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16. Abstract

The application of polymer-impregnated concrete (PIC) for improving the durability of concrete slabs, especially highway bridge decks, has been widely investigated. For this application, the slab is partially impregnated from the upper surface. Parameters such as drying time and temperature, soaking time, and curing time and temperature, which, of course, can influence the durability of concrete bridge decks after impregnation, were thoroughly investigated.

The monomer system of methyl methacrylate (MMA) with 1% (by weight) benzoyl peroxide (BP) and 10% (by weight) trimethylolpropane trimethacrylate (TMPTMA) was generally used throughout the tests. A drying temperature of more than 212° F to 300° F was acceptable for the slabs tested.

To obtain at least a 1-in. polymer depth, a soaking time of 4 hours or more was necessary. Steam was found to be an economical, safe and workable curing source for field treatment. With adequate steam, the monomer in concrete can be cured within 30 minutes. A curing temperature of at least 140° F is necessary to achieve adequate polymer depth. It is recommended that the minimum curing time and temperature of 60 minutes and 150° F, respectively, should be used for actual field impregnations.

The durability of partially-impregnated slabs was evaluated by freeze-thaw tests conducted on 10-in. × 10-in. × 6-in. concrete slabs. It was found that the durability performance of concrete was significantly increased by PIC impregnation.

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DURABILITY, STRENGTH, AND METHOD OF APPLICATION OF POLYMER-IMPREGNATED CONCRETE FOR SLABS

by

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Donald R. Paul

Research Report Number 114-4

Polymer-Impregnated Concrete for Highway Applications
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Preface

This report summarizes much of the research that has been performed in this study to identify the procedures to achieve partial impregnation of concrete bridge decks. Results of extensive durability tests are also presented.

The authors are extremely gratified by the support and encouragement of personnel of the State Department of Highways and Public Transportation. The progress that has been made that has resulted in implementation would not have been achieved without the help of John Nixon, Donald O'Connor, Maurice Ferrari, Andy Seely, Clarence Rea and others. The advice and encouragement of Jerry Bowman, John Nichols, and Ed Kristaponis of the Federal Highway Administration are sincerely appreciated.

The authors also want to acknowledge the excellent help of Dr. James T. Houston, formerly a study supervisor of this research study. Dr. Houston developed and refined many of the test methods used in this study and was responsible for making many of the specimens.

And last but not least, thanks are due to the many research assistants who have worked so diligently and untiringly to make this research effort successful. A partial list includes Ek Limsuwan, Ron Webster, Prajya Phinyawat, M. Jaber, Andy Knysh, Art Meyer, and John Wyman.

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ABSTRACT

The application of polymer-impregnated concrete (PIC) for improving the durability of concrete slabs, especially highway bridge decks, has been widely investigated. For this application, the slab is partially impregnated from the upper surface. Parameters such as drying time and temperature, soaking time, and curing time and temperature, which, of course, can influence the durability of concrete bridge decks after impregnation, were thoroughly investigated.

The monomer system of methyl methacrylate (MMA) with 1% (by weight) benzoyl peroxide (BP) and 10% (by weight) trimethylolpropane trimethacrylate (TMPTMA) was generally used throughout the tests. A drying temperature of more than 212°F to 300°F was acceptable for the slabs tested.

To obtain at least a 1-in. polymer depth, a soaking time of 4 hours or more was necessary. Steam was found to be an economical, safe and workable curing source for field treatment. With adequate steam, the monomer in concrete can be cured within 30 minutes. A curing temperature of at least 140°F is necessary to achieve adequate polymer depth. It is recommended that the minimum curing time and temperature of 60 minutes and 150°F, respectively, should be used for actual field impregnations.

The durability of partially-impregnated slabs was evaluated by freeze-thaw tests. The freeze-thaw tests were conducted on 10-in. x 10-in. x 6-in. concrete slabs. The freeze-thaw tests were conducted by placing the specimens, which contained approximately 0.25 in. ponded water on the surface, in a 24-hour cycle with the temperature ranging from -20°F to room temperature ($\approx 75°F$). It was found that the durability performance of concrete was significantly increased by PIC partial impregnation.

Actual field impregnations on highway bridge decks were conducted. Polymer depths of approximately 0.75 to 1.0 in. were achieved.

Tests were conducted to determine the effects of monomer system, polymer loading, and concrete properties on thermal expansion of PIC. It was found that after the specimens were impregnated with MMA monomer system, the change in length of specimens increased as the polymer loading increased. It was also found that the coefficient of thermal expansion increased from 4.34 microin./in. to 4.93 microin./in. of PIC.

Preliminary research was conducted to determine the structural properties of PIC. Tests were performed on various types of monomer systems and the MMA monomer system was the most favorable. The stress-strain relationships within the elastic range of PIC were investigated. It was found that the initial quality of concrete has no significant effect on the strength of the impregnated concrete.

Limited tests were also conducted to determine the behavior of PIC beams. Two rectangular beams were impregnated with MMA. It was found that the flexural stiffness of the PIC beams was also increased. The results of an analysis using the linear stress-strain relationship agree well with the test results for the PIC beam which failed in compression. However, when conventional theory was used, the analysis resulted in a rather conservative prediction for the ultimate load for a beam with a diagonal-tension failure.

KEY WORDS: polymer-impregnated, concrete, slabs, bridge decks, durability strength, method, freeze-thaw tests

SUMMARY

The partial impregnation of concrete bridges requires several steps: drying the concrete, cooling, soaking a monomer solution into the concrete, and applying heat to cure the monomer.

In this study the primary monomer system consisted of methyl methacrylate, 1% (wt) of initiator and 10% (wt) cross-linking agent. A wide range of drying times and temperatures were investigated. It was found that a surface drying temperature of 250°F to 300°F was acceptable. Data are presented on the effect of drying time, drying temperature, and moisture on the resulting polymer depth.

The effect of soaking time was found to be significant with a minimum time of 4 hrs. necessary to obtain one inch or more of polymer depth. Steam was used as a heat source for curing. A curing temperature of 150°F maintained for one hour was found to be satisfactory.

Durability of partially-impregnated slabs were evaluated by freeze-thaw tests. Water was ponded on the surface to a depth of 0.25 in. It was found that the durability performance of concrete slabs was significantly improved by PIC partial impregnation.

Actual field impregnation tests were conducted. Polymer depths of up to one inch were obtained.

The expansion of PIC was measured as a function of monomer system, polymer loading, and concrete properties. Coefficients of thermal expansion were obtained.

Structural properties of PIC were determined. Limited tests were also conducted to determine the behavior of PIC beams. It was found that both flexural strength and stiffness increased for the PIC beams as compared to control beams.

Implementation Statement

The results of this investigation establish the requirements for partial polymer-impregnation of concrete slabs. The durability is shown to be significantly improved. Field tests indicate that polymer-impregnated depths of up to one-inch can be obtained. The polymer-impregnation process has the potential of providing durable concrete bridge deck surfaces.

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CHAPTER 1

INTRODUCTION

Concrete has served as an excellent construction material for many years. However, conventional concrete has the disadvantage of being susceptible to the ingress of water. Presence of water inside the concrete frequently causes problems in concrete structures. The problems, such as freeze-thaw, corrosion of reinforcing bars are considered to be serious problems because they are progressive and finally lead to the complete failure of the structure.

Deterioration of concrete highway bridge decks involves a number and interaction of variables, such as loading pattern, material characteristics, and environmental conditions. There have been extensive investigations into the nature of deterioration of concrete bridge surfaces (Ref. 48). However, the most common types of deterioration are cracking, scaling, and spalling (Ref. 5). Freeze-thaw and corrosion of reinforcing bars seem to be the most significant causes of deterioration of concrete highway bridge decks. Studies have shown that concrete is susceptible to deterioration due to corrosion and freezing and thawing action when moisture is present (Ref.11).

In recent years, the use of polymer in concrete has been investigated to improve the strength and durability performance of concrete. In general, concrete polymer materials can be divided into three types as follows:

1) polymer-impregnated concrete (PIC),
2) polymer-concrete (PC),
3) polymer-cement concrete (PCC).

Polymer-impregnated concrete (PIC) is prepared by impregnating the dried conventional concrete with a liquid monomer system. Usually entrapped air is removed from the concrete prior to monomer application. After the pores of the concrete are filled with the monomer, polymerization of the monomer is completed by means of radiation or the thermal catalytic process.

Polymer-concrete (PC) consists of aggregate and monomer mixed together, and polymerization of the monomer is completed after placement of the mixture of monomer and concrete.

Polymer-cement concrete (PCC) is a mixture of cement, water, aggregate, and monomer. These materials, included the monomer, are mixed together and polymerization is completed after placement.

At the present time, the greatest improvements in concrete properties have been obtained with PIC. The experiments and evaluations in this report were conducted on PIC specimens.

Extensive research of PIC has been conducted by Brookhaven National Laboratory in conjunction with the U.S. Bureau of Reclamation (Ref. 16-20). The first sample of PIC was produced at Brookhaven National Laboratory in 1965 and the first measurements of physical and durability properties were conducted in 1966 by the U.S. Bureau of

Reclamation (Ref. 47). Many monomer systems have been investigated. However, methyl methacrylate (MMA) impregnated concretes have given the best results (Ref. 20). In general, the PIC has improved the following concrete properties:

1) compressive strength,
2) tensile strength,
3) modulus of elasticity,
4) water permeability,
5) water absorption,
6) resistance to abrasion, and
7) durability.

The MMA-impregnated concrete showed the most improvment in durability (Ref. 16). The freeze-thaw durability was increased by more than 360 %. The water permeability and chemical resistance were reduced to negligible values. The water absorption decreased by as much as 95 %.

The compressive strength of high pressure steam-cured concrete impregnated with MMA in glass forms and radiation-polymerized has been found to be as high as 27,000 psi. in the U.S. (Ref. 20). It has been reported that researchers in Italy have produced PIC with a compressive strength of 38,000 psi (Ref. 53). The tensile strength and modulus of elasticity have been found to increase by 292 % and 80 %, respectively (Ref. 16).

The increase in durability of PIC results because the polymer in the concrete pores significantly reduces the intrusion of water. However, the reason for increased strength is not completely understood. The possible explanations are (Ref. 5):

1) The polymer acts as a random reinforcement.
2) The polymer increases the bond between the cement paste and the aggregate.
3) The polymer repairs microcracks in concrete.
4) The polymer penetrates and reinforces the micropores in concrete.

The increase in tensile strength is related to the increase in compressive strength because the failures of concrete specimens under compressive load are essentially shear failures on oblique planes (Ref. 8). Therefore, the reasons for the increase in tensile strength are the same as the reasons for increased compressive strength.

The increase in modulus of elasticity is more difficult to understand. In general, plain concrete has a modulus (E) of 3,000,000 to 4,000,000 psi. Cross-linked polymer has a modulus which ranges from 500,000 to 750,000 psi. However, when they are combined together to produce PIC, E is increased by approximately 80 %. The reason for the increased E is not well understood as yet.

It was found (Ref. 18) that the improvement in strength is a function of polymer loading and type of monomer system. Polymer loading is defined as percent of polymer in concrete by weight.

The radiation polymerization gives slightly better strength properties than thermal catalytic polymerization (Ref. 17). Improvements in durability seem to be a function of polymer loading and the degree of success in sealing the surface of concrete (Ref. 20).

The MMA-impregnated concrete develops significantly less creep than ordinary concrete and in some cases develops no creep or even negative creep (Ref. 17 and 18). Tests were performed at the Bureau of Reclamation, Denver, Colorado, on PIC using a sustained compressive load of 800 psi for periods of 275 to 799 days (Ref. 18). Negative creep, which was found in Ref. 17, continued to exist in most MMA specimens. The average creep of thermal-catalytically polymerized MMA specimens consistently shows negative creep, averaging 62 millionths in./in. strain.

Scope and Objectives

The primary objective of this report is to investigate, develop, and evaluate the application of PIC to concrete surfaces, especially on highway bridge decks. Research was performed to develop field techniques for the polymer-impregnation of the surfaces of highway bridge decks and to evaluate the effectiveness of the protection provided by the surface impregnation. Investigations in this report could be divided into two catagories as follow:

1) PIC partial impregnation, and
2) fully-impregnated concrete.

The first part of the study involved the evaluations of the parameters which might affect the applications of PIC for partial impregnation of concrete bridge decks These parameters are

1) monomer system,
2) drying time,
3) drying temperature,
4) soaking time,
5) curing temperature,
6) curing time.

These parameters were studied and evaluated to determine the most effective method of using PIC for highway applications. The improvement in durability of concrete provided by PIC surface treatment was determined by two types of tests which were considered as a severe test conditions:

1) freeze-thaw
2) salt-water exposure

Actual field tests were conducted to impregnate concrete partially with polymer. Tests were conducted on both small and large scale field treatments.

Experiments on fully-impregnated PIC were conducted to determine the strength and thermal properties of PIC. The compressive strength, tensile strength, and modulii of elasticity of various types of PIC were determined and compared to previous studies. The volume changes of concrete during the full-impregnation treatments and the coefficients of thermal expansion of many types of PIC were determined.

Chapter 2 summarizes and reviews the monomer systems being used in polymer-impregnated concrete. The monomer system which was most often used in this research was determined.

Chapter 3 presents the results of experimental studies on drying time of concrete for surface treatments. The relationship between drying time, temperature, and polymer depth were determined. Experiments were conducted in laboratory and field conditions.

Chapter 4 presents the test results conducted on optimization of soaking time. The relationship between soaking time and polymer depth was determined.

Chapter 5 presents the experimental results of polymerization of monomer in concrete. The relationship between curing time, temperature, and polymer depth was obtained.

Chapter 6 involves the determination of the durability properties of many types of PIC. The concrete slabs were treated under field and laboratory conditions. Corrosion protection provided by field partially polymer impregnated slabs under salt-water exposure was determined. Corrosion protection provided by full-impregnated polymer concrete piles under sea water was also evaluated.

Chapter 7 presents the verification of previous research by actual field application on highway bridge decks.

Chapter 8 presents the determination of the volume change during the full-impregnation treatment of concrete. The coefficients of thermal expansion of PIC as well as of the controls were determined.

Chapter 9 discusses the compressive strength, tensile strength, and modulus of elasticity of various types of PIC. The effect of water-cement ratio on structural properties of PIC is also evaluated. A limited number of beams were also tested to determine the behavior of PIC beams under flexural loading.

Chapter 10 presents the conclusions and future research needs in this area.

It should be noted that only Colorado River aggregate was used for the tests in this dissertation. Other aggregates, such as crushed stone, may result in more improvement in properties because they are more susceptible to monomer penetration than Colorado River aggregate.

CHAPTER 2

MONOMER SYSTEMS

2.1 Introduction

There are a number of monomers which can be converted into polymers. Selections of a monomer depends on the applications of the concrete-polymer material. However, the primary concern of this report is polymer-impregnated concrete (PIC), especially PIC partial impregnation.

Several monomers, such as isobutyl methacrylate (IBMA), isodecyl methacrylate (IDMA), butyl acrylate (BA), isobutyl acrylate (IBA), and styrene (S), have been investigated in previous research (Ref. 14). Methyl methacrylate (MMA) is the monomer which is used most extensively in this dissertation. It appears to be the most promising monomer because of its high strength and excellent durability and relatively low cost. It has been widely used and considerable information is available. In commercial form, this polymer goes by such trade names as Plexiglas and Lucite. Table 2.1 provides a list of the monomers which were used in this research study. The monomer system usually consists of a monomer, an initiator, a cross linking agent, and in some cases an accelerator. The most promising system has been MMA with 1 % (wt.) of the initiator, benzoyl peroxide (BP), and 10 % (wt.) of the cross linking agent, trimethylopropane trimethacrylate (TMPTMA). For the PIC applications, the BP, a powder, is dissolved in

TABLE 2.1 PROPERTIES OF MONOMERS

Monomer	Symbol	Viscosity, centistoke	Boiling Point, °C	Glass Transition of Polymer, °C (T_g)	Density, gm./cc.
Methyl methacrylate	MMA	0.55	100	+ 105	0.940
Isobutyl methacrylate	IBMA	0.83	155	+ 48	0.883
Isodecyl methacrylate	IDMA	2.90	very high	- 41	0.870
Butyl methacrylate	BMA	0.86	163	+ 20	0.889
Butyl acrylate	BA	0.80	147	- 54	0.894
Isobutyl acrylate	IBA	0.77	145	- 43	0.883
Styrene	S	0.71	145	+ 100	0.900

the MMA,the TMPTMA is added and the mixture is then applied to concrete.

More recently, the monomer system of MMA with 0.5 % (wt.) of initiator, azobis (isobutyronitrile), AIBN (VAZO 64), and 5 % (wt.) TMPTMA has been used instead of the previous monomer system. The new system is cheaper (less TMPTMA and initiator) as compared to the previous system. However, the properties and method of application are about the same.

The purpose of this chapter is to review the previous research on monomer systems and related to current studies. Definitions and investigation of monomer system are also presented.

2.2 Definitions

Monomer - A monomer is a small, simple molecule which can be chemically linked together into a long, repeated chain-like structure and higher molecular weight, which is known as polymer. The chemical process which these linkages occur is known as polymerization.

Initiator - The initiator, often referred to as catalyst, is a chemical compound,usually peroxide, hydroperoxide, or azo. It should be noted that these compounds should be referred to as initiators, and not as catalysts since they are consumed in the reaction (Ref. 21).

Cross-Linking Agents - Cross-linking agents are the multifunctional monomers which produce cross-linking in the polymer. They are used in this study to increase the rate of polymerization.

2.3 Investigation of Monomer Systems

There are many factors which can affect the selection of the monomer systems for PIC applications. The particular concern here is viscosity, volatility, polymerization rate, safety,and cost (Ref. 3).

Common liquid monomers generally have low viscosities at ambient temperature and are suitable for PIC applications. The viscosity of the monomer system must be sufficiently low to achieve an adequate penetration within a reasonable time. From Table 2.1 it can be seen that all of the monomers used in this study had viscosities of less than one centistoke (the viscosity of water) except IDMA. In the laboratory, the rate of penetration can be increased by applying a vacuum to the concrete to remove the entrapped air prior to monomer application and by applying pressure to the monomer to increase the driving force for flow into the concrete pores. However, these approaches do not appear to be practical at the present for field impregnating large surface areas such as bridge decks.

For field impregnation, the longer soaking time is necessary to achieve adequate penetration. However, it was found (Ref. 5) that penetrations of up to 1.5 in. could be achieved with only a shallow depth of monomer ponded on the surface. Monomer penetration is believed to be caused primarily by the capillary forces rather than the pressure developed by the small head of monomer. Fig. 2.1 shows the effect of viscosity on monomer penetration into concrete. Tests

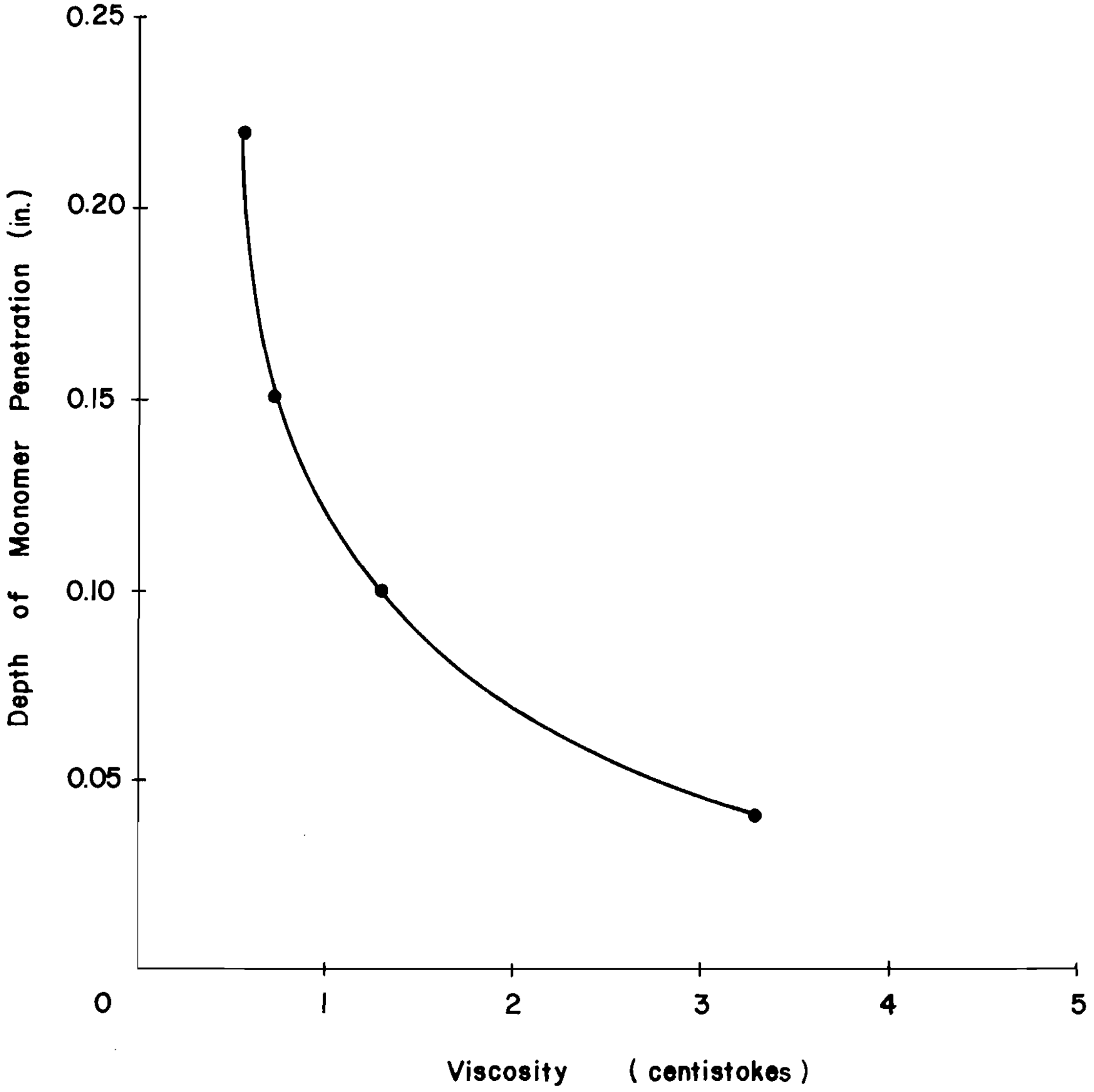


Fig. 2.1 Effect of Viscosity on Monomer Penetration into Concrete after a 30-Minute Soaking Period (Ref. 3).

were conducted at room temperature (≈ 75°F). The monomer was MMA and the viscosity was varied by the addition of acrylic co-polymer (Rohm & Haas 6906-XP). Additional details of the testing procedure are given in Ref. 3. It can be observed that the viscosity is extremely important to the time required for the impregnation of concrete. This is one of the main reasons that MMA, with a viscosity of 0.55 centistoke, is one of the most successful monomers for PIC.

Volatility of monomer is an important factor. Monomer loss due to evaporation during soaking is a function of the time required. This problem seems to be more serious with field applications. However, it was found that spreading a shallow layer of dried fine aggregate on the surface of the slab prior to monomer application minimized evaporation losses (Ref. 5).

There are several methods by which polymerization can be achieved. However, in surface impregnation it is essential that polymerization of the monomer in the concrete occur after adequate monomer penetration is achieved. Previous research (Ref. 3) had been done using an accelerator N, N-dimethyl-p-toluidine (DMPT) to decompose the initiators (usually peroxide or azo compounds) as is done in many polymer casting operations where external heat is not applied. However, adequate penetration could not be achieved, and it was believed that the monomer polymerized before penetrating to the desired depth. It became apparent that a less reactive monomer system should be used to permit adequate penetration followed by an application of external energy to achieve polymerization.

Polymerization (curing) of the monomer in concrete can be achieved by several means, including radiation and heat. For reasons of safety and cost, heat seems to be the most feasible method, especially for field application. At the present time, steam seems to be the most promising method of curing for PIC surface treatment. With adequate drying and soaking times, a polymer depth of 2 in. is now routinely obtained with steam curing.

The heating required in the polymerization process can be considered as a disadvantage. To minimize this disadvantage the initiator and the cross-linking agent were introduced to the monomer system. Decomposition of the initiator usually produces the free radicals which initiate the polymerization of monomer in concrete. There were a number of initiators which had been explored, but BP was the most extensively used initiator in this study. A cross-linking agent was added to the system to increase the molecular weight and to increase the polymerization rate (Ref. 16). TMPTMA was used as the cross-linking agent throughout this report.

Most monomer systems develop an exotherm upon polymerization. This exotherm helps initiate further polymerization, and the rate of polymerization is effectively increased. The nature of the exotherm is strongly affected by the sample size and the type of initiator.

Fig. 2.2 shows the effect of cross-linking agent on the time to peak temperature. The monomer system was 50 ml. of MMA with 2 % BP

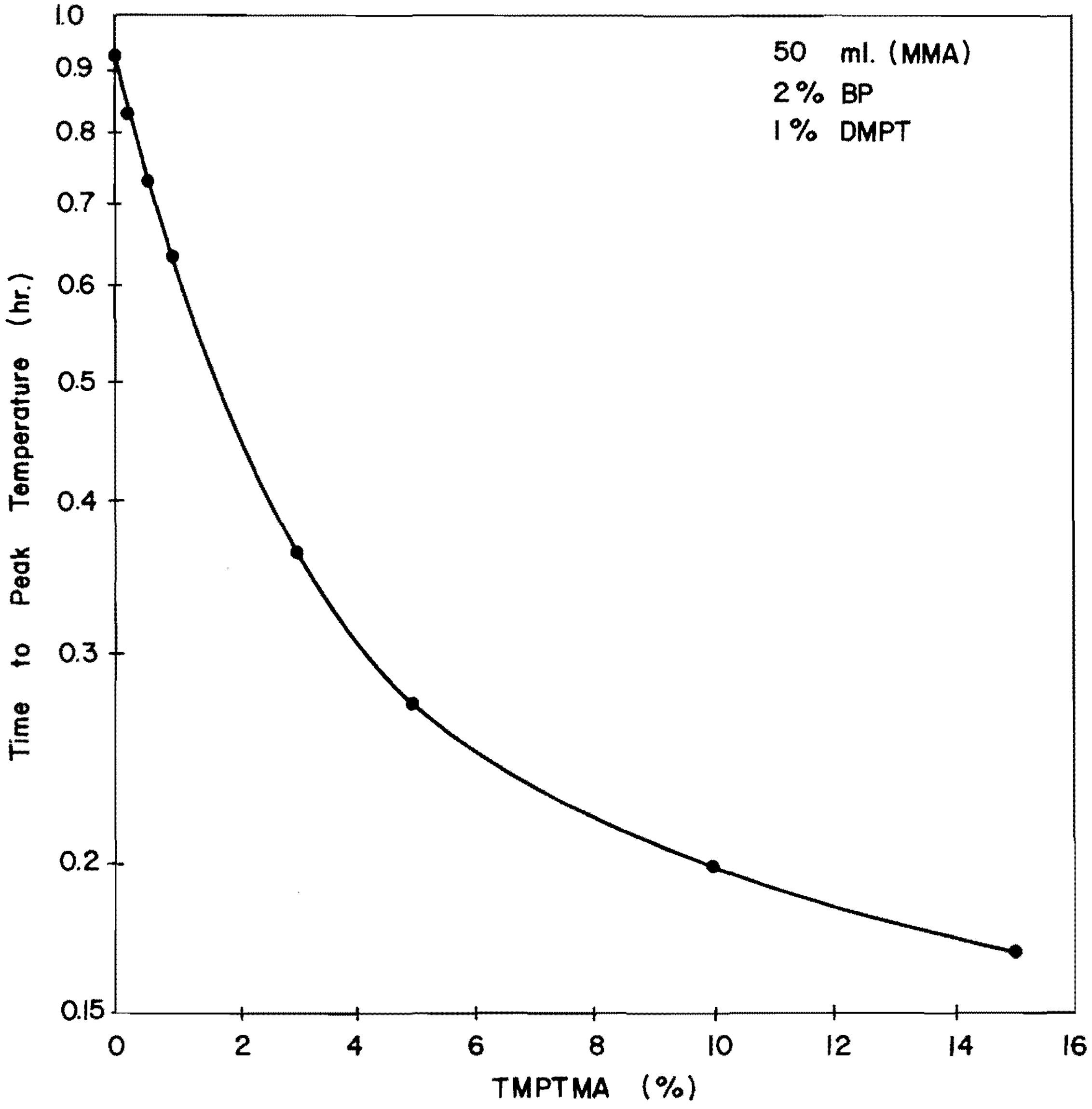


Fig. 2.2 Effect of TMPTMA Level on the Time to Peak Temperature (Ref. 3).

(wt.), and 1 % of the accelerator, DMPT. The DMPT was applied to the system so that the polymerization could occur at lower temperatures. The amount of cross-linking agent varied from zero to 15 % (wt.). It can be seen from Fig. 2.2 that the significance of the effect of cross-linking agent on the time to peak temperature decreased as the amount of TMPTMA increased.

In summary it can be stated that

1) MMA has been found to be the most logical monomer from the standpoint of economics, strength, and durability and was the most widely used in this study.
2) The usual monomer system in this study is the monomer with 1 % (wt.) initiator and 10 % (wt.) cross-linking agent.

CHAPTER 3

INVESTIGATION OF DRYING TIME AND TEMPERATURE OF CONCRETE PRIOR TO POLYMER SURFACE IMPREGNATION

3.1 Introduction

Concrete is a composite material which consists of fine aggregate, coarse aggregate, cement paste, air, and free water. Approximately 75 % of the concrete volume is made up of the aggregates and the remainder is cement and free water. Because the basic structure of the hardened paste is a rigid gel with considerable porosity and a large specific surface, it is capable of holding a substantial quantity of water under the influence of attractive forces of varying degrees of magnitude. It may be expected that the amount of the water held within the overall gel structure will vary with the humidity of the surrounding atmosphere. Some water occupies the capillary pore spaces but a significant amount occupies the gel pore spaces (Ref. 8). The free water in concrete may be an unreacted water remaining from the hydration process or the result of the absorption of the moisture from surrounding condition.

It is now obvious that adequate drying of concrete prior to the monomer application is essential to obtain the maximum monomer loading in the concrete specimens. For polymer-impregnated surface treatment, it is necessary to remove the moisture from the concrete to be impregnated because this moisture in the concrete can block the penetration of the monomer. Removal of the moisture can be accomplished by exposing concrete to external heat sources such as solar energy, ovens, hot air heaters, gas burners, or infra red heaters. The optimum drying temperature is somewhat arbitrary because of the chemical and physical changes which take place when concrete is heated. The uncombined moisture evaporates even at room temperature and all of it is lost soon after 212° F is attained. However, the water in the gel pores may not evaporate at the same temperature as the free water, but it may evaporate at higher temperatures (Ref. 8).

To expose the surface of the concrete slab to extremely high temperatures may cause some damage to the concrete. It has been found that to dry concrete slabs within a practical length of time prior to monomer application, it is necessary to raise the concrete temperature to 212° F or higher. Raising the drying temperature may affect the physical and mechanical properties of the concrete. The tests in this study were performed with drying temperatures of 350° F or less. Previous research indicated that slight damage to concrete may occur at 350° F (Ref. 9). However, at Lehigh University (Ref. 33), 3 x 6-in. cylinders were dried by propane burners for 8 hrs. under 1 to 2 in. of sand with the temperature in the sand reaching approximately 750° F, and no visible cracks were noted on the cylinders after the drying was completed.

In this study, preliminary tests were conducted by exposing the top surface 10-in. x 10-in. x 6-in. concrete slabs to a temperature of 450° F. Careful observations indicated no visible damage to the surface of the concrete slabs.

3.2 Scope

Various investigations and test programs have been conducted to determine the relationships among the drying temperature, the length of drying time, and the polymer depth. The determination of the acceptable drying temperature and the length of drying time is based on the resulting polymer depths. The tests were conducted at various lengths of drying times and temperatures. To simulate the actual partial impregnation of bridge decks, some of the 10-in.x10-in.x6-in. slabs were dried under simulated field conditions. The concrete mix and properties are shown in Appendix 2.

Tests were also performed on the water saturated specimens to evaluate the effect of relative moisture content on drying and impregnation. The tests were performed in the laboratory with a drying temperature of 300° F. To investigate the temperature build-up inside the concrete slab, another series of tests was conducted on 6-in. concrete slabs when the top surfaces were exposed to heat. The temperatures at top and bottom and at various depths inside the concrete were measured periodically. Tests were conducted for top surface temperatures of 250 , 300, and 400° F.

3.3 Investigations of Temperature Gradients in 6-in. Concrete Slab

To investigate the temperature gradients inside concrete slabs, 10-in.x10-in.x6-in. concrete slabs were selected as the specimens. Thermocouples were inserted from the bottom into the concrete slabs at depths of 0.80, 1.30, 3.0, 5.0 in. from the top surface and also placed at the top and bottom of the slabs. An electric infrared heater was used as the external heat source. Variations of the temperature at the surface of concrete slabs can be made by varying the distance between the heater and the surface of the concrete slab.

The first series of tests was performed to determine the relationship between the temperature at the surface of the concrete slab and the distance between the heater and the surface of concrete slabs. The purpose of this test was to find the distance between the heater and the surface of concrete slab that would develop a constant temperature on the surface after a certain period of drying time. The results of these tests were used as a basis for further tests.

The heater was set at various distances from the top of the concrete slabs. To simulate the actual field test condition, the test slabs were surrounded by concrete blocks to prevent the translation of the heat during the drying period. The thermocouples were placed on the surface and the temperatures were measured periodically. Fig. 3.1 shows the test results, it was found that to maintain the temperatures at the top of the concrete slabs at approximately 250 ,

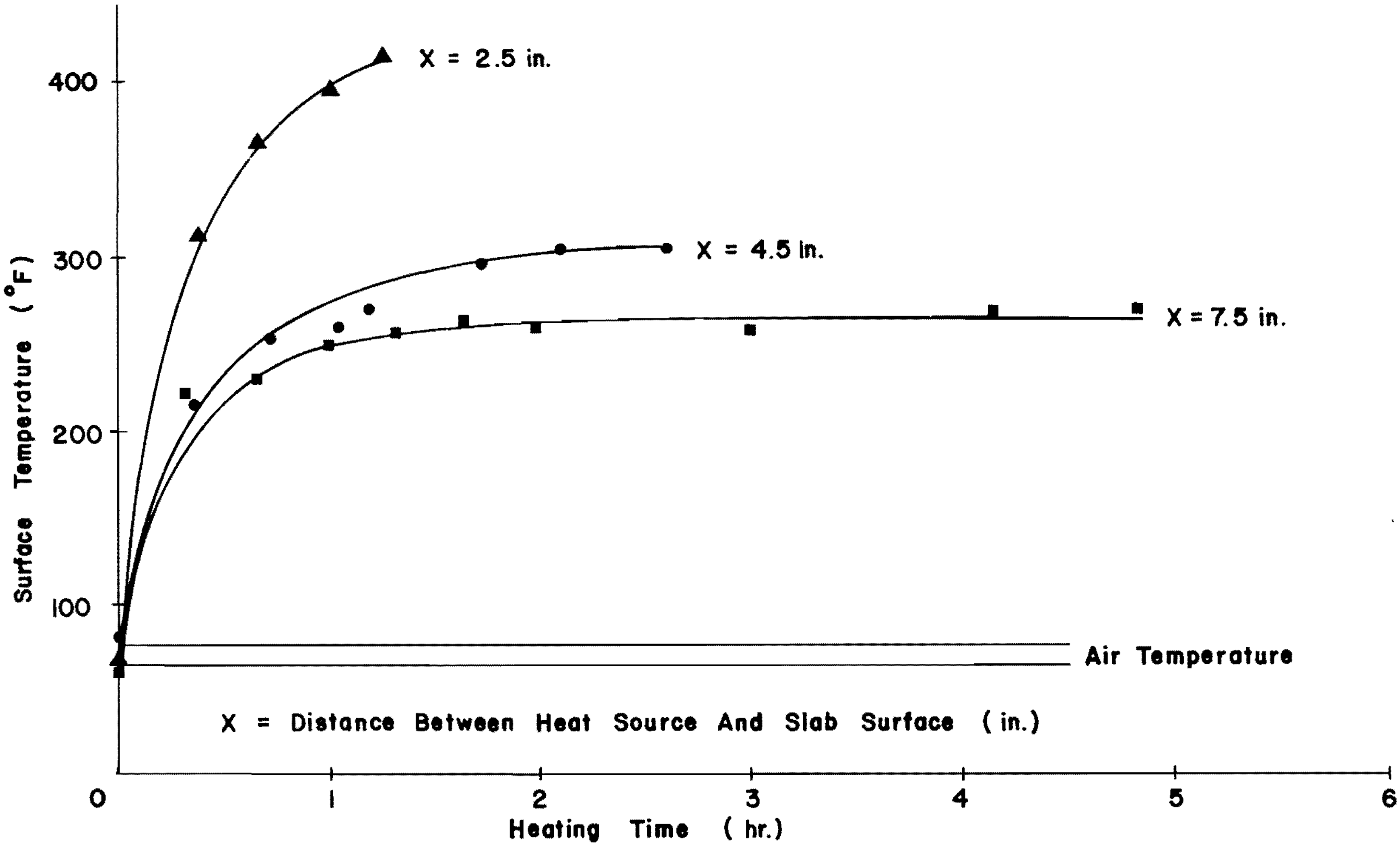


Fig. 3.1 Relationship Between the Surface Temperature and the Distance Between the Heat Source and the Concrete Slab Surface.

300, and 400° F, the distances between the heater and the surface of the concrete slabs were 7.5, 4.5, and 2.5 in., respectively.

The second series of tests was performed on the specimens with the thermocouples at various depths. The purpose of the tests was to determine the temperature gradients in 10-in.x10-in.x6-in. concrete specimens when the top surfaces were exposed to external heat. To simulate the actual field condition, the tests were performed outdoors and the specimens were surrounded by a number of concrete blocks to prevent heat loss during the test period. An electric infrared heater was used as the external heat source.

The tests were performed with the surface temperatures of 250, 300, and 400° F for a period of 3 hrs. The air temperature varied between 62° F and 78° F. Test results are presented in Table 3.1. Fig. 3.2 indicates the typical temperature gradients in a 6-in. slab for a surface temperature that increased to 320° F. Temperatures are shown for time from 30 to 180 minutes after the heat source was applied. It was found that after 1 hr. only the upper 0.50 in. of concrete is heated to 212° F for surface temperature of approximately 300° F. It takes 75 minutes to obtain 212° F at a 1-in. depth with surface temperature of 300° F. These temperature gradients are, of course, dependent upon the rate of temperature buildup on the surface.

TABLE 3.1 TEMPERATURE GRADIENTS IN 6-IN. CONCRETE SLABS HEATED FROM THE UPPER SURFACE.

Heating Time, minutes	Maximum Temperature, °F	Depth Below Surface, in.						
		0.00	0.80	1.30	3.00	5.00	6.00	Air
0	250	60	60	60	60	60	57	62
	300	80	69	68	71	71	69	78
	400	67	64	64	64	64	64	70
30	250	225	140	118	88	66	68	62
	300	235	151	134	103	82	79	78
	400	330	196	173	116	76	78	70
60	250	251	174	155	117	87	85	62
	300	260	196	186	141	104	96	78
	400	396	274	254	187	118	112	70
120	250	260	206	192	154	122	116	62
	300	302	238	228	183	143	125	78
	400	-	-	-	-	-	-	70
180	250	254	212	198	162	136	125	62
	300	320	256	248	204	164	141	78
	400	-	-	-	-	-	-	70

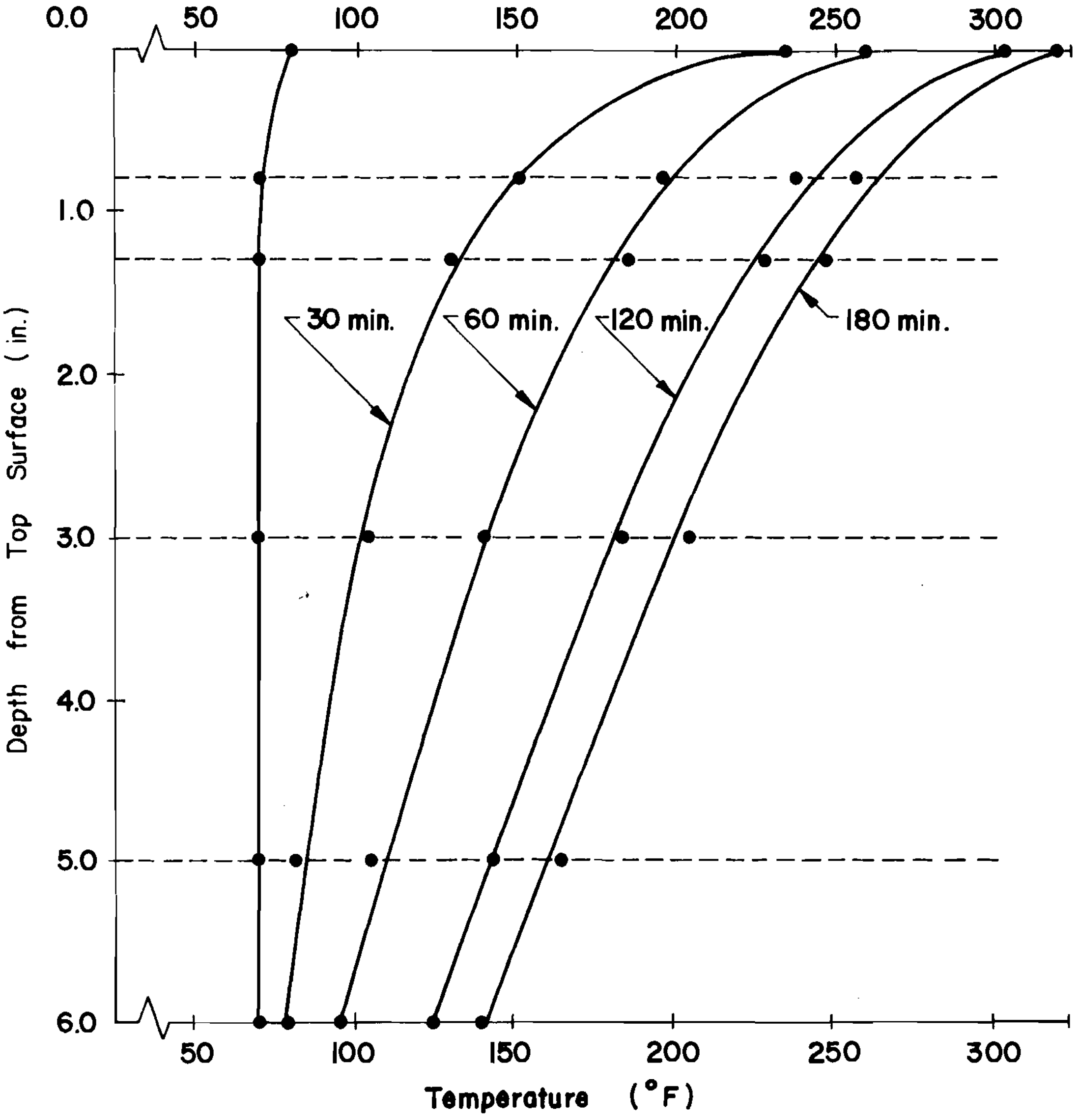


Fig. 3.2 Typical Temperature Gradient in 6-in. Concrete Slab Heated from the Upper Surface.

A computer program which was developed at The University of Texas at Austin (Ref. 61) was also used to predict the temperature gradients in concrete. The conductivity of concrete is assumed to be 0.81 btu /ft./hr./°F, the specific heat is 0.23 btu /lb./°F, and the density is 145 lbs./cu.ft..

Fig. 3.3 shows the comparison between the measured and theoretical values of the temperature gradients along the depth of a 6-in. concrete slab. These temperatures were based on a drying temperature of approximately 250°F at the surface. Within the limits of this test, the test results indicate good agreement with the predicted values.

3.4 Drying of Concrete Slabs Prior to Monomer Applications

Initially in this research, it was planned to dry concrete at less than 200°F to avoid any danger of cracking or excessive thermal expansion. However, because of the time required at the low drying temperature to achieve a particular depth of the polymer, the higher drying temperature seems to be preferable. Based on later research at The University of Texas at Austin, Lehigh University, and the Bureau of Reclamation, there seems to be little question that higher temperatures are both preferable and safe. It is also noted that the application of the PIC surface treatment can also be used for repairing any resulting fine cracks. This permits drying concrete

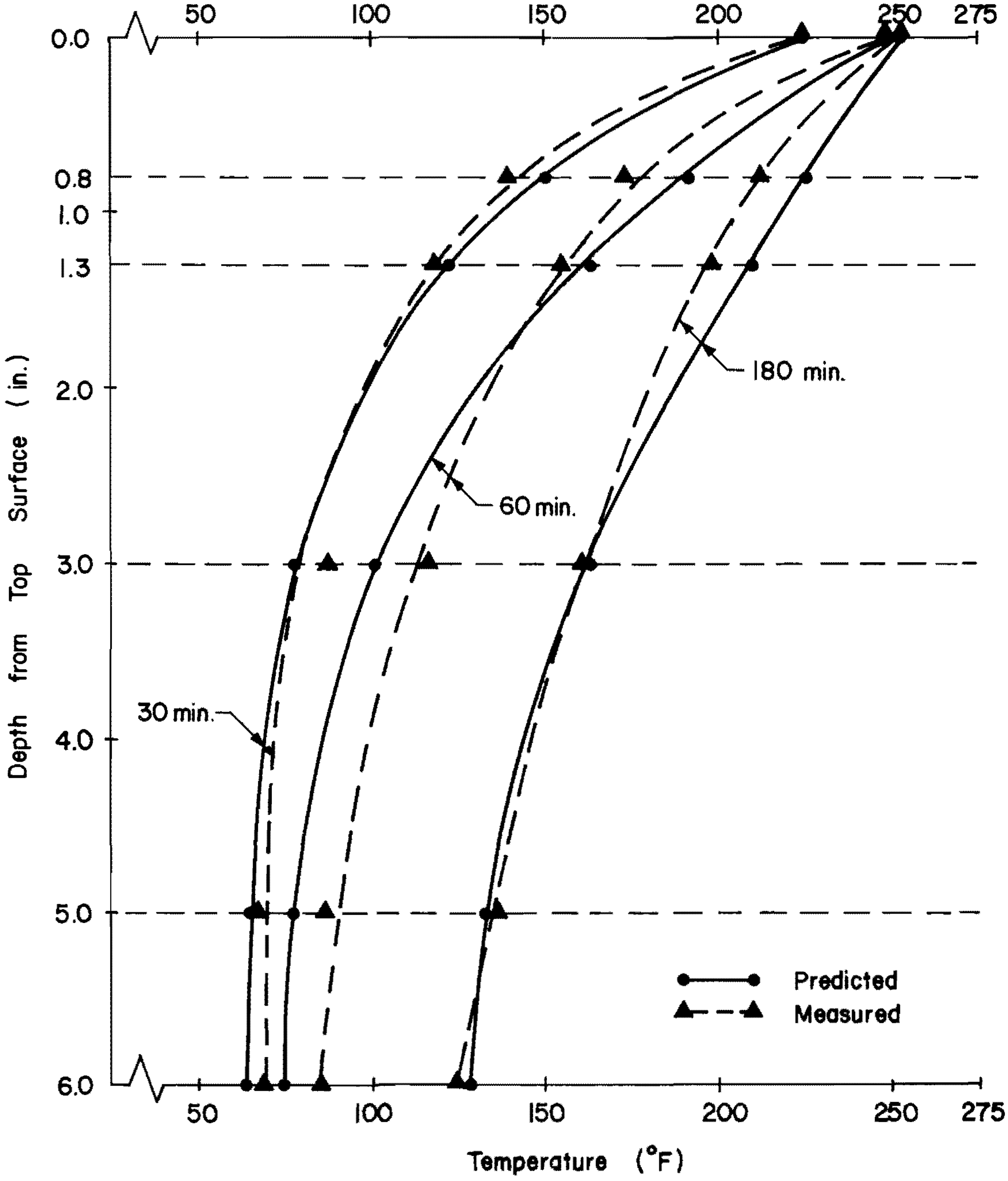


Fig. 3.3 Comparison between the Measured and Theoretical Values of Temperature Gradient in 6-in. Concrete Slabs.

at higher temperatures since if cracking occurs during the drying process the polymer impregnation will repair it.

Previous research (Ref. 8) found that to dry the 3 x 6 in. concrete cylinders to the equilibrium weight at 302, 257, and 221°F required drying times of 8, 20, and approximately 100 hrs., respectively. The effects of drying time on drying temperature are plotted in Fig. 3.4. It is indicated that increasing the drying temperature significantly decreases the drying time. At the temperature range of 200°F to 260°F, increasing the drying temperature decreases the length of drying time significantly. The significance of drying temperature on drying time is decreased when the drying temperature is higher than 260°F.

To determine the effect of drying temperature and length of drying time on polymer depth, two types of tests were used:

1) laboratory oven-dried
    a) room equilibrium specimens and
    b) water saturated specimens; and
2) field simulated
    a) room equilibrium specimens

In each test, the specimen was dried and treated with the monomer system and cured as previously discussed. After the treatment, each specimen was broken apart to measure the polymer depths. The depths were measured at 2-in. increments across the 10-in. wide slab. To minimize the edge effect, only the measured polymer depths from

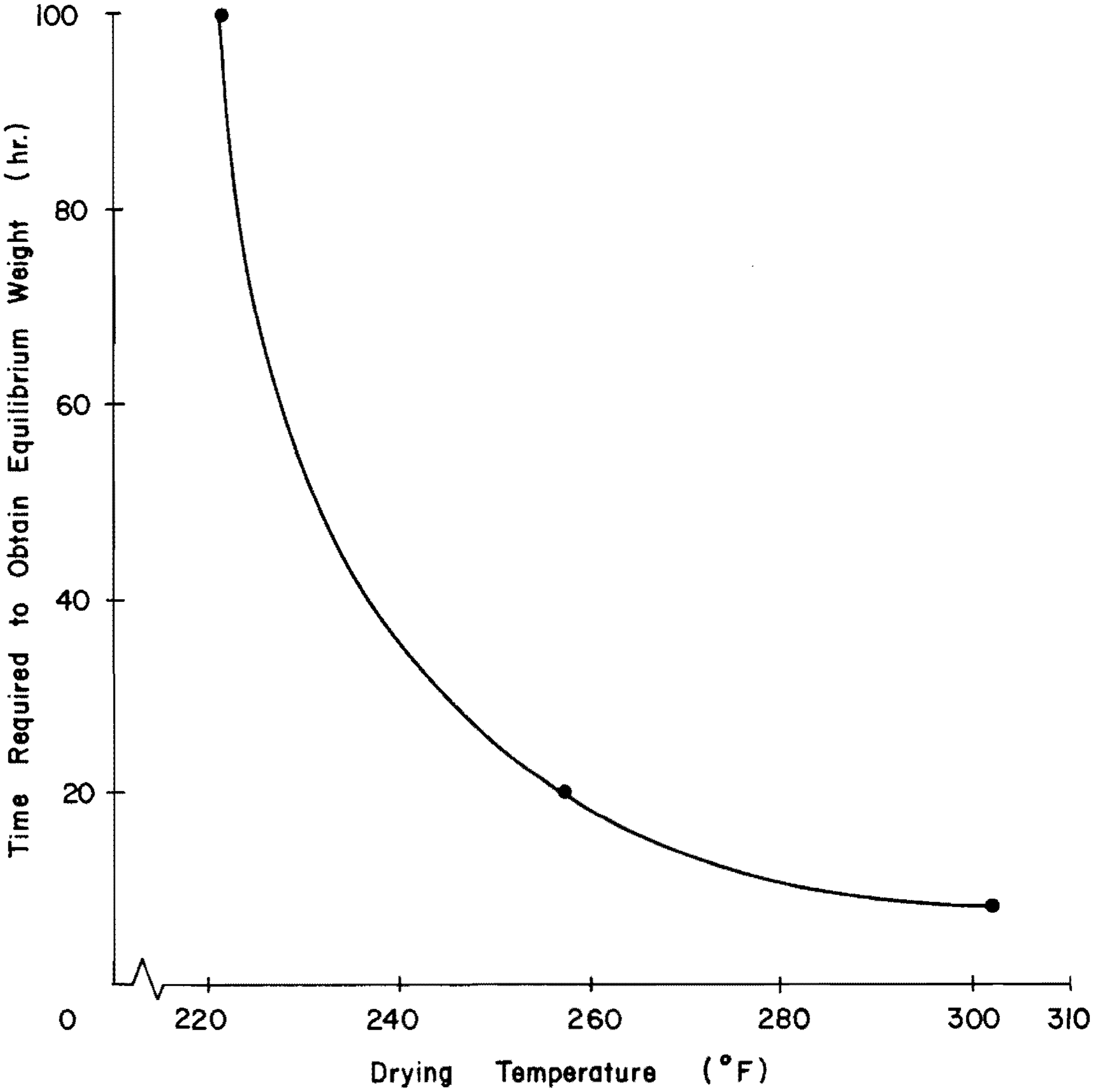


Fig. 3.4 Effect of Drying Temperature on Drying Time of 3 x 6-in. Concrete Cylinders (Ref. 8).

the interior locations were used to determine the average polymer depths.

3.4.1 Laboratory oven-dried.

a) Room equilibrium specimens. Tests were conducted on the air-dried specimens,which have approximately 30 to 50 % of relative moisture content (R.M.C.). The relative moisture content is defined as the ratio of the weight of the remaining moisture at time of test to the total weight of water at saturated surface dried condition;

$$\text{R.M.C.} = \frac{\text{Test Weight - Dried Weight}}{\text{SSD - Dried Weight}}$$

SSD = saturated surface dried weight

To determine the effect of drying time and temperature on polymer depth for partial impregnation, the experiments were performed at the test temperatures of 212, 275, and 300°F. Thermocouples were placed on the top surface of the concrete specimens to monitor the temperature periodically during drying. The specimens were placed in the oven and dried at each of the drying temperatures for approximately 2, 4, 6,and 8 hrs..

After the specimens were dried at each drying temperature, they were wrapped with polyethylene to prevent the absorption of moisture from the atmosphere. The specimens were then cooled for approximately 17 hrs. A 10-in x 10-in. steel frame was attached to the top of the surface to contain the monomer during the soaking period.

The surface of the slabs was covered with 0.25 in. of dried fine aggregate and then a volume of 400 to 500 cc. of the mixture of MMA, 1 % BP and 10 % TMPTMA was applied for a period of 17 hrs. After being cured by steam for 1 hr., the specimens were broken into approximately two equal parts to measure the depth of the polymer. The average polymer depth was determined.

Fig. 3.5 shows the effect of the drying time and temperature on the polymer depth. Fig. 3.5 (a) shows the relationship between the drying time and the drying temperature. It can be seen that with the oven drying, approximately 2.5 hrs. is required to build up the surface temperature to the test temperature. However, this time can be decreased by increasing the oven temperature. After the specimens were maintained at the test temperature for periods of 2, 4, 6, and 8 hrs., they were taken from the oven. After the specimens had cooled, they were treated as previously discussed.

Fig. 3.5 (b) shows the relationship between the drying time and polymer depth at the drying temperature of 212, 275, and 300°F. With the drying temperature of 212°F, the maximum polymer depth is less than 0.25 in. for the drying time of 10 hrs. in the oven. It may be noted that at this temperature, two specimens were left in the oven for 40 hrs. After treatments the polymer depths were measured and found to be less than 0.5 in. From these results, it can be concluded that to reduce the drying time to the acceptable level the drying temperature must be greater than 212°F.

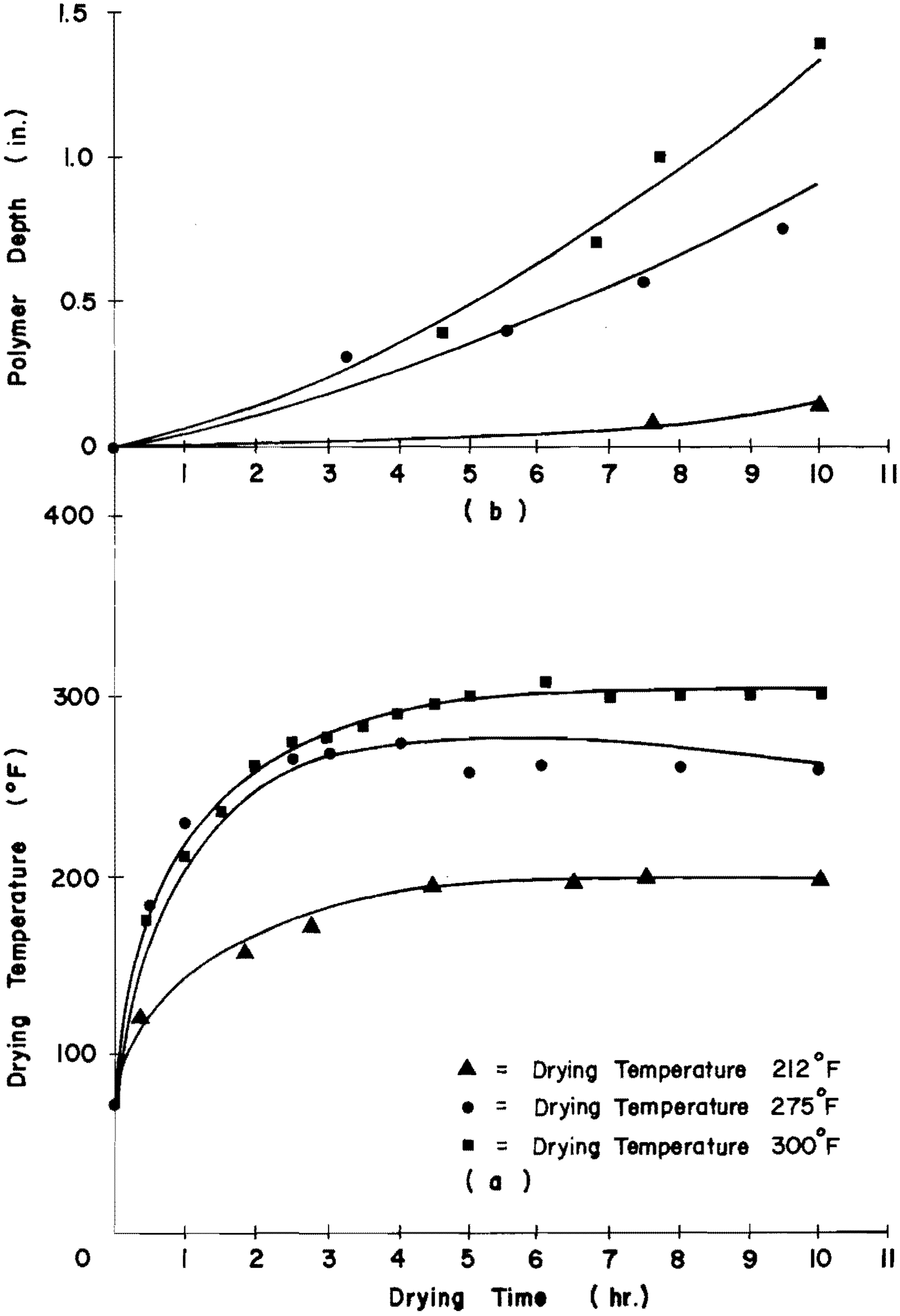


Fig. 3.5 Effect of Drying Temperature and Time on Polymer Depth (Laboratory Test).

With the drying temperatures of 250°F and 300°F there are significant increases in polymer depths for the same drying time. To obtain 0.5 in. of polymer depth it is necessary to dry the specimens at the temperatures of 300, 275, and 212°F for 4.5, 6.5, and more than 24 hrs., respectively. After 8 hours of drying, 0.2, 0.6, and 1.0 in. polymer depths were achieved with drying temperatures of 212, 275, and 300°F, respectively.

b) Water-saturated specimens. The tests were planned with the purpose of evaluating the effect of the R.M.C. on the polymer depth of the specimen prior to the drying process. It should be noted that the moisture in concrete varies with depth and the R.M.C. represents the average value. To simulate the worst possible conditions, initially saturated specimens were used.

The specimens were first oven-dried and weighed. Then six 10-in.x10-in.x6-in. concrete specimens were immersed in water for 7 days to saturated them. Along with two 10-in.x10-in.x6-in. air dried concrete specimens, they were placed in the oven. The oven temperature was set at 335°F, which produced a steady state surface temperature of 300°F. The thermocouples were attached to the surface of all specimens to monitor the temperatures periodically. The specimens were weighed periodically during the drying process. Fig.3.6 indicates the comparison of loss of weight during the drying process between the air-dried specimens and the saturated specimens. It can be seen that for a drying time of 4 hrs. or less the rate of drying of the

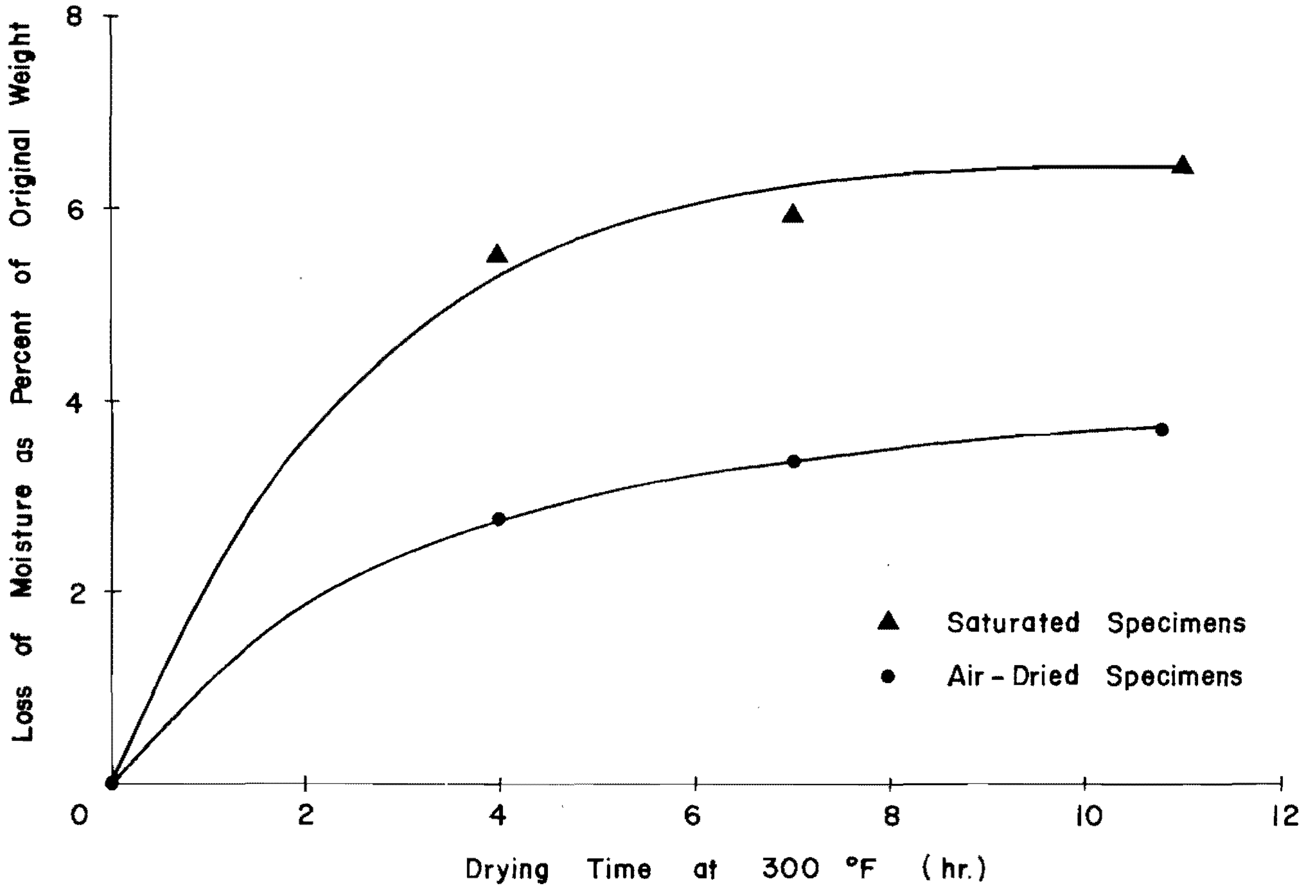


Fig. 3.6 Comparison of Drying Process between Air-Dried and Saturated Specimens.

saturated specimens is higher than that of the air dried specimens. This rate of drying tends to be equalized as the drying time increases. This can be explained by the fact that the conductivity of air is lower than that of water, and the degree of saturation of concrete strongly affects its conductivity. For a given mix design, the conductivity of concrete increases approximately linearly with the increase in R.M.C. (Ref. 4). At the beginning of the drying process, the saturated specimens have a higher thermal conductivity value than air-dried specimens because of higher R.M.C.. This causes greater evaporation of moisture inside the concrete of the saturated specimens. After 4 hrs. of drying, the loss in moisture of the saturated specimens is approximately double that of the air-dried specimens. However, as the drying time increases the R.M.C. of saturated and air-dried specimens is about the same and the rate of drying equalizes.

After the specimens were dried, they were treated as usual. Because the polymer depth of at least 0.75 in., which was believed to be adequate, was desired, the specimens were dried for 8, 10, 18, and 24 hrs. The test results are presented in Table 3.2.

The comparison between the air-dried specimens and the saturated specimens is plotted in Fig. 3.7. It is shown that, after a certain period of drying time, there is no significant difference in polymer depth after treatments. For short periods of drying the air-dried specimens achieved greater polymer depths. For a drying time of 3 hrs., the air-dried specimens developed a polymer depth of

TABLE 3.2 PARTIAL IMPREGNATION OF INITIALLY SATURATED SPECIMENS

Specimen No.	Initial Saturated Weight, lbs.	Weight Before Treatment, lbs.	Weight Loss, lbs.	Time in Oven, hrs.	Polymer Depth, in.
SOD-00*	47.87	46.12	1.75	18.40	1.45
SOD-01	48.94	45.79	3.15	18.40	1.53
SOD-02	48.27	45.14	3.13	18.40	1.33
SOD-03	50.23	47.32	2.91	7.70	1.01
SOD-04	49.22	46.22	3.00	9.50	1.51
SOD-05	49.10	45.77	3.33	24.00	1.51
SOD-06	49.33	48.50	0.83	2.20	0

SOD-00* = Air-dried specimens.

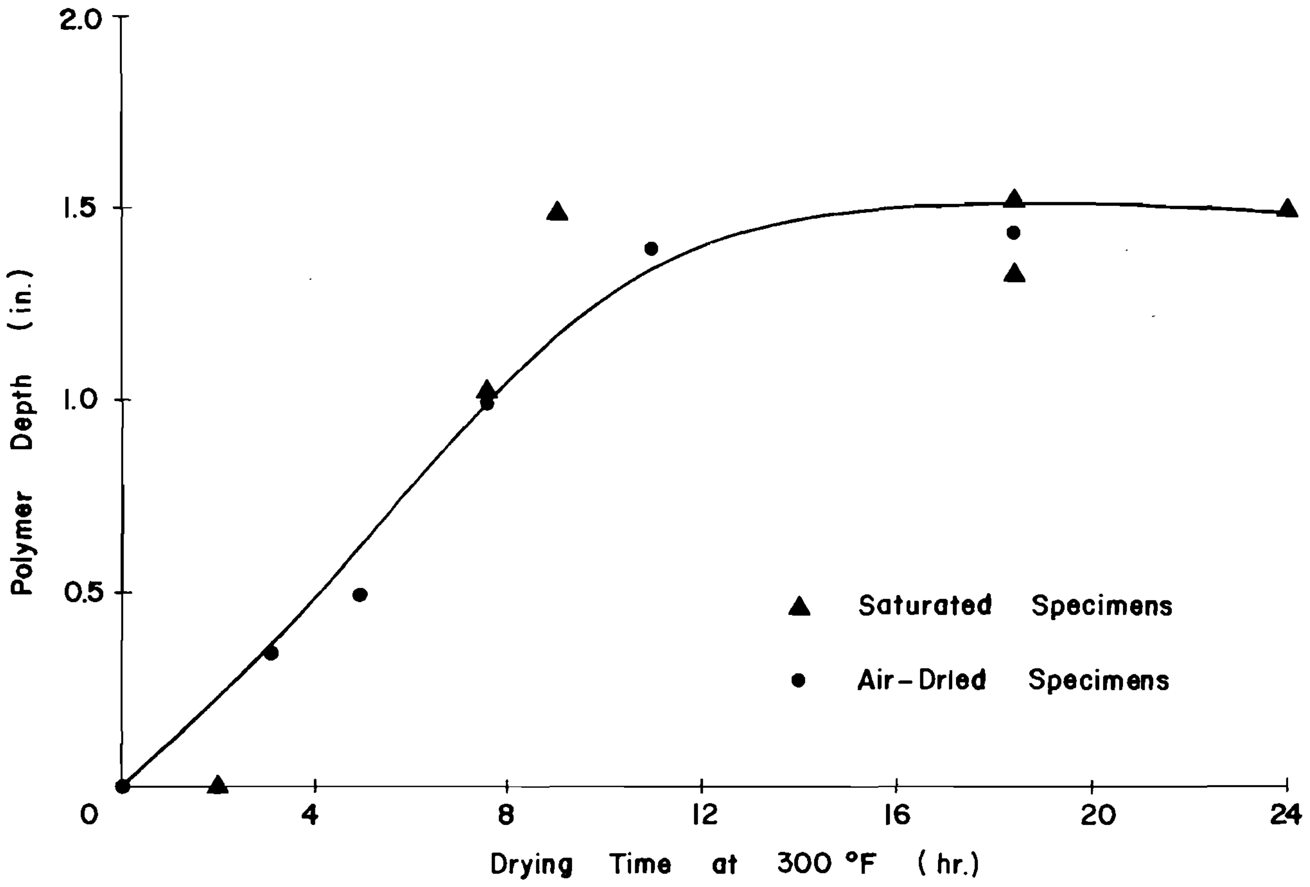


Fig. 3.7 Comparison of Polymer Depths Obtained from Air-Dried and Saturated Specimens at the Drying Temperature of 300°F.

about 0.4 in., whereas, at the drying time of approximately 2 hrs., there is no appearance of polymer in the initially saturated specimens. It can be explained that, for short periods of drying time, the amount of moisture remaining in the specimens prior to monomer application of the saturated specimens is higher than in the air-dried specimens. The moisture inside the specimens prior to the monomer application is the major factor that affects the impregnation because the moisture can block the penetration of the monomer during soaking, and an acceptable depth of polymer cannot be achieved.

For longer periods of drying time, at approximately 8 hrs. or more of the drying time, the saturated and air-dried specimens achieved the same polymer depths. It can be seen from Fig. 3.6 that after 8 hrs. of drying the rate of moisture loss of the saturated and the air-dried specimens is about equal. The rate of drying inside the concrete, at constant temperature, depends largely on the conductivity. Since the rate of drying is constant, it can be said that the conductivities of the initially saturated and dried concrete at this particular drying time are about equal. It means that the R. M. C. of the saturated and air-dried specimens prior to monomer application are practically the same. Theoretically, if the concrete specimens were taken from the same batch and treated at the same R.M.C., they will achieve the same polymer depth.

3.4.2 <u>Field tests.</u> Experiments were also conducted under field conditions at the Balcones Research Center of

The University of Texas at Austin. An infrared electric heater was used as the heat source. The 10-in x 10-in. x 6- in. specimens were placed beneath the infrared electric heater so that only the surfaces of the specimens were exposed to the heat. The thermocouples were also attached to the surface of the concrete and monitored periodically. Since previous tests had shown that the drying temperature of 212° F was not practical for drying the slab, the experiments were performed at the drying temperatures of 250, 300, and 350° F. After the specimens were dried for periods of 2, 4, 6, and 8 hrs. at each drying temperature they were treated and the polymer depth was measured, as previously discussed.

The test results are plotted in Fig. 3.8. It can be seen that increasing the drying temperature reduces the drying time. However, increasing the drying temperature beyond 300° F may physically damage the concrete. A temperature range of 250° F to 300° F is likely to be the most favorable and safe for actual field treatments. From Fig. 3.8, it may be observed that to obtain a 0.5-in. polymer depth it is necessary to dry the specimens at 350, 300, and 250° F for periods of approximately 2, 5, and more than 10 hrs., respectively.

Because the drying temperature of 300° F seemed to be the maximum safe value, another set of experiments was performed in the field at this temperature. The 10-in.x10-in.x6-in. concrete slabs with the thermocouples attached to the surface were placed beneath the heat source. To more closely simulate the actual bridge deck condition, all

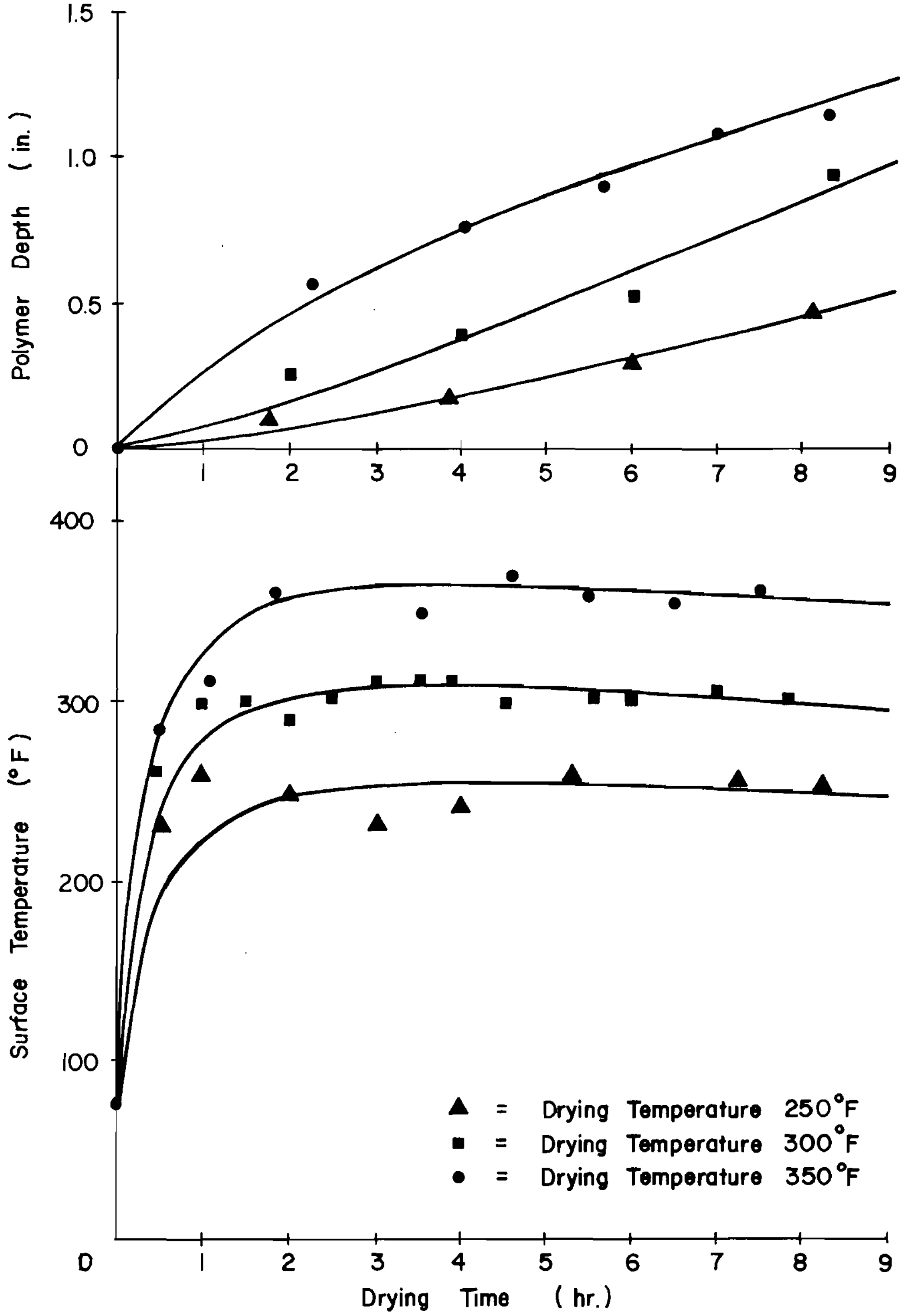


Fig. 3.8 Effect of Drying Temperature and Time on Polymer Depth (Field Test).

sides of the test specimens were surrounded with concrete slabs. The specimens were dried for approximately 2, 4, and 8 hrs., cooled, and treated, as previously discussed.

Fig. 3.9 shows the results of the experiments. From Fig. 3.2, it was observed that the temperature of 212°F was reached nearly 2.5 in. from the surface after 3 hrs.. However, the depth of polymer-impregnation after 4 and 8 hrs. of drying was only about 0.6 and 1.25 in., respectively.

3.5 Summary

Drying of concrete prior to the monomer application is the most time-consuming step in the impregnation process. From the test results, the following conclusions can be drawn:

1) The comparison of temperature gradients in 6-in. concrete slabs between the measured and theoretical values indicates very good agreement.
2) To achieve an acceptable depth (≈ 0.75 in.) of polymer for partial impregnation in relatively short periods of time, it is necessary to raise the temperature to higher than 212°F at that depth.
3) To achieve 0.75-in. polymer depth, the concrete at that depth has to remain at 212°F or higher for approximately 5 hrs.

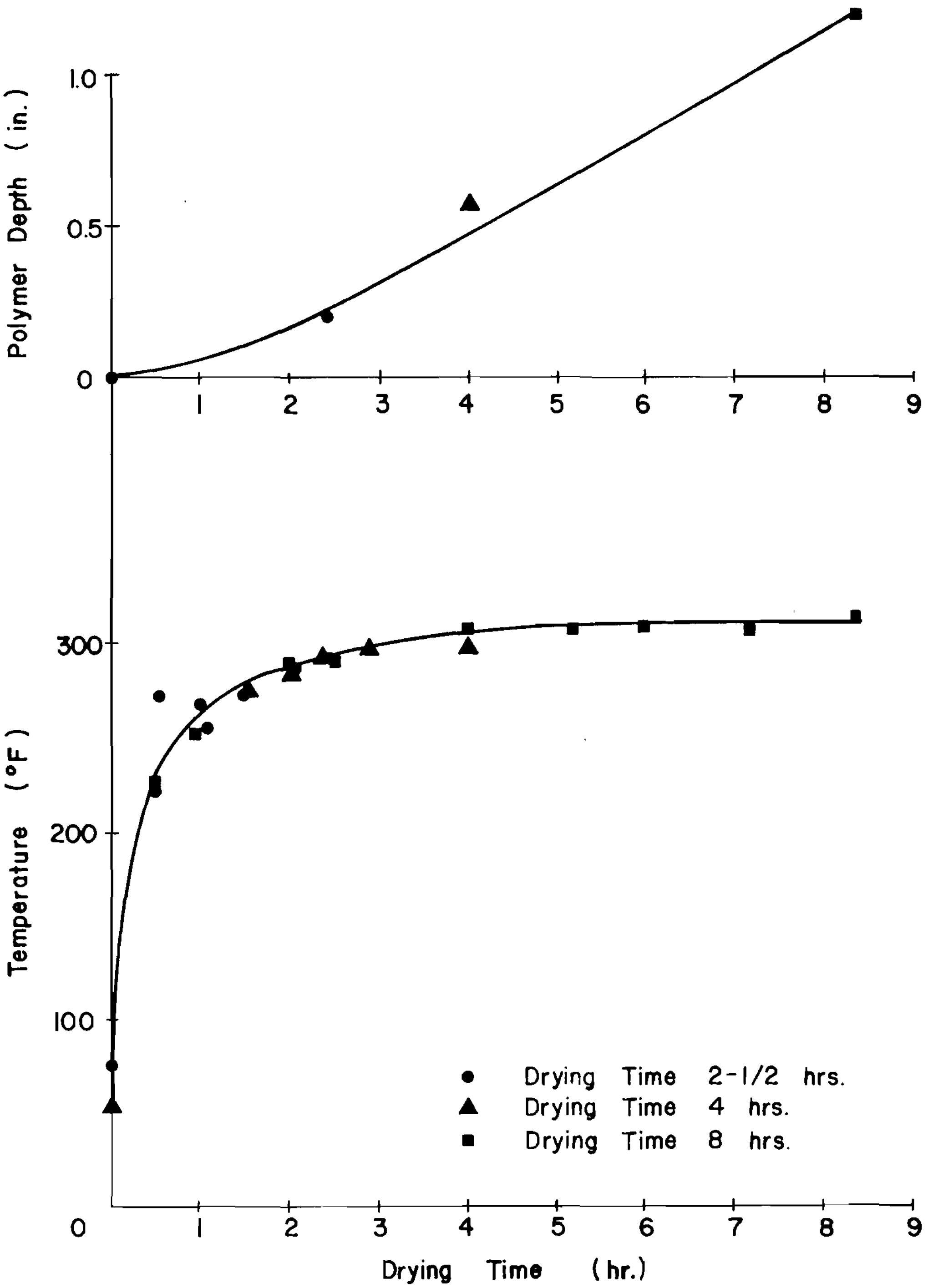


Fig. 3.9 Effect of Drying Time on Polymer Depth at the Drying Temperature of 300°F.

4) The drying temperature of 300°F is likely to be the upper limit and safe value.

5) To achieve 0.5, 1.0, and 1.5 in. polymer depths, it is necessary to dry the slabs prior to monomer application at a temperature of 300°F for 5, 8 and, 12 hrs., respectively.

6) After 8 hrs. of drying at the temperature of 300°F, there is no difference in polymer depths achieved for initially air-dried and saturated specimens.

CHAPTER 4

SOAK TIME FOR PARTIALLY POLYMER-IMPREGNATED CONCRETE

4.1 Introduction

Soaking the concrete surface with the monomer prior to curing is another important step of partial polymer-impregnation. It is well known that concrete is pervious to water. This is evidenced by its absorption of water by capillary action and by the passage of water under pressure through it. The porosity of the concrete is the major factor affecting the absorption ability of concrete. The many factors affecting the permeability of concrete can be divided into three groups (Ref. 8):

1) the influence of the constituent materials,
2) the effect of the method of preparing concrete, and
3) the influence of subsequent treatment of the concrete.

Monomers used in polymer impregnation are in liquid form and have a wide range of viscosities which significantly affect the absorption rate into concrete. The monomer viscosity controls the penetration rate of monomer into the concrete surface, with low viscosity monomers usually achieving higher penetration than more viscous monomers. For fully-impregnated PIC, the soaking process can be accomplished by surrounding the specimens with monomer.

For partially polymer-impregnated concrete such as the surface treatments of slabs, the monomer is sprayed or sprinkled onto the surface. The period of soaking time required to produce a uniform penetration of monomer into the concrete was studied. It is obvious that the monomer depth usually increases as the soaking time increases. However, the time required for a monomer to develop the desired depth of penetration depends on many factors, as follows:

1) monomer viscosity,
2) temperature of monomer and concrete,
3) condition of concrete surface,
4) relative moisture content (R.M.C.) of concrete prior to monomer application,
5) porosity of concrete, and
6) time of monomer application.

Evaporation of the monomer during soaking potentially could be a serious problem for surface treatments. To minimize evaporation and to hold the monomer in place during soaking, a 0.25 in. sand layer is used.

Previous research (Ref. 7) indicated that the penetration of monomer was significantly affected by viscosity and slab temperature. The penetration decreases as the viscosity increases. The effect of slab temperature on monomer penetration was previously studied at two slab temperatures, 75 and 125° F. It was found that, for a soak time of 30 minutes, the slab temperature of 125° F produced higher monomer penetration.

Textured surfaces were previously found to be more easily penetrated by the monomer than smooth formed surfaces for short periods of soaking time. This seems to be the result of breaking the concrete surface film formed during hydration process of the concrete (Ref. 7).

Previous research also found that the desired amount of monomer penetration can be obtained with a soaking time of 10 hrs. or more. However, the evaporation problem of monomer during soaking is more pronounced with the longer period of soaking time. To minimize this problem, coating the monomer-loaded specimens with monomer-polymer solution or wrapping the specimens with polyethylene (Ref. 17) can be used in the laboratory. However, this method does not seem to be applicable in field treatments.

For field treatment, sand on the surface was kept moist to saturated by adding monomer periodically. To minimize the evaporation problem, the slab surface was covered by polyethylene membrane during the soaking period. A soaking time of 8 hrs. or less is preferable because of the evaporation problem. These reasons led to the investigation of the parameters which can affect monomer penetration of concrete for surface treatment within the acceptable time.

4.2 Experimental Design

The emphasis of this chapter is on the partial polymer-impregnation with the primary purpose of developing the relationship between the soaking times and polymer depths. The polymer depth depends on the monomer penetration into the concrete surface. The factors affecting the monomer penetration were evaluated. To measure the penetration of monomer into the slab surface, a simple test apparatus was developed.

Attempts were made to determine the absorbability of concrete prior to monomer application. For PIC surface treatment, only the absorbability of concrete on the surface up to the required polymer depth needs to be determined. At the present time, there is no reliable method to measure the absorbability directly on the surface of concrete bridge decks prior to monomer application. These reasons have led to the development of apparatus to determine the absorbability of concrete bridge decks.

Table 4.1 shows the factorial design for the investigations of partial polymer impregnation. The first series of experiments was conducted to determine the effects of R.M.C., quality of concrete, and soaking time on monomer penetration. Tests were performed with the developed apparatus on 2-in. concrete slabs with three different concrete qualities. Measurements of monomer absorption in cubic centimeters were made periodically within 2.5 hrs.

A second series of experiments was conducted to determine the relationship between the polymer depths and the soaking time.

TABLE 4.1 FACTORIAL DESIGN FOR INVESTIGATIONS OF PARTIAL POLYMER IMPREGNATION.

Water-Cement Ratio, gal./sk.	Relative Moisture Content, %	Monomer Absorption			Polymer Depth			
		Soak Time, min.			Soak Time, hrs.			
		30	60	120	2	4	8	17
6.0	0	X	X	X	X	X	X	X
	15	X	X	X				X
	30	X	X	X				X
	60	X	X	X				X
7.5	0	X	X	X	X	X	X	X
	15	X	X	X				X
	30	X	X	X				X
	60	X	X	X				X
8.0	0				X	X	X	X
	15							X
	30							X
	60							X
9.0	0	X	X	X	X	X	X	X
	15	X	X	X				X
	30	X	X	X				X
	60	X	X	X				X

X Tests were conducted.

The emphasis was on soaking times of 8 hrs. or less.

A third series of experiment was conducted to determine the effect of R.M.C. on polymer depths.

4.3 Monomer Penetration Experiment

To determine the effect of R.M.C., quality of concrete, and soaking time on monomer penetration, 10-in.x10-in.x2-in. concrete slabs with water-cement ratios of 6.0, 7.5, and 9.0 gal./sk. were used. The mixture of MMA, 1 % (wt.) BP, and 10 % (wt.) TMPTMA was used as the monomer system throughout the tests.

4.3.1 Preparation of Specimens. The preparation of test specimens and test procedures are as follows:

1) For convenience, the specimens were oven-dried at a temperature of 212°F for 7 days to obtain the dry weight.
2) After the specimens cooled to room temperature, they were immersed in water for 7 days to obtain the saturated surface dried weight.
3) To prepare the test specimens with the particular R.M.C., the saturated specimens were placed in the oven and each specimen was taken from the oven periodically.
4) The 10-in.x10-in. steel frame and the test apparatus were attached to the concrete surface by silicone rubber adhesive.
5) The monomer system was poured into the test apparatus. The reading of monomer penetration was taken periodically for 2.5 hrs.

4.3.2 Description of Test Apparatus. The simple penetration test apparatus was developed to measure the absorption property of concrete. The apparatus consists of two glass funnels, one on each end of the glass cylinder, as shown in Fig. 4.1. The large funnel has a diameter of 11.5 cm. and is attached to the surface of concrete. The small funnel is for the purpose of pouring liquid monomer into the apparatus. The cylinder is graduated from 0 to 50 cc. over a length of 11.25 in. for measuring the monomer absorption by the concrete.

The effect of the head on the monomer penetration into the concrete slab due to the height of the monomer and the high rate of absorption of monomer by concrete at the beginning of the test could affect the test results. The following procedure was used in performing the test:

1) The test should be performed at the same temperature for each slab. In the laboratory, tests were performed at a temperature of approximately 79° F.
2) To minimize the effect of high absorption of concrete at the beginning of the test, the first reading was taken 30 seconds after the liquid monomer began to be poured into the test apparatus.
3) To minimize the difference in hydraulic pressure on concrete surfaces, the liquid monomer was always at a reading between 0 and 2 cc. at the first reading. This is equivalent to the difference in a water head of approximately 1 cm.

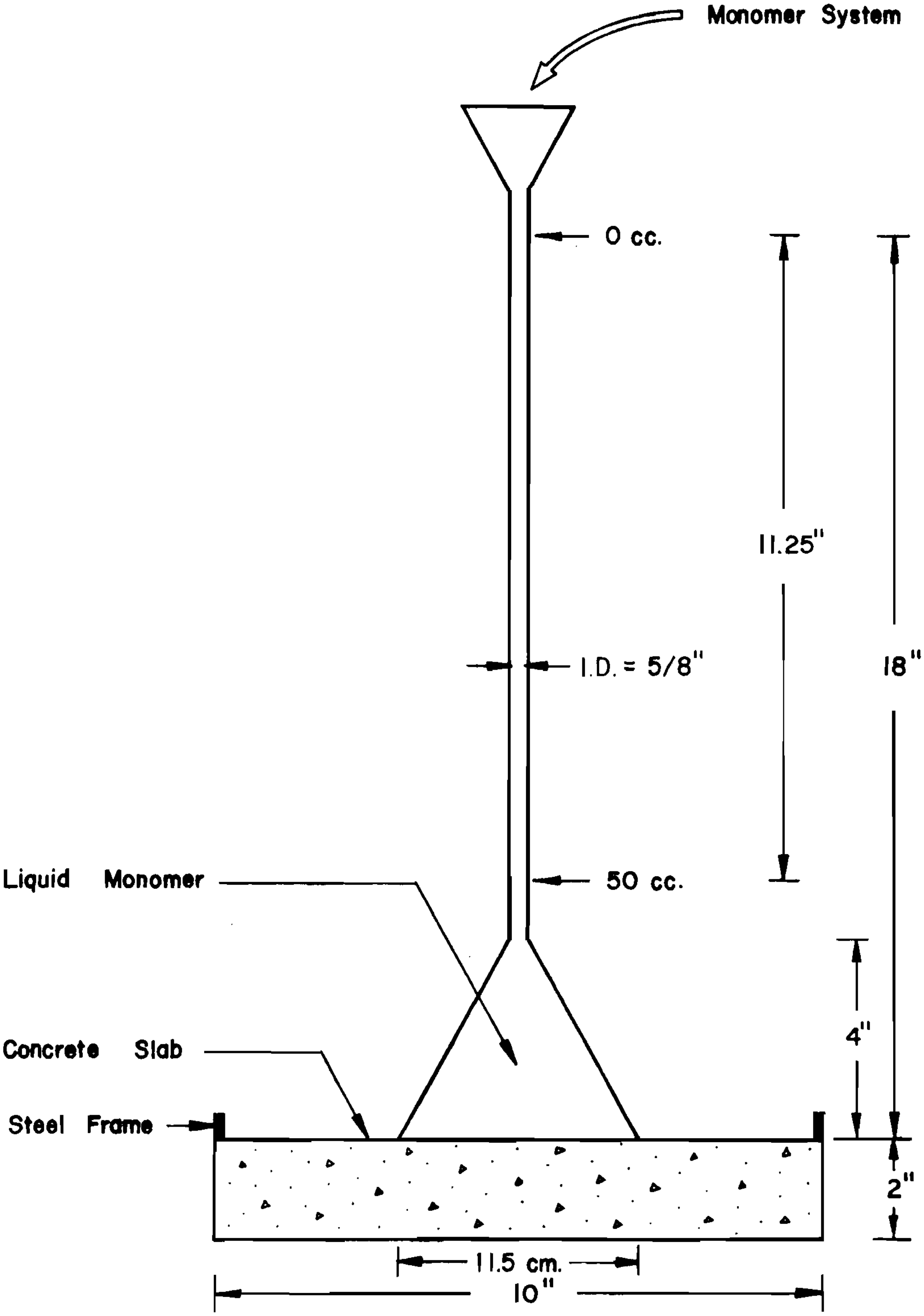


Fig. 4.1 Penetration Test Apparatus.

4.3.3 Effect of Relative Moisture Content. Tests were conducted on 10-in.x10-inx2-in. slabs with a water-cement ratio of 7.5 gal./sk.. The range of R.M.C. of test specimens was from 0 to 70 %. The tests were conducted for 2.5 hrs..

Fig. 4.2 shows the typical effects of R.M.C. on monomer absorption of concrete slabs with a water-cement ratio of 7.5 gal./sk.. The curves show this relationship at test periods of 30, 60, and 120 minutes.

From Fig. 4.2, it can be seen that the effect of R.M.C. on the monomer absorption increases as the soaking time increases. At 5 % R.M.C., the difference of monomer absorption between 30 and 120 minutes is approximately 24 cubic centimeters whereas at 60 % R.M.C. the difference is 12 cubic centimeters. It is also shown that specimens with high R.M.C. absorbed less monomer than low R.M.C. specimens. These test results agree with the fact that high R.M.C. specimens have more moisture inside the concrete to block the penetration of monomer.

Fig. 4.3 shows the relationship between the soaking time and the rate of penetration at various values of R.M.C.. Tests were conducted on the specimens with a water-cement ratio of 9.0 gal./sk.. The rate of monomer penetration is defined as the ratio between the amount of monomer absorption in cubic centimeters and the soaking time in minutes. It is observed that the rate of monomer penetration decreases as the soaking time increases. It is also seen that the

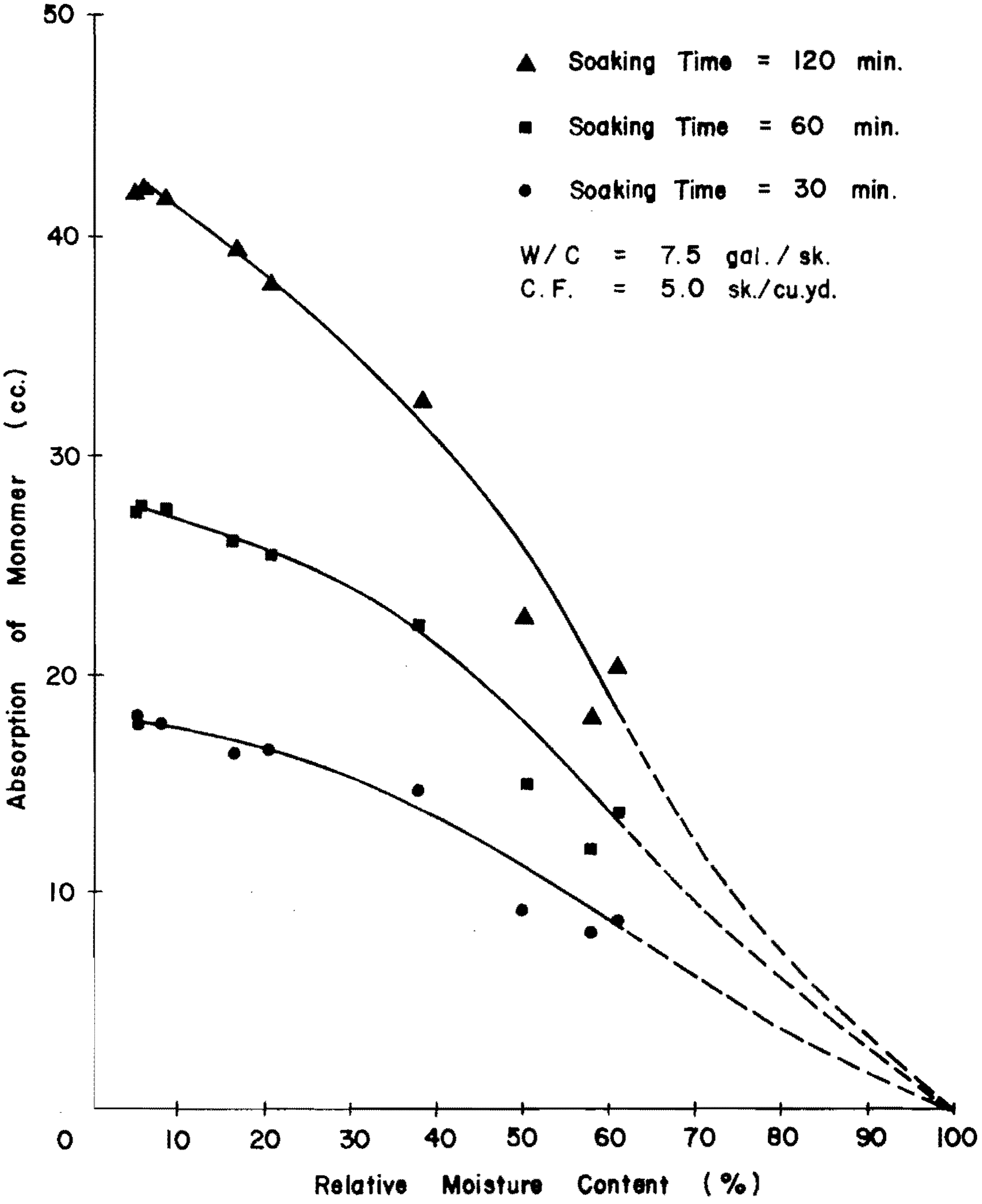


Fig. 4.2 Effect of Relative Moisture Content on Monomer Absorption.

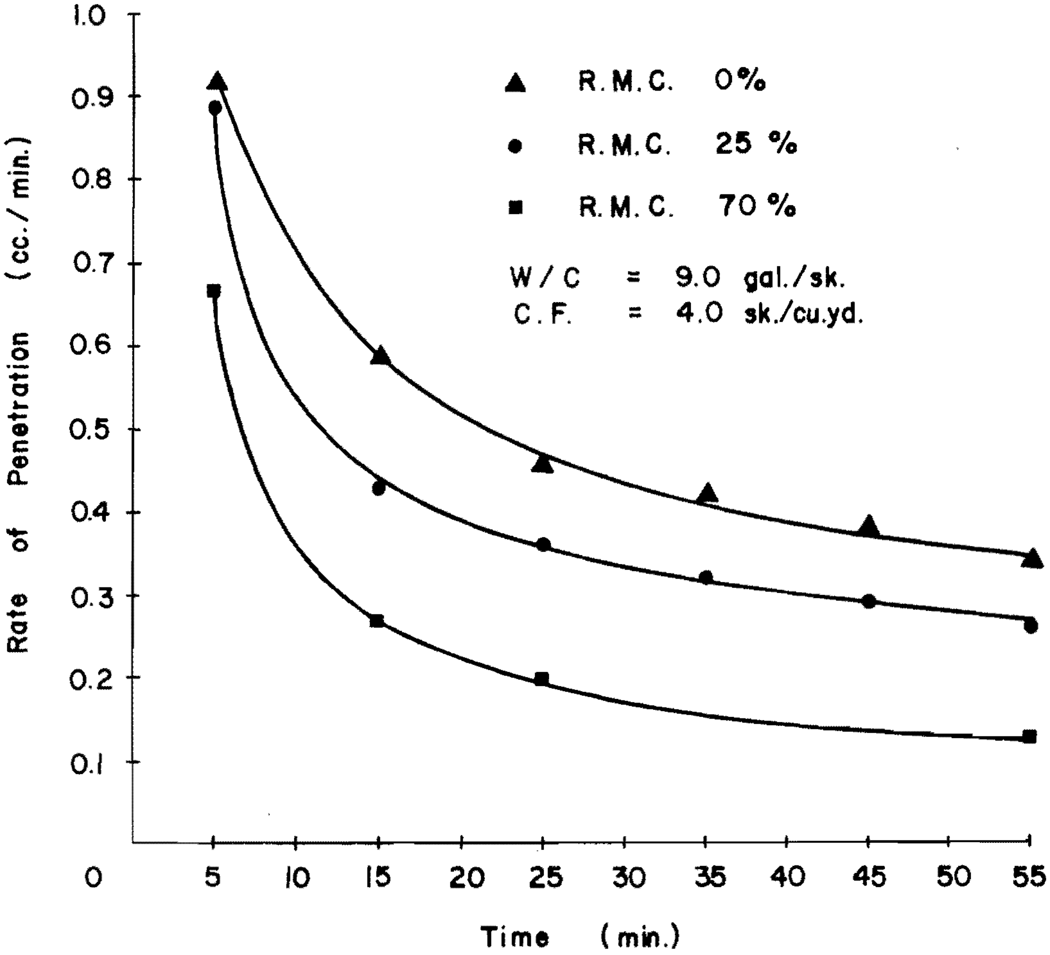


Fig. 4.3 Typical Relationship between Rate of Penetration and Time for Various Relative Moisture Contents.

rate of monomer penetration of high R.M.C. specimens is lower than the low R.M.C. specimens.

4.3.4 Effect of Water-Cement Ratio. To investigate the effect of concrete quality on monomer absorption, 10-in.x10-in.x2-in. slabs with water-cement ratios of 6.0,7.5, and,9.0 gal./sk., cement factors of 6.3,5.0, and 4.0 sk./cu.yd., were selected. To minimize the variables which may exist, tests were performed on completely dried specimens. The specimens were dried in the oven at 212°F for 7 days.

Fig. 4.4 shows the relationship between the soaking time and the absorption of monomer by the concretes. In general, lower quality concrete absorbed monomer faster than higher quality concrete. This result can be verified by the fact that the permeability of concrete increases as the water-cement ratio increases. At a 60-minute soaking time the concrete with water-cement ratios of 6.0, 7.5, and 9.0 gal./sk. absorbed 21, 27, and 30 cc. of monomer, respectively.

Fig. 4.5 shows the relationship between the soaking time and the rate of penetration of various types of concrete. It can be seen that the rate of monomer penetration increased as the water-cement ratio increased.

This method can be used to determine the absorbability of concrete prior to PIC surface treatment. The relationship between absorption, rate of penetration, concrete quality, time, and R.M.C. were previously determined. Although it may not seem to be absolutely

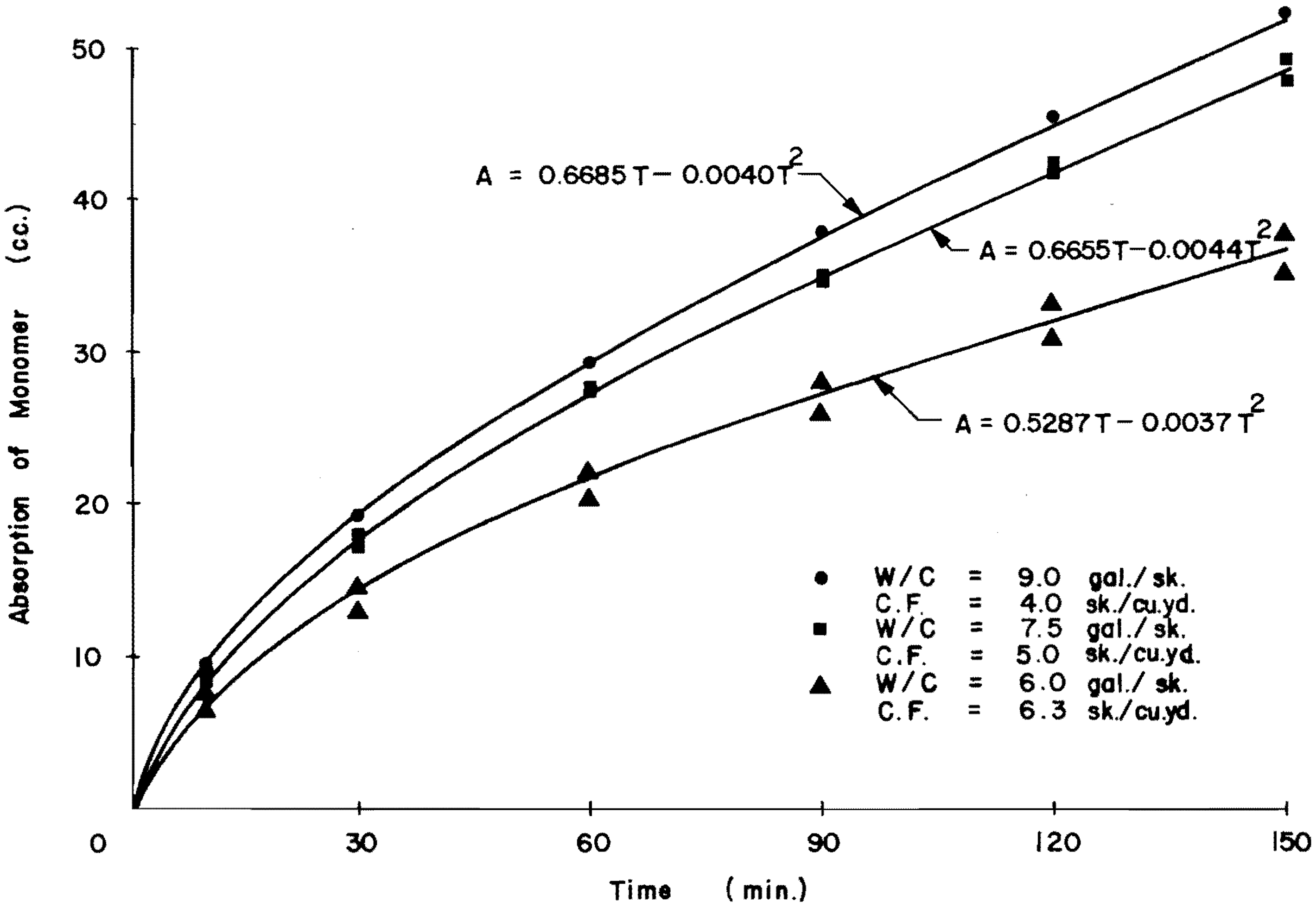


Fig. 4.4 Effect of Water-Cement Ratio on Absorption of Monomer for 2-in. Concrete Slab.

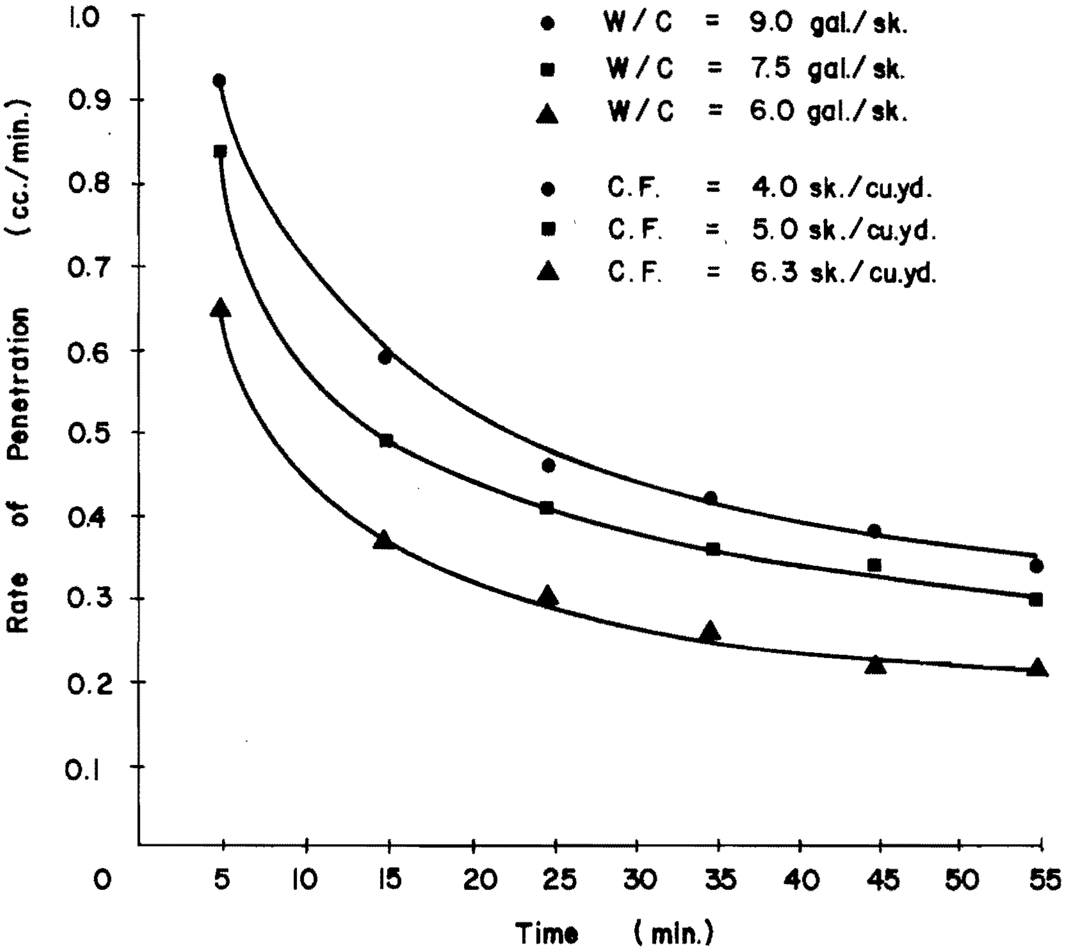


Fig. 4.5 Effect of Water-Cement Ratio on Rate of Penetration of Monomer.

the best method to determine the absorbability of concrete, at the present time this test method seems to be the best approach and can be performed if the variables are known. Because of the complex nature of the variables, such as concrete constituents, test temperature, concrete surface texture, and viscosity of monomer, this test method requires some knowledge of the concrete. It should be noted that water can not be used if the test is conducted prior to monomer application.

4.3.5 Determination of the Relationship between Monomer Absorption and Soaking Time. Attempts were made to use the polynomial regression technique to determine the relationship between the soaking time and the magnitude of monomer absorption. Based on the test data in this report, the following third degree polynomial regression equations were obtained:

For 6.0 gal./sk. concrete

$$A = 1.0972 + 0.5287T - 0.0037T^2 \; \ldots\ldots\ldots \; 4.1$$

$$R^2 = 0.9975$$

For 7.5 gal./sk.

$$A = 1.1867 + 0.6655T - 0.0044T^2 \; \ldots\ldots\ldots \; 4.2$$

$$R^2 = 0.9984$$

For 9.0 gal./sk.

$$A = 1.7551 + 0.6685T - 0.0040T^2 \; \ldots\ldots\ldots \; 4.3$$

$$R^2 = 0.9979$$

Where

A = monomer absorption, cc.;

T = soaking time, minutes;

R = coefficient of correlation.

It should be noted that the third degree terms of all the equations approach zero within 4 significant digits. All of these equations also give very high correlation coefficients. Based on observations of curve-fit and the nature of the tests, the constant in each equation can be considered to be negligible. With a soaking time between 15 to 150 minutes, the following equations are recommended:

For 6.0 gal./sk.

$$A = 0.5287T - 0.0037T^2 \quad \ldots\ldots \quad 4.4$$

For 7.5 gal./sk.

$$A = 0.6655T - 0.0044T^2 \quad \ldots\ldots \quad 4.5$$

For 9.0 gal./sk.

$$A = 0.6685T - 0.0040T^2 \quad \ldots\ldots \quad 4.6$$

These equations are plotted in Fig. 4.4.

4.4 Optimization of Soaking Time

The tests were performed on two types of specimens: 3 x 6-in. cylinders and 10-in.x10-in.x6-in. concrete slabs. Other variables, such as the quality of concrete and type of specimens, were taken into consideration. The concrete with water-cement ratios of 6.0, 7.5, 8.0, 9.0 gal./sk. were selected. The specimens were completely dried in the oven at an oven temperature of 212°F for 7 days. After the specimens cooled to room temperature, they were soaked by a monomer solution of MMA, 1 % BP, and 10 % TMPTMA from 1 to 8 hrs. in increments of 1 hr. To simulate the actual bridge deck treatment, the surface of specimens was covered with 0.25 in. of sand. The sand was kept in a moist-to-saturated condition throughout the soaking period. During the soaking period, the surface was covered with polyethylene to minimize evaporation of monomer. After soaking, the specimens were cured by steam for 1 hr. and broken apart to measure the polymer depths.

The cylinder specimens with a water-cement ratio of 9.0 gal./sk. were coated on all sides with epoxy to simulate the interior conditions of the bridge deck. The specimens were soaked and cured as usual.

Fig. 4.6 shows the relationship between the soaking time and the polymer depth for all types and shapes of specimens. It can be seen that the effect of concrete quality on the relationship of soaking time and polymer depth is relatively small. Observations show that the coated specimens gave the same results as the uncoated specimens. From Fig. 4.6 it can also be seen that the effect of soaking time on polymer depth is more significant for short periods of soaking. Polymer depths of approximately 1 in. were obtained for a soaking time of 4 hrs. whereas an 8-hr. soaking time produced approximately 1.4 in. of polymer depth. The polymer depth of approximately 2.0 in. required a 17-hr. soak.

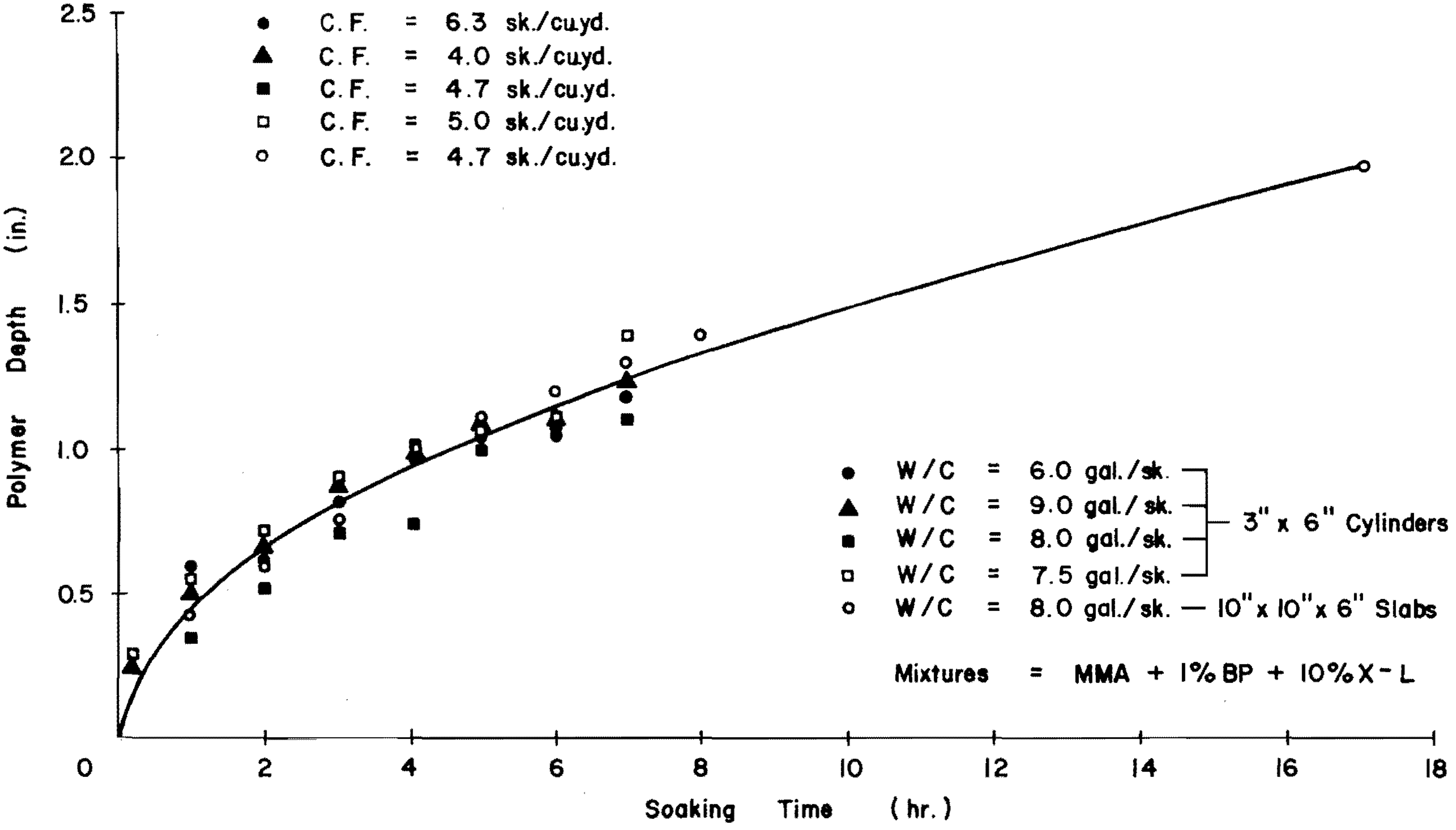


Fig. 4.6 Effect of Soaking Time on Polymer Depths.

Previous research has found that a straight line correlation exists between monomer depth and the square root of the soaking time, $\sqrt{T}$ (Ref. 62). The polymer depths were plotted against $\sqrt{T}$, in Fig. 4.7, and indicate a straight line relationship which is in agreement with Ref. 62. Attempts were made to determine the relationship between polymer depth and $\sqrt{T}$. The polynomial regression analysis was used. Based on the boundary condition of the tests and soaking time equal to or less than 8 hrs., the following equation was obtained:

$$D = 0.03733 + 0.05859\sqrt{T} \qquad 4.7$$

$$R^2 = 0.9611$$

Where

D = polymer depth, in.

Because of the nature of measurement of polymer depth, which was previously described, the constant in Eq. 4.7 can be neglected, which results in the following equation:

$$D = 0.059\sqrt{T} \qquad 4.8$$

4.5 Effect of Relative Moisture Content on Polymer Depth

Relative moisture content of the specimens prior to monomer application is the most important factor affecting the polymer loading for partial impregnation. To determine the effect of R.M.C. on polymer depth, the polymer partial impregnation tests were conducted on the concrete slabs with various R.M.C. Two and 6-in. thick concrete slabs

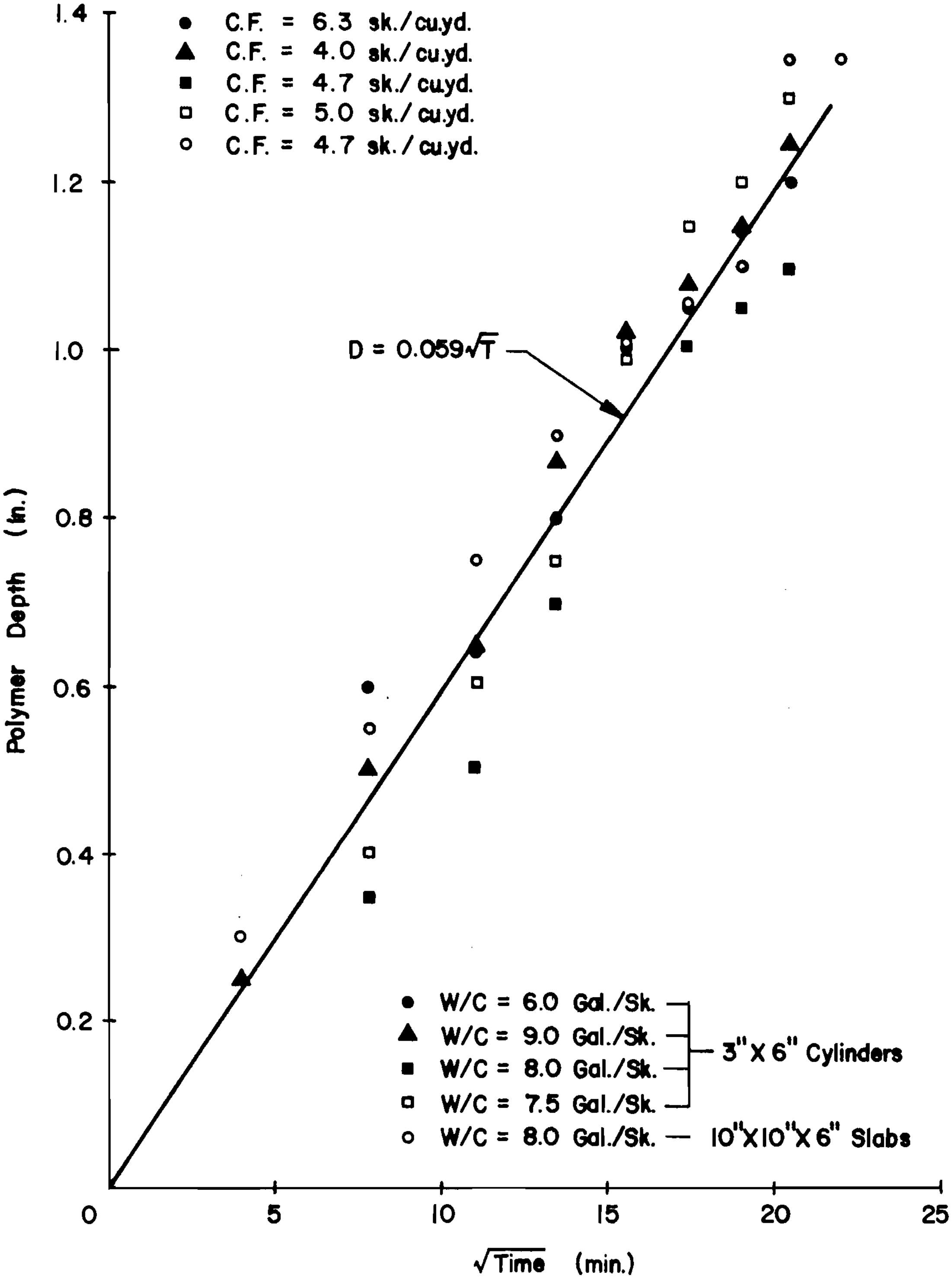


Fig. 4.7 Polymer Depth as a Function of Square Root of Soaking Time.

were selected for the test specimens. The specimens were dried in the oven at 200°F for 10 days. They were immersed in water for 7 days to obtain the saturated surface dried weight. After soaking, the specimens were placed in the oven and taken from the oven periodically to obtain various R.M.C. Determination of the relationship between the monomer absorption and R.M.C. was made as previously discussed. After the monomer absorption tests, the specimens were soaked by a solution of MMA for 17 hrs. The relative moisture content of the 10 x 10 x 2 in. specimens varied from 0 to 85 %. The tests were conducted on 44 specimens and divided into 4 groups based on the quality of concrete. The water-cement ratios of the concrete specimens were 6.0, 7.5, 8.0, and 9.0 gal./sk..

After soaking, the specimens were cured in a steam chamber and broken apart to measure the polymer depths. Fig. 4.8 shows the relationship between the R.M.C. and the polymer depth of 2-in. thick slabs with various types of concrete qualities. It is shown that at 17 hrs. of soaking, the effect of concrete quality on polymer depth is not significant. The R.M.C. is the very significant factor affecting the polymer depths. A polymer impregnation depth of 0.25 in. was achieved for the specimens with a R.M.C. of 30 % . With a 10 % R.M.C. the specimens achieved approximately 1 in. polymer depths. It can be seen that the most significant effect of R.M.C. on polymer depth occurs between zero and 25 % relative moisture content.

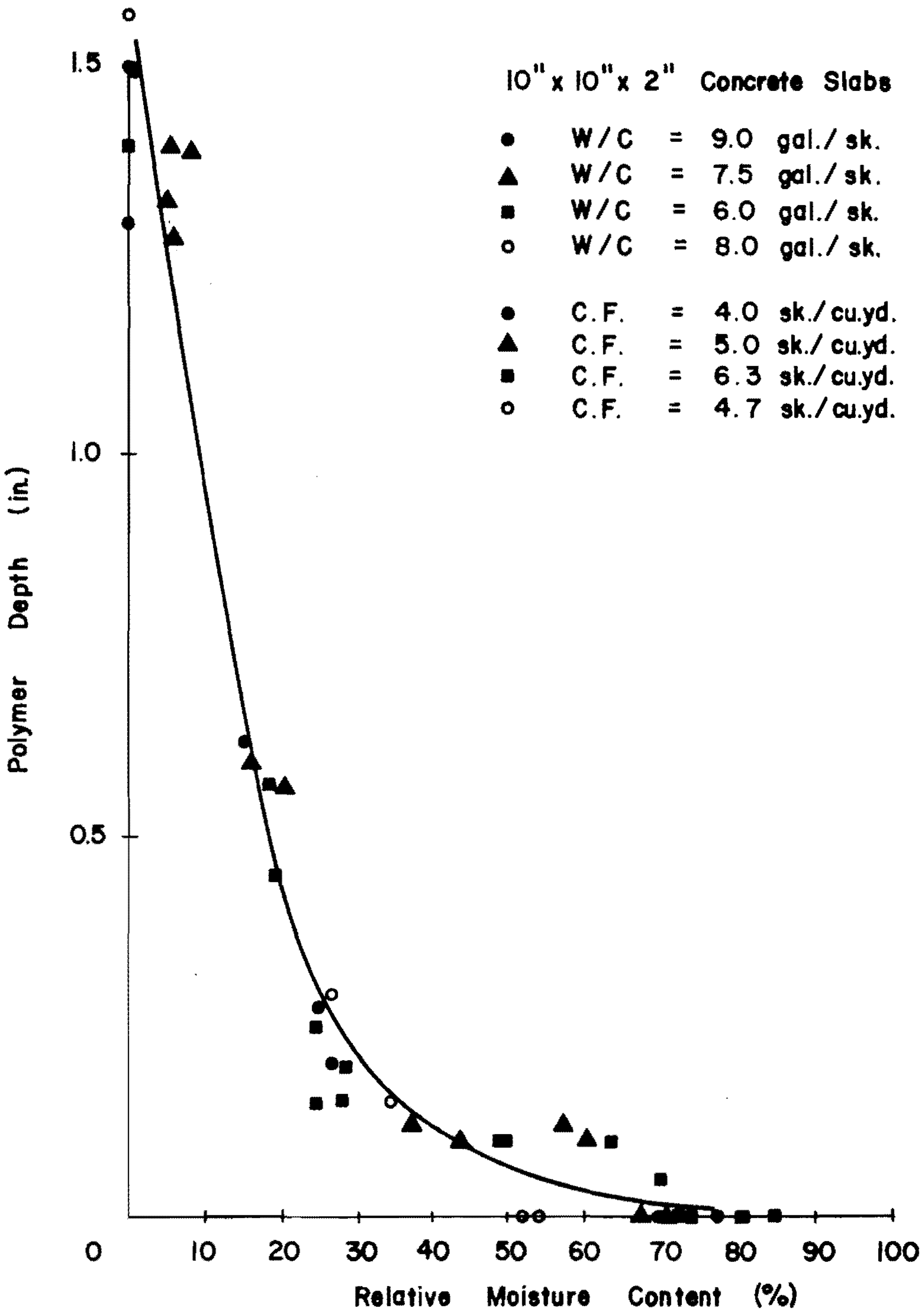


Fig. 4.8 Effect of Relative Moisture Content on Polymer Depth of Surface Treatment in 2-in. Concrete Slabs.

To determine the effect of R.M.C. on the polymer depth of 6-in. concrete slabs, fifteen 10-in.x10-in.x6-in. slabs with a water-cement ratio of 6.5 gal./sk. were tested identically as the 2-in. slabs. The R.M.C. of the tested specimens varies from zero to 57 %. Fig. 4.9 shows the effect of R.M.C. on partial impregnation of polymer concrete of 6-in. concrete slabs. It can be seen that with this range of R.M.C., the relationship between polymer depths and R.M.C. is almost a straight line. The effect of R.M.C. on polymer depth in 6-in. concrete slabs is not as significant as in 2-in. concrete slabs. From Fig. 4.9, the specimen with the R.M.C. of 40 % achieved approximately 0.6-in. polymer depth whereas only 0.2 in. of polymer depth was achieved in the 2-in. concrete slab. At the R.M.C. of 20 %, the polymer depths of 1.5 in. and 0.5 in. were achieved for 6 and 2-in. slabs, respectively. At dried condition the 6-in. slab developed approximately 2-in. polymer depth.

Attempts were made to determine equations for predicting the polymer depth, as a function of R.M.C., in both 2 and 6-in. slabs. In 2-in. slabs, a careful study of the data was made and it was found that the bilinear relationship seems to best fit the data. Therefore the input data for regression analysis was divided into two sections, based on the R.M.C., as follows:

a) $0\% \leq \text{R.M.C.} \leq 30\%$

b) $30\% \leq \text{R.M.C.} \leq 90\%$

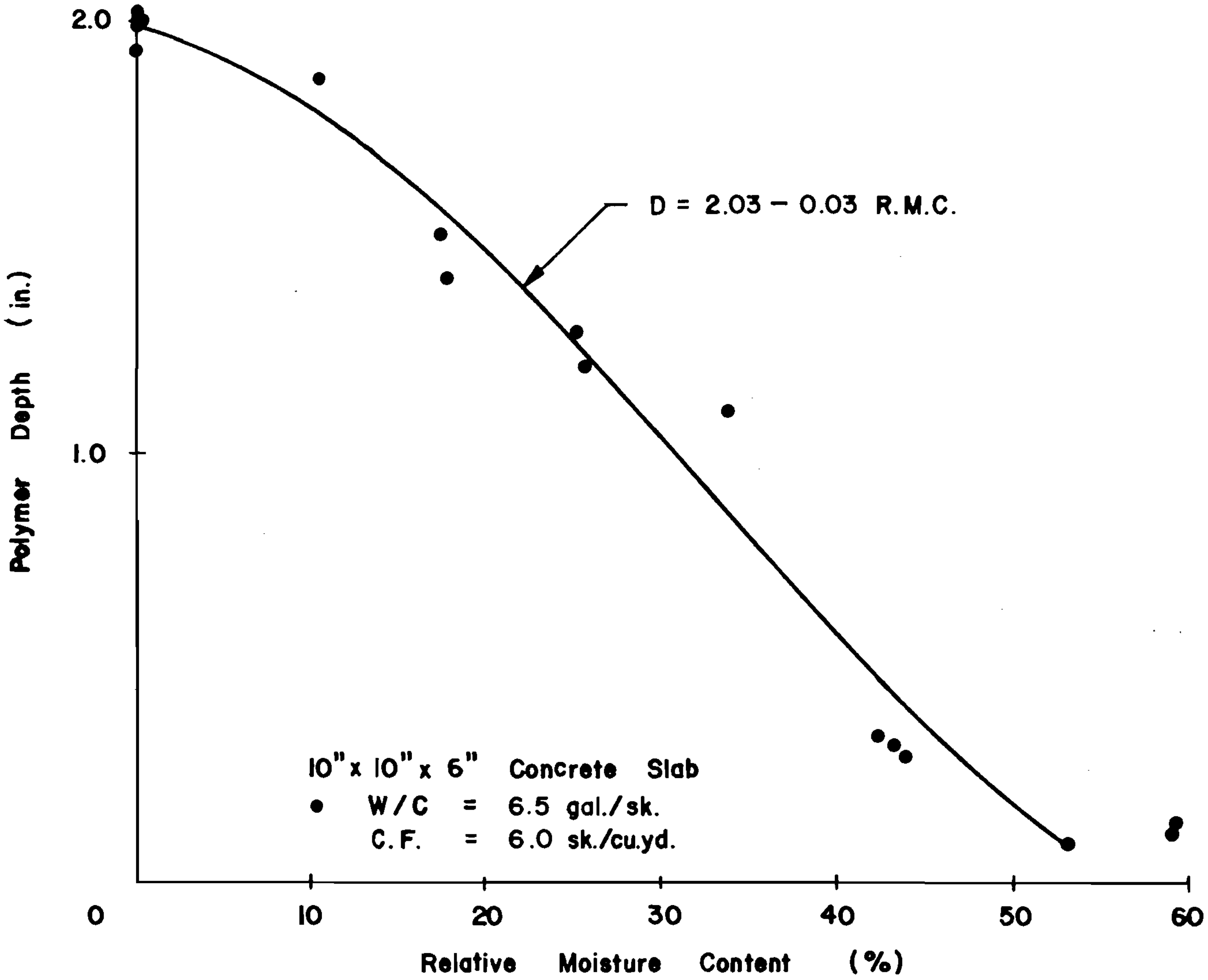


Fig. 4.9 Effect of Relative Moisture Content on Polymer Depth of Surface Treatment in 6-in. Concrete Slabs.

Based on the data boundary conditions in this report, the following equations were obtained:

1) 2-in. slab:

$0\% \leq \text{R.M.C.} \leq 30\%$

$D = 1.5156 - 0.0491\ (\text{R.M.C.})$ 4.9

$R^2 = 0.956$

$30\% \leq \text{R.M.C.} \leq 90\%$

$D = 0.3315 - 0.0066\ (\text{R.M.C.})$ 4.10

$R^2 = 0.553$

2) 6-in. slab:

$0\% \leq \text{R.M.C.} \leq 60\%$

$D = 2.0268 - 0.0339\ (\text{R.M.C.})$ 4.11

$R^2 = 0.966$

Because of the variations in measurements of polymer depth, Eq. 4.10 seems to be insignificant and may be neglected. Thus the following equations are recommended:

2-in. slab, $0\% \leq \text{R.M.C.} \leq 30\%$

$D = 1.52 - 0.05\ (\text{R.M.C.})$ 4.12

6-in. slab, $0\% \leq \text{R.M.C.} \leq 60\%$

$D = 2.03 - 0.03\ (\text{R.M.C.})$ 4.13

Eq. 4.13 is also plotted in Fig. 4.9.

4.6 Summary

From test results,it may be concluded that:

1) Relative moisture content of the specimens prior to monomer application is a very significant factor affecting penetration of monomer into the concrete.
2) Under the penetration test conditions and within a soaking time of 2.5 hrs. or less,lower quality concrete absorbed more monomer than higher quality concrete.
3) The rate of monomer penetration decreases as the soaking time increases.
4) With sufficient drying, 1 in. of polymer depth can be achieved with a soaking time of 4 hrs. or more.
5) The effect of relative moisture contents on polymer depths of 2-in. concrete slab is more significant than of 6-in. concrete slabs.
6) Because of the complex nature of variables in concrete, there is no quick reliable method to measure the absorbability of concrete.

CHAPTER 5

POLYMERIZATION OF MONOMER IN CONCRETE

5.1 Introduction

Most of the previous laboratory tests on polymerization or curing techniques have been on fully-impregnated concrete (PIC). Experiments with PIC are usually performed under laboratory conditions and complicated polymerization techniques can be accomplished without serious problems. However, the processes of polymerization of monomer in laboratory tests such as radiation or oven curing are not feasible for surface treatments in field conditions. Polymerizing the monomer in concrete bridge decks involves many complex variables, such as the nature of bridge deck structure, environmental conditions, rate of evaporation and safety, which influence selection of the methods of curing. In general, the two most common curing methods have been irradiation or thermal-catalytic.

Polymerization of the monomer requires free radicals to initiate the reaction. The free radical is generated by the decomposition of the peroxide or other initiator. To increase the rate of polymerization the TMPTMA (cross-linking agent) and the heat source were applied to the system. In general, increasing the temperature during the curing period accelerates the polymerization process. For a given monomer there are many factors which can influence the polymerization of monomer in concrete. These factors are: (Ref. 3)

1) type and amount of initiator,
2) type and amount of cross-linking agent,
3) type and amount of accelerator, and
4) temperature.

It is believed that the polymerization process of the monomer alone and the monomer inside the concrete slabs is somewhat different. Many experiments were performed to illustrate the effects of curing time and curing temperature on polymerizing of monomer inside the concrete slabs. Throughout the experiments, the mixture of MMA with 1 % (wt.) BP and 10 % (wt.) TMPTMA was used.

5.2 Scope

An investigation of previous research was made with the goal of finding the proper curing method to use for surface impregnation of concrete.

Because of the considerable safety hazards of radiation curing in field condition, only thermal-catalytic methods were used. These experiments had the general objective of finding the minimum curing time and temperature for surface treatments. Steam was used as the heat source in the curing system.

Surface impregnation of concrete involves large surface areas of concrete to be soaked with monomer. Evaporation of monomer on top of the concrete slabs may occur during soaking and curing periods. These reasons led to the investigation and studies of the effect of evaporation barrier on the temperature increase on the surface of the treated slab during the curing process.

5.3 Reviews of Previous Studies

Although it is relatively easy to produce good polymer loading for full-impregnation with the aid of a vacuum chamber, it is much more difficult to impregnate and polymerize the monomer in surface treatments at atmospheric pressure. The difficulty with polymer surface treatments can be divided into three major catagories as follow:

1) adequate monomer penetration,
2) monomer evaporation and run-off during polymerization, and
3) polymerization methods.

The relationship between the soaking time and polymer depth was discussed in chapter 4. In the preliminary phase of this chapter, many possible curing methods were discussed and related to current studies.

Oven curing - Oven curing seems to be the convenient method to cure the monomer inside the concrete in the laboratory, but the feasibility of its use in the field is considered impractical. In the field, using circulated hot air, which can be produced by forced-air heaters, for curing is feasible if there is no open flame. This method has been used to impregnate a bridge deck in Colorado.

Electric heating blanket - The disadvantage of electric heating blankets is the length of time required to build up the temperature to that required for curing. It often took more than 10 hrs. to raise the concrete slab temperature from 75°F to 125°F (Ref. 5). This slow temperature rise causes a considerable amount of monomer to evaporate before polymerization can be achieved. There is also the possibility of using solar energy for curing. However, solar heating is dependent on several variables which cannot be controlled, such as, the air temperature, cloud cover and humidity. These variable factors make it difficult to use solar energy as the curing method. Future developments in solar energy might render this method practical, however.

Reactive second treatments - The reactive second treatment was used as the curing method for polymer surface impregnation. Early experiments found that the reactive second treatment generates enough heat to induce polymerization at room temperature. A reactive monomer system consisting of MMA, 4 % (wt.) lauroyl peroxide (LP), and 4 % (wt.) of N, N-dimethyl-p-toluidine (DMPT) was found to polymerize without the addition of external heat. This solution, when ponded on the surface of the specimens already saturated with MMA, BP, and TMPTMA achieved polymerization within 2 hrs. (Ref. 3). In some cases, up to 0.75 in. of polymer depth was obtained and in addition, the sand cover was bonded to the surface. However, using the reactive second treatment gives inconsistent results and usually results in only a shallow depth of polymer in the concrete slab. In most current investigations additional heat sources are applied for curing even when the second treatment is used.

Hot water - Hot water seems to give very satisfactory results for curing. It can produce sufficiently high temperatures to cure the monomer in a short period of time. The depth of hot water ponded on the surface of the concrete is the most important factor in curing the monomer if the water is not reheated during curing. Three inches of hot water initially at 200°F induced a maximum temperature on the surface of concrete slabs of approximately 140°F (Ref. 5). The slab surface was initially at ambient temperatures of 50°F to 90°F. However, bridge decks with steep grades or super elevations would be difficult to heat with hot water unless a closed bag were used.

Steam - Steam seems to be the most practical and appropriate method to cure monomer in field applications. Previous tests indicated that steam curing is capable of generating slab temperatures in excess of 200°F in a short period of time. To use steam, an enclosure made of insulating material is placed over the slab to reduce the heat loss. Steam is supplied to the enclosed space to cure the monomer inside the concrete slab. Steam cannot be applied directly to the concrete slab because of the evaporation loss of the monomer (Ref. 14).

5.4 Curing Temperature and Length of Curing Time

To determine the relationship between the temperature on the surface of treated slabs and the curing time required, four 10-in.x10-in.x 6-in. concrete slabs with thermocouples attached to the surface were dried to constant weight and soaked with the usual MMA mixture for 4 and 21 hrs. After soaking, the specimens were cured by steam in the steam chamber and the temperatures on the slab surfaces were monitored for a period of 1 hr.

Fig. 5.1 shows the test results. It can be seen that the slab temperature could be increased to 200°F in approximately 20 minutes. Polymer depths of 1 and 2 in. were obtained in slabs that soaked for 4 and 21 hrs., respectively.

To determine the effects of curing temperature and length of curing time on polymer depths, nine 10-in.x10-in.x6-in. slabs were dried and treated as usual. Experiments were divided into three series according to the curing temperatures. An untreated specimen with the thermocouples attached to the surface and at various depths was also placed in the steam chamber to measure the curing temperature. The specimens were cured at temperatures (on the surface of controlled specimen) of 130, 150, and 170°F. Each curing temperature study consisted of three specimens cured for 30, 60, and 90 minutes. The temperature gradients in the 6-in. concrete slab during the curing period were also measured.

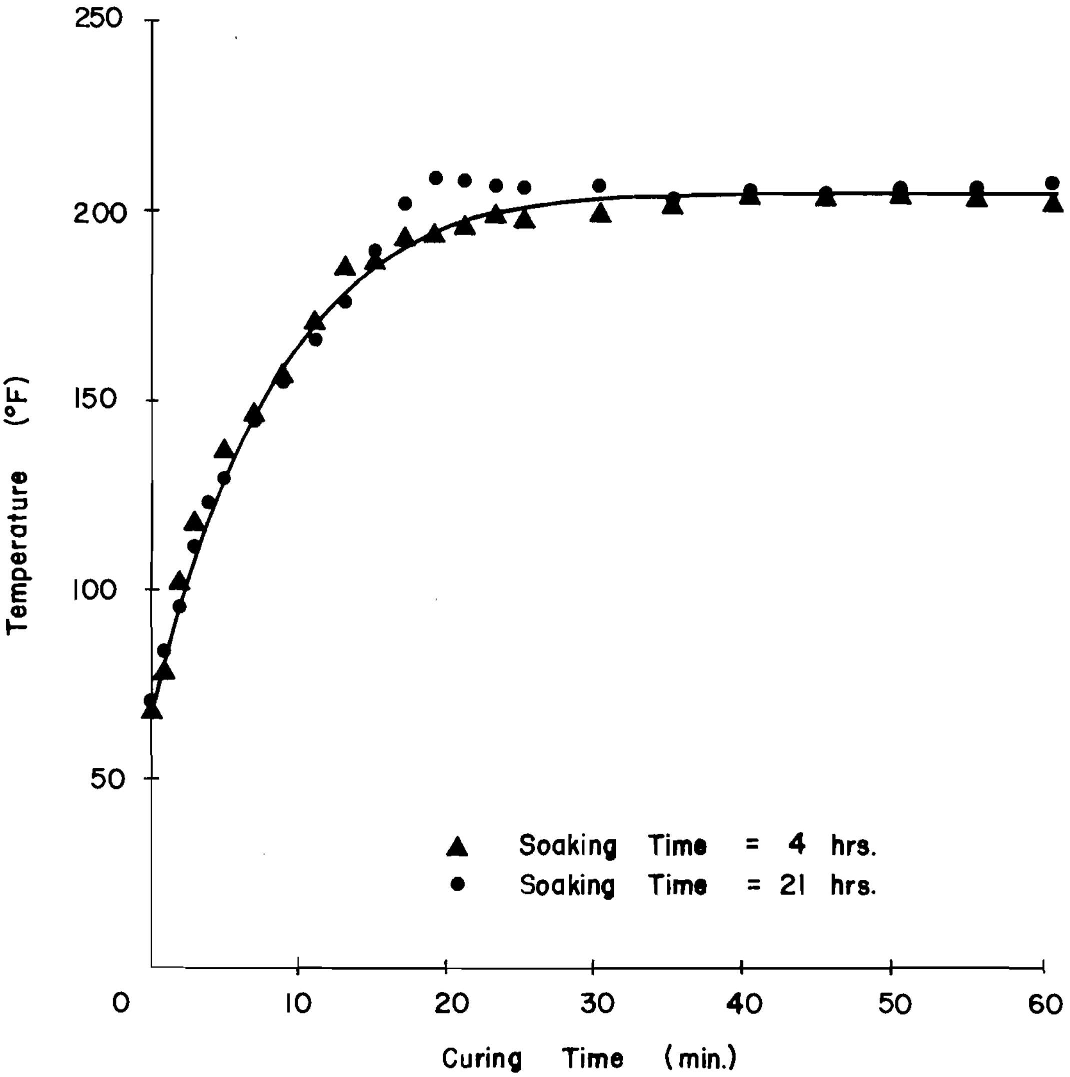


Fig. 5.1 Temperature at the Surface of PIC Slabs During Steam Cure.

After curing, each specimen was broken apart to measure the polymer depths. Fig. 5.2 indicates the polymer depths for different application times of steam and curing temperatures. At a curing temperature of 130° F, there was no appearance of polymer for any of the curing times. At curing temperatures of 150 and 170° F, polymer depths of approximately 1.5 and 1.65 in. were obtained. The effect of the three curing times on polymer depth was not significant. The 30 minute curing time was about as effective as the longer times.

It can be seen from Fig. 5.2 that as the curing temperature ranged from 130° F to 200° F, the deeper polymer depths were obtained with higher curing temperatures. At a curing temperature of 150° F, the maximum polymer depth of 1.5 in. was achieved as compared to 2.0 in. for curing temperature of 200° F. This result was believed to be caused by the higher rate of polymerization of the monomer in concrete at high curing temperature.

Fig. 5.3 indicates the temperatures at 0.8 in. from the top surface of the untreated specimen during curing. To reach a minimum curing temperature of 125° F (Ref. 3) at this depth, curing times of 80, 20, and 19 minutes were required for the surface temperature of 130, 150, and 170° F, respectively. From this result it could be seen that at 130° F, the curing time of 90 minutes was not enough to achieve polymerization because of the low rate of temperature rise in concrete. However, it was believed that if a longer period of curing time was allowed, the polymerization of monomer could be achieved if the monomer

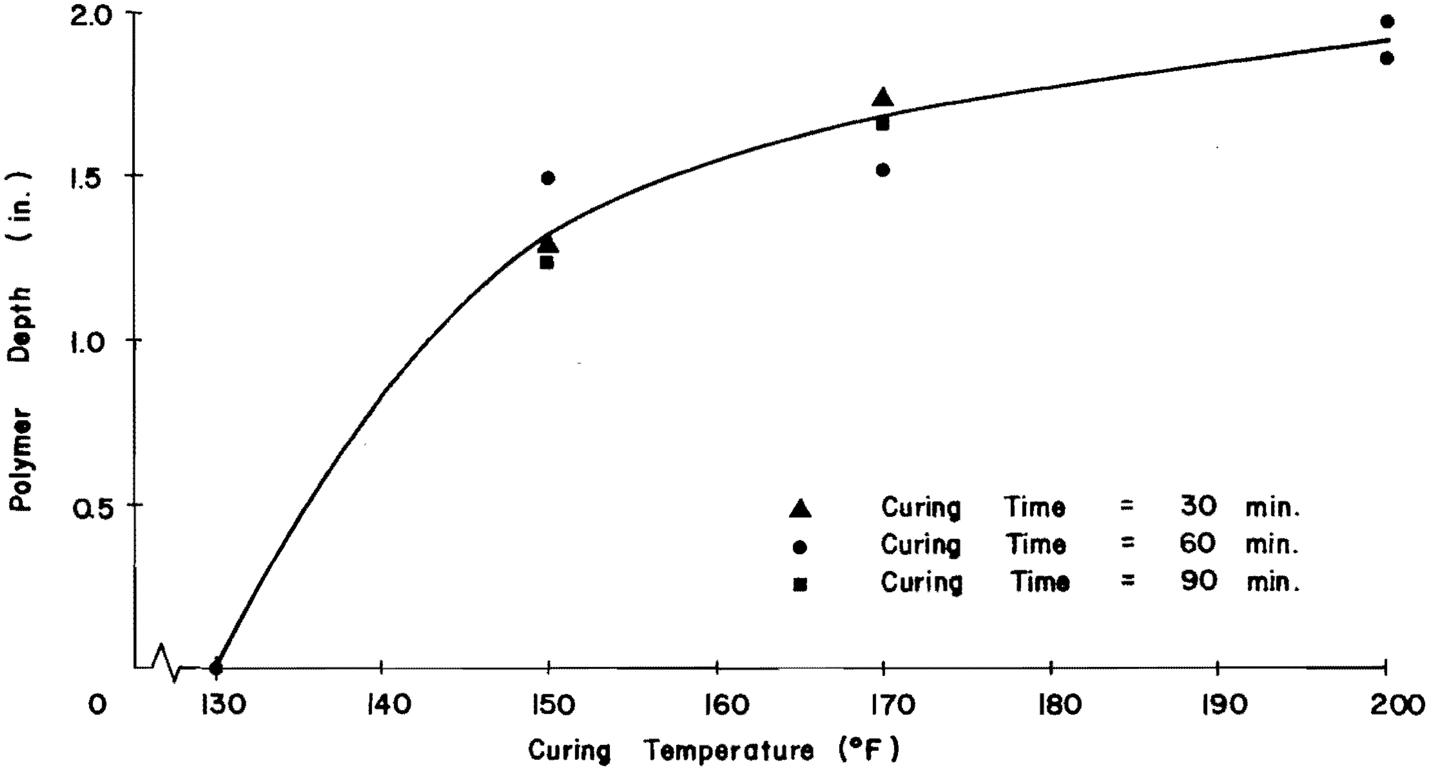


Fig. 5.2 Effect of Curing Temperature and Length of Curing Times on Polymer Depths.

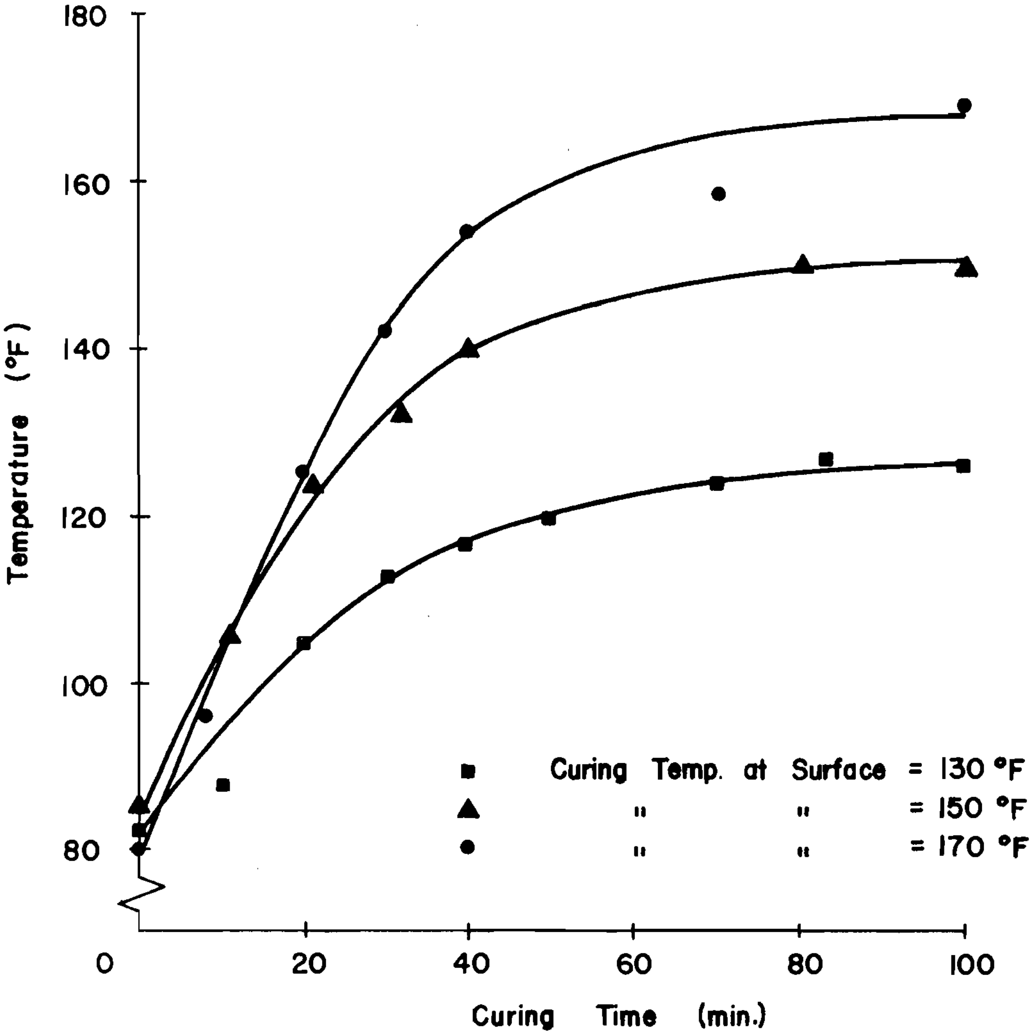


Fig. 5.3 Temperatures at 0.8 in. from the Surface of Untreated Slab During Curing Process.

was protected from evaporation. At 150 and 170°F, the temperature of 125°F was reached within 20 minutes of curing time and polymerization was achieved.

As previously discussed, polymerization of monomer is a highly exothermic reaction. For partial polymer-impregnation, after the polymerization was initiated by the external heat source, the exotherm was developed and caused the rate of polymerization to increase. Thus in general, the effect of curing time seems to be less significant after polymerization is initiated. For this reason, with sufficient curing temperature, a curing time of 30 minutes was about as effective as the longer times.

5.5 Evaporation Barrier

Previous research has shown that evaporation of the monomer on the top surface of a concrete slab may occur during polymerization. Another serious problem is that water, which is created by the condensation of steam, may soak the fine aggregate prior to polymerization of the monomer and cause difficulties in building up the temperature to the required curing temperature. To minimize these problems, the technique of covering the surface of the treated slabs prior to polymerization was selected.

To evaluate the effect of evaporation barrier on polymer depths, three pairs of 10-in.x10-in.x6-in. concrete slabs with the thermocouples attached to the surface were soaked with the regular mixture of MMA, BP, and TMPTMA for periods of 2, 4, and 21 hrs.. After the specimens were soaked, one specimen from each pair was covered with a polyethylene sheet to study the effect of the evaporation barrier on temperature rise in concrete during the curing period. These specimens were placed in the steam chamber to cure for 1 hr. and the temperatures of the covered and uncovered specimens were monitored periodically.

Fig. 5.4 shows the comparison of surface temperatures on the surfaces of treated slabs during curing of covered and uncovered slabs with various soaking time. It is apparent that with sufficient steam during curing, there is practically no difference in the temperature rise at the surface between covered and uncovered slabs.

Fig. 5.5 shows the relationship between the soaking time and polymer depth for covered and uncovered slabs. It indicates that there is very little difference in polymer depths for covered and uncovered slabs with equal soaking time. It was found that the covered slabs had less evaporation on the top surface than uncovered specimens after curing.

It should be noted that the curing process for partial impregnation of concrete should be taken to completion to avoid the effect of unpolymerized monomer on the concrete. The presence of monomer may affect the long term durability of concrete which is partially dried prior to impregnation (Ref. 64). Cracking of partially dried mortar specimens, after immersion in MMA for 5 days,

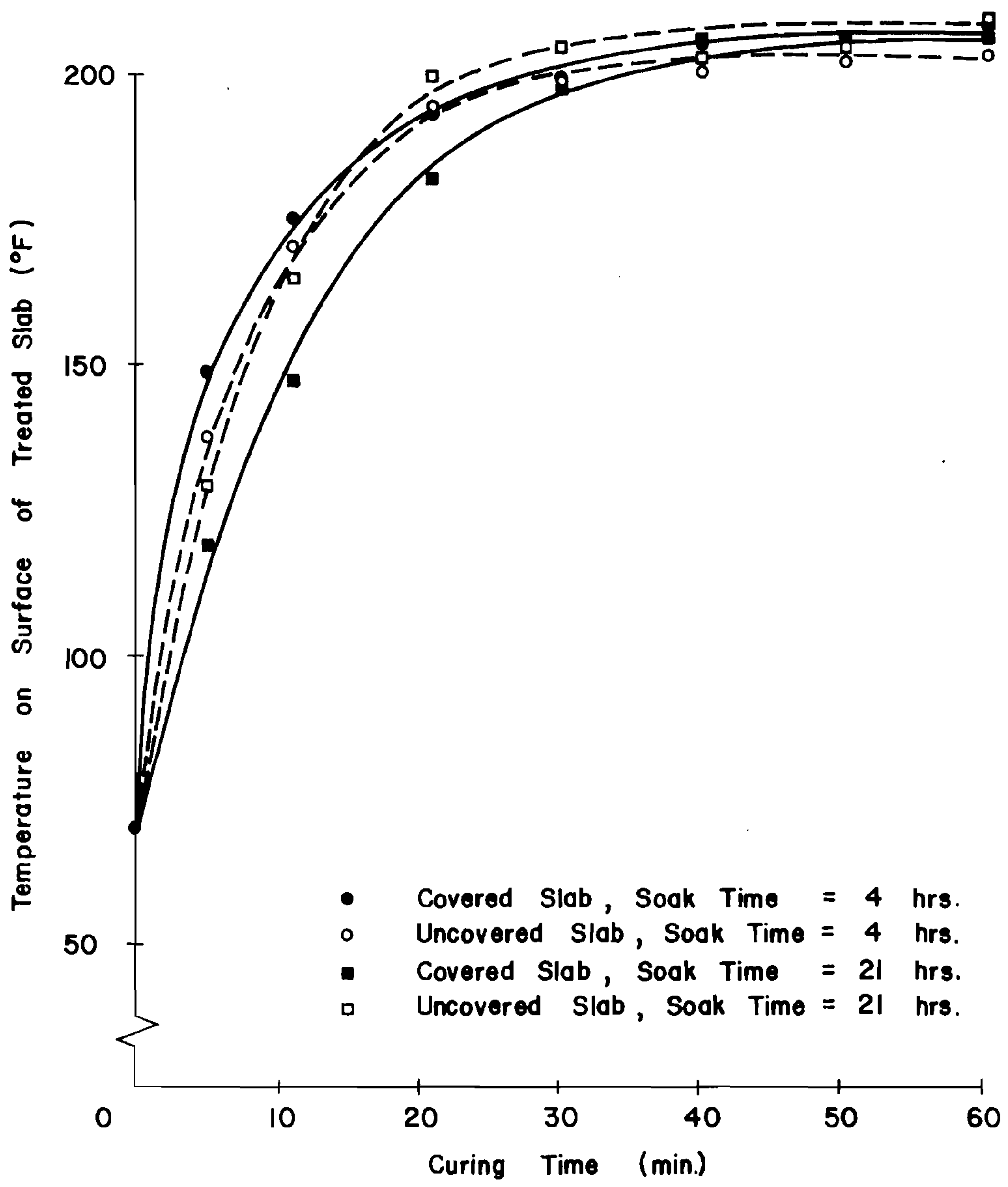


Fig. 5.4 Effect of Evaporation Barrier on Temperature Buildup on the Surfaces of Treated Slabs.

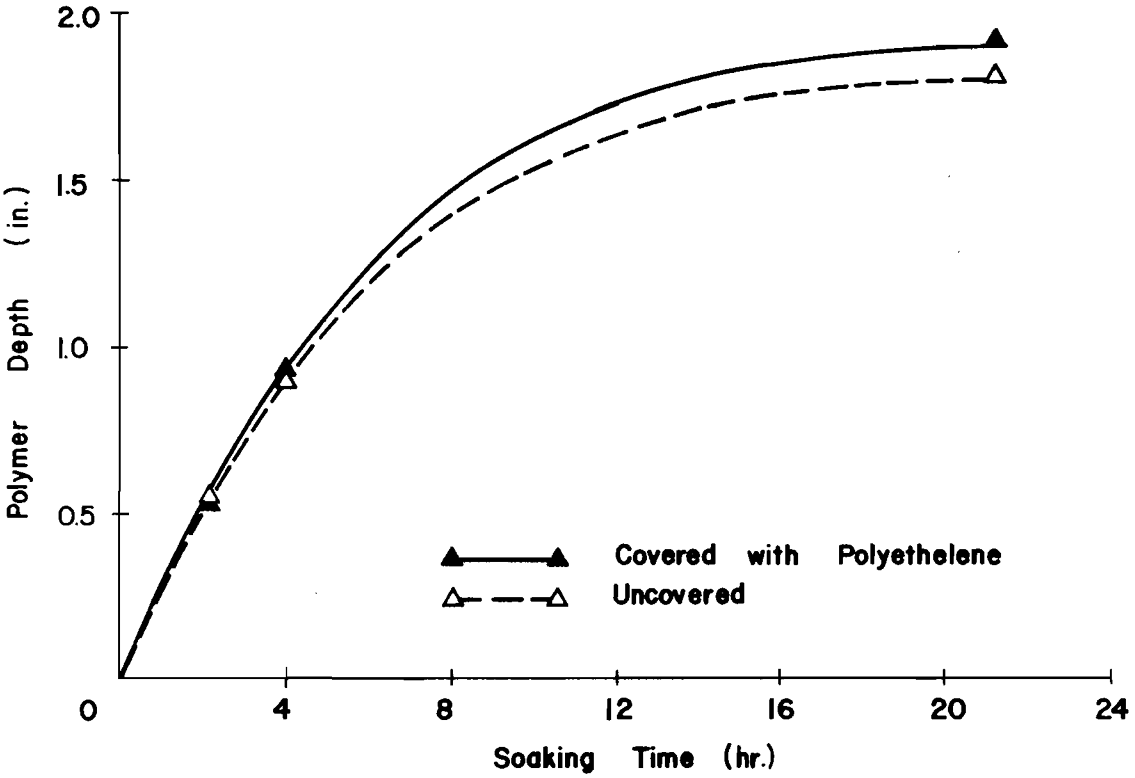


Fig. 5.5 Effect of Evaporation Barrier on Polymer Depths.

has been reported (Ref. 17). Work performed by Manning and Hope (Ref. 64) also indicated both swelling and cracking of mortar specimens which were soaked with MMA. The most severe cracking occured in the sample dried to 58 % R.M.C.. However, this problem is believed to be less significant with PIC. In the work performed at The University of Texas, no cracks were ever observed in concrete specimens,either after soaking or after polymerization.

5.6 Summary

1) Steam is likely to be the most practical curing method for polymer-impregnated surface treatments although other methods are satisfactory.
2) With the regular MMA monomer system, a minimum curing temperature of 150°F on the surface is required for polymerization.
3) With adequate curing temperature, the MMA monomer system requires a curing time of not less than 30 minutes.
4) The specimens should be covered with a protective membrane during curing to prevent the intrusion of condensed water and evaporation of monomer during the curing process.
5) The temperature rise on the surface of concrete should be monitored during curing. In general, polymerization of the monomer can be detected by observations of the temperature rise.

CHAPTER 6

DURABILITY OF POLYMER-IMPREGNATED CONCRETE

6.1 Introduction

There is little doubt that deterioration in concrete due to water intrusion is one of the major problems of concrete structures. The use of concrete in many kinds of structures has proved that in some case the watertightness of concrete may be of greater importance than strength (Ref. 8). In recent years, numerous hypotheses have been made to explain the deterioration of concrete highway bridge decks. Deterioration of concrete bridge decks can be caused by many factors,such as overloading, shrinkage,and repeated loads (Ref. 38). It is believed that the major factor that causes bridge deck deterioration is the ingress of water. Freezing and thawing of the water inside the concrete slab can result in severe damage to the concrete. Corrosion of the reinforcing bars, also a result of water penetration, is a serious problem because it is progressive and ultimately leads to cracking or spalling of the concrete.

Polymer-impregnated concrete has been shown to have greatly improved physical and mechanical properties. Partial or surface impregnation of polymer into concrete has the primary objective of providing greater resistance to the ingress of water. Previous tests have been performed to determine the skid resistance, abrasion,and watertightness properties of polymer surface impregnation specimens. It was found that polymer surface treatments provided excellent protection against freeze-thaw and water penetration. It was also found that the abrasion and skid resistance are not reduced and in some cases were improved (Ref. 3).

As a result of these advantages, more extensive tests were conducted to evaluate the durability performance of partially and fully-impregnated concrete. These test methods are

1) freeze-thaw tests,
2) long term salt-water exposure, and
3) fully-impregnated pile specimens.

6.2 Freeze-Thaw Tests

Several tests were conducted to evaluate the effect of polymer depth, reinforcing, and polymer types on freeze-thaw performance of PIC surface treatments. The procedures used in exposing the specimens to freezing and thawing, and in evaluating their relative durabilities were previously described (Ref. 3).

6.2.1 Effect of Polymer Depth.

To determine the effect of polymer depth on freeze-thaw resistance of polymer surface treatments, tests were conducted on 10-in.x10-in.x6-in. concrete which had a water-cement ratio of 6.5 gal./sk., cement factor of 6.0 sk./cu.yd., slump of 3.0 in., and a 28-day moist cured compressive strength of 6390 psi.

Twenty-one slabs were cast in the laboratory and air cured for several months. Prior to impregnation, the slabs were completely saturated in water for 7 days and dried in an oven for different lengths of time to obtain the specimens with various relative moisture contents (R.M.C.). Because the specimens were treated at various R.M.C., different polymer depths were obtained. The slabs were divided into 7 groups according to their R.M.C. prior to monomer application. Each group consisted of three slabs: one slab was broken open to measure the polymer depth, two other slabs were used for the evaluation of freeze-thaw resistance. The average R.M.C. ranged from zero to 77 %. The specimens were soaked with a mixture of MMA, 1 % BP (wt.), and 10 % TMPTMA (wt.) for a period of about 17 hrs. and cured by steam ($\approx 190^\circ$F on the surface) for 1 hr. The slab treatments and freeze-thaw test results are shown in Table 6.1. The polymer depths ranged from zero to 2 in.

Visual observations of the specimens during freeze-thaw exposure indicated that the treated slabs had higher freeze-thaw resistance than the untreated slabs. It was found that the specimens with an approximately 1/8-in. polymer depth (specimens 2, 3, 10, and 11) could sustain the freeze-thaw exposure up to an average of 79 cycles as compared to 30 cycles for the untreated specimens. The specimens with 1/8-in. to 1/4-in. faint-color polymer depths (specimens 17, 18, 19, and 21) failed at an average of 69 cycles. The lower parts of specimens 4 and 5, which had a polymer depth of 1.0 in., were

TABLE 6.1 SUMMARY OF FREEZE-THAW TESTS ON SPECIMENS (PC-23) WITH VARIOUS POLYMER DEPTHS.

Slab No.	Relative Moisture Content, %	Soak Time, hrs. & Temperature, °F	Average Polymer Depth, in.	Maximum Freeze-Thaw, cycles	Failure Mode
2	54	19 @ 78-85	1/8	77	Surface & side
3	53	19 @ 78-85	1/8-3/16	78	Surface & side
4	16.5	19 @ 78-85	1	84*	Bottom
5	23.8	19 @ 78-85	1	117*	Bottom
8	0	19 @ 78-85	2	200^+	No failure
9	0	19 @ 78-85	2	200^+	No failure
10	77	19 @ 78-85	1/8-3/16	82	Surface & side
11	74	19 @ 78-85	1/8-3/16	78	Surface & side
15	-	Control	-	29	Surface & side
16	-	Control	-	31	Surface & side
17	71	19 @ 78-85	1/4 faint	71	Surface & side
18	66	19 @ 78-85	3/16 faint	66	Surface & side
19	65	19 @ 78-85	1/8-3/16 faint	65	Surface & side
21	71	19 @ 78-85	3/8-1/4 faint	71	Surface & side

* No change was observed on the surface of the specimen.

severely deteriorated after 84 and 117 cycles of freeze-thaw, respectively. It was believed that these deteriorations were caused by the ingress of water through the bottom of slabs during treatment and testing period. With 2-in. polymer depths (specimens 8 and 9), the specimens could sustain freeze-thaw exposure of more than 200 cycles without significant damage.

Several photographs are shown in Figs. 6.1, 6.2, and 6.3 to illustrate the effect of polymer thickness on freeze-thaw resistance of selected specimens treated under laboratory conditions. Fig. 6.1 shows the typical side view of the crack pattern produced by freeze-thaw exposure of treated and untreated slabs. Cracks were generally developed on the surface and on the sides of the specimens.

Fig. 6.2 shows the surface appearances of slabs with 1/8-in. polymer depths at zero and 76 cycles freeze-thaw exposure. At 76 cycles the specimens could not retain the water during thaw cycles and testing was discontinued. In comparison with the untreated slabs which failed at 30 cycles, the 1/8-in. polymer depth slab had a freeze-thaw resistance of about 2.6 times the untreated specimens.

Fig. 6.3 shows the surface appearances of a slab with a 1-in. polymer depth (PC-23-5) at zero and 100 cycles of freeze-thaw exposure. It can be seen that with adequate polymer depth, the polymer surface treatment totally protects the exposed surface from freeze-thaw damage through 100 cycles. Close examination of the specimen (Fig. 6.3) revealed that very small surface scaling was

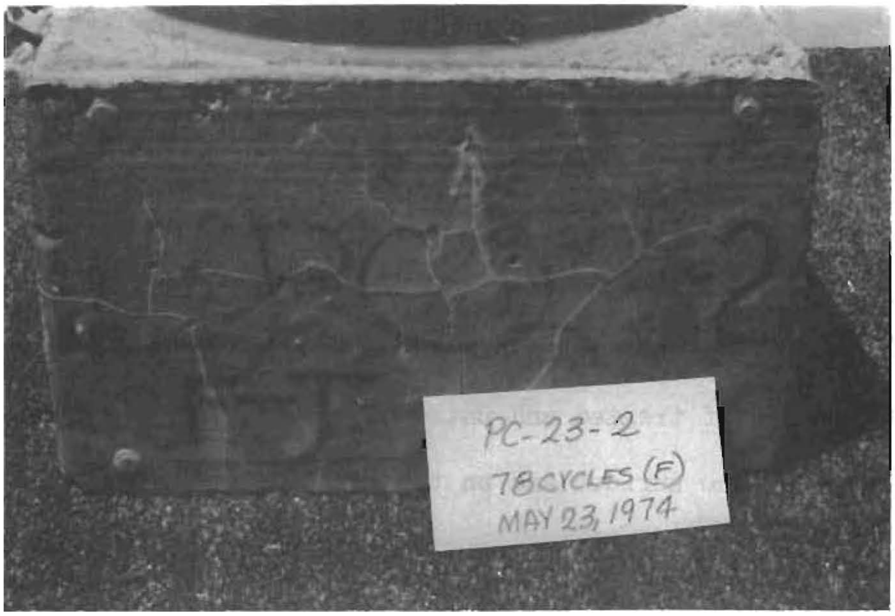


Fig. 6.1 Typical Crack Pattern Produced by Freeze-Thaw Exposure of PC-23 (Side View).

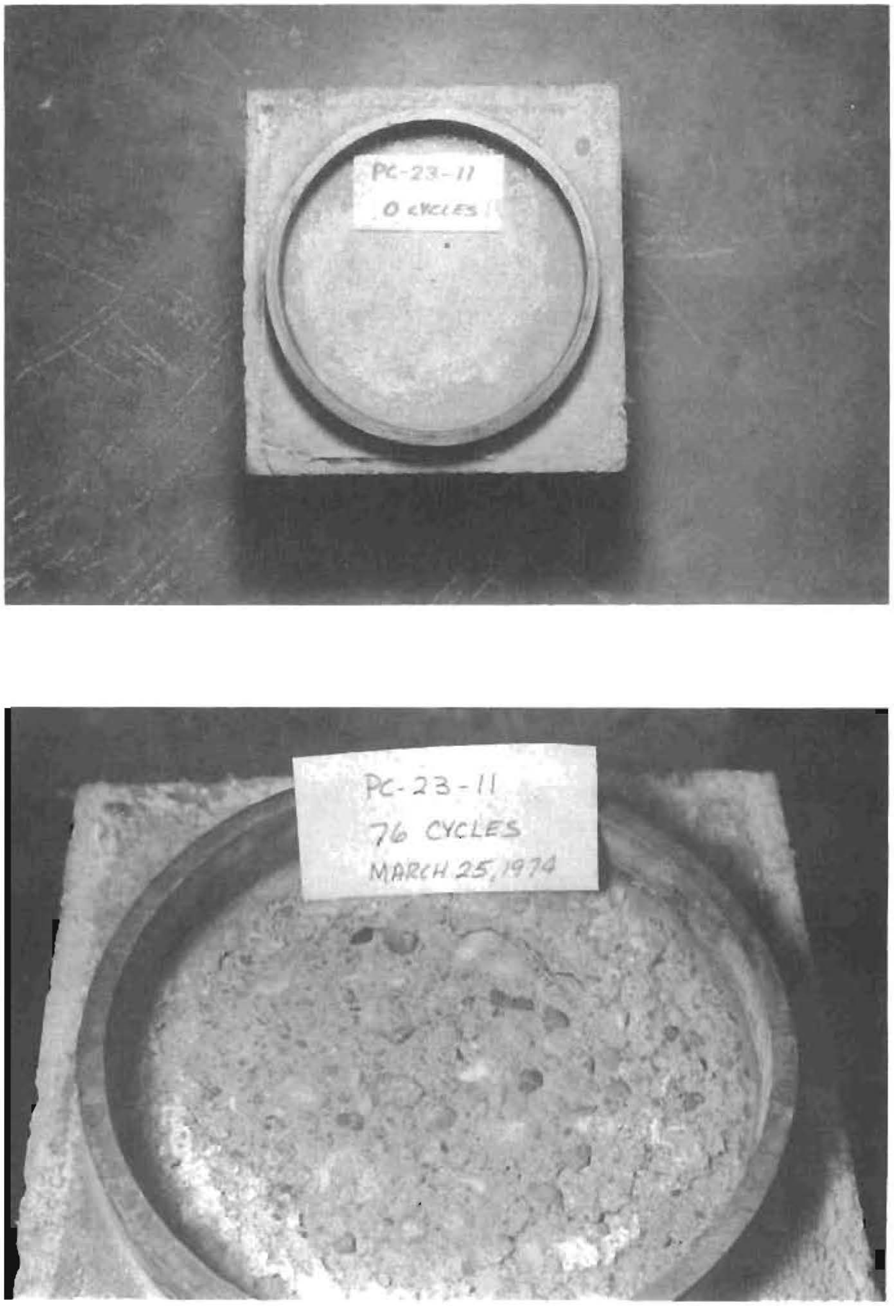


Fig. 6.2 1/8 to 3/16-Inch Thickness of MMA Laboratory Treated Slab at Zero and 76 Cycles (PC-23-11).

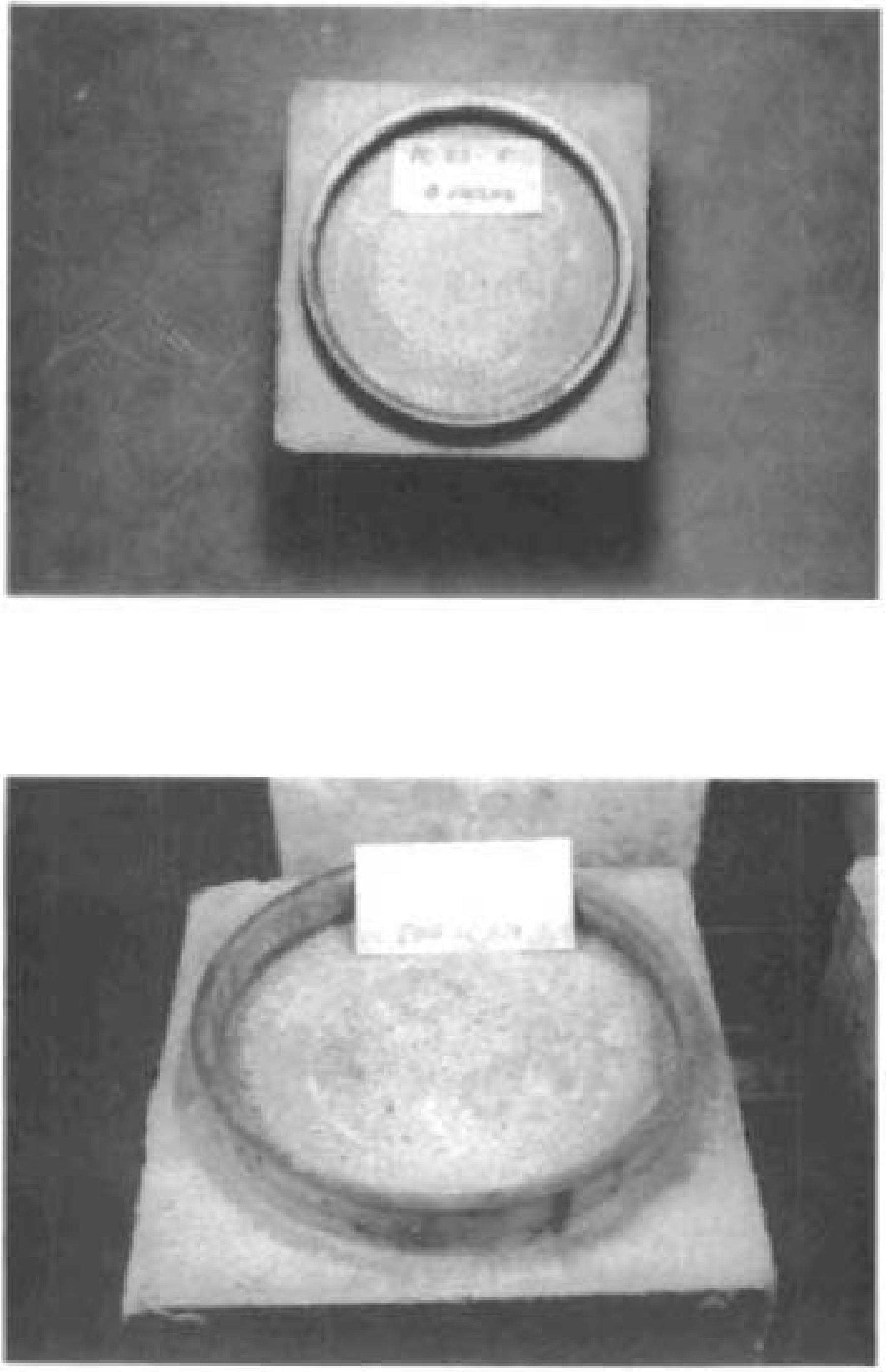


Fig. 6.3 One Inch Thickness of MMA, Laboratory Treated Slab at Zero and 100 Cycles (PC-23-5).

observed but no severe damage on the exposed surface could be detected through 100 cycles of freeze-thaw exposure.

Although visual and photographic observations were made of each specimen throughout the test period, it was determined that a more useful measure of the specimen's volume stability during testing could be provided by use of periodic strain measurements (Ref. 3). For this purpose a Berry mechanical strain guage was used to obtain readings at approximately every 10 and 5 cycles of freezing and thawing of treated and untreated slabs, respectively.

Horizontal strain measurements were made on the side of each of the freeze-thaw specimens by means of four stainless steel tabs bonded 8.0 in. apart 1/2 in. above the bottom surface and 1/2 in. below the top surface. Measurements were made with an 8-in. Berri guage with the slabs in frozen state. The results of these measurements are presented in Fig. 6.4. Strain computations are based on a comparison of the initial, unfrozen state and subsequent frozen states of the specimens. Strains at the top of the specimens are shown. It can be seen from Fig. 6.4 that all specimens exhibit thermal strain contraction in the range of 500 to 2,000 micro in./in. at the beginning of the tests. As freeze-thaw cycling progressed, the loss in volume stability of the untreated control specimens (specimens 15 and 16) was found to be much greater than for the treated specimens. Hairline cracks were observed at 19 cycles of freeze-thaw exposure. Complete loss of ponded water during the thaw cycle, which defines the failure

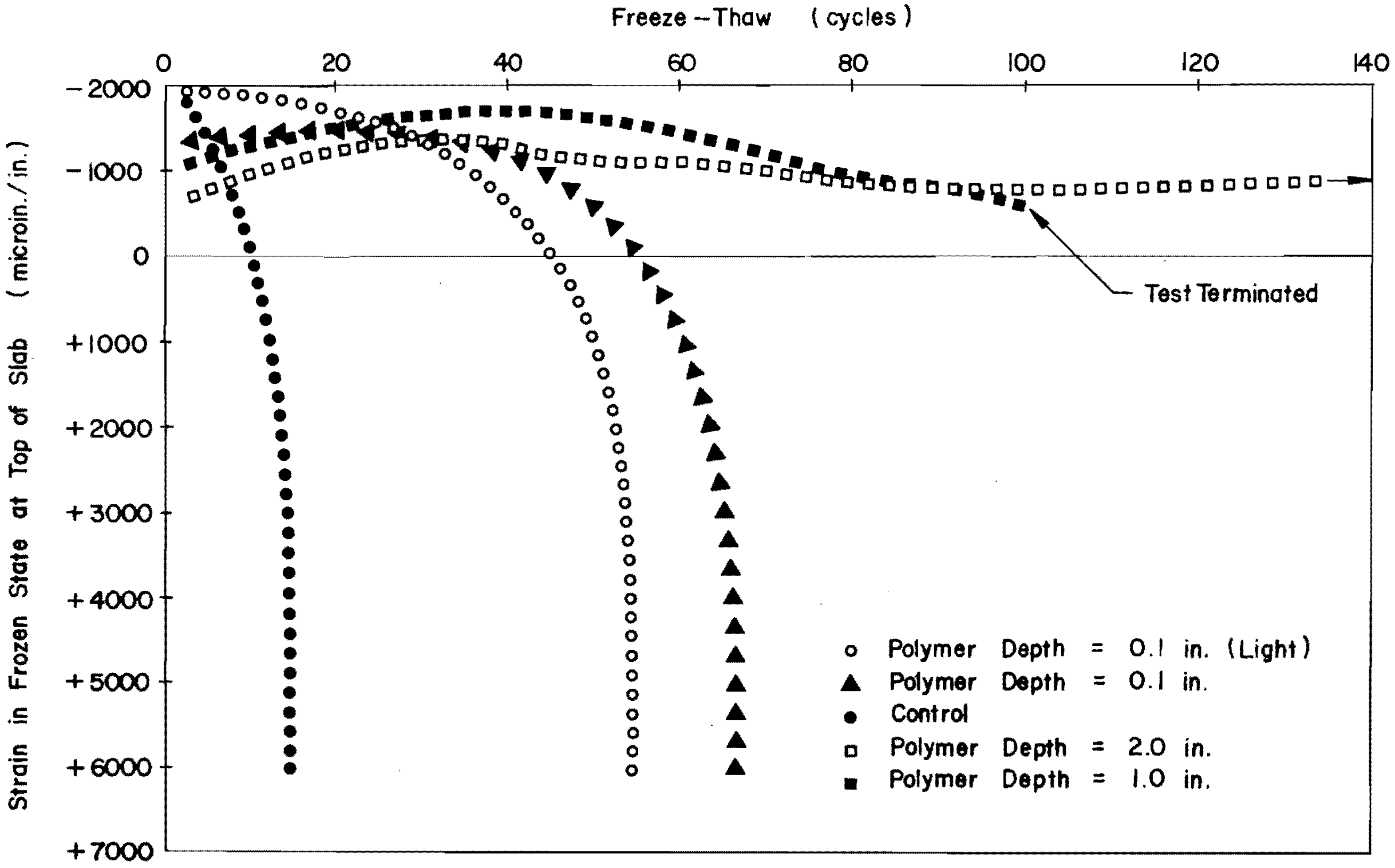


Fig. 6.4 Effect of Freeze-Thaw Exposure on Volume Stability of Specimens with Different Polymer Depths (PC-23).

of a specimen, occurred at 30 cycles. From Fig.6.4 it can be seen that the deterioration of the untreated specimens is also indicated by rapid increase in strain in the frozen state at the top of slab as the freeze-thaw cycles continued. It is believed that the increase in strain is a result of the rupture of the internal structure of concrete due to the formation of ice in saturated capillary pores. Based on Fig. 6.4 the top strain of 500 to 1,000 microin./in. seems to be the upper limit of volume stability of specimens. Whenever the top strain reached 1,000 microin./in., the specimens lost their volume stability rapidly and led to the final failures. This result agrees well with previous research (Ref. 3) which found that cracks could be observed visually whenever the top strain exceeded about 1,000 microin./in. In comparison between the strain developed on the top and bottom of specimens during freeze-thaw exposure, the top strain indicated less volume stability than the bottom strain.

The least durable treated slabs were the specimens with about 1/8-in. polymer depth. The color of polymer was relatively light as compared to other series of 1/8-in. polymer depth. The light color of polymer indicates lower polymer loading. With this series of specimens the hairline cracks were observed at 58 cycles. Failures of the specimens were found at an average of 68 cycles. The deterioration of this series of specimens was also indicated by rapid increase in strain as freeze-thaw cycling continued. The failure of this series of specimens developed due to the insufficient polymer depth and loading. If the polymer depth is not thick enough, it is unable to maintain its resistance to penetration of water into the concrete during freeze-thaw and the typical ice-expansion failure mechanism is developed (Ref. 3). However, as compared to the untreated specimens this series still provided 135 % increase in freeze-thaw resistance. It was also observed from stain measurements that the volume stability of the top of the specimen, which was indicated by the change of strain during freeze-thaw period, was less stable than that for the bottom of the specimen.

The series of the specimens which show the next lowest resistance to freeze-thaw deterioration had a 1/8-in. polymer depth with a dark color of polymer (greater loading). With this test series the hairline cracks were observed at an average of 64 cycles of freeze-thaw exposure. Failure of the specimens occurred at an average of 79 cycles. The failure is believed to be the same as for specimens 17, 18, 19, and 21. The deterioration of the specimens was also indicated by a rapid increase in strain,as shown in Fig. 6.4. The volume stability at the bottom of the specimen was also better than for the top throughout the test period.

The specimens with about 1 in. of polymer depth (specimens 4 and 5) indicated no sign of cracking on the surface during freeze-thaw testing,which was continued to an average of 100 cycles. However, cracks were observed on all sides of the slabs at approximately 56 cycles. At 84 and 117 cycles, the bottom portions of specimens 4 and 5, respectively, were completely deteriorated and the freeze-thaw test was discontinued. However, it is indicated in Fig. 6.4 that, from the volume stability standpoint, this test series maintained a stable volume throughout the test period.

The last test series consisted of specimens with about 2 in. of polymer depth (specimens 8 and 9). It is apparent from Fig. 6.4 that for this test series the specimens maintained a stable volume throughout the test period. Visual observation indicated no cracking or scaling on the surface of these specimens after 120 cycles, which is an arbitrary value. In a comparison between the strains developed on the tops and bottoms of specimens during freeze-thaw exposure, the top strain indicated better volume stability than the bottom strain. For example, at 120 cycle, PC-23-9 indicated a top strain of about -920 microin./in. as compared to a bottom strain of -470 microin./in. It should be noted that the tests were continued to 200 cycles and no cracks were observed.

6.2.2 Effect of Polymer Type. A previous report has included the results of freeze-thaw tests on concrete impregnated using three different monomers: MMA, isobutyl methacrylate (IBMA), and isodecyl methacrylate (IDMA) (3). A brief summary is presented here; additional details may be found in Reference 3.

The slabs were cast and treated under field-simulated conditions. The slabs were 43 x 40 x 5.5 in. The concrete had a water-cement ratio of 6.5 gal/sk, cement factor of 6.0 sk/cu.yd, 3-in. slump, and a 28-day, moist-cured strength of 6400 psi. The slabs were air-dried for several months prior to impregnation.

The slabs were dried with an electric curing blanket a minimum of three days. The slabs reached a surface temperature of $\sim 150^{\circ}$ F. The slabs were impregnated during the initial phase of the research when temperatures of $\sim 150^{\circ}$F were believed to be adequate and desirable.

The monomer, with 1% (wt) BP and 10% (wt) TMPTMA, was applied to a 0.25-in. cover of lightweight aggregate fines. Some slabs were cured by ponded hot water, and some cured by the electric curing blanket. The polymer impregnation was of poor quality as evidenced by the light color and shallow, splotchy depth. This was probably caused by the low drying temperatures.

The treatments are summarized in Table 6.2. Specimens 8 and 9 were treated with a second reactive monomer system consisting of MMA, 4% (wt) lauroyl peroxide, and 4% (wt) N, N-dimethyl-p-toluidine (DMPT) which, when polymerized, bonded the fine aggregate to the surface. It was theorized that the bonded fine aggregate would provide an additional surface that would have excellent skid resistance.

The results of the freeze-thaw tests indicated that the MMA specimens could withstand 120 cycles or more as compared to an average of 35 cycles for the controls. Neither of the IBMA specimens reached 120 cycles and only one IDMA specimen did. But the most significant conclusion of these tests was that even poorly impregnated slabs have significantly more resistance to freeze-thaw deterioration.

6.2.3 Effect of Reinforcing Bars. Most of the previous research on freeze-thaw durability of PIC partial-impregnations was performed on plain concrete. However, the primary objective of PIC

TABLE 6.2 SUMMARY OF FREEZE-THAW TESTS FOR FIELD TREATED SLABS.

Slab No.	Monomer System	Quantity of Monomer, ml./m^2	Soak Time, hrs., & Temperature, °F	Cure Method	Average Polymer Depth, in.	Maximum Freeze-Thaw, cycles.
1	control	-	-	-	-	30
2	MMA^a	4280	10 @ 73-94	HW^c	0.25-0.75 faint, uniform	120^e
3	IBMA^a	3600	10 @ 77-93	HW	0.75 faint, uniform	91
4	IBMA^a	4500	24 @ 75-90	HW	0.5-1.5 faint, uniform	117
5	IDMA^a	2700	10 @ 77-93	HW	0.25-0.5 very faint, uniform	120^e
6	control	-	-	-	-	40
7	IDMA^a	3600	24 @ 75-90	HW	0.25-0.5 dark to faint	117
8	MMA^a	6750	24 @ 75-93		0.5-0.75 faint, uniform	120^e
	MMA^b	900	0.25 @ 93	HW		
9	MMA^a	4500	18 @ 59-80			
	MMA^b	1350	1 @ 80	HB^d	0.25-0.5 faint, non-uniform	120^e
10	MMA	6750	24 @ 75-90	HW	0.5-0.75 faint, uniform	120^e
11	MMA	9900	18 @ 59-80	HB	0.25-0.50 faint, uniform	120^e
12	MMA	9900	24 @ 58-82	HB	0.5 very faint, uniform	120^e

[a] Monomer system included monomer, 1 % BP, 10 % TMPTMA

[b] Second monomer application included monomer, 4 % lauroyl peroxide, 4 % DMPT

[c] HW = hot water

[d] HB = heating blanket

[e] Testing terminated after 120 cycles

partial-impregnation application is for highway bridge decks, which are always reinforced. For this reason it was essential to study the effect of reinforcing bars on freeze-thaw behavior of PIC surface treatments to determine if the restraint offered by the reinforcing to the freeze-thaw expansion was significant.

With reinforcing bars, the thermal expansion of the steel, PIC, and plain concrete are different. The coefficient of thermal expansion of steel is about 6.5 microin./in./°F (Ref. 12) as compared to 4.34 microin./in./°F for plain concrete and 4.93 microin./in./°F for PIC (the test results of chapter 8). For concrete and steel to work together it is necessary that bond stress be developed between the two materials. Whenever different changes in lengths occur in the concrete or reinforcing steel, internal stress is developed.

The tests were performed on 10-in.x10-in.x6-in. concrete slabs with a water-cement ratio of 8.0 gal./sk., cement factor of 4.7 sk./cu.yd., 5 in. slump, and the 28-day moist cured compressive strength of 3820 psi. (PC-38). The types of monomer and locations of reinforcing bars are shown in Table 6.3. The bars were located in both directions. The specimens were divided into 3 groups according to the location of reinforcing bars as follow:

1) 2 - #5 bars at top and bottom (numbers 1, 2, and 3);
2) 2 - #5 bars at bottom only (numbers 4, 5, and 6);
3) 2 - #5 bars at top only (numbers 7, 8, and 9).

TABLE 6.3 SUMMARY OF FREEZE-THAW TESTS (PC-38)
FOR REINFORCED SLABS.

Batch No.	Monomer	Location of Steel	Polymer Depth, in.	Freeze-Thaw, cycles
PC-38-MMA-1	MMA	Top & bottom	2.0	200^+
MMA-2	MMA	Top & bottom	2.0	200^+
MMA-3	MMA	Top & bottom	2.0	200^+
MMA-4	MMA	Bottom	2.0	200^+
MMA-5	MMA	Bottom	2.0	200^+
MMA-6	MMA	Bottom	2.0	200^+
MMA-7	MMA	Top	-	23
MMA-8	MMA	Top	-	23
MMA-9	MMA	Top	-	25
PC-38-IBA-1	IBA	Top & bottom	-	26
IBA-2	IBA	Top & bottom	-	26
IBA-3	IBA	Top & bottom	-	26
IBA-4	IBA	Bottom	-	26
IBA-5	IBA	Bottom	-	26
IBA-6	IBA	Bottom	-	26
IBA-7	IBA	Top	2.0	120^+
IBA-8	IBA	Top	2.0	120^+
IBA-9	IBA	Top	2.0	120^+
PC-38-C-1	-	Top & bottom	-	18
C-2	-	Top & bottom	-	20
C-3	-	Top & bottom	-	20
C-4	-	Bottom	-	20
C-5	-	Bottom	-	20
C-6	-	Bottom	-	19
C-7	-	Top	-	18

The covering of the reinforcing bars was 1.5 in. from the exposed surface. Each group was treated with MMA and isobutyl acrylate (IBA) solution as usual. However, the curing temperature was not high enough, and no appearance of polymer could be detected in group 3 of the MMA slabs and groups 1 and 2 of the IBA slabs.

Each series of specimens for freeze-thaw tests consisted of 3 specimens. For the purpose of comparison, the control specimens were tested along with the treated specimens.

Visual observations of the specimens during freeze-thaw tests indicated no major change in the surface appearance of the treated slabs after 120 cycles of freeze-thaw. On the contrary, the untreated specimens failed at about 20 cycles. Selected photographs are shown in Figs. 6.5 and 6.6 to illustrate the surface appearances of the treated and untreated specimens. Fig. 6.5 shows the surface appearances of the control specimens at 0 and 20 cycles (failure) of freeze-thaw exposure. The specimens are in group 1 (specimens C1, C2, and C3) with reinforcing bars at top and bottom. Fig. 6.6 shows the surface appearances of the MMA treated specimen with the bars at top and bottom (MMA 2) at 0 and 120 cycles of freeze-thaw. In contrast with the control, no appearance of cracks or significant deterioration on the surface was observed after 120 cycles of freeze-thaw.

The complete test results are presented in Table 6.3. It can be seen that slabs with good PIC surface treatment completed 120 test cycles in good condition. The average freeze-thaw resistance

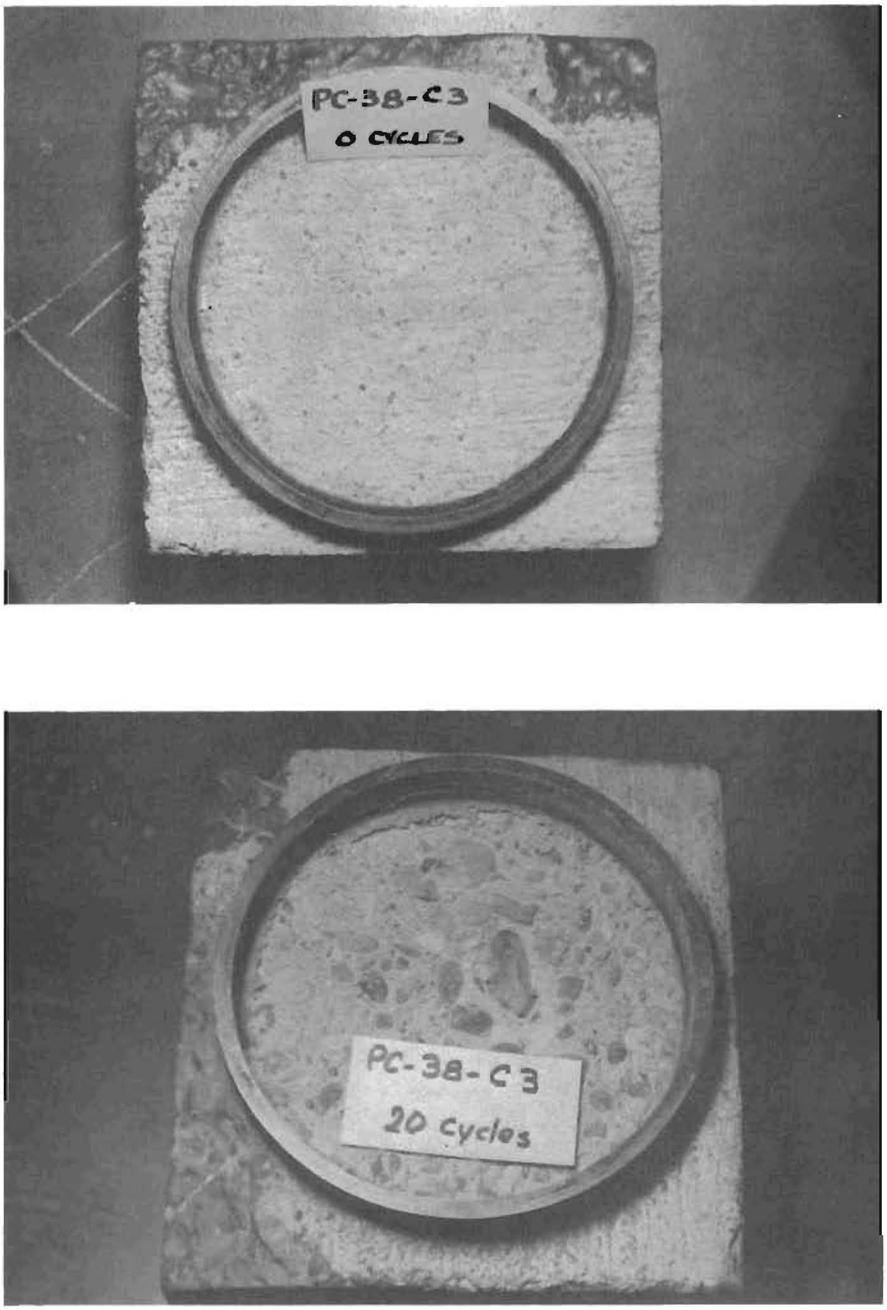


Fig. 6.5 Control Slab at Zero and 20 Cycles of Freeze-Thaw Testing (PC-38-C3).

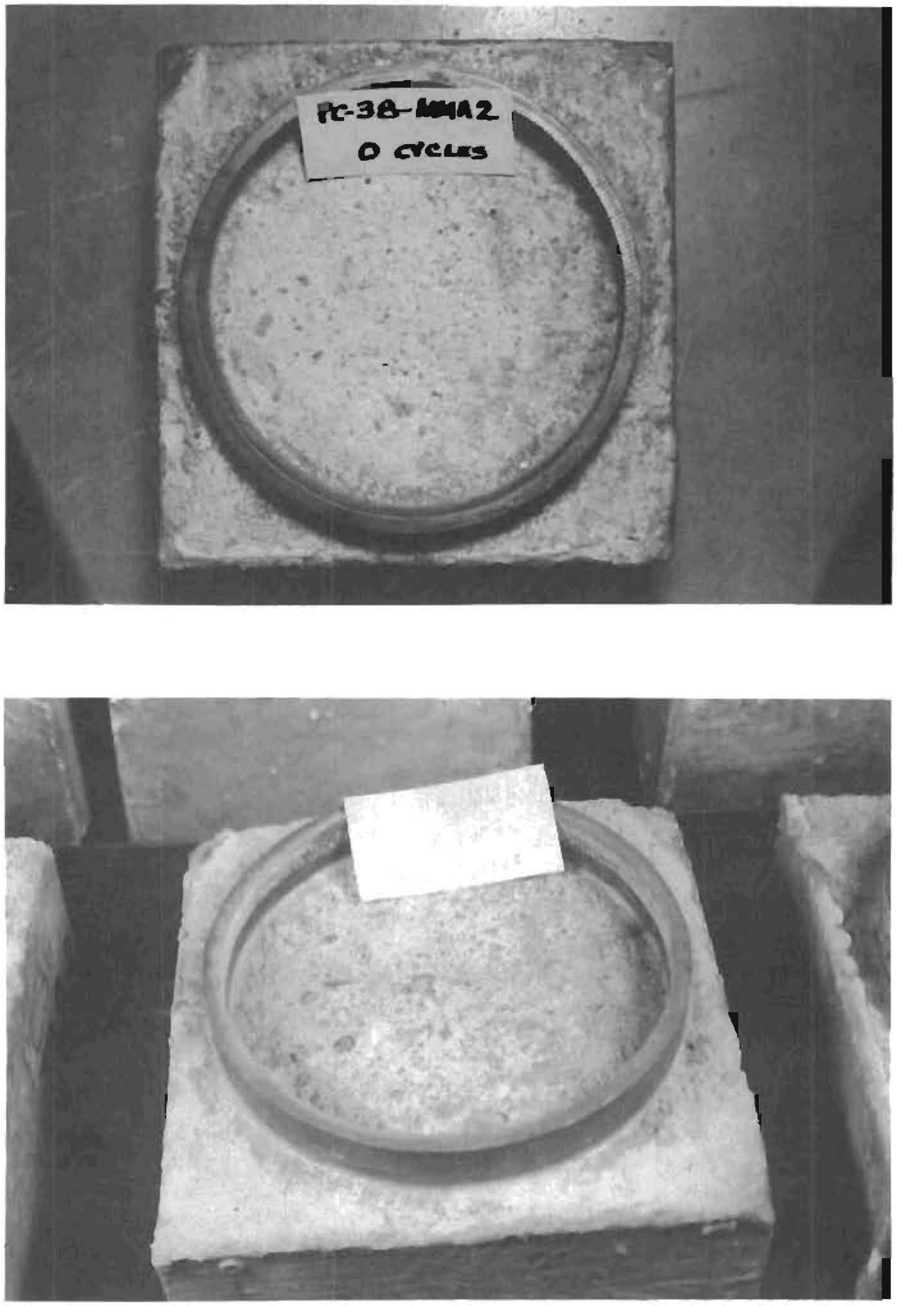


Fig. 6.6 MMA Laboratory Treated Slab at Zero and 120 Cycles of Freeze-Thaw Testing.

for all control specimens was about 19 cycles.

Fig. 6. 7 indicates the effect of freeze-thaw exposure on volume stability of MMA surface treatments. The specimens are in groups 1 and 2. For the purpose of comparison, the control specimens from groups 1 and 2 were also plotted. It can be seen from Fig. 6.7 that, at the beginning of the test, all specimens exhibited thermal contraction strains at tops of slabs about 100 to 1000 micro in./in.. The treated specimens in group 1 showed higher contraction strains on top than at the bottom of slabs. The average contraction strains at the top and bottom of slabs were 900 and 500 microin./in., respectively. This result is believed to be caused by a 15 % higher coefficient of thermal expansion of PIC as compared to the control. In contrast, the specimens in group 2 indicated lower contraction strains at the top than at the bottom. This result, perhaps, may be caused by higher coefficient of thermal expansion of steel as compared to PIC. The average contraction strains at top and bottom were 500 and 800 microin. /in., respectively.

If the PIC surface treated slabs are compared as a group to the control specimens, the data show that PIC surface treatments had much better volume stability than the controls. Cracks were observed on control specimens after 13 cycles of freeze-thaw. No cracks were observed on the surfaces of treated slabs throughout 120, cycles which is an arbitrary stopped value of freeze-thaw.

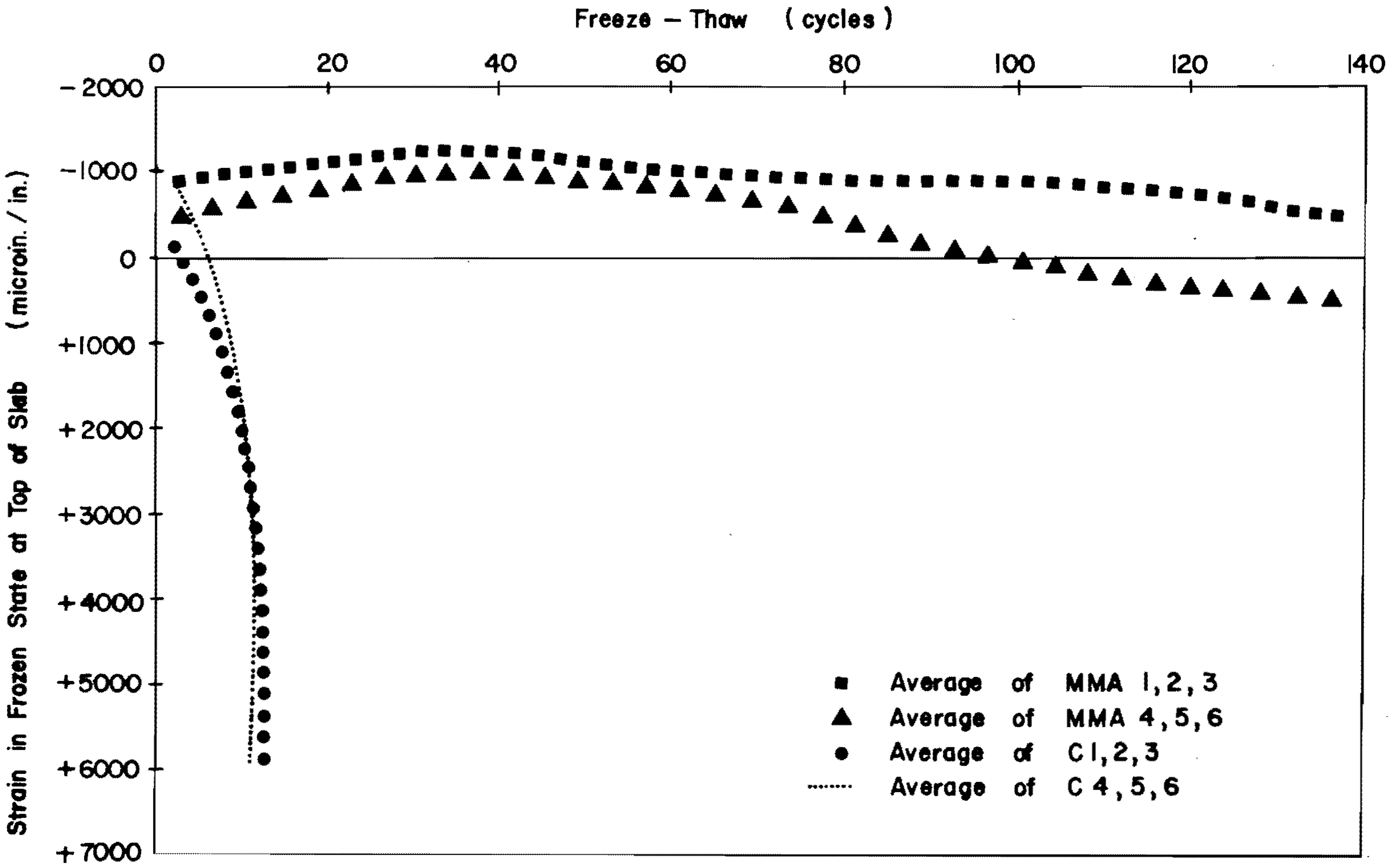


Fig. 6.7 Effect of Freeze-Thaw Exposure on Volume Stability of MMA Surface Treatment of Reinforced Slabs (PC-38).

Fig. 6. 8 shows the comparison in volume stabilities between the IBA treated and the control specimens. All the specimens had the reinforcing bars at the top of the slabs. The average contraction strain at top of IBA treated slab was 2000 as compared to 1330 microin./in. at the bottom. The control specimen contracted about 1400 microin./in. at top of slab. The rate of deterioration in volume stability of control specimens faster than that of the treated slabs. Cracks were also observed after 13 cycles of freeze-thaw for the control. There was little change in surface appearance for treated specimens after 120 cycles.

6.3 Long-Term Salt-Water Exposure

The corrosion problem of reinforcing bars in reinforced concrete structures has been more pronounced in recent years, especially in concrete bridge decks. Minimizing the corrosion problem requires the prevention of the ingress of water into reinforced concrete structures. Ordinary concrete usually provides proper protection for embedded reinforcing bars against corrosion. However, the degree of protection depends on

1) condition, location and arrangement of reinforcing bars,
2) concrete quality, and
3) environmental conditions.

The purpose of long-term salt-water exposure tests were conducted to determine the degree of corrosion protection of reinforcing bars provided by PIC surface treatments. The degree of protection is

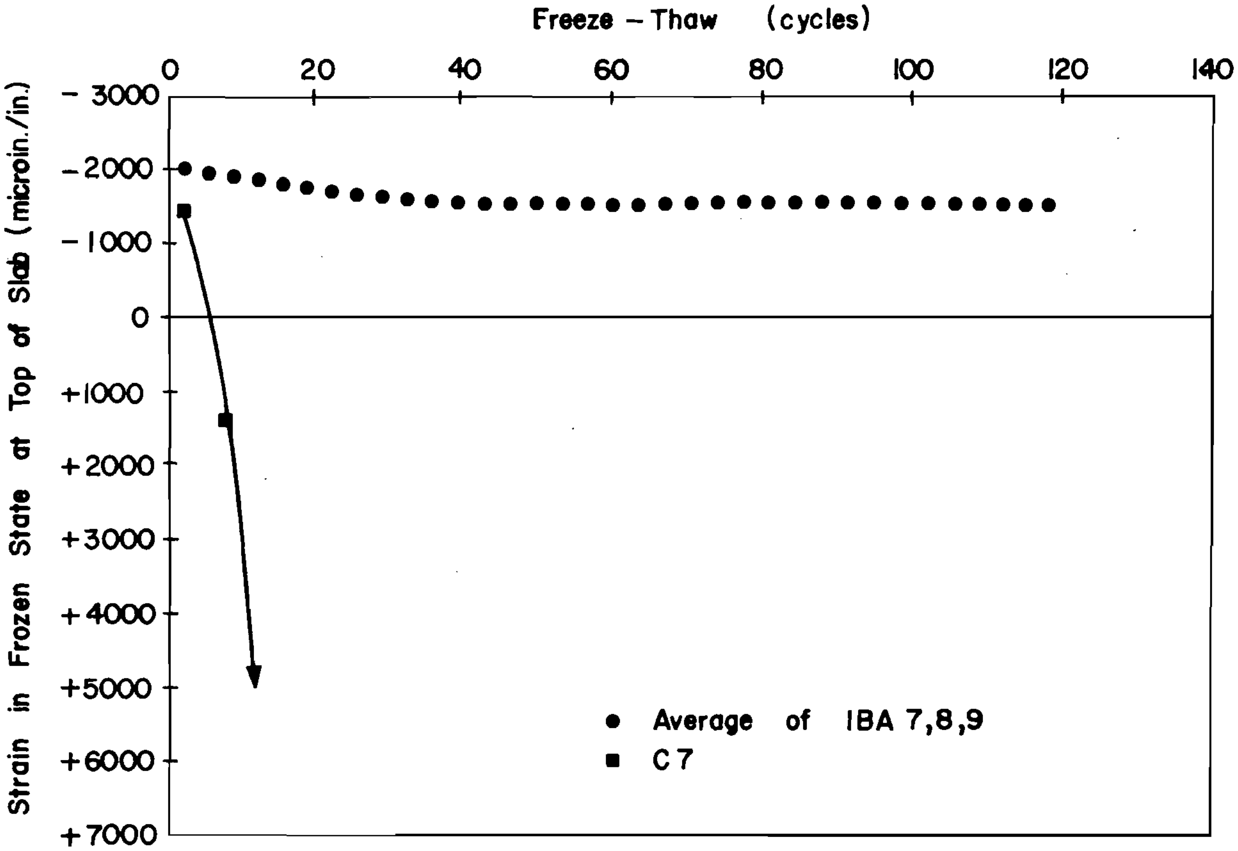


Fig. 6.8 Effect of Freeze-Thaw Exposure on Volume Stability of IBA Surface Treatment (PC-38).

indicated by the corroded area of each bar and by the chloride ion contents of the concrete. The slabs used for these tests were companion specimens to the slabs reported in Section 6.3. The slabs were reinforced with No. 8 bars with a clear cover of 1.25 in. Each slab had the same treatment and was impregnated at the same time as the slab shown in Table 6.2. A more complete discussion of the results was given in a previous report (71).

Each slab was sprayed twice daily, 5 days each week, with a 3% (wt) salt-water solution for 20 months. At the end of the test period, the unimpregnated slabs had developed cracks above the reinforcing bars with some corrosion stain in evidence on the surface. The impregnated slabs were generally in good condition. When the bars were removed it was found that 28% of the surface area of the interior bars for the control slabs was corroded as compared to 1.1% for the bars from the impregnated slabs.

The average chloride contents of the concrete above the bars were measured. The impregnated concrete slabs had an average of 19% as much chloride as the control slabs. The lowest chloride content was for slab 12, which had 268 ppm, or 4.6% of the control. However, it should be recalled that these slabs had a relatively poor impregnation. Concrete piles, which had a much better quality of impregnation, had very low chloride content after long exposure in sea water (71). Fig. 6.9 indicates the chloride content for a depth of 0.5 to 1.0 in. for unimpregnated piles (PC-16-1 and PC-37-2), and impregnated piles. All of the impregnated piles had chloride contents less than the corrosion

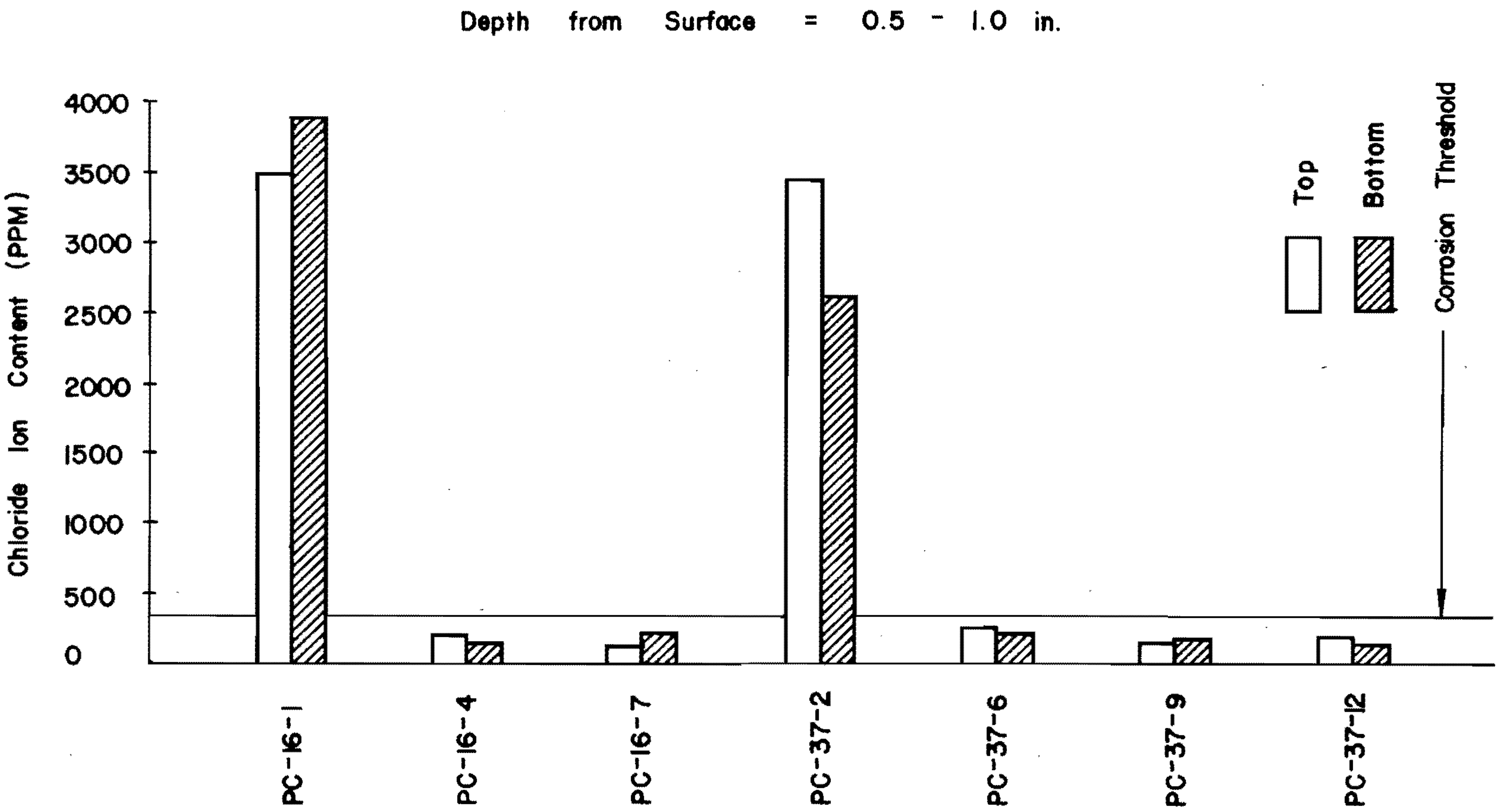


Fig. 6.9 Chloride Ion Content at a Depth of 0.5 to 1.0 in. in Concrete Piles after Sea-Water Exposure.

threshold as established by the Federal Highway Administration (43). The corrosion of bars in the fully-impregnated piles ranged from 0.03 to 0.32% of the surface area for higher quality concrete as compared to 9.9% for the controls. For lower quality concrete, the corrosion of bars in PIC ranged from 0.3 to 0.7% as compared to 47.4% for the controls.

6.4 Summary

PIC treatment slabs were subjected to freeze-thaw tests to determine the durability performance provided by the polymer. Field treated slabs and fully-impregnated piles were subjected to long term salt-water exposure to determine the corrosion protection provided by the polymer. The test results lead to the following conclusions:

1) PIC surface treatment was a very effective method for improving the freeze-thaw resistance of concrete. With only a 1/8-in. polymer depth, the freeze-thaw resistance was 70% greater than the control. With a 1.0 in. polymer depth, there was no change on the surface of the specimens and there was volume stability after 100 cycles.
2) The effect of polymer type on freeze-thaw resistance was less significant than the polymer depth. However, with MMA treatment the surface appearance of the specimens seemed to be better than the other monomer systems investigated.
3) With reinforcing steel in the concrete, there was no damage

to the PIC surfaces under freeze-thaw exposure.

4) The corrosion of the bars in control slabs was about 25 times greater than for the bars in PIC surface treatment even though the impregnation was relatively shallow and faint.

5) The average chloride ion content measured from the slab surface to the steel was 19% of the level measured in the controls.

6) The corrosion of the bars in fully-impregnated piles of higher quality concrete ranged from 0.03% to 0.32% of the surface area as compared to 9.9% for the control. For lower quality concrete, the corrosion of PIC ranged from 0.3% to 0.7% as compared to 47.4% for the controls.

CHAPTER 7

FIELD APPLICATION OF PARTIAL IMPREGNATION TECHNIQUE

7.1 Introduction

Previous chapters have discussed the successful methods and techniques of using PIC for partial impregnations. Various parameters which affected the application of PIC on surface treatments were evaluated. Many successes had been achieved under laboratory and field conditions with 10-in.x10-in.x6-in. and 5.5-in.x43-in. x40-in. concrete specimens. The 2-in. polymer depths are now routinely achieved with adequate drying time and temperature.

It is true that the problems under actual field conditions on highway bridge decks are somewhat different as compared to laboratory and small-scale field treatment conditions. The problem is complicated by the fact that the successful methods used in the laboratory may not be directly applicable in the field. However, based on studies in this report, it seems that applications of PIC for partial impregnation under field conditions are feasible and can be performed in a reasonable period of time.

The purpose of this chapter is to describe and demonstrate techniques suitable for field applications. The experiments were conducted on small slabs under field conditions on actual concrete highway bridge decks. In all tests, the monomer system of MMA with 1 % (wt.) BP and 10 % (wt.) TMPTMA was used.

7.2 Small Scale Field Treatments

All small-scale experiments were conducted on 43-in.x40-in.x 5.5-in concrete slabs. The slabs were cast, cured, and left exposed outdoors at the Balcones Research Center for several months prior to test. The slabs were placed on four concrete blocks and sloped 0.25 in./ft. to simulate the minimum slope of bridge decks. After the slabs were set in place, a forced air heater was used to dry the slabs in the canvas enclosure.

The first series of tests was conducted at a drying temperature of less than 200° F. The experiments were performed on 5 concrete slabs which had a water-cement ratio of 8.0 gal./sk., a cement factor of 4.7 sk./cu.yd., a slump of 3 in., and a 28-day moist-cure compressive strength of 4800 psi.. Thermocouples were attached to the surface of each slab to monitor the slab temperature during the drying period. The drying times for each slab ranged from 18 to 84 hrs. The drying temperatures are shown in Table 7.1.

After slabs were dried and cooled to air temperature, 1/4 in. of dried sand was spread on the top of slabs. The mixture of MMA was applied and soaked overnight (approximately 17 hrs.).

TABLE 7.1 SUMMARY OF POLYMER DEPTH ON 43 x 40 x 5.5-IN. CONCRETE SLABS.

Specimen	Monomer[a]	Drying Time, hrs.	Maximum Temperature, °F	Polymer Depth, in.
PC-39-1	MMA	18	130	0
PC-39-2	MMA	36	144	1/4 faint
PC-39-3	MMA	48	154	1/4 light
PC-39-4	MMA	60	181	1/4 - 1/2
PC-39-5	MMA	84	204	1/2 - 3/4

a = MMA with 1 % (wt.) BP + 10 % (wt.) TMPTMA.

Polyethylene was used to cover the slabs during the soaking period to reduce the evaporation of monomer.

After the completion of soaking, the canvas was used as an enclosure for injecting steam to cure the monomer. The temperature on the surface of the concrete inside the enclosure during curing ranged up to 180° F. The steam was applied for 90 minutes. After curing, they were broken open to measure the polymer depths. The results of the tests are shown in Table 7.1. It can be seen that, with the drying temperature of less than 200° F, the maximum polymer depth of 0.75 in. was achieved with a drying time of 84 hrs.

From these results, it can be seen that a drying temperature of more than 212° F is necessary to achieve an adequate polymer depth within a reasonably short period of time.

A second series of tests was conducted at a drying temperature of about 250° F. The process of drying, soaking, and curing was the same as in the first test series.

Two pairs of slabs were selected as the test specimens. The hot air heater was used for drying. To increase the drying temperature to about 250° F, the drying was conducted on one pair of slabs at a time.

Fig. 7.1 shows the relationship between the drying time and the temperature during the drying process. The studies were conducted at drying times of 4 and 8 hrs.. After drying, the slabs were

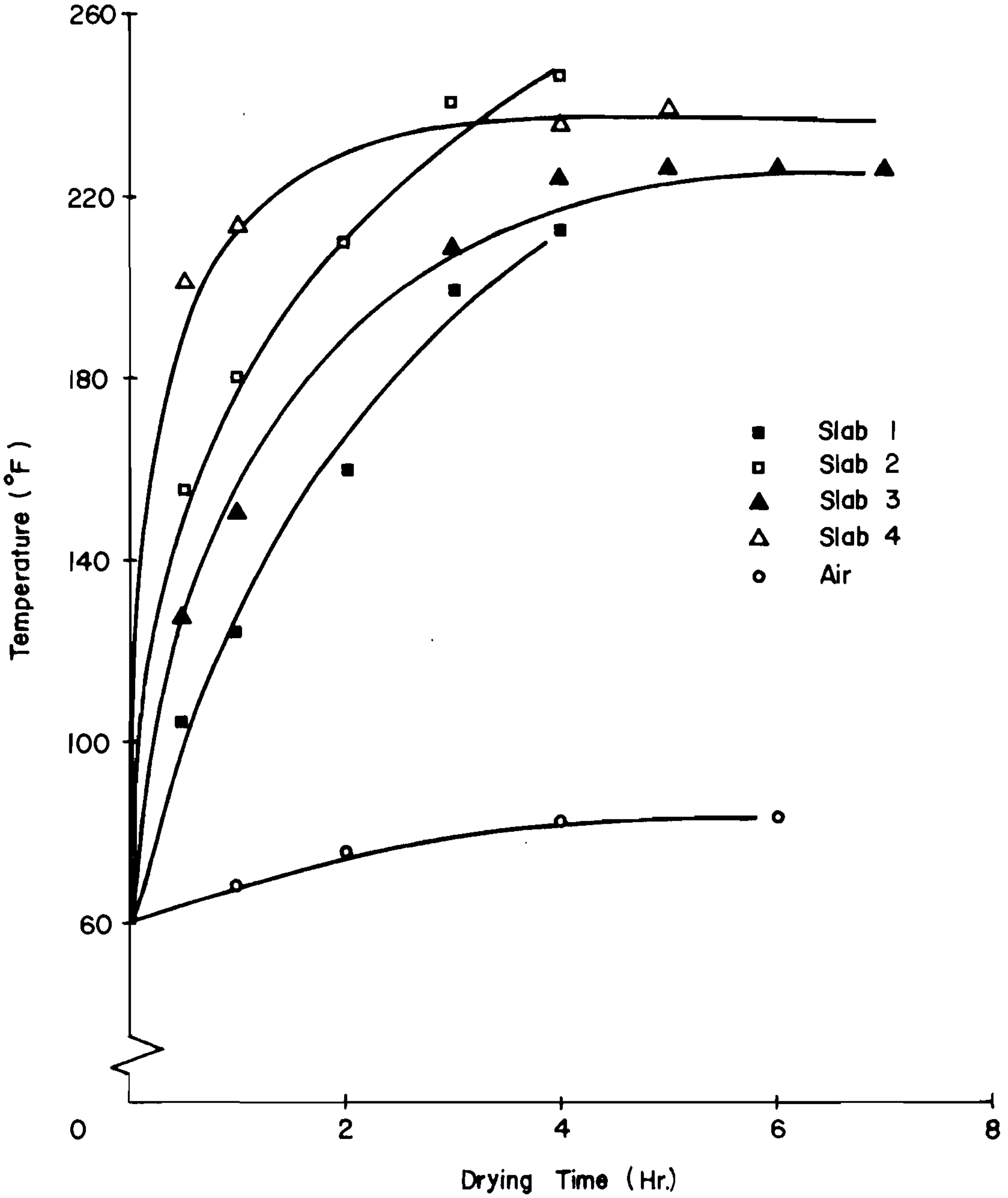


Fig. 7.1 Surface Temperature as a Function of Time.

TABLE 7.2 SUMMARY OF TEST RESULTS AT THE DRYING TEMPERATURE OF 250° F.

Slab No.	Drying Time, hrs.	Soaking Time, hrs.	Polymer Depth, in.
1	4	4	1/8 - 3/8
2	4	8	3/8 - 1/2
3	8	4	3/4 -1-1/4
4	8	8	1-1/4 - 1-3/4

soaked with a mixture of MMA for 4 and 8 hrs. and cured by steam as usual.

Table 7.2 indicates the summary of test results. It could be seen that the average polymer depth of 1.5 in. (slab 4) was obtained with the drying time of 8 hrs. and soaking time of 8 hrs..

7.3 Application of PIC on Highway Bridge Decks.

7.3.1 Bridge Deck No. 1. The first application of a PIC surface treatment on an actual bridge deck in the U.S. was performed on the southbound Mopac Bridge over the Colorado River, Austin, Texas. The treatments were performed in August 1973. A 12-ft.x12-ft. area was selected as the test area, as shown in Fig. 7.2. The bridge was new and had not been opened to traffic. However, linseed oil had already been applied to the deck. The slope of the bridge on the test section was approximately 2.0 % in the South to North direction and 6.0 % in the East to West direction.

The test area was divided into four 4-ft.x4-ft. sections as follows:

1) Section SW - The deck was sandblasted and dried by heater;
2) Section NW - The deck was not sandblasted but dried;
3) Section NE - The deck was neither sandblasted nor dried;
4) Section SE - The deck was not dried but sandblasted.

After sections SW and SE were sandblasted to remove the existing linseed oil, the 12-ft.x6-ft.x1.5-ft. canvas enclosure was

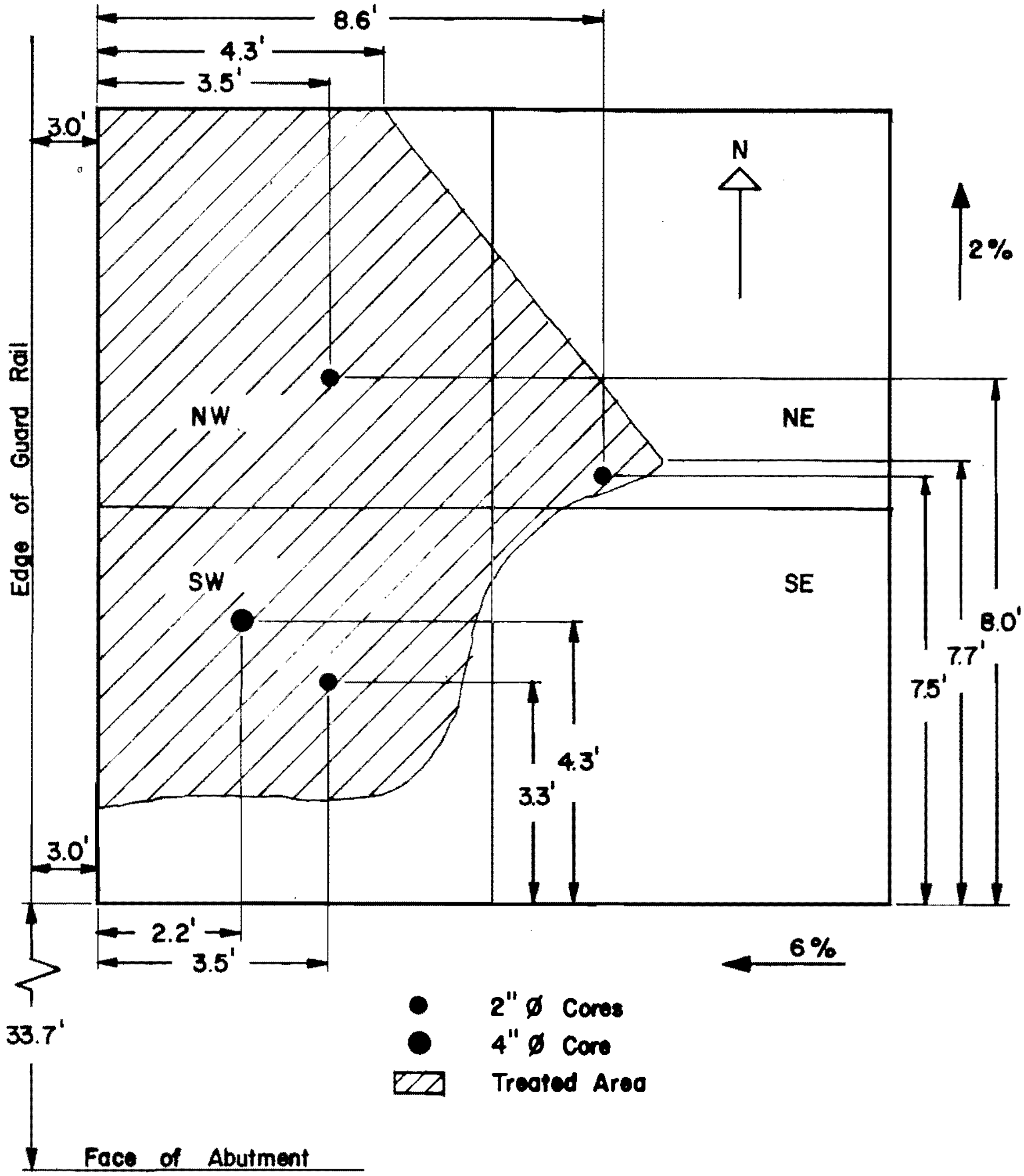


Fig. 7.2 Orientation of Tested Sections at Mopac Bridge.

placed over NW and SW sections. The hot air heater was used as the external heat source to blow the hot air into the canvas enclosure for about 40 hrs. to dry the concrete slab. The outside air temperature ranged from 80 to 100°F during the test period. The surface temperature on the concrete inside the enclosure ranged up to 270°F.

After slab sections were dried, the absorption tests were conducted on every section by using absorption apparatus described in Chapter 3. Fig. 7.3 indicates the relationship between the time and absorption of monomer for each section. It can be seen that the linseed oil treatment strongly affected the absorption properties of concrete. The sandblasted and dried section indicated the highest absorption as compared to others. After 60 minutes of the absorption test,sandblasting of the concrete prior to the test indicated 163 % and 130 % increases in monomer absorption over the non-sandblasted sections for dried and undried sections, respectively.

After the sections were cooled, about 1/4 in. of dried sand was spread over the test area. Just prior to the application of monomer, a sudden rainstorm wetted a portion of dried area. The 12-ft. x12-ft. section was covered.The bottom of the enclosure had been sealed to the slab but the water was able to get into the enclosure. The hatched area in Fig. 7.2 indicates the sand that remained dry and that was subsequently soaked with monomer. The mixture of MMA with 1 % (wt.) BP, and 10 % (wt.) TMPTMA was sprinkled and soaked overnight. Additional monomer was applied to keep the sand moist during

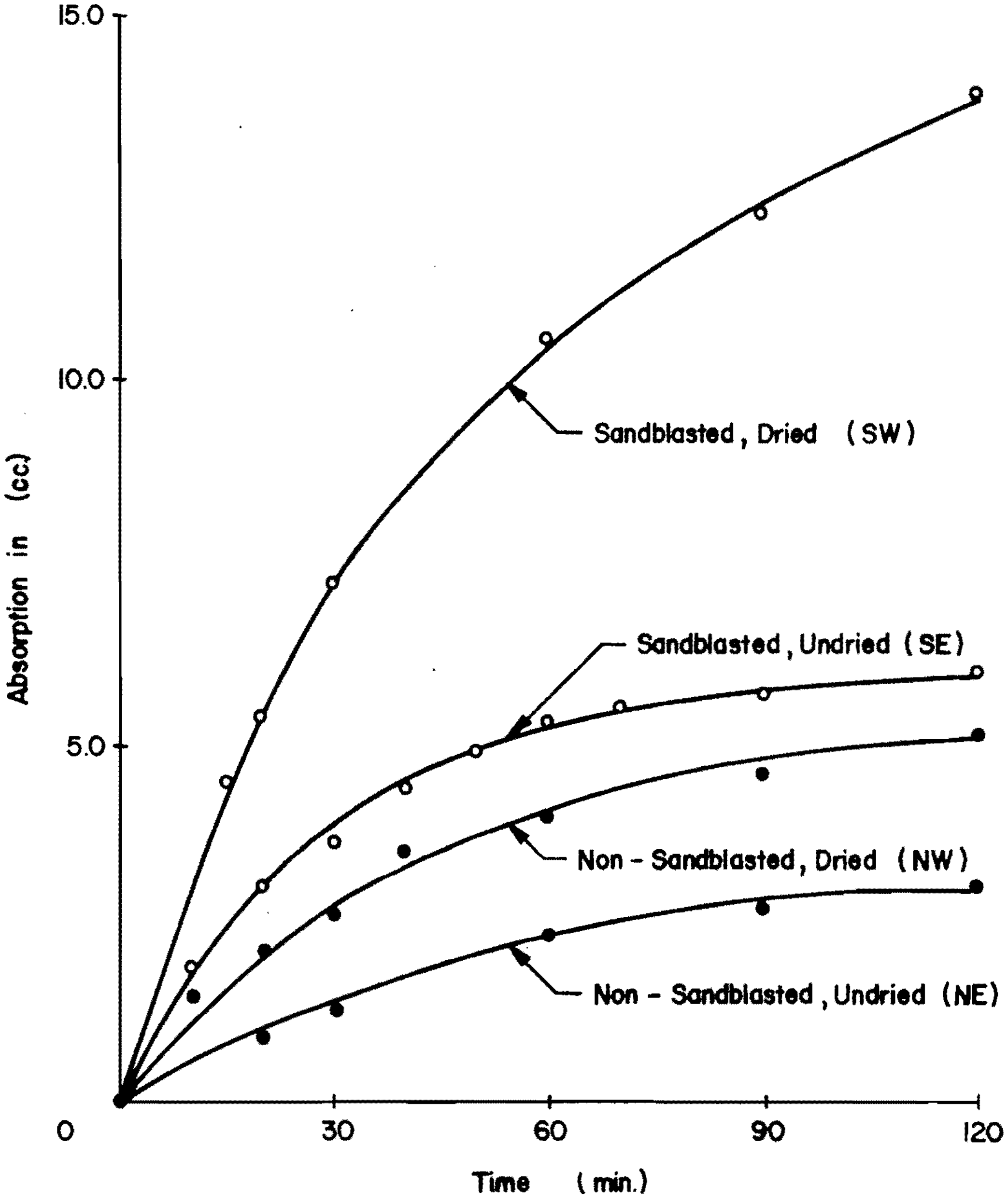


Fig. 7.3 Absorption of Monomer vs Time of Tested Section.

the soaking period.

After completion of the soaking, the canvas enclosure was placed over the soaking sections and the steam was injected into the enclosure to cure the monomer. The temperatures at the concrete surface in section SW were monitored during the curing period. Fig. 7.4 shows the relationship between the slab temperature and the curing time. The slabs were cured for 80 minutes.

The cores were taken from various locations as shown in Fig. 7.2. The 4-in. core, which was taken from section SW was subjected to the freeze-thaw test. A steel ring was attached to the top surface and water was ponded to a depth of about 1/4 in.. The procedure for freeze-thaw test is described in Appendix 3. After more than 230 cycles of freeze-thaw exposure, no change was observed on the surface of the core. The 2-in. cores were broken open to measure the polymer depth. There were no appearances of polymer in the cores from sections NW and NE. Cores from SW section showed the dark layer of polymer range from 3/4 to 1.0 in..

7.3.2 Bridge Deck No. 2. The second surface treatment of actual bridge deck was performed in November 1973 at the Berry Creek Bridge on the west access road of IH-35, approximately 3 miles north of Georgetown, Texas. The bridge deck, constructed in 1939, was believed to be porous. A 12 x 18 ft. section on the northbound lane was selected as the test area. The two variables selected for study were:

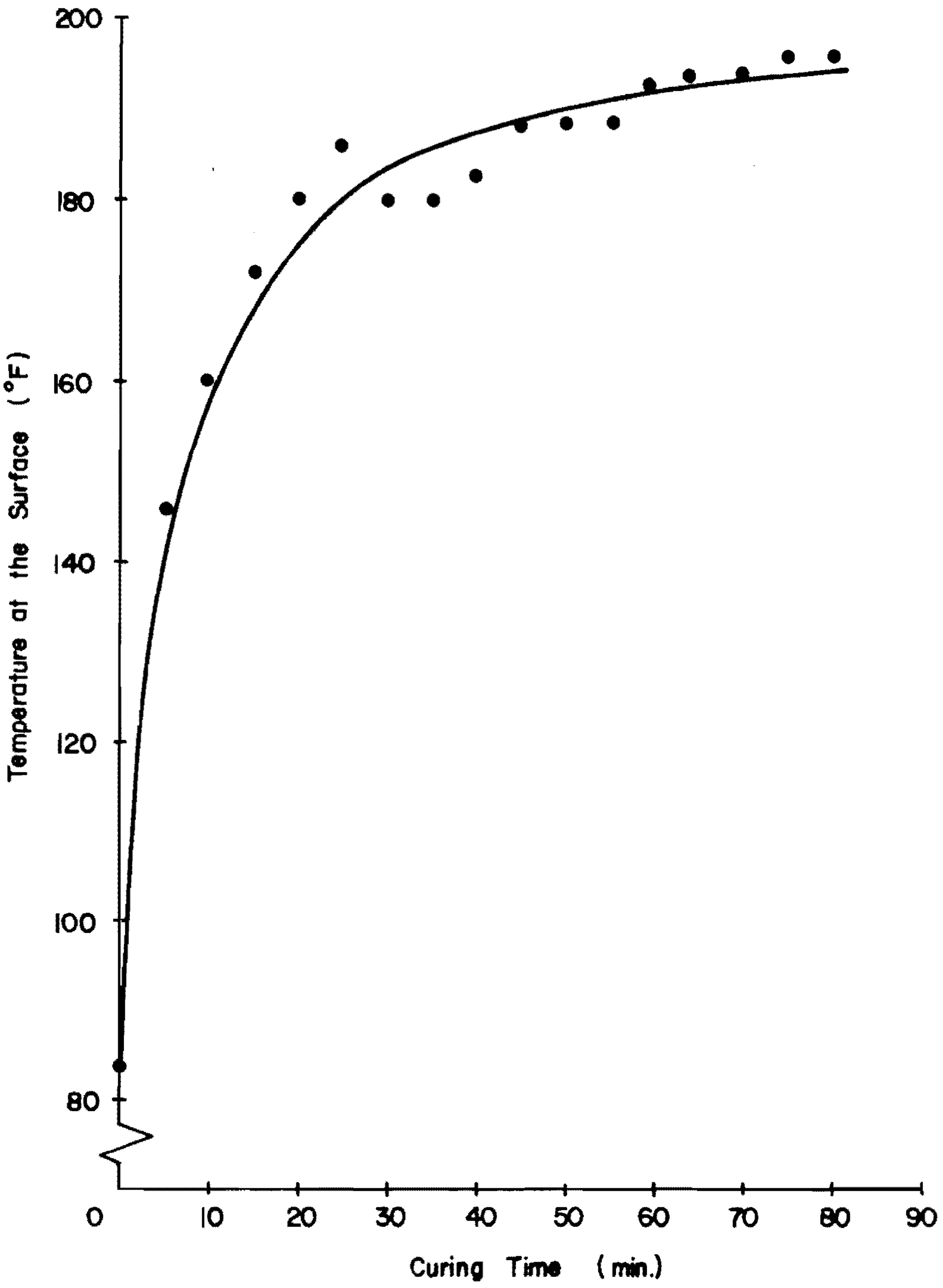


Fig. 7.4 Curing Temperature vs Curing Time for Mopac Bridge.

1) drying time and

2) effect of sandblasting.

Half the lane,an area of 6 x 18 ft., was sandblasted to remove any possible road film which may have existed on the bridge deck,although the bridge surface appeared to be relatively clean. The sandblasted and non-sandblasted sections were divided into 3 equal sections of 6 x 6 ft. in accordance with the drying time,as shown in Fig. 7.5. The sandblasted and non-sandblasted sections were dried for 1, 2, and 3 days.

After the slabs were cooled, dry sand was spread over the tested area to a depth of 1/4 in. The usual MMA mixure was hand sprinkled on the sand and kept moist during the soaking period. The slabs were soaked overnight,beginning at approximately 9 PM. At 8 AM of the following day, additional monomer was added because the sand appeared very dry, especially in the section dried for 3 days , which indicated good absorption of monomer by the old bridge deck. At about 10:30 AM, the steam was injected into the canvas enclosure through a manifold consisting of a 0.5-in. pipe with holes on each side. The holes were turned to spray the steam horizontally. However, there were difficulties because of the mechanical failure of the steam generator used for curing. Surface temperature on the concrete were generally less than 100°F and the maximum temperature was 120°F, which was not high enough to induce polymerization. From the

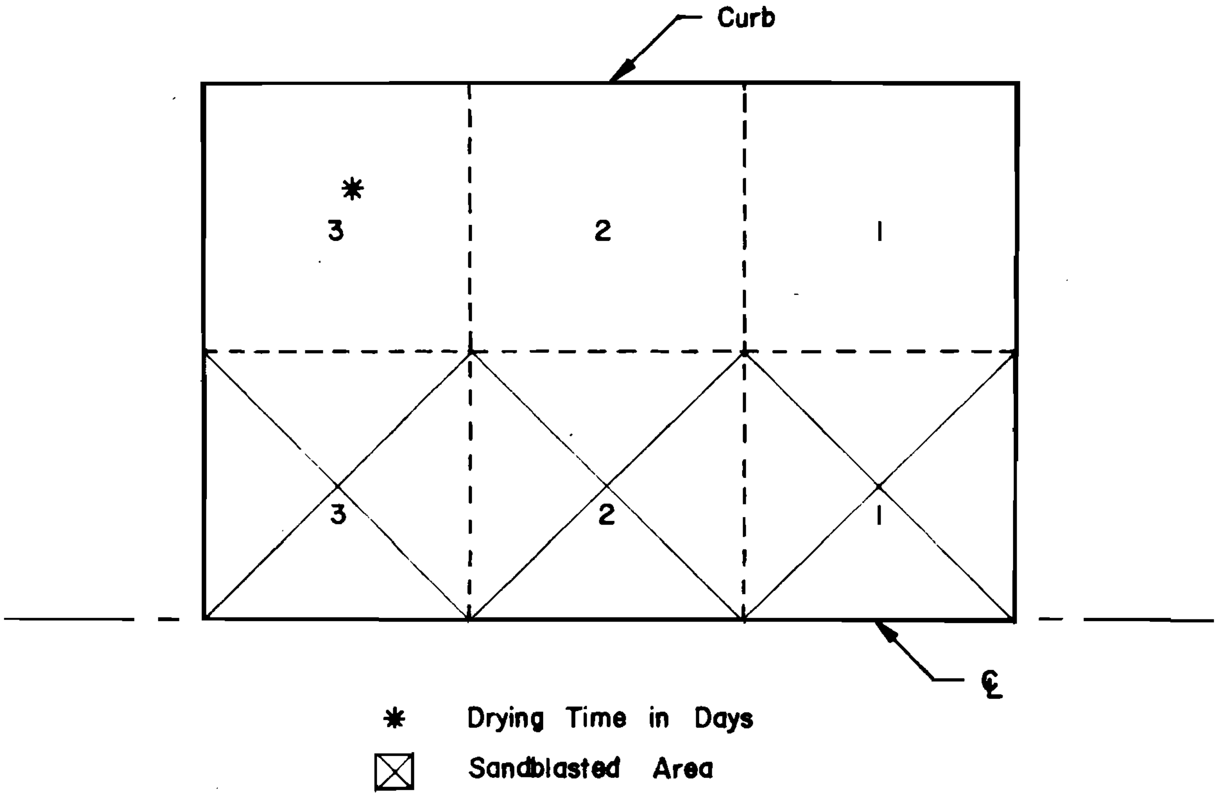


Fig. 7.5 Orientation of Test Sections of Second Bridge Deck.

standpoint of monomer penetration, it is believed that the results of this test would have been equal or better than achieved on the Mopac Bridge if the mechanical failure of the steam generator had not occurred.

7.4 Recommended Procedures for Bridge Deck Partial Impregnation.

The following procedures are recommended for partially-impregnating a bridge deck with polymer under field conditions;

1) Laboratory evaluations should be performed to determine the mechanism of monomer penetration into the concrete bridge deck. It is recommended that at least 2 cores be taken from the bridge. One core should be sandblasted to remove road film. Monomer penetration tests should be performed on both cores to determine whether the sandblasting is necessary.

2) The drying process could be accomplished by any dry-heating methods. The drying process should be done in the insulated enclosure to minimize heat loss. To avoid thermal shock, which may cause serious damage to the bridge deck, the rate of increase in temperature should be less than 2° F/minute and the maximum temperature should not exceed 300° F. A temperature range of 250° F to 280° F is recommended. This temperature range should be maintained for a minimum of 6 hrs. To minimize the unsymmetrical expansion of the decks, the area to be dried at one time should be symmetrical with respect to

the structure center line. The temperature during drying should be monitored for every 100 ft^2. It should be noted that a layer of fine aggregate to a depth of 1/4 to 1/2 in. can be spread over the treated area prior to drying. However, this step can be neglected if dry sand will be used to hold the monomer during the soaking period.

3) After the bridge is cooled to a temperature of less than 85° F, a layer of dry sand to a depth of 1/4 to 1/2 in. should be spread over the treated area to hold the monomer in place. The MMA must not be mixed with the initiator and cross-linking agent more than 1 hr. prior to the application. After mixing, the mixture of MMA should be sprayed over the area to soak it. It is recommended that soaking should start on the uppermost grade of the bridge and the sand must appear saturated. After soaking, the polyethylene membrane should be used to cover the soaked area to reduce evaporation loss of monomer. The soaked area must be protected from direct sunlight because the radiation and high temperature may initiate the polymerization before the desired polymer depth is achieved. The slab must be soaked for a minimum of 4 hrs. Additional monomer must be added if the sand appears to be dried. Open flames are not allowed in the area during the soaking period.

4) After soaking is completed, the monomer must be converted to polymer by the application of heat. Because the monomer vapor is flammable and volatile, the use of steam is recommended. The temperature on the surface of the bridge must be monitored. To minimize a shallow zone below the concrete surface that appears untreated, the temperature at the surface of concrete should reach a temperature of 150° F within 20 minutes and be maintained for at least 60 minutes. The maximum temperature on the surface of the bridge during curing should not exceed 190° F.

CHAPTER 8

VOLUME CHANGE OF POLYMER-IMPREGNATED CONCRETE

8.1 Introduction

Polymer-impregnation has been shown to be an excellent method of improving the strength and durability of concrete However, the impregnation of the concrete with the monomer,which is subsequently converted to polymer,involves a number of volume changes in the concrete. The volume changes, whether during drying, soaking or polymerization, may cause cracks to develop. Cracks and swelling in mortar specimens soaked with MMA were previously reported (Ref. 6). For these reasons,an investigation of the volume changes in concrete at various stages of the impregnation process was performed.

Since the magnitudes of exotherm and of volume change due to the conversion of monomer to polymer depend on the type and amount of the monomer being used, studies of the effect of monomer systems on volume change during the polymer-impregnation treatments were conducted. The volume change is usually specified in terms of change in length from the oven-dried condition before monomer impregnation. The measurements of volume change were made prior to monomer soaking, after soaking, and after polymerization.

Another concern of the effect of volume change in PIC applications involved partial-impregnation. After polymer impregnation, there is an interface between the PIC and the conventional concrete. At elevated temperatures these two materials may expand or contract at different rates and shear stress will be developed at the interface. Under some conditions, this stress may be critical. To obtain better understanding of this problem, experiments were conducted to determine the coefficient of thermal expansion of each type of PIC as well as the unimpregnated concrete.

The effect of concrete quality was also taken into consideration in each test. The monomer and polymer loadings of each specimen were determined.

8.2 Preparation of Specimens and Testing Procedures

Concrete prisms 11-in.x2-in.x2-in. with cement factors of 6.3, 5.0, and 4.0 sk./cu.yd. were cast in steel forms. Reference tabs, approximately 1/8 in. in diameter, were embedded at the center of both ends of the specimens. These reference tabs are compatible with the test apparatus shown in Fig. 8.1.

After the prisms were cast, they were cured in the moist room for 7 days. A pair of the specimens from each batch of concrete was treated with methyl methacrylate (MMA), butyl methacrylate (BMA), and isobutyl acrylate (IBA). Throughout the treatments, 1 % benzoyl peroxide (BP) and 10 % TMPTMA were used as the initiator and cross-linking agent, respectively. After soaking and polymerization, the specimens were weighed to determine the monomer and polymer

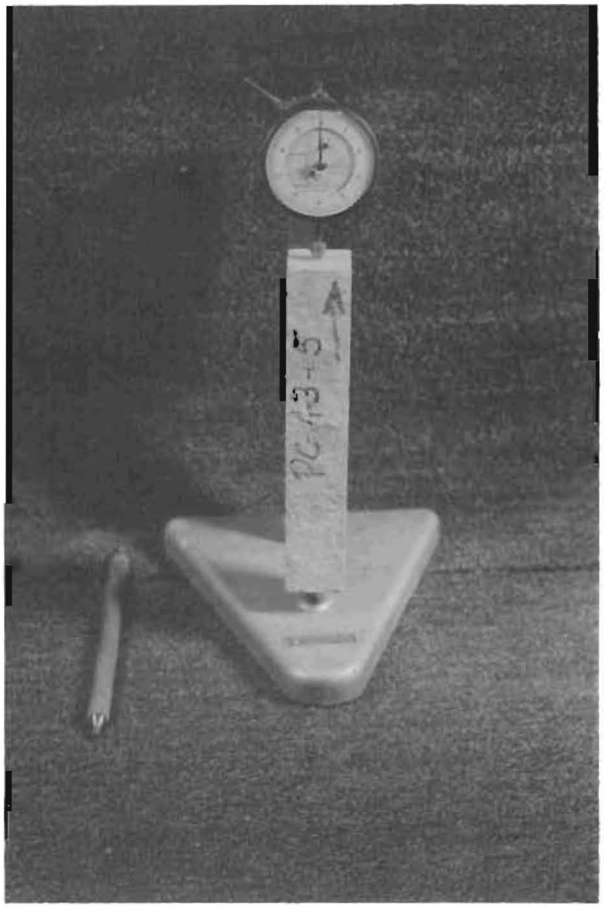


Fig. 8.1 Volume Change Test Apparatus.

loadings for each type of polymer-impregnated concrete.

To investigate the volume changes of concrete during the treatments, the lengths of the specimens and the standard bar were measured as shown in Fig. 8.1. The test temperatures varied from 78°F to 83°F. The measurements and observations were made at the following stages of the impregnation process:

1.) prior to monomer application (dried condition),
2.) after completion of soaking in monomer,
3.) after polymerization, and
4.) after the polymer-impregnated concretes were dried at 212°F to constant weight.

The specimens were weighed at the stages of (1) and (4) to determine the effect of polymer loading on volume changes.

The measurements of the coefficients of thermal expansion were conducted on the concrete with the same water-cement ratios but different batches. The measurements of the lengths of the polymer-impregnated and the unimpregnated concrete were performed at -15, 47, 120, and 156°F.

8.3 Effect of Water-Cement Ratio on Polymer Loading

The monomer and polymer loadings of the prisms of each concrete batch were determined as a percentage of the initial dried weight. The prisms were weighed after drying, after soaking, and after 7 days of drying following impregnation to obtain the initial dried weight, weight of monomer, and weight of polymer, respectively. From these results, the monomer and polymer loadings were determined.

Fig. 8.2 indicates the effect of water-cement ratio on monomer and polymer loading. In general, polymer loading increased as the water-cement ratio increased for all types of monomers. With MMA, the concrete with the water-cement ratio of 6.0 gal./sk. produced a monomer loading of about 4.85 % as compared to 5.35 % for the concrete with the water-cement ratio of 7.5 gal./sk.

The polymer loading was approximately 86 % of the monomer loading,which is an indication of the efficiency of curing 11-in.x 2-in.x2-in. concrete prisms in hot water. However, the 11-in.x2-in.x 2-in. prisms have a relatively high surface area to volume ratio. This efficiency can be improved by many techniques,such as wrapping, coating, or dipping the specimens prior to polymerization.

Fig. 8.3 shows the relationship between polymer loading, in percent of dried weight of the specimens, and the strain,based on the dried, unimpregnated state. It was found that with MMA, the specimen became shorter after polymerization. This result is apparently a result of the shrinkage of MMA during polymerization. The magnitude of strains increases approximately linearly with polymer loading. It can be concluded that for a given monomer and concrete, shrinkage strains in concrete due to polymerization of MMA increase linearly with polymer loading.

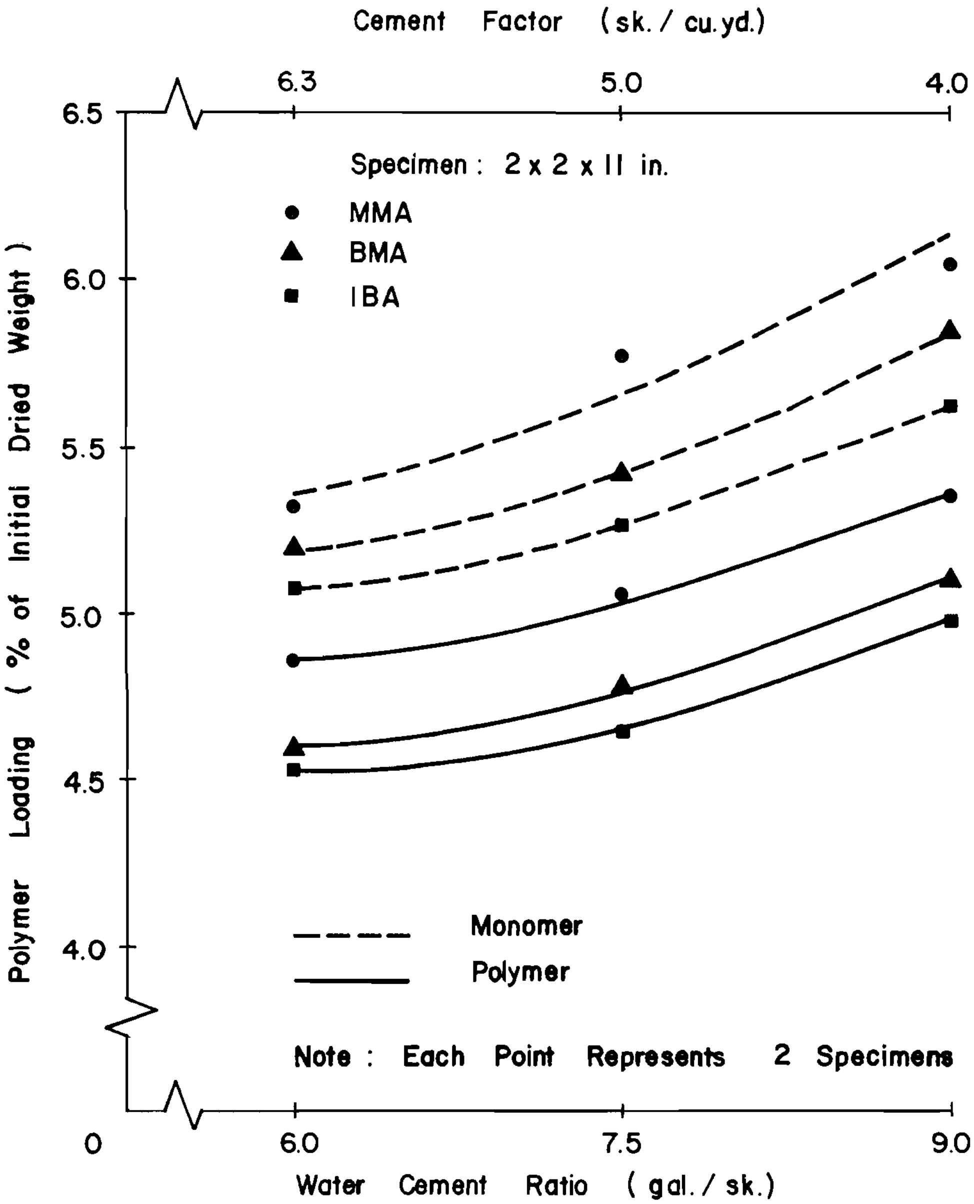


Fig. 8.2 Effect of Water-Cement Ratio on Monomer and Polymer Loading.

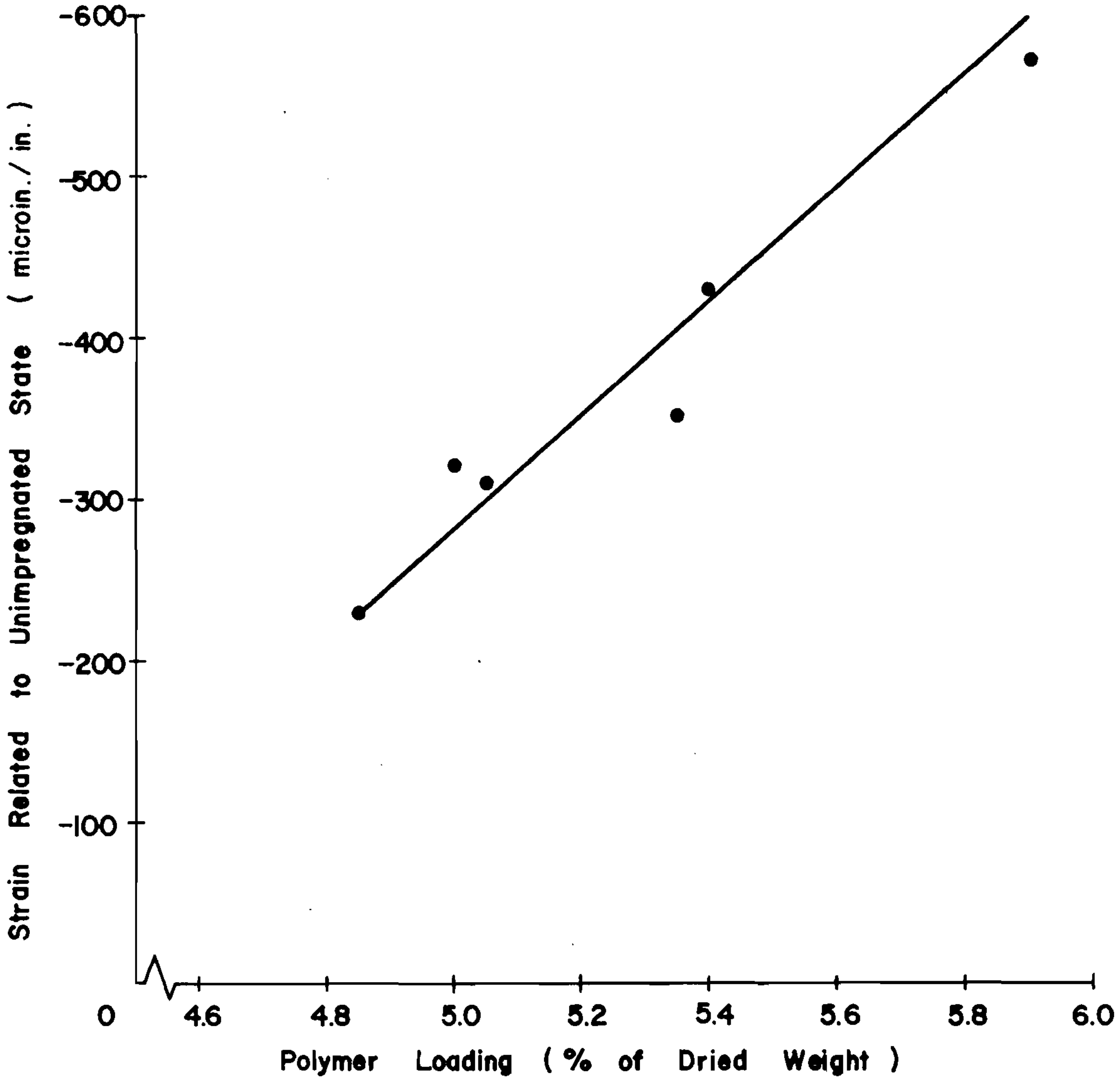


Fig. 8.3 Effect of Polymer Loading on Strain for MMA Treatments.

8.4 Volume Change

8.4.1 Effect of Types of Monomer. The monomers listed in Table 8.1 were used to evaluate the effects of volume change of PIC. From visual observations, it was found that the specimens impregnated with IBA and BMA developed some cracks after polymerization by hot water at approximately 170°F for 17 hrs. The cracks seemed to be more severe with BMA. After the PIC specimens were dried in an oven at 200°F, the cracks were more pronounced. With MMA, no cracks developed for any of the concretes.

Fig. 8.4 shows the test results. Curves show the strains as a function of concrete quality, strains related to the initial state (dried condition) and types of monomer. The results of previous investigations with MMA, IBA, and BMA (Ref. 28) are shown for comparison with this study. The horizontal axis represents the reference line of dried specimens prior to monomer application.

After soaking the specimens with monomer, all specimens expanded in comparison to the initial dried condition. There was no appearance of cracks. The range of strains varied from 10 to 75 microin./in.

After curing in hot water at 170°F for 17 hrs., the MMA and IBA specimens contracted. The maximum magnitudes of strains for MMA and IBA polymer concrete were approximately 360 and 80 microin./in., respectively. The BMA specimens expanded as compared to the

TABLE 8.1 PROPERTIES OF MONOMERS

Monomer	Viscosity, centistokes	Boiling Point, °C	Glass Transition of Polymer, °C (T_g)	Density, gm./cu.cm.
Methyl methacrylate (MMA)	0.55	100	105	0.94
Butyl methacrylate (BMA)	0.86	163	20	0.889
Isobutyl acrylate (IBA)	0.77	145	- 43	0.883

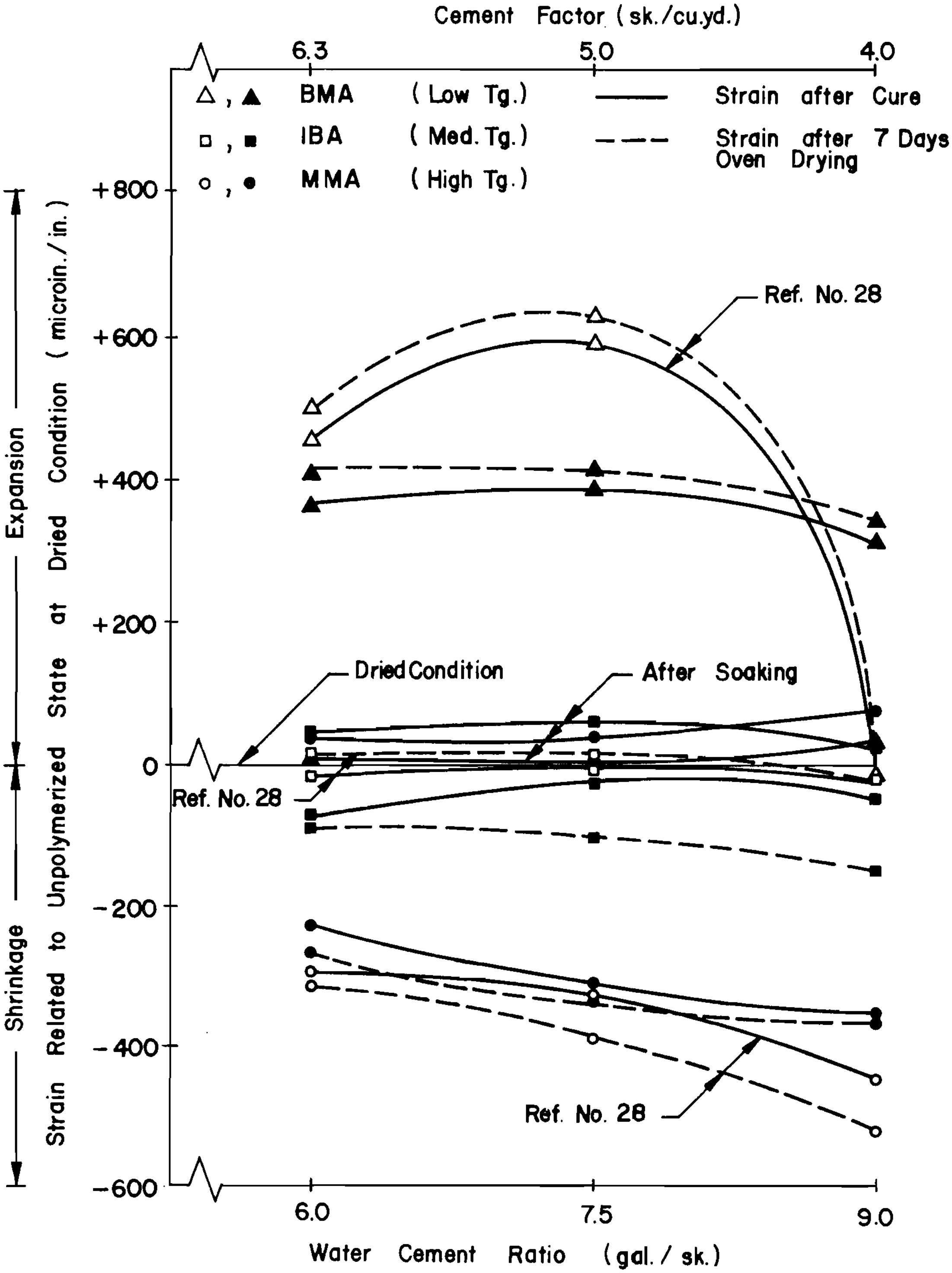


Fig. 8.4 Effect of Concrete Quality on Volume Change of Specimens during and after Treatments.

initial dried condition. The maximum magnitude of strain was 420 microin./in. Fine cracks were found in the IBA and BMA polymer-impregnated concrete at this stage of the process. It is believed that with these two monomers the specimens expanded during polymerization. This expansion was probably caused by the combination of the nature of the transition of monomer to polymer and the temperature resulting from the polymerization process.

After the PIC specimens were dried in the oven at 200°F for 7 days, the MMA and IBA developed more shrinkage, while BMA expanded. In general, the cracks were more pronounced after the final drying than after curing, which was believed to be caused by further polymerization which occurs during the drying process.

8.4.2 Effect of Water-Cement Ratio. The influence of water-cement ratio on volume change during the process of polymer-impregnation is shown in Fig. 8.4.

After soaking, all concrete expanded. The influence of the concrete quality on strain caused by the monomer's presence in the concrete seems to be small. However, there was a trend of increased strains as the water-cement ratio increased. The higher strain in concrete with high water-cement ratios was believed to be caused by higher polymer loading.

After polymerization with MMA the magnitude of strain increases as the water-cement ratio increases. A strain of

250 microin./in. was developed for a 6.0 gal./sk. water-cement ratio concrete as compared to 360 microin./in. for concrete with a water-cement ratio of 9.0 gal./sk. The significance of the effect of concrete qualities on volume change is likely to be decreased with the specimens treated by IBA and BMA. It should be noted that the measurements of lengths of the IBA and BMA specimens were performed on specimens with the visible cracks developed on the surfaces.

8.5 Coefficient of Thermal Expansion

Like most engineering materials, PIC has a positive thermal expansion,but its value depends on the composition of the concrete and the type of monomer. For ordinary concrete, the thermal expansion and the conductivity vary with the temperature and are affected by the properties of the constituents of the concrete. Since the aggregate occupies most of the concrete volume, it mainly determines the thermal characteristic of concrete.

Fully-impregnated concrete contains approximately 5 to 7 % polymer by weight of the initial dried weight of concrete. With the presence of polymer, the thermal characteristics of concrete change. The thermal expansion of PIC is somewhat complicated because of the differential expansion of its constituents. However, attempts were made to measure the coefficients of thermal expansion of MMA, IBA, and BMA polymer-impregnated concretes. Tests were performed on 11-in. x2-in.x2-in. impregnated concrete prisms. The water-cement ratios of the specimens were 6.0, 7.5, 9.0 gal./sk., cement factors of 6.3, 5.0, 4.0 sk./cu.yd. For comparison, experiments were also performed on the unimpregnated specimens with a water-cement ratio of 7.5 gal./sk.

The coefficient of thermal expansion represents the change in volume or, as usually measured on the test specimens, the change in length with the change in temperature. In this research the experiments were conducted at the temperatures of -15, 47, 120, and 156° F. At each test temperature, the specimens as well as the standard bar were suspended in an Invar frame, as shown in Fig. 8.1. The differences in length between the standard bar and the specimens were obtained. Since this experiment was performed at a temperature range of -15° F to 156° F, corrections of the change in length of the standard bar were required. After the correction of the standard bar reading, the changes in lengths of the specimens due to temperature change were obtained by subtracting the difference in readings of the specimens and standard bar at the test temperature from the difference in readings of the specimens and standard bar at the reference temperature.

8.5.1 Standard Bar Calibration. To correct the standard bar reading, the change in length with change in temperature of the standard bar needs to be determined. The tests were performed at the temperatures of 74° F and - 16° F. The length of the standard bar is 11.5 in. The calculations of the change in length for change in temperature of the standard bar are as follow:

Standard bar reading at 74°F = 0.17877

Standard bar reading at - 16°F = 0.17780

Difference in temperature = 90

Difference in reading = 0.00097

Change in length with change in temp. = 1.08×10^{-5} in./°F

Coefficient of thermal expansion = 0.09×10^{-5} in./in./°F

Standard bar reading corrections can be made by multiplying the difference of test temperatures of the standard bar by 1.08×10^{-5}. If the standard bar temperature is less than the reference temperature on the first reading, the correction is added to subsequent standard bar readings. After standard bar readings are corrected, the differences in lengths of the standard bar and the specimens at the test temperatures can be determined.

8.5.2 Test Results. The test results are presented in Fig. 8.5 for all of the concrete qualities. It can be seen that the effect of water-cement ratio on volume change of PIC is small and may be considered negligible for the monomers tested. The change in volume in terms of strains related to the dried condition varies linearly with temperature change for all types of PIC and unimpregnated specimens.

The coefficients of thermal expansion of PIC were computed by using the regression analysis technique. The analyses indicated

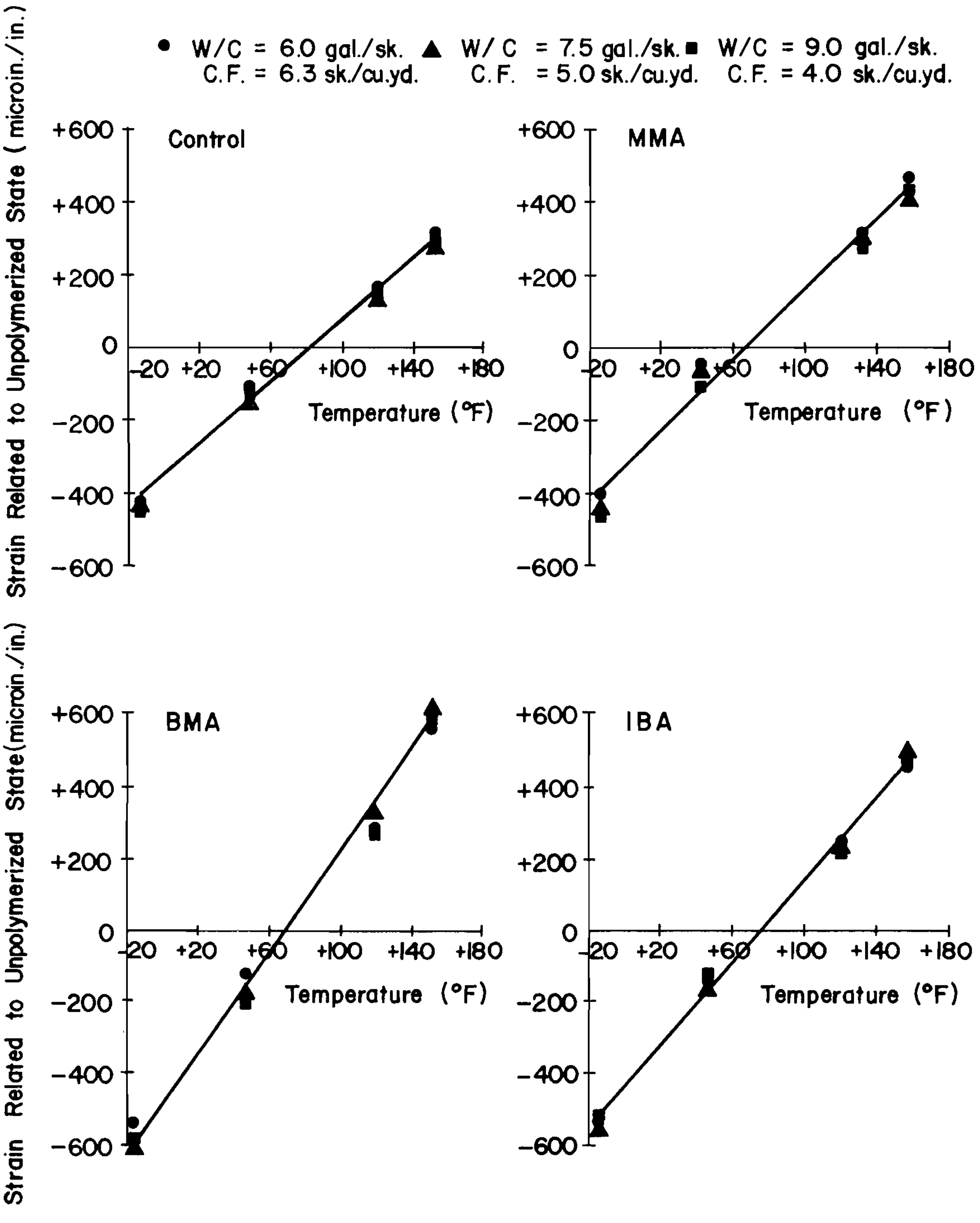


Fig. 8.5 Relationship between Temperatures and Strains of Various Types of PIC and Control.

the linear relationship between the strain and the temperature. It was found that the coefficients of thermal expansion of the control, MMA, BMA, and IBA, were 4.34, 4.93, 6.87, and 5.82 microin./in./°F, respectively. These results are also given in Table 8.2.

It can be seen from Table 8.2 that all three types of PIC indicate higher coefficients of thermal expansion than the control. The BMA indicates 58 % increase in coefficient of thermal expansion as compared to 34 and 14 % increases for IBA and MMA, respectively. These results indicate good agreement with Ref. 16 which found that the coefficient of thermal expansion of PIC was higher than the control. Ref. 45 also found that the coefficient of thermal expansion of MMA-treated specimens was 5.25 versus 4.0 microin./in./°F for the control, or an increase of 31 %. The coefficient of 4.93 obtained for MMA in this study is 6 % less than the value found in Ref. 45.

8.6 Summary

In conclusion, it was found that the most critical volume change occurs during the polymerization process. High water-cement ratio concrete achieves higher polymer loading than low water-cement ratio concrete. The strains were not sufficiently high to cause cracking in MMA-impregnated concrete although cracking did occur in BMA and IBA-impregnated concrete. The cracks were more severe with BMA-impregnated specimens. Polymer-impregnated concrete has positive

TABLE 8.2 COEFFICIENT OF THERMAL EXPANSION OF PIC AND CONTROL

PIC	Water-Cement, gal./sk.	Polymer Loading, %	Coefficient of Thermal Expansion, $\alpha \times 10^{-6}$	$\frac{\alpha\ \text{PIC}}{\alpha\ \text{control}}$
MMA - 1	6.0	5.0	4.93*	1.14
MMA - 2	7.5	5.4		
MMA - 3	9.0	5.9		
BMA - 1	6.0	4.7	6.87*	1.58
BMA - 2	7.5	5.1		
BMA - 3	9.0	5.7		
IBA - 1	6.0	4.5	5.82*	1.34
IBA - 2	7.5	4.7		
IBA - 3	9.0	5.3		
Control	7.5	-	4.34	1.00

* Based on the average of all specimens.

thermal expansion and varies linearly with increasing temperature. After polymerization the effect of water-cement ratio on coefficient of thermal expansion is negligible. As compared to the control, BMA, IBA, and MMA-impregnated concrete indicated 58, 34, and 14 % increases in coefficients of thermal expansion, respectively. The relationship between the strain, after polymerization, related to initial dried condition, and the polymer loading is linear.

CHAPTER 9

STRENGTH OF POLYMER-IMPREGNATED CONCRETE

9.1 Introduction

Polymer-impregnated concrete, as previously defined in chapter 2, is a precast hydrated cement concrete which has been impregnated with a low viscosity monomer and polymerized by radiation or by a thermal-catalytic process. Developments of other concrete-polymer composites as new construction materials are also under investigation. However, within the limits of previous test results (Ref. 17), the polymer-impregnated concrete (PIC) has the highest improvement in strength, stiffness, and durability properties.

The primary purpose of this chapter is to discuss the strength and stiffness of PIC as compared to unimpregnated control concrete. The compressive stress-strain relationships within the elastic range of the control and many types of PIC were determined. The effect of concrete mixes on strength and stiffness of PIC was evaluated. All tests were conducted on 3 x 6-in. concrete cylinders.

Two fully-impregnated beams were also tested to determine the behavior of PIC beams under flexural loading and results were compared to the control beams.

9.2 Preparation of Specimens

9.2.1 Specimens. To determine the strength and stiffness of various types of PIC, concrete cylinders (3 x 6 in.) with a water-cement ratio of 6.5 gal./sk., a cement factor of 5.6 sk./cu.yd., a slump of 5 in., and a 28-day moist-cured compressive strength of 5150 psi. were cast. After air curing for more than 3 months, the specimens were fully-impregnated with the monomer systems, which consisted of the monomer with 1 % (wt.) BP and 10 % (wt.) TMPTMA. The method of full-impregnation is described in Appendix 4. The types of monomers used are as follows:

1) methyl methacrylate (MMA),
2) butyl acrylate (BA),
3) isobutyl acrylate (IBA),
4) butyl methacrylate (BMA),
5) isobutyl methacrylate (IBMA), and
6) 45 % MMA + 55 % BA (Co-polymer)

To determine the effect of concrete qualities on strength and stiffness of PIC, three mixes with different water-cement ratio were designed, as shown in Table 9.1.

To determine the flexural strength of reinforced PIC beams, reinforced concrete beams with a water-cement ratio of 8.0 gal./sk., a cement factor of 4.5 sk./cu.yd., and a 28-day compressive strength of 3650 psi. were cast. After the specimens were air-cured for

TABLE 9.1 CONCRETE QUALITIES

Batch No.	Water-Cement Ratio, gal./sk.	Cement Factor, sk./cu.yd.	Slump, in.	Compressive Strength, psi.
PC-32	6.0	6.3	4.5	6014
PC-33	7.5	5.0	5.0	4481
PC-35	9.0	4.0	4.0	2700

several months, they were fully-impregnated with the same mixture of MMA, BP, and TMPTMA.

9.2.2 Full-Impregnation Techniques. The basic procedure for fully-impregnated precast concrete, described in detail in Appendix 3, consists of oven drying the concrete to constant weight at a temperature of about 212° F. After drying, the specimens were wrapped with polyethylene to minimize absorption of moisture from the air until the concrete had cooled to room temperature.

After cooling, the specimens were placed in a vacuum of 27 in. of Hg for about 17 hrs. After the completion of the evacuation, they were soaked with the mixture of monomer for 5 hrs..

After soaking, the 3 x 6-in. cylinders were placed under hot water for 24 hrs. to complete polymerization. The temperature of the hot water was maintained at a minimum of 140° F. The reinforced beams were cured in a steam chamber for 1 hr. at approximately 170° F.

9.3 Stress-Strain Curves of PIC

The compressive stress-strain curves of plain concrete and PIC in the elastic range are shown in Fig. 9.1. The polymer ranged from rubbery (BA) to glass-like (MMA). It should be noted that cracks were observed in the co-polymer and BMA specimens after polymerization.

The moduli of elasticity of PIC and control were calculated from the stress-strain relationship obtained during the compression

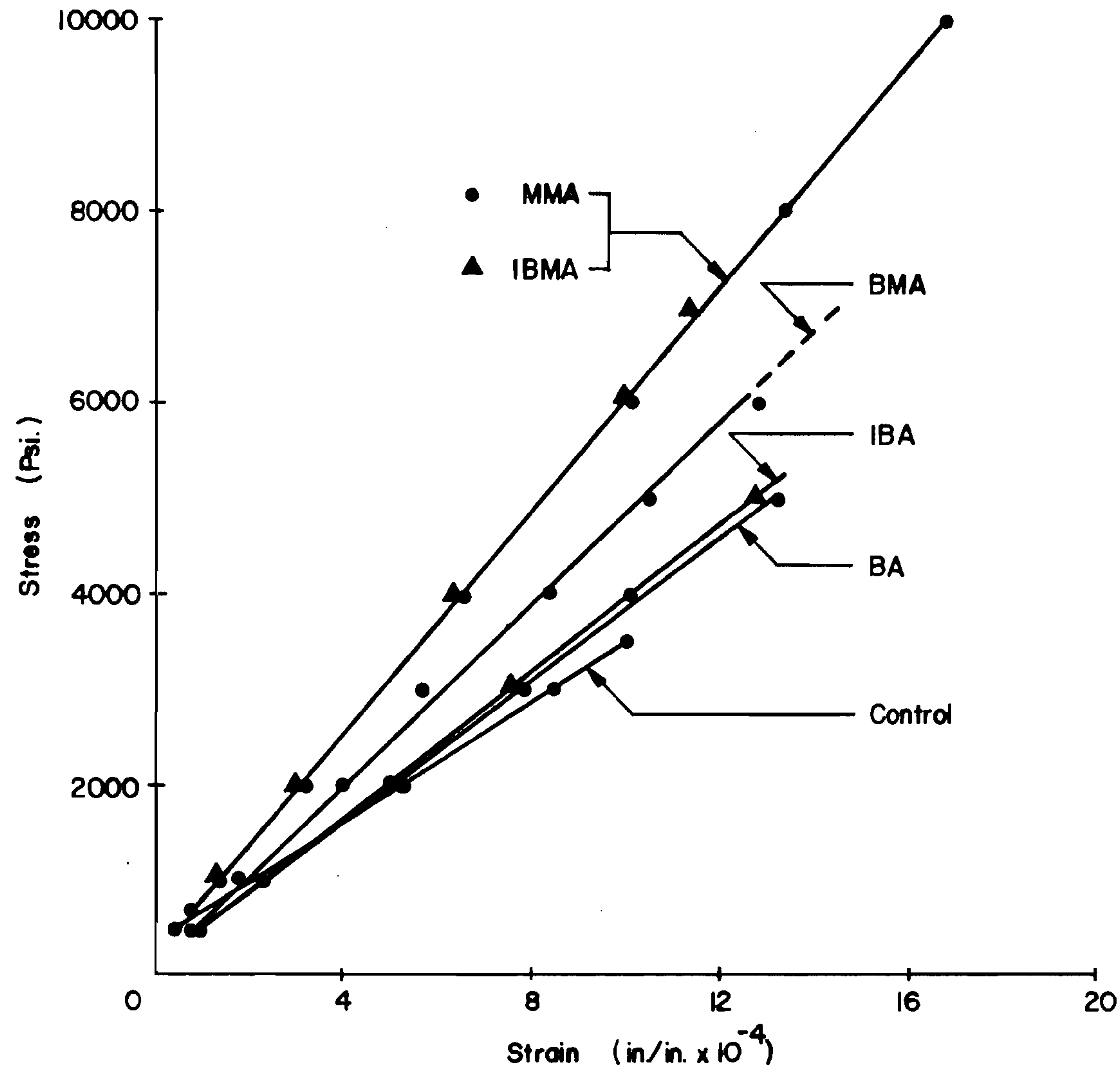


Fig. 9.1 Compressive Stress-Strain Curves of Control and PIC (Elastic Range).

tests. The first of the three specimens was loaded to ultimate load without the strain apparatus attached. The stress-strain relationships was measured on the other two specimens by means of a dial guage extensometer with a 4-in. guage length. The apparatus was removed at about 55 % of the anticipated ultimate load to prevent damage to the dial guages (Ref. 3).

The modulus of elasticity (E), compressive strength (f'_c), and ultimate split-tensile strength of each type of PIC and control are shown in Table 9.2. Each value in Table 9.2 represents the average for three specimens except the modulus of elasticity, which represents the average of two specimens. It was found that the modulus of elasticity of the PIC ranged from 3.63×10^6 to 6.00×10^6 psi. as compared to 3.10×10^6 for the control. The highest moduli of elasticity were obtained with the specimens treated by MMA and IBMA. Both types show the increase in modulus of elasticity of about 94 % over the control. The butyl acrylate (BA), which was the most rubbery polymer in this study, indicated the smallest increase in modulus of elasticity (≈ 17 %). Attempts were made to correlate the E to the $\sqrt{f'_c}$. It can be seen from Table 9.2 that E of PIC is in the range of $39{,}800\sqrt{f'_c}$ to $48{,}580\sqrt{f'_c}$ as compared to $42{,}470\sqrt{f'_c}$ for the control.

The improvement in structural properties was of significant interest in this part of the study. It was found that the highest improvements in compressive strength, and split tensile strength were obtained with the specimens treated by IBMA and MMA, respectively. The specimens treated by IBMA showed a 191 % increase in compressive strength over the control as compared

TABLE 9.2 STRUTURAL PROPERTIES OF PIC

Monomer	Glass-Transition Temperature, C	Polymer Loading, %	Compressive Strength f'_c, psi		Tensile Strength, psi		Modulus of Elasticity E, psi x 10^6		$E = X\sqrt{f'_c}$ X Values***
			PIC	PIC/C*	PIC	PIC/C*	PIC	PIC/C*	
Butyl acrylate	- 54	5.05	8,317	1.56	835	1.75	3.63	1.17	39,800
Isobutyl acrylate	- 43	4.77	8,775	1.65	785	1.65	3.74	1.21	39,900
Butyl methacrylate**	+ 20	4.96	12,750	2.39	-	-	4.61	1.49	40,980
Methyl methacrylate	+ 105	5.55	15,250	2.86	1,038	2.18	6.00	1.94	48,580
Isobutyl methacrylate	+ 48	4.83	15,500	2.91	940	1.97	6.00	1.94	48,190
45 % MMA + 55 % BA**	- 3	5.07	12,400	2.33	-	-	-	-	-
Control (C)	-	-	5,330	1.00	477	1.00	3.10	1.00	42,470

* Control

** Specimens cracked after curing

*** Modulus of elasticity as a function of square root of compressive strength.

to 186 % for the specimens treated by MMA. In contrary, the specimen treated with MMA showed a 118 % increase in split tensile strength over the control as compared to 97 % for the specimens treated by IBMA. However, from an overall standpoint, the MMA is more favorable than IBMA because:

1) The price of IBMA is higher than that of MMA (Ref. 3).
2) The viscosity of MMA is 0.55 centistokes as compared to 0.83 centistokes for IBMA.

It should be noted that the strength of PIC obtained in this chapter is rather low as compared to other studies (Refs.16-20). The strength can be increased by the application of pressure during the soaking period. However, because the primary purpose of this report is to determine the strength of PIC obtained with partial impregnation under field conditions and pressure is not feasible under these conditions, there was no application of pressure during the soaking period for the full-impregnation process.

Fig. 9.2 shows the stress-strain relationship of control and MMA-impregnated concrete obtained from Ref. 47. The specimens were 3 x 6 in. cylinders. The MMA-polymer loading was 5.4 % by weight. It can be seen that the impregnation of polymer into concrete changes the conventional concrete from plastic to elastic behavior. This is indicated by the linearity of the stress-strain of PIC in Fig. 9.2. At approximately 85 % of ultimate load, the PIC begins to indicate evidence of plastic behavior. From Fig. 9.2, the ultimate strain of PIC is about 3300 as compared to 2600 microin. /in. of the control.

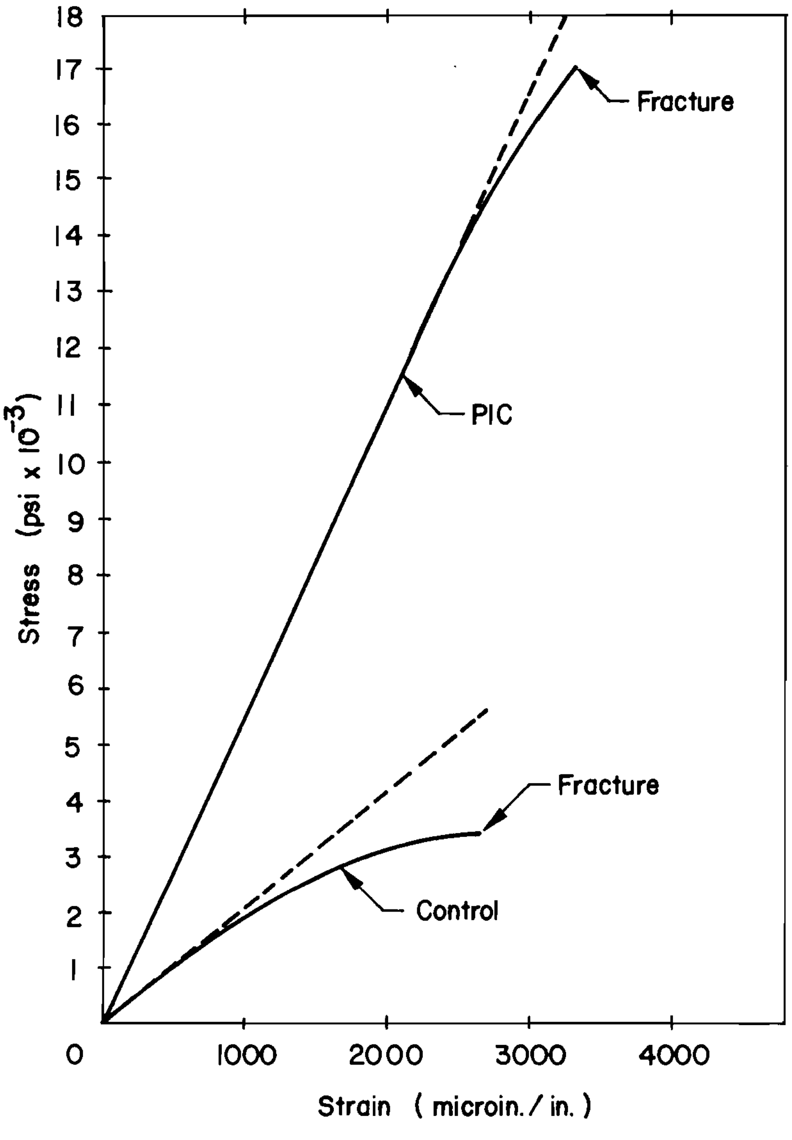


Fig. 9.2 Stress-Strain Relationship of Control and MMA-Impregnated Concrete (Ref. 47).

Fig. 9.3 indicates the effect of glass-transition temperature of polymers on compressive strength and modulus of elasticity of PIC. The glass-transition temperature (T_g) is the point which the polymers change from glass-like behavior at below T_g to soft, rubbery behavior as the temperature is raised above the T_g (Ref. 21). It can be seen from Fig. 9.3 that the ratio of compressive strength and modulus of elasticity of PIC to the control increased as the glass-transition temperature increased. It should be noted that all tests were performed in the laboratory and the temperature ranged from 70° F to 90° F.

9.4 Effect of Concrete Qualities on Structural Properties of PIC

Three types of monomers were selected to evaluate the effect of concrete quality on polymer loading, strength, and stiffness of PIC. Selections of the monomers were based on the characteristics of the polymers which were obtained after polymerization of the monomers. The characteristics of polymers were glass-like (MMA), intermediate (BMA), and rubbery (IBA) materials.

The results of tests on three different concrete qualities are shown in Table 9.3. It was obvious that the polymer loading increased as the water-cement ratio increased for all types of PIC. The specimens treated with MMA, which were of particular interest, indicated a polymer loading of 14 % greater for the concrete having

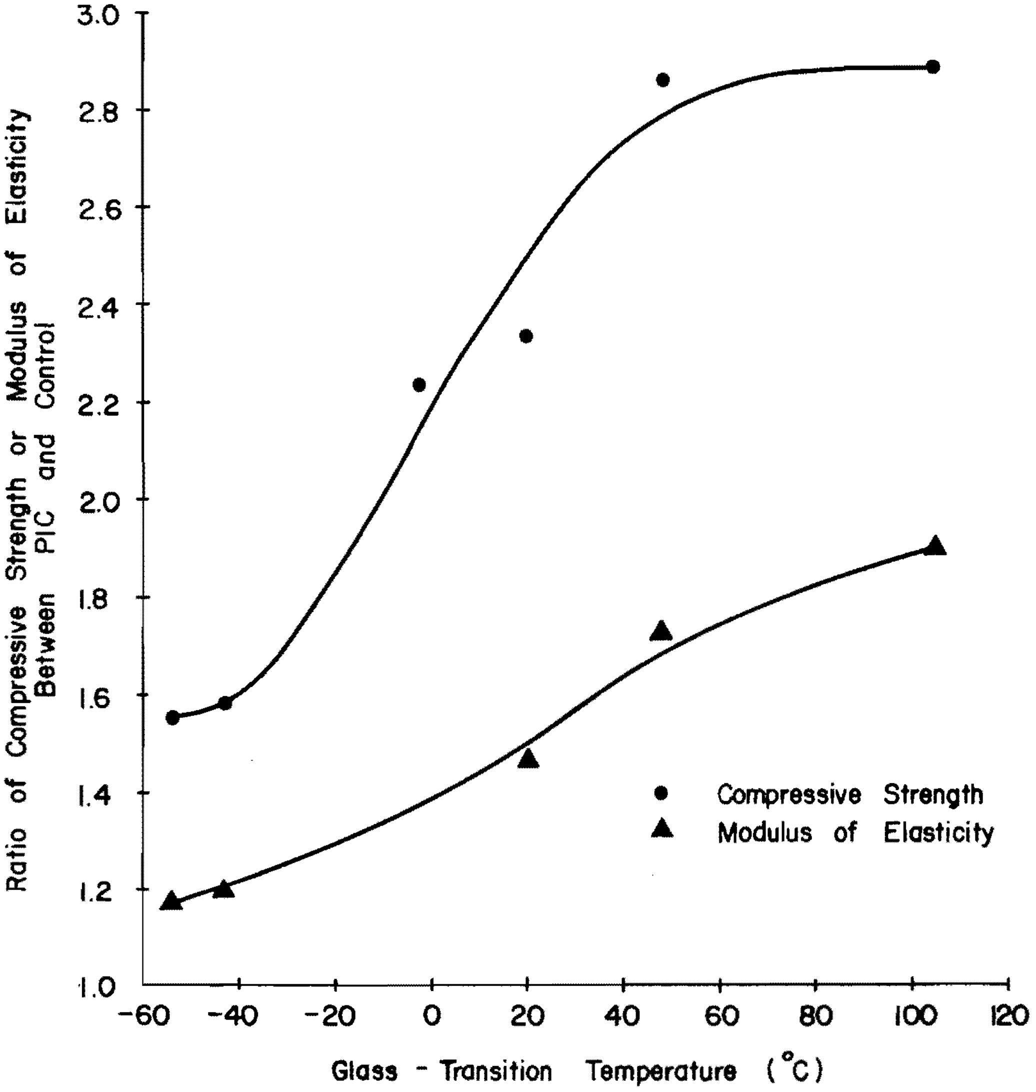


Fig. 9.3 Effect of Glass-Transition Temperature of Monomer on Strength Ratio of PIC and Control.

TABLE 9.3 PROPERTIES OF FULLY-IMPREGNATED SPECIMENS

Monomer[b]	Water-Cement Ratio, gal./sk.	Cement Factor, sk./cu.yd.	Polymer Loading, %	Compressive Strength, f'_c, psi	Tensile Strength, psi	Modulus of Elasticity, E, psi x 10^6	PIC[c] Control	$E = X\sqrt{f'_c}$ X Values[d]
Control	6.0	6.3	-	6,035	533	3.12	-	40,150
	7.5	5.0	-	4,309	476	3.00	-	45,730
	9.0	4.0	-	3,027	368	2.84	-	51,640
MMA	6.0	6.3	5.40	13,285	921	5.90	2.09	51,170
	7.5	5.0	5.75	13,994	1,037	5.83	2.25	49,320
	9.0	4.0	6.16	14,363	916	5.50	4.75	45,910
IBA	6.0	6.3	4.94	9,545	885	4.17	1.50	42,680
	7.5	5.0	5.58	8,082	878	3.75	1.88	41,670
	9.0	4.0	5.73	6,514	674	2.86	2.15	35,400
BMA[b]	6.0	6.3	4.99	9,362	798	-	1.55	-
	7.5	5.0	5.68	6,796	726	-	1.58	-
	9.0	4.0	6.04	8,307	640	-	2.74	-

[a] Monomer systems included 1 % (wt.) BP and 10 % (wt.) TMPTMA.

[b] Cracks were observed on the surface after polymerization.

[c] Ratio of compressive strength of PIC to control.

[d] Modulus of elasticity as a function of square root of compressive strength.

the water-cement ratio of 9.0 gal./sk. as compared to 6.0 gal./sk. water-cement ratio concrete.

The compression tests were conducted in accordance with the American Society of Testing Material (ASTM) C-39-64 . The control strengths decreased with increasing water-cement ratio. The IBA specimens indicated some decrease in compressive strength as the water-cement ratio increased. On the contrary, the specimens treated with MMA developed modest compressive strength increases as the water-cement ratio increased. These results agree with the previous results which were found by Fowler (Ref. 7). It was also verified by the recent conclusions of Dikeou (Ref. 45) and Manning (Ref. 23) that the presence of polymer (MMA) in concrete increases the strength of initially lower strength concrete to nearly or equal that of the impregnated, initially higher strength concrete.

The tensile strengths were determined in accordance with ASTM C 496. It can be seen that the tensile strengths of IBA decreased as the water-cement ratio increased. The specimens treated with MMA showed the highest tensile strengths in the specimens with a water-cement ratio of 7.5 gal./sk. The greatest improvement in tensile strength was obtained with specimens impregnated with MMA.

The moduli of elasticity of MMA and IBA were found to decrease as the water-cement ratio increased. Attempts were made to correlate the modulus of elasticity of PIC to the square root of compressive strength ($\sqrt{f'_c}$). As shown in Table 9.2 and 9.3, the modulus of elasticity of PIC is in the range of $35{,}400\sqrt{f'_c}$ to $51{,}200\sqrt{f'_c}$.

The BMA specimens were tested under splitting tension and compressive loading. However, there was no measurements of the stress-strain relationship because cracks were observed on the surface of specimens after the completion of polymerization. There was some indication of a slight decrease in tensile strength as the water-cement ratio increased. Under compressive loading, the specimens with a water-cement ratio of 6.0 gal./sk. indicated the highest compressive strength.

Fig. 9.4 shows the effect of polymer loading on compressive strength of MMA-impregnated concrete. The polymer loading was varied by varying the soaking and drying times. The water-cement ratios of concrete mixes ranged from 6.0 to 9.0 gal./sk. All tests were performed on 3 x 6-in. cylinders. Each point represented the average of three tests. It can be seen that the effect of polymer loading on compressive strength was significant. This result agrees well with results of previous research which were found by Sopler (Ref. 60). At 6 % (wt.) polymer loading, the MMA-impregnated concrete provided a compressive strength 215 % higher than the control.

Attempts were also made to use the polynomial regression analysis to predict the ratio of the compressive strength of PIC to that of the control from the polymer loading. The following equations were obtained:

a) First degree regression

$$P = 0.989 + 0.333\,PL \quad \ldots\ldots\ldots\ldots \quad 9.1$$

$$R^2 = 0.904$$

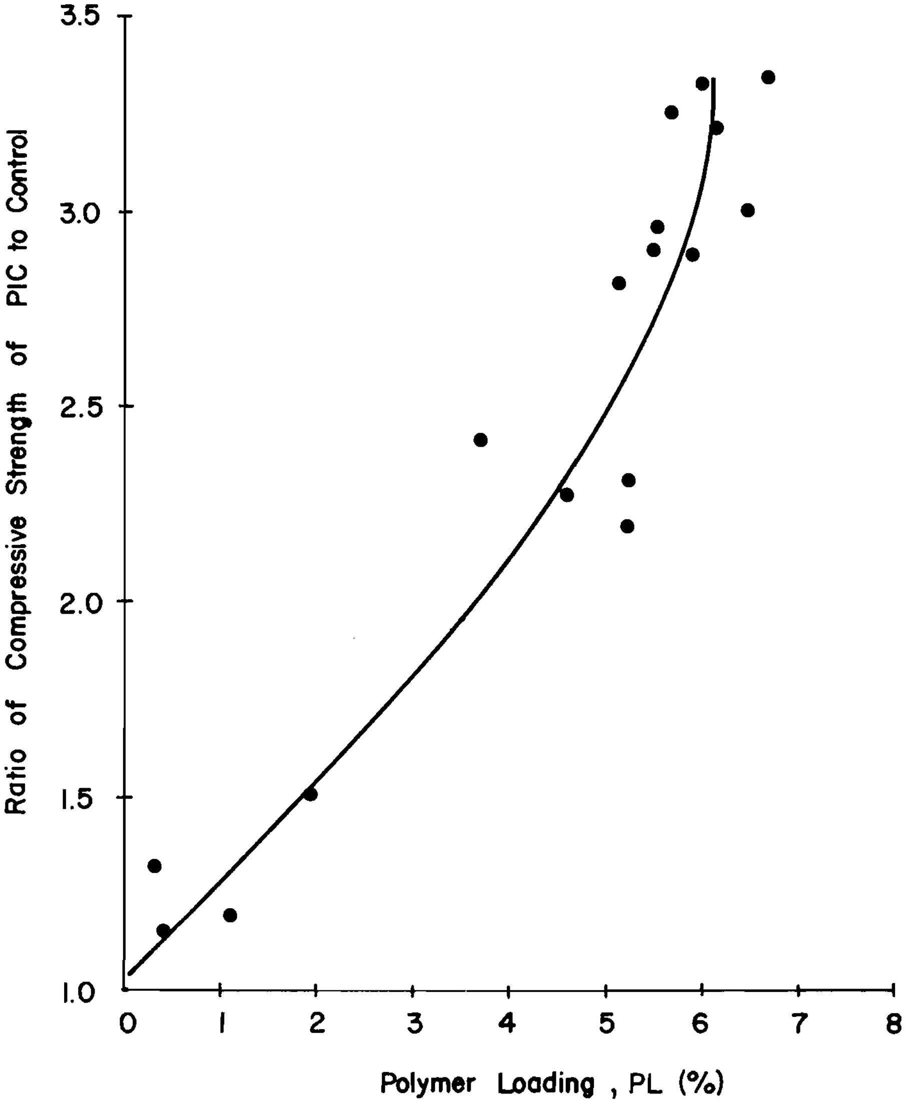


Fig. 9.4 Effect of Polymer Loading on Strength Ratio of MMA to Control.

b) Second degree regression

$$P = 1.088 + 0.194\,PL + 0.021\,(PL)^2 \quad \ldots\ldots \quad 9.2$$

$$R^2 = 0.911$$

Where:

P = ratio of compressive strength of PIC to control,

PL = polymer loading (wt.) %

Based on the simplicity of the equation and the coefficient of regression correlation, the following equation is recommended:

$$P = 1 + 1/3\,(PL) \quad \ldots\ldots\ldots\ldots\ldots \quad 9.3$$

9.5 Fully-Impregnated Beams

A limited number of beam tests were conducted to determine the behavior of PIC under flexural loading. Each rectangular beam was 4.0 in. wide, 5.5 in. deep, and 60 in. long, reinforced with one No. 5 bar. The length of reinforcing bar was 56 in. The beams were designated as Beam I and Beam II in accordance with the location of reinforcing bars. The clear covers from the tension sides of Beam I and Beam II were 1.5 in. and 3.5 in., respectively. The beams were kept moist and covered for 7 days and stored in the laboratory until treated and tested. It should be noted that these beams were prepared for use in the study which investigated the behavior of partially-impregnated reinforced beams under cyclic loading (Ref. 25). Ref. 25

contains more details of beam fabrication and material properties.

Because of the very limited number of beams tested, it is necessary to make it clear that the purpose of this part of the study was only to provide a basic insight into the behavior of rectangular reinforced PIC beams under flexural loading. These tests were conducted because no other structural tests on reinforced PIC beams have been reported. The test results are not conclusive. However, an investigation of the behavior of fully-impregnated reinforced beams is continuing at The University of Texas at Austin.

The behavior of partially-impregnated reinforced beams under static and cyclic loading was reported in Ref. 25. It was found that the beams partially-impregnated with MMA on the compression side indicated higher strength (10 to 15 %) than the controls.

9.5.1 Preparation of Specimens. The beams were dried in an oven to constant weight at a temperature of about 212° F. After drying they were weighed and placed in a vacuum chamber for the full-impregnation process. The beams were fully-impregnated with the usual mixture of MMA with 1 % (wt.) BP and 10 % (wt.) TMPTMA. The techniques of full-impregnation are described in Appendix 4.

After polymerization was achieved, these beams were again dried in the oven and weighed. The polymer loadings of Beam I and Beam II were 4.8 and 4.85 %, respectively.

9.5.2 Test Procedure. The ultimate compressive strength, split tensile strength, and modulus of elasticity of the control concrete and PIC were first determined. The tests were conducted on 3 x-6 in. cylinders because of the limitation of the machine capacity for the high strength PIC specimens. The yield strengths and moduli of elasticity of reinforcing bars were previously reported in Ref. 25.

The beams were subjected to static tests to failure to determine the ultimate loads for each beam. The beam dimensions and loading are shown in Fig. 9.5. The deflections were measured at the supports, at third points, and at mid span. Six dial guages were used to measure the deflections and were removed at approximately 75 % of the ultimate load to avoid damage.

9.5.3 Materials. Based on 3 x 6-in. cylinders and tests conducted in Ref. 25, the structural properties of the concrete, PIC and reinforcing bars are listed as follows:

a) Control

Compressive strength	4400	psi.
Split tensile strength	473	psi.
Modulus of elasticity	3.29×10^6	psi.

b) PIC

Compressive strength	13,000	psi.
Split tensile strength	896	psi.
Modulus of elasticity	5.62×10^6	psi.

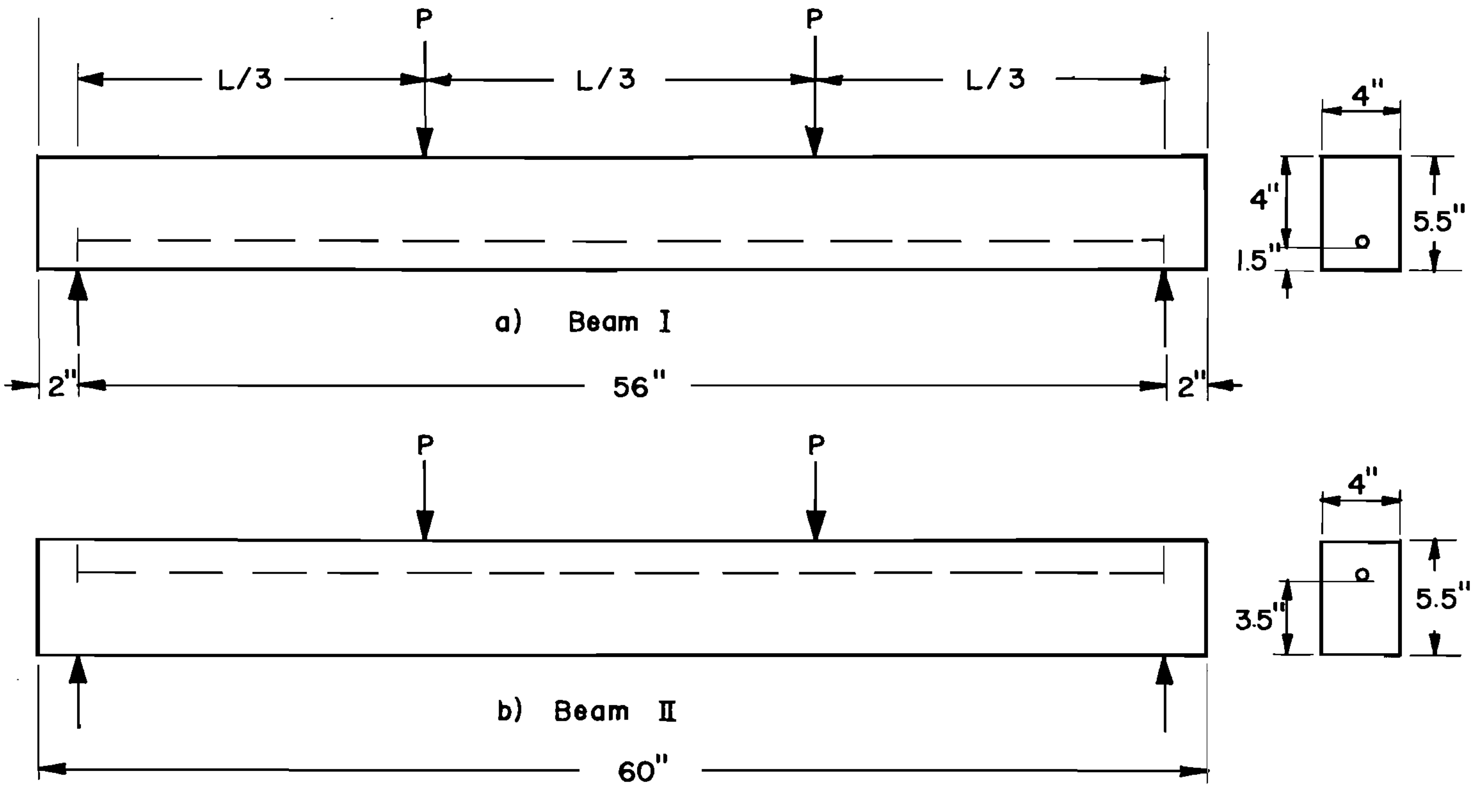


Fig. 9.5 Orientations and Loading Pattern of PIC Beams.

c) Reinforcing bars (Ref. 25)

Yield strength	68,700	psi.
Ultimate strength	111,500	psi.
Modulus of elasticity	27.84×10^6	psi.

9.5.4 Beam Tests. Fig. 9.6 shows the load deflection curves of Beam I and the control. The first visible crack was observed at 5 kips. As the load increased, more cracks developed. At 8.4 kips, all dial guages were removed and loading was continued. Finally the beam failed at the ultimate load of 10.4 kips. The mode of failure of Beam I is shown in Fig 9.7 (a). At ultimate load, cracks extended from the points of loading to the supports. The failure mode was primarily diagonal tension.

For the purpose of comparison, the load-deflection response of the control beam from Ref. 25 was also plotted in Fig. 9.6. It should be noted that the load deflection which was obtained from Ref. 25 was modified because of the difference in loading. The two point loads were 24.4 in. from each support rather than 18.7 in. for the PIC beams. By using the deflection formula $\Delta = \frac{Pa\,(3L^2 - 4a^2)}{24\,EI}$ and assuming the loads are still within the elastic range, the conversion of the deflections at 24.4 in. to those at 18.7 in. (L/3) from the supports could be made. In comparison, it could be seen that to produce a 0.2 in. deflection at mid span, the PIC required 56 % higher load than the control. In comparison with the control which failed in diagonal-tension at 6.57 kips (Ref. 25), the PIC beam indicated 58.3 % higher load than the control beam.

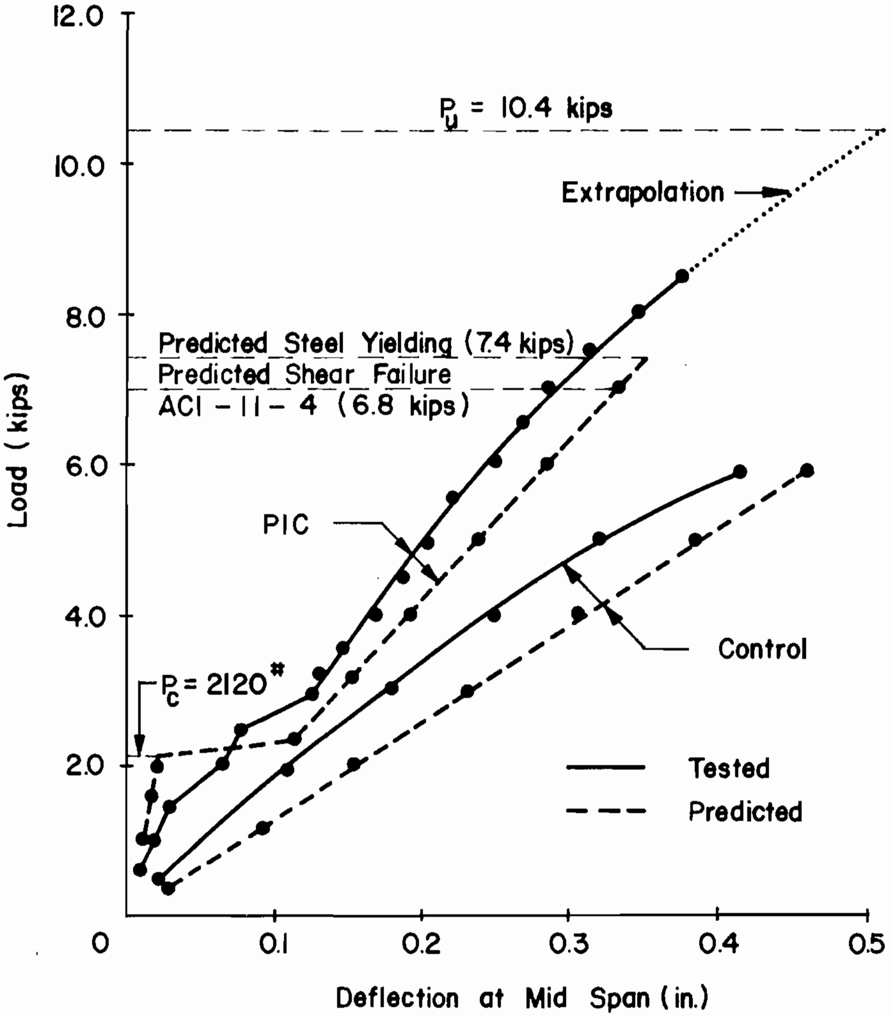
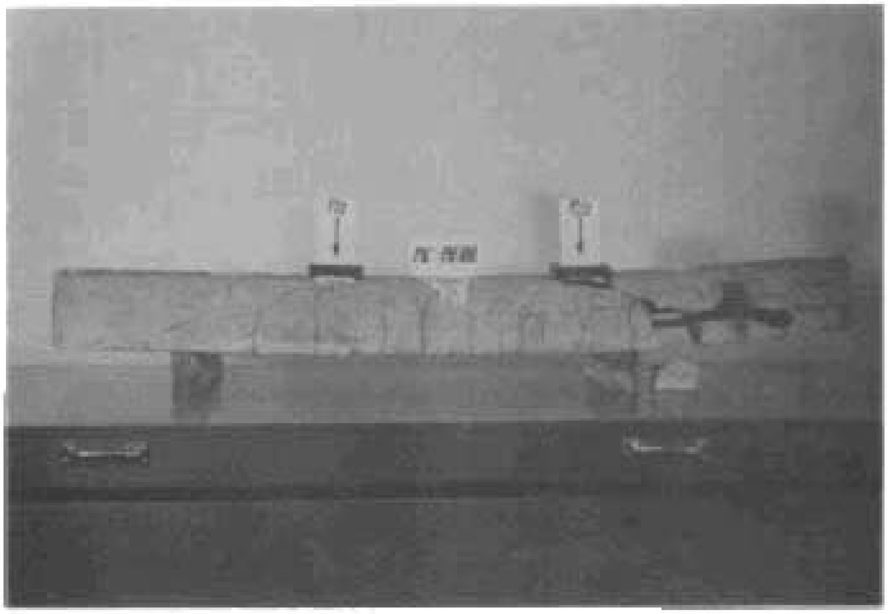
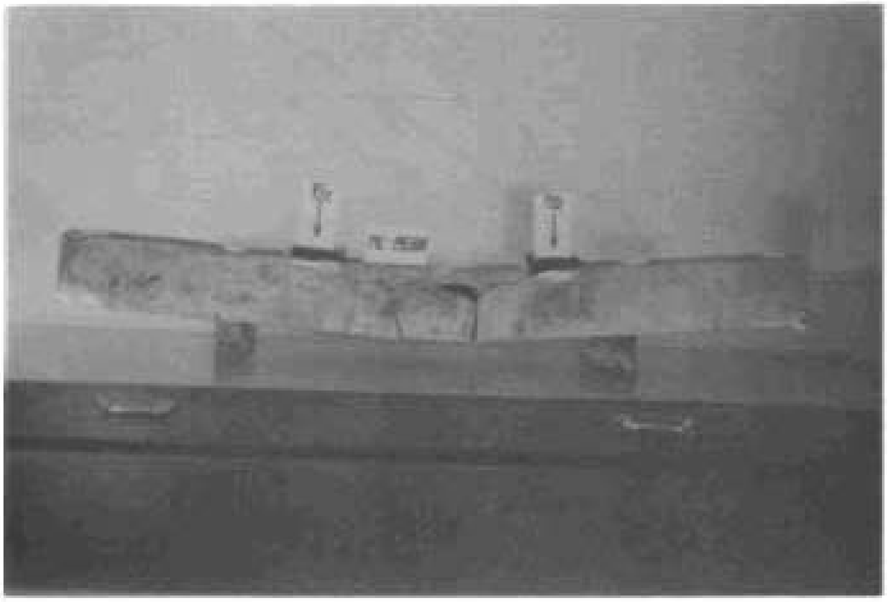


Fig. 9.6 Load-Deflection Curves of PIC Beam I and Control.



a) Beam I



b) Beam II

Fig. 9.7 Modes of Failure of Beam I and Beam II.

The orientation and loading pattern of the second beam are shown in Fig. 9.5 (b). The first crack was observed at 1900 lbs. of loading. At this point, the deflection increased and the load decreased and stabilized at 1000 lbs.. The tensile cracks propagated to the reinforcing bar. After the beam was in an equilibrium condition, it was reloaded to the ultimate load of 3700 lbs. The failure of this beam could be divided into 2 stages,as follows:

1) At a load of 1900 lbs., flexural tensile cracks developed between the load points and propagated to the reinforcing bar. After these cracks developed the deflection increased significantly.
2) At an ultimate load of 3700 lbs, failure occurred in the compression zone between the points of loading,as shown in Fig. 9.6 (b).

The load-deflecton curve of Beam II is shown in Fig. 9.8. The deflections were monitored to a load of 2400 lbs.

Figs. 9.6 and 9.8 also show the comparison of experimental and predicted load-deflection curves. To predict the relationship between the loads and deflections of both beams, the following assumptions were made:

1) At the load range from 0 to first crack, the moment of inertia (I) of PIC beam was calculated based on the transformed gross section of concrete using the elastic moduli.

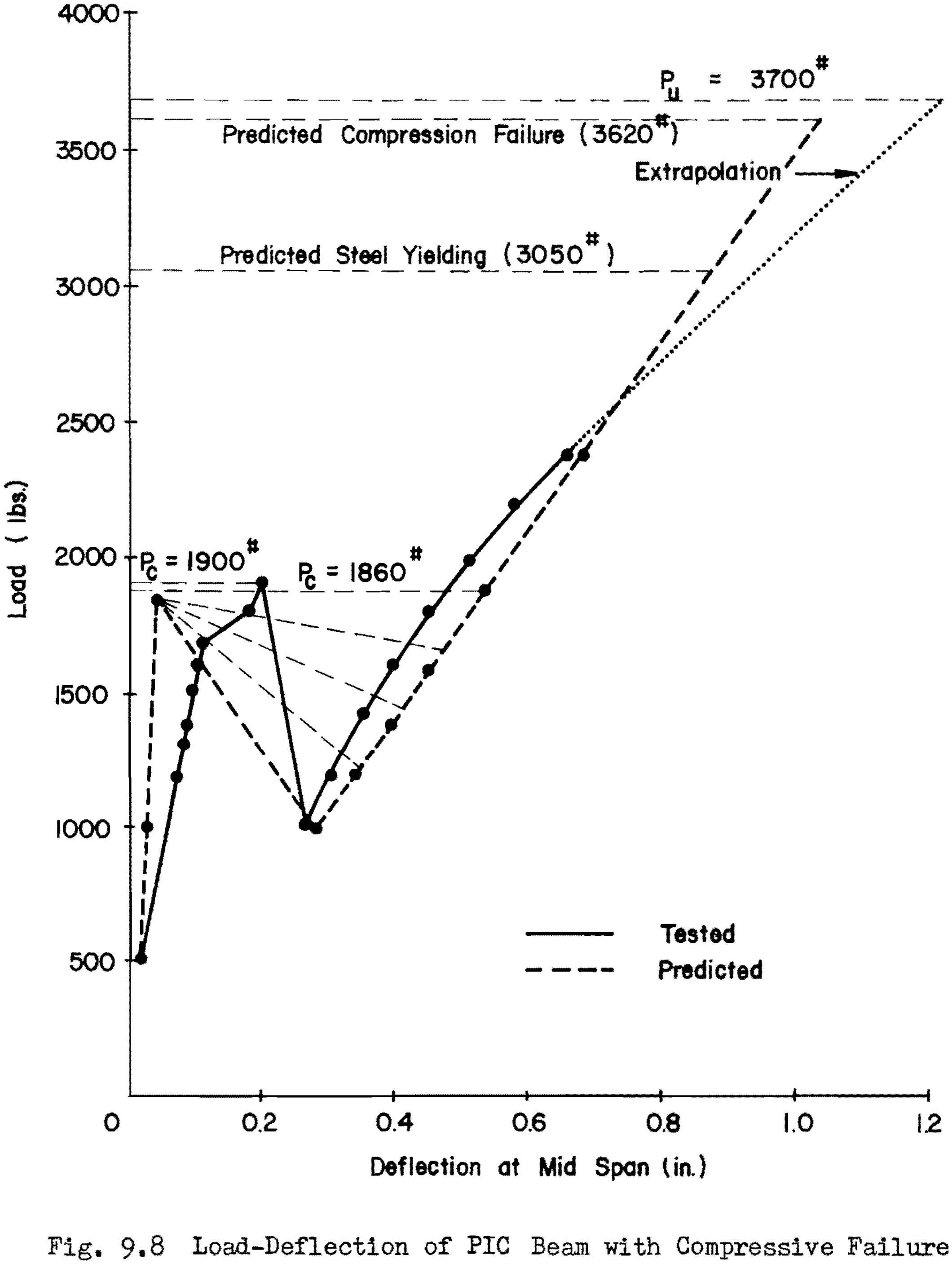


Fig. 9.8 Load-Deflection of PIC Beam with Compressive Failure.

2) At the load range from first crack to ultimate load, the moment of inertia was calculated based on the tranformed cracked section.

The calculations for load deflection are shown in Appendix 4. It can be seen from Figs. 9.6 and 9.8 that, at the load range from 0 to the load at first cracking, the predicted deflections were less than the actual test values. These results were believed to be the result of the assumption which assumed the full stiffness of the beam in this load range.

From Figs. 9.6 and 9.8, it can be seen that the predicted load-deflection relationships at the load range from first crack to ultimate load of both beams agree reasonably well with the test results.

An attempt was also made to predict the load at which cracking first occurred and the ultimate load of each beam. The results of calculations are shown in Appendix 4.

The predictions of the loads at first cracking were based on the following assumptions:

1) A transformed section was applicable with the concrete fully effective,

2) The maximum tensile stress in the beams was equal to the split tensile strength of PIC.

The predictions of the ultimate load were based on the following assumptions:

1) The transformed cracked section is applicable,

2) The compressive stress at the top fibre of the beam is equal to the compressive strength of the PIC; or

3) The tensile strength of the steel is equal to the yield strength of reinforcing bar; or

4) ACI Eq. 11-4 (Ref. 29) is application for determining ultimate shear stress:

$$v_c = 1.9\sqrt{f'_c} + 2500\,\rho_w \frac{V_d\, d}{M_u}$$

Where

v_c = nominal permisible shear stress carried by concrete, psi;

f'_c = specified compressive strength of concrete, psi;

ρ_w = ratio of area of steel to concrete;

V_u = total applied design shear force at a section, lbs.;

d = distance from extreme compression fibre to centroid of tension reinforcement, in.;

M_u = applied design load moment at a section, in-lb.

5) The stress-strain relationship of PIC is linear.

Table 9.4 shows the comparison of the predicted loads at first cracking and at the ultimate loads of both beams. It can be seen that the predicted loads at first cracking of Beam I and Beam II are 2120 and 1860 lbs., respectively. In comparison with the test results, the first cracking load of each beam is the load at the point of abrupt change in the load-deflection relationship. It was found that the loads at first cracking of Beam I and Beam II were 2000 and

1900 lbs., respectively. In comparison, the predictions of first cracking loads of both beams agree reasonably well with the test results.

The predictions of the ultimate loads of both beams are complicated because the mechanisms of stress transfer in PIC have not been defined. However, attempts were made to predict the ultimate load of each beam. The results of calculations are presented in Table 9.4. Based on the calculations, the lowest predicted ultimate load of Beam I was given by ACI Eq. 11-4 and was equal to 6790 lbs. In comparison with the test result, the prediction for diagonal tension failure is conservative. This is probably due to the fact that the assumptions for ACI Eq. 11-4 may not be applicable to high strength concrete like PIC. Because the actual failure was also primarily caused by the shear capacity of PIC, no further attempts were made to predict the ultimate load.

The lowest predicted ultimate load of Beam II was 3050 lbs., based on the yield strength of reinforcement. However, it is believed that the PIC is strong enough to allow considerable strain to occur in the steel so that strain-hardening can occur and compression failure can be developed. In addition, as was previously discussed, ACI Eq. 11-4 does not appear to be applicable for PIC. This equation gives results about 50 % lower than experimental values. Based on the compression strength of PIC, the predicted ultimate load was 3620 lbs. This approach is in close agreement with the test results

TABLE 9.4 COMPARISON OF PREDICTED AND TESTED LOADS AT FIRST CRACK AND ULTIMATE LOADS OF PIC BEAMS

Beam	Load at First Cracking, lbs.		Ultimate Load, lbs.			
	Test	Predicted	Test	Mode of Failure	Predicted	Mode of Failure
I	2,000	2,120	10,400	diagonal-tension	6,790 7,400	diagonal-tension tension
II	1,900	1,860	3,700	compression failure	3,620 3,130 3,050	compression failure diagonal-tension tension

of Beam II, which failed in compression at 3700 lbs.

9.6 Summary

1) The MMA specimens showed highest improvement in structural properties as compared to other monomer systems.
2) Within the limits of these tests, the specimens which were impregnated with MMA, which is a glass-like polymer and has low viscosity, showed minor changes in compressive strength, split tensile strength, and modulus of elasticity as the water-cement ratio changed.
3) The compressive and tensile strength of IBA polymer-impregnated concretes decreased as the water-cement ratio increased.
4) The compressive strength ratio of PIC (MMA) to control concrete increased as the polymer loading increased.
5) The use of the cracked section stiffness with the elastic modulus gave good agreement with the observed deflections.
6) The assumption of the first crack occuring at the split tensile strength of the PIC, using the transformed gross section, provided good agreement with the load at which the first crack occurred.
7) The predicted ultimate load of Beam I which failed in diagonal tension was 35 % lower than the failure load.
8) The predicted ultimate load of Beam II, which failed in

compression, indicated good agreement with the experimental load.

CHAPTER 10

CONCLUSIONS AND RECOMMENDATIONS

The studies reported herein were conducted on polymer-impregnated concrete (PIC) which provides significant improvements in structural and durability properties as compared to ordinary concrete. The monomer system which consisted of methyl methacrylate (MMA) with 1 % (wt.) benzoyl peroxide (BP) and 10 % (wt.) trimethylopropane trimethacrylate (TMPTMA) was most often used because of its low cost, low viscosity, and high strength and durability.

The first part of the study involved the evaluations of parameters which affect the applications of PIC for partial impregnation. The drying time, drying temperature, soaking time, curing time, and curing temperature which were necessary for impregnation of polymer into the concrete were determined. The relative protection provided against freeze-thaw deterioration and salt-water intrusion were investigated. Field tests were also conducted on simulated and actual bridge decks.

The second part of the study involved specimens fully-impregnated with polymer. The strength and corrosion protection provided by PIC using different monomers was determined. Limited flexural tests of reinforced PIC beams were performed and compared to theoretical predictions.

10.1 Conclusions

The conclusions are limited to the range of variables studied in this research and are as follows:

10.1.1 Application of PIC for Surface Treatments.

1) Drying of concrete prior to monomer application is the most significant step for partial polymer-impregnation. To remove the moisture from concrete in a reasonable time, it is necessary to increase the temperature at the desired depth of impregnation to at least 212°F.
2) The drying temperature significantly affects the drying time to achieve a specified depth of impregnation. It was found that drying times of 100, 20, and 8 hrs. were necessary for 3 x 6-in. cylinders to reach equilibrium weights at temperatures of 220, 256, and 300°F, respectively.
3) A drying temperature of 300°F is recommended as the upper limit for drying.
4) Relationships have been established to determine drying time and temperature to achieve different depths of impregnation. For example, to achieve a 1.0-in. polymer depth, it is necessary to dry the slab prior to monomer application at temperatures of 250 and 300°F for 9 and 6 hrs., respectively.
5) The moisture level in concrete was not found to significantly affect the drying time required to achieve a

specified depth of impregnation.

6) With adequate drying and curing, a 1-in. polymer depth can be achieved with a monomer soaking time of at least 4 hrs.

7) Many types of external heat sources can be used for polymerization of monomer in concrete. However, steam curing appears to be one of the most promising methods for partial impregnation, especially in the field.

8) Some medium is required to hold the monomer on the concrete surface during soaking. A minimum of 0.25 in. of dry sand was found to be adequate for this. Sand has been found to be adequate even on sloping surfaces.

9) A minimum curing temperature of about 150°F on the surface is necessary to achieve polymerization.

10) With an adequate curing temperature, a curing time of at least 0.5 hr. is necessary to produce polymerization and longer times (1 to 2 hrs.) are recommended to insure that all monomer is polymerized.

11) Freeze-thaw resistance of the partial impregnation depends on the polymer depth on slabs. With a 1-in. polymer depth, there was very slight surface deterioration after 100 cycles of freeze-thaw exposure as compared to unimpregnated concrete which failed at about 30 cycles.

12) Of the various monomers investigated, it was found that the MMA monomer systems provide the best improvement of

freeze-thaw resistance to the concrete.

13) However, even bars in slabs with shallow and faint polymer impregnation were found to develop only 1/24th as much corrosion as compared to bars in control slabs when subjected to salt-water exposure.

14) The average chloride content in partially-impregnated concrete slabs was about 19% of the level measured in the control.

10.1.2 Fully-Impregnated Polymer in Concrete.

1) The MMA monomer system gives the best improvement in structural and durability properties as compared to other monomer systems studied. The compressive strength of specimens impregnated with MMA was found to be about 3 to 4 times greater than for the controls.

2) Effect of concrete quality on compressive strength, tensile strength, and modulus of elasticity of specimens impregnated with MMA is small. In general, lower quality concrete exhibits strength properties about equal to the higher quality concrete after impregnation.

3) The specimens treated with rubbery monomer system indicated a decrease in compressive strength, tensile strength, and modulus of elasticity with increasing water-cement ratios.

4) The coefficient of thermal expansion of the MMA-impregnated

concrete was found to be about 15 % higher than for the control.

5) The most significant volume change of PIC occurs during polymerization.

6) After the specimens are fully-impregnated, the effect of concrete quality on volume change due to thermal expansion is small and negligible.

7) The predicted loads at first cracking of reinforced PIC beams agreed well with experimental values.

8) The predicted deflections of reinforced PIC beams from zero to first crack were smaller than the observed values.

9) The predicted deflections of reinforced PIC beams after the first cracking load agreed well with the observed deflections.

10) The predicted ultimate load of Beam I, which failed in diagonal-tension, was conservative.

11) The predicted ultimate load of Beam II, which failed in compression, agreed well with experimental values.

10.2 Recommendations

Based on this research, the following recommendations are proposed:

1) Further field applications should be made. To achieve up to 1.0 in. of polymer depth, it is recommended that:
    a) The slabs should be dried at a temperature range from 250°F to 300°F. Slab temperature should be monitored.

  - b) The drying time should be at least 5 hrs.
  - c) The slabs should be cooled to a temperature of less than 100°F prior to monomer application.
  - d) A polyethylene sheet should be used to cover the slab after soaking and during curing to minimize evaporation loss.
  - e) The slab temperature during curing should be monitored. The temperature on the surface of slabs during curing should be in the range of 150°F to 190°F.

2) Equipment for drying, soaking, and curing on the commercial scale should be developed.

3) Research to determine the structural behavior of reinforced PIC beams should be undertaken. Criteria for flexure, diagonal-tension, and anchorage failures should be established.

APPENDICES

APPENDIX 1

AGGREGATE PROPERTIES

Source: Colorado River, Austin, Texas

Type: Primarily rounded, siliceous river aggregate

SSD Bulk Specific Gravity:

Coarse Aggregate:	2.54 to 2.59
Fine Aggregate:	2.58 to 2.61

Absorption:

Coarse Aggregate:	1.8 to 2.3
Fine Aggregate:	0.9 to 1.3

Sieve Analysis (by ASTM C 136):

Coarse Aggregate:	all shipments met ASTM C 33, grade 67
Fine Aggregate:	all shipments met ASTM C 33, with fineness moduli of 2.51 to 2.79

APPENDIX 2

CONCRETE MIXES (CEMENT TYPE I)

Batch No.	Type of Test	W/C, gal./sk.	C.F., sk./cu.yd.	Slump, in.	f_c', psi
20	D	6.5	6.0	7	5290
23	D	6.5	6.0	3	6390
32	D	6.0	6.3	5	7080
33	D	7.5	5.0	5	5010
35	D	9.0	4.0	4	3400
40	D	8.0	4.7	4	4480
38	S	8.0	4.7	5	4240
42	S	6.0	6.3	6	6300
43	S	7.5	5.0	4	4600
44	S	9.0	4.0	4	3390
31	P	8.0	4.5	6	4400
38	P	8.0	4.7	5	4240

D = Drying test
S = Soaking test
P = Polymerization test
C.F. = Cement factor
f_c' = Compressive strength

APPENDIX 3

FULL IMPREGNATION TECHNIQUE

The following procedures were used to fully-impregnated concrete specimens:

1) The specimens were dried in the oven at a temperature of about 212° F to a constant weight.
2) The specimens were removed from the oven and wrapped with polyethylene to prevent the absorption of moisture from the atmosphere and allowed to cool to room temperature.
3) The specimens were placed in the vacuum chamber and subjected to a vacuum of 27 in. of Hg for 17 hrs. to remove the entrapped air from the concrete.
4) Monomer was injected into the vacuum chamber to impregnate the specimens. The vacuum was released after the specimens were under monomer. Additional monomer was added if necessary. The specimens were soaked for a minimum of 5 hrs.
5) Specimens were removed from the monomer and wrapped in polyethylene membrane to reduce evaporation loss.
6) Specimens were placed in water at 160° F to polymerize the monomer.

APPENDIX 4

PREDICTION OF LOAD-DEFLECTION RELATIONSHIP, LOAD AT FIRST CRACK, AND ULTIMATE LOAD OF PIC BEAMS

Data Available:

	Control	PIC	Steel
Compressive Strength (psi.)	4,400	13,000	-
Split Tensile Strength (psi.)	473	896	-
Yield Strength (ksi.)	-	-	68.7
Modulus of Elasticity (psi. x 10^6)	3.29	5.62	27.8

Assumptions

1) For control beam, assume transformed cracked section for all load levels.
2) For PIC
    2.1) Assume stiffness of transformed area of full cross section at load range from zero to first crack.
    2.2) Assume transformed cracked section from first crack to ultimate load.

Properties

	Control	Beam I	Beam II
Dimension (in.)	4 x 5.5	4 x 5.5	4 x 5.5
Reinforcement	1 - #5	1 - #5	1 - #5
Clear cover (in.) (from bottom)	1.5	1.5	3.5
I_1^a (in.^{4})		56.96	56.51
I_2^b (in.^{4})	12.33	11.68	1.95
P_1^c (kips)	-	2.120	1.860
P_u (concrete^d)	-	12.14	3.616
P_u (steel)	-	7.40	3.050
P_u (ACI-11-4)	-	6.80	3.130
Mode of failure	-	diagonal tension 10.40$^{\text{kips}}$	compression of concrete 3.700$^{\text{kips}}$

[a] Based on tranformed gross section.

[b] Based on transformed crack section.

[c] Load at first crack, kips.

[d] Ultimate load if concrete control, kips

TABLE A 4-1 LOAD-DEFLECTION OF PIC BEAM I AND BEAM II

Beam I			Beam II		
Load, lbs.	Actual Deflection, in.	Predicted Deflection, in.	Load, lbs.	Actual Deflection, in.	Predicted Deflection, in.
500	0.0080	0.0050	500	0.0060	0.0049
1000	0.0175	0.0097	1000	0.0140	0.0098
1500	0.0286	0.0146	1400	0.0800	0.0137
2000	0.0650	0.0195	1600	0.1010	0.0157
2500	0.0760	0.1190	1800	0.1810	0.0176
3000	0.1260	0.1420	1900	0.1970	0.0186
4000	0.1705	0.1900	1000	0.2630	0.2850
5000	0.2085	0.2370	1200	0.3100	0.3420
6000	0.2520	0.2850	1400	0.3420	0.3996
7000	0.2890	0.3320	1600	0.3970	0.4560
8000	0.3495	0.3800	1900	0.4800	0.5420
			2400	0.6600	0.6840

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