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Development, Characterization, and Fundamental Investigation of Latex-Modified Steel Fiber Reinforced Concrete

presented by

Atef Tlili

has been accepted towards fulfillment of the requirements for

Doctor of Philosophy degree in Civil Engineering

Major professor

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DEVELOPMENT, CHARACTERIZATION, AND FUNDAMENTAL INVESTIGATION OF LATEX-MODIFIED STEEL FIBER REINFORCED CONCRETE

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By

Atef Tlili

VOLUME I

A DISSERTATION

Submitted to Michigan State University in partial fulfillment of the requirements for the degree of

DOCTOR OF PHILOSOPHY

Department of Civil & Environmental Engineering

ABSTRACT

DEVELOPMENT, CHARACTERIZATION, AND FUNDAMENTAL INVESTIGATION OF LATEX-MODIFIED STEEL FIBER REINFORCED CONCRETE

By

Atef Tlili

Latex polymers and steel fibers have complementary actions in concrete. While steel fibers enhance the toughness characteristics and cracking resistance of the material, latex polymers help reduce concrete permeability and increase its durability and adhesion capacity. One may also expect that improved fiber-to-matrix bonding in the presence of latex would enhance the reinforcement efficiency of fibers. Latex may also help resolve potential workability and corrosion problems associated with the use of steel fibers and concrete.

The main thrust of this research was to investigate the mechanisms of joint action of steel fibers and latex polymers in concrete in order to develop latex-modified steel fiber reinforced concretes with balanced improvements in diverse aspects of material properties. Statistically sound methods of experimental design and analysis were adopted in order to ensure the reliability of the research outcomes. It was concluded that steel fibers and latex polymers interact favorably to produce concrete materials with substantially improved flexural strength and toughness, impact resistance, dimensional stability, freeze-thaw durability and scaling resistance. Furthermore, the corrosion inhibiting effects of latex polymer modification were successfully demonstrated, and it was concluded that the negative effects of steel fibers on the corrosion of embedded steel could be more than compensated for using latex polymers. The corrosion inhibiting action of latex polymers correlated well with its effects on reducing permeability and electrical conductivity of steel fiber reinforced concrete. Through microstructural studies the mechanisms of joint action of steel fibers and latex polymers were investigated and the failure mechanisms in latex-modified steel fiber reinforced concrete were established.

Latex-modified steel fiber reinforced concrete presents a high-performance construction material, with superior physical, mechanical and adhesion qualities, which suits demanding applications such as repair and overlay of the concrete-based infrastructure (e.g., bridges and parking structures) under severe load and environmental effects.

DEDICATED TO MY BEST FRIEND: MY WIFE BOUTHEINA,

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THE ONE I WILL GO THROUGH TIME WITH,

AND TO MY PARENTS AND FAMILY,

FOR THEIR GREAT LOVE, SACRIFICE AND ENDLESS ENCOURAGEMENT.

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INTRODUCTION

Concrete and cement-based materials date way back in history. They have been used by the ancient Egyptians in making pyramids, the Greeks in building walls, and even the Romans in erecting huge buildings and underwater structures.

As the twentieth century has progressed, Portland cement concrete - a mixture of cement, sand, gravel, and water - has emerged as the material of choice in broad areas of application. Concrete is second only to water in human material consumption. Currently about one ton of concrete is used each year for every human being on the planet. Cement- and concrete-based structures form the basis of much of the U.S. infrastructure. The value of concrete structures in the United States has been estimated at 6 trillion dollars, and more than 3 trillion dollars is expected to be needed for the maintenance, repair, and rehabilitation of these structures over the next twenty years.

The current market of construction demands very specialized materials that are different from those used in the past. Today, materials are expected to provide many properties not previously available. The modern construction industry desires materials that are corrosion resistant and chemical resistant, while maintaining the other desirable structural properties (strength and cost).

Ordinary concrete has a few disadvantages such as low tensile and flexural strengths, large drying shrinkage, and high permeability. It fails in a brittle manner under tensile stress systems and impact loads. These deficiencies generally result from the ease of initiation and propagation of microcracks, and also from the lack of post-cracking tensile resistance in conventional concrete materials. Microcracks in concrete usually start at the aggregate-cement paste interfaces as a result of the drying shrinkage, bleeding and settlement of cement paste in concrete. Under external load and environmental effects, microcracks tend to propagate and interconnect, leading to a brittle failure of concrete. Steel fiber reinforcement and polymer modification of concrete materials each can overcome some of these problems with conventional concrete.

The propagation of microcracks in concrete can be effectively hindered through the incorporation of closely-spaced short steel fibers in concrete. The arrest of microcracks by steel fibers leads to improvements in the pre-peak tension behavior and tensile strength of concrete. Upon the formation of an unstable microcrack system, which usually marks the achievement of the ultimate tensile strength in concrete, tensile deformations tend to concentrate in a limited number of macrocracks. Steel fibers can efficiently bridge these cracks and restrain their widening. Steel fibers are thus highly effective in enhancing the post-peak ductility and energy absorption capacity of concrete under tensile stress system. The stabilization of macrocracks by steel fibers in concrete results in major improvements in the compressive and flexural ductility and energy absorption capacity, impact resistance, fatigue life, and freeze-thaw durability of fiber reinforced concrete.

Nevertheless, steel fibers do present problems with the workability and flowability of the resulting fresh mixture, in addition to development of microcracks at fiber-cement matrix interfaces which possess a relatively high local water-cement ratio. Also, potentially increased electrical conductivity of concrete in the presence of steel fibers would promote the electrochemical mechanisms of steel corrosion in concrete.

In order to take full advantage of the fiber reinforcement technique in concrete, care should be taken in resolving the potential problems with uniform dispersion of fibers and workability of fresh mix as well as corrosion of steel. Also, steel fibers do not provide any significant contribution to the impermeability, durability, and adhesion capacity of concrete. Among the approaches undertaken for improving the steel fiber reinforced concrete material properties, modification of the concrete matrix by latex polymers presents quite an attractive solution. Latex polymers in the presence of steel fibers do overcome the damage to the workability of the mix associated with fiber reinforcement and also provide a better bonding between fibers and the concrete matrix. A major function of the latex polymers is to generate a plastic mass from a cement mixture with relatively low water content. Once incorporated into the mix, and as cement hydration process continues, the polymers will coalesce and bond to form a continuous polymer film. An important advantage of using latex in fiber reinforced concrete is the improvement of fiber-matrix interfacial bond characteristics. This improvement in bond is illustrated by the fact that the latex polymer (with its good bonding to both fiber and cement paste) tends to coat the fibers and provide a strong linkage between the fibers and the matrix.

Polymer latexes in fresh concrete mixtures work like lubricants. Their

presence therefore reduces the water requirements needed for achieving a certain level of workability. This leads to an increase in the hardened material strength. Polymer latexes also tend to stabilize the air void system within the fresh concrete mix, producing concretes with improved freeze-thaw durability.

The coating of steel fibers by latex polymers also presents the potentials to reduce the adverse effects of steel fibers on the electric resistivity of concrete.

Steel fiber reinforcement and polymer modification of concrete materials each can overcome some of the problems with conventional concrete. On one hand, steel fibers enhance the ductility and energy absorption capacity, flexural strength, fatigue life and impact resistance of concrete. Latex modification, on the other hand, improves the impermeability and dimensional stability as well as the strength and ductility characteristics of concrete. Latex polymers can also reduce the damage to fresh mix workability resulting from the presence of steel fibers and improve the bonding of fibers to the concrete matrix. Added ductility of concrete matrices in the presence of latex polymer may also improve their compatibility with steel fibers. Combined effects of latex modification and steel fiber reinforcement can potentially produce superior concrete materials with a highly desirable balance of mechanical properties, impermeability and durability characteristics, and adhesion capacity. Such concrete materials are particularly suitable for applications such as bridge deck overlays where severe environmental and loading effects are expected.

The objective of this research was to establish the mechanisms of joint action of latex polymers and steel fibers in concrete in order to demonstrate their compatibility for joint applications in concrete materials.

Chapter 2 presents a review of literature on steel fiber reinforcement and latex modification and their joint action in concrete. The work in this research on mix proportioning and fresh mix characterization is presented in Chapter 3. This chapter also includes an experimental study on some mechanical and physical properties of latex-modified steel fiber reinforced concrete, namely, flexural and compressive performance, impact resistance, bond strength, freeze-thaw durability, scaling resistance, and drying shrinkage.

Chapter 4 presents microstructural studies using image analysis techniques for establishing the effects of latex modification and steel fiber reinforcement on the microcracking and failure mechanisms of the resulting material.

In Chapter 5, an assessment of latex modification and steel fiber reinforcement effects on concrete permeability at different ages and with different curing regimes is provided. The applicability of different permeability test methods to steel fiber reinforced and latex-modified concretes is also investigated.

Chapter 6 presents the effects of steel fiber reinforcement and latex modification on the corrosion-inhibiting properties and electrical resistivity characteristics of concrete materials.

Chapter 7 summarizes the research program and presents its conclusions.

CHAPTER TWO

POLYMER-MODIFIED STEEL FIBER REINFORCED CONCRETE: A LITERATURE REVIEW

2.1 INTRODUCTION

Ordinary concrete has a few disadvantages such as low tensile and flexural strengths, large drying shrinkage, and high permeability. Steel fiber reinforcement and polymer modification each can overcome some of these problems with conventional concrete.

Steel fibers enhance the ductility and energy absorption capacity, flexural strength, fatigue life and impact resistance of concrete. Latex modification, on the other hand, improves the impermeability and dimensional stability as well as the strength and ductility characteristics of concrete. Latex polymers in the presence of steel fibers also overcome the damage to workability associated with fiber reinforcement and also provide a better bonding between fibers and the concrete matrix.

In short, steel fibers and latex polymers seem to be very compatible for joint application to concrete. Significant improvements to concrete mechanical, physical and durability properties could be achieved through the combined use of steel fibers and latex polymers in concrete.

2.2 CONCEPT OF FIBER REINFORCEMENT

Because cementitious materials are brittle, they have been reinforced with a variety of materials in an attempt to reduce catastrophic failure and increase toughness. The propagation of microcracks in concrete can be effectively hindered through the incorporation of closely-spaced short steel fibers in concrete. The arrest of microcracks by steel fibers leads to improvements in the pre-peak tension behavior and tensile strength of concrete. Upon the formation of an unstable microcrack system, which usually marks the achievement of the ultimate tensile strength in concrete, tensile deformations tend to concentrate in a limited number of macrocracks. Steel fibers can efficiently bridge these cracks and restrain their widening. Steel fibers are thus highly effective in enhancing the post-peak ductility and energy absorption capacity of concrete under tensile stress system (see Figure 2.1). The stabilization of macrocracks by steel fibers in concrete results in major improvements in the compressive and flexural ductility and energy absorption capacity, impact resistance, fatigue life, and freeze-thaw durability of fiber reinforced concrete.

Before fracture of the composite occurs, several possible local failures could take place (see Figure 2.1 a).^{25, 49} At some distance ahead of the crack that is travelling through the composite, fibers begin to be influenced. Near the crack tip, where the stress is very high, fibers may debond from the matrix (e.g., fiber 1 in Figure 2.1 a). This rupture of chemical bonds at the interface uses up energy from the stressed system. Also, sufficient stress may be transferred to a fiber (e.g., fiber

2) to enable it to be ultimately fractured (as in fiber 4). When total debonding occurs, the strain energy in the debonded length of the fiber is lost to the material and is dissipated as heat. A totally debonded fiber can then be pulled out from the matrix and considerable energy is lost from the system in the form of frictional energy (e.g., fiber 3). It is also possible for a fiber to be left intact as the crack propagates. This process is called crack bridging.⁴⁹ The net effect of the interaction of fibers with cracks in cement composites is the improvement of ductility and tensile strength of the resulting cement-based composite material (Figure 2.1 b).



- (a) Schematic Representation of a Crack (b) Improvements in Material Ductility Travelling Through a Composite
- Figure 2.1 Actions of Fibers in Cement Composites, and the Consequent Improvements in Material Ductility.²⁵

Nevertheless, steel fibers do present problems with the workability and flowability of the resulting fresh mixture, in addition to development of microcracks at fiber-cement matrix interfaces which possess a relatively high local water-cement ratio.

In order to take full advantage of the fiber reinforcement technique in applications to concrete, care should be taken in resolving the potential problems with uniform dispersion of fibers and workability of fresh mix. Among the approaches undertaken for improving the steel fiber reinforced concrete material properties, modification of the concrete matrix by latex polymers presents quite an attractive solution.

2.3 CONCEPT OF LATEX MODIFICATION

A latex is a colloidal dispersion of small (0.5 to 5 microns in diameter) spherical organic polymer particles held in suspension in water by surface-active agents.⁸⁰ Latexes are generally milky fluids that are white to off-white in color. It is the presence of these surface-active agents that tends to incorporate excessive amounts of entrained air in concrete; therefore, antifoaming/detraining agents are generally added to commercial latexes. In latex-modified concrete the hydration of cement together with the formation of a polymer phase proceed to yield a monolithic matrix phase with a network structure in which the hydrated cement phase and polymer phase interpenetrate into each other. Aggregates are also strongly bonded to such a comatrix phase.^{42, 47, 57, 80} A simplified model, consisting of three steps, can be used to describe the formation of polymer cement comatrix (see Figure 2.2).⁵⁷



Figure 2.2 Simplified Model of Polymer-Cement Comatrix Formation.
Initially, the polymer particles are uniformly dispersed in cement. As a result of the gradual hydration of cement, a cement gel is formed in the cement-polymer paste, and the water phase is saturated with calcium hydroxide. In step 1 of polymercement comatrix formation the polymer particles deposit partially on the surface of the mixture of cement gel and unhydrated cement particles. It is likely that the calcium hydroxide in water phase reacts with the silica surface of aggregates to form a calcium silicate layer. The bond between cement hydrates and aggregates has been attributed to the calcium hydroxide and ettringite present in the contact zone. As the hydration of cement continues, the drainage resulting from development of cement gel structure gradually confines the polymer particles to capillary pores. This process leads to the flocculation of polymer particles and formation of a continuous closelypacked layer of polymer particles on the surfaces of cement gel, unhydrated cement particles, and aggregates. This polymer layer bonds the matrix to the silicate layer over aggregate surface. At this stage, the larger pores in the mixture (cement paste pores range in size from less than one to several hundred nanometers) are found to be filled by the adhesive and autohesive polymer particles. Ultimately, with further drainage of water by cement hydration, the closely packed polymer particles on cement hydrates and aggregates coalesce in continuous films or membranes which bind the cement hydrate together and to aggregates, forming a monolithic network. Consequently, the polymer-modified concrete, when cured, consists of the hydrated cement and aggregate phases which are interconnected by a "continuous film" of latex particles. It is this continuous film which imparts the superior physical and chemical properties to latex-modified concrete.

The fact that a polymer layer is deposited on the surface of unhydrated cement particles makes the access of these particles to water rather difficult. However, water can pass around the grains, and thus hydration continues. This explains the larger increase in strength of polymer-modified concrete after 28 days as compared to conventional concrete.

The polymer film forms not only inside latex-modified concrete after a period (about 1 days) of moist curing but also on the surface, and retains the internal moisture needed for cement hydration. This partially illustrates why the optimum curing conditions are different in latex-modified and conventional concretes.

2.4 MIX PROPORTIONING AND MANUFACTURING OF LATEX-MODIFIED STEEL FIBER REINFORCED CONCRETE

Ohama³⁴⁻⁶⁰, Fukuchi³¹, and Sundara⁸⁸ have reported test results on latexmodified steel fiber reinforced concrete (LMSFRC), and Ohama⁶¹⁻⁶⁴ has presented results of tests on latex-modified steel fiber reinforced mortar (LMSFRM). All references used ordinary Portland cement and river sand (maximum size 2.5 mm, 0.1 in). The concrete mix had river gravel with maximum size of 10 or 20 mm (0.39 or 0.787 in). Sundara⁸⁸ has used crushed Granite with irregular shapes and maximum size of 20 mm. (0.787 in.) as coarse aggregate. The latex polymer used was Polyacrylic Ester (PAE) in Refs. 58, 60 and 61, and Ethylene Vinyl Acetate (EVA) in Refs. 61, 62, and 64. Natural rubber latex was used in Ref. 88. The steel fibers used were typically 0.25 x 0.55 mm (0.01 x 0.022 in) in cross-section, and 25 mm (1 in) long, or 0.5 x 0.5 mm (0.02 x 0.02 in) in cross-section, and 30 mm (1.18 in) long. Sundara⁸⁸ has used steel fibers with an aspect ratio of 70. Ohama ⁵⁸⁻⁶⁴ has used these basic mix constituents with no other admixtures for the manufacturing of LMSFRC noting that an antifoaming agent was routinely used with latex polymers in concrete. Sundara⁸⁸ added superplasticizer to these constituents for the manufacture of natural rubber latex-modified steel fiber reinforced concrete.

Table 2.1 presents the mix proportions used for latex-modified steel fiber reinforced mortar and concrete by different investigators.

Mix Proportions of Latex-Modified Steel Fiber Reinforced Concrete. Table 2.1

Ref. No.	Matrix	Weter-Cement Ratio	Sand- Cement Ratio	Gravel- Cement Ratio	Gravel Type and Max. Size	Latex-Cement Ratio	Latex Type	Steel Fiber Volume Fraction	Steel Fiber Aspect Ratio
58,	Concrete	0.553 - 0.675				0			
8		0.378 - 0.540	2.7	9	River Gravel 10 mm.	0.10	SBR	0 - 2 %	8
		0.250 - 0.390			(0.4 in.)	0.20			
62	Mortar	0.785 - 0.795				0			
		0.725 - 0.730	3.0			0.05	EVA	0 - 2 %	60
		0.675 - 0.685				0.10			
		0.630 - 0.640				0.20			
31	Concrete	0.400 - 0.600	3.1	2.1	River Gravel 20 mm. (0.8 in.)	0 - 0.20	PAE	0 - 2 %	53
88	Concrete	0.350 - 0.500	1.79	4.05	Crushed Granite 20 mm. (0.8 in.)	0 - 0.088	Natural Rubber	0 - 1.5 %	70
61,	Mortar	0.785 - 0.790				0			
8		0.670 - 0.680	3.0		-	0.05	SBR	0-2%	80
		0.630 - 0.640				0.10			
		0.550 - 0.565				0.20			
61,	Mortar	0.785 - 0.790				0			
8		0.725 - 0.730	3.0			0.05	EVA	0 - 2 %	60
		0.675 - 0.685				0.10			
		0.630 - 0.640				0.20			

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2.5 FRESH MIX PROPERTIES OF LATEX-MODIFIED STEEL FIBER REINFORCED CONCRETE

Reinforcement by steel fibers improves the physical properties of concrete but damages the fresh mix workability.^{31, 61} The modification of concrete by latex polymer dispersions would improve the fresh mix workability. The improvements in concrete workability resulting from polymer modification have been attributed to the effects of the presence of surface-active agents in polymer dispersions. Polymer modification of steel fiber reinforced concrete is thus expected to reduce the workability problems caused by the presence of steel fibers.³¹

A comprehensive presentation of the effects of steel fiber content, polymer content and water-cement ratio and the slump of concretes with 350 kg/m³ (590 lb/yd³) cement concrete is given in Figure 2.3.³¹ The figure indicates that the damage to workability by steel fiber reinforcement (up to certain fiber volume fractions) can be overcome through polymer modification.



Figure 2.3 Effects of Water-Cement Ratio, Polymer-Cement Ratio, and Steel Fiber Volume Fraction on the Slump of Concrete.³¹ (w/c=0.4-0.6, s/c=3.1, g/c=2.1, L/c=0-0.2, maximum aggregate size=20 mm (0.79 in), $V_f=0-2\%$, 0.25 x 0.55 mm (0.01 x 0.022 in), 1/d=53, PAE latex).

2.6 ACTION OF LATEX POLYMERS IN FIBER REINFORCED CONCRETE

Latex polymers enhance the fiber reinforced concrete performance by improving the concrete matrix characteristics (as they do in plain concrete), and also by providing a better bonding between fibers and cementitious matrices.

During the mixing operation of fiber reinforced concrete (FRC), a film of water forms on the fiber surfaces. Cement particles move into this film, but towards the surface of fibers the concentration of cement particles tends to decrease. The thinner the water film around the fibers, the better the fiber-matrix interfacial bond and consequently the stronger and more ductile the composite material. Latex polymers can reduce the water film thickness present around fibers during the mixing operation by reducing the required water-cement ratio for achieving a workable mix, and also by the fact that latex polymers can be surface-active agents. Latex polymer particles are also much smaller than sand and cement particles. They can fill smaller voids thus preventing the formation of much of the microcracks that tend to take place along the fiber-matrix interface. These polymer particles eventually coalesce into a monolithic film surrounding the fibers, bonding the cementitious environment to the fibers.⁴⁶ Some micrographs indicating the presence of this linkage and its desirable bonding characteristics are shown in Figure 2.4 for latex-modified carbon fiber reinforced cement.



Figure 2.4 Micrograph of the Interface Zone Between Carbon Fibers and Latex-Modified Cementitious Matrices.⁴⁶

2.7 MECHANICAL PROPERTIES OF LATEX-MODIFIED STEEL FIBER REINFORCED CONCRETE

Test results indicating the improvements in concrete mechanical properties

resulting from the joint action of latex polymers and steel fibers are discussed in this section.

2.7.1 Flexural Performance

Both steel fiber reinforcement and polymer modification are shown in Figure

2.5 to increase the flexural strength of concrete. Similar conclusions have been reached using Styrene Butadiene Rubber (SBR), Ethylene Vinyl Acetate (EVA) and Polyacrylic Ester (PAE) emulsions. Figure 2.5 indicates that the use of 2% volume fraction of steel fibers together with a latex polymer at 20% the weight of cement can increase the flexural strength of concrete to about 2.5 to 3 times that of conventional concrete.^{31, 58, 61}



Figure 2.5 Effects of Steel Fiber Reinforcement and Polymer Modification on the Flexural Strength of Concrete.⁵⁸

The flexural load-deflection relationships shown in Figure 2.6 for latexmodified steel fiber reinforced concrete with different latex and fiber contents are indicative of important improvements in the flexural ductility and toughness of concrete resulting from the joint action of steel fibers and latex polymers. These improvements can be attributed to the enhanced quality of the concrete matrix and also improvements in the fiber-matrix bond characteristics.





Figure 2.6 Effects of Latex Modification and Steel Fiber Reinforcement on the Flexural Load-Deflection Characteristics of Concrete.⁵⁸

Another indication of the desirable effects of latex modification together with steel fiber reinforcement on the flexural load-deflection characteristics of concrete is provided by Sundara⁸⁸ who has used natural rubber latex to modify steel fiber reinforced concrete (see Figure 2.7). The results indicate that natural rubber latex has a relatively small effect on the flexural strength of latex-modified concrete (LMC), but it substantially improves the ductility and toughness of the material. The flexural deflection at peak load of SFRC is also observed in Figure 2.7 to be increased through natural rubber latex modification.

The joint effects of latex modification and steel fiber reinforcement on flexural toughness in concrete are more clearly shown in Figure 2.8. Flexural toughness in this figure is defined as the area underneath the load-deflection curve up to a deflection equal to 1% of the beam span (3 mm=1/8 in). This figure indicates that the latex modification effects on toughness tend to be more pronounced as the fiber content increases, a phenomenon that can be attributed to the latex effects on the fiber-matrix interfacial bond characteristics.



Figure 2.7 Flexural Load-Deflection Curves for Natural Rubber Latex Modified Steel Fiber Concretes With Different Latex and Fiber Contents.⁸⁸



Figure 2.8 Flexural Toughness of Concrete as Influenced By Steel Fiber Reinforcement and Latex Modification.⁵⁸

2.7.2 Tensile Performance

Direct tension test results on natural rubber latex-modified steel fiber reinforced concrete (see Figure 2.9)⁸⁸ indicate that the natural rubber latex does not significantly contribute to the tensile strength of SFRC, but substantially increases the strain at peak tensile stress of the material.

Steel fiber reinforcement and polymer modification are both seen in Figure 2.10 to increase the direct tensile strength of concrete.³¹ This figure also indicates that the effectiveness of steel fibers in increasing the tensile strength of concrete tends to be higher at a polymer-cement ratio of 10% when compared with the corresponding effects of steel fibers in unmodified concrete. The tensile strength of a polymer-modified steel fiber reinforced concrete with 20% polymer-cement ratio and 2% fiber volume fraction is observed in Figure 2.8 to exceed 1.7 times that of plain concrete.



Figure 2.9 Effect of Natural Rubber Latex Modification on Direct Tensile Behavior of Steel Fiber Reinforced Concrete.⁸⁸



Figure 2.10 Effects of Steel Fiber Reinforcement and Polymer (PAE Latex) Modification on the Direct Tensile Strength of Concrete.³¹

2.7.3 Compressive Performance

Compressive strength test results on latex-modified steel fiber reinforced concrete (see Figure 2.11)⁶¹ indicate that steel fibers have relatively small effects on compressive strength while the EVA, SBR and PAE polymer latexes tend to increase the compressive strength of plain and steel fiber reinforced concretes. Improvements of as much as 70% in compressive strength of SFRC can be obtained through latex modification.

Compressive stress-strain curves have been presented by Sundara⁸⁸ for natural rubber latex-modified SFRC. Figure 2.12 indicates that natural rubber latex has relatively small effects on compressive strength but improves the ductility and toughness of plain and fiber reinforced concretes under compressive forces. The strain at peak compressive stress also tends to increase with natural rubber latex modification.



Figure 2.11 Effects of Latex Modification and Steel Fiber Reinforcement on the Compressive Strength of Concrete.⁶¹



Figure 2.12 Compressive Stress-Strain Curves of Natural Rubber Latex Steel Fiber Reinforced Concrete.³⁸

2.7.4 Impact Resistance

Fukuchi³¹ has reported the results of flexural impact tests performed on PAE latex-modified steel fiber reinforced concrete (LMSFRC). The results presented in Figure 2.13 are indicative of major improvements (up to 60 times) in flexural impact strength of concrete resulting from the joint action of steel fibers and latex polymers.

An alternative flexural impact experimental methodology (a pendulum type with notched specimens) was adopted by Sundara⁸⁸ for investigating the effects of natural rubber latex modification on the impact resistance of SFRC. The joint action of natural rubber and steel fiber is observed in Table 2.2 to increase the ultimate flexural impact resistance of concrete by as much as 20 times in this test methodology.



- Figure 2.13 Effects of Latex Modification and Steel Fiber Reinforcement on the Flexural Impact Resistance of Concrete.³¹
- Table 2.2Effects of Natural Rubber Latex Modification on the Flexural Impact
Resistance of Steel Fiber Reinforced Concrete.⁸⁸

	Volume Descent of	Volume	Number of Blows		
Material	$\begin{array}{c} \text{Material} \\ \text{Rubber, } V_{R} \\ \end{array} \begin{array}{c} \text{Percent of} \\ \text{Fibers, } V_{f} \\ \end{array}$		At First Crack	At Failure	
Plain Concrete	0	0	5 6		
Latex- Modified Concrete	2.0 %	0	21	54	
Fiber Reinforced	0	1.0 %	25	61	
Concrete (SFRC)	0	1.5 %	33	72	
Latex-	2.0 %	1.0 %	44	102	
SFRC	2.0 %	1.5 %	53	122	

2.8 WATER ABSORPTION OF POLYMER-MODIFIED STEEL FIBER REINFORCED CONCRETE

Ordinary cement mortars have the disadvantage of inferior waterproofness. Ohama⁶¹ has studied the effects of polymer modification and steel fiber reinforcement on the water absorption of the concrete matrix.

Figure 2.14 illustrates the relationship between steel fiber content, polymercement ratio and the 48-hour water absorption of steel fiber reinforced polymermodified mortars (SFRPMM). It is seen that water absorption tends to be remarkably reduced when water-cement ratio is increased irrespective of the steel fiber content. The increase in steel fiber content, however, hardly changes the water absorption of SFRPMM. Hence, the waterproofness is significantly dominated by the polymer modification effects. The water absorption of a SFRPMM with 20% polymer-cement ratio and 2% fiber volume fraction is observed in Figure 2.14 to be only 50% that of plain cement mortar.



Figure 2.14 Effects of Steel Fiber Reinforcement and Polymer Modification on the Water Absorption Characteristics of Mortar.⁶¹

2.9 DRYING SHRINKAGE OF POLYMER-MODIFIED STEEL FIBER REINFORCED CONCRETE

Various concrete structures suffer from crack formation on their surfaces

leading to leakage and thus corrosion of the reinforcement embedded in concrete.

These cracks are generally related to the large drying shrinkage movement that

characterizes cement and concrete. Steel fiber reinforcement and especially polymer modification are efficient techniques used for reducing the drying shrinkage of concrete.

Figure 2.15 shows that, similarly to ordinary unreinforced unmodified concrete, the drying shrinkage of SFRPMM does increase with increasing drying period. This drying shrinkage tends to be reduced by the addition of 2% steel fiber volume fraction. Figure 2.15 shows the relationship between the steel fiber content, polymer-cement ratio and 28-day drying shrinkage of SFRPMM. It is seen that this drying shrinkage remarkably decreases when steel fiber content and polymer-cement ratio increase. A reduction of 34% in drying shrinkage is obtained for a steel fiber content of 2% by volume and a polymer-cement ratio of 20%. This is due to the fact that, on one hand, polymer modification results in a decrease in the water-cement ratio and an improvement in the water retention, and on the other hand reinforcement with steel fibers has a restraining effect on drying shrinkage movements.

Figure 2.16 also shows the relationship between the steel fiber content, polymer-cement ratio and 84-day drying shrinkage of SFRPMM. It is also noticed that the drying shrinkage of SFRPMM decreases with increasing steel fiber content and polymer-cement ratio, irrespectively of the types of polymers used (EVA or SBR) for the same reasons stated above. In particular, results by Ohama⁶⁰ have shown that the drying shrinkage of SFRPM concrete is reduced to one half that of the unmodified unreinforced concrete when the steel fiber content is 2% by volume and the polymercement ratio is 20%.

Ohama⁶⁴ has defined a "shrinkage reduction factor" to be used for evaluating

this shrinkage reduction effect as follows:

$$R_g = \frac{(S_m - S_f) \cdot 100}{S_m}$$

where,

R_s=shrinkage reduction factor (%); **S**_m=drying shrinkage (x 10⁴) of cement mortar; and **S**_f=drying shrinkage (x 10⁴) of SFRPMM.

Figure 2.17 shows that this reduction factor R_{\star} (%) is increased when both steel fiber content and polymer-cement ratio are increased, independently of the type of polymer dispersion (EVA or SBR). It can also be seen that the shrinkage reduction factor, when compared with unreinforced unmodified cement mortar, is 36% and 33% for SBR and EVA modifications, respectively, for a 2% by volume steel fiber content and 20% polymer-cement ratio.





Figure 2.15 Measured Shrinkage Vs. Drying Period.⁶²



Figure 2.16 Effects of Steel Fiber and Latex Polymers on Drying Shrinkage.⁶²





SBR Latex



Figure 2.17 Effects of Steel Fiber Reinforcement and Polymer Modification on Shrinkage Reduction Factor of Cement Mortar.⁶⁴

2.10 BONDING OF LATEX-MODIFIED CONCRETE TO STEEL FIBERS

Fiber-matrix interfacial bond characteristics play a decisive role in the reinforcing action of fibers in concrete materials. Hence, the tensile strength and toughness characteristics of steel fiber reinforced concrete can be enhanced through improving and optimizing the bonding of concrete matrices to steel fibers. It should be noted that excessive bond strengths may encourage fiber breakage, rather than fiber pull-out, which dominate the failure mechanism of steel fiber reinforced concrete; a phenomenon which can adversely influence the toughness of fibrous concrete. The effectiveness of fibers can also be lost if bond failure occurs in the matrix at a small distance from the fiber (in the interface transition ring). Thus, to improve the fiber-matrix interface, the strength (and ductility) of both the matrix material and the fiber-matrix interface must be improved.

In steel fiber reinforced concrete a combination of adhesion, friction and mechanical interlocking constitute the fiber-matrix interfacial bond strength. Wei⁹⁵ has studied the effects of latex modification on the bond strength of mortars, with sand-cement ratio of 1.5, to steel fibers.

The plain mortar had a water-cement ratio of 0.3 by weight, and the latexmodified one had an acrylic polymer (Rhoplex E330)-cement ratio of 15% and a water-cement ratio of 0.25. Both mixes had comparable workability characteristics. Five specimens of each matrix were tested.

Figures 2.18 and 2.19 show the specimen types used to measure the tensile and interfacial bond strengths of the material, respectively. These tests were



Figure 2.18 Tensile Test Specimen.⁹⁵



Figure 2.19 Pull-Out Test Specimen.⁹⁵

Table 2.3 shows the results of the tensile strength and pull-out tests. The results very clearly show that the adhesion of the acrylic polymer resulted in major improvements (on the order of 370%) in both tensile and bond strengths as well as the work done during pull-out.

SEC. A-A

Table 2.3Effects of Latex Modification on Bond, Tensile Strength, and Work
Done During Fiber Debonding and Pull-Out.95

Water- cement ratio	Acrylic polymer/ cement (by weight), percent	Fiber embedded length, mm	Fiber diameter mm	tensile strength of matrix, kg/cm ²	Average interface bond strength, kg/cm ²	Work done during fiber pullout from matrix (per fiber), kg-cm
0.25	15	20	0.5	87.6	85.1	20.5
0.43	0	20	0.5	23.6	22.7	5.43

• cement-sand ratio = 1:1.5

• number of fibers = 16

• The workability with and without acrylic polymer was the same; values shown are an average of the results from five specimens.

Figure 2.20 shows the load-displacement curves obtained for typical plain and latex-modified mixes. The area under the curve represents the energy required for fiber debonding and pull-out. Area OAB is an upper limit to the energy associated with fiber debonding, and the remaining area is the frictional energy dissipated during fiber pull-out.



Figure 2.20 Fiber Pull-Out Load-Displacement Relationships.⁹⁵

The results presented above may be explained by two changes in the structure of the material near the fiber-matrix interface due to the addition of acrylic polymer:

1- the acrylic polymer used, being an active surface agent, may have reduced water-film thickness present around the fibers during the mixing operations; and

2- because the polymer size (0.5 to 5 microns) is much smaller than that of sand and cement, it can fill smaller voids and thus prevent the formation of much of the microcracking occurring in the matrix material and along the fiber-matrix interface.

In order to study the fiber-matrix interface it is necessary to consider an annular region surrounding the fiber (interface transition ring). During the mixing operation, a film of water forms on the fiber's surface. Cement particles move into this film and towards the surface of fibers where the lowest concentration of cement particles exists. This concentration increases as the distance from fiber surface increases. The interface transition ring is formed when the matrix has hardened and no cement particle can migrate. In general, the thinner the water film around the fiber is, the stronger is the material. Since the thickness of this film is reduced with reduced water-cement ratio, and since the addition of acrylic polymer allows a reduction in water-cement ratio, then the result of polymer modification would be a stronger material in the interface transition ring and a larger effective bond strength. Wei⁹⁵ presents microhardness test results which substantiate this hypothesis. The results indicate that the annular region between the fibers has a width of about 100 microns and that the reduction in water-cement ratio as well as the addition of polymer increase the microhardness in the interface zone and also in the bulk of cement paste. The weakest zone of interface was observed to be located at a distance of about 25 to 35 microns from the fiber surface.

Studies on the microhardness features of the material with a scanning electron microscope showed that for the material without acrylic polymer there was substantial cracking at and near the fiber interface and that the surface of the fiber was relatively clean (no cement particles) when the fiber-matrix interface was broken. However, with the acrylic polymer the surface of the fiber had cement particles adhered to it

indicating a higher bond strength. The cement materials with acrylic polymer exhibit less cracking in the region of fiber-matrix interface. This may be due to the filling of small cracks by the acrylic polymer due to its small particle size.

CHAPTER THREE

DEVELOPMENT AND BASIC CHARACTERIZATION OF LATEX-MODIFIED STEEL FIBER REINFORCED CONCRETE

3.1 INTRODUCTION

This chapter presents some results of an experimental study concerned with comprehensive characterization of concrete mixtures incorporating the following loadings of latex and steel fibers:

latex/cement (L/c)	fiber volume fraction (V_f)		
0%	0%		
10%	0%		
0%	0.75%		
10%	0.75%		

The results will reveal independent and combined effects of latex modification and steel fiber reinforcement on various aspects of the mechanical, physical and durability characteristics of concrete materials. The levels of latex and steel fiber contents considered in this phase of the study, and the corresponding mix proportions, have been chosen as optimum values based on the results generated in the previous experimental studies.

3.2 BACKGROUND

A review of the literature on the effects of steel fibers and latex polymers on the fresh mix properties of concrete materials is presented in this section. This brief literature review relates strictly to topics covered in this chapter, and it reflects mainly the experience with latex-modified concrete and steel fiber reinforced concrete. A broader view of the literature on combined effects of latex and steel fibers is presented in Chapter 2.

3.2.1 Workability Loss

The rate of workability loss with elapsed time after mixing is a powerful measure of field placability of concrete. Since concrete is usually transported to the construction site and the placement of concrete might be a lengthy process in certain field conditions, the loss of workability during transportation and placement is of major importance. The high rate of workability loss in concrete can be overcome by adding superplasticizers prior to placement.

Figure 3.1^{22} represents the slump loss vs. time curves for five different latex-modified concretes (LMC) at 0.5 water-cement ratio (w/c). Table 3.1^{22} also lists the time required for a 50% slump loss for each concrete. At 23 °C (73 °F) and 50% R.H., the slump loss vs. time characteristics of conventional concrete, Dow

Modifier A, Thermoflex 8002, Deco-Rez 4776 and Acro Dylex 1186 are similar. The Acro Dylex 1184 latex concrete required only about half as much time for 50% slump loss when compared with the other concretes.




Concrete Type	Time From Completion of Mixing to 50% Slump Loss (minutes)
Conventional Concrete, w/c=0.50	76
Dow Modifier A	83
Thermoflex 8002	80
Deco-Rez 4776	75
Acro Dylex 1184	40
Acro Dylex 1186	72
Special Latex	10

Table 3.1Loss of Slump With Time.

Studies on the slump loss of steel fiber reinforced concrete (SFRC) have indicated that for concretes with lower cement contents of 362 kg/m³ (611 lb/yd³), the rate of slump loss is higher for SFRC (see Figure 3.2⁹). For concretes with higher cement contents of 474 kg/m³ (799 lb/yd³), the rate of slump loss is the same for both plain and steel fiber reinforced concretes (see Figure 3.3⁹).



Figure 3.2 Slump Loss vs. Elapsed Time (lower Cement content).



Figure 3.3 Slump Loss vs. Elapsed Time (higher cement content).

3.2.2 Setting Time

The time of setting of concrete is a very important parameter because it helps regulate the times of mixing and transit of the concrete mixture, and also helps plan the scheduling of the finishing operations. In addition, the setting time helps determine the effectiveness of various set-controlling admixtures.

The setting time of concrete depends on the hydration rate of cement in the mixture. When modified with latex polymers, the time of setting was found to be equivalent to or slightly longer than for conventional concrete (see Figure 3.4^{43}). When reinforced with steel fibers, the initial and final setting times and the rate of setting are essentially the same for plain and steel fiber reinforced concrete.³¹



latex solids/cement

Figure 3.4 Setting Time of LMC.

3.2.3 Impact Resistance

The impact resistance of concrete materials is an important factor in the design of systems such as concrete overlays on industrial floors and airfield pavements.

Latex, due to its film formation action inside the concrete matrix, gives the material some microcrack arresting properties which can potentially lead to improvements in the impact resistance of concrete.^{4, 23} Further test data are needed for verifying the latex modification effects on the impact resistance of concrete.

Steel fiber reinforcement has been shown in various investigations⁸¹ to significantly improve the impact resistance of the concrete matrix. Figure 3.5⁸¹ shows substantial improvements in impact resistance through steel fiber reinforcement. The improvements in impact resistance in the presence of fibers can be attributed to the crack-arresting action of fibers which provides concrete with a stabilized microcrack system, increased tensile strength, and desirable post-cracking tensile resistance.



Figure 3.5 Effect of Steel Fiber Reinforcement on the Impact Resistance of Concrete.⁸¹ (hooked-end steel fibers, length-to-diameter ratio=100)

The combined effects of steel fibers and Natural Rubber Latex on the impact resistance of concrete have been investigated by Sundara et al.⁸⁸ Table 3.2⁸⁸ represents the impact resistance test results of plain and steel fiber reinforced concretes with and without Natural Rubber Latex. The combined effect of latex and steel fibers are observed to increase the impact resistance of concrete by 9 to 10 times at first crack and 18 to 20 times at failure, over that of plain concrete. This is 2 to 3 times higher than what could be achieved independently either by latex modification or steel fiber reinforcement. Latex modification alone increases the impact resistance by 4 times at first crack and 9 times at failure. This is comparable to the effects of reinforcement by 1% volume fraction of steel fibers.

Material	Percentage of	Percentage of	Number of Blows		
	DRC, V _R	Fibers, V _f	At First Crack	At Failure	
Plain Concrete	0	0	5	6	
LMC	2.0 %	0	21	54	
Fiber Reinforced Concrete (SFRC)	0	1.0 %	25	61	
	0	1.5 %	33	72	
Latex-Modified SFRC	2.0 %	1.0 %	44	102	
	2.0 %	1.5 %	53	122	

Table 3.2Impact Resistance Test Results for Plain and Steel Fiber Reinforced
Concrete Modified with Natural Rubber Latex.⁸⁸

3.2.4 Flexural Performance

Latex modification of concrete provides the material with higher flexural strengths (see Figure 3.6).⁸⁰ This increase in flexural strength can be attributed to the microcrack-arresting action of polymers in concrete, and also to the bonding they provide between the matrix and aggregates. Improvements of workability through latex modification (which reduced water requirements for achieving similar workability in latex-modified concrete) is another factor contributing to flexural strength in latex-modified concrete.



Figure 3.6 Flexural Strength of Plain and Latex- (Styrene Butadiene) Modified Concrete Vs. Age.

Previous test results⁴ have indicated that, at a polymer-cement ratio of 0.20, Styrene Butadiene, Saran, Acrylic and PVA latexes provide flexural strengths of the order of 2, 3, 1.4, and 3 times, respectively, that of plain mortar after 28 days of dry curing at 50% relative humidity.

Steel fibers have been found to increase the first-crack and ultimate flexural strengths of concrete.⁸¹ They also make major contributions to the ductility and toughness (represented by the area under the load-deflection curve) of the material. Steel fibers, with their desirable pull-out performance, are especially effective at relatively large deformations and crack widths.

3.2.5 Moisture-Sensitivity

While moist curing is preferred for unmodified concrete (cement hydration proceeds in a moist environment), the formation of polymer network inside latex-modified concrete takes place as a result of water evaporation and thus requires a dry environment.^{43, 80} Furthermore, in latex-modified concrete, a polymer film tends to be formed near the concrete surface; this film is effective in retaining the moisture inside concrete at levels sufficient for the progress of cement hydration.⁴³ In LMC, the latex film formation on the surface of the material, which prevents rapid moisture loss from the concrete, provides conditions for wet curing within the concrete even though the material is cured in a dry environment.⁴⁰ Hence, latex-modified concrete, unlike conventional concrete, gains strength more desirably in dry environments following a relatively short period of moist curing (required for initial hydration of cement).

The actual wet exposure conditions encountered in field during the life period

of the structure may negatively influence the properties of LMC. It has been suggested that the rewetting of some latex concretes during the dry curing period may cause the latex to re-emulsify or redisperse, leading to reductions in strength properties of the material.⁸⁰

3.2.6 Compressive Strength Development in Concrete

Under normal temperature conditions some components of Portland cement start to hydrate as soon as they get in contact with the mixing water; but as the products of hydration coat the anhydrase cement grains, the hydration reactions start slowing down. In general, the longer the moist-curing period the higher the strength that can be reached at a given water-cement ratio. Figure 3.7⁵⁰ shows the influence of curing conditions on concrete strength with respect to the age. It is generally recommended that Portland cement concrete be moist-cured for a minimum period of 7 days. Figure 3.7 shows that if the concrete specimen is moist-cured during the entire curing period, the maximum compressive strength increases with age. The major increase in strength is seen to occur during the first 28 days of curing.

In the case of latex-modified concrete (LMC), the material must undergo a second curing mechanism in addition to the usual hydration of cement in conventional concrete.⁸⁰ This second curing mechanism consists of the coalescence of the latex particles to form a film that must cure to gain its maximum strength. It is important to note that the curing of this latex film is best achieved in a dry environment.



Figure 3.7 Influence of Age and Curing Conditions on the Compressive Strength of Concrete.⁵⁰

3.2.7 Bond Between New and Old Concrete

Old concrete is overlaid with new concrete in many circumstances. LMC is one excellent material used in repair applications over old concrete. Soroushian⁸⁰ reports that important improvements in bond strength between fresh and old concretes can be achieved through modification of fresh concrete by latex polymers, and that these improvements are important in repair and overlay applications of concrete.

The bond strength between old and new concretes has been measured by

different test methods such as adhesion in tension, flexure and shear (see Figure 3.8).^{65, 71} As this figure shows, adhesion tests in tension can be performed using specimen types TP and TB, adhesion in flexure using specimen F, and adhesion in direct shear using specimens SSC and SSB. Figure 3.9⁶⁵ summarizes the bond strength test results of LM mortars in various adhesion tests. It can be clearly seen that the bond strength of the LM specimens depends on the test method and the latex-cement ratio. It can also be concluded that the adhesion capacity of LM mortars increases with increasing latex-cement ratio regardless of the latex type used. An exception is to be made for adhesion in slant compressive shear which attains a maximum at a latex-cement ratio of about 5%. All specimens containing a latex-cement ratio of 20% showed lower adhesion in slant shear than plain mortars. Ohama⁶⁵ also found that adhesion in slant compressive shear is much larger than that of all other test methods investigated. This can be due to the fact that in this particular adhesion test, the effects of shear and compressive stresses are combined together.







Figure 3.8 Test Methods for Adhesion in Tension, Flexure and Compressive Shear.⁶⁵

Wall et al.⁵⁴ also report results of different test methods for measuring the bond strength between fresh and hardened concrete. It has been concluded that the slant shear test is most sensitive to the variations in bond strength. In addition, the results of this test had a low coefficient of variation (noting that some investigators⁴⁴ do not agree with this conclusion). This reference also investigated the effects of different parameters on the bond strength obtained with slant shear test. Such parameters include: water-cement ratio, effect of various curing conditions, and the effect of wetting the surface of the hardened concrete before application of the overlay material. It was concluded that pre-wetting the bond surface of the substrate concrete before application of the overlay has a small effect on bond strength. No conclusions were obtained regarding the effect of water-cement ratio. Curing of specimens in 100% relative humidity for 28 days before testing yielded smaller bond strength results when compared with the air curing condition.

Kuhlmann⁴⁴ suggests a new bond test. This test has yielded a coefficient of variation less than 10%. After one day of curing, this test shows a tendency towards failure in the LMC overlay. In fact, broken aggregates appeared in the fractured LMC at early ages, thus indicating an increase in the bond to the aggregate surface due to latex modification. Figure 3.10 indicates that after 90 days of curing, failure was predominantly in the base concrete.



Figure 3.9 Effects of Test Method on Adhesion (in Tension, Flexure and Compressive Shear) of Polymer-Modified Mortars With Various Polymer-Cement Ratios.⁶⁵



Figure 3.10 Tensile Bond Test Modes of Failure.44

3.2.8 Freeze-Thaw Durability

One of the major problems in cold climates requiring heavy expenditures for repair and replacement is the damage to concrete pavements, retaining walls and bridge decks attributable to frost action (freeze-thaw cycles). The frost damage in concrete can take several forms. The most common is cracking and spalling of concrete that is caused by aggressive expansion of the cement paste matrix under repeated freeze-thaw cycles. Frost damage may also occur in the aggregates.

When water turns to ice, the corresponding volume increase (~ 9%) causes the residual water in a capillary to be compressed. This pressure can be relieved if the water can escape from the capillary to a free space by diffusing through unfrozen pores, but if the water has too far to move to an escape boundary, the capillary will tend to dilate and the surrounding material will come under stress (see Figure 3.11).⁵¹



(a) Non Air Entrained Paste

(b) Air Entrained Paste

Figure 3.11 Creation of Hydraulic Pressure in Frozen Cement Paste.⁵¹

The superposition of pressure from adjacent capillaries will eventually cause the tensile strength of the paste to be exceeded and rupture will occur. As the temperature is lowered, more capillary water is involved in freezing, increasing the hydraulic pressure, and thereby increasing microcracking and dilation. When the cement paste is saturated and non-air-entrained, the only free space is the exterior of the specimen, and the diffusion of water is too slow to relieve the hydraulic pressure. Therefore, it is very advantageous to entrain some air into the cement matrix. The inclusion of entrained air provides empty space within the paste into which the excess water can move and freeze without damage. The spacing between the air bubbles determines the average distance the water must travel to reach the free space. This distance must not be too large if the hydraulic pressure accompanying ice formation is to be relieved.

Some rocks are susceptible to damage upon freezing and thawing and may therefore contribute to concrete damage. The same concept of development of hydraulic pressure is applied to these rocks. There is thus a critical size of aggregate above which it is liable to fracture. This critical size is a measure of the maximum distance water must flow to reach the outside surface in order to relieve the hydraulic pressure. For most rocks the critical size is greater than the maximum size used for coarse aggregates. Once saturated, the critical size of the rocks can become less than the size present in the aggregate and fracture in the aggregate can occur.

Air entrainment in concrete controls the escape boundaries in the cement paste matrix. It is not the total air, but the void spacing of the order of 0.1 to 0.2 mm (0.004 to 0.008 in) within every point in the hardened cement, that is necessary for

protection of concrete against frost damage.

The effect of adding latex to the concrete matrix on its freeze-thaw resistance is shown in Figure 3.12.⁵⁷ The introduction of different latex types at a latex-cement ratio of 5% showed substantial improvements in the freeze-thaw durability of the mortar. An increase in latex-cement ratio above 5%, however, does not seem to provide any additional resistance against repeated freeze-thaw cycles.





(b) PAE-Modified

(c) EVA-Modified



Adding steel fibers to the concrete matrix does not seem to have a major effect on the freeze-thaw durability of concrete. For the case of high entrained air contents, Figure 3.13(a)¹⁰ shows that SFRC has a slightly better durability than plain concrete. For concretes with less air content, SFRC shows better durability for three out of five mixtures as seen in Figures 3.13(b) and 3.13(c).¹⁰ It is then concluded that FRC behavior is similar to that of plain concrete under freeze-thaw cycles at the same air content.

The water-cement ratio also has an important effect on the freeze-thaw durability of concrete. In general, the higher the water-cement ratio for a given degree of hydration, the higher will be the volume of large capillary pores in the hydrated cement paste, and therefore the greater the amount of freezable water will be. It was found¹⁰ that for a water-cement ratio of more than 0.4, at a cement content less than 415 kg/m³ (700 lb/yd³), and with a maximum aggregate size of 25 mm (1 in), a minimum of 6.0%, preferably 8%, air content should be used to avoid deterioration under freeze-thaw cycling. However, since in most field cases the water-cement ratio is greater than 0.4, if the structure is going to be exposed to freezing and thawing, 8% air should be specified.

The aggregate grading also affects the volume of entrained air, which tends to be decreased by an excess of very fine sand particles. Addition of admixtures such as fly ash has a similar effect. In general, a more cohesive concrete mixture is able to hold more air than either a very wet or a very stiff concrete.



(a) High Air Content



(b) Low Air Content

Figure 3.13 Comparisons Between the Freeze-Thaw Durability of Plain and Steel Fiber Reinforced Concretes.¹⁰

3.2.9 Scaling Resistance

Concrete that is adequately air-entrained for frost resistance may nevertheless be damaged by repeated application of deicing salts. Concrete that has suffered salt scaling becomes roughened and pitted due to the spalling of small pieces of mortar.

It has been suggested that the consumption of heat required to melt ice when deicer is applied causes a rapid drop in the temperature of concrete just below the surface, which may cause damage either from the effects of rapid freezing or stress caused by differential thermal strains. The additional free moisture present at the surface of concrete may encourage the growth of microscopic or macroscopic ice lenses near the surface, where ice formation can still occur. Osmosis has also been suggested as a mechanism of salt scaling. De-icing chemicals can accumulate in concrete just below its surface to form relatively concentrated solutions.⁵¹

Scaling is most likely to occur on surfaces that have been over-vibrated, troweled too early and too long, subjected to plastic shrinkage, or where excessive bleeding has occurred. Such surfaces tend to have a weak layer of paste or mortar either at the surface or just below it, and may have microcracks or bleeding channels that can transport surface solutions to lower levels. If adequate moist-curing is followed by a period of drying before de-icing chemicals are applied, scaling should not be a problem. Table 3.3⁹⁸ indicates recommended curing times for air-entrained pavements.

Polymer modification has been observed to improve the scaling resistance of mortar. Following a freeze-thaw scaling test in the presence of rock salt (ASTM C-29 modified by Illinois Highway Department),⁴⁷ as shown in Figure 3.14, the acrylic

Cement Type	Minimum Curing Period (days)				
	23 °C (73 °F)	4 ℃ (39 °F)	-4 ℃ (25 °F)		
I	7	15	>60		
ш	7	7	24		

Table 3.3 Minimum Moist-Curing Times to Develop Salt Scaling Resistance.⁹⁸



Figure 3.14 Freeze-Thaw Scaling Resistance of Acrylic Modified Mortar.47

3.2.10 Physical Properties

3.2.10.1 Water Absorption and Specific Gravity: The water absorption of concrete has been found⁶¹ to be reduced significantly by increasing the polymer-cement ratio, but hardly changed by increasing the steel fiber volume fraction as shown in Figure 3.15.



Figure 3.15 Steel Fiber Content and Polymer-Cement Ratio vs. 48-hour Water Absorption of Concrete.⁶¹

3.2.10.2 Drying Shrinkage: Dry shrinkage is a very important phenomenon in hardened concrete materials. In fact, not allowing adequately for the effects of drying shrinkage in concrete design and construction may lead to severe cracking or warping of elements of the structure due to restraints present in structures against shrinkage.⁶¹

Many factors affect shrinkage of concrete. These include:

- Paste Parameters: porosity, curing temperature, cement composition, moisture content, and admixtures.
- Concrete Parameters: aggregate stiffness, aggregate and cement contents, and thickness.
- Environmental Parameters: relative humidity, rate of drying, and time of drying.

Once a concrete matrix has dried, part of the total shrinkage that occurs is irreversible (Figure 3.16),⁵⁰ thus implying an improvement in the dimensional stability after first drying of concrete and this makes the use of precast concrete products advantageous.

Moisture loss and drying shrinkage are closely related (see Figure 3.17).⁵⁰ In this figure, domains (1) and (2) are attributed to loss of water from capillary pores, domain (3) represents loss of adsorbed water from the surfaces of C-S-H particles (which constitute much of the cement hydration products), domain (4) results from loss of water that contributes to the structure of C-S-H, and domain (5) is due to decomposition of C-S-H.



Figure 3.16 Typical Behavior of Concrete Upon Drying and Rewetting.⁵⁰



Figure 3.17 Shrinkage-Moisture Loss Relationships in Pure Cement Pastes During Drying.

Three mechanisms are known to cause drying shrinkage of cement paste, namely, capillary stress, disjoining pressure, and surface tension (see Figure 3.18).

• Capillary Stress: The larger capillaries, which are emptied at relative humidities down to about 95%, have a relatively large volume-to-surface ratio and develop rather small stresses. At lower humidities, although the capillary volume becomes much smaller, stresses rise quite rapidly. These stresses in the presence of water, cause dilation; loss of water thus leads to shrinkage.

• Disjoining Pressure: An assembly of colloidal particles, as in C-S-H surface, creates a disjoining pressure that increases with increasing thickness of the adsorbed water (i.e., increasing relative humidity). The particles will be forced apart when this disjoining pressure exceeds the Van der Waals' attractions thus creating a dilation; loss of water thus produces shrinkage.

• Surface Free Energy: When no capillary stress or disjoining pressure are present (below 40% R.H.), shrinkage is explained by changes in surface energy. A solid particle is subjected to a mean pressure of:

$P_{sfe} = 2GS/3$

where,
$$G = surface energy, J/m^2$$

S = specific surface area, m²/g

Since S is large for C-S-H, P_{ste} can be large and causes compression in the solid. The adsorption of water on the surface decreases G and leads to reduced P_{ste} values. Loss of water which causes an increased compression in the solid due to increased surface energy thus leads to shrinkage movements.



100% RH





Dry





 $d \sim 10 ~ \text{\AA}$ 60% RH

FIT 50% RH



(c)

(Ь)

Mechanisms for Causes of Drying Shrinkage of Cement Paste:(a) Capillary Stress; (b) Disjoining Pressure; (c) Surface Tension. Figure 3.18

The drying shrinkage of concrete is less than that for paste due to the existence of aggregates that have restraining effects. Generally speaking, aggregates are dimensionally stable under changing moisture conditions. The amount of restraint provided depends on their amount, stiffness, and also on the maximum size of coarse aggregates. The stresses at the cement paste-aggregate interfaces due to drying shrinkage increase as the maximum aggregate size increases. These higher internal stresses result in increasing amounts of cracking in the interfacial region.

The specimen geometry, size and shape will determine the rate of moisture loss and, therefore, the rate and magnitude of drying shrinkage. The composition of cement can also influence the shrinkage of the cement paste and/or concrete. Its effect, however, is not large.

3.3 MATERIALS AND MANUFACTURING PROCEDURES

3.3.1 Materials

The materials used in this study were cement, gravel, sand, latex, steel fibers, and regular tap water. The following is a brief description of these materials:

3.3.1.1 Cement: Regular Type I Portland cement was used in all the mixtures. Table 3.4 presents the chemical and physical properties of the cement.

Compound	CaO	SiO ₂	Al ₂ O ₃	Fe ₂ O ₃	MgO	SO3	K ₂ O
Content 63.2 21.1 5.76 2.93 2.06 (%)					2.46	0.79	
Specific Gravity 3.15							
Specific Surface (m ² /kg) 160							

Table 3.4Properties of the Portland Cement Type I Used in this Study.

3.3.1.2 Sand: Natural sand with gradation curve shown in Figure 3.19 was used in this study. This gradation satisfies the ASTM C-33 requirements for fine aggregates.

3.3.1.3 Gravel: The coarse aggregate used in this investigation was crushed limestone with maximum particle size of 12.7 mm (1/2 in). The coarse aggregate gradation curve is shown in Figure 3.20. This gradation satisfies the ASTM C-33 requirements for coarse aggregates, and is categorized by the Michigan Department of Transportation as Grade 25A.



Figure 3.19 Gradation Curve of Fine Aggregates.



Figure 3.20 Gradation Curve of Coarse Aggregates.

3.3.1.4 Latex: The latex polymer used in this research project is BASE

Styrofan 1186 Styrene-Butadiene dispersion (see Table 3.5 for properties)

Typical Properties	
Total Solids (wt. %)	47
Specific Gravity	1.01
рН (25 °C)	10
Surface Tension, mN/m (lb/in)	38 (5.7 x 10 ⁶)
Weight/Volume, kg/l (lb/U.S. gal)	1.01 (8.3)

Table 3.5Properties of the BASE Styrofan 1186 Polymer.

3.3.1.5 Steel Fibers: The steel fibers used in this research were hooked-end DRAMIX steel fibers with a length of 30 mm (1.18 in) and a diameter of 0.5 mm (0.0197 in). The key properties of these fibers are presented in Table 3.6.

Table 3.6	Typical	Properties	of Steel	Fibers	Used.
	~ ~				

Length, mm (in)	Diameter mm (in)	Aspect Ratio	Cross- Sectional Area, mm ² (in ²)	Specific Gravity	Tensile Strength MPa, ksi	Elonga- tion at Break (%)
30 (0.5)	0.5 (0.02)	60	0.196 (.000304)	7.84	1175 (170)	2.5

3.3.2 Manufacturing Procedure

Latex-modified steel fiber reinforced concrete mixtures (with different latex and fiber contents) were manufactured in a rotary-type drum mixer with a capacity of 0.04 m^3 (1.14 ft³) following the procedure described below:

1. mix the latex with water;

2. add the coarse and fine aggregates plus 2/3 of the liquid (i.e., mixture of latex and water) and mix for about 2 minutes;

3. add 2/3 of the cement (while the mixer is running) and mix for 2 to 3 minutes or until a uniform mixture is achieved;

4. add the steel fibers in a gradual manner;

5. add the remainder of the liquid and cement, and mix for about 3 minutes or until a homogeneous mix is achieved; and

6. stop the mixer for 2 to 3 minutes, and then mix for an additional 2 minutes.

3.4 EXPERIMENTAL PROGRAM

Fiber reinforced concrete researchers usually use a wide range of steel fiber volume fractions in their mixes. These volume fractions range from 0.25 up to 4% of the total concrete batch volume. In real life applications, however, fiber volume fractions below 1% are most commonly used. Soroushian⁸² reports that the best fiber performance was produced with hooked 50 mm x 0.5 mm (1.97 in x 0.02 in) steel fiber additions in volume fractions ranging from 0.5 to 1.0 % by volume. This

relatively small fiber content provides desirable performance characteristics in the hardened state without significantly damaging workability in the fresh state. Furthermore, since with this small fiber content SFRC is very comparable to unreinforced concrete, it can be more conveniently introduced to the market. In this study, in order to devise an experimental program, the latex and fiber contents were determined to provide acceptable fresh mix qualities while presenting promises for desirable hardened material properties.

3.4.1 Fresh Mix Optimization

The mix proportions obtained in this study after a series of trials and errors are presented in Table 3.7. These mixes were selected to have comparable workability characteristics, represented by a slump of 50 to 70 mm (2 to 3 in) for plain (nonmodified and latex modified) concretes, and with an inverted slump cone time of 25 sec and a VeBe time of 7 to 9 sec for the steel fiber reinforced concrete mixes (with and without latex).⁸²

V _r (%)	L/c (%)	w/c	slump, in (mm)	Vebe Time (sec)	Air Content (%)
0	0	0.43	6.0 (152)		5.5
0	10	0.32	7.0 (190)		4.5
0.75	0	0.45	5.0 (127)	6.5	6.5
0.75	10	0.34	6.0 (152)	5.0	5.0

Table 3.7Mix Proportions Used in Experimental Work.

where,	L/c = latex-cement ratio, by solids weight;
	w/c = water-cement ratio, by weight;
	s/c=2.5 = sand-cement ratio, by weight; and
	g/c=1.5= gravel-cement ratio, by weight.

The test procedures followed in this investigation are introduced below.

3.4.2 Comprehensive Characterization of the Materials

For the materials presented in Table 3.7, a comprehensive test program was conducted and fresh and hardened material properties in order to determine the individual and joint effects of steel fibers and latex polymers on diverse qualities of concrete. The test procedures followed are described in the following:

3.4.2.1 Time of Setting by Penetration Resistance: The test procedure outlined in ASTM C-403 was used for this purpose. This test consists of removing the mortar fraction of the concrete by passing it through a No. 4 (4.75 mm, 0.187 in.) sieve, rodding the mortar into the container, and then measuring the pressure required to cause a needle to penetrate 25.4 mm (1 in) into the mortar. Removable needles with bearing areas ranging from 645 to 16 mm² (1 to 0.025 in²) are used. This is done at regular intervals and a curve of penetration resistance versus time is plotted. Times of initial set and final set are defined as the times at which the penetration resistance reaches 3.5 MPa (500 lb/in²) and 27.6 MPa (4000 lb/in²), respectively.

3.4.2.2 Workability Loss: This test was performed by measuring the loss of slump and VeBe time for non-fibrous and fibrous mixes, respectively, at different

time intervals from the time cement and water came in contact. Before each workability test (at any time interval), the concrete was remixed manually.

3.4.2.3 Impact Resistance: Three cylinders 152 mm (6 in) in diameter and 63.5 mm (2.5 in) in height were prepared from each of the 4 mixes. They were moist-cured inside their molds for the first 24 hours and then demolded and cured in air until the test age of 28 days. The impact test is performed by repeatedly dropping a 4.5 kg (10 lb) hammer on the cylinder from a height of 457 mm (18 in).^{3.22} The number of blows required to cause the first visible crack and the ultimate failure represents the impact resistance of the material. Ultimate failure is assumed to occur when cracks open sufficiently so that the pieces of concrete are touching three of the four positioning lugs on the base plate. Figure 3.21a presents the impact resistance test apparatus.

3.4.2.4 Moisture Sensitivity: This test was performed on the fibrous mixes only. Four different cases were investigated. Flexure specimens with 102 mm (4 in) square cross-sections were tested by 4-point loading on a span of 305 mm (12 in) (see Figure 3.21b). The moisture-sensitivity of the material at various ages was investigated, and it was also checked if the moisture effects can be eliminated through drying. The following curing (moisture) conditions were considered: <u>Condition 1:</u> Three specimens were cured for 1 day inside their molds under wet burlap and plastic sheet and then in water until the age of 28 days when they were

<u>Condition 2:</u> Three specimens were cured for 1 day inside their molds under wet burlap and plastic sheet and then in water until the age of 26 days when they were

tested wet.

dried in air until the test age of 28 days.

<u>Condition 3:</u> Three specimens were cured for 1 day inside their molds under wet burlap and plastic sheet and then in air until an age of 26 days when they were further cured in hot water (80 °C, 176 °F) for 2 days before being tested at 28 days of age. <u>Condition 4:</u> Three specimens were cured for 1 day inside their molds under wet burlap and plastic sheet and then in air until an age of 26 days when they were further cured in water at ambient temperature for 2 days before being tested at 28 days of age.



(a) Impact Test



(b) Flexure Test. Figure 3.21 Apparatus for Impact Resistance and Flexure Tests.³

3.4.2.5 Compressive Strength Development With Time: The purpose of this test is to study the compressive strength development of concrete with age. For this purpose, eight 152 mm diameter by 305 mm high (6 in diameter by 12 in high) cylinders were prepared for each of the mixes of Table 3.7. These specimens were then tested under compression at the following curing ages:

- two tested at 1 day of age;
- two tested at 7 days of age;
- two tested at 50 days of age; and
- two tested at 100 days of age.

It should be noted that all these specimens were moist-cured inside their molds for the first 24 hours of age, and were then exposed to air.

3.4.2.6 Bond Strength: For each of the mixes of Table 3.8, three bond-to-old-concrete specimens were prepared and tested as described in Figure 3.22.

The purpose of this test is to study the bond strength of new (overlay) concrete to old (base) concrete. The test method used is a modification of the ASTM C-882 slant shear bond strength test. The specimens were manufactured by casting one half of the slant shear specimen with the base made of plain concrete and the second half of the specimen with the overlay repair material two weeks after casting the base material. The dimensions of the completed specimens were 7.62 mm x 15.24 mm (3 x 6 in) cylinders. The angle of the shear plane was approximately 30 degrees with respect to the longitudinal axis of the cylinder (see Figure 3.22).
Before casting the overlay material, the base surface to be in contact with the overlay was first abraded with sand paper and then moistened with water just before applying the overlay. This simulates the routine field practice for enhancing the bond between the two materials.

The completed specimens were kept under wet burlap for 24 hours and were then further cured for 27 days in laboratory air before being tested under compression.

The compressive strength obtained will represent the bond strength of conventional concrete to conventional concrete, LMC, SFRC, and LMSFRC. This compressive strength is calculated based on the horizontal cross-sectional area of the cylindrical specimen.

The base concrete mix proportions used are shown in Table 3.9. These mix proportions should yield a high strength concrete with 34.5 MPa (5000 psi) or more compressive strength to assure that the test truly measures the bond strength of the repair concrete and not the strength of the substrate.

Mix No.	L/c (%)	V _f (%)	w/c	AEA/c
1	0	0	0.42	0.08
2	5	0	0.32	0
3	10	0	0.30	0
4	0	0.5	0.38	0.09
5	5	0.5	0.33	0
6	10	0.5	0.30	0
7	0	1	0.39	0.1
8	5	1	0.35	0
9	10	1	0.31	0

 Table 3.8
 Selected Mix Proportions for Bond Test Experimental Work.

Table 3.9Base Material Mix Proportions.

V _f (%)	L/c	w/c	sup./c (%)	AEA/c (%)
0	0	0.35	0.75	0.19



(a) Cast Substrate Concrete

(b) Prepare Surface With Sand Paper



(c) Put Substrate Concrete in Mold (d) Finished Slant Shear Specimen

Figure 3.22 Method of Preparation of Specimens for Bond Test.

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3.4.2.7 Freeze-Thaw Durability: The purpose of this test is to study the effects of several freezing and thawing cycles on the durability of concrete. The test method used to perform this task is ASTM C-666 (procedure A, Rapid Freezing and Thawing in Water). Three specimens were prepared for each of the mixes of Table 3.7. The specimens were rectangular prisms of dimensions 76 x 102 x 406 mm (3 x $\frac{1}{2}$ 4 x 16 in). They were cured for 1 day inside their molds underneath a wet burlap and a polyethylene film, and then dry-cured in air until the age of 14 days at which time they were first immersed in water for 48 hours, and then put inside the freeze-thaw machine to start the testing cycles. The test procedure consists of subjecting the specimens alternatively to freezing and thawing periods. This is accomplished by lowering the temperature of the specimens (that are surrounded by not less than 1 mm (1/32 in) nor more than 3 mm (1/8 in) of water) from 4.4 to -17.8 °C (40 to 0 °F) and then raising it from -17.8 to 4.4 °C (0 to 40 °F) with not less than 2 nor more than 5 hours spent on the complete cycle. At least 25% of this time is used for thawing. The specimens are removed from the apparatus, in a thawed condition, at intervals not exceeding 36 cycles of exposure to the freezing-and-thawing. They are tested for fundamental transverse frequency (from which the dynamic modulus of elasticity of the material can be computed) and then returned back into the apparatus.

3.4.2.8 Scaling Resistance: The test procedure outlined in ASTM C-672 was used for this purpose. This test method determines the resistance to scaling of a horizontal concrete surface exposed to repeated freezing-and-thawing cycles in the presence of deicing chemicals.

A set of the set of



Two specimens were prepared for each of the four mixes of Table 3.7 (plain concrete; latex-modified concrete (LMC); steel fiber reinforced concrete (SFRC); and latex-modified steel fiber reinforced concrete (LMSFRC)). The prismatic specimens used were 254 x 203 x 76 mm (10 x 8 x 3 in) in dimensions (see Figure 3.23); they were moist-cured inside their molds for the first 24 hours, then demolded and cured in air. At the test age of 28 days, the flat surface of each specimen was covered with approximately 6 mm (1/4 in) of a solution of calcium chloride and water, having a concentration such that each 100 ml of solution contains 4 g of anhydrous calcium chloride. The specimens were then placed in a freezing environment for 16 to 18 hours. At the end of this time, they were removed from the freezer and placed in laboratory air at 23 ± 1.7 °C (73 ± 3 °F) for 6 to 8 hours. This cycle was repeated daily with the surface being flushed off thoroughly at the end of each 5 cycles. A visual examination was then made and the solution was replaced. This test was carried out for 50 cycles.



Figure 3.23 Specimen Used in Scaling Resistance Test.

 $(x_1, y_2, y_3) \in \mathbb{R}^{2^{-1}} \times \mathbb{R}^{2^{-1}}$

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3.4.2.9 Water Absorption and Specific Gravity: The mix proportions used in this phase of the experimental study are presented in Table 3.10. All the mixes had comparable workability characteristics represented by a slump of 63 to 89 mm (2.5 to 3.5 in).

V _f (%)	L/c (%)	w/c
0	0	0.43
0	5	0.38
0	10	0.32
0.5	0	0.44
0.5	5	0.40
0.5	10	0.33
1	0	0.45
1	5	0.42
1	10	0.38

 Table 3.10
 Mix Proportions for the Water Absorption and Specific Gravity Tests.

ASTM C-642 test procedure "Specific Gravity, Absorption, and Void in Hardened Concrete" was used for determination of the absorption capacity as well as the specific gravity of the concrete materials investigated. One specimen was tested for each of the mixes of Table 3.10.

3.4.2.10 Drying Shrinkage: The length comparator test technique (ASTM C-490) was used to test drying shrinkage. Three specimens measuring $51 \times 51 \times 254$ mm (2 x 2 x 10 in) were tested for each of the mixes of Table 3.7. All the specimens were cured inside their molds and underneath a wet burlap and a polyethylene sheet for the first 24 hours after casting. They were then stripped off

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the molds and air-dried at 22-24 °C(72-75 °F) and a relative humidity of 45-55 %. Changes in length were recorded after 1, 3, 7, 9, 14, 16, and 25 days of age.

3.5 EXPERIMENTAL RESULTS, ANALYSES AND DISCUSSIONS

This section presents the raw test data and conclusions derived based on statistical analyses of the test results generated in this investigation.

3.5.1 Time of Setting

The results of the time of setting test by "Penetration

Resistance" for mixes of Table 3.7 are presented in Table 3.11 and Figure 3.24.

Test Conditions		Test Results			
V _f (%)	L/c	T (°F)	R.H.(%)	Initial Setting Time (hrs.)	Final Setting Time (hrs.)
0	0	81	46	3.6	4.75
0	10	79	50	3.8	5.4
0.75	0	83	69	2.65	3.75
0.75	10	83	51	3.4	4.8

Table 3.11Time of Setting Test Results For Mixes of Table 3.7.

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The results presented in Table 3.11 and Figure 3.24 indicate that latex modification tends to increase both the initial and final times of setting in mixes without steel fibers. The addition of steel fibers, on the other hand, tends to lower both times of setting irrespective of the latex content in the matrix. The increased setting times in the presence of latex may result from the formation of polymer films inside cement paste which interfere with moisture movements during cement hydration, thus making water less accessible to cement particles. The formation of polymer film at the cement surface which prevents moisture loss from the specimen may also help in elongating the setting time of concrete in the presence of latex. e de la companya de l La companya de la comp La companya de la comp

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Steel fibers may reduce setting time because in this test fibers (and coarse aggregates) are actually removed from the matrix prior to conducting the penetration test on the mortar fraction of the material for measuring the setting time. The removed fibers are typically wet and thus some extra water is lost from the matrix with the fibers, and the drop in water content may lead to faster setting. There seems to exist a need for developing setting time test procedures that more suitably apply to steel fiber reinforced concrete (SFRC).

The ambient temperature during the setting tests was almost the same for all four cases. The relative humidity (R.H.), however, was noticeably higher for the SFRC mix (case 3). Since, in general, the setting time increases with increasing relative humidity, it then follows that the setting times obtained for unmodified SFRC would have been even lower if the corresponding R.H. during the setting tests was similar to those in the other cases.

3.5.2 Loss of Workability With Time

The results of the loss of slump and VeBe time (only for fibrous mixes) with time are shown in Figures 3.25a and 3.25b, respectively. Figure 3.25a seems to indicate that the loss of workability with time takes place at almost the same rate for all mixes irrespective of the latex and steel fiber contents. Figure 3.25b shows the loss in VeBe time with elapsed time for fibrous mixtures with and without latex. The latex-modified SFRC mixtures seem to be capable of maintaining their workability (under vibration) for a shorter time than the unmodified ones. More test data are needed to confirm this observation.



(a) Loss of Slump with Time (for plain and fibrous mixtures)



(b) Loss of VeBe Time with Time (only for fibrous mixtures)

Figure 3.25 Loss of Workability with Time.

3.5.3 Impact Resistance

The impact resistance test results obtained for the mixes of Table 3.7 are given in Table 3.12 and Figure 3.26.

Table 3.12 Impact Resistance Test Results for Mixes of Table 3.7.

V _f (%)	L/c (%)	Number of Blows to First Crack	Number of Blows to Failure
		29	33
0	0	25	28
		32	35
		232	233
0	10	503	503
		50	53
		60	106
0.75	0	190	247
		154	264
		900	1065
0.75	10	395	530
		148	278



Figure 3.26 Impact Resistance Test Results.

From Table 3.12 and Figure 3.26 it can be seen that latex addition increases the impact resistance of the concrete matrix by an average of 800%. Steel fiber reinforcement also increases the impact resistance by an average of 370%. When both steel fibers and latex are added to the plain concrete matrix, the improvements in impact resistance are superior (an average of 1500% increase over plain concrete). This indicates that there is an effective interaction between the steel fibers and the latex, which possibly results from the improved fiber-to-matrix bonding in the presence of latex polymers, and also from the increased compatibility of steel fibers and the matrix resulting from the reduced brittleness of the matrix incorporating latex.

A factorial analysis of variance performed on the obtained impact test results has revealed that the effects of latex and steel fibers on the impact resistance are significant at 95% level of confidence, but their interaction is not.

It is also interesting to note that when fibers are missing from the matrix, even in the presence of latex, there is only a small difference between the first crack and the failure impact resistance. This indicates that fibers, but not latex, can provide the material with post-cracking integrity.

In order to statistically investigate the effects of latex modification and steel fiber reinforcement on impact resistance, one-way analyses of variances were conducted. A one-way analysis of variance (ANOVA) performed for the cases of unmodified and latex modified plain concretes (fiber volume fraction, $V_f=0\%$) indicated that the effects of latex content on the impact resistance of plain concrete are significant with about 15.2% chance of error in this statement. A similar one-way ANOVA performed for the cases of unmodified and latex modified SFRC

 $(V_f=0.75\%)$ resulted in very similar conclusions, with about 15.3% chances of error in stating that there is a latex effect on the impact resistance at $V_f=0.75\%$.

Similar analyses were performed to investigate the effects of steel fiber reinforcement at constant latex content on the impact resistance of the concrete matrix. For the case of unmodified concrete (latex-cement ratio, L/c=0%), the one-way ANOVA showed that fiber addition has a significant positive (i.e., strengthening) effect on the impact resistance, with only 2.6% chances of error in this statement. At a constant L/c of 10%, the corresponding one-way ANOVA revealed that there is about 24.6% chance of error in stating that there is a fiber effect on impact resistance of concrete at L/c=10%.

It is worth mentioning that the scatter in impact resistance test results is relatively large. This leads to conditions where, in spite of the large differences in average values of impact resistance for different mix proportions, the chance of error in stating that such a difference exists sometimes exceeds 15%.

3.5.4 Flexural Performance

The average flexural load-deflection curves for the four mix compositions considered in this investigation are presented in Figure 3.27. The improvements resulting from latex modification and steel fiber reinforcement, and the desirable joint effects of latex and steel fibers, are obvious in this figure. The flexural strength and toughness (defined as the area underneath the flexural load-deflection curve up to a flexural deflection equal to the span length divided by 150) test results obtained for the mixes of Table 3.7 are presented in Table 3.13.





V _f (%)	L/c (%)	Flexural Strength (ksi)	Flexural Toughness (k-in)
		0.420	0.00003
0	0	0.467	0.00005
		0.522	0.00002
		0.880	0.0046
0	10	0.868	0.0058
		0.775	0.0034
		0.893	0.254
0.75	0	0.869	0.207
		0.881	0.231
		1.018	0.339
0.75	10	1.235	0.425
		1.126	0.382

Table 3.13Flexural Strength and Toughness Test Results.

3.5.4.1 Flexural Strength: From Table 3.14 and Figure 3.28, it can be seen that latex addition (L/c=10%) increases the flexural strength of plain concrete by about 87%, whereas steel fiber reinforcement ($V_r=0.75\%$) increases it slightly more by about 92%. When plain concrete is modified with both latex polymers and reinforced with steel fibers, the improvements in flexural strength are more significant (an increase of about 150% over plain concrete).

A factorial analysis of variance performed on the above results confirmed the significance of the effects of latex polymers, steel fibers, and their interaction, at 95% confidence level. The relative significance of the effects of fibers, latex, and their interaction on the flexural strength, as indicated by the factorial analysis of variance, is presented in Table 3.14.



Figure 3.28 Flexural Strength Test Results.

Table 3.14Factorial Analysis of Variance of Flexural Strength Test Results.

Factor	Importance
Steel Fibers	**
latex Polymers	**
Interaction	**

where, "**" very significant "*" significant

3.5.4.2 Flexural Toughness: Steel fiber reinforcement is observed in Table 3.15 and Figure 3.29 to have positive effects on the flexural toughness of concrete, while the improvements in toughness resulting from latex modification are relatively small. Latex modification, however, is highly effective in improving toughness characteristics in the presence of steel fibers.

A factorial analysis of variance confirmed that the effects of latex and steel fibers on flexural toughness, as well as their interaction, are important at 95% level of confidence (see Table 3.15).



Figure 3.29 Flexural Toughness Test Results.

Table 3.15 Factorial Analysis of Variance of Flexural Toughness Test Results.

Factor	Importance
Steel Fibers	**
latex Polymers	**
Interaction	•

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p. er The significant improvements in flexural toughness of SFRC resulting from latex modification (noting that latex has relatively small effects on the flexural toughness of plain concrete) can be attributed to the improvements in fiber-to-matrix interfacial bond characteristics with the addition of latex polymers. In order to confirm this, microscopic pictures were taken of the steel fibers pulled out of unmodified and latex-modified steel fiber reinforced concretes at the fractured surfaces of the flexural specimens. As shown in Figure 3.30a, the steel fibers pulling out of unmodified concrete were clean, indicating an interface shear failure. The steel fibers pulling out of LMC (Figure 3.30b) were partially coated with the polymer-cement comatrix, indicating that the interface shear strength was strong enough to encourage shear failure in the matrix further away from the interface zone.



(a) Case of Steel Fibers Pulling Out of Unmodified Concrete.



(b) Case of Steel Fibers Pulling Out of Latex-Modified Concrete.

Figure 3.30 Microscopic Pictures of Pulled Out Steel Fibers.

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3.5.5 Moisture Sensitivity

The results of tests on the sensitivity of the flexural strength of unmodified and latex modified SFRC to various moist curing conditions are presented in Table 3.16 and Figure 3.31.

Table 3.16	Moisture-Sensitivity	Test	Results.
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Curing	Flexural Strength (ksi)			
Method	SFRC	LMSFRC		
	1.137	0.822		
1	1.131	0.979		
	1.203	0.973		
	0.738	0.787		
2	0.935	0.912		
	0.953	0.966		
	0.811	0.808		
3	0.864	0.758		
	0.655	0.897		
	0.783	1.276		
4	0.939	1.211		
	0.648	1.171		



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Figure 3.31 Moisture-Sensitivity Test Results.

From this figure it may be concluded that the flexural strength of LMSFRC is larger than that of SFRC in all cases except for curing method 1 (i.e., continuous immersion in water). This and other trends observed in Figure 3.31 indicate that wet curing at early ages is detrimental to mixtures containing latex polymers, while it plays a positive role in mixtures without latex. By comparing the results of methods 1 and 2, we can deduce that the last two days of air drying in method 2 even out the flexural strength differences between SFRC and LMSFRC mixes by reducing the SFRC flexural strength values. A comparison between methods 3 and 4 shows that curing the specimens in hot water after a period of air drying reduces the flexural strength of LMSFRC below the value corresponding to curing in water at ambient temperature after the same period (26 days) of air drying.

A one-way ANOVA performed for the first two curing methods (i.e., 28 days of curing in water vs. 26 days of curing in water followed by 2 days of drying in air) showed that the flexural strength of nonmodified SFRC is significantly affected by these two curing techniques. There is, in fact, only about 1.8% chance of error in stating that these two curing methods produce different flexural strengths. However, the one-way ANOVA performed on the LMSFRC mixes for these two curing methods revealed that there are no significant effects of these curing methods on the flexural strength. There is a 65% chance of error in stating that there actually is an effect of curing on flexural strength.

A similar ANOVA was performed for methods 1 and 4 of curing (i.e., 28 days of wet curing vs. 26 days of air drying followed by 2 days of wet curing). The obtained results indicated that there are significant effects on the flexural strength of both SFR and LMSFR concretes when using curing methods 1 and 4, with about 18.9% and 1.4% chances of error in these statements, respectively.

Similar analyses have shown that there are significant effects on the flexural strength of LMSFRC when using method 3 of curing (i.e., 26 days of air drying followed by 2 days of hot water curing) vs. method 4 (i.e., 26 days of air drying followed by 2 days of wet curing) with only a 0.14% chance of error in this statement. However, there is no effect on the flexural strength of SFRC when using either of curing methods 3 or 4. There is, in fact, a 90.5% chance of error in stating that there is a difference in the flexural strength of SFRC resulting from substituting curing method 3 with 4.

In order to investigate the effects of latex modification on the SFRC mixes, one-way ANOVA studies were performed on unmodified vs. latex-modified mixes for each curing method separately. The results indicated that the effects of latex modification on the SFRC mixtures are very significant when curing method 1 or 4 is used. There are, in fact, only about 1.5 and 0.87% chances of error in the above statements, respectively. It should be noted that in curing method 1 (which involves wet curing at early ages), the effect of latex modification on flexural strength is negative, while in curing method 4 (which involves air drying at early ages), latex has positive effects on the flexural strength.

Latex modification, however, did not seem to have an effect on the flexural strength of SFRC when subjected to either curing method 2 or 3. The one-way ANOVA of results revealed that there are about 88.8 and 58.5% chances of error, respectively, if one were to state that latex modification actually has an effect on the flexural strength of SFRC when cured by method 2 (which involves wet curing at early ages) or 3 (which involves exposure to hot water at an age of 26 days).

3.5.6 Compressive Strength Development With Time

Trends in the development of compressive strength of different concrete mixtures with time are presented in Figure 3.32. It should be noted that test data could not be obtained on the 100-day strength of LMC specimens due to experimental difficulties.

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 $V_{1} = M_{1} M_{2} + \frac{1}{2} M_{1} + \frac{1}{2$



Figure 3.32 Trends in Strength Development With Time for Different Concrete Mixes.

Figure 3.32 indicates that the latex-modified concrete mixtures have a tendency to continue with strength development at later ages. This could result from the desirable internal curing conditions in the latex-modified mixtures exposed to air. The polymer film formed at the surfaces and inside the latex-modified concrete provides the material with an internal moist-curing even though the specimen is not externally moist-cured in a humid environment.

It should be noted that all the specimens were air cured after 1 day of moist-curing inside the molds. Since unmodified concrete does not possess the capability for internal moist-curing, this early-age air-drying negatively influences the strengthening with time in unmodified plain and steel fiber reinforced concrete mixtures. However, one should notice that continued moist-curing might be a costly measure.

A two-way factorial ANOVA was conducted to investigate the effects of curing time (from 7 to 100 days) on compressive strength of all concrete matrices and also the effects of the concrete matrix type on compressive strength. The results of this analysis revealed that both the curing time and matrix type have significant effects on the compressive strength of concrete. The interaction between these two factors was also found to be strong.

3.5.7 Bond to Old Concrete

The bond strength test results for all mix proportions of Table 3.7 are presented in Table 3.17.

In order to investigate the relationships between bond test results versus the percentages of steel fibers and latex polymers incorporated into the LMSFRC matrix, polynomial regression analyses along with one-way analyses of variances and a factorial analysis of variance have been performed. The dependent variable in this study is the bond strength between the old and the overlay concrete materials, whereas the independent variable could be either the fiber volume fraction (V_f) or the percent latex-cement ratio (L/c).
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V _f (%)	L/c (%)	Bond Strength (ksi)	
0	0	2.330	
0	0	1.887	
0	0	3.434	
0	5	2.984	
0	5	2.431	
0	5	2.728	
0	10	1.863	
0	10	3.693	
0	10	1.384	
0.5	0	1.822	
0.5	0	2.002	
0.5	0	2.669	
0.5	5	2.525	
0.5	5	2.900	
0.5	5	3.169	
0.5	10	3.860	
0.5	10	3.478	
0.5	10	3.968	
1	0	3.047	
1	0	2.311	
1	0	1.591	
1	5	3.199	
1	5	3.088	
1	5	3.144	
1	10	2.001	
1	10	2.566	
1	10	2.829	

Table 3.17Bond Test Results for Mixes of Table 3.8.

Figure 3.33 shows the relationship between the bond strength and the L/c for all fiber contents. The regression line and the 90% confidence interval are also shown. The bond strength seems to increase with increasing values of L/c. A factorial analysis of variance confirmed that, at a 88% confidence level, the mean bond strengths at different latex contents are different and thus there is an effect of . .

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latex modification on bond. The 88% confidence level indicates that, considering the variations in test results, there is a 12% chance of error in the statement that bond strengths at different latex contents are different.

In order to investigate the effects of V_f on bond strength, a similar linear regression analysis of test results was conducted (see Figure 3.34). This linear model shows only a slight increase in bond strength as a result of fiber addition. A comparison between Figures 3.33 and 3.34 shows that the regression line in Figure 3.33 has a higher slope than in Figure 3.34 and therefore the effect of latex content on bond strength seems to be larger than that of steel fibers. A factorial analysis of variance of test results indicated that there is a 36% chance of committing an error while making the statement that bond strengths at different fiber contents are different. This means that fiber content has a smaller effect than latex content on bond strength is concerned, a factorial analysis of variance of test results indicated that there is a variance of test results indicated that there is a smaller effect than latex content on bond strength is concerned, a factorial analysis of variance of test results indicated that there is a variance of test results indicated that there is a variance of test results indicated that there is a smaller effect than latex content on bond strength is concerned, a factorial analysis of variance of test results indicated that there is actually a relatively strong interaction between the steel fiber and latex effects on bond strength (with only 8% possibility that this statement is wrong).



Figure 3.33 Regression Line for Bond Strength Vs. L/c.



Figure 3.34 Regression Line for Bond Strength Vs. V_f.

In order to investigate the effects of latex alone at constant fiber contents (V_f) , second-degree polynomial regression models were constructed for bond strength-latex content relationships at constant V_f . In addition, a one-way analysis of variance was also performed. Figure 3.35a shows the relationship between bond strength and L/cfor plain concrete ($V_f = 0\%$). A high variation in test results can be seen for L/c = 10%. The one-way analysis of variance corresponding to this figure showed that the effects of different latex contents are not significant (there is an 85% chance of error in stating that there is some latex effect on bond strength at $V_f=0\%$). It should be noted that the slant shear bond test performed in this investigation is being seriously evaluated for revision by ASTM because it produces relatively large variations in results. Ohama^{3.12} has shown that while other test procedures indicate major improvements in bond strength with latex content, this may not be observed, especially at higher latex contents, with the slant shear test (possibly due to large variations in the results of this specific test). Repeating the same analysis with the constant fiber content of 0.5% (instead of 0%) gives the second-degree regression model shown in Figure 3.35b. A significant increase in bond strength values can be seen from this figure as L/c increases from 0 to 10%. Further investigation by one-way analysis of variance for this case shows that the effects of latex content on bond strength at a fiber volume fraction of 0.5% are significant (with only 0.4% chance of error in this statement). The results obtained at a constant fiber content of 1% (Figure 3.35c) show that the effects of latex content are less than those observed at 0.5% fiber volume fraction. The results of one-way analysis of variance also confirm this observation.



(a) $V_f = 0\%$

(b) $V_f = 0.5\%$



(c) $V_f = 1\%$

Figure 3.35 Regression Curves for Bond Strength Vs. L/c at Constant V_f .

Similar statistical analyses have been performed to investigate the fiber volume fraction effects at constant latex polymer content (L/c) on bond strength. For the case of non-modified steel fiber reinforced concrete (L/c=0%), Figure 3.36a shows no significant effect of fiber content on bond strength in non-modified concrete. The one-way analysis of variance showed that there exists an 80% chance of making an error if we state that there is some fiber content effects on the bond strength of non-modified concrete. At a constant L/c of 5%, Figure 3.36b shows an increase in bond strength with increasing fiber volume fraction. The corresponding analysis of variance showed that there is only 18% chance that this statement is wrong. In the last case for a constant L/c of 10%, the bond strength seems to reach a peak at fiber volume fraction of 0.5% and then decreases for higher V_r as seen in Figure 3.36c. The corresponding analysis of variance results showed that there is a fiber content effect on bond strength at L/c of 10% with only 11% chance of error in this statement.



(a) L/c = 0%

(b) L/c = 5%



(c) L/c = 10%

Figure 3.36 Regression Curves for Bond Strength Vs. V_f at Constant L/c.

Failure of the bonded concrete assembly may occur in the repair (overlay) concrete, at the bond line, or in the base (substrate) concrete. Figure 3.37a shows a typical failure along the bond surface especially for the case of non-modified overlay concrete, thus indicating a poor bond. When the overlay material is modified with latex polymer, however, the failed specimens illustrate the superior bonding enhanced by latex addition. Figure 3.37b shows a complete failure in the base material while the bond surface is seen to be fully bonded together. Figure 3.37c further illustrates the superior bonding effects brought by latex modification because failure occurred mostly in the base concrete that appears sheared off.

The large variation observed in bond test results urges the experimenter to be more cautious when discussing the results. The slant shear test, in fact, does not fully represent the "actual" stress conditions. Given this precaution, one may conclude from the bond test results (and their statistical analysis) presented in this section that, in general, the effect of latex modification on the bond strength between the base and the overlay concretes is larger than that of steel fiber addition. In addition, there is a noticeable interaction between latex and steel fiber effects on bond strength. From the obtained test results it can also be seen that at $V_f=0.5\%$, the increase in L/c up to 10% consistently increases the bond strength. However, at $V_f=1\%$, there is an increase and then a decrease in bond strength with increases in L/c. The highest bond strength was achieved at $V_f=0.5\%$ and L/c=10%.

These results can possibly be justified by the fact that the presence of fibers in the overlay concrete may result in increasing mechanical bonding by the interlocking of fibers at the interface between the new and old concretes. In addition, increasing

the fiber volume fraction in the matrix increases the surface area of the fibers thus consuming increasingly larger amounts of latex polymers at the fiber surfaces and hence leaving reduced amounts of latex in the matrix for enhancing the bond between the base and overlay concretes.



(a) Failure Along Bond Surface



- (b) Failure in Base Concrete
- Figure 3.37 Failure Modes of the Bond Surface Between the Base and Overlay Concretes.

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- (c) Failure in Base Concrete and Along Bond Surface
- Figure 3.37 (cont'd) Failure Modes of the Bond Surface Between the Base and Overlay Concretes.

3.5.8 Freeze-Thaw Durability

The results of the freeze-thaw durability test following ASTM C-666 (Procedure A) are presented in Figures 3.38a, b, c and d for plain, steel fiber reinforced (SFR), latex-modified (LM), and latex-modified steel fiber reinforced (LMSFR) concretes, respectively.



(a) Plain Concrete



(b) SFRC

Figure 3.38 Relative Frequency Versus Number of Freeze-Thaw Cycles.







(d) LMSFRC

Figure 3.38 (cont'd) Relative Frequency Versus Number of Freeze-Thaw Cycles.

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Figure 3.39 compares the average relative dynamic moduli of different concretes as functions of the number of freeze-thaw cycles. All the concrete mixtures considered in this investigation are observed in Figure 3.39 to perform quite desirably under the action of freeze-thaw cycles. It is worth mentioning that the concrete mixtures incorporating latex reached excellent levels of freeze-thaw durability with no need to additional air entrainment.



Figure 3.39 Comparison Between the Freeze-Thaw Durability of Different Concrete Mixtures.

A two-way factorial analysis of variance (ANOVA) was performed in order to statistically investigate the effects of latex modification and steel fiber reinforcement on the freeze-thaw durability of concrete. The results of this analysis revealed that both latex modification and steel fiber addition have significant effects on the

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freeze-thaw durability of concrete with about 12 and 10 percent chances of error, respectively, in this statement. The interaction between the two, however, was not significant as far as the freeze-thaw durability of the material is concerned.

3.5.9 Scaling Resistance

The test procedure of ASTM C-672, "Scaling Resistance of Concrete Surface Exposed to Deicing Chemicals," was used in this investigation. This test provides a measure for concrete resistance to freeze-thaw cycling in the presence of deicer salts. The results of visual observation in this test are summarized below in Table 3.18. Each result is based on tests on two specimens made from the same mix.

Table 3.18Scaling Resistance test Results.

	Number of Cycles	Rating
Plain Concrete	50	1
Latex-Modified Concrete	50	0
Steel Fiber Reinforced Concrete	50	2
Latex-Modified Steel Fiber Reinforced Concrete	50	1

where:

0 = no scaling;

1 = very slight scaling; and

2 = slight to moderate scaling

Latex modification is observed to improve the scaling resistance of both plain and steel fiber reinforced concrete; the rating of scaling resistance in each

boar plain and such hoer remining concrete, the facing of scaling resistance in

case was upgraded one level in the presence of latex.

3.5.10 Water Absorption and Specific Gravity

The water absorption and specific gravity test results are summarized in Table 3.19. The water absorption test results presented in Figure 3.40 indicate that the addition of latex to either plain or fiber reinforced concrete significantly decreases the water absorption capacity of the material. On the average, for plain mixtures, the addition of latex at 5% and 10% latex-cement ratios led to 55.2% and 76.0% reduction in water absorption, respectively. These results may be attributed to the pore filling characteristics of latex polymers when incorporated into concrete matrices. The polymers also bridge the microcracks propagating inside the matrix and at the aggregate-matrix interfaces thus closing all the channels and paths through which water may permeate. The addition of steel fibers to unmodified and latex-modified mixtures, however, had no significant effects on the water absorption capacity of concrete. A factorial analysis of variance of the results at 95% level of confidence confirmed the significance of latex modification effects on water absorption capacity, and also showed no significant effect of steel fiber reinforcement and no interaction between steel fibers and latex polymers.

The major conclusions that can be obtained from the bulk specific gravity test results (Figure 3.41) are that latex modification, steel fiber reinforcement, and their interaction have no significant effects on the specific gravity of concrete materials. This was confirmed at 95% level of confidence through factorial analysis of variance of the specific gravity test data.

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V _f (%)	L/c (%)	Water Absorption (%)	Specific Gravity
0	0	6.2	2.25
0	5	2.78	2.26
0	10	1.49	2.22
0.5	0	6.73	2.18
0.5	5	3.14	2.17
0.5	10	2.04	2.50
1	0	5.51	2.35
1	5	4.67	2.29
1	10	1.33	2.35

Table 3.19 Water Absorption and Specific Gravity Test Results.



Figure 3.40 Water Absorption Test Results.

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Figure 3.41 Specific Gravity Test Results.

3.5.11 Drying Shrinkage

The average drying shrinkage test results are shown in Figure 3.42. The drying shrinkage test results for individual mixtures are shown in Figure 3.43 (95% confidence intervals are also shown in this figure). These figures indicate that both steel fiber reinforcement and latex modification tend to reduce the average values of drying shrinkage in concrete.

After 25 days of drying, on the average, steel fiber reinforcement reduced drying shrinkage by 47%, latex modification by 29%, and steel fiber reinforcement with latex modification by 47%.

Analyses of variance performed on these data at 95% level of confidence revealed that both fiber reinforcement and latex modification have statistically significant effects on the drying shrinkage of concrete. These results also indicated the existence of an interaction between latex modification and steel fiber reinforcement effects on concrete drying shrinkage. The presence of fibers seems to reduce the potentials of latex polymers for reducing shrinkage movements. The ultimate shrinkage movements (after 25 days of drying) were statistically comparable, at 95% level of confidence, in steel fiber reinforced concrete with and without polymers.

The improvements in drying shrinkage resulting from the addition of latex polymers may be regarded as resulting from a combination of two phenomena. The first is a water-reducing effect which consists of the fact that latex modification results in concretes with lower water-cement ratios than conventional mixes. This way there is less water to be lost from capillary pores as well as less loss of adsorbed water from the surfaces of C-S-H particles (which constitute most of cement hydration products). The second effect results from the fact that latex particles form a thin polymer film that reduces the rate and extent of moisture movements; hence, retaining the water inside and reducing the extent of drying which is responsible for shrinkage movements.

Steel fiber reinforcement, on the other hand, helps in two ways. First, the fibers induce a restraining effect against shrinkage; and second, they provide additional traction due to fiber debonding, pull-out and stretching. The increased water requirements for achieving desirable workability in the presence of steel fibers, however, provide tendencies toward increased shrinkage movements in steel fiber

reinforced concrete.

The test results generated in this investigation compare favorably with those reported by Ohama (1982),⁶¹ except for the combined effect of latex modification and steel fiber reinforcement. Our results indicate no significant effects of latex modification on free drying shrinkage strains of steel fiber reinforced concrete, while Ohama (1982)⁶¹ found further reduction in drying shrinkage as a result of polymer modification of steel fiber reinforced concrete.

It is worth mentioning that this is a free drying shrinkage test while steel fibers and possibly latex polymers are expected to become really effective in the presence of a restraint where the tensile strength, post-cracking resistance and strain capacity all play important roles.



Figure 3.42 Drying Shrinkage Test Results.





Figure 3.43 Drying Shrinkage Test Results for Individual Mixtures.

3.6 SUMMARY AND CONCLUSIONS

The test results generated in this study indicated that:

1- Latex modification tends to increase both the initial and final times of setting in concrete mixtures;

2- The addition of steel fibers tends to decrease the setting time, irrespective of the latex content in the matrix.

3- The loss of workability with time takes place at almost the same rate for all mixes irrespective of the latex and steel fiber contents.

4- Latex modification and steel fiber reinforcement increases the impact resistance of the concrete matrix, with superior impact strengths obtained when steel fibers and latex polymers are used simultaneously, indicating an effective interaction between latex and steel fibers resulting from the improved bonding between the fibers and the latex-modified mixtures.

5- The separate actions of latex polymers and steel fibers in concrete lead to improved flexural strength of concrete. The combined action of latex modification and steel fiber reinforcement leads to highest flexural strength values.

6- Steel fiber reinforcement is effective in increasing the flexural toughness (area underneath the flexural load-deflection curve) of concrete. While latex modification has relatively small effects on the flexural toughness of plain concrete, it is capable of significantly improving the toughness characteristics of SFRC. These improvements can be attributed to the positive effects of latex modification on the fiber-to-matrix interfacial bond characteristics and pull-out behavior.

7- Wet curing at early ages is detrimental to mixtures containing latex polymers, while it is advantageous in mixtures without latex;

8- Exposure to hot water (compared to water at ambient temperature) after a period of air drying (26 days) reduces the flexural strength of LMSFRC;

9- The effects of latex modification on SFRC mixtures are very significant (in a negative manner) when curing is achieved by continuous immersion in water, or (in a positive manner) when curing involves air drying after a short period of moist curing. 10- Curing time, matrix type and their interaction have significant effects on the compressive strength of concrete. Latex-modified plain and steel fiber reinforced concretes tend to strengthen at later ages when exposed to air. This can be attributed to the desirable internal moist curing conditions in latex-modified mixtures provided by the polymer film formed at the surfaces and inside the concrete.

11- From the statistical analysis of the bond test results, we conclude that:

• relatively large variations exist in the slant shear test results for the assessment of bond between the new (overlay) and old (base) concretes;

• the effect of latex modification on the bond strength between overlay and base concretes tends to be larger than that of steel fiber addition;

• there is a noticeable interaction between latex modification and steel fiber addition on bond strength;

• at $V_f = 0.5\%$, the increase in latex-cement ratio up to 10% consistently increases the bond strength, while for $V_f = 1\%$, there is an increase and then a decrease in bond strength with increasing latex content. The highest bond strength was achieved at $V_f = 0.5\%$ and L/c = 10%. • for non-modified overlay concrete, failure typically occurs along the bond surface. When the overlay is modified with latex polymers, the bond improves significantly and failure generally occurs in the base material.

• fiber effects on bond may result from the mechanical bonding resulting from the interlocking of fibers at the interface. The interaction between latex and fiber effects on bond could result from the consumption of latex at the relatively large surface area of fibers, which leaves less latex available to act at the interface.

12- Statistical analyses of the test results generated in the freeze-thaw study indicated that:

• latex modification imparts improvements in the resistance to deterioration from repeated freezing-and-thawing cycles with no need to additional air entrainment;

• steel fiber addition also somewhat improves the freeze-thaw durability characteristics of air-entrained concrete matrices;

• there is no noticeable interaction between latex modification and steel fiber addition as far as the resistance to repeated freeze-thaw cycles is concerned;

13- Latex modification is observed to improve the scaling resistance of both plain and steel fiber reinforced concretes.

14- While steel fiber addition had no significant effect on the water absorption of the concrete, latex modification, due to the pore filling characteristics of latex polymers when incorporated into concrete matrices, is found to significantly decrease the water absorption capacity of both plain and fiber reinforced concretes.

15- Latex modification, steel fiber reinforcement, and their interaction had no significant effects on concrete's bulk specific gravity.

16- Both latex polymers and steel fibers have significant effects on the drying shrinkage of the resulting concrete. Analyses also revealed the existence of an interaction between the two. The presence of fibers reduces the potentials of latex polymers for reducing shrinkage movements.

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CHAPTER FOUR

MICROCRACKING CHARACTERISTICS UNDER COMPRESSION LOADING

4.1 INTRODUCTION

Concrete materials suffer from microcracking at cement-aggregate interfaces, along fiber-matrix interfaces, or around the entrapped air voids, even prior to loading. Under increased loading microcracks tend to grow and interconnect to form microcrack channels or get arrested by matrix constituent(s) such as aggregates, steel fibers, and air voids.

Latex polymers in plain and steel fiber reinforced concretes reduce the microcracking damage under external loading due to their pore filling and superior interface zone bonding.

The study reported herein is concerned with the assessment of the improvements in the matrix microstructure associated with latex modification of plain and steel fiber reinforced concrete materials. In particular, this study investigates the effects of latex modification of concrete matrix on the microcracking and failure mechanisms.

4.2 BACKGROUND

In fresh concrete, water films form around large aggregate particles once compacted. This results in a local increase in water-cement ratio around coarse aggregate particles. On the other hand, the dissolution of calcium sulfate and calcium aluminate compounds forms calcium, sulfate, hydroxyl, and aluminate compounds that combine to form ettringite and calcium hydroxide. These are crystalline products that would consist of relatively large crystals around coarse aggregates (due to the locally higher water-cement ratio) result in a more porous structure than in the bulk cement paste or mortar matrix. As hydration progresses, some poorly crystalline C-S-H and smaller crystals of ettringite and calcium hydroxide start filling the existing empty spaces between the framework created by the large ettringite and calcium hydroxide crystals. This improves the density and strength of the transition zone. The adhesion between the hydration products and the aggregate particles is by van der Waals attraction forces which depend on the volume and size of voids.⁵⁰ Since even at low water-cement ratio, the volume and size of voids at transition zone are larger than in the bulk mortar, it follows that the transition zone is weaker in strength. As curing age increases, new products may crystallize in the voids of the transition zone through chemical reactions between cement paste and aggregates. This would reduce the concentration of the calcium hydroxide in the transition zone and improves its strength.

Besides the large volume of capillary voids and oriented calcium hydroxide crystals, the presence of microcracks contributes to the weakness of the transition

zone. Many parameters affect this interface zone, namely, cement content, watercement ratio, degree of consolidation of fresh concrete, curing conditions, humidity and temperature of concrete, and aggregate size and grading. In fact, poor gradation promotes segregation during compaction. This results in formation of thick water films around coarse aggregates and interface zones making them susceptible to cracking under tensile stresses resulting from differential movements between aggregates and hydrated cement paste during drying of concrete before loading.⁵⁰

Under external loading, microcracks occur (or start to grow) at the cement-aggregate interfaces, around the entrapped air voids, or along the fiber-matrix interfaces. With increased loading, some microcracks grow and connect with each other to form large macrocracks while the propagation of some others is prevented by aggregates, air voids or fibers. The propagation of microcracks leads to increased nonlinearity of the material before the peak load is reached.

For air-dried mortar and concrete, shrinkage-induced bond cracks around large aggregate particles appear prior to any loading (Figure 4.1).⁷⁷ Under load, debonding and multiple cracking around sand grains and air voids are observed frequently (Figure 4.2), and this phenomenon seems to be more pronounced between adjacent sand grains than around isolated ones.⁷⁷ In normal weight concrete, the crack changes orientation when it encounters an aggregate by passing around it instead of crossing it (Figure 4.3). In concrete, the crack pattern is more tortuous than in mortar (Figure 4.4) because concrete cracks must propagate around the densely spaced aggregate pieces as well as sand grains. The reorientation, branching, and multiple cracking associated with the interaction of microcracks with the encountered

aggregate particles lead to the dissipation of a large amount of energy which is beneficial to the material's behavior under load.



Figure 4.1 Bond Crack Prior to Loading in Air-Dried Mortar.⁷⁷



(a) Around Sand Grain

(b) Around Air Void

Figure 4.2 Debonding and Multiple Cracking in Air-Dried Mortar.⁷⁷




Figure 4.3 Shifting of Crack Direction After Encountering Aggregates.⁷⁷



Figure 4.4 Tortuosity of Cracking.^{π}

When fibers are present in the matrix, the growth and interconnection of microcracks become a more energy absorptive process, and thus fibers enhance the pre-peak behavior and the ultimate tensile strength of fiber reinforced concrete (FRC).

Once a microcrack intersects a fiber at an angle, it generally interacts with the fiber in a manner that makes further propagation more energy absorptive (see Figure 4.5). The microcrack can advance beyond a steel fiber either along its original path as shown in Figure 4.6a (20% of the time), by shifting as shown in Figure 4.6b (30% chance), or by branching into multiple post-fiber cracks as shown in Figure 4.6c (50% of time). The microcrack encountering a fiber stays continuous, making the lateral shifts around the fibers, as can be clearly seen in the picture of the groove under a steel fiber that intersected microcracks in Figure 4.7.

Microcrack propagation at the fiber-matrix interface might take place at the interface itself leading to the separation of the matrix from the fiber by debonding (Figure 4.8a), or it might occur at a small distance (~ 20 microns) from the fiber and parallel to it by pseudo-debonding (Figure 4.8b) which separates the body of the matrix from a thin layer of interface that remains attached to the fiber.

With their microcrack-arresting action, fibers tend to increase the fracture energy and consequently the tensile strength of concrete. Fiber pull-out or rupture tends to dominate the post cracking progress of failure in steel fiber reinforced concrete (SFRC) under direct tension.



Figure 4.5 Cracking Patterns Observed at the Intersection of a Propagating Crack and a Fiber Normal to its Path.⁷⁷



Figure 4.6 Illustrations of the Crack Types (a) Parallel Running, (b) Shifting, and (c) Branching.⁷⁷



Figure 4.7 Continuous Nature of Shifted Microcracks Around a Steel Fiber.



(a) Debonding

(b) Pseudo-Debonding

Figure 4.8 Microcrack Propagation at the Steel Fiber-Cement Interface.⁷⁷

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4.3 EXPERIMENTAL PROGRAM

For each one of the mixes of Table 3.7, 12 specimens were cast. The specimens were 75 mm (3 in.) in diameter and 150 mm (6 in.) long. They were moist-cured for 48 hours inside their molds while being covered with a wet burlap and a plastic sheet, and then air-cured until the test age of 28 days.

Three specimens out of each group of fifteen were tested under compression until failure, with stresses and strains monitored throughout the test. Based on the average compressive strength (f_c) obtained from these three tests for each mix, five stress levels were selected and two specimens of the same mix were loaded to each predetermined stress level and then unloaded. The five stress levels considered in this study were:

0.00 f'_c;
0.30 f'_c (pre-peak);
0.80 f'_c (pre-peak);
1.00 f'_c (peak); and
0.90 f'_c (post-peak).

After each specimen was covered with a thin layer of epoxy, the specimen was encased (i.e., circumscribed) in fibrous mortar mount a 100 mm (4 in.) in diameter. This mount is necessary to maintain the integrity of specimens (especially those loaded to large strains) during handling and slicing for microstructural investigations. The specimens were then sectioned, one transversely and one longitudinally, to 13 mm (0.5 in.) thick slices that were cut from the center of the specimen using a diamond saw (see Figure 4.9).



(a) Longitudinal Slice(b) Transverse SliceFigure 4.9 Locations of Transverse and Longitudinal Slices.

The slices were then washed in a jet of water and allowed to dry in the lab for 24 hours. They were then stained with black indian ink, ground with silicon carbide on rotating laps over a sequence of 5 grit sizes: #180, #240, #320, #400, and #600. These specimen preparation steps help distinguish the microcracks under microscope. Figure 4.10 shows a view of the longitudinal and transverse slices after being prepared for microstructural studies.

The slices were examined for microcracking characteristics using an image analysis system of typical magnification 25 x. The cracks at the prepared faces of the slices were visible as black lines. The microstructural studies were conducted after dividing the surface area of the slice to be viewed under the microscope into about thirty 13- by 13- mm (0.5- by 0.5- in.) squares, each to be viewed as a separate field of measurement (see Figure 4.10)





Figure 4.10 Longitudinal and Transverse Slices Ready to Be Viewed Under Microscope.

Once subdivided, the slices were viewed through a microscope connected to the image analysis system. For each field of view on the microscope (which covers a 4 mm (0.16 in) by 3 mm (0.12 in) area within each 13 mm (0.5 in) by 13 mm (0.5 in) square of the mesh), the following three measurements were performed:

1- The intensity of bond cracks at aggregate-cement and fiber-cement interfaces (see Figures 4.11a and b, respectively) with intensity defined as the total crack length per unit area ;

2- The intensity of matrix cracks (see Figure 4.12, noting that both aggregate-interface and matrix cracks are shown here); and

3- Microcrack orientations defined as the average inclination of cracks with

respect to the direction of loading (performed on the longitudinal slices only).

The measurements on microcrack intensity will reveal information on the process of failure in concrete materials under increasing stress levels, as influenced by the presence of steel fibers and latex polymers. The inclination of microcracks will provide indications of the nature of failure mechanism under compression.



(a) Aggregate-Cement Bond Cracks



(b) Fiber-Cement Bond Cracks



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Figure 4.12 Typical Matrix Microcracks.

4.4 EXPERIMENTAL RESULTS AND DISCUSSIONS

The results of microstructural studies on the process of microcrack propagation and failure under compressive stresses in plain, latex-modified, steel fiber reinforced, and latex-modified steel fiber reinforced concrete materials are discussed in this section. Conclusions regarding the latex modification and fiber reinforcement effects on the failure mechanism of concrete materials are also presented.

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The results of crack intensity (i.e.; crack length per unit area of cross-section) measurements at different compressive stress levels on plain concrete specimens are presented in Figures 4.13a and b for transverse and longitudinal sections, respectively. In our judgement, the crack intensity at the transverse section is more representative of the actual conditions than that obtained at the longitudinal section. This results from the fact that microcracks tend to propagate in vertical planes. Hence, while a transverse section cuts many of these cracks, the longitudinal section occurs along or in between the approximately vertical crack planes and thus does not present a typical crack intensity.

Microcracks are observed in Figure 4.13 to be present even prior to loading (at 0% stress level). These microcracks are caused by the differential shrinkage movements, settlements and thermal strains between aggregates and cement paste, and also by the bleeding effects. They appear dominantly at the coarse aggregate-cement paste interfaces (see Figure 4.13). Similar observations using identical techniques have been made by Shah.⁷⁸ The propagation of these microcracks under compression (which starts to take place mainly at the interfaces and then extends into the matrix as previously shown in Figure 4.12) is quantitatively demonstrated in Figure 4.13. At the peak compressive stress, microcracks show a tendency to interconnect and localize, leading to the formation of macrocracks with increasing widths. The process of microcrack propagation takes place at an increasing rate in the post-peak region where microcracks tend to be unstable. Similar observations using identical techniques have been made previously by S. P. Shah and R. Sankar.⁷⁸



(a) Transverse Section



(b) Longitudinal Section

Figure 4.13 Microcrack Intensities at Different Compressive Stress Levels in Plain Concrete for Transverse and Longitudinal Sections.

The microcrack intensity in latex-modified concrete under increasing compressive stresses is presented in Figure 4.14. A comparison between Figures 4.13a and 4.14a, for the more representative transverse sections, indicates that the microcrack intensities tend to be reduced at lower stress levels with latex modification. In particular, microcracks occurring at aggregate-paste interfaces tend to be reduced substantially in the presence of latex polymers, confirming previous findings³⁹ that latex modification provides improved bonding between the cementitious paste and aggregates. Reduced microcrack intensity in the presence of latex polymers may result from the restraint of microcrack propagation by polymer films bridging across these cracks. In the post-peak region, where the interconnection and localization of microcracks lead to the formation of macrocracks with increasing widths, the failure mechanism in latex-modified concrete was comparable to that in plain concrete.

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(a) Transverse Section



(b) Longitudinal Section

Figure 4.14 Microcrack Intensities at Different Compressive Stress Levels in Latex-Modified Concrete for Transverse and Longitudinal Sections.

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The microcrack intensities at different stress levels for steel fiber reinforced concrete are presented in Figure 4.15. The appearance of fiber-interface microcracks (see Figure 4.16), which could be initiated by the same phenomena causing aggregate-interface microcracks, is the new phenomenon which seems to play an unexpected dominant effect in deciding the failure mechanism of steel fiber reinforced concrete. The formation of fiber-interface microcracks and their relatively rapid propagation (see Figure 4.17) under increasing compressive stresses in the pre-peak region seem to reduce the effectiveness of steel fibers in arresting the propagation of microcracks. In the post-peak region, however, the crack system seems to be more stable in steel fiber reinforced concrete (Figure 4.15a) than in plain concrete (Figure 4.13a) and latex-modified concrete (Figure 4.14a).

Latex modification of steel fiber reinforced concrete, as shown in Figure 4.18, seems to stabilize the fiber-interface microcracks under increasing compressive stresses. The aggregate-interface microcracks were also partly controlled by the presence of latex polymers in steel fiber reinforced concrete. Figure 4.11 illustrates these points by showing better fiber- and aggregate-interface bonds even at high compressive stress levels.

The matrix microcrack orientations at the longitudinal sections were generally less than 20 degrees from the longitudinal axis of the specimen (i.e.; the loading direction). The microcrack orientation at the aggregate- and fiber-matrix interfaces seemed to be very random.

















TRANSVERSE CRACK INTENSITY (in/in2)



(a) Transverse Section



(b) Longitudinal Section

Figure 4.15 Microcrack Intensities at Different Compressive Stress Levels in Steel Fiber Reinforced Concrete for Transverse and Longitudinal Sections.



Figure 4.16 Typical Fiber-Interface Microcracks.



Figure 4.17 Microcracking Due to Rapid Fiber-Interface Microcrack Propagation.



(a) Transverse Section



(b) Longitudinal Section

Figure 4.18 Microcrack Intensities at Different Compressive Stress Levels in Latex-Modified Steel Fiber Reinforced Concrete for Transverse and Longitudinal Sections.

4.5 SUMMARY AND CONCLUSIONS

The effects of latex modification and/or steel fiber reinforcement on the microcracking process of failure mechanism in concrete materials subjected to compression loads were studied. For this purpose concrete specimens were subjected to different levels of compressive stress in the pre-peak and post-peak regions, and were then unloaded. Image analysis techniques were employed to quantify microcrack intensities within the paste and at the aggregate- and fiber-matrix interfaces at different load levels. The results indicated that:

(1) Microcracks are present in concrete materials, dominantly at the coarse aggregate-matrix and fiber-matrix interfaces, even prior to any loading. These microcracks may be caused by differential shrinkage and thermal movements between cementitious matrices and mix inclusions (i.e., aggregates and fibers);

(2) In plain concrete, microcracks tend to originate mainly at the coarse aggregate-cement paste interfaces, and then extend into the matrix at higher compression levels; near the peak load, microcracks tend to interconnect and localize, forming macrocracks with increasing widths. There is a rather sudden growth in microcrack intensity beyond the peak compression load;

(3) In latex-modified concrete, the microcrack intensities, especially at aggregate-paste interfaces, tend to be reduced, indicating improvements in bonding between the cementitious paste and aggregates as a result of latex modification. Beyond the peak load, however, the sudden increase in microcrack intensity still takes

place in spite of latex modification;

(4) In unmodified steel fiber reinforced concrete, the fiber-matrix interface microcracks seem to play a dominant role in deciding the failure mechanism of the concrete matrix due to their relatively rapid propagation under increasing compressive stress levels; steel fibers, however, control microcrack propagation in the post-peak region; and

(5) Latex modification of steel fiber reinforced concrete seems to stabilize the fiber-matrix interface microcracks as well as the aggregate-matrix interface microcracks leading to a more stable microcrack system within steel fiber reinforced concrete materials.







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DEVELOPMENT, CHARACTERIZATION, AND FUNDAMENTAL INVESTIGATION OF LATEX-MODIFIED STEEL FIBER REINFORCED CONCRETE

By

Atef Tlili

VOLUME II A DISSERTATION

Submitted to Michigan State University in partial fulfillment of the requirements for the degree of

DOCTOR OF PHILOSOPHY

Department of Civil & Environmental Engineering

CHAPTER FIVE

PERMEABILITY

5.1 INTRODUCTION

One of the major problems encountering highway agencies in the United States and in many other parts of the world is the premature deterioration of concrete bridge deck overlays. This deterioration worsens the ride quality of these decks and requires some major repairs causing inconvenience for the travelling public and consuming major fractions of the budget of highway agencies.^{66, 97} The premature deterioration of bridge decks is due mainly to the corrosion of the embedded reinforcing steel that results from the application of salts and deicing chemicals onto the concrete deck. Once corroded, the reinforcing steel expands and causes rupture inside concrete, eventually leading to the spalling of concrete surface. Concrete permeability has strong effects on the corrosion of the reinforcing steel. The water carrying chloride ions and dissolved oxygen penetrates into the concrete and causes the corrosion of the embedded steel. Permeability of concrete also strongly influences the durability of the material in other exposure conditions.

The main thrust of this study was to assess the effects of latex modification and/or steel fiber reinforcement on concrete permeability. The applicability of

different permeability test methods to steel fiber reinforced and latex-modified concretes was also investigated.

5.2 BACKGROUND

Permeability is defined as the ease with which a particular substance, be it liquid, gas, or ion, can flow through a porous solid. Consequently, the size and continuity of the pores in this porous solid structure are key factors in determining its permeability. Permeability of concrete is, in fact, very closely related to its capillary porosity. There actually exists an exponential relationship between the two.⁵⁰

It is worth noting that capillary pores represent the part of the gross volume that has not been filled by the hydration products. Eventually, as hydration proceeds, the void space originally existing between the discrete cement particles begins to gradually fill up with hydration products. Because these products occupy more than twice the volume of the original cement solid phase alone, the volume of the capillary system decreases as hydration progresses. Accordingly, one can affirm that watercement ratio, being a representing factor of the original capillary space between cement particles, and the degree of hydration both determine the capillary porosity of the paste.

In a fresh cement paste, the capillary porosity is high because of a relatively high water content and a low degree of hydration. The flow of water is, at this stage, controlled by the size, shape, and concentration of the original cement grains. As cement hydration process progresses, since the gross volume of gel (including gel pores) is about twice the volume of the unhydrated cement, the gel gradually fills part of the originally water-filled space. A small decrease in capillary porosity is associated with considerable segmentation of large pores, thus reducing the size and number of flow channels in the cement paste.^{50, 54} At low capillary porosity (about 30%) the interconnections between the pores, which are reduced to small sizes (~100 nm or less), become very tortuous resulting in very low permeability.

In a mature and dense cement paste, the capillaries become blocked by gel and segmented so that they turn into capillary pores interconnected solely by gel pores. These are interconnected interstitial spaces between the gel particles. They are much smaller than the capillary pores (15 to 20 Å), and occupy about 28% of the total gel/paste volume. Capillary pores, on the other hand, occupy between 0 and 40% depending on the water-cement ratio and the degree of hydration. The permeability of cement is therefore seen to be not a function of its porosity only, but also to depend on the size, distribution, and continuity of the pores.

Aggregates, on the contrary, typically have less than 3% pore volume (compared to 30 to 40% capillary porosity of a typical cement paste). This would imply that aggregates are expected to have much lower permeability than the typical cement paste. Various investigations, however, revealed that the permeability of aggregates is very variable and that some aggregates may have higher permeability than cement paste which has a higher porosity. This has been attributed to the fact that the size of capillary pores in aggregates is usually much larger (> 10 μ m) than those of cement paste (10-100 nm).

Introducing aggregate particles of low permeability into a cement paste should,

theoretically speaking, reduce the permeability of the resulting concrete system. In fact, with a rational reasoning, aggregate particles would intercept the flow channels inside the cement paste, and may reduce the effective area over which flow can take place.^{50, 54} Experimental results indicate that this is not true. Incorporating aggregates into cement paste considerably increases the permeability of the resulting mortar or concrete (the larger the aggregate size, the larger the permeability). This unexpected finding is attributed to the microcracks present in the aggregate-cement paste interface zones. Water films form around large aggregate particles in fresh concrete. Calcium, sulfate, hydroxyl, and aluminate ions, produced by the dissolution of calcium sulfate and calcium aluminate compounds, combine to form ettringite and calcium hydroxide. As a result of the local high water-cement ratio in this region, the crystalline hydration products (especially calcium hydroxide and ettringite) at the interface consist of relatively larger crystals that form a more porous framework than in the cement paste or mortar matrix. The calcium hydroxide crystals are plate-like and tend to form in oriented layers. As hydration process progresses, poorly crystalline calcium silicate hydrate (C-S-H) and a second generation of smaller crystals of ettringite and calcium hydroxide start filling empty spaces existing in the porous framework (See Figure 5.1).⁵⁰ The transition zone is usually the weakest link in the concrete structure and has microcracks even without any applied loading. To recapitulate, the larger the aggregate sizes the higher the local water-cement ratio in the transition zone will be and, consequently, the weaker and more permeable the concrete will be. In other words, the porous and debonded transition zones provide for interconnection of capillary pores and thus increased permeability of concrete.





5.2.1 Factors Affecting Concrete Permeability

There are several factors that affect the permeability of concrete. They

typically fall into one of the following three main groups:

- 1. The influence of the constituent materials;
- 2. The effect of methods of preparing the concrete; and
- 3. The influence of subsequent treatment of the concrete.

5.2.1.1. Effect of Water and Cement: The permeability of concrete is very

dependent on the water/cement ratio because the latter determines the size, volume,

and continuity of capillary voids. For plastic workable mixes, the permeability

increases with the water-cement ratio as shown in Figure 5.2.50, 90, 100 Dry mixes do

not consolidate very readily; therefore, more water is required for minimum
permeability than for maximum strength. For hand-rodded concrete, the permeability increases when the water is reduced below the level which will produce a slump of about 51 to 76 mm (2 to 3 in).

Permeability decreases as the cement-voids ratio increases and this relationship appears to be more definite than that between permeability and water-cement ratio.⁹⁰ With well-cured concrete and the optimum quantity of mixing water, an increase of cement content above that in a 1:2:4 (Cement:Fine Aggregate:Coarse Aggregate) mix does not materially affect permeability. However, very wet consistencies require a richer mix and tend to produce water gain under the aggregate particles, which increases the permeability.

5.2.1.2. Effect of Aggregates: As seen in the previous Figure 5.2, the greater the maximum size of aggregate for a given water-cement ratio, the greater the flow (i.e., the coefficient of permeability); this could be due to the relatively large water voids developed on the underside of the coarser aggregate particles.

Aggregates should be sound and of low porosity. It is very important to have a well graded aggregate from the standpoint of water tightness than it is from the standpoint of strength, due to the importance of dense concrete. Sufficient fines must be used, but the mix should not be oversanded.⁹⁰

Typically, permeability coefficients for medium-strength concrete (containing 38 mm (1.5 in) aggregate, 356 kg/m³ (600 lb/yd³) cement, and an 0.5 w/c), and lowstrength concrete used in dams (76 to 152 mm (3 to 6 in) aggregate, 148 kg/m³ (250 lb/yd³) cement, and an 0.75 w/c) are of the order of $1_{\times}10^{-10}$ and $30_{\times}10^{-10}$ cm/sec (0.4.10⁻¹⁰ and $11.8_{\times}10^{-10}$ in/sec), respectively.⁵⁰



Figure 5.2 Influence of Water/Cement Ratio and Maximum Aggregate Size on Concrete Permeability. ^{50, 90, 100}

the cement results in gel development which reduces the size of the voids and increases the watertightness of the concrete. Figure 5.3^{90} shows the great increase in watertightness with curing, the change being even greater than the increase in strength with curing.

5.2.1.3. Effects of Curing Period and Temperature: Continued hydration of

The pore size distribution is strongly influenced by the curing temperature: high temperatures increase the volume of large mesopores (see Figure 5.4^{100}). This change in pore size distribution is reflected in higher permeabilities when curing temperatures are varied.



Figure 5.3 Effect of Curing Period on Permeability.⁹⁰



Figure 5.4 Effect of Curing Temperature on Pore Size Distribution.¹⁰⁰

5.2.1.4. Effects of Mineral Admixtures: Reductions in permeability are an important aspect of the beneficial effects of mineral admixtures, such as fly ashes and slags. Figure 5.5^{100} show that for cement pastes containing reactive mineral admixtures, such as fly ash and silica fume, the total porosity is reduced compared to

pastes of pure cement and a finer pore structure results. Pastes containing fly ash, however, have reduced permeability even though the much slower reactions of fly ash result in higher porosities and coarser pore structures even after 60 days of curing (see Figure 5.5^{100}).

The pozzolanic reaction seems to be able to develop a discontinuous pore system more readily. It has been found that pastes containing fly ash are more prone to damage when MIP (Mercury Intrusion Porosity) is used to measure pore sizes. It appears that mercury under pressure breaks through rather fragile barriers that isolate large pores, whereas this does not happen in pure cement pastes. This suggests that lack of adequate curing of concretes with fly ash could seriously affect its potential permeability. However, increased curing temperature greatly increases the rate of pozzolanic reaction and reduces water flow (Figure 5.4).

The pronounced reduction in pore size in pastes containing silica fume is attributed both to its high pozzolanic reactivity, and also to its very small particle size which allows it to pack efficiently between the cement grains, thereby subdividing the space. The permeability coefficients are thus greatly reduced, particularly at high w/c ratios.



Figure 5.5 Pore Size Distributions in Cement Pastes Made With and Without Mineral Admixtures (numbers beside legends represent relative permeability: control=1.0).¹⁰⁰

5.2.1.5. Effects of Chemical Admixtures: Chemical admixtures should also affect permeability if they change pore size distributions in cement pastes. Additions of calcium chloride increase the volume of fine capillary pores at the expense of large ones and provide a less permeable paste. Additions of latex polymers reduce the permeability significantly.⁴³ This is due to the fact that the structure of latex-modified concrete/mortar is such that the micropores and voids normally occurring in portland cement systems are filled with the polymer film that forms during curing, thus reducing permeability and water absorption.

Water-reducing agents should reduce permeability if they are used to lower w/c ratios. However, like calcium chloride, water reducing agents often increase drying shrinkage, which has been correlated with finer pores. Thus, they may well

reduce permeability even when the w/c is not changed. A more uniform dispersion of cement grains within reducing agent is more likely to provide a more uniform pore structure with less coarse pores.¹⁰⁰

5.2.1.6. Mortars and Concretes: It has been found that the cement-aggregate interface is more porous than the bulk paste, in all but very well-cured systems.⁶⁸ This has been demonstrated by SEM observations, but is not so easy to quantify. Only a few comparative MIP measurements on paste and mortars have been made (see Figure 5.6).¹⁰⁰ It appears likely that the interfacial zone will be a favorable pathway for water flow. This is even more likely if bond cracking occurs under local stresses caused by thermal mismatch between paste and aggregate, or restraint of paste shrinkage by the aggregate.

The permeability coefficients for concrete are generally about 100 times higher than those for comparable pastes while those for mortars are 3-10 times higher.¹⁰⁰



Figure 5.6 Pore Size Distribution For Plain Paste and Paste in Mortar.¹⁰⁰

5.2.1.7. Effect of Latex Modification: Chloride permeability is considered to be one of the most critical aspects of Latex-Modified Concrete (LMC) performance. Modifying plain concrete with latex polymers significantly enhances its impermeability characteristics.

When concrete is modified with latex, the micropores and voids get filled with a thin polymer film that acts as a barrier against permeability and water absorption.^{8,80} Besides porosity, another factor controlling the permeability of concrete is the microcrack system initiated inside concrete, especially at matrix-aggregate interfaces, due partly to restrained shrinkage movements. Latex modification reduces the amount of microcracking in concrete. Latex particles tend to form a film, during cement hydration, that surrounds the aggregates and provides a strong and flexible bonding between the cementitious matrix and aggregates.^{45,80} This improved interface condition leads to reduced microcracking at the interface, thus making the concrete more impermeable.

Kuhlmann^{8, 43} has reported that the permeability of LMC is strongly influenced by time and curing conditions. For instance, permeability decreases significantly when LMC is cured beyond the normal 28-day curing period, with largest decrease occurring within the first two months of curing (see Figure 5.7). Therefore, it is concluded that permeability is a function of time and that proper curing of concrete followed by an extended period of time prior to exposure to chloride solutions may result in a more impermeable material.

Ohama⁶⁶ has reported that the chloride ion penetration depth of LMC increases with additional immersion periods, and is reduced with an increase in polymer-cement

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ratio. Kuhlmann⁴³ also found that, even at high air contents, the air voids in LMC are very small and isolated, and therefore do not increase permeability.



Figure 5.7 Latex Solids/Cement Vs. Chloride Penetration.⁹⁷

5.2.1.8. Effect of Steel Fiber Reinforcement: Steel fiber reinforcement is not a particularly effective method of improving the impermeability of concrete;⁹ noting that some important gains in many other aspects of the material behavior can be achieved through fiber reinforcement. In actual field conditions where shrinkage movements are restrained, the effectiveness of fibers in controlling shrinkage cracks may help reduce the permeability of concrete in field conditions.

5.2.2 Methods of Permeability Testing

Various methods for assessment of concrete permeability are available.⁵²

These tests are commonly performed by measuring the permeation of liquids (hydraulic permeability), gases (air permeability), and/or ions (chloride ion permeability) into the cement/concrete matrix.

5.2.2.1. Hydraulic Permeability Measurement: Permeability to liquid water could be determined using several methods:

5.2.2.1.1 Initial Surface Absorption Test (ISAT, British Standard 1881): It basically measures the movement of water into concrete via capillary attraction. This can be remarkably rapid and can even approach fluid flow rates in saturated concrete brought about by applying hydraulic pressures close to 2.8 MPa (400 psi).

5.2.2.1.2 High Pressure Water Permeability Test: Developed at King Fahd University of Petroleum and Minerals (KFUPM), this test method has a few versions of it available. This test consists of applying, in a pressure vessel, a pressure of 6.90 MPa (1000 psi) to force water into a 70 mm diameter by 100 mm high (2.75 x 4 in) cylindrical concrete specimen.⁵

5.2.2.1.3 Von Test Method: This test method consists of measuring the amount of water that permeates through a 51 mm (2 in) thick layer of concrete in a 24-hour period.

5.2.2.2 Air Permeability Measurement: Permeability of concrete to air is of interest in structures such as pressure vessels in nuclear reactors, gas purifiers and tanks in which concrete is desired to be air-tight under some specified internal pressure. Flow of air through concrete is commonly measured with one of the following two methods:

5.2.2.1 EGG Method: The procedure for this test involves using the air pressure drop method.⁶

5.2.2.2 Standard API Method: This test procedure uses 100 x 200 mm (4 x 8 in) cylindrical concrete specimens which seal into a cell. Air is then introduced to the upstream force of the sample until a steady-state condition is achieved. At this stage, the upstream and downstream pressures are recorded. The flow rate is then determined by measuring the pressure drop across a calibrated orifice. Permeability, in darcys, is then calculated using the following equation:

$$K_{g} = \frac{2\mu Q_{g} P_{b} L}{A(P_{2}^{2} - P_{1}^{2})}$$

where: $K_g = gas$ permeability, in darcys $\mu = gas$ velocity, in Pa.s $Q_g = gas$ flow rate, in cm²/sec (in²/sec.) $P_b = barometric$ pressure, in Pa (psi) $P_1 = inflow$ pressure, in Pa (psi) $P_2 = outflow$ pressure, in Pa (psi) A = average cross-sectional area perpendicular to the line of flow, in cm² (in²) L=length of flow path, in cm (in)

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5.2.2.3 Chloride Ion Permeability Measurement: The two commonly used tests for measuring the chloride ion permeability into concrete are:

5.2.2.3.1 Rapid Chloride Permeability Test: This test was originally

developed by David Whiting in Construction Technology Laboratories (CTL).

In this test method, a potential of 60 dc Volts is applied across a 10.16 mm (4 in) in diameter by 5.08 mm (2 in) thick cylindrical concrete test specimen that has been conditioned by means of vacuum saturation. After 6 hours of testing, the total charge passed through the specimen (in Coulombs) is obtained by integration of the current passed through the specimen during the test period. This is a standard test method adopted by AASHTO T277-83 a few years ago, and recently adopted by ASTM as a standard test method (ASTM C-1202).

5.2.2.3.2 Chloride Ion Penetration By Ponding: This is an AASHTO T259-80 and T260-84 standard test method. This test method basically consists of subjecting the test slabs to continuous ponding with 3 percent sodium chloride solution to a depth of approximately 13 mm (0.5 in) for a period of 90 days. After this period of exposure, the solution is removed from the slab surfaces that are then allowed to dry. Samples are then taken from each slab at various depths for chloride ion analysis in accordance with the procedure described in AASHTO T260-84.

5.3 EXPERIMENTAL PROGRAM

5.3.1 Rapid Chloride Permeability Test

Rapid assessment of the chloride permeability of concrete was made in this investigation through the performance of the AASHTO T277-83I (also, ASTM C1202) test. This test is concerned with quick assessment of the degree of

permeability in concrete samples. The results of this test are obtained in no more than two days.

Twenty four specimens 10.16 mm (4 in) in diameter by 5.08 mm (2 in) in thickness were prepared for each mix. Twelve of these were stored in a moist curing room and the remaining twelve were cured in a laboratory environment. Eventually, three specimens from each curing condition were taken for permeability testing at each of the following ages: 28, 60, 90, and 120 days. Table 5.1 summarizes the experimental design adopted in this investigation. After conditioning the concrete specimens by vacuum saturation (Fig. 5.8a), they were bonded into the test cell with a silicone adhesive (Fig. 5.8b). The left side of cell was filled with a sodium chloride solution and the right side with an alkali solution. A voltage of 60 V dc is then applied in this test for 6 hours; the instrument automatically records and displays the electrical current passed through the specimen and integrates the current versus time to give the charge that passes (in Coulombs).

Table 5.1Experimental Design For Rapid Permeability Test For Each Curing
Age.

V _f (%)	L/c (%)				
	0		10		
	Moist	Air	Moist	Air	
0					
0.75					



(a) Vacuum Saturation Apparatus



(b) Applied Voltage Cell - Face View



Test data are then interpreted using Table 5.2.

Table 5.2Interpretation of the Chloride Permeability Output (AASHTO T277-
83I).

Chloride Permeability	Charge Passed in Coulombs
High	4000
Moderate	2000 to 4000
Low	1000 to 2000
Very Low	100-1000
Negligible	100 or less

Through this table, one can compare the chloride permeability of the concrete under test to other types of concrete. The higher the charge passed, the higher the permeability of the specimens.

5.3.2 Permeability Assessment By Ponding Test

The AASHTO T259-80 and T260-84 standard test methods for Resistance of Concrete to Chloride Ion Penetration were followed. Two slabs 76 mm (3 in) thick and 305 mm (12 in) square base were prepared for each of the four mixes in our study in accordance with the above-mentioned standard test methods.

5.5 EXPERIMENTAL RESULTS AND DISCUSSIONS

5.5.1 Rapid Chloride Permeability Test Results

An overall examination of the rapid chloride permeability test results (Figures 5.9 and 5.10) indicates that non-latex-modified steel fiber reinforced concrete

produces very high permeability values irrespective of the curing regime used. Since there is no strong reason for steel fibers to have such dramatic effect on concrete permeability, one may hypothesize that the high permeability values observed are misleading. The reliance of the rapid chloride permeability test on electrical conductivity as the means to measure chloride ion diffusivity seems to be the key problem; this test method cannot distinguish between the effects of steel fibers and chloride ion diffusion on electrical conductivity of concrete.



Figure 5.9 Rapid Chloride Permeability Test Results For The Air Curing Case.



Figure 5.10 Rapid Chloride Permeability Test Results For The Moist Curing Case.

Figures 5.11 through 5.14 present the chloride permeability test results for different matrices (and different curing regimes at different ages). While latexmodified mixtures (both plain and steel fiber reinforced) show less permeability under air drying, non-modified mixtures benefit from continuous moist curing; extended air drying actually causes an increase in non-modified concrete permeability, possibly due to shrinkage microcracking.

Latex modification is observed in Figures 5.11 through 5.14 to substantially reduce the permeability of plain and steel fiber reinforced concretes (by approximately 80% in air curing conditions and 50% in continuous moist curing).



Figure 5.11 Rapid Chloride Permeability Test Results For Plain Concrete Matrices.



Figure 5.12 Rapid Chloride Permeability Test Results For Latex-Modified Concrete Matrices.



Figure 5.13 Rapid Chloride Permeability Test Results For Steel Fiber Reinforced Concrete Matrices.



Figure 5.14 Rapid Chloride Permeability Test Results For Latex-Modified Steel Fiber Reinforced Concrete Matrices.

One-way statistical analyses of variance of the effects of curing age on permeability were performed for each mixture and each curing regime. The results indicate that curing age has statistically significant effects, at 95% confidence level, on the permeability of the latex-modified concrete matrices only when moist cured. For plain matrices, curing age has statistically significant effects on the resulting chloride permeability only at the 92% confidence level for both curing regimes. Statistical Analysis by the method of comparison of means showed no significant effect of age on the rapid chloride permeability test results.

An overall $2 \ge 2 \ge 2 \ge 4$ ANOVA of the test results revealed that the three factors: Fiber addition (A), latex modification (B), and curing type (C), have statistically significant effects on the rapid chloride permeability of concrete. However, statistical analyses also showed a very significant interaction between the effects of latex modification and steel fiber addition (AxB) on the rapid chloride permeability of concrete at 99% level of confidence. Strong interaction was also found between the effects of latex modification and curing regime (AxC) on the concrete permeability. The significance of these interaction effects implies that the former conclusions regarding the main effects are meaningless at this point. It actually means that the main effects A, B, and C are not acting independently; the main effects of the variables can be interpreted individually only if there is no evidence that the variables interact with each other. Hence the need to perform further statistical analyses.

The best way to analyze two-level factorials is by graphical means. Drawing a Normal Probability plot of the effects and fitting a straight line through the center

group of effects is the starting point. Since in a probability plot most points will fall on a straight line, those that are significant (i.e., the effects distinguishable from noise) will not. Therefore, we need to look for the points not falling on a line. After drawing the Normal Probability plot (Figure 5.15), one may note that the points that depart greatly from this line correspond to the main effects: latex modification (factor A), steel fiber reinforcement (factor B), curing type (factor C), and the two-factor interactions between factors A and B, and factors A and C. These are the points corresponding to the statistically significant effects. Significant effects and interaction plots were then drawn.



Figure 5.15 Normal Probability Plot.

Due to the significance of interactions, one needs to find out how the variation in each variable affects permeability at constant levels of the interacting variables. "Interaction curves" are used to achieve this goal. The interaction curve for the effects of curing type (moist vs. air) at constant levels of latex-cement ratio (Figure 5.16a) indicates that latex-modified mixtures produce comparable permeabilities with moist curing and air drying while non-modified mixtures benefit substantially from moist curing which reduces their permeability to about half the values obtained with air drying. Figure 5.16b further emphasizes this point and also clearly demonstrates the positive effects of latex on reducing the permeability of concrete, which are pronounced in air drying conditions.

Figure 5.17 shows the interactions between fiber reinforcement and latex modification in determining the rapid chloride permeability test results. Figure 5.17a suggests that latex modification is highly effective in controlling the adverse effects of steel fibers on rapid chloride permeability test results. Figure 5.17b further highlights the effectiveness of latex in plain and particularly steel fiber reinforced concrete. Latex polymer could reduce the chloride permeability test results by coating the steel fibers and thus providing them with an insulating layer which reduces the overall electrical conductivity of steel fiber reinforced concrete.

An overall value of the effects of different variables on rapid chloride permeability test results is presented in the "cube plot" of Figure 5.18.

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(a)

(b**)**

Figure 5.16 Interaction of Curing Condition and Latex Content in Determining Rapid Chloride Permeability Test Results.



Interaction BxA



Figure 5.17 Interaction of Latex Content and Steel Fiber Content in Determining Rapid Chloride Permeability Test Results.



Response is Permeability in Coulombs

Figure 5.18 Three-Factor Interaction Cube Plot.

Finally, for completeness, construction of two important residual plots was done. The normal probability plot of the residuals shows approximately a straight line thus indicating the normality of the error term (which is always an assumption in this type of analyses that needs to be checked for). In addition, a plot of the residuals versus the predicted values shows no pattern at all in the scatter of the points and therefore no transformations are needed.

5.5.2 Chloride Ion Penetration by Ponding Test Results

The results of the 90-day ponding test are shown in Figure 5.19. The results still indicate the superiority of latex-modified concrete impermeability characteristics as compared to plain concrete. The permeability of steel fiber reinforced concrete is seen to be slightly higher than the plain concrete one. However, the trend of drastic increase in permeability as detected by the previous test method (Rapid Chloride Permeability test) is not seen here, thus indicating the dependability of the latter test on material resistivity and electrical conductivity properties. Statistical analysis of variance did not detect any of the main effects nor interactions to have a statistically significant effect on the ponding test results. In addition, relatively weak statistical correlation was detected between the rapid chloride permeability and the ponding test results (correlation coefficient of about 0.65). The lack of any significant correlation between the two test methods is mainly attributed to the existence of steel fibers that remarkably increase the electrical conductivity of the resulting concrete matrix thus yielding extremely high values of permeability when using the rapid chloride permeability test procedure which is dependent upon the electrical conductivity of the sample being tested. This situation is, however, avoided when the 90-day ponding permeability test procedure is adopted, and hence the lack of statistical correlation between the two test procedures. The performance of this alternative permeability test method revealed that steel fiber reinforced concrete composites possess a high permeability (of the same order of magnitude, but slightly higher, as the plain concrete case) and that the use of the other test method (i.e., rapid chloride permeability) produces very misleading results.



Figure 5.19 Ponding Permeability Test Results.

5.6 SUMMARY AND CONCLUSIONS

The effects of latex modification and steel fiber reinforcement on concrete permeability were assessed through a $2 \times 2 \times 2 \times 4$ factorial experiment which considered two latex-cement ratios (0 and 10%), two steel fiber volume fractions (0 and 0.75%), two curing conditions (air and moist curing following an initial moist curing period), and four ages of permeability measurements (28 days, and 2, 3, and 4 months). Concrete permeability was assessed using the rapid chloride permeability test method (AASHTO T277 and ASTM C1202) and the Resistance of Concrete to Chloride Ion Penetration test method (AASHTO T259 and T260).

Statistical analyses of the rapid chloride permeability test results suggested, at 95% level of confidence, that:

(1) The presence of steel fibers tends to substantially increase the rapid chloride permeability test results; this could result from the reliance of this test method on concrete conductivity for measuring chloride ion diffusivity, noting that steel fibers may increase the electrical conductivity of concrete without substantially increasing chloride ion diffusivity hence, the applicability of the rapid chloride permeability test method (AASHTO T277 and ASTM C1202) to steel fiber reinforced concrete is questionable.

(2) Latex modification controlled the adverse effects of steel fibers on rapid chloride permeability, possibly by providing the fibers surfaces with an insulating layer and also substantially reducing the permeability of the matrix.

(3) Latex modification was highly effective in reducing concrete permeability. Air

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curing helped further pronounce the positive effects of latex polymers on reducing the rapid chloride permeability test results; continuous moist curing, on the other hand, was beneficial to non-modified mixtures.

(4) Curing age has statistically significant effects, at 95% confidence level, on the permeability of the latex-modified concrete matrices only when moist cured.

Statistical analysis of the ponding test results indicated that:

(5) No strong statistical correlation was detected between the rapid chloride permeability and the ponding permeability test methods because of the misleading test results obtained from the first test method due to the increase in electrical conductivity of the material incorporating steel fibers.

(6) When concrete matrices are reinforced with steel fibers, it is recommended not to use the rapid chloride permeability test method in assessing the permeability of the resulting concrete.

CHAPTER SIX

CORROSION AND ELECTRICAL RESISTIVITY STUDIES

6.1 INTRODUCTION

Corrosion of reinforcing steel in concrete structures is a problem witnessed internationally. The corrosion-damaged structures frequently show signs of severe deterioration after a few years of service only.

In the U.S. alone, it has been estimated that the economic loss due to the corrosion of steel in Portland cement concrete structures reaches a few billions of dollars per year.³⁷ In the Gulf States, many steel reinforced concrete structures that have been designed for a service life of 50 to 60 years have required extensive and costly repairs in less than 15 years.⁶⁹ Corrosion damage to parking garages in cold climates, bridge-deck surface spalling due to corrosion of embedded reinforcing steel, and corrosion in many structures due to the use of chloride-based accelerating agents, all constitute a significantly large group of reinforced concrete structures undergoing severe deterioration due to one single cause, that is corrosion.

In concrete materials, interfacial microcracks do initially exist at aggregatecement paste and steel-cement paste interfaces. Further microcracking may occur due to the corrosion of embedded steel resulting in large visible cracks. Once these

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cracks become interconnected with closely-spaced microcracks, the transport of aggressive ions and gases to the embedded steel is facilitated, therefore promoting corrosion. This corrosion leads to the formation of large amounts of expansive products and hence to cracking of concrete in a direction parallel to the reinforcement.

6.2 BACKGROUND

Portland cement acquires its adhesive property when mixed with water through a chemical reaction referred to as the hydration of cement, which results in products possessing setting and hardening characteristics. The reaction of Portland cement with water is actually the reaction of the four clinker materials:

tricalcium silicate (3CaO.SiO₂) dicalcium silicate (2CaO.SiO₂) silicates tricalcium aluminate (3CaO.Al₂O₃) tetracalcium aluminate ferrite (4CaO.Al₂O₃.Fe₂O₃) aluminates

These minerals react with water giving very insoluble precipitates of calcium silicate hydrates (like $3CaO.2SiO_2.3H_2O$) and calcium aluminate hydrates (like $4CaO.Al_2O_3.nH_2O$) together called the cement gel, as follows:

 $3CaO.SiO_2 + 6H_2O \rightarrow 3CaO.2SiO_2.3H_2O + 3Ca(OH)_2$ $3CaO.Al_2O_3 + (n-1)H_2O + Ca(OH)_2 \rightarrow 4CaO.Al_2O_3.nH_2O$

The tricalcium silicate and dicalcium silicate produce free lime $(Ca(OH)_2)$, which is very soluble. This calcium hydroxide (free lime) will react with the sodium and potassium salts (present in cement clinker) giving rise to the formation of very soluble potassium and sodium hydroxide (KOH and NaOH), according to:

$$Na_{2}SO_{4} + Ca(OH)_{2} \rightarrow CaSO_{4} + 2NaOH$$
$$K_{2}SO_{4} + Ca(OH)_{2} \rightarrow CaSO_{4} + 2KOH$$

During the first hours of the hydration of cement, the pH originates partly from production of supersaturated calcium hydroxide and partly from sodium and potassium hydroxides in the pore liquid. At later ages, the pH will be determined mainly by the sodium and potassium hydroxides in solution.

These highly soluble sodium and potassium salts give the pore solution of Portland cement a pH of greater than 13.

The iron in the reinforcement is protected against corrosion by a dense surface film of gamma Fe₂O₃ which is stable in the alkaline environment of the pore solution at the steel-concrete interface. During curing, water first reacts with cement compounds and the excess then evaporates out of the hardened structure, concentrating the Ca(OH)₂ solution in pores and voids and further increasing the corrosion resistance of steel reinforcement (i.e. Portland cement paste possesses considerable CO₂ neutralization capacity).^{50, 51, 76}

It is the existence of the protective oxide film which is insoluble in high alkaline environment that prevents the ionization of the metal and its consequent disintegration.^{50, 51, 70, 76} The passivation normally provided to the steel by the alkaline hydrated cement may be destroyed when the environment in which steel is embedded is no longer able to stabilize the passive film due to a reduction in the alkalinity of cement paste (<11.5). Alternatively, depassivation may result from the presence of aggressive anions in sufficient concentration (such as chloride ions) to encourage metallic dissolution at a rate greater than that required to maintain the passive film. The destruction of the protective film leads to the ionization of the metal and formation of corrosion products in the presence of oxygen and water.

Therefore, the presence of chloride ions, carbon dioxide, oxygen and water are necessary at or near the steel surface for the corrosion to occur and propagate.^{50, 76} The ingress of these corrosion inducing agents to the steel surface depends upon the design, volume and structure of the pore system (permeability).

6.2.1 Electrochemical Corrosion Process

Electrochemical corrosion is the form of corrosion believed to be responsible for most of concrete distress. Transfer of electricity between an electrolyte and a metal electrode consists of a chemical reaction at the surface of the electrode. The current-releasing electrode is known as anode and the electrode at which the current is collected and the electrons are being used up is termed cathode. A cell reaction is the combination of the separate electrode reactions.

For the electrochemical reaction process to take place, all of the following three conditions must be satisfied:⁷⁶

1. Existence of a potential difference between two metallic areas;

2. Existence of an electrolyte conductive to electrical current flow for carrying the current; and

3. The occurrence of appropriate electrode reactions.

This electrochemical process is represented by a Galvanic cell. It consists of dissimilar metals and alloys in contact with an electrolyte, which are capable of producing their own steady-state potentials. Once the electrodes are connected, a current flow occurs between these two electrodes and the anode (the more active metal) corrodes (see Figure 6.1).⁵⁰



Figure 6.1 Electrochemical Process of Steel Corrosion.⁵⁰

Reinforced Portland cement concrete represents an electrochemical corrosion process in that the pore fluid of steel-concrete system acts as electrolyte and the cell formation on steel surface divides the steel into anodic and cathodic regions. Some part of the reinforcing steel becomes anodic and the other cathodic. The fundamental chemical reactions occurring at the electrodes are as follows:



The primary reaction is the chemical reaction involving the ionization of metallic iron at the anode. The free electrons released at the anode will be attracted towards the cathode in order to maintain equilibrium of electric charges that flow through metal,⁷⁶ and will combine with oxygen (from the air) and water resulting in the formation of hydroxyl ions (secondary reaction). Now, the hydroxyl ions from the cathode will combine with the iron ions from the anode to form hydrous iron oxides. The latter ions migrate through the electrolyte (which allows ions to move between anodic and cathodic sites). The hydrous iron oxides (corrosion product) precipitates in the form of ferrous hydroxide (Fe(OH)₂), which is a form of rust with whitish color. Eventually, and in the presence of oxygen and water, ferrous hydroxide is further oxidized to ferric hydroxide Fe(OH)₃ as follows:

 $Fe(OH)_2 + 1/2H_2O + 1/4O_2 \rightarrow Fe(OH)_3$

2Fe(OH)₃ → Fe₂O₃.nH₂O hydrated ferric oxide (red rust)

This conversion of metallic iron to rust or $Fe(OH)_3$ involves 6 to 7 times increase in volume as illustrated in Figure 6.2.⁵⁰ This increased volume of corrosion products produces internal stresses sufficient to disrupt the surrounding concrete, thus causing expansion and cracking.



Figure 6.2 Volume increase in Metallic Iron.⁵⁰

It is worth emphasizing again that the corrosion resistance of steel in concrete would be maintained indefinitely in usual atmospheric exposures if the concrete cover could exclude air and water from the embedded steel reinforcement. When the cover is too thin or porous, corrosive damage to concrete results primarily from access of water solutions containing dissolved oxygen. No damage occurs in very dry concrete due to the absence of essential water. Nor is there much damage in continuously water-saturated concrete because oxygen has less access through the liquid phase than through air in the concrete pores. An unsaturated concrete, which results from alternate wetting and drying, allows more rapid transport of oxygen to the corroding steel surface. The oxygen dissolves in the pore water at the metal surface, enabling electrochemical corrosion. Periodic drying allows deposition and compaction of oxide corrosion products, which impose tensile stresses on the surrounding concrete.

Let us examine more carefully the effects of various factors (including those considered in this investigation) on corrosion activity.

6.2.2 Factors Affecting Corrosion Activity

6.2.2.1 Concrete Quality: Providing a good quality concrete as a basic step in retarding the corrosion phenomenon is of primary importance. The quality of corrosion resistance provided by concrete depends on many factors such as permeability, concrete cover thickness, water-cement ratio, etc.

(a) <u>Permeability</u>: The importance of concrete permeability arises from the fact that it governs the rate at which corrosive solutions can penetrate toward the embedded steel.^{40, 93} Many factors, besides admixtures (e.g. latex polymers),

influence the permeability on concrete: (i) Aggregate type, (ii) cement type, (iii) water-cement ratio, (iv) consolidation procedures, and (v) curing conditions. The water-cement ratio is of primary importance because lowering it will slow the diffusion of chlorides, carbon dioxide, and oxygen. In addition, it will reduce the pore volume and permeability. Moreover, a low water-cement ratio will increase the strength of concrete and may extend the time before corrosion-induced stresses cause cracking of concrete.²⁴ Furthermore, low permeability in addition to minimizing the permeation of corrosion-inducing agents increases the electrical resistivity of concrete which retards the flow of current associated with electrochemical corrosion and in turn the corrosion rate.

(b) Thickness of Concrete Cover: Greater thicknesses of concrete cover delay the penetration of corrosive agents to the steel surface level. In addition to keeping the w/c low for durable concrete, adequate cover thickness must also be provided. It has been observed, in accelerated corrosion tests,³⁸ that the specimens with larger cover thickness-to-bar diameter ratios take a relatively longer time to crack than the ones with smaller ratios.

6.2.2.2 Alkalinity and pH of Concrete: Many investigations have shown that the highly alkaline environment in concrete and the resulting high pH play a very important role in providing reinforcing steel with passivation against corrosion. It was found³³ that at a pH of 12.5 no corrosion occurs in the absence of salts. The presence of salt both lowers the pH of pore space electrolyte and raises the pH required to protect the reinforcing steel.

The alkalinity of the wet concrete is largely due to the presence of Ca(OH)₂
formed during the setting of cement.³⁸ Leaching of alkaline substances with water or partial neutralization by reaction with carbon dioxide or other acidic materials is considered to be very destructive in the reduction of the high-alkaline environment of concrete.

Previous investigations revealed that there is a threshold chloride alkalinity ratio (Cl⁻/OH ratio) that must be exceeded before corrosion can occur. It is reported that the protective film can be destroyed even at pH values considerably above 11.5.⁵⁰ When Cl⁻/OH⁻ molar ratios are higher than 0.6, steel seems to be no longer protected against corrosion. This could be attributed to the fact that the iron-oxide film becomes either permeable or unstable under these conditions. Therefore, the amount of chloride required for corrosion initiation is dependent on the pH of the liquid in the paste. At a pH less than 11.5 corrosion may occur without chlorides; however, at a pH greater than 11.5, a measurable amount of chloride ions is required to initiate corrosion, as illustrated in Figure 6.3.²⁸



Figure 6.3 Relationship Between Chloride, pH in the Liquid Phase, and Corrosion of Iron in Concrete.²⁸

<u>6.2.2.3 Crack Width:</u> Many codes of practice and design regulations provide permissible crack widths in different exposure conditions depending on the environment aggressiveness:

Sea water, wetting and drying	0.15	mm
Water retaining structures	0.10	mm
Deicing chemicals	0.18	mm

It has been found that minor cracks penetrating to the steel are normally not damaging and frequently heal in moist environments,³³ and that the presence of cracks in a direction transverse to the reinforcement does not lead to serious corrosion problems.¹⁹ However, when parallel to the direction of steel reinforcement, cracks may provide access to corroding substances at a greater rate.¹⁹

<u>6.2.2.4 Carbonation</u>: Carbonation is the reaction of hydrated Portland cement with carbon dioxide (CO₂) from the air or water. The CO₂ penetrates the material and reacts with Ca(OH)₂ thus forming CaCO₃. This reduction of the Ca(OH)₂ content of concrete results in a lower alkalinity of the paste system and reduces the pH to about 8.5. At this low pH the steel is no longer passive and corrosion may occur.

Carbonation is not a problem in very dry concrete or in water-saturated concrete. Maximum rates of carbonation are observed at about 50% water saturation.⁵⁰

The carbonation process increases the drying shrinkage of concrete thus promoting crack development.² Although carbonation can penetrate to some considerable distance, in most instances carbonation which substantially reduces

alkalinity is generally limited to a depth of 25 mm (1 in) or so.⁹³

6.2.2.5 Electrical Resistivity of Concrete: With the corrosion of reinforcement embedded in concrete being an electrochemical process, electrical resistivity of concrete has been found to be a dominant factor influencing the intensity of concrete reinforcement corrosion process. In fact, once the passivity of the film on steel surface is destroyed, the rate of corrosion will depend on the electrical resistivity of concrete and the availability of oxygen at the cathode. The magnitude of the corrosion current is primarily controlled by the resistivity of concrete. High resistivity reduces current and thus the probability of corrosion. In humid environments, the degree of water saturation is very high and electrical resistivity is therefore quite low. Consequently, it is the availability of oxygen for the reaction at cathode surface that becomes the principal rate-determining factor in the corrosion process, for example, for ocean structures.

The electrical resistivity of concrete ranges from around $10^3 \ \Omega$ -cm (394 Ω -in) when saturated, to $10^{11} \ \Omega$ -cm (394 x $10^8 \ \Omega$ -in) when oven-dried. For normal moist concrete, the value is around $10^4 \ \Omega$ -cm (3940 Ω -in). It has been shown that corrosion may increase seven fold when the concrete resistivity decreases from 15,000 Ω -cm (5905 Ω -in) to 6,000 Ω -cm (2362 Ω -in). The presence of significant amounts of chloride ions in concrete mix increases the concrete absorption and in turn affects the resistivity of concrete.⁸⁴ It was also found that, in a concrete with resistivity 15,000 Ω -cm (5905 Ω -in), the metal loss due to corrosion is less than 10% after a long time of service, as compared to 80% metal loss in a concrete with 7000 Ω -cm (2756 Ω -in) resistivity.¹

6.2.2.6 Effects of Latex Modification: Only one research work was found in the literature regarding the corrosion-inhibiting property of polymer-modified mortars. In this investigation, polymer-modified mortars using three types of polymers were prepared with polymer-cement ratios of 10 and 20%. The corrosion-inhibiting property of the prepared mortars was evaluated following exposure under severe corrosive conditions of carbonation and chloride penetration. Experimental results revealed that polymer-modified mortars have a superior corrosion-inhibiting effect on the embedded reinforcing steel bars with an increase in polymer-cement ratio.⁶⁷ This is due to their high resistance to carbonation and chloride ion penetration by decreasing carbonation and chloride ion penetration depths. It was also found that the corrosion-inhibiting rate of polymer-modified mortars depends not only on their carbonation and chloride ion penetration depths, but also on the polymer type.

6.2.2.7 Effects of Steel Fiber Reinforcement: Research work performed by Naaman et al.⁴¹ on the durability of steel fiber reinforced mortar (SFRM) revealed that SFRM specimens saturated in a sodium chloride solution for 2 months generally showed increases in strength and toughness compared to standard specimens cured in water for 6 days and tested at 28 days. This could be attributed to the effects of longterm moist curing, which may offset the early effects of corrosion. These studies have also showed that SFRM specimens saturated for over 6 months in a sodium chloride solution showed slight reduction in the peak stress (10%) and more important reductions in toughness (about 25%). Further studies on the long-term deterioration analysis of SFRM specimens indicated that corrosion from the concrete surface is very critical for structures having small depths (<100 mm, 4 in). However, in cases

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where the depth is relatively large (> or = 100 mm, 4 in), the influence of corrosion on the strength of uncracked members is relatively small, even if the structure is saturated in a severe corrosive atmosphere for a long period of time.⁴¹

6.2.3 Corrosion Test Techniques

Although there are variations in equipment and procedure, the main test methods for assessing reinforcement corrosion activity are:²⁶

- 1. Half-Cell Potential Mapping
- 2. Electrical Resistance Probe
- 3. Electrochemical Noise
- 4. Polarization Resistance
- 5. AC Impedance

The first three methods are known as non-perturbative, and the last two as perturbative methods. The non-perturbative test methods (1,2&3) measure changes in the steel/concrete system caused by the process of reinforcement corrosion. The perturbative test methods, on the other hand, measure the response of the steel/concrete system to an applied external perturbance from which the rate of corrosion can be determined.

6.2.3.1 Half-Cell Potential Mapping: This test, developed in the late 1950's, has been extensively used in the USA especially for the evaluation of concrete bridges (ASTM C-876). The test measures the electrode potential of steel in concrete against

a reference electrode. Potentials are measured with a high impedance voltmeter (>10 Mega Ohm) to ensure low current conditions during testing. The measured electrode potential may be affected by a number of factors that should be considered in interpreting the results. One of the most important is the quality of the cover concrete, particularly in moist conditions and contamination by carbonation and/or chlorides. A further limitation of the test is that the results only give an indication of whether corrosion is thermodynamically possible, and no information relating to corrosion kinetics can be obtained.

Measurements are normally presented in the form of equipotential contour maps from which areas of suspected corrosion are identified.

ASTM C-876 provides a classification for assessing the results of the half-cell test as:

Table 0.1 ASTM Classification of Corrosion Half-Cell Potential	Table 6.1	ASTM	Classification	of	Corrosion	Half-Cell	Potential
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Measured Potential	Statistical Risk of Corrosion		
less than -350 mV	90%		
between -350 and -200 mV	50%		
greater than -200 mV	10%		

6.2.3.2 Electrical Resistance Probe (ERP): This test uses the principle that the resistance of a conductive material is a function of its cross-sectional area. Thus, as corrosion reduces cross section, the resistance increases, enabling the progress of corrosion to be measured. The test may be carried out by comparing the resistance change between protected and exposed probes embedded in concrete.²⁶

6.2.3.3 Electrochemical Noise: This is a fairly recent development in the monitoring of reinforcement corrosion. The technique considers small fluctuations in corrosion potential of the steel/concrete system and relates these to the presence and nature of corrosion on the reinforcing steel surface.²⁶ The method uses similar equipment to the half-cell potential method except that a higher resolution voltmeter is used. Then, information relating to the corrosion process may be established from the potential/time plots or from an alternative form of data presentation. Determination of the corrosion rate is made from relationships that have been identified between the standard deviation of the measured noise and the corrosion current obtained from polarization resistance measurements. This method has been used in-situ to provide additional information to half-cell potential mapping.

6.2.3.4 Polarization Resistance: The equipment set up for this test consists of a reference electrode which is located on the concrete surface, a working electrode which is the reinforcement under test, and a counter electrode which may be located either on the concrete surface or embedded within the concrete. The potentiostat automatically controls the operations and normally has electronic IR compensation to take account of the IR drop between the reference and working electrodes resulting from the high resistance of the cover concrete.

The measurement of the corrosion rate electrochemically is based on the measurement of the anodic or cathodic corrosion current (i_{corr}) at the rest potential (E_{corr}) . During corrosion, the system is in equilibrium, i.e. production and consumption of electrical charge at the anode and cathode respectively are equal so there is no net current flow. This test relies on the linear relationship between

potential and current that is considered to exist within the range +10 to 20 mV of the rest potential.²⁶

The polarization resistance can be determined from the plot of applied potential against measured current response.

6.2.3.5 AC Impedance: The test is carried out using a transfer function analyzer and involves the application of an AC signal (typically 10 to 20 mV) to the steel/concrete test system and the modules of impedance and phase shift are considered over a range of frequencies (typically 100 kHz to 1 mHz).²⁶ The data is normally presented in the form of a Nyquist or Bode Plot and result analysis is normally carried out by consideration of an equivalent circuit. The analysis is complex, but it is possible for factors relating to the corrosion mechanism and the corrosion rate to be established.

The test has been used mainly in the laboratory to study the processes of initiation and propagation of corrosion with respect to different concrete variables and also the effectiveness of concrete repair systems.

6.2.4 Concept of Corrosion

What happens is that the surface of the corroding metal (here steel reinforcement) functions as a mixed electrode upon which coupled anodic and cathodic reactions take place.

At an anodic site, the metal atoms (Fe) pass into solution as positively charged hydrated ions (Fe^{2+}) (anodic oxidation) and the excess free electrons flow <u>through the</u> <u>metal</u> to cathodic sites where an electron acceptor (such as hydrogen ion or dissolved

oxygen) is available to consume them (cathodic reduction). This process is completed by electromigration of ions through the electrolyte, leading to the formation of a corrosion product which may be soluble (e.g. ferrous chloride) or insoluble (e.g. rust, hydrated ferric oxide).

It is then essential to have the following three features for the corrosion process:

1) A reactive metal (steel) that will oxidize anodically to form soluble ions,

2) A reducible substance which provides the cathodic reactant, and

3) An electrolyte that allows ions to move between the anodic and cathodic sites.

What we measure is actually the potential drop (i.e., difference in electrical potentials of points in the metal surface and in the neighboring electrolyte) across the electrical double layer. This potential drop is known as electrode potential. The difference between the electrode potentials at anode and cathode is the driving force for the corrosion reaction. These potentials basically provide a measure of the ease of charge transfer across the steel-concrete interface and the ease of ionization of the dissolved oxygen, respectively. The electrode potential is always lower at a local anode than at a cathode of a corrosion couple.

It is not possible to determine the absolute value of an electrochemical potential. For this reason in doing this test, one measures the potential difference between the anode or cathode and a reference half-cell electrode. This is taken as a measure of the actual potential and is quoted in volts relative to the particular reference electrode used. In a reference half-cell electrode, a fixed potential difference is always established between a metal and a solution containing its ions.

6.2.5 Electric Resistivity

The available test techniques for corrosion (potential mapping, half-cell) provide a reliable means of locating corrosion activity. It is, however, impossible to use it to determine the severity of the corrosion problem.

The steel may be corroding at an advanced rate or may be corroding so slowly that it is unlikely to be a problem within the design lifetime of the structure and hence does not need action.

The electrical resistivity of concrete is the most important parameter controlling the rate of corrosion once it has initiated. And, resistivity thresholds have been tentatively proposed to categorize the likely rate of active corrosion.

The electrical resistivity of concrete is highly dependent upon moisture content. The resistivity of concrete made from calcium aluminate cement is about ten times greater than the resistivity of concrete made form Portland cement.³² It has also been found⁵⁶ that paste and concrete are conductors similar in nature to electrolytes, and that the temperature coefficient of electrical conductivity of fresh paste is of the same order of magnitude as that of common electrolytes. Others²⁹ have found that the electrical resistivity of concrete increases with age and decreases with increasing salinity of mixing water. Several investigators have correlated the resistivity of cement pastes with its setting time. The volume of evaporable water in paste found in saturated concrete varies from about 60% at the time of mixing to about 40% when Portland cement is completely hydrated. This water contains ions, primarily Na⁺, K^+ , Ca⁺⁺, SO⁻, and OH⁻, and concentration of some ions would increase while those of others would decrease. It is then expected that conduction of electricity through moist concrete would be essentially electrolytic; many investigators support the view that conduction is by means of ions in the evaporable water. It was found that the resistivity of a certain concrete, which was about 10⁴ Ω -cm (4,000 Ω -in) after seven days of moist curing, was increased to about 10¹¹ Ω -cm (4 x 10¹⁰ Ω -in) when the evaporable water was removed by drying at 105 °C (221 °F), indicating an increase of 10 million times.

The resistance of an electrolyte, or any other material, is directly proportional to the length (L) and inversely proportional to the cross-sectional area (A):

$$R=\rho \cdot L/A$$

where, R=resistance in Ohms (Ω); ρ =resistivity in Ohm-cm (Ω -cm); L=length of specimen used; and A=cross-sectional area of specimen.

The resistivity ρ is essentially constant for a given material and is numerically equal to the resistance of a 1-cm (0.4-in) cube of the material.

Concrete may be considered as a composite system of aggregate particles with different dimensions embedded in a matrix of Portland cement paste. The resistivity of the particles, the resistivity of the matrix, and the aggregate particles is assumed to be infinite (i.e., particles are nonconductive), then, for spherical particles:

$$\rho = \rho_m \cdot \frac{(3 - V_m)}{(2 \cdot V_m)}$$

where,

 V_m = fractional volume of material; and ρ_m = resistivity of material.

6.2.6 Factors Affecting Electrical Resistivity of Concrete

6.2.6.1 Water-Cement Ratio: The effect of w/c on the resistivity of 25-mm (1-in) plain paste cubes is shown in Table 6.2.⁵³ The resistivity of paste with a w/c=0.40 is observed to be about double that of the paste having a w/c ratio of 0.60. The resistivity also increases with time of continuous moist storage. Decrease of evaporable water with continuing hydration would have possibly been a factor in this increase in resistivity. Other factors, such as continuous release of alkalies and leaching, might also influence resistivity.

The resistivities of Paste A and Concrete A (with same w/c) indicated that resistivity of concrete was about five times that of the paste.⁵³

 Table 6.2
 Effect of Water-Cement Ratio and Time of Moist Storage on Electrical Resistivity.⁵³

	w/c	Resistivity, Ohm-cm (1000 Hz,4 volts)			
Matrix Type		7 days	28 days	90 days	
Low Alkali Content	0.40 0.50 0.60	1030 790 530	1170 880 700	1570 1090 760	
High Alkali Content	0.40 0.50 0.60	1230 820 570	1360 950 730	1660 1200 790	

<u>6.2.6.2 Effect of Alkalies:</u> It has been found⁵³ that the resistance of pastes with lower alkali content is considerably higher than that of pastes with higher alkali content during the first day; at 4 days the resistances were equal, and thereafter the resistance of pastes with higher alkali content was higher.

<u>6.2.6.3 Effect of Pure Compounds</u>: The resistances of pastes of C_3S and C_3A have been studied by Monfore.⁵³ At early ages the resistance of the C_3A paste was higher than that of the C_3S paste, probably due to the absence of calcium hydroxide in the hydration products of C_3A . Later, however, the resistance of C_3S increased to a value higher than that of C_3A .

<u>6.2.6.4 Effect of Curing</u>: Experimental investigations⁵³ on the influences of normal curing in limewater at 73°F, steam curing, and autoclave curing, revealed that steam curing yielded highest resistances.

6.2.6.5 Effect of Drying: Substantial increases in resistance resulted as 10-cm (4-in) concrete cubes were dried in an atmosphere of 50% R.H. and 75°F. Removal of all of the evaporable water by oven drying increases the resistance of concrete drastically.⁵³

<u>6.2.6.6 Effect of Admixtures:</u> It has been reported⁵³ that the use of ammonium phosphate in concrete increases its electrical resistivity. At early ages, other admixtures, such as calcium chloride, generally causes decreased resistivity, but at later ages pastes with some admixtures did show moderately increased resistivity.

<u>6.2.6.7 Effect of Aggregate Type:</u> Table 6.3^{53} shows the resistivities of four types of rocks which might be used as concrete aggregates. It can be seen that the resistivities of the marble and granite are essentially infinite as compared to the

resistivity of paste, and the resistivities of the sandstone and limestone are considerably greater than that of paste.

Туре	Absorption, % by wt.	Resistivity, ohm- cm (1000 hertz, 4 volts)
Sandstone	9.2	18,000
Limestone	6.0	30,000
Marble	0.9	290,000
Granite	0.34	880,000

Table 6.3Electrical Resistivities of Rocks.

6.3 EXPERIMENTAL PROGRAM

The four concrete mixtures of Table 3.7 were also considered in this investigation: plain, latex-modified, steel fiber reinforced, and latex-modified steel fiber reinforced.

The experimental program was designed to obtain conclusions on the following issues:

a) determine the time-to-corrosion-initiation of plain versus steel fiber

reinforced concrete, and plain versus latex-modified concrete;

b) evaluate the effect of concrete quality on the corrosion-initiation time; and

c) assess the electrical resistivity of different concrete mixtures and its effect

on corrosion potential.

6.3.1 Corrosion Monitoring Test Program

The corrosion tests were conducted using 75 x 150 mm (3 x 6 in) cylindrical specimens with 12 mm (0.5 in) diameter rebars placed along their longitudinal axis. Cylindrical aluminum alloy molds were used with some modifications in order to be able to hold the steel reinforcement in the middle of the specimen and to provide a uniform cover thickness throughout. In this modified arrangement (see Figure 6.4) the cylindrical part of the mold rests on a plate having a hole in the middle such that the center of the hole coincides with the cylinder axis along which the reinforcing bar is placed. Threaded rods support this plate to a base plate. The latter has a groove aligned with the hole in the upper plate so that the reinforcing steel bar passes through the hole in the upper plate and rests on the groove provided in the base plate. Before casting the specimens, the molds were coated with a thin layer of mineral oil and then a 12.5 mm (0.5 in) diameter steel bar was placed. The pouring of concrete into the molds was performed at three layers of one-third the cylinder's height. External vibration by means of a vibrating table was applied to compact the concrete, and troweling was performed to smoothen the finished surface. The specimens were then covered with a wet burlap topped with a polyethylene sheet to prevent water evaporation from the unhydrated concrete. Demolding of specimens from the molds was done after 24 hours. The specimens were then placed in laboratory environment with a temperature of 23 to 25°C (73.4 to 77°F) and 50 to 60% Relative Humidity up to the test age of 60 days.

In the corrosion test program, the specimens were placed in a tank and partially immersed in a 5% sodium chloride solution (see Figure 6.5). The level of solution was kept at twice the cover thickness. Corrosion was monitored by obtaining half-cell potentials of the embedded steel at regular time intervals using a millivoltmeter. ASTM C-876 provides the classification shown in Table 6.1 for assessing the results of the half-cell test:



Figure 6.4 Arrangement of Molds For Specimens.



Figure 6.5 Corrosion Test Set-Up.

6.3.2 Electrical Resistivity Measurements Test

The electrical resistivity specimens were 102-mm (4-in) cubes. Four specimens were made for each of the mixes of Table 3.7; two of them were stored in moist curing room after demolding and the remaining two were kept in laboratory environment (23 to 25°C, 73.4 to 77°F, and 50 to 60% R.H.) until the test age of 30 days.

Brass plate electrodes were used to measure electrical resistivity (see Figure 6.6). Contact between these electrodes and the specimen faces was achieved by using a liquid paste with water-cement ratio of 0.5. This paste has a very low resistivity and therefore in a thin layer it has a relatively negligible resistance as compared to the resistance being measured. An HP4284A Precision LCR Meter was used to measure the electric resistance.



Figure 6.6 Electrical Resistivity Test Set-Up.

6.4 **EXPERIMENTAL RESULTS AND DISCUSSIONS**

6.4.1 Corrosion Monitoring Test Results

Figures 6.7 through 6.10 show the half-cell corrosion potential measurements for plain, latex-modified, steel fiber reinforced, and latex-modified steel fiber reinforced concretes, respectively. Each value plotted is an average of three readings obtained on three specimens from the same batch. In plain concrete, the corrosion potentials are initially seen to increase and then continue increasing at a very slow rate and somewhat stabilize. In steel fiber reinforced concrete matrices, however, these potentials increase in a relatively sharper fashion. In both latex-modified and steel fiber reinforced latex-modified concrete mixtures, the corrosion potential readings throughout the whole testing period are seen to be quite stable and steadily below (in absolute value) -100 millivolts. At the end of the testing period of about five months, the half-cell corrosion potentials reached about -380, -65, -620, and -70 millivolts for plain, latex-modified, steel fiber reinforced, and latex-modified steel fiber reinforced concretes, respectively.



Figure 6.8 Results of Corrosion Potential Measurements For Latex-modified Concrete.



Figure 6.9 Results of Corrosion Potential Measurements For Steel Fiber reinforced Concrete.



Figure 6.10 Results of Corrosion Potential Measurements For Latex-Modified Steel Fiber reinforced Concrete.

Statistical analysis of variance of the test results, performed at 100 days of testing, revealed that, at 95% confidence level, both fiber reinforcement and latex modification have statistically significant effects on corrosion activity taking place at the rebar. At 100 days, the 95% confidence intervals for corrosion potentials in plain, latex-modified, steel fiber reinforced, and latex-modified steel fiber reinforced concrete were 258.3 ± 31.6 , 59.7 ± 31.6 , 427.4 ± 31.6 , and 63.6 ± 31.6 millivolts, respectively (see Figure 6.11). This indicates that steel fibers have adverse effects on corrosion conditions of reinforcing bars whereas latex polymers add very significantly to the corrosion-inhibiting characteristics of both plain and steel fiber reinforced concretes.

Latex modification is observed to reduce the 100-day corrosion potential of plain concrete by approximately 76%. Steel fiber reinforcement, however, increased this potential by about 65%. The corrosion potential measurement of plain concrete without latex indicates a 50% statistical risk of corrosion, while with steel fibers the obtained corrosion potential indicates a 90% statistical risk of corrosion. This risk is reduced to less than 10% when concrete mixtures, whether plain or steel fiber reinforced, have latex polymers incorporated into them.

The threshold potential for corrosion (reflecting the initiation of corrosion) is taken at -250 millivolts. Half-cell potentials numerically greater than this value indicate that reinforcing steel corrosion is occurring. Figures 6.7 through 6.10 show that the corrosion threshold potential is reached after about 90 and 60 days of testing in plain and steel fiber reinforced concretes, respectively. In latex-modified mixtures (i.e., latex-modified unreinforced and latex-modified steel fiber reinforced concretes) the threshold potential was never attained at any stage of the corrosion monitoring experiment. This confirms that reinforcing bars are more likely to corrode in concrete matrices reinforced with short steel fibers than in plain concrete matrices.



Figure 6.11 100-Day Corrosion Potentials (with 95% Confidence Intervals).

6.4.2 Electrical Resistivity Test Results

Figure 6.12 presents the obtained electrical resistivity test results. These results do, in fact, support the above findings. Steel fiber reinforcement does indeed reduce the resistivity (i.e., increase the conductivity) of the resulting concrete material by about 71% when continuously moist cured and by about 63% when cured in ambient laboratory environment (see Figure 6.12), noting that conductivity is the reciprocal of resistivity. Latex modification, on the other hand, increases the electrical resistivity of concrete, by about 200% when continuously moist cured and by about 1600% when cured in ambient laboratory conditions (see Figure 6.12). Statistical analysis of variance showed that fiber reinforcement has a statistically significant effect on electrical resistivity of concrete, at 95% level of confidence. It is interesting to note that steel fibers reduced concrete resistivity by about 65% (in the moist curing case) and increased corrosion potentials also by about 65%. This provides further support for the hypothesis that adverse effects of steel fibers on corrosion condition result from reduced electrical resistivity of concrete in the presence of steel fibers. It was also found, at 95% confidence level, that latex modification and curing method (moist vs. dry) have statistically significant effects on the resulting electrical resistivity of the concrete material. Relatively strong statistical correlation was also detected between the electrical resistivity of a material and its corresponding corrosion-inhibiting property. The correlation coefficient between electrical resistivity and 100-day corrosion potential was about -0.95.

Concrete, being a composite of aggregate particles (sand and gravel) in a Portland cement matrix, normally has three possible paths for conduction of electrical current: (a) through the paste itself; (b) through the aggregate particles in contact with each other; and (c) through the aggregate and paste in series. Incorporating steel fibers into the composite adds, to the previous list, four other possible paths for conducting electrical current: (d) through the steel fibers in contact with each other; (e) through the steel fibers and aggregate in series; (f) through the steel fibers and matrix in series; and (g) through the steel fibers, aggregates and cement matrix in series. Steel fibers, due to their relatively high electrical conductivity, induce lower resistivities (i.e. higher conductivities) to the overall composite as current flows through either of the latter four paths. Consequently, steel fiber addition seems to ease the motion of electrons within the concrete matrix during any chemical or electrochemical process such as corrosion of reinforcing bars.

The adverse effects of steel fibers on rebar corrosion in concrete can thus be attributed to increased electrical conductivity of concrete materials incorporating steel fibers; this would pronounce the electrochemical process of corrosion. The possible increase in diffusivity of concrete due to fiber-matrix interface microcracking (caused by differential shrinkage and thermal strains) could further increase corrosion potentials.

The positive effects of latex modification on rebar corrosion can be attributed to decreased electrical conductivity of concrete materials incorporating latex polymers, thus remarkably slowing down the electrochemical process of corrosion. This is mainly due to the formation of a thin continuous polymer film that coats the various mix constituents (aggregate particles and steel fibers), thus providing an insulating layer and reducing the electrical conductivity of the resulting concrete material. In addition, this polymer film fills up the pores and improves the interfacial bond characteristics resulting in reduced penetration of moisture, oxygen, carbon dioxide and chloride ions into the low-permeability latex-modified mixtures thus improving the corrosion-inhibiting properties of the concrete.



Figure 6.12 Electrical Resistivity Test Results (with 95% Confidence Intervals).

6.5 SUMMARY AND CONCLUSIONS

The effects of steel fiber reinforcement and latex modification on the corrosion-inhibiting properties and electrical resistivity characteristics of concrete were investigated. Corrosion test results indicated that while latex polymers have positive corrosion-inhibiting effects when added into concrete materials, steel fiber reinforcement has adverse effects on corrosion conditions of reinforcing bars. Statistical analysis of variance of the corrosion test results revealed that, at 95% confidence level, both latex polymer modification and fiber reinforcement have statistically significant effects on corrosion activity taking place at the rebar. The 95% confidence intervals for 100-day corrosion potentials of rebars in plain, latexmodified, steel fiber reinforced, and latex-modified steel fiber reinforced concrete were 258.3 ± 31.6 , 59.7 ± 31.6 , 427.4 ± 31.6 , and 63.6 ± 31.6 millivolts, respectively. The threshold potential for corrosion (taken at -250 millivolts) was reached sooner for steel fiber reinforced concrete than for plain concrete, indicating that reinforcing bars are more likely to corrode in concrete matrices reinforced with short steel fibers than in plain concrete matrices. This threshold was never reached for the latex-modified plain and steel fiber reinforced concrete matrices.

The adverse effects of steel fibers could be illustrated with the obtained electrical resistivity test results. Steel fiber reinforcement does indeed reduce the resistivity of the resulting concrete material by about 71% when continuously moist cured and by about 63% when cured in ambient laboratory environment. Statistical analysis of variance showed that both latex modification and steel fiber reinforcement

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have statistically significant effects on electrical resistivity of the concrete material, at 95% level of confidence. The percentage increase in corrosion potential in the presence of steel fibers is comparable to the corresponding percentage reduction in electrical resistivity. This provides support for the hypothesis that the adverse effects of steel fibers on rebar corrosion in concrete result from reduced electrical resistivity of concrete in the presence of steel fibers. Electrical resistivity and corrosion potentials were also found to statistically correlate reasonably well.

At 10% latex-cement ratio, corrosion-inhibiting property of concrete was increased by 76%. The positive effects of latex polymer on corrosion inhibition seems to result from reduced conductivity (i.e., increased electrical resistivity) and also from reduced penetration of moisture, oxygen, carbon dioxide and chloride ions into the low-permeability latex-modified mixtures. In the presence of steel fibers, the coating of fibers with latex polymer seem to play a key role in reducing the electrical resistivity and corrosion potential of latex-modified steel fiber reinforced concrete.

CHAPTER SEVEN

SUMMARY AND CONCLUSIONS

An experimental study was conducted to assess the mechanisms of joint action of steel fibers and latex polymers in concrete materials in order to develop latexmodified steel fiber reinforced concretes with balanced improvements in diverse aspects of material properties.

The objectives of this research were achieved through the performance of the following tasks:

• Mix proportioning of concrete matrix with steel fibers and latex polymers for achieving acceptable fresh mix properties while providing the promise for desirable hardened material characteristics.

• Comprehensive experimental characterization of latex-modified plain and steel fiber reinforced concretes, and optimization of the latex content and curing procedure.

• Investigation of failure mechanism and microcracking under compressive loading.

• Comprehensive assessment of the effects of latex modification and/or steel fiber reinforcement of on concrete permeability using different test techniques.

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• Investigation of the effects of steel fiber reinforcement and/or latex modification on the corrosion-inhibiting properties and electrical resistivity characteristics of concrete.

Comprehensive sets of replicated experimental data were generated in this study and analyzed statistically using the analysis of variance, comparisons of means, and multiple comparison techniques in order to derive statistically reliable conclusions.

A summary of the activities related to these phases together with the corresponding conclusions are presented below. In all the discussions that follow, except when otherwise indicated, a significant effect represents a confidence level between 95 and 99%, and a highly significant effect represents confidence levels higher than 99%.

7.1 DEVELOPMENT AND BASIC CHARACTERIZATION OF LATEX-MODIFIED STEEL FIBER REINFORCED CONCRETE

The test results generated in this study indicated that:

• Latex modification tends to increase both the initial and final times of setting in concrete mixtures;

• The addition of steel fibers tends to lower the setting time, irrespective of the latex content in the matrix.

• The loss of workability with time takes place at almost the same rate for all mixes irrespective of the latex and steel fiber contents.

• Latex modification and steel fiber reinforcement increase the impact resistance of

the concrete matrix, with superior impact strengths obtained when steel fibers and latex polymers are used simultaneously, indicating an effective interaction between latex and steel fibers resulting from the improved bonding between the fibers and the latex-modified mixtures.

• The separate actions of latex polymers and steel fibers in concrete lead to improved flexural strength of concrete. The combined action of latex modification and steel fiber reinforcement leads to highest flexural strength values.

• Steel fiber reinforcement is effective in increasing the flexural toughness (area underneath the flexural load-deflection curve) of concrete. While latex modification has relatively small effects on the flexural toughness of plain concrete, it is capable of significantly improving the toughness characteristics of SFRC. These improvements can be attributed to the positive effects of latex modification on the fiber-to-matrix interfacial bond characteristics and pull-out behavior.

• Wet curing at early ages is detrimental to mixtures containing latex polymers, while it is advantageous in mixtures without latex;

• Exposure to hot water (compared to water at ambient temperature) after a period of air drying (26 days) reduces the flexural strength of LMSFRC; and

• The effects of latex modification on SFRC mixtures are very significant (in a negative manner) when curing is achieved by continuous immersion in water, or (in a positive manner) when curing involves air drying after a short period of moist curing.

• Curing time, matrix type and their interaction have significant effects on the compressive strength of concrete. Latex-modified plain and steel fiber reinforced concretes tend to continue with strength development at later ages when exposed to

air. This can be attributed to the desirable internal moist curing conditions in latex-modified mixtures provided by the polymer film formed at the surfaces and inside the concrete.

• From the statistical analysis of the bond test results, it could be concluded that:

relatively large variations exist in the slant shear test results for the assessment of bond between the new (overlay) and old (base) concretes;

the effect of latex modification on the bond strength between overlay concrete and base tends to be larger than that of steel fiber addition;
there is a noticeable interaction between latex modification and steel fiber

addition on bond strength;

■ at $V_f = 0.5\%$, the increase in latex-cement ratio up to 10% consistently increases the bond strength, while for $V_f = 1\%$, there is an increase and then a decrease in bond strength with increasing latex content. The highest bond strength was achieved at $V_f = 0.5\%$ and L/c = 10%.

for non-modified overlay concrete, failure typically occurs along the bond surface. When the overlay is modified with latex polymers, the bond improves significantly and failure generally occurs in the base material.

fiber effects on bond may result from the mechanical bonding resulting from the interlocking of fibers at the interface. The interaction between latex and

fiber effects on bond could result from the consumption of latex at the relatively large surface area of fibers, which leaves less latex available to act at the interface.

• Statistical analyses of the test results generated in the freeze-thaw study indicated

that:

Intex modification imparts improvements to the freeze-thaw resistance of concrete with no need to additional air entrainment;

steel fiber addition also somewhat improves the freeze-thaw durability characteristics of air-entrained concrete matrices;

■ there is no noticeable interaction between latex modification and steel fiber addition as far as the resistance to repeated freeze-thaw cycles is concerned;

• Latex modification is observed to improve the scaling resistance of both plain and steel fiber reinforced concretes.

• While steel fiber addition had no significant effect on the water absorption of the concrete, latex modification, due to the pore filling characteristics of latex polymers when incorporated into concrete matrices, is found to significantly decrease the water absorption capacity of both plain and fiber reinforced concretes.

• Latex modification, steel fiber reinforcement, and their interaction had no significant effects on concrete's bulk specific gravity.

• Both latex polymers and steel fibers have significant effects on the drying shrinkage of the resulting concrete. Analyses also revealed the existence of an interaction between the two. The presence of fibers reduces the potentials of latex polymers for reducing shrinkage movements.

7.2 MICROCRACKING CHARACTERISTICS UNDER COMPRESSIVE LOADING

The effects of latex modification and/or steel fiber reinforcement on the microcracking process and failure mechanism in concrete materials subjected to compression loads were studied. For this purpose concrete specimens were subjected to different levels of compressive stress in the pre-peak and post-peak regions, and were then unloaded. Image analysis techniques were employed to quantify microcrack intensities within the paste and at the aggregate- and fiber-matrix interfaces at different load levels. The results indicated that:

• Microcracks are present in concrete materials, dominantly at the coarse aggregate-matrix and fiber-matrix interfaces, even prior to any loading. These microcracks may be caused by differential shrinkage and thermal movements between cementitious matrices and mix inclusions (i.e., aggregates and fibers);

• In plain concrete, microcracks tend to originate mainly at the coarse aggregate-cement paste interfaces, and then extend into the matrix at higher compression levels; near the peak load, microcracks tend to interconnect and localize, forming macrocracks with increasing widths and there is a rather sudden growth in microcrack intensity beyond the peak compression load;

• In latex-modified concrete, the microcrack intensities, especially at aggregate-paste interfaces, tend to be reduced, indicating improvements in bonding between the cementitious paste and aggregates as a result of latex modification; beyond the peak load, however, the sudden increase in microcrack intensity still takes

place in spite of latex modification;

• In unmodified steel fiber reinforced concrete, the fiber-matrix interface microcracks seem to play a dominant role in deciding the failure mechanism of the concrete matrix due to their relatively rapid propagation under increasing compressive stress levels; steel fibers, however, control microcrack propagation in the post-peak region; and

• Latex modification of steel fiber reinforced concrete seems to stabilize the fiber-matrix interface microcracks as well as the aggregate-matrix interface microcracks leading to a more stable microcrack system within steel fiber reinforced concrete materials.

7.3 **PERMEABILITY STUDIES**

The effects of latex modification and steel fiber reinforcement on concrete permeability were assessed through a $2 \times 2 \times 2 \times 4$ factorial experiment which considered two latex-cement ratios (0 and 10%), two steel fiber volume fractions (0 and 0.75%), two curing conditions (air and moist curing following an initial moist curing period), and four ages of permeability measurements (28 days, and 2, 3, and 4 months). Concrete permeability was assessed using the rapid chloride permeability test method (AASHTO T277 and ASTM C-1202) and the Resistance of Concrete to Chloride Ion Penetration test method (AASHTO T259 and T260).

Statistical analyses of the rapid chloride permeability test results suggested, at 95% level of confidence, that:

• The presence of steel fibers tends to substantially increase the rapid chloride permeability test results; this could result from the reliance of this test method on concrete conductivity for measuring chloride ion diffusivity, noting that steel fibers may increase the electrical conductivity of concrete without substantially increasing chloride ion diffusivity hence, the applicability of the rapid chloride permeability test method (AASHTO T277 and ASTM C-1202) to steel fiber reinforced concrete is questionable.

• Latex modification controlled the adverse effects of steel fibers on rapid chloride permeability, possibly by providing the fibers surfaces with an insulating layer and also substantially reducing the permeability of the matrix.

• Latex modification was highly effective in reducing concrete permeability. Air curing helped further pronounce the positive effects of latex polymers on reducing the rapid chloride permeability test results; continuous moist curing, on the other hand, was beneficial to non-modified mixtures.

• Curing age has statistically significant effects, at 95% confidence level, on the permeability of the latex-modified concrete matrices only when moist cured.

Statistical analysis of the ponding test results indicated that:

• No strong statistical correlation was detected between the rapid chloride permeability and the ponding permeability test methods because of the misleading test results obtained from the first test method due to the increase in electrical conductivity of the material incorporating steel fibers.

• When concrete matrices are reinforced with steel fibers, it is recommended not to use the rapid chloride permeability test method in assessing the permeability of the resulting concrete.

7.4 CORROSION AND ELECTRICAL RESISTIVITY STUDIES

The effects of steel fiber reinforcement and latex modification on the corrosion-inhibiting properties and electrical resistivity characteristics of concrete were investigated. Corrosion test results indicated that while latex polymers have positive corrosion-inhibiting effects when added into concrete materials, steel fiber reinforcement has adverse effects on corrosion conditions of reinforcing bars. Statistical analysis of variance of the corrosion test results revealed that, at 95% confidence level, both latex polymer modification and fiber reinforcement have statistically significant effects on corrosion activity taking place at the rebar. The 95% confidence intervals for 100-day corrosion potentials of rebars in plain, latexmodified, steel fiber reinforced, and latex-modified steel fiber reinforced concrete were 258.3 ± 31.6 , 59.7 ± 31.6 , 427.4 ± 31.6 , and 63.6 ± 31.6 millivolts, respectively. The threshold potential for corrosion (taken at -250 millivolts) was reached sooner for steel fiber reinforced concrete than for plain concrete, indicating that reinforcing bars are more likely to corrode in concrete matrices reinforced with short steel fibers than in plain concrete matrices. This threshold was never reached for the latex-modified plain and steel fiber reinforced concrete matrices.

The adverse effects of steel fibers could be illustrated with the obtained electrical resistivity test results. Steel fiber reinforcement does indeed reduce the resistivity of the resulting concrete material by about 71% when continuously moist
cured and by about 63% when cured in ambient laboratory environment. Statistical analysis of variance showed that both latex modification and steel fiber reinforcement have statistically significant effects on electrical resistivity of the concrete material, at 95% level of confidence. The percentage increase in corrosion potential in the presence of steel fibers is comparable to the corresponding percentage reduction in electrical resistivity. This provides support for the hypothesis that the adverse effects of steel fibers on rebar corrosion in concrete result from reduced electrical resistivity of concrete in the presence of steel fibers. Electrical resistivity and corrosion potentials were also found to statistically correlate reasonably well.

At 10% latex-cement ratio, corrosion-inhibiting property of concrete was increased by 76%. The positive effects of latex polymer on corrosion inhibition seems to result from reduced conductivity (i.e., increased electrical resistivity) and also from reduced penetration of moisture, oxygen, carbon dioxide and chloride ions into the low-permeability latex-modified mixtures. In the presence of steel fibers, the coating of fibers with latex polymer seem to play a key role in reducing the electrical resistivity and corrosion potential of latex-modified steel fiber reinforced concrete.

7.5 RECOMMENDED APPLICATIONS

Steel fiber reinforced latex-modified concrete presents a unique balance of superb qualities. Practically all key aspects of concrete performance (physical, mechanical, durability and adhesion qualities) are substantially improved through the joint action of steel fibers and latex polymers. Latex modification enhances the reinforcement efficiency of steel fibers and mitigates the adverse effects of fibers on the workability and corrosion-inhibiting properties of concrete. One should be aware of the added initial costs associated with the use of both steel fibers and particularly latex polymers in concrete. In demanding applications, such as repair and overlay under severe climatic, load or industrial exposure, the superior longetivety and longterm performance of the material presents great potentials for enhancing the life-cycle cost of the material. One may also visualize non-traditional areas of concrete applications where the unique qualities of steel fiber reinforced latex-modified concrete present technical and cost advantages over alternative materials.

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APPENDICES

APPENDIX A

APPENDIX A

LATEX MODIFICATION EFFECTS ON CONCRETE MATERIALS INCORPORATING DIFFERENT FIBER TYPES

A.1 INTRODUCTION

This appendix summarizes the experimental work conducted to assess the latex modification effects on concrete materials reinforced with three different fiber types, namely polyethylene, polypropylene, and cellulose fiber. This study was concerned with the performance of materials in the fresh state, and also with the mechanical properties of the hardened materials.

A.2 EXPERIMENTAL PROGRAM

The experimental studies conducted with different fiber reinforced concrete systems are discussed below.

A.2.1 Latex-Modified Polyethylene Fiber Reinforced Concrete: The mix proportions used in this phase of the study are given in Table A.1. Four different mixes were investigated. The fresh mix characteristics were assessed by performing slump and Vebe tests. Hardened materials properties were assessed using compression, flexure, and impact resistance tests. For this purpose, three 152 mm diameter by 305 mm -height (6 x 12 in) cylindrical compression specimens, and three 152 mm -diameter by 73 mm -height (6 x 3 in) cylindrical impact specimens were prepared for each mix. All specimens were tested at the age of 28 days after being cured inside their molds underneath a plastic sheet for the first 24 hours, after which they were demolded and moist-cured for an additional 6 days, and then air-dried until the test age of 28 days.

Table A.1	Latex-Modified Polyethylene Fiber Reinforced Concrete: Mix
	Proportions and Fresh Mix Properties.

Mix No.	CA/B	S/B	L/B (%)	SF/B	SP/B	W/B	V _f (%)	Slump mm (in)	VeB e (sec)
1	1.5	1.5	0	0.10	0.01	0.40	0	51 (2.0)	5.0
2	1.5	1.5	0	0.10	0.01	0.45	0.5	38 (1.5)	8.5
3	1.5	1.5	10	0.10	0.01	0.40	0	44 (1.7)	5.3
4	1.5	1.5	10	0.10	0.01	0.428	0.5	76 (3.0)	9.1

where

A.2.2 Latex-Modified Polypropylene Fiber Reinforced Concrete: The mix

proportions are summarized in Table A.2. The experimental program was almost identical to that with polyethylene fibers except that only 2 cylindrical specimens were used for compression tests, and those specimens were subjected to moist curing only during the first 24 hours inside the molds, after which they were demolded and dried in ambient laboratory conditions.

Mix No.	CA/C	S/C	L/Ċ (%)	V _f (%)	W/C	Slump, mm (in)
1	2.5	1.5	0	0	0.43	152 (6.0)
2	2.5	1.5	0	0.1	0.34	140 (5.5)
3	2.5	1.5	10	0	0.31	191 (7.5)
4	2.5	1.5	10	0.1	0.32	165 (6.5)

Table A.2Latex-Modified Polypropylene Fiber Reinforced Concrete: Mix
Proportions and Fresh Mix Properties.

A.2.3 Latex-Modified Cellulose Fiber Reinforced Mortar: The mix proportions for this case are given in Table A.3. Four different mixes were studied. The fresh fibrous cement mixture were tested for:

(1) flow (ASTM C-230) at 1 min., 5 min., and 10 min. after mixing;

(2) air content of mortar (ASTM C-185); and

(3) setting time by penetration resistance.

1018) tests in the hardened state. The flexural specimens were prisms with 38.1 (1.5 in) square cross sections and total length of 152.4 mm (6 in), tested by 4-point loading on a span of 114.3 mm (4.5 in). The void content, specific gravity and water absorption of the hardened materials were also assessed using the broken flexure specimens (ASTM C-642).

For each mix, molded specimens were manufactured for flexure (ASTM C-

All the specimens were compacted through external vibration, and were kept inside their molds underneath a wet burlap covered with plastic sheets for 24 hours. They were then demolded. The mixture containing latex polymers were subject to air curing in laboratory environment (about 50 \pm 10 % RH and 22 \pm 3 °C, 72 \pm 5 °F

temperature) until the test age of 28 days. The unmodified mixtures, were subjected to a 5-day moist curing period after demolding before being air-cured until the test age of 28 days.

The cellulose fiber used in this investigation were Southern Softwood Kraft (Proctor and Gamble Cellulose). Some key properties of this cellulose fiber are presented in Table A.4. The Kraft pulp has to be disintegrated in water using a mortar mixer (at 450 revolutions per minute) and dried before being added to the mixture; otherwise a uniform dispersion of fibers inside the cementitious paste cannot be achieved conveniently.

 Table A.3
 Latex-Modified Cellulose Fiber Reinforced Mortar: Mix Proportions.

Mix No.	S/C	SA/C	L/C (%)	M _f (%)	W/C	SP/C (%)
1	0.5	0.02	0	0	0.30	
2	0.5	0.02	0	2	0.425	3.0
3	0.5	0.02	5	2	0.425	0
4	0.5	0.02	10	2	0.40	0

where,M_f=cellulose fiber (Kraft SSK pulp) mass fraction (% by
weight);S=Silica Sand Grade 100;SA=Set Accelerator; andSP=Super Plasticizer (% solids).

The wood fiber reinforced composites were manufactured in this study using a regular mortar mixer. The mixing procedure adopted is as follows:

 add cement, sand and 70 % of the water (with latex when applicable) and mix at low speed (140 RPM) for about 1 min. or until a uniform mixture is achieved;

- (2) turn the mixer to medium speed and gradually add the fibers and the remainder of water and superplasticizer into the mixture as the mixer is running at medium speed (285 RPM) over a period of 2 to 5 min. depending on the fiber content, taking care that no fiber balls are formed;
- (3) add set accelerator and mix for 1 min. at medium speed; and
- (4) stop the mixture for about 1 min., then finalize the process by mixing at high speed (450 RPM) for 2 min..

Table A.4Properties of Cellulose Fibers.

Manufacturer	Brand Name	Туре	Species	Avg. Length mm (in)	CSF
Procter & Gamble Cellulose	SSK	Kraft	Softwood	3.0 (0.120	700

A.3 EXPERIMENTAL RESULTS

This section summarizes the results obtained for the assessment of latex modification effects on concrete matrices reinforced with different fiber types, namely polyethylene, polypropylene and cellulose fibers. Discussions and conclusions regarding the latex modification effects on the mechanical properties on these fibrous cementitious composite materials are also presented.

A.3.1 <u>Latex-Modified Polyethylene Fiber Reinforced Concrete:</u> The flexural strength and toughness test results are shown in Figures A.1 (a) and A.1 (b). Both flexural strength and toughness are observed to increase upon the addition of latex

polymers. The addition of 10 % latex by weight of cement to plain concrete induces 20% and 25% increases in flexural strength and toughness of plain matrix, respectively. The joint action of polyethylene fibers and latex polymers produced about 60% and 150% increases in flexural strength and toughness of plain concrete, respectively, indicating promising potential joint actions between the two.





(a) Flexural Strength



V1-0.5%, L/c-10%

(b) Flexural Toughness

Figure A.1 Flexural Test Results on Latex-Modified Polyethylene Fiber Reinforced Concrete.

The compressive strength test results (see Figure A.2) indicate about 20% increase in the compressive strength of plain concrete in the presence of 10% latex polymers-cement ratio by weight. Polyethylene fiber reinforcement, however, caused a drop in strength of about 4%. Consequently, the increase in compressive strength due to both latex modification and polyethylene fiber reinforcement was only about 2.5% over that of plain concrete.



Figure A.2 Compressive Strength Test Results on Latex-Modified Polyethylene Fiber Reinforced Concrete.

The impact resistance test results, presented in Figure A.3, showed increases in impact resistance due to latex modification and/or steel fiber reinforcement; with the largest increase, about 300%, obtained by the joint action of polyethylene fibers and latex modification. Fiber reinforcement, however, was found to contribute significantly more to the impact resistance than latex modification (about 260% Vs. 30% increases, respectively).



V1-0.5%, L/c-10%

Figure A.3 Impact Resistance Test Results for Latex-Modified Polyethylene Fiber Reinforced Concrete.

A.3.2 Latex-Modified Polypropylene Fiber Reinforced Concrete: The trends obtained here are very similar to the ones obtained in the case of polyethylene fibers as far as flexural strength test results are concerned (see Figure A.4(a).). For flexural toughness, however, even though each of the latex modification (L/c = 10%) and polypropylene fiber reinforcement ($V_f = 0.1\%$) effects increased the flexural toughness of plain concrete, their combined action caused a drop in flexural toughness as compared to that of the unmodified fiber reinforced concrete (see Figure A.4(b)). More data is needed to confirm the validity of this observation.

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(a) Flexural Strength

(b) Flexural Toughness

Figure A.4 Flexure Test Results for Latex-Modified Polypropylene Fiber Reinforced Concrete.

The compressive strength test results (see Figure A.5) show that the use of latex polymers (L/c = 10%) along with polypropylene fibers ($V_f = 0.1\%$) resulted in

about 18% decrease in compressive strength of the unmodified fiber reinforced case.



Figure A.5 Compressive Strength Test Results for Latex-Modified Polypropylene Fiber Reinforced Concrete.

The combined use of latex polymers and polypropylene fibers in concrete matrices did, however, yield improvements in impact resistance of the material (see Figure A.6). An increase of about 250% in impact resistance of latex-modified polypropylene fiber reinforced concrete was obtained (see Figure A.6).



Figure A.6 Impact Resistance Test Results for Latex-Modified Polypropylene Fiber Reinforced Concrete.

A.3.3 Latex-Modified Cellulose Fiber Reinforced Mortar

A.3.3.1 Fresh Mix Properties: The fresh mix properties obtained in this

experimental study are summarized in Table A.4.

Mix	Air Content	Specific	Initial Set	Final Set
	(%)	Gravity	(min)	(min)
Plain	21.07	2.01	186	210
$M_f=2\%$, L/c=0%	22.50	1.77	233	293
$M_f=2\%$, L/c=5%	22.62	1.73	260	330
$M_f=2\%$, L/c=10%	22.40	1.79	290	355

Table A.4 Fresh Mix Properties.

These fresh mix properties indicate clearly that setting time of cementitious mixtures is increased as cellulose fibers and/or latex polymers are added to the matrix. The initial setting time increased by as much as 25, 40, and 56%, respectively, for mixes containing 2% mass fraction of Kraft fibers, 2% mass fraction of Kraft fiber with latex-cement ratio of 5%, and 2% mass fraction of Kraft fibers with a latex-cement ratio equal to 10%. These results could be attributed to the fact that Kraft pulps absorb the water available in fresh mix and thus make it difficult for cement to access the water for hydration purposes, and also latex polymers cause the formation of polymer films inside the paste which interfere with moisture movements during cement hydration, thus making water less accessible to cement particles; formation of a polymer film at the cement surface may prevent moisture loss from the specimen.

The fresh mix air content is observed to increase (see Table A.4) when cellulose fibers are added. this increase in fresh mix air content was about 7% for Kraft pulp additions at 2% mass fraction. This increase in air content could be attributed to the difficulty in compacting cement composites incorporating higher fiber contents, which leads to increased entrapped air content. In addition, the increase in latex content beyond a certain level, at a constant fiber content, tends to reduce the air content, possibly due to the corresponding reductions in water content.

Cellulose fibers are also observed in Table A.4 to reduce the unit weight (or analogously, specific gravity) of fresh cement-based materials. The reduction in unit weight was about 125 when 2% mass fraction of Kraft pulp was added. This is partly due to the fact that the air content in cementitious matrices tends to increase in the presence of fibers, as discussed earlier.

A.3.3.2 Hardened Material Properties: The average values of the hardened material properties obtained in this experimental investigation are given in Table A.5.

Mix	Void Conte nt (%)	Specific Gravity	Water Absorption (%)	Flexural Strength (ksi)	Flexural Toughness (lb.in)
Plain $M_r = 2\%$, L/c=0%	22.75	2.00	11.07	0.5694	0.3542
	24.18	1.75	13.66	0.9720	1.4981
$M_t=2\%$, L/c=5%	22.10	1.75	13.20	0.8348	5.5200
$M_t=2\%$, L/c=10%	21.80	1.82	12.83		8.3750

Table A.5Hardened Material Properties.

The hardened material void content is seen to increase in the presence of cellulose fibers, with an increase over plain cement-based materials of about 6% for Kraft pulp at 2% fiber mass content. Nevertheless, when this cellulose fiber reinforced matrix is modified with latex polymers, the hardened material void content is seen to decrease by about 8.6 and 9.8% at latex-cement ratios of 5 and 10%, respectively.

The water absorption capacity of cement-based materials is observed in Table

A.5 to increase with increasing fiber mass fraction (about 23% increase at 2% mass fraction) and decrease with increasing latex-cement ratios (about 3.4 and 6.1% decreases corresponding to 5 and 10% latex-cement ratios, respectively).

The specific gravity of hardened cement-based materials is observed in table A.5 to decrease with increasing cellulose fiber content, and then increase with latex modification of fibrous mixtures. At 2% fiber mass content, Kraft fiber reinforced composites showed a reduction in specific gravity of about 13% below that of plain matrix, and an increase of about 4% over that of fibrous matrix.

The hardened material flexural test results are shown in Figures A.7 (a) and (b). Both flexural strength and toughness can be seen to increase at 2% fiber content (about 71 and 323%, respectively). When the fibrous mixture (at 2% fiber content) is modified with 5 and 10% latex-cement ratios, the flexural toughness is observed to continuously increase (by about 268 and 459%, respectively). The flexural strength of the fibrous mixture, however, is seen to decrease at latex-cement ratio of 5% (about 14%) and then to increase at the latex-cement ratio of 10% (about 25% over that of unmodified fibrous cement matrix).



(a) Flexural Strength

(b) Flexural Toughness

Figure A.7 Flexural Test Results for Latex-Modified Cellulose Fiber Reinforced Mortar.

A.4 SUMMARY AND CONCLUSIONS

From the results of this study regarding the effects of latex modification on concrete matrices reinforced with different fiber types it was concluded that:

1. Latex modification of polyethylene fiber reinforced concrete increases both the flexural strength and toughness of the material. About 9 and 5.4% increases in flexural strength and toughness, respectively, were obtained at a fiber volume fraction $V_f=0.5\%$ and a latex-cement ratio of 10%.

2. Latex modification was found to increase the compressive strength of plain and polyethylene fiber reinforced concretes; polyethylene fiber reinforcement, however, was found to decrease the compressive strength of the material.

3. Both latex modification and polyethylene fiber reinforcement increase the impact resistance of the material, with an increase of 300% over the impact resistance of plain concrete obtained through their joint action.

4. Even though the separate action of latex modification (L/c=10%) and polypropylene fiber reinforcement ($V_f=0.1\%$) increases the flexural toughness of plain concrete, their combined action caused a drop in flexural toughness as compared to that of the unmodified polypropylene fiber reinforced concrete.

5. Latex modification (L/c=10%) of polypropylene fiber reinforced concrete resulted in about 18% decrease in compressive strength.

6. The combined action of latex modification and polypropylene fiber reinforcement caused an increase in the impact resistance of the material.

7. Both cellulose fiber reinforcement and latex modification increase the setting time of cement mortars.

8. Cellulose fibers increase the fresh mix air content while latex modification beyond a certain level, tends to reduce the air content.

9. The water absorption capacity of cement-based materials increased with increasing fiber content (by about 23% increase at 2% cellulose fiber mass fraction) and decreased with increasing latex-cement ratios (by about 3.4 and 6.1% corresponding to 5 and 10% latex-cement ratios, respectively).

10. Both flexural strength and toughness were found to increase at 2% cellulose fiber content (about 71 and 323%, respectively).

11. Latex modification of cellulose fiber reinforced mixtures increased the flexural toughness; however, the flexural strength was decreased at latex-cement ratio of 5% (by about 14%) and then increased at latex-cement ratio of 10%

APPENDIX B

APPENDIX B

INVESTIGATION OF A NEW LATEX FOR APPLICATION TO CONCRETE

B.1 INTRODUCTION

The ultimate goal of this work is to evaluate concrete materials modified with a new experimental latex (it will be referred to as "new latex" or "XU latex" in the Figures) containing antifoam. The tasks performed are concerned with the assessment of the following properties of the "new latex" latex-modified concrete:

- 1- Fresh mix properties: slump, air content and setting time;
- 2- Flexural strength and toughness;
- 3- Compressive strength;
- 4- Tensile bond strength;
- 5- Impact strength;
- 6- Abrasion resistance; and
- 7- Chloride permeability;

For comparison purposes, the same tests were performed with the Modifier A latex under similar conditions. This report presents the test results along with statistical analyses and discussions.

B.2 EXPERIMENTAL PROGRAM

Table B.1 presents the mix proportions used in this investigation. The cement in all these mixes was regular type I, the coarse aggregate was crushed limestone with a maximum particle size of 1/2 in, and the fine aggregate was natural sand. Both coarse and fine aggregates satisfied the ASTM C-33 gradation requirements.

The water content was adjusted in different mixtures, depending on the latex type and content, so as to achieve a desirable level of workability represented by a slump of 5.5 to 8.5 in. The air content was measured without adding any air-entraining admixture.

The latex-modified mixtures were manufactured in laboratory in a rotary-type drum mixer.

The experimental program designed for this investigation is shown in Table B.2; two different curing conditions (continuous moist curing and one-day moist curing followed by air drying) were considered in this investigation.

			w/c
Modifier A	L/c*	10	0.34
		15	0.32
"New Latex"	L/c	10	0.35
		15	0.33

Table B.1 Mix Proportions. $(s/c=2.6, g/c=1.7)^*$

*: s=sand; g=gravel; c=cement; w=water, and L=latex; all ratios by weight.

		Latex/Cement Ratio (%)				
		10 15				
	Cur	ing Type	Curing Type			
	Continuous Moist	1-Day Moist	Continuous Moist	1-Day Moist		
Modifier A	*	*	*	*		
"New Latex"	*	*	*	*		

Table B.2Experimental Design.

where, * represents 2 compression, 3 flexure, 3 permeability, 2 bond, 2 abrasion, and 1 setting time tests with replications done with one batch of concrete.

For each mix, the following tests were performed:

1. Fresh Mix Properties: Slump (ASTM C-143), air content (ASTM C-231)

and time of setting by penetration resistance (ASTM C-403).

2. Flexural Performance: Six 4 x 4 x 14 in. prismatic specimens were prepared for each mix; three were cured at 100% R.H. and the remaining three in ambient laboratory environment (following 24 hours of moist curing inside their molds under wet burlap covered with plastic sheet). These flexural specimens were tested following the ASTM C-1018 and the Japanese Concrete Institute (JCI-SF) test procedures by four-point loading on a span of 381 mm (12 in). A computerized data acquisition system was used for load and deflection measurements and also for the processing of test results.

3. Compressive Strength: Four 6 in. diameter by 381 mm (12 in) height cylindrical specimens were cast for each replication from each mix. Two were cured

at 100% R.H., and the other two were cured in ambient laboratory environment after the initial 24 hours of moist curing. The compression specimens were tested following the ASTM C-39 test procedures.

4. Tensile Bond Strength: Four specimens for each replication were prepared from each mix. Two were cured at 100% R.H. room, and the other two in ambient laboratory environment after 24 hours of moist curing.

5. Impact Strength: Six cylindrical specimens, 6 in. in diameter and 2.5 in. in height, were prepared from each of the mixes presented in Table B.1. Three specimens were cured at 100% R.H., and the rest in ambient laboratory environment (following 24 hours of moist curing inside their molds). The impact resistance test performed by repeatedly dropping a 10 lb. hammer on a hard steel ball supported on the cylindrical specimen from a height of 18 in. The number of blows required to cause the first visible crack and the ultimate failure are recorded. Ultimate failure is assumed to occur when the cracks open so far that the pieces of concrete are touching three of the four positioning lugs on the base plate.

6. Abrasion Resistance: The abrasion test specimens were 6 in. cubes conforming to ASTM C-779 procedures. Three specimens were cast for each of the mixes of Table B.1, and were cured in air after 24 hours of moist curing inside their molds.

7. Permeability: Rapid assessment of the chloride permeability of concrete was made in this investigation through the performance of the AASHTO T277-83I test procedure developed by the Construction Technology Laboratories (CTL). Six cylindrical specimens 4 in. in diameter by 2 in. in thickness were prepared for each of the mixes of Table B.1. Three specimens were cured at 100% R.H., and the other three were cured in ambient laboratory temperature following 24 hours of moist curing inside the molds.

B.3 EXPERIMENTAL RESULTS AND DISCUSSIONS

B.3.1. Fresh Mix Properties: Test results on slump, air content and time of setting of the fresh mixtures of Table B.1 are shown in Table B.3.

	L/c (%)	Slump	Air	Time of Se	tting (hrs.)
		mm (in)	Content (%)	Initial	Final
Modifier A	10	152 (6)	7	5	7
	15	152 (6)	8	4.8	7
"New	10	165 (6.5)	7	4.4	5.8
Latex "	15	165 (6.5)	7.5	6.3	7.9

Table B.3Fresh Mix Properties.

Figure B.1 also shows the mean values and the 95% confidence intervals of the setting time test results. Increasing the latex content from 10 to 15% by weight of cement seems to increase both the initial and final times of setting of concrete materials modified with the "new latex". In the case of concretes modified with Modifier A latex, a corresponding increase in latex content tends to slightly decrease the initial time of setting and has no apparent effect on the final time of setting. Statistical pairwise comparisons by separation of means revealed that there is no significant difference between setting times for 10% and 15% latex contents or

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(a)

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different latex types at 95% confidence level.





(b) Final Time of Setting

Figure B.1 Time of Setting Test Results.

B.3.2. Flexural Performance: The flexural strength test results for the mixes shown in Table B.1 are presented in Table B.4 and Figure B.2. It can be seen from these results that, when air cured, concretes modified with Modifier A exhibit 32 and 36% higher flexural strengths than those modified with the "new latex" at 10 and 15% latex contents, respectively. Continuous moist curing, however, reduced the difference in flexural strengths obtained with the two latexes; Modifier A produced only 23 and 2% higher flexural strengths than the "new latex" after moist curing at 10 and 15% latex contents, respectively.

Factorial analysis of variance of flexural strength test results confirmed that the latex type is a significant factor in determining the flexural strength of latex-modified concrete at 95% level of confidence. This analysis also revealed that the interaction between latex type and the curing method used is also significant at 95% level of confidence. Therefore, with continuous moist curing of the "new latex" latex-modified concrete, flexural strength gets closer to that obtained with Modifier A latex-modified concrete. It can then be concluded that, with proper moist curing of the "new latex" latex-modified concrete, one may achieve flexural strengths comparable to those of concretes modified with Modifier A latex. Pairwise comparison by separation of means indicated that there is no significant difference between both latex types for 15% latex content, at 95% confidence level.



(a) Air Curing



(b) Moist Curing

Figure B.2 Flexural Strength Test Results.

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("New	Latex"	Modifier A		
	L/c = 10%	L/c = 15%	L/c = 10%	L/c = 15%	
Air	0.5589	0.5902	0.8678	0.9375	
Curing	0.5620	0.5512	0.8918	0.8760	
Moist	0.5918	0.7253	0.7904	0.6967	
Curing	0.6110	0.6898	0.8665	0.8063	

Table B.4Flexural Strength Test Results (ksi).

B.3.3 Compressive Strength: Table B.5 and Figure B.3 show the compressive strength test results. It can be seen from these results that concrete materials modified with the experimental "new latex" present compressive strengths that are larger than those obtained with concrete modified with Modifier A in both curing conditions. When concrete was air-cured, on the average, "new latex" produced 22 and 36% higher compressive strengths than Modifier A at latex-cement ratios of 10 and 15%, respectively. Continuous moist curing seemed to reduce the compressive strengths for both latex types and contents.

Factorial analysis of variance of compressive strength test results revealed that the latex type but not curing condition had significant effects on the compressive strength of latex-modified concrete at 95% level of confidence. Interaction of latex type and curing were not significant at this confidence level. Pairwise comparisons by separation of means revealed that there is a significant difference between compressive strengths obtained with the two latex types at the 95% confidence level.

	"New Latex"		Modifier A		
	L/c=10%	L/c=15%	L/c=10%	L/c=15%	
Air Curing	6.0148	5.3211	5.4523	4.2432	
	6.7362	6.7544	4.9756	4.6378	
Moist Curing	5.7190	5.0018	4.9986	4.5580	
	5.8235	6.1133	3.8697	4.2090	







(b) Moist Curing

Figure B.3 Compressive Strength Test Results.

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Table B.	Compressive	Strength	Test	Results	(ksi)
Table D.J	Compressive	Sucingui	Test	Results	(121)

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Ai Cu Ma **B.3.4 Tensile Bond Strength:** The tensile bond strength test results are presented in Table B.6 and Figure B.4. With continuous moist curing, concretes modified with the "new latex" exhibited 6 and 5% higher tensile bond strengths than those modified with Modifier A latex at latex-cement ratios of 10 and 15%, respectively.

When air-cured, increasing the latex content from 10 to 15% decreased the tensile bond strength of concretes modified with the "new latex" (by about 13%) but increased the tensile bond strength of concretes modified with Modifier A latex (by about 3%).

Factorial analysis of variance revealed that only the curing factor was significant in determining the tensile bond strength of latex-modified concrete at the 95% confidence level. In addition, pairwise comparisons by separation of means revealed that there are no significant differences between the tensile bond strengths corresponding to the two latex types at 95% confidence level.

	"New Latex"		Modifier A		
	L/c=10%	L/c=15%	L/c=10%	L/c=15%	
Air	439.2	399.6	399.6	449.1	
Curing	507.4	436.4	483.8	459.6	
Moist	396.8	415.8	364.9	381.2	
Curing	407.4	407.4	392.5	401.0	

Table B.6Tensile Bond Strength Test Results (psi).

(b) }

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(a) ,






(b) Moist Curing

Figure B.4 Tensile Bond Strength Test Results.

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B.3.5 Impact Resistance: The impact resistance test results are summarized in Table B.7 and Figure B.5. Large Variations in impact resistance test results prevented us from making any statistically sound conclusions. When concrete was air-cured, on the average, Modifier A latex had higher impact resistance than the "new latex" by about 650% and 130% at latex-cement ratios of 10 and 15%, respectively. Continuous moist curing seemed to increase the impact resistance of the "new latex". On the average, following continuous moist curing, the "new latex" had about 300% higher and 21% lower impact resistances than Modifier A at latex-cement ratios of 10 and 15%, respectively.

	"New Latex"		Modifier A		
	L/c=10%	L/c=15%	L/c=10%	L/c=15%	
Air Curing	64	100	665	355	
	113	109	672	135	
Moist Curing	124	85	85	220	
	444	155	55	70	

 Table B.7
 Impact Resistance Test Results (no. of blows).

(b)

(2

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(a) Air Curing



(b) Moist Curing

Figure B.5 Impact Resistance Test Results.

Flexural strength of concrete depends on both toughness of the material and its compressive strength. Increasing the toughness can therefore help increase the flexural strength even if the compressive strength is low. Moist curing of the "new latex" does not do much to its compressive strength but increases its impact resistance (representing toughness) thus helping with the flexural strength. Hence, the flexural strength of the material is increased with moist curing not because of the increase in compressive strength, but because the impact resistance and thus the toughness of the material increases.

B.3.6 Abrasion Resistance: The abrasion resistance test results are summarized in Table B.8 and Figure B.6. It can be seen that, at a latex-cement ratio of 10%, concretes modified with the "new latex" are slightly more abrasion resistant. However, this behavior is reversed at a latex-cement ratio of 15%.

Factorial analysis of variance did not reveal the significance of any factor or interaction effects in deciding abrasion resistance. Pairwise comparisons by separation of means confirmed that there are no statistically significant differences between the abrasion resistance obtained with the two latex types at 95% confidence level. Table B.8 Abrasion Resistance Test Results (grams).

"New Latex"		Modifier A		
L/c = 10%	L/c=15%	L/c=10%	L/c=15%	
1.8	1.4	1.5	2.47	
1.37	2.0	1.87	0.7	



Figure B.6 Abrasion Resistance Test Results.

B.3.7 Chloride Permeability: Table B.9 and Figure B.7 show the chloride permeability test results. While Modifier A performs better than the "new latex" in dry curing conditions, moist curing influences the "new latex" effects favorably; "new latex" at 15% latex content performs slightly better than the Modifier A after moist curing.

Factorial analysis of variance of test results indicated that both latex type and content have significant effects on the chloride permeability of concrete at 95% confidence level. Curing, however, did not have a significant effect. Pairwise comparisons by separation of means showed that there is no significant difference between Modifier A and "new latex" when moist cured at 95% confidence level.

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When air-cured, a significant difference between the two latexes was obtained at 15% latex-cement ratio at 95% confidence level. Through proper moist curing, the "new latex" is capable of producing chloride permeability levels comparable to those obtained with the Modifier A latex.

	"New Latex"		Modifier A		
	L/c = 10%	L/c=15%	L/c=10%	L/c=15%	
Air Curing	1597	1002	1421	841	
	1610	1516	1096	845	
Moist Curing	1393	997	1301	1186	
	1481	1106	1325	1043	

 Table B.9
 Chloride Permeability Test Results (Coulombs).

(a)

(b) N Figu



(a) Air Curing



(b) Moist Curing

Figure B.7 Chloride Permeability Test Results.

B.4 SUMMARY AND CONCLUSIONS

This experimental investigation was performed to evaluate concrete materials modified with the experimental latex the "new latex" and compare them to concretes modified with the Modifier A latex. A $2 \times 2 \times 2$ factorial experimental design was followed with the main variables being the latex type and content and the curing technique adopted. Factorial analyses of variance (ANOVA) and statistical pairwise comparisons by separation of means were the statistical tools used in this investigation to analyze the obtained test results. The results of the tests performed in this study indicated that:

1. Latex type and content did not have statistically significant effects on setting times at 95% level of confidence.

2. Latex type has a significant effect on the flexural strength of concrete at 95% level of confidence; a significant interaction between the latex type and the curing method was confirmed at the 95% level of confidence. With proper moist curing of the "new latex"-modified concrete, one may achieve flexural strengths comparable to those of concretes modified with Modifier A latex at 15% latex content. Air-drying after a short period of moist curing is not suitable for "new latex"-modified concrete.

3. Latex type but not curing condition had significant effects on the compressive strength of latex-modified concrete at 95% level of confidence.
"new latex"-modified concrete produced compressive strengths that were 22 and 36% higher than those from Modifier A-modified concretes at 10 and 15%

latex contents, respectively.

4. Only the curing method factor was found to be significant in determining the tensile bond strength of latex-modified concrete at the 95% level of confidence. The two latex types produced statistically identical bond strengths.
5. Large variations in impact resistance test results, which are inherent to the test method used in this investigation, prevented us from making any statistically sound conclusions regarding the impact resistance of latex-modified concrete.

6. No statistically significant differences were found between the abrasion resistance obtained with the two latex types at different dosages at the 95% confidence level.

7. There is no significant difference between Modifier A and "new latex", at 95% level of confidence, as far as their effects on the chloride permeability of moist-cured concrete are concerned. Latex type has an effect on the chloride permeability in air-cured condition; however, through proper moist curing, the "new latex" is capable of producing permeability levels comparable to those obtained with the Modifier A latex.

APPENDIX C

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APPENDIX C

SIZE EFFECT IN CONCRETE MATERIALS

C.1 INTRODUCTION

In engineering design, there has always existed a difficulty in that the results of tests on small laboratory specimens can not quite accurately predict the behavior of large structural parts, particularly when the two extremes of scale differ by a wide margin.

"Size effects" is the change, usually an increase, in strength that occurs when the specimen size is decreased. Understanding of the structural size effect is important for correct interpretation of test data. Most laboratory tests are carried out on test specimens with standard dimensions from which generalizations must be made for actual structures.

This size effect phenomenon is not yet fully understood. Many theories for explaining size effects have been developed over time. This literature review summarizes the previous work on this subject with emphasis on the major theories developed, factors influencing size effect in concrete, and the experimental work performed.

C.2 FACTORS INFLUENCING SIZE EFFECTS

When investigating the behavior of concrete, it is important to recognize factors that contribute directly to the observed changes in properties with size so that one could take the necessary precautions in design and interpretation of results.

C.2.1 Compaction, Density, and Loss of Water

Compaction is an important variable affecting concrete strength; smaller specimens tend to achieve better compaction, higher density, and thus higher strength.⁴⁸ This is particularly true when standard compaction procedures are followed, involving a given period of vibration or specific number of tamping. Larger specimens will have more internal voids and entrapped air. The size effect due to this factor will be minimized when uniform compaction is achieved.

Water loss from specimens during casting can vary with size and cause different qualities in the cast material. A controlled humidity room during casting and water-tight molds are needed to minimize this source of variability and size effects.

C.2.2 Curing and Drying

Since the surface-volume ratio increases with decrease in specimen size, curing of two specimens of different size will take place at different rates. The strength of the material will vary from the surface of the specimen to its center, since hydration may not be uniform throughout the specimen at the time of testing.⁷⁴

C.2.3 Strain Rate

Higher loading rates lead to higher strengths. In a given testing machine, with the rate of cross head movement kept constant, smaller specimens will experience higher strain rates. As the specimen size is decreased, the cross head movement rate should be reduced accordingly. However, with very small specimens it is not always possible to achieve the required low rates of cross head movement with existing testing equipment, which contributes to their apparent increased strength.⁷⁴

C.2.4 The State of Stress

The state of stress, such as compression, tension and flexure, influences the strength of the specimen. The strength of compressive specimens depends on the accuracy of the loaded ends, and on parallelism, if rotating heads are not used. It is possible to achieve a higher level of capping accuracy in smaller cylinders, and therefore will have higher strength increases with an increase in strain gradient. Thus, smaller beams are expected to show higher flexural strength, as a result of the higher strain gradient.⁹⁹

C.2.5 Testing Machine and Loading Plates

Stiff end plates tend to apply uniform strain conditions to the specimen and result in higher strength than thinner plates, which tend to lead to a state of uniform stress. Also, end plates restrain lateral movements of specimens and induce lateral stresses at both ends. The higher the lateral restraint, the higher the compressive strength will be.⁷⁵

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C.3 THEORETICAL STUDIES ON SIZE EFFECT

The assumptions of random heterogeneity of materials have led to theoretical explanations of size effects based on concepts of probability, the basic philosophy being that the failure in heterogeneous materials is a statistical phenomenon. Thus, the larger the volume, the greater the chances for failure which will result in lower strength. Generally, specimens of smaller size are observed to have higher strengths. Also, the scatter in strength is generally greater in the smaller specimens. The basic approach in statistical theories of strength is to evolve a statistical distribution function that adequately characterizes the random heterogeneity of materials and the variation (scatter) in their strength. In effect, it identifies admissible forms of distribution functions with suitable parameters that reflect accurately the true material behavior and a realistic mechanism of failure for which the distribution function is applied.

C.3.1 Statistical Theories of Size Effect

C.3.1.1 Weakest Link Theory of Weibull

In this theory, the presence of a single severe defect in any of the constituent elements is adequate to cause failure of the total material. Consequently, the overall strength of a specimen subjected to uniform stress is determined by the strength of the weakest element present.^{74, 75} Weibull regarded the specimen as an ensemble of a very large number of primary elements. He considered the failure of total material the same as that of any one of the primary elements or of the weakest link. The larger the size, the greater the number of primary elements and the greater the

probability of finding a lower strength weakest link. In the case of a beam, the weakest link theory implies that the strength will depend on the stressed volume and hence on the length, width and depth of beam. According to Weibull, the strengths S_1 and S_2 of two different size specimens with stressed volumes V_1 and V_2 can be expressed as:

$$\frac{S_1}{S_2} = \left(\frac{V_2}{V_1}\right)^{\frac{1}{m}}$$
(1)

where, m = constant related to properties of the material. The smaller the value of m, the larger will be the size effect, with an infinitely large m indicating no size effect. Experimental data of mortar beam shows good agreement with a value of m=10.

C.3.1.2 Classical Theory of "Bundled Strength"

In this concept, the strength is not only determined by that of the weakest element but is dependent on the strength of the elements in the neighborhood. In this model, the specimen is assumed to be made up of parallel fibers (elements), and in such a situation the gross strength as failure is influenced by the strength of all constituent fibers. The concept of this theory lies in the assumption that the bulk specimen consists of parallel elements and that the instability or failure of one element will not lead to total fracture, rather it will be stopped before it propagates from local to bulk scale.^{30, 96} This means that the weakest volume element containing the failure is surrounded by elements of such high local strength that the stress carried by the weakest element prior to its failure can indeed be transferred to the elements. The

fracture process starts at the weakest point in the bundle, but, contrary to the weakest link model, it does not necessarily propagate unless, in a bundle of total filaments (n) of (weakest link) strengths σ_n , σ_{n-1} , ..., σ_2 , σ_1 arranged in the order of their consecutive failure, the following conditions are satisfied:

$$0 \leq \sigma_{n} \leq \frac{S}{nA} = s_{n}$$

$$\sigma_{n} \leq \sigma_{n-1} \leq \frac{S}{(n-1)A} = s_{n-1}$$

$$\vdots$$

$$\sigma_{3} \leq \sigma_{2} \leq \frac{S}{2A} = s_{2}$$

$$\sigma_{2} \leq \sigma_{1} \leq \frac{S}{A} = s_{1}$$
(2)

where, $s_i = stress$ in individual element i; and S = total applied force.

C.3.1.3 Nielsen's "Surface Theory"

Here it is assumed that failure of a beam is determined by the thin surface layer of the tension zone, and if this surface layer is assumed to be brittle, Weibull's theory may be applied to the tensioned surface. This implies that the flexural strength would decrease with increasing length and width but should be independent of depth.⁵⁵

C.3.1.4 Tucker's Strength Summation Theory

This theory is based on the assumption that the strength of a specimen is equal to the sum of strengths contributed by component parts, or elements. The specimen is assumed to have parallel link assembly. Thus, the flexural strength of a beam specimen decreases with increasing length and increasing depth, as governed by the weakest-link theory, but is independent of the width.^{91, 92}

C.3.2 Nonlinear Fracture Mechanics-Based Theories

The two common failure modes are:

- 1. Elastic or Plastic mode (horizontal line of Figures C.1 and C.2)
- 2. Linear Elastic Fracture Mechanics (lines of Figures C.1 and C.2 with slope = -1/2)

The physical evidence derived by means of experiments tells us that an interaction does exist between these two failure modes. This interaction is what causes the "transition zone". This fact introduces the necessity of considering non-linear fracture mechanics (NLFM). The failure phenomena are dependent on the material properties (K_{IC} , σ_y), on the size h, on the geometry including relative crack depth and boundary conditions. In this sense, failure can be considered a function of the material properties and of the structural shape and loading condition. Then failure is not only a result of material behavior but also of the type of structure: this is an evident "structural effect."⁷²

Concrete and mortar are heterogeneous materials which exhibit, during failure, dominant cracks that can't be adequately modelled by the classical idealization of line crack with a sharp tip. The "process zone", in which high gradient ultra-elastic deformation phenomena are present, is for metals localized in a region that is usually small compared to the usual body dimension utilized for test specimens. In concrete materials, the fracture "process zone" occurs over a relatively large region compared to the dimension of usual test specimens; in many cases the zone is of the same order of magnitude as the size of the specimen itself. Therefore, we need to take into account the phenomena occurring in the process zone. For concrete materials, these phenomena may be described by a stress-strain softening relation, or by a stressdisplacement relation with softening.

The leading parameter during fracture is "the amount of energy consumed for the creation of unitary new external surfaces" in a stressed body. During fracture (or crack propagation) this energy is furnished by the work of external forces and by the release of elastic energy stored in the body (structure). The latter depends on the loads, the elastic parameters and the geometry of the structure.

The concrete-like materials are supposed to behave according to the 3 models for strain localization: crack line model, crack band model, or arbitrary shaped damaged zone.

To predict the behavior in the non-linear range where linear elastic fracture mechanics (LEFM) can not be applied, different approaches have been proposed by Hillerborg (1986) and Bazant(1984, 1987). Their models do not include a global fracture criterion and are able to describe a progressive fracturing of the material without stress singularities. They are called Progressive Fracture Models, and they permit a description of a smooth transition from the continuous medium to the discontinuous (fully fractured) one, and they rely on two basic concepts: Strain Softening and Strain Localization.

C.3.2.1 Hillerborg's Cohesive/Fictitious Crack Model: In this model strain localization is modelled as a crack. In addition, the bulk behavior is generally assumed to be isotropic linear elastic, hence defined by an elastic modulus, E, and a Poisson's ratio, v. When the maximum principal stress reaches the tensile strength f_t , fracture starts as a cohesive (stress transferring) crack normal to the maximum tensile

stress direction. Once the cohesive crack has formed, the stress transferred through the crack faces is assumed to be dependent upon the relative displacement of the crack faces. For pure opening, the transferred stress, σ , is normal to the crack faces and is usually assumed to depend only on the evolution of (normal) crack opening w.

Hillerborg obtained the diagrams of Figure C.1.^{34, 35, 36, 72} His approach shows the possibility of describing the "transition" phenomena as a function of a nondimensional parameter represented by the ratio: d/l_{ch} where:

$$l_{ch} = \frac{E.G_F}{f_t^2}$$
(3)

d = h = the "size" of element $G_F =$ mean energy per unit area of formed crack.³⁶



Figure C.1 Theoretical Variation in Bending Strength of a Notched Beam.

The figure shows the strong reduction of "maximum load" derived from the model in comparison with the value one can obtain applying Linear Elastic Fracture Mechanics (LEFM) or Limit Analysis (LA) is evident.

To represent this aspect, the (energetic) brittleness number can also be utilized:

$$s_E = \frac{G_F}{\sigma_{\mu}.h} = brittleness number$$
(4)

where, $\sigma_u = f_t$ = ultimate tensile stress, and h="characteristic size"

Considering,

$$G_{\mathcal{K}} = \left(\frac{K_{\mathcal{K}}^2}{E}\right) = G_F \tag{5}$$

From Equation (4), we get:

$$s_E = \frac{K_{IC}^2}{E.\sigma_u.h} = \epsilon_u.s^2$$
(6)

where, $\epsilon_u = \sigma_u / E$, and h = d

Hence,

$$\frac{d}{l_{ch}} = \frac{d \cdot f_t^2}{K_{IC}^2} = \frac{1}{s^2}$$
(7)

C.3.2.2 Bazant's Crack Band Model: This model represents theories

modelling the strain localization as a softening band of constant thickness. It assumes that for concrete, the bulk behavior is generally assumed to be isotropic linear elastic, hence defined by an elastic modulus, E, and a Poisson's ratio, v. The behavior at a

point is usually (for isotropic materials) assumed to be elastic until the moment where the maximum principal stress reaches the tensile strength f_t . At this moment, the fracture starts as an array of densely and uniformly distributed cracks, normal to the maximum tensile stress direction, and distributed over the band thickness h_e . Once the crack band has formed, the crack orientation remains fixed and it is assumed that the stress tensor and the strain tensor remain uniform through the band thickness. Stresses and strains are related through a constitutive equation displaying softening.¹¹⁻ 18, 72

The model of fracture suggested by Bazant can cover the situation in which non localized cracking can be recognized. This happens for certain types of thermal and shrinkage stresses, particularly when there is a rapid drop to compression in front of cracking zone and also when concrete is bonded to some elastic elements such as reinforcement. Bazant's "crack band" model ensues from two statements: the total potential energy release U_a caused by fracture is a function of both 1) the length, a, of fracture zone and 2) the volume of the cracked zone, n.d_a.a, where d_a=maximum aggregate size, and n= constant.

Energy is expressed in terms of the non-dimensional parameters: $\alpha_1 = a/d$ and $\alpha_2 = n.d_a.a/d^2$, where d=characteristic dimension.

Now, according to Bazant, the energy U_a is expressed as:

$$U_a = A(F, E, b, d) \cdot f(\alpha_1, \alpha_2, \xi_i)$$
(8)

where f() depends on the actual geometrical shape and boundary condition of the structural element, but not on the size d.
A() depends on the loads: F, elastic parameters: E, and on the size: b, d; ξ_i are geometrical parameters.

Crack growth instability occurs when:

$$\frac{\partial U_a}{\partial a} = G_F \cdot b \tag{9}$$

where b is the thickness.

This yields:

$$\sigma_N = B \cdot f_t' \cdot [1 + \beta]^{-1/2} ; \quad \beta = \frac{\lambda}{\lambda_0}$$
 (10)

Where,

e, $\sigma_N =$ nominal strength at failure = P/b.d; P = maximum load; b = specimen thickness; d=h= characteristic dimension of size of specimen; f'_i =strength parameter; $\lambda = d/d_a$; d_a=maximum aggregate size; B, λ_0 = two empirical constants.

Figure C.2 gives a plot of the Bazant's size effect equation. From both Figure C.2 and Equation (10), one can see that for very small specimens, d is very small, so the term in brackets tends toward 1, thus σ_N is proportional to the material strength (or yield limit). This represents the failure condition of plastic Limit Analysis (LA) characterized by no size effect. It is represented by a straight horizontal line in the plot of $Log\sigma_N$ versus Logd. The other limiting case is when d is very large. In that case, the number 1 inside the brackets of equation (10) is negligible compared to β , and then σ_N is proportional to d^{-1/2}. This case represents the strongest possible size effect and corresponds to the classical Linear Elastic Fracture Mechanics (LEFM). The plot of the size effect law (i.e., equation (10)) is basically a gradual transition between the above two limiting cases: Plastic Limit Analysis (LA) for very small elements and Linear Elastic Fracture Mechanics (LEFM) for very large structural elements.



Figure C.2 Size Effect According to Strength Criteria and Linear or Nonlinear Fracture Mechanics.

C.4 EXPERIMENTAL PROGRAM

The experimental program for this study involved flexure and compression tests on specimens with different sizes. The experimental design program is shown in the following Tables.

	Height, h, mm (in.)								
	51 (2.0)	(2.0) 64 (2.5) 76 (3.0) 89 (3.5) 102 (4							
Cut*									
Cast**									

Table C.1Experimental Design For Flexure Testing.

Table C.2Experimental Design For Compression Testing.

Height-	Cylinder Diameter, mm (in.)					
Diameter Ratio	76 (3.0)		102 (4.0)		152 (6.0)	
	Cut*	Cast**	Cut*	Cast**	Cut*	Cast**
2						
5/3						
4/3						
1						

* "Cut" means that the specimen has been cut to the corresponding size from a larger specimen (102 mm (4 in) height in flexure or twice the diameter height in compression)

** "Cast" means that the specimen has been cast to the corresponding dimensions.

The "cast" versus "cut" specimens with the same dimensions are tested in order to detect the effects of fiber orientation (i.e., reorientation to fit in the corresponding mold size). The above experimental programs were applied to each one of the four mixes of Table 3.7. Three specimens were prepared for each one of cells of Tables C.1 and C.2. The flexural specimens were loaded under third-point loading on a span equal to 305 mm (12 in.). All the specimens were initially cured inside their molds under wet burlap and a polyethylene film during the first 24 hours of age. They were then stripped off of their molds and allowed to cure in ambient laboratory conditions with a temperature of 23 to 25 °C (73.4 to 77 °F) and 50 to 60% Relative Humidity up to the test age of 60 days.

C.5 EXPERIMENTAL RESULTS AND DISCUSSIONS

C.5.1 Flexural Testing

The obtained test results are plotted against the beam height in Figures C.3 through C.6. It can be noted that the flexural strengths corresponding to the beam depth of 51 mm (2 in.) do not seem to be reasonable. The reason for this could be threefold: first, in the cast specimens, this is most possibly due to the fact that, since the specimens are cast horizontally, during the casting process problems are encountered with the compaction and consolidation of the concrete materials inside such a thin specimen. Secondly, due to the relatively small beam thickness, shrinkage cracking at the surface could very possibly influence the resulting flexural strength values for the cast specimens. Thirdly, in the cut specimens, a damage to the

concrete specimen surface takes place during cutting, which produces disturbances to the bonding and interfacial conditions of fibers in concrete.

In general, however, the test results show a general trend of decreasing flexural strength with increasing beam depths. In many cases the size effect seems to be overshadowed by experimental variations and errors. In addition, the beam depth range selected in this investigation was only from 51 to 102 mm (2 to 4 in), which is considered to be a narrow range in which the size effect trend cannot be expected to be quite significant.



Figure C.3 Size Effect Regression Line For Plain Concrete.



Figure C.4 Size Effect Regression Line For Latex-Modified Concrete.



Figure C.5 Size Effect Regression Line For Steel Fiber Reinforced Concrete.



Figure C.6 Size Effect Regression Line For Latex-Modified Steel Fiber Reinforced Concrete.

Table C.3 gives the slopes of regression lines.

Table C.3Slopes For Regression Lines (in degrees).

	Plain	LMC	SFRC	LMSFRC
Cut	0.281	-2.88	-0.906	-1.18
Cast	-0.782	1.04	-2.91	-4.13
Average	-0.25	-0.92	-1.91	-2.66

From the above table and figures one can see that size effect is most pronounced in the fibrous mixtures. A comparison between the obtained results for plain mixtures versus the latex-modified mixtures shows that latex polymer addition seems to pronounce the size effect, especially in the cut specimens. This could be attributed to the fact that polymer addition results in a matrix that is more homogeneous and more sensitive to size effect. On the other hand, comparison of plain concrete with steel fiber reinforced concrete reveals that steel fiber addition increases the size effect. The reason behind this could be the fact that fibrous concrete matrices have a blunted localized fracture front in which the stress drop along the line of fracture extension is gradual and occurs over a finite and longer (compared to plain concrete) length which is a multiple of the aggregate size. This length could get very close to the structure dimensions if the structure is not very large.

The joint effect of steel fiber reinforcement and latex modification is found to further pronounce the size effect. This is mainly attributed to the improved bonding between matrix constituents (i.e., sand, gravel, fibers and cement) due to latex modification which results in better strengthening and microcrack bridging effects of the included fibers. These effects result in a larger and more stable strain-softening region and cause a more gradual drop in stress as crack front propagates through this region.

C.5.1.1 Effects of Fiber Orientation On Flexural Strength

The ultimate flexural strength σ_u of a composite steel fiber reinforced concrete containing uniaxial continuous fibers can be expressed by the law of mixtures as:

$$\sigma_{\mu} = \sigma_{\mu} \cdot V_{\mu} + \sigma_{f} \cdot V_{f} \tag{11}$$

where, $\sigma_m = \text{flexural strength of the unreinforced matrix;}$ $\sigma_f = \text{stress in fiber caused by the fiber-matrix interfacial bond stress;}$ $V_m = \text{matrix volume fraction; and}$ $V_f = \text{fiber volume fraction (equivalent volume of fibers which are effective in the direction of stress).}$

When a linear bond stress distribution is assumed, the maximum bond stress is twice the average bond stress, and the average stress in the fiber is related to the average bond stress by:

$$\sigma_r = 2\tau . l/d \tag{12}$$

The effective length of N number of fibers in the direction of stress is α .N.l, where α is the orientation factor and l is the fiber length. Therefore, the effective volume of the fibers in the direction of stress is:

$$V_{fe} = A_f \cdot l_{fe} = \frac{\pi \cdot d^2}{4} \cdot \alpha Nl$$
 (13)

and with $V_f = \frac{\pi d^2}{4}$. N. l and $V_{fe} = \alpha \cdot V_f$

Substituting Equation 12 into 11 yields:

$$\sigma_{\mu} = \sigma_{\mu} \cdot V_{\mu} + 2\tau \frac{l}{d} \cdot \alpha V_{f}$$
(14)

$$\sigma_{\mu} = \sigma_{m} \cdot (1 - V_{f}) + 2 \alpha \tau \frac{l}{d} \cdot V_{f}$$
(15)

C.5.1.2 Comparisons With Obtained Experimental Results

In order to assess the effect of the reorientation of steel fibers in fresh concrete due to the confining effects imposed by the molds, one should compare the obtained differences in flexural strength between the cast and cut specimens versus the theoretically derived differences.

Theoretically, we found that:

$$\sigma_{\mu} = \sigma_{\mu} \cdot (1 - V_f) + 2 \alpha \tau \frac{l}{d} \cdot V_f$$
(16)

It follows then that:

$$\sigma_{u_{(cont)}} = \sigma_{m} \cdot (1 - V_{f}) + 2 \alpha_{const} \cdot \tau \frac{l}{d} \cdot V_{f}$$
(17)

$$\sigma_{\mathbf{H}_{(cut)}} = \sigma_{\mathbf{M}} \cdot (1 - V_f) + 2 \alpha_{cut} \cdot \tau \frac{l}{d} \cdot V_f$$
(18)

and hence,

$$\sigma_{u_{(cont)}} - \sigma_{u_{(cont)}} = 2\tau \frac{l}{d} \cdot V_{f^*} (\alpha_{const} - \alpha_{cont})$$
(19)

This equation should be applied to our case with 1=30 mm (1.2 in), d=0.5 mm (0.02 in), $V_f=0.75\%$, $\tau=2$ MPa (300 psi) and 2.66 MPa (385 psi) for SFRC and
LMSFRC, respectively. Also, $\alpha_{cast} = 0.65$, 0.615, 0.59, 0.57 for beam heights of 51 mm (2 in), 64 mm (2.5 in), 76 mm (3.0 in), and 89 mm (3.5 in), respectively; whereas, $\alpha_{cast} = 0.55$ (obtained after evaluation of corresponding equations in Appendix D). Evaluating the theoretical difference $\sigma_{u(cast)} - \sigma_{u(cast)}$ for all different cases, we obtain the results presented in Table C.4.

Table C.4Theoretical Difference in Ultimate Flexural Strengths in Pa (psi).

	51 mm	64 mm	76 mm	89 mm
	(2 in)	(2.5 in)	(3.0 in)	(3.5 in)
SFRC	0.25	0.16	0.1	0.05
	(36)	(23.4)	(14.4)	(7.2)
LMSFRC	0.32	0.21	0.13	0.06
	(46.2)	(30.0)	(18.5)	(9.2)

This difference has also been evaluated for the obtained experimental results and tabulated in Table C.5.

 Table C.5
 Experimental Difference in Ultimate Flexural Strength in Pa (psi).

	51 mm	64 mm	76 mm	89 mm
	(2 in)	(2.5 in)	(3.0 in)	(3.5 in)
SFRC	0.26	0.76	0.94	0.59
	(37)	(110)	(136)	(86)
LMSFRC	1.1	2.4	2.16	1.9
	(157)	(353)	(313)	(275)

Figure C.7 illustrates the comparison between the theoretical and experimental differences in flexural strengths.



Figure C.7 Comparisons Between Theoretical and Experimental Flexural Strength Results.

C.5.2 Compression Testing

Regression lines of the obtained compressive strength test results for all four height-to-diameter ratios are plotted against the cylinder diameter and shown in Figures C.8 through C.11. It can be noted that the compressive strengths corresponding to cylinders with height-to-diameter ratio equal to 1 do not seem to be reasonable. This could be attributed to a few reasons. First, this is most likely due to the end effects of the loading machine because of the small height of the specimen. This actually follows from the Saint-Venant's principle which stipulates that the forces at the ends of the test cylinder have a nonuniform distribution and that stress distribution gets uniform only away from the supports. With cylinders having a height as big as the diameter, the stress distribution remains nonuniform even away from the ends. In addition, the loading platens of the testing machine would, in this type of situations, restrain lateral movements of the specimen and induce lateral stresses at both ends, thus affecting the mode of failure and yielding misleading results. Moreover, due to the small size, shrinkage cracking at surfaces could play a role, especially for plain mixtures. Furthermore, there are practical problems and difficulties due to the effects of compaction in such short specimens for the case of cast specimens, in addition to the effects of cutting process on such short specimens.

In general, however, the test results show overall trends of decreasing compressive strength values with increasing cylinder diameter for constant height-todiameter ratios. These conclusions are not as clearly obvious for plain mixtures (which are prone to more severe drying shrinkage effects that could overshadow size effects).



PLAIN (cut)



Figure C.8 Size Effect Regression Lines For Plain Concrete Under Compression.



Figure C.9 Size Effect Regression Line For Latex-Modified Concrete Under Compression

5

5.5

6

3.5

4

4.5

Cylinder Diameter (in.)



Figure C.10 Size Effect Regression Line For Steel Fiber Reinforced Concrete Under Compression.



Figure C.11 Size Effect Regression Line For Latex-Modified Steel Fiber Reinforced Concrete Under Compression.

As for the flexural test investigations, one can see that size effect is most pronounced in the fibrous mixtures. Comparing plain concrete versus the steel fiber reinforced concrete results reveals that steel fiber addition strongly influences size effect. The joint action of steel fiber reinforcement and latex modification on size effect is found to further pronounce size effect. This is mainly attributed to the improved bonding between matrix and inclusions (i.e., sand, gravel, and fibers) due to latex modification, thus resulting in better strengthening and microcrack bridging effects of the included fibers. These effects result in a larger and more stable strainsoftening region and cause a more gradual drop in stress as crack front propagates through this region.

APPENDIX D

APPENDIX D

FIBER ORIENTATION FACTOR - DERIVATION OF EQUATIONS

The contribution of a fiber to the load transfer of the matrix element in which it is fully embedded depends on its orientation with respect to the loading direction.^{7,} 20, 21, 27, 73, 79, 85, 86, 87, 89 This contribution can be presented by the ratio of the total projected fiber length in the loading direction, L_{proj}, and the sample volume, V:

$$(L_{\nu})_{proj} = \frac{L_{proj}}{V}$$
(1)

This is the relevant approach in the case of dense parallel microcracking, where every fiber contributes to the stress transfer at one or more crack surfaces.

For crack spacings in the order of magnitude of the fiber length, a considerable portion of the fibers does not contribute to the stress transfer at the crack surfaces. In this case, the effectiveness of the fibers, the so-called fiber orientation factor or geometric efficiency factor β , is determined by the total projected length in the direction of the load of those fibers intersecting the crack per unit volume of material. Hence, we can write:

$$\beta = \frac{L_{proj}}{Volume}$$
(2)

The probability of hitting a fiber is given by:

$$Prob(hit) = L_{proj} = Projected \ Length \ of \ an \ Arbitrary \ Fiber$$
(3)

Many factors could influence the value of the orientation factor, these include:

· geometric boundary conditions of the specimen that could be restricting the orientation of the fibers;

· vibration during concrete construction which may cause reorientation of the fibers in horizontal planes; and

· location in cross-section (top versus bottom) with respect to the casting and vibration directions.

Steel fibers, when uniformly dispersed in an infinitely large volume of concrete, are expected to be randomly oriented, with equal probabilities of being oriented in different directions in space. From the unit sphere of Figure 1, we get:

$$L_{proj} = \cos\theta \cdot \cos\phi \tag{4}$$

A pyramidal element of the unit sphere is considered here with infinitely small cross-sectional dimensions (see Figures D.1 and D.2), containing all the fibers with an orientation falling in the range θ to $\theta + d\theta$ and $\phi + d\phi$. The dimensions of the basis of the trapezium on the surface of the unit sphere amount to $d\phi$ and $\cos\theta d\theta$; and therefore we obtain:

Geometric Efficiency Factor
$$\beta = \frac{\iint L_{proj} \cdot dA}{\iint dA}$$
 (5)

from which,



Hence, only 50% of the total fiber length of randomly dispersed fibers is oriented in a particular direction.

In practice, however, fibers are not randomly oriented mainly due to, among other factors, specimen boundaries restricting their random orientation within the concrete matrix. For the case of non-random fiber dispersions, we can obtain in the same way appropriate values for the orientation factor:

(a) <u>Two Boundaries</u>: When two boundaries are present to restrict the fiber orientation (see Figure D.3), the orientation factor in the z-direction of stress can be obtained by considering the effects of these two parallel boundaries as follows:

(6)

$$\alpha_{1} = \begin{cases} \frac{\frac{h}{2}}{\int \beta_{1} \cdot dy} \\ \frac{\frac{d_{f}}{2}}{2} & \cdots \text{ for } h < l_{f} \\ \frac{\frac{h}{2}}{2} & \int \beta_{1} \cdot dy \\ \frac{\int \beta_{1} \cdot dy}{\int \beta_{1} \cdot dy} & \int \beta_{1} \cdot dy \\ \frac{l_{f}}{\frac{f}{2}} & \frac{l_{f}}{2} & + (1 - \frac{l_{f}}{h})(0.50) & \cdots \text{ for } h \ge l_{f} \end{cases}$$
(7)

where,

$$\beta_{1} = \frac{\int_{0}^{\frac{\pi}{2}} \int_{\gamma_{0}}^{r} l_{f} \cos^{2}\theta \cdot \cos\phi \cdot d\theta \, d\phi}{l_{f} \int_{0}^{\frac{\pi}{2}} \int_{\gamma_{0}}^{r} \cos\theta \cdot d\theta \, d\phi} \qquad (8)$$

$$\gamma = \sin^{-1}(\frac{2y}{l_{f}}) ;$$

$$\gamma_{0} = \sin^{-1}(\frac{d_{f}}{l_{f}}) \qquad (=\gamma \ at \ y = \frac{d_{f}}{2}) .$$

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(b) Four Boundaries: When four boundaries are present to restrict the fiber orientation (see Figure D.4), the orientation factor in the z-direction of stress can be obtained as follows:

$$\alpha_{2} = \begin{cases} f(b,h) & \cdots \text{ for both sides } (b,h) < l_{f} \\ & \int_{f}^{\frac{h}{2}} \beta_{1} \cdot dx \\ \frac{l_{f}}{b} f(l_{f},h) + (1 - \frac{l_{f}}{b}) \frac{\frac{d_{f}}{2}}{\frac{h}{2}} \\ & \cdots \text{ for only one side } h \leq l_{f} \end{cases}$$

$$(9)$$

$$\frac{l_{f}^{2}}{bh} f(l_{f},l_{f}) + \frac{l_{f}(b+h-2l_{f})}{bh} \frac{\frac{d_{f}}{2}}{\frac{l_{f}}{2}} \\ & \cdots \text{ for both sides } (b,h) \geq l_{f} \end{cases}$$

where,

$$f(m,n) = \int_{\frac{d_f}{2}}^{\frac{m}{2}} \frac{\frac{n}{2}}{\frac{d_f}{2}} \frac{\beta_2 dx dy}{(m/2)(n/2)} ;$$

$$\beta_2 = \frac{\int_{\frac{v_0 v_0}{2}}^{\frac{v_0}{2}} l_f \cos^2\theta \cos\phi d\theta d\phi}{\int_{\frac{v_0 v_0}{2}}^{\frac{v_0 v_0}{2}} l_f \cos\theta d\theta d\phi} ;$$

$$\gamma_0 = \sin^{-1}(d_f/l_f) ;$$

$$\gamma = \sin^{-1}(2y/l_f) ;$$

$$\delta = \sin^{-1}(2x/l_f) .$$
(10)



Figure D.1 Element of the Unit Sphere in Spherical Coordinates.



Figure D.2 Approximate Dimensions of a Spherical Block.



Figure D.3 Case of Steel Fibers Restrained by Two Boundaries.



Figure D.4 Case of Steel Fibers Restrained by Four Boundaries.

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