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16. Abstract

The durability of concrete containing fly ash subjected to various curing conditions was investigated in this research program. One Type A and two Type B fly ashes were used at 0, 25, and 35 percent replacement of cement by volume. Test specimens were cured at 50, 73, and 100°F and 50 and 100 percent relative humidity. The two durability tests performed were the abrasion resistance test and the resistance to deicing scaling test.

The abrasion test was performed at 14 days test age according to ASTM C944. Depth of penetration was used to evaluate the abrasion resistance of concrete. The scaling resistance test was performed at 28 days test age according to ASTM C672. The deicing scaling damage was assessed by visual inspection at various stages during the test. At the completion of the scaling test, the chloride concentration of the concrete was determined at various depths from the exposed concrete surface.

The test results show that strength is the most important factor influencing the abrasion resistance of concrete. The curing practices were found to influence the abrasion resistance of the concrete in that they affected the concrete strength.

No relationship could be established between the deicer scaling resistance of concrete and the water cementitious ratio, the compressive strength, or the curing practices. Moist-cured plain concrete, however, was found to be, in most cases, the most resistant to deicer scaling. The chloride penetration test results revealed that moist-cured concrete is much more resistant to chloride penetration than similar air-dried concrete.

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ABRASION RESISTANCE AND SCALING RESISTANCE OF CONCRETE CONTAINING FLY ASH

by

Karim M. Hadchiti
and
Ramon L. Carrasquillo

Research Report Number 481-3

Research Project 3-5/9-87-481

"Durability and Performance of Concrete Containing Fly Ash"

Conducted for

Texas
State Department of Highways and Public Transportation

In Cooperation with the
U.S. Department of Transportation
Federal Highway Administration

by

CENTER FOR TRANSPORTATION RESEARCH
BUREAU OF ENGINEERING RESEARCH
THE UNIVERSITY OF TEXAS AT AUSTIN

August 1988

The contents of this report reflect the views of the authors who are responsible for the facts and accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Federal Highway Administration. This reports does not constitute a standard, specification, or regulation.

PREFACE

This is the third in a series of reports summarizing the durability and performance of concrete containing fly ash. The first report addressed the sulfate resistance of concrete containing fly ash. The second report addressed the effect of fly ash on the temperature rise in concrete. This report addresses the abrasion resistance and scaling resistance of concrete containing fly ash when submitted to various curing conditions. Other reports in the series will address freeze-thaw durability, creep and shrinkage at early ages, and fly ash characterization.

This is part of Research Project 3-5/9-87-481, entitled "Durability and Performance of Concrete Containing Fly Ash." The study described in this report was jointly conducted by the Center for Transportation Research, Bureau of Engineering Research and the Phil M. Ferguson Structural Engineering Laboratory at the University of Texas at Austin. The work was co-sponsored by the Texas State Department of Highways and Public Transportation and The Federal Highway Administration.

The overall study was directed and supervised by Dr. Ramon L. Carrasquillo.

ABSTRACT

The durability of concrete containing fly ash subjected to various curing conditions was investigated in this research program. One Type A and two Type B fly ashes were used at 0, 25, and 35 percent replacement of cement by volume. Test specimens were cured at 50, 73, and 100°F and 50 and 100 percent relative humidity. The two durability tests performed were the abrasion resistance test and the resistance to deicing scaling test.

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The test results show that strength is the most important factor influencing the abrasion resistance of concrete. The curing practices were found to influence the abrasion resistance of the concrete in that they affected the concrete strength.

No relationship could be established between the deicer scaling resistance of concrete and the water cementitious ratio, the compressive strength, or the curing practices. Moist-cured plain concrete, however, was found to be, in most cases, the most resistant to deicer scaling. The chloride penetration test results revealed that moist-cured concrete is much more resistant to chloride penetration than similar air-dried concrete.

IMPLEMENTATION

This report summarizes the findings of the abrasion resistance tests and scaling resistance tests performed on concrete containing fly ash. Factors such as the fly ash content, type of fly ash used, and the curing practices were investigated.

The results from this study provide specific recommendations for the resident engineer in order to ensure adequate durability of concrete containing fly ash.

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CHAPTER 1
INTRODUCTION

The use of fly ash as a partial replacement of cement in concrete production has been on the rise for the last two decades. The primary reasons behind this are the economic advantages that concrete containing fly ash offers over conventional concrete and the need for a stronger and more durable concrete in various construction applications.

1.1 Research Significance

The production of concrete containing fly ash in Texas has recently gained widespread applications especially in highway construction. As of January 1986, the Texas State Department of Highways and Public Transportation has included fly ash in its concrete specifications. Previous work done in Texas on concrete containing fly ash has touched on most of the durability aspects and has laid much of the ground work for future research. Due to the lack of in-depth knowledge in this field, the implementation of a proposed study of the performance and long term durability of fly ash concrete was urgently needed.

1.2 Definition and Classification of Fly Ash

1.2.1 Definition of Fly Ash. Fly ash is a by-product of the coal burning process in power generation plants. It consists of a very fine material which mixes with flue gases and escapes the combustion chamber. The fly ash particles are then collected by means of mechanical or electrostatic precipitators before the gases are released into the atmosphere. When the coal particles are in the combustion chamber, they are exposed to very high temperatures for a brief period of time followed by rapid cooling, causing the molten particles to undergo complex reactions which form the fly ash particles. The rapid heating and cooling process of the coal particles causes the fly ash particles to condensate into a spherical shape.

Fly ash consists primarily of the oxides of calcium, silica, aluminum, iron, and sulfur trioxide. Traces of magnesium, sodium, potassium, and phosphorus can be found in various amounts depending on the nature and origin of the coal.[28]

1.2.2 Fly Ash Classification. The factors which influence the most the physical and chemical properties of fly ash are found in the composition of the coal, the degree of pulverization, the design of the combustion unit, and the methods of collecting and processing the ash.[1,41] The most important factor, however, is the composition and sources of the coal.[9] The most common types of coal used are: a) anthracite and bituminous coals, (also referred to as eastern coals) mainly found in the eastern and north central states, and b) sub-bituminous and lignite coals (also known as western coals), generally found in the western and southwestern states. The eastern coals are cleaner and have a higher carbon content than the western coals; their energy potential and burning efficiency is therefore higher.[31]

The American Society for Testing and Materials (ASTM) and the Texas State Department of Highways and Public Transportation (TSDHPT) divide fly ash into two groups based on the source of the coal and on the chemical and physical characteristics, respectively. ASTM Class F fly ash and Texas Type A fly ash typically originate from the burning of anthracite or bituminous coals, whereas ASTM Class C fly ash and Texas Type B fly ash originate from the burning of sub-bituminous and lignite coals. Tables 1.1 through 1.4 list the chemical and physical requirements of fly ash according to ASTM C618-85 and Texas SDHPT D-9- 8900.[4,5] ASTM Class F and TSDHPT Type A fly ashes have a low calcium oxide content and exhibit only pozzolanic properties, whereas ASTM Class C and TSDHPT Type B fly ashes have a high calcium oxide content and exhibit both cementitious and pozzolanic properties. The pozzolanic activity is controlled by the amount of fly ash used, the particle size, the reactivity of the silica, and the availability of calcium hydroxide. The long term effect of the pozzolanic reaction is an increase in the proportion of calcium silicate hydrate (C-S-H) in the hydrated paste matrix at the expense of calcium hydroxide liberated during the hydration of cement.[32]

1.2.3 Physical Properties of Fly Ash. The physical properties of fly ash greatly influence the properties of fresh concrete.

The size and shape of the fly ash particles depend primarily on the burning efficiency of the plant and the type of collection system used. Electrostatic precipitators are more efficient than mechanical precipitators, resulting in the collection of a greater percentage of fine fly ash particles.

The fineness of fly ash is also important because it affects the pozzolanic activity. There is a general agreement that finer ashes have a higher pozzolanic activity than coarser fly ashes.

The density of fly ash depends on the composition of the ash. High density ashes are rich in iron whereas low density ashes have high alumina, silica, and carbon contents. Ashes with a high carbon content are usually grey or black while ashes with a high iron content are tan-colored.[24,16]

1.2.4 Chemical Properties of Fly Ash. The chemical properties of fly ash, such as sulfate, influence the performance of hardened concrete mainly in strength and durability such as sulfate resistance.

The total oxides include the sum of silica, alumina, and iron oxide (SiO_2 + Al_2O_3 + Fe_2O_3). These compounds react with hydrated lime released during the hydration of cement to form a secondary gel rich in calcium silicate hydrate. This reaction, also known as pozzolanic activity, contributes primarily to the long term compressive strength gain in the concrete.[9,16,24]

The sulfur trioxide (SO_3) content is limited to a maximum of five percent in order to avoid any disruptive sulfate reaction from occurring in the hardened concrete.[16] This will also reduce the risks of delayed setting time due to any excess SO_3.

The restriction placed on the magnesium oxide (MgO) content limits or prevents any expansion caused by the formation of magnesium hydroxide which, in some cases, may prove to have deleterious effects on the concrete.

The alkali content in the fly ash is limited to a maximum of 1.5 percent. Beyond this limit, tests have shown that alkali-aggregate reaction is likely to occur and cause undesirable expansions in the hardened concrete.[24] It has also been found that if proper percent replacement is used, any expansion due to alkali-aggregate reactions is reduced.[12]

Table 1.1 ASTM C618-85 Chemical Requirements of Fly Ash[4]

	Class F	Class C
Silicon dioxide (SiO_2 plus aluminum oxide (Al_2O_3) plus iron oxide (FeO_3), min., %	70.0	50.0
Sulfur trioxide (SO_3), max., %	5.0	5.0
Available Alkalies, as Na_2O, max., %	1.5	1.5
Moisture content, max., %	3.0	3.0
Loss on ignition, max., %	6.0	6.0

Table 1.2 ASTM C618-85 Physical Requirements of Fly Ash[4]

	Class F	Class C
Fineness, amount retained on 325 sieve, max., %	34.0	34.0
Pozzolanic activity index with portland cement, at 28 days, min., % of control	75.0	75.0
Water requirement, max., % of control	105.0	105.0
Soundness, autoclave expansion or contraction, max., %	0.8	0.8
Increase of drying shrinkage of mortar bars at 28 days, max., %	0.03	0.03
Reactivity with cement alkalies, mortar expansion at 14 days, max., %	0.02	0.02

Table 1.3 Texas SDHPT Departmental Material Specification D-9-8900, Physical Requirements[5]

	Type A	Type B
Fineness - retained on 325 sieve (45 cm), max., %	30.0	30.0
Variation in percentage points retained on 325 sieve from the average of the last 10 samples (or less provided 10 have not been tested) shall not exceed	5.0	5.0
Pozzolanic activity index with portland cement as a min. % of control at 28 days	75	75
Water requirement, max. % of control	100	100
Soundness, autoclave expansion or contraction, max., %	0.8	0.8
Increase of drying shrinkage of mortar bars at 28 days, max., %	0.03	0.03
Reactivity with cement alkalies mortar expansion at 14 days, max., %	0.020	0.020
Specific gravity, max. variation from average %	5.0	5.0

Drying shrinkage shall be tested in accordance with ASTM C157.
Alkali reactivity shall be tested in accordance with ASTM C441.
Specific gravity shall be tested in accordance with ASTM C188. All other physical requirements shall be tested in accordance with ASTM C311.

Table 1.4 Texas SDHPT Departmental Material Specification D-9-8900, Chemical Requirements[5]

	Type A	Type B
Silicon dioxide (SiO_2) plus aluminum oxide (Al_2O_3) plus iron oxide (FeO_3), min., %	65.0	50.0
Sulfur trioxide (SO_3), max., %	5.0	5.0
Calcium oxide (CaO), variation in percentage points of CaO from the average of the last 10 samples (or less provided 10 have not been tested) shall not exceed plus or minus	4.0	4.0
Magnesium oxide (MgO), max., %	5.0*	5.0*
Available alkalies, as Na_2O, max., % (when used in conjunction with reactive or potentially reactive aggregates)	1.5	1.5
Moisture content, max., %	2.0	2.0
Loss on ignition, max., %	3.0	3.0

* When the autoclave expansion or contraction limit is not exceeded, an MgO content above 5.0% may be acceptable.

The loss on ignition (LOI), or carbon content of the fly ash, primarily affects the required dosage of air entraining admixture. Carbon is known to have a strong affinity for organic chemical admixtures. The higher the carbon content in the fly ash, the higher the dosage of the air entraining admixture needed to entrain a given percentage of air. It is therefore desirable to use a fly ash with as low a loss on ignition as possible.

1.2.5 Advantages and Disadvantages of Concrete Containing Fly Ash. Significant improvements in both fresh and hardened concrete properties can be achieved from the use of fly ash. The factors influencing the fresh concrete properties the most are the spherical shape and fineness of the fly ash particles. Hardened concrete properties are mostly affected by the pozzolanic properties and the chemical composition of the fly ash. Some of the possible advantages of using fly ash in concrete are:[1,5,16,23,31,32,41]

(a) improved workability,

(b) reduced bleeding,

(c) reduced segregation,

(d) reduced heat of hydration,

(e) reduced drying shrinkage,

(f) increased resistance to sulfates,

(g) reduced permeability,

(h) increased ultimate tensile and compressive strength,

(i) reduced alkali reactivity, and

(j) reduced cost.

Some of the possible disadvantages resulting from the use of fly ash in concrete include:[16,31,41]

(a) lower early strength,

(b) delayed removal of formwork due to slower strength gain,

(c) increased dosage of organic air entraining admixture, and

(d) reduced sulfate resistance of concrete.

1.3 Objective of the Research

The primary objective of this research is to study the long term durability of concrete containing fly ash. The results of the study will help modify existing guidelines and develop new ones to ensure the proper selection and use of fly ash in concrete. The durability aspects described herein are the abrasion resistance and scaling resistance of concrete containing fly ash subjected to various curing conditions. Other durability aspects of concrete containing fly ash including freeze-thaw resistance, sulfate resistance, and alkali- aggregate reaction are being investigated under Research Studies 481 and 450.

1.4 Research Plan

The study includes the selection of two Texas Type B and one Texas Type A fly ashes to be used in the concrete mixes. A total of 21 batches were cast containing 0, 25, and 35 percent replacement of cement with fly ash by volume. The mix proportions of the trial batches and actual mixes were done in accordance to Construction Bulletin C-11 of the Texas Highway Department Construction Division. The mixes were also designed to meet the strength specifications for continuously reinforced concrete pavements according to the Texas SDHPT 1982 Standard Specifications for Construction of Highways, Streets, and Bridges. Curing of the specimens was performed at 50, 73, and 100°F temperatures with 50 and 100 percent relative humidity for each of the respective curing temperatures. Two durability aspects of concrete are the focus of the part of the study reported herein. The abrasion resistance test was conducted according to ASTM C944-80 *Standard Test Method for Abrasion Resistance of Concrete or Mortar by the Rotating Cutter Method.* The scaling resistance was determined according to ASTM C672-84 *Standard Test Method for Scaling Resistance of Concrete Surfaces Exposed to Deicing Chemicals.* For each of the two durability tests performed, companion cylinders were tested in compression at the beginning of each test according to ASTM C39-84 *Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens.*

1.5 Format

This report is divided into six chapters. A review of the literature and research done relevant to this study is discussed in Chapter 2. The test procedures and materials used throughout the experimental program are presented in Chapter 3. The test results are presented in Chapter 4 and discussed in Chapter 5. Chapter 6 contains the summary, conclusions, and recommendations.

CHAPTER 2
LITERATURE REVIEW

2.1 Introduction

Although the use of fly ash in construction has increased significantly in the last 10 to 15 years, the earliest works on record involving fly ash concrete date back to the 1930's. In 1936 a seawall at the State Line Plant in Chicago was constructed using fly ash.[1] The wall is still serving its purposes to this date and appears to be in fair condition. In 1938, fly ash was also used to cast pavements for the Northside Sewage Treatment Plant in Chicago.[30] In 1942 the Bureau of Reclamation used fly ash in the repair of the Hoover Dam tunnel spillway in Arizona. The successful construction of the Hungry Horse Dam in 1948 using concrete containing fly ash paved the way for several mass concrete projects that followed.[1,38] In 1958 the U.S. Army Corps of Engineers used fly ash in the construction of the Sutton Dam in West Virginia and the Hartwell Dam between South Carolina and Georgia. By then, fly ash had gained international interest. In 1970, the Georgia Power Company built a steam generating plant in Atlanta using fly ash concrete. The interest in fly ash concrete kept growing mainly because of the increased durability and the economic advantages it offers over conventional concrete.

2.2 Effect of Curing Conditions on the Durability of Concrete

Proper curing of concrete is a must if it is to attain its intended properties. A sufficient supply of moisture is necessary for complete hydration and development of adequate strength and durability. The primary parameters affecting the curing conditions of concrete are the effects of relative humidity and temperature in the cement paste.[35] Even if the concrete is not subjected to fully saturated conditions, it will continue to hydrate. The lower the relative humidity in the paste becomes, the slower the hydration of cement is. If the relative humidity in the paste falls below 80 percent, the hydration of cement ceases altogether.

The external relative humidity influences the strength development at early ages as it contributes to the drying out of the concrete. When concrete is cured at high temperatures it experiences an increase in early strength due to the faster hydration of the cement. At later ages however there is a decrease in strength. Lower curing temperatures, on the other hand, contribute to a slower rate of strength development thus resulting in lower early strength and higher ultimate strength. Figure 2.1 shows the effect of curing temperatures on strength gains after 1 and 28 days. There is no definite explanation as to why strength decreases at later ages when concrete is cured at high temperatures. It is believed that at higher temperatures, the crystal formation in the cement paste is larger thus making the concrete more porous and susceptible to cracking.[2] The loss in strength at later ages is attributed to microcracking and the formation of weak zones within the concrete due to the nonuniform distribution of the hydration products.[35]

When fly ash is used in concrete mixes, extended periods of moist curing are required in order to achieve the best results and obtain full benefits from the use of fly ash. Moisture is necessary to maximize the pozzolanic reaction which contributes to the strength gain at later ages.[1,43] Previous research on concrete containing fly ash recommends a minimum of seven days

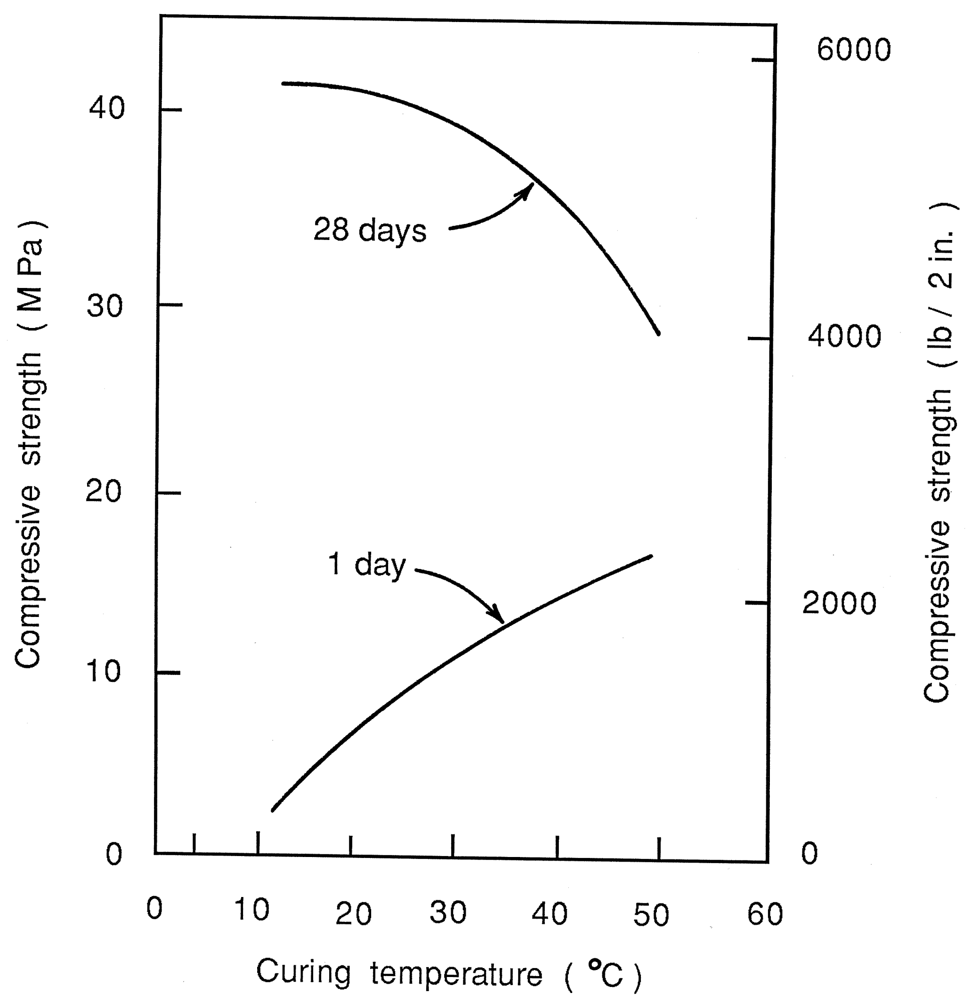

Fig. 2.1 Effect of curing temperature on strength gain after 1 and 28 days[35]

moist curing for concrete exposed to 100°F temperature and 33 percent relative humidity and a minimum of three days moist curing for concrete exposed to 40°F temperatures and 55 percent relative humidity.[5] As opposed to plain concrete, concrete containing fly ash shows higher strength gains at later ages when it is moist cured at high temperatures. Figure 2.2 "shows the general way in which the temperature reached during early ages of curing influences the 28-day strength of concrete."[6]

Curing practices have also been found to greatly affect the abrasion resistance and scaling resistance of concrete. If all of the factors involved in a concrete job such as mix proportions, materials used, and placing and finishing techniques are performed according to specifications, the curing practices then greatly influence the porosity and permeability of the concrete. This will in turn affect the durability and strength gain in the top layer of the specimens.[40]

2.3 Effect of Fly Ash on Strength

Previous research has shown that when fly ash is used as a partial replacement to cement on a percent volume or weight basis, the compressive strength development of concrete is lower at early ages but much higher at later ages.[1,16,24,31,37,41,42] Cook[9,47], however, did research on Type B fly ash and found that concrete mixes containing fly ash as a cement replacement on a percent weight basis were able to attain early strength comparable to, if not better than, conventional concrete. Figure 2.3 shows the compressive strength development of a Type B fly ash as studied by Cook.

Type B ashes, known for their high calcium oxide content, contribute to strength through cementitious and pozzolanic reactions, whereas Type A ashes, known for their low calcium oxide content, contribute through pozzolanic reaction only. It is the cementitious properties of Type B ashes that are responsible for the early strength gain in fly ash concrete. On the other hand, the pozzolanic reaction which occurs over a long time uses the calcium hydroxide liberated by the cement hydration to combine with the silicates and increase the proportions of calcium silicate hydrate (C-S-H) in the cement paste.[32] Several attempts have been made to try to increase the early strength of fly ash concrete.[1] In 1958 Washa[29] found as did several others "that in order to achieve comparable early strengths, fly ash concrete mixes required fly ash replacement in excess of the replaced cement." Practices such as increasing the fineness of fly ash or intergrinding the fly ash with cement have also shown to increase early strength. Figure 2.4 shows the general trend in strength development of concrete containing fly ash as opposed to conventional concrete. It is worth noting from the graph that no specific age can be set as a break point where fly ash concrete will outperform plain concrete. Factors such as the source, chemical and physical composition of the fly ash, the water to cement plus fly ash ratio, the curing conditions, and the percent fly ash replacement of cement all play a crucial role in determining the rate of strength development of fly ash concrete.[6,42]

The early strength gain of concrete containing fly ash is of primary interest in highway construction especially since there is concern over wear damage during early traffic. The strength of the concrete pavement has been shown in most cases to be the most important factor controlling abrasion resistance.

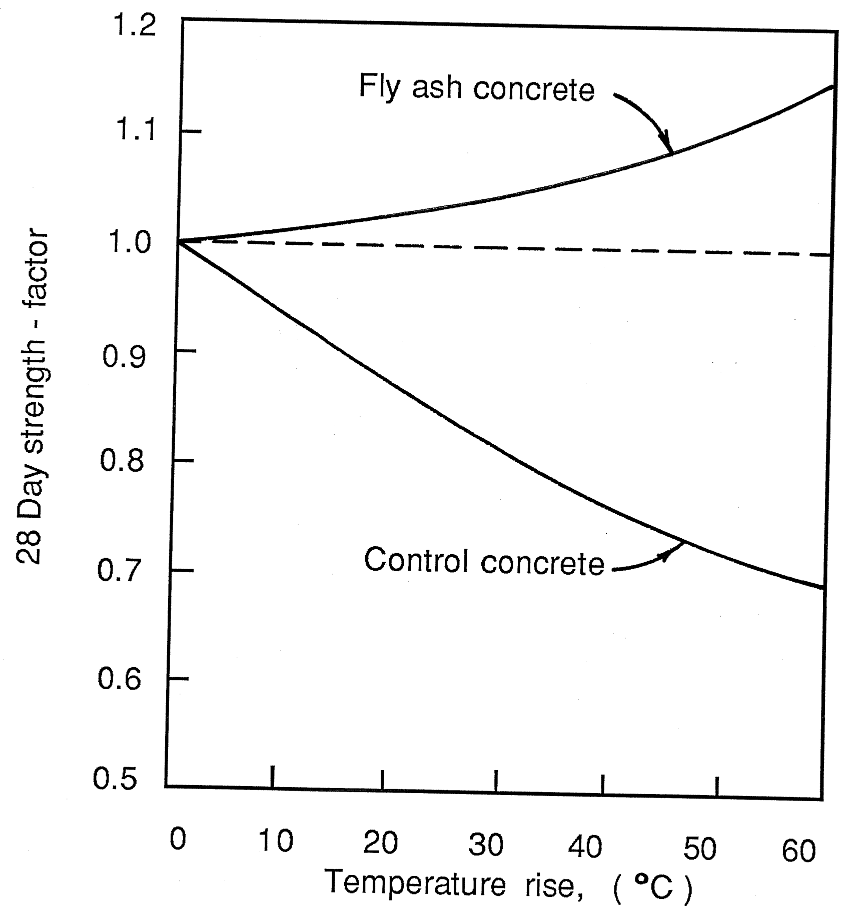

Fig. 2.2 Effect of temperature rise during curing on the compressive strength development of concrete.[6]

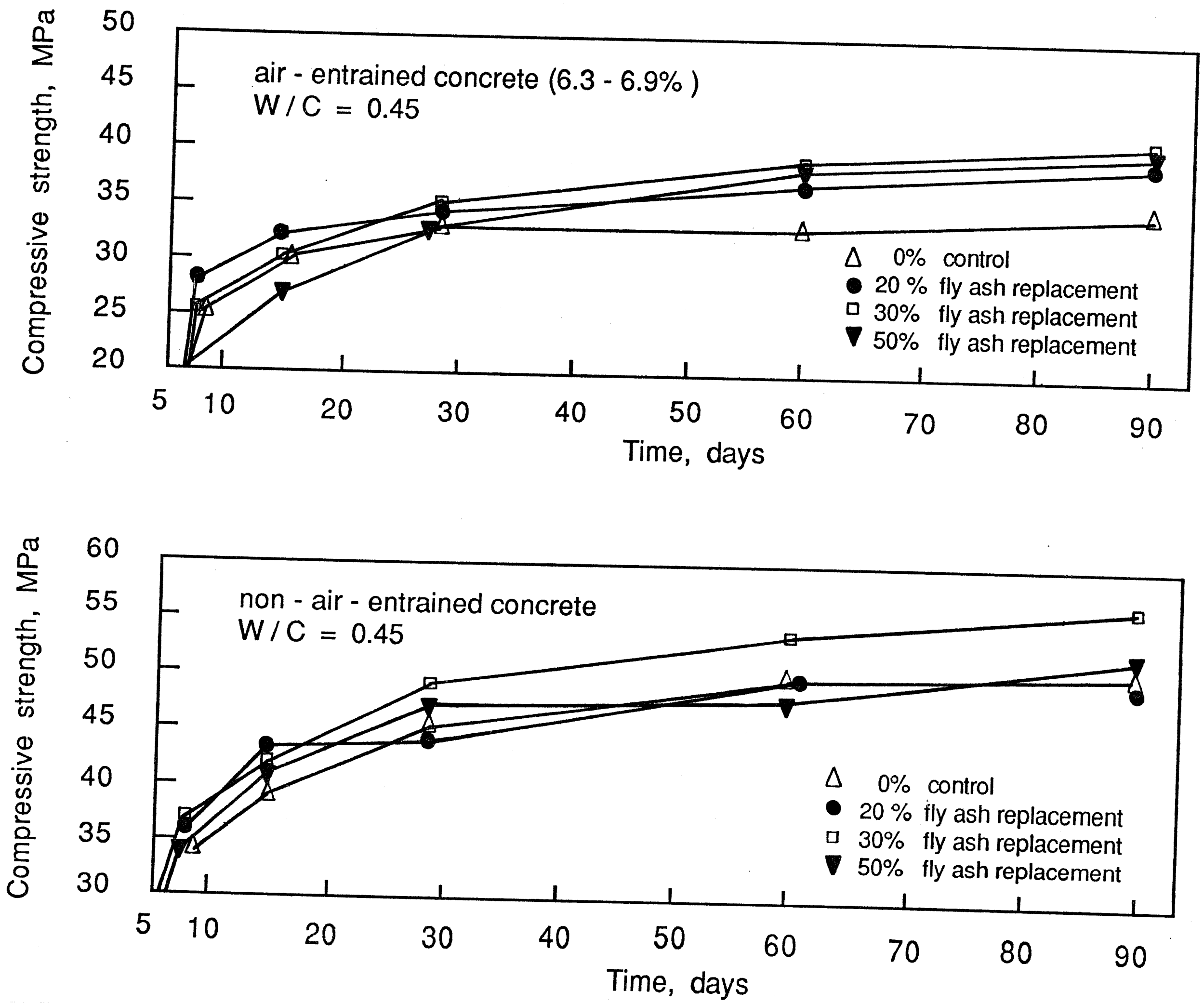

Fig. 2.3 Compressive strength development of concrete containing high calcium fly ash.[6]

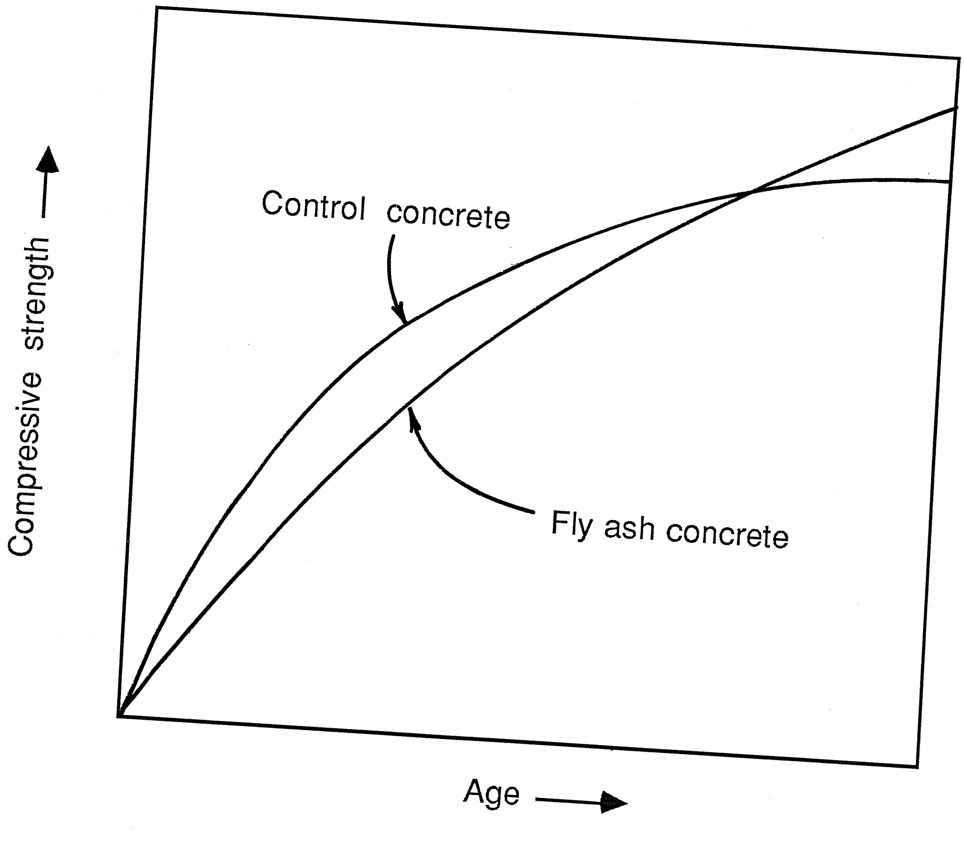

Fig. 2.4 Relative rates of strength increase of plain concrete proportioned by simple replacement of cement.[6]

2.4 Abrasion Resistance of Concrete

2.4.1 Introduction. The wear of concrete surfaces can take one of several forms. Cavitation, which stems mainly from poor design details, is caused by the sudden formation and collapse of low pressure bubbles in liquids flowing at high velocities.[3] Erosion, which is another form of wear, is caused by the action of abrasive materials carried by fluids. Both cavitation and erosion usually occur on dams, spillways, pipes and tunnels. A third form of wear related to concrete pavements or industrial floors is abrasion. It is the result of rubbing, scraping, skidding or sliding of objects on concrete surfaces. The scope of this research is limited to the investigation of abrasion only.

Up until the mid-seventies, there was no specific standard test for determining the abrasion resistance of concrete.[3] The American Society for Testing and Materials then adopted ASTM C779, *Standard Test Method for Abrasion Resistance of Horizontal Concrete Surfaces.*[4] This test method actually covers three different procedures for determining the abrasion resistance of concrete. It includes the Revolving Disc Method, the Ball Bearing Method, and the Dressing-Wheel Method. At about that time, ASTM C418 *Standard Test Method for Abrasion Resistance of Concrete by Sandblasting* was also issued and approved. In 1980, ASTM C944, *Standard Test Method for Abrasion Resistance of Concrete or Mortar Surfaces by the Rotating-Cutter Method*, was issued and has been successfully used in the quality control of concrete for use in highways subjected to traffic.

None of the tests stated above is adequate for all types of abrasion conditions. If all the abrasion test methods were performed on concrete from the same batch, the test results would be quite different for each case. The tests are therefore devised to compare the relative performance of concrete mixes based on weight loss, volume loss, visual inspection, or depth of penetration.

2.4.2 Factors Affecting Abrasion. The primary factors affecting the abrasion resistance of concrete are:

a. compressive strength,

b. aggregate properties,

c. surface finishing,

d. curing, and

e. use of surface hardeners or toppings.

Previous research has shown that the most important factor influencing the abrasion resistance of concrete is the compressive strength. Air entrainment in concrete usually reduces the strength and therefore the abrasion resistance. Backstrom and Witte[46] studied the effects of entrained air on the abrasion resistance of concrete. Their test results revealed that for equal strengths, air entrained concrete is as resistant to abrasion as plain concrete. They also found that as the voids (air plus water) to cement ratio increases, both the compressive strength and abrasion resistance decreased. Figures 2.5 to 2.7 illustrate their findings.

The abrasion resistance of the aggregate is another important factor in determining the abrasion resistance of concrete.[3] This is especially true when exposed aggregates are used as a surface finish or when lower strength concrete is used. Gravel, for example, is much stronger than crushed limestone and therefore is more resistant to wear.

Proper finishing and curing practices significantly increase the abrasion resistance of concrete. Best results are obtained when finishing is delayed until all the bleed water has evaporated and when the concrete has sufficiently stiffened. Fentress[13] studied among other things the relative effect of finishing and curing practices on the wear resistance of concrete made from the same batch. The finishing techniques included wood float, steel trowel and hard steel trowel. Figure 2.8 shows the relative depth of wear for the different types of finish. The curing techniques included moist burlap for three days, curing compound applied immediately after the surface was finished, no curing, and delayed application of the curing compound. It can be seen from Fig. 2.9 that the wear resistance of the concrete increased when proper curing practices were followed. Kettle[20] conducted a similar study, however, he tried to simulate as close as possible the finishing and curing techniques currently used in the concrete construction industry. The finishing and curing practices included hand finish (H.F.), power finish (P.F.), repeated power finish (R.P.F.), vacuum dewatering (V.D.) followed by power finish, air curing (A.C.), wet burlap (W.B.), polyethelyne sheeting (P.S.), and three kinds of liquid curing compounds (C.C.1., C.C.2, C.C.3). Kettle found that abrasion resistance of concrete depends heavily on the finishing techniques applied with repeated power finishing yielding the best results in most cases. Proper curing also played a significant role in controlling the wear resistance. This became more apparent as the water cement ratio of the concrete mixes increased. Figures 2.10 through 2.13 illustrate the performances of one of the mixes used in the study. Senbetta[40] also showed that through proper curing the abrasion resistance of concrete is enhanced. His tests were performed according to ASTM C779 using the Revolving Disc Method and according to ASTM C418, *Abrasion Resistance of Concrete by Sandblasting*. Figures 2.14 and 2.15 show that both test results indicate similar trends and that curing has a pronounced effect on the abrasion resistance.

The application of special toppings, dry shakes, or iron aggregates to the concrete surface[3] or the incorporation of either polypropylene or steel fiber to the concrete mix have also been shown to increase the abrasion resistance. Hoff[17] and Vondran[45] found that, for approximately equal strength, steel fiber reinforced concrete exhibited the most resistance to wear followed by polypropylene fiber reinforced concrete and plain concrete respectively when tested using ASTM C779, the Ball Bearing Test Method, and ASTM C779, the Dressing-Wheel Test Method, respectively.

2.4.3 Recommendations for Abrasion Resistant Concrete. As mentioned in the previous section, there are several factors involved in controlling the abrasion resistance of concrete surfaces. To obtain abrasion resistant concrete, the following measures should be followed.[3]

The concrete should have adequate strength levels; 4000 psi minimum compressive strength at the time of exposure to services is recommended. This could be achieved through proper mix proportioning by keeping the water cement ratio and the air content low.

The finishing operation of the concrete surface should be delayed until all the bleed water has evaporated and the concrete is stiff enough to withstand some pressure.

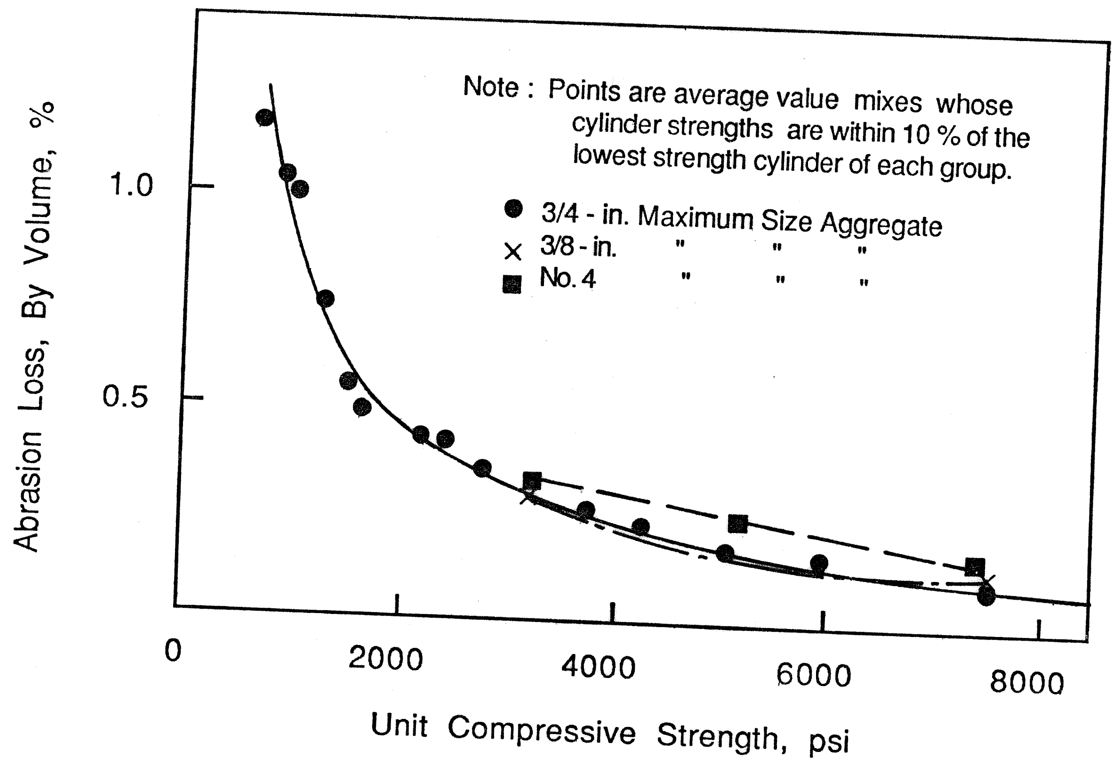

Fig. 2.5 As compressive strength increases abrasion loss decreases.[46]

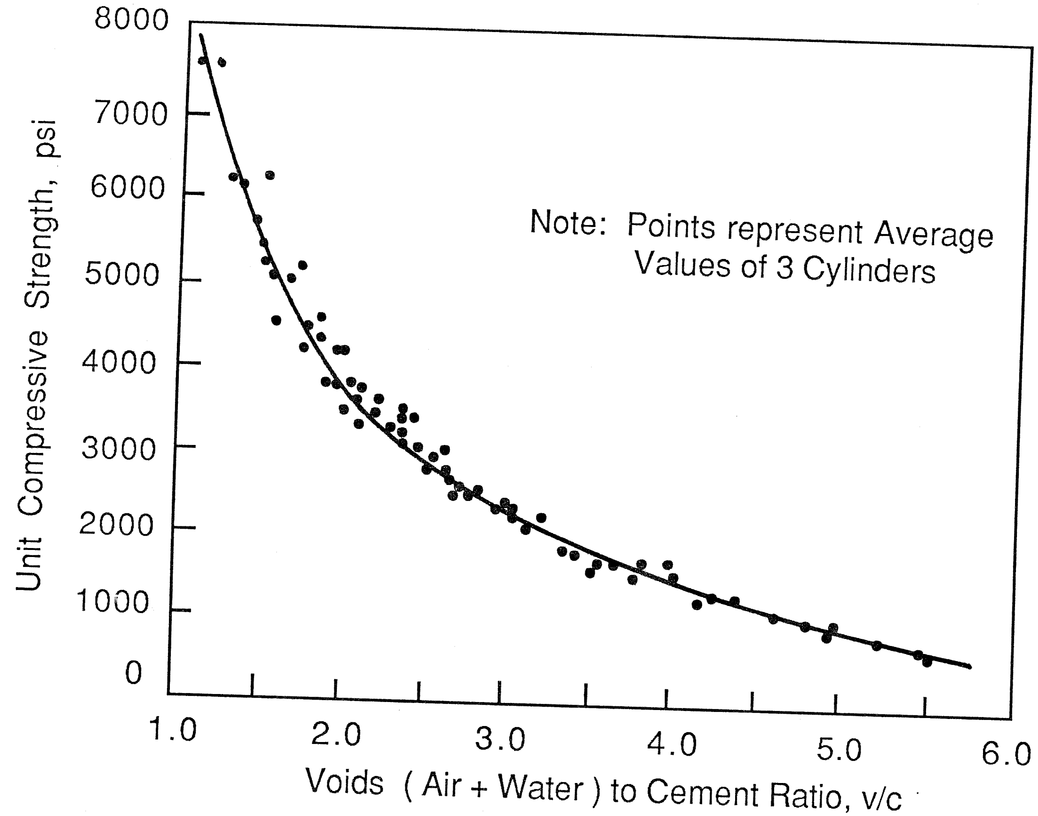

Fig. 2.6 Compressive strength is dependent on the voids cement ratio regardless of water cement ratio, water, cement, or air content.[46]

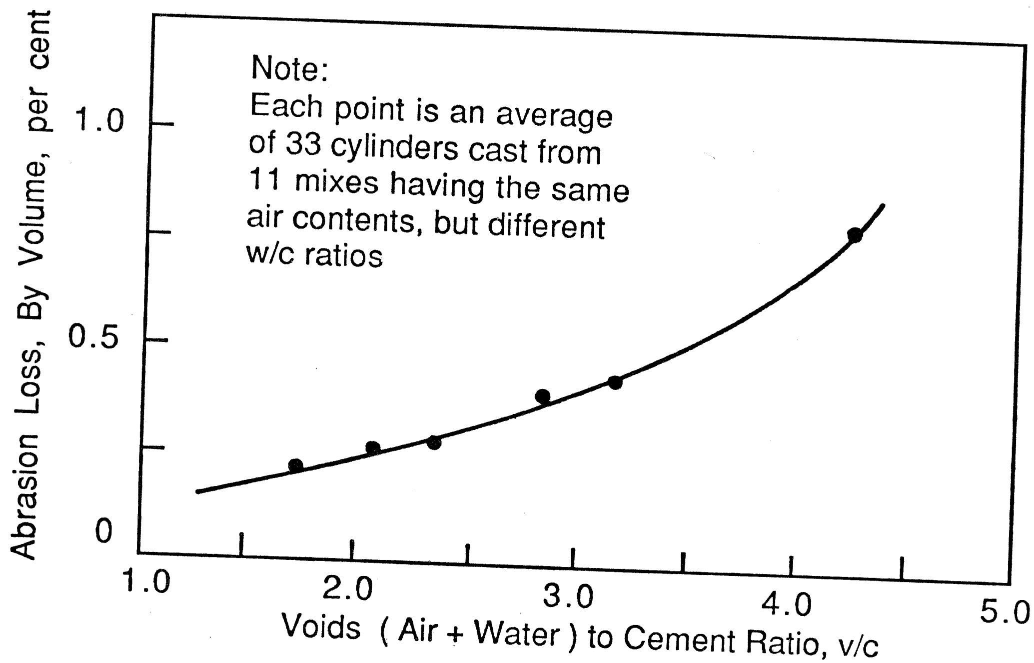

Fig. 2.7 As the voids-cement ratio increases the abrasion loss increases.[46]

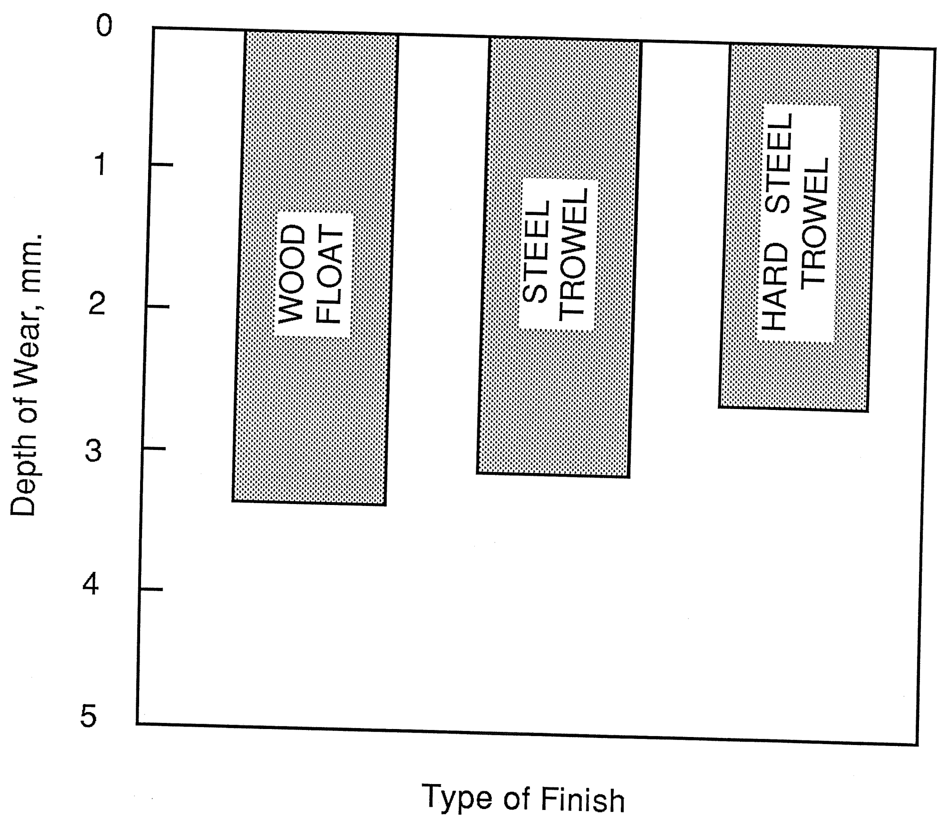

Fig. 2.8 The depth of wear in the test varied with the type of finish.[13]

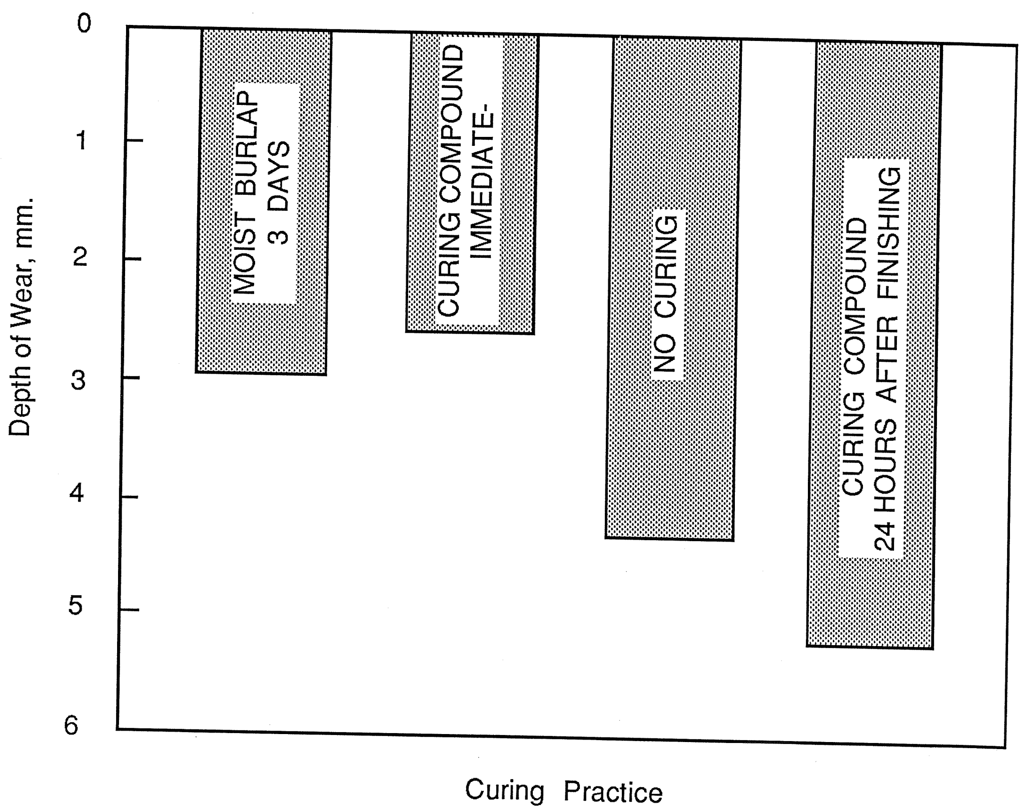

Fig. 2.9 The depth of wear was low when good curing practices were used.[13]

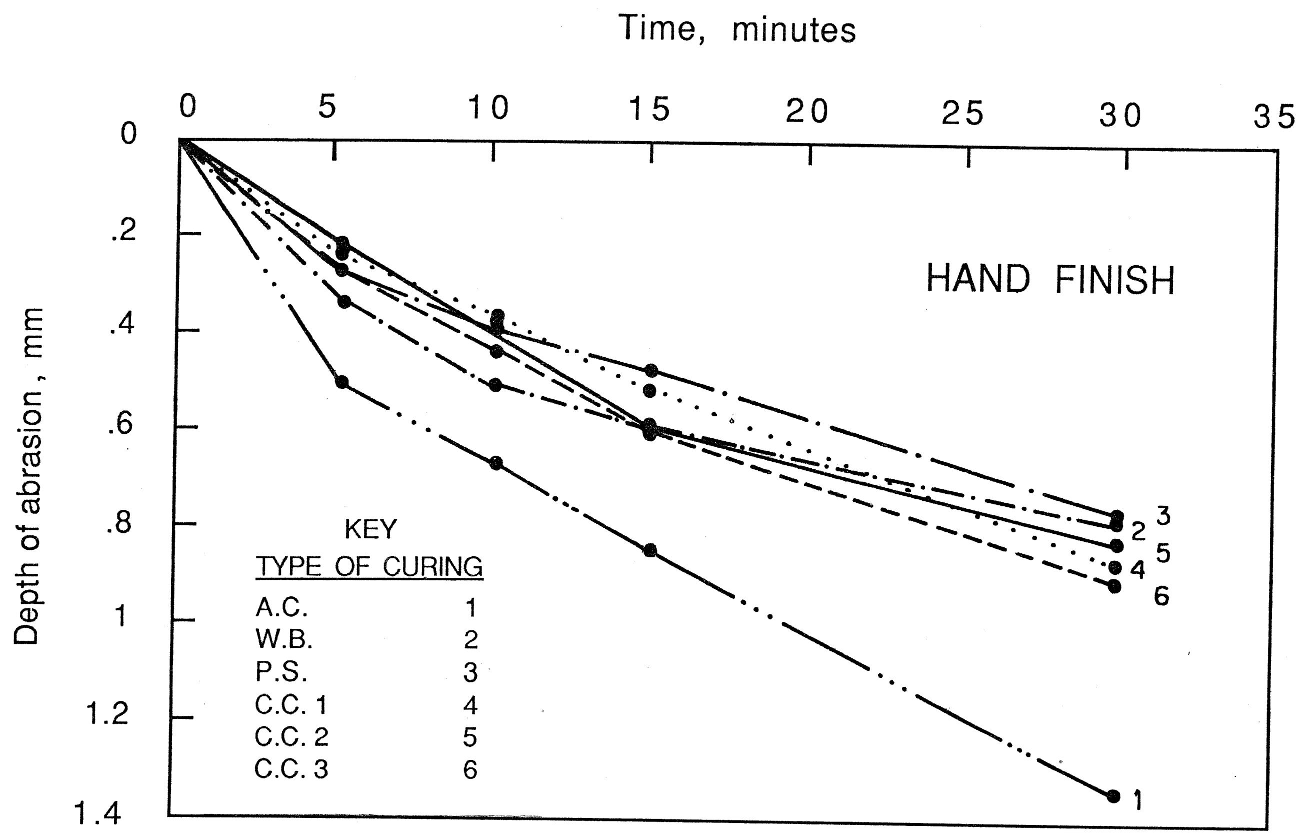

Fig. 2.10 Rate of abrasion for the hand finished specimens.[20]

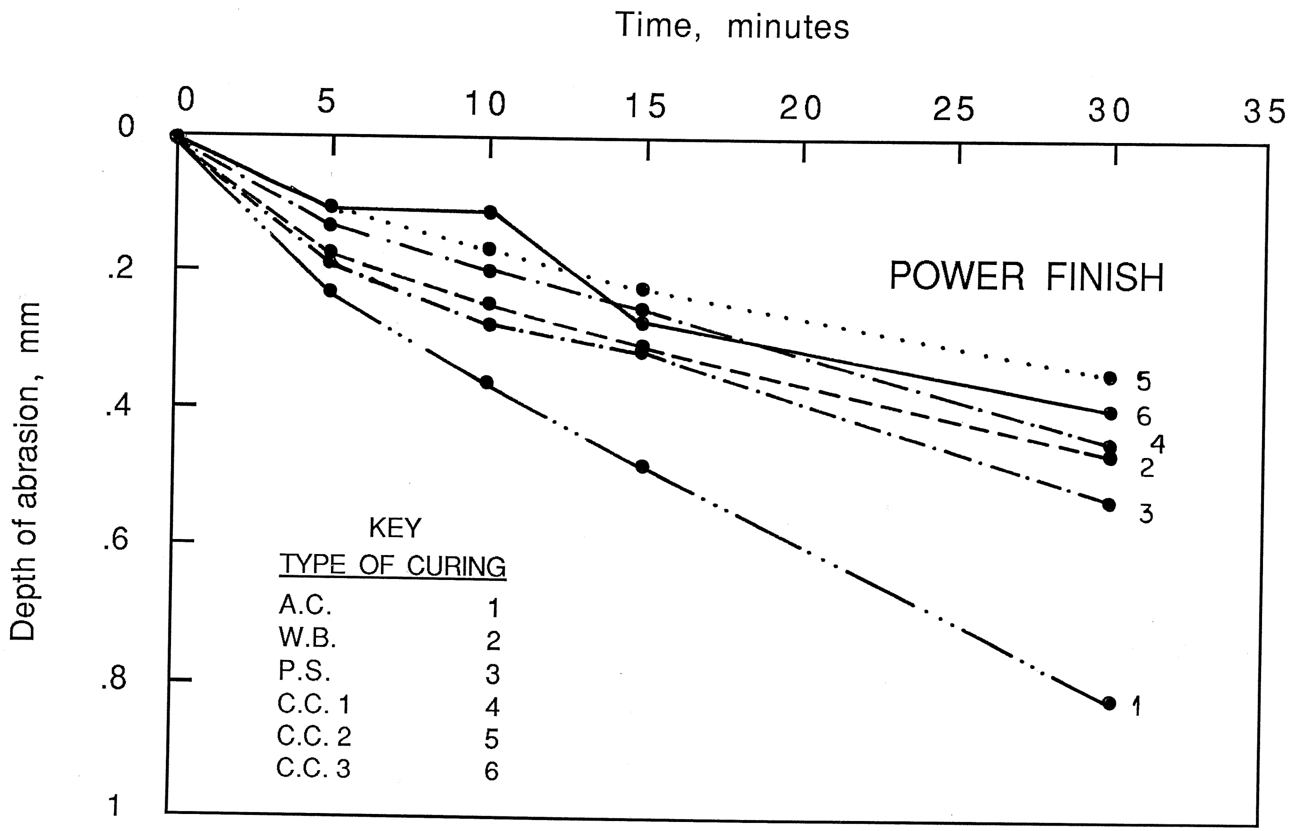

Fig. 2.11 Rate of abrasion for the power finished specimens.[20]

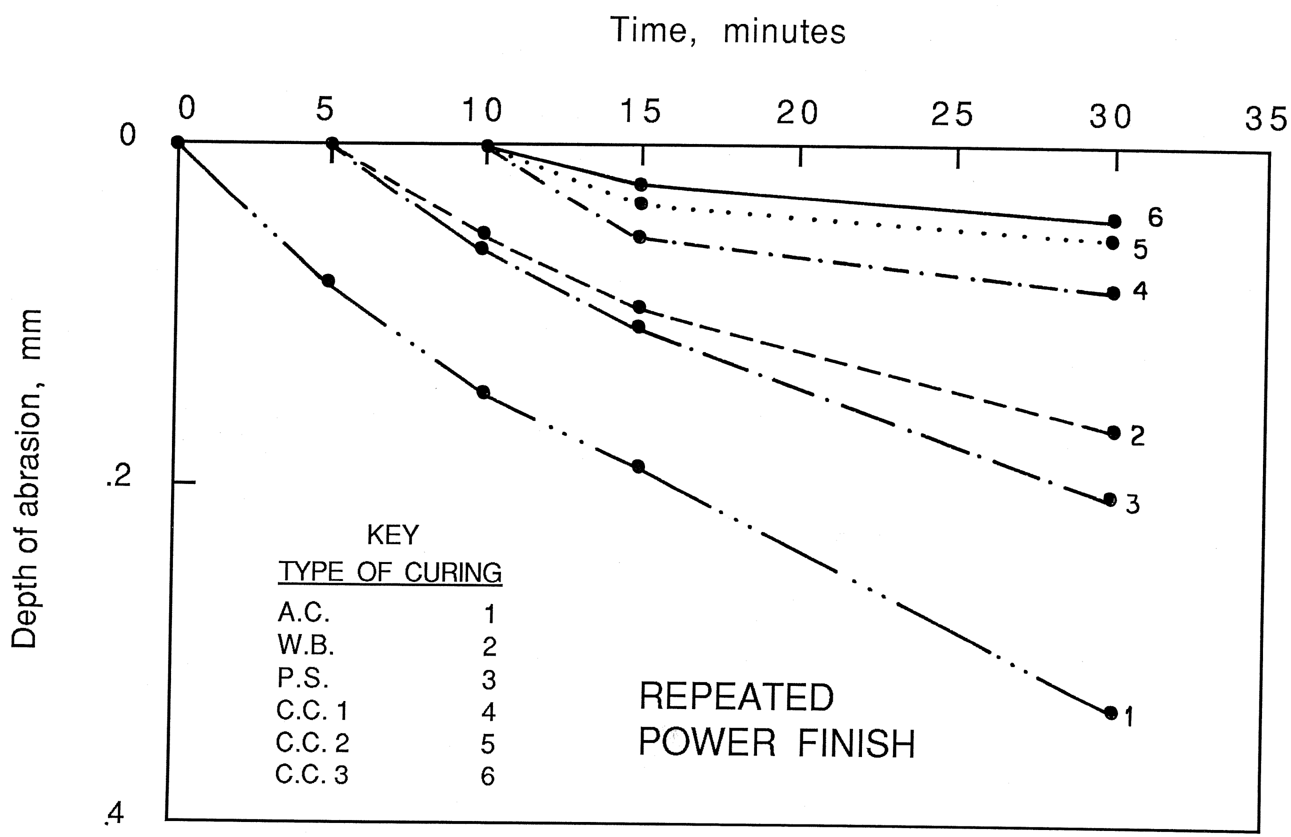

Fig. 2.12 Rate of abrasion for the repeated power finished specimens.[20]

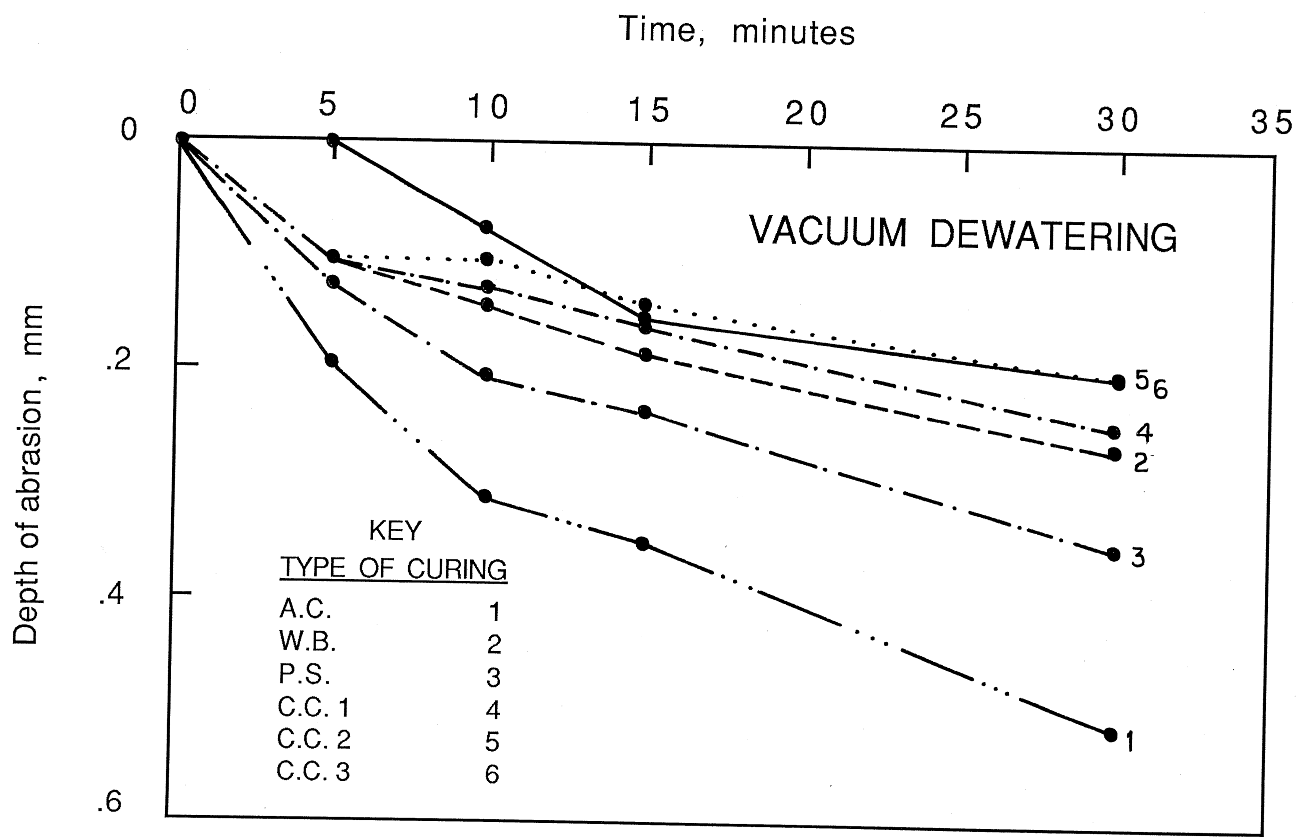

Fig. 2.13 Rate of abrasion for the specimens subjected to vacuum dewatering.[20]

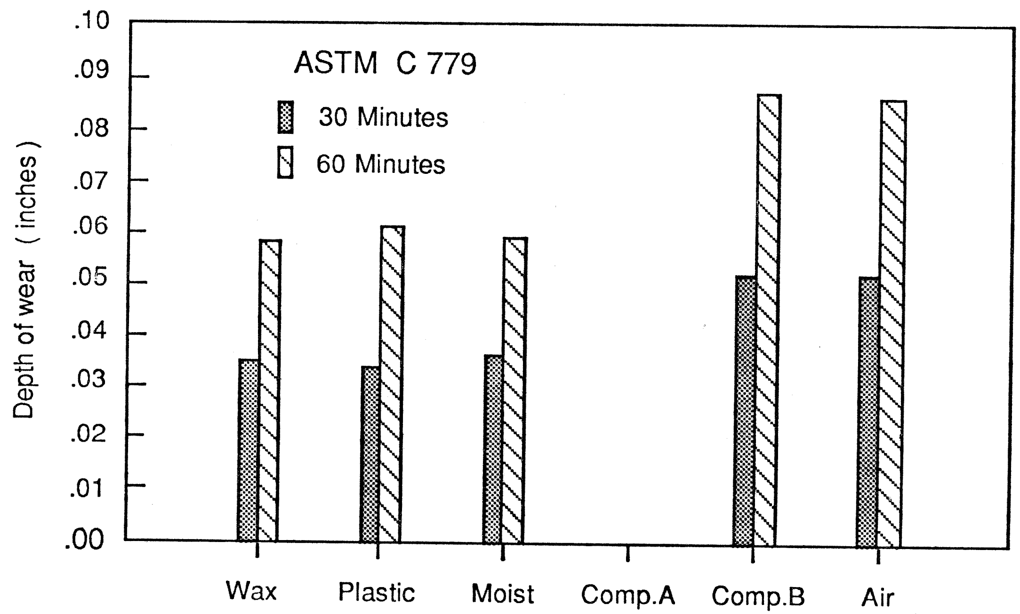

Fig. 2.14 Effect of curing on abrasion resistance.[40]

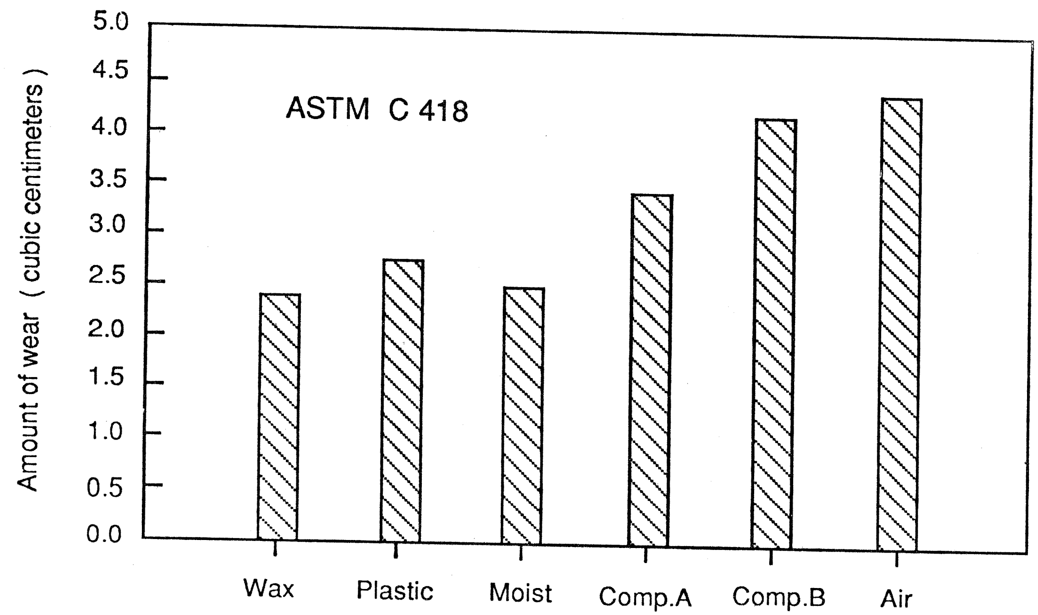

Fig. 2.15 Abrasion resistance as influenced by curing.[40]

Adequate curing of the concrete should begin immediately after the finishing operation is completed and continued for a minimum period of seven days.

Where the concrete is likely to be subjected to excessive abrasion, the use of hard, tough coarse aggregates in the top layer or the use of special toppings or dry shakes at the surface will greatly lengthen the service life of the concrete.

2.5 Effect of Fly Ash on Air Entrainment

Air entrainment is one of the most important factors in improving the durability of concrete subjected to freezing and thawing and deicing chemicals.[35] During freeze-thaw cycles, entrained air in concrete serves as a pressure-relief point to which the water can migrate to upon freeze without causing any damage to the paste.[26] Figure 2.16 illustrates the effect of air entrainment on the frost resistance of concrete. Entrained air also reduces bleeding and makes the concrete less permeable. This will in turn improve the scaling resistance of concrete by reducing the potential for the penetration of any deicer salt solution beyond the concrete surface.

There has been some concern as to whether concrete containing fly ash is as durable as conventional concrete. Larson[26] and several others[24,25,42,43] found that when concrete containing fly ash is compared to plain concrete on an equal strength and air content basis, they both exhibited comparable freeze-thaw durability.

Laboratory tests conducted on concrete containing fly ash have shown that for a constant dosage of air entraining admixture, the air content of concrete containing fly ash was less than that of plain concrete.[10,14,25,26,34] There is general agreement that fly ash concrete requires more air entraining admixture than concrete without fly ash to entrain the same amount of air. The loss of air in concrete containing fly ash is mainly attributed to the carbon content of fly ash and its adsorption capabilities.[26,34,41,42] Minnick[34] was the first to establish that the organic compounds of the air entraining admixture adsorbed to the carbon. He further noted that once the carbon becomes saturated, air entrainment should occur in a normal fashion. A more recent study by Gebler and Kleiger[14] seems to substantiate these findings. The results of their study revealed that for an increase in organic matter content, carbon content, and loss on ignition of the fly ash, the air entraining admixture dosage also increases. They also found that concrete containing ASTM Class C (TSDHPT Type B) fly ash require less air entraining admixture than concretes containing ASTM Class F (TSDHPT Type A) fly ash.

The loss on ignition (LOI) is defined as the percentage of residual organic matter composed mainly of unburned carbon in fly ash. Research[8,41,43] has shown that as the LOI of a fly ash increase, the dosage of air entraining admixture required to produce a given amount of entrained air also increases. Since the carbon content is the primary factors affecting the loss of air in concrete containing fly ash, limits on the LOI of fly ash were included in the specification. ASTM C618-85 and the Texas SDHPT D-9-8900 place a 6 and 3 percent limit, respectively, on the LOI of acceptable fly ash.

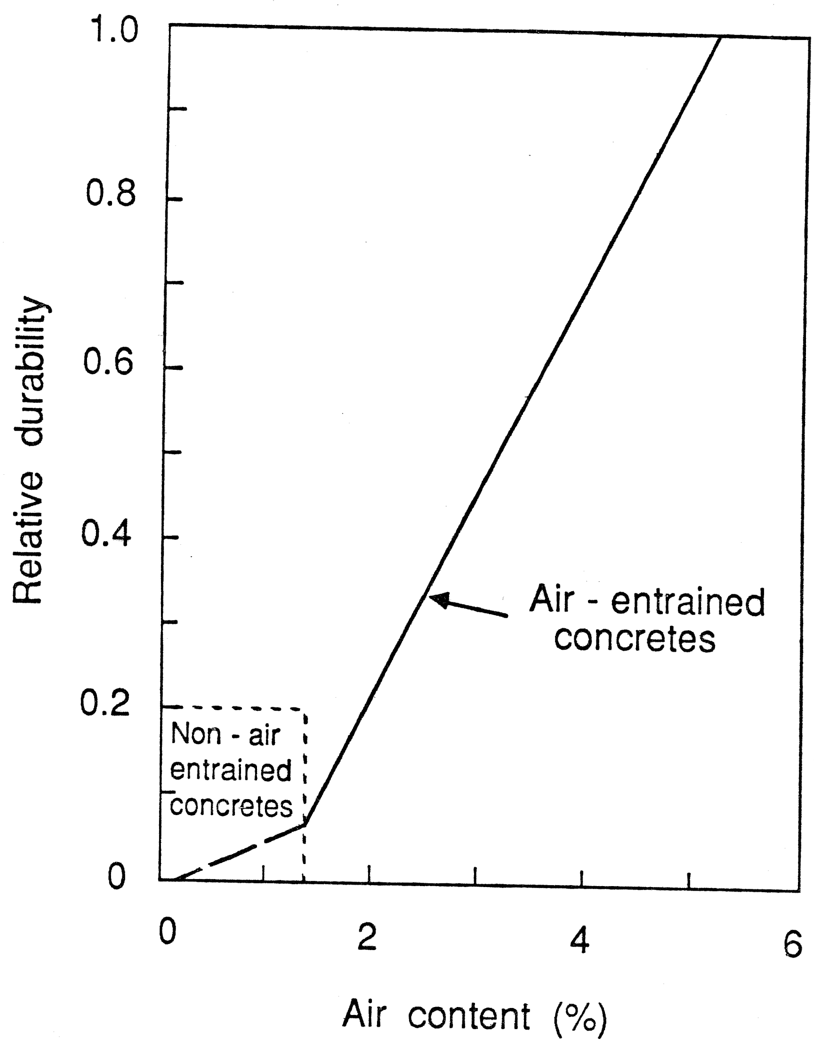

Fig. 2.16 Effect of air entrainment on frost resistance of concrete.[35]

2.6 Scaling Resistance of Concrete

2.6.1 Mechanism of Scaling. Scaling occurs when the top surface of the concrete flakes or peels off. It is mainly caused by repeated freezing and thawing in the presence of deicing chemicals. The scaling mechanism of concrete subjected to salt solutions is very complex. There is general agreement that the mechanism involved is mostly physical in nature rather than chemical. Verbeck and Kleiger[44] conducted research using a variety of chemically dissimilar materials and obtained comparable scaling for every case. The scaling mechanism is based on the osmotic pressure theory.[39] When salt is used to melt ice, the salt solution slowly penetrates the concrete surface and fills the pores of the cement paste. The capillary pores which are filled much faster than the cement gel pores attain a higher salt concentration. Upon freezing, ice builds up in the capillary pores and the salt concentration of the unfrozen water is further increased. As the concentration of the solution keeps increasing, the capillary pores are transformed into osmotic pressure cells capable of attracting weaker solutions from the surrounding cement gel pores. This causes the capillaries to grow and exert pressures on the cement gel which may eventually rupture. The larger the difference in concentration between the capillary pore solution and the cement gel pore solution, the greater the pressure exerted on the cement paste is. Air entrainment has proven to enhance the scaling resistance of concrete. It acts as a pressure relief point and provides a path for water osmosis. Litvan,[27] however, has shown that air entrainment does not always effectively prevent scaling. His research suggests that the best protection against scaling is reduced porosity.

2.6.2 Factors Affecting the Scaling Resistance. The most important factors affecting the scaling resistance of concrete are:

a. mix proportioning and materials used,

b. curing conditions,

c. concentration of the salt solution, and

d. air entrainment.

Proper mix proportioning is essential in order to obtain scaling resistant concrete. The water cement ratio (w/c) of the concrete mix should not exceed 0.45.[3] Johnston[30] found that as the w/c increased, the scaling resistance of concrete decreased. The use of good sound aggregates is also necessary to ensure proper durability. Porous and lightweight aggregates should be avoided. The incorporation of polypropylene fibers in the concrete mix has been shown to improve the scaling resistance of concrete.[45] Polypropylene fibers tend to increase the cohesiveness and reduce the bleeding and permeability of the concrete.

Curing practice greatly affect the scaling resistance of concrete. The concrete should be allowed to attain adequate strength and undergo period of drying before it is subjected to freezing and thawing in the presence of salt solutions. Kleiger[21] found that continuously moist cured concrete scaled more than concrete which was allowed to partially dry. Pigeon[36], conducted scaling resistance tests on concrete containing silica fume. His tests revealed that the use of a curing compound instead of seven days moist curing improved the scaling resistance of concrete. Figure 2.17 shows the scaling test results for one of his mixes. In another study on concrete containing fly ash, Kleiger and

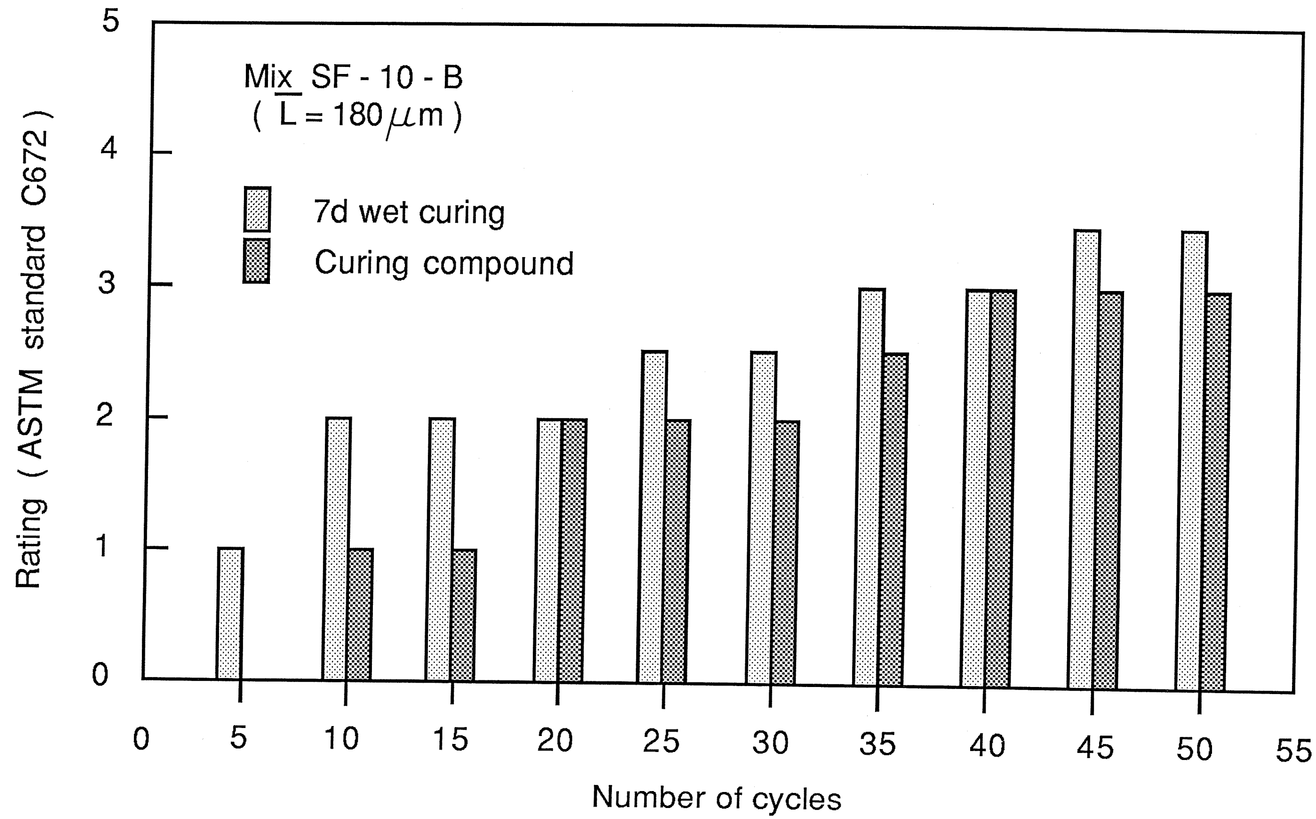

Fig. 2.17 Scaling test results for a mix containing 10 percent silica fume.[36]

Gebler[22] found that plain concrete was more resistant to scaling than fly ash concrete regardless of the curing conditions provided.

Verbeck and Kleiger[44] studied the effect of using various concentrations of either calcium chloride, sodium chloride, ethyl alcohol, or urea as a deicer. They were able to obtain comparable scaling with the different solutions used. They also found that the scaling resistance of concrete is dependent on the concentration of the solution used. When the solutions had a moderate concentration of 2 to 4 percent, the specimens suffered a much greater rate of scaling deterioration than when the concentrations were higher at about 8 to 16 percent.

Air entrainment in concrete is known to considerably improve the freeze-thaw durability and scaling resistance of concrete. Research[22,27,39,44] has shown that air entrained concrete is much more resistant to scaling than regular concrete. Air entrainment in the concrete helps in relieving the pressures caused by the frost action. Litvan[27] however suggests that air entrained concrete can suffer greater freeze thaw damage than plain concrete. He believes that air entrainment is beneficial to protect the concrete against freezing only when the air bubbles are dry. When prolonged periods of wetting occur, the air bubbles become filled with salt solution which has lower vapor pressure than pure water. This causes the air voids to remain saturated at lower relative humidities. Upon renewed freezing, the pores are already saturated and the concrete suffers more damage because its porosity is larger. Litvan further states that porosity is the most important factor controlling the scaling mechanism. Senbetta and Malchow[40] found that the capillary porosity of concrete near the surface can be reduced by as much as 80 percent if proper curing practices are applied.

CHAPTER 3
MATERIALS AND TEST PROCEDURES

3.1 Materials

All the materials and test procedures used in this research are in accordance with the 1985 Annual Book of American Society for Testing and Materials Standards Sec. 4 on Construction and/or the Manual of Testing Procedure by the Texas State Department of Highways and Public Transportation. The materials used include portland cement, fly ash, coarse aggregate, fine aggregate, chemical admixtures, and water.

3.1.1 Portland Cement. The cement used in the study was an ASTM C150-85 Type I-II cement. It meets the specifications for both Type I and Type II cement. The chemical and physical properties of the Type I-II cement used are presented in Table 3.1.

Table 3.1 Chemical and Physical Properties of the Type I-II Portland Cement Used

Chemical Composition		Percent by Weight
Silicon Dioxide (SiO_2)		21.8
Aluminum Oxide (Al_2O_3)		4.2
Ferric Oxide (Fe_2O_3)		3.3
Calcium Oxide (CaO)		65.2
Magnesium Oxide (MgO)		0.6
Sulfur Trioxide (SO_3)		2.8
Loss on Ignition		0.9
Insoluble Residue		0.2
Free Lime		0.9
Tricalcium Silicate (C_3S)		57
Tricalcium Aluminate (C_3A)		6
Total Alkalies, Na_2O equivalent		0.63
Physical Properties		
Specific Surface		
Blaine		3350 cm^2/gm
Wagner		1890 cm^2/gm
Compressive Strength		
1 day		2030 psi
3 days		3640 psi
7 days		4670 psi
Time of Setting	*Vicat*	*Gilmore*
Initial	91 min.	132 min.
Final	210 min.	244 min.

Table 3.2 Fly Ash Chemical and Physical Composition

Study Designation	Type A A	Type B B1	Type B B2
Si + Al + Fe Oxides, %	78.48	64.99	59.38
Calcium Oxide, %	6.97	22.44	29.86
Magnesium Oxide, %	0.81	3.88	5.63
Sulfur Trioxide, %	0.26	1.97	2.48
Available Alkalies, Na_2O equivalent, %	0.31	2.35	1.67
Loss on Ignition, %	0.04	0.28	0.03
Fineness, amount retained on 325 sieve, %	13.30	17.30	18.90
Pozzolanic activity with cement, % of control	97.09	100.00	91.55
Shrinkage, %	0.007	-0.003	-0.007

3.1.2 Fly Ash. One Texas SDHPT Type A and two Texas SDHPT Type B fly ashes were used in the study. The Texas Type B fly ashes were chosen in such a way so as to have one with a higher calcium oxide content (CaO) than the other. All the fly ashes used are approved for commercial use by the Texas SDHPT. Table 3.2 contains the physical and chemical properties of the different fly ashes used. The fly ashes were incorporated in the concrete mixes at 0, 25 and 35 percent partial replacement of portland cement by volume.

3.1.3 Coarse Aggregate. The coarse aggregate used throughout the testing program was 3/4-in. nominal maximum size crushed limestone. The aggregate met the specifications of the Texas Highway Department Grade No. 5 and ASTM C33 Gradation No. 67 for coarse aggregates. The bulk specific gravity at SSD, the absorption capacity, and the dry rodded unit weight were determined according to Texas Specifications Test Method TEX-403-A and Tex-404-A. The specific gravity at SSD was 2.50, the absorption capacity was 3.50 percent, and the dry rodded unit weight was 85.41 pounds per cubic foot.

3.1.4 Fine Aggregate. The fine aggregate used in the study was a siliceous river sand from the Colorado River. During the duration of the study, sand from two different shipments was used. For the first truck load, the bulk specific gravity at SSD and the absorption capacity were determined according to Texas SDHPT specifications and were found to be 2.63 and 0.63 percent, respectively. For the second truck load, the sand properties were provided by the distributor to be 2.60 for the specific gravity and 1.20 percent for the absorption capacity.

3.1.5 Chemical Admixtures. A commercially available neutralized vinsol resin air entraining agent and a water reducer- retarder were used in the mix proportions. The chemical admixtures met the specifications of the Texas SDHPT and ASTM C260 and C494. The dosage of air entraining admixture necessary to entrain the amount of air required was determined by trial and error using trial batches. The dosage of water reducer-retarder used was 4 oz/cwt corresponding to the median of the range provided in the manufacturer's specifications.

3.1.6 Water. The mixing water used during batching was regular tap water meeting both ASTM and Texas SDHPT specifications.

3.2 Mix Proportions

The mix proportions were designed according to the Texas SDHPT 1982 Standard Specification for Construction of Highways, Streets and Bridges Item 366, Continuously Reinforced Concrete Pavement. The mix designs were produced to meet a flexural strength when tested in center point loading of at least 575 psi at 7 days. In order to account for the 10 to 15 percent higher strength obtained in the laboratory as compared to the same concrete batched under field conditions, the mix proportions for this study were selected on the basis of a minimum 7-day flexural strength of between 630 and 660 psi. As a result, it is essential that the mix proportions selected for this study will meet the 575 psi minimum strength requirements if the concrete is produced under actual field conditions. Fly ash was used in the amount of 25 and 35 percent replacement of the absolute volume of portland cement. The design factors involved in the mix proportions are shown in Table 3.3. When fly ash was used, it was necessary to increase the cementitious factor in order to meet the 7-day strength design requirement. Since most of the batching operation was taking place in the hot summer months, it was necessary to use a water reducer-retarder so as to allow for enough time for handling, placing and finishing of the concrete. After batching, the mix proportions were adjusted for yield due to any variations in the mixing water and in the air content. The nomenclature of the different mixes used throughout the study is shown in Table 3.4. The type A fly ash used is referred as fly ash A, whereas the two Type B fly ashes used are referred to as fly ash B1 and fly ash B2. The adjusted mix proportions for every concrete batch are given in Appendix A Table A.1.

3.3 Mixing Procedure

The mixing procedure was done according to ASTM C192-81 *Standard Test Method of Making and Curing Concrete Test Specimens in the Laboratory.* The trial batches were mixed in a 3-cubic foot rotary drum mixer while the final batches for casting specimens were done in a 9-cubic foot rotary drum mixer. The mixing procedure was done in the following manner. The coarse aggregate was loaded into the mixer with approximately one quarter of the total mixing water needed for the mix. The initial water was added to saturate the coarse aggregate and prevent it from absorbing any of the admixtures used. The air entraining admixture was then mixed in with the second quarter of water required and added to the coarse aggregate. The mixer was then started and allowed to make a few revolutions to help initiate the formation of the air bubbles. The water reducer-retarder was added with the third quarter of water after loading the fine aggregate and the cementitious materials. Mixing continued while the final quarter of mixing water was gradually added to the mix. The concrete was mixed for three minutes followed by a three minute rest followed by a final two minute mixing period. The mixer was then stopped and a slump test was performed. If the slump was between 1 in. and 3 in., the concrete was considered to have met the design criteria for slump and other fresh concrete tests were performed. If, however, the slump was between 0 in. and 1 in., more water was added to the mix and the concrete was mixed for an additional two to three minutes, at the end of which another slump test was performed.

3.4 Test Specimens

The flexural strength, abrasion resistance, and resistance to deicing scaling specimens were cast using reusable metal molds. Compressive strength specimens were cast in plastic cylindrical molds which were discarded after one use.

Table 3.3 Design Factors for the Various Concrete Mixes

Plain Concrete	
Cement Factor (CF)	5.1 sacks/c.y.
Coarse Aggregate Factor (CAF)	0.68
Water Factor (WF)	5.5 gal/sack
Air Factor (AF)	5.0%
Type A Fly Ash 25% Replacement	
Cement + Fly Ash Factor (CF)	5.75 sacks/c.y.
Coarse Aggregate Factor (CAF)	0.68
Water Factor (WF)	4.2 gal/sack
Air Factor (AF)	5.0%
Type A Fly Ash 35% Replacement	
Cement + Fly Ash Factor (CF)	6.0 sacks/c.y.
Coarse Aggregate Factor (CAF)	0.68
Water Factor (WF)	4.5 gal/sack
Air Factor (AF)	5.0%
Type B Fly Ash 25% Replacement	
Cement + Fly Ash Factor (CF)	5.25 sacks/c.y.
Coarse Aggregate Factor (CAF)	0.68
Water Factor (WF)	5.15 gal/sack
Air Factor (AF)	5.0%
Type B Fly Ash 35% Replacement	
Cement + Fly Ash Factor (CF)	5.5 sacks/c.y.
Coarse Aggregate Factor (CAF)	0.68
Water Factor (WF)	4.9 gal/sack
Air Factor (AF)	5.0%

3.4.1 Finishing Procedure. Immediately after consolidation, all the test specimens were struck off with a wooden float. The flexural strength and compressive strength specimens were then finished with a steel trowel. For the abrasion and scaling specimens, finishing with a steel trowel was delayed until all the bleed water had evaporated and the concrete was sufficiently stiff to withstand moderate thumb pressure. The waiting period ranged anywhere from two to three hours depending on the temperature and relative humidity the day the batching was done.

3.4.2 Curing Practices. Immediately after finishing, the compressive strength cylinders were sealed with especially designed plastic caps to prevent any moisture loss from the specimens. The flexural strength specimens were covered with a double layer of wet burlaps topped by a polyethylene sheet. The abrasion and scaling specimens were put in a wooden enclosure with saturated burlaps around the sides of the molds and a polyethylene sheet over the whole enclosure. In no instance did the burlaps or polyethylene sheet come into contact with the concrete surface. All the test specimens were demolded at 24 hours and cured in one of the following fashions. The curing conditions consisted of three curing temperatures, 50, 73 and 100°F and two relative humidities, 50 and 100 percent. For a given batch cured at one of the given temperatures, half the abrasion and

Table 3.4 Nomenclature for the Different Mix Designs

Mix Designation	Curing Conditions		Fly Ash	
	R.H. %	Temp. °F	Type	% by Volume
50-50	50	50	N/A	0
100-50	100	50	N/A	0
50-50A-25	50	50	A	25
200-50A-25	100	50	A	25
50-50B1-25	50	50	B1	25
100-50B1-25	100	50	B1	25
50-50B2-25	50	50	B2	25
100-50B2-25	100	50	B2	25
50-50A-35	50	50	A	35
100-50A-35	100	50	A	35
50-50B1-35	50	50	B1	35
100-50B1-35	100	50	B1	35
50-50B2-35	50	50	B2	35
100-50B2-35	100	50	B2	35
50-73	50	73	N/A	0
100-73	100	73	N/A	0
50-73A-25	50	73	A	25
100-73A-25	100	73	A	25
50-73B1-25	50	73	B1	25
100-73B1-25	100	73	B1	25
50-73B2=25	50	73	B2	25
100-73B2-25	100	73	B2	25
50-73A-35	50	73	A	35
100-73A-35	100	73	A	35
50-73B1-35	50	73	B1	35
100-73B1-35	100	73	B1	35
50-73B2-35	50	73	B2	35
100-73B2-35	100	73	B2	35
50-100	50	100	N/A	0
100-100	100	100	N/A	0
50-100A-25	50	100	A	25
100-100A-25	100	100	A	25
50-100B1-25	50	100	B1	25
100-100B1-25	100	100	B1	25
50-100B2-25	50	100	B2	25
100-100B2-25	100	100	B2	25
50-100A-35	50	100	A	35
100-100A-35	100	100	A	35
50-100B1-35	50	100	B1	35
100-100B1-35	100	100	B1	35
50-100B2-35	50	100	B2	35
100-100B2-35	100	100	B2	35

scaling specimens along with companion compressive strength cylinders were cured at 50 percent relative humidity while the other half was cured in a saturated lime bath. At all times the seven day flexural strength specimens and a series of control cylinders were cured in a saturated lime bath at 73°F meeting the specification of ASTM C511-80 *Standard Specification for Moist Cabinets, Moist Rooms, and Water Storage Tanks Used in Testing of Hydraulic Cements and Concretes.*

3.5 Fresh Concrete Properties

3.5.1 Slump. The mix design called for a slump between 1 and 3 inches. Anytime the slump was less than 1 in., it was adjusted by the addition of water to the mix. If the slump exceeded the upper limit by more of 1/4 in., the batch was discarded. The slump test was performed according to ASTM C143-78, *Standard Test Method for Slump of Portland Cement Concrete* and Texas SDHPT procedure Tex-415- A, *Slump of Portland Cement Concrete.*

3.5.2 Air Content. The concrete mixes were designed to have an air content of 5 ± 1 percent. The air content of the concrete was measured according to ASTM C173-78, *Standard Test Method for Air Content of Freshly Mixed Concrete by the Volumetric Method* and Texas SDHPT procedure Tex-416-A, *Air Content of Freshly Mixed Concrete, B: Volumetric Method.*

3.5.3 Unit Weight. The unit weight of the concrete was determined using a 1/2-cubic foot steel measure according to ASTM C138-81, *Standard Test Method for Unit Weight, Yield, and Air Content of Concrete* and Texas SDHPT procedure Tex-417-A, *Weight per Cubic Foot and Yield of Concrete.*

3.6 Hardened Concrete Properties

3.6.1 Introduction. The hardened concrete properties tested throughout the experimental program included: a) flexural strength, b) Compressive strength, c) abrasion resistance, and d) resistance to deicing scaling. All the hardened concrete tests were performed according to applicable ASTM and Texas SDHPT specifications. When applicable, any variations in the test procedure is stated in detail.

3.6.2 Flexural Strength. The flexural strength test (modulus of rupture) was performed on 6 × 6 × 20-in. prisms loaded on an 18-in. span using a Rainhart Series 416 Beam Tester. The test was done according to ASTM C293-79, *Standard Test Method for Flexural Strength of Concrete Using Simple Beam with Center Point Loading* and Texas SDHPT procedure Tex-420-A, *Flexural Strength of Concrete.*

3.6.3 Compressive Strength. The compressive strength specimens were cast in 6 × 12-in. cylinder molds. The specimens were kept in their respective curing environment until testing. Compressive strength tests were performed at 14 and 28 days according to ASTM C39-84, *Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens* and Texas SDHPT procedure Tex-418-A, *Compressive Strength of Molded Concrete Cylinders* using a 600- kip capacity compression test machine. The test specimens were tested using reusable neoprene unbonded caps instead of the conventional high strength sulfur capping compound.

3.6.4 Abrasion Resistance. The abrasion resistance test was performed at 14 days on 6 × 6 × 6-in. specimens. The test was done according to ASTM C944-80, *Standard Test Method for Abrasion Resistance of Concrete or Mortar Surfaces by the Rotating Cutter Method.* The testing machine consisted of a drill press rotating at a speed of 200 rpm and exerting a force of 10 kgf (98N) on the surface being tested. The rotating cutter consisted of 24 grinding dressing wheels. The washers placed between the dressing wheels had a smaller diameter than the wheels. The use of a smaller diameter washer than that of the wheels was decided based on the results of a preliminary test program. Details of this test program are given in reference 15. A total of five 2-minute periods of abrasion were performed on every surface tested. The depth of wear on the surface of the specimens tested was measured to determine the abrasion resistance of the concrete. Figure 3.1 illustrates how the measurements were done. Measurements were made at the end of each 2- minute period at four different locations using a micrometer mounted on an Invar reference frame. The Invar reference frame was seated on demec gage points glued to the surface of the specimens prior to start of testing. The average of the four readings at each measuring time was used as the depth of wear for each specimen.

3.6.5 Scaling Resistance. The scaling test was performed at 28 days on $7\frac{1}{2} \times 13\frac{1}{2} \times$ 3-in. deep specimens according to ASTM C672-84, *Standard Test Method for Scaling Resistance of Concrete Surfaces Exposed to Deicing Chemicals.* The test specimens were cured according to the test curing conditions for 14 days after which they were stored at 73°F and 50 percent relative humidity for another 14 days before testing. At 28 days, a silicone rubber caulking dike was built around the edges of the specimens to hold the salt solution in place. The deicing agent used consisted of a 4 percent solution of calcium chloride. The scaling test was designed to measure the scaling resistance of concrete subjected to deicing salt solutions. At the end of test cycles, the chloride penetration beyond the surface of the concrete was determined using a commercially available *Cl Test System* manufactured by James Instruments, Inc. The test procedure consisted of drilling through the concrete surface and collecting samples of concrete powder at different depths. A measured sample of powder was then dissolved in a chloride extracting liquid. The chloride concentration of the concrete was determined using an activated electrode and an electrometer.

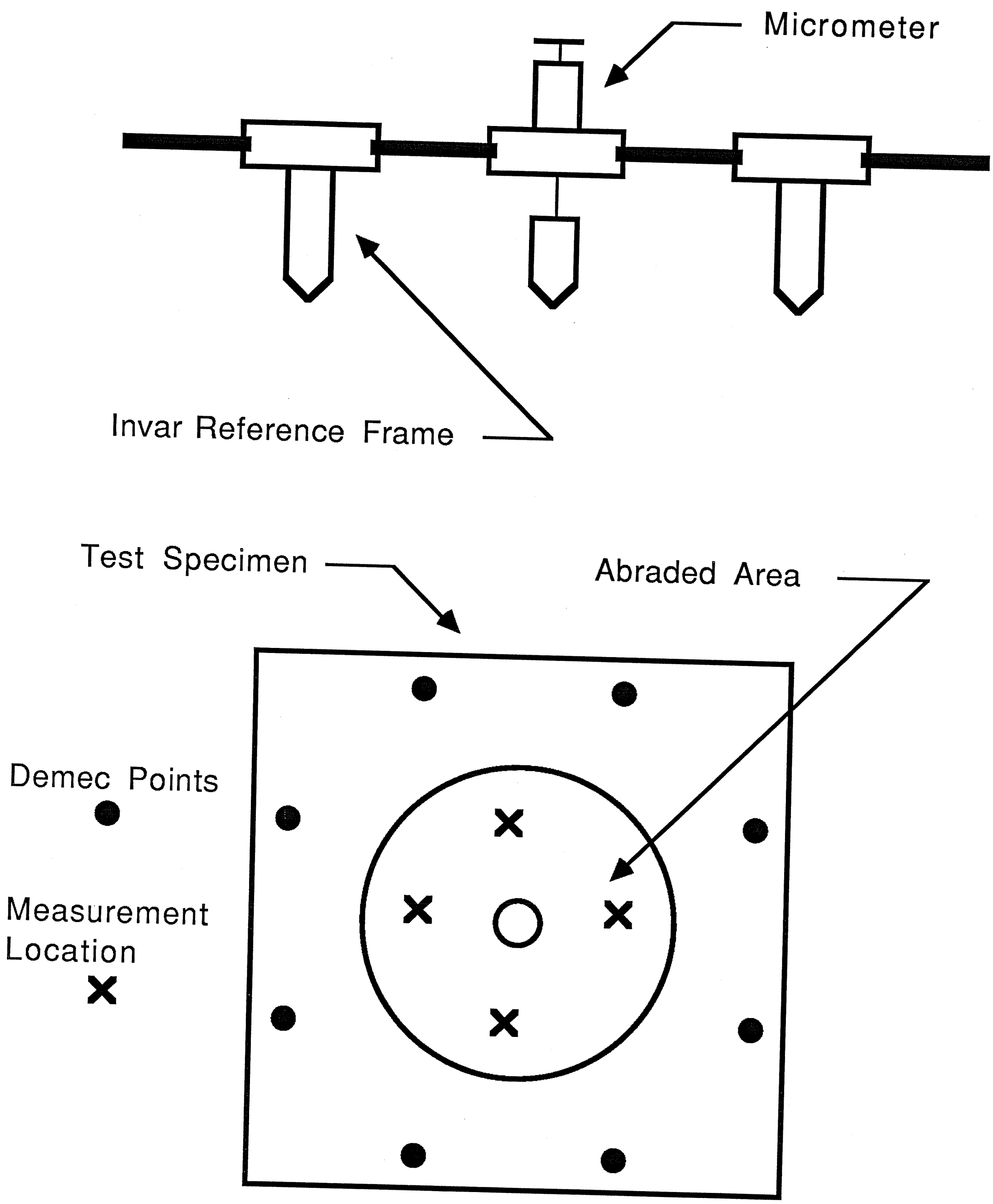

Fig. 3.1 Depth of penetration was used to measure the abrasion resistance of concrete.

CHAPTER 4
PRESENTATION OF THE TEST RESULTS

4.1 Introduction

The abrasion resistance and scaling resistance of concrete containing fly ash subjected to different curing conditions were studied in this experimental program. The test results are presented in this chapter both in tabular and graphical form to facilitate the ease in interpretation and understanding of the data.

The nomenclature of the various mixes used is as shown in Table 3.4. Table A.1 in Appendix A contains the mix proportions used in this study. The cementitious factor, type of fly ash used, percent replacement, and curing conditions are shown on all graphs.

4.2 Fresh Concrete Testing

Tables 4.1 to 4.3 present the values of the three fresh concrete properties tested for each batch. These include the slump, the unit weight, and the air content. In addition, the water to cementitious ratio [W/(C+FA), water to cement + fly ash by weight] is also shown. The slump of all the concrete mixes ranged between 1 in. and 3 1/4 in. The air content varied between 4.75 and 6.5 percent. When batching, the ambient temperature varied between 70 and 95°F with the majority of the mixes batched between 85 and 95°F.

4.3 Flexural Strength

The results of the 7-day flexural strength test are shown in Tables 4.4 to 4.6. Each test result represents the average value obtained by testing three companion beam specimens. Regardless of the curing condition of the concrete, the flexural strength specimens were always moist cured at 73°F until testing. The flexural strength test (modulus of rupture) was performed to determine whether the various concrete mixes complied with the Texas SDHPT specification requirements. All the mixes used in this study met the minimum required 7-day flexural strength.

4.4 Compressive Strength

The compressive strength test was performed at 14 and 28 days on two companion cylinders for each of the respective curing conditions. The 14-day test was done the day the abrasion resistance test was performed, while the 28-day test coincided with the start of the scaling resistance test. The results of the compressive strength test are illustrated in Figs. 4.1 to 4.21. The results of the mixes cured at 50°F are shown first followed by the mixes cured at 73°F followed by the mixes cured at 100°F.

4.5 Abrasion Resistance

Depth of penetration was used to determine the abrasion resistance of concrete. The test was performed at 14 days on three companion specimens. The test data is presented in Appendix A in Tables A.2 through A.4 and plotted in Figs. 4.22 to 4.42. Each graph illustrates the average depth of wear of concrete specimens made from the same batch and cured at the same temperatures but at different relative humidities.

Table 4.1 Fresh Concrete Properties for Mixes Cured at 50°F

Mix Designation	Slump (in.)	Unit Weight (lb/ft^3)	Air Content (%)	W/(C + FA) (by weight)
50-50 100-50	2.50	139	6.50	0.50
50-50A-25 100-50A-25	1.25	142	5.50	0.43
50-50B1-25 100-50B1-25	3.00	139	6.25	0.47
50-50B2-25 100-50B2-25	2.00	140	5.75	0.47
50-50A-35 100-50A-35	1.50	141	6.00	0.41
50-50B1-35 100-50B1-35	1.00	144	4.75	0.46
50-50B2-35 100-50B2-35	2.00	140	6.00	0.45

Table 4.2 Fresh Concrete Properties for Mixes Cured at 73°F

Mix Designation	Slump (in.)	Unit Weight (lb/ft^3)	Air Content (%)	W/(C + FA) (by weight)
50-73 100-73	1.75	141	5.75	0.49
50-73A-25 100-73A-25	1.50	141	5.75	0.44
50-73B1-25 100-73B1-25	1.50	141	6.00	0.51
50-73B2-25 100-73B2-25	2.00	141	6.00	0.45
50-73A-35 100-73A-35	2.00	143	6.00	0.43
50-73B1-35 100-73B1-35	3.25	141	5.25	0.46
50-73B2-35 100-73B2-35	2.00	141	5.50	0.46

Table 4.3 Fresh Concrete Properties for Mixes Cured at 100° F

Mix Designation	Slump (in.)	Unit Weight (lb/ft^3)	Air Content (%)	W/(C + FA) (by weight)
50-100 100-100	2.75	139	5.75	0.51
50-100A-25 100-100A-25	2.25	139	6.25	0.43
50-100B1-25 100-100B1-25	2.75	139	6.50	0.44
50-100B2-25 100-100B2-25	2.00	140	6.00	0.48
50-100A-35 100-100A-35	3.25	138	6.00	0.42
50-100B1-35 100-100B1-35	3.25	141	5.25	0.47
50-100B2-35 100-100B2-35	1.50	142	4.75	0.48

Table 4.4 Flexural Strength Test Results for Mixes Cured at 50°F

Mix Designation	Average Modulus of Rupture (psi)
50-50 100-50	776
50-50A-25 100-50A-25	775
50-50B1-25 100-50B1-25	678
50-50B2-25 100-50B2-25	705
50-50A-35 100-50A-35	694
50-50B1-35 100-50B1-35	715
50-50B2-35 100-50B2-35	677

Table 4.5 Flexural Strength Test Results for Mixes Cured at 73° F

Mix Designation	Average Modulus of Rupture (psi)
50-73 100-73	761
50-73A-25 100-73A-25	692
50-83B1-25 100-73B1-25	738
50-73B2-25 100-73B2-25	721
50-73A-35 100-73A-35	685
50-73B1-35 100-73B1-35	674
50-73B2-35 100-73B2-35	669

Table 4.6 Flexural Strength Test Results for Mixes Cured at 100°F

Mix Designation	Average Modulus of Rupture (psi)
50-100 100-100	685
50-100A-25 100-100A-25	650
50-100B1-25 100-100B1-25	718
50-100B2-25 100-100B2-25	722
50-100A-35 100-100A-35	682
50-100B1-35 100-100B1-35	669
50-100B2-35 100-100B2-35	659

4.6 Scaling Resistance

The scaling resistance test was started at 28 days on three companion specimens. Visual inspection of the test specimens was done after 5, 10, 15, 25, and 50 cycles. The damage was assessed according to ASTM C672-84 on a scale from 0 to 5 with zero indicating no deterioration and 5 indicating severe damage. Figures 4.43 through 4.63 illustrate the progress of deicing scaling observed for the concrete at the different stages of testing. In addition to visual inspection, a photograph of the specimens was taken at the end of the last test cycle.

4.7 Chloride Penetration

At the completion of the scaling resistance test the chloride penetration beyond the concrete surface was examined as previously explained in Chapter 3. The chloride concentration was determined at depths of 0 to 1/4 in., 1/2 to 3/4 in., and 1 to 1-1/4 in. Dust samples were collected at each depth from three different locations on the surface of the specimens. The chloride penetration results are shown in Figs. 4.64 to 4.84.

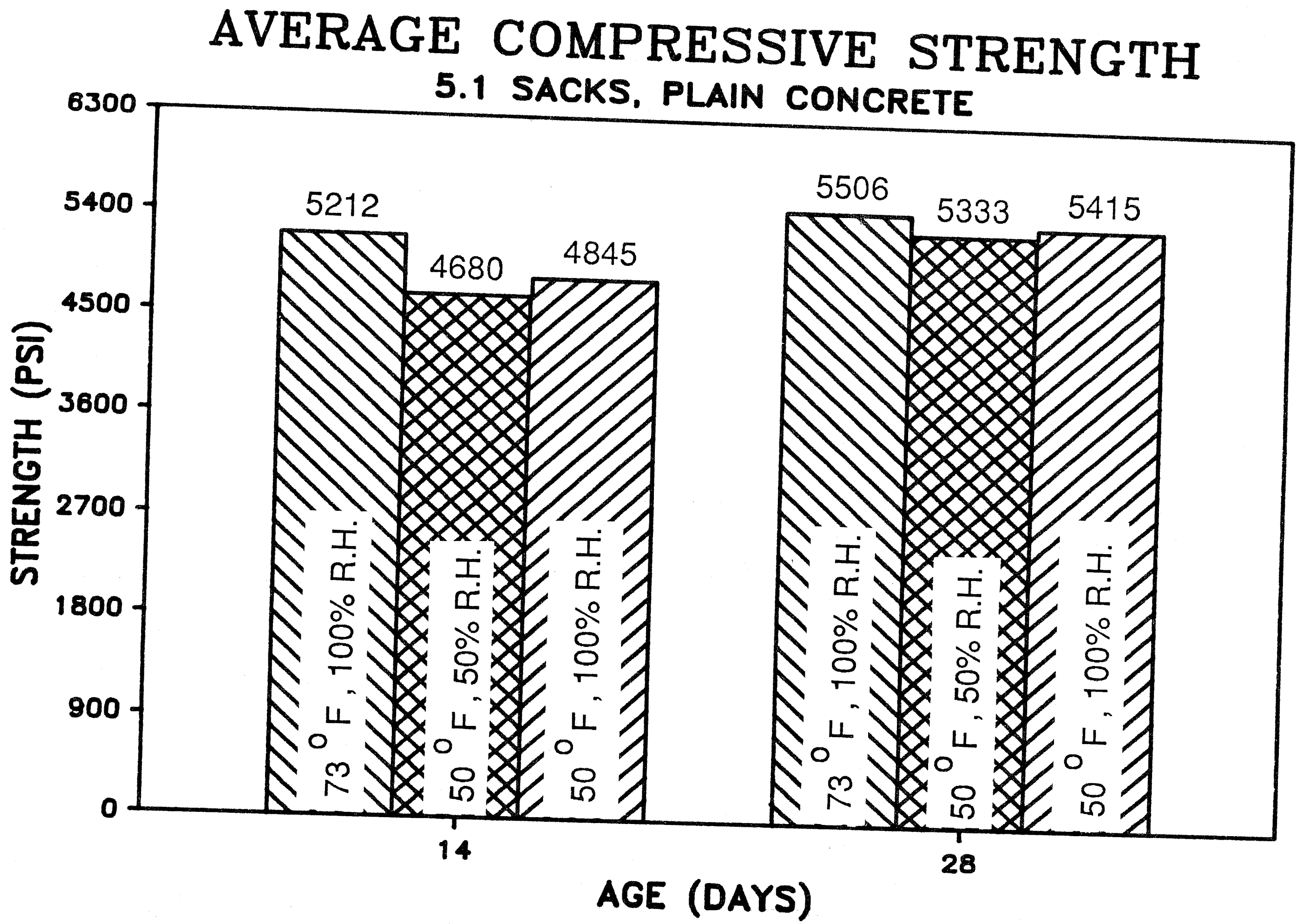

Fig. 4.1 Compressive strength results for the 50-50 and 100-50 mixes.

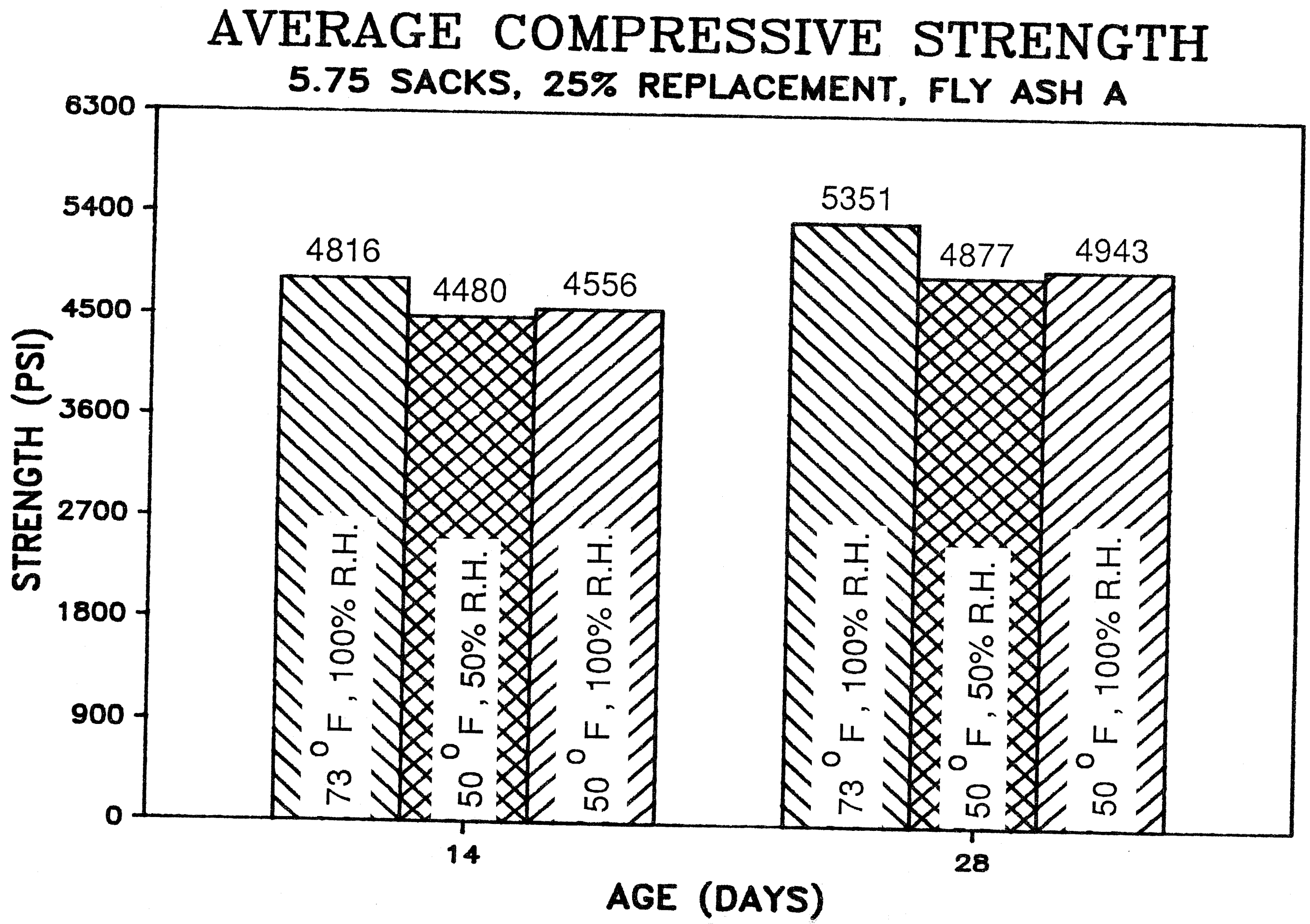

Fig. 4.2 Compressive strength results for the 50-50A-25 and 100-50A-25 mixes.

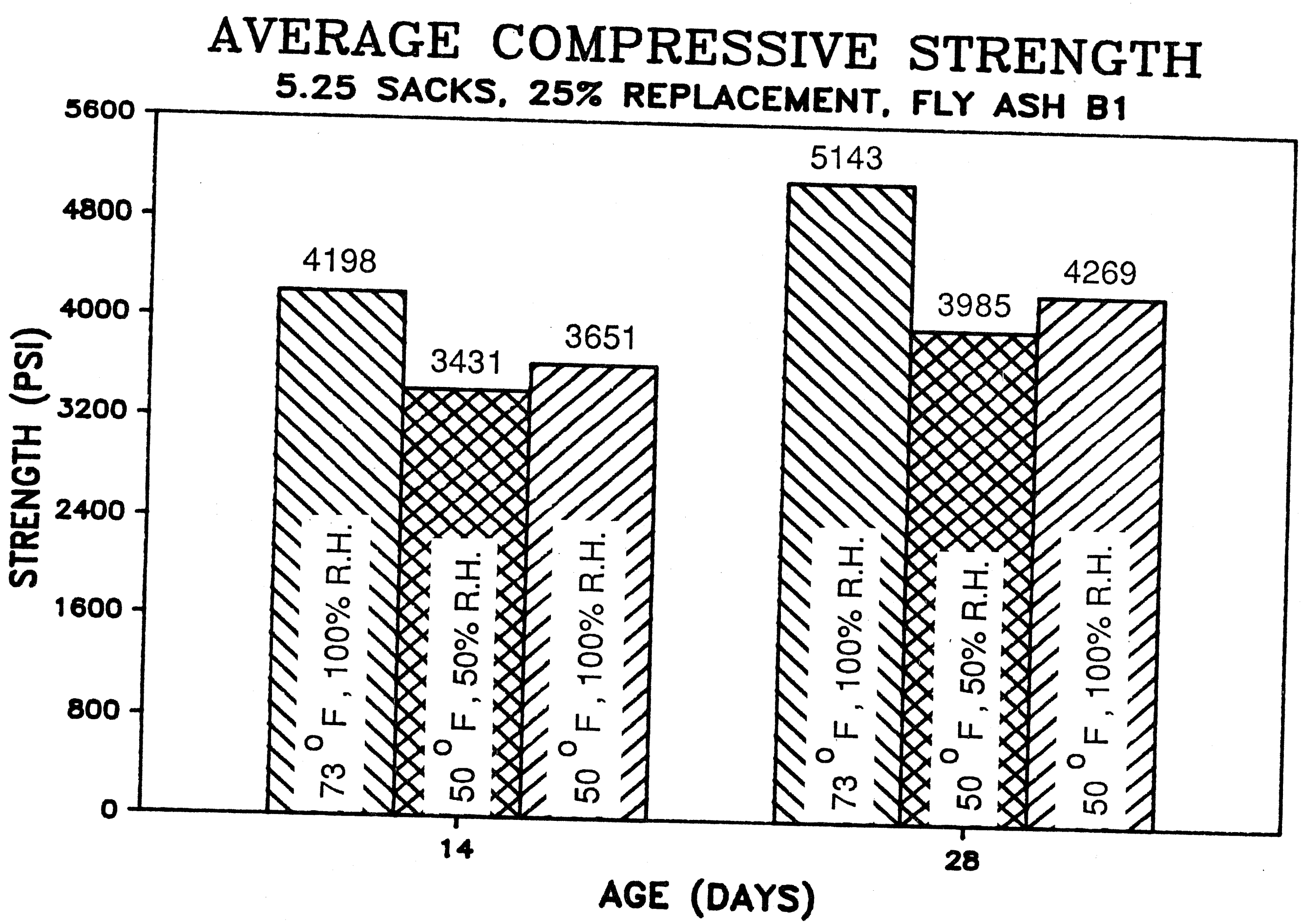

Fig. 4.3 Compressive strength results for the 50-50B1-25 and 100-50B1-25 mixes.

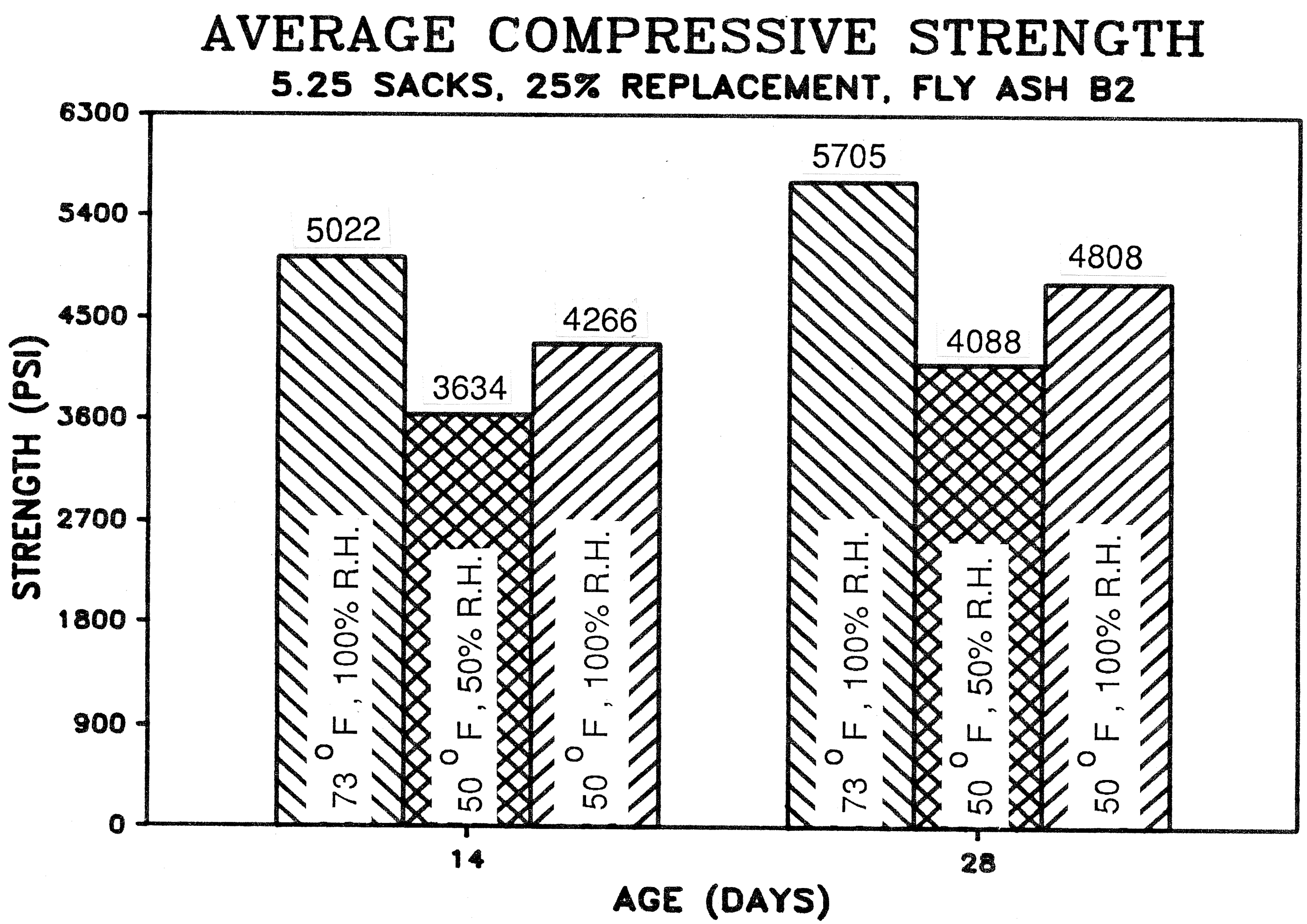

Fig. 4.4 Compressive strength results for the 50-50B2-25 and 100-50B2-25 mixes.

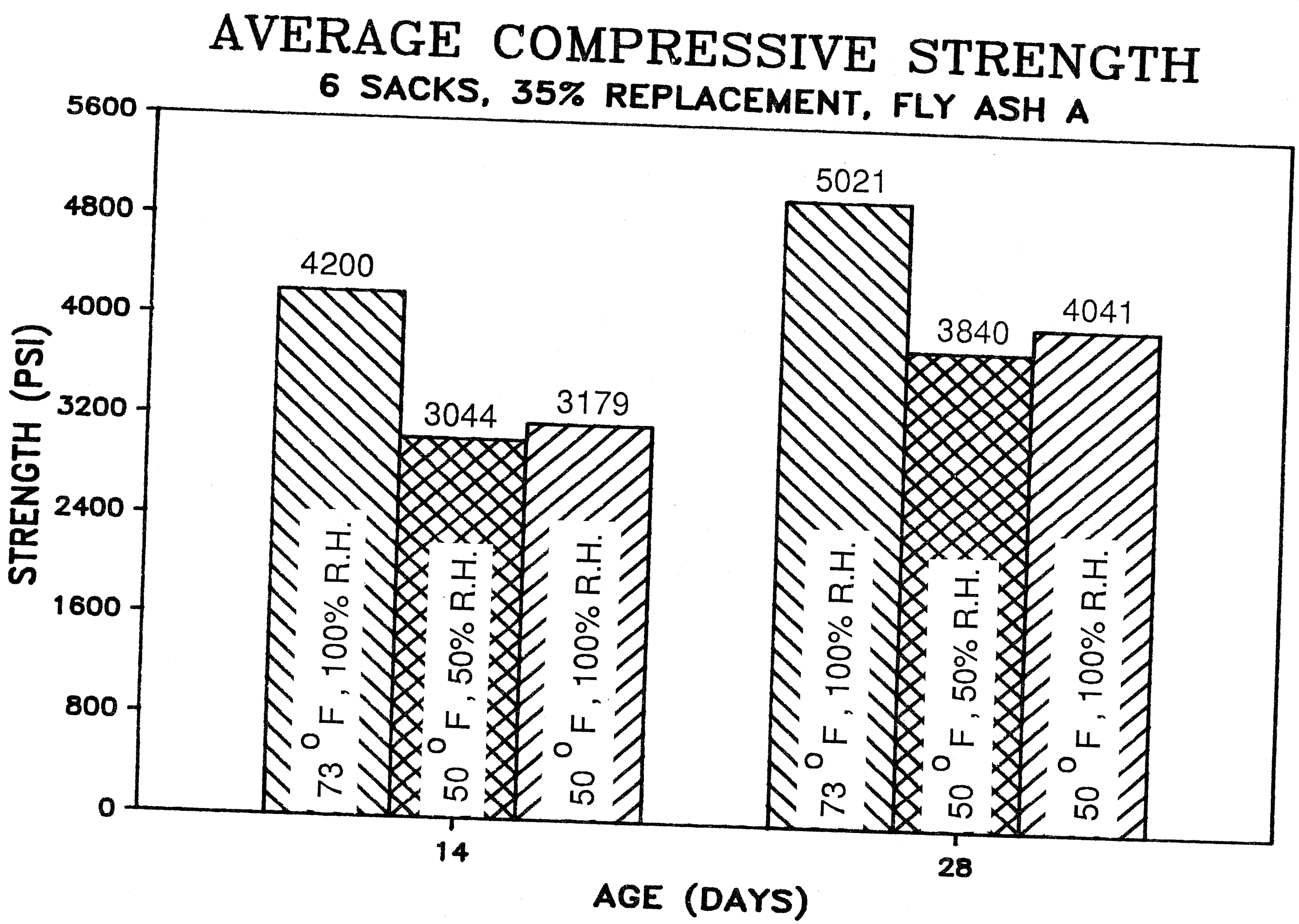

Fig. 4.5 Compressive strength results for the 50-50A-35 and 100- 50A-35 mixes.

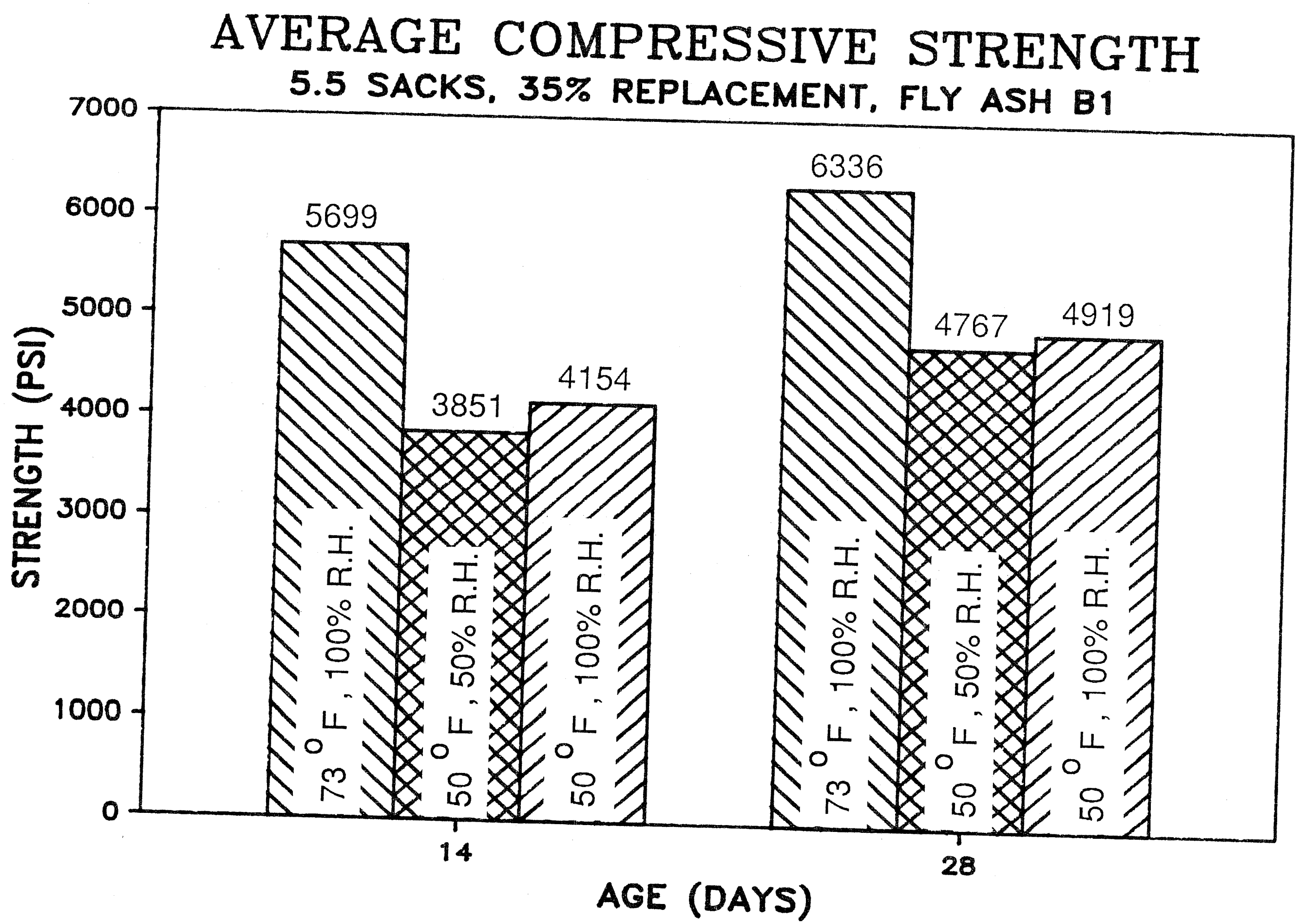

Fig. 4.6 Compressive strength results for the 50-50B1-35 and 100-50B1-35 mixes.

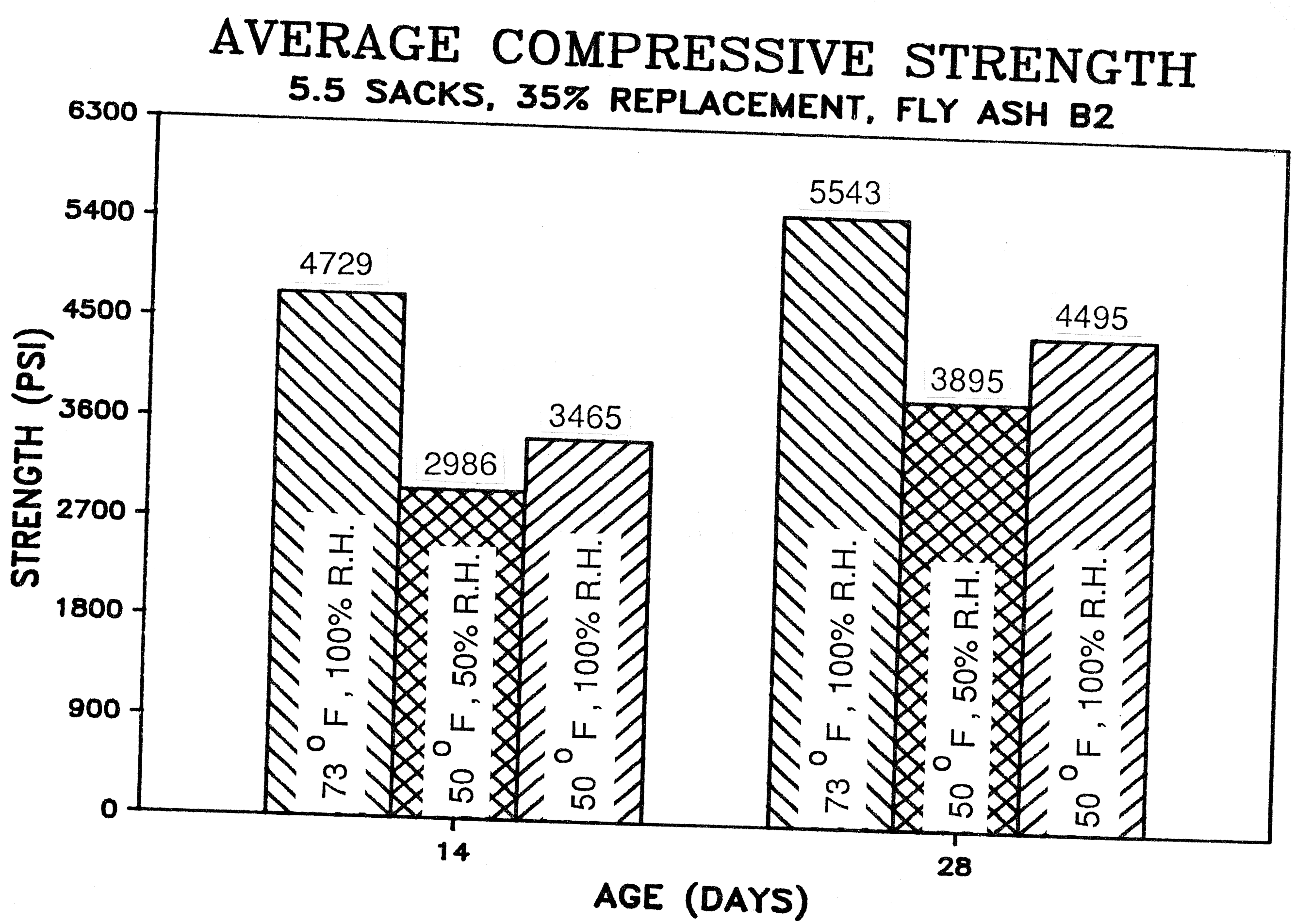

Fig. 4.7 Compressive strength results for the 50-50B2-35 and 100-50B2-35 mixes.

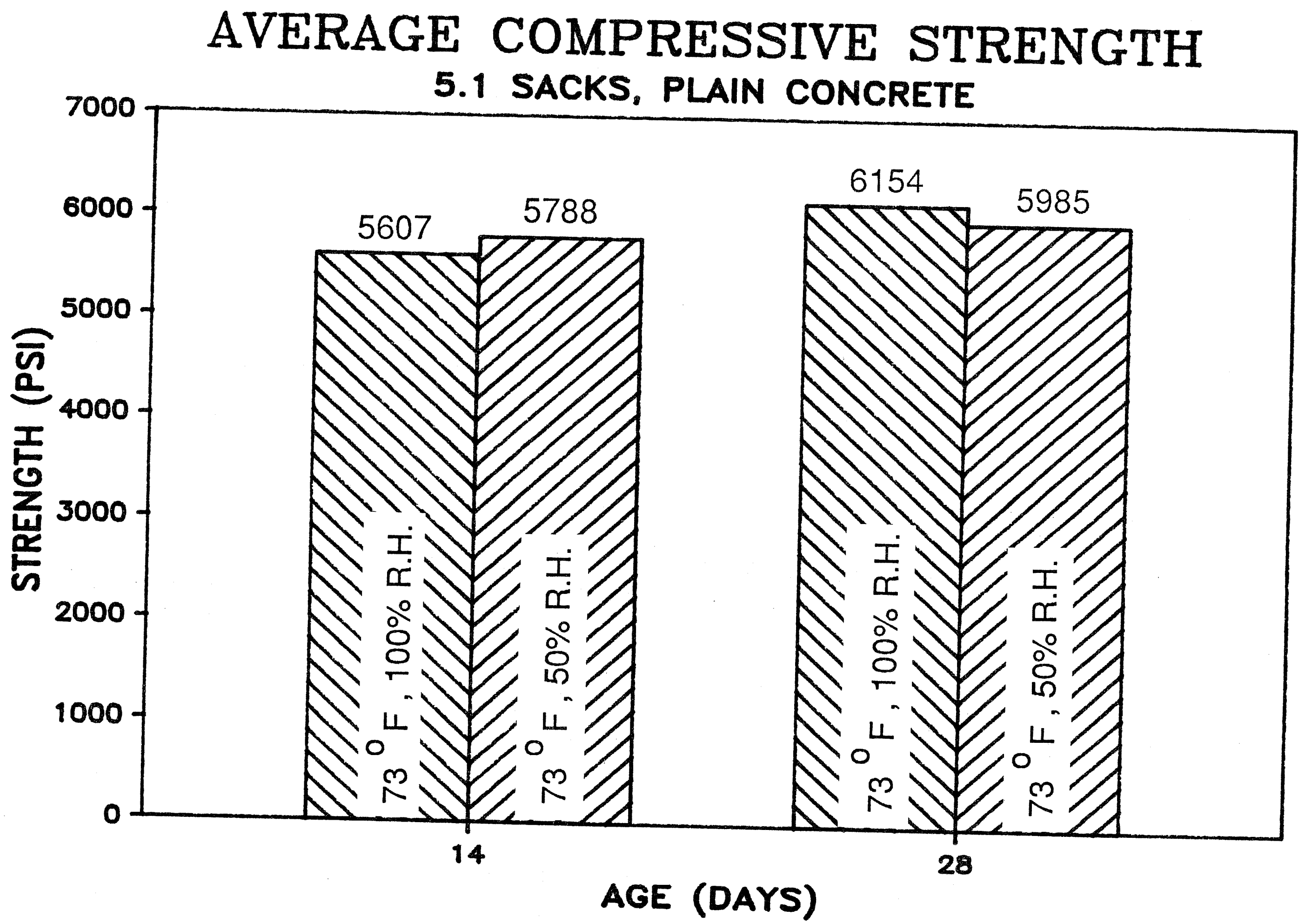

Fig. 4.8 Compressive strength results for the 50-73 and 100-73 mixes.

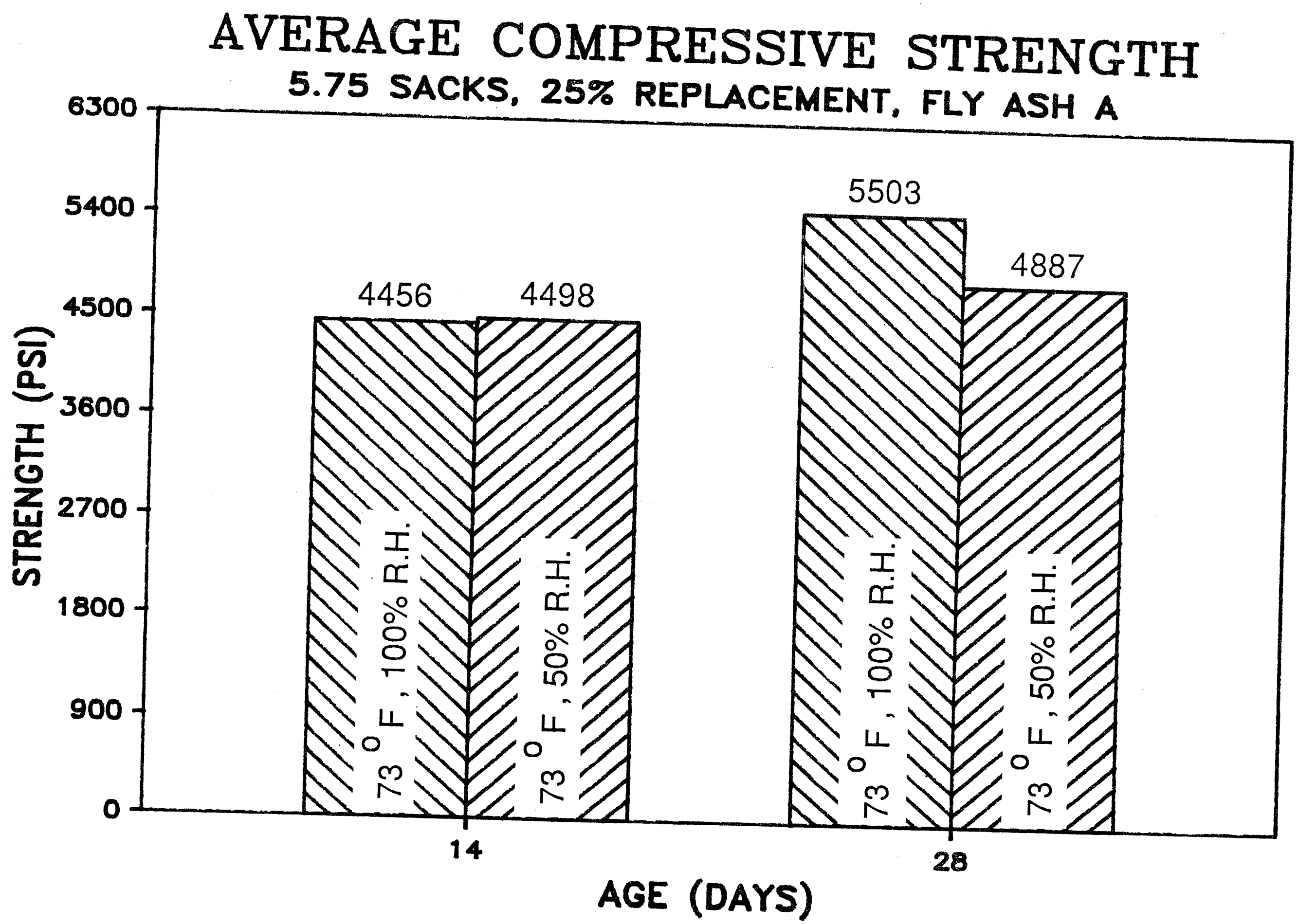

Fig. 4.9 Compressive strength results for the 50-73A-25 and 100-73A-25 mixes.

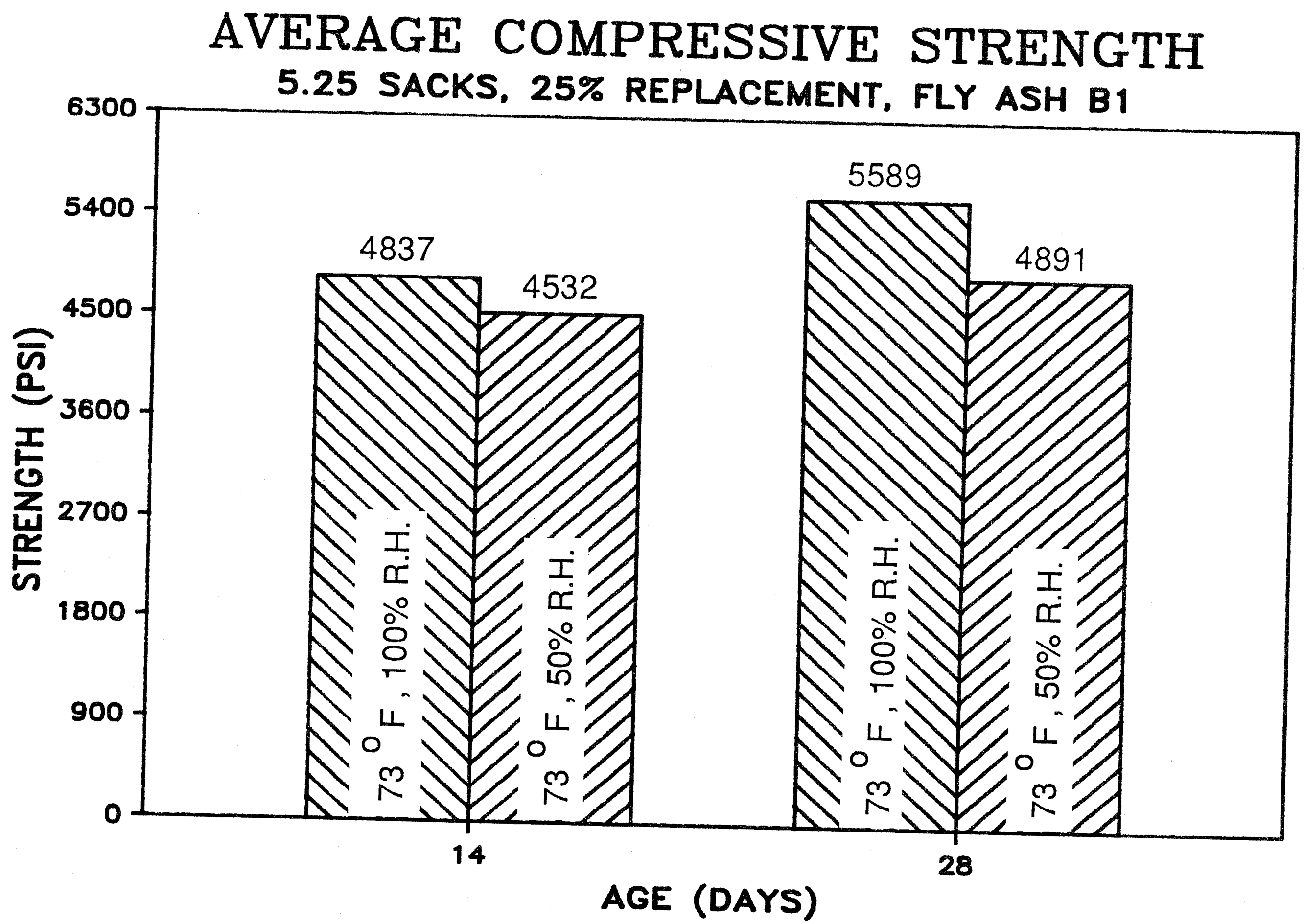

Fig. 4.10 Compressive strength results for the 50-73B1-25 and 100-73B1-25 mixes.

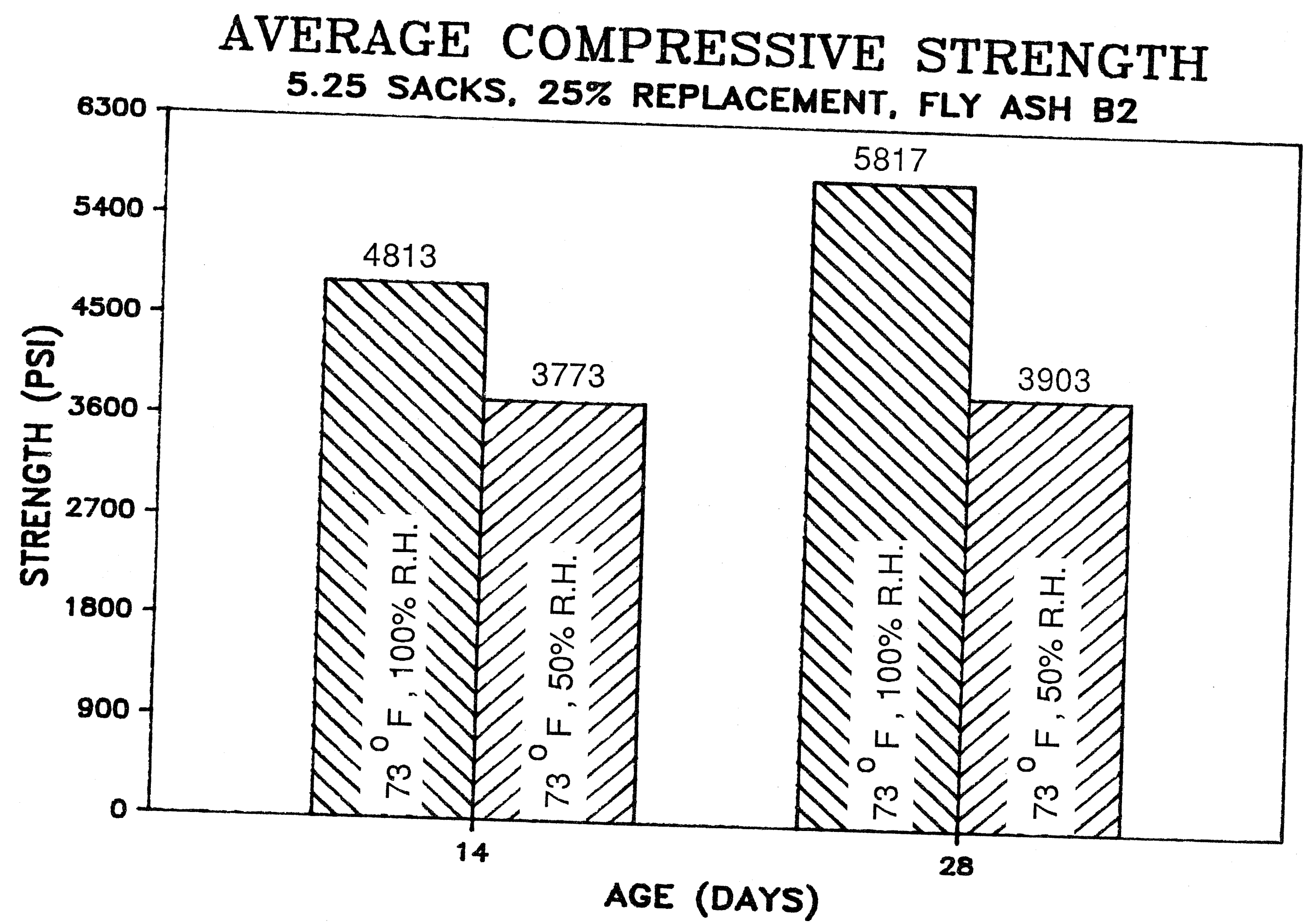

Fig. 4.11 Compressive strength results for the 50-73B2-25 and 100-73B2-25 mixes.

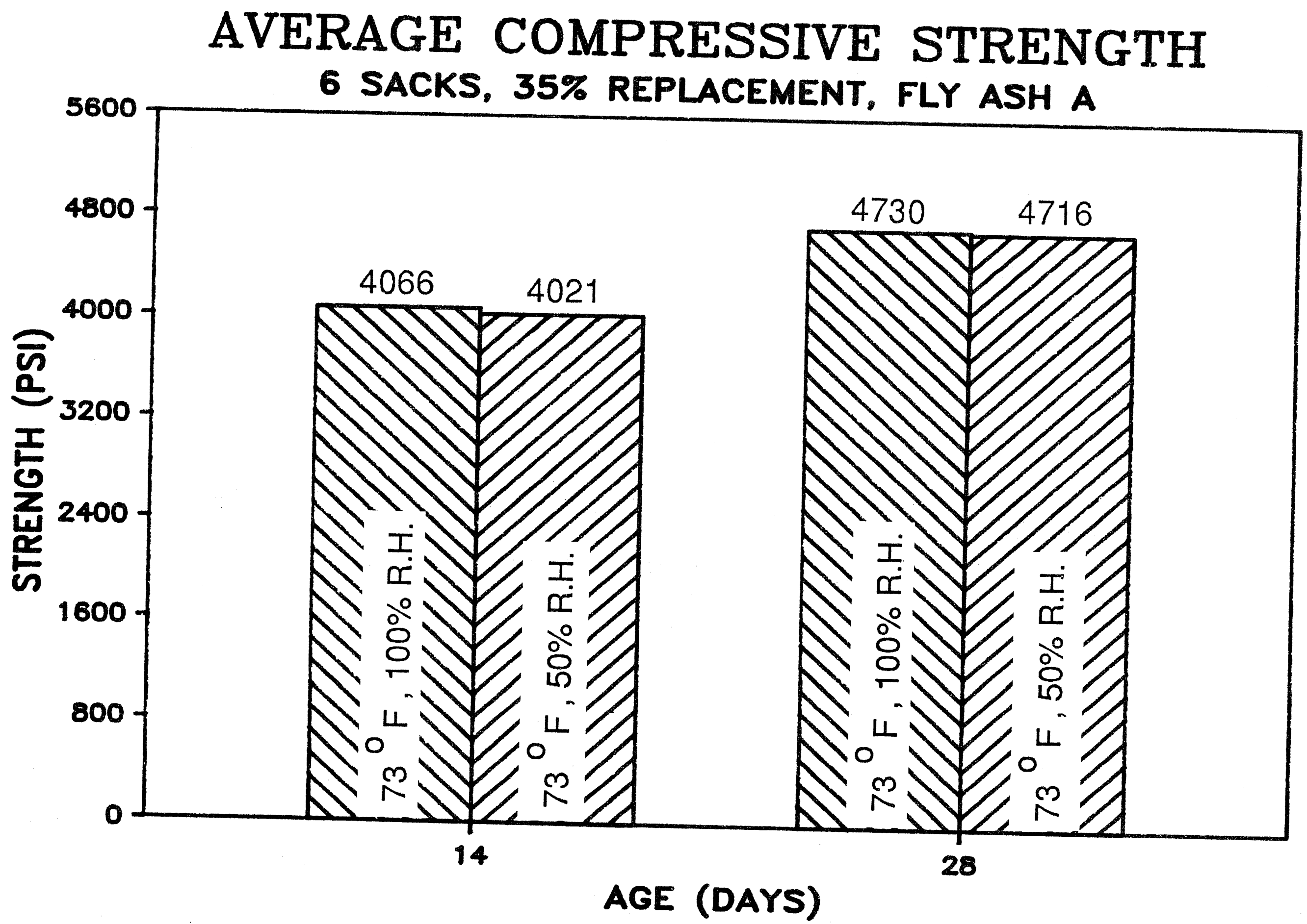

Fig. 4.12 Compressive strength results for the 50-73A-35 and 100-73A-35 mixes.

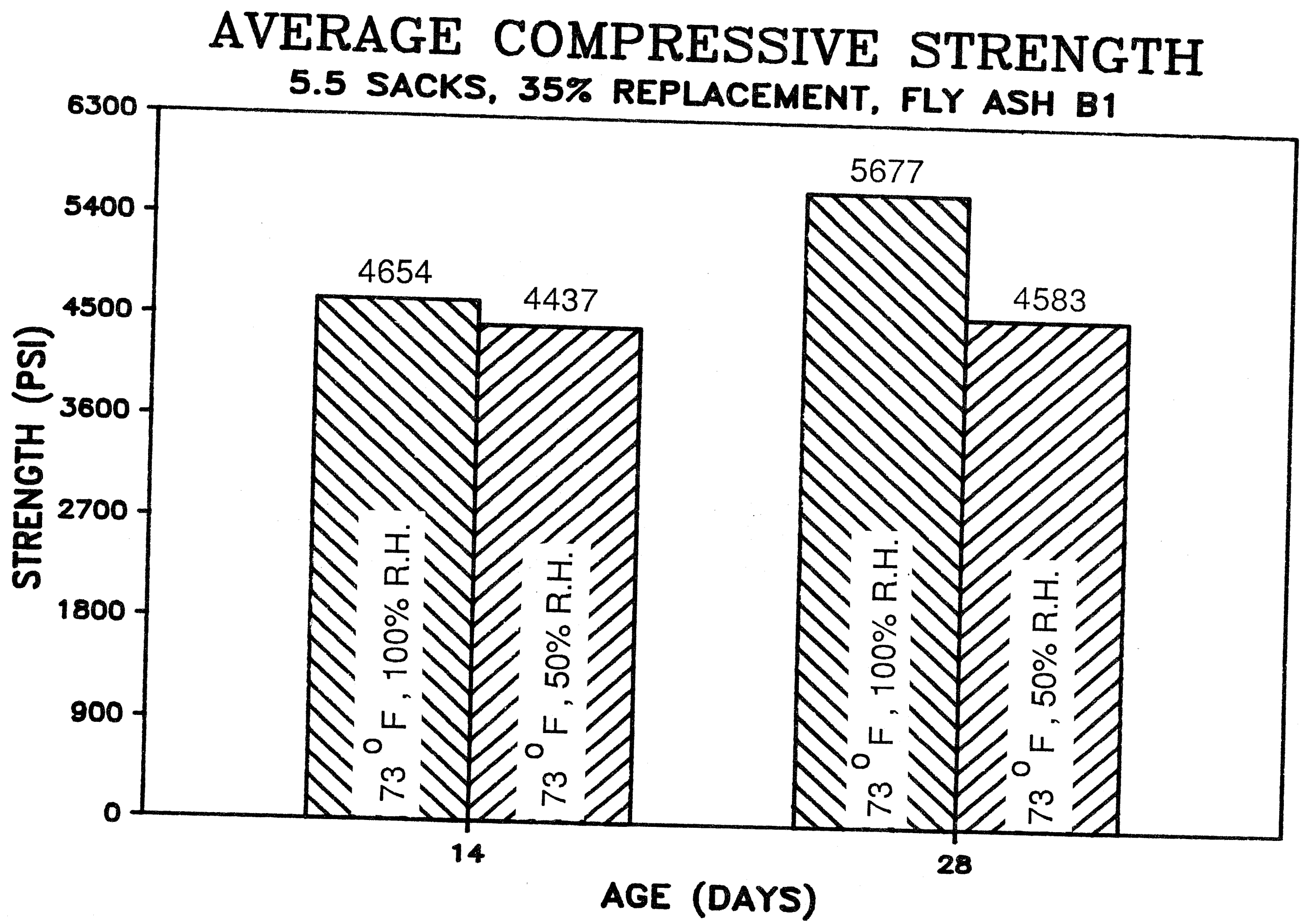

Fig. 4.13 Compressive strength results for the 50-73B1-35 and 100-73B1-35 mixes.

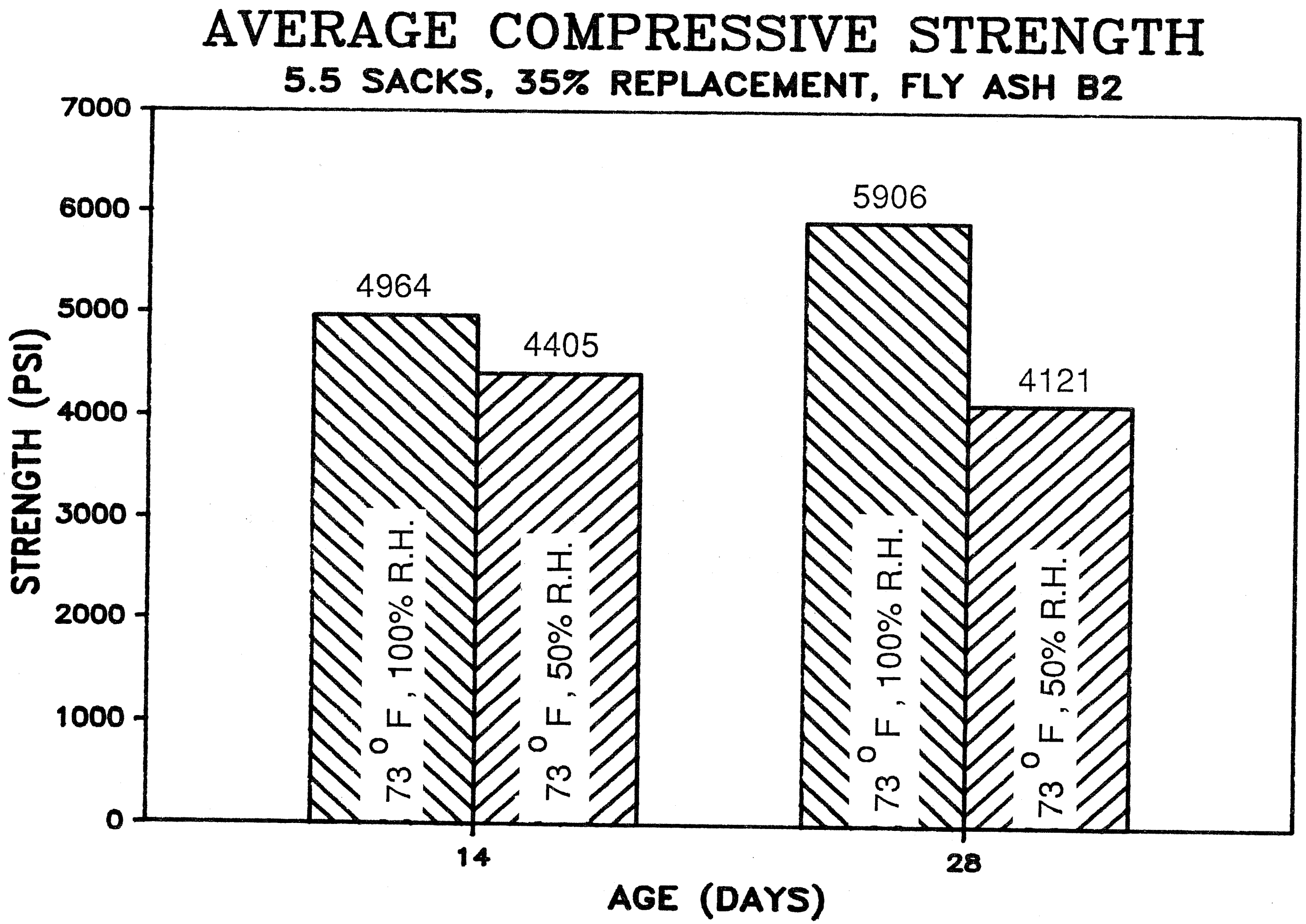

Fig. 4.14 Compressive strength results for the 50-73B2-35 and 100-73B2-35 mixes.

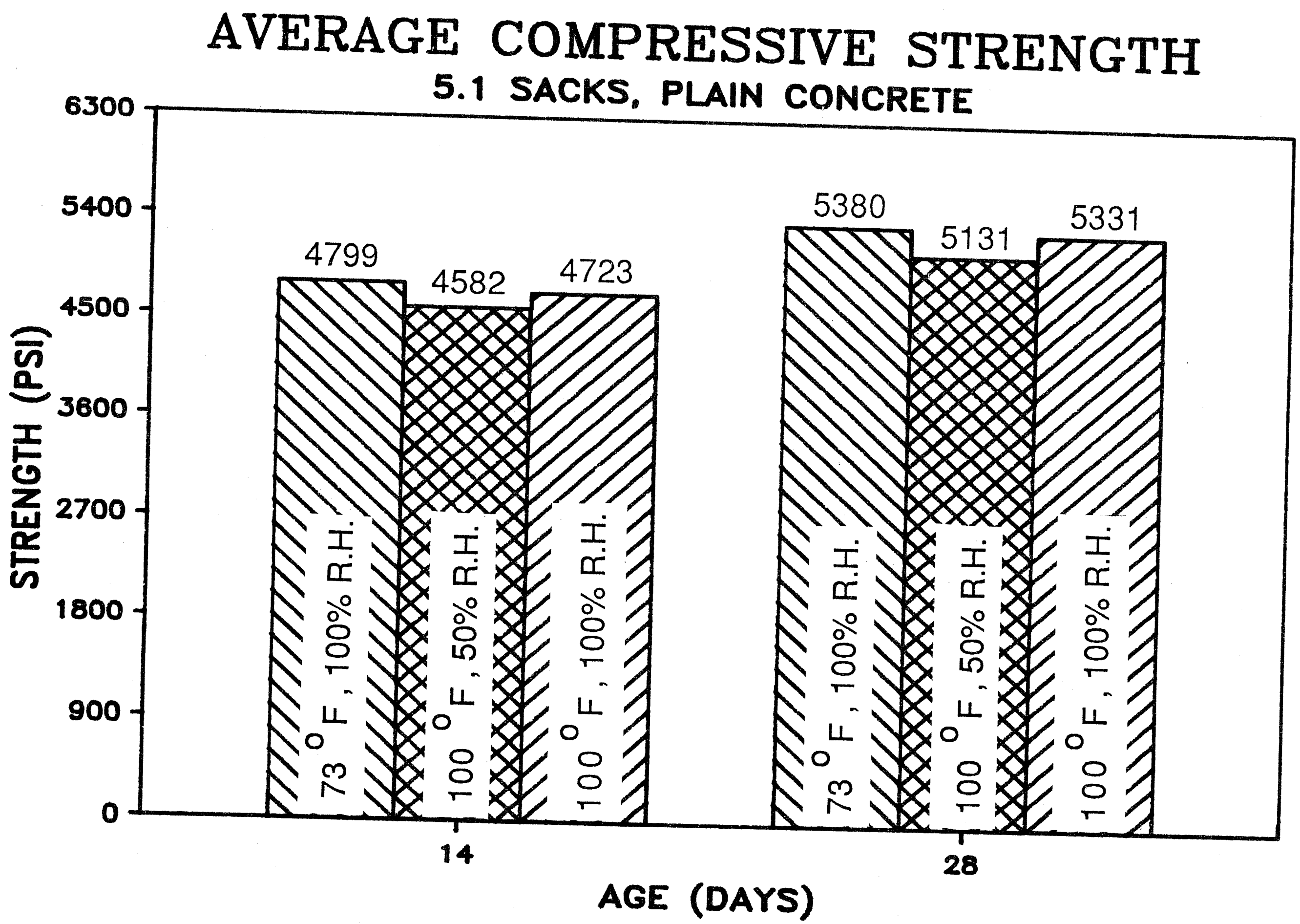

Fig. 4.15 Compressive strength results for the 50-100 and 100-100 mixes.

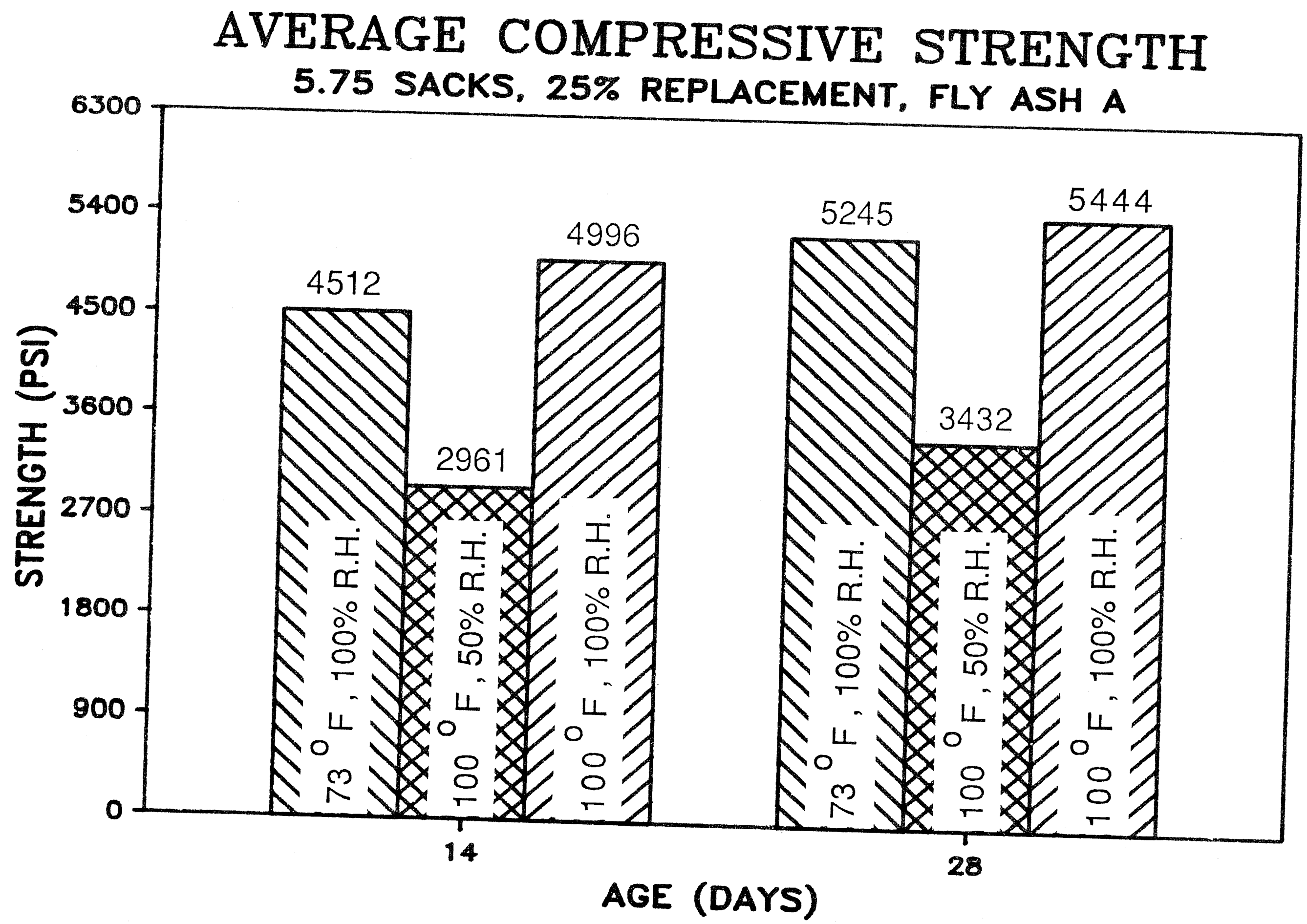

Fig. 4.16 Compressive strength results for the 50-100A-25 and 100-100A-25 mixes.

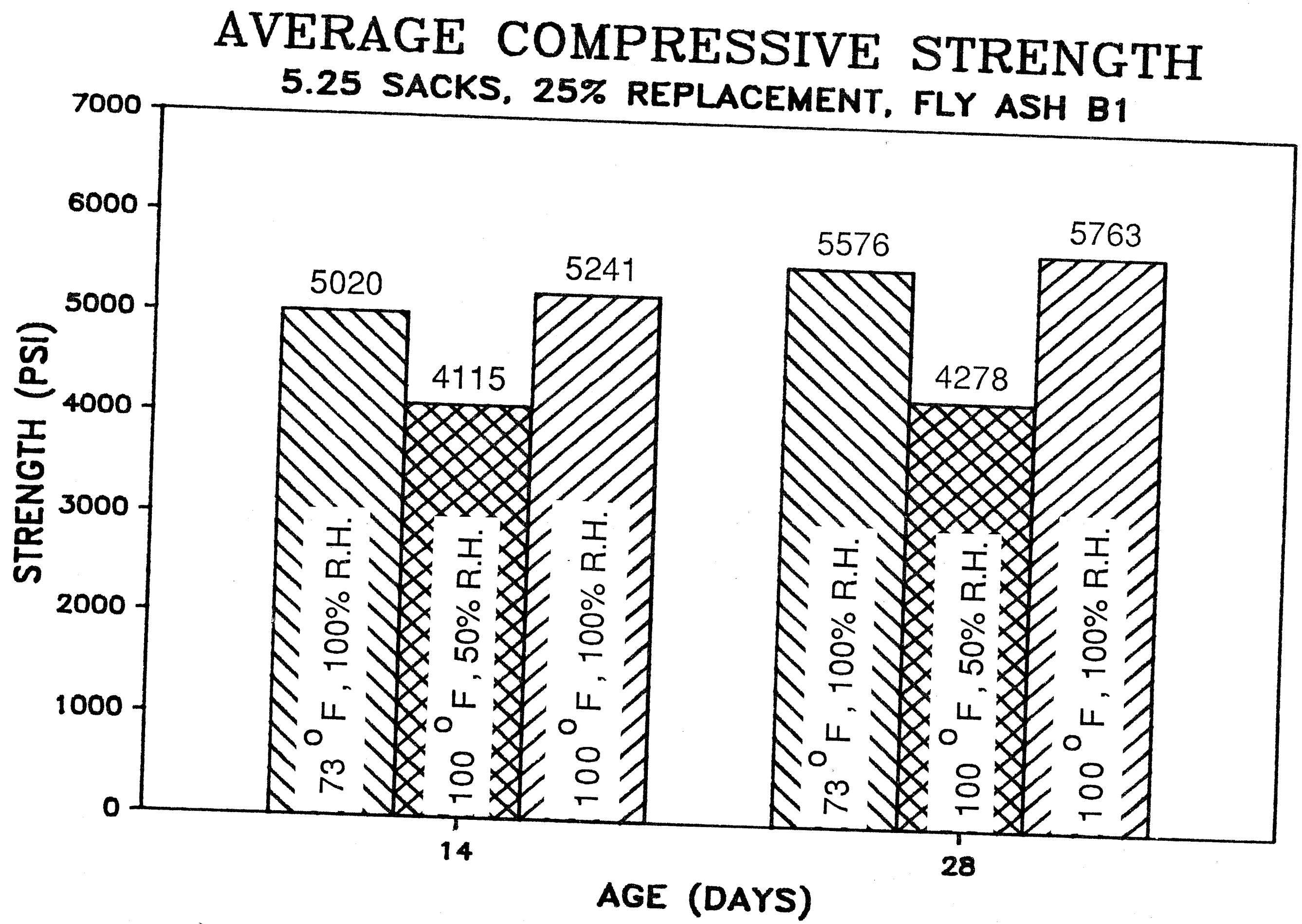

Fig. 4.17 Compressive strength results for the 50-100B1-25 and 100-100B1-25 mixes.

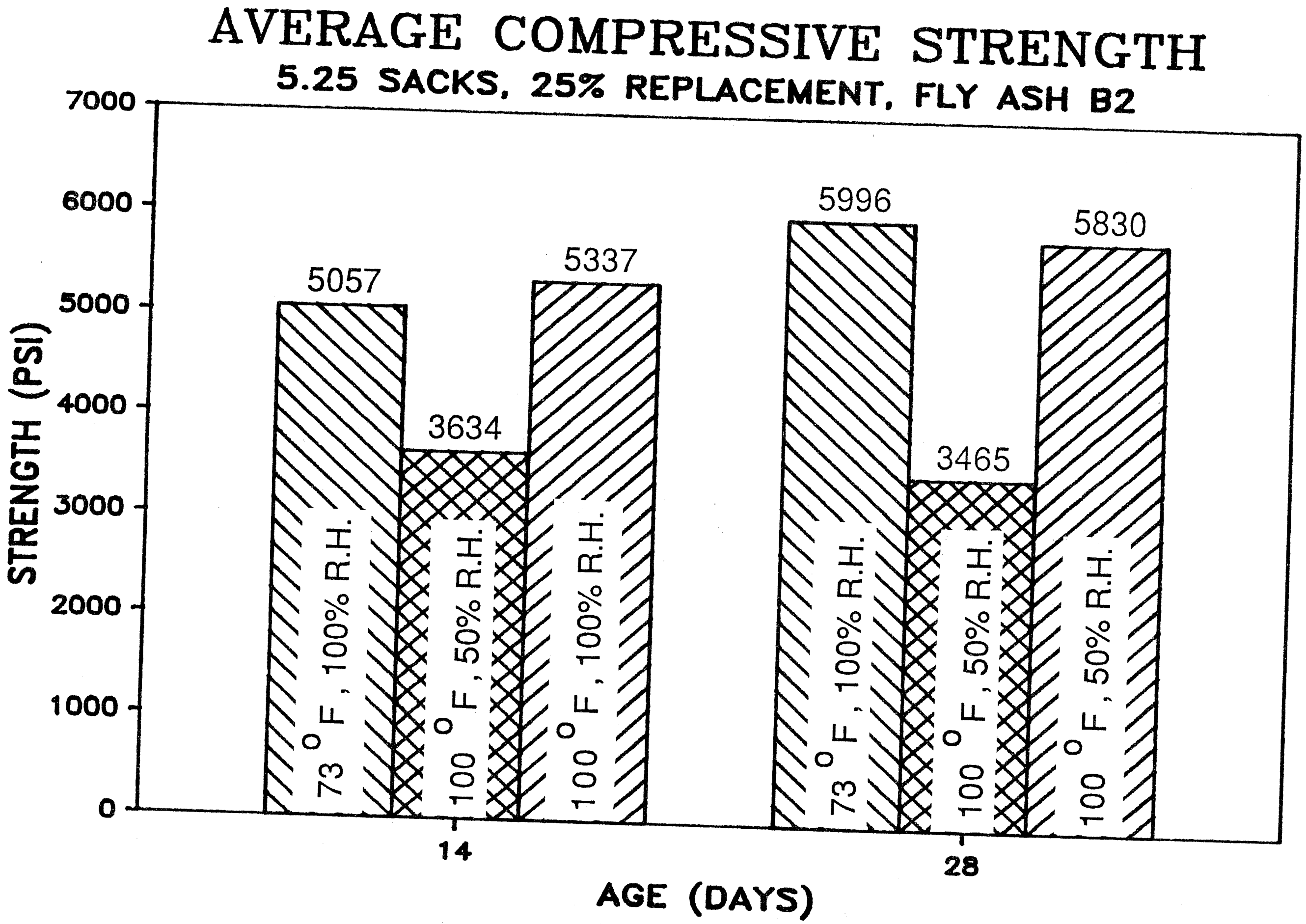

Fig. 4.18 Compressive strength results for the 50-100B2-25 and 100-100B2-25 mixes.

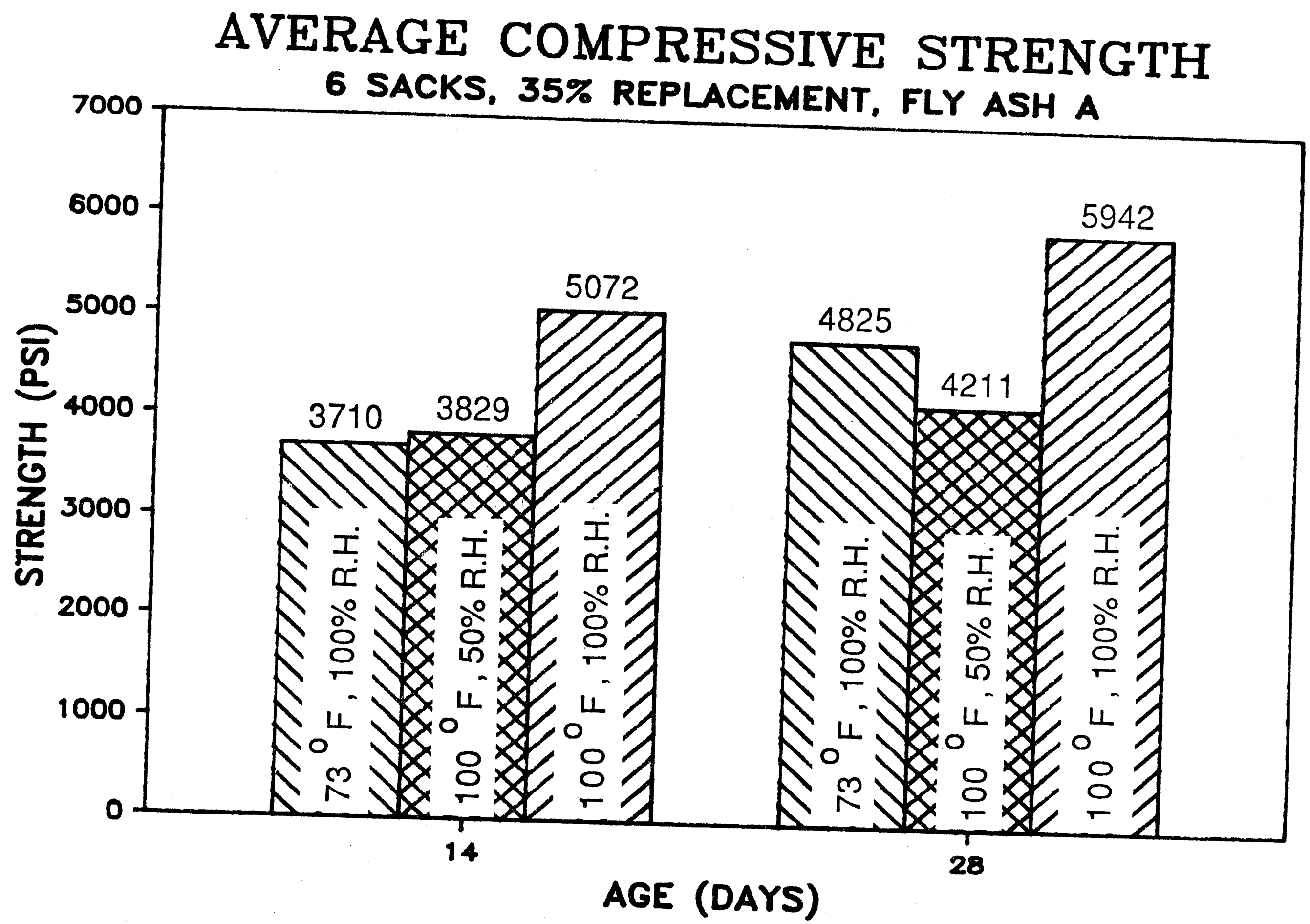

Fig. 4.19 Compressive strength results for the 50-100A-35 and 100-100A-35 mixes.

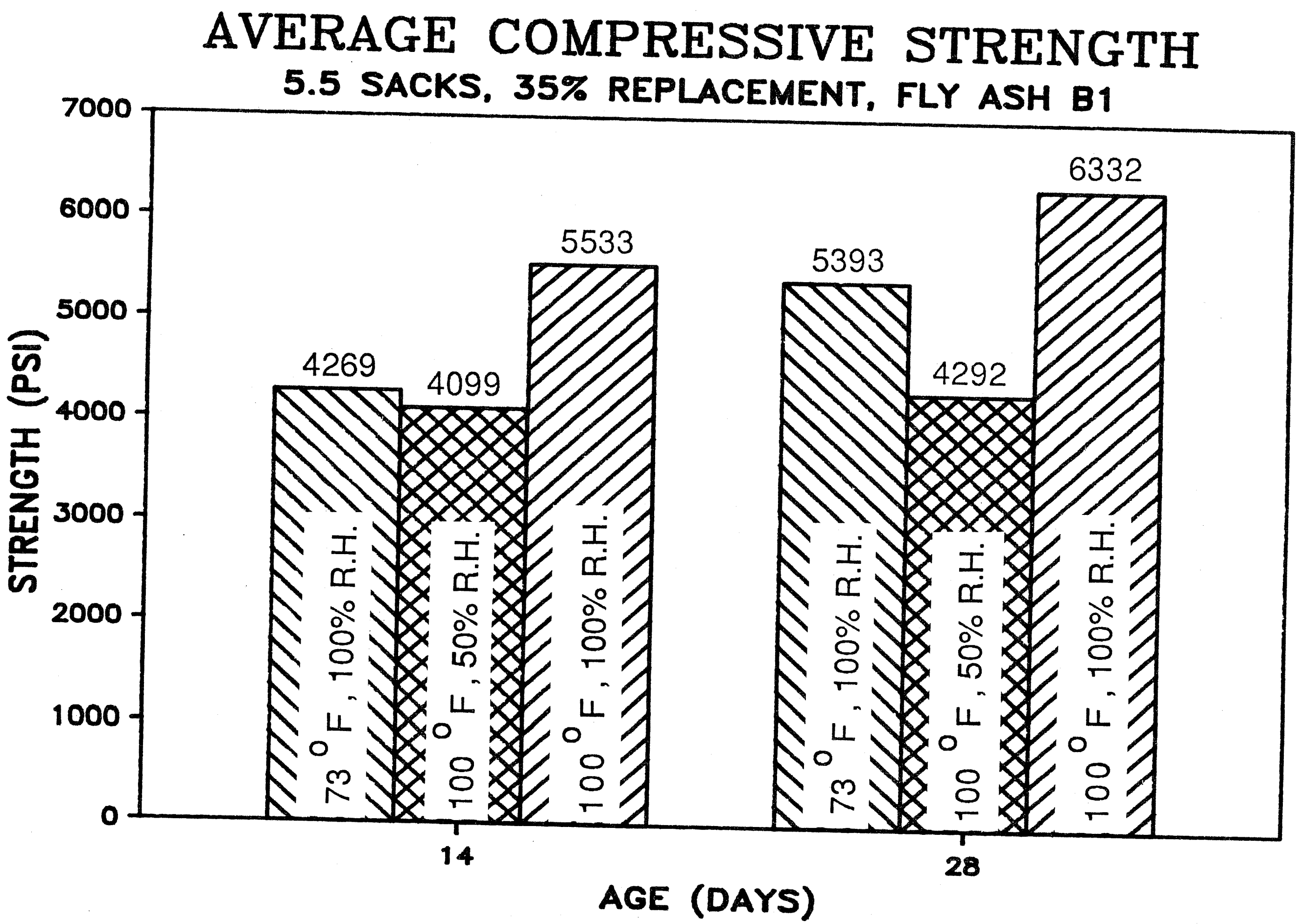

Fig. 4.20 Compressive strength results for the 50-100B1-35 and 100-100B1-35 mixes.

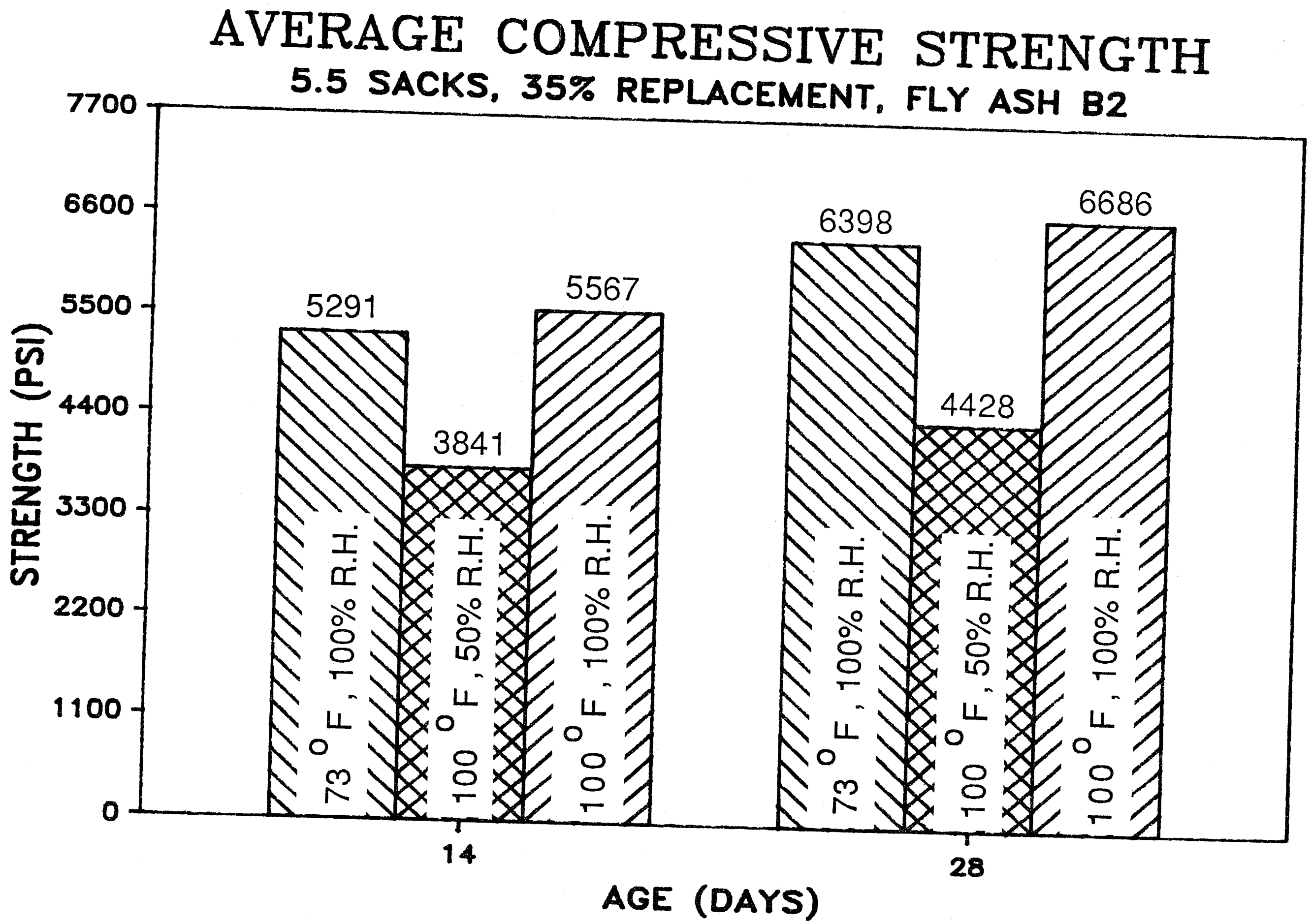

Fig. 4.21 Compressive strength results for the 50-100B2-35 and 100-100B2-35 mixes.

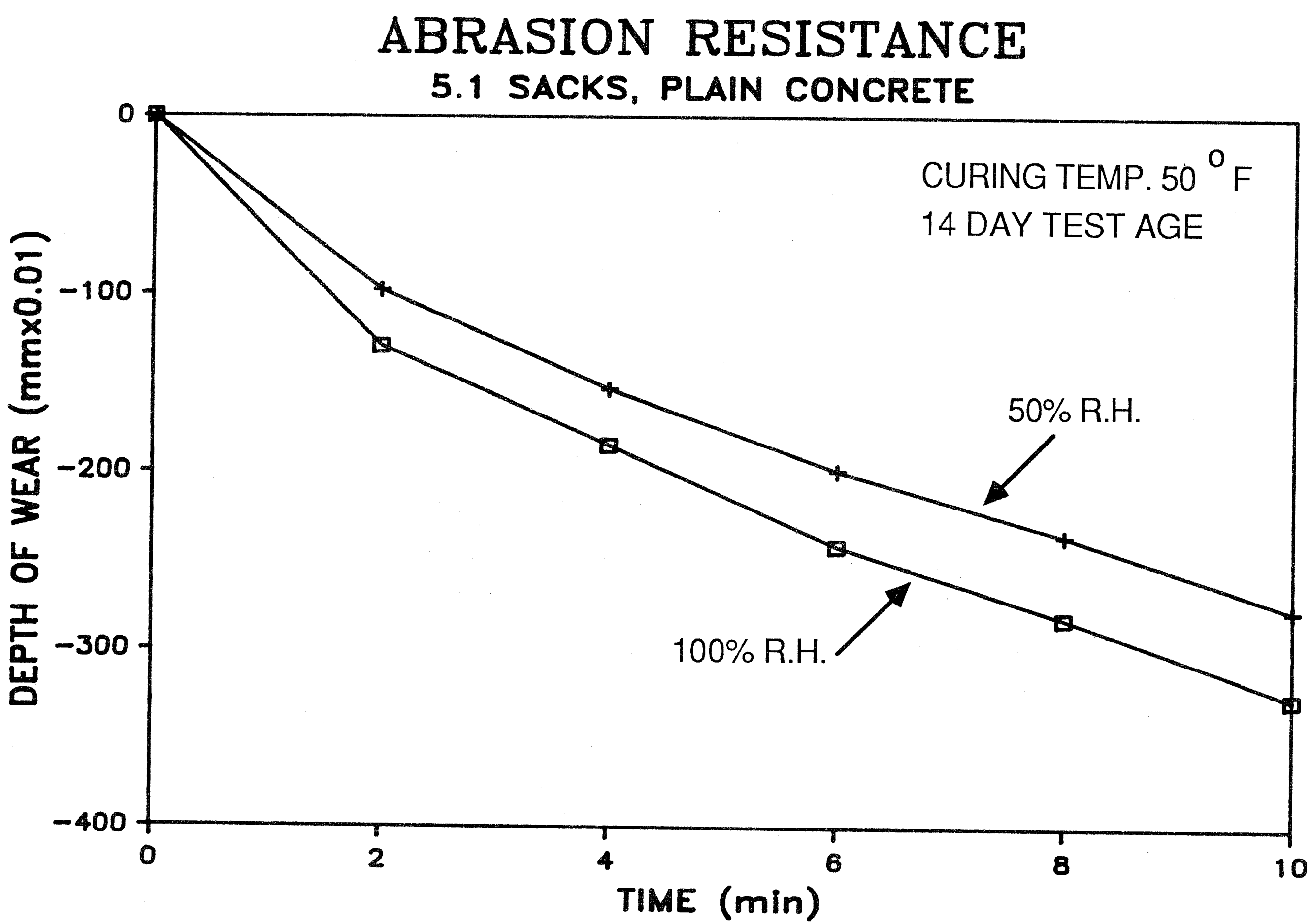

Fig. 4.22 Abrasion resistance of concrete for the 50-50 and 100- 50 mixes.

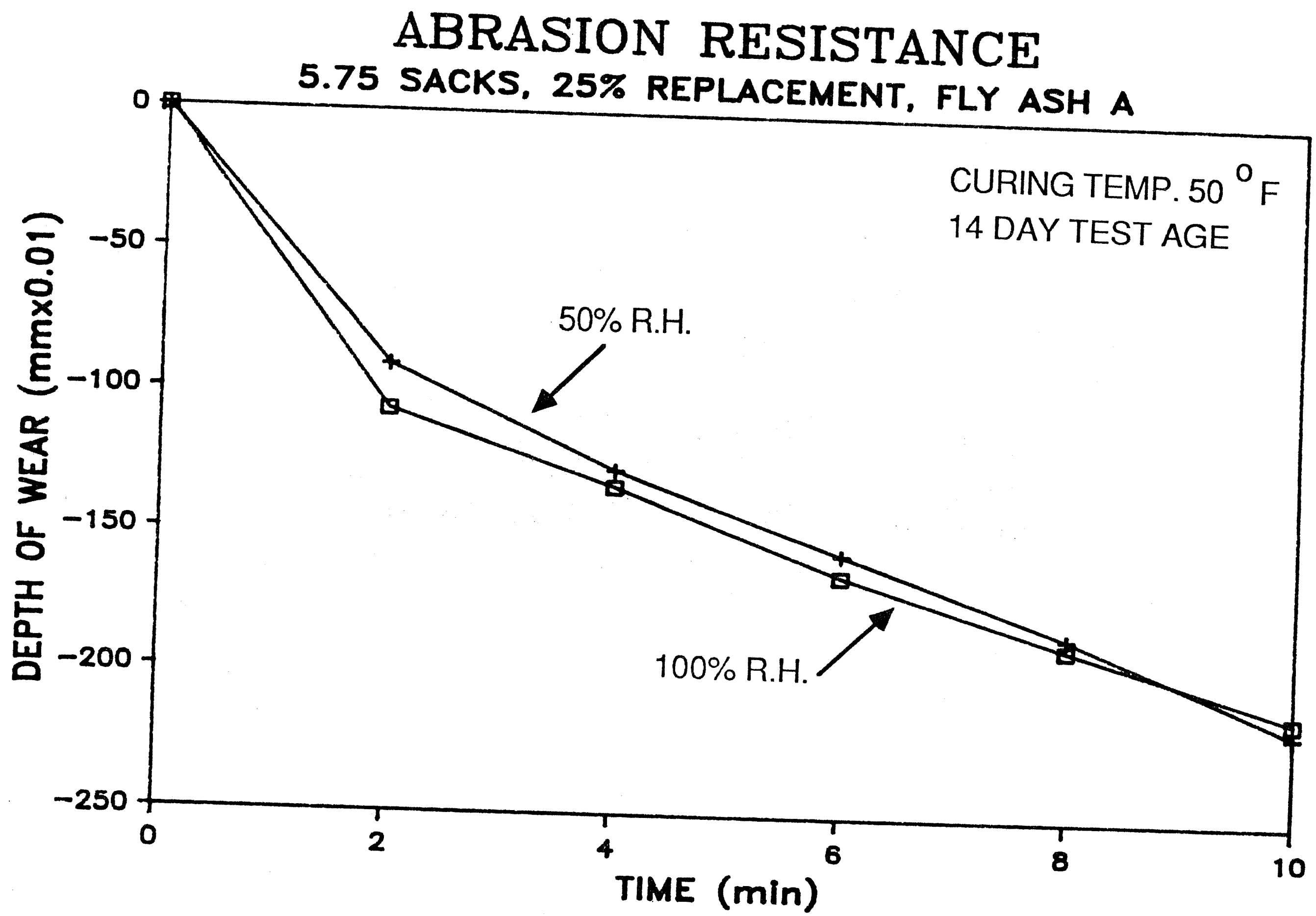

Fig. 4.23 Abrasion resistance of concrete for the 50-50A-25 and 100-50A-25 mixes.

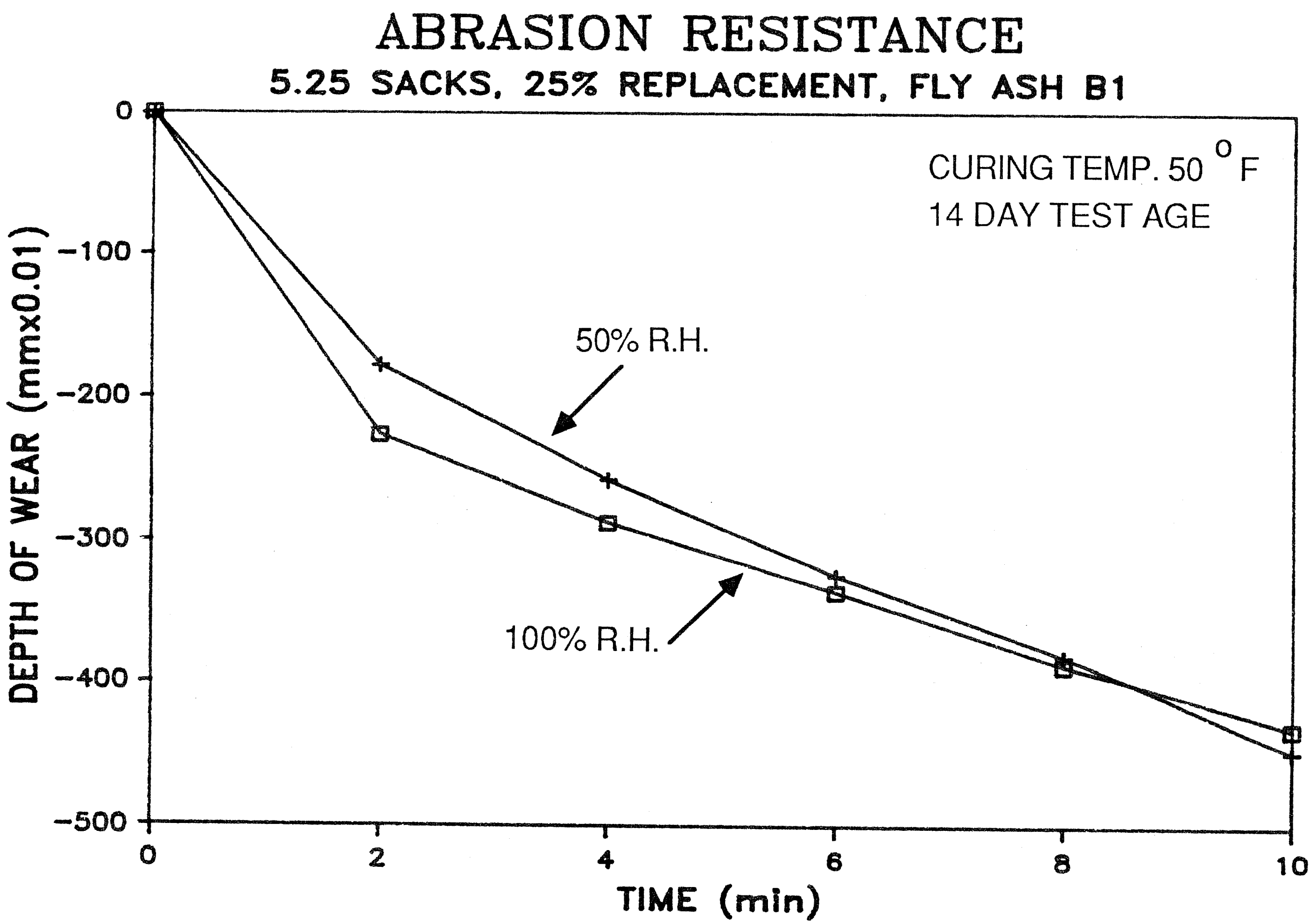

Fig. 4.24 Abrasion resistance of concrete for the 50-50B1-25 and 100-50B1-25 mixes.

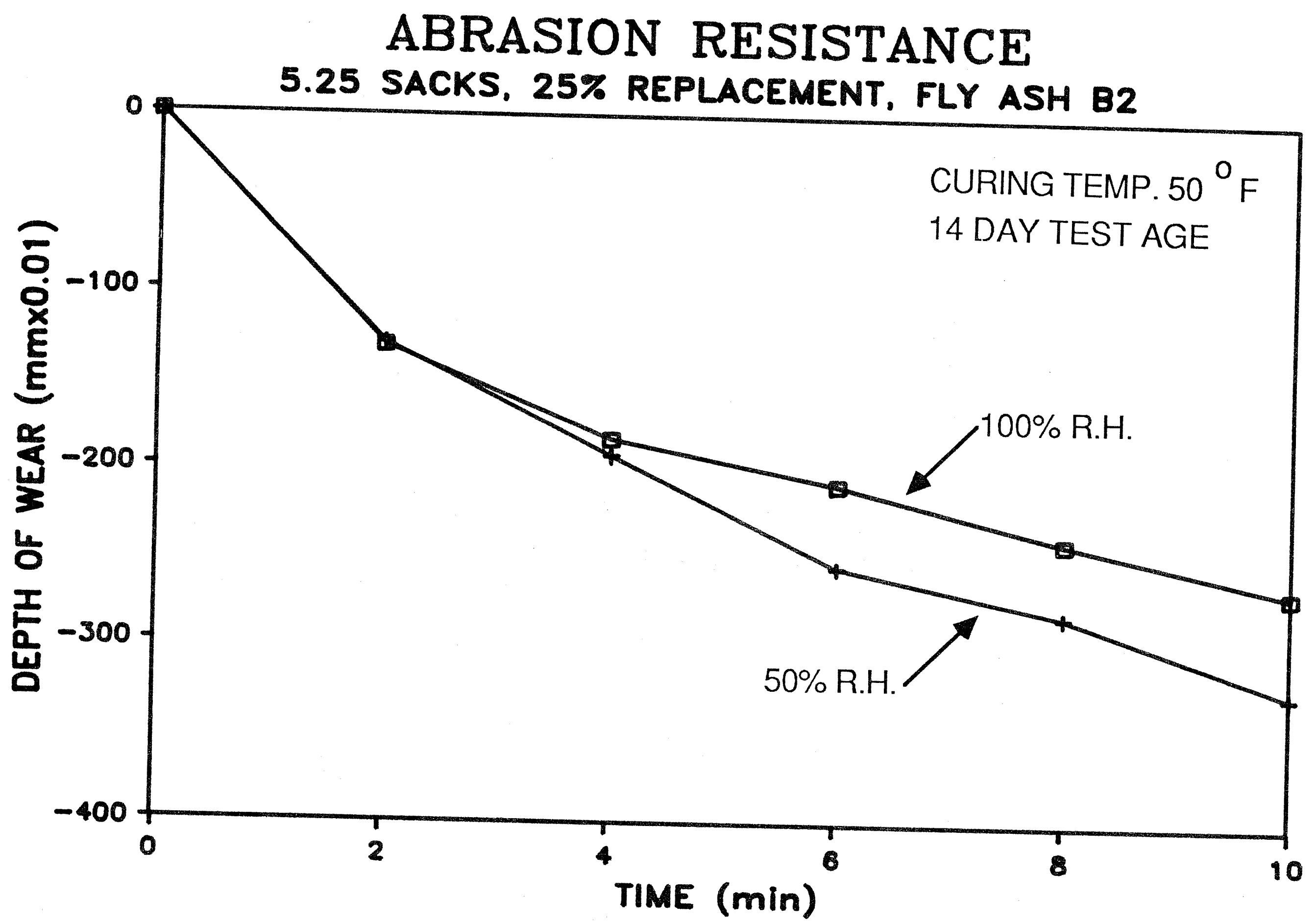

Fig. 4.25 Abrasion resistance of concrete for the 50-50B2-25 and 100-50B2-25 mixes.

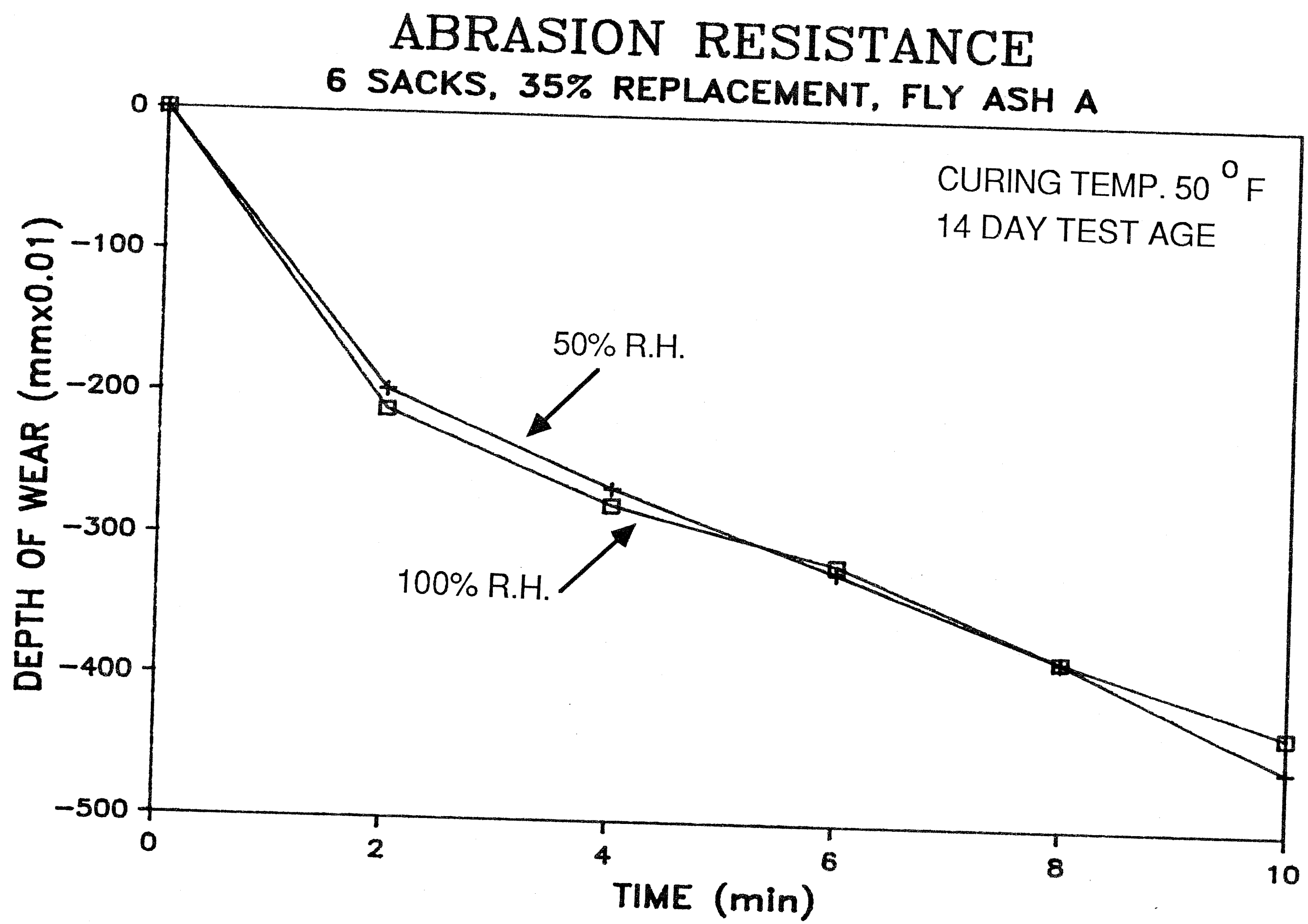

Fig. 4.26 Abrasion resistance of concrete for the 50-50A-35 and 100-50A-35 mixes.

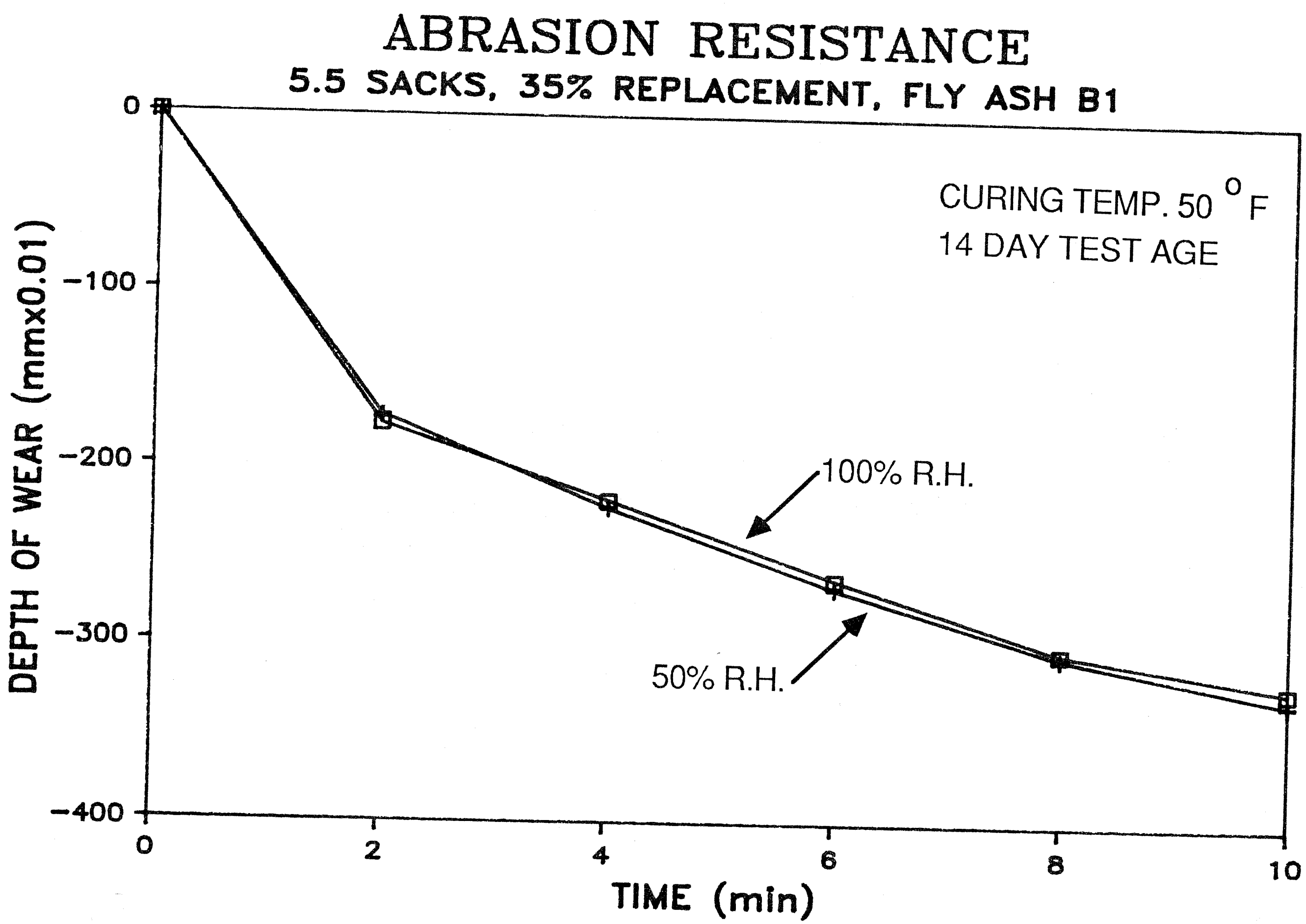

Fig. 4.27 Abrasion resistance of concrete for the 50-50B1-35 and 100-50B1-35 mixes.

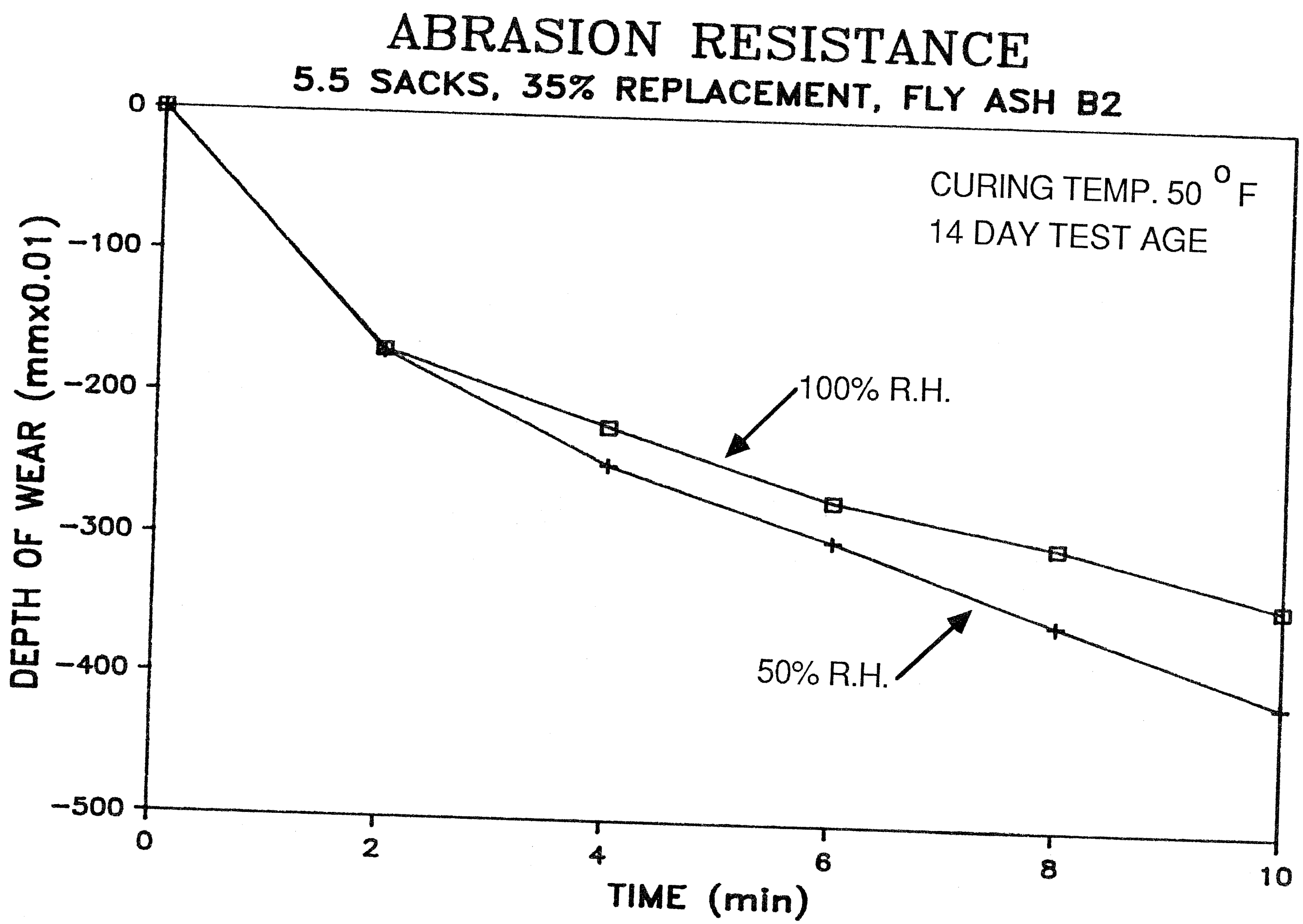

Fig. 4.28 Abrasion resistance of concrete for the 50-50B2-35 and 100-50B2-35 mixes.

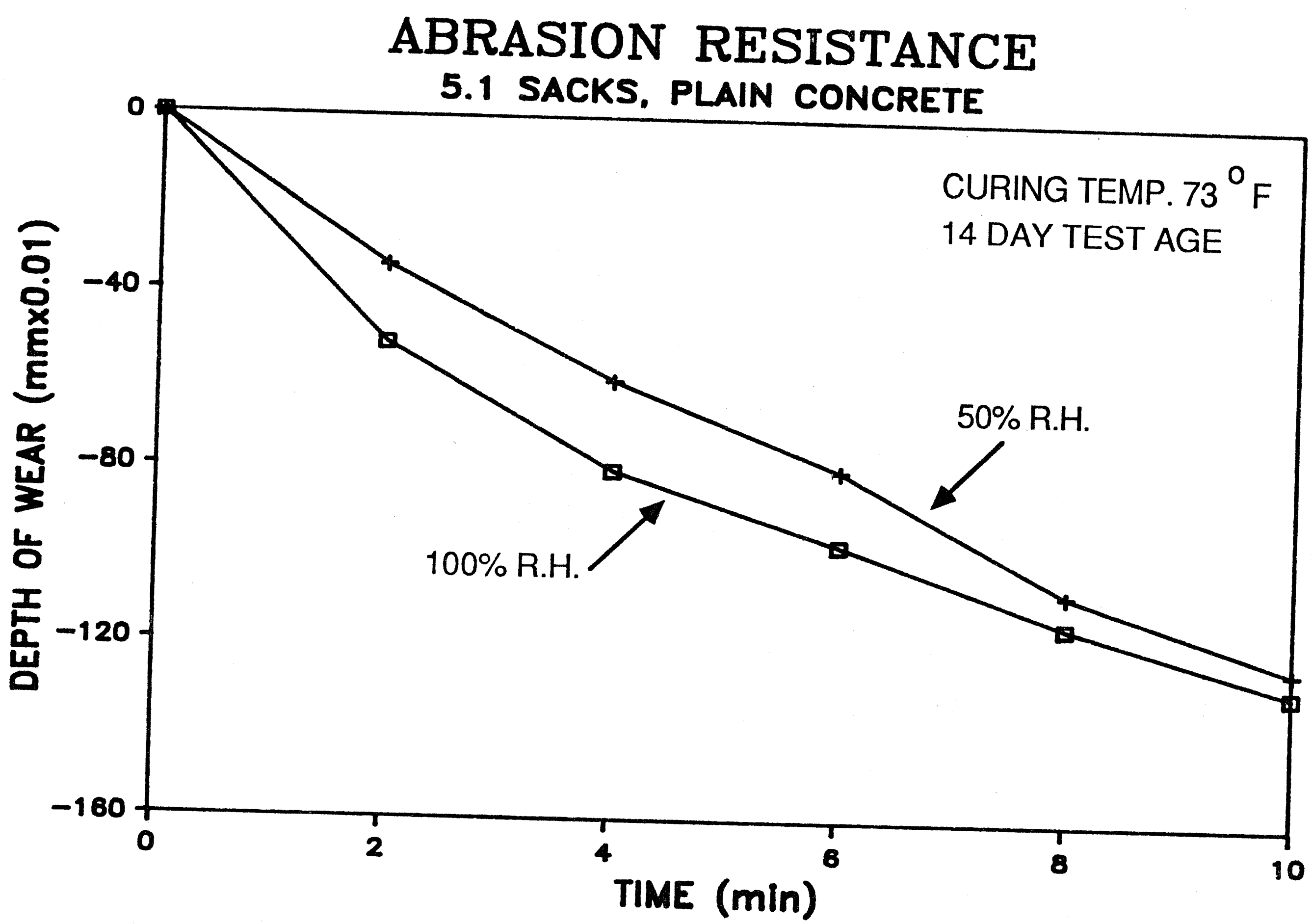

Fig. 4.29 Abrasion resistance of concrete for the 50-73 and 100- 73 mixes.

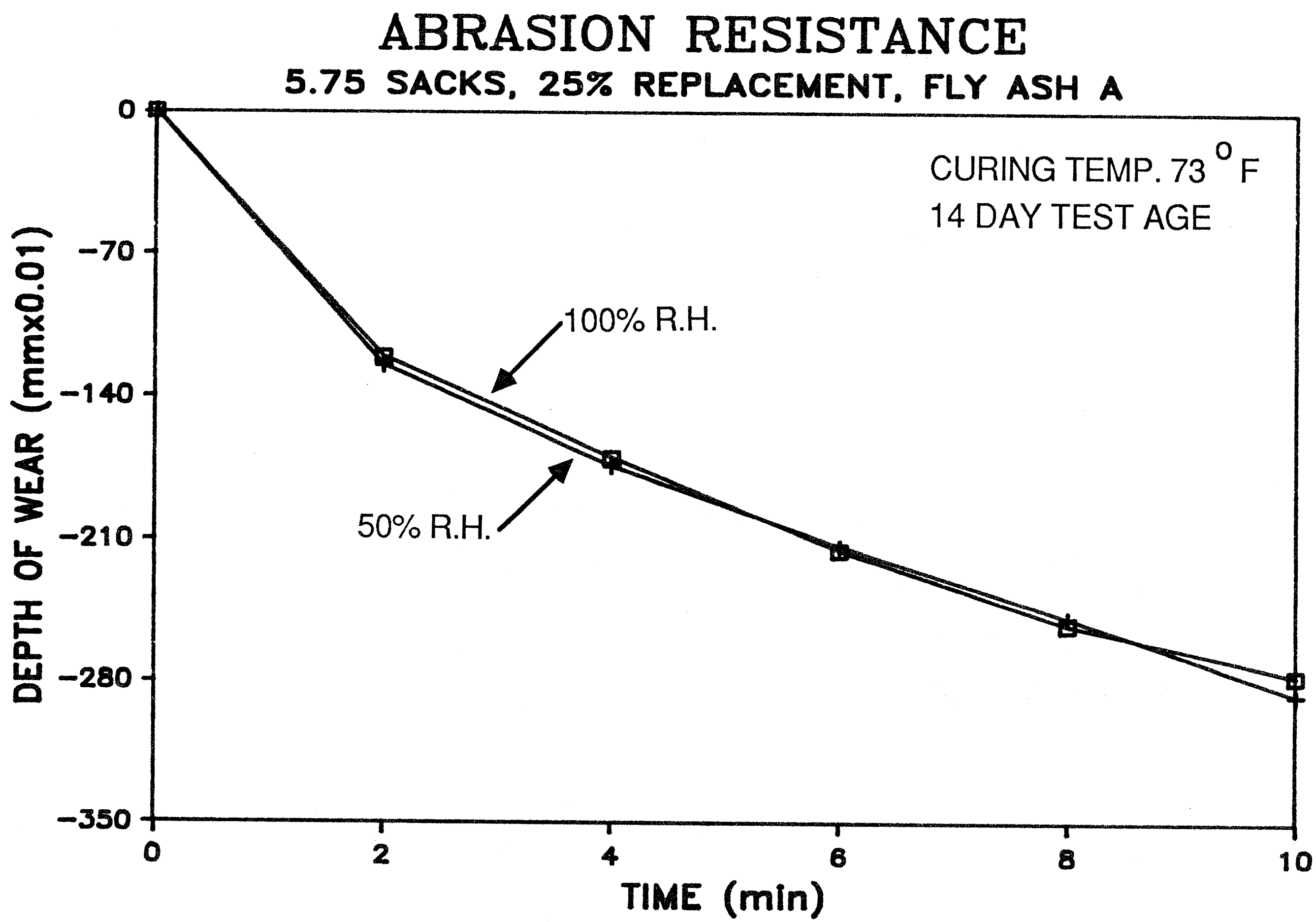

Fig. 4.30 Abrasion resistance of concrete for the 50-73A-25 and 100-73A-25 mixes.

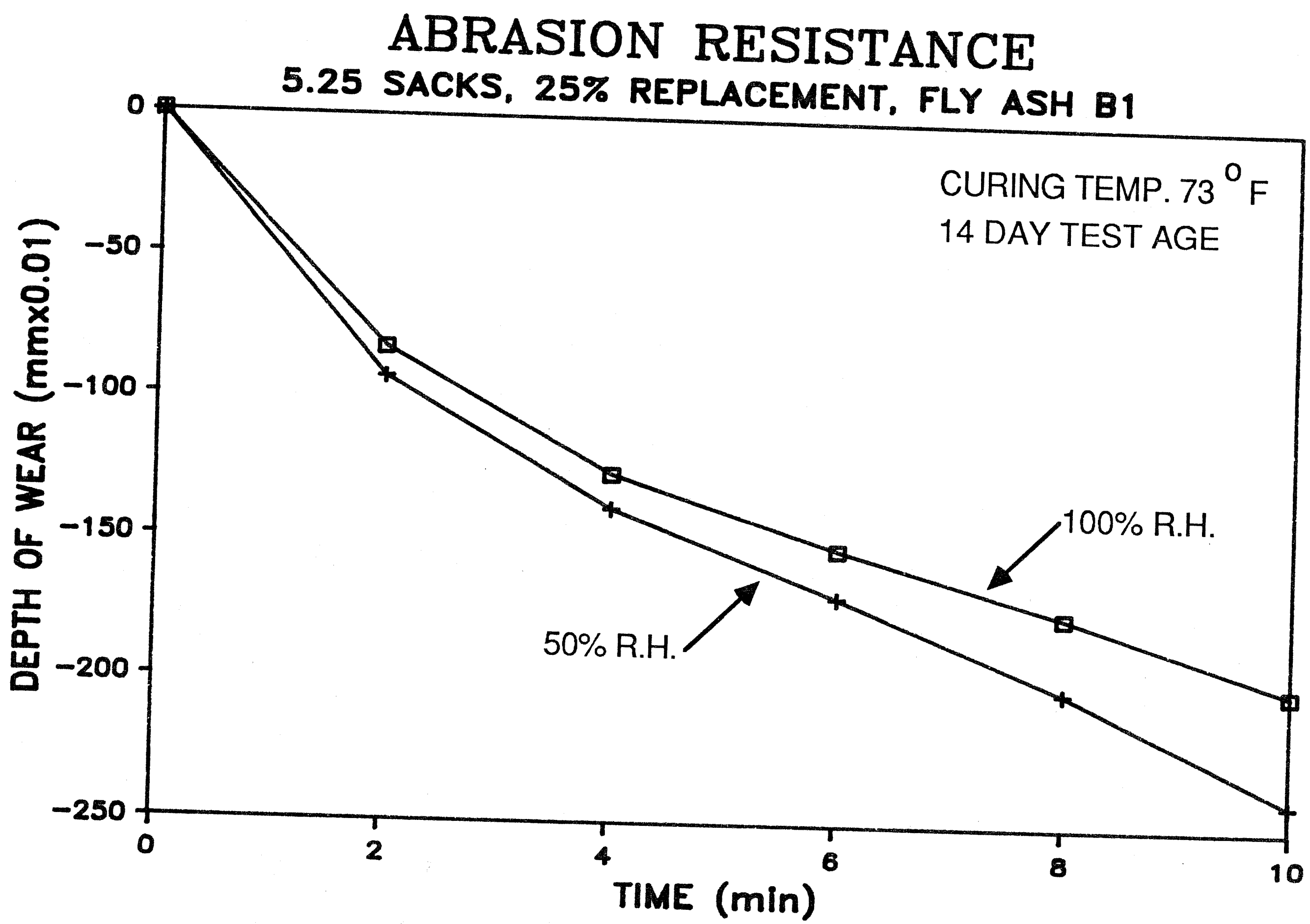

Fig. 4.31 Abrasion resistance of concrete for the 50-73B1-25 and 100-73B1-25 mixes.

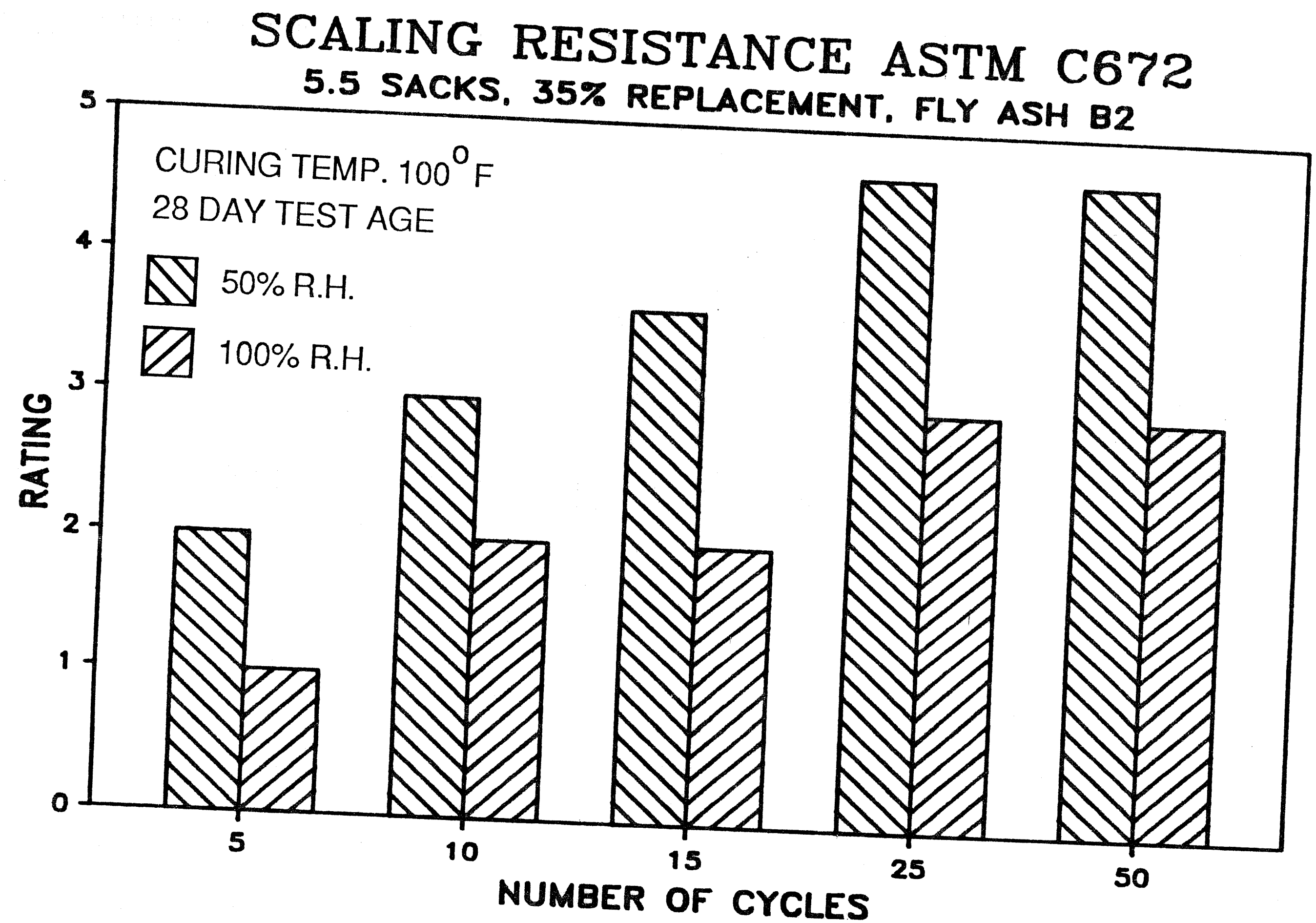

Fig. 4.63 Scaling resistance of concrete at different stages for the 50-100B2-35 and 100-100B2-35 mixes.

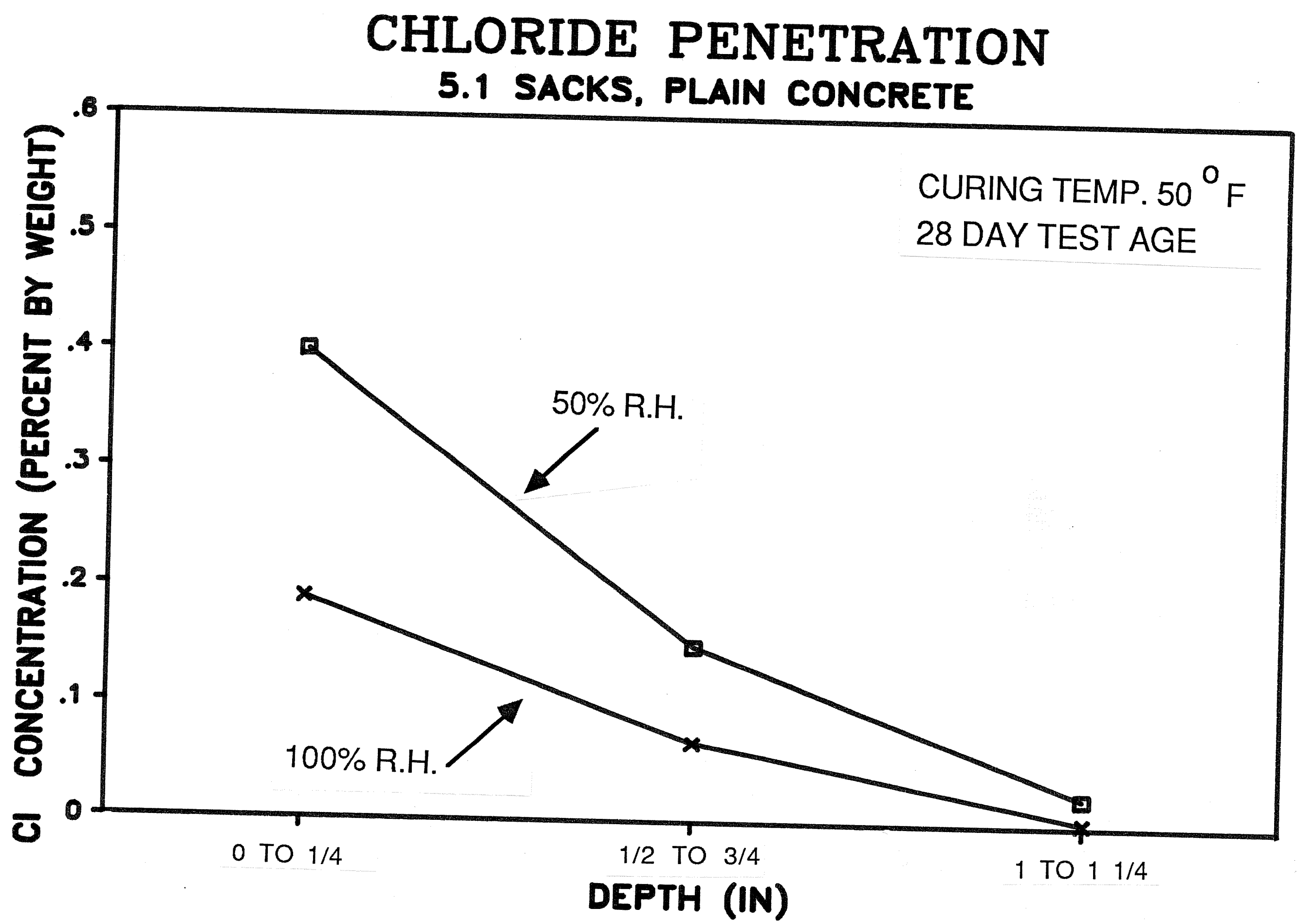

Fig. 4.64 Chloride penetration results for the 50-50 and 100-50 mixes.

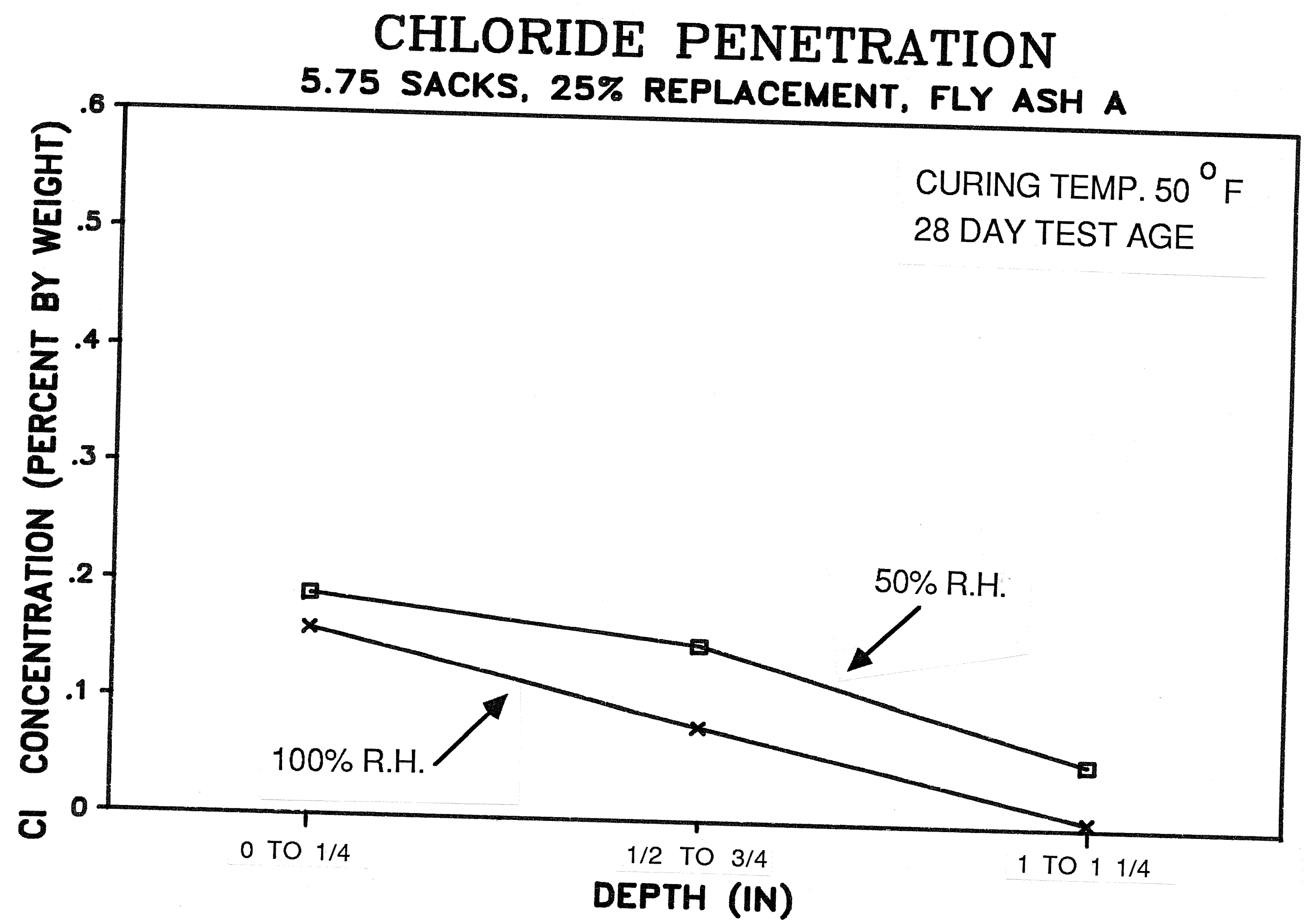

Fig. 4.65 Chloride penetration results for the 50-50A-25 and 100-50A-25 mixes.

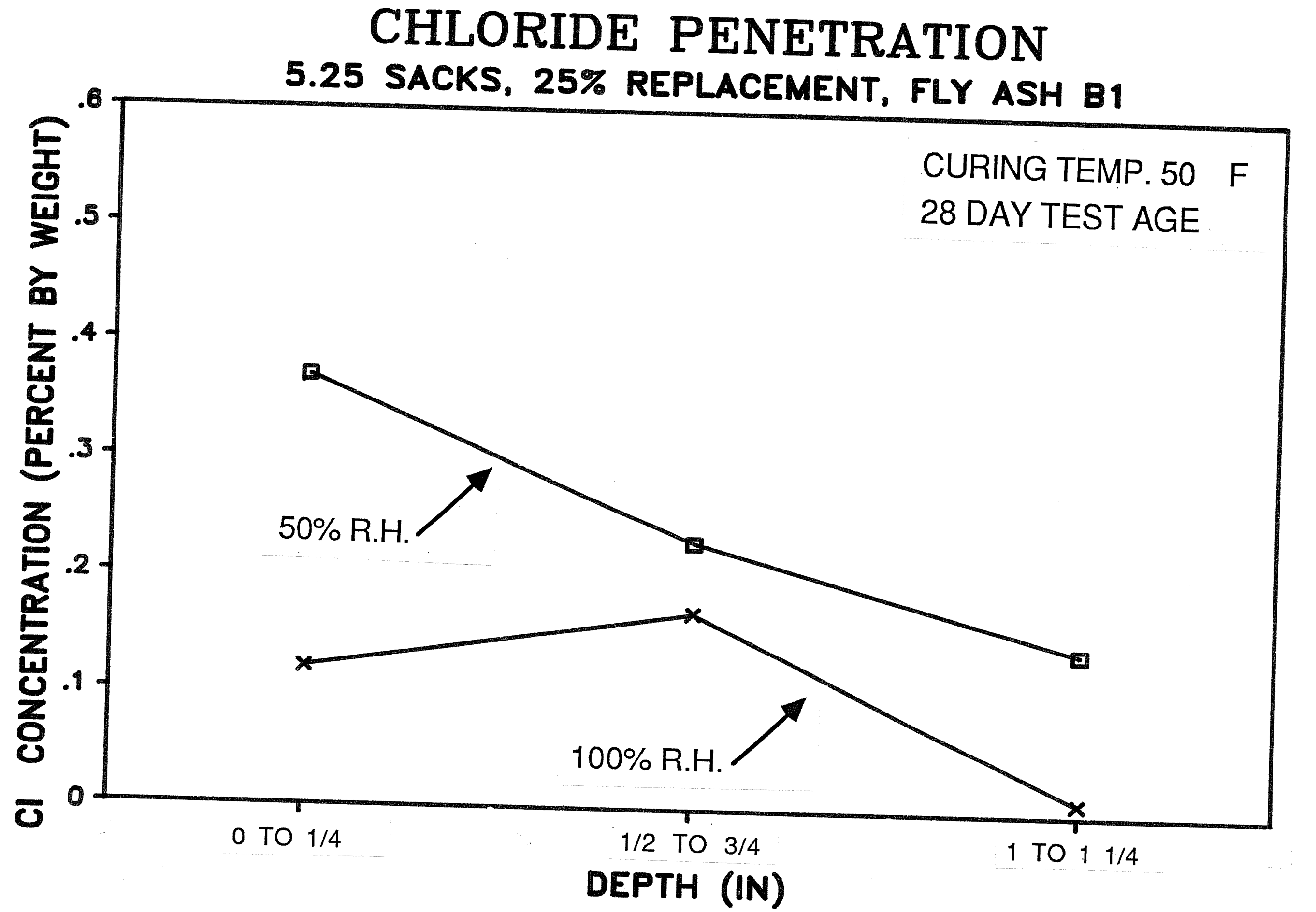

Fig. 4.66 Chloride penetration results for the 50-50B1-25 and 100-50B1-25 mixes.

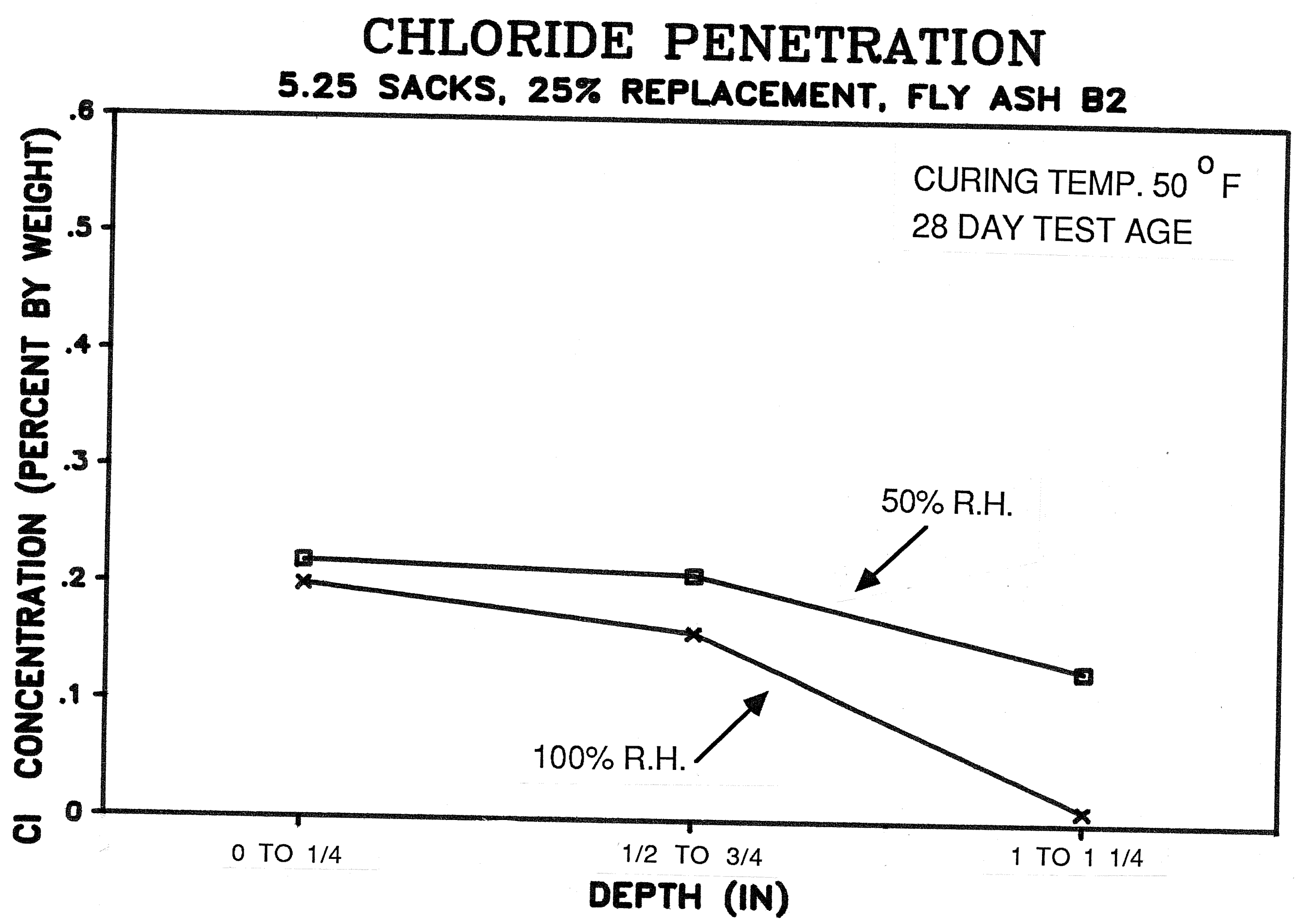

Fig. 4.67 Chloride penetration results for the 50-50B2-25 and 100-50B2-25 mixes.

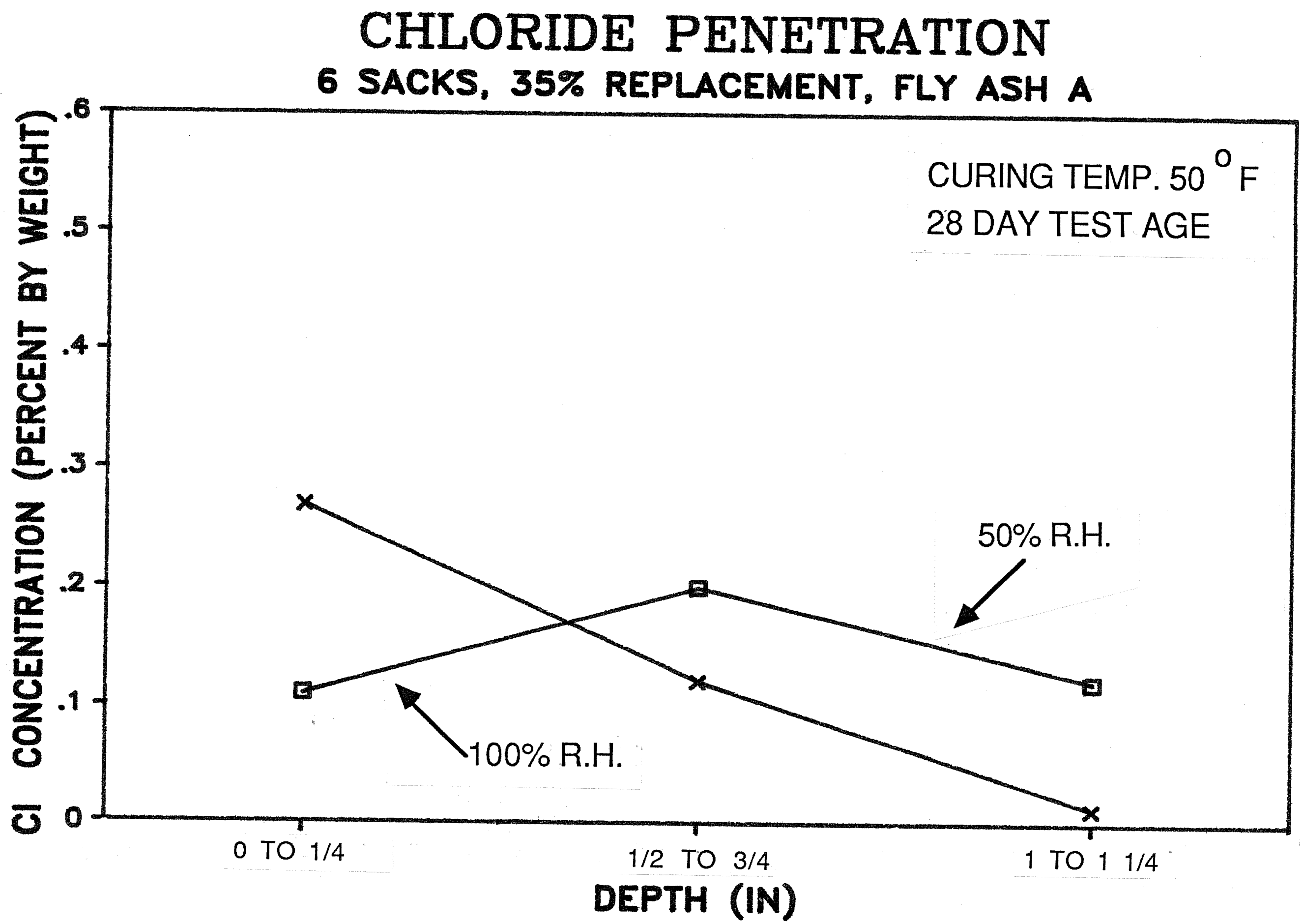

Fig. 4.68 Chloride penetration results for the 50-50A-35 and 100-50A-35 mixes.

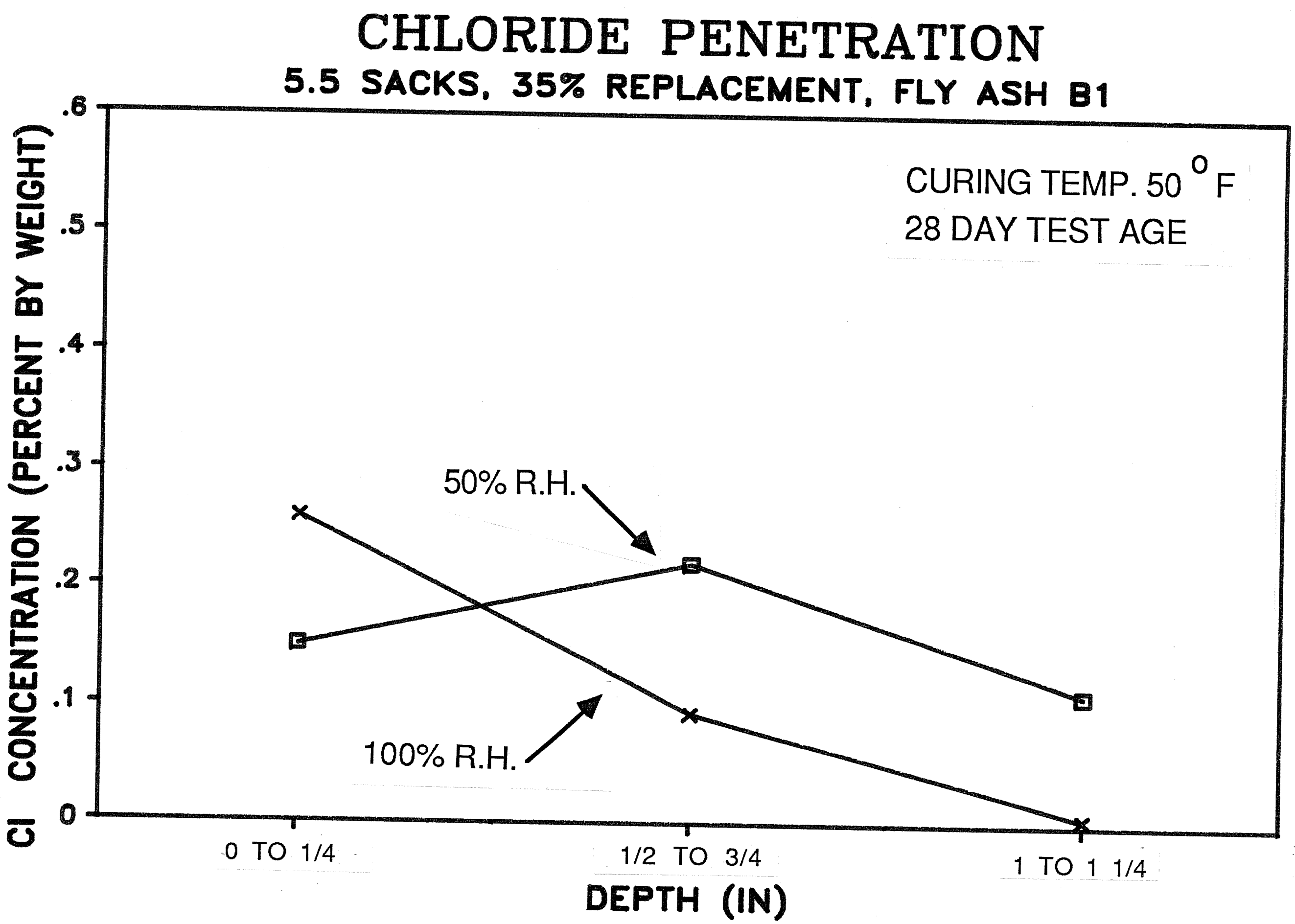

Fig. 4.69 Chloride penetration results for the 50-50B1-35 and 100-50B1-35 mixes.

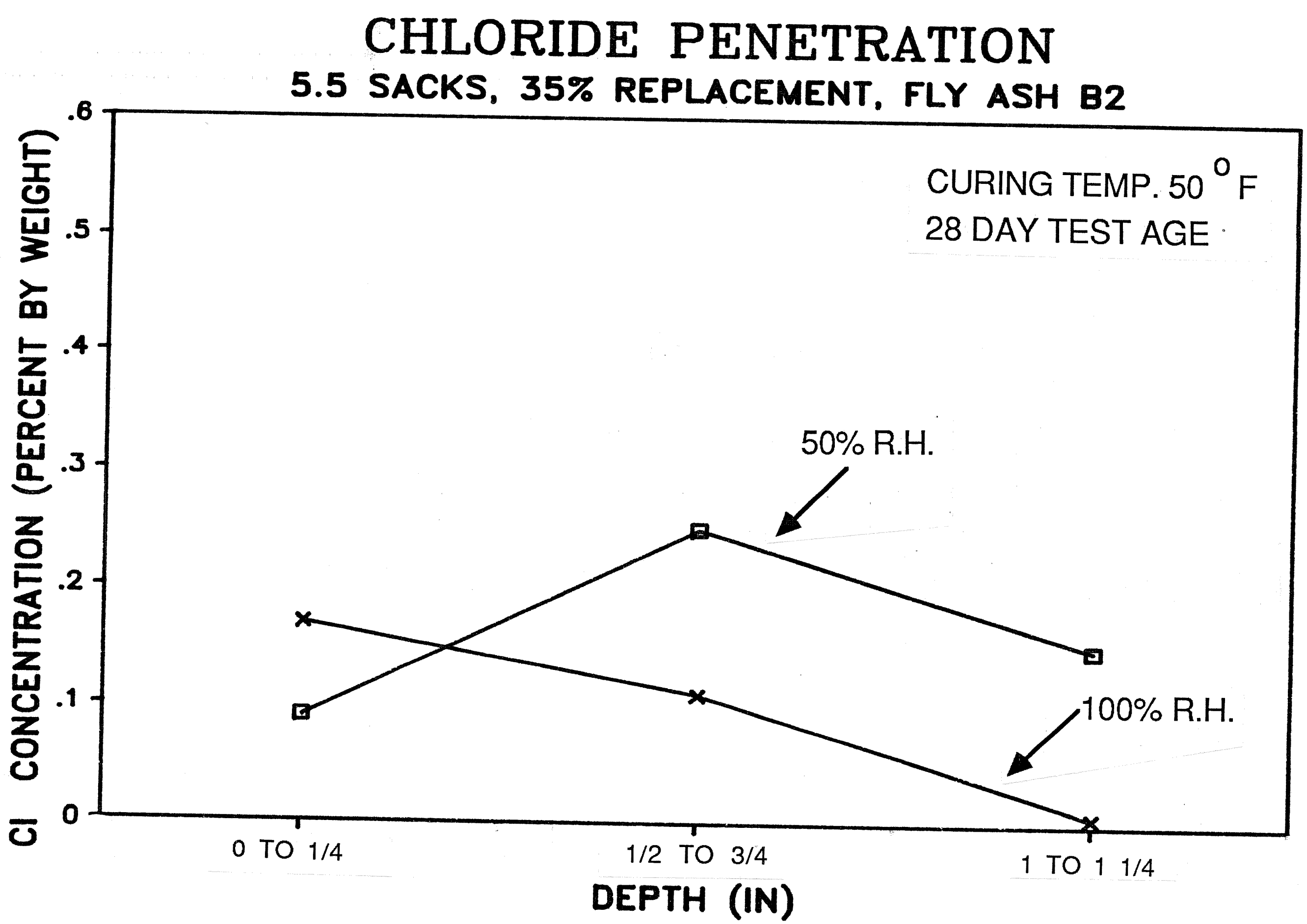

Fig. 4.70 Chloride penetration results for the 50-50B2-35 and 100-50B2-35 mixes.

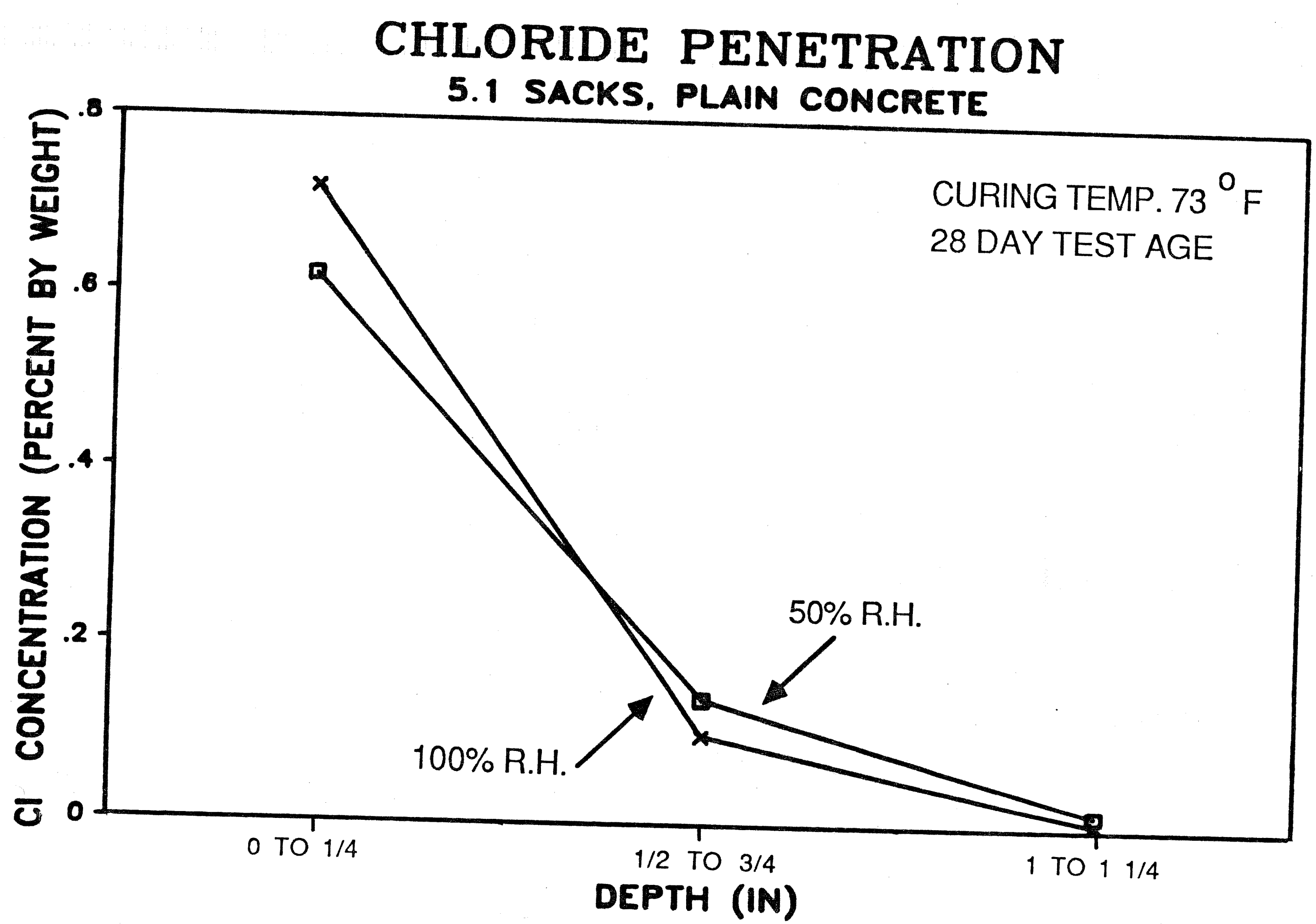

Fig. 4.71 Chloride penetration results for the 50-73 and 100-73 mixes.

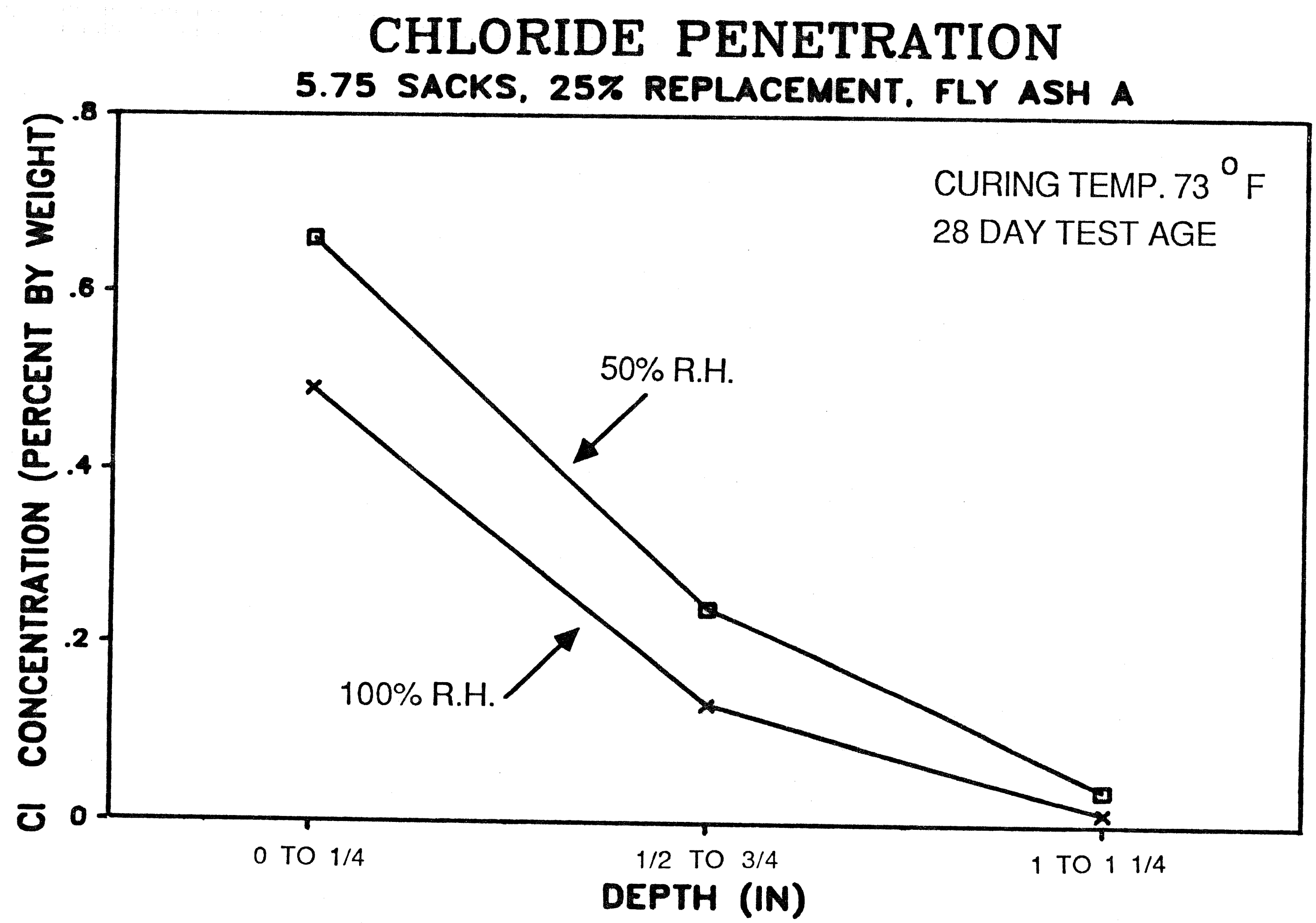

Fig. 4.72 Chloride penetration results for the 50-73A-25 and 100-73A-25 mixes.

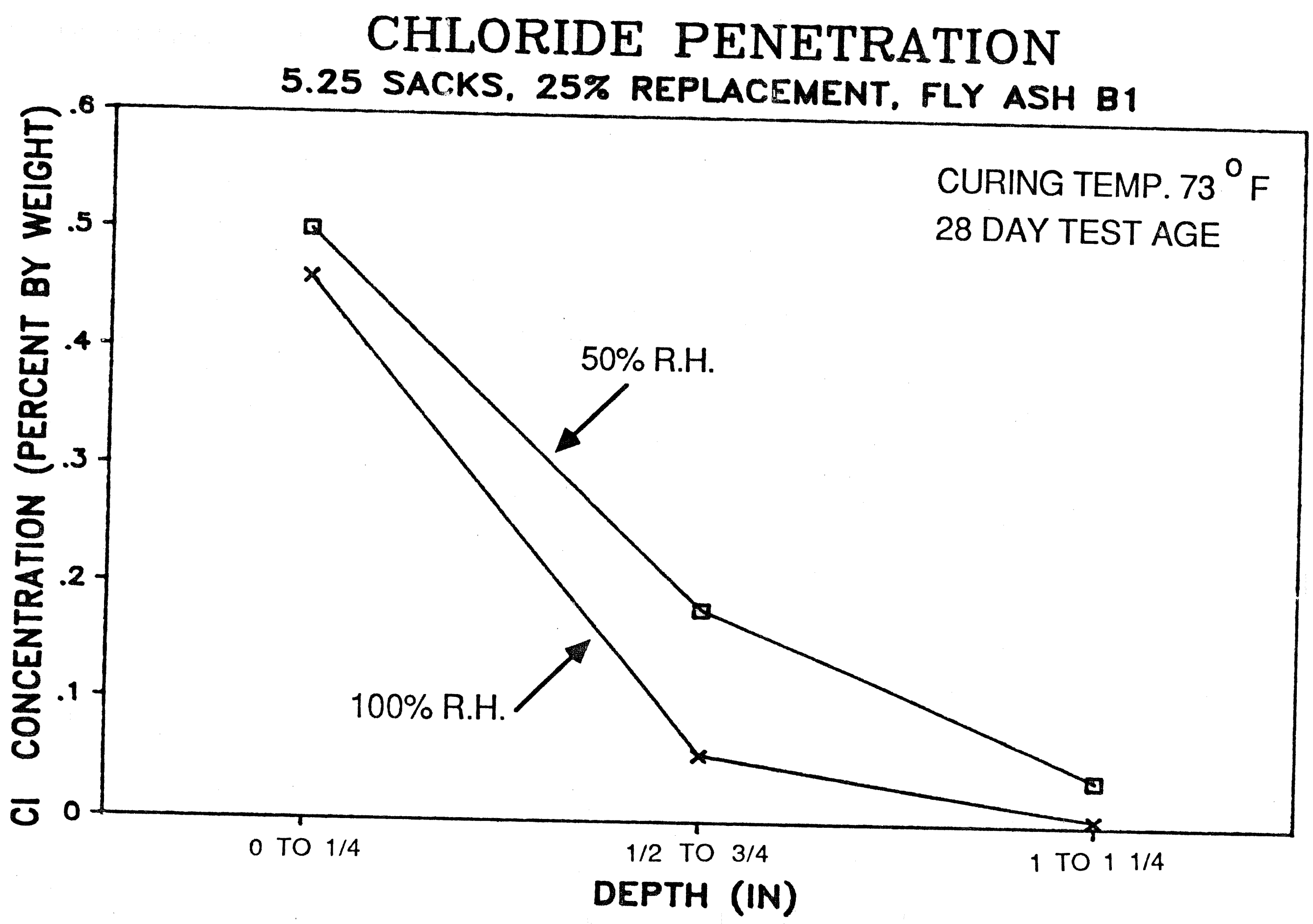

Fig. 4.73 Chloride penetration results for the 50-73B1-25 and 100-73B1-25 mixes.

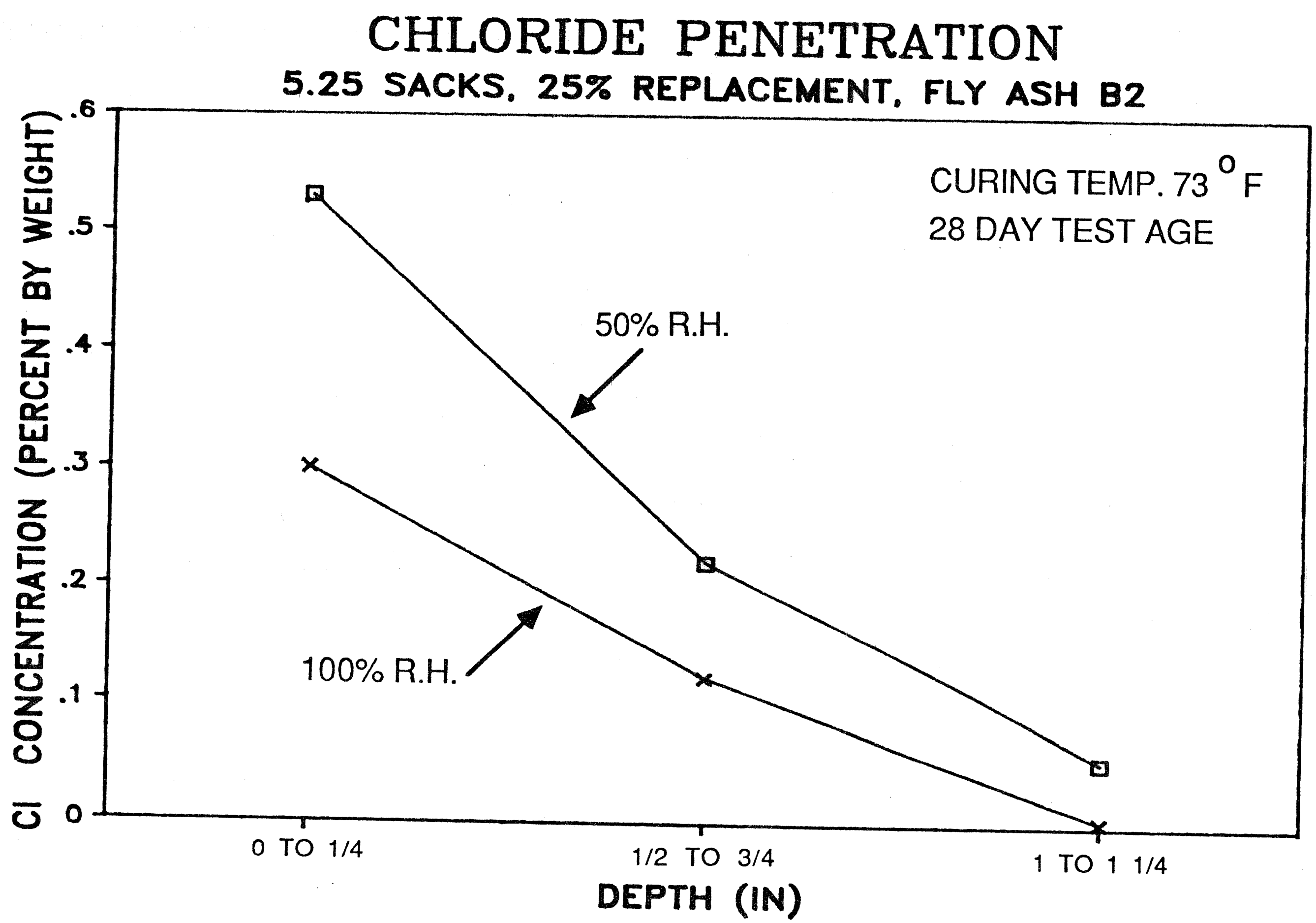

Fig. 4.74 Chloride penetration results for the 50-73B2-25 and 100-73B2-25 mixes.

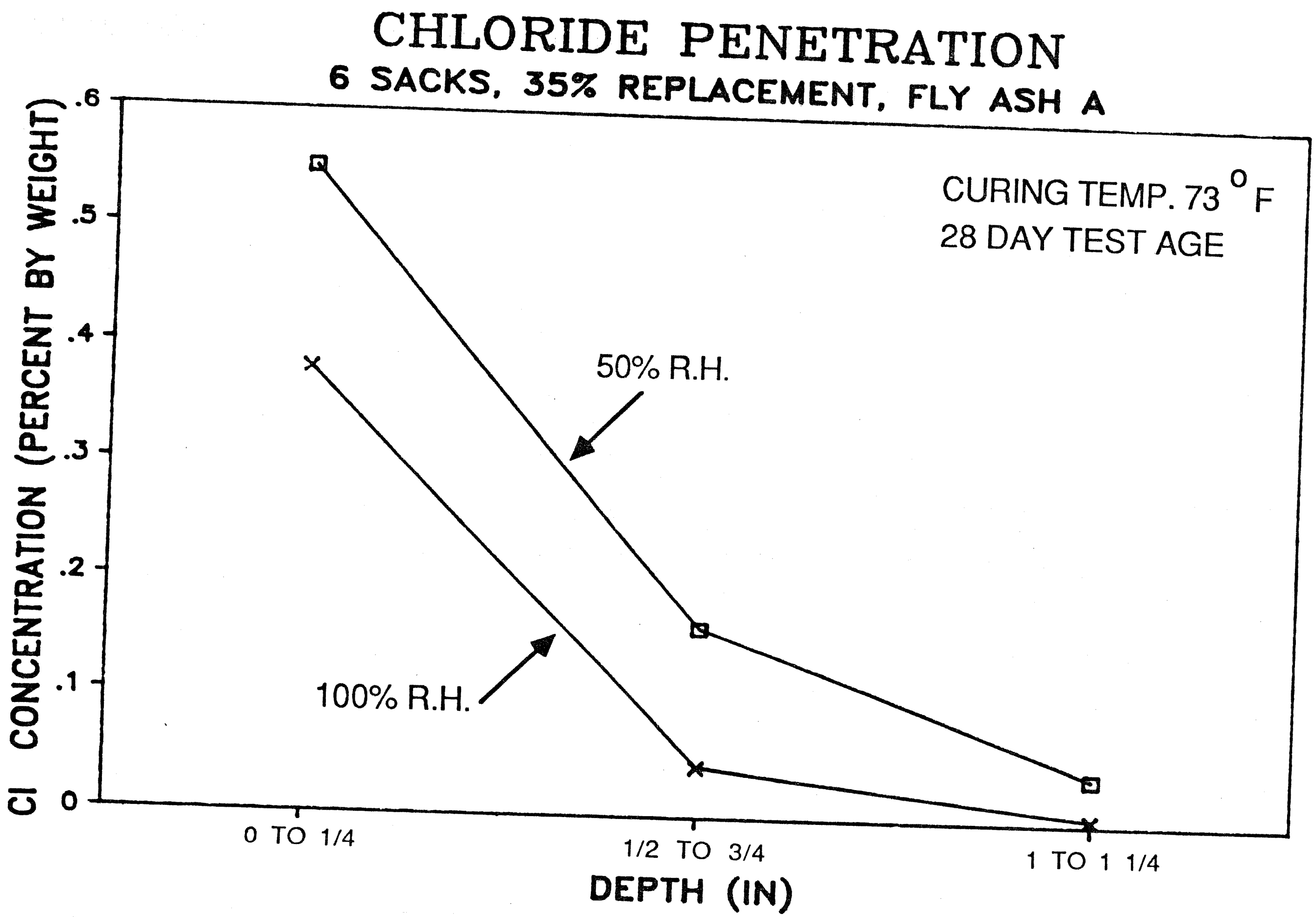

Fig. 4.75 Chloride penetration results for the 50-73A-35 and 100-73A-35 mixes.

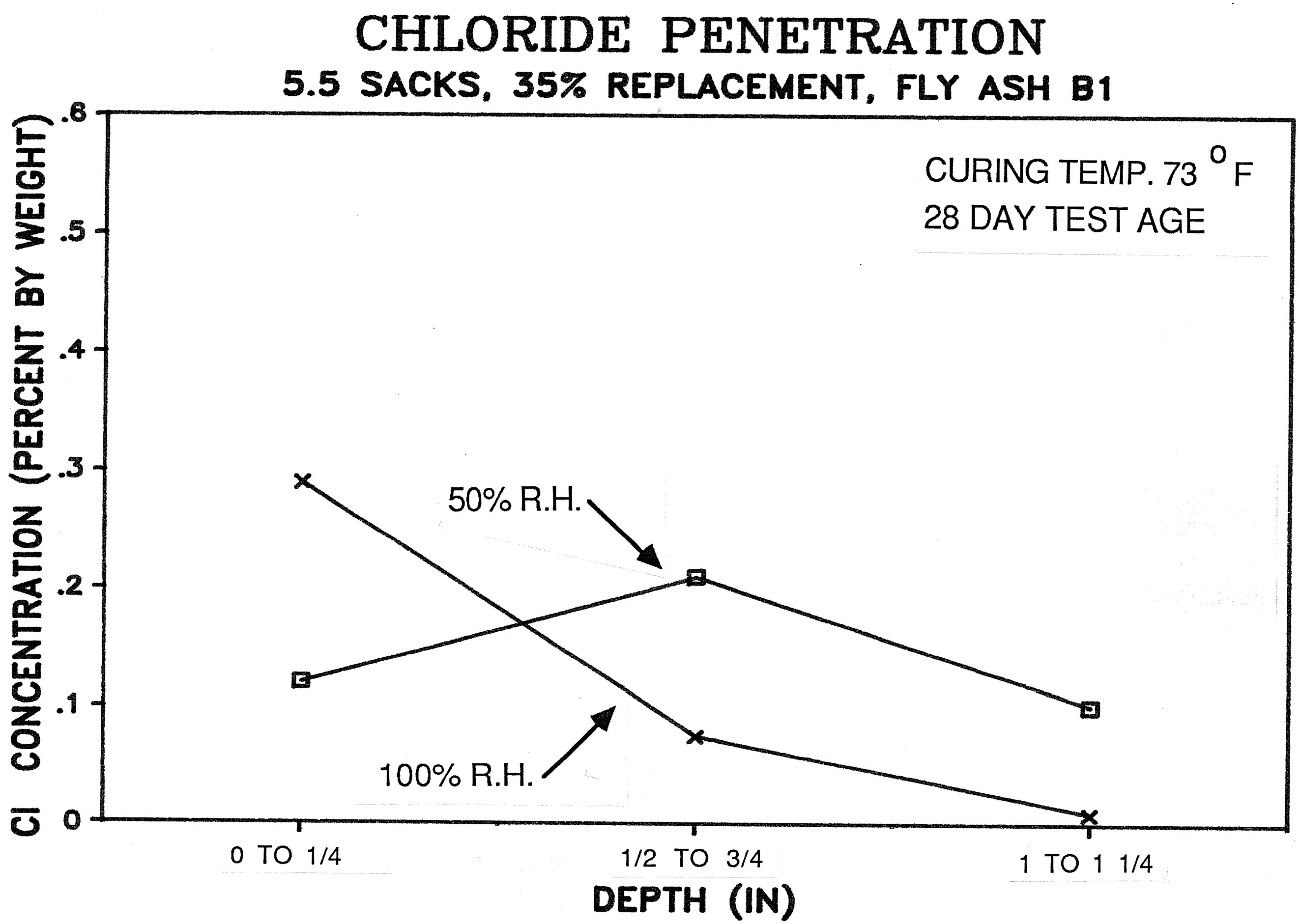

Fig. 4.76 Chloride penetration results for the 50-73B1-35 and 100-73B1-35 mixes.

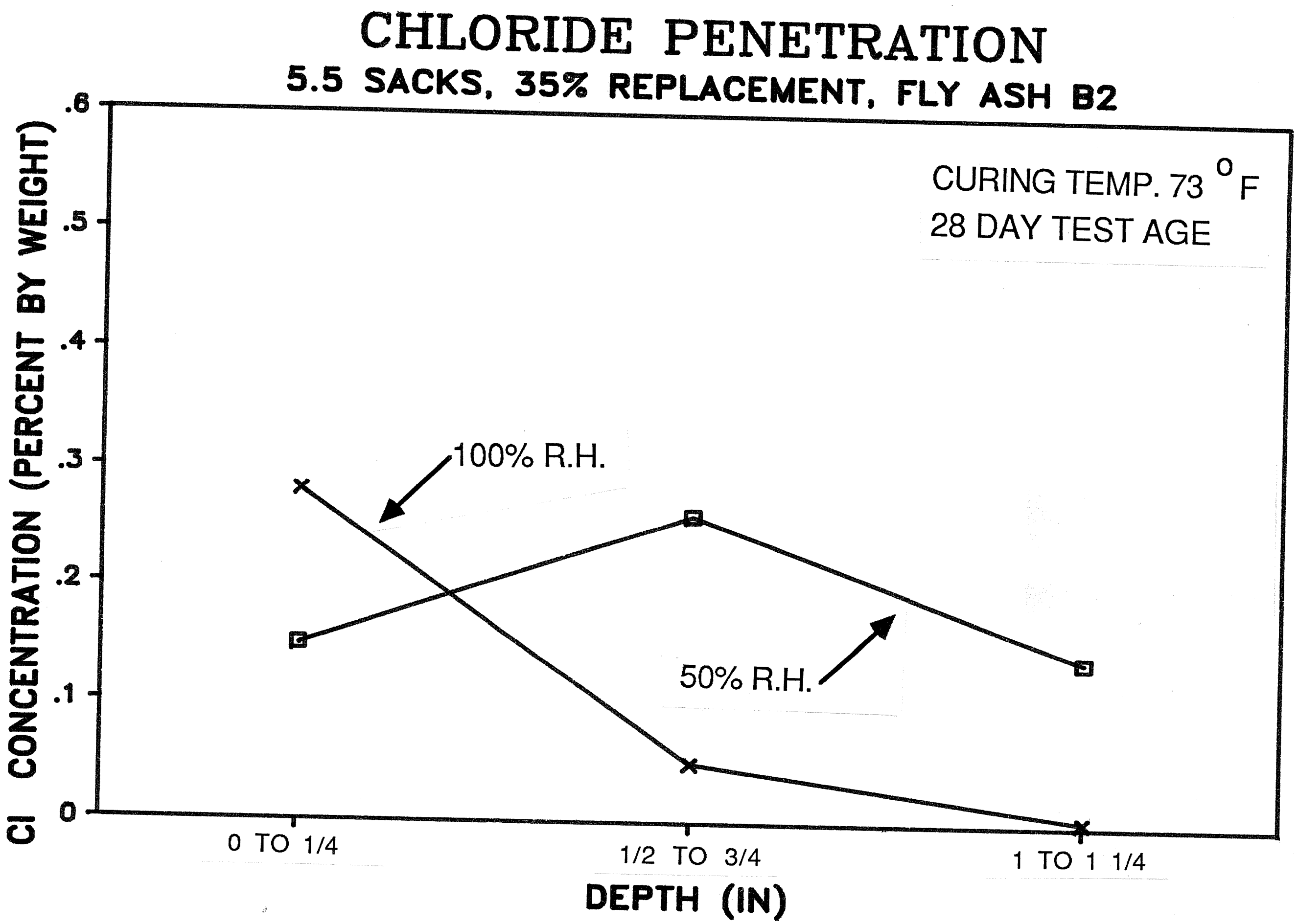

Fig. 4.77 **Chloride penetration results for the 50-73B2-35 and 100-73B2-35 mixes.**

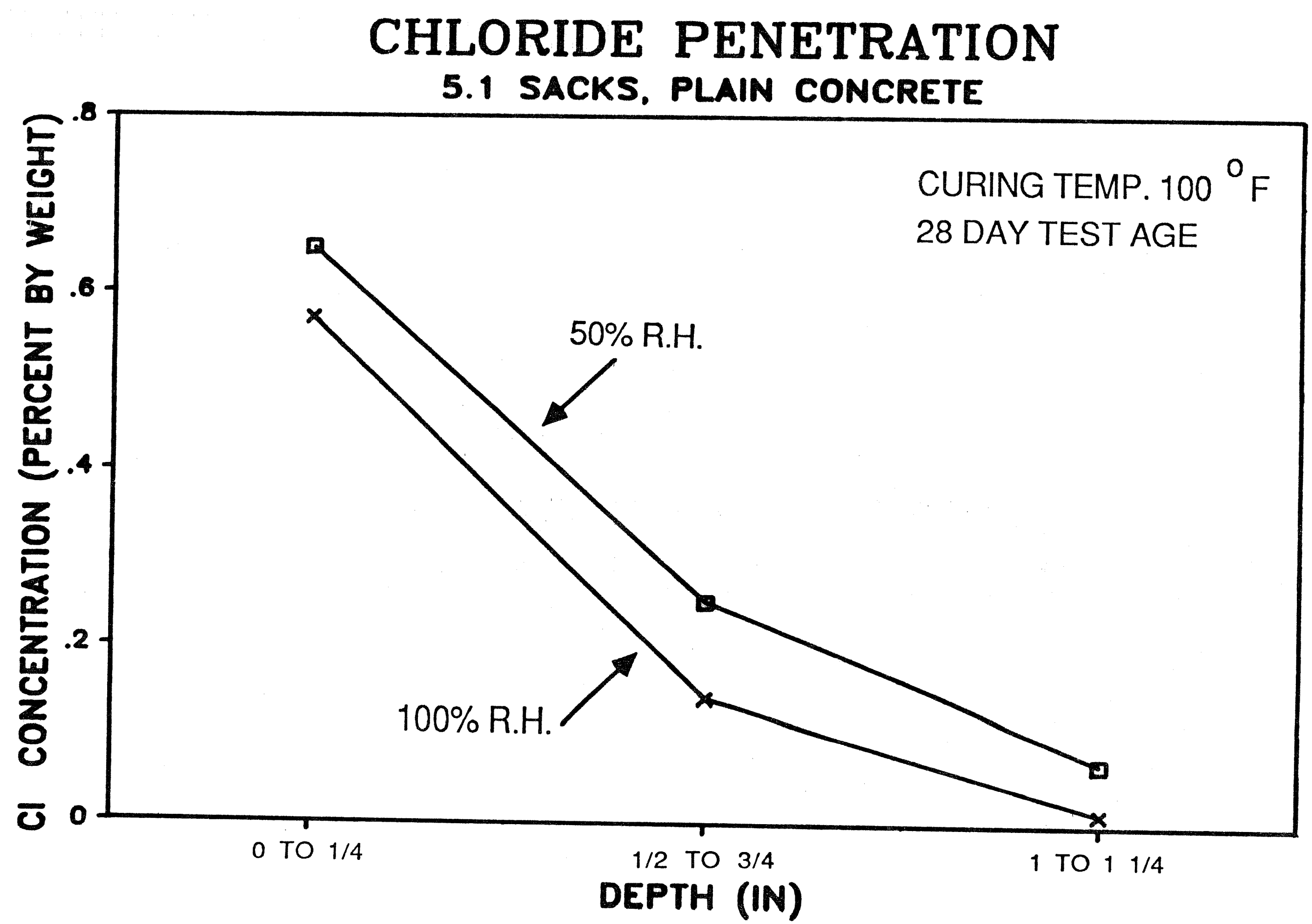

Fig. 4.78 Chloride penetration results for the 50-100 and 100-100 mixes.

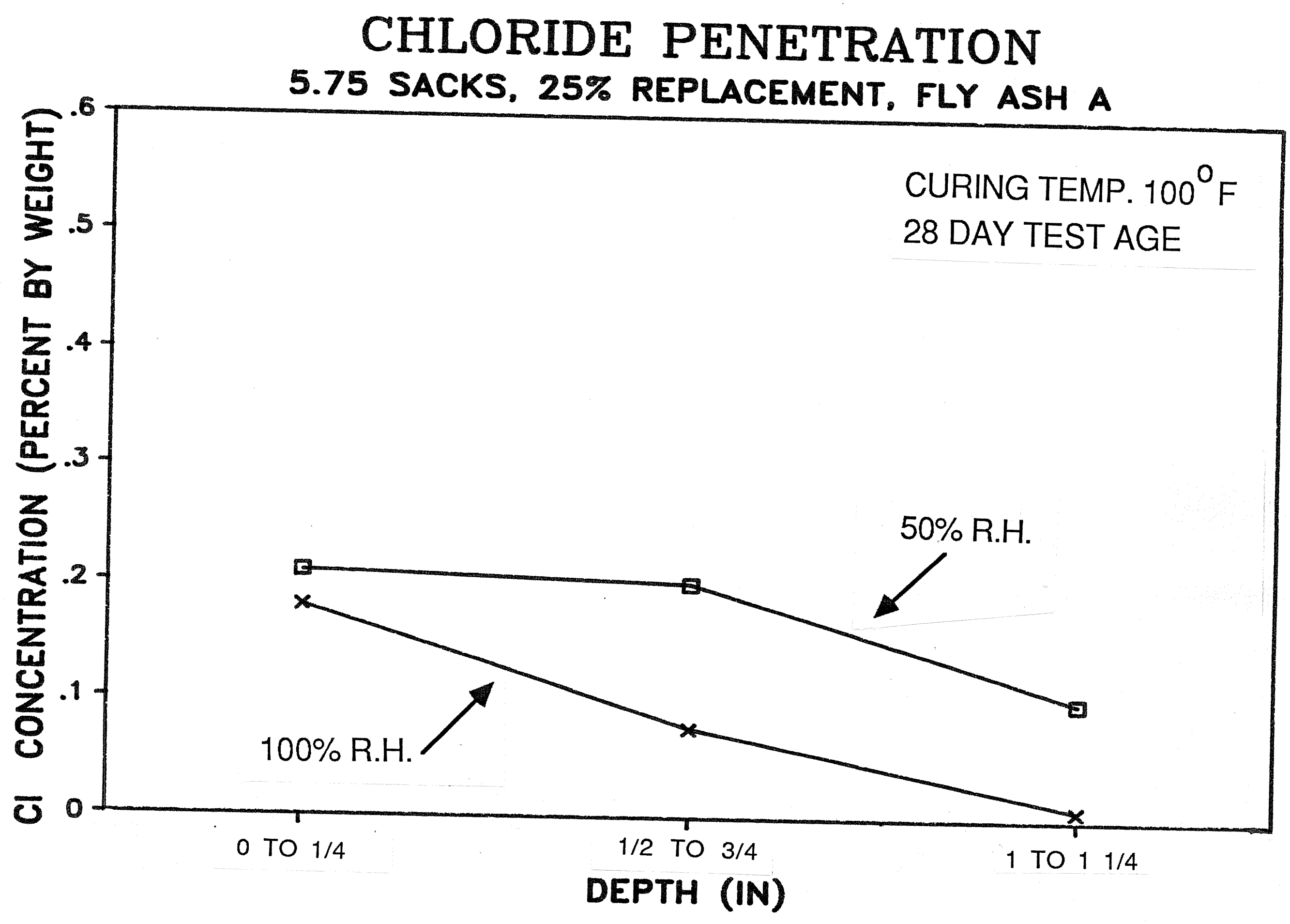

Fig. 4.79 Chloride penetration results for the 50-100A-25 and 100-100A-25 mixes.

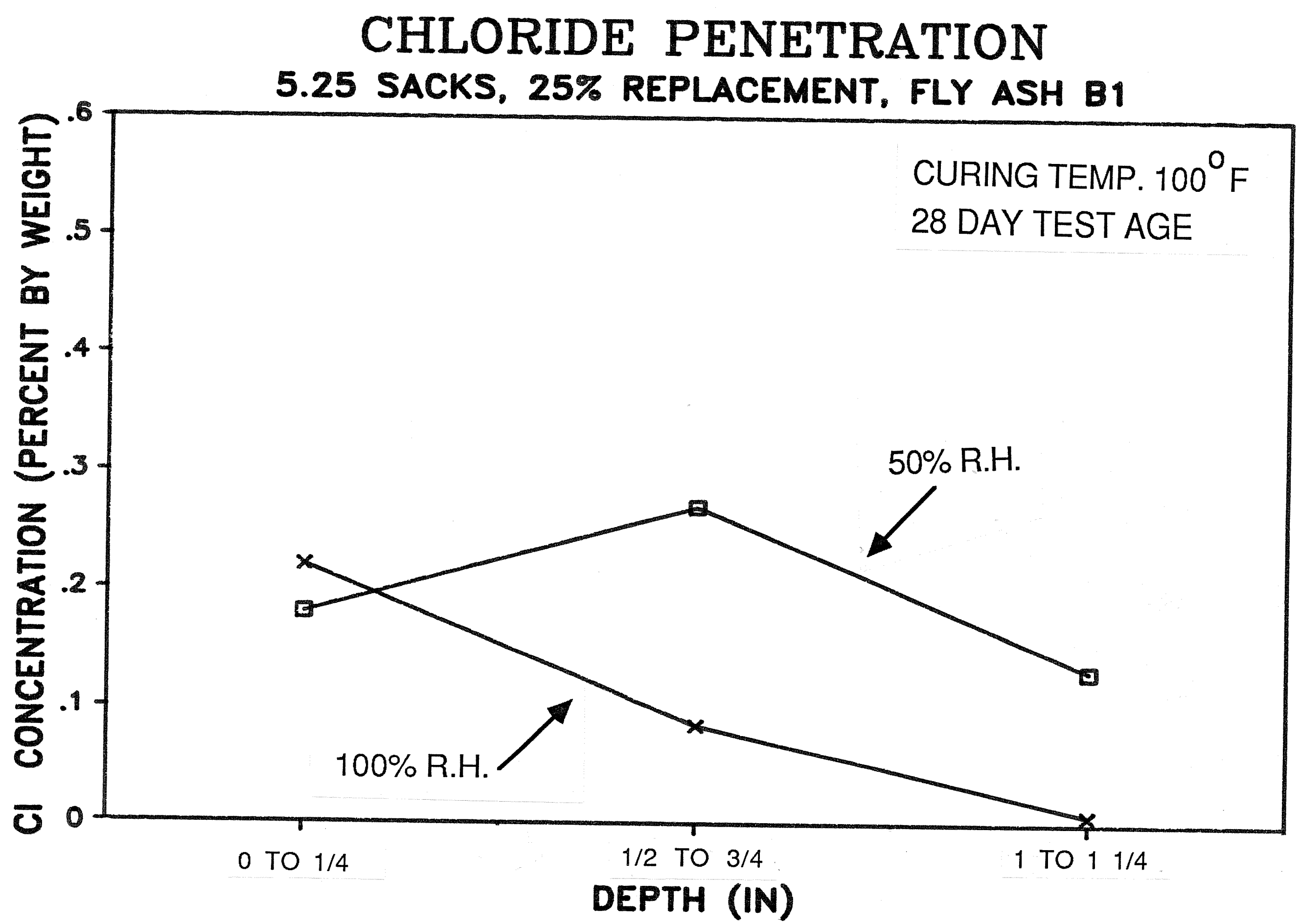

Fig. 4.80 Chloride penetration results for the 50-100B1-25 and 100-100B1-25 mixes.

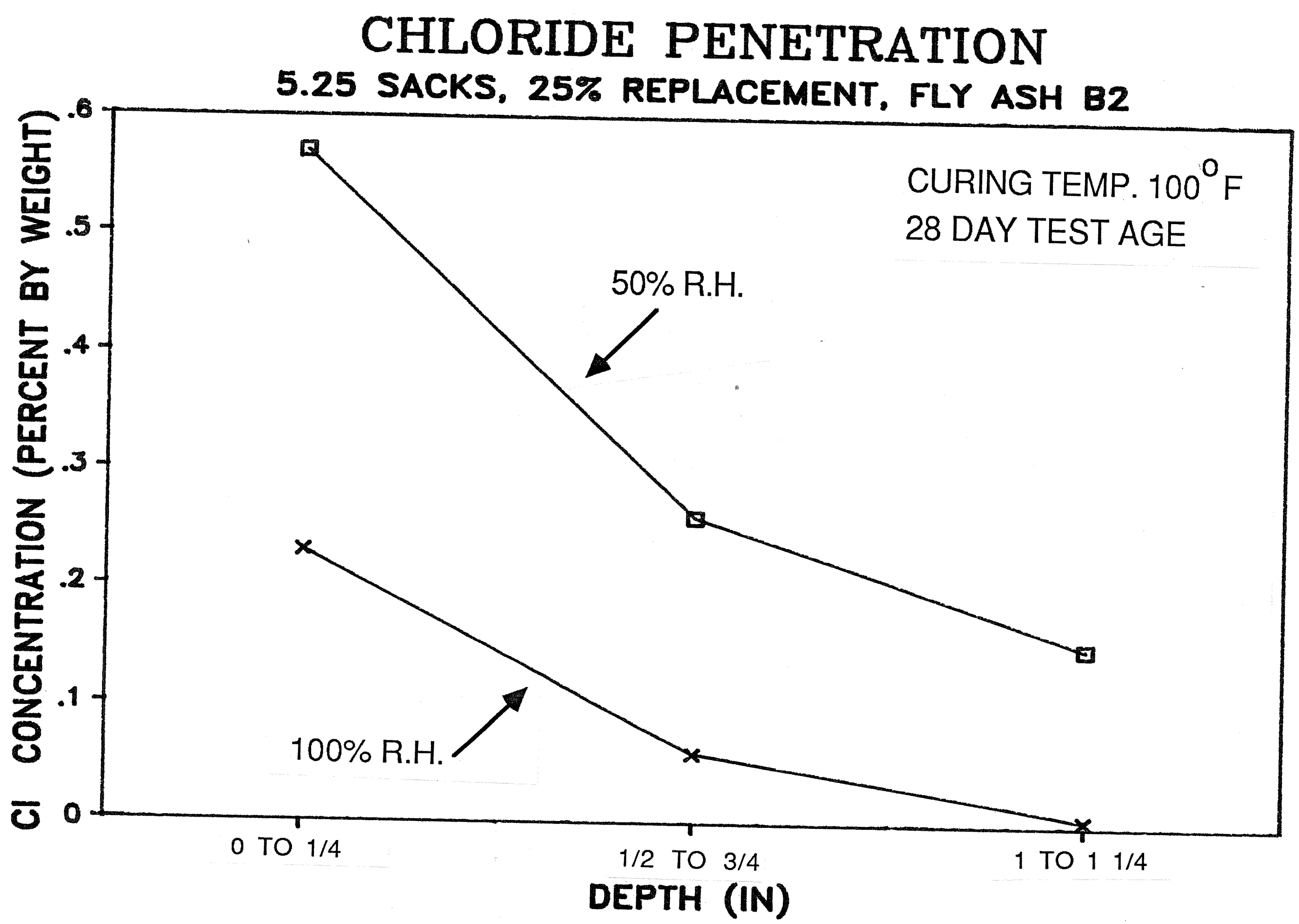

Fig. 4.81 Chloride penetration results for the 50-100B2-25 and 100-100B2-25 mixes.

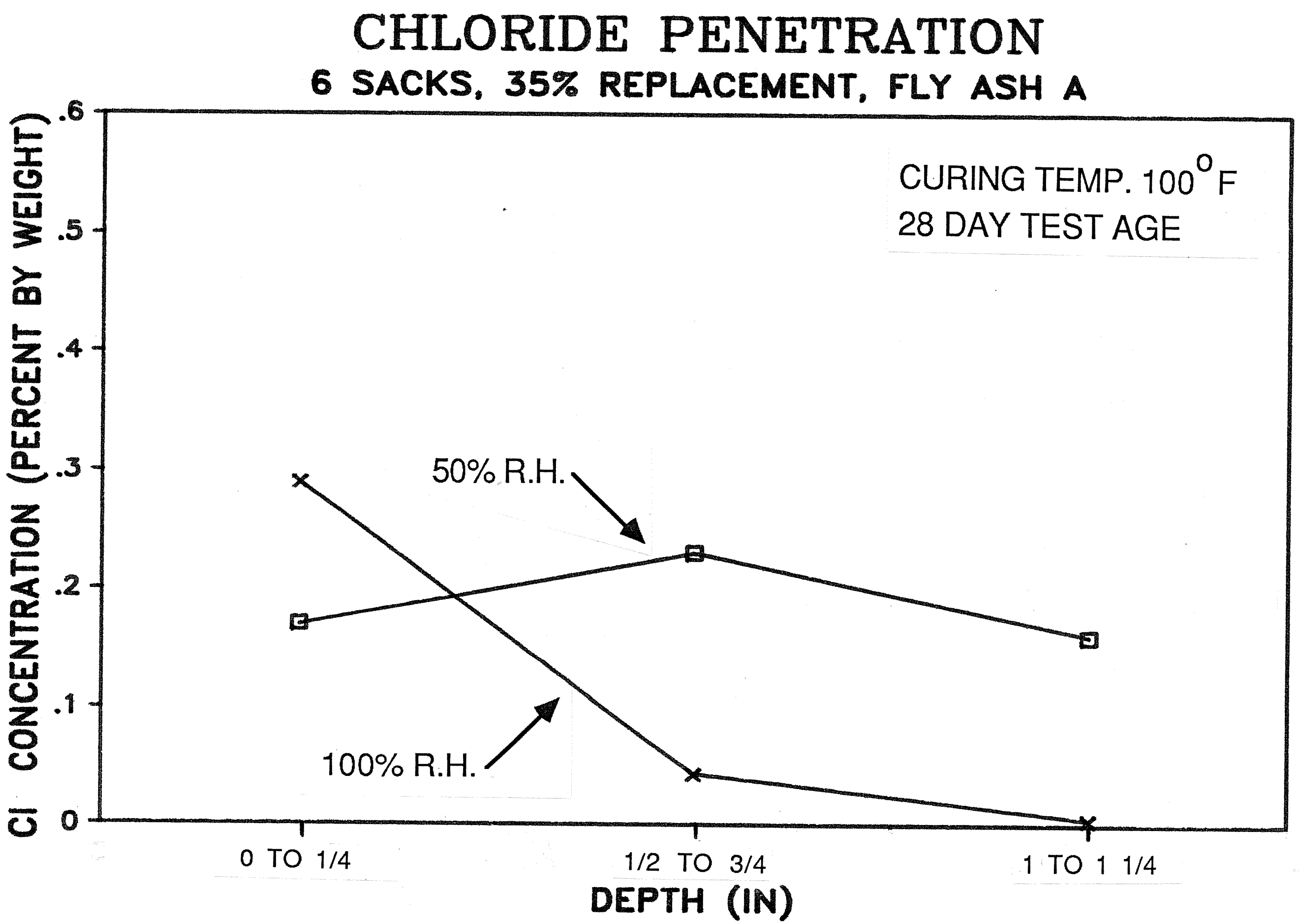

Fig. 4.82 Chloride penetration results for the 50-100A-35 and 100-100A-35 mixes.

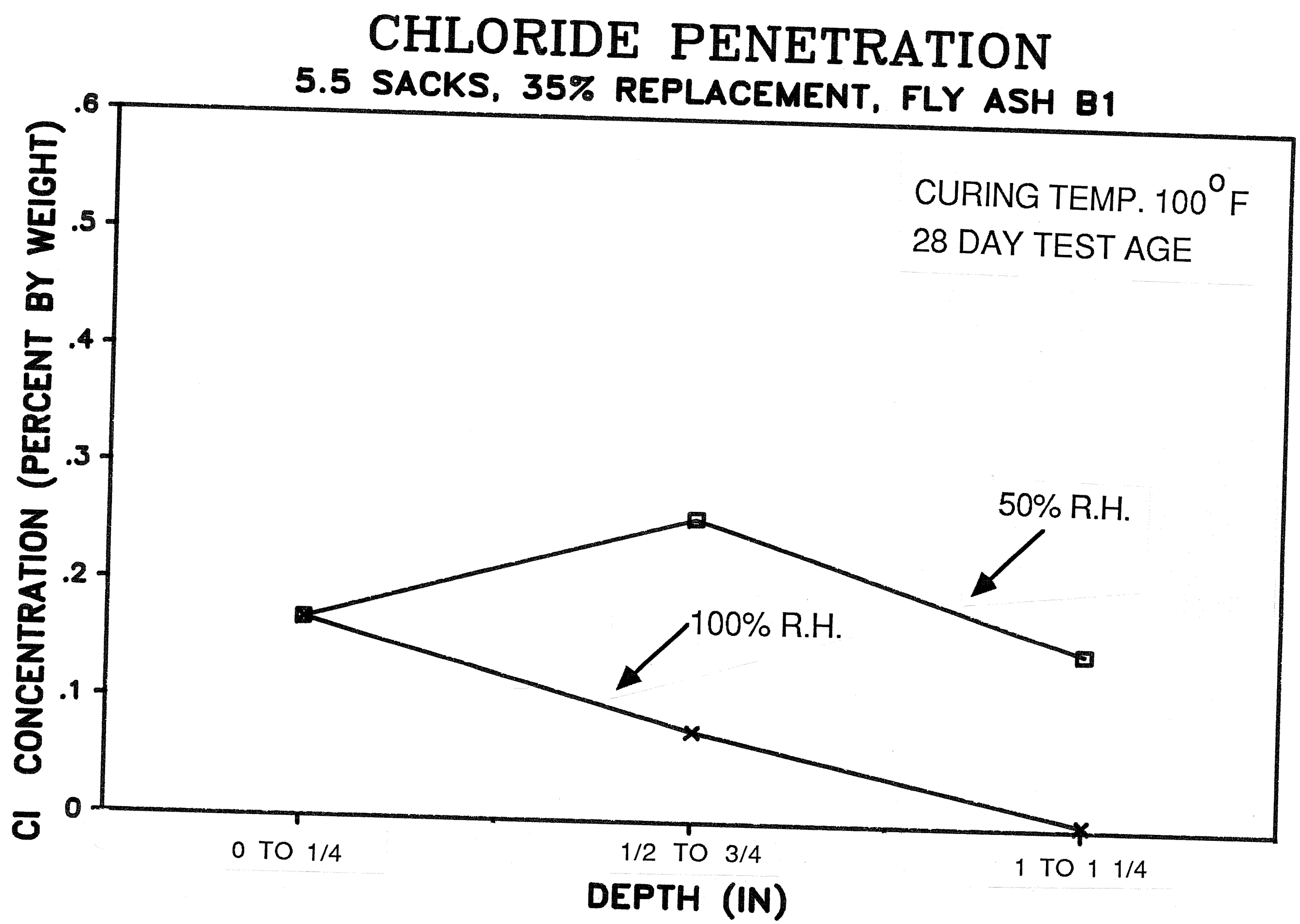

Fig. 4.83 Chloride penetration results for the 50-100B1-35 and 100-100B1-35 mixes.

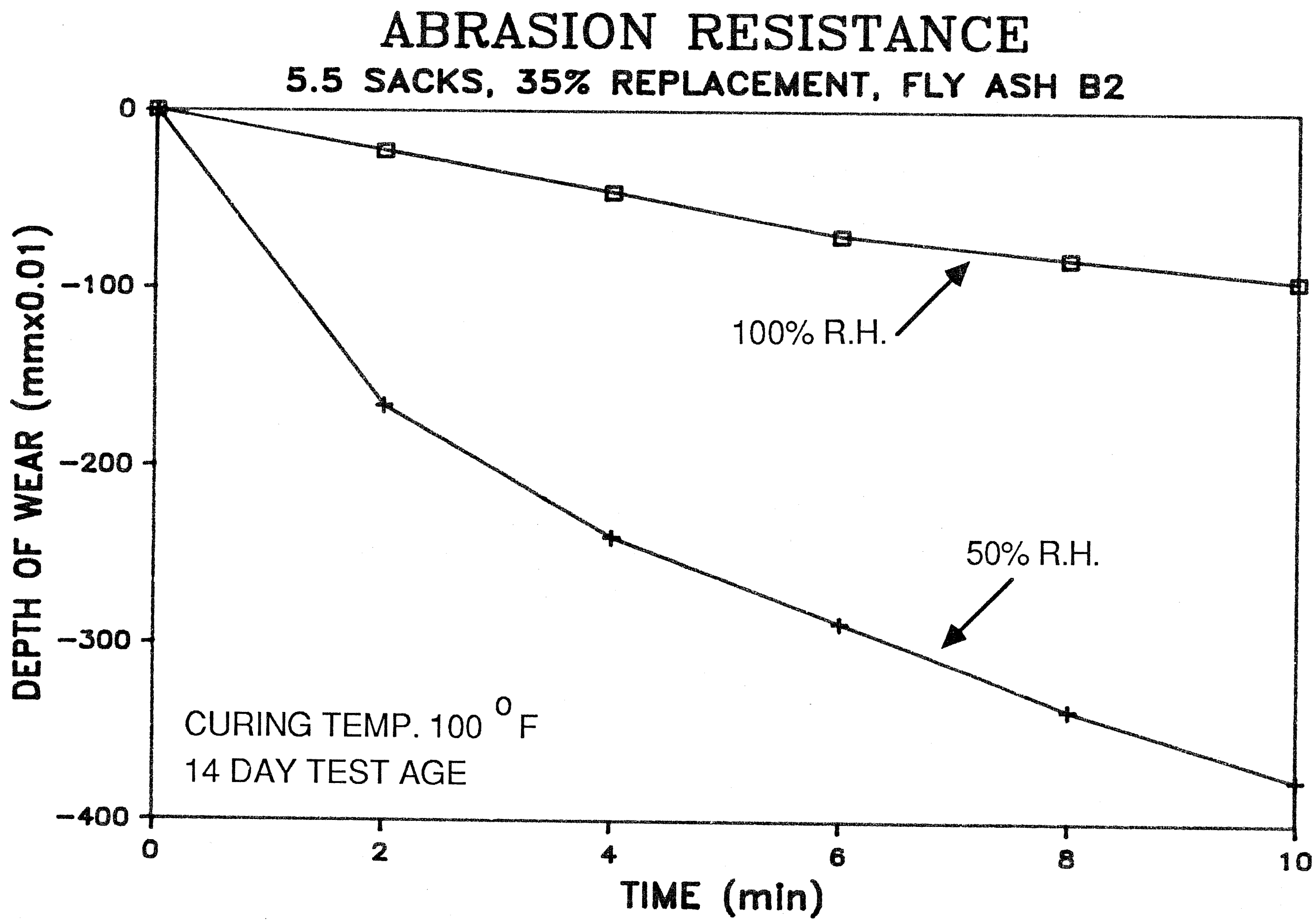

Fig. 4.42 Abrasion resistance of concrete for the 50-100B2-35 and 100-100B2-35 mixes.

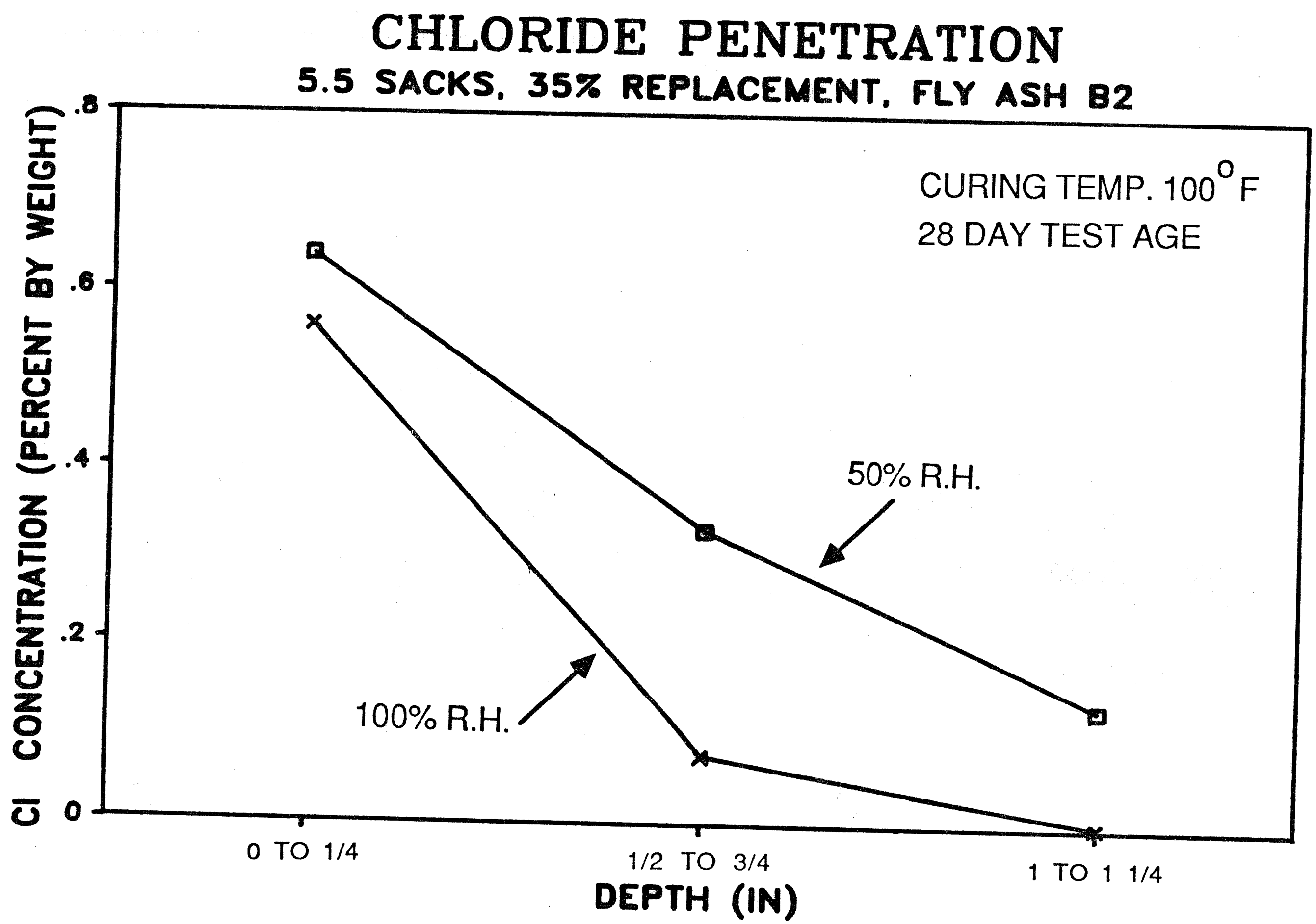

Fig. 4.84 Chloride penetration results for the 50-100B2-35 and 100-100B2-35 mixes.

CHAPTER 5
DISCUSSION OF THE TEST RESULTS

5.1 Introduction

The test results presented in Chapter 4 are the subject of discussion in this chapter. The influence of the fresh concrete properties, the compressive strength, the curing conditions, and the use of fly ash on the abrasion resistance and resistance to deicing scaling of concrete containing fly ash are the main focus of discussion. The compressive strength was used to compare and correlate the test results of the various mixes investigated in this study. General trends in the data are presented and discussed in detail. Wherever applicable, the test results of this experimental program are compared with the findings of previous research.

Any findings or conclusions regarding the durability of concrete containing fly ash are restricted to the properties of the fly ashes used in this study.

5.2 Effect of Fly Ash on Fresh Concrete Properties

The dosage of air entraining admixture required to entrain a specified amount of air was found generally to increase with an increase in the fly ash content of concrete. Figure 5.1 illustrates the relationship between the air entraining admixture dosage and the fly ash content for the various mixes used. This increase is mainly attributed to the carbon content of fly ash and its strong affinity for organic compounds. As previously mentioned in Sec. 3.2 titled "Mix Proportions," the mixes containing the Type A fly ash required more air entraining admixture than any of the mixes containing either of the two Type B fly ashes used. Plain concrete containing no fly ash required the least amount of admixture to entrain an equal amount of air. The increase in the cementitious factor for the mixes containing fly ash increased the amount of fines in the mixes thereby also contributing to the increase in air entraining admixture dosage.

All the concrete mixes were designed to have a slump between 1 and 3 in. Figure 5.2 shows that plain concrete mixes required a greater amount of water to achieve a similar slump than similar concretes containing fly ash. The fly ash particles which are spherical act like ball bearings thus reducing the amount of water needed to attain a given slump.

5.3 Effect of Fly Ash on the Flexural Strength

The 7-day flexural strength test results for all the mixes are summarized in Fig. 5.3. It could be observed from the figure that all the mixes met the minimum design flexural strength requirement as specified by the Texas SDHPT Specification Item 366. After adjustment for laboratory test conditions, the minimum allowed flexural strength was found to be 630 psi. In order to obtain comparable 7-day flexural strength test results from all the mixes, it was viewed necessary to increase the cementitious factor (CF) for the mixes containing fly ash. For a given fly ash, the CF increased with an increase in the percent replacement of fly ash. This increase in the CF is mainly attributed to the fact that the test was performed at an early age. At 7 days the pozzolanic activity of the fly ash has not yet contributed significantly to the strength gain of the concrete. The type A fly ash used required a higher cementitious factor to attain the required strength than the type B fly ashes used. As opposed to Type A fly ash, Type B fly ash contains a relatively high calcium oxide content and therefore contributes more to the early strength through the cementitious reaction.

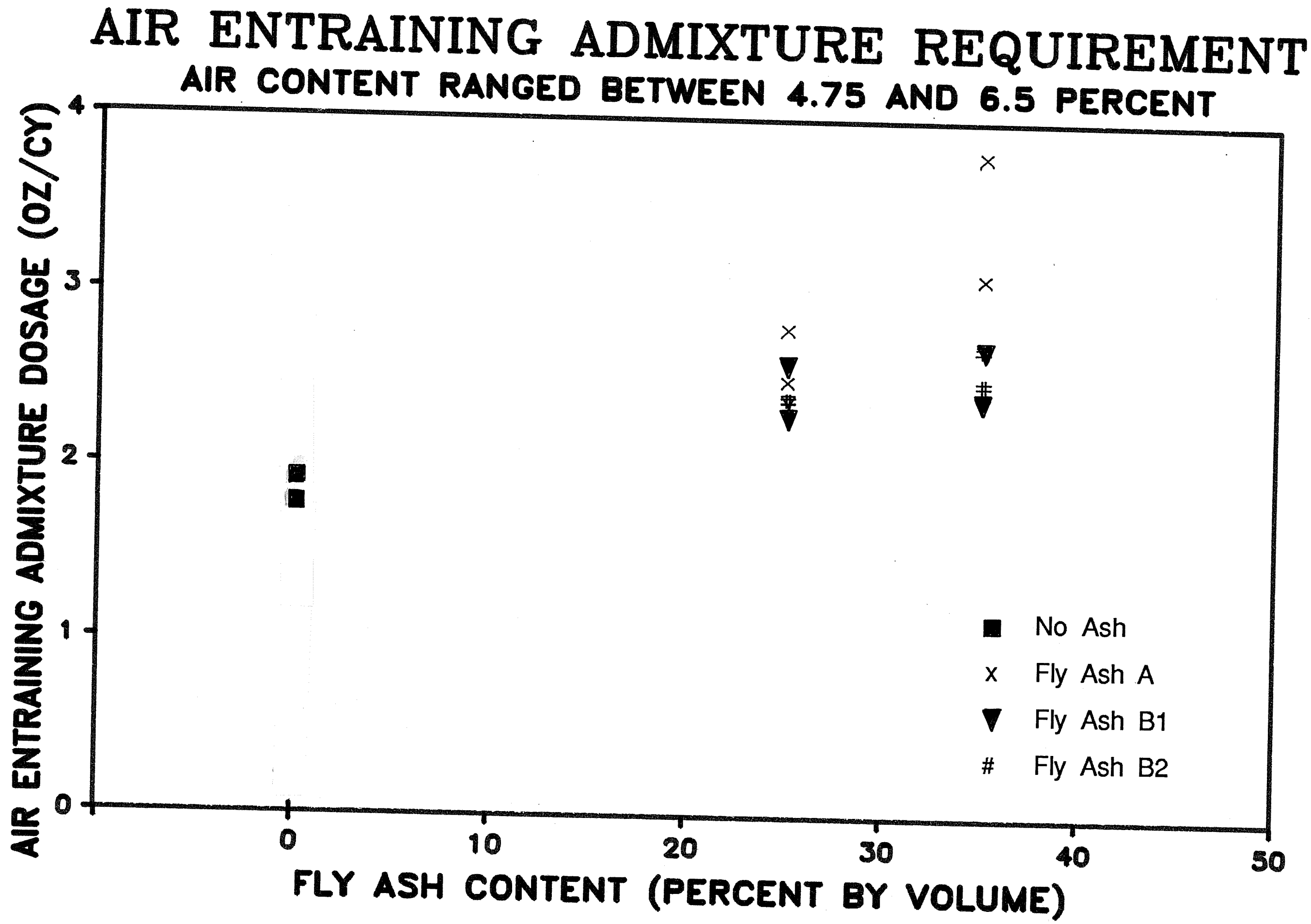

Fig. 5.1 Effect of fly ash on the air-entraining admixture dosage.

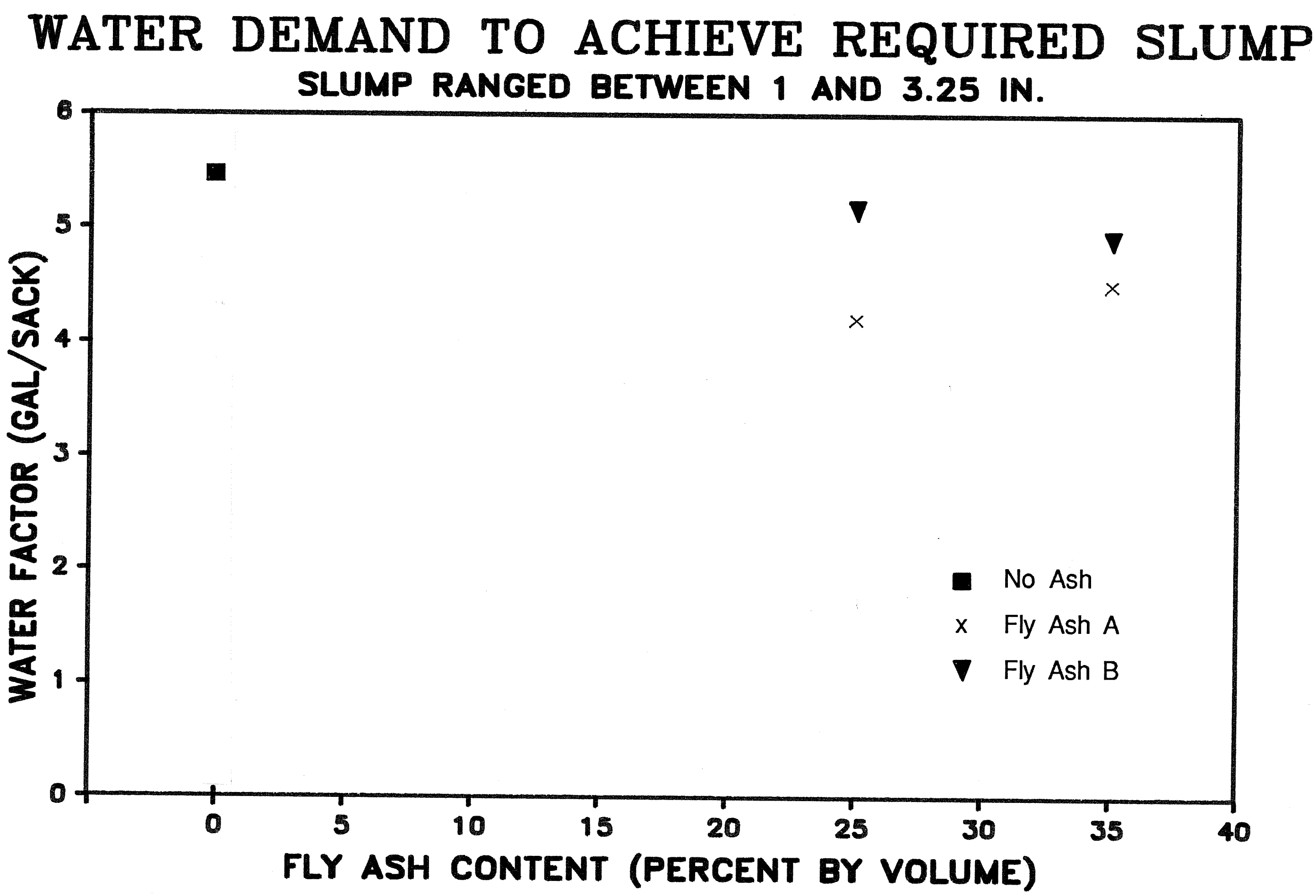

Figure 5.2 Effect of fly ash on the water demand to achieve the required slump.

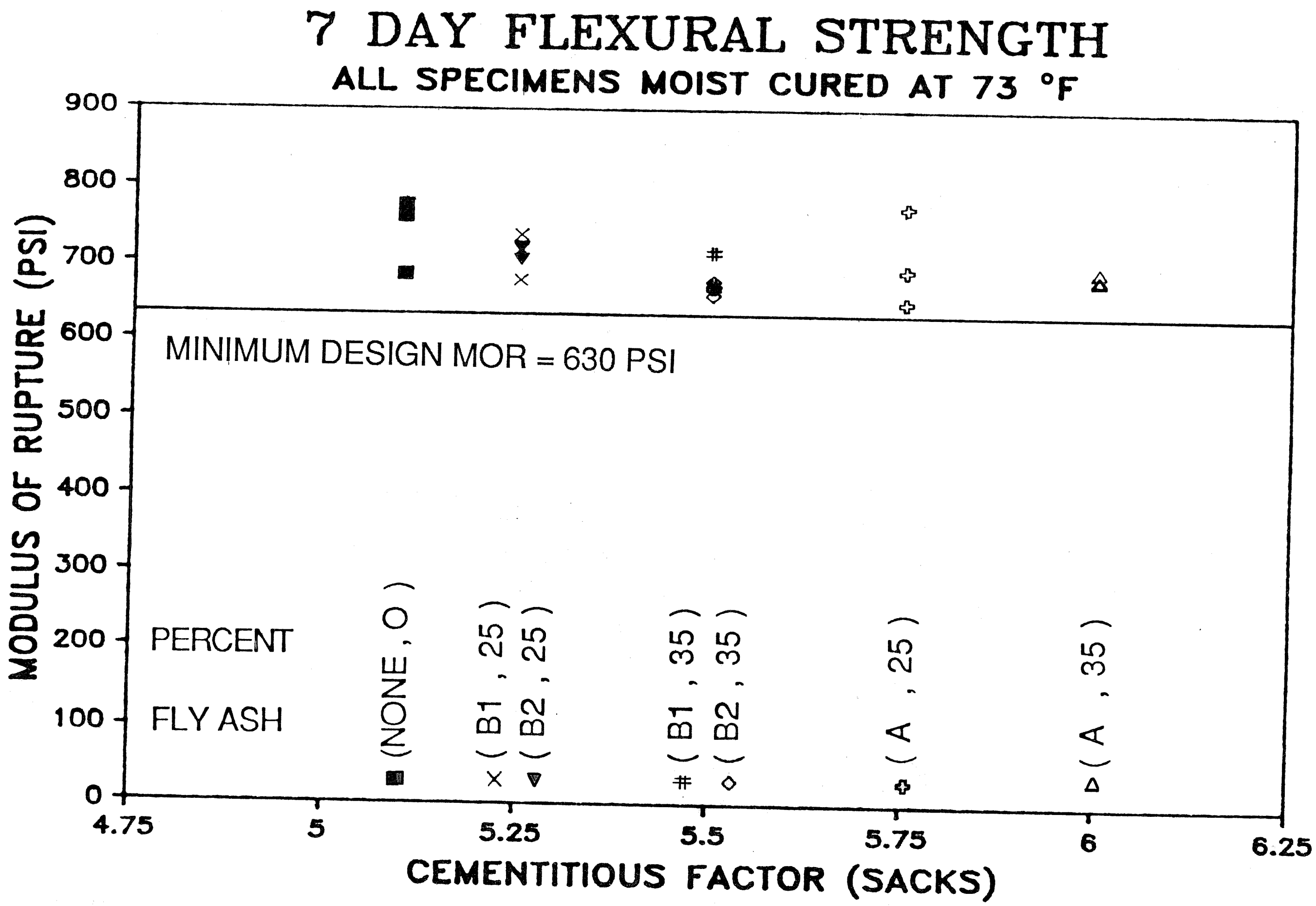

Figure 5.3 Seven-day flexural strength test results for the control specimens moist cured at 73°F.

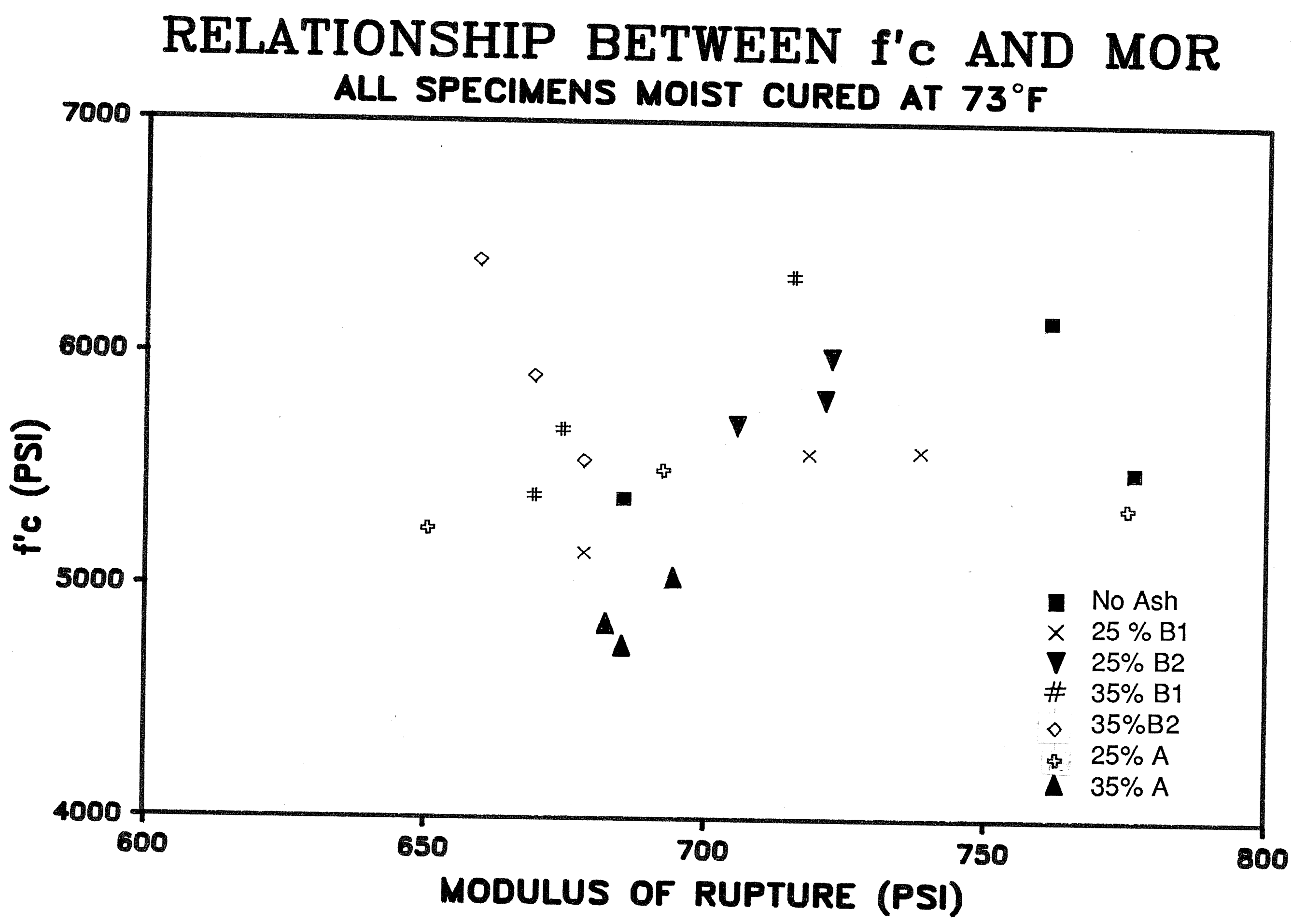

Figure 5.4 No relationship could be established between f'_c and MOR.

Figure 5.4 illustrates the relationship between the strength and the modulus of rupture for the most cured specimens at 73°F. Note that since the test results presented in Figure 5.4 represent a very narrow range of values for the modulus of rupture, no relationship could be established between f'_c and the modulus of rupture.

5.4 Effect of Curing Conditions on the Compressive Strength

The 7 series of concrete mixes containing fly ash investigated in this study were batched 3 times each, once for every curing condition. The 14- and 28-day compressive strengths versus cementitious factor for the 21 mixes used are plotted in Figs. 5.5 and 5.6. The test results presented in the figures are for the series of control cylinders moist cured at 73°F. The test data indicates that with the exception of the mix containing 35 percent replacement to cement with fly ash A the average compressive strength results for both the 14 and 28-day tests were comparable to those of plain concrete.

Figure 5.7 reveals that regardless of the cementitious factor, fly ash content, and type of fly ash used, the ratio between the 14- and 28- day compressive strength was constant and independent of the cementitous factor.

Figures 5.8 to 5.14 illustrate the trends in the compressive strength gain for the various mixes cured at 50, 73, and 100°F at 50 and 100 percent relative humidity, respectively. The 14-day and 28-day test data shows a general increase in the compressive strength with an increase in curing temperature at 100 percent relative humidity. This was not the case, however, for the control mix containing no fly ash. For that particular mix, the test data shows a decline in the strength both at 14 and 28 days when the test specimens are moist cured at 100°F. For the portion of the test specimens cured at 50 percent relative humidity, the compressive strength at 14 and 28 days was almost always lower than that of the specimens moist cured at the same curing temperature. A closer inspection of the test results for the mixes containing fly ash reveals the following general trend in the data. For a given mix cured at the three different temperatures, the difference in compressive strength between the moist cured specimens and the specimens cured at 50 percent relative humidity both at 14 and 28 days increases with an increase in temperature. This difference becomes more evident at 28 days and especially when the specimens are cured at 100°F. Figures 5.15 and 5.16 illustrate the typical trend in the compressive strength gain for concrete containing fly ash cured at both 50 and 100 percent relative humidity and tested at 14 and 28 days, respectively. In fact, in all cases but one, the strength of the specimens cured at 50 percent relative and 100°F is lower than that of the specimens cured at 50 percent relative humidity and 73°F. The primary reason behind this increase in strength differential is that for a constant relative humidity, the evaporation rate increases with an increase in temperature. At lower temperatures the specimens cured at 50 percent relative humidity retain more of the original moisture for the hydration of the cement paste for a longer period of time. As the temperature increases, more water evaporates and the hydration process is less complete. For the plain concrete mix, the difference in the strength between moist curing and curing at 50 percent relative humidity is not as nearly as pronounced as that of the mixes containing fly ash.

It is therefore concluded from the test results that concrete containing fly ash requires a more prolonged period of moist curing for adequate strength development than does plain concrete. These findings are in agreement with previous work done.[1,5,43] As opposed to plain concrete, the

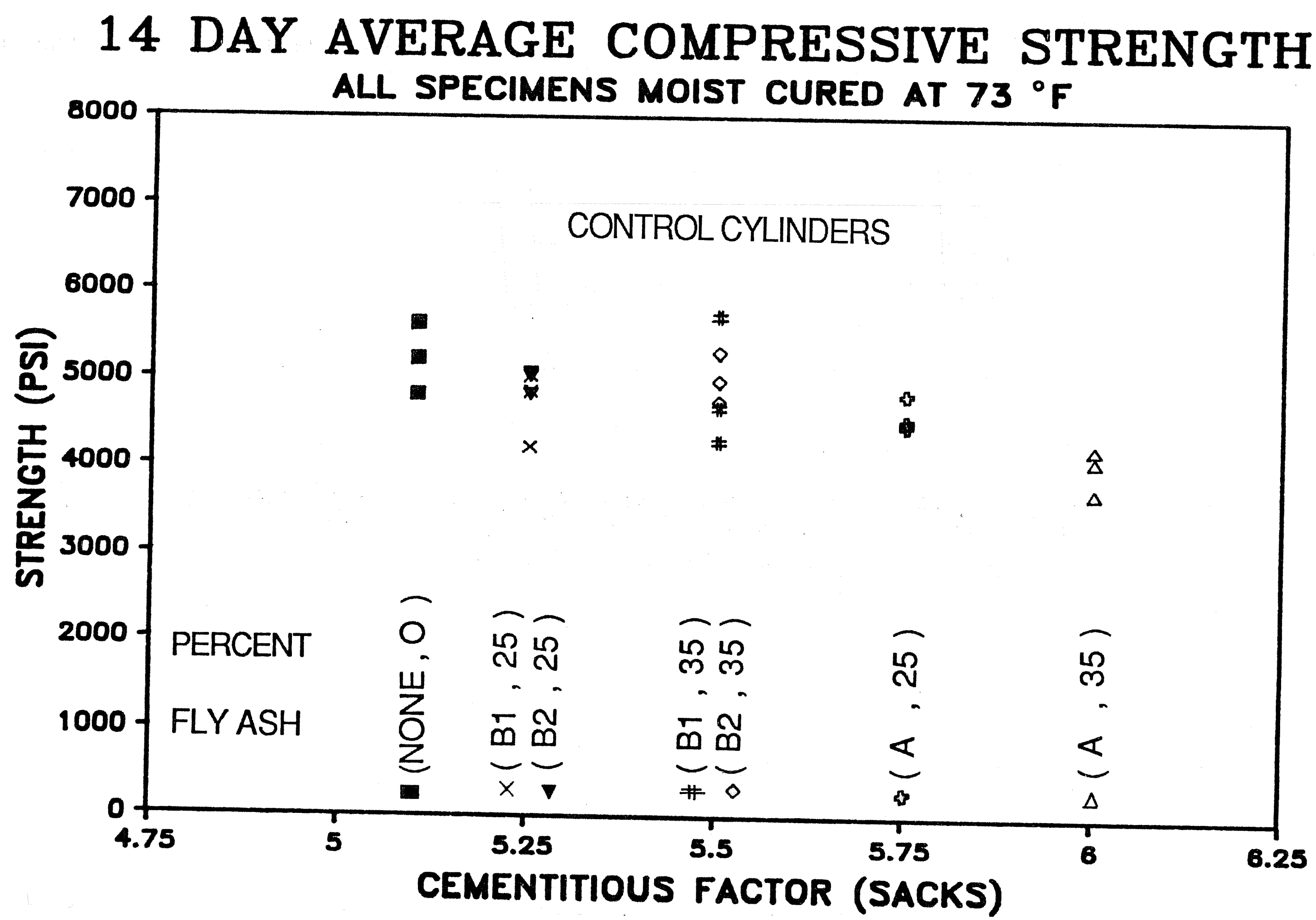

Figure 5.5 Fourteen-day compressive strength test results for the control specimens moist cured at 73°F.

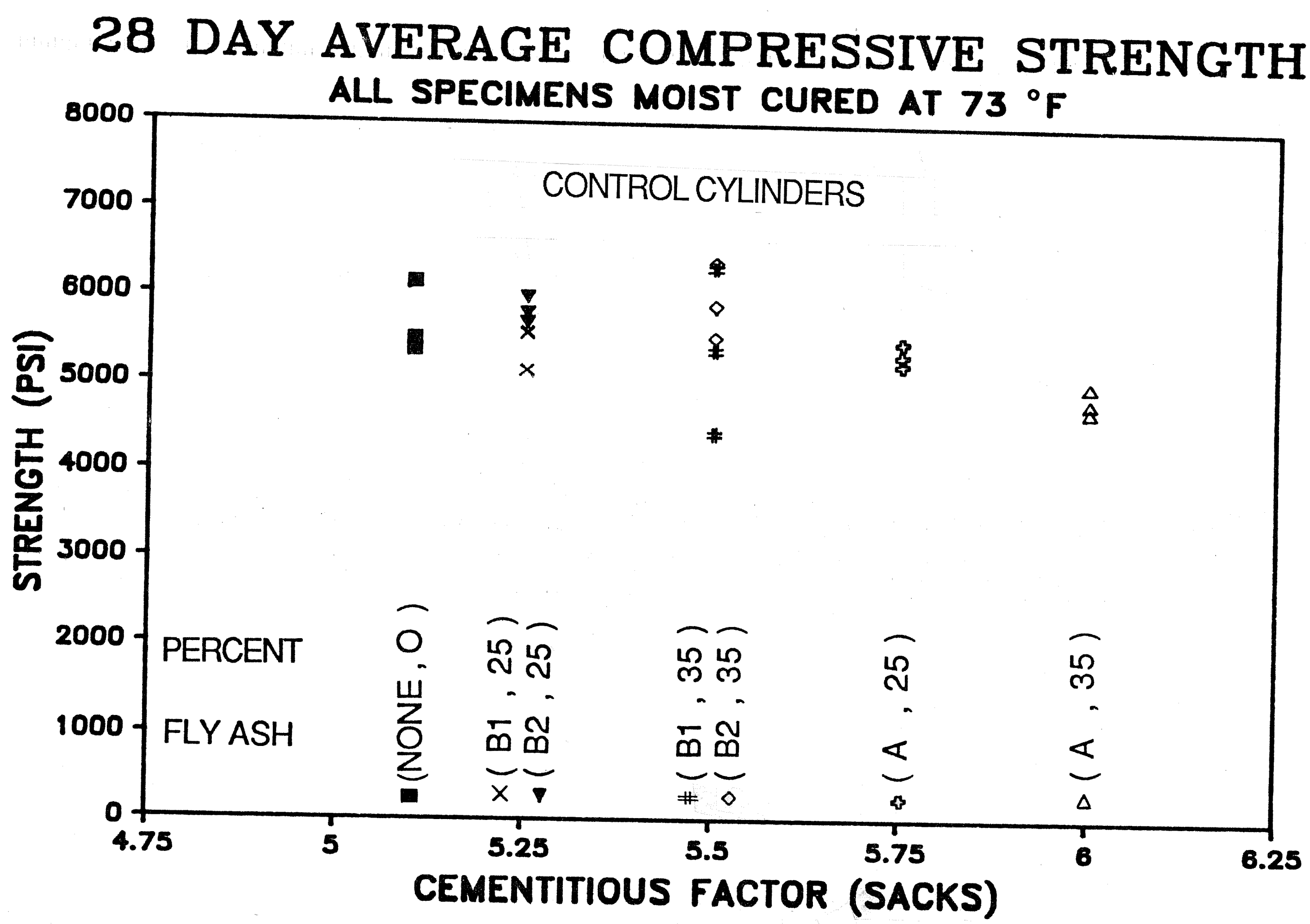

Figure 5.6 Twenty-eight day compressive strength test results for the control specimens moist cured at 73°F.

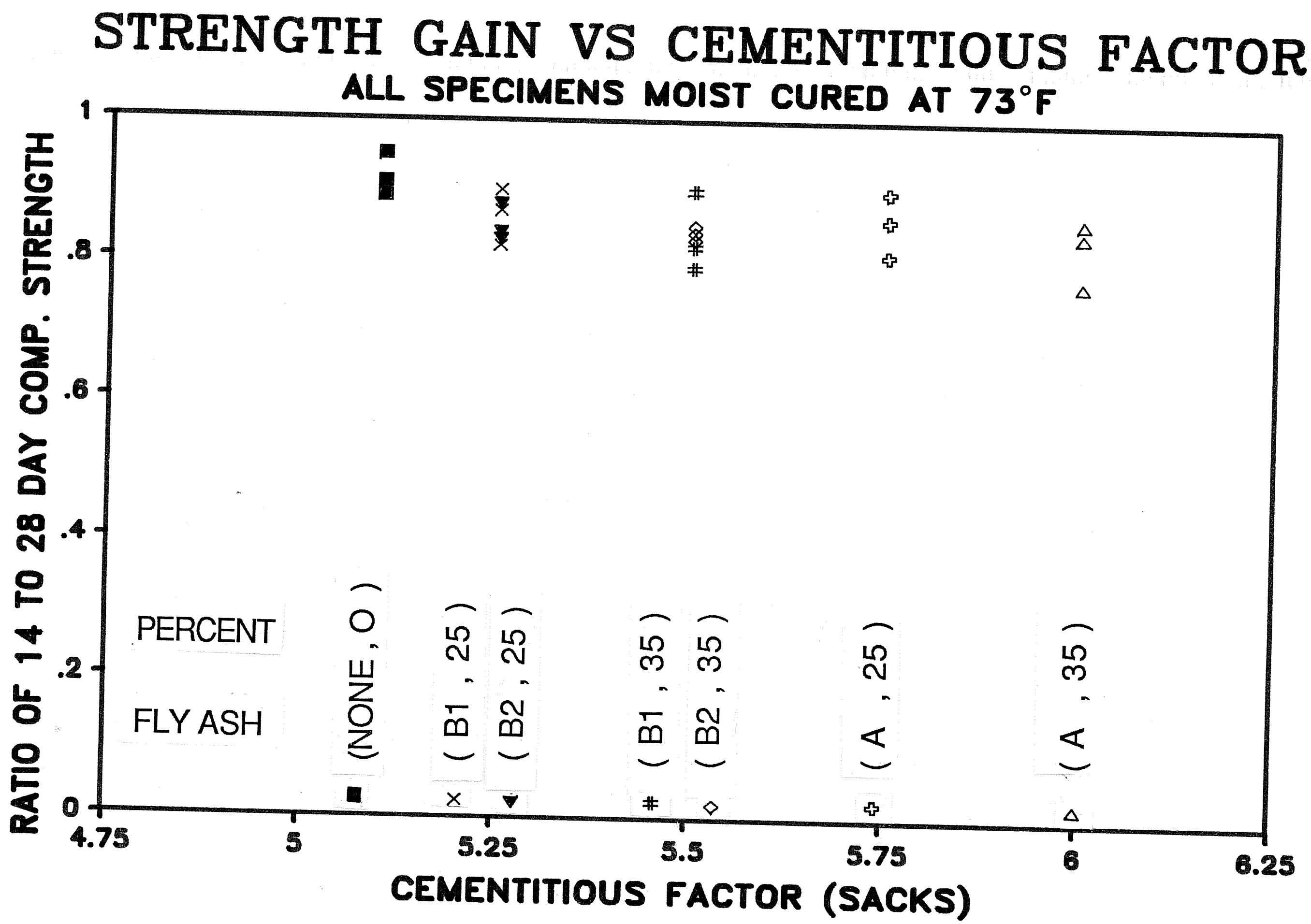

Figure 5.7 The relationship between the ratio of 14- to 28-day strength and the cementitious factor was constant in general.

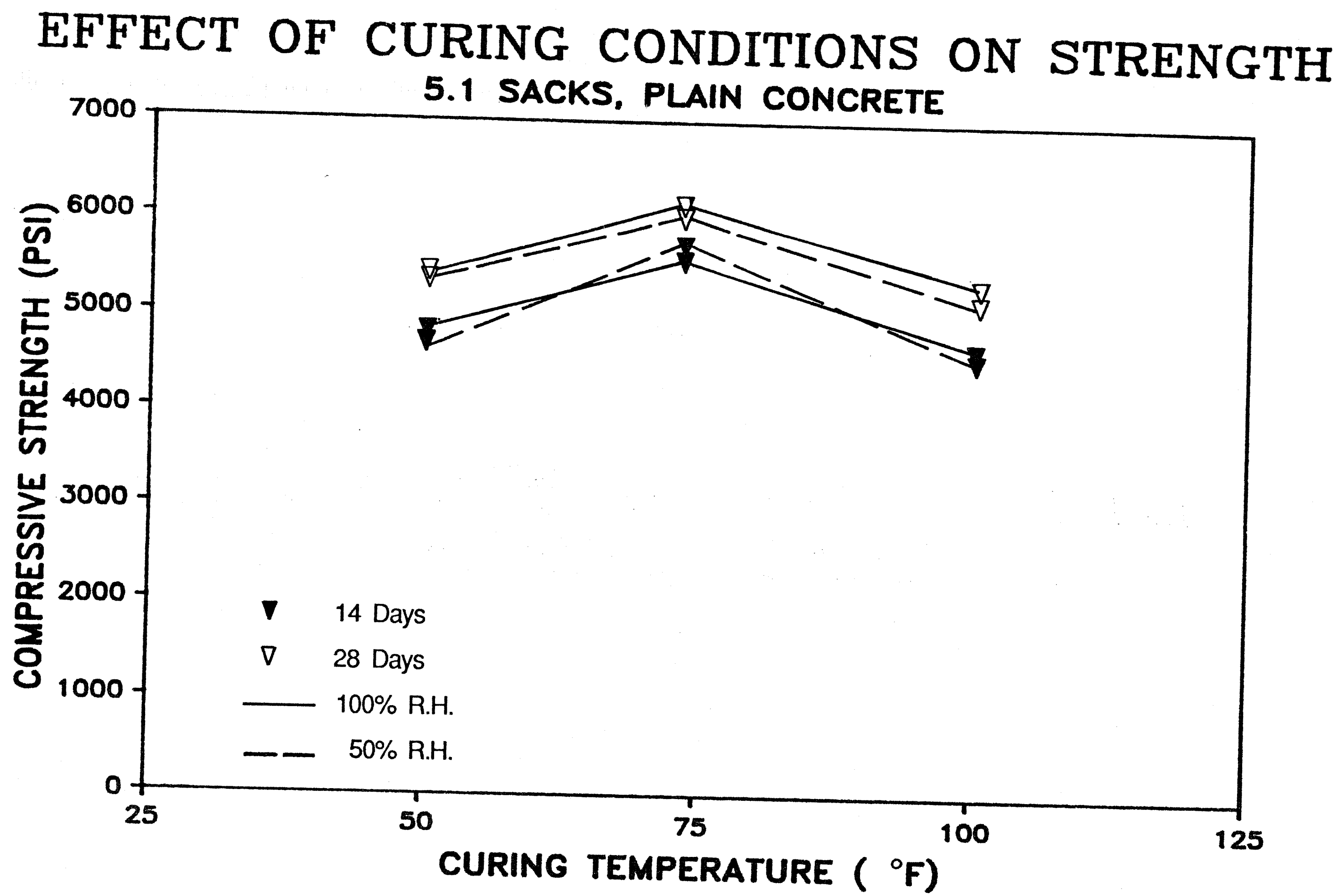

Figure 5.8 Effect of curing conditions on the 14- and 28-day compressive strength for the plain concrete mixes.

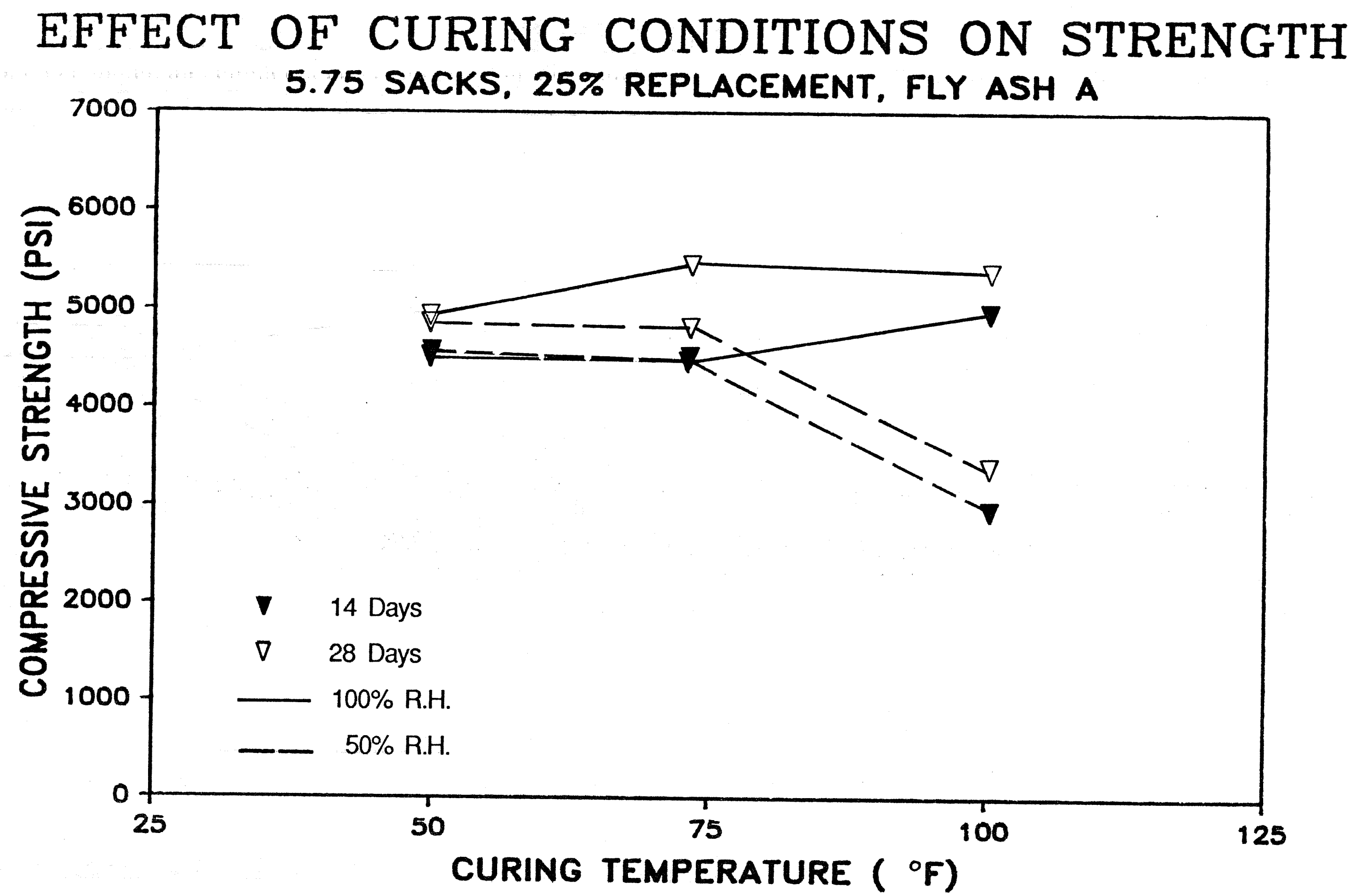

Figure 5.9 Effect of curing conditions on the 14- and 28-day compressive strength for the mixes containing 25 percent fly ash A.

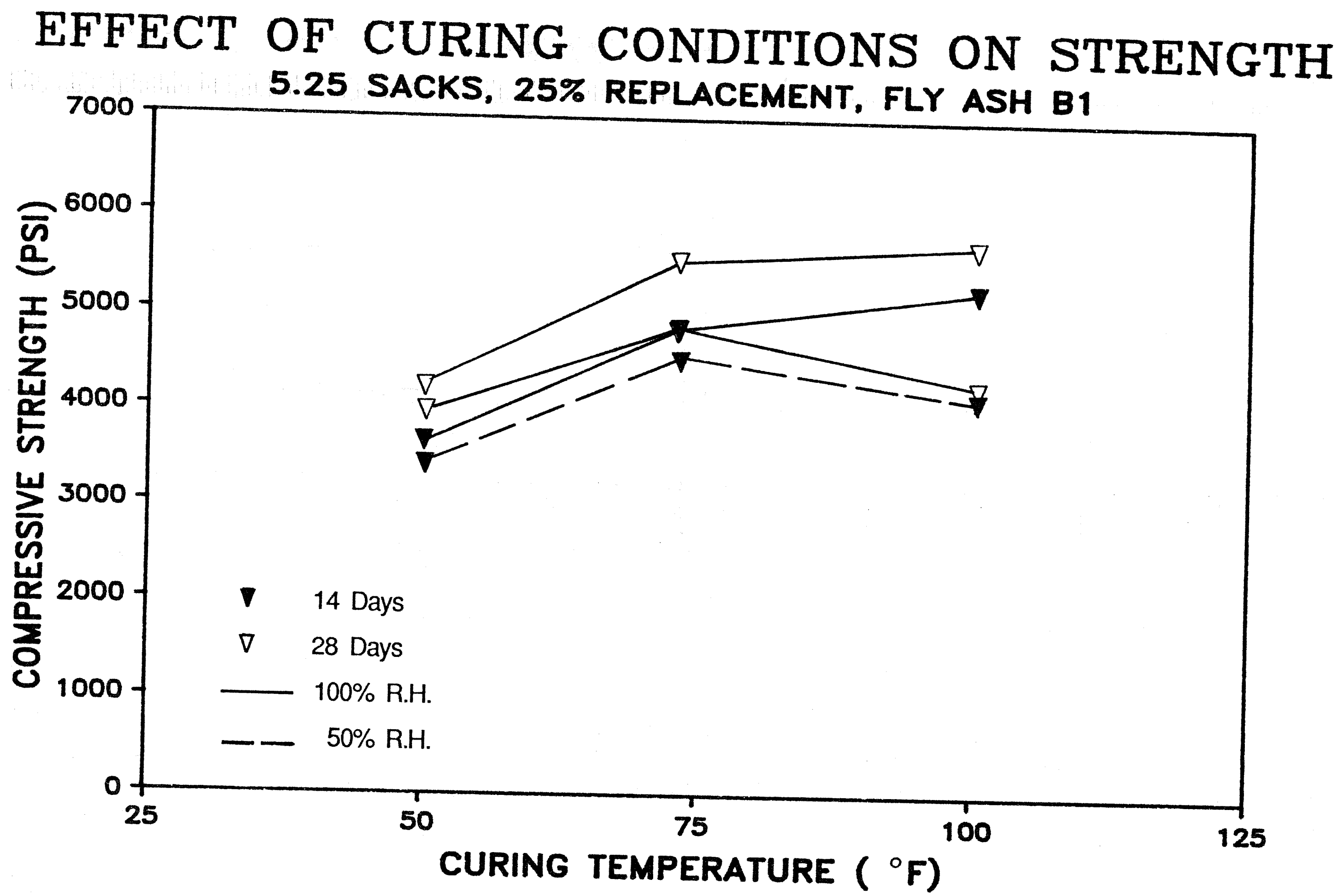

Figure 5.10 Effect of curing conditions on the 14- and 28-day compressive strength for the mixes containing 25 percent fly ash B1.

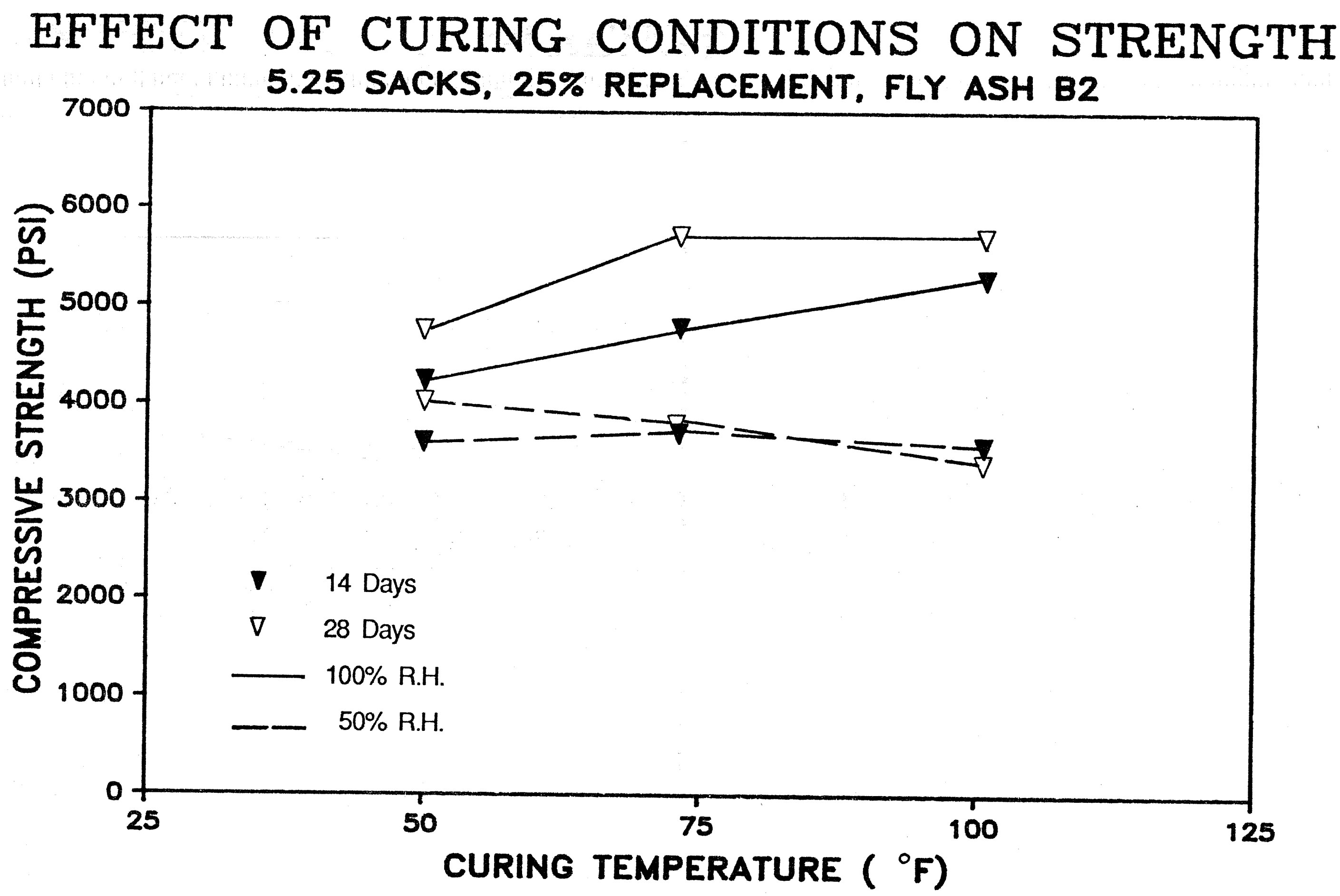

Figure 5.11 Effect of curing conditions on the 14- and 28-day compressive strength for the mixes containing 25 percent fly ash B2.

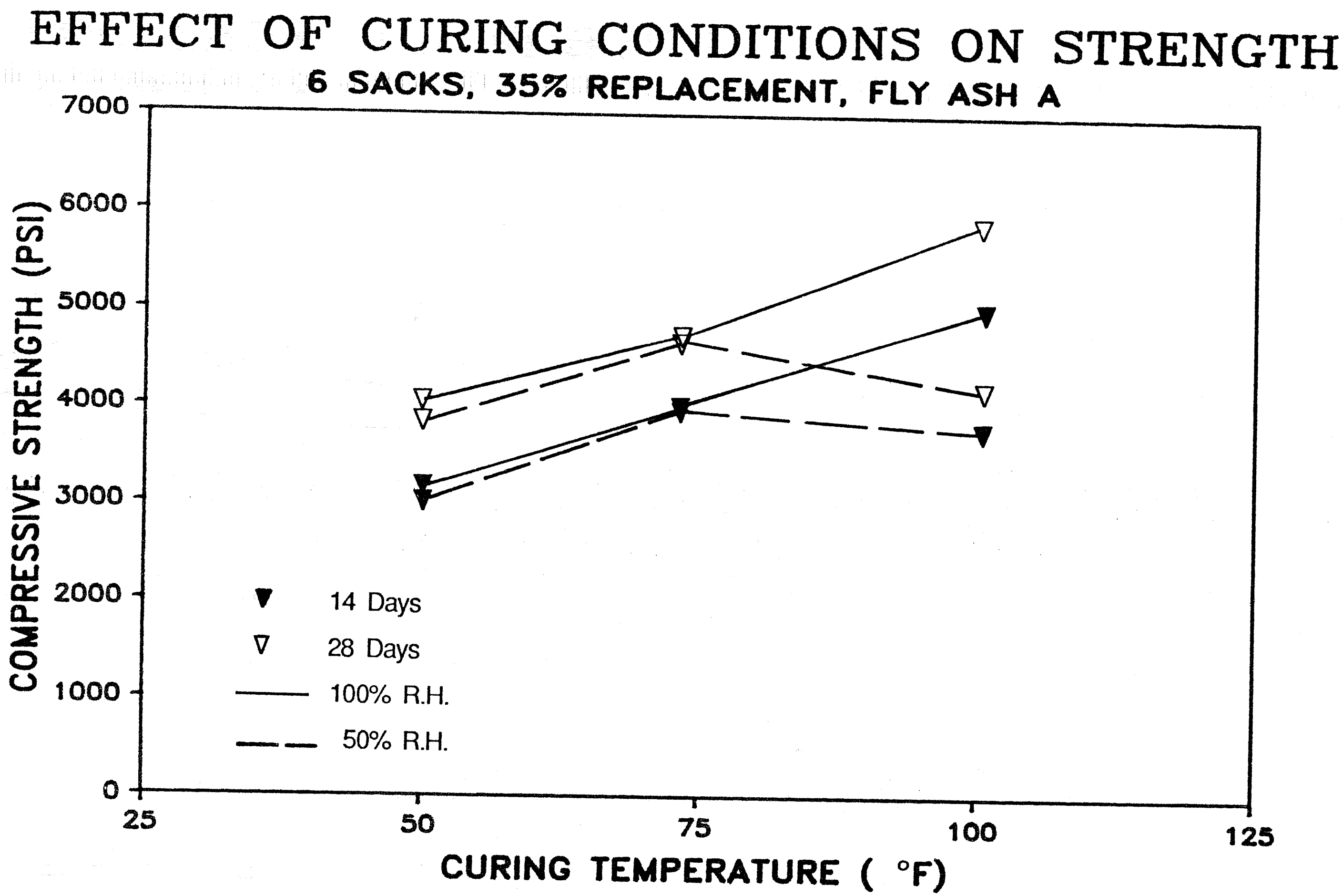

Figure 5.12 Effect of curing conditions on the 14- and 28-day compressive strength for the mixes containing 35 percent fly ash A.

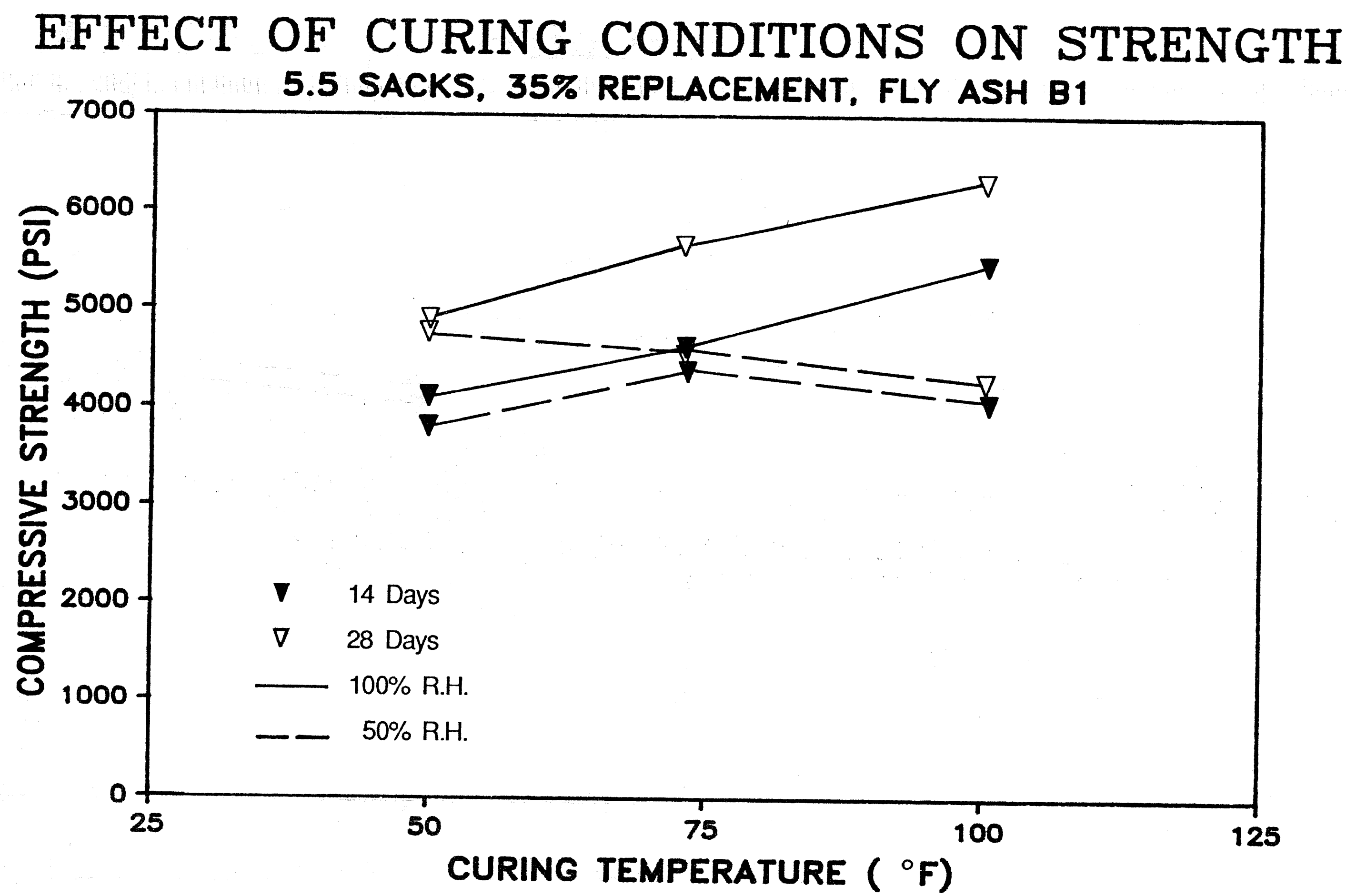

Figure 5.13 Effect of curing conditions on the 14- and 28-day compressive strength for the mixes containing 35 percent fly ash B1.

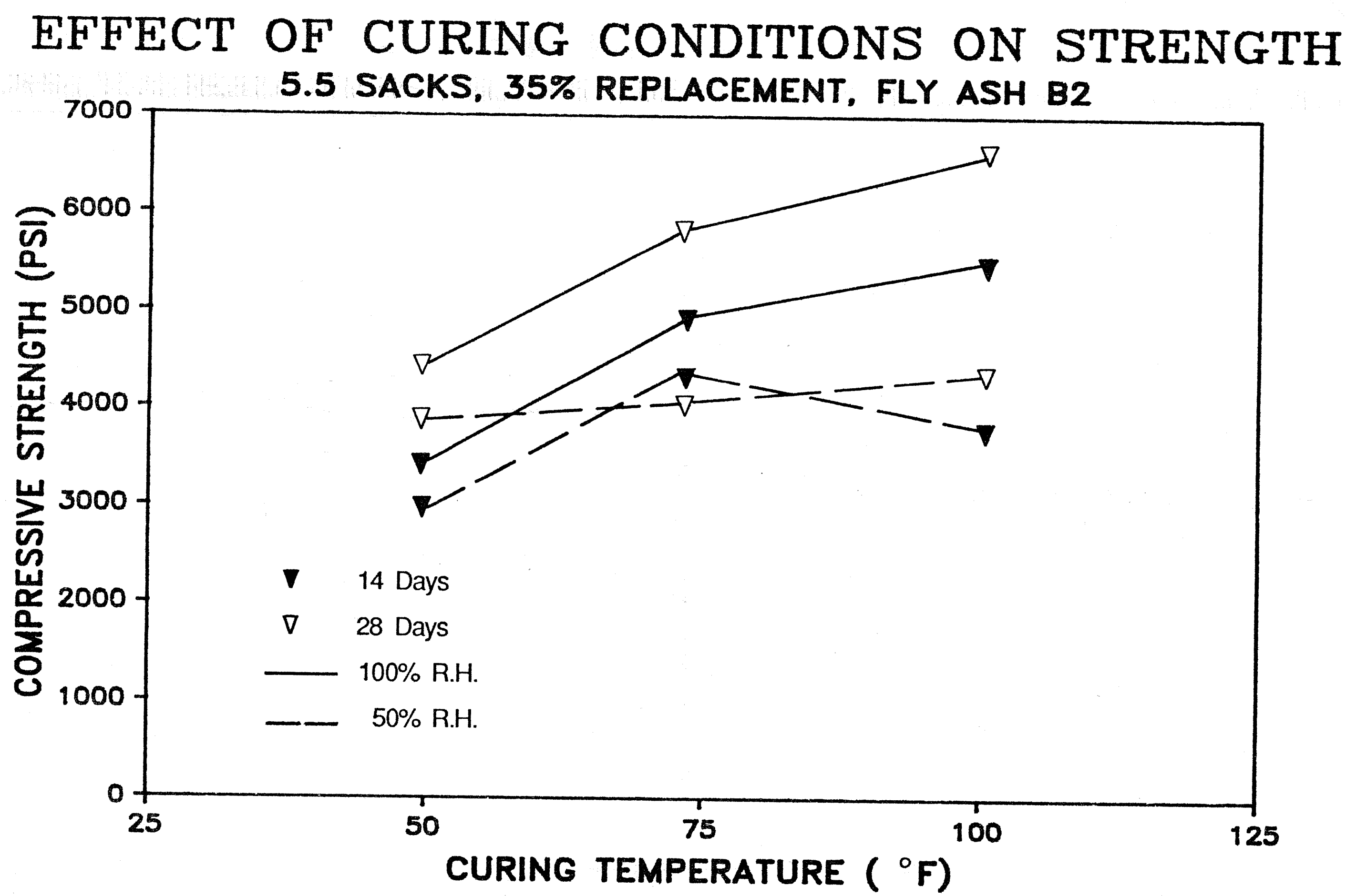

Figure 5.14 Effect of curing conditions on the 14- and 28-day compressive strength for the mixes containing 35 percent fly ash B2.

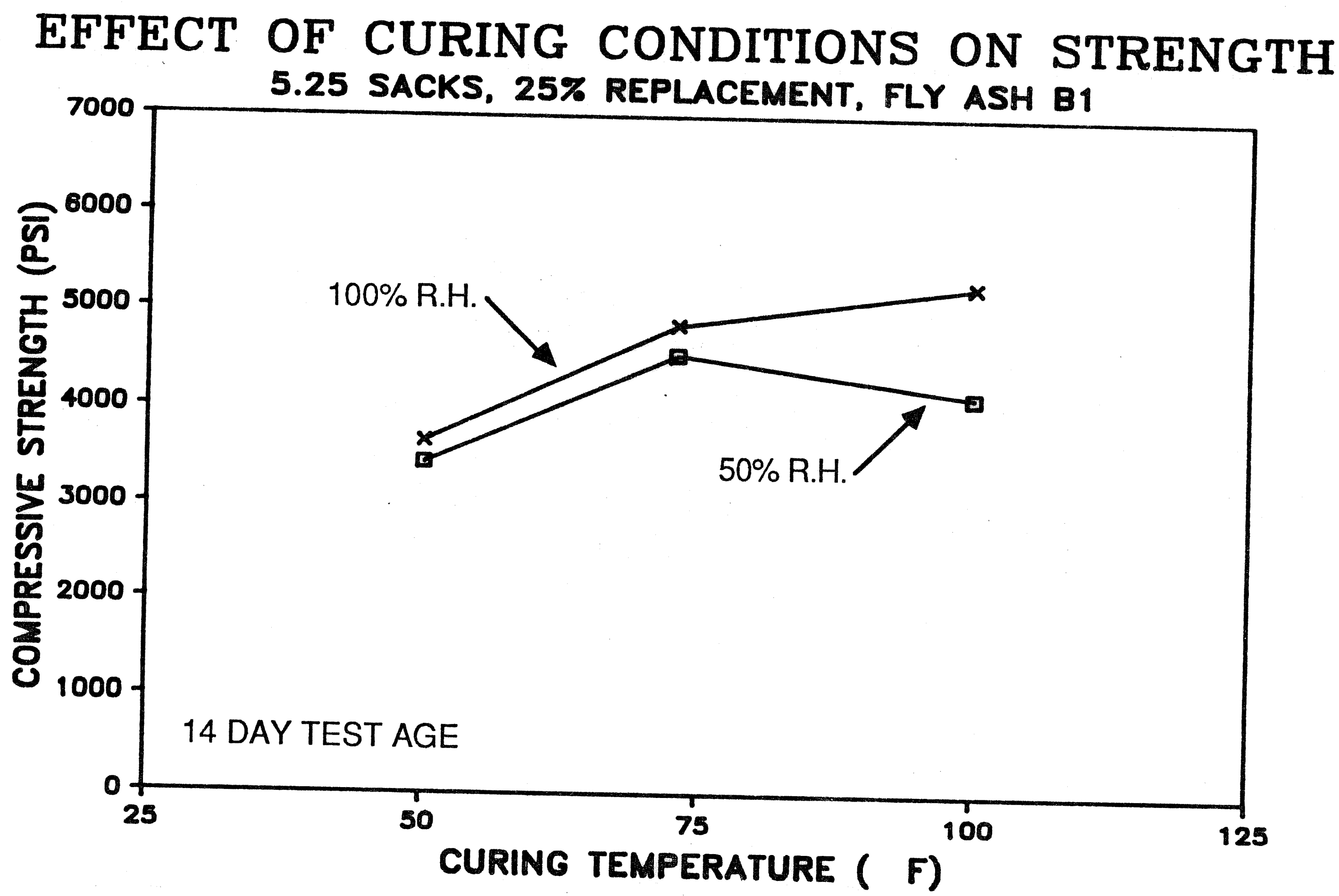

Figure 5.15 Typical trend for the 14-day compressive strength gain for concrete containing fly ash.

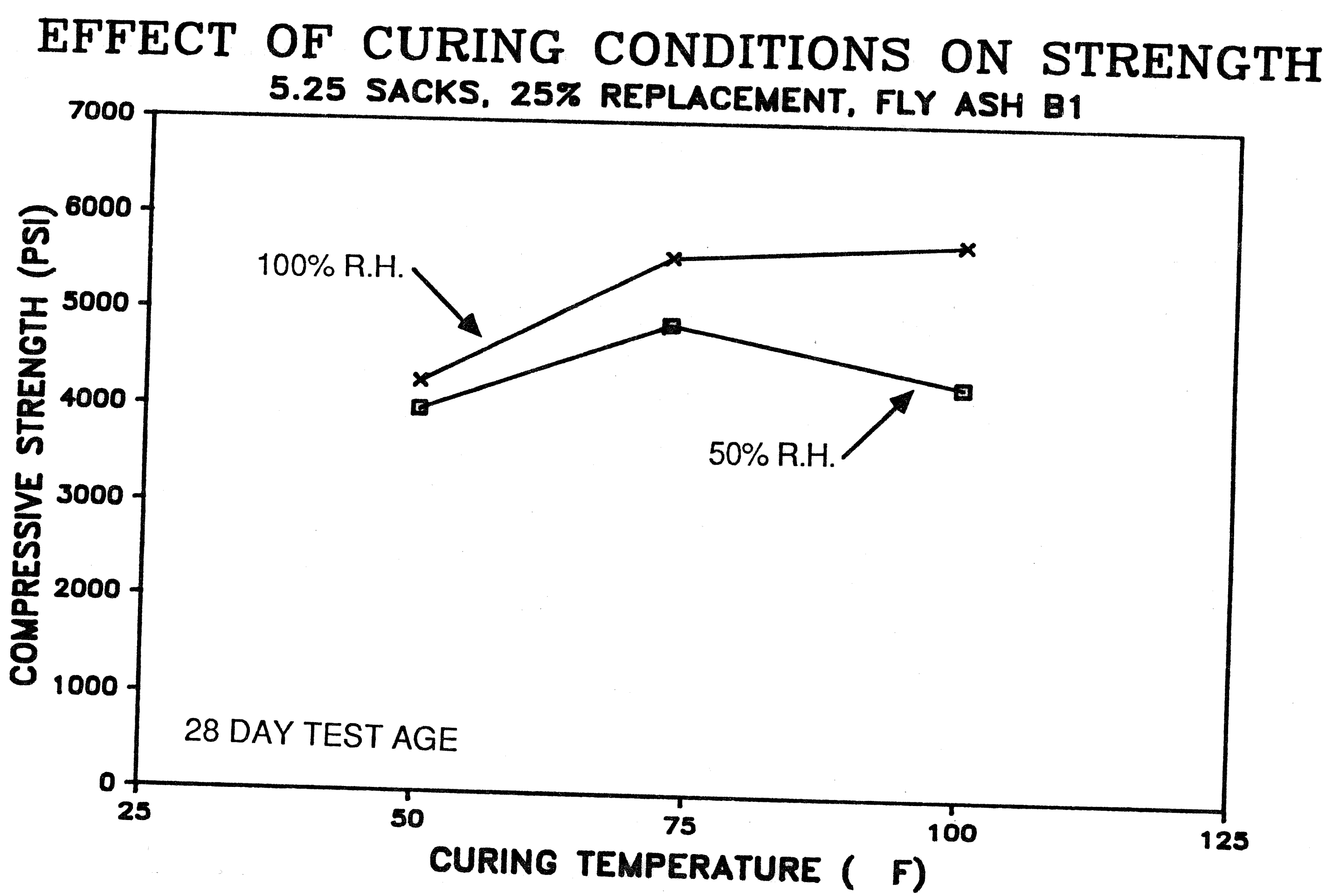

Figure 5.16 Typical trend for the 28-day compressive strength gain for concrete containing fly ash.

strength of moist cured concrete containing fly ash was found to keep increasing with prolonged curing as temperature increased. It is believed that at higher temperatures the pozzolanic activity is initiated earlier and enhanced, thus contributing to the strength gain faster.[6]

5.5 Abrasion Resistance of Concrete

5.5.1 Effect of Strength. The most important factor influencing the abrasion resistance of concrete was found to be the compressive strength at the day of testing. As discussed in the previous section, the curing conditions played a primary role in the strength development of concrete. Figure 5.17 clearly shows that as the strength increases the abrasion resistance of concrete increases. The best fitting straight line was obtained by performing a linear regression analysis of the data. Since some of the data was bivariate, (more than one value for the final depth of wear for every value of the compressive strength) the sample correlation coefficient (R) was used to associate between the depth of wear and the compressive strength rather than trying to estimate one variable from the other. The sample correlation coefficient varies between the limits -1 and 1. When R has an absolute value equal to 1, the relationship in the data is perfect. When R is equal to zero the variables are independent. For the data presented in Fig. 5.17, R was found to be equal to 0.81. This indicates that there is good agreement among the data points. The relationship between strength and abrasion resistance found herein is in agreement with the results of previous research.

As suggested by previous research,[46] a correlation between the compressive strength and voids-cementitious (water + air to cement + fly ash) ratio was attempted for the various curing conditions. Figures 5.18 and 5.19 reveal that the variables tilt towards being independent. The R factors for the two graphs were too small to establish any relationship between the compressive strength and the voids-cementitious ratio. This is contrary to what was expected and contradicts the findings of Witte and Backstrom.[46] It is believed that the different curing temperatures and relative humidities along with the incorporation of fly ash in the concrete mixes made it impossible to draw any conclusions regarding the relationship between compressive strength and the voids-cementitious ratio. It should also be noted that for a given mix with a certain voids cementitious ratio the test specimens which were cured at one of the curing temperatures at both 50 and 100 percent relative humidity behaved quite differently. The effect of curing conditions on the strength development was discussed in details in Sec. 5.4.

5.5.2 Effect of the Finishing Technique. The abrasion specimens were first finished with a wood float followed by several passes of a steel trowel. The concrete was not finished until all bleed water had evaporated and the surface of the concrete was stiff enough to withstand moderate thumb pressure. A typical profile of the incremental depth of wear versus time for the various curing conditions is illustrated in Figs. 5.20 through 5.22. The graph reveals that the test specimens suffered the most wear in the initial 2 minute period under the rotating cutters. In the following four 2 minute periods of abrasion, the depth of wear became smaller and relatively constant. The same observation was made for all the abrasion resistance test specimens. This is mainly due to the fact that the concrete near the surface is weaker. Although trowelling of the concrete was delayed until all the bleed water had evaporated, it nevertheless brought more moisture to the concrete surface. The water cementitious ratio near the surface was therefore higher, resulting in a weaker concrete layer near the top of the specimens.

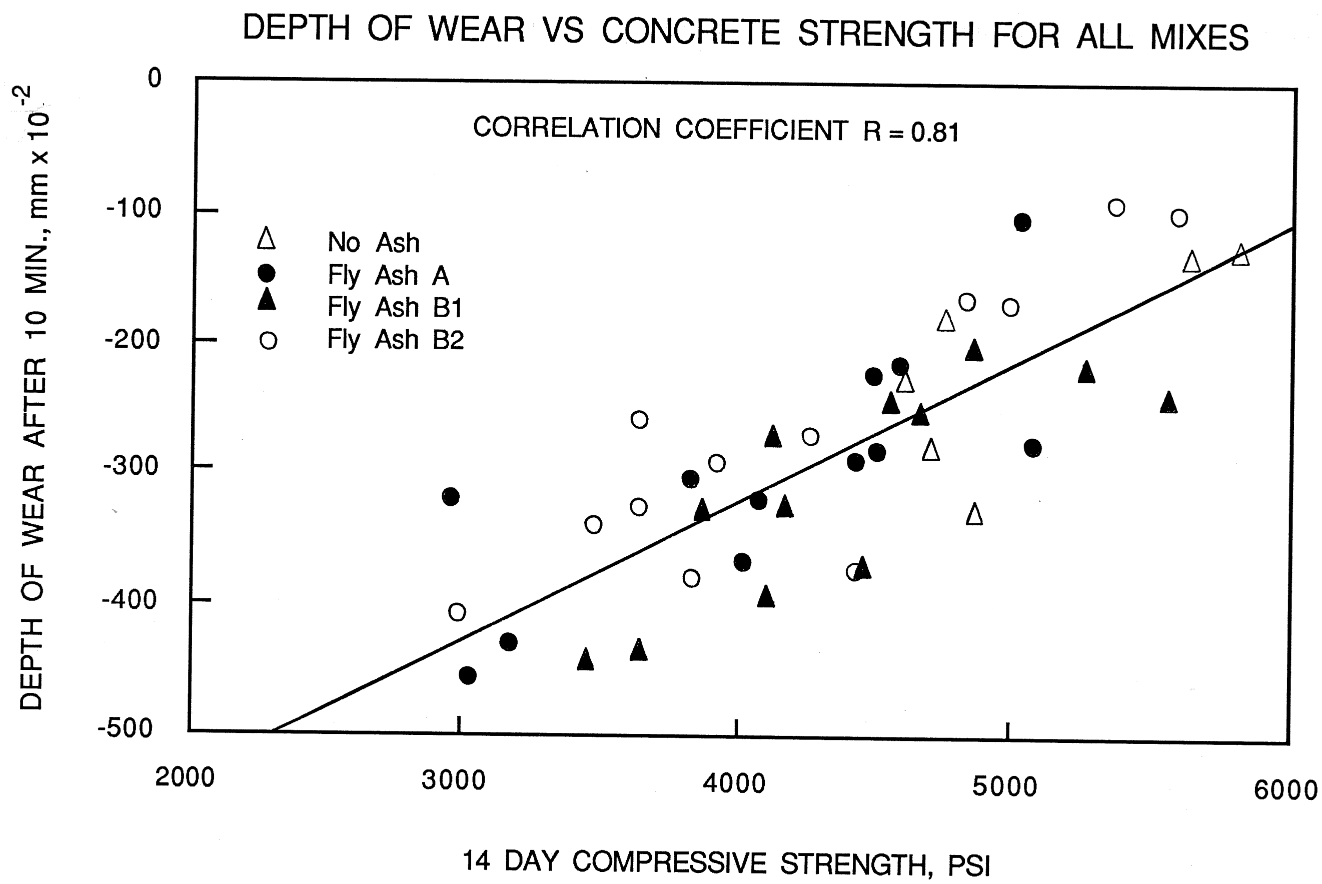

Figure 5.17 **Relationship between abrasion resistance and the 14-day compressive strength for all the mixes examined.**

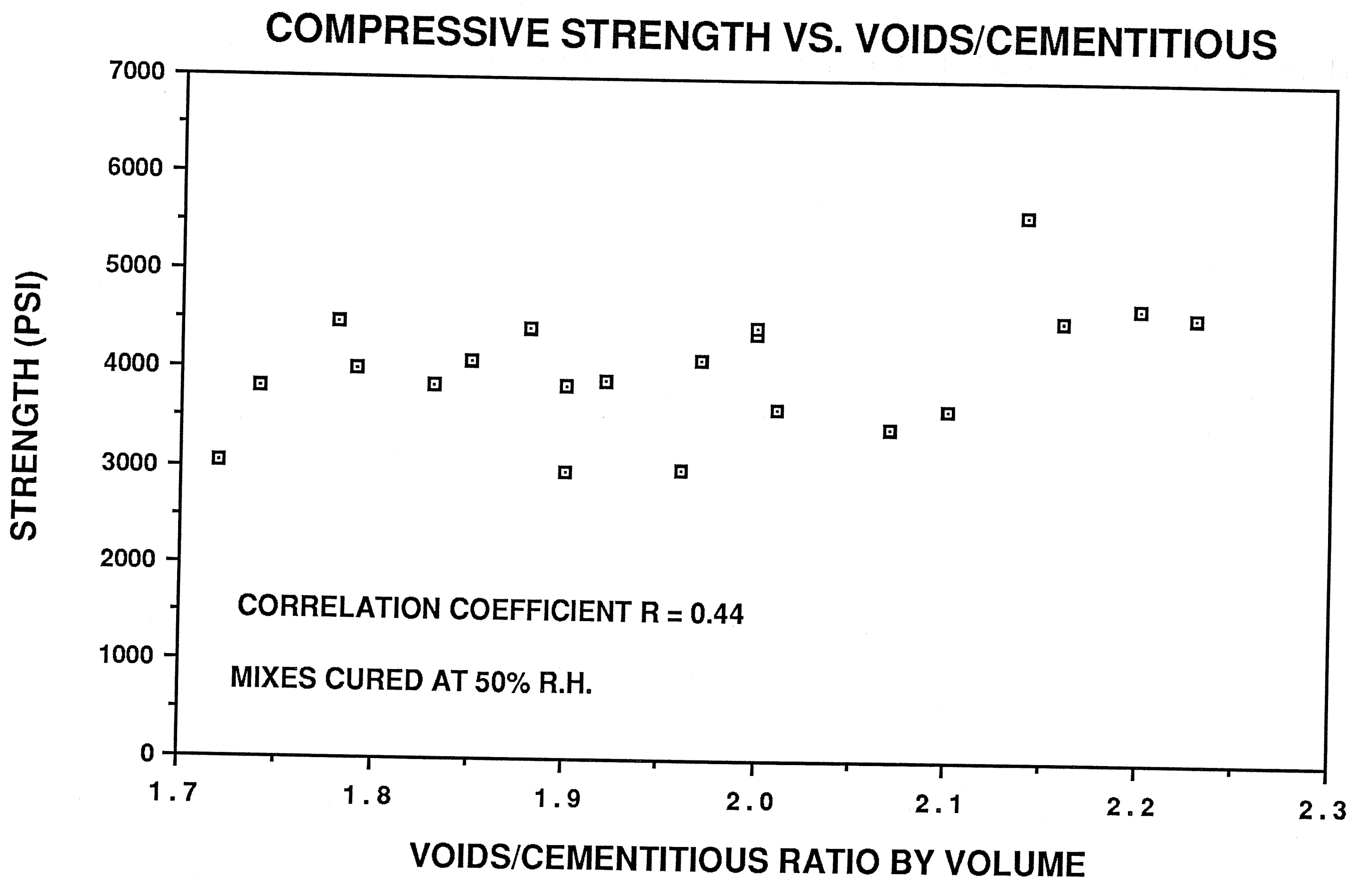

Figure 5.18 Plot of compressive strength versus voids - cementitious ratio for the mixes cured at 50 percent relative humidity.

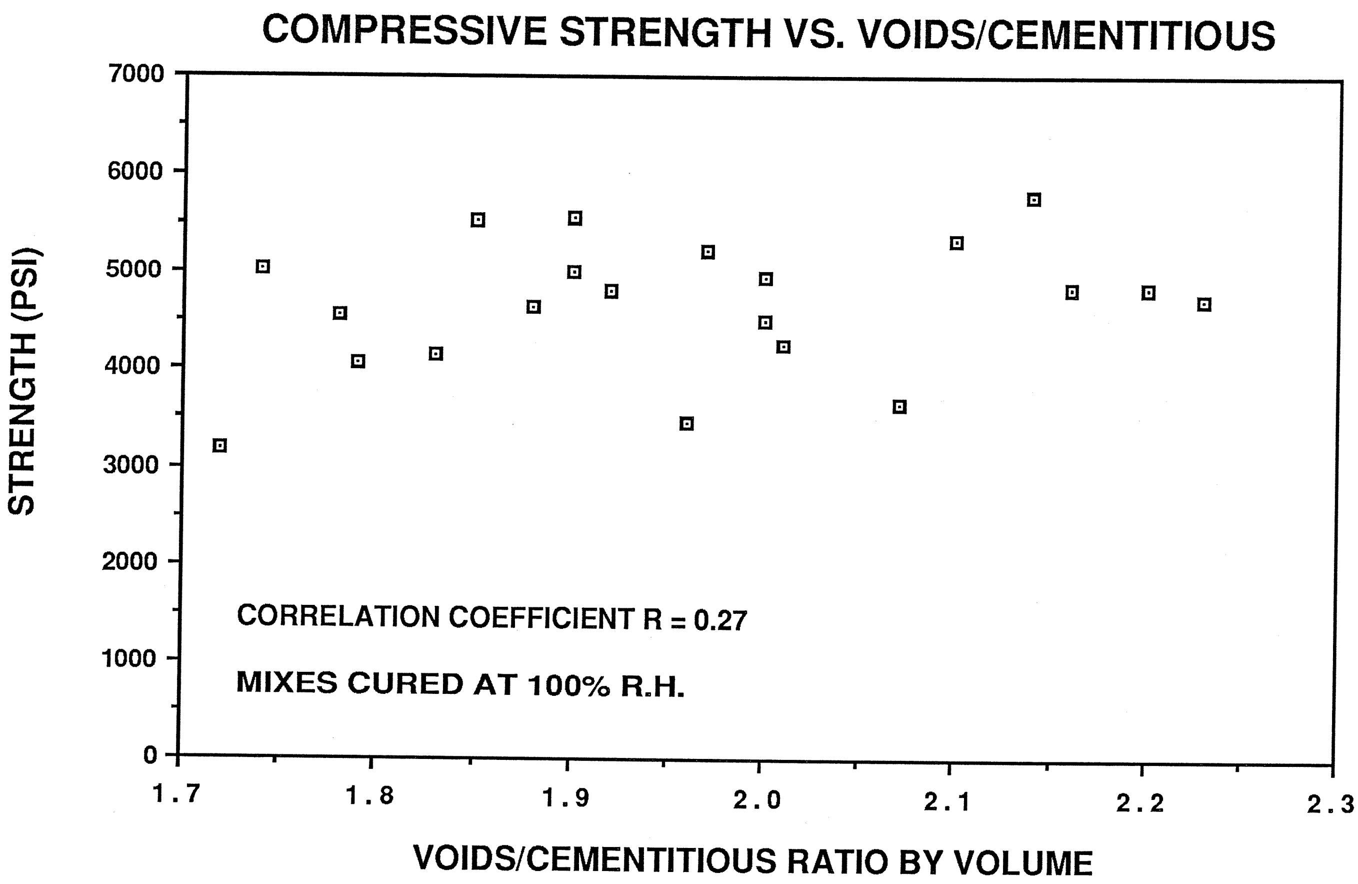

Figure 5.19 Plot of compressive strength versus voids - cementitious ratio for the mixes cured at 100 percent relative humidity.

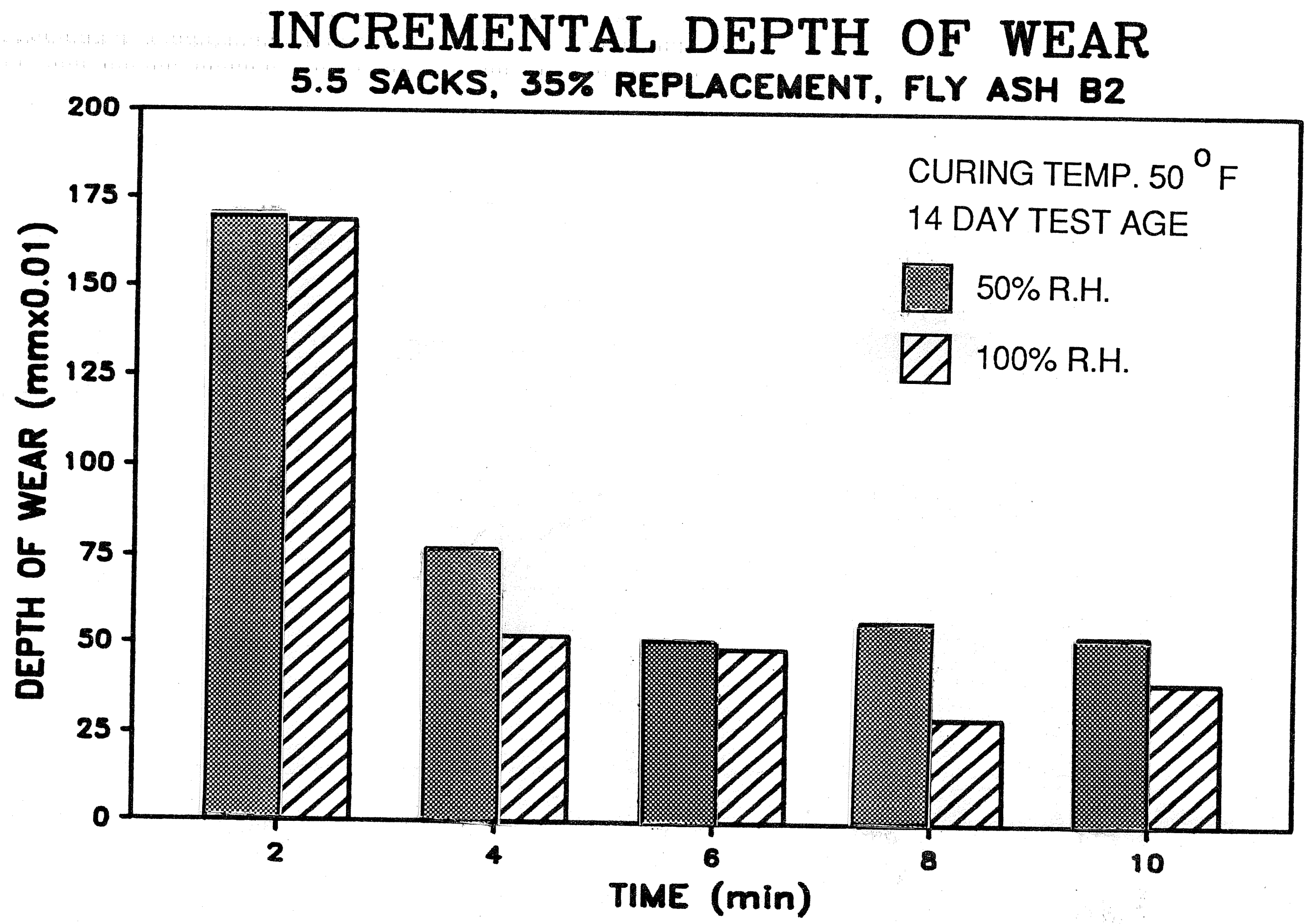

Figure 5.20 Typical incremental depth of wear for concrete specimens cured at 50°F.

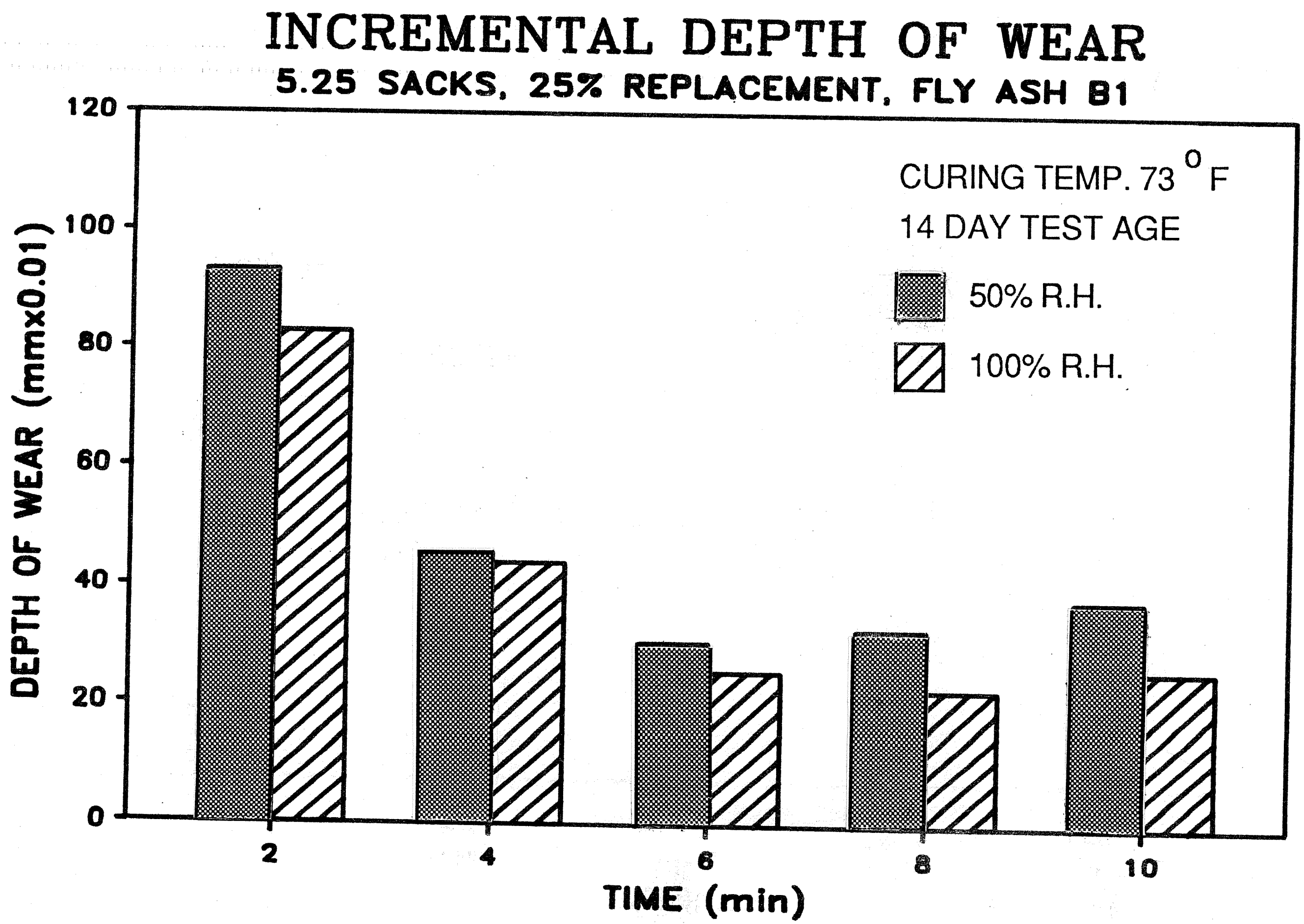

Figure 5.21 Typical incremental depth of wear for concrete specimens cured at 73°F.

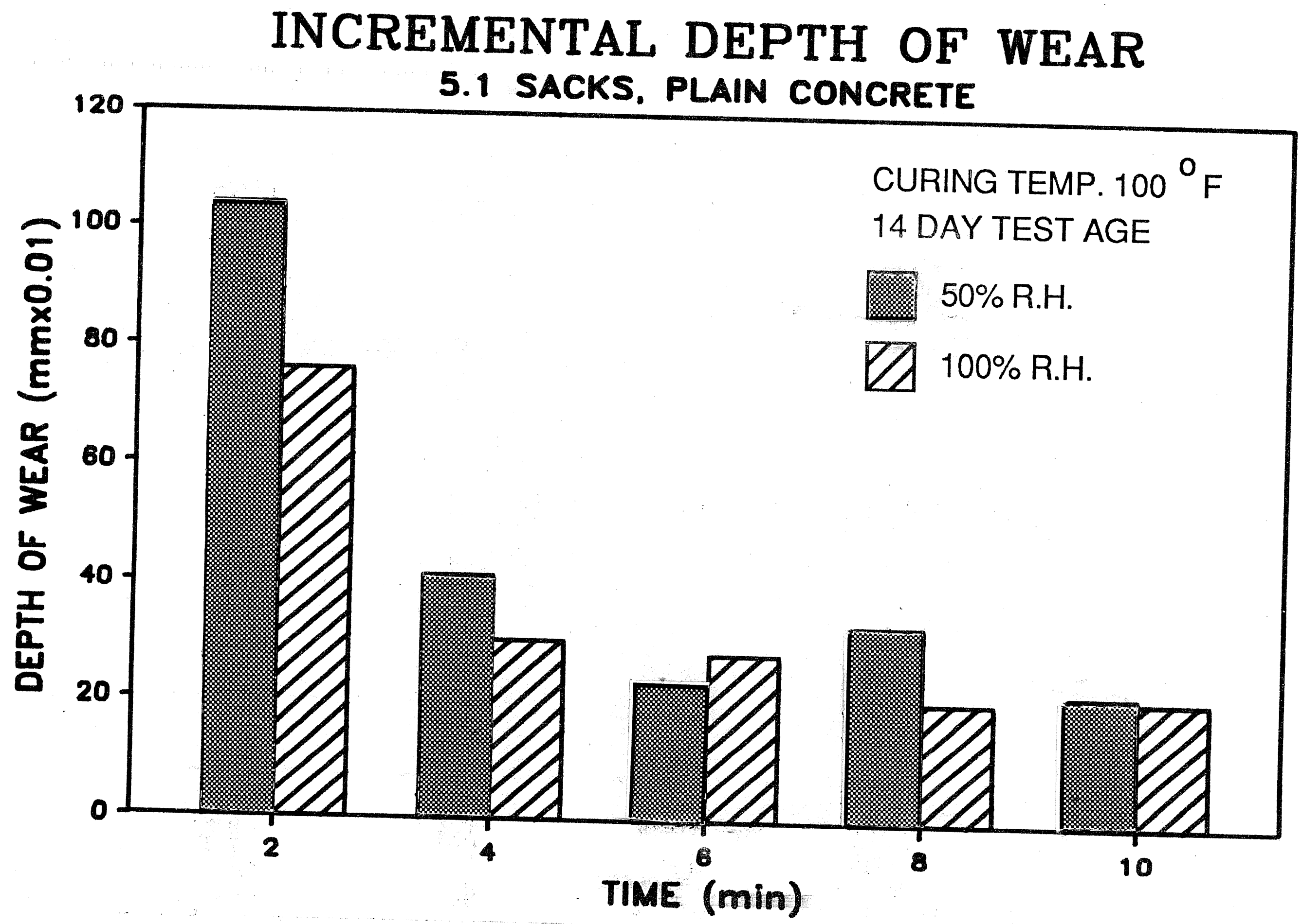

Figure 5.22 Typical incremental depth of wear for concrete specimens cured at 100°F.

These findings and observations are in agreement with the test results reported by Kettle.[20] Kettle cited that when compared to power finish, hand finishing of concrete significantly reduced its abrasion resistance. The increase in abrasion resistance of concrete when subjected to power finish is attributed to an increase in compaction and a reduction of the water cement ratio near the surface of the concrete.

5.5.3 Effect of Curing Conditions. The curing temperature influenced the abrasion resistance of concrete mainly in that it affected the strength of the concrete. The trend in the compressive strength gain of concrete subjected to various curing conditions was discussed in detail in Sec. 5.4.

The curing conditions were found to greatly affect the strength of the concrete especially near the surface. It is believed that at higher temperatures, the specimens cured at 50 percent relative humidity suffer a greater loss in strength in the outermost layers of the concrete than similar specimens cured at lower temperatures. As stated earlier, when temperature increases, the rate of evaporation at a constant relative humidity increases. This causes a more significant moisture loss near the surface of the concrete, which in turn prevents the complete hydration of the paste matrix. When compared to specimens cured at 100 percent relative humidity, the specimens cured at 50 percent relative humidity exhibited less abrasion resistance. A close inspection of Figs. 5.20 to 5.22 presented in Sec. 5.5.2 indicates that for the first 2-minute interval of abrasion time the difference in the incremental depth of wear between the moist cured specimens and the specimens cured at 50 percent relative humidity increases with an increase in curing temperature. For the following four 2-minute periods of abrasion the difference becomes relatively smaller.

In summary, the abrasion resistance of concrete is greatly influenced by the structure of the surface matrix of the specimens. The curing practices play a primary role in the abrasion resistance of concrete. Proper curing of the surface becomes more critical at higher temperatures due to the lack of moisture available for the formation of the hydration products. The test results presented herein are in agreement with the findings of Kettle[20] and Fentress[13] who studied the effects of curing practices on the abrasion resistance of concrete.

5.5.4 Effect of the Aggregate Properties. The aggregate used throughout this experimental program was 3/4-in. crushed limestone. The toughness and hardness of the aggregate influences the abrasion resistance of concrete to a great extent. Since the aggregate used in this study was relatively soft, the compressive strength of the concrete at the time of testing and the structure of the cement paste matrix played a primary role in determining the abrasion resistance of the concrete. In study 1117 (also conducted at the University of Texas) abrasion resistance was performed on specimens made with hard siliceous river gravel. A comparison of the abrasion test results between the specimens made with crushed limestone and those made with river gravel revealed the following. For a specified compressive strength, the specimens made with gravel exhibited a much higher resistance to abrasion than the specimens made with crushed limestone. In the case of crushed limestone, strength was found to be the governing factor affecting the abrasion resistance of concrete. For gravel however, the aggregate was so strong that no clear relationship could be established between the abrasion resistance and the strength of the concrete.

5.5.5 Effect of Fly Ash on Abrasion Resistance. The 14- day abrasion resistance test results show that the strength is the single most important factor affecting abrasion. For the concrete mixes containing fly ash studied in this research, the cementitious factor was increased accordingly to compensate for the slow early strength gain. When compared at relatively equal strengths, concrete containing fly ash is as resistant to abrasion as plain concrete. Fly ash was found to affect the abrasion resistance of concrete in that it affected its strength when the concrete containing fly ash was subjected to poor curing conditions.

Previous research[1] suggests that when compared to conventional concrete, concrete containing fly ash exhibits less resistance to abrasion at early ages. In the previous research referred to however, the concrete containing fly ash tested was weaker than the plain concrete at the time of testing.

5.6 Scaling Resistance and Chloride Penetration of Concrete

5.6.1 Scaling Resistance of Concrete.

5.6.1.1 Effect of the water-cementitious ratio. The water- cementitious ratio [W/(C+FA)] for the mixes used in this study varied between 0.41 and 0.51. There was no clear indication that the scaling resistance of concrete decreases with an increase in the W/(C+FA) ratio for mixes within the range from 0.41 to 0.51. It can be seen from Figure 5.23 that the scaling resistance of concrete containing fly ash was independent from the W/(C+FA) ratio.

The air content for the mixes studied ranged between 4.75 and 6.5 percent. Although no trend could be established here, previous research(22,27,39,44) has shown that air-entrained concrete is more resistant to deicer scaling than non-air-entrained concrete.

5.6.1.2 Effect of the Coarse Aggregate. As stated earlier, the aggregate used in this study was 3/4-in. crushed limestone with a high absorption capacity of 3.5 percent. As a result, the damage due to the freeze-thaw action in the presence of a deicer salt solution occurred mostly over and around the porous coarse aggregate. In Study 1117, a similar scaling resistance test was performed on air-entrained specimens made with a low absorption capacity gravel. A comparison of the test results indicates that concrete made with low absorption capacity aggregates is more resistant to deicer scaling than concrete made with a high-absorption capacity aggregate.

5.6.1.3 Effect of Curing Conditions. Figures 5.24 through 5.26 illustrate the effects of curing temperature and relative humidity on the deicer scaling resistance of concrete containing fly ash. Although no clear relationship could be established, the following observations can be made.

In general,

a. deicer scaling resistance of concrete containing no fly ash was similar or better than that of companion mixes containing fly ash; and

b. Damage due to deicer scaling resistance is highly dependent on the quality of the finishing and curing of the concrete.

Regardless of the curing temperature, when comparing the moist cured concrete test results, it is clear that plain concrete is the most resistant to scaling. This is in agreement with previous work done by Kleiger and Gebler.(22)

5.6.1.4 Effect of Strength. Figure 5.27 shows the effect of the 28-day compressive strength on the scaling resistance of concrete for all the mixes examined throughout the study. It is clear that

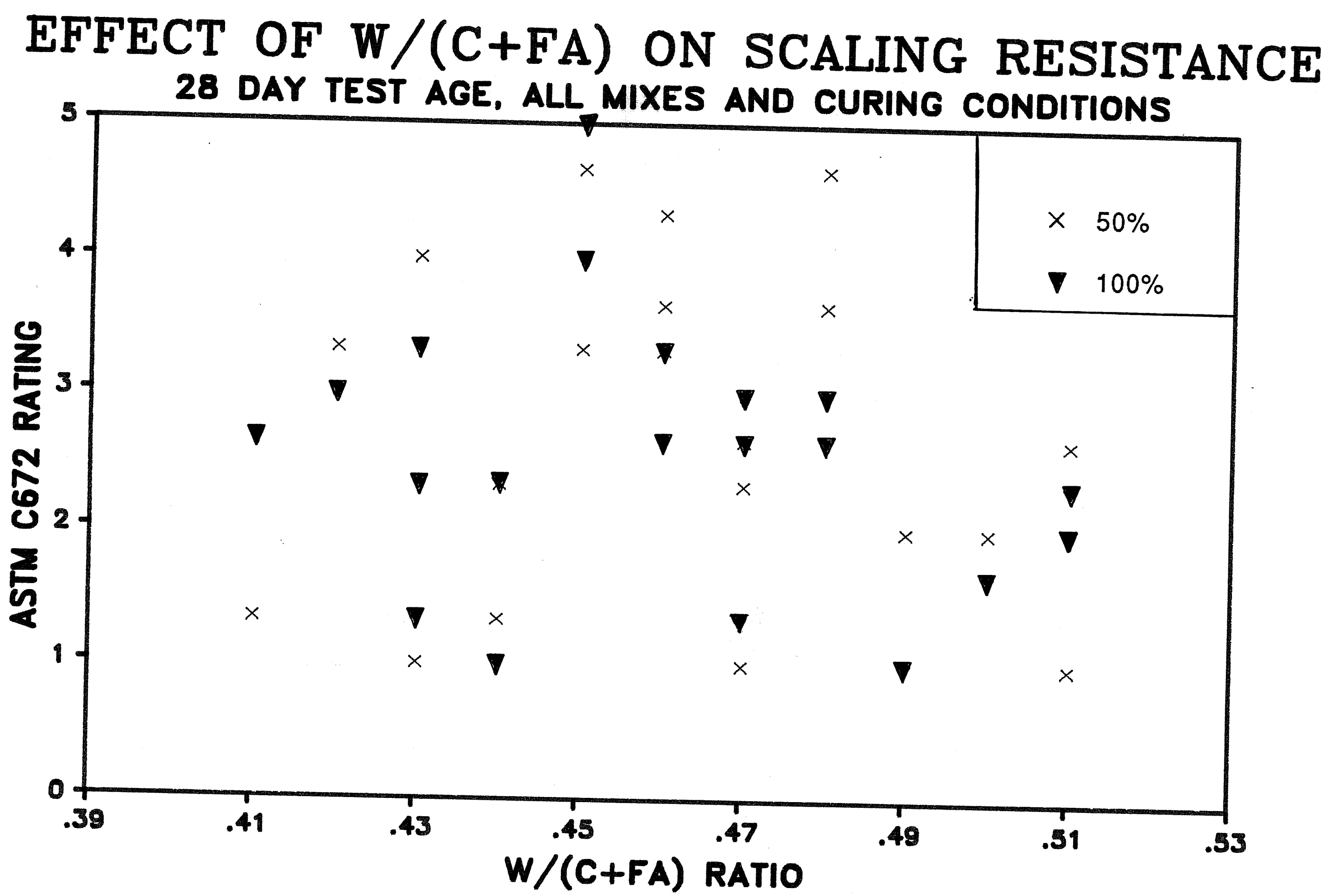

Figure 5.23 Scaling resistance was independent from the W/(C+FA).

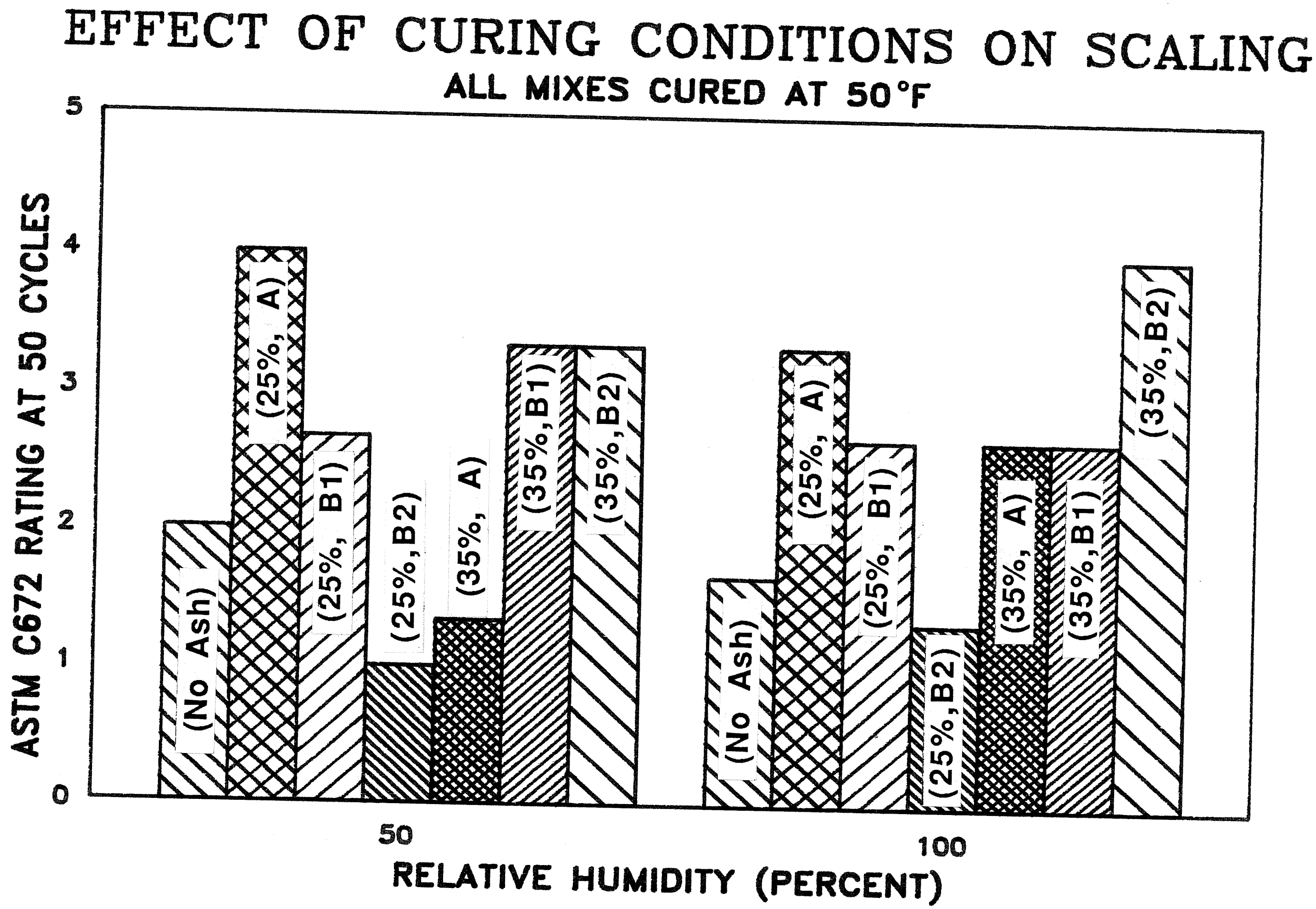

Figure 5.24 ASTM C672 rating after 50 cycles for the deicer scaling damage for the mixes cured at 50°F.

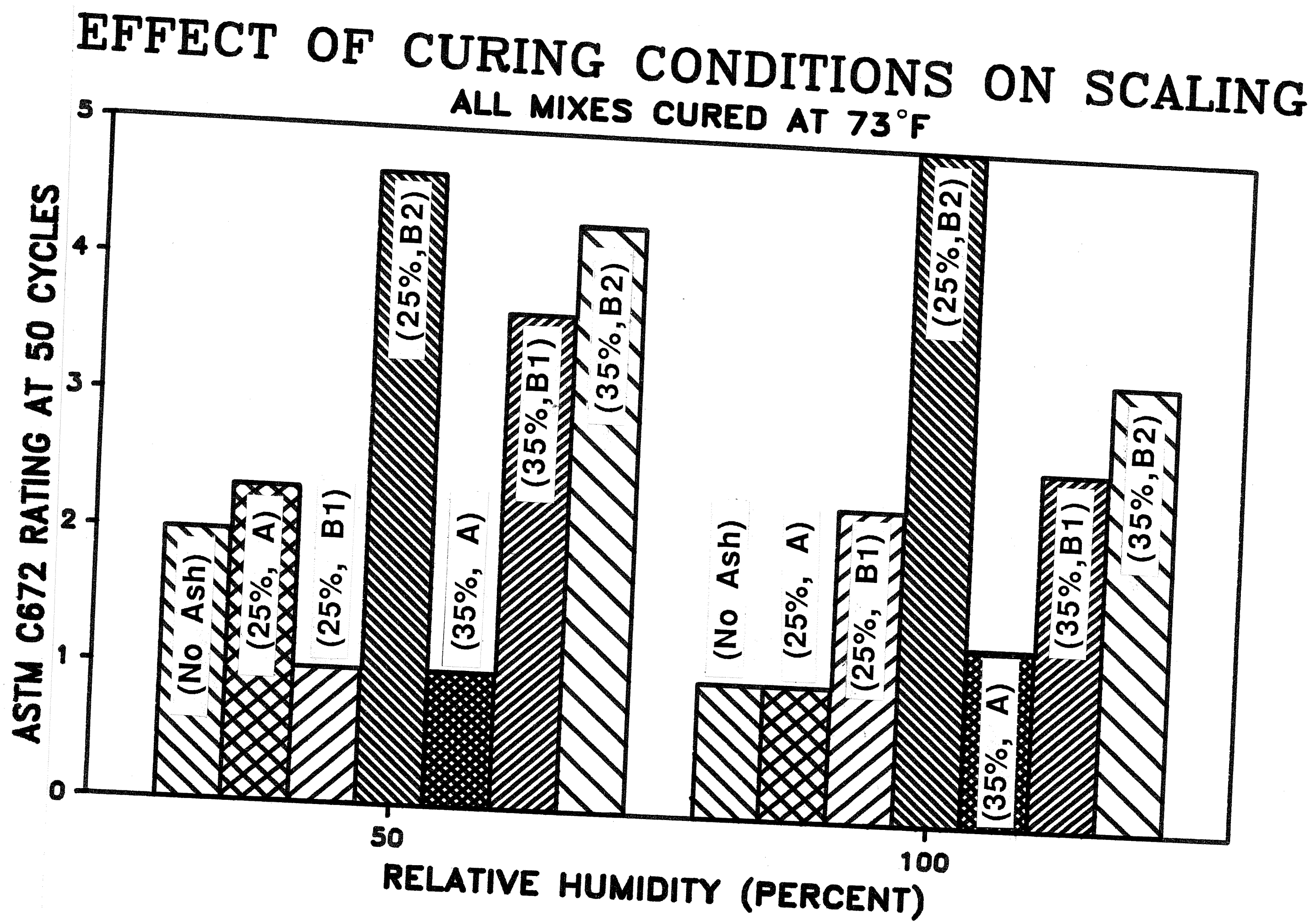

Figure 5.25 ASTM C672 rating after 50 cycles for the deicer scaling damage for the mixes cured at 73°F.

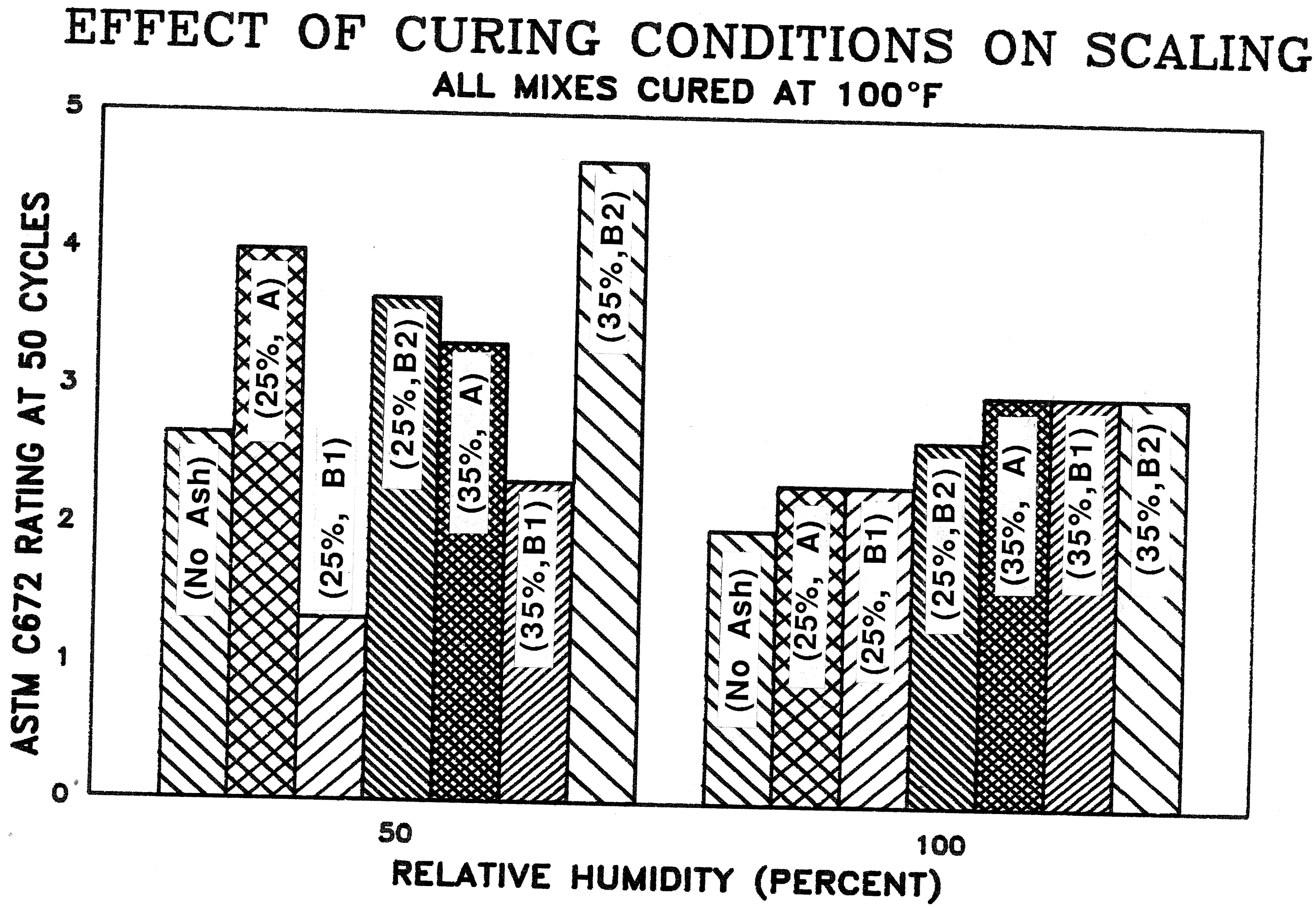

Figure 5.26 ASTM C672 rating after 50 cycles for the deicer scaling damage for the mixes cured at 100°F.

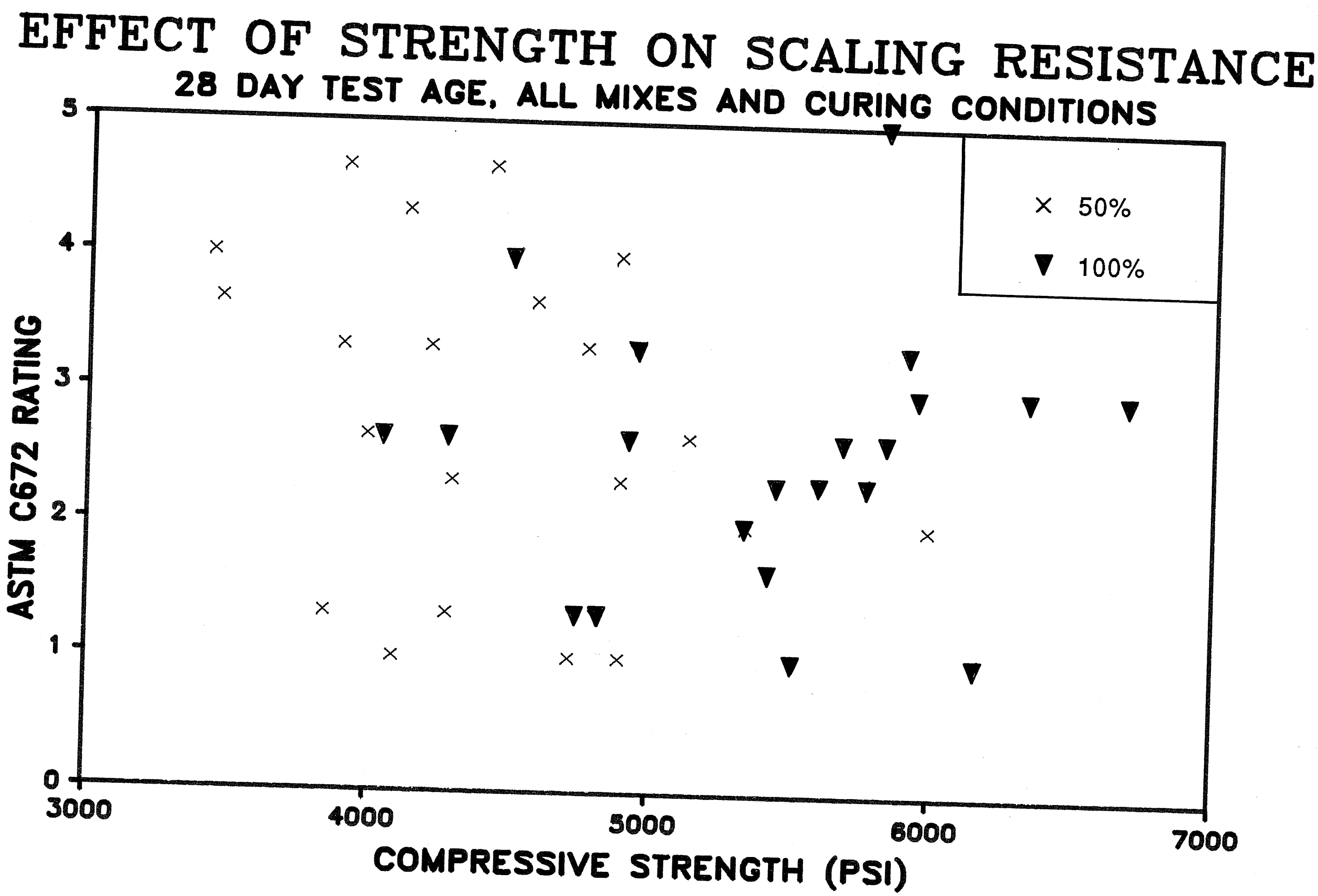

Figure 5.27 No relationship could be established between strength and scaling resistance.

no relationship could be established between the compressive strength and the scaling resistance of the concrete containing fly ash.

5.6.2 Chloride Penetration. The chloride content within the concrete was measured at depths of 0 to 1/4-in., 1/2 to 3/4-in. and 1 to 1-1/4-in. The discussion of the test results will focus only on the data obtained from the 1/2 to 3/4-in. and 1 to 1-1/4-in. depths. The data obtained for the 0 to 1/4-in. depth was too variable due to the effect of the amount of spalling that had occurred in the chloride content of the concrete at such a shallow depth.

The chloride penetration test data for the 1/2 to 3/4 and 1 to 1-1/4- in. depths is illustrated in Figs. 5.28 to 5.30. It can be seen from these figures that the concrete specimens cured at 50 percent relative humidity and 100°F exhibited higher chloride concentrations both at 3/4 and 1-1/4-in. depths than any other concrete specimens tested. This is mainly due to the fact that 100°F and 50 percent relative humidity represent the worst curing conditions and there is not enough moisture available for the complete hydration of the cement paste, thus leading to a more porous concrete. Figure 5.30 clearly shows that at 100°F, the chloride concentration of the concrete containing fly ash cured at 50 percent relative humidity and measured at 1-1/4-in. depth, is higher than that of the moist cured concrete when measured at 3/4-in. depth. Furthermore, at all temperatures plain concrete cured at 50 percent relative humidity had a considerably lower chloride concentration at 1-1/4-in. depth than similar concrete containing fly ash.

Regardless of the curing temperature, concrete cured at 50 percent relative humidity showed much higher chloride concentrations at both 3/4 and 1-1/4-in. depths than similar moist cured concrete. For all curing temperatures, the chloride concentration at 1-1/4-in. depth for the moist cured specimens was negligible at less than 0.02 percent.

The chloride concentration of moist cured concrete containing fly ash tended in general to decrease with an increase in curing temperature. At 100°F all the moist cured mixes containing fly ash out- performed the plain concrete mix. It is believed that this is the result of fly ash reducing the porosity of concrete by the formation of a secondary gel initiated by the pozzolanic activity of the fly ash.

In summary, the chloride penetration is heavily dependent on the curing conditions of the concrete. Although not evident from the visual inspection of the scaling test, moist cured concrete is by far more resistant to the chloride penetration than similar air-dried concrete. When fly ash is used in concrete mixes, moist curing for a longer period of time is crucial in order to reduce the amount of chloride penetration.

Overall, test results indicate that the lower the permeability of the concrete, the lower the chloride penetration. Without a doubt, temperature and relative humidity are the main variables affecting the permeability of concrete as a result of curing conditions.

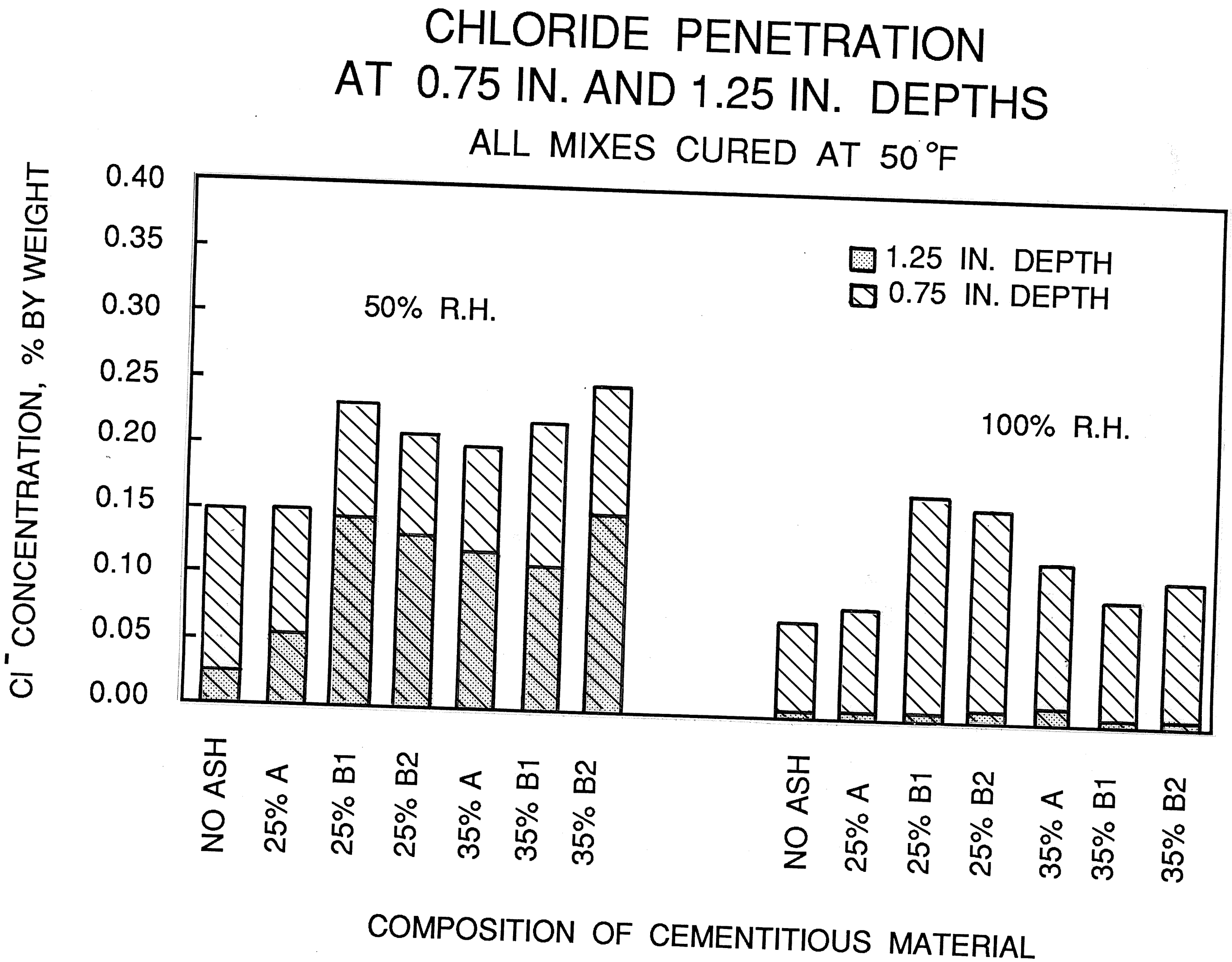

Figure 5.28 **Chloride penetration test results for the mixes cured at 50°F.**

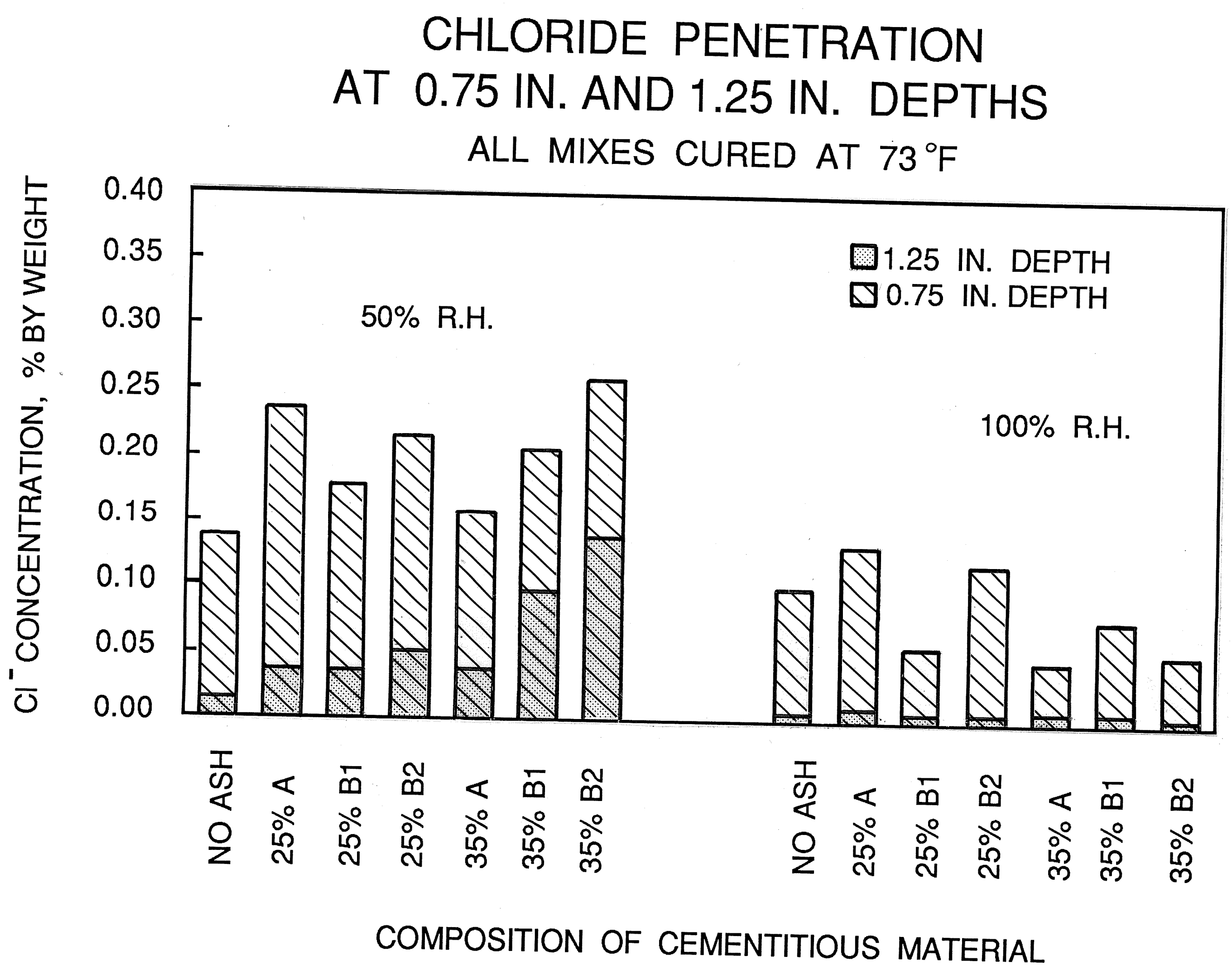

Figure 5.29 Chloride penetration test results for the mixes cured at 73°F.

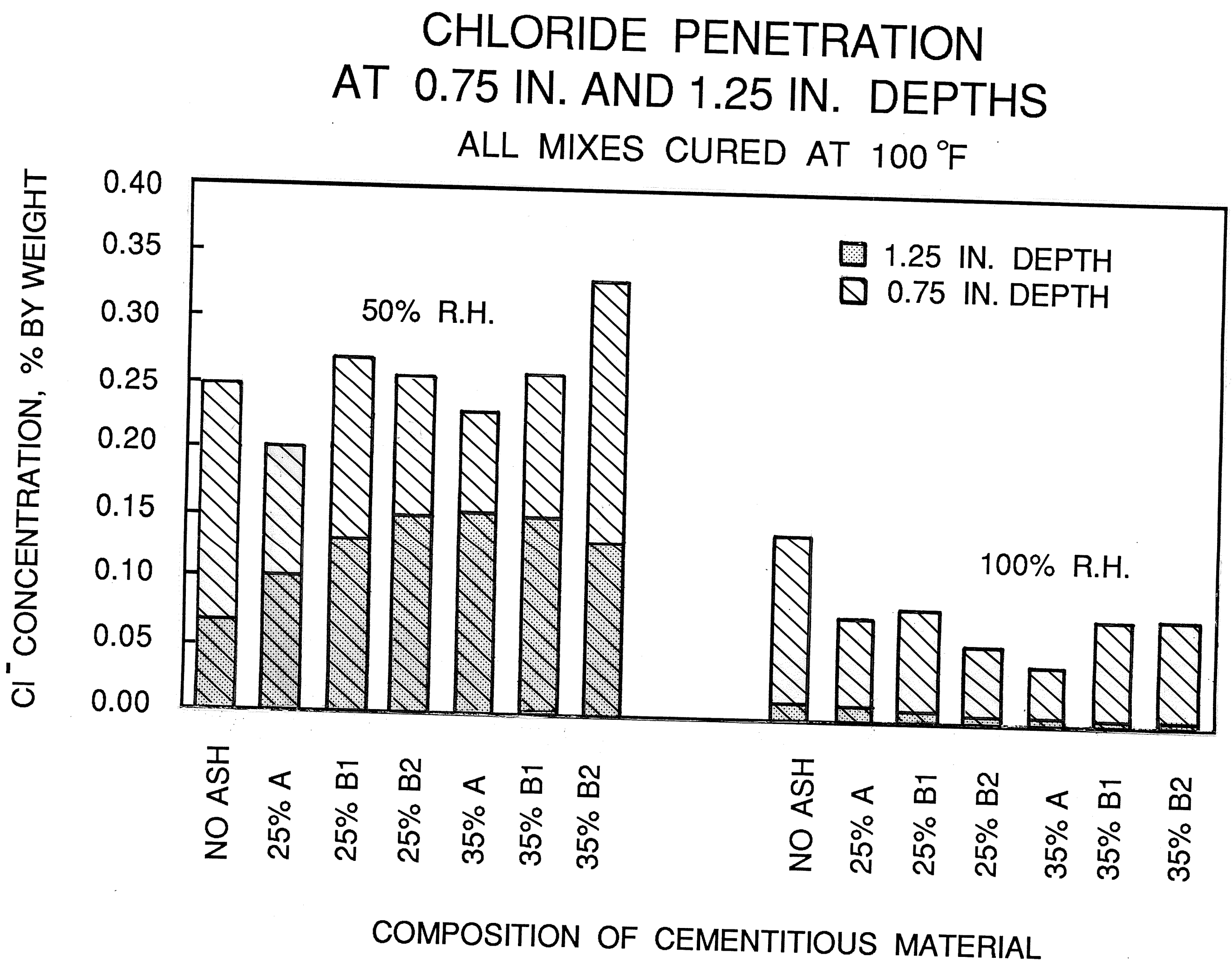

Figure 5.30 Chloride penetration test results for the mixes cured at 100°F.

CHAPTER 6
SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

6.1 Summary

The use of concrete containing fly ash in highway construction has been increasing steadily for the last 10 years and will continue to increase in the future. This is mainly due to the economic and technical advantages that concrete containing fly ash offers over conventional concrete. The abrasion resistance and resistance to deicer scaling of concrete containing fly ash are the scope of investigation in this study.

The results of the study indicate that the strength, the curing practices, the finishing procedure, and the aggregate properties all played a crucial role in the abrasion resistance of concrete containing fly ash. However, strength was found to be the factor most influencing the abrasion resistance of concrete. The general trend in the data indicates that abrasion resistance of concrete increases with an increase in the compressive strength of the concrete with or without fly ash.

From the scaling test results, no clear relationship could be established between the deicer scaling resistance of the concrete and the water-cementitious ratio, compressive strength, air content, or curing practices. Moist-cured plain concrete was found to be the most resistant to scaling. From the chloride penetration test results it was evident that the curing conditions are the most important factor affecting the chloride concentration in the concrete at various depths. Moist cured concrete was found to be much more resistant to chlorides than similar air dried concrete. The curing practices were especially important when concrete containing fly ash was used.

6.2 Conclusions

The primary objective of the study was to develop the necessary guidelines for the production of durable concrete. Although several of these guidelines may be taken for granted, their application becomes of primary importance when fly ash is used as a partial replacement to cement in the concrete mix. To obtain abrasion resistant concrete the engineer must be aware of the following:

1. The use of fly ash reduces the rate of early strength development for a given strength concrete. When compared to plain concrete, concrete containing fly ash may require a higher cementitious factor to be able to meet the same 7-day flexural strength requirement as specified by the Texas SDHPT Specification Item 366. The increase in the cementitious factor is more pronounced for Type A fly ash than for Type B fly ash.
2. The 14-day and 28-day compressive strength of concrete containing fly ash increases with an increase in curing temperature provided the specimens are moist cured.
3. Prolonged periods of moist curing for concrete containing fly ash are important for adequate strength development. Poorly cured fly ash concrete exhibited greater strength losses than conventional concrete especially at higher curing temperature.
4. Concrete strength is the governing factor affecting the abrasion resistance of concrete. In general an increase in the compressive strength was accompanied by an increase in abrasion resistance.

5. The finishing practices greatly influence the abrasion resistance of concrete by affecting the water cementitious ratio near the top of the slab. The finishing operation should be undertaken only after all the bleed water has evaporated and the concrete has stiffened sufficiently.

6. The curing temperature and relative humidity affect the abrasion resistance of concrete in that they influence its strength gain. For higher temperatures and low relative humidities, the loss of moisture from the concrete surface causes a deficiency in the hydration products thus resulting in a weak concrete surface. The structure of the surface matrix is therefore crucial for the production of abrasion resistant concrete.

7. The toughness of the aggregate influences the abrasion resistance of concrete. When concrete is made with soft aggregates, the compressive strength and the paste matrix are the primary parameters affecting abrasion. When tough hard aggregates are used, the strength of the aggregate rather than the compressive strength of the concrete becomes the main factor controlling the abrasion resistance of concrete.

8. Concrete containing fly ash is as resistant to abrasion as plain concrete provided the two have comparable compressive strengths.

9. No clear relationship could be established between the deicer scaling resistance of concrete an the water-cementitious ratio, the compressive strength, or the curing practices.

10. The porosity of the coarse aggregate was found to be a factor in determining the deicer scaling resistance of the concrete. Porous crushed limestone concrete specimens suffered more scaling than concrete specimens made with a low absorption capacity gravel.

11. Regardless of the curing temperature, moist-cured conventional concrete was found to be the most resistant to deicer scaling.

12. The chloride concentration of the concrete at various depths was found in all cases to be much higher for the specimens cured at 50 percent relative humidity than similar moist-cured specimens. The highest chloride concentrations were recorded for those specimens air cured at 50% RH and 100°F.

13. At 100°F., the moist-cured fly ash concrete specimens had lower chloride concentrations than plain concrete due to the contribution of pozzolanic reaction in reducing the permeability of the concrete.

14. For similar strength concrete, fly ash concrete requires moist curing for a longer period of time than plain concrete without fly ash in order to ensure adequate permeability of the concrete.

6.3 Recommendations

Based on the results of this research program, the abrasion resistance of concrete containing fly ash is best controlled by achieving proper strength development. This can be easily accomplished by proper mix design, proper finishing techniques, and prolonged periods of moist curing. Where adequate strengths can not be achieved, the use of hard, tough aggregates in the concrete mix design or near the surface of the slab will enhance the abrasion resistance of the concrete.

Although no clear relationship could be established between the scaling resistance of concrete and the water-cementitious ratio, the compressive strength or the curing condition, the test results indicate that moist-cured plain concrete is the most resistant to deicer scaling. The chloride concentration at various depths from the concrete surface is best controlled by controlling the porosity of the concrete. This is achieved by proper curing practices which include prolonged periods of moist curing when fly ash is used in the concrete mix. Keeping the chloride penetration to a minimum is crucial as the chlorides are one of the main factors contributing to the corrosion of the steel reinforcement in the concrete.

The results of this study have answered and clarified several questions about the durability of concrete containing fly ash. It is recommended however, that more research be conducted regarding the abrasion resistance of concrete when subjected to actual finishing and curing conditions in the field. In the case of scaling resistance of concrete containing fly ash, more research is needed in the area of mix design in order to try to enhance the performance of fly ash concrete subjected to freeze-thaw cycles in the presence of deicer salt solutions.

APPENDIX A
MATERIAL PROPERTIES AND MIX PROPORTIONS

Table A.1 Concrete Mix Proportionsper Cubic Yard of Concrete

Batch	Mix	Coarse Agg. (lb)	Fine Agg. (lb)	Cement (lb)	Fly Ash (lb)	Water (lb)	AEA (oz)	Reducer Retarder (oz)
1	50−50 100−50	1539	1486	470	??0	234	1.9	18.8
2	50−50A−25 100−50A−25	1542	1536	399	104	216	2.8	15.0
3	50−50B1−25 100−50B1−25	1582	1503	366	101	221	2.6	14.6
4	50−50B2−25 100−50B2−25	1591	1511	368	107	221	2.4	14.7
5	50−50A−35 100−50A−35	1563	1496	365	148	211	3.8	14.6
6	50−50B1−35 100−50B1−35	1619	1493	336	150	225	2.7	13.5
7	50−50B2−35 100−50B2−35	1598	1473	332	156	222	2.7	13.3
8	50−73 100−73	1556	1498	476	??0	232	1.8	19.0
9	50−73A−25 100−73A−25	1533	1526	396	103	220	2. 5	15.9
10	50−73B1−25 100−73B1−25	1575	1479	364	100	237	2.3	14.6
11	50−73B2−25 100−73B2−25	1594	1496	368	107	215	2.4	14.7
12	50−73A−35 100−73A−35	1552	1468	363	147	221	3.1	14.5
13	50−73B1−35 100−73B1−35	1602	1460	333	149	223	2.4	13.3
14	50−73B2−35 100−73B2−35	1604	1463	334	157	225	2.5	13.3
15	50−100 100−100	1544	1487	472	??0	242	1.8	18.9
16	50−100A−25 100−100A−25	1531	1524	396	103	213	2.5	15.8
17	50−100B1−25 100−100B1−25	1593	1496	368	107	207	2.4	14.7
18	50−100B2−25 100−100B2−25	1579	1482	365	106	228	2.4	14.6
19	50−100A−35 100−100A−35	1560	1476	365	147	213	3.1	14.6
20	50−100B1−35 100−100B1−35	1607	1465	334	149	227	2.4	13.4
21	50−100B2−35 100−100B2−35	1607	1465	334	157	235	2.5	13.4

Table A.2 Abrasion Test Results for the Mixes Cured at 50°F.

TIME (minutes)	SPEC. #1 mmx0.01	SPEC. #2 mmx0.01	SPEC. #3 mmx0.01	AV. DEPTH OF 3 SPEC
		MIX 50-50		
0	0	0	0	0
2	-113	-77	-105	-98
4	-186	-126	-151	-154
6	-235	-167	-199	-200
8	-280	-194	-237	-237
10	-325	-240	-271	-279
		MIX 100-50		
0	0	0	0	0
2	-124	-144	-122	-130
4	-180	-191	-186	-186
6	-240	-242	-245	-243
8	-276	-281	-292	-283
10	-304	-326	-354	-328
		MIX 50-50A-25		
0	0	0	0	0
2	-84	-100	-88	-91
4	-131	-130	-122	-127
6	-171	-157	-142	-156
8	-217	-179	-159	-185
10	-217	-196	-186	-218
		MIX 100-50A-25		
0	0	0	0	0
2	-109	-86	-126	-107
4	-142	-111	-147	-133
6	-181	-143	-169	-164
8	-208	-171	-188	-189
10	-236	-189	-213	-213

Table A.2 Continued

MIX 50-50B1-25

TIME (minutes)	SPEC. #1 mmx0.01	SPEC. #2 mmx0.01	SPEC. #3 mmx0.01	AV. DEPTH OF 3 SPEC
0	0	0	0	0
2	-196	-158	-183	-179
4	-272	-237	-267	-259
6	-339	-288	-350	-326
8	-398	-334	-417	-383
10	-470	-405	-467	-447

MIX 100-50B1-25

TIME (minutes)	SPEC. #1 mmx0.01	SPEC. #2 mmx0.01	SPEC. #3 mmx0.01	AV. DEPTH OF 3 SPEC
0	0	0	0	0
2	-242	-215	-225	-227
4	-306	-282	-276	-288
6	-349	-336	-327	-337
8	-408	-393	-368	-390
10	-444	-444	-406	-431

MIX 50-50B2-25

TIME (minutes)	SPEC. #1 mmx0.01	SPEC. #2 mmx0.01	SPEC. #3 mmx0.01	AV. DEPTH OF 3 SPEC
0	0	0	0	0
2	-130	-139	-120	-130
4	-188	-209	-182	-193
6	-243	-287	-237	-256
8	-276	-293	-279	-283
10	-318	-341	-318	-326

MIX 100-50B2-25

TIME (minutes)	SPEC. #1 mmx0.01	SPEC. #2 mmx0.01	SPEC. #3 mmx0.01	AV. DEPTH OF 3 SPEC
0	0	0	0	0
2	-126	-117	-151	-131
4	-178	-162	-211	-184
6	-206	-183	-236	-208
8	-250	-207	-264	-240
10	-287	-230	-288	-268

Table A.2 Continued

MIX 50-50A-35

TIME (minutes)	SPEC. #1 mmx0.01	SPEC. #2 mmx0.01	SPEC. #3 mmx0.01	AV. DEPTH OF 3 SPEC
0	0	0	0	0
2	-182	-202	-209	-198
4	-244	-291	-259	-264
6	-307	-354	-307	-322
8	-348	-438	-361	-382
10	-411	-541	-414	-456

MIX 100-50A-35

TIME (minutes)	SPEC. #1 mmx0.01	SPEC. #2 mmx0.01	SPEC. #3 mmx0.01	AV. DEPTH OF 3 SPEC
0	0	0	0	0
2	-206	-225	-203	-211
4	-262	-298	-269	-276
6	-294	-350	-306	-317
8	-341	-412	-392	-382
10	-376	-454	-460	-430

MIX 50-50B1-35

TIME (minutes)	SPEC. #1 mmx0.01	SPEC. #2 mmx0.01	SPEC. #3 mmx0.01	AV. DEPTH OF 3 SPEC
0	0	0	0	0
2	-156	-186	-171	-171
4	-210	-243	-215	-223
6	-253	-301	-248	-268
8	-292	-344	-280	-305
10	-330	-359	-298	-329

MIX 100-50B1-35

TIME (minutes)	SPEC. #1 mmx0.01	SPEC. #2 mmx0.01	SPEC. #3 mmx0.01	AV. DEPTH OF 3 SPEC
0	0	0	0	0
2	-172	-184	-173	-176
4	-221	-242	-193	-218
6	-253	-283	-251	-262
8	-289	-337	-281	-302
10	-311	-359	-296	-322

Table A.2 Continued

TIME (minutes)	SPEC. #1 mmx0.01	MIX 50-50B2-35 SPEC. #2 mmx0.01	SPEC. #3 mmx0.01	AV. DEPTH OF 3 SPEC
0	0	0	0	0
2	-214	-155	-142	-171
4	-268	-238	-238	-248
6	-325	-297	-276	-299
8	-362	-374	-332	-356
10	-403	-464	-360	-409

TIME (minutes)	SPEC. #1 mmx0.01	MIX 100-50B2-35 SPEC. #2 mmx0.01	SPEC. #3 mmx0.01	AV. DEPTH OF 3 SPEC
0	0	0	0	0
2	-200	-151	-156	-169
4	-283	-188	-193	-221
6	-346	-243	-222	-270
8	-400	-265	-236	-300
10	-466	-296	-260	-340

Table A.3 Abrasion Test Results for the Mixes Cured at 73°F.

TIME (minutes)	SPEC. #1 mmx0.01	SPEC. #2 mmx0.01	SPEC. #3 mmx0.01	AV. DEPTH OF 3 SPEC
		MIX 50-73		
0	0	0	0	0
2	-32	-31	-40	-34
4	-50	-65	-66	-60
6	-66	-91	-83	-80
8	-92	-110	-121	-108
10	-114	-122	-137	-124

TIME (minutes)	SPEC. #1 mmx0.01	SPEC. #2 mmx0.01	SPEC. #3 mmx0.01	AV. DEPTH OF 3 SPEC
		MIX 100-73		
0	0	0	0	0
2	-64	-47	-45	-52
4	-93	-73	-76	-80
6	-110	-90	-92	-97
8	-123	-108	-114	-115
10	-133	-120	-136	-129

TIME (minutes)	SPEC. #1 mmx0.01	SPEC. #2 mmx0.01	SPEC. #3 mmx0.01	AV. DEPTH OF 3 SPEC
		MIX 50-73A-25		
0	0	0	0	0
2	-134	-119	-124	-125
4	-181	-162	-180	-174
6	-208	-208	-226	-214
8	-236	-247	-265	-250
10	-264	-294	-304	-287

TIME (minutes)	SPEC. #1 mmx0.01	SPEC. #2 mmx0.01	SPEC. #3 mmx0.01	AV. DEPTH OF 3 SPEC
		MIX 100-73A-25		
0	0	0	0	0
2	-114	-119	-131	-121
4	-158	-172	-180	-170
6	-202	-209	-237	-216
8	-229	-251	-280	-253
10	-264	-272	-299	-278

Table A.3 Continued

MIX 50-73B1-25

TIME (minutes)	SPEC. #1 mmx0.01	SPEC. #2 mmx0.01	SPEC. #3 mmx0.01	AV. DEPTH OF 3 SPEC
0	0	0	0	0
2	-95	-80	-104	-93
4	-131	-148	-138	-139
6	-164	-177	-168	-169
8	-182	-216	-209	-202
10	-225	-242	-253	-240

MIX 100-73B1-25

TIME (minutes)	SPEC. #1 mmx0.01	SPEC. #2 mmx0.01	SPEC. #3 mmx0.01	AV. DEPTH OF 3 SPEC
0	0	0	0	0
2	-77	-88	-83	-83
4	-110	-126	-145	-127
6	-139	-150	-169	-152
8	-159	-176	-191	-175
10	-182	-208	-213	-201

MIX 50-73B2-25

TIME (minutes)	SPEC. #1 mmx0.01	SPEC. #2 mmx0.01	SPEC. #3 mmx0.01	AV. DEPTH OF 3 SPEC
0	0	0	0	0
2	-112	-113	-142	-122
4	-175	-144	-220	-180
6	-197	-168	-278	-214
8	-246	-205	-320	-257
10	-277	-252	-365	-298

MIX 100-73B2-25

TIME (minutes)	SPEC. #1 mmx0.01	SPEC. #2 mmx0.01	SPEC. #3 mmx0.01	AV. DEPTH OF 3 SPEC
0	0	0	0	0
2	-39	-96	-66	-67
4	-92	-111	-93	-99
6	-153	-143	-114	-137
8	-182	-163	-139	-161
10	-210	-182	-158	-183

Table A.3 Continued

MIX 50-73A-35

TIME (minutes)	SPEC. #1 mmx0.01	SPEC. #2 mmx0.01	SPEC. #3 mmx0.01	AV. DEPTH OF 3 SPEC
0	0	0	0	0
2	-151	-124	-163	-146
4	-229	-210	-207	-215
6	-270	-301	-259	-276
8	-318	-362	-295	-325
10	-361	-402	-328	-363

MIX 100-73A-35

TIME (minutes)	SPEC. #1 mmx0.01	SPEC. #2 mmx0.01	SPEC. #3 mmx0.01	AV. DEPTH OF 3 SPEC
0	0	0	0	0
2	-130	-159	-163	-151
4	-190	-212	-208	-203
6	-230	-261	-252	-248
8	-262	-306	-285	-284
10	-289	-332	-321	-314

MIX 50-73B1-35

TIME (minutes)	SPEC. #1 mmx0.01	SPEC. #2 mmx0.01	SPEC. #3 mmx0.01	AV. DEPTH OF 3 SPEC
0	0	0	0	0
2	-151	-195	-147	-164
4	-203	-258	-205	-222
6	-263	-322	-257	-280
8	-313	-378	-304	-332
10	-351	-435	-336	-374

MIX 100-73B1-35

TIME (minutes)	SPEC. #1 mmx0.01	SPEC. #2 mmx0.01	SPEC. #3 mmx0.01	AV. DEPTH OF 3 SPEC
0	0	0	0	0
2	-129	-82	-104	-105
4	-177	-112	-158	-149
6	-234	-144	-190	-189
8	-283	-171	-217	-224
10	-327	-184	-244	-252

Table A.3 Continued

MIX 50-73B2-35

TIME (minutes)	SPEC. #1 mmx0.01	SPEC. #2 mmx0.01	SPEC. #3 mmx0.01	AV. DEPTH OF 3 SPEC
0	0	0	0	0
2	-141	-201	-197	-180
4	-199	-252	-281	-244
6	-239	-294	-339	-291
8	-281	-332	-377	-330
10	-315	-386	-427	-376

MIX 100-73B2-35

TIME (minutes)	SPEC. #1 mmx0.01	SPEC. #2 mmx0.01	SPEC. #3 mmx0.01	AV. DEPTH OF 3 SPEC
0	0	0	0	0
2	-85	-62	-40	-62
4	-112	-93	-82	-96
6	-141	-122	-97	-120
8	-161	-144	-122	-142
10	-179	-162	-149	-162

Table A.4 Abrasion Test Results for the Mixes Cured at 100°F.

TIME (minutes)	SPEC. #1 mmx0.01	SPEC. #2 mmx0.01	SPEC. #3 mmx0.01	AV. DEPTH OF 3 SPEC
		MIX 50-100		
0	0	0	0	0
2	-98	-94	-120	-104
4	-138	-137	-162	-146
6	-166	-154	-188	-169
7	-205	-187	-217	-203
10	-226	-199	-249	-225

TIME (minutes)	SPEC. #1 mmx0.01	SPEC. #2 mmx0.01	SPEC. #3 mmx0.01	AV. DEPTH OF 3 SPEC
		MIX 100-100		
0	0	0	0	0
2	-90	-62	-76	-76
4	-118	-88	-113	-106
6	-139	-113	-152	-135
8	-158	-129	-179	-155
10	-185	-154	-192	-177

TIME (minutes)	SPEC. #1 mmx0.01	SPEC. #2 mmx0.01	SPEC. #3 mmx0.01	AV. DEPTH OF 3 SPEC
		MIX 50-10		
0	0	0	0	0
2	-137	-147	-151	-145
4	-205	-207	-215	-209
6	-272	-234	-246	-251
8	-330	-271	-296	-299
10	-351	-300	-312	-321

TIME (minutes)	SPEC. #1 mmx0.01	SPEC. #2 mmx0.01	SPEC. #3 mmx0.01	AV. DEPTH OF 3 SPEC
		MIX 100-100A-25		
0	0	0	0	0
2	-33	-39	-45	-39
4	-63	-58	-65	-62
6	-76	-65	-84	-75
8	-88	-77	-100	-88
10	-98	-91	-113	-100

Table A.4 Continued

MIX 50-100B1-25

TIME (minutes)	SPEC. #1 mmx0.01	SPEC. #2 mmx0.01	SPEC. #3 mmx0.01	AV. DEPTH OF 3 SPEC
0	0	0	0	0
2	-128	-119	-147	-131
4	-167	-159	-185	-170
6	-201	-196	-219	-205
8	-221	-230	-254	-235
10	-249	-265	-286	-266

MIX 100-100B1-25

TIME (minutes)	SPEC. #1 mmx0.01	SPEC. #2 mmx0.01	SPEC. #3 mmx0.01	AV. DEPTH OF 3 SPEC
0	0	0	0	0
2	-112	-111	-94	-105
4	-143	-152	-115	-137
6	-179	-197	-133	-170
8	-203	-224	-154	-194
10	-230	-249	-172	-217

MIX 50-100B2-25

TIME (minutes)	SPEC. #1 mmx0.01	SPEC. #2 mmx0.01	SPEC. #3 mmx0.01	AV. DEPTH OF 3 SPEC
0	0	0	0	0
2	-84	-104	-90	-93
4	-132	-167	-134	-144
6	-173	-207	-168	-183
8	-222	-251	-205	-226
10	-266	-278	-226	-257

MIX 100-100B2-25

TIME (minutes)	SPEC. #1 mmx0.01	SPEC. #2 mmx0.01	SPEC. #3 mmx0.01	AV. DEPTH OF 3 SPEC
0	0	0	0	0
2	-29	-20	-42	-30
4	-50	-35	-59	-48
6	-58	-49	-73	-60
8	-71	-63	-87	-74
10	-81	-75	-96	-84

Table A.4 Continued

MIX 50-100A-35

TIME (minutes)	SPEC. #1 mmx0.01	SPEC. #2 mmx0.01	SPEC. #3 mmx0.01	AV. DEPTH OF 3 SPEC
0	0	0	0	0
2	-162	-122	-127	-137
4	-206	-180	-171	-186
6	-241	-226	-211	-226
8	-271	-263	-252	-262
10	-327	-296	-292	-305

MIX 100-100A-35

TIME (minutes)	SPEC. #1 mmx0.01	SPEC. #2 mmx0.01	SPEC. #3 mmx0.01	AV. DEPTH OF 3 SPEC
0	0	0	0	0
2	-131	-131	-108	-123
4	-188	-180	-137	-168
6	-247	-218	-170	-212
8	-283	-247	-197	-242
10	-312	-273	-232	-272

MIX 50-100B1-35

TIME (minutes)	SPEC. #1 mmx0.01	SPEC. #2 mmx0.01	SPEC. #3 mmx0.01	AV. DEPTH OF 3 SPEC
0	0	0	0	0
2	-156	-151	-199	-169
4	-243	-203	-252	-233
6	-300	-252	-317	-289
8	-374	-287	-359	-340
10	-437	-322	-413	-391

MIX 100-100B1-35

TIME (minutes)	SPEC. #1 mmx0.01	SPEC. #2 mmx0.01	SPEC. #3 mmx0.01	AV. DEPTH OF 3 SPEC
0	0	0	0	0
2	-90	-118	-113	-107
4	-111	-158	-168	-146
6	-157	-188	-207	-184
8	-200	-213	-237	-217
10	-235	-232	-255	-241

Table A.4 Continued

MIX 50-100B2-35

TIME (minutes)	SPEC. #1 mmx0.01	SPEC. #2 mmx0.01	SPEC. #3 mmx0.01	AV. DEPTH OF 3 SPEC
0	0	0	0	0
2	-167	-170	-164	-167
4	-253	-224	-244	-240
6	-298	-264	-304	-289
8	-352	-306	-352	-336
10	-390	-338	-399	-376

MIX 100-100B2-35

TIME (minutes)	SPEC. #1 mmx0.01	SPEC. #2 mmx0.01	SPEC. #3 mmx0.01	AV. DEPTH OF 3 SPEC
0	0	0	0	0
2	-29	-33	-8	-23
4	-44	-46	-49	-46
6	-64	-77	-71	-71
8	-75	-90	-88	-84
10	-87	-100	-101	-96

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