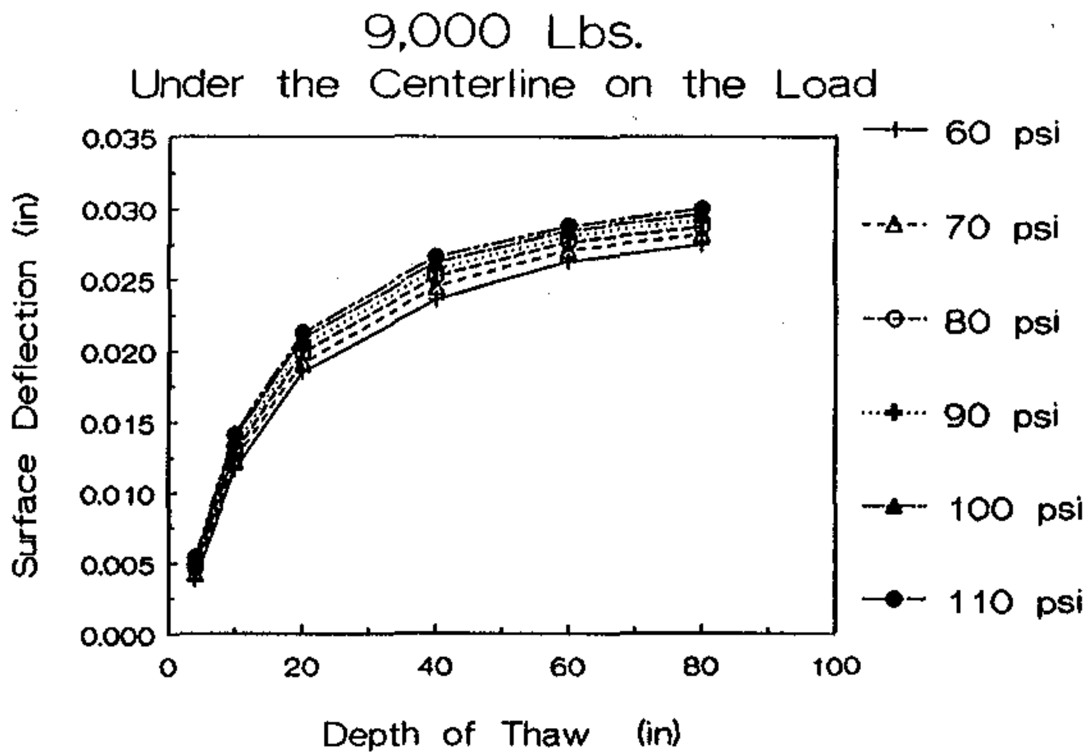


Figure 5.1 Variation of Surface Deflection with Thaw Depth and Tire Pressure (9,000 lb Wheel Load)

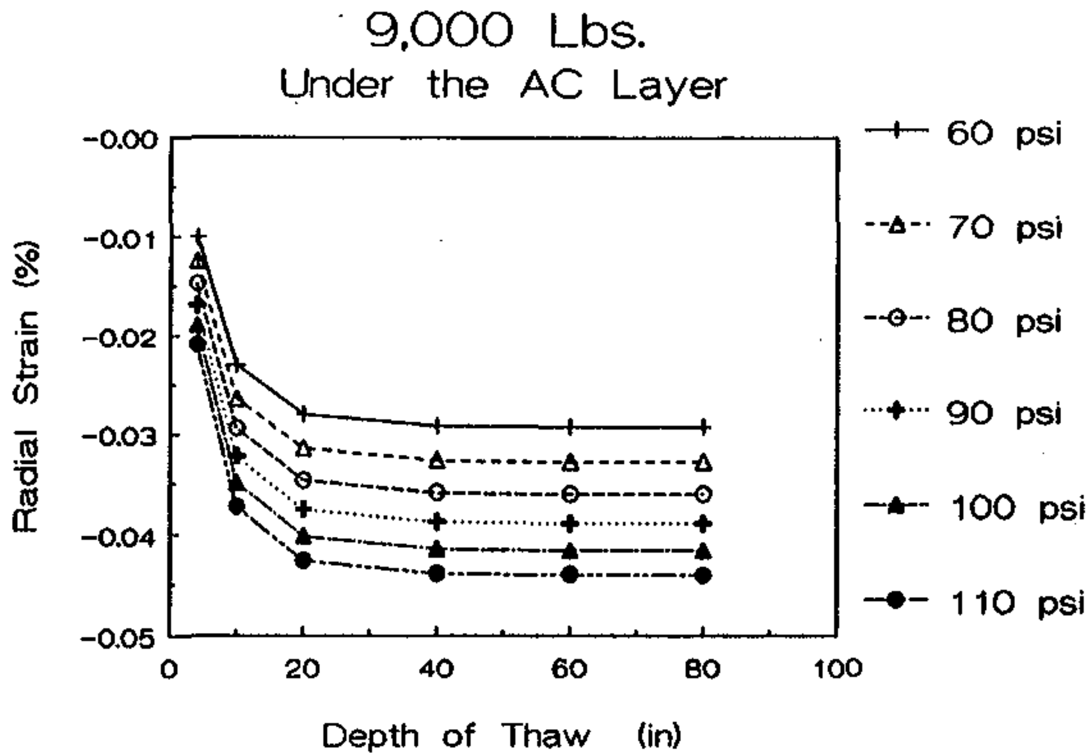


Figure 5.2 Variation of Radial Strain on the Underside of the Asphalt Concrete Layer with Thaw Depth and Tire Pressure (9,000 lb Wheel Load)

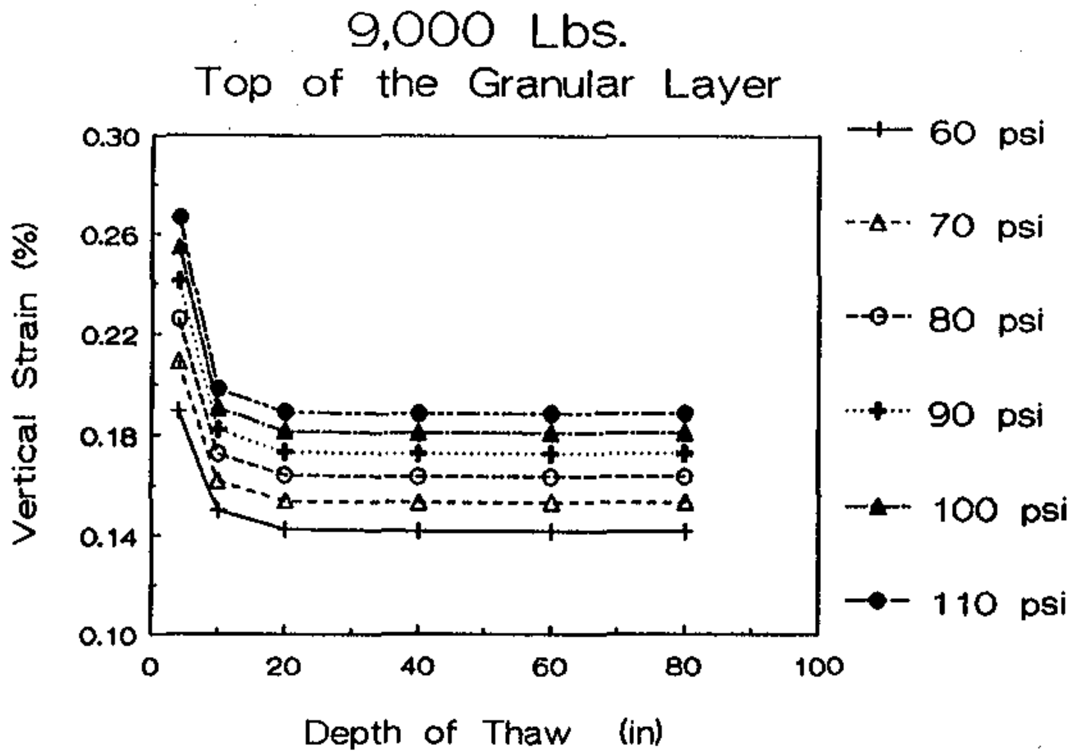


Figure 5.3 Variation of Vertical Strain on Top of the Base Course with Thaw Depth and Tire Pressure (9,000 lb Wheel Load)

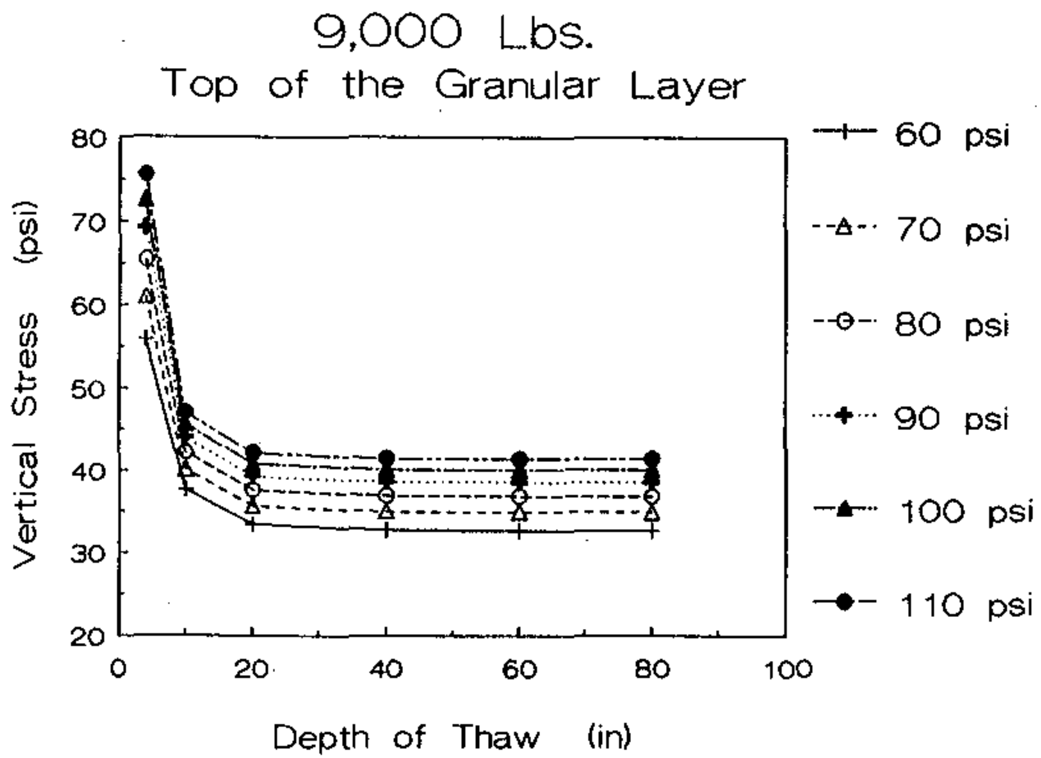


Figure 5.4 Variation of Vertical Stress on Top of the Base Course with Thaw Depth and Tire Pressure (9,000 lb Wheel Load)

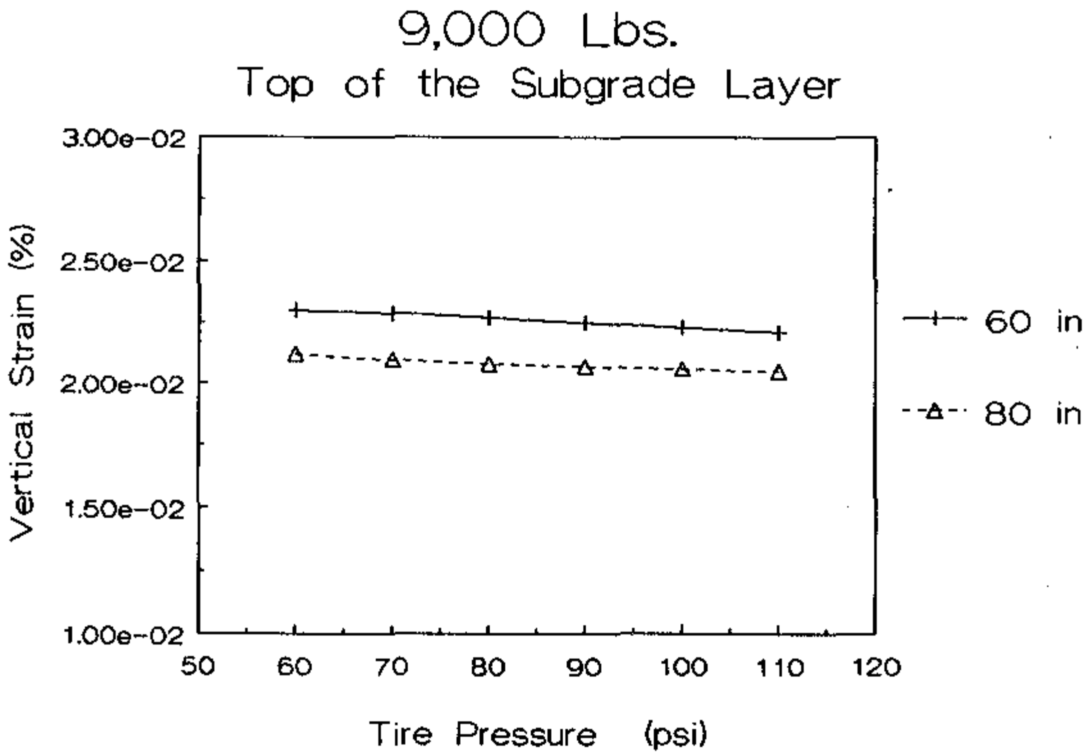


Figure 5.5 Variation of Vertical Strain on Top of the Subgrade with Tire Pressure for Thaw Depth of 60 inches and 80 inches (9,000 lb Wheel Load)

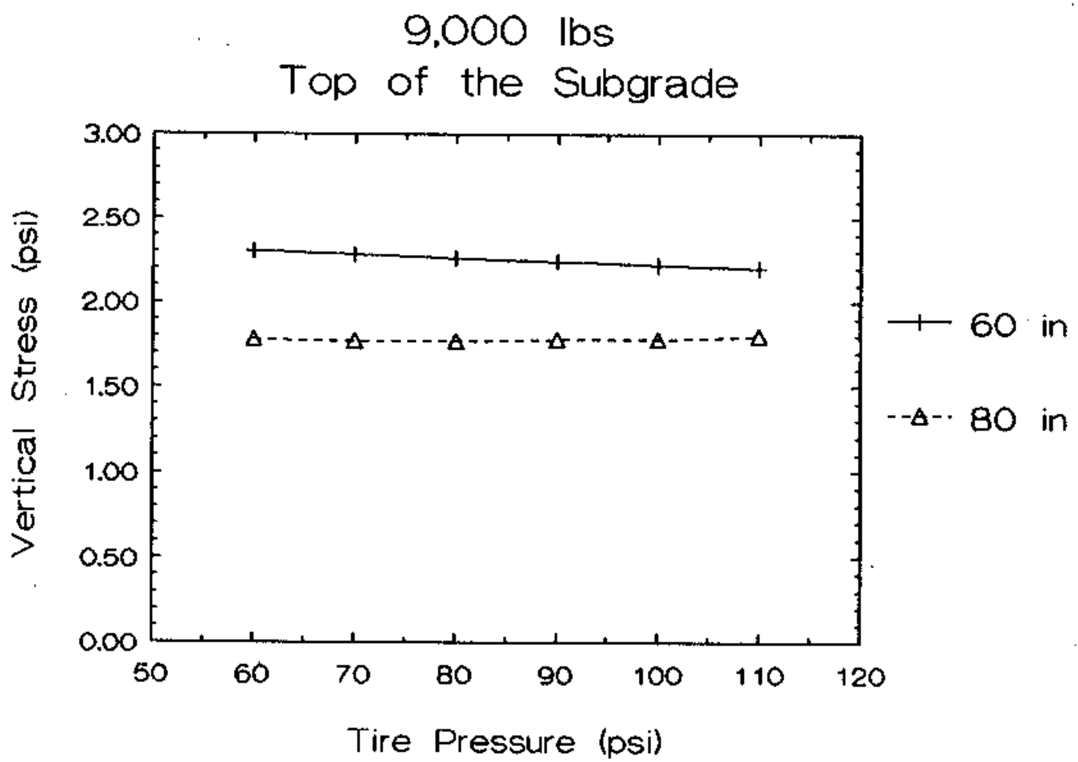


Figure 5.6 Variation of Vertical Stress on Top of the Subgrade with Tire Pressure for Thaw Depth of 60 inches and 80 inches (9,000 lb Wheel Load)

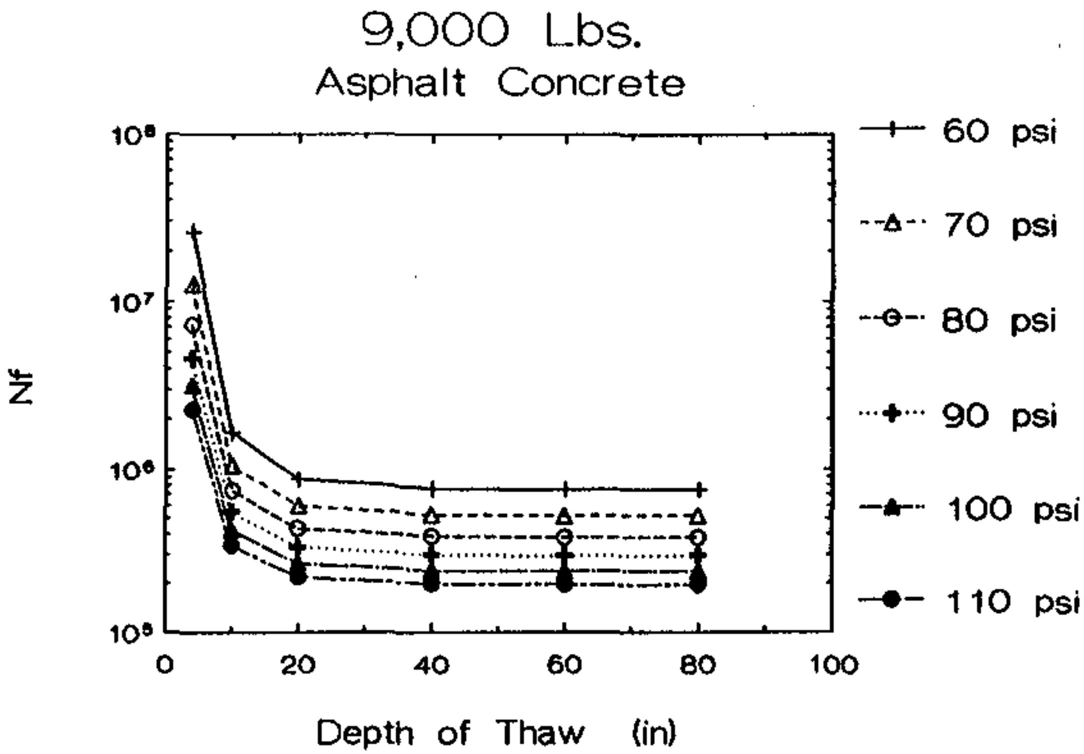


Figure 5.7 Variation of Fatigue Life of Asphalt Concrete with Thaw Depth and Tire Pressure (9,000 lb Wheel Load)

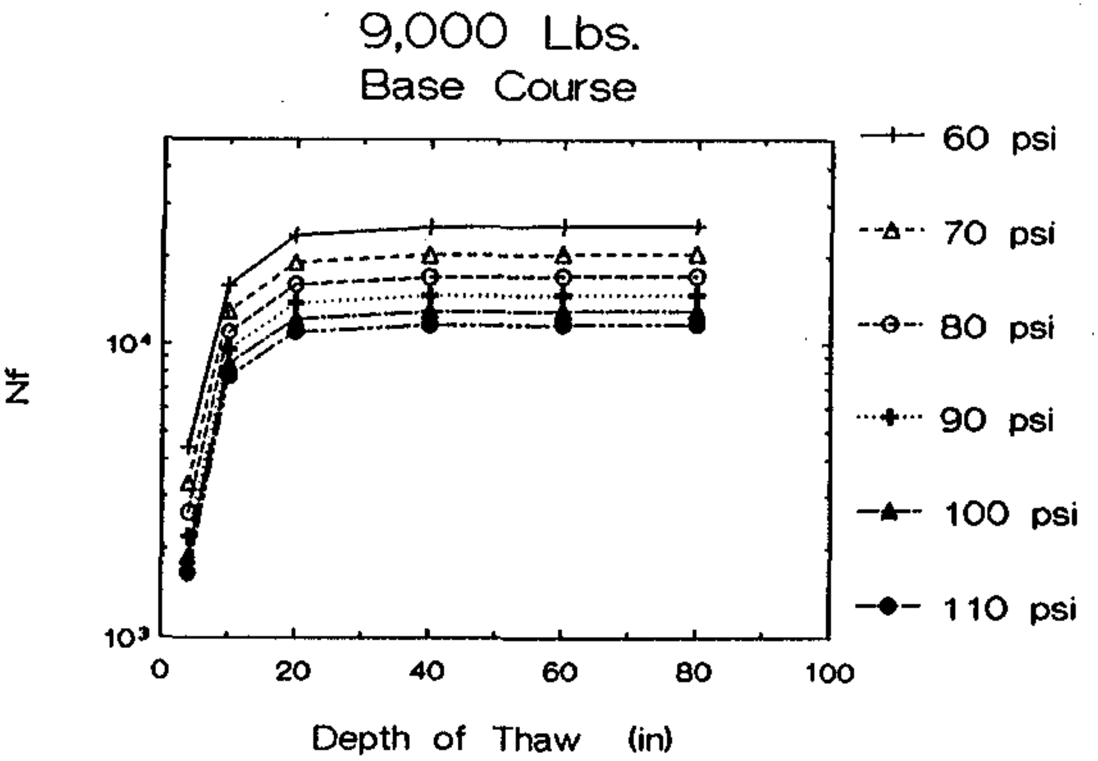


Figure 5.8 Variation of Load Repetitions for Base Course Failure with Thaw Depth and Tire Pressure (9,000 lb Wheel Load)

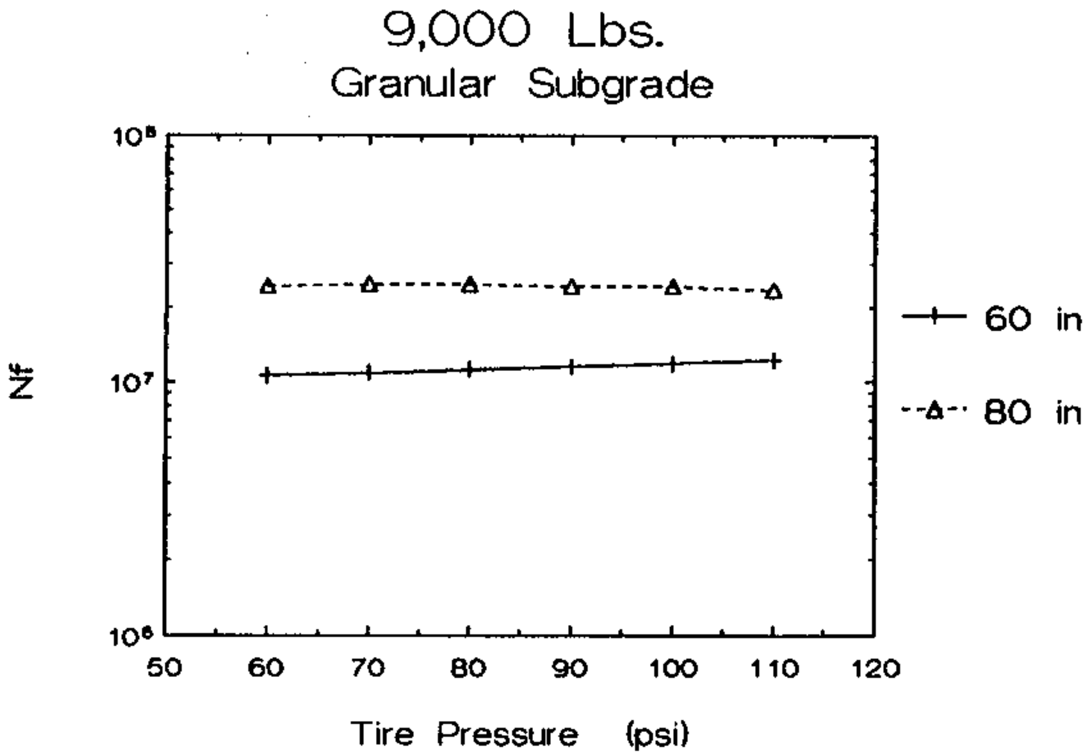


Figure 5.9 Failure Repetitions for Coarse-Grained Subgrade as a Function of Tire Pressure for Thawing of 60 inches and 80 inches (9,000 lb Wheel Load)

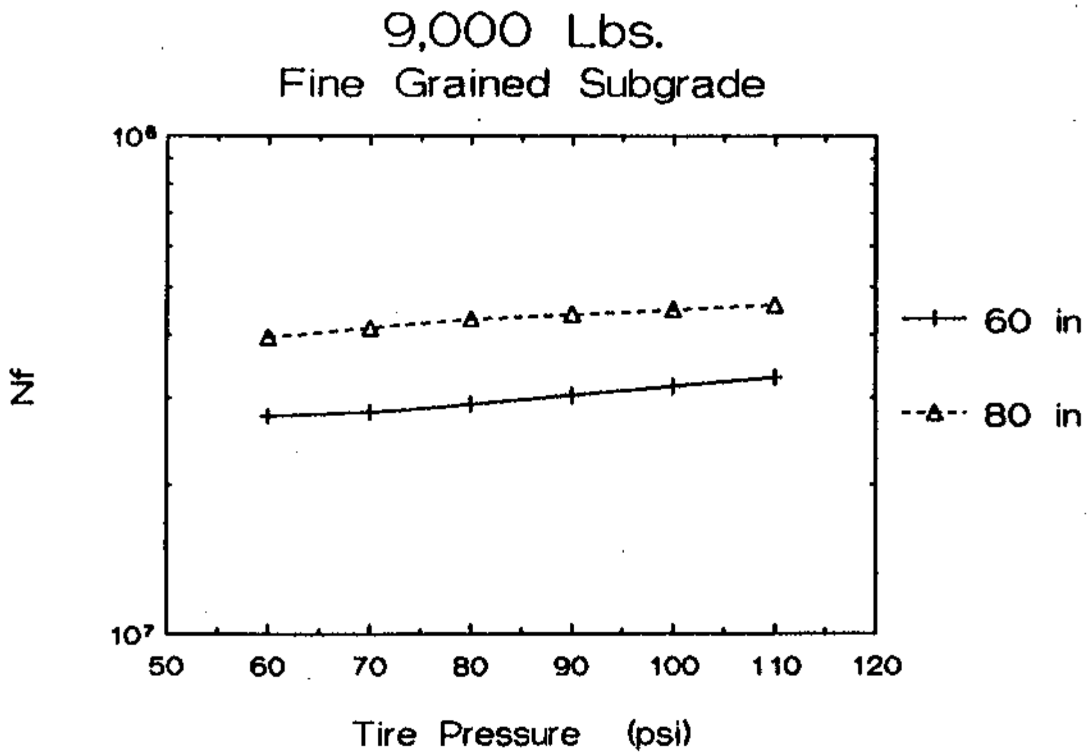


Figure 5.10 Failure Repetitions for Fine-Grained Subgrade as a Function of Tire Pressure for Thawing of 60 inches and 80 inches (9,000 lb Wheel Load)

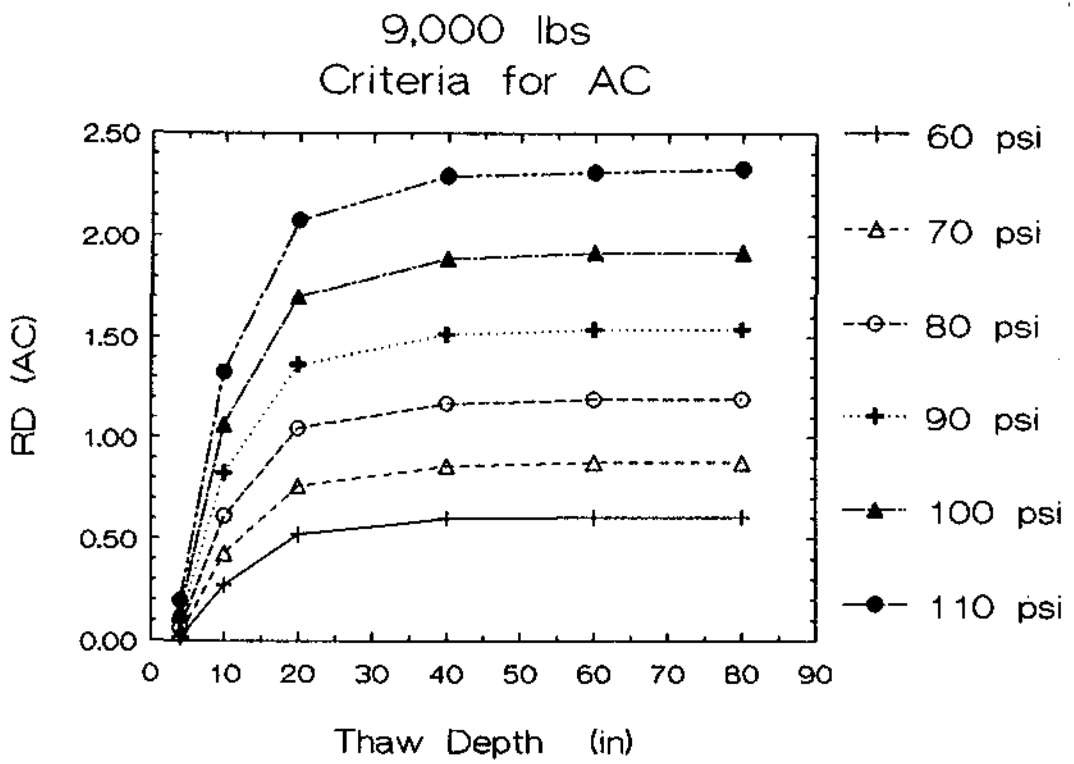


Figure 5.11 Influence of Tire Pressure and Thaw Depth on Damage of the Asphalt Concrete Layer (9,000 lb Wheel Load)

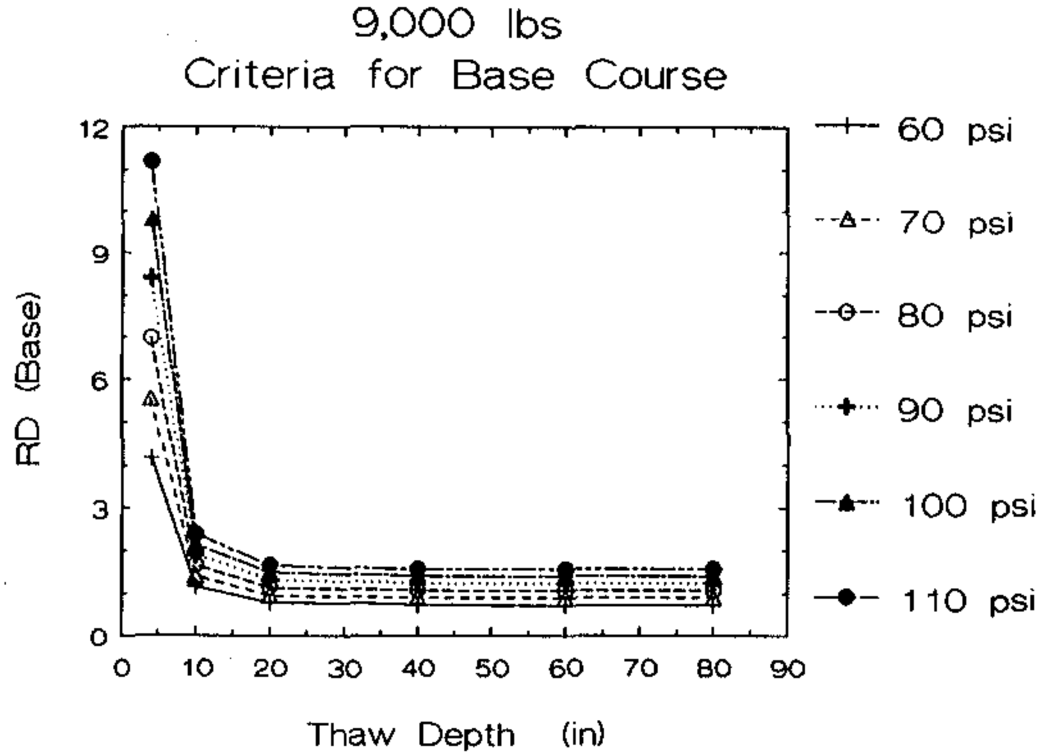


Figure 5.12 Influence of Tire Pressure and Thaw Depth on Damage of Base Course (9,000 lb Wheel Load)

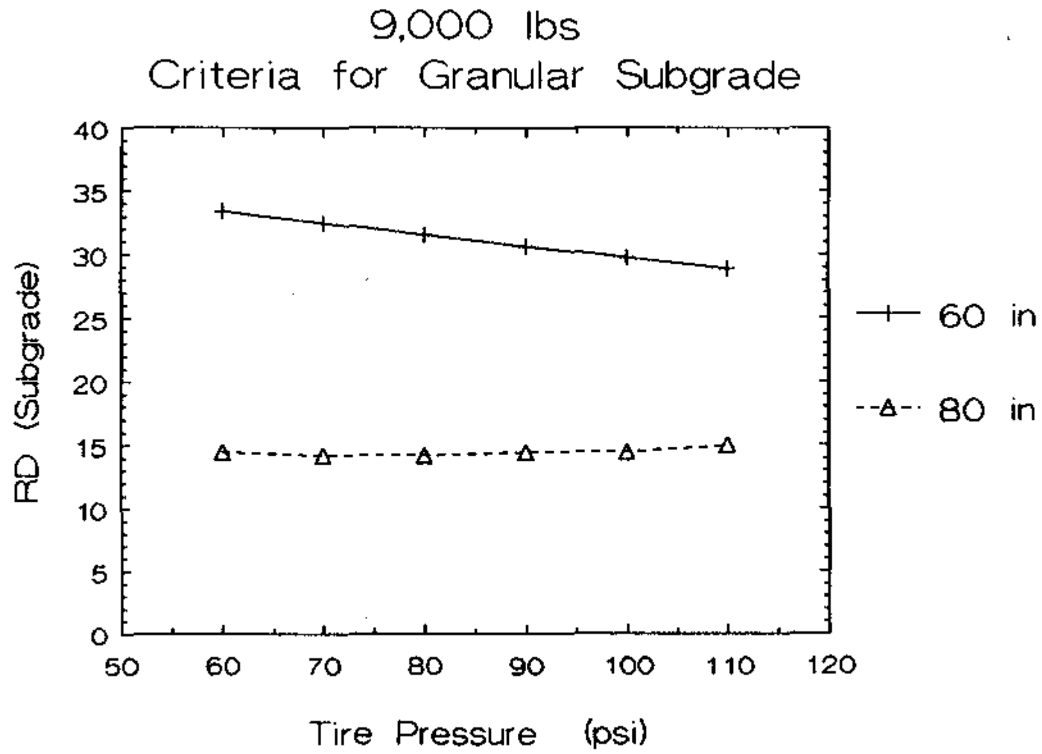


Figure 5.13 Influence of Tire Pressure and Thaw Depth on Damage of Coarse-Grained Subgrade (9,000 lb Wheel Load)

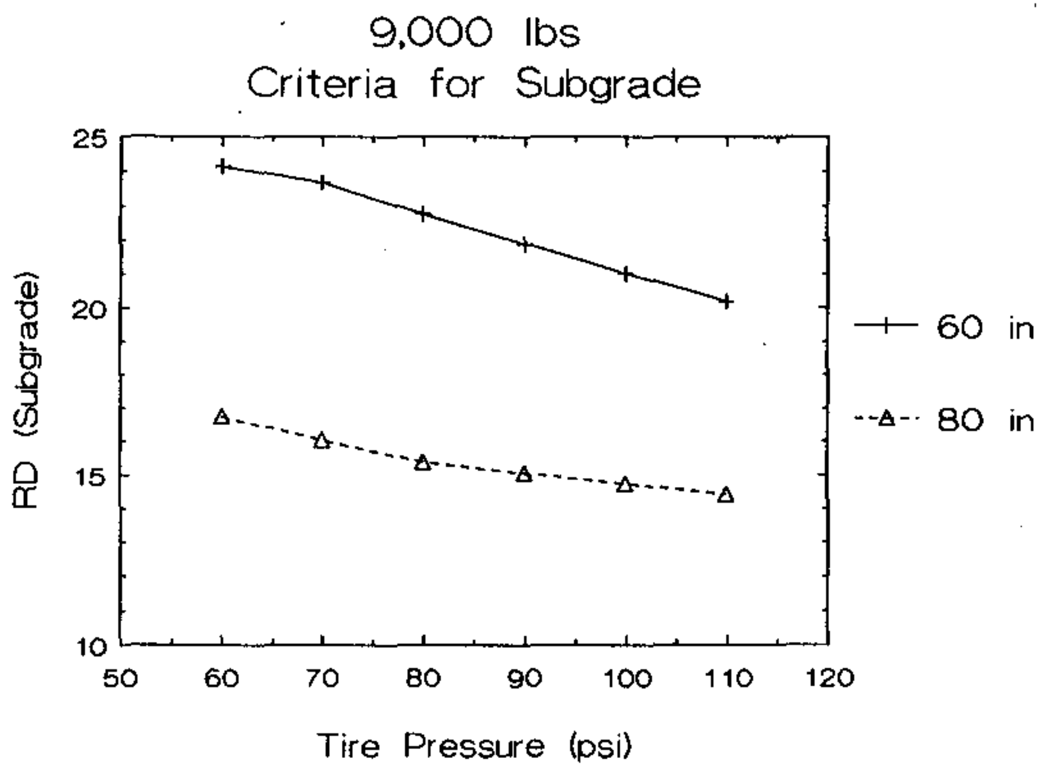


Figure 5.14 Influence of Tire Pressure and Thaw Depth on Damage of Fine-Grained Subgrade (9,000 lb Wheel Load)

CHAPTER SIX

SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

6.1 SUMMARY AND CONCLUSIONS

Pavement Field Studies and Damage Under No Load Restrictions

1. The extent of pavement weakening was determined by using FWD deflection profiles and backcalculation analyses. In almost all cases, "weakening" of the granular base was the most critical factor with regard to pavement damage during spring-thaw.

2. A criterion for selecting weak pavement sections was tentatively proposed. This criterion utilizes predictions of remaining life repetitions for both "average" and most "critical" pavement sections when no springtime load restrictions are applied. Accordingly, if no restrictions are applied, highway segments that exhibit a 15 percent or more reduction in "average" remaining life in comparison with the anticipated design life, are candidates for strengthening. Strengthening *should be* applied if the "critical" remaining life is less than 30 percent of the design life, and *need to be* applied if the "critical" remaining life is greater.

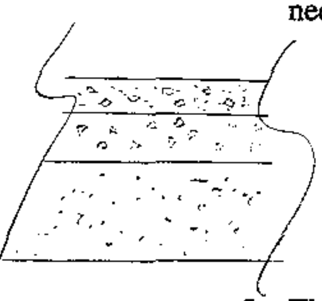
3. If this criterion is used to identify "weak" pavements of the Steese/Elliot and Haines highways, and if it is decided to strengthen sections that exhibit remaining life less or equal to 3 years, then the estimated mileage which must be maintained or rehabilitated for the Steese/Elliot and the Haines highways, in order to eliminate the need for load restrictions, will be approximately 3 miles (i.e. 11 percent) and 2 miles (i.e. 5 percent), respectively. The proposed criteron needs to be verified in relation to maintenance requirements, costs, and field performance.

WEAK PAVEMENT SECTIONS!

(A) NO RESTRICTIONS!
AVE. Remaining life 15% 2010 of life – Candidates
30% " " " – SHOULD BE
>30% " " " NEED TO BE

(B) Remaining life of 3YRS
STEESE 3 MILES (11%)
HAINES 2 miles

4. Proposed design thicknesses for "weak" pavement sections required to eliminate the need for springtime load restrictions are as follows:



34"

4 inches hot-mix asphalt surface

6 inches open-graded base, preferrably bituminous-treated

24 inches non-frost susceptible granular subbase

5. The proposed pavement should be properly drained and should consist of non-frost susceptible materials with strength and durability adequate to resist applied traffic and extreme climatic conditions.

6. FWD *center* deflections do not correlate in general with spring-thaw pavement damage, particularly for small thaw depths, and should not be used as criteria for load restrictions. On the other hand, the FWD surface *deflection bowl* can be used to backcalculate the critical pavement response parameters and the corresponding remaining life repetitions. This could be used to assess the duration and extent of load restrictions.

7. Variations of pavement surface temperatures were significant for a given testing period for both the Steese/Elliot and Haines highways. It is clear that the surface temperature varies depending on the location and time of the day and does not seem to correlate to thaw depth.

8. Extensive field tests were conducted during 1994 and 1995 on the Steese/Elliot highway to monitor rutting and roughness and to determine how much damage can be induced if no load restrictions are applied. Results in terms of EALs indicate that although the Southbound traffic lane is about 73 percent smaller than the Northbound traffic lane, the corresponding change in rutting and IRI is much smaller. In this

LESS - WEIGHT = TRAFFIC ?

case, the average decrease in rutting and IRI of Southbound pavements in comparison with Northbound pavements is about 14 percent (ranges between -35 and 40 percent) and 3 percent (ranges between -11 and 19 percent), respectively. For "critical" sections, the average decrease in rutting and IRI is 20 percent (ranges between -23 and 45 percent) and 10 percent (ranges between -15 and 48 percent), respectively. These values, particularly the IRI results, are much lower than expected and may indicate that damage associated with frost heave and foundation instabilities is more significant than load related damage.

KEY QUESTION:
WILL THERE BE DAMAGE IF NO TRAFFIC GOES OVER HIWAY?
IF NO - then frost would form initially, set the conditions in motion

Pavement Thaw Initiation and Propagation

1. Analyses of ground temperature data from a number of pavement sites in the Central and Northern regions indicate that thaw progression did not seem to be influenced by fluctuations of surface temperature (measured 3 inches below pavement surface) but followed essentially a trend similar to the "average" surface temperature.
2. No conclusion could be reached concerning the significance of night-time refreezing on load restriction since night-time ground temperature data were not available for this study. Limited data using day-time ground temperature data from Northern Region sites indicate that refreezing could occur. The refreezing rate is estimated to be 40 ^{O}F-hr/in.

Ground temperature data were also used to suggest probabilistic criteria for timing and removal of load restrictions. These criteria were based on thaw initiation and progression below the pavement surface for both "critical" sites and "normal" sites.

THE REAL QUESTION IS:
GIVEN limiting bearing ability of soils (roads) during a thawing period - what is the max. ranges of weight that will not distress the soil (base course) to extent that damage doesn't occur

The limitations of these criteria were discussed particularly in relation to upgrading the proposed models as more ground temperature data become available.

Tire Pressure and Pavement Damage

1. Pavement analysis for different load magnitudes and tire pressure show that for thin pavements (2 inch asphalt surface) maximum damage during spring-thaw occurs when thawing initiates in the base. Results also show that damage in the granular base represents the most significant contribution to overall pavement damage. Fatigue of the asphalt concrete surface is also critical but to a lesser degree. The subgrade has the least critical damaging effect on the pavement sections analyzed.
2. Maximum base damage does not coincide with maximum surface deflection. This indicates that the application of spring load restrictions should be based on the vertical stress on top of the granular base rather than maximum surface deflections.
3. The critical pavement damaging period occurs during the first 20 inches of thawing. No further reduction in damage is predicted for greater thaw depths.
4. Reducing tire pressure could significantly limit pavement damage. For example, a reduction of tire pressure from 110 psi to 60 psi for a 9,000 lb wheel load would reduce maximum damage by a factor of 3.
5. Reducing the tire pressure for a standard 9,000 lb wheel load from 110 psi to 77 psi is equivalent in terms of pavement damage to applying a 75 percent spring load restriction limit while keeping the tire pressure equal to 110 psi.
6. The results obtained were limited to selective pavement geometries and material properties using multilayer elastic analyses. The analyses used the simplifying assumption that the contact pressure between the tire and the pavement surface is

equal to the tire inflation pressure. Field verification of analytical predictions is required before final recommendations on the effects of tire pressue on pavement damage can be established.

6.2 RECOMMENDATIONS

1. Alternatives to springtime load restrictions for Alaskan roads may include strengthening "weak" pavement sections and reducing tire inflation pressure. The selection of candidate pavements for no load restriction application would depend on the extent of spring-thaw weakening, the significance of climate related damage in comparison with load associated damage, pavement maintenance and rehabilitation costs, and trucking operations costs.
2. The most critical time for road damage to occur is when thawing initiates in the pavement base. For "restricted" routes, load restriction should be applied when thawing starts in the base. Based on multilayer elastic analyses (Chapter 5), FWD backcalculation of stiff layer (Chapter 4), and observed pavement "dry" conditions (Chapter 3), the thaw depth corresponding to minimum pavement damage varies between 2 feet and 5 feet. It is recommended that load restrictions be removed when the thaw depth reaches 3.5 feet.
3. Since maximum surface deflections do not coincide with the most critical damage period of the pavement, ground temperature measurements rather than FWD center deflections should be used to determine the time for applying and removing load restrictions.

remaining life of the pavement. It could also be used to monitor the gain in pavement strength during thaw propagation thereby providing additional information for removing load restrictions.

6.3 FUTURE RESEARCH NEEDS

The following are recommended areas of future research that will complement the work presented in this study:

1. Determine spring-thaw weakenening effects on major routes in Alaska and identify corresponding springtime restriction needs and alternatives. Of particular interest in this case is the evaluation of pavement damage associated with frost heave and foundation instability relative to damage resulting from traffic loads.
2. Evaluate the influence of reduced tire pressure on pavement damage using large scale accelerated pavement tests and field studies.
3. Develop improved models for predicting thaw initiation and propagation using field temperature data. This will provide a better assessment of spring-thaw weakening periods for Alaskan roads.
4. Analyze truck traffic data on major Alaskan highways and determine equivalent axle load applications for different periods in order to obtain better estimates of pavement remaining life and the corresponding load restriction needs.
5. Develop improved criteria for the behavior of granular bases in Alaskan roads for different loading, moisture, and freeze-thaw conditions.

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APPENDIX A

STEESE/ELLIOT HIGHWAY RWD DATA
(1993)

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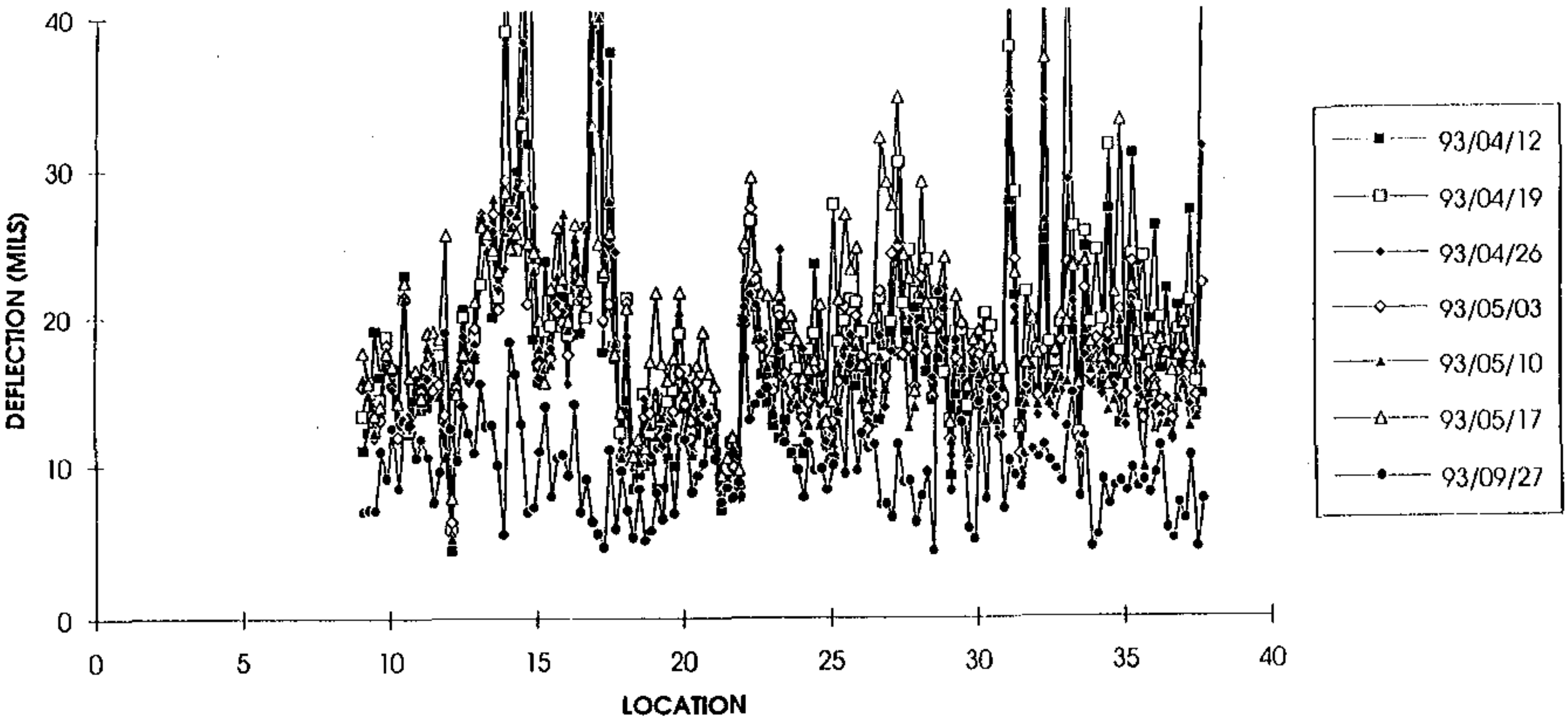
APPENDIX A -- STEESE/ELLIOT HIGHWAY FWD DATA (1993)

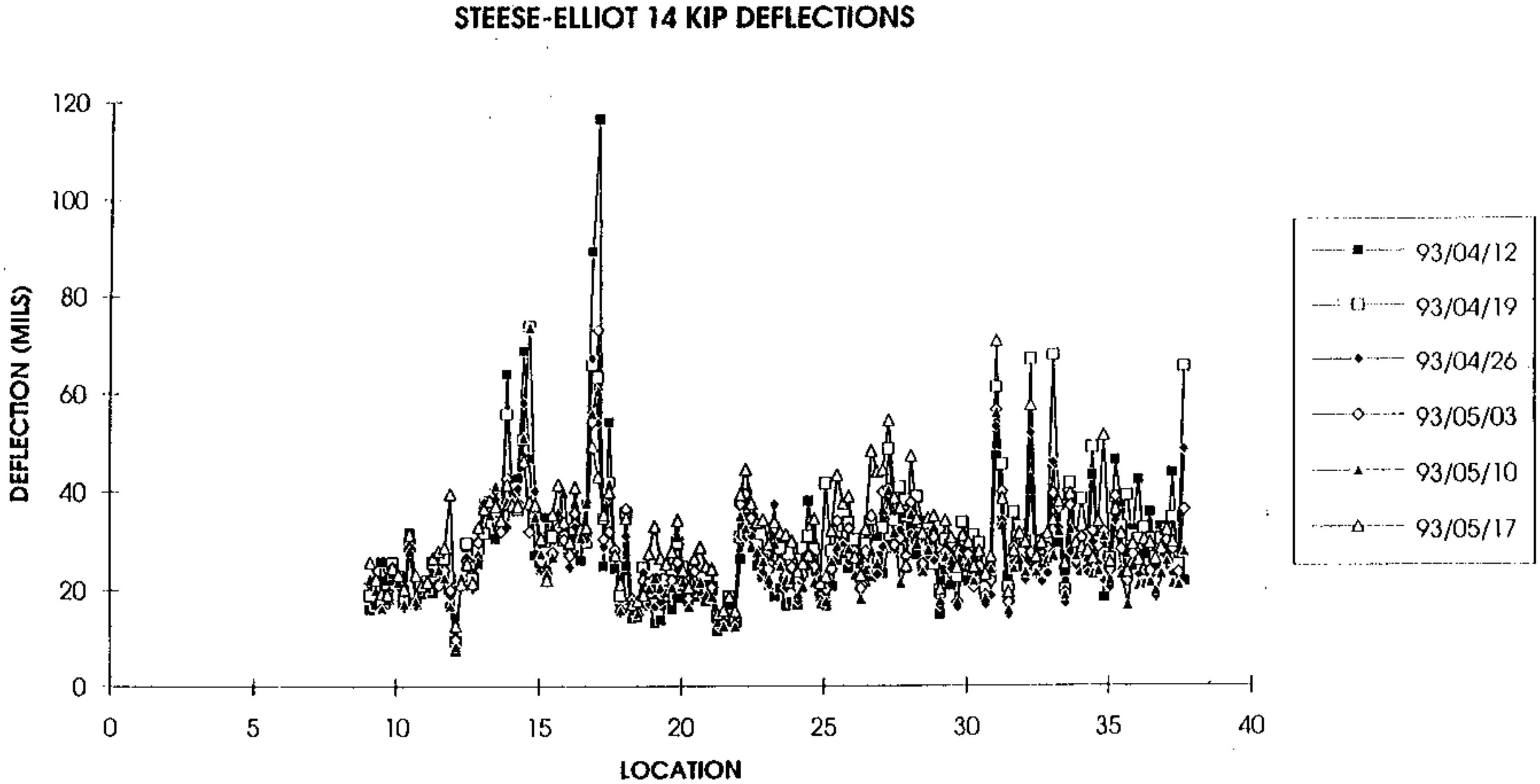
APPENDIX A-1

STEESE/ELLIOT

FWD SURFACE DEFLECTIONS FOR 9K AND 14K

STEESE-ELLIOT 9 KIP DEFLECTIONS



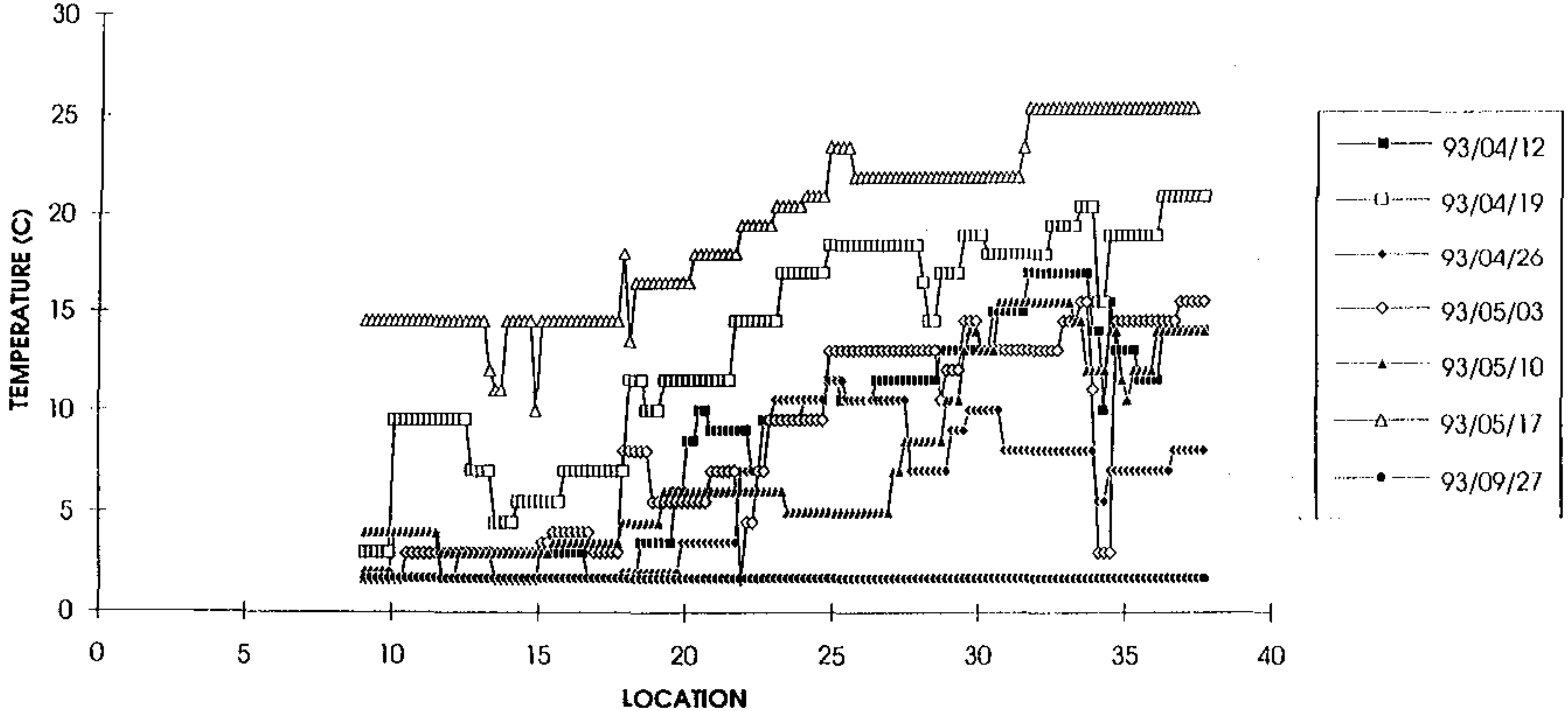

STEESE-ELLIOT 14 KIP DEFLECTIONS
DEFLECTION (MILS)
120
100
80
60
40
20
0
0
5
10
15
20
25
30
35
40
LOCATION
93/04/12
93/04/19
93/04/26
93/05/03
93/05/10
93/05/17

APPENDIX A-2

STEESE/ELLIOT

PAVEMENT SURFACE TEMPERATURE

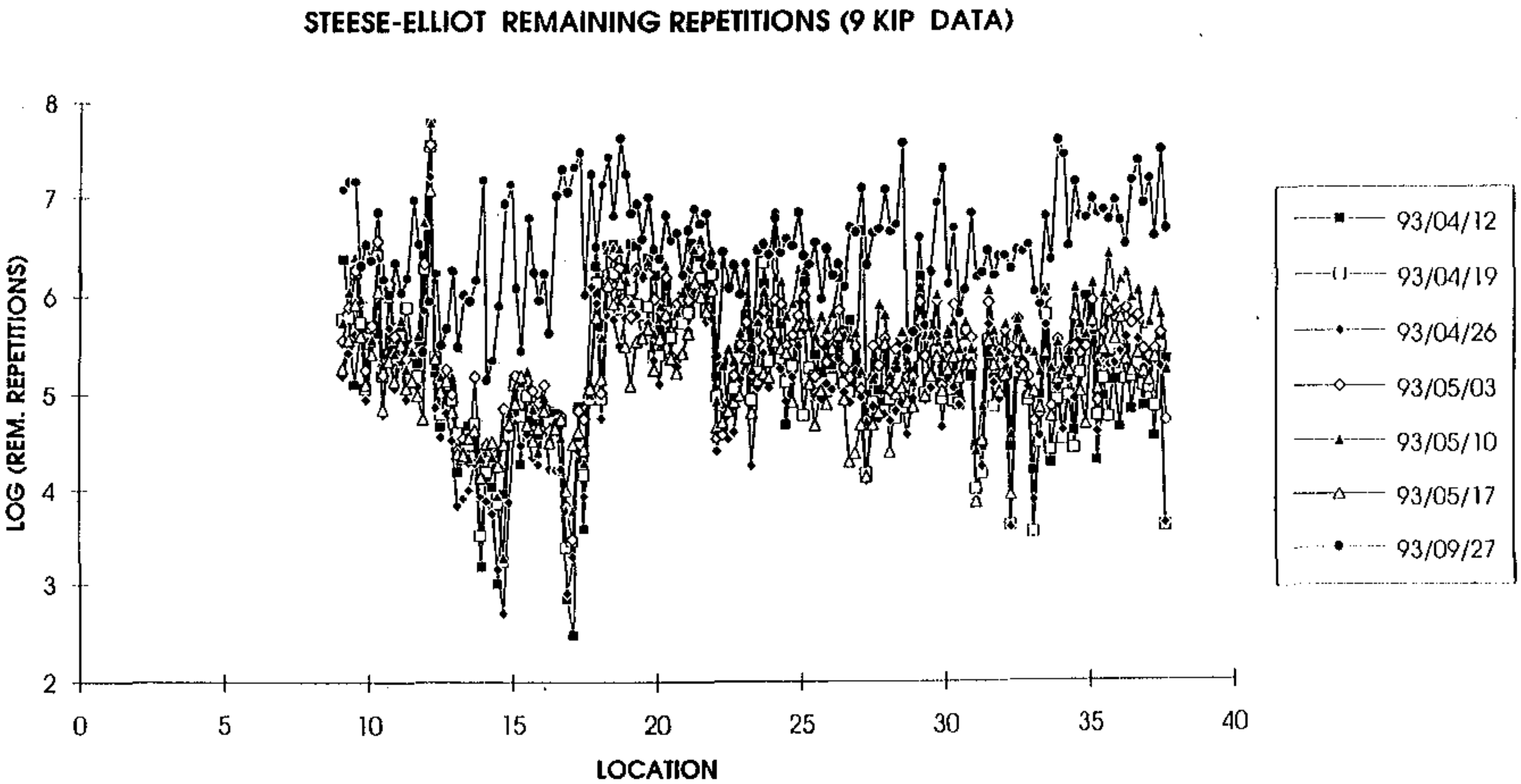
STEESE-ELLIOT HWY. SURFACE TEMPERATURE (C)

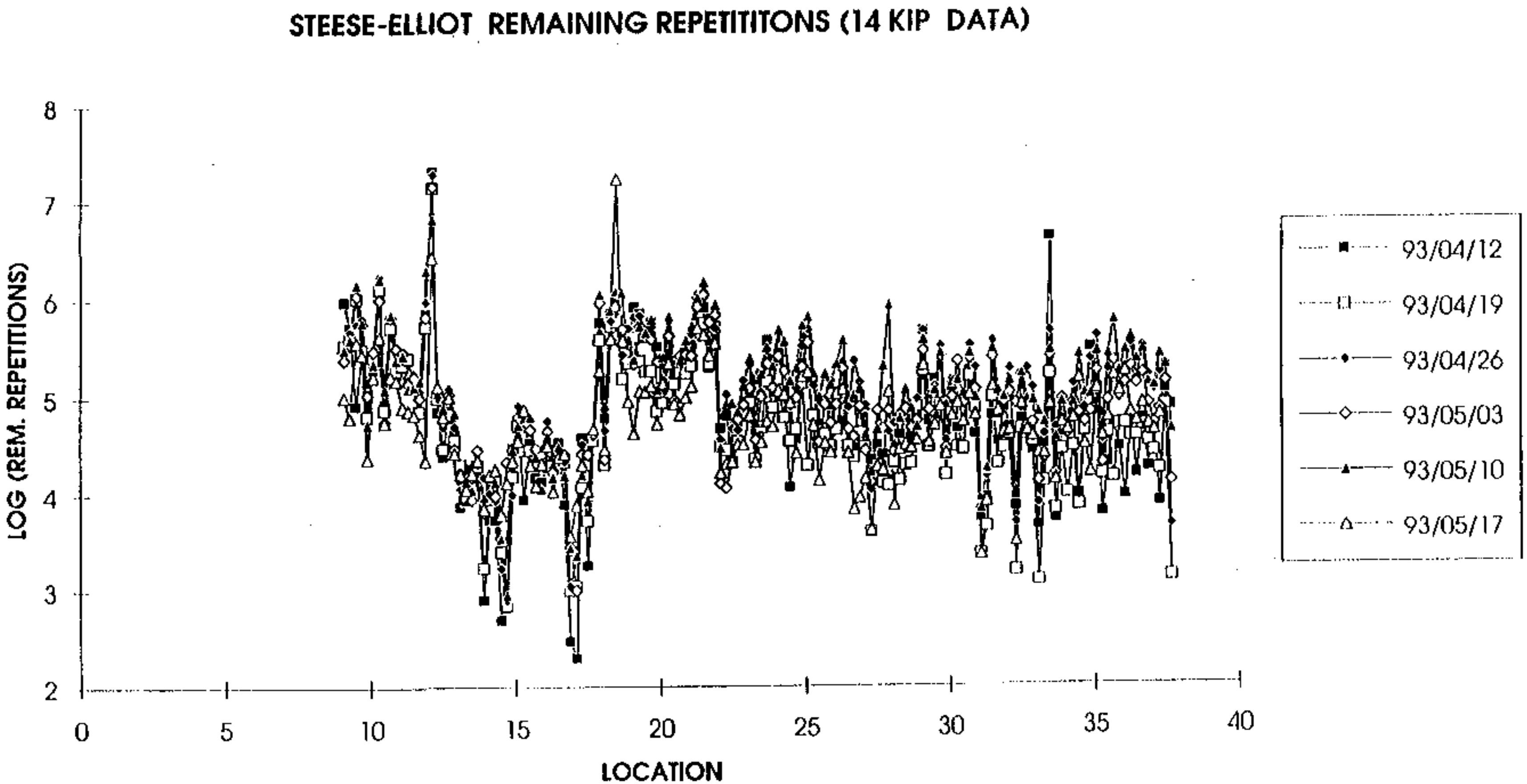


APPENDIX A-3

STEESE/ELLIOT

REMAINING LIFE REPETITIONS


STEESE-ELLIOT REMAINING REPETITIONS (9 KIP DATA)
LOG (REM. REPETITIONS)
LOCATION
8
7
6
5
4
3
2
0
5
10
15
20
25
30
35
40
93/04/12
93/04/19
93/04/26
93/05/03
93/05/10
93/05/17
93/09/27

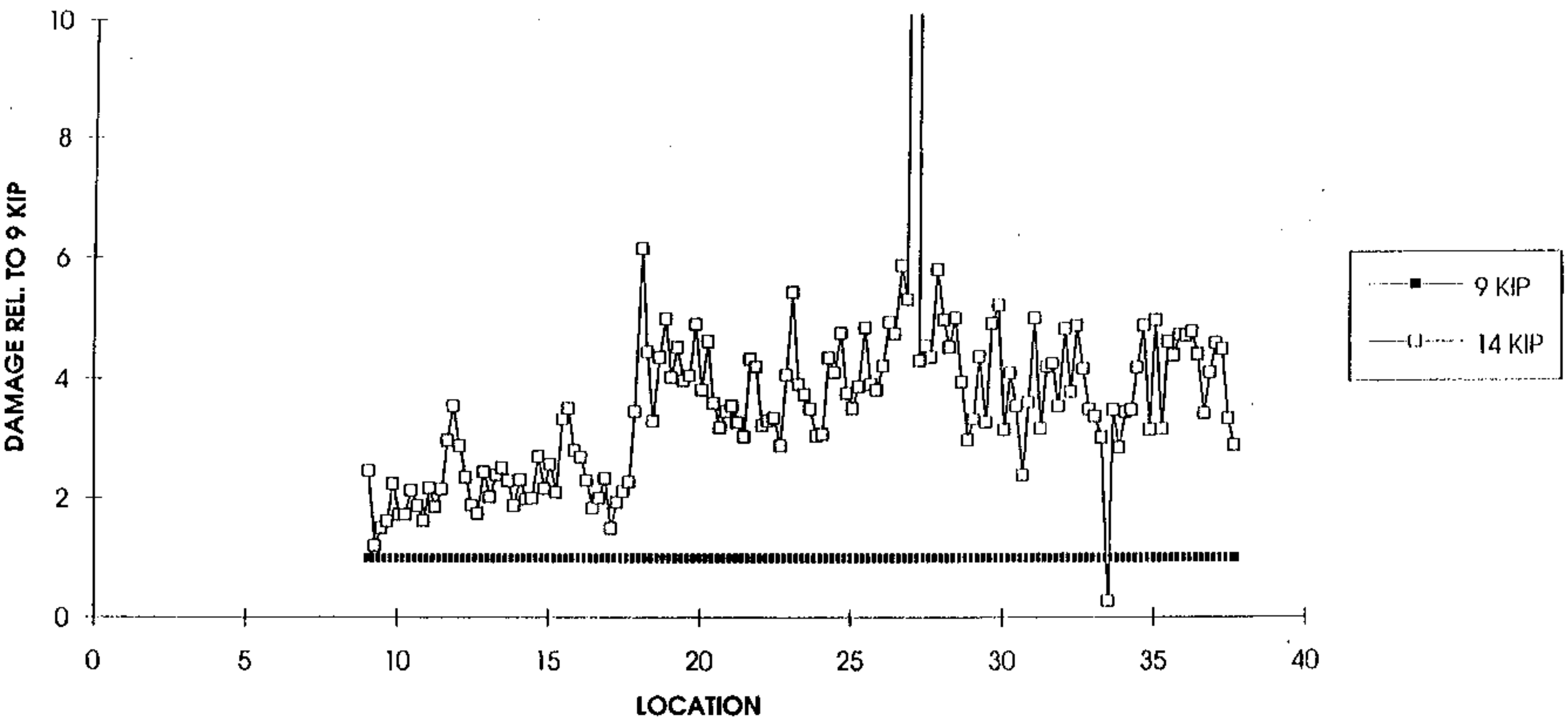

STEESE-ELLIOT REMAINING REPETITITONS (14 KIP DATA)
LOG (REM. REPETITIONS)
8
7
6
5
4
3
2
0
5
10
15
20
25
30
35
40
LOCATION
93/04/12
93/04/19
93/04/26
93/05/03
93/05/10
93/05/17

APPENDIX A-4

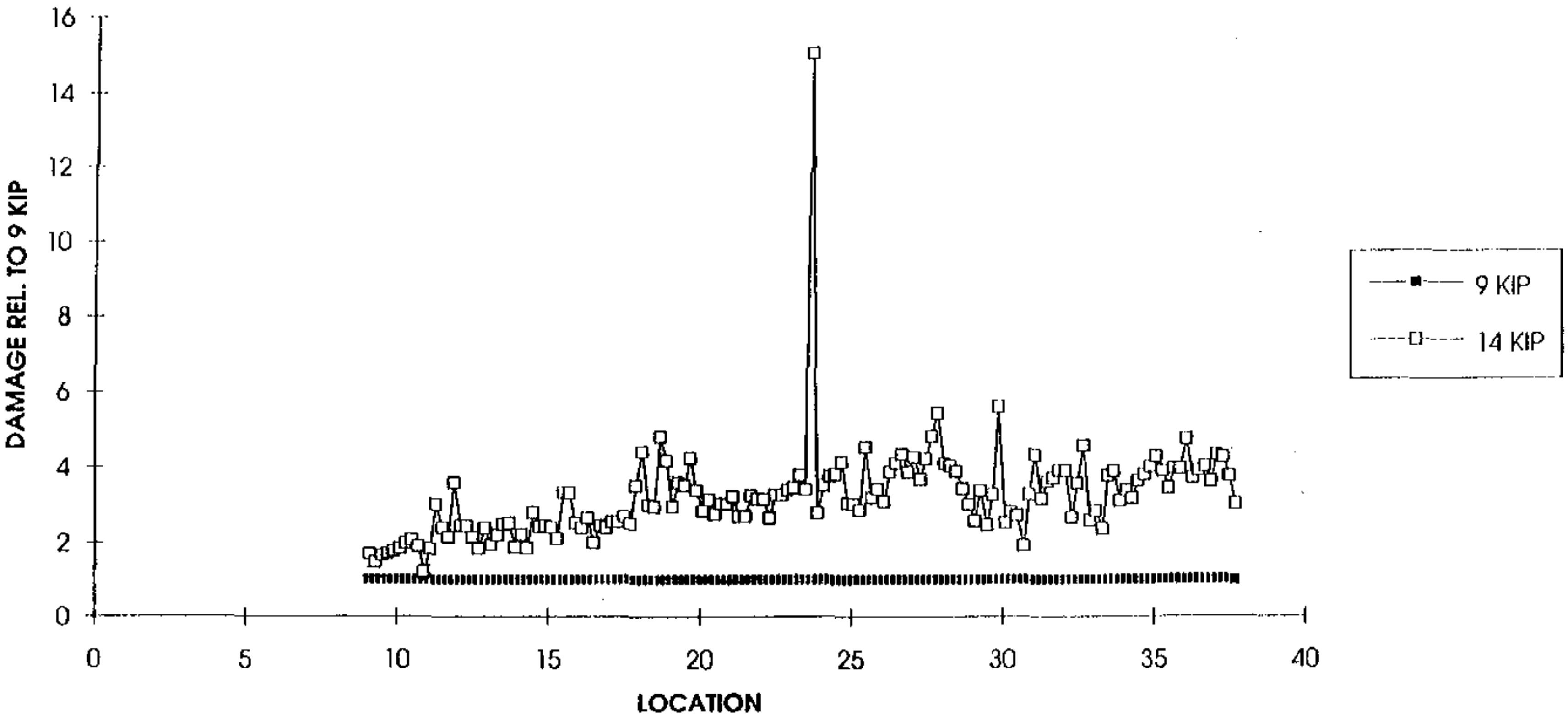
STEESE/ELLIOT

PAVEMENT DAMAGE RELATIVE TO 9K LOAD FOR A GIVEN TESTING PERIOD

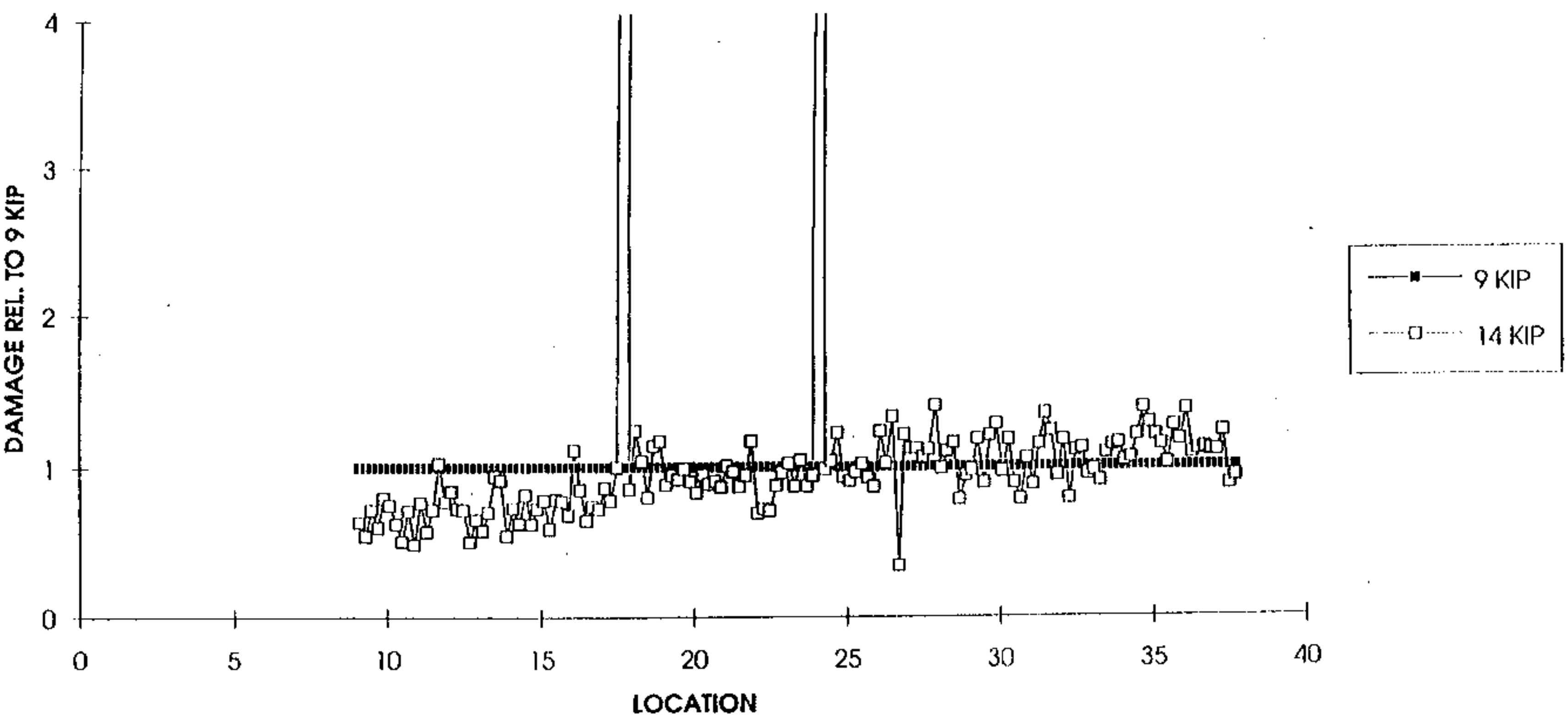
STEESE-ELLIOT RELATIVE BASE DAMAGE 93/04/12

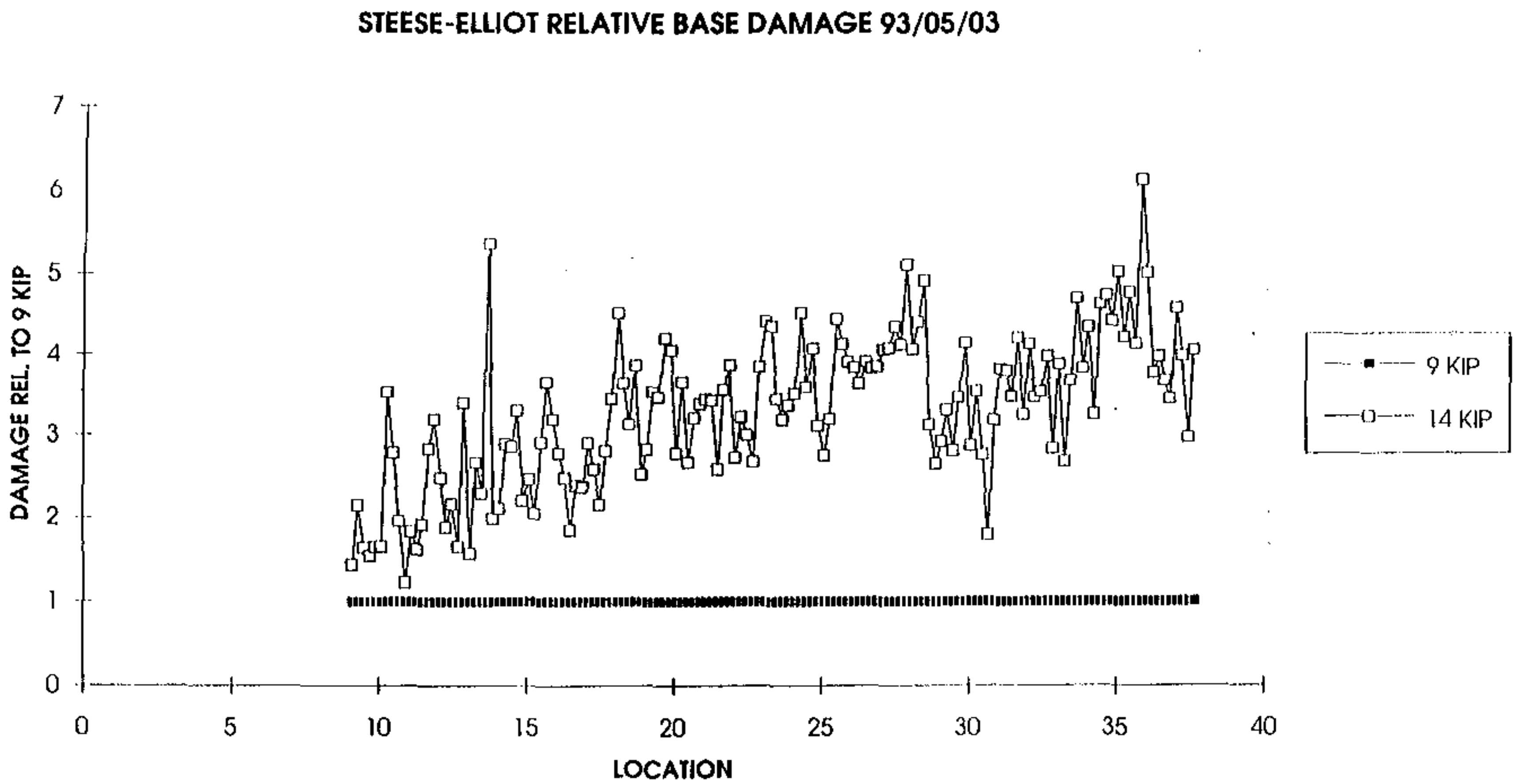


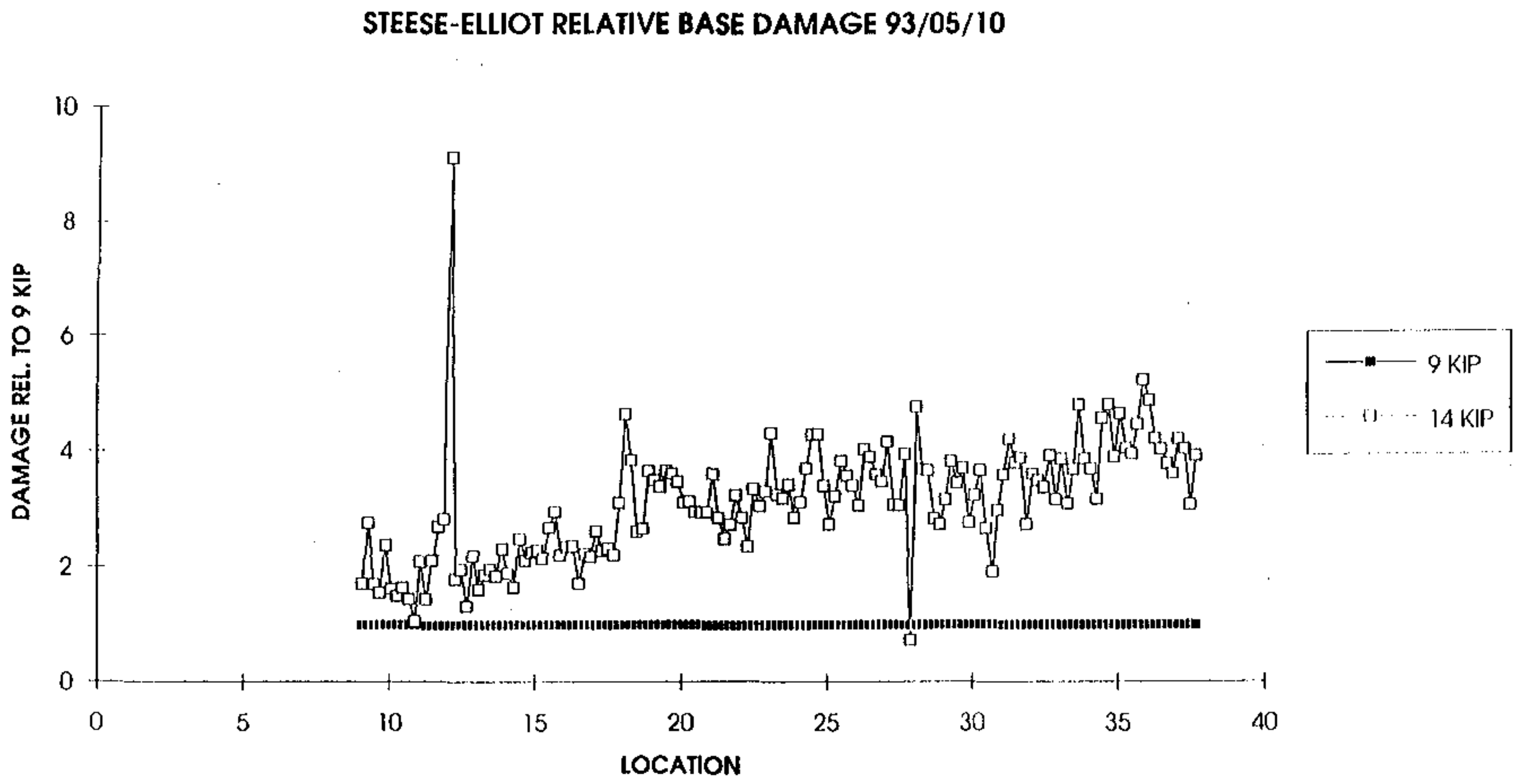
STEESE-ELLIOT RELATIVE BASE DAMAGE 93/04/19



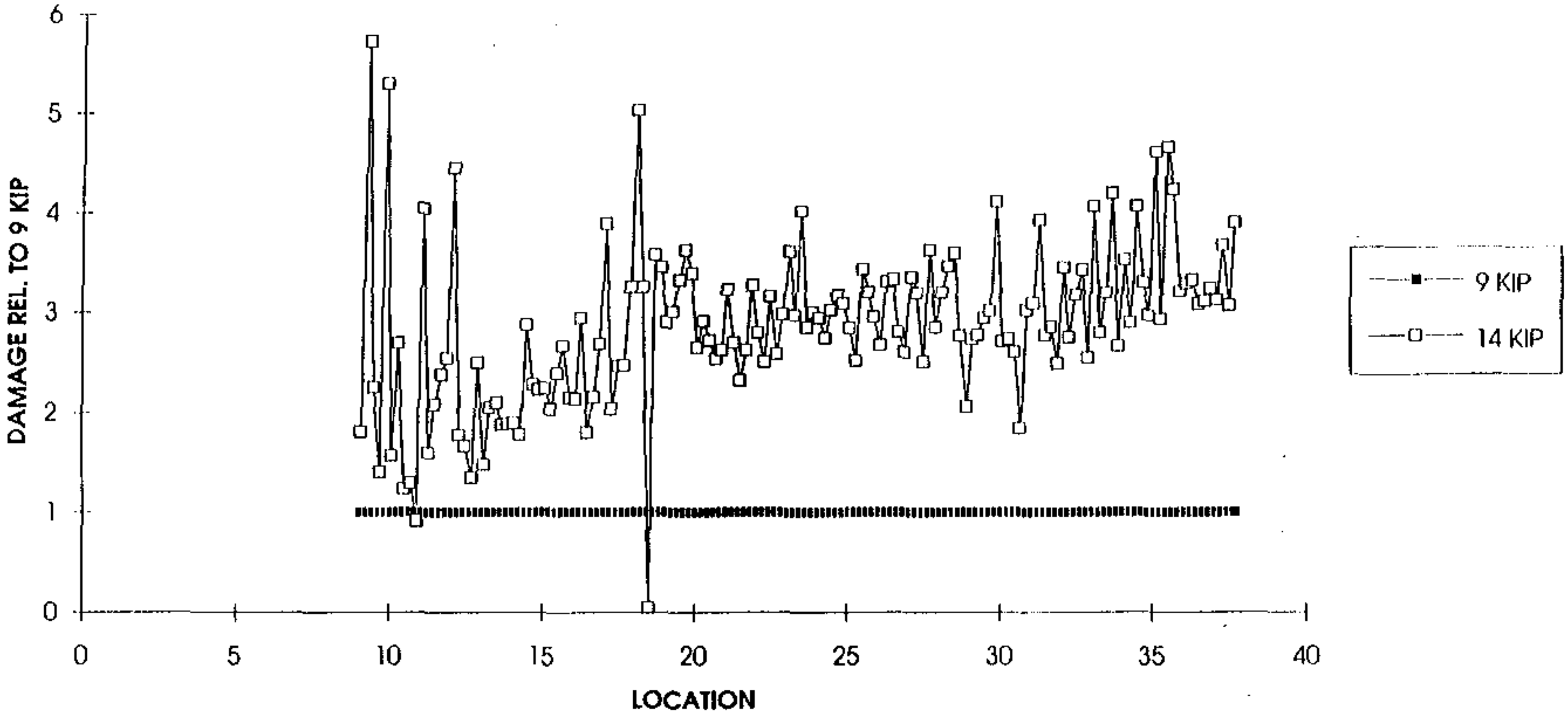
STEESE-ELLIOT RELATIVE BASE DAMAGE 93/04/26




STEESE-ELLIOT RELATIVE BASE DAMAGE 93/05/03
DAMAGE REL. TO 9 KIP
7
6
5
4
3
2
1
0
0
5
10
15
20
25
30
35
40
LOCATION
9 KIP
14 KIP


STEESE-ELLIOT RELATIVE BASE DAMAGE 93/05/10
DAMAGE REL. TO 9 KIP
10
8
6
4
2
0
0
5
10
15
20
25
30
35
40
LOCATION
9 KIP
14 KIP

STEESE-ELLIOT RELATIVE BASE DAMAGE 93/05/17

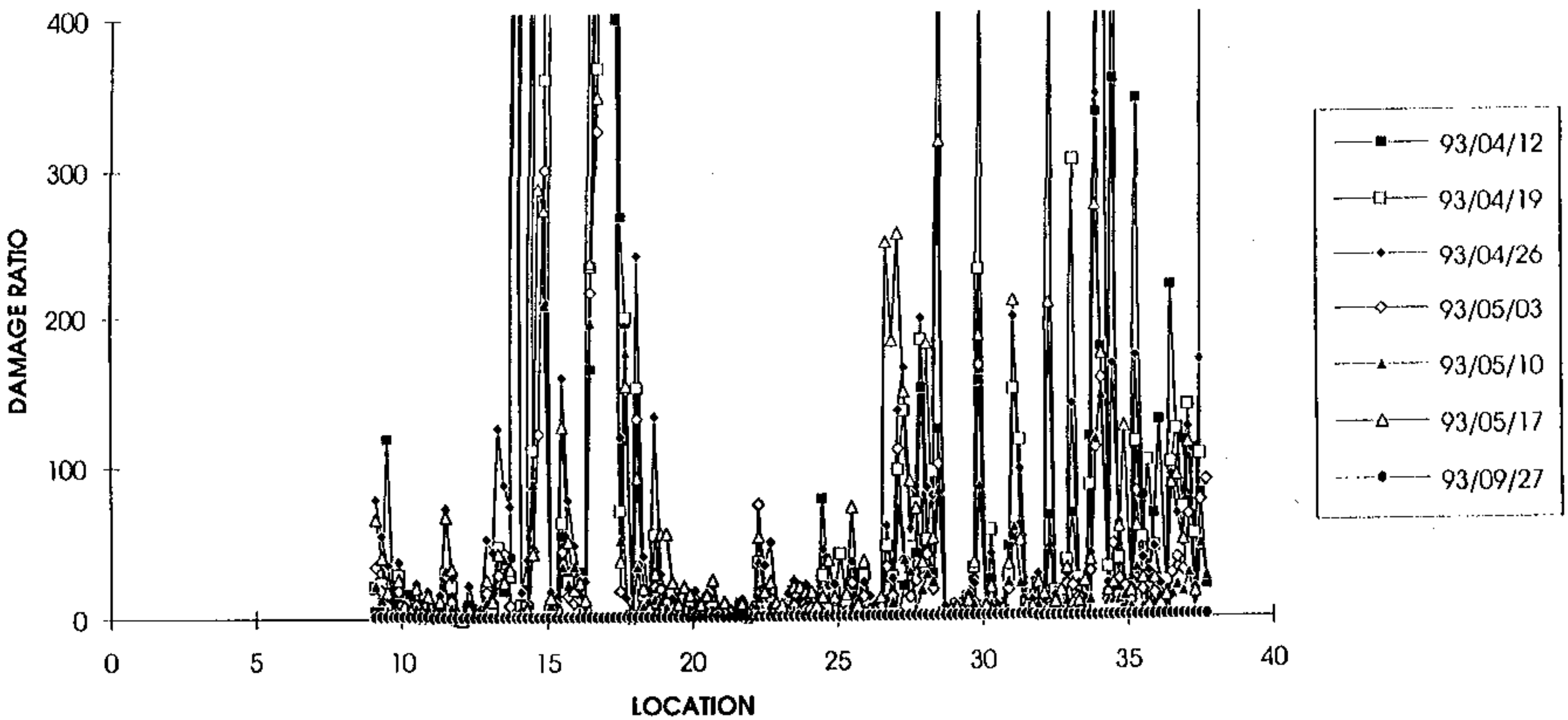


APPENDIX A-5

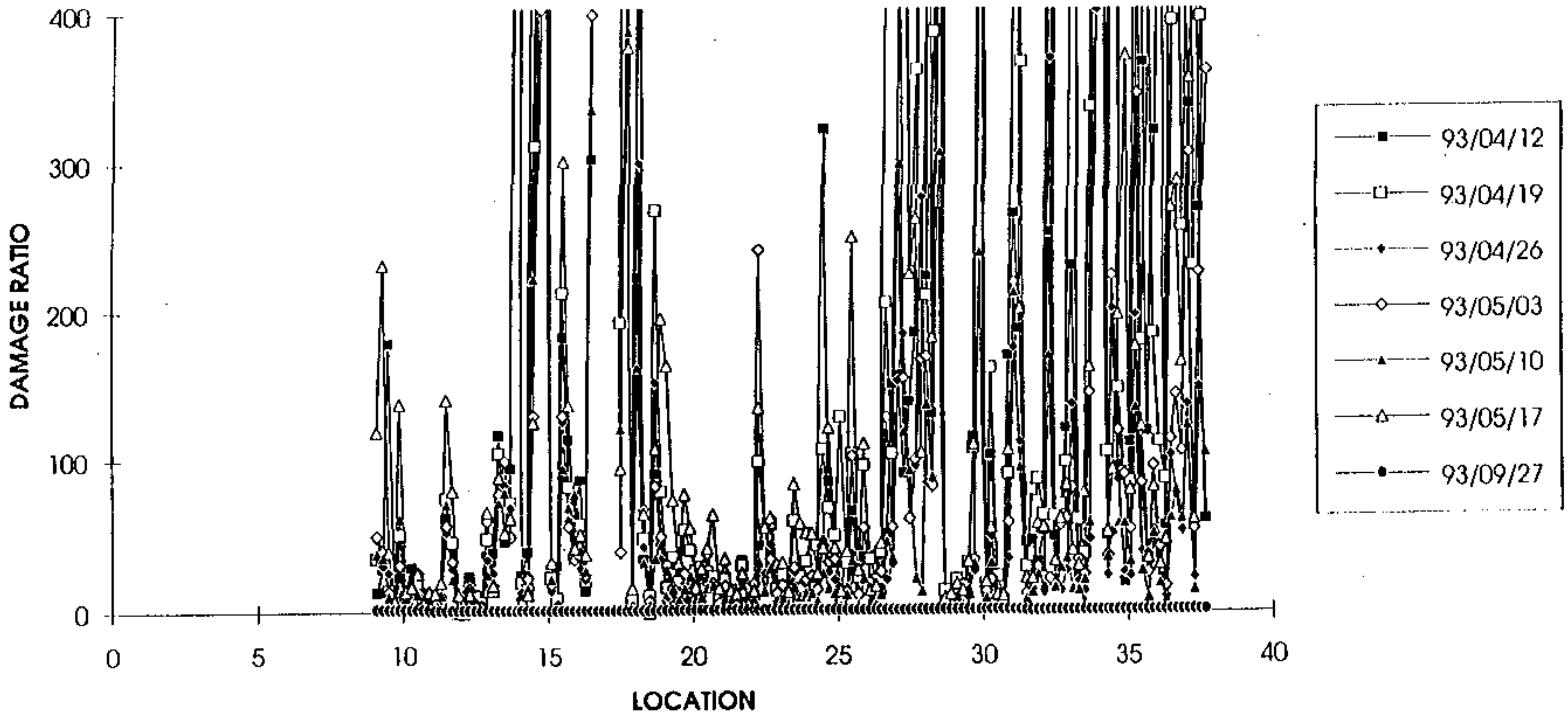
STEESE/ELLIOT

PAVEMENT DAMAGE RELATIVE TO 9K LOADING DURING 09/27/93

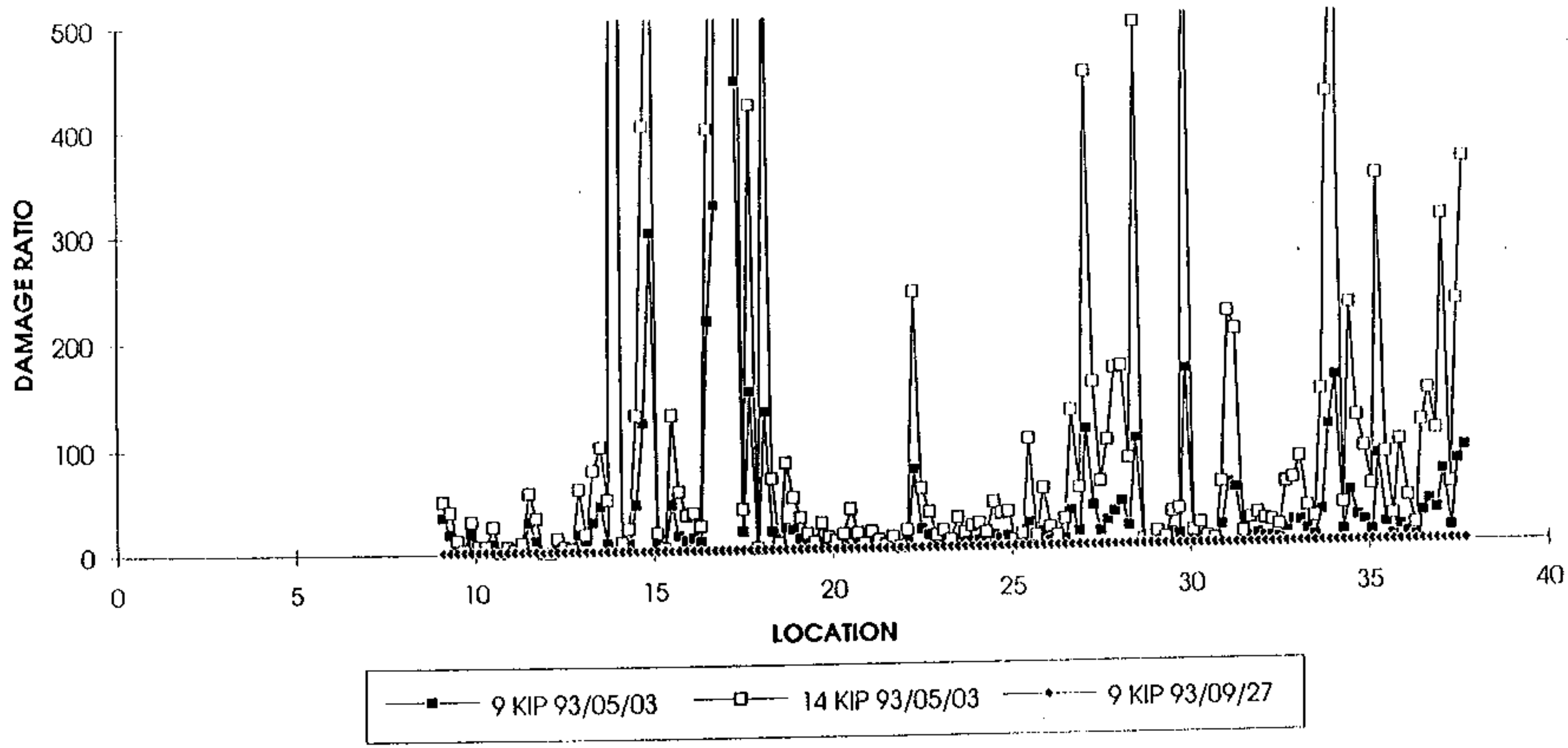
STEESE-ELLIOT 9 KIP DAMAGE RELATIVE TO 9 KIP ON 93/09/27



STEESE-ELLIOT 14 KIP DAMAGE RELATIVE TO 9 KIP ON 93/09/27



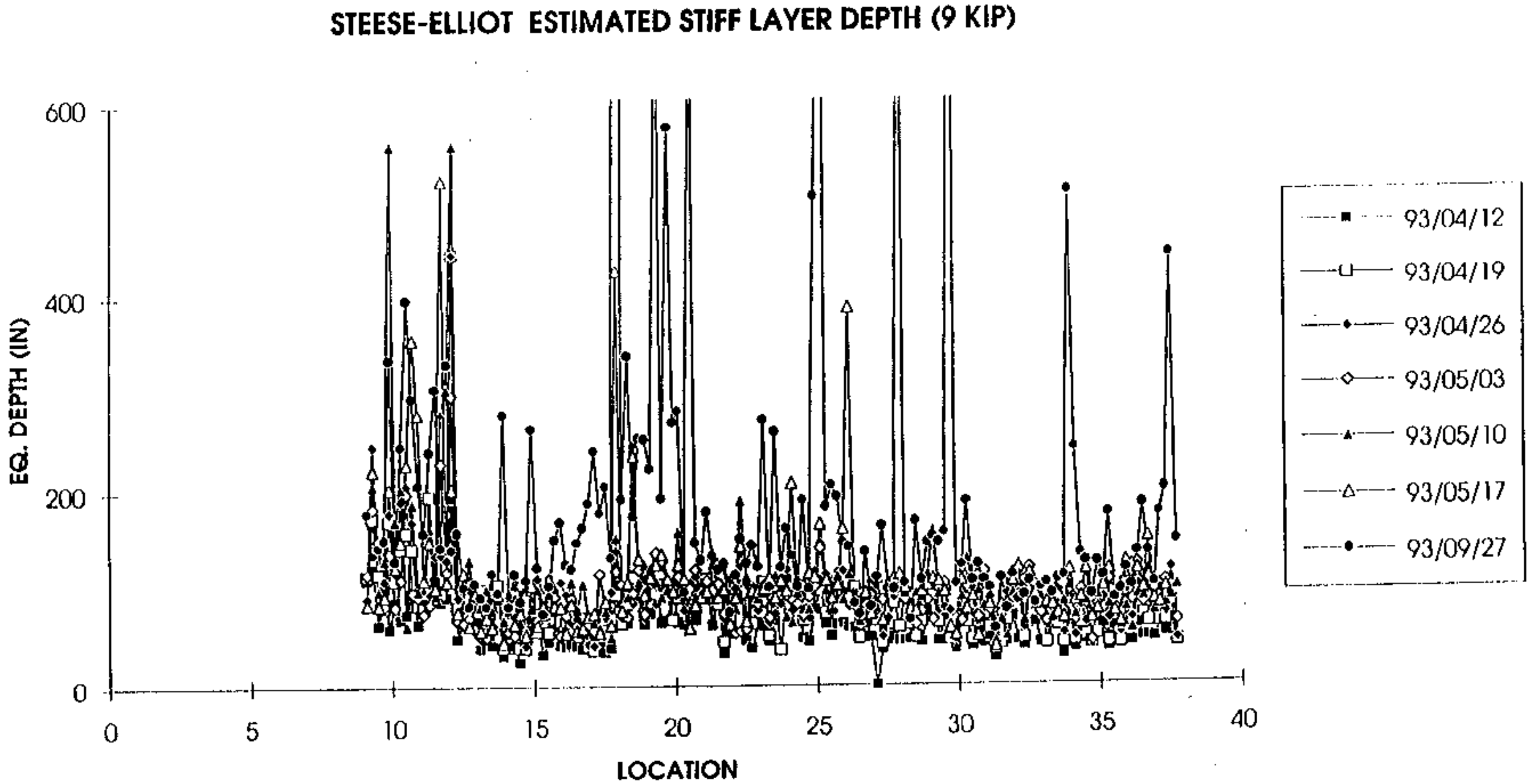
STEESE-ELLIOT 93/05/03 DAMAGE RELATIVE TO 9 KIP ON 93/09/27



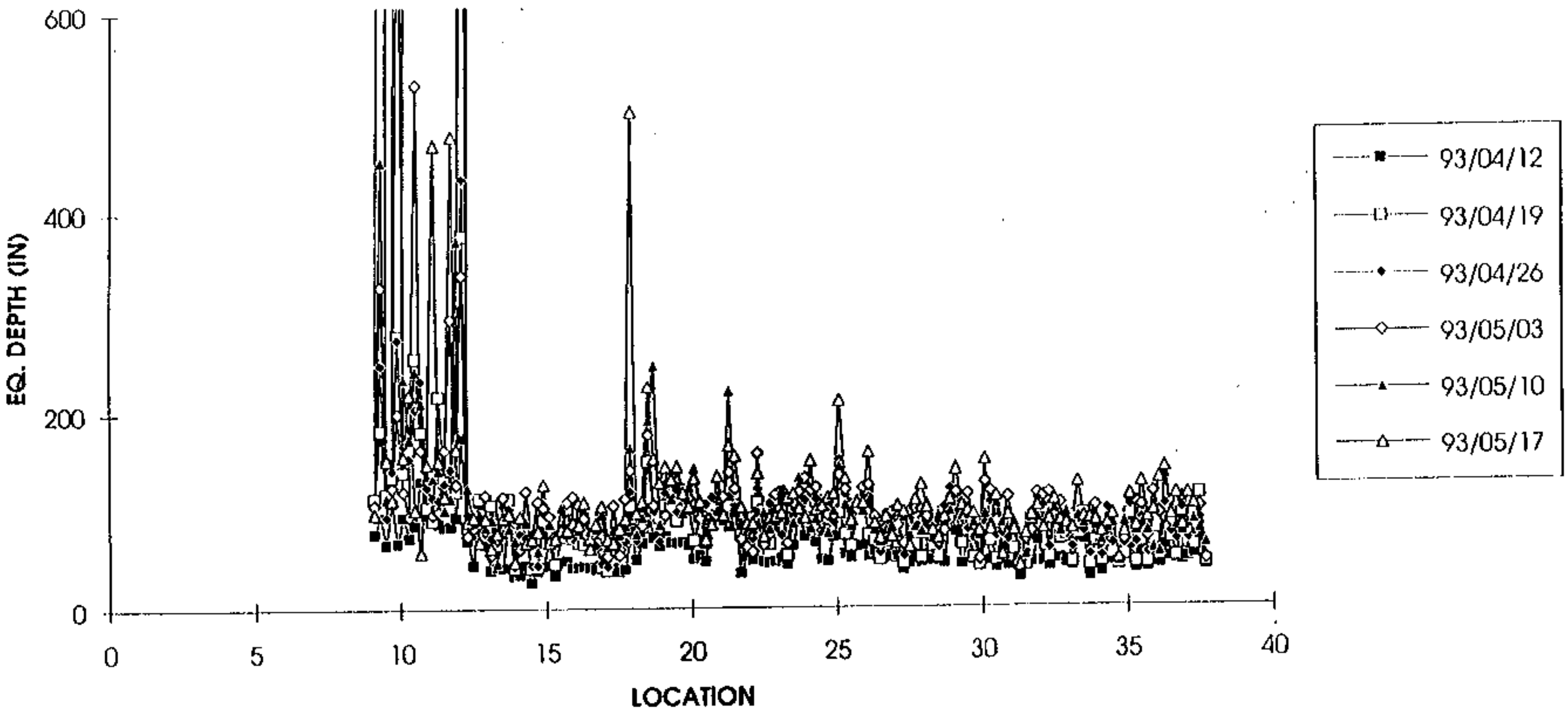
APPENDIX A 6

STEESE/ELLIOT

BACKCALCULATED THAW DEPTH (DEPTH TO STIFF LAYER) USING FWD SURFACE DEFLECTIONS


STEESE-ELLIOT ESTIMATED STIFF LAYER DEPTH (9 KIP)
EQ. DEPTH (IN)
600
400
200
0
0
5
10
15
20
25
30
35
40
LOCATION
93/04/12
93/04/19
93/04/26
93/05/03
93/05/10
93/05/17
93/09/27

STEESE-ELLIOT ESTIMATED STIFF LAYER DEPTH (14 KIP)



APPENDIX B

HAINES HIGHWAY FWD DATA
(1993)

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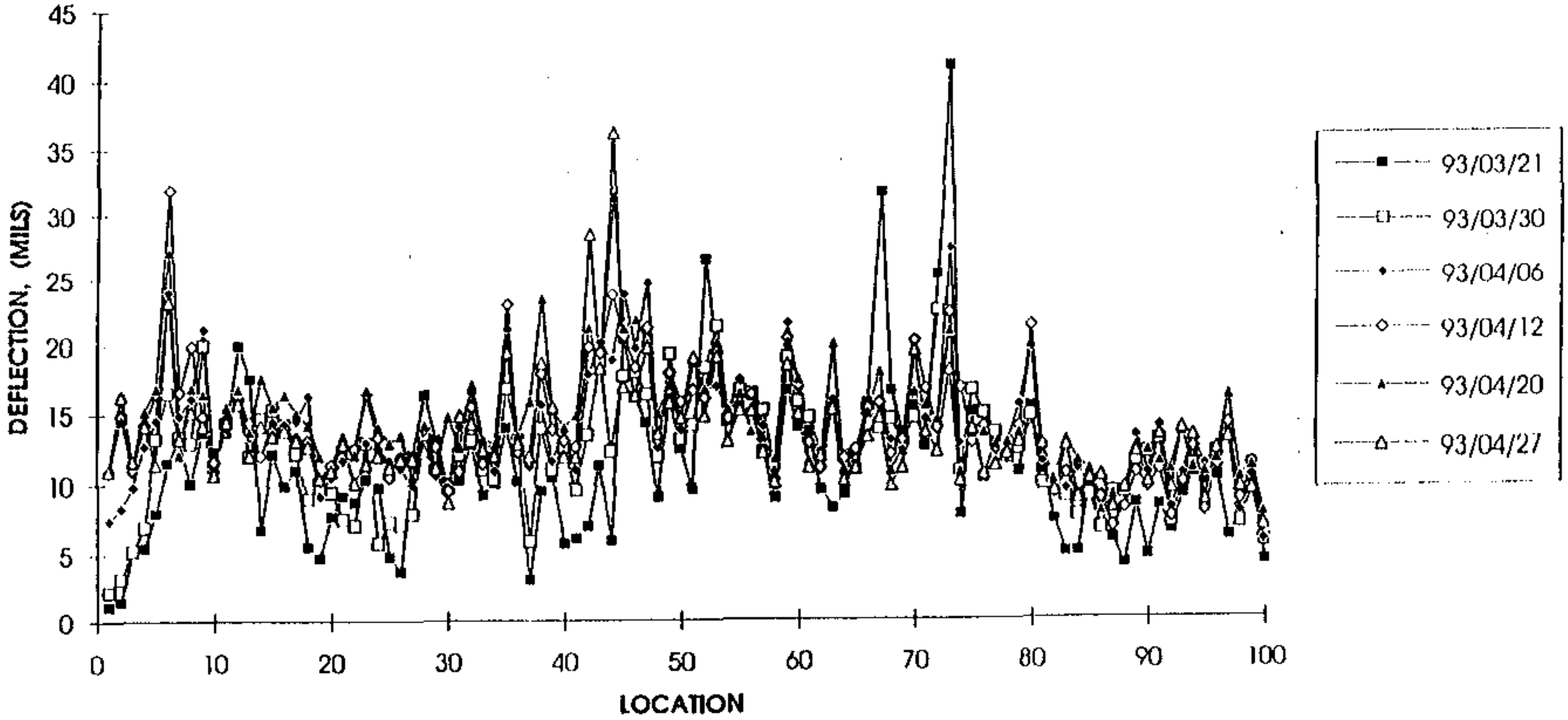
APPENDIX B -- HAINES HIGHWAY FWD DATA (1993)

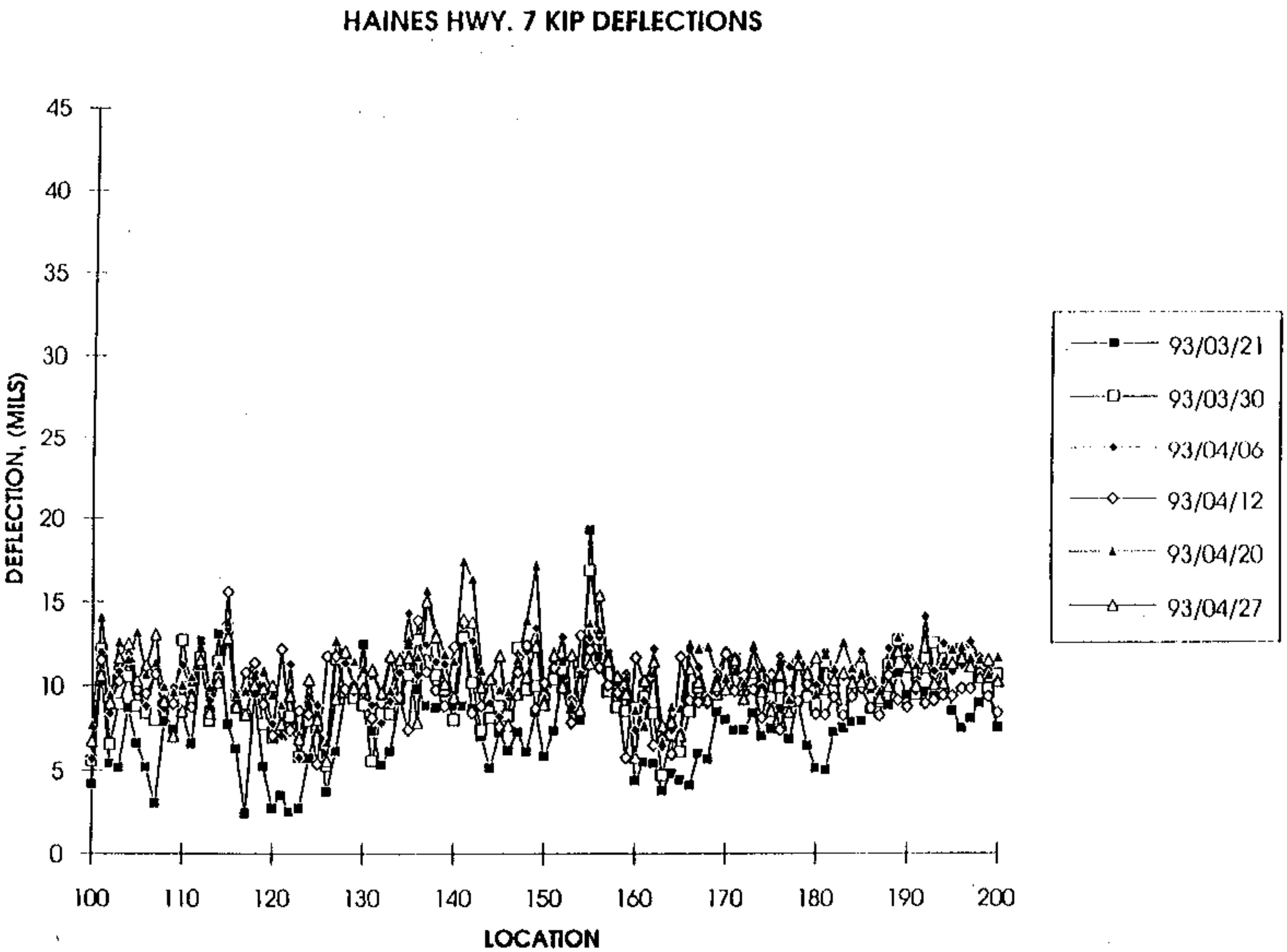
APPENDIX B-1

HAINES

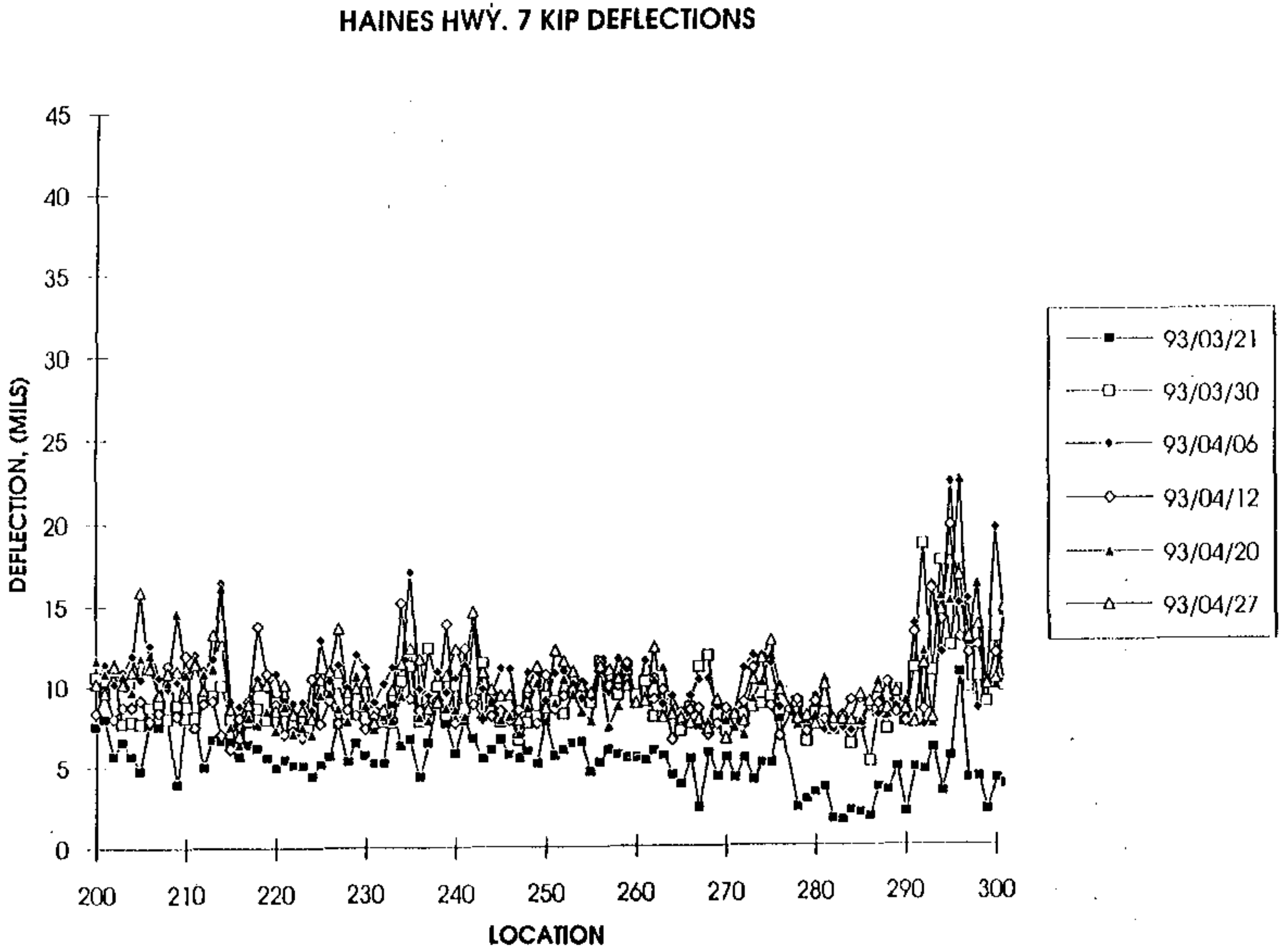
FWD SURFACE DEFLECTIONS

HAINES HWY. 7 KIP DEFLECTIONS

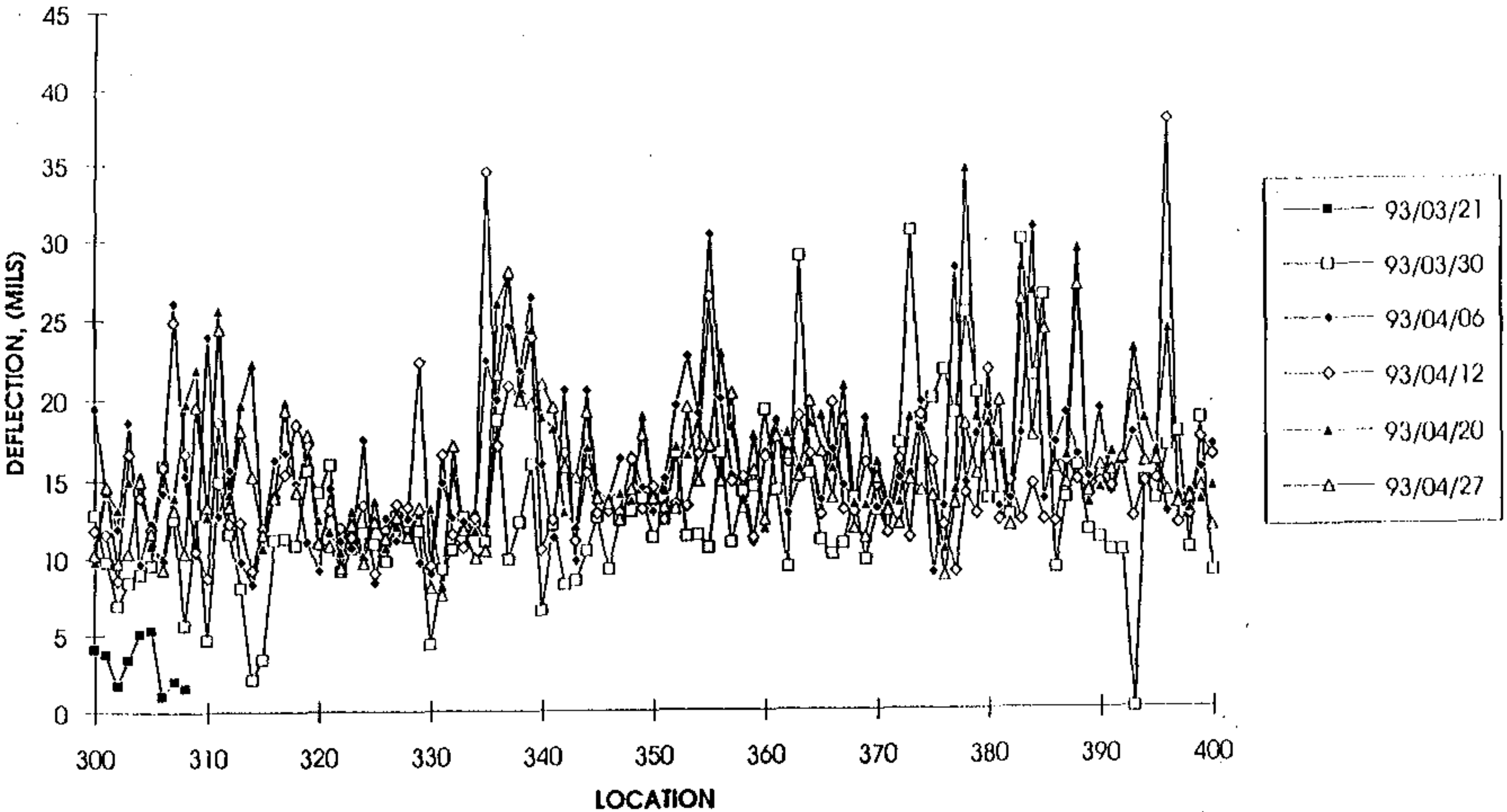




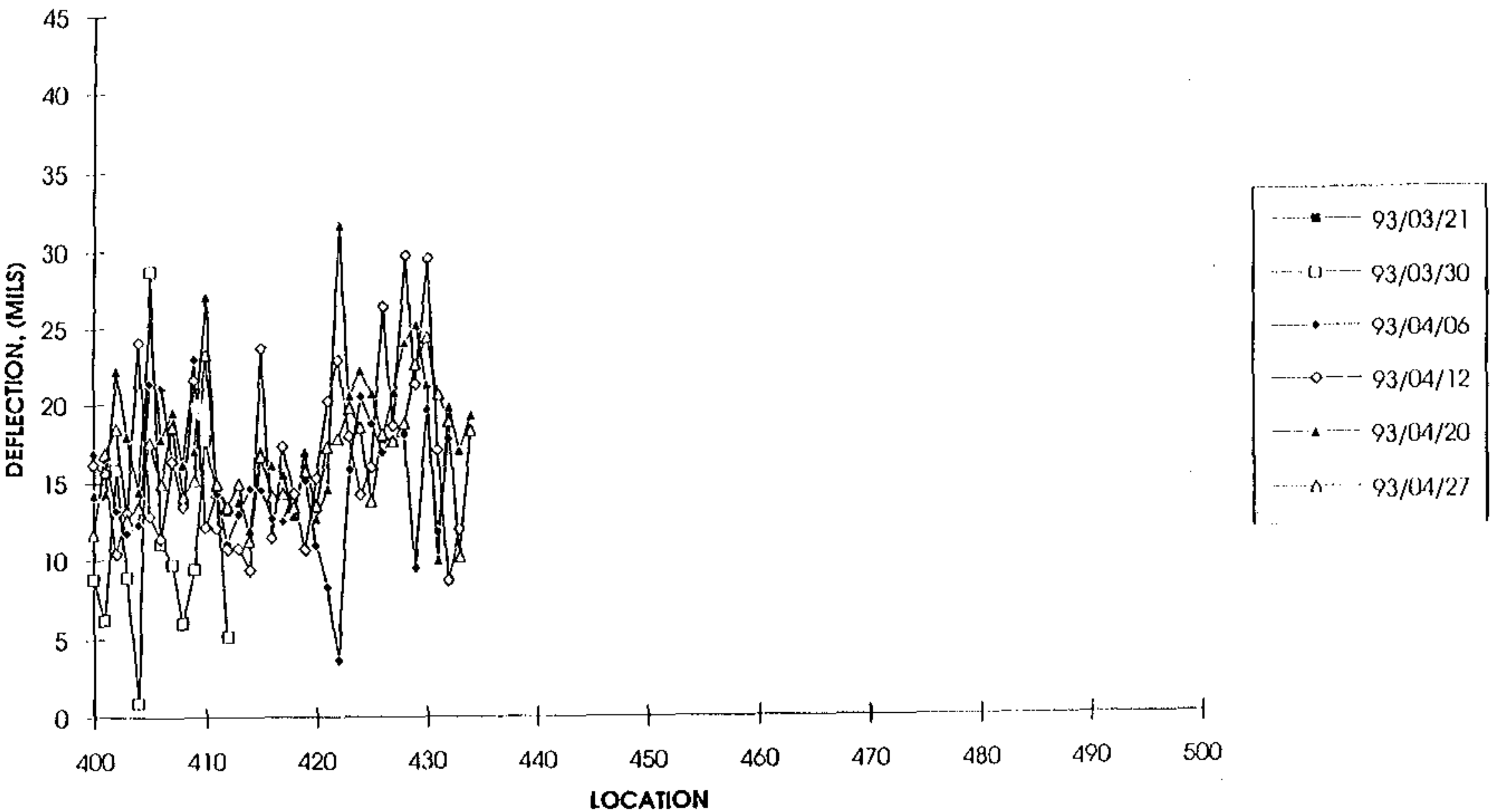
HAINES HWY. 7 KIP DEFLECTIONS
DEFLECTION, (MILS)
45
40
35
30
25
20
15
10
5
0
100
110
120
130
140
150
160
170
180
190
200
LOCATION
93/03/21
93/03/30
93/04/06
93/04/12
93/04/20
93/04/27

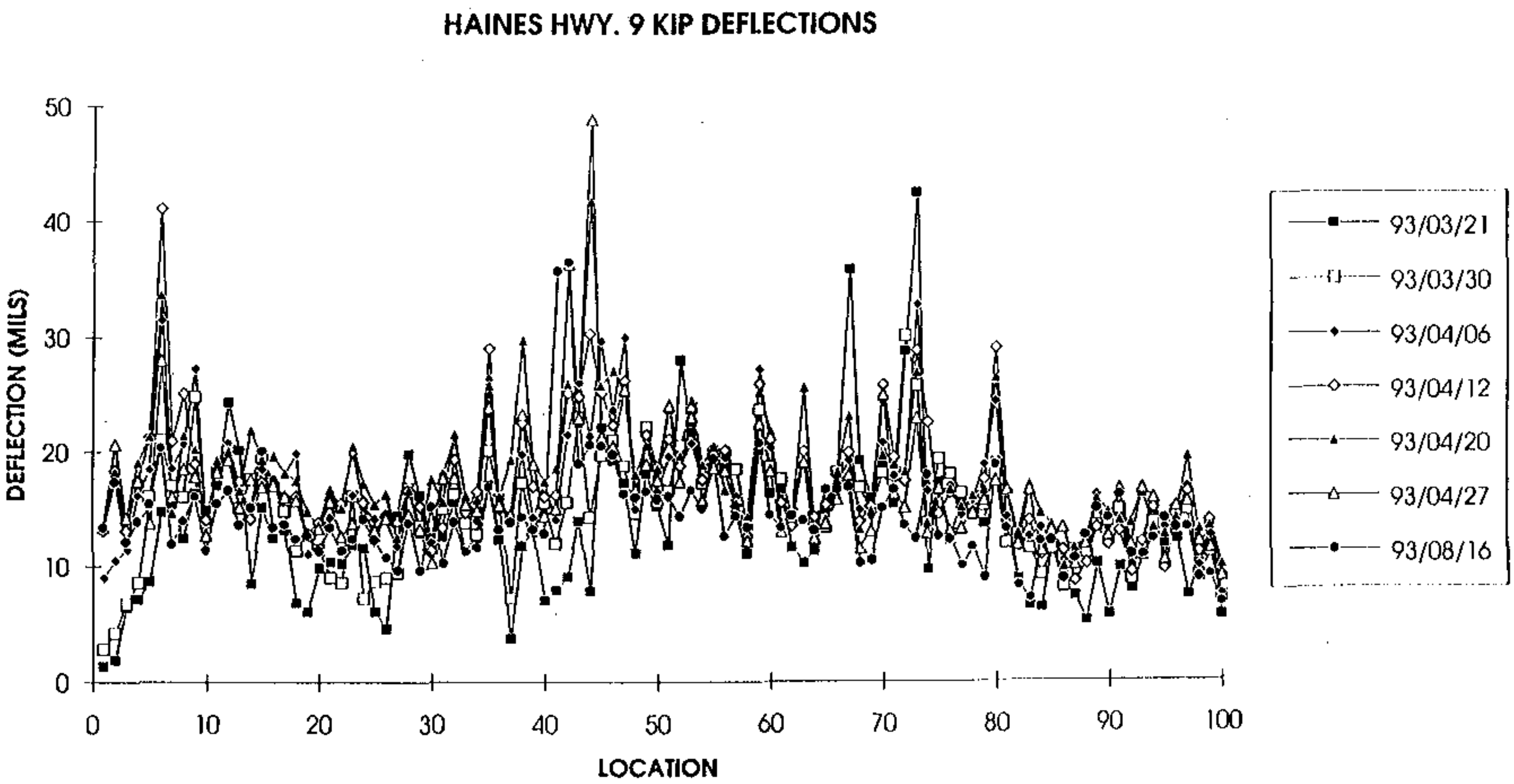

HAINES HWY. 7 KIP DEFLECTIONS
DEFLECTION, (MILS)
45
40
35
30
25
20
15
10
5
0
200
210
220
230
240
250
260
270
280
290
300
LOCATION
93/03/21
93/03/30
93/04/06
93/04/12
93/04/20
93/04/27

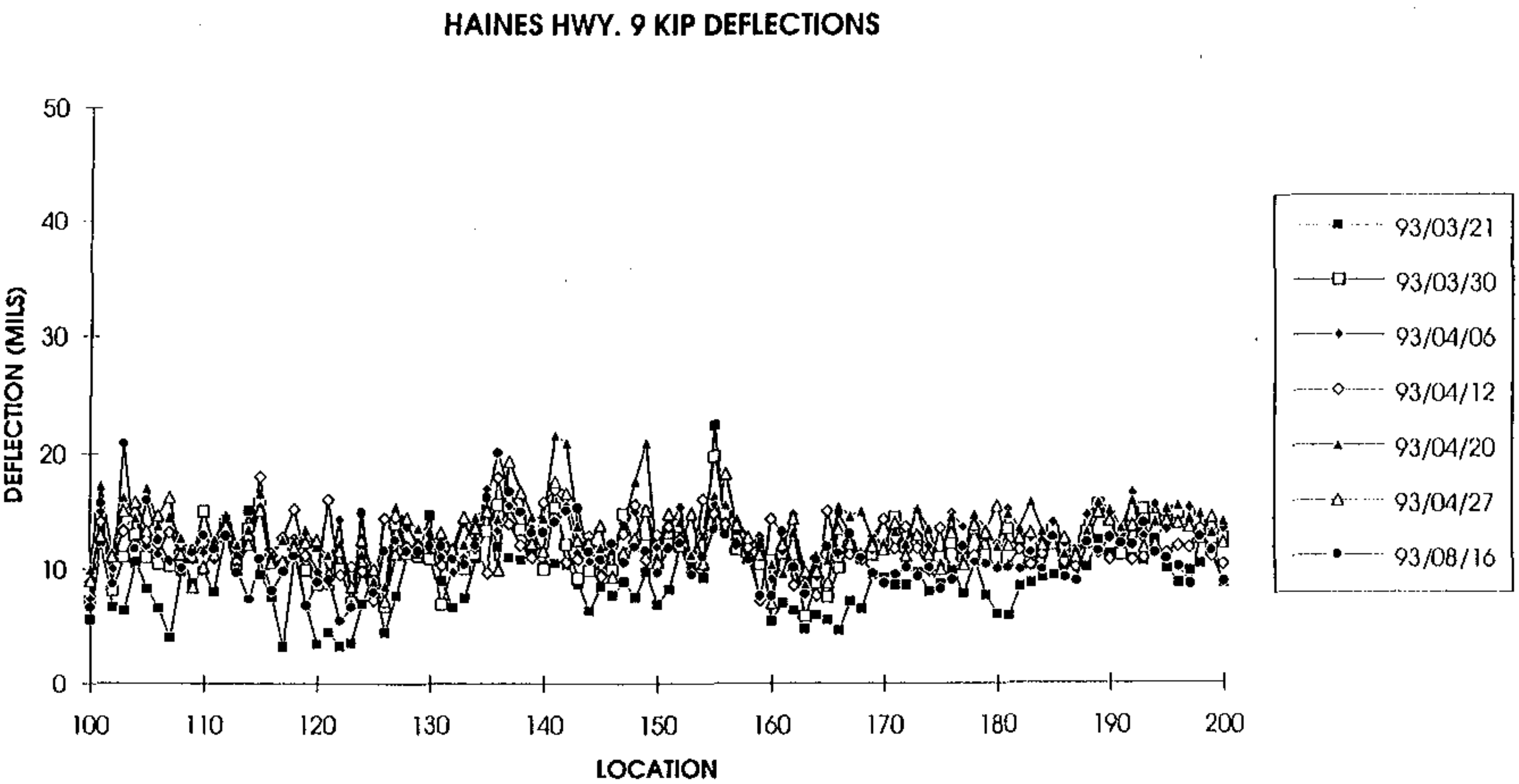
HAINES HWY. 7 KIP DEFLECTIONS

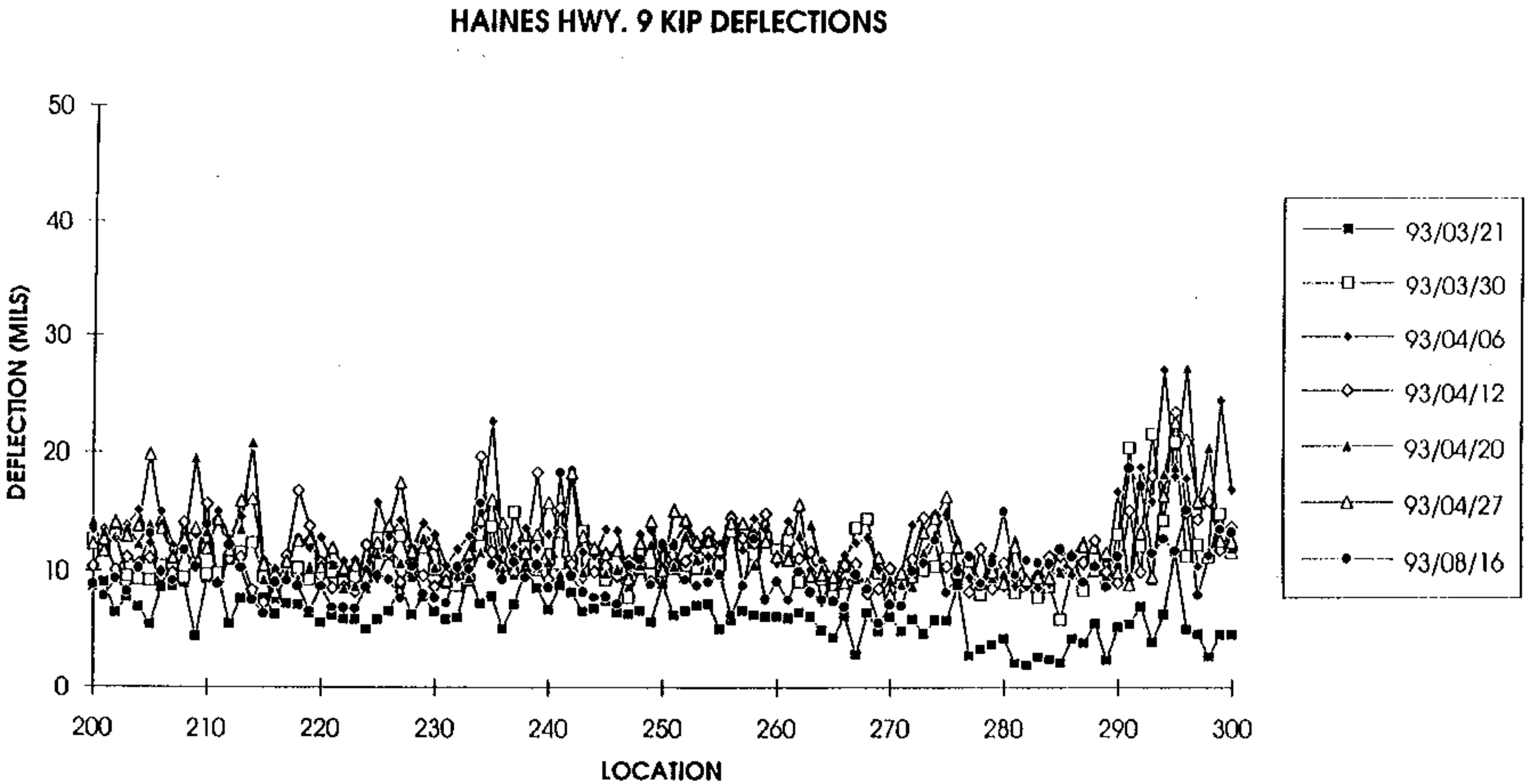


HAINES HWY. 7 KIP DEFLECTIONS

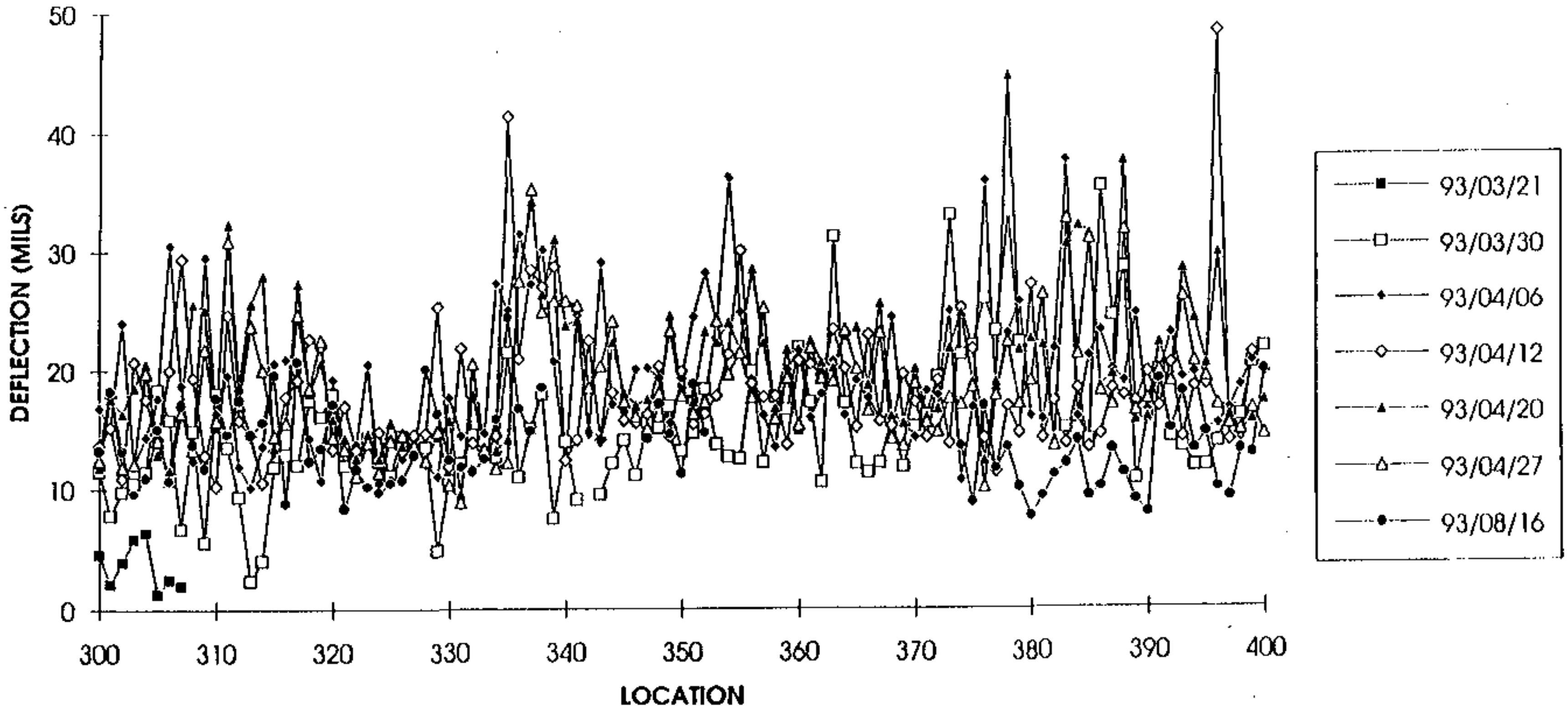



HAINES HWY. 9 KIP DEFLECTIONS
DEFLECTION (MILS)
LOCATION
0
10
20
30
40
50
60
70
80
90
100
93/03/21
93/03/30
93/04/06
93/04/12
93/04/20
93/04/27
93/08/16

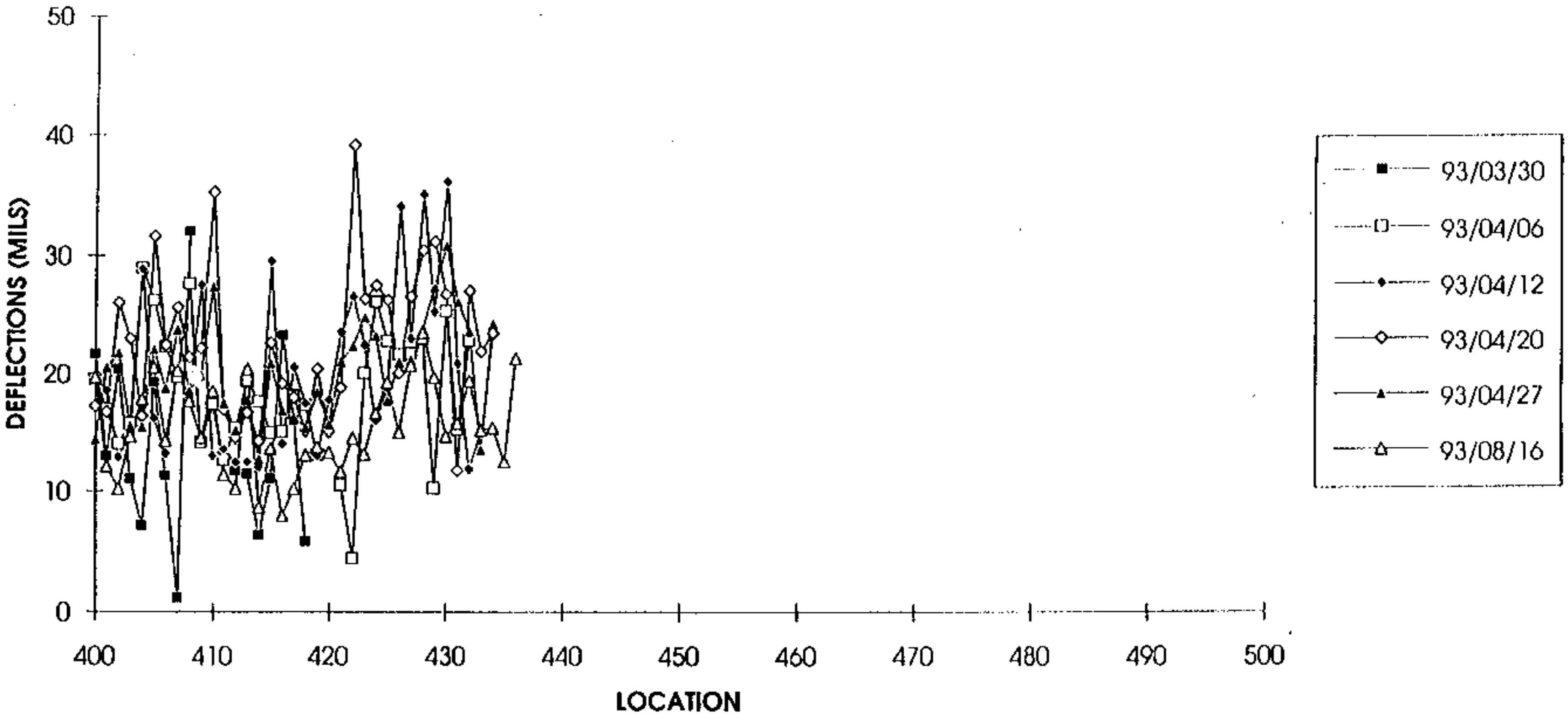

HAINES HWY. 9 KIP DEFLECTIONS
DEFLECTION (MILS)
0
10
20
30
40
50
LOCATION
100
110
120
130
140
150
160
170
180
190
200
93/03/21
93/03/30
93/04/06
93/04/12
93/04/20
93/04/27
93/08/16

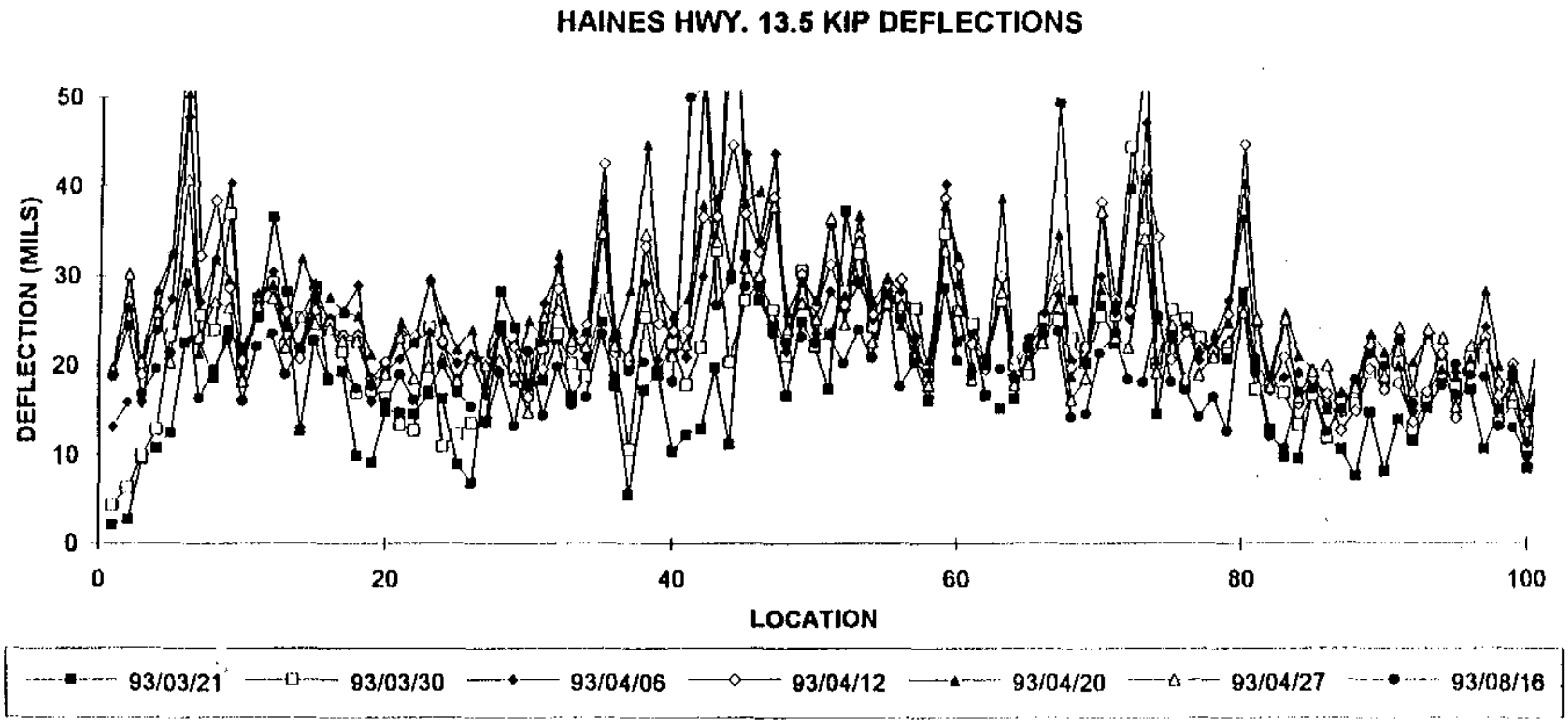

HAINES HWY. 9 KIP DEFLECTIONS
DEFLECTION (MILS)
0
10
20
30
40
50
200
210
220
230
240
250
260
270
280
290
300
LOCATION
93/03/21
93/03/30
93/04/06
93/04/12
93/04/20
93/04/27
93/08/16

HAINES HWY. 9 KIP DEFLECTIONS

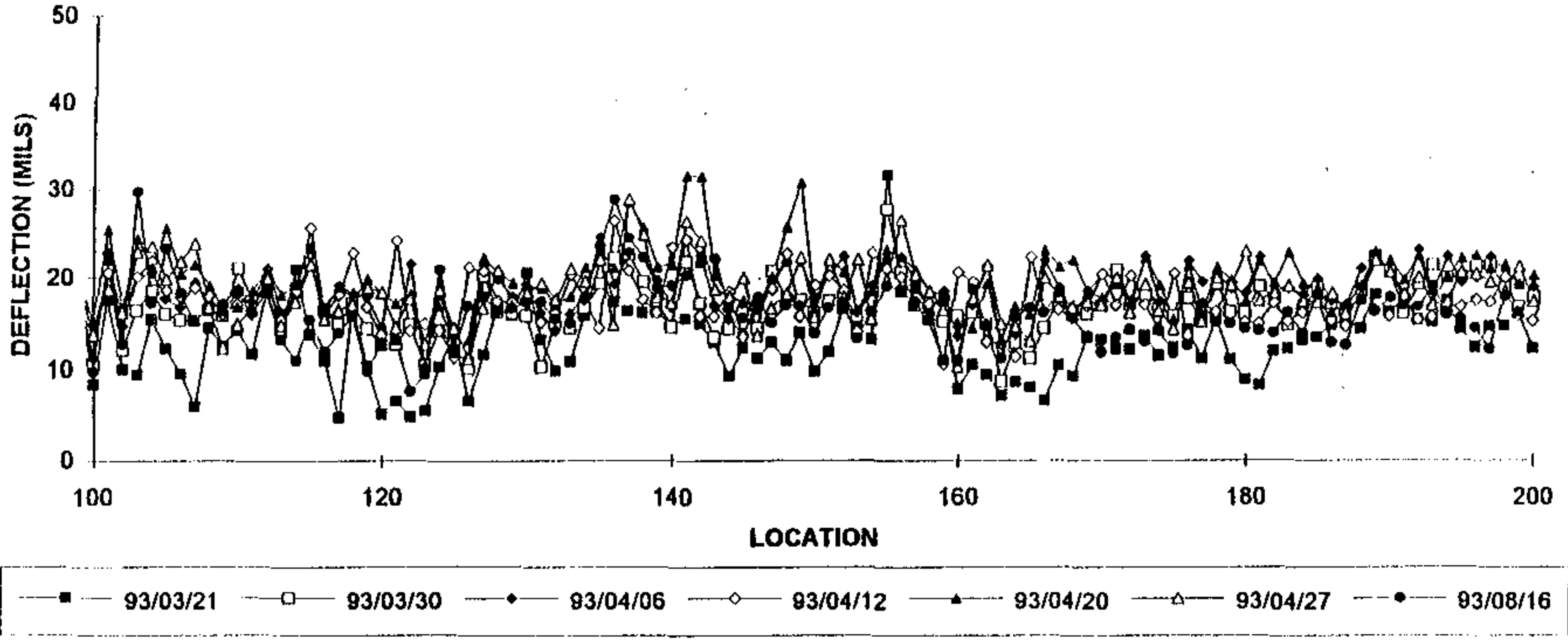


HAINES HWY. 9 KIP DEFLECTIONS

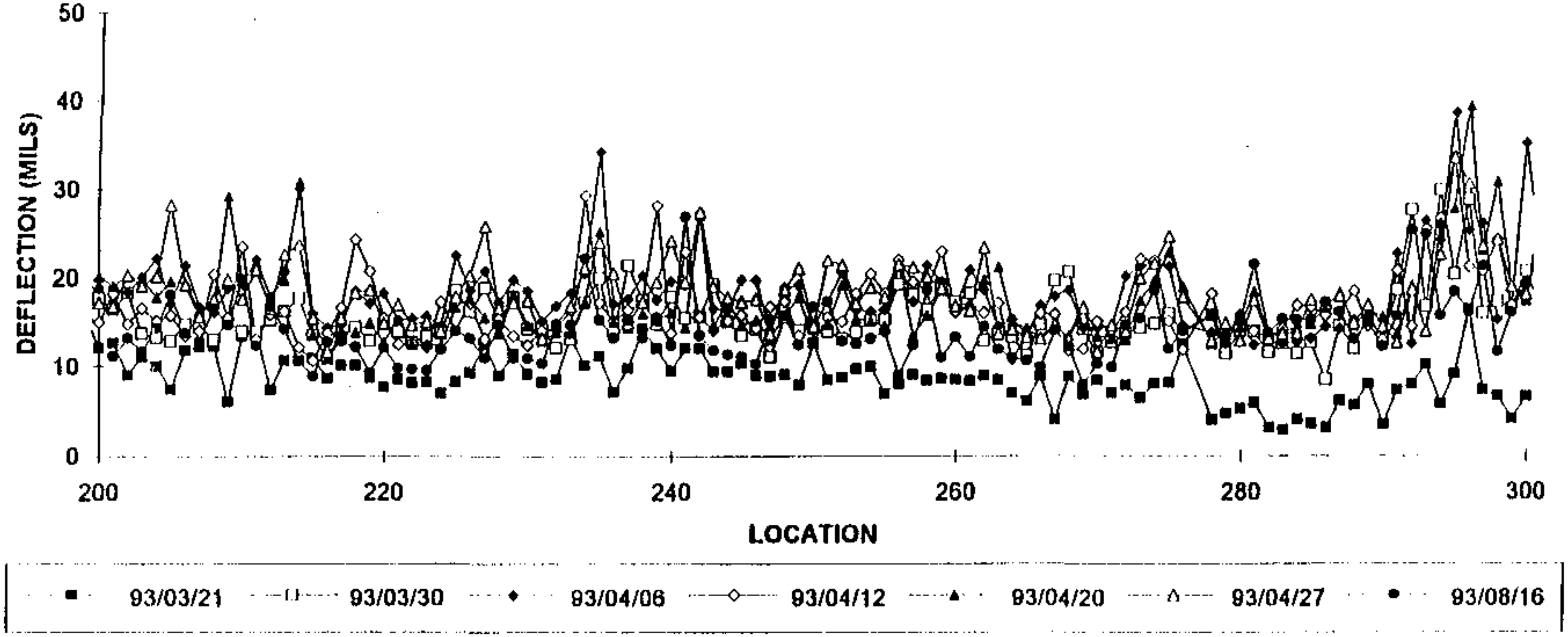


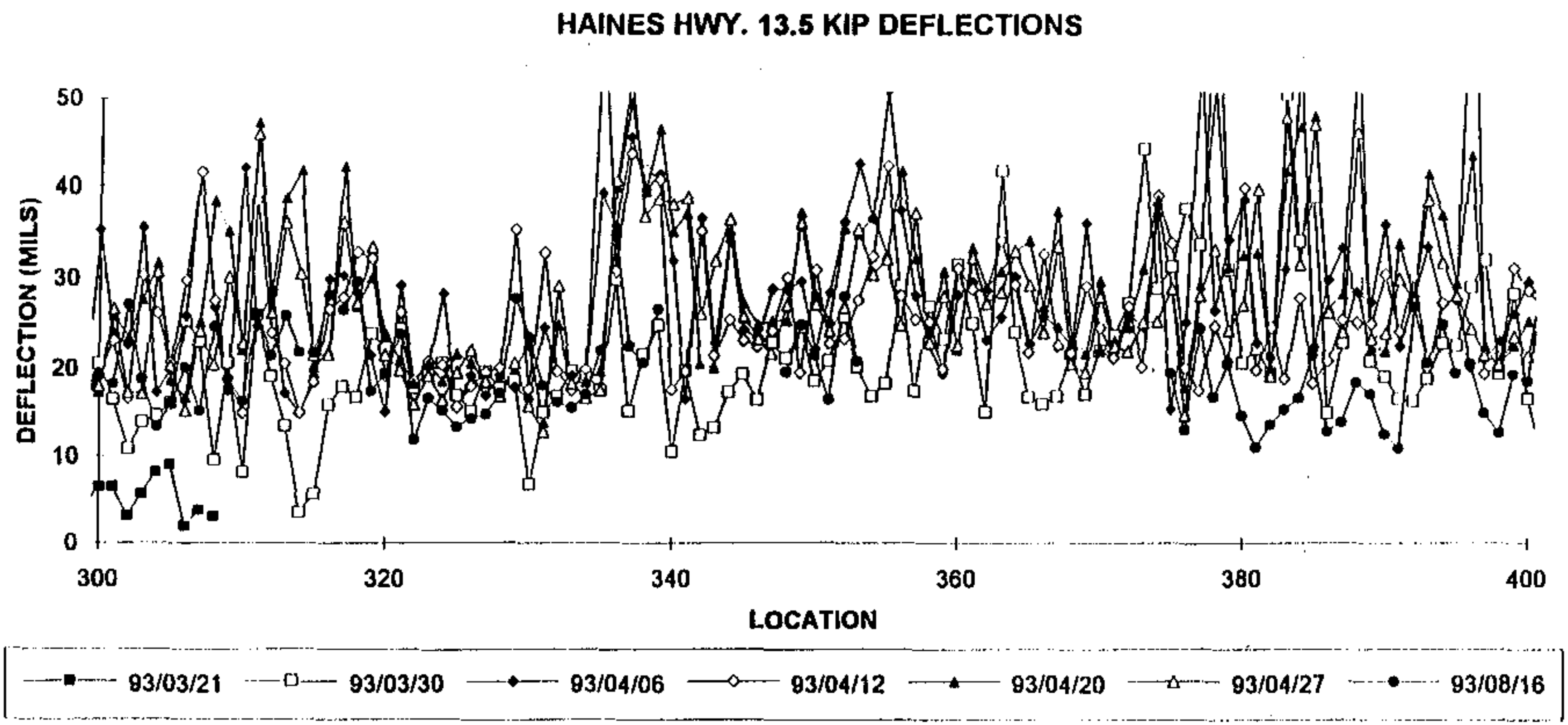

HAINES HWY. 13.5 KIP DEFLECTIONS
DEFLECTION (MILS)
0
10
20
30
40
50
0
20
40
60
80
100
LOCATION
93/03/21
93/03/30
93/04/06
93/04/12
93/04/20
93/04/27
93/08/16

HAINES HWY. 13.5 KIP DEFLECTIONS

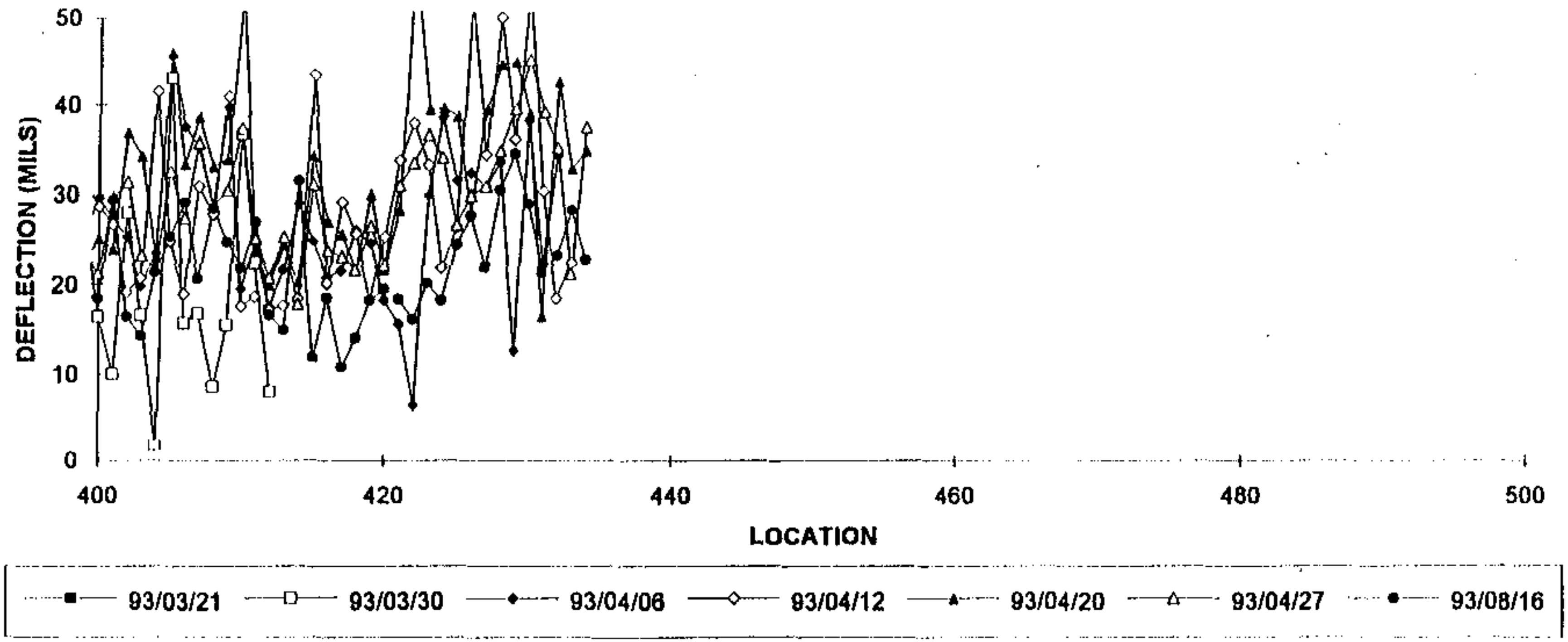


HAINES HWY. 13.5 KIP DEFLECTIONS




HAINES HWY. 13.5 KIP DEFLECTIONS
DEFLECTION (MILS)
0
10
20
30
40
50
300
320
340
360
380
400
LOCATION
93/03/21
93/03/30
93/04/06
93/04/12
93/04/20
93/04/27
93/08/16

HAINES HWY. 13.5 KIP DEFLECTIONS

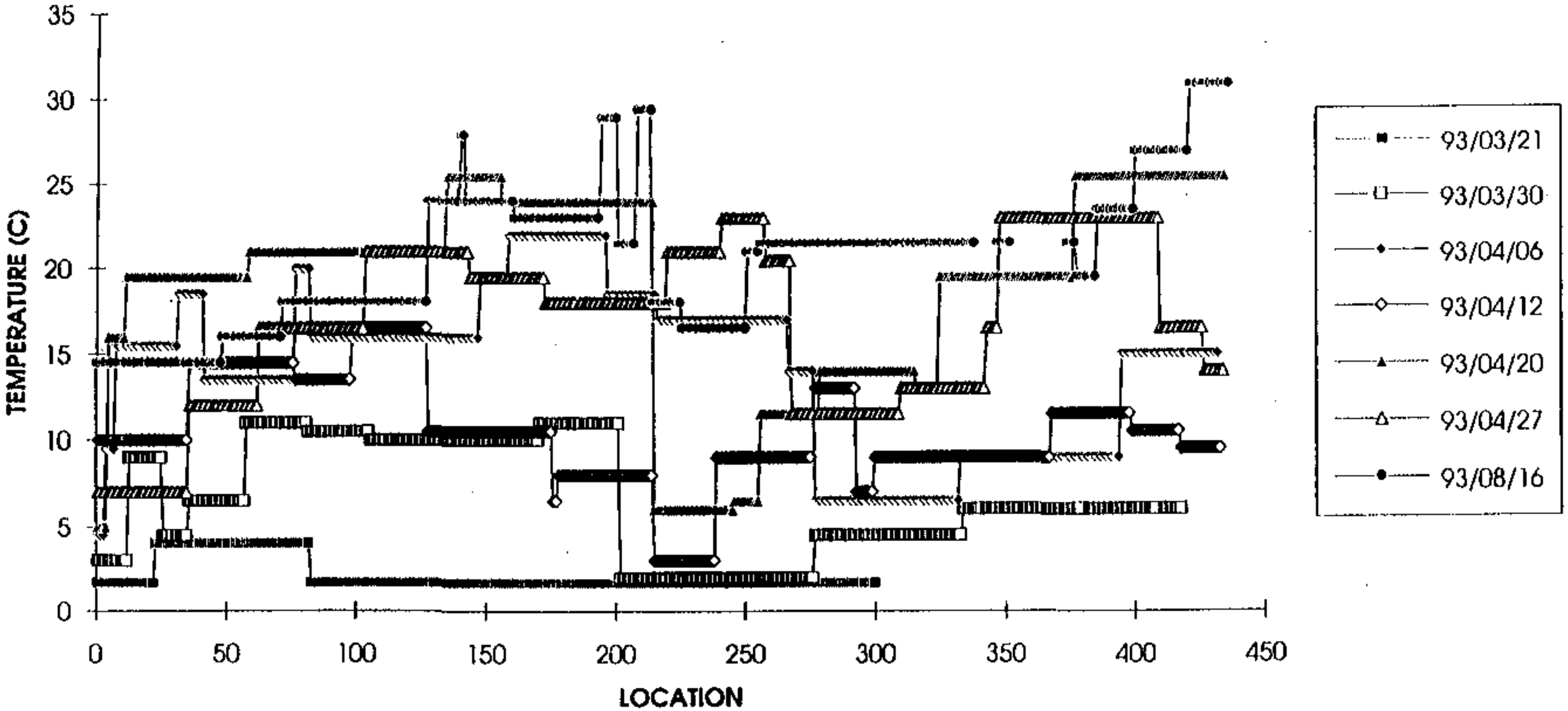


APPENDIX B-2

HAINES

PAVEMENT SURFACE TEMPERATURE

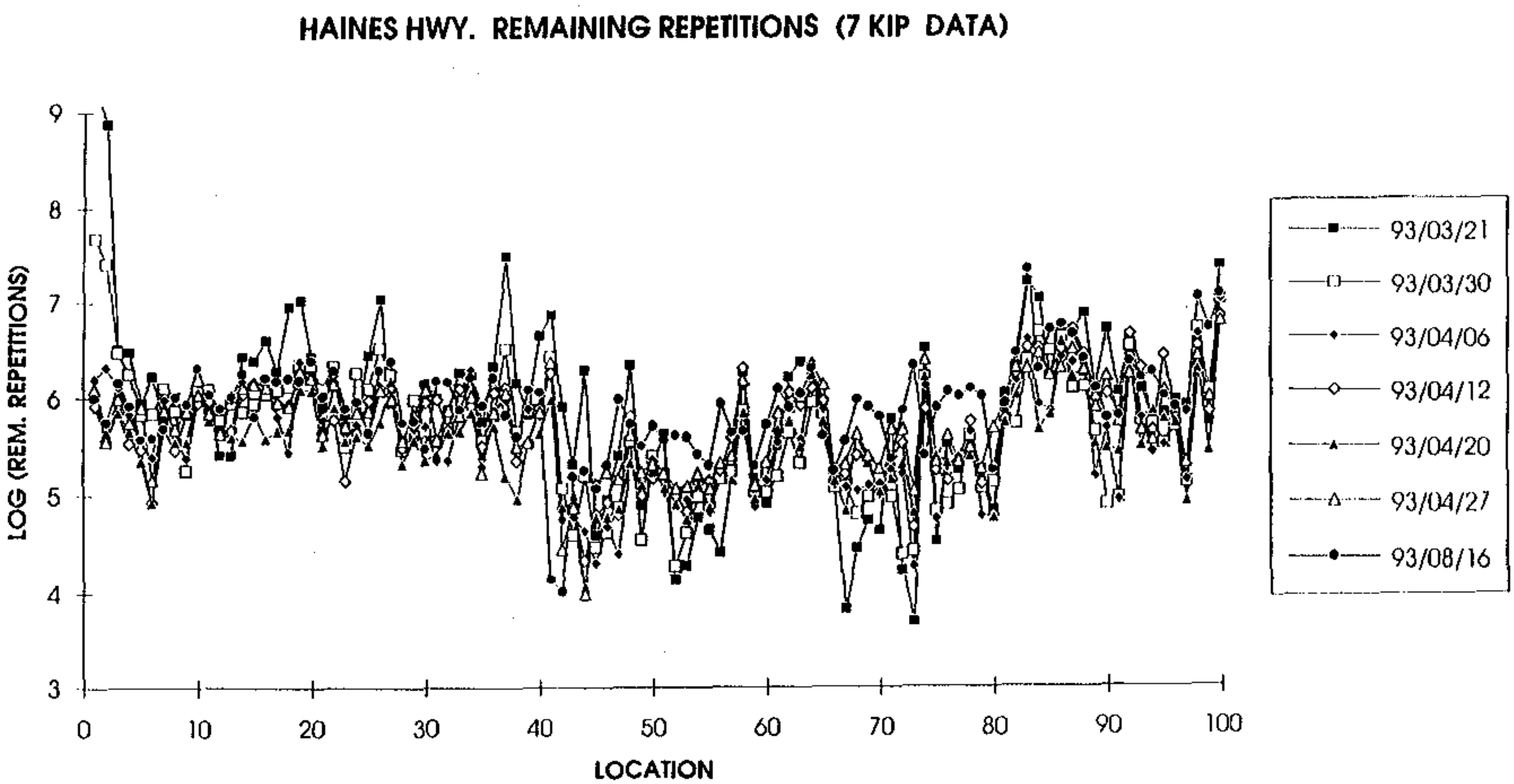
HAINES HWY. SURFACE TEMPERATURES FOR DROPS 1, 2 & 3



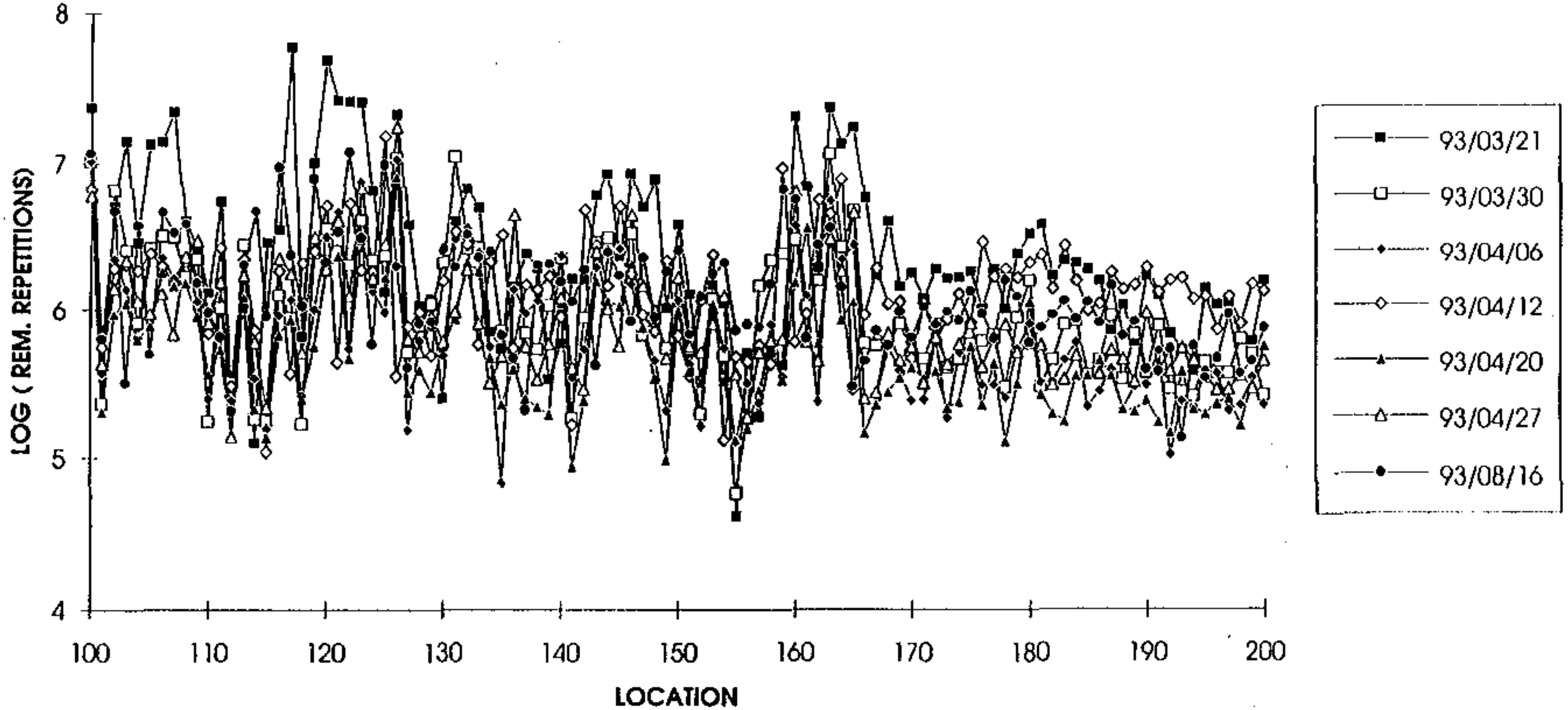
APPENDIX B-3

HAINES

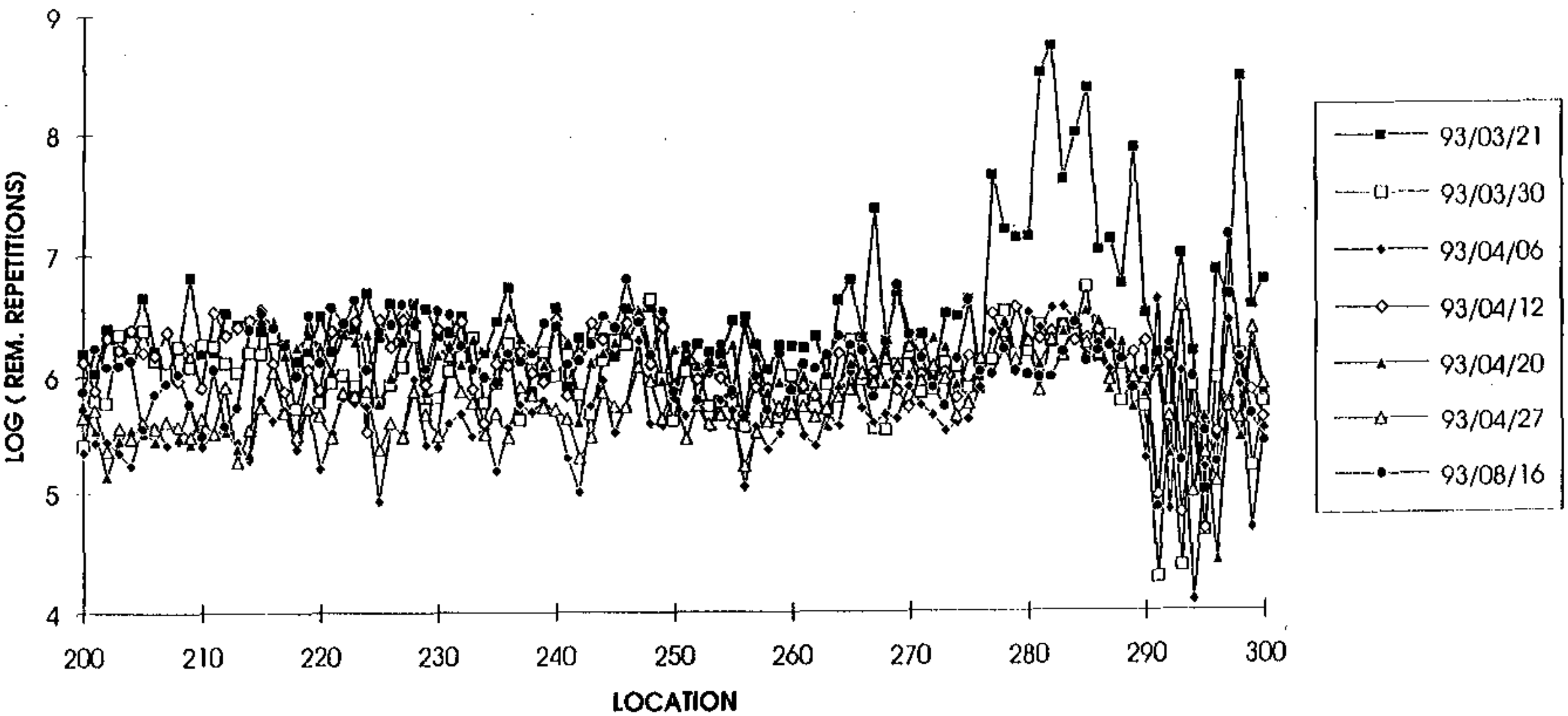
REMAINING LIFE REPETITIONS


HAINES HWY. REMAINING REPETITIONS (7 KIP DATA)
LOG (REM. REPETITIONS)
9
8
7
6
5
4
3
0
10
20
30
40
50
60
70
80
90
100
LOCATION
93/03/21
93/03/30
93/04/06
93/04/12
93/04/20
93/04/27
93/08/16

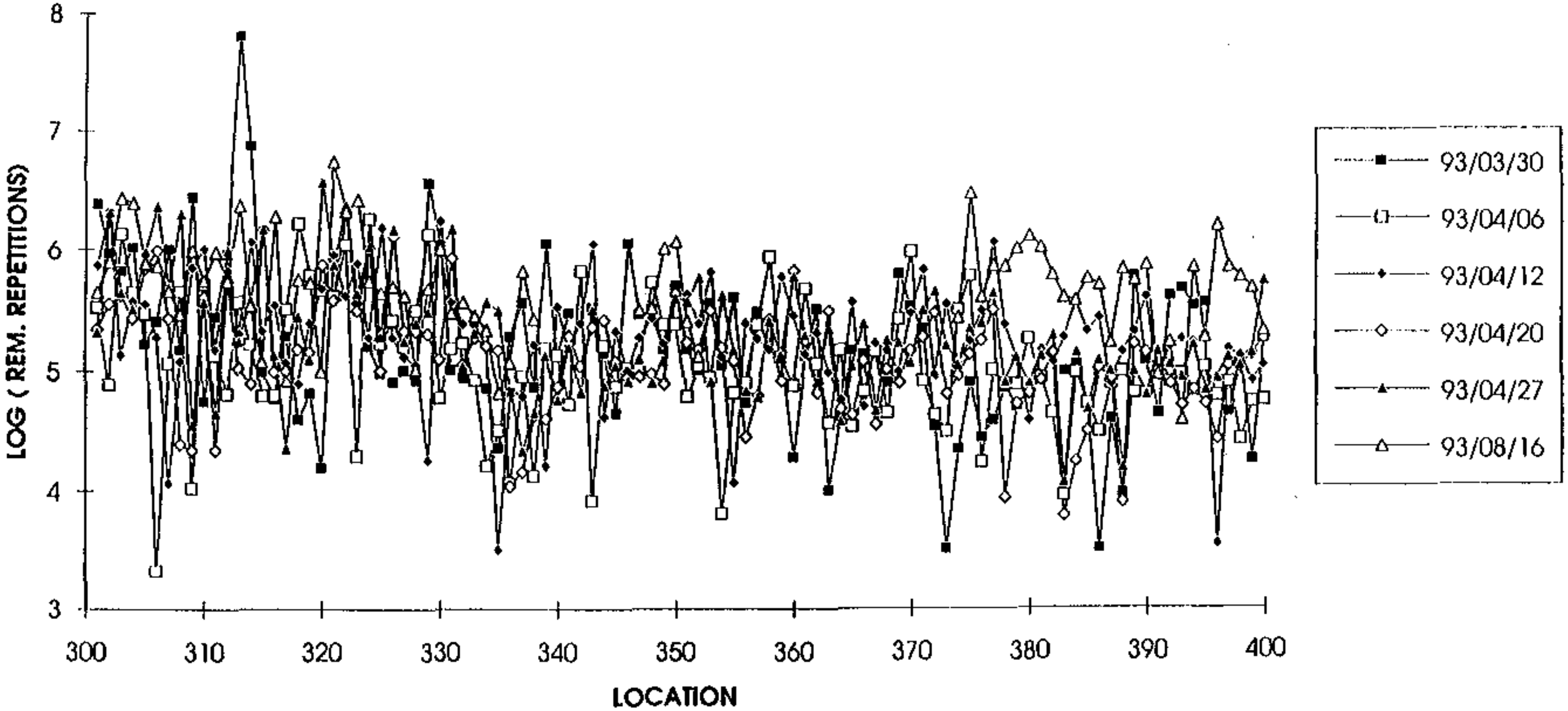
HAINES HWY. REMAINING REPETITIONS (7 KIP DATA)



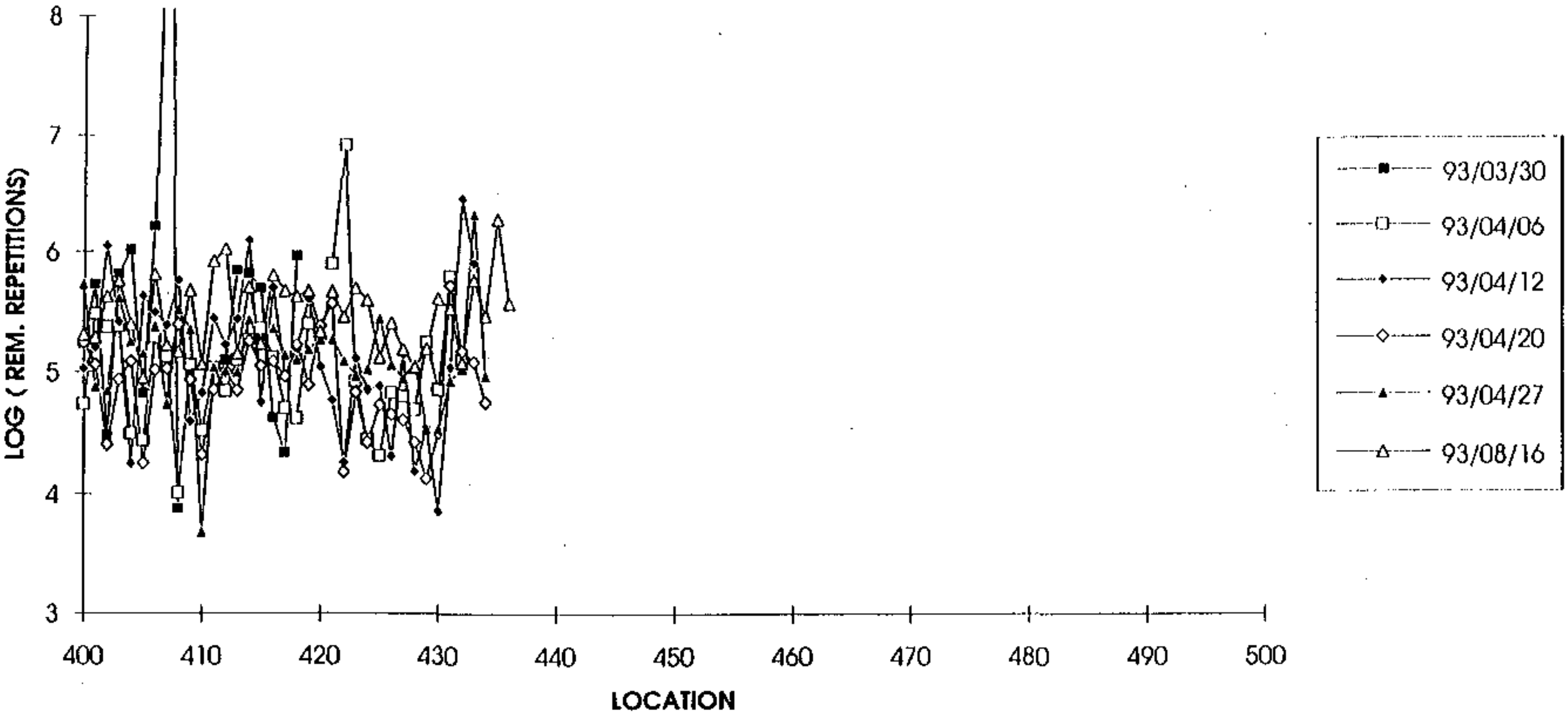
HAINES HWY. REMAINING REPETITIONS (7 KIP DATA)

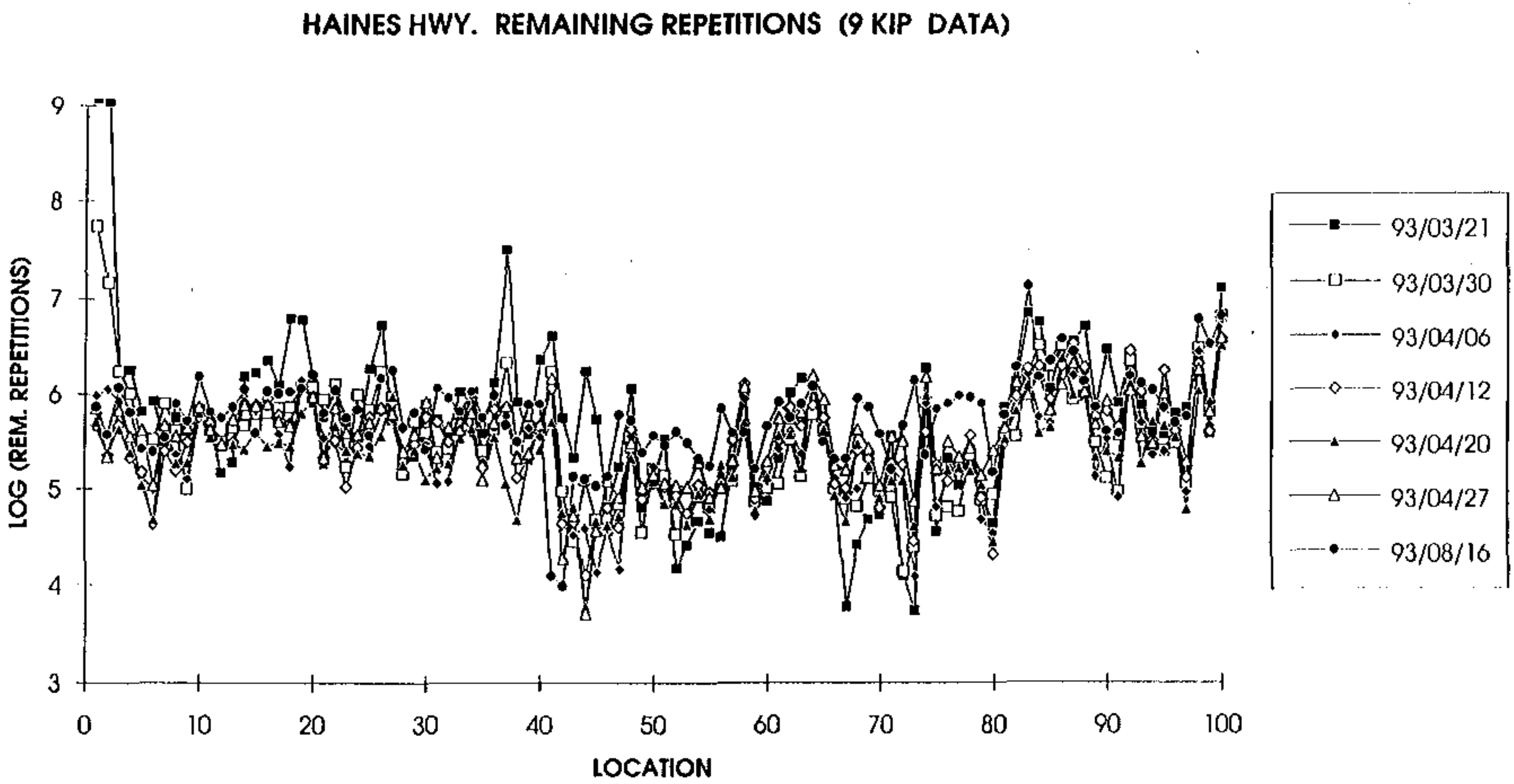


HAINES HWY. REMAINING REPETITIONS (7 KIP DATA)

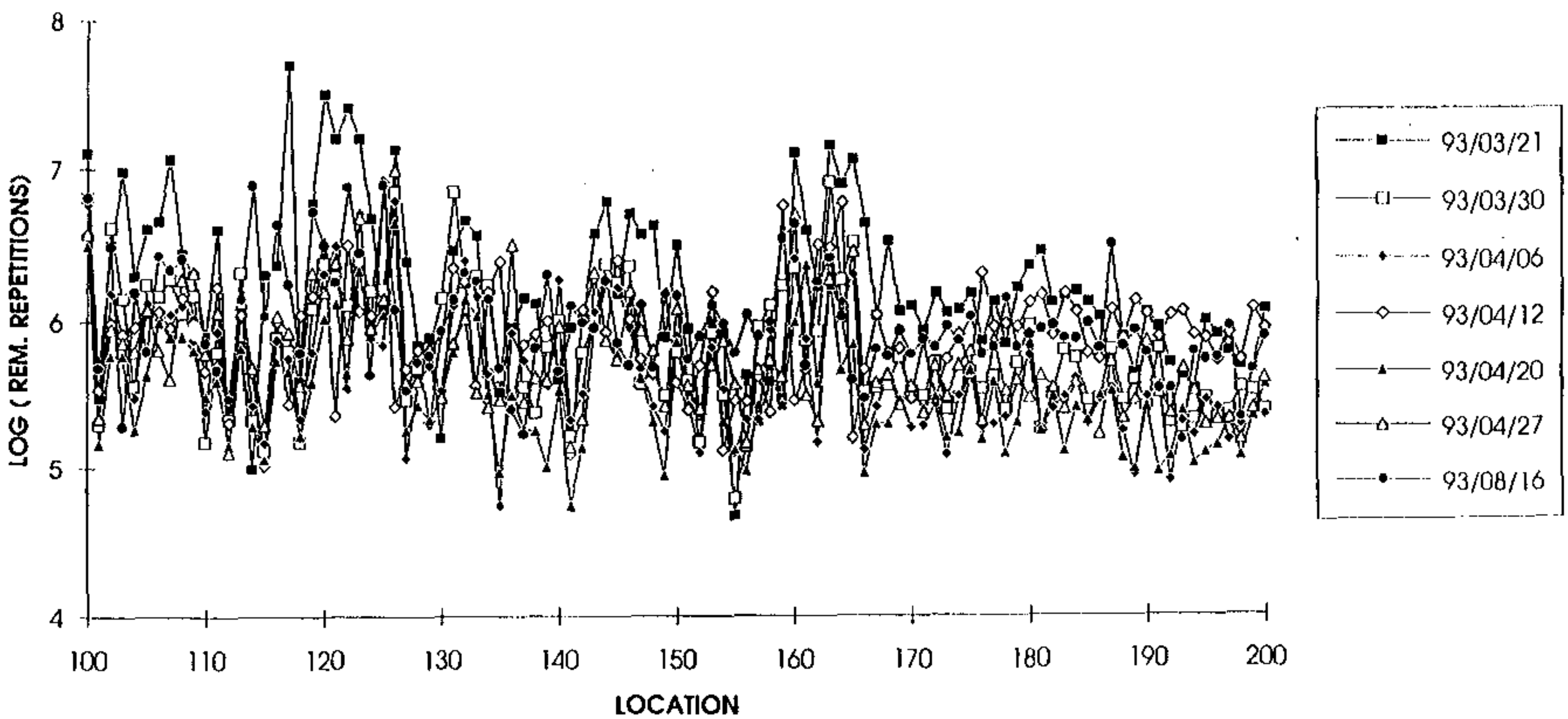


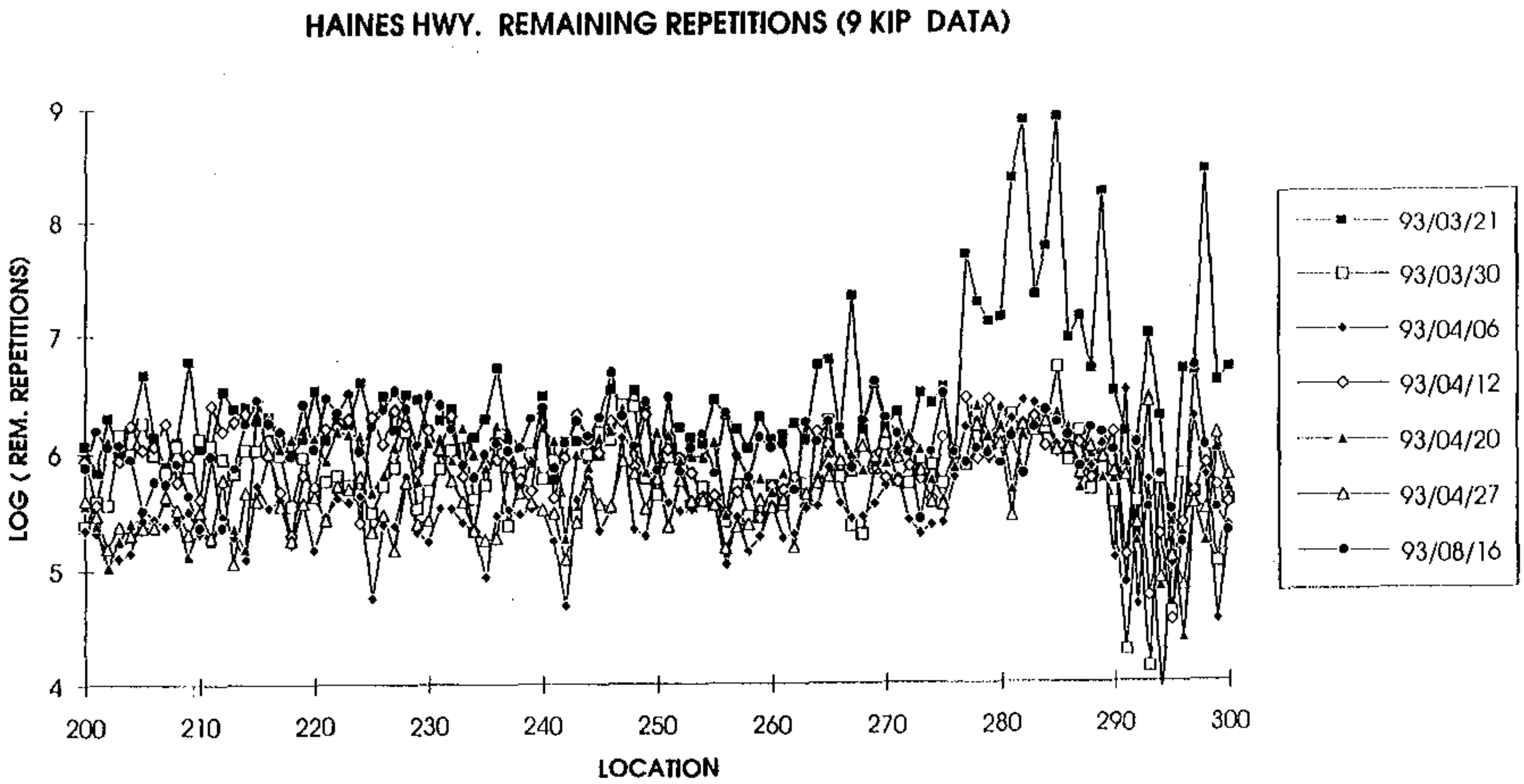
HAINES HWY. REMAINING REPETITIONS (7 KIP DATA)




HAINES HWY. REMAINING REPETITIONS (9 KIP DATA)
LOG (REM. REPETITIONS)
9
8
7
6
5
4
3
0
10
20
30
40
50
60
70
80
90
100
LOCATION
93/03/21
93/03/30
93/04/06
93/04/12
93/04/20
93/04/27
93/08/16

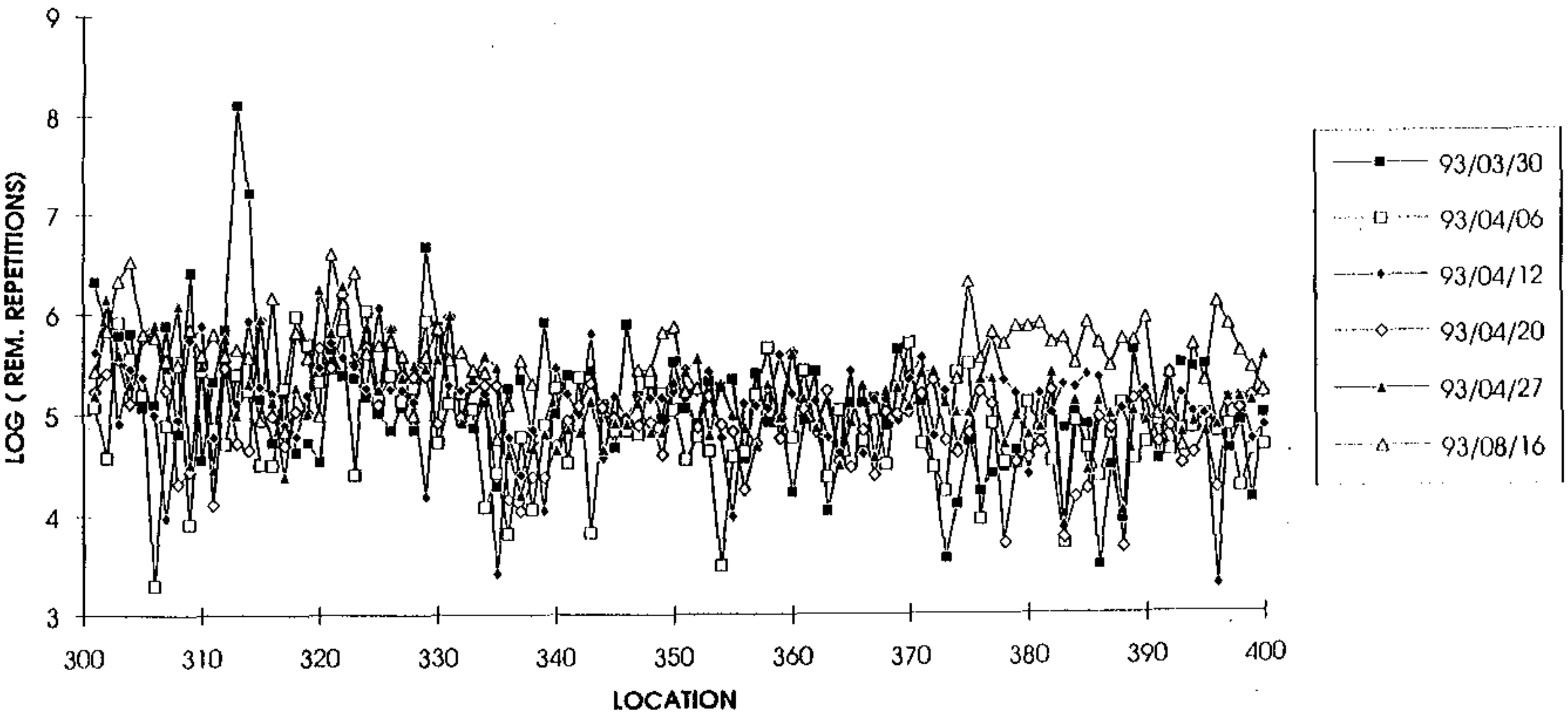
HAINES HWY. REMAINING REPETITIONS (9 KIP DATA)

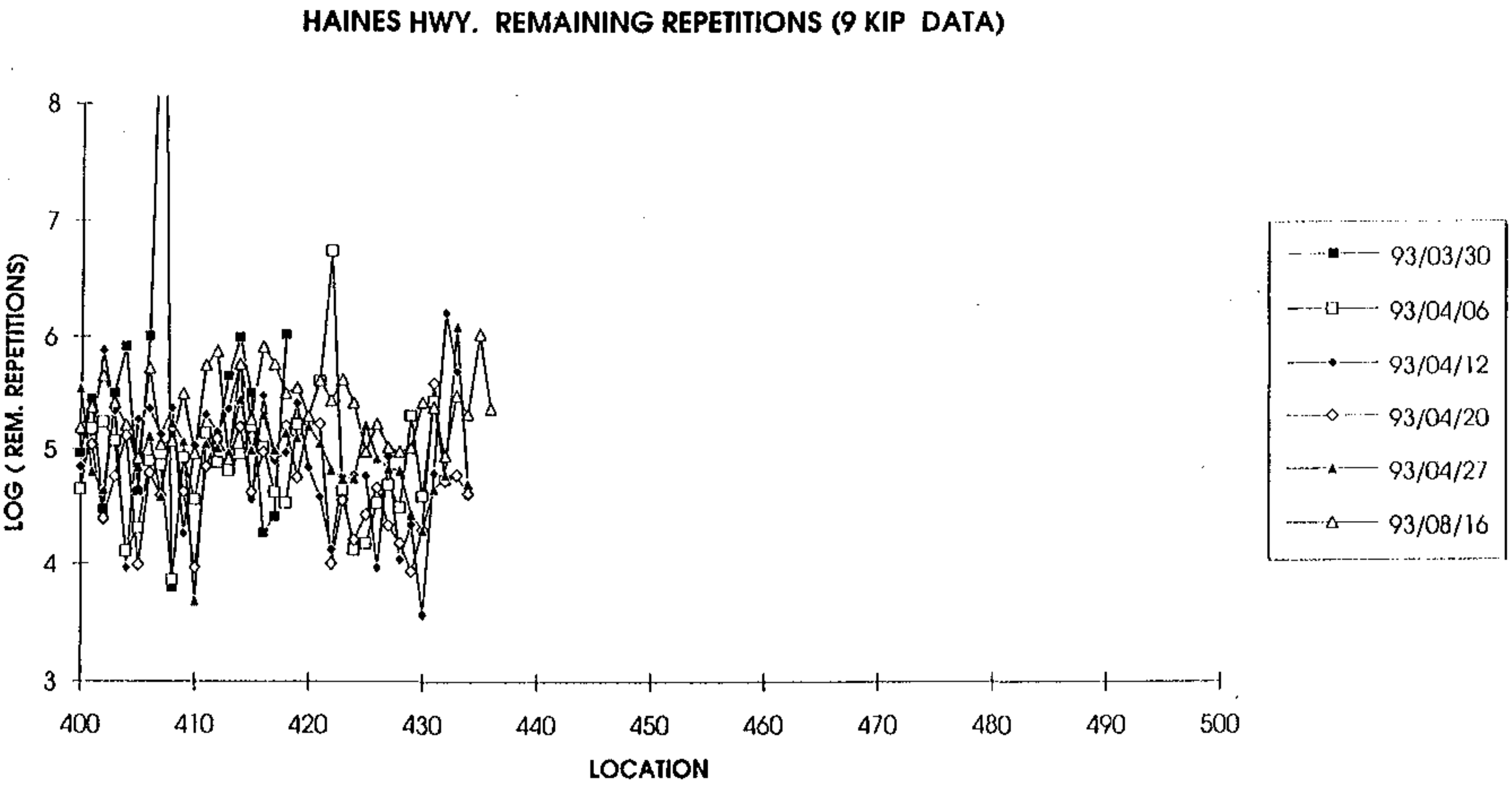




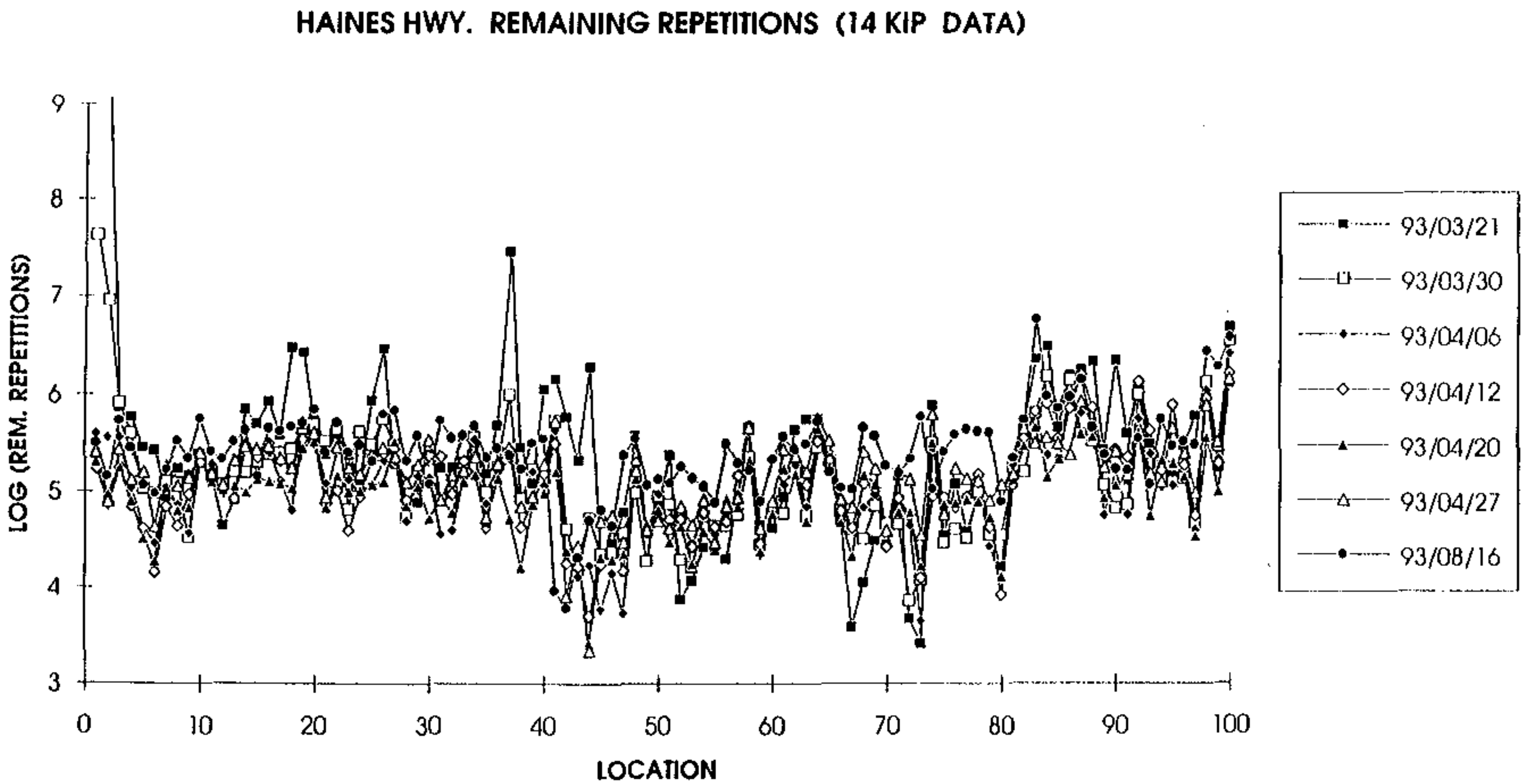
HAINES HWY. REMAINING REPETITIONS (9 KIP DATA)
LOG (REM. REPETITIONS)
9
8
7
6
5
4
200
210
220
230
240
250
260
270
280
290
300
LOCATION
93/03/21
93/03/30
93/04/06
93/04/12
93/04/20
93/04/27
93/08/16

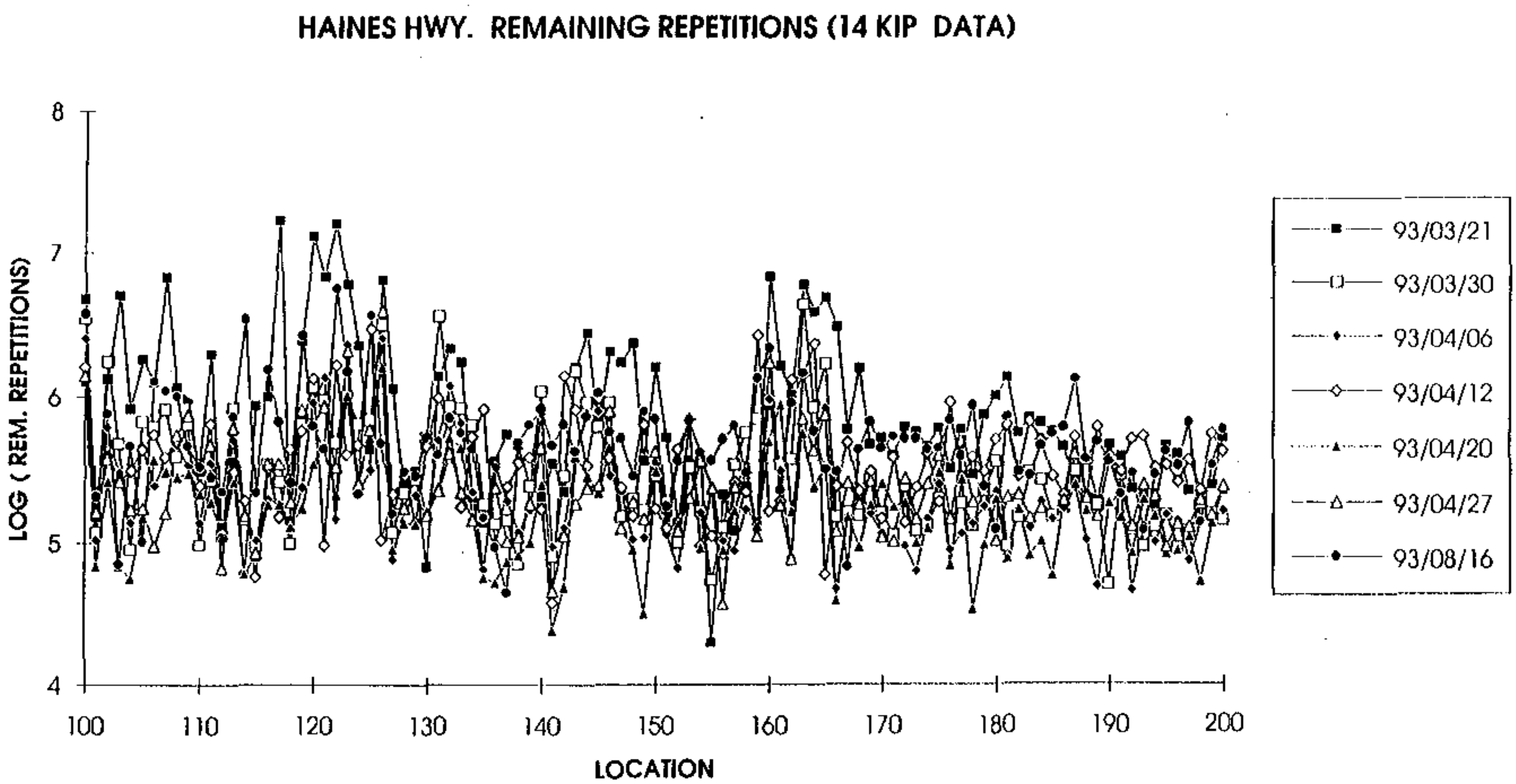
HAINES HWY. REMAINING REPETITIONS (9 KIP DATA)





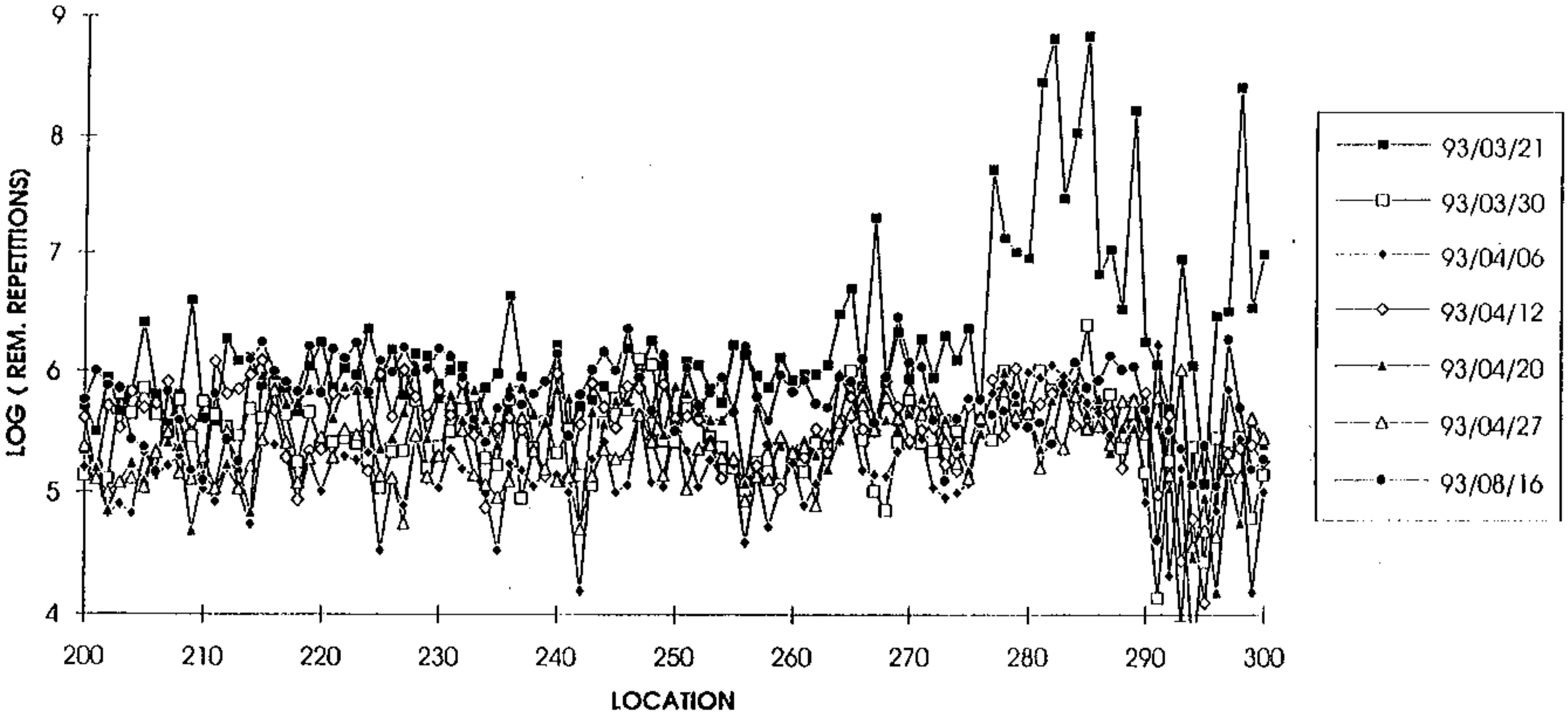
HAINES HWY. REMAINING REPETITIONS (9 KIP DATA)
LOG (REM. REPETITIONS)
8
7
6
5
4
3
400
410
420
430
440
450
460
470
480
490
500
LOCATION
93/03/30
93/04/06
93/04/12
93/04/20
93/04/27
93/08/16

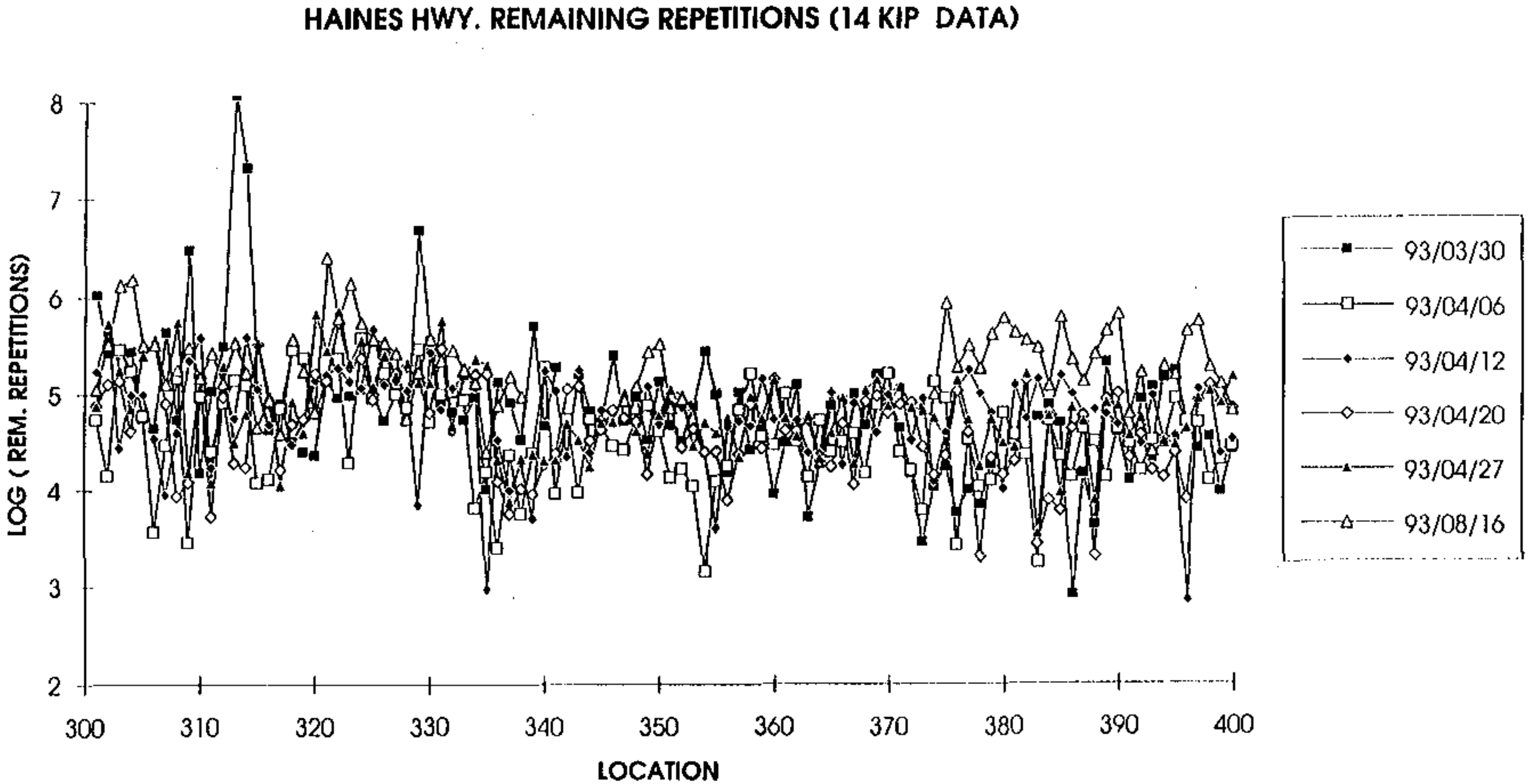

HAINES HWY. REMAINING REPETITIONS (14 KIP DATA)
LOG (REM. REPETITIONS)
LOCATION
93/03/21
93/03/30
93/04/06
93/04/12
93/04/20
93/04/27
93/08/16



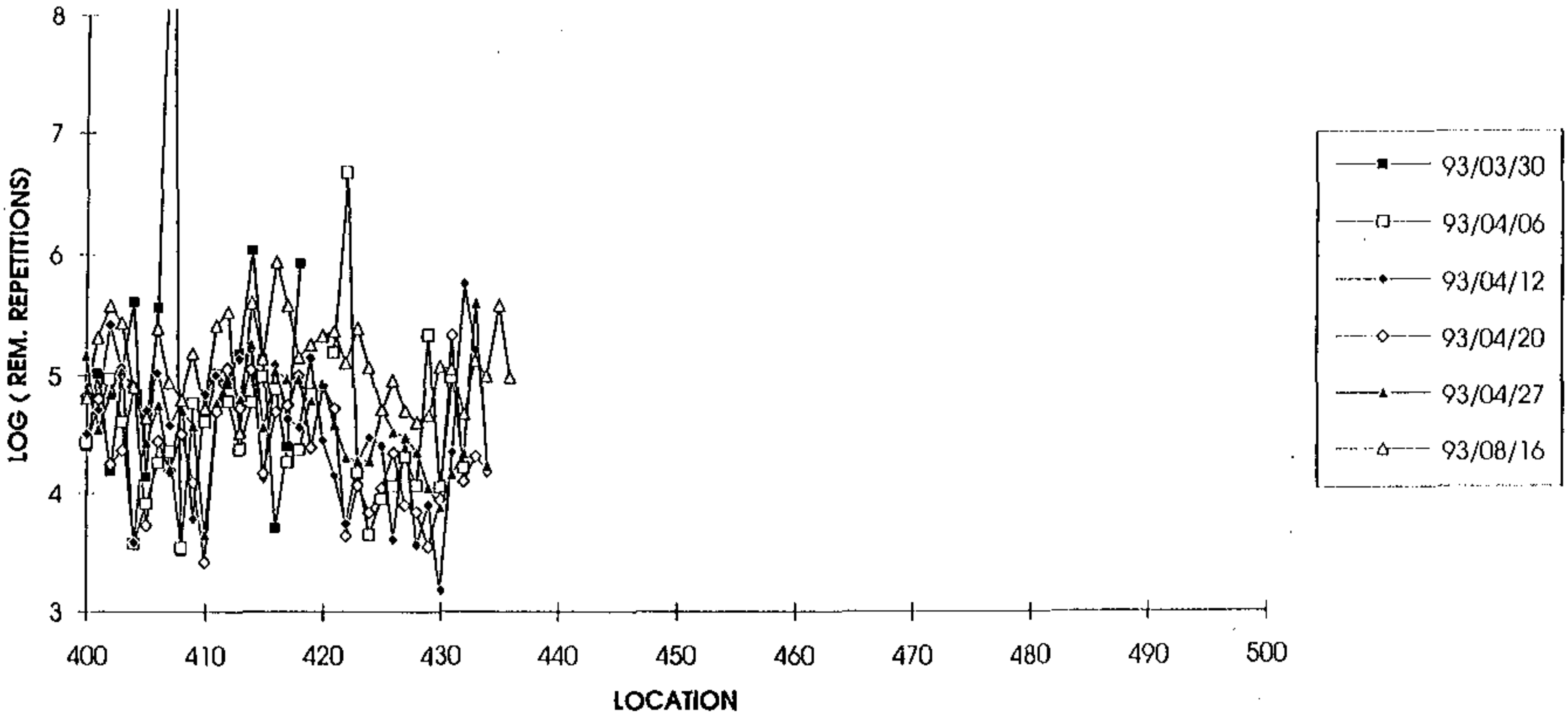
HAINES HWY. REMAINING REPETITIONS (14 KIP DATA)
LOG (REM. REPETITIONS)
8
7
6
5
4
100
110
120
130
140
150
160
170
180
190
200
LOCATION
93/03/21
93/03/30
93/04/06
93/04/12
93/04/20
93/04/27
93/08/16

HAINES HWY. REMAINING REPETITIONS (14 KIP DATA)




HAINES HWY. REMAINING REPETITIONS (14 KIP DATA)
LOG (REM. REPETITIONS)
8
7
6
5
4
3
2
300
310
320
330
340
350
360
370
380
390
400
LOCATION
93/03/30
93/04/06
93/04/12
93/04/20
93/04/27
93/08/16

HAINES HWY. REMAINING REPETITIONS (14 KIP DATA)

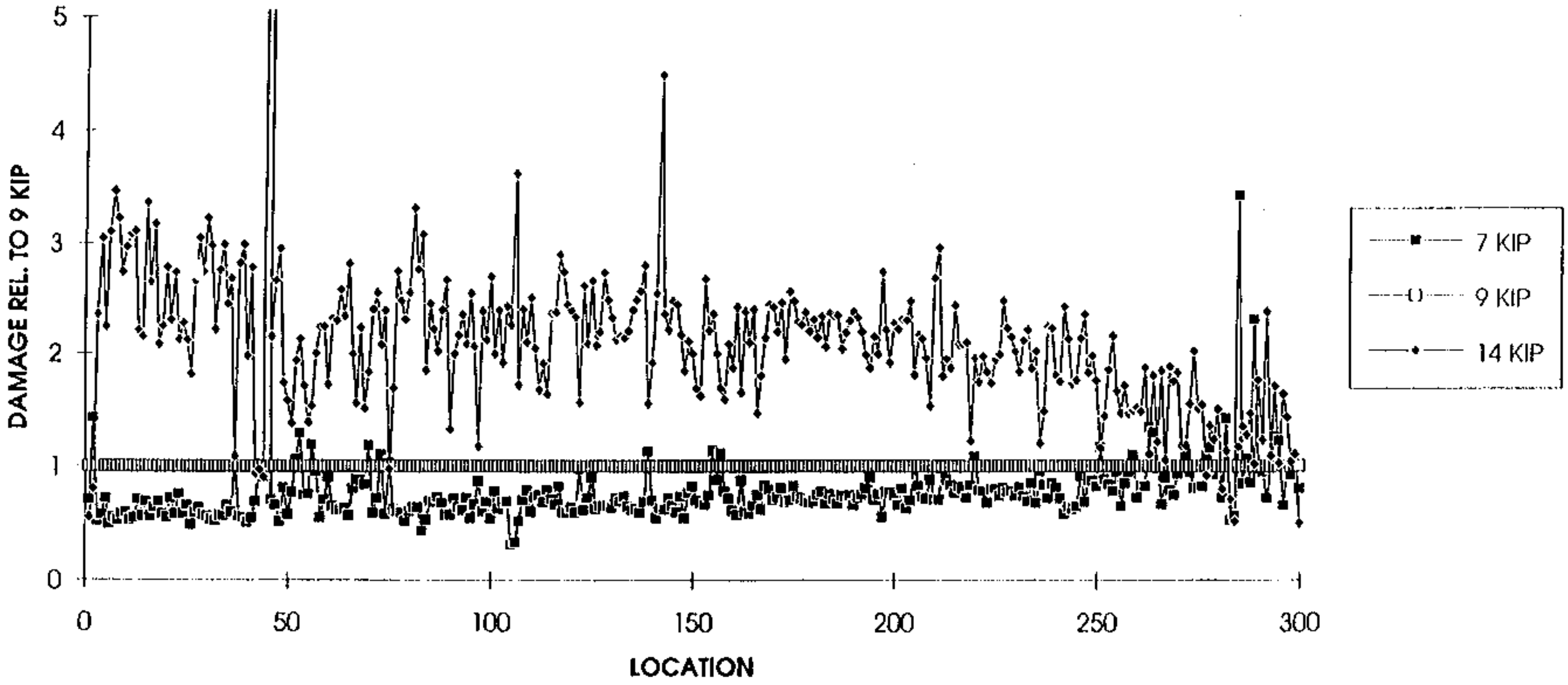


APPENDIX B-4

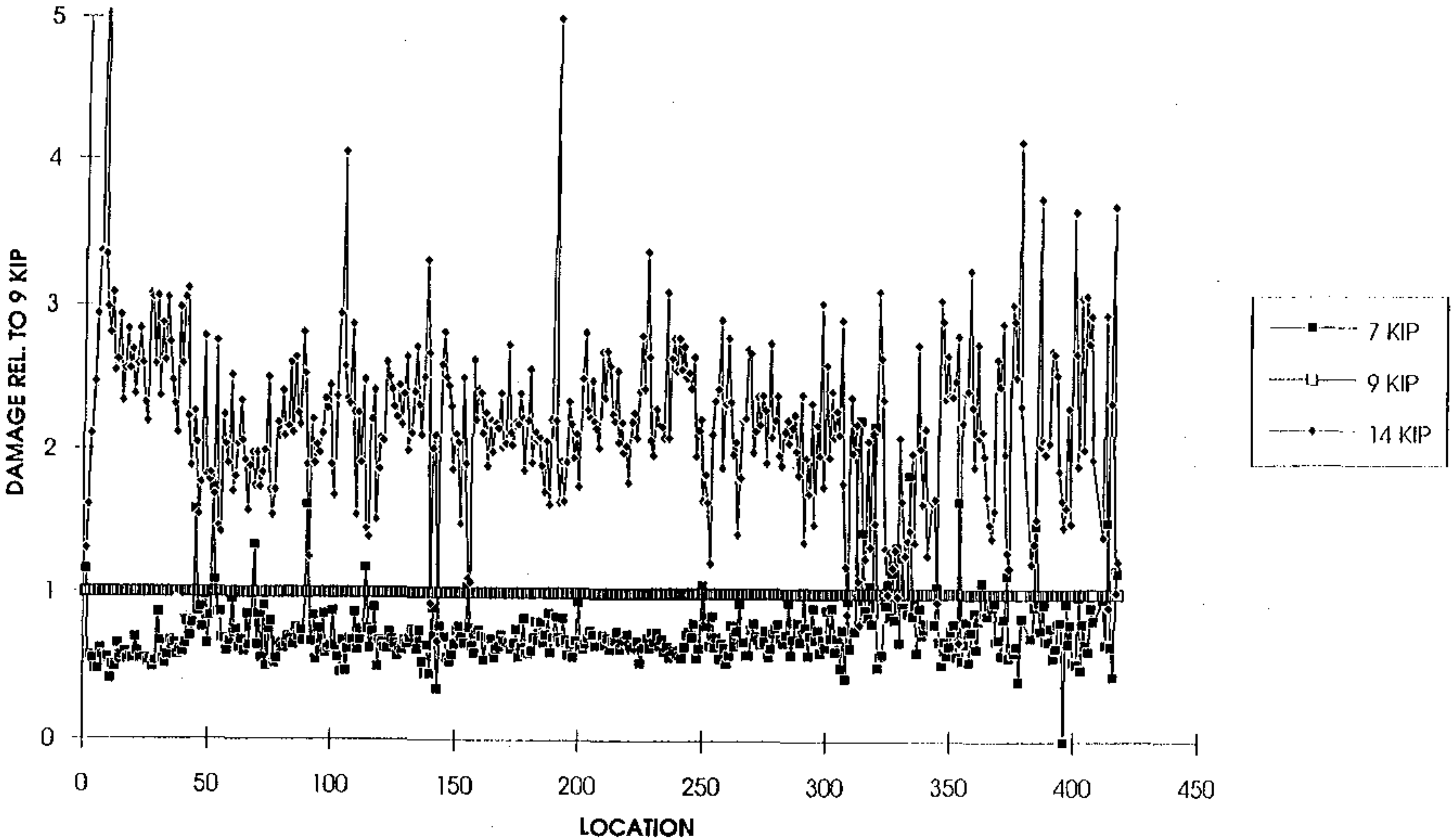
HAINES

PAVEMENT DAMAGE RELATIVE TO 9K LOAD FOR A GIVEN TESTING PERIOD

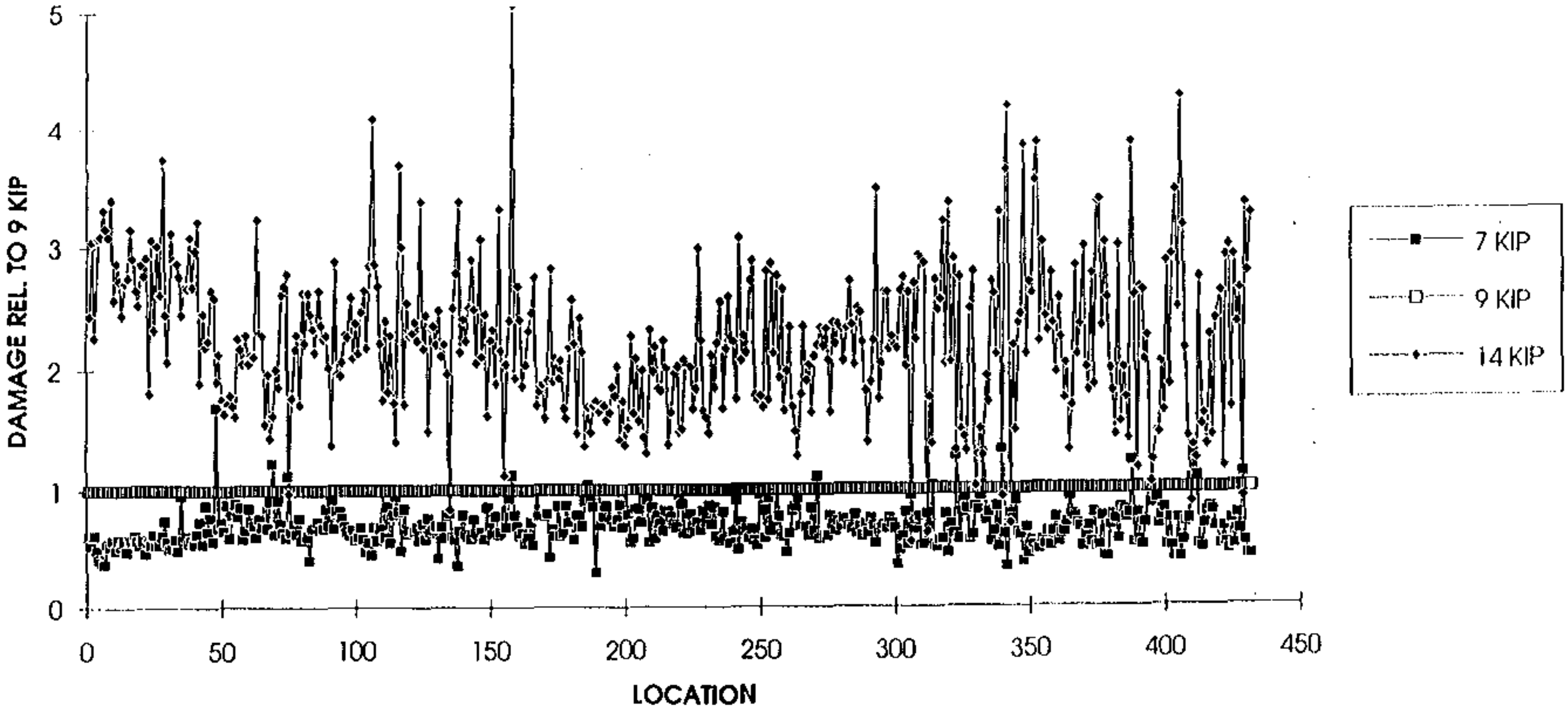
HAINES HWY. RELATIVE BASE DAMAGE 93/03/21



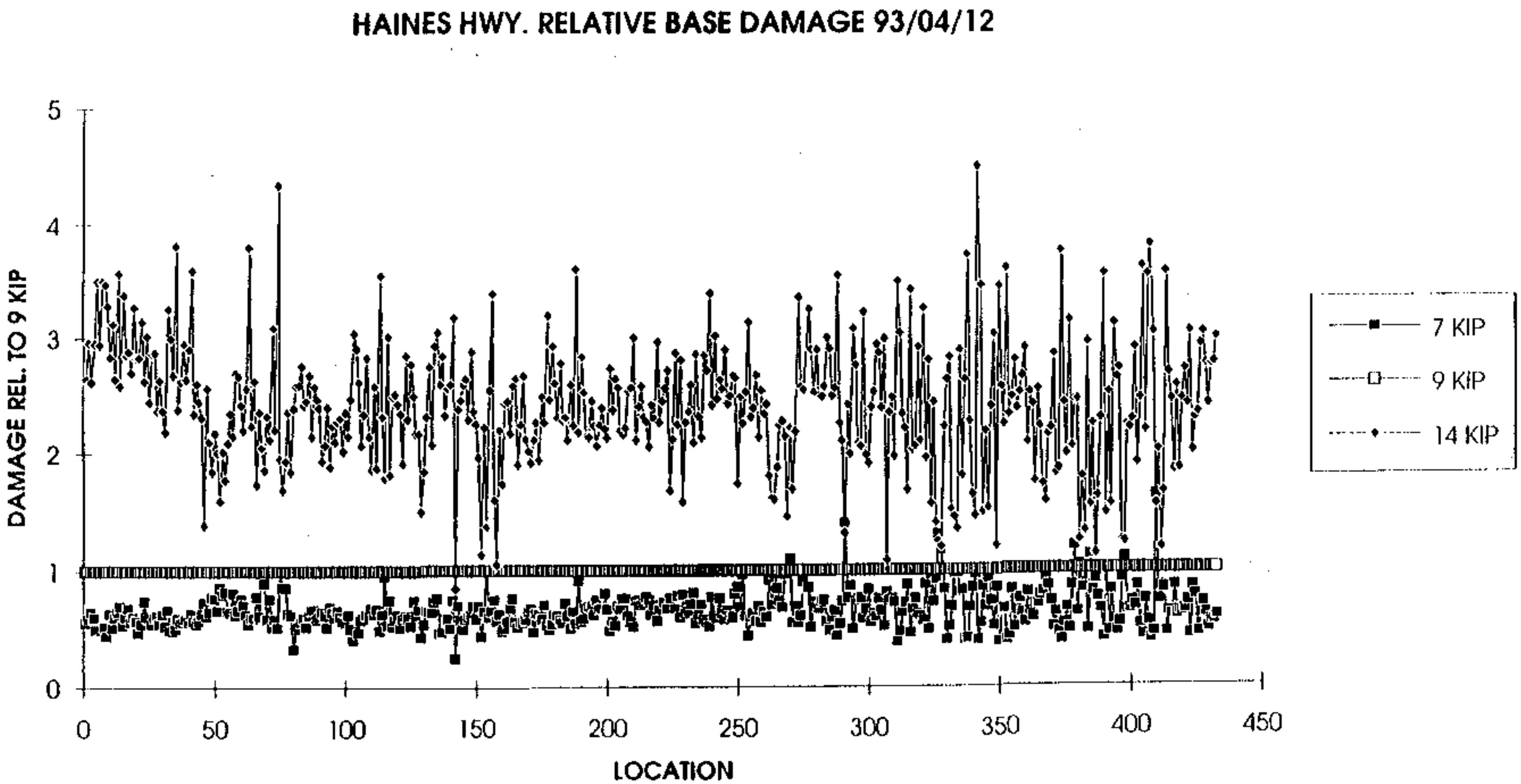
HAINES HWY. RELATIVE BASE DAMAGE 93/03/30



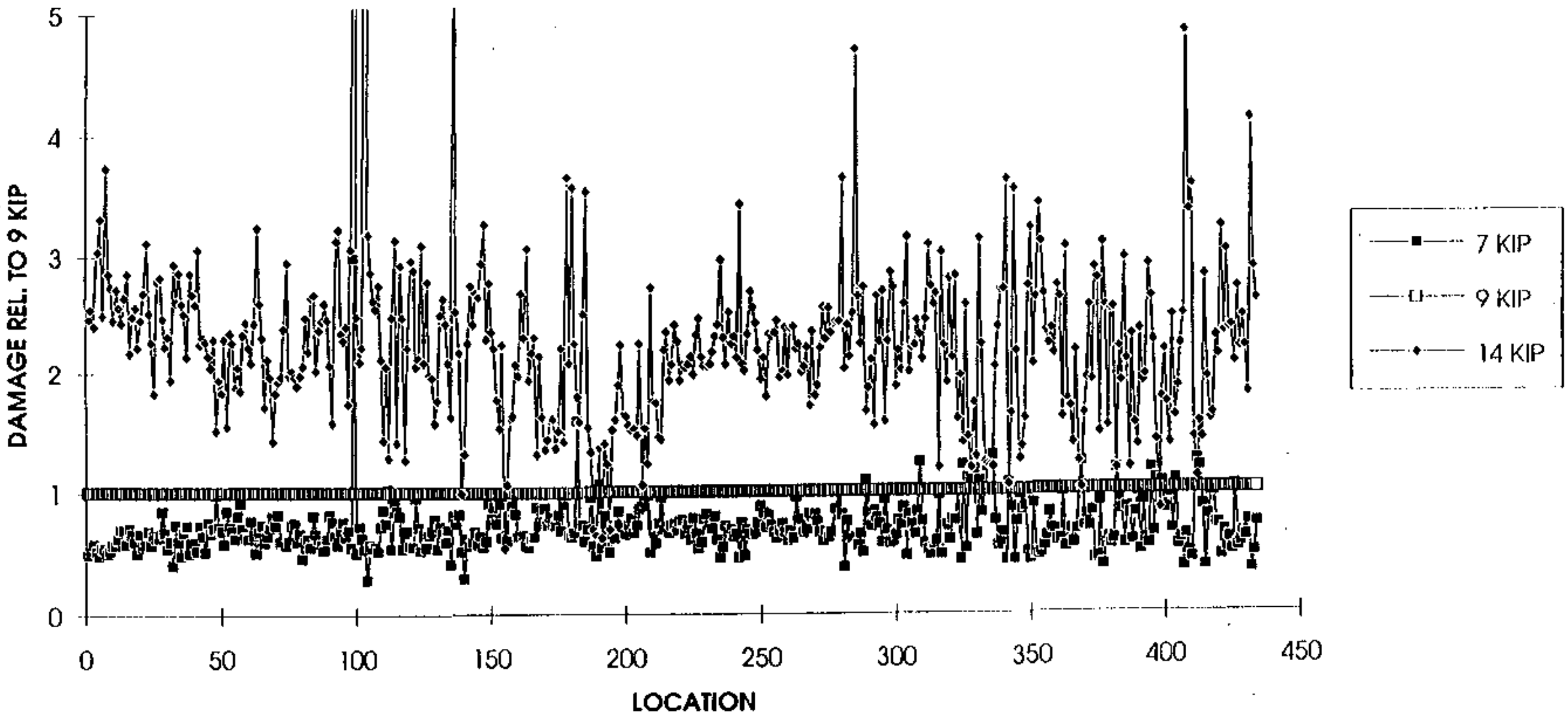
HAINES HWY. RELATIVE BASE DAMAGE 93/04/06



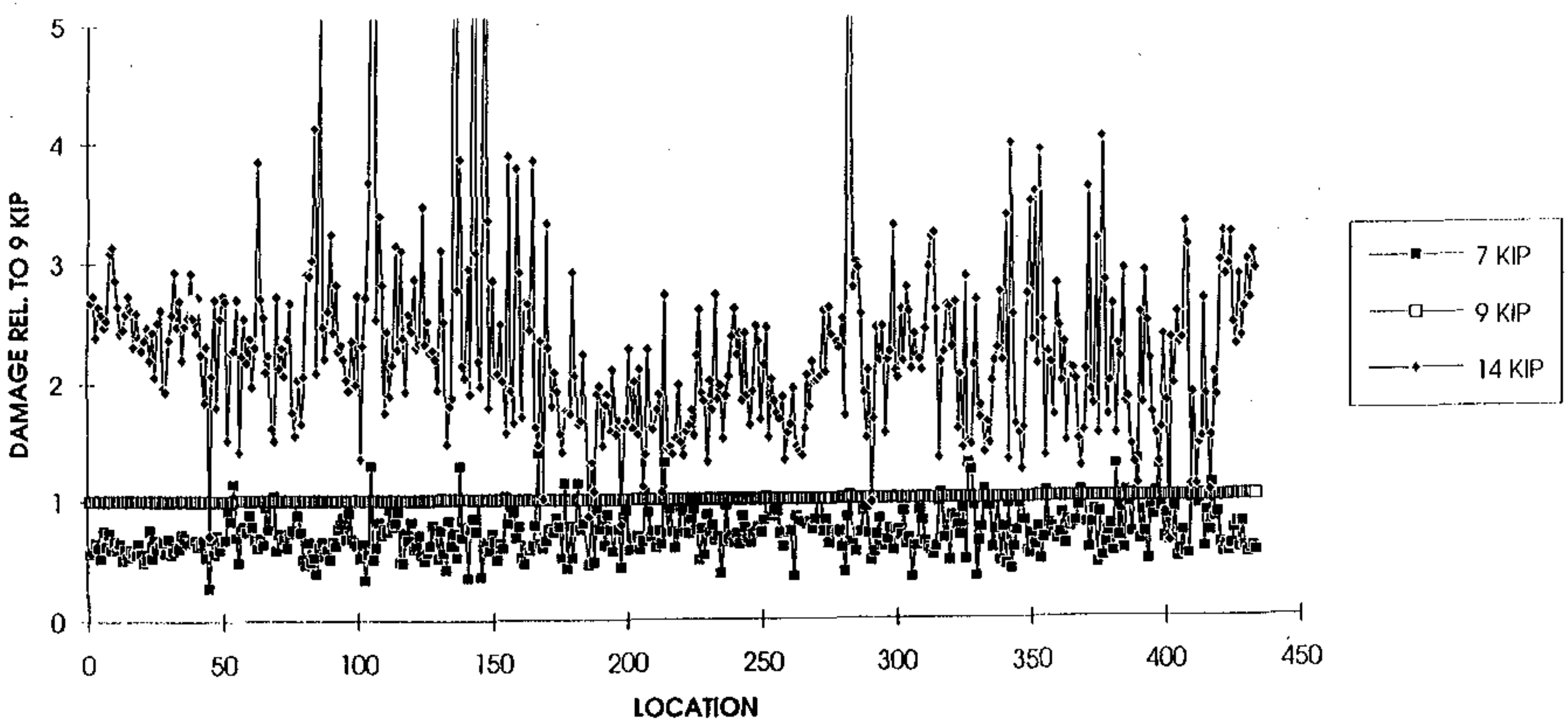
B-4-3

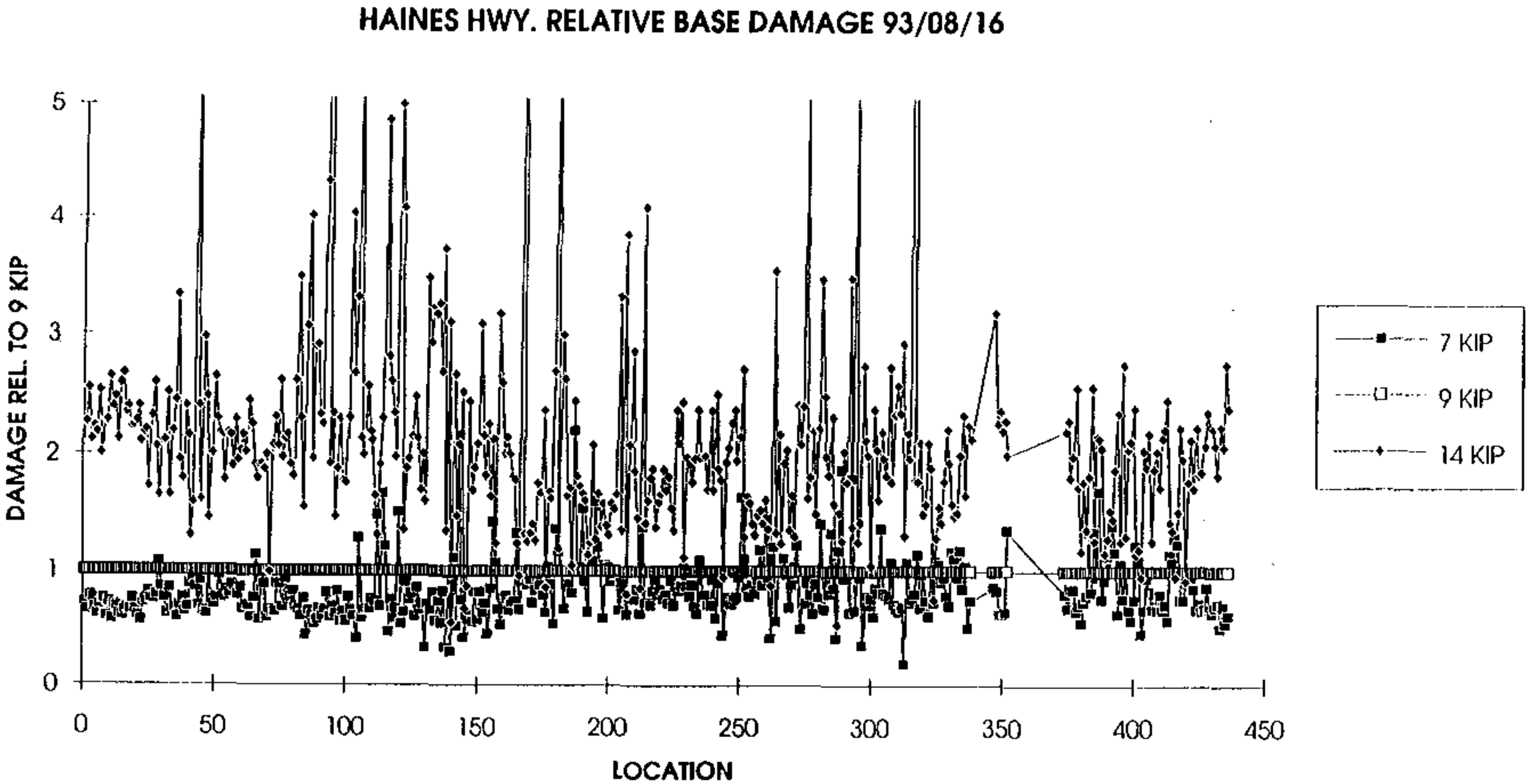

HAINES HWY. RELATIVE BASE DAMAGE 93/04/12
DAMAGE REL. TO 9 KIP
LOCATION
0
1
2
3
4
5
0
50
100
150
200
250
300
350
400
450
7 KIP
9 KIP
14 KIP

HAINES HWY. RELATIVE BASE DAMAGE 93/04/20



HAINES HWY. RELATIVE BASE DAMAGE 93/04/27

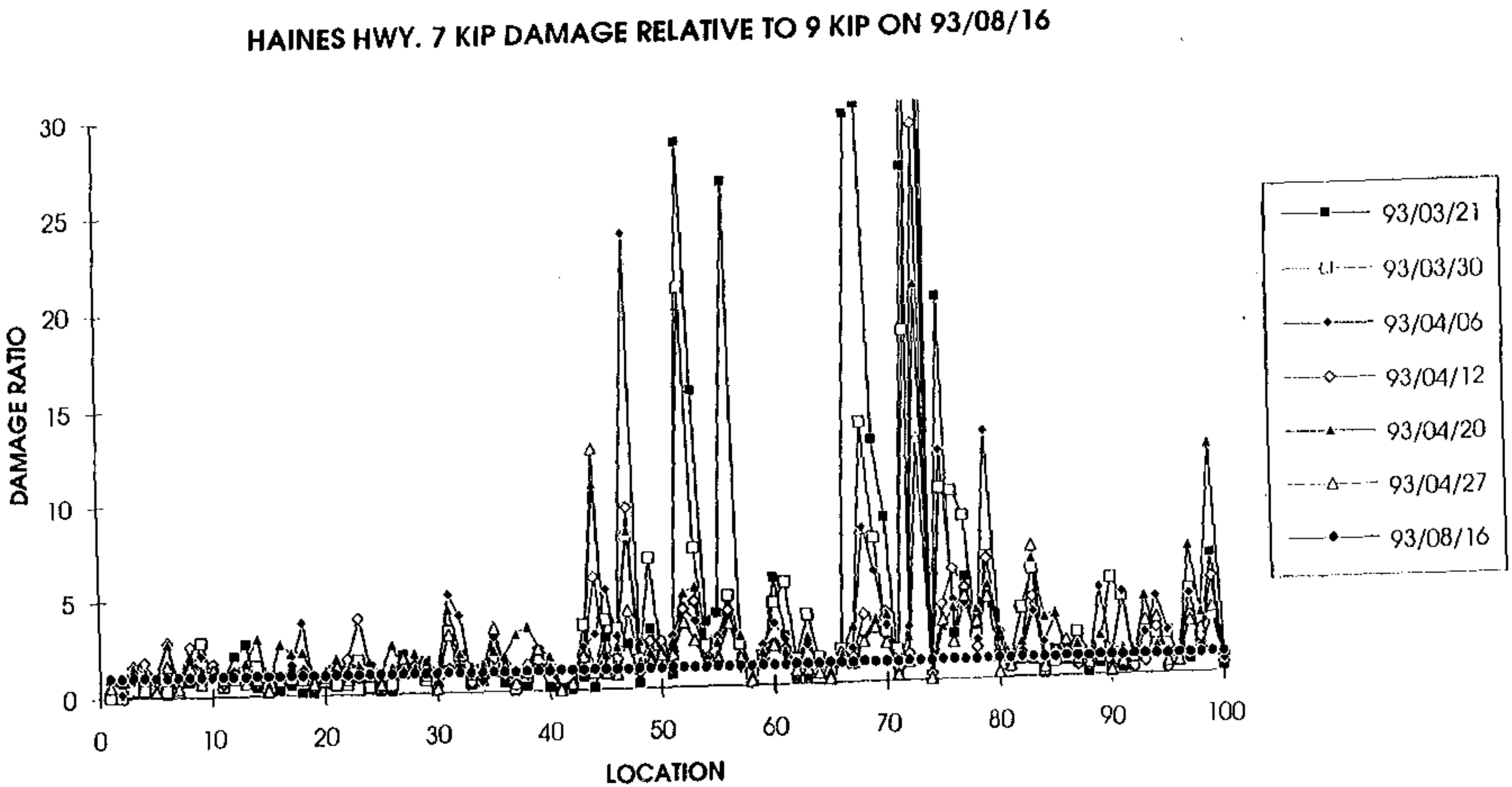



HAINES HWY. RELATIVE BASE DAMAGE 93/08/16
DAMAGE REL. TO 9 KIP
LOCATION
0
1
2
3
4
5
0
50
100
150
200
250
300
350
400
450
7 KIP
9 KIP
14 KIP

APPENDIX B-5

HAINES

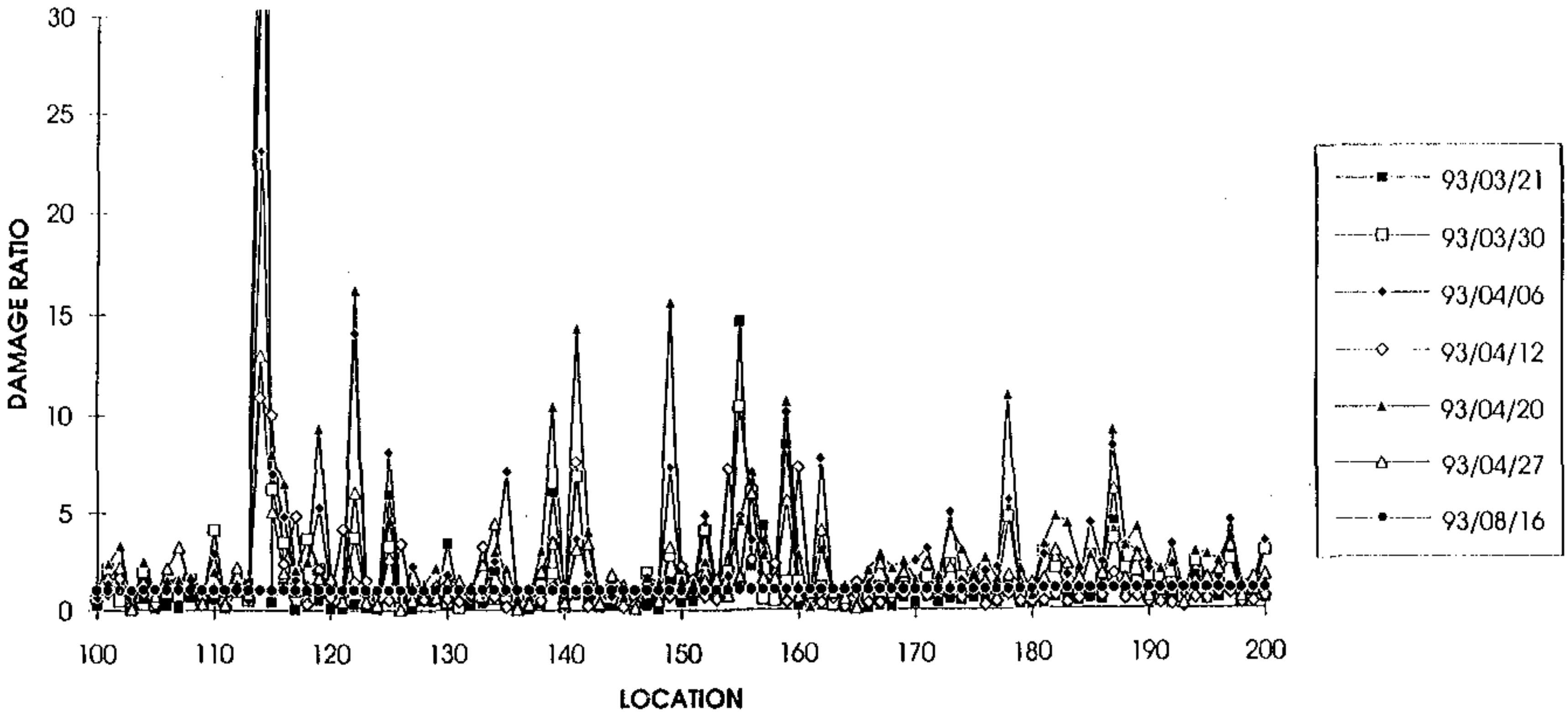
PAVEMENT DAMAGE RELATIVE TO 9K LOADING DURING 08/16/93



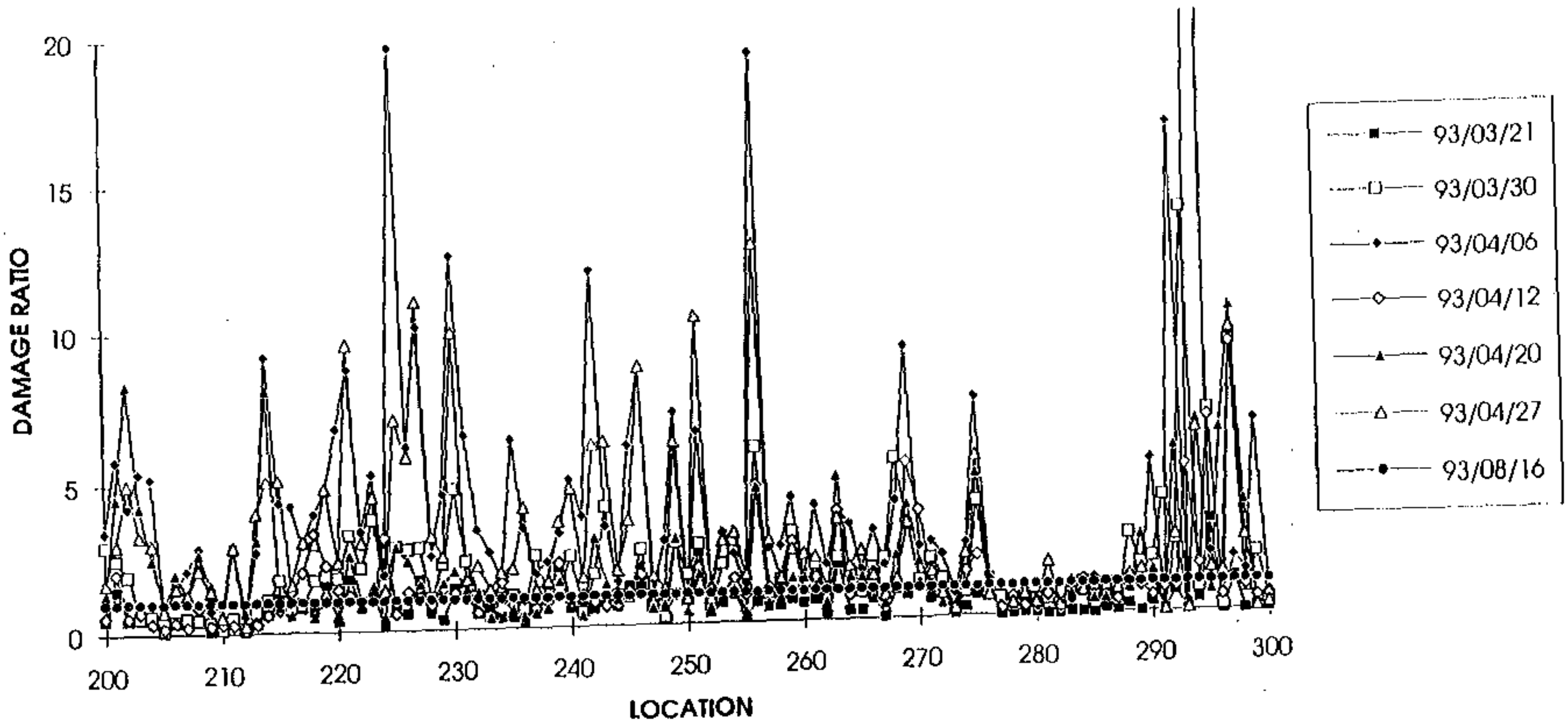
HAINES HWY. 7 KIP DAMAGE RELATIVE TO 9 KIP ON 93/08/16
DAMAGE RATIO
0
5
10
15
20
25
30
LOCATION
0
10
20
30
40
50
60
70
80
90
100
93/03/21
93/03/30
93/04/06
93/04/12
93/04/20
93/04/27
93/08/16

B-5-1

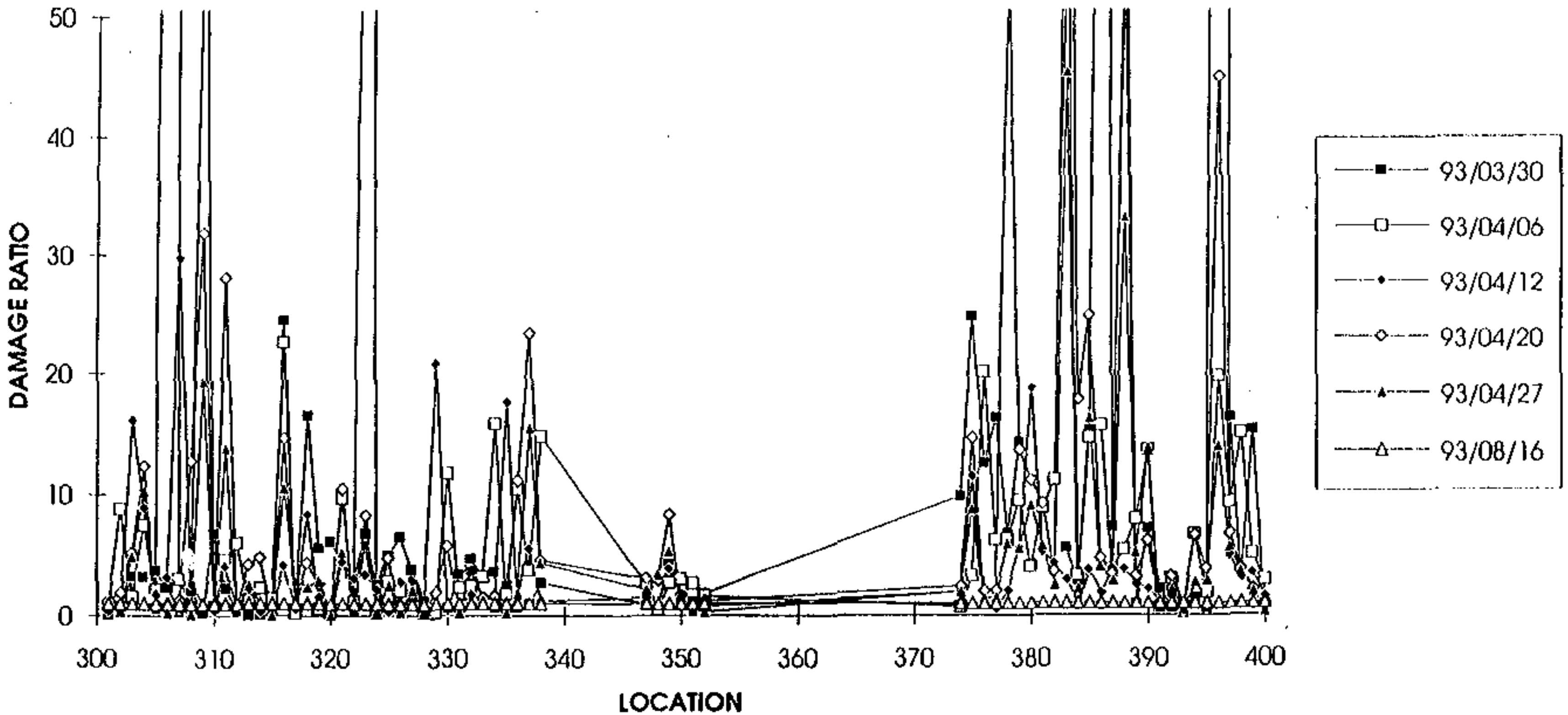
HAINES HWY. 7 KIP DAMAGE RELATIVE TO 9 KIP ON 93/08/16

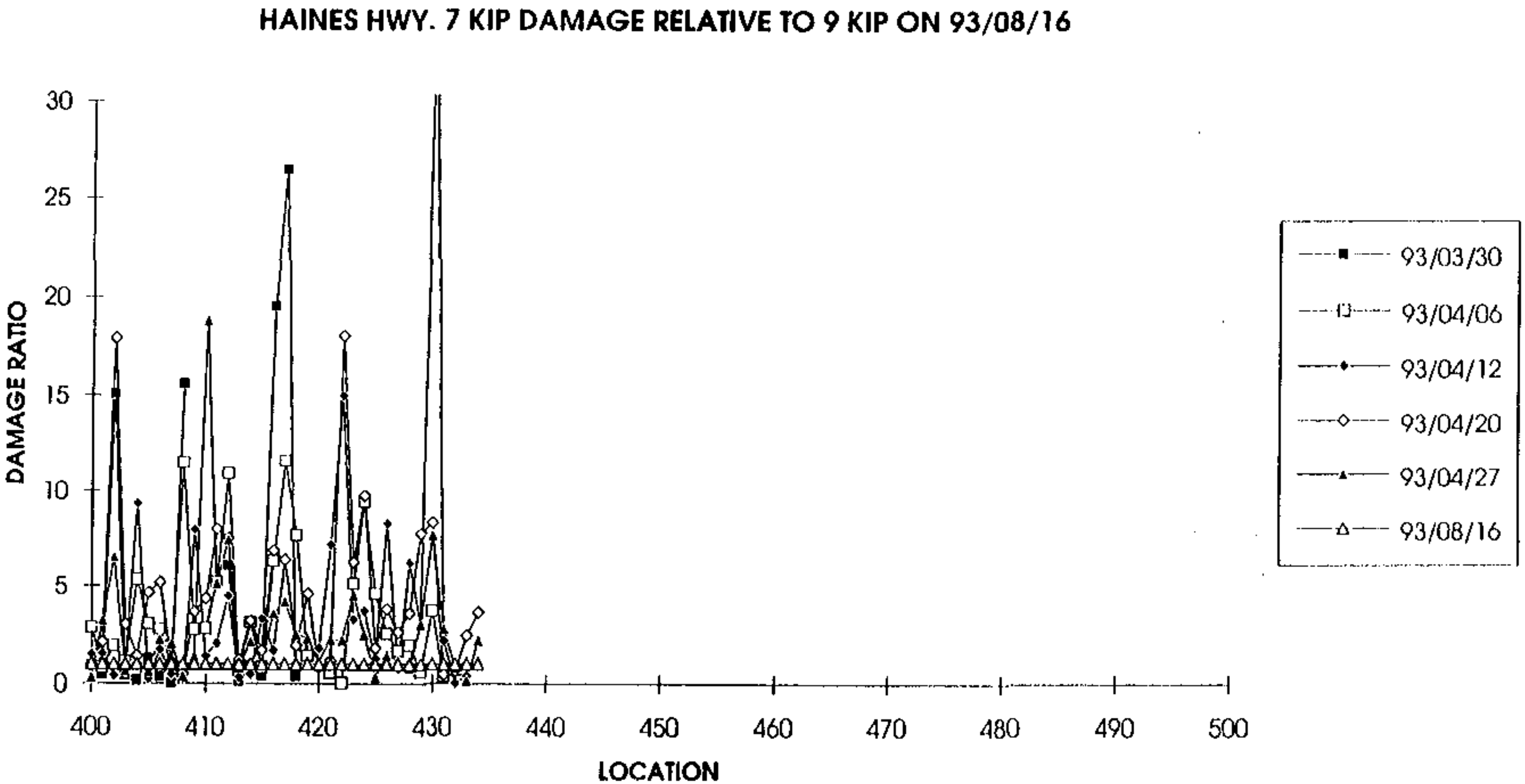


HAINES HWY. 7 KIP DAMAGE RELATIVE TO 9 KIP ON 93/08/16



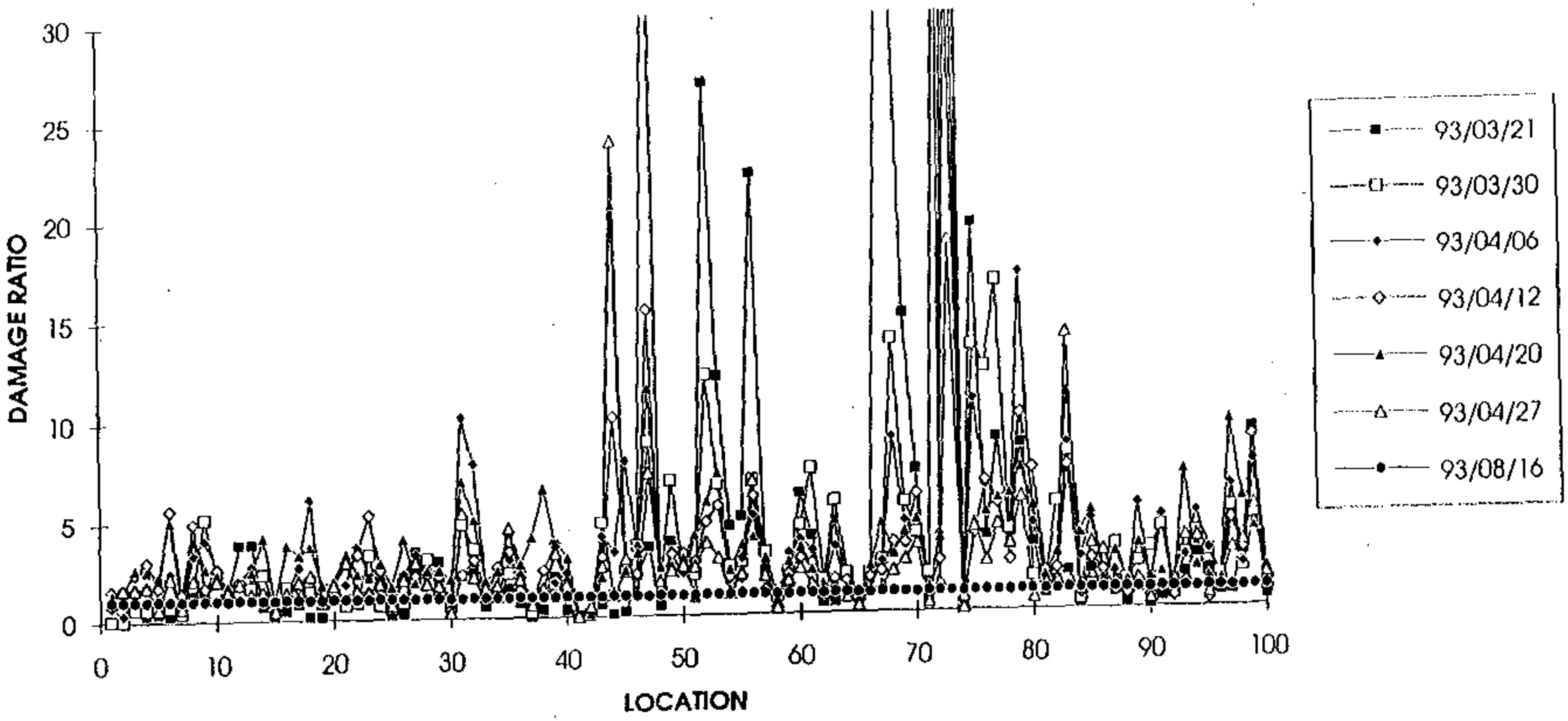
HAINES HWY. 7 KIP DAMAGE RELATIVE TO 9 KIP ON 93/08/16



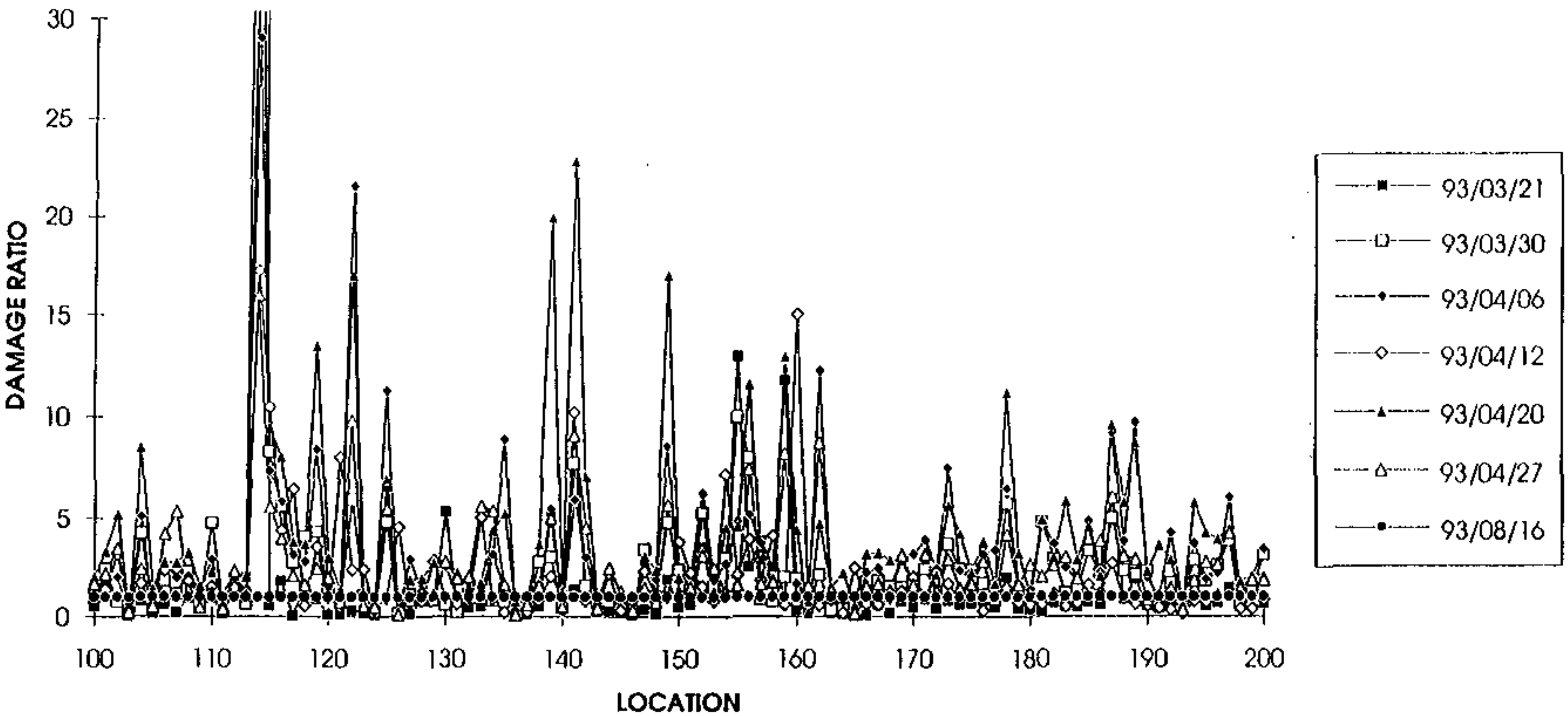


HAINES HWY. 7 KIP DAMAGE RELATIVE TO 9 KIP ON 93/08/16
DAMAGE RATIO
0
5
10
15
20
25
30
400
410
420
430
440
450
460
470
480
490
500
LOCATION
93/03/30
93/04/06
93/04/12
93/04/20
93/04/27
93/08/16

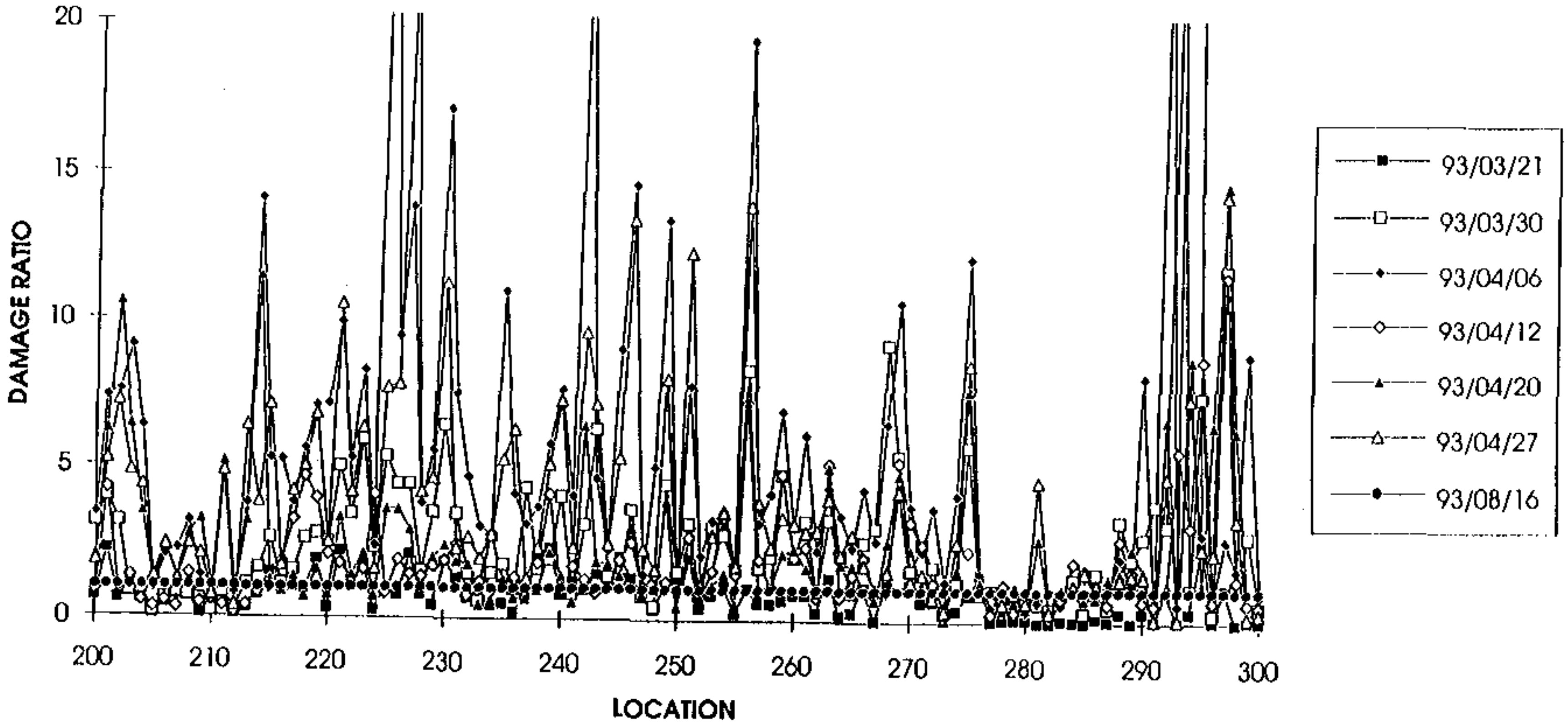
HAINES HWY. 9 KIP DAMAGE RELATIVE TO 9 KIP ON 93/08/16

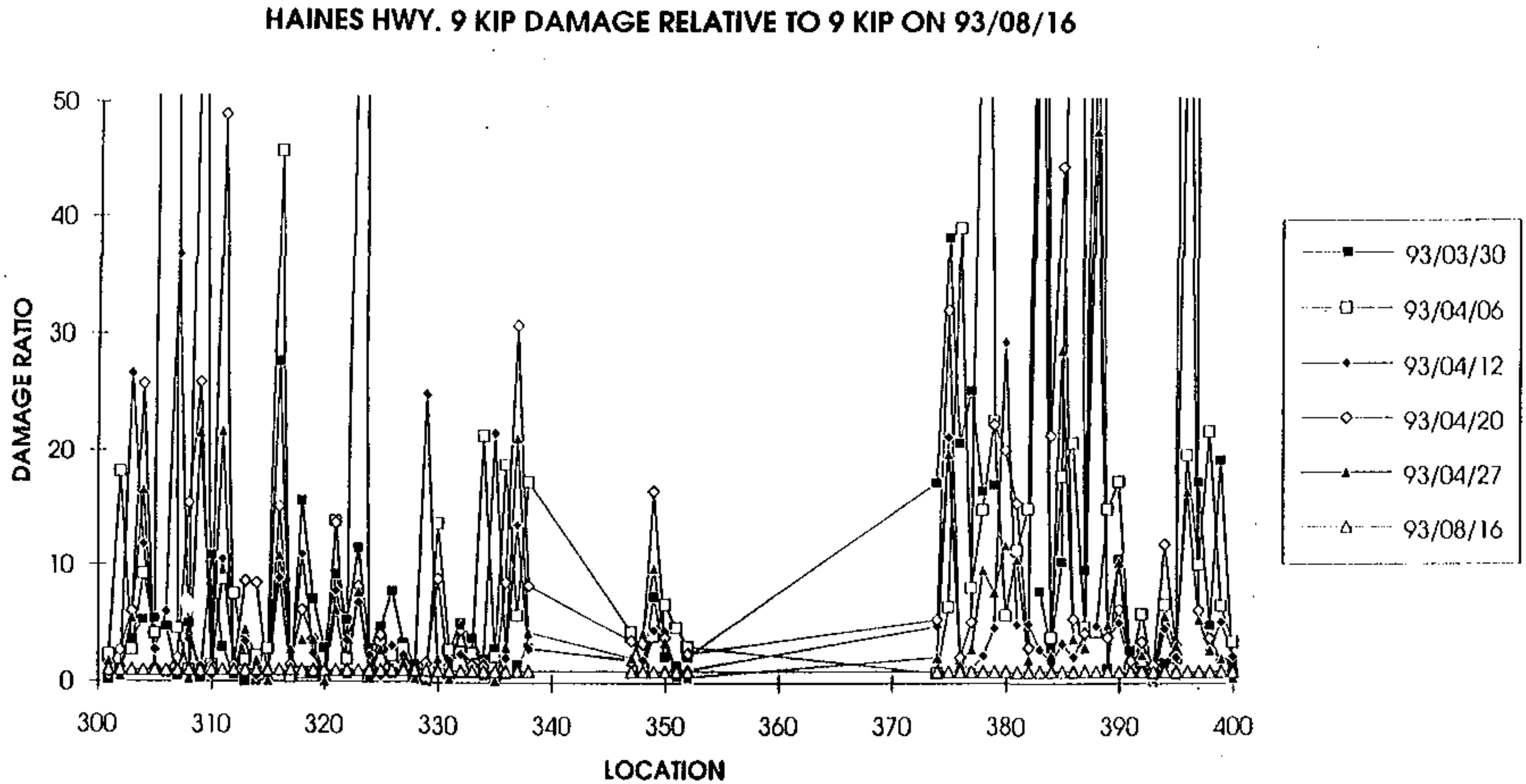


HAINES HWY. 9 KIP DAMAGE RELATIVE TO 9 KIP ON 93/08/16

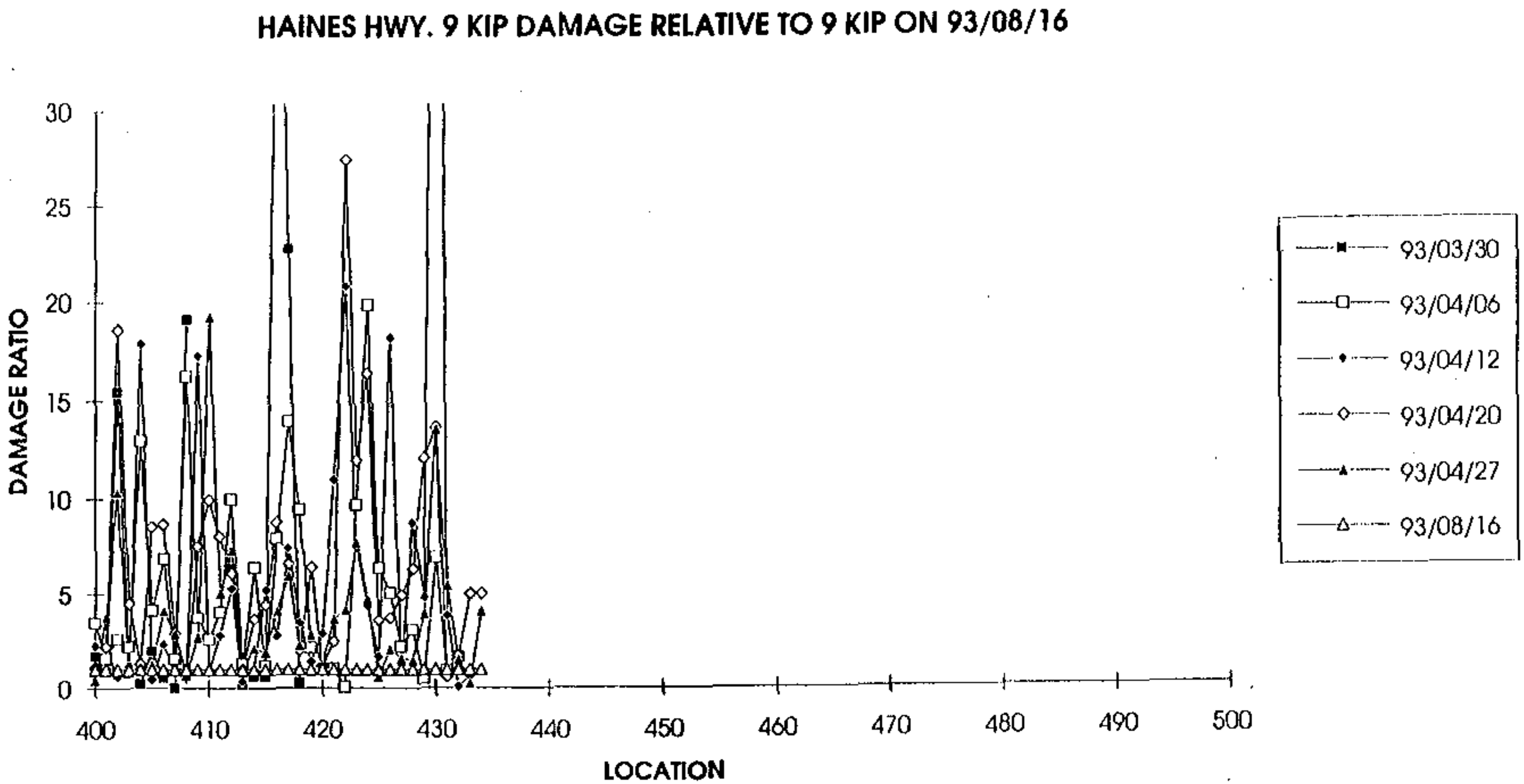


HAINES HWY. 9 KIP DAMAGE RELATIVE TO 9 KIP ON 93/08/16

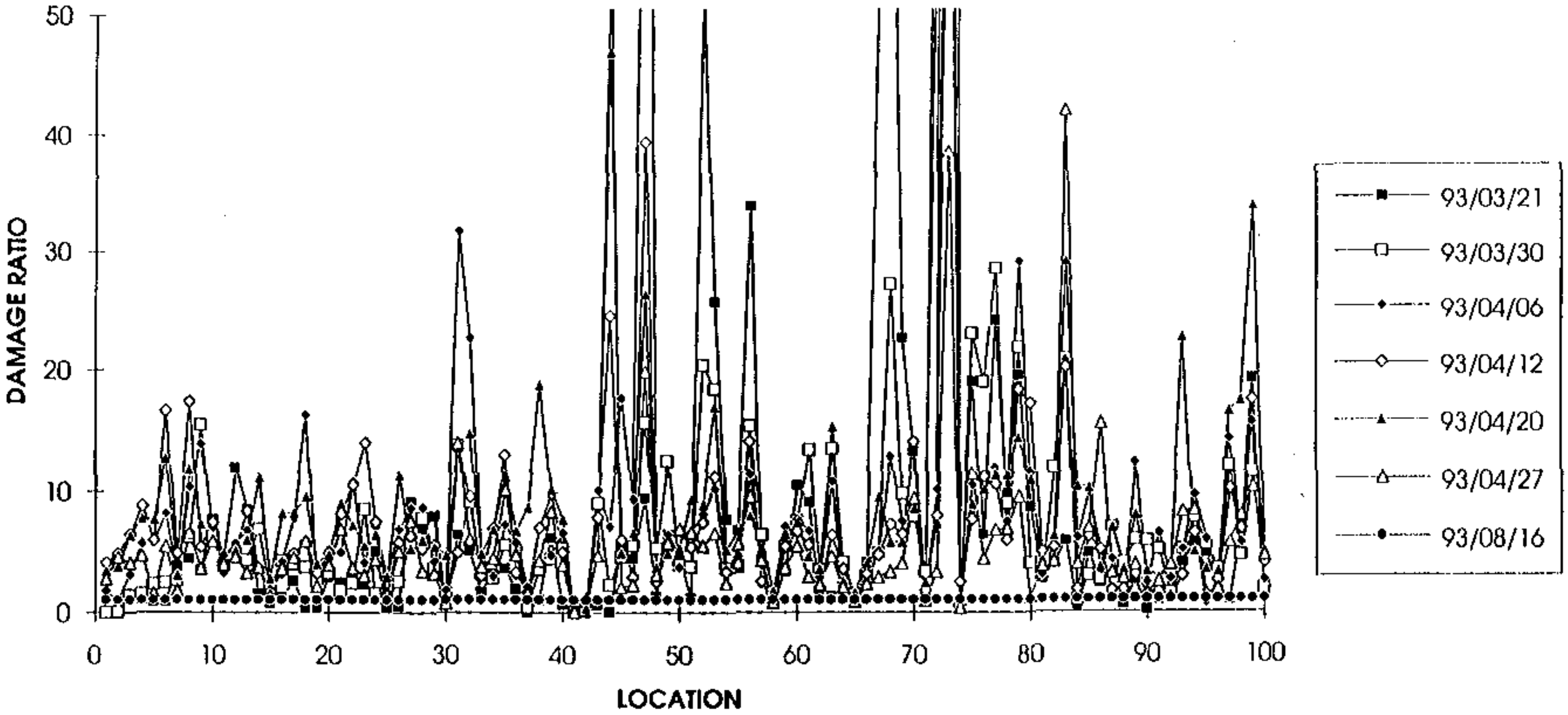




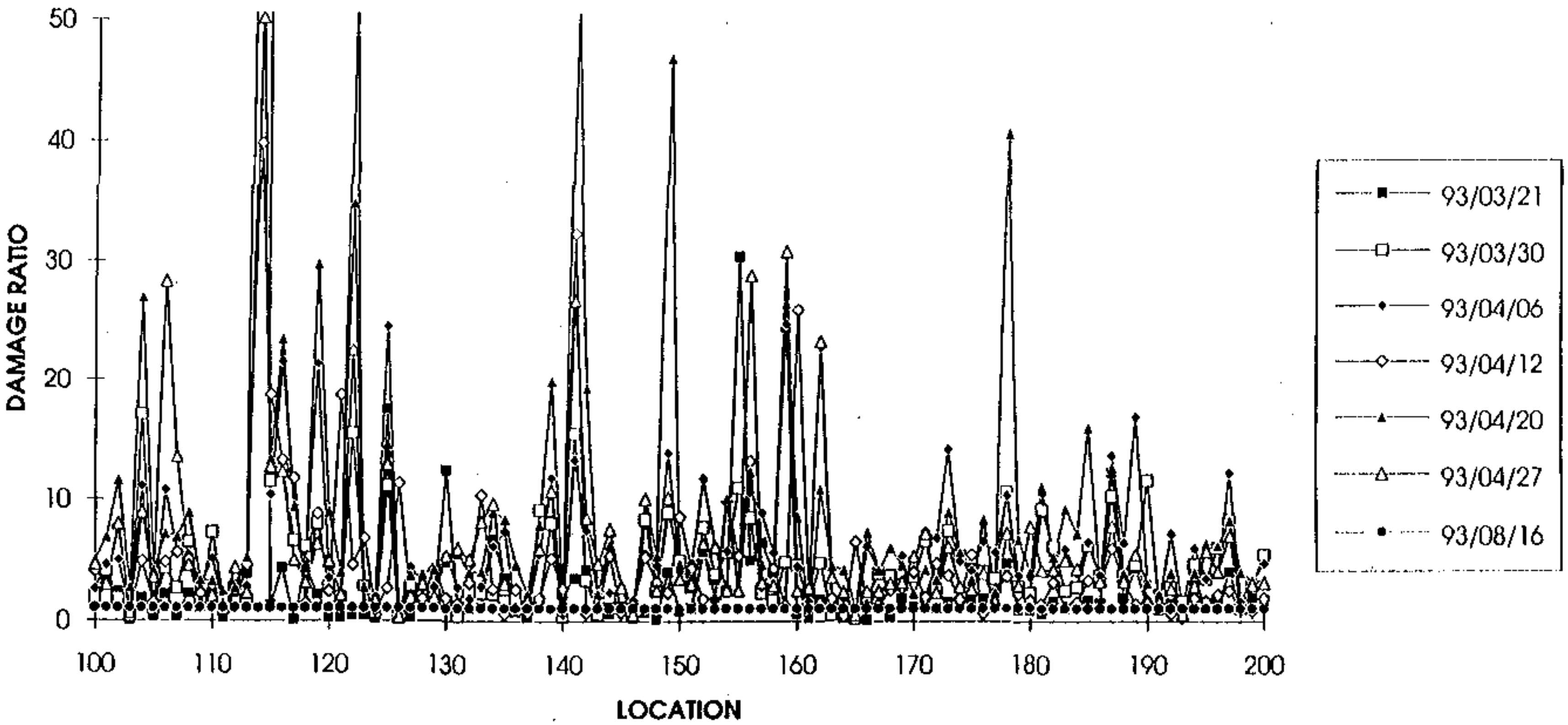
HAINES HWY. 9 KIP DAMAGE RELATIVE TO 9 KIP ON 93/08/16
DAMAGE RATIO
50
40
30
20
10
0
300
310
320
330
340
350
360
370
380
390
400
LOCATION
93/03/30
93/04/06
93/04/12
93/04/20
93/04/27
93/08/16


HAINES HWY. 9 KIP DAMAGE RELATIVE TO 9 KIP ON 93/08/16
DAMAGE RATIO
0
5
10
15
20
25
30
400
410
420
430
440
450
460
470
480
490
500
LOCATION
93/03/30
93/04/06
93/04/12
93/04/20
93/04/27
93/08/16

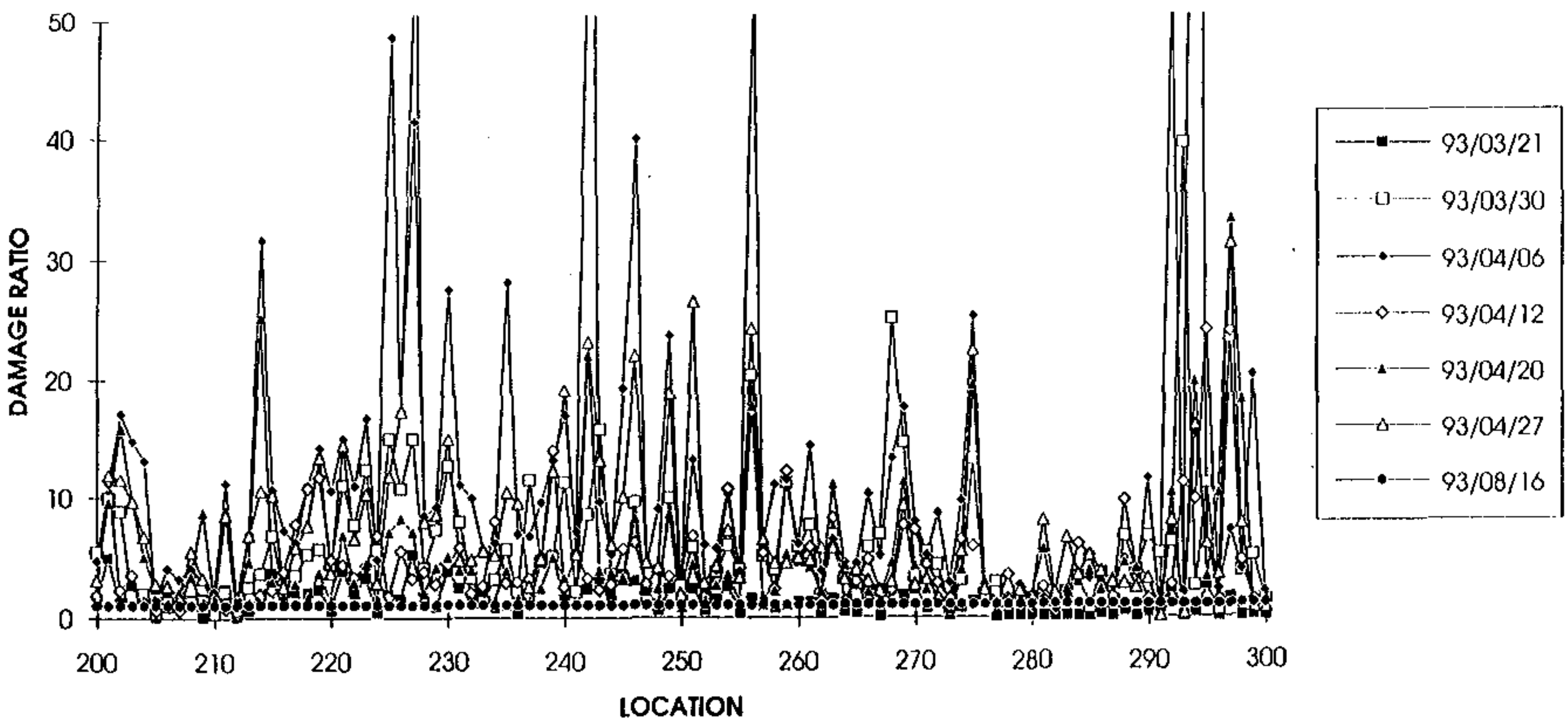
HAINES HWY. 14 KIP DAMAGE RELATIVE TO 9 KIP ON 93/08/16



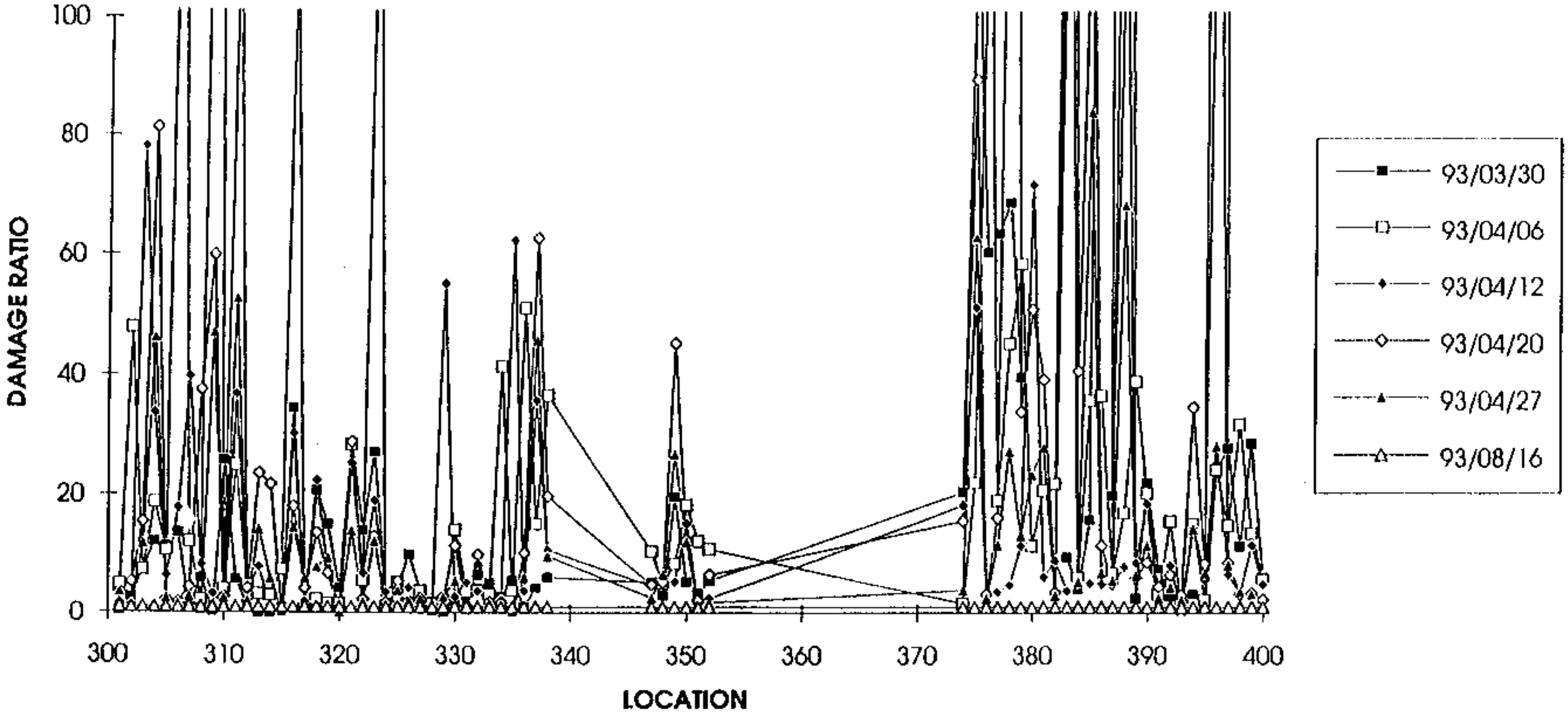
HAINES HWY. 14 KIP DAMAGE RELATIVE TO 9 KIP ON 93/08/16



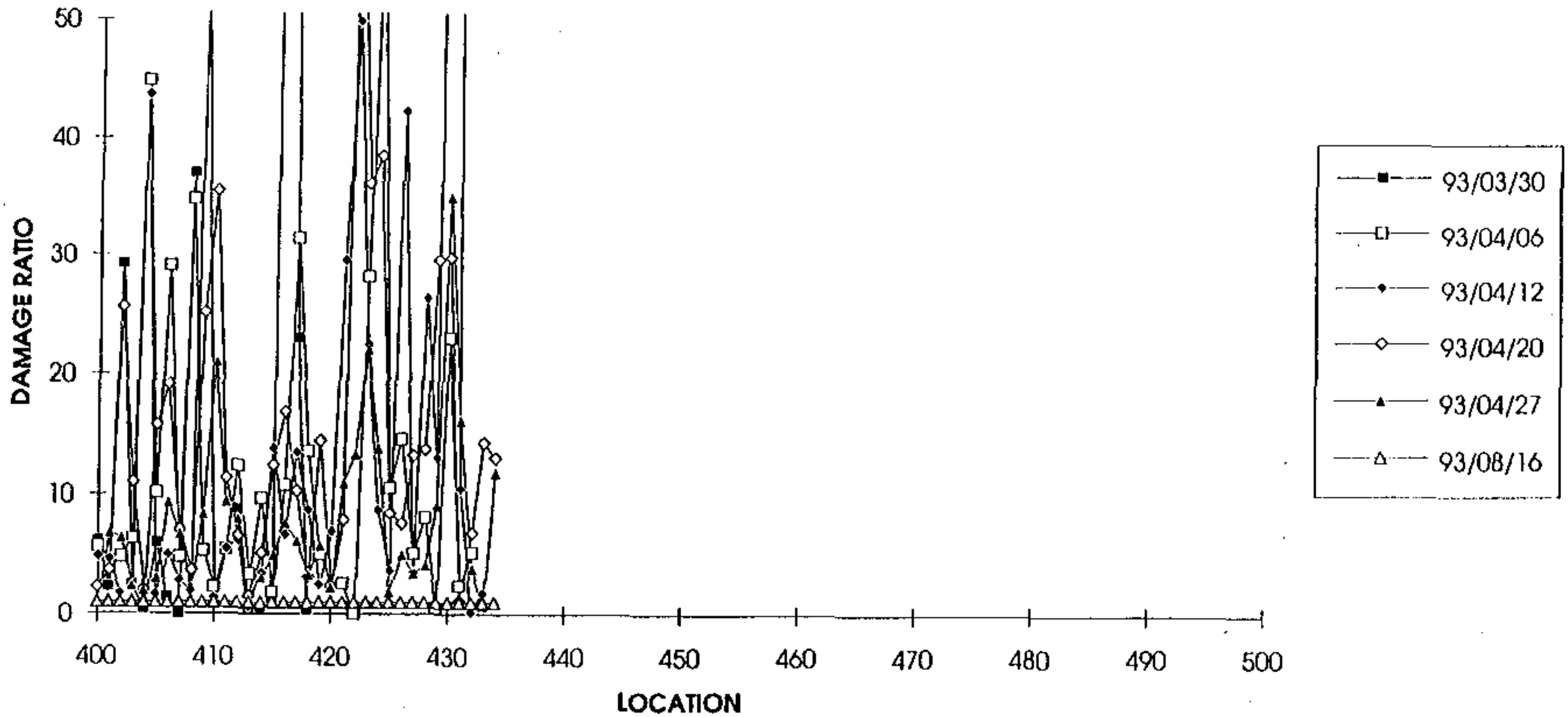
HAINES HWY. 14 KIP DAMAGE RELATIVE TO 9 KIP ON 93/08/16



HAINES HWY. 14 KIP DAMAGE RELATIVE TO 9 KIP ON 93/08/16



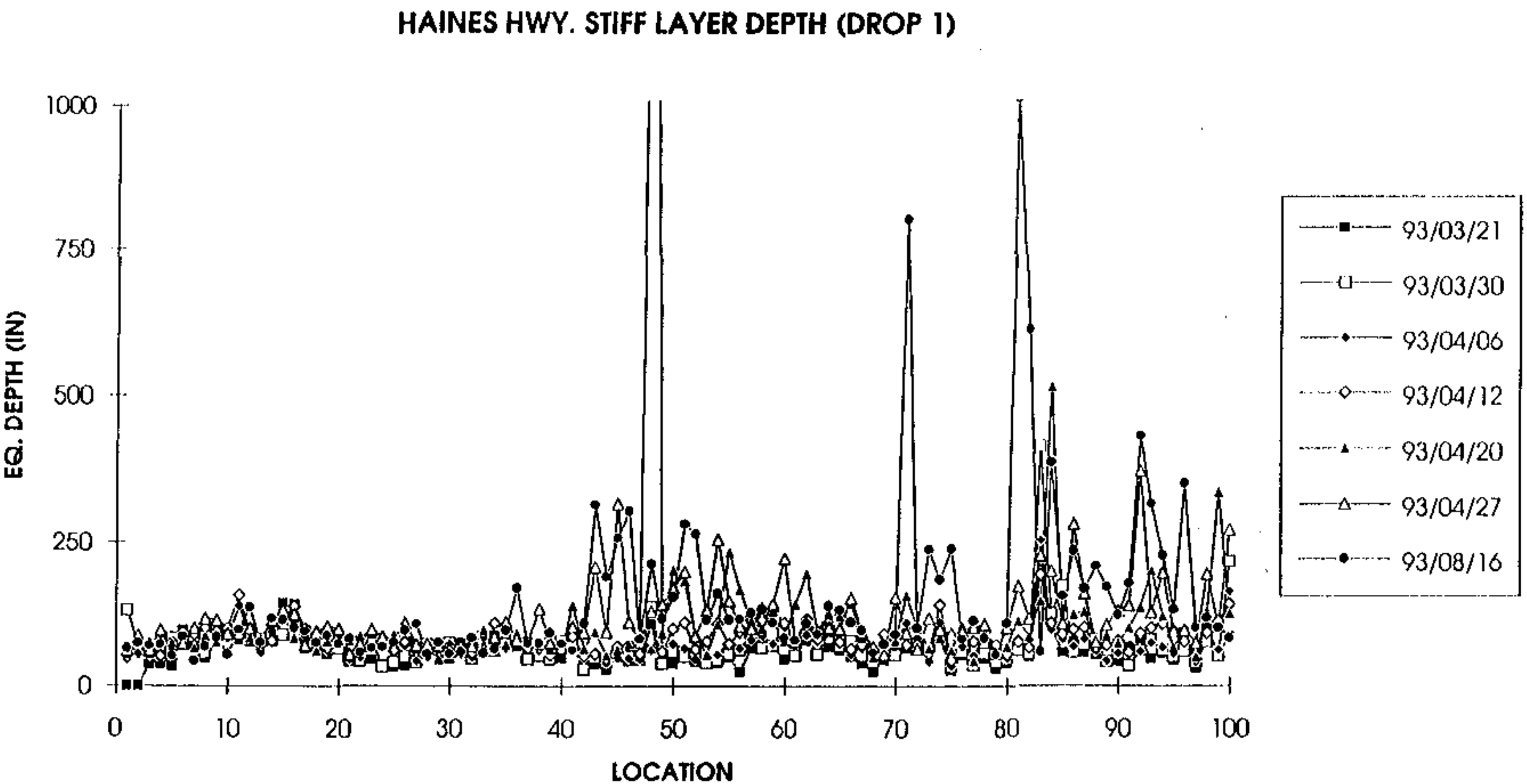
HAINES HWY. 14 KIP DAMAGE RELATIVE TO 9 KIP ON 93/08/16



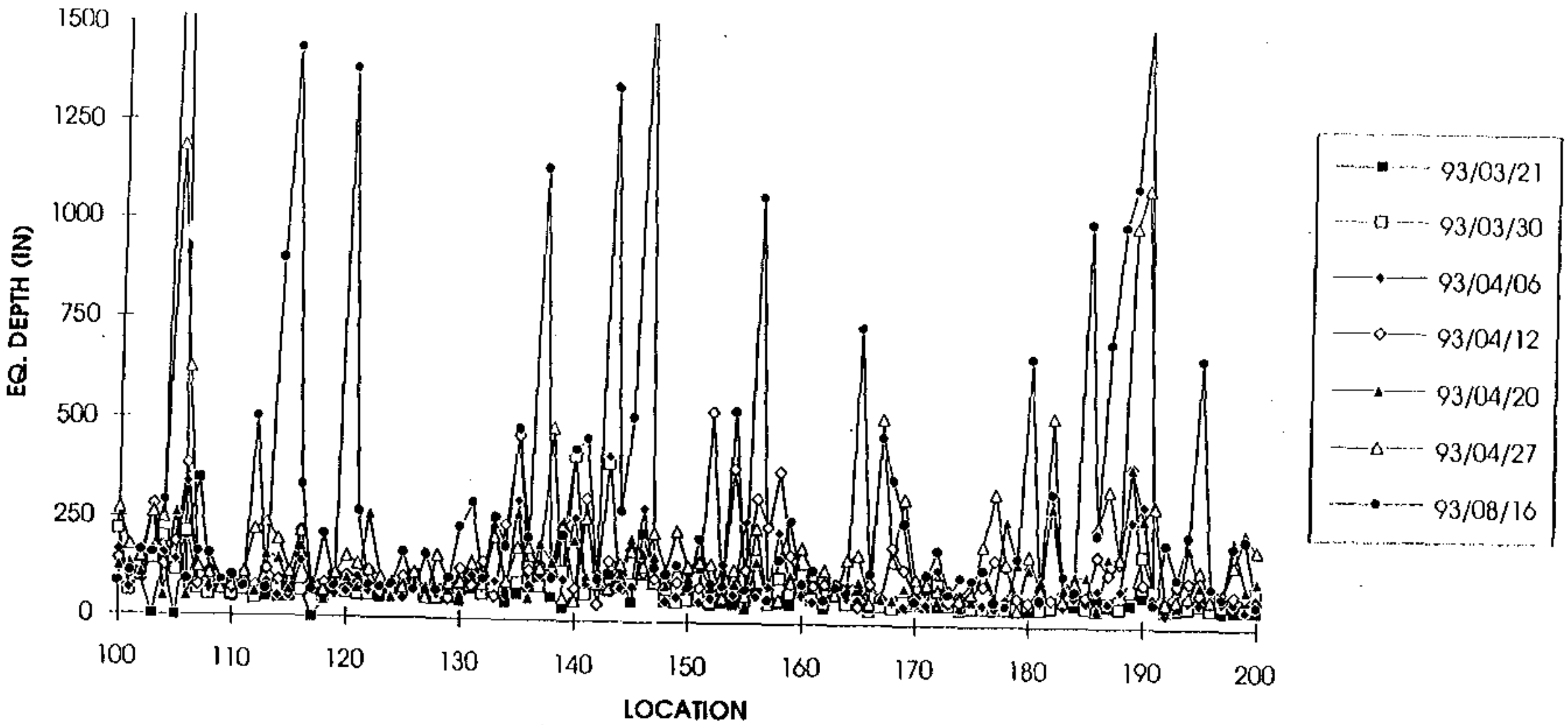
APPENDIX B 6

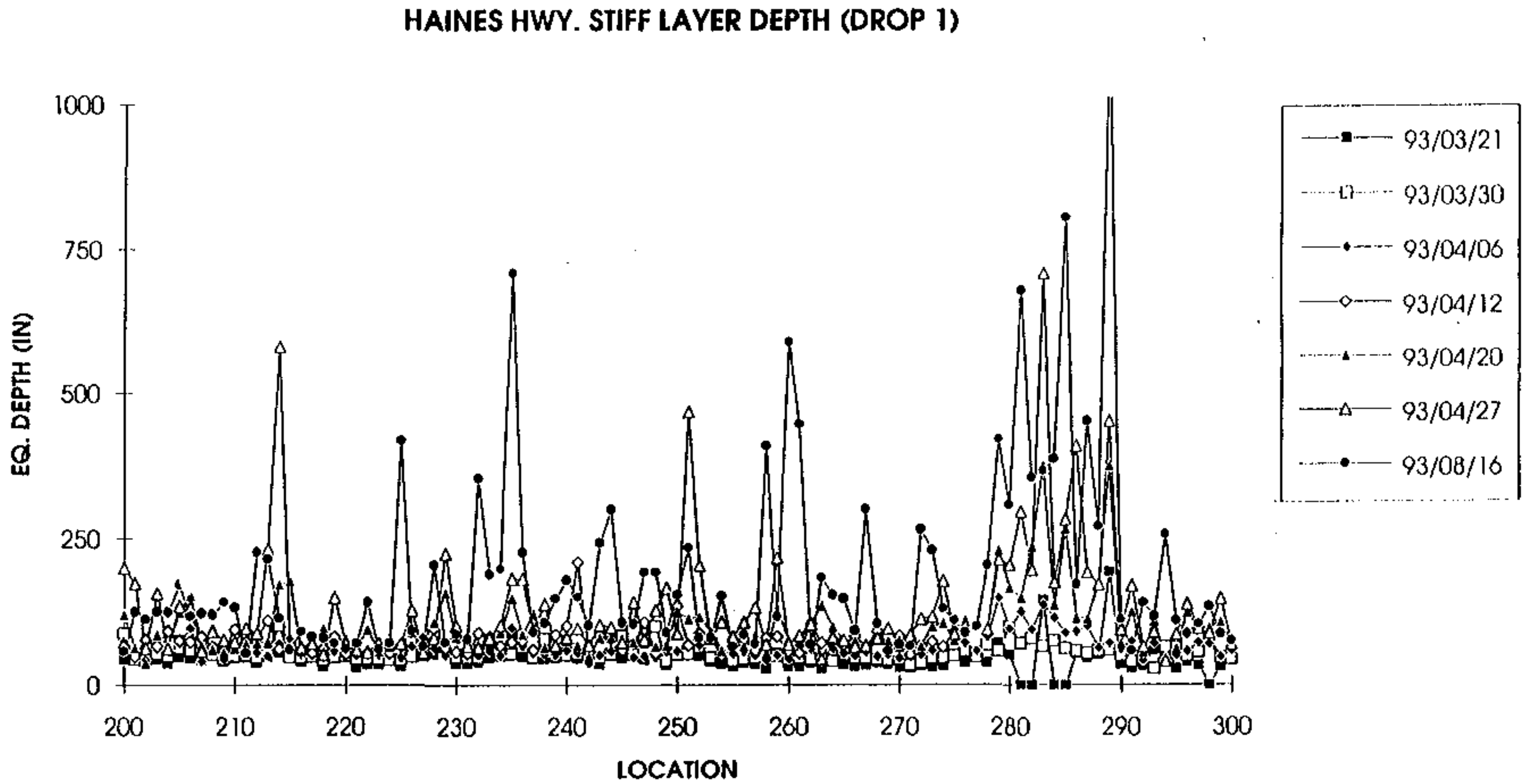
HAINES

BACKCALCULATED THAW DEPTH (DEPTH TO STIFF LAYER) USING FWD SURFACE DEFLECTIONS

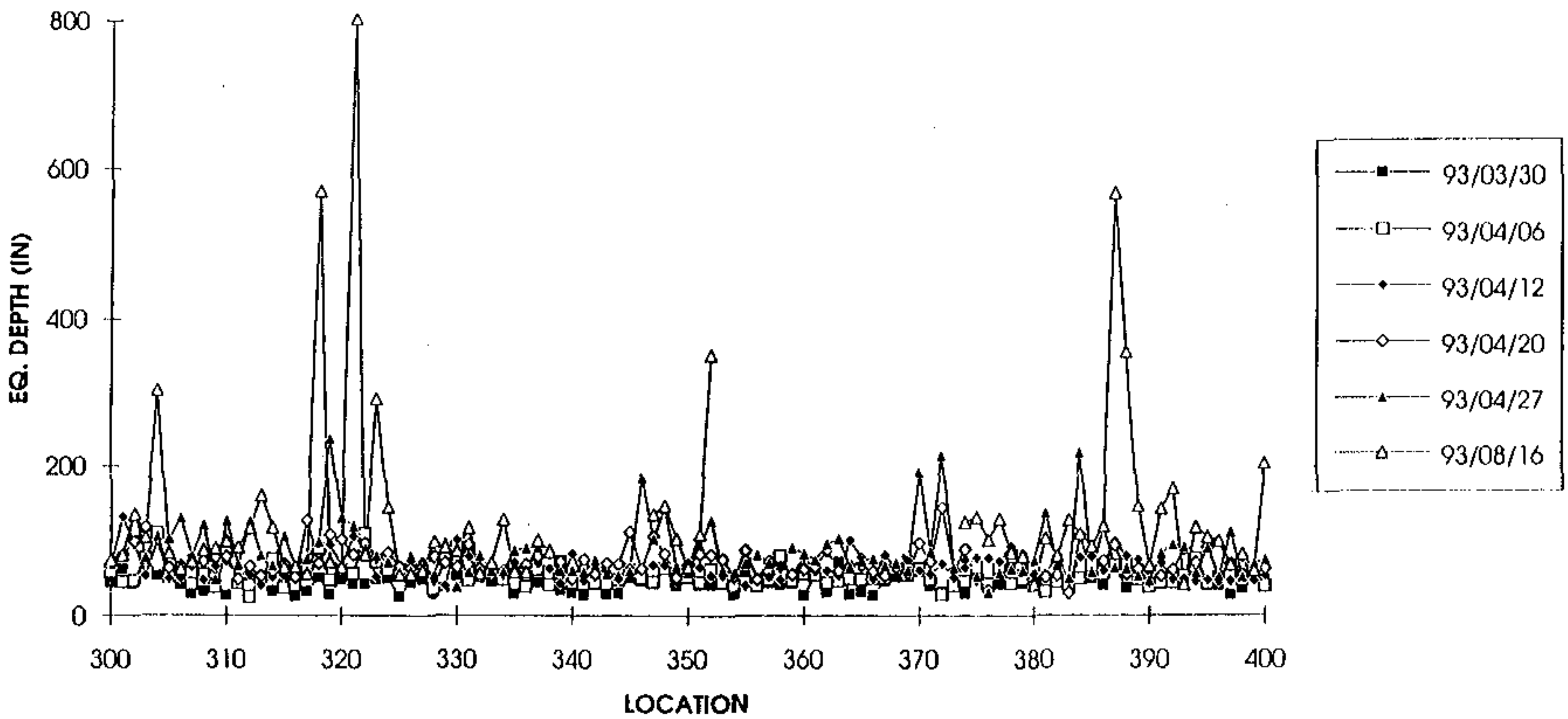

HAINES HWY. STIFF LAYER DEPTH (DROP 1)
EQ. DEPTH (IN)
1000
750
500
250
0
0
10
20
30
40
50
60
70
80
90
100
LOCATION
93/03/21
93/03/30
93/04/06
93/04/12
93/04/20
93/04/27
93/08/16

HAINES HWY. STIFF LAYER DEPTH (DROP 1)



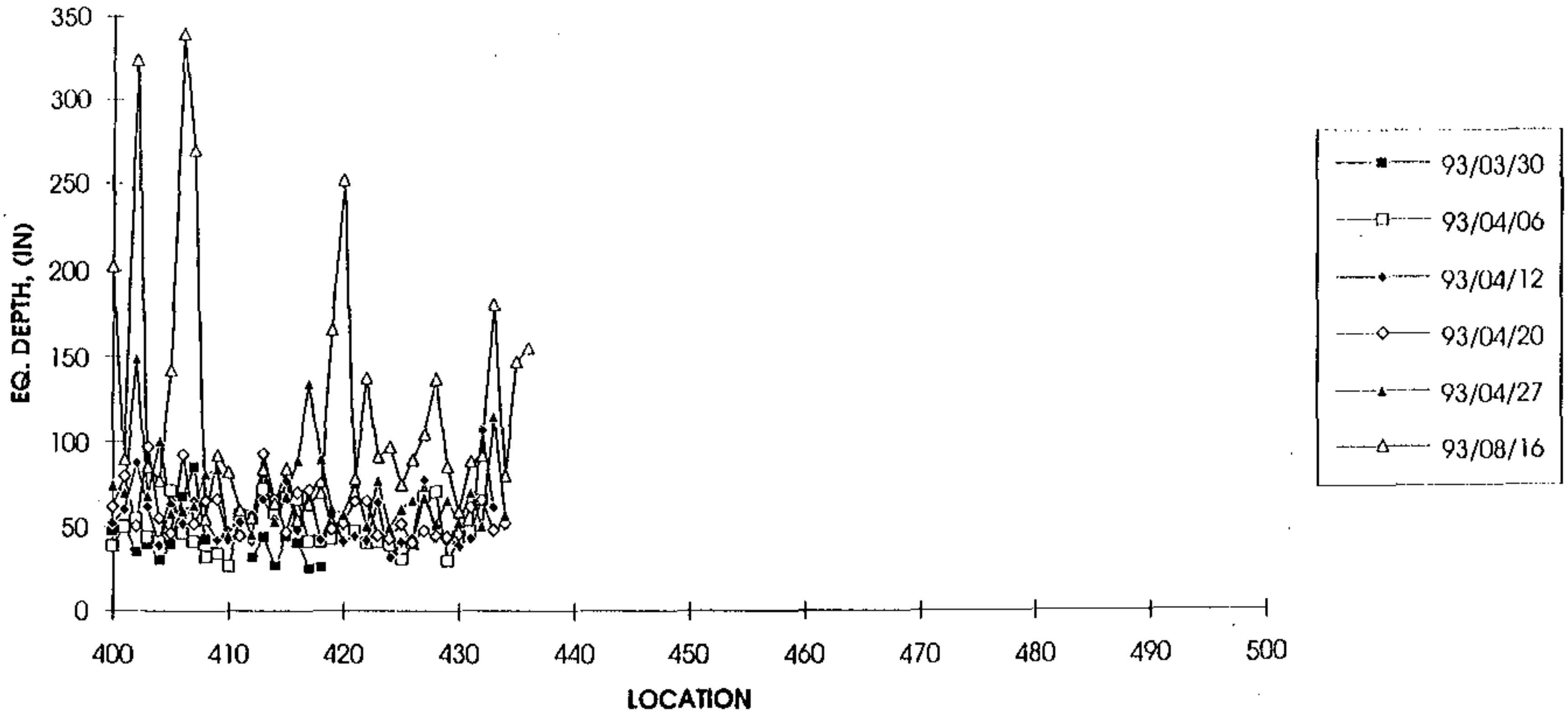

HAINES HWY. STIFF LAYER DEPTH (DROP 1)
EQ. DEPTH (IN)
1000
750
500
250
0
200
210
220
230
240
250
260
270
280
290
300
LOCATION
93/03/21
93/03/30
93/04/06
93/04/12
93/04/20
93/04/27
93/08/16

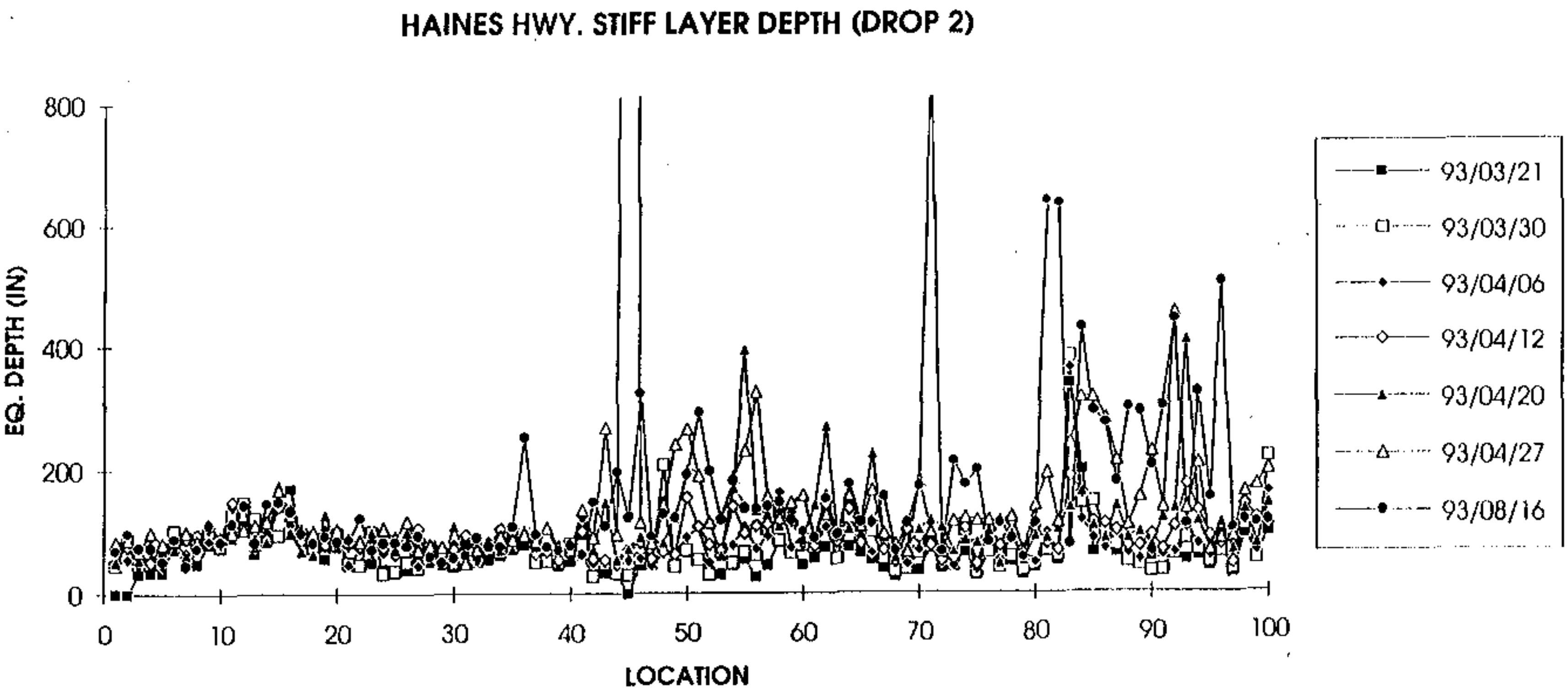
HAINES HWY. STIFF LAYER DEPTH (DROP 1)



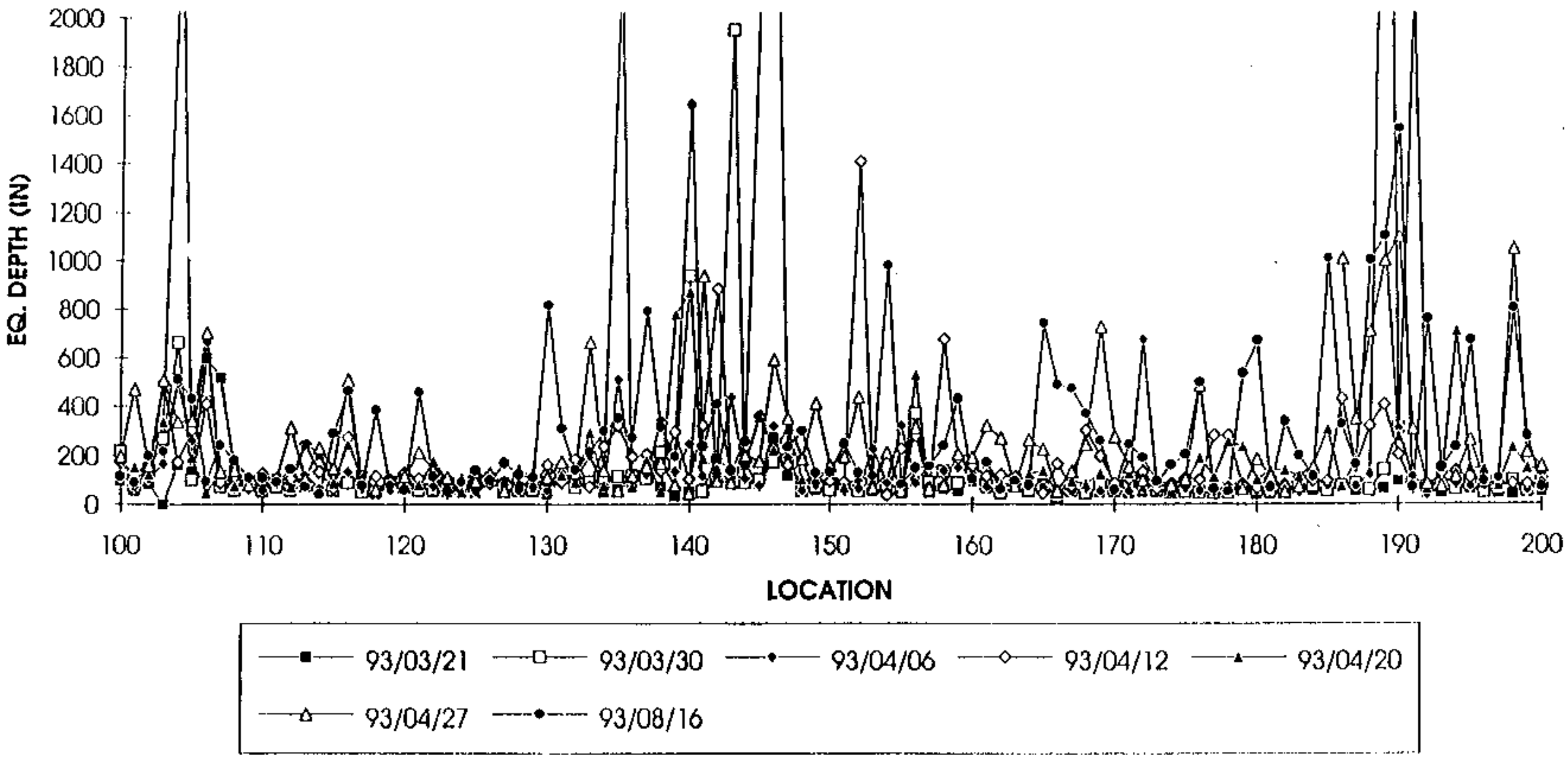
B-6-5

HAINES HWY. STIFF LAYER DEPTH (DROP 1)

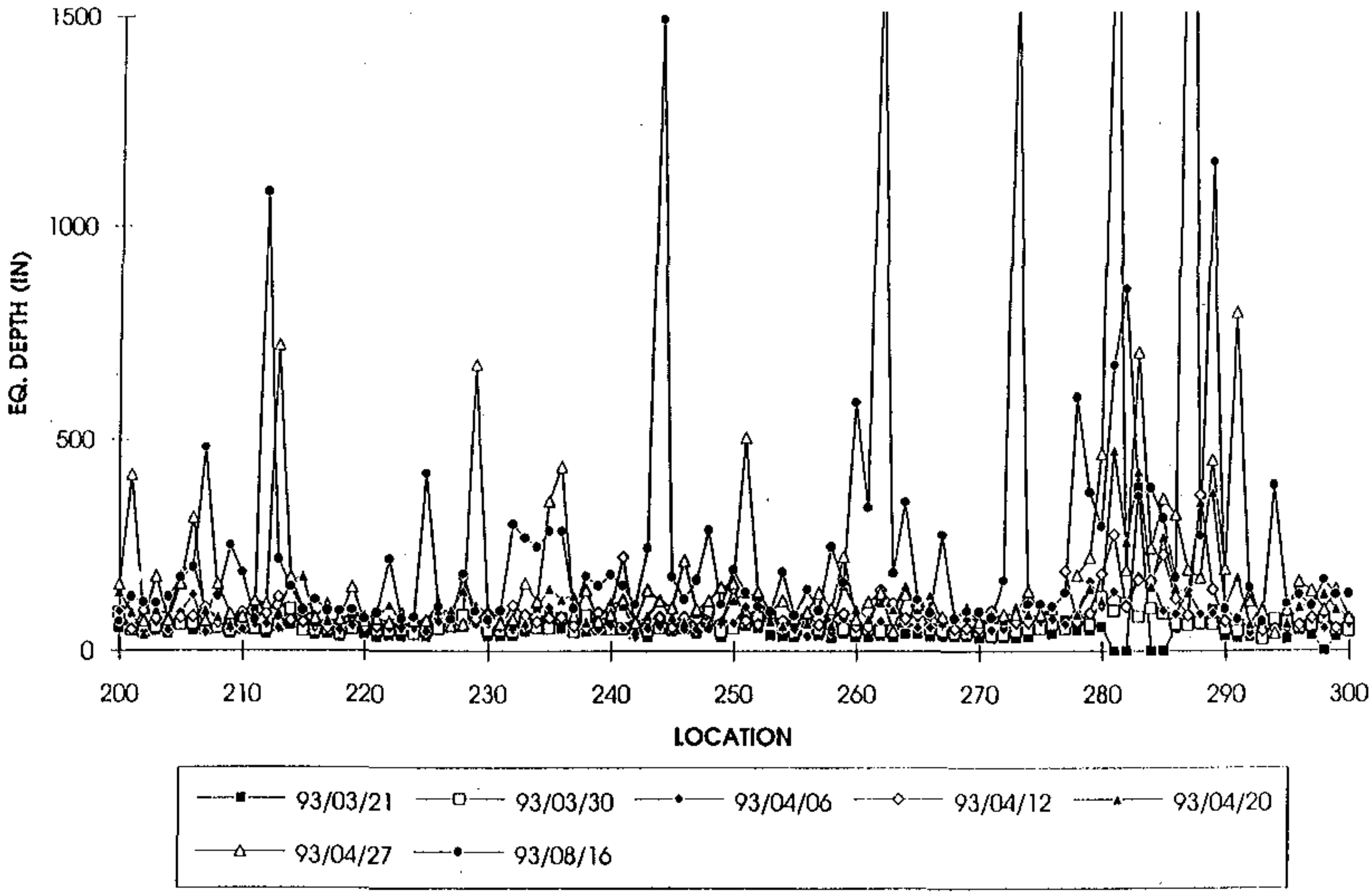


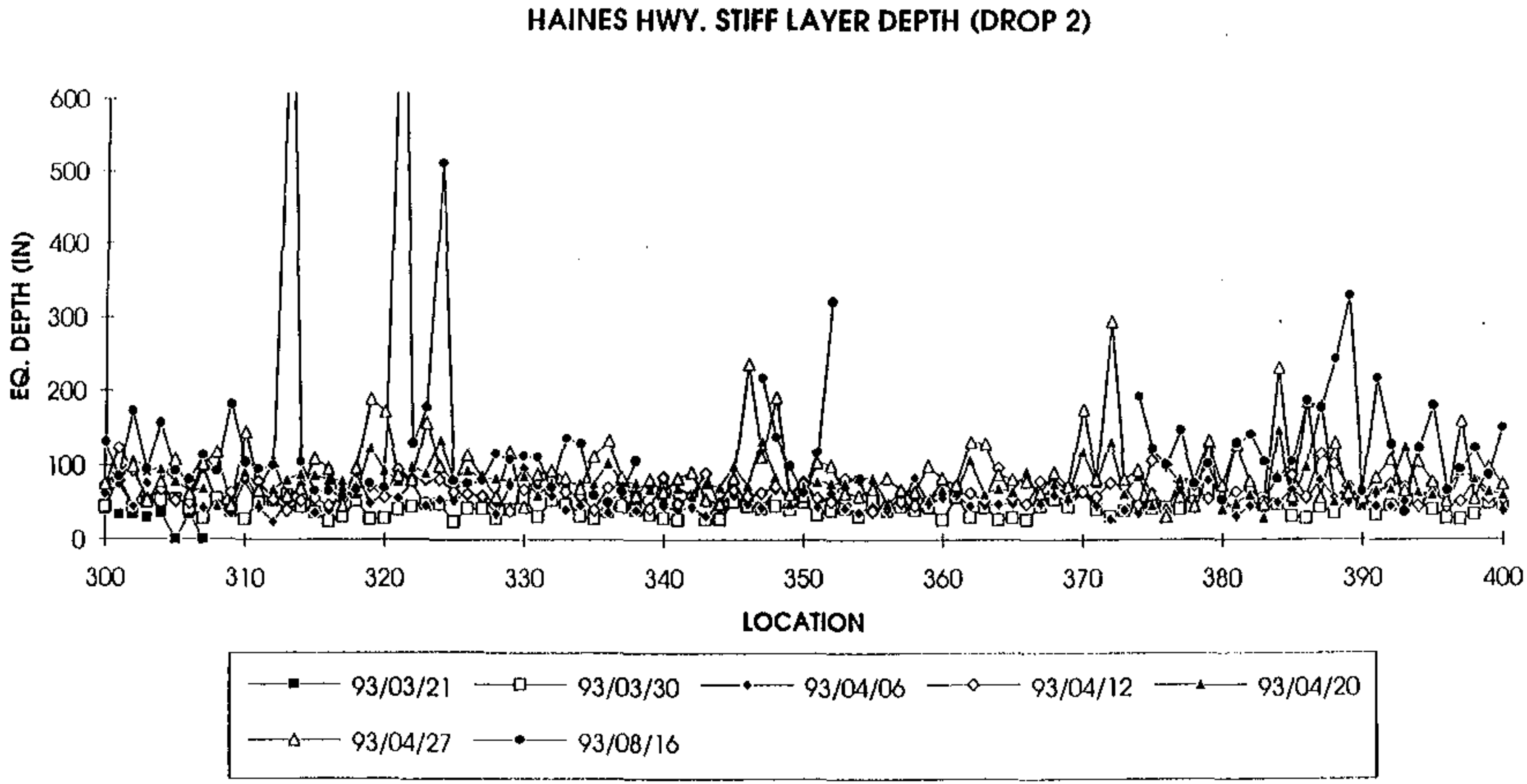

HAINES HWY. STIFF LAYER DEPTH (DROP 2)
EQ. DEPTH (IN)
800
600
400
200
0
0
10
20
30
40
50
60
70
80
90
100
LOCATION
93/03/21
93/03/30
93/04/06
93/04/12
93/04/20
93/04/27
93/08/16

HAINES HWY. STIFF LAYER DEPTH (DROP 2)

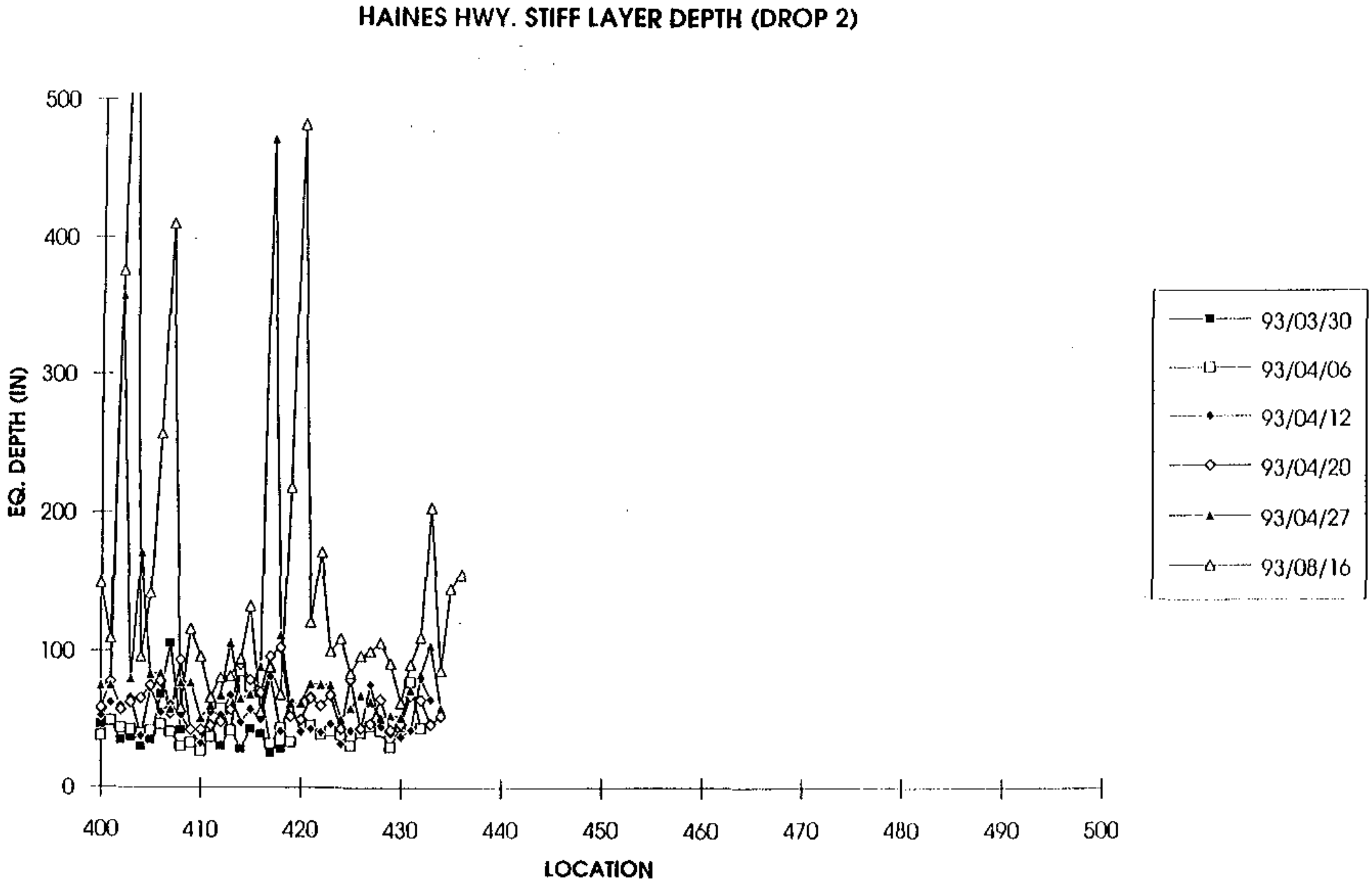


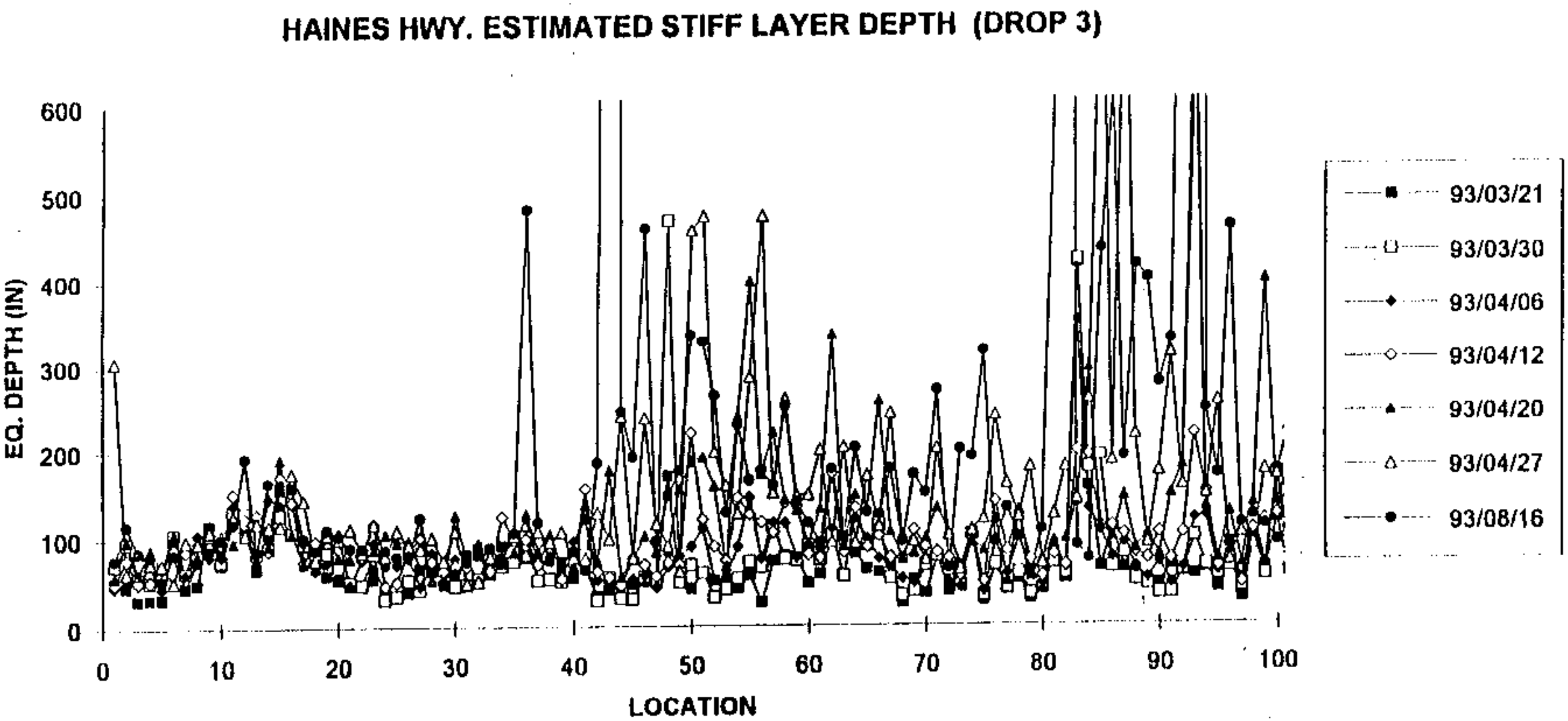
HAINES HWY. STIFF LAYER DEPTH (DROP 2)

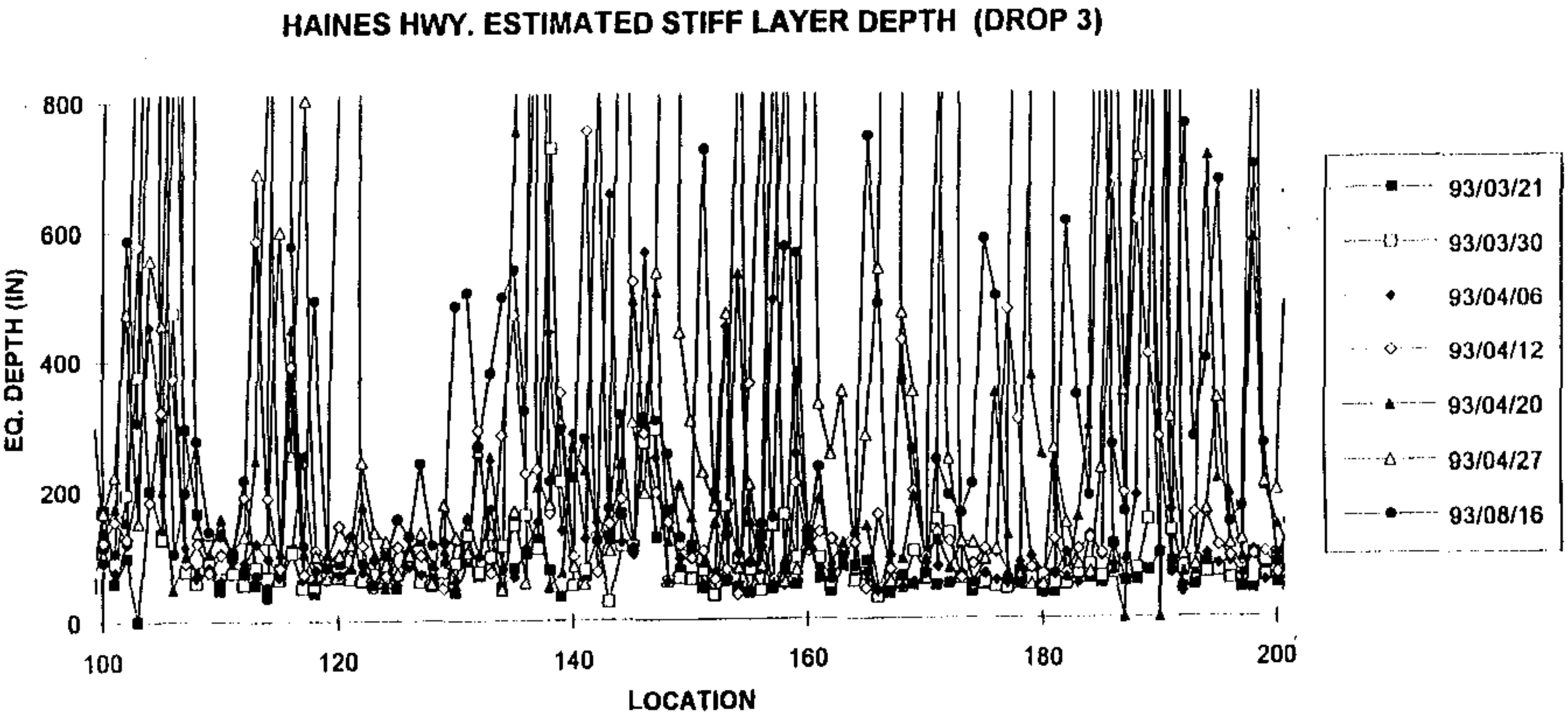


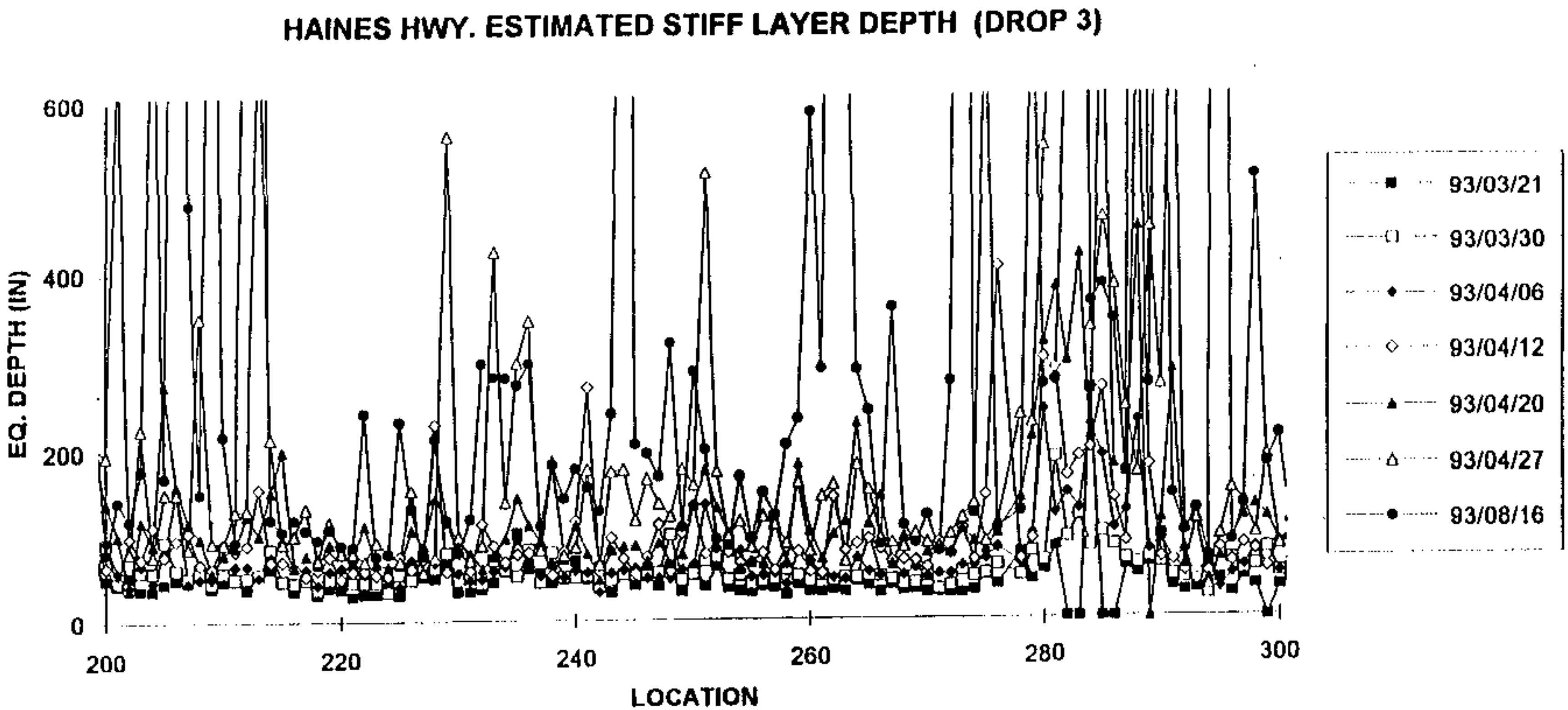


HAINES HWY. STIFF LAYER DEPTH (DROP 2)
EQ. DEPTH (IN)
600
500
400
300
200
100
0
300
310
320
330
340
350
360
370
380
390
400
LOCATION
93/03/21
93/03/30
93/04/06
93/04/12
93/04/20
93/04/27
93/08/16


HAINES HWY. STIFF LAYER DEPTH (DROP 2)
EQ. DEPTH (IN)
500
400
300
200
100
0
400
410
420
430
440
450
460
470
480
490
500
LOCATION
93/03/30
93/04/06
93/04/12
93/04/20
93/04/27
93/08/16

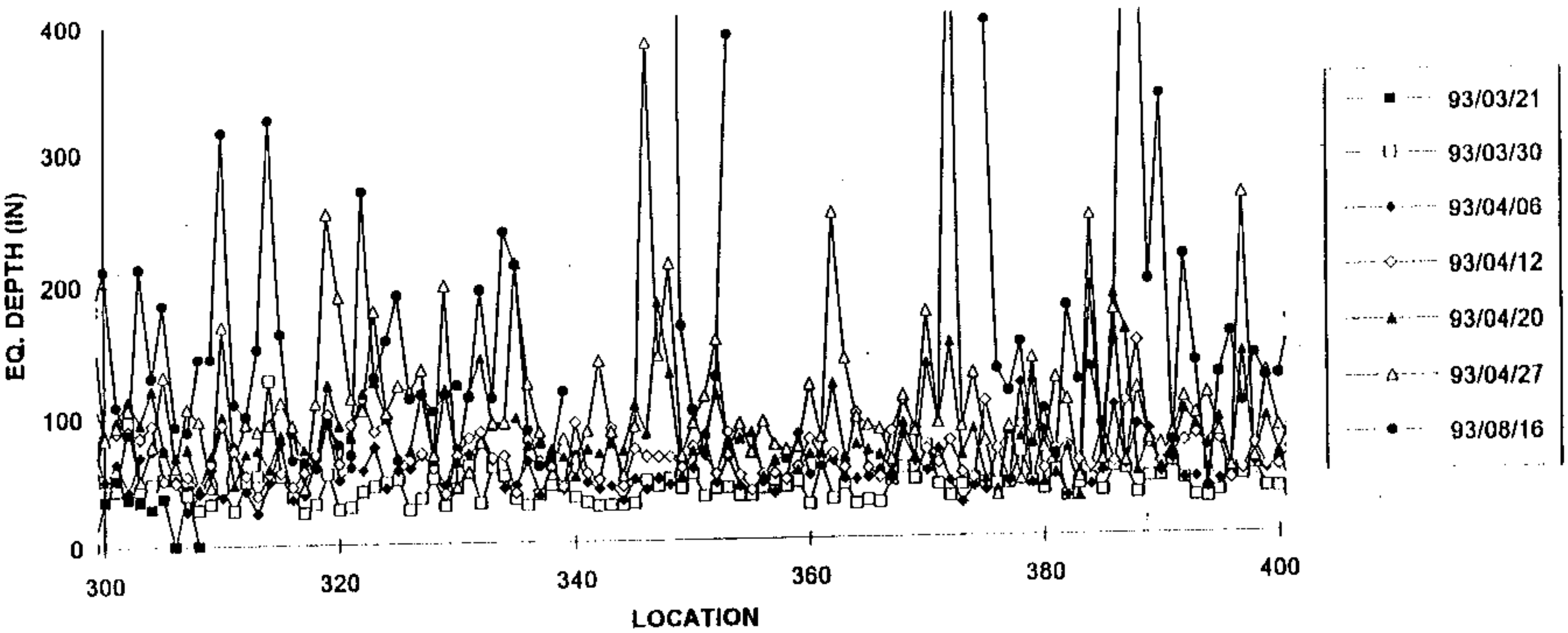

HAINES HWY. ESTIMATED STIFF LAYER DEPTH (DROP 3)
EQ. DEPTH (IN)
600
500
400
300
200
100
0
0
10
20
30
40
50
60
70
80
90
100
LOCATION
93/03/21
93/03/30
93/04/06
93/04/12
93/04/20
93/04/27
93/08/16

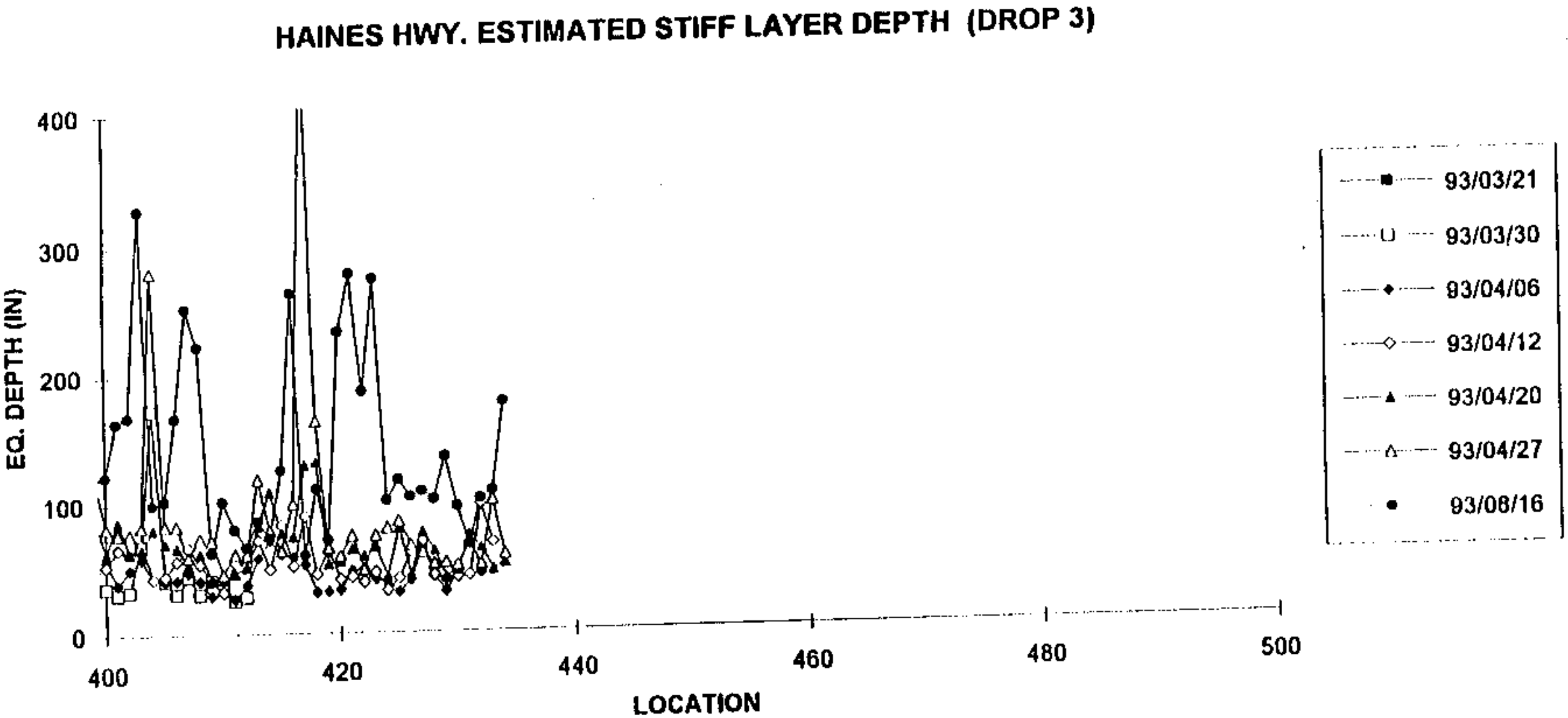

HAINES HWY. ESTIMATED STIFF LAYER DEPTH (DROP 3)
EQ. DEPTH (IN)
800
600
400
200
0
100
120
140
160
180
200
LOCATION
93/03/21
93/03/30
93/04/06
93/04/12
93/04/20
93/04/27
93/08/16



HAINES HWY. ESTIMATED STIFF LAYER DEPTH (DROP 3)
600
400
200
0
EQ. DEPTH (IN)
200
220
240
260
280
300
LOCATION
93/03/21
93/03/30
93/04/06
93/04/12
93/04/20
93/04/27
93/08/16

HAINES HWY. ESTIMATED STIFF LAYER DEPTH (DROP 3)





HAINES HWY. ESTIMATED STIFF LAYER DEPTH (DROP 3)
EQ. DEPTH (IN)
400
300
200
100
0
400
420
440
460
480
500
LOCATION
93/03/21
93/03/30
93/04/06
93/04/12
93/04/20
93/04/27
93/08/16

APPENDIX C

ROUGHNESS AND RUT MEASUREMENT DATA
ON THE STEESE/ELLIOT HIGHWAY
(1994-1995)

TABLE OF CONTENTS

APPENDIX C -- ROUGHNESS AND RUT MEASUREMENT DATA ON THE STEESE/ELLIOT HIGHWAY (1994-1995)

APPENDIX C-1

STEESE/ELLIOT NORTHBOUND LANE
RUT MEASUREMENT USING STRAIGHT-EDGE
(DOT DATA - 1994)

RUT MEASUREMENT OF ELLIOT HIGHWAY
(by DOT, Alaska)

CDS Miles	Measured Rut (mm) 4/28/94	5/5/94	5/12/94	5/19/94	5/26/94	9/28/94
11.50	2.0	2.7	2.4	2.5	2.7	2.8
11.70	9.1	9.0	9.7	9.7	8.9	9.1
11.90	7.0	7.1	6.9	6.8	8.0	-
12.10						
12.30	2.3	2.3	3.0	2.3	2.9	2.7
12.50	6.7	6.1	6.2	6.0	5.8	5.7
12.70	7.0	5.0	4.5	4.5	4.5	4.2
12.90	4.8	5.4	5.4	5.4	5.3	5.1
13.10	4.1	4.2	3.8	3.5	3.8	3.9
13.30	5.8	5.7	5.9	5.9	5.9	5.5
13.50	11.6	10.9	10.2	10.2	9.4	9.4
13.70	5.3	6.2	5.8	5.0	5.5	4.8
13.90	8.6	8.0	7.6	7.8	7.7	8.0
14.10	7.3	6.9	6.8	6.5	6.7	7.1
14.30	7.0	6.9	7.9	7.1	6.2	7.5
14.50	11.8	11.7	11.6	11.8	11.2	11.1
14.70	11.1	11.4	11.4	11.5	11.1	10.3
14.90	5.9	6.4	6.2	6.4	6.4	6.0
16.70	14.2	14.0	14.0	12.9	12.6	11.9
16.90	10.8	11.4	11.1	11.5	11.2	10.4
17.10	8.8	7.9	8.0	8.6	7.7	7.7
17.30	4.2	4.4	4.3	4.3	4.3	4.2
17.50	6.8	6.8	6.8	6.4	6.8	6.1
17.70	5.4	5.2	5.1	4.8	4.5	4.2
17.90	8.1	8.5	8.5	8.6	8.1	7.6
18.10	2.6	2.7	2.3	2.2	2.0	2.1
18.30	0.5	1.0	0.8	1.0	0.4	1.0
18.50	1.1	1.2	1.3	1.2	1.1	1.0
18.70	1.1	1.3	1.9	1.5	1.6	1.6
18.90	1.7	1.6	1.3	1.4	1.7	1.8
19.10	0.3	0.3	0.6	0.4	0.2	0.3
19.30	3.6	3.4	3.8	3.9	4.0	4.2
19.50	3.2	2.9	3.6	3.3	3.3	3.4
19.70	3.2	3.1	3.2	3.7	3.5	3.1
19.90	3.2	4.3	5.3	6.3	6.4	21.2

20.10	1.4	1.1	1.2	0.9	0.6	0.9
20.30	0.8	0.8	0.6	0.6	0.6	0.6
20.50	1.7	2.4	1.9	2.2	1.7	2.3
20.70	1.4	1.2	1.3	0.8	0.9	0.9
20.90	5.8	5.9	5.5	6.0	6.8	7.6
21.10	2.2	2.1	2.3	2.4	2.1	1.8
21.30	3.1	3.0	3.2	3.0	3.0	3.0
21.50	2.8	2.9	3.0	2.9	2.8	2.7
30.10	1.6	1.2	1.4	1.4	1.8	3.7
30.40	4.6	3.3	3.5	3.5	3.4	4.4
30.50	7.6	6.7	8.3	8.0	8.1	11.6
30.70	5.4	5.6	5.2	5.3	5.8	-
30.90	6.7	6.9	6.8	6.9	6.6	6.9
31.10	3.7	3.4	4.2	3.5	3.2	3.6
31.30	5.0	5.1	4.9	4.9	4.8	5.1
31.50	8.4	5.8	5.3	5.6	5.2	4.6
31.70	4.1	4.1	4.0	4.1	3.8	3.6
31.90	6.6	6.5	6.2	6.1	5.8	5.9
32.10	7.0	5.8	6.1	5.9	5.7	5.4
32.30	7.7	5.0	5.5	5.0	4.8	4.3
32.50	4.4	5.2	4.8	4.3	5.4	4.6
32.70	4.4	4.9	4.6	4.8	4.2	4.1
32.90	5.4	4.8	4.9	4.9	5.1	4.6
33.10	8.0	9.1	11.6	11.4	10.9	10.6
33.30	2.5	1.6	1.7	1.2	1.2	2.7
33.50	2.7	2.4	3.1	1.9	1.8	2.2
33.70	3.6	2.8	3.1	3.6	3.2	2.1
33.90	3.7	4.5	4.4	4.5	4.3	4.5
34.10	6.6	6.9	6.7	6.8	6.5	5.8
34.30	0.9	1.9	1.7	2.4	2.0	3.1
34.50	8.4	12.0	13.3	12.9	12.5	11.6
34.70	4.5	4.0	3.8	4.2	3.9	3.6
34.90	4.3	4.7	4.4	4.1	4.2	4.2
35.10	4.9	4.2	4.1	3.7	3.7	3.6
35.30	6.5	6.0	6.4	5.7	5.8	5.1
35.50	4.5	4.4	4.3	4.0	3.7	4.0
35.70	6.3	7.7	6.7	6.8	6.4	5.7
35.90	3.5	4.2	3.8	3.5	3.8	3.9

36.10	7.1	6.5	6.2	6.6	6.0	6.1
36.30	5.0	4.3	4.2	4.5	4.3	3.7
36.50	12.6	12.5	11.8	11.9	11.4	11.7
36.70	5.1	4.4	4.6	4.3	4.5	4.3
36.90	4.9	5.3	5.1	5.4	5.4	5.4
37.10	4.0	4.6	4.2	4.4	4.3	3.9

APPENDIX C-2

STEESE/ELLIOT NORTHBOUND AND SOUTHBOUND LANES RUT MEASUREMENT USING STRAIGHT-EDGE (UAF/TRC DATA - 1994/1995)

RUT MEASUREMENT OF ELLIOT HIGHWAY

CDS Miles	Measured Rut (mm)					
	Sep. 8. 94		June 1. 95		Sep. 1. 95	
	North Bound	South Bound	North Bound	South Bound	North Bound	South Bound
11.00	-	-	-	-	3.0	3.0
11.25	-	-	-	-	5.0	11.0
11.50	-	-	-	-	6.0	9.0
11.75	-	-	-	-	7.0	6.0
12.00	-	-	-	-	3.0	3.0
12.25	-	-	-	-	10.0	4.0
12.50	4.0	3.5	6.0	6.0	6.0	5.0
12.75	4.5	3.0	5.0	5.0	3.0	4.0
13.00	4.0	2.5	5.0	5.0	7.0	8.0
13.25	7.0	4.5	23.0	9.0	4.0	2.0
13.50	4.0	4.0	-	-	7.0	2.0
13.75	-	-	-	-	22.0	5.0
14.00	5.0	4.0	4.0	3.0	16.0	10.0
14.25	6.0	5.5	10.0	7.0	6.0	3.0
14.50	5.0	3.5	4.0	8.0	5.0	6.0
14.75	5.5	4.0	7.0	5.0	6.0	3.0
15.00	4.0	3.0	8.0	6.0	83.0	40.0
15.25	46.0	29.0	84.0	50.0	5.0	4.0
15.50	3.0	4.0	7.0	5.0	7.0	15.0
15.75	4.5	5.5	8.0	5.0	4.0	4.0
16.00	4.0	3.5	3.0	3.0	14.0	6.0
16.25	5.0	2.5	5.0	3.0	13.0	11.0
16.50	21.0	3.5	79.0	21.0	51.0	9.0
16.75	5.0	3.0	5.0	4.0	90.0	20.0
17.00	82.0	23.0	90.0	33.0	4.0	10.0
17.25	17.0	30.0	3.0	13.0	3.0	6.0
17.50	3.0	5.0	5.0	5.0	33.0	10.0
17.75	4.0	2.5	10.0	4.0	1.0	3.0
18.00	-	-	6.0	5.0	3.0	8.0
18.25	4.5	2.0	3.0	4.0	2.0	3.0
18.50	2.5	2.5	3.0	2.0	1.0	1.0
18.75	1.0	2.5	2.0	3.0	21.0	8.0
19.00	2.0	2.0	1.0	3.0	53.0	2.0
19.25	4.5	2.5	19.0	19.0	0.0	5.0
19.50	-	-	4.0	7.0	34.0	45.0
19.75	-	-	8.0	8.0	7.0	4.0

20.00	-	-	18.0	6.0	52.0	12.0
20.25	-	-	5.0	5.0	25.0	15.0
20.50	5.0	2.0	22.0	9.0	2.0	2.0
20.75	5.0	2.5	18.0	10.0	4.0	2.0
21.00	3.5	3.0	6.0	6.0	4.0	2.0
21.25	-	-	2.0	2.0	3.0	1.0
21.50	3.5	4.0	2.0	2.0	2.0	1.0
21.75	2.5	3.0	-	-	50.0	8.0
22.00	4.0	2.0	2.0	2.0	11.0	9.0
22.25	-	-	27.0	27.0	27.0	5.0
22.50	-	-	5.0	4.0	70.0	36.0
22.75	7.0	3.5	10.0	7.0	49.0	7.0
23.00	-	-	25.0	24.0	37.0	2.0
23.25	-	-	6.0	4.0	12.0	15.0
23.50	-	-	5.0	3.0	5.0	3.0
23.75	-	-	6.0	8.0	2.0	0.0
24.00	4.5	1.5	2.0	4.0	5.0	2.0
24.25	4.5	2.0	2.0	3.0	2.0	1.0
24.50	4.5	4.0	5.0	3.0	3.0	5.0
24.75	4.0	2.5	2.0	4.0	43.0	7.0
25.00	3.0	2.5	3.0	3.0	3.0	4.0
25.25	66.0	12.0	100.0	65.0	91.0	15.0
25.50	-	-	21.0	21.0	6.0	3.0
25.75	-	-	9.0	9.0	6.0	1.0
26.00	3.0	3.0	6.0	4.0	3.0	0.0
26.25	5.0	2.0	21.0	21.0	4.0	2.0
26.50	4.0	3.0	2.0	1.0	19.0	7.0
26.75	4.0	1.0	3.0	3.0	11.0	2.0
27.00	4.0	4.0	5.0	2.0	7.0	3.0
27.25	6.0	4.0	7.0	6.0	5.0	3.0
27.50	8.0	4.5	9.0	6.0	3.0	2.0
27.75	10.0	2.0	107.0	4.0	5.0	1.0
28.00	4.0	2.5	3.0	2.0	2.0	1.0
28.25	3.5	2.5	1.0	1.0	25.0	1.0
28.50	3.0	1.0	2.0	2.0	9.0	3.0
28.75	4.0	1.0	1.0	1.0	5.0	3.0
29.00	7.5	2.5	7.0	2.0	54.0	1.0
29.25	2.5	3.0	1.0	1.0	4.0	1.0
29.50	16.0	8.0	4.0	3.0	3.0	2.0
29.75	4.0	2.0	9.0	6.0	2.0	1.0

30.00	3.0	3.0	5.0	4.0	3.0	2.0
30.25	3.0	2.0	4.0	3.0	2.0	1.0
30.50	1.5	1.0	3.0	2.0	7.0	5.0
30.75	-	-	5.0	1.0	6.0	4.0
31.00	7.0	2.5	3.0	3.0	3.0	2.0
31.25	2.5	2.0	2.0	2.0	3.0	2.0
31.50	6.0	3.5	5.0	5.0	5.0	0.0
31.75	3.5	2.0	4.0	3.0	6.0	5.0
32.00	5.0	1.5	5.0	3.0	4.0	3.0
32.25	5.5	4.0	3.0	3.0	4.0	2.0
32.50	2.0	1.0	7.0	5.0	7.0	2.0
32.75	4.5	2.5	6.0	5.0	7.0	3.0
33.00	5.0	2.5	4.0	3.0	6.0	3.0
33.25	4.0	2.0	10.0	6.0	5.0	2.0
33.50	2.5	1.5	4.0	3.0	19.0	4.0
33.75	3.0	6.0	2.0	1.0	1.0	0.0
34.00	4.5	5.0	4.0	3.0	10.0	1.0
34.25	2.0	2.5	1.0	1.0	3.0	0.0
34.50	9.0	4.0	7.0	4.0	4.0	1.0
34.75	2.5	1.0	4.0	4.0	7.0	5.0
35.00	3.5	1.0	3.0	2.0	90.0	25.0
35.25	4.5	2.5	4.0	3.0	7.0	5.0
35.50	11.0	5.0	5.0	5.0	10.0	1.0
35.75	7.0	3.0	6.0	6.0	4.0	9.0
36.00	7.0	1.0	6.0	6.0	5.0	2.0
36.25	9.0	3.0	6.0	6.0	21.0	1.0
36.50	5.0	4.0	3.0	2.0	11.0	4.0
36.75	10.0	4.0	4.0	3.0	7.0	1.0
37.00	9.0	2.5	8.0	8.0	7.0	2.0
37.25	8.0	3.0	5.0	5.0	3.0	1.0
37.50	7.0	4.5	22.0	11.0	-	-
37.75	5.0	2.5	2.0	2.0	-	-

APPENDIX C-3

STEESE/ELLIOT NORTHBOUND AND SOUTHBOUND LANES ROUGHNESS AND RUT MEASUREMENTS USING PROFILOMETER (DOT DATA - 1995)

RUT AND IRI MEASUREMENT OF ELLIOT HIGHWAY

(by DOT, Alaska)

CDS Miles From	To	Rut Depth (mm) and IRI Feb. 21, 95 Rut	Feb. 21, 95 IRI	May 11, 95 Rut	May 11, 95 IRI	Aug. 31, 95 Rut	Aug. 31, 95 IRI	Aug. 31, 95 * Southbound Rut	Aug. 31, 95 * Southbound IRI
10.90	10.82	-	-	-	-	-	-	5	2.03
11.00	10.90	-	-	-	-	-	-	13	5.19
11.11	11.20	-	-	-	-	2	1.84	4	3.31
11.20	11.30	-	-	-	-	10	3.95	3	2.52
11.30	11.40	5	3.11	4	1.45	4	3.24	4	2.95
11.40	11.50	3	7.21	2	1.86	1	3.00	4	3.58
11.50	11.60	6	2.69	3	1.16	5	2.56	4	2.03
11.60	11.70	5	6.09	1	1.23	4	3.53	5	2.23
11.70	11.80	9	7.28	2	1.68	2	1.92	7	1.79
11.80	11.90	11	8.61	2	1.73	3	2.41	7	1.94
11.90	12.00	9	4.49	3	1.72	3	2.14	5	1.98
12.00	12.10	13	8.75	4	2.04	2	2.12	5	2.32
12.10	12.20	9	10.86	20	3.05	1	2.07	5	2.21
12.20	12.30	6	5.27	5	1.82	1	2.46	5	1.95
12.30	12.40	6	3.58	7	2.55	5	1.88	5	1.67
12.40	12.50	5	1.90	2	2.08	4	1.52	6	1.64
12.50	12.60	4	1.78	-1	1.51	5	1.22	4	1.68
12.60	12.70	6	1.85	3	1.30	6	1.41	4	2.87
12.70	12.80	5	1.81	1	1.27	5	1.65	6	1.34
12.80	12.90	3	3.66	-3	1.10	3	2.90	7	1.71
12.90	13.00	4	1.74	-1	1.36	6	1.31	5	2.08
13.00	13.10	3	2.00	-3	1.48	7	1.65	8	2.19
13.10	13.20	5	1.86	-1	1.55	5	1.44	7	1.79
13.20	13.30	5	2.17	2	1.53	9	1.50	6	1.67
13.30	13.40	1	3.10	2	1.62	8	1.59	8	2.15
13.40	13.50	1	2.49	6	2.11	9	1.74	5	2.06
13.50	13.60	5	2.71	-1	1.86	9	1.57	5	2.26
13.60	13.70	-8	3.04	4	1.64	0	1.30	5	1.71
13.70	13.80	-8	2.81	4	2.20	2	1.29	4	3.87
13.80	13.90	-6	3.00	2	1.93	4	1.43	3	1.88
13.90	14.00	2	4.37	3	1.80	6	2.36	6	1.78

14.00	14.10	4	3.74	1	1.74	2	2.63	7	1.88
14.10	14.20	2	2.08	-1	1.76	5	1.59	5	2.02
14.20	14.30	-2	2.57	1	1.45	10	1.68	8	1.69
14.30	14.40	0	2.11	2	1.87	8	1.73	7	1.54
14.40	14.50	-1	3.13	-0	1.55	5	1.48	6	1.77
14.50	14.60	-4	2.63	0	3.09	9	1.28	5	1.97
14.60	14.70	-0	2.18	-2	5.31	5	1.47	6	1.87
14.70	14.80	3	1.85	1	1.62	7	1.07	5	2.05
14.80	14.90	5	1.91	0	1.66	6	1.13	7	2.14
14.90	15.00	3	1.72	-2	1.32	6	1.38	7	1.86
15.00	15.10	9	1.81	-1	2.57	6	1.73	7	1.80
15.10	15.20	5	2.11	-2	4.23	6	1.61	22	2.33
15.20	15.30	6	1.87	-2	3.32	5	1.86	3	1.43
15.30	15.40	12	3.21	-2	2.22	22	2.63	3	1.44
15.40	15.50	17	5.97	-1	1.20	7	1.98	10	2.23
15.50	15.60	8	2.27	0	1.22	5	2.01	6	1.95
15.60	15.70	11	2.72	-0	0.94	10	2.35	5	1.60
15.70	15.80	5	1.78	5	3.11	5	1.66	7	1.91
15.80	15.90	6	1.86	13	4.58	5	1.24	5	1.57
15.90	16.00	5	1.76	4	1.77	7	1.55	4	1.43
16.00	16.10	5	1.99	5	2.44	5	1.02	3	1.70
16.10	16.20	4	1.79	2	2.73	4	1.23	3	1.95
16.20	16.30	2	1.81	5	7.39	5	1.21	5	1.66
16.30	16.40	2	2.42	1	4.27	3	1.30	5	1.74
16.40	16.50	2	2.99	-2	2.14	4	1.40	9	1.58
16.50	16.60	4	2.61	0	1.75	6	1.51	6	1.55
16.60	16.70	7	3.01	10	3.78	12	2.05	5	1.33
16.70	16.80	0	2.72	1	3.03	9	1.72	5	1.72
16.80	16.90	2	2.22	2	1.79	10	1.67	10	1.86
16.90	17.00	9	2.60	4	2.01	15	2.14	8	1.70
17.00	17.10	16	3.92	4	1.81	19	2.77	8	1.85
17.10	17.20	5	2.31	4	1.43	5	1.35	5	1.70
17.20	17.30	7	2.51	3	1.19	6	1.59	7	2.11
17.30	17.40	5	2.59	2	1.45	3	1.40	9	1.56
17.40	17.50	5	2.22	1	1.21	4	1.63	6	1.61
17.50	17.60	6	1.71	-0	2.75	6	1.35	5	2.10
17.60	17.70	7	2.40	3	1.91	6	1.75	7	4.40
17.70	17.80	5	2.18	4	3.04	4	2.04	4	3.08
17.80	17.90	6	4.13	-1	3.15	6	4.14	1	2.12
17.90	18.00	6	4.81	0	3.61	5	3.16	3	1.21

18.00	18.10	3	1.99	-0	1.27	3	1.77	4	1.46
18.10	18.20	3	2.00	-0	0.91	5	1.18	9	3.83
18.20	18.30	4	2.14	0	2.23	5	1.66	6	2.98
18.30	18.40	5	3.40	-3	3.23	4	2.86	4	1.64
18.40	18.50	3	4.67	-3	1.17	2	3.09	5	1.38
18.50	18.60	3	2.77	-3	1.20	3	1.38	6	1.14
18.60	18.70	3	2.25	1	0.78	3	1.92	6	1.09
18.70	18.80	3	1.45	3	3.04	4	1.35	4	1.01
18.80	18.90	4	1.44	13	5.58	4	0.90	9	3.72
18.90	19.00	4	1.51	11	3.39	4	1.02	8	2.58
19.00	19.10	10	3.55	3	5.43	13	3.04	6	1.52
19.10	19.20	14	2.89	-3	2.29	8	3.08	6	1.69
19.20	19.30	7	2.03	3	2.73	7	1.76	11	4.09
19.30	19.40	7	2.06	6	1.63	11	2.35	7	4.86
19.40	19.50	9	3.20	8	3.74	13	3.70	9	4.59
19.50	19.60	8	6.87	11	4.69	7	3.35	6	1.30
19.60	19.70	9	3.25	1	3.56	5	3.31	8	1.36
19.70	19.80	5	1.70	-1	5.19	5	1.45	8	2.84
19.80	19.90	7	2.01	-1	2.36	8	1.77	16	3.64
19.90	20.00	18	3.18	0		21	3.57	8	1.14
20.00	20.10	11	2.45	11	3.87	14	2.47	9	2.62
20.10	20.20	6	1.28	12	2.47	7	1.30	8	2.16
20.20	20.30	9	1.87	17	3.66	10	2.06	9	1.84
20.30	20.40	5	1.82	5	5.64	11	1.97	11	1.85
20.40	20.50	4	1.21	5	4.14	7	1.15	8	2.28
20.50	20.60	5	1.35	7	1.81	6	1.01	5	1.19
20.60	20.70	6	1.54	5	1.46	7	1.41	4	2.97
20.70	20.80	5	1.50	6	1.38	5	1.18	7	2.09
20.80	20.90	4	3.12	5	4.54	5	2.68	7	2.28
20.90	21.00	6	1.72	3	2.14	8	1.64	8	2.62
21.00	21.10	6	3.14	1	1.12	7	2.74	4	1.89
21.10	21.20	5	3.39	-1	1.19	4	2.78	3	0.88
21.20	21.30	3	2.88	1	1.32	3	1.81	4	0.74
21.30	21.40	2	1.39	-2	1.11	4	1.14	4	1.37
21.40	21.50	3	1.00	0	1.18	4	0.81	3	2.42
21.50	21.60	3	2.78	0	1.04	3	1.49	2	1.07
21.60	21.70	4	3.39	1	1.01	3	2.10	2	0.75
21.70	21.80	3	1.88	3	1.50	4	1.17	3	0.69
21.80	21.90	3	1.50	4	2.38	4	0.86	5	2.82
21.90	22.00	4	1.30	1	2.17	3	0.77	10	3.50

22.00	22.10	8	3.83	11	3.81	8	3.01	11	4.24
22.10	22.20	12	6.26	4	4.92	16	5.72	10	4.04
22.20	22.30	4	3.13	7	4.78	15	3.76	8	2.37
22.30	22.40	3	4.93	1	2.61	7	6.14	10	2.01
22.40	22.50	6	2.74	5	2.33	7	2.37	10	2.03
22.50	22.60	8	3.13	11	3.69	18	2.66	7	4.61
22.60	22.70	4	1.94	13	3.51	13	1.67	2	4.70
22.70	22.80	10	3.33	-2	2.64	11	2.74	6	2.60
22.80	22.90	5	6.37	3	3.05	13	4.23	12	2.46
22.90	23.00	7	3.49	7	3.24	5	4.25	7	2.34
23.00	23.10	7	3.22	6	1.78	2	5.83	9	1.86
23.10	23.20	8	2.37	2	1.27	6	3.17	4	1.82
23.20	23.30	11	2.24	0	1.23	20	2.81	8	2.26
23.30	23.40	11	2.59	2	1.15	14	2.92	5	4.07
23.40	23.50	7	4.40	2	1.22	9	2.58	7	1.87
23.50	23.60	7	6.09	4	1.49	9	4.39	18	2.84
23.60	23.70	8	2.01	2	0.83	9	1.33	6	1.55
23.70	23.80	5	1.67	2	1.32	9	1.67	7	1.88
23.80	23.90	7	1.51	3	1.11	7	1.22	6	2.84
23.90	24.00	8	2.66	6	1.21	7	1.89	3	1.34
24.00	24.10	4	3.74	6	1.12	7	2.34	5	1.18
24.10	24.20	5	3.54	4	1.21	5	1.49	5	0.93
24.20	24.30	6	3.24	11	1.56	3	0.87	4	0.92
24.30	24.40	2	3.85	5	1.10	5	1.05	4	0.91
24.40	24.50	8	3.02	5	1.61	8	0.94	6	0.89
24.50	24.60	5	2.56	6	1.45	9	0.97	4	0.87
24.60	24.70	6	3.09	10	1.58	5	0.94	4	0.90
24.70	24.80	-3	2.69	3	1.21	6	0.89	3	3.35
24.80	24.90	2	2.72	2	1.61	5	0.81	6	2.09
24.90	25.00	8	3.82	-1	1.58	5	2.16	8	2.33
25.00	25.10	6	3.04	1	1.23	7	1.98	11	3.60
25.10	25.20	1	6.71	1	1.20	7	3.32	12	3.43
25.20	25.30	4	5.84	2	1.26	10	2.86	4	3.30
25.30	25.40	0	3.86	2	1.09	10	4.05	9	2.71
25.40	25.50	6	3.43	-1	2.51	5	3.90	13	3.66
25.50	25.60	1	2.62	4	2.17	8	2.52	12	2.94
25.60	25.70	8	2.94	13	2.82	13	3.22	16	3.40
25.70	25.80	4	3.21	8	2.01	19	3.79	6	3.06
25.80	25.90	4	2.64	6	1.39	10	2.21	5	2.25
25.90	26.00	3	3.87	3	1.24	3	3.14	4	1.90

26.00	26.10	2	3.01	4	1.36	7	3.18	5	0.88
26.10	26.20	5	2.70	8	2.85	12	2.57	4	1.17
26.20	26.30	4	1.80	6	1.72	7	1.25	4	0.80
26.30	26.40	3	2.48	14	2.50	3	1.19	2	0.97
26.40	26.50	1	2.17	8	3.26	3	0.98	3	0.93
26.50	26.60	5	1.67	2	1.18	6	1.06	7	0.93
26.60	26.70	3	1.52	5	1.12	5	1.01	4	0.78
26.70	26.80	-1	1.81	3	1.10	8	0.96	5	0.83
26.80	26.90	1	1.52	7	1.49	6	0.69	4	0.78
26.90	27.00	1	1.89	4	1.23	9	1.00	4	0.78
27.00	27.10	0	1.54	2	1.15	6	0.86	5	0.96
27.10	27.20	-1	1.65	2	1.11	9	1.06	5	0.94
27.20	27.30	-1	1.88	-1	4.58	5	0.95	7	1.04
27.30	27.40	-1	1.78	0	2.66	3	0.83	6	0.96
27.40	27.50	3	1.94	5	6.03	12	1.36	6	0.93
27.50	27.60	0	1.80	-4	10.41	6	1.05	4	0.81
27.60	27.70	-3	2.34	4	8.44	7	1.02	5	0.96
27.70	27.80	-2	2.44	4	1.54	8	1.16	3	0.98
27.80	27.90	7	1.83	5	1.20	10	1.63	5	2.98
27.90	28.00	6	2.21	2	1.01	6	0.98	3	1.11
28.00	28.10	1	2.45	1	1.52	4	1.75	5	0.92
28.10	28.20	3	2.51	5	1.24	3	1.30	4	0.96
28.20	28.30	5	1.72	7	1.75	6	1.08	4	0.93
28.30	28.40	5	2.16	5	1.18	5	1.10	5	1.01
28.40	28.50	7	1.86	1	1.42	5	1.12	4	1.81
28.50	28.60	4	2.31	2	1.07	4	1.15	6	1.48
28.60	28.70	3	2.93	4	1.12	2	2.34	7	1.46
28.70	28.80	7	2.37	5	0.99	9	2.46	7	1.37
28.80	28.90	4	2.36	2	0.88	15	2.50	7	1.23
28.90	29.00	3	2.01	3	0.90	11	1.96	5	1.44
29.00	29.10	3	2.00	6	1.19	9	1.22	6	1.21
29.10	29.20	4	1.96	6	1.78	6	1.18	8	1.54
29.20	29.30	5	2.08	1	1.31	8	1.20	6	1.92
29.30	29.40	6	3.38	1	1.08	4	3.36	9	2.33
29.40	29.50	5	3.58	3	1.35	9	1.85	4	1.39
29.50	29.60	6	2.91	3	1.14	14	2.49	4	0.82
29.60	29.70	2	2.42	3	0.99	11	3.29	6	1.16
29.70	29.80	-1	1.60	4	1.24	5	0.89	6	1.10
29.80	29.90	7	2.10	6	1.35	8	0.98	3	0.78
29.90	30.00	6	2.65	5	1.17	6	1.06	3	0.82

30.00	30.10	7	2.10	4	1.27	10	1.54	3	0.87
30.10	30.20	8	2.45	1	1.03	7	1.06	3	1.09
30.20	30.30	4	2.46	3	1.31	5	1.13	3	3.00
30.30	30.40	4	2.15	2	1.09	5	1.02	5	2.25
30.40	30.50	3	3.76	1	0.93	1	3.04	5	3.15
30.50	30.60	4	3.10	-1	0.94	3	2.78	6	2.40
30.60	30.70	7	5.16	3	1.00	9	2.78	1	2.32
30.70	30.80	11	4.31	0	1.32	4	2.08	4	2.23
30.80	30.90	9	3.70	-3	1.14	7	2.81	4	1.06
30.90	31.00	8	1.89	2	1.73	7	1.57	3	0.92
31.00	31.10	1	2.55	-2	1.12	8	1.12	4	0.97
31.10	31.20	1	2.28	1	1.17	5	0.93	4	1.09
31.20	31.30	5	2.46	8	1.36	6	1.17	5	1.10
31.30	31.40	9	2.46	6	1.13	6	1.14	4	1.03
31.40	31.50	7	3.79	2	1.06	7	1.40	5	1.25
31.50	31.60	5	2.98	3	1.62	9	1.12	4	0.79
31.60	31.70	3	3.05	0	1.29	8	1.07	4	1.12
31.70	31.80	0	2.30	1	1.22	5	0.93	3	1.03
31.80	31.90	4	2.10	3	1.07	7	1.05	3	0.82
31.90	32.00	9	1.70	4	1.16	8	0.91	6	0.83
32.00	32.10	8	1.80	5	1.43	6	0.87	5	0.98
32.10	32.20	2	1.85	6	1.67	8	0.91	3	0.76
32.20	32.30	8	2.45	7	2.57	9	1.14	4	1.06
32.30	32.40	8	3.00	6	1.74	10	1.36	5	1.24
32.40	32.50	2	2.45	1	1.28	5	0.90	5	0.97
32.50	32.60	2	2.26	-4	1.60	5	1.22	3	0.84
32.60	32.70	2	2.12	1	1.46	4	1.11	3	0.89
32.70	32.80	6	1.93	7	1.28	7	1.15	4	0.96
32.80	32.90	5	2.40	7	1.63	6	0.83	4	0.76
32.90	33.00	5	2.99	5	1.81	8	1.00	5	1.12
33.00	33.10	9	2.39	7	1.80	8	1.31	5	1.04
33.10	33.20	5	2.13	-4	2.04	7	0.95	3	0.83
33.20	33.30	5	2.63	3	2.34	6	0.94	3	0.76
33.30	33.40	4	2.82	2	1.59	4	1.04	2	0.97
33.40	33.50	5	1.63	-1	2.05	6	1.35	3	0.78
33.50	33.60	3	2.42	5	2.01	5	0.84	4	0.93
33.60	33.70	0	3.06	4	1.67	4	0.90	5	0.89
33.70	33.80	4	2.30	1	1.51	4	1.09	6	0.93
33.80	33.90	6	2.10	4	1.67	5	0.98	7	1.04
33.90	34.00	5	2.34	4	2.03	2	1.11	4	1.03

34.00	34.10	7	1.93	3	1.42	2	1.22	5	1.19
34.10	34.20	4	2.19	1	2.08	7	1.09	3	1.03
34.20	34.30	3	2.39	1	1.51	11	1.25	3	1.06
34.30	34.40	7	2.04	-0	1.59	7	0.98	6	0.82
34.40	34.50	9	2.30	1	1.27	6	1.32	5	0.80
34.50	34.60	8	2.11	2	2.30	2	0.92	4	1.12
34.60	34.70	5	2.64	-	-	4	1.19	5	0.91
34.70	34.80	3	2.94	-	-	7	0.92	3	1.15
34.80	34.90	4	2.77	-	-	7	1.08	4	1.08
34.90	35.00	-1	3.48	-	-	8	1.01	4	0.87
35.00	35.10	6	2.19	-	-	7	1.39	6	0.88
35.10	35.20	3	2.30	-	-	15	2.80	6	0.80
35.20	35.30	9	2.86	-	-	8	1.56	7	1.57
35.30	35.40	-	-	-	-	10	1.41	6	1.24
35.40	35.50	-	-	-	-	4	1.22	6	1.29
35.50	35.60	-	-	-	-	12	1.39	7	1.35
35.60	35.70	-	-	-	-	9	1.40	5	1.26
35.70	35.80	-	-	-	-	10	1.44	5	1.24
35.80	35.90	-	-	-	-	9	1.54	5	1.19
35.90	36.00	-	-	-	-	8	1.46	6	1.34
36.00	36.10	-	-	-	-	7	2.02	7	1.78
36.10	36.20	-	-	-	-	7	1.77	3	1.68
36.20	36.30	-	-	-	-	7	1.79	5	1.35
36.30	36.40	-	-	-	-	8	1.71	6	1.48
36.40	36.50	-	-	-	-	9	2.01	7	1.52
36.50	36.60	-	-	-	-	8	1.58	5	1.18
36.60	36.70	-	-	-	-	8	1.31	4	1.15
36.70	36.80	-	-	-	-	8	1.75	5	1.39
36.80	36.90	-	-	-	-	8	1.30	3	1.32
36.90	37.00	-	-	-	-	8	1.53	4	1.58
37.00	37.10	-	-	-	-	8	1.29	7	1.20
37.10	37.20	-	-	-	-	8	1.75	5	1.28
37.20	37.30	-	-	-	-	5	1.20	4	1.45
37.30	37.40	-	-	-	-	5	1.33	3	1.51
37.40	37.50	-	-	-	-	5	1.53	3	1.35
37.50	37.60	-	-	-	-	-	-	4	2.42

APPENDIX D

GROUND TEMPERATURE DATA - CENTRAL REGION

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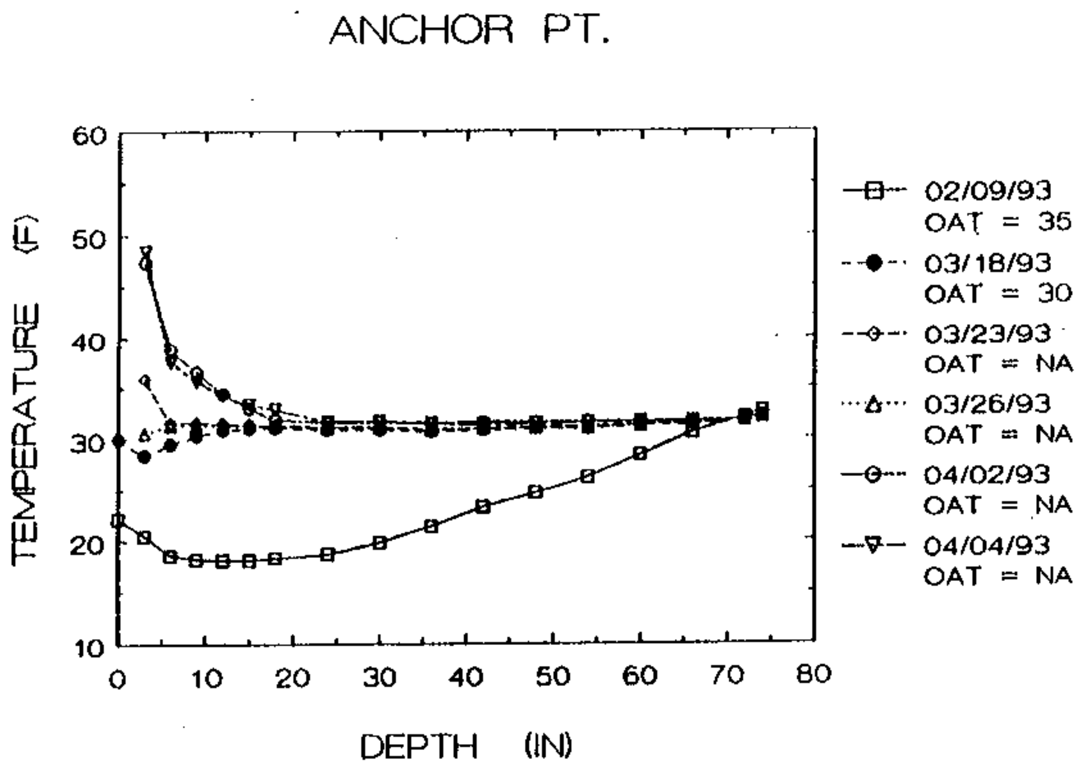
APPENDIX D -- GROUND TEMPERATURE DATA - CENTRAL REGION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03/18/93
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03/26/93
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04/02/93
OAT = NA
04/04/93
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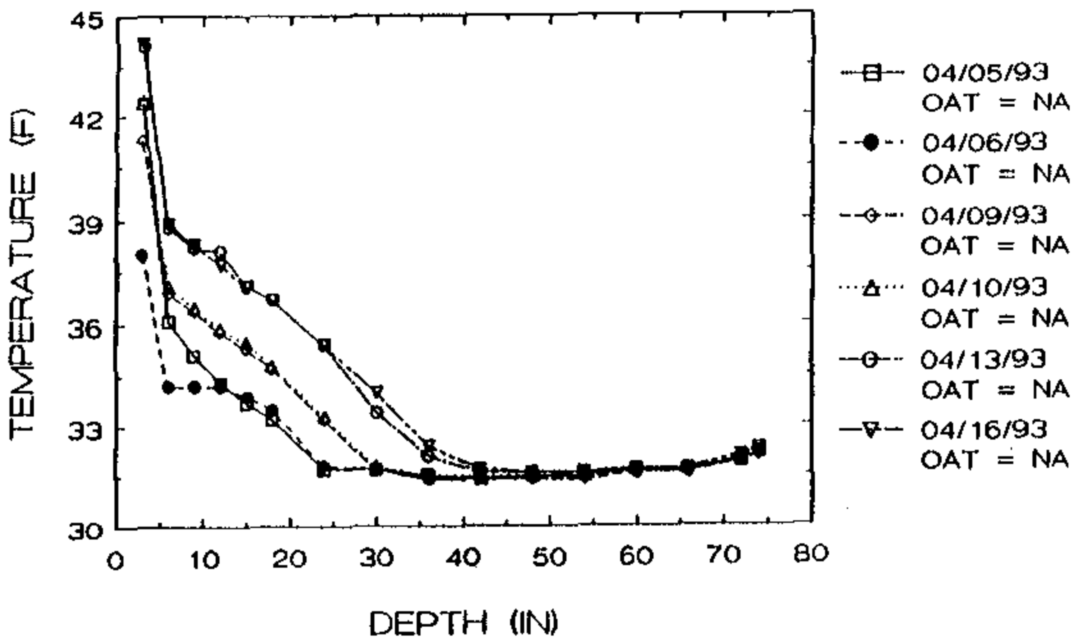

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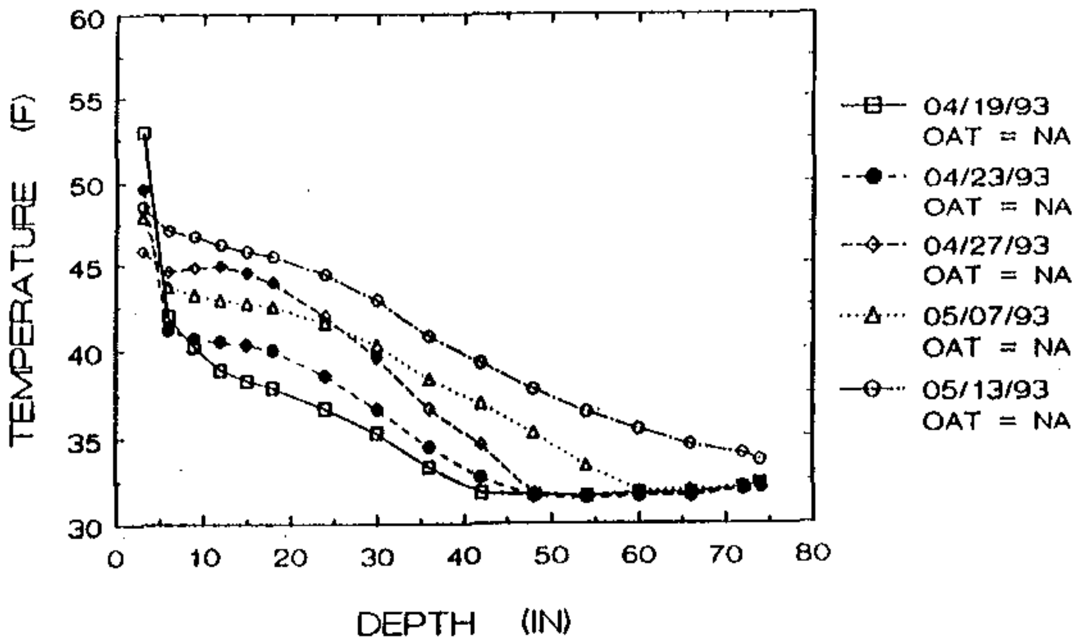

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05/07/93
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05/13/93
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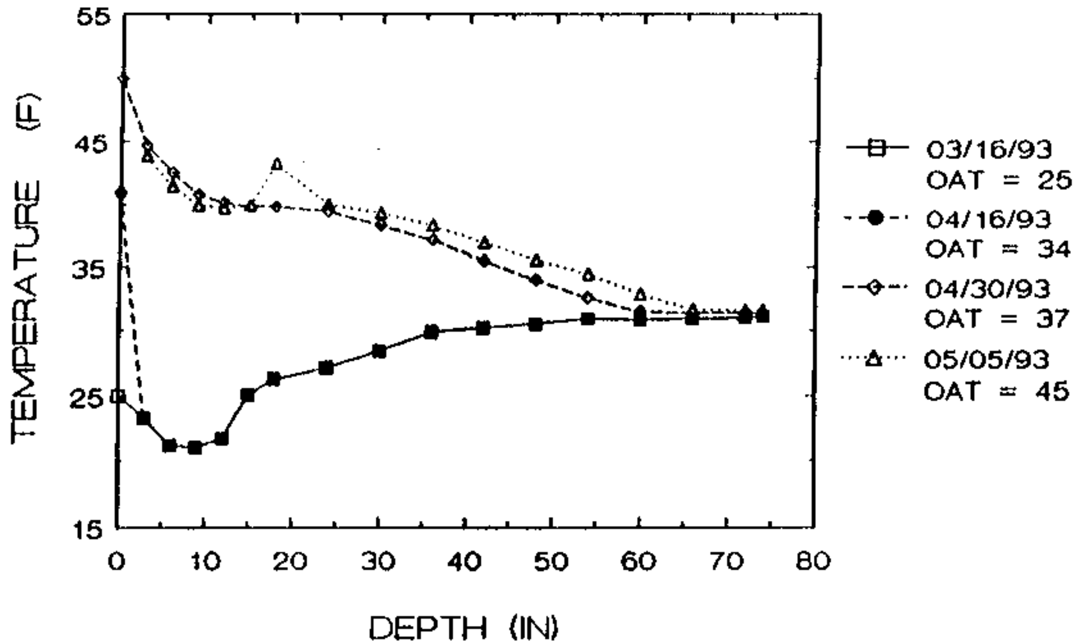
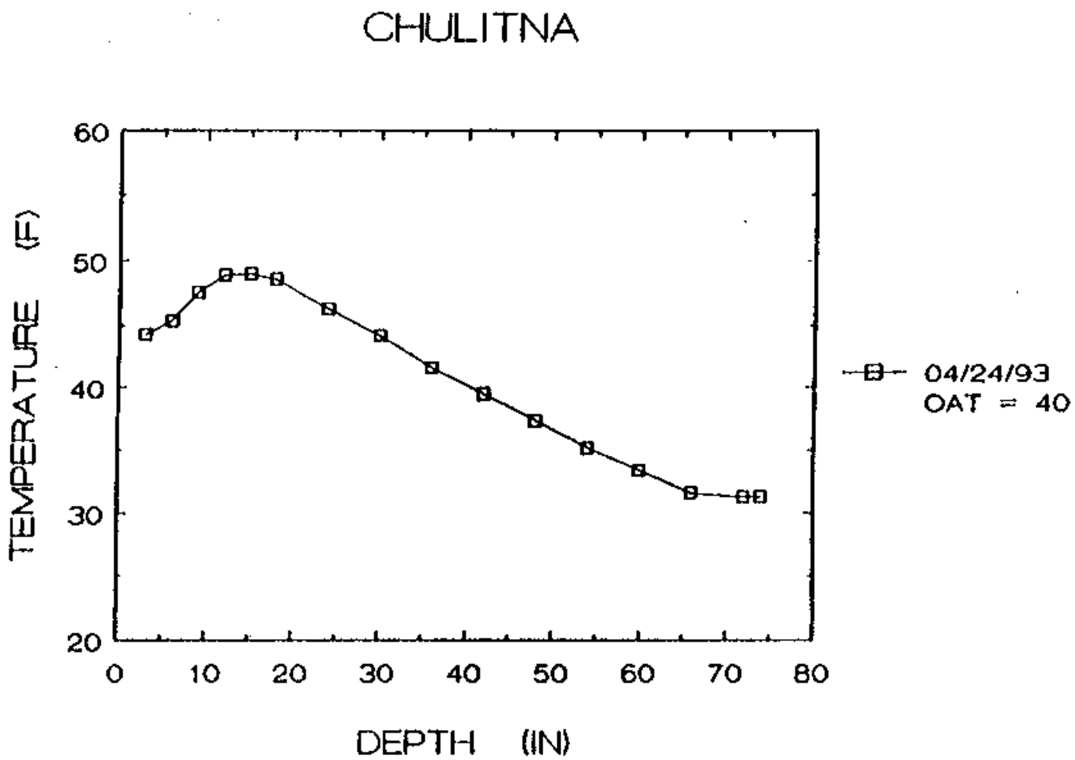
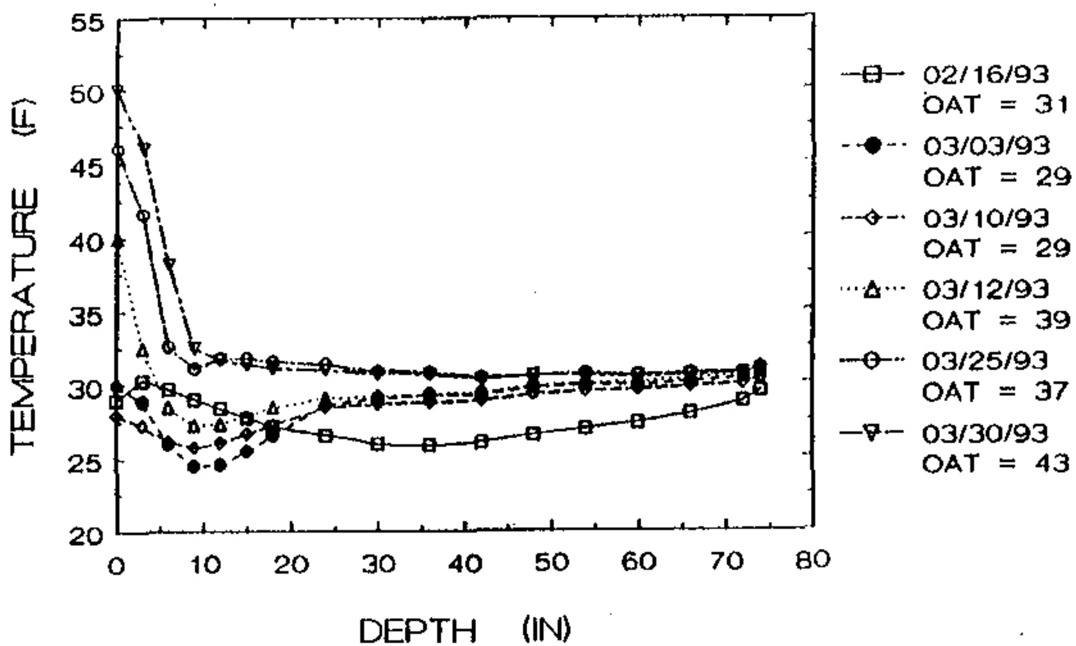
BERTHA CREEK

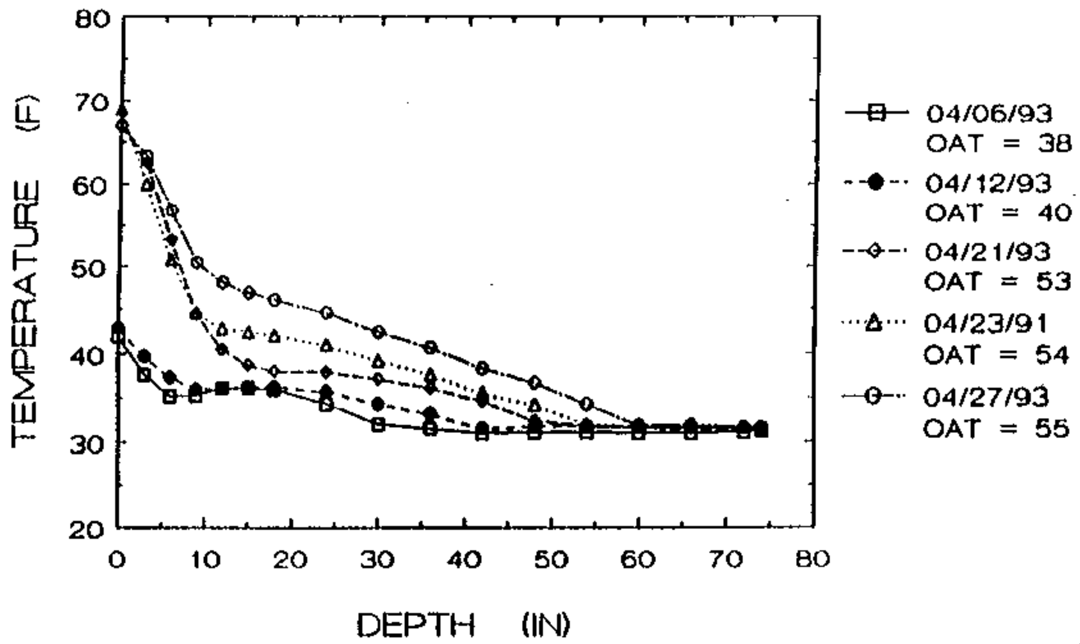



CHULITNA
TEMPERATURE (F)
DEPTH (IN)
60
50
40
30
20
0
10
20
30
40
50
60
70
80
04/24/93
OAT = 40

GLENN HWY.

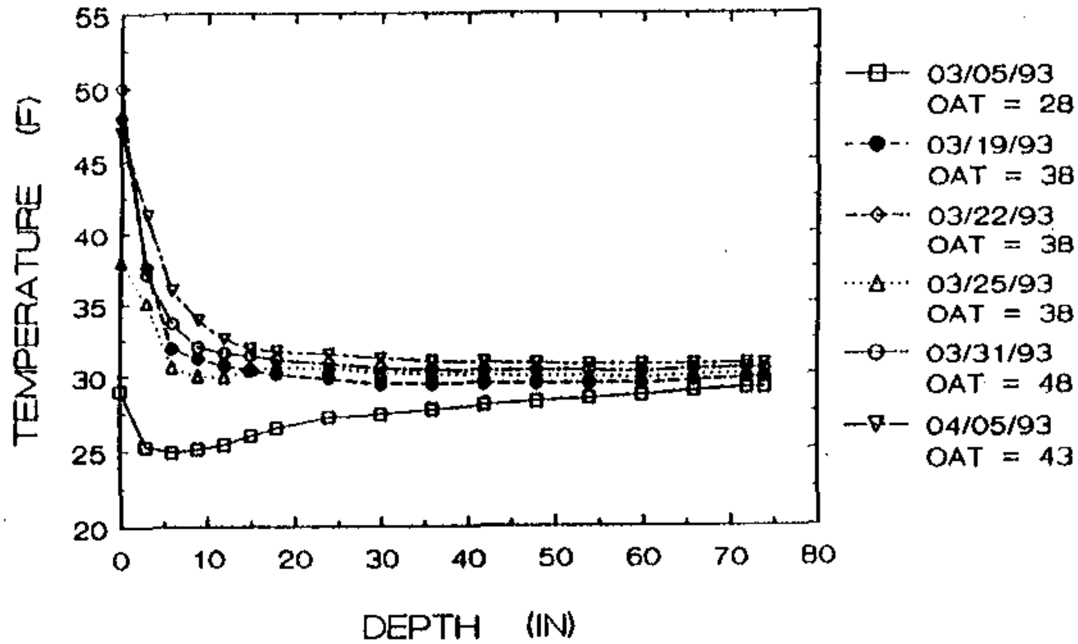

TEMPERATURE (F)
55
50
45
40
35
30
25
20
0
10
20
30
40
50
60
70
80
DEPTH (IN)
02/16/93
OAT = 31
03/03/93
OAT = 29
03/10/93
OAT = 29
03/12/93
OAT = 39
03/25/93
OAT = 37
03/30/93
OAT = 43

GLENN HWY.



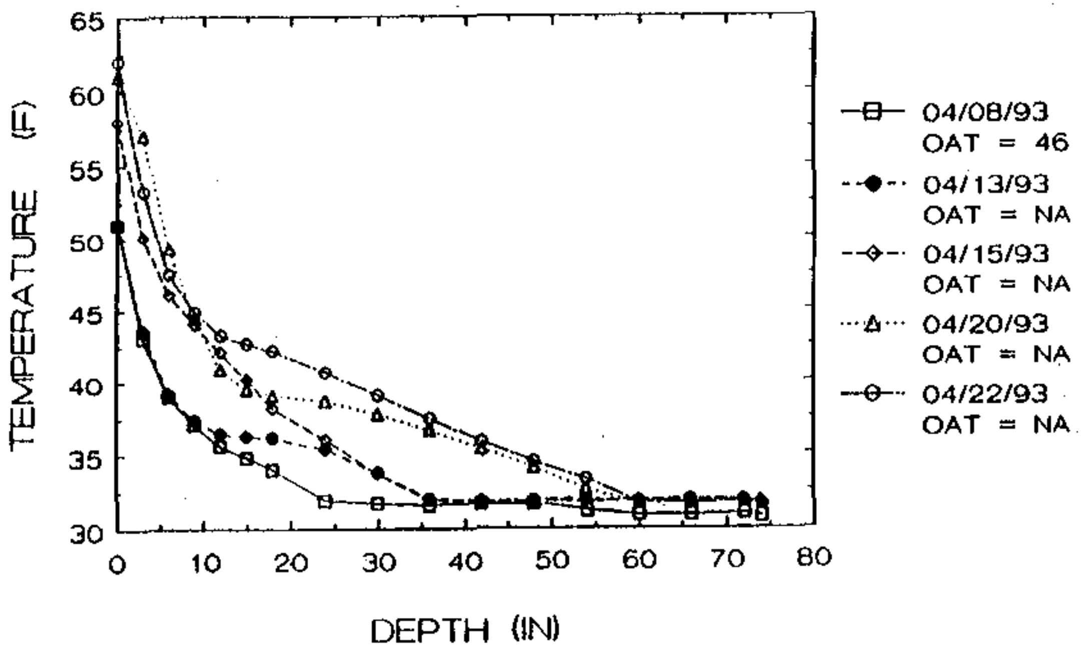
80
70
60
50
40
30
20
TEMPERATURE (F)
0
10
20
30
40
50
60
70
80
DEPTH (IN)
04/06/93
OAT = 38
04/12/93
OAT = 40
04/21/93
OAT = 53
04/23/91
OAT = 54
04/27/93
OAT = 55

GLENN MP53



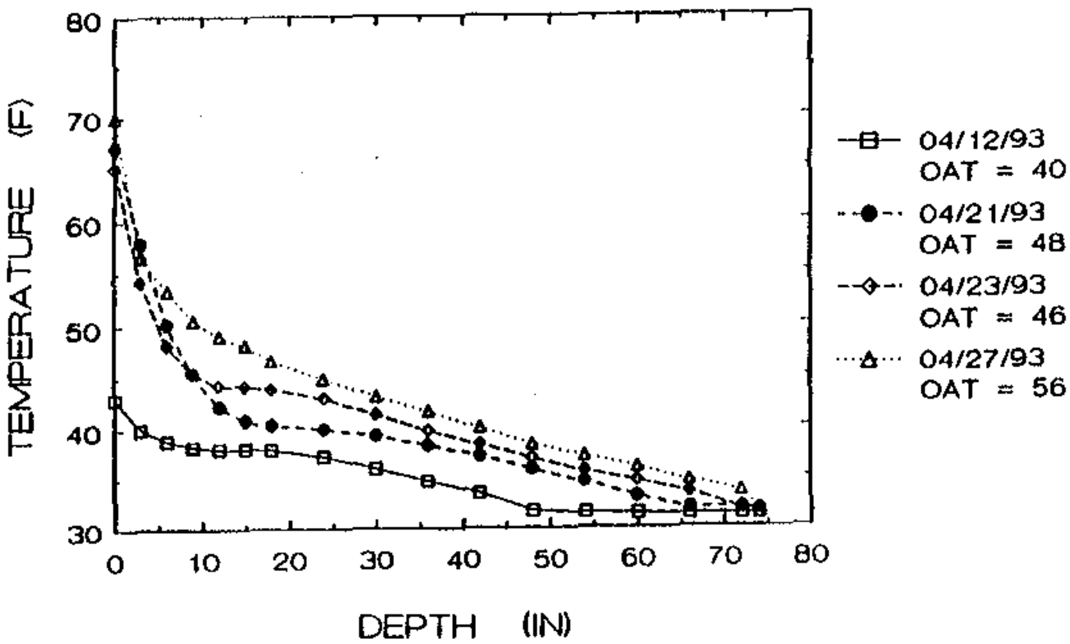
TEMPERATURE (F)
DEPTH (IN)
03/05/93
OAT = 28
03/19/93
OAT = 38
03/22/93
OAT = 38
03/25/93
OAT = 38
03/31/93
OAT = 48
04/05/93
OAT = 43

GLENN MP53

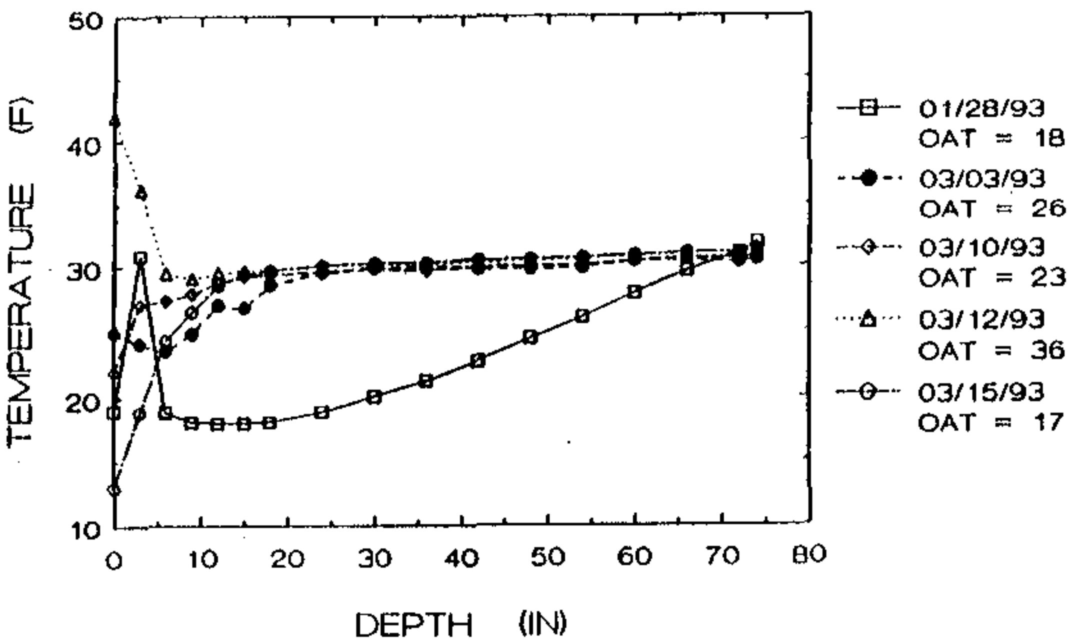


TEMPERATURE (F)
65
60
55
50
45
40
35
30
0
10
20
30
40
50
60
70
80
DEPTH (IN)
04/08/93
OAT = 46
04/13/93
OAT = NA
04/15/93
OAT = NA
04/20/93
OAT = NA
04/22/93
OAT = NA

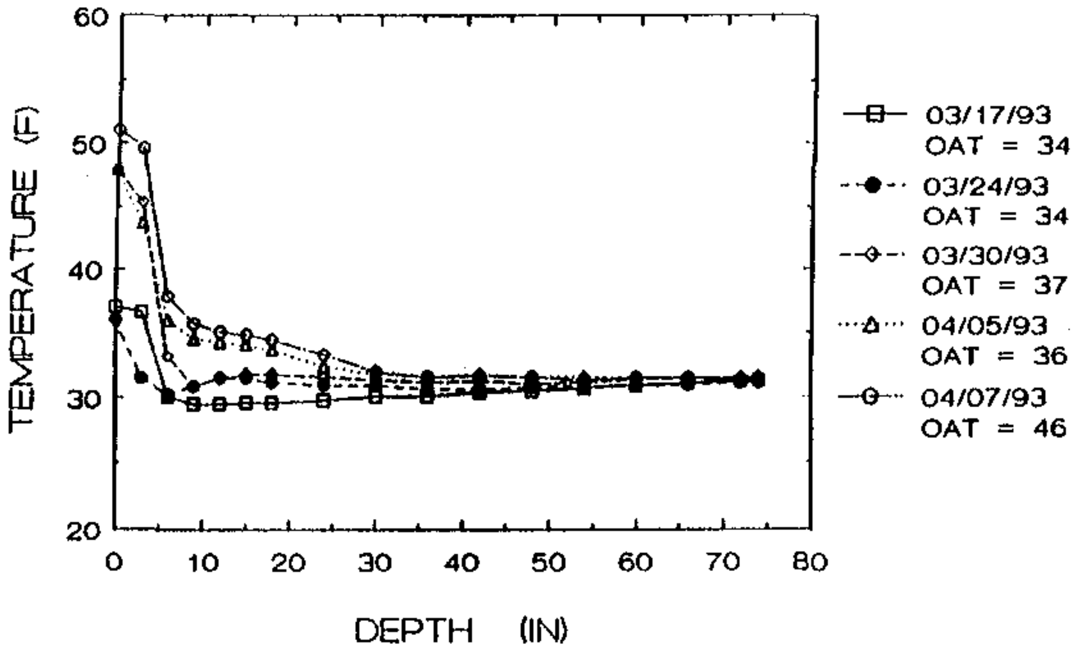
GLENN/BRAGAW



NEW SEWARD

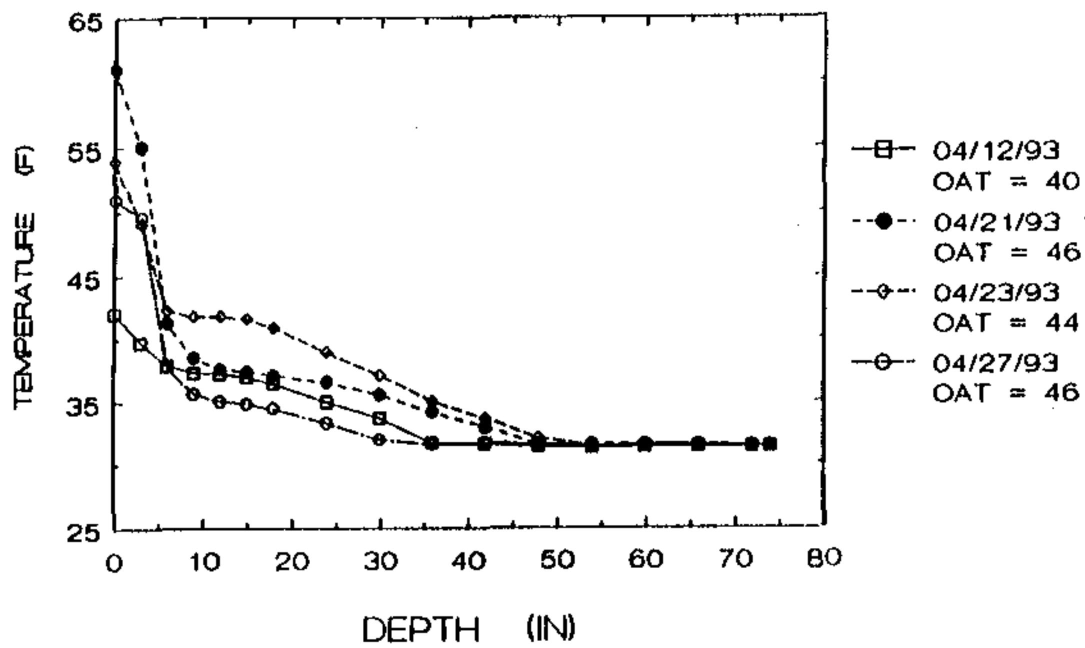

TEMPERATURE (F)
50
40
30
20
10
0
10
20
30
40
50
60
70
80
DEPTH (IN)
01/28/93
OAT = 18
03/03/93
OAT = 26
03/10/93
OAT = 23
03/12/93
OAT = 36
03/15/93
OAT = 17

NEW SEWARD



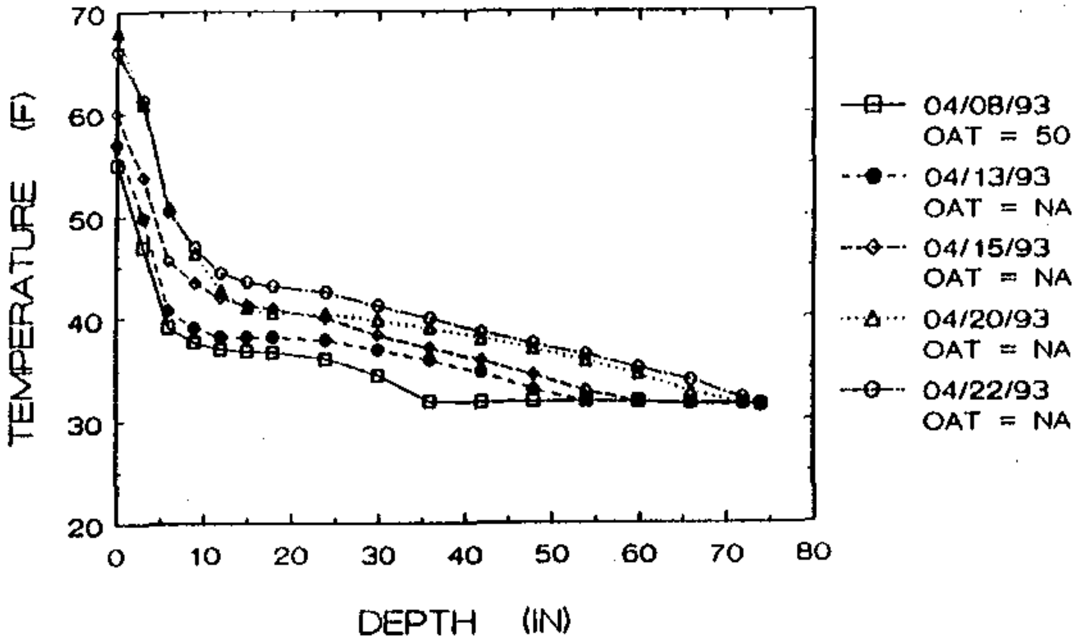
TEMPERATURE (F)
60
50
40
30
20
0
10
20
30
40
50
60
70
80
DEPTH (IN)
03/17/93
OAT = 34
03/24/93
OAT = 34
03/30/93
OAT = 37
04/05/93
OAT = 36
04/07/93
OAT = 46

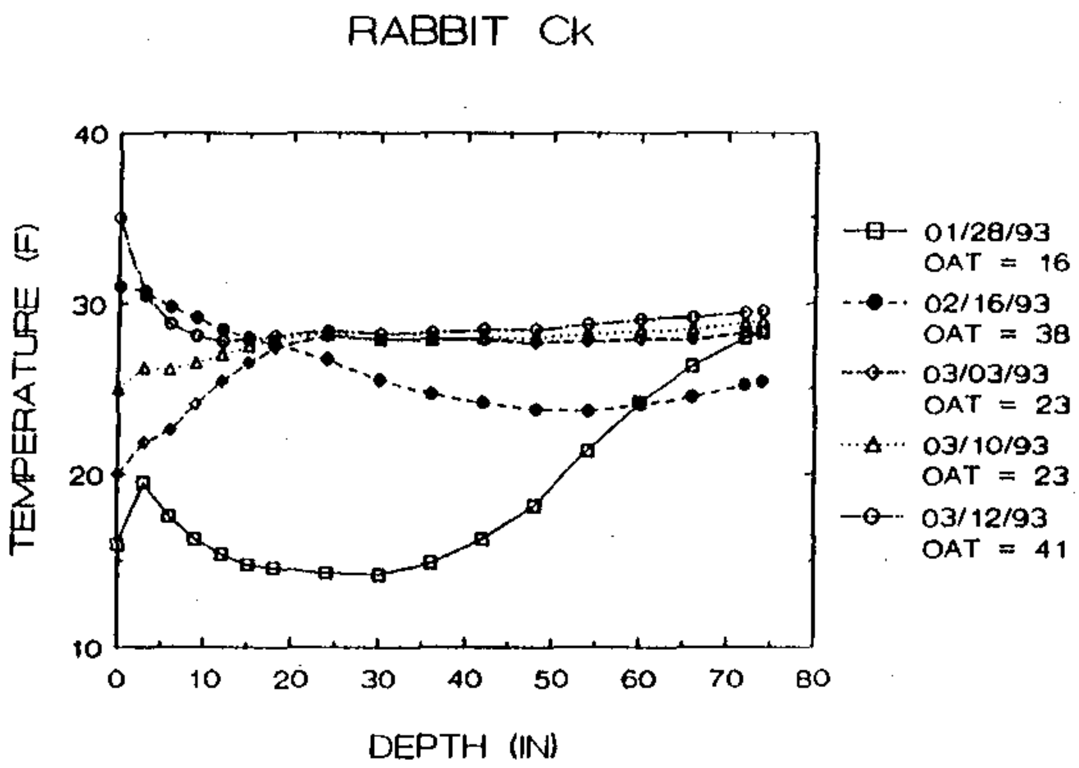
NEW SEWARD

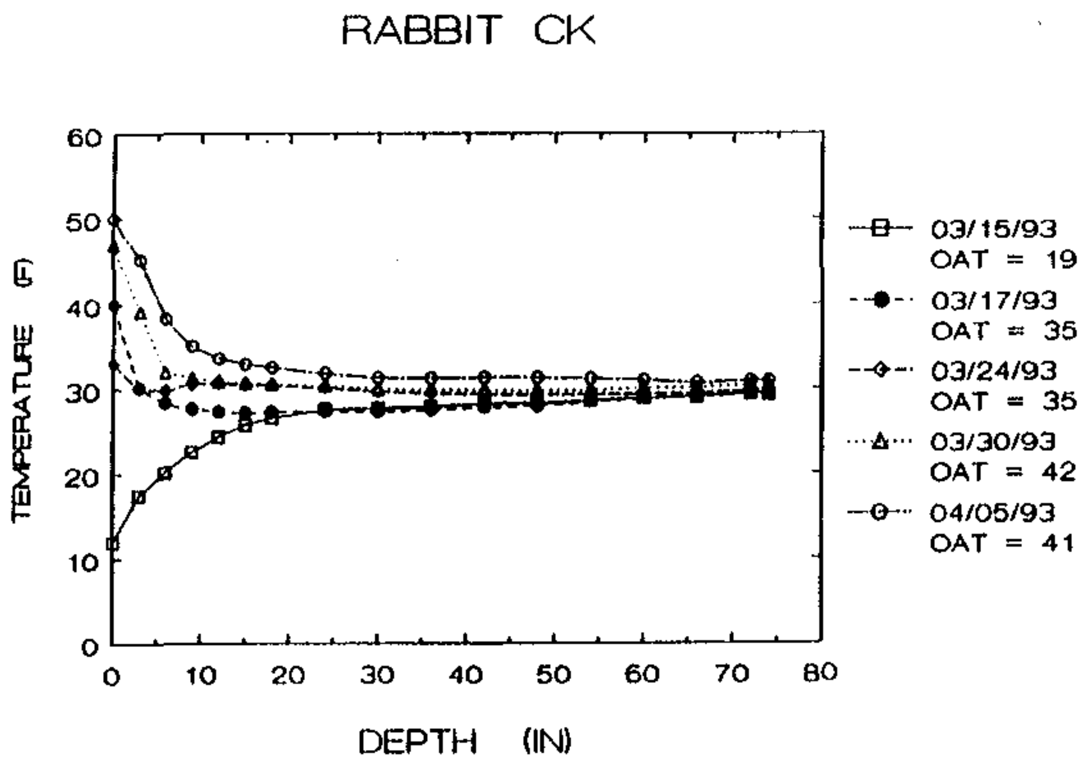


65
55
45
35
25
0
10
20
30
40
50
60
70
80
TEMPERATURE (F)
DEPTH (IN)
04/12/93
OAT = 40
04/21/93
OAT = 46
04/23/93
OAT = 44
04/27/93
OAT = 46

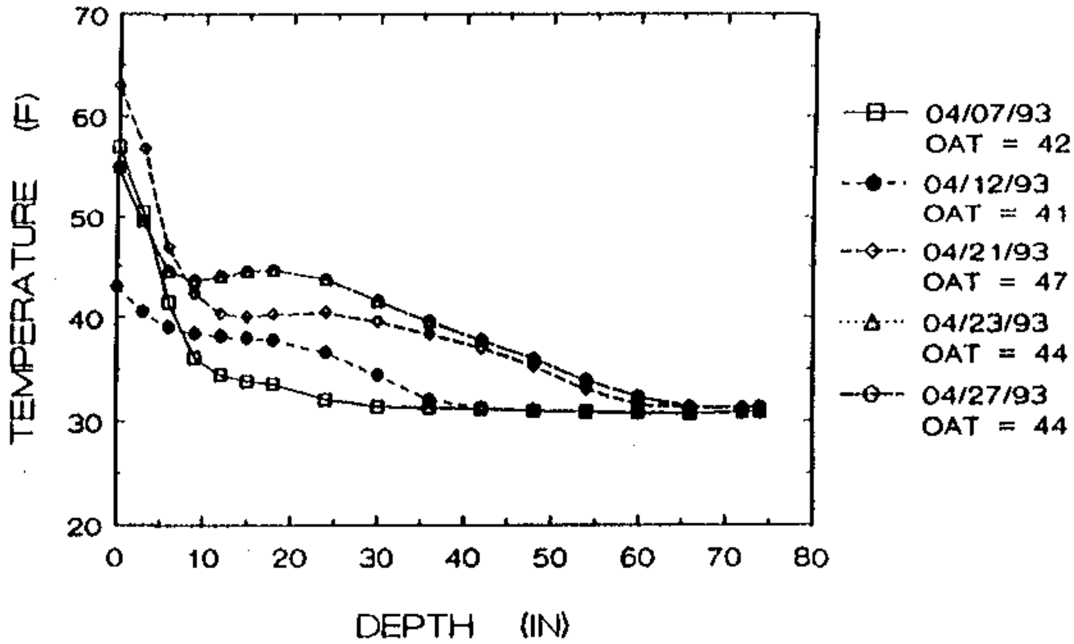
PALMER/WASILLA




RABBIT Ck
TEMPERATURE (F)
DEPTH (IN)
40
30
20
10
0
10
20
30
40
50
60
70
80
01/28/93
OAT = 16
02/16/93
OAT = 38
03/03/93
OAT = 23
03/10/93
OAT = 23
03/12/93
OAT = 41

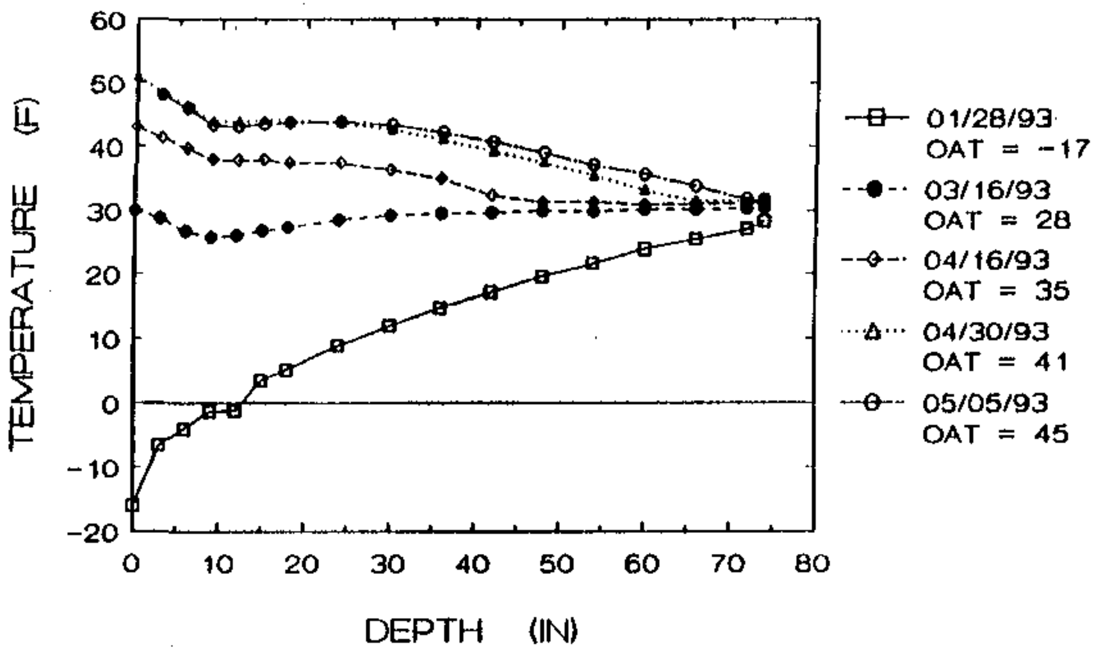

RABBIT CK
TEMPERATURE (F)
DEPTH (IN)
03/15/93
OAT = 19
03/17/93
OAT = 35
03/24/93
OAT = 35
03/30/93
OAT = 42
04/05/93
OAT = 41

RABBIT CK

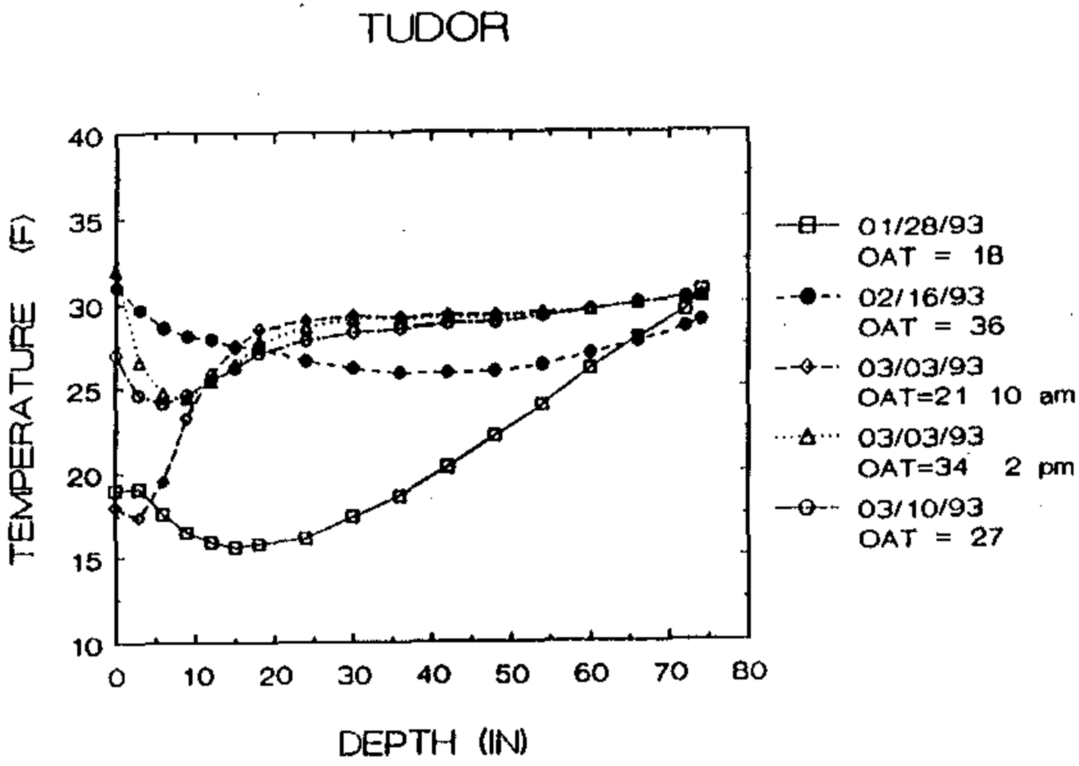


TEMPERATURE (F)
70
60
50
40
30
20
0
10
20
30
40
50
60
70
80
DEPTH (IN)
04/07/93
OAT = 42
04/12/93
OAT = 41
04/21/93
OAT = 47
04/23/93
OAT = 44
04/27/93
OAT = 44

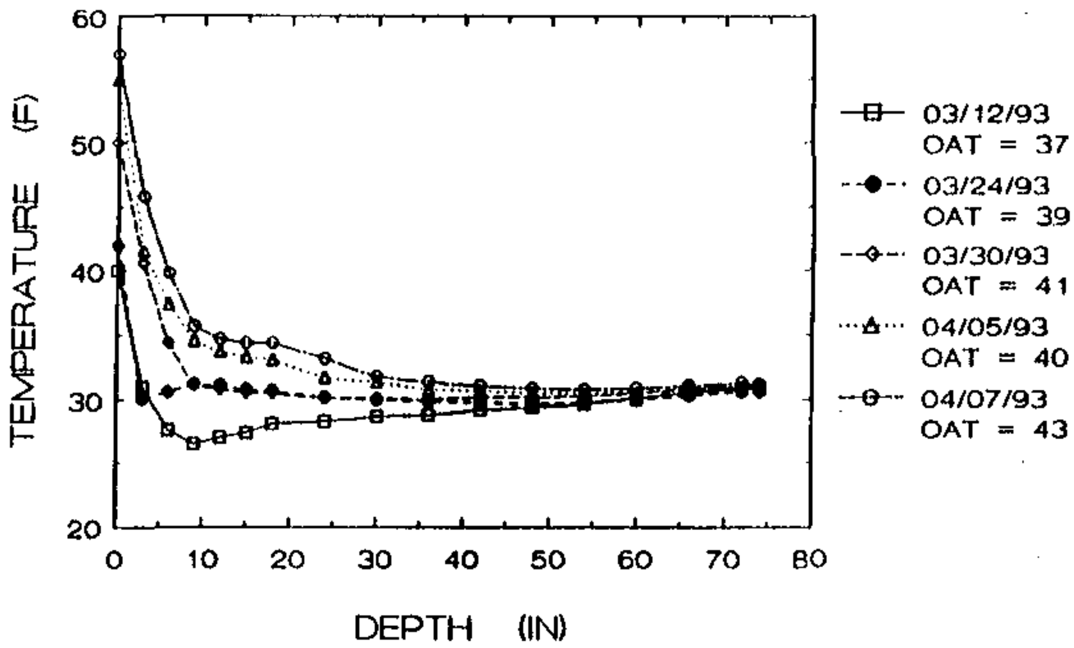
SUMMIT LAKE



TEMPERATURE (F)
60
50
40
30
20
10
0
-10
-20
0
10
20
30
40
50
60
70
80
DEPTH (IN)
01/28/93
OAT = -17
03/16/93
OAT = 28
04/16/93
OAT = 35
04/30/93
OAT = 41
05/05/93
OAT = 45

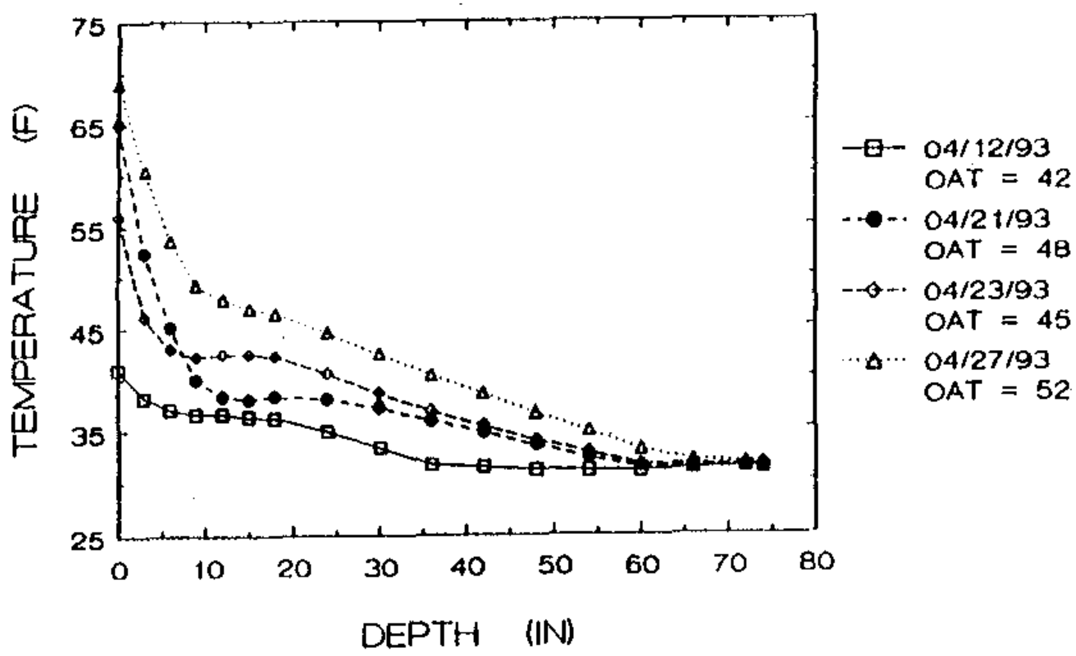

TUDOR
TEMPERATURE (F)
DEPTH (IN)
40
35
30
25
20
15
10
0
10
20
30
40
50
60
70
80
01/28/93
OAT = 18
02/16/93
OAT = 36
03/03/93
OAT=21 10 am
03/03/93
OAT=34 2 pm
03/10/93
OAT = 27

TUDOR

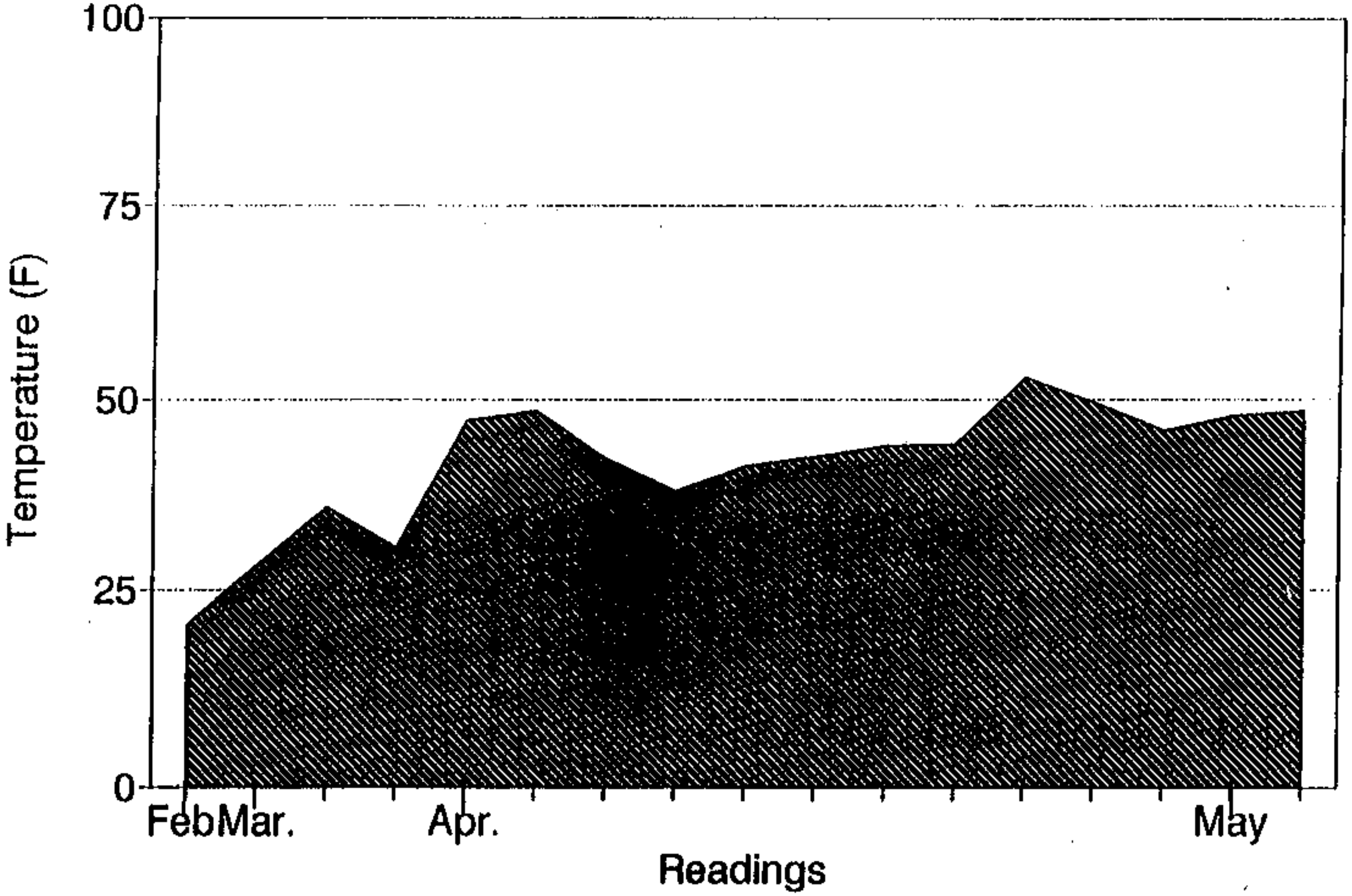


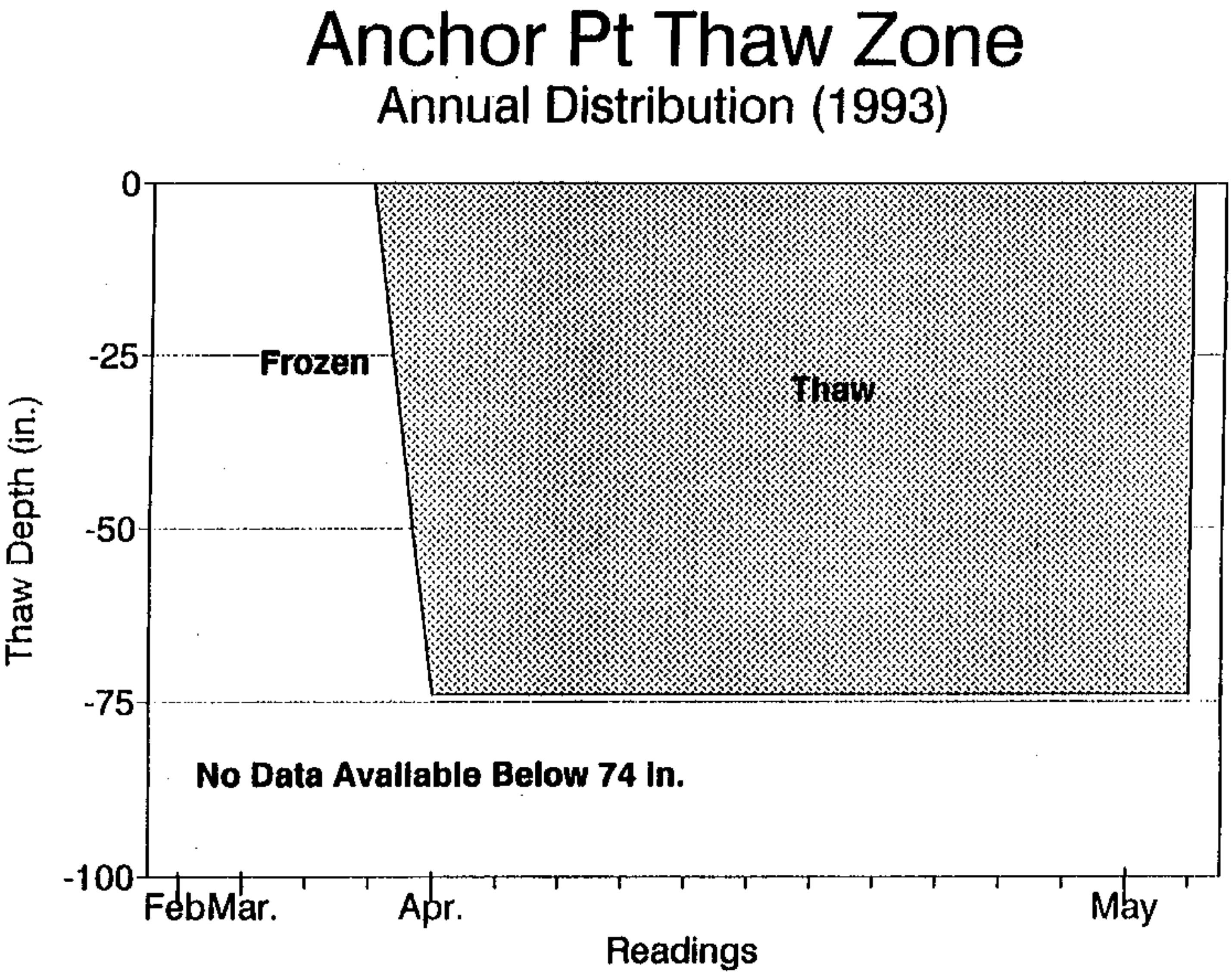
60
50
40
30
20
TEMPERATURE (F)
0
10
20
30
40
50
60
70
80
DEPTH (IN)
03/12/93
OAT = 37
03/24/93
OAT = 39
03/30/93
OAT = 41
04/05/93
OAT = 40
04/07/93
OAT = 43

TUDOR


TEMPERATURE (F)
75
65
55
45
35
25
0
10
20
30
40
50
60
70
80
DEPTH (IN)
04/12/93
OAT = 42
04/21/93
OAT = 48
04/23/93
OAT = 45
04/27/93
OAT = 52

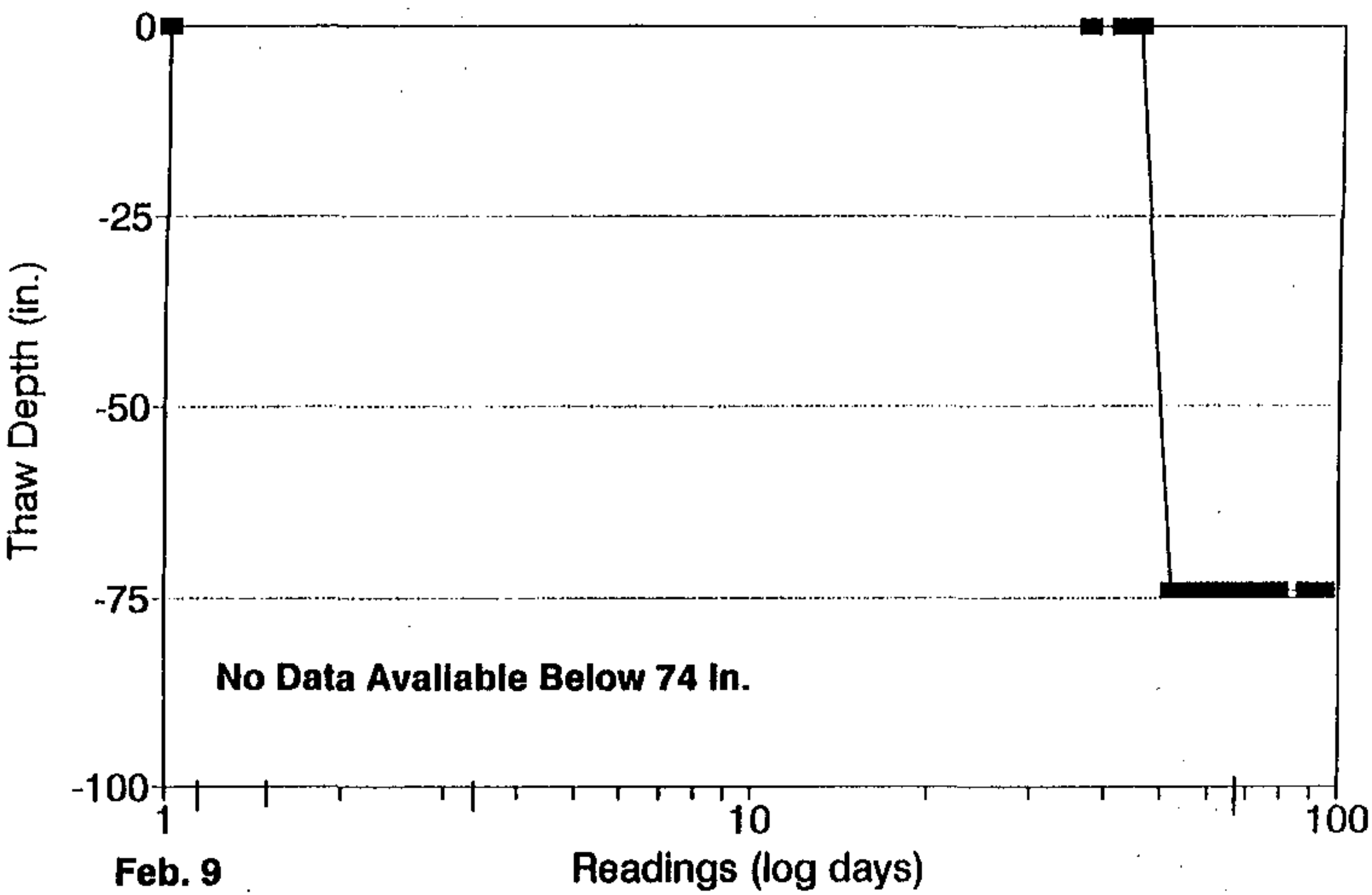
Anchor Pt Temperature
Annual Distribution (1993)

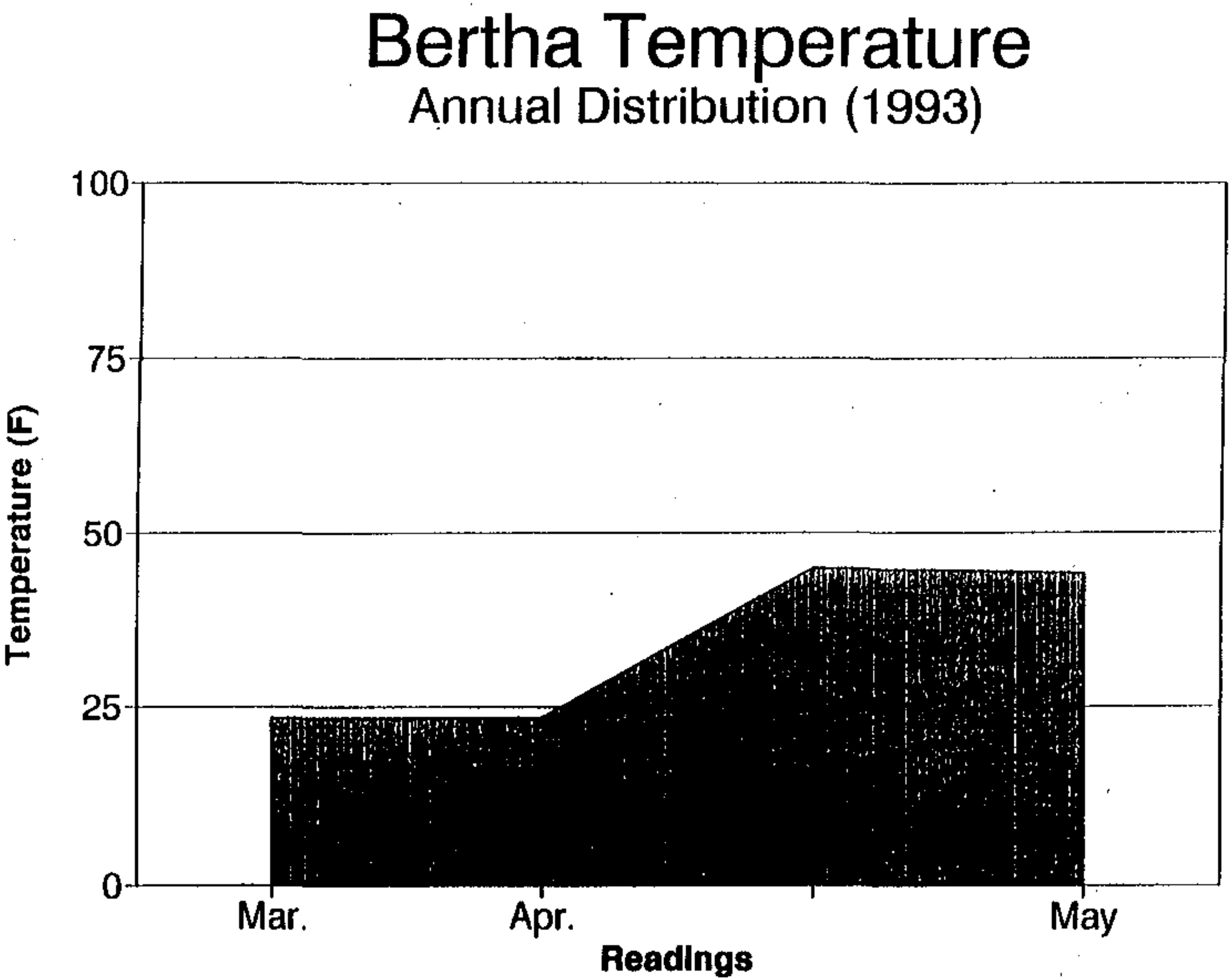



Anchor Pt Thaw Zone
Annual Distribution (1993)
0
-25
-50
-75
-100
Thaw Depth (in.)
Frozen
Thaw
No Data Available Below 74 In.
Feb
Mar.
Apr.
May
Readings

Anchor Pt Thaw Zone

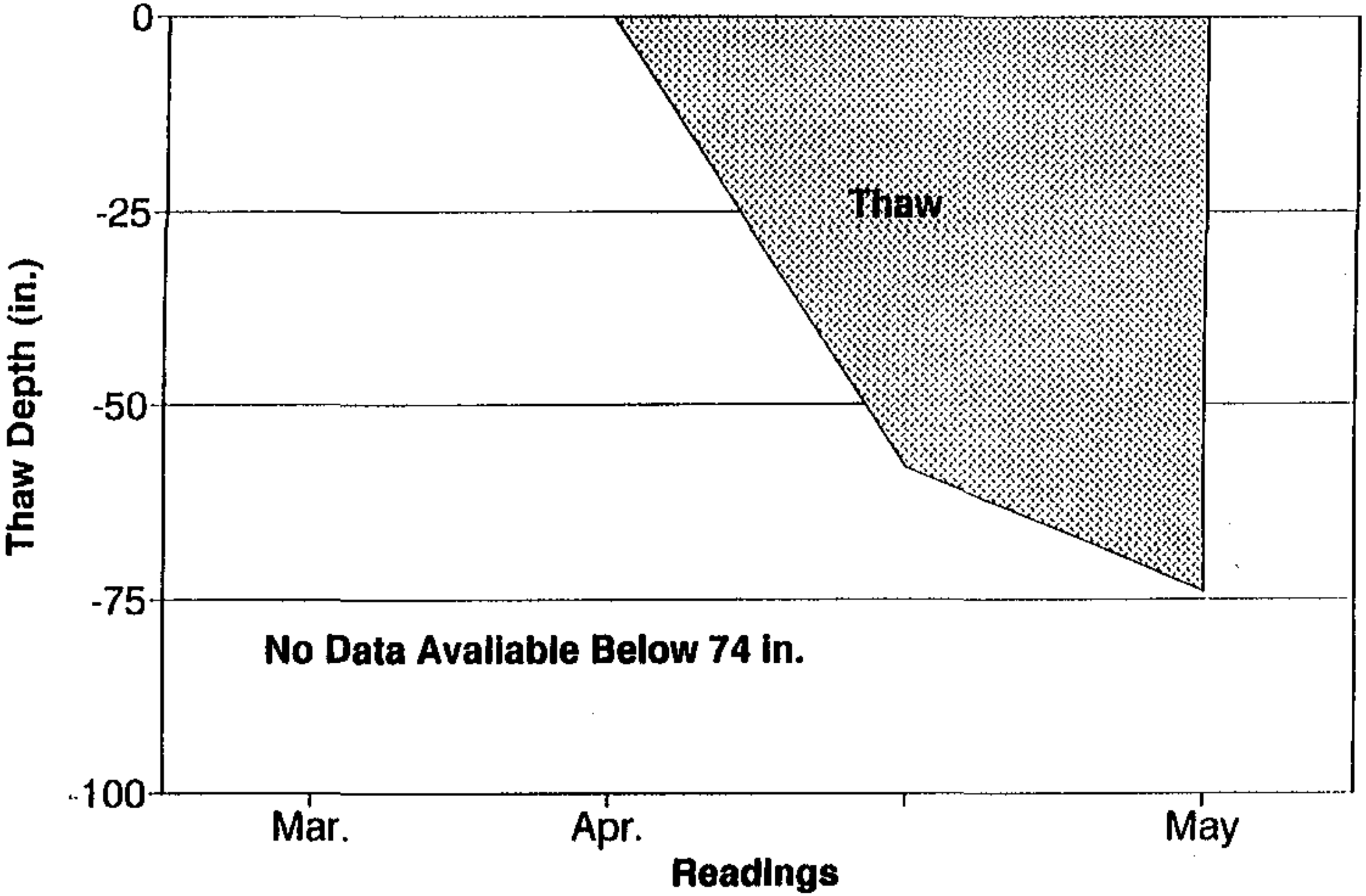
Annual Distribution (1993)

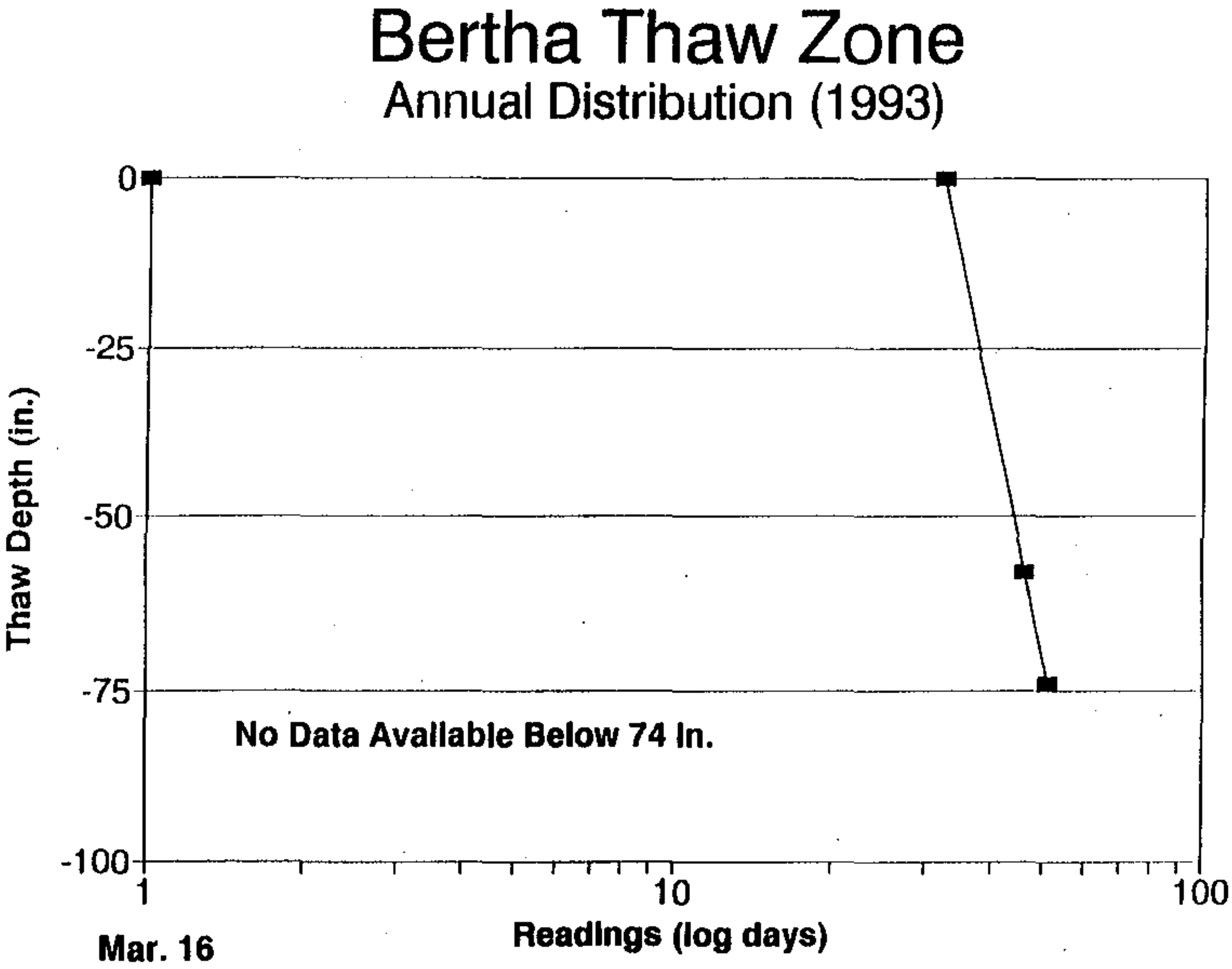



Bertha Temperature
Annual Distribution (1993)
Temperature (F)
100
75
50
25
0
Mar.
Apr.
May
Readings

Bertha Thaw Zone

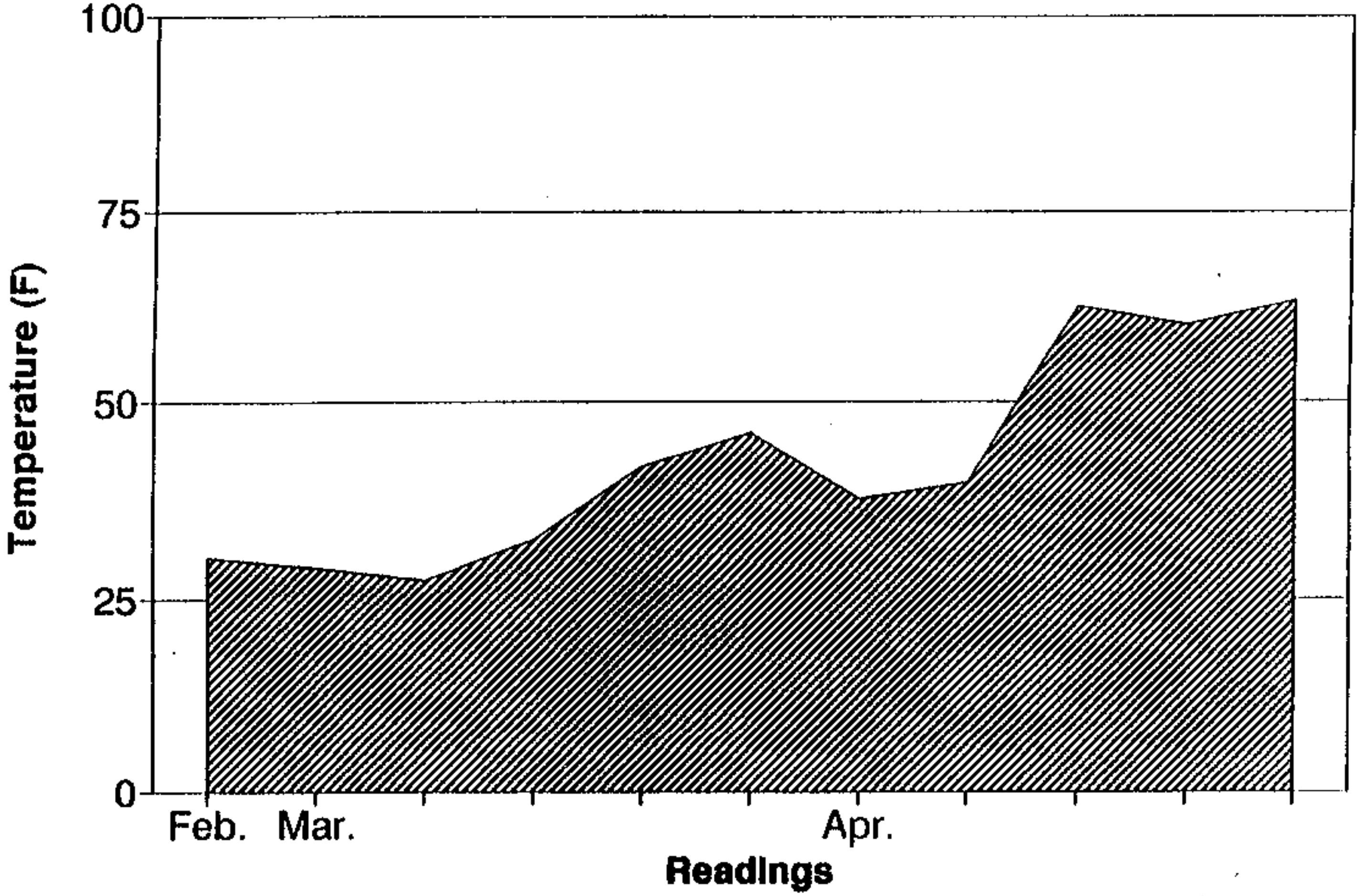
Annual Distribution (1993)

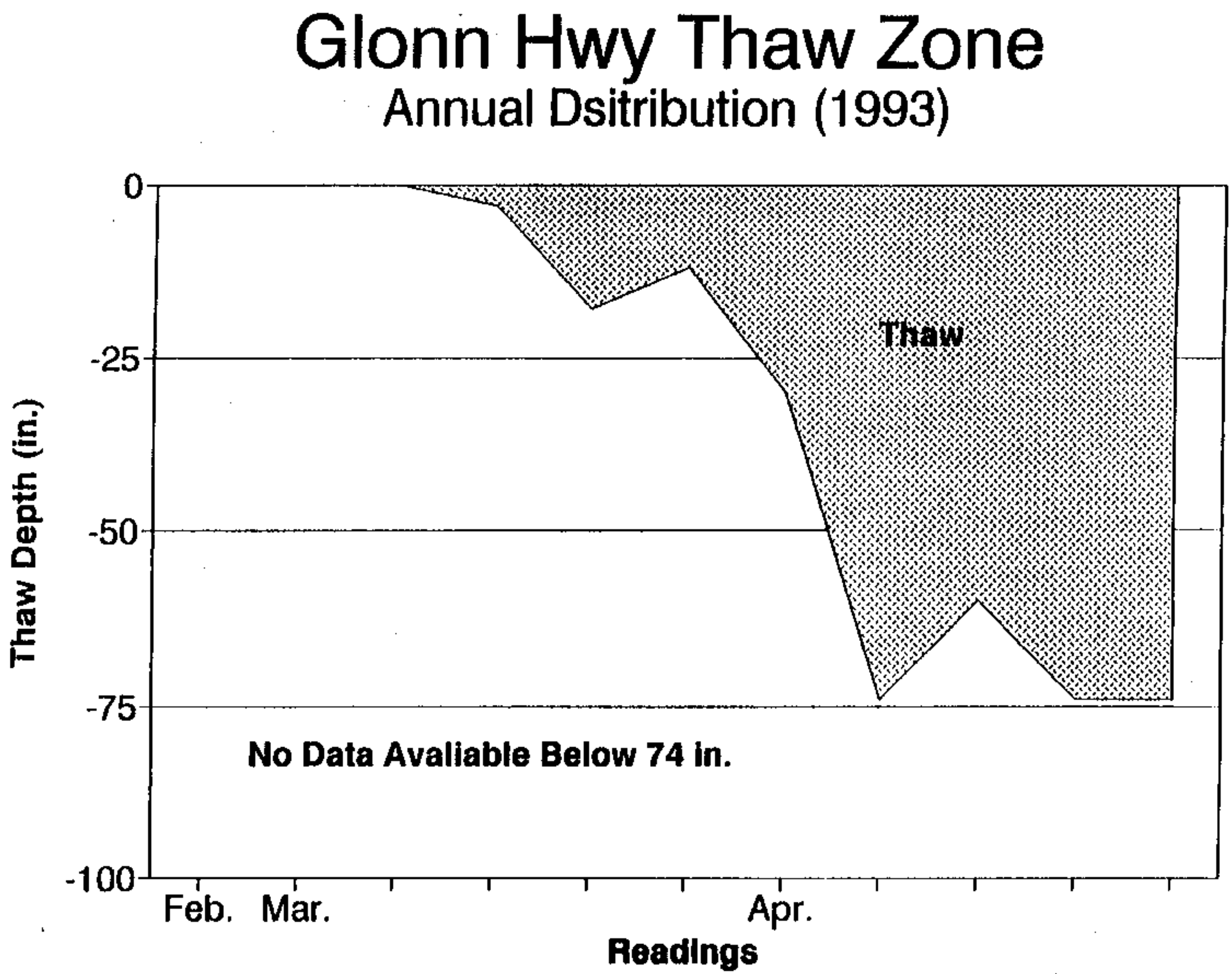



Bertha Thaw Zone
Annual Distribution (1993)
Thaw Depth (in.)
0
-25
-50
-75
-100
No Data Available Below 74 In.
1
10
100
Readings (log days)
Mar. 16

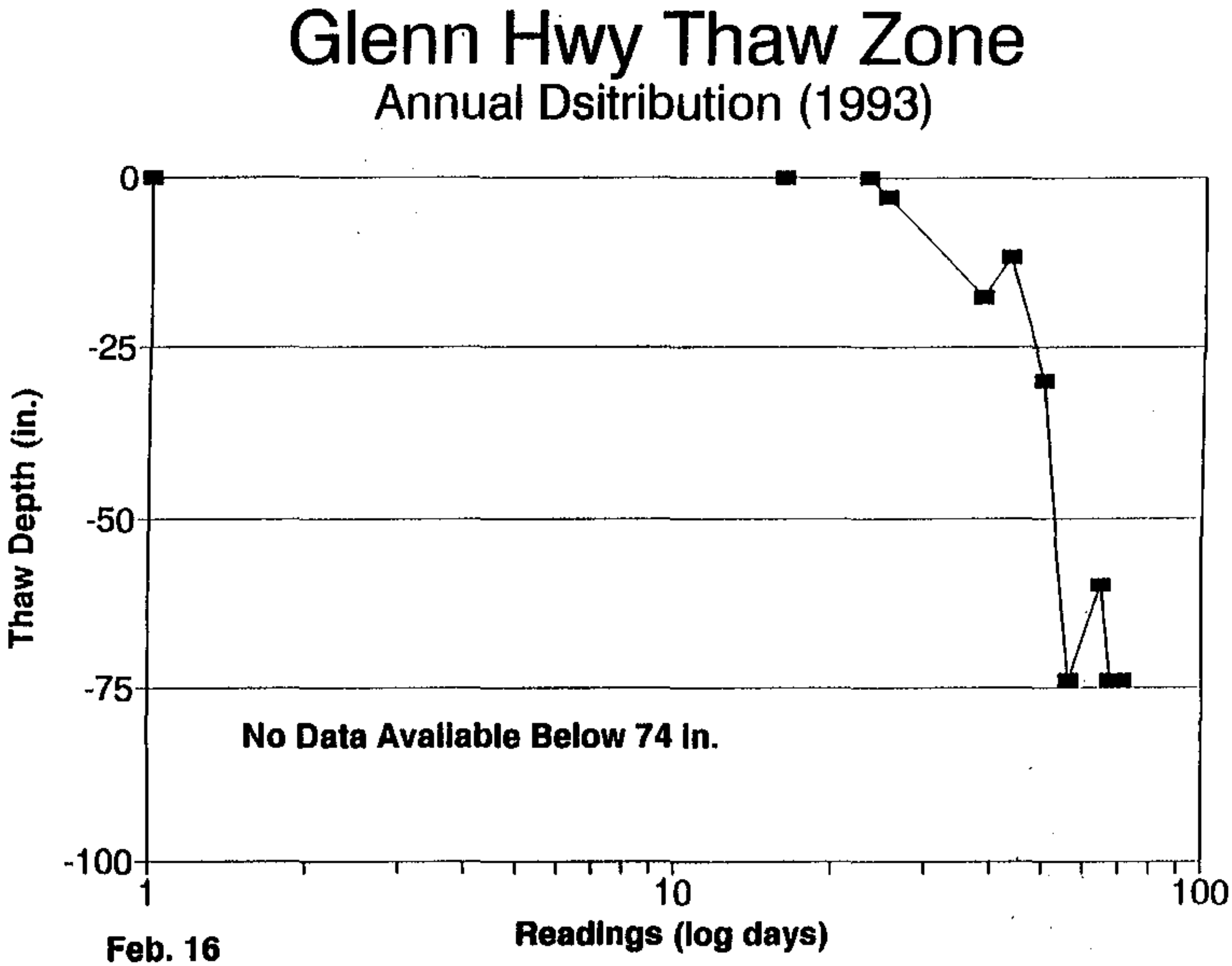
Glenn Hwy Temperature

Annual Dsitribution (1993)



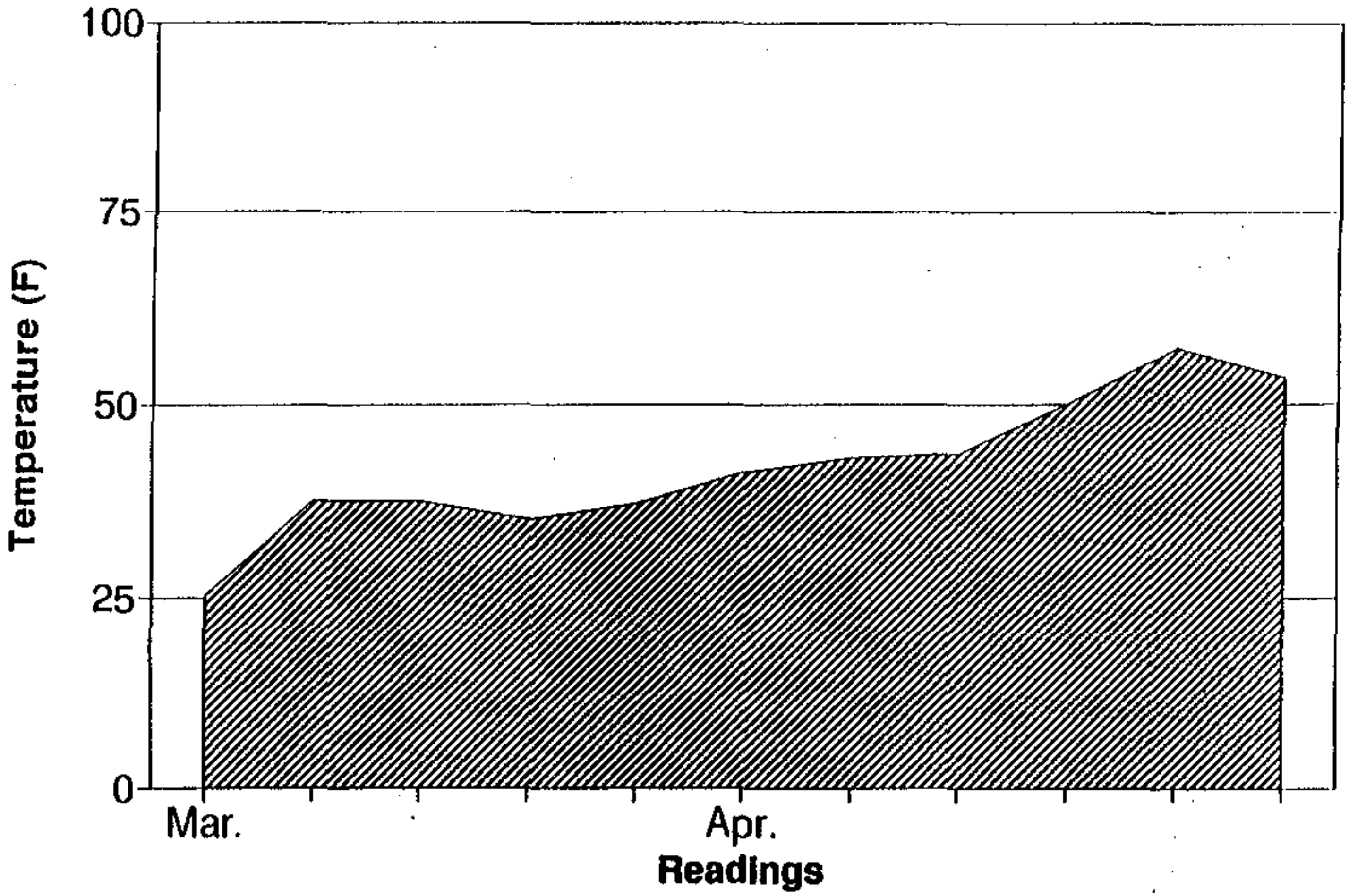


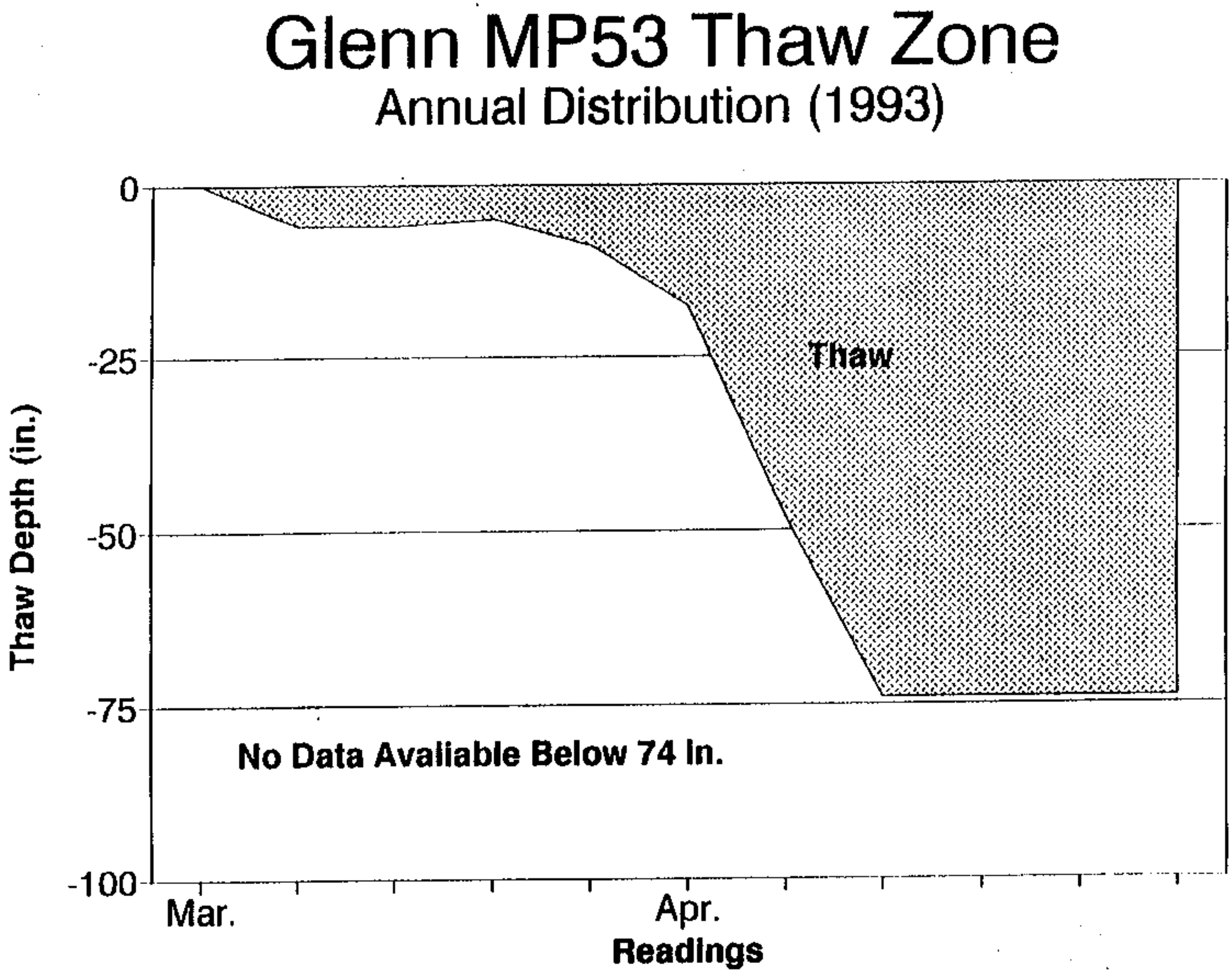
Glonn Hwy Thaw Zone
Annual Dsitribution (1993)
0
-25
-50
-75
-100
Thaw
No Data Avaliable Below 74 in.
Thaw Depth (in.)
Feb.
Mar.
Apr.
Readings


Glenn Hwy Thaw Zone
Annual Dsitribution (1993)
0
-25
-50
-75
-100
1
10
100
Thaw Depth (in.)
Readings (log days)
Feb. 16
No Data Available Below 74 in.

Glenn MP53 Temperature

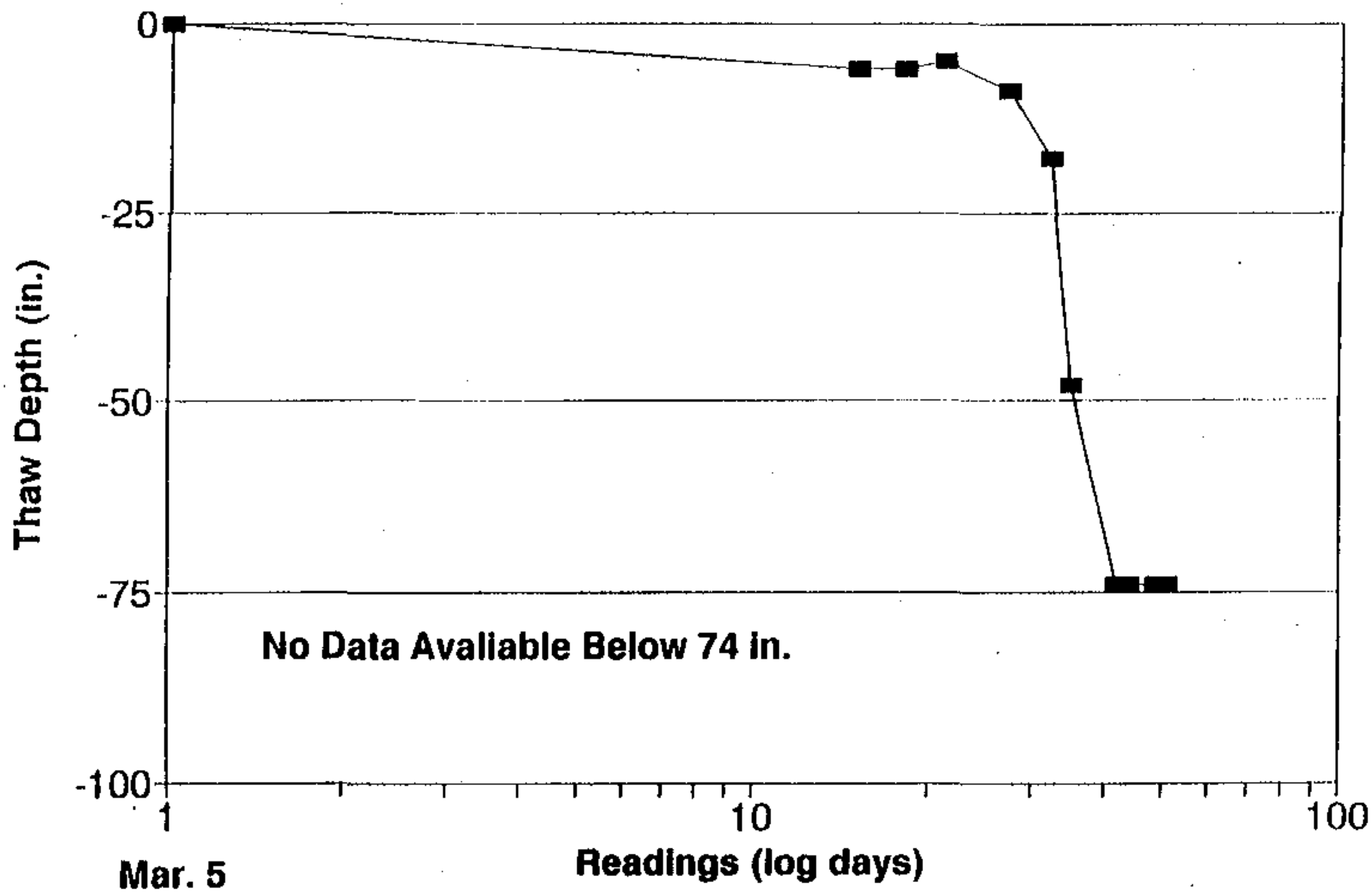
Annual Distribution (1993)

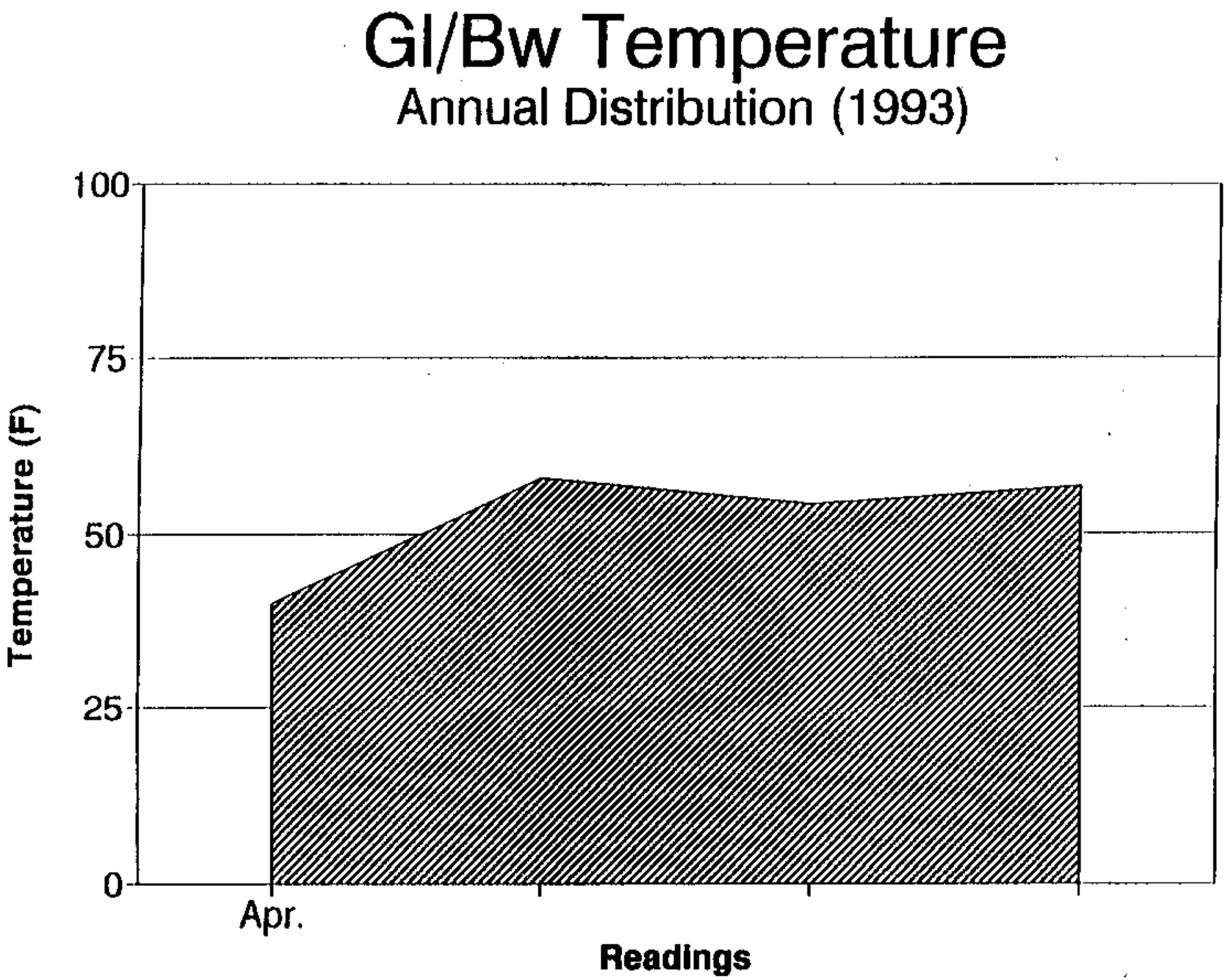



Glenn MP53 Thaw Zone
Annual Distribution (1993)
0
-25
-50
-75
-100
Thaw Depth (in.)
Thaw
No Data Avaliable Below 74 In.
Mar.
Apr.
Readings

Glenn MP53 Thaw Zone

Annual Distribution (1993)

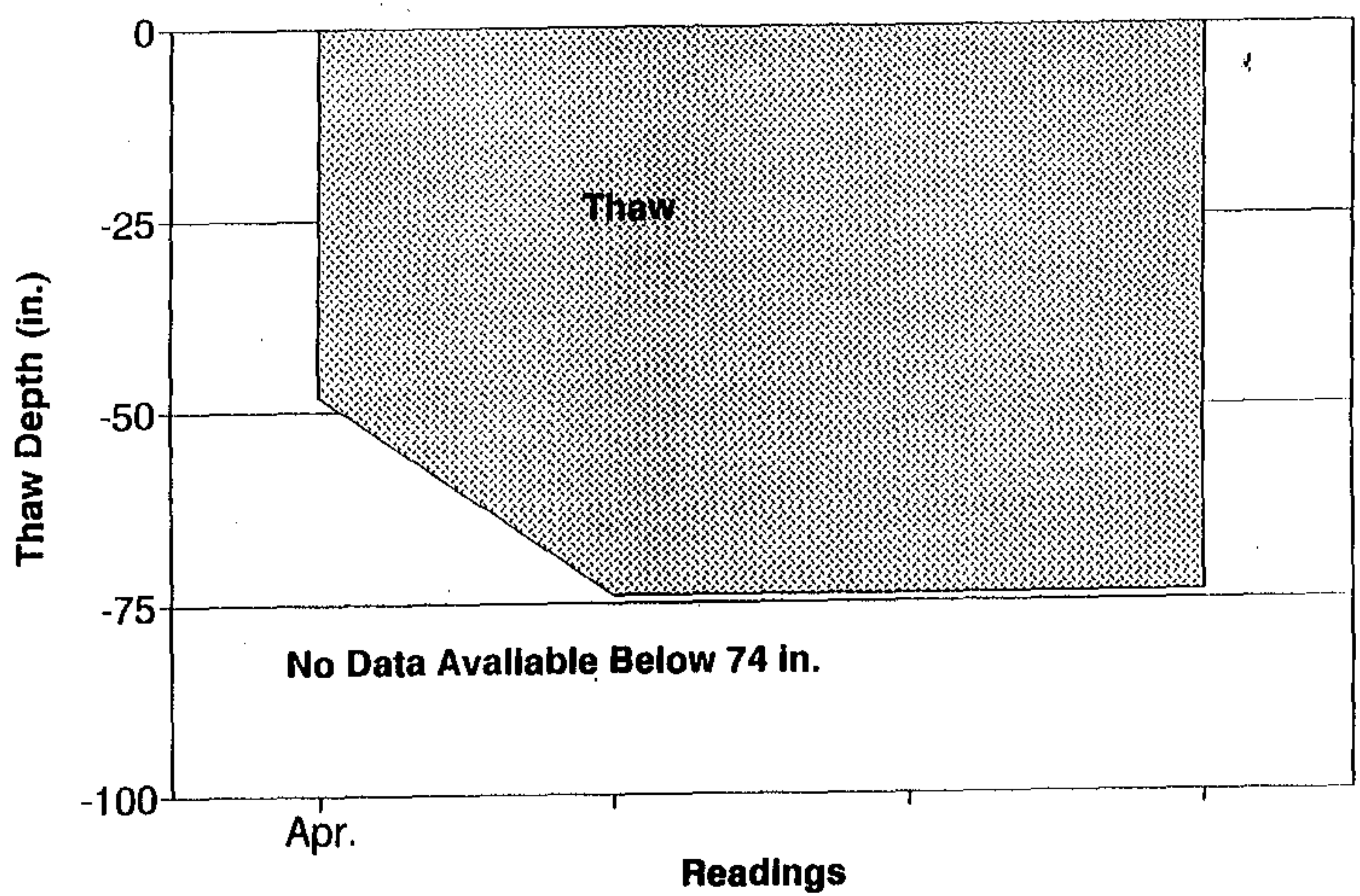




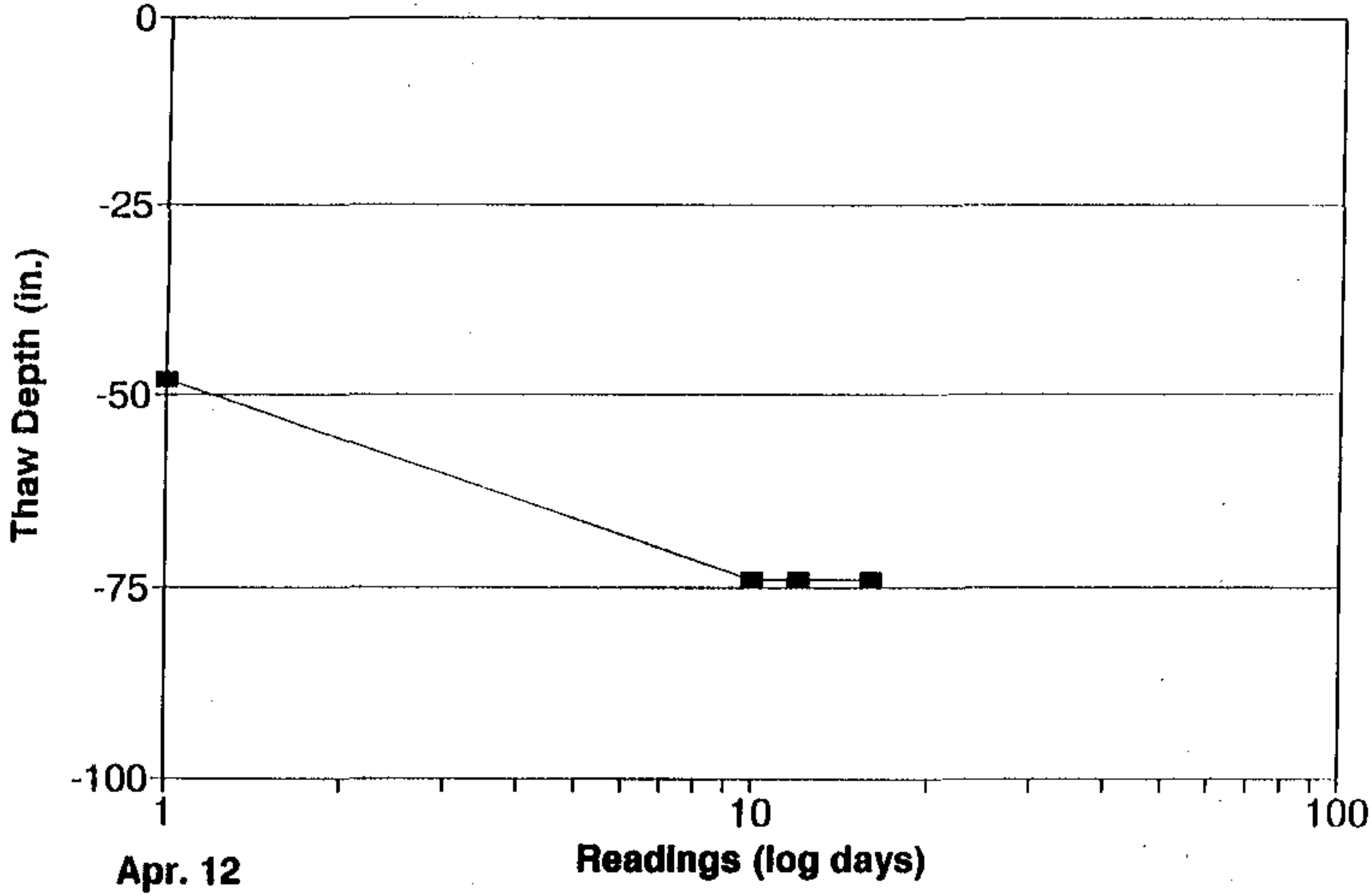
Gl/Bw Temperature
Annual Distribution (1993)
100
75
50
25
0
Temperature (F)
Apr.
Readings

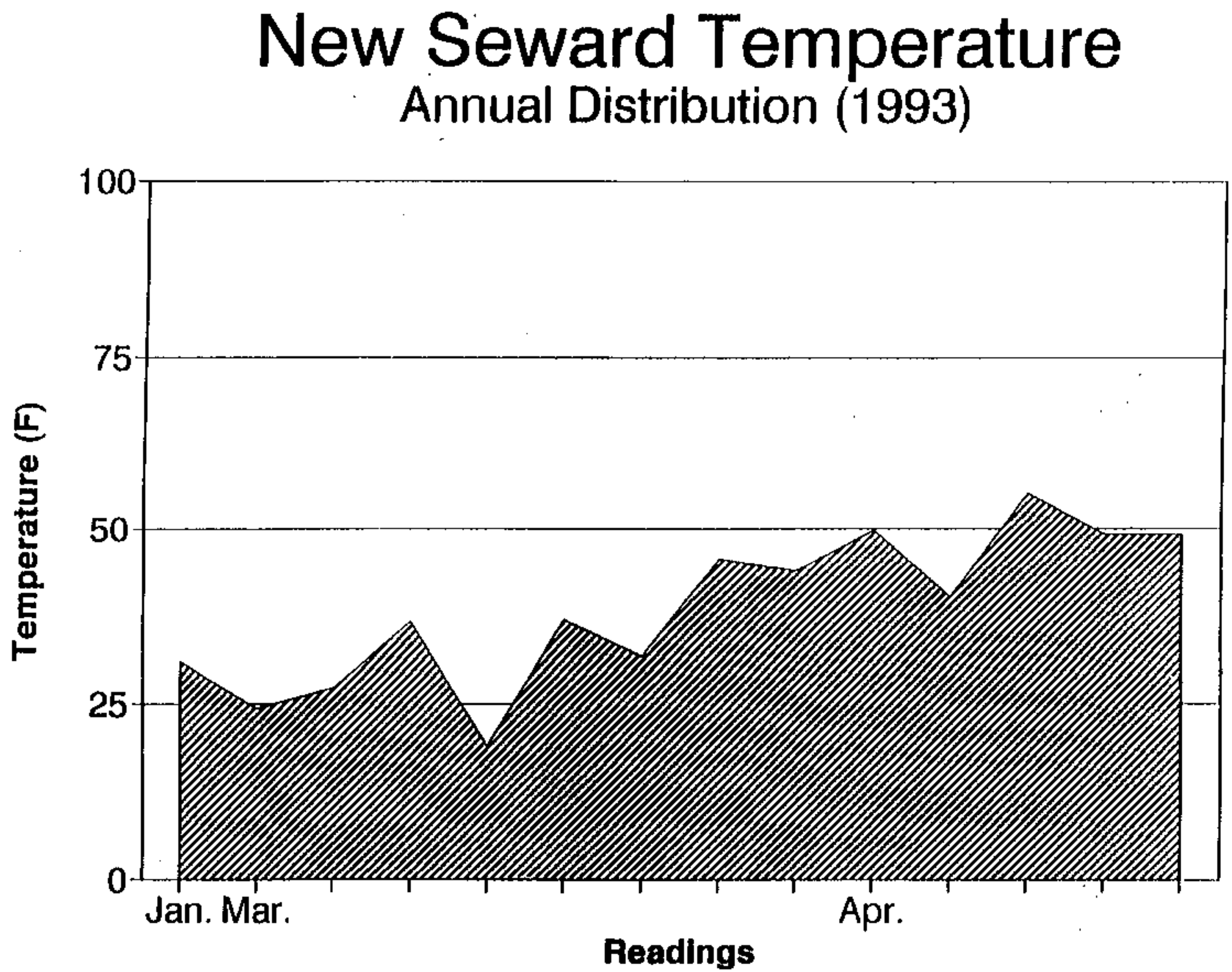
Gl/Bw Thaw Zone

Annual Distribution (1993)



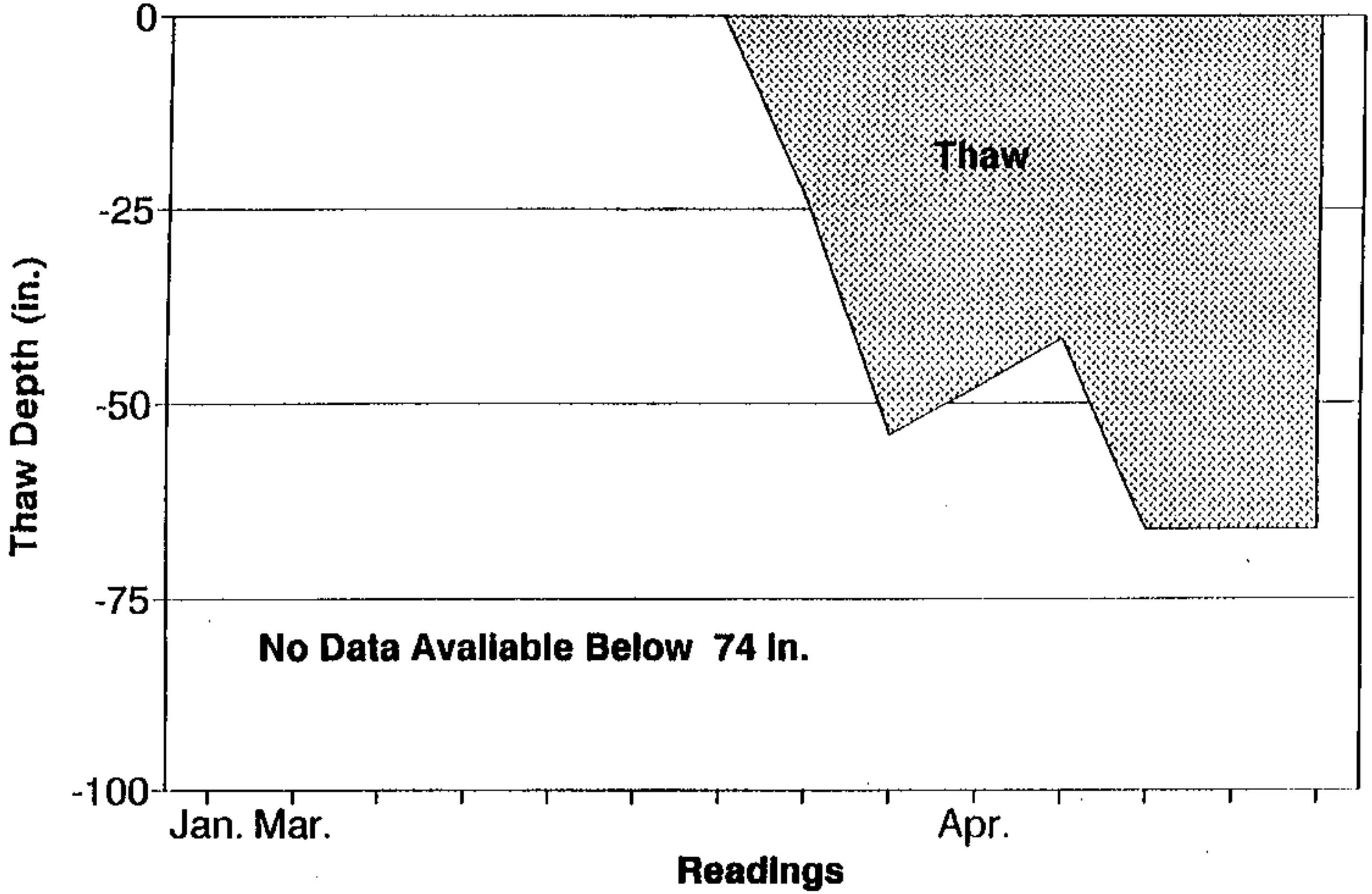
Gl/Bw Thaw Zone
Annual Distribution (1993)

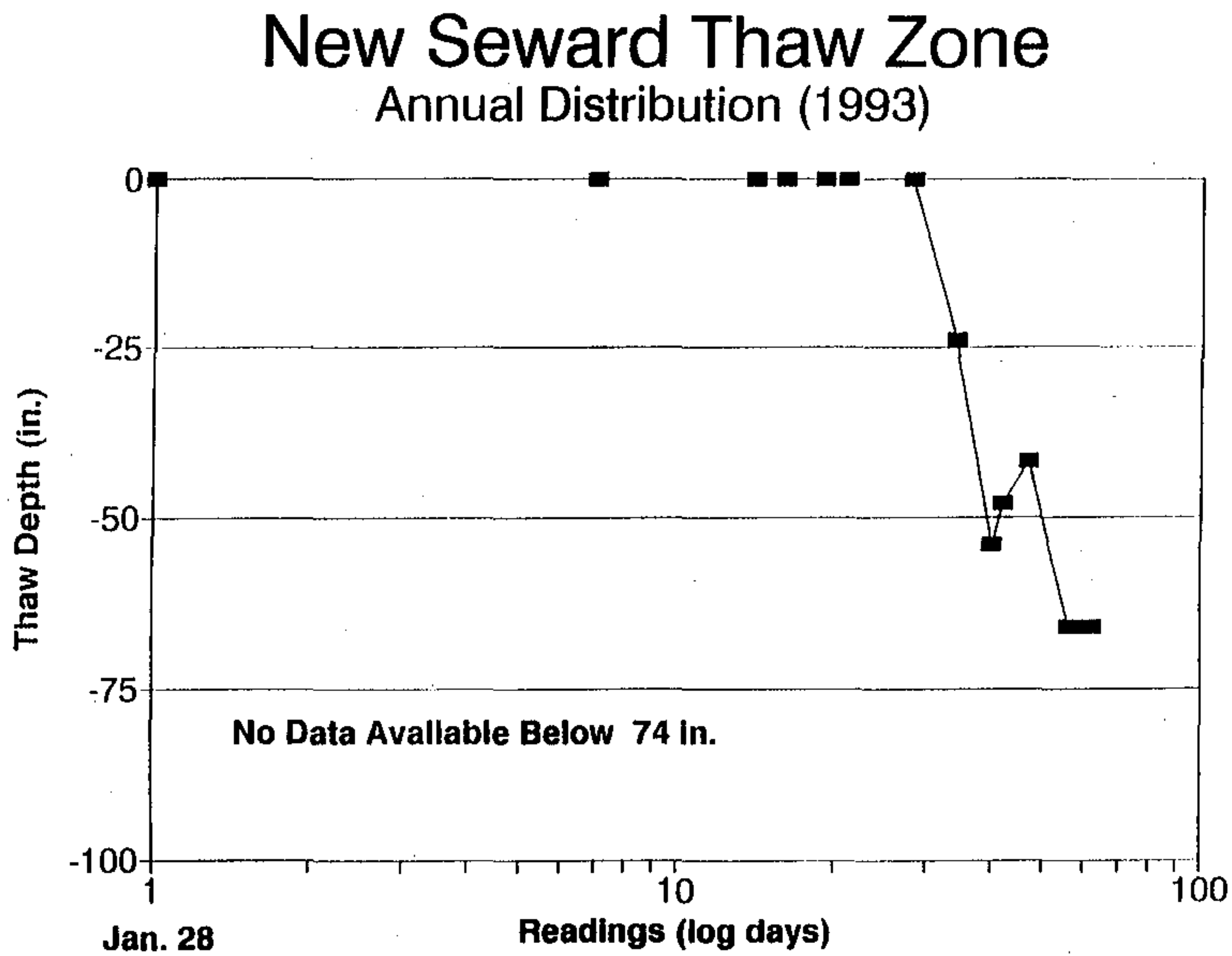



New Seward Temperature
Annual Distribution (1993)
100
75
50
25
0
Temperature (F)
Jan. Mar.
Apr.
Readings

New Seward Thaw Zone

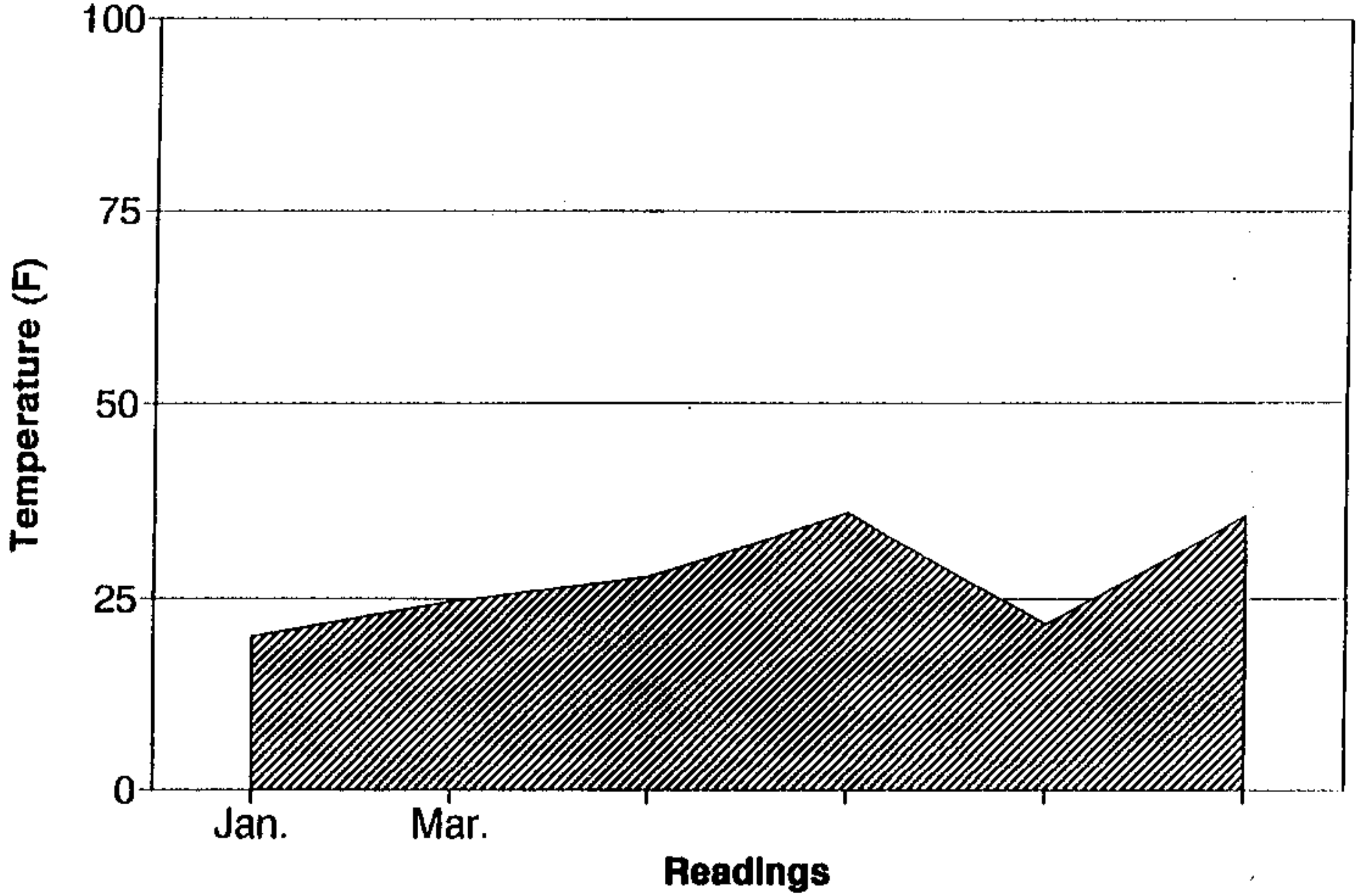
Annual Distribution (1993)

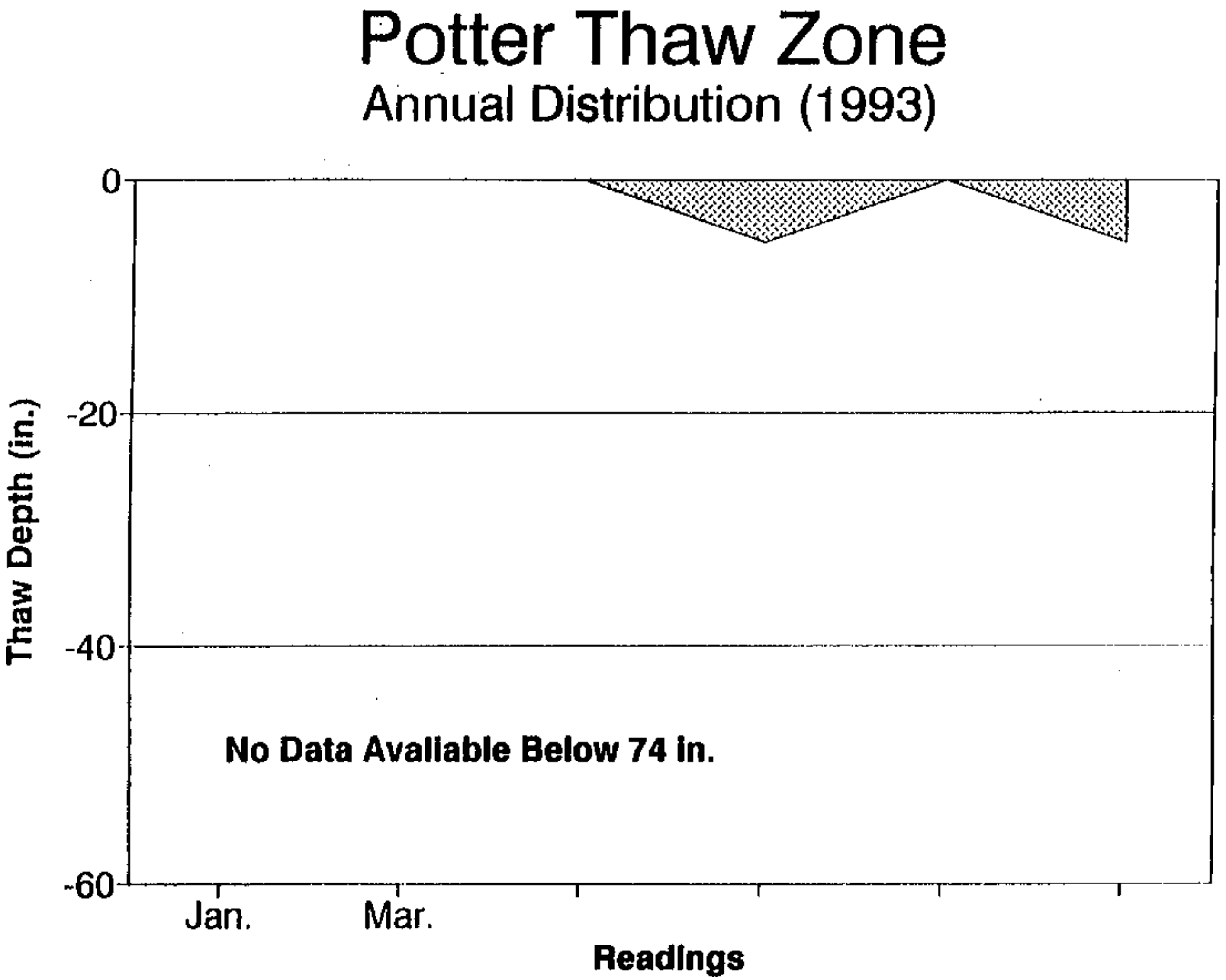




New Seward Thaw Zone
Annual Distribution (1993)
Thaw Depth (in.)
0
-25
-50
-75
-100
1
10
100
Jan. 28
Readings (log days)
No Data Available Below 74 in.

Potter Temperature
Annual Distribution (1993)

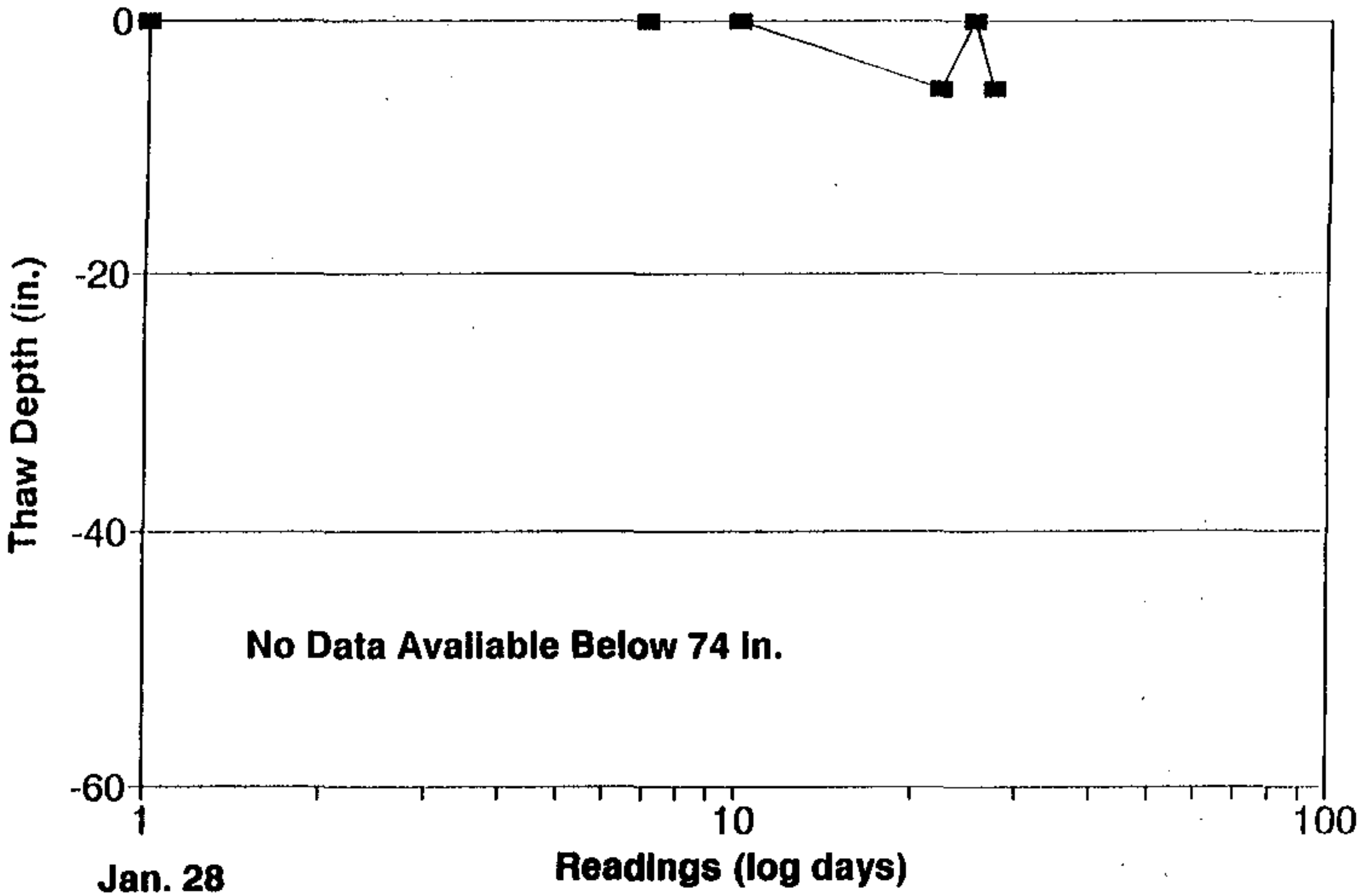




Potter Thaw Zone
Annual Distribution (1993)
Thaw Depth (in.)
0
-20
-40
-60
No Data Available Below 74 in.
Jan.
Mar.
Readings

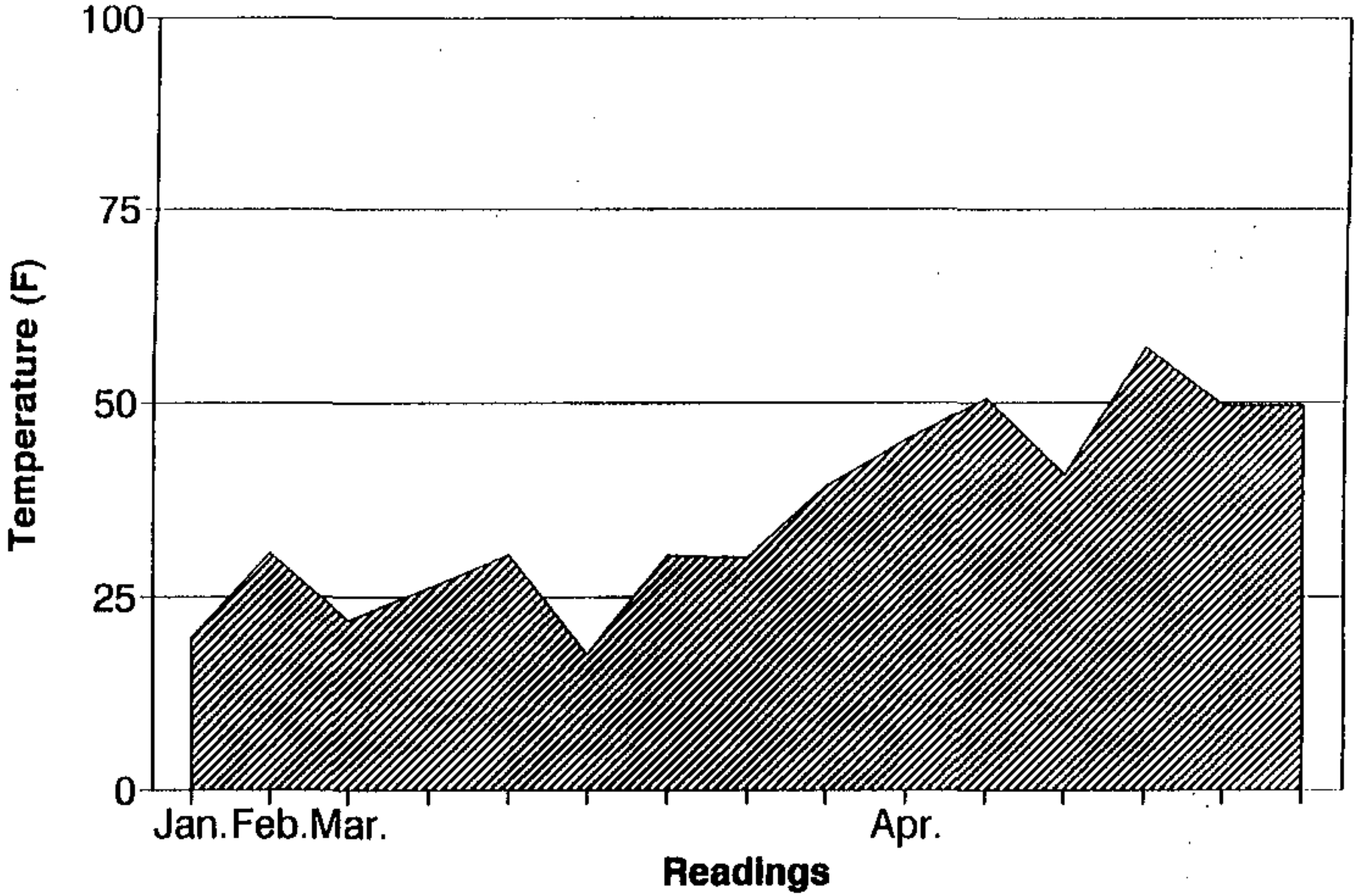
Potter Thaw Zone

Annual Distribution (1993)



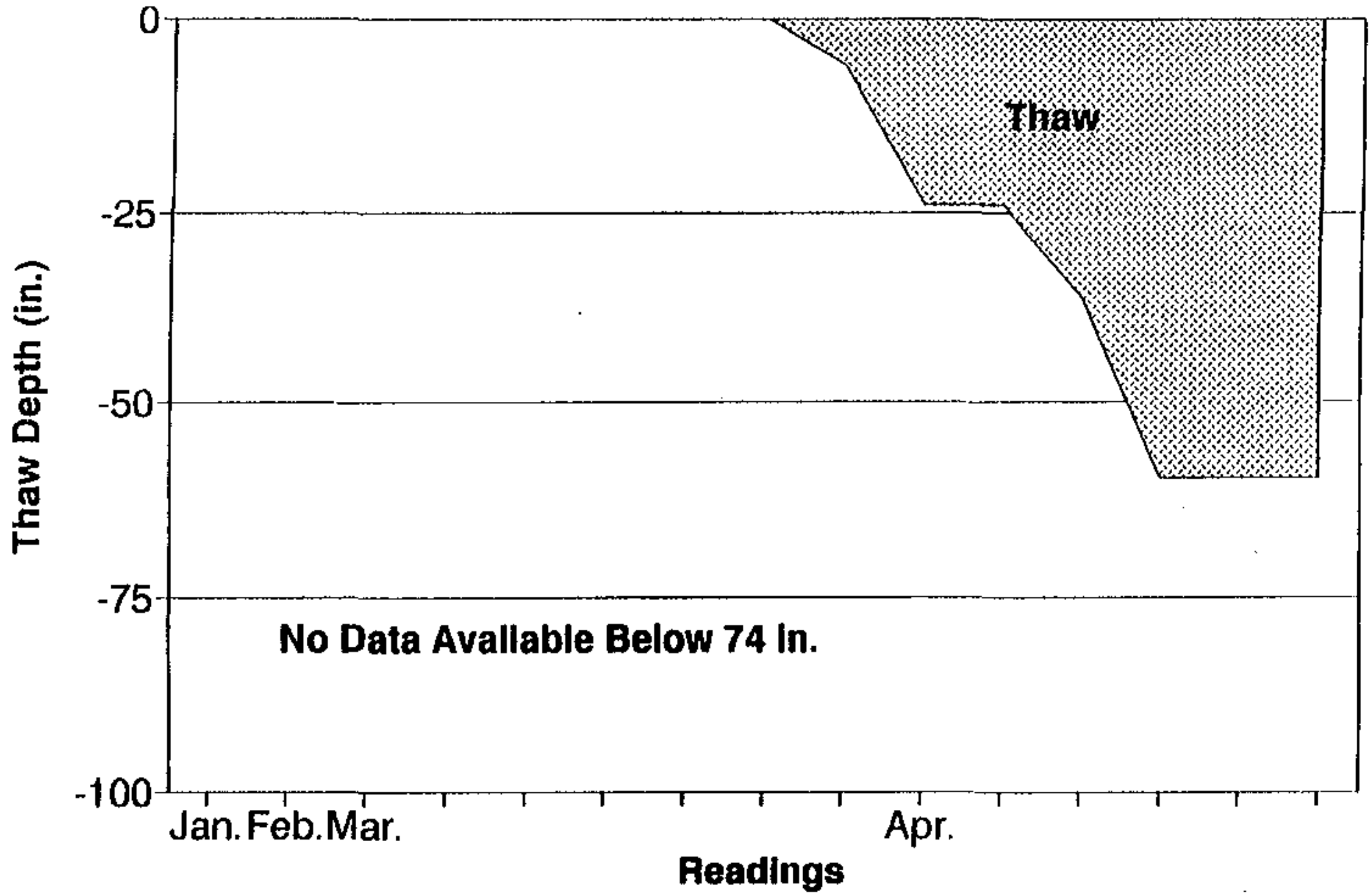
Rabbit Temperature

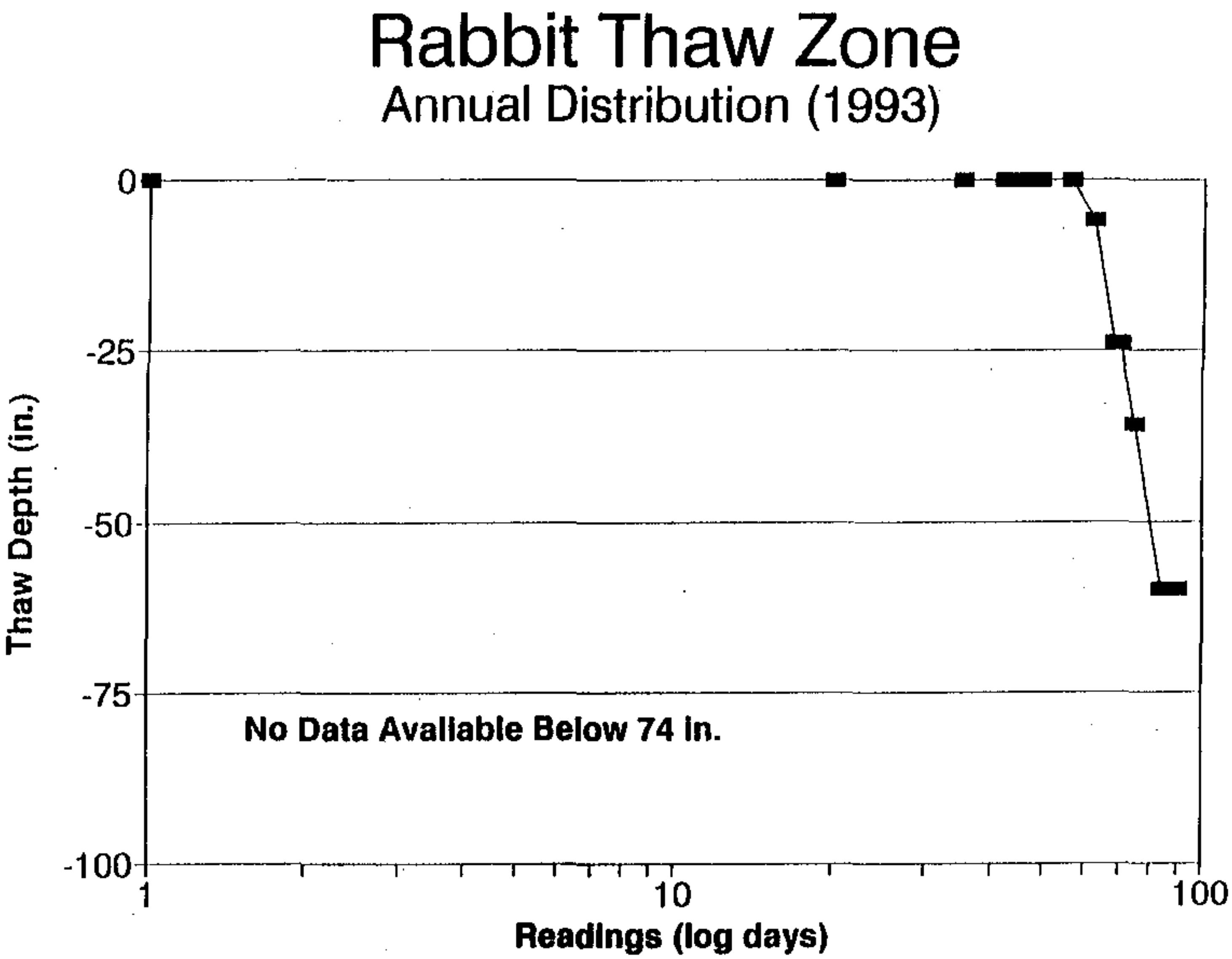
Annual Distribution (1993)



Rabbit Thaw Zone

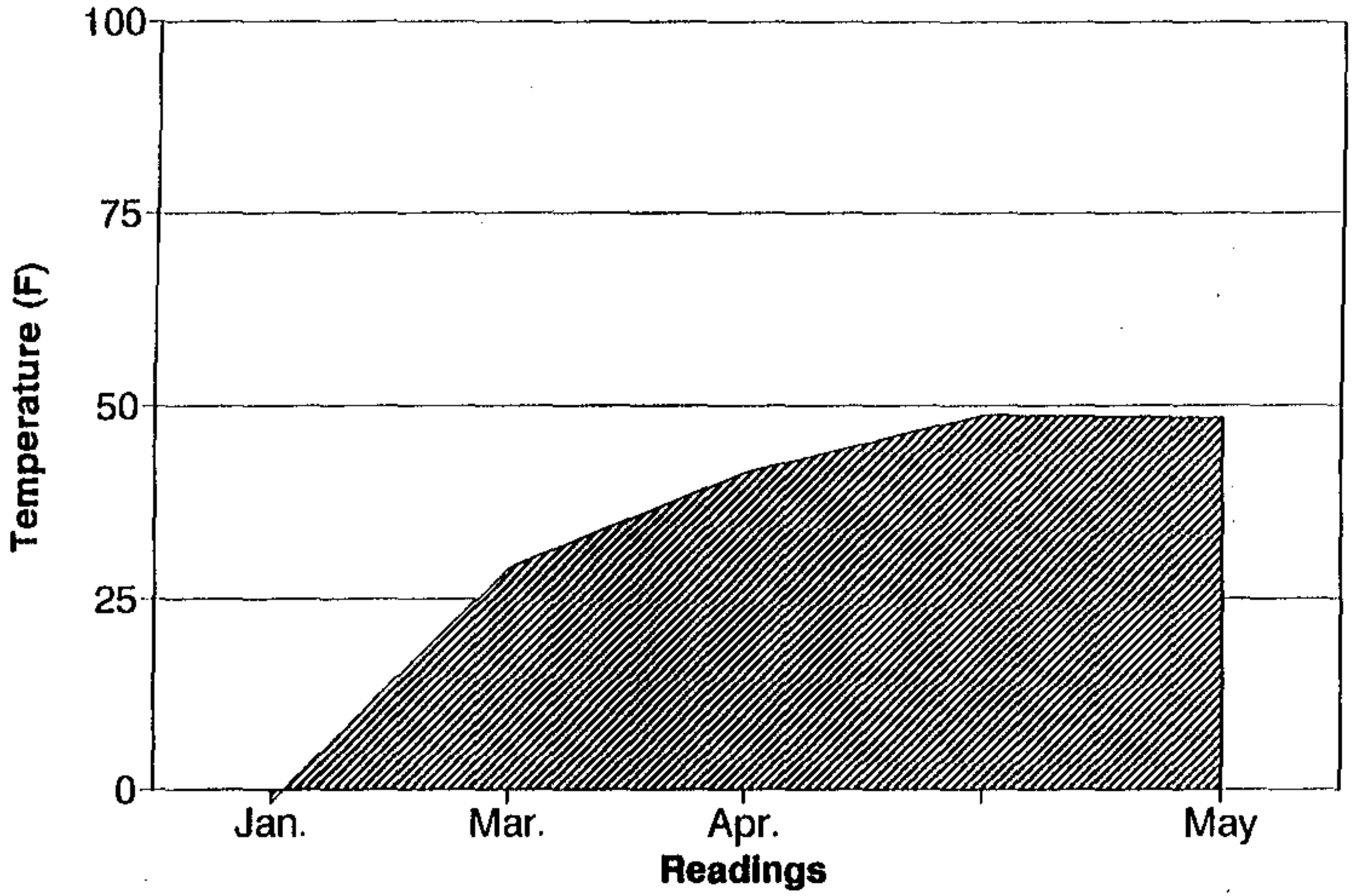
Annual Distribution (1993)




Rabbit Thaw Zone
Annual Distribution (1993)
0
-25
-50
-75
-100
Thaw Depth (in.)
No Data Available Below 74 in.
1
10
100
Readings (log days)

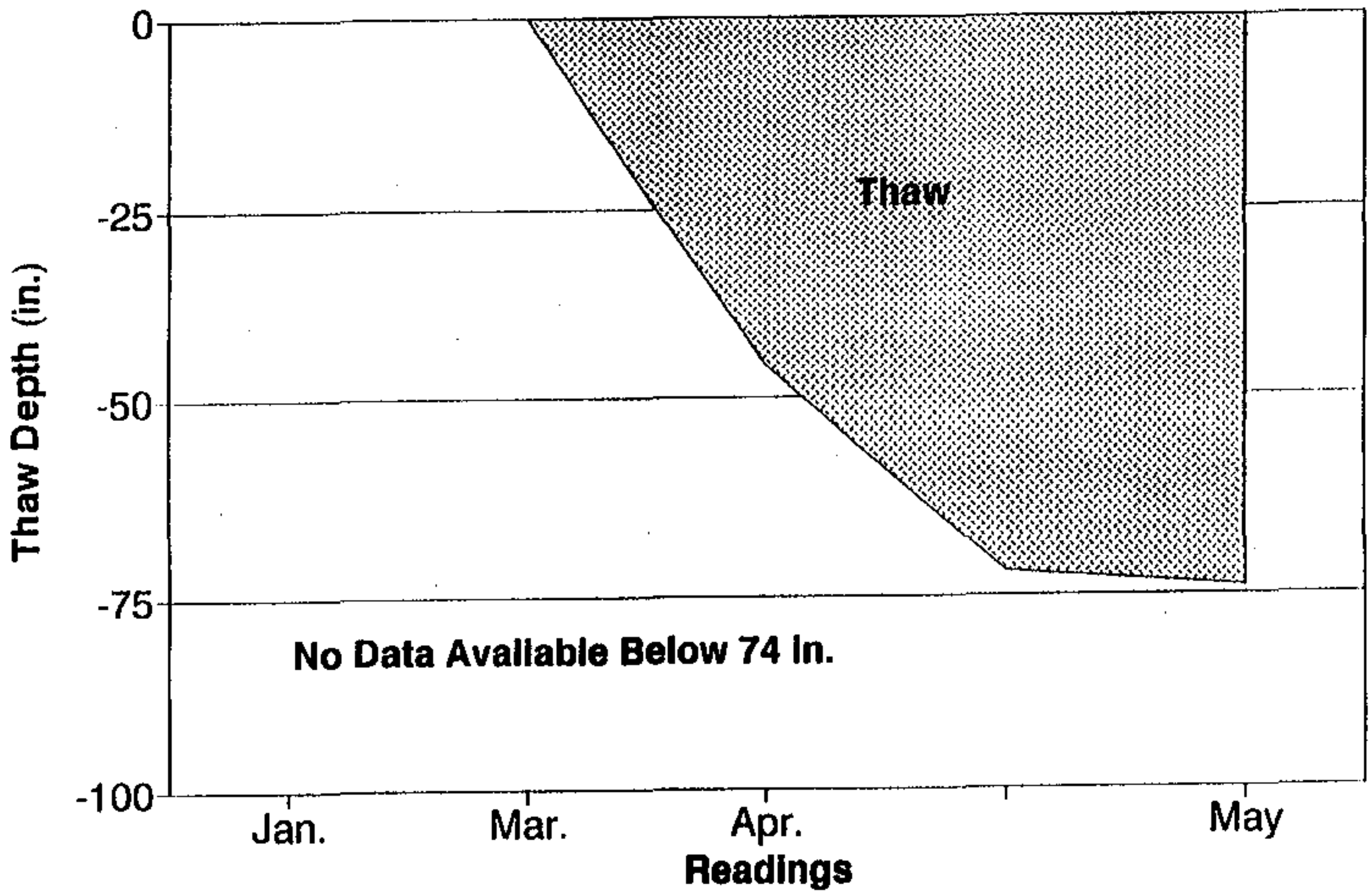
Summit Temperature

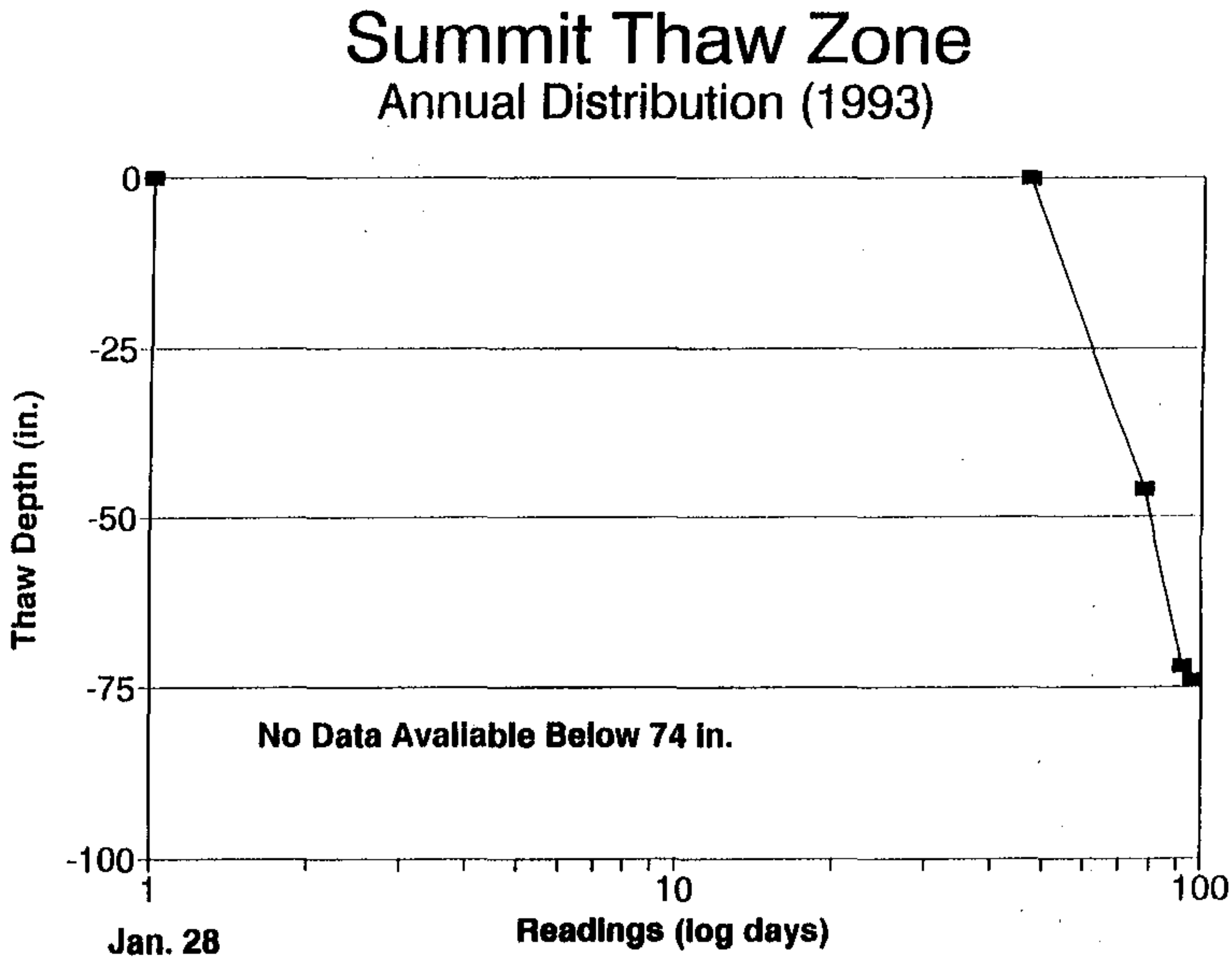
Annual Distribution (1993)



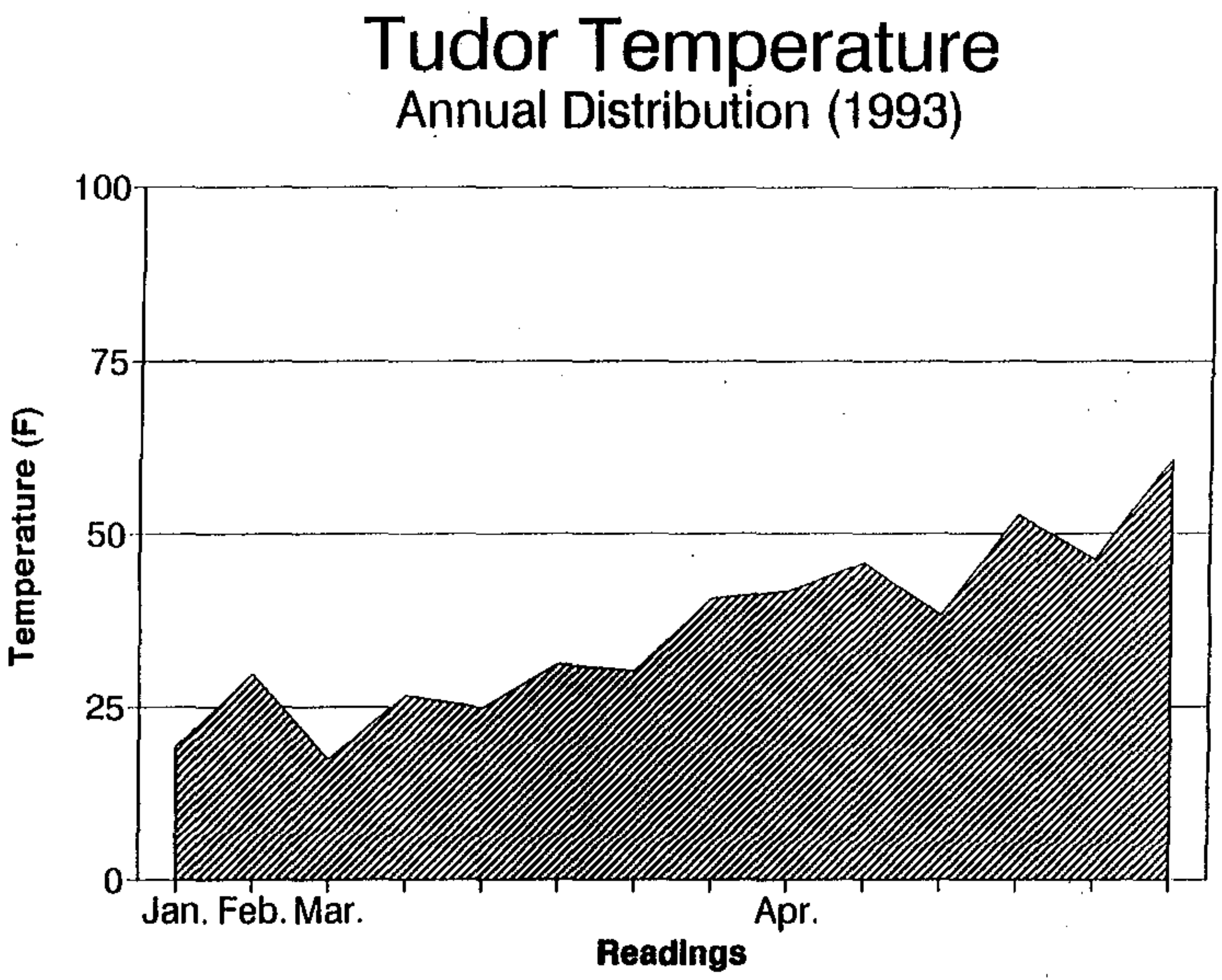
Summit Thaw Zone

Annual Distribution (1993)



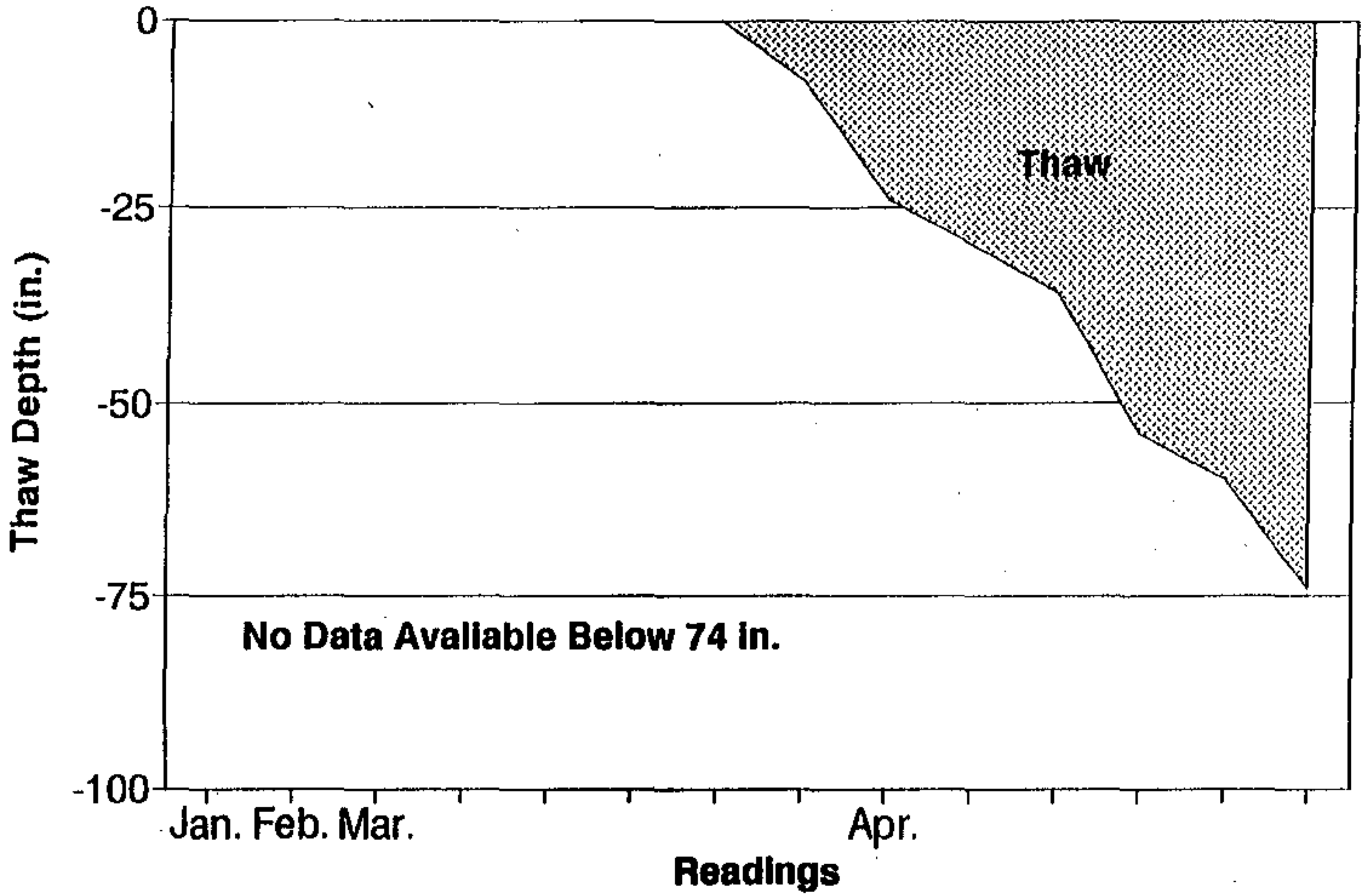


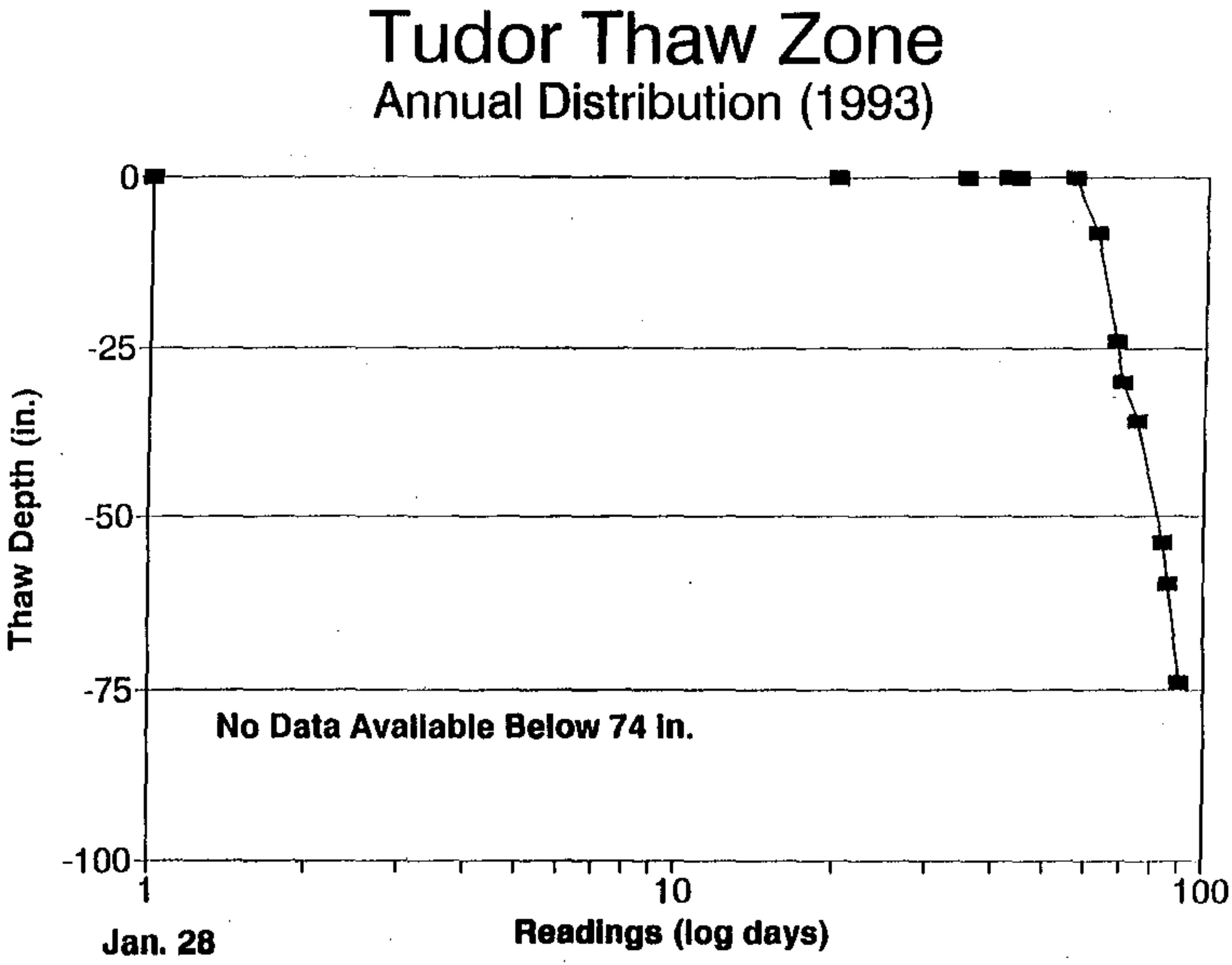
Summit Thaw Zone
Annual Distribution (1993)
Thaw Depth (in.)
0
-25
-50
-75
-100
No Data Available Below 74 in.
1
10
100
Jan. 28
Readings (log days)


Tudor Temperature
Annual Distribution (1993)
100
75
50
25
0
Temperature (F)
Jan. Feb. Mar.
Apr.
Readings

Tudor Thaw Zone

Annual Distribution (1993)




Tudor Thaw Zone
Annual Distribution (1993)
Thaw Depth (in.)
0
-25
-50
-75
-100
1
10
100
Jan. 28
Readings (log days)
No Data Available Below 74 in.

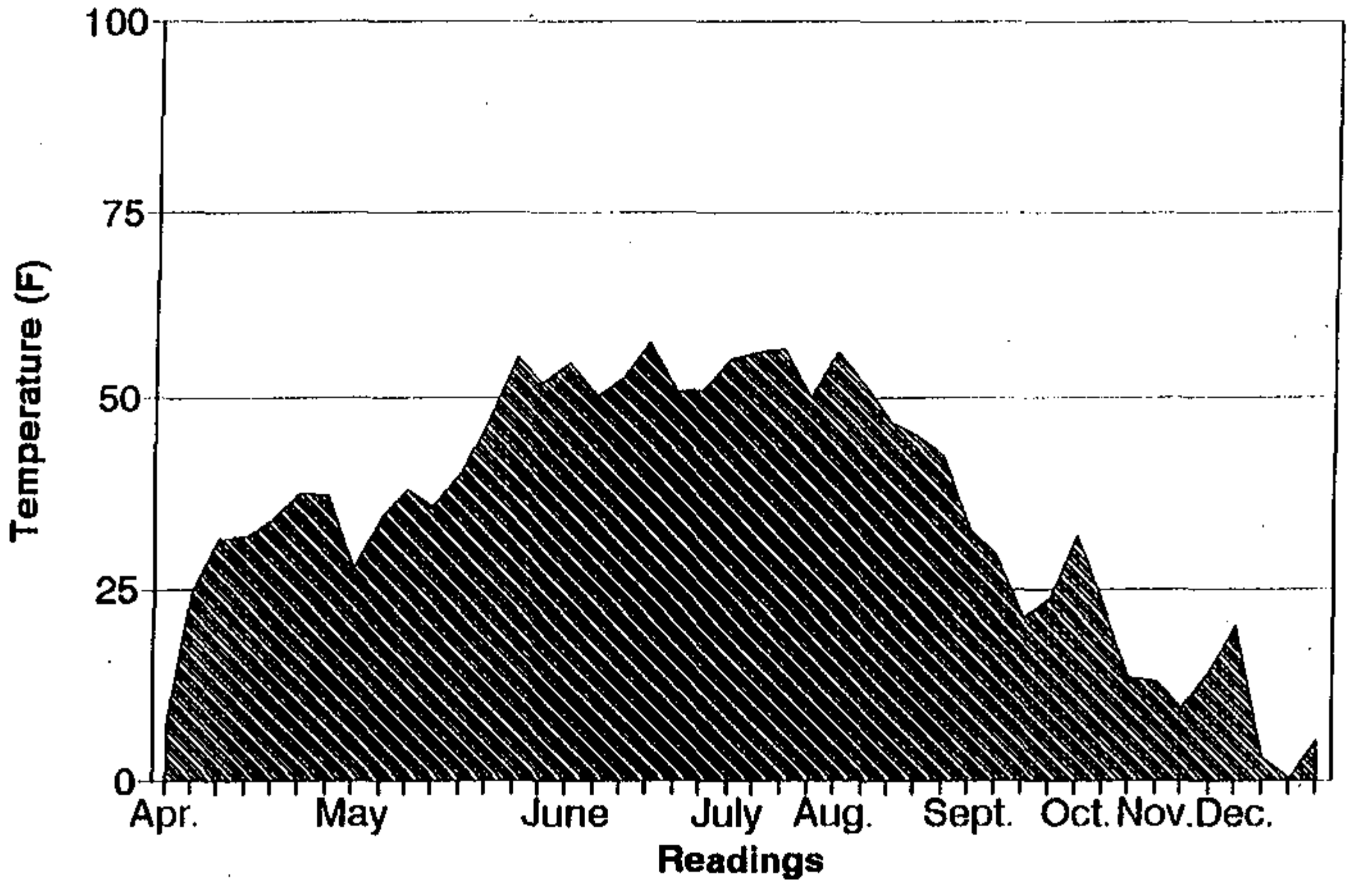
APPENDIX E

GROUND TEMPERATURE DATA - NORTHERN REGION

TABLE OF CONTENTS

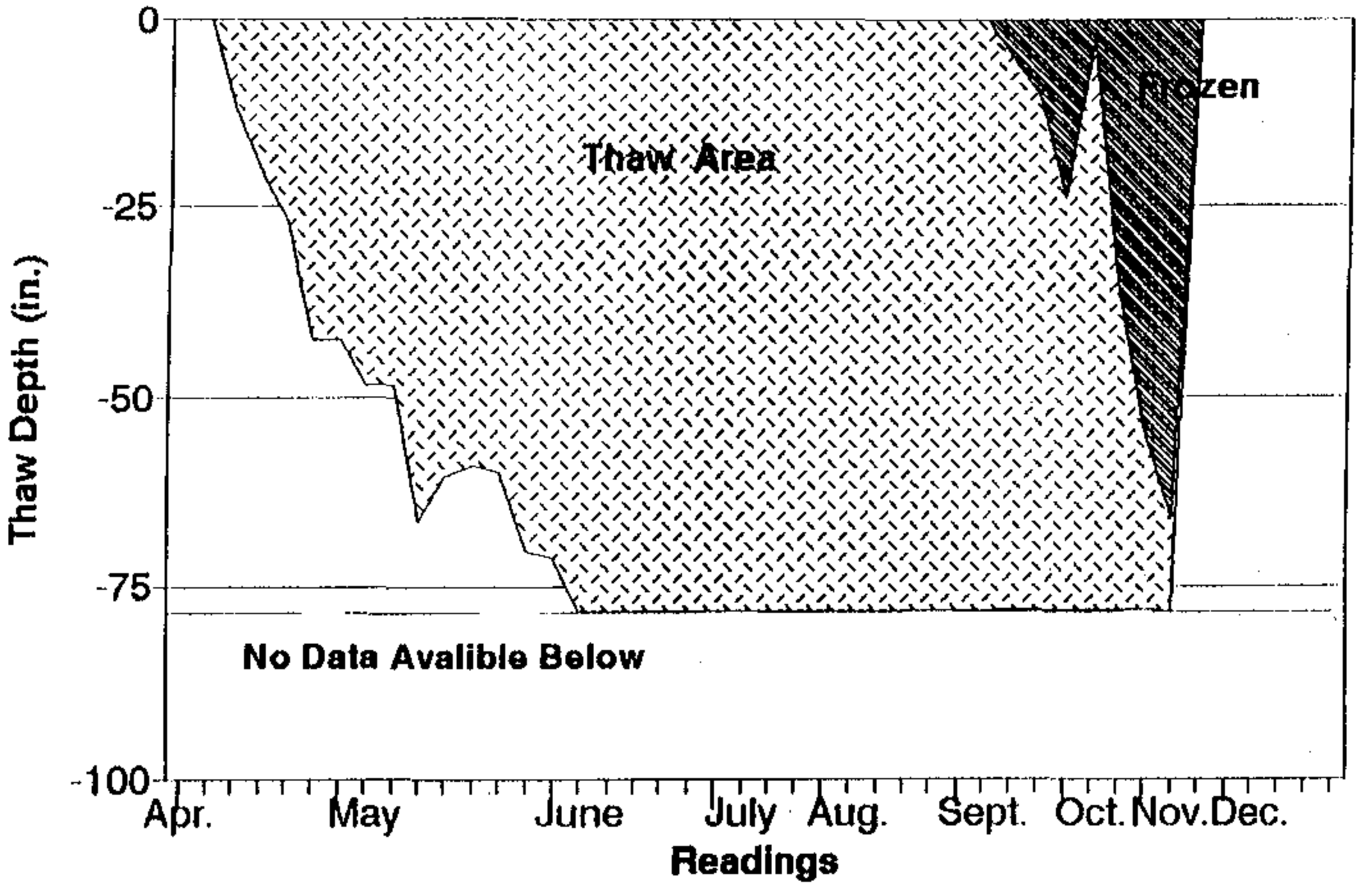
APPENDIX E -- GROUND TEMPERATURE DATA - NORTHERN REGION

Cantwell Temperature
Annual Distribution (1992)



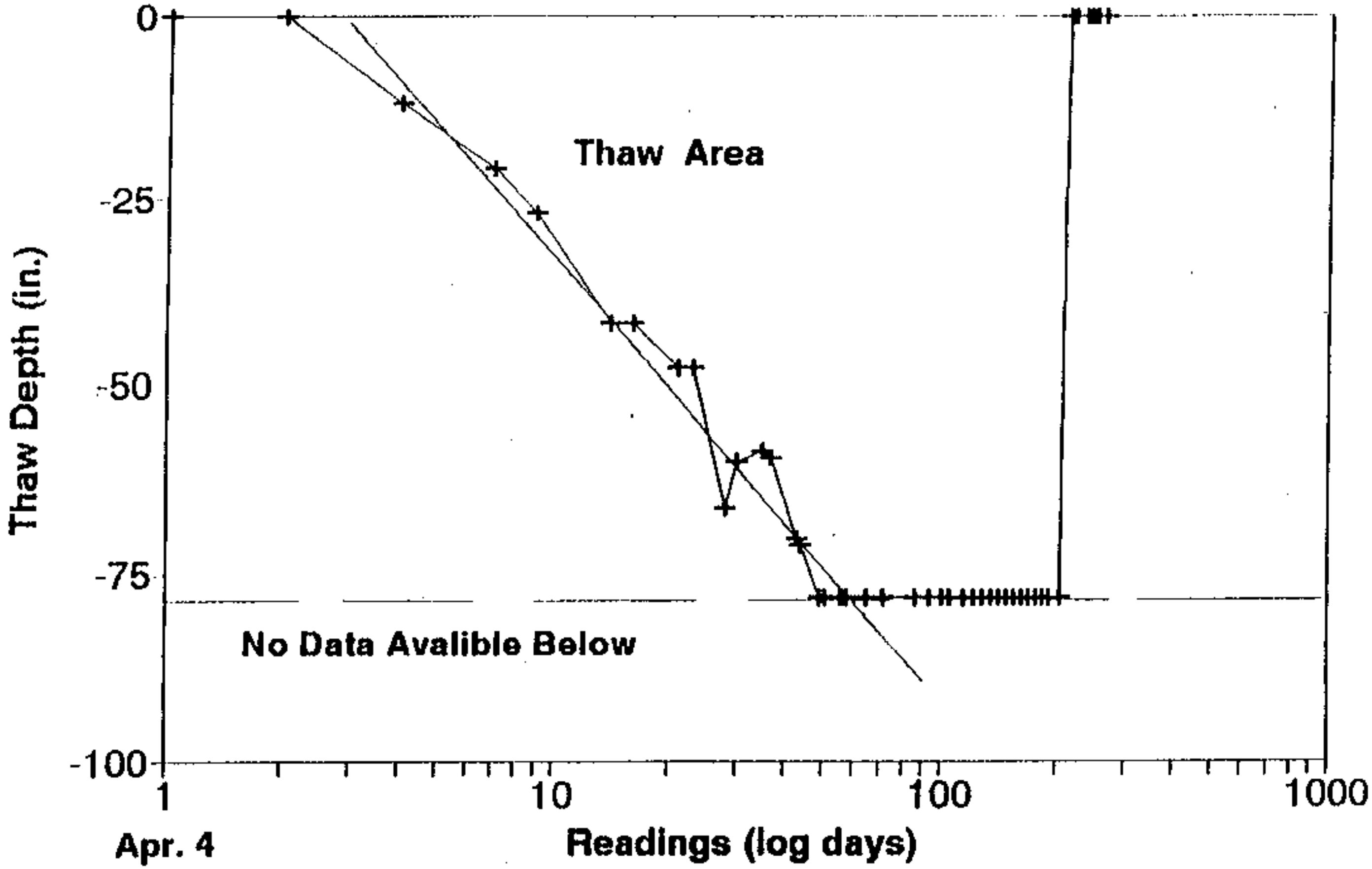
Cantwell Thaw Zone

Annual Distribution (1992)

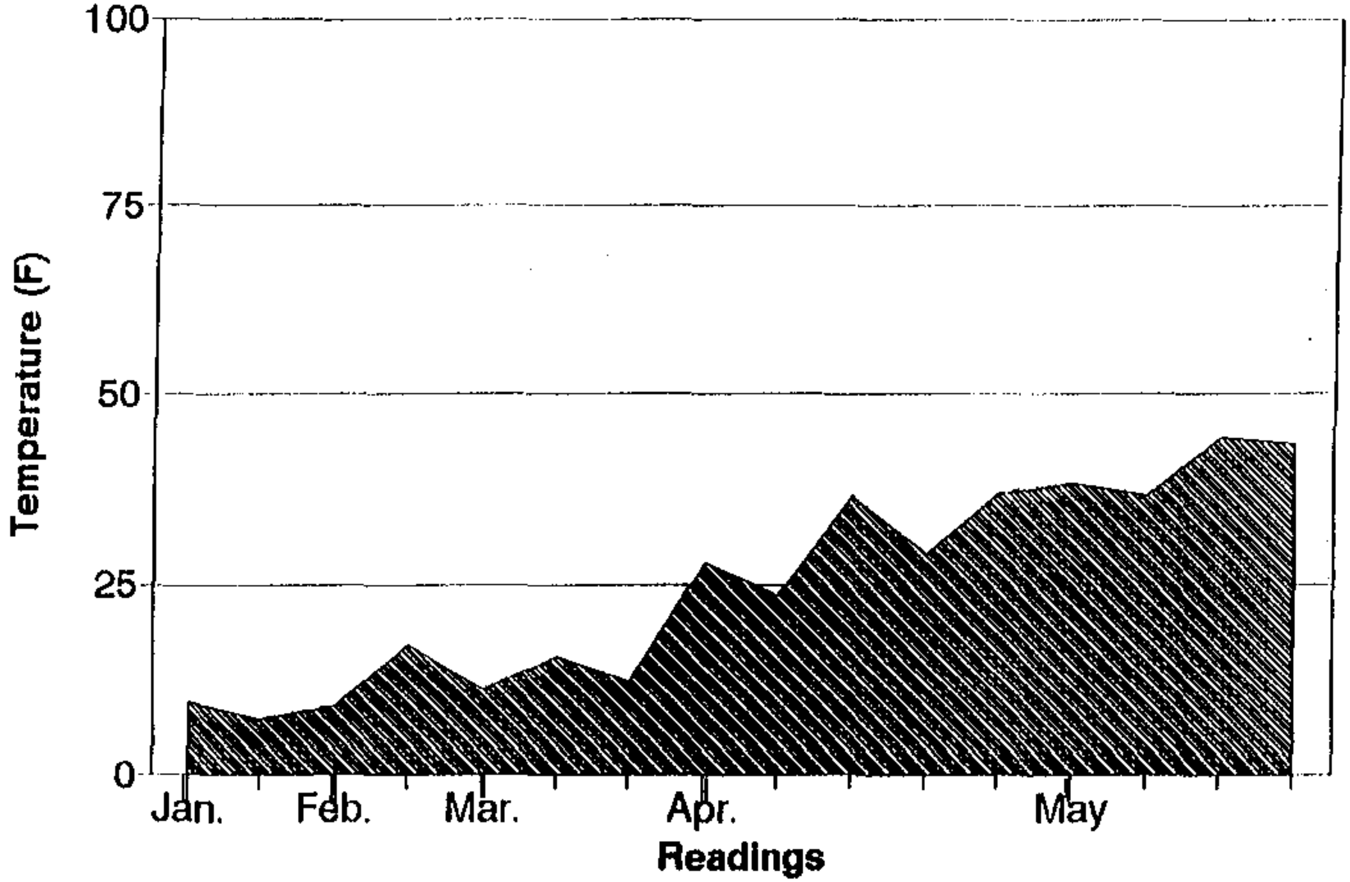


Cantwell Thaw Zone

Annual Distribution (1992)

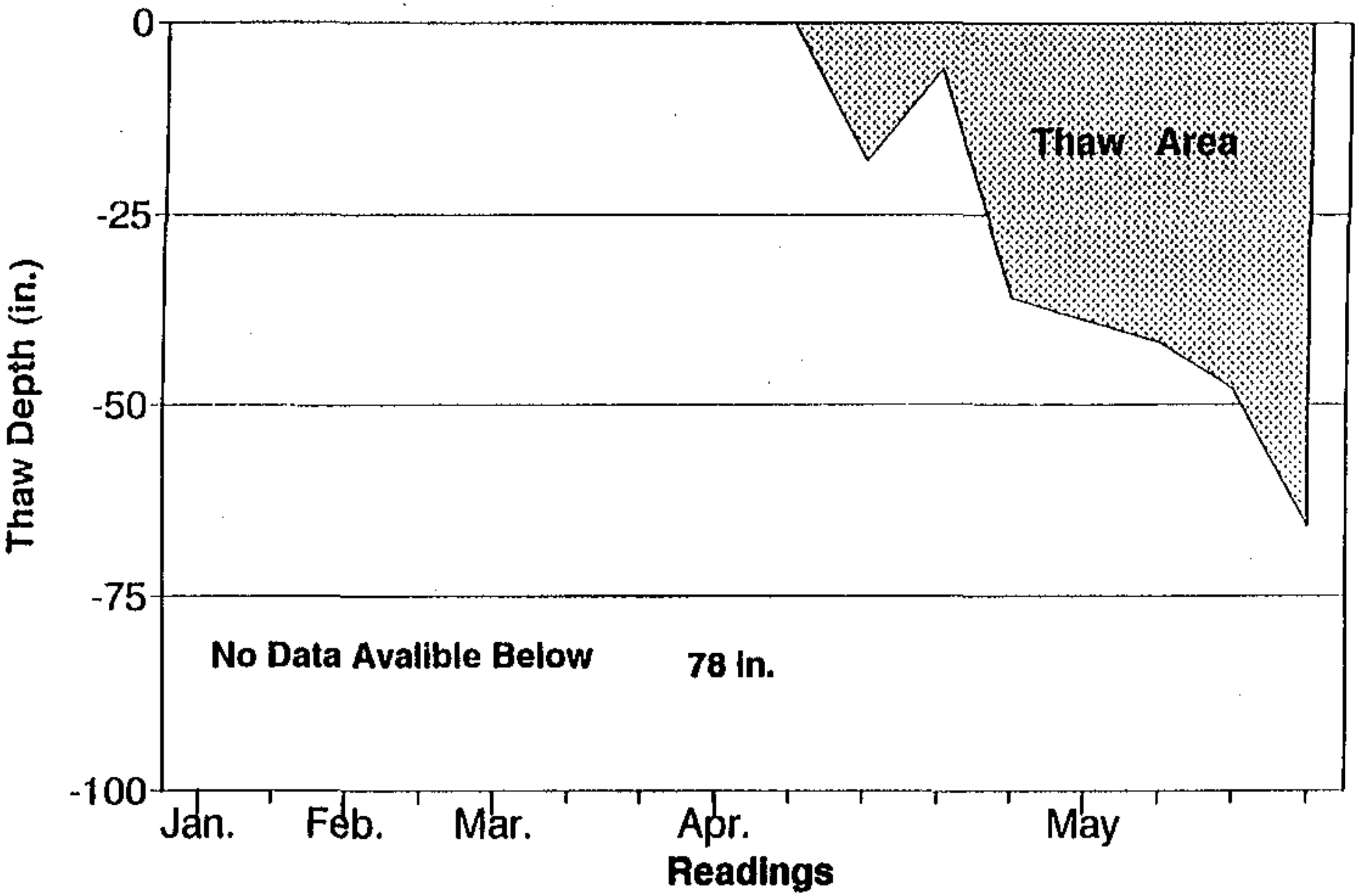


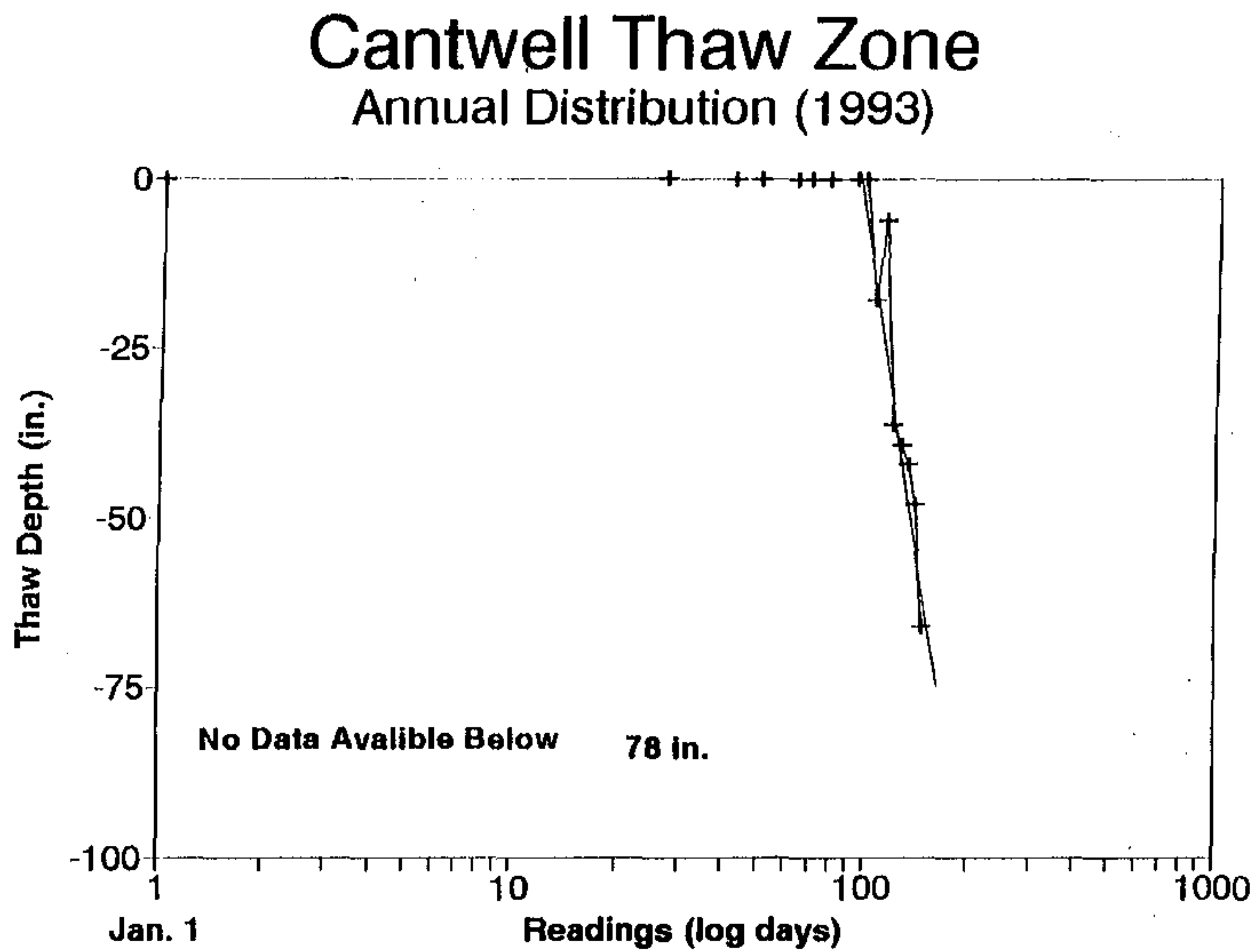
Cantwell Temperature
Annual Distribution (1993)



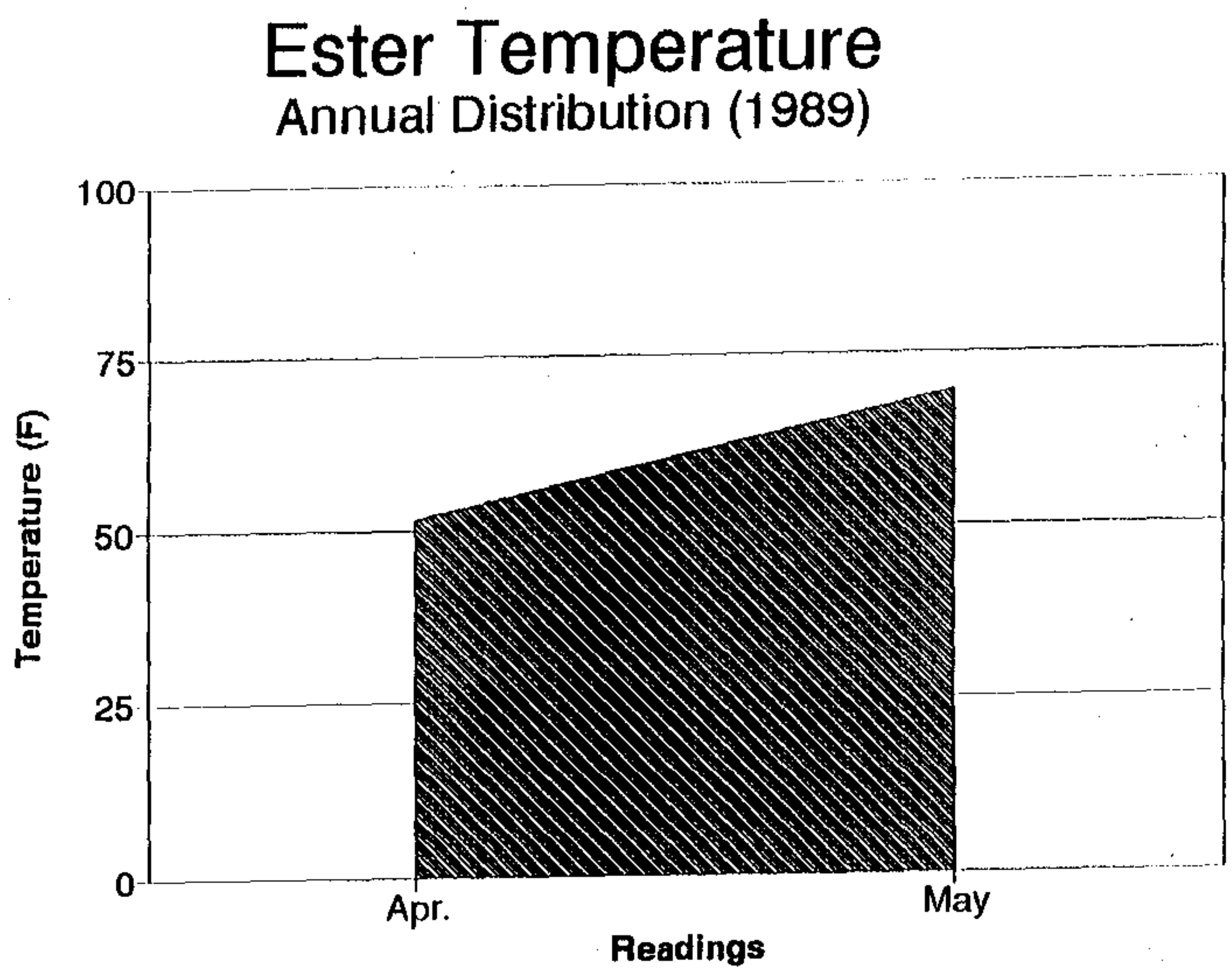
Cantwell Thaw Zone

Annual Distribution (1993)



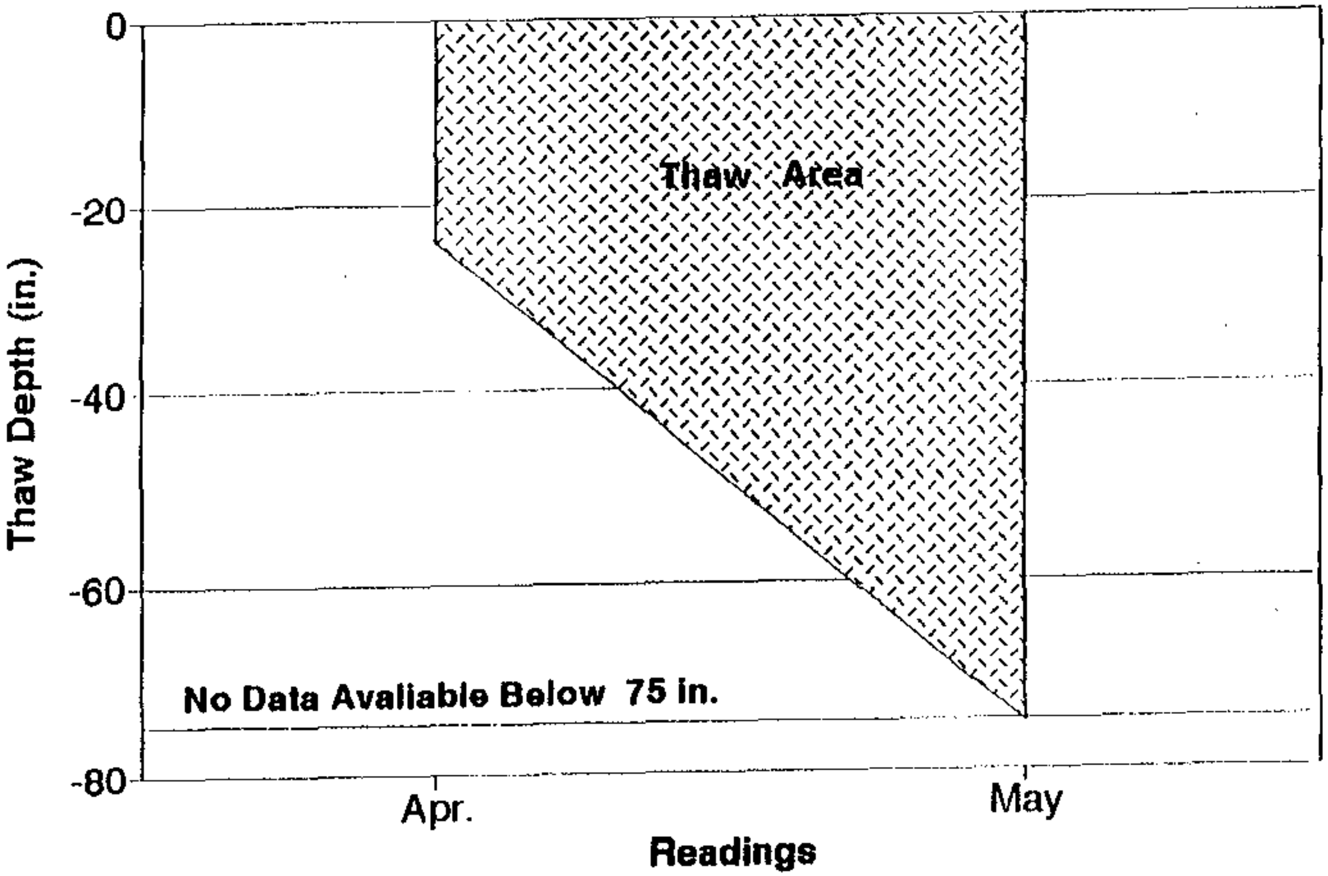


Cantwell Thaw Zone
Annual Distribution (1993)
0
-25
-50
-75
-100
Thaw Depth (in.)
No Data Avalible Below
78 in.
1
10
100
1000
Jan. 1
Readings (log days)


Ester Temperature
Annual Distribution (1989)
100
75
50
25
0
Temperature (F)
Apr.
May
Readings

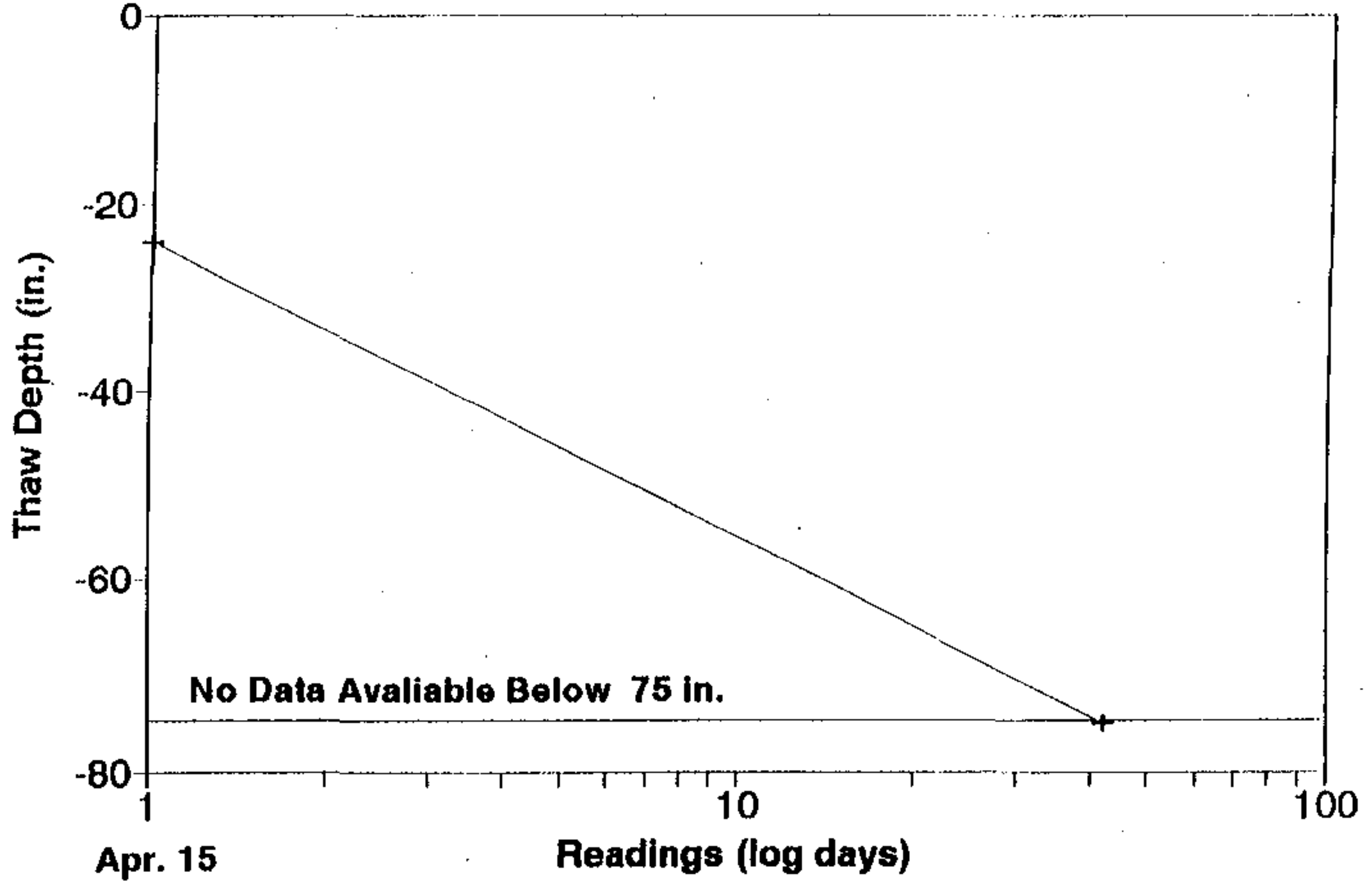
Ester Thaw Zone

Annual Distribution (1989)



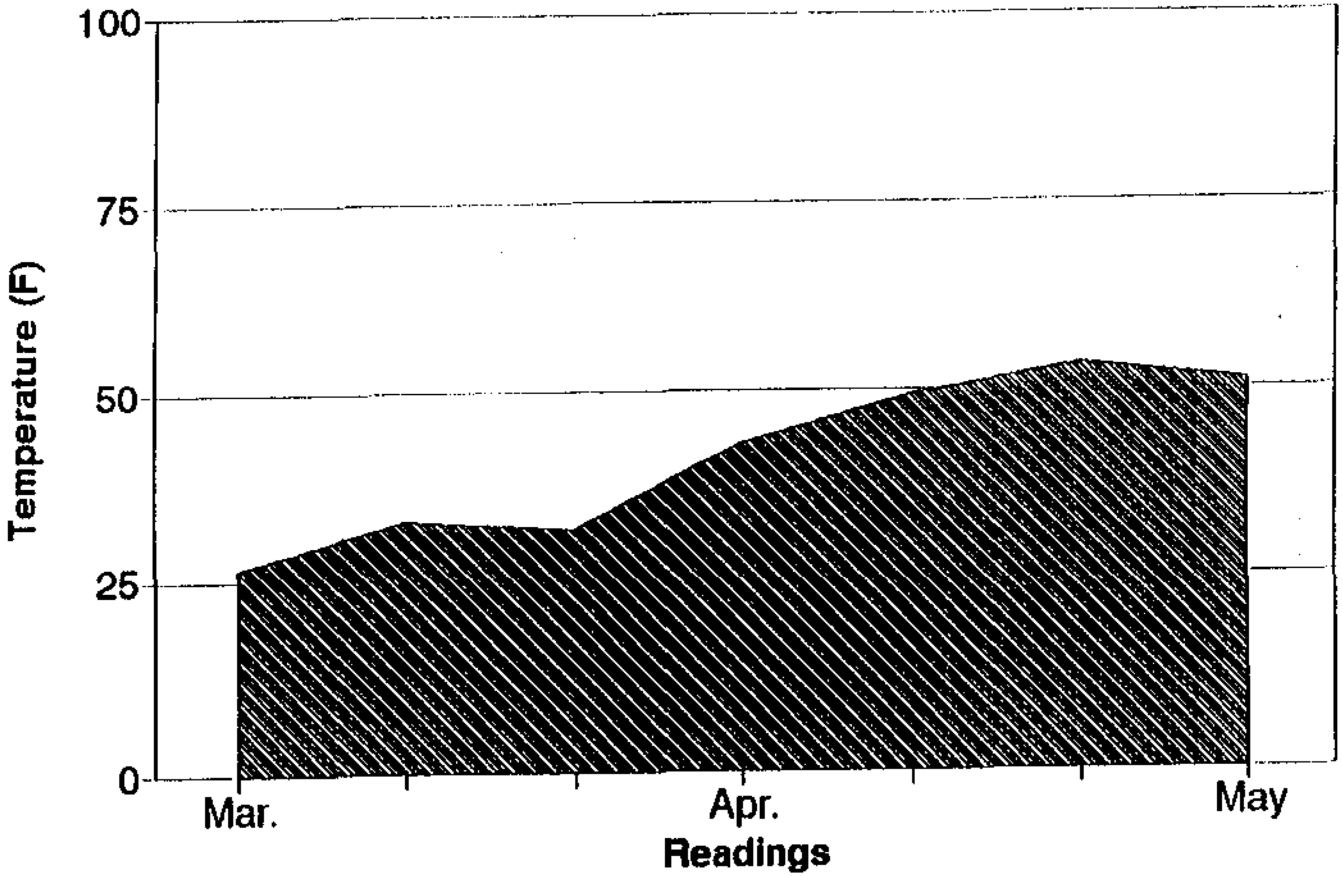
Ester Thaw Zone

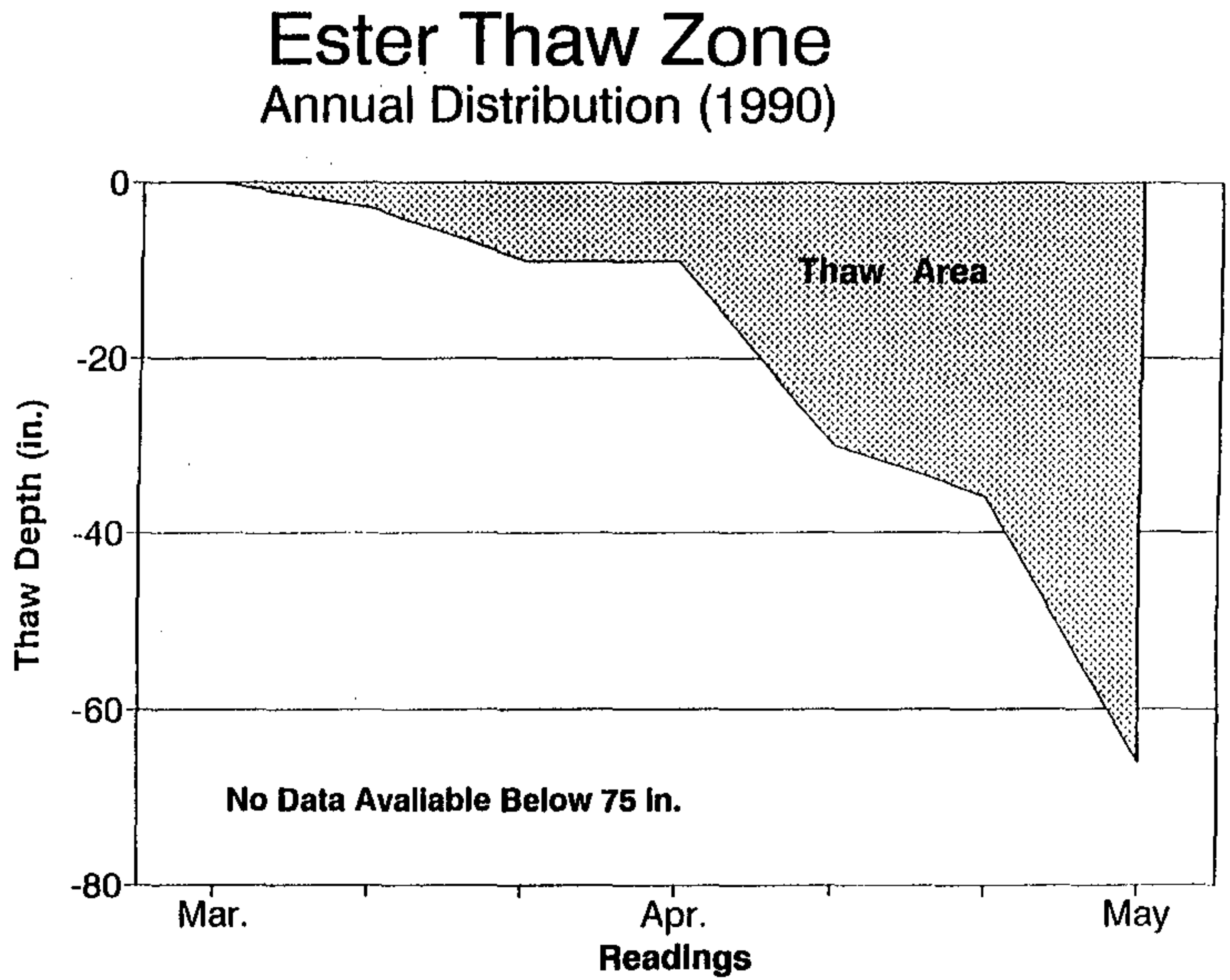
Annual Distribution (1989)



Ester Temperature

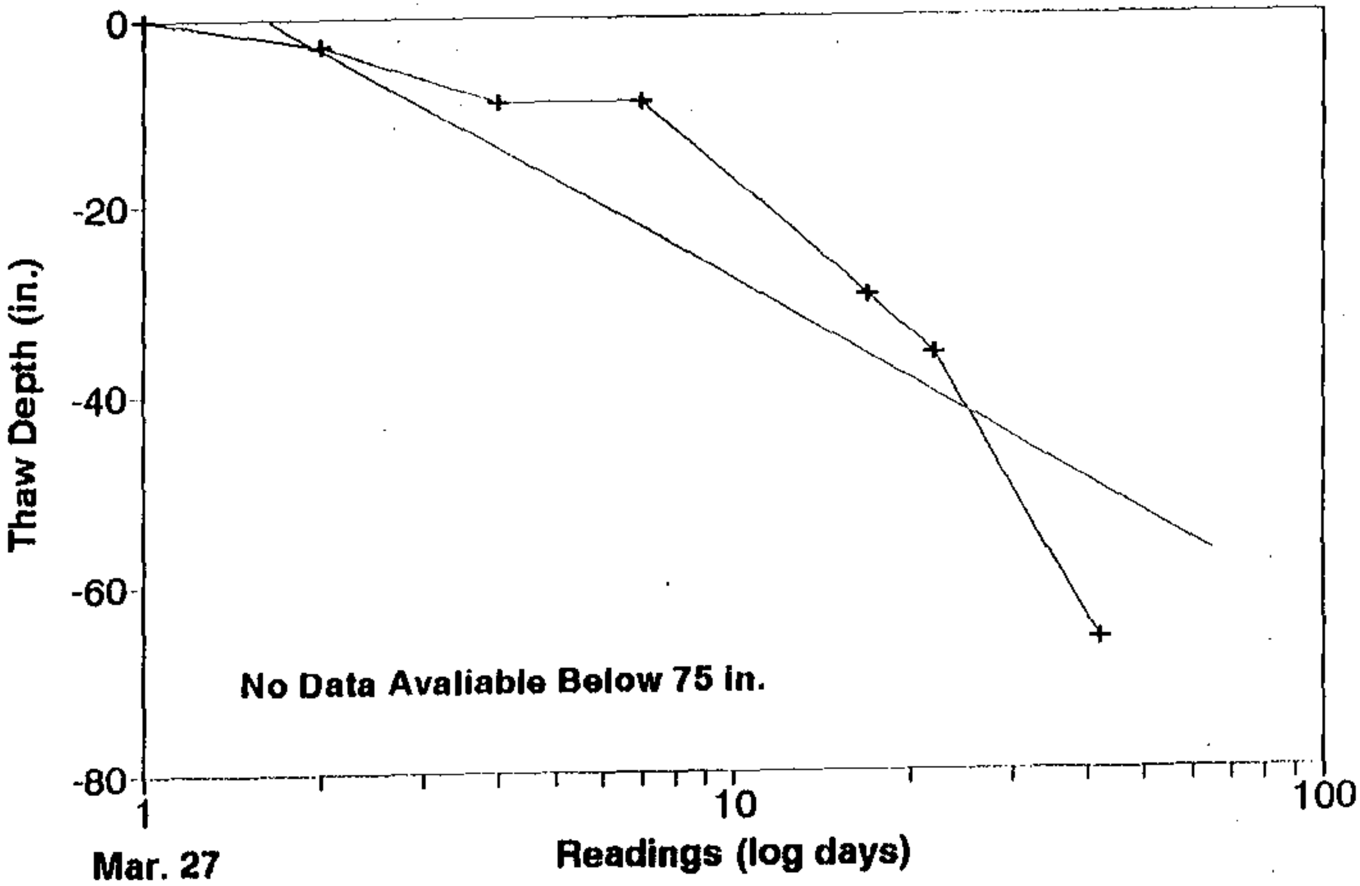
Annual Distribution (1990)



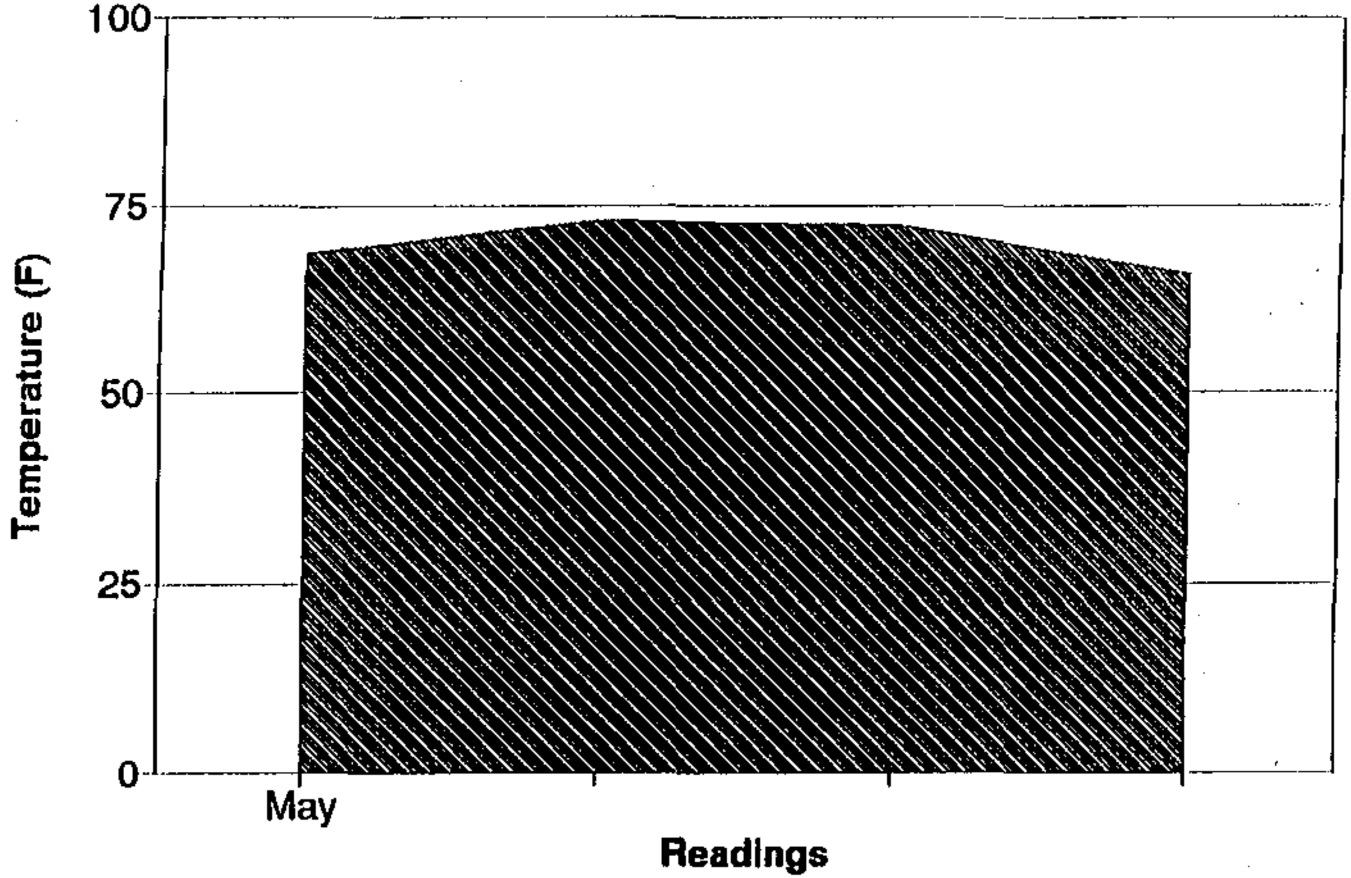


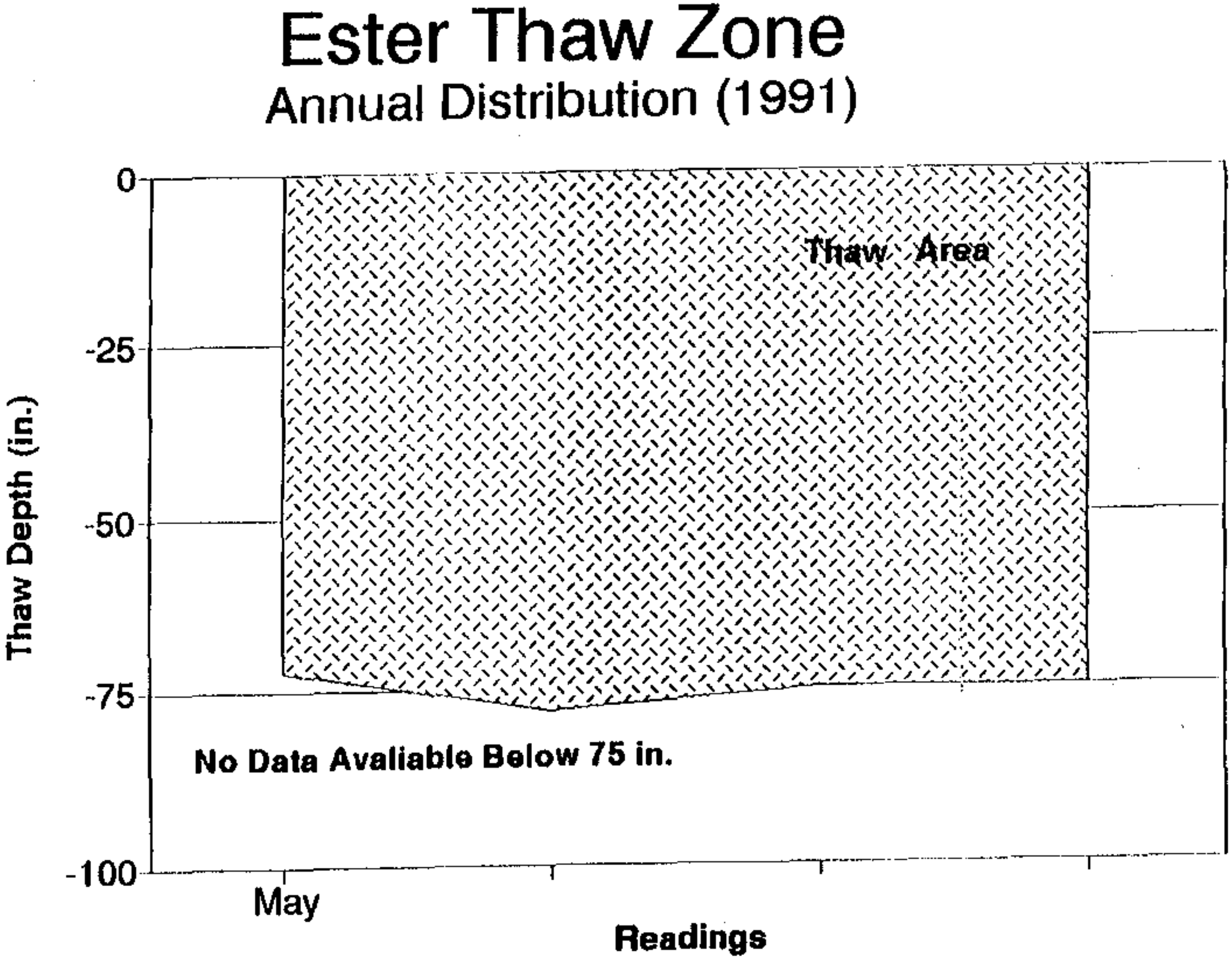
Ester Thaw Zone
Annual Distribution (1990)
Thaw Area
No Data Available Below 75 in.
0
-20
-40
-60
-80
Thaw Depth (in.)
Mar.
Apr.
May
Readings

Ester Thaw Zone
Annual Distribution (1990)

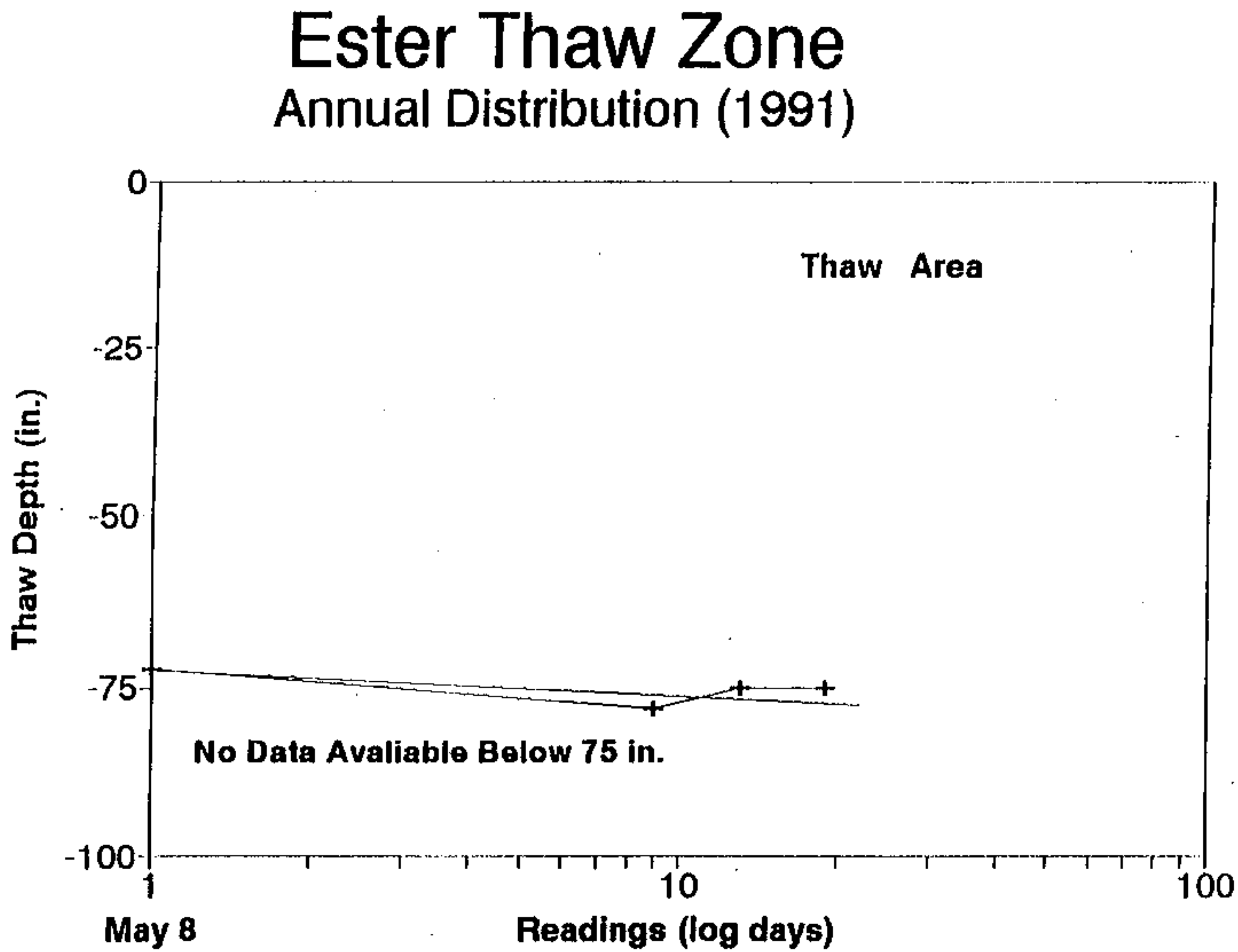


Ester Temperature
Annual Distribution (1991)



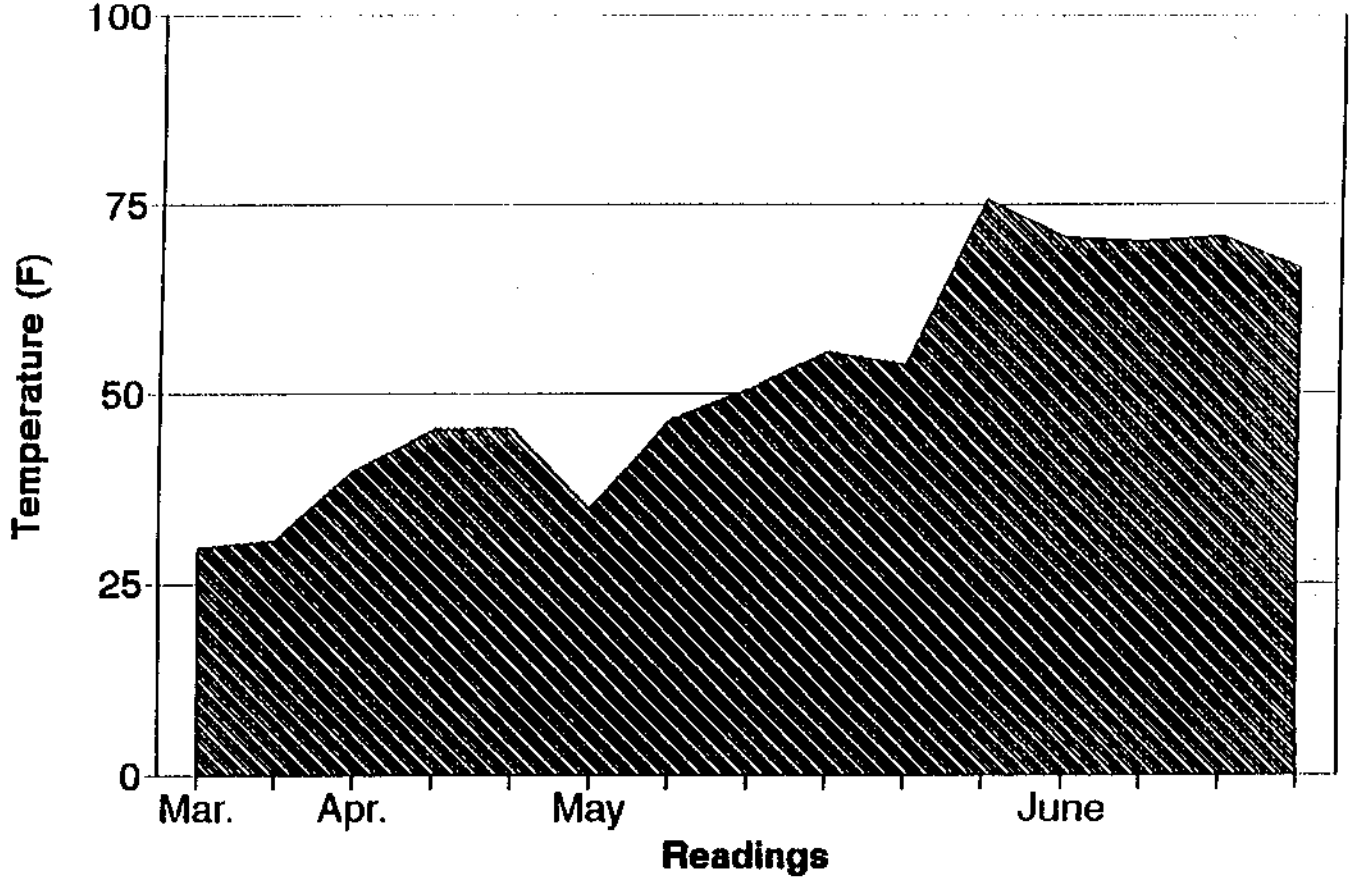


Ester Thaw Zone
Annual Distribution (1991)
0
-25
-50
-75
-100
Thaw Depth (in.)
Thaw Area
No Data Avaliable Below 75 in.
May
Readings


Ester Thaw Zone
Annual Distribution (1991)
Thaw Area
Thaw Depth (in.)
0
-25
-50
-75
-100
No Data Avaliable Below 75 in.
1
10
100
May 8
Readings (log days)

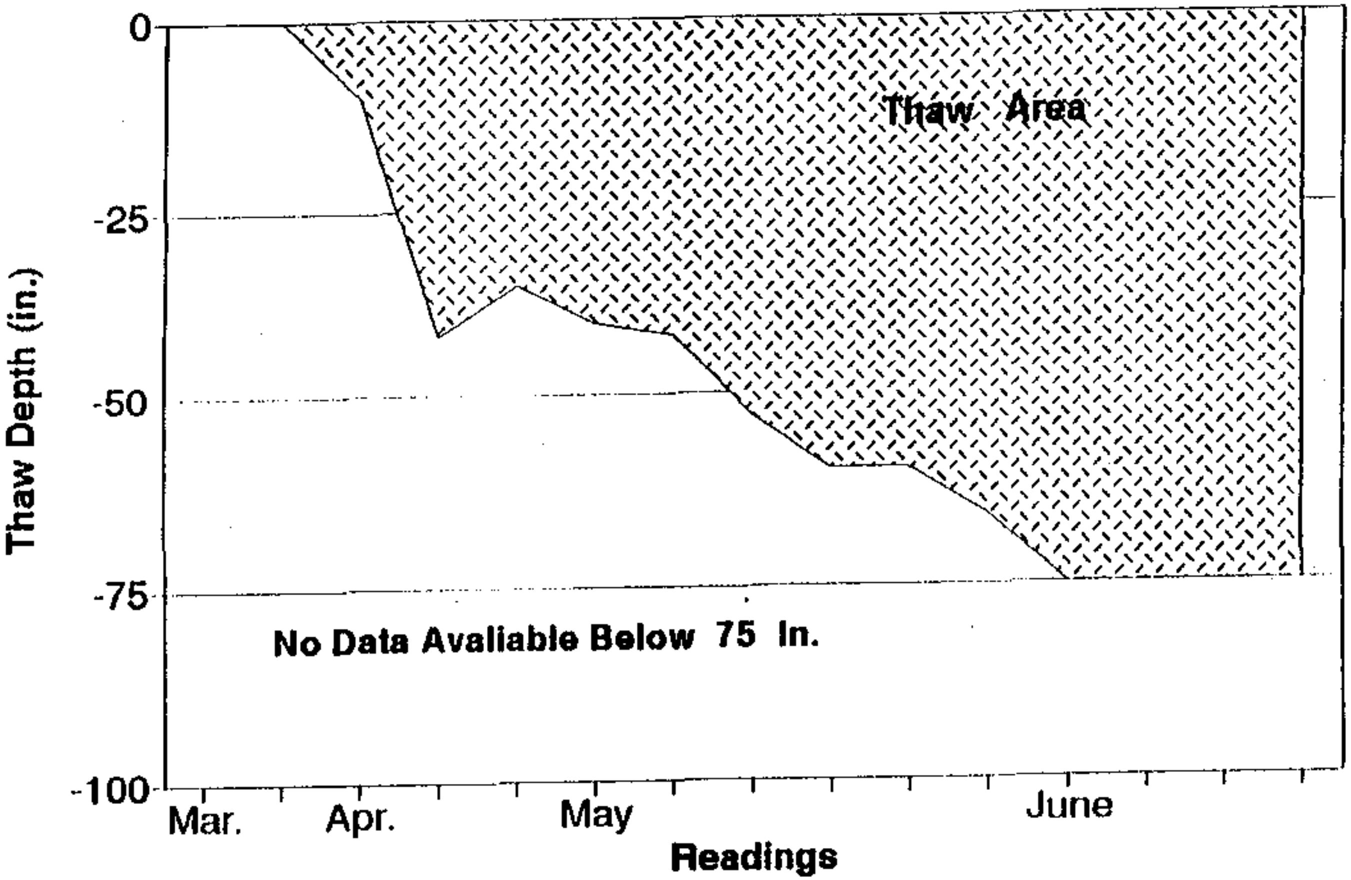
Ester Temperature

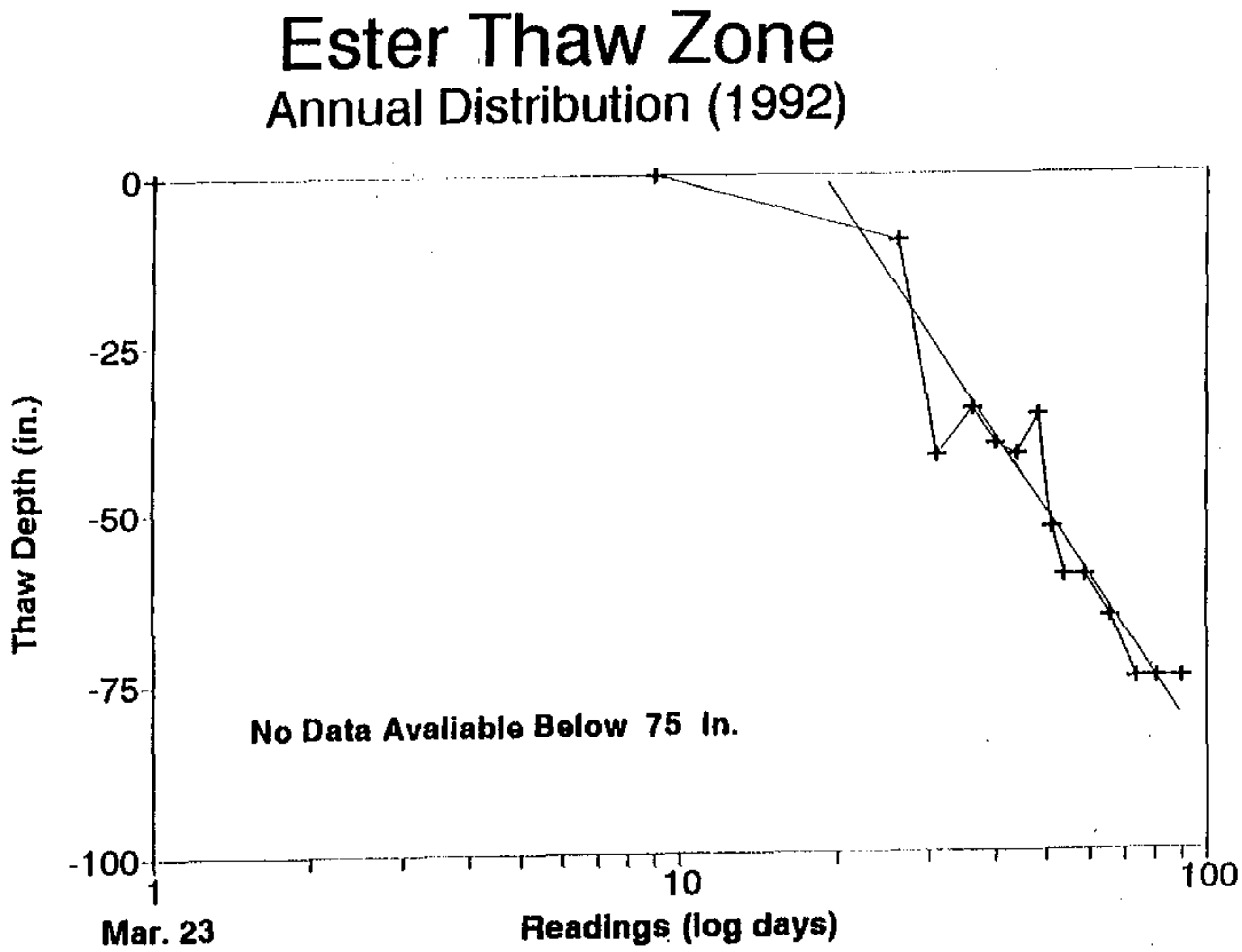
Annual Distribution (1992)



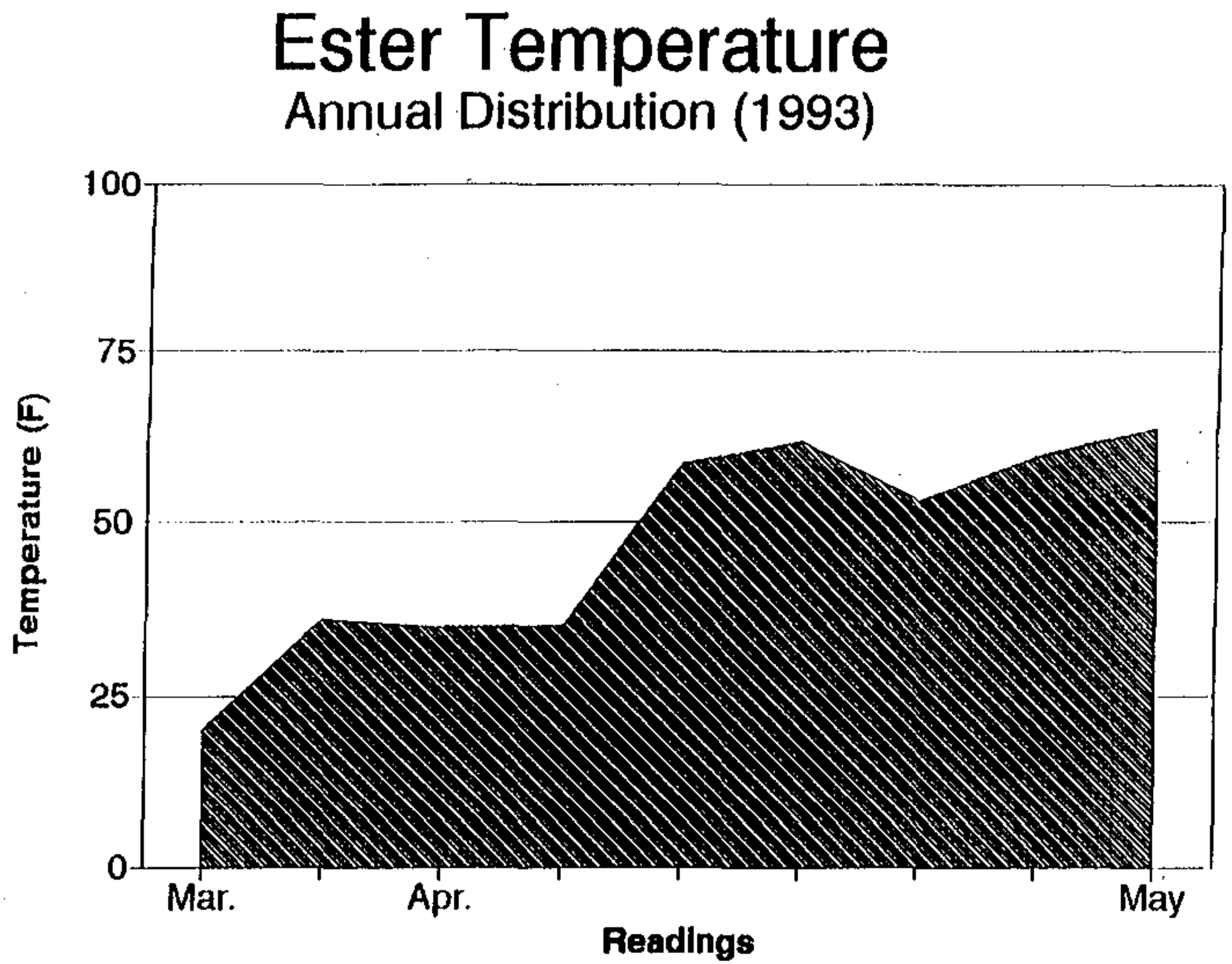
Ester Thaw Zone

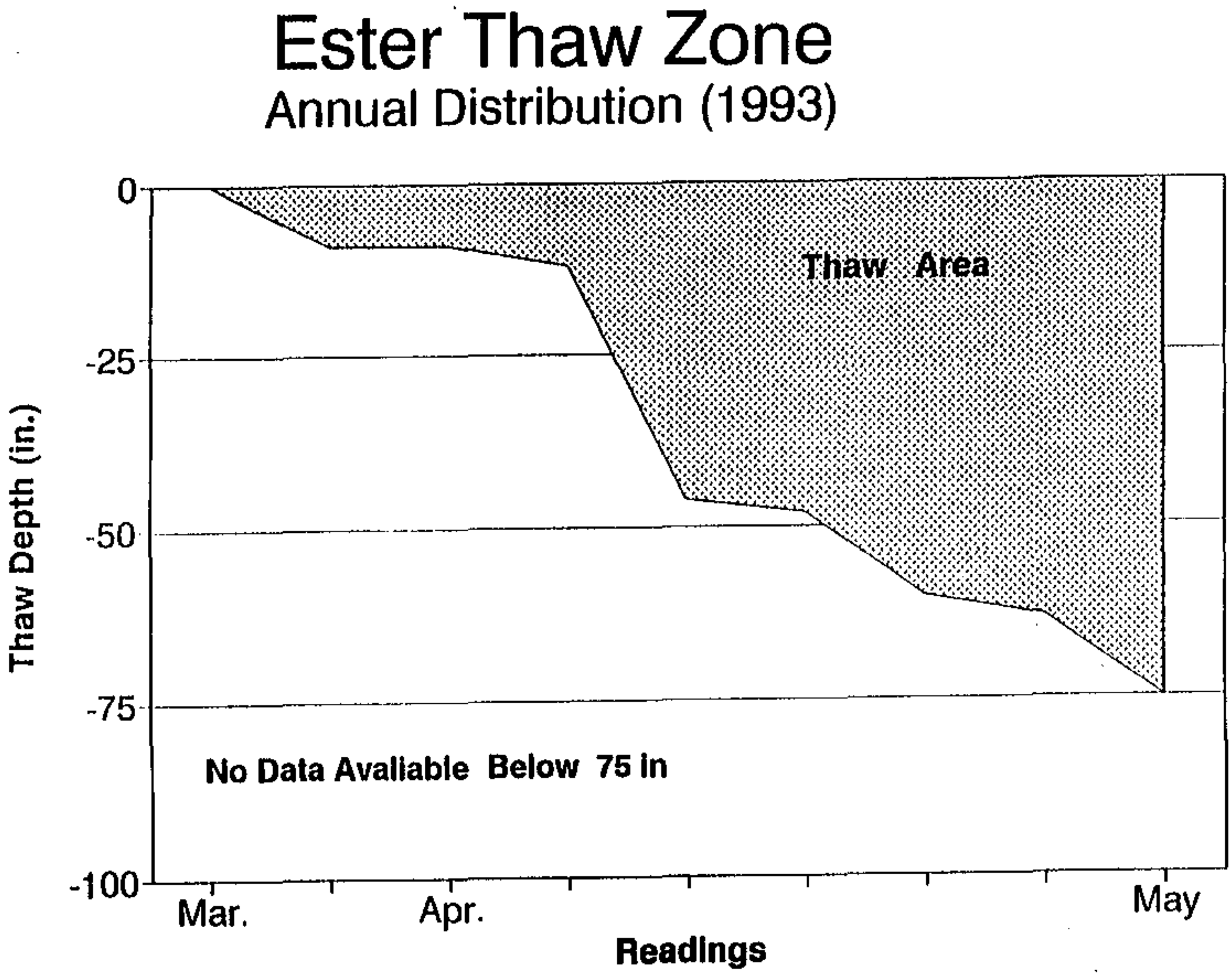
Annual Distribution (1992)





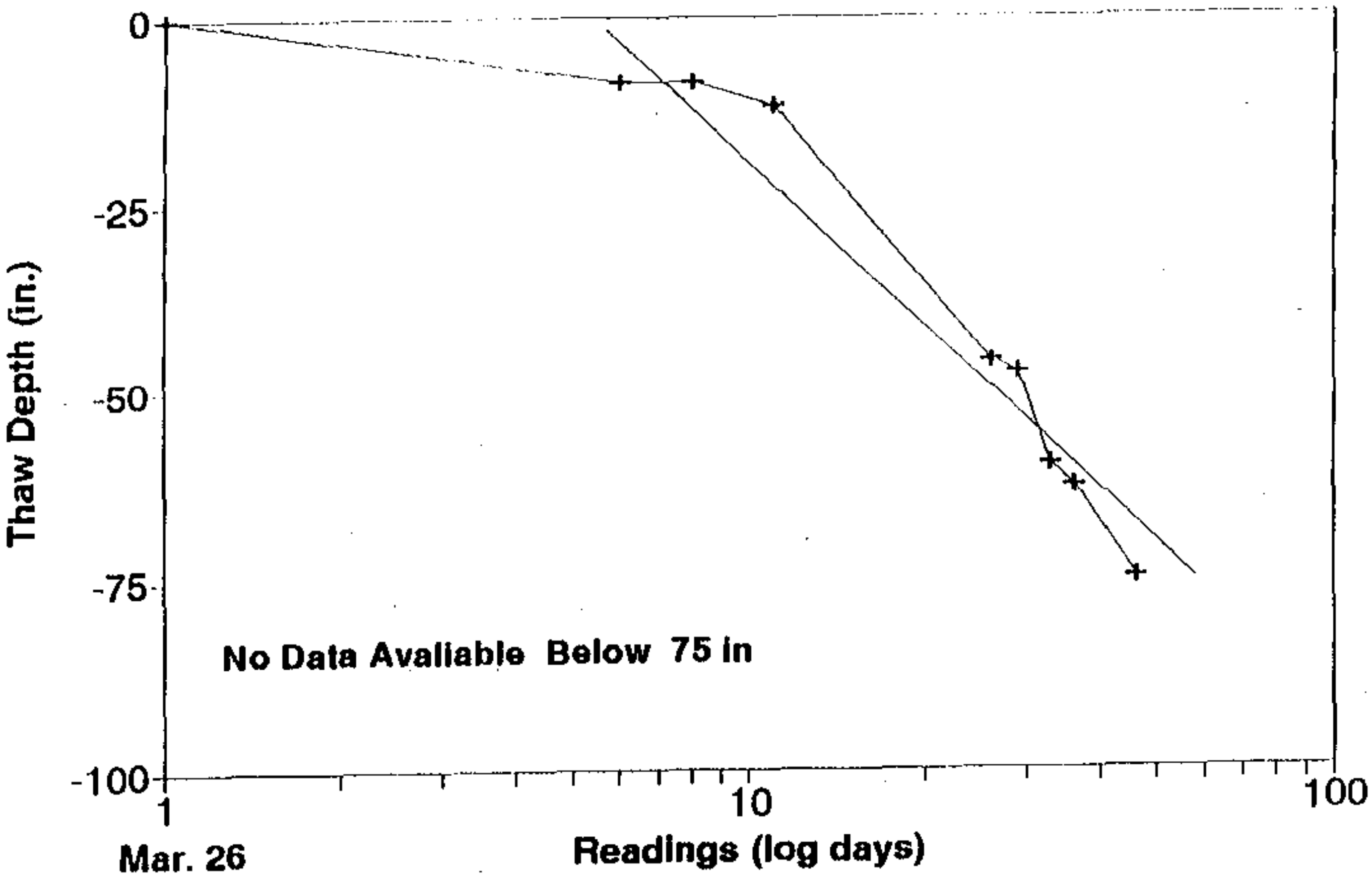
Ester Thaw Zone
Annual Distribution (1992)
Thaw Depth (in.)
0
-25
-50
-75
-100
No Data Avaliable Below 75 In.
1
10
100
Mar. 23
Readings (log days)

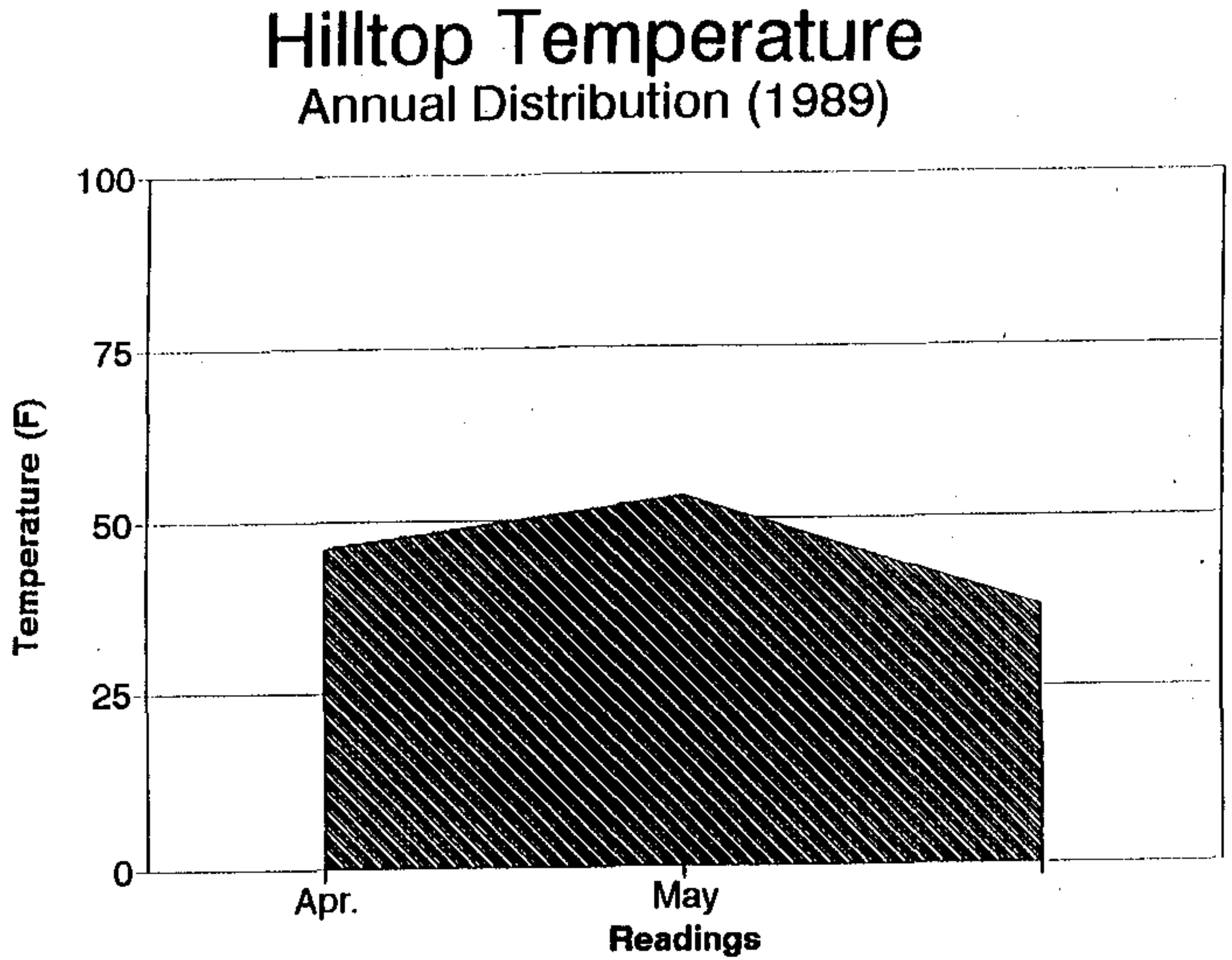

Ester Temperature
Annual Distribution (1993)
Temperature (F)
100
75
50
25
0
Mar.
Apr.
May
Readings

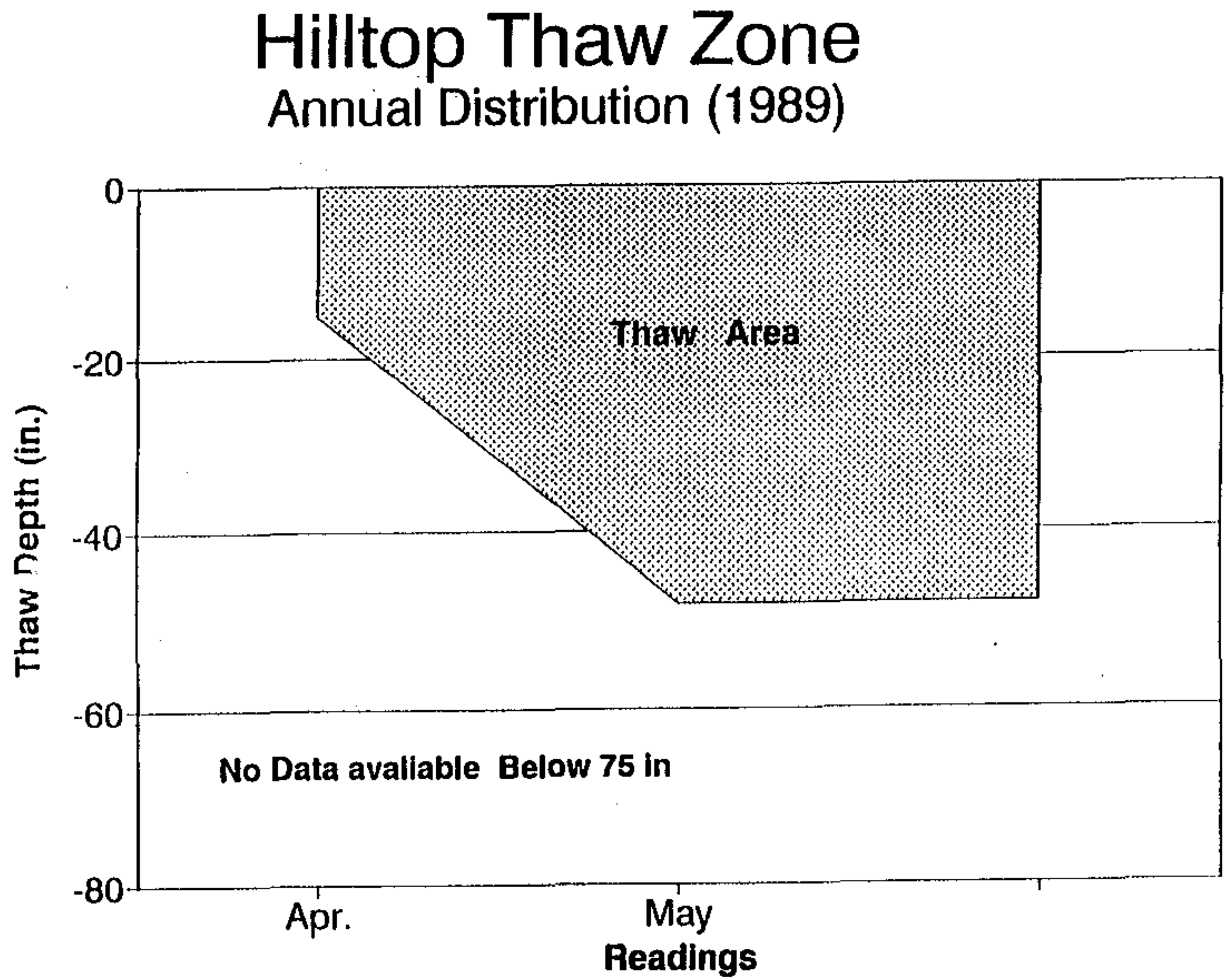

Ester Thaw Zone
Annual Distribution (1993)
0
-25
-50
-75
-100
Thaw Depth (in.)
Thaw Area
No Data Avaliable Below 75 in
Mar.
Apr.
May
Readings

Ester Thaw Zone

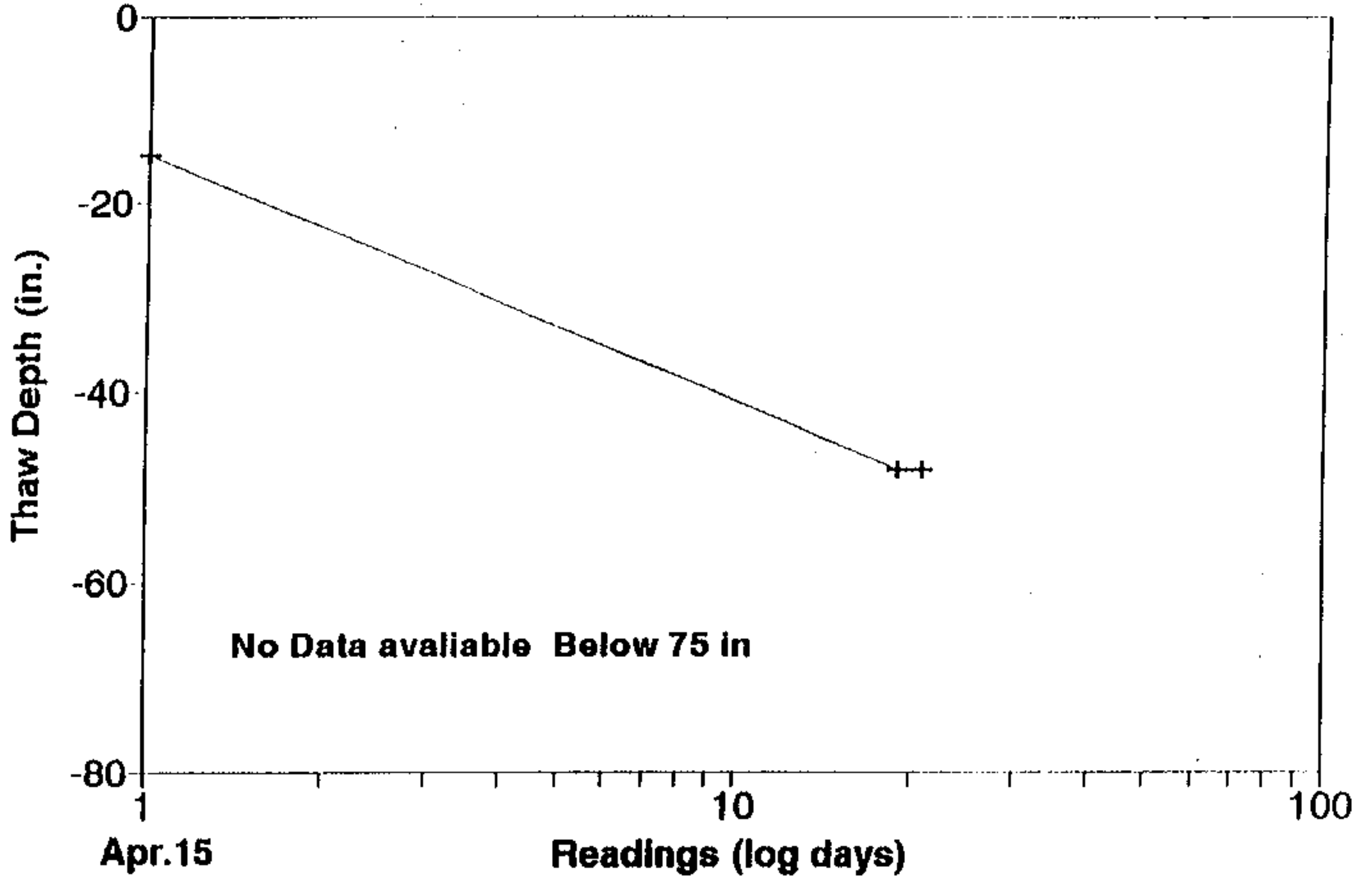
Annual Distribution (1993)




Hilltop Temperature
Annual Distribution (1989)
100
75
50
25
0
Temperature (F)
Apr.
May
Readings

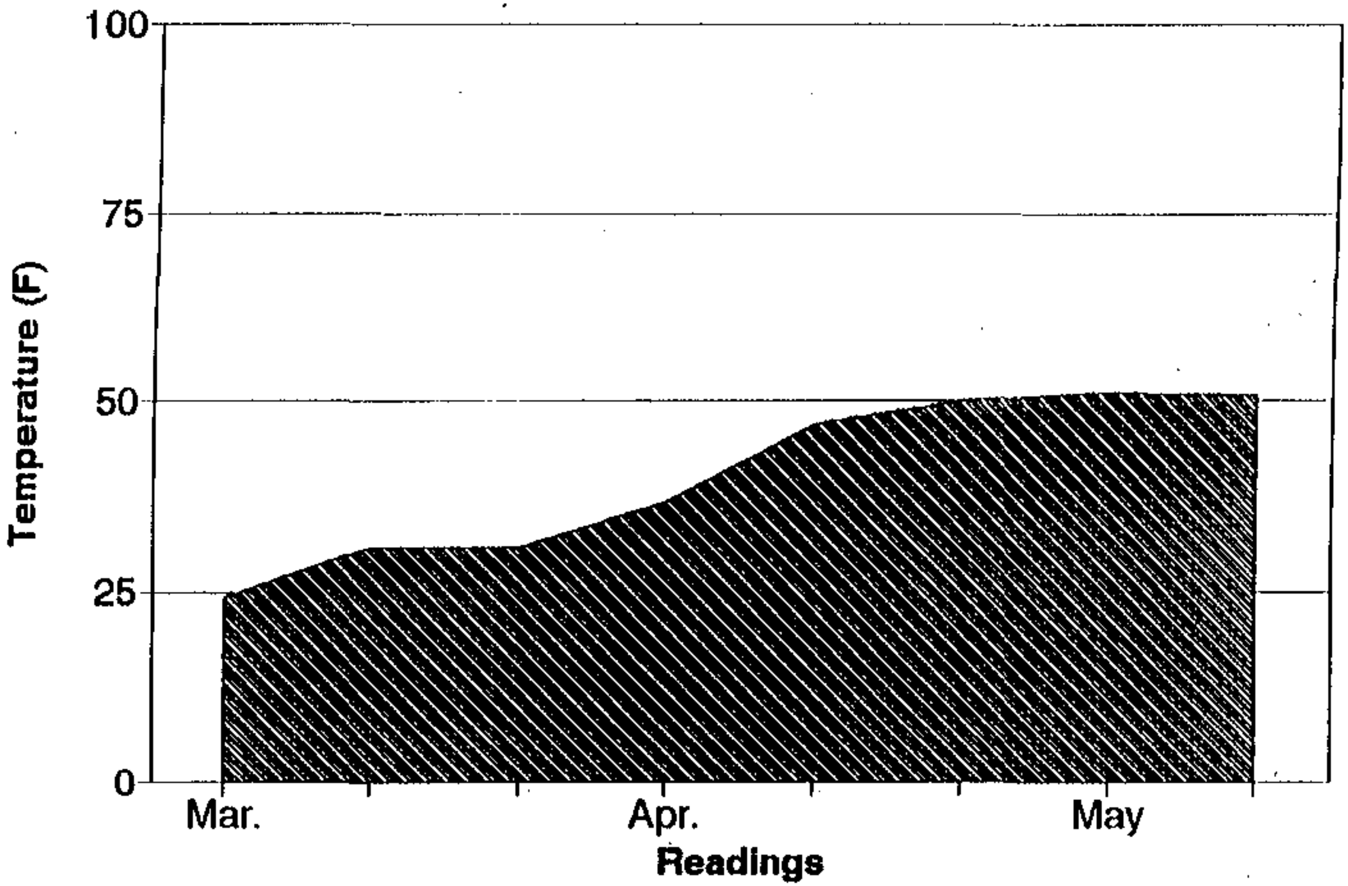

Hilltop Thaw Zone
Annual Distribution (1989)
0
-20
-40
-60
-80
Thaw Depth (in.)
Thaw Area
No Data available Below 75 in
Apr.
May
Readings

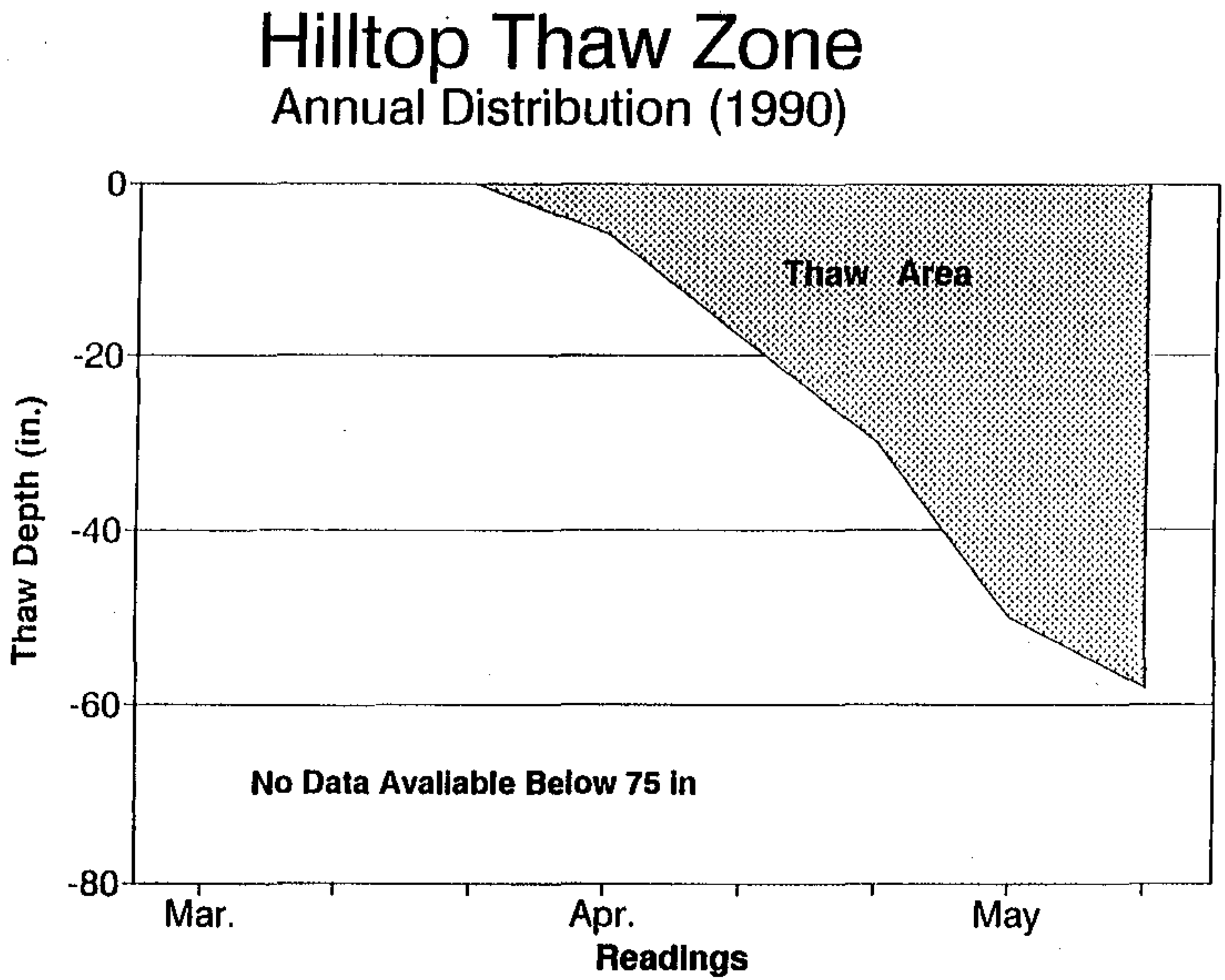
Hilltop Thaw Zone
Annual Distribution (1989)



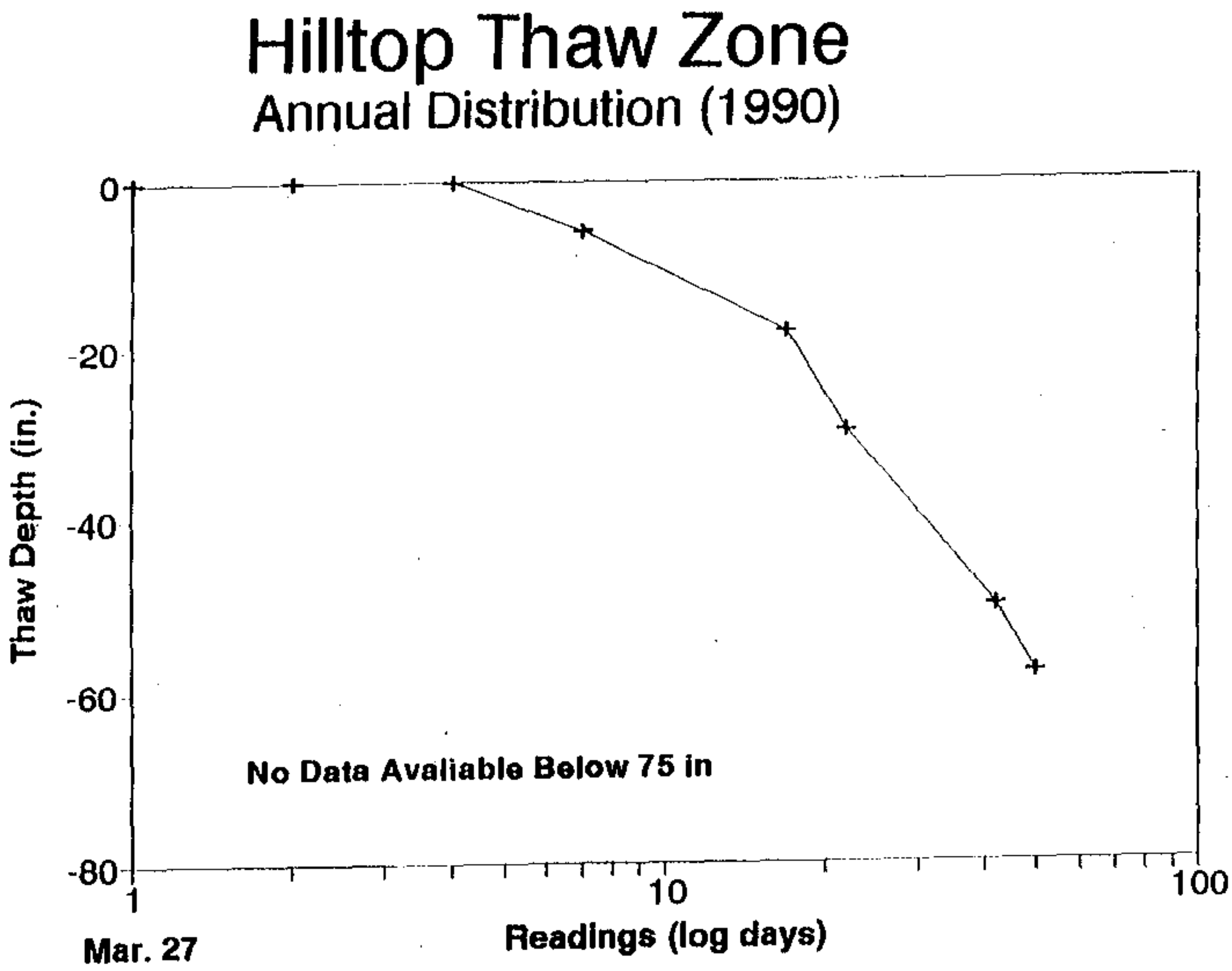
Hilltop Temperature

Annual Distribution (1990)





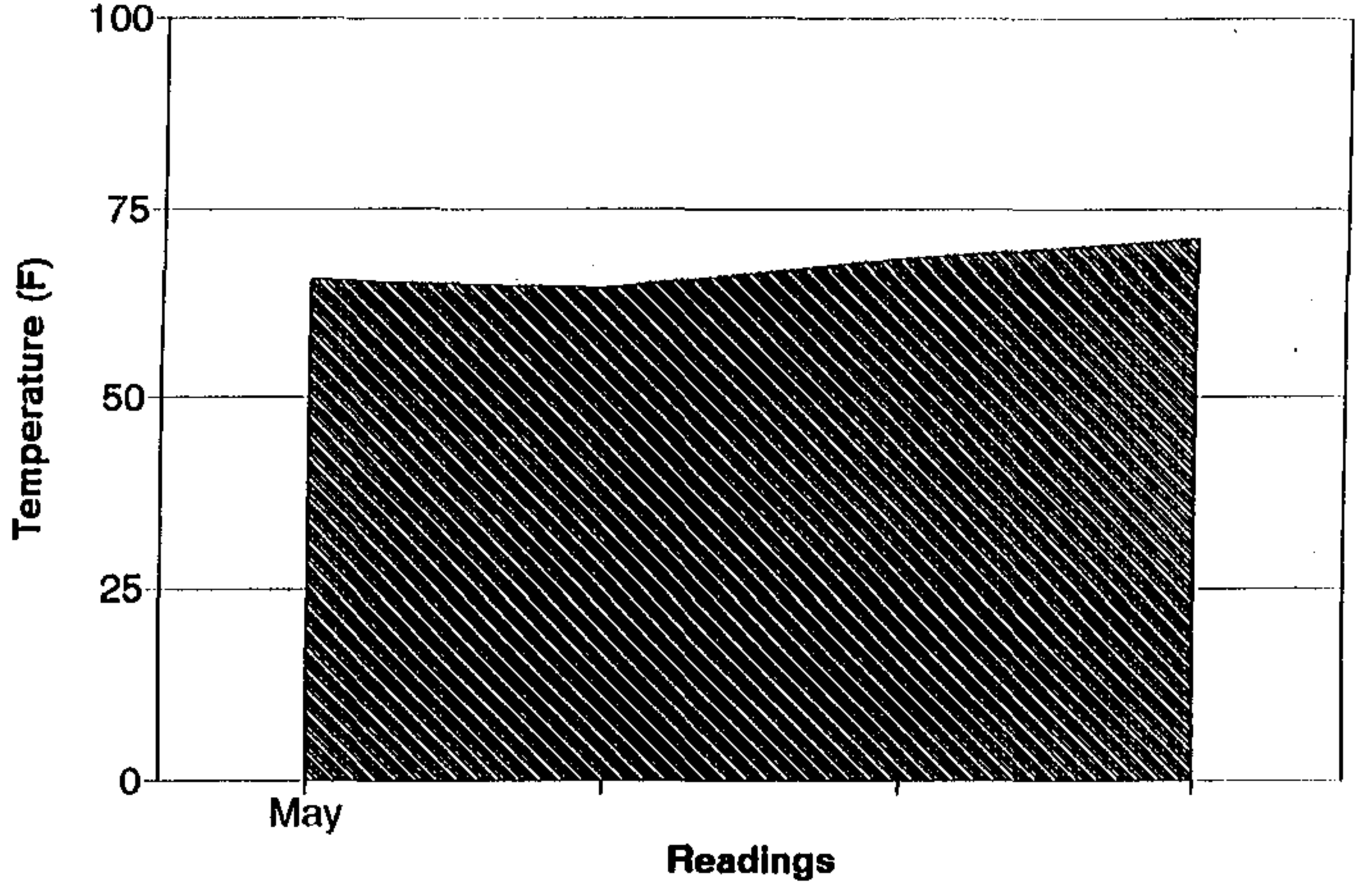
Hilltop Thaw Zone
Annual Distribution (1990)
0
-20
-40
-60
-80
Thaw Depth (in.)
Thaw Area
No Data Avaliable Below 75 in
Mar.
Apr.
May
Readings

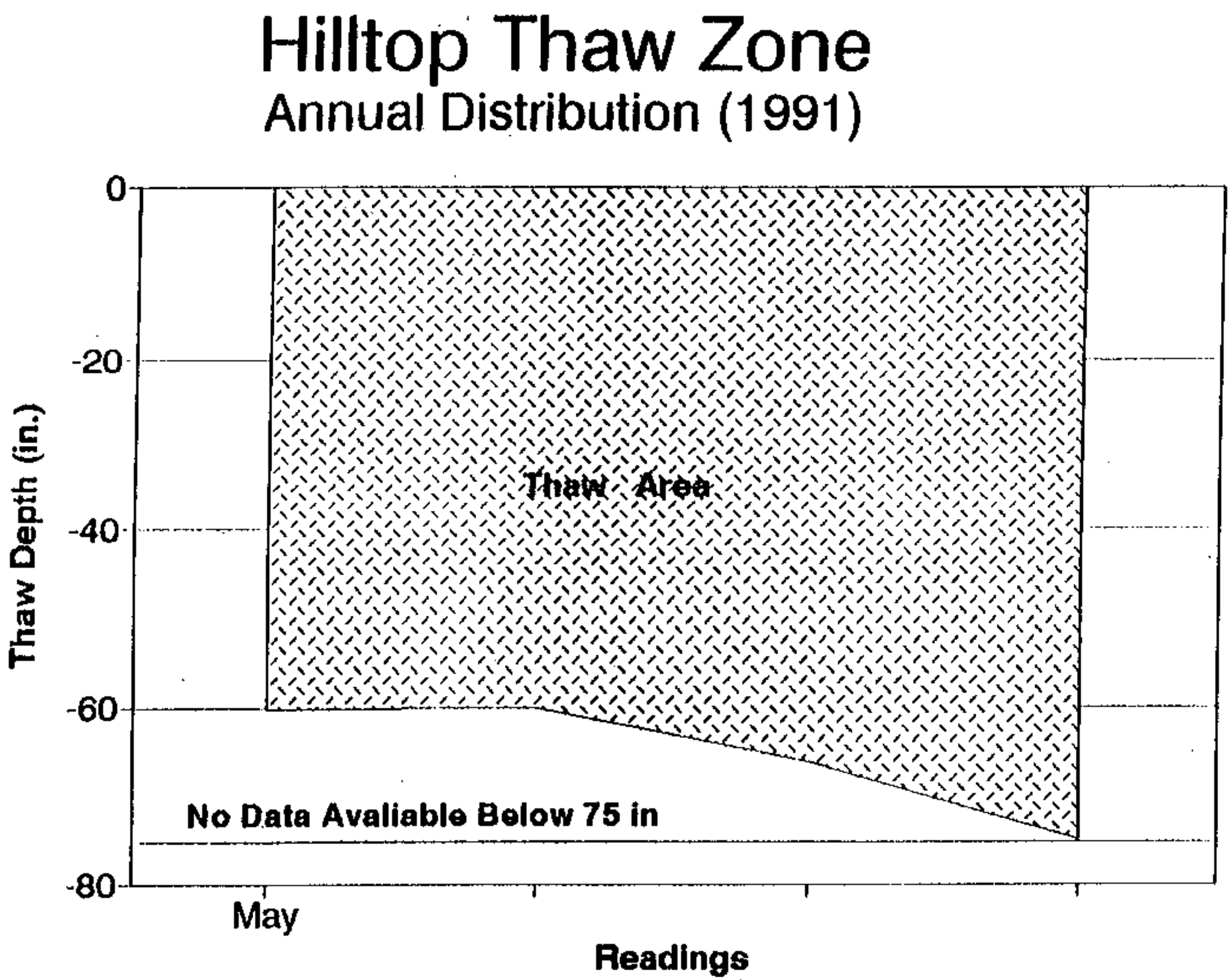


Hilltop Thaw Zone
Annual Distribution (1990)
0
-20
-40
-60
-80
Thaw Depth (in.)
No Data Avaliable Below 75 in
1
10
100
Mar. 27
Readings (log days)

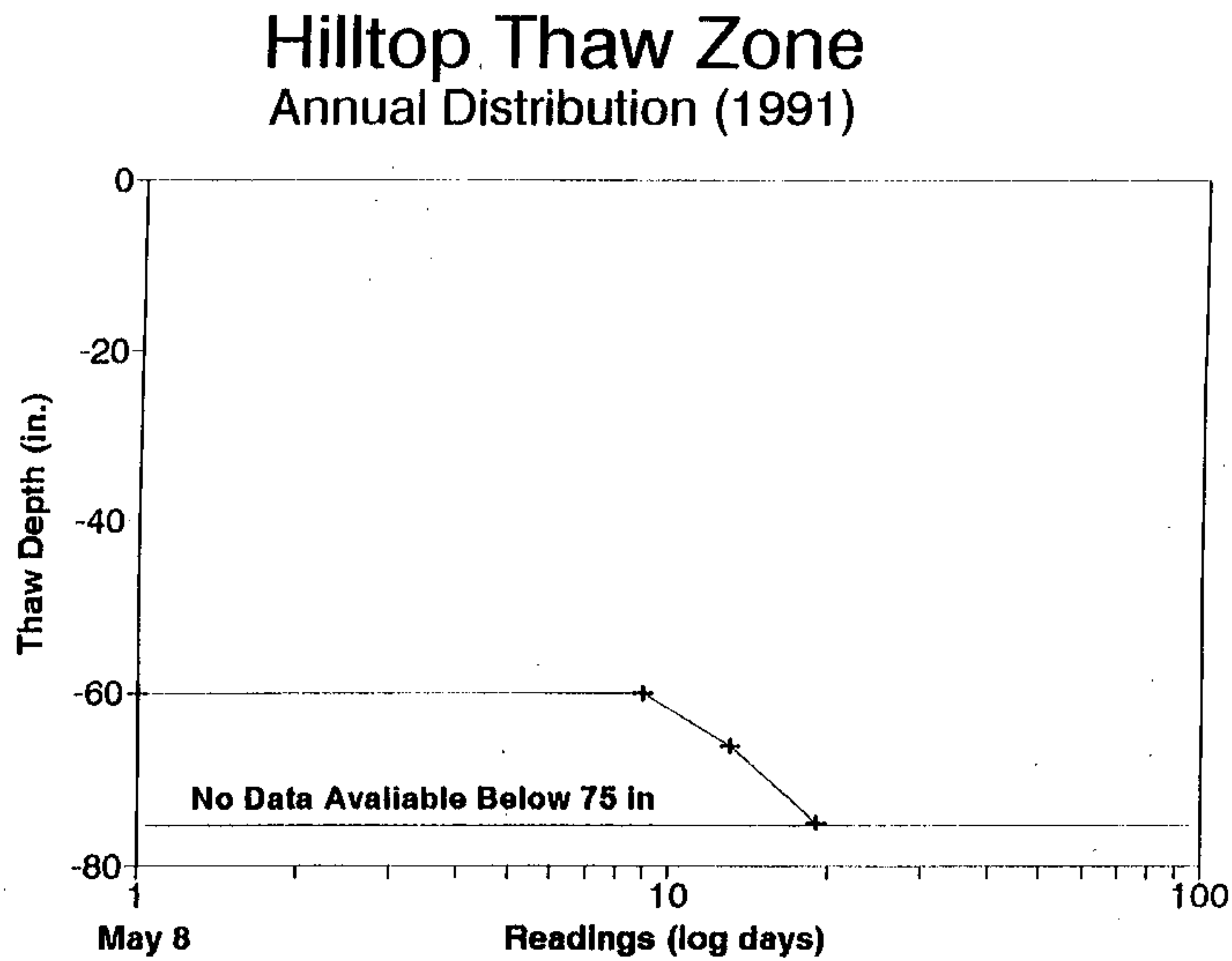
Hilltop Temperature

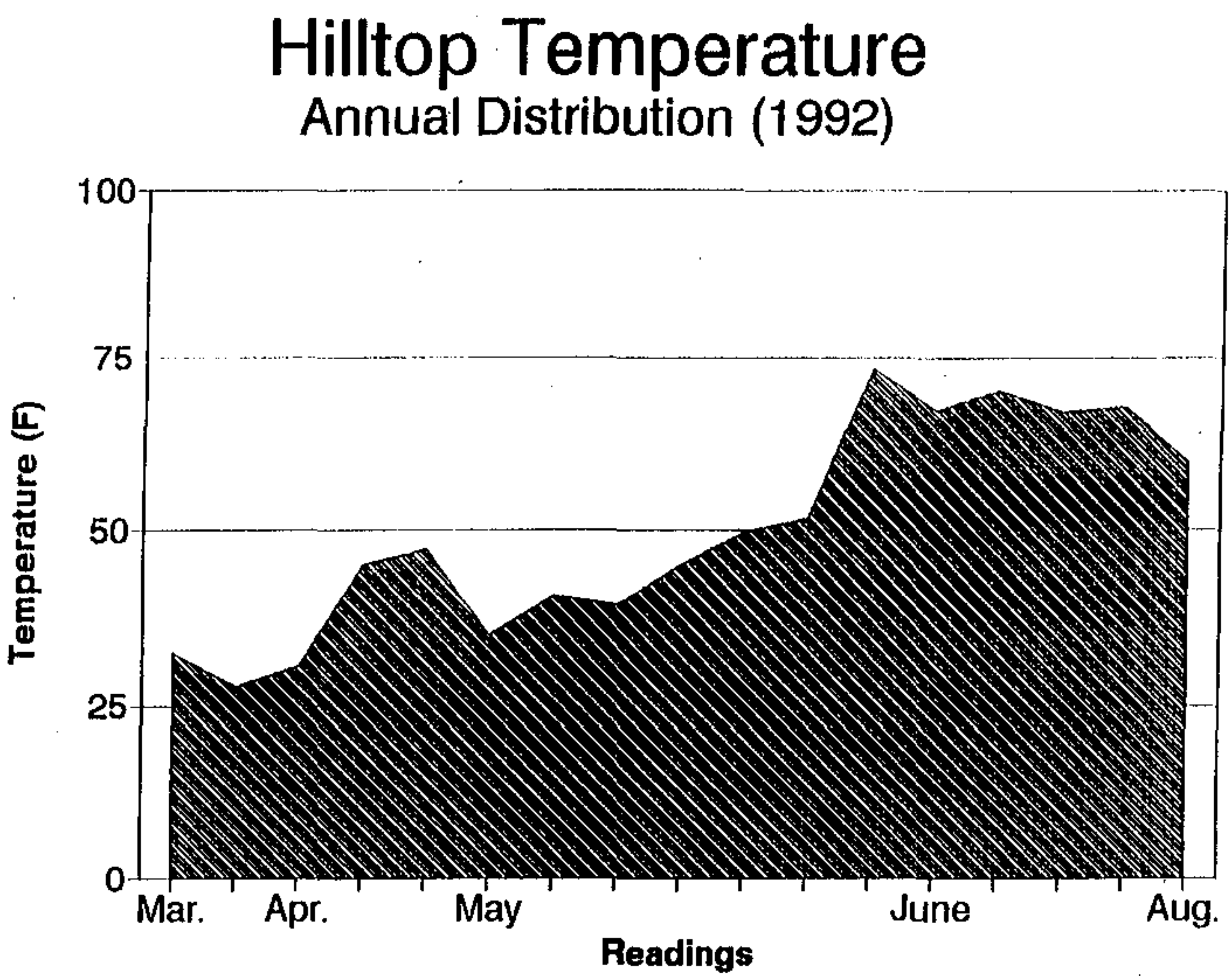
Annual Distribution (1991)





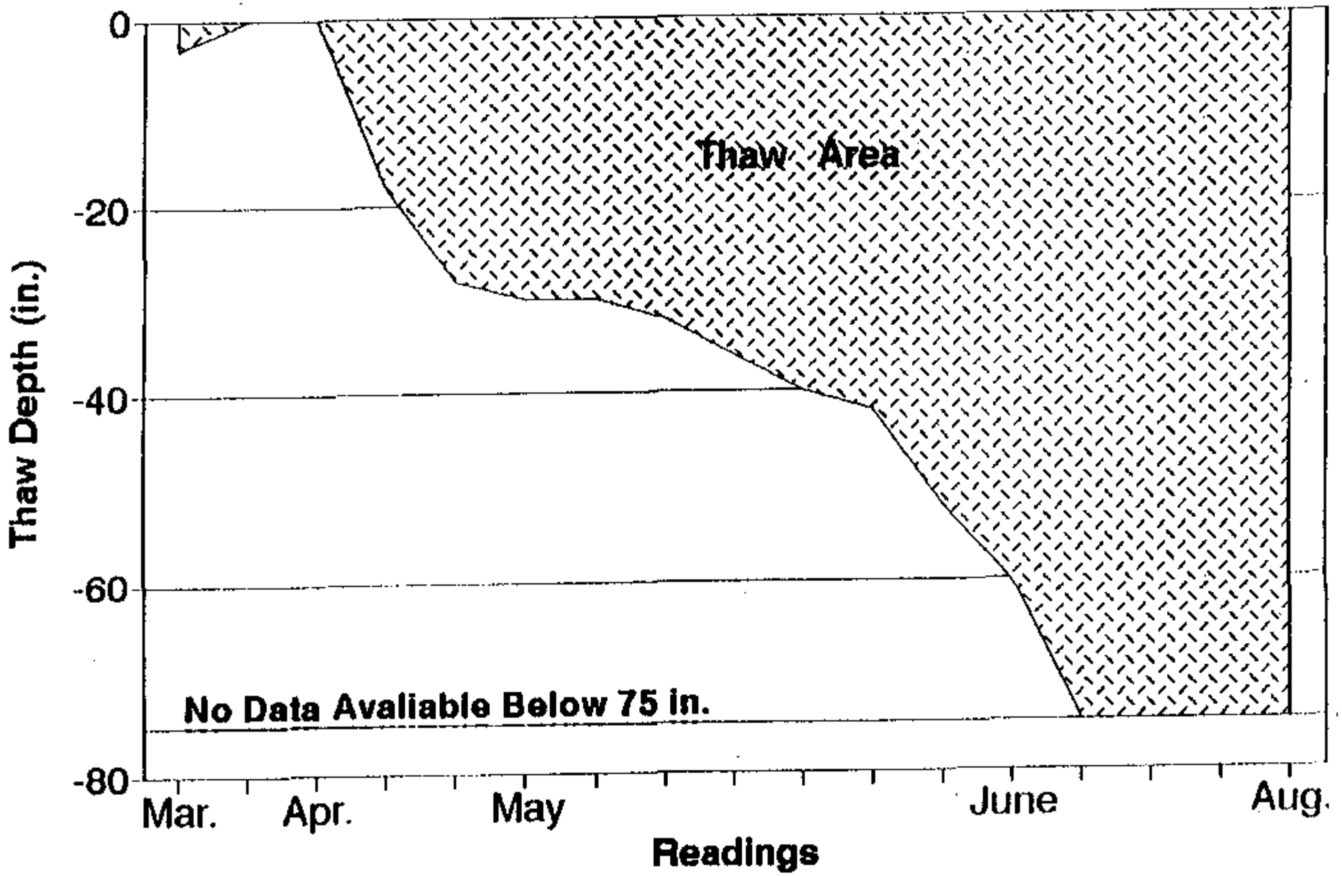
Hilltop Thaw Zone
Annual Distribution (1991)
0
-20
-40
-60
-80
Thaw Depth (in.)
Thaw Area
No Data Avaliable Below 75 in
May
Readings

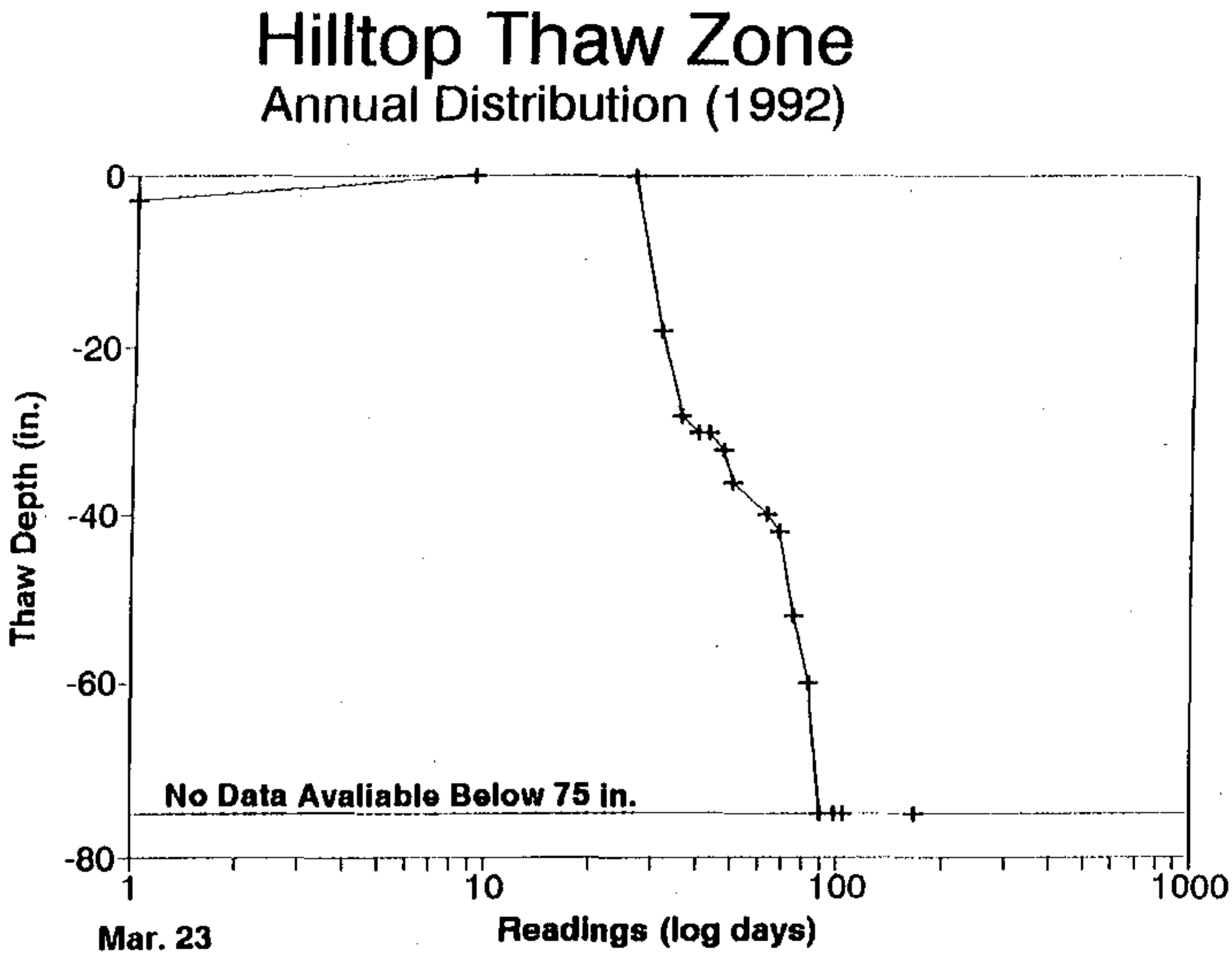

Hilltop Thaw Zone
Annual Distribution (1991)
Thaw Depth (in.)
0
-20
-40
-60
-80
No Data Avaliable Below 75 in
1
10
100
May 8
Readings (log days)


Hilltop Temperature
Annual Distribution (1992)
100
75
50
25
0
Temperature (F)
Mar.
Apr.
May
June
Aug.
Readings

Hilltop Thaw Zone

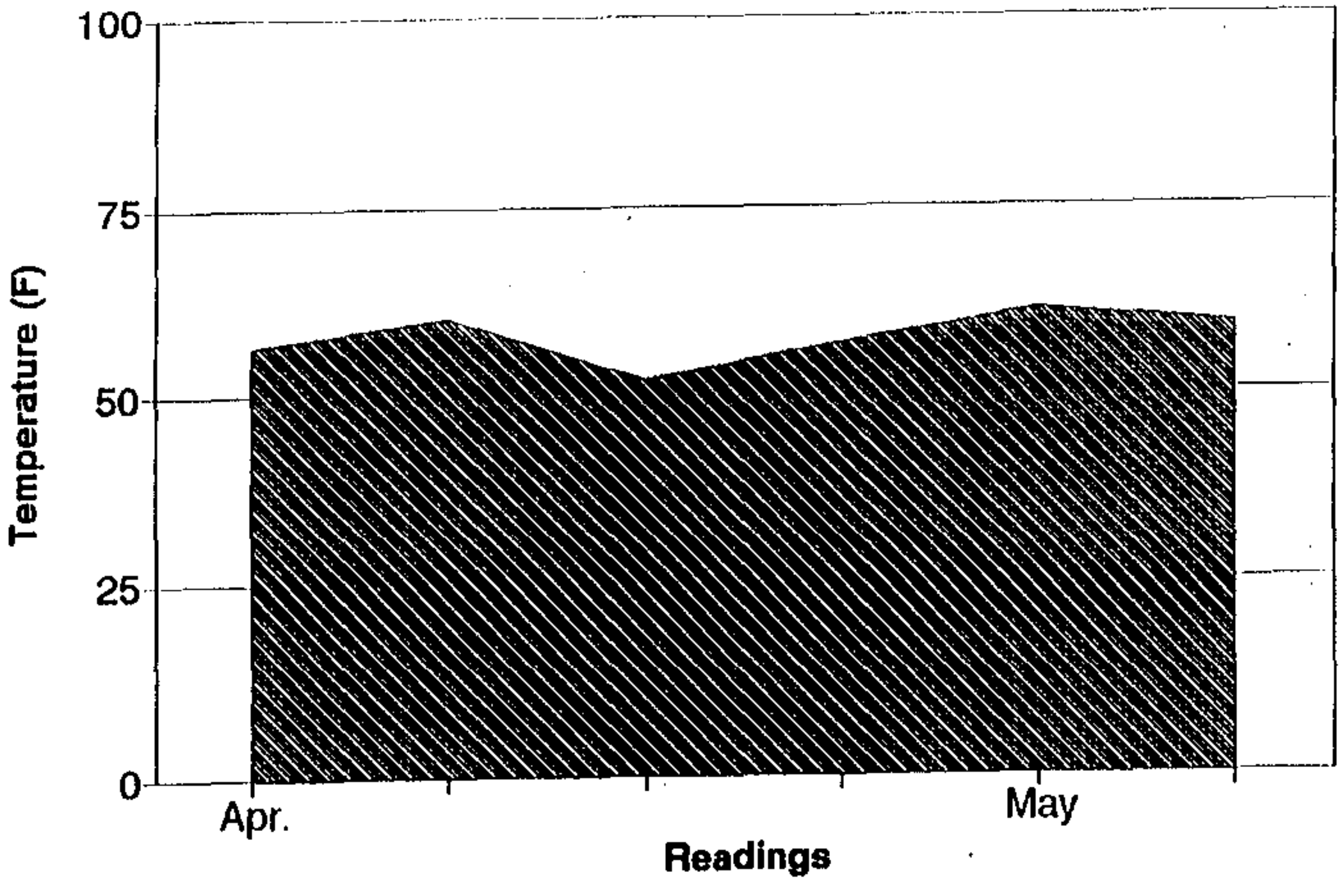
Annual Distribution (1992)

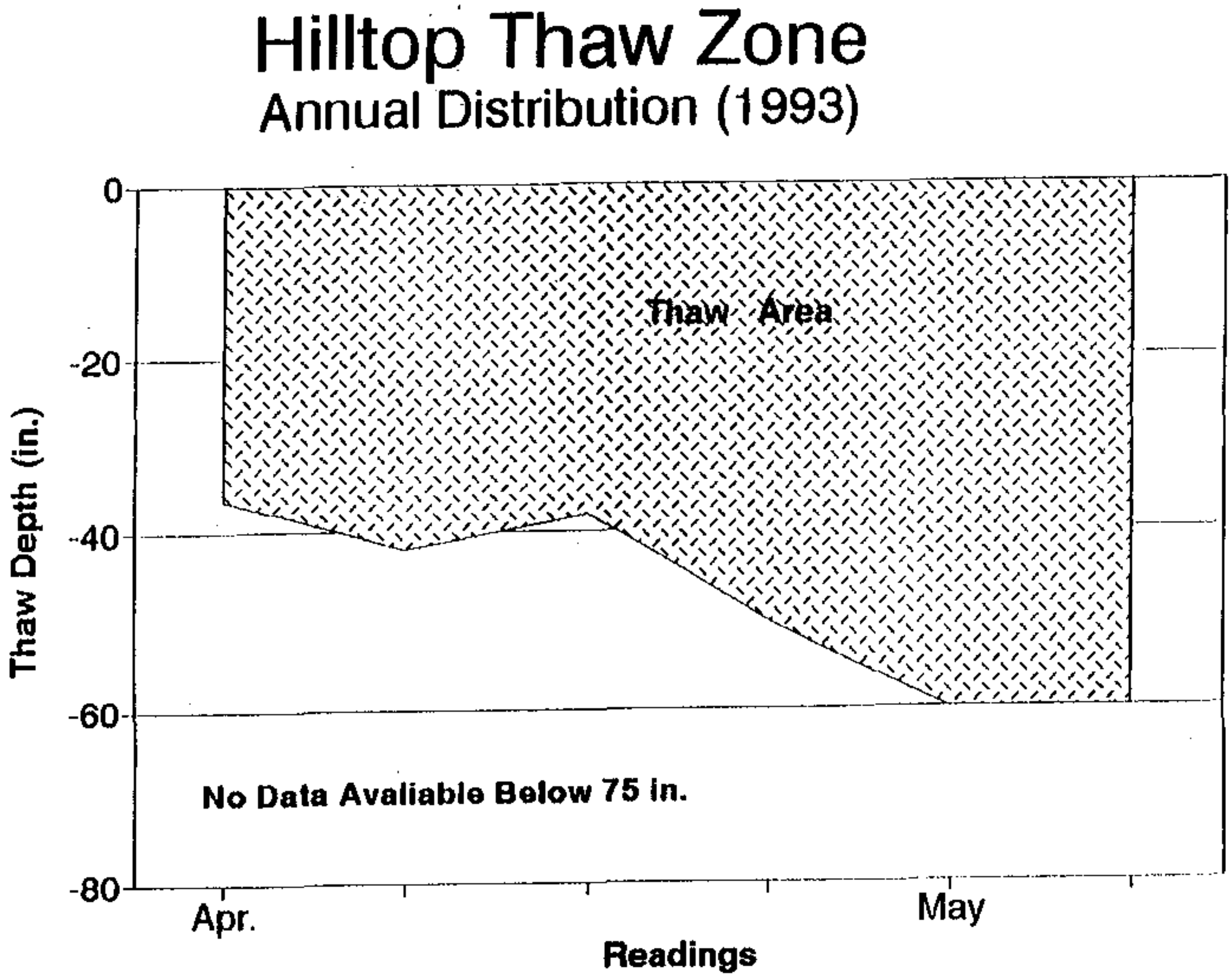


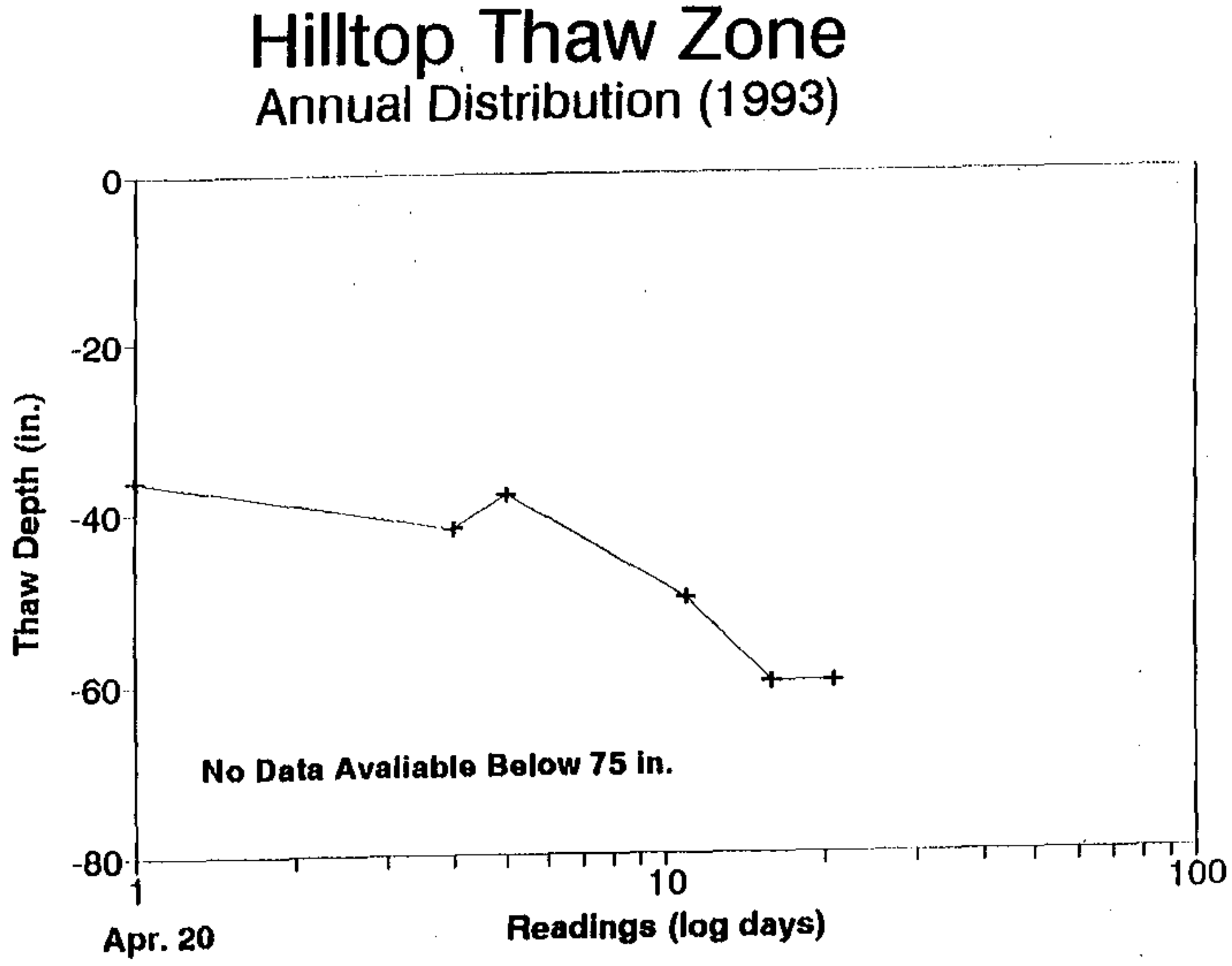

Hilltop Thaw Zone
Annual Distribution (1992)
Thaw Depth (in.)
0
-20
-40
-60
-80
No Data Avaliable Below 75 in.
1
10
100
1000
Mar. 23
Readings (log days)

Hilltop Temperature

Annual Distribution (1993)



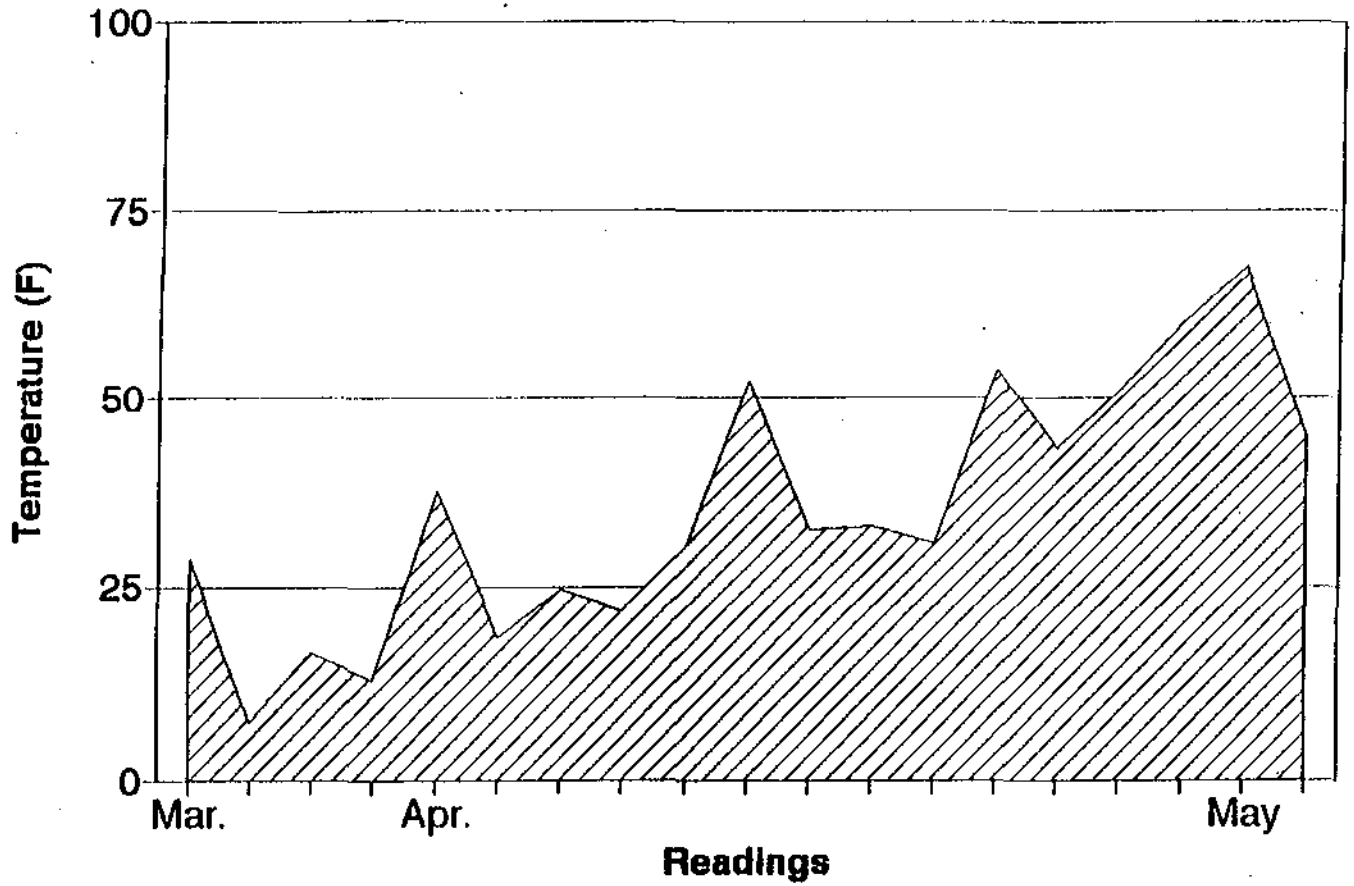

Hilltop Thaw Zone
Annual Distribution (1993)
Thaw Area
No Data Avaliable Below 75 in.
Thaw Depth (in.)
0
-20
-40
-60
-80
Apr.
May
Readings



Hilltop Thaw Zone
Annual Distribution (1993)
0
-20
-40
-60
-80
Thaw Depth (in.)
No Data Avaliable Below 75 in.
1
10
100
Apr. 20
Readings (log days)

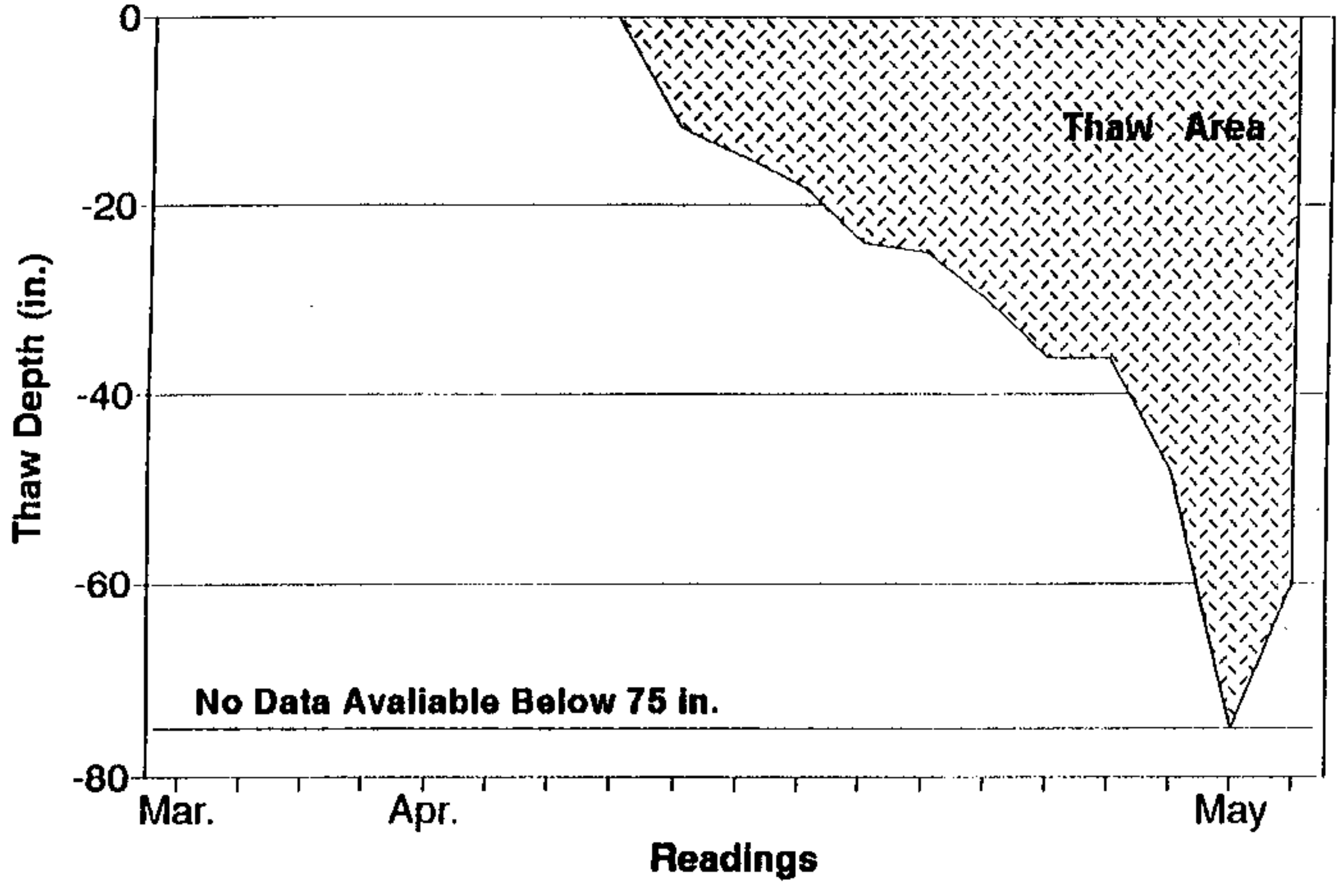
Peger Temperature

Annual Distribution (1989)



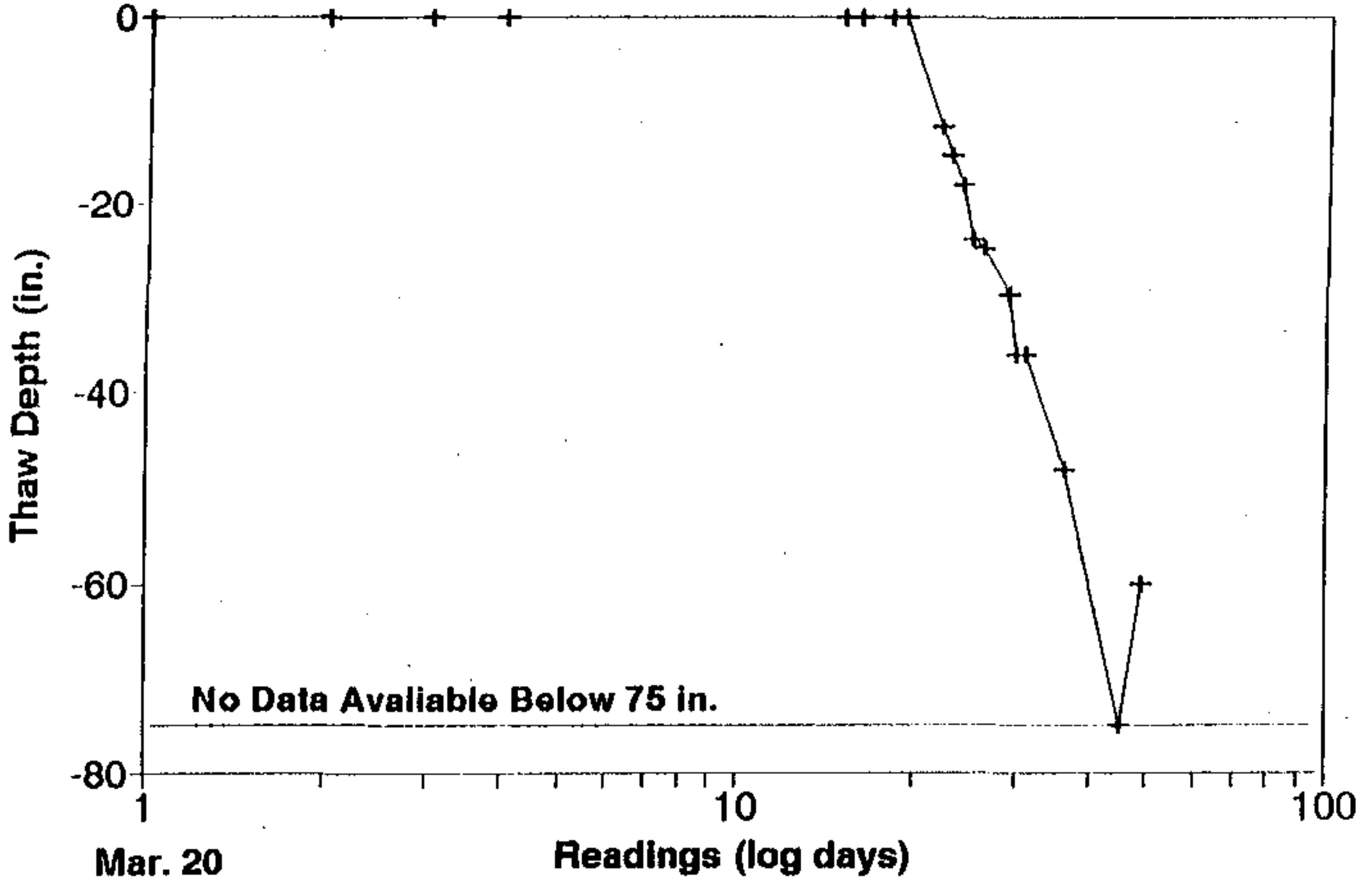
Peger Thaw Zone

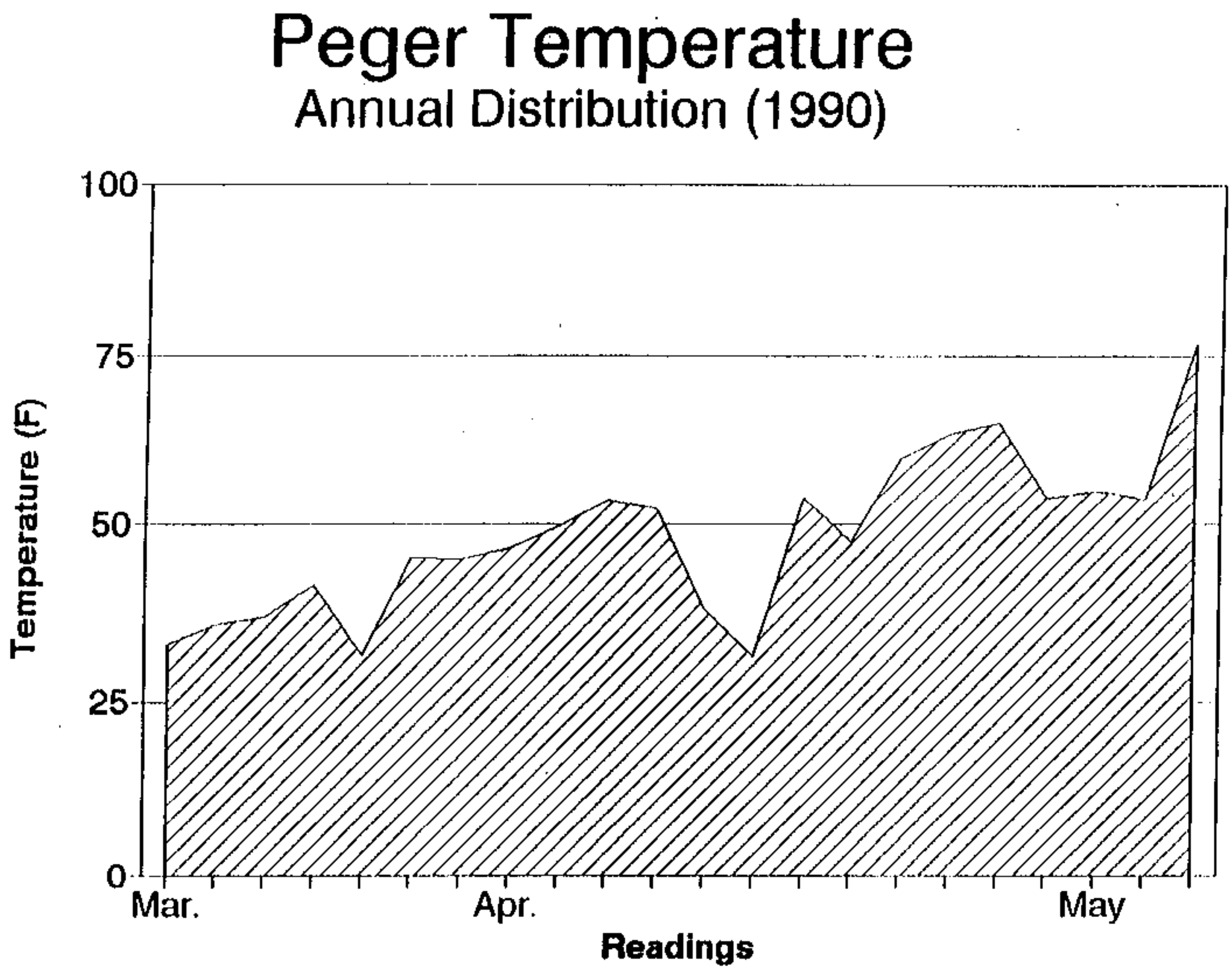
Annual Distribution (1989)



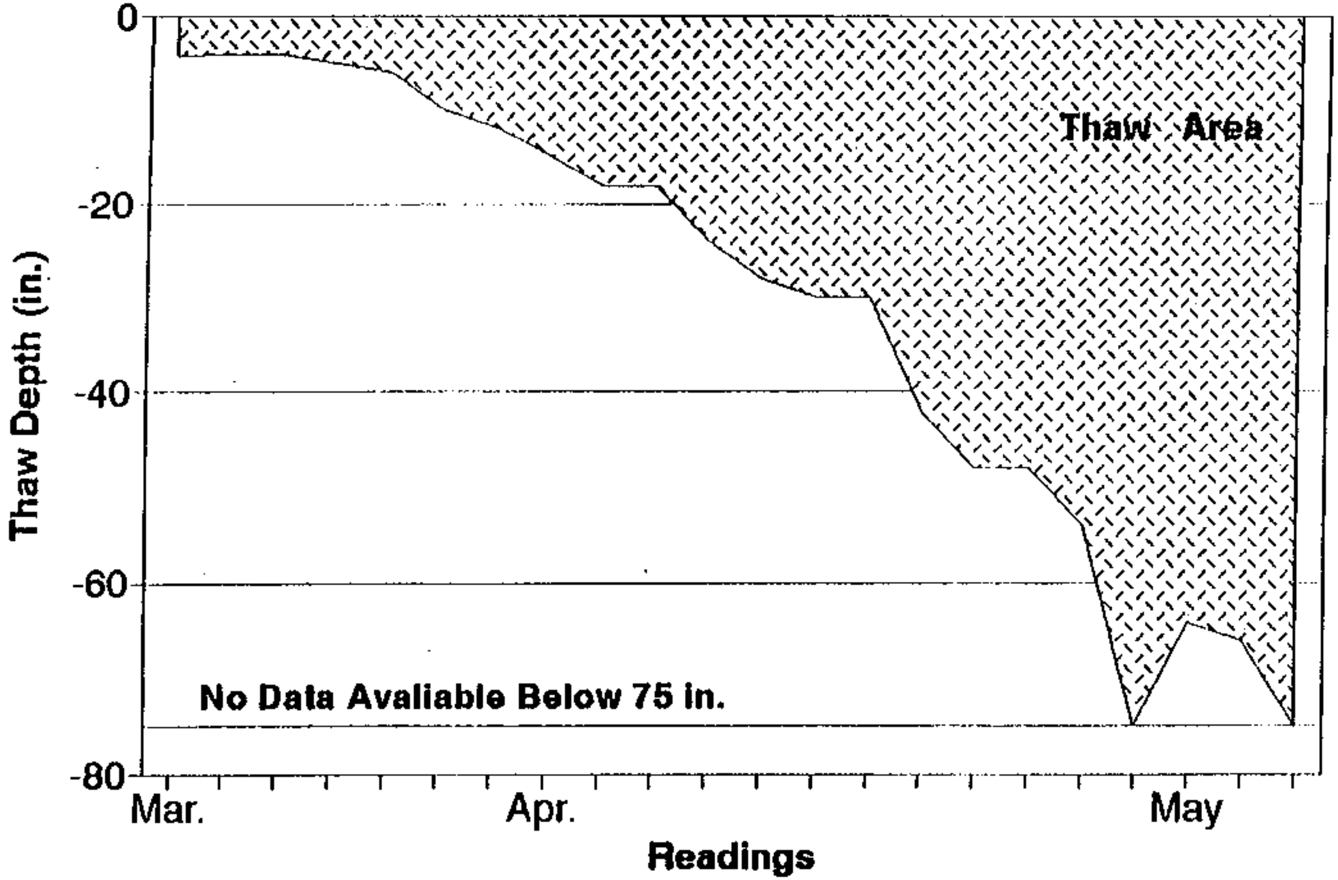
Peger Thaw Zone

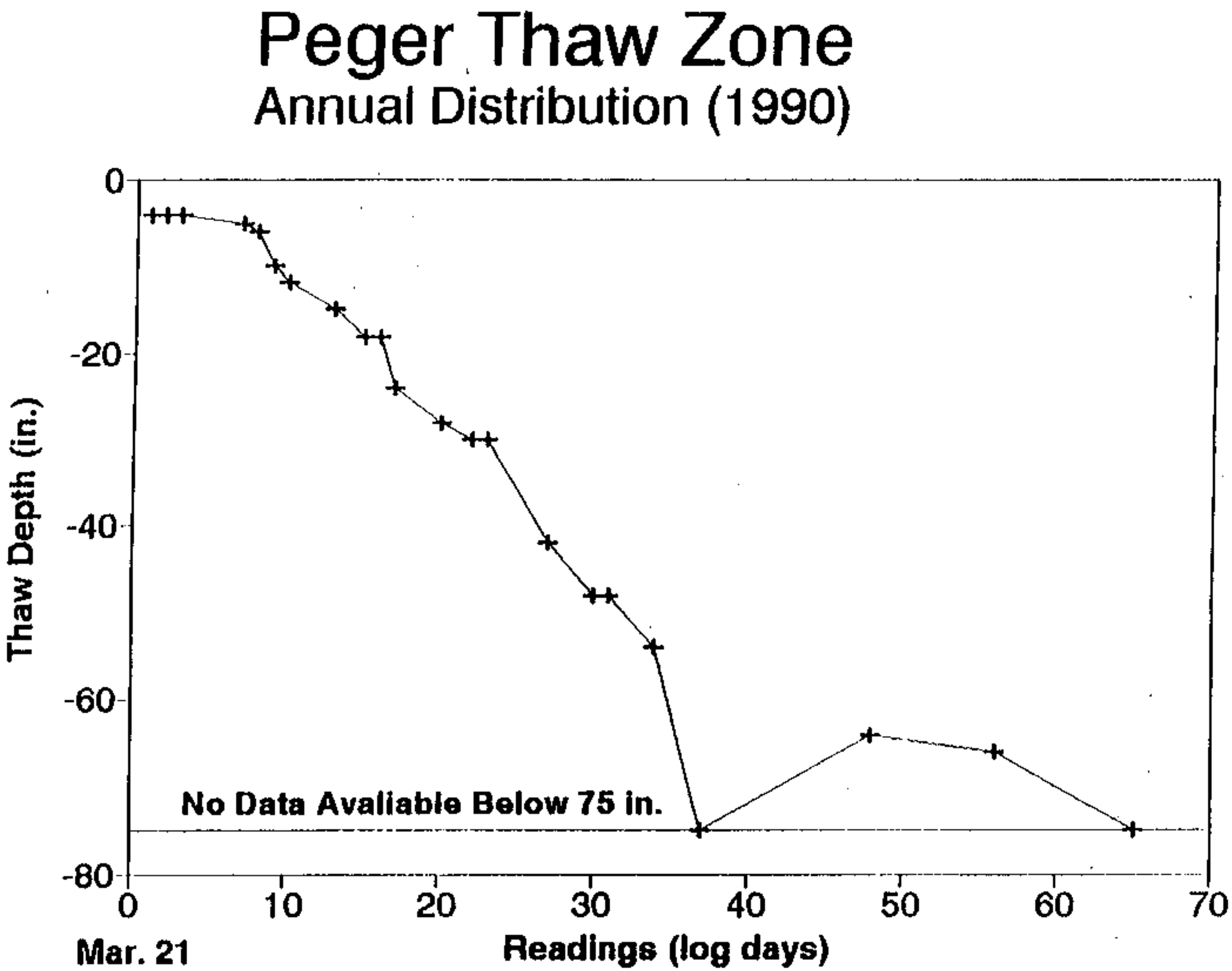
Annual Distribution (1989)



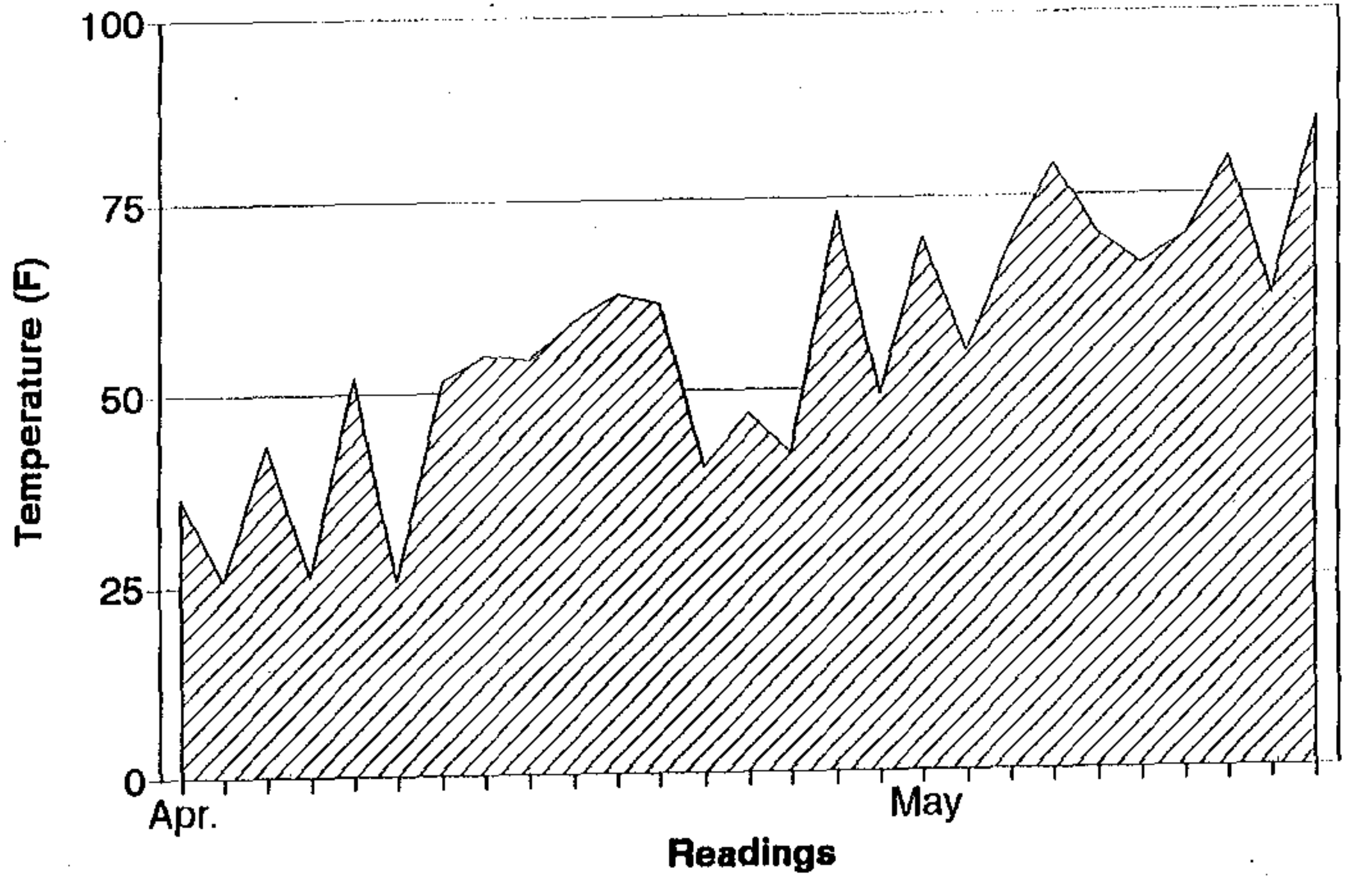

Peger Temperature
Annual Distribution (1990)
100
75
50
25
0
Temperature (F)
Mar.
Apr.
May
Readings

Peger Thaw Zone
Annual Distribution (1990)



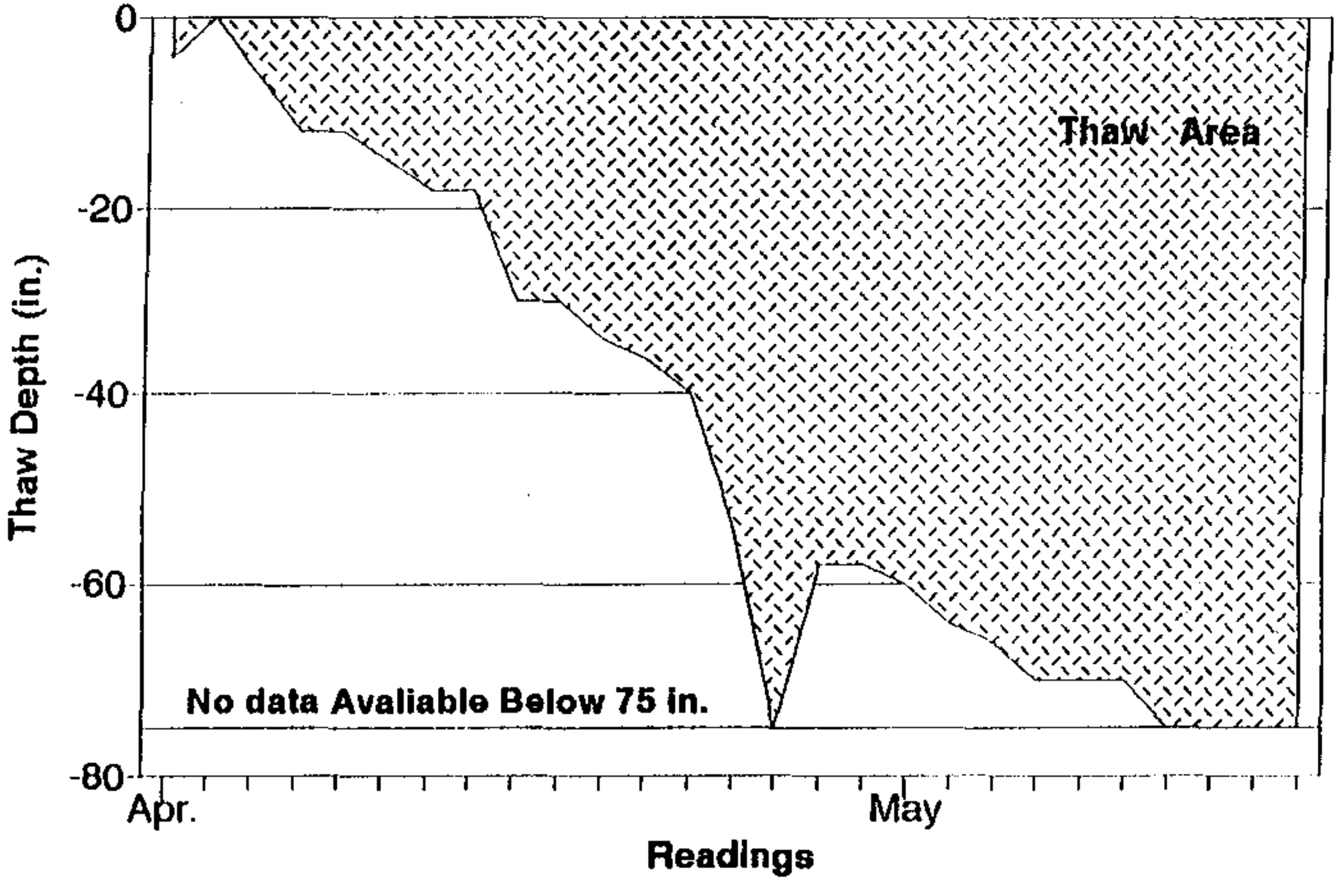

Peger Thaw Zone
Annual Distribution (1990)
Thaw Depth (in.)
0
-20
-40
-60
-80
No Data Avaliable Below 75 in.
0
10
20
30
40
50
60
70
Mar. 21
Readings (log days)

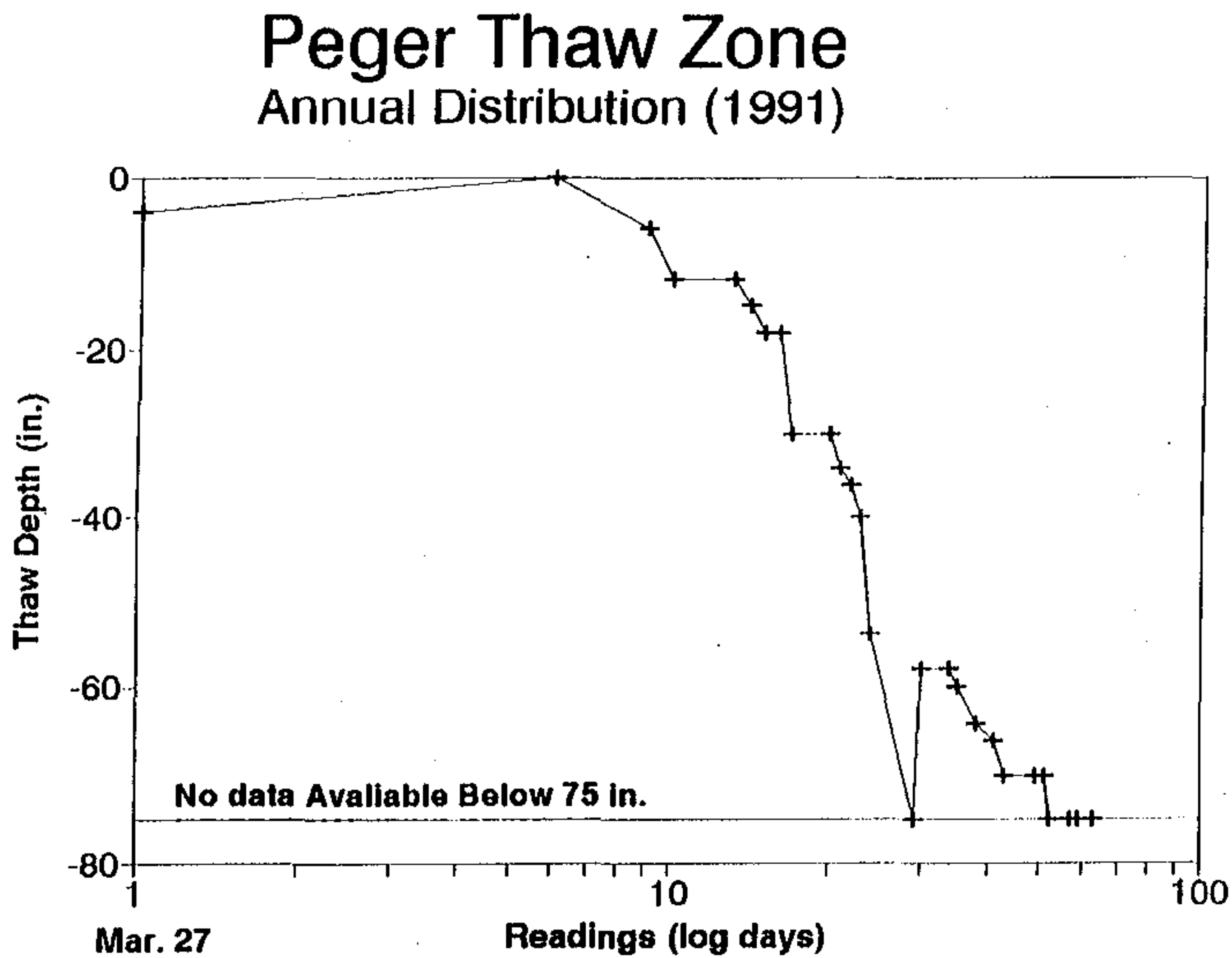
Peger Temperature
Annual Distribution (1991)



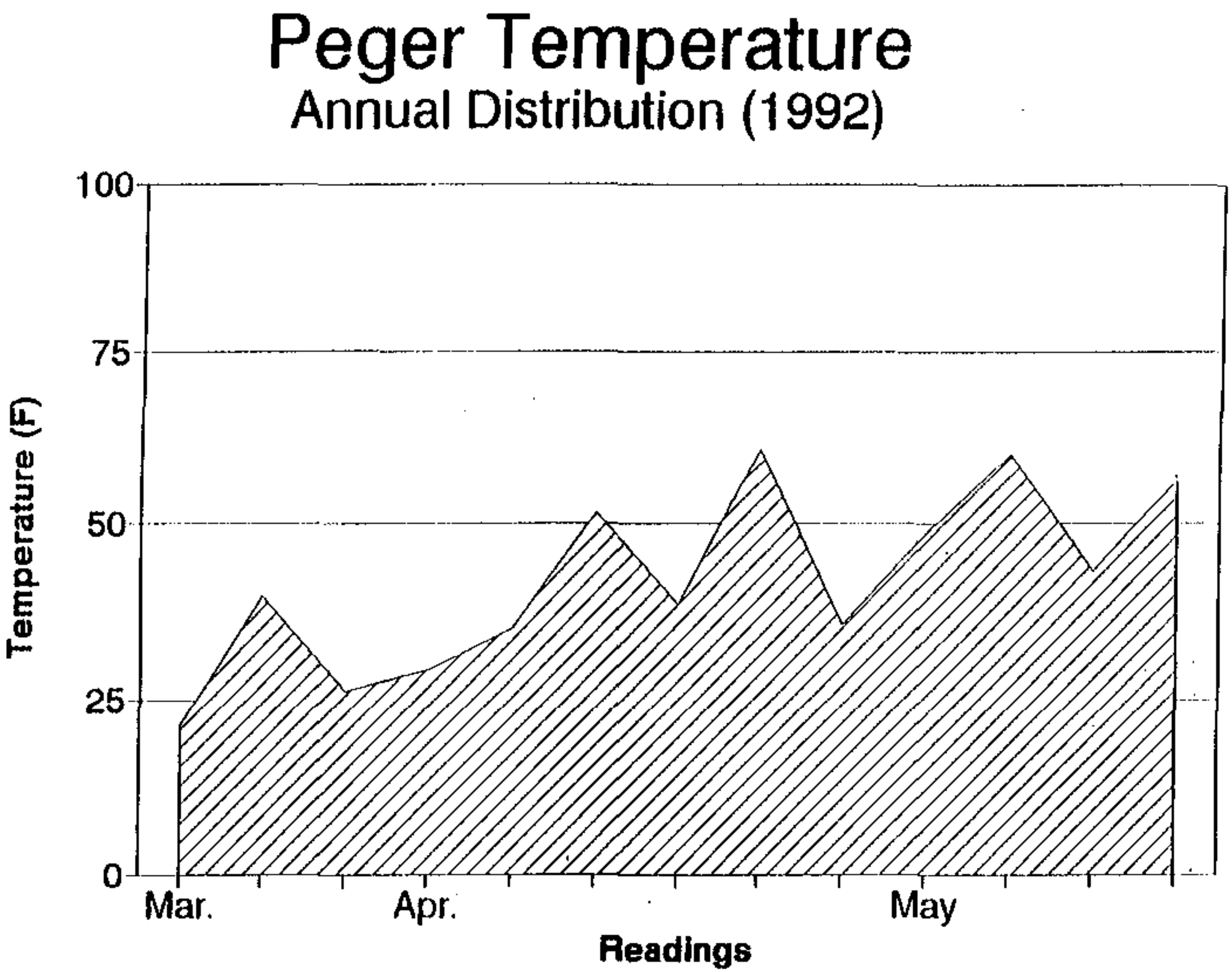
Peger Thaw Zone

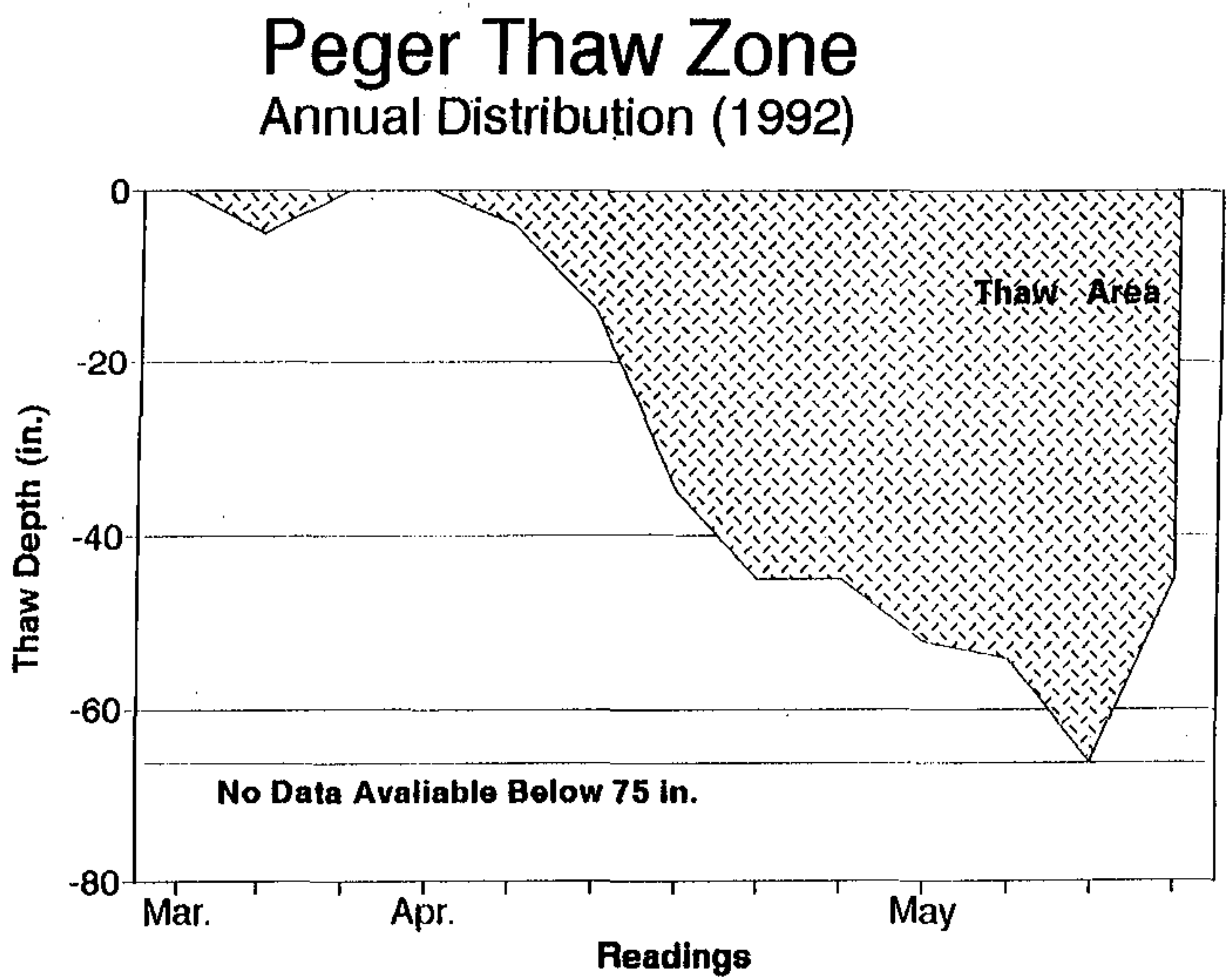
Annual Distribution (1991)



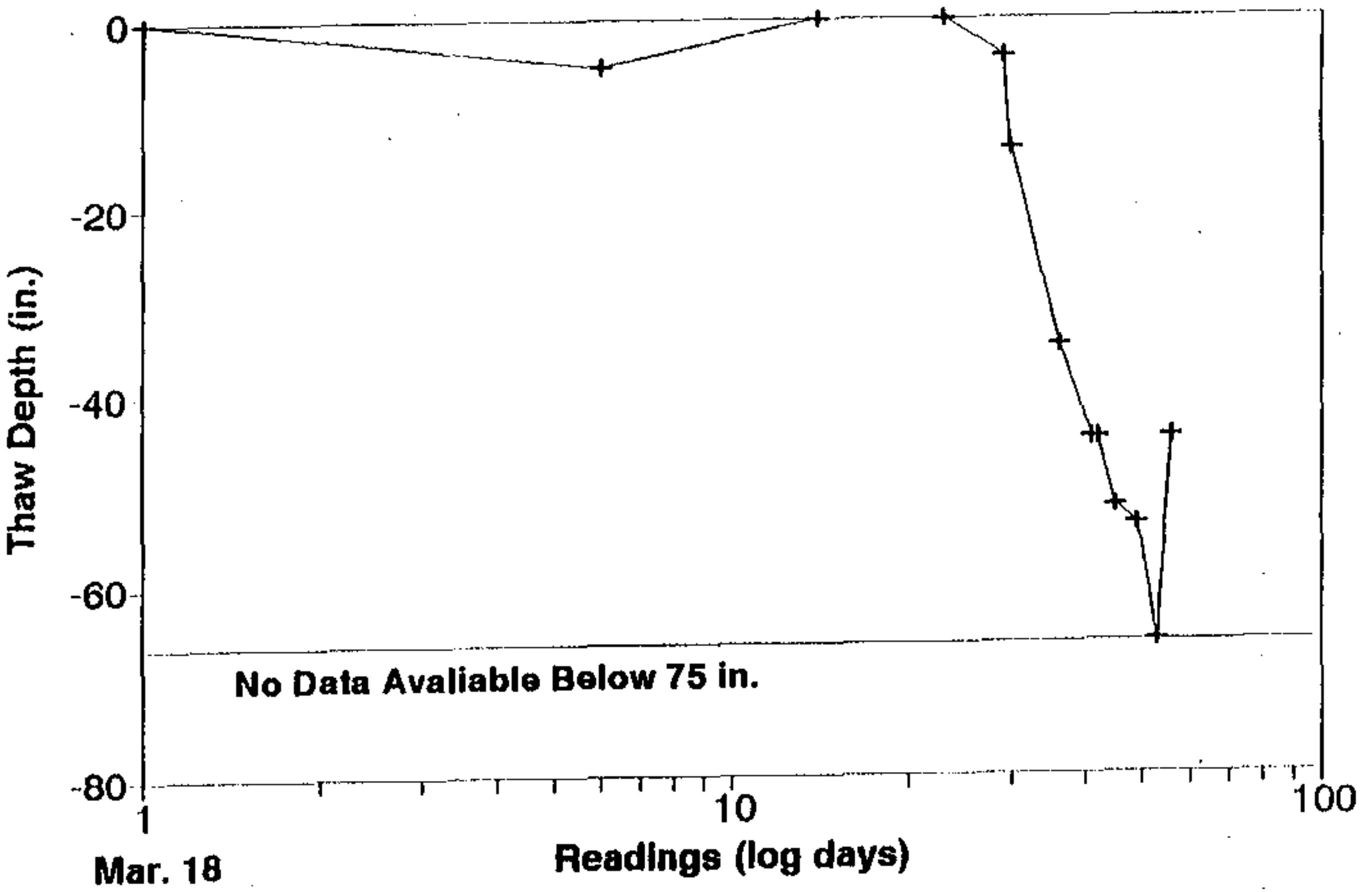


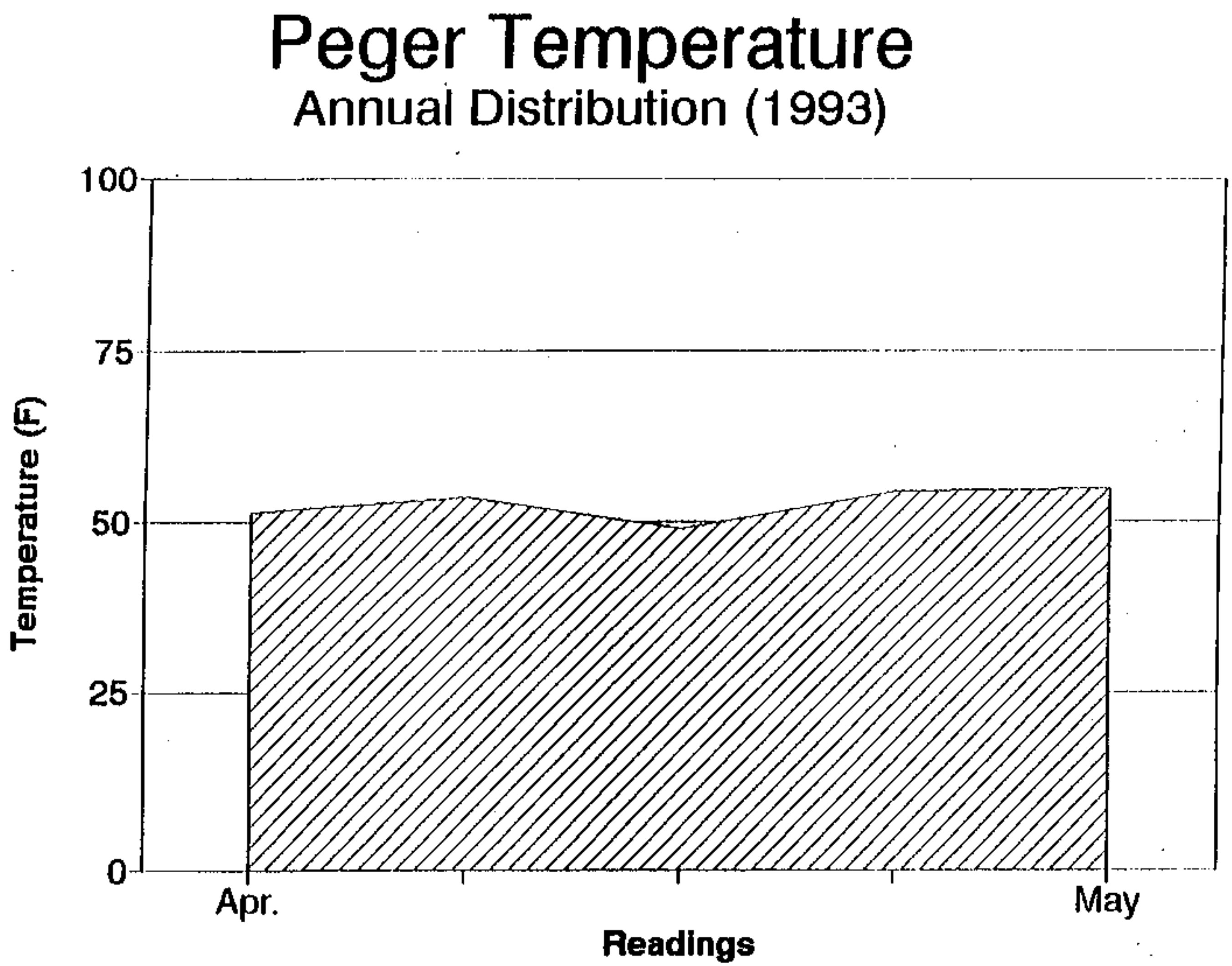
Peger Thaw Zone
Annual Distribution (1991)
Thaw Depth (in.)
0
-20
-40
-60
-80
No data Avaliable Below 75 in.
1
10
100
Mar. 27
Readings (log days)


Peger Temperature
Annual Distribution (1992)
100
75
50
25
0
Temperature (F)
Mar.
Apr.
May
Readings

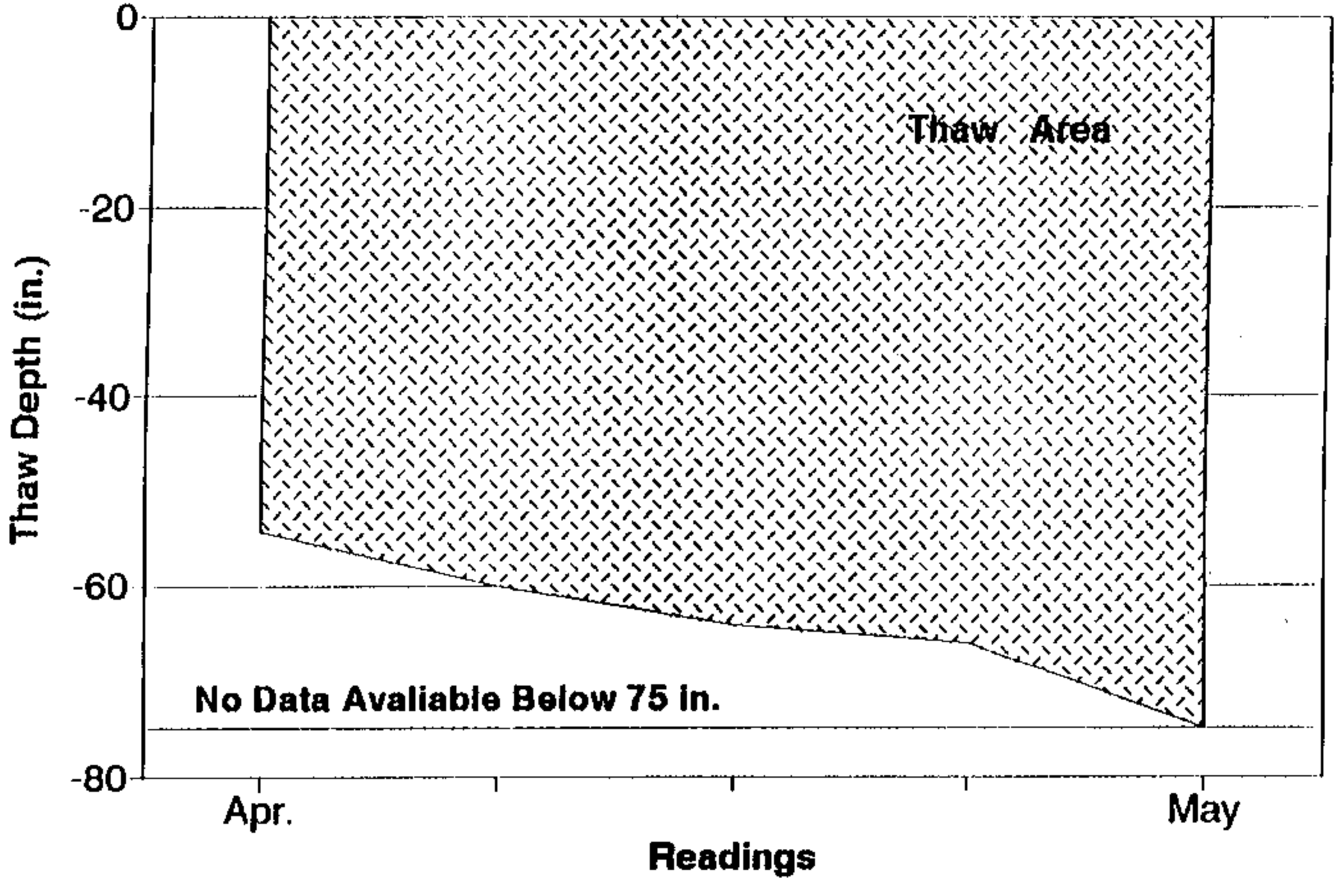

Peger Thaw Zone
Annual Distribution (1992)
0
-20
-40
-60
-80
Thaw Depth (in.)
Thaw Area
No Data Avaliable Below 75 in.
Mar.
Apr.
May
Readings

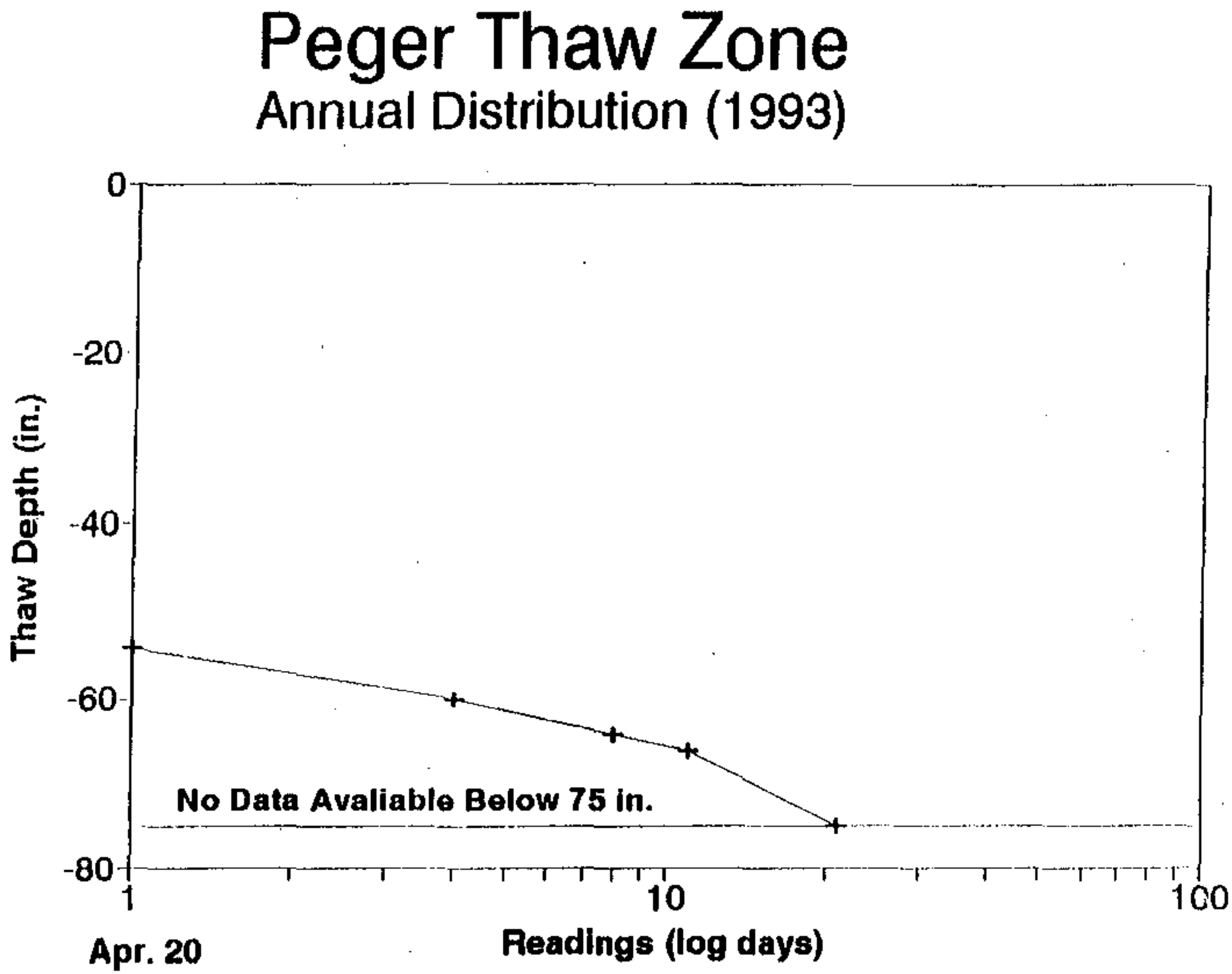
Peger Thaw Zone
Annual Distribution (1992)




Peger Temperature
Annual Distribution (1993)
100
75
50
25
0
Temperature (F)
Apr.
May
Readings

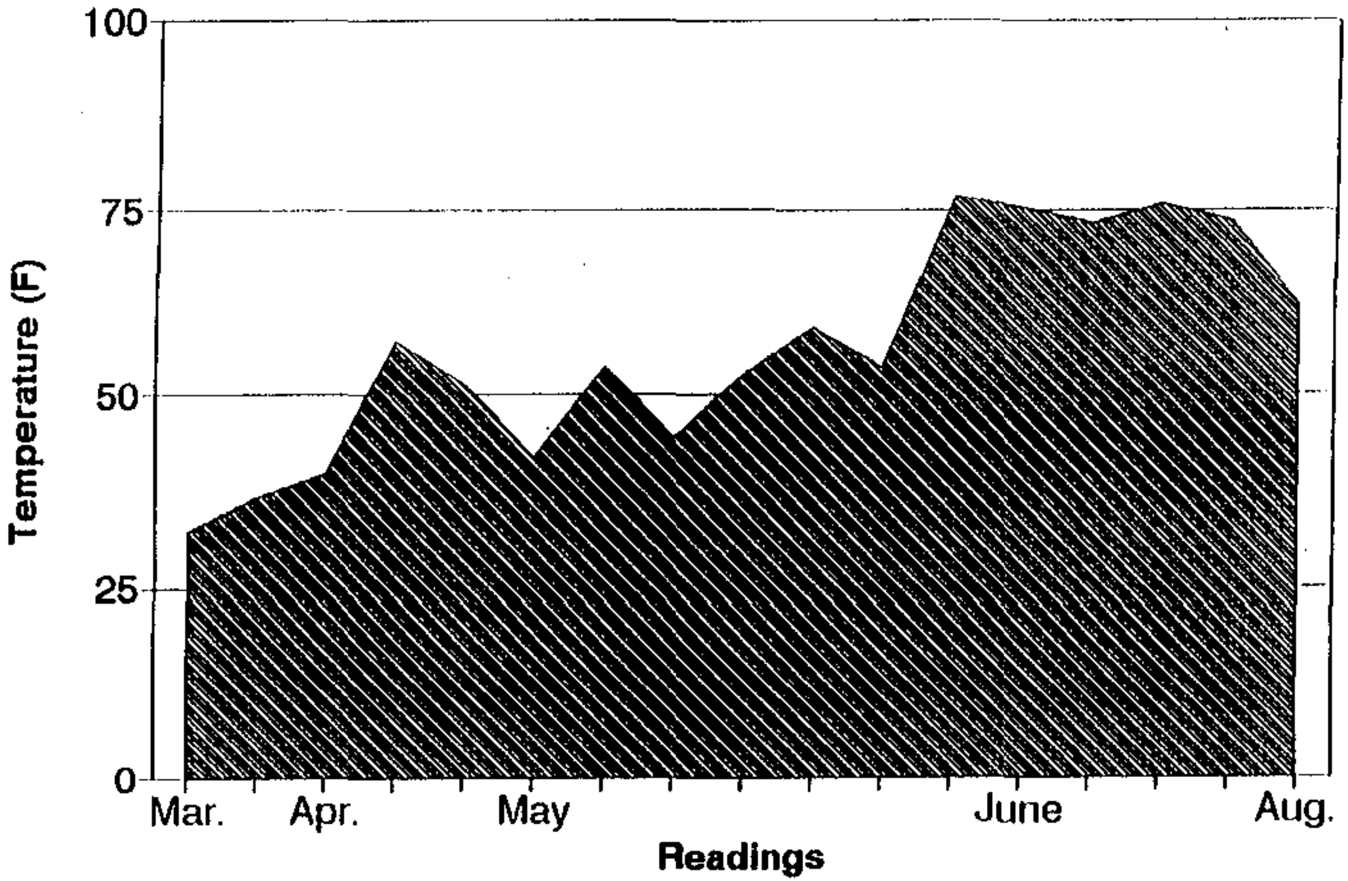
Peger Thaw Zone
Annual Distribution (1993)

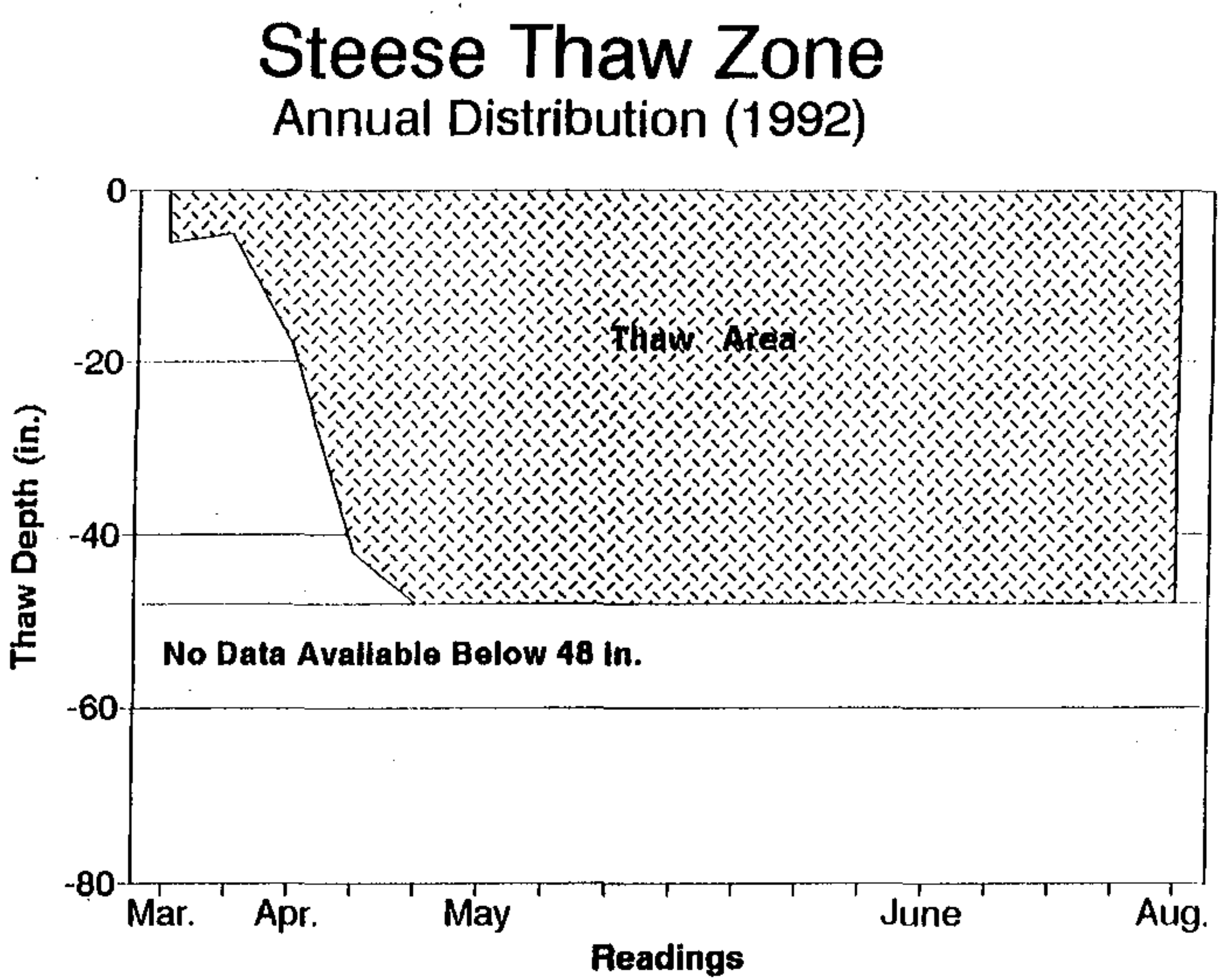


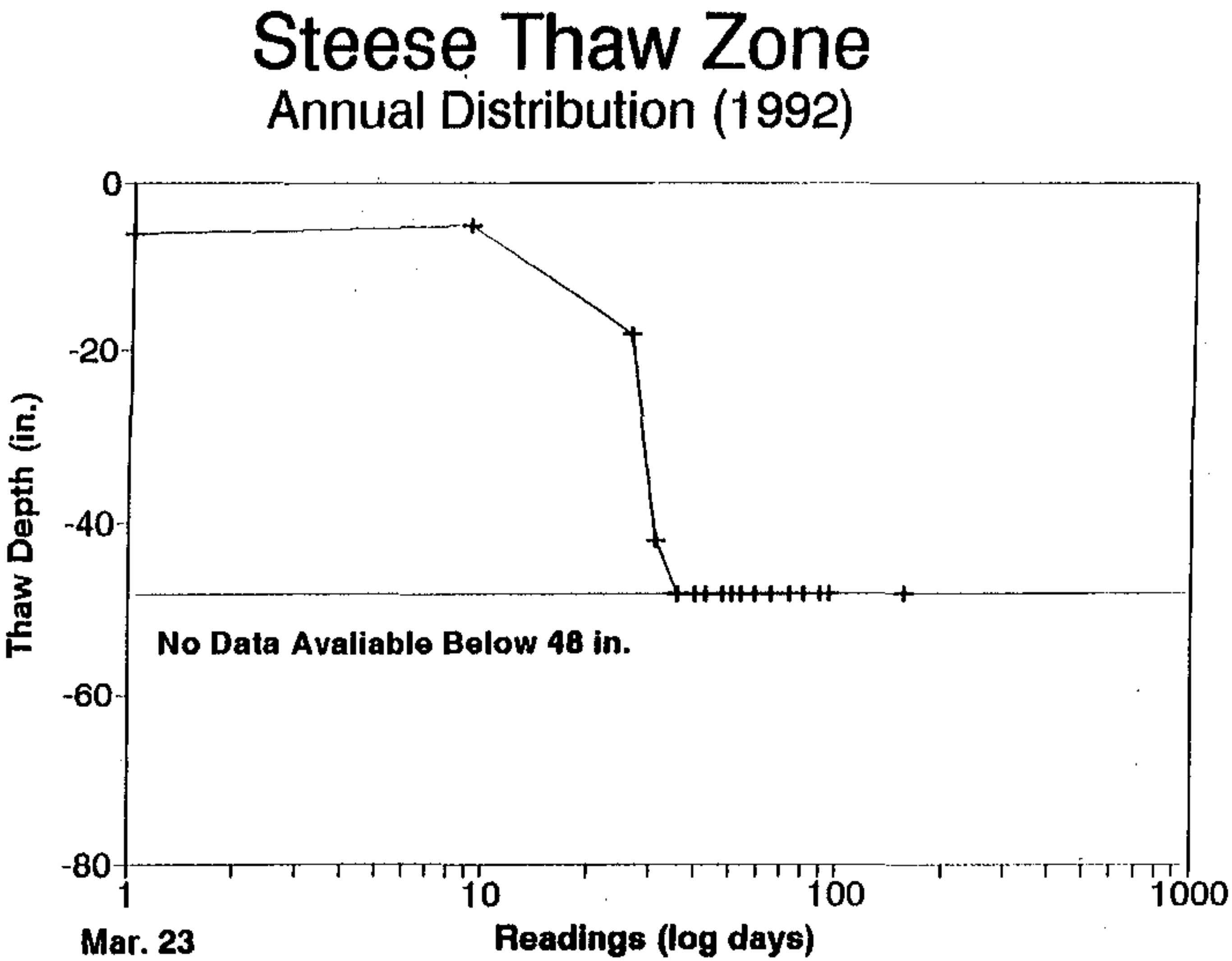


Peger Thaw Zone
Annual Distribution (1993)
0
-20
-40
-60
-80
Thaw Depth (in.)
No Data Avaliable Below 75 in.
1
10
100
Apr. 20
Readings (log days)

Steese Temperature
Annual Distribution (1992)

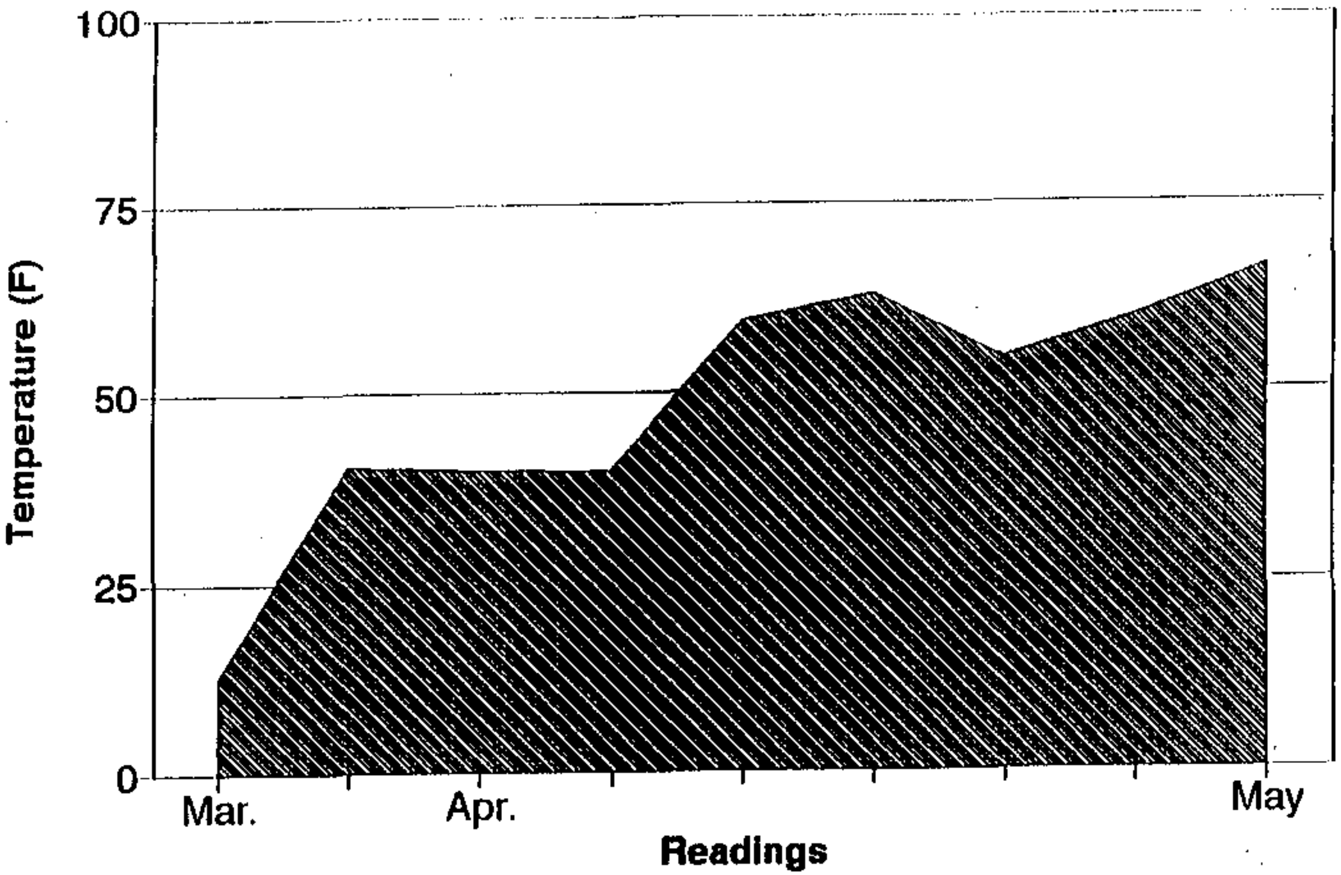



Steese Thaw Zone
Annual Distribution (1992)
Thaw Area
No Data Available Below 48 in.
Thaw Depth (in.)
0
-20
-40
-60
-80
Mar.
Apr.
May
June
Aug.
Readings

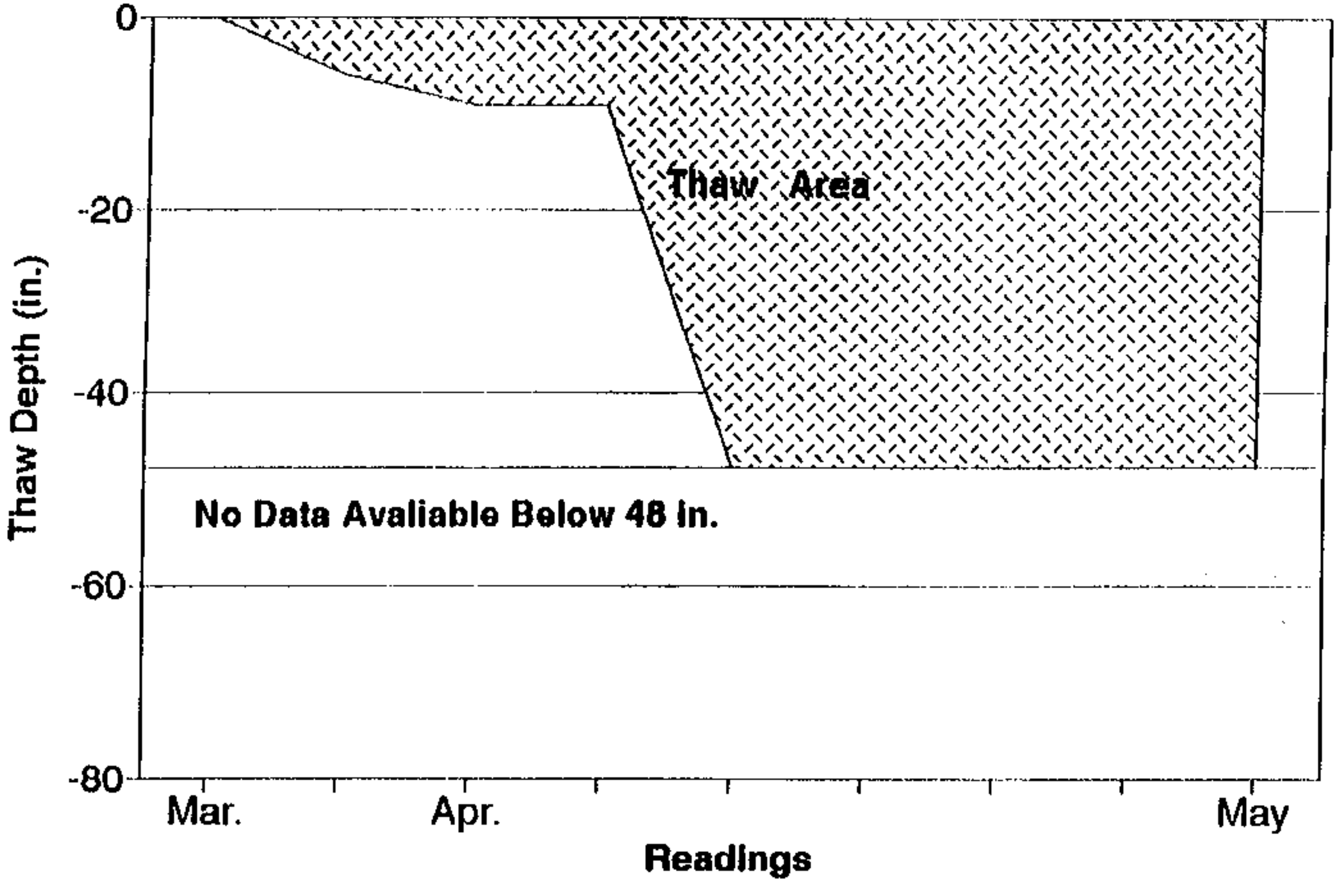


Steese Thaw Zone
Annual Distribution (1992)
0
-20
-40
-60
-80
Thaw Depth (in.)
No Data Avaliable Below 48 in.
1
10
100
1000
Mar. 23
Readings (log days)

Steese Temperature
Annual Distribution (1993)

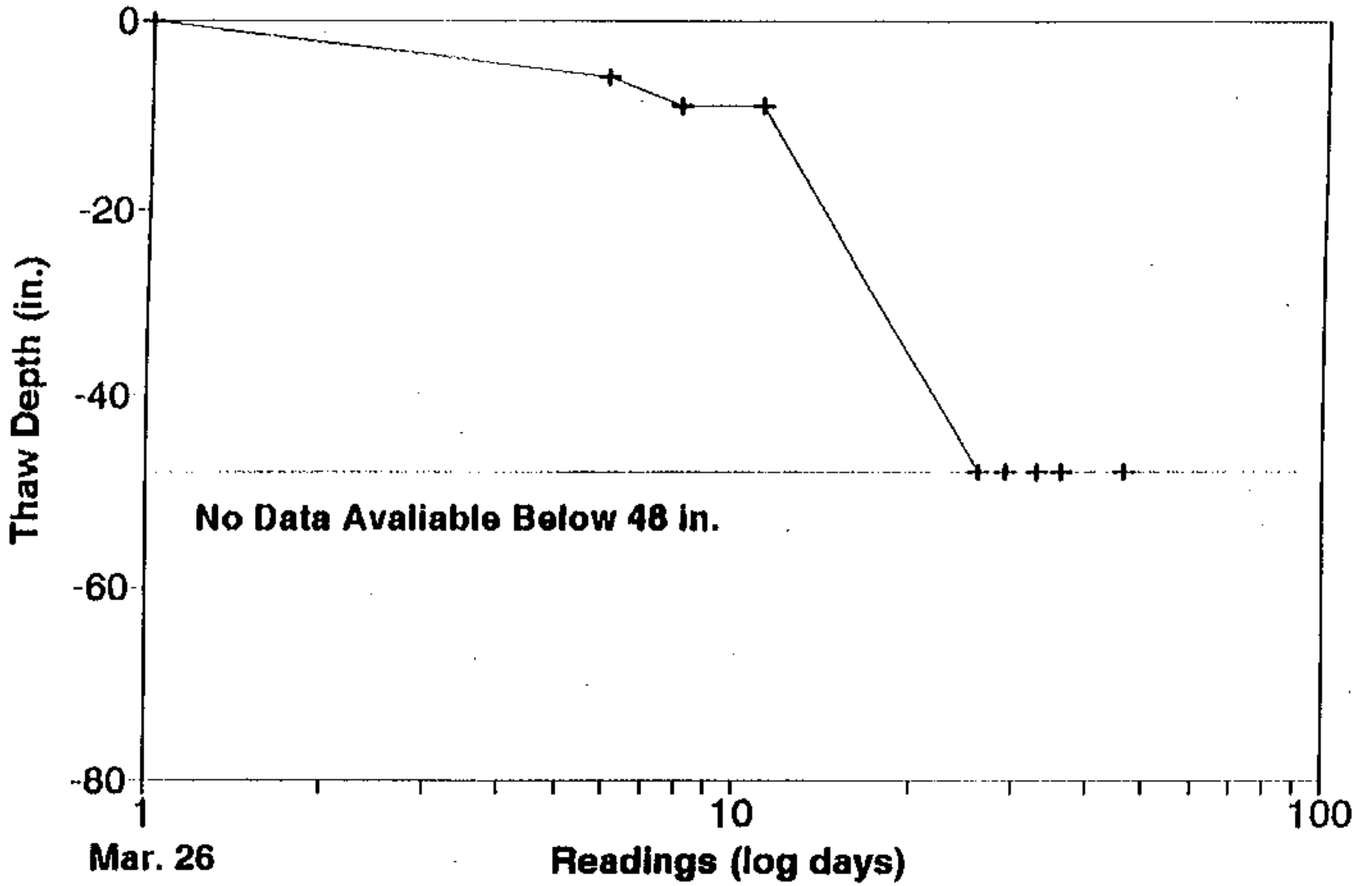


Steese Thaw Zone
Annual Distribution (1993)

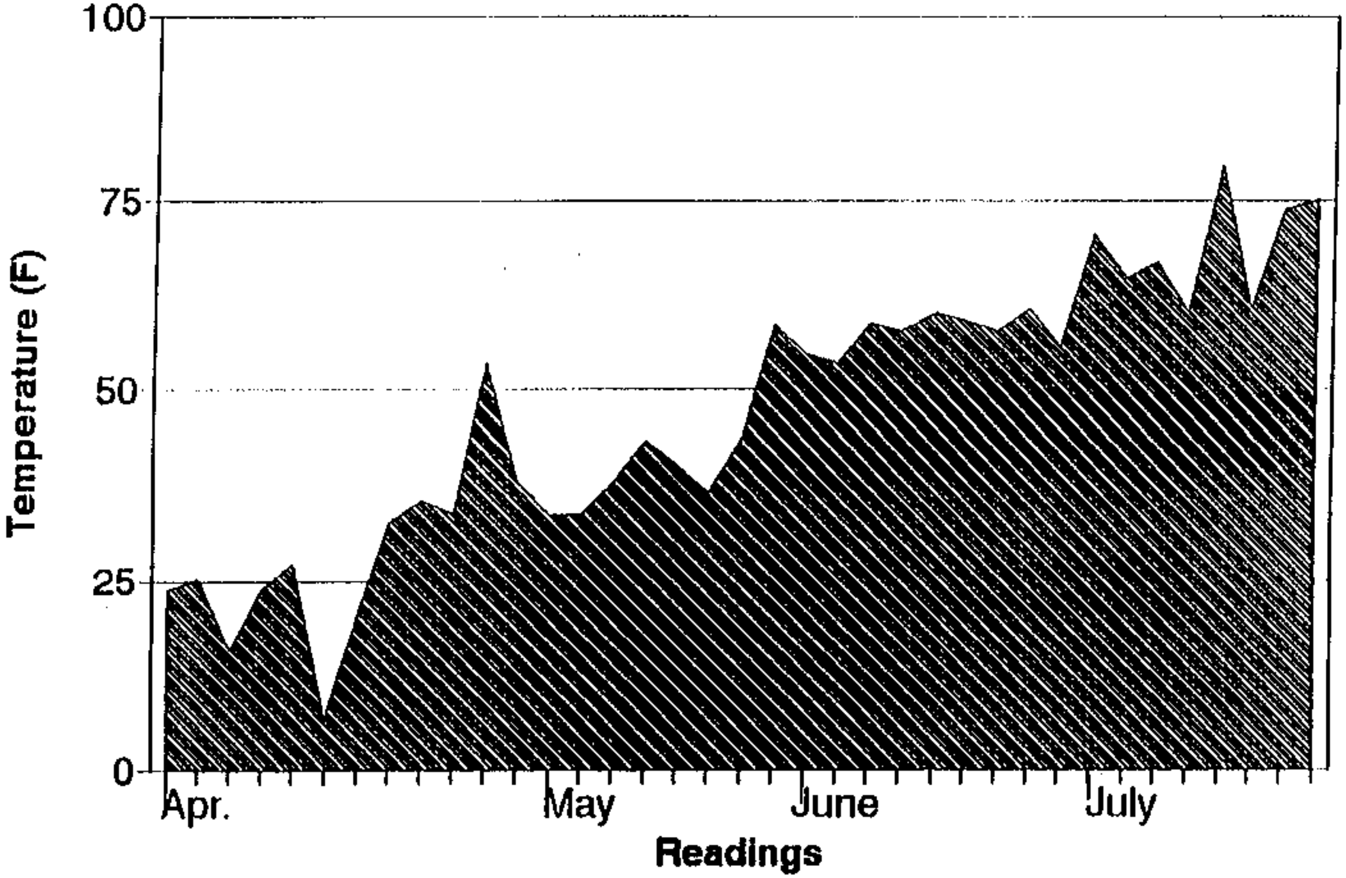


Steese Thaw Zone

Annual Distribution (1993)

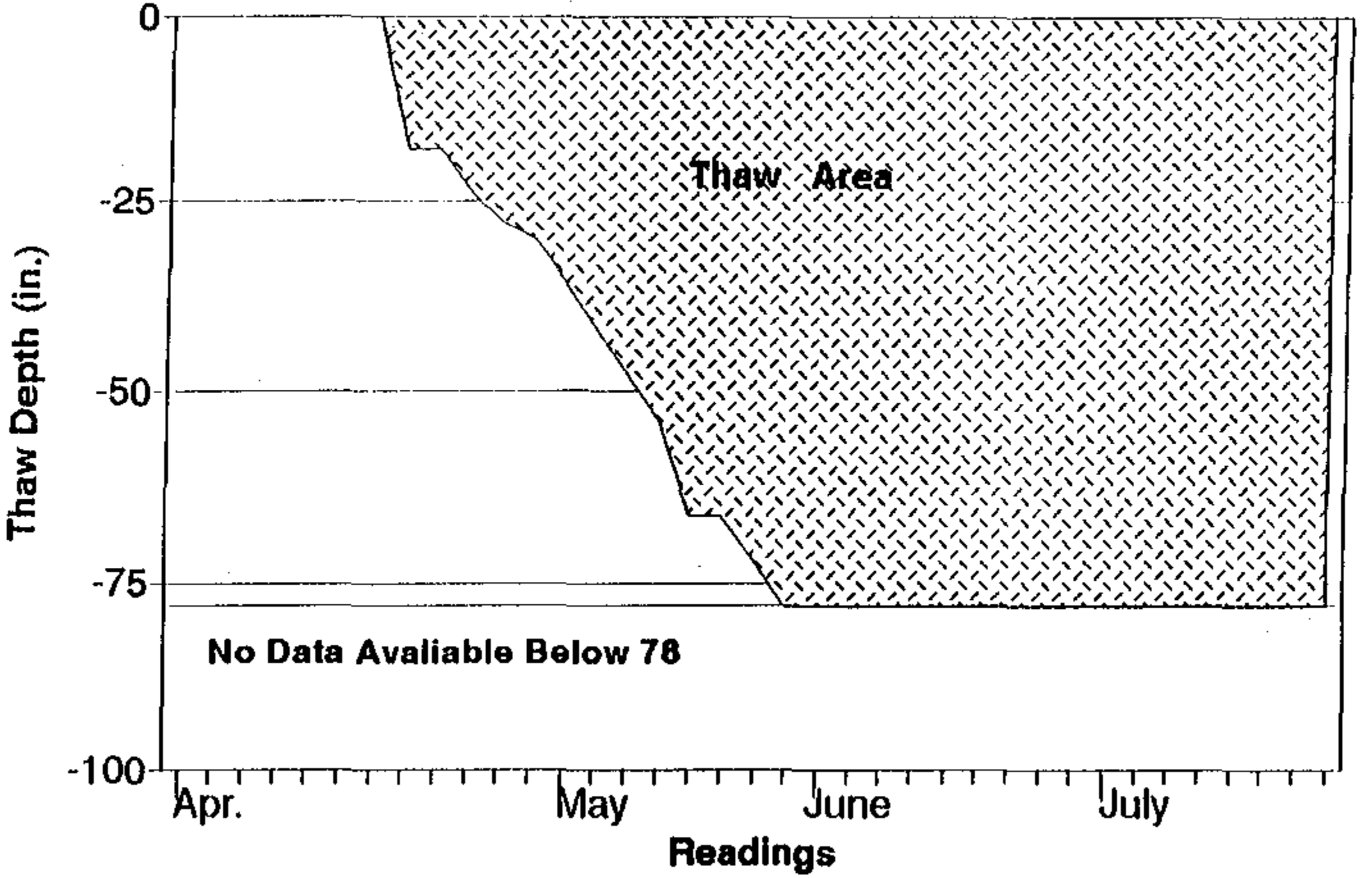


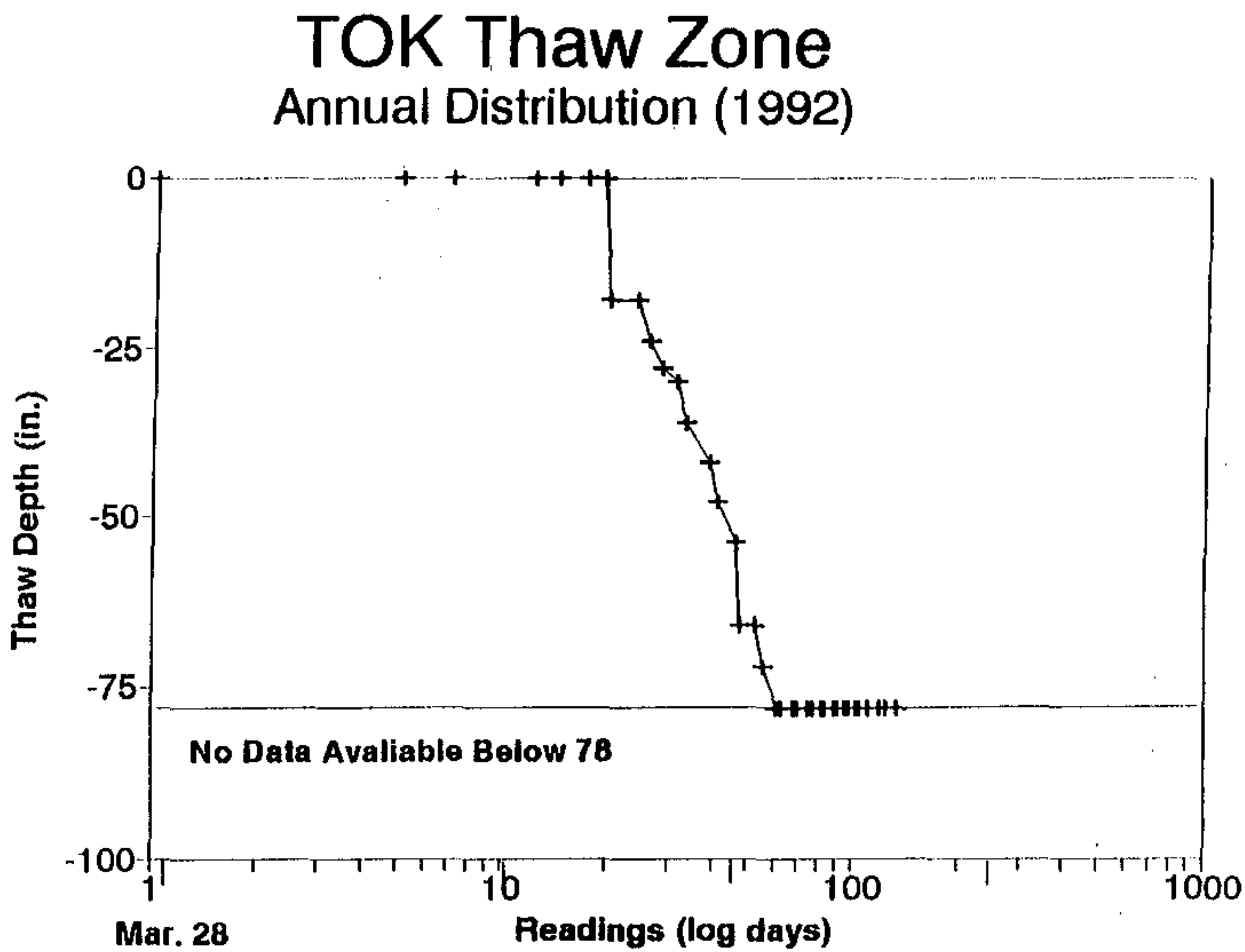
TOK Temperature
Annual Distribution (1992)



TOK Thaw Zone

Annual Distribution (1992)




TOK Thaw Zone
Annual Distribution (1992)
0
-25
-50
-75
-100
Thaw Depth (in.)
No Data Avaliable Below 78
1
10
100
1000
Mar. 28
Readings (log days)

APPENDIX F

INFLUENCE OF TIRE PRESSURE, AND THAW DEPTH ON PAVEMENT RESPONSE AND DAMAGE

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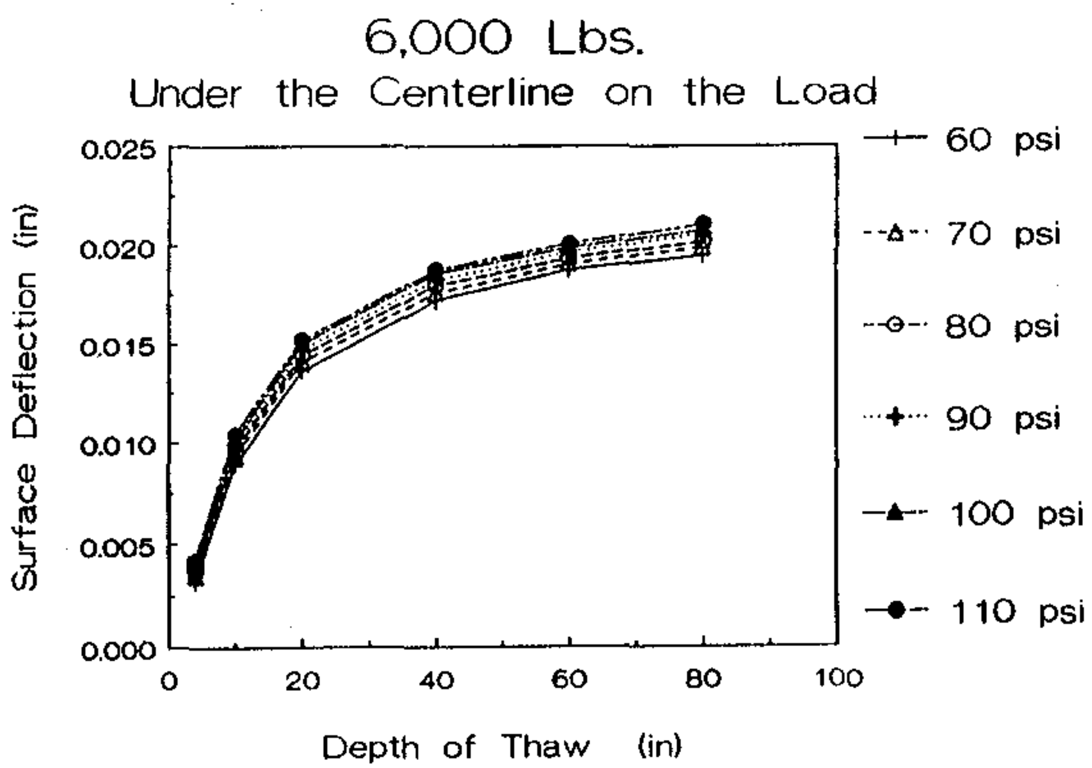
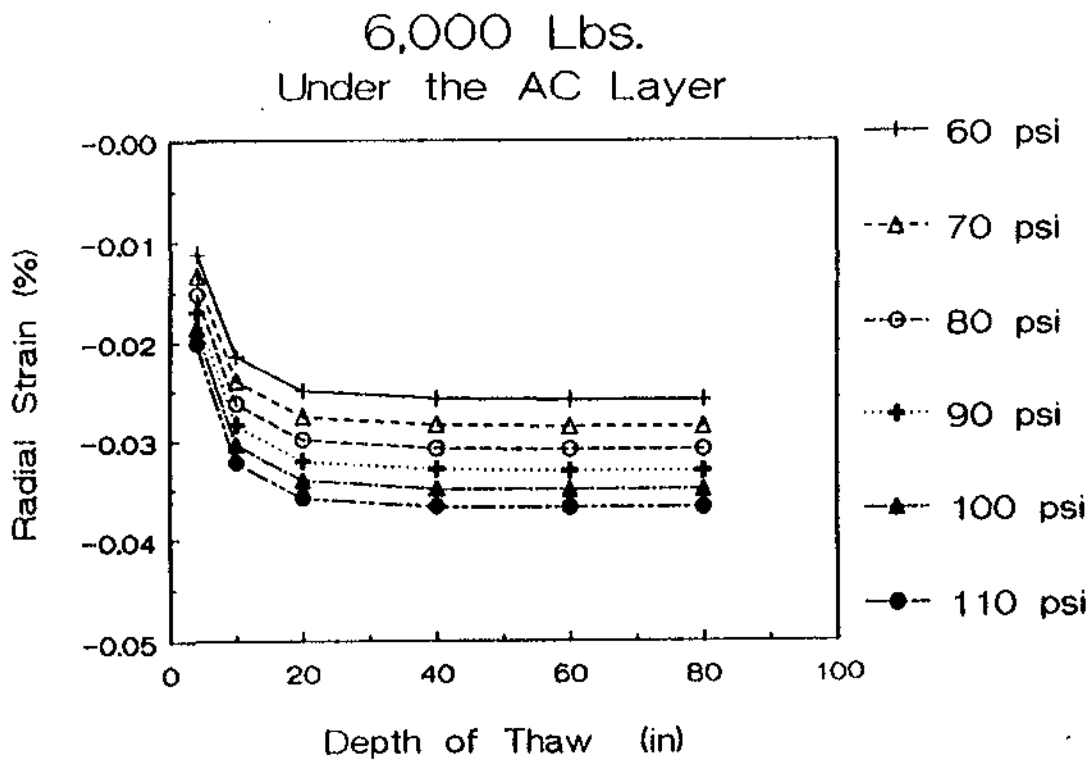
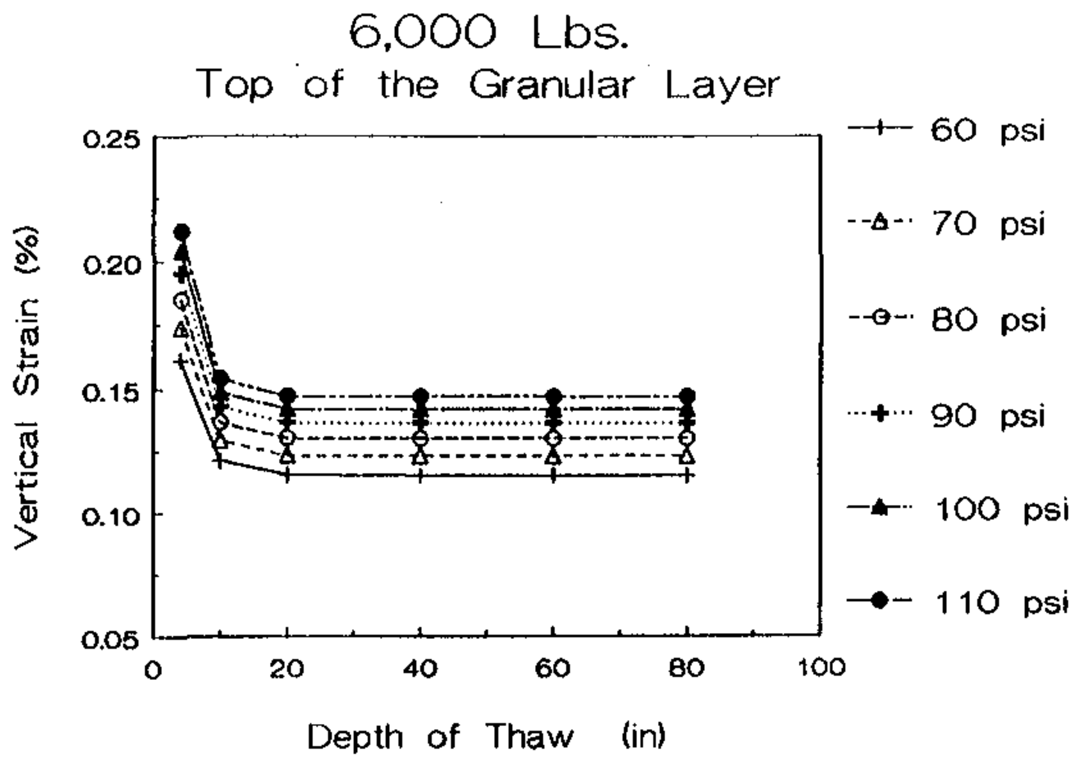
APPENDIX F -- INFLUENCE OF TIRE PRESSURE, AND THAW DEPTH ON PAVEMENT RESPONSE AND DAMAGE


6,000 Lbs.
Under the Centerline on the Load
Surface Deflection (in)
0.025
0.020
0.015
0.010
0.005
0.000
0
20
40
60
80
100
Depth of Thaw (in)
60 psi
70 psi
80 psi
90 psi
100 psi
110 psi

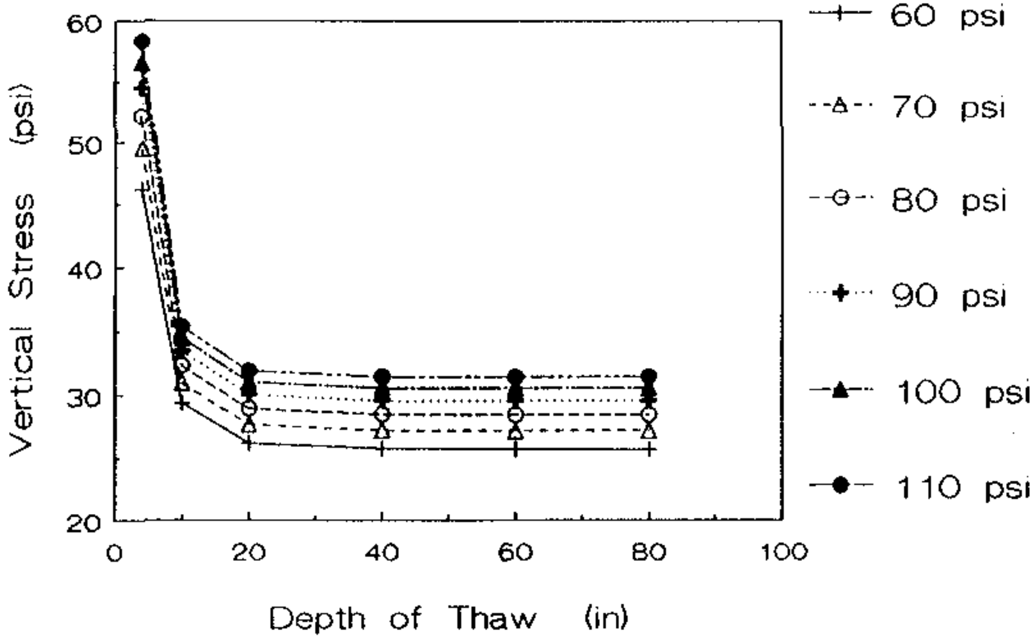


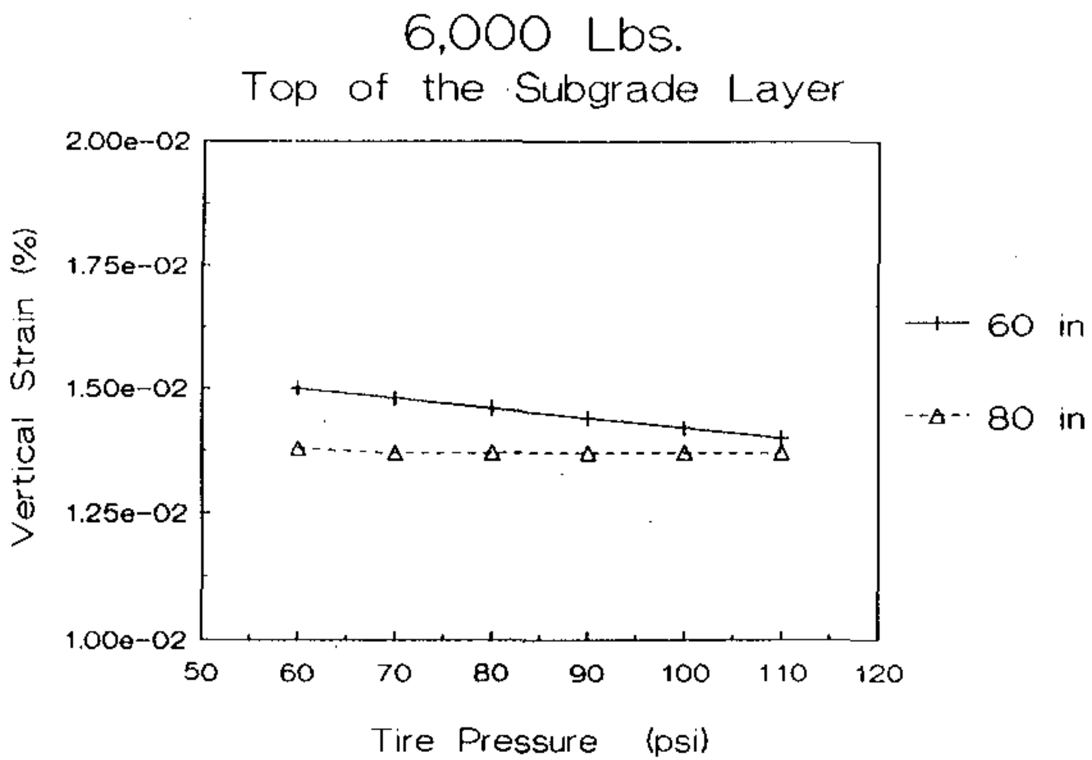
6,000 Lbs.
Under the AC Layer
Radial Strain (%)
-0.00
-0.01
-0.02
-0.03
-0.04
-0.05
0
20
40
60
80
100
Depth of Thaw (in)
60 psi
70 psi
80 psi
90 psi
100 psi
110 psi

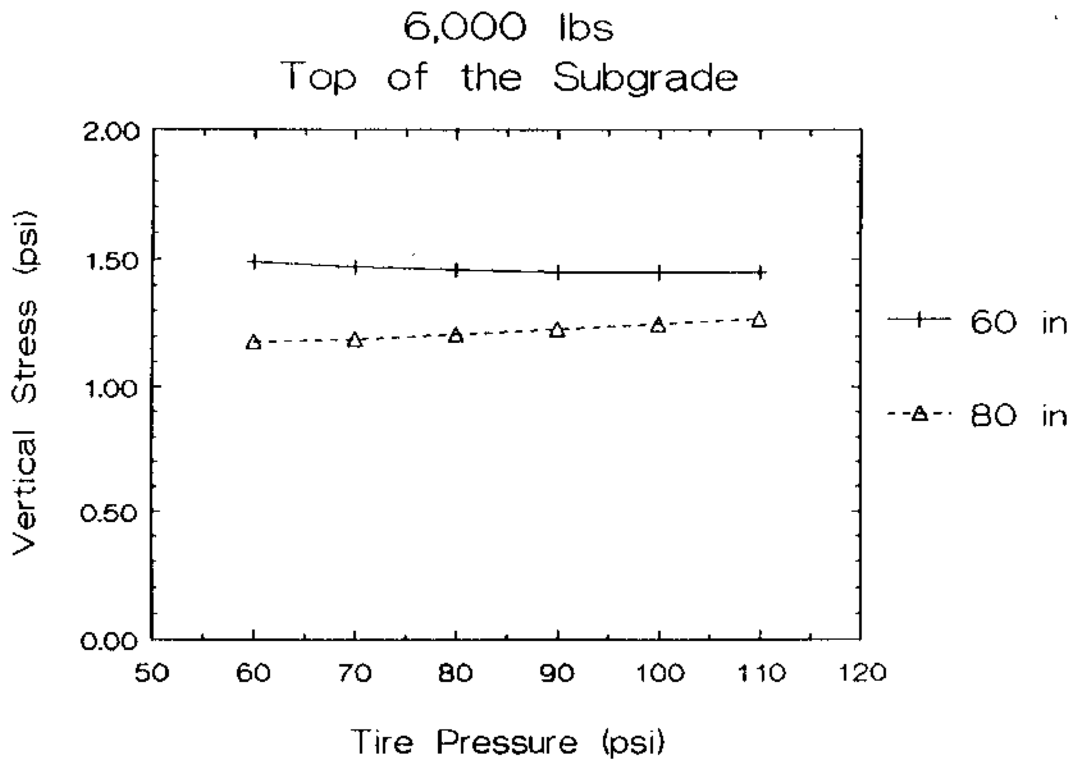

6,000 Lbs.
Top of the Granular Layer
Vertical Strain (%)
0.25
0.20
0.15
0.10
0.05
0
20
40
60
80
100
Depth of Thaw (in)
60 psi
70 psi
80 psi
90 psi
100 psi
110 psi

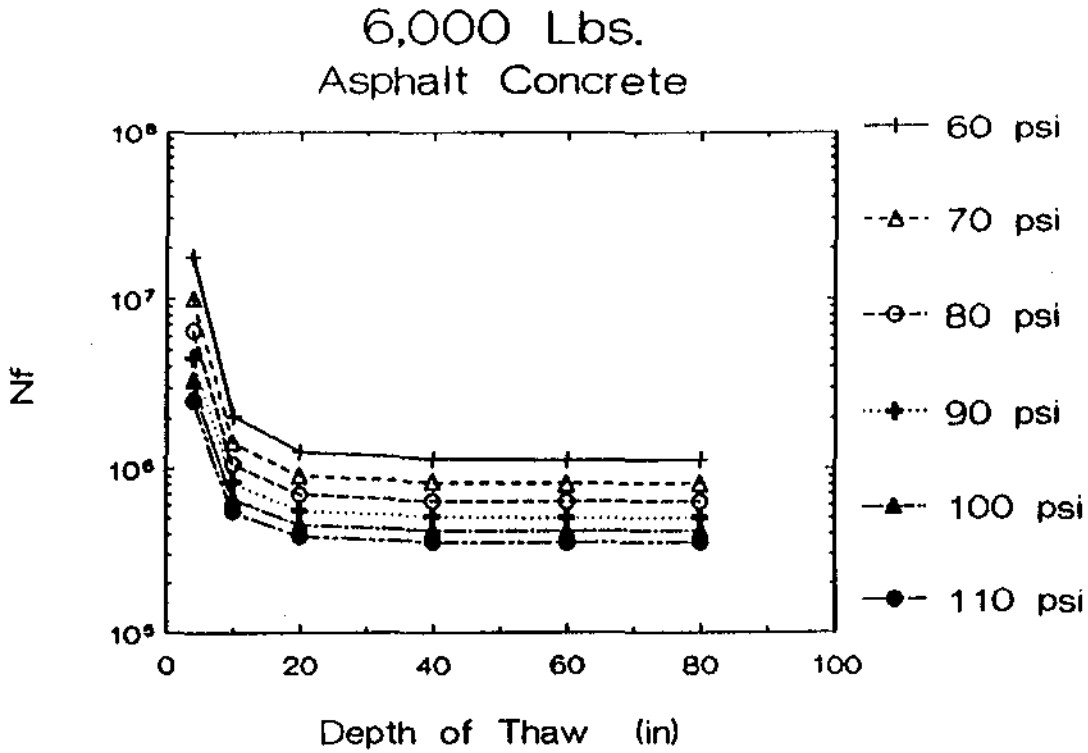
6,000 Lbs.

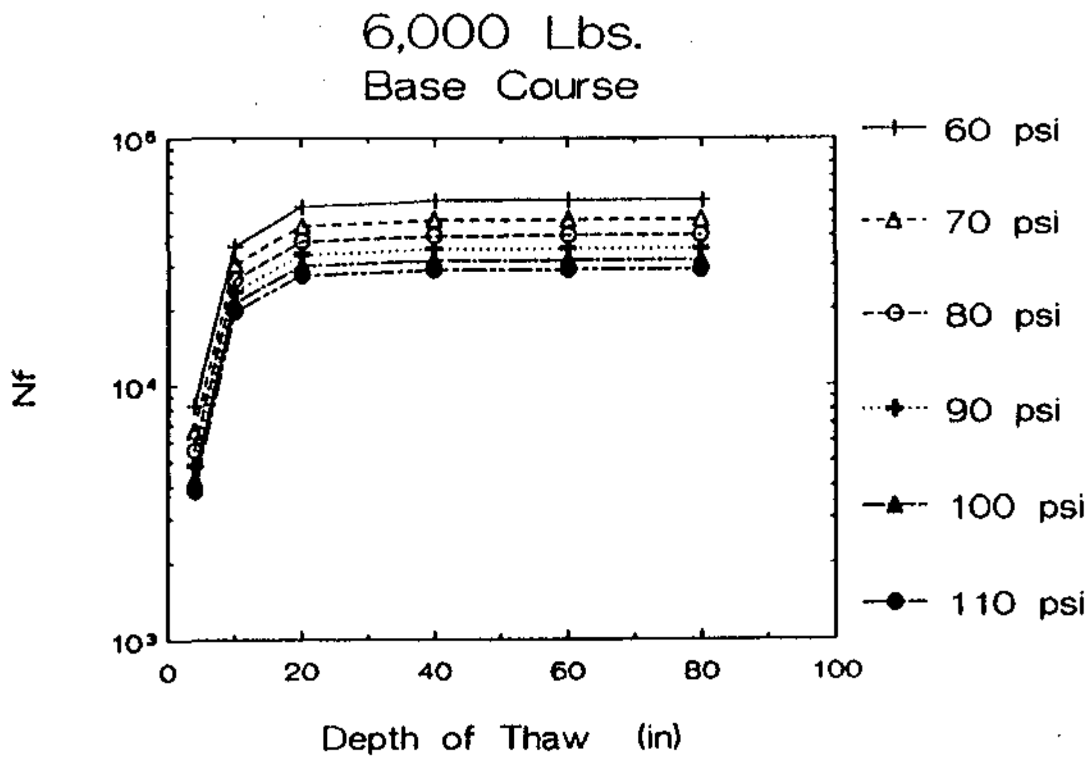
Top of the Granular Layer



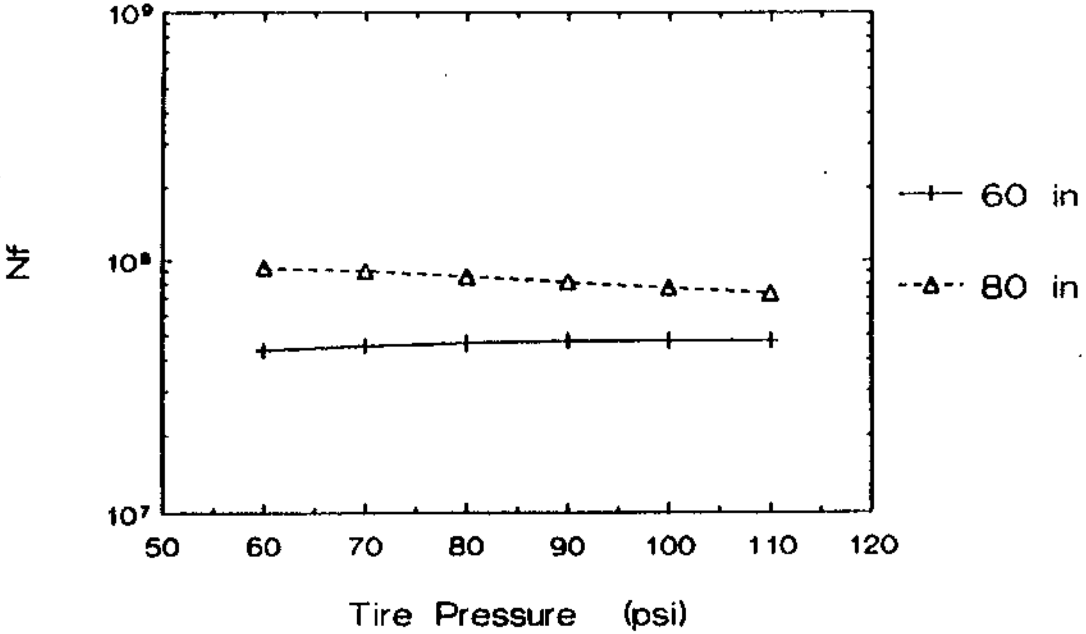

6,000 Lbs.
Top of the Subgrade Layer
Vertical Strain (%)
2.00e-02
1.75e-02
1.50e-02
1.25e-02
1.00e-02
50
60
70
80
90
100
110
120
Tire Pressure (psi)
60 in
80 in

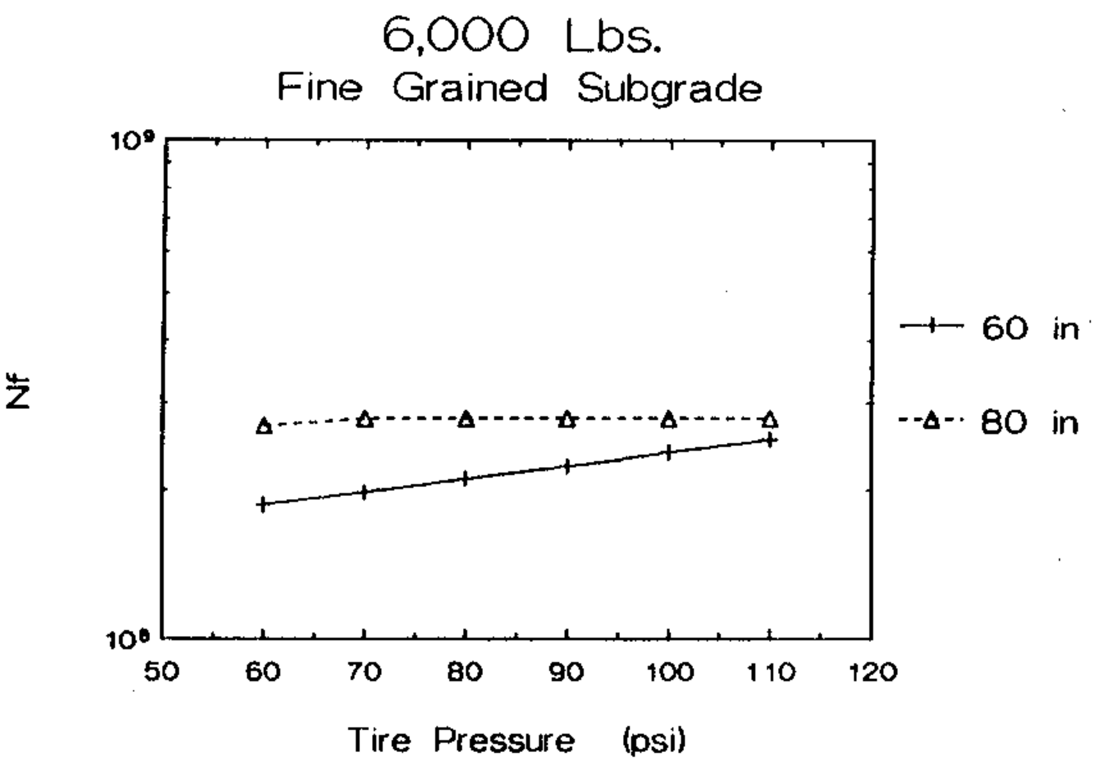

6,000 lbs
Top of the Subgrade
Vertical Stress (psi)
2.00
1.50
1.00
0.50
0.00
50
60
70
80
90
100
110
120
Tire Pressure (psi)
60 in
80 in


6,000 Lbs.
Asphalt Concrete
10^8
10^7
10^6
10^5
Nf
0
20
40
60
80
100
Depth of Thaw (in)
60 psi
70 psi
80 psi
90 psi
100 psi
110 psi

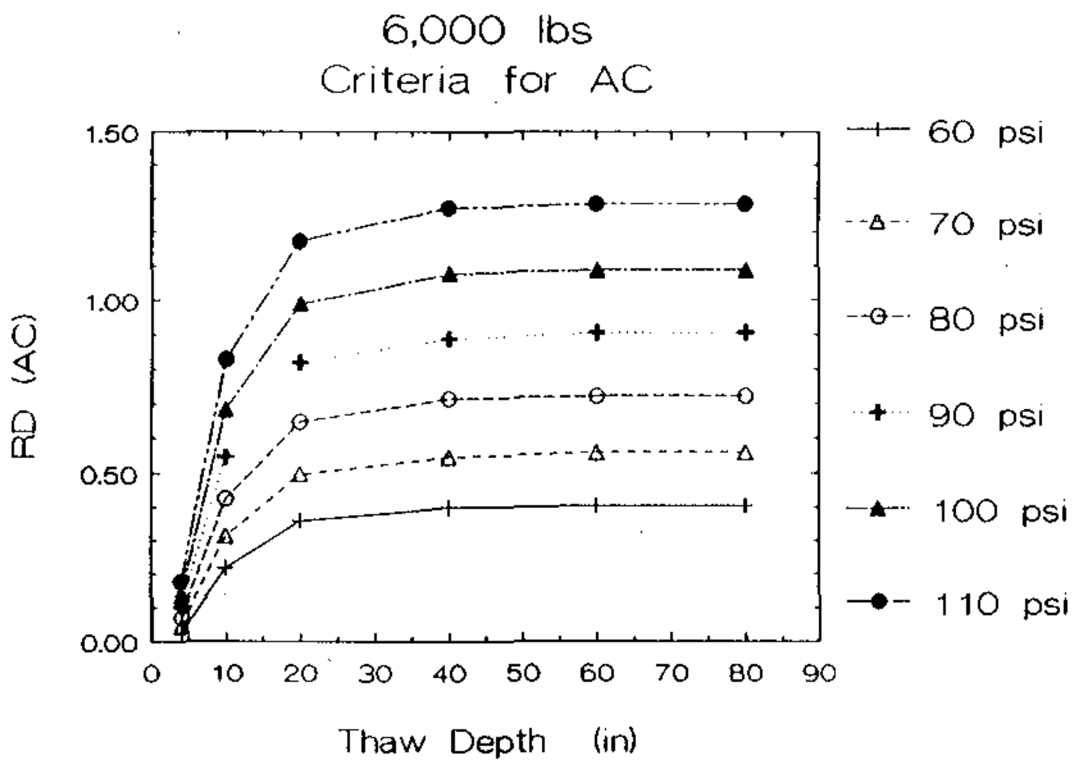

6,000 Lbs.
Base Course
Nf
10^5
10^4
10^3
0
20
40
60
80
100
Depth of Thaw (in)
60 psi
70 psi
80 psi
90 psi
100 psi
110 psi

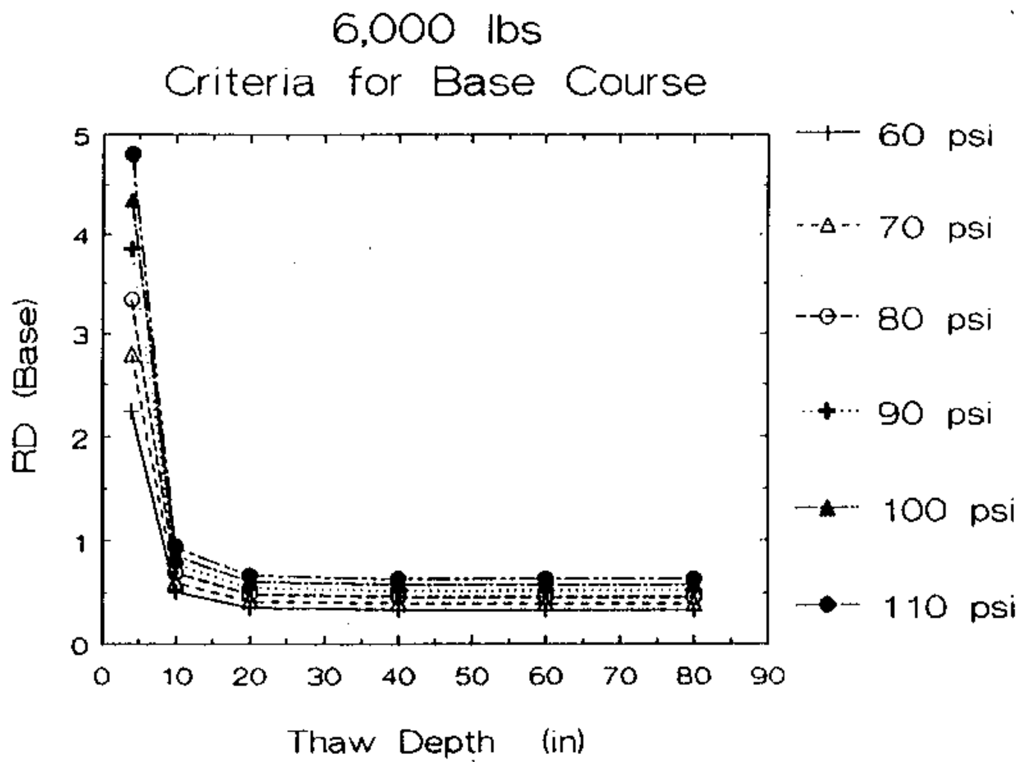
6,000 Lbs.
Granular Subgrade

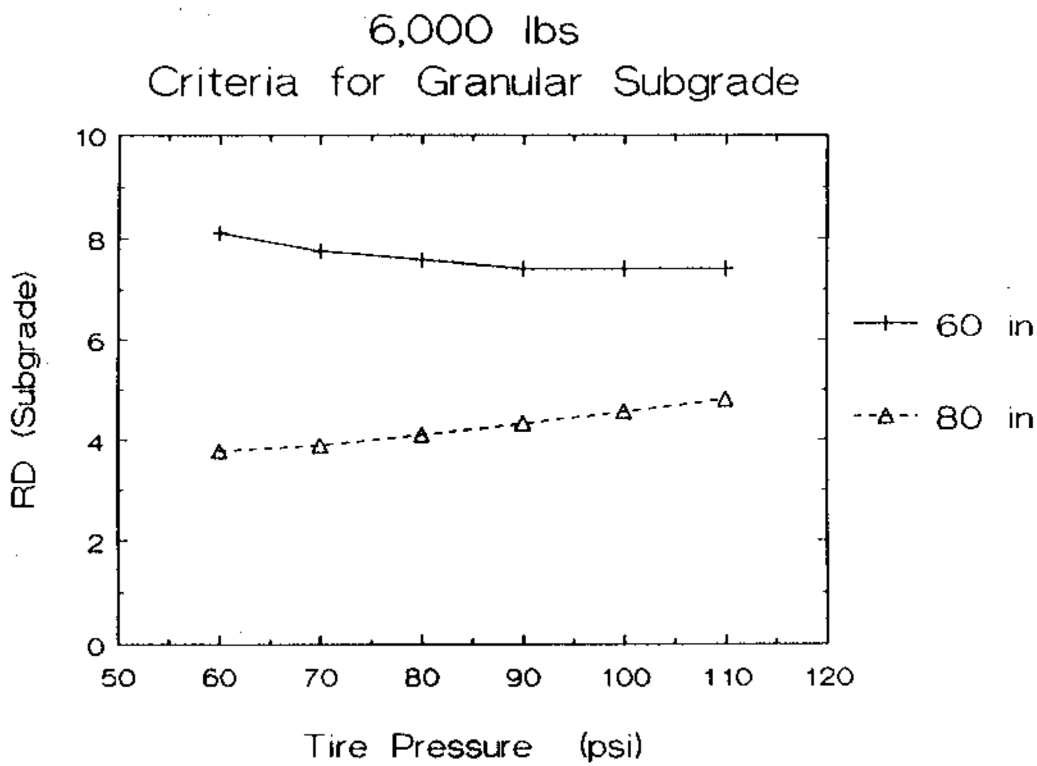




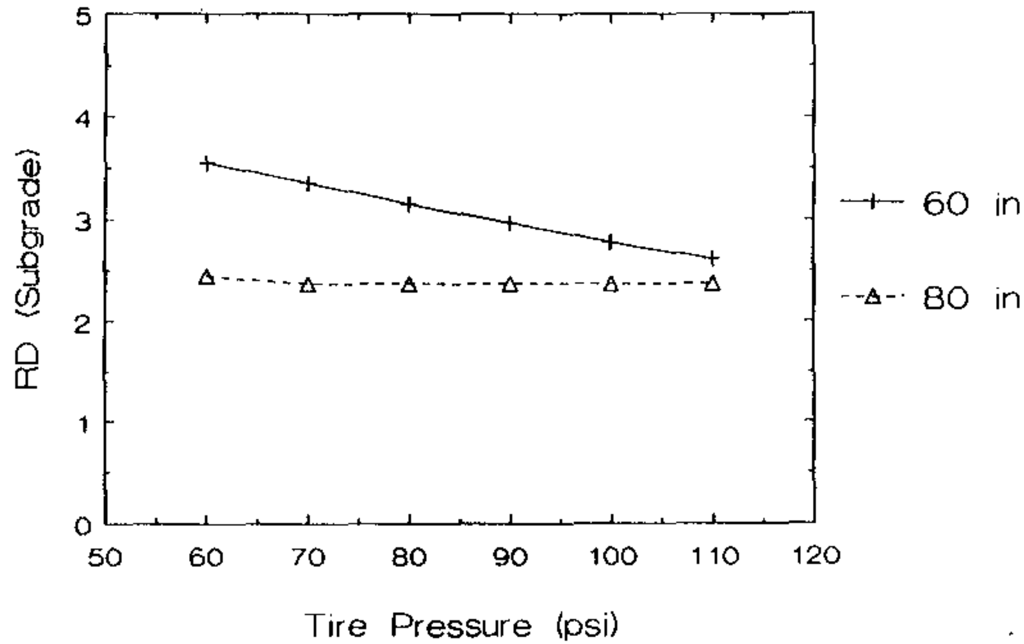
6,000 Lbs.
Fine Grained Subgrade
Nf
Tire Pressure (psi)
60 in
80 in

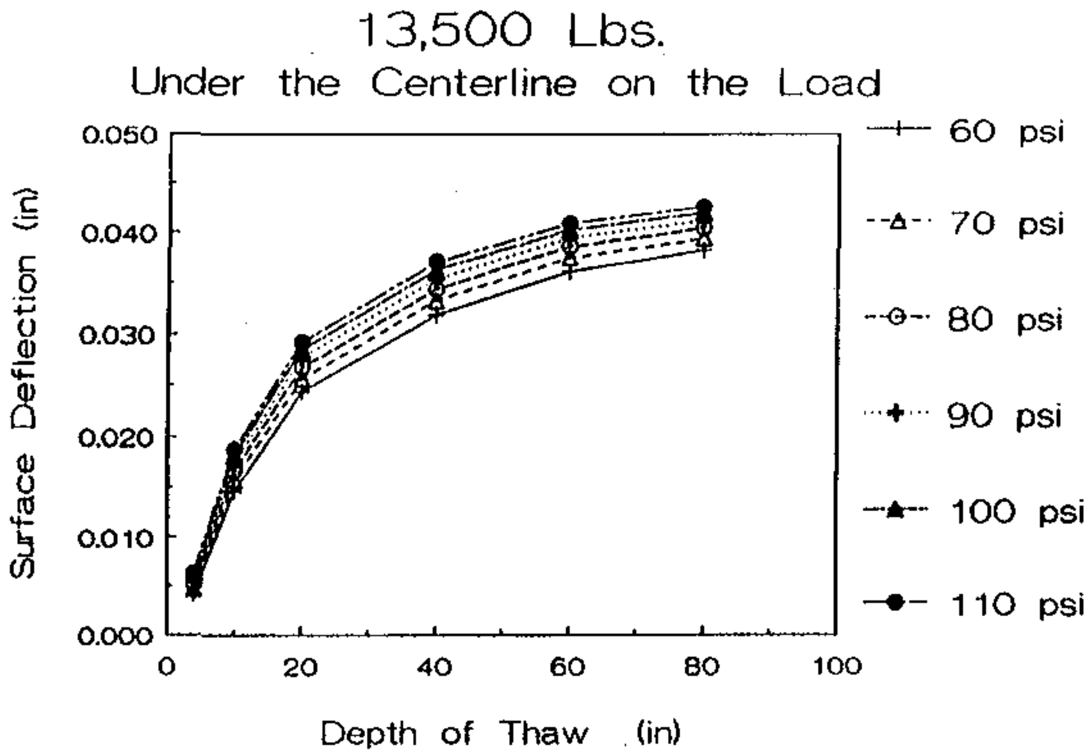

6,000 lbs
Criteria for AC
RD (AC)
Thaw Depth (in)
60 psi
70 psi
80 psi
90 psi
100 psi
110 psi
0.00
0.50
1.00
1.50
0
10
20
30
40
50
60
70
80
90


6,000 lbs
Criteria for Base Course
RD (Base)
Thaw Depth (in)
60 psi
70 psi
80 psi
90 psi
100 psi
110 psi
0
1
2
3
4
5
0
10
20
30
40
50
60
70
80
90

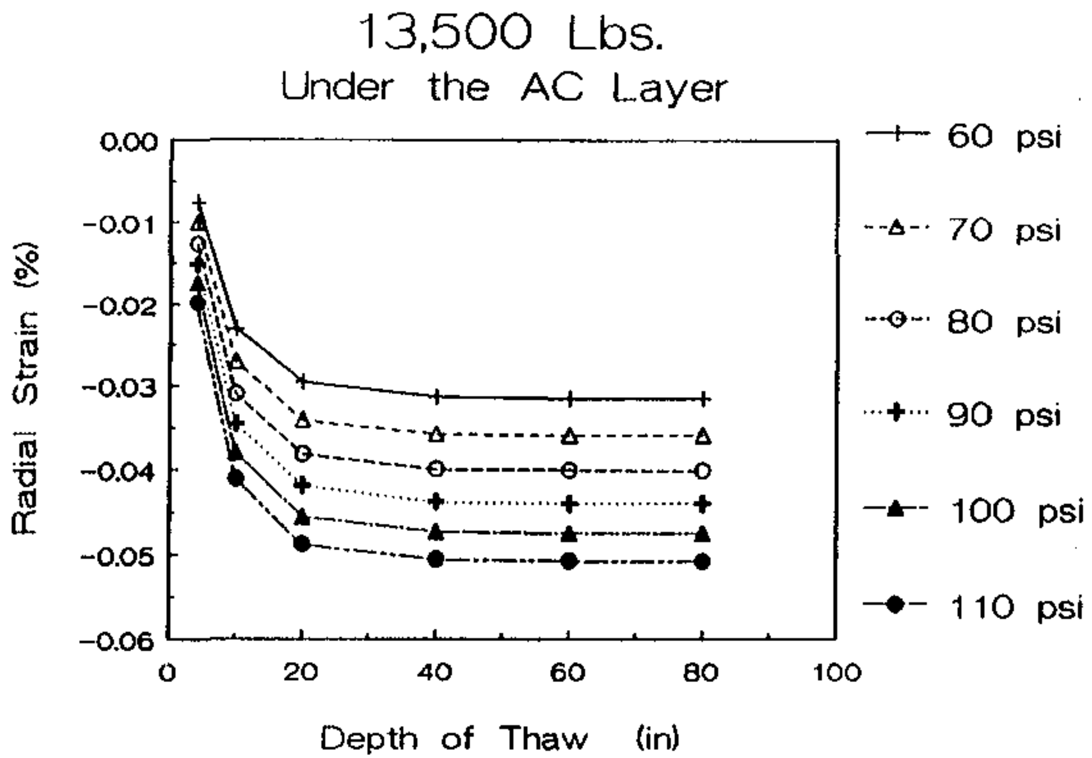

6,000 lbs
Criteria for Granular Subgrade
RD (Subgrade)
Tire Pressure (psi)
0
2
4
6
8
10
50
60
70
80
90
100
110
120
60 in
80 in

6,000 lbs
Criteria for Subgrade



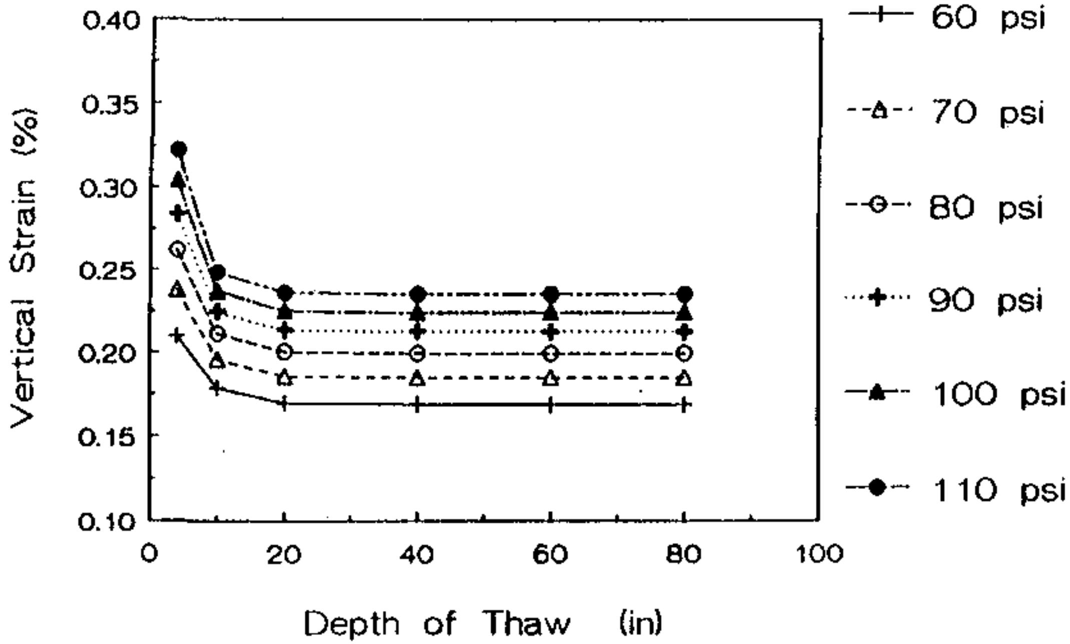


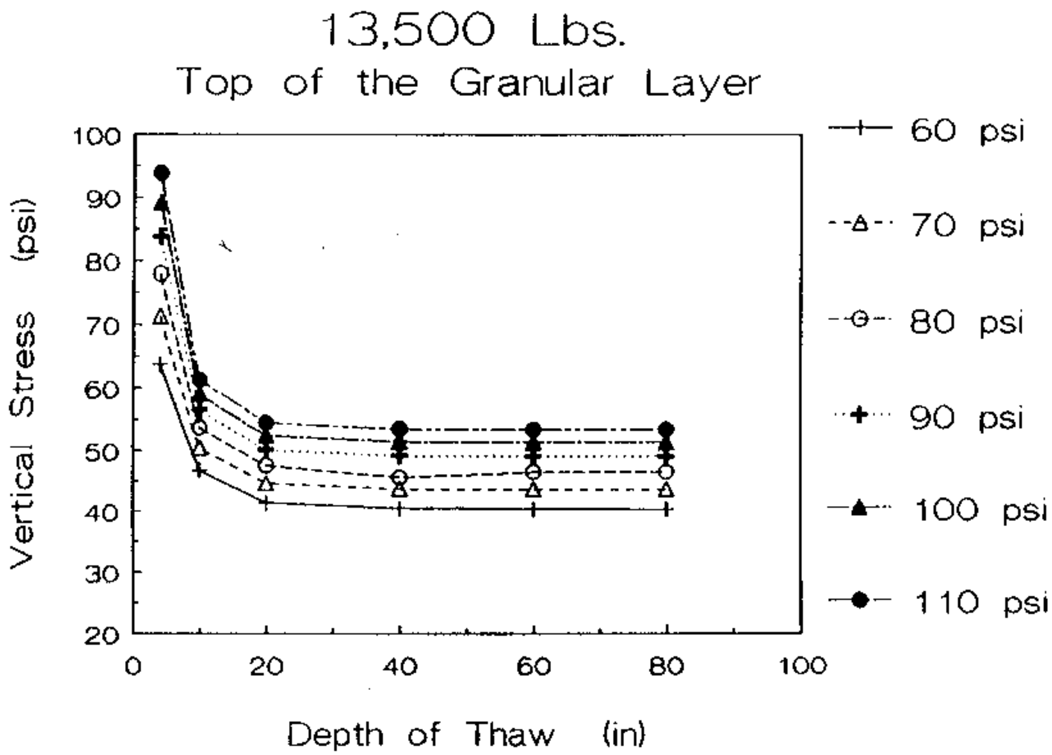
13,500 Lbs.
Under the Centerline on the Load
Surface Deflection (in)
0.050
0.040
0.030
0.020
0.010
0.000
0
20
40
60
80
100
Depth of Thaw (in)
60 psi
70 psi
80 psi
90 psi
100 psi
110 psi

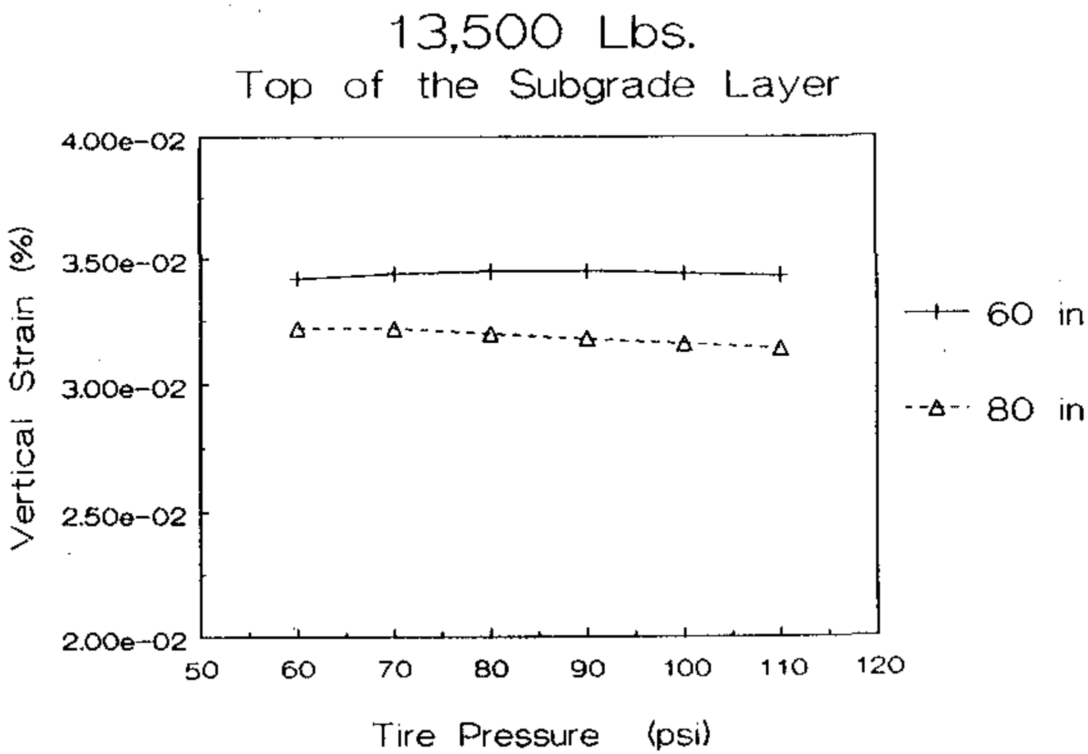

13,500 Lbs.
Under the AC Layer
Radial Strain (%)
0.00
-0.01
-0.02
-0.03
-0.04
-0.05
-0.06
0
20
40
60
80
100
Depth of Thaw (in)
60 psi
70 psi
80 psi
90 psi
100 psi
110 psi

13,500 Lbs.

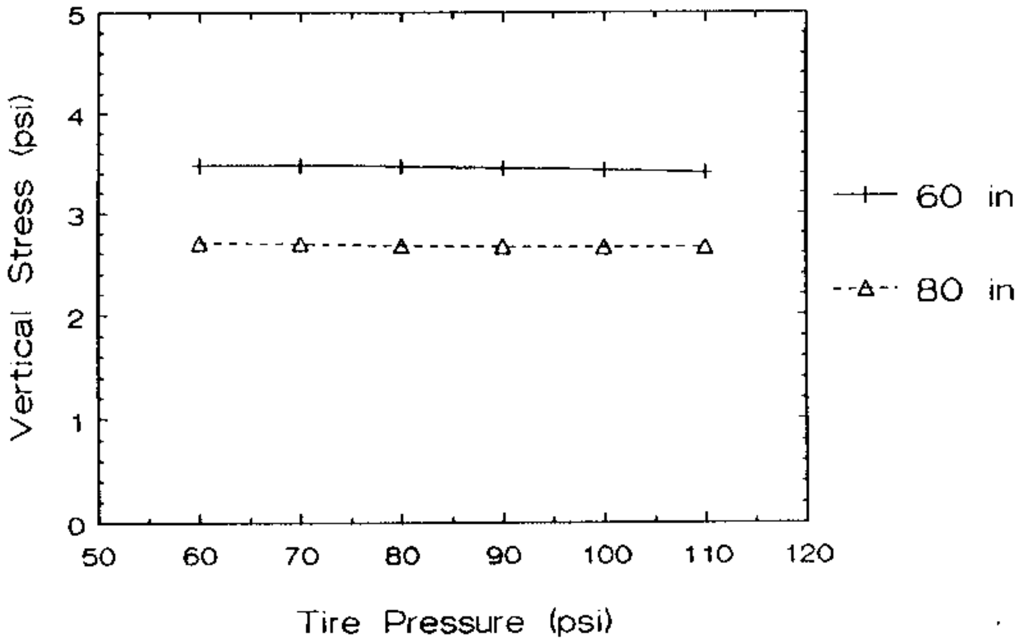
Top of the Granular Layer

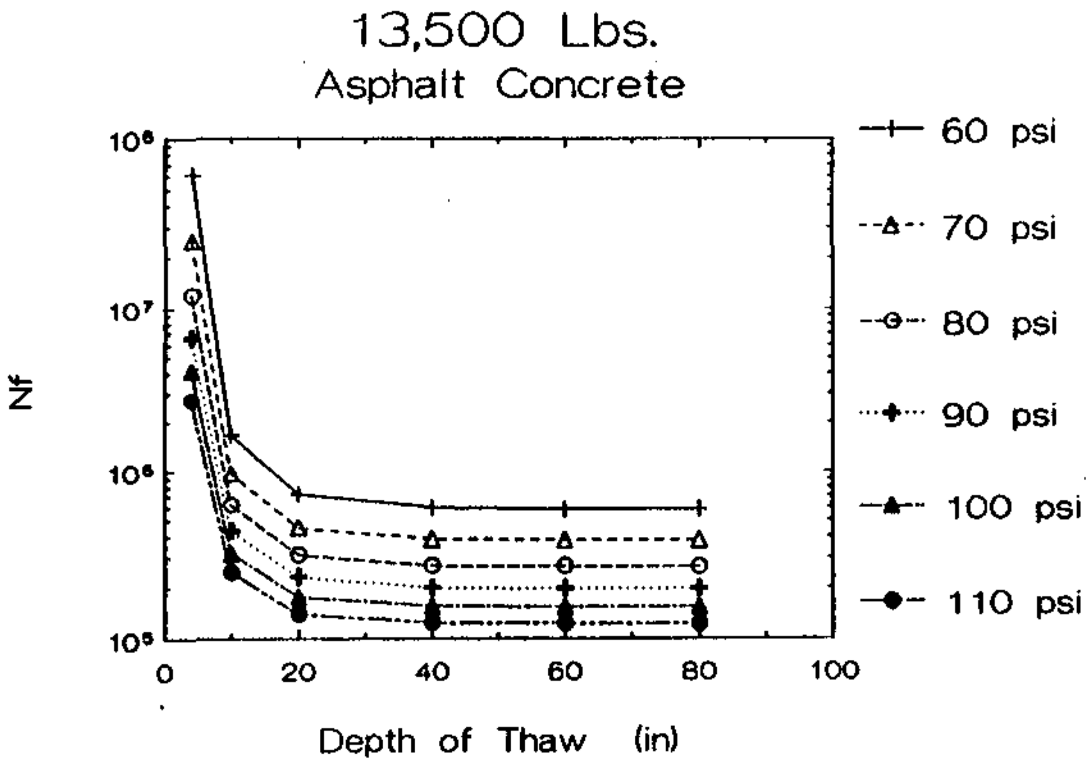


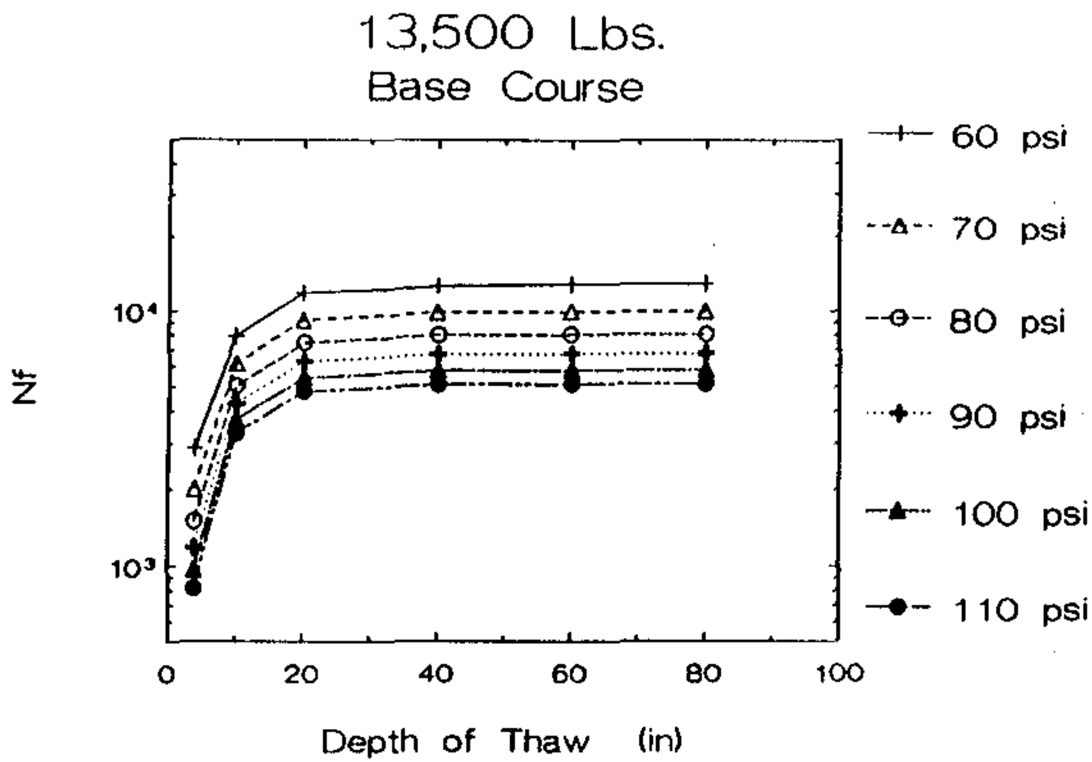

13,500 Lbs.
Top of the Granular Layer
Vertical Stress (psi)
Depth of Thaw (in)
100
90
80
70
60
50
40
30
20
0
20
40
60
80
100
60 psi
70 psi
80 psi
90 psi
100 psi
110 psi


13,500 Lbs.
Top of the Subgrade Layer
Vertical Strain (%)
4.00e-02
3.50e-02
3.00e-02
2.50e-02
2.00e-02
50
60
70
80
90
100
110
120
Tire Pressure (psi)
60 in
80 in

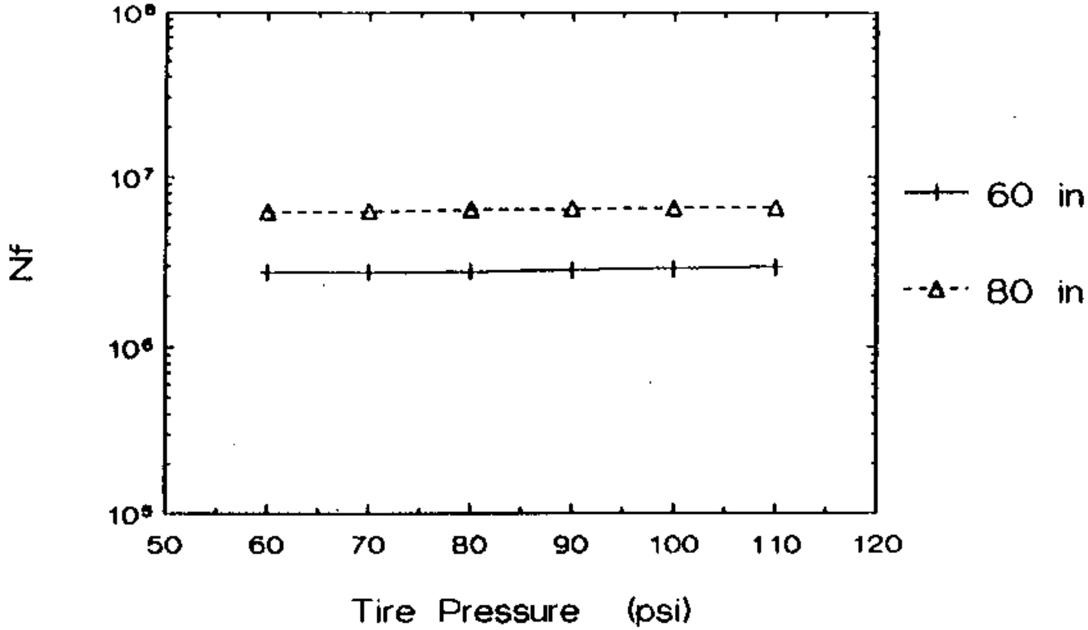
13,500 lbs
Top of the Subgrade

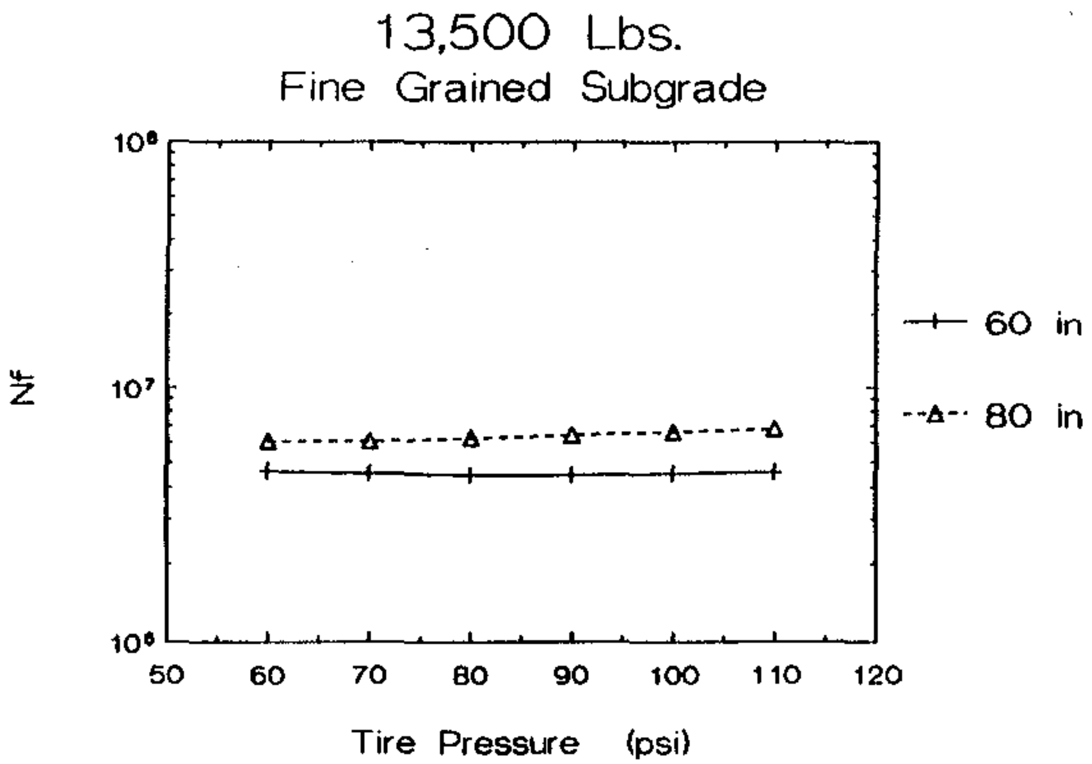


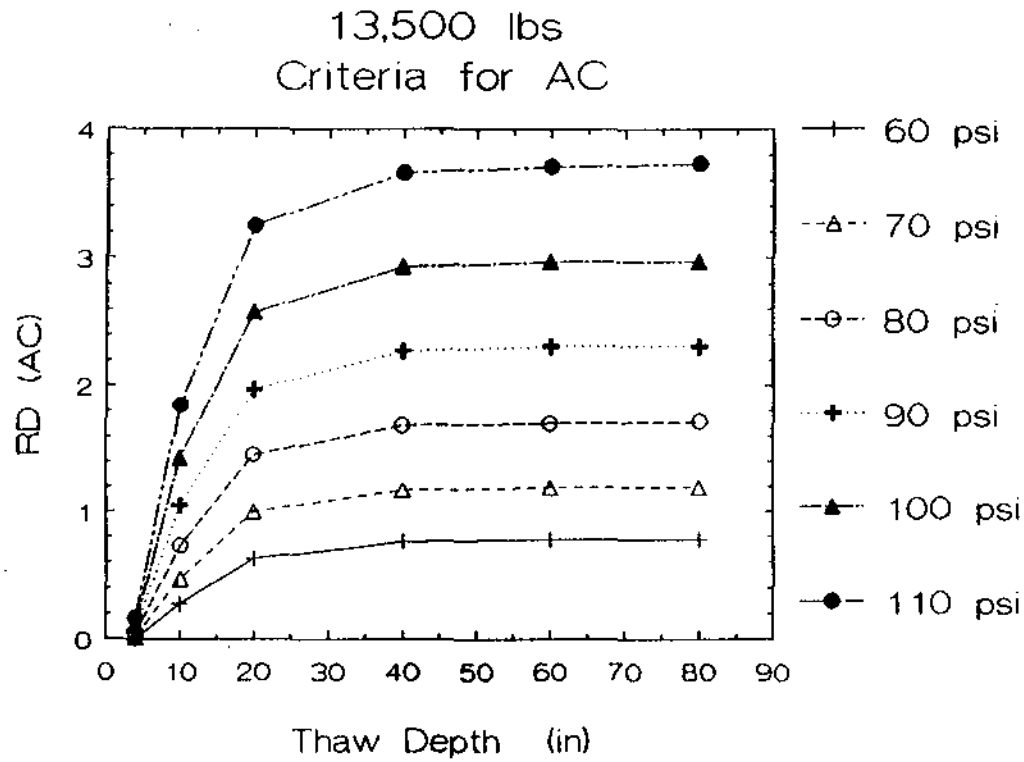

13,500 Lbs.
Asphalt Concrete
10⁸
10⁷
10⁶
10⁵
Nf
0
20
40
60
80
100
Depth of Thaw (in)
60 psi
70 psi
80 psi
90 psi
100 psi
110 psi

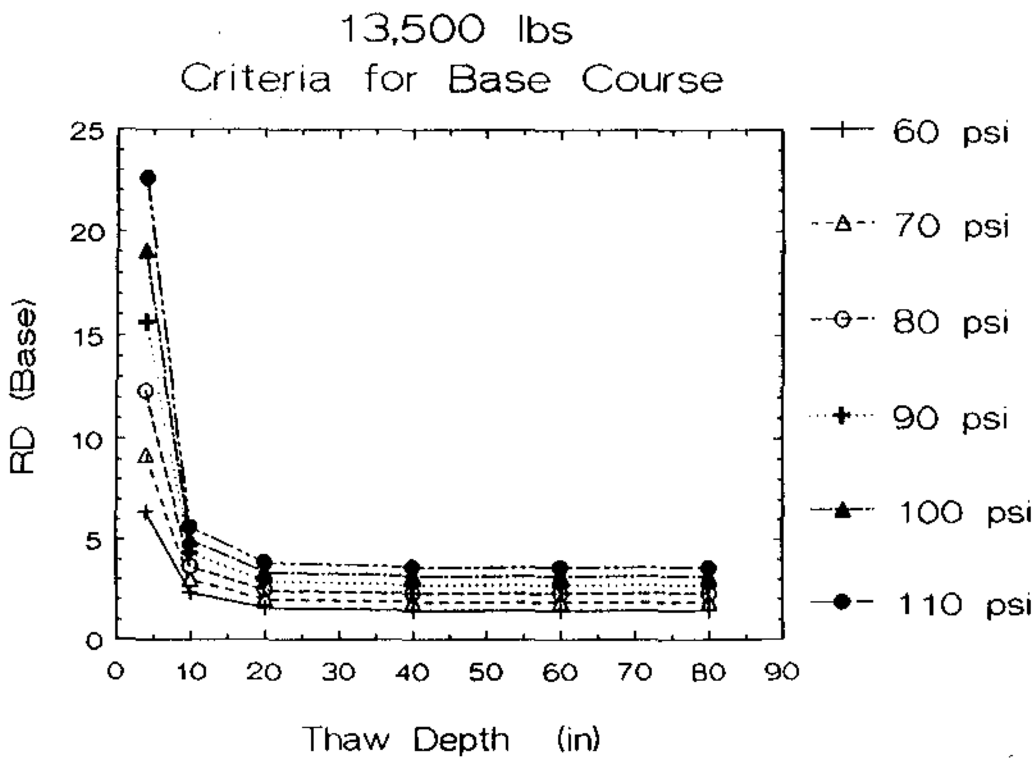

13,500 Lbs.
Base Course
Nf
10⁴
10³
0
20
40
60
80
100
Depth of Thaw (in)
60 psi
70 psi
80 psi
90 psi
100 psi
110 psi

13,500 Lbs.
Granular Subgrade

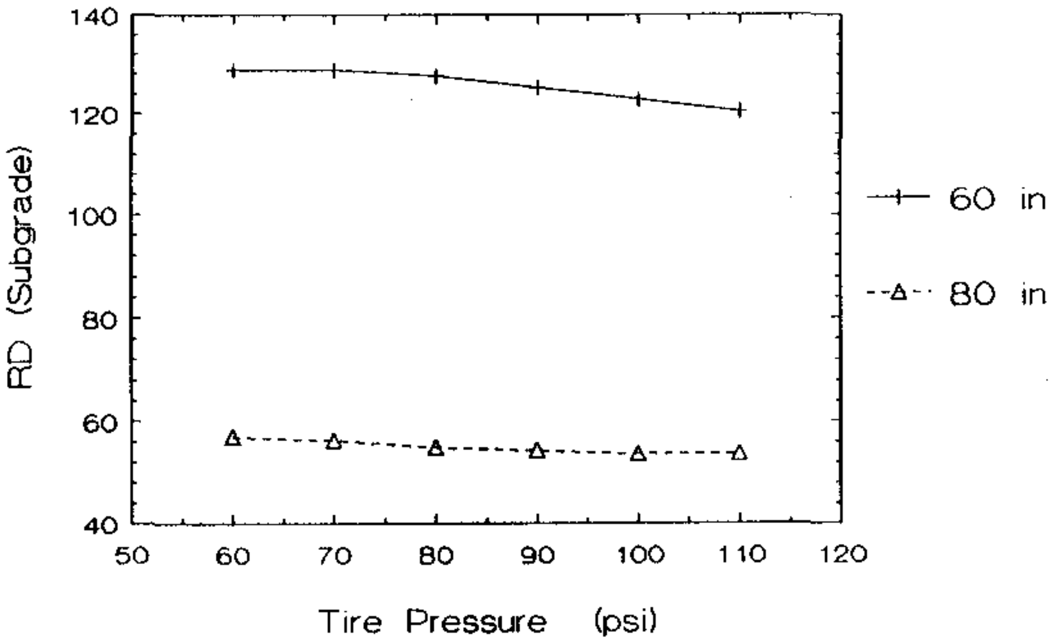


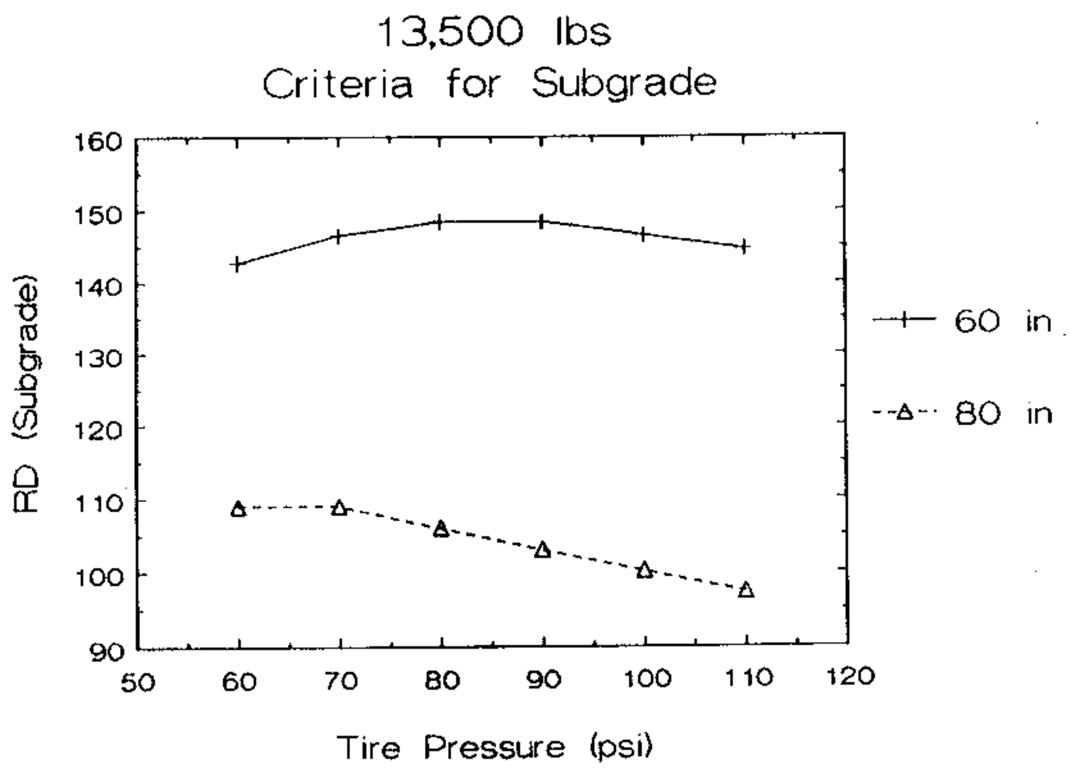

13,500 Lbs.
Fine Grained Subgrade
Nf
10^8
10^7
10^6
50
60
70
80
90
100
110
120
Tire Pressure (psi)
60 in
80 in


13,500 lbs
Criteria for AC
RD (AC)
Thaw Depth (in)
0
1
2
3
4
0
10
20
30
40
50
60
70
80
90
60 psi
70 psi
80 psi
90 psi
100 psi
110 psi


13,500 lbs
Criteria for Base Course
RD (Base)
Thaw Depth (in)
0
5
10
15
20
25
0
10
20
30
40
50
60
70
80
90
60 psi
70 psi
80 psi
90 psi
100 psi
110 psi

13,500 lbs
Criteria for Granular Subgrade





13,500 lbs
Criteria for Subgrade
RD (Subgrade)
160
150
140
130
120
110
100
90
50
60
70
80
90
100
110
120
Tire Pressure (psi)
60 in
80 in