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I am submitting herewith a dissertation written by Rohi Mohammed Salem entitled "Strength and Durability Characteristics of Recycled Aggregate Concrete." I have examined the final copy of this dissertation for form and content and recommend that it be accepted in partial fulfillment of the requirements for the degree of Doctor of Philosophy, with a major in Civil Engineering.

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STRENGTH AND DURABILITY CHARACTERISTICS OF RECYCLED AGGREGATE CONCRETE

A Dissertation

Presented for the

Doctor of Philosophy

Degree

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ABSTRACT

Recycling demolished concrete as an alternative source for the production of new concrete can help solve the growing waste disposal crisis and the problem of depleted natural aggregates. However, to make such recycling feasible, the strength and durability of the recycled aggregate concrete (RAC) must also be assured. This study was an attempt to provide that assurance by conducting an extensive investigation regarding the strength and frost resistance of RAC. The following three phases were carried out:

(1) Three replacement ratios (0.50, 0.75, 1.00) of the recycled aggregate were considered to determine their effects on the physical properties of the RAC and to obtain an acceptable quality of concrete with maximum utilization of recycled aggregate in place of natural aggregate.

(2) Four cases for both RAC and natural aggregate concrete (NAC) were considered to determine the main factors that affect the freeze-thaw durability of both concretes:

(**a**) Using a medium water -cement ratio of 0.47;

(**b**) Increasing the fly-ash content from 14 percent to 28 percent;

(**c**) Lowering the water-cement ratio from 0.47 to 0.29 ;

(**d**) Using air entrainment (5 percent air content).

(**3**) Three different types of recycled aggregates were derived from three highly used structural grade concretes (3,000, 4,000, and 5,000 psi) to determine the effects of various original concrete strengths on the new strength of the RAC.

Regression analyses were performed to establish the relationships between the various properties of the RAC and the NAC obtained in this study, and the best fit equations were compared to those usually used by the ACI Code.

From the strength point of view, the recycled aggregate compared well with the natural aggregate in most cases and, therefore, could be considered for various potential applications. From the frost resistance point of view, the recycled aggregate had negative effects on the durability performance of the RAC. However, with the use of mineral and chemical admixtures, particularly air entrainment and fly-ash , the RAC could be as durable as the NAC. Some of the relations commonly used for the prediction of the concrete properties of the NAC cannot be used for the RAC. Therefore, some alternative relations were proposed for the prediction of the physical properties of the RAC.

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CHAPTER 1

INTRODUCTION

1.1 BACKGROUND

With the increases in human development and human knowledge, especially in the industrial age, the concept of recycling old material and reusing it in another form has become both a necessity and a challenge. That challenge ranges from avoiding or minimizing the solid waste problem on one hand to achieving an acceptable and sound use of old material without extra expenses on the other. Concrete recycling is no exception.

Recycled aggregate concrete is referred to as concrete produced using old crushed concrete in place of natural aggregates (mainly coarse aggregates). Recycled aggregates are referred to as aggregates produced by the crushing of the old concrete. Such aggregates can be coarse or fine aggregates.

As with other types of recycling, there are economic and environmental incentives for recycling old concrete. From an economic point of view, quality aggregates are in short supply in many places where concrete technology is needed, making it necessary to import quality aggregates from distant locations. Importing such aggregates can be quite expensive because of transportation costs, which can significantly increase the overall budget of a project.

From the environmental point of view, concrete is the most widely used construction material. The world consumption of concrete is approximately 4.5 billion tons a year (Mehta, 1986). It has also been estimated by the Environmental Resources Ltd. (1980) that every year, approximately 50 million tons of concrete are discarded in the European Economic Community, 60 million tons are discarded in the U.S.A., and 10 to 20 million tons are discarded in Japan. It has been further estimated that, by the year 2000, these figures will increase approximately threefold. If these huge amounts of rubble could be recycled, the amount of land needed for disposal sites would be reduced and the land used for aggregate extraction would not be depleted as quickly. Already, the shortage of dumping sites resulting from environmental regulations and urban development is forcing the construction industry to seek alternatives to minimize transportation and disposal costs. Thus, large-scale recycling of demolished concrete will not only help to conserve the depleted natural resources of gravel and sand but will also be a strong factor in helping to solve the growing waste disposal problem. As a result of these economic and environmental benefits, the concept of recycling demolished concrete and reusing it in another form has gained momentum in the last four decades.

One major area where recycled concrete has been successfully used is in the reconstruction of concrete pavements. The first successful utilization of an old pavement as aggregates for the new pavement was in the state of Iowa. The state conducted its initial recycling project in 1976 on U.S. 75 in Lyon County. The 42-year- old concrete pavement was crushed into aggregates, which were used in a new one-mile long and 10-inch thick highway pavement. The project was considered a success, which prompted the Iowa Department of Transportation to continue with other larger-scale projects. Among those was a 1978 project in which a 17-mile long and 27- inch thick highway concrete pavement was crushed and used as aggregates for the new pavement. On this project, 28,124 tons of coarse aggregates and 12,661 tons of fine crushed aggregates were utilized in the reconstruction of the new pavement. This eliminated a total of 40,875 tons (20,602 cubic yards) of concrete rubble to be disposed of and saved almost the same amount of virgin aggregate to be used. In addition to that, 200 tons of steel were saved for recycling. By recycling instead of using natural conventional aggregates, a total saving of more than $115,000 has been estimated. Those successful projects encouraged other states to initiate similar projects on their highway systems. In 1980, the Minnesota Department of Transportation recycled a 16-mile plain concrete pavement on U.S. 59 from Worthington to Fulda in southern Minnesota. It is estimated that the saving on this recycling project was approximately $600,000. Many state highway agencies have tried recycling concrete pavements after concluding that there would be considerable savings in terms of money and the environment. It is estimated that over 1,000 lane miles of concrete pavements have been recycled into new pavements (Yrjanson, 1981). Such recycling projects are expected to increase since there are 80,000 miles of concrete pavements on the Federal Aid System, and out of these, 40 percent are rated to be in fair to poor condition. There will be a time when all 80,000 miles of the system need to be reconstructed, and recycling should be considered a vital alternative whenever economic and environmental benefits are likely.

Besides the use of recycled concrete in concrete pavements, recycled concrete has also been used in aggregate base, lean concrete base, and asphalt pavements (Burke et al. , 1992). Recycling concrete to produce structural grade concrete for purposes other than pavements can become technically feasible if its strength and durability characteristics are completely understood.

1.2 PROBLEM DESCRIPTION

1.2.1 Phase I

Since the strength of recycled aggregate concrete (RAC) has been found in several studies to be lower than that of conventional natural aggregate concrete (NAC), it is necessary to determine the influence of varying contents of the recycled aggregate on the physical properties of the RAC. In this phase, the replacement ratio, r, is defined as :

$$r = RA / (RA+NA) \qquad (1.1)$$

where RA is the weight of recycled coarse aggregate and NA is the weight of natural aggregate. Four replacement ratios (0, 0.50, 0.75, 1.00) were considered to determine the optimum recycled aggregate content. The replacement ratio, r = 0, which was the natural aggregate concrete, served as the base or the standard, and its mix target design strength was 5,000 psi at 28 days.

1.2.2 Phase II

Even though much is known about the mechanical proprties of recycled aggregate concrete, an important question remains about its freeze and thaw durability, and it appears that wide-spread utlizatation of such concrete will be limited until this question is addressed. Phase II of this research addressed that question by clarifying the circumstances in which recycled aggregate concrete may be regarded as frost-resistant and by determining the major factors that affect its durability.

Four cases of concrete mixes were considered for both the recycled and natural aggregete concretes to determine the main factors that affect their durability. Case I consisted of using a medium water-cement ratio of 0.47 (base mixes). Case II consisted of varying the base mixes by changing the water-cement ratio from 0.47 to 0.29 to determine whether lowering the water-cement ratio is alone enough to produce freeze-thaw resistant concretes in the absence of entrained air. Case III consisted of varying the fly-ash content from 14 percent to

28 percent by the weight of cement to test the hypothesis that adequate strength and freeze-thaw durability could be attained in both recycled and natural concretes when significant amounts of potland cement in the mix are replaced by fly-ash without a need of air entrainment. Case IV consisted of varying the base mixes by entraining them with an air content of 5 % to produce frost resistant concretes and to evaluate the negative effects on the compressive strength and other mechanical properties as a result of air entrainment.

Compression tests and modulus of rupture tests were also performed for each case. These tests, in essence, defined the mix; that is, the fundamental quantities of compressive and tensile strengths were known for each mix for which durability tests are performed.

1.2.3 Phase III

The effects of strength variations of the original concrete from which the recycled aggregates were derived on the new strength of the recycled aggregate concrete were studied in this phase. Understanding these effects is necessary to determine whether the stockpiling of different grades of concrete together is feasible for the practical production of a specific grade of recycled aggregate concrete. This information is particularly significant in a crushing site that receives demolished concretes of non-uniform quality from different sources simultaneously.

To determine the effects of the original concrete from which the recycled aggregate were derived on the recycled aggregate concrete, three highly used structural grade concretes (3000, 4000, and 5000 psi) were crushed and used as recycled coarse aggregate.

1.3 LITERATURE REVIEW

1.3.1 Compressive Strength

1.3.1.1 Recycled Aggregate (Coarse) Aggregate

Frondistou-Yannas (1977) reported that the compressive strength of concrete containing recycled aggregate was within the range of 4 to 14 percent lower than that of natural concrete. Ravindrarajah et al. (1987) found that the strength of recycled aggregate concrete was consistently about 10% lower than the strength of corresponding concrete made with conventional aggregates. Nishibayashi and Yamura (1988) found that the compressive strength of recycled aggregate concrete was 15% to 30% lower than that of conventional concrete of the same water-cement ratio.

Hansen and Narud (1983) reported that the compressive strength of recycled aggregate concrete heavily depends on the strength of the original concrete when other factors are held the same. They stated that "if the water-cement ratio of the original concrete is the same as or lower than that of the recycled concrete, then the new strengths will be as good as or better than the original strengths and vica versa". They were able to make recycled aggregate concrete with w/c = 0.40 having a compressive strength of 4930 psi after 38 days by employing recycled aggregate from an original concrete with w/c = 1.20 having a compressive strength of 2030 psi. However, when they tested recycled aggregate concrete at the same w/c = 0.40 but with recycled aggregate from original concrete with w/c = 0.40, they were able to get a compressive strength of 8850 psi. This considerable difference between the two compressive strengths (4930 psi and 8850 psi) at the same w/c of 0.40 is due in large to the compressive strength (the water-cement ratio) of the original concrete from which the recycled aggregates were derived. However, other investigations by Nixon (1978), Yrjanson (1981) and Ravindrarajah and Tam (1985) reported that a low compressive strength of the original concrete is not detrimental to the compressive strength of the recycled aggregate concrete. For instance, Ravindrarajah and Tam (1985) reported that compressive strengths of 4790 psi and 4640 psi were obtained using recycled aggregates from original concretes with compressive strengths of 3840 psi and 2900 psi respectively.

It can be seen from the above results that more research is needed to understand the strength effects of the original concrete on the behavior of the recycled aggregate concrete.

1.3.1.2 Recycled Aggregate (Coarse and Fine) Concrete

Buck (1973) studied concrete mixes that contained recycled aggregate as coarse aggregate and mixes that contained recycled concrete as both fine and coarse aggregate. He found that recycled concrete aggregates can best be used as a substitute for coarse aggregate only. Pearson (1988) reported that concrete mixes that used both recycled fine and coarse aggregates were less workable and lower in strength than those that used recycled coarse aggregates and natural sand. Therefore, most DOTs will not allow the use of the recycled fines in their recycling projects (Johnson and Han, 1992). On the other hand, Rasheeduzzafar and Khan (1984) found no significant influence on compressive strength between mixes that contained both recycled fine and coarse aggregates and mixes that only contained recycled coarse aggregates and natural sand.

1.3.2 Modulus of Elasticity

Frondistou-Yannas (1977) found up to 40% lower modulus of elasticity for recycled aggregate concrete made with coarse recycled aggregate and natural sand compared to corresponding conventionl concrete. That finding is to be expected since large amounts of old mortar which have comparatively low moduli of elasticity are attached to original aggregate particles. Rasheeduzzafar and Khan (1984) found up to 18% lower modulus of elasticity for recycled aggregate concrete made with coarse recycled aggregate and natural sand compared with the modulus of elasticity of a corresponding conventional concrete. Hansen and Boegh (1985) found the modulus of elasticity to be 14% to 28% lower for recycled aggregate concrete than for corresponding conventional aggregate. Ravindrarajah and Tam (1985) confirmed the above results by reporting a reduction of 30% in the modulus of elasticity when using recycled coarse aggregate instead of natural coarse aggregate in concrete.

From the above findings, it can be seen that the modulus of elasticity for recycled aggregate concrete is consistently lower than that of corresponding conventional concrete by as much as 40%.

1.3.3 Tensile and Flexural Strength

Ravindrarajah and Tam (1985) found that the flexure and tensile strengths of recycled aggregate concrete is within the range of ± 15% of those of conventional concrete. They suggested that the tensile strength of recycled aggregate concrete may be comparable to that for the natural-aggregate concrete owing to the presence of higher cement paste content in the former. This is in contrary to Malhotra (1976) who reported lower flexural strength for the recycled aggregate concrete than that of control concrete made with the same conventional aggregate.

1.3.4 Durability

Durability is one of the most important characteristics of concrete and is determined by its ability to endure its physical and environmental surroundings without compromising the functional properties and structural quality of the original design. In temperate regions, damage to concrete structures that are exposed to repeated freezing and thawing cycles is one of the major problems requiring heavy costs for repair and replacement. For example, in industrially developed countries it is estimated that more than 40 percent of the total resources of the building industry are spent on repair and maintenance of existing structures and less than 60 percent on new installations (Mehta, 1986) .

Buck (1977) found the freeze-thaw resistance of original and recycled aggregate concrete to be not significantly different. Mulheron and O'Mahony (1988) reported that concrete made with recycled aggregates when subjected to freeze-thaw action has better or similar durability compared to an equivalent concrete mix made with conventional aggregates. They explained this enhanced durability by the greater porosity of the recycled aggregate which provides a sufficient number of air-filled macro-pores. Those pores act as a relief for the building pressure resulting from ice formation during the freeze-thaw cycles.

Contrary to Mulheron and O'Mahony, Nishibayashi and Yamura (1988) reported that the freeze-thaw durability of recycled aggregate concrete is considerably inferior to that of normal concrete and even can not be improved by air entrianment. They attributed the lower durability to the presence of old mortar in the recycled aggregate concrete which is easily damaged by repeated freeze-thaw cycles. Comparing the results obtained by Mulheron and O'Mahony (1988) on one side and Nishibayashi and Yamura (1988) on the other side reveals a direct contradiction. That contradiction, along with the limited research on this important aspect of durability, requires more extensive research to clarify the circumstances in which recycled aggregate concrete may be regarded as frost-resistant and to determine the major factors that affect its durability.

CHAPTER 2

FREEZE-THAW MECHANISM

2.1 BACKGROUND

Current designers of concrete structures in temperate regions are becoming more conscious of the serious damage caused by repeated cycles of freeze-thaw actions. That damage requires substantial repair or replacement of a structure before the structure's life expectancy is reached. Consequently, the issue of durability has gained increased interest in the last decade forcing engineers not only to design structures to meet a specified strength but also to withstand long-term effects of their surroundings.

According to ACI Committee 201 (1977), durability of Portland cement concrete is defined as "its ability to resist weathering actions, chemical attack, abrasion or any other process of deterioration". A durable concrete is expected to maintain its shape and quality and provide its intended service throughout the structure's life expectancy.

Maintenance and reconstruction of structures due to durability problems can be considerably reduced or eliminated by specifying the right combinations and proportions of materials which will insure both durability and strength. Whiting and Stark (1987) have shown that concrete made with sound aggregates, low water-cement ratio and a proper air content is likely to be frost resistant when it is allowed to reach maturity before exposure to aggressive surroundings.

2.2 FREEZE-THAW MECHANISM

2.2.1 General

The deterioration of concrete can be viewed as a function of the combined effects of its complex microstructure and the specified environmental conditions to which it is exposed. Thus, concrete which is durable under specific environmental conditions might not be durable under other conditions. For instance, a non-air concrete may be durable in Texas, and the same concrete may not be durable in Michigan where it is exposed to repeated freezing and thawing cycles.

Discussing the microstructure of the hydrated cement paste and the various roles played by different kinds of pores is necessary before discussing the freeze and thaw mechanisms of concrete. Such discussion below is based on the explanations given by Metha (1986), Brunauer (1962), and Cordon (1967).

2.2.2 Microstructure of Hydrated Cement Paste

Portland cement is made of five major crystalline compounds. They are : Tricalcium Silicate (C_3S), Dicalcium Silicate (C_2S), Tricalcium Aluminate (C_3A), Tetracalcium Aluminoferrite (C_4AF) and Gypsum (CSH_2) where C = CaO, S = SiO_2, A = Al_2O_3, F = Fe_2O_3, S = SO_3 and H = H_2O.

Those five compounds are originally produced by pulverizing clinker (a product of high-temperature reactions between calcium oxide, silica, alumina, and iron oxide) with a small amount of sulfate.

When Portland cement is mixed with water, the five compounds ionize and precipitate three main solid phases. Metha (1986) covers in detail the types, amounts, and characteristics of these three solid phases, that can be resolved by an electron microscope. The most important component is the hydrated form of calcium silicate hydrate , abbreviated C-S-H, which makes up 50 to 60 percent of the volume of solids in a completely hydrated Portland cement paste. Because of the resemblance of the C-S-H to the natural mineral tobermorite, it is sometimes called "tomermorite gel". This gel plays a key role in determining the strength and dimensional stability of the hardened paste and concrete.

The exact structure of C-S-H is not yet known, but there are several models proposed to explain the properties of the material. According to the models proposed by Powers (1958) and Brunauer (1962), the material has a layered structure with an enormous surface area on the order of 100 to 700 m^2/g depending upon the measurement technique used. The strength of the material is mainly derived from van der Waals forces between the surface areas at an average spacing estimated at 18 A°.[1] Other models include the Feldman- Sereda (1970) model which visualizes the C-S-H structure as being composed of an irregular or kinked array of layers that are randomly arranged to produce interspaces of different shapes and sizes (5A° to 25A°).

The second largest compound formed in the hydrated process is calcium hydroxide, $Ca(OH)_2$ which makes up 20 to 25 percent of the total volume of solids in the hydrated cement paste. Calcium hydroxide is a well defined crystalline substance with hexagonal-prism morphology. Because of its much larger sizes, and thus lower surface area, the van der Waals forces are limited so they contribute little to the strength of the hydrated paste.

The last hydrate is made up of calcium sulfoaluminates which occupy 15 to 20 percent of the solid volume. This compound plays a minor role in the structure-property relationship of the hydrated cement paste.

[1] 0.1 nanometer (nm) = 1 angstrom (A°) , nm = 10^{-9} m, µm = 10^{-6} m

2.2.3 Types of Voids / Pores

In addition to the solid phases described above, the hydrated cement paste also contains various voids throughout its structure, which have a significant effect on its properties especially resistance to freezing and thawing actions. The various kinds of voids and their significance are discussed in this subsection .

1. Gel Pores

These are the interlayer voids or spaces within the C-S-H structure. Powers (1958) estimated the width of pores to be 18A° and determined that they account for 28 percent of the gel volume, while Felman and Sereda (1970) suggested that the pores vary from 5 to 25A°. Since those pores or voids are very small, they don't have adverse effect on the strength and permeability of the hydrated cement paste. However, since water in these voids is held by hydrogen bonding, the water's removal under certain conditions may accelerate the drying and shrinkage mechanism (Mehta, 1986).

2. Capillary Pores

These are the spaces, or cavities, not filled with solid hydration products in the hydrated cement paste. They are formed by water in excess of that required for the hydration process. Porosity is a popular method used to calculate the total volume of capillary voids in the hydrated cement paste. The two most important characteristics of concrete porosity are the size and distribution of the pores, and those characteristics entirely depend on the water-cement ratio and the degree of hydration. The pores vary in size from 0.01 to 0.05μm for well-hydrated cement pastes with low water-cement ratios, while they may be as large as 3 to 5μm at early stages of hydration and high water-cement ratios (Mehta, 1986). Figure 2.1 shows the dimensional range of the solids and pores in a hydrated Portland cement paste. Capillary pores smaller than 50 nm are assumed to be detrimental to drying shrinkage and creep, whereas pores larger than 50 nm are considered harmful to strength and permeability.

3. Air Voids

In Portland cement concrete, there are two kinds of air voids that may exist, namely entrapped and entrained air voids. One difference between the two is that the entrapped air voids are unintentionally included in the cement paste during mix operation, while entrained air voids are intentionally introduced by the addition of air entraining admixture (AEA) to develop immunity against frost actions. Since both types of air voids are much larger than the capillary pores, the air voids are considered detrimental to the strength and impermeability of the cement paste.

2.2.4 Water in the Hydrated Cement Paste

Water can exist in the hardened cement paste in one of three forms. These forms are described in the following :

1. Capillary Water

Capillary water forms in large capillaries (usually larger than $50A^o$) between hydrated cement grains in the hardened paste. Its physical properties may be viewed as that of bulk water because it is free from the influence of attraction forces of the solid surfaces of the gel. From a behavior point of view, water in capillary pores larger than 50 nm diameter is considered as "free water" because its removal does not cause any volume change, although its removal affects the strength and permeability of the concrete paste. Water held by capillary tension in capillary pores smaller than 50 nm in diameter can cause shrinkage on removal.

2. Adsorbed Water

This is the water that is held under the influence of attractive forces to the walls of the tiny gel pores. It is estimated by Metha (1986) that up to 6 molecular layers of water can be physically held by hydrogen bonding. A significant portion of the adsorbed water can be removed or lost by drying the paste to 30 percent relative humidity. When this water is removed or lost, shrinkage of the hydrated cement paste occurs.

3. Hydrated Water

Powers (1945) referred to the hydration water as that which is chemically combined with the cement and as such considered to be an integral part of the structure of the hydrated cement paste. It is lost only under heating the sample to ignition (i.e. 1000° C).

2.2.5 Mechanism of Frost Damage

2.2.5.1 Hydraulic Pressure Theory

Powers (1949) was the first to describe the mechanism by which damage due to repeated cycles of freezing and thawing takes place. According to his hypothesis, in water soaked paste, the capillary and gel pores are saturated with water. When water begins first to freeze in the capillary pores, the resulting increase of volume (due to 9% expansion of water as it changes state) requires a space for accommodation. If space for that expansion is unavailable, the water and the surrounding paste will experience increasing pressure until excess water in the capillary is forced to the nearest void. Since the porous cement paste has an extremely low coefficient of permeability, hydraulic pressure is generated through the freezing process. He suggested that the magnitude of the hydraulic pressure depends on the rate of freezing, degree of saturation and coefficient of permeability.

Powers hypothesized that the resistance to the flow of water in concrete is proportional to the distance water has to travel for pressure relief (length of the flow path or escape boundaries) and to the concrete's permeability. He concluded that there must be a critical path length beyond which the pressure would overcome the strength of the material. He estimated the critical path length for most concrete to be on the order of 0.01 inches. Further research provided the present maximum critical spacing factor of 0.008 inches. In this regard, the benefits of entrained air were explained in terms of the shortening of flow-paths to release the building pressure. This hypothesis is now widely accepted by the engineering community.

2.2.5.2 Osmotic Pressure Theory

Later studies by Powers (1956) and Helmath (1961) concluded that the hydraulic pressure theory was not entirely consistent with the experimental findings. They noticed that the movement of water during the freezing process of the cement paste was toward, not away from, the capillaries (freezing sites) as had been previously found. The water in the capillaries is not usually pure, typically containing several soluble substances such as alkalis, chlorides, and calcium hydroxide. Solutions other than pure water freeze at lower temperatures. As freezing starts, the solution concentration in the unfrozen pores adjacent to the frozen ones (ice sites) increases, and this gives rise to osmotic pressure which causes water to flow toward the ice as the solution attempts to reach equilibrium within itself. The resulting flow of water toward the ice allows further growth of the body of ice (ice lensing). When the pores become full of ice and solution, any further ice-accretion will result in a dilation of that pore, which can cause the paste to fail. Experiments have verified the shrinkage of the paste due to movement of water out of the unfrozen capillaries.

2.2.5.3 Other Theory

The mechanisms just described do not appear to be the only causes of damage to the cement paste due to frost action. Beaudoin and McInnis (1974) reported that when benzene, which actually contracts upon freezing, was used as a pore liquid instead of water, expansion and damage of the cement paste have also been observed.

The water held in the larger capillaries of the cement paste freezes at the normal freezing point. However, the remaining gel water (both interlayer and adsorbed in the gel pores) freezes at continuously lower temperatures due to its increased surface tension and higher pore water pressure. It is estimated that the gel water does not freeze above -78° F, and it stays in a super cooled state while water in the capillaries freezes (Litvan, 1976).

According to Litvan (1976), as the freezing process starts, there is a thermodynamic inequlibrium between a low energy state due to the frozen water in the capillaries and a high energy state due to the supercooled water in the gel pores. The difference between the two states of energy results in the movement of water to the low energy sites (the capillary pores) where it can freeze. When there is no accommodation in the capillary pores, internal pressure increases and causes the system to expand.

As can be seen, the physical process of frost action by which concrete is damaged is not totally understood and is perhaps a combination of several mechanisms. The main cause might be the development of hydraulic or osmotic pressure, adsorption of water from gel pores, or probably all of these depending upon the conditions involved. However, the common driving agent behind each mechanism is the movement of water within the cement paste, which in turn increases pressure on the capillary wall resulting in expansion of the cement paste. If this is the case, then regardless of mechanism, concrete freeze-thaw durability depends on the water-cement ratio, the degree of saturation, the concrete permeability, and the air content.

2.3 FACTORS AFFECTING FROST RESISTANCE

2.3.1 Water-Cement Ratio

Through extensive testing, researchers have determined that for the hydration of each gram of cement, 0.24g of water is consumed and 0.18g of water is adsorbed to the wall of the gel voids. The amount of freezable water in the capillary pores can be determined if the water-cement ratio and the degree of hydration are known. Figure 2.2a shows the changes in the porosity as the hydration percentage increases from 0 to 100 percent. As the hydration process continues, the total solid volume (hydrated products and gel pores) keeps increasing , which causes a decrease in the volume of the capillary pores. Powers and Brownyard (1947) found that a water-cement ratio less than 0.35 and a well cured sample result in zero water in the capillaries after completed hydration as shown in Figure 2.2b. Theoretically, if there is no freezable water in the cement paste, the concrete will be immune against frost damage, and this cannot happen until all water resides either as gel water or is combined as hydration products. Other researchers like Verbeck and Klieger (1976) confirmed the foregoing discussion by affirming that higher water- cement ratios result in more and larger capillary pores where more freezable water can be available as shown in Fig. 2.3.

2.3.2 Degree of Saturation

The critical degree of saturation may be defined as that level above which concrete is likely to be damaged when exposed to freezing temperatures. Concrete, after adequate curing, may well fall below the critical degree of saturation. However, when exposed to moist conditions, the concrete may again reach or exceed the original critical degree of saturation if its permeability characteristics are inadequate.

Whitside and Sweet (1950) calculated the percent of saturation after submerging the specimens in water. They found that samples over 91% saturated were always vulnerable to frost actions. The importance of these saturation effects is well reflected in the freeze-thaw testing methods. Under ASTM C666 "Resistance of Concrete to Rapid Freezing and Thawing", two procedures for freeze-thaw testing of concrete are allowed: Procedure A, rapid freezing and thawing in water, and Procedure B, rapid freezing in air and thawing in water. Researchers have found that identical samples tested under both procedures always performed significantly poorer using Procedure A because the sample is subjected to a moist environment.

2.3.3 Permeability

The permeability of concrete is a physical property related to the rate of the flow of water into the porous cement paste or the ability of water to penetrate the hardened paste. The flow of water through concrete obeys Darcy's law shown below :

$Q = K_p \, dp/ds$ where,

Q : flow rate

K_p : permeability coefficient of the cement paste

dp/ds : pressure gradient within the paste

The permeability coefficient depends strongly on the capillary porosity in which the latter is a function of the water-cement ratio and the age of hydration. Mindess and Young (1981) have shown that as the hydration process continues, the capillary pores are reduced in size resulting in a decrease in their interconnections, thus producing less permeable concrete as shown in Table 2.1.[1]

Table 2.2 taken also from Mindess (1981) shows the effects of the water-cement ratio on the curing time required to produce a discontinuous system of capillaries. As can be seen, the lower the water-cement ratio, the less curing time

[1] All tables and figures may be found in the appendixes

required to produce discontinuity in the system. This discontinuity is due to the decreased volume of large pores which produces a less permeable paste. From the general durability point view, low permeability is a very positive feature. When exposed to outside water sources, concrete with low permeability will not be saturated and, as a result, will not be vulnerable to freeze-thaw damage. However, if the concrete is not well cured and capillary voids remain filled with water, the decrease in the capillary size due to low permeability will result in higher hydraulic pressure being applied to the paste during the freezing process.

2.3.4 Air Content

As mentioned earlier, entrapped air and entrained air both constitute the total amount of air present in concrete. The presence of entrained air is desired in concrete because it acts as a reservoir to receive unfrozen water as it flows either to or from the freezing sites through the capillary pore system. As a result, the hydraulic pressure within a certain radius of the void is reduced below the tensile strength of the cement paste.

Adding a small amount of certain air entraining agents to the concrete mixture, can produce air bubbles with diameters in the range of 0.003 to 0.05-in. and a maximum spacing factor of 0.008-in. Washa (1993) defined the maximum spacing factor to be "an index related to the maximum distance of any point in the cement paste from the periphery of an air void" and it is typically considered to be in the range of 0.008in. However, the air entrainment criterion that is used to produce quality durable concrete is the percentage of air content in the concrete mixture, and it varies as the maximum coarse-aggregate size is changed. For an equivalent degree of durability, concrete with large aggregates needs less air entrainment than concrete with small aggregates. Table 2.3 of the ACI Committee 212 report (1971) shows air contents that are needed for different aggregate sizes.

Powers (1958) was among the first to offer an explanation of the air entrainment for the frost resistance of concrete. He observed that a saturated cement paste without air entrainment dilated by 1600 millionths of an inch during freezing (-24C), and on thawing to its original temperature, it retained a residual permanent elongation of 500 millionths (figure 2.4a). With 2 percent entrained air, the specimen showed 800 millionths of maximum dilation and a residual elongation of 50 millionths under similar conditions (figure 2.4b) . With 10 percent air content, the specimen showed no dilation on freezing and no residual elongation on thawing but showed a slight contraction (figure 2.4c). This contraction may be due to the loss of adsorbed water from C-S-H gel.

Although air entrainment was considered a revolutionary breakthrough in the concrete construction industry because it eliminated the freeze-thaw damage, many researchers (Gutman 1988, Klieger 1956, and Whiting 1983) reported strength losses in the range of 2 to 10 percent per one percent increase in entrained air. In addition, researchers concluded that for most applications, 4 to 6 percent of air content is required to provide a durable resistant concrete. This amount of air can add up to a 30 percent loss of compressive strength in order to obtain frost resistant concrete. This reduction in strength is particularly noticeable in high strength concrete (12,000 psi-15,000 psi) in which 20 percent loss due to air entertainment can represent 2,000 to 4,000 psi. Thus, even though proper air entrainment can be utilized to obtain a frost-resistant concrete, it can also adversely affect the original compressive strength

2.3.5 Aggregates

From the traditional perspective, aggregates are usually looked at as inner filler which is responsible for the dimensional stability of concrete, and their strength role is usually overlooked due to the fact that their strength is several times more than the cement paste and the transition zone. Nevertheless, aggregates often play a significant role in concrete durability, and different physical characteristics produce varying strength and durability.

The damage of the freeze-thaw mechanism in the cement paste is also generally applicable to aggregates. The resistance of aggregates depends on the development of high internal pressure when the water inside the aggregates freezes and causes volumetric change. This pressure is a function of the porosity of the aggregates (size, number, and distribution of pores) and the permeability of the pore's system.

According to Verbeck and Landgren (1960), aggregates may be classified into three categories. The first category contains aggregates of low permeability and high strength, in which the internal pressure caused by the freezing of water is accommodated without producing cracks. The second category consists of aggregates of intermediate permeability (i.e., with pore's diameter of less than 500 nm). Once saturated and upon freezing, the hydraulic pressure depends on the freezing rate and the distance the water has to travel to reach an escape boundary to relieve the building pressure. For most aggregates, the distance to this boundary is much higher than the critical distance in the case of the cement paste because of the aggregate's higher permeability. It has been also shown by Bloem (1963) that smaller aggregate sizes are more resistant to frost damage than larger sizes of the same type of aggregate. This finding gave rise to the concept of the critical size of aggregate above which the aggregate is likely to be fractured. This critical size may be considered as a measure of the maximum distance water must flow to reach the outside boundary to relieve the internal pressure. This size depends on the freezing rate, the degree of saturation, and the permeability of the aggregate, and, because of these variables, there is no single critical size for an aggregate type. Nevertheless, for most aggregates, the critical size is greater than the maximum size used for coarse aggregates.

The third category of aggregates are those with high permeability. Upon freezing and thawing actions, the high permeability causes water to flow easily in and out of the aggregates causing no damage to the aggregates themselves but initiating durability problems by allowing water to accumulate in the transition zone between the aggregate and the cement paste.

There are two types of fractures or damage of concrete usually associated with the use of unsound aggregates. The first one, known as popout, is caused by the rapid disruption of harder but saturated pieces of aggregates. This happens when aggregates larger than the critical size are present in concrete. This popout by itself does not affect the strength of the structure, but it allows water to enter into the structure through accesses provided by the popout and ultimately will result in deterioration. The second type, known as D-cracking, occurs in highway and airfield pavements. Here, cracks form parallel to joints and then connect, forming a crude capital D (Mehta 1986). This type of cracking is associated with coarse aggregates having a specific pore-size distribution described by a large volume confined to a narrower pore size range (0.1 to 1mm).

CHAPTER 3

MATERIAL SELECTION AND SPECIMEN FABRICATION

3.1 MATERIALS

3.1.1 Natural Concrete

The materials used in this research for the natural concrete were typical of those normally used in the production of commercial Portland cement concrete. These materials included Portland cement, fly ash, coarse and fine aggregates, and chemical admixtures. The properties of these materials are described below.

(1) Portland Cement

The cement used was Type I Portland cement manufactured by Dixie Cement Company. Its physical and chemical properties are given in Table 3.1.

(2) Fly Ash

Fly ash, classified as class F according to ASTM C618, was used in this study. Detailed information on the chemical and physical properties of the fly ash is given in Table 3.2.

(3) Coarse Aggregate

A 3/4-inch nominal maximum size crushed limestone was used as coarse aggregate in this study. The specific gravity of the natural coarse aggregates in the saturated dry condition (S.S.D) was 2.67 .

(4) Fine Aggregate

The natural aggregate was a natural river sand. This aggregate was kept in the saturated surface dried condition until it was used. The specific gravity of the fine aggregate was 2.65.

(5) Chemical Admixtures

An air entraining agent (AEA) and a high range water reducer (HRWR) were used in this study. The air entraining agent was a micro-air admixture complying with the requirement of ASTM C260 "Air Entraining Admixtures for Concrete", while the high range water reducer complied with the requirement of ASTM C494 "Chemical Admixtures for Concrete". Both admixtures are commercially available.

(6) Water

The mix water used throughout the research program was regular tap water. The specific gravity of the water, which was used in the mix proportion calculations, was assumed to be 62.4 1b/cf.

3.1.2. Recycled Aggregate Concrete

The materials that were used in the recycled aggregate concrete mixes were the same materials as those used in the natural concrete mixes, except that the natural coarse aggregates were replaced by recycled aggregates. Recycled aggregates were produced by crushing the original (old) concrete, which was obtained from two main sources. The first source was concrete manholes and pre-cast concrete pipes, which were crushed by the supplier (ESSROC) to an approximately 3-inch maximum size and then reduced to a 3/4-inch maximum size through manual crushing in the Civil Engineering laboratory. The compressive strength of the original concrete was approximately 4,600 psi.

The second source was old 6 by 12-inch concrete cylinders, which were used in previous laboratory experiments and then given to the author for this research project. The cylinders were then separated into three different grades according to their compressive strenths of 3000, 4000, and 5000 psi. Each concrete cylinder was then split in half and fed to a laboratory jaw crusher, which was set at an opening of 1-inch with the breaker jaws in a closed position. The crushed concrete was then screened into a four size fraction by using a sieve shaker. Then, the screened concrete was re-combined to produce concrete aggregates that conform to the standards of ASTM C33.

The recycled aggregates obtained from the first source were utilized in phase I of the study, and the recycled aggregates obtained from the second source were used in phases II and III. The specific gravity of the recycled aggregates in the saturated surface dry condition ranged from 2.40 to 2.55. Grading of the coarse aggregates was performed according to ASTM C136 and the results are shown in Table 3.3 .

3.2 SPECIMEN FABRICATION

3.2.1 Mixing of Concrete

All mix designs used in this research were developed using the absolute volume method. Tables 3.4 through 3.6 gives the mix proportion for both the recycled aggregate concrete and the natural concrete. All mixing was conducted under laboratory conditions in a 7 cubic foot capacity concrete mixer in accordance with the provisions of ASTM C192 "Standard Method of Making and Curing Concrete Test Specimens in the Laboratory". Prior to mixing, the aggregates were air dried to a low moisture content. The aggregates and cement were added and mixed for about 3 minutes before water was added. After water was added, the following procedure was followed : three minutes of initial mixing followed by three minutes of rest, followed by two minutes of mixing. A slump test was then run to determine if it was within the targeted range. If the slump was too low, a water reducing admixture was added to the mixture, which was then mixed for two more minutes. Another slump test was run, and this was repeated until the described slump was reached. For mixes that contained air entraining admixture, the solution was added along with the water, and the air content test was run along with the slump test.

3.2.2 Casting of Test Specimen

After the mixing was completed, the concrete was discharged into a large hopper, from which concrete was extracted as needed for fresh concrete tests and for casting beams and cylinders. For beam specimens, standard 3- inch by 4 -inch by 16-inch steel and plastic molds were used, while for the cylinder specimens, 6-inch by 12-inch plastic molds were used. Molding was done according to the specification of ASTM C31. Casting was accomplished in 3 layers and compaction was done by rodding. Upon completion of casting, all molds were immediately covered with wet burlap and plastic sheeting for 24 hours to prevent loss of moisture.

3.2.3 Curing of Concrete Specimens

Approximately 20 to 24 hours after initial curing, specimens were stripped and kept in the standard moist room where curing continued at about 73 F° (23 C°) and 100 percent relative humidity until the time of testing. The continuation of curing under these conditions was very important for the development of the physical properties of the concrete.

CHAPTER 4

TESTING PROCEDURES

4.1 TEST PROGRAM

A total of 372 concrete specimens were tested for the physical properties of both the RAC and the NAC and more than 290 concrete prisms were tested for the freeze and thaw durability of both concretes . Table 4.1 gives the test program followed in this study. Procedures for the various tests performed are summarized below.

4.2 FRESH CONCRETE TESTING

Slump tests were performed on each mixture in accordance with ASTM C143 "Standard Test Method for Slump of Portland Cement Concrete". Mixes containing air entraining admixture were also tested for air content according to ASTM C231 "Standard Test Method for Air Content of Freshly Mixed Concrete by the Volume Method". Air content tests were conducted periodically on non-air entrained mixes in order to estimate the amount of entrapped air. Table 4.1 gives a listing of each mix and its fresh concrete properties.

4.3 FREEZE-THAW TEST

Portland cement concrete in temperate regions experiences periodic cycles of freezing and thawing. These conditions are reflected in the laboratory by controlling the temperature and moisture using automatic equipment capable of producing continuous periodic cycles. The freeze-thaw machine that was used for testing in this research has the capacity of testing 16 specimens at a time and features automatic time and temperature control. The freezing and thawing cycle consisted of alternately lowering the temperature of the specimens from 40 F° to 0 F° (4.4 C° to -17.8 C°) and raising it from 0 F° to 40 F° (-17.8 C° to 4.4 C°).

The concrete's resistance to freezing and thawing was determined by testing 3 -inch by 4 -inch by 16 -inch concrete prisms in accordance with ASTM C666 "Standard Test Method for Resistance of Concrete to Rapid Freezing and Thawing". This specification allows two procedures : Procedure A requires the freezing and thawing to be performed with the specimens continuously surrounded by water, whereas Procedure B allows the specimens to be surrounded by air during the freezing phase. Since Procedure A is usually found to be the more severe of the two tests, it was selected for use in this research so that any conclusions would be on the conservative side for the expected performance of the concrete in service. After 14 days of standard moist curing as specified by the test method, three specimens from each mix were weighed and tested for fundamental transverse frequency according to ASTM C215 "Standard Test Method for Fundamental Transverse, Longitudinal and Torsional Frequencies of Concrete Specimens". Figure 4.1 shows the test set-up for the fundamental frequency test. At the end of the thawing cycle, the samples were placed in the freeze-thaw machine at 40 F°.

The specimens were tested for resonant frequencies and weights every 36 cycles until either 300 cycles were completed or the squared value of the fundamental frequency was reduced to 60 percent of the square of the initial frequency value.

The relative dynamic modulus of elasticity (RDM) was calculated as follows :

$$RDM = (nc^2/n^2)\times 100$$

where : RDM = relative dynamic modulus of elasticity after C cycles of freezing and thawing, percent.

c = number of cycles at time of testing.

nc = fundamental transverse frequency after c cycles of freezing and thawing.

n = fundamental transverse frequency at zero cycles, in Hertz.

The durability factor (DF) was determined as follows :

$$DF = (PN/M)\times 100$$

where :

DF = durability factor of test specimen.

P = relative dynamic modulus of elasticity at N cycles, percent.

N = number of cycles at which P reaches a specified minimum value for discontinuing the test or the specified number of cycles at which exposure is to be terminated, whichever is fewer.

M = specified number of cycles at which exposure is terminated.

The scale for the durability factor ranges from 0-100. The 100 stands for the best possible frost resistance performance, and the 0 stands for the worst possible performance. In the literature, there is no subjective rating of the durability factor, and interpretation of the rating varies from user to user. For instance, some agencies consider values of DF greater than 80 to be excellent while 60 to 80 to be good.

The change in weight due to freezing and thawing cycles is calculated as follows :

$$W = \{(W1-W2)/W1\}\times 100$$

where : W = weight change of specimen at C cycles of freezing and thawing , percent.

W1 = weight of the specimen at the beginning of the test, lbs.

W2 = weight of the specimen after C cycles, lbs.

4.4 COMPRESSIVE STRENGTH

One of the primary objectives of this research was to evaluate and compare the strength of concrete mixtures containing natural aggregates with the strength of concrete mixtures containing recycled aggregates. Prior to testing and to prevent a stress concentration during testing, 6- inch by 12-inch cylinders were capped using a proprietary compound containing sulphur inside steel retaining rings in accordance with the specifications of ASTM C617 "Practice for Capping Cylindrical Concrete Specimens". The compressive strength test was performed on a 300,000 pound capacity "Forney Compression Testing Machine" in accordance with ASTM C39 "Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens". Strength tests were performed in uniaxial compression up to failure on all mixtures at 7and 28 days. In addition, some of the mixes were also tested at 3 and 90 days. Figure 4.2 shows the set-up for this test.

4.5 MODULUS OF RUPTURE (FLEXURAL STRENGTH)

Flexural strength tests were performed in 3-inch by 4-inch by 16-inch prisms according to ASTM C78 "Standard Test Method for Flexural Strength of Concrete Using Simple Beam with Third-Point Loading". The modulus of rupture was calculated by the relation :

$$f_r = PL/bd^2$$

where :

f_r = flexural strength or modulus of rupture, in psi.

P = maximum applied load, in Ibs.

L = span length in inches.

b = average width of specimen, in inches.

d = average depth of specimen, in inches.

If fracture occurred outside of the middle third of the span length, but by not more than 5 % of the span length, the modulus of rupture was calculated as follows :

$$f_r = 3Pa/bd^2$$

where a is the average distance between the line of fracture and the nearest support measured on the tension surface of the beam, in inches.

When failure occurred outside of the middle third of the span length by more than 5 % of the span length, the test results were discarded. Figure 4.3 shows the set-up for this test .

4.6 SPLITTING TENSILE STRENGTH TEST

Splitting tensile strength tests were performed occasionally to compare with the modulus of rupture. Those tests were performed in accordance with ASTM C496 “Standard Test Method for Splitting Tensile Cylindrical Concrete

Specimens". Cylindrical specimens measuring 6-inch by 12-inch were used in this test. The tensile strength was computed as follows :

$$f_t = 2P/\pi DL$$

where : f_t = tensile strength in psi.

P = maximum applied load in 1b.

L = length of cylinder, in inch.

D = diameter of cylinder, in inch.

4.7 MODULUS OF ELASTICITY

The elastic modulus of elasticity (Young's Modulus) test was performed in accordance with ASTM C469 "Standard Test Method for Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression". Figure 4.2 shows the set-up for the test performed. A compressometer was used to measure the deformation of the cylindrical specimen as it was loaded. The deformation was recorded at every 10-kip load interval up to approximately 60 % of the ultimate compressive strength.

The static elastic modulus was calculated from the stress-strain diagram according to :

$$Ec = (s_2 - s_1) / (\varepsilon_2 - 0.00005)$$

where : Ec = chord modulus of elasticity, psi.

s_2 = stress corresponding to 40 % of ultimate load.

s_1 = stress corresponding to a strain of 0.00050 .

$\varepsilon 2$ = longitudinal strain produced by stress S2.

CHAPTER 5

DISCUSSION OF RESULTS

5.1 PROPERTIES OF RECYCLED AGGREGATE

5.1.1 Grading of Recycled Aggregates

Gradation of aggregates plays a key role in the concrete mix proportions. When a range of graded aggregates is used, the smaller particles fill the spaces between the larger aggregates, thus lowering the void content and decreasing the cement paste requirement.

Figure 5.1 shows an actual grading curve for a typical recycled aggregate. It can be seen that all gradings fall within the allowable limits for the respective aggregate as specified in ASTM C33. The recycled aggregate was produced by crushing original concrete in a jaw crusher. From the practical point of view, it may be concluded that producing a well-graded recycled aggregate that complies with ASTM requirements would not be difficult.

5.1.2 Shape and Texture

Recycled aggregate is angular in shape with well defined edges. The angular shape of recycled aggregate, which results in a high surface-to-volume ratio, might be in part responsible for the reduced workability of the fresh recycled aggregate concrete.

The surface texture of the recycled aggregates was rough and porous due to the presence of old attached mortar. A rough surface provides better bonding and interlocking of the aggregates within the cement paste; hence, a higher compressive strength may be expected for recycled concrete made with recycled aggregates as compared with natural aggregate concrete made with limestone coarse aggregates.

5.1.3 Absorption

The water absorption of recycled aggregates given in Table 5.1 ranged from 4.41 percent to 5.10 percent, depending on the source from which they were derived. Those values were certainly much higher than the corresponding value of the natural aggregates, which had only a 0.30 percent absorption capacity. This difference may be due to the higher absorption of the old mortar attached to the recycled aggregate. Due to its significant effects on both the fresh and hardened concrete properties as shown later, the high absorption capacity of the recycled aggregates was considered the most remarkable characteristic in their physical properties when compared with natural aggregates.

5.1.4 Specific Gravity

The specific gravity of the recycled aggregates in saturated surface dry conditions (S.S.D) as given in Table 5.1 ranged from 2.4 to 2.55, while that of the natural aggregates was 2.67. Again, the noticeable difference was due to the relatively low density of the old mortar, which was attached to the recycled aggregates. The low value of the specific gravity of recycled aggregates is expected to have some effects on the weight of the recycled aggregate concrete on which the fundamental natural frequency depends.

5.2 PROPERTIES OF FRESH RECYCLED AGGREGATE CONCRETE

5.2.1 Workability

The workability of all mixes as given in Table 5.2 was observed in terms of the slump test. Mixes that contained recycled aggregate were harsher and stiffer than mixes that contained natural aggregates. To achieve the same level of slump for both mixes, more superplasticizer was added to mixes of recycled aggregate concrete. This lower workability for the recycled concrete could be attributed to the highly absorbent mortar, which was attached to the recycled aggregates. For the same reason, loss of workability was also noticed to be faster for recycled aggregate concrete than for natural concrete, especially after the first 5 to 10 minutes.

5.2.2 Air content

The entrapped air content of fresh recycled aggregate concrete given in Table 5.2 was found to be slightly higher than that of fresh natural concrete. This might be due to the high void content of the recycled aggregates.

5.3 PHASE I

5.3.1 Compressive Strength and Modulus of Rupture

One of the primary objectives of this study was to evaluate and compare the physical properties of both recycled aggregate concrete containing various amounts of recycled aggregates and natural aggregate concrete. Test results are summarized in Tables 5.3 through 5.5 and shown in graphical forms in Figures 5.2 through 5.7 .

As shown in Figure 5.2, for a replacement ratio of 0.50, the compressive strength of recycled aggregate concrete was 14 percent and 7 percent higher than those of the natural concrete at 7 days and 28 days, respectively. The modulus of rupture, as shown in Figure 5.3, increased by 3 percent at 7 days and decreased by 4 percent at 28 days. At a replacement ratio of 0.75, the compressive strength of the recycled aggregate concrete, as shown in Figure 5.4, was 12 percent higher than that of the natural concrete at 7 days and almost even at 28 days. The modulus of rupture of the recycled aggregate concrete increased by 2 percent at 7 days and decreased by 5 percent at 28 days. From these results, it appears that replacement of natural aggregates with recycled aggregates at fifty and seventy-five percent ratios was found to be advantageous and beneficial to the development of the compressive strength of the recycled aggregate concrete and not to have significant adverse effects on the modulus of rupture.

Complete replacement of natural coarse aggregates with recycled aggregates (R =1.0) was also advantageous to the development of the compressive strength of the recycled aggregate concrete. The compressive strength, as shown in Figure 5.6, increased by 16 percent and 2 percent at 7days and 28 days, respectively. However, the modulus of rupture as shown in Figure 5.7 increased by 2 percent and decreased by 9 percent at 7 days and 28 days, respectively.

Figures 5.8 and 5.9 show a summary of the compressive strengths with the various replacement ratios at 7 days and 28 days, respectively. As seen at 7 days testing, the compressive strength of the RAC for all replacement ratios was higher than that of the NAC, with the highest occurring at R = 1.00. At 28 days, the same trend was observed. The observed increases in the compressive strength of the RAC may be due to the angular shape and rough texture of the recycled aggregates which may have provided better bonding and interlocking between the cement paste and the recycled aggregates themselves compared with those of natural concrete. Another cause of this trend could be the absorbent nature of the recycled aggregates, which absorbed some of the water during mixing and caused a reduction of the actual water cement ratio of the RAC mixes. It is important to mention here that the compressive strength of the original concrete from which the recycled aggregates were derived was about 4,600 psi at 28 days, which is close to the design compressive strength of 5,000 psi. If this value were considerably less than the design compressive strength, additional investigation would be necessary. Phase III of this investigation deals with this concern.

Figures 5.10 and 5.11 show a summary of the modulus of rupture at 7 days and 28 days. As seen at 7 days, the modulus of rupture of the RAC at all replacement ratios was higher than those of the NAC. However, at 28 days, the modulus of rupture of RAC at all replacement ratios was lower than that of NAC and had decreased by as much as 10 percent. This increase in the flexural strength of the RAC at early ages and decrease at later ages may be attributed to the rough-textured recycled aggregates. According to Mehta (1986), a stronger physical bond between the rough-textured aggregate and the cement paste is responsible for the increased tensile strength at early ages. At later ages, however, when chemical interaction between the aggregate and the paste begins to take effect, the effect of the surface texture may not be as important.

5.3.2 Modulus of Elasticity and Splitting Strength

The compressive strength and the modulus of rupture of the RAC and the NAC were determined and compared at various replacement ratios in the previous sub-section. It was concluded that the use of the recycled aggregate to replace all natural aggregate is surely feasible without producing adverse effects on the aforementioned properties of concrete. However, a complete investigation of other physical properties of concrete, such as the modulus of elasticity and the splitting tensile strength, are needed to provide an overall picture of the behavior of the RAC verses the NAC.

The stress-strain curves for the RAC and the NAC are given in Figure 5.12. As shown in this figure, strains for the RAC were higher than those of the NAC under the same loads mainly due to the lower elastic modulus of the recycled aggregates. Nevertheless, the shape of the stress-strain curve for the RAC was similar to that of the NAC, which leads to the conclusion that there would be no obstacles in the structural design process based on the theory of plasticity.

Table 5.6 gives all the physical properties of the RAC and the NAC, including the tensile splitting strength and the modulus of elasticity. As shown in Figure 5.13, the tensile strength of the RAC was 28 percent and 5 percent higher at 7 and 28 days, respectively. However, a reduction of 3 percent occurred at 90 days. This trend is similar to that of the development of the modulus of rupture in which there was a fast rate of increase at early ages and a relatively low rate of decrease at later ages, and the same explanation that was given before applies to this trend as well.

As shown in Figure 5.14 , the modulus of elasticity of the RAC was lower than that of the NAC by 9 percent and 16 percent at 7 days and 28 days, respectively. This reduction is due to the old mortar with a comparatively low modulus of elasticity, which was attached to the recycled aggregates. The lower values for the modulus of elasticity of the RAC can cause higher deflection of structures made of recycled aggregates. However, with structures in which deflection is not a significant problem, any disadvantage may cease to exit.

5.3.3 Development of Physical Properties

Table 5.7 gives the development of the physical properties for both RAC.3 and NAC.3 at 3, 7 and 28 days for a replacement ratio of R = 1.00. As shown in Figure 5.15, the rate of development of the compressive strength at an early age of the RAC was faster than that of the NAC. For instance, the RAC developed 84 percent of the 28-day strength in 7 days, while 75 percent of the NAC was attained in 7 days. For the tensile strength as shown in Figure 5.16, the rate of development was very fast at 7 days and almost the same at 28 days. For the modulus of elasticity, however, the natural concrete values at all ages were higher than those of the recycled aggregate concrete as shown in Figure 5.17. It may be concluded from the above results that the strength properties of the RAC developed at a faster rate during the early ages of maturity than those of NAC.

5.4 PHASE II

5.4.1 Case I

5.4.1.1 Electrodynamic Determination of the Modulus of Elasticity

The dynamic modulus of elasticity is generally used to determine the frost resistance of concrete because it is non-destructive to concrete specimens. Since the test involves a vibrating technique, the modulus is known as a dynamic modulus. The variation in the value of the modulus over the entire duration of freezing and thawing cycles provides a good indication of the variation in the strength of the concrete specimen. The strength variation, in turn, reflects the degree of the freeze-thaw resistance.

The modulus of elasticity was determined by subjecting the specimen to longitudinal vibration to determine its natural frequency. The dynamic modulus was determined at intervals of 36 cycles until the specimen had been subjected to 300 cycles or until its relative dynamic modulus of elasticity reached 60 % of the initial modulus, whichever occurred first.

The relative dynamic modulus values for Case I are listed in Table 5.9. At 36 cycles of freezing and thawing, the relative dynamic modulus (RDM) of the natural concrete reached 33 % of its intial value, while it took 72 cycles for the recycled aggregate concrete to reach 45 % of its initial value. Figure 5.18 shows the RDM for both concretes versus the number of cycles. As can be seen, there was a sudden drop in the RDM for both the RAC and the NAC at 36 and 72 cycles, respectively. The short term survival of both concretes may be mainly due to the existence of a significant amount of freezable water in the capillary pores in the cement paste, which upon freezing produced expansive internal pressure that damaged the concrete.

5.4.1.2 Weight Change

The weight change of concrete specimens over the freezing and thawing cycles is another indication of the deterioration of concrete. Weight change provides an idea of the amount of moisture absorbed due to the cracking of the specimen caused by the expansion of the cement paste.

Table 5.8 lists the weight change percentages for both the RAC and the NAC. As seen in Figure 5.19, the weight of both concretes increased as the number of cycles increased, and the weight increase of the NAC was higher than that of the RAC, which indicates that more water was absorbed by the NAC. The deterioration shown by the weight increase trend of both the RAC and the NAC was consistent with that shown by the drastic drop in the relative dynamic modulus of elasticity.

The durability factors for both the RAC and the NAC are shown in Figure 5.20. It is obvious that the durability factors for both concretes were very low, resulting in a very poor durability performance. Although the durability factor for the RAC was higher than that of the NAC, the effects of using recycled aggregates in concrete cannot be completely understood at this point in the investigation. The higher durability factor of the RAC may be attributed to less water in the capillary pores in the cement paste, which was a result of the high absorption nature of recycled aggregates, leading to lower hydraulic pressure in the RAC. Nevertheless, it may be concluded that testing concrete under the conditions used (high w/c, non-air concrete) will result in non-frost resistant concrete and that other ways to provide immunity against frost action are needed.

5.4.2 Case II : High Strength Concrete

5.4.2.1 Physical Properties

The physical properties of both the RAC and the NAC for a water-cement ratio of 0.29 are given in Table 5.10. As shown in Figure 5.21, a reduction in the water cement ratio increased the compressive strength as expected for both the RAC and the NAC. However, the rate of increase in the RAC's compressive strength was not equivalent to that of the NAC's. For instance, at 28 days of age, the NAC's compressive strength increased from 5523 psi to 7416 psi (34% increase), while the RAC's increased from 5635 psi to 6578 psi (14% increase).

Other properties of the RAC, as shown in Figures 5.22 to 5.23, experienced the same trend as well.

The reduction in the properties of the RAC may be due to the lower original compressive strength (4,600 psi) from which the recycled aggregates were derived compared to the required design compressive strength of 7,000 psi for both the RAC and the NAC. As a result, the failure of the RAC may have been mainly caused by the use of recycled aggregates, while the failure of the NAC may have been mainly caused by the failure of the cement paste. Visual observation supported this explanation, where in the RAC, the failure cracks passed through the recycled aggregates splitting them, along with the mortar. On the other hand, in the NAC, failure cracks passed through the mortar and around the natural aggregates causing bond failure at the aggregate-paste interface. Therefore, the recycled aggregates derived from the 4,600 psi original concrete may have been weaker than the crushed limestone. The effects on the physical properties of the RAC of using various strengths of original concretes, from which the recycled aggregates are derived, need more in-depth investigation. This situation is dealt with in Phase III of this study.

5.4.2.2 Durability Characteristics

One primary hypothesis tested in this study was whether lowering the water-cement ratio without using air entrainment would give concrete immunity against frost damage. It was concluded earlier that using a water-cement ratio of 0.47 provided no such immunity for either the RAC or the NAC. However, a lower water-cement ratio might bring about a higher level of durability.

Tables 5.11 through 5.14 gives the relative dynamic modulus of elasticity for both the NAC and RAC. Figure 5.24 shows the relative dynamic modulus of elasticity versus the number of cycles for a water-cement ratio of 0.29. As shown, the NAC survived more than 300 cycles of freezing and thawing without showing any loss in its RDM, while the RAC experienced a significant decrease in its RDM after about 100 cycles. In fact, the test for the RAC was interrupted after 254 cycles because its RDM decreased below 60 percent of its initial value.

The excellent immunity of the NAC against the freeze-thaw action might be due to its improved water pore system, which resulted from decreasing the water-cement ratio. In other words, lowering the water cement ratio reduced the number and volume of the capillary pores in the cement paste, thus eliminating the chances of freezable water, which is the main cause of internal expansive pressure.

In the case of the RAC, the decrease in the water cement ratio from 0.47 to 0.29 apparently did not offer full immunity against frost action, although its durability performance improved slightly as indicated by its RDM. The lack of full immunity may be attributed to the existence of significant amounts of freezable water in the cement paste's capillary pores. These capillary pores were not modified or improved enough to provide such immunity, a fact consistent with the lower compressive strength of the RAC compared to that of the NAC. Another possible reason for the lack of immunity could be the absorption and porosity characteristics of the recycled aggregates. These characteristics may have caused the recycled aggregates to become easily saturated and, upon freezing, to develop internal pressure within the aggregates themselves. The resulting internal pressure may either have exceeded the tensile strength of the recycled aggregates and caused their failure or have expelled some of the water from within to the surrounding cement paste and helped to damage it. A combination of all of these mechanisms may have led to the early deterioration of the RAC. Case III of this investigation sheds more light on the durability role played by the recycled aggregates.

Figure 5.25 shows the weight change versus the number of freezing and thawing cycles. As shown, the NAC had less than 0.2 % weight gain throughout the testing period. This very slow weight increase rate indicates the low absorption and permeability of the NAC due to its dense cement paste and the low absorption of the natural aggregates. The absorbed water had no negative effects on the freeze-thaw resistance of the NAC, and perhaps it might have helped in the hydration process of the cement paste since the weight gain remained almost stable after about 100 cycles of freezing and thawing. On the other hand, the RAC showed a sharp weight gain throughout the test period, indicating its high absorption characteristics. Increases in weight for the RAC consistently caused negative performances in its RDM, suggesting a negative correlation between weight gain and reductions in the RDM.

Figure 5.26 shows the durability factor for both the RAC and the NAC. The durability factor of the NAC was 100 percent, suggesting full frost resistance, while that of the RAC was about 24 percent, indicating very poor durability. It may be concluded that lowering the water-cement ratio was a very significant factor for the full development of the freeze-thaw durability of the NAC, while it failed to provide such durability for the RAC. Other means to provide frost resistance for the RAC are needed in order to guarantee the feasibility of using recycled aggregates in concrete. Since everything is the same in the NAC and the RAC except the coarse aggregates, the recycled aggregates seem to be the cause of the negative effects on the development of the strength and durability of the RAC.

5.4.3 Case III : High Fly-Ash Content Concrete

5.4.3.1 Workability

It was found that increasing the fly-ash content from 14 percent to 28 percent by weight of cement resulted in a much more workable mix. This increase in workability may be due to the mechanical action of fine spherical particles present in the ash, replacing some of the original angular cement particles which required more water. This is in agreement with Berry and Malhotra (1988) and Malhtra (1987), who suggested that fly ash particles act as ball bearings and reduce friction between other mortar constituents, resulting in the need for less water for a specified workability.

5.4.3.2 Physical Properties

The physical properties of the high fly-ash content of both the RAC and the NAC are listed in Table 5.15. Figure 5.27 shows the 28-day compressive strength at 14 percent and 28 percent fly-ash contents. As shown in the figure, the compressive strength of the RAC decreased from 5635 psi to 5152 psi (i.e., a 9 percent reduction), and that of the NAC decreased from 5523 psi to 4952 psi (i.e., an 11 percent reduction). A similar corresponding reduction in the modulus of rupture, as shown in Figure 5.28, also occurred, in which the modulus of rupture of both the RAC and the NAC decreased by 14 percent and 10 percent, respectively. Although this study did not seek to determine the critical amount of fly-ash in either the RAC or the NAC that would cause adverse effects on their physical properties, a 28 percent replacement by weight of cement did not appear to cause significant reductions in their measured physical properties.

5.4.3.3 Durability Characteristics

One of the main objectives of this study was to determine whether increasing the fly-ash content without using air entrainment would improve the freeze-thaw resistance of either the RAC or the NAC. Tables 5.16 and 5.17 give the RDM of both the RAC and the NAC throughout the test period. As shown in Figure 5.29, both the NAC and the RAC showed significant improvement in their RDM, decreasing only to 94 percent and 84 percent of their initial values, respectively, at about 320 cycles. This significant improvement might have been due to the refinement of the water pore system caused by the vital distribution of particle-size fly ash. Since fly ash is known to be finer than cement, it can be expected that fly-ash concretes will have more cementitious particles per unit volume allowing a greater packing density and resulting in a more refined cement paste. Consequently, this would have reduced the amount of capillary pores in both the RAC and the NAC, resulting in a less porous paste.

As shown in Figure 5.30, there was a slow rate of increase in the weight of the NAC up to 180 cycles, after which there was a continuous decrease. This may be explained by the observed increase of scaling, which overshadowed the increase of weight due to the ingress of water. As a result, the net long-term effect was a loss in weight. Even though there was a moderate amount of scaling for the RAC, its RDM exhibited little corresponding reductions. In the RAC a constant weight increase occurred throughout the test period.

The refinement of the water pore system of both the RAC and the NAC due to the increase of the fly-ash content did not provide equal durability performance in both concretes. As shown in Figure 5.31, the NAC exhibited an excellent durability performance (Durability Factor , D.F. = 94), while the RAC achieved only a good durability performance (D. F. = 84). The difference between the two durability performances was slight; nevertheless, this difference should be attributed to the use of recycled aggregates in the RAC, since this was the only difference between the two concretes. As mentioned previously, the high absorption capacity of the recycled aggregates greatly distinguished them from the corresponding natural aggregates. This high absorption capacity provided a high potential for saturation of the recycled aggregates, which upon freezing may have expelled some of their water to the surrounding cement paste, hindering its durability performance. This explanation excludes the possibility that the freezing of the water inside the recycled aggregates would have caused internal damage to the aggregates themselves and completely damaged the durability of the RAC. This exclusion may be justified by the slight negative effects on the durability performance of the RAC that occurred; but if this explanation were considered, the disruption of the recycled aggregates would have occurred and caused a very poor durability performance, which was not the case. Case IV of this phase lends support and validity to the argument for excluding this explanation.

5.4.4 Case IV : Air Entrained Concrete.

5.4.4.1 Physical Properties

Air entrainment of concrete is considered to be a breakthrough in concrete technology because it provides immunity against frost damage. A 5 to 6 percent air content is usually needed to accomplish such immunity. In this case of the study, both the RAC and the NAC were entrained with 5 percent air content. The physical properties of both RAC and NAC are given in Table 5.20. The effects of such air entrainment on the physical properties of both the RAC and the NAC were strongly noticeable. For instance, as shown in Figure 5.32, the compressive strength of the NAC decreased from 5623 psi for zero percent air content to 4233 psi for 5 percent air content (i.e., a 25 percent reduction), and that of the RAC decreased from 5635 psi for zero percent air content to 4339 psi for 5 percent air content (i.e., a 23 percent reduction). These reductions in the compressive strengths of both the NAC and the RAC indicated that, consistent with other researchers, for every one percent of air content, there was a 4 to 5 percent reduction in compressive strength.

5.4.4.2 Durability Characteristics

A major question tested in this study was whether using air entrainment would provide equivalent frost resistance for both the RAC and the NAC. Table 5.21 through 5.24 give the natural frequencies and the relative dynamic moduli for both the RAC and the NAC throughout the test period. As shown in Figure 5.34, both concretes survived more than 300 cycles of freezing and thawing without showing any reductions in their relative dynamic modulus of elasticity. This significant improvement, compared to Case I in which there was no air entrainment, was solely due to the positive effects of the entrained air bubbles in the concrete, which serve as reservoirs to receive water as it flows from the freezing sites. As a result, the internal hydraulic pressure within the cement paste, which normally causes damage to the concrete, was relieved. The curves in figures 5.35 show the percent of weight change for both the RAC and the NAC. As shown, there was a slow rate of increase in the weight of both the RAC and the NAC. Durability factors for both concretes are shown in Figure 5.36. As shown, both concretes behaved extremely well (D.F.=100). Thus, it may easily be concluded that air entrainment is as effective for the RAC as for the NAC, and the notion that the RAC cannot be air entrained was proven not valid.

5.5 PHASE III

Table 5.25 shows the physical properties of recycled aggregate concretes made with recycled aggregates that were derived from three different original concretes with compressive strengths of 3000 psi, 4000 psi, and 5000 psi. As shown in Figures 5.37 and 5.38, with a constant water-cement ratio of 0.40, the variations in the resulting compressive strengths of the RAC indicated the effects of the original concretes from which the recycled aggregates were derived. For instance, compared to natural aggregate concrete, at 7 days of age, the RACs derived from 3000 psi and 4000 psi original concretes showed 13 percent and 6 percent reductions, respectively, in their compressive strength; however, the RAC derived from 5000psi original concrete showed a 7 percent increase in its compressive strength. The same trend continued at 28 days of age where the RACs derived from 3000 and 4000 psi original concretes showed 15 percent and 8 percent reductions, respectively, in their compressive strength, while the RAC derived from 5000 psi original concrete showed a 4 percent increase. However, when comparing the original strengths from which the recycled aggregates were derived with the new compressive strengths of the RACs, it is obvious that it is feasible to make recycled aggregate concretes that are stronger than the corresponding original concretes provided the new mix design compressive strength is higher than the original compressive strength of old concretes. For example, with a 6000 psi design compressive strength, 5000 psi, 5390 psi, and 6086 psi compressive strengths were achieved for the RACs derived from original concretes of 3000 psi, 4000 psi and 5000 psi, respectively. From the above discussion, it may be concluded that the compressive strength of the RAC is largely determined by both the strength of the original concrete from which the recycled aggregates were derived and the design mix of the RAC. If the new mix design strength is equal or higher than the original strength, then the expected strength of the RAC will be equal or higher than the original strength.

Figure 5.39 shows the modulus of rupture for the three types of RAC compared with the NAC at 28 days of age. As shown, the modulus of rupture was almost equal for the three types of RAC and almost 13 percent lower than that of the NAC. Therefore, the strength of the original concrete, from which the recycled aggregates were derived, apparently had little affect on the modulus of rupture of the RAC. As shown in Figure 5.40, the modulus of elasticity of the three types of the RAC was less than that of the NAC.

Figure 5.41 shows the relative dynamic modulus of elasticity for the three types of RAC and the NAC. As shown in the figure, with an average air content of 5 to 6 percent, no concretes showed a reduction in their RDM. As also shown in Figure 5.42, the slow change in weight corresponds to the constant RDM for all concretes, except in the case of the RAC derived from 3000 psi original concrete.

Here some scaling took place, leading to a reduction in the net weight in the final stage of testing. Nevertheless, the durability factor for all concretes was excellent, indicating high durability performance, as shown in Figure 5.43. In fact, the quality of the original concrete from which the recycled aggregates were derived had no effect on the durability performance of the RACs, and that was mainly due to the significant effects of air entrainment.

It may be concluded from the above results that it is feasible from both strength and durability criteria to produce recycled aggregate concrete from original concrete with compressive strengths as low as 3000 psi. This recycled aggregate concrete could provide adequate frost resistance and acceptable physical properties compared to the NAC.

5.6 RELATIONSHIPS BETWEEN CONCRETE PROPERTIES

5.6.1 Introduction

This section presents the results of regression analyses that were performed to establish the relationships between the various properties of the RAC and the NAC obtained in this study. Regression analyses were performed to relate one concrete property to another, and the best fit equations were compared to the equations usually used by the ACI Code.

5.6.2 Seven and Twenty-Eight Day Relation

The 7-day compressive strength of concrete is usually believed to be between 60 to 70 percent of the 28-day compressive strength under standard moist curing. Hool and Pulver (1937) proposed the following relation for the 7-day and the 28-day strengths :

$$S_{28} = S_7 + 30\sqrt{S_7} \qquad (5.1a)$$

where S_{28} and S_7 are the 28-day and 7-day compressive strength (in psi) respectively. In his investigation, Oluokun et al. (1991) concluded that the above relation does not correlate well with research results, and he proposed the following relation:

$$S_{28} = 8.87\ (S_7)^{0.774} \qquad (5.1b)$$

Regression analysis to estimate an empirical relationship between the 7-day and the 28-day compressive strength of both the NAC and the RAC was performed on the results obtained in this study. The following relations are proposed by the author for both the NAC and the RAC, respectively:

$$S_{28} = 11.17\ (S_7)^{0.742} \qquad (5.2)$$

$$S_{28} = 10.73\ (S_7)^{0.742} \qquad (5.3)$$

A comparison between the equations of Hool and Pulver, Oluokun et al., and this author's proposed relations is given in Tables 5.34 and 5.35 for both the NAC and the RAC, respectively. As shown in Table 5.34 for the NAC, the total average ratio (predicted / actual) for the relation of Oluokun et al. and this author's proposed relation is 3.5 percent and 0 percent, respectively, indicating a good correlation of their results. However, Hool and Pulver's relation resulted in a 12.2 percent, indicating systematic overestimation of the 28-day compressive strength. Nevertheless, the coefficient of variation for all the relations of the 28-day compressive strength is essentially the same.

The relation of Hool and Pulver and the relation of Oluokun et al. were formulated for natural concrete and when applied to the RAC, as seen in Table 5.35, they resulted in 18.2 and 8.5 percent total average overestimation, respectively. However, the author's proposed relation derived specifically for the RAC resulted in only a 0.5 percent total average overestimation. Therefore, the prediction of the 28-day relationship of the RAC using NAC relations does not correlate well with the actual development of the RAC's compressive strength.

5.6.3 Relationship of the Compressive Strength and the Modulus of Rupture

The compressive strength and the modulus of rupture are closely related. As the compressive strength increases, the modulus of rupture also increases. One of the most commonly used equations for relating the compressive strength and the modulus of rupture in the field of structural concrete is the following ACI equation:

$$f_r = 7.5\sqrt{f'_c} \qquad (5.4)$$

where f_r = modulus of rupture, psi.

f'_c = compressive strength, psi.

Regression analyses were performed to establish empirical relationships between the compressive strength and the modulus of rupture obtained in this study using the following regression model

$$f_r = A\sqrt{f'_c} \qquad (5.5)$$

The following equations are proposed for the NAC and the RAC respectively :

$$f_r = 12.51\sqrt{f'_c} \qquad (5.5a)$$

$$f_r = 11.37\sqrt{f'_c} \qquad (5.5b)$$

The ACI equation was compared with the proposed NAC and RAC relations, and the results are given in Tables 5.36 and 5.37. As shown in these tables, the ACI equation resulted in a significant underestimation (sometimes as much as a 40 percent underestimation) of the modulus of rupture of both concretes, thus generating conservative results. However, the proposed relations resulted in less than a 12 percent over / underestimation.

5.6.4 Relationship of Compressive Strength and Splitting Tensile Strength

The following equation to estimate the splitting tensile strength of concrete is proposed by ACI Committee 363 (1984):

$$f_t = 7.4\sqrt{f'_c} \qquad (5.6)$$

where f_t = splitting tensile strength, psi.

f'_c = the compressive strength, psi.

Regression analyses performed on the results obtained in this study, using the form of equation 5.7, gave the following relations for both the NAC and the RAC, respectively:

$$f_t = 7.44\sqrt{f'_c} \qquad (5.7)$$

$$f_t = 7.12\sqrt{f'_c} \qquad (5.8)$$

Tables 5.38 and 5.39 give a comparison between the ACI equation and those relations obtained in this study. As show in these tables, the ACI equation provides an excellent representation of the compressive and tensile strength relationships.

5.6.5 Relationship of the Modulus of Rupture and the Splitting Tensile Strength

Although both the modulus of rupture and the splitting tensile strength are usually used to estimate the tensile strength of concrete, they do not yield equivalent results. According to Troxell (1968), the splitting tensile strength ranges from 50 to 75 percent of the modulus of rupture. Other researchers like Shah and Ahmed (1985) have estimated the splitting tensile strength to be between 65 to 70 percent of the modulus of rupture. This difference is mainly due to the use of the flexure formula, which assumes a linear stress-strain relationship throughout the cross-section of the beam specimen, to calculate the modulus of rupture.

Regression analyses were performed on the results obtained in this study using the following linear regression model

$$f_t = Af_r \qquad (5.9)$$

The following equations are proposed for both the NAC and the RAC respectively:

$$f_t = 0.62f_r \qquad (5.10)$$

$$f_t = 0.66f_r \qquad (5.11)$$

As shown in these two equations, the splitting tensile strength was approximately 60 percent and 65 percent of the modulus of rupture for the NAC and the RAC, respectively, which agrees with the range mentioned by the aforementioned researchers. The comparison of the results are summarized in Tables 5.40 and 5.41 for both the NAC and RAC respectively.

5.6.6 Relationship of the Compressive Strength and the Modulus of Elasticity

According to the ACI Code (1989), the empirical relationship between the compressive strength and the modulus of elasticity is given by

$$Ec = 33\rho^{3/2}\ (f'_c)^{1/2} \qquad (5.12)$$

ρ = unit weight of concrete, lb/ft^3

For normal weight concrete approximately equal to 144 pounds per cubic feet, the above equation becomes the well known form :

$$Ec = 57{,}000\ (f'_c)^{1/2} \qquad (5.13)$$

This form of the equation was used as a model in this study. Regression analyses were performed on the experimental data and the following relations are obtained for both the NAC and the RAC respectively :

$$Ec = 65{,}900(f'_c)^{1/2} \qquad (5.14)$$

$$Ec = 51{,}683(f'_c)^{1/2} \qquad (5.15)$$

Based on the average unit weight of both the NAC and RAC, the above equations become :

$$Ec = 34.89\ \rho^{3/2}\ (f'_c)^{1/2} \qquad (5.16)$$

$$Ec = 30.35\ \rho^{3/2}\ (f'_c)^{1/2} \qquad (5.17)$$

The coefficients in the NAC equations 5.15 and 5.17 are higher than those in the ACI equations 5.14 and 5.13 by about 16 percent and 6 percent respectively. Therefore, the use of the ACI equations (i.e. eq. 5.13 and 5.14) to estimate the modulus of elasticity of the NAC will result in underestimating the modulus of elasticity. This finding is in agreement with Oluokun et. al., who reported that the use of the approximate equation 5.14 generally results in an underestimation of the modulus of elasticity. Table 5.42 gives a comparison of the measured modulus of elasticity of the NAC with the ACI equations 5.13 and 5.14.

The coefficients of the RAC equations 5.16 and 5.18 are lower than those of the ACI equations 5.14 and 5.13 by about 9 percent and 10 percent respectively. Therefore, the use of the ACI equations to estimate the modulus of elasticity of the RAC will result in unconservative estimates. Table 5.43 shows a comparison between the ACI equation 5.14 and the proposed equation for the RAC.

CHAPTER 6

CONCLUSIONS AND RECOMMENDATIONS

6.1 CONCLUSIONS

The following conclusions are made concerning the physical properties of recycled aggregates :

(I) The production of well-graded recycled aggregates that meet ASTM specifications is feasible.

(II) Due to the old attached mortar in recycled aggregates, their water absorption capacity is higher than that of limestone coarse aggregates.

(III) The specific gravity of recycled aggregates is lower than that of the natural aggregates.

The following conclusions are made concerning the physical properties of both the recycled aggregate concrete and the natural aggregate concrete:

(I) The replacement of natural aggregates with recyled aggregates was found to be achievable and, in some cases, advantageous to the development of the compressive and tensile strengths of the RAC. This was specifically the case when the new mix design strength of the RAC was not significantly different from the strength of the original concrete from which the recyled aggregates were derived.

(**II**) The modulus of elasticity of the RAC was found to be always lower than that of the NAC. However, reductions in the RAC should not be considered obstacles to the feasibility of using recycled aggregates in concrete if appropriate structural applications are selected.

(**III**) The strength variations of the original concretes, from which the recycled aggregates were derived, had some effect on the new physical properties of the RAC. This may cause some concerns for the practical production of high quality RAC in a crushing site that receives non-uniform quality demolished concretes from different sources simultaneously. However, these effects on the properties of the RAC were not significantly detrimental to the production of medium quality RAC. In fact, a 5000 psi compressive strength RAC was produced from the recycled aggregates derived from a 3000 psi original concrete. Therefore, a medium strength concrete may be produced from a mixture of low-strength, non-uniform concretes.

(**IV**) The use of high absorption recycled aggregate in the RAC resulted in a reduced freeze-thaw resistance due to the saturation of the aggregate during the test. This conclusion is valid for all of the RACs tested for freeze-thaw durability, except when air entrainment was involved.

(**V**) Lowering the water-cement ratio to a certain level was highly beneficial to the development of the freeze-thaw durability of the NAC. At a water-cement ratio of 0.29, the freeze-thaw durability of the NAC performed extremely well. However, using the same water-cement ratio for the RAC failed to provide full durability.

(**VI**) The use of a higher fly-ash content in both the RAC and the NAC resulted in significant improvements in their freeze-thaw durability and caused no significant reductions in their corresponding physical properties.

(**VII**) The use of entrained air was far more effective at improving the freeze-thaw durability of the RAC than the other two methods used, i.e., lowering the water-cement ratio and increasing the fly-ash content. Therefore, air entrainment is clearly the most efficient method tested in this study for producing freeze-thaw resistance in the recycled aggregate concrete. However, even though air entrainment provided excellent freeze-thaw durability, it caused significant reductions in the physical properties of both the RAC and the NAC. Nevertheless, the necessity of achieving frost-resistant concrete typically outweighs the negative effects on the corresponding physical properties, especially for low and medium strength concretes where the reductions will not be as great as they would be for high strength concrete.

(**VIII**) The following relationships between compressive strength and modulus of elasticity, compressive strength and modulus of rupture, splitting tensile strength and modulus of rupture for the RAC are proposed :

$$Ec = 30.35\,\rho^{3/2}\ (f'_c)^{1/2} \qquad (6.1)$$

$$f_r = 11.37\,\sqrt{f'_c} \qquad (6.2)$$

$$f_t = 7.12\sqrt{f'_c} \qquad (6.3)$$

(**VX**) The results of this study provide strong support for the feasibility of using recycled aggregates instead of natural aggregates for the producton of concrete. Provided that the strength and durability characteristics of the resulting recycled aggregate concrete are both assured, using recycled aggregate concrete as a viable option in some projects could provide monetary savings and/or environmental benefits. The overall findings in this study provide this assurance.

6.2 RECOMMENDATIONS

(**I**) The operations of crushing old concrete for this study resulted in a significant amount of unused fine aggregates. Instead of dumping these unused aggregates, they could possibly be used in non-structural type materials, such as flowable fill, which consists mostly of fine aggregates. Research is needed to determine the feasibility of recycling these unused fine aggregates for non-structural type materials since in large recycling projects, huge amounts of these materials will be left over.

(II) The resulting RACs in this study were medium strength concretes. Research may be necessary to determine whether it is feasible to produce frost-resistant, high strength RAC without air entrainment.

(III) The optimum replacement of cement by fly-ash in both RAC and NAC to obtain the highest durability performance should be investigated.

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APPENDICES

APPENDIX A
TABLES

Table 2.1 Effect of Age of Cement Paste on its Permeability Coefficient, w/c = 0.50. (Mindess, 1981).

AGE (days)	Kp (m/s)	
Fresh paste	1.00E-05	independent of w/c
1	1.00E-08	capillary pores interconnected
3	1.00E-09	
4	1.00E-10	
7	1.00E-11	
14	1.00E-12	
28	1.00E-13	
100	1.00E-16	Capillary pores discontinuous
240	1.00E-18	

Table 2.2 Curing Time Required to Produce a Discontinuous System of Capillaries. (Mindess, 1981).

W/C Ratio	Curing Time (days)
0.40	3
0.45	7
0.50	28
0.60	180
0.70	365
>0.70	not possible

Table 2.3 Total Air Content for Frost-Resistant Concrete. (ACI, 1971)

Nominal maximum aggregate size (in)	Air content (%)	
	Severe exposure	Moderate exposure
3/8	71/2	6
1/2	7	51/2
3/4	6	5
1	6	41/2
11/2	51/2	41/2
2	5	4
3	41/2	31/2

Table 3.1 Chemical and Physical Properties of Cement.

CHEMICAL DATA		PHYSICAL DATA	
COMPOSITION	PERCENT		
Silicon Dioxide	20.5	Compressive Strength	
Aluminum Dioxide	5.0	1 Day	2090 psi
Ferric Oxide	3.4	3 Day	3520 psi
Calcium Oxide	63.7	7 Day	4670 psi
Magnesium Oxide	1.0	Set Time :	
Sulfur Trioxide	2.7	Initial Set (min.)	118
Loss in Ignition	0.95	Final Set (min.)	237
Insoluble Residue	0.22	Specific Surface	$366 M^2/Kg$
Tricalcium Silicate	59	Air Content	10%
Tricalcium Aluminate	7		

Table 3.2 Chemical and Physical Properties of Fly Ash.

Chemical Analysis Results, %		ASTM C618
Silicon Dioxide	49.8	----
Iron Oxide	11.9	----
Aluminum Oxide	24.2	----
Sum of Silicon, Iron oxide and & Aluminum Oxide	85.9	70% min.
Magnesium Oxide	1.11	----
Sulfur Trioxide	1.56	5 % max.
Moisture Content	0.23	3 % max.
Loss of Ignition	3.52	12 % max.
Calcium Oxide	1.62	----

Physical Analysis Results, %			ASTM C618
Uniformity Requirements			
% Retained # 325 Sieve : 15.0	Average :17.6	-2.6%	5% max. from averg.
Specific Gravity : 2.41	Average : 2.3	3.5%	5 % max. from averg.
Fineness (Amount Retained on #325 Sieve)		15.0%	34 % max.

Table 3.3 Mechanical (Sieve) Analysis of Recycled Coarse Aggregate.

Sieve	Fraction Retained,(1b)	Fraction Retained, %	Cumulative Retained, %	Cumulative Passing, %
3/4 in	168	3.3	3.3	96.7
1/2 in	3414	67	70.3	29.7
3/8 in	1335	26.2	96.5	3.5
No.4 (4.75mm)	109	2.2	98.7	1.3
No.8 (2.36mm)	17	0.30	99	1
Pan	52	1.0	100	0

Table 3.4 Mix Proportion for RAC and NAC of Phase I.

Mix ID	Cement lb.	Fly Ash lb.	Gravel lb.	Recycled Conc. lb	Sand lb.	Water lb.
RAC 50%	612	102	910	810	1163	337
RAC 75%	612	102	455	1216	1163	337
RAC.1	612	102	---	1620	1163	337
RAC.2	612	102	---	1620	1163	337
RAC.3	612	102	---	1620	1163	337
NAC.1	612	102	1820	---	1163	337
NAC.2	612	102	1820	---	1163	337
NAC.3	612	102	1820	---	1163	337

Table 3.5 Mix Proportion for RAC and NAC of Phase II.

Mix ID	Cement lb.	Fly Ash lb.	Gravel lb.	Recycled Conc. lb	Sand lb.	Water lb.
RAC.4	612	102	---	1620	1163	337
RAC .5	900	127	---	1512	1115	300
RAC.6	514	200	---	1620	1163	337
RAC.7	612	102	---	1620	1163	337
NAC.4	612	102	1820	---	1163	337
NAC.5	900	127	1698	---	1115	300
NAC.6	514	200	1820	---	1163	337
NAC.7	612	102	1820	---	1163	337

Table 3.6 Mix Proportion for RAC and NAC of Phase III.

Mix ID	Cement 1b.	Fly Ash 1b.	Gravel 1b.	Recycled Conc. 1b	Sand 1b.	Water 1b.
RAC.7	612	102	---	1722	1030	286
RAC .8	612	102	---	1722	1030	286
RAC.9	612	102	---	1722	1030	286
NAC.8	612	102	1820	---	1030	286

Table 4.1 Test Program

Test Type	MIXES											
	RAC50%, RAC75% NAC50% RAC75%		NAC.1 RAC.1		NAC.2 RAC.2		NAC.3 RAC.3		NAC.4,NAC.5, NAC.6,NAC.7, NAC.8		RAC.4,RAC.5, RAC.6,RAC.7, RAC.8,RAC.9	
	Age (Day)	No. of Samples	Age (Day)	No. of Samples	Age (Day)	No. of Samples	Age (Day)	No. of Samples	Age (Day)	No. of Samples	Age (Day)	No. of Samples
Compressive Strength 6"× 12" Cylinder	7 28	3 EA 3 EA	7 28	3 EA 3 EA	7 28 90	3 EA 3 EA 3 EA	3 7 28	3 EA 3 EA 3 EA	7 28	3 EA 3 EA	7 28	3 EA 3 EA
Modulus of Rupture 3"×4"×18" Prism	7 28	3 EA 3 EA	7 28	3 EA 3 EA	7 28 90	3 EA 3 EA 3 EA	3 7 28	3 EA 3 EA 3 EA	28	3 EA	28	3 EA
Modulus of Elasticity 6"× 12" Cylinder					7 28 90	3 EA 3 EA 3 EA	3 7 28	3 EA 3 EA 3 EA	7 28	3 EA 3 EA	7 28	3 EA 3 EA
Tension (Split Cylinder) 6"× 12" Cylinder					7 28 90	3 EA 3 EA 3 EA	3 7 28	3 EA 3 EA 3 EA				
Freeze and Thaw 3" ×4" ×18" prism									Each 36 Cycles per week	3 EA	3 EA	3 EA

Table 5.1 Properties of Recycled and Natural Aggregates.

Source of Aggregates	Specific Gravity (O.D.)	Specific Gravity (S.S.D.)	Water Absorption, %
Natural Limestone	2.7	2.67	0.30
Pre-Cast Conc. Pipes	2.85	2.40	5.10
Rec. Conc. Cylinders			
f'_c : 5000 psi	2.70	2.55	4.70
f'_c : 4000 psi	2.80	2.54	4.95
f'_c : 3000 psi	2.75	2.55	4.41

Table 5.2 Slumps and Air Contents for RACs and NACs.

Mix ID	Slump (in)	Natural Air Content, %	Entrained Air Content, %
RAC 50%	9.5	---	---
RAC 75%	9.5	---	---
RAC.1	10	---	---
RAC.2	8	---	---
RAC.3	6	2.2	---
RAC.4	6.5	2.5	---
RAC.5	6	3.0	---
RAC.6	5	2.0	5
RAC.7	6	2.5	5.5
RAC.8	6	2.3	5.5
RAC.9	5.5	2.1	6
NAC.1	10	---	---
NAC.2	8	---	---
NAC.3	6	2.1	---
NAC.4	6	1.5	---
NAC.5	6	2.1	---
NAC.6	5.5	1.9	5
NAC.7	5	1.5	5.5

Table 5.3 Compressive Strength and Modulus of Rupture of NAC and RAC with R = 0.50.

Test Age, days	Compressive Strength (psi)		Modulus of Rupture (psi)	
	NAC	RAC	NAC	RAC
7	3784	4303	742	767
28	5523	5930	947	911

Table 5.4 Compressive Strength and Modulus of Rupture of NAC and RAC with R = 0.75.

Test Age, days	Compressive Strength (psi)		Modulus of Rupture (psi)	
	NAC	RAC	NAC	RAC
7	3784	4244	742	760
28	5523	5506	947	897

Table 5.5 Compressive Strength and Modulus of Rupture of NAC and RAC with R = 1.00, [Mix RAC.1 and NAC.1] .

Test Age, days	Compressive Strength (psi)		Modulus of Rupture (psi)	
	NAC	RAC	NAC	RAC
7	3784	4397	742	760
28	5523	5635	947	860

Table 5.6 Physical Properties of the RAC and the NAC. [RAC.2 and NAC.2]

Test Age	Modulus of Elast. (ksi)		Tensile Strength (psi)		Compressive Strength (psi)		Modulus of Rupture (psi)	
	NAC	RAC	NAC	RAC	NAC	RAC	NAC	RAC
7-Day	3263	2976	342	439	3219	3891	643	655
28-Day	4034	3378	445	469	4255	4858	808	797
90-Day	5733	4292	539	522	5482	5883	928	874

Table 5.7 Development of Physical Properties of the RAC and the NAC. [RAC.3 and NAC.3].

Test Age (days)	Compressive Strength (psi)		Modulus of Elasticity (ksi)		Modulus of Rupture (psi)		Splitting Tensile Strength (psi)	
	NAC	RAC	NAC	RAC	NAC	RAC	NAC	RAC
3	2830	3938	3587	3104	593	703	345	407
7	3513	4521	4098	3224	685	717	413	513
28	4657	5347	4438	3354	803	803	510	517

Table 5.8 Physical Properties of the RAC and the NAC, Case I.

Test Age (days)	Compressive Strength (psi)		Modulus of Elasticity (ksi)		Modulus of Rupture (psi)	
	NAC	RAC	NAC	RAC	NAC	RAC
7	3790	4383	4117	3224	---	---
28	5523	5636	4550	3354	950	860

Table 5.9 Relative Dynamic Modulus of Elasticity and Percentage of Weight Change of RAC and NAC, Case I.

No of cycles	Relative Dynamic Modulus of Elasticity				Percentage of Weight Change			
	RAC		NAC		RAC		NAC	
	Act.	Ave.	Act.	Ave.	Act.	Ave.	Act.	Ave.
36	90		36		0.25		0.55	
	91	89	33	33	0.12	0.23	0.20	0.52
	87		29		0.31		0.81	
72	54	45	---	---	0.48		1.10	
	49		---		0.48	0.46	0.23	0.69
	34		---		0.42		0.72	

Table 5.10 Physical Properties of RAC and NAC, Case II.

Test Age (Days)	Compressive Strength (psi)		Modulus of Elasticity (ksi)		Modulus of Rupture (psi)		Tensile Strength (psi)	
	NAC	RAC	NAC	RAC	NAC	RAC	NAC	RAC
7	6236	4728	5067	3967	884	753	530	522
28	7416	6413	5350	4208	1048	812	619	531

Table 5.11 Fundamental Transverse Frequencies and Relative Dynamic Modulus of Elasticity for NAC, Case II.

No. of Cycles	TRANS. FREQ. (HZ)				% RDM
	Specimen A	Specimen B	Specimen C	Average	
36	2440	2390	2395	2408	100
72	2425	2380	2385	2397	99
108	2425	2379	2395	2400	99
144	2425	2375	2395	2398	99
180	2424	2375	2385	2395	99
216	2425	2325	2385	2378	98
252	2425	2375	2385	2395	99
288	2425	2385	2385	2398	99
324	2450	2400	2405	2418	100

Table 5.12 Fundamental Transverse Frequencies and Relative Dynamic Modulus of Elasticity for RAC, Case II .

No. of Cycles	TRANS. FREQ. (HZ)				% RDM
	Specimen A	Specimen B	Specimen C	Average	
36	2220	2215	2225	2220	100
72	2215	2200	2205	2207	99
108	2215	2195	2195	2202	98
144	2205	2157	2180	2181	96
180	2175	1980	2092	2082	88
216	2040	1610	1950	1867	71
252	1780	---	1760	1180	28

Table 5.13 Weight Change Percentage for NAC, Case II.

No. of Cycles	Weight (1b)				Weight Change Percentage
	Specimen A	Specimen B	Specimen C	Average	
0	17.49	16.95	16.70	17.05	0
36	17.5	16.96	16.72	17.06	0.08
72	17.52	16.97	16.74	17.08	0.18
108	17.52	16.97	16.74	17.08	0.18
144	17.53	16.98	16.75	17.09	0.23
180	17.54	16.97	16.74	17.08	0.22
216	17.54	16.97	16.74	17.08	0.22
252	17.54	16.97	16.74	17.08	0.22
288	17.55	16.98	16.74	17.09	0.25
324	17.55	16.98	16.74	17.09	0.25

Table 5.14 Weight Change Percentage for RAC, Case II .

No. of Cycles	Weight (1b)				Weight Change Percentage
	Specimen A	Specimen B	Specimen C	Average	
0	16.30	16.56	16.53	16.46	0
36	16.32	16.58	16.55	16.48	0.12
72	16.34	16.61	16.57	16.51	0.26
108	16.35	16.63	16.58	16.52	0.34
144	16.36	16.65	16.6	16.54	0.45
180	16.39	16.69	16.62	16.57	0.63
216	16.42	16.73	16.64	16.60	0.81

Table 5.15 Development of Physical Properties of the RAC and the NAC, Case III.

Test Age, days	Compressive Strength (psi)		Modulus of Elasticity (ksi)		Modulus of Rupture (psi)	
	NAC	RAC	NAC	RAC	NAC	RAC
7	3678	3932	3750	3150	---	---
28	4952	5152	4200	3275	855	739

Table 5.16 Fundamental Transverse Frequencies and Relative Dynamic Modulus of Elasticity for NAC, Case III .

No. of Cycles	TRANS. FREQ. (HZ)				% RDM
	Specimen A	Specimen B	Specimen C	Average	
36	2195	2165	2195	2185	100
72	2195	2165	2195	2185	100
108	2195	2165	2190	2183	100
144	2195	2165	2175	2178	99
180	2185	2155	2130	2157	97
216	2170	2145	2125	2147	97
252	2185	2150	2120	2152	97
288	2168	2146	2103	2139	96
324	2160	2130	2060	2117	94

Table 5.17 Fundamental Transverse Frequencies and Relative Dynamic Modulus of Elasticity for RAC, Case III .

No. of Cycles	TRANS. FREQ. (HZ)				% RDM
	Specimen A	Specimen B	Specimen C	Average	
36	2010	1985	1995	1997	100
72	2010	1985	1995	1997	100
108	2010	1985	1995	1997	100
144	2010	1985	2000	1998	100
180	1980	1955	1990	1975	98
216	1940	1920	1980	1947	95
252	1915	1890	1970	1925	93
288	1870	1855	1944	1890	90
324	1825	1790	1890	1835	84

Table 5.18 Weight Change Percentage for NAC, Case III .

No. of Cycles	Weight (1b)				Weight Change Percentage
	Specimen A	Specimen B	Specimen C	Average	
0	16.63	16.54	16.79	16.65	0
36	16.64	16.56	16.81	16.67	0.10
72	16.67	16.6	16.85	16.71	0.32
108	16.68	16.62	16.87	16.72	0.42
144	16.69	16.62	16.86	16.72	0.42
180	16.69	16.62	16.85	16.72	0.40
216	16.68	16.62	16.85	16.72	0.38
252	16.68	16.6	16.86	16.71	0.36
288	16.65	16.59	16.84	16.69	0.24
324	16.64	16.55	16.84	16.68	0.14

Table 5.19 Weight Change Percentage for RAC, Case III .

No. of Cycles	Weight (1b)				Weight Change Percentage
	Specimen A	Specimen B	Specimen C	Average	
0	15.88	15.61	15.79	15.76	0
36	15.89	15.62	15.8	15.77	0.06
72	15.9	15.62	15.81	15.78	0.11
108	15.92	15.65	15.83	15.79	0.25
144	15.94	15.66	15.84	15.81	0.34
180	15.95	15.68	15.84	15.82	0.40
216	15.96	15.69	15.84	15.83	0.44
252	15.97	15.7	15.84	15.84	0.49
288	15.98	15.7	15.84	15.84	0.51
324	15.97	15.71	15.84	15.84	0.51

Table 5.20 Development of Physical Properties of the RAC and the NAC, Case IV.

Test Age, days	Compressive Strength (psi)		Modulus of Elasticity (ksi)		Modulus of Rupture (psi)	
	NAC	RAC	NAC	RAC	NAC	RAC
7	2914	3453	3558	2808	---	---
28	4233	4339	4026	3150	813	728

Table 5.21 Fundamental Transverse Frequencies and Relative Dynamic Modulus of Elasticity for NAC, Case IV .

No. of Cycles	TRANS. FREQ. (HZ)				% RDM
	Specimen A	Specimen B	Specimen C	Average	
36	2240	2270	2290	2267	100
72	2240	2265	2280	2262	100
108	2240	2270	2290	2267	100
144	2240	2270	2290	2267	100
180	2240	2270	2290	2267	100
216	2240	2270	2290	2267	100
252	2240	2270	2290	2267	100
288	2240	2270	2290	2267	100
324	2240	2270	2290	2267	100

Table 5.22 Fundamental Transverse Frequencies and Relative Dynamic Modulus of Elasticity for RAC, Case IV .

No. of Cycles	TRANS. FREQ. (HZ)				% RDM
	Specimen A	Specimen B	Specimen C	Average	
36	2190	2150	2130	2157	100
72	2185	2145	2120	2150	99
108	2195	2145	2120	2153	100
144	2190	2150	2130	2157	100
180	2190	2150	2130	2157	100
216	2190	2150	2130	2157	100
252	2190	2150	2130	2157	100
288	2190	2150	2130	2157	100
324	2190	2150	2130	2157	100

Table 5.23 Weight Change Percentage for NAC, Case IV .

No. of Cycles	Weight (1b)				Weight Change Percentage
	Specimen A	Specimen B	Specimen C	Average	
0	16.83	17.33	17.56	17.24	0.00
36	16.84	17.33	17.57	17.25	0.04
72	16.85	17.34	17.57	17.25	0.08
108	16.85	17.35	17.57	17.26	0.10
144	16.85	17.36	17.58	17.26	0.14
180	16.86	17.37	17.58	17.27	0.17
216	16.86	17.37	17.58	17.27	0.17
252	16.86	17.38	17.58	17.27	0.19
288	16.86	17.38	17.58	17.27	0.19
324	16.86	17.39	17.58	17.28	0.21

Table 5.24 Weight Change Percentage for RAC, Case IV .

No. of Cycles	Weight (1b)				Weight Change Percentage
	Specimen A	Specimen B	Specimen C	Average	
0	16.8	16.71	16.65	16.72	0.00
36	16.81	16.72	16.66	16.73	0.06
72	16.81	16.72	16.67	16.73	0.08
108	16.82	16.72	16.67	16.74	0.10
144	16.83	16.73	16.67	16.74	0.14
180	16.84	16.73	16.67	16.75	0.16
216	16.84	16.74	16.68	16.75	0.20
252	16.84	16.74	16.69	16.76	0.22
288	16.85	16.74	16.69	16.76	0.24
324	16.85	16.75	16.69	16.76	0.26

Table 5.25 Physical Properties of Recycled Aggregate Concretes and Natural Aggregate Concrete for Phase III.

Mix ID	Age (Days)	Compressive Strength (psi)	Modulus of Elasticity (Ksi)	Modulus of Rupture (psi)
NAC.8	7	4930	4825	---
	28	5862	5875	1050
RAC.8	7	4293	3083	---
	28	5031	3690	900
RAC.9	7	4636	3960	---
	28	5390	4375	917
RAC.10	7	5284	4475	---
	28	6086	4583	914

Table 5.26 Fundamental Transverse Frequencies and Relative Dynamic Modulus of Elasticity for NAC, Phase III

No. of Cycles	TRANS. FREQ. (HZ)				% RDM
	Specimen A	Specimen B	Specimen C	Average	
36	2275	2315	2280	2290	100
72	2265	2300	2275	2280	99
108	2270	2310	2285	2288	100
144	2282	2321	2295	2299	101
180	2290	2325	2300	2305	101
216	2289	2322	2297	2303	101
252	2290	2325	2300	2305	101
288	2290	2325	2300	2305	101
324	2290	2325	2300	2305	101

Table 5.27 Fundamental Transverse Frequencies and Relative Dynamic Modulus of Elasticity for RAC.7, Phase III.

No. of Cycles	TRANS. FREQ. (HZ) Specimen A	Specimen B	Specimen C	Average	% RDM
36	1978	1949	---	1964	100
72	1978	1950	---	1964	100
108	1987	1962	---	1975	101
144	1990	1970	---	1980	102
180	1990	1970	---	1980	102
216	1990	1970	---	1980	102
252	2000	1970	---	1985	102
288	2000	1970	---	1985	102
324	2000	1970	---	1985	102

Table 5.28 Fundamental Transverse Frequencies and Relative Dynamic Modulus of Elasticity for RAC.8, Phase III.

No. of Cycles	TRANS. FREQ. (HZ)				% RDM
	Specimen A	Specimen B	Specimen C	Average	
36	2205	2225	2155	2195	100
72	2190	2220	2145	2185	99
108	2200	2225	2155	2193	100
144	2204	2235	2158	2199	100
180	2210	2235	2160	2202	101
216	2210	2235	2160	2202	101
252	2210	2235	2160	2202	101
288	2210	2235	2160	2202	101
324	2210	2235	2160	2202	101

Table 5.29 Fundamental Transverse Frequencies and Relative Dynamic Modulus of Elasticity for RAC.9, Phase III.

No. of Cycles	TRANS. FREQ. (HZ) Specimen A	Specimen B	Specimen C	Average	% RDM
36	2158	2115	2177	2150	100
72	2158	2122	2175	2152	100
108	2160	2125	2182	2156	101
144	2165	2130	2185	2160	101
180	2165	2130	2185	2160	101
216	2165	2130	2190	2162	101
252	2165	2130	2190	2162	101
288	2165	2130	2190	2162	101
324	2165	2130	2190	2162	101

Table 5.30 Weight Change Percentage for NAC.8, Phase III.

No. of Cycles	Weight (1b)				Weight Change Percentage
	Specimen A	Specimen B	Specimen C	Average	
0	16.93	17.42	17.14	17.16	0.00
36	16.94	17.43	17.16	17.18	0.08
72	16.95	17.45	17.17	17.19	0.16
108	16.96	17.46	17.18	17.20	0.21
144	16.97	17.46	17.18	17.20	0.23
180	16.97	17.46	17.18	17.20	0.23
216	16.98	17.46	17.19	17.21	0.27
252	16.97	17.47	17.18	17.21	0.25
288	16.98	17.47	17.2	17.22	0.31
324	16.94	17.44	17.2	17.19	0.17

Table 5.31 Weight Change Percentage for RAC.7, Phase III.

No. of Cycles	Weight (1b)			
	Specimen A	Specimen B	Average	Weight Change Percentage
0	15.82	15.54	15.68	0.00
36	15.84	15.57	15.71	0.16
72	15.86	15.58	15.72	0.26
108	15.86	15.59	15.73	0.29
144	15.85	15.58	15.72	0.22
180	15.85	15.58	15.72	0.22
216	15.83	15.57	15.70	0.13
252	15.82	15.55	15.69	0.03
288	15.72	15.5	15.61	-0.45
324	15.54	15.33	15.44	-1.56

Table 5.32 Weight Change Percentage for RAC.8, Phase III.

No. of Cycles	Weight (1b)				Weight Change Percentage
	Specimen A	Specimen B	Specimen C	Average	
0	16.87	17.04	16.56	16.82	0.00
36	16.88	17.05	16.58	16.84	0.08
72	16.9	17.07	16.59	16.85	0.18
108	16.91	17.07	16.59	16.86	0.20
144	16.91	17.08	16.59	16.86	0.22
180	16.92	17.08	16.6	16.87	0.26
216	16.91	17.09	16.6	16.87	0.26
252	16.91	17.08	16.6	16.86	0.24
288	16.92	17.09	16.6	16.87	0.28
324	16.93	17.09	16.6	16.87	0.30

Table 5.33 Weight Change Percentage for RAC.9, Phase III.

No. of Cycles	Weight (1b)				Weight Change Percentage
	Specimen A	Specimen B	Specimen C	Average	
0	16.9	16.63	16.6	16.71	0.00
36	16.91	16.64	16.62	16.72	0.08
72	16.91	16.65	16.63	16.73	0.12
108	16.91	16.64	16.64	16.73	0.12
144	16.91	16.67	16.64	16.74	0.18
180	16.92	16.65	16.63	16.73	0.14
216	16.92	16.65	16.64	16.74	0.16
252	16.92	16.66	16.64	16.74	0.18
288	16.91	16.65	16.64	16.73	0.14
324	16.88	16.63	16.62	16.71	0.00

Table 5.34 Comparison of Prediction Equations for the 28-Day Compressive Strength of NAC.

Actual		Hool and Pulver		Oluokun		Proposed	
S_7 (psi)	S_{28} (psi)	S_{28} (psi)	Hool/ Actual	S_{28} (psi)	Oluokun/ Actual	S_{28} (psi)	Proposed/ Actual
3784	5523	5629	1.019	5215	0.944	5045	0.914
3219	4255	4921	1.157	4602	1.081	4475	1.052
3513	4657	5291	1.136	4924	1.057	4775	1.025
6236	7416	8605	1.160	7677	1.035	7309	0.986
3678	4952	5497	1.110	5102	1.030	4940	0.998
2914	4233	4533	1.071	4260	1.006	4156	0.982
4930	5862	7036	1.200	6400	1.092	6140	1.047
Avg.			1.122		1.035		1.000
COV, %			5.4		4.8		4.8

Table 5.35 Comparison of Prediction Equations for the 28-Day Compressive Strength of RAC.

Actual		Hool and Pulver		Oluokun		Proposed	
S_7 (psi)	S_{28} (psi)	S_{28} (psi)	Hool/ Actual	S_{28} (psi)	Oluokun/ Actual	S_{28} (psi)	Prop./ Actual
4397	5635	6386	1.133	5858	1.040	5418	0.961
3891	4858	5762	1.186	5329	1.097	4948	1.019
4521	5347	6538	1.223	5985	1.119	5531	1.034
4728	6413	6791	1.059	6196	0.966	5718	0.892
3932	5152	5813	1.128	5372	1.043	4987	0.968
3453	4339	5216	1.202	4858	1.120	4528	1.044
4293	5031	6259	1.244	5750	1.143	5322	1.058
4636	5390	6679	1.239	6103	1.132	5635	1.045
5284	6089	7465	1.226	6753	1.109	6209	1.020

Avg.	1.182		1.085		1.005
COV, %	5.3		5.3		5.4

Table 5.36 Comparison of Prediction Equations for the Modulus of Rupture of NAC.

Actual		ACI		Proposed	
f_r (psi)	f_c (psi)	f_r (psi)	ACI/ Actual	f_r (psi)	Proposed/ Actual
947	5523	557	0.589	930	0.982
808	4255	489	0.605	816	1.010
803	4657	512	0.637	854	1.063
1048	7416	646	0.616	1077	1.028
855	4952	528	0.617	880	1.030
1050	5862	574	0.547	958	0.912

Avg.	0.602		1.004
COV, %	4.8		4.8

Table 5.37 Comparison of Prediction Equations for the Modulus of Rupture of RAC.

Actual		ACI		Proposed	
f_r (psi)	f_c (psi)	f_r (psi)	ACI/ Actual	f_r (psi)	Proposed/ Actual
860	5635	563	0.655	854	0.992
797	4858	523	0.656	792	0.994
803	5347	548	0.683	831	1.035
812	6413	601	0.740	911	1.121
739	5152	538	0.728	816	1.104
900	5031	532	0.591	806	0.896
917	5390	551	0.600	835	0.910
914	6089	585	0.640	887	0.971

Avg.	0.662		1.003
COV, %	7.6		7.6

Table 5.38 Comparison of Prediction Equations for the Splitting Tensile Strength of NAC.

Actual		ACI		Proposed	
f_c (psi)	f_t (psi)	f_t (psi)	ACI/ Actual	f_t (psi)	Proposed/ Actual
4255	445	483	1.085	485	1.091
4657	510	505	0.990	508	0.996
7416	619	637	1.029	641	1.035
6873	650	613	0.944	617	0.949
6626	625	602	0.964	606	0.969
Avg.			1.002		1.008
COV, %			5.6		5.6

Table 5.39 Comparison of Prediction Equations for the Splitting Tensile Strength of RAC.

Actual		ACI		Proposed	
f_c (psi)	f_t (psi)	f_t (psi)	ACI/ Actual	f_t (psi)	Proposed/ Actual
4858	469	516	1.100	496	1.058
5347	517	541	1.047	521	1.007
6413	531	593	1.116	570	1.074
6838	592	612	1.034	589	0.995
6484	637	596	0.935	573	0.900

Avg.	1.046		1.007
COV, %	6.8		6.8

Table 5.40 Prediction of Splitting Tensile Strength for the NAC.

Actual		Proposed	
f_r (psi)	f_t (psi)	f_t (psi)	Proposed /Actual
808	445	501	1.126
803	510	498	0.976
1048	619	650	1.050
998	650	619	0.952
952	625	590	0.944

Avg.	1.010
COV, %	7.6

Table 5.41 Prediction of Splitting Tensile Strength for the RAC.

Actual		Proposed	
f_r (psi)	f_t (psi)	f_t (psi)	Proposed/Actual
797	469	526	1.122
803	517	530	1.025
812	531	536	1.009
876	592	578	0.977
878	637	579	0.910

Avg.	1.008
COV, %	7.7

Table 5.42 Comparison of Measured Modulus of Elasticity with ACI Prediction for the NAC.

Actual		ACI Exact		ACI Approximate	
f'_c (ps)	E (ks)	Ec (ksi)	Exact/ Actual	E (ksi)	Approx./ Actual
3219	3263	3509	1.075	3234	0.991
4255	4034	4034	1.000	3718	0.922
5482	5733	4579	0.799	4220	0.736
2830	3587	3290	0.917	3032	0.845
3513	4098	3665	0.894	3378	0.824
4657	4438	4220	0.951	3890	0.876
3790	4117	3807	0.925	3509	0.852
5523	4550	4596	1.010	4236	0.931
6235	5067	4883	0.964	4501	0.888
7416	5350	5326	0.995	4909	0.917
3678	3750	3750	1.000	3457	0.922
4952	4200	4352	1.036	4011	0.955
4930	4825	4342	0.900	4002	0.829
5862	5875	4735	0.806	4364	0.743

Avg.	0.948		0.874
COV, %	8.5		8.5

Table 5.43 Comparison of Measured Modulus of Elasticity with ACI Prediction for the RAC.

Actual		ACI Approximate		Proposed	
f'_c (psi)	E (ksi)	E (ksi)	Approx./Actual	E (ksi)	Proposed/Actual
3891	2976	3556	1.195	3224	1.083
4858	3378	3973	1.176	3602	1.066
5883	4292	4372	1.019	3964	0.924
3938	3104	3577	1.152	3243	1.045
4521	3224	3833	1.189	3475	1.078
5347	3335	4168	1.250	3779	1.133
4383	3224	3774	1.170	3422	1.061
5635	3354	4279	1.276	3880	1.157
4728	3967	3919	0.988	3554	0.896
6413	4208	4565	1.085	4139	0.984
3932	3150	3574	1.135	3241	1.029
5152	3275	4091	1.249	3710	1.133
4293	3083	3735	1.211	3386	1.098
4636	3960	3881	0.980	3519	0.889
5390	4375	4185	0.957	3794	0.867
5284	4475	4143	0.926	3757	0.840
6086	4583	4447	0.970	4032	0.880
5031	3690	4043	1.096	3666	0.993
3453	2808	3349	1.193	3037	1.082
4339	3150	3755	1.192	3404	1.081

Avg.	1.120		1.016
COV, %	10.4		9.8

APPENDIX B
FIGURES

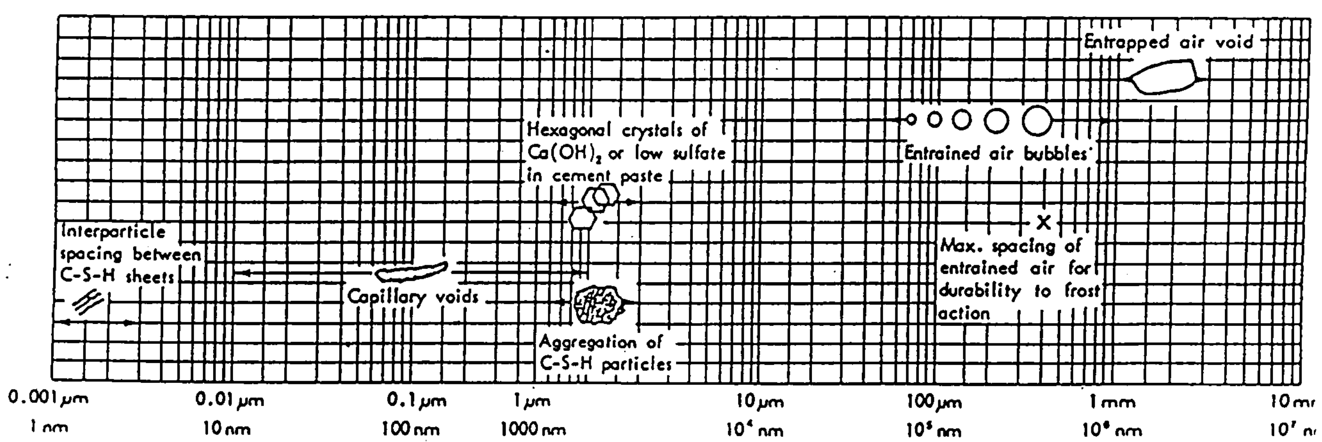

Figure 2.1 Dimensional range of solids and pores in a hydrated paste

(Mehta, 1986).

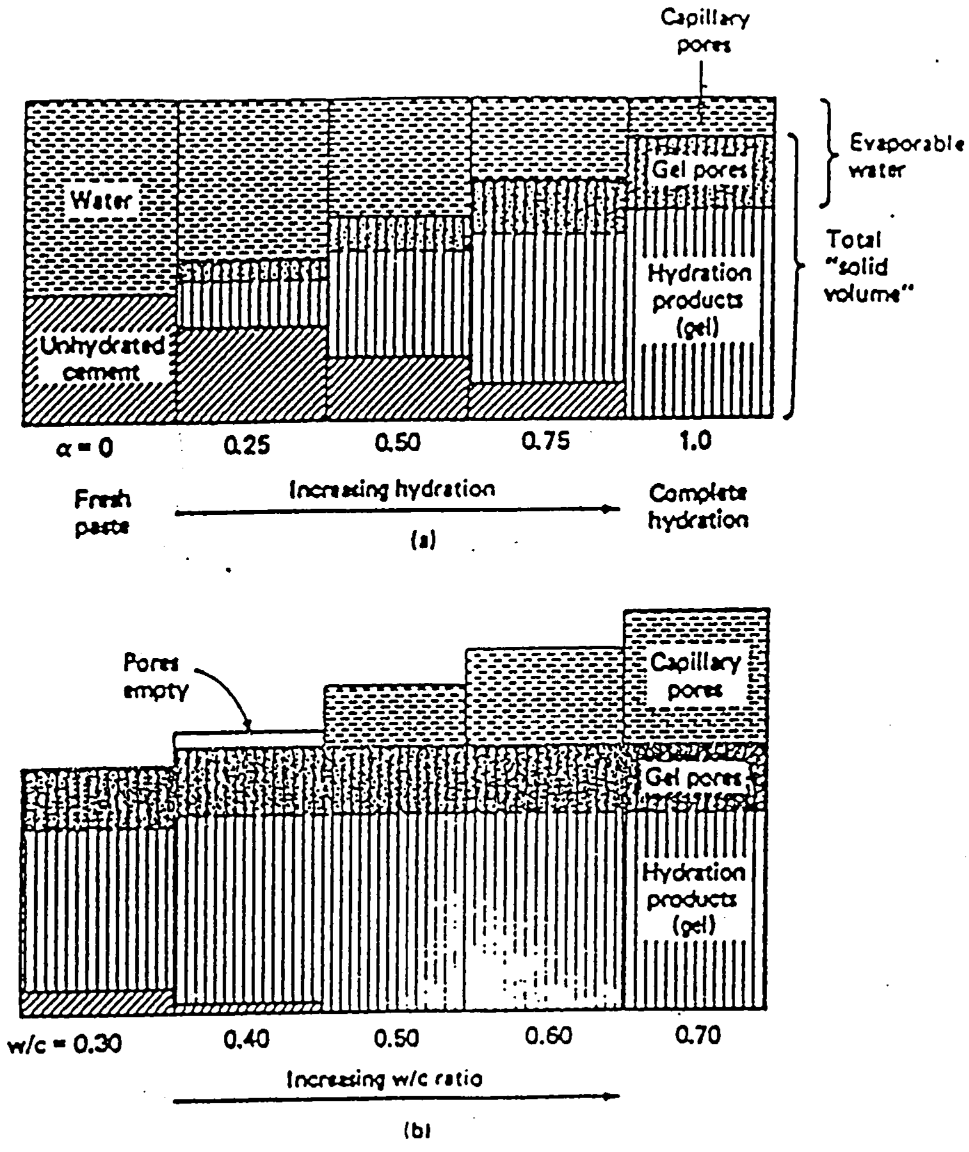

Figure 2.2 Porosity Relationships in Concrete (Mindess, 1981).

a) Hydration percentage at constant w/c = 0.51

b) Changing w/c (hydration ratio = 1.0)

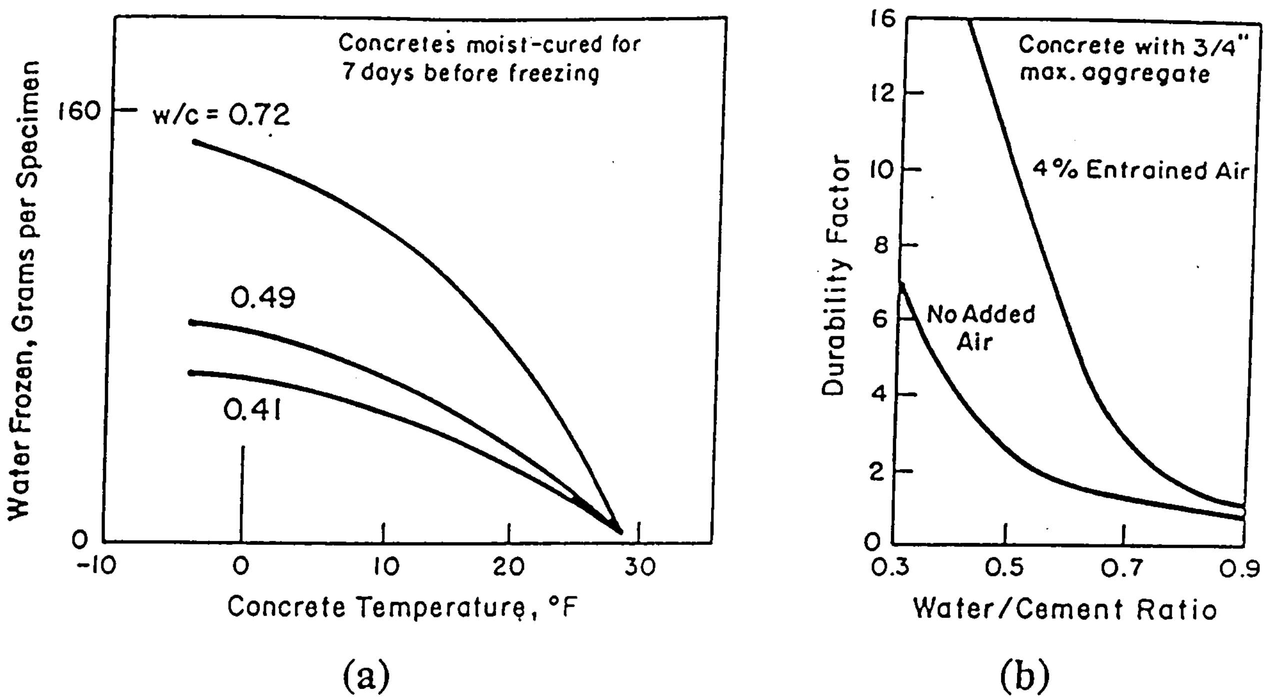

Figure 2.3 Influence of water cement-ratio and air content on the freeze-thaw durability of concrete. (Verbeck and Klieger, 1958).

a) The amount of freezable water in concrete, with a given water-cement ratio, increases with decrease in temperature.

b) High durability shown as combination of low water-cement ratio and increase in air-entrainment.

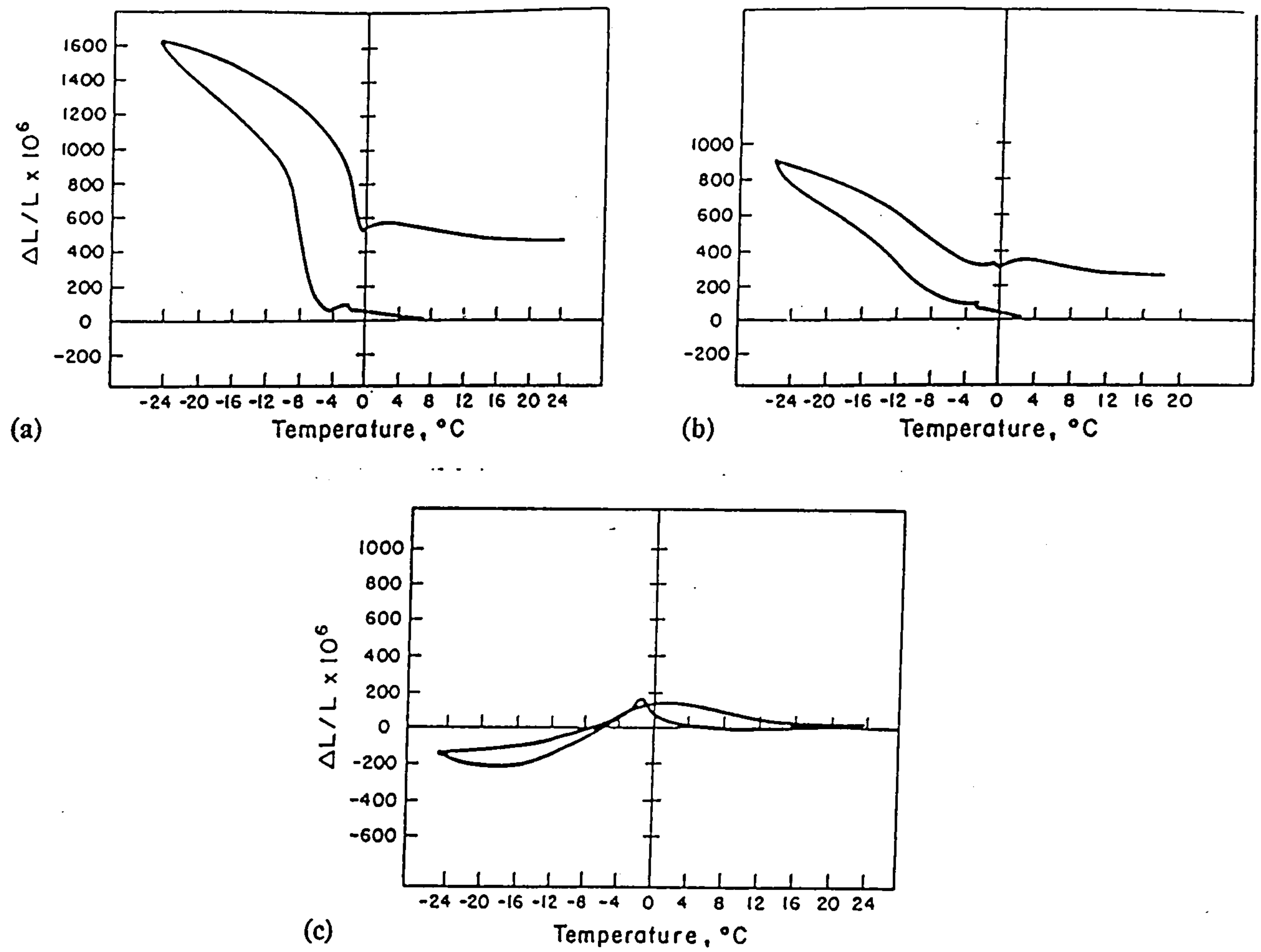

Figure 2.4 Response of Saturated Cement Paste to Freezing and Thawing, Both with and without Entrained Air. (Powers, 1958).

a) Specimen without air-entrainment,

b) Specimen with 2% air-entrainment, and

c) Specimen with 10% air-entrainment.

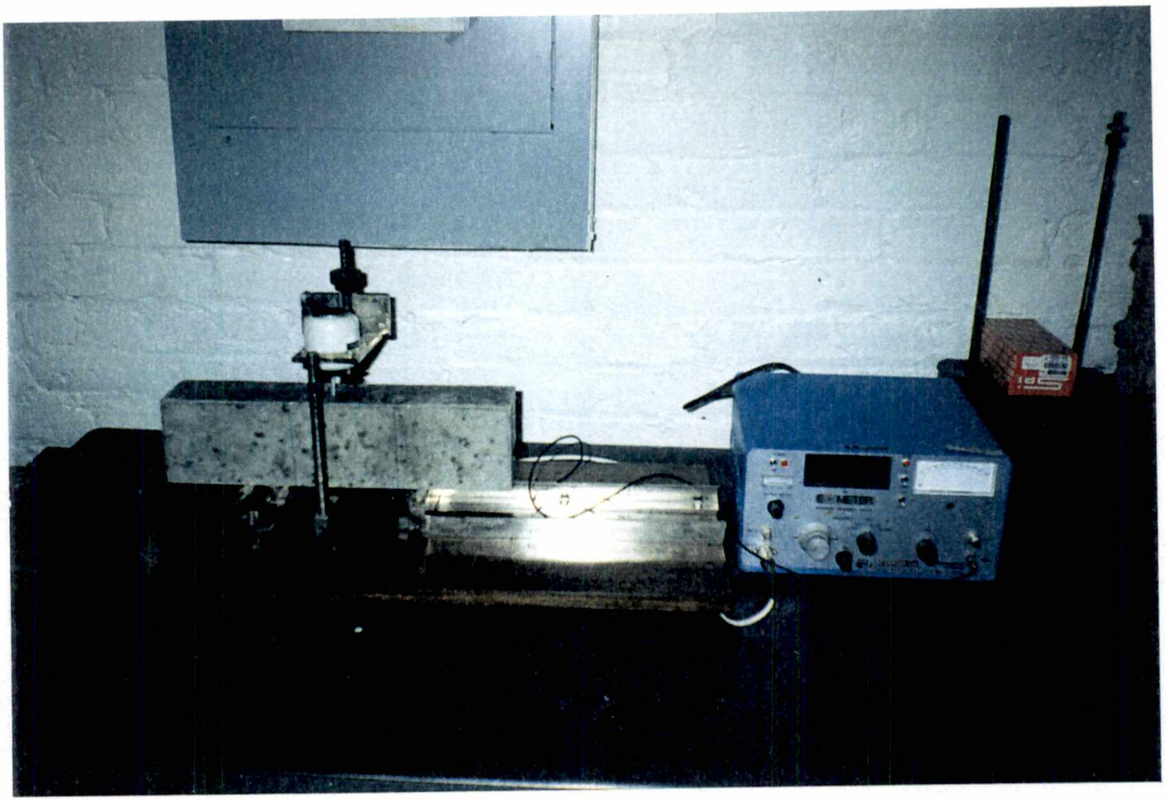

Fig. 4.1 Fundamental Frequency Set-up Test.

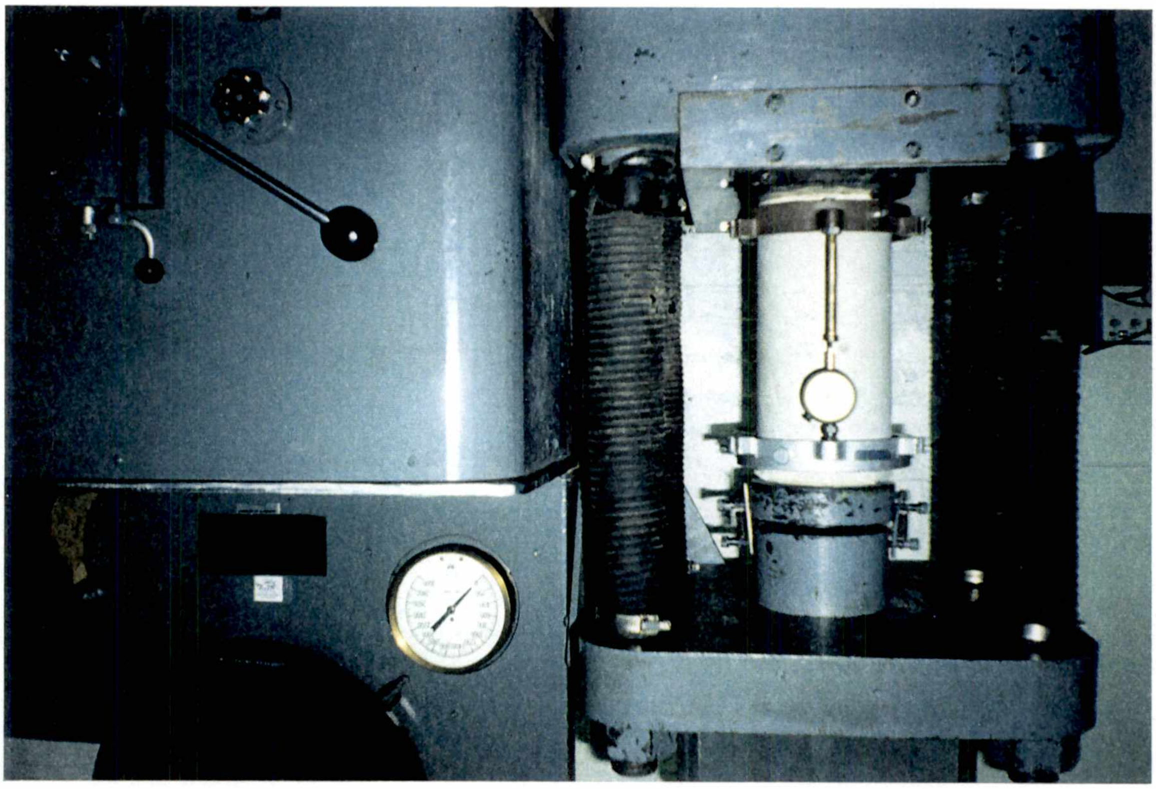

Figure 4.2 Test Set-up for Measuring the Modulus of Elasticity and Compressive Strength of Cylindrical Concrete Specimens.

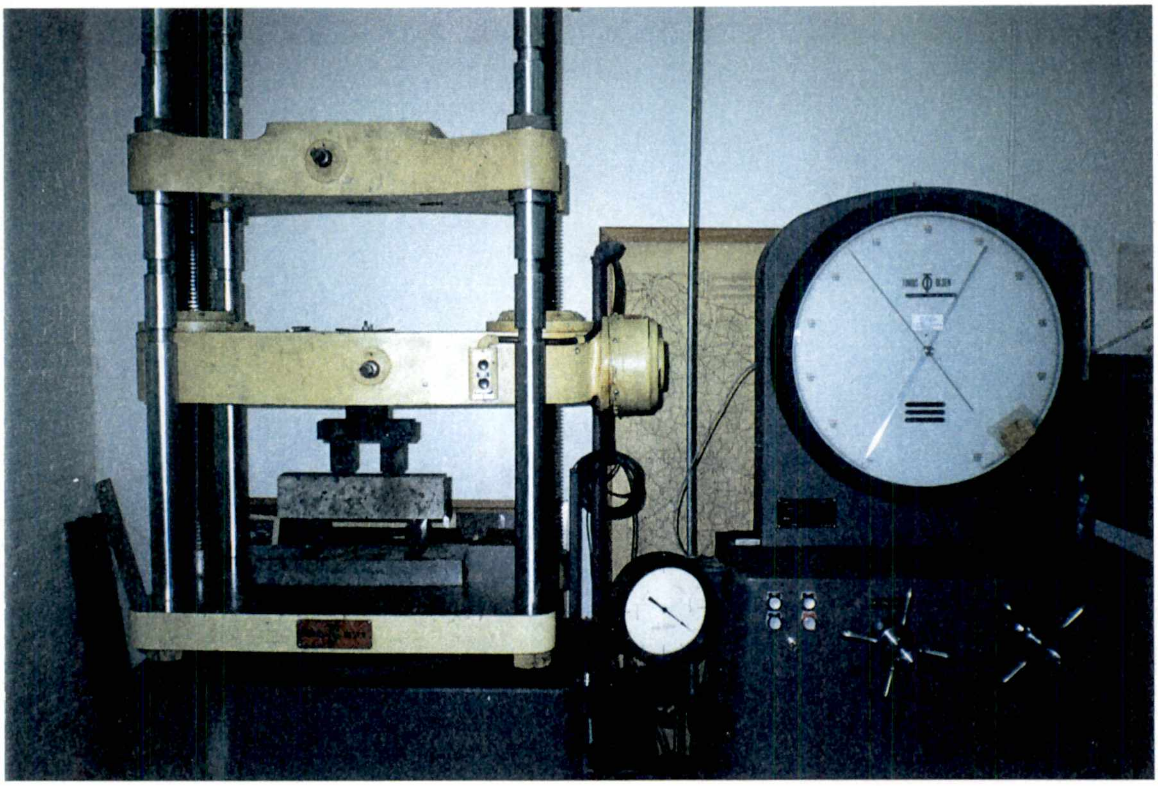

Fig. 4.3 Set-up for Flexural Test of Concrete by Third-Point Loading Method.

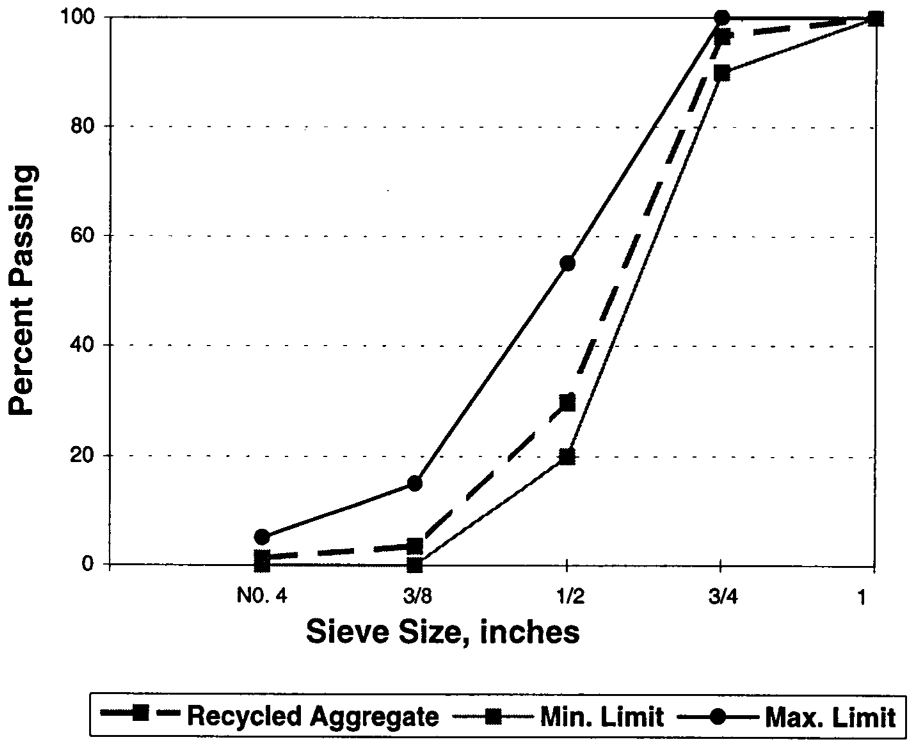

Figure 5.1 Gradation Chart for Typical Recycled Aggregate with ASTM C33 Limits for a 3/4 inch Max. Size Coarse Aggregate.

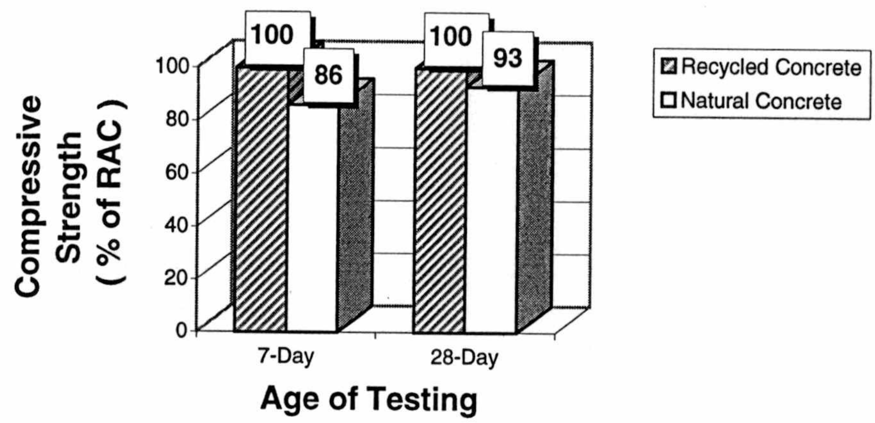

Figure 5.2 Relative Compressive Strength of Recycled and Natural Concretes for R = 0.50

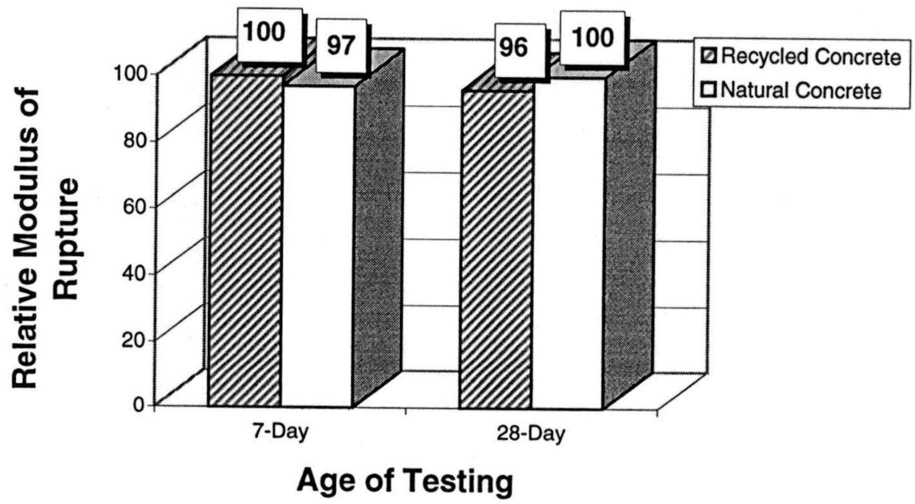

Figure 5.3 Relative Modulus of Rupture of Recycled and Natural Concretes for R = 0.50

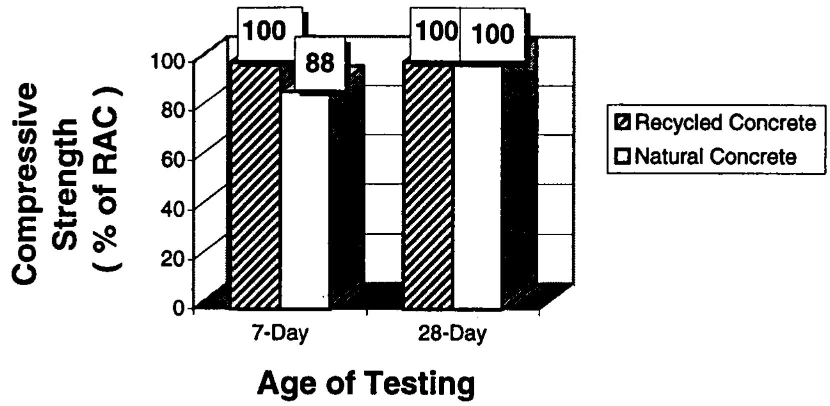

Figure 5.4 Relative Compressive Strength of Recycled and Natural Concretes for R = 0.75

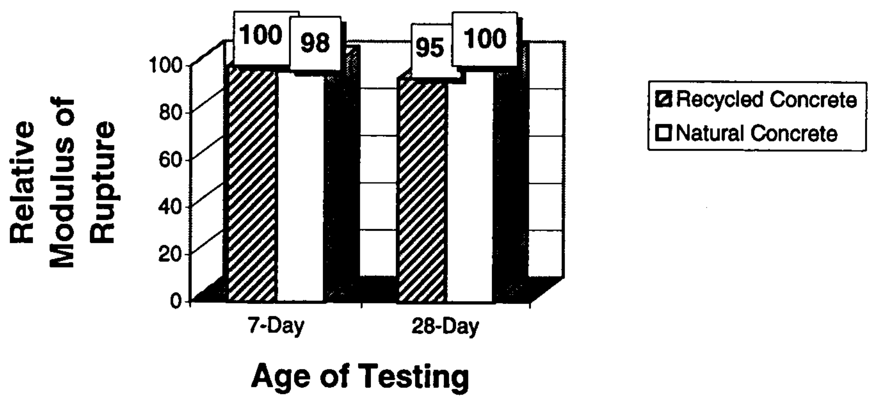

Figure 5.5 Relative Modulus of Rupture of Recycled and Natural Concretes for R = 0.75

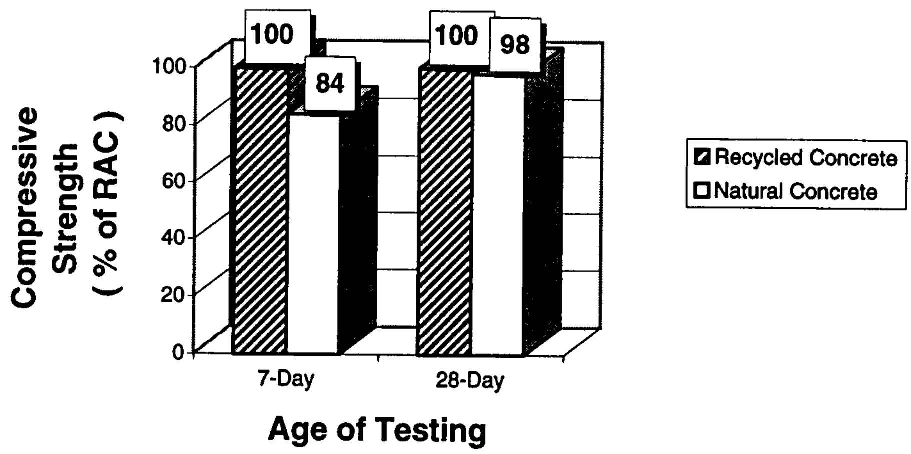

Figure 5.6 Relative Compressive Strength of Recycled and Natural Concretes for R = 1.00

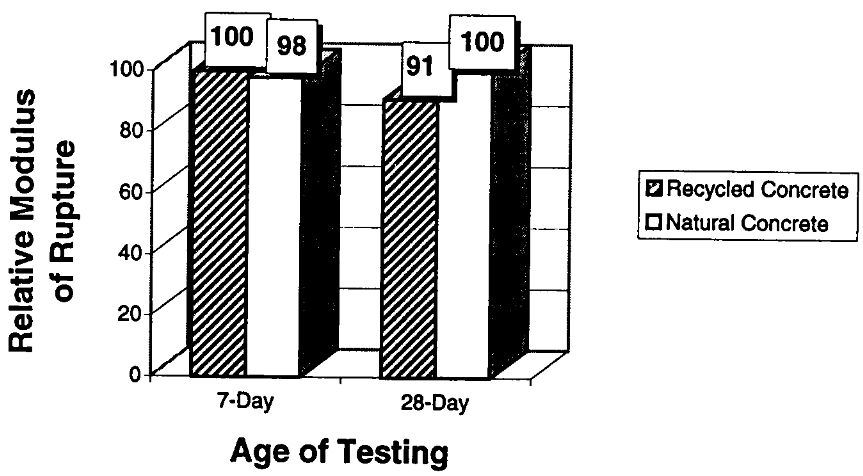

Figure 5.7 Relative Modulus of Rupture of Recycled and Natrural Concretes for R = 1.00

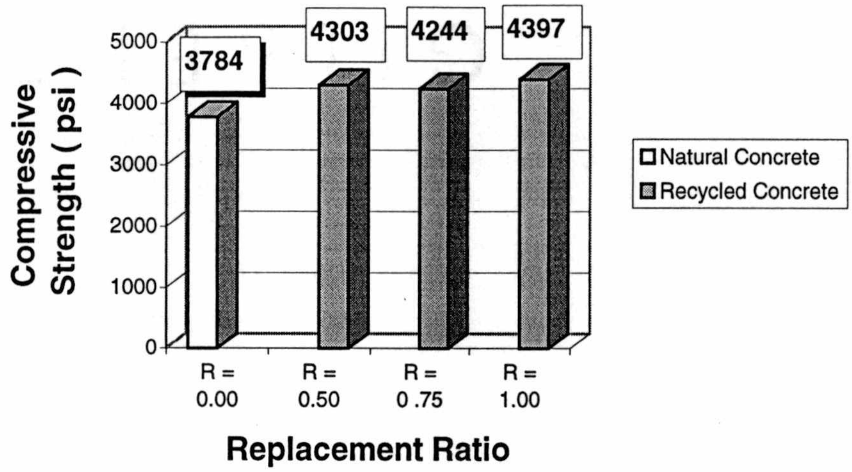

Figure 5.8 Summary of 7-Day Compressive Strength with Different Replacement Ratios.

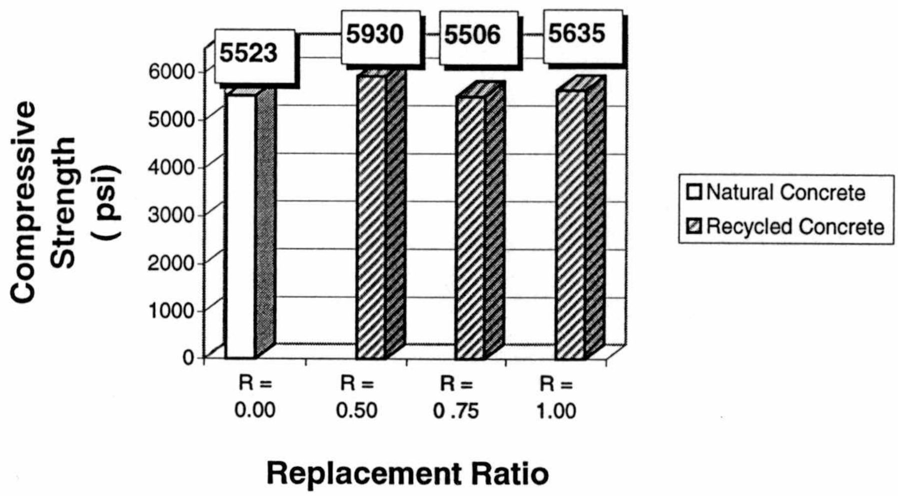

Figure 5.9 Summary of 28-Day Compressive Strength with Different Replacement Ratios.

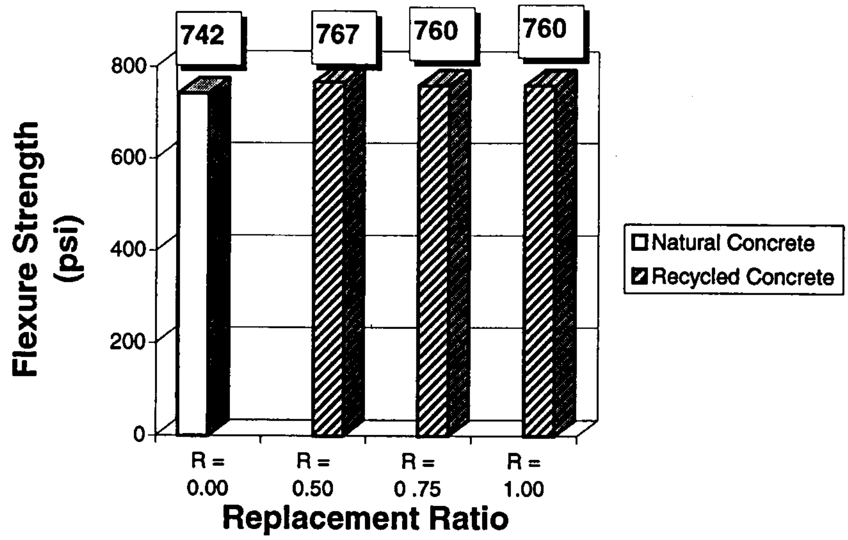

Figure 5.10 Summary of 7-Day Modulus of Rupture with Different Replacement Ratios.

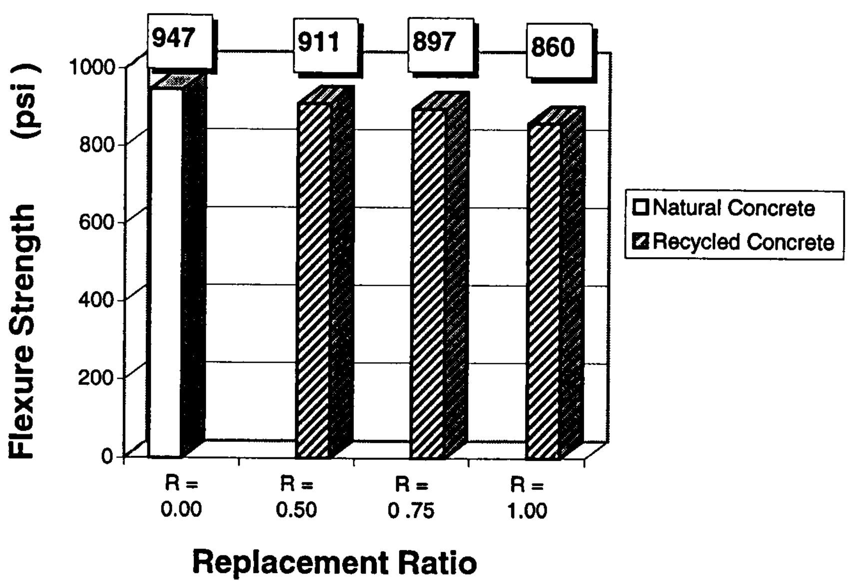

Figure 5.11 Summary of 28-Day Modulus of Rupture with Different Replacement Ratios.

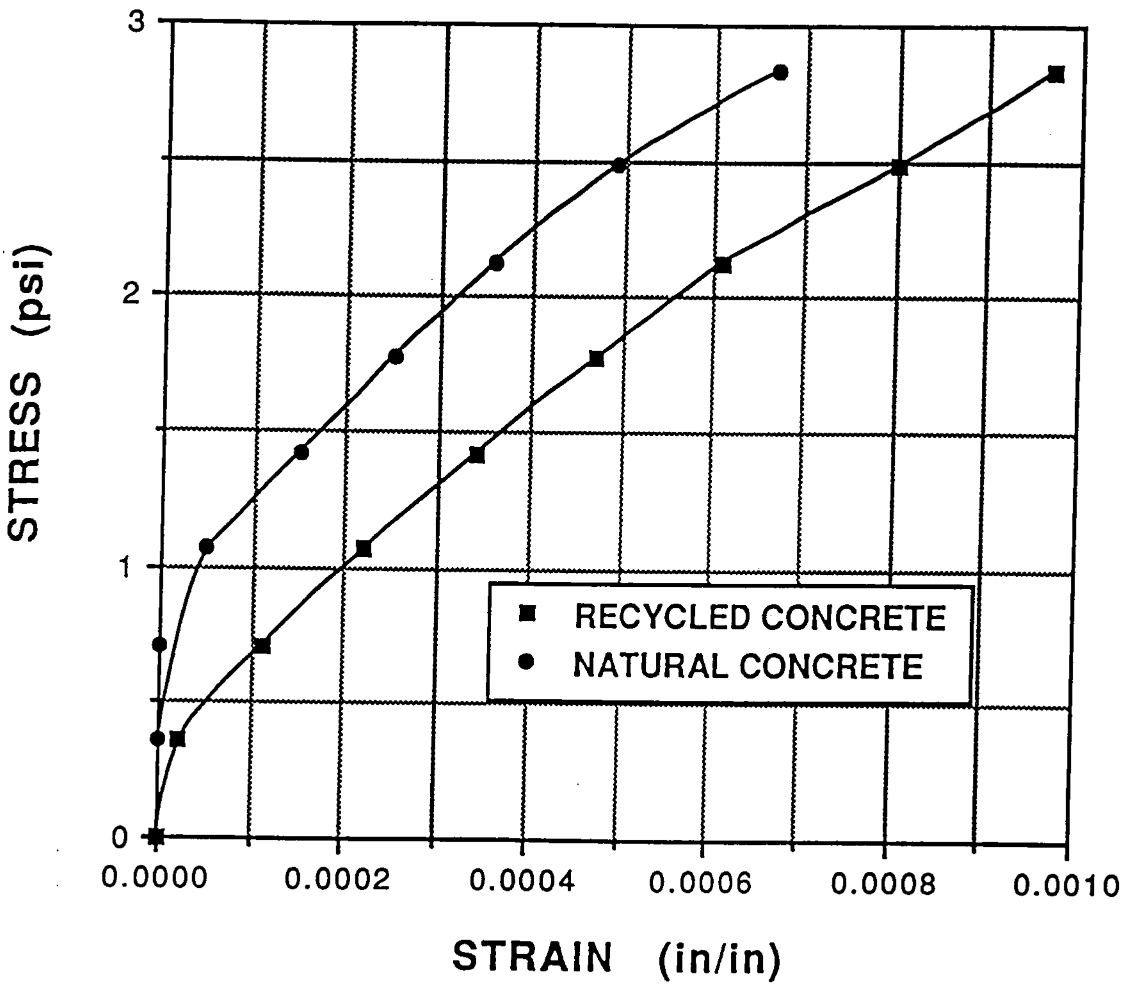

Figure 5.11 Stress-Strain Curves for the RAC and NAC.

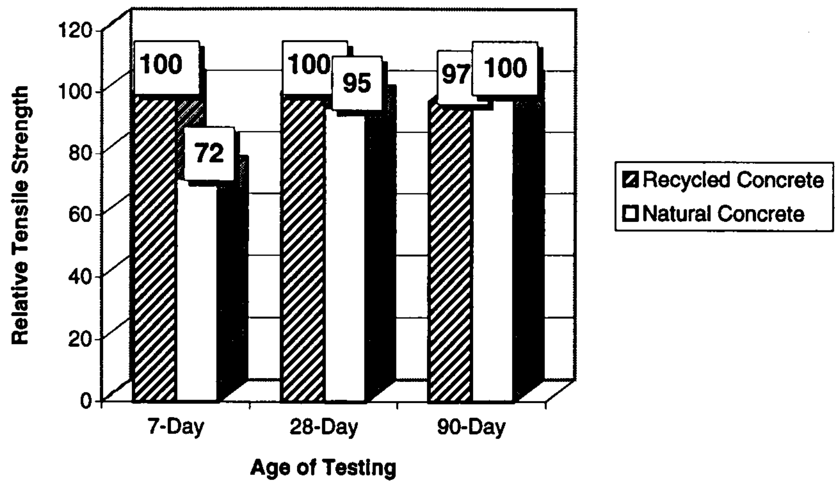

Figure5.13 Relative Tensile Strength of Recycled and Natural Concretes for R = 1.0 .

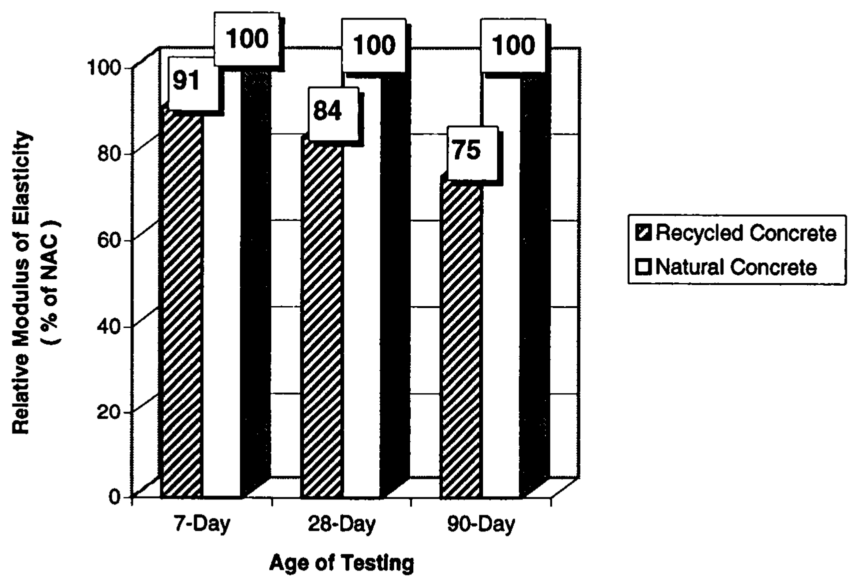

Figure 5.14 Relative Modulus of Elasticity of Recycled and Natural Concretes for R = 1.0

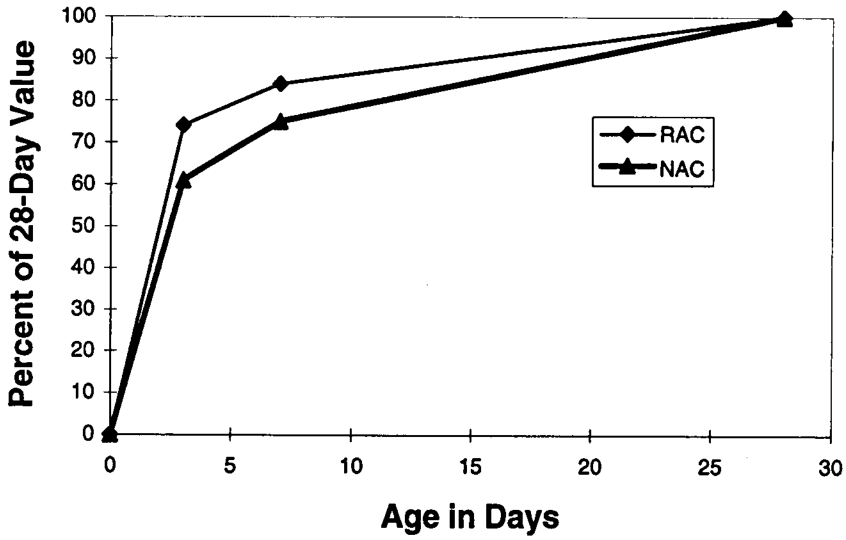

Figure 5.15 Compressive Strength Development of RAC and NAC Versus Age.

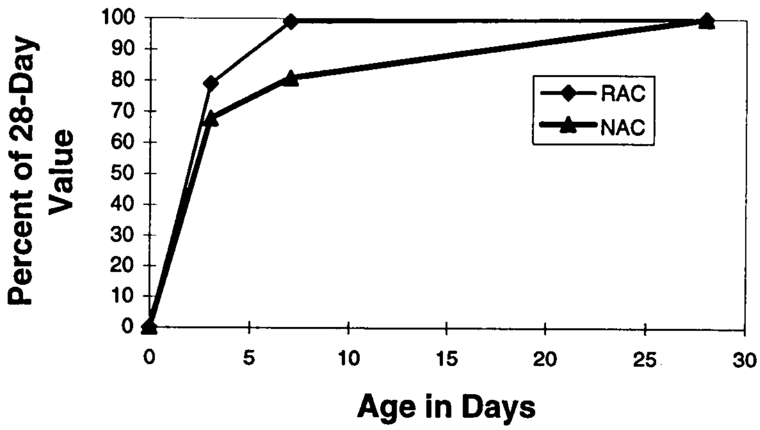

Figure 5.16 Tensile Strength Development of RAC and NAC Versus Age.

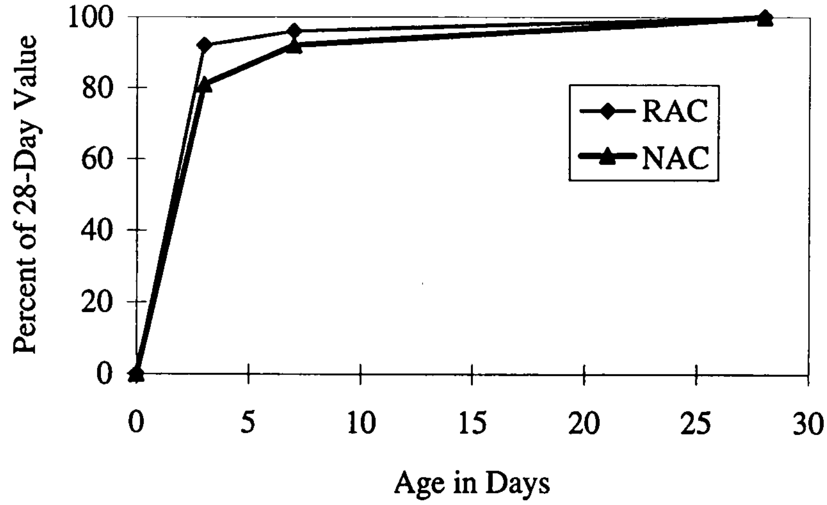

Figure 5.17 Development of Modulus of Elasticity of RAC and NAC Versus Age.

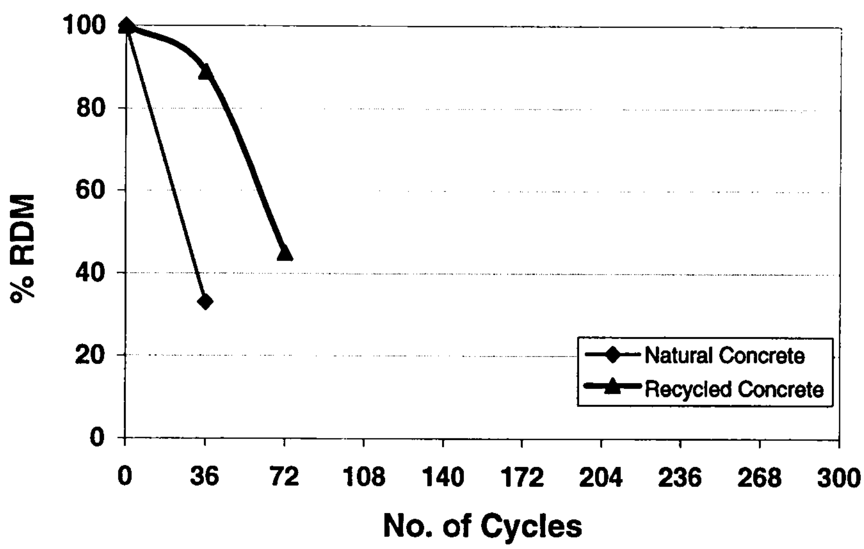

Figure 5.18 Relative Dynamic Modulus of Elasticity for Case I.

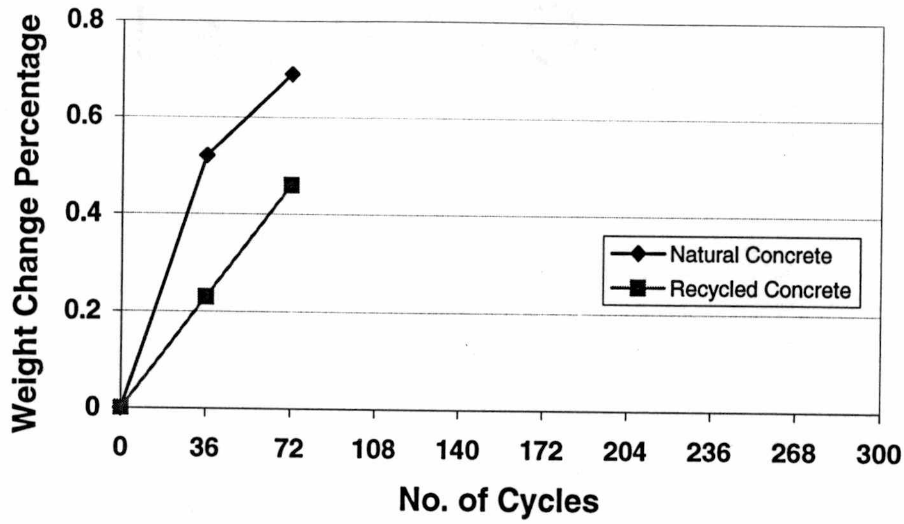

Figure 5.19 Weight Change Percentage for Case I.

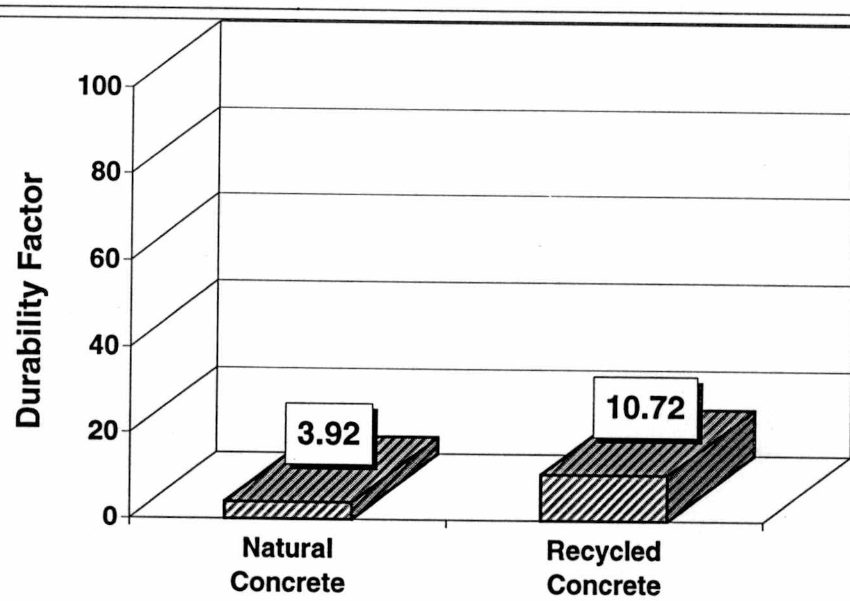

Figure 5.20 Durability Factor for RAC and NAC of Case I.

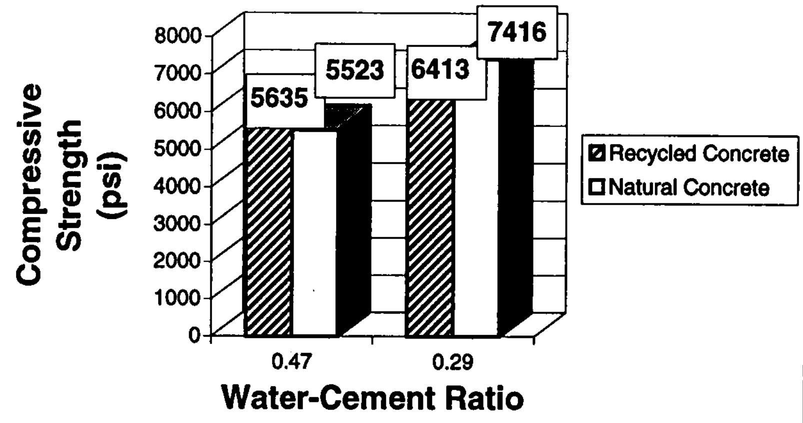

Figure 5.21 Effects of Water-Cement Ratio on 28-Day Compressive Strength of the RAC and NAC.

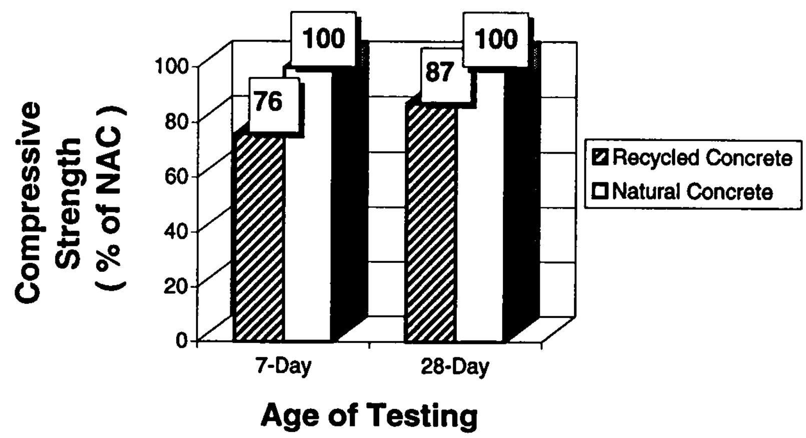

Figure 5.22 Relative Compressive Strength of Recycled and Natural Concretes for Case II.

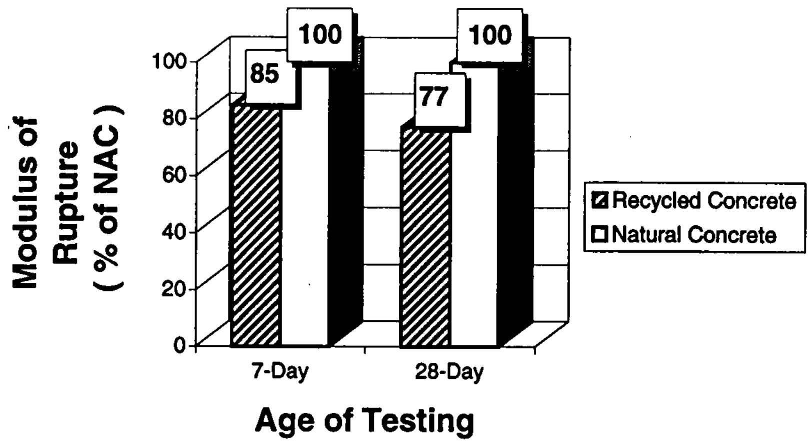

Figure 5.23 Relative Modulus of Rupture of Recycled and Natural Concretes for Case II.

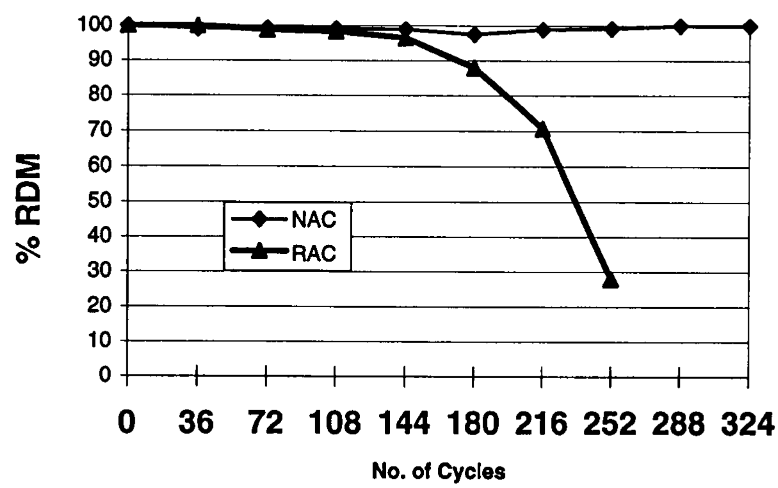

Figure 5.24 Relative Dynamic Modulus of Elasiticity of RAC and NAC for Case II.

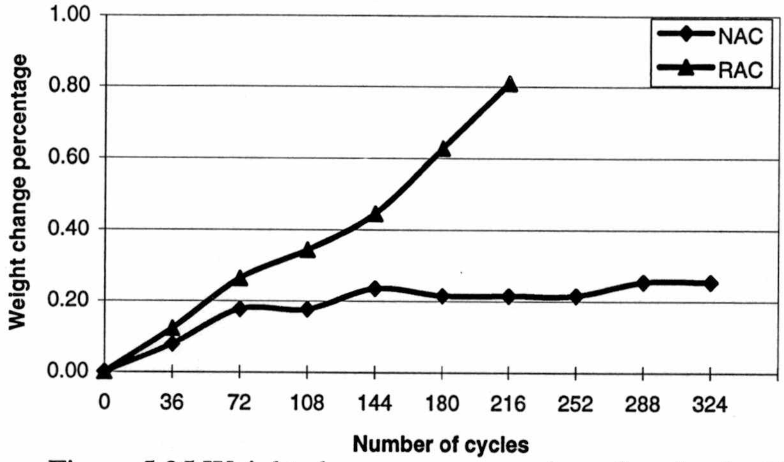

Figure 5.25 Weight change versus number of cycles for Case II.

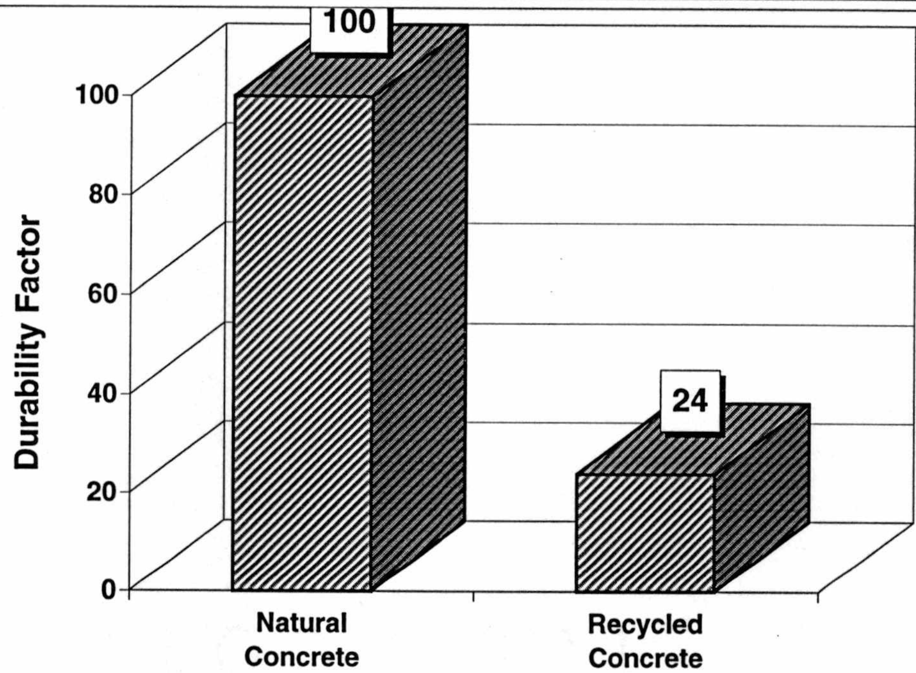

Figure 5.26 Durability Factor of RAC and NAC of Case II.

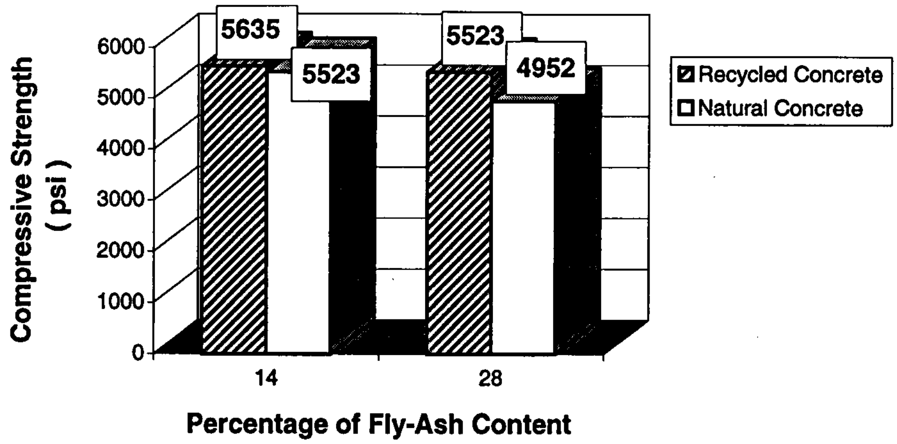

Figure 5.27 Compressive Strength of RAC and NAC at Various Fly- Ash Contents at 28-Day.

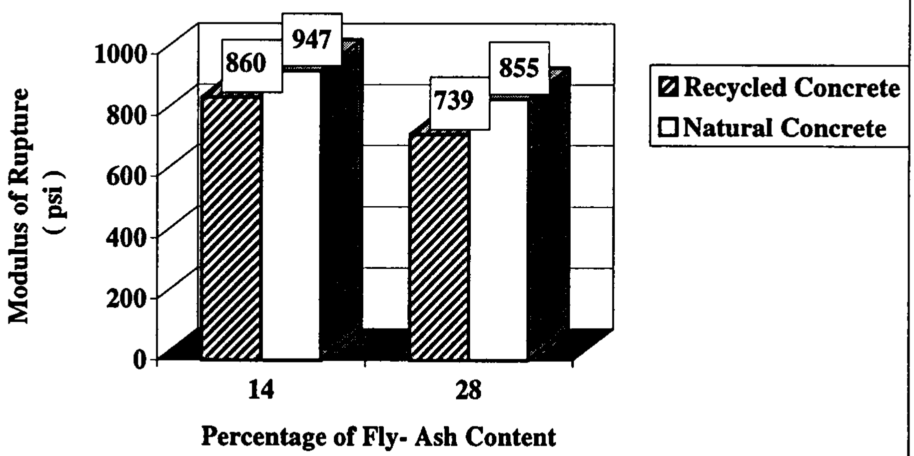

Figure 5.28 Modulus of Rupture of RAC and NAC with Various Fly- Ash Contents at 28-Day.

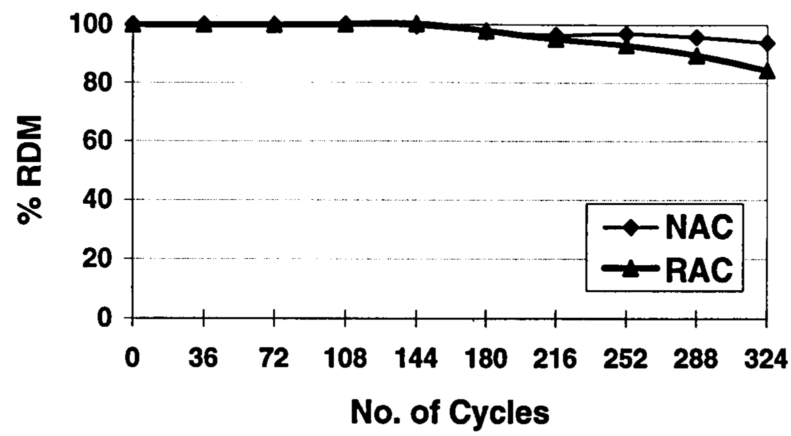

Figure 5.29 Relative Dynamic Modulus of Elasticity for High Fly-Ash Content Concrete.

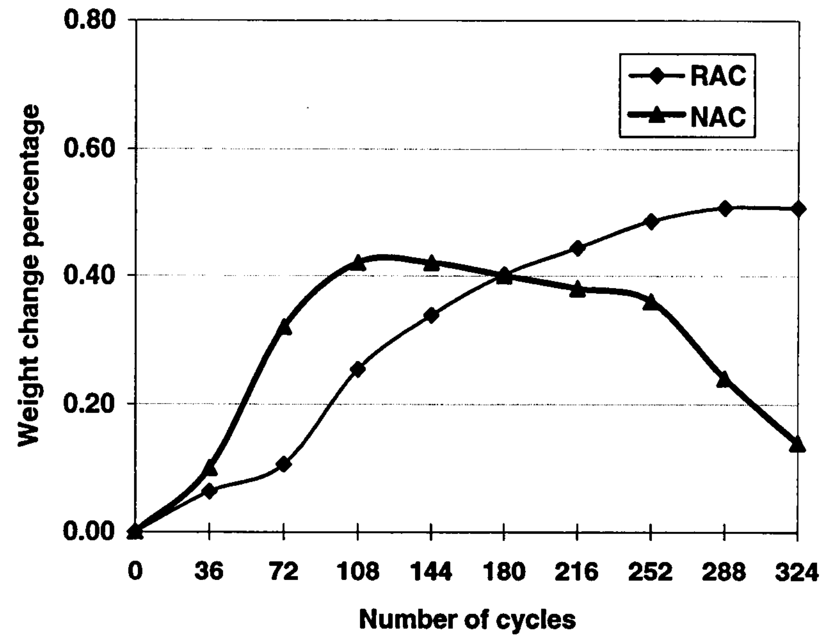

Figure 5.30 Weight change for High Fly-Ash Content.

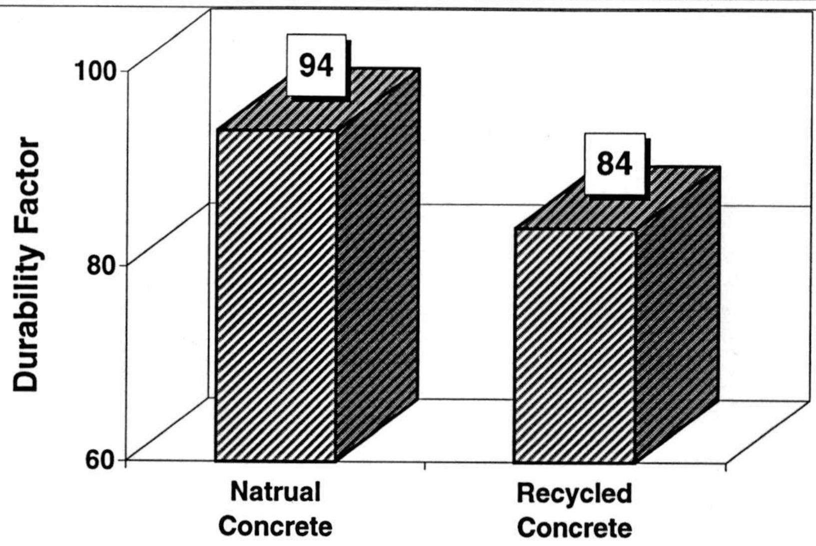

Figure 5.31 Durability Factor for RAC and NAC for High Fly-Ash Content.

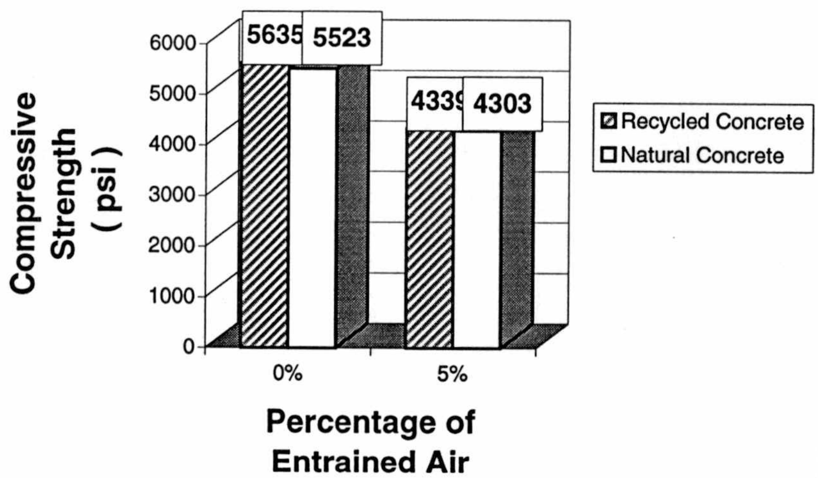

Figure 5.32 Compressive Strength of RAC and NAC with Various Air Contents .

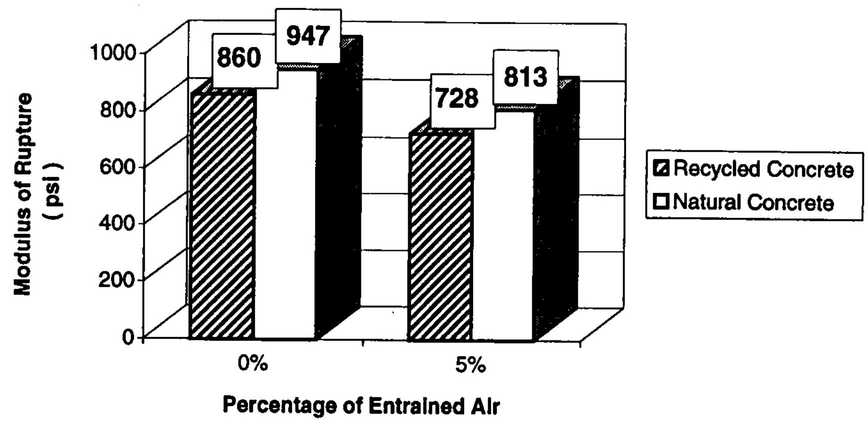

Figure 5.33 Modulus of Rupture of RAC and NAC with Various Air Contents.

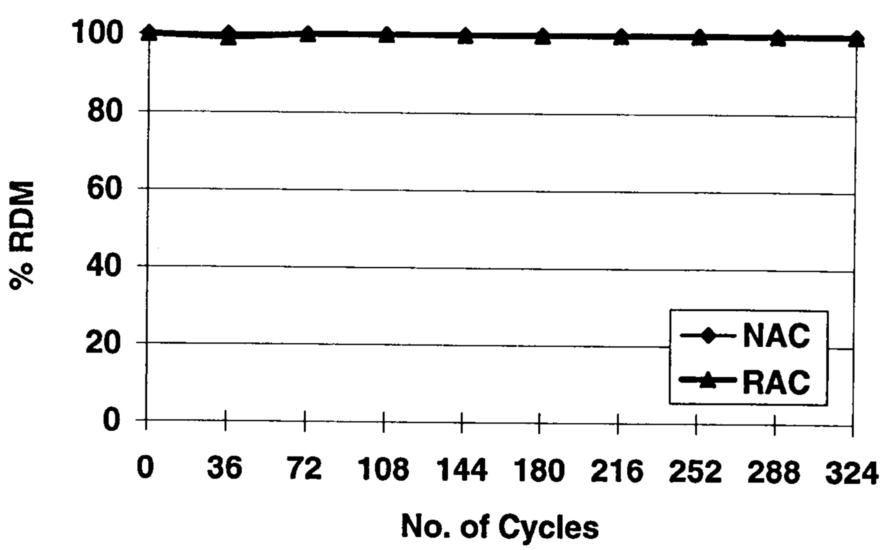

Figure 5.34 Relative Dynamic Modulus of Elasticity of RAC and NAC With Entrained Air.

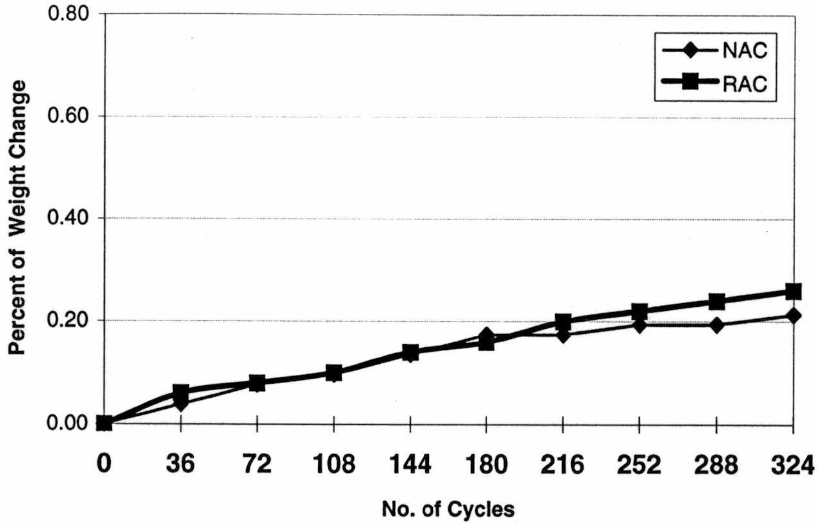

Figure 5.35 Percent Weight Change for RAC and NAC with Entrained Air

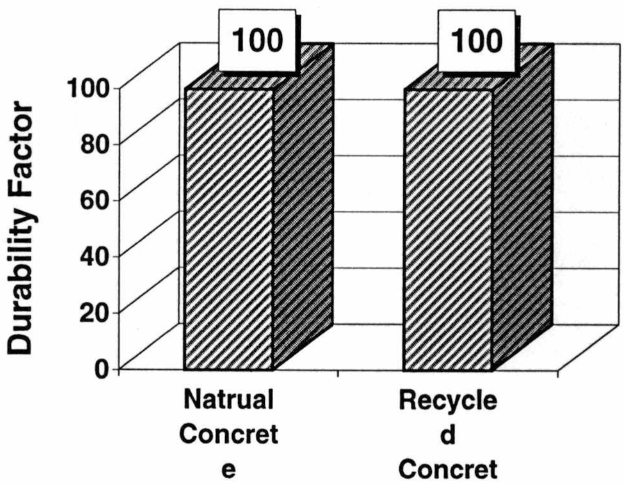

Figure 5.36 Durability Factor for RAC and NAC wit 5% Air Content.

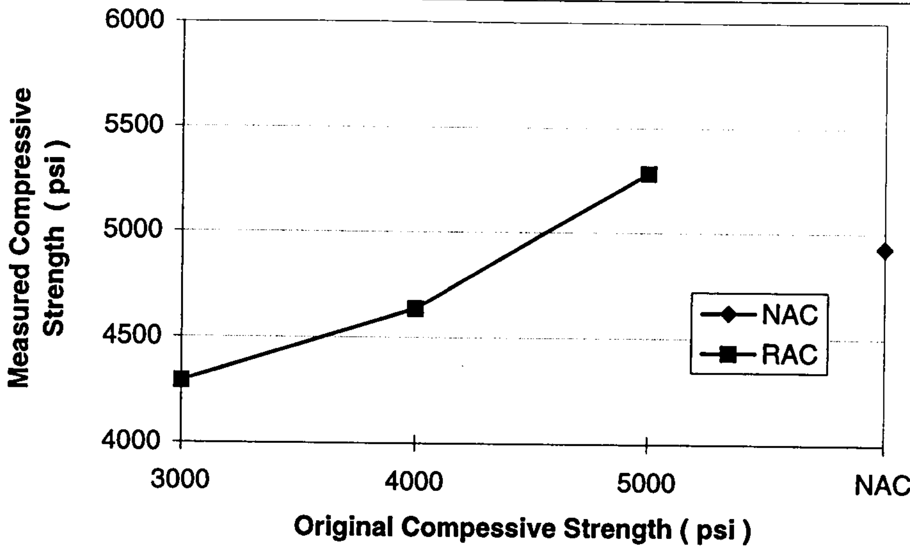

Figure 5.37 Compressive Strength of RAC's and NAC for Phase III at 7 days.

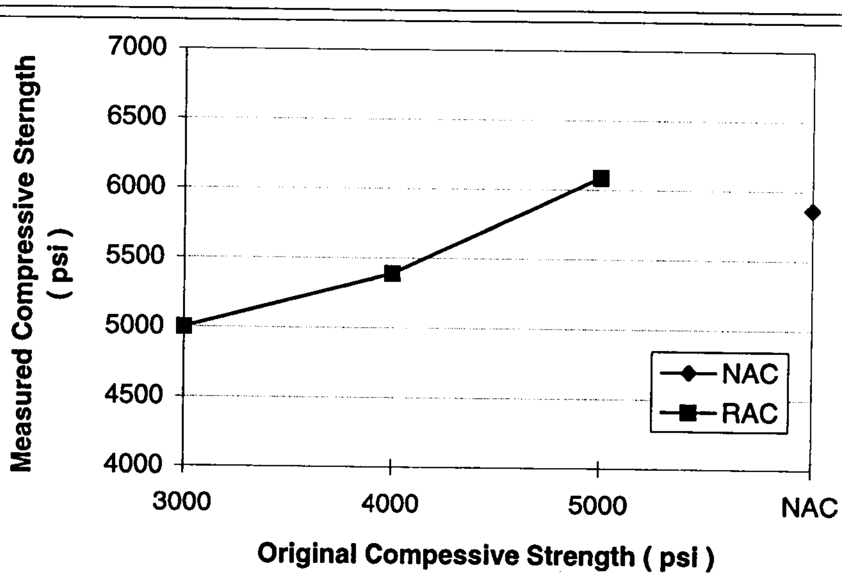

Figure 5.38 Compressive Strength of RAC's and NAC for Phase III at 28 days.

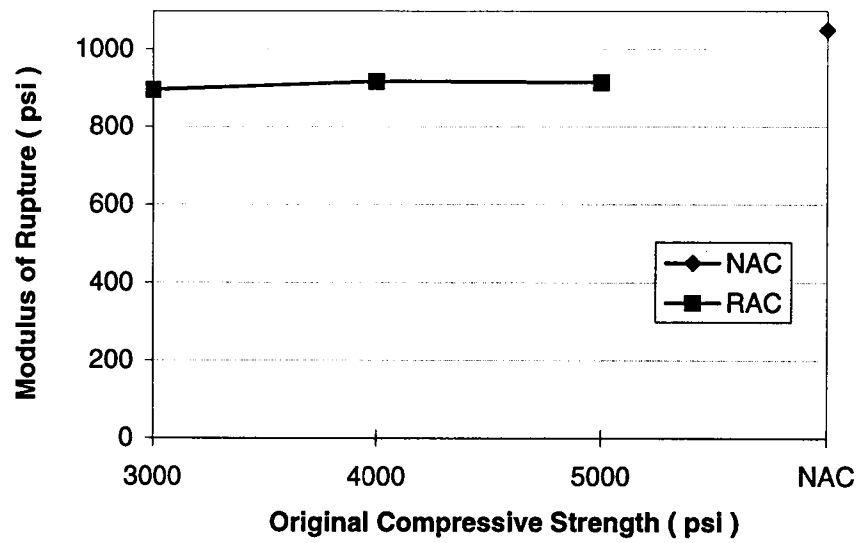

Figure 5.39 Modulus of Rupture of RAC's and NAC for Phase III at 28 days.

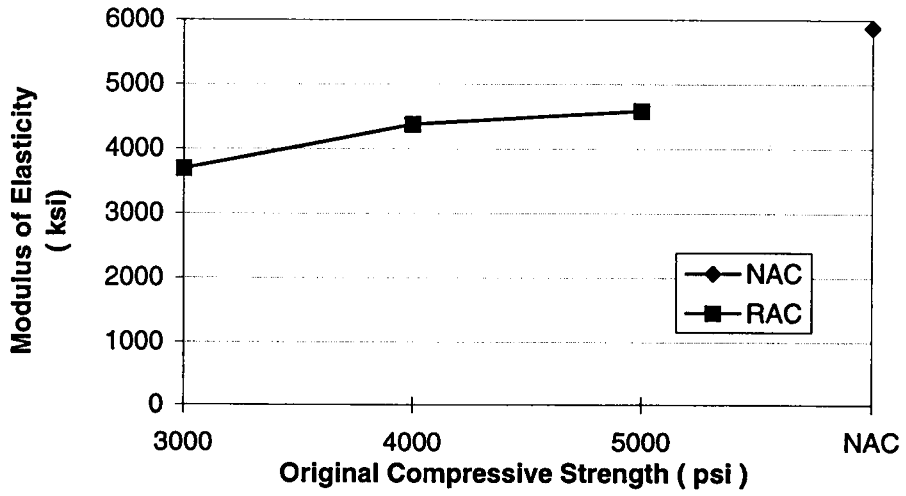

Figure 5.40 Modulus of Elasticity of RAC's and NAC for Phase III at 28 days.

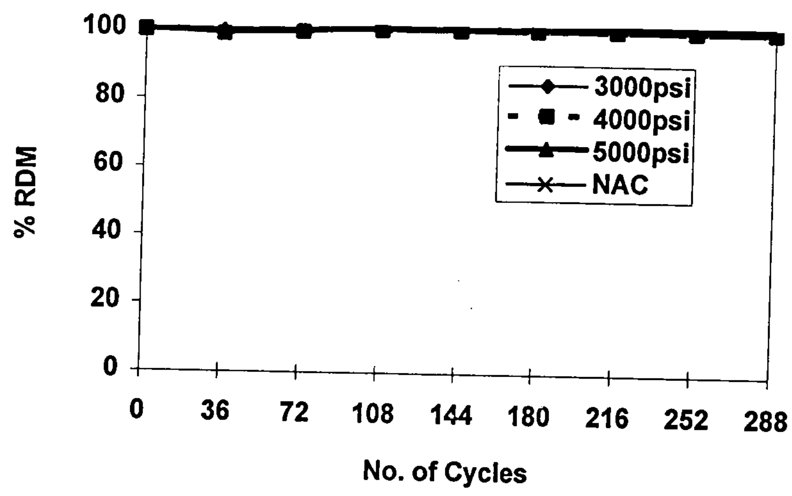

Figure 5.41 RDM for RAC with Different Original Compressive Strengths and RDM for NAC.

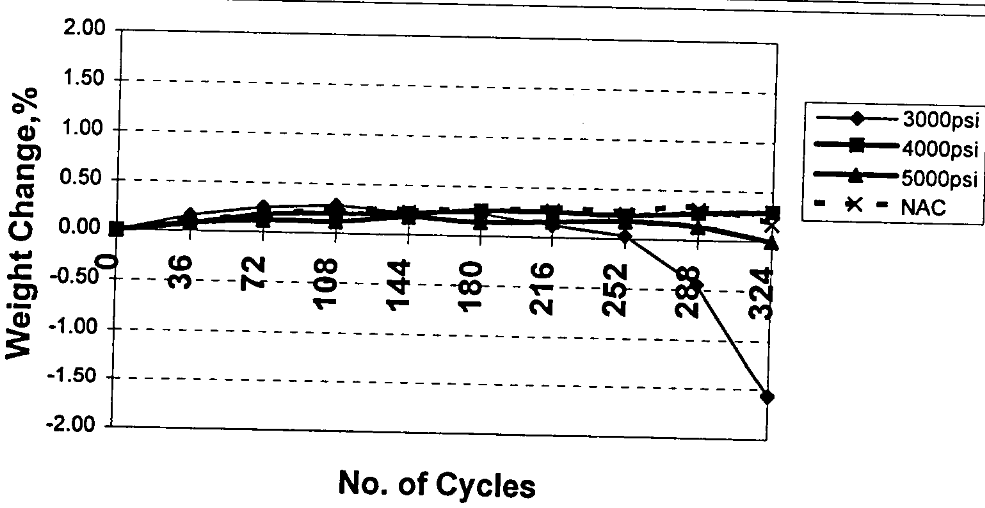

Figure 5.42 Weight Change Percentage for RAC with Different Original Compressive Strengths .

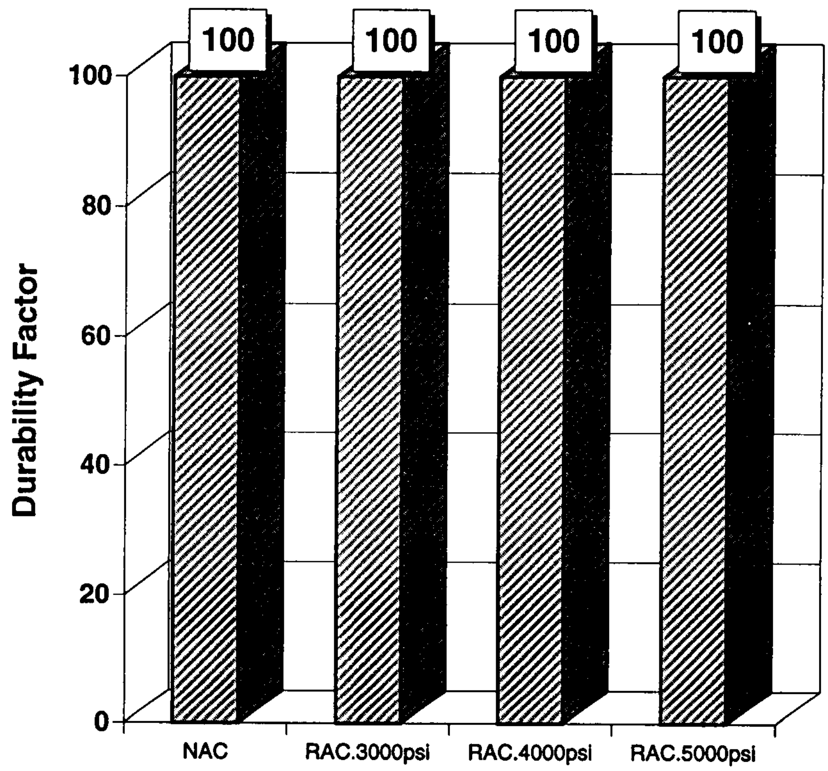

Figure 5.43 Durability Factor for RAC's and NAC

VITA

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