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ENHANCEMENT OF THE CRACKING RESISTANCE AND TOUGHNESS CHARACTERISTICS OF CONCRETE WITH RECYCLED SYNTHETIC INCLUSIONS

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ENHANCEMENT OF THE CRACKING RESISTANCE AND TOUGHNESS CHARACTERISTICS OF CONCRETE WITH RECYCLED SYNTHETIC INCLUSION

By

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ABSTRACT

ENHANCEMENT OF THE CRACKING RESISTANCE AND TOUGHNESS CHARACTERISTICS OF CONCRETE WITH RECYCLED SYNTHETIC INCLUSIONS

By

Aly I. Eldarwish

The main intent of this research was to determine the feasibility of utilizing recycled plastics and tires, through mechanical size reduction, as soft reinforcing inclusions in concrete.

Plastic flakes obtained through size reduction of high-density polyethylene and mixed plastic wastes were used at relatively high volume fractions as replacement for fine aggregate in light-weight concrete, and as an additive at relatively low volume fractions in normal-weight concrete. Plastic inclusions were effective in enhancing the flexural toughness, post-cracking impact strength and resistance to shrinkage cracking of light-weight concrete. The flexural strength of light-weight concrete could be maintained at an optimum plastic volume fraction; the compressive strength, however, was reduced in the presence of plastics. The positive effects of plastics could be attributed to their micro crack arrest and deflection, and particularly crack bridging effects. In compression, the adverse

effects of stress redistribution and concentration resulting from the presence of low-modulus inclusions seem to overshadow any positive effects resulting from the interaction of plastics with cracks. In normal-weight concrete, relatively low volume fraction of plastic flakes had significant effects on reducing the restrained shrinkage crack widths; slight improvements in flexural toughness were also observed in the presence of plastics. Various aspects of the long-term performance of both the light-weight and normal-weight plastic-concrete composites under severe exposures were also investigated. While long-term performance was generally good, plastic flakes at relatively high volume fractions were observed to have adverse effects on de-icer salt scaling resistance; the normal-weight concrete with optimum plastic flake content exhibited satisfactory scaling resistance.

Tire inclusions were observed to be dimensionally unstable in concrete under moisture effects; coated tire particles at relatively low volume fractions, however, performed satisfactorily in reducing restrained shrinkage crack widths in normal-weight concrete.

DEDICATED TO: MY FATHER PROFESSOR IBRAHIM ELDARWISH, MY MOTHER NAGIA ELDARWISH, AND MY SISTER INGY ELDARWISH, FOR THEIR GREAT LOVE AND ENDLESS ENCOURAGEMENT.

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Last but not least, I would like to express my appreciation and gratitude to my wonderful family, my father professor Ibrahim A. Eldarwish, my mother Nagia Eldarwish, and my sister Ingy Eldarwish, who stood by me during my ups and downs throughout my graduate studies in the U.S.A. Their emotional support, understanding, love and constant encouragement made my life much happier and my work much easier.

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CHAPTER 1

INTRODUCTION

1.1 CONCRETE MATERIALS

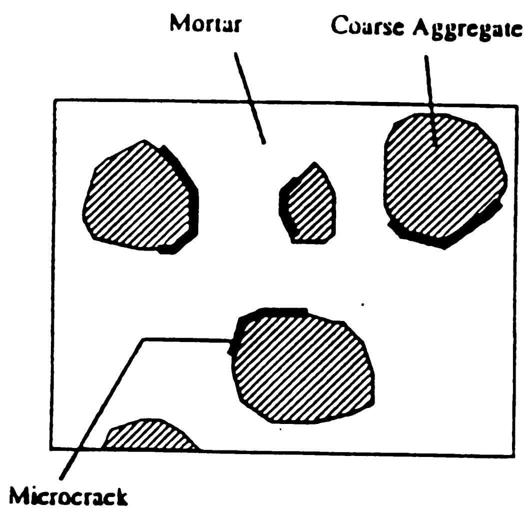
The most widely used construction material is concrete, commonly made by mixing Portland cement with sand, aggregate, water and some admixtures. The consumption of concrete in the U.S. is close to two tons per year for every U.S. resident. No other material except water is consumed in such tremendous quantities. Concrete has emerged as the most widely used engineering material because: (1) it possesses excellent resistance to water; (2) structural concrete elements can be easily formed into a variety of shapes and sizes; (3) concrete is usually the cheapest and most readily available material on the job site; (4) compared to most other engineering materials, the production of concrete requires considerably less energy input; and (5) large amounts of many industrial wastes can be recycled as a substitute for various virgin materials in concrete.

There are some key advantages associated with recycling in concrete construction, including: (1) the potential to develop of large-volume markets for waste products; (2) potentially reduced need for purification of waste; and (3) long-term removal of recycled materials from the waste stream, noting that concrete products typically have a service life exceeding 40 years. Improvements in some key aspects of concrete performance can make important contributions toward developing a more reliable infrastructure. Recycling of

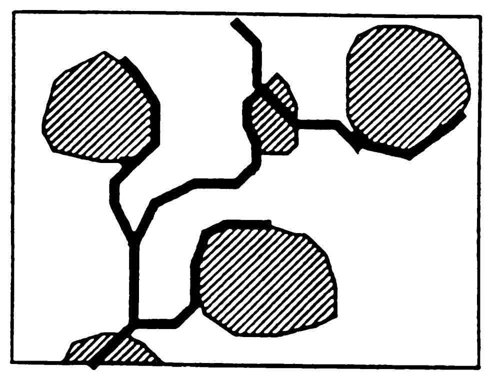
plastics in concrete can help overcome problems with the brittleness, low cracking resistance and the relatively high unit weight of concrete. It can also help control shrinkage cracking of concrete. The study presented herein is concerned with the recycling of plastics as reinforcing and light-weight inclusions in light-weight concrete, and as reinforcing inclusions in normal-weight concrete.

1.2 HYPOTHESIS: Reinforcing Action of Plastic Inclusions

It is now known that even before the application of external loads, microcracks already exist in the transition zone between the mortar matrix and coarse aggregates in concrete as shown in Figure 1a (Hsu et al, 1963). The number and width of these cracks in concrete would depend, among other factors, on the bleeding characteristics and strength of the transition zone, and the curing history of concrete. Under ordinary curing conditions (when a concrete element is subjected to drying shrinkage or thermal strains), due to the differences of dimensional movements and elastic moduli, differential strains will be set up between the matrix and the coarse aggregates, generating the cracks in the transition zone. Under load and environmental effects, the transition zone microcracks begin to increase in length, width and number, initially within the transition zone and later into the matrix and (in the case of light-weight aggregates) through the aggregates (Figure 1b). The relatively low fracture energies required for the propagation of cracks in brittle concrete matrices result in relatively low toughness, impact resistance and tensile strength of concrete.



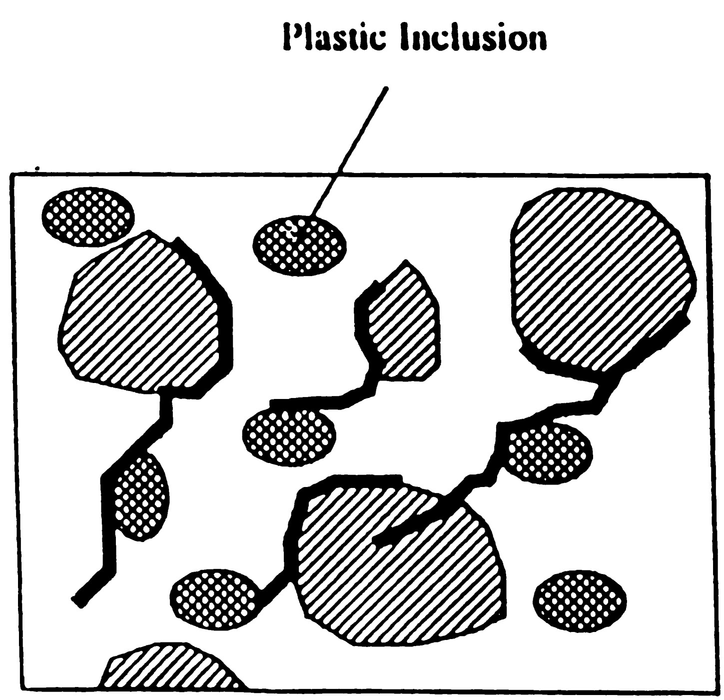
(a) Prior to Load Environmental Effects



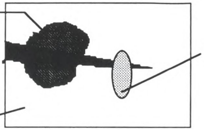
(b) Under Load/Environment Effects

FIGURE 1.1 Concrete Microcracks (Hsu et al, 1963)

In plastic-concrete composites, the encounter of propagating microcracks with the tough and well-bonded plastics leads to relaxed intensity of stresses at the crack tips, a phenomenon which increases the fracture energy and thus the toughness and impact resistance of the composite as well as its resistance to shrinkage cracking. Delayed propagation of microcracks encountering the plastic inclusions would take place within the plastic-cement paste transition zone rather than through the plastic inclusions (Figure 2a). Eventually, when increased load levels lead to interconnection and rapid growth of microcracks, the bridging of plastic inclusions across the resulting cracks helps control the crack widths and maintain the integrity of the composite after cracking (Figure 2b).



(a) Arrest and Deflection of Microcracks



(b) Bridging of Cracks

FIGURE 1.2 Mechanism of Action of Plastic Inclusions in Concrete

1.3 OBJECTIVES

The main thrust of this research is to assess the feasibility of using recycled plastics as reinforcing inclusions in concrete.

The work reported herein is divided into 3 parts. Part I deals with recycled plastics as soft inclusions in light-weight concrete; part II considers recycled plastics as additives, to control shrinkage cracking, in normal-weight concrete. Part III deals with recycled tire as reinforcing inclusion in normal-weight concrete.

Chapter 2 gives a background on recycled plastics, surface treatment through sulfonation, and light-weight concrete. Chapter 3 is concemred with determining the influential variables in the production of light-weight concrete incorporating recycled plastics. Chapter 4 deals with optimizing the influential

variables identified in Chapter 3. Theoretical modeling of light-weight concrete incorporating plastics is presented in Chapter 5 which emphasizes compressive behavior. Long-term durability characteristics and environmental impact of light-weight concrete materials incorporating recycled plastics are assessed in Chapter 6.

Part II, which deals with recycled plastics as reinforcing additives in normal-weight concrete, is divided into two chapters. The first chapter (Chapter 7) deals with optimizing the potentially influential variables in application of recycled plastics to normal weight concrete. The composites were optimized considering their resistance to cracking under restrained drying shrinkage. Both short- and long-term properties of the optimized composite were evaluated in Chapter 8 in order to ensure the added values of using recycled plastics in normal-weight concrete.

In the final part of this research (Chapter 9), potential chemical and physical causes of dimensional instability of tires in concrete, causing pop-out were investigated. Following the elimination of pop-out problem through coating of tire particles, the effectiveness of recycled tires as reinforcing inclusions in normal-weight concrete was demonstrated.

The summary and conclusions of research as well as the recommendations for further research are presented in Chapter 10.

PART 1: PLASTIC LIGHT-WEIGHT CONCRETE

CHAPTER 2

BACKGROUND

2.1 RECYCLED PLASTICS

2.1.1 PLASTIC TYPES

Plastics can be classified into four groups: commodity thermoplastics, engineering thermoplastics, thermosets and multicomponent plastics. Commodity thermoplastics are produced at low costs in high volumes. They include the five resins that account for about two-thirds of all plastic sales: low-density polyethylene (LDPE), high-density polyethylene (HDPE) polyvinyl chloride (PVC), polypropylene (PP) and polystyrene (PS). They also include polyethylene terephthalate (PET), which has only recently been used in sizable quantities for packaging. Other commodity thermoplastics include acrylonitrile butadine styrene (ABS) and nylon. In general, commodity thermoplastics can be reheated and reshaped many times and are therefore eminently recyclable.

Engineering thermoplastics are produced at high costs and in low volumes. Examples include polycarbonate and polytetrafluoroethylene

(PTFE). Engineering thermoplastics are used in the construction, electric/electronic and transportation markets. These plastics are not considered a major component of the municipal solid waste, MSW (Facing America's Trash, 1990). Engineering thermoplastics can be used in structural panels, building insulation and other long-life construction markets.

Thermosets, compared to thermoplastics are low-volume materials but they still compromise about 20% of the US plastics market. Two resins, phenolic and polyurethane, are sold in sizable quantities. They can be used in building and construction, transportation, and furniture. Unlike thermoplastics, thermosets generally are not considered recycable because they do not soften when heated and thus can not be remolded.

Multicomponent plastics and laminations are combinations of different plastics or of plastics and other material such as paper or metal foil. These materials are primarily used for packaging. Multicomponent plastics provide an economical way of combining the needed properties of different materials. Recently, available plastic ketchup bottles are made of several plastics including an exterior plastic for appearance and strength and an interior plastic to resist fats and acids. This combination of materials makes these plastics technically difficult to recycle except into mixed plastic products. Medical devices also are fabricated of or contain a variety of polymeric resins such as polyolefins. Most of the recycling handlers, reclaimers and distributors refer to the thermoplastics that are not pure (i.e. colored or mixed thermoplastics) as commingled plastics.

2.1.2 PROCESSING PROCEDURES FOR RECYCLED PLASTICS

To produce virgin substitute resins suitable for use by manufacturers, post-consumer plastics must go through 3 stages: separation, washing and granulation.

Granulators are designed and engineered to meet the specific requirements on the size, shape, amount, and type of plastic scrap being produced. It is usually a mechanical procedure. The number and design of the knife blades and their rotational speed, rotor design and screen size are all essential to the reliable production of high quality scrap recycling. Granulators all work on a similar principle; they consist of knives mounted around the perimeter of a cutting chamber. There can be 1 to 6 stationary knives and as many as 32 rotating knives depending on the rotor design. Although granulate is easy to use, there has been problems with contamination.

Recycling collectors are broadly using Granulators to process scrap plastics. A brief review of the granulation process and how it has been applied to processing is presented below.

The purpose of a granulator is to reduce materials to a uniform and consistent size so it can be reclaimed for use in any of the various plastic manufacturing processes. A granulator is a very simple piece of equipment which has 3 major components:

(a) the hopper, designed to accept the subject feed stock;

(b) the cutting chamber, where size reduction takes place; and

(c) the base, where granulated material is collected .

Several factors must be considered to determine the throughput capability of a granulator. These factors are:

(i) hopper opening and configuration;

(ii) rotor design and knife configuration;

(iii) available screen area and hole size;

(iv) horsepower; and

(v) method of evacuation.

The proper rotor design is essential. An open rotor will increase and help prevent heat build-up in the cutting chamber because of better air flow. It has been reported that HDPE has been known to plasticize in cutting chambers (Changon, 1989).

A slant knife design rotor with counter angled rotor to bed knives provide cleaner and more uniform granulate with less dust, see Figure 2.1.

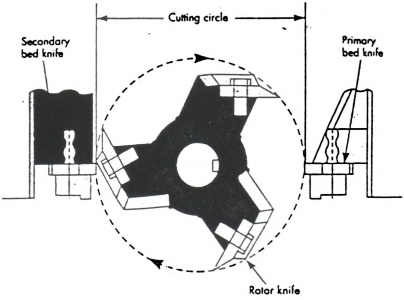


FIGURE 2.1 A Schematic Presentation of the Granulator Cutting Chamber (Changon, 1989)

It has been reported by several people that an absolute necessity is to feed the blown bottles to the cutting chamber on a tangent. A tangential feed cutting chamber presents the feedstock to the down stroke of the rotor and eliminates "rotor bounce".

The open screen area and the diameter of the holes in the screen will determine how quickly the granulated product can exit the cutting chamber.

Following the granulation process, the separation process takes place. The separation process is the backbone of any plastics recycling process. Automated separation technologies have been developed. Such technologies separate out component resins by combination of floatation, electrostatic processes and air classification.

The first separation procedure used is gravity separation. In this method, separation is by blowing air through a porous deck shaped like a trapezoid. The lighter material or the material with the lowest terminal velocity will escape from the deck and will be collected while the remainder will be conveyed onto the next stage of processing. The final stage of separation is the is the electrostatic separation.

The last step for preparing the recycled plastics is the washing process. Water hoses are used to wash the recycled plastics.

Sulfonation of plastics is used in this investigation to make the plastic surfaces wettable and provide for chemical bonding of the cement-based matrix to plastic surfaces (see Figure 2.2).

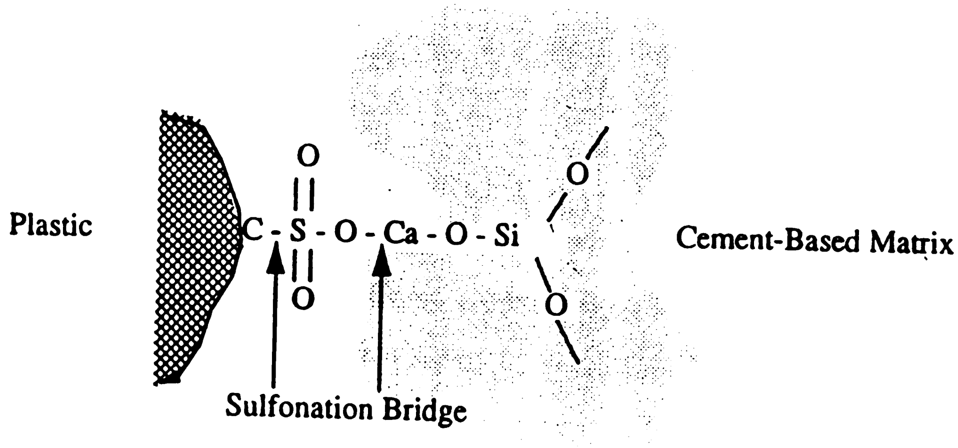


FIGURE 2.2 Chemical Bonding of Sulfonated Plastic Surfaces to the Cement-Based Matrix

2.2 SULFONATION

Chemical modification of the surface of engineering polymers is an attractive approach to the problem of economically producing a barrier or a permelective structure with tailored properties.

2.2.1 RATIONALE FOR THE USE OF SURFACE SULFONATION

In a study by Walles, 1990, involving the use of fluorine treatments resulted in containers with a good barrier to gasoline, however, upon repeated flexing, a loss of barrier properties can results. The barrier layers in these cases were up to 0.1m thick. Additional studies using 25m (82 ft) films of polystyrene, polyethylene and polypropylene exposed to fluorine for a longer time led to total pulverization of the films. The loss in mechanical properties appears to be associated with the localization of energy associated with the C-F bond formation, which in turn can break neighboring C-C-bonds of the polymer backbone. Under certain rigorously controlled treatments, the use of fluorine has proven workable, but it was felt that a less aggressive agent allowed added process flexibility to sulfonation.

Surface chlorination can produce good barrier layers with less tendency to provide flexure failures. Unfortunately, chlorine reacts too slowly to be practical as a treatment agent in the absence of promotion by ultra-violet light.

Chemical activation of the chlorination reaction was shown to be possible using a mixture of 10% SO_3 and 90% Cl_2 as such or further diluted with air or nitrogen at 25 ^{o}C (77 ^{o}F). This procedure produced good barriers to gasoline, particularly to the ethanel-and methanel- containing types.

Treatments of the inside of high density polyethylene (HDPE) automotive gas tanks with about 20% SO_3 in air followed by air purging with subsequent neutralization with NH_3 gas resulted in an excellent gasoline barrier.

It was reported by Walles (1990) that concentrations of about 75 and 200 micrograms SO_3 per cm^2 (0.484 to 1.29 psi) of surface reduces permeation losses by 90 and 99% in ambient temperature permeation tests.

2.2.2 NATURE OF THE BARRIER LAYER

Much work has been done on solution sulfonation, but solution sulfonation causes problems such as polymer swelling and solvent contamination (Tradiff, 1993). A gaseous sulfonation method is preferred, using sulfur trioxide gas. The small gas molecule can diffuse farther into a polymer sample than the bulky liquid group, resulting is a more thorough and efficient surface modification. Sulfur trioxide gas is difficult to use due to its high affinity for water. Much of the work that has been done to characterize gaseous sulfonation has been conducted by Walles (Walles, 1992).

The sulfur trioxide molecule is a resonance hybrid, which means that the oxygen atoms are equivalent. The oxygen atoms are stingily bound, with a large degree of double bond character. Due to the binding nature of the oxygen atoms, the sulfur atom is strongly electron-deficient and the oxygen atoms are electron-rich (Walles, 1992). A typical reaction with a polymer is;

$$-[CH_2CH_2]_n - CH_2CH_2- \quad + \quad SO_3 \text{ -------->} -[CH_2CH_2]_3-CH_2CHSO_3H-$$

where the sulfonic acid group ($-SO_3H$) is added to the polymer chain. In reactions with polymers, the sulfur atom attacks the electron-rich sites, abstracts a proton, and the oxygen atoms accept the acidic protons. Since sulfur trioxide often exists as a complex, it has been found that the modification surface group may initially exist as SO_3SO_3H, with one SO_3 group being more strongly bound than the other. With a water wash after sulfonation, one SO_3 group is removed as H_2SO_4, thus producing the SO_3H group on the polymer chain.

The sulfonation reaction is a reversible reaction. Due to its strong affinity for water, the sulfonic acid group may be removed with water, producing sulfuric acid. To prevent the loss of the sulfonic acid group, the sulfonated material is neutralized with ammonia or ammonium hydroxide;

$$-[CH_2CH_2]_n-CH_2CHSO_3H- + NH_3 \text{-------}> -[CH_2CH_2]_n\, CH_2CHSO_3^-NH_3^+-$$

The neutralized sulfonic acid group ($CHSO_3^-NH_3^+$) is a stable complex which resists the attack of solvents such as water due to the strong attraction between the sulfonic acid ion and the ammonium ion.

The neutralized sulfonic group, as well as the unneutralized sulfonic acid group, is polar thus making the polymer water-wettable. Water-wettability is important for the use of recycled plastics in concrete materials.

All commercially available films containing either a CH or an NH bond have been found to be treatable via the sulfonation process.

The time required to produce a desirable barrier improvement varies with the type of polymer, its degree of crystallinity and particular penetrant being considered.

Polyethylene generally requires a degree of sulfonation equivalent to about 0.1 mg SO_3 per cm^2 ($142*10^{-5}$ lb/in^2). For polypropylene a degree of sulfonation as low as 0.015 mg SO_3 per cm^2 can be effective ($213*10^{-6}$ lb/in^2).

2.2.3 SULFONATION

The process of sulfonation starts with delivering a stream of sulfur trioxide reagent and maintaining the reagent stream within narrow, but adjustable concentration limits. The sulfur trioxide reagent is a reagent of sulfur trioxide in a carrier. The carrier may be either a liquid halocarbon or an inert gas such as dry air.

In either way, the process for generating the reagent includes the step of introducing a source of sulfur trioxide into a suitable vessel such as a reagent generator. The source of sulfur trioxide may be gaseous sulfur trioxide or sulfur trioxide in a liquid vehicle, such as oleum counting 10-90% sulfur trioxide. That step may actually include the introduction of a source of sulfur trioxide into a separate contact chamber and, from there, to the reagent of SO_3 in carrier. The process also includes the step of introducing the carrier to the reagent generator, either as a liquid halocarbon or an inert gas. Finally, the process includes the step of introducing oleum into the reagent generator.

In this approach, a process for generating and recycling a solution of sulfur trioxide in a liquid halocarbon as the carrier is provided. The process includes the steps of introducing a source of SO_3, a halocarbon, and oleum into a reagent generator. The SO_3, halocarbon, and oleum are contacted and mixed in the reagent generator to provide a reagent solution of SO_3 in the halocarbon. Additionally, the oleum phase which forms is removed from the reagent generator for recycling. The sulfur trioxide in halocarbon solution phase is sent from the reagent generator to a parts treatment chamber located in the parts treatment loop. This separation of the two phases is readily accomplished due to the immiscibility of the halocarbon and oleum as well as the differences in density between the two and the high solubility of sulfur trioxide in the halocarbon.

The process for generating SO_3 reagent solution also includes the surface treatment of polymer resins in a treatment chamber in the parts treatment loop. Figure 2.3 shows a schematic picture of the sulfonation apparatus.

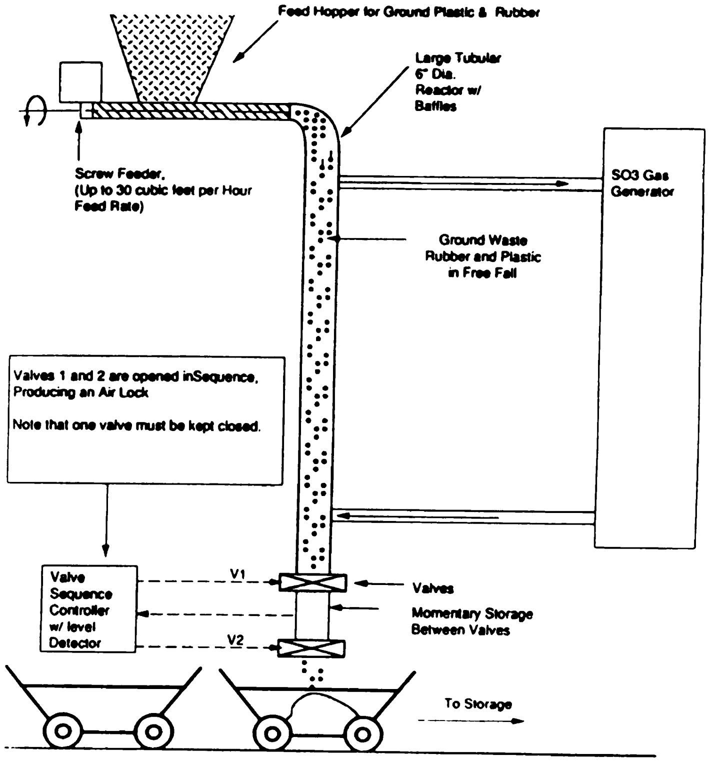


FIGURE 2.3 Schematic Picture of the Sulfonation Apparatus (Walles et al, 1990)

2.3 LIGHT-WEIGHT CONCRETE

Light-weight concrete may weigh from 320 kg/m^3 (20 pcf) to 1844 kg/m^3 (115 pcf), depending on the type of light-weight aggregate used and the method of production. The strength of light-weight aggregate is roughly proportional to its density. The advantages of light-weight concrete generally result from the decrease in member size because of reduced weight, increase in fire resistance, and insulation against heat and sound when light-weight concrete is used. Light-weight concrete, however, is typically 30-50% more expensive than normal-weight concrete (Nilson and Winter, 1986), and has a greater porosity and more drying shrinkage than ordinary concrete.

2.3.1 CLASSIFICATION

Light-weight concrete can be classified in accordance to its density, or based on the purpose for which it is to be used. Distinctions can be made between structural and insulating light-weight concrertes. Structural light-weight concretes has a unit weight between 1443-1844 kg/m^3 (90-115 pcf), with a 28-day compressive strength not less than 17 MPa (2500 psi). Insulating light-weight concrete has a density lower than 802 kg/m^3 (50 pcf) and a 28-day compressive strength between 0.69-6.9 MPa (100-1000 psi). Light-weight concrete can be classified as moderate strength light-weight concretes with properties in between those of structural and insulating concretes (Neville, 1982).

A variety of light-weight aggregates have been used for the production of light-weight aggregate concrete. Figure 2.4 shows the classification of light-weight concretes produced with different light-weight aggregates.

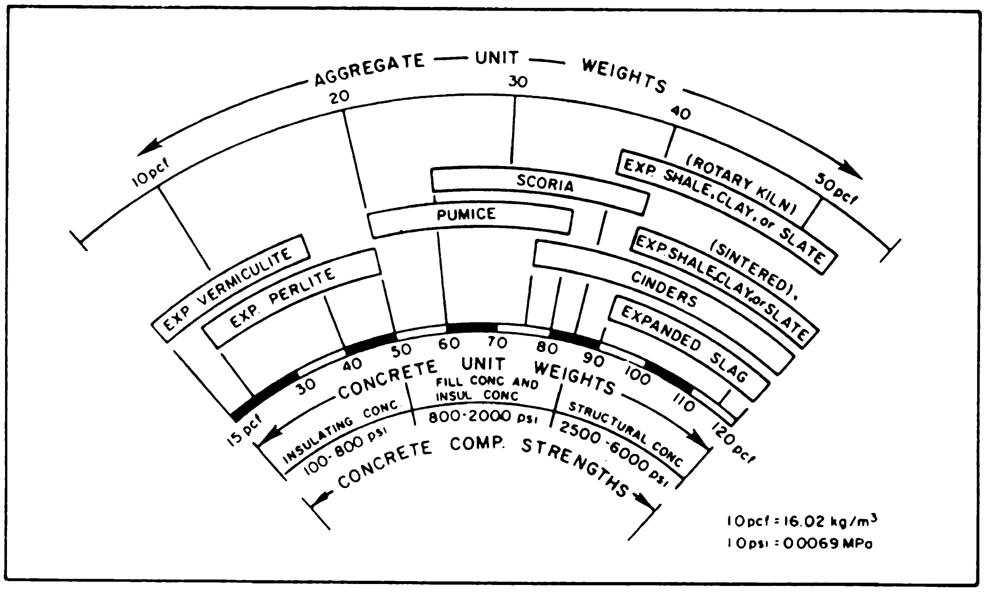


FIGURE 2.4 Classification of Light-Weight Concretes (Mehta, 1993)

2.3.2 LIGHT-WEIGHT AGGREGATES

There are many types of aggregates available which are classified as being light-weight. Their individual characteristics are so variable that it is impossible to consider a typical light-weight aggregate to produce a similar (standard) concrete mix for which any type of light-weight aggregate can be used with equal success.

The process by which light-weight aggregates are obtained has a considerable bearing on their resulting characteristics and hence, the uses to which they can be put. Their chemical composition is variable, some are inert while others may be corrosive to steel.

Light-weight aggregates can be classified into two groups, natural and manufactured, including recycled waste products and industrial by-products. Pumice, which are common rocks of volcanic origin which occur in many parts of the world and are light enough and generally strong enough to be used as light-weight aggregates. Their lightness is due to their being spongy laves, the cells having being formed by gases expanding with release of pressure when the material was still plastic. Pumice is usually light colored or nearly white and has a fairly even texture of small interconnected cells. Pumice is the oldest known light-weight aggregate and from about 100**BC** onwards was commonly used as an aggregate in the concrete roofs and walls of Roman buildings, notably baths and temples, the best known surviving example being the Pantheon in Rome, in the dome of which pumice concrete was used. In the mid-nineteenth century pumice as an aggregate was revived in Germany, where there were large deposits of the rock, and its use subsequently spread to other parts of Europe. Considering its mode of formation, it is not surprising that pumice varies in quality in different localities, and that it is not all equally suitable as aggregate. While the strength of pumice aggregate is an important factor in the achievements of strength in concrete

produced from it, the particle grading and the degree of compaction attainable under different manufacturing conditions influence the density and strength of concrete.

Another natural light-weight aggregate is Diatomite, which is essentially a hydrated amorphous silica derived from the skeleton remains of microscopic aquatic plants called diatoms. When pure, diatomite has an average density of 448 kg/m^3 (28 pcf), but due to impurities such as sand clay, it may be much heavier than this. It is mined in many parts of the world. Diatomite has many uses, as a filtering aid in sugar refining, and as a workability aid in concrete. In the USA, it is used for concrete for the insulation of high temperature furnaces. Low-grade diatomite and diatomaceaous earth are sintered in rotary kilns at about 1100°C (2012°F) to produce light-weight aggregates, such as Diacrete, Raycrete and Arrox.

2.3.3 PECULIARITIES OF LIGHT-WEIGHT CONCRETE MIX PROPORTIONING

Light-weight aggregates tend to segregate and float on the surface of concrete in high-consistency mixtures. This problem can be controlled by limiting the maximum slump and by entraining air in fresh concrete mixtures made with light-weight aggregates.

A major factor necessitating adjustments in proportioning and control procedures of normal-weight concrete when applied to light-weight concrete is the greater water absorption and higher absorption rate of light-weight aggregates. Light-weight aggregate mixtures are usually proportioned by trial on a cement and air content basis at the required consistency rather than on a water-to-cement ratio basis, because water-to-cement ratio can not be established with accuracy due to absorption of water by light-weight aggregates.

In a recent study, it was observed that during mixing of light-weight aggregate concrete the cement paste will penetrate most of the open pores in a surface layer of the aggregate. The amount of paste penetration depends on the micro structure of the surface layer of the light-weight aggregate, the particle size distribution of the cement, and the viscosity of the paste (Zhang and Gjorv, 1992).

The particular properties of light-weight aggregates pose special problems in calculating mix proportions for lightweight concrete. The absolute volume method, which is the basis of the American Concrete Institute method for proportioning normal-weight concrete, can not be used with confidence for lightweight concrete. This is due to :

1. Variations in bulk specific gravity of lightweight aggregates;
2. Changes in lightweight aggregate moisture content;
3. Difficulty to quantify the absorption, penetration of cement paste.

2.3.4 PECULIARITIES IN MANUFACTURING LIGHTWEIGHT CONCRETE

Production of concrete with light-weight aggregate involves all the procedures and precautions that are necessary for ordinary concrete. After mix design, the ingredients should be mixed according to ASTM C-94 as in the case of normal weight concrete. The problem is more difficult when light-weight aggregates are used because of greater variations in absorption, specific gravity, moisture content and gradation of aggregates. Uniform results can be obtained if the unit weight and slump tests are performed frequently, and the water content of the mix is adjusted as necessary to compensate for variations in properties.

Dry light-weight aggregates should not be used at the mixing stage because continuous water absorption by dry light-weight aggregates will cause concrete to segregate and stiffen before placement is completed.

In order to ensure uniformity, light-weight aggregates should be presaturated. Presaturation of aggregates will help achieve uniformity but may reduce freeze-thaw durability (particularly if concrete is not allowed to dry before exposure to freezing temperature).

Workability of freshly made light-weight aggregate concrete requires special attention because, with high consistency mixtures, the aggregate tends to segregate and float on the surface. To combat this tendency, it is necessary to limit the maximum slump and to entrain air. Approximately, 5-7% air entertainment is generally required to lower the mixing water requirement while maintaining the desired slump and reduce the tendency for segregation and bleeding. A slump of 51-76 mm (2-3 inches) represents a relatively high workability. A slump in excess of 51-102 mm (2-4 inches) may cause segregation with light-weight aggregate particles floating to the top. The tendency towards floating of larger particles of light-weight aggregates may be improved by adjusting the grading of aggregates. Concretes made with light-weight aggregates may be difficult to place and finish because of porosity and angularity of the aggregates. The placeablity of concrete can be improved by adding air-entraining agents.

2.3.5 MECHANICAL PROPERTIES OF LIGHT-WEIGHT AGGREGATE CONCRETE

The following properties of light-weight aggregate concrete will be discussed in this section: Compressive, tensile and flexural strengths, modulus of elasticity and Poison's ratio, impact resistance, ductility, shrinkage, abrasion resistance, permeability and creep characteristics.

2.3.5.1 Compressive Strength

Design compressive strengths of 21-28 MPa (3000-4000 psi) at 28 days are common for structural light-weight concrete. Light-weight aggregates with controlled microporosity have been developed to produce 69-76 MPa (10000-11000 psi) light-weight concretes which generally weigh 1435 to 1735 kg/m^3 (Zhang and Gjorv, 1992).

The following factors associated with light-weight aggregates may contribute towards the differences in compressive strength of light-weight concretes:

i) Aggregate strength;

ii) Aggregate stiffness;

iii) Aggregate surface texture;

iv) Differences in surface area of the aggregate particles;

v) Aggregate shape

Concretes made with round aggregates have 28-day compressive strengths of about 6-8 MPa (870 to 1160 psi) higher than those obtained with elongated aggregates (length/thickness=4.0). The differences in surface area associated with the differences in shape contribute to strength variations (Soroushian et al, 1989). This could be due to the fact that the shape of light-weight aggregates affects stresses concentrations in concrete under load causing differences in the compressive strength of light-weight aggregate concretes made with different aggregate shapes (Gzuryszkiewiez, 1973).

2.3.5.2 Modulus of Elasticity and Poison's Ratio:

The modulus of elasticity of light-weight concrete E_c tends to increase with increasing strength of concrete. Modulus of elasticity of light-weight concrete is typically lower than that of normal-weight concrete. The following equation is recommended for calculating E_c with compressive strengths ranging from 21-48 MPa, 3000 to 7000 psi (Mehta, 1993).

$$E_c = (w_c)^{1.5} \sqrt{33 * f'_c}$$

It has been reported that the elastic properties of the light-weight aggregates are characteristic of the material property and are therefore not affected by particle shape (Gzuryszkiewiez, 1973). Thus, it can be assumed that Young's modulus of all shapes is the same.

Typical Poison's ratio for light-weight concretes range from 0.17 to 0.21.

2.3.5.3 Ductility

Ductility is the ability to sustain inelastic deformations after peak load without significant drop in load-resisting capacity prior to collapse. The ductility of reinforced concrete members depends on the stress-strain behavior and ductility of concrete materials.

Very little information is available on complete compressive stress-strain behavior of light-weight concretes.

2.3.5.4 Shrinkage

The shrinkage of light-weight concrete is generally greater than that for normal weight concrete with similar mix proportions and consistency. Some light-weight aggregates produce concretes having relatively low shrinkage. light-weight aggregates usually give higher shrinkage because they have lower modulus of elasticity and thus produce smaller restraint against the shrinkage movements of the cement paste. Aggregates with a larger fractions of fines smaller than #200 sieve (75 microns) show higher shrinkage movement as they lead to larger void content.

Light-weight aggregate concrete exhibits higher moisture movements and a somewhat higher ultimate shrinkage when compared with normal-weight concrete. It has been reported that the shrinkage of light-weight concrete might be 6 to 38% higher than that of normal-weight concrete.

CHAPTER 3

DETERMINATION OF INFLUENTIAL VARIABLES IN THE PRODUCTION OF PLASTIC-LIGHTWEIGHT CONCRETE

3.1 INTRODUCTION AND OBJECTIVES

The main thrust in this phase of research was to check the validity of the hypotheses presented in Chapter 1 regarding microcrack arrest and deflection, and bridging of cracks by plastic inclusions in concrete. Two different types of recycled plastics were used, and the plastic content was optimized. An effort was made to modify the matrix through partial substitution of cement with fly ash in order to achieve better interface characteristics. Mechanical and physical properties of light-weight concrete incorporating recycled plastics were assessed with emphasis on flexural performance. The results obtained in this study were analyzed statistically using the analysis of variance and multiple comparison techniques in order to derive statistically reliable conclusions.

3.2 EXPERIMENTAL PROGRAM

An experimental program was conducted at this stage of research on light-weight plastic-concrete composites. This experimental program was concerned with identifying the influential variables in the production of light-weight plastic concrete materials.

The experimental program (Table 3.1) was designed based on the statistical concepts of factorial analysis of variance. The purpose of this experimental program was to investigate the following three variables: plastic type, plastic content, and composition of the cementitious paste. The composites were optimized considering their flexural performance (strength, toughness, and initial stiffness), compressive strength, and impact resistance. In Table 3.1, each "*" reflects one batch of concrete for which the standard number of specimens were tested; two batches (i.e. two replications) were considered for each mix.

TABLE 3.1 Experimental Program For Light-Weight Concrete Incorporating Recycled Plastics

Plastic Type	Replacement Level of Sand With Plastics, by volume			
	7.5%		15%	
	Paste Composition		Paste Composition	
	W/O Fly Ash	With Fly Ash	W/O Fly Ash	With Fly Ash
HDPE	* *	* *	* *	* *
MIXED	* *	* *	* *	* *
CONTROL	* *	* *		

3.3 MATERIALS AND MIX PROPORTIONS

The basic mix ingredients were Type I Portland cement, light-weight coarse aggregate, light-weight fine aggregate, recycled HDPE and MIXED plastics, Class F fly ash, water, and air-entraining agent. The chemical composition of the cement are shown in Tables 3.2. Sulfonated plastics were used in this investigation, and results are compared with some results obtained with sulfonated plastics in section 3.6.

TABLE 3.2 Chemical Compostion of Type I Cement Used in This Investigation

Chemical Composition	Percent
Tricalcium Silicate (C_3S)	43.3
Dicalcium Silicate (C_2S)	26.3
Tricacium Aluminate (C_3A)	11.0
Tetracalcium Aluminoferrite (C_4AF)	8.6
Insoluble Residue	0.12

The light-weight aggregate used in this investigation (Tufflite) was volcanic rock-based with a maximum aggregate size of 12mm (0.5 in). The specific gravity of light-weight coarse and fine aggregates were 1.2 and 1.5, respectively.

The gradation of the light-weight fine aggregates, see Figure 3.1, incorporating the two replacement levels of recycled plastics satisfied ASTM C-330 requirements for structural light-weight concrete.

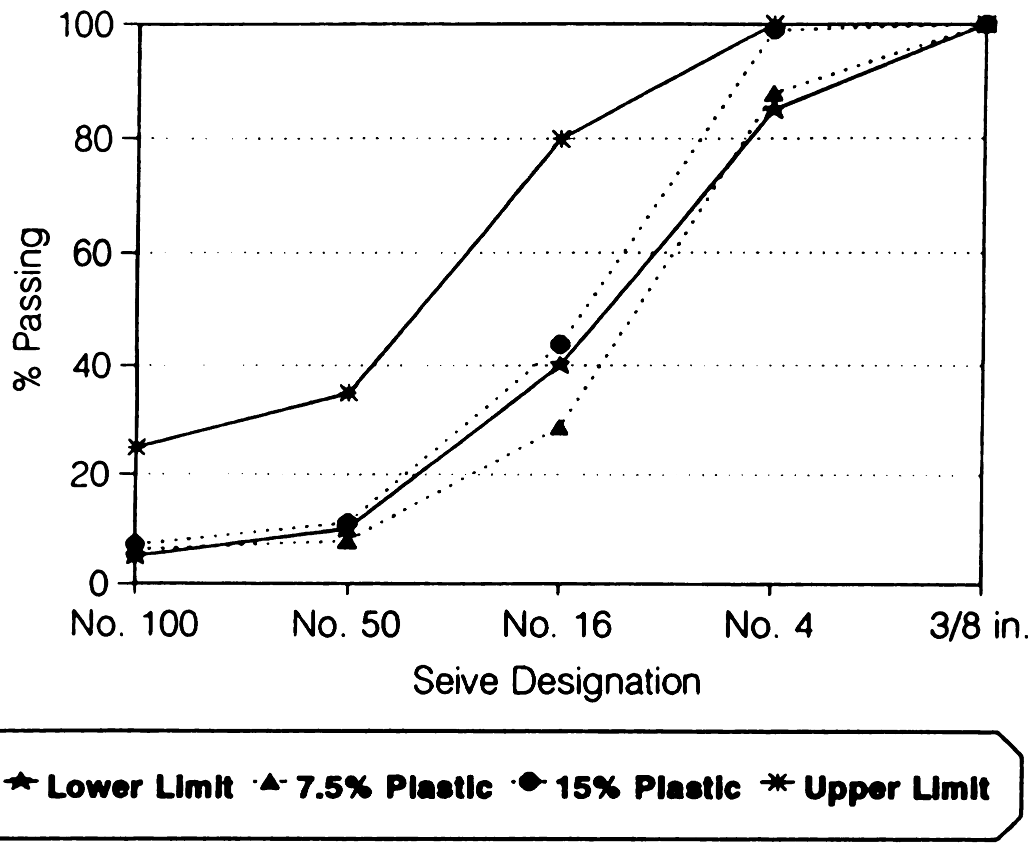


FIGURE 3.1 Light-Weight Aggregate Gradation

Recycled "MIXED" plastic, as will be referred to in this research, is a combination of high density polyethylene (HDPE), polystyrene (PS), polypropylyne (PP), polyvinyl chloride (PVC), polyethylene terephlthalate (PET), and acrylonitrile butadiene styrene (ABS), all obtained from the solid waste stream.

Table 3.3 shows the percentages, by weight, of these plastics in municipal solid waste and those used in this investigation.

TABLE 3.3 Types and Percentages of Different Plastics, by Weight

Plastic Type	MSW*	Used
HDPE	21	31
PP	16	24
PS	16	24
PVC	7	10
PET	4	6
ABS	3	5
OTHER	33	-

* MSW: Muncipal Solid Waste

Both recycled HDPE and "MIXED" plastic particles are irregular (relatively flat) in shape. The specific gravity of different types of plastics ranged from 0.9 to 1.1. Both HDPE and "MIXED" plastics have a nominal planar dimension of 10 mm (3/8 in). The average and the 95% confidence interval for the thicknesses of the recycled HDPE and "MIXED" plastic are shown in Table 3.4.

Table 3.4 Average Thickness of the Different Types of Plastics

Plastic Type	Thickness, mm (in)
HDPE	1.90 (0.075)
PP	1.60 (0.063)
PS	2.40 (0.095)
PVC	1.70 (0.067)
PET	1.04 (0.041)
ABS	2.31 (0.091)

Different trial mixtures were done to optimize the cement content and the fine aggregate/coarse aggregate ratio, in order to achieve the maximum replacement level of fine aggregates with HDPE or "MIXED" plastics. The cement content and fine aggregate/coarse aggregate ratio were 450 kg/m^3 (750

lb/yd^3) and 4 (by volume), respectively. It should be noted that a relatively high fine/coarse aggregate ratio was necessary in order to achieve desirable workability, compactability and finishability when part of the light-weight fine aggregate was replaced with HDPE or "MIXED" plastic. Both the cement content and fine aggregate/coarse aggregate ratio were kept constant in all mixtures. The water content was adjusted to give comparable slumps of 38-51 mm (1.5-2.0 in). An air entraining agent (water based) was used at 0.06% by weight of cement to produce resistance against frost attack. Table 3.5 presents the optimized mix proportions.

Table 3.5 Mix Proportions, lb/yd^3*

Matrix Comp.	Cement	Coarse Agg.	Fine Agg.	Recycled Plastic	Water	AEA %
Control	750	170	850	-	735	0.06
7.5% Plastic	750	180	719	120	698	0.06
15% Plastic	750	193	579	258	638	0.06

*1 lb/yd^3 = 0.594 kg/m^3; AEA = Air Entraining Agent, by weight of cement

3.4 TEST PROCEDURES

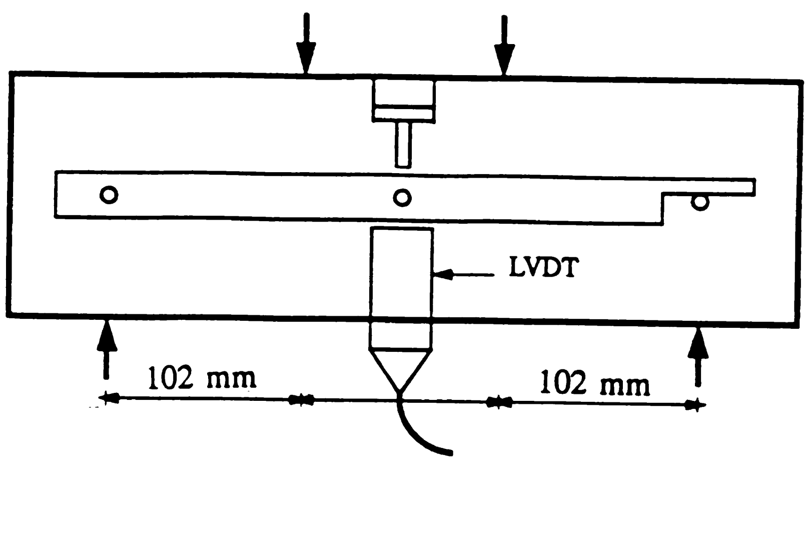
The fresh mix workability was assessed by the slump test (ASTM C-143) and the hardened unit weight was measured following the ASTM C-567 procedures.

For the hardened materials, the flexural strength and toughness, compressive strength, impact resistance and restrained shrinkage cracking

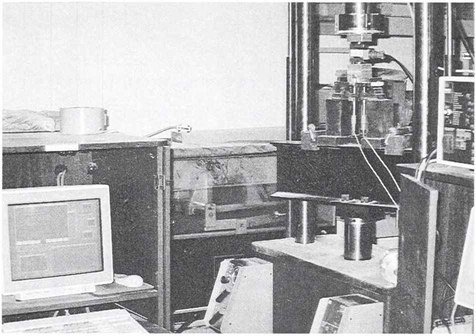
characteristics were investigated experimentally in order to develop an overall understanding of the various aspects of material behavior.

The flexural tests were conducted by four-point loading on a span of 12 in (305 mm). Deflections were measured at the center of the specimen with respect the loading point (Figure 3.2). This method of displacement measurement eliminates any errors associated with the rigid body movements of the specimen or penetration at support and loading points into the specimen.

Flexural loading was displacement-controlled at a quasi-static deflection rate of 1/1000 times the span length per minute. These flexural tests produced load-deflection curves which were characterized by flexural strength (modulus of rupture) and toughness. The Japanese Concrete Institute approach was followed for calculating flexural toughness, defined as the area underneath the flexural load-deflection curve up to a deflection equal to the span length divided by 150 (JCI-Sp, 1979).



(a) Japanese Standard Flexural Test Set-Up



(b) Photograph of Flexural Test Apparatus

FIGURE 3.2 Flexural Testing

The compressive tests were performed according to the Japanese code (JCI-SP, 1978); in this approach compressive toughness is defined as the area underneath the compressive load-deflection curve up to a strain of 0.0075.

In the flexural and compression tests, both load and deflection were monitored through about the test in order to obtain complete load-deflection relationship.

The impact test was conducted following the procedures recommended by ACI Committee 544. This test measures the amount of impact energy (represented by the number of blows) necessary to start a visible crack in the concrete incorporating recycled plastics and then to continue opening that crack until failure. The equipment for impact test (Figure 3.3) consists of a standard 4.54-kg (10-lb) compaction hammer repeatedly and recording the number of blows

required to cause the first visible crack on the specimen top surface and then failure.

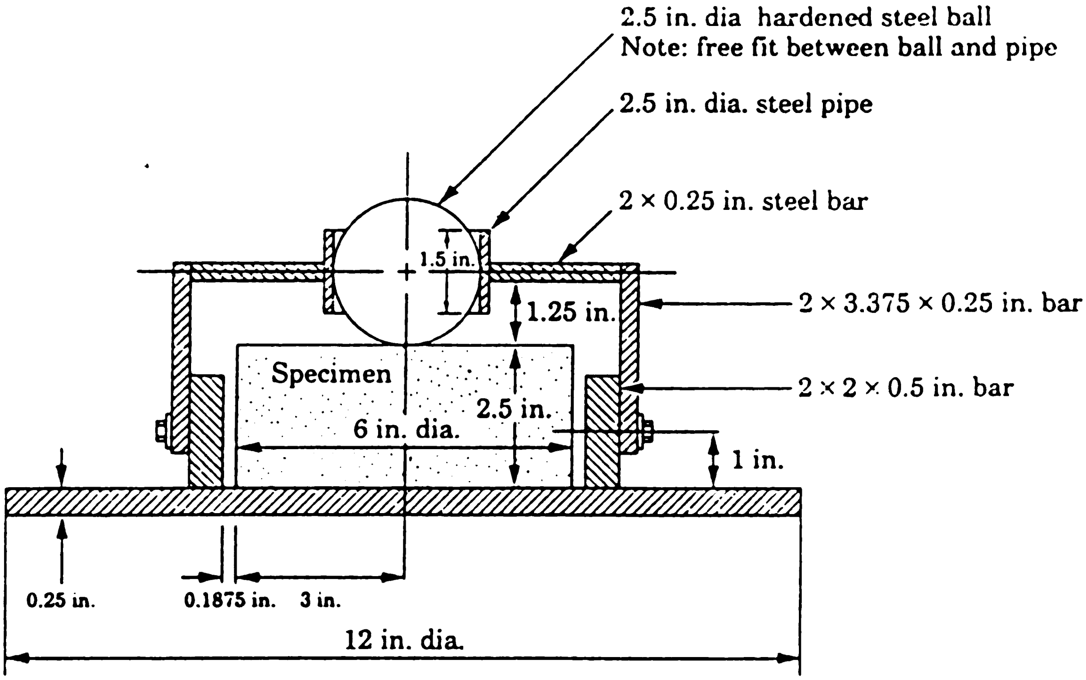


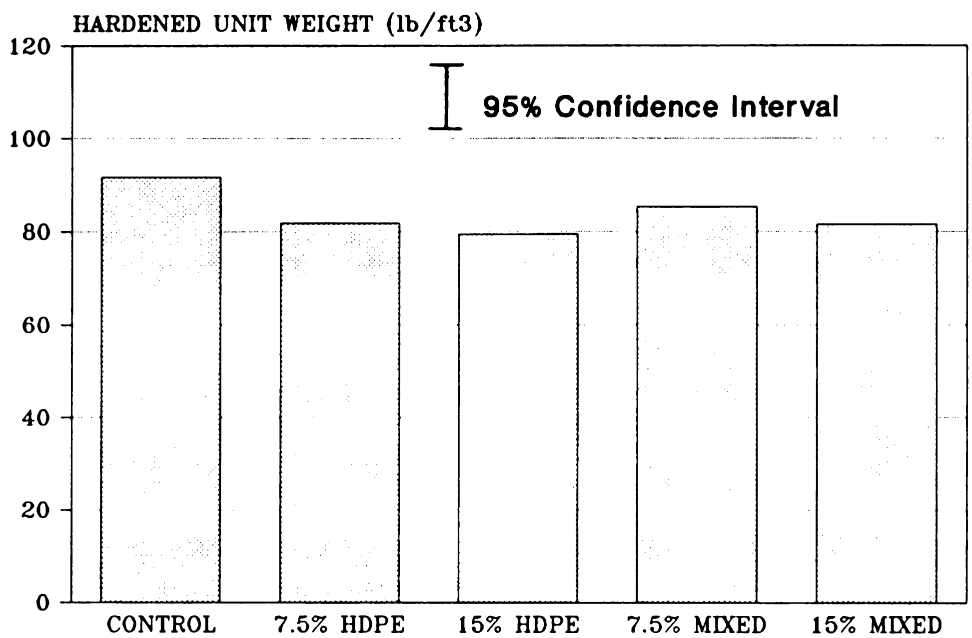
FIGURE 3.3 Impact Resistance Test Set-Up

Ring type specimens were used for restrained drying shrinkage tests on mortar. The specimen is cast in two equal layers, leveled by trowel, and then covered with plastic sheets for 6 hours. The specimen is then exposed to air at approximately 23°C (73°F) and 40% R.H. Restraint of shrinkage movements by the steel ring inside the specimen creates internal tangential tensile stresses which cause cracking.

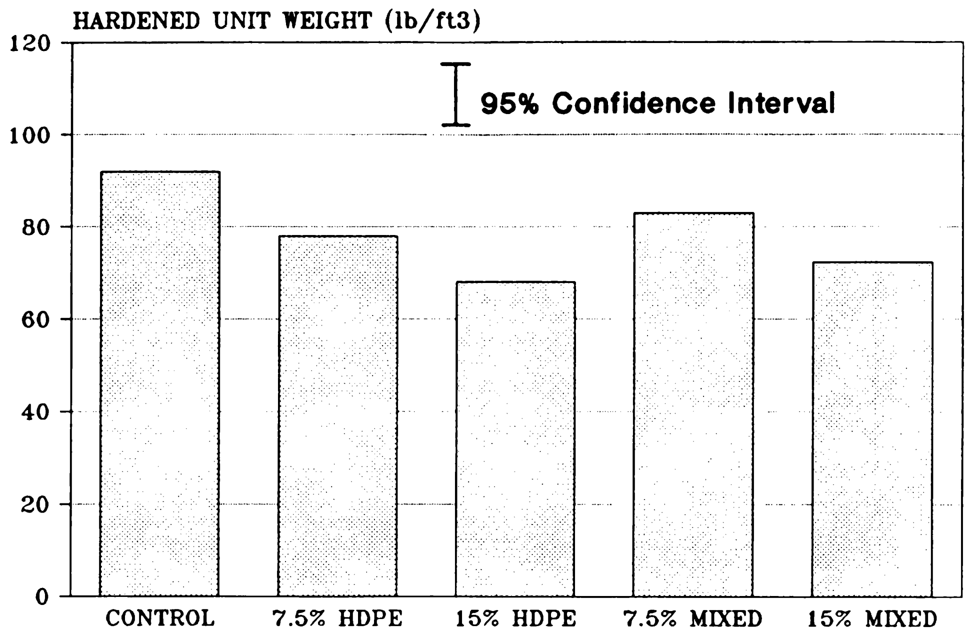
3.5 TEST RESULTS AND DISCUSSION

3.5.1 Hardened Unit Weight

The hardened unit weight test results are presented in Figure 3.4. The addition of recycled plastics tends to reduce the hardened unit weight which adds value to concrete properties. The reduction in hardened unit weight can be attributed to the fact that the light-weight sand used in this investigation had a higher specific gravity than that of the recycled plastics used.



(a) Without Fly Ash

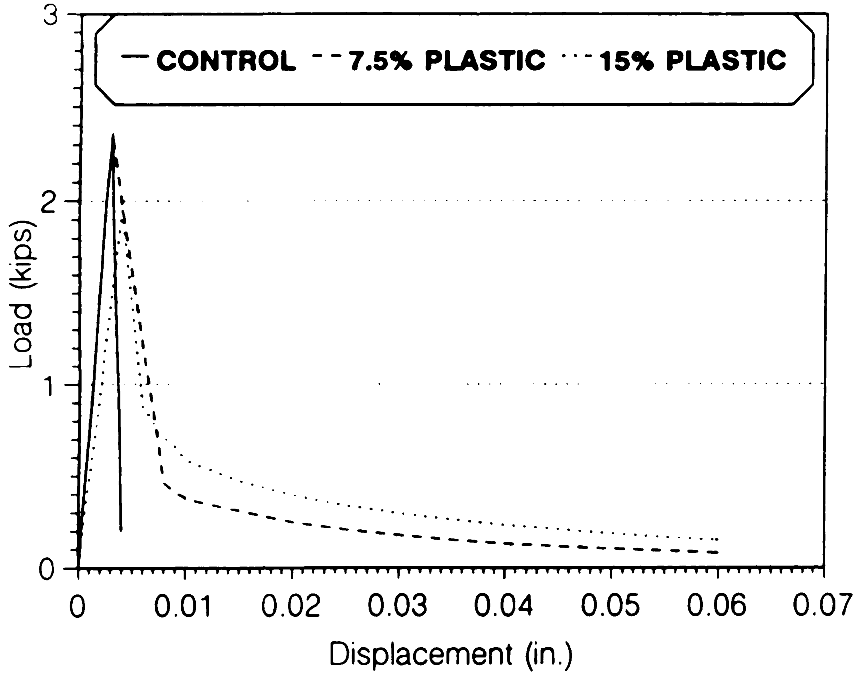


(b) With Fly Ash

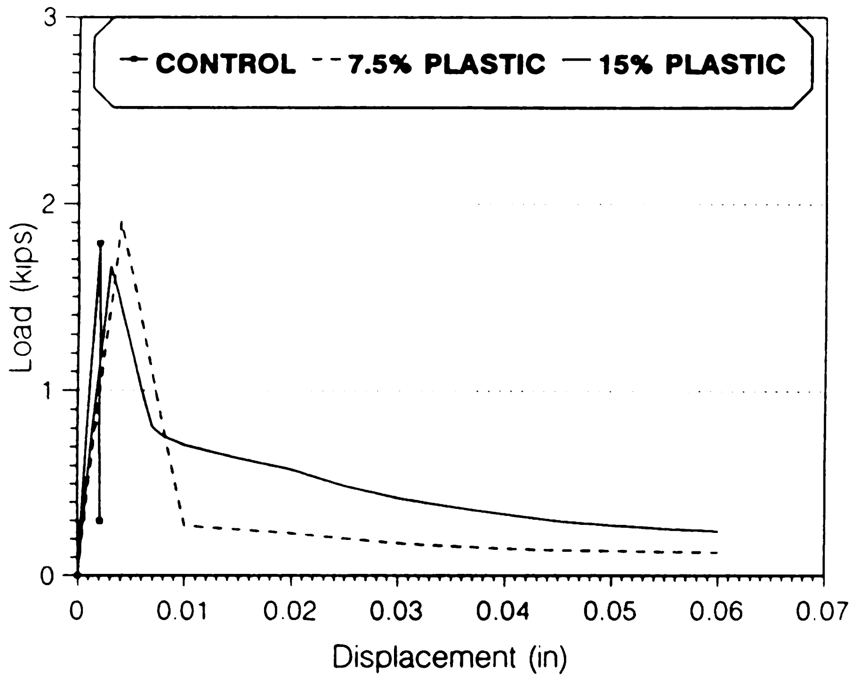
FIGURE 3.4 Hardened Unit-Weight Test Results (Means and 95% Confidence Level)

3.5.2 Flexural Performance

Typical 28-Day flexural load-deflection curves for light-weight concrete and plastic concretes incorporating 7.5%, 15% (without fly ash), and 7.5% and 15% (with fly ash) recycled plastics, respectively, are shown in Figure 3.5. Table 3.6 and Figure 3.6 present the flexural strength test results, and Table 3.7 and Figure 3.7 presents the flexural toughnesses test results.



(a) Without Fly Ash

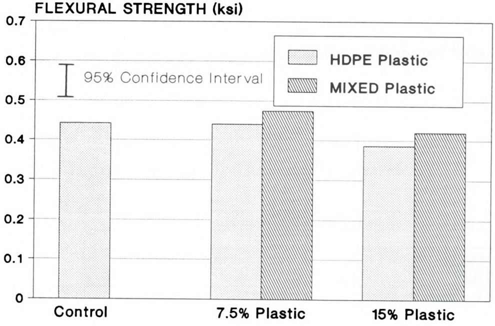


(b) With Fly Ash

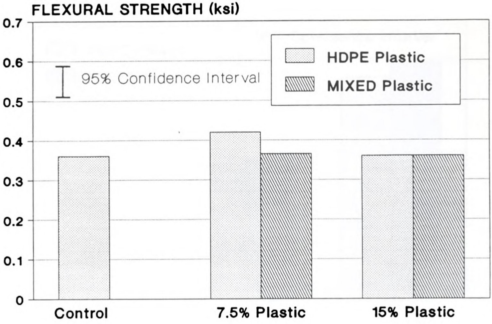
FIGURE 3.5 Typical 28-day Flexural Load-Deflection Curves

TABLE 3.6 Flexural Strength Test Results at 28 Days: Means, ksi

Plastic Type	Replacement Level of Sand With Plastics, by volume			
	7.5%		15%	
	Paste Composition		Paste Composition	
	W/O Fly Ash	With Fly Ash	W/O Fly Ash	With Fly Ash
HDPE	0.44 0.43	0.41 0.43	0.39 0.35	0.35 0.37
MIXED	0.40 0.48	0.36 0.37	0.42 0.40	0.35 0.37
CONTROL	0.44 0.43	0.35 0.37		



(a) Without Fly Ash

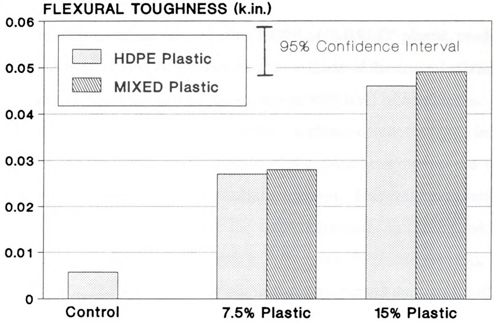


(b) With Fly Ash

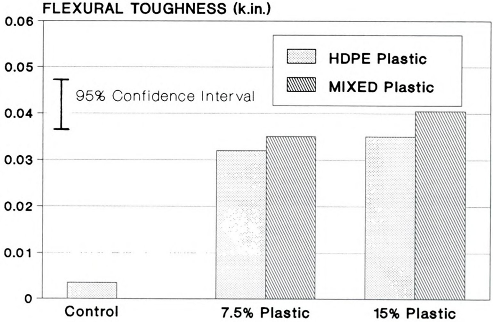
FIGURE 3.6 Flexural Strength Test Results at 28 Days (Means and 95% Confidence Interval)

TABLE 3.7 Flexural Toughness Test Results at 28 Days: Means, k.in

Plastic Type	Replacement Level of Sand With Plastics, by volume			
	7.5%		15%	
	Paste Composition		Paste Composition	
	W/O Fly Ash	With Fly Ash	W/O Fly Ash	With Fly Ash
HDPE	0.027	0.029	0.046	0.057
	0.038	0.034	0.035	0.058
MIXED	0.028	0.034	0.049	0.04
	0.025	0.036	0.043	0.041
CONTROL	0.005	0.004		
	0.006	0.003		



(a) Without Fly Ash



(b) With Fly Ash

FIGURE 3.7 Flexural Toughness Test Result at 28 Days (Means and 95% Confidence Interval)

Statistical analysis of the results through comparison of means indicated that the addition of plastics, whether HDPE of "MIXED" plastic, produce flexural strengths that are statistically comparable to those of the control mixtures without plastics. This was confirmed statistically at 95% level of confidence.

Analysis of variance showed that the plastic content, and the interaction between plastic content and composition of cementitious matrix (i. e. use of fly ash) had significant effects on flexural toughness. This was confirmed statistically at 95% level of confidence. The flexural toughness increased 4.5 and 8 times that of the control light-weight concrete at 7.5% and 15% plastic contents, respectively. The addition of fly ash further pronounced the positive effects of plastics on flexural toughness. The positive effects of fly ash can be attributed to their capability to enhance the structure and properties (bonding) at the transition zone between plastics and cement-based matrix.

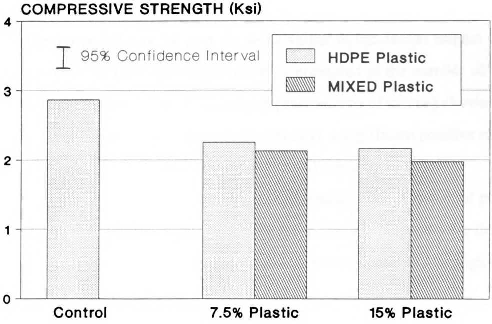
In general, the positive effects of plastics on flexural toughness reflect their capability to bridge cracks and mitigate brittle modes of failure in concrete materials by their pull-out resistance across cracks.

3.5.3 Compressive Behavior

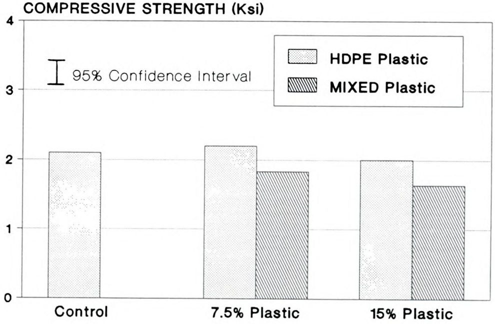
The compressive strength test results (means and 95% confidence intervals) for different plastic light-weight concretes at 28 days are shown in Table 3.8 and Figure 3.8. It was confirmed statistically, at 95% level of confidence, that plastics had adverse effects on compressive strength when fly ash was not used. However, comparison of means of the compressive strength test results showed that 7.5% plastics (HDPE or "MIXED") in the presence of fly ash produced compressive strength comparable, at 95% level of confidence, with that of the control fly ash concrete. At 15% plastic contents, HDPE with fly ash also produced a compressive strength comparable to that of the control fly ash concrete. The general drop in compressive strength with the addition of plastics may be attributed to the relatively low modulus of elasticity of plastics which leads to a redistribution of stresses into the more rigid inorganic matrix. It should, however, be noted that limits on load-carrying capacity and service life of concrete structures are generally provided by the resistance of concrete to tensile stresses and impact loads. Concrete is fairly strong in compression and concrete structures rarely fail due to material failure in compression. In the presence of fly ash, one may expect that plastic-concrete composites may reach compressive strengths approaching that of control concrete.

TABLE 3.8 Compressive Strength Test Results at 28 Days: Means, k.in

Plastic Type	Replacement Level of Sand With Plastics, by volume			
	7.5%		15%	
	Paste Composition		Paste Composition	
	W/O Fly Ash	With Fly Ash	W/O Fly Ash	With Fly Ash
HDPE	2.25 2.36	2.22 2.18	2.13 1.57	1.89 1.78
MIXED	2.16 2.14	2.05 1.96	1.97 1.87	1.62 1.67
CONTROL	2.87 2.90	2.08 2.12		



(a) Without Fly Ash



(b) With Fly Ash

FIGURE 3.8 Compressive Strength Test Results at 28 Days (Means and 95% Confidence Interval)

3.5.4 Impact Resistance

Table 3.9 and Figure 3.9 give the mean values of the 28-day impact resistance test results for light-weight concrete, presented as the number of blows to first crack and failure. Statistical analysis (comparison of means) showed, at 95% level of confidence, that recycled plastics have a significant positive effect on the impact resistance of concrete beyond the initial crack up to failure.

The improvements in ultimate impact resistance in the presence of plastics further validate the hypothesis that tough plastic inclusions help enhance the fracture energy and toughness characteristics of concrete materials through bridging across cracks.

TABLE 3.9 Impact Resistance Test Results at 28 Days: Means (# of Blows)

Plastic Type	Replacement Level of Sand With Plastics, by volume							
	7.5%				15%			
	Paste Composition				Paste Composition			
	W/O Fly Ash		With Fly Ash		W/O Fly Ash		With Fly Ash	
	Initial	Final	Initial	Final	Initial	Final	Initial	Final
HDPE	5.8	8.0	5.8	8.0	6.6	9.4	8.8	12.2
MIXED	7.4	10.0	3.4	5.4	5.4	9.0	5.4	7.2
CONTROL	5.8	6.4	4.2	4.4				

3.5.5 Restrained Drying Shrinkage

Figure 3.10 shows the crack width versus time in restrained shrinkage test on light-weight concrete. Figure 3.11 compares the shrinkage cracking conditions of control and plastic concretes. The addition of recycled plastics to light-weight concrete helps control the drying shrinkage cracks. This can be attributed to the fact that recycled plastics (HDPE or "MIXED") act as reinforcing inclusions that arrest microcracks and bridge across cracks to restrain their widening.

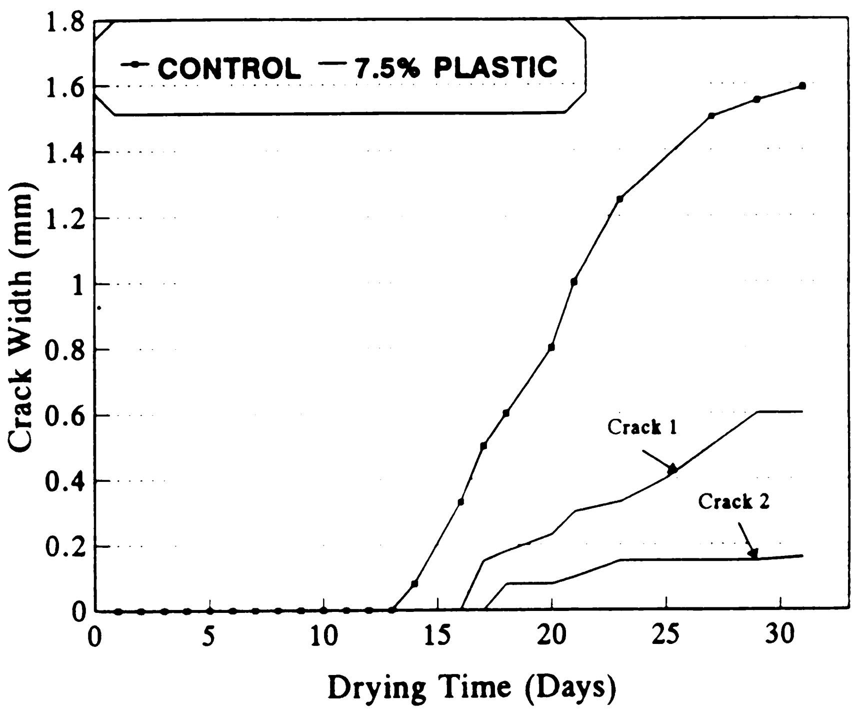
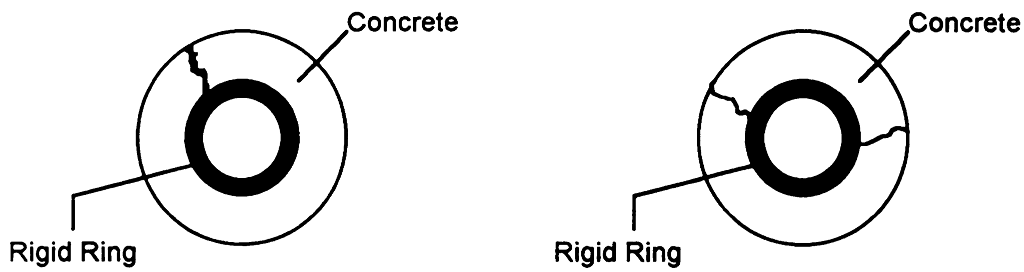


FIGURE 3.10 Crack Width Vs. Drying Time

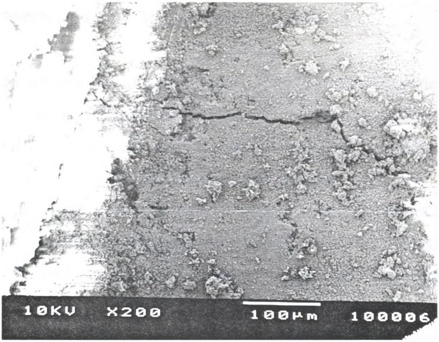


(a) Control Concrete (b) Plastic Concrete

FIGURE 3.11 Shrinkage Cracking Conditions

3.5.6 Microstructural Observations

In order to confirm the crack arrest and bridging mechanisms of plastic inclusions in concrete, fracture surfaces of flexural specimens were observed under a Scanning Electron Microscope. A typical arrest of microcracks by plastics is shown in Figure 3.12a (at 200x magnification), and Figure 3.12b (at 50x magnification) presents a typical condition of plastic inclusions bridging cracks. These observations further validate the hypotheses of this investigation.



(a) Arrest of Microcrack

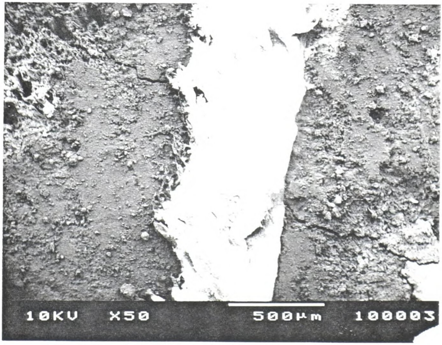
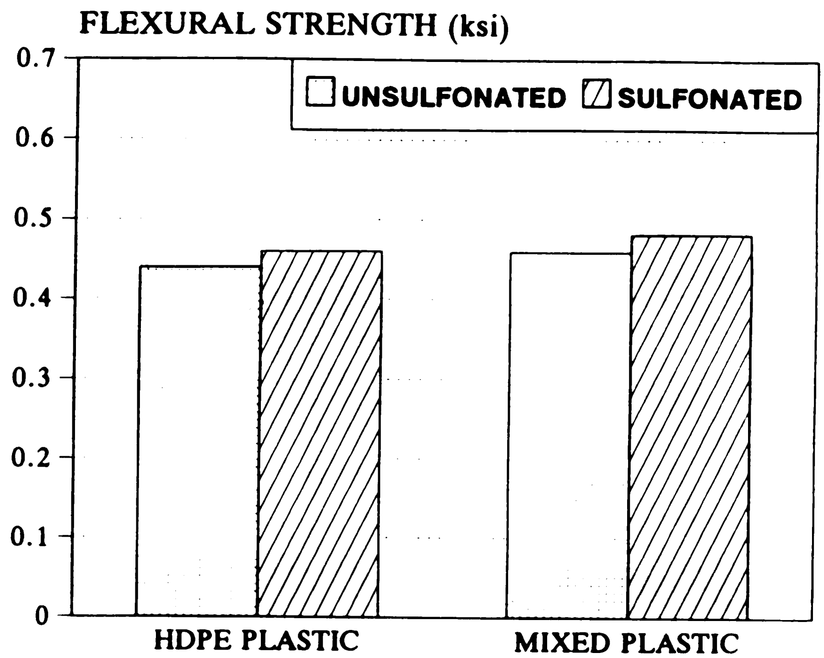


FIGURE 3.12 SEM Micrographs of Plastic-Concrete Composites

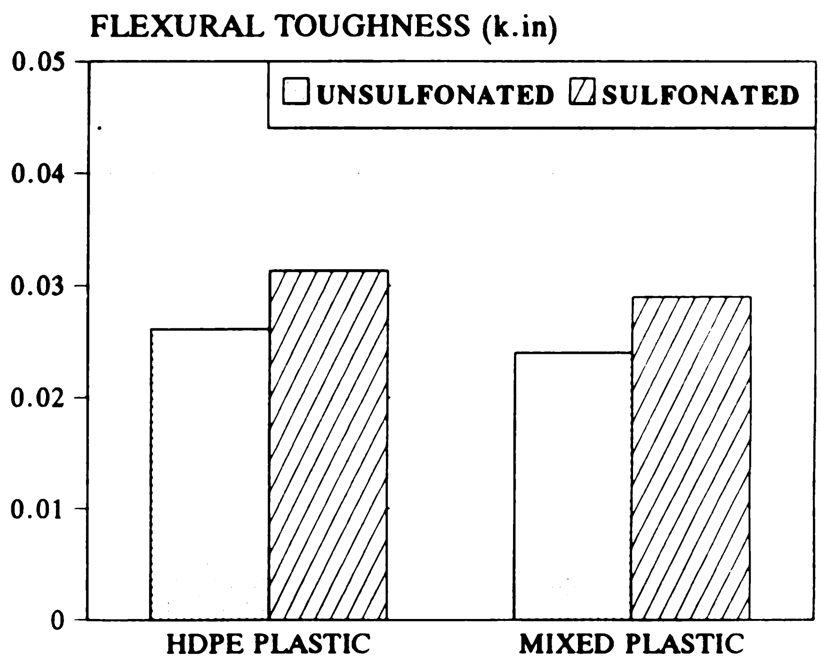
3.6 ASSESSMENT OF THE SULFONATION EFFECTS

In order to further investigate the effects of sulfonation, some critical tests were performed on specific concrete mixtures incorporating unsulfonated recycled plastics. These results are compared in this section with those of corresponding concrete mixtures incorporating sulfonated recycled plastics.

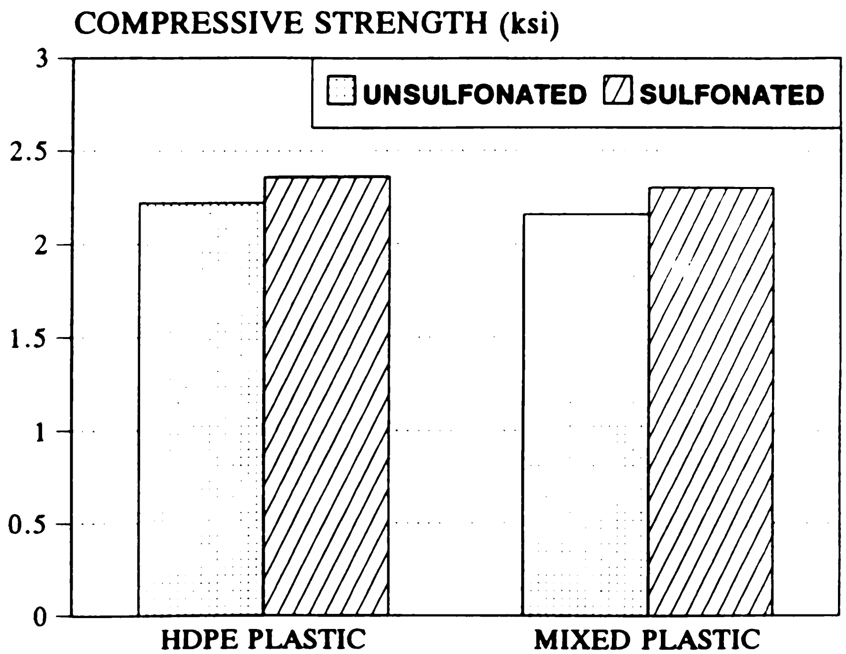
For the mix with 7.5% HDPE without fly ash, as shown in figures 3.13a and 3.13b, sulfonation of the recycled plastics helped increase both flexural strength and toughness by 2 and 65%, respectively. This increase was due to the improved bond between the cementitious matrix and recycled plastic inclusion. However, the addition of sulfonated recycled plastics in light-weight concrete produced comparable compressive strength and impact resistance to those with unsulfonated recycled plastics (see Figure 3.13c and d). It is important to note that increased level of sulfonation of the recycled plastics may show some improvements in some other mechanical properties.



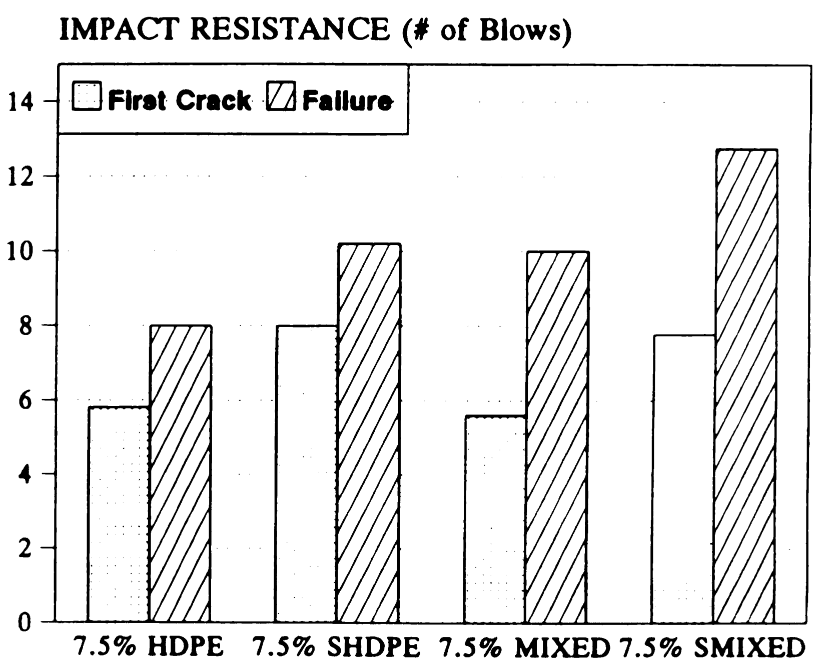
(a) Flexural Strength, ksi



(b) Flexural Toughness, k.in



(c) Compressive Strength, ksi



(d) Impact Resistance (# of Blows)

FIGURE 3.13 Assessment of The Sulfonation Effects

3.7 SUMMARY AND CONCLUSIONS

The effects of partial substitution of light-weight aggregates with recycled plastics on concrete properties were investigated. Two plastic types (HDPE and "MIXED"), two levels of replacement of fine aggregate (7.5% and 15% plastics by total volume of concrete) were used, and two paste compositions (without and with fly ash) were considered.

The hardened material mechanical properties were assessed through flexure, impact, compression and restrained drying shrinkage tests. The following conclusions were derived through analyses of the generated data:

1. The addition of recycled plastics to light-weight concrete helps reduce the drying shrinkage crack widths.

2. Recycled plastics at 7.5 and 15% volume fractions produce flexural strengths comparable to that of control concrete mix. However, flexural toughness increases by 4.5 and 8 times, respectively. This was confirmed statistically at 99% level of confidence.

3. Compressive strength test results were indicative of the adverse effects of recycled plastics on compressive strength; however, the addition of fly ash to the cementitious matrix produced compressive strengths comparable to control mixtures. It should be noted that concrete is fairly strong in compression, and brittle failure mode and low crack resistance are the main problems which are targeted to be overcome through the use of reinforcing inclusions. Furthermore, since the reduction in compressive strength is

accompanied with the reduction in unit weight, the situation would be improved if one looks at the compressive strength-to-weight ratio.

4. Recycled plastics have a significant positive effect on the impact resistance of concrete beyond the initial crack up to failure.

5. Although some of the flexural performance of the plastic light-weight concrete incorporating sulfonated recycled plastics improved in some cases, the overall mechanical properties (i.e. compressive strength and impact resistance) produced test results that are comparable to these that were unsulfonated. It is important to note that by increasing the level of sulfonation one may see some improvement in these mechanical properties.

CHAPTER 4

OPTIMIZATION OF INFLUENTIAL VARIABLES

4.1 INTRODUCTION

The only influential variable identified in the previous phase of the study (plastic content) was selected to be optimized for the production of light-weight concrete incorporating sulfonated recycled plastic flakes. The composites were optimized considering their flexural performance (strength and toughness), compressive strength, and post-cracking impact resistance i.e. the difference between the number of blows to initial cracking and final break).

The optimized plastic light-weight concrete composites identified in this phase of research were produced with sulfonated recycled plastic flakes, and their mechanical properties (flexural performance, compressive strength, and impact resistance) where compared with control mixtures (i.e. without plastics).

4.2 OPTIMIZATION EXPERIMENTAL PROGRAM

The experimental program for optimization is presented in Table 4.1. Various levels of the influential variable (plastic content) are considered in this experimental program for the production of plastic light-weight concrete.

TABLE 4.1 Optimization Experimental Program

Experiment #	Plastic Content, % by total volume
1	7.5
2	11.25
3	15.0
4	0

For each of the mixtures presented in table 4.1, two replications (i.e. two batches) in two blocks (i.e. with two replications with HDPE and two with "MIXED" plastics) were considered. It should be noted that HDPE and "MIXED" plastics have been observed to perform similarly in concrete. For each batch of concrete the standard number of specimens specified by ASTM were tested. The binder in those experiments was Type I Portland cement.

Materials, production and curing procedures were the same as these presented in Chapter 3.

4.3 TEST RESULTS AND ANALYSIS

Typical flexural load-deflection curves produced for the light-weight concrete incorporating recycled plastics (at different plastic contents) are shown in Figure 4.1. Flexural strength and toughness test results are shown in Table 4.2, and Table 4.3 shows the compressive strength and post-cracking impact resistance. Figure 4.2 presents the test results and regression curves indicating the effects of plastic content on various material properties.

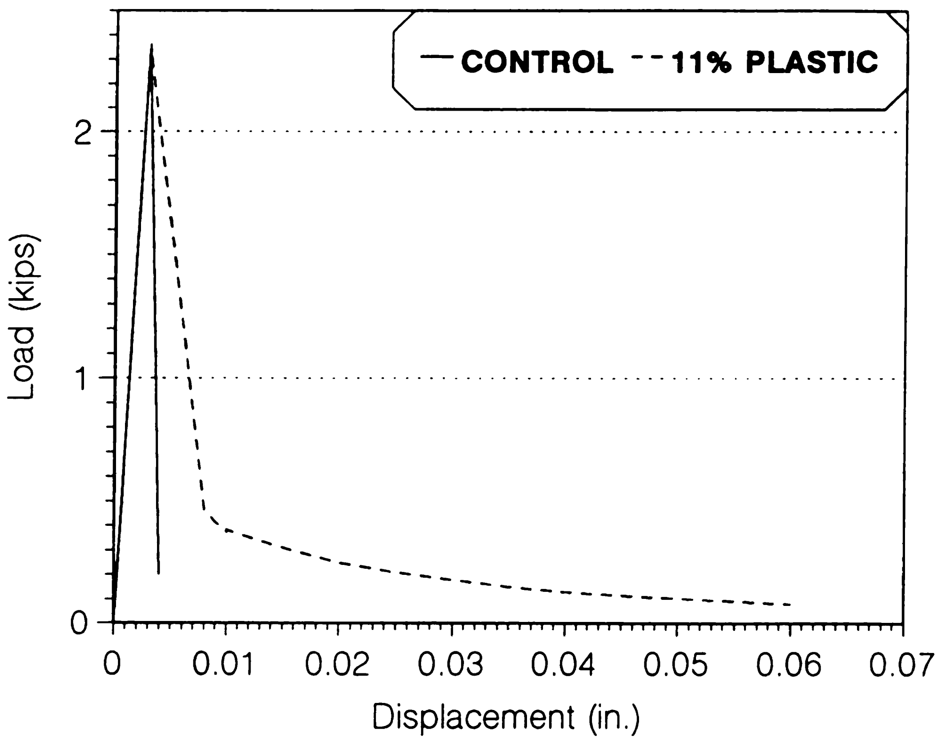


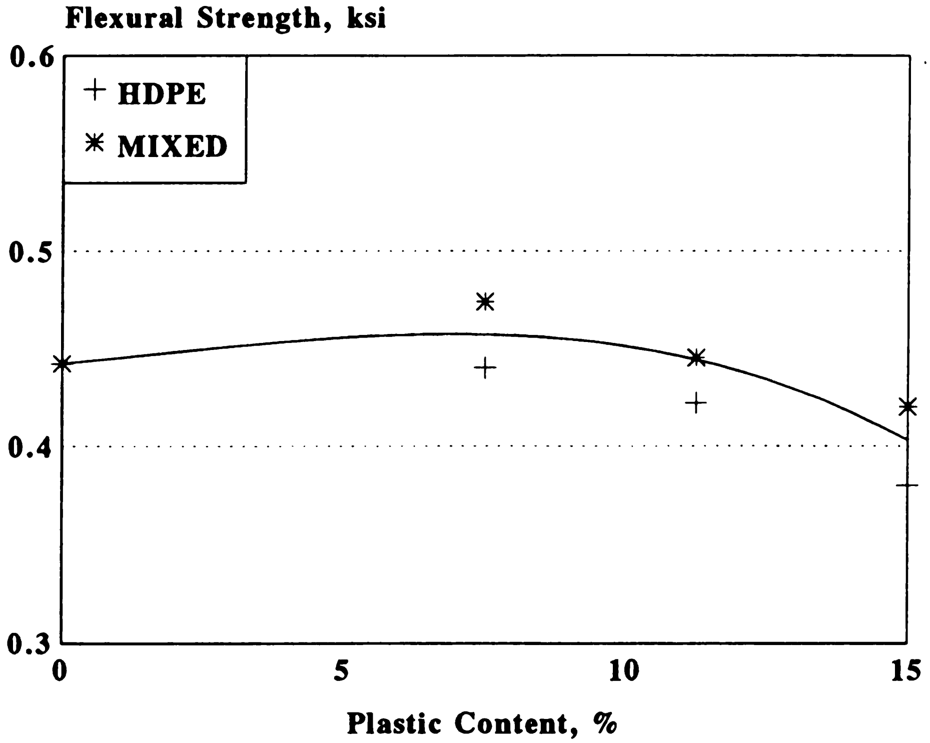
FIGURE 4.1 Typical Flexural Load-Deflection Curves

TABLE 4.2 Flexural Performance

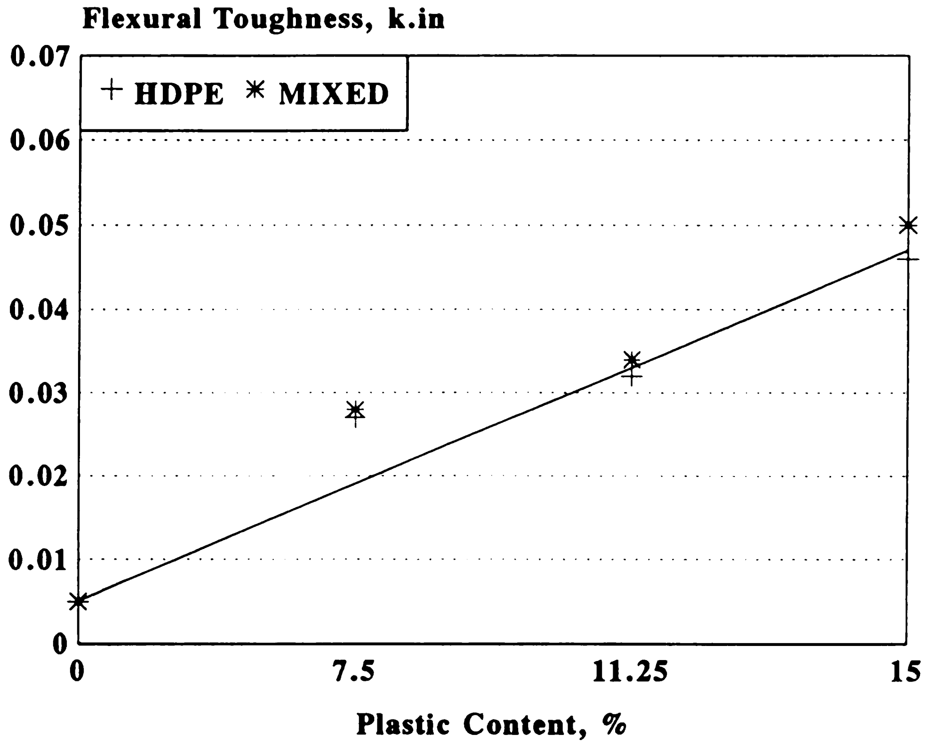
Experiment #		Flexural Strength (ksi)	Mean Flexural Strength	Flexural Toughness (k.in)	Mean Flexural Toughness
1	HDPE (1)	0.467 0.422 0.430	0.460	0.025 0.027 0.026	0.028
	HDPE (2)	0.443 0.474 0.425		0.031 0.033 0.026	
	MIXED (1)	0.503 0.480 0.439		0.026 0.028 0.023	
	MIXED (2)	0.511 0.488 0.446		0.022 0.027 0.024	
2	HDPE (1)	0.438 0.437 0.402	0.432	0.031 0.035 0.030	0.035
	HDPE (2)	0.398 0.440 0.408		0.023 0.034 0.043	
	MIXED (1)	0.461 0.428 0.447		0.046 0.042 0.040	
	MIXED (2)	0.441 0.442 0.440		0.038 0.033 0.029	
3	HDPE (1)	0.406 0.385 0.369	0.371	0.046 0.048 0.047	0.048
	HDPE (2)	0.360 0.363 0.435		0.043 0.051 0.049	
	MIXED (1)	0.431 0.401 0.454		0.049 0.046 0.051	
	MIXED (2)	0.429 0.433 0.412		0.048 0.049 0.053	
CONTROL		0.467 0.456 0.0.41	0.442	0.003 0.006 0.006	0.005

TABLE 4.3 Compressive Strength and Impact Resistance Test Results

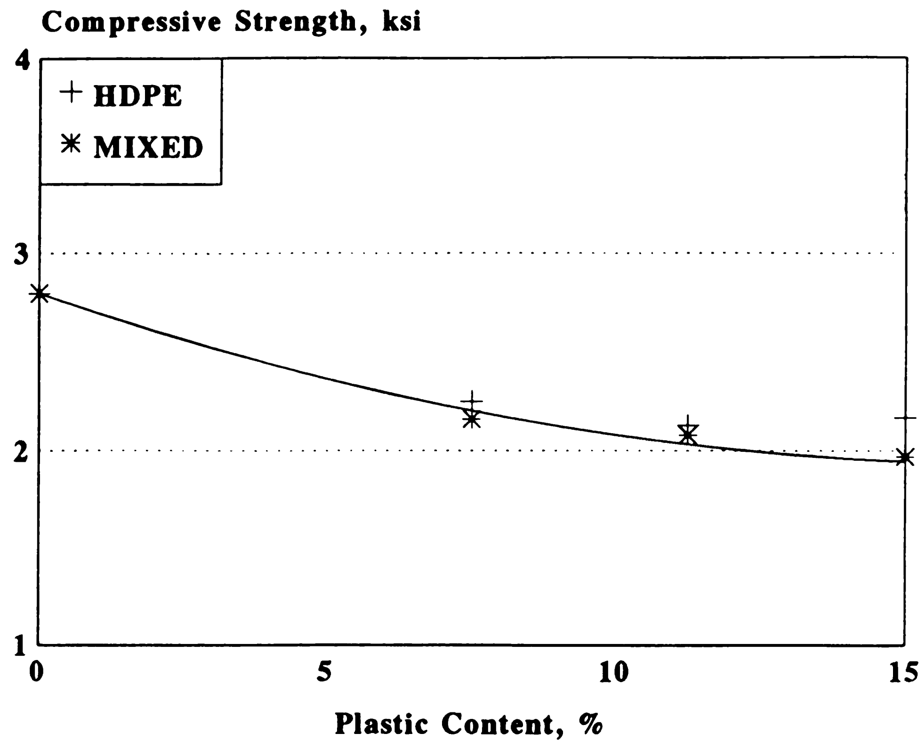
Experiment #		Compressive Strength (ksi)	Mean Compressive Strength	Diff. between Initial and Final Crack (# of blows)	Mean Diff. bet. Initial and Final Crack
1	HDPE (1)	2.27 2.22 2.20	2.26	2,3,2,1,3	2.4
	HDPE (2)	2.28 2.10 2.15		3,2,2,2,2	
	MIXED (1)	2.22 2.15 2.13		4,3,2,2,2	
	MIXED (2)	2.33 2.27 2.34		2,2,2,3,1	
2	HDPE (1)	2.15 1.96 2.06	2.19	4,3,2,3,5	3.15
	HDPE (2)	2.03 2.00 2.11		2,6,4,3,3,	
	MIXED (1)	2.25 2.26 2.24		3,3,2,2,1	
	MIXED (2)	2.75 2.25 2.315		3,3,3,4,4	
3	HDPE (1)	2.15 2.09 2.10	1.78	1,3,2,6,2	4.3
	HDPE (2)	1.52 1.60 1.58		6,6,5,5,5	
	MIXED (1)	1.84 1.94 2.14		6,3,3,4,2	
	MIXED (2)	1.56 1.36 1.56		6,6,5,5,5	
CONTROL		3.08 2.74 2.80	2.87	0,0,0,1,1	0.6



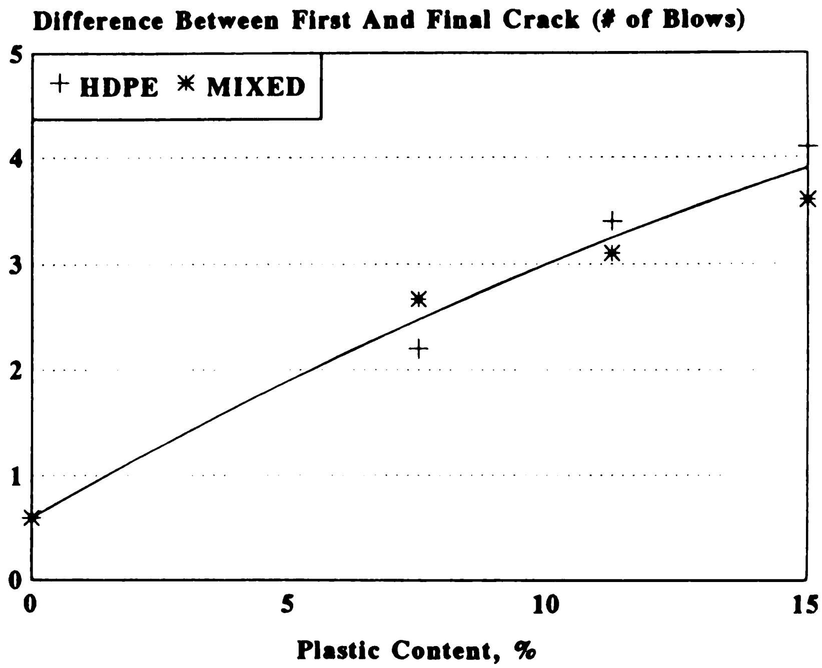
(a) Flexural Strength, ksi



(b) Flexural Toughness, k.in



(c) Compressive Strength, ksi



(d) Difference Between First and Final Crack in Impact Resistance

FIGURE 4.2 Regression Curves for Different Light-weight Concrete Mixtures

The results presented in Figure 4.2 suggest that increasing plastic content leads to improved flexural toughness and post-cracking impact resistance but damages compressive strength. These trends were confirmed statistically, through analysis of variance, at 95% level of confidence (in the case of compressive strength, the increase in plastic content beyond 7.5% up to 15% did not cause any statistically significant drop at 95% level of confidence). Plastic contents exceeding 11% started to also damage flexural strength.

The optimum plastic content would depend on the importance of different properties for various applications. If one can tolerate a drop of roughly 25% in compressive strength, it seems that approximately 11% plastic content can provide major improvements in flexural toughness and post-cracking impact resistance without any adverse effects on flexural strength.

4.4 EVALUATION OF THE OPTIMIZED COMPOSITE

Flexural performance, compressive strength and impact resistance tests were carried out on the optimized composite with 11% of HDPE or "MIXED" plastics. The optimized plastic light-weight concrete mixtures were evaluated versus control light-weight concrete mixtures (no plastics).

4.4.1 FLEXURAL PERFORMANCE

The flexural load-deflection curves for the plastic light-weight concretes are presented in Figure 4.3. Analysis of variance, Table 4.4, shows that light-weight concrete composites with 11% plastic content produced flexural strengths statistically comparable to those of control mixtures (without plastic); however,

flexural toughness increased by about 6 times when compared with control mixtures (see Figures 4.4 and 4.5). This was confirmed at 95% level of confidence.

Table 4.4 Results of the Analysis of Variance (Flexural Strength and Toughness)

Flexural Strength					
	Sum of Squares	DF	Mean-Square	F-Ratio	P-Value
	0.000705	1	0.000705	1.691312	0.22260
Error	0.004170	10	0.000417		
Flexural Toughness					
	0.000010	1	0.000010	0.353388	0.565408
Error	0.000285	10	0.000029		

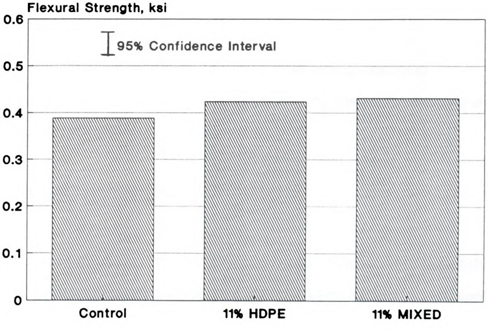


FIGURE 4.4 Flexural Strength Test Results (Means and 95% Confidence Intervals)

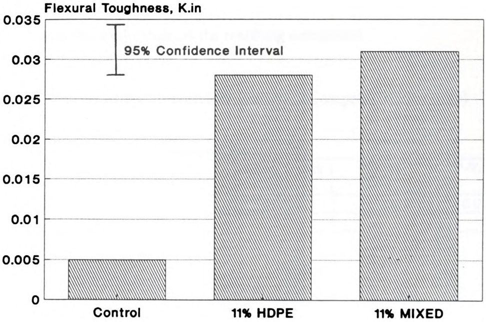


FIGURE 4.5 Flexural Toughness Test Results (Means and 95% Confidence Intervals)

The higher flexural toughness of plastic light-weight concretes incorporating sulfonated recycled plastic flakes at 11% (optimized content) shows the reinforcement efficiency of the plastic flakes.

4.4.2 COMPRESSIVE STRENGTH

Optimized composites with sulfonated recycled plastic flakes are observed in Figure 4.6 to show reduced compressive strength when compared with control composites (i.e. no plastics). This was confirmed statistically at 95% level of confidence. The reduction of compressive strength was about 25%. Table 4.5 shows the results of the analysis of variance. It is important to note that concrete

rarely fails in compression, thus this drop in compressive strength can be tolerated pending the specific application of the resulting composite.

Table 4.5 Results of the Analysis of Variance (Compressive Strength)

Compressive Strength					
	Sum-of-Squares	DF	Mean-Square	F-Ratio	P-Value
	0.257	1	0.257	11.362	0.007
Error	0.226	10			

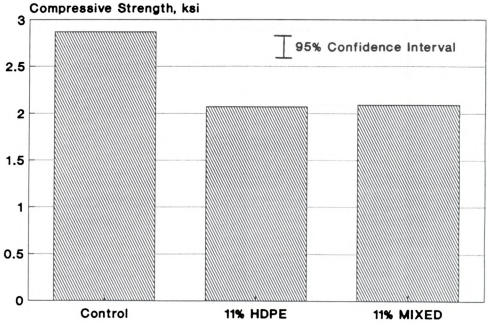


FIGURE 4.6 Compressive Strength Test Results (Means and 95% Confidence Intervals)

The drop in compressive strength of plastic light-weight concrete can be due to the addition of soft inclusions in a rigid inorganic matrix (see Chapter 5).

4.4.3 IMPACT RESISTANCE

Figure 4.7 shows the post-cracking impact resistance test results (presented as the number of blows between initial cracking and failure). The addition of sulfonated recycled plastic flakes at 11% (optimum content) causes major improvements in the post-cracking impact resistance of the resulting composite. This can be attributed to the reinforcing action of the tough plastic flakes when incorporated in a brittle cementitious matrix.

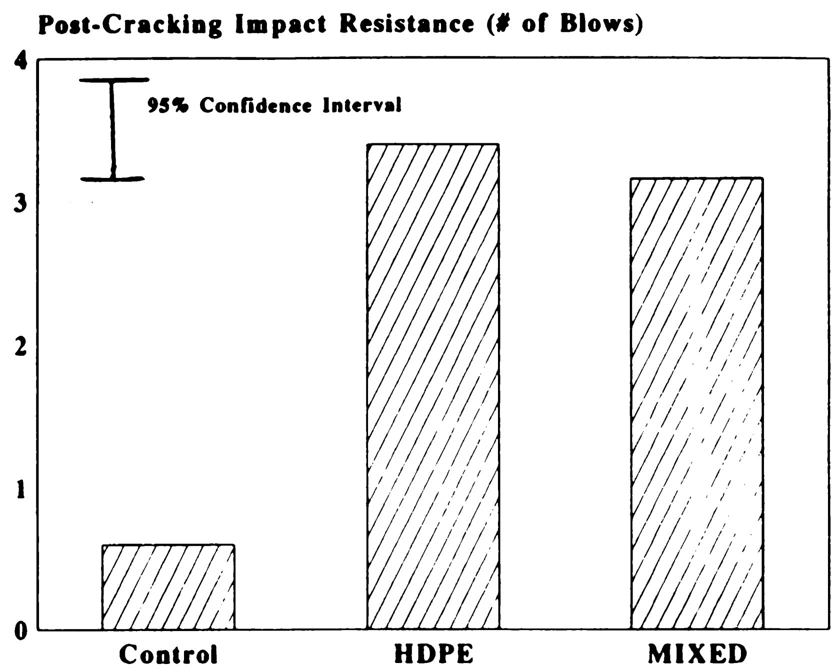


FIGURE 4.7 Impact Resistance Test Results

4.5 SUMMARY AND CONCLUSIONS

The influential variable (plastic content) in the production of light-weight concrete incorporating recycled plastics was optimized. Optimization was based on flexural strength and toughness, compressive strength, an impact resistance with sulfonated plastics. The optimized mixture was then compared to the control concrete mixture (i.e. without plastics). The following conclusions could be derived:

(1) Increasing the plastic content leads to improved flexural toughness and post-cracking impact resistance; but adversely influences the compressive strength; excess plastic contents may also damage flexural strength; and

(2) The optimum plastic content would depend on the importance of different properties for various applications; if one can tolerate a drop of 25% in compressive strength, when compared to control mixtures, 11% plastic content (by total volume) will improve flexural toughness (about 6 times) and post-cracking impact resistance (about 3 times) without adversely influencing flexural strength, this was confirmed statistically at 95% level of confidence.

CHAPTER 5

THEORETICAL MODELING

5.1 INTRODUCTION

The drop in compressive strength and elastic modulus are the key adverse effects of plastics in light-weight concrete. Due to their low elastic modulus, plastic inclusions tend to redistribute compressive stresses to the stiffer concrete matrix; they also act as stress risers producing sharp local increases in stress values. The nature of plastic-concrete interface could also be influential. While the crack-arrest action of plastics is also expected to play a role in compression, the work reported herein intends to partially understand the role plastic inclusions play under compression through modeling of plastic-concrete composites. Linear elastic finite element analysis techniques were used to investigate stress distributions and deformations in plastic-concrete composites subjected to compressive stresses. The effects of plastic content and interface conditions on compressive performance were investigated.

"ANSYS" which is a computer program for finite element analysis and design was used in numerical analysis of plastic-concrete composites

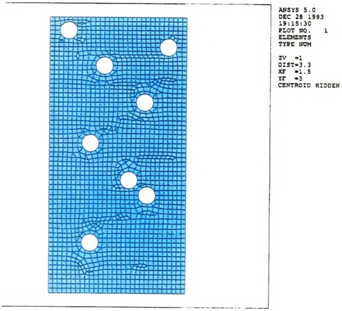
under compression. The maximum compressive stresses, maximum tensile stresses and average displacements of different concretes were obtained through "ANSYS" while the modulus of elasticity of the composites were computed numerically.

5.2 MODELING UNDER COMPRESSION

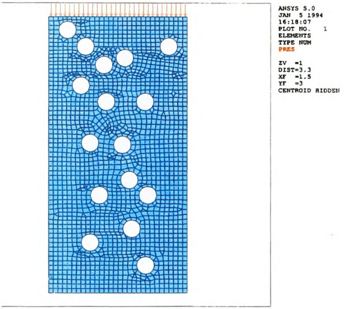
Two different inclusions (plastics and empty void), two different volume fraction of inclusions (7.5% and 15%), and two different interface conditions (perfect and imperfect bonds), were investigated. A two-dimensional elastic finite element analysis on a plate 152 mm (6.0 in) high and 305 mm (12 in) wide subjected to a compressive stress of 7 MPa (1000 psi) was performed; the inclusions were assumed circular with a diameter of 10 mm (0.375 in), and the imperfect coating was represented by a 1.00 mm (0.04 in) thick interface zone with lower modulus and higher Poisson's ratio than concrete matrix. The key properties used for the plastic inclusions, the bulk concrete matrix and the interface zone (in case of imperfect bond) are presented in Table 5.1. A control plate (without any soft inclusions) was also considered. At each volume fraction, the inclusions (voids or plastics) were distributed randomly in the plane (see Figure 5.1)

TABLE 5.1 Material Properties

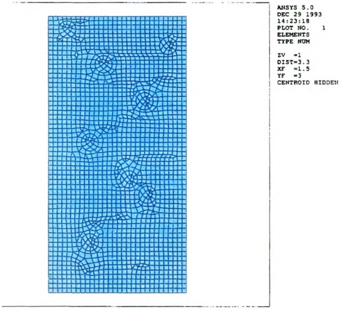
Material	Modulus of Elasticity (10^7), psi	Poisson's ratio
Light-Weight Concrete	2.6	0.3
Plastic Particles	0.02	0.45
Coating Material	1.31	0.35



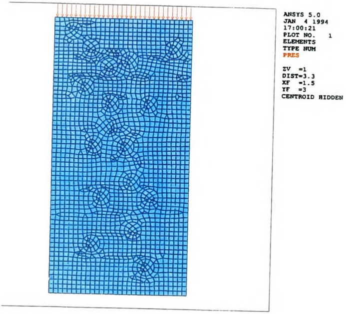
(i) 7.5% Volume Fraction



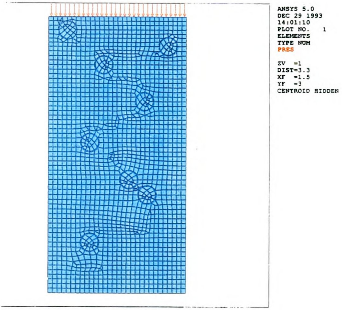
(ii) 15% Volume Fraction
(a) Empty Voids



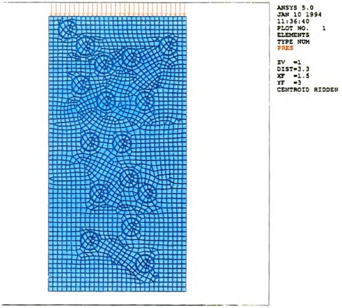
(i) 7.5% Volume Fraction



(ii) 15% Volume Fraction
(b) Plastics With Perfect Bond



(i) 7.5% Volume Fraction

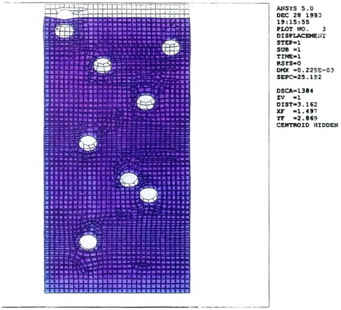


(ii) 15% Volume Fraction
c) Plastics With Imperfect Bond (i.e. coated)

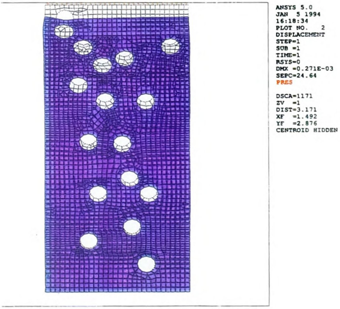
FIGURE 5.1 Geometric Configurations

5.3 RESULTS AND DISCUSSION

Figures 5.2, 5.3 and 5.6 present the deformations, longitudinal stresses (in the direction of compressive loading) and transverse stresses, respectively, for different modeling conditions.

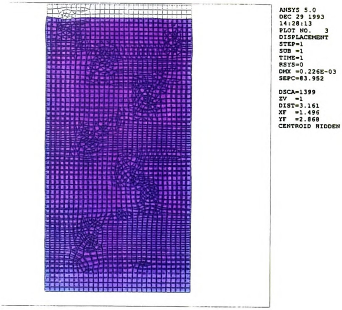


(i) 7.5% Volume Fraction

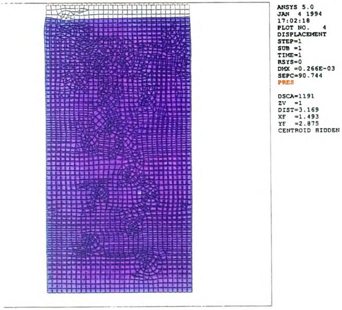


(ii) 15% Volume Fraction

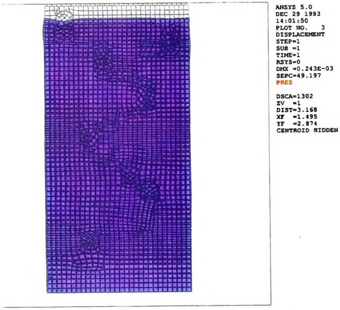
(a) Empty Voids



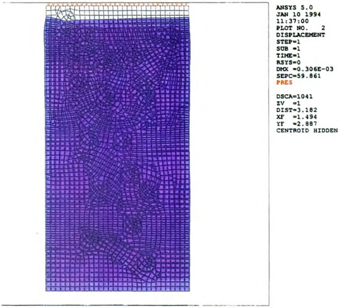
(i) 7.5% Volume Fraction



(ii) 15% Volume Fraction
(b) Plastics With Perfect Bond



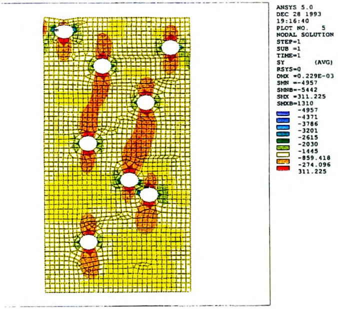
(i) 7.5% Volume Fraction



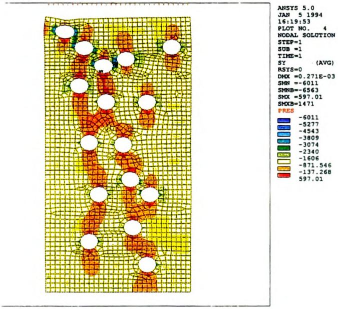
(ii) 15% Volume Fraction

(c) Plastics With Imperfect Bond

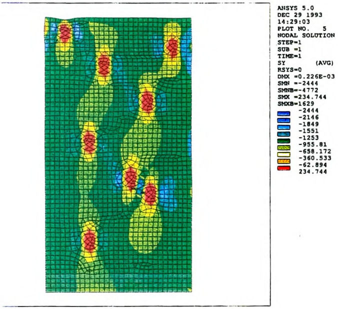
FIGURE 5.2 Deformations



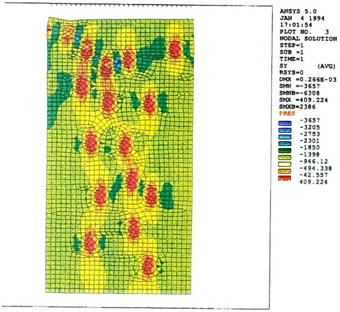
(i) 7.5% Volume Fraction



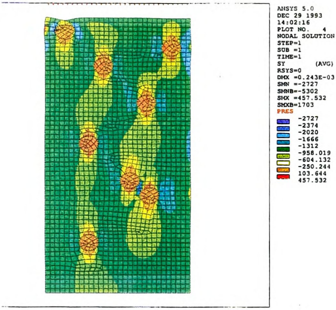
(b) 15% Volume Fraction
(a) Empty Voids



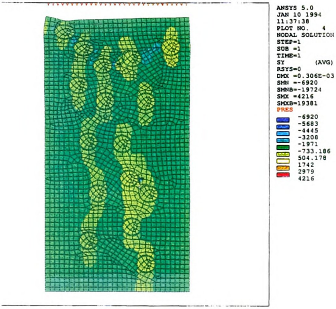
(i) 7.5% Volume Fraction



(ii) 15% Volume Fraction
(b) Plastics With Perfect Bond



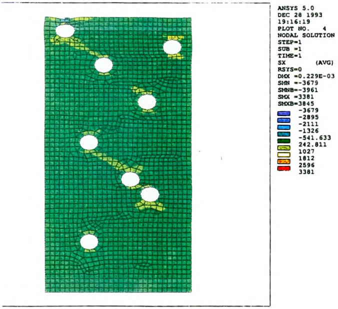
(i) 7.5% Volume Fraction



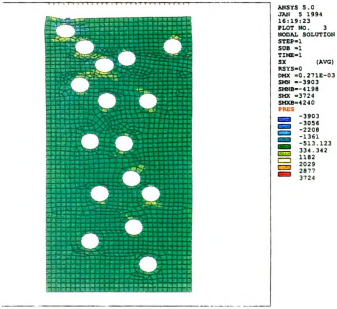
(ii) 15% Volume Fraction

(c) Plastics With Imperfect Bond

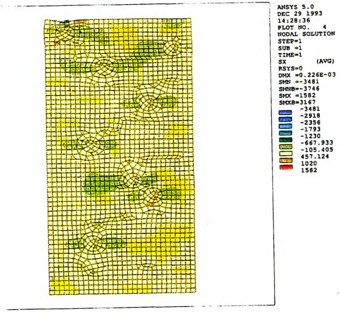
FIGURE 5.3 Longitudinal Stresses



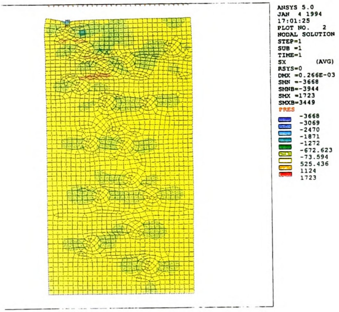
(i) 7.5% Volume Fraction



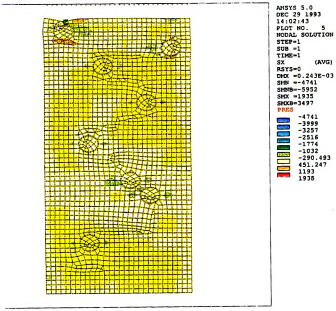
(b) 15% Volume Fraction
(a) Empty Voids



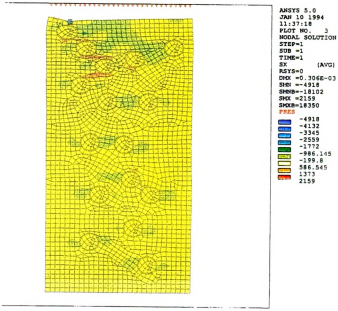
(i) 7.5% Volume Fraction



(ii) 15% Volume Fraction
(b) Plastics With Perfect Bond



(i) 7.5% Volume Fraction

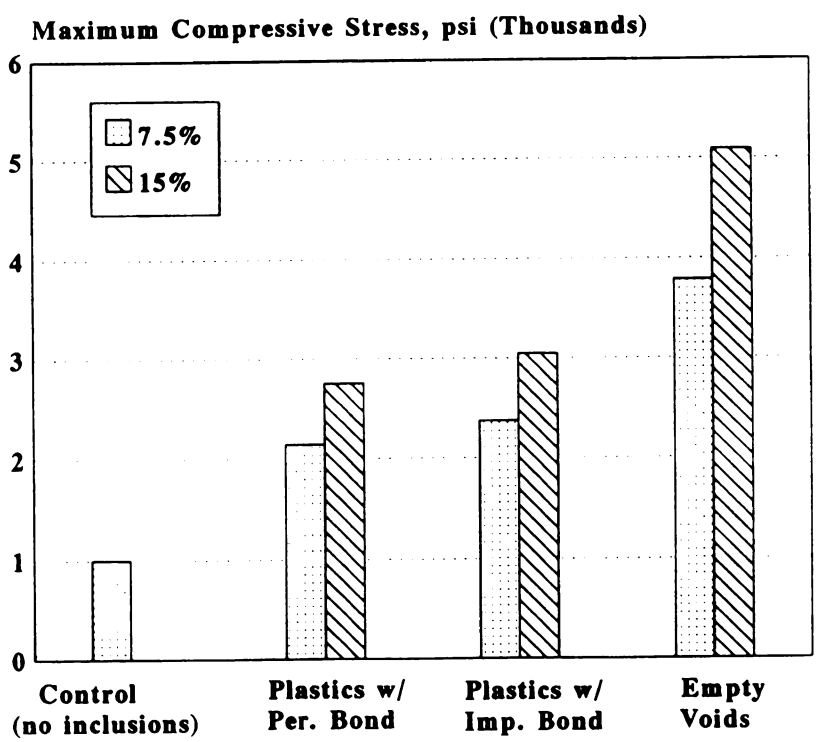


(ii) 15% Volume Fraction

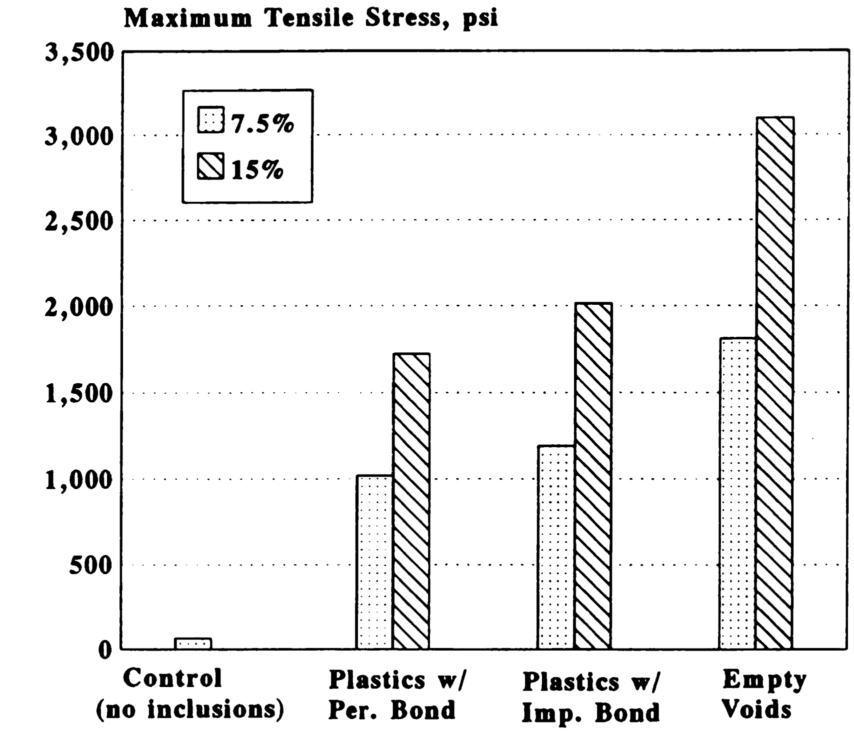
(c) Plastics With Imperfect Bond

FIGURE 5.4 Transverse Stresses

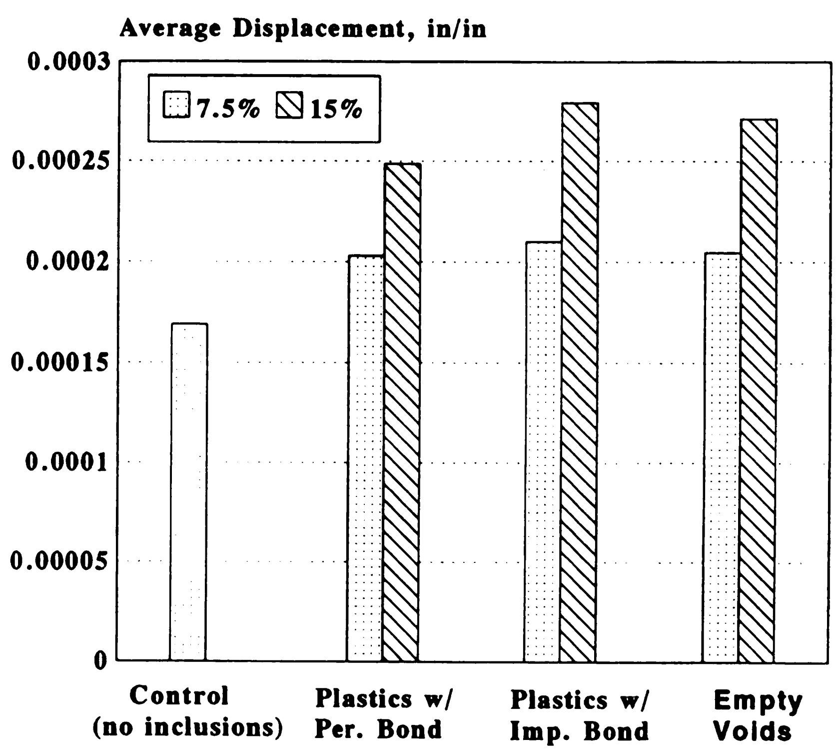
The effects of inclusions (plastics or empty voids) on some key aspects of light-weight concrete performance are summarized in Figure 5.5. Increasing levels of maximum compressive and particularly tensile stresses are generated by the inclusion of bonded plastics, imperfectly bonded plastics and empty voids (see Figures 5.5 a and b); the increase in the inclusion volume fraction leads to increased maximum compressive and particularly tensile stresses. These elevated stresses are consequences of the relatively low elastic modulus (and also the relatively high Poisson's ratio) of the inclusions. Apparently, any positive effects of soft plastic inclusions on the arrest, deflection and bridging of cracks (not accounted for in this modeling) are overshadowed by the adverse effects of plastics on the maximum stress levels under compression. The presence of soft inclusions is also observed in Figures 5.5 c and 5.5 d to increase displacements and reduce the elastic modulus of light-weight concrete.



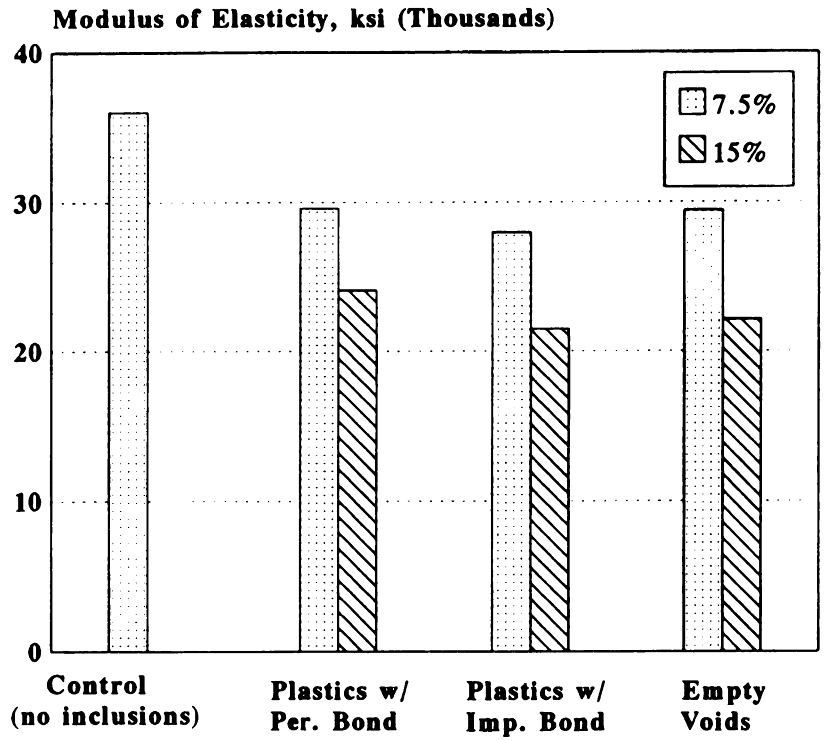
(a) Maximum Compressive Stress



(b) Maximum Tensile Stresses



(c) Average Displacement



(d) Modulus of Elasticity

FIGURE 5.5 Effects of Soft Inclusions on Light-Weight Concrete Properties

Figure 5.6 shows elastic moduli of plastic-lightweight concrete composites as predicted by the theoretical model and measured in laboratory experiments. The laboratory specimens were prepared and tested as presented earlier in Chapter 3; the test procedure followed ASTM C-469. The predictions of elastic modulus compare reasonably well (considering phenomenological nature of the model) with experimental results. The model, however, does not consider the arrest and bridging of cracks by the soft inclusions and thus is only capable of illustrating some aspects of the soft inclusion effects on certain properties of light-weight concrete.

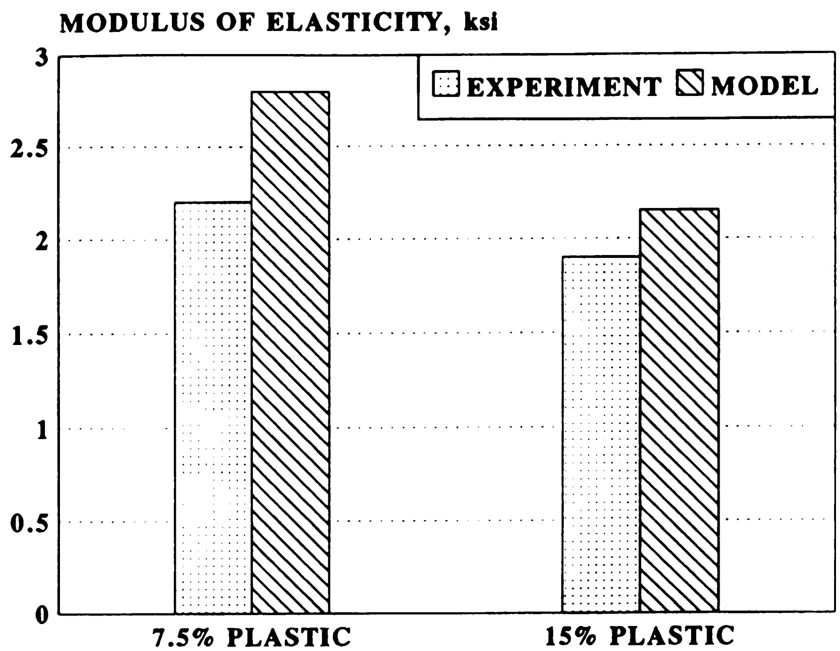


FIGURE 5.6 Experimental Results Versus Theoretical Model Predictions

5.4 SUMMARY AND CONCLUSIONS

A phenomenological model based on linear elastic using finite element analysis techniques was used to investigate some effects of plastic inclusions on some aspects of light-weight concrete behavior under compressive stresses. The inclusions were either empty voids or plastics which were perfectly or imperfectly bonded to the concrete matrix. Two different volume fractions of inclusions were considered. The analytical results, which did not account for the arrest and bridging of cracks by plastics, indicated that:

(1) The redistribution and concentration of stresses induced in concrete by soft inclusions (with relatively low elastic modulus and relatively high

Poisson's ratio) leads to increased maximum compressive and particularly tensile stresses, increased displacements and reduced elastic modulus of light-weight concrete;

(2) Increasing levels of maximum stress and deformation are observed with perfectly bonded plastics, imperfectly bonded plastics, and empty voids; increased volume fraction of the soft inclusions leads to increased maximum stress and deformation;

(3) The experimentally observed negative effects of plastics on compressive performance suggest that any positive effects of plastic inclusions (through the arrest and bridging of cracks, which are not accounted for in this analytical study) would be overshadowed by the adverse effects of plastics on maximum compressive and particularly tensile stresses.

CHAPTER 6

LONG-TERM DURABILITY CHARACTERISTICS AND ENVIRONMENTAL IMPACT OF PLASTIC LIGHT-WEIGHT CONCRETE

6.1 INTRODUCTION

Concrete materials incorporating recycled plastic flakes as light-weight aggregates and reinforcing fibers for enhancing toughness characteristics, impact resistance and shrinkage cracking characteristics at reduced unit weight present new developments in concrete technology. In order to fully develop this class of materials it is important to ensure their satisfactory long-term performance under severe environmental effects; this was the main thrust of the work reported in this chapter.

The long-term durability tests performed on light-weight plastic-concrete composites include: freeze-thaw durability (ASTM C-666), scaling resistance (ASTM C-672), and exposure to temperature cycles.

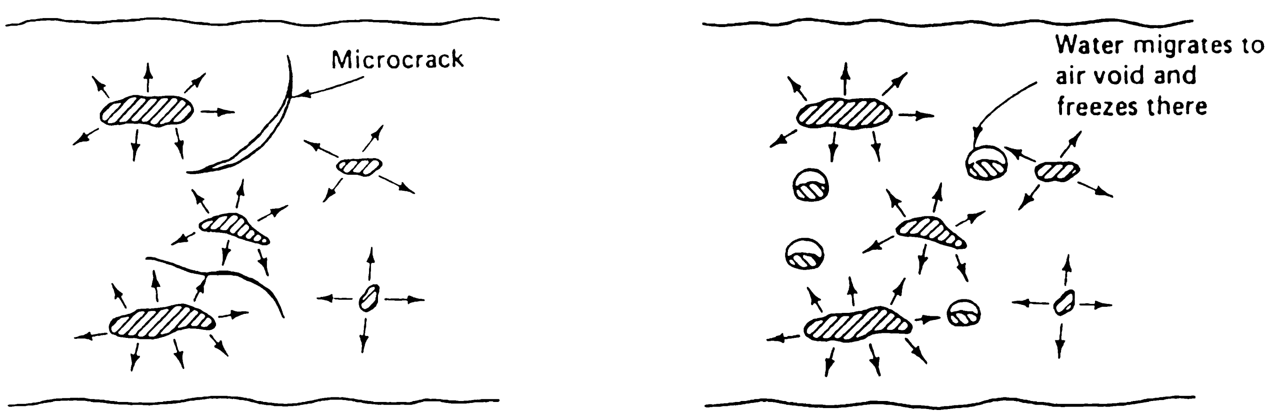
The results obtained in this study were analyzed statistically using analysis of variance and multiple comparison techniques in order to derive statistically reliable conclusions.

6.2 BACKGROUND

6.2.1 FREEZE-THAW DURABILITY

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

When water turns to ice, the corresponding volume increase (about 9%) causes the residual water in capillary pores 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 6.1).



(a) Non Air-Entrained Paste (b)Air-Entrained Paste

FIGURE 6.1 Creation of Hydraulic Pressure in Frozen Paste (Mindess et al 1981).

The superposition of pressure from adjacent capillaries will eventually cause the tensile strength of paste to be exceeded and rupture will occur. As the temperature is progressively lowered, more capillary water is involved in freezing, increasing the hydraulic pressures and thereby increasing microcracking and dilation. In a saturated non-air-entrained paste, the only free space is the exterior of the specimen, and the diffusion of water to the outside is very slow to relieve the hydraulic pressure. Thus, the inclusion of entrained air provides empty space within the paste to which the excess water can move and freeze without damage. The bubbles act as "safety valves" and the spacing factor determines the average distance the water must travel to reach the free space. This distance must not be too great if hydraulic pressure is to be relieved.

Air entertainment in concrete provides escape boundaries in the cement paste matrices. 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 water-to-cement ratio also has an important effect on the freeze-thaw durability of concrete. In general, the higher 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. It was found that for a water-cement ratio of more than 0.4, at a cement content less than 415 kg/m^3 (700 lb/yd^3), a maximum aggregate size of 25 mm (1 in), and an air content of 6 to 8% should be used to avoid deterioration under freeze-thaw cycling (Mindess et al, 1981). But since in most field conditions the water-cement ratio is greater than 0.4, if the structure is going to be exposed freezing and thawing, sufficient entrained 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 number of different tests have been developed to assess the long-term frost resistance of concrete. These involve subjecting concrete to different freeze-thaw cycles and measuring the progressive internal damage by monitoring weight loss, length change (dilation), and decrease in strength or dynamic modulus of elasticity. Tests differ in the nature of freeze-thaw cycles used and the condition of the specimen during the test.

6.2.2 DE-ICER SALT SCALING

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

The exact causes underlying salt scaling are not known, but they probably involve more than one process. It has been reported that the consumption of heat required to melt ice when the de-icer is applied causes a rapid drop in the temperature of the concrete just below the surface, which may cause damage from either the effects of rapid freezing or the stress caused by differential thermal strains. The additional free moisture now present at the surface of the 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 the concrete just below the surface to form relatively concentrated solutions. When rainwater accumulates on the surface, the phenomenon of osmosis occurs, where water flows

to equalize concentration differences. Considerable hydraulic pressures can be created by this effect, causing rupture of the paste. The most damage usually occurs at places where rainwater can accumulate and remain for some time.

Scaling is most likely to occur on the surfaces that have been over vibrated, trawled too early or too long, subjected to plastic shrinkage, or where excessive bleeding occurs. Such surfaces tend to have a weak layer of paste or mortar either at the surface or just below, and may have microcracks or bleeding channels that can transport surface solutions to lower levels. Careful attention to mix design, placing and finishing should eliminate many potential problems. Table 6.1 indicates the recommended curing times for air entrained pavements. If adequate moist curing is followed by a period of drying before de-icing chemicals are applied, scaling should not be a problem.

TABLE 6.1 Minimum Moist-Curing Times to Develop Salt Scaling Resistance (Woods, 1968)

Cement Type	Minimum Curing Period (days)		
	23°C (73°F)	4°C (39°F)	-4°C (25°F)
I	7	15	>60
II	7	12	35
III	7	7	24

6.3 EXPERIMENTAL PROGRAM

An experimental program has been designed based on the statistical concepts of factorial analysis of variance (2^2 factorial design). This experimental program (Table 6.2) investigates the following two variables: plastic type (HDPE and MIXED), plastic content (two different levels, 20 and 40% replacement of fine light-weight aggregate by volume corresponding to 7.5% and 15% by total

volume of concrete). The sulfonation of plastics was considered in the case of light-weight concrete (for both HDPE and MIXED plastics) for freeze-thaw ductility, scaling resistance and exposure to temperature cycles.

TABLE 6.2 Experimental Program For Light-Weight Concrete Incorporating Recycled Plastics.

Replacement Level of Sand with Plastics, by volume*	Plastic Type	
	HDPE	MIXED
20%	-	-
40%	-	-
Control	-	

*20% and 40% replacements of fine aggregate correspond to 7.5% and 15% of total volume.

The mix proportions used in the phase of research are similar to these presented earlier in Chapter 3.

6.4 TEST PROCEDURES

6.4.1 FREEZE-THAW DURABILITY

The purpose of this test is to study the effects of repeated 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). Two specimens were prepared for each of the mixes in Table 6.2. The specimens were rectangular prisms of 76x102x406 mm (3x4x16 in) in dimensions. They were cured for one day inside their molds underneath a wet burlap and a polyethylene film, and then moist cured at 23°C (74°F) and 100% R.H. until the age of 28 days;

thereafter, they were immersed in a cold water bath at 7°C (45°F) for two 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 apparatus, in a thawed condition, at intervals not exceeding 36 cycles of exposure to the freezing-and-thawing. The freeze-thaw damage was assessed through measurement of the fundamental transverse frequency of specimens when simply supported, see Figure 6.2, from which the dynamic modulus of elasticity (P_c), was derived using the following equations:

$$P_k = (n_1)^2 / (n) \times 100$$

Where n = fundamental transverse frequency at 0 freeze-thaw cycles;

n_1 = fundamental transverse frequency at k freeze-thaw cycles.

The length change (L_c) of the specimens was calculated using the following equation:

$$L_c = (l_2 - l_1)/l_g \times 100$$

Where L_c = length change of test specimen after C cycles of freezing and thawing, %;

l_1 = length compactor reading at 0 cycles;

$l_2 =$ length compactor reading at c cycles;

$l_g =$ the effective gage length between the innermost ends of the gage studs.

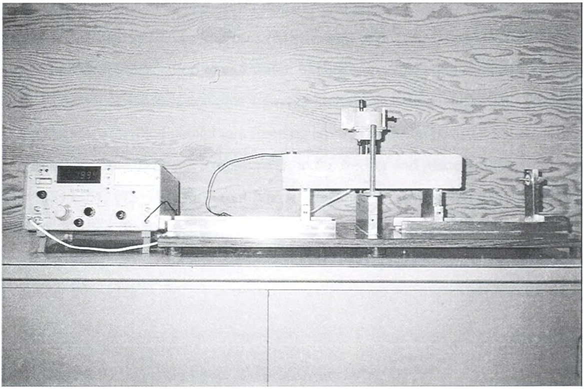


FIGURE 6.2 Test Set-Up for Measurement of Fundamental Transverse Frequency

6.4.2 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. Two specimens were prepared for each of the nine mixes presented in

Table 6.2. The prismatic specimens were 254x203x76 mm (10x8x3 in) in dimensions (see Figure 6.3); they were moist-cured inside their molds for the first 24 hours, then demolded and cured in a curing room at 23°C (74°F) and 100% R.H. until the age of 28 days. The specimens were left to dry in room temperature at 23°C (74°F) and 60% R.H. for one day. At the age of 29 days, the flat finished surface of each specimen was covered with approximately 6 mm (0.25 in) of a solution of calcium chloride and water, having a concentration such that each 100 ml (6.1 in^3) of solution contains 4 gms 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$\pm$1.7°C (73$\pm$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.

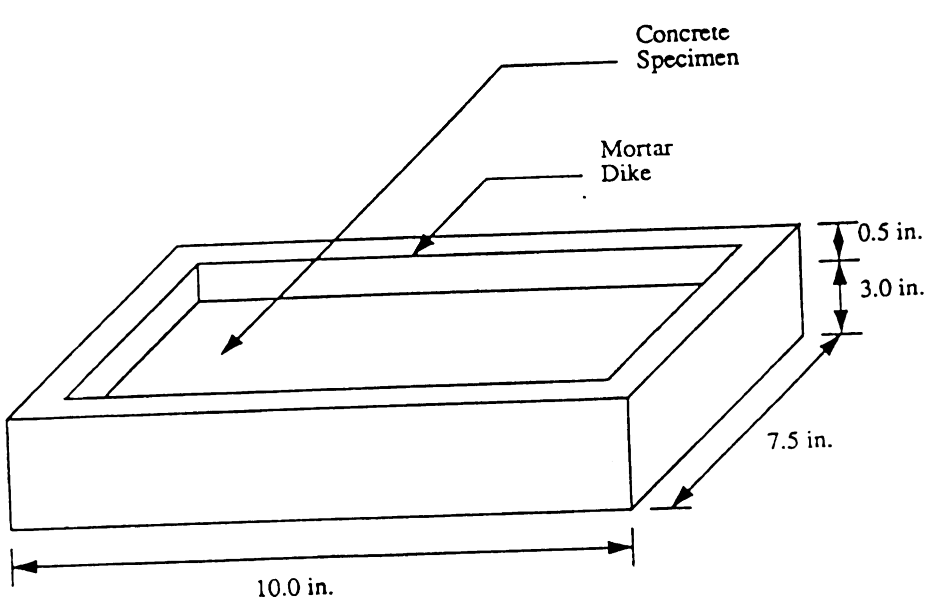


FIGURE 6.3 Specimen Used in Scaling Resistance Test.

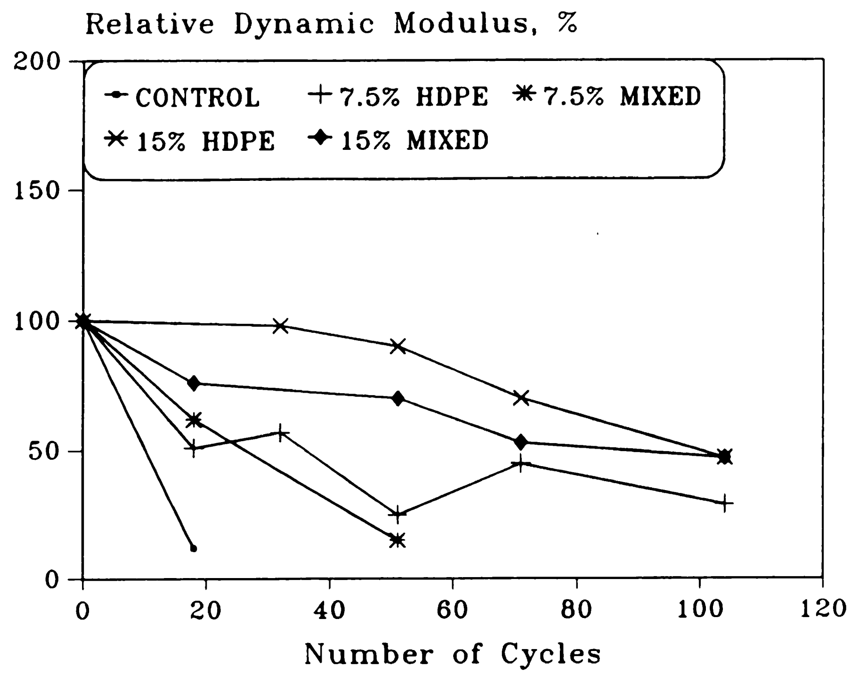
6.4.3 EXPOSURE TO TEMPERATURE CYCLES

In this test, after 28 days of moist curing, the specimens were subjected to cycles of temperature change. This was accomplished by subjecting the specimens to a temperature of 1.7°C (35°F) for a period of 12 hours followed by exposing the specimens to a temperature of 30°C (86°F) for another 12 hours. This cycle was repeated 30 times before testing. Specimens for flexure strength and toughness, 100x100x350 mm (4x4x14 in), were the tested in flexure.

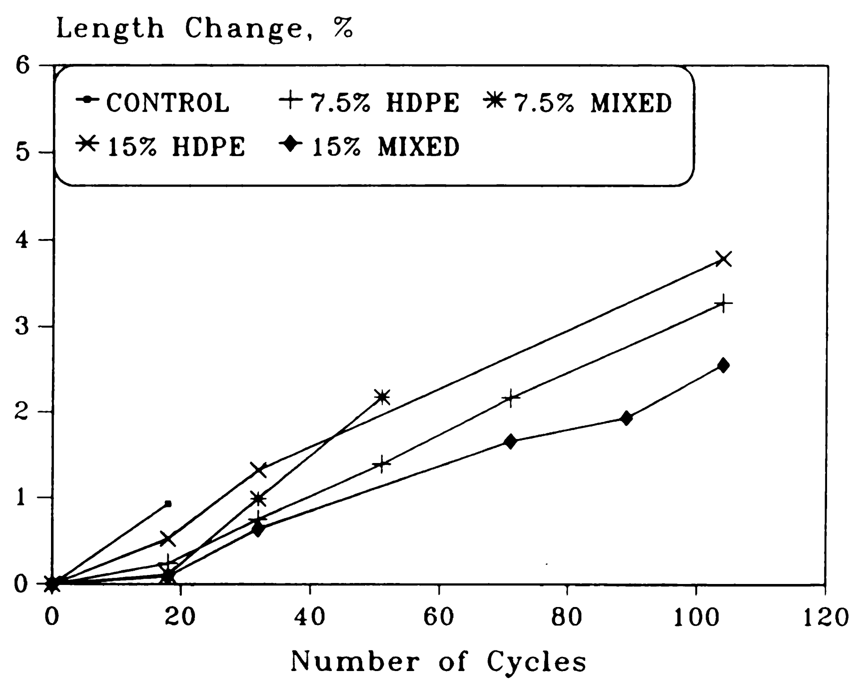
6.5 EXPERIMENTAL RESULTS AND DISCUSSION

6.5.1 FREEZE-THAW DURABILITY

The relative dynamic modulus test results for light-weight concrete incorporating recycled plastics are presented in Figure 6.4(a). HDPE and MIXED plastics produced essentially similar performance. Each value plotted is an average of two specimens. Statistical analysis (using split plot analysis of variance with repeated measurement and separation of means) of results between 12 and 48 cycles confirmed the improvements in freeze-thaw characteristics obtained through incorporating plastics into light-weight concrete.



(a) Relative Dynamic Modulus



(b) Length Change in Percent

FIGURE 6.4 Freeze-Thaw Durability Test Results

As seen in figures 6.4a and b, the control mixture showed poor freeze-thaw durability characteristics. This poor behavior was attributed to the specific light-weight aggregate, Tufflite, used in this investigation. The fine pore system in Tufflite provides it with an extremely high capacity for absorption, about 50 percent, and apparently does not allow quick escape of water to allow release of the pressure associated with ice formations. The addition of recycled plastics to light-weight concrete improved the freeze-thaw durability characteristics of the resulting composite. This could be due to the fact that plastic particles act as reinforcing inclusions, bridging across cracks and mitigating the widening of cracks and disintegration of concrete.

6.5.2 SCALING RESISTANCE

Table 6.3 shows the scaling resistance test results. Recycled plastics, at relatively high volume fractions of 7.5 and 15% by total volume of concrete, had an adverse effect on scaling resistance. This effect, however, is not reflected in Table 6.3 because the specific light-weight concrete used in this investigation itself produced the lowest scaling resistance specified by ASTM. In the presence of plastics, the surface damage under de-icer salt scaling effects occurred at a faster rate.

TABLE 6.3 Scaling Resistance Test Results

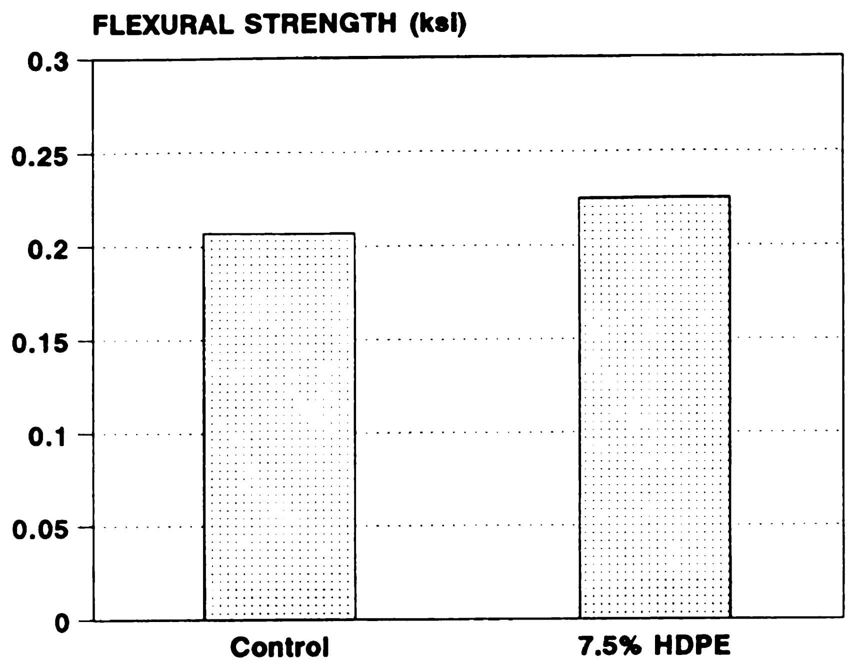
	# of Cycles	Rating
Control	50	5
7.5% PLASTIC*	50	5
15% PLASTIC*	50	5

* HDPE and "MIXED" produced comparable results

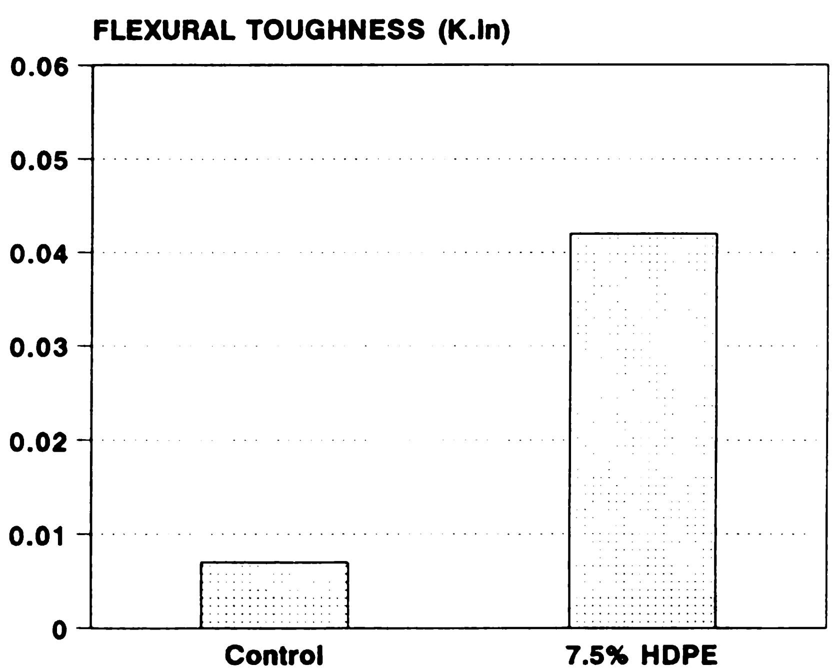
6.5.3 EXPOSURE TO TEMPERATURE CYCLES

The flexural strength and toughness test results associated with exposure to temperature cycles of light-weight concretes incorporating recycled plastics (HDPE) are shown in Figure 6.5. Exposure to temperature cycles led generally consistent damage to control specimens and those incorporating plastics at 7.5% by total volume; the average strength drops of control specimens and those with 7.5% by total volume of plastics were 60% and 52%, respectively. The major drop in strength of control concrete, which was also reflected in plastic concretes; is due to the weakness of the specific light-weight aggregates used in this investigation. Analysis of variance and comparison of means of the flexural strength test results indicated that all drops in strength after 30-day exposure to temperature cycles are statistically comparable at 95% level of confidence.

After 30 day exposure to temperature cycles, the flexural toughness increased about 6 times that of control at 7.5% plastic content. This was confirmed statistically at 99% level of confidence. Hence, if the light-weight aggregate type is selected to provide resistance to temperature cycles, the pronounced toughening mechanism of plastics could highly be advantageous under temperature cycles.



(a) Flexural Strength After Exposure to Temperature Cycles



(b) Flexural Toughness After Exposure to Temperature Cycles

FIGURE 6.5 Effect of Exposure To Temperature Cycles

6.6 ENVIRONMENTAL IMPACT

Table 6.4 present the leaching test results for plastic light-weight concrete incorporating 7.5% HDPE. The results show that the addition of recycled plastics to light-weight concrete produced environmentally acceptable composites.

TABLE 6.4 Toxicity Characteristics Test Results For Plastic Light-Weight Concrete, ug/L

Analyte	Result	Limit Allowed
Arsenic	<200	200
Barium	<100	100
Cadmium	<100	100
Chromium	<100	100
Copper	<100	100
Lead	<100	100
Selenium	<100	100
Silver	<100	100
Zinc	<100	100
Mercury	<10	10
Aluminium	<2000	2000
Beryllium	<2000	2000
Boron	<4000	4000
Calcium	680000	6000
Iron	490	50
Manganese	<50	50
Molybdenum	<2000	2000
Nickel	<200	200

6.7 ASSESSMENT OF SULFONATION EFFECTS

The effects of sulfonation of recycled plastics on chloride permeability, freeze-thaw durability characteristics, and scaling resistance of light-weight concrete mixtures were investigated through comparing the results obtained with sulfonated and unsulfonated plastics. Sulfonated recycled plastics produced

freeze-thaw durability characteristics and scaling resistance which were comparable to those obtained with usulfonated plastics (see Figure 6.6 for freeze-thaw durability test results).

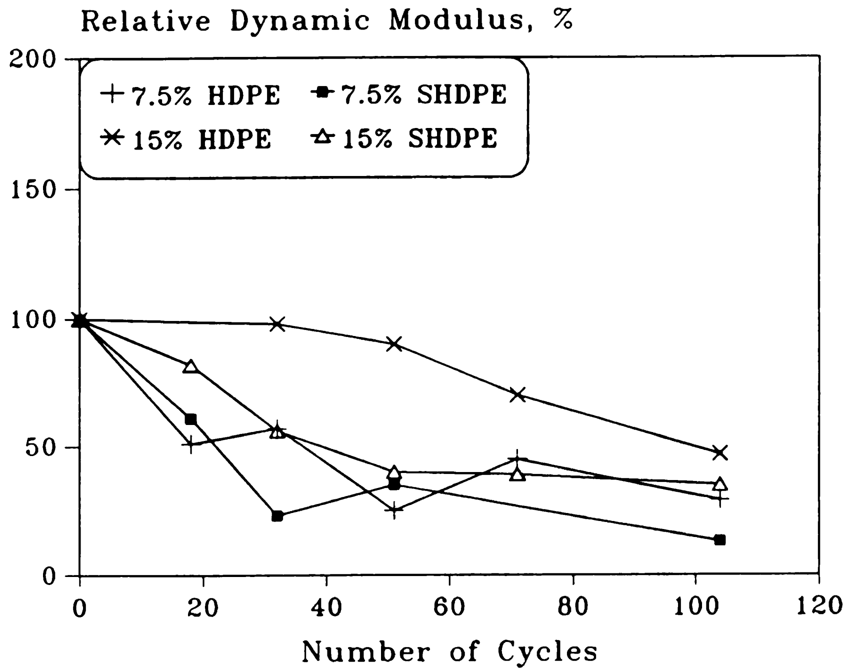


FIGURE 6.6 Assessment of Sulfonation Effects on Freeze-Thaw Durability Characteristics of Light-Weight Concrete.

6.8 SUMMARY AND CONCLUSIONS

The long-term durability of light-weight plastic-concrete campsites (incorporating recycled sulfonated HDPE and "MIXED" plastics as partial substitutes for light-weight aggregate in light-weight concrete) were assessed through investigating the freeze-thaw durability characteristics (ASTM C-666), scaling resistance (ASTM C-672), and resistance to temperature cycles of the

materials. Two different plastic contents, 20 and 40% replacement of fine light-weight aggregate by volume corresponding to 7.5% and 15% by total volume of plastics in the case of light-weight concrete were considered. The effects of sulfonation on freeze-thaw durability and scaling resistance were investigated. It was concluded that:

In light-weight concrete: (1) The addition of recycled plastics to light-weight concrete improved the freeze-thaw durability characteristics of the resulting composite noting that the specific light-weight aggregate used in this investigation made concrete highly susceptible to frost attack; (2) Recycled plastics had a negative effect on scaling resistance of concrete, again noting the very low scaling resistance of the control light-weight concrete associated wit the frost susceptibility of the specific light-weight aggregate used in this study; (3) the addition of recycled plastic flakes to light-weight concrete produced environmentally acceptable composites; and (4) Sulfonated plastics produced freeze-thaw durability characteristics and scaling resistance which were comparable to those obtained with unsulfonated plastics.

CHAPTER 7

SHRINKAGE CHARACTERISTICS OF NORMAL-WEIGHT CONCRETE

7.1 INTRODUCTION

The potentially influential variables (recycled plastic content and recycled plastic width) were selected to be optimized in application to normal-weight concrete. The optimization experimental design was formulated based on response surface analysis techniques. The composites were optimized considering their resistance to cracking under restrained dying shrinkage.

7.2 BACKGROUND

Volumetric changes in hardened and plastic concrete are significant in magnitude and have a serious influence on the performance and durability of concrete structures. Inadequate allowance for the effects of shrinkage in concrete can lead to cracking or warping of concrete slabs (Zollo, 1984). Shrinkage, the reduction in the bulk volume of concrete, is only a fraction of the volume of the

water loss, with larger part of released water coming from the pores in the concrete. Furthermore, shrinkage dose not depend on water loss alone, but also on the actual deformability of concrete.

The duration of shrinkage in concrete could be divided into three phases, as shown in Figure 7.1 (Kral and Geuaver, 1979). During the first phase, concrete adopts itself to the mold (low resistance to deformation); it does not shrink at all in this phase (except in the vertical direction, see Figure 7.1) and does not suffer serious cracking. In the second phase, the evaporation of water causes rapid shrinkage. Thus, the possibility of cracking arises as the tensile strength of concrete increases more slowly than the induced stresses. Then, in the third phase, the rapidly increasing resistance against deformation, as shown in Figure 7.1, will be accompanied by a retardation in shrinkage and faster increase of the tensile strength. Due to the combination of both effects, the danger of cracking at early ages ceases in the third phase.

At the beginning and during the period of initial hardening (i.e., in green concrete), the fracture stress of concrete is going through a minimum while the tensile strength is low, see Figure 7.2 (Chatterji, 1982). As result, concrete cracks easily.

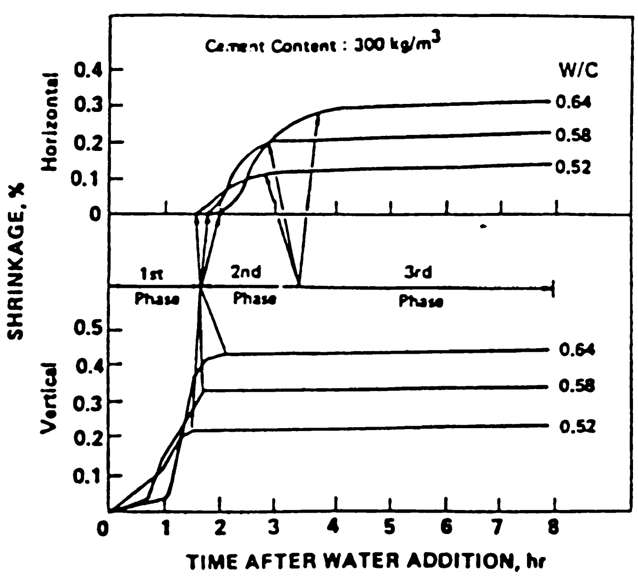


FIGURE 7.1 Effect of Water-Cement Ratio on Vertical and Horizontal Shrinkage (Karl and Geuaver, 1979)

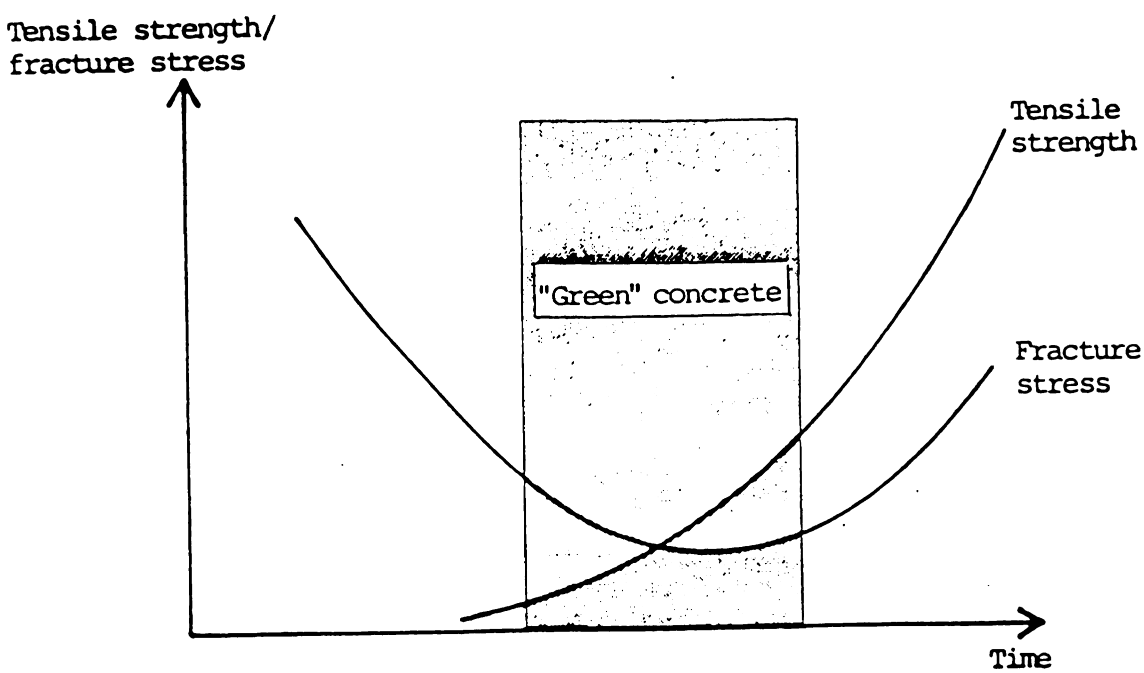


FIGURE 7.2 Principle of Stress and Tensile Strength Development of Concrete at Early Stage (Chatterji, 1982)

Here are some of the many factors that influence the shrinkage of concrete, which will consequently affect cracking:

1. Material composition;
2. Mix design;
3. Construction Methods;
4. Temperature and relative humidity of the construction environment;
5. Steel Reinforcement;
6. Curing practices and conditions;
7. Rate of evaporation;
8. Size of the structure;
9. Age of the concrete;
10. Structural loads;
11. Capillary stresses;
12. Desiccating pressure; and
13. Changes in surface pressure.

Un-restrained (free) shrinkage is rarely found in typical concrete structures. Restraints are always present, either internal or external, resulting from support conditions, reinforcement, or due to non-uniform drying. These restraints induce tensile stresses which approach the tensile strength of concrete and cause cracking.

Recent investigation of concrete structures have revealed that cracking can occur inside the bulk of concrete structures with or without any visible sign on the outer surfaces. It is also indicated that under certain conditions both micro- and macro-cracks can form shortly after or even before the formwork is completely removed. It is more probable that the micro-cracks will widen further due to both mechanical and environmental stresses. To lengthen the service life of a concrete

structure, it will be necessary to avoid the formation of the early cracks as much as possible.

Shrinkage is a time-dependent phenomenon; if it takes place during the early age, it is called "plastic shrinkage". If it occurs after the concrete has hardened it is called "Drying shrinkage". Other identifiable types of shrinkage, such as carbonation shrinkage and autogenous shrinkage, while not caused by the same mechanisms, are similar in effects to drying shrinkage.

In plastic concrete the stress distribution properties of plastics and their ability to transfer tensile stresses across cracks are very important. When concrete reinforced with steel bars is subjected to tension, it has been shown that the predominant parameter determining the crack spacing and the crack width is the specific surface of the reinforcement (the surface area of reinforcement per unit volume of concrete). It can be shown that concrete incorporating discrete plastic inclusions of small particle size has higher specified surface area of reinforcement than conventionally reinforced concrete (Krenchel and Shah, 1987). Hence, when plastic concrete is subjected to restrained shrinkage, the resulting strains can be distributed over several cracks of limited width instead of a few cracks with unacceptably large widths as is the case in unreinforced concrete.

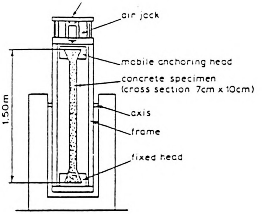
Many other factors govern the performance of plastic-concrete subjected to restrained shrinkage. These include the potential extent of shrinkage, degree of restraint, time-dependent constitutive properties of concrete, and the plastic to matrix interfacial bond characteristics (Grzybowski and Shah, 1990; Dahl, 1989; and Dahl and Holand, 1985).

There are currently no standard tests to assess cracking due to restrained shrinkage. Table 7.1 presents a summary of the restrained shrinkage test methods and conditions reported by different investigators. Some researchers have tried to use long specimens with flared ends (Figure 8.3 a) that were restrained, and used

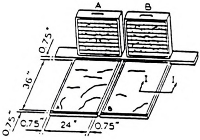
small cross-sectional dimensions at 2.75 x 4 in. (71 x 102 mm) to produce shrinkage cracking. Two other key types of specimen have been used by other investigators for restrained shrinkage tests, namely plate-type specimens (rectangular) and ring-type specimens, see Figures 7.4 b, 7.4c and 7.4d (Krenchel and Shah, 1987; and Grzybowski and Shah, 1990). In the case of plate specimens, when the restraint against shrinkage movement is provided in two directions, a biaxial state of stress is produced. The results obtained from this type of test may depend on the specimen geometry in addition to the material properties. The restrained shrinkage tests using steel rings were conducted as early as 1939 to 1942 (Grzybowski and Shah, 1990). The ring type restrained shrinkage test apparatus has been found to provide a high and nearly constant degree of restraint, producing consistent results with cement paste and mortar. In this test a concrete ring is cast directly around a heavy steel ring. As concrete dries, the concrete ring tends to shrink while the steel ring provides the restraint and prevents the shrinkage movements. The steel ring also serves as a sensitive dynamometer to compute the induced tensile stresses in concrete by measuring the steel strains using strain gages.

TABLE 7.1 Summary of Restrained Shrinkage Test Methods and Conditions Reported By Different Investigators

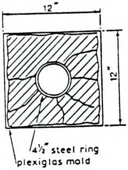
Ref #	Specimen		Curing Condition
	Type	Dimension	
6	Ring	6.7/9.8 in. 1.6 in. thick	Moist cured for 1-2 weeks then ambient atmosphere
7	Ring	6.07/7.36 in. 5.5 in. thick	68^0F, 100% R.H. for 2.5 hrs. then dry at 40% R.H.
8	Ring	20/23 in. 4.0 in thick	61^0F, 50% R.H. (chamber)



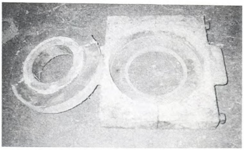
(a) Flared Ends



(b)Plate (Panel)



(c) Plate With Steel Ring



(d) Ring Type

FIGURE 7.3 Restrained Shrinkage Test Specimen Types (Mirza, 1992)

7.3 EXPERIMENTAL PROGRAM

The main thrust of this phase of research was to determine the optimum geometry and volume fraction of plastics for the control of restrained drying shrinkage cracking. The experimental program for optimization through response surface analysis (using the "Design Expert" software) is presented in Table 7.2. Various combinations of the two influential variables (recycled plastic content and plastic width) are considered in this experimental program. Recycled plastic flakes with maximum size of 10 mm (3/8 in) were also used at different volume fractions (in addition to the experimental program of Table 7.2). the materials considered in this phase of the study were subjected to the restrained shrinkage test.

TABLE 7.2 Optimization Experimental Program

	Variables	
Experiment #	Plastic Width, mm	Plastic Content (by total volume), %
1	1.0	0.250
2	2.0	0.250
3	1.0	5.000
4	5.5	1.500
5	5.5	2.625
6	5.5	2.625
7	2.0	2.625
8	5.5	5.000
9	5.5	2.625
10	5.5	2.625
11	5.5	2.625
12	5.5	2.625
13	5.5	2.625

7.4 MATERIALS, MIX PROPORTIONS AND TEST PROCEDURES

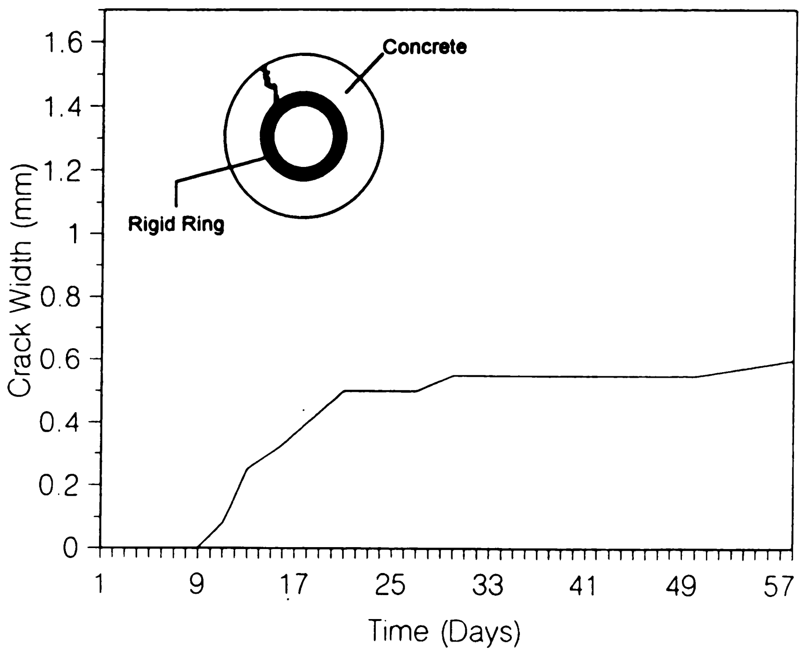
The basic mix ingredients of the mortar mixtures used in this phase of research were Type I portland cement, normal-weight fine aggregate, water, and recycled HDPE plastics with variable widths ranging from 1 mm (0.04 in) to 5.5 mm (0.22 in) were used. The length of plastic particles was kept constant at 25.4 mm (1.0 in) for all mixtures. The plastic inclusions had a thickness of 1.90 mm (0.075 in); they were basically milk jugs cut to the specified dimensions.

The mortar mixtures used in this phase had cement-to-fine aggregate-to-water ratios of 1 : 2.2 : 0.5

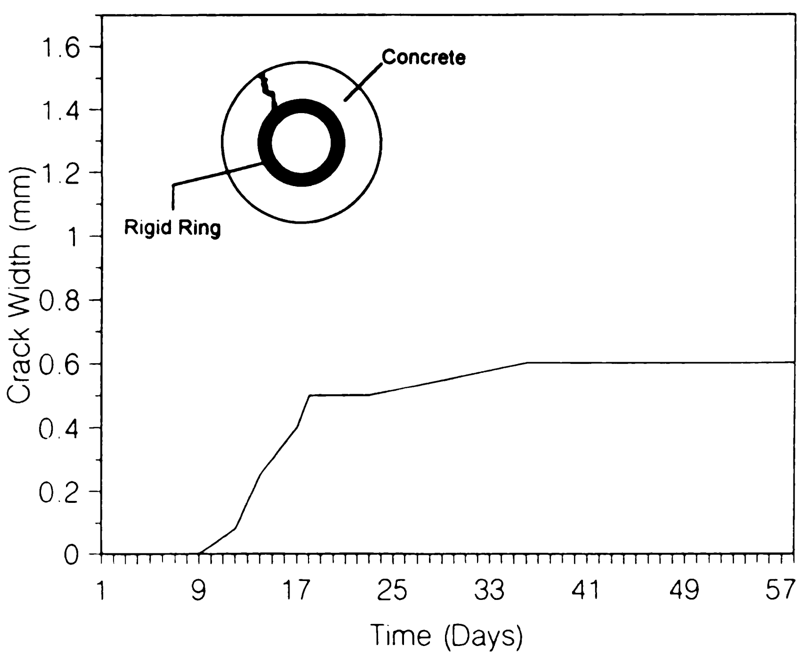
Ring type specimens (see Chapter 3) were used for restrained drying shrinkage tests on mortar specimens. The specimen was cast in two equal layers, leveled by trowel, and then covered with plastic sheets for 6 hours. The specimens were then exposed to air at approximately 23^0C (73^0F) and 40% R.H. Restraint of shrinkage movements by the steel ring inside the specimen creates internal tangential tensile stresses which cause cracking.

7.5 TEST RESULTS AND ANALYSIS

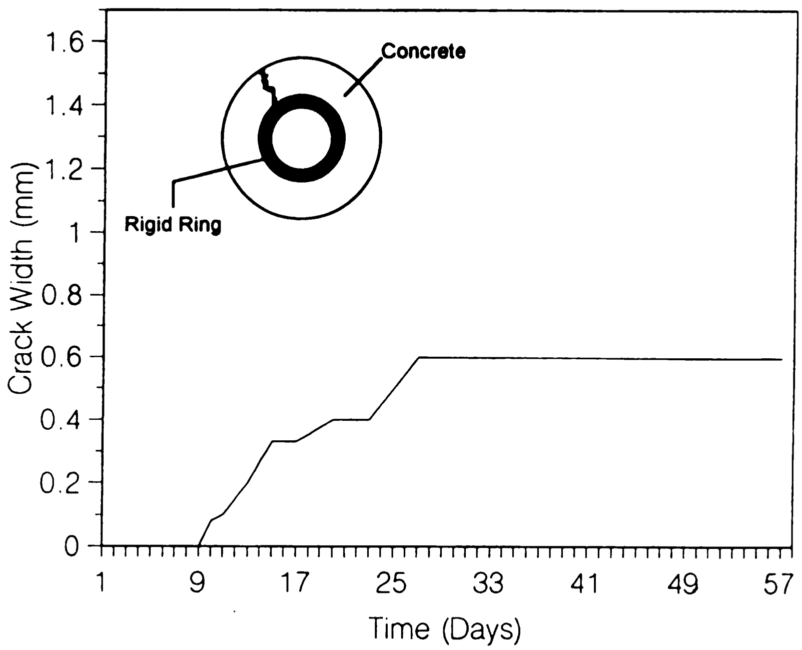
The test results obtained in this phase of study are presented in Figure 7.4 in the form of crack width versus air-drying period. Typical effects of plastic particle width on maximum crack width in restrained drying shrinkage are presented in Figure 7.5. Figure 7.6 shows the effect of fiber widths on restrained drying shrinkage cracking at same volume fractions.



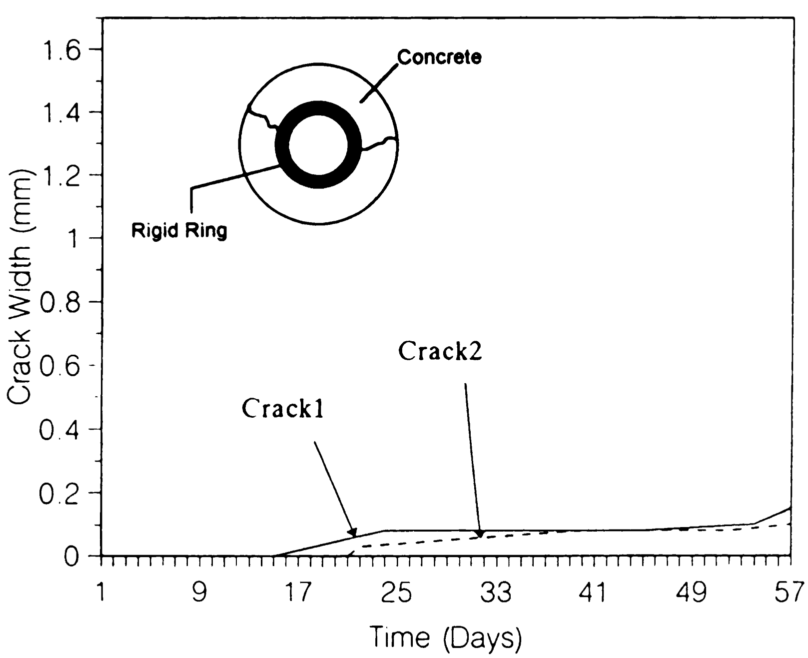
(a) Width = 1 mm @ Volume Fraction = 0.25%



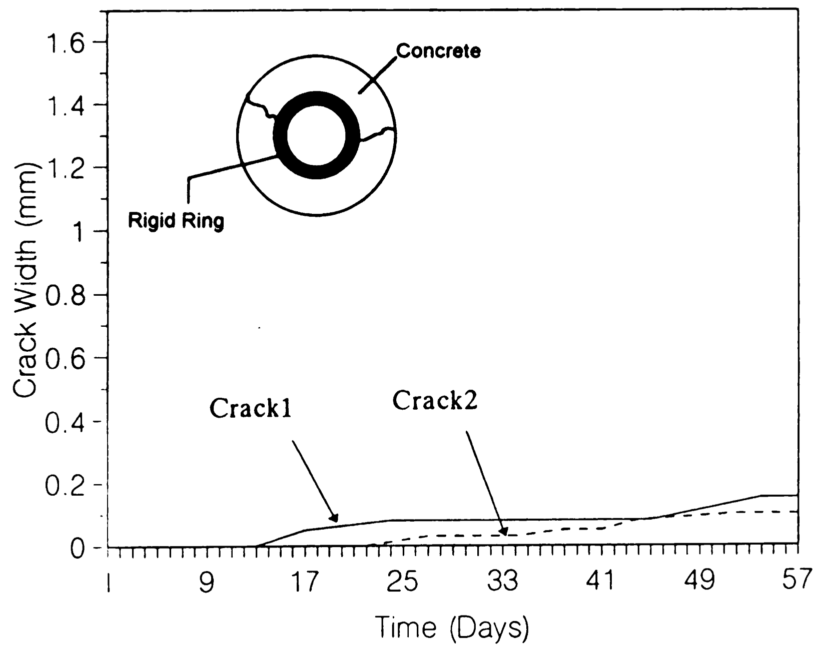
(b) Width = 2 mm @ Volume Fraction = 0.25%



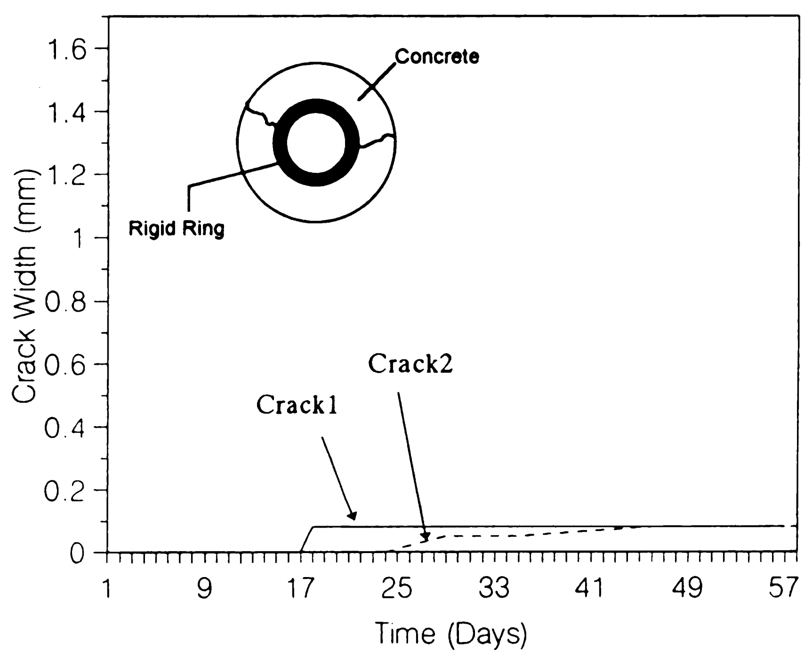
(c) Width = 5.5 mm @ Volume Fraction = 1.5%



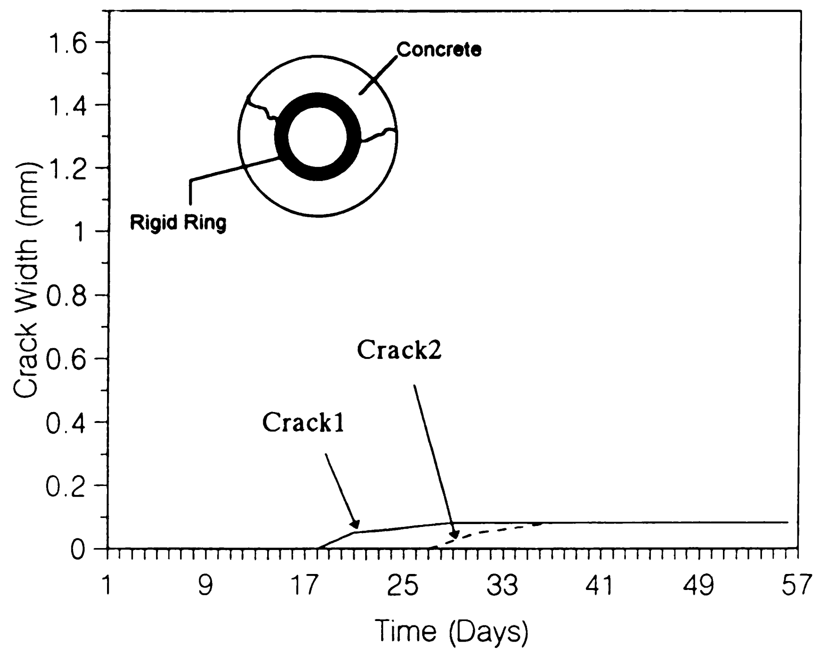
(d) Width = 2 mm @ Volume Fraction = 2.625%



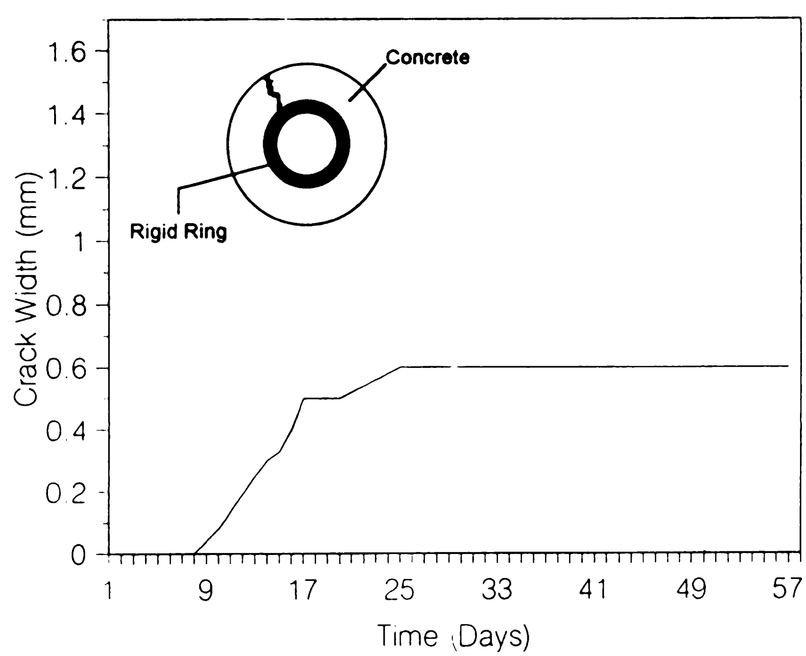
(e) Width = 5.5 mm @ Volume Fraction = 2.625%



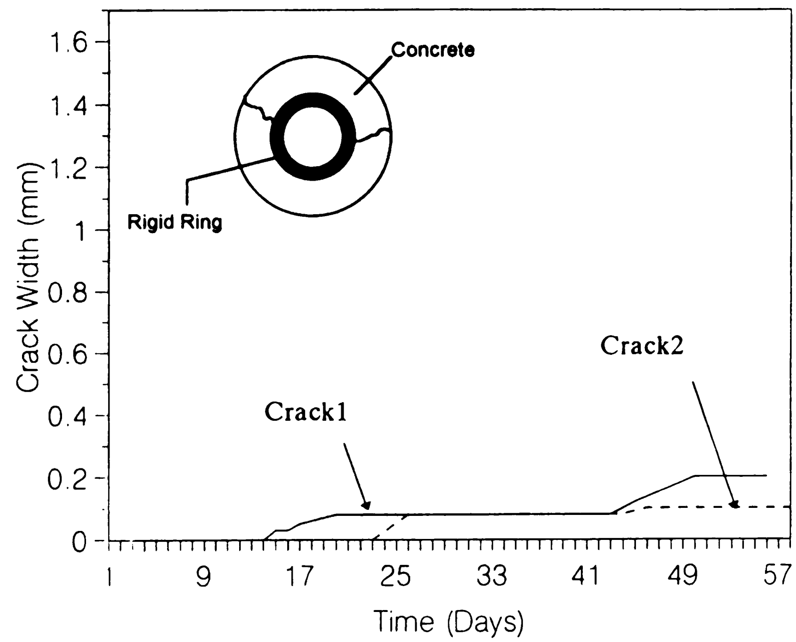
(f) Width = 1 mm @ Volume Fraction = 5%



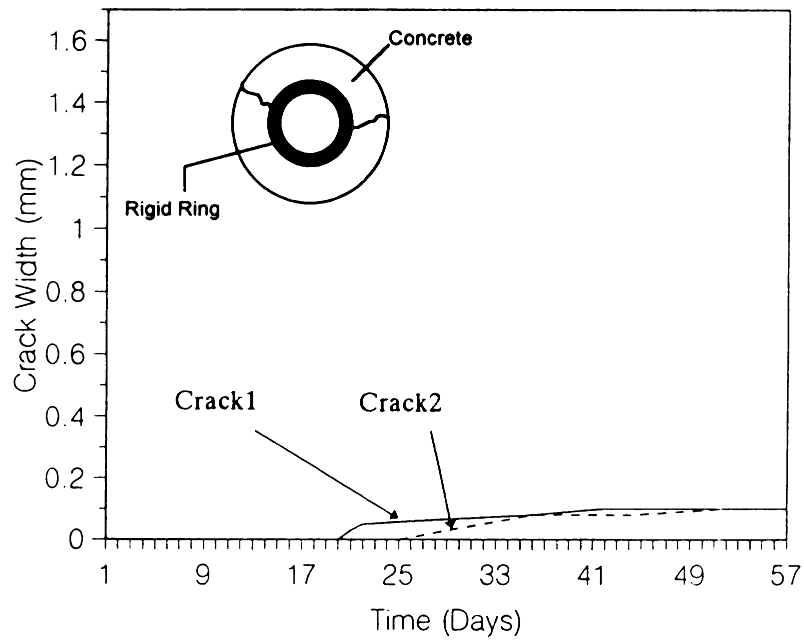
(g) Width = 5.5 mm @ Volume Fraction = 5%



(h) Flakes @ Volume Fraction = 0.25%



(i) Flakes @ Volume Fraction = 2.625%



(j) Flakes @ Volume Fraction =5%

FIGURE 7.4 Test Results

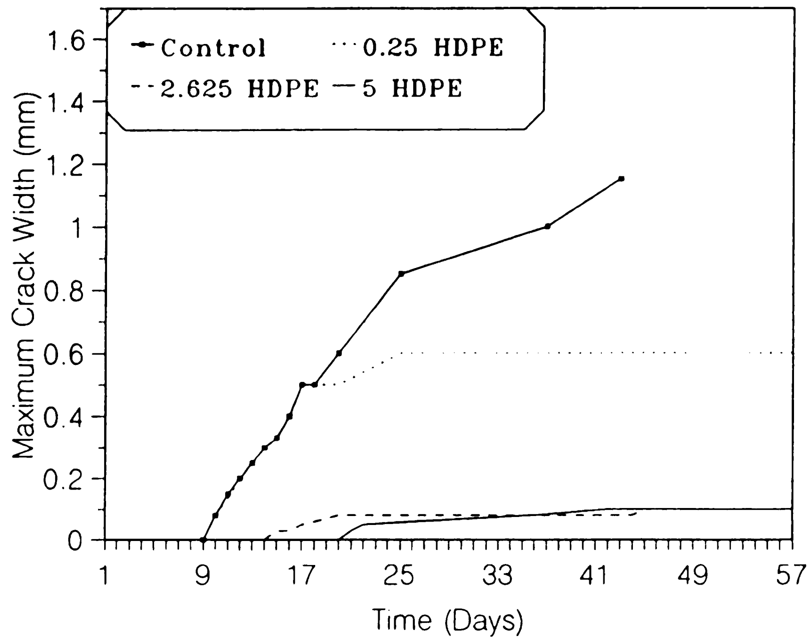
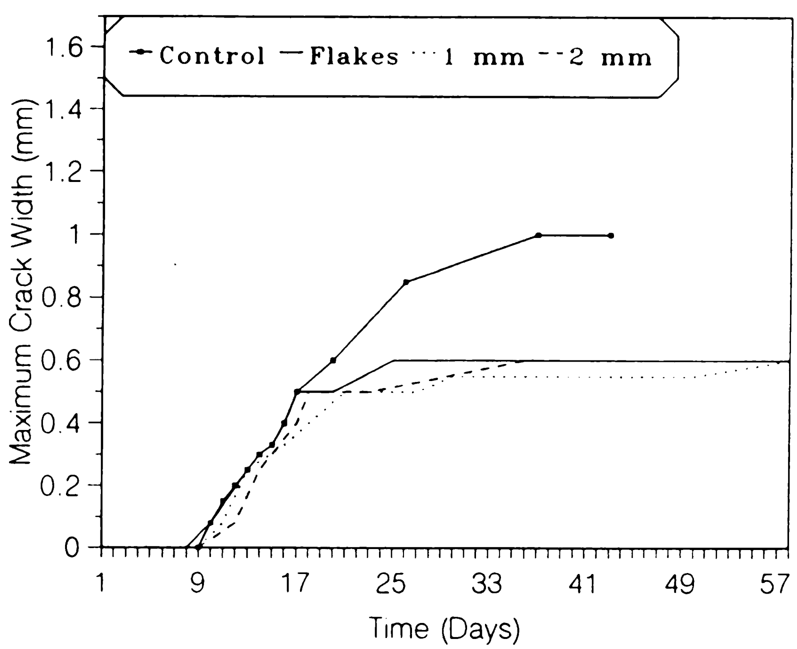
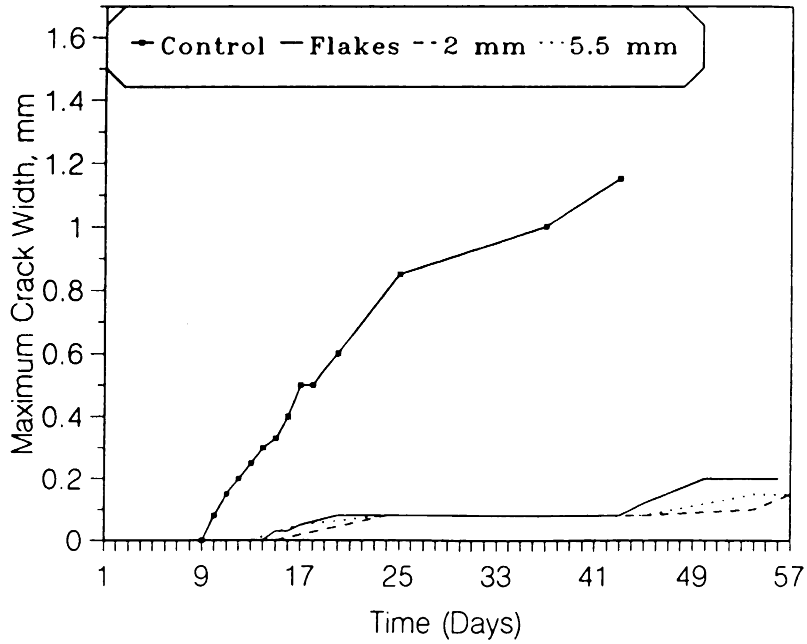


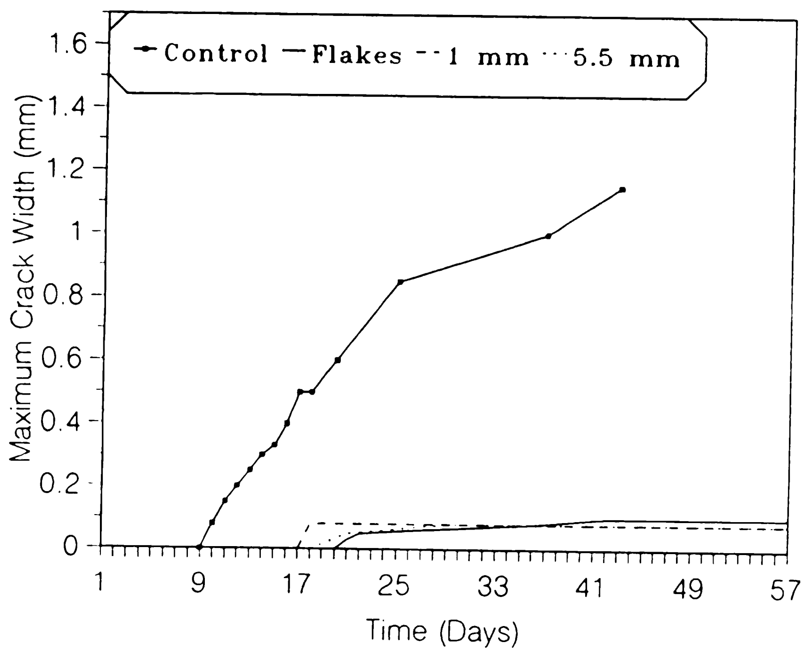
FIGURE 7.5 **Restrained Drying Shrinkage Versus Drying Time**



(a) Volume Fraction of 0.25%



(b) Volume Fraction of 2.625%

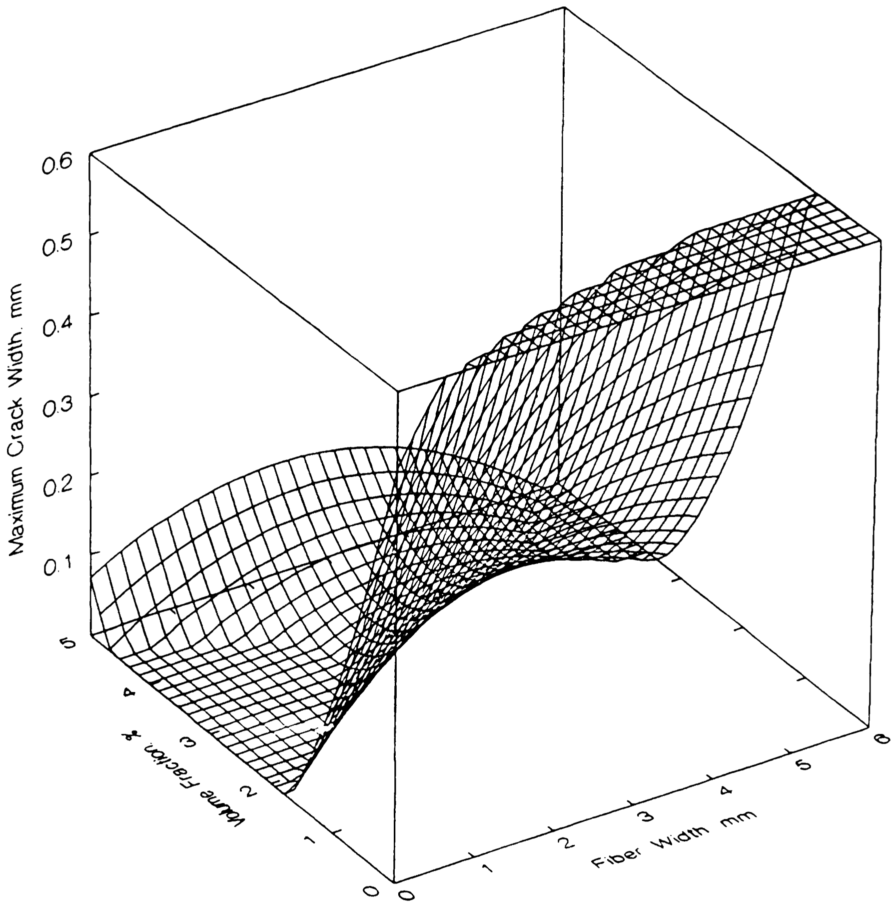


(c) Volume Fraction of 5%

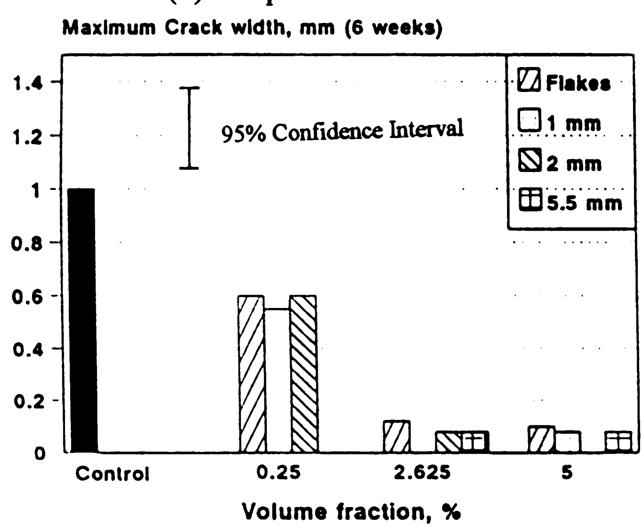
FIGURE 7.6 Effect of Fiber Width at Similar Volume Fractions on Restrained Drying Shrinkage Cracking

It can be seen in Figure 7.4 that fiber volume fraction has an important role on restrained shrinkage cracking. This is due to the reinforcing action of the plastic particles in brittle concrete matrices. Concretes at same volume fractions produced restrained shrinkage cracking characteristics that were statistically comparable at different plastic widths (Figure 7.5). Plastic flakes also produced results comparable with those obtained with plastic "fibers" having different widths.

Figure 7.7(a) shows the response surface of the results. Analysis of test results, at the age of 6 weeks showed that the maximum crack width dropped by about 40% at 0.25% volume fractions when compared to control mixtures without plastics. The reduction in crack width was about 85% for both 2.625% and 5% plastic contents, as shown in Figure 7.7(b). The drop in restrained shrinkage crack width with increasing plastic volume fractions was confirmed statistically at 95% confidence level.



(a) Response Surface



(b) Effect of Fiber Volume Fraction on Restrained Drying shrinkage Cracking at 6 Weeks of Age

FIGURE 7.7 Optimization Test Results

The optimum levels of the influential variables derived from the response surface analysis of results were:

Recycled Plastic Content: 2%

Recycled Plastic Width: Due to the insignificant effects of plastic width, a range of widths would be acceptable.

The above optimum conditions correspond to the minimum plastic content which effectively reduces restrained shrinkage crack widths. The optimized mixture with a plastic width of 5.5 mm (0.22 in) at volume fraction of 2% was then compared to control mixtures (i.e. without plastics) with respect to restrained drying shrinkage. Figure 7.8 shows the comparison of restrained drying shrinkage test results (crack width versus time). The addition of recycled plastic distributes the cracks and thus reduces the crack widths. This can be attributed to the fact that recycled plastics act as reinforcing inclusions that arrest microcracks and bridge across cracks to restrain their widening. Statistical analysis showed that at 2% (optimum) plastic content, plastic flakes produced shrinkage cracking characteristics comparable to those obtained with plastic "fiber" at a width of 5.5 mm (0.22 in.), as shown in Figure 7.9; this was confirmed statistically at 95% confidence levels. Hence, plastic flakes obtained through a simple mechanical grinding process would produce satisfactory results as far as the control of restrained shrinkage cracking is concerned.

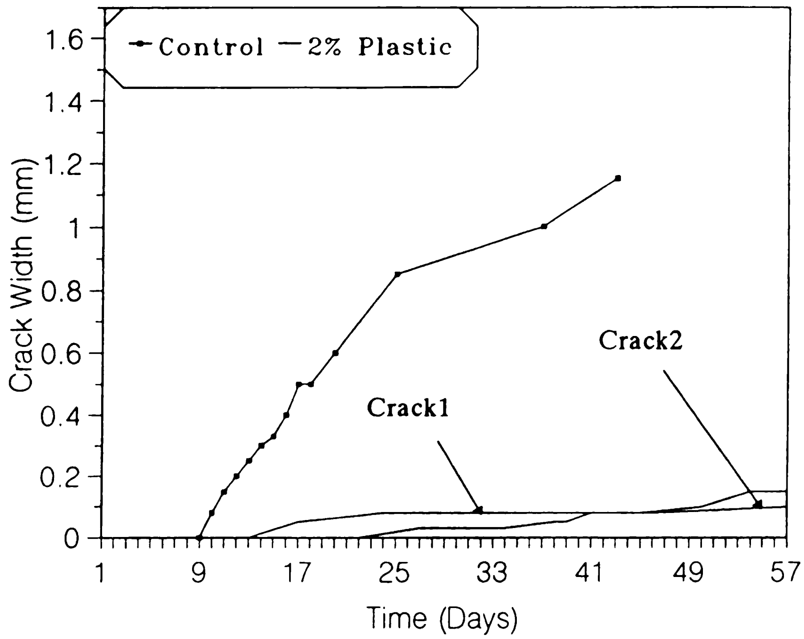


FIGURE 7.8 Crack Width Versus Time in Days

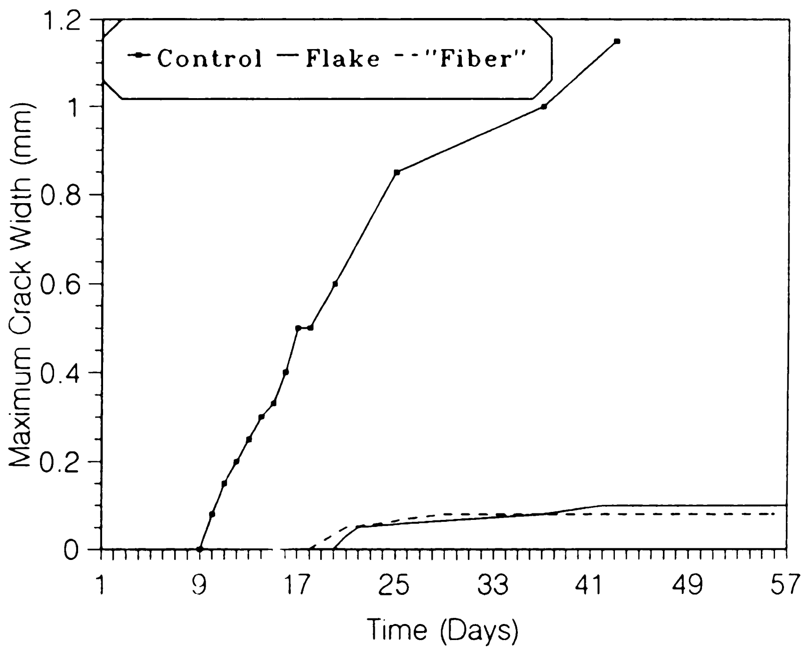


FIGURE 7.9 Comparison Between Plastic Flakes and Plastic Fibers.

7.6 SUMMARY AND CONCLUSIONS

Normal-weight concretes incorporating recycled HDPE plastics, as secondary reinforcing inclusions (additives), were considered in this phase of research. Plastic volume fraction and plastic width were optimized in order to achieve a condition where minimum plastic content would effectively reduce the restrained shrinkage crack width. The optimized composite was then compared to control mixtures (i.e. without plastic) and also with mixtures incorporating ground plastic flakes at the optimum plastic content. The conclusions derived are summarized below.

1) Recycled plastics can effectively reduce the restrained shrinkage crack widths in concrete; this can be attributed to the capability of plastics to bridge across cracks, restrain their opening, and provide post-cracking tensile resistance across cracks.

2) While plastic volume fraction significantly influences restrained shrinkage crack widths, the width of plastic particles did not have statistically significant effects.

3) A plastic volume fraction of 2% represent the minimum (optimum) plastic content where restrained shrinkage crack widths were effectively controlled; ground plastic flakes and plastic "fibers" where both effective at this volume fraction.

CHAPTER 8

EVALUATION OF THE OPTIMIZED PLASTIC NORMAL-WEIGHT CONCRETE MIXTURES

8.1 INTRODUCTION

As mentioned in the previous chapter, the addition of recycled plastics to concrete helps control the drying shrinkage cracks. This valuable contribution of recycled plastics to concrete shrinkage cracking could be most beneficial if other qualities of concrete (both short-and long-term properties) are not damaged (or are improved) in the presence of plastics. The aim of this chapter is to evaluate the effect of the optimum plastic content on some mechanical properties (flexural strength and toughness (ASTM C-78), compressive strength (ASTM C-39), and impact resistance (ACI Committee 544), and long-term durability characteristics (freeze-thaw resistance (ASTM C-666) and scaling resistance (ASTM C-672) of concrete.

8.2 MATERIALS, MIX PROPORTIONS AND TEST PROCEDURES

The basic mix ingredients of the concrete materials used in this phase of research were: type I Portland cement, normal-weight aggregate (natural sand and

crushed limestone with maximum size of 12 mm, 0.5 in), water, recycled (ground) HDPE plastics with maximum particle size of 10 mm (3/8 in), and air entraining agent (water-based). The gradation of both coarse and fine aggregates met the ASTM C-33 requirements). The concrete mixtures used in this phase of research had water/cement, fine aggregate/cement and coarse aggregate/cement ratios of 0.42, 2 and 2.5, respectively. The optimum plastic content of 2% by total volume of concrete was used in the production of the plastic normal-weight concrete mixtures. Air entraining agent was used at 0.06% by weight of cement to produce resistance against frost attack. Table 8.1 presents the mix proportions.

TABLE 8.1 Mix Proportions for Normal-Weight Concrete (lb/yd^3)*

Mix	Cement	Water	Coarse	Fine	Plastic	AE%
Control	670	281	1675	1340	---	0.06
Plastic	655	275	1637.5	1310	31	0.06

* lb/yd^3 = 0.594 kg/m^3; AE = Air Entraining Agent by weight of cement

Conventional mixing and curing procedures (ASTM C-192) were used to prepare the control and plastic concrete materials. External vibration was found to be suitable for consolidating concrete specimens incorporating recycled plastics. The optimum vibration time was found to be 25 ± 5 seconds at a frequency of 80 Hz.

Details of test specimens and the test procedures for determining the mechanical properties and long-term durability characteristics have been introduced in Sections 3.4 and 6.5, respectively.

8.3 TEST RESULTS AND DISCUSSION

8.3.1 Mechanical Properties

8.3.1.1 Flexural Strength and Toughness

Figure 8.1 shows the mean values of flexural strength. Analysis of variance and comparison of means showed that plastic-concrete with 2% plastics produced a flexural strength comparable to that of control mixtures. The addition of recycled plastics produced a ductile mode of failure rather than the conventional brittle mode of failure, as shown in Figure 8.2. Flexural toughness of plastic normal-weight concrete improved by about 1.5 times when compared to control mixtures. This was confirmed statistically at 99% level of confidence.

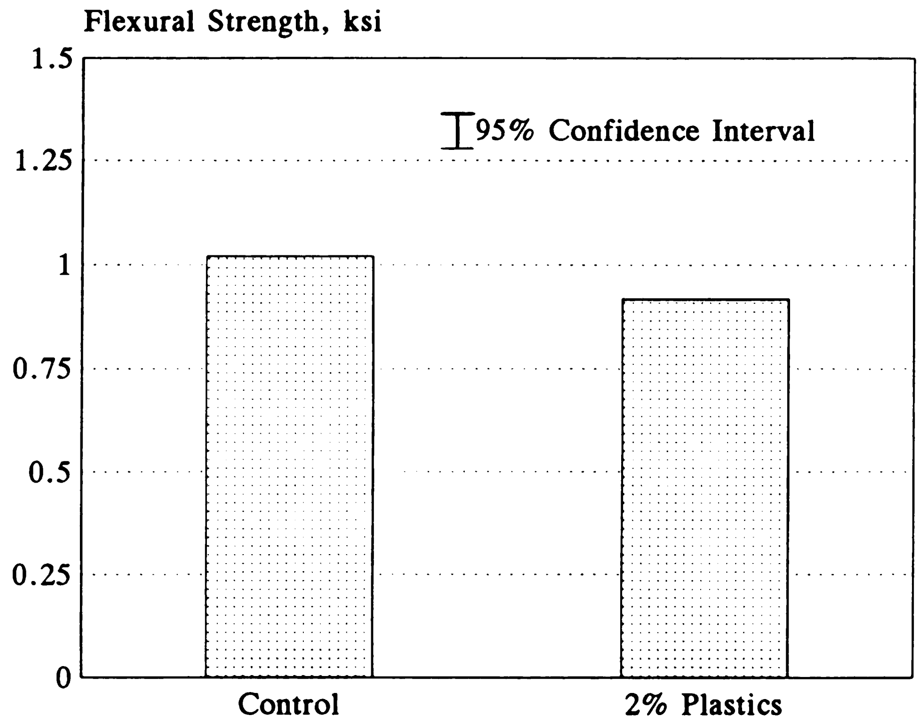


FIGURE 8.1 Flexural Strength Test Results (Means and 95% Confidence Intervals)

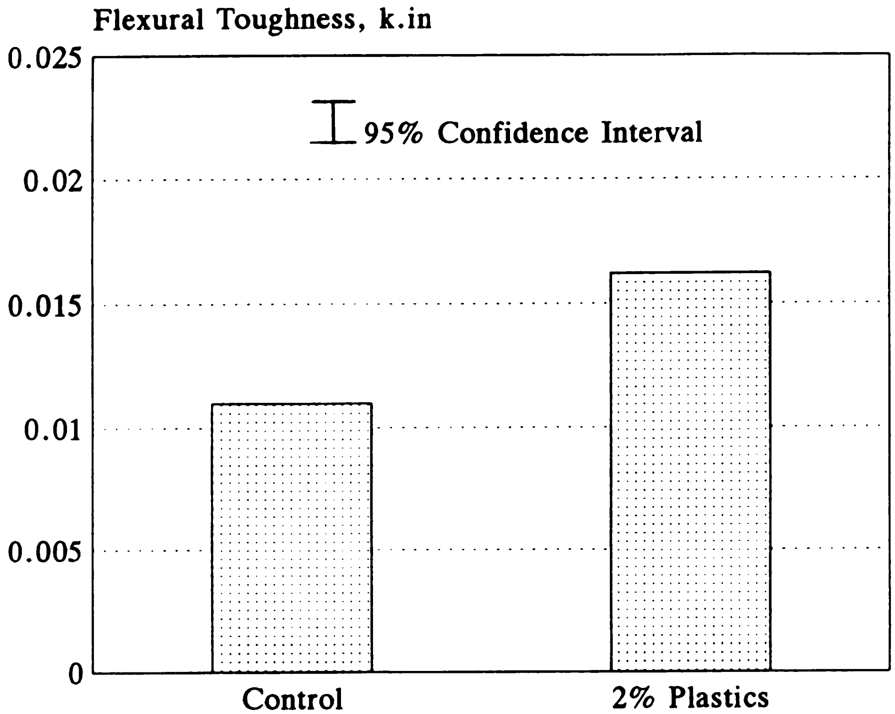


FIGURE 8.2 Flexural Toughness Test Results (Means and 95% Confidence Intervals)

8.3.1.2 Compressive Strength

The 28-day compressive strength test results for different mixtures are shown in Figure 8.3. Statistical analysis of variance and comparison of means indicated that the addition of 2% plastics (optimized plastic content) produced compressive strengths comparable to those of control normal-weight concrete mixtures. This was confirmed statistically at 95% level of confidence.

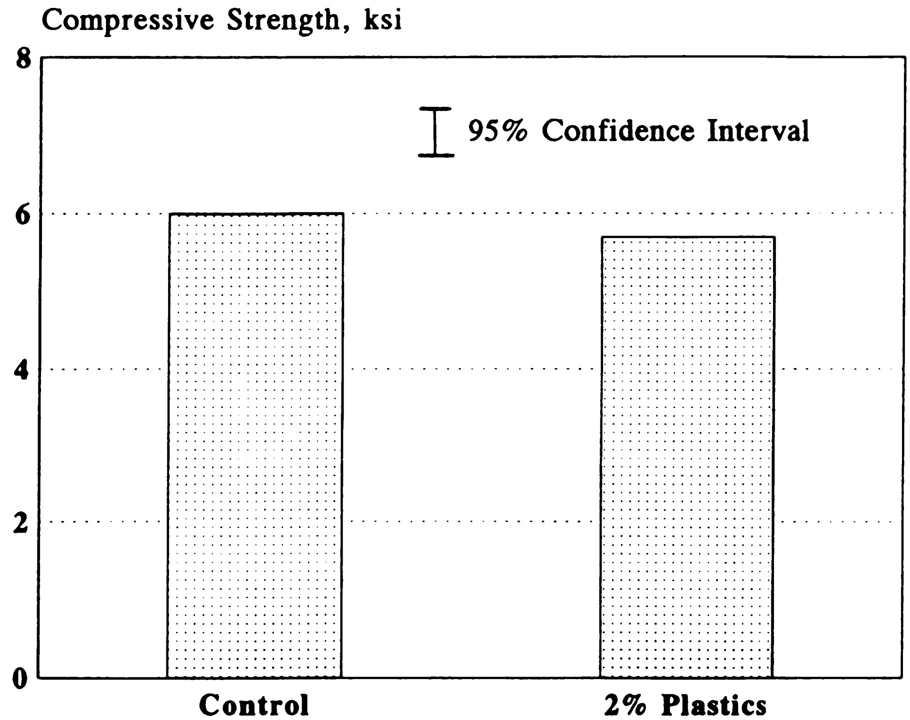


FIGURE 8.3 Compressive Strength Test Results (Means and 95% Confidence Intervals)

8.3.1.3 Impact Resistance

Figure 8.4 gives the mean values of the 28-day impact resistance test results, presented as number of blows to first crack and failure. Statistical analysis of variance (comparison of means) showed, at 95% level of confidence, that recycled plastics have a positive effect on the impact resistance beyond the initial crack up to failure.

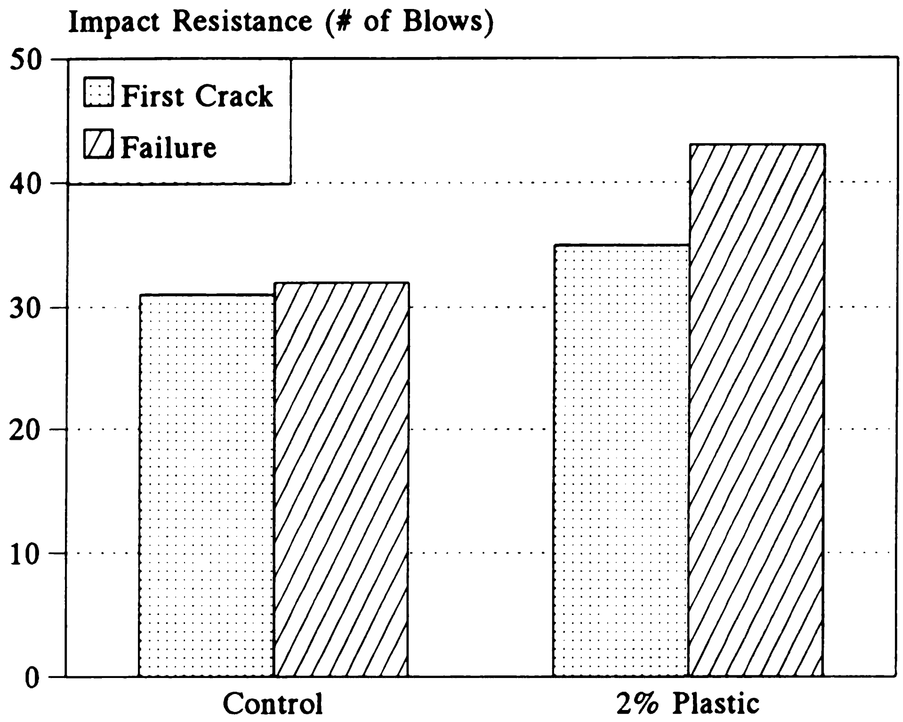


FIGURE 8.4 Impact Resistance Test Results (Means)

8.3.2 Long-Term Durability Characteristics

8.3.2.1 Freeze-thaw Durability

The relative dynamic modulus test results for normal-weight concrete incorporating recycled plastics are presented in Figure 8.5. Optimized mixtures incorporating 2% recycled plastics produced essentially similar performance when compared to control mixtures (i.e. no plastics).

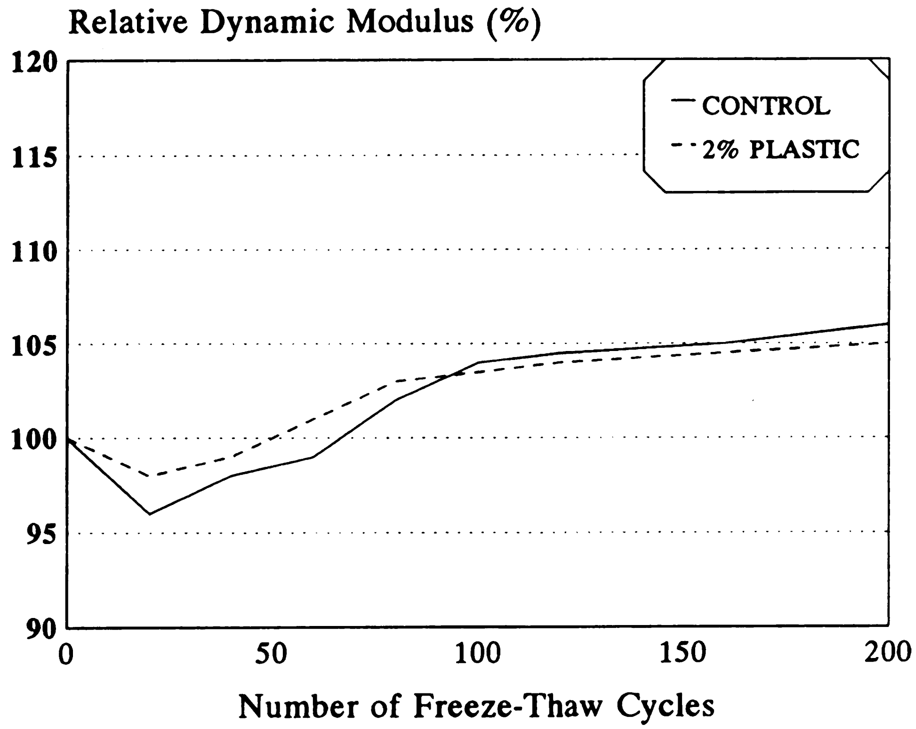


FIGURE 8.5 Freeze-Thaw Durability Test Results

8.3.2.2 Scaling Resistance

Table 8.1 shows the de-icer salt scaling resistance test results. Recycled plastics, at relatively low volume fractions, produced concretes with scaling resistance comparable to that of control mixtures. However, at higher volume fractions, recycled plastics had a negative effect on scaling resistance of resulting concrete (as observed in some preliminary tests at 5% or more plastic volume fractions). This can be attributed to the fact that plastic particles, which are impermeable, tend to block the bleeding water from escaping to the surface, producing cavities underneath plastics where later ice can form to pressure plastics out of concrete surfaces. The action of impermeable flat particles in preventing

water (e.g. osmosis) movements may also contribute to scaling damage at high plastic contents.

TABLE 8.2 Scaling Resistance Test Results

	# of Cycles	Rating
Control	50	1
Plastic Concrete (2% by total Volume)	50	1

8.4 SUMMARY AND CONCLUSIONS

The effects of recycled plastics as secondary reinforcing inclusions (additives) in normal weight concrete were investigated through evaluating concrete mixtures, with optimum plastic (ground HDPE) content of 2% versus control mixtures (without plastics). The results indicated that:

(1) Recycled plastics enhance the restrained shrinkage cracking characteristics of normal-weight concrete without adversely influencing any of the short-term mechanical properties of the material;

(2) The optimized mixture gave comparable flexural and compressive strengths when compared to control mixtures;

(3) The addition of recycled plastics, at 2% by total volume (optimum content), improved both flexural toughness and post cracking impact resistance of the concretes;

(4) Recycled plastics had no adverse effects on freeze-thaw durability of concrete; and

(5) Recycled plastics at the optimum volume fraction of 2% did not have a negative effect on the scaling resistance of concretes exposed to de-icer salts; however, at higher volume fractions one may see some adverse effects of plastics on the scaling resistance of normal-weight concrete.

PART III: TIRE CONCRETE

CHAPTER 9

RECYCLING OF TIRES IN CONCRETE MATERIALS

9.1 INTRODUCTION

Approximately 240 million used vehicle tires are generated in the United States each year. About 25% of these tires are reused or processed. The remaining 170 million are discarded in landfills or illegal disposal sites, or stockpiled above ground.

Tires usually contain a variety of rubber compositions as well as other constituents such as carbon black, synthetic fibers, steel wires, sulfur and zinc oxide, each contributing certain properties to the overall performance of tire. Rubber compounds designed for a specific function will usually be similar but not identical in composition and properties, although in some cases there can be significant differences between compounds in tires of various types. When coupled with cement and moisture, tire constituents tend to swell inside concrete matrices; this eventually leads to pop-out at outer surfaces of concrete.

Recycled tire inclusions can potentially relieve the stress intensity at sharp crack tips in brittle concrete matrices, and thus produce a composite material with enhanced resilience, toughness, energy absorption capacity, impact resistance and

cracking strength. The relatively low density of recycled tire inclusions would also improve the thermal insulation properties of concrete. Many applications of recycled tires as reinforcing inclusions such as slabs on grade can take advantage of the cracking resistance and toughness characteristics of tire-concrete composites.

9.2 BACKGROUND

About 240 million automotive, truck, and off-road tires are discarded in the United States each year. This is approximately equal to one waste tire per person per year. Additionally, there are 33.5 million tires that are retreated and an estimated 10 million that are reused each year as second-hand tires. It is estimated that 7% of the discarded tires are currently being recycled into new products and 11% are converted to energy. Nearly 78% are being landfilled, stockpiled, or illegally dumped, with the remainder being exported.

Tires are difficult to landfill. Whole tires do not compact well, and they tend to work their way up through the soil to the top. As a result, tire stockpiles, which cost less than landfills, have sprung up all over the country. It is estimated that between 2 and 3 billion tires are stockpiled in the U.S. at present, with at least one pile containing over 30 million tires. Tire stockpiles are unsightly and are a threat to public health and safety. Not only are tire piles excellent breeding grounds for mosquitoes, but they are also fire hazards.

Tire recycling activities include the use of whole tires or processed tires for useful purposes. Whole tire applications include reefs and breakwaters, playground equipment, erosion control, and highway crash barriers. Processed tire products include mats and other rubber products, rubberized asphalt, playground gravel substitute, and bulking agent for sludge composting.

Scrap tire combustion is practiced in power plants, tire manufacturing plants, cement kilns, pulp and paper plants, and small package steam plants (EPA, 1991).

9.3 PRODUCTION AND DISPOSAL

9.3.1 Volume of Production

It is commonly accepted in the tire industry that about one tire per person per year is discarded. Since there is no industry group or governmental agency that monitors tire disposal in the United States, the best estimates that can be made are based on tire production. The Rubber Manufacturers Association (RMA) records the number of original equipment, replacement, and export tires that are shipped each year in the United States. In 1990, a total of 264,262,000 tires were shipped. The RMA data include new tire imports, but not imported used tires.

Figure 9.1 shows the estimated disposition of the 240 million scrap tires generated in 1990. About 16.3 million were recycled, 26 million were recovered for energy, and about 12 million were exported, leaving 188 million for landfilling, stockpiling, or illegal dumping. Figure 9.1 shows that in 1990, 17.4% of the tires scrapped were recycled or burned for energy (EPA, 1991).

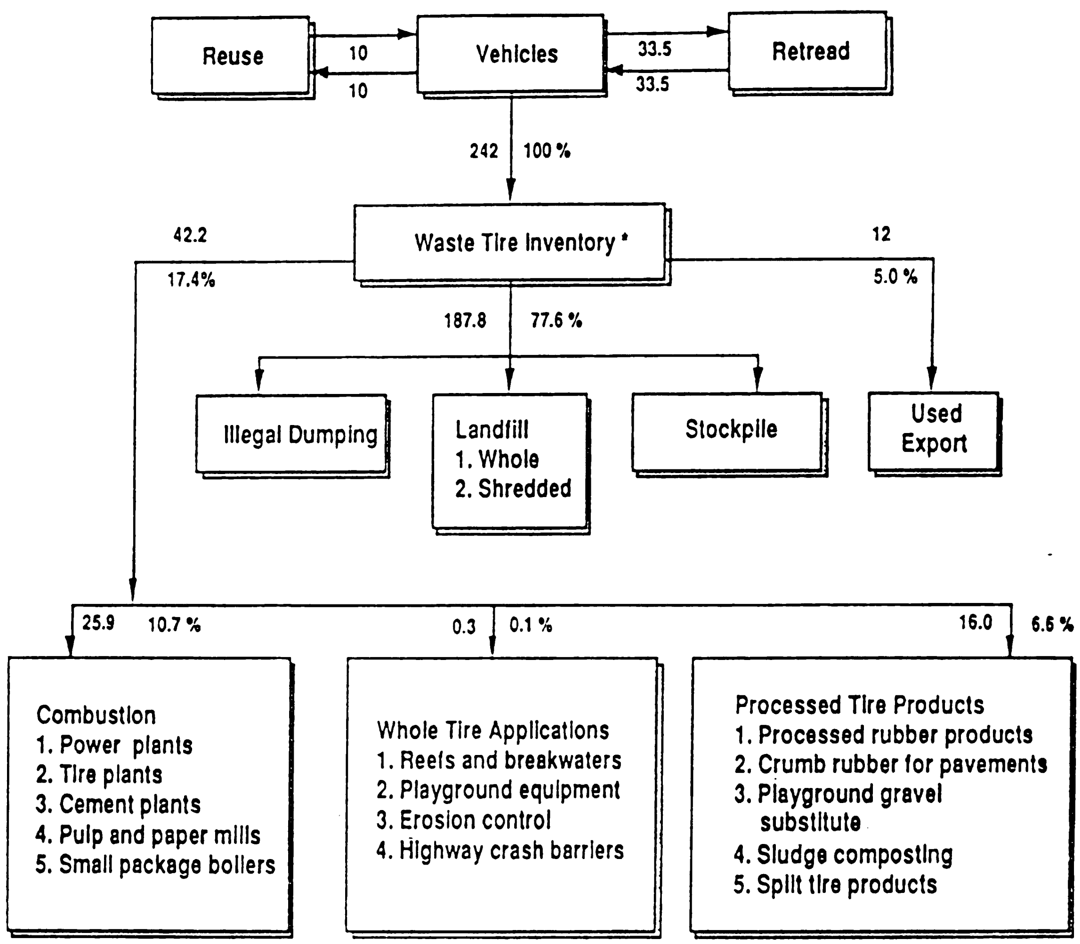


FIGURE 9.1 Flow Diagram Showing Estimated Destination of Scrap Tires in 1990 (In Millions of Tires and Percent; EPA, 1991)

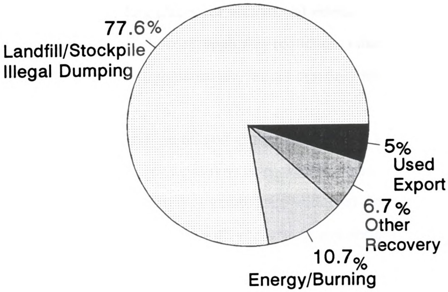


FIGURE 9.2. Destination of Waste Tires in 1990 (Fletcher, 1980).

9.3.2 Disposal Problems

A wide range of scrape tire disposal technique are available or being investigated. Numerous diverse schemes, such as artificial tire reefs, dock fenders, synthetic turf, motor way crash barriers, fillers for building and road materials, and filling in river beds, railway cutting and marshlands have been contemplated, but inevitably have had only minor impact on the total disposal problem.

Methods such as tire splitting, and reclaim and crumb recovery (most of which is recycled to the tire industry) are carried out, but do not handle large truck tires and produce by-product wastes with similar disposal problems.

An increasing majority of landfills refuse to accept any tires because they do not biochemically degrade when buried and can "float" to the top, breaking

through the cover causing damage which requires costly repairs. Tire piles provide excellent breeding places for vermin and insets.

The recycling of rubber through size reduction, either by shredding and grinding or by cryogenic fragmentation, would certainly attack a large market but is expensive.

In the United States, an oil and ground-tire mixture has been burned to produce carbon black for re-use in new tires, while a stoker-fired boiler has been operated using a mixture of 10% shredded tire with coal. Biochemical degradation, by mixing steel-free tire particles with yeast/fungi and certain chemicals, has also been reported for producing a soil conditioner, although, without a market, the process is uneconomic.

Dumping is generally an expensive operation, is a waste of a value resource, and is normally discouraged by local authorities because such sites are fire risks as well as being potentially unsightly. Where sites are not scarce, inaccessibility can be a problem even when the normally low bulk-density of tires is more than doubled by shredding. Several furnaces, operating worldwide, have been designed specifically for burning tires. However, incineration of tires without heat recovery is disadvantageously placed in an energy-conscious environment. The incorporation of a waste-heat recovery system is attractive, provided economical operation is possible. The high calorific value of a tire is troublesome, the air-fuel ratio must be maintained within close limits to ensure the required fuel-vaporization rate and maintain the ignition temperature; insufficient quantities of air result in the production of explosive mixtures. Even so, the combustion is violent and flame extinction is always a possibility. The relatively high sulfur content (up to 2%) results in sulfur dioxide formation and subsequent corrosion problems. To comply with the requirements of the Clean Air Act, it is necessary to install sophisticated gas clean-up plant. The high temperature of

operation, 871°C (1600°F) in some cases, necessitates temperature resistant construction materials with associated additional costs. The high steel content of tires can cause major problems with moving grate incinerator. For efficient and acceptable treatment, specially designed incinerators are therefore required, although tire-shredding and/or mixing with general waste materials can reduce this problem and allow the use of conventional units (Fletcher, 1980).

According to the Maine DEP (1989), fire is a particularly important problem, since tire fires emit heavy black smoke into the air, and release petroleum and other chemical contaminants into surface and ground water. Once ignited, tire fires are extremely difficult to contain and extinguish, requiring large amounts of manpower and equipment.

Anyone who has ever handled an old tire realizes the impossibility of dumping all the water out of one. Such water creates breeding habitat for mosquitoes, known to carry yellow fever and many forms of encephalitis. According to the Maine DEP (1989), "In 1986, the US. Environmental Protection Agency estimated that health care costs for scrap tire related encephalitis totaled $5. 4 million. This cost includes medical care as well as lost earning due to illness".

9.4 PROPERTIES OF TIRE

9.4.1 Chemical Composition

Today's tire is made to last under extremely severe physical, thermal, and chemical conditions and is practically indestructible. Over years tires have developed to be chemically complex, precision engineered and designed not to

come apart. A tire only becomes a scrap tire because the tread is worn off or it has been physically damaged. The material out of which a tire is made the fabric, fiber glass, wire and rubber remain essentially as good as when they were introduced to make a new tire. The most common tire rubber is styrene butadiene copolymer (SBR), containing about 25% by weight styrene. In combination with SBR, other elastromers such as natural rubber (cis-polyisoprene), synthetic cis-ployisoprene, and cis-polybutadience are also used in tires in varied amounts. A typical recipe for tire rubber is given in Table 9.1 (Fader, 1990).

Table 9.1 Typical Chemical Composition of Tire Rubber (Fader, 1990)

Component	Weight (%)
SBR	62.1
Carbon Black	31.0
Extender Oil	1.9
Zinc Oxide	1.9
Stearic Acid	1.2
Sulfur	1.1
Accelerator	0.7

A typical analysis of a de-beades scrap tire suggests that tire is a mixture of rubber, rayon, nylon, polyester, carbon-black, and natural and synthetic rubbers. Steel (up to 17%), non-ferrous metal (mainly as the vulcanizing agent, zinc oxide) and sulfur are also present in significant quantities in the whole tires.

9.4.2 Physical Properties

A pneumatic tire generally fulfills the following functions: (1) Allows a comparatively free and frictionless motion of the vehicle through rolling; (2)

Distributes vehicle weight over a substantial area of ground surface, thus avoiding excessive stress on the latter and on the wheel; (3) Cushion the vehicle against road shocks; (4) Transmits engine torque to the road surface with a low power consumption; (5) Permits, through tire adhesion, the generation of substantial braking, driving and steering loads; and (6) Ensures lateral and directional stability. No other device exists today which can fulfill these varied function in an efficient manner as the pneumatic tire. To fulfill these purpose, passenger-car radial ply tires are usually built of one to four radial plies of rayon, nylon or polyester. The belt or breaker consists of either two layers of steel cords, or four to six layers of textile cords. These are generally thicker than the cords used in the radial portion of the casing and therefore have a considerably increased stiffness. For truck and heavy-duty tires, it is useful to have a single layer of radial cords and two or three layers in the belt, all of steel construction. Often, additional reinforcing strips of cross-biased fabric extend a short way up the sidewalls of radial-ply tires. The introduction of fibrous glass as a substitute reinforcement for textiles in carcass design deserves mention. Having received special chemical treatment, glass fibers permit greater adhesion to rubber and give increased stability in withstanding dynamic stress when embedded in elastomeric material. Moisture and low elongation no longer appear to be problems in using fibers for rubber reinforcement. Table 9.2 (Moore, 1975) compares some of the physical properties of glass, rayon, nylon and polyester reinforcement in tires, while Table 9.3 (Rodrigues, 1970) shows the typical properties of cross-linked rubber compounds of the most commonly used rubber tires.

TABLE 9.2 Physical Properties of Reinforcement Structures in Tires (Moore, 1975)

	Glass	Rayon	Nylon	Polyester
Ultimate tensile strength psi*1000	407	94	122	104
Ultimate Elongation, %	4.83	9.8	19.3	18.5
Toughness, psi	9,900	5,800	10,200	9,900
Modulus, 1000 psi	8,450	960	630	570
Breaking strength, lbf	79.0	39.1	33.2	32.1
Filament Dia. (mls)	17	26	21	24

TABLE 9.3 Typical Properties of Cross-Linked rubber Compounds Used in Tires (Rodrigues, 1970)

	Polyisoprene (natural rubber, also made synthetically)	Polybutadine	Styrene-butadine random copolymer 25 wt% styrene SBR
Gum Stock (cross-linked, unfilled):			
Density, gm/cm^3	0.93	0.93	0.94
Tensile Str., psi	300-2500	200-1000	200-400
Reinforced Stock:			
Tensile Str., psi	3000-4000	2000-3500	2000-3500
Elong. at Break, %	300-700	300-700	300-700
Resistance To:			
Acid	Good	Good	Good
Alkali	Good	Good	Good
Oxidation	Good	Good	Good

9.5 RESOLVING OF POP-OUT OF TIRE PARTICLES

9.5.1 Objectives

Preliminary work on tire-concrete composites indicated that dimensional instability of tire in the presence of moisture, particularly at elevated temperatures, is a problem; surface pop-out is a result of the dimensional movements of tire particle.

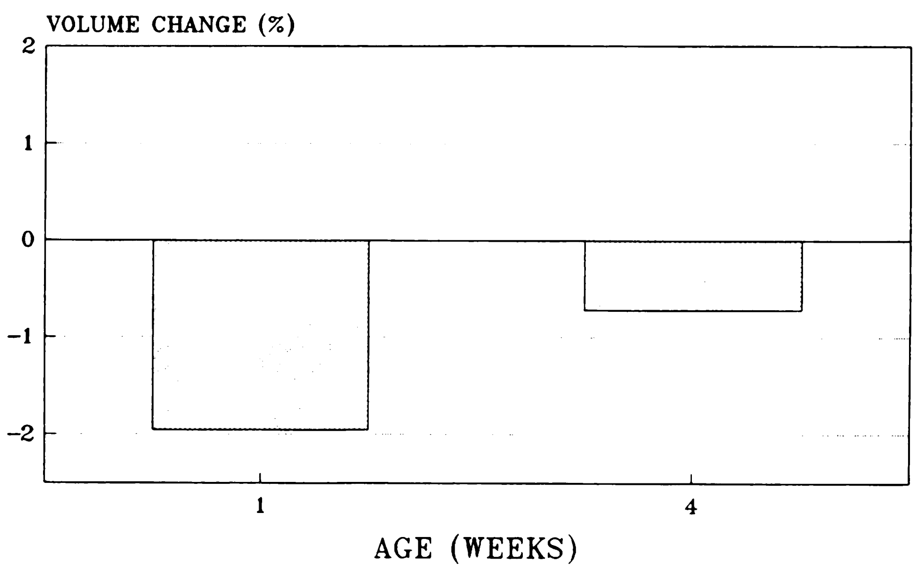
Potential chemical and physical causes of dimensional instability of tires causing pop-out were investigated in order to resolve the problem. For this purpose, concrete specimens incorporating tire particles, subjected to different treatments (aimed at controlling the chemical or physical causes of pop-out) were exposed to different environments where pop-out could occur.

9.5.2 Approach

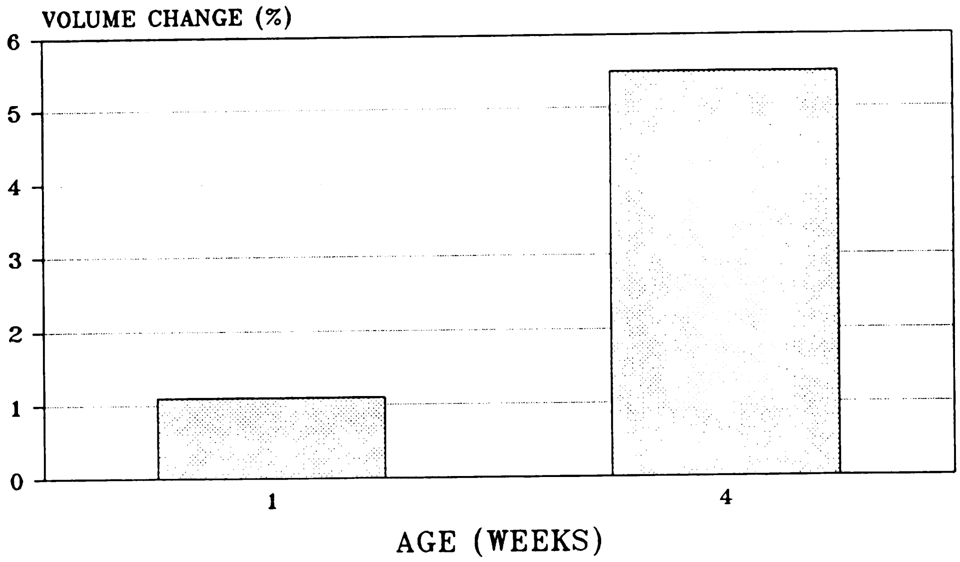
Based on alternative hypotheses regarding the pop-out mechanisms, different treatments were considered for mitigating the pop-out problem. These treatments included: immersion in lime solution, immersion in cold and hot water and other chemicals, and coating with epoxy and acrylic latex. Immersion of tire particles in lime solution for one week prior to mixing in concrete helped reduce the pop-out problem in moist conditions at ambient temperature 23^{0}C (73^{0}F), but when immersed in water at high temperatures of approximately 60^{0}C (140^{0}F), it actually pronounced pop-out.

To further investigate the pop-out problem of tires in concrete materials at high temperatures, different chemical pretreatments were investigated with limited success. These treatments included immersion in solutions of coal fly ash, coal fly ash with lime, cold and hot water, sodium hydroxide, aluminum oxide, Kaolin, calcium chloride, sodium silicate, potassium hydroxide, aluminum sulfate, and silica fume slurry. These treatments were supposed to remove different chemical causes of pop-out or physical inhibit it. The lack of any considerable success led to the hypothesis that pop-out is a physical problem caused by expansion of tire in the presence of moisture (and particularly at high temperatures) rather than a chemical one.

Tires were subsequently immersed in cold/hot water at different time intervals in order to measure the volume change and water absorption. Figures 9.3a and 9.3b shows the volume changes of the tire particles in cold and hot water, respectively.



(a) Ambient Water Immersion



(b) Hot Water Immersion

FIGURE 9.3 Volume Change of Tire Particles

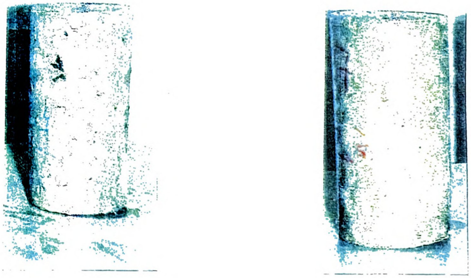
It is observed in Figure 9.3 that tire particles, when immersed in ambient temperature water, tend to shrink and at longer periods of time start to expand, but when immersed in hot water they expand from the beginning. The volume change of tires at different water temperatures is partly due to thermal expansion. It should be noted that the expansion of tires in moist/hot condition is more severe than that in moist/cold condition. This further explains why tire particles at moist/hot condition tend to cause more severe pop-out than in moist/cold environments.

In order to prevent tire expansion upon exposure to moisture, tire particles were coated with HydroEpoxy 104[1] (water based), which is commercially available. This resulted in eliminating the tire pop-out(problem (see Figure 9.4).

[1] HydroEpoxy 104 is a commercially available Epoxy produced by ACME Chemicals Ltd, New York.

The combination of reducing the amount of water reaching the tire particles and providing an elastic layer around the tire particles prevented the tire particles from causing pop-out in concrete materials. This was partly due to the fact that as tires particles tend to expand, the thin elastic layer around the particles was able to withstand the pressure caused by the tires and act as a "bumper" to prevent the tires from applying pressure on the thin concrete surface which leads to pop-out However, this method proved to be relatively costly due to the high cost of HydroEpoxy 104.

Acrylic latex, Synthemul 40401-00[2] which is water based and commercially available, was then used to coat the tire particles. This method produced same results as the HydroEpoxy coating at reduced cost.



(a) Uncoated Tire Particles (b) Coated Tire Particles

FIGURE 9.4 Epoxy Coating Effects on Pop-Out in Hot Water

[2] Synthemul 40401-00 is a commercially available Acrylic latex produced by Reichhold Chemicals, Ltd.

9.6 TIRES AS REINFORCING INCLUSION IN CONCRETE

9.6.1 Objective

Following the elimination of pop-out problem through coating of tire particles with polymeric materials, the effectiveness of coated and uncoated tires as reinforcing inclusions in concrete was demonstrated. The fact that coating increases the cost of recycled tire particles encourages higher-value use of the coated particles (e.g. as reinforcing inclusions rather than light-weight aggregates). Coated tire particles were incorporated into concrete at 2.5 and 5% volume fractions and uncoated tires at 5% volume fraction. Control mixtures without tire were also considered. The effects of tire inclusions on the following properties of concrete were investigated: restrained drying shrinkage, flexural strength and toughness, compressive strength, impact resistance and chloride permeability.

9.6.2 Materials and Mix Proportions

The basic ingredients of the concrete materials used in this phase of the research were type I Portland cement, normal-weight aggregate (fine and coarse with a maximum size of 12 mm, 0.5 in.), water, recycled tires with maximum size of 12mm (0.5 in.), and air entraining agent (water based). The gradation of coarse and fine aggregates met the ASTM C-33 requirements.

The concrete mixtures used in this phase had fine aggregate/cement and coarse aggregate/cement ratios of 2 and 2.5, respectively. Air entraining agent was used at 0.06% by weight of cement to produce resistance against frost attack. The

water content was adjusted for all mixtures to give similar slumps (44.5 to 57.2 mm, 1.75" to 2.25"). Water-cement ratio ranged from 0.39 to 0.43.

For the normal-weight mortar mixtures, the cement:sand:water ratio was 1:2.4:0.5. The flow (ASTM C-230) ranged from 80 to 95%.

9.6.3 Test Procedures

For the hardened materials, the flexural strength and toughness, compressive strength, impact resistance, restrained shrinkage cracking characteristics and chloride permeability properties were investigated experimentally in order to develop an overall understanding of various aspects of material behavior.

The flexural test specimens were 100x100x350 mm (4"x4"x14") prisms, and the compressive strength test specimens were 3"x6" (75x15 mm) cylinders. Flexural and compression tests were conducted following ASTM C-78 (four point loading) and C-39 procedures. Midspan deflection as well as loads were monitored in flexure tests. The impact test was conducted following the procedures recommended by ACI Committee 544. This test measures the amount of impact energy (represented by the number of blows) necessary to start a visible crack in concrete and then continue the opening of crack until failure.

Ring type specimens are used for restrained drying shrinkage test on mortar. The specimen is cast in two equal layers, leveled by trowel, and then covered with plastic sheets for 6 hours. The specimens are then exposed to air at approximately 23°C (73°F) and 40% R.H. Restraint of shrinkage movements by the steel ring inside the specimen creates internal tangential tensile stresses which cause cracking.

Permeability tests were conducted using AASHTO T-277 (Rapid Determination of the Chloride Permeability of Concrete). This test measures the amount of charge passed through a concrete specimen subjected to permeation of chloride ions at 60 VDC for 6 hours. The total charge passed (in Coulombs) is related to chloride ion permeability. The more permeable the concrete, the higher would be the Coulombs. A cylindrical specimen. 102 mm (4 in) in diameter by 51 mm (2 in.) in thickness is used for this test.

9.6.4 Test Results and Discussion

9.6.4.1 Flexural Strength and Toughness

Figures 9.5 and 9.6 show the flexural strength and toughness test results of normal-weight concrete incorporating recycled tires.

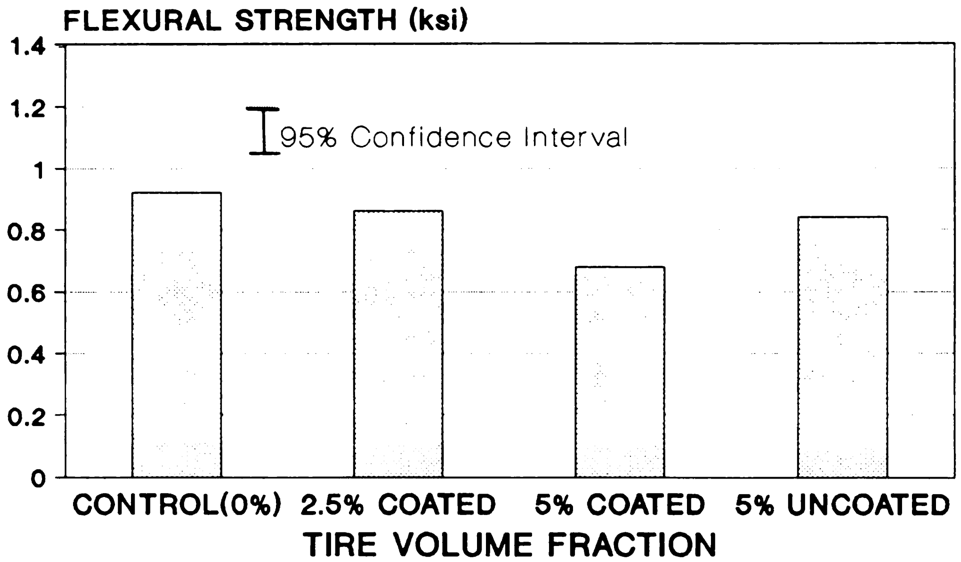


FIGURE 9.5 Flexural Strength Test Results at 28-Days (Means and 95% Confidence Level)

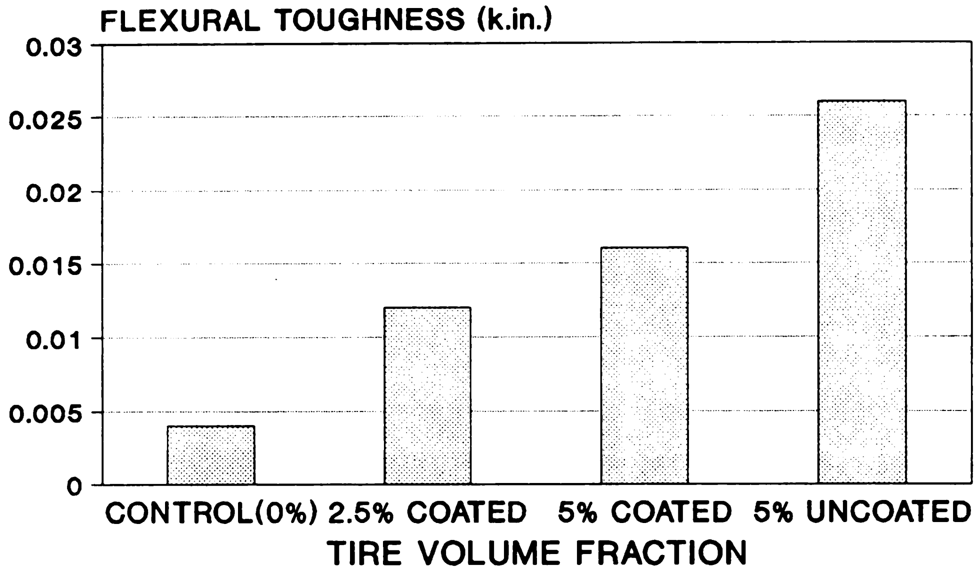


FIGURE 9.6 Flexural Toughness Test Results at 28-Days

Although the flexural strength of concrete incorporating recycled tires dropped by 6, 35 and 9% at 2.5 and 5% coated, and 5% uncoated, respectively, when compared to control mixtures (without tires), they were statistically comparable at 95% level of confidence, except for 5% treated tire. The drop in flexural strength, particularly at 5% addition of coated tire can be attributed to the weak bond between coated tire inclusions and cementitious matrices.

Figure 9.6 is indicative of the positive effects of recycled tires on the post-peak ductility and toughness of concrete materials in flexure. The flexural toughness of concrete incorporating 2.5 and 5% coated tire, and 5% uncoated (as additives), improved by about 3, 4 and 6.5 times, respectively, when compared to control mixture. The significant increase in flexural toughness in the presence of recycled tires can be attributed to the reinforcing action of the tough tire inclusions in the brittle cementitious matrices. In spite of the potentially weak bonding of the tire particles to concrete matrix, the fact that cracks can not go through tire inclusions but have to follow longer paths around their interfaces seem to enhance the fracture properties of concrete.

9.6.4.2 Compressive Strength

The compressive strength test results are shown in Figure 9.7. The addition of tire inclusions as additives in normal-weight concrete had an adverse effect on compressive strength.

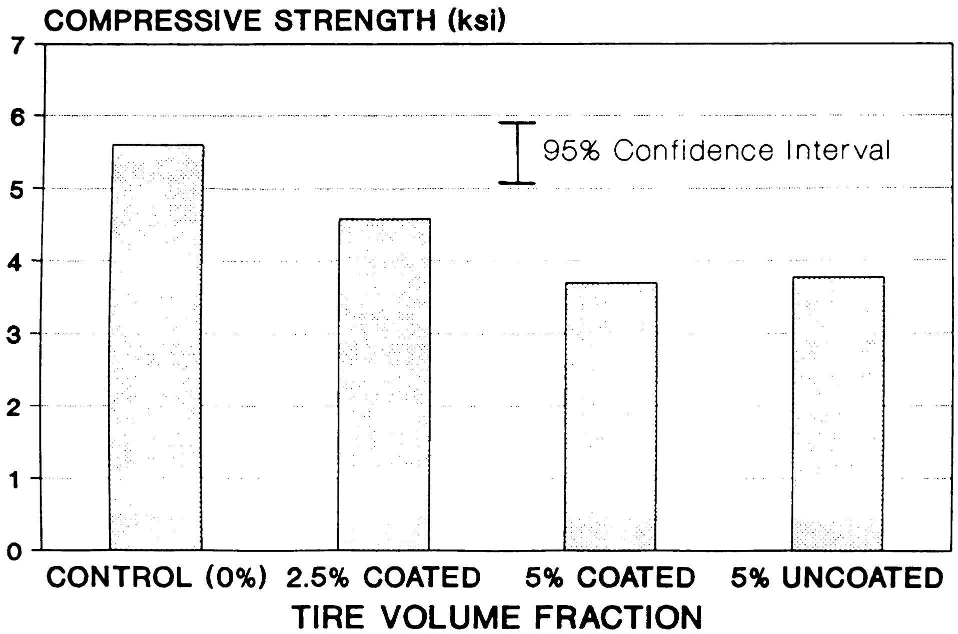


FIGURE 9.7 Compressive Strength Test Results at 28-Days (Means and 95% Confidence Level)

The compressive strength dropped by 18, 33 and 32% when 2.5 and 5% coated, and 5% uncoated tires were added to normal-weight concrete, respectively.

The reduction in compressive strength was confirmed statistically at 95% level of confidence. The drop in compressive strength was expected because of lower modulus of elasticity of tire when compared to that of concrete (about 10 times less) leading to a redistribution of stresses into concrete which is a more rigid matrix. It should, however be noted that limits on load-carrying capacity and service life of concrete structures are generally provided by the resistance of concrete to tensile stresses and impact loads. Concrete is fairly strong in compression and concrete structures rarely fail in compression.

9.6.4.3 Impact Resistance

Figure 9.8 shows the impact resistance test results measured as the number of blows up to the initial crack and failure. The difference between the initial crack and failure represent the post-cracking energy absorption capacity of the material.

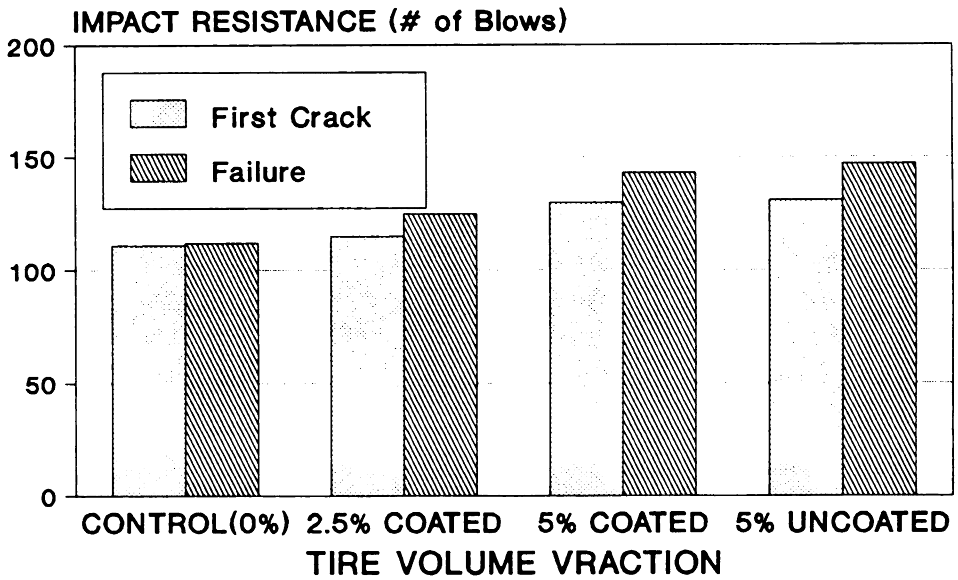


FIGURE 9.8 Impact Resistance Test Results at 28-Days

One-way analysis of variance and comparison of means showed that, at 99% level of confidence, the recycled tires have a significant positive effect on the impact resistance of concrete beyond the initial crack up to failure. The improvements in impact resistance between initial crack and failure in the presence of recycled tires in concrete materials further validates the hypothesis that tough tire inclusions help enhance the fracture energy and toughness characteristics of concrete materials through bridging of cracks by the tire inclusions.

9.6.4.4 Restrained Drying Shrinkage

Figure 9.9 shows the maximum crack width versus time in restrained shrinkage test on normal-weight concrete. The addition of recycled tires to normal-weight concrete helps delay and control the drying shrinkage cracks. This can be attributed to the fact that recycled tire inclusions act as reinforcing inclusions that arrest microcracks and bridge across cracks to restrain their widening.

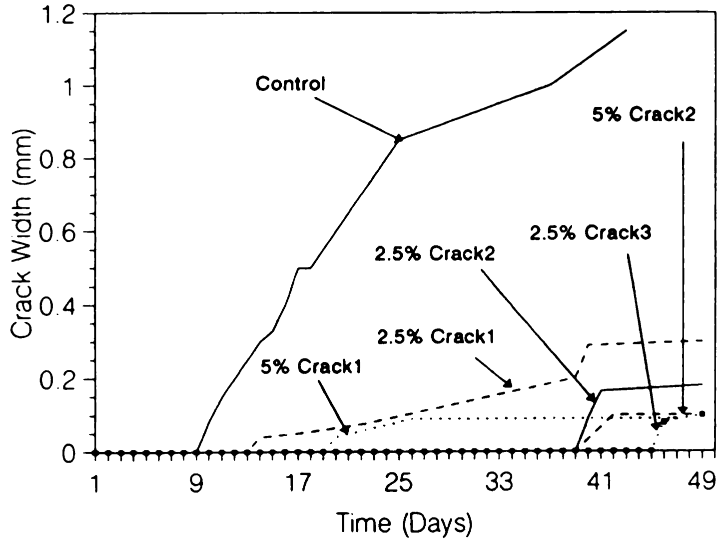


FIGURE 9.9 Restrained Drying Shrinkage Vs. Time

9.6.4.5 Permeability

The chloride permeability test results (Figure 9.10) indicates that incorporating coated tire particles at 2.5 and 5% volume fractions and uncoated tires at 5% volume fraction produced permeability levels comparable to that of control. Although the addition of 5% coated tires as additive in normal-weight concrete increased permeability by an average of 6%, one way analysis of variance and separation of means showed that this increase was not statistically significant at 95% level of confidence. In actual feild conditions, control of restrained shrinkage cracking by tire inclusions could lead to reduced permeability of tire-concrete composites.

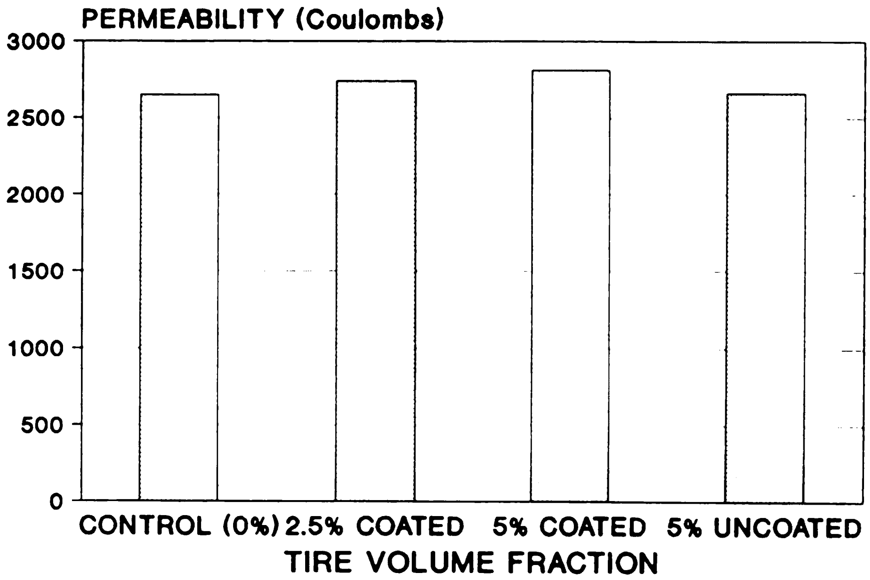


FIGURE 9.10 Permeabilty Test Results.

9.7 SUMMARY AND CONCLUSIONS

The dimensional instability of tire particles in the presence of moisture, which is pronounced at elevated temperatures and eventually leads to surface pop-out was investigated. Following the elimination of pop-out through coating of tire particles with polymeric materials, the effects of tire inclusions on: restrained drying shrinkage, flexural strength and toughness, compressive strength, impact resistance were investigated. The environmnetal impact of tire concrete was alo checked. The following conclusions were derived:

1. Coating of tire particles with polymeric materials proved to eliminate surface pop-out of concrete particles even in extreme conditions of immersion in hot water.

2. The addition of recycled tires to normal-weight concrete helps delay restrained drying shrinkage cracking and reduce the crack width.

3. Coated recycled tires at 2.5% and uncoated recycled tire at 5% volume fraction produced flexural strengths comparable to that of control concrete mixtures. However, statistical analysis of variance showed that coated tire at 5% volume fraction reduced flexural strength. This was confirmed statistically at 95% level of confidence.
4. The flexural toughness of concrete incorporating 2.5 and 5% coated tire, and 5% uncoated tire, increased by about 3, 4 and 6.5 times when compared to control concrete mixtures. this was cnfirmed statistically at 99% level of confidence.

5. Compressive strength test results were indicative of the adverse effects of recycled tires on compressive strength. It should be noted that concrete is fairly strong in compression, and brittle failure mode and low crack resistance are the main problems which are targeted to be overcome through the use of soft reinforcing inclusions.

6. Recycled tires have a significant possitive effect on impact resistance of concrete beyond the initial crack upto failure.

7. Chloride permeability test results showed that coated tire and uncoated tire produced results which were statistically comparable to those obtained with control concrete.

CHAPTER 10

SUMMARY AND CONCLUSIONS

10.1 INTRODUCTION

Recycling in construction presents the potentials for high-volume use of waste materials in products with long service life, while avoiding costly separation and purification steps. This research focused on the use of recycled plastic flakes as reinforcing and light-weight inclusions in light-weight concrete, and as secondary reinforcing inclusions in normal-weight concrete. The effectiveness of recycled tires as secondary reinforcement in normal-weight concrete was also demonstrated.

10.2 PLASTIC LIGHT-WEIGHT CONCRETE

The effects of partial substitution of light-weight aggregates with recycled plastic flakes on concrete properties were investigated. The short-and-long-term properties of the materials were assessed. The following conclusions were derived through analysis of the generated data:

(1) The addition of recycled plastics to light-weight concrete helps reduce the drying shrinkage crack widths;

(2) Recycled plastics at 7.5 and 15% volume fractions produce flexural strengths comparable to that of control concrete mix. However, flexural toughness increases by 4.5 and 8 times, respectively. This was confirmed statistically at 99% level of confidence;

(3) Compressive strength test results were indicative of the adverse effects of recycled plastics on compressive strength; however, the addition of fly ash to the cementitious matrix produced compressive strengths comparable to control mixtures. It should be noted that concrete is fairly strong in compression, while brittle failure mode and low crack resistance are the main problems which are targeted to be overcome through the use of reinforcing inclusions. Furthermore, since the reduction in compressive strength is accompanied with the reduction in unit weight, the situation would be improved if one looks at the compressive strength-to-weight ratio;

(4) Recycled plastics have a significant positive effect on the post- cracking impact resistance of concrete;

(5) Some aspects of the flexural performance of light-weight concrete materials incorporating plastics are improved through sulfonation of plastics; the overall mechanical properties (i.e. compressive strength and impact resistance), however, were not substantially improved by the sulfonation of plastics;

(6) Increasing the plastic content leads to improved flexural toughness and post-cracking impact resistance; but adversely influences the compressive strength; excess plastic contents may also damage flexural strength;

(7) The optimum plastic content would depend on the importance of different properties for various applications; if one can tolerate a drop of 25% in compressive strength, when compared to control mixtures, 11% plastic content (by total volume) will improve flexural toughness (about 6 times) and post-cracking impact resistance (about 3 times) without adversely influencing flexural strength; this was confirmed statistically at 95% level of confidence;

(8) Theoretical studies of plastic-lightweight concrete composites under compression indicated that the redistribution and concentration of stresses induced in concrete by soft inclusions (with relatively low elastic modulus and relatively high Poisson's ratio) leads to increased maximum compressive and particularly tensile stresses, increased displacements and reduced elastic modulus of light-weight concrete; increasing levels of maximum stress and deformation are observed with perfectly bonded plastics,

imperfectly bonded plastics, and empty voids; increased volume fraction of the soft inclusions leads to increased maximum stress and deformation;

(9) The experimentally observed negative effects of plastics on compressive performance suggest that any positive effects of plastic inclusions (through the arrest and bridging of cracks) would be overshadowed by the adverse effects of plastics on maximum compressive and particularly tensile stresses;

(10) The addition of recycled plastics to light-weight concrete improved the freeze-thaw durability characteristics of the resulting composite, noting that the specific light-weight aggregate used in this investigation made concrete highly susceptible to frost attack;

(11) Recycled plastics had a negative effect on scaling resistance of concrete, again noting the very low scaling resistance of the control light-weight concrete associated with its frost susceptibility; and

(12) Sulfonated plastics produced levels of freeze-thaw durability characteristics and scaling resistance which were comparable to those obtained with unsulfonated plastics.

10.3 PLASTIC NORMAL-WEIGHT CONCRETE

Normal-weight concretes incorporating recycled HDPE plastics, as secondary reinforcing inclusions (additives) were considered in this research. Plastic volume fraction and plastic width were optimized in order to achieve a condition where minimum plastic content would effectively reduce the restrained shrinkage crack width. The optimized composite was then compared to control mixtures (i.e. without plastic). The conclusions derived are summarized below.

(1) Recycled plastics can effectively reduce the restrained shrinkage crack widths in concrete; this can be attributed to the capability of plastics to bridge across cracks, restrain their opening, and provide post-cracking tensile resistance across cracks;

(2) While plastic volume fraction significantly influenced restrained shrinkage crack widths, the width of plastic particles did not have statistically significant effects;

(3) A plastic volume fraction of 2% represented the minimum (optimum) plastic content where restrained shrinkage crack widths were effectively controlled; ground plastic flakes and plastic "fibers" used in this investigation where both effective at this volume fraction;

(4) The optimized mixture gave comparable flexural and compressive strengths when compared to control mixtures;

(5) The addition of recycled plastics, at 2% by total volume (optimum content), improved both flexural toughness and post cracking impact resistance of concrete;

(6) Recycled plastics had no adverse effects on freeze-thaw durability of concrete; and

(7) Recycled plastics at the optimum volume fraction of 2% did not have a negative effect on the scaling resistance of concretes exposed to de-icer salts; however, at higher volume fractions one may see some adverse effects of plastics on the scaling resistance of normal-weight concrete.

10.3 TIRE CONCRETE

The dimensional instability of tire particles in the presence of moisture, which is pronounced at elevated temperatures and eventually leads to surface pop-out, was investigated. Following the elimination of pop-out through coating of tire particles with polymeric materials, the effects of tire inclusions on some mechanical properties were determined. The following could be concluded:

(1) Coating of tire particles with polymeric materials proved to eliminate surface pop-out of concrete particles even in extreme conditions of immersion in hot water;

(2) The addition of recycled tires to normal-weight concrete helps delay restrained drying shrinkage cracking and reduce the crack width;

(3) Coated recycled tires at 2.5% and uncoated recycled tire at 5% volume fraction produced flexural strengths comparable to that of control concrete mixtures. However, statistical analysis of variance showed that coated tire at 5% volume fraction reduced flexural strength. This was confirmed statistically at 95% level of confidence;

(4) The flexural toughness of concrete materials incorporating 2.5 and 5% coated tire, and 5% uncoated tire, increased by about 3, 4 and 6.5 times, respectively, when compared to control concrete mixtures;

(5) Compressive strength test results were indicative of the adverse effects of recycled tires on compressive strength. It should be noted that concrete is fairly strong in compression, and brittle failure mode and low crack resistance are the main problems which are targeted to be overcome through the use of soft reinforcing inclusions (e.g. plastics and tire);

(6) Recycled tires have a significant positive effect on impact resistance of concrete beyond the initial crack up to failure; and

(7) Chloride permeability test results showed that coated tire and uncoated tire produced results which were statistically comparable to those obtained with control concrete.

APPENDIX A

APPENDIX A

EFFECT OF AGGREGATE PROPERTIES ON CONCRETE PROPERTIES

A.1 INTRODUCTION

Aggregates are the major constituents of concrete, they usually occupy about 75% of the concrete's volume. They are critical to the performance of fresh mixed and hardened concrete. In addition to serving as an inexpensive filler, they have certain positive benefits to concrete. This report will give a brief review on the effect of aggregate properties on some Portland cement concrete properties.

A.2 EFFECT OF AGGREGATE GRADING

It has been reported by the American Concrete Institute (ACI, 1978) that there are several reasons for specifying both grading limits and maximum aggregate size in concrete applications. Aggregates having a smooth grading curve and neither a deficiency nor excess of any one particle size will generally produce mixtures with fewer voids between particles. Since cement costs more than aggregate and the cement paste requirement for concrete increases with increasing void content of the combined aggregates, it is desirable to keep the void content aslow as possible. If there is not enough sand to fill the voids between coarse aggregate particles, the space must be filled with cement paste. These mixtures also tend to be harsh and difficult to finish. On the other hand, aggregate combinations produce uneconomical concrete because of the larger surface area of finer particles. When the surface area increases, concrete will be too stiff and hard to place and compact. If we make the paste more fluid by adding water, the concrete strength and durability will suffer.

The maximum size of coarse aggregate used in concrete also has an effect upon surface area and economy. Usually, as the maximum size of well-graded coarse aggregate increases, the amount of paste required to produce concrete of a given slump or consistency decreases. Figure A.1 shows the effect of increasing maximum size upon the void content of a well graded aggregate.

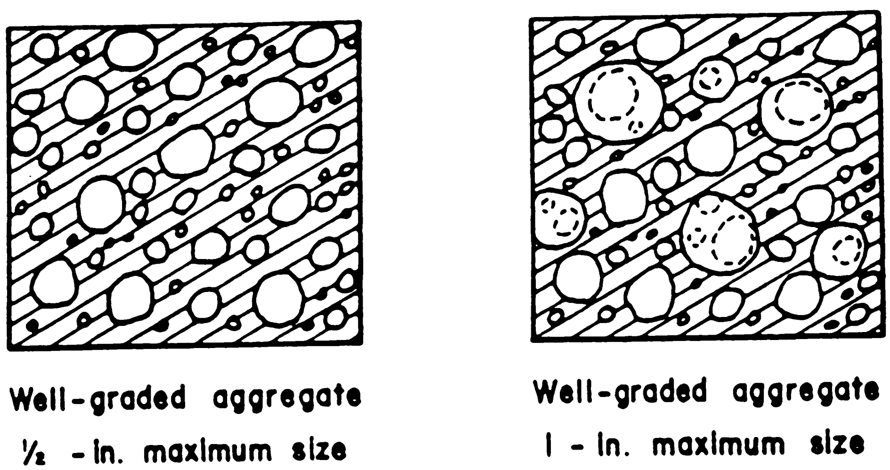


FIGURE A.1 Effect of Increasing Maximum Size On the Void Content of a Well-Graded Aggregate (ACI, 1978)

One of the most important characteristics of the fine aggregate grading is the amount of material passing numbers 50 and 100 sieves. Inadequate amounts of materials in these size ranges can cause excessive bleeding, difficulties in pumping concrete, and difficulties in obtaining smooth trawled surfaces.

In a study by Czuryszkiewicz (1973), it has been reported that the size of aggregate has less influence on the strength of concrete having a higher water-cement ratio; however, for lower water-cement ratios a difference of approximately 15 Mpa (2200 psi) was observed between a concrete having a maximum aggregate size of 76 mm (3 in.) and one having a maximum aggregate size of 9.5 mm (3/16 in.); the one with larger aggregates gave less strength. They concluded that the smaller size aggregates produce higher strength concrete at a given water-cement ratio, as discussed previously; however, the water and cement requirements of the small maximum size aggregate mixes are much greater than for the large aggregate mixes. A similar trend for the effect of aggregate size on flexural strength (Walker, 1960), although the difference in strengths are not so great. Walker also reported that shrinkage and loss of water during drying increased with size. In the range of aggregates sizes from 19 to 63.5 mm (3/4 in. to 2.5 in.), differences in shrinkage amounted to only about 0.5 percentage points, which would seem to be of little or no significance.

It has been reported that small size aggregates would increase the volume changes. The drying shrinkage is closely related to the quantity of water and hence reflect the greater amounts required by the small-size materials (Czuryskiewicz, 1973).

A.3 EFFECT OF PARTICLE SHAPE AND SURFACE TEXTURE

The shape and surface texture of the individual aggregate particles has an important influence on the workability of freshly mixed concrete and the strength of the hardened material. Fine aggregate particle shape and texture affect concrete in one major way (ACI, 1978). Angular rough sands will require more mixing water in concrete than rounded smooth fine aggregates to obtain the same level of slump or workability (thus decreasing both the strength and eventually durability). This, in turn, will affect the water-cement ratio if the cement content is to be held constant; or it will require an adjustment in the cement content if a certain water-cement ratio is needed. The influence of fine aggregate shape and texture on the strength of hardened concrete is almost entirely related to its influence on the resulting water-cement ratio of concrete if the fine aggregate has a grading within the normally accepted limits.

Coarse aggregate shape and texture also affects mixing water requirements and water-cement ratio in a similar manner to fine aggregates. But coarse aggregate particles, due to their much smaller ratio of surface area to volume, affect strength through a more complex relationship involving the bond characteristics to cement paste and concrete water-cement ratio (ACI, 1978). It has been demonstrated by different researchers that the microcracks between the paste or mortar and the surfaces of the largest coarse aggregate particles play critical roles in the response of concrete to bond. Angular rough-textured aggregates, for example, have an increased surface area for bond to the cement paste when compared to similar size round particles, and this provides favorable conditions in concrete.

Considering all of the factors relating to aggregates which have an effect on concrete strength, the following appear to be most important:

i) The water-cement ratio of the concrete as affected by fine and coarse aggregate shape and texture.

ii) The surface area available for bond to the cement paste; here the shape and texture of the largest particle is most important.

iii)The surface texture of the largest pieces affect interfacial bond strength. The mineralogy and crystal structure of these pieces will also affect bond strength.

iv)As the size of the larger particles is increased then there is more likelihood of a paste aggregate bond failure since stresses at the interface will be higher than those for smaller particles.

Factors which give higher intrinsic bond strength are relatively unimportant in fine aggregates because of the large total surface area available for bond and the lower stresses around small particles. Likewise, the larger surfaces of angular sands compared to rounded sands are of no particular benefit to bond strength. This leads to the conclusion that fine aggregate shape and texture affect the amount of mixing water required for a given slump level and that the effects of different fine aggregates on concrete strength can be predicted from a knowledge of their effects on mixing water and water-cement ratio.

For a coarse aggregate the situation is quite different, and the final effects on strength are more difficult to predict due to the importance of bond strength characteristics in the larger particles. This is the fundamental reason why different maximum sizes of coarse aggregate, different gradings, and different sources of coarse aggregate will produce different water-cement ratio vs. strength curves.

A.4 EFFECT OF STRENGTH AND MODULUS OF ELASTICITY OF AGGREGATES

Clearly, the compressive strength of concrete cannot significantly exceed that of the major part of the aggregate contained, although it is not easy to state what is the strength of the individual particles. It has been reported that inadequate strength of aggregate represents a limiting case as the properties of aggregate have some influence on the strength of concrete even when the aggregate by itself is strong enough not to fracture prematurely (Neville, 1986). It has also been reported by different researchers that if we compare concrete made with different aggregates we can observe that the influence of aggregate on the strength of concrete is qualitatively the same whatever the mix proportions, and is the same regardless of whether the concrete is tested in compression or in tension. It is possible that the influence of aggregate on the strength of concrete is due not only to the mechanical strength of the aggregate but also, to a considerable degree, to its absorption and bond characteristics.

In general, the strength and elasticity of aggregate depend on its composition, texture and structure, as discussed previously. Thus a low strength may be due to the weakness of constituent grains or the grains may be strong but not well knit or commented together.

The modulus of elasticity of aggregate is rarely determined; this property however, can not be neglected as the modulus of elasticity of concrete is generally increased with increasing modulus of elasticity of the constituent aggregate; the concrete modulus depends on other factors as well. The modulus of elasticity of aggregate also affects the magnitude of creep and shrinkage that can be realized by the concrete. It should be noted that the required strength of aggregate is considerably higher than the normal range of concrete strengths because the actual stresses at the points of contact of individual particles within the concrete may be far in excess of the nominal compressive stresses applied. On the other hand, aggregate with moderate or low strength and modulus of elasticity can be valuable in preserving the durability of concrete. It has also been reported that the volume changes of concrete, arising from thermal reasons, lead to a lower stress in the cement paste when the aggregate is compressible. Thus compressibility of aggregate would reduce distress in concrete while a strong and rigid aggregate might lead to cracking of the surrounding paste (Neville, 1986).

A.5 EFFECT OF POROSITY AND ABSORPTION OF AGGREGATES

The porosity of aggregate, its permeability, and absorption, influence such properties of aggregate as the bond between it and the cement paste, the resistance of concrete to freezing and thawing, as well as its chemical stability and resistance to abrasion. The specific gravity of aggregate also depends on its porosity and, as a consequence, the yield of concrete for a given weight of aggregate is affected. The pores in aggregate vary in size over a wide range, the largest being large enough to be seen under a microscope or even with the naked eye, but even the smallest aggregate pores are generally larger than the gel pores in the cement paste. It was reported that pores smaller than 4μm are of special interest as they are generally believed to affect the durability of aggregate under alternating freezing and thawing (Mehta, 1993).

Some of the aggregate pores are wholly within the solid. Other pores are open to the surface of the particle. The cement paste, because of its viscosity, cannot penetrate to a great depth into any but the largest of the aggregate pores, so that it is the gross volume of the particle that is considered solid for the purpose of calculating the aggregate content in concrete.

Although there is no clear-cut relation between the strength of concrete and the water absorption of aggregate used, the pores at the surface of the particle affect the bond between the aggregate and the cement paste and thus exert some influence on the strength of concrete (Neville, 1986).

It has also been reported that the porosity of the aggregates not only affect the workability of concrete but also the mixing procedures, pumbability and finishing. Recycled plastics do not absorb water, thus problems associated with workability are reduced.

Normally, it is assumed that at the time of setting of concrete the aggregate is in a saturated and surface-dry condition. If the aggregate is batched in a dry condition, it is assumed that sufficient water will be absorbed from the mix to bring the aggregate to a saturated condition, and this absorbed water is not included in the net or effective mixing water. It was reported that when dry aggregate is used, the particles become quickly coated with cement paste, which prevents further inpress of water necessary for saturation. This is particularly so with coarse aggregate, where water has further to travel from the surface of the particle. As a result, the effective water-cement ratio is higher than would be the case if full absorption of water by the aggregate had been possible. This effect is significant mainly in rich mixes where rapid coating of aggregate can take place, while in lean, wet mixes, the saturation of aggregate proceeds undisturbed.

The absorption of water by aggregates also results in some loss of workability with time, but it has been reported (ACI, 1989) that beyond 15 minutes the loss becomes small.

A.6 EFFECT OF BOND OF AGGREGATE

Bond between aggregate and cement paste is an important factor in the strength of concrete, especially the flexural strength. The full role of bond in cement is only being realized now. Bond is due, in part, to the interlocking of the aggregate and the paste owing to the roughness of the surface of the former. A rougher surface results in a better bond. A better bond is also usually obtained with softer, porous and mineralogically heterogeneous particles (Nichols et al, 1970).

Generally, texture characteristics which permit no penetration of the surface of the particles are not conductive to good bond. In addition, bond is affected by other physical and chemical properties of aggregate, related to its mineralogical and chemical compositions and to the electrostatic condition of the particle surface. Due to the hydrophilic nature of recycled plastics, the bond between the recycled plastics and the cementitious matrix is weak resulting in strength and durability problems.

A.7 EFFECT OF THERMAL PROPERTIES OF AGGREGATE

There are three thermal properties of aggregate that may be significant in the performance of concrete: coefficient to thermal expansion, specific heat, and conductivity. The last two are of importance in mass concrete or where insulation is required, but not in ordinary structured work.

The coefficient of thermal expansion of aggregate influences the value of this coefficient for concrete containing the given aggregate: the higher the coefficient of the aggregate, the higher the coefficient of the concrete, but the latter depends also on the aggregate content in the mix and on the mix proportions in general.

It was reported that there is another aspect of the problem. It has been suggested that if the coefficient of thermal expansion of the coarse aggregate and of the cement differ too much, a large change in temperature may introduce differential movement and a break in the bond between the aggregate particles and the surrounding paste. However, possibly because the differential movement is affected also by other forces, such as those due to shrinkage, a large difference between the coefficients is not necessarily detrimental when the temperature does not vary outside the range of 4 to 60 ^{o}C (39 to 140^{o}F), for example. Nevertheless, when the two coefficients differ by more than 5.5×10^{-6} / ^{o}C (3×10^{-6}/ ^{o}F) the durability of concrete subjected to freezing and thawing may be affected. Recycled plastics, due to their higher thermal coefficient will cause problems with durability, specially freezing and thawing and scaling.

A.8 EFFECT OF AGGREGATE PROPERTIES ON

A.8.1. MIXTURE PROPORTIONS

The grading and particle shape of aggregates influence the proportions needed to obtain workable freshly mixed concrete and at the same time provide needed hardened concrete properties with reasonable economy. The amount of water needed to obtain a desired slump or workability depends on the maximum size of the coarse aggregate, particle shape and texture of both the fine and coarse aggregates, and particle size range of coarse aggregate.

Increased angularity and roughness of the coarse aggregate can also increase the mixing water requirement (and needed mortar content) of concrete for a given level of workability; however, its effect is generally as great as the shape and texture properties of fine aggregate. Large amounts of flat and elongated pieces of aggregate in concrete can make it too harsh for some placement methods resulting in voids, honeycombing, or pump blockage.

A.8.2. FRESH MIX PROPERTIES

Slump and Workability

The strength, appearance, permeability, and general serviceability of concrete is dependent on the effective placement and consolidation of freshly mixed concrete without undesirable voids and honeycombing. It must be workable enough for a given formwork, reinforcement spacing, placement procedure, and consolidation technique to completely fill spaces around the reinforcement and flow into corners. Aggregate grading, and in particular the maximum aggregate size, as mentioned earlier, controls the amount of paste in a properly designed mix and, therefore, has an important influence.

The effect of aggregate on the cohesive properties of a concrete mixture depends on factors such as the maximum size of the coarse aggregate, if larger than 10 mm (3/8 in), the overall combined grading fine and coarse (and percentage of sand on the basis of total aggregate), and the amount of clay size fines present.

Another source of slump loss, as mentioned earlier, is the absorption of mixing water by the aggregates.

A.8.3. SHRINKAGE

The shrinkage of concrete is less than that of neat cement owing to the restraining influence of the aggregate and may be one-fifth to one-tenth, or even less, of that of neat cement. The aggregate is surrounded by cement paste which, in shrinking, places the aggregate under compression and itself becomes subjected to tensile forces. These tensile forces may be greater than the strength of the paste, in which case cracking will occur and shrinkage measurement will be unreliable. From this reasoning it would be expected that aggregate with high modulus of elasticity would give a concrete with less shrinkage than aggregate with a low modulus of elasticity. Table A.1 shows the relationship between drying shrinkage and elastic modulus.

TABLE A.1 Relationship Between Drying Shrinkage and Elastic Modulus (Orchard, 1973)

Aggregate Type	Elastic Modulus, $x10^6$ psi	Absorption, % by Volume	Drying Shrinkage at 1 yr $x\ 10^{-6}$
Basalt	13.63	3.3	300
Rounded Quartz	12.27	4.7	180
Crushed Quartz	3.37	6.6	330
Marble	6.61	8.0	250
Granite	6.18	5.5	290
River Gravel Mixed	5.60	3.2	280
Calcareous Sandstone	2.80	9.7	1020
Ferruginous Sandstone	1.37	13.6	630

It has been reported by that the differences in shrinkage cannot be explained entirely by differences in the mineralogical composition of the aggregate and the anomalies may be due to cracking of the mortar paste; thus the shrinkage of the concrete using the calcareous sandstone is much greater than that of the concrete using the ferrugunous sandstone in spite of the fact that the ferruginous sandstone has a lower modulus of elasticity (Orchard, 1973) .

The shrinkage of concrete is largely governed by the compressibility of the aggregate and its own shrinkage properties on drying. Carlson (1938) made some concrete using rubber as an aggregate. This concrete contracted almost as much as the corresponding neat cement paste and about 8 times as much as ordinary concrete.

It has been reported by several researchers that aggregate shape has little effect on shrinkage, except in so far as it affects the amount of mixing water required to maintain workability.

It has been shown by Carlson (1938) that below a No. 4 sieve the aggregate size had little effect on shrinkage; the abrupt reduction in shrinkage with aggregate above No. 4 sieve size indicates, however, that cracking of the cement paste probably occurred in these regions and the cement and aggregate were both of a type likely to cause cracking . The aggregate grading, and in particular the maximum aggregate size, as mentioned earlier, controls the amount of paste in a properly designed mix and, therefore, as the paste has an important influence it would be expected that the minimum drying shrinkage would be obtained with the concrete using an aggregate grading promoting the minimum amount of paste. In general, the shrinkage reduced considerably as the aggregate size is increased. It is also known that the reduction in shrinkage is obtained by using a large maximum size of aggregate on account of the reduction in the water-cement ratio which can be effected and due to the increased likelihood of cracking of the cement paste.

A.8.4 CREEP

It is reasonable to suppose that if creep occurs in the mortar paste the progressive effect will be to transfer more and more applied load on to the aggregate and further deformation will then be governed by creep of the aggregate particles.

Orchard (1973) reported that the effect of modulus of elasticity of the aggregate does not appear to follow a regular law and variability in the test results may be due to the effect of breakdown of the bond between the mortar paste and the aggregate with consequent internal cracking. There is little difference in the creep of concrete made with sharp or with rounded aggregate, but concrete made with porous aggregate generally has a grater creep.

It has been reported that there appears to be considerable conflict on the effect of the water absorption of the aggregate and its modulus of elasticity on the amount of creep.

A.8.5. DURABILITY

a) Resistance to Freezing and Thawing:

Concrete containing frost-resistance paste may not be totally resistant to freezing and thawing if it contains aggregate particles which became critically saturated. An aggregate particle is considered to be critically saturated when there is insufficient unfilled pore space to accommodate the expansion of water which accompanies freezing. It has been reported (ACI, 1989) that there is a critical particle size above which the particle will fail under repeated freezing-thawing cycles of critically saturated aggregate. This size is dependent on pore structure, permeability, and tensile strength of the particle.

It was also reported that experience has yet to show that fine aggregates are directly associated with freezing-thawing deterioration of concrete. Some porous coarse aggregates can, on the other hand, cause deterioration of concrete due to freezing (Spitzner, 1989).

Various properties related to the pore structure within the aggregate particles, such as absorption, porosity, pore size, and pore distribution, or permeability, may be indicators of potential durability problems when used in concrete which will become saturated and freeze in service. Generally, it is absorption values, caused principally by medium-sized pore spaces in the range of 0.1 to 0.5μm, that are most easily saturated and contribute to deterioration of concrete.

b) Permeability

Theoretically, the introduction of aggregate particles of low permeability into a cement paste is expected to reduce the permeability of the system because the aggregate particles should intercept the channels of flow within the cement paste matrix. Test data, however, show that this is not the case. Figure A.2 shows that the addition of aggregate to a cement paste or a mortar increases the permeability considerably; in fact the larger aggregate size, the greater the coefficient of permeability.

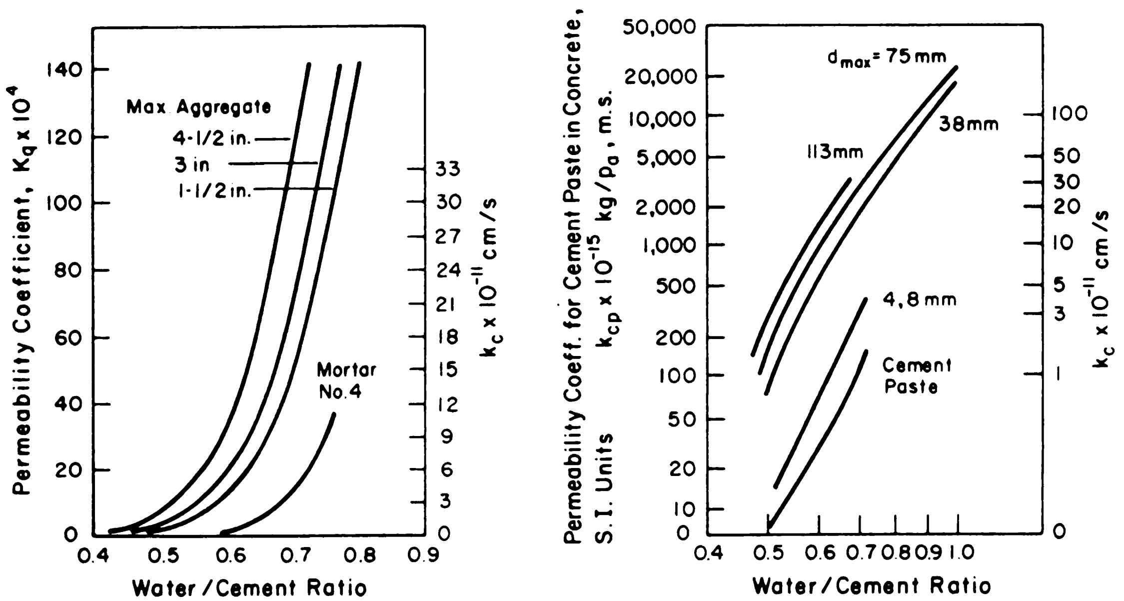


FIGURE A.2 Influence of Maximum Aggregate Size and Water-Cement Ratio on Concrete Permeability (Mehta, 1993)

The explansion as to why the permeability of mortar or concrete is higher than the permeability of the corresponding cement paste lies in microcracks that are present in the transition zone between the aggregate and the cement paste. It is known that the aggregate size and grading affect the bleeding characteristics of a concrete mixture which, in turn, influences the strength of the transition zone.

Since strength and permeability are related to each other through the capillary pores, thus the factors influencing the strength will influence the permeability. As far as aggregates are concerned, the aggregate size and grading, thermal and drying shrinkage strains, and aggregate permeability will influence the permeability characteristics of concrete.

c) **Fire Resistance**

The fire resistance of concrete depends largely on the type of aggregate; aggregates which have been subjected to heat during their formation or manufacture being the best and siliceous aggregates the poorest.

For this purpose, the Building Regulations deviled the aggregates into two classes:

Class I: Foamed slag, pumice, blast-furnace slag, pelleted fly ash, crushed brick and burnt clay products (excluding expanded clay), well burnt clinker and crushed lime stone;

Class II: Flint-gravel, granite, and all crushed natural stones other than limestone.

Expanded clay aggregates and other lightweight aggregates prepared by sintering would presumably come with Class I.

The failure of concrete under the action of fire is due to differential expansion between the hot surface layers and the cooler concrete behind, and to the opposing actions of the cement which shrinks owing to the loss of moisture to a greater extend than it expands due to the rise in temperature and of the aggregate which expands continuously with the rise in temperature. These phenomena lead to cracking and sapling and in the case of reinforced concrete to the exposure of the reinforcement to the fire. Once the reinforcement is exposed, it conducts the heat rapidly and accelerates the effects of unequal expansion. Flints, siliceous gravels, and granites are perhaps the worst aggregates and concrete having these aggregates does not offer high resistance to fire.

In reinforced concrete an important factor is the thermal transmittance of the concrete and in this respect the lightweight aggregate concretes made with foamed slag, pumice, expanded clays, and sintered materials offer the best protection.

c) Resistance to Wetting and Drying

The influence of aggregate or durability of concrete subjected to wetting and drying is also controlled by the pore structure of the aggregate. Differential swelling accompanying moisture gain of an aggregate particle with a fine-textured pore system may be sufficient to cause failure of the surrounding paste and results in the development of a popout. The amount of stress developed is proportional to the modulus of elasticity of the aggregate. In some cases this may be a contributing factor to popouts.

d) Resistance to Heating and Cooling:

Heating and cooling induce stresses in any non homogeneous material. If the temperature range is great, damage may result. For aggregates commonly used, and for temperature changes ordinarily encountered, this is not usually a critical factor in concrete. However, it has been reported that there has been speculation that large differences in the coefficient of expansion or thermal diffusing between the paste and the aggregate can result in damaging stresses in concrete subject to normal temperature change.

e) Abrasion Resistance:

Abrasion resistance and localized impact resistance of concrete is a property which is highly dependent on the quality of both the cement paste and the aggregate at and near the surface receiving localized impact and abrasive stresses. In those cases where the depth of wear is not great, there will be little exposure of coarse aggregate, and only the presence of hard and strong fine aggregate in good quality cement paste may be necessary to provide needed surface toughness.

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