ACCELERATED FREEZE-THAW TESTS OF

EXPANSIVE CONCRETE

By

PAUL WILLIAM ERDNER

Bachelor of Science

Oklahoma State University

Stillwater, Oklahoma

1968

Submitted to the Faculty of the Graduate College
of the Oklahoma State University
in partial fulfillment of the requirements
for the Degree of
MASTER OF SCIENCE
May, 1969

ACCELERATED FREEZE-THAW TESTS OF

EXPANSIVE CONCRETE

Thesis Approved:

Robert L. Janes
Thesis Adviser

James V. Parcher

Phillip G. Manke

N. D. Durham
Dean of the Graduate College

ACKNOWLEDGMENTS

The writer wishes to express his indebtedness and sincere appreciation to the following individuals and organization:

To Professor R. L. Janes, for his friendship and guidance as advisor during the writing of this thesis and as Chairman of the Advisory Committee;

To Professor J. V. Parcher and Professor P. G. Manke as members of the Advisory Committee;

To the Oklahoma State University Civil Engineering Department for financial aid in the form of a research assistantship;

To Mr. Eldon Hardy for his assistance in preparing the figures for this thesis;

To Mr. Cecil Sharp for his assistance and suggestions in helping prepare some of the laboratory testing equipment.

To Samuel Ng and Pat Erdner for their help in taking pictures;

To Mrs. Nancy Wolfe for her typing excellence.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

Figure Page

CHAPTER I

INTRODUCTION

As a result of the findings made by Klein and Troxell (1), two basic formulations of expansive cement have been developed in the United States. One is proportioned to expand and compensate for the shrinkage which occurs in conventional portland cement as it hardens. The product of this formulation is called "shrinkage compensated cement." The other is proportioned to create greater expansion in the concrete when it hardens which will pre-stress the high-tensile steel embedded in the concrete. This is called "self-stressing cement." The former eliminates shrinkage cracks in concrete, while the latter produces concrete that prestresses itself.

Although concretes with expansive cements have been studied for the past 30 years very little data exists on their freeze-thaw resistance. Since proposed uses of expansive concretes includes pavements, parking lots, and other severe exposures, knowledge of their durability under severe conditions is needed.

The present study was undertaken to evaluate the freeze-thaw resistance of concretes made with "ChemComp cement." As defined by its manufacturer, Chemically Prestressed Concrete Corporation, "ChemComp cement" is a compensated shrinkage type of hydraulic cement which is produced by pulverizing clinker(s) consisting essentially of hydraulic silicates and of anhydrous calcium aluminosulfates (2).

The behavior of the unrestrained expansive concrete when exposed to severe weather conditions was investigated by molding beams of different water-cement ratios and air contents, and then subjecting the specimens to accelerated freeze-thaw tests. Throughout the testing of the beams, changes in weight, length, and the dynamic modulus of elasticity were measured as a means of determining the durability of the specimens.

CHAPTER II

REVIEW OF LITERATURE

Expansive cements and concretes, although seemingly new engineering materials, have had a long history of development. During the past 79 years, there have been recurrent efforts to develop expansive cements.

The principal accomplishments in the development of expansive cement have been made in France, the Soviet Union, and in the United States.

Early in the development of expansive cement, the Lafarge Company found that the expansive reaction could be obtained by using a mixture of gypsum or plaster, aluminous cement, and portland cement, but the reaction was both erratic and difficult to control (3).

During the 1940's major developments in expansive cement and concrete were made in France principally by Lossier (4). His expansive cement consisted of a mixture of portland cement and an expansive ground clinker claimed to be calcium sulfoaluminate. His method of expansion control consisted in regulating the availability of water to the concrete and insuring termination of reactions by introducing blast furnace slag to the mixture. In 1952, Lafuma reported that the above clinker consisted essentially of dicalcium silicate, calcium sulfate, and various calcium aluminates and ferrites.

However, little has been made of this expansive cement, as difficulty is experienced in controlling the expansion. Also, the magnitude of self-stressing was found generally insufficient for prestressed concrete components of economical design.

In using the French expansive cement for making expansive concrete, conventional methods of mixing and placing were employed, but to terminate the expansion, besides introducing the slag terminator to the mixture, special methods of curing had to be used.

The expansive cements being used in the Soviet Union may be termed as "stressing cements." They consist of interground mixtures of portland cement, calcium aluminate cement, and either gypsum or plaster of Paris.

More recently considerable research with self-stressing cement has been undertaken by Mikhailov (5) as reported in 1960. Hydrothermal methods of curing under close control are used to achieve the desired expansive self-stressing reaction. Water-cement ratios as low as 0.16 are reported to be used in the USSR in pneumatically applied expansive mortars.

The earliest major investigation in the field of expansive cement and concrete in the United States was the work of Klein and Troxell (1) at the University of California at Berkeley. In this research, calcium aluminosulfate clinkers and blast furnace slag were introduced as admixtures to the concrete mix to produce expansion in plain concretes. Though both unrestrained and restrained expansions achieved were unsatisfactory for self-stressing of steel and concrete, the important contribution was the definite proof of a stable compound in the system $CaO.Al_2O_3.SO_3$, by determining the x-ray diffraction pattern and those of the several hydrates of the particular calcium aluminosulfate anhydrite. Important data was provided on both the theoretical and applied chemistry of the expansive reaction enabling subsequent development of expansive cements capable of producing expansive concretes for application to prestressing structural elements.

Further studies of improved expansive cements and concretes and their performance were reported by Klein, Karby, and Polivka (3). The expansive component comprised from 20 to 30 percent of the expansive cement, and the expansive cement contents were 7 to 10 bags per cubic yard of concrete. The expansion was controlled by the composition of the expansive component and by the relative amounts of expansive component and portland cement.

As a result of the research that has been conducted at the University of California the amount of expansion that occurs in the concrete can be very closely controlled.

The University of California expansive cement consists of a blend of portland cement component and a calcium-sulfoaluminate anhydrite expansive component (6). The portland cement component is of high tricalcium silicate and low tricalcium aluminate content, and the expansive component is made by grinding a clinker of calcium aluminosulfate composition. The clinker is usually a true anhydrous calcium sulfoaluminate, the remainder being 15 to 20 percent free lime.

The expansive cement that was developed at Berkeley is now commercially produced by the Chemically Prestressed Concrete Corporation and several major United States cement companies.

The types of expansive concretes that are being produced in the United States have been classified as "self-stressing" or "shrinkage compensating," depending upon the amount and composition of the expansive component in the cement. As described in a report by the Portland Cement Association, self-stressing concretes are those that have expansive properties which, if restrained, develop compressive stresses in the concrete of such magnitude that they will be reduced only moderately by subsequent shrinkage (7). Shrinkage compensating concretes exhibit a lower potential for expansion under restraint; hence subsequent shrinkage results in a return to approximately the original volume, thus lessening the tendency for the development of tensile stresses leading to cracking when drying.

Investigations that have been made of the properties of the Klein expansive concrete have shown these characteristics of the material (8):

1. The higher the proportion of expansive component, the larger the ultimate expansion.

2. Although low expansive component shrinkage compensated concrete retains relatively the same strength (despite some slight expansion) as ordinary concrete, high expansive cement without self-stressed reinforcing reduces significantly the structural strength of a given slab.

3. On the other hand, the greater the restraining forces (among them self-stressed reinforcing bars), the greater the load strength of the slab.

4. Strength of self-stress, in high expansion component concrete, is adversely affected by a too high water content in the mix.

5. Total restrained expansion of the pour is reached within seven days. In fact, much of the mix's expansion takes place within the initial 24 hours. As with portland cement concrete, the strength

of the slab continues to increase, despite the cessation of expansion, during the normal cure cycle and beyond.

6. From tests performed by Klein and Bertero (9) it was found that the rate of expansion is related to the curing temperature, with the rate of expansion greater for high temperatures.

Further studies by Bertero (10) have been undertaken to compare the strength of self-stressing concrete with regular reinforced concrete. From his investigations he found that it is doubtful that use of uniaxial restraint alone would be practical for fabrication of chemically prestressed prismatic structural elements. This is due to the considerable drop in strength of the resulting expansive cement concrete after certain age, when it is compared with the strength which may be obtained by the use of ordinary reinforced concrete.

The main reason for the drop in strength as well as in stiffness after a certain age is the transverse expansion that occurs after certain age.

From the above observations he concludes, successful use of expansive concrete in the fabrication of self-stressed prismatic structural elements such as beams and columns, seems to require restraint of the expansion in all directions. It should be recognized that Bertero's conclusions can be assumed to apply only to the particular expansive cement concrete mix, type and size of specimen, and conditions of restraint and curing used in his investigation.

By being able to control the amount of expansion and by wisely taking advantage of the characteristics of the expansive concrete, the material has many potential uses in construction and may have several advantages over conventional portland cement concrete. Li (11) in an article from the Journal of the American Concrete Institute, published in 1965, has listed several potential uses of both formulations of the expansive concrete. He states that "self-stressing concrete" should find applications in precast concrete pressure pipes, precast architectural panels, highway pavement, airport runway pavement, sidewalks, and tunnel linings. The other formulation "shrinkage compensated concrete" should find application in thin shell concrete roofs, folded plate concrete roofs, thin concrete roof slabs, with the possibility of eliminating the need for additional waterproofing, in bridge decks with the promise of reducing or eliminating cracks and hence deterioration, and in the manufacture of non-shrinkage concrete wall block.

During the past several years very few projects have used expansive concrete for construction purposes. Most of the structures that have been built using expansive concretes have performed very well, while a few have not. It is believed that the reason for the poor performance of some of the structures was due to poor construction practices. It appears that the reason for the lack of use of the relatively new material is because of the initial cost of the material and also the performance of expansive concrete under various conditions is not known.

The initial cost of expansive cement is $1.40 to $2.00 per barrel higher than ordinary cement (12). However, if the expansive concrete will perform as expected, the additional initial cost can easily be justified by the amount that will be saved in maintenance costs.

To evaluate the performance of the expansive concrete, further investigations will have to be made to get a better understanding of the characteristics of the material. One of the more important characteristics of any construction material is its ability to withstand severe exposure conditions. For many years the ability of concrete to perform under severe exposure conditions has been determined by subjecting the material to alternate cycles of freezing and thawing.

The earliest evaluation of the mechanism of frost action in mortar and concrete appears to have been confined to the use in 1837 of sodium sulfate as a substitute for freezing water (13). Alternate soaking in the solution and washing with pure water resulted in specimens which were not long in showing deterioration.

Many investigators in a 20-year period following 1885 studied the effects of frost on mortars and concrete. Atmospheric freeze-thaw tests were used. In the early 1900's a controlled temperature room was first used to observe the action of frost on mortars made with fresh water and brine.

In the following 20 years some work was done on freeze-thaw testing methods, including studies of low-temperature testing versus the use of sodium sulfate. Field concreting methods were concurrently improving, for there seemed to be indications the carelessness and poor construction practices were more to blame for frost damage than were the cement or aggregates used. The idea that water-cement ratio had a significant bearing on the problem was beginning to evolve.

Later studies showed that there was less deterioration of the mortar with low water-cement ratios when exposed to frost effects. Through the 1940's, theories and investigations were concerned with the effects on durability and permeability of curing time, water-cement ratio, thermal coefficients, bleeding, and freezable water in concrete pores. Blends of cements and rates of freezing were studied.

Measurements of concrete deterioration by changes in the modulus of elasticity were first reported. The major contribution introduced during this period was the use of air entrainment and its acceptance as a factor affecting the durability of concrete.

As a result of the research that has been made concerning the basic mechanisms that cause concrete deterioration, it has been found that there are three principal phenomena that cause concrete to lose quality when exposed to freezing and thawing: 1) buildup of hydraulic pressure in the gel structure of the cement paste, 2) growth of capillary ice during sustained cold periods, and 3) deterioration caused by concrete aggregates (14).

The several freezing and thawing mechanisms which cause deterioration in concrete do not operate under the same climatic conditions. For example, rapid freezing and thawing is unusually severe in developing hydraulic pressure in the gel structure, but is too rapid to permit the buildup of ice crystals due to accretion. Also, laboratory specimens which are tested submerged in water are more likely to reach critical saturation than exposed structures in the field. It is difficult, therefore, to establish testing procedures which properly evaluate freezing and thawing resistance for all field conditions.

There are several test methods used to evaluate the freeze-thaw resistance of concrete. The type of freeze-thaw tests to be used usually depends upon the laboratory equipment available, the cost of conducting the tests, and/or the intensity of exposure that the specimens are to be subjected to.

While the freeze-thaw tests are being carried on, the amount of deterioration is determined by weighing the specimens, measuring their change in lengths, and/or determining the dynamic modulus of elasticity. The length and weight measurements are easily determined; however, the determination of the dynamic modulus of elasticity is slightly more complex.

The dynamic modulus of elasticity is usually determined by using sonic test methods. As stated by Whitehurst (15) the expression "sonic testing" is used to include two general areas of concrete testing. The areas included are: 1) that involving determination of the resonant frequency of a concrete specimen or some characteristic of the specimen when vibrating at or near resonance, and 2) that involving determination of the velocity with which a small mechanical impulse travels through the concrete. The area of sonic testing used in this report was that of determining the resonant frequency of the specimens. This method of testing will be described in further detail in Chapter 3.

Powers (16) published one of the first reports of testing the resonant frequency of concrete in 1938. He determined the resonant flexural vibration frequency of concrete specimens, usually 2 x 2 x 9½ inches, by supporting them at their nodal points, striking them with a hammer, and matching the musical tone produced with a suitably calibrated tone source. Two such tone sources were used in this study, a set of Deagen orchestra bells and a homemade sonometer. Powers felt that the maximum error likely to occur in matching the frequency of the beam to the calibrated source was about 3 percent in the case of the orchestra bells and much less when the sonometer was used.

There are several rather serious objections to this type of test (15). Although some concrete specimens ring resoundingly when struck, others emit a very dull sound, and an unusually good ear is required to detect and match their tone. Further, the natural frequency of vibration of a specimen may be lowered as much as 10 percent by being struck by a series of sharp blows.

In spite of these shortcomings, the basic concept of a relatively simple, nondestructive, quantitative test for concrete was highly appealing and quickly pursued by others engaged in studies of concrete durability.

As the work of others showed the definite merits of this type of test, the technique was taken under consideration by the American Society for Testing and Materials. The Society accepted the method in 1947. The method has been revised several times and in its present form was adopted in 1960 as ASTM C215-60, "Standard Method of Test for Fundamental Transverse, Longitudinal, and Torsional Frequencies of Concrete Specimens" (17).

There have been many investigations to determine the durability of concrete made with portland cement, but there have been very few tests to determine the durability of concrete made with expansive cement. The results of tests seem to differ from laboratory to laboratory even if the same method of testing is used in both test sites. The reason for this peculiar phenomena is discussed in a report by the Highway Research Board (18) in which it is believed that this difference in results is due to different amounts of loss in moisture during freezing and testing. The amount of moisture loss during freezing depends on the temperature of the cooling coils, the rate of circulation of air, position in the freezer, and other factors. The amount of moisture lost during testing depends on the amount of time the specimens are exposed to the atmosphere.

Because of variations that have been observed from laboratory to laboratory it seems that comparison of results from other freeze-thaw tests will have little meaning.

The only report that has been found on the results of freeze-thaw tests of unrestrained expansive concrete showed poor durability of the mixes (7). However, as the restraint was increased the durability of the specimens increased.

As a result of the numerous investigations and observed field performances of portland cement concrete, it is generally accepted that durable concrete may be produced by applying the following methods (19):

1. Produce a high-quality cement paste by limiting the amount of water in proportion to cement used in the concrete mix.

2. Use only high-quality aggregates that have satisfactory performance records in durable concrete.

3. Use the proper amount of air-entraining agent to produce an air bubble spacing that will protect the concrete against the development of hydraulic pressure and the growth of capillary ice.

4. Prevent rapid drying of pavement surfaces, and never give a final finish to a concrete slab until bleeding has stopped.

As already mentioned, the durability of expansive concrete may be increased by restraining the expansion.

CHAPTER III

EXPERIMENTAL METHODS AND MATERIALS

Materials

Cementing Material

The cementing material used in these tests was produced by Texas Industries Inc. The specifications of the TXI 4-C ChemComp cement were developed by the cooperative efforts of technical representatives of the Chemically Prestressed Concrete Corporation and five ChemComp producers. The chemical and physical requirements of the cements are shown in Tables I and II, respectively.

TABLE I

CHEMICAL REQUIREMENTS OF 4-C CHEMCOMP CEMENT

Aluminum oxide (Al_2O_3), max., percent	10.0
Ferric oxide (Fe_2O_3), max., percent	5.0
Magnesium oxide (MgO), max., percent	5.0
Sulfur trioxide (SO_3), max., percent	12.0
Loss on ignition, max., percent	3.0
Insoluble residue, max., percent	0.75

TABLE II

PHYSICAL REQUIREMENTS OF 4-C CHEMCOMP CEMENT

Fineness, specific surface, SQ-CM/GR:	
Air permeability test (ASTM C-204):	
Average value, min.	2800
Minimum value, any one sample	2600
Expansion, Restrained: (7 day)	
Percent, not less than	.03
Percent, not more than	.10
Time of setting:	
Gillmore test (ASTM C-266):	
Initial set, minutes, not less than	45
Final set, hours, not more than	10
Setting of concrete mixtures by penetration resistance (ASTM C-403-67T):	
Initial set, hours, not less than	2.5
Final set, hours, not more than	12
Air content of mortar (ASTM C-185) max., percent by volume, less than	12
Compressive strength, PSI, (ASTM C-109):	
Not less than the values specified for the ages indicated below:	
3 days in moist air	1200
3 days in moist air, 4 days in water	2100
3 days in moist air, 25 days in water	3500

<u>Sand</u>

A well-graded river sand was utilized in the concrete mixtures studied in this investigation. The material was obtained from the Ote Sand Gravel Company at Ponca City, Oklahoma. Tests to determine the specific gravity and the percent absorption of the fine aggregate were performed according to the procedure set forth by the American Society for Testing and Materials (ASTM C-128) (20). The results of these tests showed the bulk specific gravity to be 2.63 and the amount of absorption to be 0.19 percent. Tests were also performed to determine the gradation of the fine material. The results of several sieve analysis of the sand are shown in Table III.

TABLE III

SIEVE ANALYSIS OF THE SAND

Sieve Size	Percent Passing
3/8"	100.0
# 4	98.7
# 8	84.5
# 16	60.6
# 30	35.1
# 50	11.4
#100	0.8

Fineness Modulus = 3.10

Coarse Aggregate

A crushed limestone was used for the coarse aggregate in the concrete mixtures. The material was obtained from the Quapaw Rock Plant at Drumright, Oklahoma. The specific gravity and the percent absorption were determined by the procedure set forth by the American Society for Testing and Materials (ASTM C-127) (21). Results of these tests showed the bulk specific gravity to be 2.65 and the amount of absorption to be 1.07 percent. A sieve analysis of the coarse material gave the results shown in Table IV. The grain size distribution of the coarse and fine aggregate is shown in Figure 1.

TABLE IV

SIEVE ANALYSIS OF THE COARSE AGGREGATE

Sieve Size	Percent Passing
1"	99.4
3/4"	64.5
1/2"	13.0
3/8"	2.6
#4	0.0

Admixtures

The only admixture used in the fabrication of the specimens was an air-entraining agent. The same type of air-entraining agent, darex,

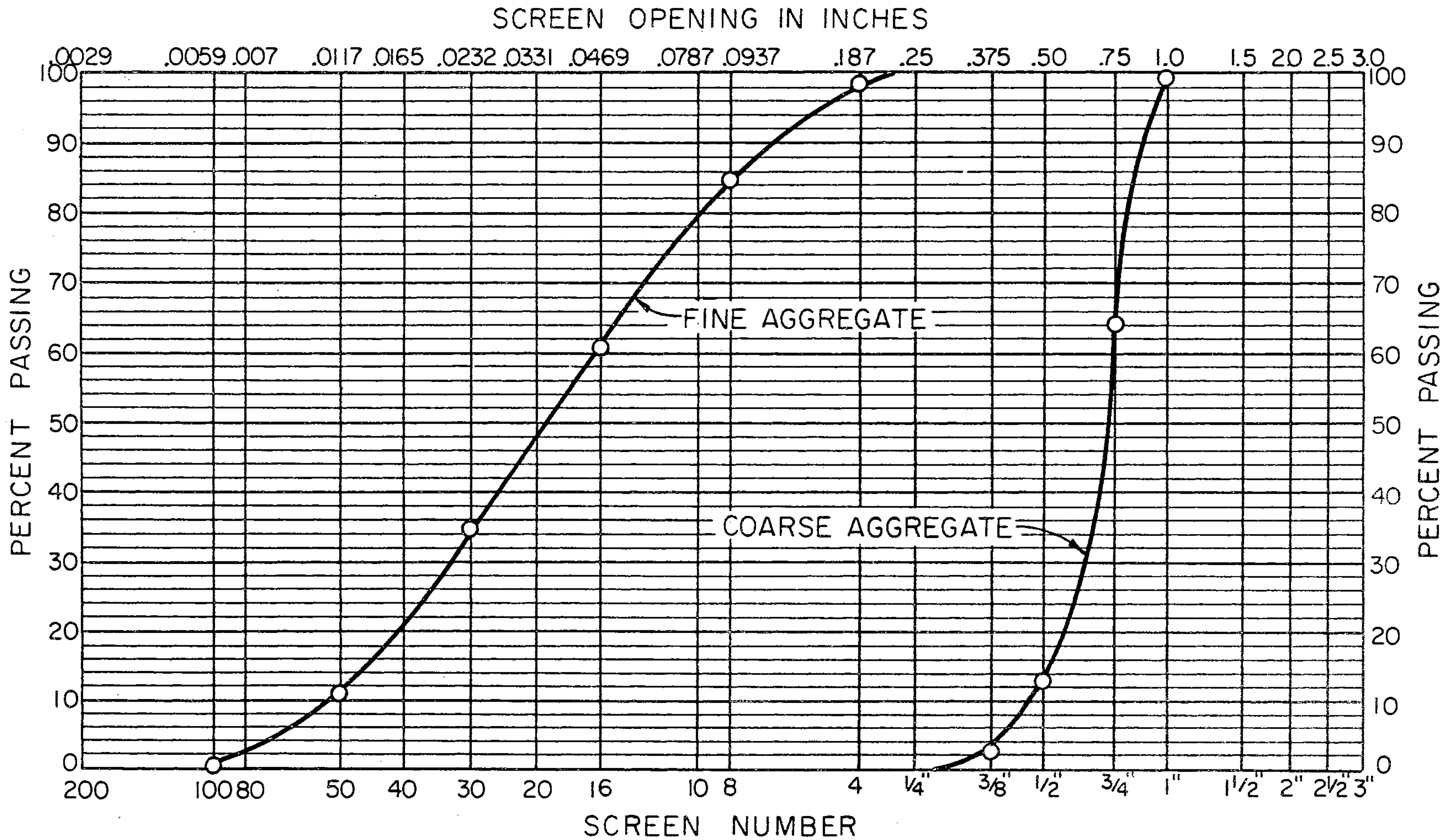


Figure 1. Grain Size Distribution of Coarse and Fine Aggregate

was used in all of the specimens containing entrained air.

Concrete

The concrete mixtures tested contained different water-cement ratios and different air contents. The water-cement ratios varied from approximately five to eight gallons per sack and the air contents varied from approximately two to twelve percent. To maintain a constant water-cement ratio and a slump of four inches, ± 1/2 inch, the ratio of sand to coarse aggregate had to be varied with varying air contents of the mixture. A summary of the concrete mix data is given in Table V. Each specimen number in the table represents three specimens.

Fabrication of Specimens

Prior to use, the aggregates were soaked in water for five days and then allowed to dry in the atmosphere for one day. At the end of the one day drying period, the aggregates were assumed to be in a saturated surface dry condition. Determination of the moisture contents of the aggregates very closely supported this assumption.

Each batch contained sufficient concrete to cast six 3x3x12 inch beams. The batches were mixed for approximately five minutes in a Homart Utility Mixer (Figure 2). Immediately after mixing, the slump was determined, then the air content of the mixture was measured by the air pressure method (ASTM C-231) (22). The apparatus used for making these determinations is shown in Figure 3.

If the desired consistency of the mixture was achieved, the concrete was then cast in water tight molds. The molds were made of an exterior type medium density overlaid plywood, with opaque resin

TABLE V

EXPANSIVE CONCRETE MIX DATA

Mix No.	Date Made D.M.Y.	Batch Proportions Cement (lbs)	C.A. (lbs)	F.A. (lbs)	Water (lbs)	A.E.A. (ml)	Slump (in)	Test Results Air Content %	W/C Ratio (gal/sack)
C- 2	1-7-68	8.62	18.31	16.26	3.68	0.0	3.5	1.7	4.81
C- 5	1-7-68	8.62	18.31	16.26	3.93	0.4	3.5	2.6	5.14
C- 8	1-7-68	8.62	18.31	16.26	3.93	2.0	3.7	4.5	5.14
C-11	1-7-68	8.62	18.31	16.26	3.93	4.0	4.5	9.5	5.14
C-14	2-7-68	7.66	24.60	20.55	4.24	0.0	4.5	3.0	6.24
C-17	2-7-68	7.66	24.90	22.85	4.24	1.7	3.5	7.2	6.24
C-20	2-7-68	7.66	26.30	24.39	4.24	3.0	4.5	12.5	6.24
C-23	2-7-68	7.66	26.48	24.05	4.24	2.5	4.2	10.7	6.24
C-26	21-9-68	6.90	24.81	26.55	4.37	0.0	4.5	5.5	7.14
C-29	21-9-68	6.90	26.36	28.13	4.37	2.0	3.5	8.5	7.14
C-32	21-9-68	6.90	26.88	28.16	4.37	3.0	3.5	10.5	7.14
C-35	21-9-68	6.90	26.60	27.46	4.37	1.0	3.5	7.5	7.14
C-38	22-9-68	6.00	23.25	26.57	4.31	2.0	4.5	10.0	8.10
C-41	22-9-68	6.00	24.73	26.57	4.31	0.0	3.5	4.7	8.10
C-44	22-9-68	6.00	24.94	26.57	4.31	0.8	4.5	7.5	8.10
C-47	22-9-68	6.00	26.86	28.22	4.31	1.8	4.5	8.5	8.10

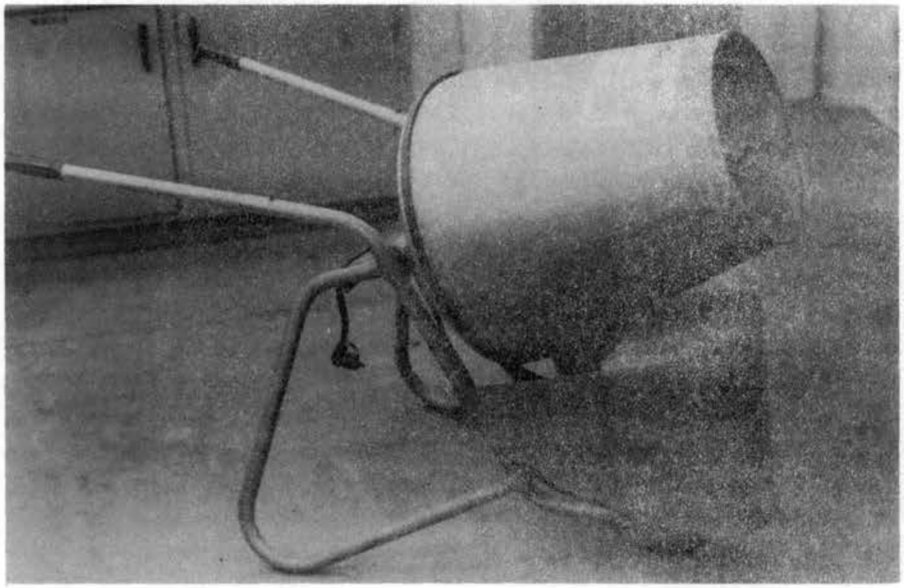


Figure 2. Homart Utility Mixer

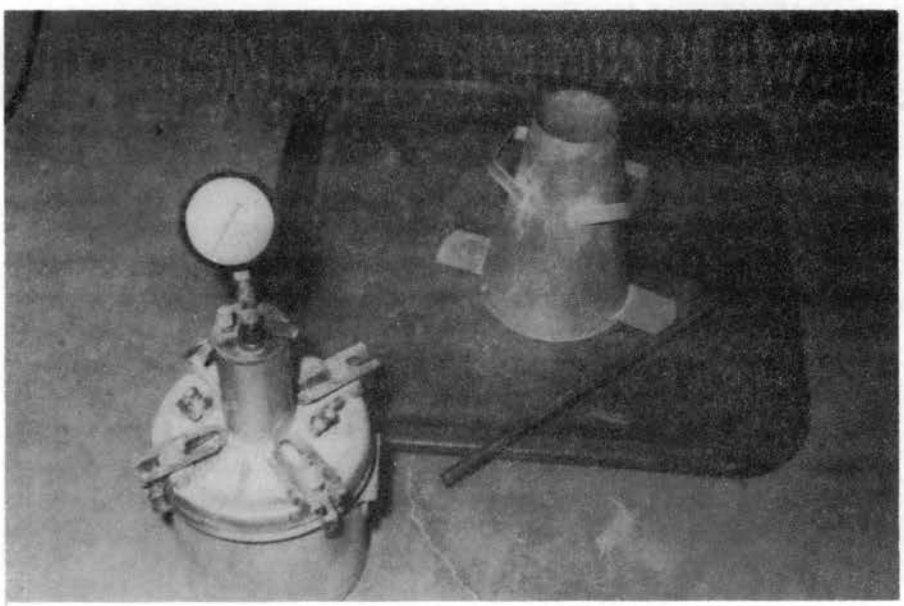


Figure 3. Air Content Measuring Apparatus,
Slump Cone, and Tamping Rod

impregnated fiber overlay heat-fused to both sides. To insure against the loss of water from the molds, the internal intersections of the faces of the molds were lined with paraffin wax. Just prior to placing the concrete, the internal faces of the molds were lined with a thin film of oil to prevent bonding of the concrete to the walls of the form. A partially disassembled form is shown in Figure 4.

To facilitate consolidation, the specimens were cast on a Syntron, Model C-2 vibrator (Figure 5). The concrete was placed in two layers with three specimens being made from each batch.

Only 24 specimens could be fabricated in one group due to the size limitations of the freezer. For each group one additional specimen was molded which contained a chromelalumel thermocouple at the center. This specimen was used for measuring the temperature during freezing and thawing cycles. The positioning of the thermocouple is shown in Figure 6.

Curing

The specimens were placed in a moist room for 12 days. After the 12 day curing period the specimens were taken from the moist room for one day. During this time contact points (Figure 6) were placed in the beams by drilling a small hole in the ends of the specimen, then cementing the contact points in the holes with epoxy cement. The specimens were then placed back in the curing room to complete a 14 day curing period.

The temperature of the moist room varied between 75°F and 80°F.

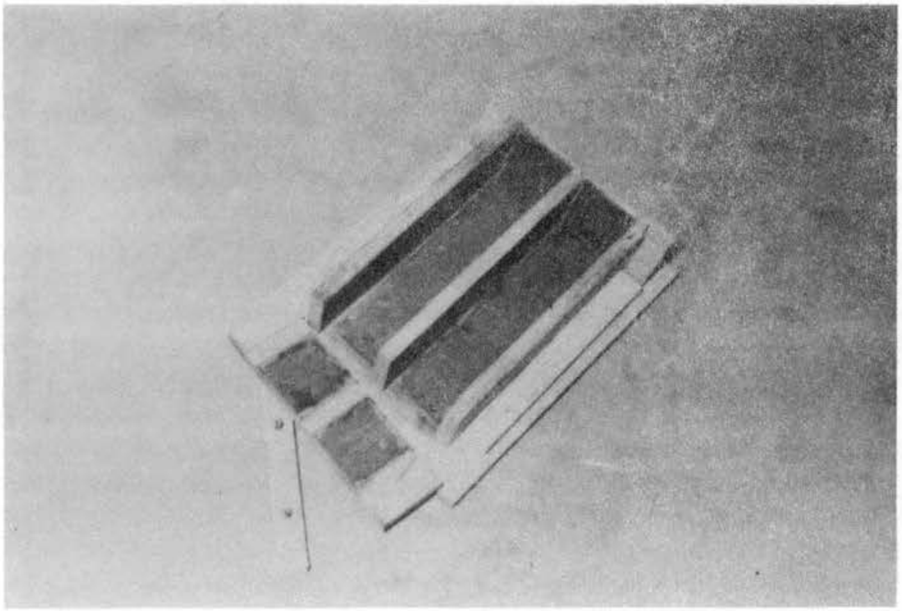


Figure 4. Partially Disassembled Form

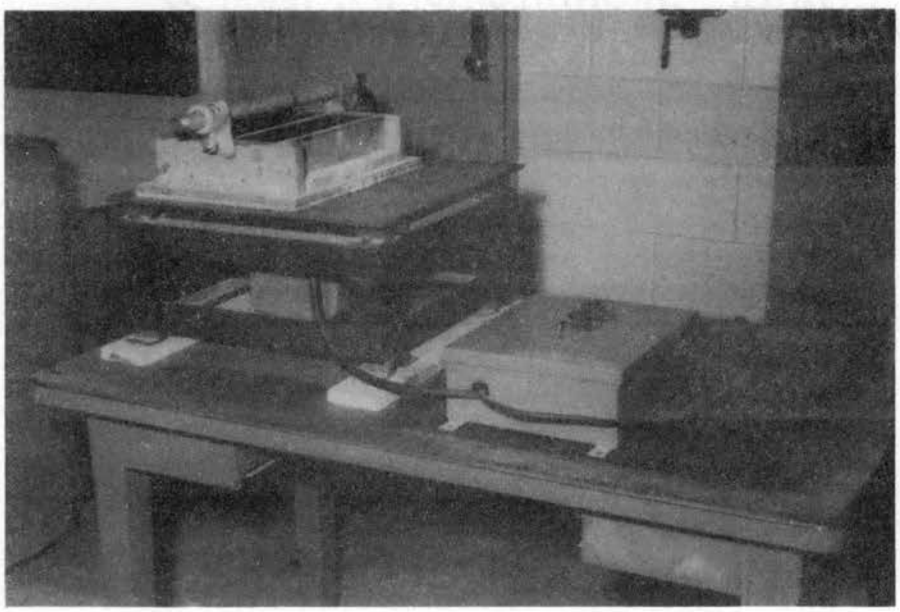


Figure 5. Syntron Model C-2 Vibrator

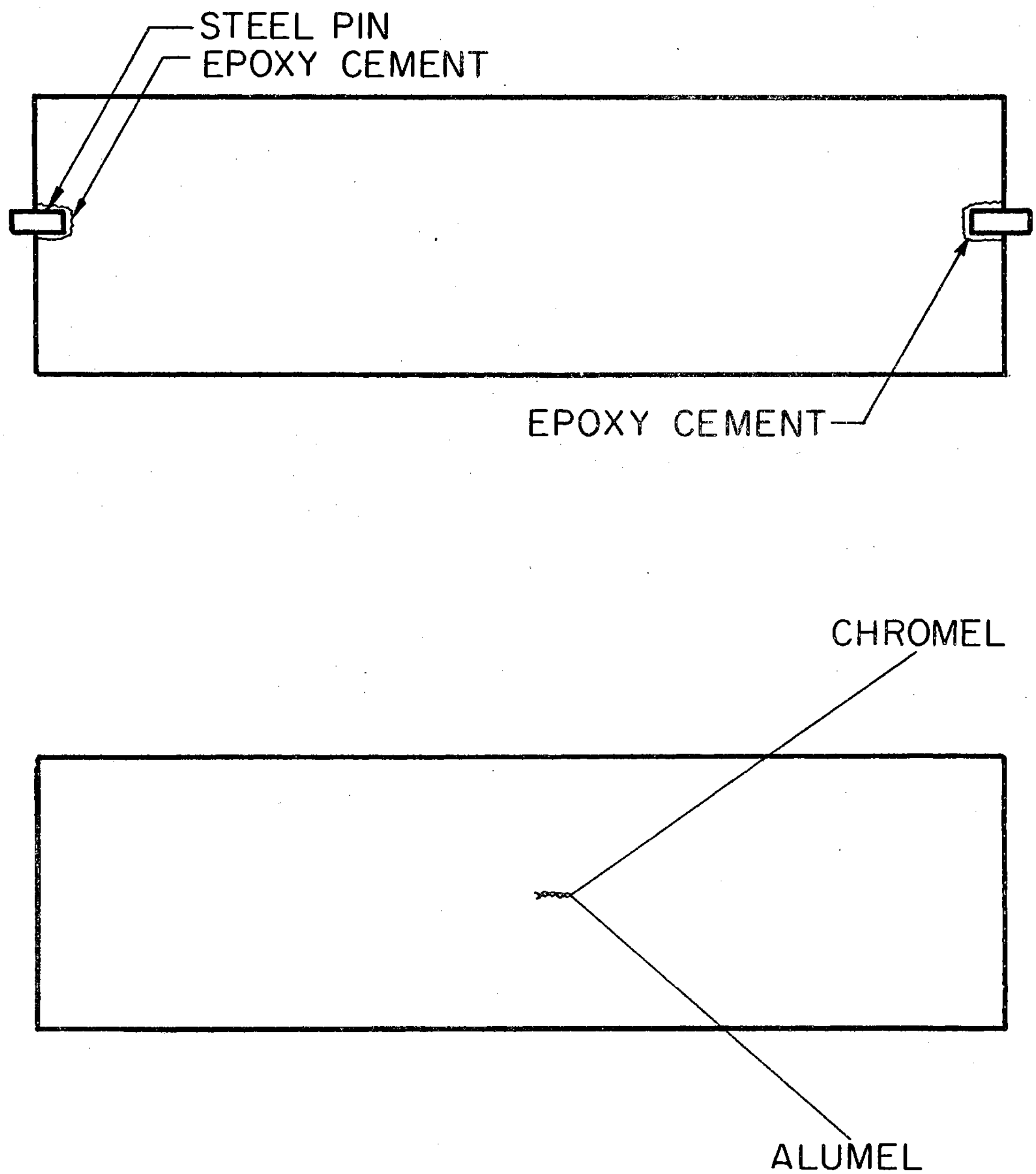


Figure 6. Positioning of Contact Points and Thermocouples in the Specimens

Testing Equipment

Sonic Testing Equipment

The sonic testing device used in these tests was the Soiltest CT-366 Sonometer. The sonometer is designed to provide a convenient means for the determination of the resonant frequency of a material as set forth in ASTM's Standard Method of Test for Fundamental Transverse, Longitudinal, and Torsional Frequencies of Concrete Specimens (23).

Special features of the CT-366 (Figure 7) include a built-in cathode-ray oscilloscope which eliminates the need for a separate unit. The pick-up assembly is designed to accept specimens of a standard size and to adapt easily to odd sizes. The adjustable driver unit can be used with samples including a wide range of heights without the need for separate blocks or supports.

In operation, the sonometer consists of two basic functions, the driving circuit and the pick-up circuit (Figure 8). The driving circuit consists of (1) a variable frequency oscillator to provide the range of audio frequencies needed in performing sonic tests, (2) a power amplifier which supplies power to energize the driving transducer, and (3) the driver transducer which mechanically couples the audio frequency power from the amplifier to the specimen in the form of vibrational energy.

The pick-up circuit is made up of (1) the pick-up transducer which, when resting on an oscillating specimen, changes the vibrational energy into electrical energy similar in operation to an ordinary phonograph record player, (2) an amplifier, and (3) the indicating devices. A meter and a cathode-ray oscilloscope are connected to the

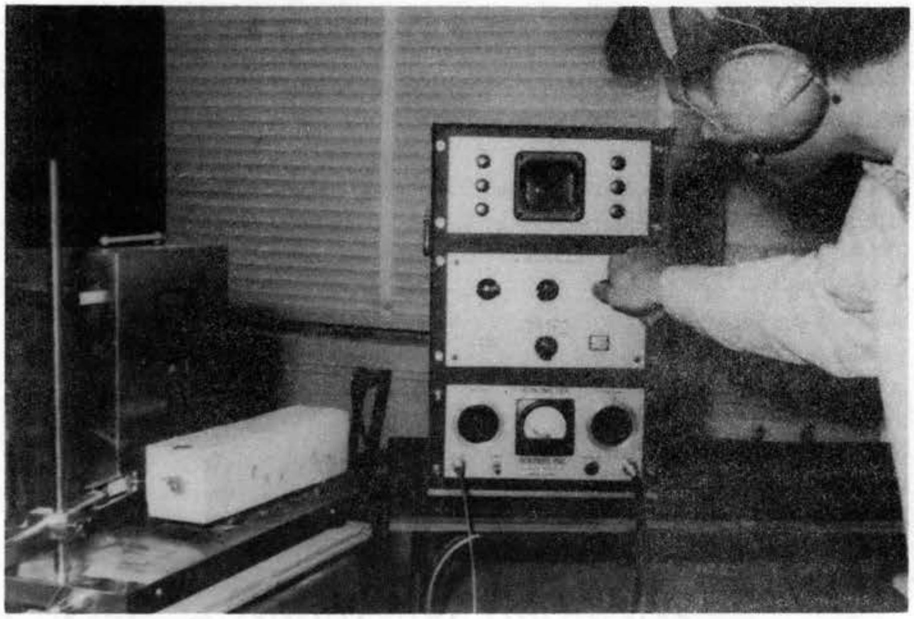


Figure 7. Soiltest CT-366 Sonometer

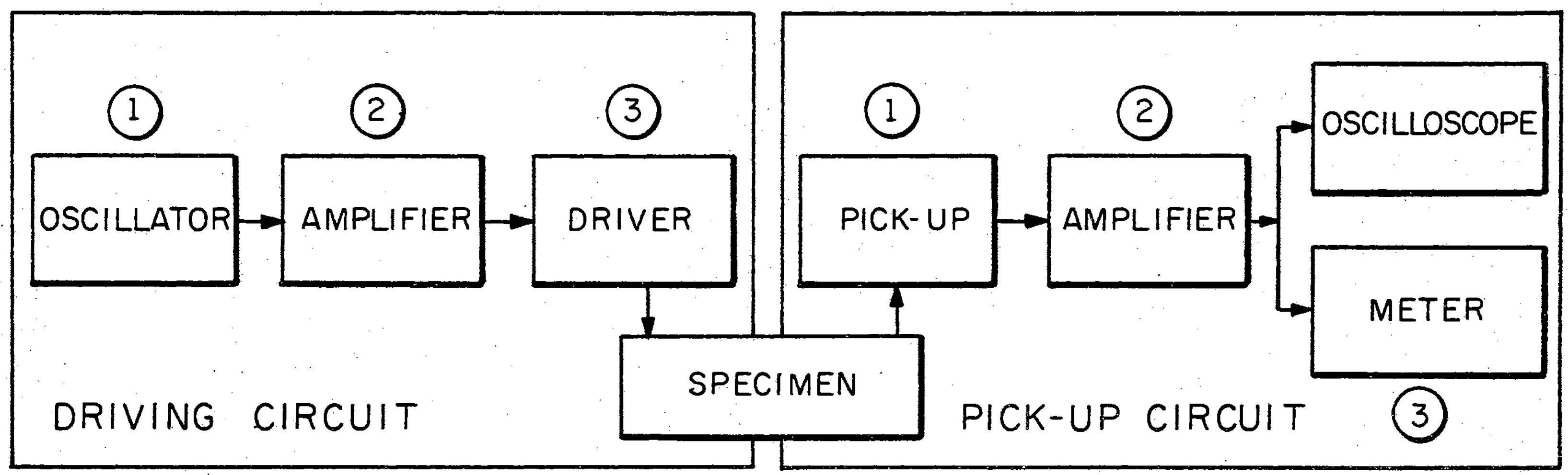


Figure 8. The Driving Circuit and Pick-up Circuit of the Soiltest CT-366 Sonometer

pickup circuit for use in the verification of resonance and in determining the mode of resonance. The position of the pick-up and driver are shown in Figure 9 for the three types of vibration.

Temperature Control Equipment

A Lab-Line Model 3922 controlled temperature cabinet with an operating range of -150°F to +150°F was used to reduce the temperature from 42°F ± 3°F to 8°F ± 3°F. The cabinet, a Leeds-Northrup potentiometer, and the specimens during a thaw cycle are shown in Figure 10.

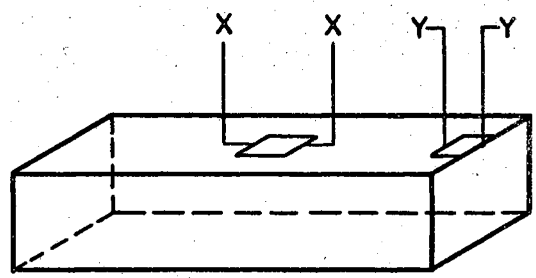
Length Measuring Equipment

A Soiltest CT-384 length comparator (Figure 11) was used to measure the lengths of the specimens throughout the freezing and thawing tests. The dial gage on the length comparator was graduated in increments of 0.0001 inch.

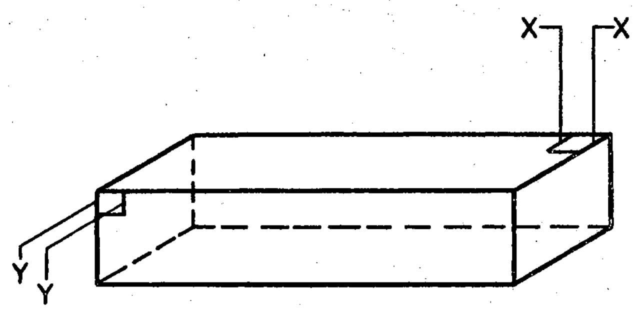
Testing Procedure

Immediately after the curing period, the lengths, weights and resonant frequencies of the specimens were determined. The lengths of the specimens were measured with the apparatus shown in Figure 11. The specimens were then weighed to the nearest one-hundredth pound. After weighing, the fundamental transverse resonant frequency of each specimen was determined by the use of the sonometer.

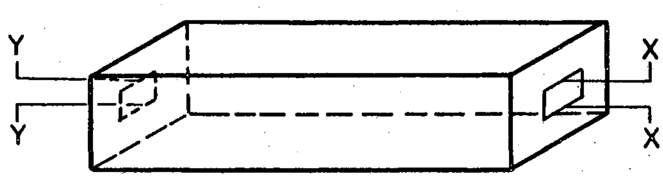
After vibration, the specimens were immersed in water for 30 minutes, then subjected to their first freeze cycle, which was 3 1/2 hours. All freeze cycles were 3 1/2 hours ± 15 minutes, while during thawing the specimens were immersed in water for 30 minutes ± 5



(a) DRIVER AND PICKUP POSITIONS FOR TRANSVERSE RESONANCE



(b) DRIVER AND PICKUP POSITIONS FOR TORSIONAL RESONANCE



(c) DRIVER AND PICKUP POSITIONS FOR LONGITUDINAL RESONANCE

XX - TO DRIVING UNIT
YY - TO PICKUP UNIT

Figure 9. Pick-up and Driver Positions for the Three Types of Vibration

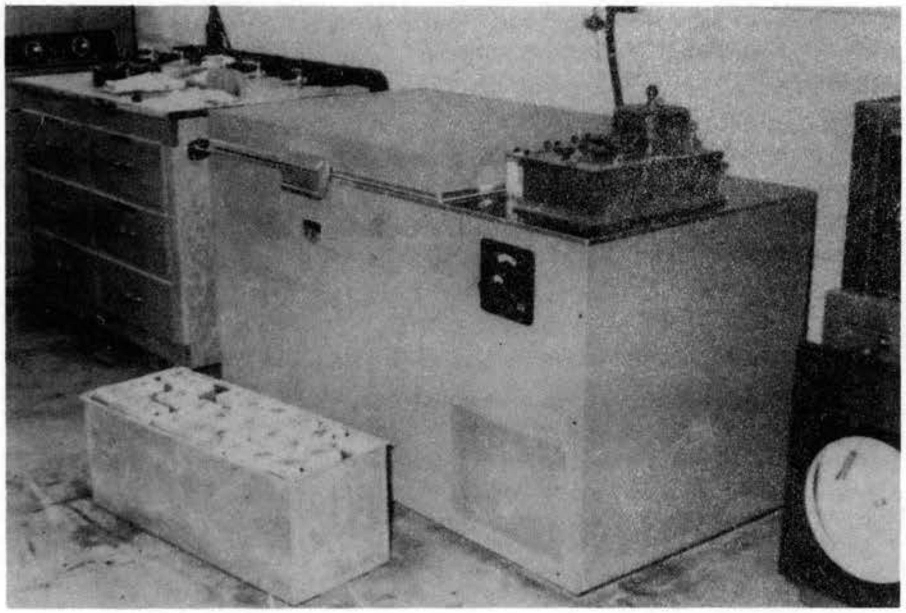


Figure 10. Lab-line Model 3922 Controlled Temperature Cabinet, Leeds-Northrup Potentiometer, and Specimens During a Thaw Cycle

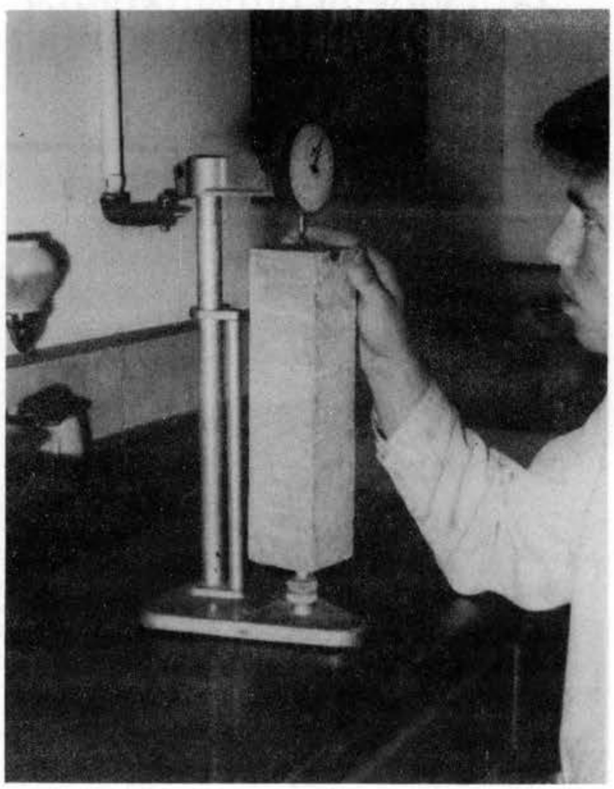


Figure 11. Soiltest CT-384 Length Comparator

minutes. The arrangement of the specimens in the freezer is shown in Figure 12. A fan was placed in the freezer to provide a uniform temperature within the controlled temperature cabinet during the freeze cycle.

The temperature changes during a typical freeze-thaw cycle are shown in Figure 13. Either five or six cycles were run per day, seven days per week. Many investigators have concluded that it does not make any difference how many freeze-thaw cycles are run per day, as long as the specimens are kept in a frozen condition when regular freezing and thawing tests are not being conducted. By keeping the specimens frozen the process of hydration is prevented.

After a desired number of freeze-thaw cycles, the lengths, weights, and fundamental transverse frequencies are again determined. These measurements were taken intermittently throughout the freeze-thaw testing until a total of 200 cycles were completed. The results of these measurements were used to plot graphs of percent change in weight vs. number of freeze-thaw cycles, percent change in length vs. number of freeze-thaw cycles, and relative dynamic modulus of elasticity vs. number of freeze-thaw cycles.

The percent change in weight was calculated from the following relation:

$$\text{PERWA} = \frac{(\text{Weight} - \text{Original Weight})(100)}{\text{Original Weight}} \qquad (3\text{-}1)$$

Where:

PERWA = percent change in weight.

The percent change in length was calculated from the following

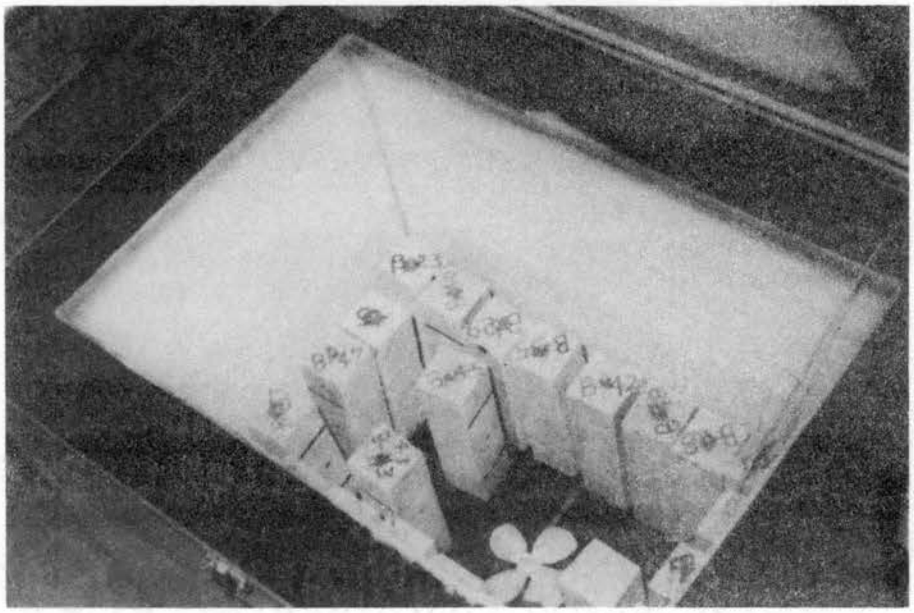


Figure 12. Positioning of Specimens During a Freeze Cycle

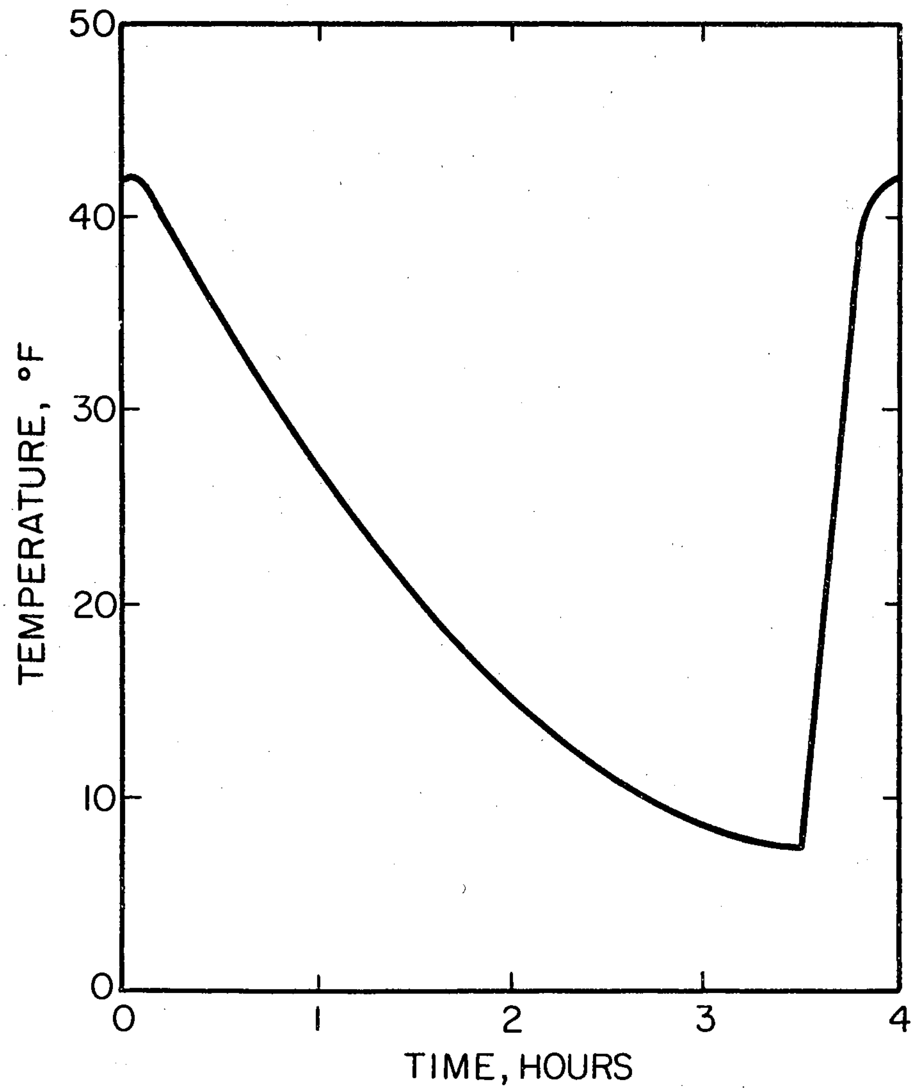


Figure 13. Temperature Changes During a Typical Freeze-Thaw Cycle

relation:

$$\text{PCHLEN} = \frac{(\text{Length - Original Length})(100)}{\text{Original Length}} \tag{3-2}$$

where:

PCHLEN = Percent change in length.

The determination of the relative dynamic modulus of elasticity involves considerably more calculations than the percent change in length and percent change in weight. Young's dynamic modulus of elasticity may be determined from the following relations (23):

$$\text{Dynamic E} = \text{CWn}^2 \tag{3-3}$$

where:

Dynamic E = Young's dynamic modulus of elasticity in pounds per square inch;

W = weight of specimen in pounds;

n = fundamental transverse frequency in cycles per second;

$C = \dfrac{0.00245L^3T}{bt^3}$ sec.^{2} per square inch for a prism;

L = length of the specimen in inches;

t, b = dimensions of cross-secion of prism in inches, t being the direction in which it is driven, and

T = a correction factor which depends on the ratio of the radius of gyration, K, to the length of the specimen, L, and on Poissons ratio, μ.

In an article from the _Proceedings of the American Society for Testing and Materials_, Pickett (24) discusses a relation that was developed by Goens for estimating a value which he calls T_1. For

Poisson's ratio equal to one-sixth, Goens developed the following relation:

$$T_1 = 1 + (81.79)\left(\frac{K}{L}\right)^2 - \frac{1314\left(\frac{K}{L}\right)^4}{1 + 81.09\left(\frac{K}{L}\right)^2} - (125)\left(\frac{K}{L}\right)^4 \qquad (3\text{-}4)$$

The value of T is then found by multiplying T_1 by a correction factor which depends on Poisson's ratio (μ), the radius of gyration, and the length of the specimen. Applying the correction results in:

$$T = T_1 \left[\frac{1 + (0.26\mu + 3.22\mu^2)\left(\frac{K}{L}\right)}{1 + 0.1328\,\frac{K}{L}}\right] \qquad (3\text{-}5)$$

After T is found, Young's dynamic modulus of elasticity is easily calculated.

In the tests conducted Poisson's ratio was estimated to be one-sixth, even though another investigator has found it to be slightly higher for expansive concrete (10). It should also be recognized that Poisson's ratio depends on the degree of saturation of the specimen. By inspection of Figure 1 in the article by Pickett (24), the correction factor in Equation (3-5) changes only slightly for Poisson's ratio between one-sixth and one-third, therefore the estimate of Poisson's ratio to be equal to one-sixth for all specimens seems reasonably accurate.

For these tests, the relative dynamic modulus of elasticity was defined to be:

$$\text{PERCEN} = \frac{(\text{DYNE})(100)}{\text{ORDYNE}} \qquad (3\text{-}6)$$

where:

PERCEN = relative dynamic modulus of elasticity,

DYNE = Young's dynamic modulus of elasticity after 200 freeze-thaw cycles,

ORDYNE = Young's dynamic modulus of elasticity at 0 freeze-thaw cycles.

To reduce the chances for errors in the calculation of Young's dynamic modulus of elasticity, the IBM 1620 computer was used to make these calculations. The computer program that was used to make the computations appears in the Appendix of this report.

CHAPTER IV

DISCUSSION AND RESULTS

In view of the wide variety of concrete materials, mix proportions, curing procedures, and test conditions employed, with a consequent wide variety of results obtained, it is not surprising that a considerable amount of uncertainty exists as to the meaning of the published results of freeze-thaw tests of concrete (25, 18). A survey of previous investigations on the subject show, however, that significant general conclusions can be drawn.

Dry hardened concrete, or concrete in which the hardened cement paste and the aggregate are both below some critical degree of saturation (about 91 percent) by water, is highly resistant to freezing and thawing.

Water-soaked hardened concrete may be damaged by freezing and thawing, and the locus of damage may be the hardened cement paste, or the aggregate, or both. In any of these cases, the damage arises basically from the fact that the process of freezing water within the paste or aggregate gives rise to the internal pressures which may be high enough to exceed the strength of the paste or the aggregate. The aggregate particles and the hard cement paste may appear to be solid bodies; they are so only superficially. Some aggregates are capable of absorbing several percent of water (25). The hard cement paste may include, to the extent of a fifth of its own volume, capillary space capable of holding water. The capillary structure of cement paste is very fine. Because of this, and in view of the large volume of freezeable water it may hold, it is not surprising that the cement paste may be damaged by freezing and thawing.

Damaging pressure in hardened cement paste may arise in any of several ways (14). First, there may not be sufficient empty space available to accomodate the additional solid volume produced when the contained water freezes. This possibility has given rise to the degree of saturation theory of frost damage. Second the pressure may be hydraulic, due to resistance of the fine-grained capillary structure to the flow of water displaced by the advancing ice front. Third, the pressure may be caused by the growth of ice bodies extracting water from the surrounding gel.

As mentioned in the Review of Literature, entrained air is used in portland cement concrete to increase its durability. It is to provide in the hardened paste a multiple of air voids separated from one another by only a few thousandths of an inch, so that excess water displaced during the freezing process has only a short distance to move before finding relief space. The extruded water, frozen in the air bubbles, is able to extract additional water from the surrounding paste and thus prevent the growth of ice bodies in the paste. In these ways, pressure is limited, and if the air voids are close enough together the pressure will not exceed the strength of the paste.

The methods currently used to test the frost resistance of concrete have in many cases shown results different from those when the concrete is exposed to natural weathering. Many concretes that have performed well in the field under natural weathering conditions have been shown in the laboratory to have poor durability. In an effort to make laboratory results to more nearly represent natural field conditions, the specimens were not saturated before freeze-thaw tests were begun; although this is done in most laboratory testing procedures. By using this method of laboratory testing the initial degree of saturation is decreased which will cause the specimens to undergo less deterioration, for a specified number of cycles, than would normally occur during accelerated freeze-thaw tests.

Effect of Water-Cement Ratio

As can be seen in Table VI, there were no large differences in the results for specimens with different water-cement ratios. It is believed that the lack of large differences is related to the degree of saturation of the specimens and an insufficient number of exposure cycles. However, Figures 14, 15, 16, and 17 show that the relative dynamic modulus of elasticity begins to decrease for water-cement ratios larger than six gallons per sack. The relative dynamic modulus of elasticity depends upon the change in weight and change in fundamental transverse frequency of the specimens. Therefore specimens with a relative dynamic modulus of elasticity greater than 100 would indicate almost no deterioration has occurred (investigators have shown that the relative dynamic modulus of elasticity may increase by 10 percent between a 14 day and a 28 day curing period), as is shown by the results for mixes containing five and six gallons per sack, and the results that show a relative dynamic modulus of elasticity less than 100 would indicate a weight loss and/or a decrease in the fundamental transverse frequency, as is shown by the results for the mixes

TABLE VI

RESULTS OF FREEZE-THAW TESTS

Mix Number	Number of Freeze-Thaw Cycles	Water-Cement Ratio	Change in Weight	Young's Dynamic Modulus of Elasticity x 10^{-6}	Relative Dynamic Modulus of Elasticity	Change in Length
		(gal/sack)	%	(Psi)		%
C- 2	200	4.81	-.1722	6.74	101.74	-.0069
C- 5	200	5.14	-.2084	6.36	101.55	-.0117
C- 8	200	5.14	-.1797	6.06	101.36	-.0048
C-11	200	5.14	-.1861	5.08	102.91	-.0096
C-14	200	6.24	-.2444	6.09	102.46	.0198
C-17	200	6.24	-.5809	5.59	101.25	-.0021
C-20	200	6.24	-.4222	4.37	101.57	.0018
C-23	200	6.24	-.4509	4.83	101.01	-.0042
C-26	200	7.14	-.5076	5.61	99.94	.0084
C-29	200	7.14	-.5935	5.16	99.88	.0052
C-32	200	7.14	-.4534	4.76	100.03	.0066
C-35	200	7.14	-.5516	5.19	97.82	.0052
C-38	200	8.10	-.3774	4.64	99.14	.0071
C-41	200	8.10	-.5109	5.49	97.88	.0060
C-44	200	8.10	-.4842	4.97	98.79	.0092
C-47	200	8.10	-.3323	5.00	100.14	.0086

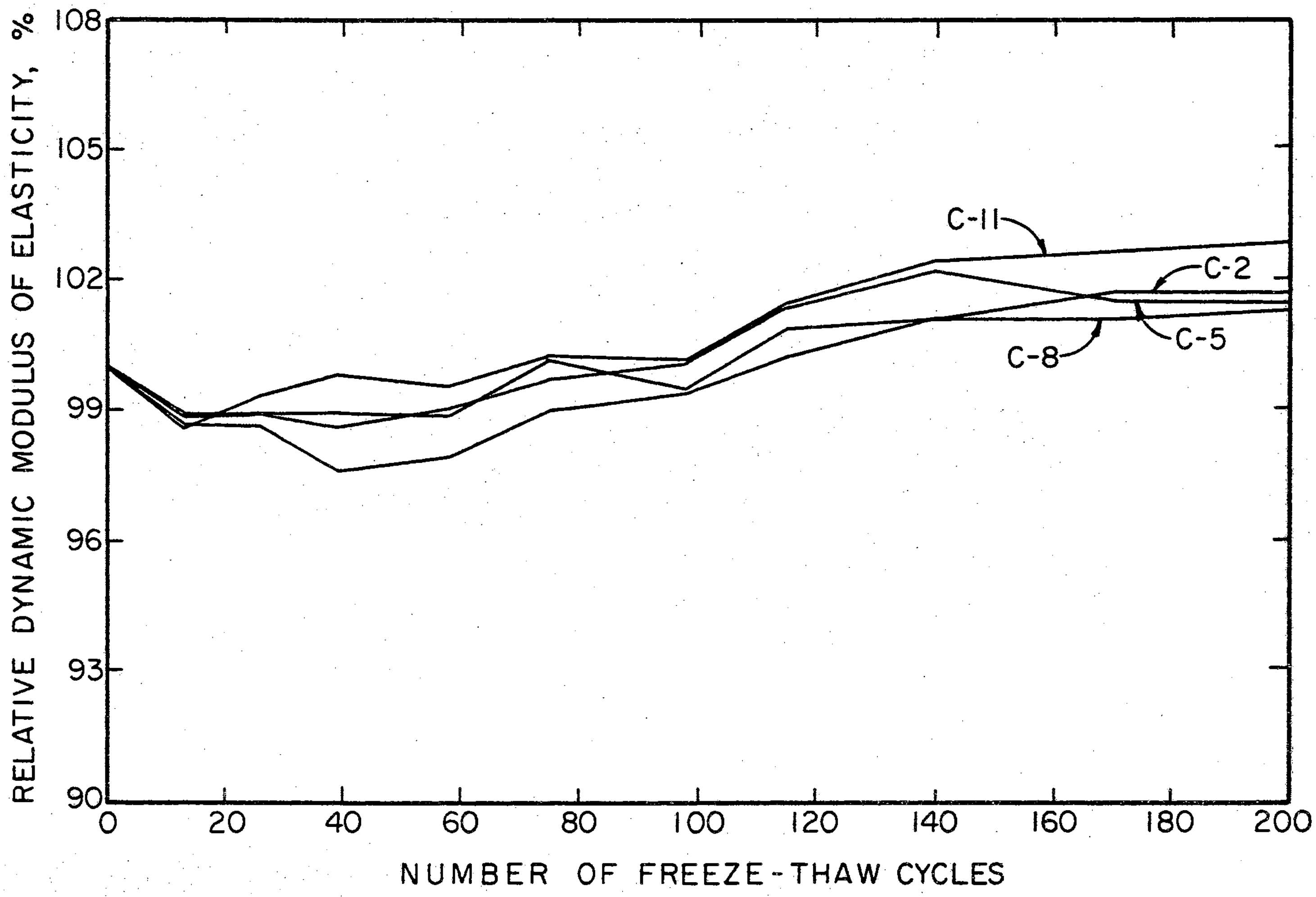


Figure 14. Relative Dynamic Modulus of Elasticity vs. Number of Freeze-Thaw Cycles (W/C $\approx$ 5 Gallons Per Sack)

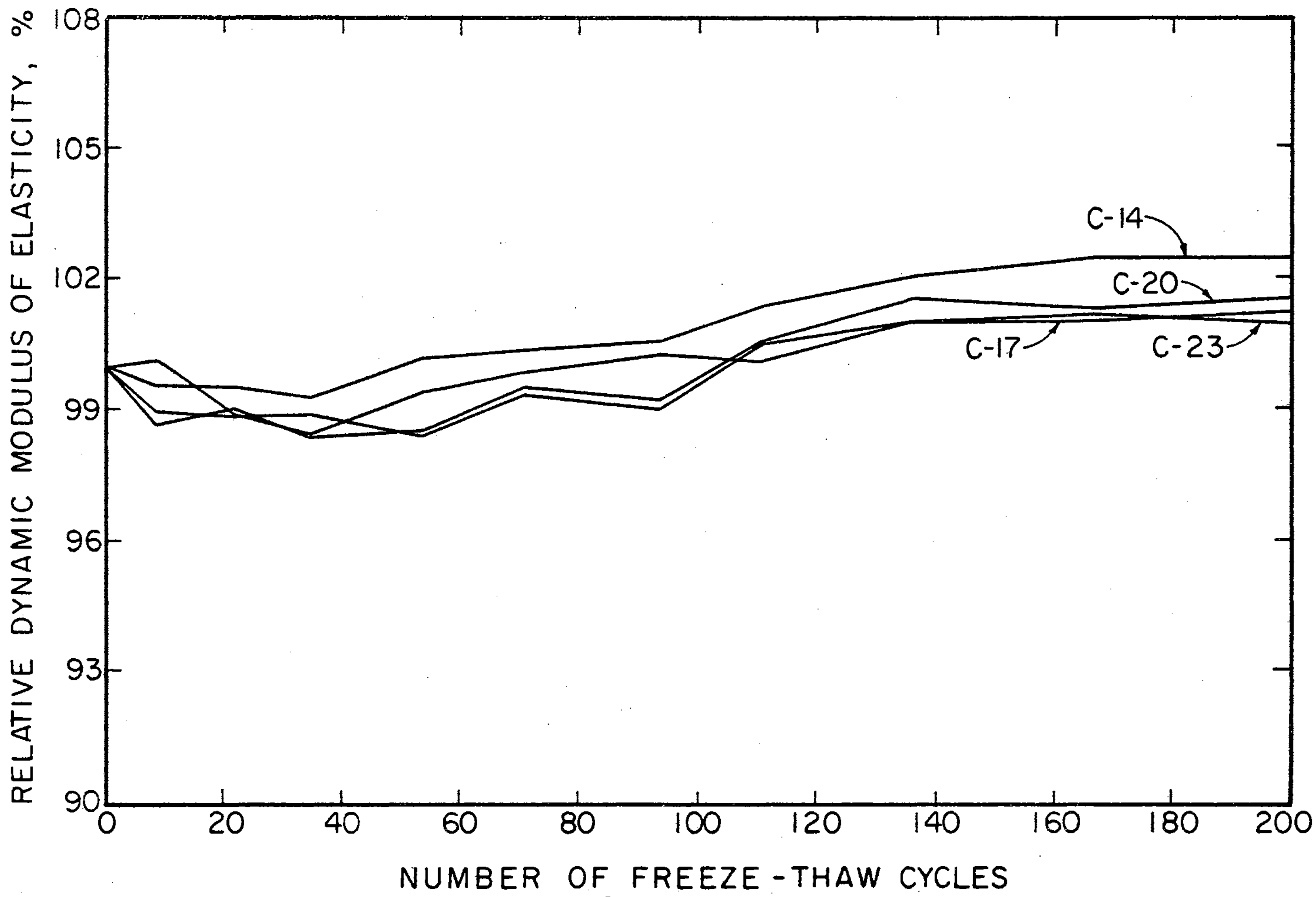


Figure 15. Relative Dynamic Modulus of Elasticity vs. Number of Freeze-Thaw Cycles (W/C ≈ 6 Gallons Per Sack)

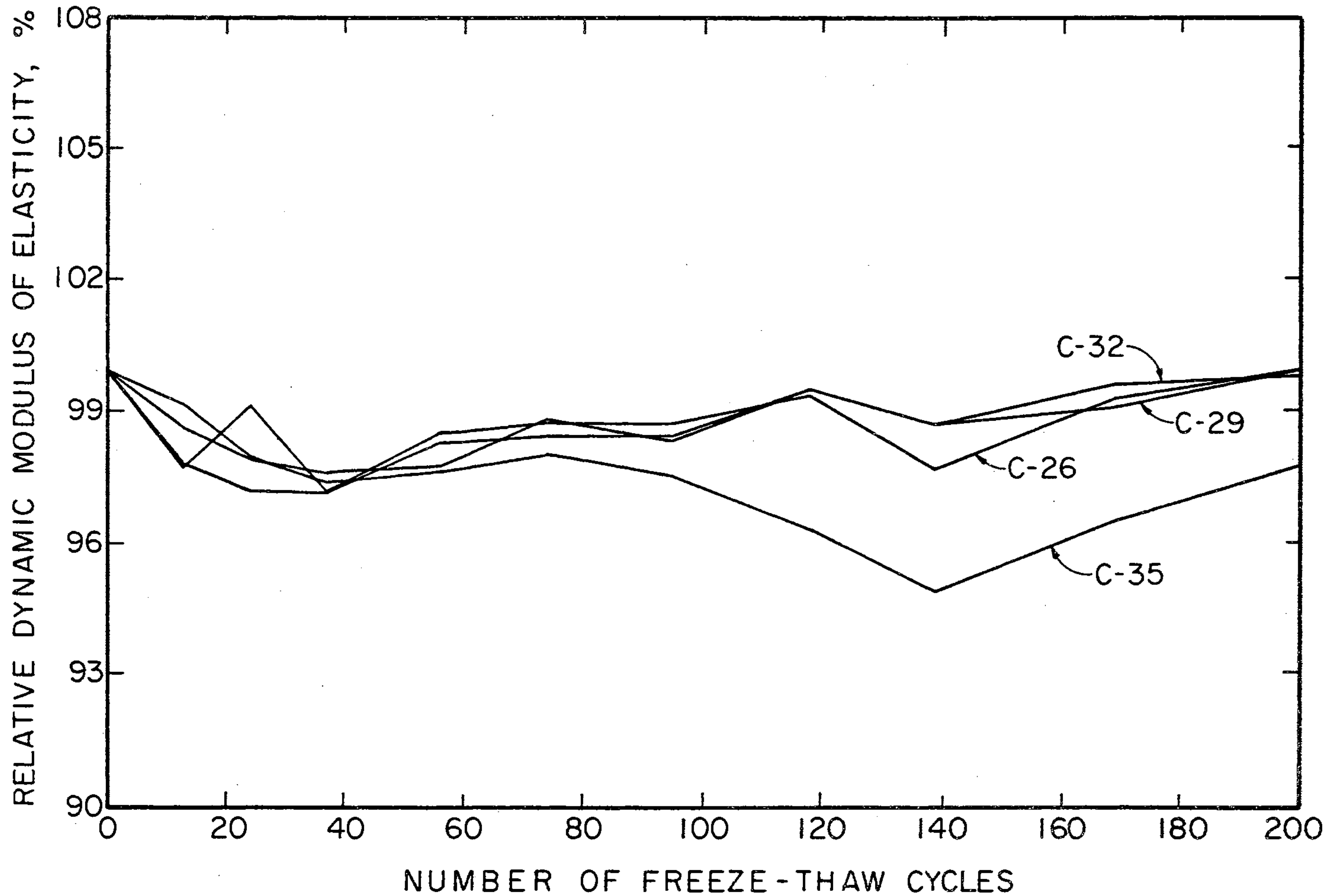


Figure 16. Relative Dynamic Modulus of Elasticity vs. Number of Freeze-Thaw Cycles (W/C $\approx$ 7 Gallons Per Sack)

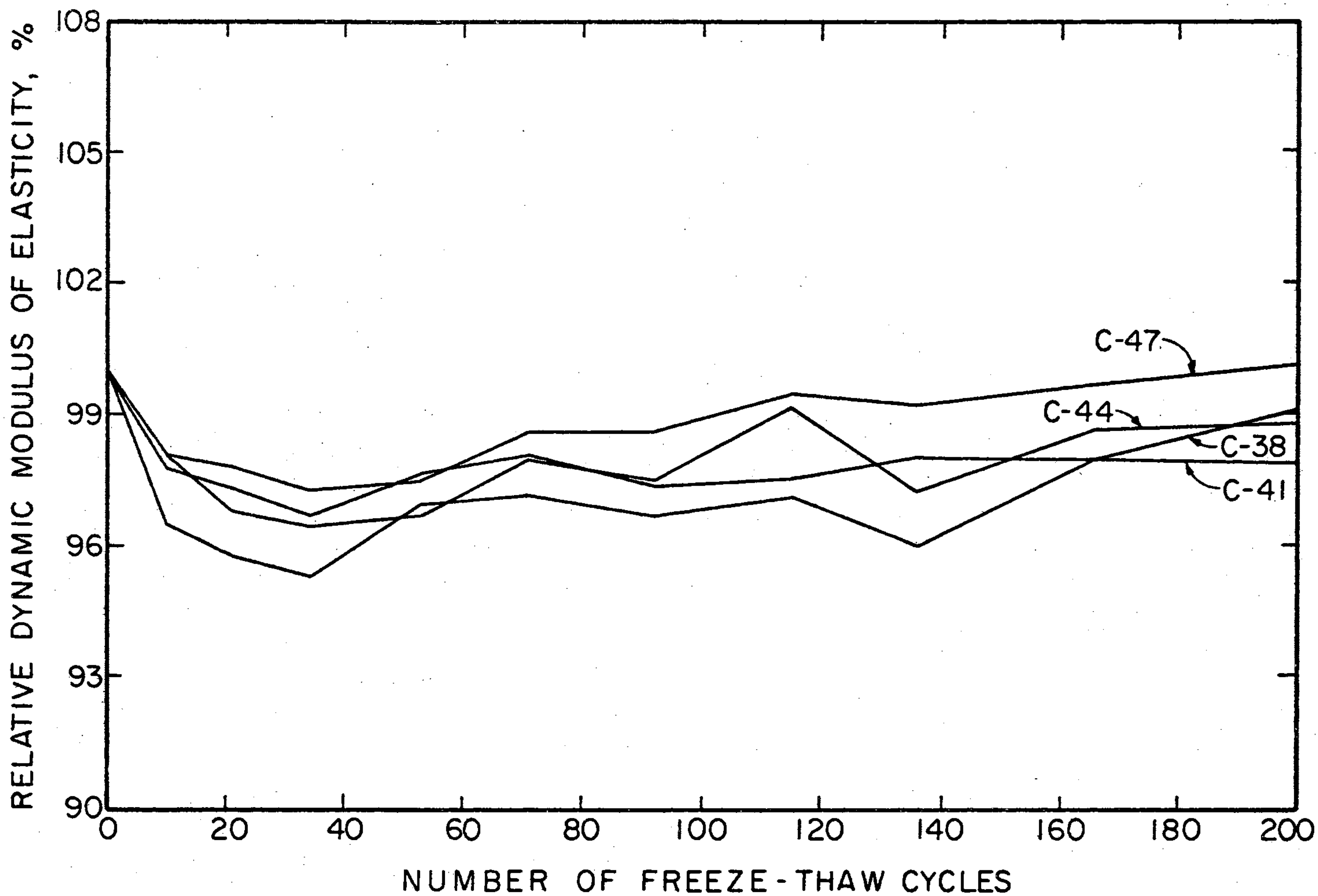


Figure 17. Relative Dynamic Modulus of Elasticity vs. Number of Freeze-Thaw Cycles (W/C $\approx$ 8 Gallons Per Sack)

containing seven and eight gallons per sack.

Use of the change in weight as a method of determining durability has been given some criticism by several investigators. It is usually argued that weight is lost by surface scaling of the specimen while at the same time the weight of the specimen is increased by absorption of water during the thaw cycle. However, an observation of Figures 18, 19, 20, and 21 shows that all specimens lost weight after 200 freeze-thaw cycles, which would indicate the amount of surface scaling was greater than the amount of water absorbed during the thaw cycles.

In several laboratory testing procedures percent change in length is used as the method of determining the relative durability of the concrete. Before inspecting Figures 22, 23, 24, and 25, it should be recognized that the change in length is also dependent on the richness of the mix and the amount of air in the mixture. However, investigations of the 4-C ChemComp cement show that almost all of the expansion occurs during the initial 14 day curing period for both air-entrained and non-air-entrained expansive cement (2). Since the measurements in these tests were not made until after the freeze-thaw tests were begun the measurement of the percent change in length is believed to give a good indication of the relative durability of the mixes. An increase in length would indicate that internal cracking had taken place in the specimen while a decrease in length would indicate that no internal cracking had occurred in the specimen and that its internal strength had probably increased.

Inspection of Figures 22, 23, 24, and 25 show that the percent change in length increased with increasing water-cement ratios. A positive increase is shown in all of the mixes made with seven and

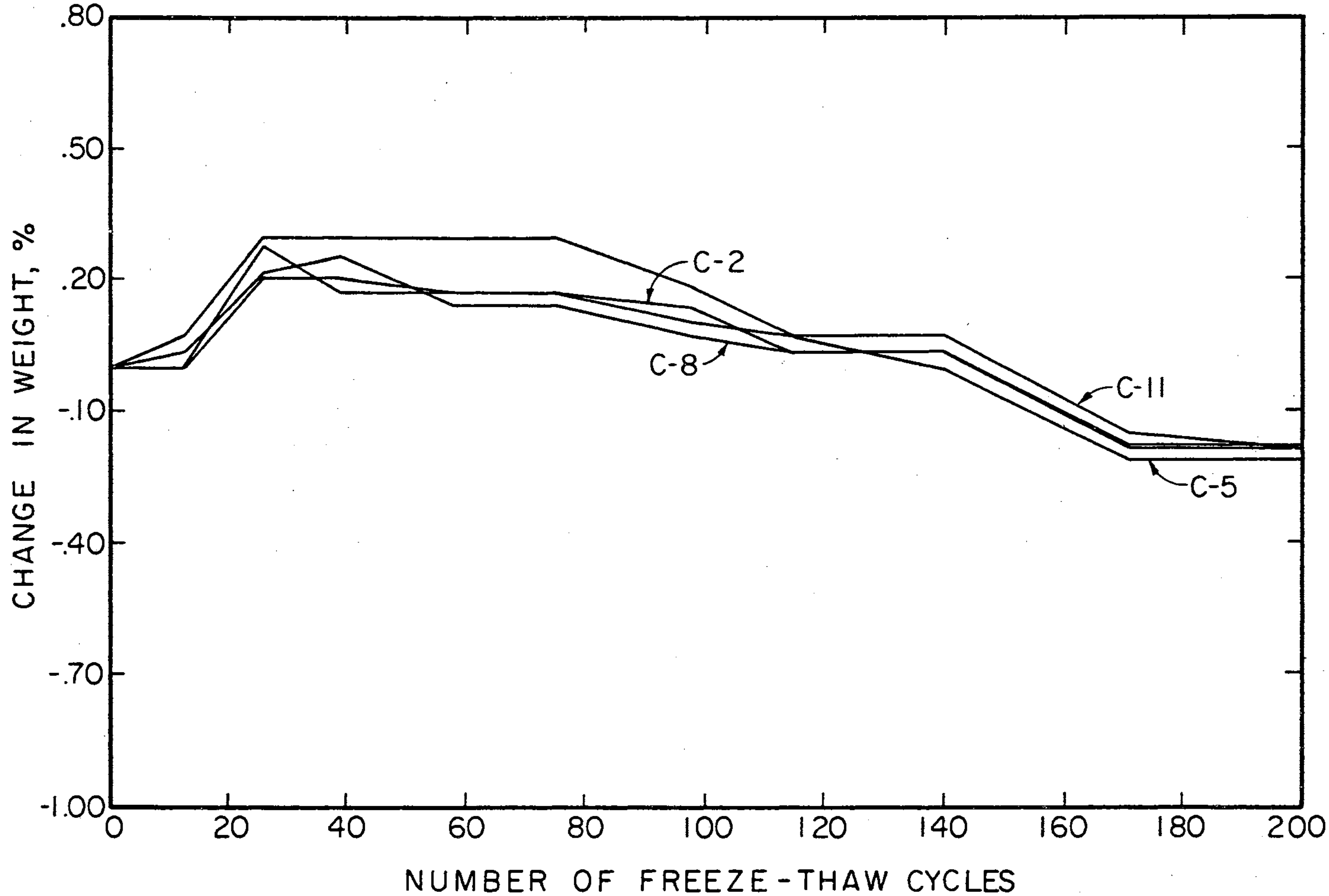


Figure 18. Change in Weight vs. Number of Freeze-Thaw Cycles (W/C ≈ 5 Gallons Per Sack)

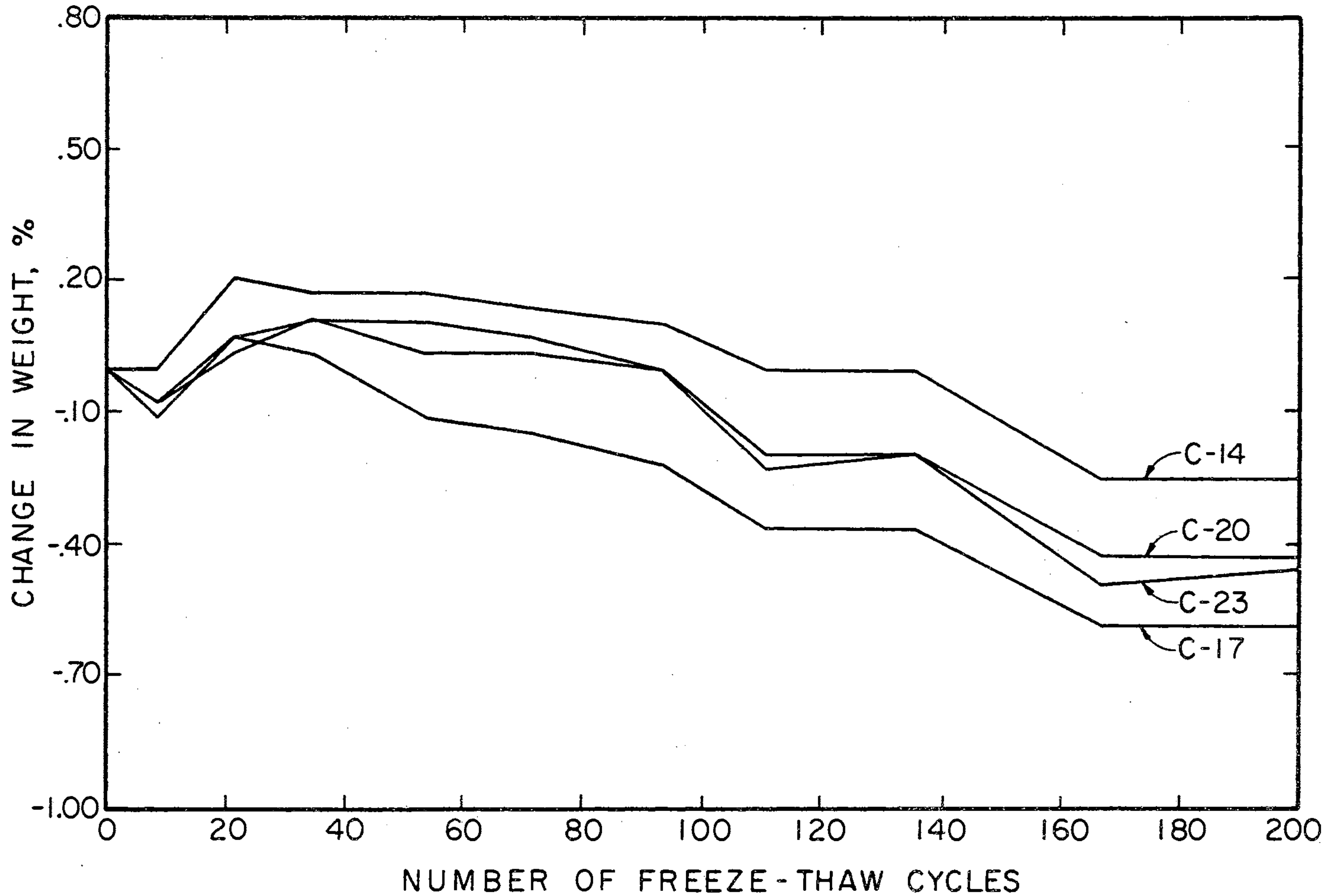


Figure 19. Change in Weight vs. Number of Freeze-Thaw Cycles
(W/C $\approx$ 6 Gallons Per Sack)

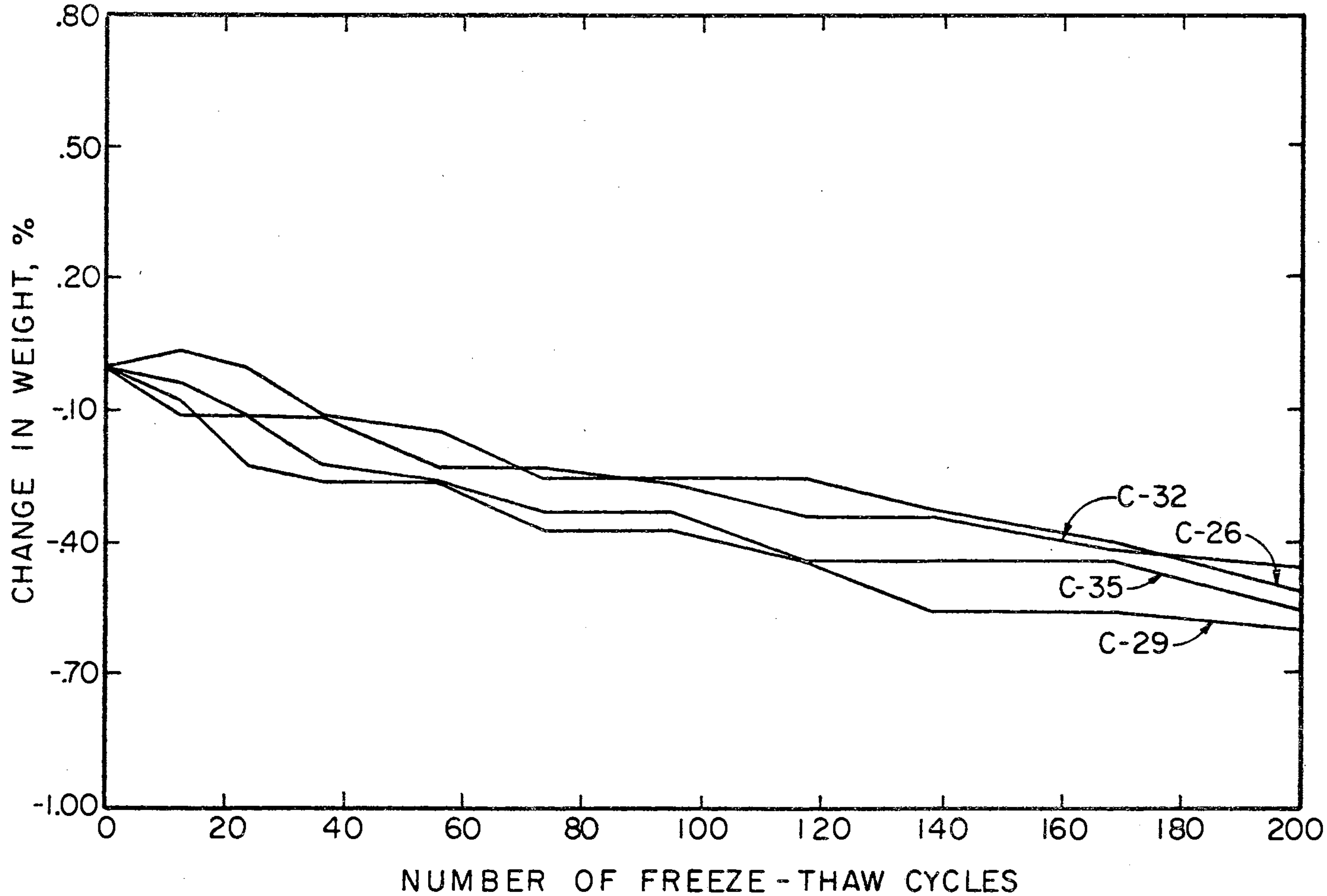


Figure 20. Change in Weight vs. Number of Freeze-Thaw Cycles
(W/C ≈ 7 Gallons Per Sack)

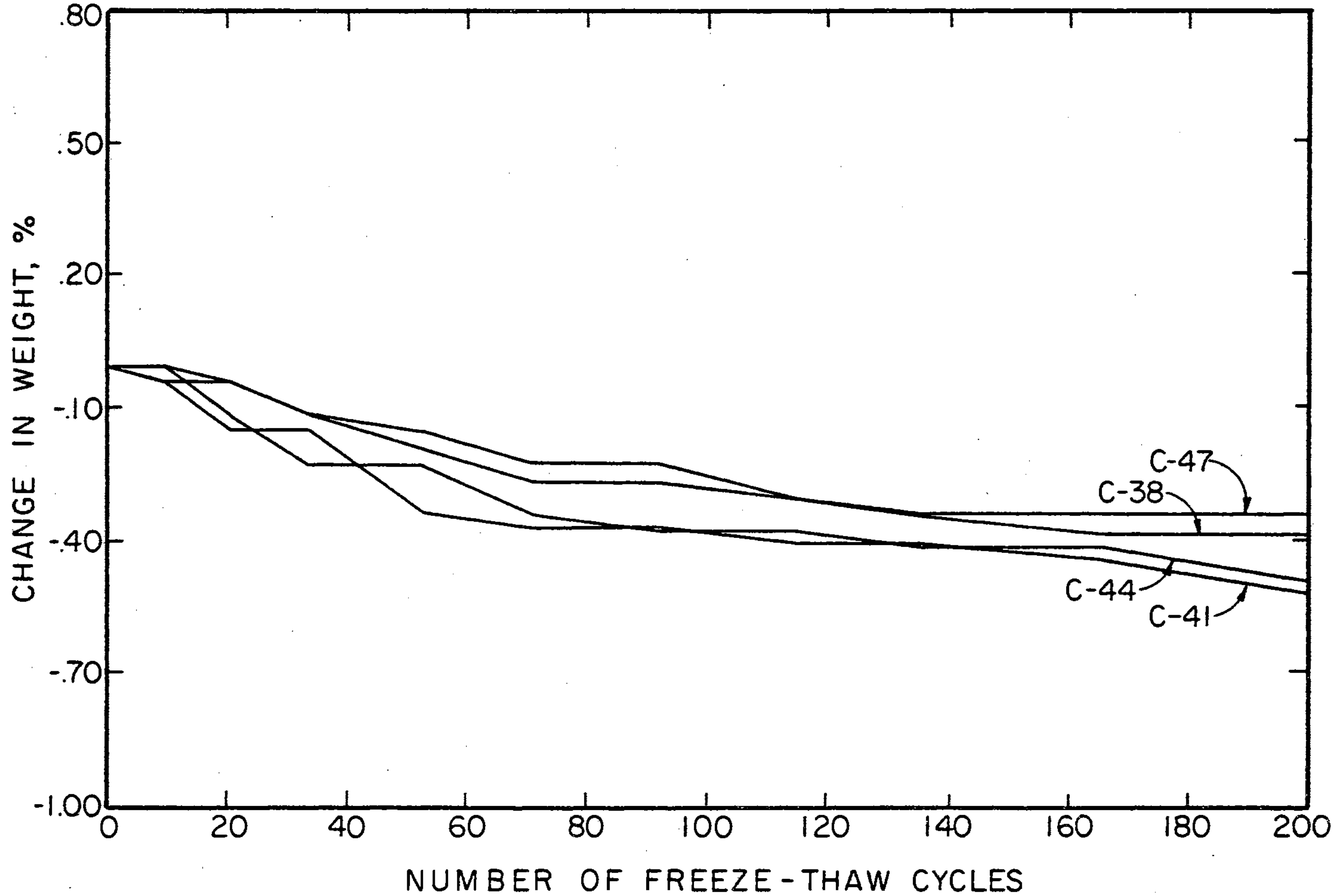


Figure 21. Change in Weight vs. Number of Freeze-Thaw Cycles
(W/C ≈ 8 Gallons Per Sack)

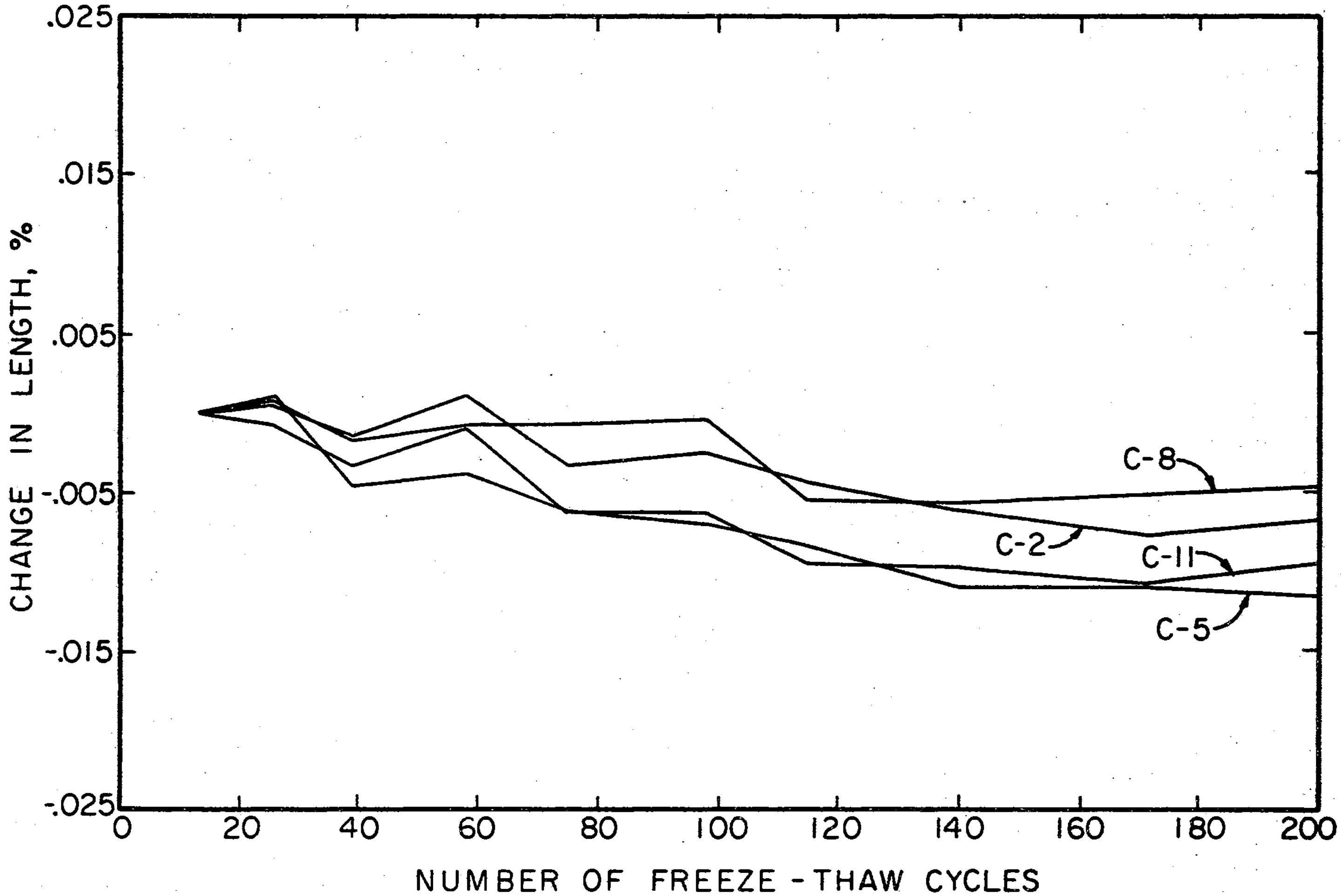


Figure 22. Change in Length vs. Number of Freeze-Thaw Cycles
(W/C $\approx$ 5 Gallons Per Sack)

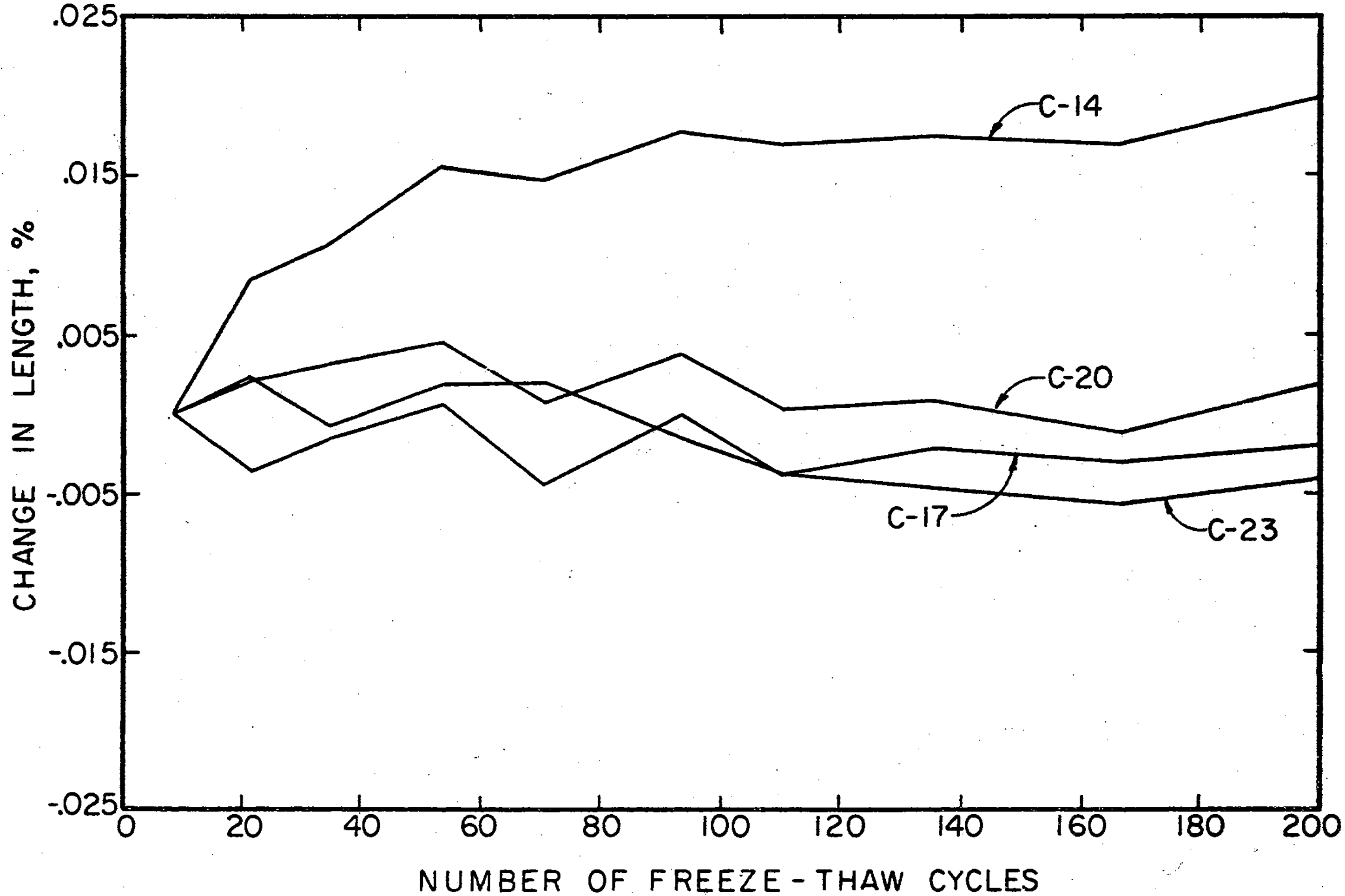


Figure 23. Change in Length vs. Number of Freeze-Thaw Cycles
(W/C $\approx$ 6 Gallons Per Sack)

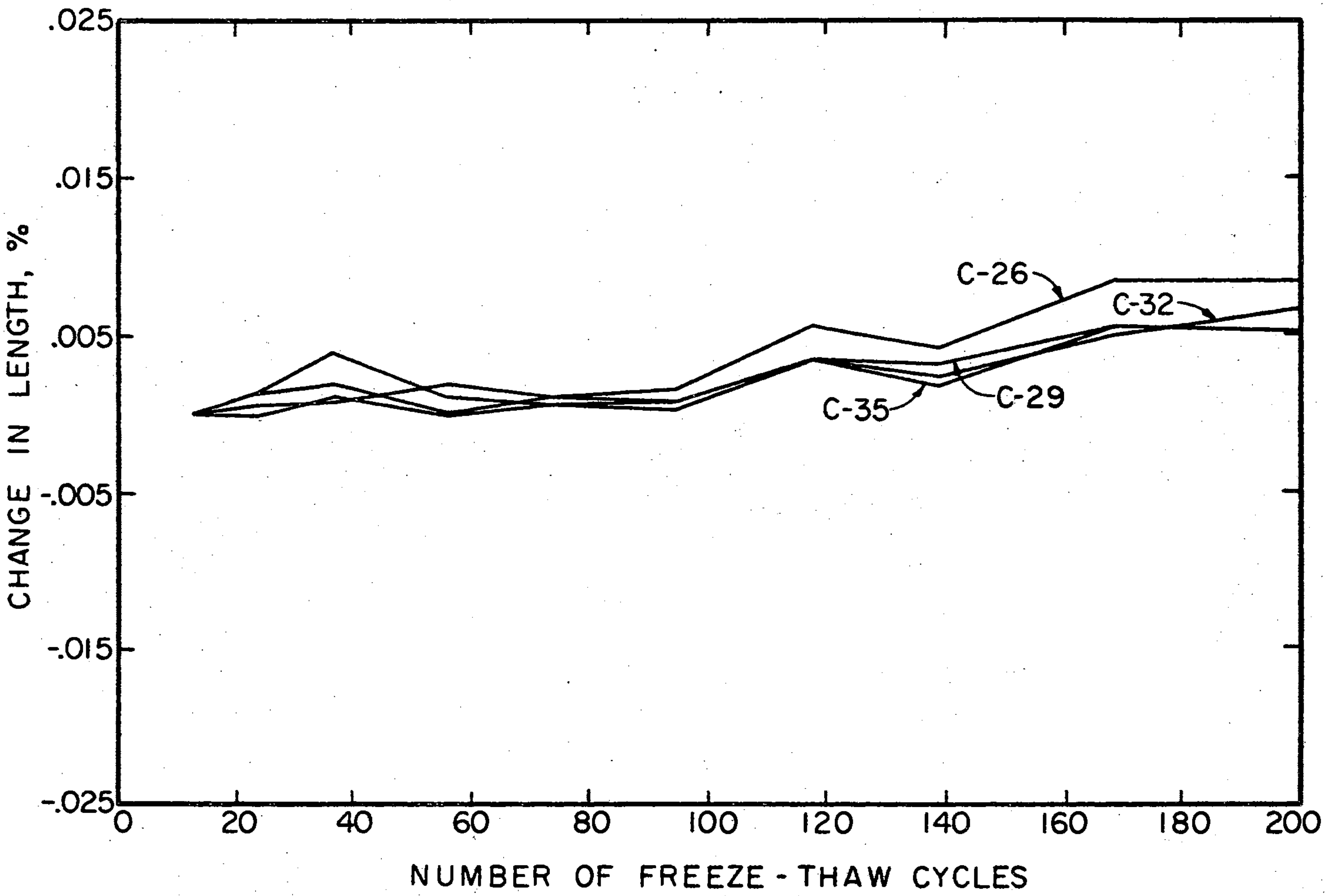


Figure 24. Change in Length vs. Number of Freeze-Thaw Cycles
(W/C ≈ 7 Gallons Per Sack)

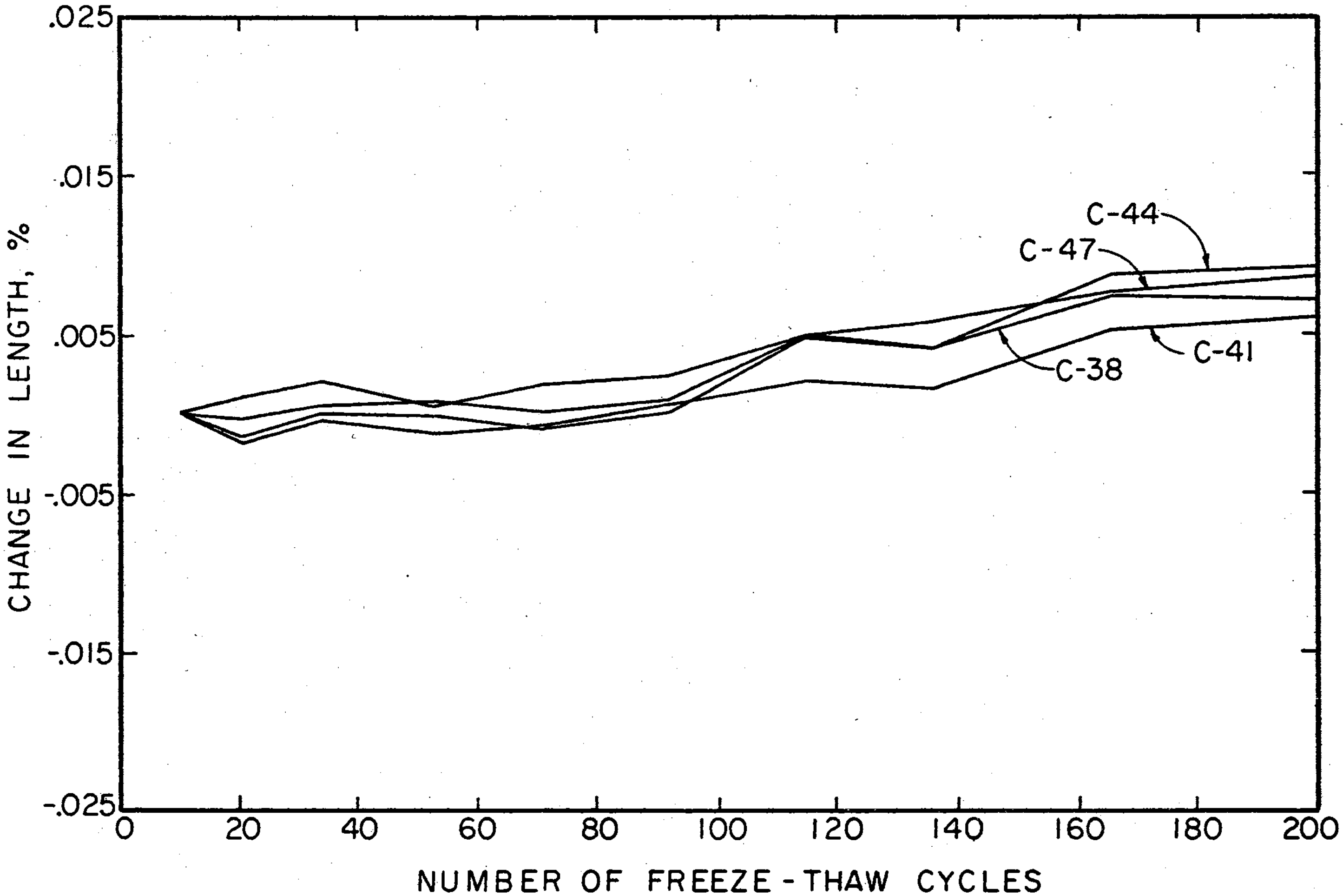


Figure 25. Change in Length vs. Number of Freeze-Thaw Cycles
(W/C $\approx$ 8 Gallons Per Sack)

eight gallons per sack which indicates a slight amount of internal cracking, while most of the mixes made with five and six gallons per sack showed no increase or a slight decrease in the percent change in length.

The results of the percent change in length and the relative dynamic modulus of elasticity agree very well as a means of determining the relative durability of the mixes.

Effect of Air Entrainment

After a careful study of the results no definite conclusions may be stated about the effect of using entrained air as a means of increasing the durability of expansive concrete. However it is believed that if the specimens had been exposed to more freeze-thaw cycles the effect of using an air-entraining agent may have become more apparent.

Tests by the Missouri State Highway Department on portland cement concrete exposed to natural field conditions over a period of 20 years also showed that air-entrainment had very little effect on the durability of the mixture (26). The results obtained by the Missouri Highway Department support the belief that the method of laboratory testing used in this study (that is, no pre-saturation of the specimens) may give a good indication of how the concrete will actually perform when exposed to natural conditions.

In Figure 26 is shown a specimen that contained 12 percent air. In these tests it was observed that all specimens containing more than 10 percent air showed damaging surface scaling.

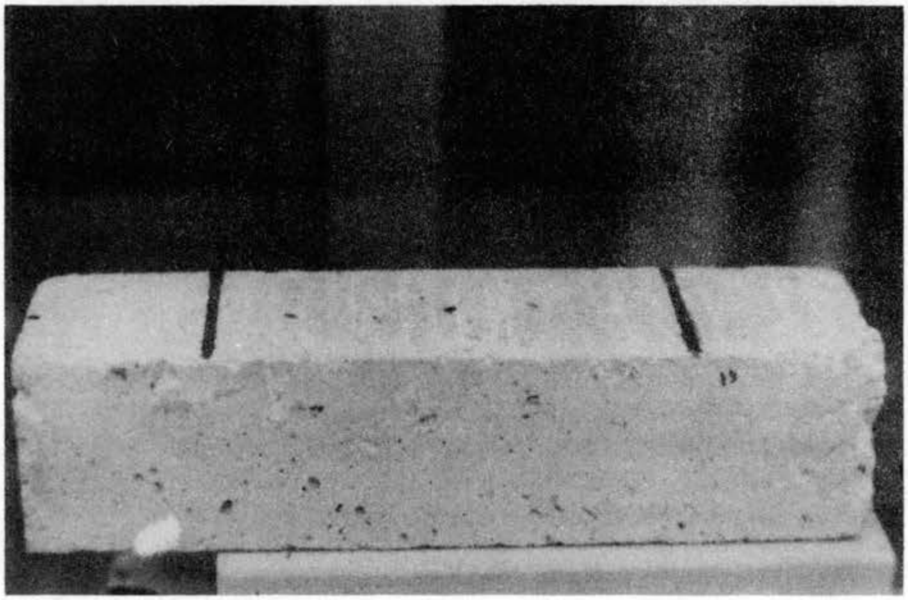


Figure 26. Specimen that Contained 12% Air

CHAPTER V

CONCLUSIONS AND RECOMMENDATIONS

This study was centered on the determination of the relative durability of unrestrained expansive concrete by exposing specimens to accelerated freeze-thaw tests. The method of test used was to freeze the specimens in air and thaw them in water. In an attempt to obtain laboratory results that would more nearly predict the behavior of expansive concrete under natural field conditions the specimens were not saturated before exposing them to accelerated freeze-thaw tests.

Conclusions

From the results of this study the following conclusions can be drawn:

1. In all specimens, the relative dynamic modulus of elasticity increased throughout the last 150 cycles. The probable reason for this behavior is believed to be related to the degree of saturation of the specimen; that is the degree of saturation was probably below that which is needed to cause damaging internal cracking of the specimen.

2. From the results of the percent change in relative dynamic modulus of elasticity all of the specimens showed a high degree of durability after 200 cycles. The degree of durability decreased as the water-cement ratio increased.

3. The percent change in length of the specimens increased as the

water-cement ratios increased which indicates that there was a slight amount of internal cracking in the specimens with high water-cement ratios.

4. The percent change in weight probably is not a very good measure of deterioration because weight is lost by surface scaling while weight is gained by absorption of water. However in these tests the weights of all specimens were decreasing which would indicate that the weight lost by surface scaling was greater than the weight gained by water absorption. The net weight losses along with the high degree of durability given by the relative dynamic modulus of elasticity would also support the belief that the specimens were saturated only near the surface.

5. The air content of expansive concrete should probably not be greater than 10 percent. The specimens that contained more than 10 percent air showed severe surface scaling for all water-cement ratios.

6. After 200 freeze-thaw cycles, no definite statement can be made as to the amount of entrained air that should be used in expansive concrete. However it is believed that if the number of cycles was increased the effect of air entrainment would be much more conclusive.

7. If concrete specimens are not saturated before exposing them to laboratory freeze-thaw tests, the results may conform to natural exposure conditions.

Recommendations

The method used to determine the relative durability of the specimens was limited to testing a small number of specimens at one time. Also the testing procedure required inconvenient work hours over a long period of time. If further tests are to be made on the durability of concrete it is suggested that automatic freeze-thaw equipment be used. Automatic equipment may be purchased or it can be built at a relatively small cost (27, 28, 29). By using automatic equipment the specimens can easily be exposed to more cycles per day and more specimens can be tested at one time.

It is suggested that further studies be made to compare the relative durabilities of portland cement concrete, prestressed portland cement concrete, unrestrained expansive concrete, restrained expansive concrete, and self-stressing expansive concrete.

BIBLIOGRAPHY

(1) Klein, A. and G. E. Troxell. "Studies of Calcium Sulfoaluminate Admixtures for Expansive Cements." Proceedings of the American Society for Testing and Materials, Vol. 58, 1958, pp. 986-989.

(2) Chemically Prestressed Concrete Corporation, Tentative Specifications for ChemComp Cement (September 1967).

(3) Klein, Alexander, Tsevi Karby, and Milos Polivka. "Properties of an Expansive Cement for Chemical Prestressing." Proceedings of the American Concrete Institute, Vol. 58, No. 1 (July 1961), pp. 59-82.

(4) Lossier, H. and A. Caquot. "Expanding Cements and Their Application - Self-Stressed Concrete." Review of Le Genie Civil 121, No. 8, by R. L. Bertin. Proceedings of the American Concrete Institute Journal, 41: 238-240 (1945).

(5) Mikhailov, V. V. "Stressing Cement and the Mechanism of Self-Stressing Concrete Regulation." Proceedings of the Fourth International Symposium on the Chemistry of Cement, Washington, D.C., Vol. II, 1960, pp. 927-955.

(6) Lin, T. Y. and A. Klein. "Chemical Prestressing of Concrete Elements Using Expanding Cements." Proceedings of the American Concrete Institute, Vol. 60, No. 9 (September 1963), pp. 1187-1218.

(7) Gustaferro, A. H., N. Greening, and P. Klieger. "Expansive Concretes - Laboratory Tests of Freeze-Thaw and Surface Scaling Resistance." Journal of the Portland Cement Association, Vol. 8, No. 1, January 1966.

(8) Joseph, J. "Expansive Concrete." Progressive Architecture, Vol. 44, November 1963, pp. 178, 180, 182.

(9) Klein, A. and V. Bertero. "Effects of Curing Temperature and Creep Characteristics of Expansive Concretes." Proceedings of A.S.T.M., Vol. 63, 1963, pp. 1008-1025.

(10) Bertero, V. V. "Curing Effects on Expansion and Mechanical Behavior of Expansive Cement Concrete." Proceedings of the American Concrete Institute, Vol. 64, No. 2, February 1967, pp. 84-96.

(11) Li, Shu-t'ien. "Expansive Cement Concretes - A Review." American Concrete Institute Journal, Vol. 62, No. 6, Pt. 1, June 1965, pp. 689-706.

(12) "Expansive Concrete Opens Era of New Concepts." Chemical and Engineering News, Vol. 42, No. 32, August 10, 1964, pp. 38-39.

(13) Larson, T., et al. "A Critical Review of Literature Treating Methods of Identifying Aggregates Subjected to Destructive Volume Changes When Frozen in Concrete and a Proposed Program of Research." Highway Research Board Special Report 80, 1964.

(14) "Expansive Cement: A New Approach to Reduce Concrete Cracking." Architecture Record, Vol. 140, No. 3, September 1966, pp. 231-232.

(15) Whitehurst, E. A. Evaluation of Concrete Properties from Sonic Tests, American Concrete Institute, Detroit, Michigan and The Iowa State University Press, Ames, Iowa, 1966.

(16) Powers, T. C. "Measuring Young's Modulus of Elasticity by Means of Sonic Vibrations." Proceedings of A.S.T.M., Vol. 38, Part II, 1938, pp. 460-467.

(17) American Society for Testing and Materials, 1964 Book of A.S.T.M. Standards, 1964, pp. 140-144.

(18) Report on Cooperative Freezing and Thawing Tests of Concrete, Highway Research Board, Special Report No. 47, 1959.

(19) Cordon, W. A. Freezing and Thawing of Concrete-Mechanisms and Control, American Concrete Institute, Detroit, Michigan and The Iowa State University Press, Ames, Iowa, 1966.

(20) American Society for Testing and Materials, 1968 Book of A.S.T.M. Standards, 1968, pp. 89-91.

(21) American Society for Testing and Materials, 1968 Book of A.S.T.M. Standards, 1968, pp. 86-89.

(22) American Society for Testing and Materials, 1968 Book of A.S.T.M. Standards, 1968, pp. 172-178.

(23) American Society for Testing and Materials, 1968 Book of A.S.T.M. Standards, 1968, pp. 155-159.

(24) Pickett, Gerald. "Equations for Computing Elastic Constants from Flexural and Torsional Resonant Frequencies of Vibration of Prisms and Cubes." Proceedings of the American Society for Testing and Materials, Vol. 45, 1945, pp. 846-859.

(25) Woods, Hubert. "Observations on the Resistance of Concrete to Freezing and Thawing." Journal of the American Concrete Institute, Vol. 26, No. 4, December 1954, pp. 345-349.

(26) Axon, E. O., L. T. Murray, and R. M. Rucker. "Laboratory Freeze-Thaw Tests Vs. Outdoor Exposure Tests." A Summarization of Missouri State Highway Department Research Investigation 31-10C, January 1969.

(27) Legg, F. E. "Freeze-Thaw Durability of Michigan Concrete Coarse Aggregate." Highway Research Board Bulletin 143, January 1956, pp. 3-4.

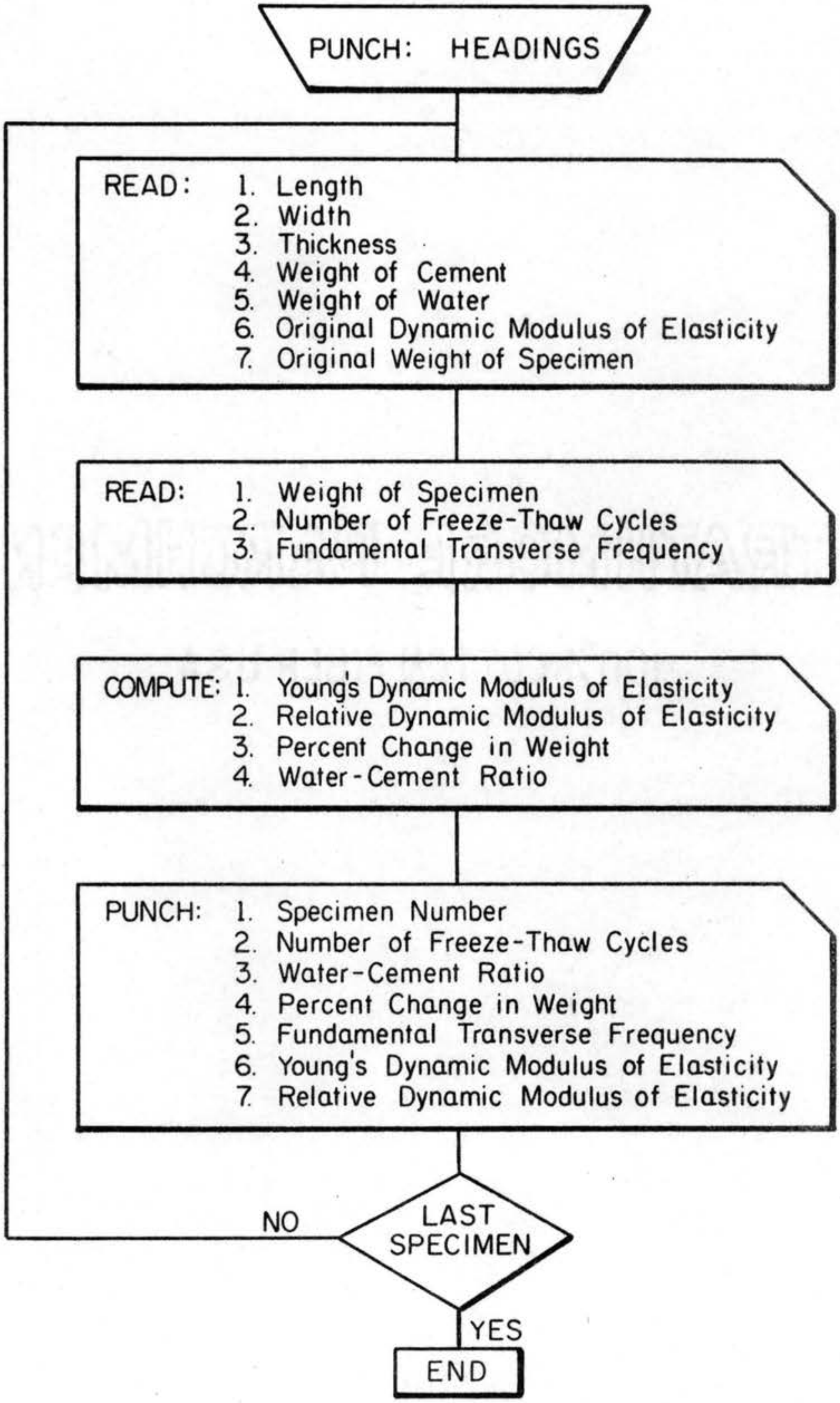
(28) Gordon, W. A. "Automatic Freezing and Thawing Equipment for a Small Laboratory." Highway Research Board Bulletin 259, January 1960.

(29) Cook, H. K. "Automatic Equipment for Rapid Freezing and Thawing of Concrete in Water." Highway Research Board Bulletin 259, January 1960, pp. 7-27.

APPENDIX

COMPUTER PROGRAM FOR DATA REDUCTION

FLOW CHART OF PROGRAM



PROGRAM LISTING

```
C  THIS PROGRAM IS USED TO COMPUTE THE WATER-CEMENT RATIO,PERCENT
C  CHANGE IN WEIGHT,YOUNG'S DYNAMIC MODULUS OF ELASTICITY,AND THE
C  RELATIVE DYNAMIC MODULUS OF ELASTICITY OF EACH SPECIMEN.
      N=0
      K=3
      POIRA=0.167
      PUNCH 60
60    FORMAT (25X,37HFREEZE-THAW DETERIORATION OF CONCRETE,//)
      PUNCH 90
90    FORMAT (2X,8HSPECIMEN,3X,6HFREEZE,3X,5HWATER,3X,7HPERCENT,2X,10HTR
     1ANSVERSE,4X,7HDYNAMIC,7X,7HPERCENT)
      PUNCH 70
70    FORMAT (3X,6HNUMBER,5X,4HTHAW,4X,6HCEMENT,3X,7HOF ORG.,2X,9HFREQUE
     1NCY,2X,10HMODULUS OF,6X,7HOF ORG.)
      PUNCH 80
80    FORMAT (13X,6HCYCLES,3X,5HRATIO,4X,6HWEIGHT,3X,9HCYC./SEC.,2X,10HE
     1LASTICITY,6X,7HMODULUS)
      PUNCH 82
82    FORMAT (20X,9HGAL./SAK.,/)
50    READ 10,ENGTH,WIDTH,THICK,WATECE,WATEWA,ORDYNE,ORWATE
10    FORMAT (5F10.2,E14.8,F5.2)
      READ 20,WATE,ITCYC,TRFRE
20    FORMAT (F10.2,I10,F10.0)
      CDIVL=(THICK)/((3.464)*(ENGTH))
      RAGYR=THICK/3.464
      HA=1.0+((81.79)*((RAGYR/ENGTH)**2.0))
      GA=((1314.0)*((RAGYR/ENGTH)**4.0))
      EA=(1.0+(81.09)*((RAGYR/ENGTH)**2.0))
      DA=GA/EA
      FA=(125.0)*((RAGYR/ENGTH)**4.0)
      TPRIME=HA-DA-FA
      TSNUM=1.0+((0.26*POIRA)+(3.22*((POIRA)**2.0)))*(CDIVL)
      TSDNOM=1.0+(0.1328*CDIVL)
      T=((TSNUM)/(TSDNOM))*TPRIME
      C=(0.00245*(ENGTH**3.0)/((WIDTH)*(THICK**3.0)))*T
      DYNE=C*WATE*(TRFRE**2.0)
      PERCEN=(DYNE/ORDYNE)*100.0
      PERWA=((WATE-ORWATE)/ORWATE)*100.0
      GALWA=WATEWA/8.33
      SACKS=WATECE/94.0
      WACERA=GALWA/SACKS
      N=N+1
      IF(N-K)30,30,34
34    K=K+3
      PUNCH 100
100   FORMAT (80X)
30    PUNCH 40,N,ITCYC,WACERA,PERWA,TRFRE,DYNE,PERCEN
40    FORMAT (3X,2HB-,I2,I10,F10.2,F10.4,F10.2,E15.7,3X,F7.2)
      GO TO 50
      END
```

VITA

Paul William Erdner

Candidate for the Degree of

Master of Science

Thesis: ACCELERATED FREEZE-THAW TESTS OF EXPANSIVE CONCRETE

Major Field: Civil Engineering

Biographical:

Personal Data: Born in El Reno, Oklahoma, November 29, 1944, the son of Mr. and Mrs. Paul A. Erdner

Education: Graduated from Kingfisher High School, Kingfisher, Oklahoma, in May, 1963; attended Central State College, Edmond, Oklahoma, 1964 and 1965; received Bachelor of Science degree from Oklahoma State University in 1968, with a major in Civil Engineering; completed requirements for the Master of Science degree at Oklahoma State University in May, 1969.

Professional Experience: Engineering Trainee, Mobil Oil Corporation, Summer 1966 and 1967; Research Assistant, School of Civil Engineering, Oklahoma State University, 1968.