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**INVESTIGATION OF PHYSICAL PROPERTIES
OF CONCRETE AT
EARLY AGES**

**A Dissertation
Presented for the
Doctor of Philosophy
Degree
The University of Tennessee, Knoxville**

Francis Ajiboye Oluokun

May 1989

To my parents, my wife, Oyeronke, and my son, Oyekunle.

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Many people and experiences make up my history prior to this study. Realizing the impossible task of mentioning all, I will attempt to express my sincere gratitude and appreciation to those most responsible for providing encouragement and motivation throughout the course of the work presented in this dissertation.

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ABSTRACT

An investigation was conducted of the physical properties of concrete at early ages. Four different concrete mixes with 28 day strength varying from about 4,000 to 10,000 pounds per square inch were tested. Tests were performed on 6 by 12-inch concrete cylinders at ages of 6 hours and 12 hours, and at 1, 2, 3, 7, and 28 days. Physical properties measured at each age included the compressive strength, the static elastic modulus, the splitting tensile strength, and Poisson's ratio. Investigation was also conducted on the effects of steam curing (as used in a typical precast, prestressed concrete plant) on these physical properties. The applicability of some physical property prediction equations to concrete at early ages as well as to high strength concrete was looked into.

The compressive strength, the tensile strength, and the elastic modulus are all related, and an increase in one is, in general, similarly reflected in increases in the others. The period within the first 12 hours after casting was observed to be the period of fastest development of physical properties. The elastic modulus developed much faster than either the tensile or the compressive strengths which developed at approximately the same rate. Poisson's ratio did not change substantially with both richness of mix and age of concrete.

The commonly accepted relationship that the tensile strength and the elastic modulus of concrete are proportional to the 0.5 power of the cylinder compressive strength with time was found to be accurate for the elastic

modulus but inaccurate for the tensile strength. In fact, the tensile strength was found to be proportional to the 0.79 power of cylinder strength with respect to age. An alternate relation between the tensile and the compressive strengths was proposed.

Plowman's relation for compressive strength prediction has been found to be inaccurate at low maturities. An alternate relation was proposed to take care of these deficiencies. Similarly, a more representative relation between the seven and twenty-eight day strengths was proposed as an alternate to Hool's relation of 1935.

Steam curing increased physical property development at early ages at the expense of strength development at later ages. The adverse effect at later ages was relatively more pronounced in the tensile strength development.

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NOMENCLATURE

D	Diameter of test cylinder
E _c	Static elastic modulus at 28 days of standard moist cure
E _{cx}	Static elastic modulus at ages other than 28 days
f _c '	Compressive strength at 28 days of standard moist cure
f _{cx}	Compressive strength at ages other than 28 days
f _t	Tensile strength at 28 days of standard moist cure
f _{tx}	Tensile strength at ages other than 28 days
K	Intercept of regression line
L	Length of test cylinder
M	Maturity
P	Load at failure
R	Slope of regression line
r	Radius of test cylinder
Se	Predicted compressive strength at age "e"
T _i	Final temperature
T	Datum temperature
W	Dry unit weight of concrete

Greek Symbols

μ	Poisson's Ratio
ε	Unit strain
π	3.14159

CHAPTER I

INTRODUCTION

In the early 1950's, the use of precast prestressed concrete deck panels (or plank as it was formerly called) was proposed for a series of underpasses for the Illinois Toll Highway. These precast prestressed deck panels have two major functions:

- (1) Initially, the panels are to carry the weight of about six inches of cast-in-place concrete which is cast on top of the deck panels to complete the deck slab;
- (2) Finally, the panels bond to the cast-in-place concrete to become an integral load carrying part of the complete bridge system through composite action.

Recently it has been observed that many of the deck panels are developing cracks in a pattern consistent with the orientation of the prestressing strands. Among the many recent studies of beam-slab bridge systems is the work of Fagundo, Tabatabai, Soongswang, Richardson and Callis [42]. They conducted a series of tests on several bridge decks to determine the degree of composite action and structural adequacy of each bridge system. The bridge systems tested included (1) moderate and short span stay-in-place decks with regular fireboard support all the way to the end of the panel (no grout bearing was provided); (2) modified stay-in-place decks with strand extensions and grout support for the panel; and (3) standard cast-in-place decks. Their extensive

testing revealed that several composite deck bridges with precast panels have experienced severe cracking and spalling, some after only a short time in operation.

Other research was conducted by Buckner and Turner [27] to determine the effects of repetitive loading on serviceability and strength of composite panel form bridges. Six simply supported bridge decks were tested. The decks were made up of three precast, pretensioned panels spanning in the direction of traffic and acting compositely with the cast-in-place topping. All the decks were tested to failure. It was observed that the primary mode of failure was a ductile flexural mode. In addition, transverse cracking of the topping slab was observed in two of the six bridge decks. It was also noticed that these cracks developed during the early period of curing, approximately within one week after the concrete topping was cast. This is a period within which concrete may not have attained its specified design strength. Although these cracks were attributed to relatively poor curing conditions, they caused an increase in measured service load deflection of about twenty-five percent compared with the uncracked decks. This is a reflection of the impact of early age cracking on overall serviceability of the structural system.

Jones and Furr [72] conducted a field study of three existing highway bridges constructed by Texas Highway Department in 1963. The three bridges utilized the use of precast, prestressed concrete deck panels as an integral part of their decks. The investigation was undertaken to establish the condition of these structures after seven years of service and to detect any possible discontinuity of action between beams, panels and decks. The study included

mapping of crack patterns in the cast in place deck, soundings to detect delamination between prestressed panels and cast-in-place deck, corings, and load tests. Although evidence of distress or lack of composite action was undetected in the bridges tested, some transverse cracking was found in the cast-in-place deck of the heavily traveled bridges. Most of these cracks were found to coincide with the transverse butt joints between prestressed panels. Core samples revealed that these cracks extended approximately halfway through the cast-in-place deck.

Often times prestressed panel cracking is observed during routine inspections at the prestress plant prior to delivery to the job site. In most cases, these cracks form during prestressing at the time of form removal when concrete has not attained its specified design strength. Other cracks have been observed during the Federal Highway Administration's (FHWA) mandated periodic bridge inspection. This is an indication that the problem has been present although possibly undetected for a considerable period of time.

Among the considerable amount of research done on stay-in-place precast, prestressed panels primarily from the Texas Highway Department, the Pennsylvania Department of Transportation, and the Florida Department of Transportation were the works of Barnoff and Rainey [20] and Barnoff and Orndorff [19]. These works included the laboratory casting and testing of panels and construction and testing of a full scale bridge. The deck of the full scale bridge was formed with precast, prestressed panels. They found that there were longitudinal cracks along the top of the center strand when strands were released. This was before the concrete attained its final cure and specified

design strength, indicating that the cracking problem is a function of concrete strength which in turn depends on time of cure. In addition to the longitudinal cracks, transverse cracks appeared in the top of the decks above the joints in the panels. Again, in agreement with Jones and Furr [72], cores showed that these cracks extended approximately halfway through the cast-in-place concrete in every case. These cracks were attributed to thermal and shrinkage effects. Barnoff and Rainey [20] also observed that flexural cracks were formed at the time panels were removed from the casting beds. These cracks were attributed to the relatively low strength of concrete at the time the panels were removed from their beds.

In some construction, cast-in-place concrete is often subjected to flexural action beginning at the time it is cast. A bridge deck being widened while traffic is maintained on adjacent lanes is an example. Records that were kept by Furr and Fouad [47] on bridge widening construction showed that transverse bending occurred in the fresh concrete as a result of traffic moving on the old deck. Britton [25] in another study has reported flexural cracks in continuous bridge decks before the concrete had its final set. These cracks occurred in the vicinity of the supports at the top surface of the deck. These cracks were attributed to changes in deflection and rotation of formwork over the support during construction and to concrete placement. These research results showed clearly that flexural action of any form on concrete in the early period of its life is detrimental to its satisfactory performance and appearance. This detriment has been attributed to the low physical properties of concrete early in its life.

Flexural action on early age concrete is not peculiar to the bridge construction industry alone. The building construction industry has reported cracks in concrete due to significant construction loads on concrete at early ages. Typically, in multi-story building construction, cast-in-place reinforced concrete slab and beam structures involve casting the new slab into forms which are supported by one or more previously cast floor slabs. In the study conducted by Fu and Gardner [46] on comparison of immediate and time dependent deformations of one way slabs, it was revealed that high early age short duration construction loads are imposed upon the supporting slabs. These early age construction loads can be of comparable magnitude to the design service loads and are applied to concrete slabs which have not achieved their specified strength. It was also observed that due to immaturity of concrete slabs with reduced modulus of elasticity, the immediate deflections due to the construction loads are relatively large. Since creep effects are dependent upon the magnitude of applied stress relative to the concrete strength developed, creep deflections due to construction loads are large. They also concluded that construction loads applied to immature concrete slab results in significant non-recoverable creep deflections which can not be ignored in long term deflection considerations. The paper further suggested that concrete strength at form stripping should be specified to ensure meeting deflection and other serviceability criteria.

Fu [45] in his own study of the effect of high construction loads on short and long term deflections of slabs confirmed the observations and conclusions of Fu and Gardner [46]. In addition, flexural cracks were also reported in building

slabs subjected to high early age construction loads compared to the slab capacity.

Shah, Miller, and Virding [132] conducted a study of early age shear strength of reinforced concrete beams. They discovered that:

- (1) Mode of failure of reinforced concrete beams with shear reinforcement changes with age from shear to flexure.
- (2) Ductility of beams at early ages is less than the ductility at maturity.
- (3) Effectiveness of shear reinforcement is low at early ages of beams.
- (4) The American Concrete Institute method for evaluating strength is not necessarily valid at early ages of concrete.

All these observations were attributed to immaturity of concrete at early ages and its consequent relatively low physical properties.

From the series of research activities and test results discussed above, it is evident that either in precast, prestressed concrete or in cast-in-place concrete construction, concrete is vulnerable to flexural cracking, excessive short and long term deformations, and other associated serviceability and performance problems. These cracks and excessive deformations are frequently caused by flexural actions on immature concrete between the time it is cast and the final setting time. This has been attributed to the relatively low strength of concrete at that age. In other cases, cracks may be formed after final set but

before the concrete has attained sufficient tensile strength to resist the imposed flexural stresses.

CHAPTER II

STATEMENT OF THE PROBLEM

2.1 Introduction

In the preceding chapter, the effects of early flexural actions and loading on immature concrete were discussed. Several experimental and theoretical research results were cited to substantiate these observed adverse effects. The terms immature concrete, cracks, and excessive deflections were frequently used. In this section an explanation is given of what an immature concrete is. This section also discusses concrete immaturity as it affects both the precast, prestressed concrete and cast-in-place concrete industries. The impacts of cracking, excessive deflections, and other serviceability problems related to early flexural actions on structures are also discussed. Mention is also made of the different circumstances in the concrete industry that have necessitated some significant application of stresses to concrete at early ages, before concrete attains its final cure and specified design strength.

2.2 Concrete Immaturity and Flexural Actions

Concrete, in a very broad sense, can be defined as any product or mass made by the association of some aggregate and a cementing medium. Concrete can be best described as being in the liquid state at the time it is cast. After casting, and with time, it undergoes a period of phase transformations. Concrete changes from the liquid to the plastic state and eventually it attains the solid state. It is a well established fact that during these phase changes, the

period when concrete sets, and for a short time thereafter, concrete has little ability to resist stress. The ability to resist stress is attained in the curing process. As a method of acceptability, curing is said to be complete twenty-eight days after casting even though, in fact, it continues to gain strength. Conventionally, concrete is considered immature for flexural actions anytime between the time it is cast and twenty-eight days of normal curing. Therefore, ideally, premature stressing of concrete or loading of any kind should be avoided. Gilkey [56] recommended that during the curing period, shocks from explosives, from heavy falling objects, or from other construction operations should be avoided because of their adverse effects on long term performance of concrete structures.

Daily turn-around of forms is the basis of the economy of the precast, prestressed concrete industry. This can only be achieved if a relatively high compressive strength is attained at about sixteen hours after mixing the last concrete of the day. Consequently, successful factory operation requires a curing procedure which will insure that the required release strength is attained. To achieve the required release strength, low pressure steam curing methods are usually employed. Many studies have been undertaken to confirm the strength development effectiveness of steam curing. Keong [74] revealed that steam curing accelerates concrete strength development. This observation was also confirmed by Hansen [60] and Shideler and Chamberlin [133]. It is worth noting that steam curing as used by the precast, prestressed industry, does not insure that concrete attains its specified design strength before strands are released. At the time of strand release, a fractional, though appreciable, part of the

specified strength may have been developed anyway. Consequent to the relatively low strength of concrete at time of strand release, cracking has been reported by Britton [25], Furr and Fouad [47] and Barnoff and Rainey [20] in their independent studies of precast, prestressed bridge deck panels.

Contrary to the recommendations above and in detriment to the structural integrity of concrete, recent pressure on the concrete construction industry to provide shorter construction schedules has led to flexural actions on immature structural members. The shorter construction schedule has been necessitated by the need for earlier occupancy of structures and construction cost savings. Additionally, recent catastrophic failures of concrete structures [65, 111] under construction revealed the inherent danger in the application of significant loads to immature concrete. From the discussions in the preceding chapter, it is well established that early age flexural actions on immature concrete have resulted in cracking, excessive immediate and long term deflections, and some other associated serviceability problems. These deficiencies in immature concrete have been attributed to the relatively low physical properties of concrete at the age it is first subjected to flexural actions. Gardner [49] observed that concrete slabs of high rise buildings have shown some excessive deformation tendencies when forms and shoring were removed before concrete had its final cure. Therefore, for early form stripping, post-tensioning, pretensioning and other flexural actions, it is imperative that the physical properties of concrete be monitored to maintain structural, as well as human safety. Verification of concrete strength with a high degree of confidence is therefore a necessity during the critical early days following concrete placement. At such an early

age, the concrete may have attained only a fraction of its twenty-eight day physical properties. Effects of early age flexural actions on immature concrete become very pronounced when the applied load is a substantial percentage of the actual design load.

2.3 Effects of Cracking and Excessive Deflections on Structures

Cracking has been linked with the early age flexural actions on immature concrete. It is very important to state here that this is not the only cause of cracking in concrete; it is one of numerous different situations that can result in cracking in concrete. These situations range from minute internal microcracking that occurs even on the application of a modest amount of stress to undesirable large cracks caused by unfavorable interactions with the environment. Generally, cracks do not affect the load carrying ability of concrete; however, in extreme cases, cracks have been reported to have a serious detrimental effect on the structural integrity of concrete. Cracks also have adverse effects on durability and deflections as reported by Buckner and Turner [27]. Cracks have been found to be points of access to the body of concrete for aggressive agents. Buildings have been considered unfit for habitation on the basis of excessive deflections. Cracking, once formed and under conducive situations, can propagate and thus become very detrimental to structural integrity and serviceability. Cracking, excessive deflections, and other serviceability problems should therefore be minimized if not totally prevented.

The overall evaluation of cracking problems in precast, prestressed or cast-in-place concrete construction or even in concrete deck panels is far beyond the scope of this study. The extent of the problem with deck panels is illustrated by Figure 2-1 which is a graphic representation of the different phases of the problem. The total cracking and serviceability problems include a broad spectrum of potentially important variables; however, the literature survey discussed in the previous chapter revealed that the first and potentially the most important area to investigate relates to the physical properties of concrete at early ages. This is the age when structural members are initially stressed and the age when cast-in-place concrete is subjected to early age construction loads and other early flexural actions. Thus, this study concentrated on the investigation and consequent determination of the early age physical properties of concrete.

The determination of early age physical properties of concrete is at the fore-front of concrete technology as is evidenced by the American Concrete Institute (ACI) formation of a new technical committee 231 on properties of concrete at early ages. Francis Young, Chairman of ACI 231, states in the preface to ACI SP-95 [10]:

The environment that concrete is subjected to very early in its lifetime (the first few days) is crucial to its performance. However, the concrete will be affected by conditions which may not be reflected by the external environment. Concrete is subjected to internal heating, and perhaps self-diccation, during hydration. The thermal and hygric gradients within the concrete may be much greater than is realized, causing it to be subjected to considerable internal damage, even though it appears sound. Dr. Idorn has drawn attention to this problem in several forums, the most recent being the 1983 Raymond E. Davis lecture (see Concrete International, February 1984, pp. 13-20). His experience as a consulting engineer

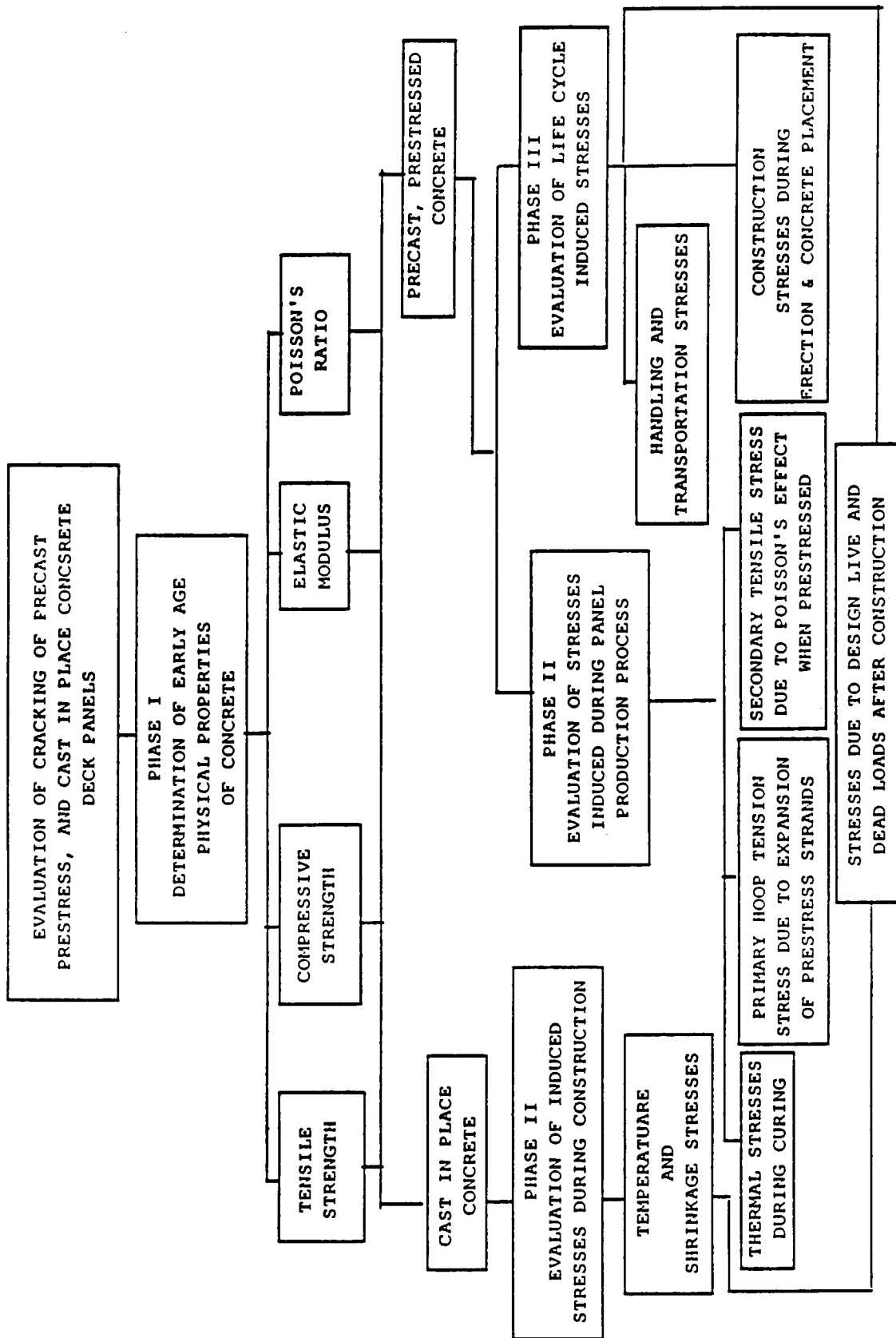


Figure 2-1: Evaluation of Cracking of Concrete

has led him to conclude that the early age history of concrete is crucial to determining how durable that concrete will be. The concrete's environment will also be important in determining how properties, other than compressive strength, develop in the early stages. This knowledge is necessary for correctly predicting concrete deflections and overall crack resistance. Such considerations are important elements in ensuring safe construction practices.

The main objectives of the present research are as follows:

- (1) Obtain a fundamental understanding of the development of physical properties in concrete particularly within the first three days after casting.
- (2) Generate data describing the time variation of development of physical properties in concrete with particular reference to the early age of concrete.
- (3) Evaluate the existing relationships between all the physical properties and their applicability to concrete at early ages and perhaps develop and propose alternate relationships.
- (4) Determine whether or not the existing strength relationships are valid for predicting strengths in high strength concrete.
- (5) Develop and propose empirical relationships between various concrete physical properties and time of cure.

To achieve these objectives, the relevant literature pertaining to experimental and theoretical aspects of physical strength development in concrete were assembled and reviewed. Available data were also summarized. This was done for two main reasons: (1) to give the writer a comprehensive

knowledge of what has been done to date on the subject and (2) to identify areas related to concrete strength development which have not been adequately considered in previous studies of early age concrete strength development.

The literature review as discussed extensively in the next chapter of this study points to an urgent need for the present study of strength development in concrete at early ages. An experimental and analytical study was undertaken and is described in the subsequent sections. The data generated along with the developed and proposed strength relationships are applicable to both precast, prestressed and cast-in-place concrete construction. It is believed that appropriate use of the information obtained should help to mitigate the concrete cracking problem at early ages and help in predicting concrete deflections more accurately.

CHAPTER III

PREVIOUS EXPERIMENTAL AND THEORETICAL STUDY

3.1 Introduction

One of the most important phases of a research project is a comprehensive review and summary of previous related experimental and theoretical studies. This section is, therefore, primarily concerned with the summary of relevant works that have been undertaken on the development of physical properties of concrete. Factors that have been known to affect strength development in concrete are reviewed and summarized along with previously postulated hypotheses for the development of physical properties. Also, available strength prediction formulas and techniques for predicting physical strength parameters are presented and discussed. A detailed discussion of the existing strength prediction equations including different opinions of concrete researchers are presented.

3.1 General Observations of Record

Comprehensive studies of the physical strength development in concrete started before the year 1900. Most of the early studies concentrated on concrete strength development at maturity. Early age studies did not start until very recently. Consequently, very little information is available in the literature on early age physical properties of concrete. That not much has been done on early age physical properties can be attributed to the fact that the primary concern of designers for a long time has been with physical properties at a mature age, conventionally at 28 days. Another reason for the apparent

tardiness in early age studies is the difficulty of handling the material before it has had its final set. But probably the most important reason for this tardiness is that for a long time construction schedules did not lead to the need for early age strength evaluation. Usually, structures were allowed to mature before they were subjected to flexural actions. The relatively scanty literature on physical properties of concrete at early ages was confirmed by Furr and Fouad [47], Hansen [60], and Fu and Gardner [46] in their independent and very recent (1986) studies of properties of concrete at early ages.

The physical properties of concrete at early ages cannot be investigated thoroughly without any link with the observations and results of numerous previous studies on strength of concrete at maturity. Any meaningful strength investigation must be conducted with due consideration for all the factors that have been found to influence concrete strength development. Moreover, a proper understanding of previous methods for concrete strength evaluation is necessary to the overall understanding and consequent evaluation of previously obtained research results. It is, therefore, considered necessary to highlight some early observations of record on investigation of concrete physical properties at relative maturity that are considered pertinent to the investigation of concrete strength development at early ages.

Investigations into the various physical properties of concrete can be traced to the time when Feret, the French investigator, first recognized the water to cement ratio versus strength relationship. This was about two decades prior to the well-publicized Abrams [2] pronouncements of 1918 on the effect of water to cement ratio on concrete strength. Since then, several research results have

established that the development of the physical properties of concrete, most especially compressive strength, to which other physical properties are related, is influenced by numerous environmental and material factors. The most important of these established influential factors, as it relates to this study, are:

- 1) the water to cement ratio
- 2) type, size, and grading of aggregates
- 3) moisture condition
- 4) curing method
- 5) temperature and duration of cure

Other conditions such as the shape and size of test specimens, testing machine used, and other test conditions are assumed to be fixed since standard methods as specified by the American Standard for Testing and Materials (ASTM) are used in this study.

The effects of the factors listed above, as revealed by the results of previous investigations, are discussed in the following sections.

3.2.1 Effect of Water to Cement Ratio

Abrams' [2,5] water to cement ratio pronouncement of 1918 has been described as the most useful and significant advancement in the history of concrete technology. Abrams performed and published extensive experimental research works on the relationship of concrete compressive strength and water to cement ratio. His most important formulation was the inverse proportionality between water to cement ratio and strength of concrete. As

opined by Gilkey [57], Abrams formulated statements that have, for many concrete practitioners, elevated the ratio of water to cement ratio to the near-sacred status of the law of proportioning for concrete. Abrams [2] conducted works on design of concrete mixtures, water to cement ratio as a basis for concrete quality, proportioning concrete mixtures, and the effect of time of mixing on the strength and wear resistance of concrete. Tests carried out showed the controlling influence of the water to cement ratio on strength of workable concrete mixes. He also proposed a relationship (discussed later) between strength and water to cement ratio of concrete. It was his opinion that the water to cement ratio law is valid regardless of mix and size and grading of aggregate as long as the concrete is plastic. Abrams water to cement ratio law states that

For a given cement and conventional aggregates in workable mixtures, under similar conditions of placement, curing and test, the strength of concrete is solely a function of the ratio of cement to the free water in the plastic mixture.

This pronouncement gained unquestioned and near universal acceptance in its entirety. The water to cement ratio assertion survived for almost half a century without a change although not without any challenge. Most of the later challenges were in the area of the inert filler role that Abrams accorded the aggregates. In fact Abrams' inert filler concept of aggregate is evidenced in one of his fundamental principles on proportioning concrete mixes where he states:

It is not necessary, or desirable, that the aggregate be proportioned according to any fixed grading; wide variations in gradings of aggregate may occur without affecting the quantity of mixing water or the quality of the concrete.

Strong, in his belief that concrete strength was dependent on only one factor--the water to cement ratio, Abrams [2] enunciated a mathematical relationship between concrete strength and water to cement ratio as

$$S = \frac{A}{B^x} \quad (3.1)$$

where S is the compressive strength of the concrete

x (an exponent) is water to cement ratio, and

A and B are constants for a given material and test conditions

For an average Portland Cement concrete cured under normal temperature and moisture conditions, Abrams gave the relationship between compressive strength and water to cement ratio as

$$S_7 = \frac{9200}{(14)^x} \quad (3.2a)$$

For the seven-day strength and

$$S_{28} = \frac{14000}{(8.2)^x} \quad (3.2b)$$

For the twenty-eight day strength

Powers and Brownyard [119] in a different study agreed with Abrams that the water to cement ratio is inversely proportional to the strength of concrete. They also revealed that the water to cement ratio has a major influence on the porosity and durability of concrete.

3.2.2 Effect of Type, Size, and Grading of Aggregate

In 1922, Walker [143] conducted field tests to determine proportions for concrete. He performed tests on 6-in. by 12-in. cylinders stored in a moist

place. Testing was done at 28 days. He recognized that the strength of concrete is a function of (1) size and grading of aggregate, (2) quantity of cement in the mix, and (3) plastic condition of concrete. Compressive strength interrelationship with size and grading of aggregate was emphasized. This observation was contrary to the inert filler concept proposed by Abrams [2, 5].

Hutchinson [70], in his study of proportioning concrete materials with special reference to highway construction, performed compressive tests on 6-in. by 12-in. cylinders. The tests included practically every gradation of coarse aggregate used in normal concrete work. Data on the effect of variation in cement content as well as in the ratio of fine to coarse aggregate were furnished. From the results and reasonable interpretation of the curves, it was concluded among other things that (1) the compressive strength and the resistance to impact of concrete were increased as the size of coarse aggregate was decreased, (2) relative compressive strengths of concretes containing the smaller sizes of coarse aggregate increased at a greater rate than those containing the larger sizes, and (3) an economic relationship existed between the amount of cement in the mixture and the maximum size of the coarse aggregate used. This showed that the type and grading of aggregate are important factors in concrete strength evaluation.

Investigations were also made of the variations of compressive strength of concrete within the water to cement ratio law. This was done by Collier [34]. Three series of standard concrete tests overing about 450 specimens were carried out to study (1) effect of various consistencies, (2) effect of various percentages of sand, and (3) effect of various gradings of sand in concrete

having a number of different water to cement ratios. It was found from this study that the compressive strength of concrete of the same water to cement ratio decreases as the percentage of sand decreased, and also decreased as the fineness modulus of sand decreased. An explanation for this observation was advanced that there is a limit to the amount of cement paste that sand in concrete can hold in a homogeneous condition. If any amount of paste above this is used, the cement and water tend to separate before hardening begins, which results in a loss of strength. This study supports the fact that concrete strength also depends on the proportion and grading of sand and not on water to cement ratio alone.

In another study conducted by Walker and Bloem [144] on the relationship of concrete strength to maximum size of aggregate, four different maximum sizes of aggregate, $3/8$, $3/4$, $1-1/2$, and $2-1/2$ -in., were used in concrete mix preparation. All tests were conducted at 28 days using 6-in. by 12-in. cylinders in accordance with ASTM standards. Two different sources of coarse aggregate were used for each grading. It was observed in both cases that there was a reduction in water requirement, for a given slump as maximum size was increased. In spite of this reduction in water requirement and consequent reduction in water to cement ratio, strength did not improve with increased aggregate size. Large sizes of coarse aggregate tend to reduce measured strength because its effect is enough, in most cases, to offset the effect of the associated reduction in water to cement ratio. They also observed from the data collected that highest strengths were secured with a maximum size of $3/4$ in., slightly lower and about equal strengths with $3/8$ and $1-1/2$ -in., and lowest

with 2-1/2-in. maximum size. This is further evidence that concrete strength does not depend on water-cement ratio alone; it also depends on gradation and size of coarse aggregate.

An experimental investigation was conducted by Walker, Bloem, and Gaynor [145] on the effects of aggregate size on properties of concrete. This study was to supplement the earlier work of Walker and Bloem [144] discussed above. This investigation covered a broader range of classes of concrete and test variables for the purpose of checking the degree of applicability of their earlier finding. The tests were made with maximum sizes of aggregate ranging from 3/8 to 2-1/2-in. using three cement factors, utilizing between 4 and 8 sacks per cubic yard. They measured compressive and flexural strengths at ages 7, 28, and 91 days. Tests included tensile splitting tests which were done at 28 and 91 days. Again, from the results and data collected, they concluded that increasing the maximum size of coarse aggregate may not necessarily be beneficial to concrete strength. In addition, they indicated that drying and shrinkage was not substantially increased by reduced size of aggregate down to about 3/4-in. They clearly concluded again that the size of coarse aggregate exerted some influence on concrete strength independently of water to cement ratio; for a given water to cement ratio, strength became less as maximum size of coarse aggregate was increased. They also observed that the decreased strength resulting from use of large aggregate sizes was most pronounced in the concretes of higher cement factors. Lean mixes showed very slight increases in strength with increasing aggregate sizes.

Gilkey [57] was so critical of the inert-filler concept of the aggregate function suggested by Abrams [2] that he took another look at the water to cement ratio versus strength. Though he agreed that the work of Abrams was probably the best known and most influential publication in the history of concrete, he opposed the idea of the inert-filler function of the aggregate. He considered the inert filler concept to be very unfortunate because of its failure to recognize the influence of amount, grading, surface texture, stiffness, and size of aggregate on concrete strength. He conducted tests on concrete of varying sizes of aggregate using 6-inch by 12-inch cylinders. Tests were made with the cylinders in the usual 28-days standard-cured condition. From his test results he concluded that Abrams water to cement ratio pronouncement, as initially formulated, constituted an oversimplification that blocked sound concrete thinking. Based on the data collected during his investigations, Gilkey suggested that the water to cement ratio versus strength pronouncement should be revised to read substantially as follows:

For a given cement and acceptable aggregate, the strength that may be developed by a workable mixture of cement, aggregate and water (under the same mixing, curing, and testing conditions) is influenced by the following:

- (a) the ratio of cement to mixing water
- (b) the ratio of cement to aggregate
- (c) the grading, surface texture and stiffness of aggregate
- (d) the maximum size of the aggregate.

He also concluded that where more than one of the foregoing are appreciably present as simultaneous variables, each of them will exercise a proportionate influence on the strength of the concrete.

Another experimental and analytical study by Cordon and Gillespie [37] focuses on the variables in concrete aggregates and portland cement paste which influence the strength of concrete. Their study was undertaken not only to verify the findings of earlier researchers like Abrams [2], Walker and Bloem [144], and Gilkey [57], but also to find explanations for the apparent refutation of established concepts regarding the water-cement ratio versus strength relationship. They made sixty-nine concrete mixes with water to cement ratios of 0.40, 0.55, and 0.70 by weight. These mixes included No. 4, 3/8, 3/4, 1-1/2, and 3-in. maximum size aggregates (MSA). Strength tests conducted included compressive strength, flexural strength, and triaxial strength. Two 2-inch by 2-in. by 16-in. prisms and two 6-in. by 12-in. cylinders were made for strength tests with each mix containing aggregates up to 3/8-in. Two 3-in. by 3-in. by 16-in. prisms and two 6-in. by 12-in. cylinders were cast with concrete from each mix containing 3/4-in. MSA. One 6-in. by 6-in. by 30-in. beam and two 6-in. by 12-in. cylinders were cast for mixes containing 1-1/2 in. MSA. In agreement with earlier researchers, and from the results of their tests, they upheld the validity of the water to cement ratio law. They concluded that with other things equal, the strength of concrete varies inversely with the ratio of water to cement. They cleverly pointed out that the water to cement ratio law as pronounced by Abrams [2] is not valid if changes are made in the aggregate size and grading and other variables, other than the ratio of water to cement. Their laboratory test results indicated that cylinders tested in unconfined compression failed because of some combination of paste failure, failure of bond between cement paste and aggregate particles, and failure of aggregate

particles. They finally concluded that size and grading of coarse aggregate influenced concrete strength. It was also observed that concrete strength increased with decreasing maximum aggregate size. For all mixes that they tested, they observed a general decreasing pattern in strength with increased aggregate size. Strength was maximum with aggregate size between 3/8 and 1-1/2-in. maximum. They also remarked, in agreement with Walker and Bloem [144], that the size of aggregate has less influence on the strength of concrete having a high water to cement ratio while its influence on rich mixes with low water to cement ratio was remarkably noticeable.

3.2.3 Effect of Moisture Condition

Several concrete researchers like Young [149], Popovic [116], Ramakrishnan [121], and McGhee [95] have established the importance of moisture to strength development in concrete. The first established requirement for the proper curing of concrete is the preservation of an adequate moisture content in concrete following the completion of placing operations.

According to Young [149] and Ramakrishnan [121], regardless of method of cure, complete hydration of cement which requires adequate moisture, is a necessity for strength development during the curing procedure. They are of the opinion that hydration is an integral part of the concrete curing process.

In concrete hardening, the hydration process can be classified into four distinct periods:

- (1) A period of rapid dissolution and exothermic chemical reactions immediately after cement and water are first brought together. It lasts about 15 minutes.
- (2) During the second period, the rate of chemical reactions subsides to a lower level. This is a period of dormancy and plasticity which lasts about 40-120 minutes depending on cement composition.
- (3) The third period of another rapid chemical reaction follows; it usually lasts about 3 to 6 hours. During this period, concrete loses its plasticity and sets. Final set occurs about six hours after mixing. According to McGhee [95] the end of this period marks only 15 percent completion of hydration.
- (4) During the fourth and last period, chemical reaction continues at a diminishing rate until the conditions necessary for hydration no longer exist.

Generally, reaction with water is designated hydration. All the four periods above require adequate moisture. Young [149] remarked that if internal relative humidity drops below about 80 percent as a result of evaporation, self-desiccation, or any form of lack of moisture, hydration will stop, and strength will cease to develop further. It is, therefore, desirable to provide additional moisture during curing to ensure maximum hydration and consequent strength development.

Ramakrishnan [121] and Popovic [116], in their different studies of effect of moisture condition on strength development, have revealed higher strength development in moist-cured concrete than in air cured concrete. The higher

strength was attributed to availability of moisture (in moist-cured) for maximum hydration and strength development. Powers and Brownyard [119] also showed that hydration would cease when chemically combined water became equal to about half the initial mixing water. They further stated that maximum hydration is not assured without an additional source of moisture, particularly when the water to cement ratio is less than 0.5.

Gause and Tucker [51] demonstrated the reduction in relative humidity during self-desiccation. They sealed freshly prepared neat cement pastes in bottles and, from time to time, measured the relative humidity inside the bottle. In all cases they found that the relative humidity, starting at 100 percent, decreased with increasing age. Self-desiccation, a process in which the moisture content of fresh concrete is reduced by its own moisture consuming hydration procedure, has adverse effects on the development of all physical properties of concrete. External sources of moisture have the most influence on strength development through its control in the drying out of concrete in its early life.

In his study of factors that influence concrete strength, Price [120] confirmed the importance of availability of moisture to the strength development in concrete. He also noted that strength development in concrete may continue for a number of years if moisture is available. The later strength, though of little value when the structure has to support full load at early ages, adds to the safety of the structure. Gonnerman [59], Timms [139], and Gilkey [54], in their independent studies of moisture effect on concrete strength development, showed considerable evidence that hydration of cement, which

results in strength development, may cease at any time because of lack of adequate available moisture.

3.2.4 Effect of Curing Method

Optimum curing, as defined by most concrete researchers, is the act of maintaining controlled conditions for freshly placed concrete for some definite period following the placing or finishing operations to assure the proper hydration of the cement and proper hardening of the concrete. Proper curing is a necessity for optimum development of the physical properties of concrete. As discussed in the preceding section, adequate supply of moisture is required for maximum hydration and resultant strength development. The requirements for an acceptable curing method include:

- (1) Preservation of an adequate water content in the concrete
- (2) The maintenance of fairly constant temperature above freezing
- (3) Protection of the structure from damaging mechanical disturbances like load, stresses, heavy shocks, or excessive vibration during the early part of the curing period.
- (4) The passage of adequate time necessary for the hydration of cement and hardening of the concrete to the degree necessary for safe use of the structure.

To fulfil these requirements, the curing methods that are commonly used can be classified as (a) the normal cure which consists of moist-curing and air-dry curing methods, and (b) accelerated cure.

NORMAL CURE: The moist-curing and air-dry curing take place at a temperature that is about equal to the average temperature of the environment to which concrete will be exposed. In the moist-curing method, concrete is kept continuously moist by using one of the following two methods:

(1) **Water curing:** using processes that supply additional moisture as well as prevent moisture loss

(2) **Sealed curing:** processes that prevent loss of moisture only

Supply of water to the concrete can be achieved by spraying, sprinkling, using saturated coverings, or by ponding; while waterproof paper, plastic sheeting, and curing membranes are mostly used for sealed curing.

In the air-dry curing method, concrete is left to cure in air without any sealing and without additional source of moisture other than the moisture that may be in the unsaturated air.

Numerous studies have been undertaken to investigate and compare the strength development in concrete under moist curing and air-dry curing methods. Price [120] in his studies of factors that influence concrete strength, indicated that continuously moist-cured concretes produced higher strength than continuously air-cured concretes at all ages. He also observed that compressive strength increases at a decreasing rate as the moist-curing period increases and that strength development stops at an early age with air-cured concrete. Figure 3-1 obtained from Price [120] shows that the development of strength stops at an early age when the concrete specimen is exposed to dry air with no previous moist curing. Furthermore, it could be observed that concrete exposed to dry

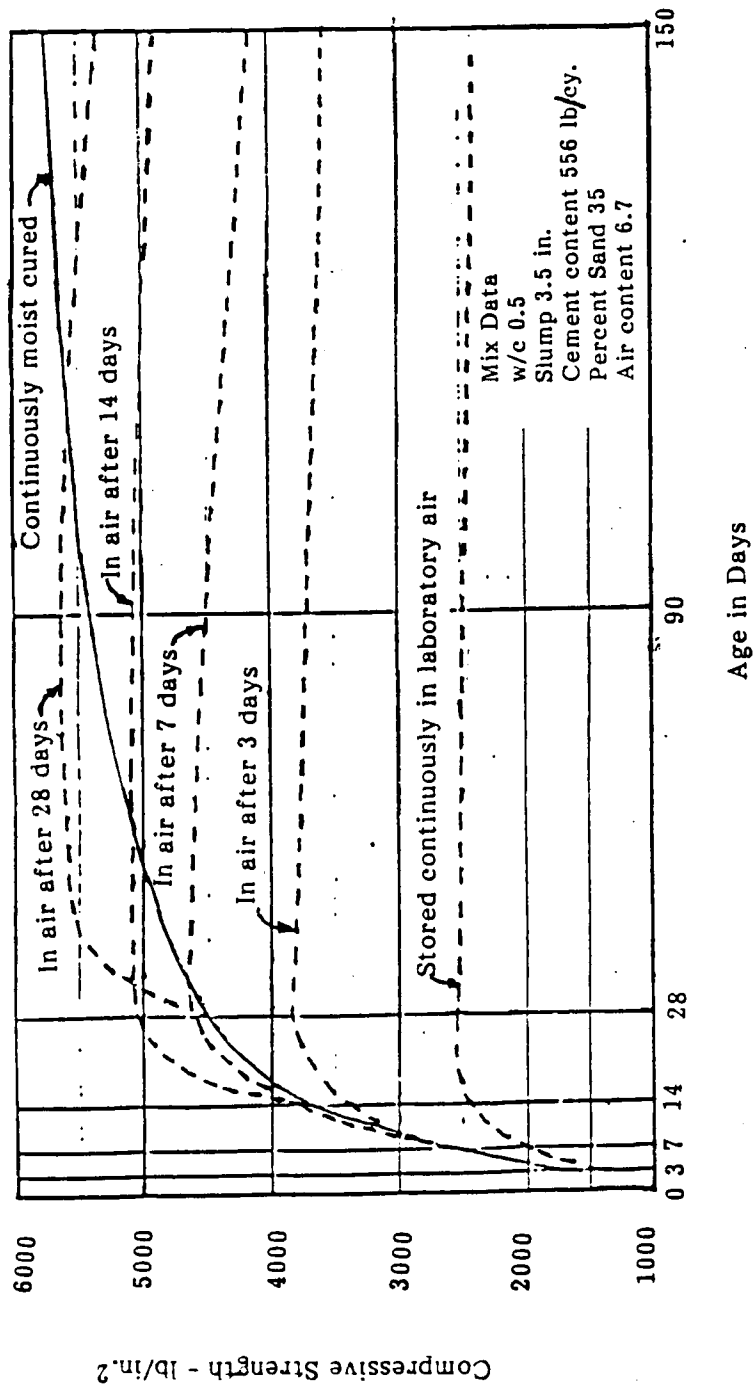


Figure 3-1: Compressive Strength of Concrete Dried in Laboratory Air After Preliminary Moist Curing (Price)

air from the time it is cast is about 42 percent as strong at 6 months as concrete continuously moist-cured. He also inferred that concretes cured in water at about 70 degree Fahrenheit ($^{\circ}\text{F}$) were found to be stronger than those cured in the fog room at 100 percent relative humidity.

Powers [118], in his discussion on cement hydration in relation to curing of cement, revealed the adverse effect of the use of sealing methods to keep the concrete moist during hydration. Sealing only prevents loss of moisture by evaporation. During hydration, combined moisture in concrete is used up, and with time, a significant reduction in the moisture content of concrete results. This reduction in moisture content, known as self-desiccation, has been found by Gause and Tucker [51] and Young [149] to be detrimental to strength development. Powers observed that because of self-desiccation, sealing as a curing method resulted in relatively incomplete hydration which was evident from the lower strength observed in the sealed specimens. He also indicated that sealing should not be used to cure concrete with water to cement ratio less than 0.5 because of the pronounced effect of self-desiccation in this range of water to cement ratio. From the results of his studies, he concluded that only half of the water present in the paste can be used during hydration, even if the total amount of water present is less than the water required for chemical combination. Finally, he observed that in general, moist-cured concrete had higher strength than air-cured concrete of the same mix proportions.

Copeland and Bragg [36] studied self-desiccation in portland cement paste. They prepared cement pastes of various water to cement ratio and subjected them to different curing conditions. Their research showed that the relative

humidity of sealed paste decreased with time of cement hydration; at a water to cement ratio of 0.44, the rate of hydration of sealed specimen dropped below that of moist-cured specimens after the first 10 days. They also observed that sealed specimens required 45 days to reach the same amount of hydration produced in 28 days by moist curing. This effect was observed to be greater at later ages and with concrete of lower water to cement ratio. It was also detected that for water to cement ratios equal to or greater than 0.53, sealed and moist cured specimens hydrated at the same rate throughout the one year test period. Hence, they concluded that if a curing membrane were perfectly impervious, sealed-curing would be just as effective as moist-curing for lean mixes with water to cement ratio equal to or greater than 0.5. This conclusion agreed with Powers' [118] earlier observation discussed above.

A study of methods of curing concrete was conducted by Gonnerman [59]. He observed that air-cured specimens gave strengths about 85 percent of that obtained for water-cured specimens at 7 days. This observed difference in strength became more marked at later ages. From the results of testings performed on specimens moist-cured at various durations, he indicated that strength development increased with duration of moist-curing. The strength development was observed to be rapid at early ages and very slow at duration above seven days.

Shacklock and Keene [130] investigated the compressive and flexural strengths of concrete with and without entrained air. Specimens were cured under standard moist conditions and in laboratory air. Test results showed that the compressive and flexural strengths were about 11 and 30 percent lower for

specimens cured in air of the laboratory and tested dry than for specimens cured in water and tested wet.

In an investigation of the crushing strengths of concrete made with three different cements in Nigeria, Ackroyd and Rhodes [7] studied the age-strength relationship of concrete cured under hot and humid climate. They concluded that concrete cured under hot and humid climatic conditions reached its ultimate strength much earlier than in temperate climates. This was attributed to higher moisture content and higher temperature of the hot and humid climate that accelerates hydration and strength development.

Other researchers like Shober [135], Bellander [22], and Teychenne [138], in their independent studies, agreed that concrete was susceptible to inadequate curing and low strength development when air-cured, particularly rich mixes with water to cement ratios less than 0.5.

ACCELERATED CURE: Accelerated curing is a relatively newer method that is mostly used by the precast, prestressed concrete industry. The relatively recent introduction of precast, prestressed concrete into the different areas of the concrete construction industry has brought a great pressure on the prestressing plants to produce at a faster rate than the rate that normal curing methods (as discussed previously) can accommodate. This demand pressure on the precast, prestressed plants has necessitated an economic daily turn around of molds which can only be achieved through an accelerated curing procedure. The most commonly used method to achieve accelerated curing is the low-pressure steam curing method in which concrete is cured with saturated steam at atmospheric pressure, necessarily at temperatures below 100 degrees

centigrade ($^{\circ}\text{C}$) or 212 degrees Fahrenheit ($^{\circ}\text{F}$). Its primary purpose is to accelerate concrete strength development so that prestressing strands can be released, forms can be stripped and re-used at frequent intervals, and concrete products can be stored or put to use at an early age. Since the inception of the use of steam curing at atmospheric pressure about sixty years ago, considerable experimental work has been undertaken to investigate its effects on concrete strength development.

Among the important observations relating to the effectiveness of steam curing of concrete at atmospheric pressure is that of Shideler and Chamberlin [133]. In their investigation of early strength of concrete as affected by steam curing temperatures, they tested 990 6-in. by 12-in. concrete cylinders steam-cured for various periods and at temperatures ranging from 100 to 212 $^{\circ}\text{F}$. Strength tests were performed at ages ranging from 6 hours to 28 days. Companion specimens moist-cured at 70 $^{\circ}\text{F}$ were also kept and tested for comparison. The first series of tests consisted of steam curing the cylinders immediately after they had been cast; in the second series, the initial steam curing was followed by moist curing at 70 $^{\circ}\text{F}$; and the third series consisted of cylinders that were subjected to preliminary fog curing at 70 $^{\circ}\text{F}$ prior to steam curing. All the five types of cement were used. The results indicated that 28 days of continuous moist curing produced higher 28-day compressive strengths than any of the steam curing or fog and steam curing combinations. It was also found that the apparent advantages of steam curing appeared to be limited to the early age strength development. The results also indicated that the optimum steam curing temperatures were between 130 and 165 $^{\circ}\text{F}$ and that a

presteaming period of 2 to 6 hours, depending upon the temperature, produced strengths 15 to 40 percent higher at the age of 24 hours than when steam curing was started immediately after concrete placement. It was also noted that concrete steam cured at temperatures of 100 to 130°F for periods up to 3 days, followed by moist curing, continued to gain strength under moist curing and had only slightly less strength at 28 days than continuously moist cured specimen. Specimens steam cured at a temperature of 165°F for periods up to 3 days showed little gain in strength during subsequent moist curing and at 28 days had compressive strengths about 75 percent of continuously moist cured specimens. They also revealed that temperatures above 165°F and rapid temperature rise (above 40°F/hour) were detrimental to the strength development in concrete. The effects of steam curing was found to be very advantageous for the first 24 hours; this marked advantage decreased rapidly with age and had little value after 48 hours.

Saul [127, 128] did a summary of the conclusions drawn from experimental works carried out at the Cement and Concrete Association Research Station and elsewhere. He noted that concrete steam cured with a slow initial temperature rise gained strength at first slightly more rapidly and later more slowly than did normal cured concrete. He also observed that concrete steam cured with a rapid initial temperature rise (212°F within two hours from time of mixing) gave an adverse effect on strength beyond 24 hours. Furthermore, strength of concrete raised to 212°F within two hours from time of mixing did not obey the strength versus maturity concept (discussed in the next section). He also recognized the existence of optimum steam curing temperature, and he

attributed the adverse effect of too rapid initial temperature rise on the gain of strength with an adverse effect upon volume change and too rapid rate of hydration.

The effect of steam curing on the important properties of concrete was investigated by Higginson [64]. The investigation was to determine the effects of steam temperatures on compressive strength, modulus of elasticity, durability, and permeability. The results of his study agreed totally with the existence of optimum steam temperature and presteaming period and other observations by Shideler and Chamberlin [133] and by Saul [127]. In addition, he observed that permeability of concrete was increased by steam curing. He observed a variable trend in the results of static modulus of elasticity and Poisson's ratio tests, but generally he noted that modulus of elasticity increased as strength increased and that Poisson's ratio showed nearly the same value for all ages and curing conditions.

Another major study on the optimum steam curing procedure in precasting plants was undertaken by Hanson [60]. He investigated the effect of various steam curing procedures on the compressive strength, direct tensile strength, and elastic properties of concrete. He used steaming procedures that are compatible with the practices of precast concrete industries. Data obtained showed an optimum presteaming period of about 5 hours combined with a temperature rise rate of 40°F per hour up to a constant temperature of 150°F. He also stressed the adverse effects of short presteaming periods in the neighborhood of only one hour. In addition to the observations made by previously discussed researchers, he noted that the effect of variable steam

curing procedures on modulus of elasticity and splitting tensile strengths were similar to those on compressive strength but of lower degree of magnitude. It was also observed that optimum steam curing conditions for modulus of elasticity and tensile strength would differ from those required for optimum compressive strength. He noted that with only one hour presteaming period, regardless of the rate of temperature rise, concrete was susceptible to some horizontal cracking.

Verbeck and Copeland [141], in another investigation, concluded that a longer period of curing at lower temperature leads to a higher optimum strength than when higher temperature is applied for a shorter time. They also indicated that for any one period of curing, there is a temperature which leads to an optimum strength. For example, for a curing period of 4 hours, the optimum temperature is about 78°C (172°F). Komar, a Soviet researcher, in his study of building materials and components, revealed that concrete attains about 60 to 80 percent of its 28-day strength if steam-cured for 10 to 14 hours at a temperature between 80 to 90°C. The United States Bureau of Reclamation (USBR) has also observed that the greatest acceleration in strength gain and a minimum loss in ultimate strength were obtained at temperatures between 130°F and 165°F. This is in agreement with Shideler and Chamberlin [133], Saul [127], and Hansen [60].

In his study of the influence of external factors on compressive strength development, Keong [75] noted that steam curing should only be applied to Portland cement, as high-alumina and supersulphated cement would be adversely

affected by high temperature especially at high pressure. He agreed with all the observations made by the previous researchers herein cited.

3.2.5 Effect of Temperature and Duration of Cure

It has been firmly established by early concrete researchers that strength development of plain concrete is a function of age, curing method, and temperature of cure, all other conditions being equal. In their independent studies on steam curing of concrete, Nurse [109] and Saul [127] have shown that the strength of a given concrete mix is a function of the combined effects of temperature and duration of cure.

Saul [128] summarized the conclusions drawn from experimental works carried out at the Cement and Concrete Association Research Station and elsewhere. He noted that, at normal and fixed temperatures, concrete gained strength with time. Strength gain was observed to be initially rapid, and later progressed more slowly. He also observed that the rate of strength development decreased with decreasing temperature of curing with fixed curing time. He attributed this to the stimulation of hydration rate with temperature increase. It was also noted that concrete cured at much higher temperatures (above 130°F) reached the ultimate strength faster. This ultimate strength was observed to be lower than the value obtained for lower temperature curing. Nurse [109], in his study of steam curing of concrete, agreed with the observations made by Saul [128].

Klieger [78] conducted an investigation into the effect of mixing and curing temperature on concrete strength. He found that the compressive and flexural

strengths obtained from different types of Portland cement concretes initially increased with an increase in initial curing temperature; but the higher the early age curing temperature, the lower the ultimate strength of concrete. It was also noted that the variation of compressive and flexural strengths with temperature for concrete made with different cements were similar in behavior but varied in degree. He observed that high mixing temperature was detrimental to strength development, but noted that the stimulating effect of high temperature of curing outweighed the detrimental effect of high mixing temperature.

From the results of an investigation of the factors influencing concrete strength undertaken by Price [120], it was concluded that concrete continuously cured at higher temperatures had higher strengths at all fixed ages between 1 and 28 days. But high temperature curing resulted into lower strength development at ages over 28 days. Timms and Withey [140] performed an experimental investigation into the temperature effects on compressive strength of concrete. They confirmed the observation that concrete strength development increased with temperatures (above freezing) of curing. McDaniel [94], in his own study, confirmed that concrete strength development increased with time or duration of cure, all other conditions being fixed. He also noted, in agreement with other researchers, that strength development rate was very rapid at the early ages (during the first 7 days after casting), but slower at later ages.

Another important observation was made by Gardner and Poon [50] during their investigation of time and temperature effects on tensile, bond, and

compressive strengths. They noted that temperature influenced tensile and bond strength development of concrete in much the same manner as it does compressive strength development. They also observed, like McDaniel [94] and Saul [128], that strength development increased with increasing curing temperature and that the greater the curing temperature, the greater the eventual ultimate strength provided curing is continuous at temperatures below 100°F.

3.3 The Concept of Maturity

Attempts have been made by early concrete researchers to accurately determine concrete compressive strength at early ages and use the result to predict strength at much later ages. Opinion of researchers vary as to the variables to be considered in evaluating early properties of concrete. While there is agreement that time of cure is the major and undisputable variable parameter in early age strength evaluation, some early age researchers have proposed the use of combined temperature and duration of cure as a better variable. This led to the introduction of the concept of maturity into concrete strength evaluation procedures. The maturity concept, which accounts for the simultaneous effects of both temperature and time of cure on strength development, has been used to predict concrete strength at any age and any curing temperature. Because of its fairly wide usage, the concept of maturity as used by concrete researchers is described in detail below.

Maturity, as simply defined by most concrete researchers, is the summation of the integrals of time and temperature of concrete above a datum

temperature. The datum temperature for maturity may be defined as the curing temperature at which the strength of concrete remains constant irrespective of age. The maturity concept, sometimes referred to as the law of gain of strength with maturity, proposed that concrete of the same mix at the same maturity (reckoned in temperature-time) has approximately the same strength whatever combination of temperature and time make up that maturity. Therefore, maturity (M) is given by the expression

$$M = \int_0^t [T_1(t) - T] dt \quad (3.3)$$

where T_1 is temperature of concrete at time t

T is the datum temperature.

According to the law of strength gain with maturity proposed by early researchers like Plowman [115], strength (S) was found to be directly proportional to maturity (M):

$$S \propto [T_1(t) - T] dt \quad (3.4)$$

In earlier tests before Plowman [115], Saul [128], and Bergstrom [23], the datum temperature was taken as freezing temperature; and according to Plowman, this proved satisfactory until results were compared for concrete cured at different temperatures when agreement was poor. Saul [128] in his study of temperature effects on strength development revealed that strength continued to develop at negative temperatures, down to -10°C (14°F). Bergstrom [23] conducted another study to determine the correct datum temperature, and his findings agreed with Saul that at a temperature of -10°C (14°F), there was no strength development in concrete. Bergstrom [23] also accepted -10°C (14°F) as the

temperature at which concrete strength does not develop. These and other research results like those of Timms and Withey [140] and Wiley [147] led to the acceptance of a value between 11°F and 14°F as the datum temperature (T) in the United States and Canada.

3.4 Previous Observations on Physical Properties at Early Ages

One of the early observations of record on strength development in concrete was made by Plowman [115]. He conducted an investigation to establish the relationship between concrete strength and maturity. After careful examination of strength-maturity experimental results from various sources, he proposed an expression relating the strength at any maturity to that of a given maturity. According to Plowman's equation, the percentage of the strength obtained at a maturity of 35,600 degree Fahrenheit hours (°F. hr.), (28 days at 64°F), is given by the expression

$$A + B \log \left[\frac{\text{Maturity } ^\circ F h}{10^3} \right] \quad (3.5)$$

where A and B are constants which are related linearly to the strength at any age. The datum temperature used was 11°F.

Plowman's equation, though approximately applicable to early and later ages, was claimed to be independent of:

- (1) the quality of the cement
- (2) the water cement ratio
- (3) the aggregate cement ratio
- (4) the curing temperature below 100°F (38°C)

(5) the shape of the test specimen

As shown in Figure 3-2 taken from Plowman [112], the relationship between compressive strength and the logarithm of maturity is claimed to be linear. The claim by Plowman [115] that the proposed equation was independent of the factors listed above came under serious criticism from other early researchers.

It has been well established that water to cement ratio has a very significant influence on strength development; the equation as suggested by Plowman contradicted research findings that water to cement ratio is inversely proportional to rate of strength development.

Price [120] conducted a study of the effect of temperature on strength development and observed that concrete strength development is not independent of temperature even at temperatures below 100°F as claimed by Plowman. Ridley [123], in his investigation into the manufacture of high strength concrete in a tropical climate, confirmed that strength development in concrete is not independent of temperature. He noted that concrete cast in tropical countries has an apparently lower ultimate strength.

Klieger [78], during his study of effect of mixing and curing temperature on concrete strength, revealed that the strength and maturity relation is not independent of curing temperature; at early ages, a higher temperature resulted in a higher strength, but beyond the age of 7 days the situation changed radically. In Klieger's [76] contribution to and assessment of Plowman's equation, along with the accompanying independent factors, he observed that the relation departed considerably from linearity at ages greater than 28 days and that at temperatures of 55°F and lower, the relation may be non-linear even

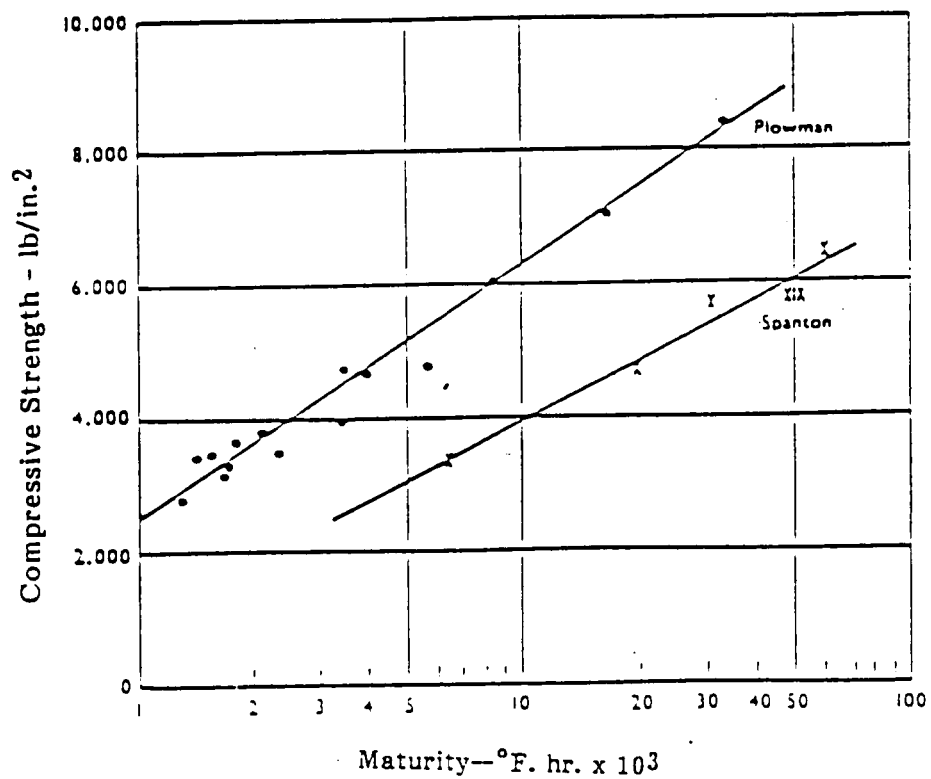


Figure 3-2: Compressive Strength Versus Log Maturity.
Plowman [115]

at early ages. He also noted that Plowman's equation cannot be correct for relatively high temperature curing since the initial period of curing does not become as strong when cured at a lower temperature. Moreover, high temperature curing has been known to result in lower ultimate strengths. In general, as observed by Klieger [76], the higher the temperature, the larger the differences between Plowman's percentage and actual measured percentage even at 28 days or earlier.

Marshall [92], in evaluating Plowman's equation, indicated that maturity as determined from accelerated curing cannot be used in Plowman's equation. Its use resulted in about 12 percent over-estimation of the potential 28-day strength. This showed that maturity as used in Plowman's equation is not independent of temperature and time combination.

Verbeck and Helmuth [142] also indicated that Plowman's equation is not independent of temperature since the rapid initial rate of high temperature curing retards the subsequent hydration. This produces a non-uniform distribution of products of hydration within the paste.

Lew and Reinhard [8] and Narayanan [105], in their studies of properties of concrete, confirmed that Plowman's equation cannot be right because the strength-maturity relation depends on the water to cement ratio. Low water to cement ratio showed higher strength at the same maturity.

Consequent to the series of constructive criticisms attracted by Plowman's equation, he modified his equation as shown below:

Percentage of strength at a maturity of $35,600^{\circ}\text{Fh}$

$$= A + B \log \left[\frac{\text{Maturity}}{10^3} \right] - C \quad (3.6)$$

where $C = 10$ for maturities greater than $35,600^\circ\text{Fh}$

or $C = 0$ for maturities less than $35,600^\circ\text{Fh}$

All other constants are as previously defined. This slight modification attempted to take care of the deficiencies of the equation at later ages, but the effects of water to cement ratio, curing temperature, and quality of cement were not incorporated into the relation. Since then, other researchers have used the maturity concept with some modifications to predict compressive strength at any age including early ages.

Hudson and Steele [67] conducted a study on the developments in the prediction of potential strength of concrete from results of early age tests. Data were obtained from experimental results of 200 cylinders with four different water to cement ratios, moist cured at 73°F and tested at 1, 2, 3, and 28 days. From the regression analysis of their results, they proposed a modified Plowman's equation of the form

$$S_M = S_m + b [\log M - \log m] \quad (3.7)$$

where

S_M is the predicted normal compressive strength at maturity M

S_m is the measured strength at maturity m

M is in degree hours of maturity under moist curing at 73°F (standard conditions)

m is in degree hours ($^\circ\text{F hour}$) of maturity of specimen at time of early test after conditioning ($^\circ\text{F} \times \text{hours}$) + C

C is degree hours ($^{\circ}\text{F}$ hour) of maturity due to autogeneous heating, method of preparation, and residual temperature at time of test

b is the slope of prediction line.

Equation 3.7 is claimed to be perfectly linear and differ from Plowman's in that the line of prediction was projected from the actual value of the early test (S_m) instead of the intercept which was constant in Plowman's equation. For predicting strength at 28 days, equation 3.7 was simplified to

$$S_{28} = S_m + b(4.699 - \log m) \quad (3.8)$$

where 4.699 is logarithm of maturity of the concrete specimen at 28 days and 73°F . Other parameters are as stated in equation 3.7.

When equations 3.7 and 3.8 were applied to concrete of varying water to cement ratio, they were found to be satisfactory for concrete with moderate and high water to cement ratio only. Predictions were unreliable when low strengths could be attributed to cement deficiency--high water to cement ratio. As a result of this, an alternate equation which is parabolic in nature was proposed as

$$S_{28} = a(S_m)^b \quad (3.9)$$

where S_{28} is compressive strength of specimens cured for 28 days under standard conditions,

a is value of intercept of line of relationship with the Y axis on a log-log plot,

S_m is compressive strength of specimen tested at a particular age or maturity and,

b is slope of line of relationship on a log-log plot.

Equation 3.9 was tested and found to be satisfactory for predicting strength of concrete tested at a particular maturity. To obtain a more general relation for general application, they combined equations 3.7, 3.8, and 3.9 to provide an equation of the form

$$\text{Log } S_{28} = 2.9844 + 0.75 \log S_e - 0.51 \log m \quad (3.10)$$

where

S_{28} is predicted 28-day strength

S_e is compressive strength of specimens tested at an early age and having a maturity m , and

M is degree hours of maturity at the time of test.

Equation 3.10, which was simplified to the form

$$S_{28} = 965 \frac{(S_e)^{0.75}}{M^{0.51}} \quad (3.11)$$

was claimed to be independent of the cement content of concrete up to a cement content of about 600 pounds per cubic yard of concrete.

It could be observed that equations 3.7 through 3.11 do not predict strength at early ages directly, but if the 28-day strength is known (as is the case with most design mixes), strength at an early age can be predicted. This can be done by solving for S_e in the equation 3.11; this gives

$$(S_e)^{.75} = \frac{S_{28}(m)^{0.51}}{965} \quad (3.11A)$$

$$S_e = \frac{(S_{28})^{1.33}(M)^{0.68}}{9536} \quad (3.11B)$$

Another important observation about equations 3.7 through 3.11 is that the equations are not truly general. Their applicability is limited to concrete mixes with not more than about 600 pounds of cement per cubic yard of concrete. A more general strength prediction equation without any limit on cement content and with wider range of application is therefore needed.

It should be noted that most of the early investigations of concrete properties at early ages were concerned with the establishment of relations that could predict compressive strength at any age as a function of maturity of a given concrete with known water to cement ratio. Later, concrete researchers have attempted investigations into other physical properties like elastic modulus, splitting tensile strength, and pullout strength at early ages.

Lew and Reichard [87] investigated the mechanical properties of concrete at early ages using the concept of maturity with 10°F as the datum temperature. They investigated compressive strength, modulus of elasticity, splitting tensile strength, and pull out strength. Conventional 6-in. by 12-in. cylinders were used for compressive strength, modulus of elasticity, and tensile strength tests. Tests were made on specimens cured at three different temperatures of 35, 55, and 73°F and at ages of 1, 2, 3, 5, 7, 14, 21, 28, and 42 days. Their results showed that the compressive strength of concrete of a known water to cement ratio can be related adequately to the maturity for each curing temperature. They detected that there was good correlation among regression lines (as shown in Figure 3-3) for the different temperatures; and the possibility of expressing all the data, irrespective of the curing temperature, by a single regression line. Comparison of variances, slope of individual regression lines, and relative

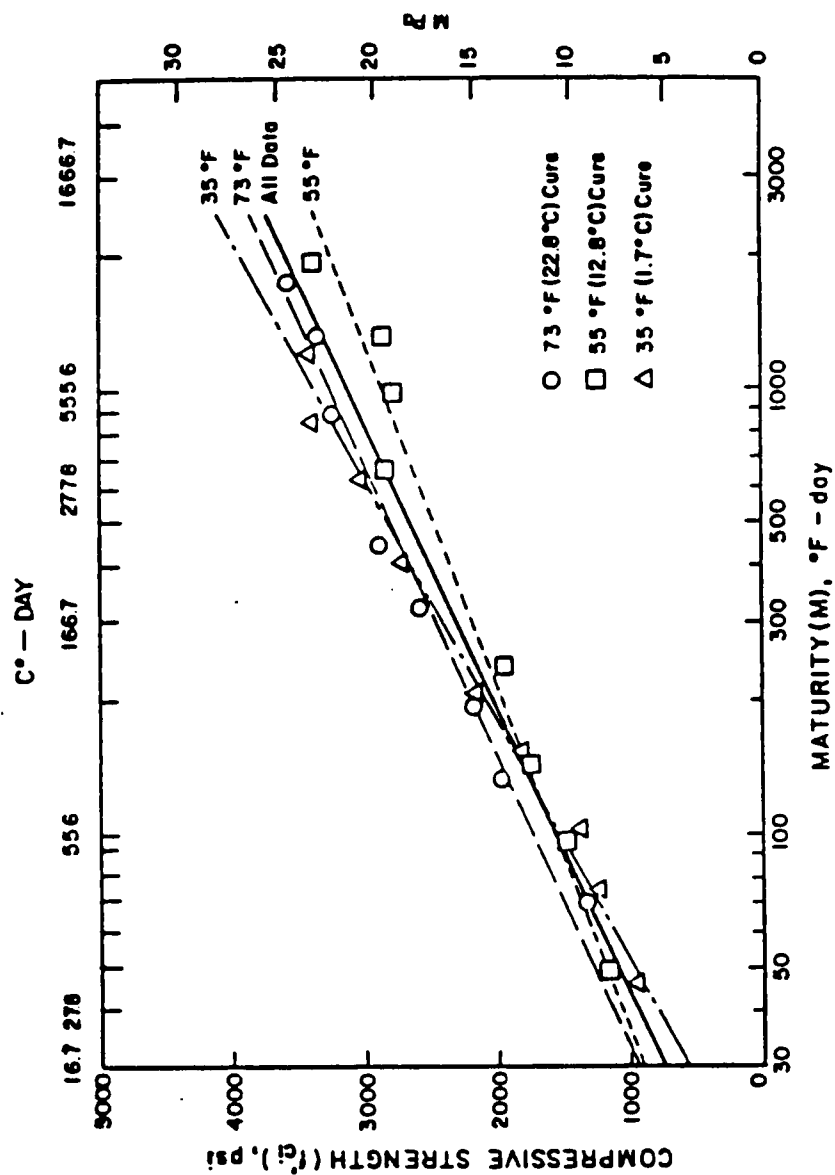


Figure 3-3: Compressive Strength Versus Log of Maturity
(Lew and Reichard)

positions of individual regression lines was done. This confirmed that the use of one regression line was reasonable and quite representative of their experimental results. This observation was also found to be true for elastic modulus and splitting tensile strength data. This justifies the use of 73°F as standard curing temperature. Another observation made was that the rate of increase in the splitting tensile strength is about the same as that of compressive strength. They also noticed that the rate of pull out strength and elastic modulus developments were slightly greater than that of the compressive strength at early ages. It is worth noting that this study did not consider water to cement ratio as a variable, but the paper agreed that the regression strength prediction equation is influenced by the water to cement ratio factor.

Another study by Parsons and Naik [111] on early age concrete strength determination by maturity confirmed that maturity could be used to establish strength prediction equations. Tests were performed on 6-in. by 12-in. cylinders at temperatures between 37°F and 73°F and at outdoor temperature. In agreement with Lew and Reichard [87], data points on a strength-logarithm maturity curve were so close that a single regression line was used as shown in Figure 3-4. They predicted a regression equation that can be used to estimate compressive strength of concrete at early ages, the first 7 days after casting. They further suggested that future research was needed to improve on the existing knowledge on strength development at early ages. They observed that most existing regression equations have an error of about ± 500 pounds per square inch (psi) from the measured strengths. They further suggested that

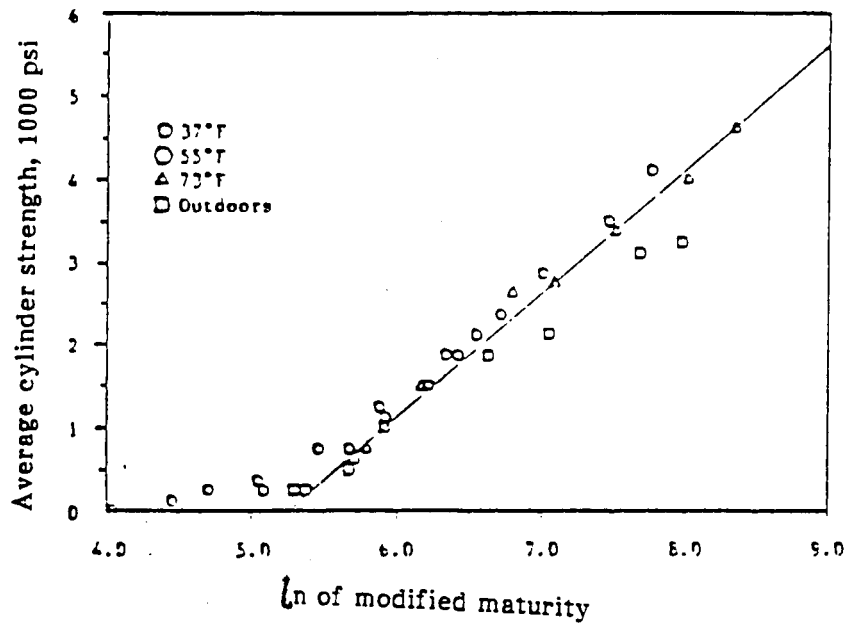


Figure 3-4: Cylinder Strength Versus Ln Modified Maturity for Mixture of Type I Cement, Gravel and w/c = 0.6 (Parsons and Naik)

more investigations on the datum temperature be carried out to ensure more accurate prediction of concrete strength.

Among the most recent investigations of the physical properties of concrete at early ages was the work of Fouad and Furr [43] on the behavior of portland cement mortar in flexure at early ages. Mortar beams of 6-in. by 2-in. by 19-in. were cast and cured at a controlled room temperature. Tests were conducted at ages of 3, 5, 7, 9, and 11 hours from time of mixing. The specimens were loaded in the middle while they were simply supported at the ends. They observed that the average curvature decreased from $149 \times 10^{-5} \text{ in}^{-1}$ at first cracking at 3 hours to a minimum of $27 \times 10^{-5} \text{ in}^{-1}$ at 5 hours test age which corresponds approximately to the setting age of the material. The modulus of elasticity was found to increase rapidly between 3 and 5 hour test ages; thereafter it increased at a much slower rate. They noted that the flexural strength increased slowly between 3 and 5 hours test ages and thereafter it increased at a much faster rate. Generally, they observed that elastic modulus developed more rapidly than the flexural strength during the early ages. These results are very useful in estimating the extent of damage to immature concrete when subjected to early age flexural actions.

3.5 Existing Strength Relations

Starting from the time of Abrams [2], several strength prediction equations have been proposed. These equations range from those that predict strength versus water to cement ratio, 28 day versus 7 day strengths, up to design

equations that relate all other physical strength parameters to the compressive strength at 28 days of moist curing. The validity of these design equations at early ages of concrete are investigated.

Several values have been given to the ratio of 7 day strength (S_7) to the 28 day strength (S_{28}). Abrams [5] stated that S_7/S_{28} varies between 0.4 and 0.5 when concrete is air cured depending on the richness of concrete mix. Young [149] gave an average value of about 0.7 to the same strength ratio for concrete without flyash. Recent research results have indicated some deviations from these previously observed ratio ranges.

Furthermore, Hool and Pulver [67] suggested an equation relating the 7 day strength to the 28 day strength as

$$S_{28} = S_7 + 30 \sqrt{S_7} \quad (3.12)$$

The American Concrete Institute (ACI) building code [8] recommends an empirical relationship between elastic modulus (E_c) and compressive strength (f_c') as

$$E_c = 33\rho^{3/2}(f_c')^{1/2} \quad (3.13)$$

where ρ is the unit weight of concrete. For normal weight concrete of about 145 pounds per cubic feet (lb/ft³), according to ACI

$$E_c = 57,000 [f_c']^{0.5} \quad (3.14)$$

This equation is an indication that E_c is directly proportional to the 0.5 power of the compressive strength. The ACI equation for elastic modulus prediction was suggested by Pauw [113] and is based on accumulated test results from several investigators. Most of the tests covered a strength range of about 2000 to 6000 pounds per square inch (psi).

Carrasquillo, Nilson, and Slate [30] indicated that the ACI equation for predicting elastic modulus resulted in overestimation when applied to medium and high strength concrete. They suggested that

$$E_c = 40,000 [f_c']^{0.5} + 1.0 \times 10^6 \quad (3.15)$$

for normal weight concrete with medium or high strength.

Shah and Ahmad [131] conducted an investigation into the physical properties of high strength concrete. They discovered that the secant modulus of elasticity of medium and high strength concrete does not conform to the conventional ACI formulation. A comparison of experimentally determined values of the secant modulus of elasticity with those predicted by the expression recommended by ACI 318 was done. This revealed that the ACI 318 equation predicted lower and higher values of elastic modulus for concrete under 5000 psi and above 5000 psi, respectively. This agrees with the observation of Carrasquillo et al [28] mentioned earlier. Consequent to this observation, the elastic modulus relation was modified to read

$$E_c = w^{2.5} (\sqrt{f_c'})^{0.65}$$

$$E_c = w^{2.5} (f_c')^{0.325} \quad (3.16)$$

where w is the dry unit weight of concrete. This proposed equation is claimed applicable to both normal and high strength concretes.

In another study done by Klink [81] on aggregates, elastic modulus and Poisson's ratio of concrete, it was indicated that the actual elastic modulus of concrete was much higher than those predicted by any of the strength prediction relations. The result of his study is another indication that the use of ACI 318

expression for elastic modulus results in an underestimation. A substitute expression was proposed as

$$E_c = 14.6 w^{1.75} (f_c')^{0.5} \quad (3.17)$$

for the actual modulus of elasticity evaluation.

Other empirical relations proposed for predicting the elastic secant modulus included

$$E_c = 26 w^{1.5} (f_c')^{0.5} \quad (3.18)$$

proposed by Ahmad in his dissertation, and

$$E_c = 27.55 w^{1.5} (f_c')^{0.5} \quad (3.19)$$

proposed by Jobse and Moustafa [71].

It is worth noting here that some concrete researchers agree that the ACI 318 expression for elastic modulus does not agree with experimentally measured values. The ACI 318 equation underestimates the elastic modulus of normal strength concrete while it overestimates that of high strength concrete.

Experimentally determined moduli of elasticity have been shown [44, 131] to be dependent on properties and proportions of coarse aggregate (all other conditions held constant). It has been shown [131] that the larger the aggregate size, the higher the modulus of elasticity. Wetness of specimen has been known to affect the modulus of elasticity. Other influential factors are methods of strain measurements (strain gage, mechanical compressometer, and transducers).

Equations 3.13 through 3.19 relate the elastic modulus to the compressive strength. This is an indication that all concrete researchers agree that the static modulus can be related to the compressive strength.

The ACI building code [8] represents the relationship between concrete tensile strength (f_t) and compressive strength by the relation

$$f_t = 6[f_c']^{0.5} \quad (3.20)$$

Shah and Ahmad [131] revealed that tensile strengths of concrete with compressive strength between 6,000 and 12,000 psi do not conform to the conventional ACI formulation. They, therefore, proposed an alternate relation for concrete compressive strength up to 12,000 psi as

$$f_t = 4.34 [f_c']^{0.55} \quad (3.21)$$

ACI Committee 363 [9] proposed another relation in the form of

$$f_t = 7.4 [f_c']^{0.5} \quad (3.22)$$

as an upper bound.

Gardner and Poon [50] suggested that splitting tensile strength is not necessarily proportional to the 0.5 power of compressive strength. They proposed that the tensile strength is proportional to the 0.8 power of the cylinder strength with respect to age.

From the detailed literature review discussed in the foregoing sections, it is clear that a considerable amount is known about the physical properties of concrete at maturity (28 days of moist curing). On the contrary, relatively very little is known about concrete physical properties at early ages - within about 72 hours of casting. As mentioned earlier, the relative lack of adequate knowledge about the properties of concrete at early ages can be attributed to a number of factors. These factors include the fact that for a very long time concrete design and construction practices and occupancy requirements did not lead to the need for research into early properties evaluation. Lately, catastrophic

failures during construction, excessive cracking and other serviceability problems have revealed an urgent need for research into the physical properties of concrete at early ages.

It should be recognized that an appreciable amount of research activities have been undertaken in the area of early age physical property evaluation. Most of these efforts have been geared towards adequate early age compressive strength prediction. Efforts have also been made to determine the effect of variation of time and temperature of curing on concrete compressive strength. Also there have been research activities in the realm of elastic modulus variation with duration and temperature of normal standard curing.

To the best of this writer's knowledge, the inter-relationship between the various physical properties of concrete at early ages have not been clearly established. Moreover, the links between the physical properties at early ages and the conventional 28-day properties have yet to be precisely defined.

The relations discussed in section 3.5 of this study, particularly equations 3.13 through 3.19 for prediction of elastic modulus and splitting tensile strength, were primarily derived from concrete tested at maturity (28-day strength). Their applicability to early age strength development has not been very well established.

It has been established that steam curing at atmospheric pressure increases the rate of compressive strength development in concrete. To the best of the writer's knowledge, an accurate comparison of rate of strength development at early ages during steam curing and at 28 days has not been done.

CHAPTER IV

SCOPE OF EXPERIMENTAL INVESTIGATIONS

4.1 Introduction

In the preceding chapter, a comprehensive discussion of previous research results were presented. A literature survey indicated that there is an urgent need to investigate the rate of development of various physical properties at early ages. Such an investigation is important to ensure proper and accurate prediction of these properties and to establish the different relationships between the several physical property parameters at early ages. The sections that follow represent a discussion of the physical property parameters investigated in this study.

4.2 Physical Properties

This study investigates the development of the following physical property parameters at early ages:

- (1) Compressive strength (f_c')
- (2) Modulus of elasticity in compression (E_c)
- (3) Splitting tensile strength (f_t) and
- (4) Poisson's ratio (ν)

These parameters have not been randomly chosen; they have been selected on the basis of their importance to safe concrete design and construction practices. Compressive strength, to which all other physical strength parameters seem to

be related, is considered the most important physical property of concrete; it is this property that influences the operation of concrete prefabrication plants. Release of strands in the precast, prestressed concrete industry without detrimental losses requires a knowledge of compressive strength development. Furthermore, adequate knowledge of compressive strength development is necessary for safe form stripping and early load applications.

The static elastic modulus is another important characteristic of concrete. It is a basic parameter necessary for estimating prestress losses, and immediate and time-dependent deformations due to early loading. While it is generally assumed that concrete performance is mostly governed by its compression capabilities, tensile strength which directly influences cracking at prestress release and shear capacity is important with respect to the appearance and durability of concrete structural members. Variation of tensile strength with time is an important factor in concrete shear strength prediction at different curing times. The variation of static elastic modulus of concrete is also useful in predicting early age buckling loads of long columns. Knowledge of Poisson's ratio is needed in slab-like structures.

Therefore, the variation of these physical property parameters with time of cure is investigated and their inter-relationship at early ages is established. Empirical relations between these properties at early ages are proposed.

To the best of the writer's knowledge, it has not been proven that physical properties development at early ages conforms to the relations discussed in section 3.5 of this study. Therefore, in addition to verifying these relations, this study investigates their conformity with strength development trends at early

ages. Where non-conformity exists, alternate relations are proposed for physical property development at early ages (say within 72 hours after casting).

CHAPTER V

EXPERIMENTAL INVESTIGATION METHODS

5.1 Introduction

In this chapter, methods for the preparation of the concrete mixes used in this study are discussed, including the type and size of aggregate and type of cement. Also highlighted here are the concrete test cylinder fabrication method, curing methods, and testing procedures.

5.2 Materials

Concrete mixes containing Type I cement were used in this study as this is the cement most widely used in practice. Aggregates were selected in accordance with the American Society for Testing and Materials (ASTM) C33 and with due considerations for the effects of aggregate size and gradings on strength as revealed by previous studies. Coarse aggregate used was ASTM Number 67 with 100 percent of the crushed stone less than 1-inch and 90 to 100 percent was about 3/4 inch. The coarse aggregate, supplied by American Limestone Company of Knoxville, had a saturated surface dry bulk specific gravity of 2.80. The fine aggregate used was crushed limestone with sizes varying from 300 to 100 micrometer and specific gravity of 2.60. These aggregates were free of any organic impurities and other deleterious impurities. Pure and drinkable water was used in the mix preparation. The Type I cement used satisfied the requirements of ASTM C150.

In addition to the three mixes prepared in the laboratory, a sample was obtained from a producer of precast, prestressed, concrete; this sample was used for investigation of the effect of steam curing on the development of concrete properties at an early age.

5.3 Fabrication of Test Cylinders

Three concrete mixes were prepared: 4000, 6000, and 9000 pound per square inch (psi) mixes. This preparation was done by varying the water to cement ratio while aggregate proportion and sizes were kept fairly constant. It is the opinion of the writer that the test results obtained from these mixes are quite representative of mixes between 2000 and 10,000 psi at 28 days of standard moist curing. All mixes are standard with no admixtures whatever. Prior to mixing, the aggregate was air dried to a low moisture content. The sand and gravel were mixed for about 2 minutes before adding the cement which was then allowed to mix with the dry aggregates for another 2 minutes before water was added. The concrete was further allowed to mix for about 5 minutes in a 7.5 cubic feet capacity mixer. The slump and air content was measured before casting.

Standard 6 by 12-inch plastic moulds were used. These plastic molds have been chosen in accordance with ASTM C470. The plastic material is non-reactive with cement, strong enough to prevent deformation, and has no absorption quality. Molding was done according to the specifications of ASTM C31. Casting was accomplished in 3 or 4 layers while compaction was done through rodding. All cylinders were struck off level with the top of the mold

immediately after filling and covered with wet burlap and plastic sheeting to prevent loss of moisture during the period of the initial setting of concrete.

The prestressed concrete producer's sample was similarly cast and both moist and steam cured for comparison. These molded specimens were left for about 5 hours before they were steam cured.

Table* 5-1 describes the concrete mix designs used in this study. Concrete mix A is the producer sample; other mixes are laboratory mixed concrete.

5.4 Curing Procedure

Wet burlap was used to cover the molds and specimens until specimens were removed from the molds. A polyethylene sheet was placed over the burlap to minimize evaporation. At about 20 to 24 hours after casting, specimens were stripped and transferred to the standard moist room where curing continued at about 73°F (23°C) in accordance with ASTM C192. Most of the test results on which strength design relations are based were performed on moist cured concrete specimens, hence the choice to use moist curing at 73°F in this study. This way, results obtained and relations proposed from this study will be comparable to the existing physical property relations.

Both moist and steam curings were used for the precast, prestressed producer sample. Steam curing, as practiced by typical prestressing plants, was used. The presteaming period was five hours, and rate of temperature rise was about 40°F per hour up to a maximum temperature of about 130°F. The steaming period was 15 hours.

*Tables 5-1 through 6-23 may be found in Appendix A.

5.5 Testing Equipment

Two testing machines were used, the Tinius Olsen Universal testing machine with a load capacity of 120,000 pounds and the Forney Compression Testing Machine with 300,000 pound load capacity. The Tinius Olsen machine was calibrated in load per voltage output. This enabled the equivalent voltage output of the load to be read directly by the digital data acquisition system. The Tinius Olsen machine was used for loading the specimens during the static modulus of elasticity determination, while the Forney machine was used to break the specimens and for the tensile strength evaluation by split cylinder tests.

The strains were measured with the Structural Behavior Engineering Laboratories Deformation Jacket (DJC). This jacket is equipped with six Linear Variable Differential Transformer Transducers (LVDT). Three of these LVDT's measure the axial strain while the other three measure the lateral strain. These LVDT's are arranged at 60 degree angles from each other. This arrangement of the LVDT's gave the opportunity of getting an average of three readings for strains in each direction. This approach reduces the error in strain measurement and ensures a higher degree of accuracy of physical property evaluation.

Each of the LVDT's was calibrated to displacement per voltage output using the Signal Conditioner. This allowed the input of deformation as voltage output into the digital data acquisition system. Before testing, linearity of displacement versus voltage output of each of the LVDT's was established as shown in Figures* 5-1 through 5-7. From these figures, the sensitivity of each

LVDT was determined and used to convert voltage output back to deformation through the digital data acquisition system.

All the data were read and stored digitally using the data acquisition system. The data acquisition system consisted of the electronic data acquisition plate installed in an IBM personal computer and the Structural Behavior Engineering Laboratories (SBEL) Servo software. The Servo software controlled the rate at which deformations were measured.

5.6 Testing Procedures

A total of about 300 concrete cylinders were tested to determine the variation of the various physical properties with time of cure, particularly at early ages. Table 5-2 is a graphic representation of the test program. This program was of sufficient magnitude to develop trends of various physical property developments in concrete at early ages. Furthermore, the test program is more than adequate either to verify or to invalidate the existing strength prediction relations. Procedures for the various tests performed are summarized below.

5.6.1 Compressive Strength, Elastic Modulus, and Poisson's Ratio

The cylinders that were tested for compressive strength also provided the data for static elastic modulus and Poisson's ratio. Prior to testing, and to prevent stress concentration during testing, the specimens were capped with a proprietary compound containing sulphur in accordance with the specifications

*Figures 5-1 through 6-56 may be found in Appendix B.

of ASTM C617. Compressive strength testing was performed on a 300,000 pound capacity testing machine that is capable of loading at a rate of 20-50 pounds per square inch per second as recommended by ASTM C39. Compressive strength tests were performed in uniaxial compression up to failure.

Simultaneously with the compressive strength test, the longitudinal and lateral deformations were measured by the use of the DJC with the axial and lateral LVDT's. The strains in the two directions were computed using the data acquisition software.

The compressive strength (S) was calculated from the relation

$$S = P/\pi r^2 \quad (5.1)$$

where P is load at failure and

r is the radius of the specimen.

The static elastic modulus (Young's modulus) was calculated from the stress-strain diagram. There are three different ways to do this: the initial tangent method, the secant modulus method, and the chord modulus method. The initial tangent modulus, which is the slope of the tangent to the curve at the origin, applies only to a small portion of the stress-strain diagram. Since it does not apply to an appreciable part of the stresses and strains, it is considered by Young and Mindess [149] to have little practical applicability and importance. The secant modulus, which is the slope of a secant drawn from the origin to any point on the curve, has some elements of non-linearity. Its value depends on the level of applied stress chosen. Both the initial tangent and the secant moduli are not easily determined in compression; furthermore, they do not exclude the effect of closing up pre-existing cracks at the initial stage of load application.

Hence, the chord modulus as recommended by ASTM C469 is used in this study. The chord modulus is the slope of the line drawn between the two fixed points on the stress-strain diagram. The lower point, established to eliminate the effect of cracks on the initial portion of the stress-strain curve, is the point where strain is 0.00005, while the upper point is the point where stress is equal to 40 per cent of the ultimate stress. The chord modulus is calculated from the relation

$$E_c = \frac{S_2 - S_1}{\epsilon_2 - .00005} \quad (5.2)$$

where S_2 is the stress corresponding to 40 percent of the ultimate stress

S_1 is the stress corresponding to a strain of .00005

ϵ_2 is the longitudinal strain corresponding to S_2 .

Poisson's ratio, the ratio of transverse strain to longitudinal strain, is determined from the relation

$$\mu = \frac{\epsilon t_2 - \epsilon t_1}{\epsilon_2 - .00005} \quad (5.3)$$

where ϵt_2 and ϵt_1 are transverse strains at mid-height of the test specimen produced by S_2 and S_1 respectively. S_2 , S_1 , and ϵ_2 are the same as defined in equation 5.2 above.

5.6.2 Splitting Tensile Strength

In general, three kinds of test methods are available for estimating the tensile strength of concrete: the direct tension test, the beam or modulus of rupture test, and the split cylinder or Brazilian test. Although the term "tensile

strength" has no absolute meaning, it is usually expressed in terms of a specific test procedure.

ASTM has no recommended procedure for the direct tensile strength measurement. This can be attributed to the difficulty of ensuring that the load is truly axial in direct tension. Furthermore, and according to McNeely and Lash [96], some eccentricities of load have resulted in underestimation of tensile strength, particularly in brittle materials. The beam test is used to determine the tensile strength, and the maximum fiber stress calculated at failure in such a test is known as the modulus of rupture. Because of the attendant microcracking during testing, the ultimate tensile strength observed is not the same as the stress at which cracking first occurs. L'Hermite [89] has revealed that cracking sounds may be heard by using an ordinary phonograph pick-up attached to the concrete cylinder at about 70 percent of the ultimate strength. Although the modulus of rupture as obtained from the beam test is not representative of the true tensile strength of concrete, concrete researchers like Young and Mindess [149], L'Hermite [89], and Shah and Ahmad [131] have indicated that the true tensile strength is between 65 to 70 percent of the modulus of rupture.

Previous investigations have indicated that the split cylinder method is a more reliable measure of tensile strength than the modulus of rupture beam test. The most important advantage of the split cylinder test is the approximate uniformity of tensile stress over the diametral area of the test cylinder. Moreover, its simplicity affords the opportunity to test a large number of specimen within a relatively shorter period of time.

The split cylinder test is accomplished by loading the test cylinder along two diametrically opposite lines. The load is distributed over a width of about one-twelfth the diameter of the cylinder by a strip of plywood. The cylinder fails suddenly by splitting along a vertical plane. Hondros [66] has shown that, according to the theory of elasticity, the vertical diameter of the test cylinder is a plane of principal tensile stress for about 80 percent of its length and, the stress is approximately constant for about 60 percent of the length.

The split cylinder test was used in this study in accordance with the specifications of ASTM C496. The tensile strength was computed from the relation

$$f_t = \frac{2P}{\pi DL} \quad (5.4)$$

where f_t is the tensile strength in psi

P is the magnitude of load at failure in pounds

D and L are the cylinder diameter and length respectively in inches.

All observations, a summary of test results, and a discussion of test results are presented in the next chapter.

CHAPTER VI

PRESENTATION AND DISCUSSION OF TEST RESULTS

6.1 Introduction

The testing was done on concrete cylinders cast from four different concrete mixes with 28-day compressive strengths ranging from approximately 4000 to approximately 9000 psi. The compressive and tensile strengths, elastic modulus, and Poisson's ratio were measured at various ages. The test for each age was done three times and the average of test results for each of the mixes are summarized in Tables 6-1 through 6-4. The variation of physical property development with maturity is illustrated in Figures 6-1 to 6-3, while variation with age is shown in Figures 6-4 to 6-6.

6.2 Physical Property Development at Early Ages

Tables 6-1 to 6-4 which are illustrated in Figures 6-1 through 6-6 show that each of the physical properties investigated increased with age of concrete. As expected, the richer the concrete mix, the higher the value of each of the physical properties at a given age as shown in Figures 6-3 to 6-6.

One of the main objectives of this study was to determine the rate of development of physical properties of concrete at early ages and to establish their relative relationships. In as much as the 28-day values of these physical properties obtained from concrete moist cured at 73°F are the acceptable standard reference values, the compressive and the splitting tensile strengths and the elastic moduli are normalized by dividing them by their respective 28-

day values from a 73°F cure. These normalized values for each mix are shown in Tables 6-5 to 6-9 and, when plotted against age of concrete as shown in Figures 6-7 to 6-11, can be used to compare the rates of development of each of these physical properties with respect to their 28-day values.

For all the concrete mixes investigated in this study, and as shown in Figures 6-7 to 6-11, the rate of the elastic modulus development is fastest at early ages (first 3 days) but relatively slower than that of both the tensile and the compressive strengths at later ages. This observation is even true for steam cured concrete as illustrated in Figure 6-9. The rate of development of the tensile and compressive strengths are not appreciably different during the later ages.

The variations in the development of the various physical properties of concrete become more obvious when the different property developed is plotted in the form of a bar chart against curing period as shown in Figures 6-12 to 6-14. These plots show the rates of development for the first 3 days using the normalized values of each of the parameters investigated. It could be seen from these figures that the development of physical properties in concrete is fastest within the first 12 hours after casting, specifically, within 6-12 hours for normal and medium strength concrete and 0-12 hours for high strength concrete. In all mixes the elastic modulus developed fastest while the tensile and the compressive strengths developed at about the same rate, generally with less than 5 percent difference.

By the end of the first 12 hours after casting, the percentage of the 28-day value of the elastic modulus developed (as shown in Tables 6-5 to 6-9) varied

from 42.7 in normal strength (4000 psi) concrete through 55.8/64.1 in medium strength (6000-7000 psi) concrete to 70.0 in high strength (about 9000 psi) concrete. Within the same period, the percentage of the 28-day value of the tensile strength developed varied from 15.2 through 28.0 to 47.0 in normal, medium, and high strength concrete, respectively, while that of the compressive strength varied from 16.5 through 27.7 to 43.9 in normal, medium, and high strength concrete, respectively. The differences between the rates of development of each of these physical properties narrowed down with increasing age. After the age of 12 hours, as shown in Figures 6-12 through 6-14, the rate of elastic modulus development decreased faster than those of the compressive and the tensile strengths. The fastest drop in the rate of elastic modulus development occurred between 12 to 24 hours after casting in all mixes. Beyond the age of 12 hours the rates of development of the tensile and the compressive strengths were slightly higher than that of the elastic modulus development. Rate differences continued to narrow down considerably after the age of 24 hours. In fact, by the end of the third day after casting, the preceding percentages for the elastic modulus changed to 69.5, 77.2/81.6, 87.9 for normal, medium and high strength concrete, respectively. In the same period the percentages for the tensile strength became 60.7, 64.7/70.4 and 79.0 while that for the compressive strength rose to 51.1, 60.0/64.6 and 74.6 for the normal, medium and high strength concretes, respectively.

Figures 6-15 through 6-17 compare rates of development of each of the physical strengths as affected by mix richness. Generally, the richer the mix the higher the rate of development of all the physical properties. Figures 6-18

through 6-20, which are bar charts to compare rates of physical property development at early ages as affected by richness of concrete mix, show that the richer the mix the faster the rate of physical property development within the first 12 hours after casting. This period is followed by periods of slower rates of development; and the richer the mix the slower the rate of physical strength development, particularly after the first 2 days of casting. In fact, the rate of the elastic modulus development, which was fastest during the first 12 hours, dropped off most rapidly afterwards. That the rate of strength development is proportional to the richness of the mix is not surprising since rate of hydration on which physical property development depends increases with concrete mix richness.

From the results of this study as shown in Tables 6-1 through 6-4, Poisson's ratio was found to be insensitive to mix proportion, and richness of mix as well as to age of concrete above 6 hours. In all the mixes tested low Poisson's ratios ranging from 0.1316 through 0.1512 were obtained at the age of 6 hours. It is the writer's opinion that whether or not a concrete mix is really concrete at the age of 6 hours is highly debatable. One thing is clear: at 6 hours concrete is still somewhat plastic, and its properties are still very much in their initial formation stages. With this in mind one can conclude from this study that the Poisson's ratio of concrete is relatively insensitive to the age of concrete. This observation about Poisson's ratio agrees with earlier observations made by Higginson [64] and Klink [81].

From the foregoing, it is evident that the values of the physical properties used in concrete design are never fully developed at early ages. The relatively

low physical properties at early ages (as discussed earlier) are accountable for the cracking and other serviceability problems observed when concrete structures are subjected to substantial early age loading. The early age cracking reported in concrete deck panels by Buckner and Turner [27], Fagundo et al [42], Furr and Fouad [47], and the cracking observed at the time of strand release in prestressed deck panels by Barnoff and Rainey [20], Barnoff and Orndorff [19], and the Federal Highway Administration are, in this writer's opinion, all attributable to the relatively low tensile strength of concrete at strand release. Also, the excessive immediate and long term deformations observed by Fu and Gardner [46] and by Shah et al [132] when concrete structures were subjected to substantial early age construction loads are attributable to the relatively low values of the physical properties of concrete at early ages.

6.3 Physical Property Prediction Relations

Several attempts have been made to relate the tensile strength and the elastic modulus to the compressive strength of concrete. Efforts have been made to establish a prediction relation between concrete compressive strength, age, and temperature of cure. Also, prediction of the 28-day strength from the early age strength determinations have been studied. The applicability of some of these relations to concrete at early ages was investigated as discussed in the following sections.

6.3.1 Compressive Strength Prediction

Among the several attempts made at predicting the compressive strength of concrete was the reknowned work of Plowman [115] in which he related the rate of concrete compressive strength development to its maturity. For concrete made from normal Type I portland cement, Plowman represented the relation between the compressive strength developed at a given maturity as a percentage of its 28 day strength and maturity as

$$\frac{S_e}{S_{28}} = A + B \log_{10} \left(\frac{M}{1000} \right)$$

where A and B are constants and

M is the maturity in °F. hr.

S_e is the compressive strength at maturity M

S_{28} is the 28-day compressive strength

Constants A and B obtained from the regression line of S_e/S_{28} versus the logarithm of maturity were determined by Plowman, and recommended values are zoned and tabulated in Table 6-10. Plowman claimed that his equation is completely linear at any maturity and therefore could be applied to concrete regardless of the temperature and time that make up the maturity.

The claim that Plowman's relation is completely linear was disputed by Klieger [76] in his discussion of Plowman's equation. He showed that the relation was essentially linear between the ages of 3 days to 28 days. This observation of non-linearity at early and much later ages was arrived at when Plowman's relation was applied to test results from other sources. Marshall

[92], in his comments on Plowman's equation, agreed with Klieger's observation of non-linearity at early and at much later ages (ages less than 3 days and greater than 28 days). Figure 6-21 obtained from Marshall [92] shows that the relationship between compressive strength and logarithm of maturity is not completely linear. This is contrary to Plowman's claims and is an indication that Plowman's relation is not necessarily applicable to concrete strength prediction at early ages. In fact, Plowman, in his reply to Klieger's and Marshall's contributions agreed that the proposed relation was inaccurate at early ages with low maturity values, particularly at zero maturity.

The observed deficiencies in Plowman's equation, particularly at early ages, requires some modifications to ensure improved accuracy in compressive strength prediction. This study used the maturity concept to propose an alternate relation for strength prediction. Maturity is a function of temperature and age of concrete. At early ages, particularly during the first 24 hours of mixing concrete, the temperature of concrete is not necessarily a function of the ambient temperature alone. To a larger extent it is a function of the exothermic (heat generating) hydration reaction that dominates the curing process at this initial stage. It is therefore more realistic to use the actual concrete temperature at the early ages rather than ambient temperature in calculating the maturities to be used in establishing strength prediction relations that will be applicable to early ages. The actual concrete temperature was measured by three thermocouples embedded in a concrete cylinder during casting. Figure 6-22 shows the variation of concrete temperature with age during the first day of hydration. The richer the mix, the higher the heat

generated during hydration. From Figure 6-22, an estimation of additional maturities in excess of maturities based on 73°F curing temperature was made and tabulated as shown in Table 6-11. The values in this table have been incorporated into the maturities for each testing period for the mixes tested as shown in Tables 6-2 to 6-4.

The alternate equation proposed is of the form:

$$f_{cx} = f_{c'} \left(1 - e^{\frac{-\gamma M}{10000}} \right) \quad (6.1)$$

where f_{cx} is strength at a given maturity, M

$f_{c'}$ is 28 day compressive strength

γ is a constant

Relation (6.1) can be written as

$$\frac{f_{cx}}{f_{c'}} = \left(1 - e^{\frac{-\gamma M}{10000}} \right)$$

from which

$$e^{\frac{-\gamma M}{10000}} = \frac{f_{c'} - f_{cx}}{f_{c'}}$$

and

$$\left[\frac{f_{c'} - f_{cx}}{f_{c'}} \right] = e^{\frac{-\gamma M}{10000}}$$

$$\log \left[\frac{f_{c'} - f_{cx}}{f_{c'}} \right] = \gamma \log e^{\frac{-M}{10000}} \quad (6.2)$$

The slope of the log-log plot of equation (6.2) for each of the mixes gave the constant γ . The values of γ obtained from Figure 6-23 are tabulated below.

Compressive Strength Range (psi)	Mix B	Mix C	Mix D
	2,500 - 5,000	5,001 - 7,500	7,501 - 10,000
γ	1.028	1.457	2.826

The table above can be used to choose the value of the applicable constant γ if the 28 day strength can be reasonably estimated.

Since the real test of any relation is its ability to fit data other than data from which it has been derived, the proposed relation as well as Plowman's relation were applied to data from other sources. The resulting comparison of their accuracies are shown in Tables 6-12 to 6-15. Figures 6-24 through 6-26 show the predicted strength in percentage of 28 day strength for each relation. Figures 6-24 and 6-25 show that at early ages (when maturity is low), especially at ages less than one day, Plowman's relation resulted in negative strength predictions which are clearly inaccurate. The two relations agree reasonably well on predictions between 1 day and 28 days for normal and medium strength concrete as shown in Table 6-12. There are significant differences in the prediction of the two relations for high strength concrete as shown in Figure 6-26, most especially above the age of 1 day. When compared with test results from other sources as shown in Tables 6-13 through 6-15, the proposed and Plowman's relations gave average errors tabulated below:

Average Percentage Error		
Age of Prediction	Plowman	Proposed
1 Day	8.16	6.20
3 Days	5.96	7.10
7 Days	7.62	4.63

This table shows that the errors are within acceptable limits. Furthermore, the proposed relation is slightly more accurate than Plowman's at ages above 1 day and significantly more accurate than Plowman's relation at early ages (below 1 day) since Plowman's relation resulted in negative values at ages below 1 day for normal and medium strength concretes.

It should be stated here that the proposed relation is applicable to concrete up to the age of 28 days only, whereas Plowman's equation has a wider age application. However, in the context of concrete construction practices, it is the gain of strength from placement to 28 days which is important, and the proposed equation covers the age range.

6.3.2 Seven Day and Twenty-eight Day Strengths Relation

It is generally believed by concrete researchers that between 60 and 70 percent of a concrete's 28 day strength is developed in the first 7 days of standard moist cure. The exact value depends to a very large extent on richness of the mix. Hool and Pulver [67] suggested that the relationship between the seven day and twenty-eight day strengths can be represented by the relation

$$S_{28} = S_7 + 30 \sqrt{S_7} \quad (6.2a)$$

where S_{28} is the 28 day compressive strength

S_7 is the 7 day compressive strength

This relation implies that $S_{28} - S_7$ is directly proportional to the 0.5 power of S_7 . To check the validity of this relation, $S_{28}-S_7$ was plotted against S_7 on a log-log scale as shown in Figure 6-28. Correlation of the points with the regression line was poor when compared with the log-log plot of S_{28} against S_7 shown in Figure 6-27. From the properties of the regression line of Figure 6-28, it was established that $S_{28} - S_7$ was proportional to the 0.123 power of S_7 with 2.703 as the constant of proportionality. The regression line of Figure 6-27 gave a better correlation; its slope and intercept are 0.774 and 8.87 respectively. This results in the relations

$$S_{28} = 8.87 (S_7)^{0.774} \quad (6.3)$$

Again, to test the proposed equation (6.3) and compare its accuracy with that of Hools and Pulver, they were both applied to test results from other sources. The comparisons is shown in Table 6-16. The error ranges are shown in the table below:

Error Range (%)		Average Error (%)	
Hool and Pulver	Proposed	Hool and Pulver	Proposed
5.90 - 16.01	0.23 - 5.31	12.52	2.72

From the table above, it can be concluded that the proposed relation $S_{28} = 8.87 (S_7)^{0.774}$ is a better representation of S_7 and S_{28} relationship than equation 6.2a.

6.3.3 Static Modulus of Elasticity

The chord modulus of elasticity is defined as the slope of the chord of the uniaxial stress-strain curve between the point corresponding to 40 percent of maximum stress and the point corresponding to a strain of 0.00005. The chord moduli for all the mixes were experimentally determined for each testing age and recorded as shown in Tables 6-1 through 6-4.

It has been generally accepted by concrete researchers as well as the American Concrete Institute (ACI) that the static elastic modulus of concrete is proportional to the 0.5 power of its compressive strength. This relation was derived from results of tests performed on standard moist cured concrete specimens by many concrete researchers. The study reported herein investigated the validity of the 0.5 power proportionality between the elastic modulus and the compressive strength.

Figure 6-29 is a plot of the static elastic modulus versus the compressive strength on a log-log scale for each testing age. The properties of the regression lines for each testing age are shown in Table 6-17. The slopes of the regression lines revealed that the elastic modulus is in fact proportional to the 0.5 power of the compressive strength at all testing ages except at the age of 6 hours when the elastic modulus was found to be proportional to the 0.794 power of the compressive strength. The constants of proportionality for each age are reasonably constant. They vary slightly between 63,562 and 64,868 except at the age of 6 hours with its constant of proportionality as low as 6,884.

Figure 6-30 is a log-log plot of the elastic modulus versus the compressive strength for each mix. Again, the properties of the regression lines as shown in Table 6-18 upheld the 0.5 power relationship between the elastic modulus and the compressive strength. The regression lines are so close that all the test data, irrespective of mix proportion and age at testing, could be represented by a single regression line as shown in Figure 6-33. The properties of the resulting single regression line still confirmed the 0.5 power relation between the elastic modulus and the compressive strength. It can be seen in Figure 6-31 that there is a poor correlation between the regression line and points corresponding to the age of 6 hours (where the compressive strength was less than 500 psi). Figure 6-32 is the same plot but with points lower than 500 psi compressive strength deleted. The properties of the new regression line which is included in Table 6-18 showed that for the mixes tested the elastic modulus and the compressive strength relation can be represented by:

$$E_c = 63,096 (f_c')^{0.5} \quad \text{for } f_c' \geq 500 \text{ psi} \quad (6.4)$$

Based on the average dry unit weight of the mixes tested

$$E_c = 31.770 W^{1.5} \left[f_c' \right]^{0.5} \quad (6.4a)$$

Further confirmation of the 0.5 power relation between the elastic modulus and the compressive strength was obtained from the plot of elastic modulus versus the square root of compressive strength as shown in Figure 6-33. Again, the slope and intercept of the regression line implied that

$$E_c = 63,313 \left(\sqrt{f_c'} \right)^{0.9973}$$

from which

$$E_c = 63,313 \left(f'_c \right)^{0.4987} \quad (6.5)$$

The 0.5 power relation between the elastic modulus and the compressive strength appears to be valid for elastic modulus prediction for concrete with compressive strength greater than 500 psi at any age.

Comparisons of experimentally determined values of the static elastic modulus with those predicted by the expression recommended by ACI 318-83, Section 8.15, are given for each mix in Figures 6-34 through 6-38. Figure 6-36 for Mix B (a normal strength concrete mix) shows that both the exact and the approximate relations of ACI for predicting the elastic modulus under-estimated the values of the elastic moduli corresponding to compressive strengths between 500 psi and 4100 psi. For the compressive strength values below about 500 psi, the ACI relations over-estimated the elastic modulus values. Figure 6-34 and 6-37 for Mixes A and C (medium strength concrete mixes) show close (<5%) agreement between the measured and the exact ACI predicted values. The only exception was at the testing age of 6 hours in Mix C where there was an overestimation of about 48.2 percent. The approximate ACI relation underestimated the elastic modulus values at all ages except at the age of 6 hours (with compressive strength of 382 psi). Figure 6-38 for Mix D (high strength concrete mix) showed slight over-estimations at all ages by exact ACI formulation while the approximate formulation showed under-estimations.

The percentage over/under estimations associated with both the exact and the approximate formulations of the ACI compared to data obtained in this study are shown in Tables 6-19 through 6-22. The exact formulation showed an average underestimation of about 10.31 percent while the approximate formulation of the ACI showed an average under-estimation of 18.74 percent for mix B (normal strength concrete). For the medium strength concrete, the situation changed to an average of 3.26 percent over-estimation and 9.41 percent under-estimation attributable to the exact and approximate ACI relations, respectively. Mix D (high strength mix) indicated an average of 5.08 percent over-estimation and 8.58 percent under-estimation for the exact and approximate ACI formulations, respectively.

Table 6-23 shows the comparison of measured elastic moduli with the ACI predicted values for tests done by Lew and Reichard [87]. The mix used by Lew and Reichard is comparable to Mix B used in this study. Comparison of the percentage under-estimation for the ages of 1, 2, 3, 7 and 28 days showed close agreement with the data reported herein (Tables 6-20 and 6-23). The difference in percentage under-estimation was less than 5 percent at each age except at the age of 1 day.

When all the elastic modulus data obtained from this study, irrespective of mix proportion and age at testing, are combined and plotted against compressive strength as shown in Figure 6-39, a general comparison with the predictions of the ACI formulations can be made. As illustrated in this figure, the use of the exact ACI formulation resulted in an over-estimation of the elastic modulus of concrete with compressive strength lower than about 1500 psi, a small under-

estimation when compressive strength was between 1500 and 5000 psi, and an over-estimation for compressive strengths in excess of 5000 psi. The approximate ACI formulation consistently resulted in under-estimation of the elastic modulus for concrete with compressive strengths in excess of about 1000 psi.

Figure 6-40 shows that for ages greater than approximately 6 hours, the ratio of the static elastic modulus to the square root of the compressive strength is approximately constant. In fact, this observation is clearly shown in Table 6-17 in which the intercepts of plots of Figure 6-29 were very close.

From this study, it can be concluded that the approximate ACI formulation, which is most widely used for the elastic modulus evaluation, can result in as high as 18.74 percent underestimation or 9.41 percent overestimation in normal and medium/high strength concretes, respectively. Whether or not these percentages are acceptable is debatable and is dependent on the purpose(s) for which the predicted elastic modulus is to be used.

As mentioned earlier, Shah and Ahmad [131] observed that the values of experimentally determined moduli of elasticity depend on the properties and proportion of coarse aggregate. It was observed that with the same consistency and water to cement ratio, the larger the maximum size of coarse aggregate and the coarser the grading, the higher the elastic modulus. It was also noted that the drier the concrete at the time of the test, the lower the elastic modulus--wet concrete is stiffer although often weaker. The method of measuring the deformation (strain gage, mechanical compressometer, and

transducers) has been found to be highly influential on elastic modulus evaluation [131].

Freedman [44], in a different study conducted at the Concrete Technology and Services Section of the Portland Cement Association, agreed completely with the observations of Shah and Ahmad [131]. In addition, he noted that the elastic modulus of concrete increased with increasing modulus of the paste which, in turn, increased with the richness of the concrete mix. It was indicated that concretes may be expected to comply with the ACI formulations only to within ± 20 percent. The maximum average error observed from this study, 18.74 percent, is within this limit for normal strength concrete, and the average error of 9.41 percent for medium/high strength concrete is certainly well within the limits.

Whether or not the limit itself is acceptable should be decided with due considerations for the influential factors discussed earlier. One important observation about these factors is that the elastic modulus increases with increase in aggregate size, whereas Walker [145], Hutchinson [70], Gilkey [57], and Collier [34] have shown that compressive strength decreases with increase in aggregate size. This implies that when proportioning concrete mixes to favor the development of compressive strength (as is usually done), it may be done at the expense of the elastic modulus development. In this study, the coarse aggregate used was Number 67 with 1-inch maximum aggregate size. The use of, for example, $1\frac{1}{2}$ or 2-inch maximum size of coarse aggregate would probably have resulted in higher values of elastic modulus but in lower compressive

strength values. So there is generally a trade-off between the elastic modulus and the compressive strength developments.

It is, therefore, the opinion of this writer, based on the study reported herein, that in view of the many possible variables affecting experimental data on modulus of elasticity, it is likely that any of the relations discussed in Section 3.5 of this study can be reasonably used for elastic modulus estimation and that the development of a more accurate relation is perhaps unwarranted. This study concludes that the ACI formulation for the elastic modulus evaluation are applicable to concretes at any age greater than 6 hours or at any age when the compressive strength is greater than 500 psi.

6.3.4 Splitting Tensile Strength

The tensile strength of concrete, a quantity that affects both resistance to cracking at prestress release and shear capacity, was investigated. The test results for each mix are shown in Tables 6-1 through 6-4.

The simple 0.5 power law relationship between the tensile strength and the compressive strength was investigated. Figure 6-41 shows plots of the tensile strength versus the compressive strength for each testing age on a log-log scale. The properties of the regression lines are shown in Table 6-24. These slopes (R) and intercepts (K) represent the values of the constants at each age in the general relation

$$f_t = K[f_c']^R \quad (6.5a)$$

From the properties of the regression lines, it was determined that the tensile strength is proportional to the 0.79 power of the cylinder compressive strength

at all ages except at the age of 6 hours where the power was found to be 0.72. Figure 6-42 is a plot of tensile strength versus compressive strength for each mix. The properties of the regression lines are shown in Table 6-25 where the 0.79 power relationship between the tensile and the compressive strengths are further confirmed.

The properties of the regression lines of Figures 6-41 and 6-42 are so close that a single regression line could be used for the combination of all the data points regardless of the mix proportions and age at testing. This plot is shown in Figure 6-43. It was observed that points with values of compressive strength less than 1000 psi had poor correlations with the regression line. These points correspond to the data obtained for the 6-hour testing period when the power relationship has been found to deviate slightly from 0.79. With these points deleted from Figure 6-43, the resulting plot is shown in Figure 6-44. The resulting associated regression line indicates that the 0.79 power law is definitely representative of the true relationship between the tensile and compressive strengths of concrete. The properties of the regression line show that the relation at any age can be represented by

$$f_t = 0.584 [f_c']^{0.79} \quad (6.6)$$

for $f_c' > 1000$ psi. Regression analysis of Figure 6-45 shows that the 0.5 power law between the tensile and the compressive strengths is not necessarily valid. The slope of the regression line in Figure 6-45, a log-log plot of tensile strength versus the square root of compressive strength, was found to be 1.590. This implies that

$$f_t \propto (\sqrt{f_c'})^{1.590} = (f_c')^{0.7950} \quad (6.7)$$

If the 0.5 power law had proved to be valid, the slope of Figure 6-45 would have been 1.0.

To confirm further the 0.79 power law, test data from other sources were assembled and plotted as shown in Figure 6-46. The properties of the regression lines are shown in Table 6-26. The slopes and intercepts obtained show that the 0.79 power law is a general representation. Figure 6-48 also confirms that the 0.5 power is not valid.

Figure 6-47, Tensile Strength Versus Compressive Strength (for $f_c' < 1000$ psi), revealed that the power relationship is a little different at ages when the compressive strength is less than 1000 psi. The relationship at these ages could be represented by the relation

$$f_t = 0.928 (f_c')^{0.6} \quad (6.8)$$

where f_c' is in pounds per square inch.

The hypothesis that the ratio of tensile strength to the 0.5 power of the compressive strength is constant at all ages cannot be substantiated from this study or from other data. On the contrary, the ratio of the tensile strength to the 0.79 power of the compressive strength (a plot of which is shown in Figure 6-49) was found to be reasonably constant at an average of 0.6002 at all ages above 6 hours. The 0.6002 value is not substantially different (2.77% difference) from the 0.584 value obtained and used in the formulation of equation (6.6).

The proposed equation (6.6) was applied to data from other sources to test its validity. Table 6-27 shows the comparison of the proposed equation with the ACI relation for tensile strength prediction. The Table below summarizes the errors in each of the relations as obtained from Table 6-27.

Source	Average Error (%)	
	ACI	Proposed
Gardner and Poon	18.13	22.38
Walker and Bloem	17.28	10.88
Lew and Reichard	8.49	10.14
Price	28.76	20.76
Grieb and Warner	17.57	3.62

The total weighted percentage average errors are 17.25 and 9.36 for the ACI and the proposed relations, respectively. This comparison shows a better accuracy (about 45.73 percent better) for the proposed relation given in equation (6.6).

6.4 Effects of Steam Curing

Most concrete researchers like Saul [127], Higginson [64], Price [120], and Hansen [60] are consistent in their findings that steam curing at atmospheric pressure is highly beneficial to the early age compressive strength development. It is also generally accepted by researchers that the early age benefits derived from steam curing are detrimental to strength development at later ages. Opinions vary as to the effects of steam curing on other physical properties like the elastic modulus, tensile strength, and Poisson's ratio. This study investigated the effects of steam curing on all of these physical properties of

concrete. The summary of test results for steam cured Mix A is shown in Table 6-1.

Figure 6-50 shows a comparison of compressive strength development in steam and moist cured concretes. As expected, this figure shows that steam curing significantly accelerated the strength development within the first one day. At the end of the first day (when steam curing was discontinued), the rate of gain in strength in the steam cured concrete diminished. In fact, there was very little (184 psi) gain in strength between the end of the first and the third days. Moist-cured concrete gained strength steadily, and by the third day, it developed more compressive strength than the steam cured concrete. At 7 and 28 days, the moist cured concrete continued to develop higher compressive strengths than the steam cured concrete. This result is in complete agreement with observations of earlier researchers discussed earlier.

Steam curing seems to have a more lasting effect on elastic modulus development than on development of the compressive strength. Figure 6-51 shows that steam curing provided a beneficial effect within the ages of 1 to 3 days. The most increase in the elastic modulus development was observed shortly after steam curing had been discontinued. Then, on the seventh day of curing, the moist cured value of elastic modulus became slightly higher than the steam-cured value. Beyond the age of 7 days, the elastic modulus of the moist cured concrete developed faster than that of steam cured concrete. This fact shows that the benefit of steam curing to the elastic modulus development is limited to the first 3 days and that steam curing is detrimental to the elastic modulus development at ages above 3 days.

The development of tensile strength is the least benefited. As shown in Figure 6-52, steam curing benefited the tensile strength development only up to about 14 hours. From this study, it appears that the beneficial effect of steam curing on tensile strength ceased even before steam curing was discontinued. By the end of the first day (when strands are usually released), the moist cured concrete had developed more tensile strength than the steam cured. Thereafter, and up to 28 days, the moist cured concrete consistently developed higher values of tensile strength than the steam cured. As stated earlier, tensile strength controls resistance to cracking at prestress release. The relatively slow rate of tensile strength development in steam cured concrete may be at least partially responsible for the cracking that has been reported in precast, prestressed bridge deck panels during strand release and form removal.

Using the 28-day moist cured values as standard, Table 6-6 shows the various physical properties developed at each testing age as a percentage of the respective 28-day values. The detrimental effect of steam curing on physical property development is most pronounced on tensile strength as is evidenced by the percentage of the 28-day moist cured values developed by each property.

Taking the moist-cured value for each variable at each testing age as an acceptable standard, the effect of steam curing on each property is shown in Figures 6-53 through 6-55. Points above the reference line represent increase in property development due to steam curing, while points below the reference line are indications of reduction in physical property development due to steam curing. Figure 6-56 shows the increase or decrease in property development

caused by steam curing. This figure shows that the higher the percentage increase in development of a property at early ages, the less lasting the beneficial effect of steam curing.

As shown in Table 6-1, Poisson's ratio is found to be insensitive to steam curing. Generally, the measured Poisson's ratio showed practically the same value for all ages and conditions of cure. This observation agrees with the previous observations of Higginson [64], Hanson [60], and Klink [81].

CHAPTER VII

CONCLUSIONS AND RECOMMENDATIONS

7.1 Conclusions

An experimental investigation of the physical properties of concrete at early ages was conducted. This investigation included the evaluation of the physical properties of elastic modulus, tensile strength, compressive strength, and Poisson's ratio at ages varying from 6 hours to 28 days. Also, the effect of steam curing on these concrete properties was investigated. The results obtained were compared with the predictions of various physical property relations proposed in the literature.

From the analysis of test results, it was found that the relationship between the elastic modulus and the compressive strength is approximately constant from the age of 12 hours. The ACI 318 relation for elastic modulus evaluation was found to be essentially valid at ages 12 hours and above. Contrary to the ACI 318 formulation for tensile strength prediction, the tensile strength was found to be proportional to the 0.79 power of the compressive strength. The relationship proposed (Equation 6.6) for tensile strength prediction is valid for ages 12 hours and above. Thus, the important point is made that the prediction of modulus of elasticity and tensile strength from compressive strength tests is consistently valid for all ages of concrete 12 hours and beyond.

It was found that, in general, all the physical properties developed fastest within the first 12 hours after mixing. This period of relatively rapid physical

property development coincides with the period of most vigorous hydration reaction. It was also found that the elastic modulus developed fastest during the early ages while the tensile strength developed just slightly faster than the compressive strength. It was observed that the faster the early age development of a particular physical property, the slower its development at later ages. Consequently, the elastic modulus with the fastest early age development was found to be slowest in development at ages greater than 3 days. This observation was in agreement with earlier observation of Lew and Reichard [87]. As previously observed by Klink [81] and Higginson [64], Poisson's ratio was found to be insensitive to the age, the richness of concrete mix, and the curing method.

Test results showed that the elastic modulus of concrete is in fact proportional to the 0.5 power of the compressive strength. This 0.5 power relationship between the elastic modulus and the cylinder compressive strength was found to be valid at all ages corresponding to a compressive strength greater than 500 psi. The relations for the elastic modulus prediction (most especially the ACI formulation) discussed in section 3.5 were considered satisfactory in view of the experimental variability in the evaluation of elastic modulus.

The 0.5 power relationship between the splitting tensile strength and the cylinder compressive strength of concrete was found to be inaccurate. A 0.79 power relationship was found to be a much more realistic representation. Therefore, a better tensile strength prediction relation than the ACI relation was formulated and proposed.

Steam curing was found to be beneficial to the development of the elastic modulus, the compressive strength, and the tensile strength only at the early ages. The beneficial effect was found to be least lasting on the tensile strength development. Steam curing was found to be detrimental not only to later age development of the compressive strength but also to the later age development of both the tensile strength and the elastic modulus. Steam curing had no significant effect on Poisson's ratio.

Plowman's relation for predicting compressive strength as a function of maturity was found to be inaccurate at early ages, especially during the first one day of casting. An alternate improved relation that incorporated the heat generated during hydration was formulated and proposed. Hool and Pulver's equation relating the 28 day strength to the 7 day strength was found to be accurate within about 13 percent. More interestingly, the 0.5 power relation between $S_{28} - S_7$ and S_7 did not have good correlation with test data. A more accurate and more representative 0.774 power relation was developed and proposed.

7.2 Recommendations

The investigation reported herein considered three different concrete mixes: 4000, 6000, and approximately 9000 psi concretes (at 28 days). The alternate relation to Plowman's equation proposed herein includes the use of hydration temperature in calculating the maturities used in the relation. The tabulated additional maturities due to hydration were obtained from single mix testings. The accuracy of the proposed relation, although better than Plowman's

at early ages, can be improved by evaluating the hydration temperature of several mixes in each of the following categories: normal, medium, and high strength concrete mixes, and by using the average of each group to evaluate the early age maturities.

Aggregate sizes, most especially of the coarse aggregate, have been found to be highly influential on the compressive strength and the elastic modulus development in concrete. In fact, it has been shown by researchers that as the compressive strength increased with decreasing size of coarse aggregate, the elastic modulus actually decreases. It would be interesting to investigate the possibility of the existence of an optimum balanced aggregate size that equally benefits both the compressive strength and elastic modulus. Moreover, findings about the effects of aggregate size on the compressive strength and the elastic modulus have been determined using the 6 by 12 inch concrete cylinders. One is curious on what the optimum aggregate size would be if different sizes of test specimens such as 4 by 8 inch and 9 by 18 inch test cylinders were used.

Results obtained from steam cured concrete indicated that steam curing could be detrimental to tensile strength development even at very early ages (first day of casting). This fact could be partially responsible for the cracks observed in precast deck panels during form removal and handling in the prestressed concrete plants. To further confirm this observation, a more comprehensive evaluation of the effects of steam curing on the tensile strength of concrete, particularly at early ages, (up to the age of strand release and form removal), would be potentially very useful.

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APPENDICES

APPENDIX A

TABLES

TABLE 5-1
CONCRETE MIX DATA

Mix ID	Cement Type	Quantities Per Cubic Yard of Concrete						Gravimetric Air Content (%)	Water to Cement Ratio	Slump in.
		Water		Cement		Aggregate, lb.				
						Gal.	lb.			
A	I	35.00	292	8.00	752			1,242	1,730	2.0
B	I	32.0	267	3.72	350	1,400	1,900	4.50	0.763	3.50
C	I	32.00	267	5.32	500	1,400	1,900	4.50	0.534	2.50
D	I	37.00	309	10.00	940	1,200	1,900	5.00	0.329	1.50

TABLE 5-2: TEST PROGRAM

TEST CRITERIA	CONCRETE STRENGTH									
	MIX B 4000 PSI MOIST CURED		MIXES A & C 6000 PSI MOIST CURED		MIX D 9000 PSI MOIST CURED		MIX A 6000 PSI STEAM CURED			
	AGE	NO. OF SAMPLES	AGE	NO. OF SAMPLES	AGE	NO. OF SAMPLES	AGE	NO. OF SAMPLES		
COMPRESSION 6" DIA X 12" CYLINDER	6 HR	3 EA	6 HR	3 EA	6 HR	3 EA	8 HR	3 EA		
	12 HR	3 EA	12 HR	3 EA	12 HR	3 EA	14 HRS	3 EA		
	24 HR	3 EA	24 HR	3 EA	24 HR	3 EA	24 HRS	3 EA		
	2 DAYS	3 EA	2 DAYS	3 EA	2 DAYS	3 EA	2 DAYS	3 EA		
	3 DAYS	3 EA	3 DAYS	3 EA	3 DAYS	3 EA	3 DAYS	3 EA		
TENSION (SPLIT CYLINDER) 6" DIA X 12" H CYLINDER	7 DAYS	3 EA	7 DAYS	3 EA	7 DAYS	3 EA	7 DAYS	3 EA		
	28 DAYS	3 EA	28 DAYS	3 EA	28 DAYS	3 EA	28 DAYS	3 EA		
	6 HR	3 EA	6 HR	3 EA	6 HR	3 EA	8 HR	3 EA		
	12 HR	3 EA	12 HR	3 EA	12 HR	3 EA	14 HRS	3 EA		
	24 HR	3 EA	24 HR	3 EA	24 HR	3 EA	24 HRS	3 EA		
ELASTIC MODULUS (SAME SPECIMENS AS COMPRESSION TEST)	2 DAYS	3 EA	2 DAYS	3 EA	2 DAYS	3 EA	2 DAYS	3 EA		
	3 DAYS	3 EA	3 DAYS	3 EA	3 DAYS	3 EA	3 DAYS	3 EA		
	7 DAYS	3 EA	7 DAYS	3 EA	7 DAYS	3 EA	7 DAYS	3 EA		
	28 DAYS	3 EA	28 DAYS	3 EA	28 DAYS	3 EA	28 DAYS	3 EA		
	6 HR	3 EA	6 HR	3 EA	6 HR	3 EA	8 HR	3 EA		
POISSON'S RATIO 3" DIA X 6" H CYLINDER	12 HR	3 EA	12 HR	3 EA	12 HR	3 EA	14 HRS	3 EA		
	24 HR	3 EA	24 HR	3 EA	24 HR	3 EA	24 HRS	3 EA		
	2 DAYS	3 EA	2 DAYS	3 EA	2 DAYS	3 EA	2 DAYS	3 EA		
	3 DAYS	3 EA	3 DAYS	3 EA	3 DAYS	3 EA	3 DAYS	3 EA		
	7 DAYS	3 EA	7 DAYS	3 EA	7 DAYS	3 EA	7 DAYS	3 EA		
	28 DAYS	3 EA	28 DAYS	3 EA	28 DAYS	3 EA	28 DAYS	3 EA		

TABLE 6-1

SUMMARY OF TEST RESULTS FOR CONCRETE MIX A

Age	Steam f_c (psi)	Moist f_c (psi)	Steam E_c (10^6 psi)	Moist E_c (10^6 psi)	Steam f_t (psi)	Moist f_t (psi)	Steam v	Moist v
8 Hrs	1,957	1,548	2.4586	2.2677	243	177	0.1438	0.1374
14 Hrs	3,259	2,942	3.3438	3.2875	336	289	0.1841	0.1876
24 Hrs	4,025	3,596	4.2926	3.7994	346	366	0.1815	0.1839
2 Days	4,111	3,979	4.3537	4.0057	391	405	0.1805	0.1834
3 Days	4,209	4,339	4.4744	4.1840	398	420	0.1807	0.1862
7 Days	4,904	5,176	4.5816	4.5960	444	507	0.1906	0.1948
28 Days	6,198	6,720	4.7224	5.1316	534	597	0.1961	0.1875

TABLE 6-2

SUMMARY OF TEST RESULTS FOR CONCRETE MIX B

Age	Maturity °F. Hr.	Average f_{cx} (psi)	Average E_c (10^6 psi)	Average f_t (psi)	Average ν
6 Hrs	474	188	0.4123	16	0.1316
12 Hrs	934	669	1.9446	69	0.1959
24 Hrs	1,673	1,134	2.3731	155	0.1982
2 Days	3,161	1,768	2.8145	182	0.1990
3 Days	4,649	2,075	3.1370	277	0.1876
7 Days	10,601	2,694	3.5527	305	0.1924
28 Days	41,849	4,060	4.5592	455	0.1856

TABLE 6-3

SUMMARY OF TEST RESULTS FOR CONCRETE MIX C

Age	Maturity F. Hr.	Average f_{cx} (psi)	Average E_c (10^6 psi)	Average f_t (psi)	Average ν
6 Hrs	475	382	0.8465	31	0.1512
12 Hrs	994	1,772	2.7774	193	0.1805
24 Hrs	1,915	2,709	3.2357	281	0.1895
2 Days	3,403	3,466	3.7396	415	0.1907
3 Days	4,891	3,840	3.8448	447	0.1909
7 Days	10,843	5,140	4.4385	563	0.1819
28 Days	42,091	6,402	4.9785	690	0.1901

TABLE 6-4

SUMMARY OF TEST RESULTS FOR CONCRETE MIX D

Age	Maturity °F. Hr.	Average f_{cx} (psi)	Average E_c (10^6 psi)	Average f_t (psi)	Average ν
6 Hrs	527	1,461	2.230	121	0.1440
12 Hrs	1,118	3,882	4.0707	361	0.1814
24 Hrs	2,077	5,211	4.5243	481	0.1886
2 Days	3,565	6,413	5.0901	583	0.1878
3 Days	5,053	6,702	5.1368	607	0.1834
7 Days	11,005	7,268	5.2512	630	0.1832
28 Days	42,253	8,842	5.8440	768	0.1902

TABLE 6-5
INCREMENTAL PHYSICAL PROPERTY DEVELOPMENT FOR MIX A (MOIST CURED)

Age	Compressive Strength			Elastic Modulus			Tensile Strength		
	Compressive Strength (psi)	$\frac{f_{cx}}{f_c'}$ (%)	Incremental $\frac{f_{cx}}{f_c'}$ (%)	Elastic Modulus (10 ⁶ psi)	$\frac{E_{cx}}{E_c}$ (%)	Incremental $\frac{E_{cx}}{E_c}$ (%)	Tensile Strength (psi)	$\frac{f_{tx}}{f_t}$ (%)	Incremental $\frac{f_{tx}}{f_t}$ (%)
8 hrs	1,548	23.03	23.03	2.2677	44.19	44.19	177	29.63	29.63
14 hrs	2,942	43.78	20.75	3.2875	64.06	19.87	289	48.40	18.77
1 day	3,596	53.51	9.73	3.7994	74.04	9.98	366	61.24	12.84
2 days	3,979	59.21	5.70	4.0057	78.06	4.02	405	67.78	6.54
3 days	4,339	64.56	5.35	4.1840	81.53	3.47	420	70.37	2.59
7 days	5,176	77.02	12.46 (3.12/day)	4.5960	89.56	8.03 (2.01/day)	507	84.94	14.57 (3.64/day)
28 days	6,720	100.00	22.98 (1.09/day)	5.1316	100.00	10.44 (0.50/day)	597	100.00	15.06 (0.72/day)

TABLE 6-6
INCREMENTAL PHYSICAL PROPERTY DEVELOPMENT FOR MIX A (STEAM CURED)

Age	Compressive Strength			Elastic Modulus			Tensile Strength		
	Compressive Strength (psi)	$\frac{f_{cx}}{f_c}$ (%)	Incremental $\frac{f_{cx}}{f_c'}$ (%)	Elastic Modulus (10 ⁶ psi)	$\frac{E_{cx}}{E_c}$ (%)	Incremental $\frac{E_{cx}}{E_c}$ (%)	Tensile Strength (psi)	$\frac{f_{tx}}{f_t}$ (%)	Incremental $\frac{f_{tx}}{f_t}$ (%)
8 hrs	1,957	29.12	29.12	2.4586	47.91	47.91	243	40.70	40.70
14 hrs	3,259	48.50	19.38	3.6542	71.21	23.30	336	56.28	15.58
1 day	4,025	59.90	11.40	4.2926	83.65	12.44	346	57.96	1.68
2 days	4,112	61.19	1.29	4.3537	84.84	1.19	391	65.49	7.53
3 days	4,209	62.63	1.44	4.4744	87.19	2.35	398	66.67	1.18
7 days	4,904	72.98	10.35 (2.59/day)	4.5816	89.28	2.09 (0.52/day)	444	74.37	7.70 (1.93/day)
28 days	6,198	92.23	19.25 (0.92/day)	4.7224	92.03	2.75 (0.13/day)	534	89.45	15.08 (0.72/day)

TABLE 6-7
INCREMENTAL PHYSICAL PROPERTY DEVELOPMENT FOR MIX B

	Compressive Strength			Elastic Modulus			Tensile Strength		
	Compressive Strength (psi)	$\frac{f_{cx}}{f_c'}$ (%)	Incremental $\frac{f_{cx}}{f_c'}$ (%)	Elastic Modulus (10 ⁶ psi)	$\frac{E_{cx}}{E_c}$ (%)	Incremental $\frac{E_{cx}}{E_c}$ (%)	Tensile Strength (psi)	$\frac{f_{tx}}{f_t}$ (%)	Incremental $\frac{f_{tx}}{f_t}$ (%)
Age									
6 hrs	188	4.60	4.60	0.4122	9.04	9.04	16	3.50	3.50
12 hrs	669	16.47	11.87	1.9446	42.65	33.61	69	15.24	11.74
1 day	1,134	27.92	11.45	2.3731	52.05	9.40	155	33.98	18.74
2 days	1,768	43.55	15.63	2.8145	61.73	9.68	182	40.00	6.02
3 days	2,075	51.11	7.56	3.1370	69.53	7.80	276	60.71	20.71
7 days	2,694	66.36	15.25 (3.81/day)	3.5527	77.92	8.39 (2.10/day)	305	67.00	6.29 (1.57/day)
28 days	4,060	100.00	33.64 (1.60/day)	4.5592	100.00	22.08 (1.05/day)	455	100.00	33.0 (1.57/day)

TABLE 6-8
INCREMENTAL PHYSICAL PROPERTY DEVELOPMENT FOR MIX C

Age	Compressive Strength			Elastic Modulus			Tensile Strength		
	Compressive Strength (psi)	$\frac{f_{cx}}{f_c'}$ (%)	Incremental $\frac{f_{cx}}{f_c'}$ (%)	Elastic Modulus (10 ⁶ psi)	$\frac{E_{cx}}{E_c}$ (%)	Incremental $\frac{E_{cx}}{E_c}$ (%)	Tensile Strength (psi)	$\frac{f_{tx}}{f_t}$ (%)	Incremental $\frac{f_{tx}}{f_t}$ (%)
6 hrs	382	5.96	5.96	0.8165	17.00	17.00	31	4.18	4.48
12 hrs	1,772	27.68	21.72	2.7774	55.79	38.79	193	28.01	23.53
1 day	2,709	42.32	14.64	3.2357	64.99	9.20	281	40.70	12.69
2 days	3,466	54.14	11.82	3.7396	75.11	10.12	415	60.17	19.47
3 days	3,840	59.98	5.84	3.8448	77.23	2.12	447	64.74	4.57
7 days	5,140	80.30	20.32 (5.08/day)	4.4385	89.15	10.85 (2.71/day)	563	81.59	16.85 (4.21/day)

TABLE 6-9
INCREMENTAL PHYSICAL PROPERTY DEVELOPMENT FOR MIX D

	Compressive Strength				Elastic Modulus			Tensile Strength		
	Compressive Strength (psi)	$\frac{f_{cx}}{f_{c'}}$ (%)	Incremental $\frac{f_{cx}}{f_c}$ (%)	Elastic Modulus (10 ⁶ psi)	$\frac{E_{cx}}{E_c}$ (%)	Incremental $\frac{E_{cx}}{E_c}$ (%)	Tensile Strength (psi)	$\frac{f_{tx}}{f_t}$ (%)	Incremental $\frac{f_{tx}}{f_t}$ (%)	
Age										
6 hrs	1,461	16.52	16.52	2.230	38.16	38.16	121	15.81	15.81	
12 hrs	3,882	43.90	27.38	4.0707	69.66	31.50	361	47.03	31.22	
1 day	5,211	58.93	15.03	4.5243	77.42	7.76	481	62.69	15.66	
2 days	6,413	72.53	13.60	5.0901	87.10	9.68	583	75.93	13.24	
3 days	6,596	74.60	2.07	5.1368	87.90	0.80	607	79.04	3.11	
7 days	7,268	82.20	7.60 (1.9/day)	5.2512	89.86	1.96 (.49/day)	630	82.03	2.99 (.75/day)	
28 days	8,842	100.00	17.80 (.85/day)	5.8440	100.00	10.14 (.48/day)	768	100.00	17.97 (.86/day)	

TABLE 6-10
ZONED CONSTANTS A AND B FOR PLOWMAN'S EQUATION
PLOWMAN [115]

Compressive Strength (psi) After							Constants		Zone
Maturity in °F Hr.	35,600	26,700	17,800	8,900	3,800		A	B	
Age at 64 °F (Days)	28	21	14	7	3				
	0-2,500	0-2,300	0-2,000	0-1,600	0-1,000		-7	68	I
	2,500-5,000	2,300-4,800	2,000-4,000	1,600-3,200	1,000-2,200		6	61	II
	5,000-7,500	4,800-7,100	4,000-6,300	3,200-5,200	2,200-3,800		18	54	III
	7,500-10,000	7,100-9,300	6,300-8,500	5,200-7,000	3,800-6,000		30	46.5	IV

TABLE 6-11
ADDITIONAL MATURITY (°F HR) DUE TO HYDRATION
BASED ON 73°F CURING TEMPERATURE

Age	Compressive Strength Range in psi		
	2500-5000	5,001-7,500	7,501-10,000
6 hrs	102	103	156
12 hrs	190	250	374
1 day	194	428	589
2 days	-	-	624

TABLE 6-12
COMPARISON OF PLOWMAN'S AND PROPOSED EQUATIONS

28-Day Strength	2500 < f_c' ≤ 5000 (psi)				5000 < f_c' ≤ 7500 (psi)				7500 < f_c' ≤ 10,000 (psi)			
	Maturity (°F Hr)	Percent of 28-day Strength		Maturity (°F Hr)	Proposed	Plowman	Percent of 28-day Strength		Maturity (°F Hr)	Proposed	Plowman	Percent of 28-day Strength
Age		Proposed	Plowman									
0	0	0	6	0	0	18.00	0	18.00	0	0	30	0
6 hrs	474	4.76	-20.20	475	6.69	-5.06	6.69	-5.06	527	13.84	10.03	13.84
12 hrs	934	9.15	-1.83	994	13.48	11.06	13.48	11.06	1,118	27.09	24.03	27.09
1 day	1,673	15.80	16.53	1,915	24.35	27.32	24.35	27.32	2,077	44.40	38.03	44.40
2 days	3,161	27.74	34.89	3,403	39.09	43.58	39.09	43.58	3,565	63.49	52.02	63.49
3 days	4,649	37.99	45.63	4,891	50.96	53.09	50.96	53.09	5,053	76.02	60.21	76.02
7 days	10,601	66.37	68.08	10,843	79.40	72.96	79.40	72.96	11,055	95.54	77.32	95.54
14 days	21,017	88.47	86.44	21,259	95.48	89.21	95.48	89.21	21,421	99.76	91.32	99.76
28 days	41,849	98.65	104.81	42,091	99.78	105.47	99.78	105.47	42,253	100.00	105.32	100.00

TABLE 6-13
COMPARISON OF PLOWMAN'S AND PROPOSED EQUATIONS
AT AGE ONE DAY

Source	Measured			Plowman %	Error %	Proposed %	Error %
	S ₂₈ (psi)	S ₁ (psi)	$\frac{S_1}{S_{28}} \times 100$				
Timms and Withey 70°F	7,450	1,460	19.60	26.15	+6.55	21.57	+1.97
	5,290	780	14.74	26.15	+12.58	21.57	+6.83
	4,520	650	14.38	15.21	+0.83	15.80	-1.42
	3,040	290	9.54	16.53	+7.00	15.80	+6.26
Glaniville, Collins and Matthews 65°F	4,100	914	22.29	12.37	-9.92	15.80	-6.49
	4,906	453	9.23	12.37	+3.14	15.80	+6.57
	2,562	277	10.81	12.37	+15.36	15.80	+5.01
Gardner and Poon 72°F	3,540	919	25.96	16.53	-9.43	15.80	-10.16
Hanson 73°F	5,950	1,960	32.94	20.57	-12.37	19.51	-13.43
Shideler and Chamberlin 70°F	4,680	920	19.66	15.21	-4.45	15.80	-3.81

TABLE 6-14
COMPARISON OF PLOWMAN'S AND PROPOSED EQUATIONS
AT AGE THREE DAYS

Source	Measured			Plowman %	Error %	Proposed %	Error %
	S ₂₈ (psi)	S ₃ (psi)	$\frac{S_3}{S_{28}} \times 100$				
Timms and Withey 70°F	7,450	3,710	49.80	51.92	+2.12	50.96	+1.16
	5,290	2,430	45.94	51.92	+5.98	50.96	+5.02
	4,520	1,890	41.81	45.63	+3.82	37.99	-3.82
Glanville 64°F	7,500	3,800	50.67	49.41	-1.26	48.01	-2.66
	6,000	2,700	45.00	49.41	+4.41	48.01	+3.01
	4,000	1,500	37.50	41.48	+3.98	35.18	-2.32
Plowman 60.1°F	8,390	4,730	56.38	55.50	-0.88	63.18	+6.8
Glanville et al 65°F	4,906	1,641	33.45	41.48	+8.03	35.18	+1.73
	4,100	2,045	49.87	41.48	-8.31	35.18	-14.69
	2,562	885	34.54	41.48	+6.94	35.18	-0.64
Gardner and Poon 72°F	3,540	1,632	46.10	45.63	-0.47	37.99	-8.11
Popovic 73°F	6,240	4,530	72.60	53.09	-19.51	50.96	-21.64
	7,950	5,550	69.81	60.21	+9.60	76.02	+6.21
Lew and Reichard 73°F	3,560	1,990	55.89	45.63	-10.26	37.99	-17.91
Shideler and Chamberlin 70°F	4,680	2,250	48.08	44.31	-3.76	37.99	-10.81

TABLE 6-15
COMPARISON OF PLOWMAN'S AND PROPOSED EQUATIONS
AT AGE SEVEN DAYS

Source	Measured			Plowman %	Error %	Proposed %	Error %
	S28 (psi)	S7 (psi)	S7 x 100 S28				
Timms and Withey 70°F 1934	7,450	5,510	73.96	71.80	-2.17	77.83	+3.87
	5,290	4,080	77.13	71.80	+5.33	77.83	+0.70
	4,520	3,000	66.37	68.08	+1.79	66.37	0
	3,040	1,930	63.35	68.08	+4.60	66.37	+3.02
Glaniville 64°F 1948	7,500	5,700	76.00	69.28	-6.72	74.22	-1.78
	6,000	4,300	71.67	69.28	-2.39	74.22	+2.55
	4,000	2,600	65.00	63.93	-1.07	67.35	+2.35
Ridley 74°F 1959	6,604	5,897	89.00	73.33	-15.67	79.83	-9.17
	8,018	7,285	90.80	77.64	-13.16	95.52	+4.72
	5,986	5,117	85.00	73.33	-11.67	79.83	-5.17
Saul 1960 73°F	7,067	5,678	80.00	72.96	-7.04	79.40	-0.60
	8,580	7,170	84.73	77.32	-7.41	95.54	+10.84
Nurse - 1949 64°F	6,050	4,620	76.36	69.28	-7.08	74.22	-2.14
	5,300	3,770	71.13	69.28	-1.85	74.22	-3.09
Ackroyd and Rhodes 80°F 1964	5,110	4,495	88.0	75.46	-12.54	85.88	-2.22
	7,600	7,070	93.0	79.49	-13.54	96.65	+3.65
	6,085	4,840	79.5	75.46	-4.04	85.88	+6.38
	7,600	6,515	85.7	79.49	-6.21	96.65	+10.95
	5,990	4,750	79.3	75.46	-3.84	85.88	+6.58
Popovic 72°F 1986	7,950	7,060	88.8	76.99	-11.81	97.03	+8.23
	6,240	5,280	84.62	72.57	-12.05	78.96	-5.66
	6,230	5,630	90.37	72.57	-17.80	78.96	-11.41
	7,620	7,060	92.65	76.99	-15.66	97.03	+4.38

TABLE 6-15 (Continued)

Source	Measured			Plowman %	Error %	Proposed %	Error %
	S ₂₈ (psi)	S ₇ (psi)	$\frac{S_7}{S_{28}} \times 100$				
Hanson 72°F 1963	5,950	4,820	81.01	72.57	-8.44	78.96	-2.05
Walker and Bloem 73°F 1960	5,665	4,680	82.6	72.96	-9.64	79.40	-3.20
Gardner and Poon 73°F	3,540	2,554	72.15	68.08	-4.07	66.37	-5.78
Lew and Reichard 73°F	3,560	2,590	72.75	68.08	-4.67	66.37	-6.30
Shideler and Chamberlin 70°F 1949	4,680	3,170	67.74	66.77	-0.97	64.84	-2.90

TABLE 6-16
PREDICTION OF 28-DAY STRENGTH FROM 7-DAY STRENGTH

Source	Measured		Hool and Pulver		Proposed	
	S ₂₈ (psi)	S ₇ (psi)	S ₂₈ (psi)	Percentage Error	S ₂₈ (psi)	Percentage Error
Gardner and Poon (1976)	3,540	2,554	4,070	+14.97	3,728	+5.31
Popovics (1976)	4,710	3,518	5,297	+12.47	4,771	+1.30
Ackroyd & Rhodes (1964)	5,420	3,950	5,835	+7.67	5,216	-3.76
	6,085	4,840	6,927	+13.8	6,099	+0.23
Nurse (1949)	5,300	3,770	5,612	+5.90	5,032	-5.06
Ridley (1959)	7,034	5,774	8,054	+14.5	6,987	-0.67
Saul (1951)	8,720	7,220	9,769	+12.03	8,299	-4.83
	8,580	7,270	9,828	+14.5	8,343	-2.76
Timms and Whitney (1934)	5,290	4,080	5,996	+13.3	5,347	+1.08
Hanson (1963)	5,950	4,820	6,903	+16.01	6,080	+2.19

TABLE 6-17
PROPERTIES OF THE REGRESSION EQUATION OF LOG-LOG PLOT
OF ELASTIC MODULUS VERSUS COMPRESSIVE STRENGTH FOR
EACH AGE

Concrete Age	Slope R	Intecept K
6 hrs	0.7935	6,884
12 hrs	0.5028	64,868
1 day	0.5008	64,003
2 days	0.5013	63,652
3 days	0.5028	63,713
7 days	0.4997	63,562
28 days	0.5012	63,768

TABLE 6-18
INTERCEPTS AND SLOPES OF REGRESSION EQUATIONS
OF THE LOG-LOG PLOT OF ELASTIC MODULUS VERSUS
COMPRESSIVE STRENGTH FOR EACH MIX

Concrete Mix	Slope R	Intercept K
A	0.4991	62,661
B	0.5074	60,308
C	0.4928	63,826
D	0.4993	62,738
Combined	0.5019	63,096

TABLE 6-19
COMPARISON OF MEASURED ELASTIC MODULUS WITH ACI PREDICTION
(MIX A)

Concrete Mix A (Moist Cured) Ec in 106 psi						
Age	Compressive Strength (psi)	Ec Measured	Ec Exact ACI	Over/Under Estimation (%)	Ec Approximate ACI	Over/Under Estimation (%)
8 hrs	1,548	2.2680	2.5294	+11.52	2.2420	+1.15
14 hrs	2,942	3.2880	3.4877	+6.07	3.0920	-5.96
1 day	3,596	3.7990	3.8557	+1.49	3.4180	-10.03
2 days	3,979	4.0060	4.0559	+1.25	3.5950	-10.26
3 days	4,339	4.1840	4.2352	+1.22	3.7540	-10.28
7 days	5,176	4.5960	4.6257	+0.065	4.1010	-10.78
28 days	6,720	5.1316	5.2709	+2.71	4.6726	-8.94

TABLE 6-20
COMPARISON OF MEASURED ELASTIC MODULUS WITH ACI PREDICTIONS
(MIX B)

Concrete Mix B (E_c in 10^6 psi)						
Age	Compressive Strength (psi)	E_c Measured	E_c Exact ACI	Over/Under Estimation (%)	E_c Approximate ACI	Over/Under Estimation (%)
6 hrs	188	0.4123	0.8653	+109.87	0.7820	+89.67
12 hrs	669	1.9446	1.6309	-16.13	1.4740	-24.20
1 day	1,134	2.3731	2.1234	-10.52	1.9192	-19.13
2 days	1,768	2.8145	2.6519	-7.09	2.3969	-14.84
3 days	2,075	3.1370	2.8726	-8.43	2.5964	-17.23
7 days	2,694	3.5527	3.2738	-7.85	2.9589	-16.71
28 days	4,060	4.5592	4.0186	-11.85	3.6321	-20.33

TABLE 6-21
COMPARISON OF MEASURED ELASTIC MODULUS WITH ACI PREDICTIONS
(MIX C)

Concrete Mix C (E_c in 10^6 psi)						
Age	Compressive Strength (psi)	E_c Measured	E_c Exact ACI	Over/Under Estimation (%)	E_c Approximate ACI	Over/Under Estimation (%)
6 hrs	382	0.8465	1.2564	+48.42	1.1138	+31.58
12 hrs	1,772	2.7774	2.7063	-2.56	2.3991	-13.62
1 day	2,709	3.2357	3.3461	+3.41	2.9663	-8.33
2 days	3,466	3.7396	3.7853	+1.22	3.3556	-10.27
3 days	3,840	3.8448	3.9846	+3.64	3.5323	-8.13
7 days	5,140	4.4385	4.6102	+3.87	4.0869	-7.92
28 days	6,402	4.9785	5.1445	+3.33	4.5606	-8.39

TABLE 6-22
COMPARISON OF MEASURED ELASTIC MODULUS WITH ACI PREDICTIONS
(MIX D)

Concrete Mix D (E_c in 10 ⁶ psi)						
Age	Compressive Strength (psi)	E_c Measured	E_c Exact ACI	Over/Under Estimation (%)	E_c Approximate ACI	Over/Under Estimation (%)
6 hrs	1,461	2.230	2.5049	+12.33	2.1785	-2.31
12 hrs	3,882	4.0707	4.0831	+0.03	3.5511	-12.76
1 day	5,211	4.5243	4.7314	+4.58	4.1148	-9.05
2 days	6,413	5.0901	5.2484	+3.11	4.5646	-10.32
3 days	6,702	5.1368	5.3231	+3.63	4.6295	-9.88
7 days	7,268	5.2512	5.5872	+6.40	4.8593	-7.46
28 days	8,842	5.8440	6.1626	+5.45	5.3597	-8.29

TABLE 6-23
COMPARISON OF MEASURED ELASTIC MODULUS WITH
ACI PREDICTION [LEW AND REICHARD]

Age (Days)	Compressive Strength (psi)	Ec (106 psi) Measured	Ec (106 psi) Approx. ACI	Over/Under Estimation (%)
1	1,330	3.32	2.078	-37.39
2	1,990	3.03	2.543	-16.08
3	2,190	3.28	2.667	-18.68
5	2,590	3.76	2.901	-22.85
7	2,890	3.66	3.064	-16.28
14	3,230	3.65	3.239	-11.25
21	3,330	3.91	3.289	-15.88
28	3,560	4.48	3.401	-24.09

TABLE 6-24
PROPERTIES OF THE REGRESSION EQUATIONS OF FIGURE 6-41

Age	Slope R	Intecept K
6 hrs	0.7169	0.485
12 hrs	0.7749	0.573
1 day	0.7810	0.607
2 days	0.7842	0.603
3 days	0.7917	0.623
7 days	0.7873	0.612
28 days	0.7922	0.626

$$f_t = K[f'c']^R$$

TABLE 6-25
PROPERTIES OF THE REGRESSION EQUATIONS OF FIGURE 6-42

Concrete Mix	Slope R	Intercept K
A	0.7818	0.60
B	0.7925	0.510
C	0.7979	0.532
D	0.7820	0.576
Combined	0.790	0.584

$$f_t = K[f'c']^R$$

TABLE 6-26
 PROPERTIES OF THE REGRESSION EQUATIONS OF FIGURE 6-46

Source	Slope R	Intercept K
Gardner & Poon	0.8098	0.647
Walker & Bloem	0.7894	0.542
Len & Reichard	0.7951	0.625
Price	0.7673	0.581
Grieb & Werner	0.7854	0.612

$$f_t = K[f_c]^R$$

TABLE 6-27
COMPARISON OF ACI AND PROPOSED TENSILE STRENGTH RELATIONS

Source	Measured		ACI Equation		Proposed Equation	
	Compressive Strength (psi)	Tensile Strength (psi)	Tensile Strength (psi)	Percentage Error	Tensile Strength (psi)	Percentage Error
Gardner and Poon (1976) 72°F	919	159	181.90	+14.40	128.07	-19.45
	1,632	272	242.40	-10.88	201.59	-25.89
	2,554	386	303.22	-21.45	287.16	-25.61
	3,048	423	331.30	-21.68	330.21	-21.94
	3,540	459	356.99	-22.22	371.64	-19.03
Walker and Bloem (1960) 72°F	2,780	389	316.35	-18.68	307.05	-21.07
	2,910	411	323.67	-21.25	318.34	-22.55
	2,980	426	327.54	-23.11	324.40	-23.85
	5,035	504	425.75	-15.53	490.89	-2.60
	5,090	528	428.07	-18.93	495.13	-6.23
	4,915	510	420.64	-17.52	486.63	-4.58
	4,675	517	410.24	-20.65	462.96	-10.45
	5,665	485	451.60	-6.89	538.82	+11.10
	5,545	516	446.79	-13.41	529.78	+2.67
	5,100	515	428.49	-16.80	495.90	+3.71
Lew and Reichard (1978) 72°F	1,330	200	218.81	+9.41	171.50	-14.25
	1,990	251	267.70	+6.65	235.78	-6.06
	2,190	292	280.78	-3.84	254.31	-12.91
	2,590	312	305.35	-2.13	290.35	-6.94
	2,890	329	322.56	-1.96	316.61	-3.77
	3,230	386	340.99	-11.66	345.69	-10.44
	3,330	406	346.24	-14.72	354.11	-12.78
	3,560	434	357.99	-17.51	373.30	-13.99
Price (1951) 70°F	1,000	110	189.74	+72.49	136.90	+24.45
	2,000	200	268.30	+34.15	236.72	+18.36
	3,000	275	328.63	+19.50	326.09	+18.58
	4,000	340	379.47	+11.61	409.30	+20.38
	5,000	400	424.26	+6.07	488.20	+22.05

TABLE 6-27 (CONTINUED)

	Measured		ACI Equation		Proposed Equation	
Source	Compressive Strength (psi)	Tensile Strength (psi)	Tensile Strength (psi)	Percentage Error	Tensile Strength (psi)	Percentage Error
Grieb and Warner 1962 73°F	4,120	450	385.12	-14.42	418.97	-6.90
	4,680	495	410.46	-17.08	463.35	-6.39
	5,150	495	430.58	-13.01	499.74	-0.96
	5,380	505	440.09	-12.85	517.29	+2.43
	5,640	505	450.60	-10.77	536.94	+6.32
	5,650	515	450.99	-12.43	537.69	+4.41
	5,020	510	425.11	-16.65	489.74	-3.97
	5,010	525	424.68	-19.11	488.97	-6.86
	5,600	525	448.99	-14.48	533.93	+1.70
	5,560	525	447.39	-14.78	530.91	+1.13
	5,770	545	455.76	-16.37	546.69	+0.31
	5,620	540	449.80	-16.70	535.43	-0.85
	5,890	545	460.48	-15.51	555.66	+1.96
	5,770	550	455.76	-17.13	546.69	-0.60
	5,810	555	457.34	-17.60	549.68	-0.96
	5,500	560	444.97	-20.54	526.38	-6.00
	5,980	560	463.98	-17.15	562.34	+0.42
	6,110	565	468.99	-16.99	571.99	+1.24
	6,200	580	472.44	-18.54	578.63	-0.24
	6,450	615	481.87	-21.65	596.99	-2.93
	6,140	620	470.15	-24.17	574.20	-7.39
	6,050	625	466.69	-25.33	567.54	-9.19
	5,940	605	462.43	-23.57	559.38	-7.54
	5,640	595	450.60	-24.27	536.94	-9.76
	6,160	575	470.91	-18.10	575.68	+0.12

APPENDIX B

FIGURES

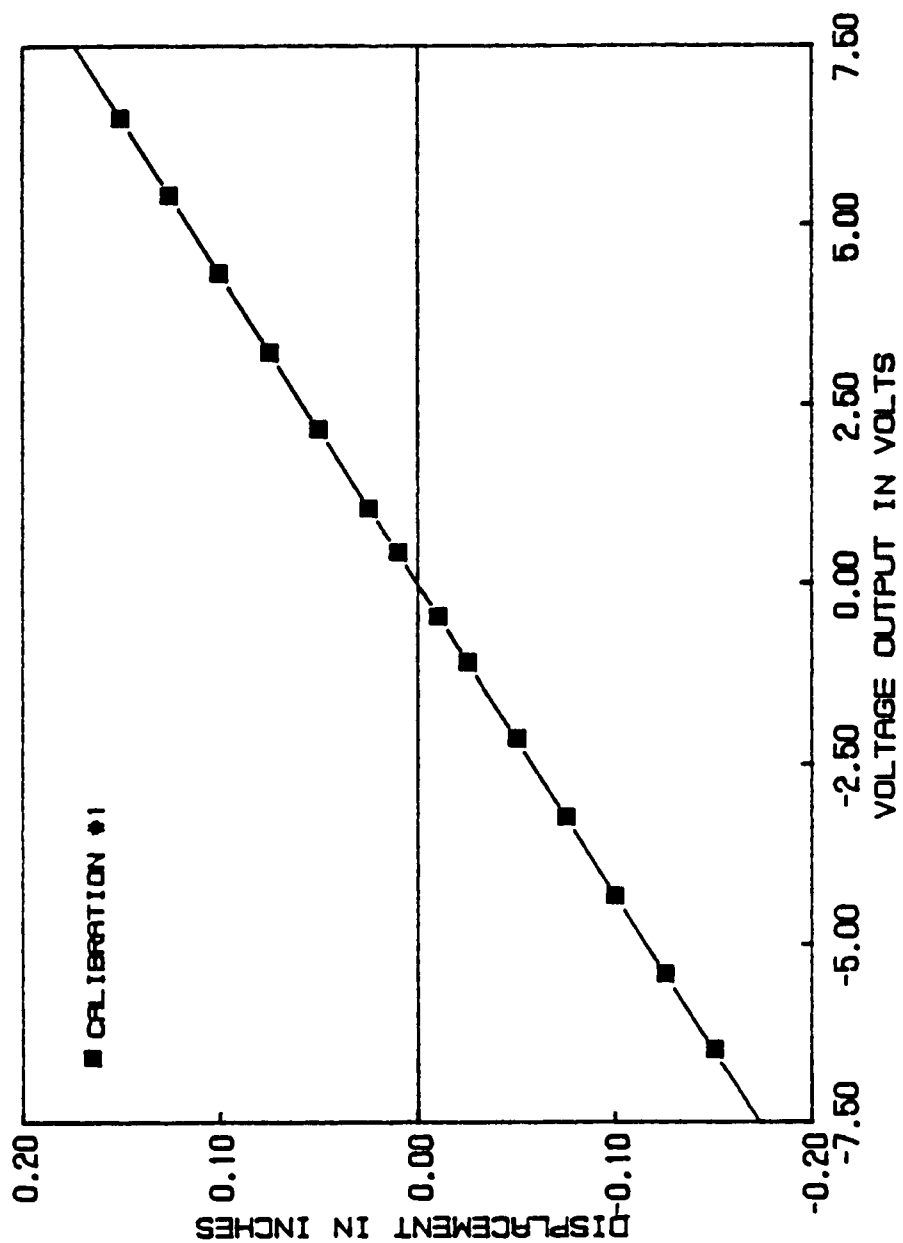


Figure 5-1: Calibration of Transducer 1

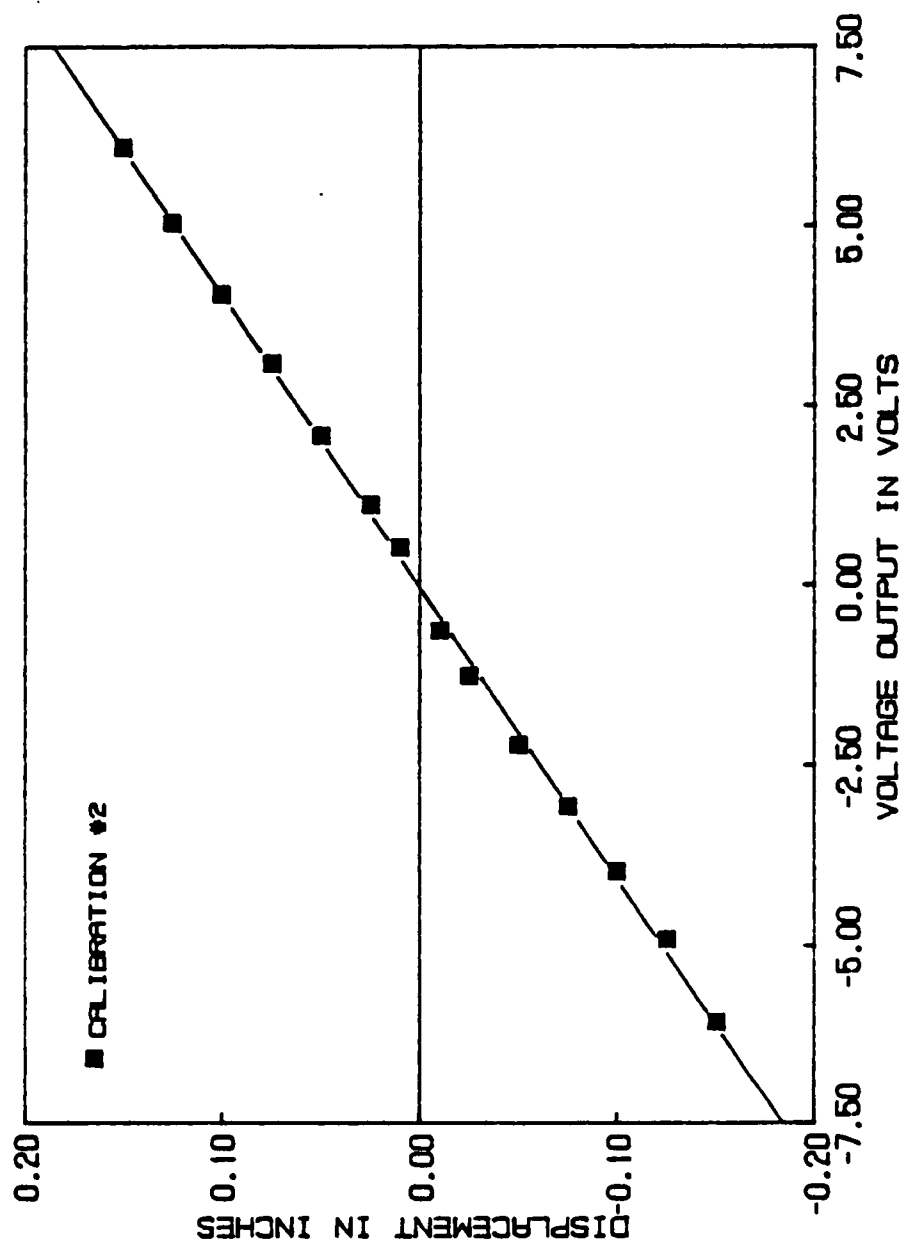


Figure 5-2: Calibration of Transducer 2

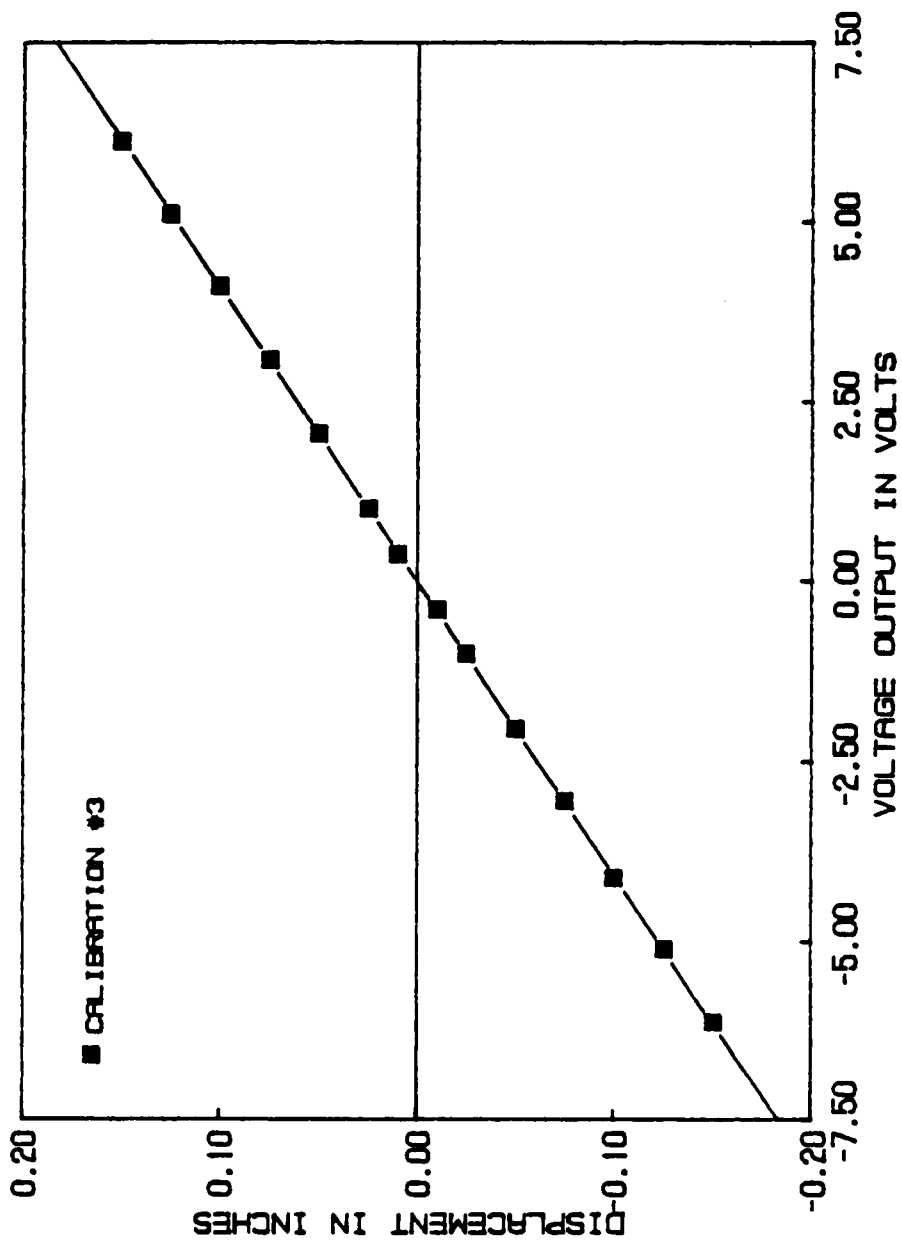


Figure 5-3: Calibration of Transducer 3

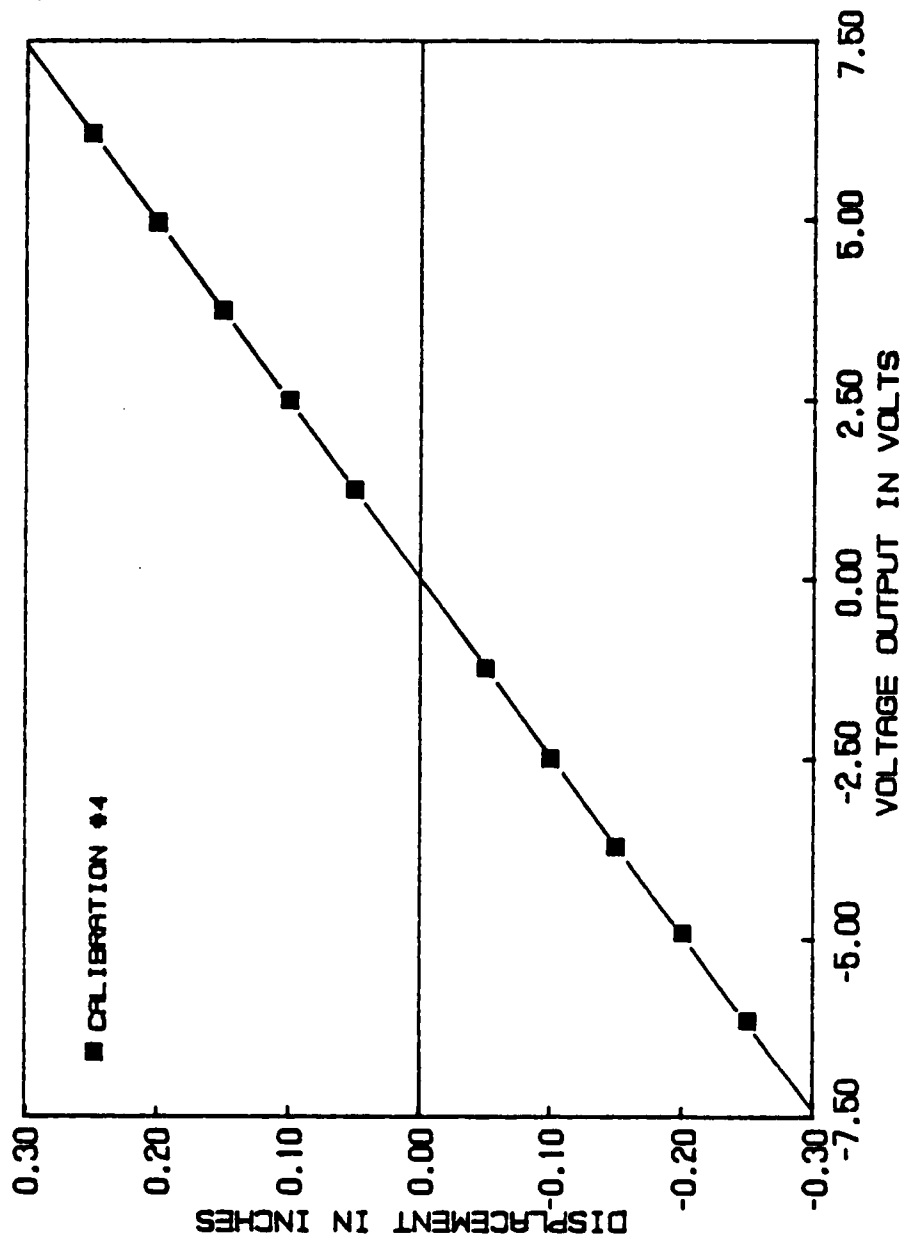


Figure 5-4: Calibration of Transducer 4

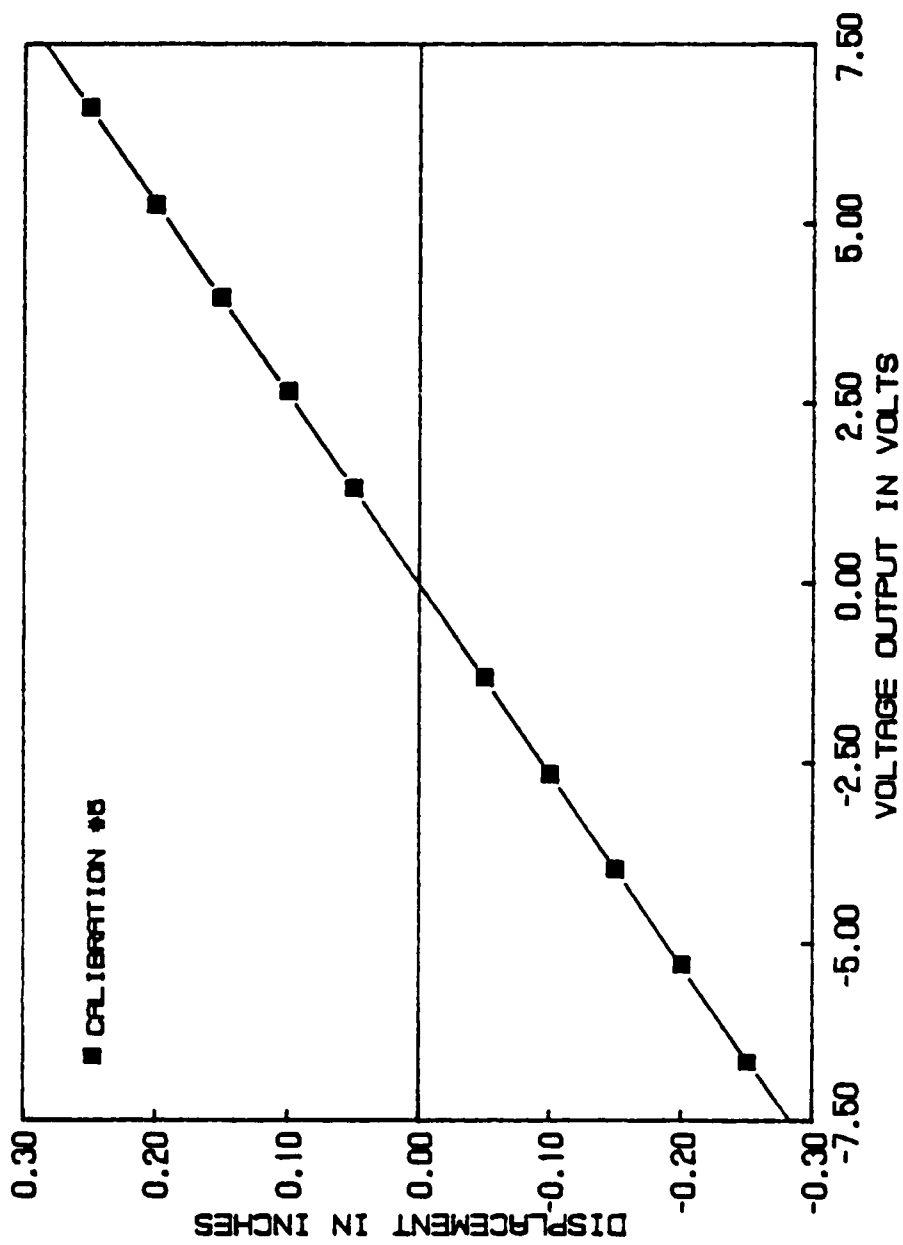


Figure 5-5: Calibration of Transducer 5

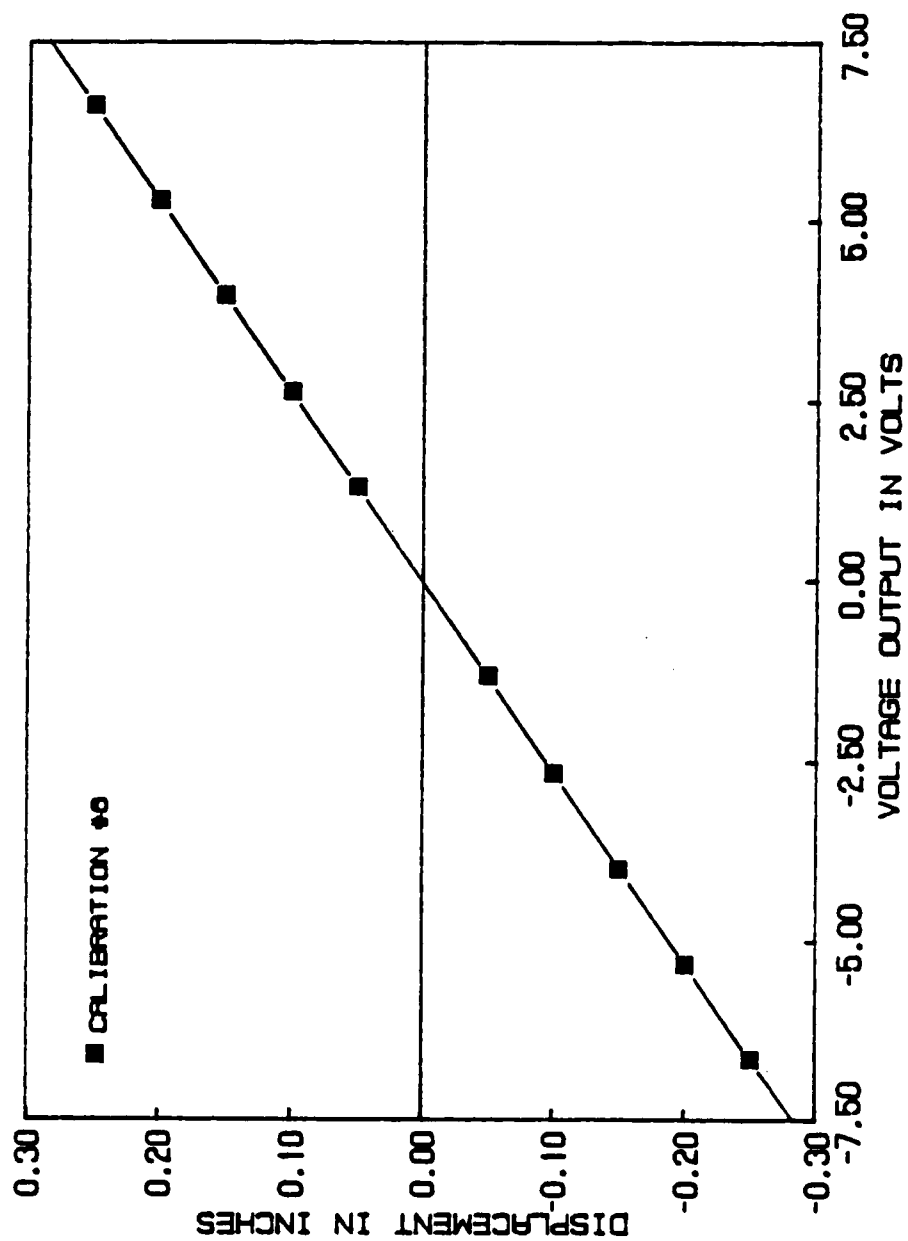


Figure 5-6: Calibration of Transducer 6

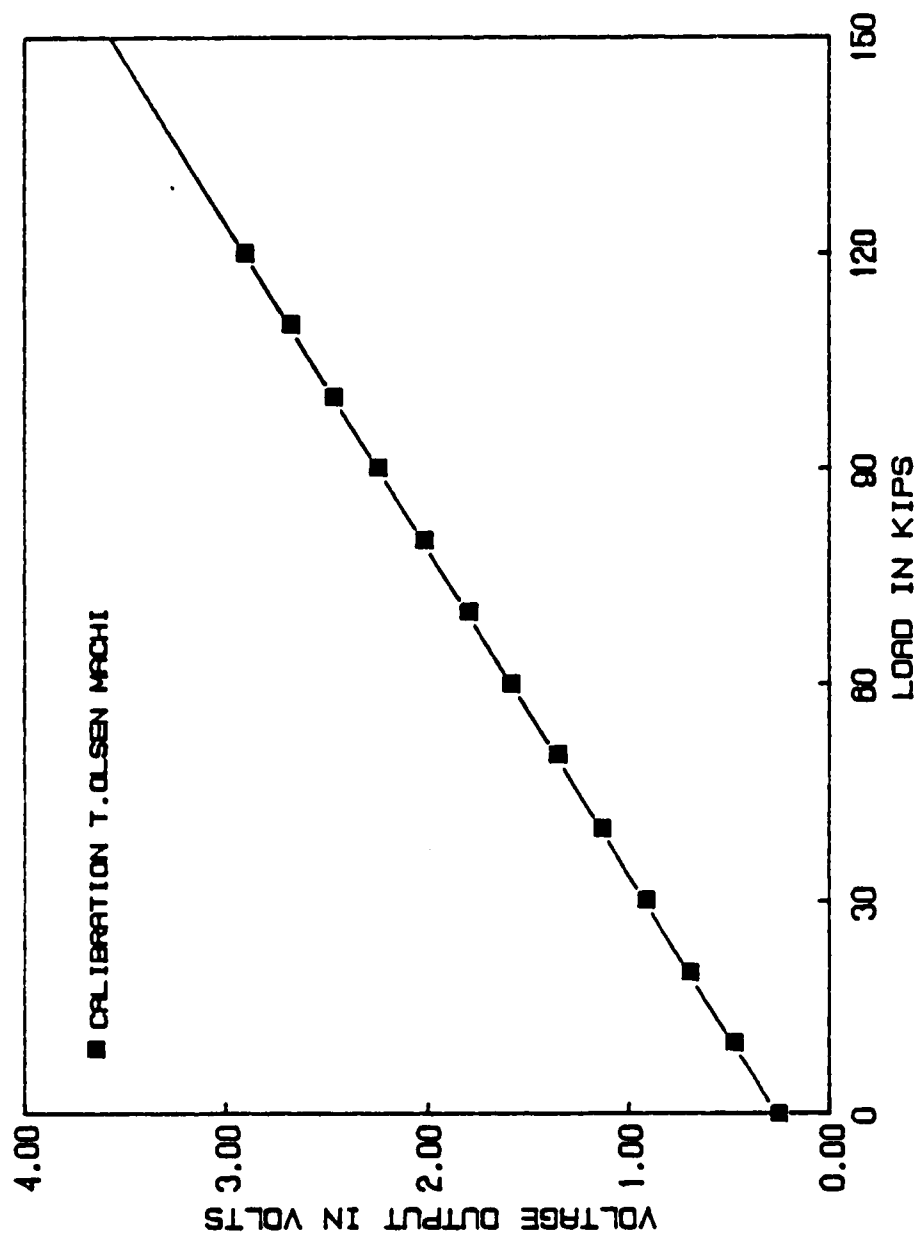


Figure 5-7: Calibration of T. Olsen Testing Machine

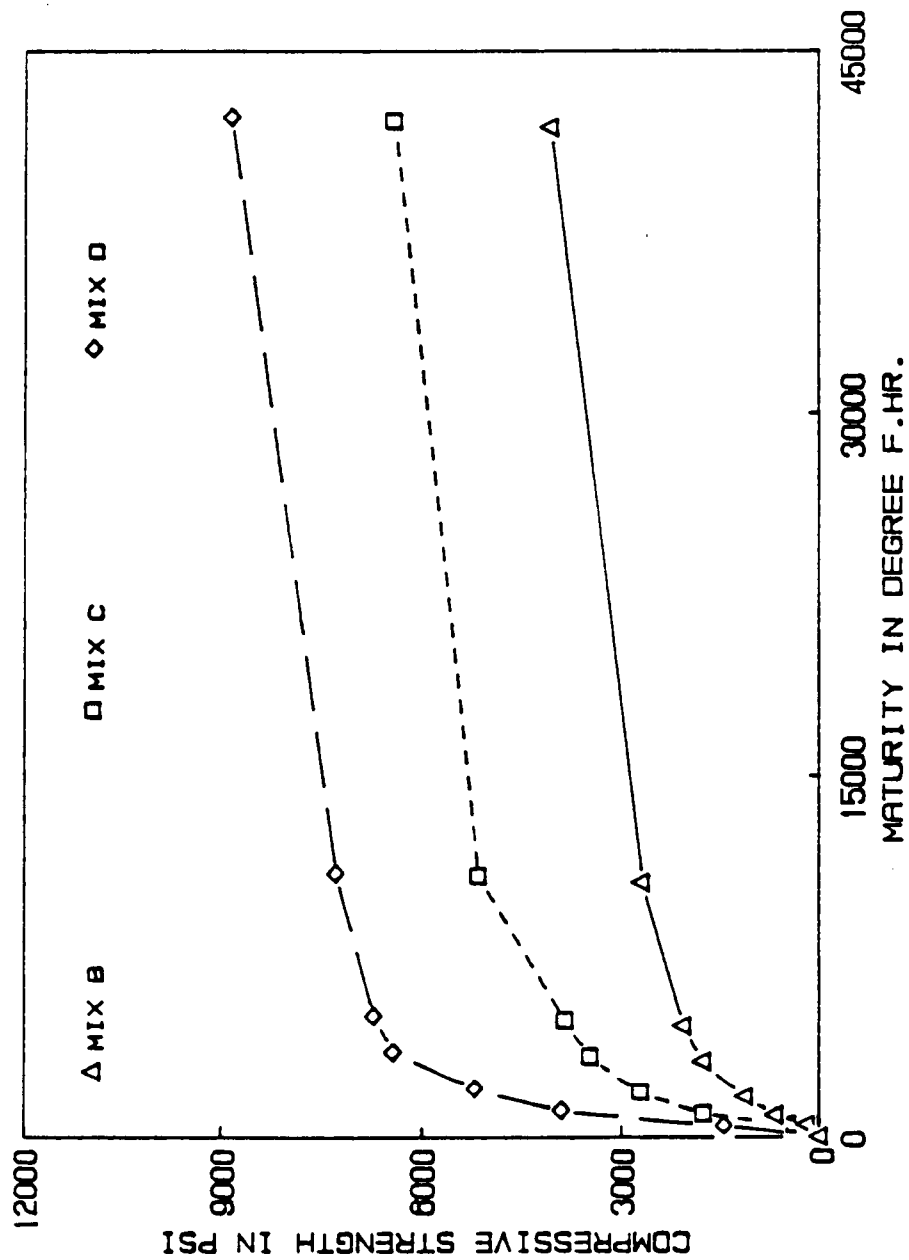


Figure 6-1: Compressive Strength Development Versus Maturity

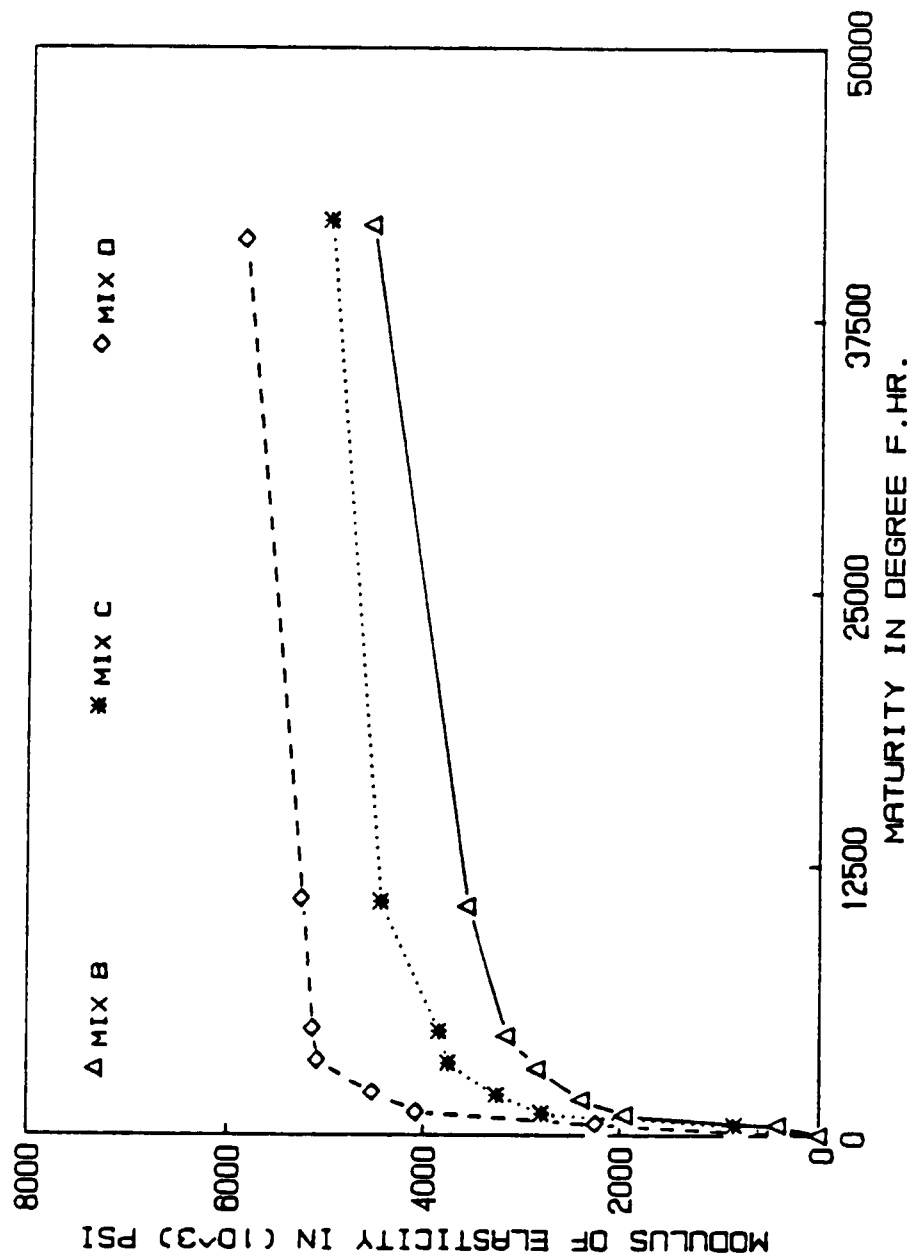


Figure 6-2: Elastic Modulus Versus Maturity

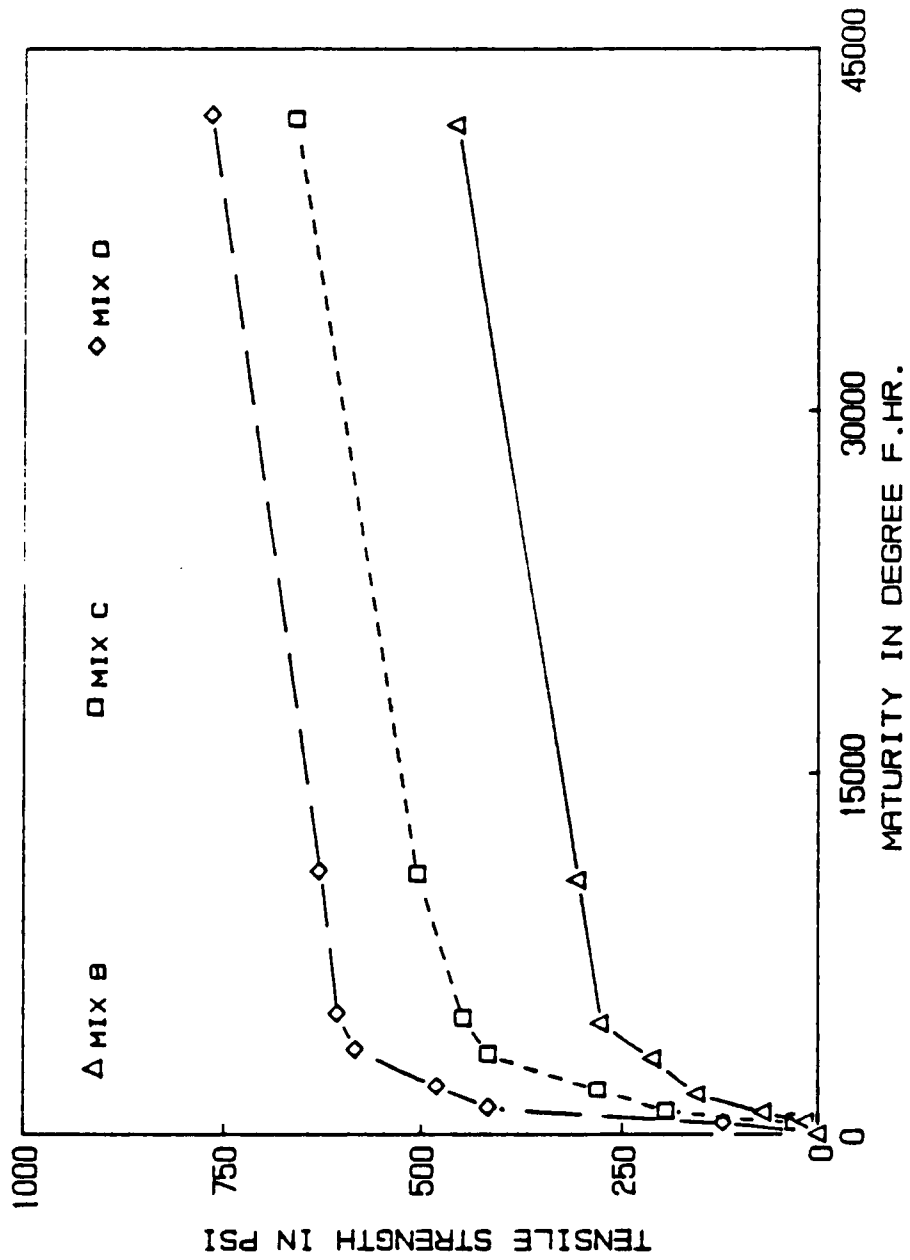


Figure 6-3: Tensile Strength Versus Maturity

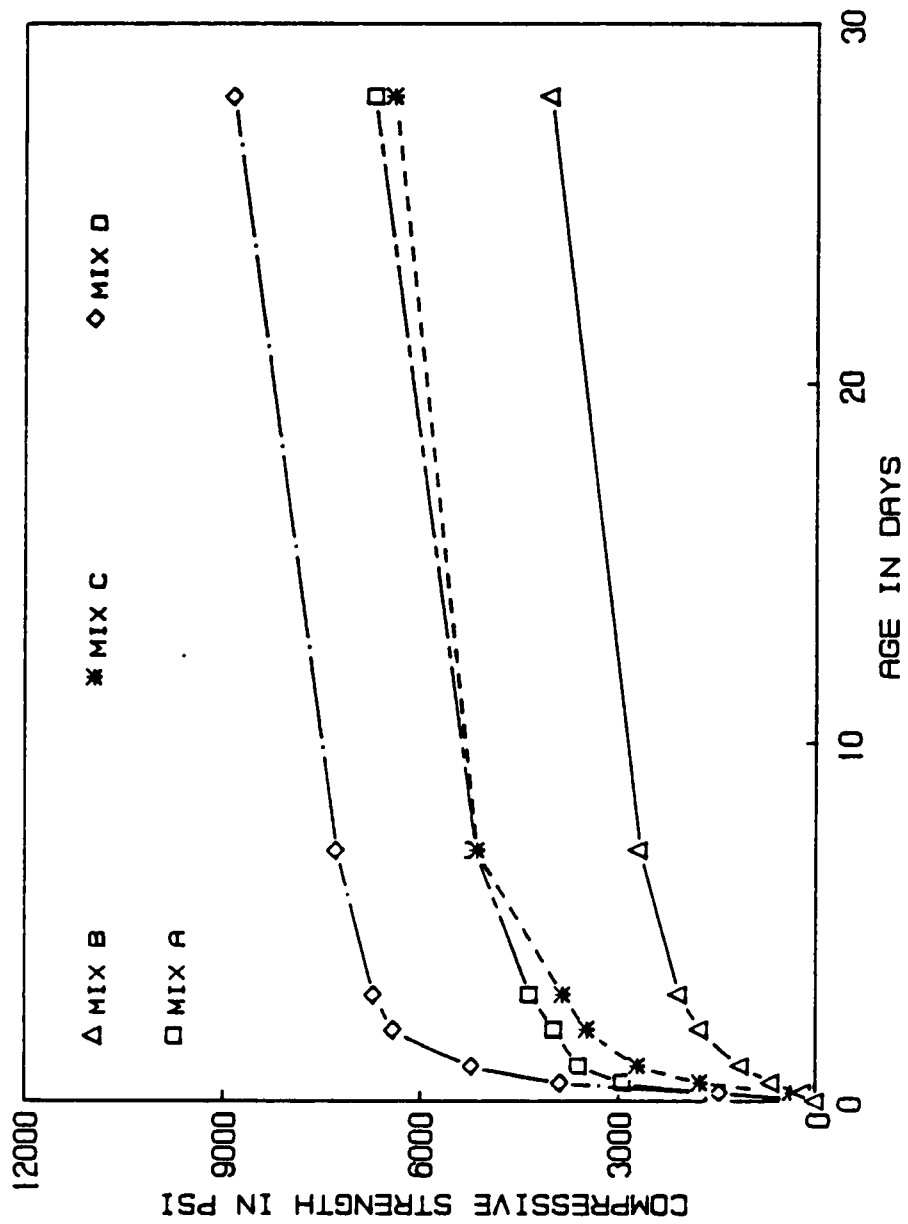


Figure 6-4: Compressive Strength Versus Age

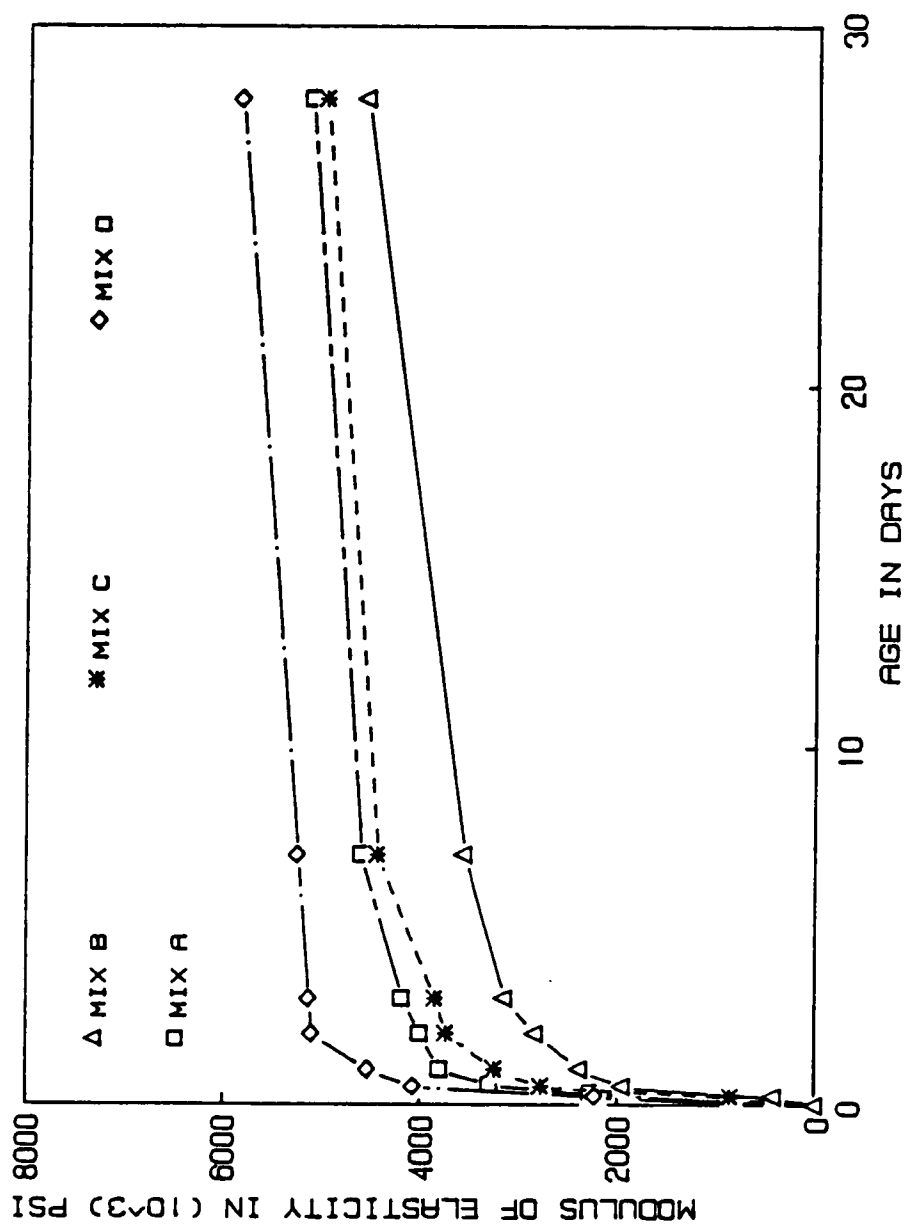


Figure 6-5: Elastic Modulus Versus Age

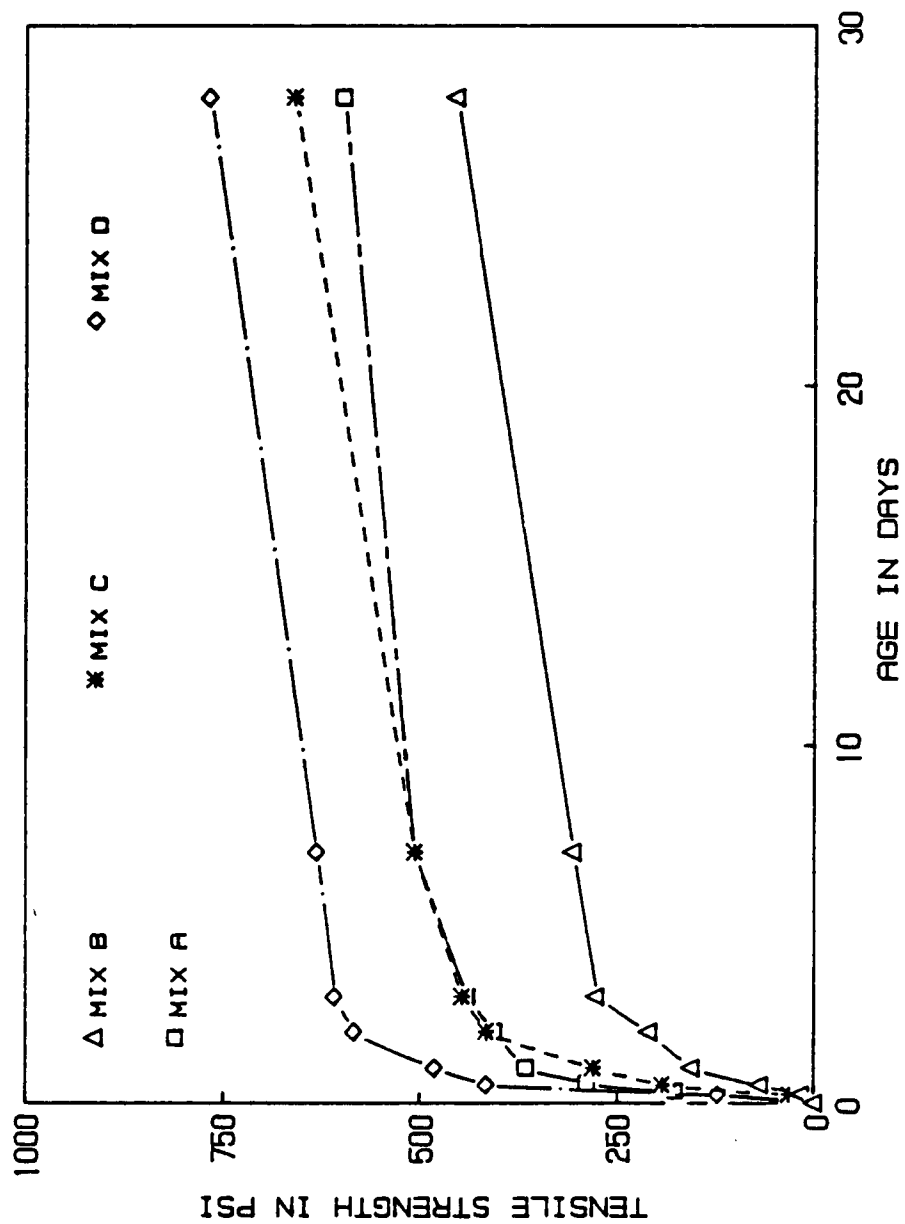


Figure 6-6: Tensile Strength Versus Age

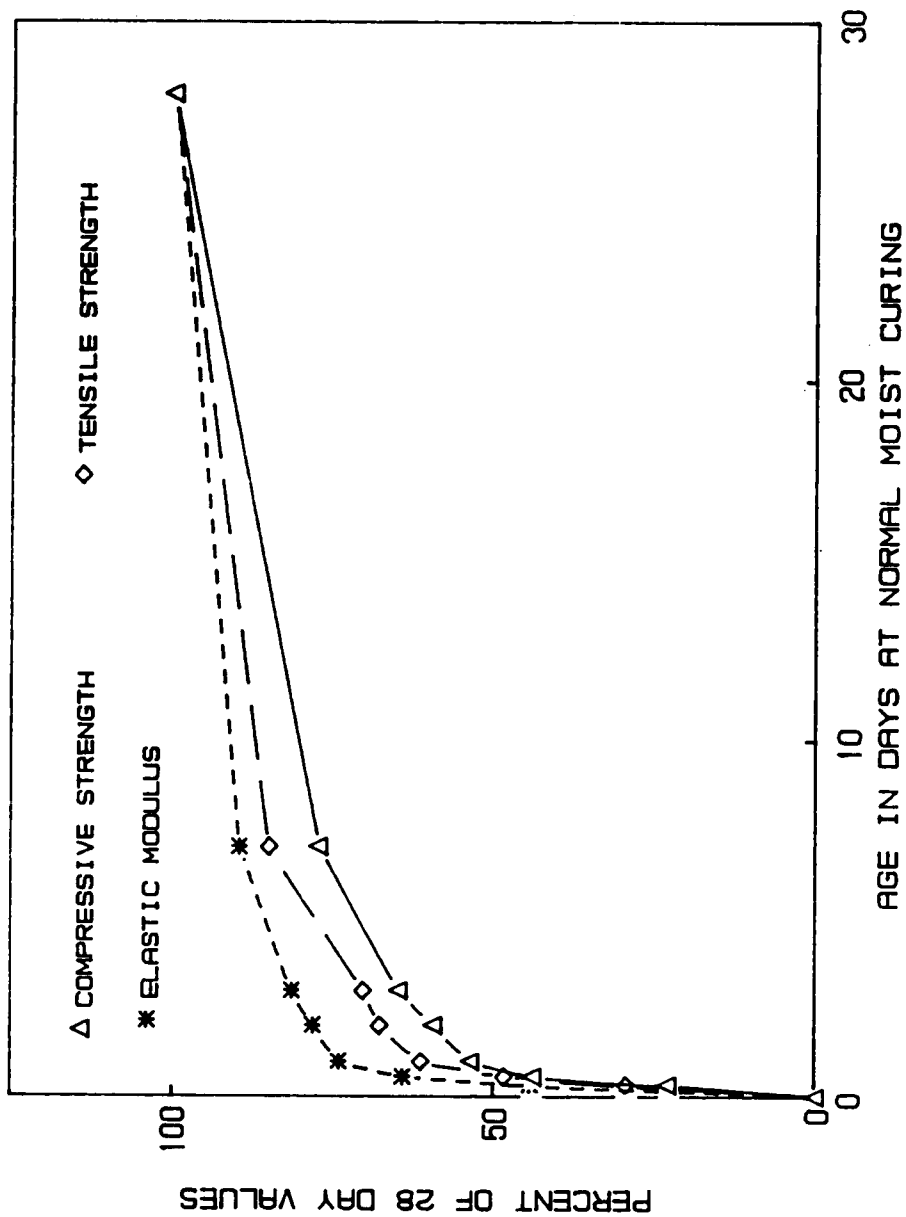


Figure 6-7: Comparison of Physical Property Development Using Normalized Values (Mix A, Moist Cured)

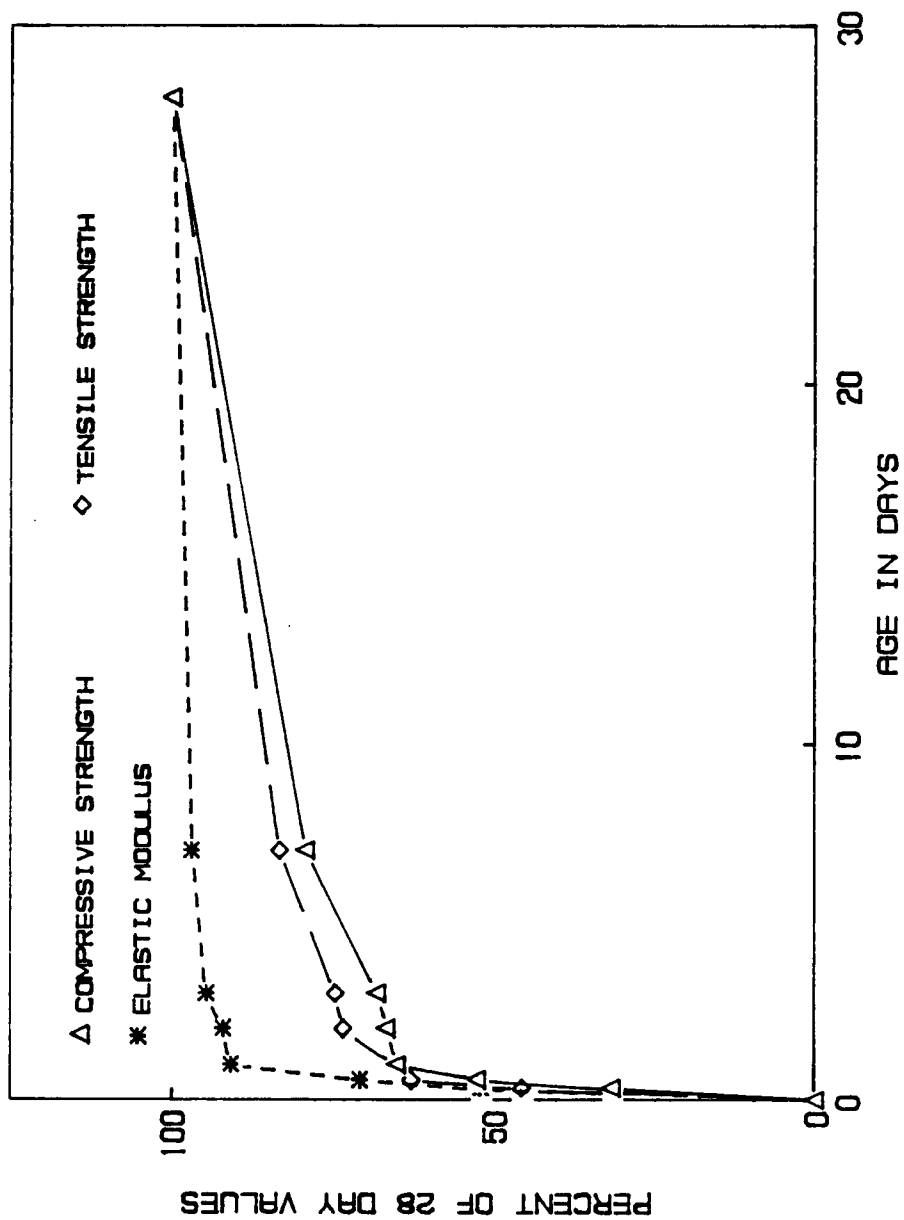


Figure 6-8: Comparison of Physical Property Development Using Normalized Values (Mix A, Steam Cured)

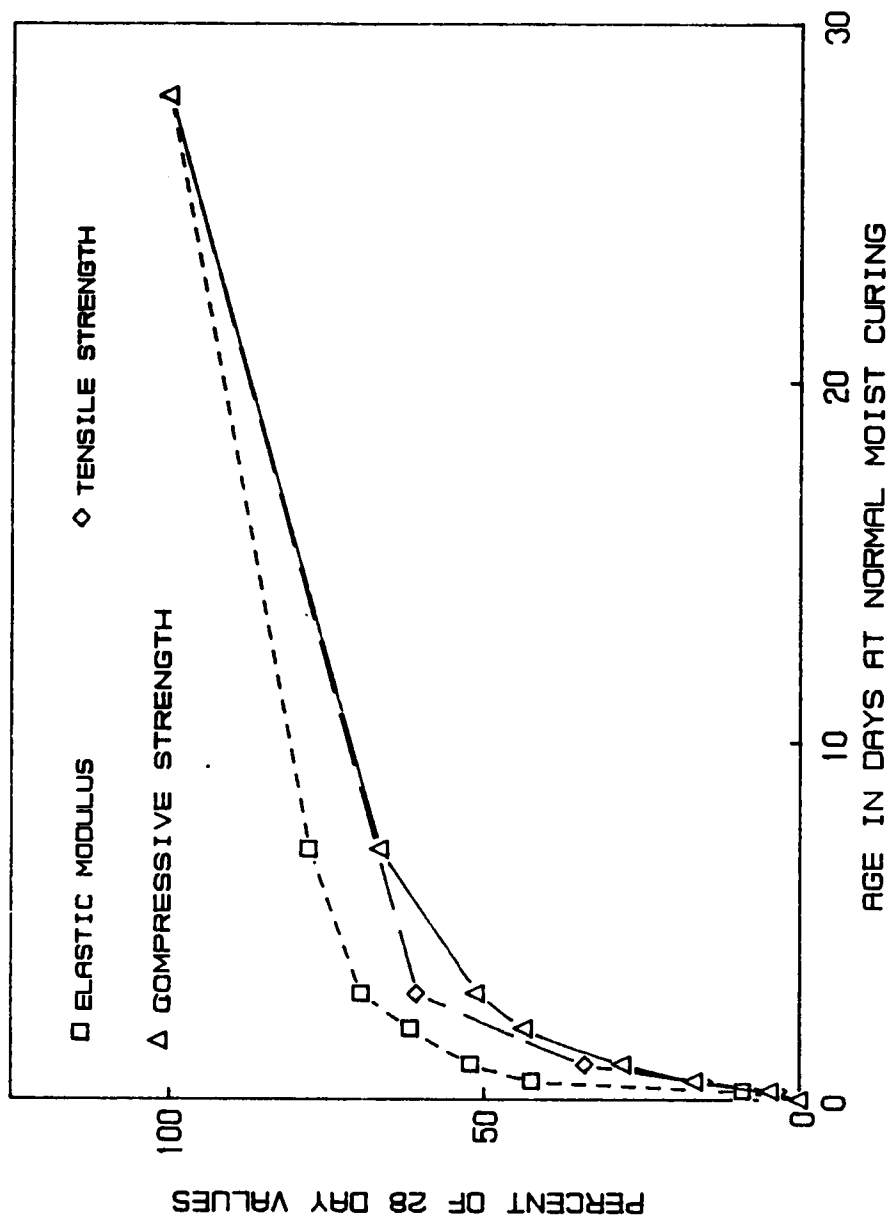


Figure 6-9: Comparison of Physical Property Development Using Normalized Values (Mix B)

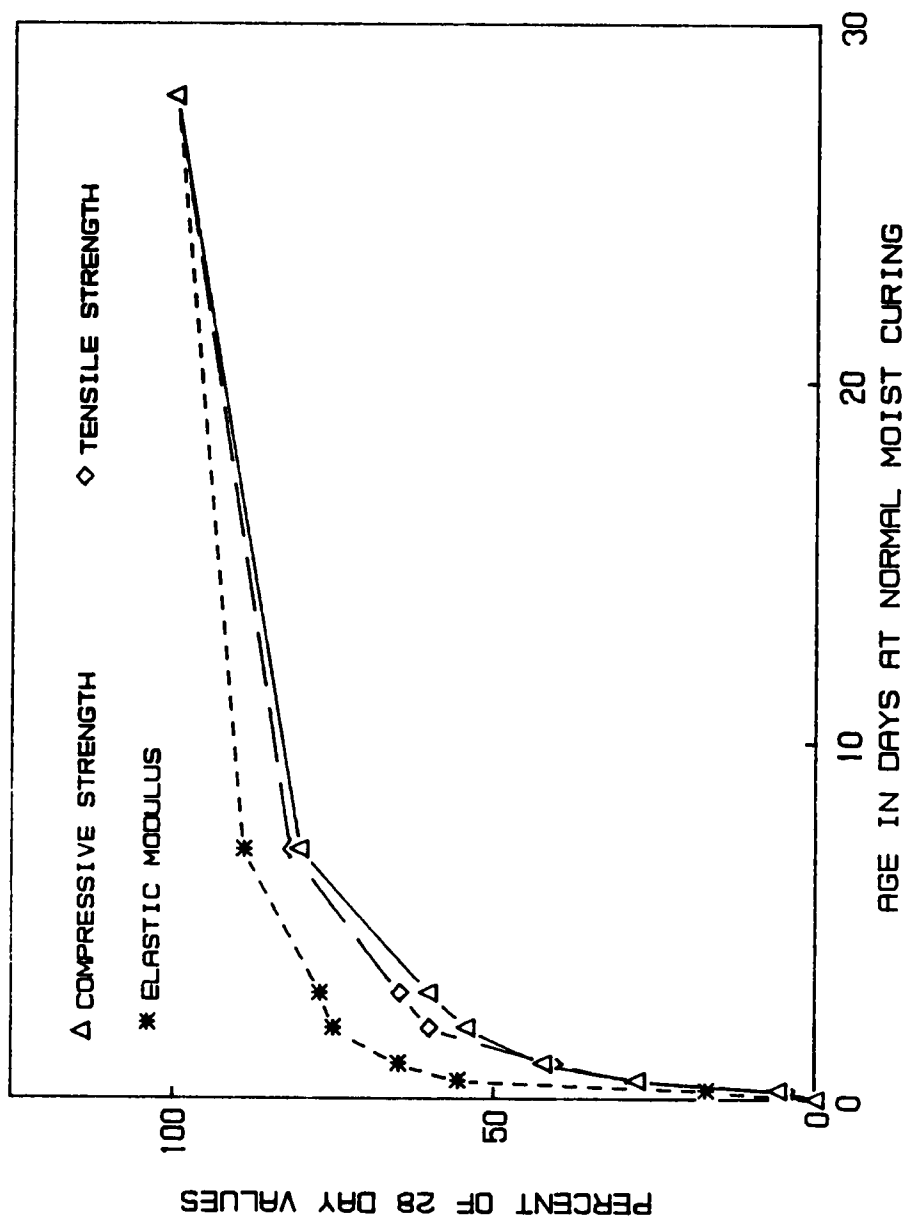


Figure 6-10: Comparison of Physical Property Development Using Normalized Values (Mix C)

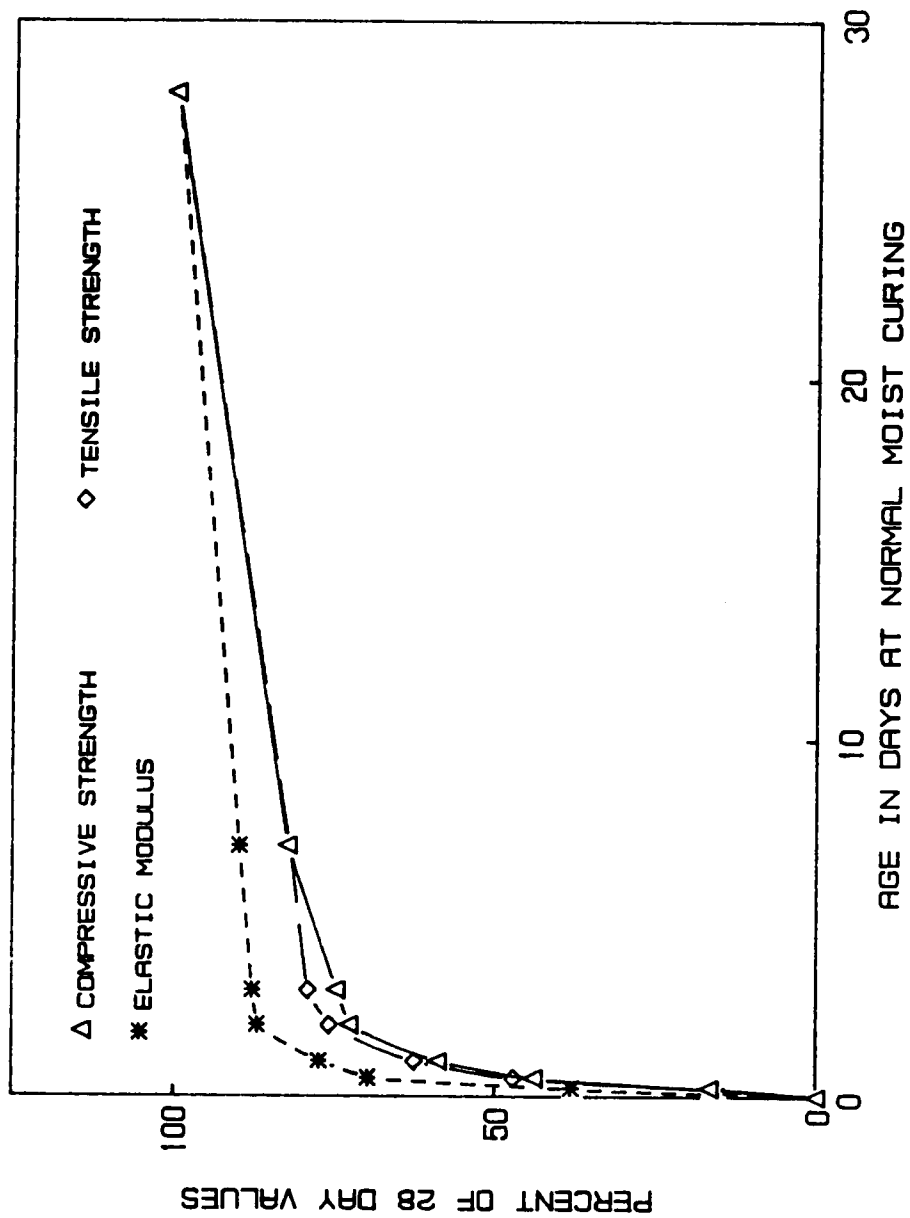


Figure 6-11: Comparison of Physical Property Development Using Normalized Values (Mix D)

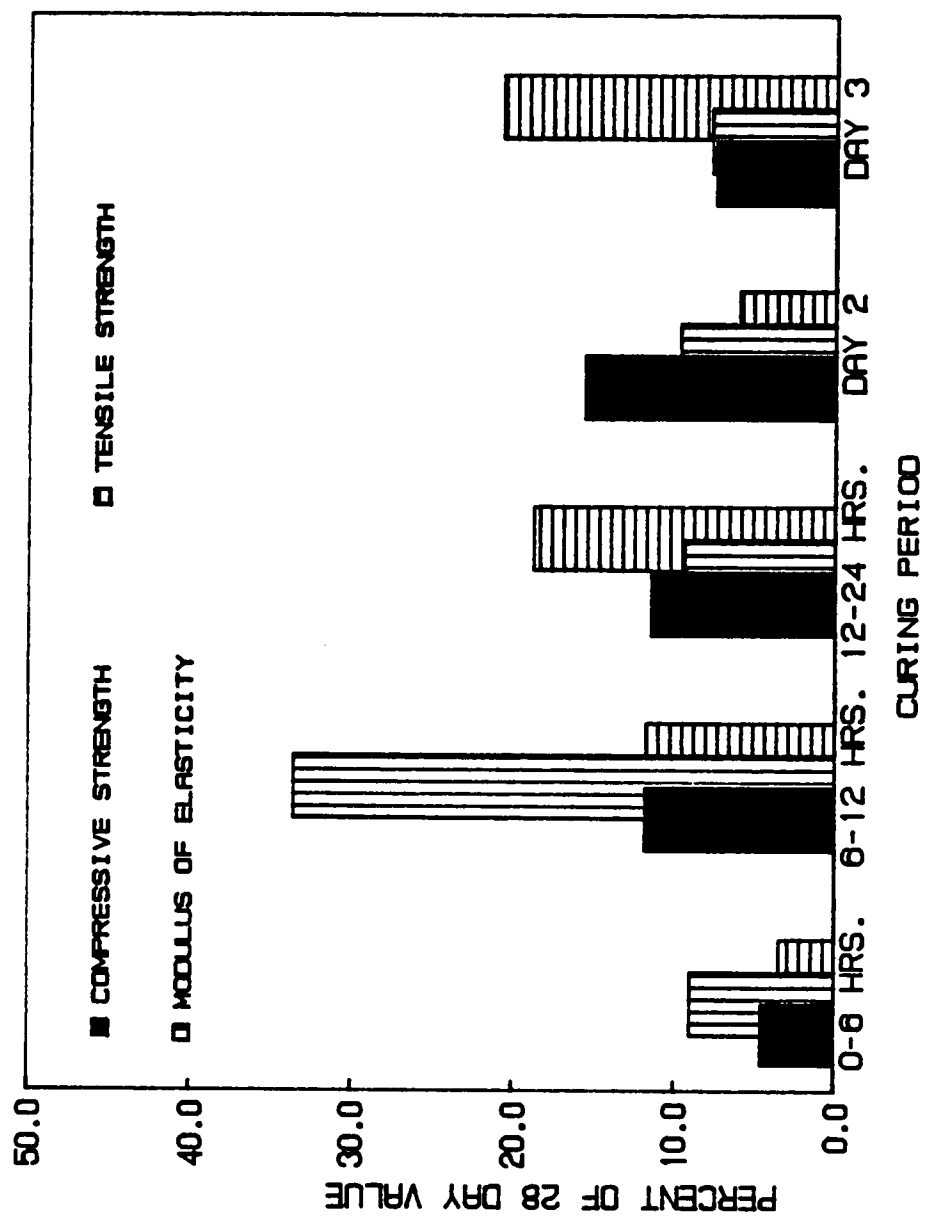


Figure 6-12: Periodic Development of Physical Properties in Mix B
(First Three Days)

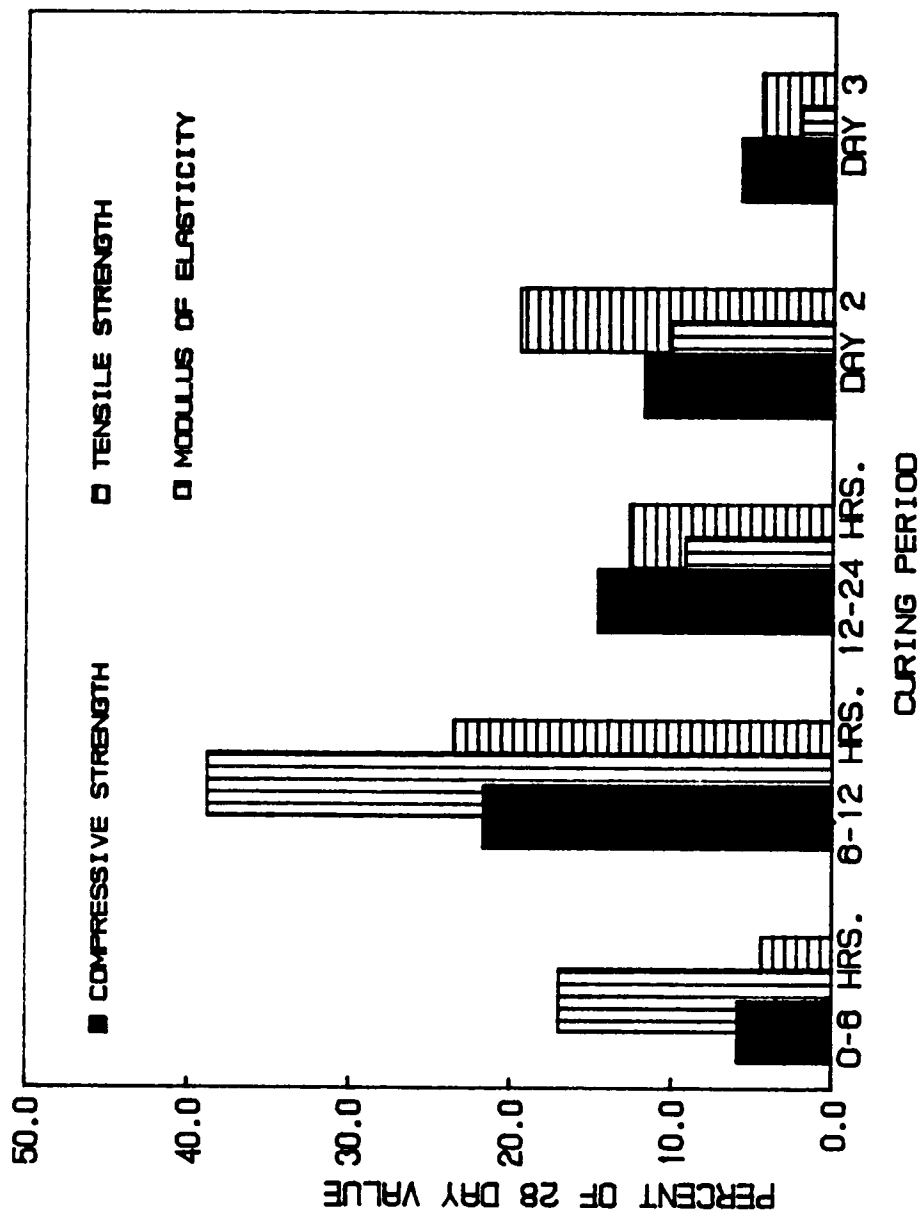


Figure 6-13: Periodic Development of Physical Properties in Mix C
(First Three Days)

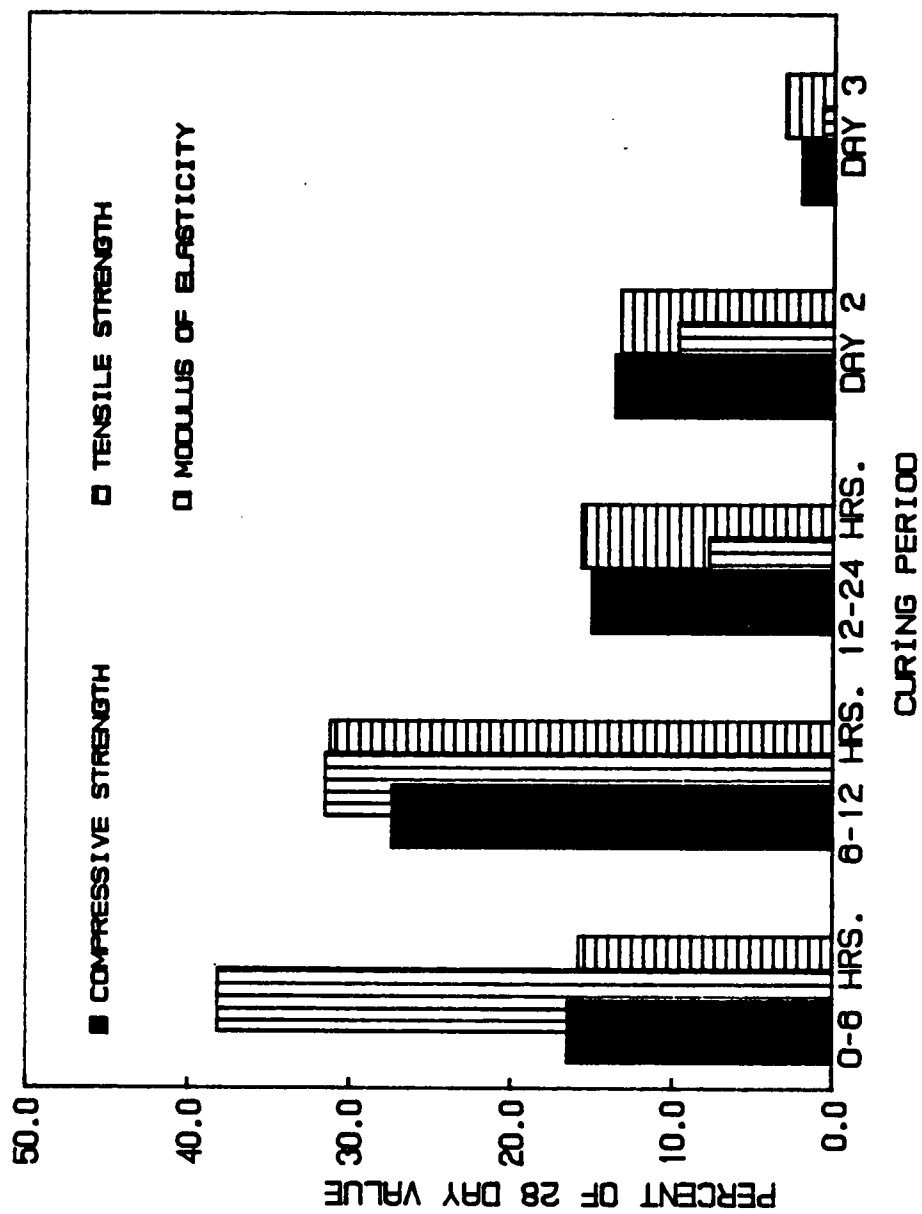


Figure 6-14: Periodic Development of Physical Properties in Mix D
(First Three Days)

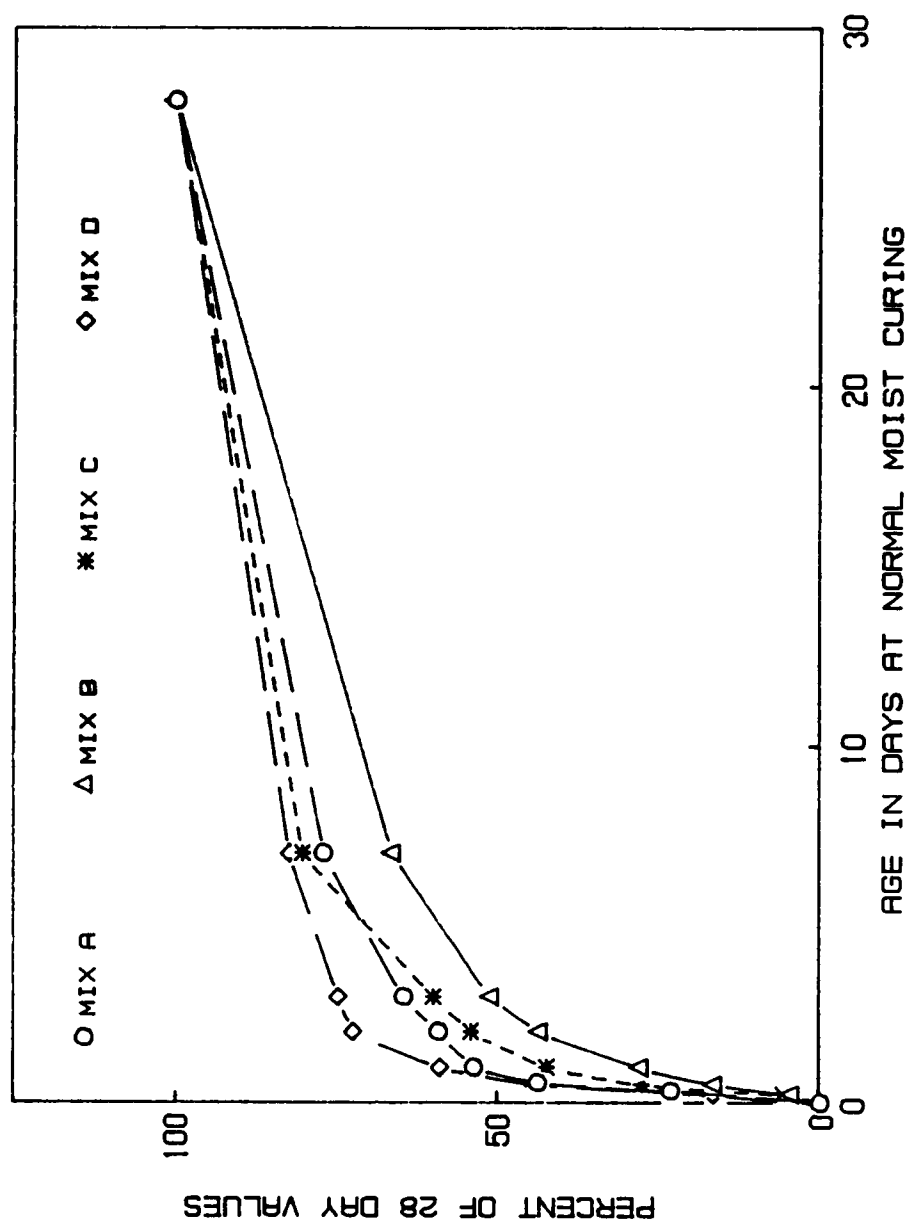


Figure 6-15: Comparison of Compressive Strength Development Using Normalized Values

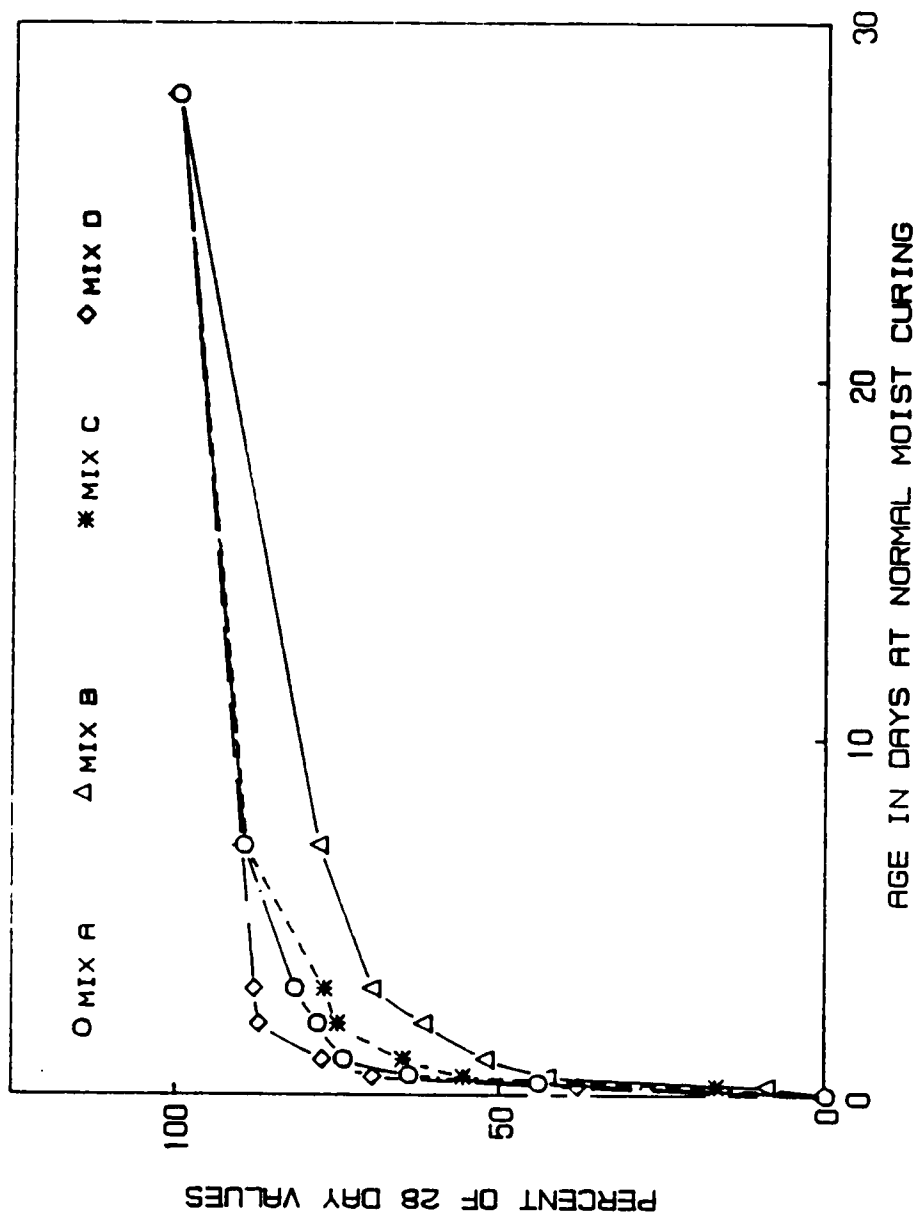


Figure 6-16: Comparison of Elastic Modulus Development Using Normalized Values

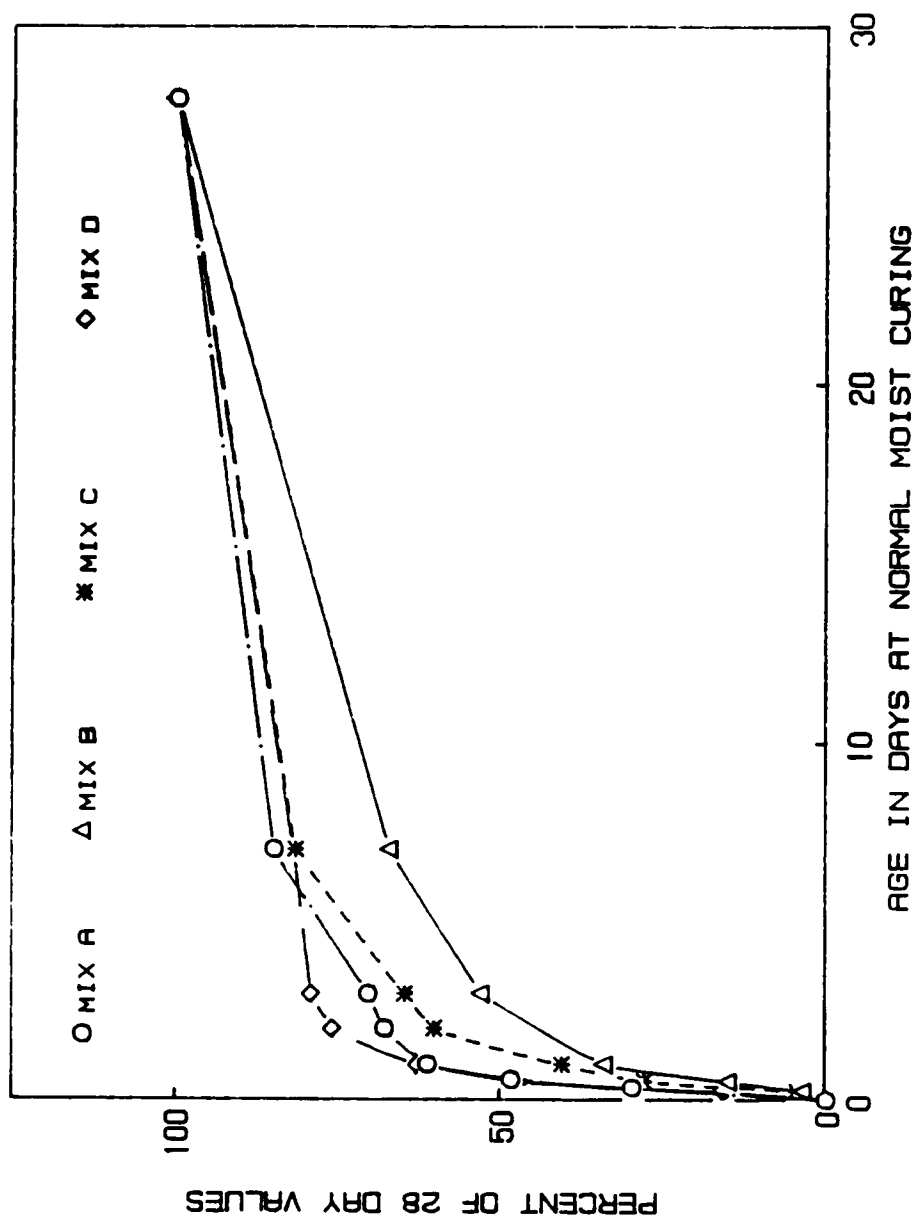


Figure 6-17: Comparison of Tensile Strength Development Using Normalized Values

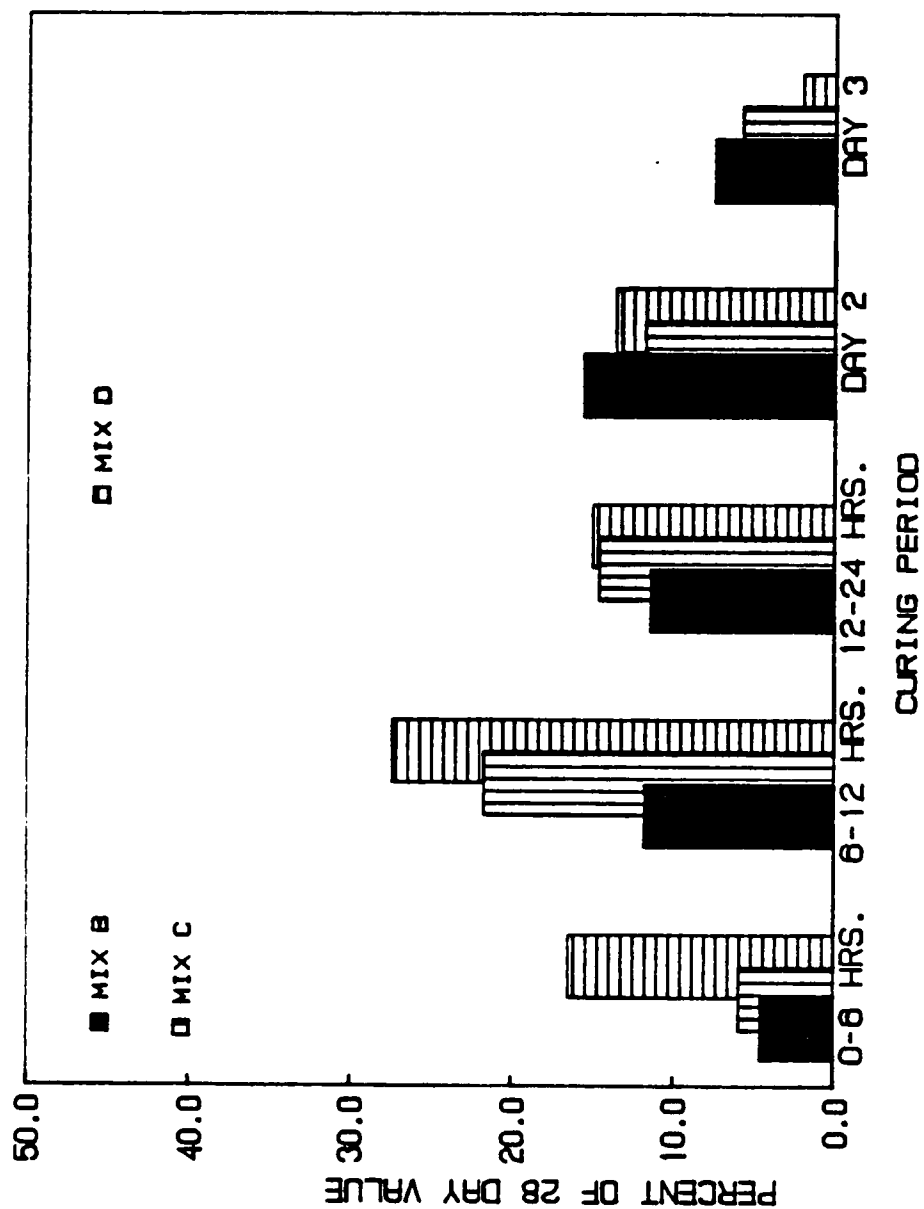


Figure 6-18: Comparison of Rate of Compressive Strength Development (First Three Days)

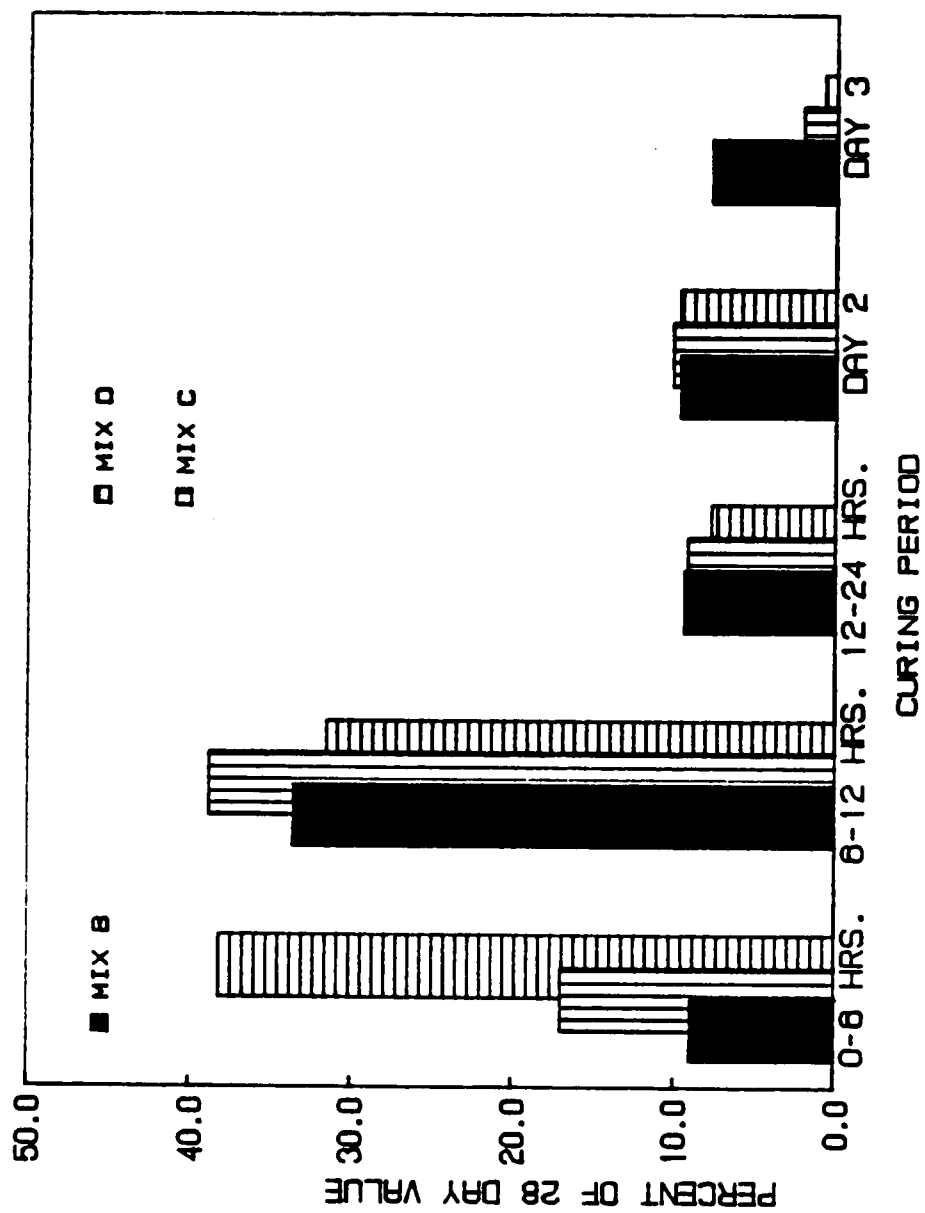


Figure 6-19: Comparison of Rate of Elastic Modulus Development (First Three Days)

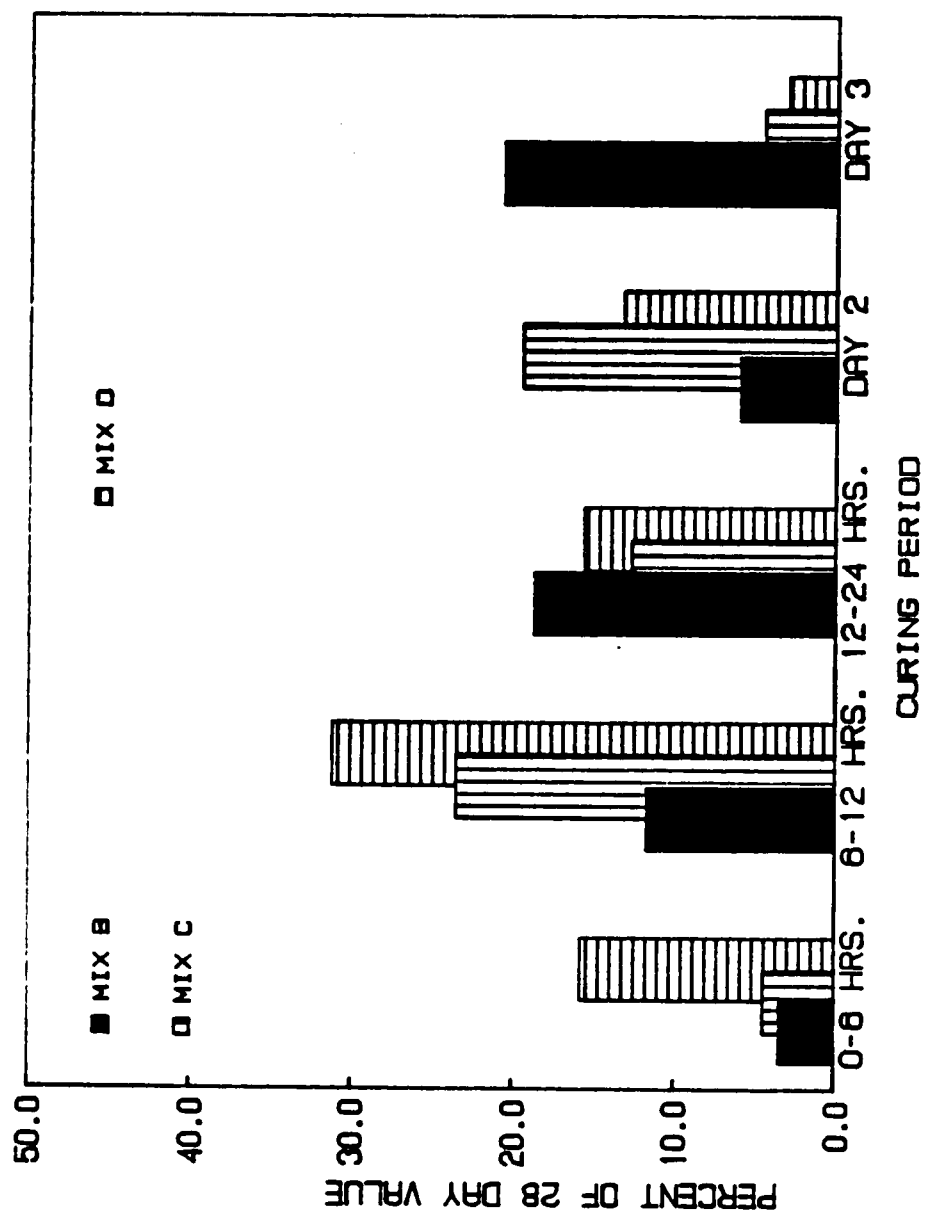


Figure 6-20: Comparison of Rate of Tensile Strength Development (First Three Days)

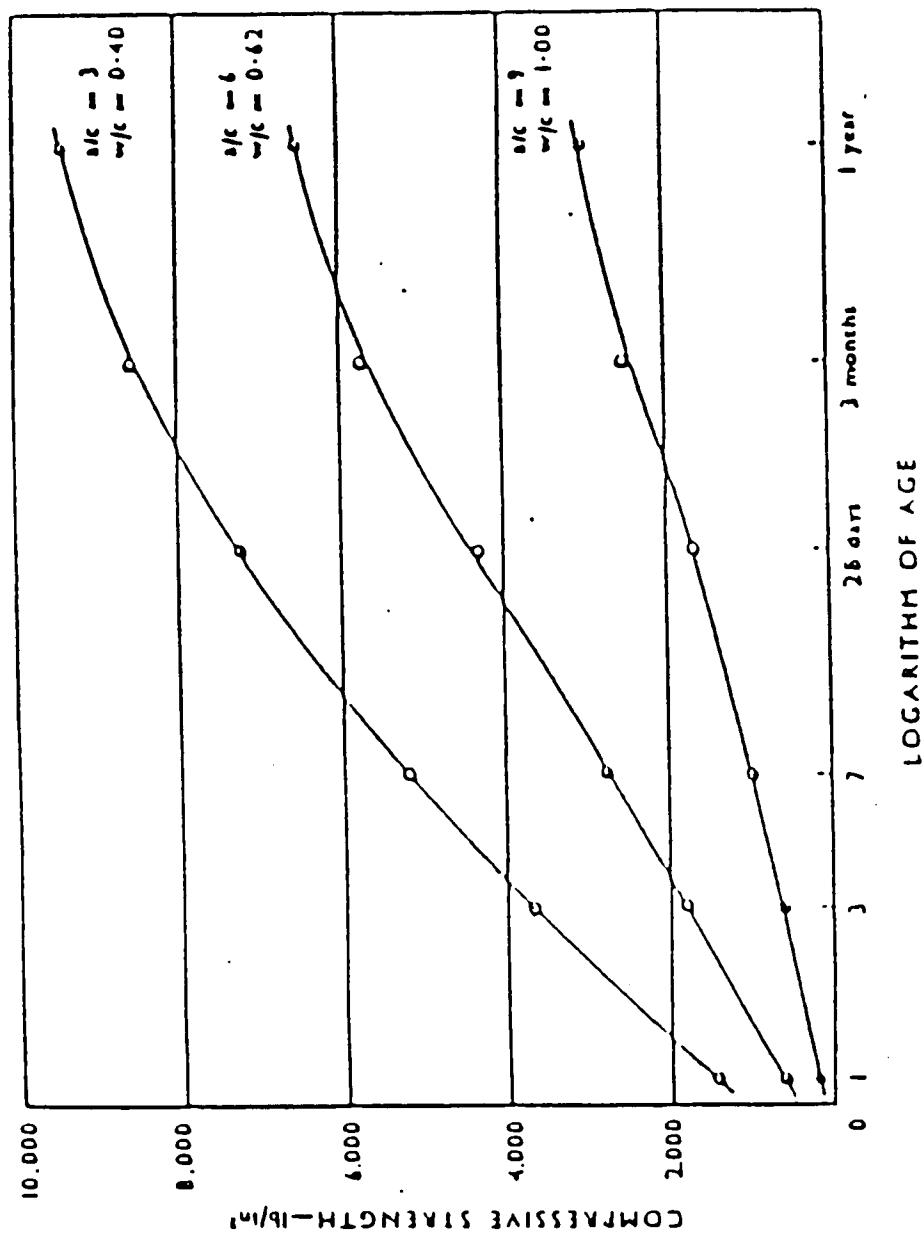


Figure 6-21: Results of Tests Carried out by the Cement and Concrete Association (Marshall)

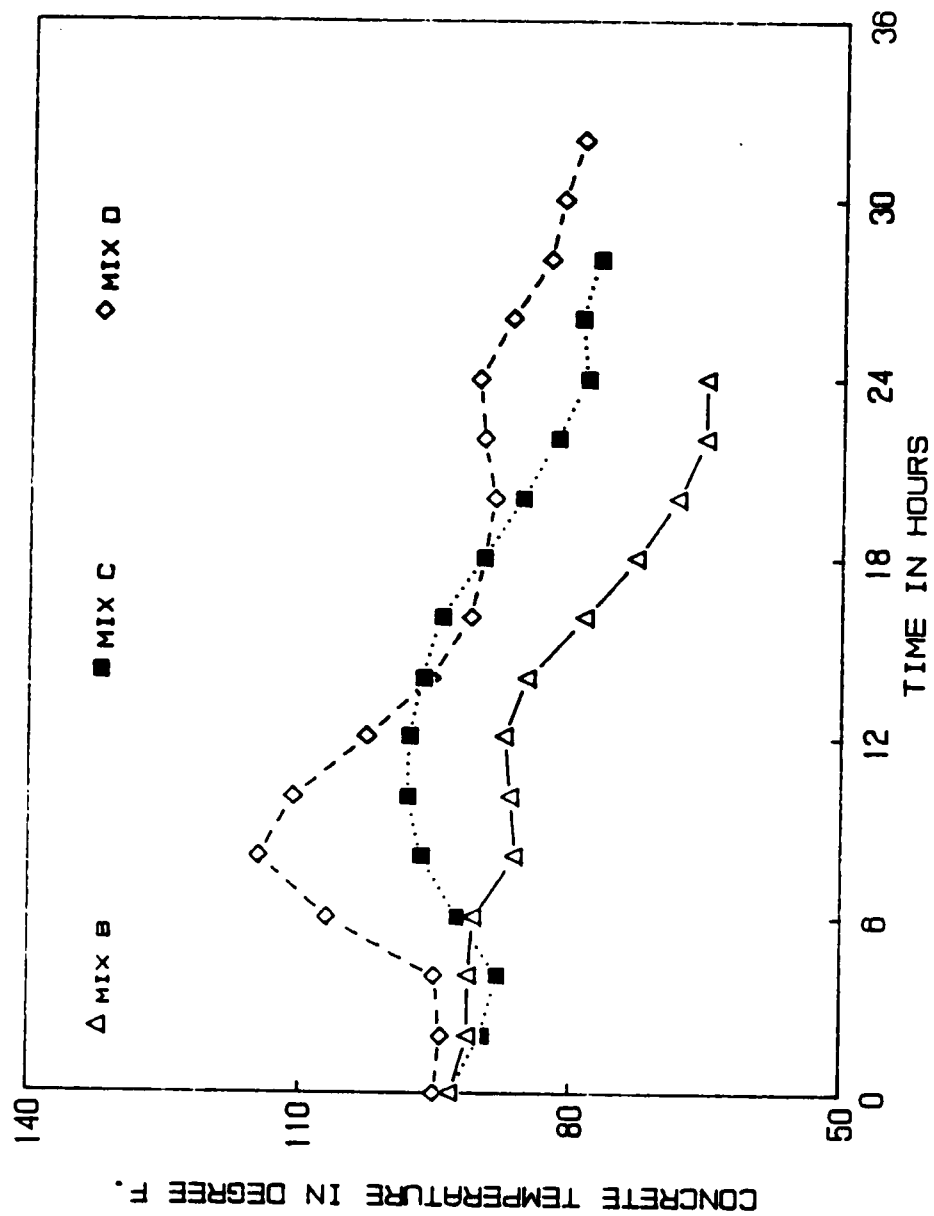


Figure 6-22: Hydration Temperature Versus Time

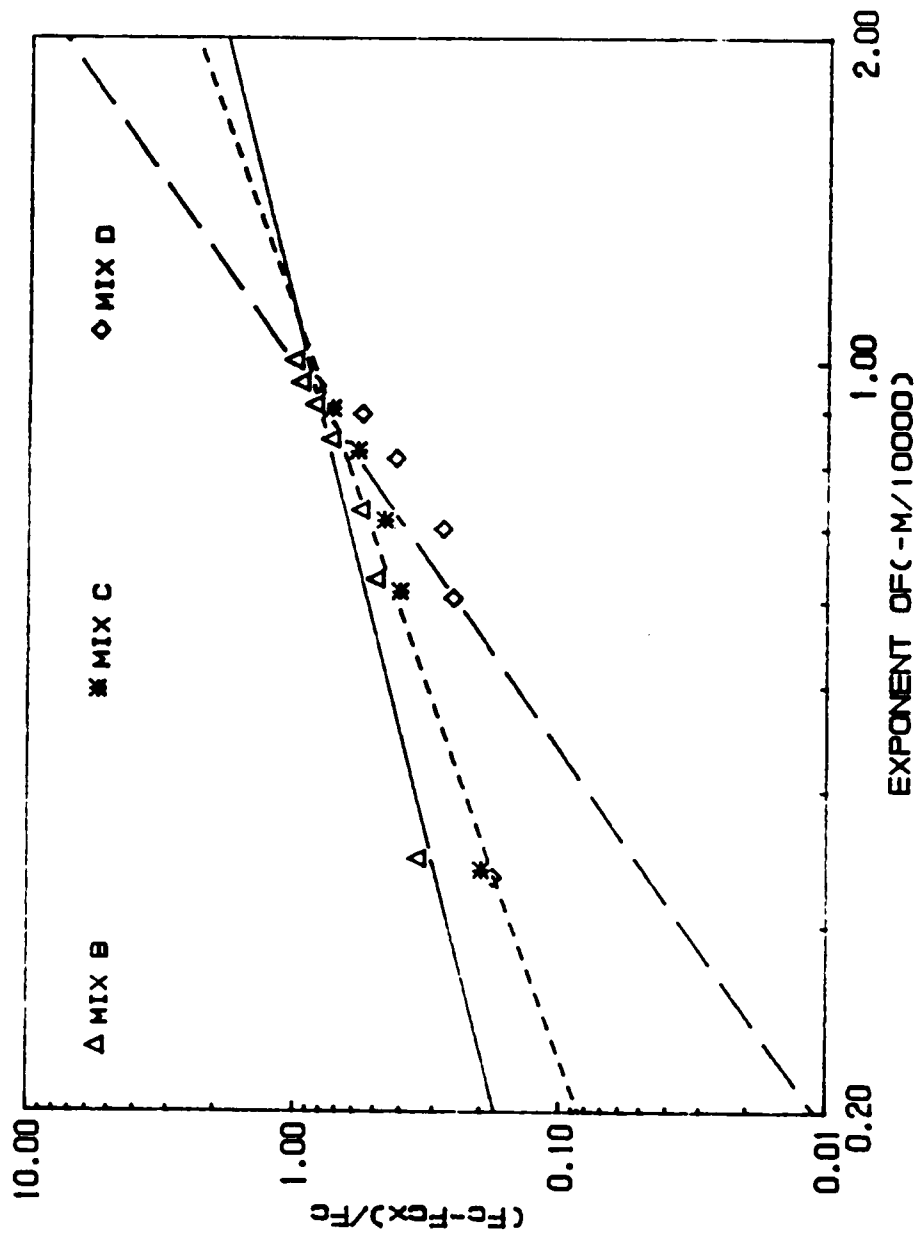


Figure 6-23: $\log (f_c' - f_{cx}) / f_c'$ Versus Log Exponent of $(-M/10000)$

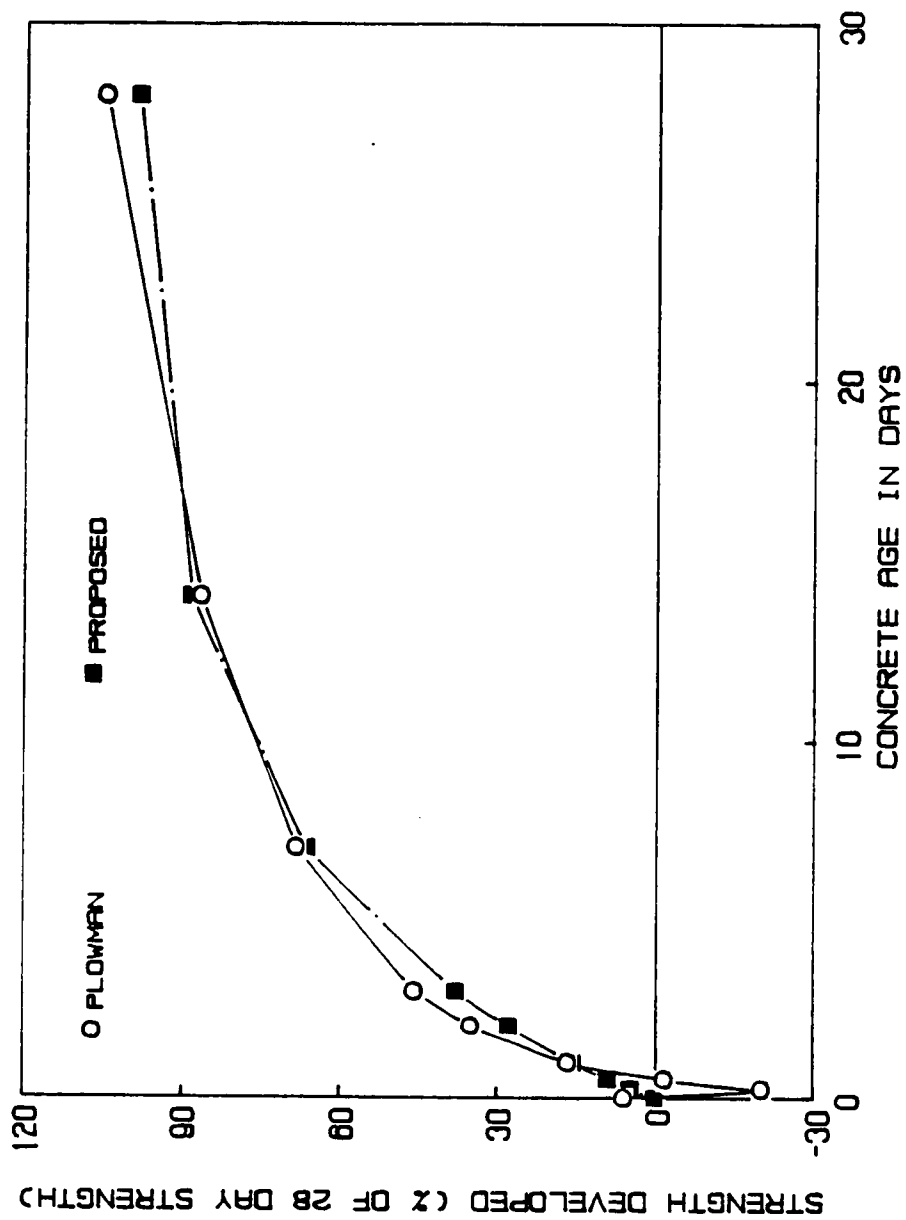


Figure 6-24: Comparison of Plowman's and Proposed Equations
($2500 \leq f_c \leq 5000$)

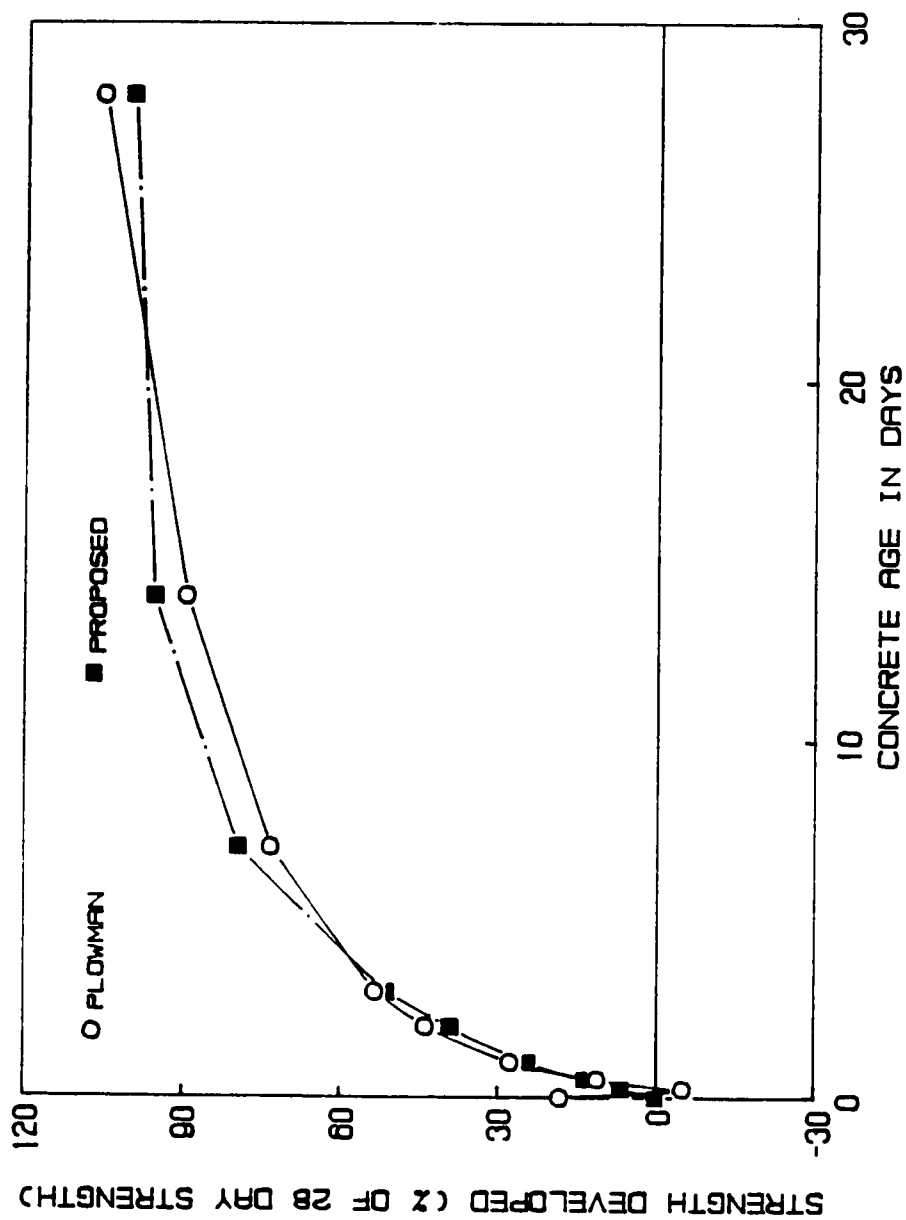


Figure 6-25: Comparison of Plowman's and Proposed Equations
($5000 \leq f_c \leq 7500$)

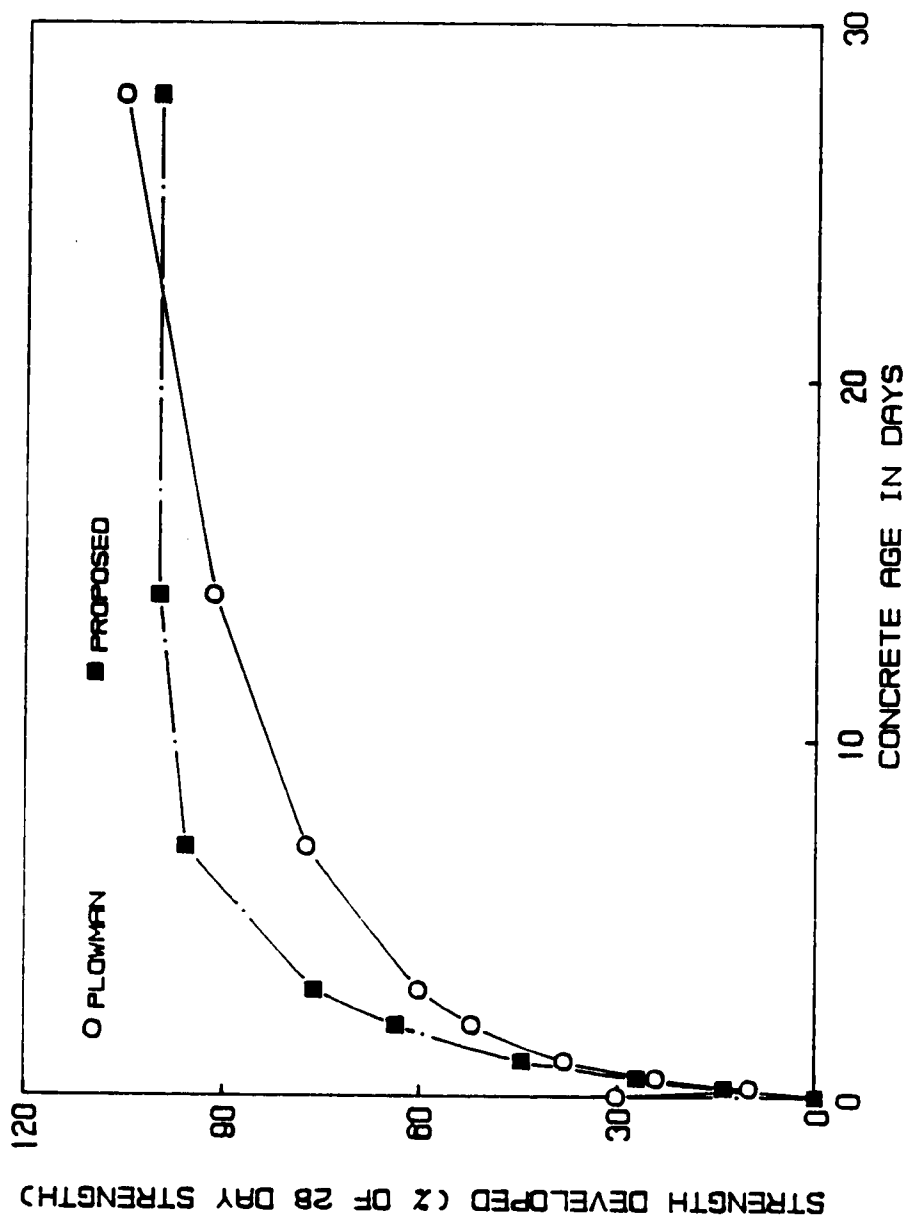


Figure 6-26: Comparison of Plowman's and Proposed Equations
($7500 \leq f_c \leq 10000$)

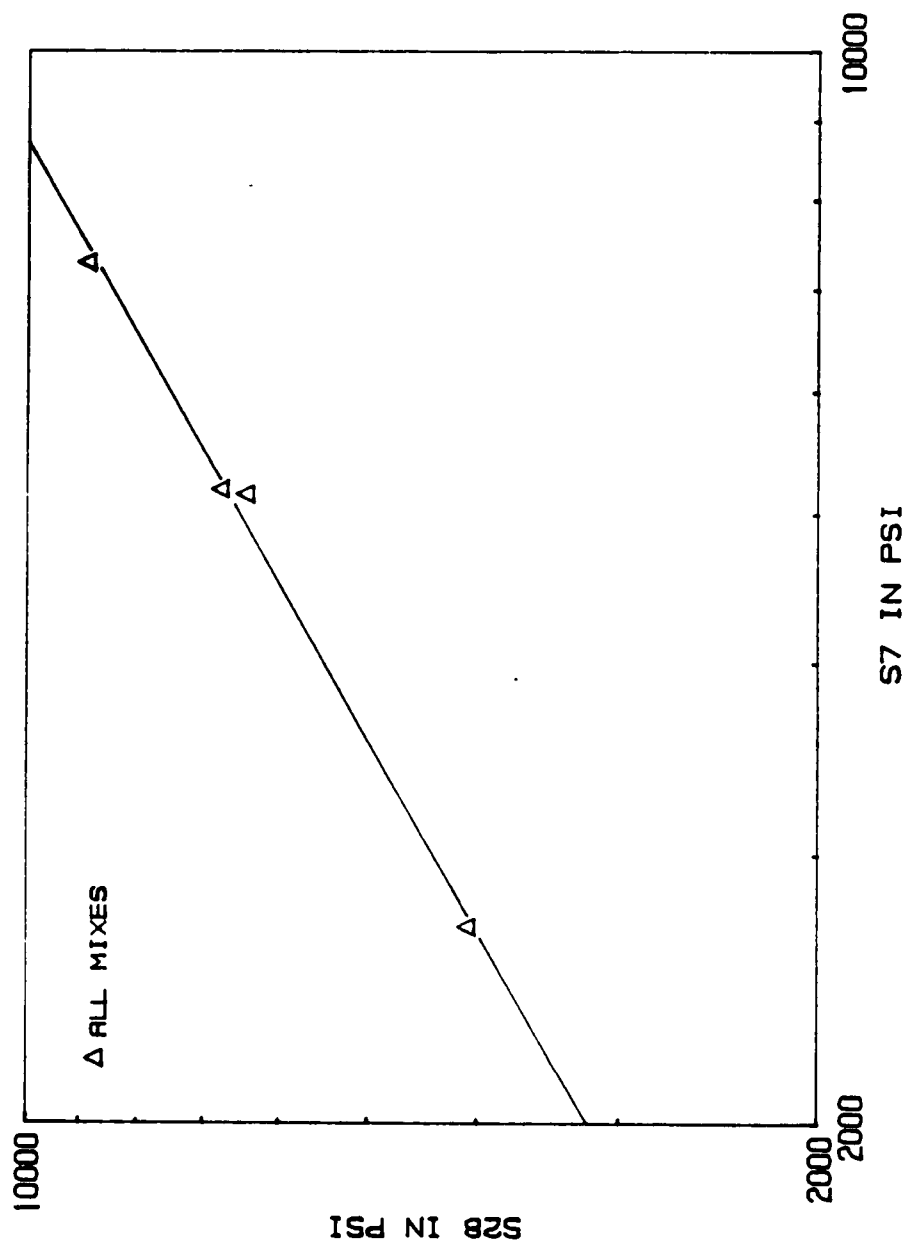


Figure 6-27: Log S28 Versus Log S7

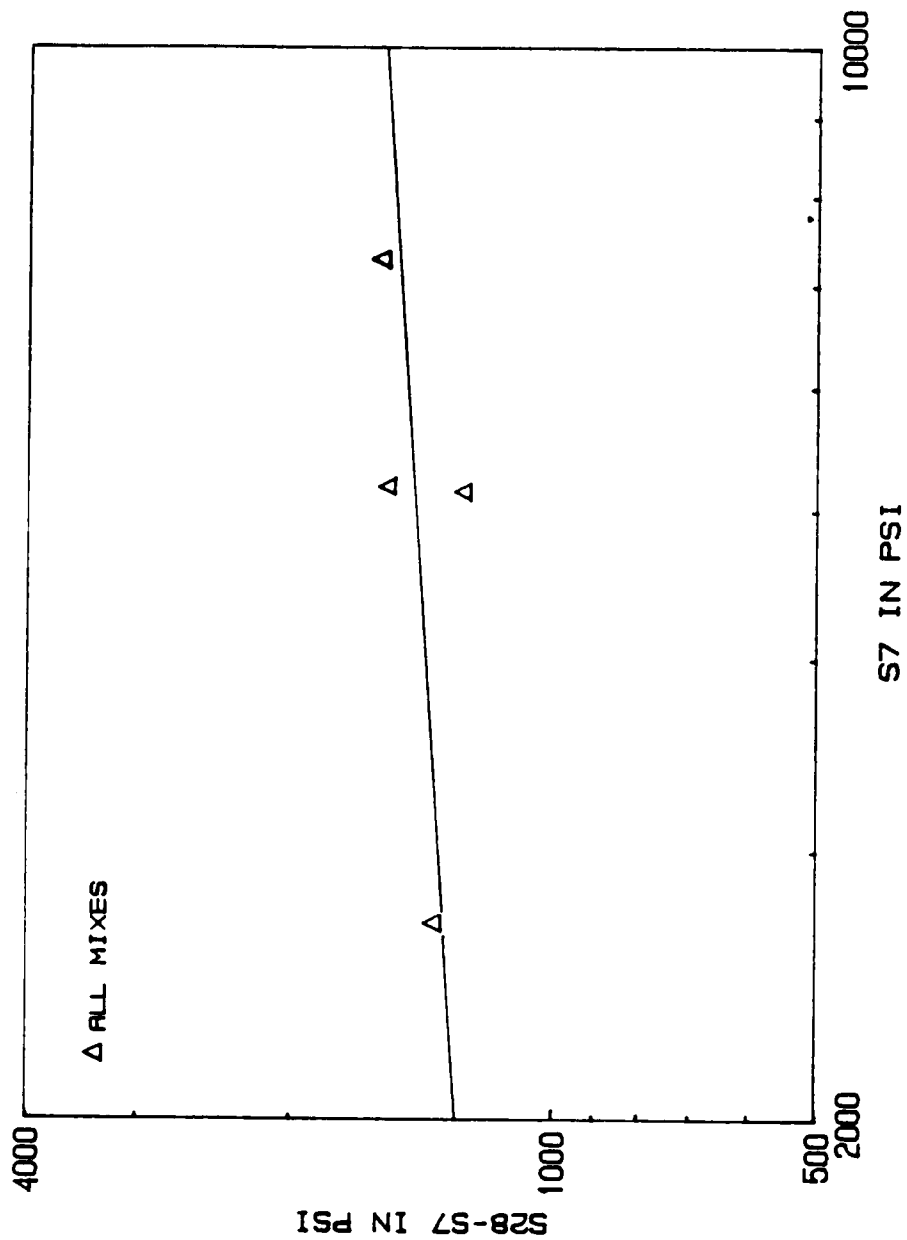


Figure 6-28: Log (S₂₈-S₇) Versus Log S₇

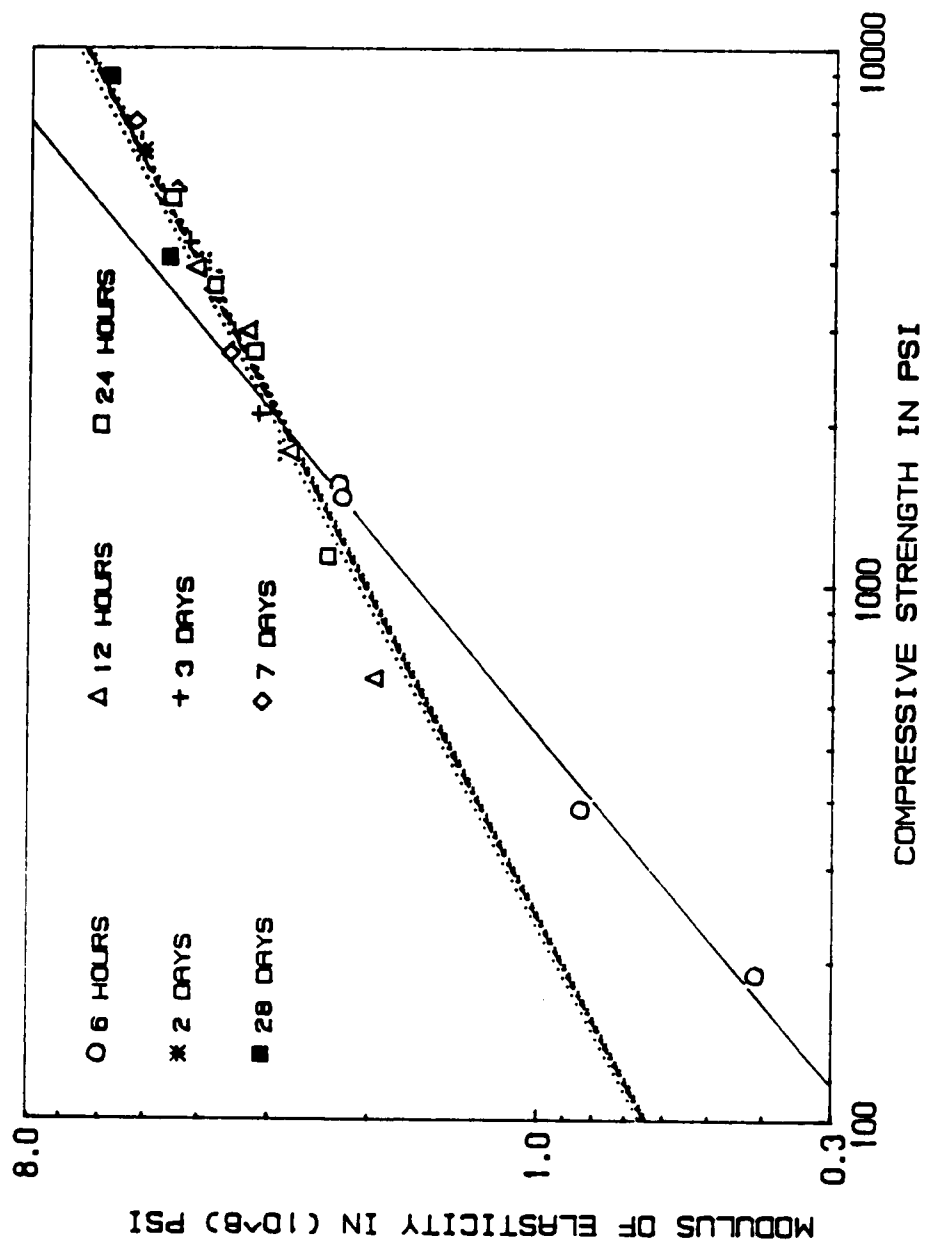


Figure 6-29: Log-Log Plot of Elastic Modulus Versus Compressive Strength (Each Age)

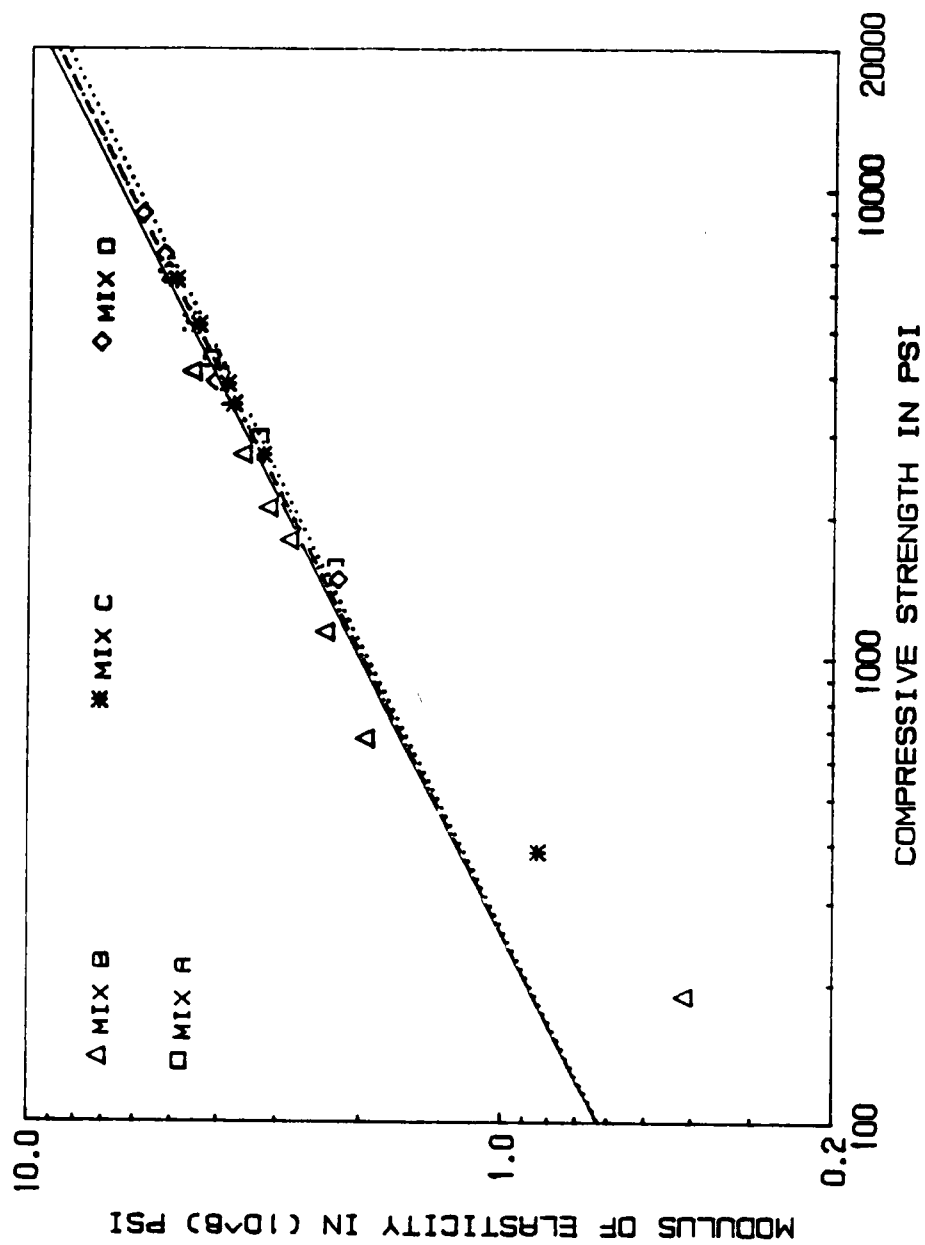


Figure 6-30: Log-Log Plot of Elastic Modulus Versus Compressive Strength (Each Mix)

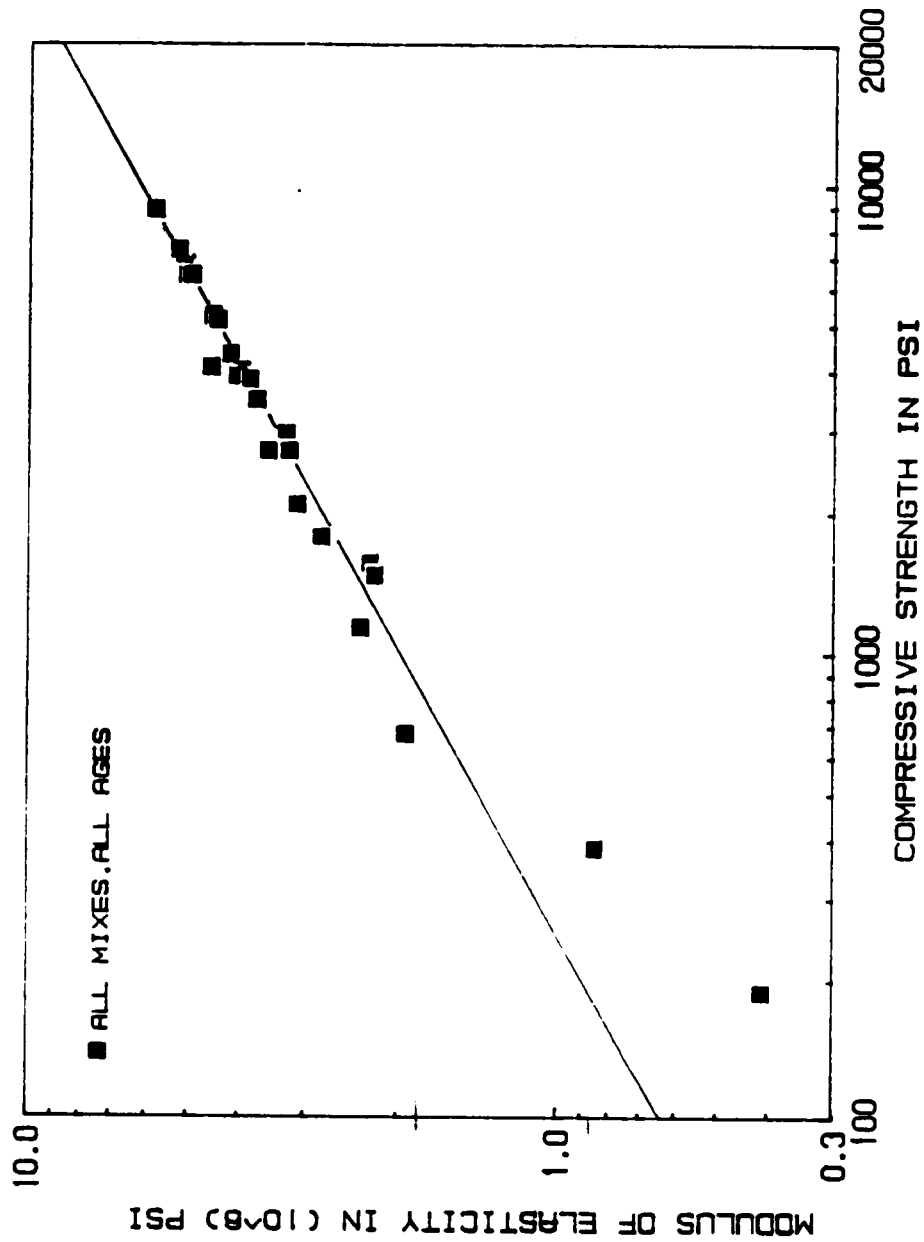


Figure 6-31: Log-Log Plot of Elastic Modulus Versus Compressive Strength (All Data Combined)

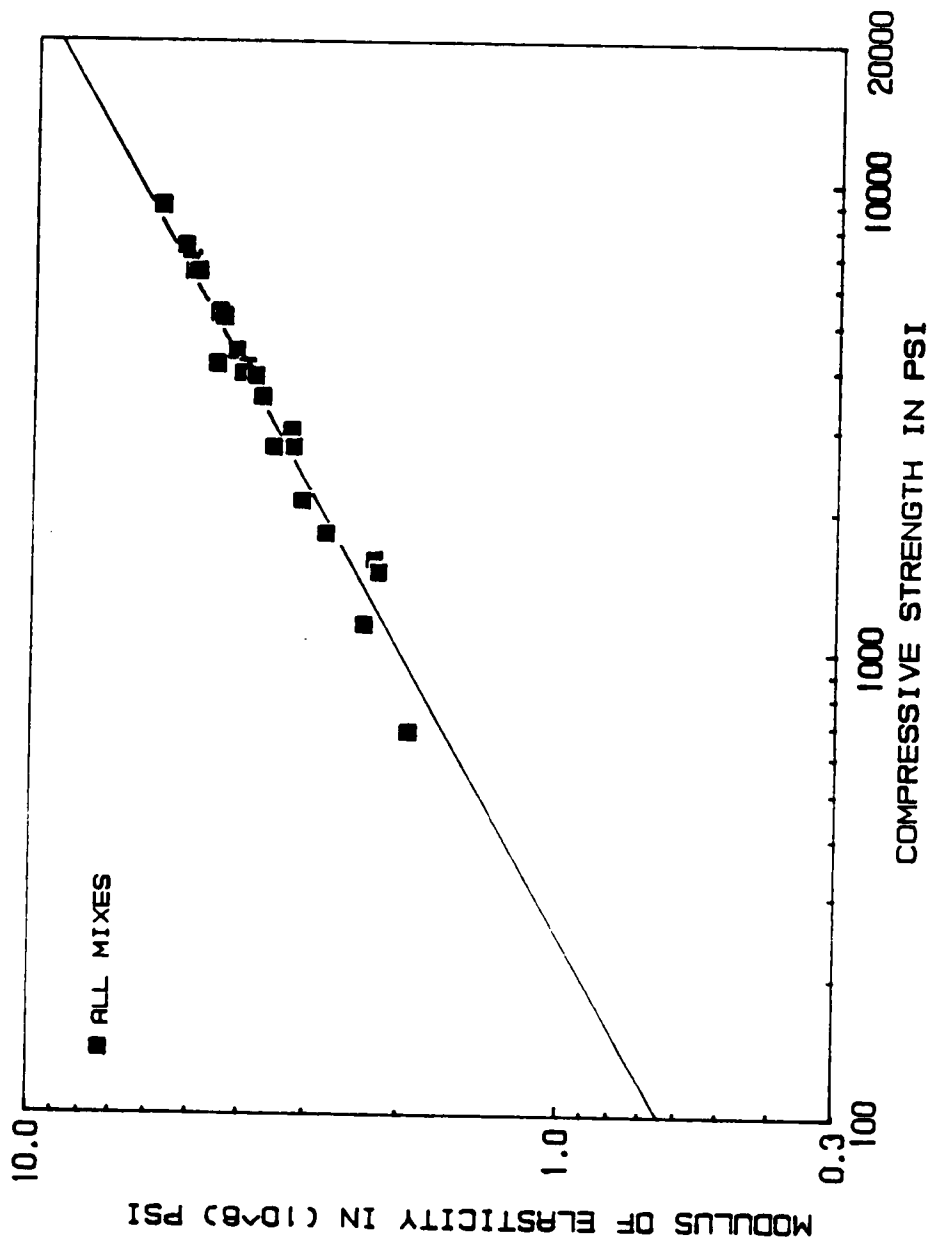


Figure 6-32: Log-Log Plot of Elastic Modulus Versus Compressive Strength for $f_c \geq 500$ psi

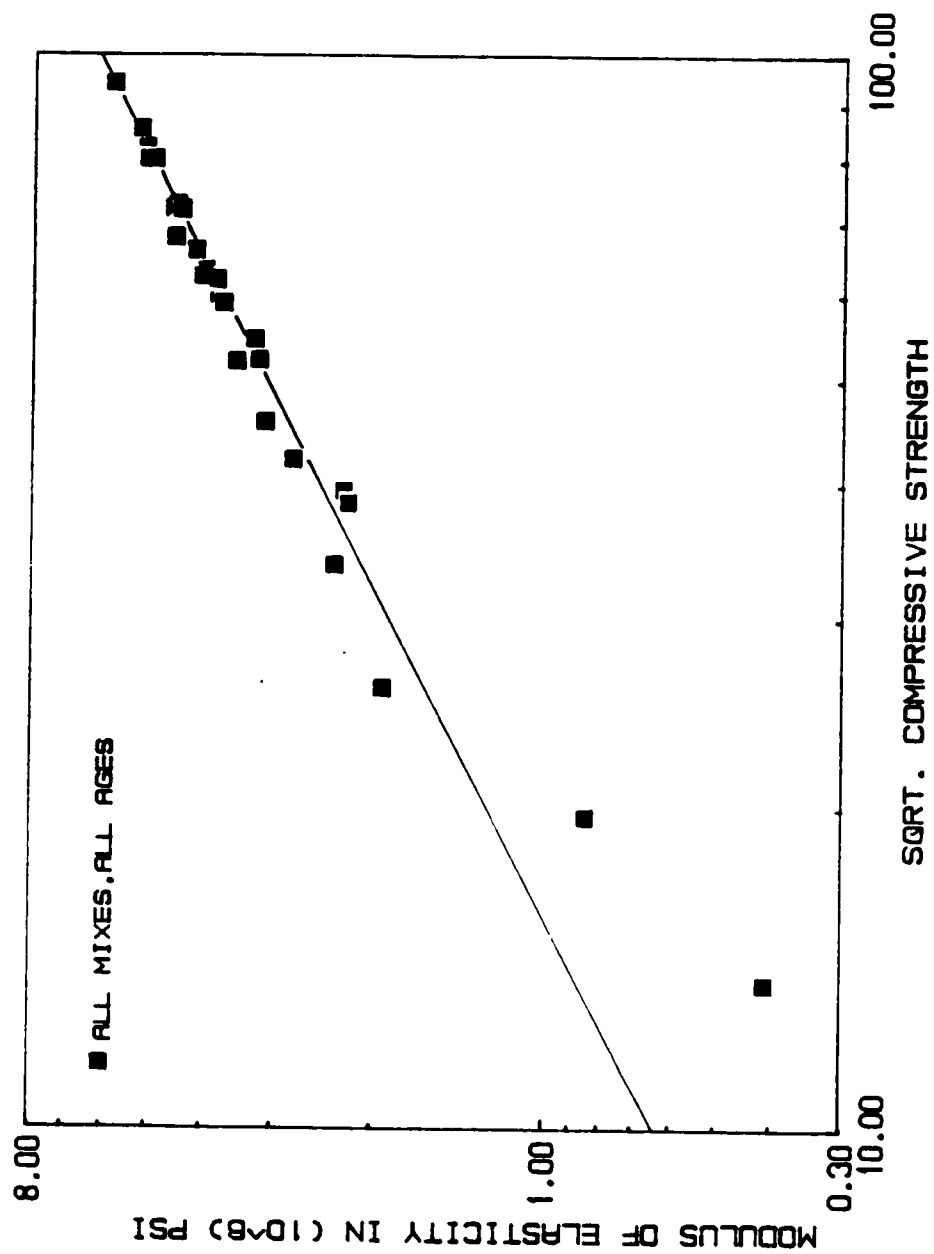


Figure 6-33: Log-Log Plot of Elastic Modulus Versus Square Root of Compressive strength (All Data Combined)

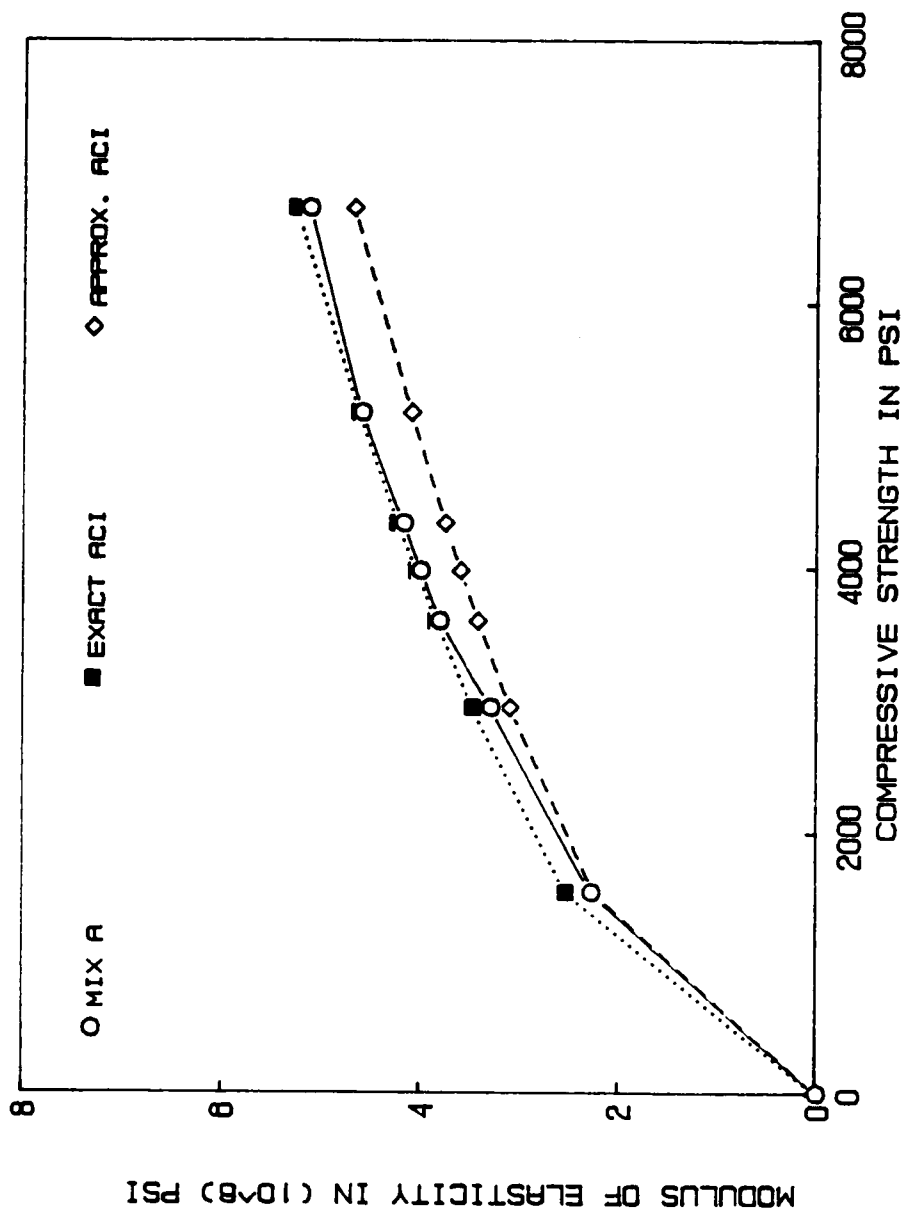


Figure 6-34: Comparison of Measured Elastic Modulus with ACI Predictions (Mix A, Moist Cured)

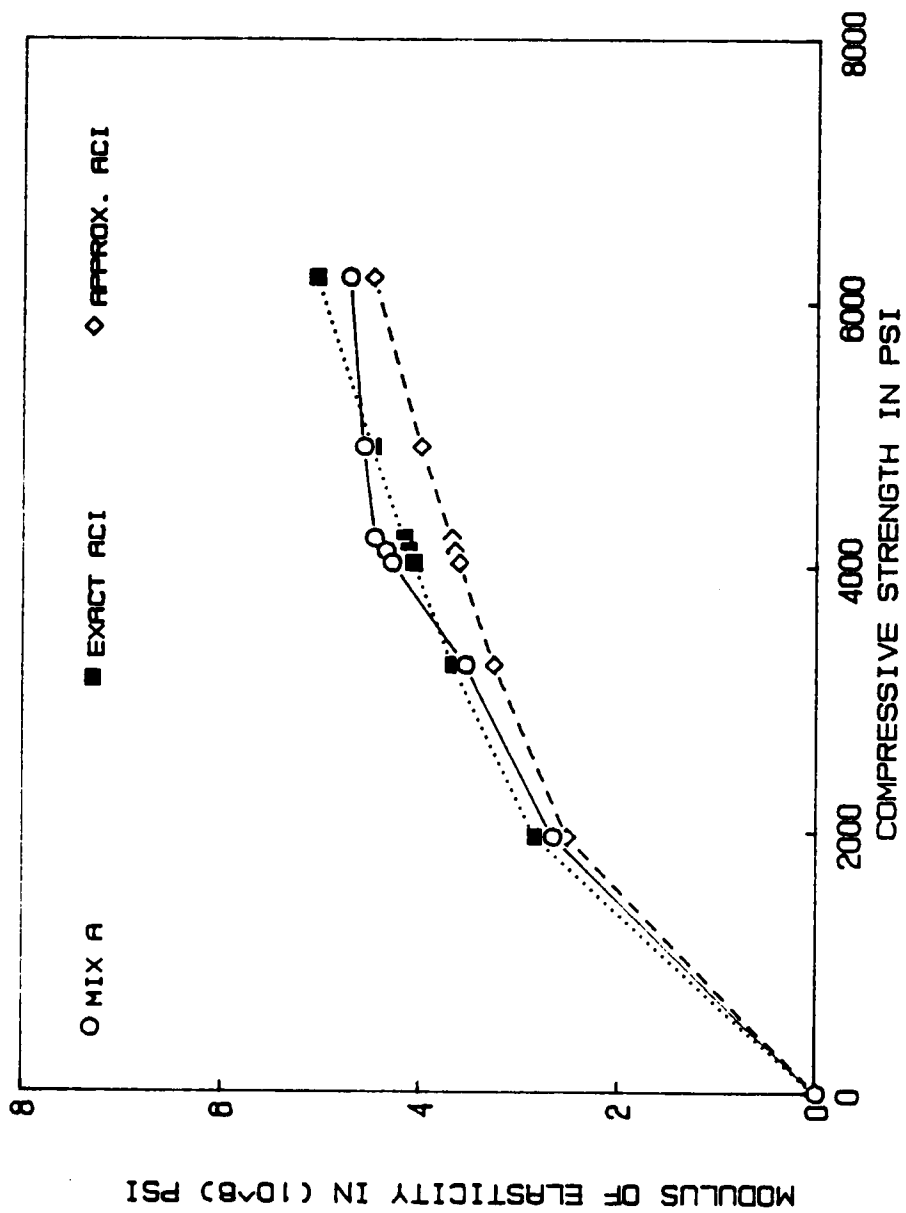


Figure 6-35: Comparison of Measured Elastic Modulus with ACI Predictions (Mix A, Steam Cured)

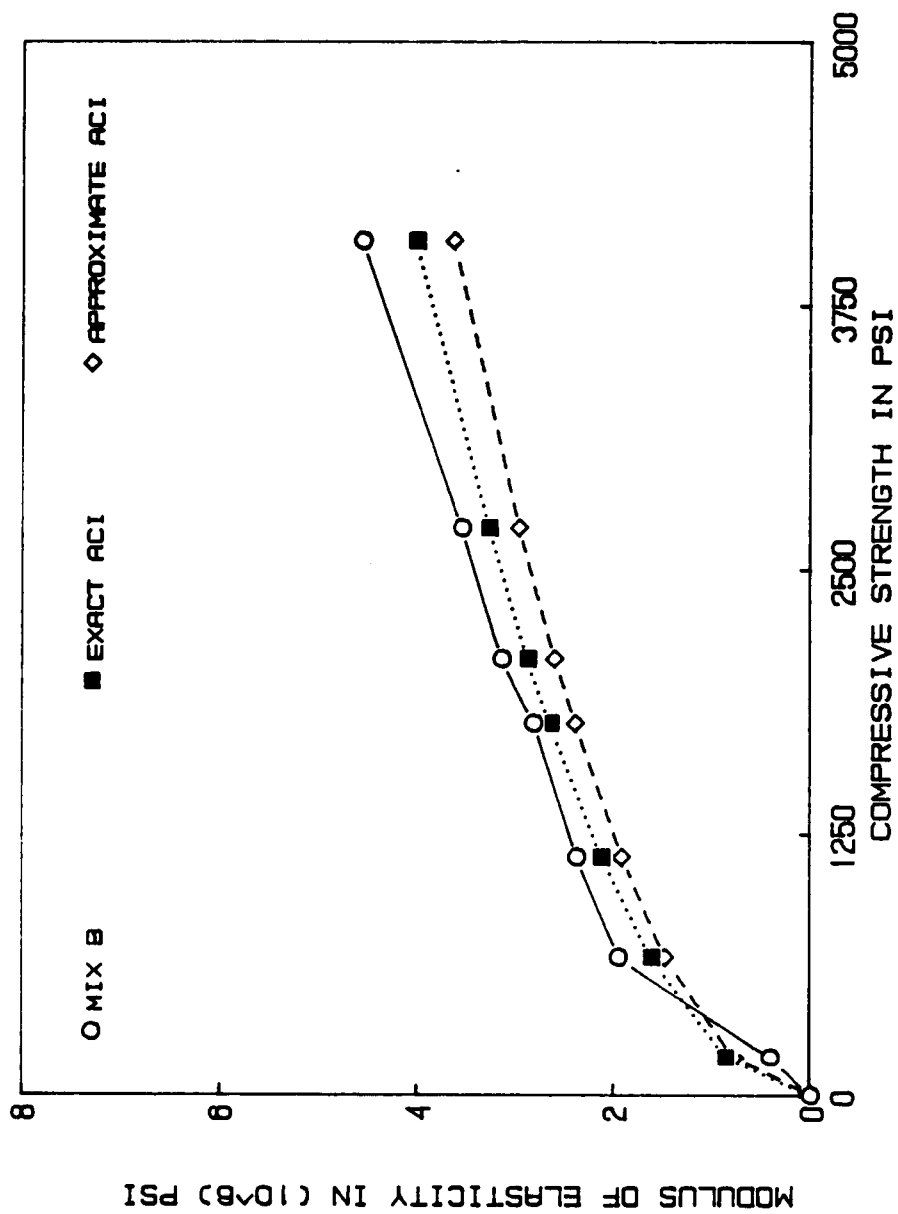


Figure 6-36: Comparison of Measured Elastic Modulus with ACI Prediction (Mix B)

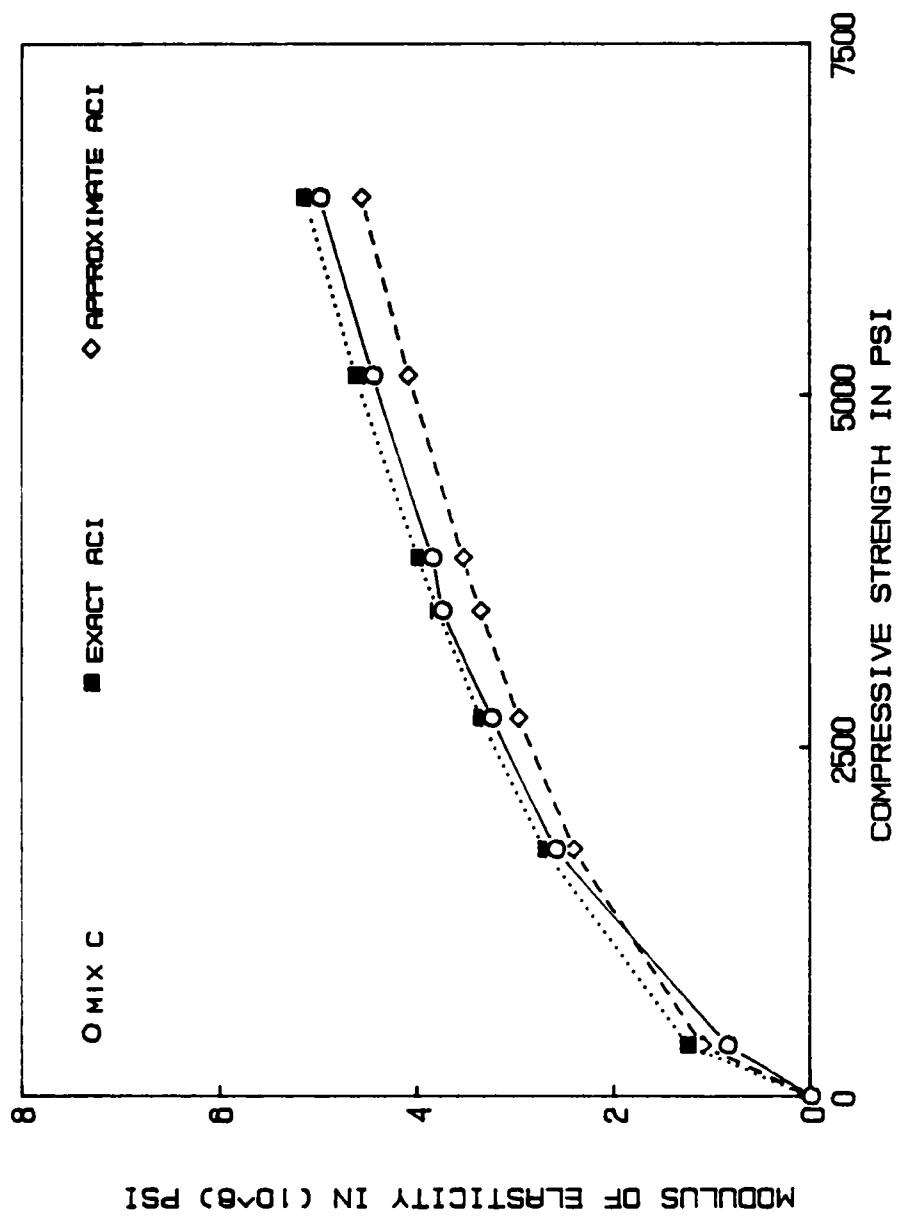


Figure 6-37: Comparison of Measured Elastic Modulus with ACI Prediction (Mix C)

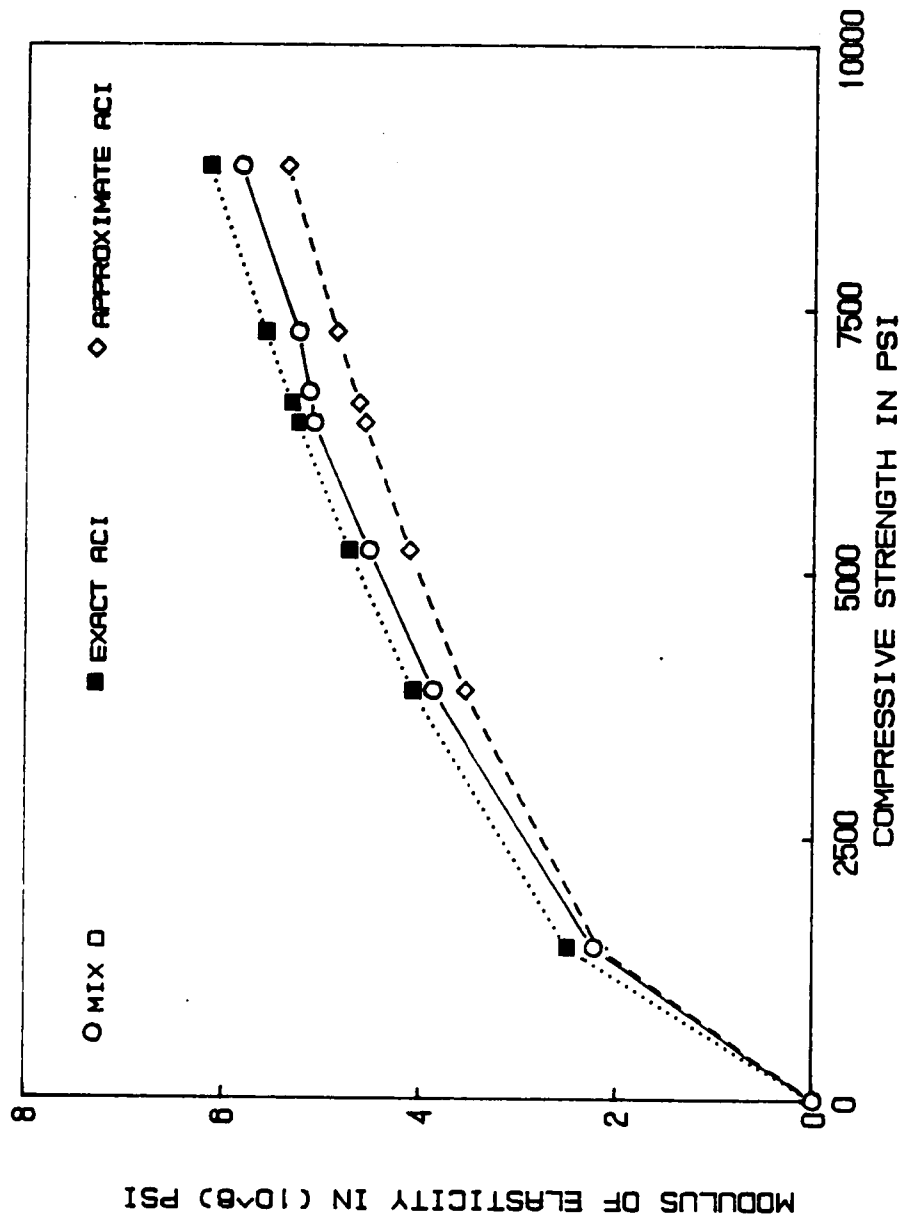


Figure 6-38: Comparison of Measured Elastic Modulus with ACI Prediction (Mix D)

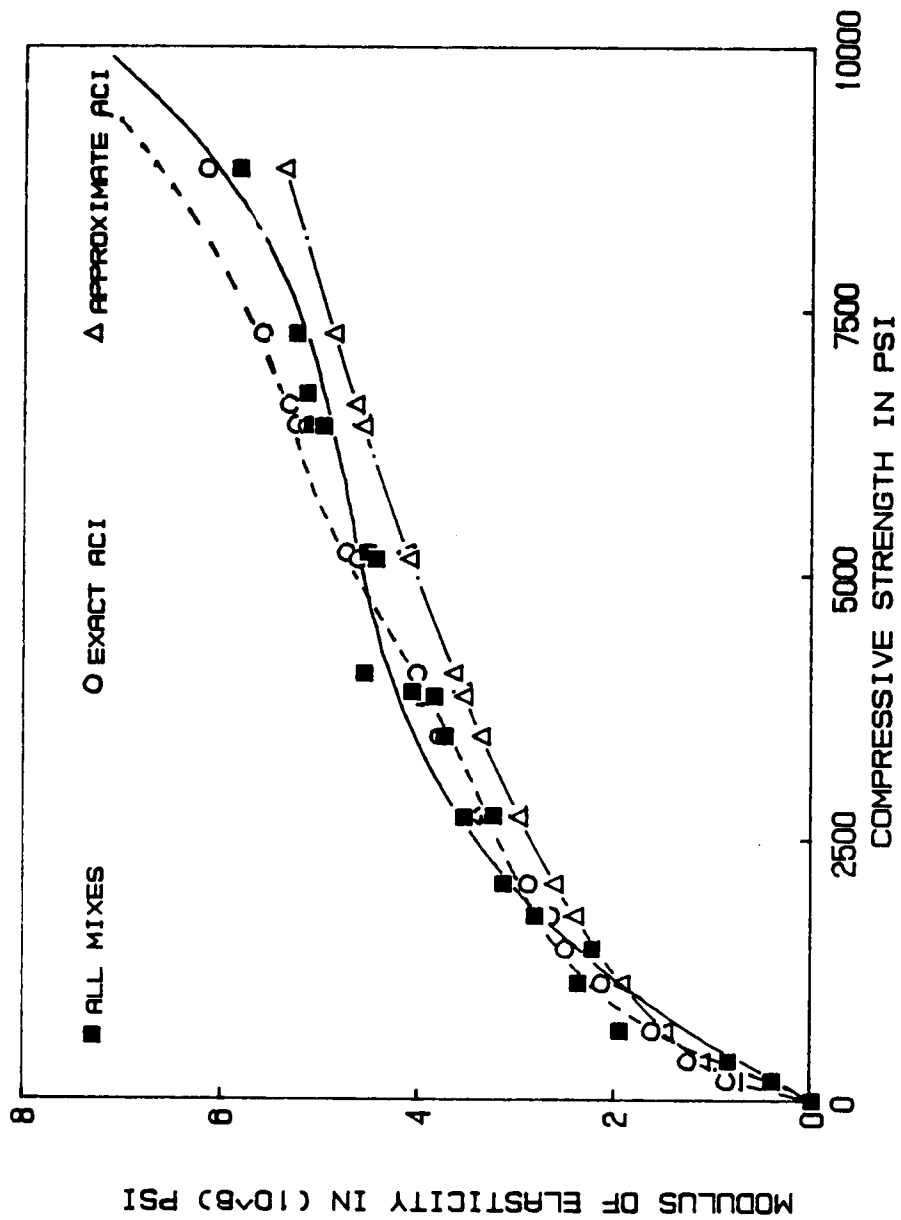


Figure 6-39: Comparison of Measured Elastic Modulus with ACI Prediction (All Data Combined)

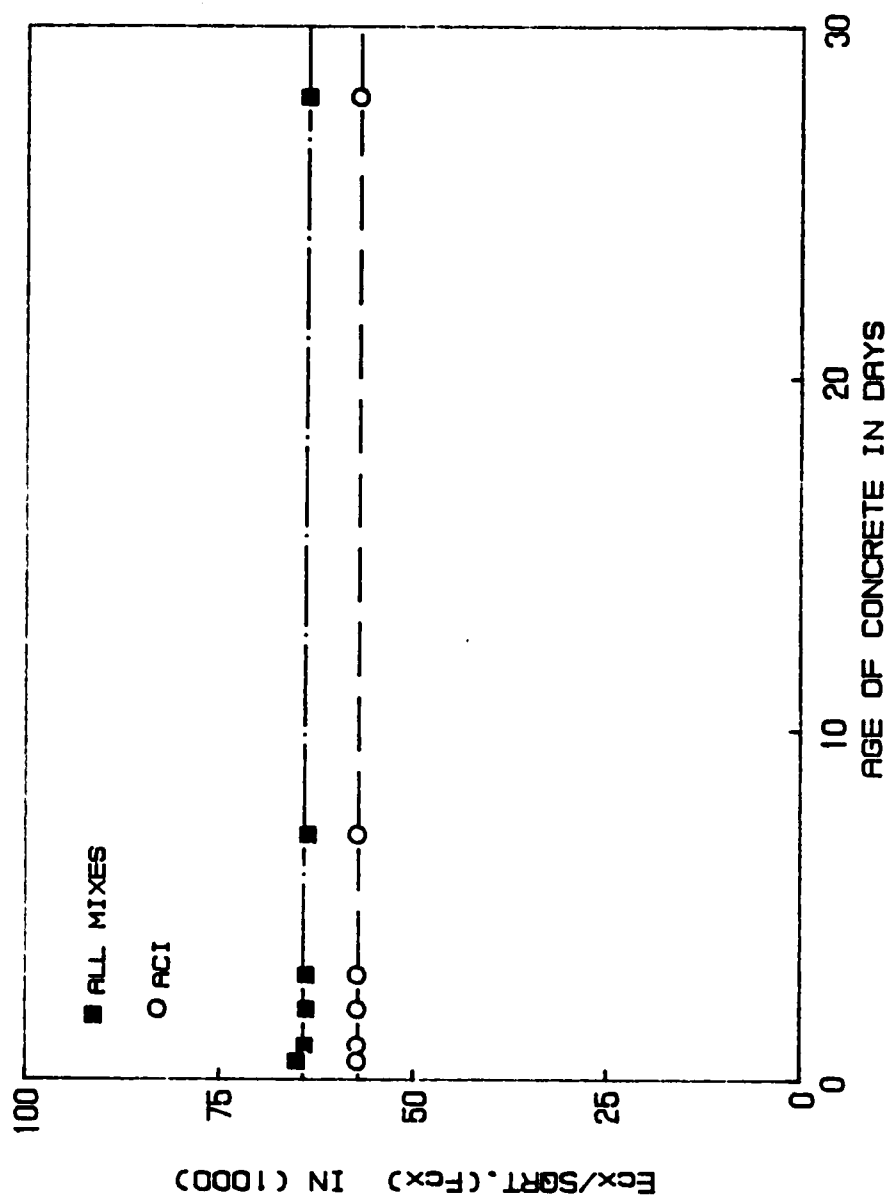


Figure 6-40: Ratio of Elastic Modulus to 0.5 Power of Compressive Strength Versus Age

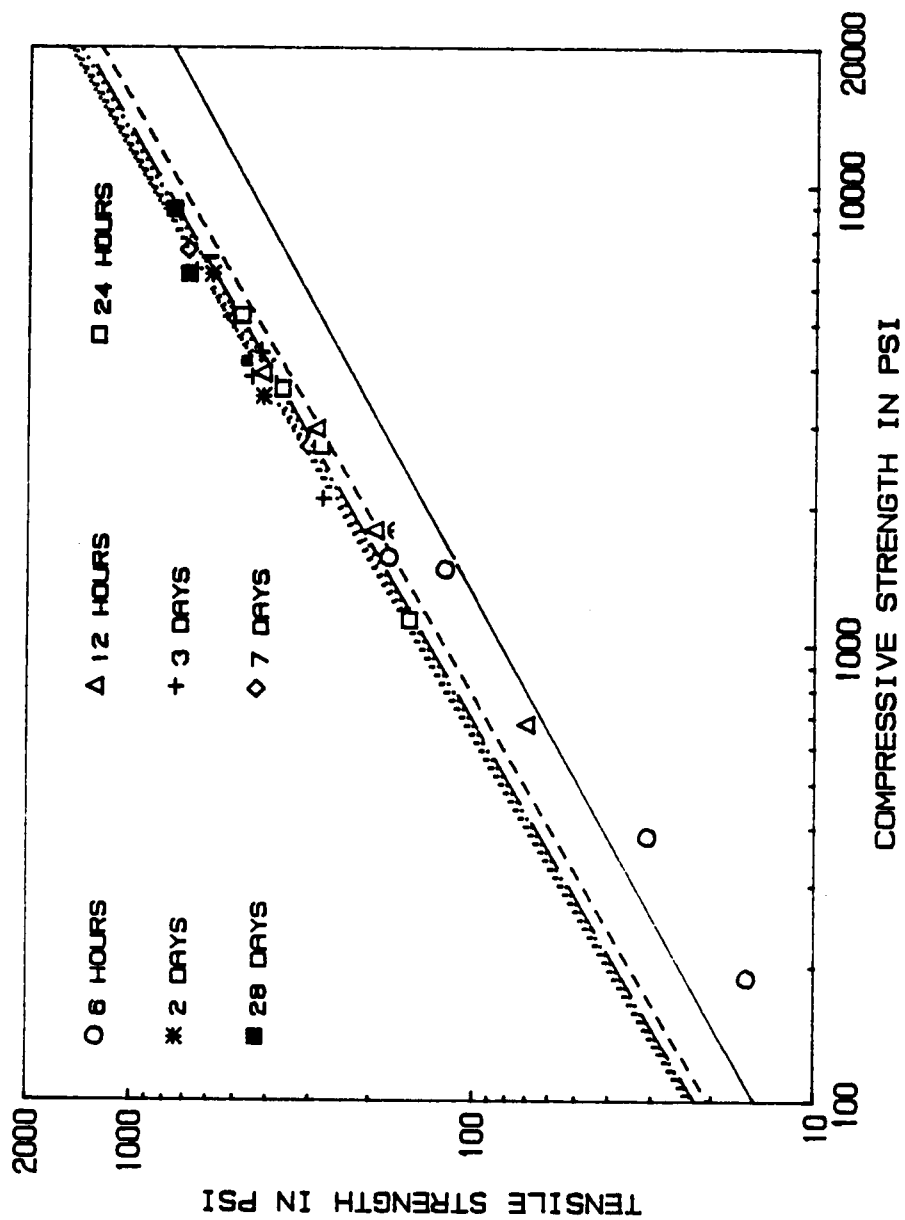


Figure 6-41: Log-Log Plot of Tensile Strength Versus Compressive Strength (Each Age)

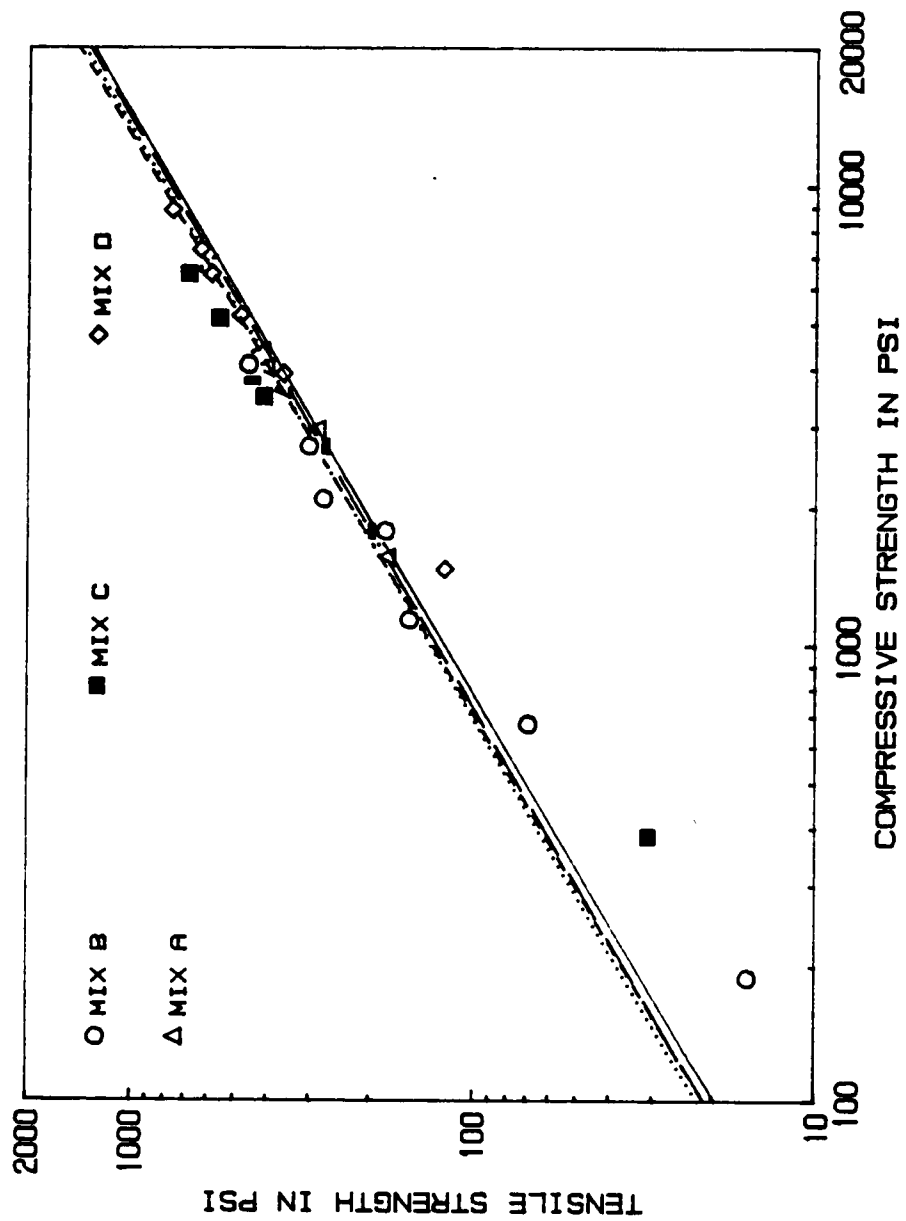


Figure 6-42: Log-Log Plot of Tensile Strength Versus Compressive Strength (Each Mix)

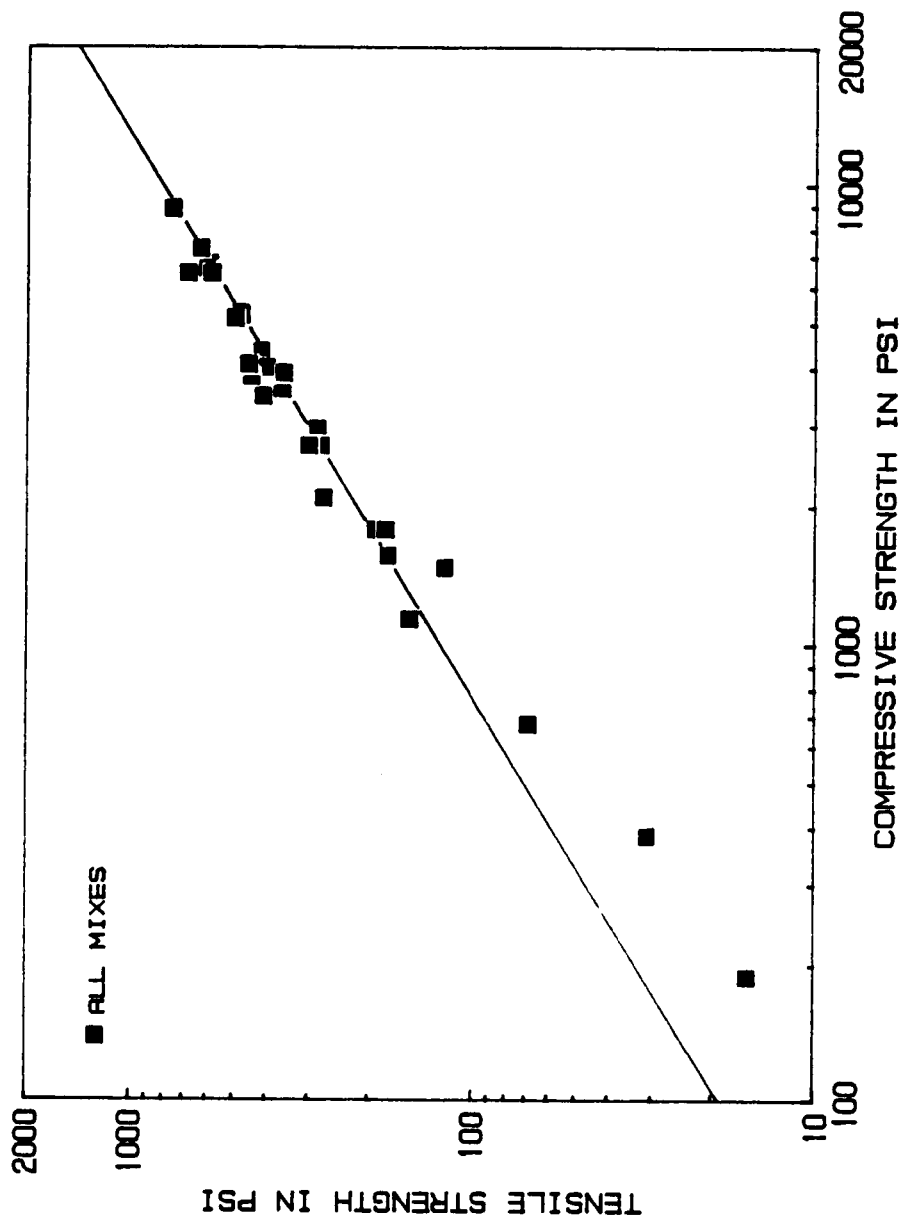


Figure 6-43: Log-Log Plot of Tensile Strength Versus Compressive Strength (All Mixes, All Ages Combined)

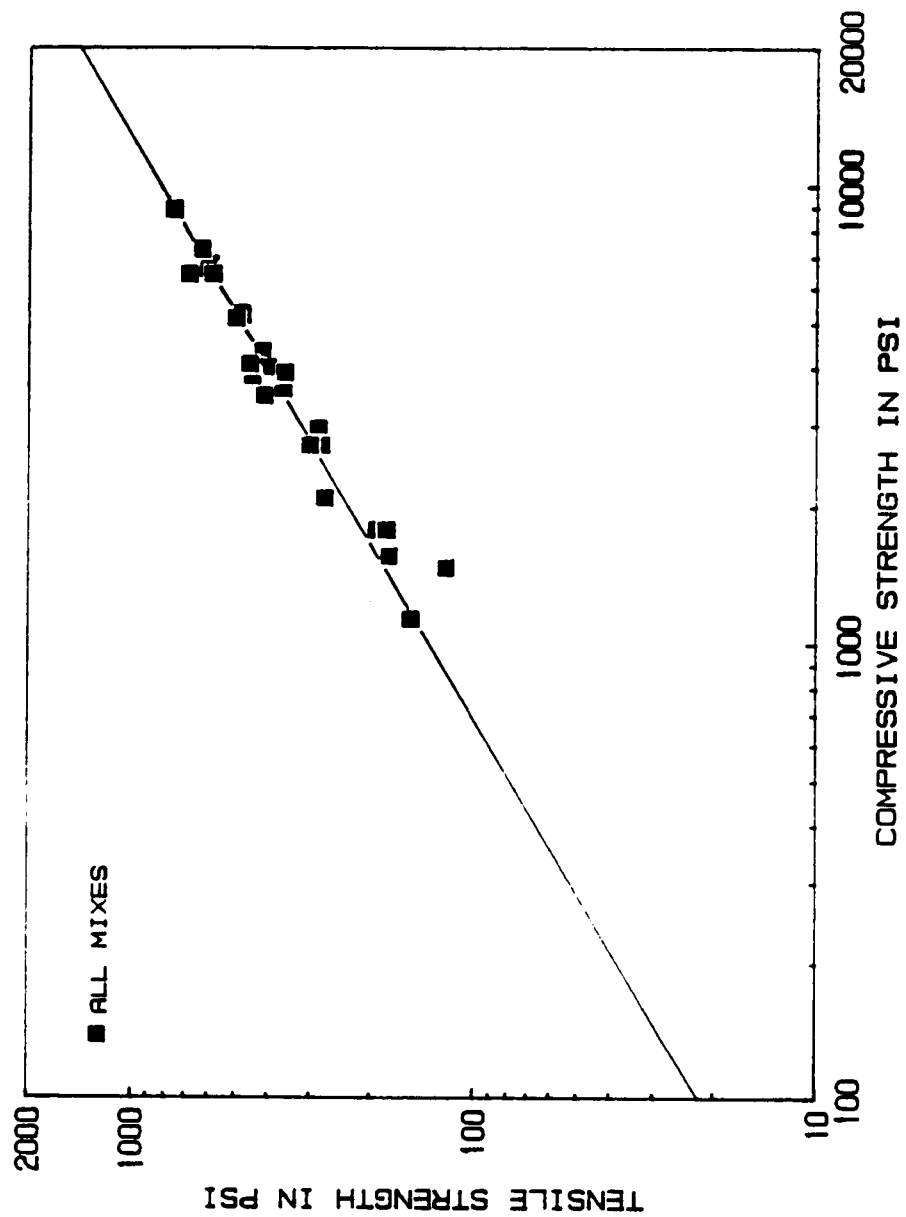


Figure 6-44: Log-Log Plot of Tensile Strength Versus Compressive Strength ($f_c \geq 1000$ psi)

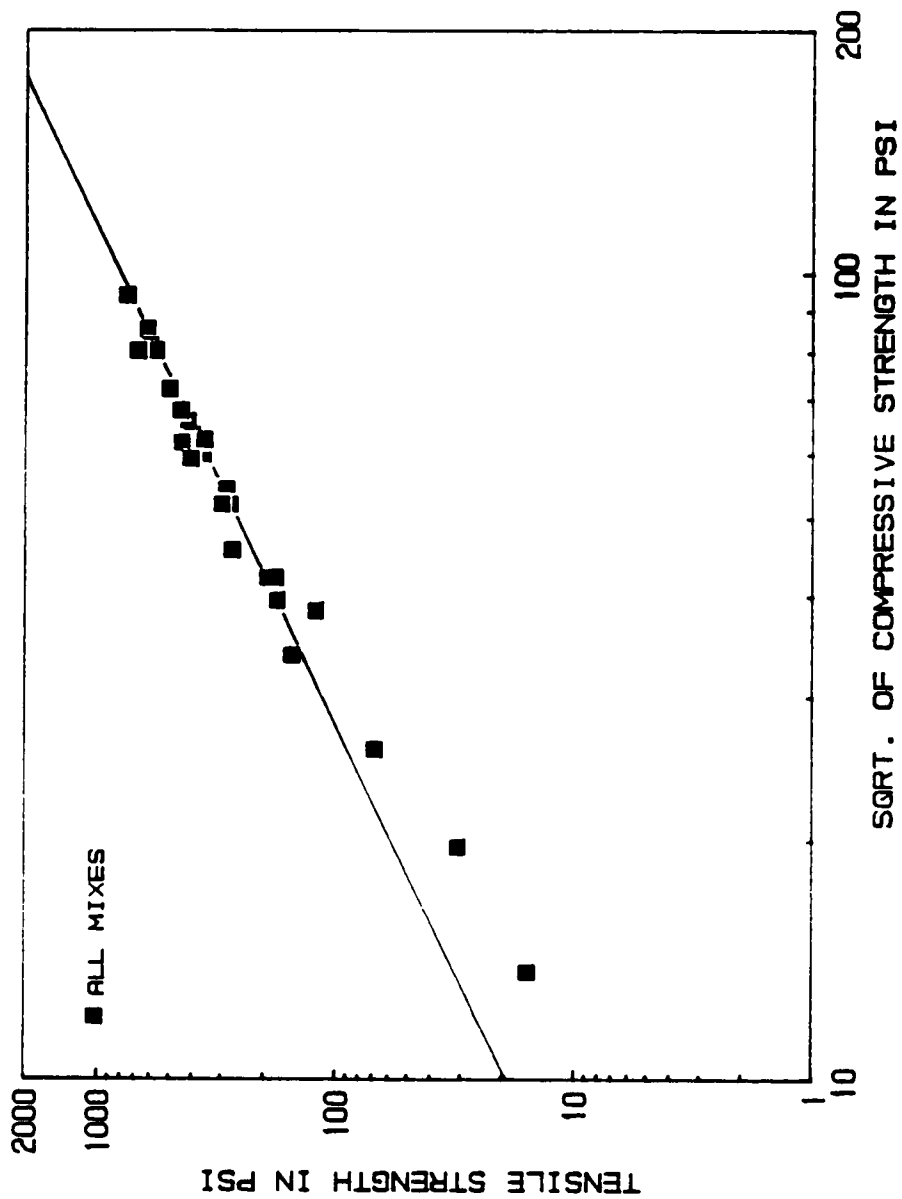


Figure 6-45: Log-Log Plot of Tensile Strength Versus Square Root of Compressive Strength (All Mixes, All Ages Combined)

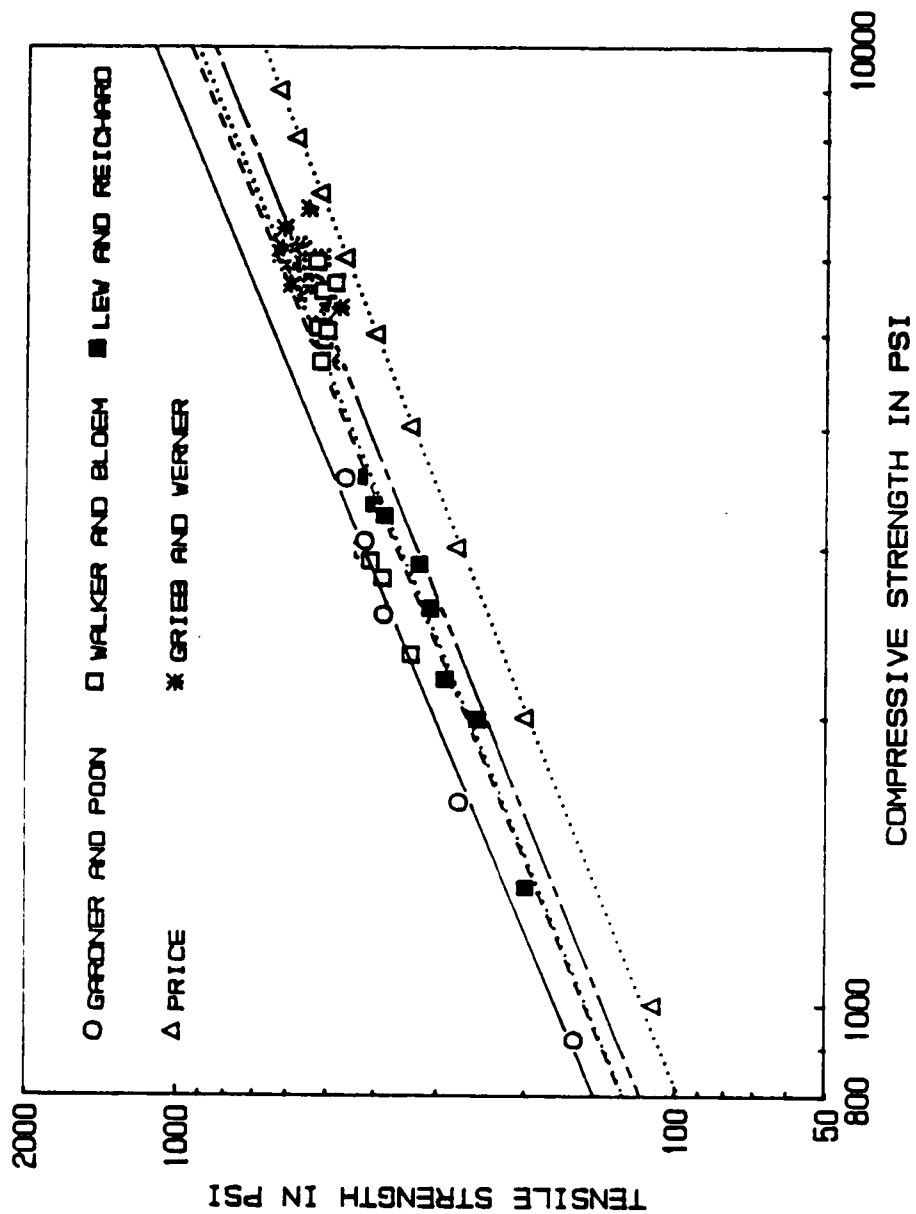


Figure 6-46: Log-Log Plot of Tensile Strength Versus Compressive Strength (Other Sources)

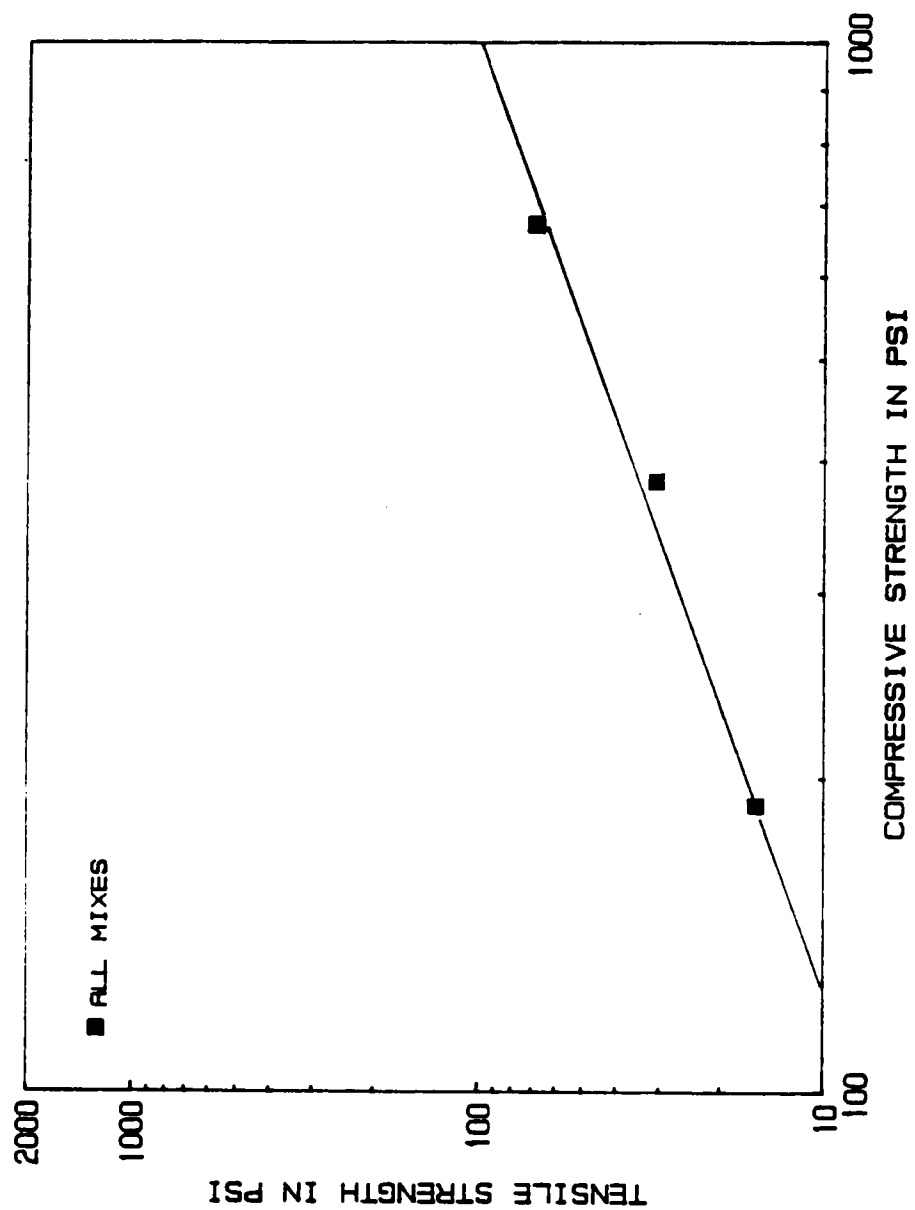


Figure 6-47: Log-Log Plot of Tensile Strength Versus Compressive Strength (For $f_{cx} < 1000$ psi)

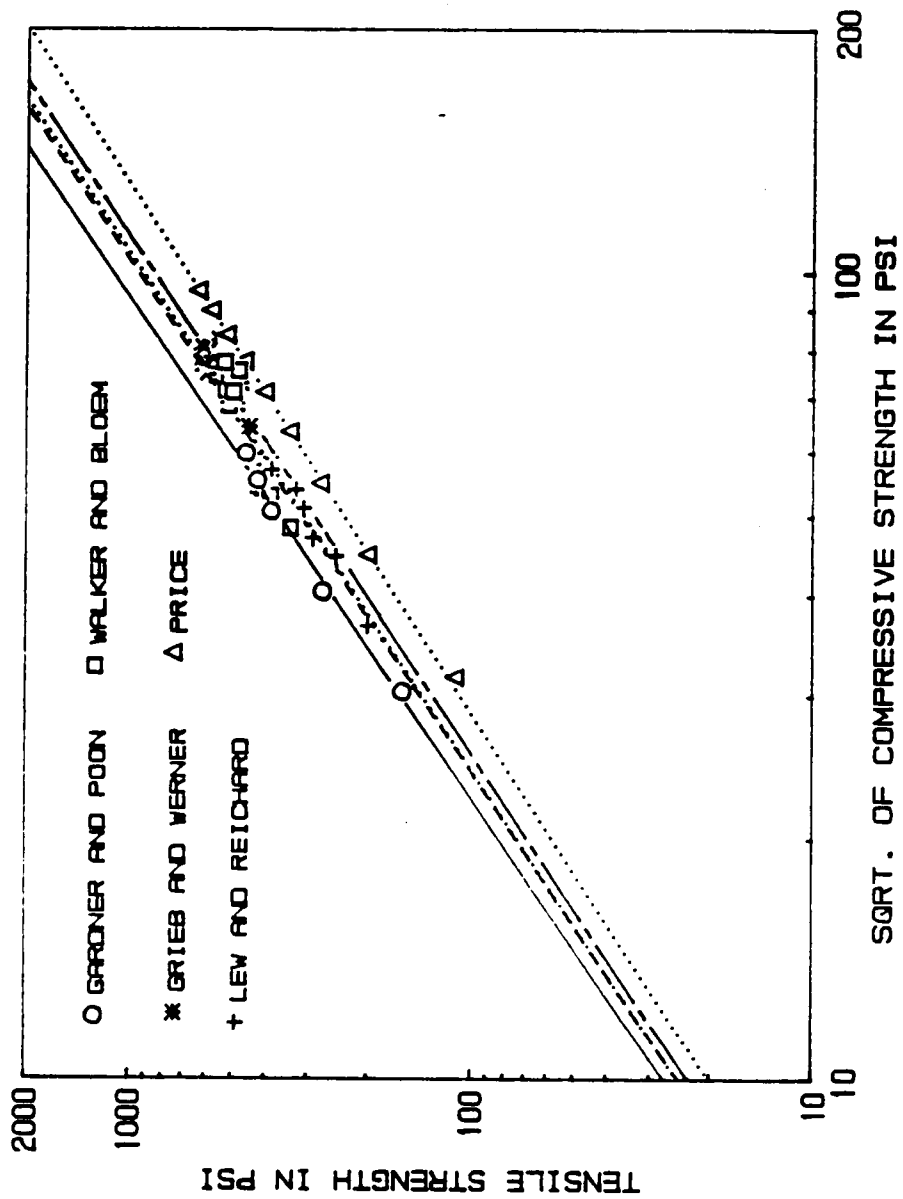


Figure 6-48: Log-Log Plot of Tensile Strength Versus Square Root of Compressive Strength (Other Sources)

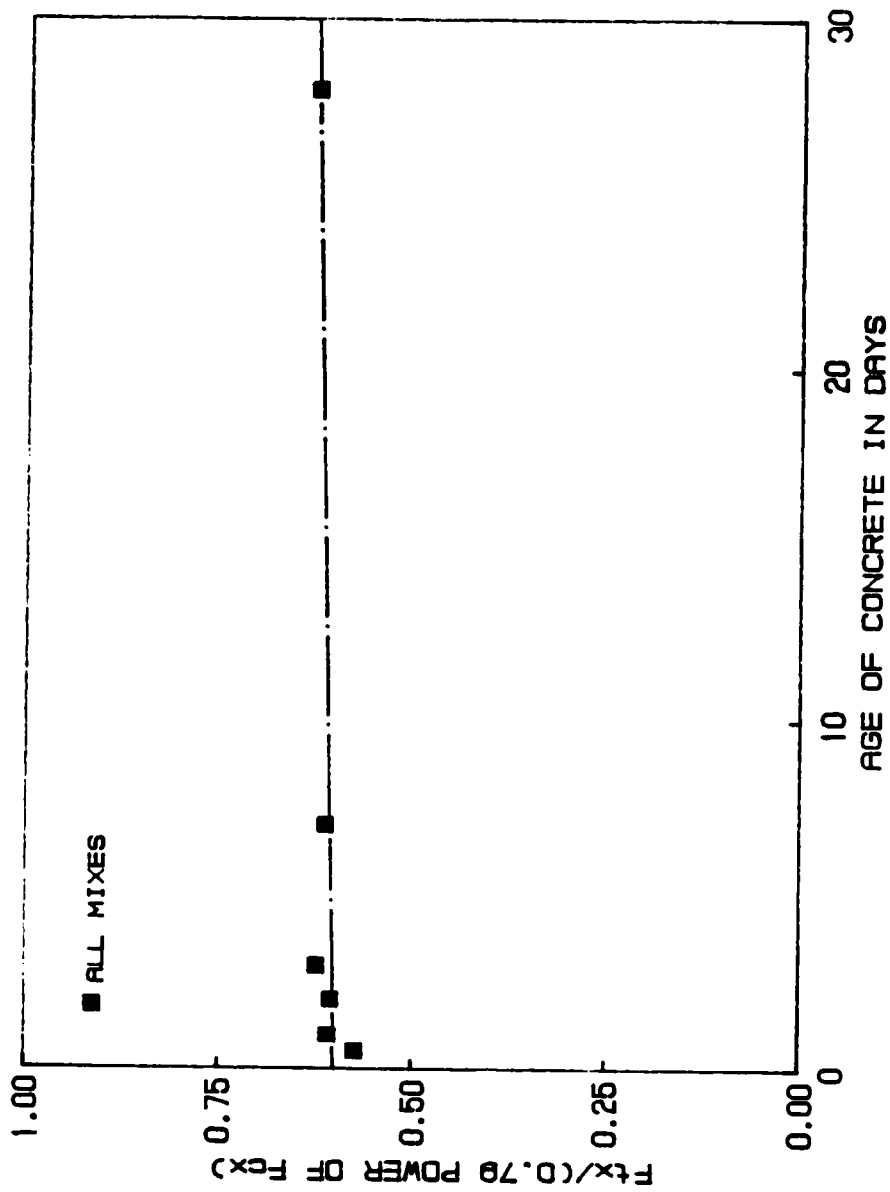


Figure 6-49: Ratio of Tensile Strength to 0.79 Power of Compressive Strength Versus Age

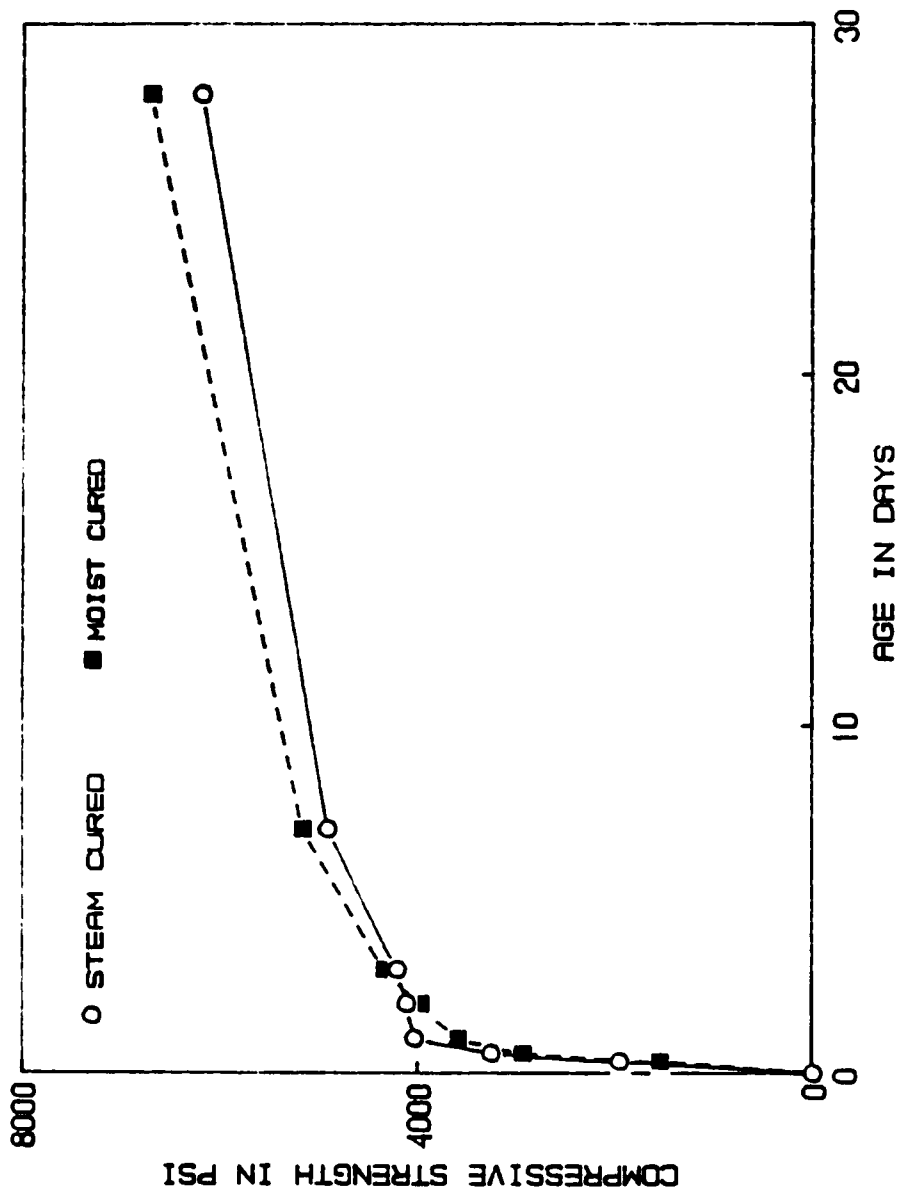


Figure 6-50: Comparison of Compressive Strength Development in Moist and Steam Cured Concrete

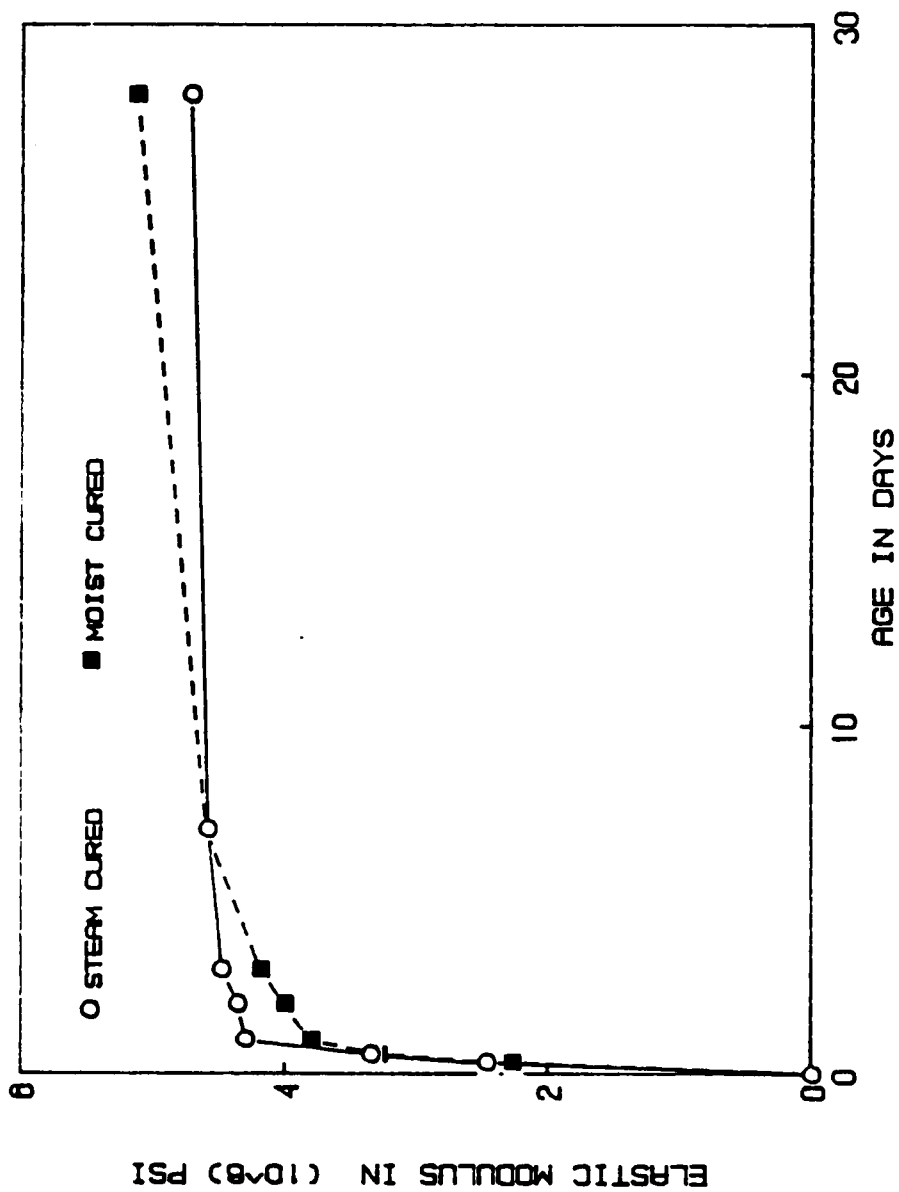


Figure 6-51: Comparison of Elastic Modulus Development in Moist and Steam Cured Concrete

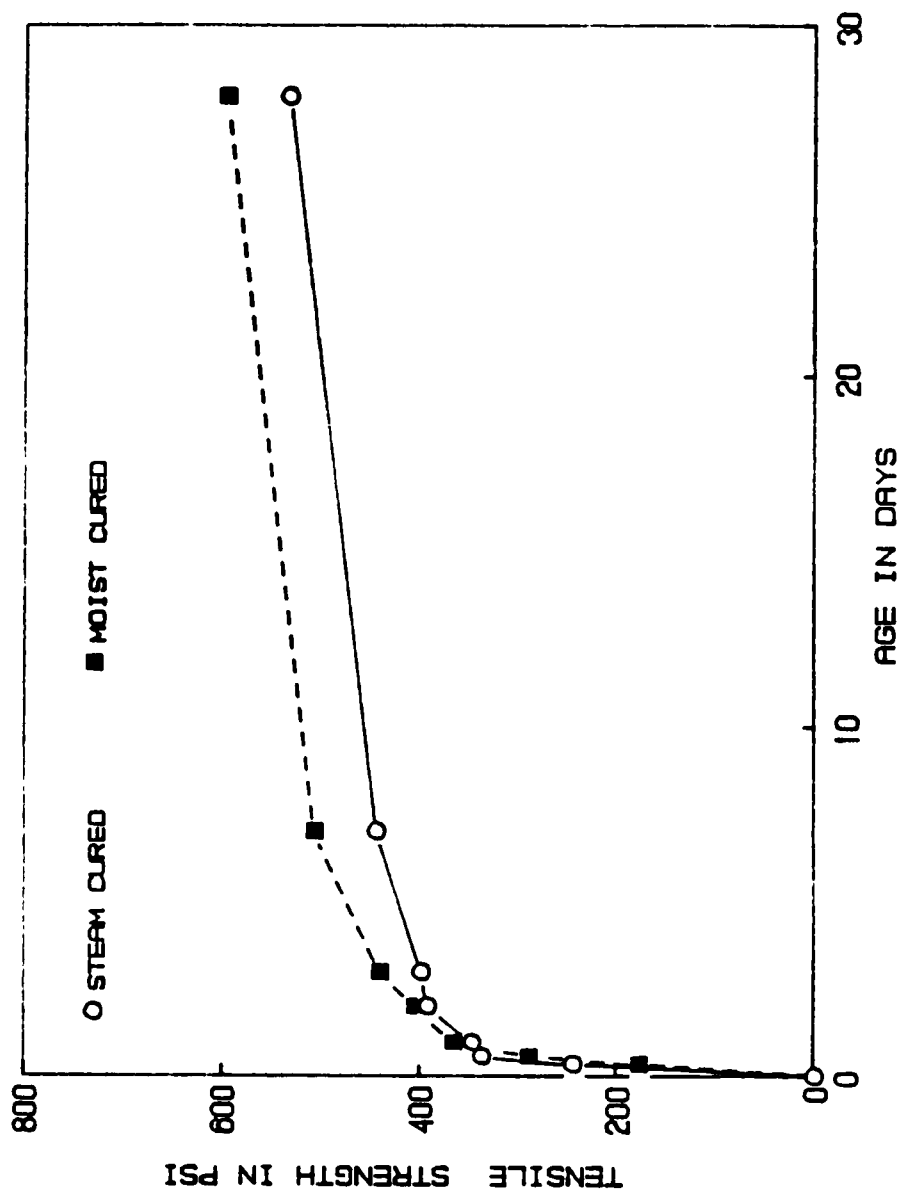


Figure 6-52: Comparison of Tensile Strength Development in Moist and Steam Cured Concrete

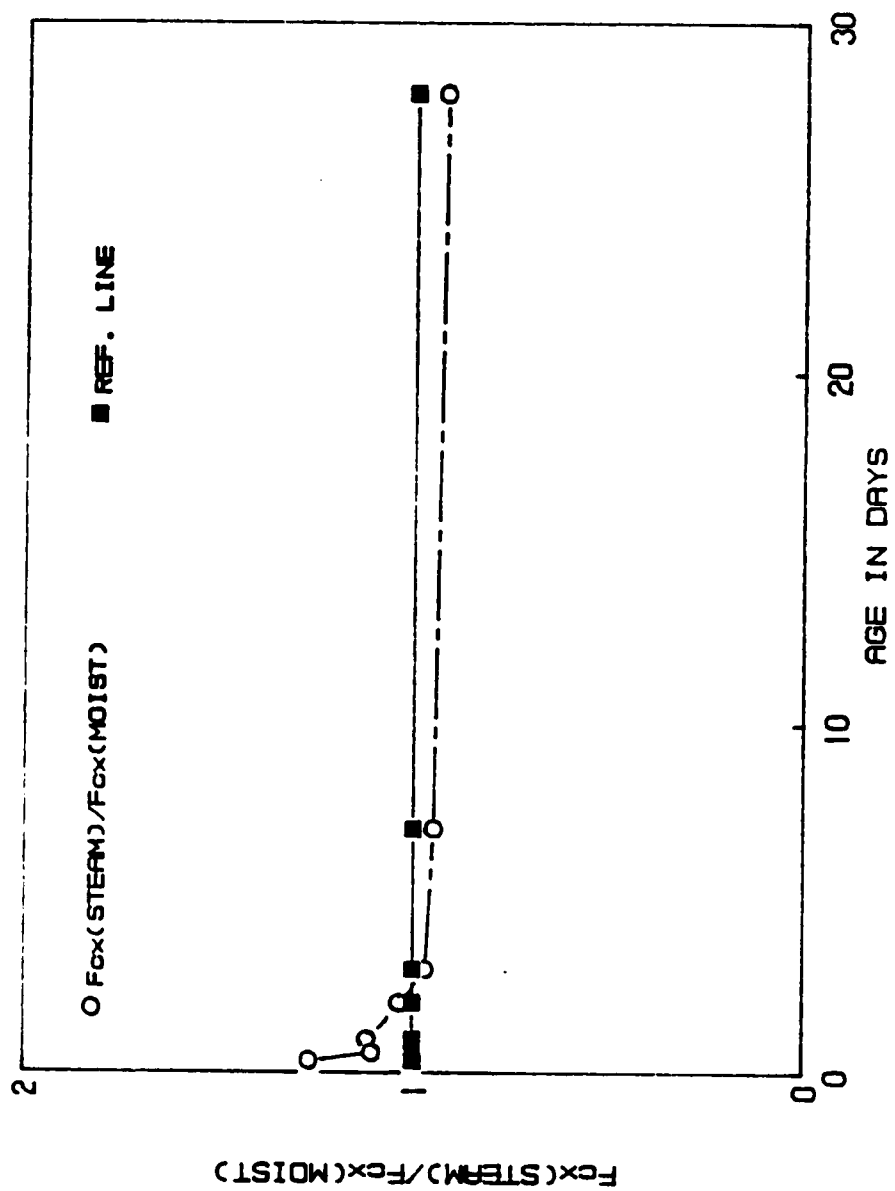


Figure 6-53: Normalized Compressive Strength for Steam Cured Versus Age

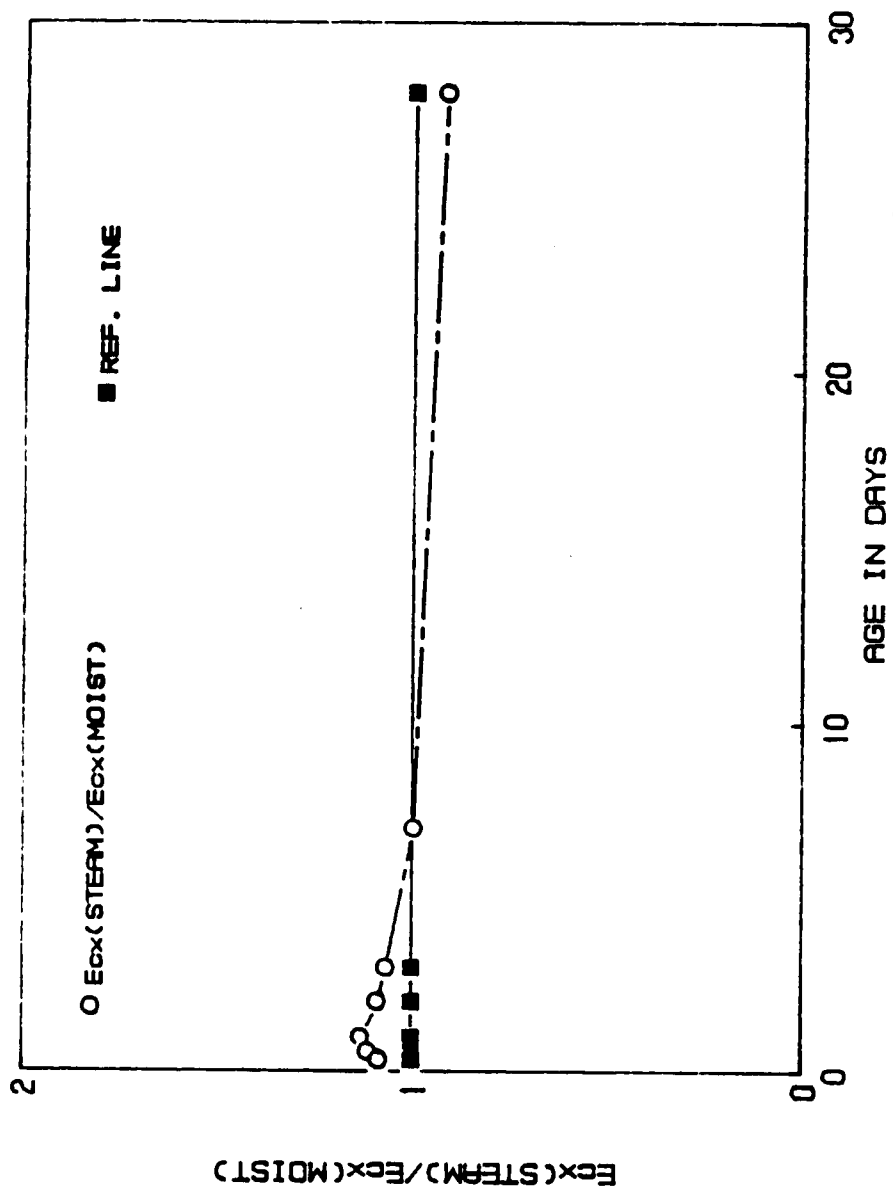


Figure 6-54: Normalized Elastic Modulus for Steam Cure Versus Age

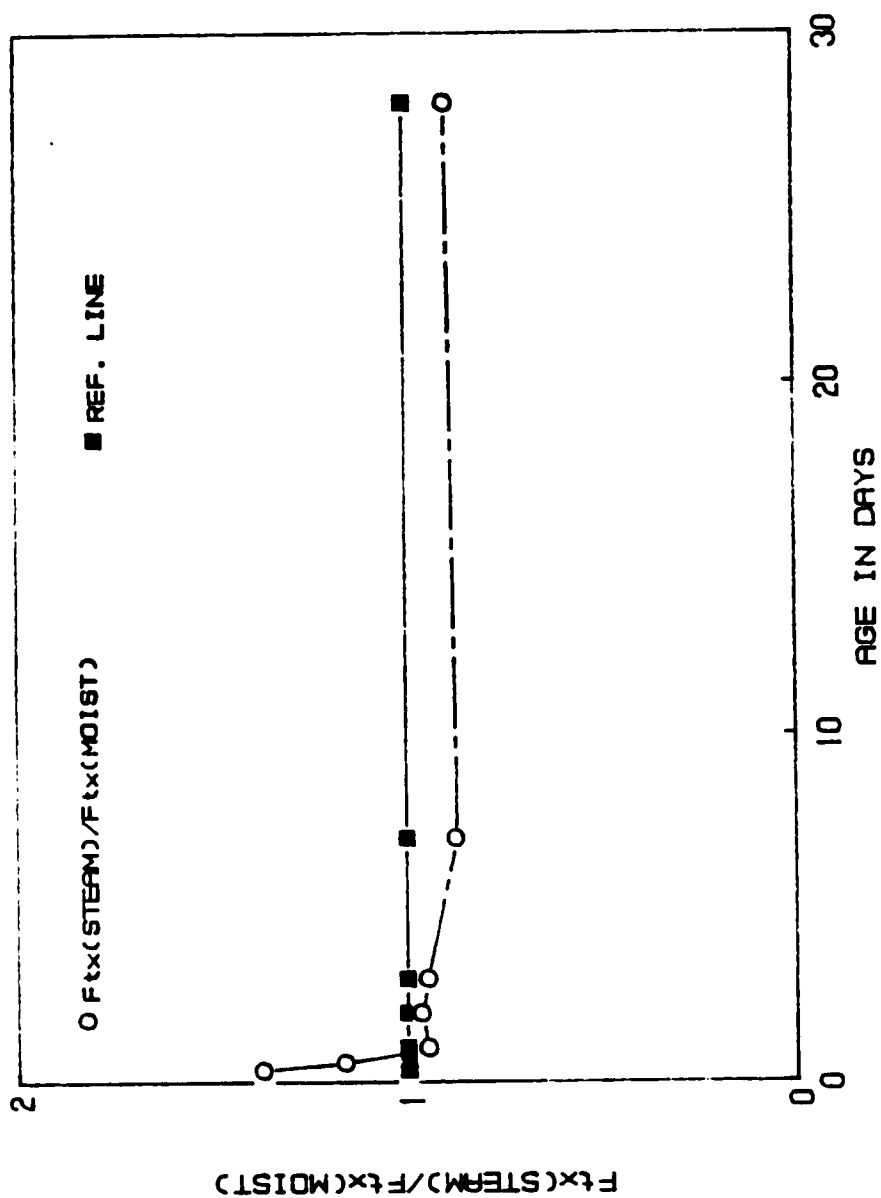


Figure 6-55: Normalized Tensile Strength for Steam Cure Versus Age

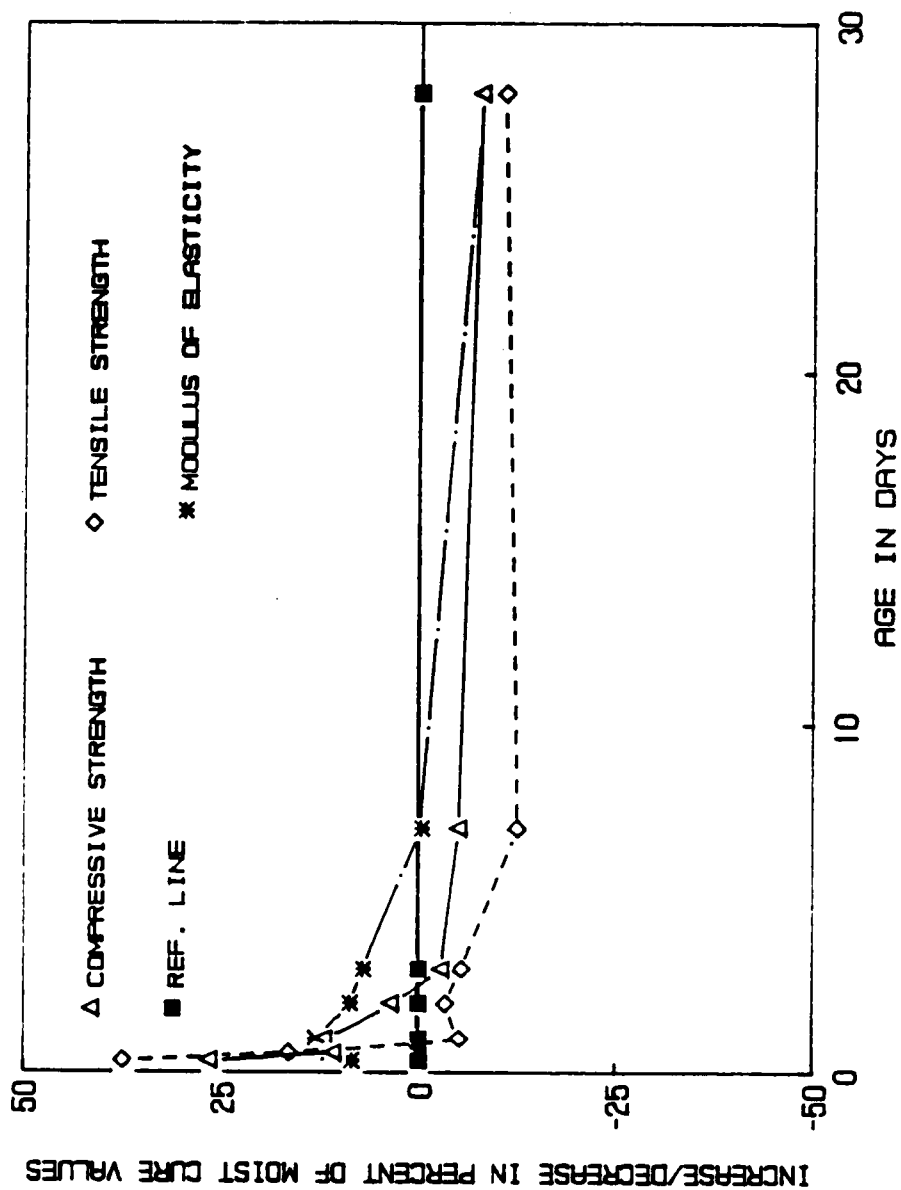


Figure 6-56: Increase or Decrease in Physical Property Development Due to Steam Curing

VITA

Francis Ajiboye Oluokun was born in Ogbomoso, Oyo State, Nigeria on April 19, 1950. He attended elementary school in that city and was graduated from Ogbomoso Grammar School (Paku) in December 1969. The following September he enrolled at the University of Ife, Nigeria, for a brief period of time. He later attended the University of Tennessee, Knoxville, and in December 1979, he received a Bachelor of Science degree in Civil Engineering. In August 1980 he received his professional Master of Engineering degree in Civil (Structural) Engineering.

After graduation, Mr. Oluokun was employed by Bechtel Power Corporation, USA, as a structural design engineer until 1981. At this time he returned to his home country and he was employed by the University of Ibadan, Nigeria, as a lecturer of Civil Engineering until 1986. After six years as a practicing engineer Mr. Oluokun returned to the University of Tennessee to pursue his doctoral degree in Civil Engineering. He received his Doctor of Philosophy degree in May, 1989.

Mr. Oluokun is a registered engineer in Nigeria. He is a member of the Nigerian Institute of Structural Engineers, American Concrete Institute, Chi Epsilon, Gamma Beta Pi, Tau Beta Pi, and an associate member of the American Society of Civil Engineers.