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IR-02-01

Laboratory Evaluation of the Effects of Omitting Air-Entrainment on the Fresh and Hardened Properties of ALDOT Drilled Shaft and High Performance Bridge Concrete Mixtures

Submitted to

Highway Research Center
Auburn University

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JANUARY 2002

Final Report

on

Highway Research Center Research Project

Laboratory Evaluation of the Effects of Omitting Air-Entrainment on the Fresh and Hardened Properties of ALDOT Drilled Shaft and High Performance Bridge Concrete Mixtures

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ABSRACT

Entrained air has been used in concrete in the past to provide freeze-thaw resistance, reduce permeability, better overall durability, and improved workability. With the development of high performance concrete mixtures, some researchers suggest that entrained air is not needed in many instances where fly ash and/or micro silica are used to achieve good workability, low permeability, and durable concrete. Testing this on two different Alabama Department of Transportation (ALDOT) concrete mixtures was the impetus and purpose of this research.

Four different concrete mixtures were compared in fresh concrete properties/workability, and the hardened strength, permeability, and freeze-thaw durability properties. The four concrete mixtures included BR-A, ALDOT high performance bridge concrete, DS-A, ALDOT drilled shaft concrete, BR, ALDOT high performance bridge concrete without air entrainment, DS, ALDOT drilled shaft concrete without air entrainment. The investigation was limited to testing of laboratory prepared mixtures under ambient laboratory conditions.

Through extensive laboratory testing, the BR mixture showed superior results over its air-entrained counterpart, while the DS mixture did not. The BR mixture was better in every category tested than the BR-A mixture. It is believed the BR mixture would be a good replacement for the ALDOT standard high performance bridge concrete (BR-A). However, because this study was limited to laboratory testing under ideal environmental conditions, it is recommended that field verification testing be conducted before omitting A/E from the high performance bridge concrete mixture. It is also

believed that the DS mixture is not a good replacement for the ALDOT standard drilled shaft concrete (DS-A).

ACKNOWLEDGMENTS

This research was funded by the Highway Research Center (HRC) and Civil Engineering Department at Auburn University, and the PI is grateful to these sponsors for their support of the work. Also, the PI is grateful to Blue Circle Cement Company and Master Builders Technologies for their supplying of the cement and aggregate, and the concrete admixtures respectively used in this research. Lastly, special thanks are due to Mr. Sergio Rodriguez, Concrete Engineer of the Alabama Department of Transportation, and to Mr. Oscar Harper of Master Builders for their technical assistance in the concrete mixture designs.

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1. INTRODUCTION

1.1 Statement of Problem

The Alabama Department of Transportation (ALDOT) requires air-entrainment (A/E) in concrete exposed to salt water directly, in seal footing, drilled shafts, bridge substructures, bridge superstructures, prestressed concrete girders and piles, box culverts, retaining walls, and concrete safety barriers. Class "C" or "F" fly ash is allowed in almost all concrete produced for ALDOT and is required in all concrete for seal footings and in some prestressed girders, piles, and drilled shafts. Concrete produced with A/E with class "F" fly ash sometimes leads to problems in maintaining a consistent and acceptable level of A/E. Varying and larger doses of A/E are often needed to maintain the desired level of air with mixtures containing class "F" fly ash, and if too much A/E is added, the concrete strength may be lowered below an acceptable level.

Entrained air has been used in concrete in the past to provide freeze-thaw resistance, reduced permeability, better overall durability, and improved workability. With the development of high performance concrete mixtures, some researchers suggest that entrained air is not needed in many instances where fly ash and/or micro silica are used to achieve good workability, low permeability, and durable concrete. A pilot investigation of the performances of ALDOT's high performance bridge and drilled shaft concrete mixtures with and without A/E would be helpful in assessing the importance of using A/E in these mixtures. This was the impetus and purpose of this study.

1.2 Objective

The objective of this study was to compare the fresh concrete properties/workability, and the hardened strength, permeability, and freeze-thaw durability properties of ALDOT's high performance bridge and drilled shaft concrete mixtures when made with and without A/E to evaluate the effect of A/E on the performances of these two mixtures.

1.3 Work Plan

A brief plan of work to accomplish the above objective is given below.

1. Literature Review. A review of pertinent literature will be performed and a summary prepared.

2. Identify the exact mixture proportions and secure the necessary materials
   For the following 4 test mixtures:

   - ALDOT high performance bridge concrete
   - ALDOT high performance bridge concrete without A/E
   - ALDOT drilled shaft concrete
   - ALDOT drilled shaft concrete without A/E

3. Perform the following tests on each of the 4 mixtures in (2) above.

   - Concrete Temperature
   - Unit Weight
   - Air Content
   - Slump
   - Compressive Strength (7 day, 28 day, 56 day)
   - Rapid Chloride Ion Penetration
   - Freeze-Thaw Durability

4. Present, analyze, and compare results from the testing in (3) above to assess the relative performances of the mixtures and the importance of A/E on the performances.

5. Draw appropriate conclusions and recommendations and prepare final research report.

1.4 Scope

The investigation was limited to testing of laboratory prepared mixtures under ambient laboratory conditions. A more complete spectrum of mixture variations, parameters tested, and environmental placement conditions should be performed before any actions to possibly omit the use of A/E from high performance bridge and/or drilled shaft concretes are taken.

2. BACKGROUND AND LITERATURE REVIEW

2.1 Background

As indicated in Chapter 1, class "F" fly ash is allowed in almost all concrete produced for ALDOT, and it is required in concrete for seal footings and some prestressed girders, piles, and drilled shafts. The problem with using class "F" fly ash is that it sometimes leads to problems such as "eating" the air-entrainment. "Eating air" is an effect of the physical properties and chemical composition of fly ash. The fineness of the fly ash can make it difficult to develop or hold entrained air, and class "F" fly ash has a relatively high carbon level which causes it to partially absorb the air-entraining agent (AEA).

The current solution to this problem is to add more air-entrainment than needed to compensate for the air lost due to the addition of the class "F" fly ash. The technicians at the concrete ready-mix plant, who daily monitor the air levels in the batch mixes, usually carry out this overcompensation using a standard deviation chart of the level of air in the concrete. A mistake can lead to significant strength losses and even strength test failures in the field. An air content too low can cause failure in durability in the field. Differences in LOI or Loss on Ignition, measurements in the fly ash can throw off the uniform carbon content of the mix and cause high fluctuations in the levels of entrained air. The loss on ignition is a measurement of the amount of unburned carbon remaining in the fly ash and is an indicator of its suitability for use as a cement replacement in concrete.

Entrained air has been used in concrete in the past to not only provide freeze-thaw resistance, but to also achieve better overall durability and workability. It has been the opinion of researchers that air-entrainment must be used to provide durability to the concrete. "When concrete is used to construct roads and bridges subjected to freezing temperature, freeze-thaw durability is an important concern. To provide freeze-thaw resistance, air entrainment needs to be introduced" [21].

With the development of high performance concrete mixtures, some researchers have suggested that entrained air may no longer be needed. Mr. Vance Robinson, a quality control specialist at The Concrete Company in Columbus Georgia said, "A good water/cement ratio in high strength concrete should be able to provide adequate durability." It is the opinion of Mr. Robinson as well as others who work with admixtures and the design of concrete mixes, that working with air-entrainment and class "F" fly ash can cause a lot of problems in providing quality control.

The ALDOT requires that drilled pier shafts and high strength concrete bridge girders use air entrainment throughout as well as class "F" fly ash in some cases. ALDOT uses air-entrainment mainly to provide freeze-thaw durability and improve workability (without adding additional water to the mix). The inherent problem with using air-entrainment in high strength concrete is that the air reduces the strength of the concrete. It introduces voids or air bubbles that could have otherwise been filled with cement or aggregate. One proposed solution to the durability issue of high strength concrete could be simply to use a good (low) water to cement ratio. This is usually done anyway to achieve high strength, and thus the addition of A/E maybe unnecessary and in fact hurtful. Also, winter temperatures in Alabama are rather mild and hard freezes are uncommon, and when they occur only the top 6 inches or so of the ground will freeze. Because of this, the need to provide A/E in drill shaft concrete to protect it from freeze-thaw durability problems is questionable.

This research investigates the effects of removing air entrainment from high strength concrete in bridge girders and drilled pier shafts in the state of Alabama, via testing laboratory specimens of high performance bridge and drilled shaft concrete, with and without air entrainment. Prior to the experimental testing, a literature review on AEA and its combined usage with class "F" fly ash was conducted and is included on the following pages. The review also discusses the level of freezing and the annual number of freeze-thaw days in the state of Alabama.

2.2 Literature Review

The primary literature of importance to this investigation is that pertaining to the effect of air-entrainment on the fresh and hardened properties of concrete. Of particular interest is its effect on the durability, as assessed by freeze-thaw and rapid chloride ion penetration permeability testing, of high performance concrete. Also of interest is the depth of frost penetration in the ground and the cold weather eliminate conditions typically found in Alabama. A review of the literature in these area is reported on in the following sections and in Chapter 3.

The ACI, "Guide to Durable Concrete", [1] reports that there is general agreement that cement paste can be made completely immune to damage from freezing temperatures by means of entrained air, unless special exposure conditions result in filling of the air voids. Without entrained air, the paste matrix surrounding the aggregate particles may fail when it becomes critically saturated and is frozen. However, if the matrix contains an appropriate distribution of entrained air voids characterized by a spacing factor less than about 0.008 in. (0.20 mm), freezing does not produce destructive stress. Unfortunately, air entrainment alone does not preclude the possibility of damage of concrete due to freezing. Freezing phenomena in aggregate particles must also be taken into consideration.

Reference [1] reports that concrete which will be exposed to a combination of moisture and cyclic freezing requires the following:

1. Design of the structure to minimize exposure to moisture
2. Low water-cement ratio
3. Air entrainment
4. Suitable materials
5. Adequate curing
6. Special attention to construction practices

Too little entrained air will not protect cement paste against cyclic freezing. Too much air will unduly penalize the strength. Recommended air contents of concrete are given in Table 2.1. It can be noted that air contents are given in Table 2.1 for two conditions of exposure - severe and moderate. These values are reported to provide about 9 percent of air in the mortar for severe exposure, and about 7 percent for moderate exposure.

Shah, et al. [18] report that "while it is widely recognized that ASTM C666, *Standard Test Method for Resistance of Concrete to Freezing and Thawing,* provides a useful comparative testing procedure, there is not any adequate correlation between accelerated lab test results and the observed behavior of mature concretes exposed to

Table 2.1. Recommended Air Contents for Frost-Resistant Concrete [1]

Nominal maximum aggregate size, in. (mm)		Average air content, percent*	
		Severe exposure†	Moderate exposure‡
⅜	(9.5)	7½	6
½	(12.5)	7	5½
¾	(19)	6	5
1½	(38)	5½	4½
3§	(75)	4½	3½
6§	(150)	4	3

*A reasonable tolerance for air content in field construction is ±1½ percent.

†Outdoor exposure in a cold climate where the concrete may be in almost continuous contact with moisture prior to freezing, or where deicing salts are used. Examples are pavements, bridge decks, sidewalks, and water tanks.

‡Outdoor exposure in a cold climate where the concrete will be only occasionally exposed to moisture prior to freezing, and where no deicing salts will be used. Examples are certain exterior walls, beams, girders, and slabs not in direct contact with soil.

§These air contents apply to the whole mix, as for the preceding aggregate sizes. When testing these concretes, however, aggregate larger than 1½ in. (38 mm) is removed by handpicking or sieving and the air content is determined on the minus 1½ in. (38 mm) fraction of the mix. (The field tolerance applies to this value.) From this the air content of the whole mix is computed.

There is conflicting opinion on whether air contents lower than those given in the table should be permitted for high strength (more about 5500 psi) (37.8 MPa) concrete. This committee believes that where supporting experience and/or experimental data exists for particular combinations of materials, construction practices, and exposure, the air contents may be reduced by approximately 1 percent. [For maximum aggregate sizes over 1½ in. (38 mm), this reduction applies to the minus 1½ in. (38 mm) fraction of the mix.]

natural freezing and thawing." A proposed modification to ASTM C666 recommends a 14 day air drying period before the sample is exposed to the first freezing cycle, to improve the correlation between the laboratory test data and observed field performance. It should be noted that the excellent freeze-thaw performance of drilled shaft concrete in Alabama to date will provide some degree of correlation for our freeze-thaw laboratory results and help to render them more conclusive.

In existing laboratory testing procedures (ASTM C666), identical freeze-thaw cycles are repeated in rapid succession. This does not simulate actual field conditions. Thus questions arise whether test results are applicable to actual differences in field exposure conditions. One such question pertains to the length of time concrete is held continuously below freezing temperatures. It has been reasoned that longer individual freeze periods at sufficiently low temperatures permit greater time for remaining unfrozen water to migrate to existing ice crystals. This would permit further growth of ice crystals and resulting distress that would not otherwise develop with shorter freeze cycles, despite a greater number of cycles under the latter condition.

To address this question, Stark [20] conducted laboratory testing in which concretes were subjected to two types of freeze-thaw cycling, long cycles and short cycles. Four sets of concrete specimens with different air-void characteristics were included in the test. Some sets were well air-entrained with 5 ±% air and a void-spacing factor $L < 0.008$ in., and some were under air-entrained. Weight and dynamic modulus of elasticity were monitored during the test period.

Temperature extremes for both freeze-thaw cyclings were approximately 55° F and -20°F. Based on previous work [22], the temperature of initial freezing of the concrete was selected as 26°F. All data analyses are based on this temperature. Tests were conducted on standard 3x3x11-1/4 in. concrete specimens continuously immersed in 4% solution of sodium chloride (NaCl).

Short cycling consisted of two cycles per day Monday through Friday, one cycle on Saturday, and none on Sunday. During overnight periods, most of Saturday, and all day Sunday, the concretes were held continuously at -20°F. Time above 26°F for each cycle lasted about four hours. Thus, 11 cycles per week were run in the short freeze-thaw cycle, and consisted of 124 hours of freezing below 26°F and 44 hours of thawing above 26°F.

Long cycling consisted of maintaining the concretes at -20°F for seven consecutive days, interrupted only by one four-hour thaw period above 26°F, i.e., the concrete underwent 164 hours of freezing below 26°F per week in the long cycling testing. Thus, for a one week period, 11 short freeze-thaw cycles were run, and one long cycle run.

Since the weight loss and dynamic modulus of elasticity results were similar and weight loss is a more direct measure of loss of surface mortar or surface deterioration, Stark [20] presented his results in graphical form of weigh loss vs. number of cycles or total test time, etc. These are shown in Figs 2.1-2.4.

Comparative effects of the long and short freeze-thaw cycles are indicated in Fig 2.1, where weight loss data are plotted for mixes with relatively high void-spacing factor,

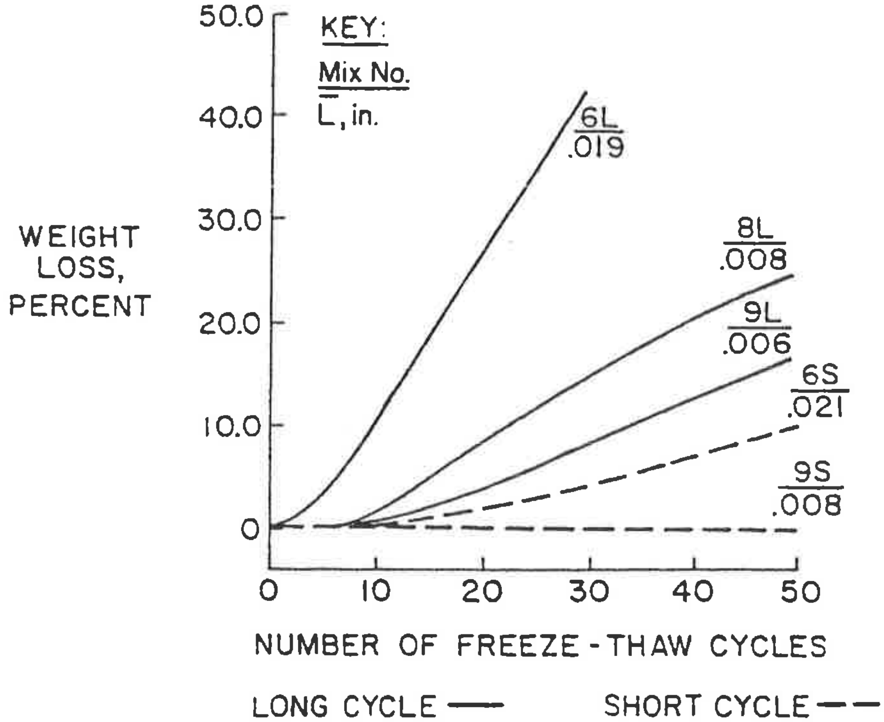


Figure 2.1. Comparison of Effects of Long and Short Freeze-Thaw Cycles on Weight Loss [20].

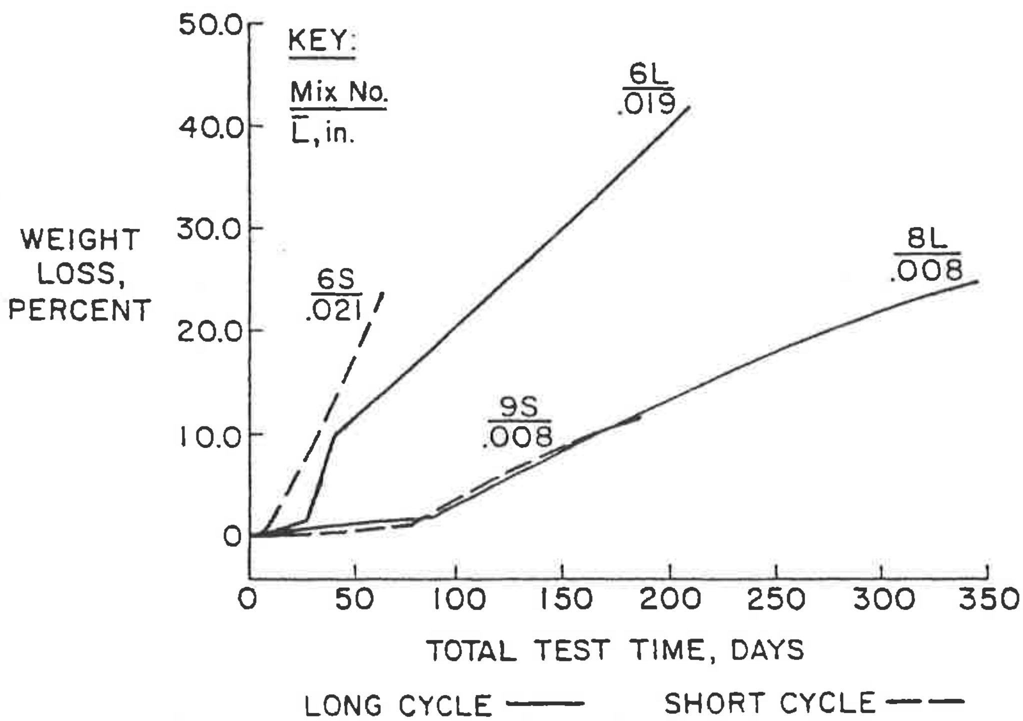


Figure 2.2. Relationship of Cumulative Test Time in Short and Long Cycle Exposure to Weight Loss [20].

L values (mixes 6L and 6S) and for mixes with L values at the 0.008 in. limit (mixes 8L and 9S). For a given number of cycles and L value, greater weight losses occurred in the long freeze-thaw cycle.

In addition to number of cycles, per se, the data suggest that several time-related factors associated with the type of freeze-thaw cycle also affected durability. These include cumulative length of freeze time (below 26°F) and of thaw time (above 26°F).

The relationship of overall test time to weight loss is shown in Fig. 2.2. For the concretes with high L values (mixes 6L and 6S), the short cycle produced somewhat greater rate of weight loss than the long cycle. However, for concretes with L values of 0.008 in. (mixes 8L and 9S), rate of weight loss was the same regardless of type of cycle. This suggests that number of freeze-thaw cycles was not the only controlling factor but that length of freezing or thawing time, individually, had decisive effects on durability.

An indication of the importance of cumulative lengths of freezing time and of thawing time are given in Figs 2.3 and 2.4. Note that these figures seem contradictory. However difference in behavior indicated in Fig 2.3 appears to be more normal data scatter than in Fig. 2.4.

Stark's results appear to be in agreement with Helmuth's [9] theory of freeze-thaw deterioration due to ice accretion. Briefly stated, Helmuth's theory is that when water freezes, ice crystals grow in entrained air voids and capillary pores in the paste matrix of the concrete. Under these conditions unsaturated flow develops in which water or solution is drawn to the growing ice crystal. The driving force for this flow is a thermodynamic free energy gradient that develops between the ice and unfrozen water,

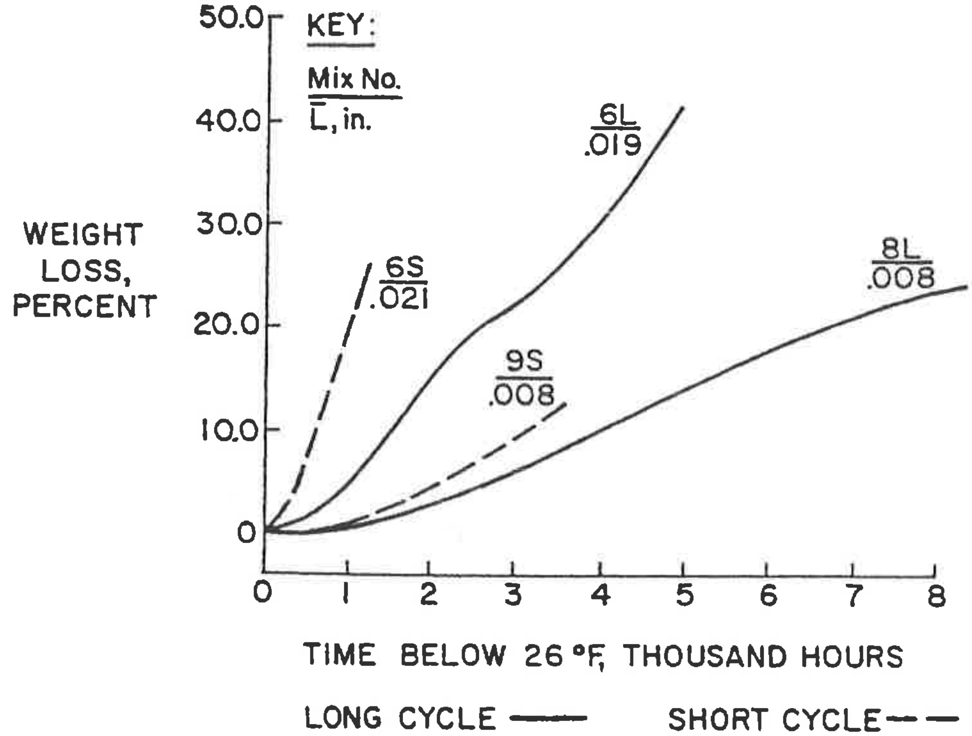


Fig. 2.3. Relationship Between Cumulative Time Below 26°F and Weight Loss [20].

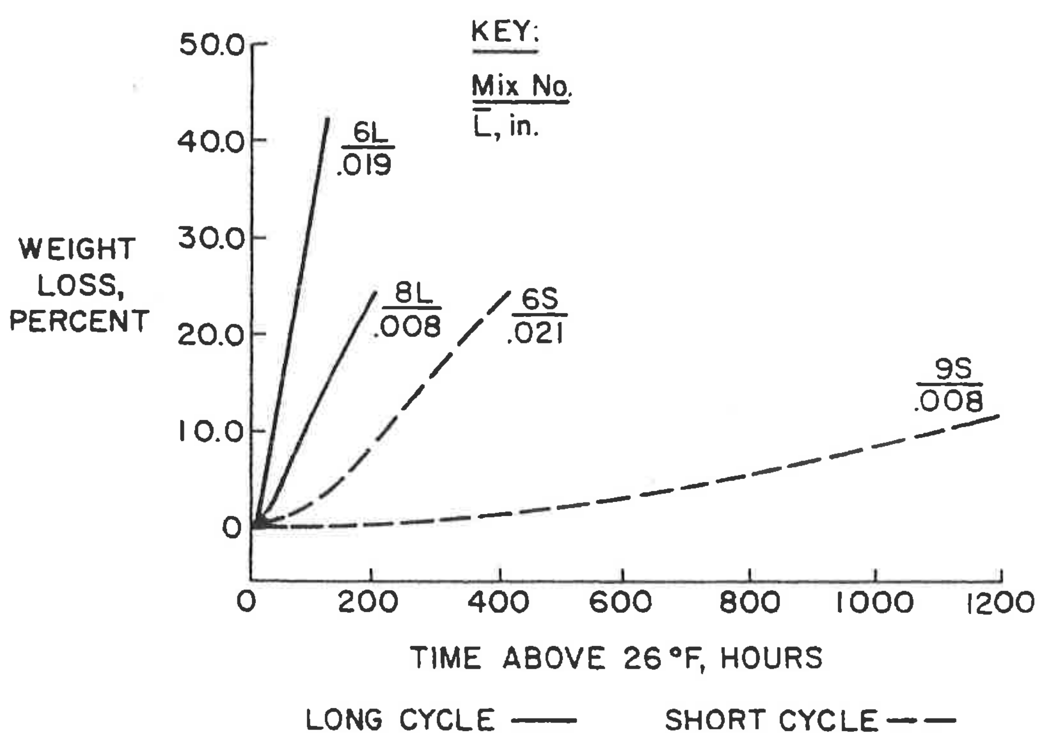


Fig. 2.4. Relationship Between Cumulative Time Above 26°F and Weight Loss [20].

with the ice being at the lower free energy level.

The longer the period during which a gradient exists, the longer the period during which moisture can diffuse to existing ice. Also, as temperatures continue to drop, new sites for ice formation can develop to which water also can be drawn. In addition, the more water that freezes, the greater the ice pressures that may develop. Distress may develop when ice pressures generated exceed the tensile strength of the paste.

Based on his data, Stark [20] concludes the following:

1. Longer continuous periods of freezing have more detrimental effects on the freeze-thaw durability of concrete than shorter periods, even in concrete that is considered to be properly air-entrained.

2. Cumulative length of thawing period also effects the freeze-thaw durability of concrete.

3. Results appear to agree with the ice accretion theory of frost action in concrete as developed by Helmuth.

Durability and High Performance Concrete. High performance concrete (HPC) has many definitions. The Federal Highway Administration (FHWA) definition states that "HPC is concrete that has been designed to be more durable and, if necessary, stronger than conventional concrete" [15].

One of the good properties of high performance concrete is its low permeability. Figure 2.5 shows the decrease of permeability (by a factor of approximately1/2.5) as the W/C ratio decreases from 0.5 to 0.3, and also the benefit of silica fume in reducing rapid chloride permeability. High performance concrete usually has a very dense structure of cement paste with a discontinuous capillary pore system. Figure 2.6 relates capillary porosity to permeability. Nawy [15] reports that capillary continuity hardly exists in dense

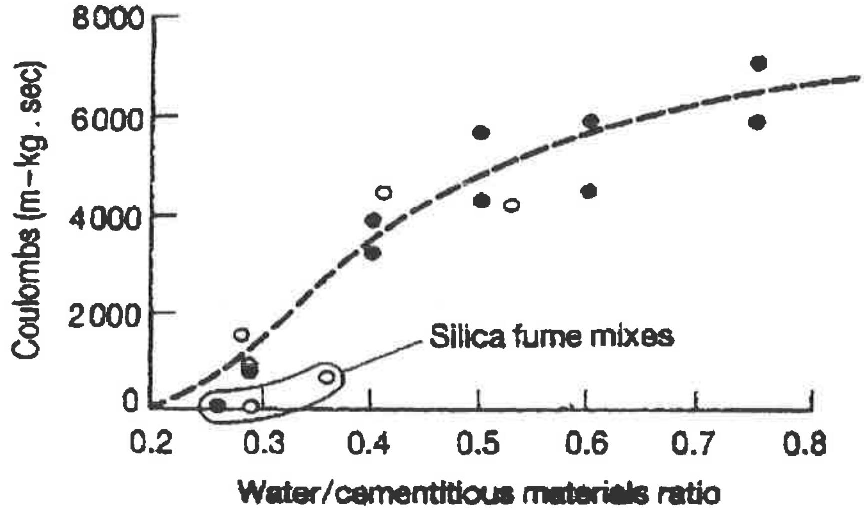


Fig. 2.5. Rapid Chloride Permeability vs. W/C Materials Ratio [15].

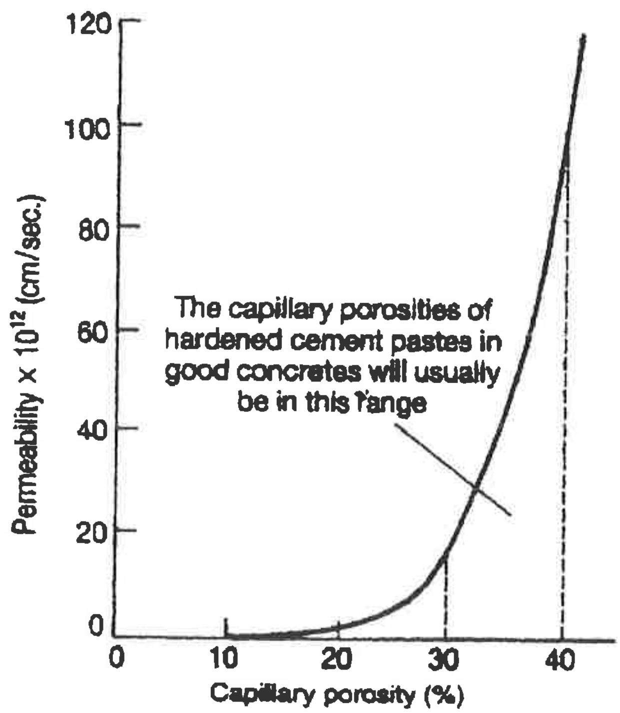


Fig. 2.6. Permeability of Portland Cement Paste with Capillary Porosity [15].

concrete such as that with W/C = 0.4, where capillarity almost stops in less than 3 days.

This dense structure also yields a high resistance to external penetration of water and other liquids. Research has suggested that the resistance is especially true with regards to chloride penetration, as was shown in Figure 2.5. The amount of penetration is usually considered to be negligible. Long-term performance with respect to freeze-thaw and wetting-drying depends on the degree of water penetration. This can be determined by the coefficient of permeability of the concrete matrix. High strength concrete that incorporates a high replacement level of Portland cement with a cementitious pozzolan, like silica fume, possesses the lowest permeability. See Figure 2.7. However, Figure 2.8, shows that if the silica fume content is raised to 20-30% of the cementitious content, the freeze-thaw resistance becomes only marginal.

High performance concrete has very good durability with respect to freezing and thawing. The main reason is that high performance concrete has a naturally low water/cement ratio. "In HPC where the aggregate size is normally small (3/8 to 1/2 in.), the W/C ratio is very low, and the quality control is maximized, freezing and thawing effects can be reduced considerably, if not eliminated altogether" [15]. A low W/C content leaves only a small amount of freezable water in the cement. Because of this, and because air-entraining agents tend to reduce the concrete strength, some researchers believe that air-entrainment is not needed and is in fact harmful in high strength concrete mixtures.

However, other researchers have found that larger doses of air-entraining agent may be needed in high performance concrete. This is particularly true in high strength

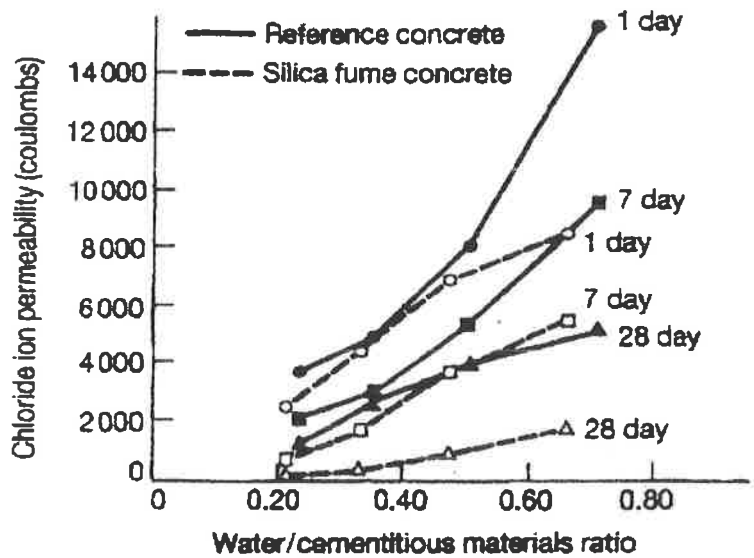


Fig.2.7. W/C Ratio vs. Chloride Ion Permeability [15].

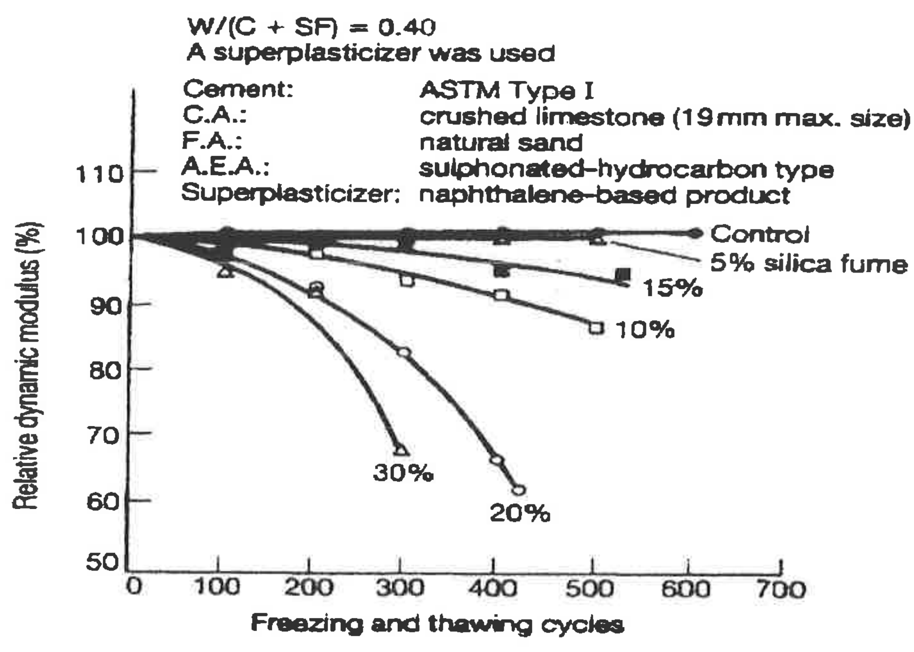


Fig. 2.8. High-Strength Concrete Performance in Freeze-Thaw Resistance for Different Silica Fume Contents [15].

mixtures and mixtures containing large quantities of fly ash. They say that air-entrainment is essential to provide the necessary amount of freeze-thaw protection. It is generally accepted that if properly air-entrained, concrete in general is not affected by freeze-thaw cycles. Fiorato [5] found that high strength concrete could only perform well if it is properly air-entrained. Figure 2.9 shows that the durability factor of non-air-entrained concretes is 5-40% of those that were air-entrained.

In service placement, under service conditions, such as bridge decks or dense overlays on decks, the surface of the concrete would in all likelihood dry out before it is ever exposed to any kind of freezing temperature, pending of course that it is not poured under freezing conditions. With its low permeability it is not very likely that the concrete will become resaturated.

Concrete deteriorates in freezing temperatures if it is exposed to the environment. Examples of this are bridge decks and in pavements. In the case of bridge girders, they are somewhat shielded from the environment because they are underneath the deck which acts as an umbrella to the weather. Deterioration that occurs normally in exposed situations is caused when moisture is allowed to seep into the concrete and fill up the voids. When the moisture freezes in cold weather, the ice causes concentrated stresses on the walls of the pores. These concentrated stresses cause cracking and spalling of the concrete surface, which in turn allows more moisture to enter and thus propagate the deterioration of the concrete. The serviceability of the structural element is gradually reduced. When the dilation or expansion pressure due to the expanding frost in the pores exceed the tensile strength of the surrounding concrete enclosure localized fracture results.

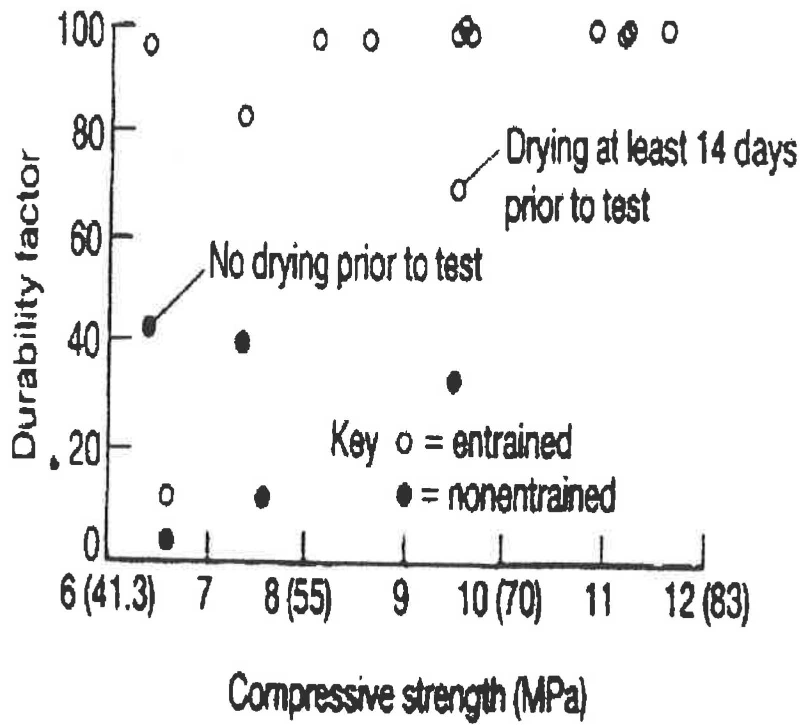


Fig. 2.9. Effect of Air-Entrainment on the Durability of High-Strength Concrete [9].

This fracture will then lead to surface scaling.

"The disintegration of concrete due to cycles of wetting, freezing, thawing, drying, and the propagation of resulting cracks is a matter of great importance" [15]. The addition of tiny evenly distributed air bubbles can increase the concretes resistance to freezing and thawing. These tiny air bubbles are introduced by using air-entrainment.

Air Entrainment and High Performance Concrete. Air-entraining agents are vinsol resins that introduce small air bubbles (0.05-1 mm. in diameter) that are evenly spaced throughout the concrete mixture. Air-entrainment is used to increase workability, as well as increase frost resistance, and increasing the concrete's resistance to deicing chemicals. Most air-entraining agents are readily available in a liquid form. They can also be found as powders, flakes, or semisolids. As stated above, the air bubbles reduce the stresses caused by frost introduction into the concrete by entering moisture.

The amount of the admixture needed to achieve a required air content is dependant on the size, shape, and grading of the aggregate that was used in the mixture. The finer the size of the aggregate used, the larger the percentage of admixture that is needed as indicated in Table 2.1. Other factors that change the amount of the air-entrainment needed are as follows: type and condition of the mixer, use of fly ash or other pozzolans, and the degree of agitation of the mixture.

The percentage of air-entrainment used should never exceed 8%, because the concrete compressive strength decreases as a function of the percentage of air. Davis et al. [4] report that the strength reduction of concrete is evident when the air content of the mixture exceeds 4%. Entrained air reduces the strength of the high performance concrete because the air bubbles created in the concrete cannot be fully compensated by a reduction in water content in the presence of a superplasticizer. It is difficult to have both air entrainment and a very low water/cement ratio in the same mixture. When using air entrainment it is generally desirable to have a maximum water/cement ratio so that freeze/thaw cycles will not damage the concrete.

Good quality control must be exercised when using air-entrainment in concrete. Overmixing and overvibration can cause the entrapped air bubbles to surface, and thereby cause the air-entrainment to lose its abilities to lessen the concentrated loads in the pores. Also, large unevenly distributed air voids can weaken the concrete and leave it impotent to resisting frost cycles. Other variations such as temperature, aggregate grading, fly ash, superplasticizers, and in mixing and placing are unavoidable and cause many problems in the air content.

Air Entrainment and Fly Ash. Fly ash is an admixture of fine particle ash that accumulates elecrtrostatically from the exhaust fumes of coal-fired power stations. It is considered an artificial pozzolan and its particles have at least the same fineness as the cement particles. The level of carbon found in the ash itself determines the class of fly ash. In some cases, the carbon content can be as high as 3%. Because of its pozzolanic properties, using fly ash can result in high strength mixtures.

"The inception of the Clean Air Act Amendments in 1990 has prompted many power generating plants to retrofit their facilities to accommodate low NO_x burners. The resulting changes to the combustion process undoubtedly reduces NO_x emission, but are often accompanied by higher unburned carbon contents" [11].

The carbon content in the fly ash is important because it has an adverse effect on the air entrainment in concrete. It has been proposed that fly ash carbon can sometimes absorb surfactants used in air entraining agents. Consequentially, there is a lower percentage of entrained air in the concrete. High carbon fly ash (such as Class "C" or "F") or varying carbon content is commonly avoided when air entrainment in concrete is a necessity. The high carbon content in the fly ash necessitates a need for more air-entraining agent.

Frost Resistance of High Strength Concrete. In producing normal strength concrete there are sometimes problems in attaining a stable air void system, and in the presence of a superplasticizer, the establishment of a good and stable air void system is an even bigger problem (Shah et al. [18]). When dealing with high-strength concrete, there is sometimes a requirement for entrained air, which decreases the concrete strength. There has been much interest in finding out whether a frost resistant high strength concrete can be produced without air entrainment.

In 1981, Okada et al. [17] reported very good frost resistance of non-air-entrained high-strength concrete with W/C ratios in the range of .25 to .35. Similar observations were noted by Foy et al. [6] and Gagne et al. [7]. They observed good frost resistance at W/C content of .20 to .30, respectively.

Using the guidelines set forth by ASTM C666-A, Malhotra et al. [12] also conducted freeze-thaw testing of concrete samples. The W/C ratios of these samples were around 0.3 to 0.35, which was significantly higher than those of Okada, Foy or Gagne [17, 6, 7]. Malhotra concluded that air-entrainment was necessary for those samples to be considered frost resistant. Hammer and Sellevold [8] tested non air-entrained concrete with 0 and 10% silica fume and a W/C ratio that varied from 0.25 to 0.40. They found that even in the lowest W/C ratios some of the specimens were damaged during testing. However, Hammer and Sellevold suggested that damage could have been due to thermal fatigue caused by too large of a difference between the thermal expansion coefficients of aggregate and binder rather that because of ice formation.

Depth of Frost Penetration. Sowers [19] reports that when the daily mean temperature remains below 32°F for a period longer than three or four days, the soil moisture at the ground surface freezes. The longer and the more intense the cold spell, the greater the depth to which the freezing extends. The temperature deep in the ground remains nearly constant throughout the year, while the temperature just below the ground surface fluctuates with the air temperature. After a period of cold weather in which the air temperature is below freezing, a thermal gradient is established in which the air temperature is below freezing, a thermal gradient is established in which the 32°F point is below the ground surface, as shown in Fig. 2.10. This point defines the frost line.

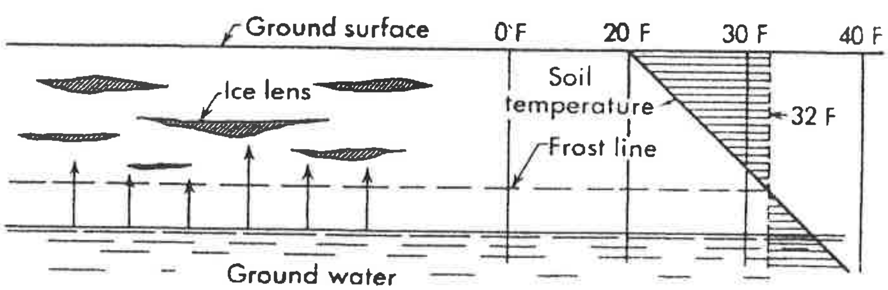


Figure 2.10. Formation of Ice Lenses in Zone of Freezing [19].

Neither the frost line nor the thermal gradient is fixed; both vary with the duration and intensity of the cold. The frost line is not found at a uniform depth, for it depends on the density, saturation, and composition of the soil. Above the frost line the temperature is below the ordinary freezing point for water.

As indicated above, the depth below the ground surface to which a 32°F temperature extends is termed the frost line. Above the frost line, freezing occurs and ice lenses will form if the soil and water conditions are right. The depth of the frost line depends primarily on three factors: the air temperature, the length of time the air temperature is below 32°F, and the ability of the soil to conduct heat. The lower the air temperature and the longer it remains below 32°F, the greater the depth of the frost line; and the higher the thermal conductivity of the soil, the greater the depth of frost penetration. Figure 2.11 shows the approximate maximum depth of frost penetration in the United States. On mountain tops the depths will be greater and in highly organic soils or coarse gravels above the water table, the depth will be smaller. Figure 2.12 shows an approximate frost-depth map of the U.S. by Bowles [2]. Comparing the maps of Figs 2.11 and 2.12, one can appreciate the approximate nature of the values shown. An average of the two indicates a range of frost-depth penetration of approximately 0-18 in. for Alabama.

It should be noted that whereas the freezing temperature of water in the soil is 32°F as it is in air, as indicated earlier, previous research [22] has determined the temperature of initial freezing of water in concrete to be around 26°F.

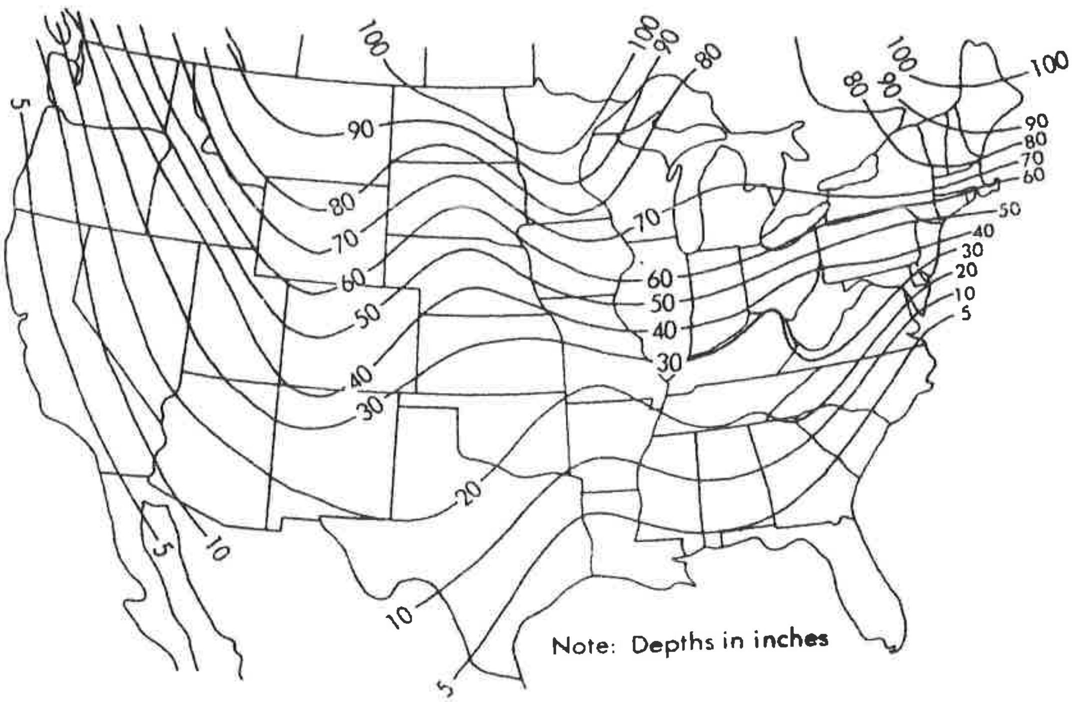


Fig.2.11. Maximum Depth of Frost Penetration in the United States [19].

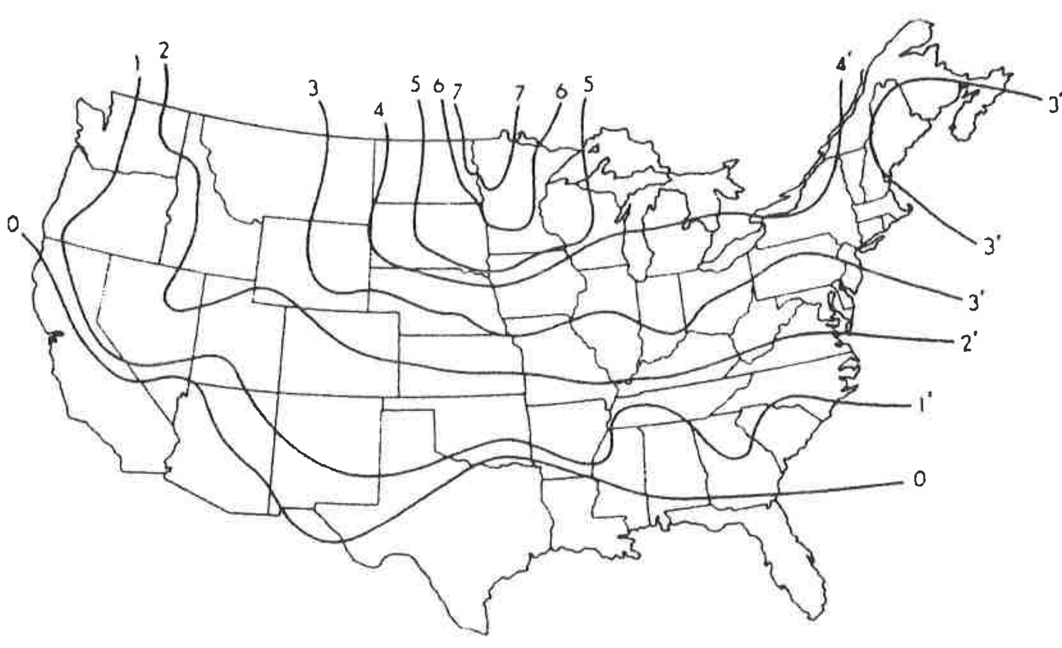


Fig. 2.12. Approximate Frost-Depth Contours for the United States, Based on a Survey of a Selected Group of Cities [2]

3. Cold Weather Climate Conditions in Alabama

3.1 General

The climate in Alabama is generally such that there are hot summers and mild winters. The July temperatures average about 85°F while the winter temperatures average 46°F. The average annual temperature is about 60°F in north Alabama to about 70°F in the south. The range of temperatures in Alabama is from -27°F (in 1966) to 112°F (in 1925). The average yearly rainfall is approximately 65 inches. Alabama is sometimes prone to an occasional hurricane in the summer or fall.

El Niño has had an effect on precipitation. In Mobile and Baldwin counties, El Niño brings an additional 3.2 inches of precipitation annually. In the northern part of Alabama El Niño has meant a drier temperature in the winter and an earlier spring. Rainfall and snow reduction in the north has been recorded as much as 1.9 inches.

3.2 Annual Number of Freezing Days

According to the National Climatic Data Center (NCDC), a division of NOAA weather, the number of days that the temperature drops below 32°F in Alabama is approximately 44 days per year. Using NCDC data from weather stations located in major cities in Alabama and the southeastern region of the United States, approximate mean annual number of days that the temperature drops below 32°F contour lines were estimated for Alabama and the southeastern region. These contour lines are shown in Figs 3.1 and 3.2. Also shown on these figures are the NCDC mean annual numbers of days that the temperature drops below 32°F data at major cities/weather stations in Alabama and the southeast. The data summarized in Figs 3.1 and 3.2 are current and represent the

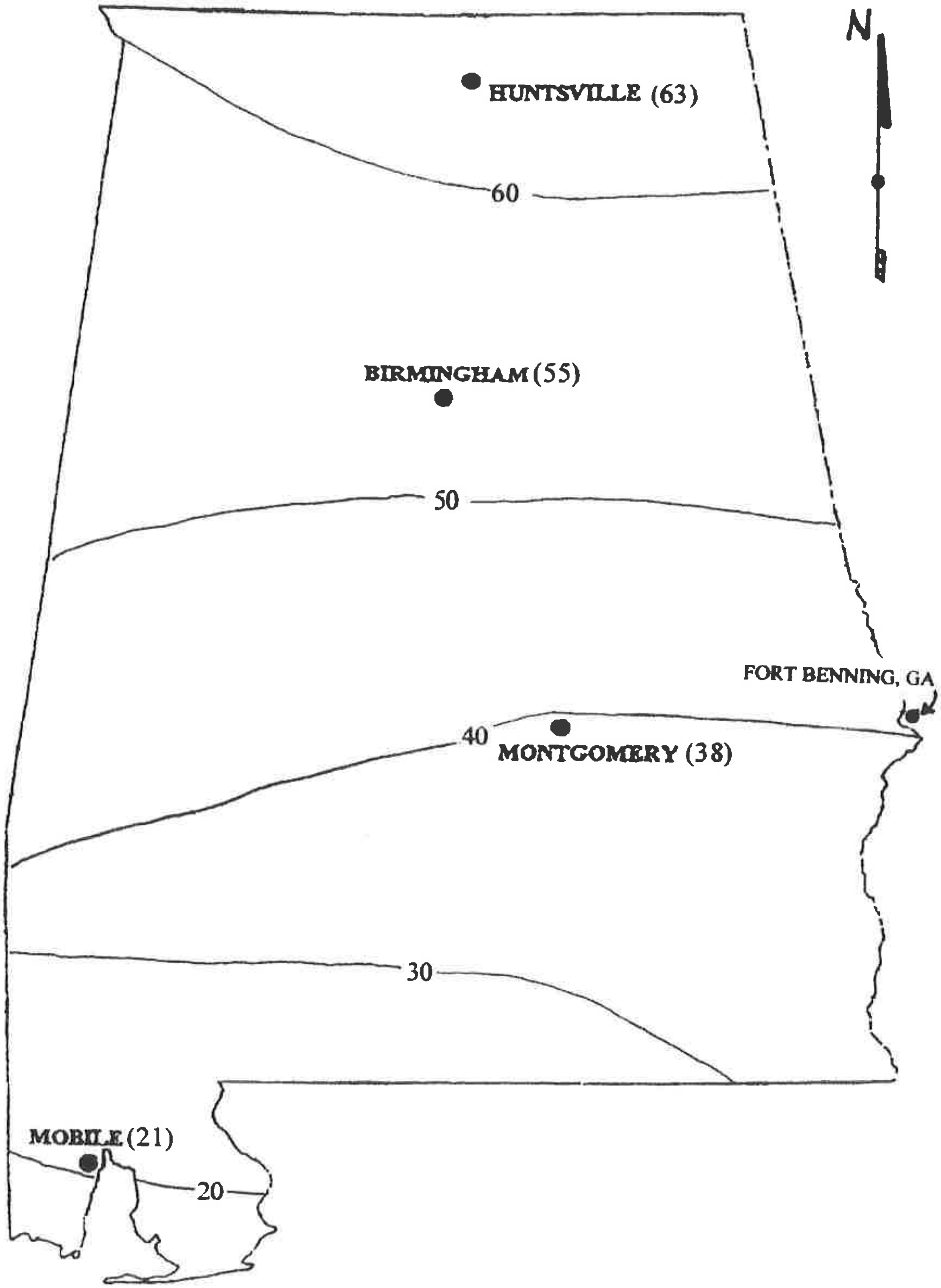


Figure 3.1. Average Annual Number of Freezing Days (below 32°F) in Alabama.

Fig. 3.2. Average Annual Number of Freezing Days (below 32°F) in Southeast

means over a 35-year period from 1964-1999. Note in Fig. 3.1 that the average annual number of freezing days in Alabama is substantial, varying from approximately 20 in the extreme southern portion of the state (Mobile and below), to 30 in south Alabama, to 60 in north Alabama. It should be noted that south Alabama, and particularly the extreme southern portions, are where most of the concrete pile foundations are employed in the state.

With El Niño providing for warmer winters the number of freeze/thaw days could decrease. This would be better for high performance or drilled shaft concrete without air entrainment. Based on a state average of 44 day per year freeze/thaw cycle (NCDC), a freeze/thaw cycle test of 300 cycles would be able to effectively simulate 6.82 years of freezing conditions on our test batches. In north Alabama the average number of freezing days is as many as 63 days. Based on this a freeze/thaw cycle test of 300 cycles would be able to effectively simulate only 4.76 years of freezing conditions on our test batches.

3.3 Number of Days and Duration of Time Below 26°F

Since the literature indicates the freezing temperature of water in concrete is approximately 26°F rather than 32°F, it was decided to investigate the number of days that the temperature dropped below 26°F. Hourly temperature data for the months of December 2000 and January 2001 (our coldest 2 months of the 2000-2001 winter) was secured from the U.S. Army weather station at Fort Benning, which is located in Columbus, Georgia. This weather station is located just across the eastern border of Alabama as shown in Fig. 3.1. The data for each of the 62 days were examined, and it was determined that in 37 of the 62 day (60% of the days) the temperature dropped below

32°F. Temperature-Time plots were constructed for each of these 37 days, and a typical daily plot is shown in Fig. 3.3. It was further determined that on 28 of the days (45% of total days), the temperature continued to drop below 32°F and on below 26°F. The 32°F and 26°F temperature lines are shown in Fig. 3.3. The 37 Temperature-Time plots are presented in Appendix A.

The duration of the time below 32°F for the 37 days is shown in histogram form in Fig. 3.4, and the median duration was approximately 8 hours. Similarly, the duration below 26°F for the 28 days is shown in Fig. 3.5 and for this case the median duration was approximately 2 hours.

It is the opinion of many of the weather personnel at Fort Benning, especially that of A1C Russell Ward, that the Winter of 2000-2001 (over which the data was collected) was abnormally cold. This claim can be substantiated by evidence from the NCDC and the histograms from Fort Benning. Using NCDC data collected over the past 35 years throughout the state of Alabama, the months of December and January on average comprised 58.9% of the total freezing days per year. The NCDC data gives an average of 25.5 freezing days for the normal winter months of December and January. Using the data collected from Fort Benning for the months of December and January for the winter of 2000-2001, there were 37 days where the temperature dropped below 32°F. This is an increase of freezing days of 31.1%. Alternatively, if the trend for the past 35 years holds true, then the months of December and January for the Winter of 2000-2001 also comprised 58.9% of the total freezing days per year. Extrapolated this means that there were approximately 62.8 freezing days this year. This also shows that there was a 30%

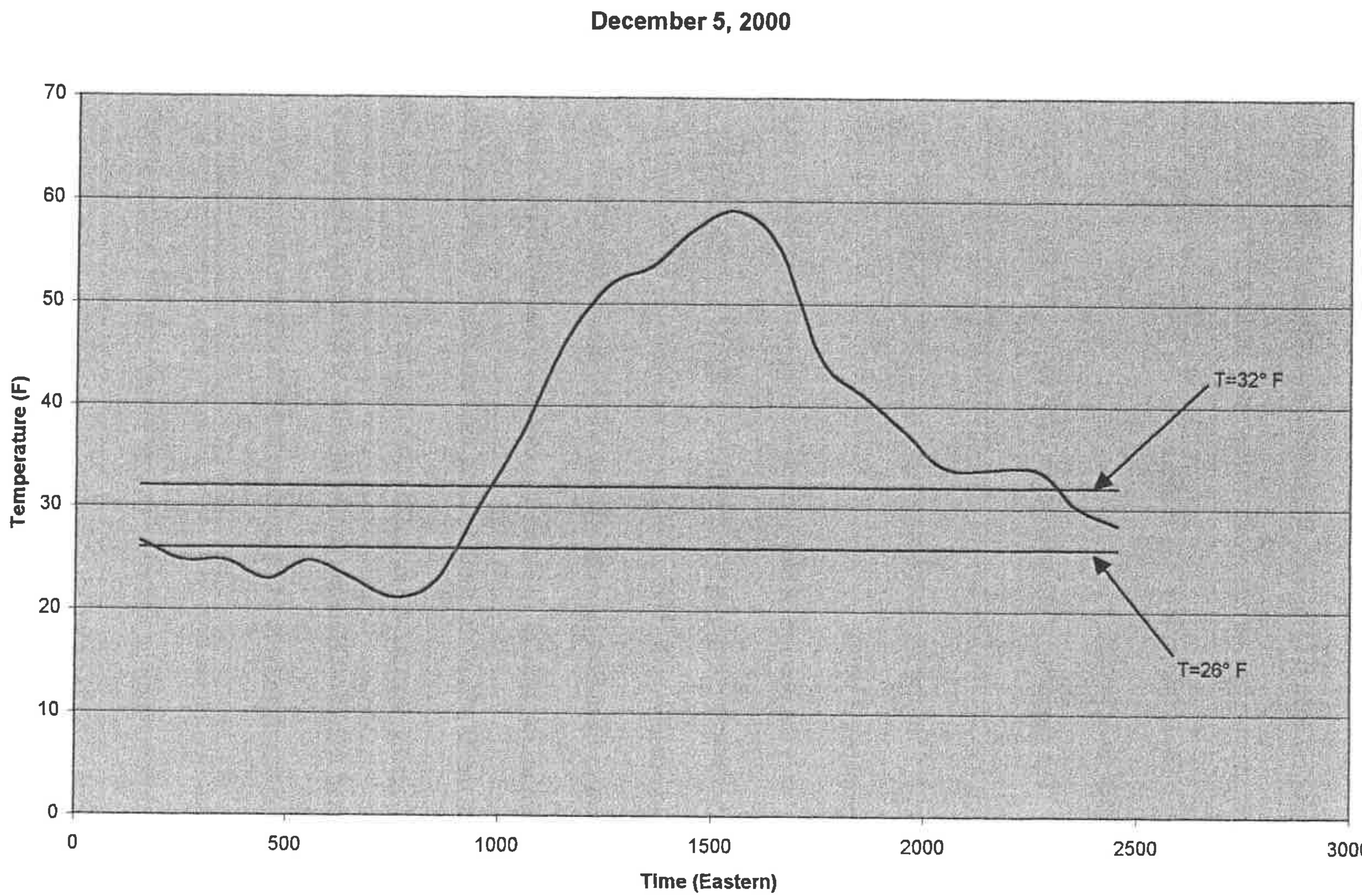


Figure 3.3 Typical Daily Temperature-Time Plot of Ft. Benning Data

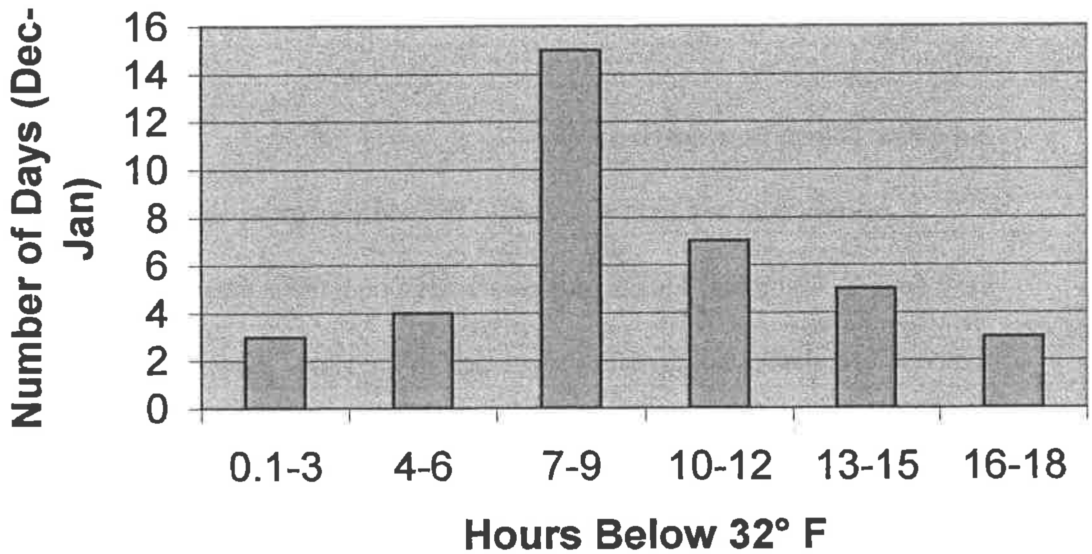


Fig. 3.4. Histogram of Duration of Time Below 32°F for 37 Freezing Days.

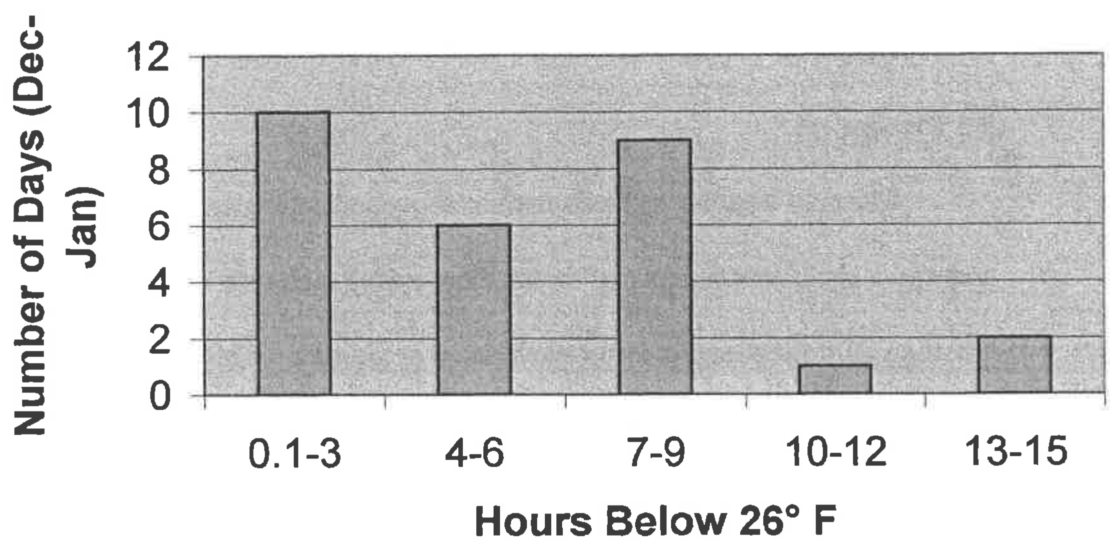


Fig. 3.5. Histogram of Duration of Time Below 26°F for 28 Below 26°F Days.

increase in number of freezing days. Thus the Winter of 2000 - 2001 was not only unseasonably cold, it also had 30 – 31% more subfreezing days than a normal Alabama winter.

3.4 Frost Depth Penetration

As indicated in Chapter 2, due to the warm climate, the ground does not freeze very deep in Alabama. This is particularly true in south Alabama, which is where most of the concrete pile foundations are employed in the state. According to Highway Research Board (HRB) Publication 211 [10], the maximum depth of frost penetration in inches in the state of Alabama is 17 in. That report seems to indicate that when designing drilled pier shafts in Alabama, air-entrained concrete could be placed to a depth of 2 feet below ground level and nonair-entrained concrete below this depth and this would satisfactorily meet the requirements of freeze/thaw conditions in Alabama. Figure 3.6 shows the frost depth penetration in Alabama and throughout the U.S. from HRB Publication 211 [10].

3.5 Closure

The cold weather and information above indicates that all regions of Alabama undergo a significant number of freeze/thaw cycles each year and thus during the service life of an outdoor concrete structure located in the state. This coupled with the fact that average annual rainfall is high make condition for potential freeze/thaw damage to concrete structures in the state very real. This indicates that the current practice of using air entrainment in outdoor/exposed concrete should be continued.

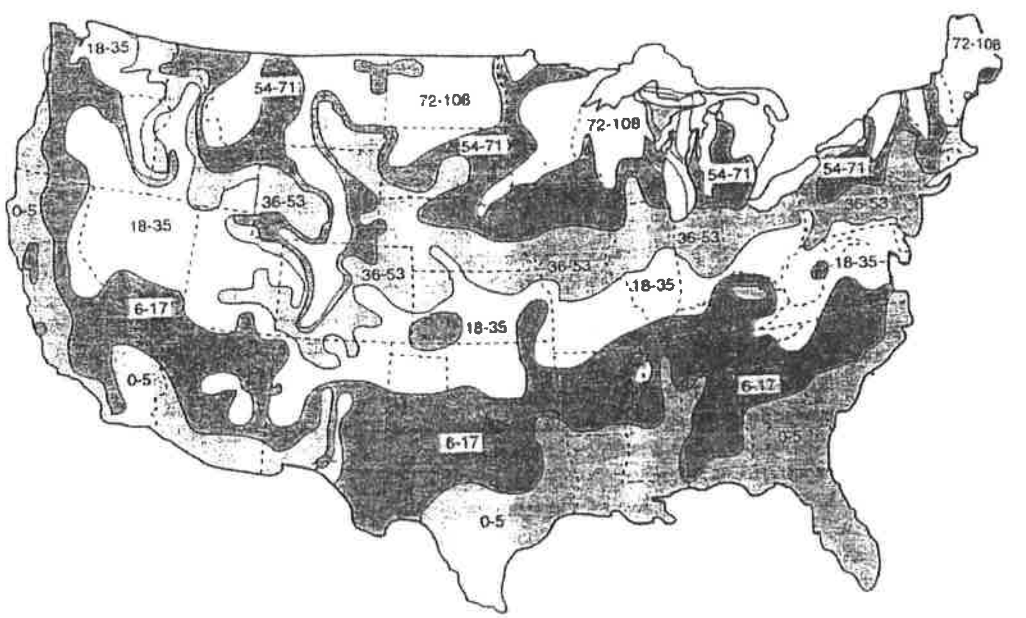


Fig. 3.6. Frost Depth Penetrations for the United States (from Frost Action in Roads and Airfields, HRB Publication 211 [10]).

The fact that frost depth penetration varies from about 0-17 inches in the state (from south to north Alabama), and this would be even smaller for freezing of water in concrete, the need for air-entrainment in piling and drilled shafts is very questionable and The fact that frost depth penetration varies from about 0-17 inches in the state (from south to north Alabama), and this would be even smaller for freezing of water in concrete, the need for air-entrainment in piling and drilled shafts is very questionable and in fact probably not needed. Even in the upper regions of these structural members where they are exposed to subfreezing temperatures, because of their vertical orientation, they should not be in a saturated condition and thus freeze/thaw should not be a problem. An exception to this would be in regions where the water table is less than 18 inches below the ground surface. However, even for piling and drilled shafts, an air content of 5% is still probably a good design practice. The air reduces cost, improves workability of the mixture (and thus less water can be used), helps achieve a quality placement and consolidation of the in-place concrete, and provides freeze/thaw protection at a minimal reduction in compressive strength.

4. LABORATORY TESTING PROGRAM

4.1 General

The objective of this research was to identify whether or not air entrainment could be eliminated from the mix designs for high performance concrete bridge girders and drilled pier shafts. The research plan to determine this was to conduct laboratory testing to evaluate the fresh and hardened properties of ALDOT's high performance bridge and drilled shaft concrete mixture with and without A/E.

In general, air-entrainment in concrete most strongly affect the concrete's

- workability
- strength
- permeability
- freeze-thaw durability

Thus, concrete mixtures with and without A/E should be tested to evaluate these properties/characteristics to assess the most significant affects of the A/E. Slump, compressive strength, rapid chloride ion penetration and freeze-thaw durability testing is obviously required to assess the above properties/characteristics. The specific concrete mixtures examined and the fresh and hardened properties tested are presented below.

4.2 Concrete Test Mixtures

ALDOT's high performance concrete test bridge project constructed in 1999 on Alabama Highway 199 in Macon County, Alabama employed two different high performance concrete mixture designs. Both mixtures contain A/E, and one was taken as the standard high performance concrete mixture design and used in this study. ALDOT's standard drilled shaft concrete mixture (which contains A/E) was taken as the standard for comparison of the drilled shaft mixtures. Thus, ALDOT's high performance bridge concrete mixture with and without A/E and their standard drilled shaft concrete mixture with and without A/E were the four mixtures examined in this study. These mixtures, with their designations in this report, are shown below.

BR-A: ALDOT standard high performance bridge concrete with 5% entrained air

BR : ALDOT standard high performance bridge concrete without air entrainment

DS-A: ALDOT standard drill shaft concrete with 5% entrained air

DS : ALDOT standard drill shaft concrete without air entrainment

The mixture proportions are shown in Table 4.1.

4.3 Fresh Concrete Property Testing Program

The fresh concrete property testing program was conducted to evaluate the workability/construction ease of the candidate mixtures. It consisted of conducting the following tests on each of the mixtures under ambient laboratory conditions.

- Concrete temperature
- Unit weight
- Air content
- Slump

Two replicas of each of the tests for each of the mixtures were made.

4.4 Hardened Concrete Property Testing Program

The hardened concrete property testing program was conducted to evaluate the important mechanical properties and characteristics of the hardened concrete for each of the mixtures. Standard ASTM type of testing conducted and the properties tested and evaluated for each of the mixtures are shown below.

- Compressive strength at 7, 28, and 56 days
- Rapid Chloride Ion Penetration
- Freeze-Thaw Durability

Again, two replicas of each of the tests for each of the mixtures were made.

Table 4.1. Concrete Test Mixture Proportions

Component	Mixture			
	BR-A	BR	DS-A	DS
Cement (lbs)	640	640	557	557
Fly Ash (Class C) (lbs)	160	160	139	139
Water (lbs)	300 (36 gals)	300	271	271 (32.5 gals)
Water-Cementious Matl. Ratio	0.38	0.38	0.39*	0.39
Fine Aggregate (lbs) (Sand)	990	1065	1070	1152
Coarse Aggregate (lbs) (#57 Limestone)	1950	2104	1925	2050
Air Entrainment (oz/cwt) (MB AE 90)	0.45 (5%)	0	0.45 (5%)	0
Retarder (oz/cwt) (MB Pozzolith 100-XR)	2.0	2.0	-	-
Water Reducer (oz/cwt) (MB Polyheed 977)	10.0	10.0	-	-
Delvo (oz/cwt)	-	-	5.17	5.17
Min. 28-Day Strength (psi)	6000	6000	4000	4000

BR-A - ALDOT high performance bridge concrete
BR- ALDOT high performance bridge concrete without A/E
DS-A - ALDOT drilled shaft concrete
DS - ALDOT drilled shaft concrete without A/E
* Should provide slump of 7"-8"

4.5 Other Mixtures Tested

When first attempting to prepare the mixtures shown in Table 4.1, the class "C" fly ash was mislabeled in the laboratory and Type I portland cement was used in its place. Thus, all four initial mixtures had no fly ash, but rather the sum of the cement and fly ash pounds shown in Table 4.1 of cement. This error was caught early, and specimens of the correct mixture proportions were prepared. However, since we had already prepared the all cement specimens and tested their fresh properties, these properties are reported in a later chapter. Also, we decided to retain the erroneous mixture freeze-thaw bars and test them to better assess the effect of fly ash on the mixture freeze-thaw properties.

5. LABORATORY EQUIPMENT, SPECIMENS AND PROCEDURES

5.1 General

All testing conducted in this investigation was done in the laboratory under ambient conditions, and was done in accordance with the appropriate ASTM standard. Two replica specimens for each test were prepared and tested for each set of parameters to help assure accuracy and credibility of the data. As indicated earlier, the concrete mixtures tested were ALDOT standard high performance bridge and drill shaft concretes with and without air-entrainment and are identified in this work as follows:

BR-A: ALDOT standard high performance bridge concrete with 5% entrained air

BR : ALDOT standard high performance bridge concrete without air entrainment

DS-A: ALDOT standard drill shaft concrete with 5% entrained air

DS : ALDOT standard drill shaft concrete without air entrainment

Mixtures where cement was used inadvertently as a substitute for fly ash (see Chapter 4) are designated as,

BR-A/AC

BR/AC

DS-A/AC

DS/AC

Concrete mixture proportions are given in Chapter 4. A brief discussion of the raw materials used in the concrete mixtures is given below along with discussions of the test specimens, curing, and testing procedures and equipment.

5.2 Raw Materials Used

The concrete mixtures were formulated using Type I-II Portland cement, class C fly ash, air entrainment, regular and mid-range water reducers, hydration control admixture and coarse and fine aggregates. A brief discussion of each ingredient follows:

- Type I-II Portland Cement - The Type I-II cement was Lehigh Portland cement manufactured by Lehigh Portland Cement Company to meet the requirements of ASTM C150. This cement was obtained from Lowe's Home Improvement in Opelika, Alabama.
- Class C Fly Ash - The fly ash used in this research was Class C fly ash. It was manufactured by Holnam in Quinton, Alabama at Plant Miller. The fly ash was in accordance with ASTM C 618. Fly ash was added to the mixtures to improve workability.
- Air Entraining Agent - The air entraining agent used in this research was MB AE-90 which is manufactured by Master Builders Technologies and is in accordance with ASTM C260. MB AE-90 was applied to two of the mixtures in order to provide entrained air in the concrete mixture in an attempt to reach the mixture design's specification for air content.
- Water Reducing Agent - The water reducing/retarding agent used in this research was Pozzolith 100XR manufactured by Master Builders Technologies and meets the requirements of ASTM C494, Type B and D. Pozzolith 100XR was applied to the mixtures in an attempt to achieve the

mixture design specifications of slump with the designated water-cement ratio and also to provide improved workability to the mixtures.

- Mid Range Water Reducing Agent - The mid range water reducer used in this research was Polyheed 997 manufactured by Master Builders Technologies and meets the requirements of ASTM C494, Type A and F. Polyheed 997 was applied to the mixtures in an attempt to achieve the mixture design specifications of slump with the designated water-cement ratio and also to provide improved workability to the mixtures.
- Hydration Control Admixture - The hydration control admixture used in this research was Delvo Stabilizer manufactured by Master Builders Technologies in accordance with ASTM C494, Type B and D. Delvo is normally added to drilled shaft concrete in order to retard the setup time of the concrete. Delvo helps to remedy the problem of drilled shaft concrete setting in layers between truck pours.
- Aggregates - The coarse and fine aggregates used in this research were #67 limestone and river sand respectively. The aggregate was from Shorter, Alabama at the Pinkston Pit and was obtained from the Blue Circle Cement plant in Auburn, Alabama.

5.3 Concrete Mixing Procedure

The concrete mixing procedure used in this research was performed as per ACI 223 Method B. The procedure was as follows:

1. Add the batch ingredients to the mixer. (All dry ingredients were added half at a time and then the remaining halves were added to ensure good mixture. The mixer drum was then turned 3 times around to mix the dry ingredients well. The water and the add mixtures were then added.)
2. Start the mixer and mix for 3 minutes.
3. Stop the mixer and rest for 3 minutes.
4. Start the mixer and mix for an additional 2 minutes.
5. Stop the mixer and run tests to determine the fresh concrete properties.

5.4 Specimen Curing

The hardened concrete properties were determined after specimens had experienced lab curing using the following procedure. After 24 hours the cylinder molds for Strength and Rapid Chloride Ion Penetration tests were stripped from their molds and placed in the wet curing chamber (for moist curing) until time for their use. The rectangular bar specimens for the Freeze-Thaw Durability test were also removed from their molds after 24 hours, but were then placed in a lime bath for 14 days.

5.5 Standard Testing Conducted

After the concrete was mixed, tests were conducted to assess both the fresh and hardened concrete properties. Since the tests for fresh concrete are common, they will not be discussed in detail here. Some of the hardened tests such as Freeze-Thaw test and Rapid Chloride Ion Penetration test are not as common and will be discussed in detail later in the chapter (Section 5.7).

Fresh Concrete Properties. Testing conducted on all of the test mixtures to assess their fresh concrete properties was as follows:

- Air Content (ASTM C231) - Conducted on every mixture to ensure that the mixture met the requirements for air as specified by ALDOT.
- Slump (ASTM C143) - Conducted on every mixture to ensure that the mixture met the requirements for slump as specified by ALDOT.
- Unit Weight (ASTM C138) - Conducted on every mixture to meet requirements of ALDOT.
- Concrete Temperature (ASM C1064) - Conducted on every mixture by placing a thermometer into the fresh concrete and reading the temperature after temperature reading stabilized.

All mixture ingredients were carefully measured and allowed to sit at room temperature for at least 24 hours before mixing to simulate actual mixing conditions that would occur at a batch plant.

Hardened Concrete Properties. Testing conducted on all of the test mixtures to assess their hardened concrete properties was as follows:

- Compressive Strength (ASTM C39) - Conducted on every mixture at 7, 28, and 56-days to ensure that the mixture met the strength requirements of ALDOT.
- Freeze-Thaw (ASTM C666) - Conducted on all mixtures to determine the durability of the concrete as required by ALDOT.
- Rapid Chloride Ion Penetration/Permeability (ASTM C1202) - Conducted

on all specimens to determine permeability of concrete as required by ALDOT.

5.6 Freeze-Thaw Durability Testing

The Freeze-Thaw durability test was conducted in accordance with ASTM C666, Procedure A. The test specimens (2 for each mixture) were molded in 3 x 4 x 16 in. steel molds and were initially cured by following the procedure set forth in ASTM C31. After hardening for one day, the samples were removed from their mold and placed in a lime bath for 14 days. After 14 days, a sonometer was used to measure each specimen's initial fundamental transverse frequency. The specimen's were then placed in a freeze-thaw cabinet to undergo rapid freeze-thaw cycles. Figure 5.1 shows a photograph of the sonometer and freeze-thaw cabinet. The testing was in accordance with ASTM C666, and in this test the samples are set in 1/32 in. to 1/8 in. of water throughout the freeze-thaw cycle. The samples went through a freeze-thaw cycle approximately every 3 to 4 hours. A single freeze-thaw cycle consists of lowering the temperature in the cabinet from 40° F to 0° F, and then from 0° F back to 40° F. Samples were tested with the sonometer every 30 cycles in the thawed state for the fundamental transverse frequency. The test was conducted for 300 freeze-thaw cycles. After the 300th cycle, the samples were removed and the final transverse frequency was measured. The samples resistance to freeze-thaw is given as a Durability Factor (DF) by the following equation.

$$DF = (n_1^2) / (n^2) * 100$$

where

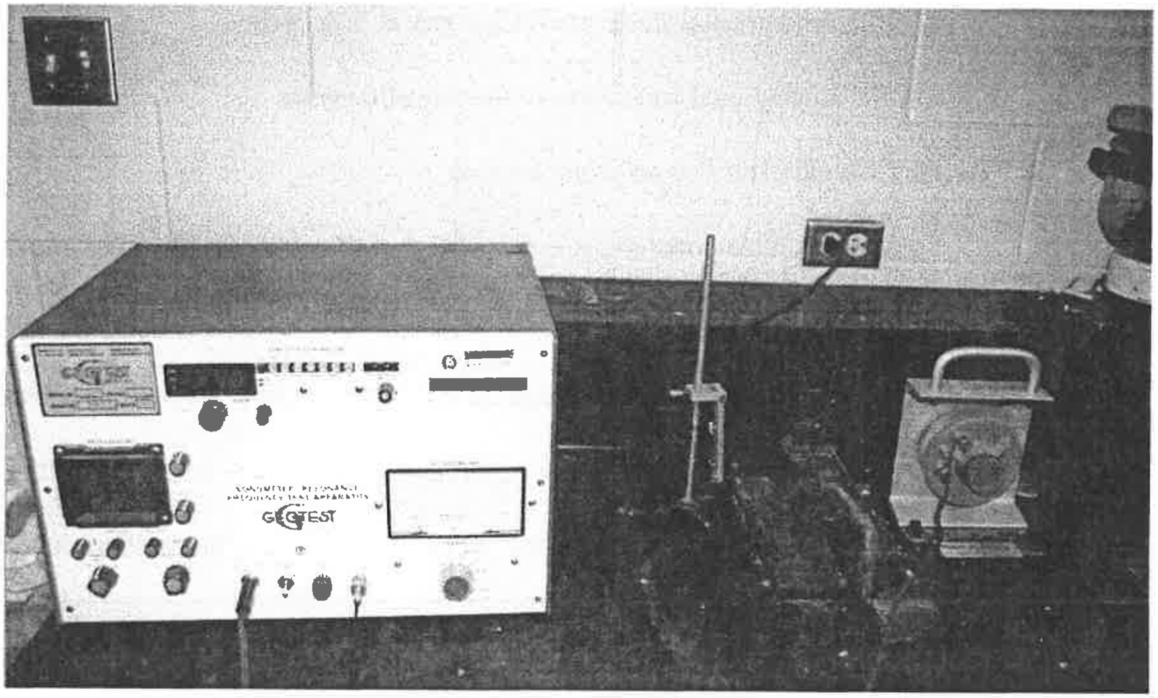


Fig. 5.1. Freeze-Thaw Sonometer and Cabinet

n_1 = the fundamental transverse frequency at 300 cycles

n = the fundamental transverse frequency at 0 cycles

5.7 Rapid Chloride Ion Penetration Testing

To attain durable concrete, it is essential that the concrete be rather impermeable to water and chloride ion penetration. The rapid chloride ion penetration test is a relatively quick and east method to assess the permeability and penetration resistance of concrete. Thus, the permeability of the concrete samples was measured according to ASTM C1202. Photographs of the instrumentation and equipment used in the rapid chloride ion penetration test are shown in Fig. 5.2.

The test consists of monitoring the amount of electrical current or charges (in coulombs) that pass through a 2 in. thick disk cut from a 4 x 8 in. concrete cylinder. The monitoring lasts for a 6 hour period. A 60-volt current is maintained across the ends of the sample while they are immersed in solutions. On one end, the sample is placed in a sodium chloride solution, and the other is placed in a sodium hydroxide solution. The total charge passed in the 6 hour period has been found to be related to the ability of the sample to resist chloride ion penetration. The test is conducted after the sample has been moisture cured for 56 days. The permeability of the samples is then classified based on the charge passed through it as follows:

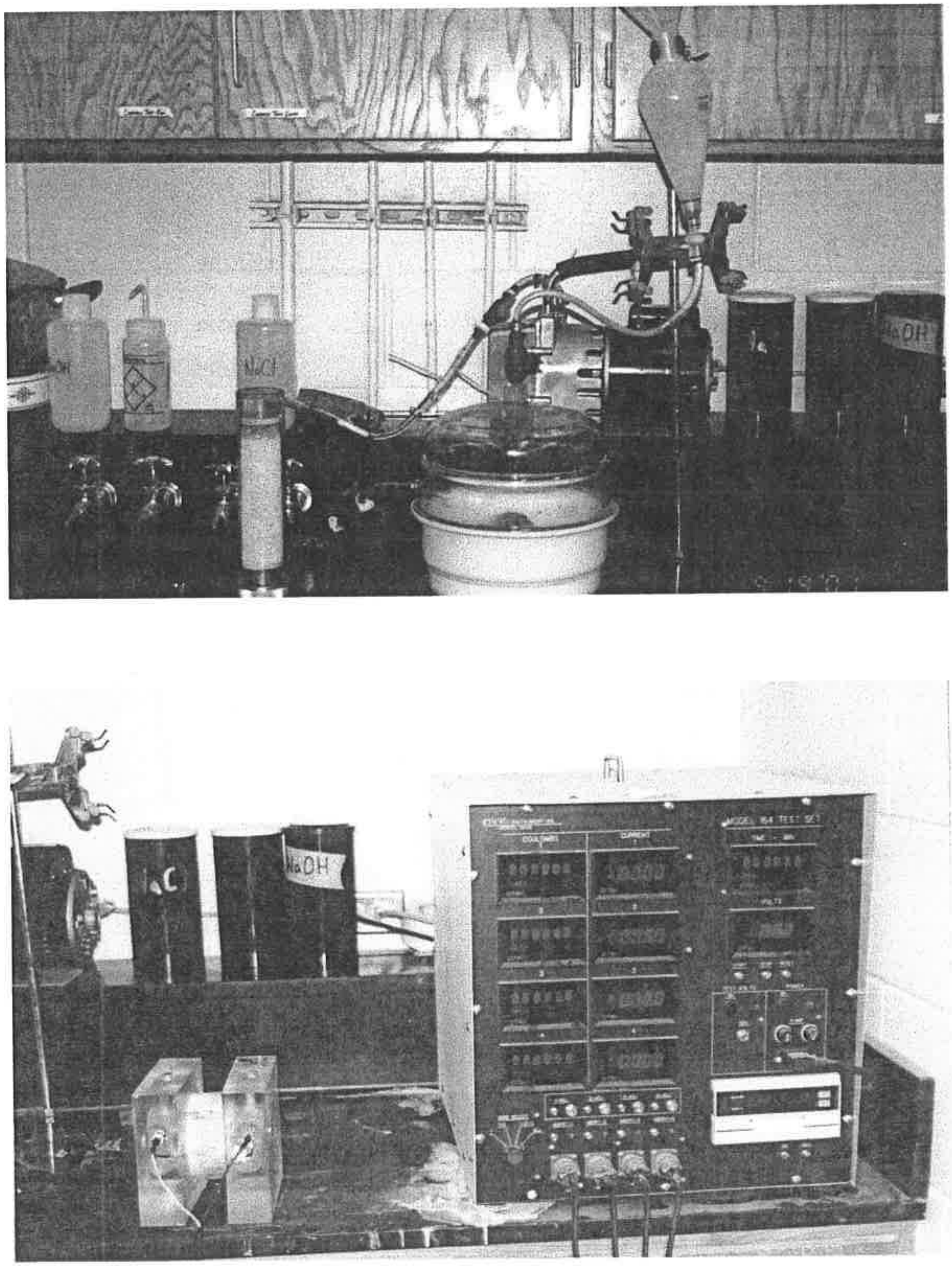


Fig. 5.2. Rapid Chloride Ion Penetration Test Instrumentation

Charge Passed (Coulombs)	Chloride Ion Permeability Classification
> 4000	High
2000 – 4000	Moderate
1000 – 2000	Low
100 – 1000	Very Low
<100	Negligible

6. PRESENTATION OF LABORATORY RESULTS

6.1 General

As mentioned in Chapters 4 and 5, laboratory testing was conducted on the following four concrete mixture designs:

- BR-A: ALDOT high performance bridge concrete
- BR: ALDOT high performance bridge concrete without A/E
- DS-A: ALDOT drilled shaft concrete
- DS: ALDOT drilled shaft concrete without A/E

The testing was a rather comprehensive investigation of each mixture's fresh and hardened properties in hopes of identifying whether high performance bridge and drilled shaft concrete mix designs without A/E would perform as well in both fresh and hardened properties (especially freeze/thaw) as did the ALDOT standard mixtures.

In presenting the results, the fresh properties are discussed first, followed by the hardened concrete properties. Fresh property results from the special all-concrete mixes (designated as /AC) are included in the corresponding sections. The only hardened concrete properties of the AC mixes tested were the freeze-thaw preparations. Those results are included in the figures for the freeze-thaw testing.

6.2 Fresh Concrete Property Results

The fresh properties of the concrete mixtures were tested immediately with the following properties/parameters being tested: concrete temperature, unit weight, air content, and slump. Results from each of these are presented and discussed below. The results shown are the average of two replica tests for each concrete mixture.

Concrete Temperature: The concrete temperature was observed at the completion of the mixing of the concrete. The results from the temperature test are shown in Tables 6.1 and 6.2 and Figures 6.1 and 6.2. The concrete temperature ranges for the mixtures were all within 1 degree of each other. The measured ingredients were kept inside the laboratory until time for mixing. This would account for the mixture temperatures being slightly lower (approximately 2° F) than ambient temperatures. Bar charts of the concrete temperature results are shown in Figures 6.1 and 6.2.

Unit Weight: The unit weights of the mixtures were measured upon completion of the mixing, and the results are shown in Tables 6.1 and 6.2. The unit weights of the concrete mixtures were all approximately the same with the exception of the BR-A mixture, which had a lowest weight of all the mixes. This can probably be best accounted for by the fact that the BR-A mixture had the highest air content and least weight of aggregate (see Table 4.1) of all of the mixes, and it and the BR mixture had the highest water content. The high content of air filled more of the container that would have normally been filled with concrete. A bar chart of unit weights is shown Figure 6.3, for convenience of comparisons.

Air Content: The air content of the mixtures were observed at the completion of the concrete mixing. The results can be seen in Tables 6.1 and 6.2, and Figures 6.4 and 6.5. The air contents all fluctuated within 1% of the targeted air content levels. This is probably due to air meter calibration/accuracy, and/or mixture variations. The air content of the mixtures without A/E (BR and DS mixtures) were both measured to be either 1% or 1.5% air content.

Slump: The slump of the mixtures were measured upon the completion of the mixes. The slumps were within 0.25 inches of each other, with 3 of the 4 mixtures having 8 inch slumps. The mixtures were all designed to have high slumps so that the workability of the mixes would be good. The results can be seen in Tables 6.1and 6.2 and Figures 6.6 and 6.7.

6.3 Hardened Concrete Property Results

The following hardened properties of the concrete mixtures were evaluated: compressive strength, rapid chloride ion penetration, and freeze-thaw durability. Results for each of these are presented below and are the average of two replica tests for each of the concrete mixtures.

Compressive Strength: The compressive strength of each of the concrete mixtures was measured at 7, 28, and 56-days. Results of the compressive strength tests are shown in Table 6.3 and Figures 6.8 – 6.11. Figures 6.9 – 6.11 show compressive strength of each of the samples (at 7, 28, and 56-days) on separate bar charts for convenience of comparison. The compressive strength data shown in the table and figures were calculated by averaging the compressive strength of the two individual cylinders that were broken at each curing time.

The BR and DS mixtures were much stronger than their counterparts. This can be accounted for because there was no A/E added to the mixtures. The entrained air caused air voids that resulted in a loss of strength. The BR-A mixture did not meet its 28-day strength requirements, but did come very close. This was attributed to the fact that the concrete was 1% overaired. The BR-A mix had the highest A/E level, and the lowest strength of all the mixes. Obviously A/E does reduce concrete strength as expected.

Rapid Chloride Ion Penetration: The rapid chloride ion test was conducted on each of the mixes after wet curing for 56 days. This process is described in ASTM 1202. Results of the rapid chloride ion penetration test are shown in Table 6.4. These results showed that all of the mixtures fell into the "moderate" category of permeability. This was surprising since high performance concrete is supposed to have a low permeability. The results also showed that A/E did not have a large effect on permeability. In fact, the air-entrained mixtures gave slightly lower charges passed, or lower permeability, relative to their non air-entrained counterparts. Side-by-side comparisons of the rapid chloride ion penetration results of each of the mixtures are shown in Figures 6.12 and 6.13.

Freeze-Thaw Durability: The freeze-thaw durability results were observed approximately every 30 cycles for 300 cycles. Results for the freeze-thaw durability are shown in Tables 6.5 – 6.8 and in Figures 6.14 – 6.19. As discussed in Chapter 4, there were four AC mixes poured due to inadvertent mislabeling of ingredients in the lab. These mixes contained no fly ash but were tested against their corresponding mixes with fly ash in the freeze-thaw cabinet. In this way we were able to assess the effects of fly ash, as well as A/E on the mixtures freeze-thaw properties.

The results indicated that the mixtures with and without fly ash preformed about the same in durability, except that the BR mixture with fly ash preformed much better than its counterpart without fly ash. Neither of the DS mixtures completed the test. They broke in half at approximately 210-240 cycles. Therefore on the bar graphs of Figures 6.17 and 6.18, they are shown as having zero durability factor. The BR mixtures did very well and actually had a higher durability factor than its counterpart with A/E. Although there was almost no scaling in the BR samples or any of the mixtures with A/E, there was a great deal of scaling and cracking in the DS samples. Figure 6.19 shows some photographs of the test specimens after removal from the freeze-thaw chamber after 300 freeze-thaw cycles. Note that in these photos, Figure 6.19a shows the following mixtures (from left to right): BR-A, BR-A/AC, BR, BR/AC. Figure 6.19b shows the following mixtures (from left to right): DS-A, DS-A/AC, DS, DS/AC. In Figure 6.19a the differences in physical appearance of the bars is only slight variations in color. In Figure 6.19b, it is easily discernable that the DS/AC mixture had the worse durability of all the mixes. It was reduced to small rubble, and had to be stacked together in order to photograph it. Although the DS mixture does not look as bad as the DS/AC mixture from the photograph, it is broken in half with a line that can be seen along the exposed limestone rock.

It was observed that over certain intervals for some of the tests that the frequency and durability factor of the concrete actually rose. This was due to the aging of the concrete. As the test period increased, the concrete became stronger and its stiffness became greater, and therefore in some cases, the durability of the sample became greater.

Table 6.1 Fresh Concrete Property Testing Results*

Concrete Property/Parameter	Concrete Mixture			
	BR-A	**BR**	**DS-A**	**DS**
Concrete Temperature ASTM C1064 (° F)	76	76	75	76
Unit Weight ASTM C138 (lbs/cf)	144.0	153.9	153.9	156.5
Air Content ASTM C231 (%)	6	1	4	1
Slump ASTM C143 (inches)	8	8	7 3/4	8

*Results are the average of two replica tests

Table 6.2 Fresh Concrete Property Testing Results for AC Mixes*

Concrete Property/Parameter	Concrete Mixture			
	BR-A/AC	BR/AC	DS-A/AC	DS/AC
Concrete Temperature ASTM C1064 (^{o} F)	82	79	79	80
Unit Weight ASTM C138 (lbs/cf)	N/A	N/A	N/A	N/A
Air Content ASTM C231 (%)	6	1.5	4	1
Slump ASTM C143 (inches)	8	8	7 3/4	7

*Results are the average of two replica tests

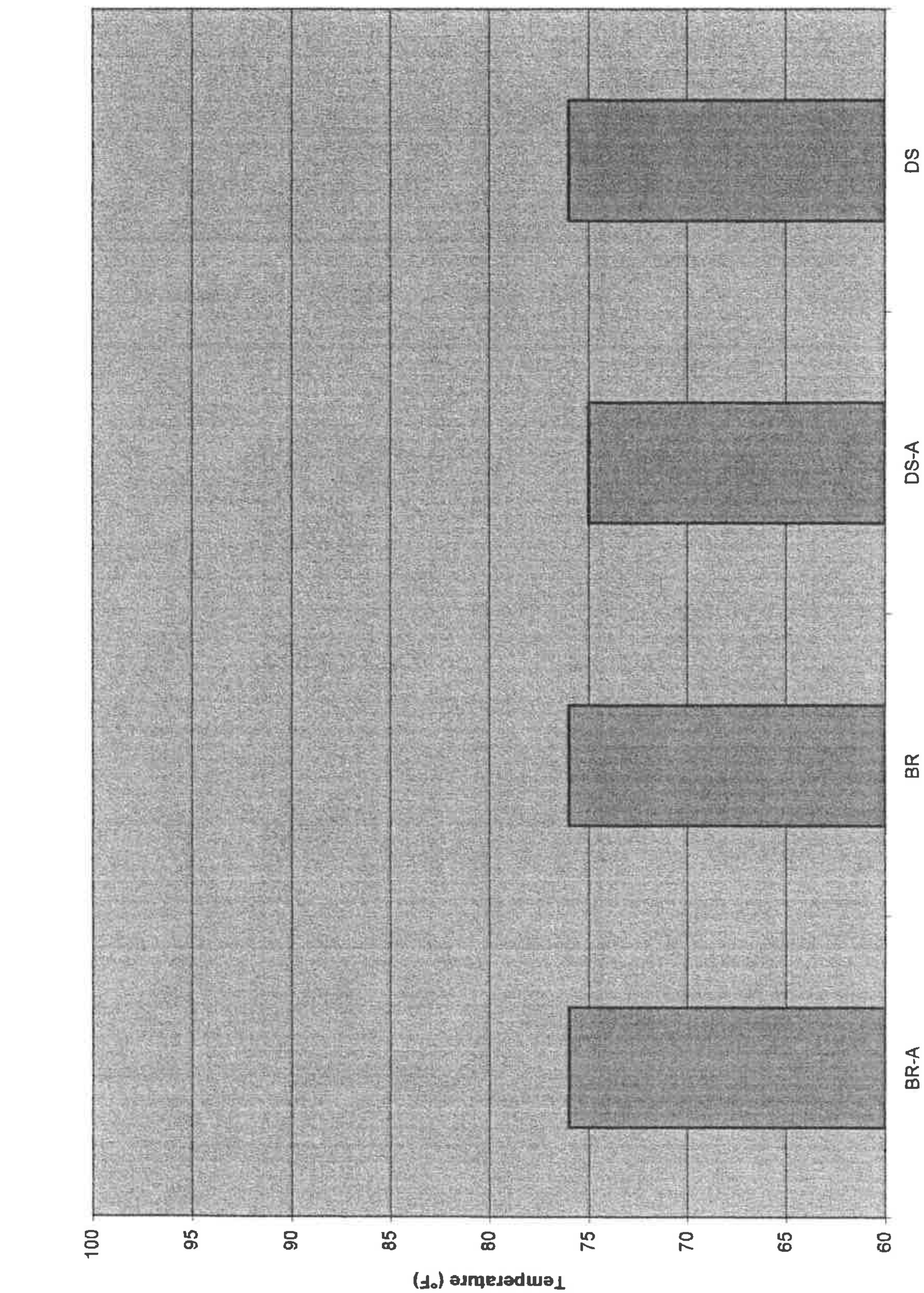


Figure 6.1 Concrete Temperature of Mixtures

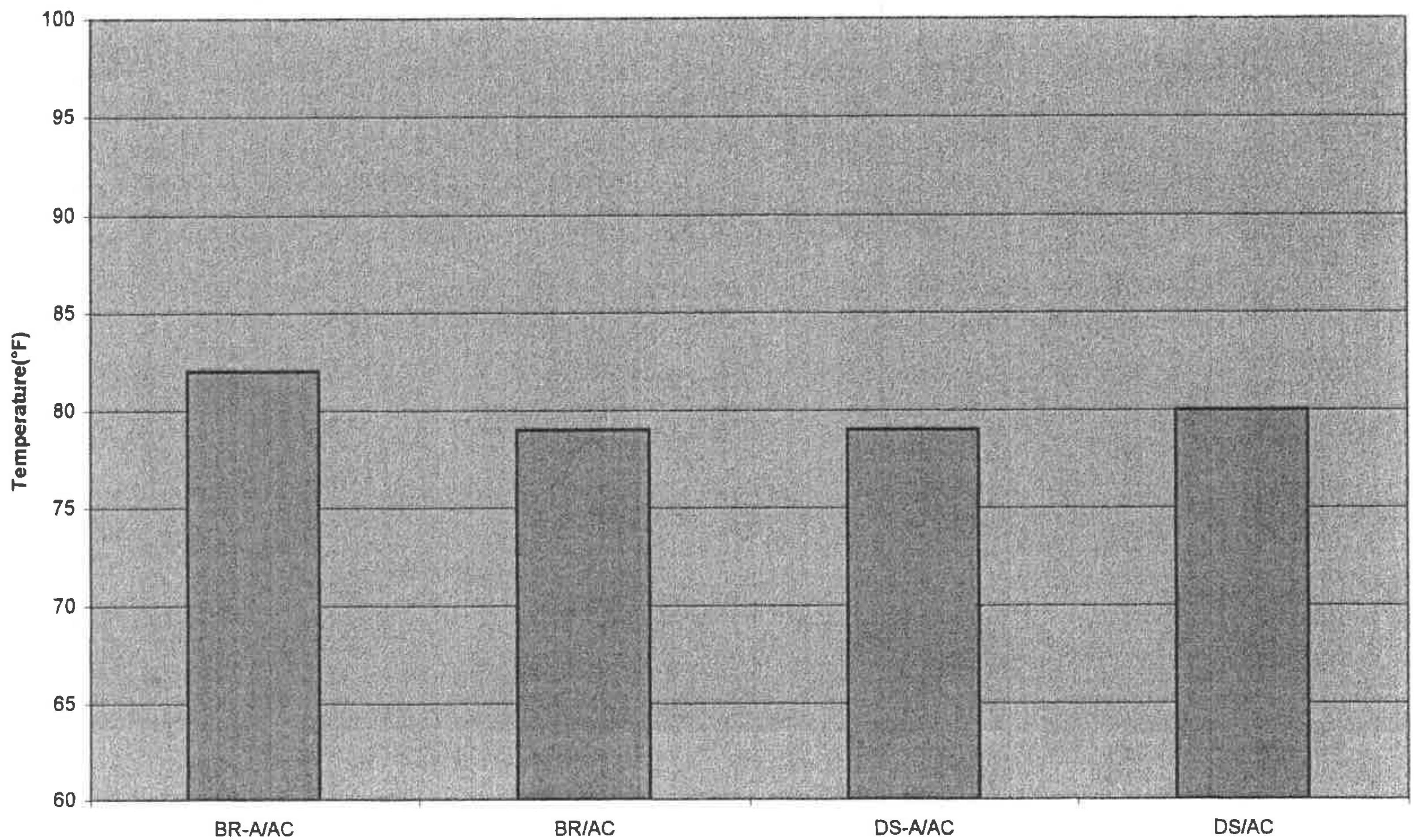


Figure 6.2 Concrete Temperature of AC Mixtures

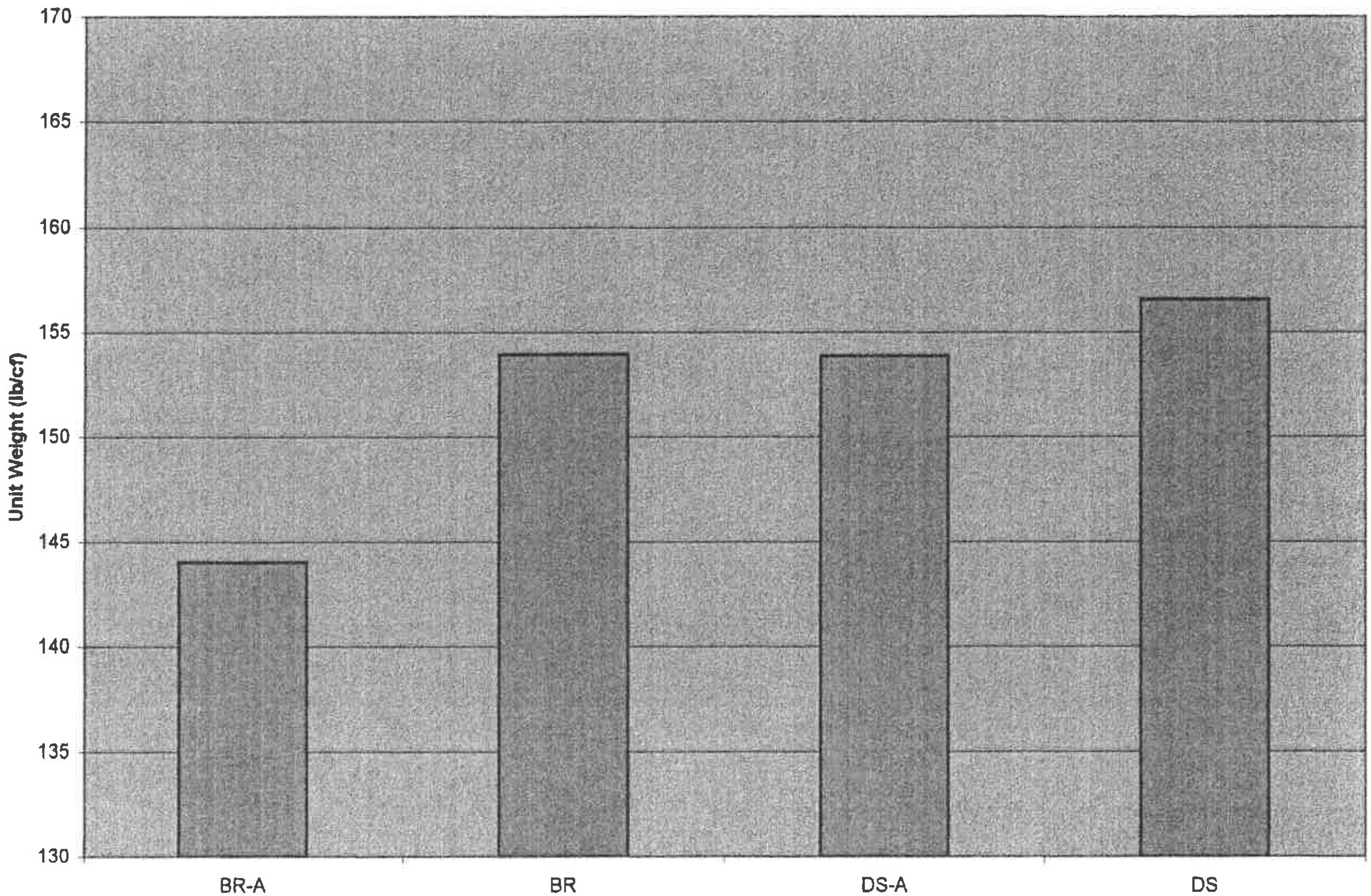


Figure 6.3 Concrete Unit Weights of Mixtures

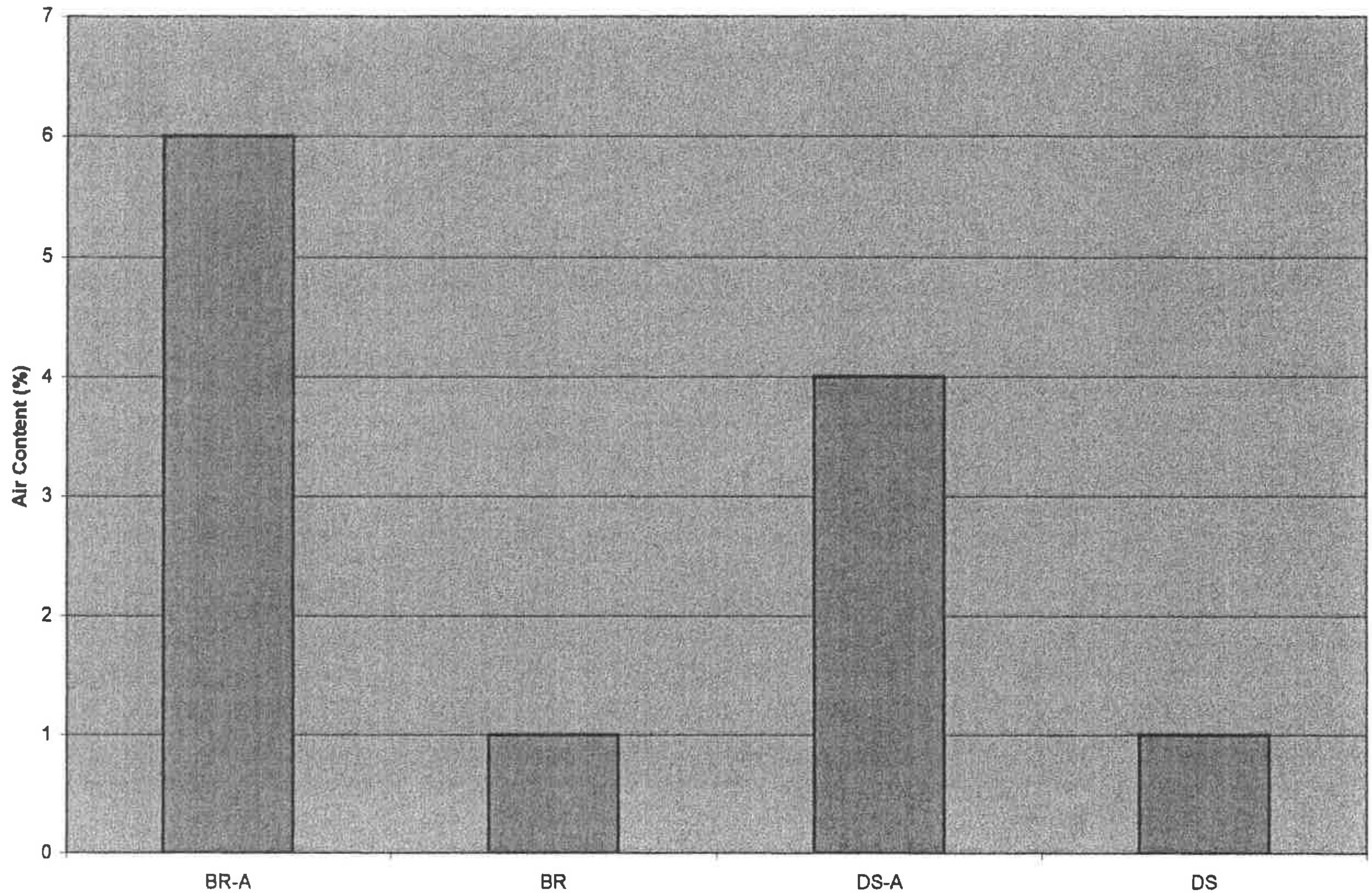


Figure 6.4 Air Content of Concrete Mixtures

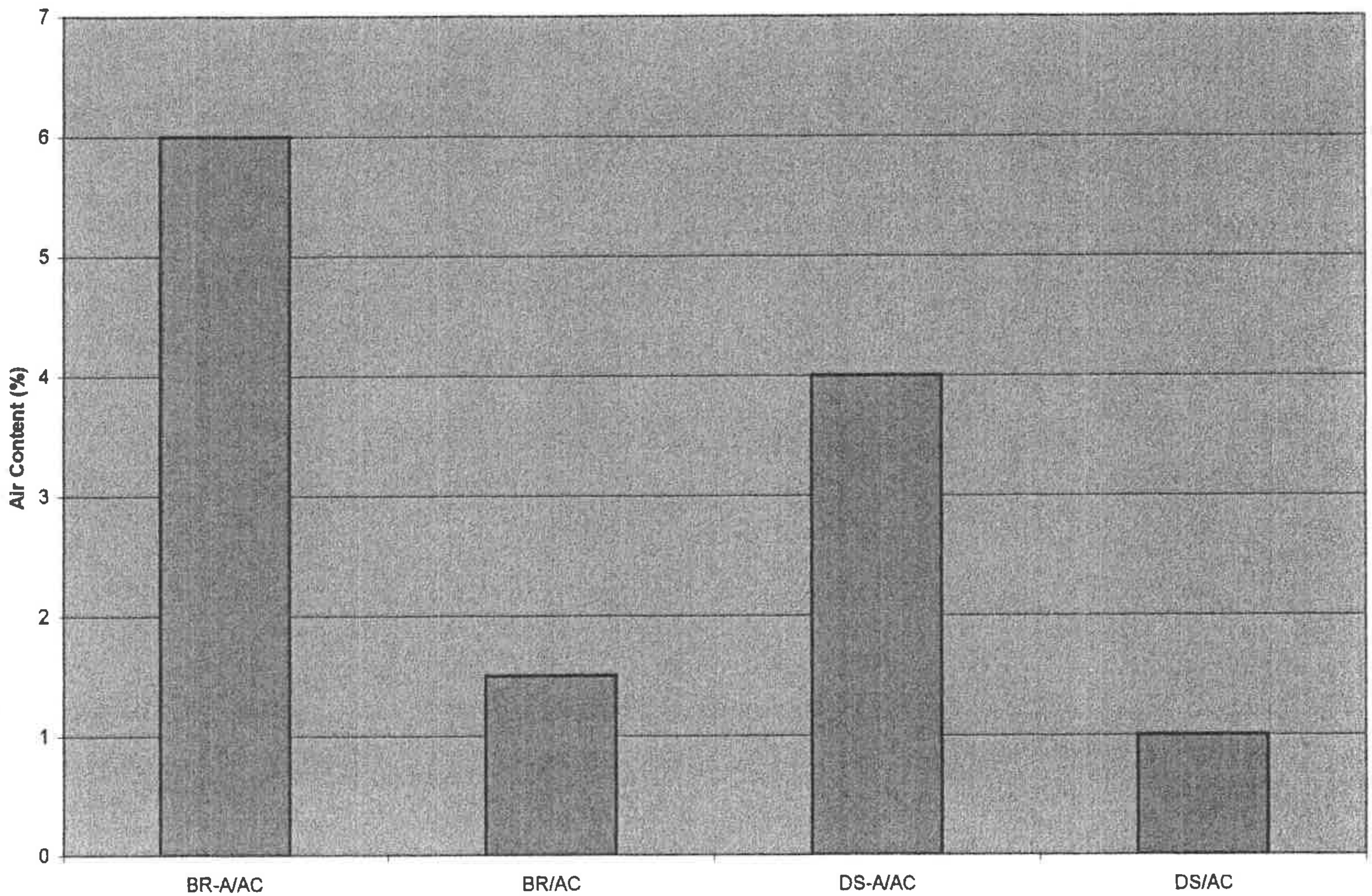


Figure 6.5 Air Content of AC Mixtures

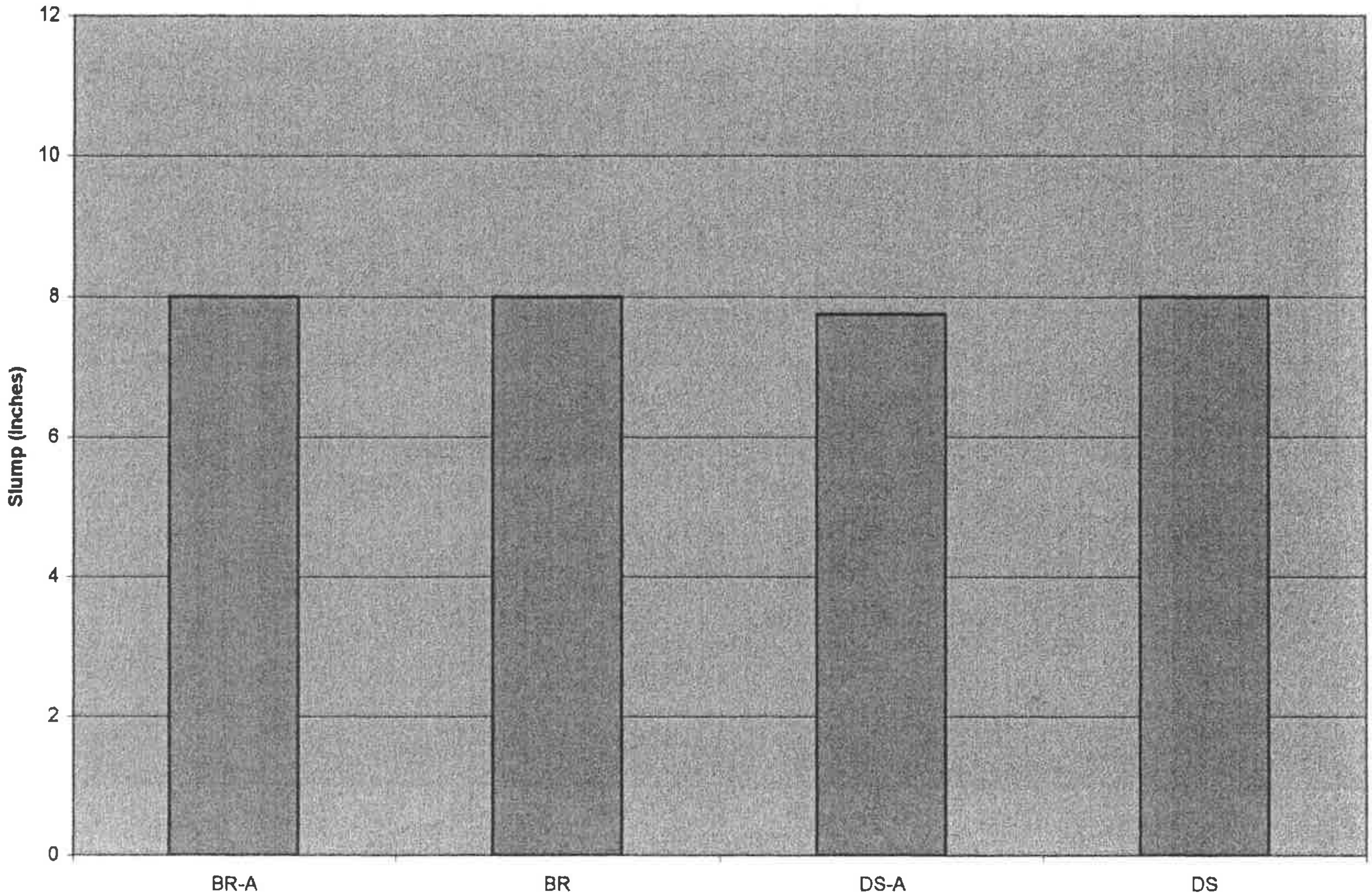


Figure 6.6 Slump of Concrete Mixtures

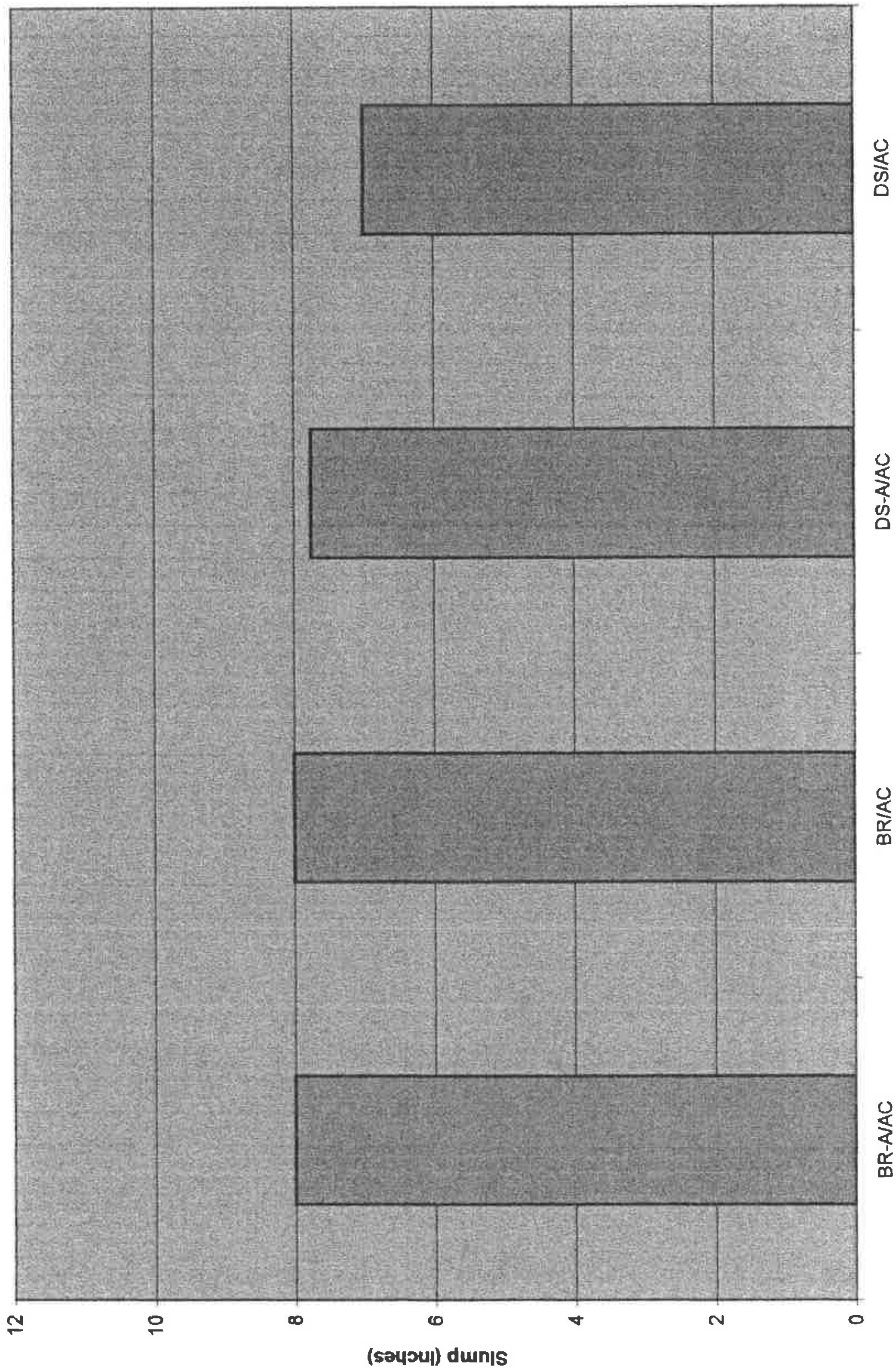


Figure 6.7 Slump of AC Mixtures

Table 6.3 Compressive Strength Results of Concrete Mixtures at 7, 28, 56-Days*

Concrete Property/Parameter		**Concrete Mixture**			
		BR-A	**BR**	**DS-A**	**DS**
Concrete Strength ASTM C39 (psi)	**7-Day**	4297	5769	5376	5669
	28-Day	5819	7858	6485	7997
	56-Day	6167	9469	7719	8673

*Results are the average of two replica tests on 4" x 8" cylinders capped with neoprene

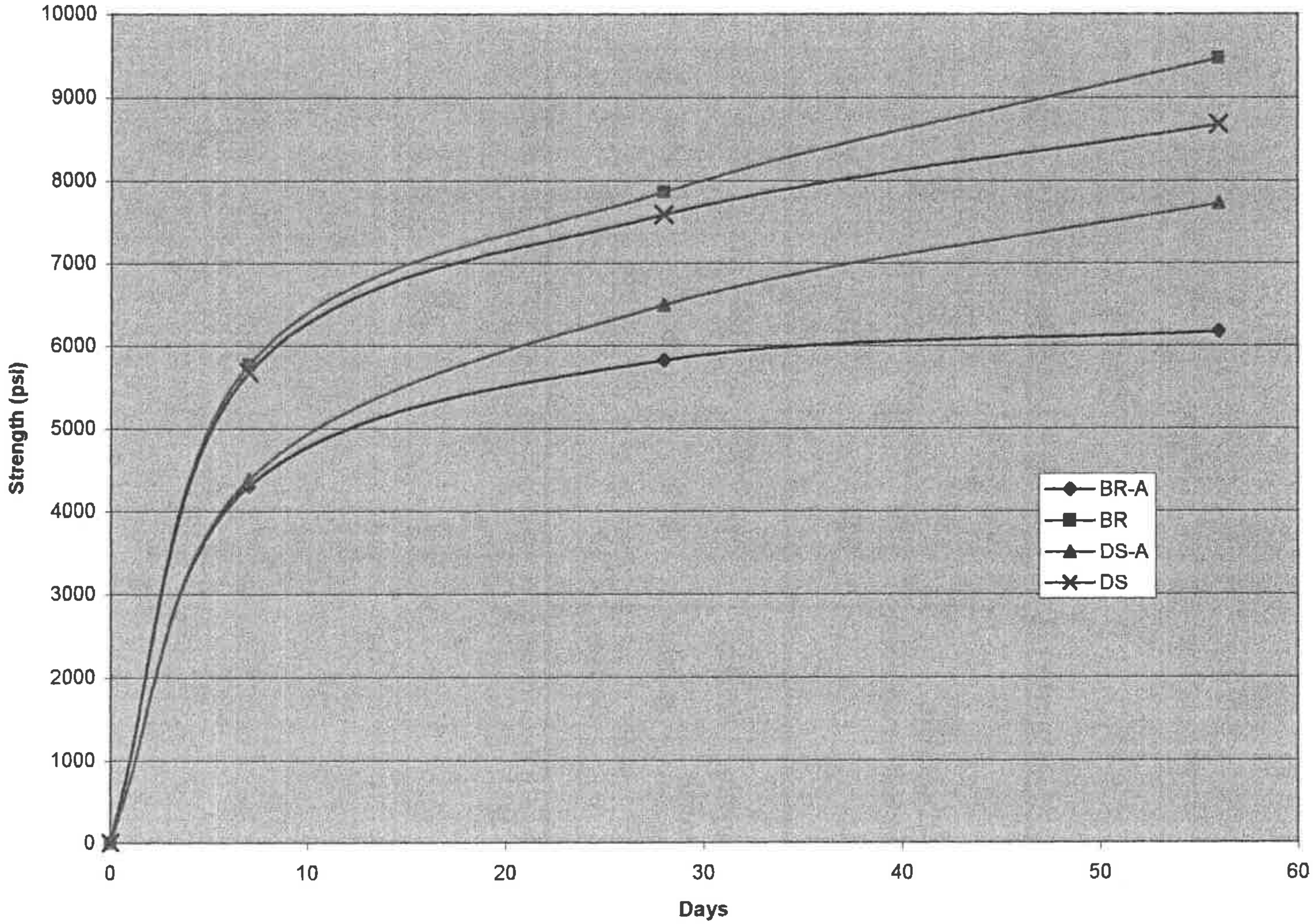


Figure 6.8 Concrete Compressive Strength vs. Age Curves for Mixtures

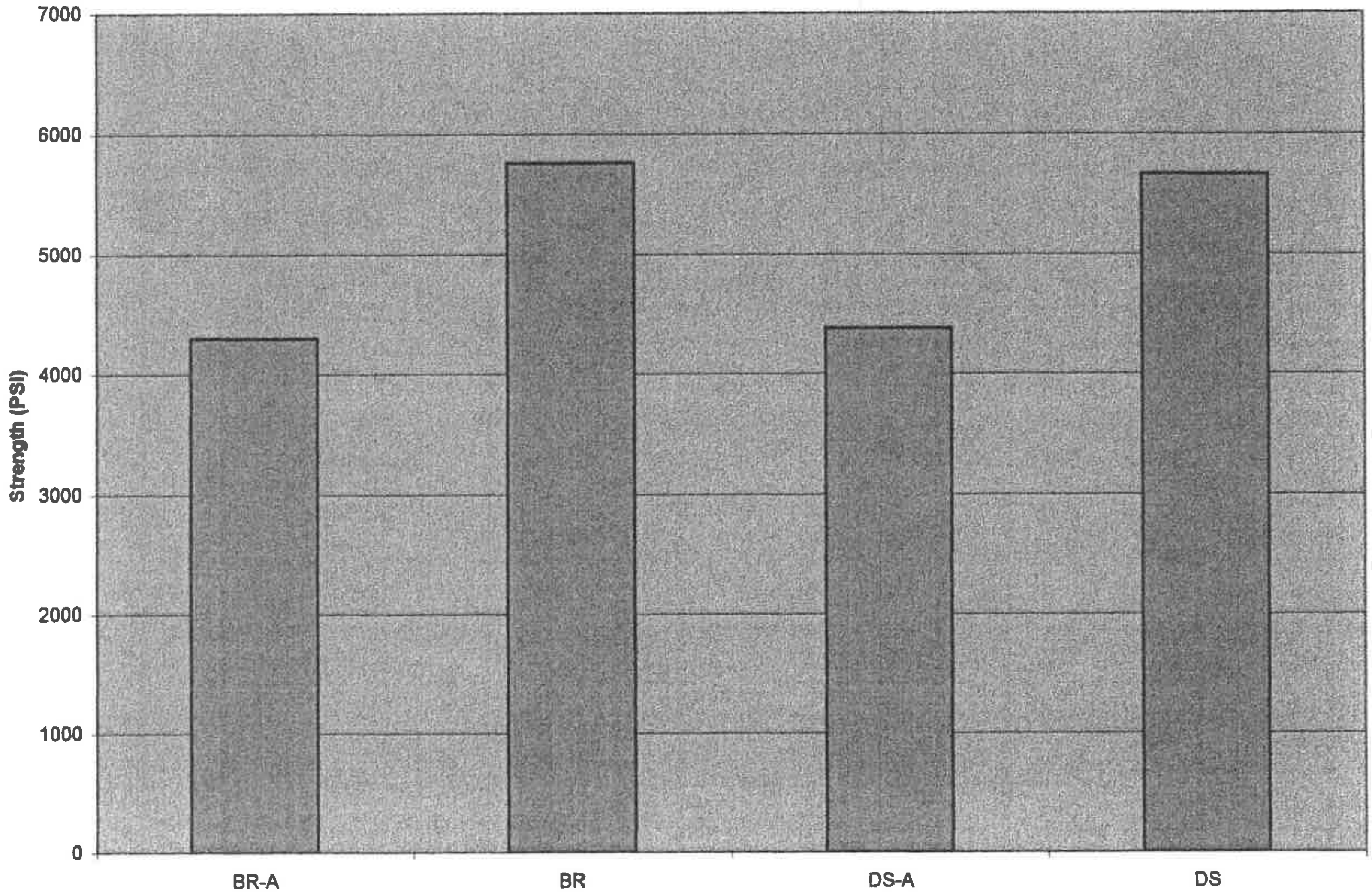


Figure 6.9 Compressive Strength of Concrete Mixtures at 7-Days

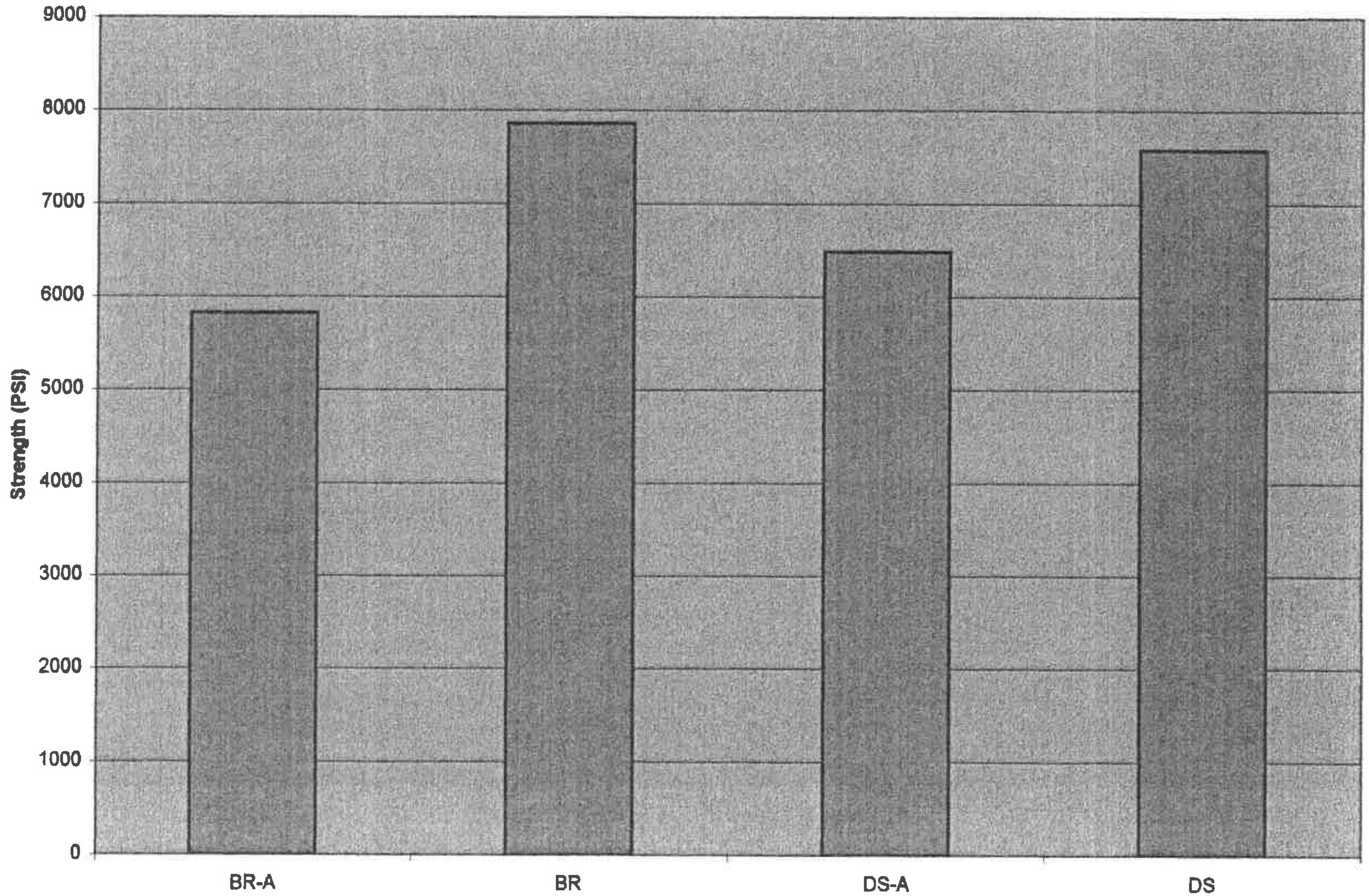


Figure 6.10 Compressive Strength of Concrete Mixtures at 28-Days

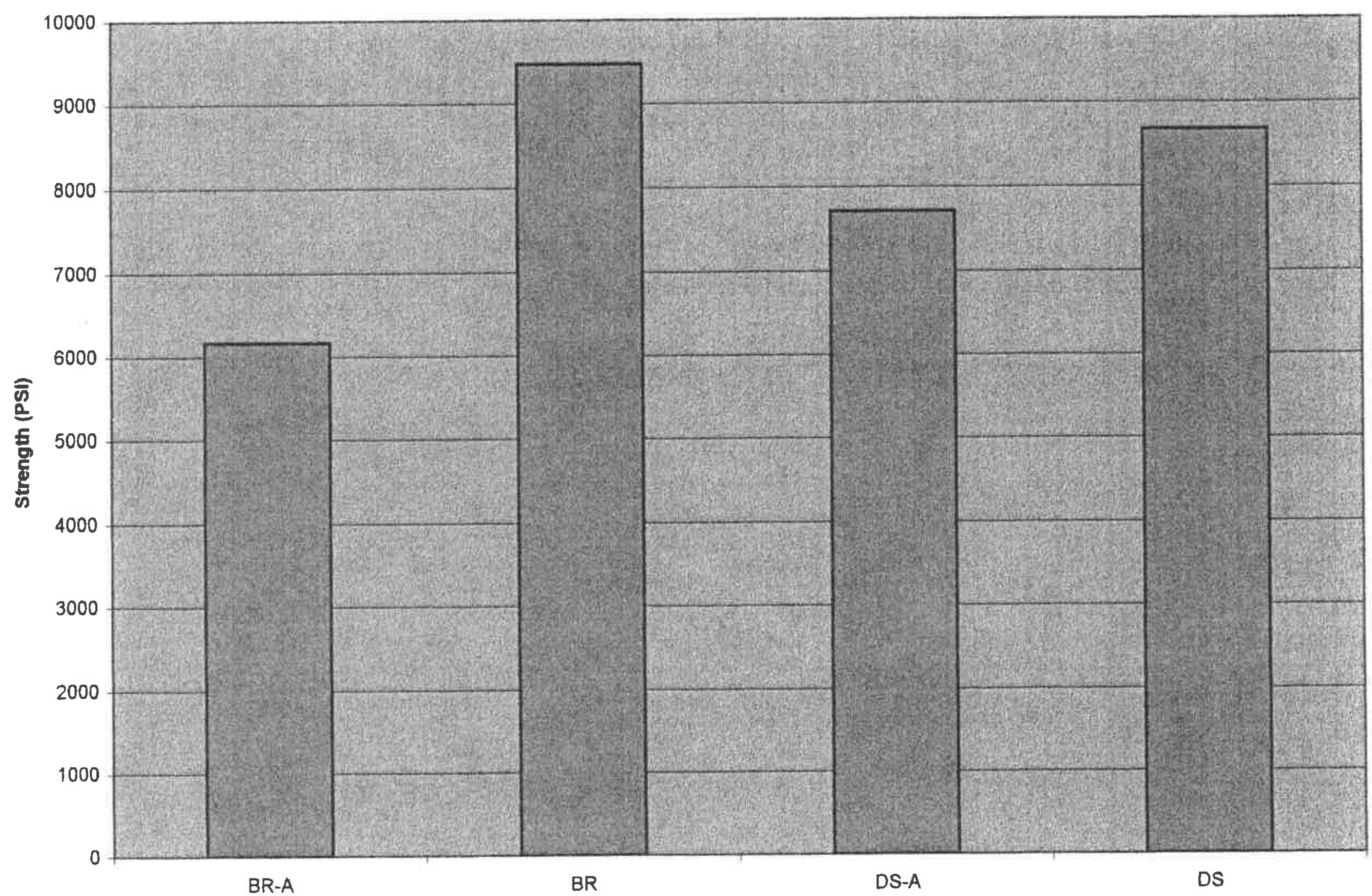


Figure 6.11 Compressive Strength of Concrete Mixtures at 56-Days

Table 6.4 Rapid Chloride Ion Penetration Results for Concrete Mixtures*

Concrete Mixture	Charge Passed (Coulombs)	ASTM C1202 Rating
BR-A	3006	Moderate
BR	3240	Moderate
DS-A	2111	Moderate
DS	2304	Moderate

*Results are the average of two replica tests

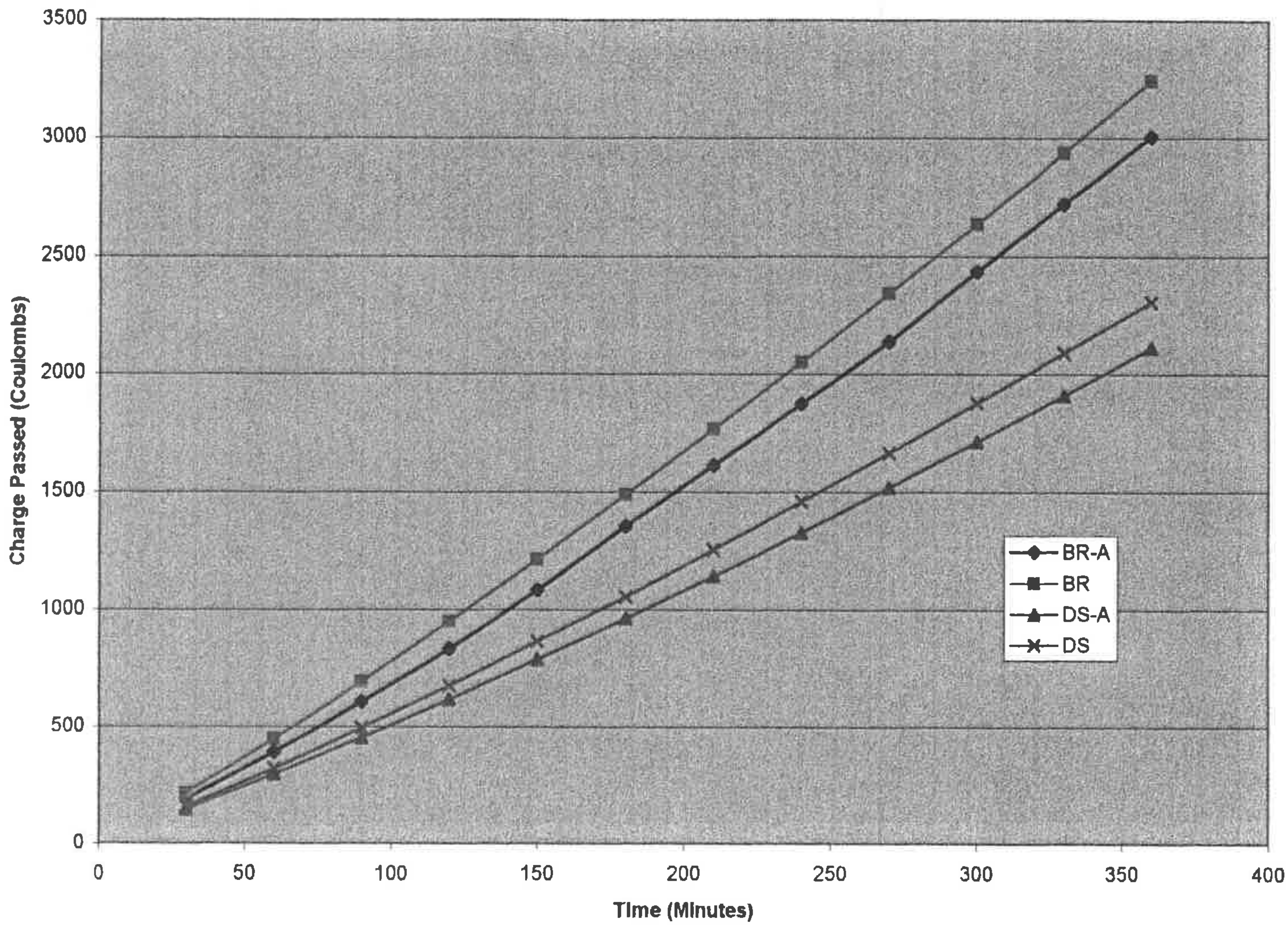


Figure 6.12 Total Charge Passed vs. Time for Concrete Mixtures

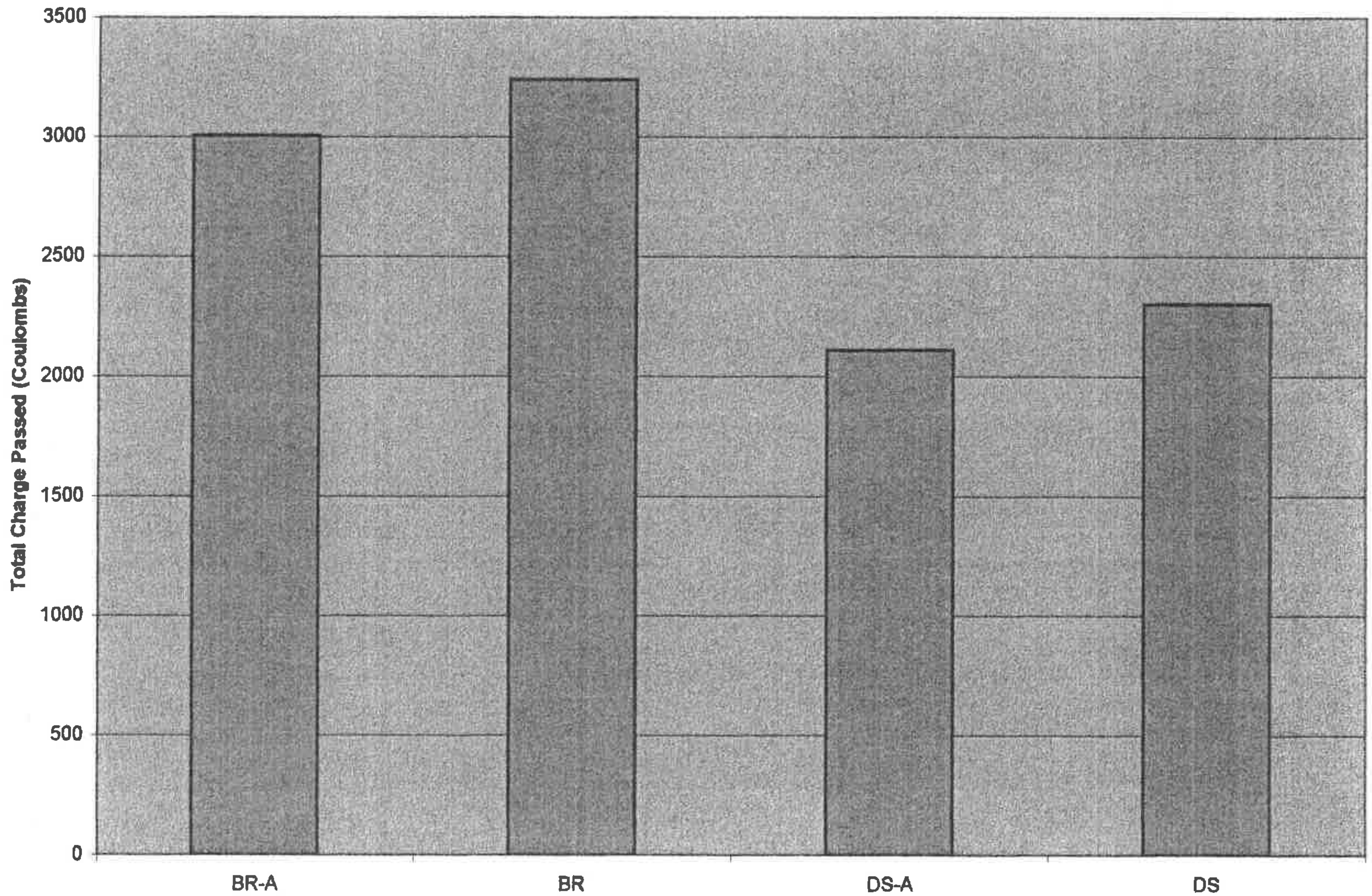


Figure 6.13 Rapid Chloride Ion Penetration Test Results of Concrete Mixtures

Table 6.5 Freeze-Thaw Durability Results for Concrete Mixtures*

Concrete Mixture	Initial Frequency (Hz)	Final Frequency (Hz)	Durability Factor (%)
BR-A	1724	1617	88.0
BR	1819	1750	92.6
DS-A	1806	1760	95.0
DS	1890	0	0

*Results are the average of two replica tests

Table 6.6 Freeze-Thaw Durability Results for AC Mixtures*

Concrete Mixture	Initial Frequency (Hz)	Final Frequency (Hz)	Durability Factor (%)
BR-A/AC	1826	1777	94.7
BR/AC	1916	1591	69.0
DS-A/AC	1954	1881	92.7
DS/AC	1947	0	0

*Results are the average of two replica tests

Table 6.7 Durability Factors for Concrete Mixtures*

Cycle	BRA	BR	DSA	DS
0	100	100	100	100
30	95.9	96.5	97.7	67.3
60	95.2	95.8	97.0	42.0
90	95.8	96.1	97.5	34.4
120	94.4	96.0	97.8	22.6
150	94.4	95.9	97.6	11.2
180	92.7	93.6	97.6	3.1
210	92.7	93.0	97.4	0.6
240	89.2	96.2	95.3	0
270	89.0	93.9	95.3	0
300	88.0	92.6	95.0	0

*Results are the average of two replica tests

Table 6.8 Durability Factors for AC Mixtures*

Cycle	BR-A/AC	BR/AC	DS-A/AC	DS/AC
0	100	100	100	100
30	96.6	96.9	97.1	53.0
60	96.1	95.6	96.5	27.0
90	96.4	96.9	96.45	10.5
120	96.4	93.7	96.2	2.9
150	96.3	92.4	95.9	0.5
180	95.9	85.8	94.5	0
210	96.1	83.0	94.5	0
240	95.2	80.9	93.8	0
270	95.0	74.6	91.9	0
300	94.7	69.0	92.7	0

*Results are the average of two replica tests

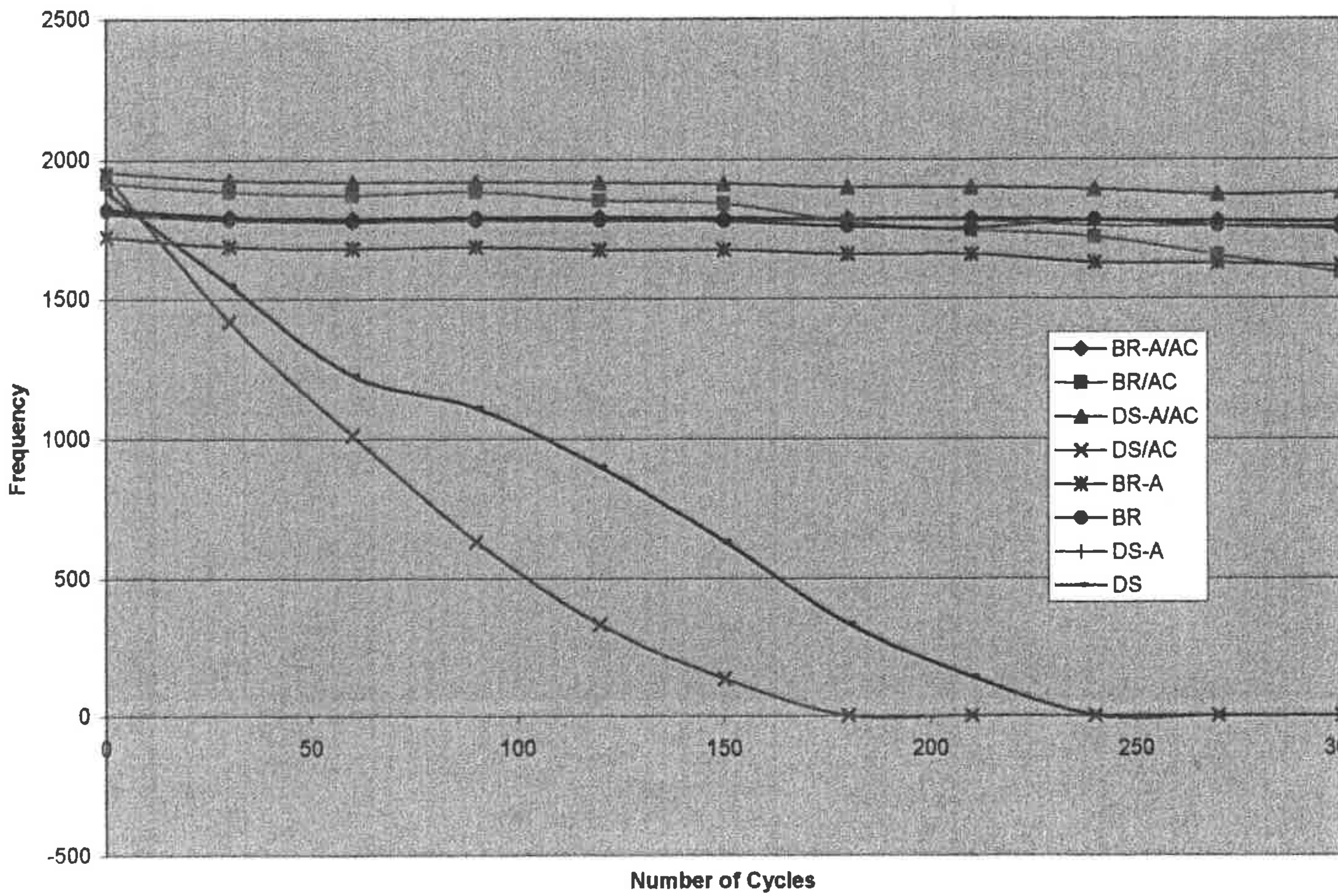


Figure 6.14 Cycle Number vs. Natural Frequency of All Concrete Mixtures

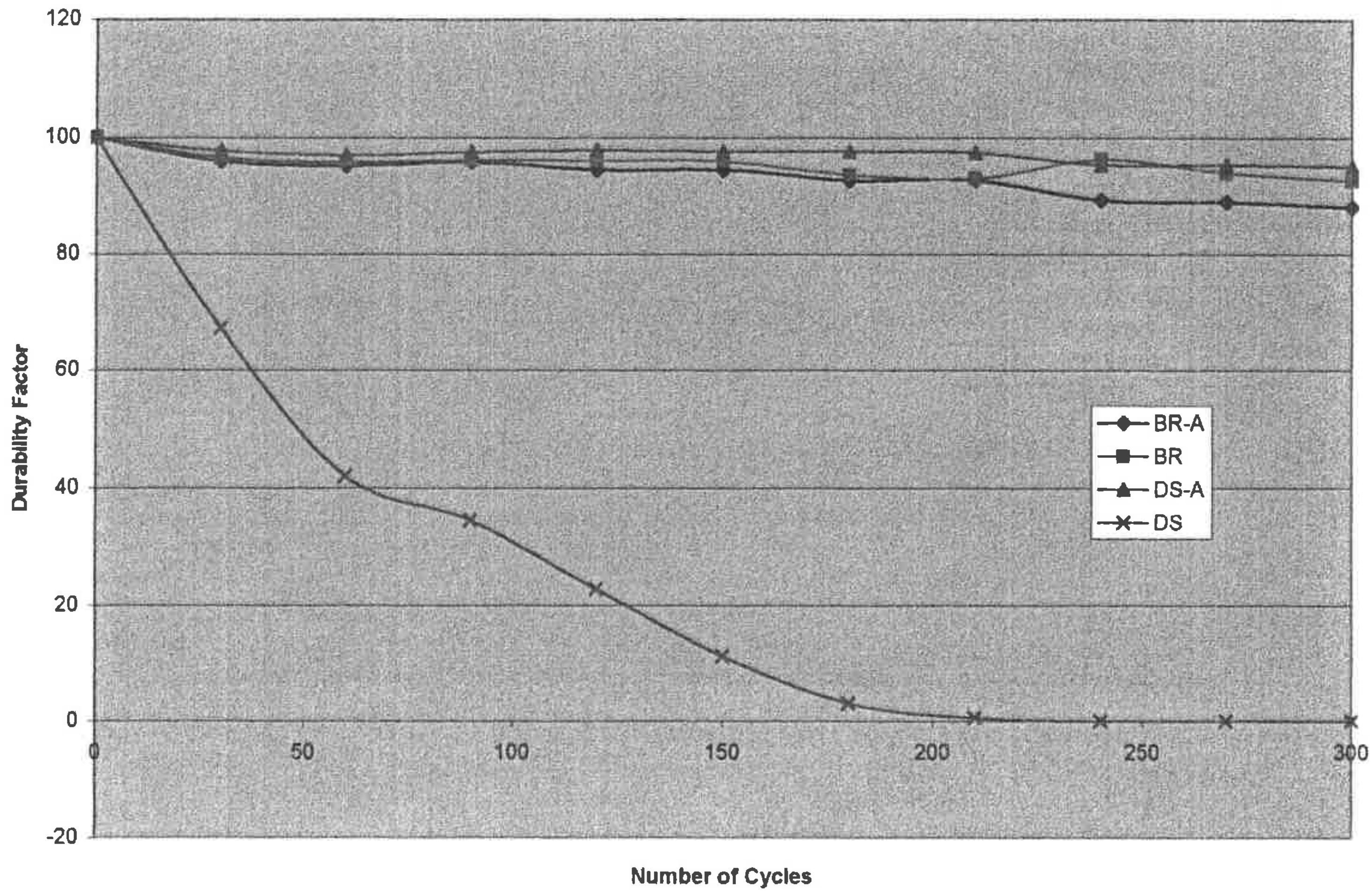


Figure 6.15 Cycle Number vs. Durability Factor of Concrete Mixtures

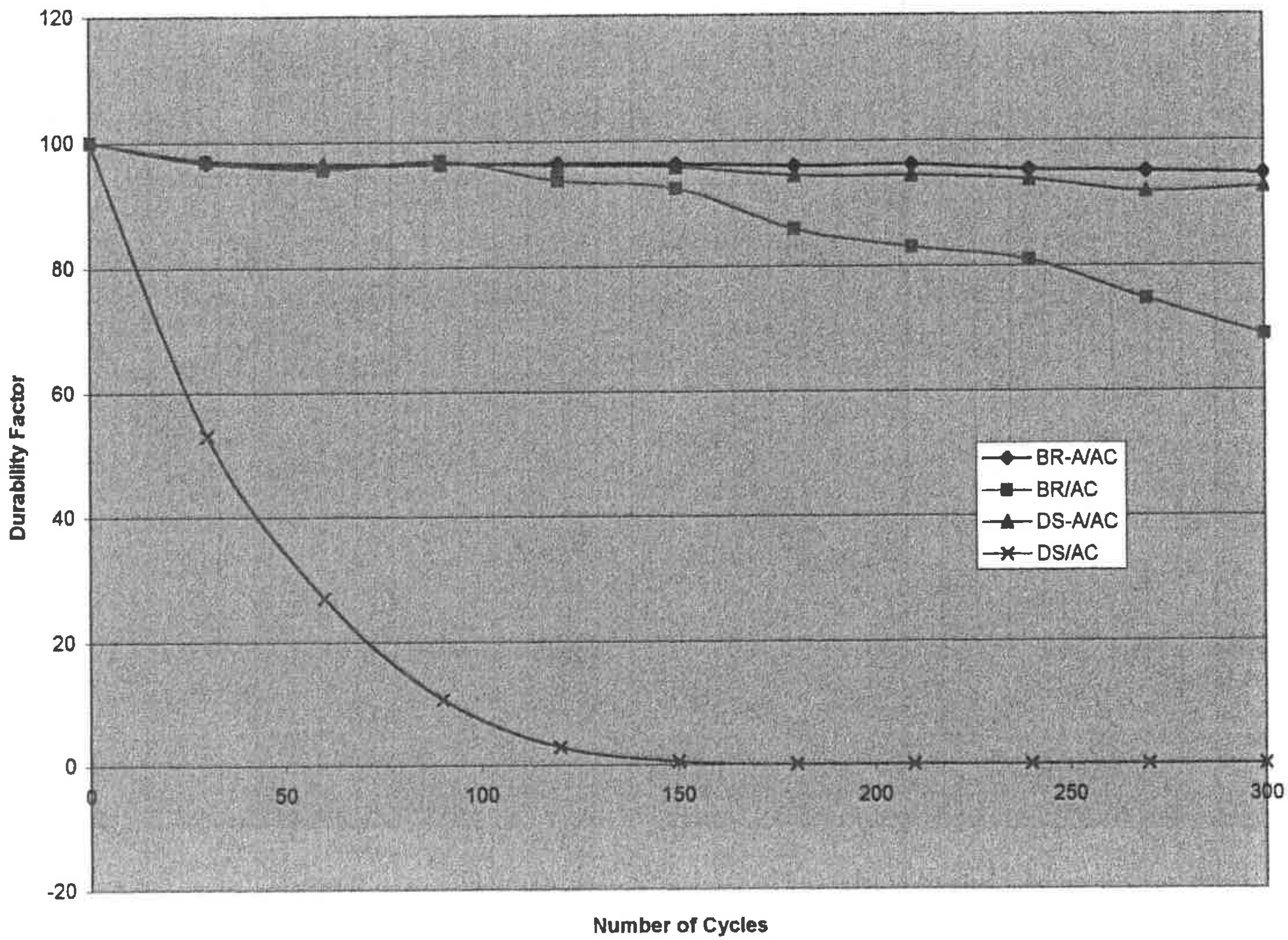


Figure 6.16 Cycle Number vs. Durability Factor of AC Mixtures

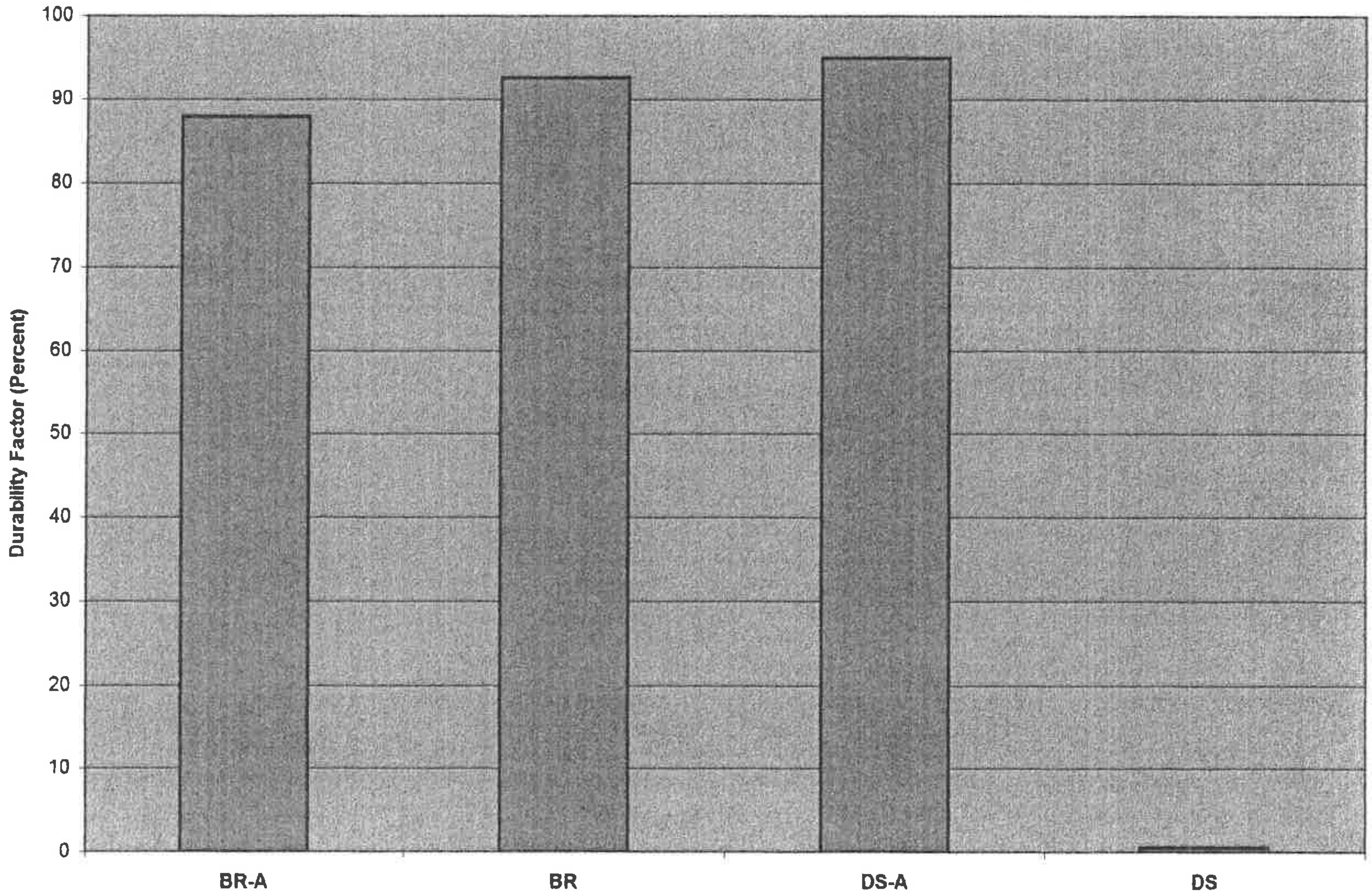


Figure 6.17 Durability Factor of Concrete Mixtures

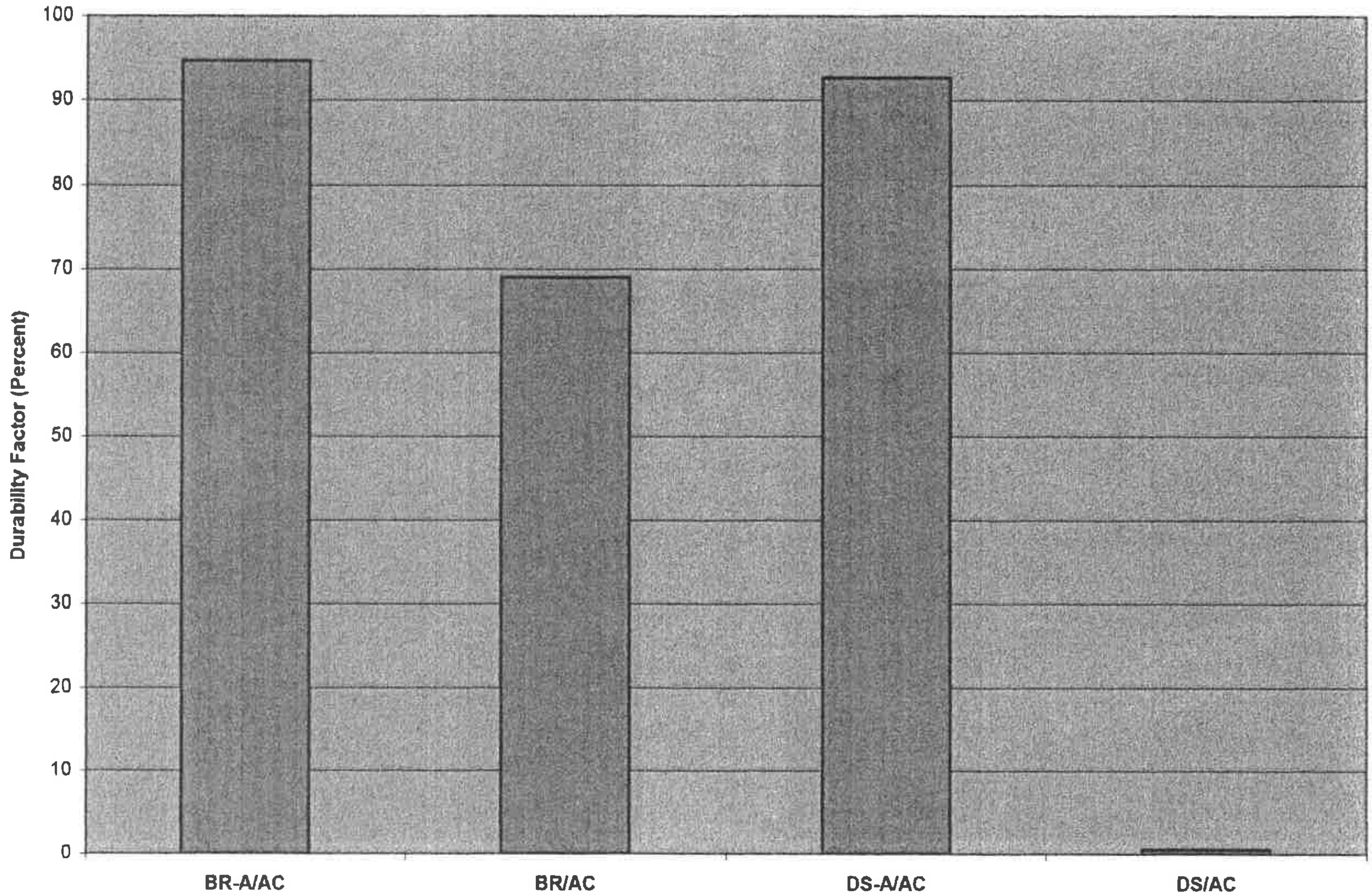


Figure 6.18 Durability Factor of AC Mixtures

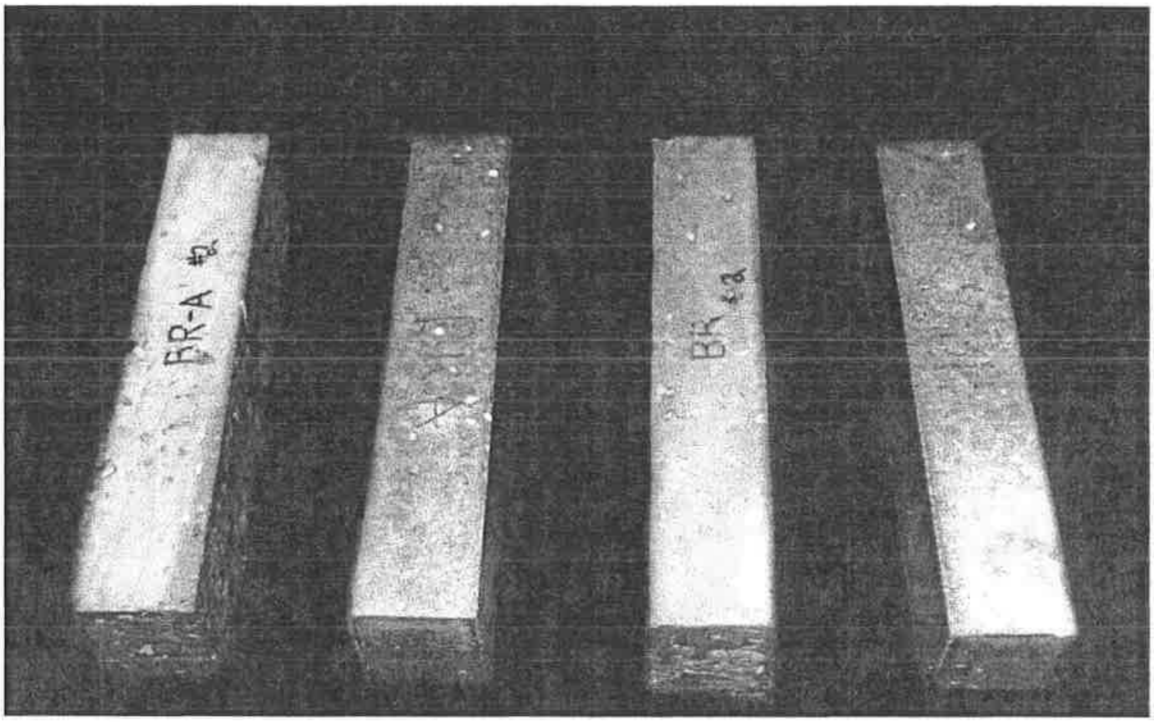


Figure 6.19.a (left to right) BR-A, BR-A/AC, BR, BR/AC after 300 freeze-thaw cycles

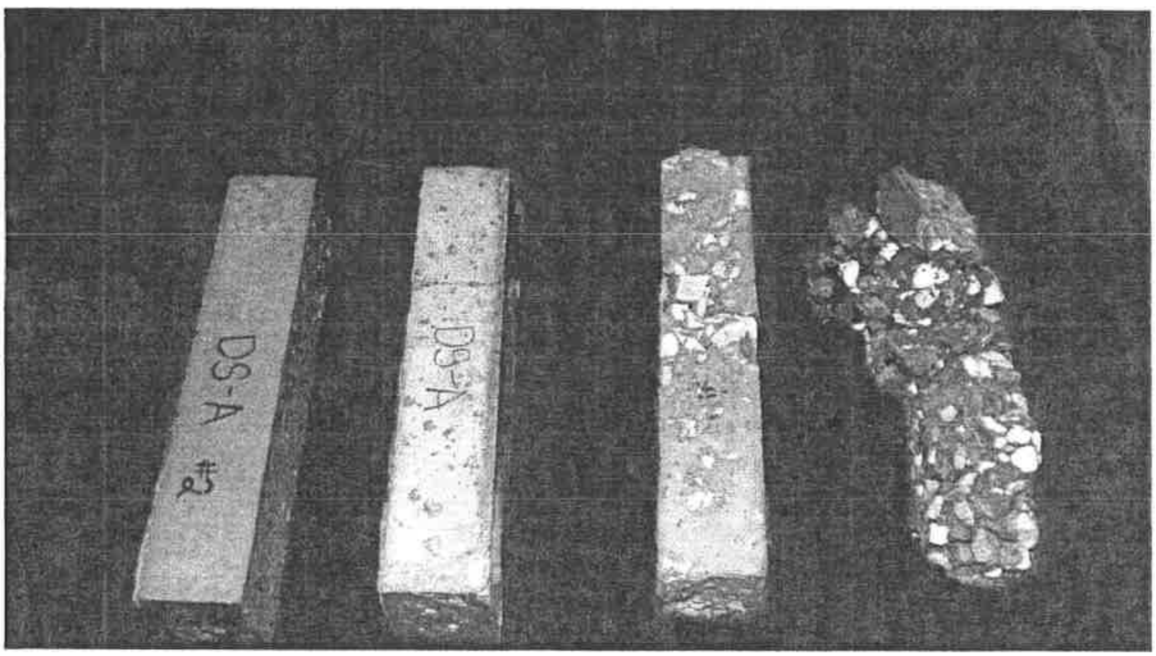


Figure 6.19.b (left to right) DS-A, DS-A/AC, DS, DS/AC after 300 freeze-thaw cycles

7. CONCLUSIONS AND RECOMMENDATIONS

7.1 General

Entrained air has been used in concrete in the past to provide freeze-thaw resistance, reduced permeability, better overall durability, and improved workability. With the development of high performance concrete mixtures, some researchers suggest that entrained air is not needed in many instances where fly ash and/or micro silica are used to achieve good workability, low permeability, and durable concrete. A pilot investigation of the performances of ALDOT's high performance bridge and drilled shaft concrete mixtures with and without A/E was viewed as being helpful in assessing the importance of using A/E in these mixtures. This was the impetus and purpose of this investigation. Fresh concrete properties/workability and the hardened compressive strength, permeability, and freeze-thaw durability properties of the mixtures were evaluated and compared. The investigation was limited to testing of laboratory prepared mixtures under ambient laboratory conditions.

The cold weather data and information gathered and discussed in Chapters 2 and 3 indicate that all regions of Alabama undergo a significant number of freeze-thaw cycles each year, and thus during the service life of an outdoor concrete structure located in the state. This coupled with the fact that average annual rainfall in Alabama is high make condition for potential freeze-thaw damage to concrete structures in the state very real. However, due to its basically warm climate, the ground does not freeze very deep in Alabama. This is particularly true in south Alabama, which is where most of the concrete pile foundations are employed in the state. According to Highway Research Board

(HRB) Publication 211, the maximum depth of frost penetration in the state of Alabama is approximately 17 inches, and the depth of penetration of freezing of water in concrete (approximately 26°F) would be even less. Additionally, even in the upper regions of piling and drilled shafts where they are exposed to subfreezing temperatures, because of their vertical orientations, they should not be in a saturated condition and thus freeze-thaw should not be a problem. An exception to this would be in regions where the water table is less than 17 inches below the ground surface.

7.2 Conclusions

Based on results from the literature review, and performing laboratory testing on fresh and hardened concrete properties of the four concrete mixtures, it is the opinion of the authors that high performance bridge concrete without A/E (designated BR in this study) would be an acceptable replacement for ALDOT's standard high performance bridge concrete. It was stronger in compressive strength, its permeability was in the same range as the standard high performance bridge concrete, and it had a better durability factor than the standard high performance bridge concrete. Its fresh properties were all comparable to the standard high performance mixture (except for air content which was substantially less).

It is also the opinion of the authors that the drilled shaft concrete mixture without A/E (designated DS in this study) is not an acceptable substitute for ALDOT's standard mix for drilled shaft concrete. Although its strength was greater and its permeability was in the same range as the standard, it did not successfully complete the freeze-thaw testing. This could lead to freeze-thaw failures at locations throughout Alabama.

Specific conclusions based on laboratory testing of the fresh and hardened properties of the four concrete mixtures and the all-cement mixtures are as follows:

BR-A: The BR-A mixture was the ALDOT standard high performance bridge concrete with A/E. It did not perform as well as the experimental BR mixture. It yielded the lowest unit weight. Its compressive strength was the lowest of all of the mixtures, and did not even meet the 28-day strength requirements. It also had a somewhat lower durability factor. This was surprising because it should have theoretically been better than the BR mixture because of the addition of A/E.

BR: The BR mixture preformed the best of all the mixtures. It met or exceeded all of the fresh properties of the standard BR-A mixture. Its compressive strength at 7-days nearly met its 28-day strength requirements. The final 56-day strength was approximately 9500 psi which was154% higher than the ALDOT standard BR-A mixture. It had approximately the same permeability rating as the standard mixture. It preformed somewhat better in freeze-thaw without the addition of A/E. Based on laboratory testing only, it would seem that the BR mixture is an appropriate replacement for the ALDOT standard high performance bridge concrete mixture.

DS-A: The DS-A mixture was ALDOT's standard drilled shaft mixture with A/E. It preformed better than the experimental DS mixture, which contained no A/E. Although it was weaker than the DS mixture in compression tests, it was able to meet all strength requirements. The DS-A mixture was also able to complete the freeze-thaw test (unlike the DS mixture). The DS-A mixture provided the highest durability factor of all the mixture at 95%.

DS: The DS mixture was the experimental drilled shaft mixture without A/E. It had the highest compressive strength of all the mixtures, as well as the highest unit weight and best permeability. However, the DS mixture was the worst of all the mixtures in durability. It did not even finish the freeze-thaw testing. It therefore had a durability factor of 0%. Based solely upon this, it cannot be considered as a viable replacement for the ALDOT standard DS-A concrete mixture.

All-Cement (AC) Mixtures: The all-cement mixtures tested approximately the same as their fly ash counterparts in the fresh property testing. The only hardened concrete property tested on the AC mixtures was the freeze-thaw testing. The BR/AC, DS-A/AC, and DS/AC mixtures did not perform as well as their corresponding mixtures with fly ash in the freeze-thaw testing. The BR-A/AC mixture preformed somewhat better that the BR-A mixture. Overall, the freeze-thaw performances of the all-cement (AC) mixtures were inferior to those of their counterpart mixtures containing fly ash.

7.3 Recommendations

Based on the results of this study, the BR concrete mixture appears to be a viable candidate for use as a high performance bridge concrete to yield stronger concrete that performs just as well as the standard high performance mixture containing A/E in freeze-thaw durability. The BR mixture exceeded or was comparable to all of the fresh and hardened concrete properties of the standard ALDOT high performance bridge concrete (BR-A). This being the case, the following recommendations are made regarding the BR mixture.

1. Because this study was limited to testing of laboratory prepared mixtures under ambient laboratory conditions, a more complete spectrum of mixture variations,

parameters tested, and environmental placement conditions should be preformed before any actions to omit the use of A/E from high performance bridge concrete are taken.

2. A "test" beam employing the BR mixture should be constructed to be placed in the field in north Alabama adjacent to other beams that employ the ALDOT standard high performance mixture. This beam should then be monitored and evaluated to assess its performance in freeze-thaw durability under actual field conditions.

Based on the results of this study, the drilled shaft concrete mixture without A/E, i.e. the DS mixture, is not an acceptable substitute for ALDOT's standard drilled shaft concrete mixture. Also, while the need for air-entrainment in piling and drilled shafts is very questionable and in fact is probably not needed, unless there are compelling reasons to do otherwise, an air content of 5% in these members is probably still a good design practice. The air reduces concrete cost, improves workability of the mixture (and thus less water can be used in the mixture), helps achieve a quality placement and consolidation of the in-place concrete, and provides freeze-thaw protection at a minimal reduction in compressive strength.

8. REFERENCES

1. ACI, "Guide to Durable Concrete," ACI 102.2R-77, Detroit, Michigan, 1982.

2. Bowles, J.E., Foundation Analysis and Design, McGraw Hill Co., New York, N.Y., 1968.

3. Christy, John, "El Niño Means Cool Winters for Alabama, Wet on Gulf Coast, and Drier as you go North," Internet Article, University of Alabama Huntsville, www.uah.edu/News/archivedNews/Recent/120897-01.html

4. Davis, Harmer E., Gerorge Earl Troxell, George F.W. Hauk, The Testing of Engineering Materials, 4th Ed., John Wiley and Sons, Inc., New York, 1996.

5. Fiorato, A.E., "PCA Research on High Strength Concrete," ACI Compilation 17, American Concrete Institute, Farmington Hills, Michigan, 1994.

6. Foy, C., M. Pigeon, and N. Bauthia, "Freeze-Thaw Durability and Deicer Salt Scaling Resistance of a .25 Water-Cement Ratio Concrete," Cement and Concrete Research, 1988.

7. Gagne, R., M. Pigeon, and P.C. Aitcin, "Durabilité au gel bétons de hautes performances mécaniques," Materials and Structures, RILEM, 1990.

8. Hammer, T.A., and E.J. Sellevold, "Frost Resistance of High Strength Concrete," ACI SP – 121, 1990.

9. Helmuth, R.A., Discussion of Paper VI-2, "Frost Action in Concrete," *Chemistry of Cement, Proceedings of the Fourth International Symposium,* Washington, 1960, pp 829-833.

10. Highway Research Board, "Frost Action in Roads and Airfields," HRB Publication 211, National Academy of Sciences – National Research Council, Washington D.C., 1952.

11. Hill, Russell, S. Sarkar, R. Rahbone, and J. Hower, "An Examination of Fly Ash Carbon and its Interactions with Air Entraining Agent," Cement and Concrete Research, Vol. 27, No. 2, Elsevier Science Ltd. 1997, pp. 193-204.

12. Malhotra, V.M., K. Painter, and A.Bilodeau, "Mechanical Properties and Freezing and Thawing Resistance of High-Strength Concrete Incorporating Silica Fume," CANMET - ACI International Workshop on Condensed Silica Fume in Concrete, 1987.

13. McCarthy, David F., Essentials of Soil Mechanics and Foundations, Prentice Hall, New Jersey, 1997.

14. National Climatic Data Center, "Mean Number of Days with Minimum Temperature 32 Degrees F or Less," Internet Article, NCDC, www.ncdc.noaa.gov/ol/climate/online/ccd/min32temp.html

15. Nawy, Edward G, Fundamentals of High Strength High Performance Concrete, Longman, Rutgers University, 1996.

16. Neville, A.M., Properties of Concrete, 4th Ed., John Wiley and Sons, New York, 1996.

17. Okada, E., M. Hisaka, Y. Kazama, and K. Hattori, "Freeze-Thaw Resistance of Superplasticized Concretes. Developments in the Use of Superplasticizers," ACI SP-68, 1981.

18. Shah, S.P., and S.H. Ahmad, High Performance Concrete Properties and Applications, McGraw-Hill, New York, 1994.

19. Sowers, G.B. and Sowers, G.F., Introductory Soil Mechanics and Foundations, 3rd Edition, MacMillan Co., New York, N.Y., 1970.

20. Stark, David, "Effect of Length of Freezing Period on Durability of Concrete," PCA Research and Development Bulletin RD 096T, PCA, 1989.

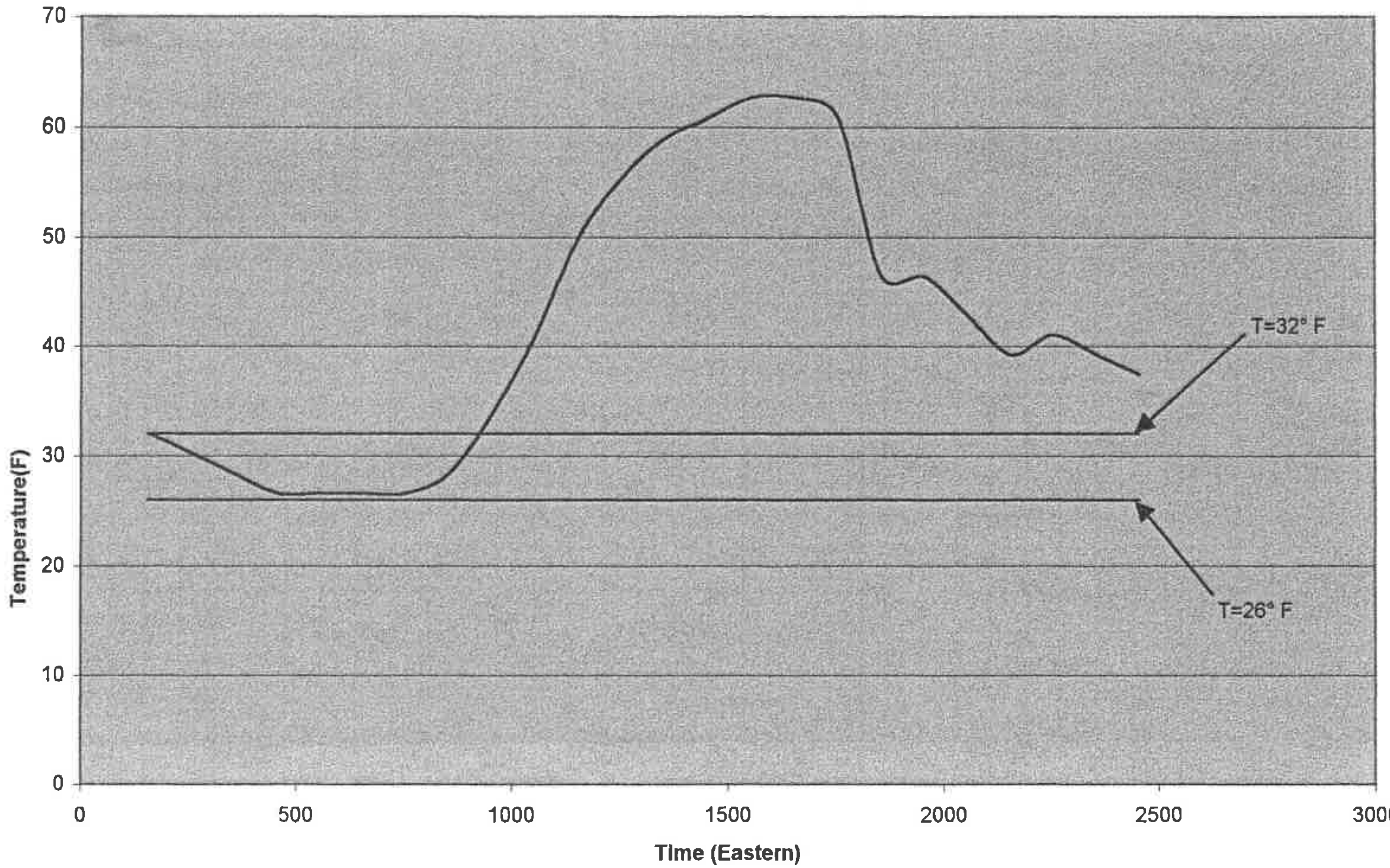
21. Thanakorn, Pheeraphan and C. Leung, "Freeze-Thaw Durability of Microwave Cured Air-Entrained Concrete," Cement and Concrete Research, Vol. 27, No. 3, Elsevier Science Ltd, 1997, pp. 427-435.

22. Verbeck, George and Klieger, Paul, "Calorimeter-Strain Apparatus for Study of Freezing and Thawing Concrete," Highway Research Board Bulleting 176, 1958, pp. 9-22.

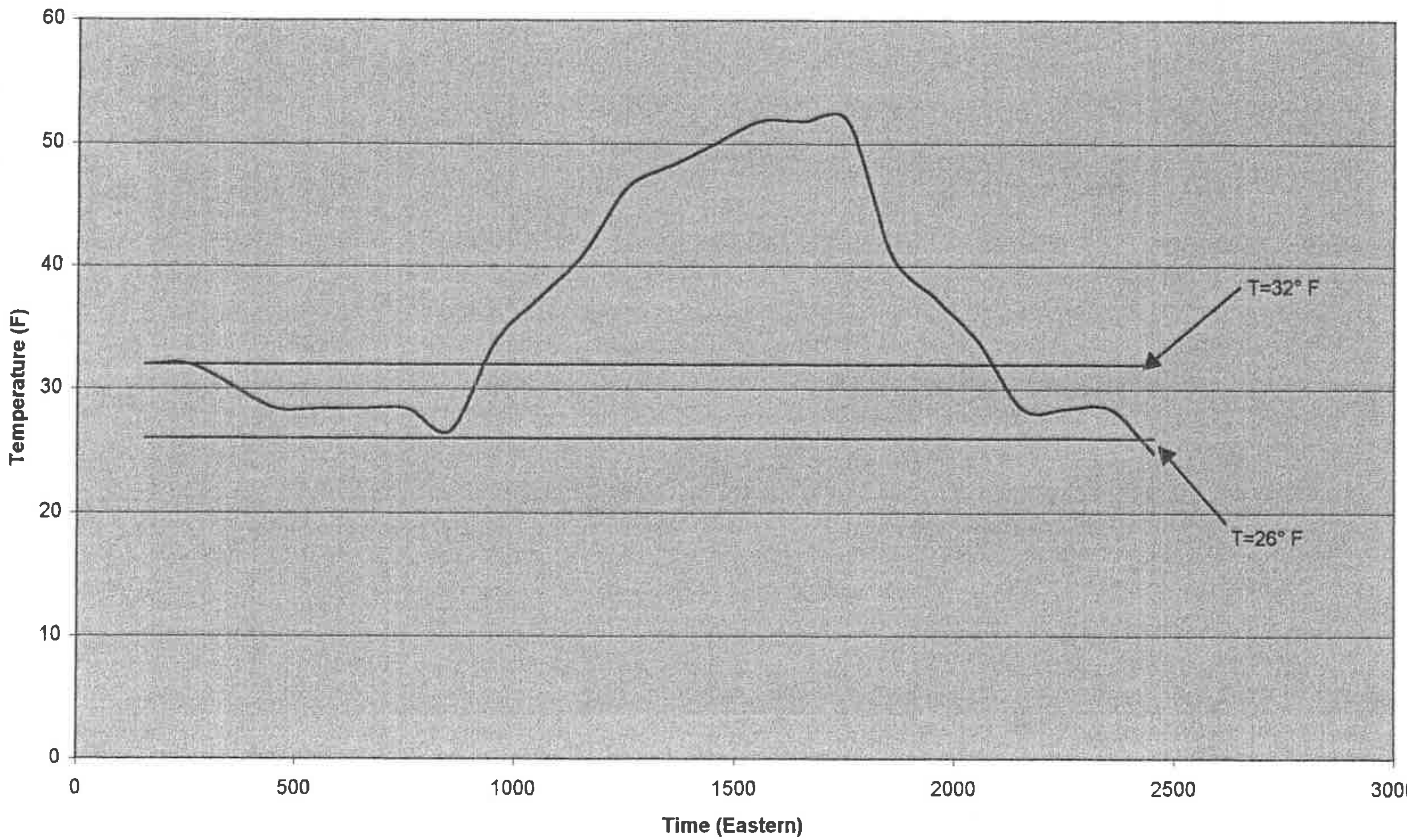
APPENDIX A

**Daily Temperature – Time Plots
of Fort Benning, Georgia Weather Data
for Subfreezing Days for Period
December 1, 2000 – January 31, 2001**

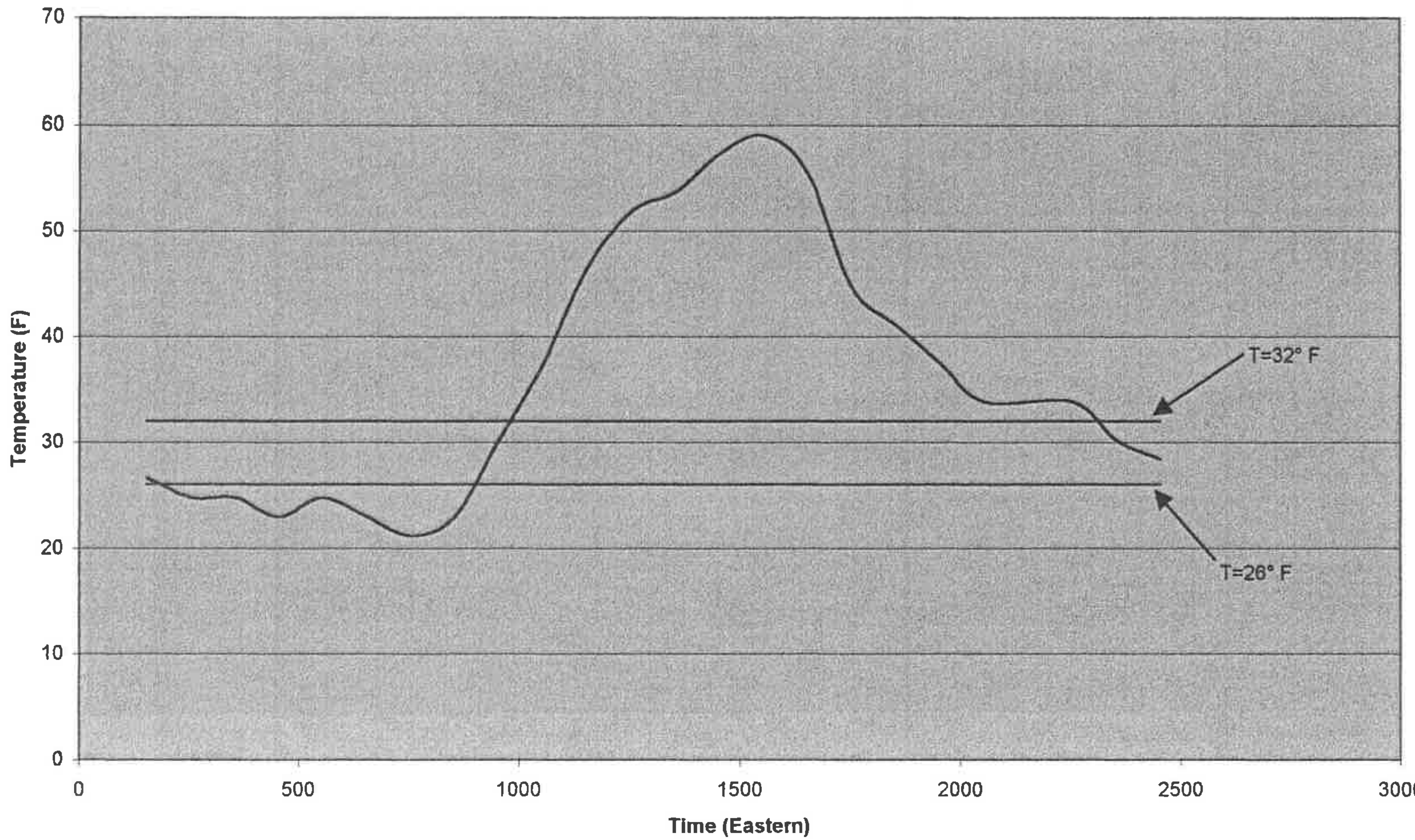
December 1, 2000



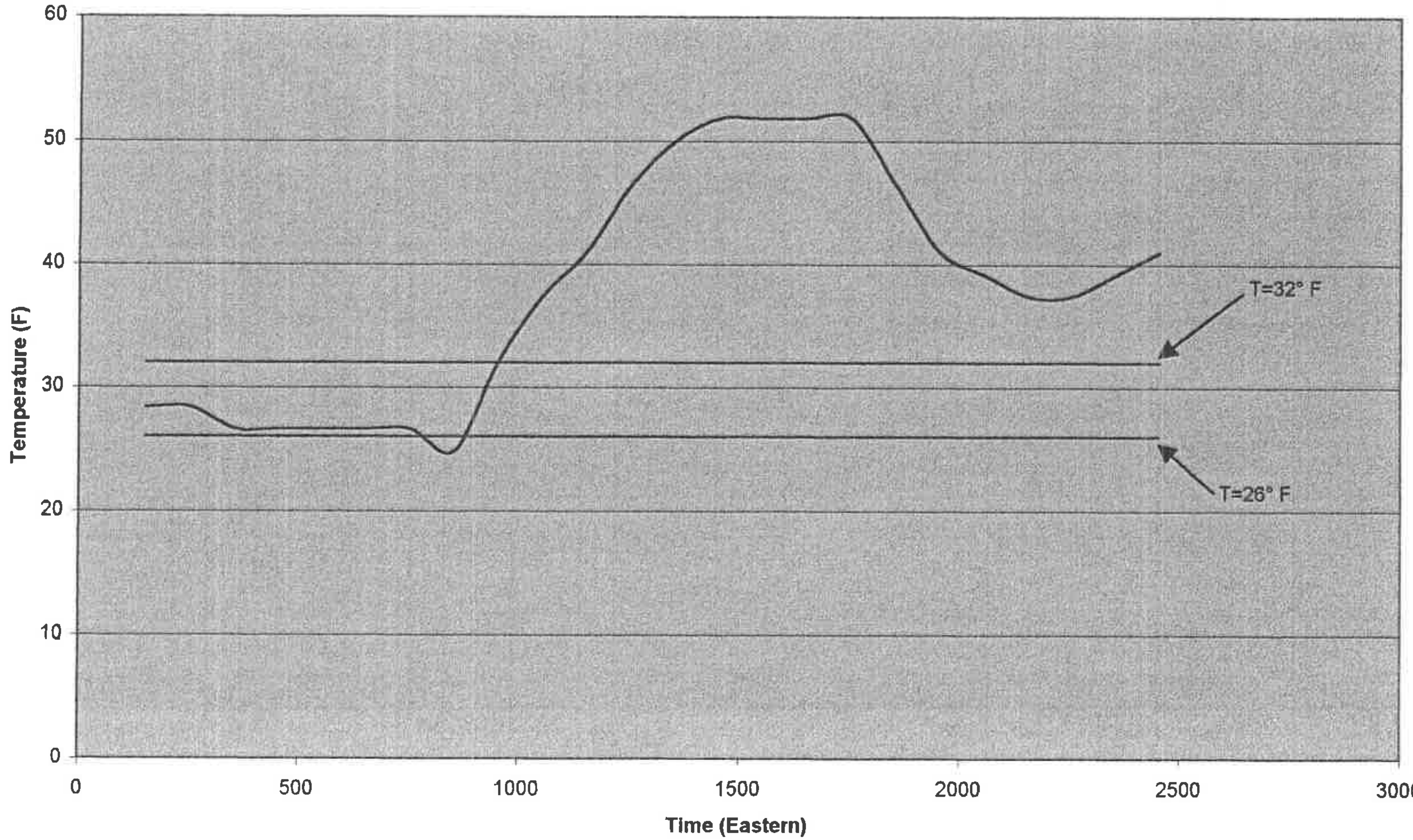
December 4, 2000



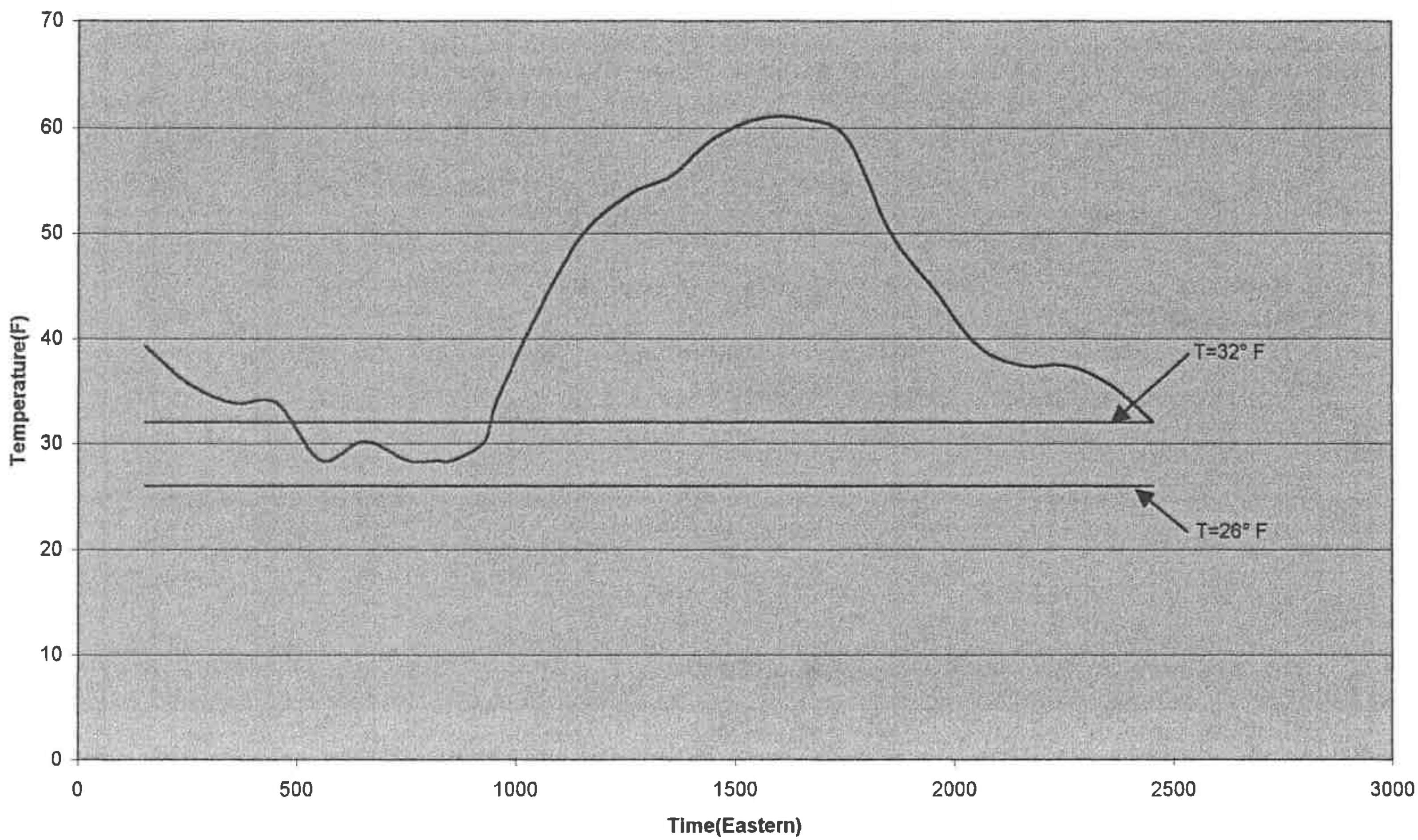
December 5, 2000



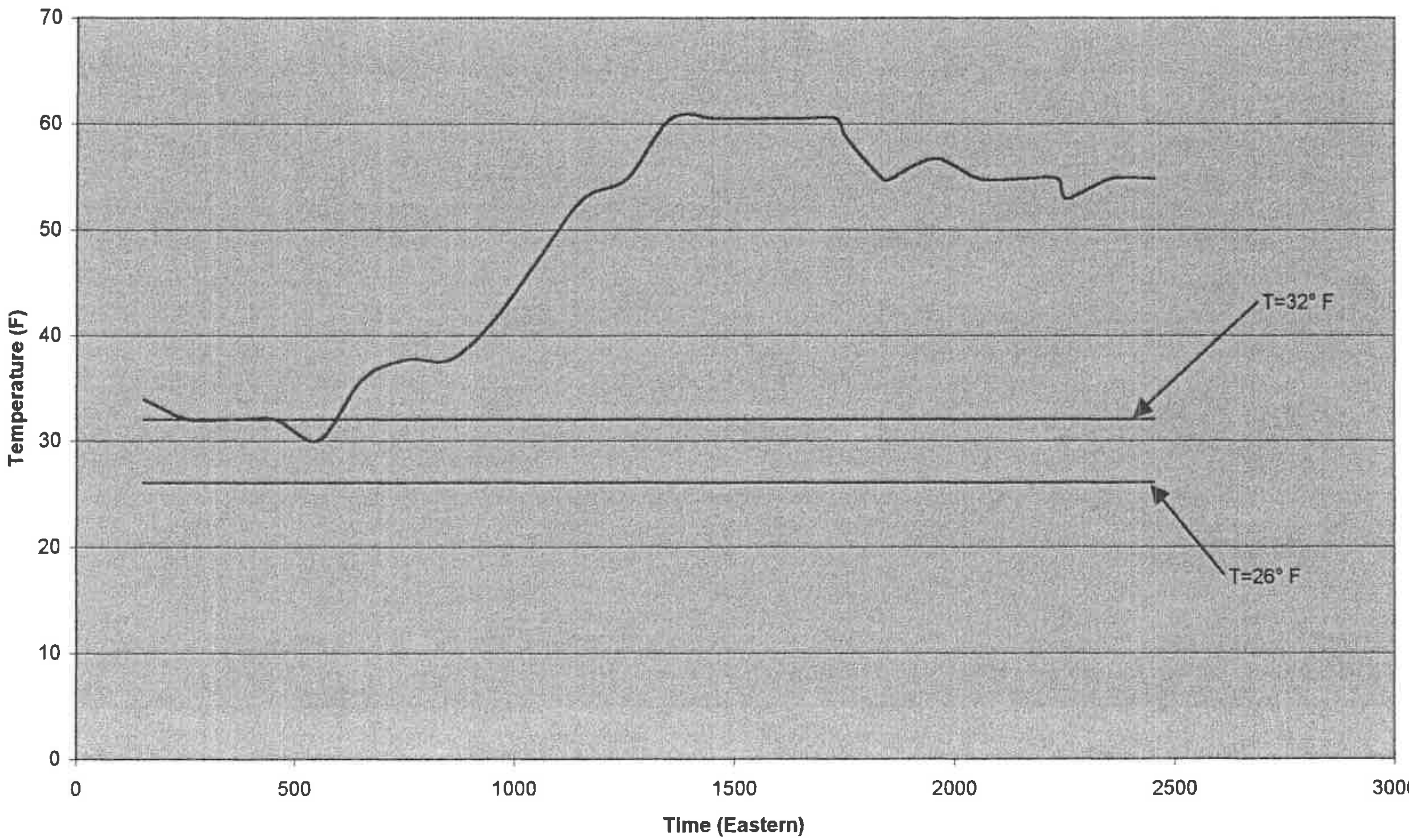
December 6, 2000

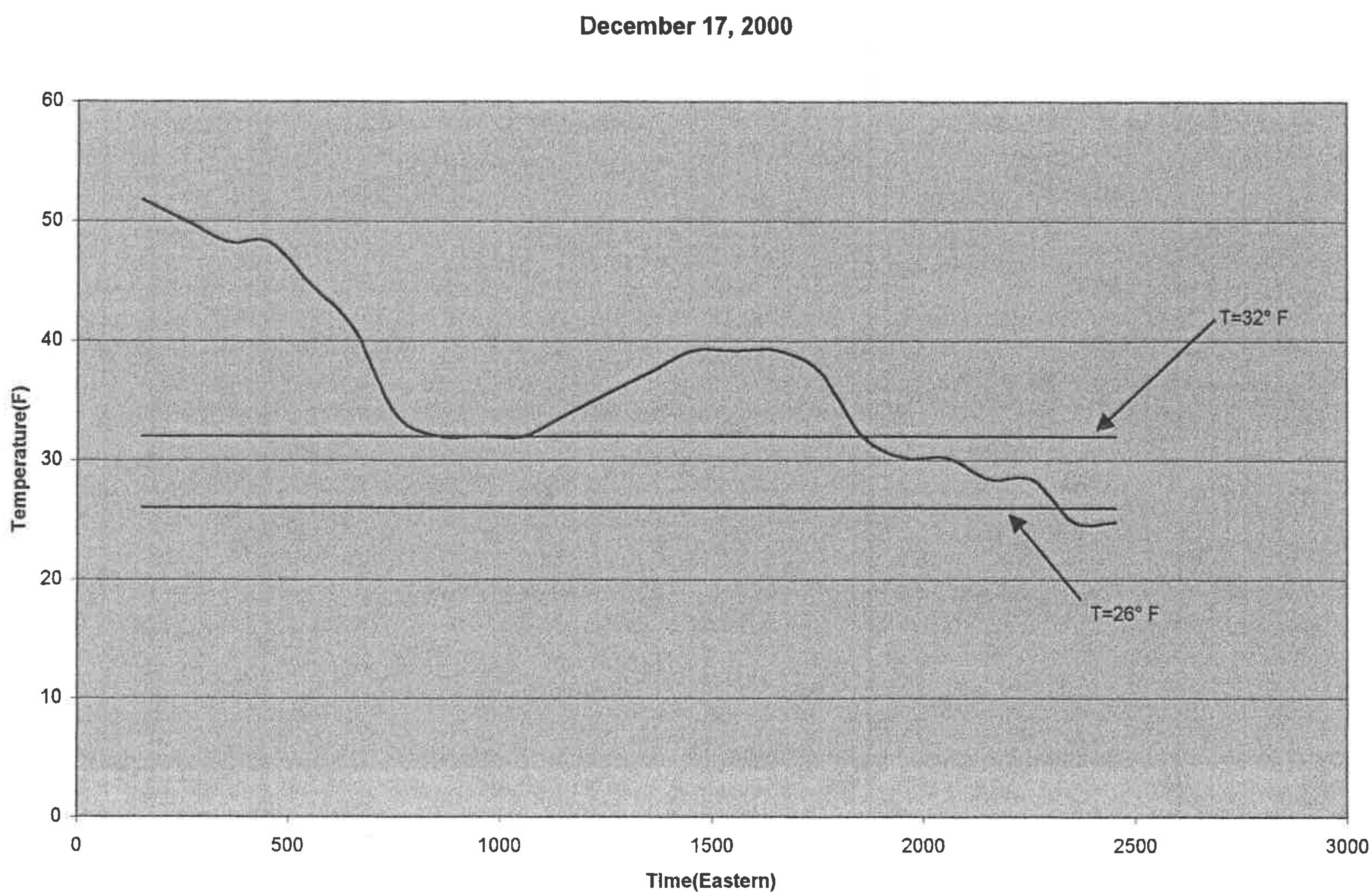


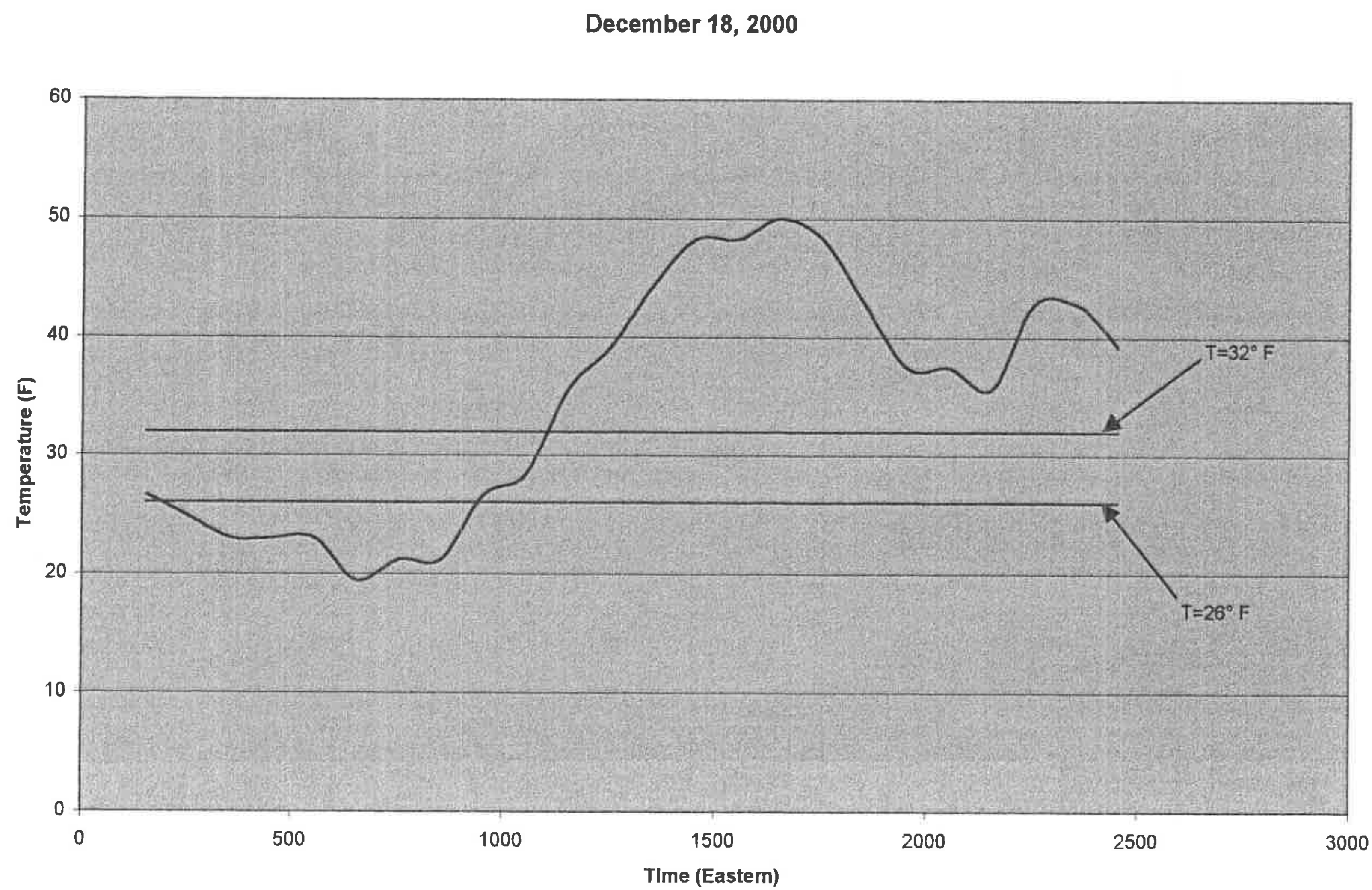
December 7, 2000



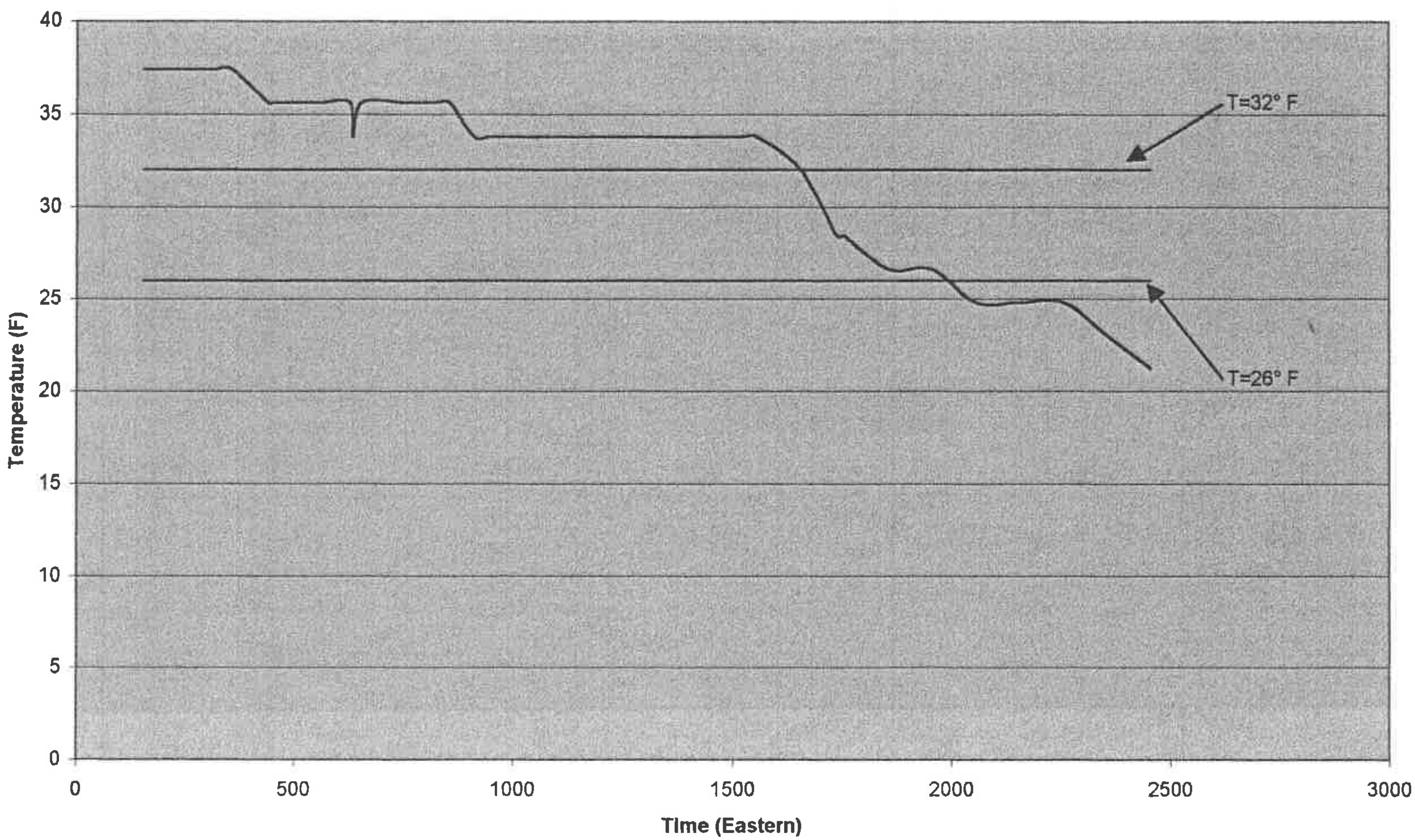
December 8, 2000



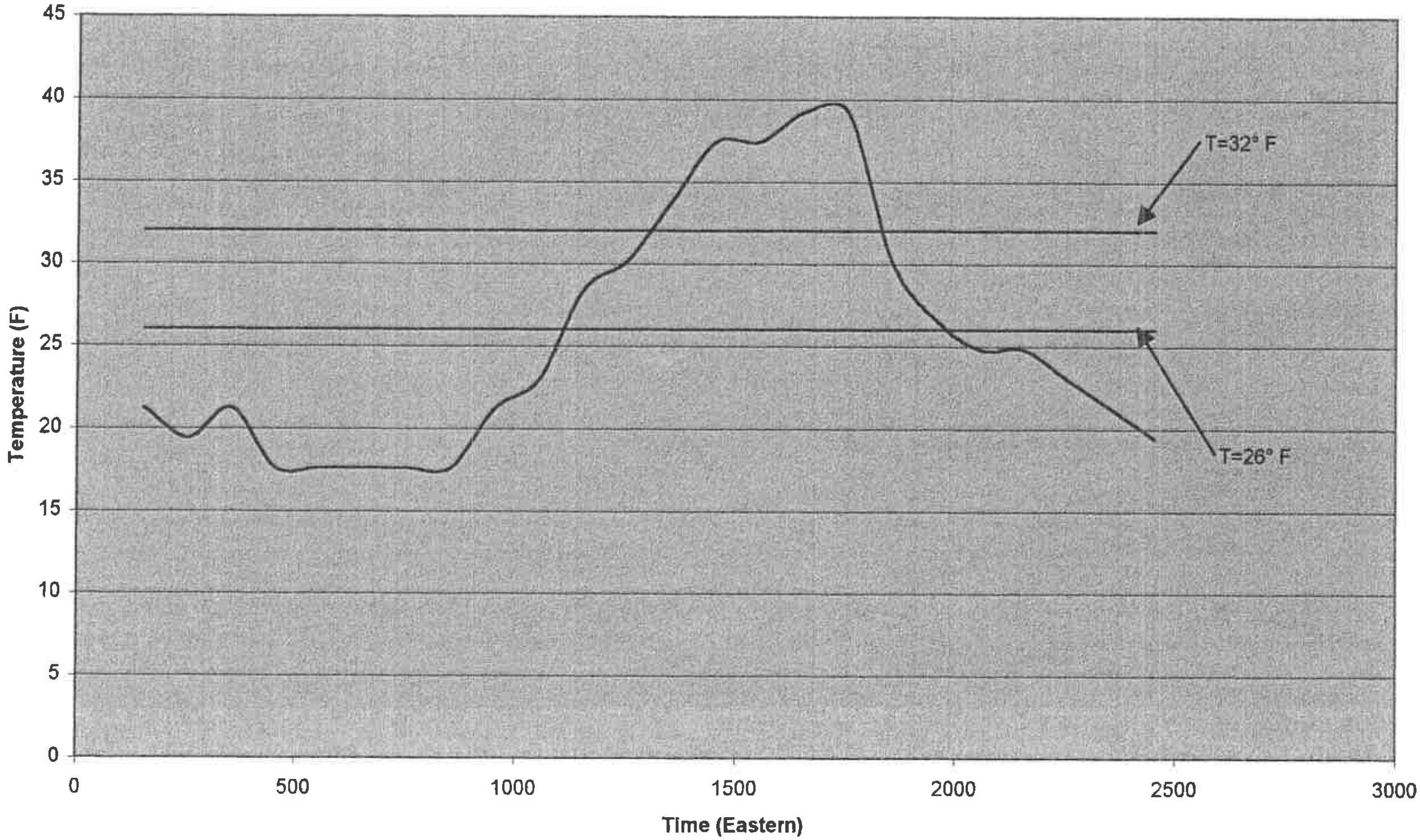

December 17, 2000
Temperature(F)
60
50
40
30
20
10
0
0
500
1000
1500
2000
2500
3000
Time(Eastern)
T=32° F
T=26° F


December 18, 2000
Temperature (F)
60
50
40
30
20
10
0
0
500
1000
1500
2000
2500
3000
Time (Eastern)
T=32° F
T=26° F

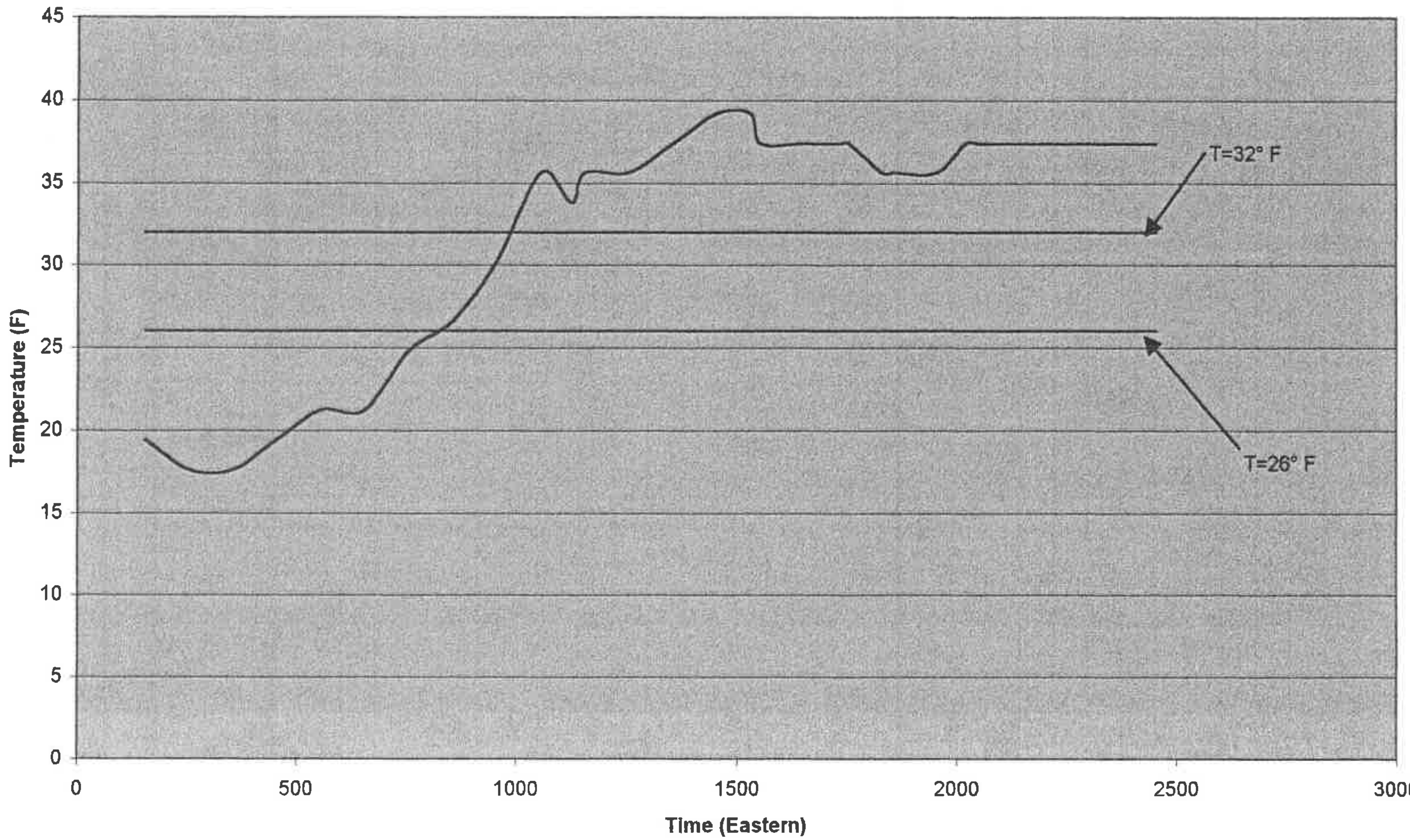
December 19, 2000



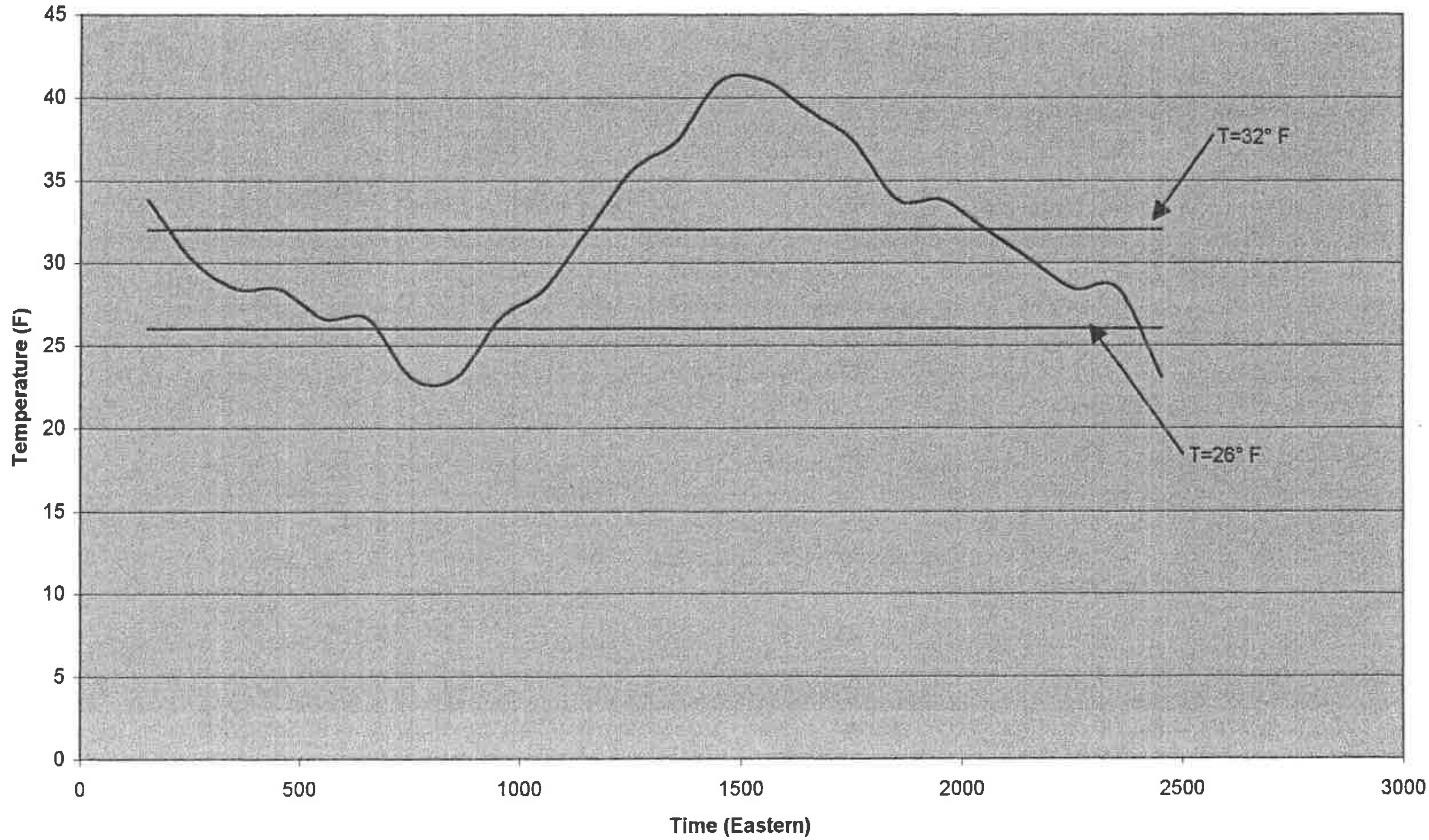
December 20, 2001



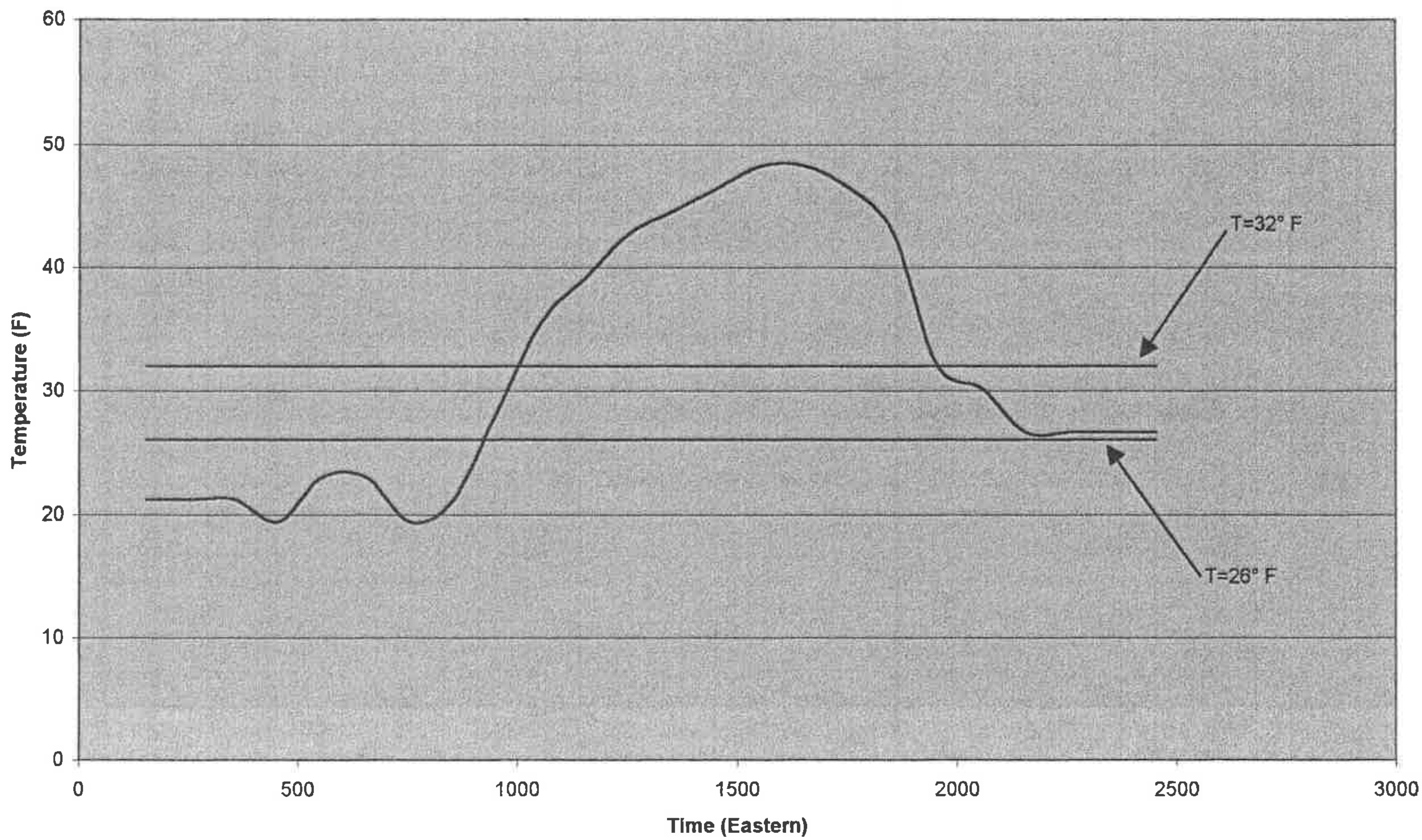
December 21, 2000



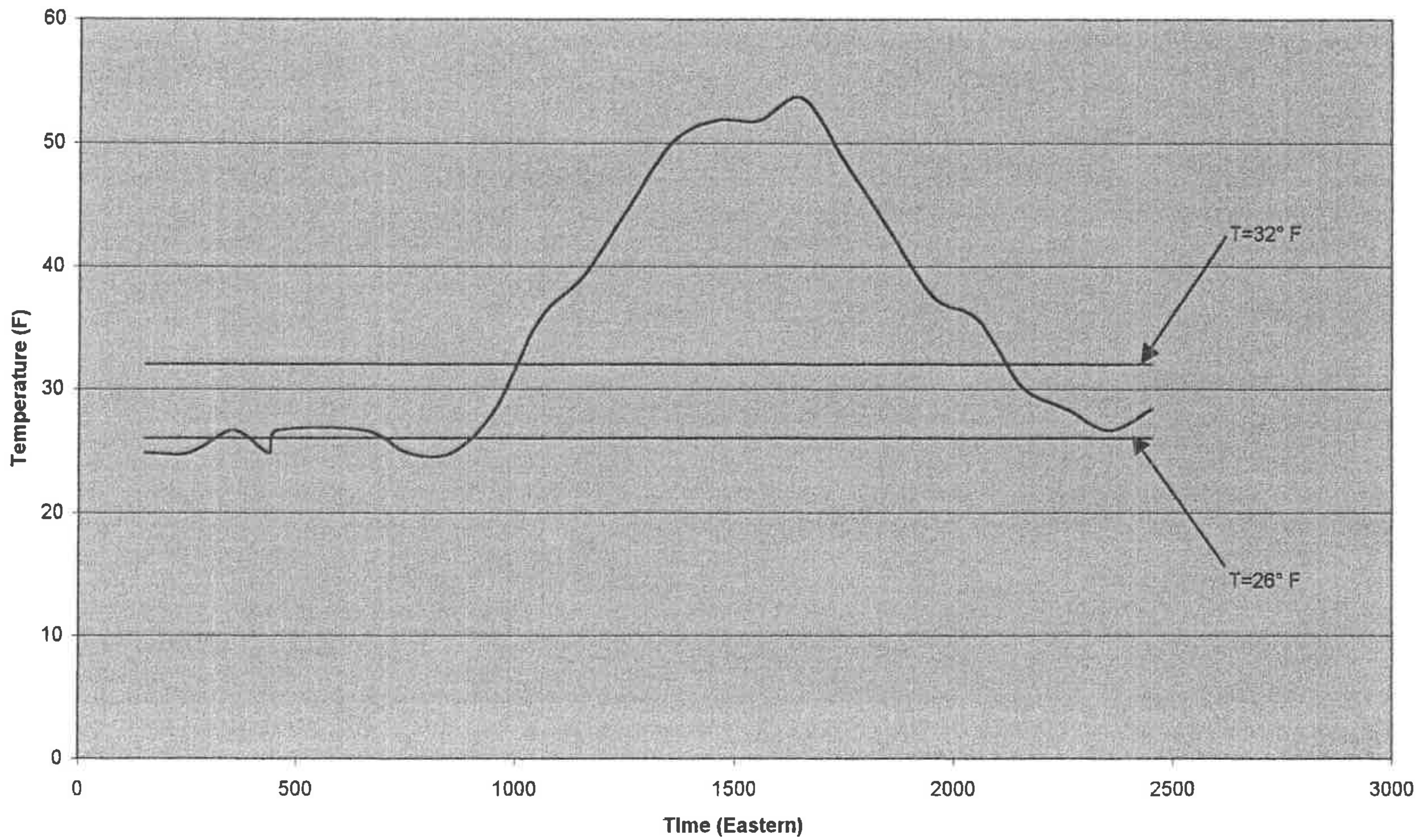
December 22, 2000

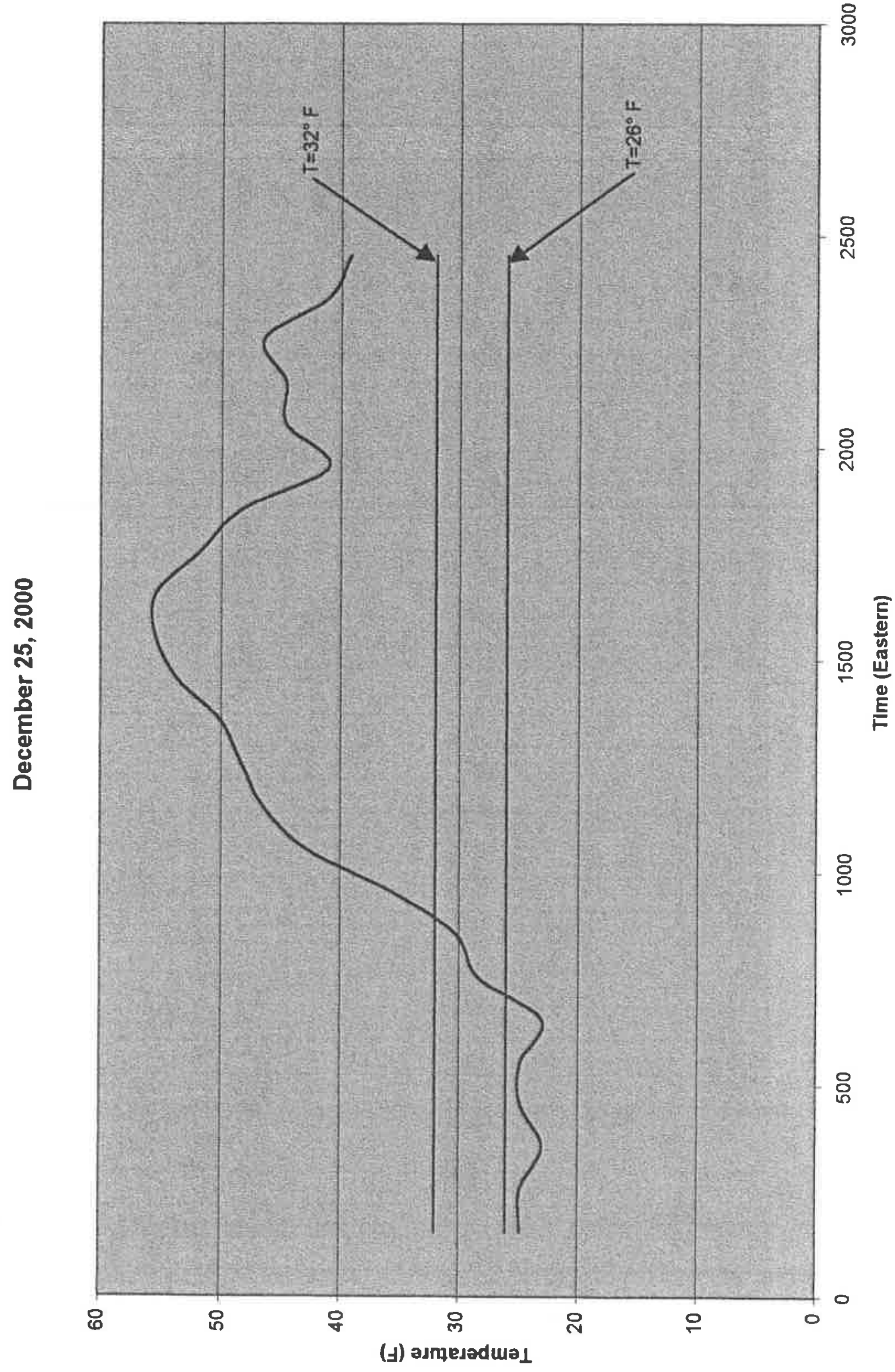


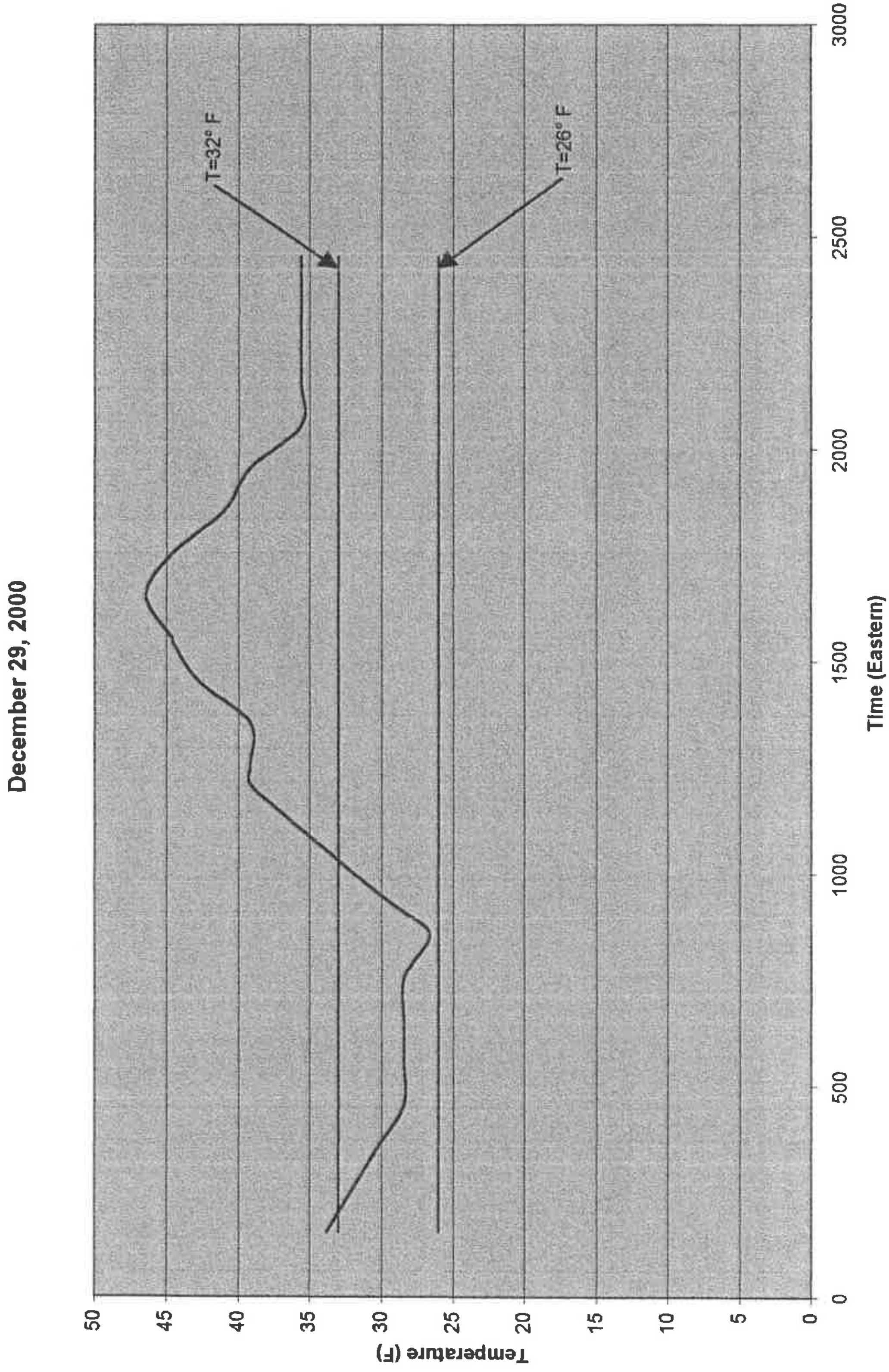
December 23, 2000



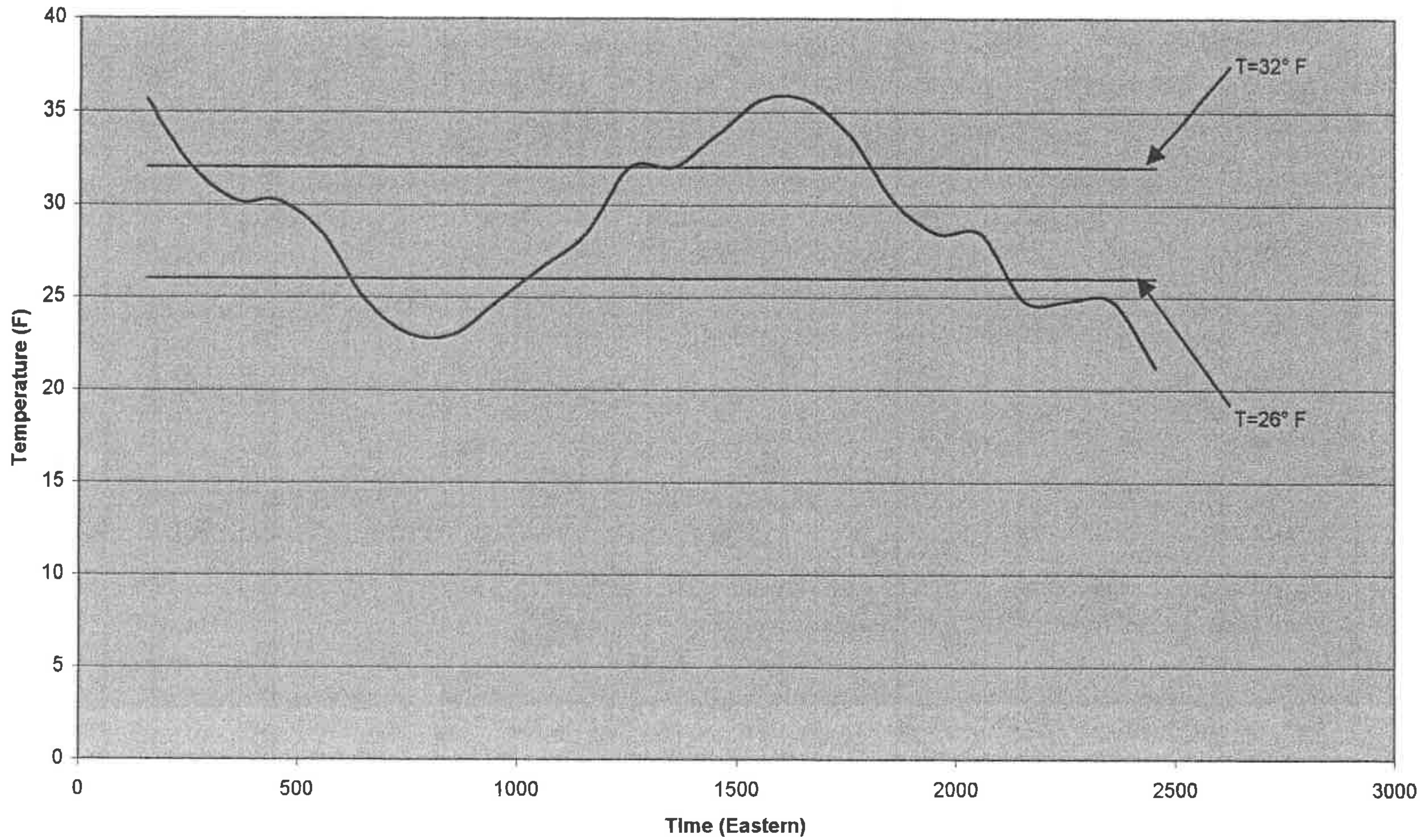
December 24, 2000

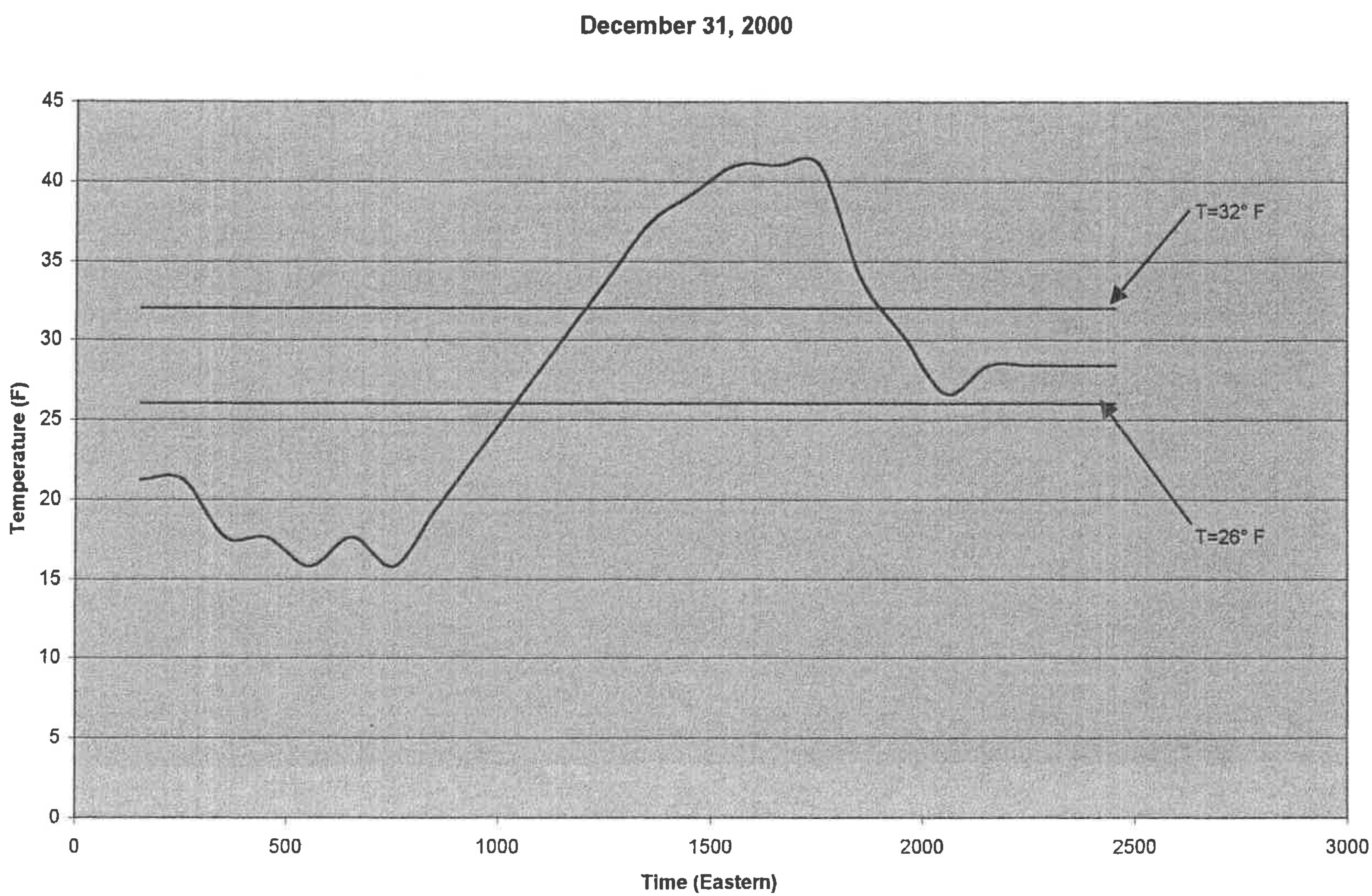


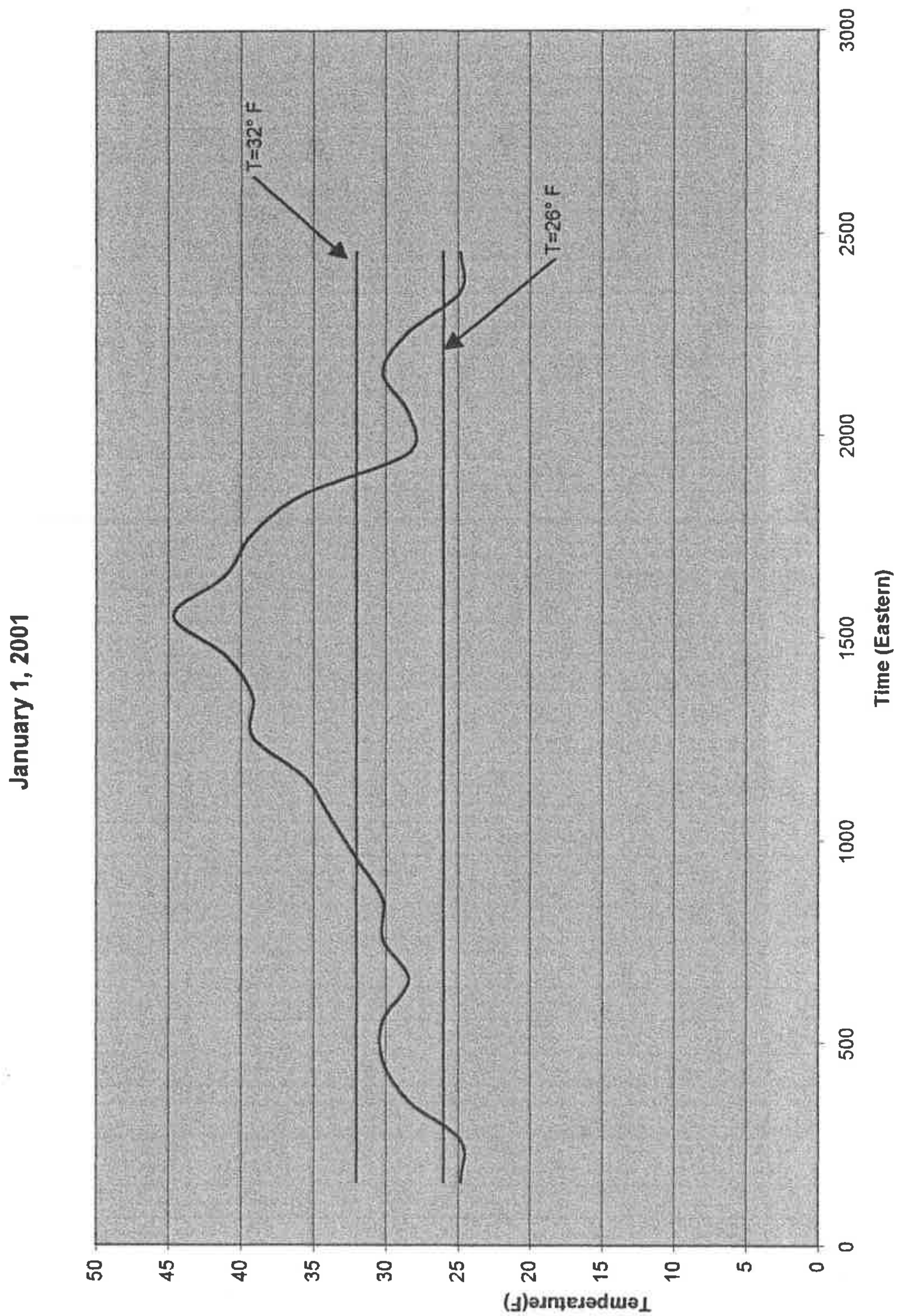

December 25, 2000
Temperature (F)
Time (Eastern)
T=32° F
T=26° F
0
10
20
30
40
50
60
0
500
1000
1500
2000
2500
3000


December 29, 2000
Temperature (F)
Time (Eastern)
T=32° F
T=26° F
0
5
10
15
20
25
30
35
40
45
50
0
500
1000
1500
2000
2500
3000

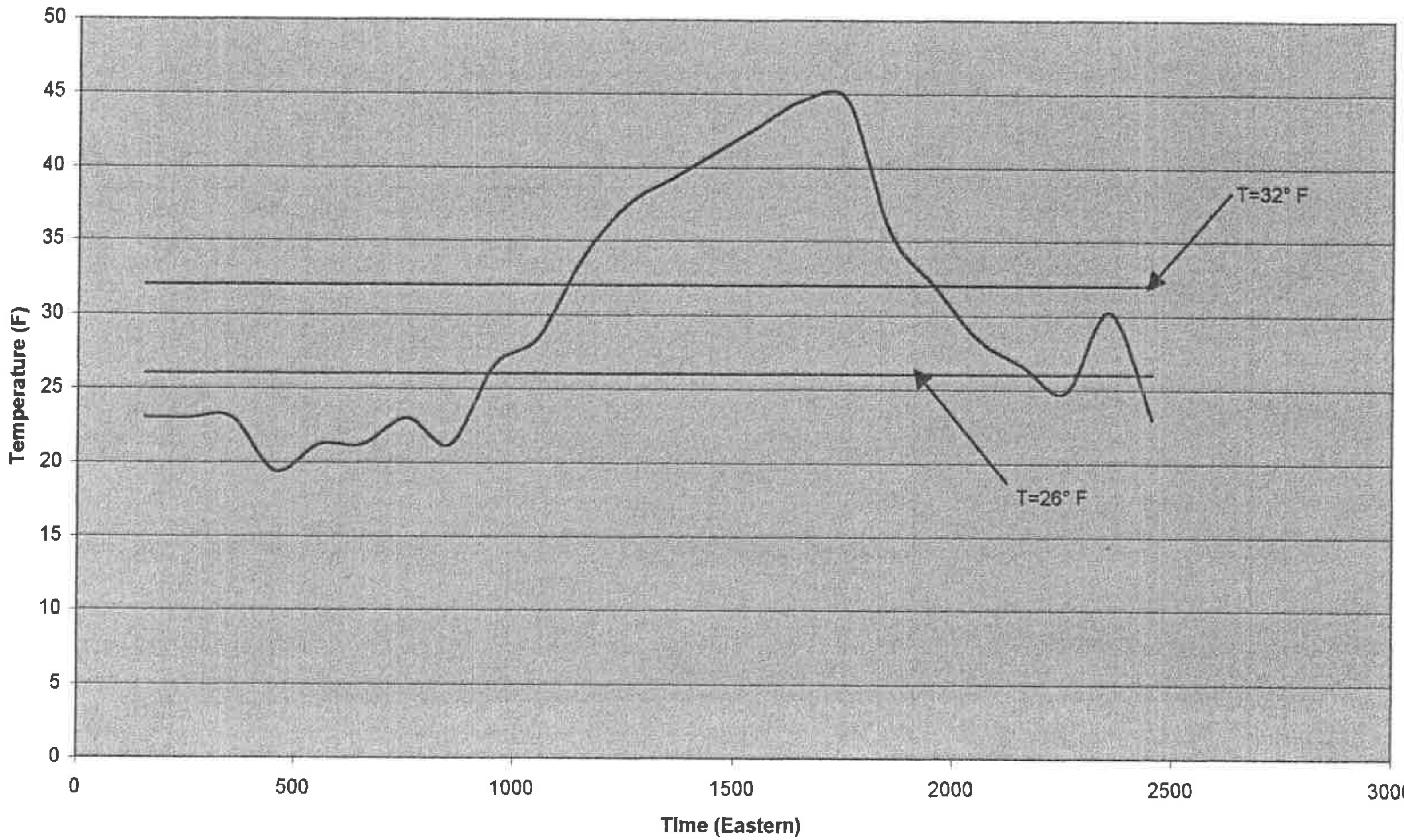
December 30, 2000



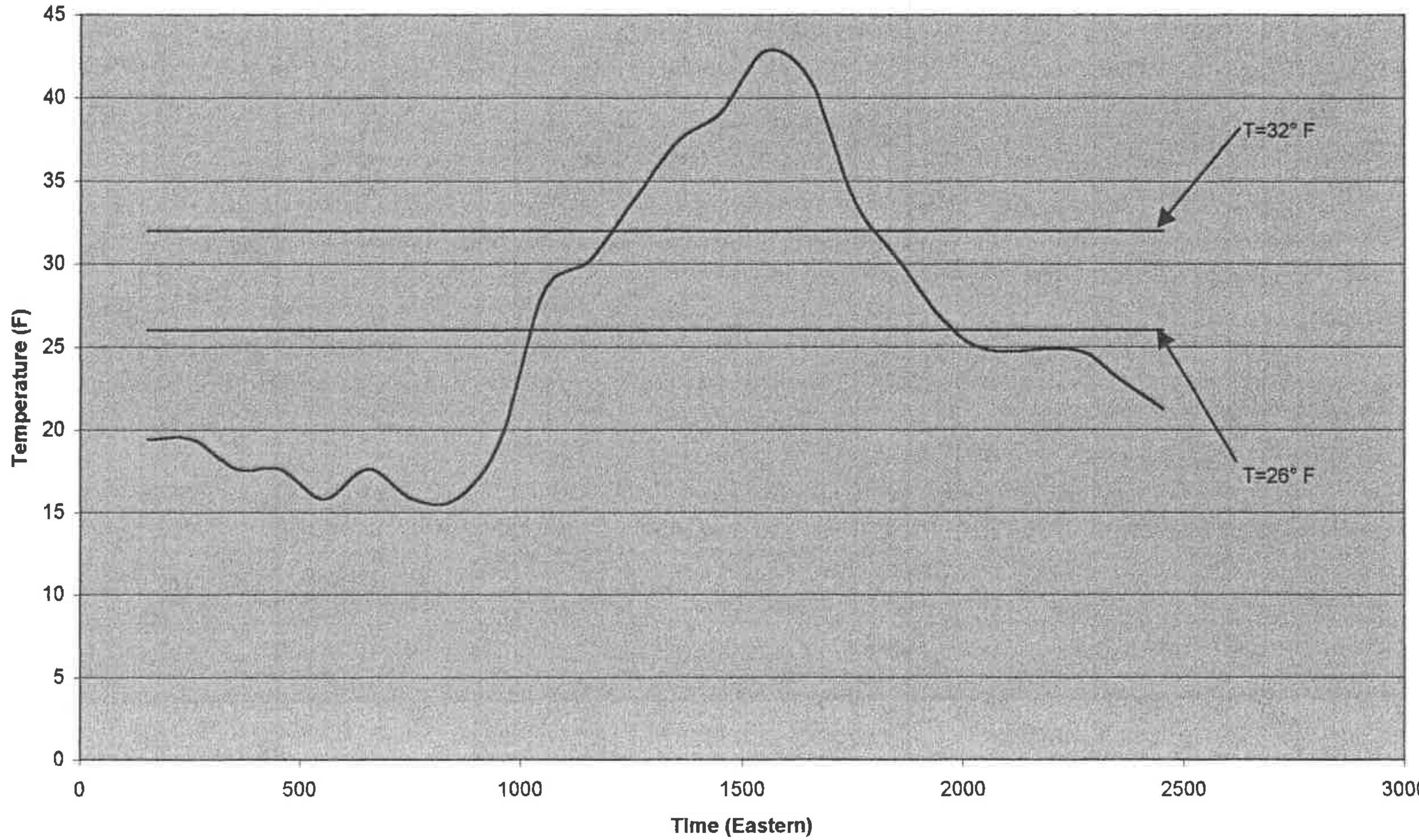

December 31, 2000
Temperature (F)
45
40
35
30
25
20
15
10
5
0
0
500
1000
1500
2000
2500
3000
Time (Eastern)
T=32° F
T=26° F


January 1, 2001
T=32° F
T=26° F
Temperature(F)
0
5
10
15
20
25
30
35
40
45
50
0
500
1000
1500
2000
2500
3000
Time (Eastern)

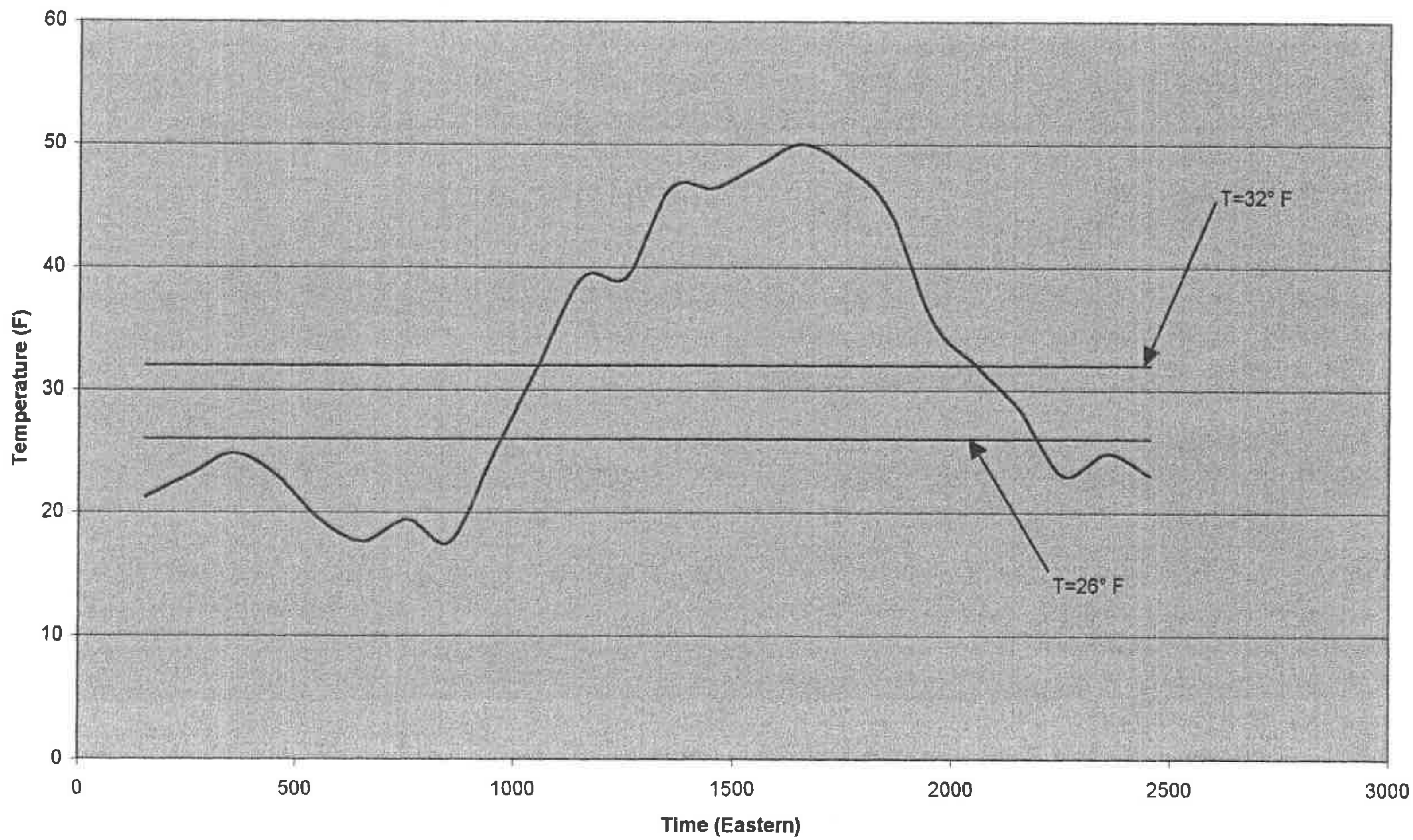
January 2, 2001

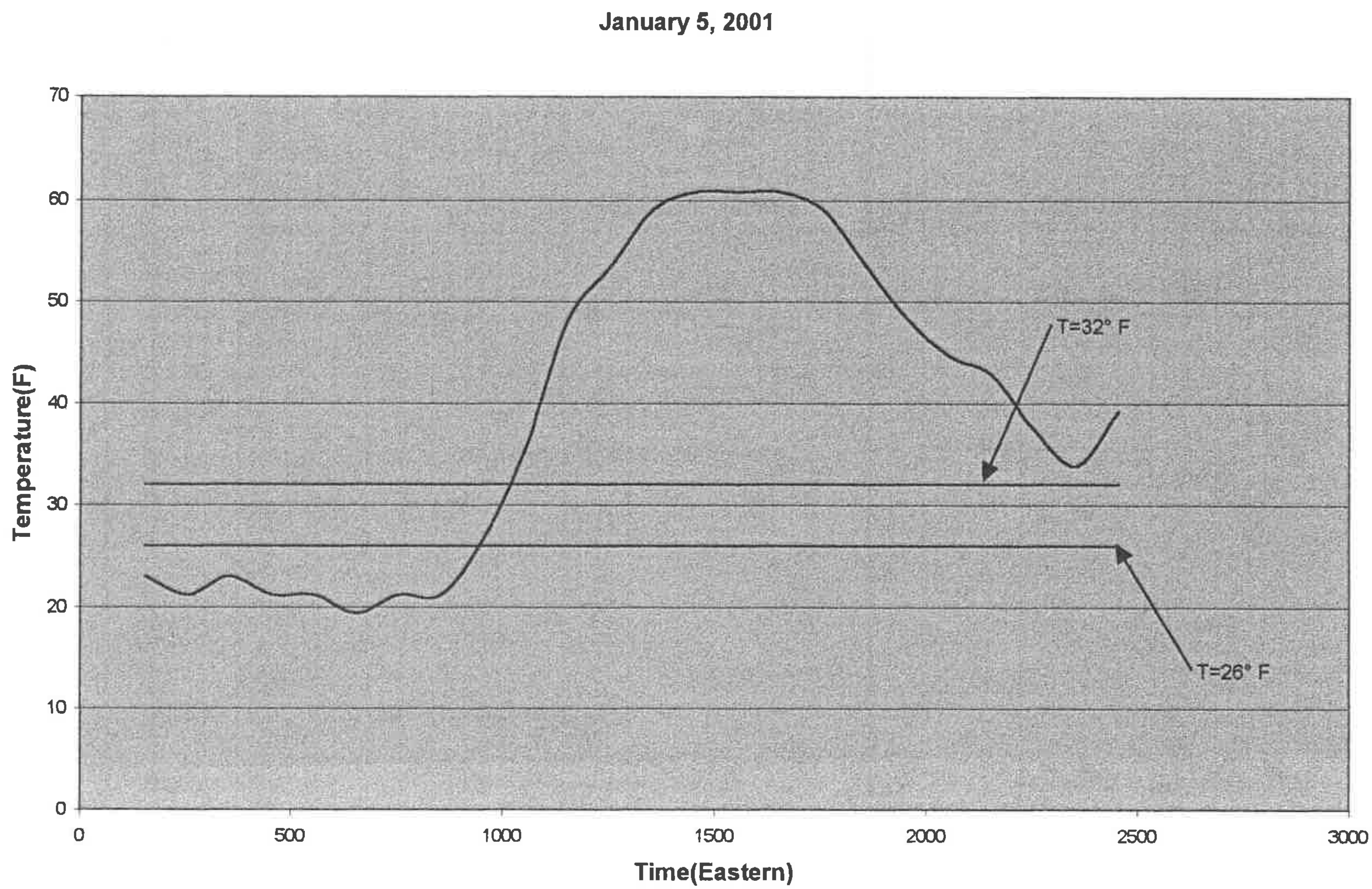


January 3, 2001

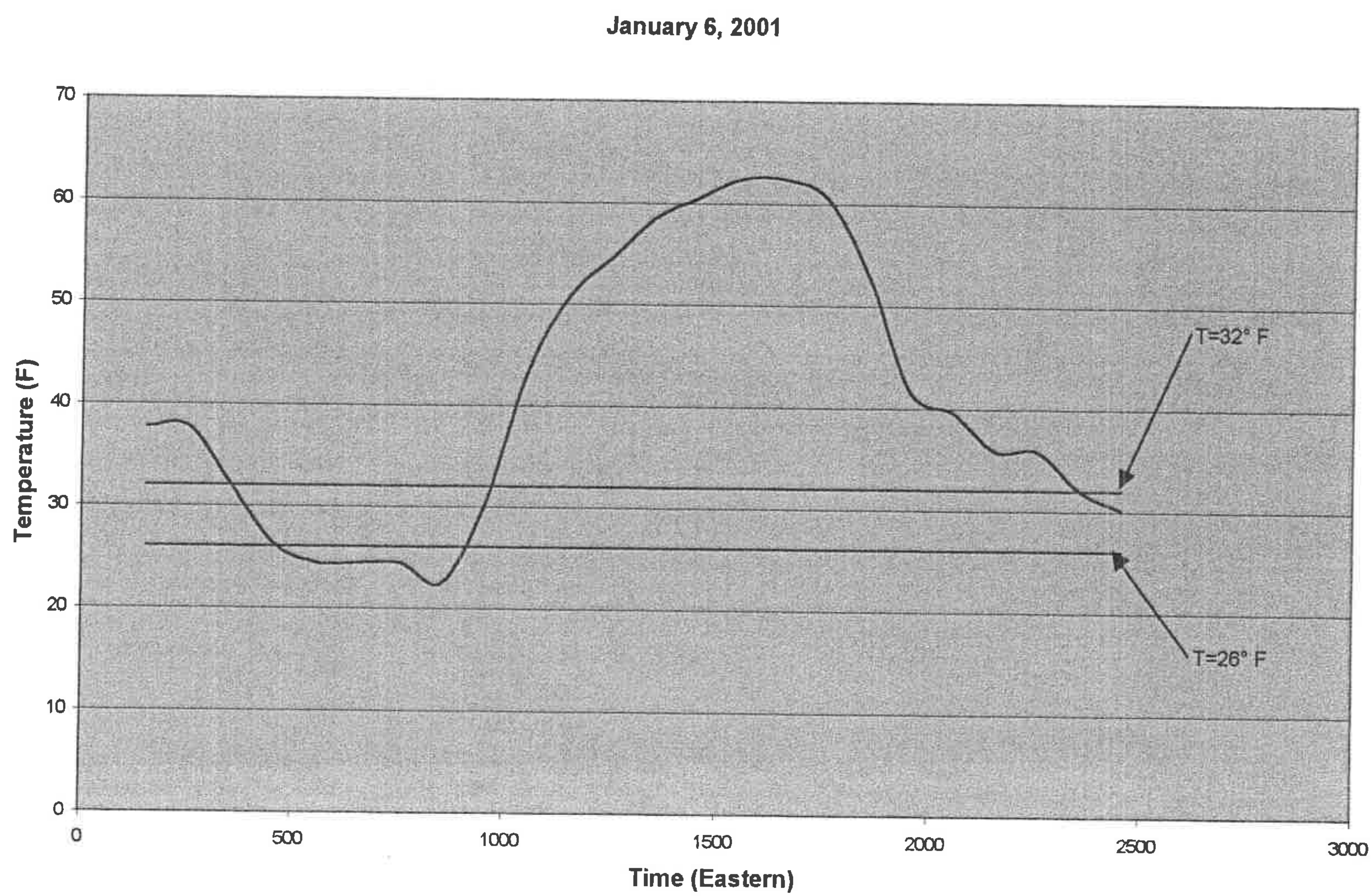


January 4, 2001

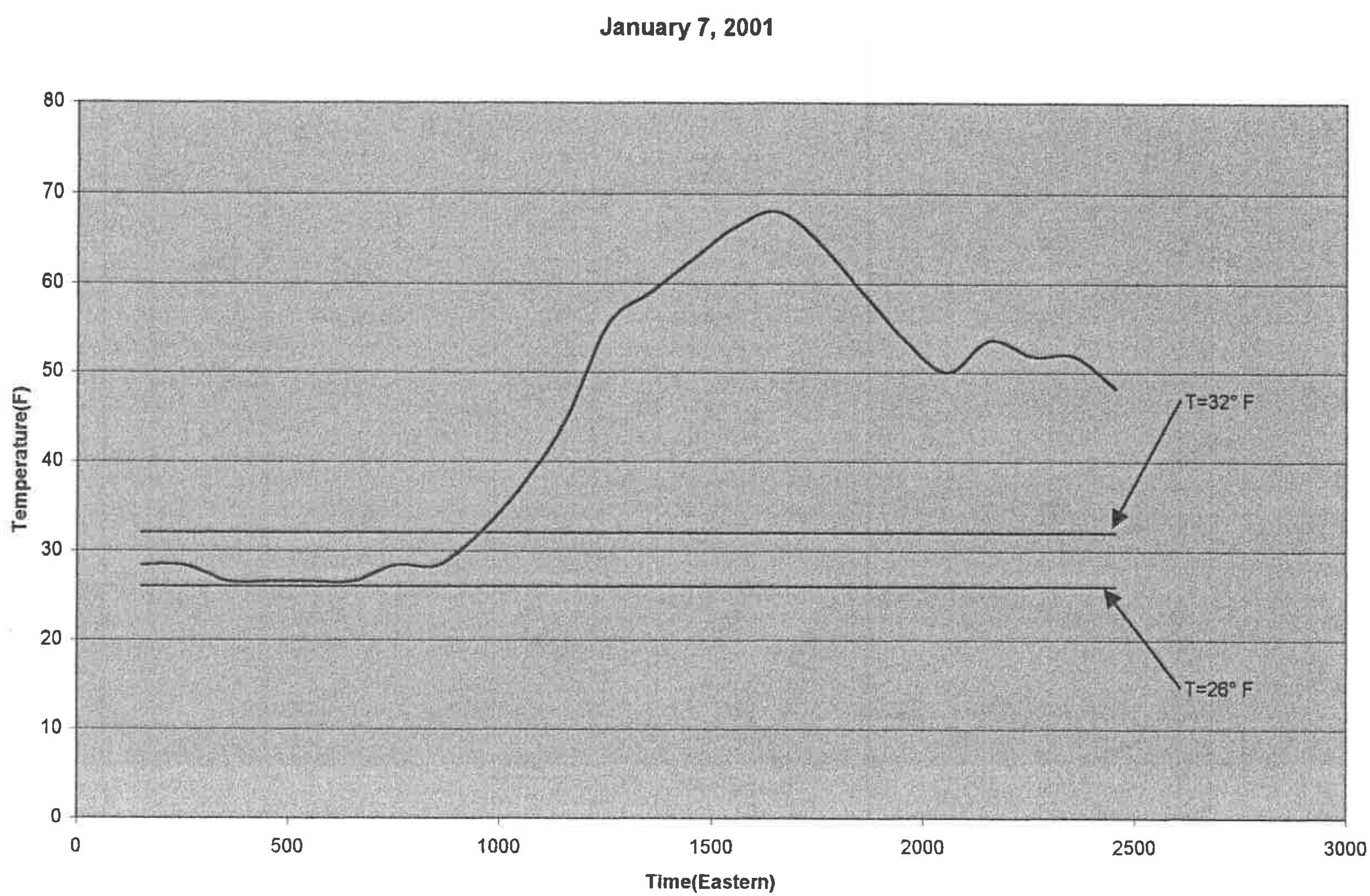


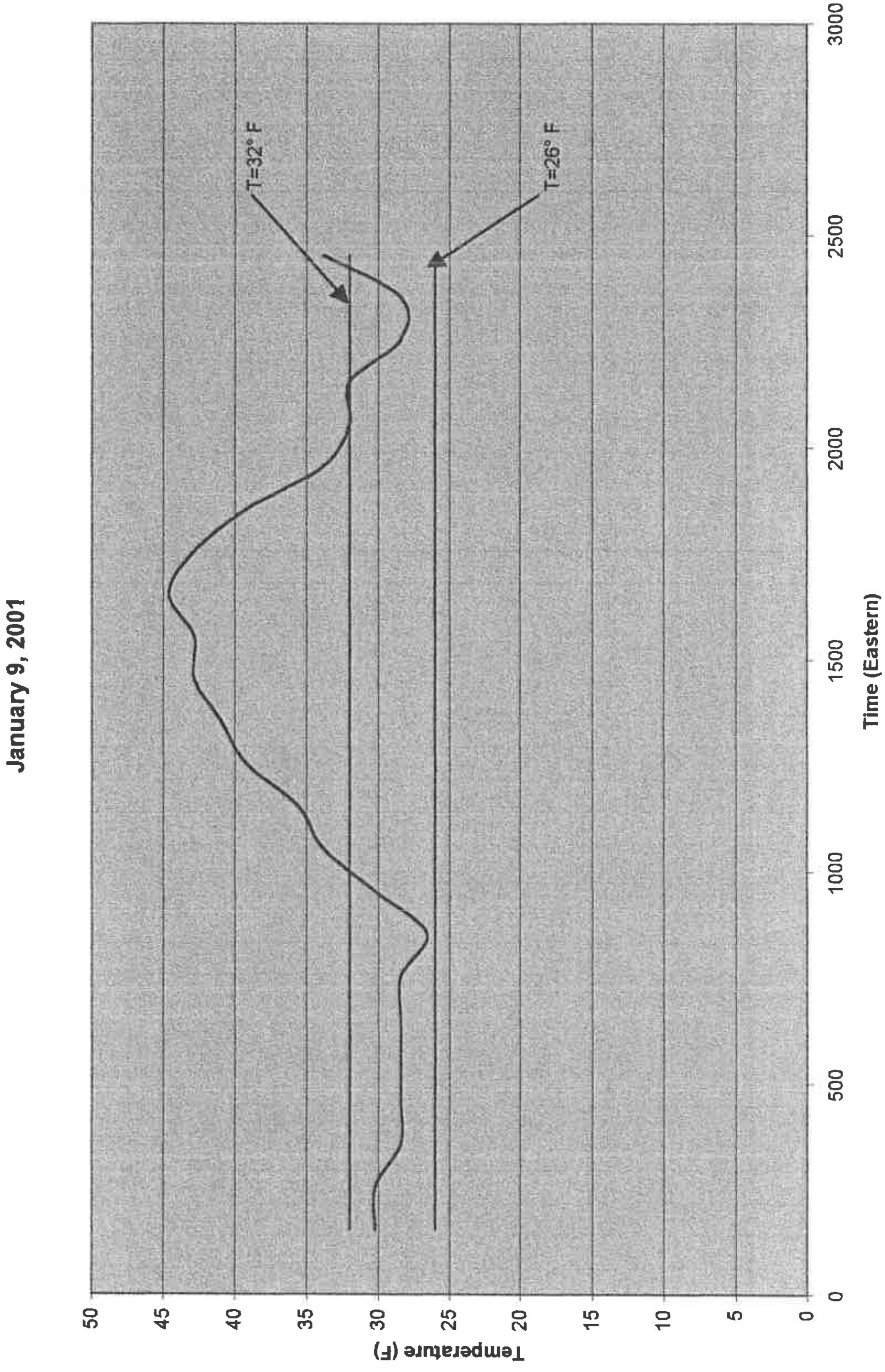


January 5, 2001
Temperature(F)
70
60
50
40
30
20
10
0
0
500
1000
1500
2000
2500
3000
Time(Eastern)
T=32° F
T=26° F

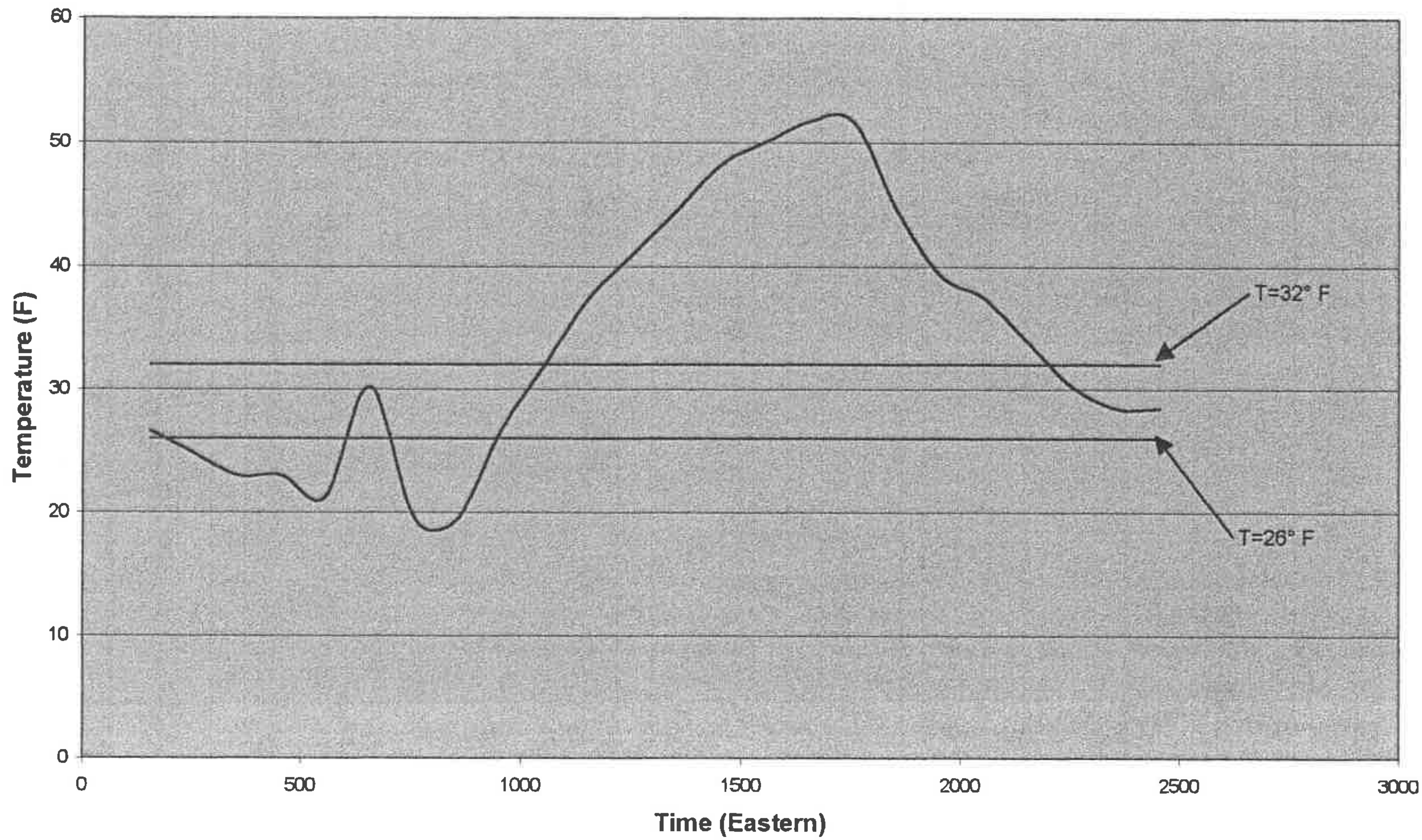


January 6, 2001
Temperature (F)
70
60
50
40
30
20
10
0
500
1000
1500
2000
2500
3000
Time (Eastern)
T=32° F
T=26° F

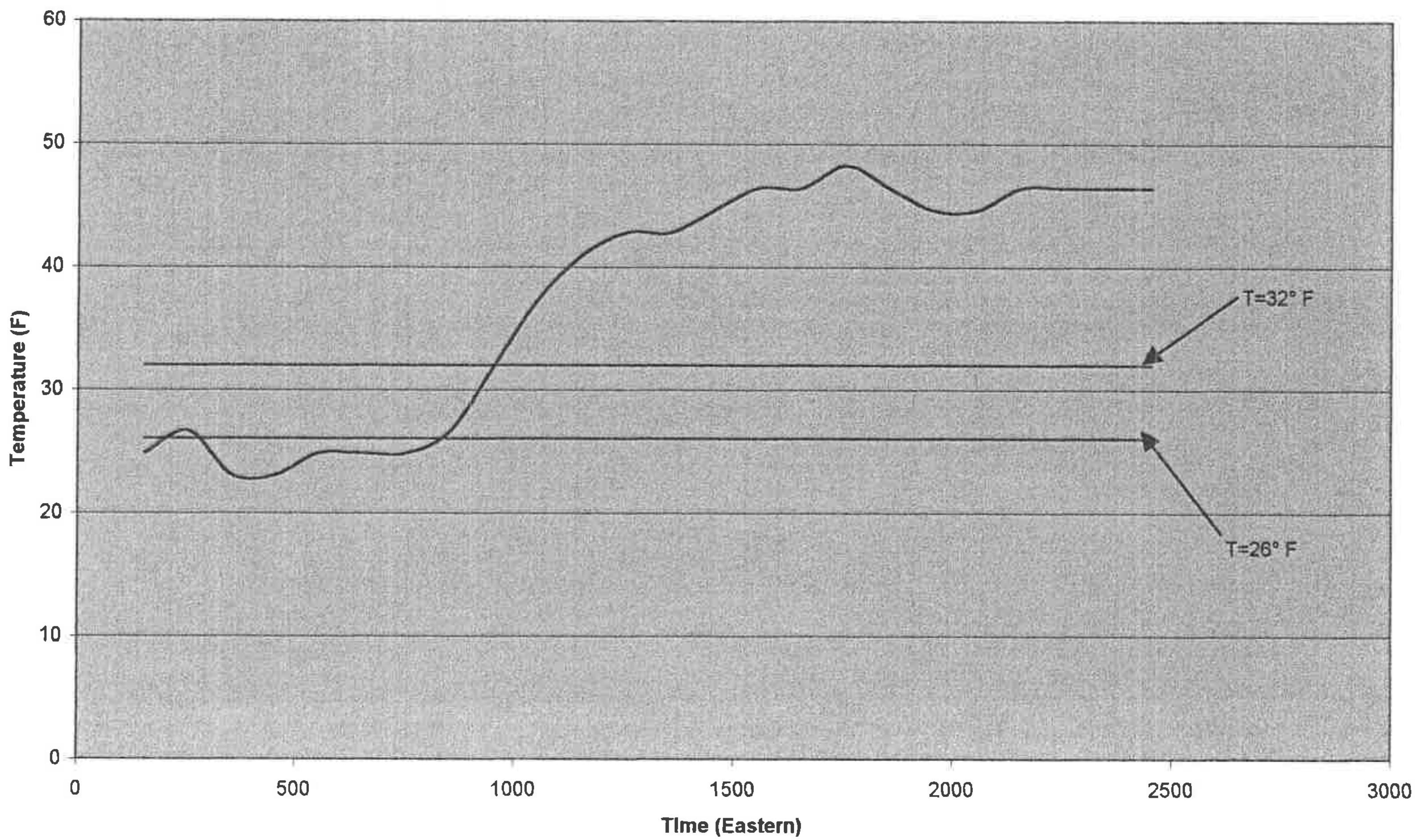

January 7, 2001
Temperature(F)
80
70
60
50
40
30
20
10
0
0
500
1000
1500
2000
2500
3000
Time(Eastern)
T=32° F
T=26° F


January 9, 2001
T=32° F
T=26° F
Temperature (F)
Time (Eastern)
0
5
10
15
20
25
30
35
40
45
50
0
500
1000
1500
2000
2500
3000

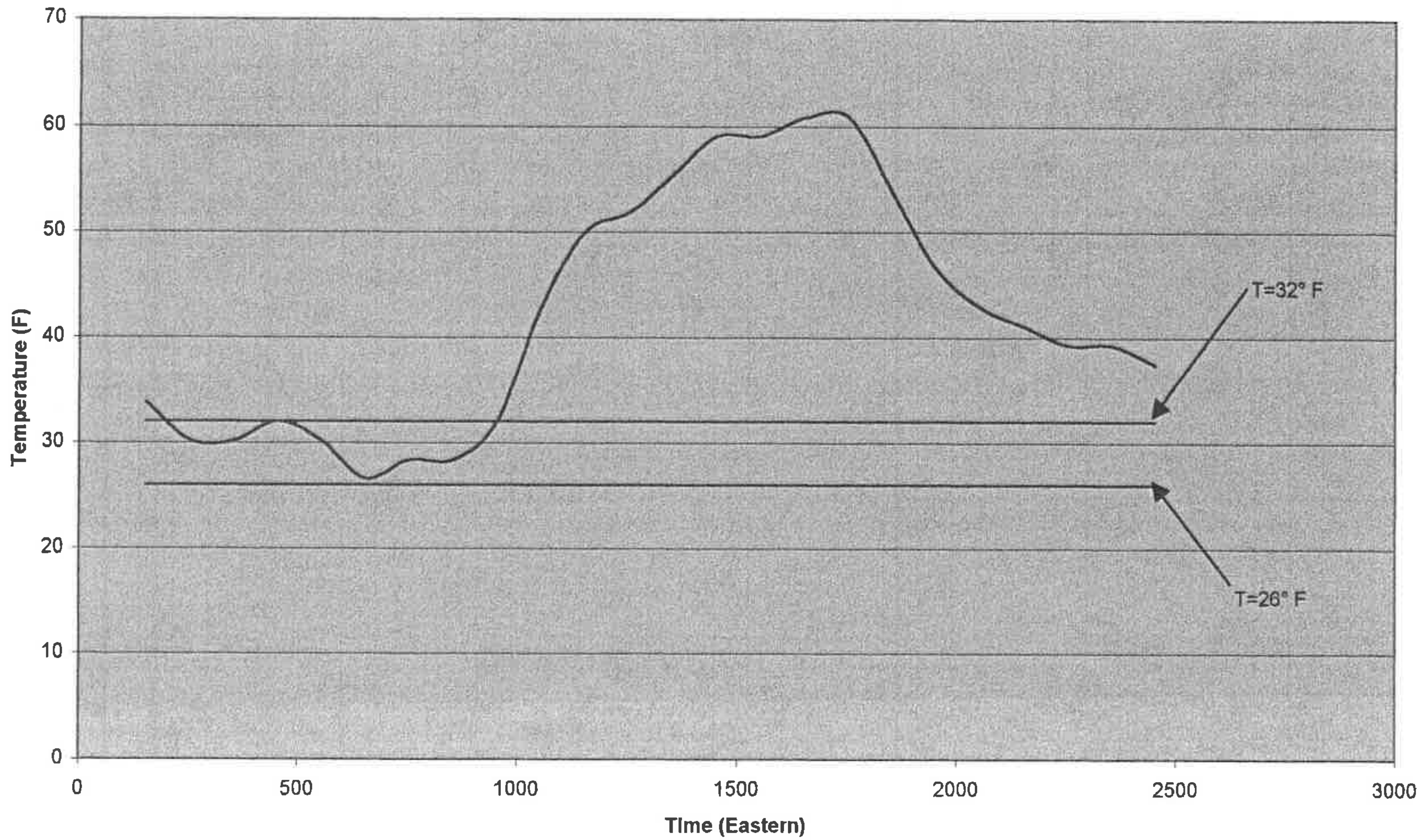
January 10, 2001



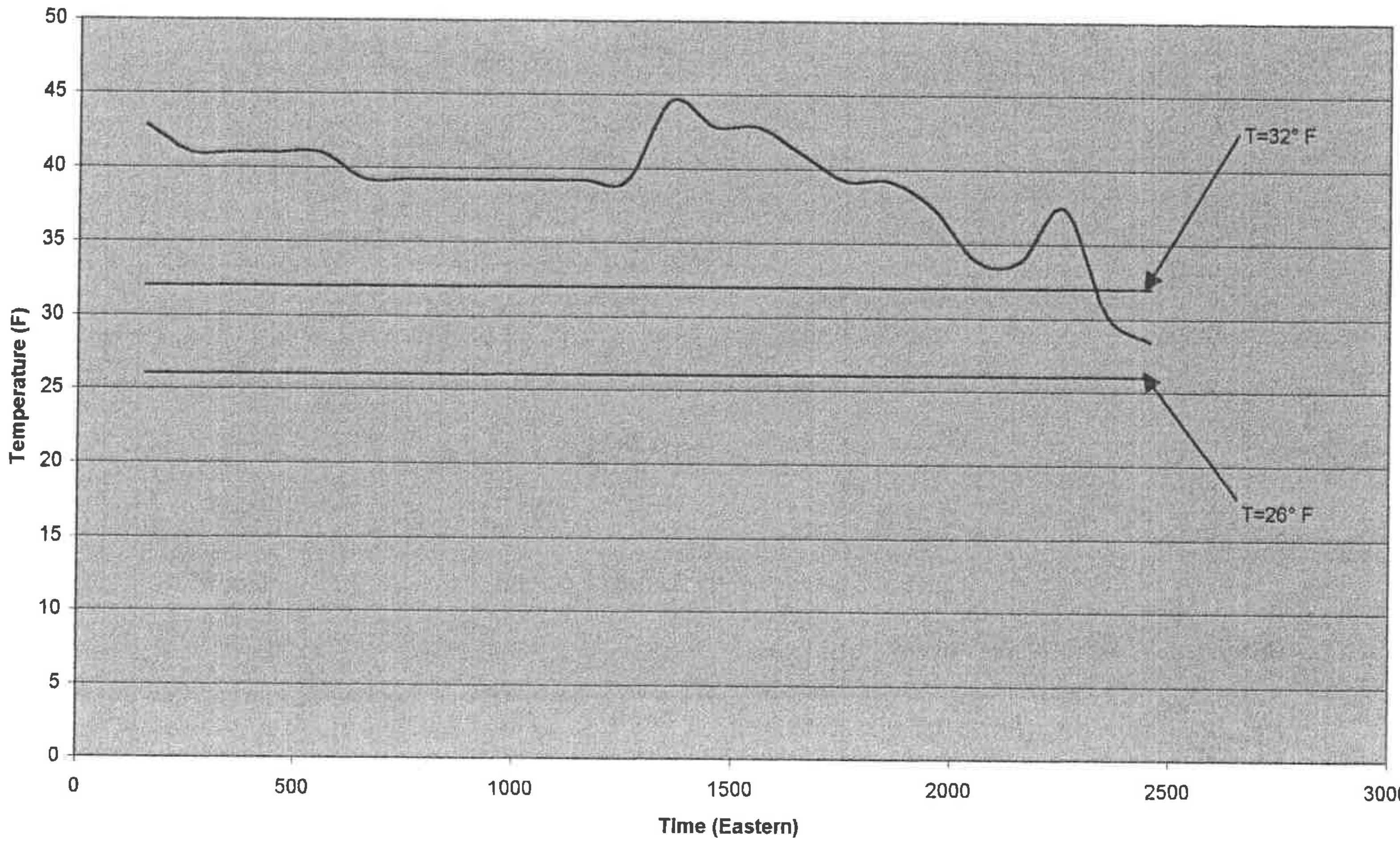
January 11, 2001



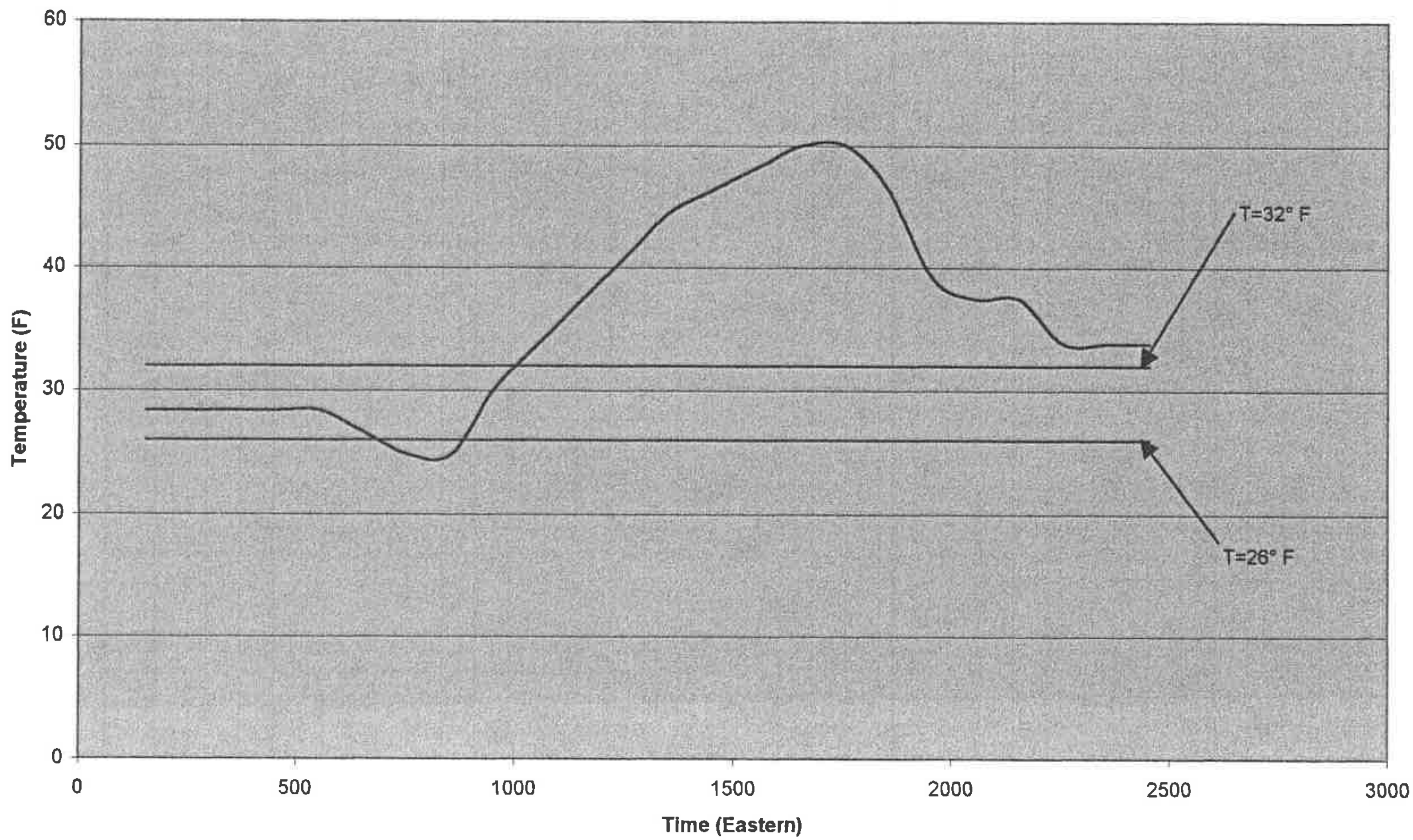
January 13, 2001



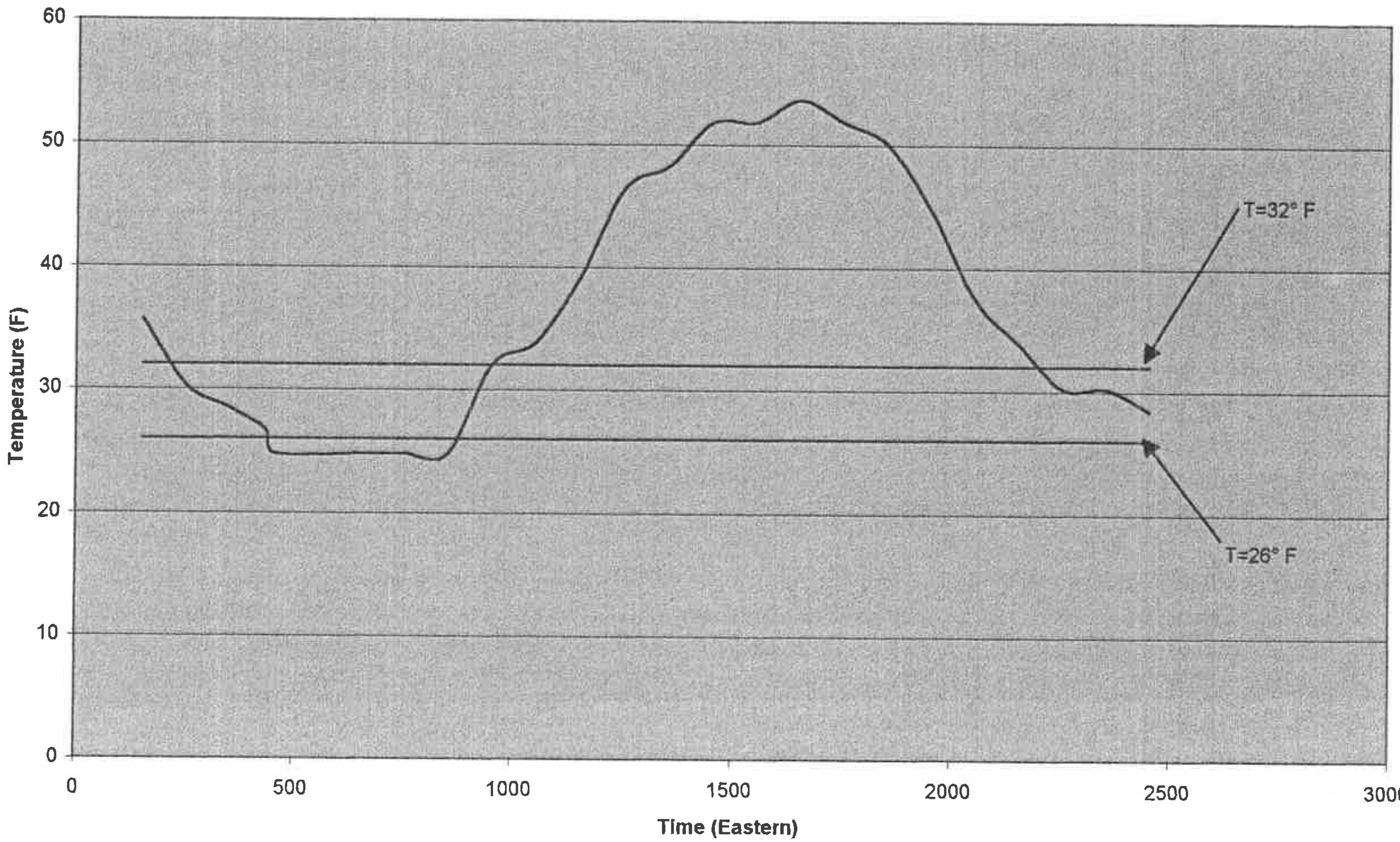
January 20, 2001



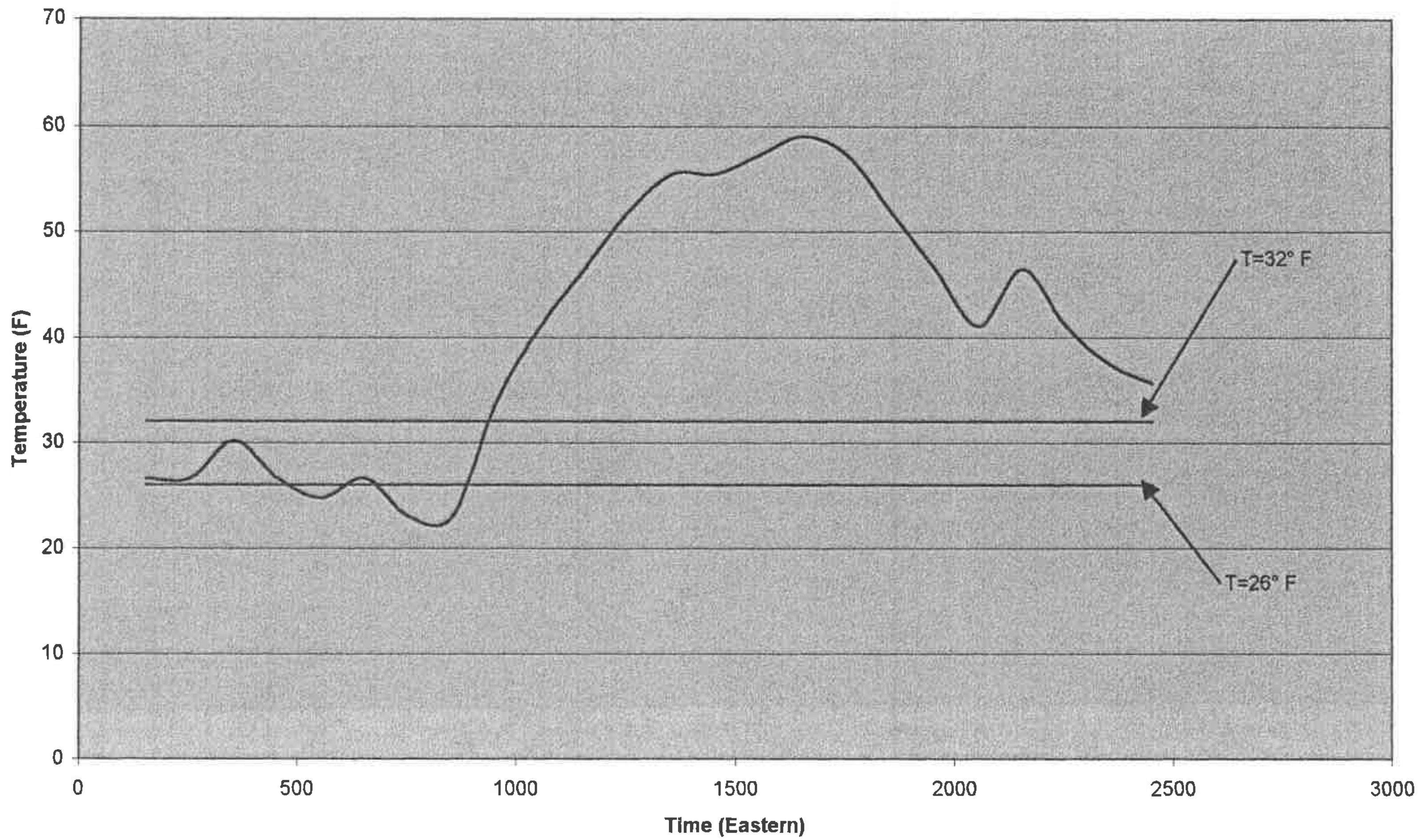
January 21, 2001



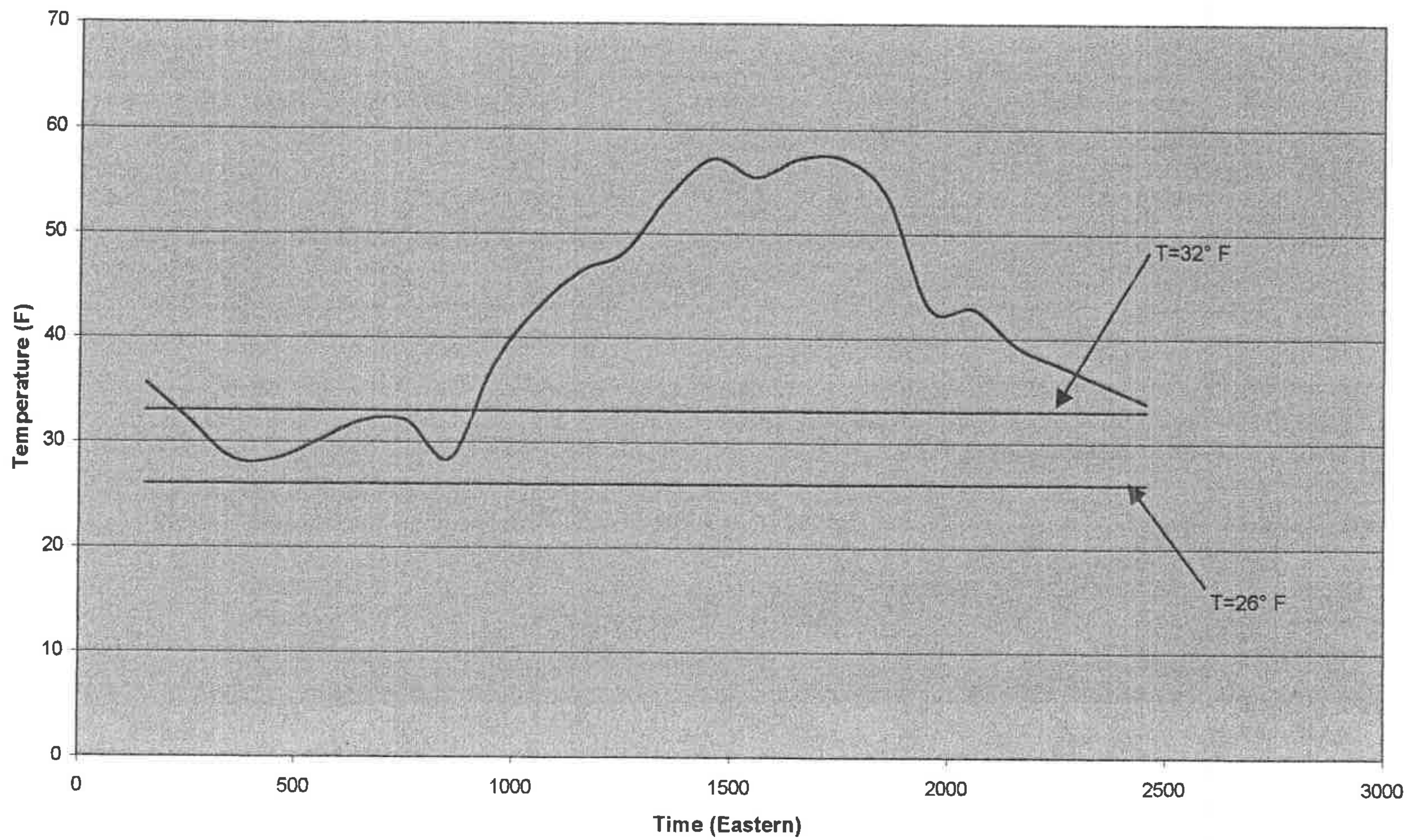
January 22, 2001



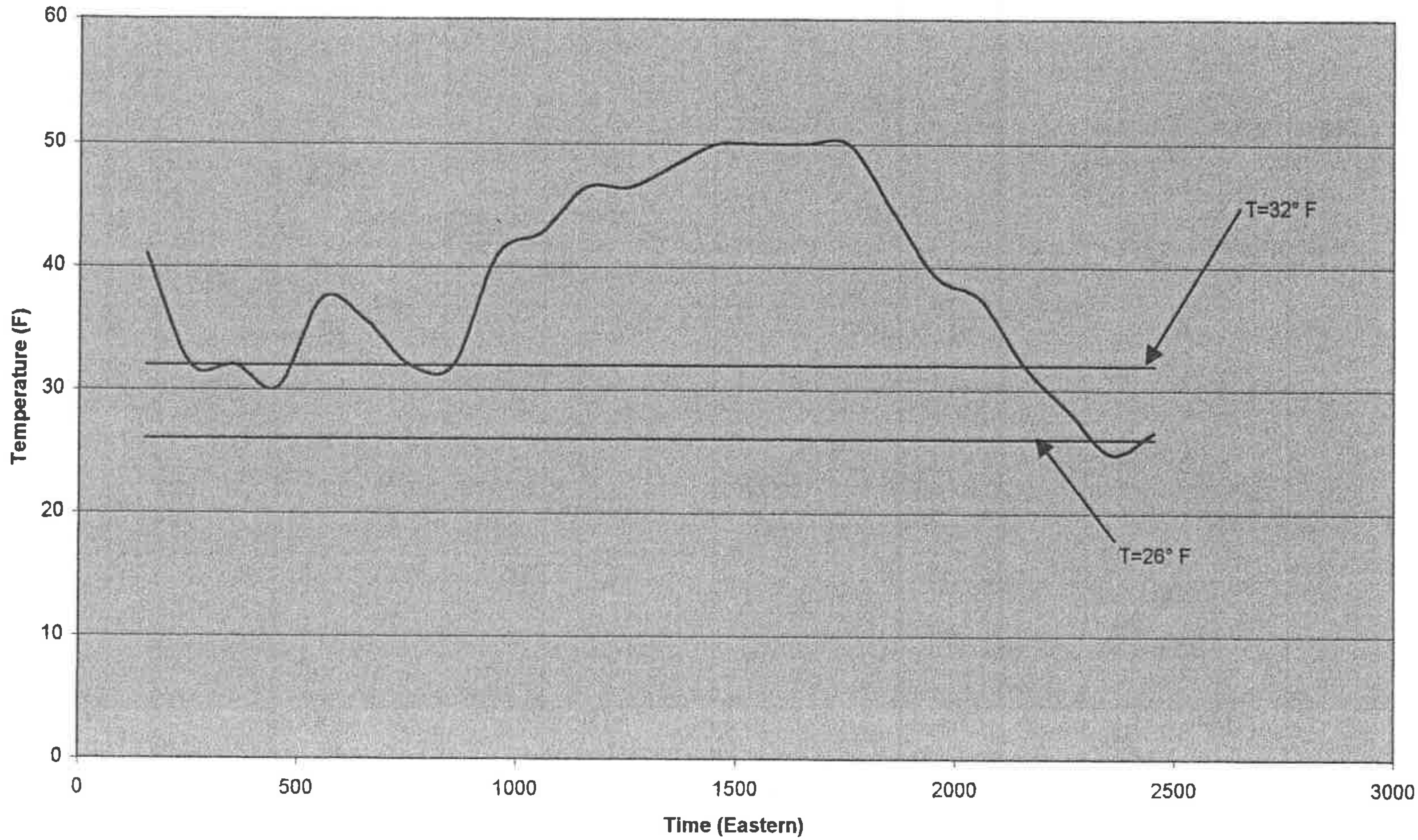
January 23, 2001

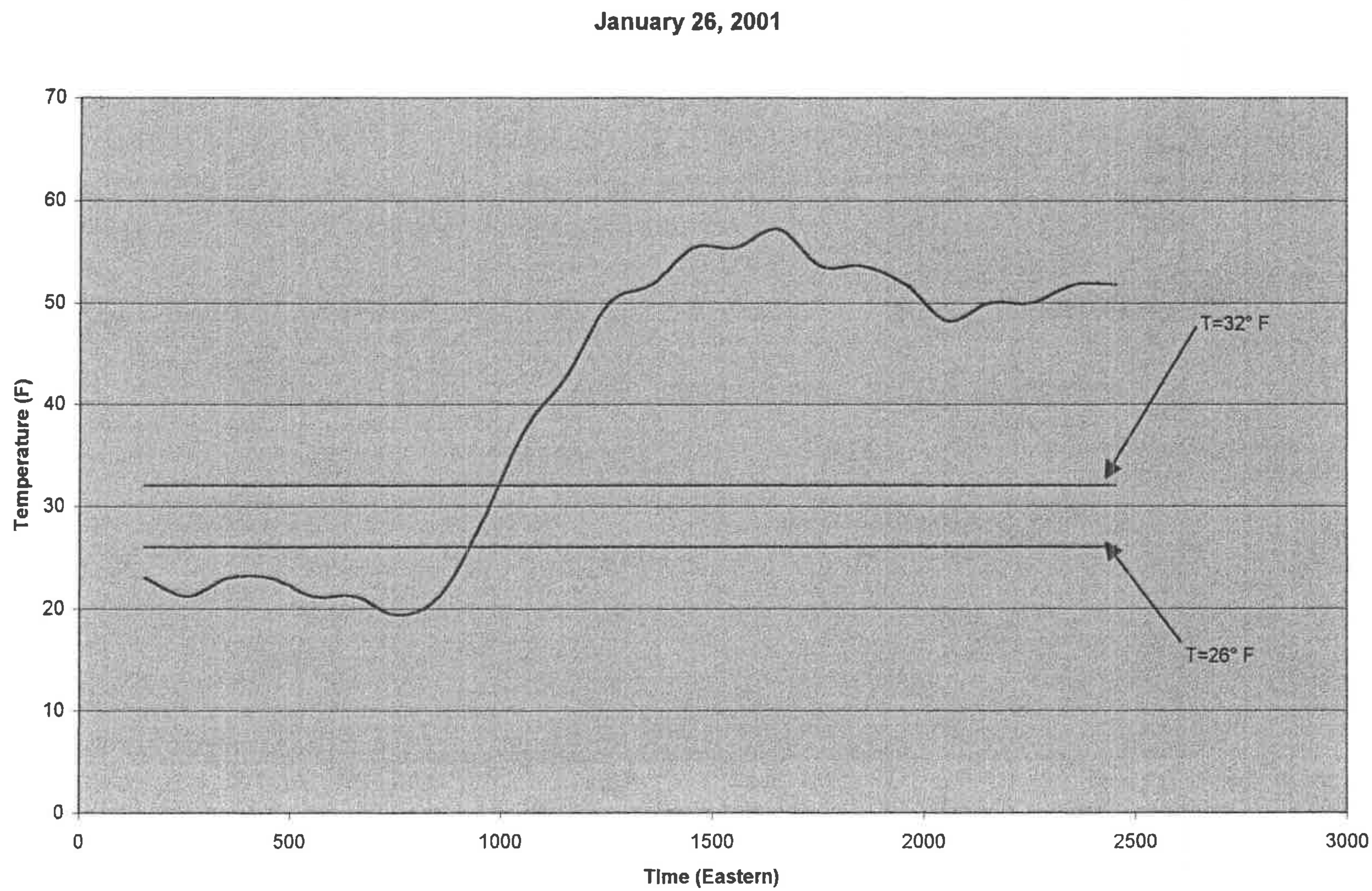


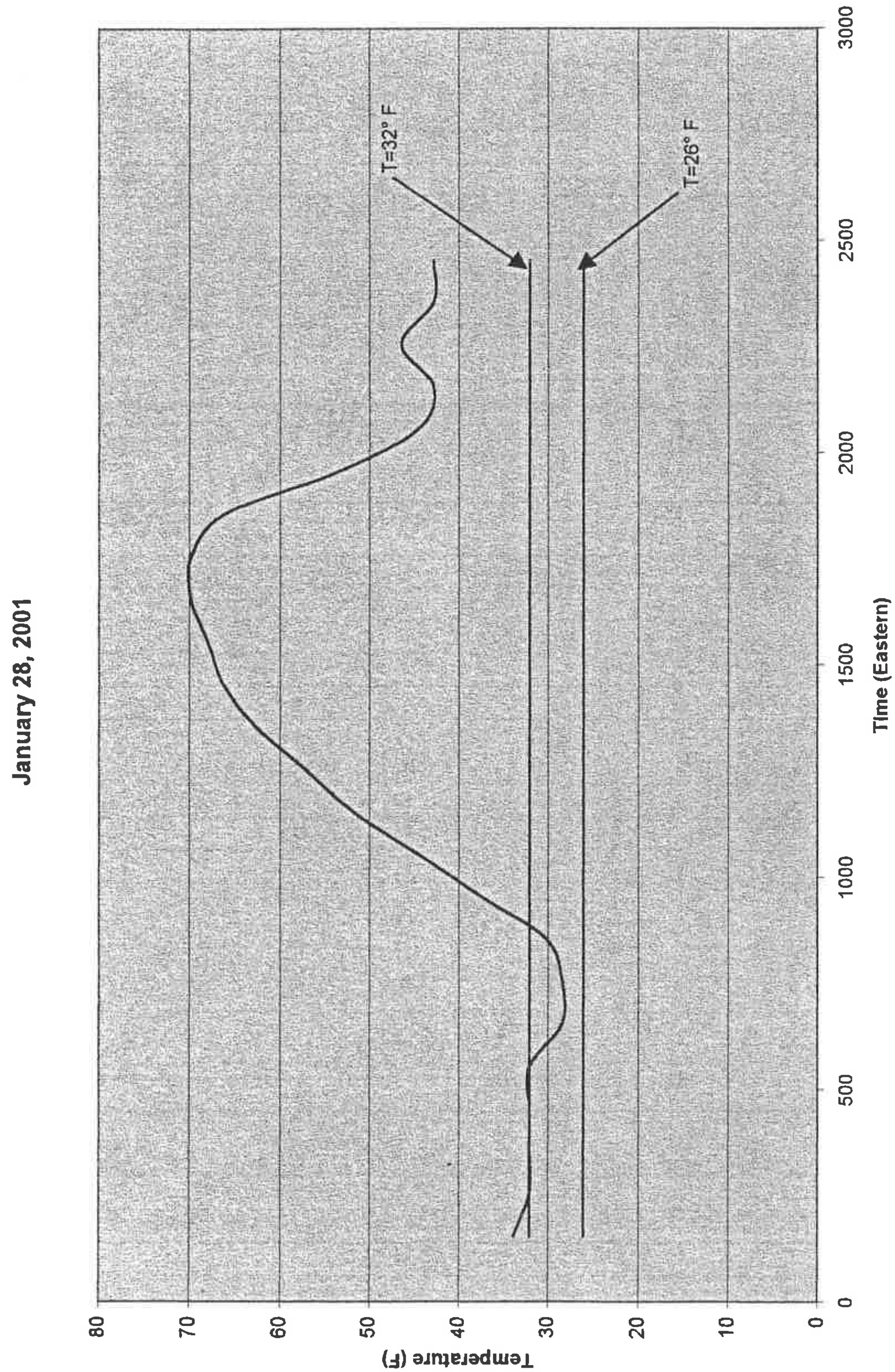
January 24, 2001



January 25, 2001




January 26, 2001
Temperature (F)
70
60
50
40
30
20
10
0
0
500
1000
1500
2000
2500
3000
Time (Eastern)
T=32° F
T=26° F


January 28, 2001
Temperature (F)
Time (Eastern)
T=32° F
T=26° F
0
10
20
30
40
50
60
70
80
0
500
1000
1500
2000
2500
3000