EFFECT OF PRE-STRESSING ON THE DURABILITY OF PORTLAND CEMENT CONCRETE

by

Khan Shahid Jamil

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I. INTRODUCTION

Due to the phenomenal pace of modern construction and the accompanying economic considerations, the use of pre-stressed concrete has advanced to such a stage that detailed information as regards its various physical characteristics has become of paramount importance to the engineers. Pre-stressed concrete offers many distinct advantages and is responsible for many new improvements introduced during the recent past in the field of civil engineering. The increased use of pre-stressed concrete in bridge construction is becoming more and more evident. If certain factors such as cost of labor, especially applicable in the United States, are held down by further mechanization and improvisation of the methods of construction, it is stipulated that the pre-stressed concrete will hold tremendous potentialities for eventual use in the construction of highway and airfield pavements. Already, several airfield pavements using this construction material have been built on an experimental basis in the United States, and the results of their field performance data on a wide range of loadings, foundation and weather conditions and construction methods are being evaluated so as to check existing design theories and develop valid criteria for establishing dimensions, magnitude of prestress and for properly accounting for various foundation conditions.

Presently, the methods adopted in the construction of prestressed pavements are, generally speaking, more complex than those required for conventional pavements. It is, therefore, imperative that simple and economical construction methods be developed so that full advantage of this versatile material may be taken in the construction of highway and airfield pavements. In Europe, prestressed concrete has been used on a relatively larger scale in the construction of highway and airfield pavements because the labor is cheaper there (7).

The advantages which a prestressed concrete pavement offers over a conventional rigid one are twofold:

First, design investigations and field testing done so far indicate that a more efficient use of construction materials in terms of required pavement thickness is permissible. This can best be illustrated by the following example which relates to the Vienna Airport (1). The price of one square yard of six inch thick prestressed slab is $5.00 which is the same as that of a nine inch non-prestressed slab. The same load-carrying capacity, however, could only be obtained by an 11 inch thick non-prestressed slab.

Secondly, prestressed pavements can be designed with fewer joints and with less probability of cracking than conventional rigid pavements, thereby promising extended pavement life and reduced maintenance costs under normal conditions.

The effect of freezing and thawing action on ordinary concrete has been investigated quite extensively, but this is not true in the case of prestressed concrete which in itself is a relatively new field. Due to the widespread use of this material during the last twenty years in bridge construction, where, however, the effect of natural weathering action is not so severe, and its potential use in pavement construction (in particular airfield pavements), which is more vulnerable to natural freezing and thawing action, it has become of immense necessity that further research in this direction be carried out, although it is generally believed that concrete under compression will exhibit excellent resistance to weathering (2).

There is another compelling reason which warrants further research in this field. Sources of aggregates of known satisfactory performance are being rapidly depleted in many areas of the United States. Some of the known deposits of excellent aggregates are being zoned out of existence by being reserved for residential areas. It has therefore become all the more imperative that efforts be made to utilize the aggregates which are known to perform poorly when subjected to adverse weathering action in the field. The highway and airfield paving industry being one of the major users of aggregates, and also pavements being the most vulnerable to natural weathering action, it is expected that it will go

a long way in relieving the scarcity of usable aggregates if an improved performance of a poor performing aggregate is realized, in the field, by prestressing the concrete made from it.

Accordingly, the objectives of this thesis are two-fold:

1) To study the durability of prestressed concrete made of poor performing aggregate when subjected to the freezing and thawing action.

2) To compare the freezing and thawing effects on prestressed concrete with that of ordinary concrete.

II. PAST HISTORY OF RESEARCH

Review of Literature

In 1949, Pendley conducted a study involving behavior of beams which were restrained from expansion and subjected to freezing and thawing (3). Although the beams he tested were not exactly prestressed in the sense we know, it was found that restraint of concrete reduced the magnitude of deterioration caused by exposure. He opined that the increase in durability of restrained beams was apparently due to prevention of cracking perpendicular to the longitudinal axis of the beam. It was noticed that the cracking pattern in the restrained beams was parallel to the beam axis as against the random pattern observed in the unrestrained beams.

An acceptable explanation of concrete failure when it is subjected to repeated cycles of freezing and thawing was the hypothesis presented by Powers (4). This hypothesis presents the action of frost as being the cause of hydraulic pressure that tends to cause the deterioration of concrete. This hydraulic pressure depends upon many factors, which, in order of importance, are as follows:

1) The permeability of the material through which the water must flow to escape from the saturated region on the surface of the test specimen during the cooling phase of the cycle;

2) Degree of saturation; and

3) The absorptivity of the test specimens.

When freezing starts, the water near the outer edges of the concrete freezes, thereby blocking the outer drainage of the inside water. As freezing continues the still unfrozen water is pushed inside by the resulting expansion in the volume due to formation of ice. Since in normal concretes the size of the pores is very small, the water moves inside with very high velocities, thus developing very high hydraulic pressures. When concrete sets, the quantity of water inside reduces due to hydration and evaporation which results in the formation of pores or "capillary voids." The hydraulic pressures generated during freezing depend partly on the distance between these capillary voids. Air entrainment provides in the concrete a system of small air voids which serve as pressure relief points, and thus prevent the formation of excessive hydraulic pressure during freezing. The influence of a uniaxial compressive stress, such as induced by linear prestressing, on the ability of concrete to resist internal stresses developed during freezing by the generation of hydraulic pressures, has been of much interest in the recent past. It has been suggested that in the absence of, or in combination with, entrained air, the prestressing force may have an influence on the ability of concrete to resist weathering action in the field.

Gutzwiller and Musleh (5), after investigating the effects of prestressing on the durability of concrete, concluded that 1) the post-tensioning improves its durability against cyclic freezing and thawing; that 2) continuously post-tensioned concrete having a minimum ultimate strength of 5000 psi after 28 days is effectively more durable than the unstressed concrete having ultimate strength of 3000 psi after 28 days; and that 3) continuously prestressed concrete is effectively more durable than the intermittently stress-released concrete of the same mix. The level of prestress used in this study was 2000 psi.

A durability test program was conducted by Kansas State University under the direction of the Illinois Prestressed Concrete Association and on behalf of the Prestressed Concrete Institute (2). A group of test specimens were programmed to determine the effect of three primary variables on the durability of concrete; the variables being level of prestress in the concrete, method of curing (moist and steam), and type of cement (Type I and III). Beam specimens were pretensioned and the coarse aggregate used was crushed limestone with known good performance. Results of these experiments indicated that the effects of the introduced variables on the durability of the concrete could not be differentiated or even noticed. However, it was concluded that the high

quality concrete, particularly when under compression, is a highly durable construction material from the standpoint of weathering action, and that the use of Type III cement as a means in obtaining high strength concrete at an early age, will not be detrimental to the durability of the concrete.

Klieger (6) found that a prestressing force providing a compressive stress of 350 - 400 psi in the concrete had no significant influence on the resistance to freezing and thawing of either air-entrained or non-air-entrained concretes.

Laboratory Freezing and Thawing Tests of Concrete

The role of accelerated freezing and thawing as a test method to determine the resistance of concrete subject to severe weathering action has been very controversial. It is impossible to predict the durability of concrete in the field from laboratory tests because one cannot get precisely the same concrete in a test specimen as one would in a structure and then be able to subject that specimen to the same temperature changes, moisture gradients and load conditions as the structure itself in service. The purpose of the laboratory tests is to detect differences in resistance to freezing and thawing of different concretes that may be correlated with the field performance of similar concretes. Many experiments have been conducted with a view to develop

a single standard freezing and thawing test capable of yielding information that will indicate the potential resistance of the concrete. Presently, there are four standardized tentative methods recommended by the ASTM for the aforesaid purpose. These methods are fully described under ASTM Standards C290, C291, C292 and C310, and are respectively:

1) Rapid freezing and thawing in water,
2) Rapid freezing in air and thawing in water,
3) Slow freezing and thawing in water or brine, and
4) Slow freezing in air and thawing in water.

So far it has not been established which of the aforesaid methods gives the best results. However, none of these methods were intended to be used for predicting the behavior of prestressed concrete. These standard test methods do not give the same indications of durability because of differences in freezing rates and exposure to water. Also, even minor variations in the various phases of the test such as mixing, casting, curing and freezing and thawing may contribute to giving widely different results.

It is therefore essential that care be taken to maintain the same conditions throughout the duration of tests so that results obtained can give true indication of differences in behavior of different types of concretes. It may be noted that these tests do not intend to duplicate the experiences of concrete in the field.

III. MATERIALS AND PROCEDURE

Programming of Test Specimens

Variables. As mentioned earlier, the purpose of this thesis was to study the effect of prestress on the durability of concrete made of poor performing aggregate. As such it was decided to include in the experiment the following two variables:

1) Level of prestress:

Magnitude of prestress currently being used throughout the world varies between 90 to 1138 psi and 100 to 935 psi in case of highways and airfield pavements respectively in the longitudinal direction; in transverse direction it is much less than that in longitudinal direction (7). The maximum level of prestress used in the United States is of the order of 700 psi. Accordingly, it was decided to adopt the level of prestress for this study as 600 psi, being a reasonable average. The specimens were initially prestressed to this level before the commencement of the test and the force was allowed to remain on them permanently throughout the duration of the test. The behavior of these prestressed specimens was then compared with that of companion specimens having zero prestress.

2) Proportion of bad performing aggregate in the concrete:

Altogether two types of concrete mixes were designed.

In mix design A, midwestern chert river gravel with a known reputation of bad performing aggregate was used as the coarse aggregate. In mix design B, the proportion of chert was reduced to fifty per cent while crushed limestone with a known good performance was used as the remaining fifty per cent of coarse aggregate.

In order to fully appreciate the effect of variables introduced in an experiment of this nature, it is essential that it be conducted within the shortest possible period. The variability, or the error, is much greater for mixtures made over an extended period than for those produced over a short interval, and could be so great as to obscure the effects of the variables which are the subject of the research. The entire mixing program of this study was completed within two weeks.

Constants. The factors which were held as constant as possible throughout the investigations may be itemized as follows:

1) Dimensions of specimens
2) Mixture design
3) Method and period of curing
4) Type of cement

Number of specimens. Three batches from each concrete mix design were made. Each batch provided enough concrete

for three prestressed beams along with three non-prestressed companion beams. In all, nine specimens for each condition which was to be studied were made. The results of freezing and thawing tests on a total of thirty-six beams are thus reported in this study.

Materials

Cement. A blend of three brands of low alkali type I cement from the same shipment was used for all mixes to avoid differences in results due to any change in the quality of cement.

Fine Aggregate. A single high quality quartzite sand was used throughout the investigation. Its source and physical properties are given in Table 1.

Coarse Aggregate. Two different coarse aggregates in the proportions shown in Table 3 were used. The source and physical properties of these aggregates are shown in Table 2.

Preparation of Specimens

Concrete Mix Design. The mixes were batched on a weight basis. The so-called b/bo method, as described by Goldbeck and Gray (8), was used for designing the mixes. This method of mix design is based on the laboratory established values of the volume of coarse aggregate required per unit volume of concrete for a properly workable concrete, and also on the cement and total water required per unit volume of

Table 1. Physical Properties of Fine Aggregate

Specific Gravity (Bulk Dry)	2.590
Specific Gravity (Bulk S. S. D.)	2.600
Absorption, per cent	0.341
Fineness Modulus	2.630

Table 2. Physical Properties of Coarse Aggregates

Aggregate	Chert	Crushed Limestone
Source	Missouri	Virginia
Specific Gravity (Bulk Dry)	2.420	2.774
Specific Gravity (Bulk S.S.D.)	2.510	2.781
Absorption, per cent	3.625	0.378
Unit weight, dry, rodded, pcf	86.7	

Table 3. Summary of Concretes Studied

Mix Design Designation	Percent of Coarse Aggregate			
	Chert		Crushed Limestone	
	3/4" - 1/2"	1/2"-1/4"	3/4" - 1/2"	1/2"-1/4"
A	50	50	-	-
B	25	25	25	25

concrete for the desired strength, slump, size and type of coarse aggregate. In this method of mix design "b" is defined as the solid volume of coarse aggregate per unit volume of concrete, b_o as the solid volume of coarse aggregate per unit volume of coarse aggregate, and b/b_o as the dry-rodded volume of coarse aggregate per unit volume of concrete.

Each mix was designed for 0.6 cubic foot of concrete which was slightly more than what was required to mold six beams of 3" x 3" x 14" size from the same batch.

Preparation of Aggregates. Air-dried coarse aggregates were kept under one and one-half centimeters of vacuum for about an hour and then saturated under water at atmospheric pressure for 24 hours prior to mixing. The fine aggregate was thoroughly mixed with slightly more water than needed for complete absorption and then left under a waterproof plastic sheet for 24 hours prior to mixing. The same method of saturation was followed while carrying out tests to determine the specific gravity and absorption of the aggregates.

Molding of Specimens. The mixing of the concrete was done in a 1.5 cubic foot Lancaster tub mixer. The mixing drum of this mixer rotates at 11 rpm in a clockwise direction while two blades, one near the rim of the drum and the other near the pivot, rotate at 40 rpm in a counterclockwise direction. This arrangement facilitates a thorough mixing of the

concrete. The coarse and the fine aggregates were placed in the drum and dry-mixed for one minute. Thereafter, cement was added and mixer was allowed to run for another minute before adding the water. First, only half the quantity of water required as per the mix design and containing the required quantity of air-entraining agent (neutralized vinsol resin solution) was added. The mixer was then run for three more minutes and more water was added until a slump of about three inches was achieved. An attempt to maintain the entrained air content at the desired level of 5.5 per cent was met with limited success. A minimum of three air content measurements were made with a Chase air meter. Properties of each mix batch are shown in Table 4.

The concrete was then placed into the forms. The specimens were prepared essentially according to ASTM designation C192-59. For specimens which were to be post-tensioned, a 5/8" diameter steel dummy bar wrapped with waxed paper was fixed in position beforehand at the center of the mold lengthwise. This dummy bar when taken out after 24 hours of molding of the specimens at the time of stripping of forms provided a duct of sufficient diameter for accomodating the tensioning 1/2" diameter steel bar. In most cases a thermocouple was placed in the center of the specimen. Brass inserts, serving as carriers for stainless steel strain plugs, were placed in the open side of the concrete specimen at ten inch centers.

Table 4. Mixing Data

Mix Number	W/C Ratio (by weight)	Cement Factor (bags/cu.yd.)	% Air
1A	0.626	5.27	4.0
2A	0.610	5.26	5.0
3A	0.670	5.16	5.0
1B	0.598	5.56	3.0
2B	0.598	5.55	3.25
3B	0.608	5.53	3.3

Curing of Specimens. After curing for one day, wrapped in a moisture proof plastic, the beams were removed from the forms and carefully identified with the notations selected for the study. Soon after the specimens were cured in a moist room in lime-saturated water having a temperature of approximately 70°F for a period ranging from 13 to 18 days.

Prestressing of Specimens

Method of Prestressing. Due to the limitations imposed upon the dimensions of the specimens by the size of the freezing and thawing apparatus, it was decided to adopt the post-tensioning method for prestressing the specimens. The other method - pre-tensioning - would have required longer specimens, in order to develop the desired value of prestress through bond in the concrete, than those which could be conveniently accomodated in the freezing and thawing apparatus at our disposal. The beams were prestressed in a manner such that compressive stress distribution within the specimen would be symmetrical with respect to horizontal and vertical axes and uniform throughout the longitudinal axis of the beam.

Equipment. The tensioning bars used were 1/2" diameter Stress-Steel high strength bars having a yield strength of 135,000 psi, ultimate tensile strength of 170,000 psi and a modulus of elasticity of 33,000,000 psi. The bars were 18

inches in length and had 2-1/2" long threads at each end. In order to avoid the damage to the threads of the bars, a small piece of another bar was attached at each end with the help of threaded couplers while tensioning the bar in a Universal testing machine. Steel end plates of the dimensions 2-7/8" x 2-7/8" x 1/2" thick, with 17/32" inch diameter holes drilled in their centers, along with suitable 1/2" diameter nuts were used at each end of the post-tensioned beams.

For accurately determining the magnitude of the prestress in the concrete, a BLH SR-4 Strain Gage (Type A-8) was fixed on each bar. After the surface of the bar was thoroughly cleaned of foreign matter with the help of sand paper and SR-4 Precoat Solvent, the SR-4 Cement Solvent was applied in a thin layer on the cleaned surface. This was followed by gently putting the gauge on the bar and wrapping a rubber band around it in such a manner that a small pressure would be exerted on the gauge during the period of curing of cement, thus ensuring adequate bond between the gauge and the underlying surface of the bar. After about eight hours of drying the bar at low temperature (70°F), the rubber band was removed and the gauge was checked with an ohm-meter for the minimum resistance required for satisfactory performance of the gauge, which was 120 micro-ohms. The lead wires were then soldered to the wires coming out

from the gauge. The strain gauge was then further dried at low temperature for expelling any moisture which might be present and thereafter covered with a layer of Cerese Wax, which was furnished with the SR-4 Kit, for waterproofing the gauge. This was done in view of the fact that the beams were to be prestressed 24 hours before they became due for undergoing the freezing and thawing tests, and in the meantime they would have to be replaced in the curing room. This procedure was adopted for allowing for losses such as those due to elastic shortening and anchorage to take place before the beam was transferred to the freezing and thawing apparatus. Accordingly, at the time of prestressing, a provision was made for these losses so that when the specimen was put in the apparatus for undergoing freezing and thawing tests, a net level of stress equal to 600 psi would be available within the beam. This initial prestress was never taken off the beam throughout the duration of the test.

Procedure of Prestressing. The specimens were post-tensioned when they were from 13 to 18 days old. The prestress was allowed to remain on the specimen permanently. The stress steel bars were placed, through the holes in the end plates, in the duct at the center of the beam such that about a two-inch long portion of the bar remained outside the beam at each end. Grooves were made in the end-plates for taking out lead wires from the strain gauge fixed on

the tensioning bar. The end plates were then placed in position so that they were butting the two ends of the beam. Thereafter, one-half inch diameter nuts were tightened on the two exposed ends of the tensioning bar such that they were loosely touching the two end plates. The lead wires from the SR-4 strain gauge were then plugged into a SR-4 strain indicator. Another SR-4 gauge fixed on a dummy steel bar was also connected to the strain indicator, thus acting as a compensating gauge so as to offset any effect due to temperature differential on the measuring gauge readings. Initial readings were then recorded from the strain indicator. Calculations in respect to load on the beam and stress and strain on the tensioning bar for the required level of prestress are shown in Appendix I.

In earlier stages of the experiment, a 120,000 pounds Universal Testing Machine was used for supplying the necessary force to the tensioning bar. After the gauge dial of the machine was showing slightly more load than needed on the bar, the nut on one end of the bar was tightened until such time that the dial on the SR-4 strain indicator gave the desired computed final reading, this final reading being the sum total of initial SR-4 indicator reading and the computed strain, for the given amount of stress, on the tensioning bar. Upon getting the desired final reading on the strain

indicator, the machine was released of load and the final reading was rechecked on the indicator. Discrepancy, if any, was made good by reloading the machine and adjusting the nut on the bar.

Later on, as the experiment progressed, it was found that the required amount of load could be quite conveniently put on the bar with the help of a pair of wrenches which tightened the nuts by applying manual pressure. The same procedure, recorded earlier, was adopted for putting on the desired value of load on beam. Thereafter, the specimen was re-placed in the curing room for eventually taking out the next day and transferring it to the freezing and thawing chamber. During the period when the specimens were being prestressed, their companion non-prestressed specimens were also taken out and were subjected to approximately the same sort of physical environments and conditions as those being undergone by the prestressed specimens. In this way, it was ensured that all specimens, both prestressed and non-prestressed, underwent identical conditions as far as possible, so that true evaluation of the results furnished by the two groups of specimens could be made.

Testing of Specimens

Freezing and Thawing Apparatus. All specimens after curing were placed in the freezing and thawing apparatus

which was similar to that developed by the Engineering Experiment Station of the Utah State University and was manufactured by the Logan Refrigeration Company of Logan, Utah. This equipment exposed the specimens to approximately seven cycles of rapid freezing and thawing in water in accordance with ASTM designation C290-61T. The cabinet is six feet, ten inches long by two feet ten inches wide by ten inches deep with four inches insulation on all sides. A six foot commercial cooling plate upon which rested copper containers holding the concrete specimens, attached to a one-half horsepower compressor, lowers the temperature to 0°F during the freezing phase of the freezing and thawing cycle. Thawing is accomplished with electric resistance strip heaters placed in direct contact with the containers which heat the specimen to 40°F. The copper containers were one-quarter inch larger than the actual dimensions of the specimens, thus leaving one eighth inch layer of water on all four sides and top and bottom of the specimen. When the temperature at the center of the control beam placed in the center of the apparatus reached 40°F, the heater was turned off and compressor started automatically. This process was reversed when the temperature dropped down to 0°F. A recording thermometer kept a continuous record of the temperature at the center of the control beam. The average time for a complete cycle of freezing and thawing was about 3.5 hours.

Testing Procedure. Just prior to placing a specimen into freezing and thawing apparatus, it was carefully weighed, tested for the fundamental frequency (ASTM C215-60) that corresponded to the zero cycle of freezing and thawing and 100 per cent relative modulus of elasticity (this is dealt with in detail under the next sub-title of this study) and measured for length using a Whittemore Strain Gage. If the specimen contained a thermocouple, it was connected to a continuous six-channel temperature recorder and the initial temperature was recorded. Length and temperature changes were made at approximately ten minute intervals throughout the freezing phase of the first cycle of freezing and thawing. As the specimens were lying in a horizontal position in copper containers placed on the cooling plate, no difficulty was encountered while taking strain gage readings throughout the duration of the test, because one face of the specimen (although covered with water) was exposed at all times. Protective collars (in the form of ordinary bottle caps) were placed around the strain plugs for facilitating taking strain measurements quickly at all times. When a specimen did not have a thermocouple, temperature was taken from the control beam. The specimen was subjected to freezing and thawing cycles until it had lost 50 per cent of its original dynamic modulus of elasticity or had experienced

100 cycles of freezing and thawing. At regular intervals during this period, the specimens were measured for length, weight and fundamental transverse frequency.

Measurement of Specimen Deterioration. The most commonly used method for measuring the effect of freezing and thawing on concrete is by determining a Durability Factor. The ASTM method for calculating the durability factor is given by the following formula:

$$DF = \frac{PN}{M} \quad . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; (1)$$

where P is the dynamic modulus of elasticity expressed as a percentage of its original value or relative dynamic modulus of elasticity;

N is the number of cycles for 50 per cent reduction in dynamic modulus of elasticity, or at which the test is to be terminated;

and M is the specified number of cycles at which the test is to be terminated.

A relationship exists between the fundamental transverse frequency of a concrete specimen and its modulus of elasticity. As described in ASTM specification C215-58, this relationship can be expressed as:

$$E = CWn^2 \quad . \; . \; . \; . \; . \; . \; . \; . \; . \; (2)$$

where E is Young's modulus of elasticity, in pounds per

square inch;

W is weight of specimen, in pounds;

n is fundamental transverse frequency, in cycles per second; and

C refers to a constant for any given specimen.

For getting an absolute value of the modulus of elasticity, the above equation can be used, but for comparing the durability of various specimens, it is only necessary to obtain relative values. The ASTM specifications allow the assumption that C and W remain constant, for a given specimen, throughout the durability test, although disintegration of the specimens probably tends to change these values. Considering the initial fundamental transverse frequency of a specimen to represent Young's modulus at 100 percent relative value, the subsequent frequency values can be related by the following formula:

$$P_c = \frac{n_c^2}{n_o^2} \times 100 \quad \ldots\ldots (3)$$

where P_c is the relative modulus of elasticity, in percent, after C cycles of freezing and thawing;

n_c is fundamental transverse frequency after C cycles of freezing and thawing, in cycles per second; and

n_o is fundamental transverse frequency at zero cycles of freezing and thawing, in cycles per second.

The relative modulus of elasticity was computed for each specimen following a measurement of natural frequency. The durability was then computed using equation number 1, mentioned earlier.

The natural frequency measuring unit in this experiment consisted of a variable frequency oscillator, a variable frequency driving unit to induce controlled frequency vibrations in the specimen, a crystal pick-up and a voltmeter to measure the magnitude of the voltage produced by the current flowing from the pickup. When the induced vibration is the same as the natural frequency of the specimen, resonance occurs and is indicated by a sudden increase in the voltmeter reading. Thus the maximum voltage occurs at the fundamental frequency of vibration of the specimen. The frequency is read directly from the oscillator dial.

IV. RESULTS AND DISCUSSION

Definition of Terms

Before presenting the results, it will be appropriate that several terms which are used herein be defined. DF_{100} is the one hundred cycle durability factor as calculated by the procedure described in ASTM designation C290-61T. (It has been dealt with in detail in the preceding chapter of this study.)

Another term which needs definition is the minimum 5° temperature slope, b_1. This is the minimum slope that can be found, within a $5^{o}F$ or more range, on the length change vs. temperature curve obtained during the first freeze of each specimen (9). This slope occurred generally when the temperature at the center of the specimens was somewhere between $36^{o}F$ and $17^{o}F$. This minimum slope results when the formation of ice and resulting hydraulic pressure cause expansion that either in part or completely counteracts the natural contraction due to cooling. b_1 is in units of 10^{-4} inches per ^{o}F and ranged from +0.571 where little or no expansion occurred to -1.600 where there was much more expansion than contraction.

A third term used is ΔL which is total change in the length of a specimen from the beginning to the end of the test, units being 10^{-4} inches.

Presentation of Results

Figure 1 shows the results of the freezing and thawing tests for all specimens. Relative dynamic modulus of elasticity has been plotted against the cycles of freezing and thawing. The symbol N stands for non-prestressed specimens while P signifies specimens which were pre-stressed, and the following alphabet A or B designates the mix design. Each curve represents the average values of nine specimens made from three different batches of the same mix design. The durability factors, DF_{100}, are tabulated for each specimen in Table 5, along with the average values for each batch and mix. Table 6 shows the values of minimum 5° slope, b_1, for each individual specimen as also the averages for each batch and mix. Table 7 and Figure 2 show the relationship between DF_{100} and b_1 for all specimens; the values shown are averages of three specimens made from each batch. In Table 8 are tabulated the values of final length change, ΔL, undergone by the specimens, along with respective relative modulus of elasticity in percent.

Discussion of Results

Mix Design A. The effect of prestressing on the durability of concrete which was wholly made from an aggregate of known bad performance, is very glaring. It will be seen from Figure 1 that while the prestressed specimens showed a

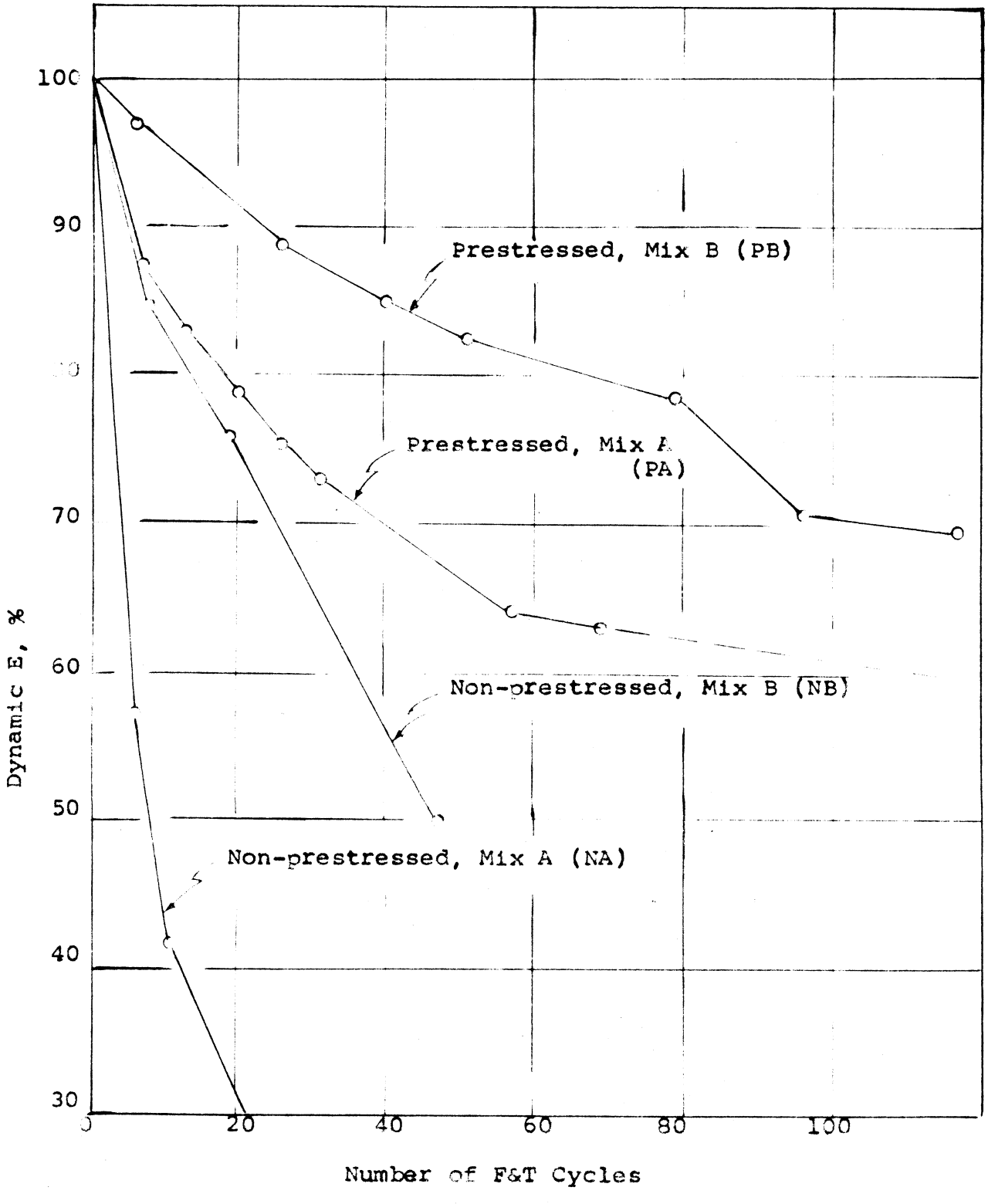

Figure 1. Dynamic E, %, vs. Cycles of F&T.

Table 5. Durability Factors at 100 Cycles

Mix Designation	Beam No.	Non-Prestressed (N)	Avg.	Beam No.	Prestressed (P)	Avg.
1A	1	3.0		1	37.7	
	2	2.6	3.4	2	41.7	41.5
	3	4.7		3	45.2	
2A	1	3.8		1	32.7	
	2	4.6	4.0	2	35.5	34.1
	3	3.5		3	34.1	
3A	1	6.7		1	39.6	
	2	6.3	6.5	2	45.8	43.9
	3	6.6		3	46.2	
Average			4.6			39.8
1B	1	25.0		1	74.4	
	2	26.5	23.5	2	75.5	75.0
	3	19.1		3	75.0	
2B	1	19.7		1	66.3	
	2	16.6	16.6	2	67.3	71.2
	3	13.4		3	80.0	
3B	1	16.6		1	74.4	
	2	16.9	16.6	2	70.5	70.6
	3	16.2		3	66.8	
Average			18.9			72.3

Table 6. Minimum 5° Temperature Slope, b_1, 10^{-4} in./°F

Mix Designation	Beam No.	Non-Prestressed (N)	Avg.	Beam No.	Prestressed (P)	Avg.
1A	1	-0.875		1	0.000	
	2	-0.833	-0.865	2	+0.200	+0.233
	3	-0.888		3	+0.500	
2A	1	-1.600		1	-0.714	
	2	-0.667	-0.956	2	-0.286	-0.500
	3	-0.600		3	-0.500	
3A	1	-0.250		1	+0.571	
	2	-0.750	-0.583	2	+0.500	+0.500
	3	-0.750		3	+0.430	
Average			-0.801			+0.078
1B	1	-0.250		1	+0.500	
	2	0.000	-0.483	2	+0.400	+0.367
	3	-1.200		3	+0.200	
2B	1	0.000		1	0.000	
	2	0.000	-0.250	2	-0.166	+0.078
	3	-0.750		3	+0.400	
3B	1	0.000		1	+0.166	
	2	+0.143	+0.103	2	-0.400	+0.055
	3	+0.166		3	+0.400	
Average			-0.210			+0.167

Table 7. Relationship Between Minimum 5° Temperature Slope, b_1 (10^{-4} in./°F) and 100 Cycle Durability Factor, All Mix Designs

Mix Designation	DF_{100}	b_1	Mix Designation	DF_{100}	b_1
1AN	3.4	-0.865	1AP	41.5	+0.233
2AN	4.0	-0.956	2AP	34.1	-0.500
3AN	6.5	-0.583	3AP	43.9	+0.500
Average	4.6	-0.801	Average	39.8	+0.078
1BN	23.5	-0.483	1BP	75.0	+0.367
2BN	16.6	-0.250	2BP	71.2	+0.078
3BN	16.6	+0.103	3BP	70.6	+0.055
Average	18.9	-0.210	Average	72.3	+0.167

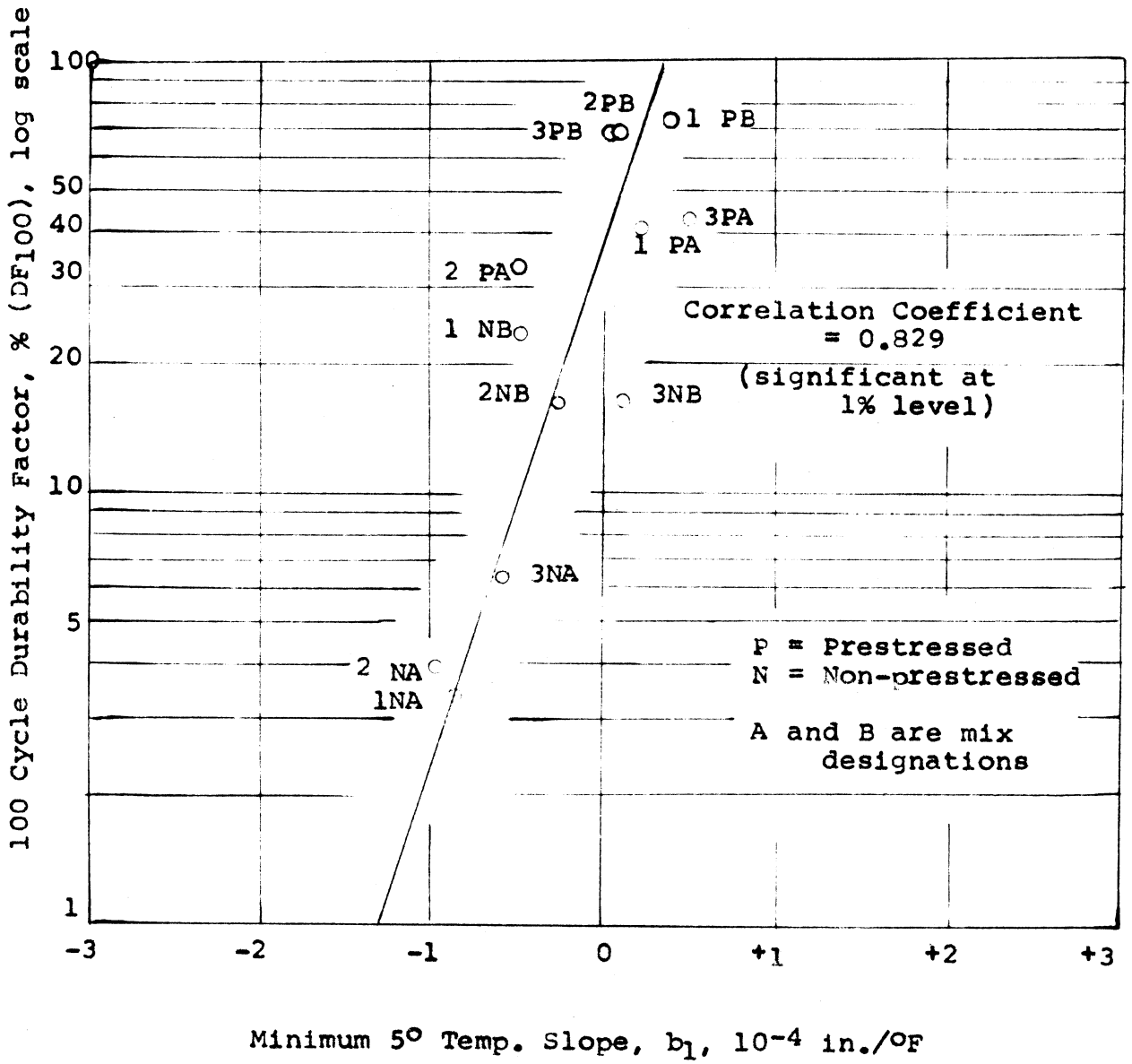

Figure 2. Relationship Between Durability Factor, DF_{100} and Minimum 5° Temperature Slope, for All Mixes.

Table 8. Relationship Between Relative Modulus of Elasticity, %E, and Final Length Change, ΔL, (in in. $\times 10^{-4}$).

Mix Designation	Beam no.	Non-prestressed (N) ΔL	Avg.	%E	Avg.	Prestressed (P) ΔL	Avg.	%E	Avg.
1A	1	+62		49.9		-14		53.8	
	2	+87	+76	42.6	57.1	-57	-21	59.5	59.2
	3	+80		78.8		+8		64.2	
2A	1	+68		62.6		+28		54.5	
	2	+59	+62	52.0	57.4	-18	+5	59.2	56.8
	3	+59		57.7		-		-	
3A	1	+114		23.0		-23		69.4	
	2	+108	+109	21.8	22.5	-61	-29	66.4	67.6
	3	+106		22.8		-3		67.0	
Average			+82		45.7		-15		61.2
1B	1	+39		53.2		-16		87.6	
	2	+50	+57	56.5	50.1	-5	-9	88.8	88.2
	3	+83		40.7		-7		88.3	
2B	1	+96		49.2		+2		62.6	
	2	-	+55	-	69.3	-28	-7	63.5	67.2
	3	+14		89.5		+4		75.4	
3B	1	+3		87.0		-17		76.7	
	2	+29	+28	65.2	71.6	-23	-24	72.0	72.5
	3	+53		62.5		-31		68.7	
Average			+47		63.7		-13		76.0

relative modulus of elasticity of 87.5 per cent at the end of 13 cycles of freezing and thawing, that of non-prestressed specimens had dropped down to 41.7 per cent by the end of 11 cycles. In fact, the deterioration in non-prestressed concrete was so fast that all beams showed a relative modulus of elasticity of less than 50 per cent at the end of six cycles. The average value of DF_{100} for all specimens made with this mix design is 4.6 in the case of non-prestressed specimens while in the case of prestressed specimens the figure is as high as 39.8, an improvement of about 800 per cent over the former value of DF_{100}. The respective values of b_1 are -0.801 and +0.078. While the non-prestressed specimens, by the time their relative modulus of elasticity had fallen to 50 per cent or less invariably showed transverse cracks on their exposed surfaces, which in some cases extended down to the sides for some distance, the prestressed beams at the same level of relative modulus of elasticity developed, in some cases only, hair-line cracks which were approximately aligned in the longitudinal axis of the beams. Disintegration of aggregate particles exposed to surface was noticeable as also a high degree of surface scaling on the non-prestressed specimens - the latter could be attributed to a high water-cement ratio in the concrete mixes.

Mix Design B. The same trend that was observed in specimens made from Mix Design A was also followed in this

case. As mentioned earlier, these specimens were made using poor performing aggregate as only 50 per cent of coarse aggregate, while the remaining portion was comprised ofcrushed limestone of known good performance. From Figure 1 it will be seen that while the average value of relative modulus of elasticity had dropped down to 50.1 per cent at the end of 47 cycles of freezing and thawing in the case of non-prestressed beams, it was 82.5 for prestressed specimens by the end of 51 cycles. The average value of DF_{100} in case of non-prestressed specimens made from this mix design is 18.9, while it has risen to 72.3 for specimens which were prestressed, which is approximately four times that of the former figure. The average values of b_1 are respectively -0.210 and +0.167 for non-prestressed and prestressed specimens. The surface scaling of non-prestressed specimens made with this mix design was not very severe. However, the same cracking pattern as reported in specimens made from Mix Design A was also observed here.

Comparison of Results of Mix Designs A and B. It will be observed from Figure 1 and Table 5 that by prestressing, a concrete made wholly of a poor performing aggregate can be made to perform better than the non-prestressed concrete in which the proportion of poor performing aggregate had been reduced to 50 per cent of coarse aggregate, when

subjected to laboratory freezing and thawing cycles. The average value of DF_{100} for prestressed specimens made from Mix Design A is 39.8 while that of non-prestressed specimens made from Mix Design B (in which only 50 per cent poor performing aggregate was used) is 18.9. It will be further noted that while substitution of 50 per cent of poor performing aggregate with the good performing aggregate resulted in the increase of DF_{100} from 4.6 to 18.9 for non-prestressed concrete, i.e. an improvement of more than 300 per cent on the first value, the same proportion was not maintained in case of prestressed specimens where the two values of DF_{100} are respectively 39.8 and 72.3.

From Table 4 it will be noticed that in all batches of Mix Design B percentage of entrained air was less than that of Mix Design A, but probably this was compensated for by the higher values of water-cement ratio and lower values of cement-factor obtained in the later mix.

Figure 2 shows a semi-log plot of durability factor, DF_{100}, vs. minimum 5° temperature slope, b_1, based on average values of all batches made from the two mix designs. A semi-log plot was used to straighten an otherwise curved line due to very high b_1 values for very low durability factors. The correlation coefficient as calculated was 0.829 which is more than what is considered minimum in order

to make the relationship significant at the one per cent level. (10, 11)

It is interesting to note the relationship between DF_{100} and b_1 as tabulated in Table 7. As the value of DF_{100} approaches 39.8, the value of b_1 rises from a higher negative value to a lower negative value and finally to a positive value which is further enhanced for the respective higher value of DF_{100}. This phenomenon tends to confirm the suggestion of Walker (9) that those aggregates having durability factors of 30 or less have a value of b_1 which is less than zero.

There is also a marked improvement in the expansive characteristics of the concrete when it is prestressed as will be seen from Table 8. Upon comparing the results of the specimens made from the two mix designs, it will be seen that while substitution of 50 per cent poor performing aggregate with good performing aggregate resulted in considerable reduction in the magnitude of expansion undergone by the specimens during the tests in the case of non-prestressed concrete, the effect was not so pronounced on the magnitude of contraction undergone by the prestressed specimens. The non-prestressed specimens in all instances showed a total length increase while the beams which were prestressed, in most cases, displayed a shortening.

These results, therefore, indicate that the durability of the concrete made of poor performing aggregate can be improved to a considerable extent by prestressing. However, the magnitude of this improvement tends to diminish with the increasing proportion of good performing aggregate in the concrete.

V. CONCLUSIONS

From the results of the freezing and thawing tests the following conclusions may be drawn:

1. The post-tensioning of concrete made of poor performing aggregate improves its effective durability against cyclic freezing and thawing by a factor of 2 to 4.

2. The magnitude of improvement in the durability of concrete which can be secured by prestressing tends to diminish with the increasing proportion of aggregates having a high resistance to freezing and thawing in the concrete.

3. Continuously post-tensioned concrete made wholly of poor performing aggregate is effectively more durable than unstressed concrete in which 50 per cent of good performing aggregate (and the balance poor performing aggregate) has been used.

4. A definite relationship exists between durability factor (DF_{100}) and the minimum temperature slope (b_1).

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APPENDIX NO. I

Calculations for inducing prestress in the specimens:

Net cross-sectional area of the concrete

$$= (3 \times 3) - \frac{\pi}{4}\left(\frac{5}{8}\right)^2$$

$$= 9 - 0.317$$

$$= 8.69 \text{ sq. in.}$$

Initial prestress $= 600$ psi

Add 10% for losses $= \underline{60}$

$= 660$ psi

Total force to be applied on the beam

$$F = 660 \times 8.69$$

$$= 5735 \text{ lbs.}$$

Average dia. of the tensioning bar $= 0.514$ in.

Area of this bar, A $= \frac{\pi}{4}(0.514)^2$

$= 0.2075$ sq. in.

Stress in bar induced due to the prestressing force, p $= \frac{F}{A} = \frac{5735}{0.2075}$

$= 27,600$ psi

Modulus of elasticity of this high strength steel bar, E $= 33 \times 10^6$ psi

Strain in the bar, e $= \frac{p}{E}$

$$= \frac{27,600}{33 \times 10^6}$$

$$= 837 \times 10^{-6} \text{ in./in.}$$

$$= 837 \text{ micro inches}$$

Let e_1 = Initial reading on SR-4 strain indicator,

e_2 = Final reading on SR-4 strain indicator,

Then

$$e_2 = e_1 + 837$$

or Residual strain on the bar

$$e = e_2 - e_1$$

ABSTRACT

EFFECT OF PRESTRESSING ON THE DURABILITY OF PORTLAND CEMENT CONCRETE

In view of the fact that prestressed concrete is extensively used in bridge construction and that it holds potentialities for eventual use in pavement construction, and that methods must be investigated to utilize an otherwise rejected aggregate which cannot be used for its poor performance under natural weathering, it has become of paramount importance that further efforts be made to evaluate the effect of prestressing on the durability of concrete.

The purpose of this thesis was twofold: 1) To study the durability of prestressed concrete made of poor-performing aggregate; 2) To compare the freezing and thawing effects on prestressed concrete with those on ordinary concrete.

Two mix designs having different proportions of poor performing aggregate were used in this study. Half the number of specimens were post-tensioned after they had been cured for a period of 13 to 18 days in water, and were then re-placed in the curing room for 24 hours. Level of prestress was 600 psi. Before transferring the specimens - both prestressed and non-prestressed - into freezing and thawing apparatus, they were tested for fundamental transverse frequency and initial weight and length measurements were recorded. Thereafter, transverse frequency, weight, length

change and temperature change measurements were made periodically. The relative dynamic modulus of elasticity and durability factor were then calculated for each specimen.

On the basis of the results furnished by these tests, it may be concluded that prestressing improves the durability of concrete made of poor performing aggregate and that the magnitude of improvement in the durability of concrete tends to diminish with increasing proportion of good performing aggregate.