Evaluation of Concrete and Related Materials for Desalination Plants
Second Annual Progress Report

United States Department of the Interior

Office of Saline Water • Research and Development Progress Report No. 390

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Evaluation of Concrete and Related Materials for Desalination Plants Second Annual Progress Report

By United States Department of the Interior, Bureau of Reclamation, Denver, Colorado, for the Office of Saline Water, J.A. Hunter, Director; Everett N. Sieder, Chief, Distillation Division; Paul G. Tomalin, Project Engineer

USBR-OSW Agreement No. 14-01-0001-544
Work Orders No. 1 and 6

UNITED STATES DEPARTMENT OF THE INTERIOR • Stewart L. Udall, Secretary
Max N. Edwards, Assistant Secretary for Water Pollution Control

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The Department works to assure the wisest choice in managing all our resources so each will make its full contribution to a better United States—now and in the future.

FOREWORD

This is one of a continuing series of reports designed to present accounts of progress in saline water conversion and the economics of its application. Such data are expected to contribute to the long-range development of economical processes applicable to low-cost demineralization of sea and other saline water.

Except for minor editing, the data herein are as contained in a report submitted by the contractor. The data and conclusions given in the report are essentially those of the contractor and are not necessarily endorsed by the Department of the Interior.

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ABSTRACT

Laboratory tests were conducted to investigate concrete under the temperature, pressure, salinity, and structural stress conditions in a sea water distillation plant. The studies consisted of concretes containing both natural and limestone coarse aggregates exposed to variety of temperature-pressure-salinity conditions; panel and module structural studies; evaluation of coatings, sealants, and polymeric products; corrosion studies on reinforcing steel; concrete microstructural investigaions; and stable binder investigations. Significant conclusions are: (1) Untreated portland cement concrete will not withstand leaching by distilled water. (2) The high quality concrete tested has not been detrimentally affected by 2-yr exposure to 100 deg F flowing synthetic sea water. (3) Some coating materials are showing promise for use on concrete and steel to 250 deg F. (4) Degassing and drying of concrete or mortar surfaces prior to coating applications are necessary to prevent disruptions from entrapped gases and water. (5) No significant corrosion of steel reinforcing bars or pretensioning rods has occurred. Several stable binders exposed to sea water at 250 deg F appear promising.

DESCRIPTORS-- concrete/ *concrete technology/ concrete testing/ portland cement/ sea water/ pozzolans/ prestressed concrete/ aggregates/ mastics/ reinforced concrete/ chemical analysis/ *protective coatings/ *binders/ *demineralization/ saline water/ *performance tests/ reinforcing steel/ deterioration/ admixtures/ *high temperature research
IDENTIFIERS-- brines/ concrete properties/ distilled water

SECTION I

INTRODUCTION

Improved feasibility of large-scale desalting plants is in part dependent upon the development of improved designs and the use of construction materials which can lead to substantial cost reductions. The extensive use of concrete and related materials in this type of construction has been proposed as a cost-saving item. Very little experience is available concerning the use of this material and the accessory materials necessary for use with concrete under the environmental conditions that will be encountered in a sea water distillation plant.

On August 21, 1964, and May 7, 1965, the Bureau of Reclamation entered into agreements with the Office of Saline Water to perform the necessary research to fill the present gaps that exist in our knowledge concerning the use of concrete under the conditions of temperature, pressure, salinity, and structural stresses that will exist in an evaporator-type desalting plant. The latter agreement has subsequently been amended several times.

Following are excerpts taken from the two agreements regarding the objectives of the work and the scope:

(A) Agreement No. 14-01-0001-454 (August 21, 1964)

Objectives:

(1) Determine through fabrication and testing of suitable specimens the potential performance of portland cement concrete and degree of corrosive attack when subjected to the operating conditions of interest. Such specimens shall consist of:

(a) Unreinforced concrete;

(b) Conventionally reinforced unfractured and slightly fractured concrete; and

(c) Prestressed steel reinforced unfractured and slightly fractured concrete.

(2) Examine through literature search, analysis of available data, or experimentation, methods for improving performance and corrosion resistance of concrete. These may be variations in portland cement and/or concrete formulations, use of selected protective coatings, and other methods which may be suggested.

(3) Determine through searches of pertinent literature and performance of design studies the shape of orifices between chambers of the flash evaporation structure required to eliminate possible cavitation in the orifices.

(4) Determine through literature search, analysis of available data, and experimentation, methods for sealing portland cement concrete against internal and external pressures superimposed upon thermal stresses including:

(a) Permeability and cracks;

(b) Piping penetrations in concrete;

(c) Fluid flows between zones of different pressures;

(d) Expansion joints in concrete; and

(e) Closures for access ports.

(B) Agreement No. 14-01-0001-544, Work Order No. 1 (May 7, 1965)

Objectives:

(1) To provide data relating to the use of concrete and accessory materials which may be helpful in evaluating the conceptual design studies now under contract by the OSW.

(2) To provide design data relating to the use of concrete and accessory materials for use in the first large prototype plants which will be built in accordance with the report to the President on the Program for Advancing Desalting Technology with construction planned to start in 1967 to be completed by 1970.

Scope:

The work shall consist of five related studies, identified as follows:

Part I. Evaluation of concretes containing both natural and limestone coarse aggregate under a variety of temperature-pressure-brine conditions.

Part II. Wall section model studies. (Subsequently changed to panel and module structural studies.)

Part III. Selection and evaluation of accessory coatings, sealants, and polymeric products.

Part IV. Concrete reinforcement materials.

Part V. Concrete microstructural investigations.

(C) Agreement No. 14-01-0001-544, Work Order No. 6 (November 9, 1966)

Objectives:

Determine combinations of portland cements, pozzolans, slags, by-product ferric oxide, anhydride, etc., which when cured (and used) over a temperature range of practical interest in desalination plants will form a binder of concrete of highest resistance to saline and other water attack, and possess required physical properties. The results will indicate the potential application of concrete for all sizes of plants to be used for purifying any raw water, except that of pH of about 7.0 and lower.

The program has an intermediate range payoff. The results of tests would screen various potential binders and accelerate considerably the evaluation only of promising concretes for stability for intended environments of desalination plants if required. This is a fundamental program requiring no prior information to be initiated. Facilities such as closed circuits at different temperatures are already available for the final evaluation of the concretes made with promising binders.

Scope:

The basic approach is the systematic synthesis of various binders of concrete which will form at temperatures anticipated in desalination plants. At the lower temperatures a complex high-lime calcium silicate, containing Al_2O_3, Fe_2O_3, SO_3, etc., as substituents in the atomic lattice, is formed. Properties relate to kinds and amounts of substituents, with the Fe_2O_3 probably governing stability to saline water attack. At higher temperatures from about 260° F and higher, a low-lime calcium silicate (tobermorite) forms and properties of it also relate to the substiuents. These potential binders, as small paste samples, are first being examined as to phase content, by X-ray diffraction, differential thermal analysis and other means. Permeability of the paste controlled by initial water content will be another parameter in overall evaluations. The binders of the more promising compositions, free of undesirable phases as alpha dicalcium silicate hydrate, the hydrogarnets, unreacted raw materials, etc., will be tested as pastes for strength using 1-inch cubes. These screenings operations will eliminate undesirable compositions. The promising products will then be tested as mortars and/or concretes for resistance to saline water attack, and other required properties. Also surface coatings such as Fe_2O_3, certain phosphates and other insoluble materials will be applied to selected compositions and tested for resistance to distilled water.

This annual report is a comprehensive coverage of the progress of all work done to May 15, 1967, under the aforementioned agreements. In order to facilitate updating, this report has been divided into two general sections, Sections I and II, five independent sections corresponding to the five parts in Work Order No. 1 of the second agreement, and one Section VIII covering Work Order No. 6. Section I is a general introduction to the entire program. Section II contains a number of selected general conclusions covering the entire five parts of Work Order No. 1 as discussed in Sections III through VII and Work Order No. 6 as discussed in Section VIII. More detailed and possibly less significant conclusions are found in each of the independent sections dealing with the five parts of Work Orders No. 1 and 6.

SECTION II

GENERAL CONCLUSIONS

General conclusions and those with seemingly the most importance, based on results of tests performed on the high-quality concretes used in this investigation, are included in the following compilation. Complete and more detailed conclusions can be found within the various sections of this report.

Concrete Studies

1. Conventional portland cement concrete uncoated and untreated will not withstand the leaching effects of warm- to hot-flowing mineral-free distilled water.

2. Concrete has not been detrimentally affected by 2 years' exposure to the 100° F flowing synthetic sea water brine; therefore, it appears that it would be suitable for use under these conditions for an extended period of time.

3. The interior concrete exposed to the 290° F synthetic sea water brine for 18 months is sound; however, the exterior concrete was moderately deteriorated to a depth of 5 to 8 mm, and shows chemical alteration, extensive microfractures, and some separation by large cracks. The life expectancy of this concrete under the subject environment cannot be predicted at this time, but it is known that some sacrificial concrete (amount not determined at this time), in addition to that which would normally cover the reinforcing steel beneath the exposed surfaces will be required.

4. In general, concretes containing high-quality natural aggregate and crushed limestone aggregate have not been detrimentally affected by 180 days' exposure to the 203°, 225°, or 250° F synthetic sea water brine, although some minor surface alteration has occurred on all of the specimens. Because of the early age, no definite prediction can be made as to service life of the concrete under these environments.

5. Limestone aggregate in the concrete where directly exposed to the hot brine as on a cut surface underwent dissolution. Such a phenomenon may preclude the use of limestone aggregate in concrete of structures subjected to such exposures.

6. No significant corrosion of the steel reinforcing bars or the steel pretensioning rods has occurred as a result of exposure to flowing synthetic sea water brine regardless of temperature or length of exposure.

7. Correlation between the actual test results and the theoretical values is accurate enough that a time-dependent-temperature distribution of a reinforced concrete shell for use as an evaporator vessel can be calculated using the theoretical-numerical-analysis method.

Coatings and Related Materials Studies

8. Coating materials listed in Table V-1 are not suitable for extended 350° F wet exposure.

9. EPT and butyl-rubber materials generally appear to date to be least affected by any exposure condition; the exceptions to this point out the importance of formulation within families of elastomers. (April 1967)

10. Neoprene rubber coatings as a class are showing good performance to date on both steel and concrete at all test temperatures through 290° F. Elastic properties are showing little change with increased exposure time. One type of neoprene joint sealant is showing good promise for extended service at 290° F. (April 1967)

11. The phenolic and epoxy modified phenolics as a class are showing excellent performance on steel at all test temperatures up through 290° F. These coatings are showing good performance on concrete with just occasional occurrence of small cracks amenable to repair through 290° F temperatures. (April 1967)

12. Degassing and drying of concrete or mortar surfaces prior to coating applications will be necessary to prevent coating disruptions from entrapped gases and water as the temperature of coated surfaces increases above the application temperature.

Stable Binder Studies

13. Results of screening tests of many combinations of cementitious materials with portland cement as base disclosed several compositions of satisfactory strength at 210° to 290° F. Compositions were selected in which the known, and one newly discovered, deleterious compounds would not form. Pastes of these compositions in concentrated sea water at 250° F show no signs of being attacked (up to 48 days at last examination).

14. Tests under way on lumnite cement pastes and mortars show, contrary to previous belief on harmful effects of elevated temperature, that the mortar may be a satisfactory product to use at elevated temperatures.

SECTION III

PART I--EVALUATION OF CONCRETES

III. 1. Introduction

The research effort of the Office of Saline Water, hereafter referred to as OSW, has shown the need to investigate the suitability of concretes as a material of construction for saline water desalination plants. Studies indicate that the use of concrete may have a significant economic advantage over the use of other construction materials. On August 21, 1964, the Bureau of Reclamation entered into an agreement with the OSW to investigate, in part, the corrosion resistance of concrete subjected to distilled water and brine at temperatures and pressures from 100° F and 1.0 psia to 350° F and 134 psia. The maximum brine temperatures and pressures were subsequently reduced to 290° F and 58 psia, respectively, by memorandum from OSW. On May 7, 1965, the Bureau of Reclamation entered into a second agreement with OSW to further investigate, among other things, concretes containing two types of aggregate and under three additional environmental conditions.

Concrete fabricated for testing under the original agreement has now been subjected to the 100° and 290° F brine environments for 24 and 18 months, respectively. These tests are continuing. Similar concrete was moist cured for various ages prior to being subject to the two distilled water environments for 126 days. These tests have subsequently been terminated; however, a small concrete dome is being subjected to 203° F vapor and distilled water. Concretes fabricated for testing under the second agreement mentioned have now been subjected to the 203°, 225°, and 250° F brines for 180 days.

III. 2. Test Specimens and Materials

III. 2. 1. Aggregates

Due to the limited nature of the initial 1964 program only a single concrete containing high-quality natural aggregate was used. However, to facilitate determination of incipient deterioration by X-ray diffraction, differential thermal analysis and petrographic examination a few specimens were cast using a concrete containing crushed quartz coarse aggregate. In early 1966, two new concrete series were placed under test, one containing crushed limestone coarse aggregate and the other, natural aggregate similar to that previously used. In all cases, the same fine natural aggregate was used. Following is a petrographic description of each of the three aggregates:

Sample No.	Source
M-5036 (P-7682)	Plus No. 4 gravel from the Lawrence Canyon pit, sec 23, T 11 S, R 5 W, SBB&M. H. W. Rohl Company, a subsidiary of Concrete and Service Company, Oceanside, California. Minus No. 4 sand from the Palo Plant pit, sec 23, 4, 25, 27, T 9 S, R 2 W, SBB&M. H. W. Rohl Company.

The 3/4-inch size fraction rock consists of about 82.4 percent amphibole schist, amphibolite, and minor granitic schist, 10.7 percent granite-diorite series rocks, and 6.9 percent quartz pebbles, quartzite and metasandstone, Table III-1. The 3/8-inch size fraction rock consists of about 75.2 percent amphibole schist, amphibolite and minor granitic schist, 14.3 percent granite-diorite series rocks, and 10.5 percent quartz pebbles, quartzite and metasandstone. Of the total rock constituency in the 3/4-inch size fraction about 78.5 percent is hard, dense and fine to medium grained and is considered satisfactory for use in concrete, while 18.6 percent is hard to firm, moderately weathered, absorptive, fractured and considered only fair. The other 2.9 percent is firm to crumbly or friable, highly absorptive, deeply weathered, moderate to highly fractured and is termed poor, Table III-2. The percentages in the 3/8-inch fraction size for the aforementioned divisions are 69.5, 26.7, and 3.8 percent, respectively.

The gravel is mostly angular and subangular with about 13 percent of the particles in the 3/4-inch size being flat. About 1 percent of the particles in the 3/4-inch size contain a calcium carbonate coating. The No. 4 size material appears similar to the

Table III-1

PETROGRAPHIC EXAMINATION OF COARSE AGGREGATE
Sample No. M-5036
Corrosion of Concrete Investigation for OSW

Rock types	Description of rock types	Physical quality	Percentage by particle count	
			3/4 inch	3/8 inch
Amphibole schist, amphibolite, and minor granitic schist	Hard, dense, fine- to medium-grained, schistose, somewhat weathered and fractured.	Satisfactory	65.8	51.0
	Hard to firm, moderately weathered, absorptive, and fractured with fracture surfaces generally being weathered.	Fair	13.7	21.7
	Firm to crumbly, highly absorptive, deeply weathered, moderate to highly fractured, argillaceous, tends to become soft and crumbly in water.	Poor	2.9	2.5
			82.4	75.2
Granite-diorite series rocks	Hard, dense, fine- to medium-grained, somewhat weathered and fractured.	Satisfactory	7.3	10.5
	Hard to firm, moderately absorptive, weathered, and fractured, somewhat argillaceous.	Fair	3.4	2.5
	Firm to friable, highly absorptive, deeply weathered, moderately fractured, argillaceous, tends to become somewhat soft in water.	Poor	-	1.3
			10.7	14.3
Quartz pebbles, quartzite and metasandstone	Hard, dense, fine-grained quartzite and metasandstone, minor amount of schistose quartz, somewhat weathered and fractured.	Satisfactory	5.4	8.0
	Hard to firm, moderately weathered and fractured, somewhat argillaceous.	Fair	1.5	2.5
			6.9	10.5

Table III-2

SUMMARY OF QUALITY OF COARSE NATURAL AGGREGATE
Sample No. M-5036
Corrosion of Concrete Investigation for OSW

		Percentage by particle count	
		3/4 inch	3/8 inch
Physical quality	Satisfactory	78.5	69.5
	Fair	18.6	26.7
	Poor	2.9	3.8
Chemical quality	Alkali-reactive	--	--

3/8-inch material in both composition and physical and chemical quality; however, there is an increase in monomineralic granules of quartz and feldspar and also an increase in granite-type particles. Presented in Table III-3 is a physical evaluation of the aggregate.

The coarse sand is mostly angular in shape. There is a very minor amount of calcium carbonate coating on only a few of the examined particles. The fine sand is mostly angular and is composed of decreasing amounts of the fine-grained rock types found in the coarse sand, and increasing amounts of monomineralic grains of quartz, feldspar, mica, amphibole, magnetite, clay (montmorillonite type), and a few miscellaneous minerals, Table III-6. About 2 percent of the fine sand is unsound and no minerals known to be deleteriously reactive were observed. That portion of the sand removed by washing, which was less than 1 percent, by weight, consisted of the same material as in the fine sand but with an increased amount of clay. The test for salts show a slight indication of the presence of chloride. The gradation of sand is less than desirable due to the high percentage of coarse material present. Gravel M-5036 is petrographically of good quality for use as concrete aggregate; however, Sand M-5036 is petrographically only fair for use as a concrete aggregate. Presented in Tables III-4 and III-5 are physical evaluations of this natural sand.

Sample No.	Source
M-5037 (P-7683)	Crushed quartz from the M & S Company, Inc., at Sedalia, Colorado. This material is designated as Rampart Range No. 1 grade.

This aggregate is composed of hard, dense, crushed, angular quartz fragments, containing well-healed fractures with some of the fracture surfaces having been stained. Some particle surfaces contain a thin layer of clay and some staining.

Sample No.	Source
M-5192	Crushed limestone coarse aggregate from limerock deposit of Guernsey Stone Company quarry at Guernsey, Wyoming. Quarry is located within S1/2 sec 25, T 27 N, R 66 W, sixth principal meridian.

The crushed-rock particles are angular, with some having rounded edges, and tend to become more thin, flat, elongate, and/or chiplike in the smaller size fractions.

The 3/4-inch crushed rock consists of about 91.4 percent limestone, 2.9 percent dolomitic limestone, and 5.7 percent chert nodules and cherty particles. The limestone is hard, dense, light brown to buff, and finely crystalline. A few of the particles have a moderately weathered surface. Tight fractures are present in many of the particles, but are generally well healed by calcite and in some instances by reddish brown siderite. Trace amounts of clay were observed in a few of the particles.

The dolomitic limestone is hard, dense, fine grained, crystalline, and almost white to pinkish in color. A few tight fractures are present in some of the particles and are well cemented with calcite and in minor instances with quartz.

The chert nodules are hard, dense, and dark gray. The cherty particles are embedded in buff colored limestone. The surfaces of a few of the particles are moderately weathered and a few particles are somewhat fractured.

An acid insoluble determination was made on a few small chips of rock selected to be representative of the aggregate sample. This portion of the rock was composed of about 91 percent calcite (including minor dolomite) and 9 percent acid insoluble constituents which consisted almost entirely of quartz with only a trace of orthoclase feldspar, as determined by optical examination and X-ray analysis. No clay minerals were detected in the residue. The quartz consisted almost entirely of microcrystalline quartz. If there was any chert present, it was completely masked by the quartz.

An X-ray analysis was also performed on the original material that was used for the acid insoluble analysis. This material was composed of about 93 percent calcite,

Table III-3
AGGREGATE QUALITY EVALUATION
Corrosion of Concrete Investigation for OSW

STATE California		AGGREGATE	REPORT NO. CM-	
LAT. 33° N	LONG. 117° W		LAB SAMPLE NO. M- 5036	REG.
TYPE OF MATERIAL Gravel			DATE REC'D. 9-9-64	

NAME OF DEPOSIT Lawrence Canyon Plant

LOCATION near Oceanside, California in

SEC. 23	T 11S	R 5 W	MERIDIAN San Bernardino	

OWNERSHIP H. W. Rohl Company

VOLUME	OVERBURDEN

FEATURE

PROJECT

REMARKS *A subsidiary of Concrete Service Company, Oceanside, California

GRADING (DES. 4,5,6)(CUM. % RETAINED)					
SIEVE	PIT RUN	3"-1½"	1½"-¾"	¾"-#4	FINE AGG.
6 IN.	1/				
3½ IN.					
3 IN.					
2½ IN.					
1¾ IN.					
1½ IN.	0				
1¼ IN.	-				
⅞ IN.	-				
¾ IN.	43				
⅝ IN.	-				
⅜ IN.	80				
5/16 IN.	-				
NO. 4	100				
NO. 5					
NO. 8					
NO. 16					
NO. 30					
NO. 50					
NO. 100					
PAN					
F.M.	7.23				
% SAND					

TEST RESULTS	6"-3"	3"-1½"	1½"-¾"	¾"-⅜"	⅜"-#4	FINE AGG.	WASHED FINE AGG.
SP. GR., S.S.D. (DES. 9,10)			2.94	2.93	2.87		
ABSORPTION, PERCENT (DES. 9,10)			0.7	0.9	1.2		
ORGANIC IMPURITIES, COLOR (DES. 14)	—	—	—	—	—		
PERCENT SILT (DES. 16)							
PERCENT LIGHTER THAN SP. GR.							
CLAY LUMP, % (DES. 13)							
SAND EQUIVALENT	—	—	—	—	—		
NA_2SO_4 LOSS, 5 CYC WGTD % LOSS (DES. 19)	—	—				—	

L.A. ABRASION (DES. 21) GRADING	"A"	"B"	"C"	"D"
% LOSS, 100 REV.	5.4			
% LOSS, 500 REV.	28.2			

FREEZING AND THAWING DATA								
CONCRETE							RIPRAP	
W/C RATIO	SLUMP INCHES	% AIR METER	H_2O LBS/YD^3	28-DAY STR. 3"x6" CYL.	WGT. LOSS, %	CYCLES	WGT. LOSS, % 3 INCH CUBE	CYCLES

ALKALI-AGGREGATE REACTIVITY DATA								
MATERIALS	SAND				GRAVEL			
CEMENT NO.								
SODA EQUIVALENT								
TEST AGG. %	100	100	50	25	100	100	50	25
EXP % - 6 MO.								
EXP % - 12 MO.								

1/Grading on sample as received in Laboratory.

Table III-4
AGGREGATE QUALITY EVALUATION
Corrosion of Concrete Investigation for OSW

STATE California		AGGREGATE	REPORT NO. CM-
LAT. 33° N	LONG. 117° W		LAB SAMPLE NO. M-5036A · REG. 3
TYPE OF MATERIAL Sand			DATE REC'D. 9-9-64
NAME OF DEPOSIT Pala			
LOCATION At Pala, California in			
Sec 23, 24,25 & SEC. 26 & 27	T 9 S	R 2 W	MERIDIAN San Bernardino
OWNERSHIP H. W. Rohl Company*			
VOLUME		OVERBURDEN	
FEATURE			
PROJECT			
REMARKS *A subsidiary of Concrete Service Co., Oceanside, California			

GRADING (DES. 4,5,6)(CUM. % RETAINED)

SIEVE	PIT RUN	3"-1½"	1½"-¾"	¾"-#4	FINE AGG.
6 IN.					
3½ IN.					
3 IN.					
2½ IN.					
1¾ IN.					
1½ IN.					
1¼ IN.					
⅞ IN.					
¾ IN.					
⅝ IN.					
⅜ IN.					
5/16 IN.					
NO. 4					0
NO. 5					-
NO. 8					18
NO. 16					35
NO. 30					54
NO. 50					71
NO. 100					89
PAN					100
F.M.					2.67
% SAND					

TEST RESULTS	6"-3"	3"-1½"	1½"-¾"	¾"-⅜"	⅜"-#4	FINE AGG.	WASHED FINE AGG.
SP. GR., S.S.D. (DES. 9,10)						2.75	2.76
ABSORPTION, PERCENT (DES. 9,10)						1.1	1.1
ORGANIC IMPURITIES, COLOR (DES. 14)	—	—	—	—	—	Clear	
PERCENT SILT (DES. 16)						8.0	
PERCENT LIGHTER THAN SP. GR.							
CLAY LUMP, % (DES. 13)							
SAND EQUIVALENT	—	—	—	—	—	66	93
NA_2SO_4 LOSS, 5 CYC WGTD % LOSS (DES. 19)	—	—				—	6.6

L.A. ABRASION (DES. 21) GRADING	"A"	"B"	"C"	"D"	
% LOSS, 100 REV.					
% LOSS, 500 REV.					

FREEZING AND THAWING DATA

CONCRETE							RIPRAP	
W/C RATIO	SLUMP INCHES	% AIR METER	H_2O LBS/YD³	28-DAY STR. 3"x6" CYL.	WGT. LOSS, %	CYCLES	WGT. LOSS, % 3 INCH CUBE	CYCLES

ALKALI-AGGREGATE REACTIVITY DATA

MATERIALS	SAND				GRAVEL			
CEMENT NO.								
SODA EQUIVALENT								
TEST AGG. %	100	100	50	25	100	100	50	25
EXP % - 6 MO.								
EXP % - 12 MO.								

PETROGRAPHIC DESCRIPTION:

Table III-5
AGGREGATE QUALITY EVALUATION
Corrosion of Concrete Investigation for OSW

STATE California		AGGREGATE	REPORT NO. CM-	
LAT. 33° N	LONG. 117° W		LAB SAMPLE NO. M-5036A-1	REG. 3
TYPE OF MATERIAL Sand			DATE REC'D. 7-12-65	
NAME OF DEPOSIT Pala				
LOCATION At Pala, California in				
Sec 23,24,25 & SEC. 26 & 27	T 9 S	R 2 W	MERIDIAN San Bernardino	
OWNERSHIP H. W. Rohl Company*				
VOLUME		OVERBURDEN		
FEATURE				
PROJECT				
REMARKS *A subsidiary of Concrete Service Company, Oceanside, California				

GRADING (DES. 4,5,6)(CUM. % RETAINED)

SIEVE	PIT RUN	3"-1½"	1½"-¾"	¾"-#4	FINE AGG.
6 IN.					
3½ IN.					
3 IN.					
2½ IN.					
1¾ IN.					
1½ IN.					
1¼ IN.					
⅞ IN.					
¾ IN.					
⅝ IN.					
⅜ IN.					
5/16 IN.					
NO. 4					0
NO. 5					-
NO. 8					18
NO. 16					37
NO. 30					59
NO. 50					78
NO. 100					94
PAN					100
F.M.					2.86
% SAND					

TEST RESULTS	6"-3"	3"-1½"	1½"-¾"	¾"-⅜"	⅜"-#4	FINE AGG.	WASHED FINE AGG.
SP. GR., S.S.D. (DES. 9,10) 1/							2.74
ABSORPTION, PERCENT (DES. 9, 10) 1/							0.8
ORGANIC IMPURITIES, COLOR (DES. 14)	—	—	—	—	—		
PERCENT SILT (DES. 16)						2.7	0.5
PERCENT LIGHTER THAN SP. GR.							
CLAY LUMP, % (DES. 13)							
SAND EQUIVALENT	—	—	—	—	—	89	97
NA_2SO_4 LOSS, 5 CYC WGTD % LOSS(DES. 19)	—	—				—	

L.A. ABRASION (DES. 21) GRADING	"A"	"B"	"C"	"D"
% LOSS, 100 REV.				
% LOSS, 500 REV.				

FREEZING AND THAWING DATA

CONCRETE							RIPRAP	
W/C RATIO	SLUMP INCHES	% AIR METER	H_2O LBS/YD^3	28-DAY STR 3"x 6" CYL	WGT. LOSS, %	CYCLES	WGT. LOSS, % 3 INCH CUBE	CYCLES

ALKALI - AGGREGATE REACTIVITY DATA

MATERIALS	SAND				GRAVEL			
CEMENT NO.								
SODA EQUIVALENT								
TEST AGG. %	100	100	50	25	100	100	50	25
EXP % - 6 MO.								
EXP % - 12 MO.								

1/30-minute specific gravity and absorption.

Table III-6

PETROGRAPHIC EXAMINATION OF COARSE SAND
Sample No. M-5036A
Corrosion of Concrete Investigation for OSW

Rock and mineral types	Percentage by particle count		
	No. 8	No. 16	No. 30
Amphibole schist, amphibolite, and minor granitic schist	42	33	17
Granite-diorite series rocks	36	41	22
Quartz grains, quartzite, and metasandstone	20	21	46
Feldspar grains	2	5	14
Mica flakes	-	Tr	1
Percent unsound	4	3	2
Percent alkali reactive	0	0	0
Percent flat particles	5	-	-

2 percent dolomite, and 5 percent quartz with a trace of montmorillonite-type clay. As the amount and distribution of both the insoluble material and dolomite in the rock are variable, other analysis of individual rock samples would be expected to show some variation from figures from both the acid insoluble and X-ray analyses.

Crushed rock, Sample No. M-5192, is marginally deleteriously reactive with high-alkali cement due to the presence of chert nodules and cherty particles. These alkali-reactive particles constitute about 5.7, 5.0, and 5.0 percent of the 3/4- and 3/8-inch, and No. 4 size aggregate, respectively.

A review of the petrographic examinations of previously tested samples from the same source as Sample No. M-5192 shows this quarry to be quite variable in both the amount of limestone and dolomite present and the amount of acid insoluble constituents within these rocks. The rock types encountered in this quarry range from an almost pure limestone to cherty limestone to dolomitic limestone to cherty dolomite to calcitic dolomite to an almost pure dolomite. The acid insoluble constituents range from less than 1 to almost 10 percent.

As the rock from this quarry is quite variable, the results of this examination quantitatively apply only to the small sample examined, and are only indicative of the types of rock which might be expected in a shipment from the quarry. To insure having a homogeneous limestone aggregate from this quarry, rock should be selected by a geologist or petrographer familiar with the various rock types occurring in the quarry. However, crushed rock Sample No. M-5192 is petrographically of good quality for use as concrete aggregate. Presented in Table III-7 is a physical evaluation of this aggregate.

III. 2. 2. Cements

The cement used in the mixes from which the main test specimens were cast was portland, Type V, low alkali, sulfate resistant; Tables III-12 and III-13. This cement was chosen for use in the primary concrete because of its potential resistance to sulfate attack, as well as its resistance to alkali-aggregate reaction. This cement was obtained from the Riverside Cement Company, Oro Grande, California, and was so chosen because of its proximity to the proposed location of a large distillation plant. The first shipment of this cement exhibited false set and was returned to the manufacturer. The second shipment, M-4986, met all specifications and was accepted and used in specimens fabricated for test under the initial Agreement No. 14-01-0001-454.

Cement, Laboratory Sample No. M-5207, used in the concrete for testing under Agreement No. 14-01-0001-544, also exhibited false set; however, because of the time factor, and because of past experience which has shown that false set is not generally detrimental to the final concrete product,1/ this cement was accepted and used.

In order to obtain some comparative data on the effects that different cements may have on the resistance of concrete to the severe exposures anticipated, a few concrete specimens were cast from mixes containing Types I, II, III, and IV cements, Tables III-8, III-9, III-10, and III-11. Specimens containing Types I, II, and III cements are presently undergoing exposures in the 250°, 225°, and 203° F brine environments, respectively.

III. 2. 3. Pozzolan

Fly ash pozzolan from the Chicago Fly Ash Company, Chicago 4, Illinois, was used in all of the concrete prepared in this program. It was chosen because of its proven consistency and high quality when used on Bureau projects. Presented in Table III-14 are data pertaining to its physical and chemical properties and constituents.

III. 2. 4. Admixtures

A 5 percent solution of neutralized vinsol resin (NVX) air-entraining agent, and Placewel, an adipic acid-type water-reducing agent were used in all of the concrete utilized in this test program.

1/Refers to references on page 155.

Table III-7
AGGREGATE QUALITY EVALUATION
Corrosion of Concrete Investigation for OSW

STATE Wyoming		AGGREGATE ~~RIPRAP~~	REPORT NO. CM-
LAT.	LONG.		LAB SAMPLE NO. M- 5192 — REG. 7
TYPE OF MATERIAL Crushed limestone			DATE REC'D. 7-9-65
NAME OF DEPOSIT Guernsey			
LOCATION			
SEC. 25	T 27N	R 66W	MERIDIAN 6th PN
OWNERSHIP Guernsey Stone Company			
VOLUME		OVERBURDEN	
FEATURE			
PROJECT			
REMARKS			

GRADING (DES. 4, 5, 6) (CUM. % RETAINED)					
SIEVE	PIT RUN	3"-1½"	1½"-¾"	¾"-#4	FINE AGG.
6 IN.					
3½ IN.					
3 IN.					
2½ IN.					
1¾ IN.					
1½ IN.					
1¼ IN.					
⅞ IN.					
¾ IN.					
⅝ IN.					
⅜ IN.					
5/16 IN.					
NO. 4					
NO. 5					
NO. 8					
NO. 16					
NO. 30					
NO. 50					
NO. 100					
PAN					
F.M.					
% SAND					

TEST RESULTS 1/	6"-3"	3"-1½"	1½"-¾"	¾"-⅜"	⅜"-#4	FINE AGG.	WASHED FINE AGG.
SP. GR., S.S.D. (DES. 9,10)			2.69	2.70	2.69		
ABSORPTION, PERCENT (DES. 9, 10)			0.2	0.3	0.4		
ORGANIC IMPURITIES, COLOR (DES. 14)	—	—	—	—	—		
PERCENT SILT (DES. 16)							
PERCENT LIGHTER THAN SP. GR.							
CLAY LUMP, % (DES. 13)							
SAND EQUIVALENT	—	—	—	—	—		
Na_2SO_4 LOSS, 5 CYC WGTD % LOSS (DES. 19)	—	—	1.9			—	

L.A. ABRASION (DES. 21) GRADING	"A"	"B"	"C"	"D"
% LOSS, 100 REV.	7.3			
% LOSS, 500 REV.	32.8			

FREEZING AND THAWING DATA								
CONCRETE							RIPRAP	
W/C RATIO	SLUMP INCHES	% AIR METER	H_2O LBS/YD³	28-DAY STR. 3"x6" CYL	WGT. LOSS, %	CYCLES	WGT. LOSS, % 3 INCH CUBE	CYCLES

ALKALI-AGGREGATE REACTIVITY DATA								
MATERIALS	SAND				GRAVEL			
CEMENT NO.								
SODA EQUIVALENT								
TEST AGG. %	100	100	50	25	100	100	50	25
EXP % - 6 MO.								
EXP % - 12 MO.								

~~[illegible]~~

24 hour specific gravity and absorption

	3/4 to 1-1/2 inch	3/8 to 3/4 inch	No. 4 to 3/8 inch
Sp gr	2.70	2.70	2.69
Abs	0.3	0.3	0.4

1/30-minute specific gravity and absorption.

Table III-8

TYPE I CEMENT TEST DATA
Corrosion of Concrete Investigation for OSW

Project:			
Sample No.	M-4998	Specifications No.:	Federal SS-C-192d & g
Date Received:	7-23-64	Amount Received:	40 sacks
Type of Cement:	I1/	Mill:	Portland, Colo., Ideal Cement Co.
Letter of Transmittal: From:		Date:	

PHYSICAL PROPERTIES		CHEMICAL ANALYSIS	
Specific Gravity:	3.13	Silicon Dioxide (SiO_2)	20.84 Percent
Autoclave Expansion:	0.046 Percent	Aluminum Oxide (Al_2O_3)	6.11 Percent
Normal Consistency:	26.2 Percent	Ferric Oxide (Fe_2O_3)	2.25 Percent
Initial Set:	Hours 4 Minutes 00	Calcium Oxide (CaO)	65.25 Percent
Final Set:	Hours 6 Minutes 45	Magnesium Oxide (MgO)	1.33 Percent
Compressive Strength of 2- by 2-inch Cubes		Sulfur Trioxide (SO_3)	2.12 Percent
1-day	psi	Sodium Oxide (Na_2O)	0.08 Percent
3 days	2,183 psi	Potassium Oxide (K_2O)	0.46 Percent
7 days	3,708 psi	Total Alkalies, as Na_2O	0.38 Percent
28 days	5,408 psi	Loss on Ignition	1.09 Percent
Percent Air in Mortar:	8.8	Insoluble Residue	0.16 Percent
False Set:			
Initial, 1/4-min pen., mm	36	CHEMICAL COMPOUNDS	
5-min pen., mm	22/	Tricalcium Silicate (C_3S)	56.86 Percent
Remix 1/4-min pen., mm	37	Dicalcium Silicate (C_2S)	16.94 Percent
Remix 5-min pen., mm	37	Tetracalcium	
Percent Passing No. 325 Sieve:		Aluminoferrite (C_4AF)	6.84 Percent
Blaine Surface, cm^2/g:	3,177	Tricalcium Aluminate (C_3A)	12.39 Percent
Heat of Hydration:		Calcium Sulfate ($CaSO_4$)	3.60 Percent
3 days	cal/gram		
7 days	cal/gram		
28 days	cal/gram		
Other Tests:			

Remarks:

1/Shall meet low-alkali and false-set limitations.

2/This cement meets all specifications requirements except the false-set limitations.

Table III-9

TYPE II CEMENT TEST DATA
Corrosion of Concrete Investigation for OSW

Project:
Sample No. M-4700 Specifications No.: 1/
Date Received: Amount Received:
Type of Cement: 1/II Mill: 40 sacks each from 12 different mills
Letter of Transmittal: From:
Date:

PHYSICAL PROPERTIES		CHEMICAL ANALYSIS	
Specific Gravity:	3.17	Silicon Dioxide (SiO_2)	22.62 Percent
Autoclave Expansion:	0.023 Percent	Aluminum Oxide (Al_2O_3)	4.79 Percent
Normal Consistency:	25.2 Percent	Ferric Oxide (Fe_2O_3)	3.50 Percent
Initial Set: Hours	4 Minutes 20	Calcium Oxide (CaO)	62.62 Percent
Final Set: Hours	6 Minutes 45	Magnesium Oxide (MgO)	2.20 Percent
Compressive Strength of 2- by 2-inch Cubes		Sulfur Trioxide (SO_3)	1.97 Percent
1-day	psi	Sodium Oxide (Na_2O)	0.16 Percent
3 days	2,543 psi	Potassium Oxide (K_2O)	0.46 Percent
7 days	3,183 psi	Total Alkalies, as Na_2O	0.46 Percent
28 days	5,100 psi	Loss on Ignition	1.46 Percent
Percent Air in Mortar:	8.9	Insoluble Residue	0.24 Percent
False Set:			
Initial, 1/4-min pen., mm	34	CHEMICAL COMPOUNDS	
5-min pen., mm	25	Tricalcium Silicate (C_3S)	Percent
Remix 1/4-min pen., mm		Dicalcium Silicate (C_2S)	Percent
Remix 5-min pen., mm		Tetracalcium	
Percent Passing No. 325 Sieve:		Aluminoferrite (C_4AF)	Percent
Blaine Surface, cm^2/g:	3,394	Tricalcium Aluminate (C_3A)	Percent
Heat of Hydration:		Calcium Sulfate ($CaSO_4$)	Percent
3 days	cal/gram		
7 days	cal/gram		
28 days	cal/gram		
Other Tests:			

Remarks:

1/This cement meets the requirements of Federal Specifications SS-C-192d & g, and ASTM, Designation C150-63 for Type II cement, including the optional limits for low-alkali and false set.

TABLE III-10

TYPE III CEMENT TEST DATA
Corrosion of Concrete Investigation for OSW

Project:

Sample No. M-4997 Specifications No.: Federal SS-C-192d & g

Date Received: 7-23-64 Amount Received: 40 sacks

Type of Cement: 1/III Mill: Portland, Colo., Ideal Cement Co.

Letter of Transmittal: From:

Date:

PHYSICAL PROPERTIES			CHEMICAL ANALYSIS		
Specific Gravity:	3.21		Silicon Dioxide (SiO_2)	20.45	Percent
Autoclave Expansion:	0.022	Percent	Aluminum Oxide (Al_2O_3)	6.12	Percent
Normal Consistency:	26.6	Percent	Ferric Oxide (Fe_2O_3)	2.28	Percent
Initial Set: Hours 3 Minutes	00		Calcium Oxide (CaO)	64.66	Percent
Final Set: Hours 5 Minutes	30		Magnesium Oxide (MgO)	1.26	Percent
Compressive Strength of 2- by 2-inch Cubes			Sulfur Trioxide (SO_3)	2.76	Percent
1-day	1,971	psi	Sodium Oxide (Na_2O)	0.08	Percent
3 days	3,008	psi	Potassium Oxide (K_2O)	0.52	Percent
7 days	5,150	psi	Total Alkalies, as Na_2O	0.42	Percent
28 days		psi	Loss on Ignition	1.58	Percent
Percent Air in Mortar:	8.9		Insoluble Residue	0.26	Percent
False Set:					
Initial, 1/4-min pen., mm	35		CHEMICAL COMPOUNDS		
5-min pen., mm	2/0		Tricalcium Silicate (C_3S)		Percent
Remix 1/4-min pen., mm	36		Dicalcium Silicate (C_2S)		Percent
Remix 5-min pen., mm	35		Tetracalcium Aluminoferrite (C_4AF)		Percent
Percent Passing No. 325 Sieve:			Tricalcium Aluminate (C_3A)		Percent
Blaine Surface, cm^2/g:	4,524		Calcium Sulfate ($CaSO_4$)		Percent
Heat of Hydration:					
3 days		cal/gram			
7 days		cal/gram			
28 days		cal/gram			
Other Tests:					

Remarks:

1/Shall meet low alkali and false-set limitations.

2/This cement meets all specifications requirements except the false-set limitation.

Table III-11

TYPE IV CEMENT TEST DATA
Corrosion of Concrete Investigation for

Project:
Sample No. 2341
Date Received: 5-41
Type of Cement: IV
Letter of Transmittal: From:
Specifications No.: Federal SS-C-192d & g
Amount Received:
Mill: Southwestern Portland Cement Co.
Victorville, California
Date:

PHYSICAL PROPERTIES

Specific Gravity: 3.21
Autoclave Expansion: -0.002 Percent
Normal Consistency: 23 Percent
Initial Set: Hours 4 Minutes 20
Final Set: Hours 6 Minutes 30
Compressive Strength of 2- by 2-inch Cubes
1-day ____ psi
3 days ____ psi
7 days 1,217 psi
28 days 2,988 psi
Percent Air in Mortar:
False Set:
Initial, 1/4-min pen., mm
5-min pen., mm
Remix 1/4-min pen., mm
Remix 5-min pen., mm
Percent Passing No. 325 Sieve:
Blaine Surface, cm^2/g: 3,030
Heat of Hydration:
3 days ____ cal/gram
7 days ____ cal/gram
28 days ____ cal/gram
Other Tests:

CHEMICAL ANALYSIS

Silicon Dioxide (SiO_2) 26.12 Percent
Aluminum Oxide (Al_2O_3) 3.27 Percent
Ferric Oxide (Fe_2O_3) 2.78 Percent
Calcium Oxide (CaO) 62.72 Percent
Magnesium Oxide (MgO) 1.90 Percent
Sulfur Trioxide (SO_3) 1.54 Percent
Sodium Oxide (Na_2O) 0.16 Percent
Potassium Oxide (K_2O) 0.30 Percent
Total Alkalies, as Na_2O 0.36 Percent
Loss on Ignition 0.74 Percent
Insoluble Residue 0.27 Percent

CHEMICAL COMPOUNDS

Tricalcium Silicate (C_3S) 25.6 Percent
Dicalcium Silicate (C_2S) 55.6 Percent
Tetracalcium Aluminoferrite (C_4AF) 8.4 Percent
Tricalcium Aluminate (C_3A) 3.9 Percent
Calcium Sulfate ($CaSO_4$) 2.6 Percent

Remarks:

Physical test results are from retest made October 1965; original tests conducted in 1941.

Table III-12

TYPE V CEMENT TEST DATA
Corrosion of Concrete Investigation for OSW

Project:
Sample No. M-4986
Date Received: 6-30-64
Type of Cement: 1/V
Letter of Transmittal: From:

Specifications No.: Federal SS-C-192d & g
Amount Received: 4,000 lbs
Mill: Oro Grande, Riverside Cement Co.,
Oro Grande, California
Date:

PHYSICAL PROPERTIES

Specific Gravity: 3.16
Autoclave Expansion: 0.017 Percent
Normal Consistency: 25.0 Percent
Initial Set: Hours 3 Minutes 50
Final Set: Hours 6 Minutes 20
Compressive Strength of 2- by 2-inch Cubes
1-day psi
3 days 2,707 psi
7 days 3,670 psi
28 days 5,392 psi
Percent Air in Mortar: 6.6
False Set:
Initial, 1/4-min pen., mm 35
5-min pen., mm 34
Remix 1/4-min pen., mm
Remix 5-min pen., mm
Percent Passing No. 325 Sieve:
Blaine Surface, cm^2/g: 3,673
Heat of Hydration:
3 days cal/gram
7 days cal/gram
28 days cal/gram
Other Tests:

CHEMICAL ANALYSIS

Silicon Dioxide (SiO_2) 23.50 Percent
Aluminum Oxide (Al_2O_3) 3.62 Percent
Ferric Oxide (Fe_2O_3) 3.11 Percent
Calcium Oxide (CaO) 63.90 Percent
Magnesium Oxide (MgO) 2.06 Percent
Sulfur Trioxide (SO_3) 2.00 Percent
Sodium Oxide (Na_2O) 0.05 Percent
Potassium Oxide (K_2O) 0.63 Percent
Total Alkalies, as Na_2O 0.46 Percent
Loss on Ignition 1.12 Percent
Insoluble Residue 0.13 Percent

CHEMICAL COMPOUNDS

Tricalcium Silicate (C_3S) 47.00 Percent
Dicalcium Silicate (C_2S) 32.01 Percent
Tetracalcium Aluminoferrite (C_4AF) 9.45 Percent
Tricalcium Aluminate (C_3A) 4.34 Percent
Calcium Sulfate ($CaSO_4$) 3.40 Percent

Remarks:

1/Shall meet low-alkali and false-set limitations. This cement meets the specifications requirements.

Table III-13

TYPE V CEMENT TEST DATA
Corrosion of Concrete Investigation for OSW

Project:

Sample No. M-5207	**Specifications No.:** Federal SS-C-192g
Date Received: 7-21-65	**Amount Received:** 20,000 lbs
Type of Cement: 1/V	**Mill:** Oro Grande, Riverside Cement Co., Riverside, California
Letter of Transmittal: From:	**Date:**

PHYSICAL PROPERTIES			**CHEMICAL ANALYSIS**		
Specific Gravity:	3.19		Silicon Dioxide (SiO_2)	23.50	Percent
Autoclave Expansion:	0.032	Percent	Aluminum Oxide (Al_2O_3)	3.46	Percent
Normal Consistency:	24.6	Percent	Ferric Oxide (Fe_2O_3)	3.08	Percent
Initial Set: Hours	4	Minutes 05	Calcium Oxide (CaO)	64.06	Percent
Final Set: Hours	5	Minutes 55	Magnesium Oxide (MgO)	2.36	Percent
Compressive Strength of 2- by 2-inch Cubes			Sulfur Trioxide (SO_3)	2.01	Percent
1-day		psi	Sodium Oxide (Na_2O)	0.04	Percent
3 days	2,679	psi	Potassium Oxide (K_2O)	0.52	Percent
7 days	3,683	psi	Total Alkalies, as Na_2O	0.38=	Percent
28 days		psi	Loss on Ignition	1.10	Percent
Percent Air in Mortar:	7.8		Insoluble Residue	0.04	Percent
False Set:					
Initial, 1/4-min pen., mm	33		**CHEMICAL COMPOUNDS**		
5-min pen., mm	2/0		Tricalcium Silicate (C_3S)	48.74	Percent
Remix 1/4-min pen., mm	21		Dicalcium Silicate (C_2S)	30.69	Percent
Remix 5-min pen., mm	20		Tetracalcium Aluminoferrite (C_4AF)	9.36	Percent
Percent Passing No. 325 Sieve:			Tricalcium Aluminate (C_3A)	3.96	Percent
Blaine Surface, cm^2/g:	3,395		Calcium Sulfate ($CaSO_4$)	3.42	Percent
Heat of Hydration:					
3 days		cal/gram			
7 days		cal/gram			
28 days		cal/gram			
Other Tests:					

Remarks:

1/Requested Type V with low-alkali and false-set limitations.
2/Passes all specifications requirements except false-set limitation.

Table III-14
POZZOLAN TEST DATA
Corrosion of Concrete Investigation for OSW

Project:
Sample No. M-4962 Specifications No. USBR pozzolan dated 3-1-61
Date Received: 5-26-64 Amount Received: 1,000 lbs
Identification of material: Flyash, Chicago
Letter of Transmittal:
Date:
Name of Deposit: Location: Chicago
Ownership: Estimated Quantity:

PETROGRAPHIC ANALYSIS

Description of Material:

PHYSICAL PROPERTIES

Grinding Time: Hours Minutes
Fineness:
Air Permeability 7,828 cm^2/cubic centimeter
Percent Retained on No. 325 Sieve 11.0
Specific Gravity: 2.50
Water Requirement: 93 Percent of Control
Difference in Percent Drying Shrinkage (Test bar minus control bar): -0.010

Reduction of Reactive Expansion at 14 Days: 61 Percent

Compressive Strength of 2-inch Cubes		PSI	Percent of Control
Control Cement:	Sealed cured, 7 days		
	Sealed cured, 28 days	6,133	
	Sealed cured, 90 days		
With Pozzolan Replacement:	Sealed cured, 7 days		
	Sealed cured, 28 days	5,242	85
	Sealed cured, 90 days		

Compressive Strength of 2- by 4-inch Cylinders of Pozzolan-lime Mortar at 7 days, psi: 1,041

CHEMICAL ANALYSIS

Silicon dioxide (SiO_2) plus Aluminum Oxide (Al_2O_3) plus Ferric Oxide (Fe_2O_3) 81.56 Percent
Magnesium oxide (MgO) 0.83 Percent
Sulfur trioxide (SO_3) 2.03 Percent
Loss on ignition 1.03 Percent
Moisture content 0.03 Percent
Exchangeable alkalies as Na_2O 1.23 Percent

Remarks: Free CaO 2.83 Percent
Total CaO 9.52 Percent

This pozzolan meets the requirements of USBR Specifications for pozzolan, dated 3-1-61.

III. 2. 5. Concrete Mix Data

At the outset of this investigative program time was at a premium and as a result only one concrete was chosen for initial testing and evaluation. This one concrete contained, in addition to one of the better natural concrete aggregates (1-1/2-inch maximum size) from southern California, Table III-1, constituents combined in proportions which we judged would result in a concrete of high quality and potentially of high sulfate resistance. These constituents were (1) cement, Type V, low-alkali, selected to resist potential internal chemical reaction attending the combination of aggregate having detrimental amounts of reactive materials and the alkalies (Na_2O and K_2O) in cements; and the aggressive attack on the hardened concrete by soluble sulfates in the ground and/or ground water; (2) air-entraining agent, neutralized vinsol resin (NVX) to provide an air-void system to improve workability, durability, and impermeability of the concrete; and (3) a water-reducing agent, Placewel, to provide further benefits similar to those accruing to the use of NVX and also an increase in compression strength with a possible decrease in cement content. The proportions for 1 cubic yard of concrete are presented in Table III-15 under Mix No. 1.

At the same time that the aforementioned mix was prepared a small companion mix, Mix No. 2, Table III-15, was also made using identical constituents except for the coarse aggregate which was quartz, Laboratory Sample No. M-5037. Specimens cast from this mix were used primarily for petrographic investigations.

Subsequently six additional concrete mixes were made; two large and four small. From the two large mixes; one containing 1-1/2-inch maximum size natural aggregate and comparable to Mix No. 1, Table III-15, and the other containing 1-1/2-inch maximum size crushed limestone coarse aggregate and natural fine aggregate, Mix No. 3, Table III-15, the main test specimens were cast for the new tests as outlined under the section entitled "Test Environments." The four small mixes all utilized 1-1/2-inch maximum size natural aggregate, Laboratory Sample No. M-5036-1 but each contained a different type of cement; namely, Types I, II, III, and IV, Mixes No. 4, 5, 6, 7, respectively, Table III-15. All of the concrete mixes used in this program contained NVX air-entraining agent, Placewel water-reducing agent, fly ash pozzolan and Denver tapwater.

III. 2. 6. Specimen Types and Sizes

Specimens cast from the aforementioned concrete mixes consisted of three sizes: 6- by 12-inch cylinders, 4- by 4- by 30-inch bars, and 3- by 7- by 2-inch bars. The number and type of specimens cast from each of the seven concrete mixes is presented in Table III-16. In addition to the standard size specimens initially cast from Mix No. 1, two rectangular concrete orifices, 4 by 9 by 8 inches were cast inside of a 16-inch-diameter section of steel pipe for use in cavitation tests. Also a concrete pipe section of 7 inches inside diameter, 15 inches outside diameter and 22 inches long was cast for the purpose of subjecting the concrete to a differential pressure in conjunction with the high-temperature brine environment. Just recently two small domes were cast using Mix No. 1. They are being subjected to 204° F steam and the subsequent distillate. (For more detail refer to section on distilled water.)

Concrete for all specimens was mixed for 5 minutes in a 3-1/2-cubic-foot rotating drum mixer. After the mixing of each batch and prior to the casting of specimens, the air content, unit weight, and slump were measured to maintain the desired control. If the desired properties of a particular batch were not within the tolerances allowed, the batch was discarded, adjustments made, and a new batch prepared. In the small mixes one batch provided sufficient concrete to cast the desired number of specimens; however, the larger Mixes No. 1 and 3, Table III-15, required numerous batches. To eliminate the effects of possible differences in the individual batches, specimens were grouped so that concrete to be tested at any one age would have been cast from two or three different batches. Also both bars and cylinders were cast from the same batches.

All specimens except those 3 by 7 by 2 inches in size received an initial 28 days of moist curing (100 percent relative humidity, 73.4° F) followed by at least 90 days of drying. In the case of the original specimens cast from Mixes No. 1 and 2, the drying

Table III-15

CONCRETE MIX DESIGN DATA
Corrosion of Concrete Investigation for OSW

Mix No.	1	2	3	4	5	6	7
CONSTITUENT							
Cement: Type	V	V	V	I	II	III	IV
Sample No.	M-4986	M-4986	M-5207	M-4998	M-4700	M-4997	M-2341
Quantity--lb/cu yd	564	563	563	567	564	567	566
Fly ash pozzolan: Sample No.	M-4962	M-4962	M-5043	M-5043	M-5043	M-5043	M-5043
Quantity--lb/cu yd	141	141	141	141	141	142	141
Water, Denver tap--lb/cu yd	248	260	260	261	250	261	251
Aggregate, coarse3/: Type	Natural	Quartz	Lime-stone	Natural	Natural	Natural	Natural
Sample No.	M-5036	M-5037	M-5192	2/M-5036	M-5036-1	M-5036-1	M-5036-1
Quantity--lb/cu yd	2,201	1,905	1,944	2,181	2,193	2,191	2,206
Aggregate, fine: Type	Natural	Natural	Natural	Natural	Natural	Natural	Natural
Sample No.	M-5036	M-5036	2/M-5036-1	M-5036-1	M-5036-1	M-5036-1	M-5036-1
Quantity--lb/cu yd	1,020	1,067	1,066	1,009	1,014	1,013	1,019
Air-entraining agent (NVX)--cc/cu yd	275	355	144	129	184	145	144
Water-reducing agent--cc/cu yd	610	608	607	611	608	611	610
Water-cement ratio	0.44	0.46	0.46	0.46	0.44	0.46	0.44
Water to cement + pozzolan ratio	0.35	0.37	0.37	0.37	0.35	0.37	0.36
Slump--inches	3.5	3.4	3.5	3.8	3.8	3.7	3.6
Entrained air (grav)%	4.4	4.6	4.6	4.0	4.5	4.0	4.2
Mixing temperature °F	74	74	70	66	68	67	69

1/5 percent solution; 2/Sample No. M-5036-1 was obtained from same pit as M-5036 only at a later date. The two samples are comparable. 3/1-1/2-inch maximum size.

Table III-16

SCHEDULE OF CONCRETE TEST SPECIMENS
Corrosion of Concrete Investigation of OSW

Mix No.	1				2		3		4		5		6		7	
	Orig Spec		New Spec		No.	Envir	No.	Envir	No.	Envir	No.	Envir	No.	Envir	No.	Envir
	No.	Envir	No.	Envir												
6- by 12-inch cylinder[1]	74	Brine	102	Brine	8	Brine	102	Brine	6	Brine	6	Brine	6	Brine	6	Brine
6- by 12-inch cylinder[1]	74	Dry	105	Dry	10	Dry	105	Dry	2	Dry	2	Dry	2	Dry	2	Dry
6- by 12-inch cylinder[1]	-	-	12	Fog[3]	-	-	12	Fog	4	Fog	4	Fog	4	Fog	4	Fog
6- by 12-inch cylinder[1]	-	-	8	Perm[4]	-	-	4	Perm	-	-	-	-	-	-	-	-
4- by 4- by 30-inch bars	26	Brine	60	Brine	4	Dry	36	Brine	2	Brine	2	Brine	2	Brine	2	Brine
4- by 4- by 30-inch bars	12	Dry	30	Dry	4	Dry	30	Dry	2	Dry	2	Dry	2	Dry	2	Dry
4- by 4- by 30-inch bars	24	Dry[2]	-	-	-	-	-	-	-	-	-	-	-	-	-	-
3- by 7- by 2-inch bars	24	Dist H_2O	-	-	8	Dist H_2O	-	-	-	-	-	-	-	-	-	-

[1] Companion specimens, 28 days fog cure followed by continuous drying.
[2] Cast for brine specimens but have not been used.
[3] 100 percent RH, 73.4° F.
[4] Permeability specimens.

was in laboratory air (20 to 30 percent relative humidity plus or minus 80° F). Drying of the specimens cast from Mixes No. 3 through 7 was at 50 percent relative humidity and 73.4° F. Following the drying time certain designated specimens were then subjected to hot synthetic sea water brine while companion specimens continued in the drying atmosphere. Each time that tests are conducted upon the concrete specimens in the brine, similar tests are conducted upon the companion control specimens. The 3- by 7- by 2-inch distilled water specimens received an initial moist curing time varying in amounts from 28 to 280 days. Following the moist curing these specimens received a minimum of 35 days' drying prior to their subjection to the distillate.

All of the 6- by 12-inch cylinders were cast in steel laboratory molds and consolidated by small electric vibrators. The 4- by 4- by 30-inch bars were cast in both brass and wooden molds using a vibrating table as the method of consolidation. The small distilled water specimens were cast in wooden molds also on the vibrating table.

The large concrete bars cast from Mixes No. 1 and 3 are of three types, each type purposely suited to fill a specific need. Following is an outline of these three types:

1. Plain--These bars contain only concrete and are standard length and weight change specimens.

2. Reinforced--These bars have one 3/4- or 7/8-inch-diameter by about 23-inch-long deformed reinforcing bar embedded near the longitudinal centroid. Measurements of the rates of corrosion of the steel are the prime tests to be conducted upon these bars.

3. Prestressed--These bars have four 1/4-inch solid high-strength pretensioning wires, each embedded equidistant from the longitudinal centroid. End anchorage is maintained by the use of "Prescon" buttons bearing against 4- by 4- by 1-inch high-strength steel plates. Initial stress in the concrete was about 1,000 psi. Determination of the rate of corrosion due to the high-temperature brine environment are the principal data to be obtained from tests conducted upon these specimens, although other tests such as length and weight change are also being made.

Also, three sets of the original bars were coated, each with a different coating material for testing in the 100° and 290° F brines.

Two of the coatings were epoxies, one of which was glass fiber filled, and the third was a silicone alkyd. These coatings are discussed in detail in Section V.

III. 3. Test Equipment

III. 3. 1. Distilled Water Test Systems

The low-temperature distilled water system, Figure III-1, consists of a test chamber fabricated from a 55-gallon steel drum. Mounted in the side of the chamber near the bottom is a 220-volt, 5-kw electric immersion heating element that is controlled by a thermostat which has its sensing element strapped to the heating element. Also mounted to the side of the chamber is a water glass used for observing the source water level. Attached to the inside of the lid for part of the test was a 50-foot-long, 1/2-inch-diameter condensing tube rolled into a plus or minus 1-foot coil. Each of the six test specimens is supported upon a rack located midway between the top and bottom and directly beneath one of six 1/4-inch copper overflow tubes. These tubes protrude from the sides of a collection distribution container, and allow the collected distillate to overflow onto the test specimens.

Subsequent to the completion of the main distilled water test series and just recently, the low temperature distilled water test equipment was modified somewhat to accommodate a concrete dome. Essentially the condensing coil as previously described was eliminated and replaced in one case by three 1/4-inch-diameter copper condensing circuits embedded in a concrete dome, Figure III-2.

The high-temperature distilled water test system, Figures III-3 and III-4, consists principally of a 2-foot-diameter by 3-foot-deep autoclave coupled with an externally

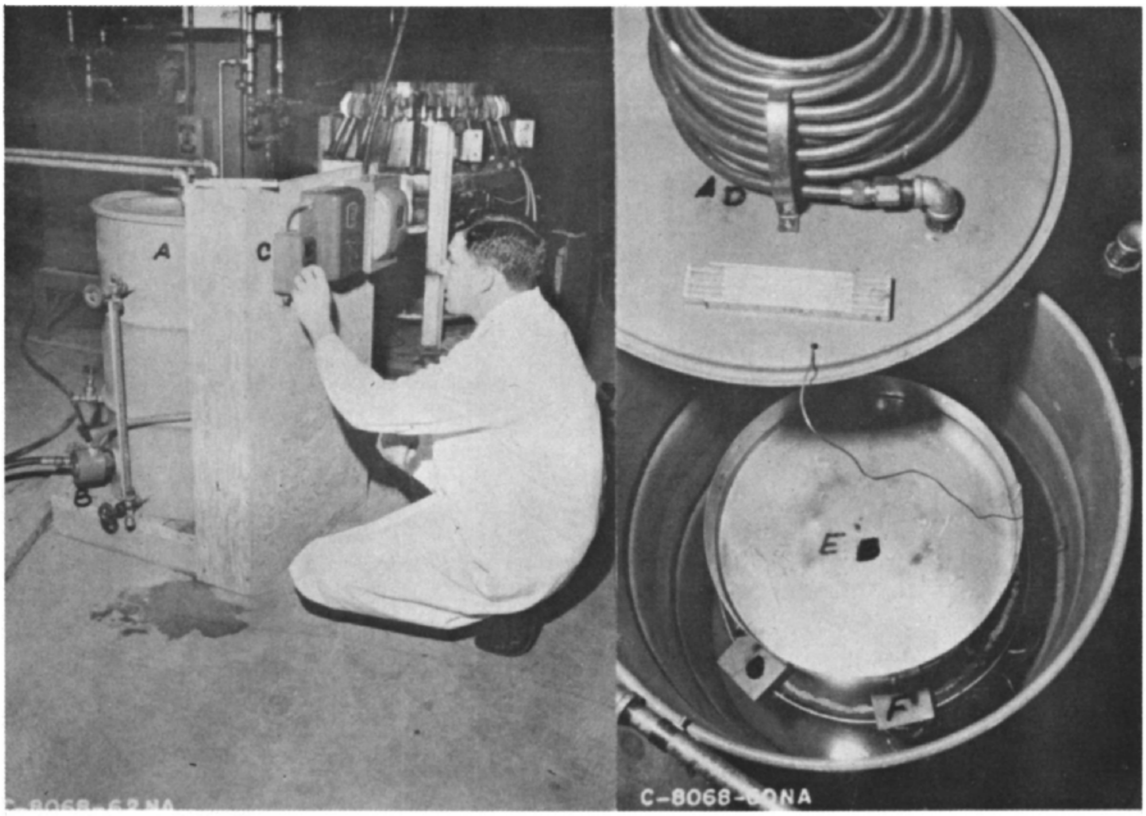


Figure III-1: Low-temperature distilled water test chamber controls, and specimen rack.

(A) Test chamber
(B) Electric immersion heater
(C) Thermostat temperature control
(D) 50-foot-long condensing tube
(E) Distillate collection and distribution container
(F) Concrete test specimen

Photos (left) PX-D-55296 NA and (right) PX-D-55297 NA

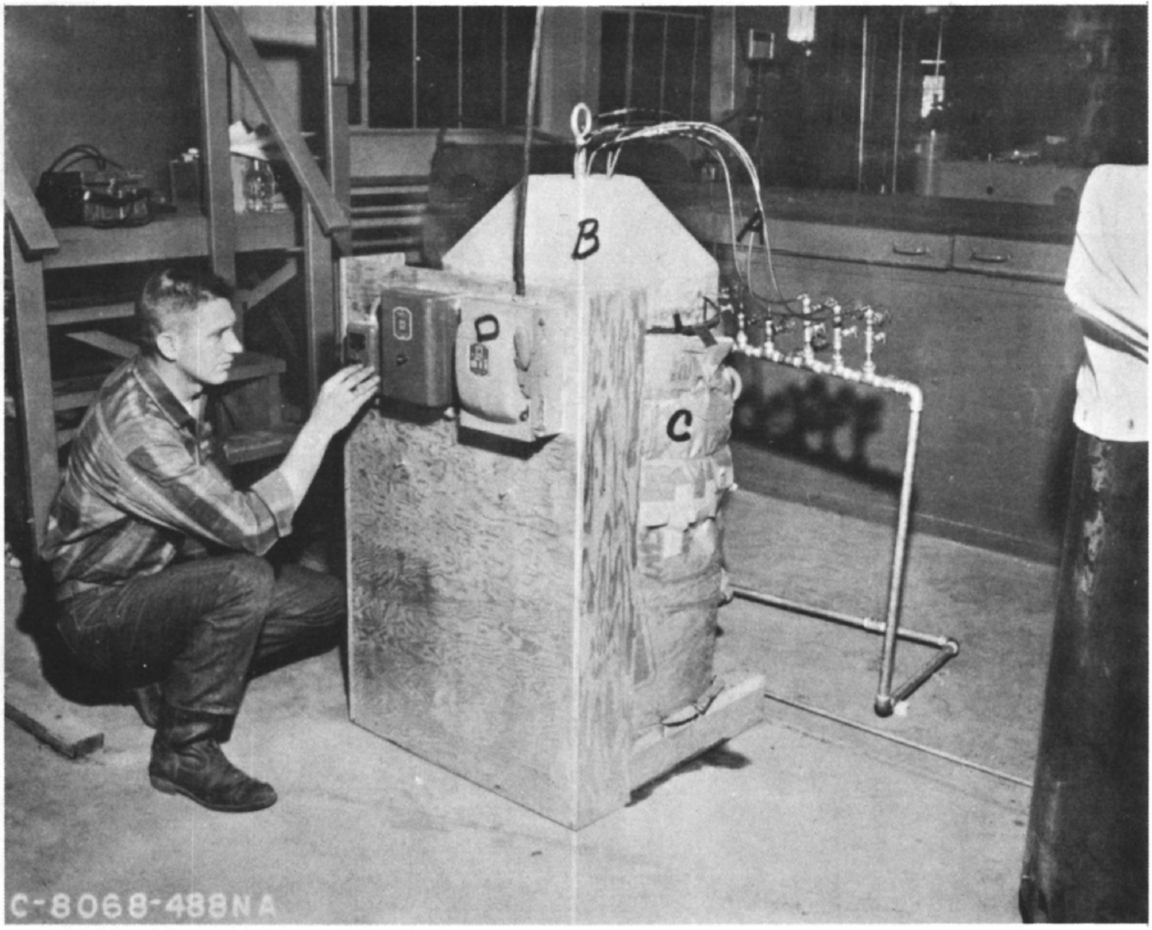


Figure III-2. Low-temperature distilled water test chamber and controls modified to accommodate a concrete dome.

(A) Partially embedded condensing circuits
(B) Concrete dome
(C) Test chamber
(D) Controls and electrical equipment

Photo PX-D-60850

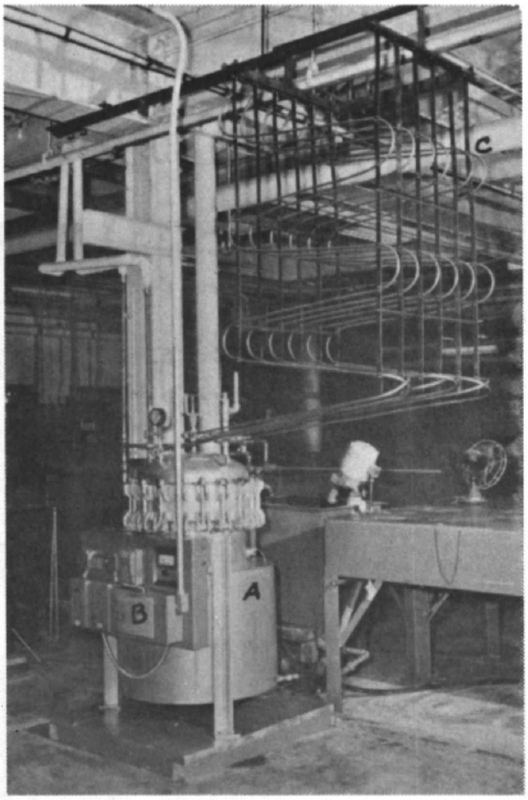


Figure III-3. High-temperature distilled water test chamber, controls, and reflux condenser.

- (A) Test autoclave
- (B) Temperature controls
- (C) Six-part reflux condenser

Photo PX-D-55284 NA

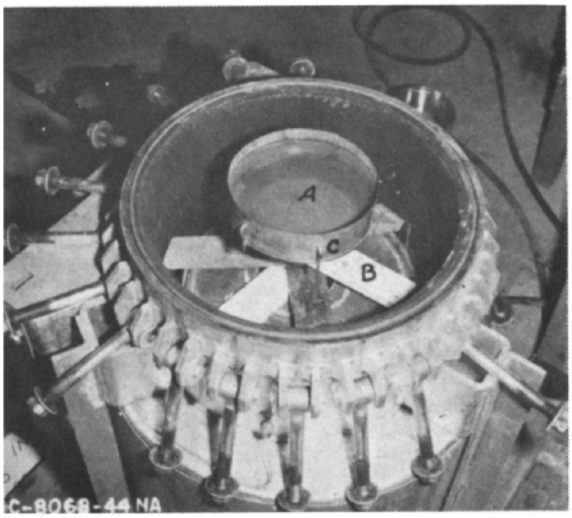


Figure III-4. Interior view of high-temperature test chamber.

- (A) Distillate collection and distribution container
- (B) Test specimen
- (C) Distillate overflow tube

Photo PX-D-55285 NA

mounted six-part reflux condenser made up of six 22-foot-long, 1/2-inch-diameter vertical serpentined tubes, each one valved at the upper extremity. The specimen support system is identical to that described in the low-temperature distilled water discussion.

III. 3. 2. Synthetic Sea Water Test Systems

The synthetic sea water test systems will be discussed in two divisions: (1) Original systems which operated for about 2 years; and (2) new systems which have been operating about 1 year.

Each of the two original systems, Figure III-5, operated continuously at 100° and 290° F until they were replaced with the new systems. They consisted primarily of a continuous closed pipe loop approximately 130 feet long. Specimen test chambers which consisted of three sections each 12 inches in diameter by 20 feet long, were located approximately 17 feet above the ground floor. These test sections were coupled to 8-inch-diameter vertically positioned pipes which in turn were coupled to 6-inch-diameter horizontally positioned pipes approximately 2 feet above the floor. Coupled to these pipe systems were the following equipment items necessary to maintain circulation, temperature and pressure of brine in the two systems:

1. Twenty-horsepower electric-motor-operated centrifugal chemical pumps, used to circulate brine in both the 100° and 290° F systems.

2. One-half-horsepower electric-motor-operated screw pump, used to maintain pressure in the 290° F system.

3. One-half-horsepower electric-motor-operated turbine vane pump, used to maintain water under pressure in packing gland of large 290° F system circulating pump.

4. One-third-horsepower electric-motor-operated piston pump, used to maintain vacuum in 100° F system.

5. One-third-horsepower electric-motor-operated piston pump, used to maintain cool waterflow into 100° F system heat exchanger.

6. Five-kw electric immersion heating element, used to maintain temperature in the 100° F system.

7. Fifteen-kw electric immersion heating element, used to maintain temperature in the 290° F system.

8. Electric temperature and pressure-recording controllers, used to control the the temperature and pressure or vacuum in both the 100° and 290° F systems.

Because of the necessity to maintain a specific brine velocity in the test sections, manometers were coupled through piezometer tubes to calibrated elbow meters to provide differential pressure readings from which discharge rates and velocities could be computed. Control of the velocities was maintained by the use of 8-inch gate valves positioned in the main loops downstream from the circulating pumps.

Concrete test specimens were held in position in the steel test sections by movable dollies, Figure III-6. These dollies were fabricated of 1-inch-diameter galvanized pipe, stainless steel specimen and axle supports, coupling brackets and axles, and bronze wheels. Design is such that they can be uncoupled in short sections to facilitate handling.

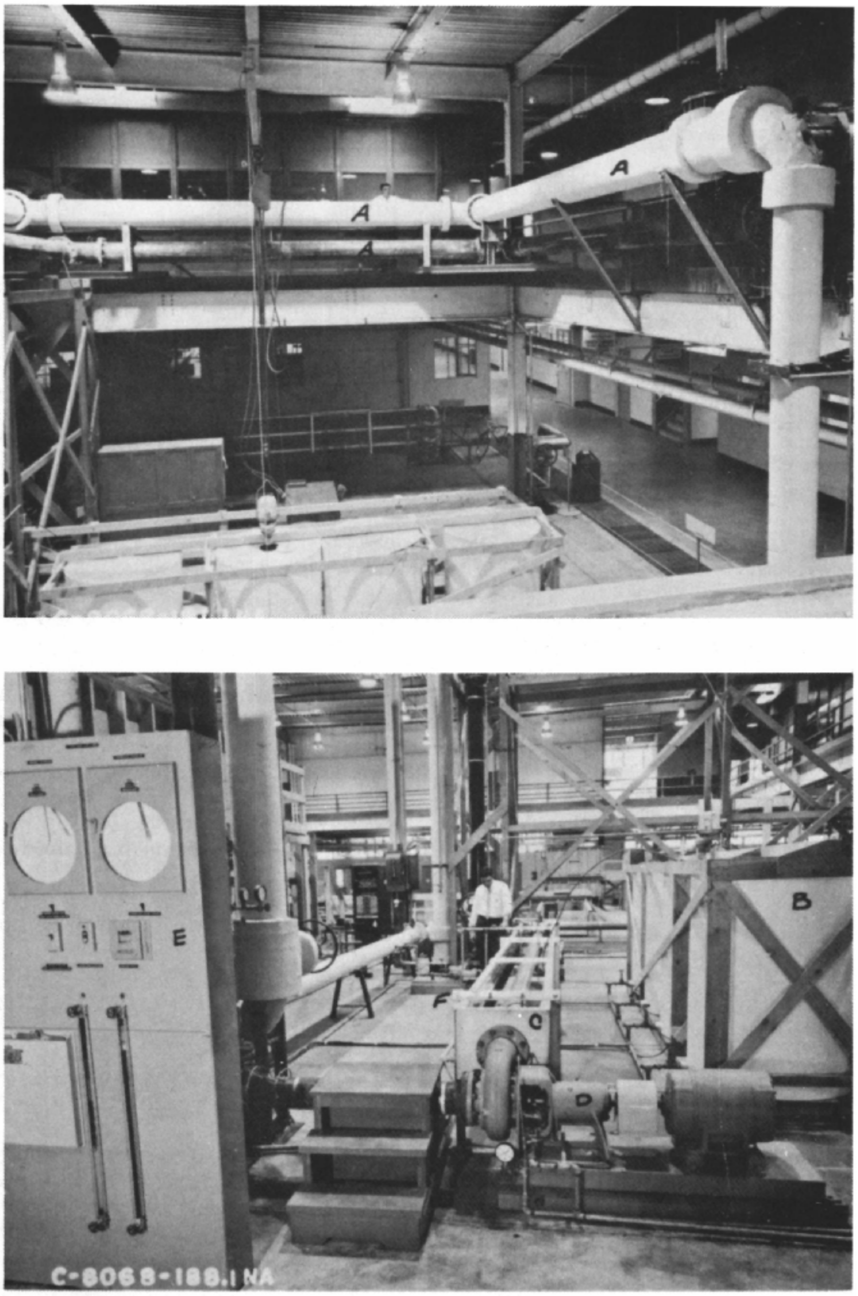


Figure III-5. Original high- and low-temperature synthetic sea water test systems.

(A) 12-inch-diameter specimen test sections
(B) 360-gallon brine mixer tanks
(C) Low-temperature test system heat exchanger
(D) Centrifugal brine circulating pump
(E) Automatic controls for test systems
(F) Pressure and vacuum control pumps

Photos (top) PX-D-55288 NA and (bottom) PX-D-55289 NA

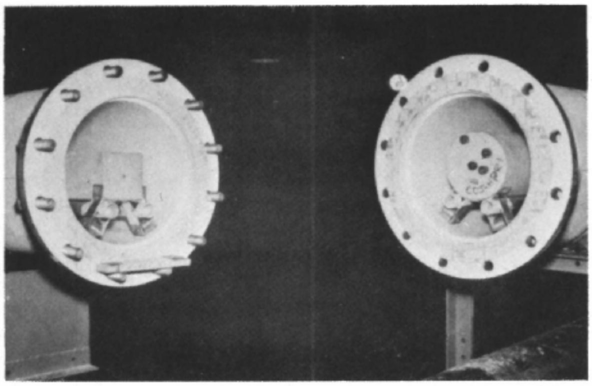


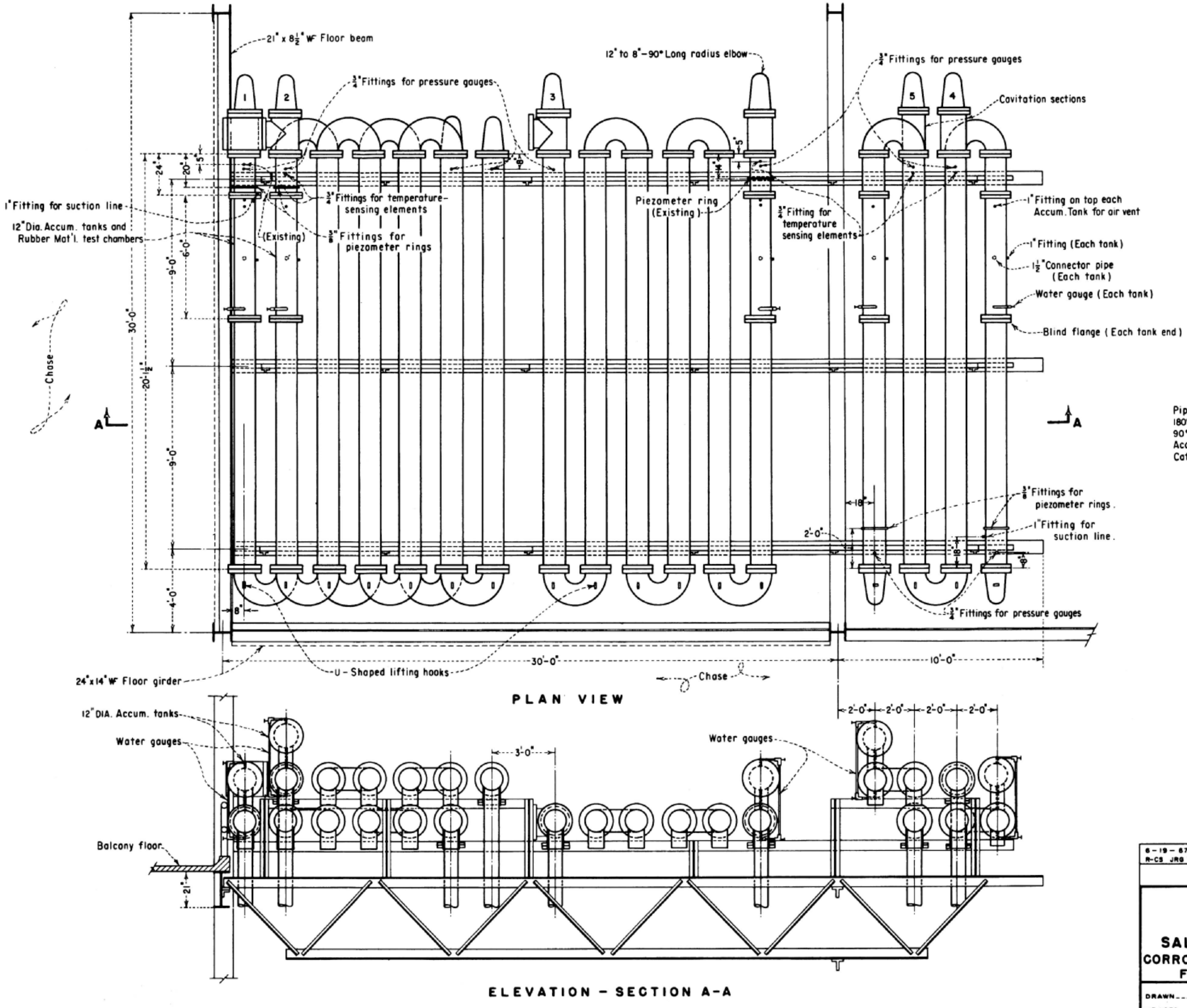
Figure III-6. Concrete test specimens are positioned in the center of the brine test sections on movable dollies. A 90° elbow has been removed here to show interior arrangement. Photo PX-D-55281 NA

When construction of the new synthetic sea water test systems began, the aforementioned two test loops were dismantled and rebuilt as a part of the five new test systems, Drawings No. CCI-DR-1, CCI-DR-2, and CCI-DR-3. Concrete specimens under brine test at the time were transferred to the two new test chambers. This fabrication procedure was adopted to most efficiently utilize the available space. The end result was five independent synthetic sea water test systems operating under five different environmental conditions, Table III-17, Figure III-7. Built into each of the five test loops was an 8-inch-diameter by 14-foot-long bypass loop to be used for the testing of coating, sealing, and filler materials, Figure III-9. These small loops are designed to be used in conjunction with, or independently of, the main concrete test chambers.

Concrete test chambers consisted of 12-inch-diameter by 20-foot-long parallel steel pipe sections 2 feet apart, 18 feet above the floor, and connected at the ends by 180° return bends, Figure III-8. The 203°, 225° and 250° F systems utilize six such sections each, while the 100° and 290° F systems utilize only three sections each. Each of the five systems operates completely independent of any of the others and each is fully automatic. In other words, temperature and pressure recording controllers, Figure III-10, coupled with electric strip heaters and pumps, respectively, provide close control, thereby allowing the equipment to operate on a continuous basis. Maximum pressures are controlled first by electric-solenoid-operated pressure-release valves coupled to the aforementioned pressure controllers; and second by pressure-activated blowoff valves. Electrical supporting components such as relays, contactors, transformers, fuse, etc., are housed in a compartmentalized cabinet, Figure III-11.

Brine is circulated through the systems at about 4 feet per second with 20-horsepower electric-motor-operated end-suction open impeller centrifugal chemical pumps cast from high alloy stainless steel. All pipe utilized in the closed loop test systems is standard wrought steel unlined and specimens are positioned in the test chambers on dollies identical to those used in the original tests.

Periodically, at predesignated times, concrete specimens are removed from the brine environment and subjected to tests including those for compressive strength, length and weight change, static and dynamic moduli of elasticity, X-ray diffraction, differential thermal analysis and petrographic examination.



NOTES

Pipe 12" I.D., 12.75" O.D. S Wrought Steel.
180° Return bends, short radius wgt. w/flange – 324 lbs.
90° Elbows, long radius, wgt. w/flange – 222 lbs.
Accum. tanks – Approx. 40-gal.
Catwalks omitted for clarity

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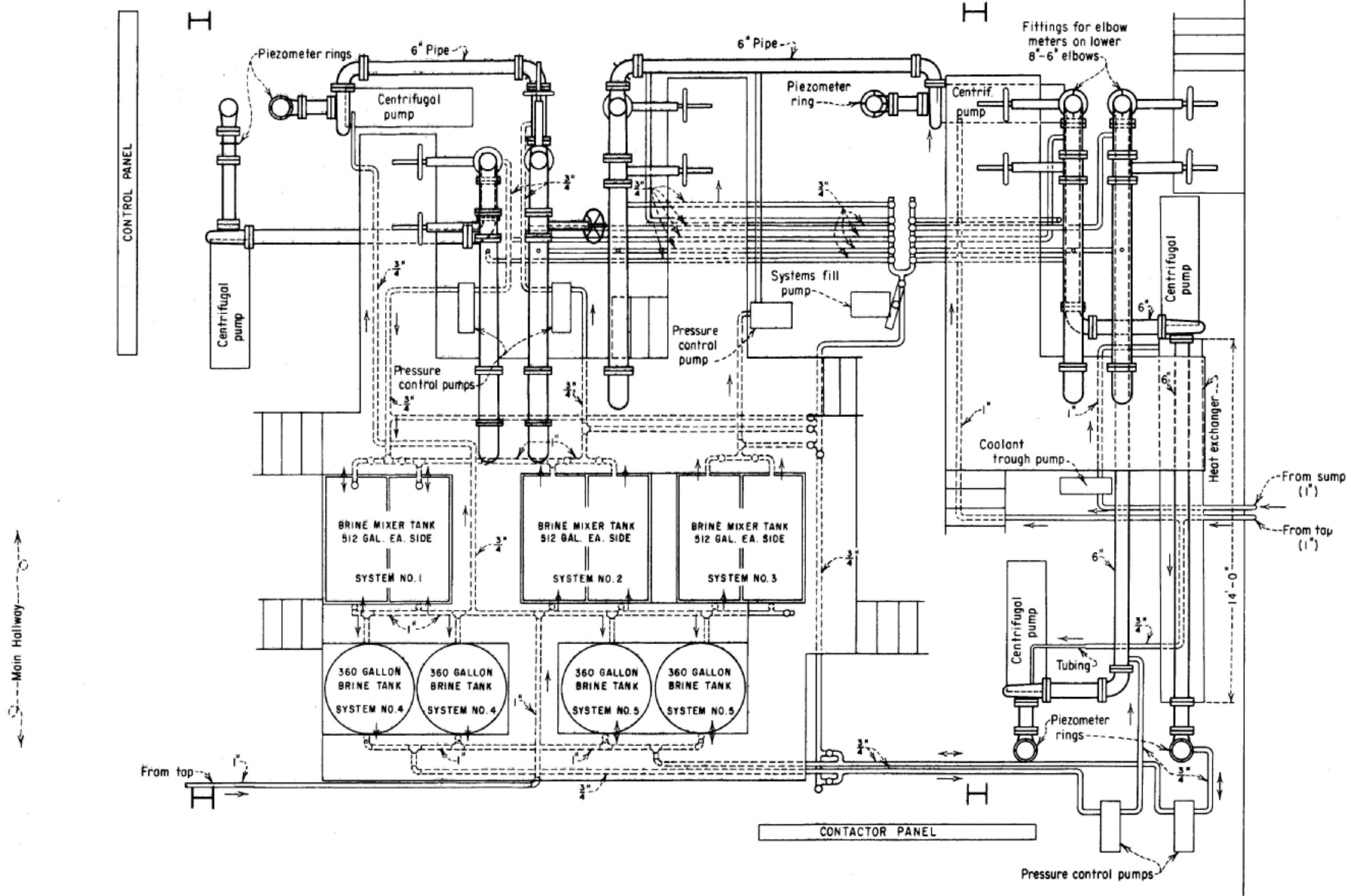
DIVISION OF RESEARCH

SALINE WATER TEST SYSTEMS
CORROSION OF CONCRETE INVESTIGATION
FOR OFFICE OF SALINE WATER

DRAWN H. F. N. SUBMITTED
TRACED H. E. W. RECOMMENDED
CHECKED J. R. G. APPROVED

DENVER, COLORADO, APRIL, 1965 | CCI-DR-1



PLAN OF GROUND FLOOR

6-19-67 R.C.S.-J.R.G. | MINOR REVISIONS

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CORROSION OF CONCRETE INVESTIGATION
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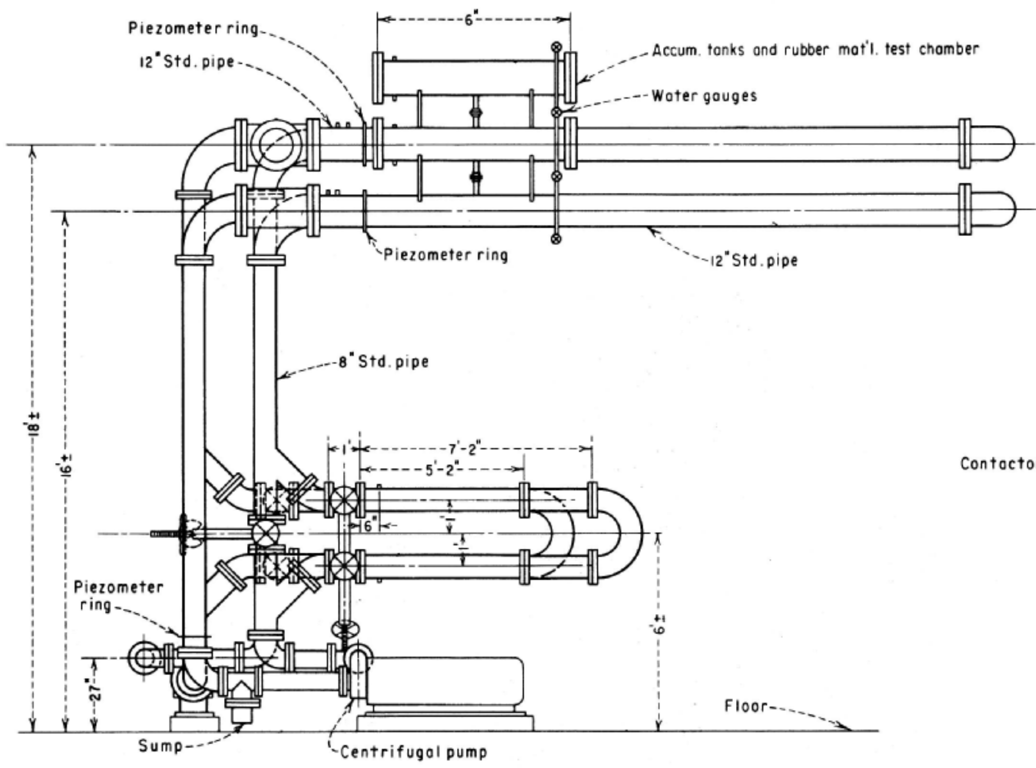
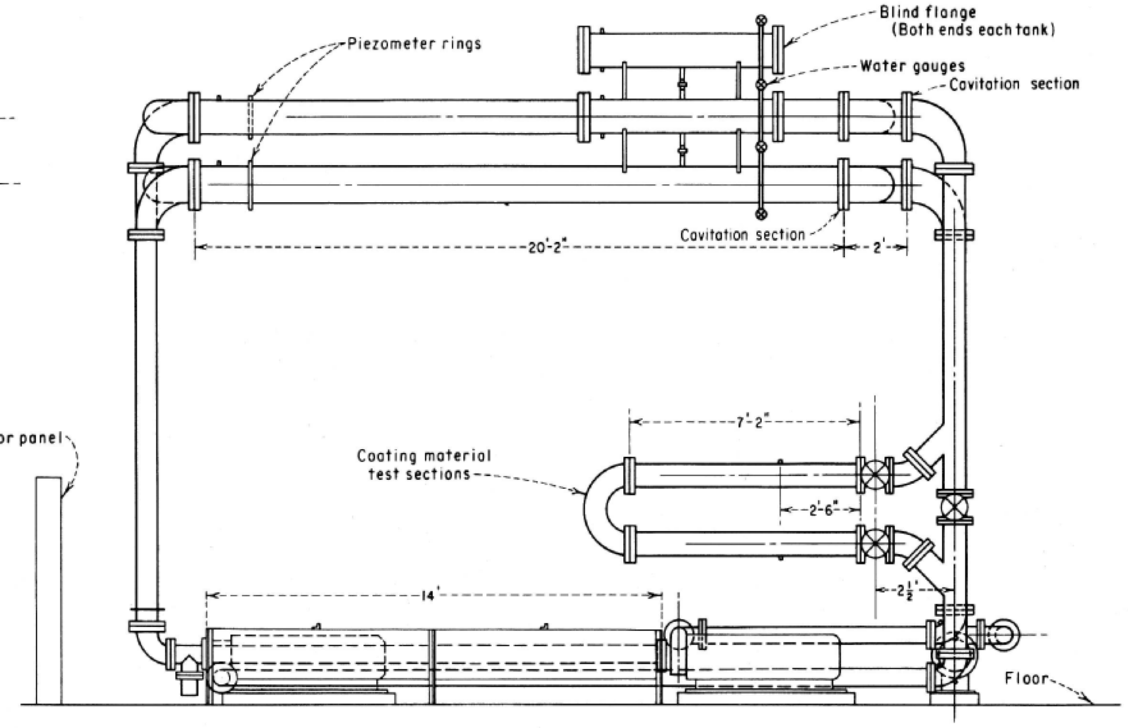
DRAWN J.R.G. SUBMITTED
TRACED L.H.M. RECOMMENDED
CHECKED APPROVED

DENVER, COLORADO, APRIL 19, 1965

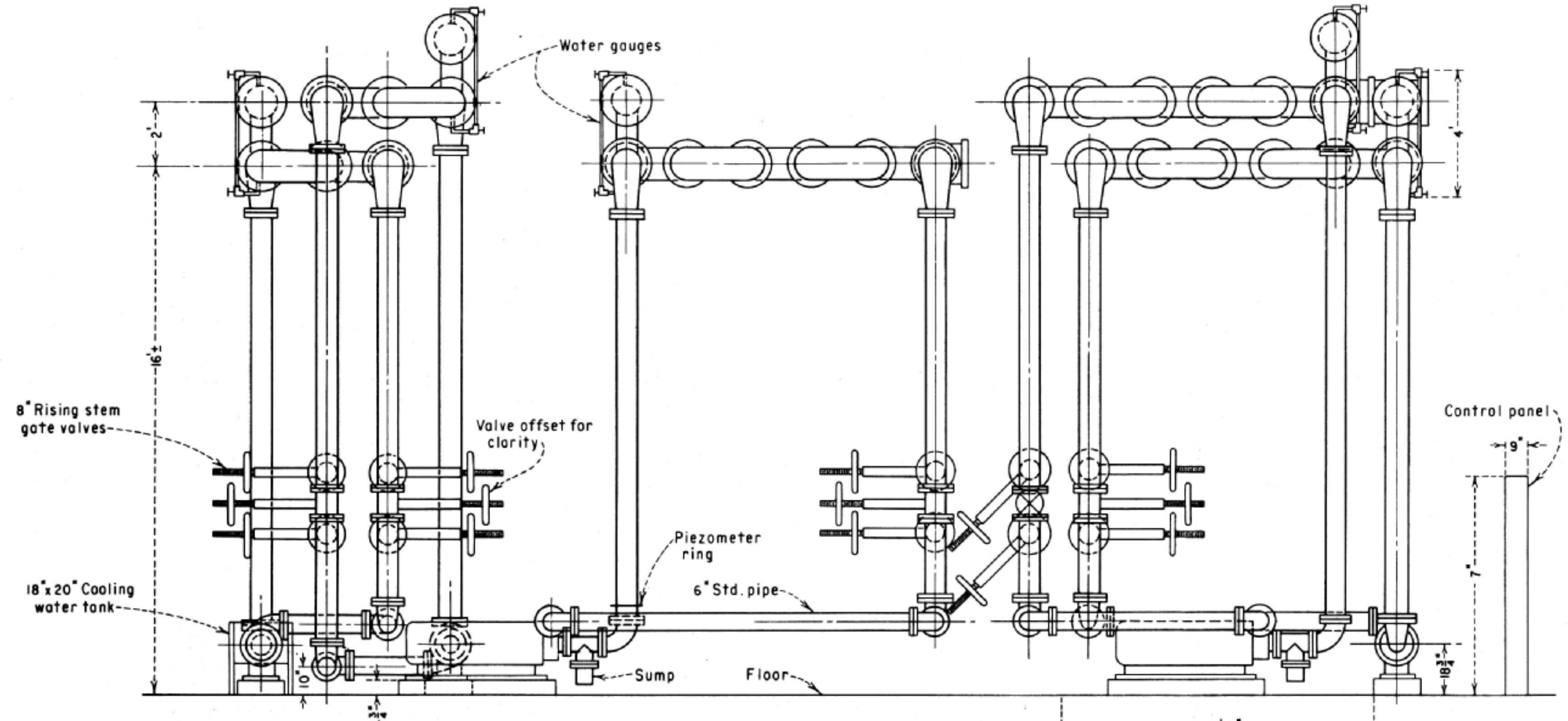
CCI-DR-2



ELEVATION WEST SIDE



ELEVATION EAST SIDE



ELEVATION NORTH SIDE

NOTES

For clarity catwalks and supports have been omitted from this drawing.

6-19-67 R.C.S.-J.R.G.	MINOR REVISIONS

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SALINE WATER TEST SYSTEMS
CORROSION OF CONCRETE INVESTIGATION
FOR OFFICE OF SALINE WATER

DRAWN H.F.W. SUBMITTED
TRACED W.G.C. RECOMMENDED
CHECKED J.R.G. APPROVED

DENVER, COLORADO, MAY, 1965 **CCI-DR-3**

Table III-17

OPERATING DATA OF THE SALINE WATER TEST SYSTEMS
Corrosion of Concrete Investigations for OSW

System No.	1	2	3	4	5
On line date	January 1965	February 1965	August 1966	May 1966	June 1966
Description of Saline Solution					
a. Dissolved solids content	175,000 ppm	38,400 ppm	73,000 ppm	73,000 ppm	73,000 ppm
b. Solution temperature	100° F	290° F	203° F	225° F	250° F
c. System pressure	0.95 psia	58 psia	12 psia	19 psia	30 psia
d. Solution velocity	±4 fps	±4 fps	±4 fps	±4 fps	±4 fps
e. Solution velocity in orifice	10 fps	10 fps	-	-	-
f. Dissolved oxygen	0.2-0.8 ppm	0 ppm	0 ppm	0 ppm	0 ppm
Description of Test Chambers					
a. Length for concrete tests	60'	60'	120'	120'	120'
b. Diameter for concrete tests	12"	12"	12"	12"	12"
c. Length for coatings test	14'	14'	14'	14'	14'
d. Diameter for coatings tests	8"	8"	8"	8"	8"
e. Length for rubber tests	6'	6'	6'	6'	6'
f. Diameter for rubber tests	12"	12"	12"	12"	12"
Description of Test Specimens					
a. Sizes: Conc cyls	6 x 12"	6 x 12"	6 x 12"	6 x 12"	6 x 12"
Conc bars	4 x 4 x 30"	4 x 4 x 30"	4 x 4 x 30"	4 x 4 x 30"	4 x 4 x 30"
Coated concrete	6 x 6 x 1"	6 x 6 x 1"	6 x 6 x 1"	6 x 6 x 1"	6 x 6 x 1"
Coated steel	6 x 6 x 1/8"	6 x 6 x 1/8"	6 x 6 x 1/8"	6 x 6 x 1/8"	6 x 6 x 1/8"
Rubber samples	6 x 6 x 1/8"	6 x 6 x 1/8"	6 x 6 x 1/8"	6 x 6 x 1/8"	6 x 6 x 1/8"
b. No. in System: Conc' cyl	18	18	36	36	36
Conc bars	16	16	32	32	32
Coated conc	48	48	48	48	48
Coated steel	120	120	120	120	120
Rubber smpls	240	240	240	240	240

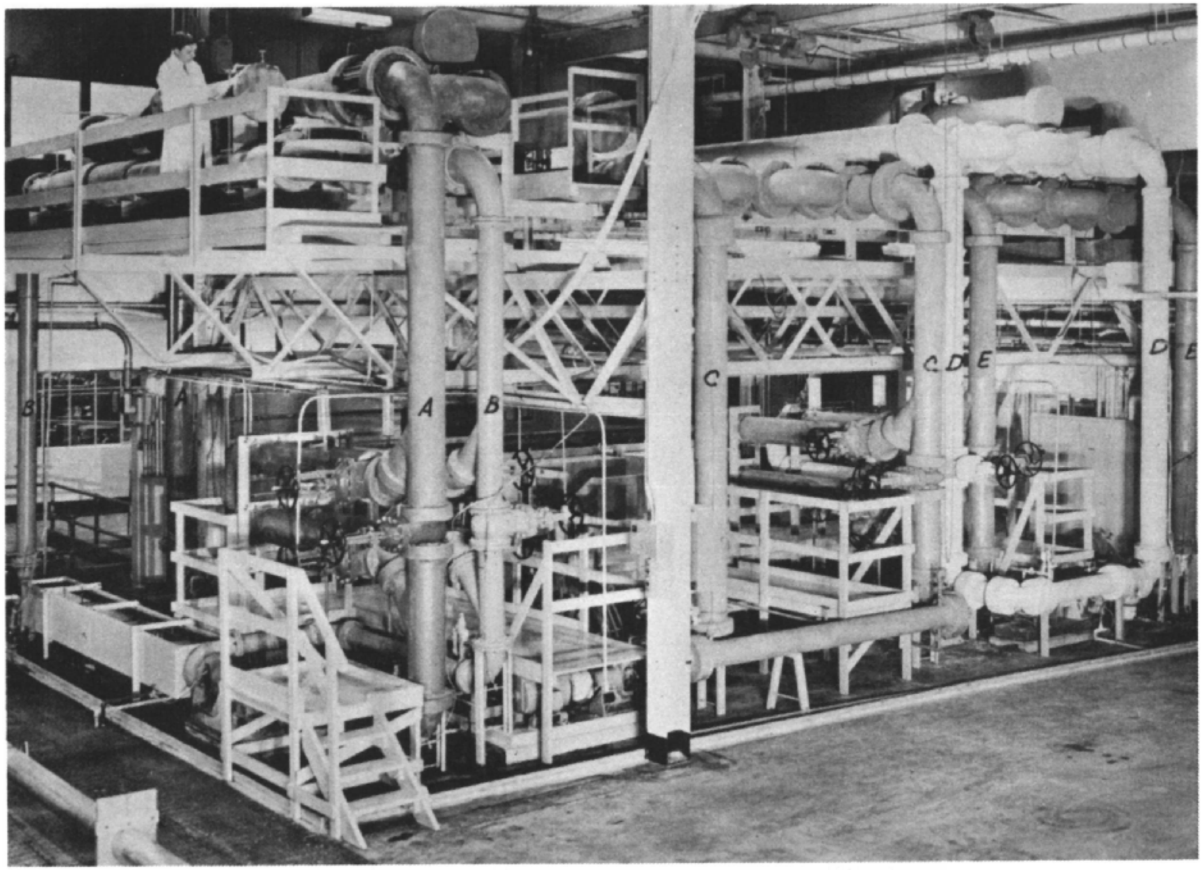

Figure III-7. Saline water test equipment. Photo

(A) 290° F system
(B) 100° F system
(C) 250° F system
(D) 225° F system
(E) 203° F system

Photo PX-D-58619

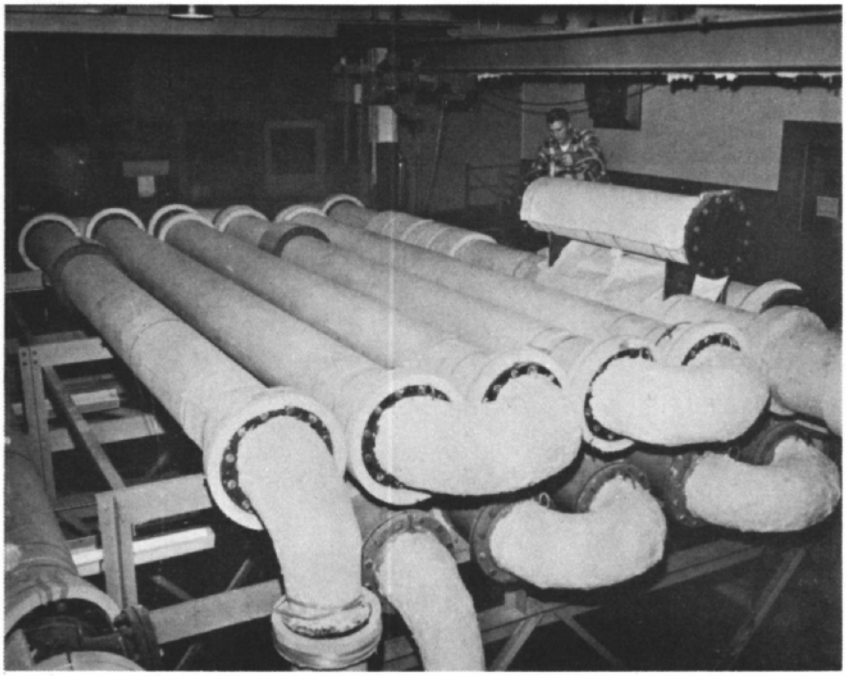

Figure III-8. Closeup view of 225° (top) and 203° F (bottom) concrete specimen test chambers. Photo PX-D-60851

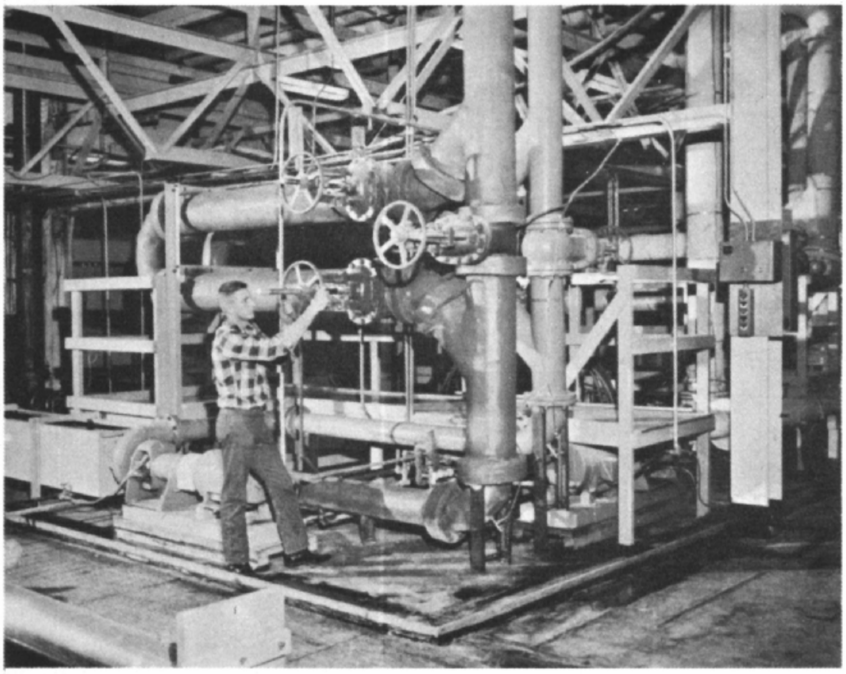

Figure III-9. View of typical coating material test chamber being valved into brine system. Photo PX-D-60852

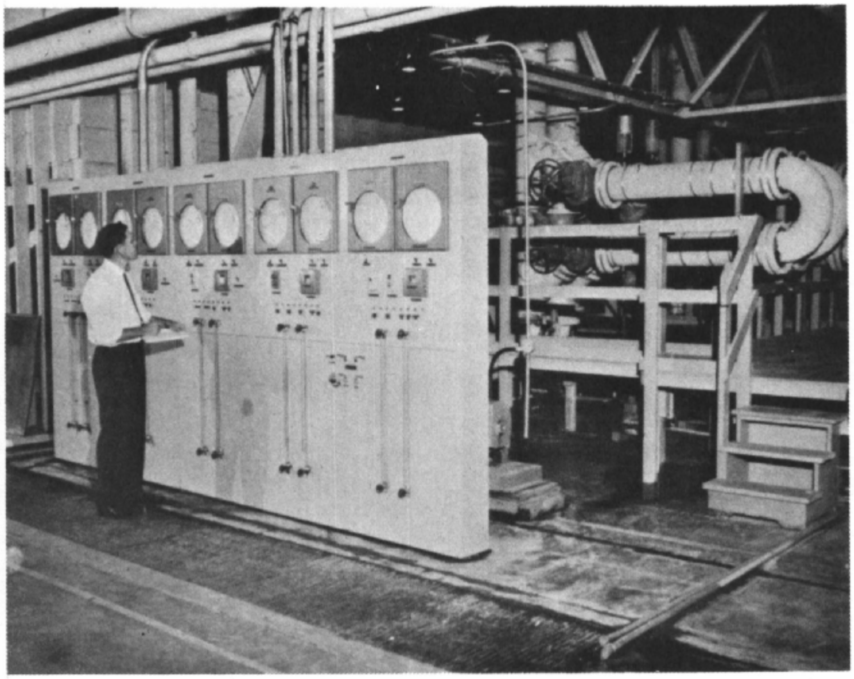

Figure III-10. View of instrument panel which houses all controls for operating saline water test systems. Photo PX-D-60854

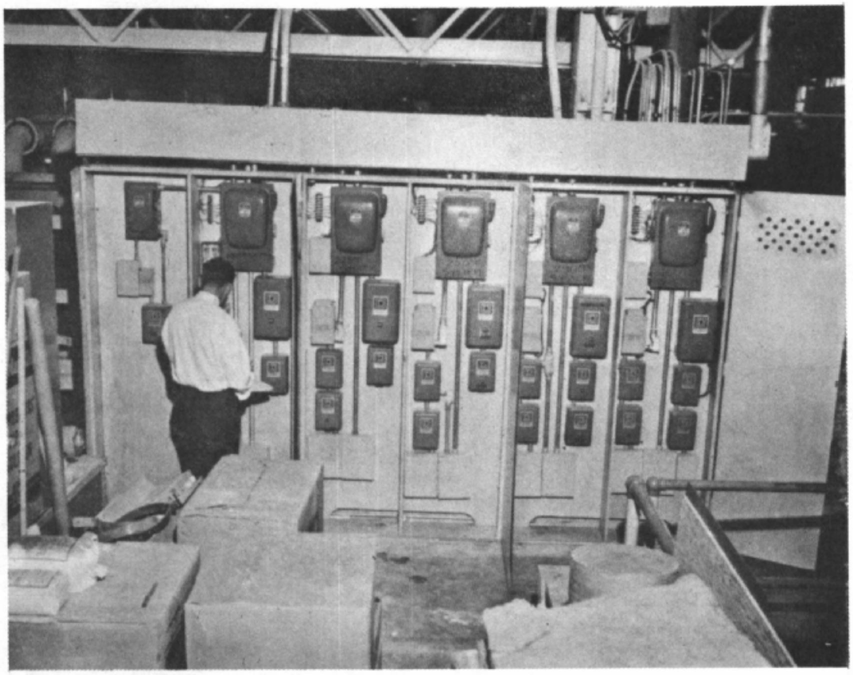

Figure III-11. View showing arrangement of contactors, relays, transformers, etc., used in operation of saline water test equipment. Photo PX-D-60855

III. 4. Test Environments

III. 4. 1. Distilled Water Environments

Test environments maintained within the two distilled water test systems are as follows:

1. Low-temperature distilled water system; source water temperature 203° F, distillate temperature 150° F (203° F for the recently initiated dome tests), pressure 12.2 psia (atmospheric pressure at Denver), distillate pH about 7.

2. High-temperature distilled water system; source water temperature 350° F, distillate temperature somewhat less than 350° F, pressure 135 psia, distillate pH about 7.

Source water used in these two distilled water test systems was Denver tap. Condenser tubes were copper and the collection and distribution containers were stainless steel.

III. 4. 2. Synthetic Sea Water Environments

Test environments maintained within the two original synthetic sea water systems, which have subsequently been rebuilt and which have been in operation for about 2 years are shown in Table III-17 under Systems No. 1 and 2. Systems No. 3, 4, and 5, Table III-17 are the newer systems, identical in operation and conformation to the others of twice the test chamber capacity.

The synthetic sea water that was utilized in the original Test Systems No. 1 and 2 was prepared using a proprietary mixture of salts and Denver tapwater. Lake Products Company, Inc., 1254 Grover Road, St. Louis (Lemay), Missouri, was the source of this formulation of 10 granular inorganic compounds, most commonly found dissolved in the major oceans of the world. The type and amount of each salt compound found within this formulation is such as to meet ASTM Specification D-1141-52, Formula A, for Substitute Ocean Water. In Table III-18 are recorded the various chemicals used in the synthetic sea water and the percentages of each. Table III-19 presents a typical analysis of the sea salt components that have been used for the synthetic sea water, commonly referred to as brine.

Table III-18

SUBSTITUTE OCEAN WATER
ASTM1/ Specification D-1141-52, Formula A
Corrosion of Concrete Investigation for OSW

NaCl	58.490%
$MgCl_2 \cdot 6H_2O$	26.460%
Na_2SO_4	9.750%
$CaCl_2$	2.765%
KCL	1.645%
$NaHCO_3$	0.477%
Kbr	0.238%
H_3BO_3	0.071%
$SrCl_2 \cdot 6H_2O$	0.095%
NaF	0.007%

Density of sea water equals 1.025 at 15° C

1/American Society for Testing and Materials.

Table III-19

TYPICAL ANALYSIS OF SEA SALT COMPONENTS
Corrosion of Concrete Investigation for OSW

$NaCl$	Sodium Chloride		99.9%
	Calcium Sulfate		.08%
	Magnesium Chloride		.006%
	Calcium Chloride		.014%
$MgCl_2\cdot 6H_2O$			
	Magnesium Chloride		99.0%
	Calcium Chloride	max	0.5%
	Alkali Chlorides	max	1.0%
	Magnesium Hydroxide	max	0.2%
Na_2SO_4			
	Sodium Sulfate		99.97%
	Iron		.0004%
	Zinc		.0000%
	Insoluble		.0018%
$CaCl_2$			
	Calcium Chloride		94-97%
	Calcium Hydroxide		0.20%
	Sulfate	max	0.06%
	Iron	max	0.02%
	Heavy Metals	max	0.004%
KCl	Potassium Chloride		99.95%
	Insoluble		.01%
	Sulfate		.01%
	Bromide		.02%
	Sodium		.005%
	Calcium and Magnesium		.002%
	Lead		.002%
$NaHCO_3$			
	Sodium Bicarbonate		99.0%
	Arsenic	max	.0004%
	Heavy Metals	max	.0005%
KBr	Potassium Bromide	min	99.0%
	Iron	max	0.002%
	Sulfide		0.0005%
	Heavy Metals (Pb)	max	0.001%
	Chloride	max	0.3%
	Insoluble	max	0.3%
H_3BO_3			
	Boric Acid		99.90%
	Chloride		0.006%
	Sulfate		0.04%
	Arsenic		0.000%
	Heavy Metals		0.000%
	Iron		0.0003%
$SrCl_2\cdot 6H_2O$			
	Strontium Chloride		99.0%
	Heavy Metals (Pb)	max	0.005%
NaF			
	Sodium Fluoride	min	99.0%
	Insoluble		0.020%
	Chloride	max	0.005%
	Sulfate	max	0.02%
	Sulfite	max	0.0005%
	Heavy Metals (Pb)	max	0.003%
	Iron	max	0.003%

Procedures used in preparation of the brine involved combining an appropriate quantity of the sea salts into the proper quantity of actively agitated Denver tapwater. After a suitable mixing time the prepared brine was then pumped into the test systems.

It was found to be impossible to dissolve enough sea salts to obtain a concentration of 175,000 ppm specified for the 100° F tests because of the low solubility of calcium salts. Since this problem would be encountered in practice and would be solved by the use of ion exchange resins or other means of water softening, the final test solution was modified to duplicate a softened water by the substitution of sodium for a large percentage of the calcium and magnesium. The composition of the softened sea water is given in Table III-20. The 290°, 250°, 225°, and 203° F systems use the standard ASTM formulation sea salt, Table III-18; however, each one is subjected to an acid treatment prior to use to reduce the quantity of carbonates. Presented in Table III-21 is a schedule of concentrations and solutions for each of the five systems.

Table III-20

SOFTENED SEA SALT
FOR CONCENTRATED SUBSTITUTE OCEAN WATER
Corrosion of Concrete Investigation for OSW

Salt	Percentage
NaCl	69.40%
$MgCl_2 \cdot 6H_2O$	17.12%
Na_2SO_4	10.61%
$CaCl_2$	0.725%
KCL	1.718%
$NaHCO_3$	0.0
KBr	0.249%
H_3BO_3	0.075%
$SrCl_2 \cdot 6H_2O$	0.100%

Fluctuations in brine concentration and constituency due to usage was monitored through periodic analysis. Using standardized and proven techniques and procedures for chemically analyzing the used brine, results revealed that although variations in constituency did occur, no definite trends or patterns could be established.

Some variations in brine concentration have existed throughout the life of the test. Scheduled changing of the brine in each of the systems coupled with periodic brine adjustments have facilitated maintenance of concentration approximating those desired.

It has been standard practice to periodically monitor the pH of the used brine. Very little change in pH with use has been noted in any of the five environments except that of the 290° F system. The 38,400 ppm TDS brine used at 290° F seems to undergo a continual drop in pH while being used. No explanation for this phenomenon is presently available. It is necessary to maintain the pH above 7.0 since concrete is very subject to acid attack. To combat the possibility of low pH, a sample of the brine is periodically obtained, the pH is determined, and then adjusted by the use of sodium hydroxide.

III.5. Tests and Procedures

III.5.1. High- and Low-temperature Distilled Water Tests

All of the 3- by 7- by 2-inch concrete bars made for this test received an initial 28 days of moist cure (100 percent relative humidity, 73.4° F). Two sets, six specimens, were removed from the fog after this initial 28-day cure and allowed to dry in uncontrolled laboratory air until such time as the distilled water tests were commenced. One set of specimens received 35 days' drying prior to low-temperature distilled water testing, and the other set 49 days' drying prior to high-temperature distilled water testing. Consecutive sets of specimens received a total of 28, 56, 154, 182, and 280 days' moist cure. Each set of specimens was subject to the distilled water for 126 days.

Table III-21

SYNTHETIC SEA WATER SOLUTIONS AND TREATMENTS
For 5 Test Environments
Corrosion of Concrete Investigation for OSW

System No.	1	2	3	4	5
Dissolved solids, ppm	175,000	38,400	73,000	73,000	73,000
Brine temperature °F	100	290	203	225	250
Salt mix	1/B	2/A	A	A	A
Mix quantity oz/gal H_2O	31.75	6.23	12.36	12.36	12.36
Treatment after mixing:					
1. Acidify with H_2SO_4 to pH 3.7-3.8	No	Yes	Yes	Yes	Yes
2. Neutralize with NaOH to pH 7.3-7.5	If required	Yes	Yes	Yes	Yes

1/Softened sea salt, Table III-20.
2/Standard ASTM sea salt, Table III-18.

Arrangement of specimens in the two distilled water systems was such that the hot distillate would drop about 2 inches from the collection and distribution container outlet tubes onto the inclined, flat-formed surface of the concrete test specimens. The distillate, after reaching the specimen, would then flow the length of the specimen, down over the end and back into the source reservoir. A maximum of six test specimens were tested in each system.

Due to the surface tension inherent in the distillate and the equipment configuration, it was very difficult to obtain an equal amount of flow over each of the six specimens. Therefore, to reduce the effects of this phenomenon the positions of the specimens were rotated approximately monthly. Other than the aforementioned time, the systems were shut down only for inserting or removing specimens. Each time that the test systems were opened for inserting or replacing specimens, photographs of both the new and old specimens were taken as well as a determination of the weight. Visual observations were also recorded each time. It was decided midway through the tests that the weight measurements were of little value due to the number of variables that effected them and thus, they were eliminated.

More recently the low-temperature distilled water test equipment has been modified to accommodate a concrete dome which is undergoing a vapor condensation exposure approaching that expected in flash desalination structures. The dome configuration is such that concrete surfaces in three different planes receive the 203° F vapor, Figure III-12. Also the exposed area of the first dome tested encompassed two types of surfaces: (1) a hand sacked or rubbed finish, and (2) the untouched steel form surface. Only the untouched steel form surface is being tested on the second dome. The concrete from which the domes were cast contains the same mix proportions as Mix No. 1 in Table III-15.

Operation of the distilled water dome tests are closely controlled by the maintaining of constant temperatures. Source water for the vapor is maintained at a very active boil, 203° F, and vapor temperature remains constant at 203° F. The domes which have walls 2-1/2 inches thick, are cooled sufficiently to cause condensation, in the case of the first dome, by the circulation of cooling water through three 1/4-inch copper tube circuits embedded in the concrete of the dome, Figure III-12. Although cooling water into the dome averaged about 85° F, a very nonuniform temperature distribution was obtained. The external surface of the second dome is uniformly cooled by an air-conditioning unit. Temperature of the condensed moisture, commonly referred to as product water, approaches 203° F. The average quantity of product water produced by the first dome tests was 15 gallons per day, whereas, it totals about 5 gallons per day with the uniform cooling of the second dome.

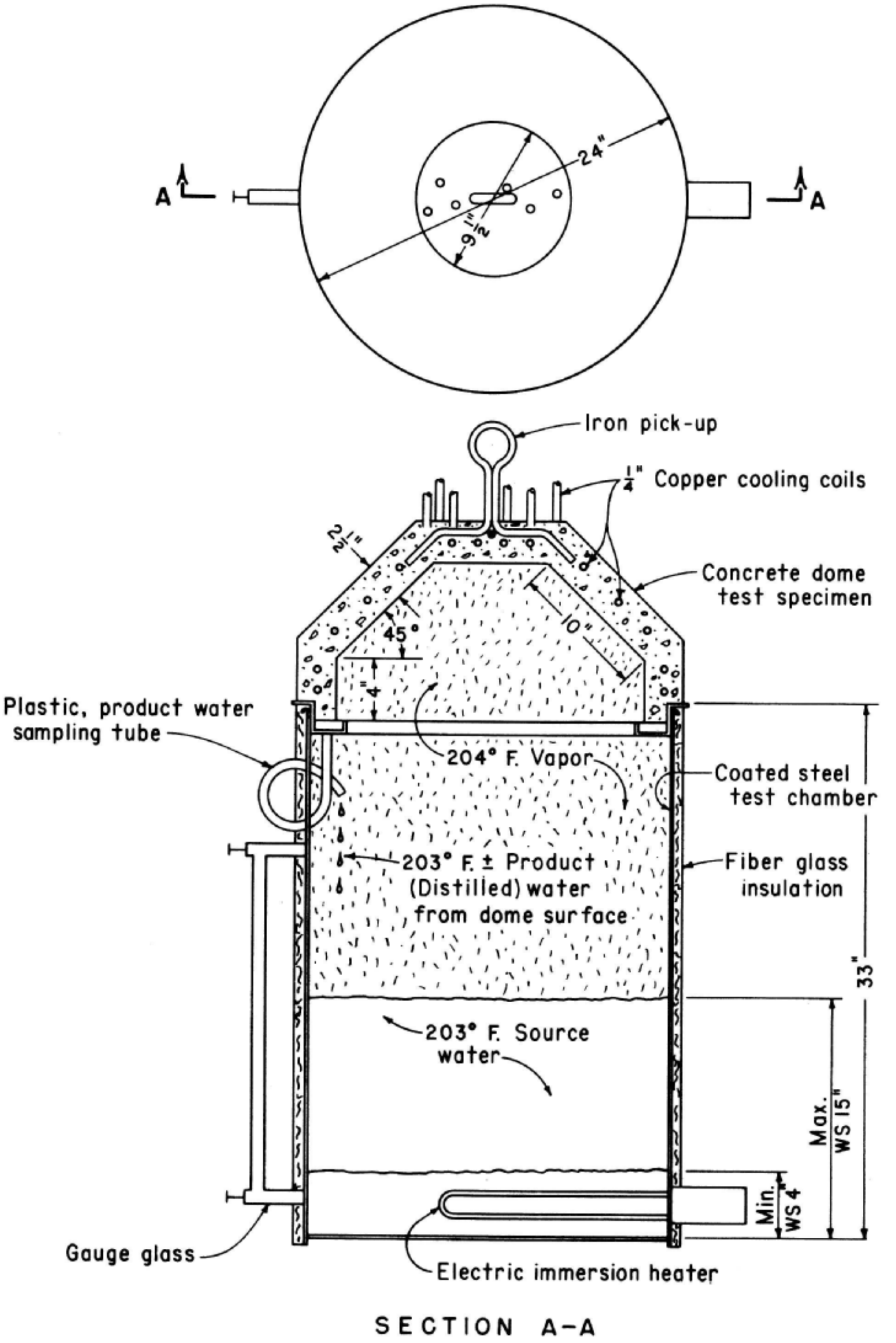

FIGURE III – 12 : DISTILLED WATER DOME SPECIMEN AND THE ASSOCIATED TEST EQUIPMENT

Physical testing of the dome after a given length of vapor-condensate exposure consists primarily of visual observations, photographs, such as Figure III-13, depth of leaching measurements, and later petrographic studies. At periodic intervals during the life of the test, samples of the product water and in some cases the source water are extracted for chemical analysis.

A chemical investigation conducted in conjunction with the distilled water tests involved determining the amount of free lime available in concrete and mortar specimens after varying lengths of moist curing. Specimens utilized for these tests consisted of 2- by 2- by 2-inch neat cement fly ash mortar cubes and 1- by 1-1/2- by 2-inch concrete bars. The concrete specimens were of the same mix as the distilled water specimens. However, the mortar specimens consisted of four different mixes, each with a different cement and pozzolan content. Pozzolan quantities were 0.0, 9.1, 16.7, and 23.1 percent by weight of the total cementitious material for the four mixes. Quantity of cement in each case was reduced by the same amount as the pozzolan was increased.

III. 5. 2. Synthetic Sea Water Tests

Twenty-four hours following casting of the 6- by 12-inch concrete cylinders the steel molds were removed. Three days following the casting of the companion 4- by 4- by 30-inch bars the molds were removed and initial values of length, weight, and dynamic modulus of elasticity determined. All specimens cast for the brine tests received an initial 28 days of moist curing at 100 percent relative humidity and 73.4° F, followed by a period of drying.

Original specimens were dried in laboratory air at approximately 30 percent relative humidity, and 80° F; however, the new specimens cast for the extended brine tests were dried under controlled conditions, 50 percent relative humidity and 73.4° F. Length of drying time varied for different specimens, depending upon their use. For instance, specimens designated to go into the 100° F brine dried for 95 days, whereas, their companion specimens underwent continuous drying.

Prior to testing in the brine environments, one each of the original reinforced and prestressed bars were intentionally cracked in flexure. Two each of the newer, but similar specimens, were intentionally cracked subsequent to initial insertion in brine. This was done to provide an adit for the brine to the reinforcing thereby providing additional valuable specimens for corrosion studies. Specimens were placed into the 12-inch-diameter brine test sections in such a way that companion specimens were widely separated. Such an arrangement of the test specimens was made to minimize any possible side effects occurring from positioning. Outside of the aforementioned arrangement, the only division made in specimen location was that cylinders of like concrete were placed together in one 20-foot leg of each test section and the bars of like concrete were placed together in the remaining two legs, Figure III-6.

In the cases of the 203°, 225°, and 250° F systems, the limestone aggregate concrete occupied three adjacent legs of the test chamber in each case while the natural aggregate concrete specimens occupied the remaining three legs of each chamber system.

Operation of the test systems is intended to be continuous except for short periods of time required for specimen testing. In recent months such has been the case; however, prior to this a great deal of difficulties were encountered with the mechanical seals on the circulating centrifugal pumps used in our test systems.2/ Originally, these pumps were equipped with packed stuffing boxes as recommended by the pump manufacturers. At a later date it was determined that the dilution of the test brine caused by the stuffing box cooling lubricating water could not be tolerated further and conversion from the packed stuffing boxes to mechanical seals was effected. The pump company, although still not advocating them, recommended that a John Crane Type 9 mechanical seal be used. After a number of months of constant seal failure on pumps used by all of the five brine environmental test systems, this seal was rendered useless for our situation. In brief, failures seemed to result from brine scale, possible fretting action between seal and shaft, and actual component failures. Seals in the pumps have subsequently been converted to John Crane Type I, and their operations for the most part have been satisfactory. An identical replacement pump has also been obtained which considerably reduces the down time required to correct pump failures. As a point of

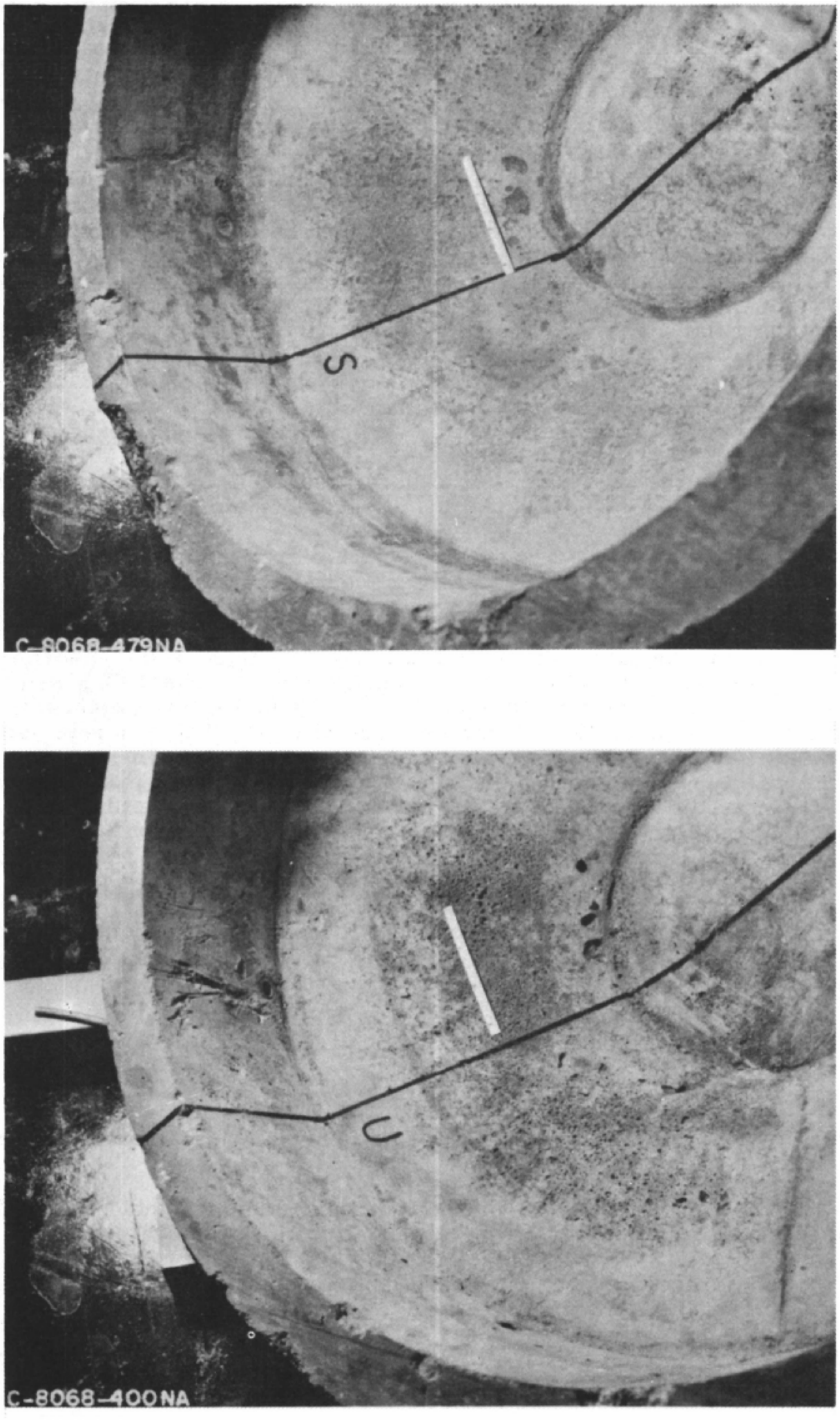

Figure III-13. Views of interior of dome used in vapor condensation studies. Top photograph shows sacked surface and bottom photograph shows untouched formed surface. Photos (top) PX-D-60895 and (bottom) PX-D-60896 interest, no deterioration of the pump impellers or volutes has yet been detected in any of the circulating pumps.

Concrete specimens were removed from two systems and subjected to standard testing after 28, 90, 180, 270, 365, and 547 days' exposure to the 100° and 290° F brines environments and further after 2 years' exposure to the 100° F brine. Concretes undergoing 203°, 225°, and 250° F brine exposure were tested after 28, 90, and 180 days in the environments. At each testing age, all specimens were subjected to nondestructive tests. In addition, 3- by 6- by 12-inch cylinders were tested destructively at each of the test ages except in the cases of the 290° F, 18 months' test and the 100° F, 2-year test. Previous rescheduling of the test dates of concrete in the 100° and 290° F brine to allow for the testing of an additional set of specimens after 180 days of exposure and an additional test series after 270 days of exposure, has caused a lag in the total exposure time undergone by the specimens designated for destructive testing.

Tests conducted upon the 6- by 12-inch concrete cylinders included pulse velocity determinations, specific gravity, elasticity, and compressive strength. Tests conducted upon the 4- by 4- by 30-inch bars included length change, weight change, dynamic modulus, microstructural change and corrosion. (See Sections VI and VII for steel corrosion and microstructural information.) General tests conducted upon the concrete included coefficient of expansion, and air and water permeability.

As a general rule, all specimens removed from the brine systems were placed in a controlled temperature area at 73.4° F for 1/2 to 1 day prior to testing to obtain a standard temperature.

Tests, to provide information on potential cavitation damage to the concrete circumventing rectangular-shaped orifices, were made in the 100° and 290° F synthetic sea water brine. Concrete units containing 4- by 9- by 8-inch-long rectangular orifices were cast inside sections of 16-inch-diameter steel pipe. The concrete used was identical to that from which the natural aggregate concrete specimens were cast, Mix No. 1, Table III-15; further, these cavitation sections were cast at the same time and from the same batches as the test specimens. A steel facing plate was cast into the concrete on the upstream face and adjacent to the lip of the orifice to prevent loss of the sharp 90° lip. Brine flow through these cavitation orifices was maintained at 10 feet per second. Four 1/4-inch copper piezometer tubes were placed in the floor of the orifice near the upstream edge for possible use in measuring differential pressures. These tubes were coupled to manometers located on the control panel.

III. 6. Test Results

III. 6. 1. High- and Low-temperature Distilled Water Tests

III. 6. 1. 1. Conclusions. --

Based upon the results of the high- and low-temperature distilled water tests the following conclusions seem apropos:

1. Portland cement concrete uncoated and untreated will not withstand the leaching effects of warm to hot flowing mineral-free distilled water.

2. Fly ash pozzolan in the concrete mixes was ineffective in reducing leaching by warm to hot distilled water.

3. Increasing the moist curing time of the concrete to 280 days does not reduce its susceptibility to leaching by the distilled water.

4. Severity of matrix leaching by the distillate increases as the temperature of the distillate increases.

5. Removal of the free lime from the concrete does not make the concrete immune to leaching by distilled water.

6. Increasing the amount of fly ash pozzolan used in the mix concrete from 9.1 to 16.7 percent of the total cementitious material reduced the quantity of free lime; however, little benefit in free lime reduction was gained by increasing the pozzolan beyond 16.7 to 23.1 percent.

III.6.1.2. Discussion.--

Coincidental with the start of the distilled water tests two small chemical investigations were undertaken to determine the amount of free lime available in concrete at various stages of its curing. Pearson and Galloway3/ presented data which indicated that the use of fly ash pozzolan would tie up all available free lime after about 210 days of curing. With no free lime available then leaching of the concrete matrix by distilled water would be inhibited as would the destructive attack by soluble sulfates.

Results of tests conducted upon mortar specimens containing varying combinations of cement and pozzolan indicated a maximum reduction in free lime content of 2.70 percent, by weight, of cement plus pozzolan at 210 days and 3.06 percent at 553 days' age, Table III-22. Curves plotted from data included in Table III-22, indicate that a small benefit in free lime reduction was realized when the amount of fly ash used was increased from 9.1 to 16.7 percent, but practically no benefit was realized when the amount was increased further to 23.1 percent, Figure III-14.

Results of free lime determination tests conducted upon 1- by 1-1/2- by 2-inch concrete specimens were more variable than those conducted upon the mortar specimens, Table III-23. Apparently the reason for this is that the quantity of coarse aggregate varies from sample to sample. Additional moist curing helps in reducing the amount of free lime in concrete although, not substantially, and large amounts of lime remain even after 553 days of fog curing. Also, increasing the length of moist curing of concrete appears to reduce the effects drying has on the free lime content, Figure III-15.

High- and low-temperature distilled water tests proved quite conclusively that a normal high-quality concrete cannot suitably withstand the leaching effects of mineral-hungry waters even at only moderate temperatures. The severity of leaching of the matrix from the concrete by the low-temperature distillate was somewhat less (Figure III-16) than that which occurred from exposure to the high-temperature (290° F) brine, Figure III-17. However, in both cases deterioration of the surfaces was sufficient to preclude the immediate use of unprotected concrete in areas of distilled water exposure.

Another important observation to be made from the distilled water tests is that apparently no additional resistance to leaching is developed by the concrete because of extended periods of moist curing. The top photographs in Figures III-16 and III-17 show distilled water leaching of concrete having had 28 days' moist cure; whereas, the two bottom photographs are specimens having had the same distilled water exposure but a 180-day moist cure. No significant differences can be detected. The differences in amount of leaching between companion specimens resulted from our equipment configuration and not from any differences in the concrete. Specimens containing quartz aggregate have an "X" in the identification number.

Realizing, from the aforementioned information, that the leaching which was taking place was probably free lime, doubt still remained as to whether leaching of the matrix by distilled water would cease if all available free lime were eliminated. A petrographic analysis of the concrete subjected to the high-temperature brine indicated that no free lime existed in those specimens after the first 28 days of exposure. Therefore, a number of 1-inch-thick slices of this concrete, which had 28 days' moist cure, 92 days' drying and 180 days in the 290° F brine, were exposed for 126 days to the 150° F distilled water. Leaching of the matrix was apparently as severe as that which occurred on the previously discussed specimens, Figure III-18.

Table III-22

REPORT OF FREE LIME CONTENT (CaO) OF CUBES OF NEAT CEMENT AND CEMENT-POZZOLAN MIXTURES AT VARIOUS AGES AND UNDER VARIOUS CURING CONDITIONS

Cement-fly ash--B8946, Fly ash--M4962, Cement--M-4986
Corrosion of Concrete Investigation for OSW

Test age, days	Type of cure	Sample No.	%1/ cement	% pozzolan	% free lime from cement fraction	% free2/ lime from pozzolan fraction	% total free lime from cement and pozzolan (theoretical)	% free lime found in cubes	Change in free lime content, %
3	Fog	CFI-01	100	0.0	11.11	0.00	11.11	11.11	0.00
		-11	90.9	9.1	10.10	0.25	10.35	11.05	+0.70
		-21	83.3	16.7	9.25	0.47	9.72	9.89	+0.17
		-31	76.9	23.1	8.54	0.65	9.19	9.64	+0.45
28	Fog	CFI-02	100	0.0	12.16	0.00	12.16	12.16	0.00
		-12	90.0	9.1	11.05	0.25	11.30	11.12	-0.18
		-22	83.3	16.7	10.13	0.47	10.60	9.90	-0.70
		-32	76.9	23.1	9.35	0.65	10.00	9.36	-0.64
56	28 Fog	CFI-03	100	0.0	13.31	0.00	13.31	13.31	0.00
	+	-13	90.9	9.1	12.10	0.25	12.35	11.47	-0.88
	28 Air	-23	83.3	16.7	11.09	0.47	11.56	11.02	-0.54
		-33	76.9	23.1	10.23	0.65	10.88	9.70	-1.18
56	Fog	CFI-04	100	0.0	12.74	0.00	12.74	12.74	0.00
		-14	90.9	9.1	11.58	0.25	11.83	11.11	-0.72
		-24	83.3	16.7	10.61	0.47	11.08	9.65	-1.43
		-34	76.9	23.1	9.80	0.65	10.45	9.50	-0.95
210	Fog	CFI-05	100	0.0	14.48	0.00	14.48	14.48	0.00
		-15	90.9	9.1	13.16	0.25	13.41	11.99	-1.42
		-25	83.3	16.7	12.06	0.47	12.53	9.83	-2.70
		-35	76.9	23.1	11.13	0.65	11.78	9.14	-2.64
553	Fog	CFI-06	100	0.0	15.02	0.00	15.02	15.02	0.00
		-16	90.9	9.1	13.65	0.25	13.90	12.20	-1.70
		-26	83.3	16.7	12.51	0.47	12.98	10.23	-2.75
		-36	76.9	23.1	11.55	0.65	12.20	9.14	-3.06

1/Water to cement plus pozzolan ratio for all mixes was 0.40
2/Free lime in fly ash = 2.80 percent

Table III-23

REPORT OF FREE LIME CONTENT OF CONCRETE BARS CUT FROM ORIGINAL SPECIMEN NO. CCI-DWL-12 AND CURED UNDER VARIOUS CONDITIONS

Corrosion of Concrete Investigation for OSW

Set No.	Specimen No.	Date cast	Length of cure, days		Test date	Percent[1] free lime
			Fog	Dry		
1	1	9-21-64	28	0	10-19-64	1.72
	2		28	28	11-16-64	1.98
	3		28	154	3-22-65	1.60
2	1	9-21-64	56	0	11-16-64	1.86
	2		56	28	12-14-64	1.38
	3		56	154	4-19-65	1.71
3	1	9-21-64	154	0	2-22-65	1.50
	2		154	28	3-22-65	1.50
	3		154	154	7-26-65	1.74
4	1	9-21-64	182	0	3-22-65	1.60
	2		182	28	4-19-65	1.51
	3		182	154	8-23-65	1.52
5	1	9-21-64	280	0	6-28-65	1.63
	2		280	28	7-26-65	1.48
	3		280	154	11-29-65	1.30

[1] Percent, by weight, of test sample.

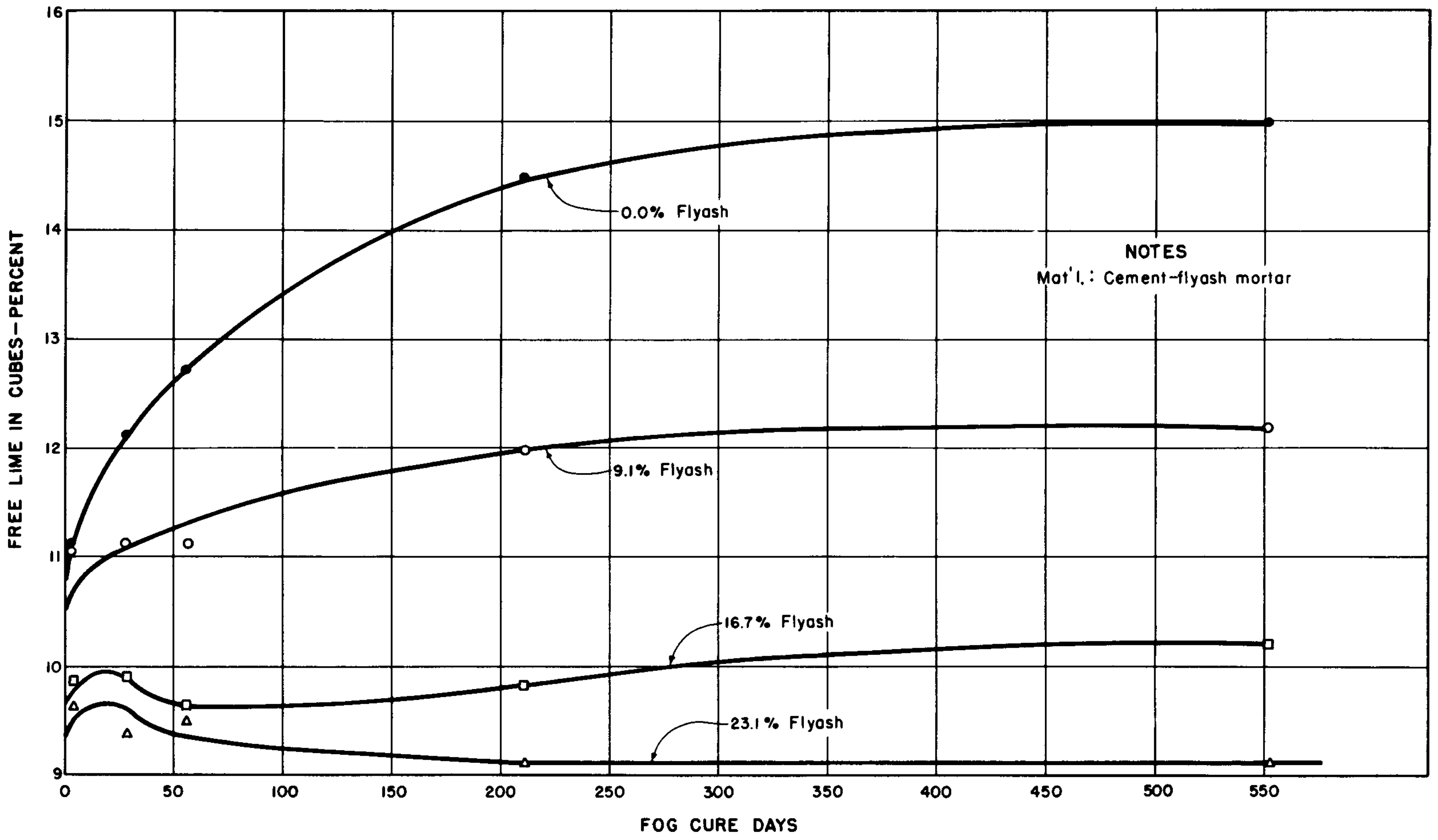

FIGURE III-14: THE QUANITITY OF FREE LIME FOUND IN CONCRETE VARIES WITH THE PERCENTAGE OF FLYASH POZZOLAN USED AND THE LENGHT OF MOIST CURE

FIGURE III-15

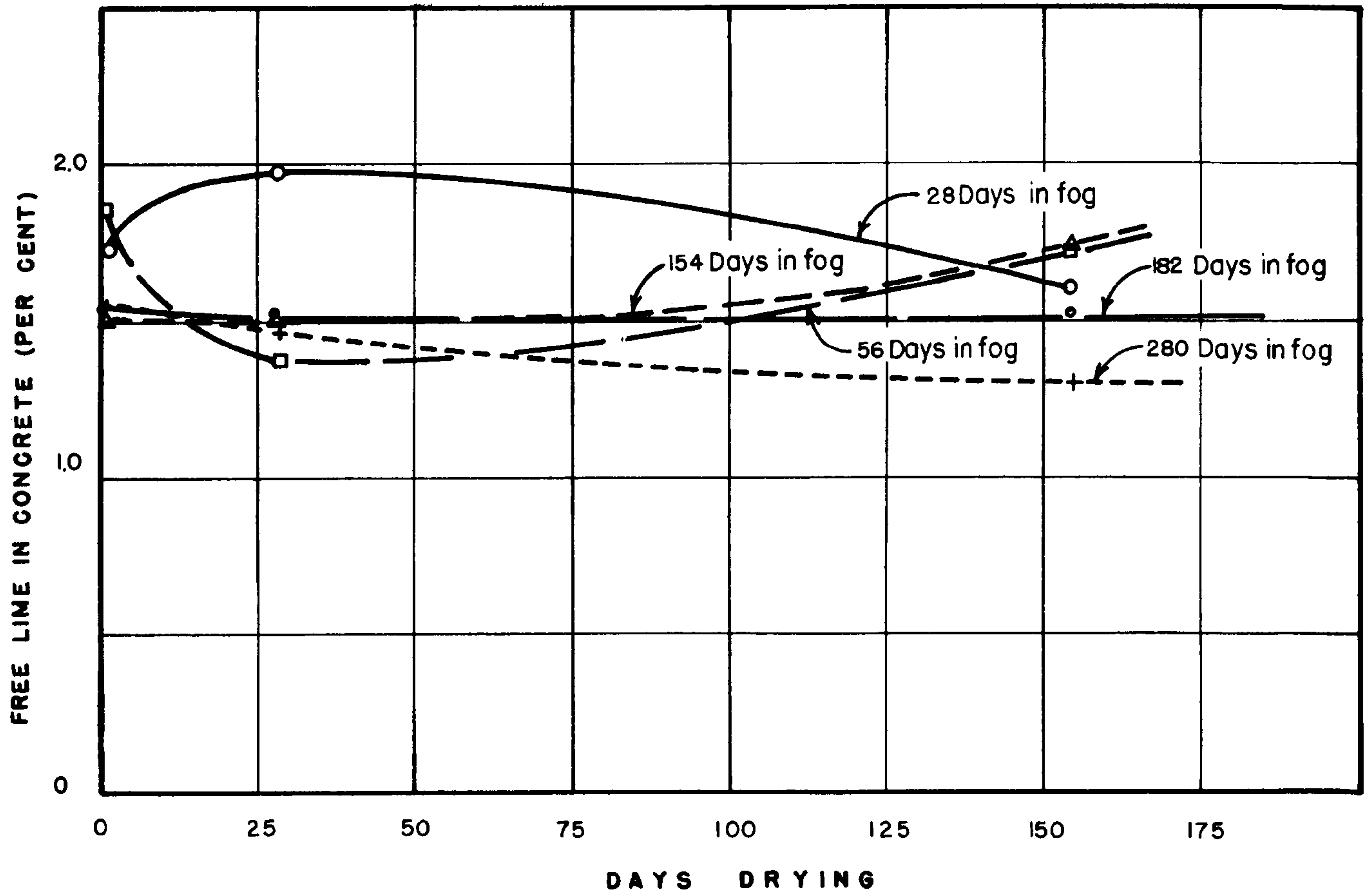

FIGURE III-15: The quantity of free lime found in cement flyash concrete varies with the length of moist cure.

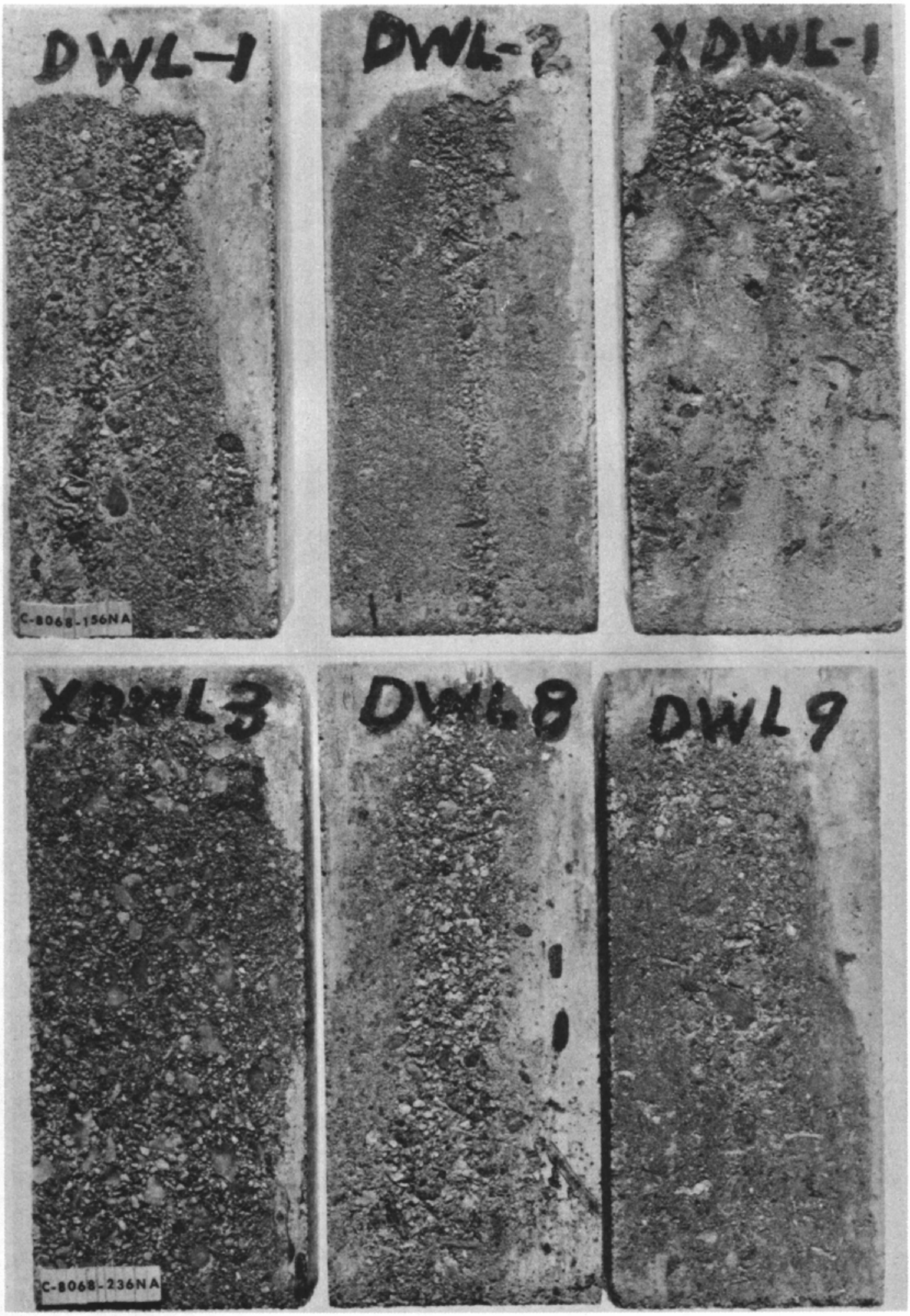

Figure III-16. Concrete having received 28 days' initial fog cure (top photo) appears to be equally as resistant to 126 days' exposure to 150° F distilled water as concrete having had an initial 180 days' fog cure (bottom photo). Photos (top) PX-D-55243 NA and (bottom) PX-D-55244 NA

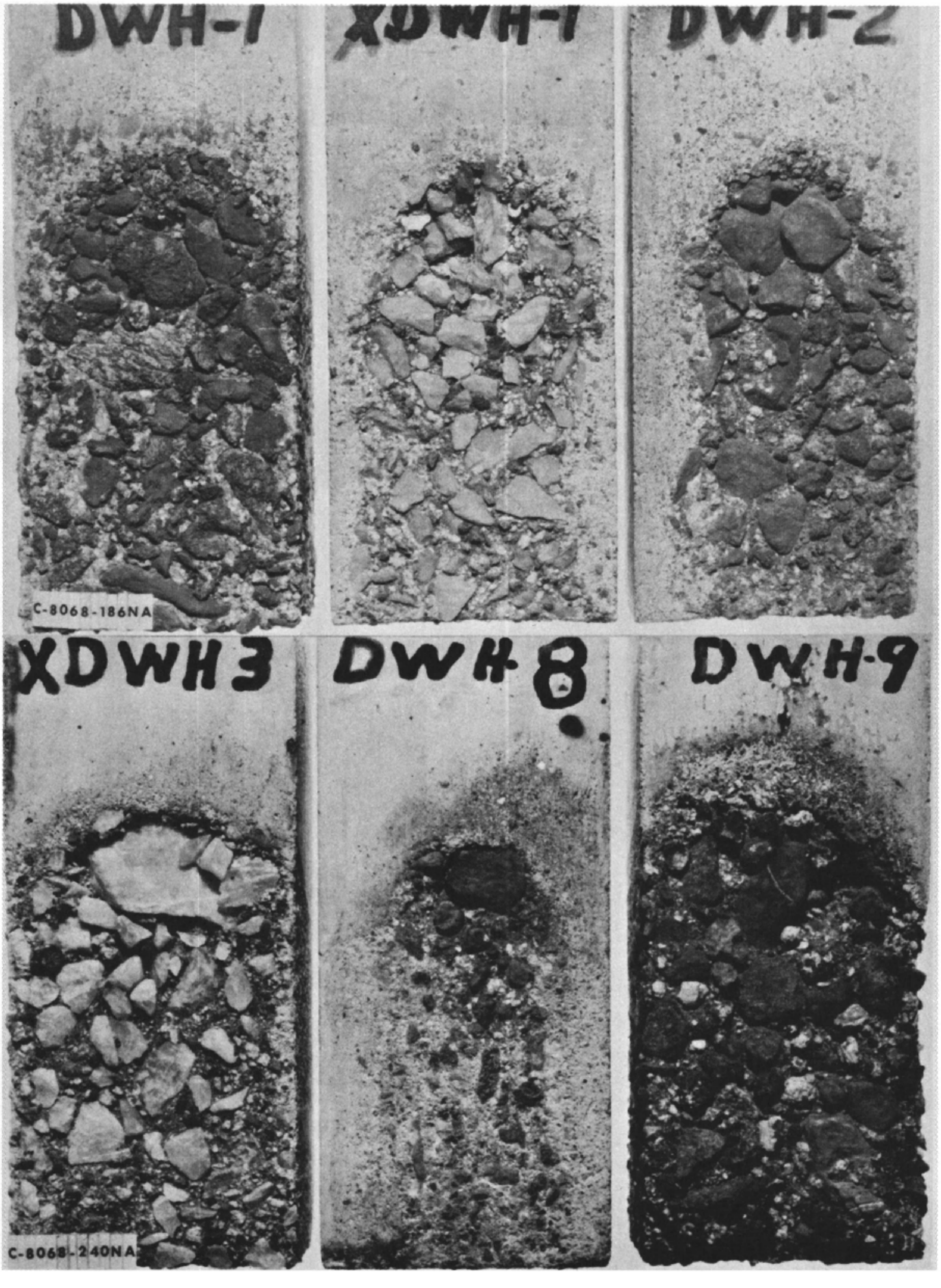

Figure III-17. Concrete having received 28 days' initial fog cure (top photo) appears to be equally as resistant to 126 days' exposure to 350° F distilled water as concrete having had an initial 180-day fog cure (bottom photo). Photos (top) PX-D-55241 NA and (bottom) PX-D-55242 NA

Figure III-18. Concrete containing no available free lime is still susceptible to leaching by hot distilled water. Photo PX-D-55290 NA

III. 6. 2. Synthetic Sea Water Tests

III. 6. 2. 1. Conclusions. --

Based upon the results of synthetic sea water tests conducted upon concrete exposed to the five environments the following conclusions seem appropriate:

1. Natural aggregate concrete has not been detrimentally affected by 2 years' exposure to the 100° F synthetic sea water brine.

2. Natural aggregate concrete has undergone peripheral deterioration to a depth of about 5 to 8 mm as a result of 1.5 year's exposure to 290° F synthetic sea water brine.

3. Concrete on the interior of the specimens subjected to the 290° F brine apparently remained sound throughout the 1.5 years' exposure, although exhibiting somewhat less compressive strength.

4. Natural aggregate and crushed limestone aggregate concretes have not been detrimentally affected by 180 days' exposure to the 203°, 225°, or 250° F synthetic sea water brine, although some minor surface alteration has occurred on all of the specimens.

5. No cavitation damage was evident in the 4- by 9- by 8-inch concrete orifices after 2 years' exposure to 100° F brine or 1 year's exposure to 290° F brine flowing at 10 feet per second. However, the invert surface of the orifice in the hot loop has been affected adversely by erosion of the altered peripheral concrete.

6. Compressive strength of the concrete exposed to the 100° F brine environment increased with age from 7, 390 psi after 28 days in brine to 8, 770 psi after 18 months in brine; whereas, for the same two test ages compressive strength of companion control specimens (28 days' fog plus continuous drying) decreased with age from 7, 960 to 7, 740 psi, respectively.

7. Compressive strength of the concrete exposed to the 290° F brine environment is adversely affected, decreasing with age from 7, 590 psi after 31 days in brine to 6, 150 psi after 18 months in brine. Compressive strengths of companion control specimens (28 days in fog followed by continuous drying) varied somewhat over the test time, but not significantly. The retrogression in compressive strength of the brine-exposed concrete apparently resulted in part from the reduction in cross-sectional area of the sound concrete.

8. In general, both the natural aggregate and crushed limestone aggregate concretes, with one exception, exhibited compressive strength increases during 180 days' exposure to the 203°, 225°, and 250° F brines.

9. Moduli of elasticity of the natural aggregate concrete subjected to the five environments gradually increased with time, and were greater than the moduli of the continuously dried control concrete.

10. Poissons' ratio of concretes subjected to the 100° and 290° F brine environments increased very slightly with age, high-temperature concrete having a ratio about 15.8 percent greater at 1 year's exposure than the low-temperature concrete. Poissons' ratio of the continuously dried concrete was somewhat less than the concrete in either of the brine environments.

11. Moduli of elasticity of the crushed limestone aggregate concrete followed trends similar to the aforementioned natural aggregate concrete through 90 days' exposure to the 203°, 225°, and 250° F environments, however, after 180 days' exposure, they have shown a slight decrease.

12. Pulse velocity values of control concrete as well as those of all concrete subjected to the five brine environments have remained generally above 15, 000 feet per second during the entire exposure and indicate very good quality concrete.

13. Natural aggregate concrete exposed for 18 months and 1 year to the 100° and 290° F brine environments, respectively, continues to show a slight increase in unit weight, an indication of continuing absorption of moisture into the concrete.

14. No deleterious sulfate attack, as measured by abnormal expansion, has taken place in any of the concretes under any of the exposures.

III. 6. 2. 2. Discussion. --

(Note: A detailed petrographic discussion of the microstructural aspects of the tested concrete can be found in Section VII. Also a detailed account of the metal corrosion can be found in Section VI.)

III. 6. 2. 2. 1. 100° F brine tests. --

The general condition of the concrete subjected for 2 years to the 100° F synthetic sea water brine is excellent. Concrete specimens have shown no signs of deterioration of any type; further, all dynamic and static tests indicate that the concrete remains sound. This concrete has had an extraneous coating material adhering to the surfaces from initial exposure to the brine; however, this coating, as described in Section VII, is not a result of nor does it indicate any deterioration of the concrete surface.

Compressive strengths of the concrete increased with continued 100° F brine exposure to 8, 980 psi at 1 year, then dropped slightly to 8, 770 psi at 547 days. This 547-day strength is 18. 7 percent greater than the 28-day strength, Table III-24 and Figure III-19. Continuously dried control concrete (all specimens received an initial 28-day fog cure) showed a steady reduction in compressive strength from the maximum of 7, 960 psi at 148 days' age. This reduction continued to at least 854 days' total age. Such compressive strength drop is not unusual and is not considered particularly significant since the rate of decrease is leveling off and even the lowest strength is far greater than that generally required for design purposes.

Compressive strength development of continuously moist-cured concrete for control purposes was considered secondary in importance to that of concrete cured similarly to the potential prototype, and, therefore, only sufficient specimens were fabricated to provide test results to 1 year's total age. Compressive strength of this continuously moist-cured concrete was 9, 600 psi at 1 year's age.

Values of moduli of elasticity and Poisson's ratio of the control concrete and the concrete subjected to the 100° F brine generally increased with time and were in range normally expected of good quality concrete, Table III-24 and Figure III-20. The values recorded are well above average for good concrete and indicate no deterioration of any kind.

As might be expected, the unit weight of the concrete exposed for 18 months to the 100° F synthetic sea water brine increased somewhat to 159. 7 pounds per cubic foot while the unit weight of the companion drying concrete decreased slightly to 155. 1 pounds per cubic foot, Table III-24. This indicates that the concretes are still absorbing and expelling water or solution, respectively, although not to any detriment.

Other nondestructive tests, dynamic modulus and sonic, or pulse, velocity were conducted upon the concrete to determine its soundness and internal integrity. Briefly, the pulse velocity test involves transmitting then measuring the time it takes a sound wave to travel through the specimen. The pulse velocity of the concrete exposed to the 100° F synthetic sea water brine has remained above 15, 000 feet per second (Table III-24 and Figure III-22) which by accepted standards indicates very good quality concrete. That of the control drying concrete has been but slightly less, between 14, 000 and 15, 000 feet per second, which indicates good quality concretes with no deterioration apparent. Rather than use the resonant frequency, weight, and physical dimensions of the concrete specimens to compute its dynamic modulus, we are using the accepted practice of evaluating concrete soundness by the change in the square of the resonant frequency, Table III-25, Figure III-21. Resonant frequency of the concrete subjected

Table III-24

COMPRESSIVE STRENGTH AND ELASTIC PROPERTIES
6- by 12-inch Concrete Cylinders
100° F Low-temperature System
Corrosion of Concrete Investigation For OSW

Specimen No.	Fog cure (days)	Dry cure (days)	In brine (days)	Total age (days)	Compressive strength (psi)	E (million psi)	r	Unit weight pounds per cu foot	Pulse velocity foot per second
CCI-L1	28	92	28	148	7,910	4.82	0.17	155.8	15,100
CCI-L2	28	92	28	148	6,010	4.54	0.14	156.4	14,700
CCI-L3	28	92	28	148	8,260	5.54	0.20	158.2	16,100
Average	28	92	28	148	7,390	4.97	0.17	156.8	15,300
CCI-L1C	28	120	0	148	8,070	4.71	0.14	153.9	15,700
CCI-L2C	28	120	0	148	7,150	4.91	0.17	157.6	15,400
CCI-L3C	28	120	0	148	8,670	4.90	0.13	160.1	15,600
Average	28	120	0	148	7,960	4.84	0.15	157.2	15,600
CCI-L4	28	99	90	217	7,600	5.26	0.19	157.6	15,800
CCI-L5	28	99	90	217	7,390	4.97	0.19	155.1	15,600
CCI-L6	28	99	90	217	8,560	5.43	0.19	157.0	16,100
Average	28	99	90	217	7,850	5.22	0.19	156.6	15,800
CCI-L4C	28	189	0	217	7,680	4.63	0.17	156.4	15,300
CCI-L5C	28	189	0	217	7,680	4.59	0.17	153.9	14,900
CCI-L6C	28	189	0	217	8,450	4.66	0.17	157.0	14,900
Average	28	189	0	217	7,940	4.63	0.17	155.8	15,000
CCI-L7	28	101	180	309	7,990	5.28	0.17	159.5	16,800
CCI-L8	28	101	180	309	8,420	5.32	0.14	160.1	16,900
CCI-L9	28	101	180	309	8,770	5.64	0.19	160.7	17,000
Average	28	101	180	309	8,390	5.41	0.17	160.1	16,900
CCI-L7C	28	281	0	309	6,620	3.99	0.15	157.0	13,900
CCI-L8C	28	281	0	309	7,530	4.37	0.14	157.0	14,400
CCI-L9C	28	281	0	309	8,370	4.47	0.13	158.2	14,400
Average	28	281	0	309	7,510	4.28	0.14	157.4	14,200

Table III-24--Continued

COMPRESSIVE STRENGTH AND ELASTIC PROPERTIES
6- by 12-inch Concrete Cylinders
100° F Low-temperature System
Corrosion of Concrete Investigation for OSW

Specimen No.	Fog cure (days)	Dry cure (days)	In brine (days)	Total age (days)	Compressive strength (psi)	E (million psi)	r	Unit weight pounds per cubic foot	Pulse velocity foot per second
CCI-L10	28	102	270	400	7,430	5.24	0.18	158.2	15,100
CCI-L11	28	102	270	400	8,600	5.70	0.19	158.2	16,100
CCI-L12	28	102	270	400	9,270	6.00	0.20	158.2	15,600
Average					8,430	5.65	0.19	158.2	15,600
CCI-L10C	28	372	0	400	6,440	4.30	0.17	155.8	14,400
CCI-L11C	28	372	0	400	7,610	4.51	0.17	155.8	14,600
CCI-L12C	28	372	0	400	8,220	4.86	0.17	156.4	14,900
Average					7,420	4.56	0.17	156.0	14,600
CCI-L13	28	135	365	528	8,540	5.83	0.20	157.6	16,100
CCI-L14	28	135	365	528	8,880	5.84	0.19	160.1	16,300
CCI-L15	28	135	365	528	9,520	5.61	0.19	160.7	16,200
Average					8,980	5.76	0.19	159.5	16,200
CCI-L13C	28	500	0	528	7,780	4.60	0.14	154.5	14,600
CCI-L14C	28	500	0	528	6,560	4.36	0.16	153.9	14,400
CCI-L15C	28	500	0	528	7,570	4.64	0.14	158.9	14,700
Average					7,300	4.53	0.15	155.8	14,600
CCI-L16	28	259	547	834	8,750	5.74	0.19	158.2	16,100
CCI-L17	28	259	547	834	9,040	5.96	0.16	160.1	15,900
CCI-L18	28	259	547	834	8,510	5.72	0.18	160.7	16,500
Average					8,770	5.81	0.18	159.7	16,200
CCI-L16C	28	806	0	834	7,840	4.53	0.14	153.9	15,100
CCI-L17C	28	806	0	834	7,130	4.46	0.16	154.5	14,900
CCI-L18C	28	806	0	834	8,260	4.99	0.14	157.0	14,900
Average					7,740	4.66	0.15	155.1	15,000

Table III-25

FREQUENCY SQUARED OF NATURAL AGGREGATE CONCRETE
4- by 4- by 30-inch Bars
100° F System
Corrosion of Concrete Investigation for OSW

			5/$F^2 \times 10^{-5}$		8/$F^2 \times 10^{-5}$		9/$F^2 \times 10^{-5}$	
Days1/ drying	Days brine	Total age	Spec 1 & 2	Spec 1c & 2c	Spec 3 & 4	Spec 3c & 4c	Spec 5 & 6	Spec 5c & 6c
0	0	28	4/4.72	4/4.62	4.80	4.69	3.60	3.54
7	0	35	3.97	4.40	4.90	4.90	3.64	3.60
14	0	42	4.32	4.42	4.56	4.62	3.54	3.52
21	0	49	4.36	4.45	4.66	4.66	3.64	3.60
35	0	63	4.36	4.49	4.69	4.53	3.60	3.60
49	0	77	4.26	4.42	4.49	4.40	3.64	3.60
63	0	91	4.29	4.36	4.29	4.56	3.54	3.60
88	0	116	4.10	4.33	4.29	4.29	3.60	3.58
2/120	3/28	148	4.72	4.62	6/4.83	4.76	7/3.84	3.58
189	90	217	5.08	4.22	5.04	4.06	4.29	3.58
281	180	309	5.16	4.16	5.26	4.05	4.42	3.60
372	270	400	5.21	4.20	5.04	4.03	4.49	3.62
465	365	493	5.21	4.07	5.33	4.03	4.56	3.72
721	547	749	5.18	4.33	5.40	4.22	4.56	3.72
934	730	962	5.28	4.29	5.40	4.20	4.62	3.74

1/All specimens received an initial 28-day fog cure.

2/Days drying from here on only applies to Specimens 1c and 2c, 3c and 4c, 5c and 6c.

3/Days brine only applies to Specimens 1 and 2, 3 and 4, 5 and 6.

4/These values are F_0 as referred to in Figure III-14.

5/Units for frequency (F) are cycles per second.

6/No. 3 specimen only from here on.

7/No. 5 specimen only from here on.

8/Reinforced concrete specimens.

9/Pretensioned concrete specimens.

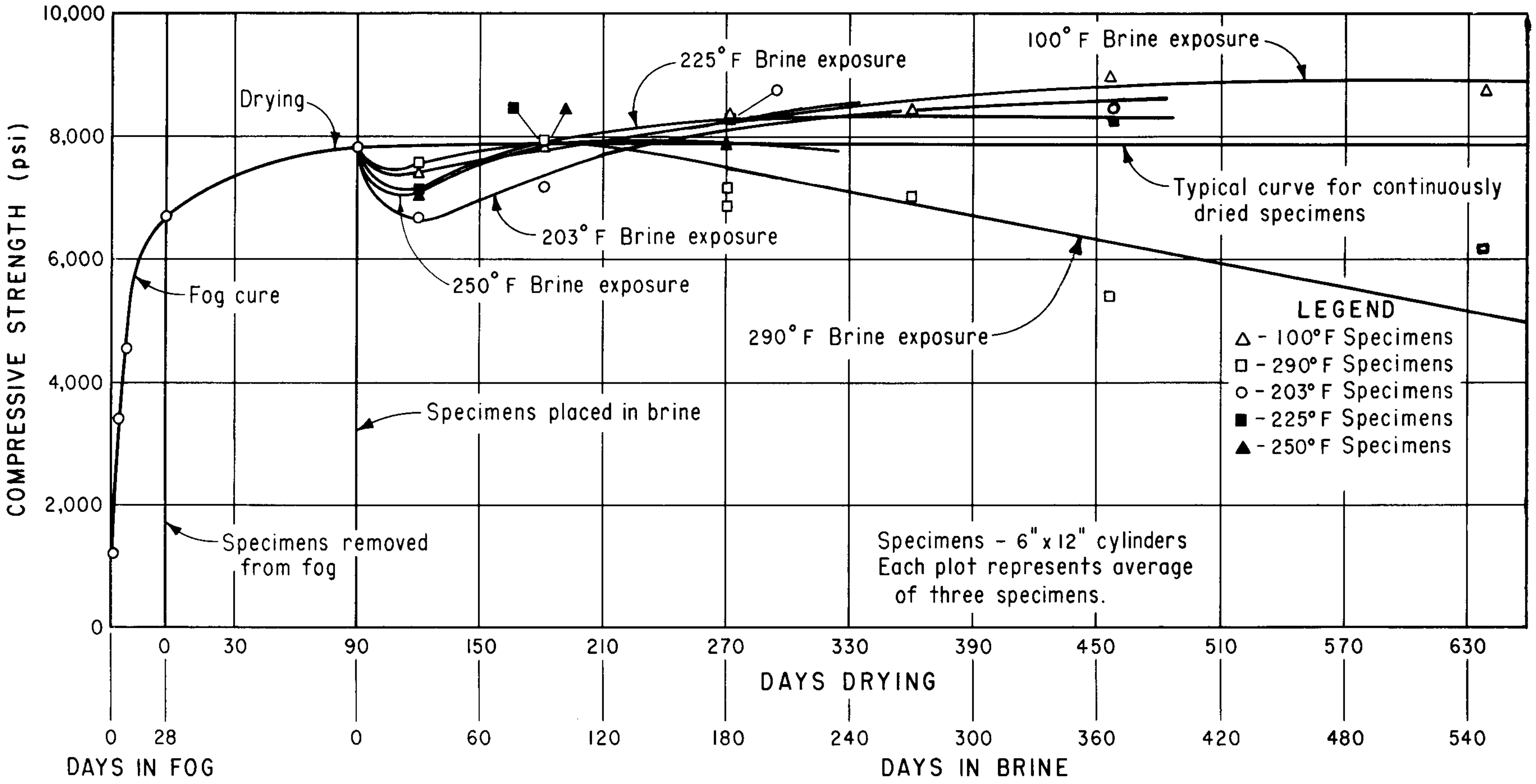

FIGURE III-19 - COMPRESSIVE STRENGTHS OF NATURAL AGGREGATE CONCRETES SUBJECTED TO FOG, DRYING, AND VARIOUS BRINE ENVIRONMENTS

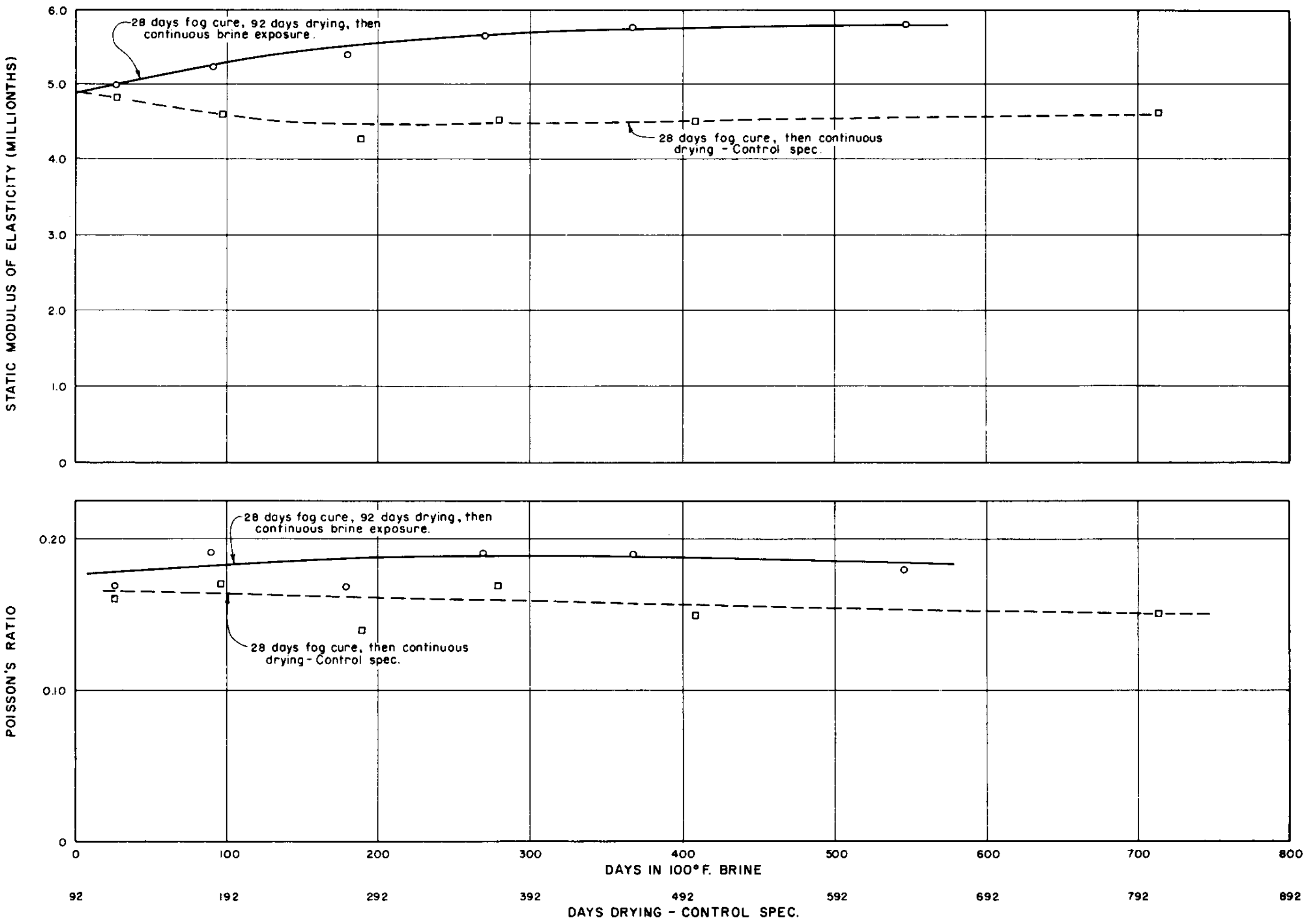

FIGURE III - 20: MODULUS OF ELASTICITY AND POISSON'S RATIO OF NATURAL AGGREGATE CONCRETE SUBJECTED TO 100°F SYNTHETIC SEA WATER BRINE, AND CONTINUOUSLY DRIED CONTROL CONCRETE. POINTS PLOTTED AT EACH AGE REPRESENTS THE AVERAGE OF THREE DIFFERENT SPECIMENS.

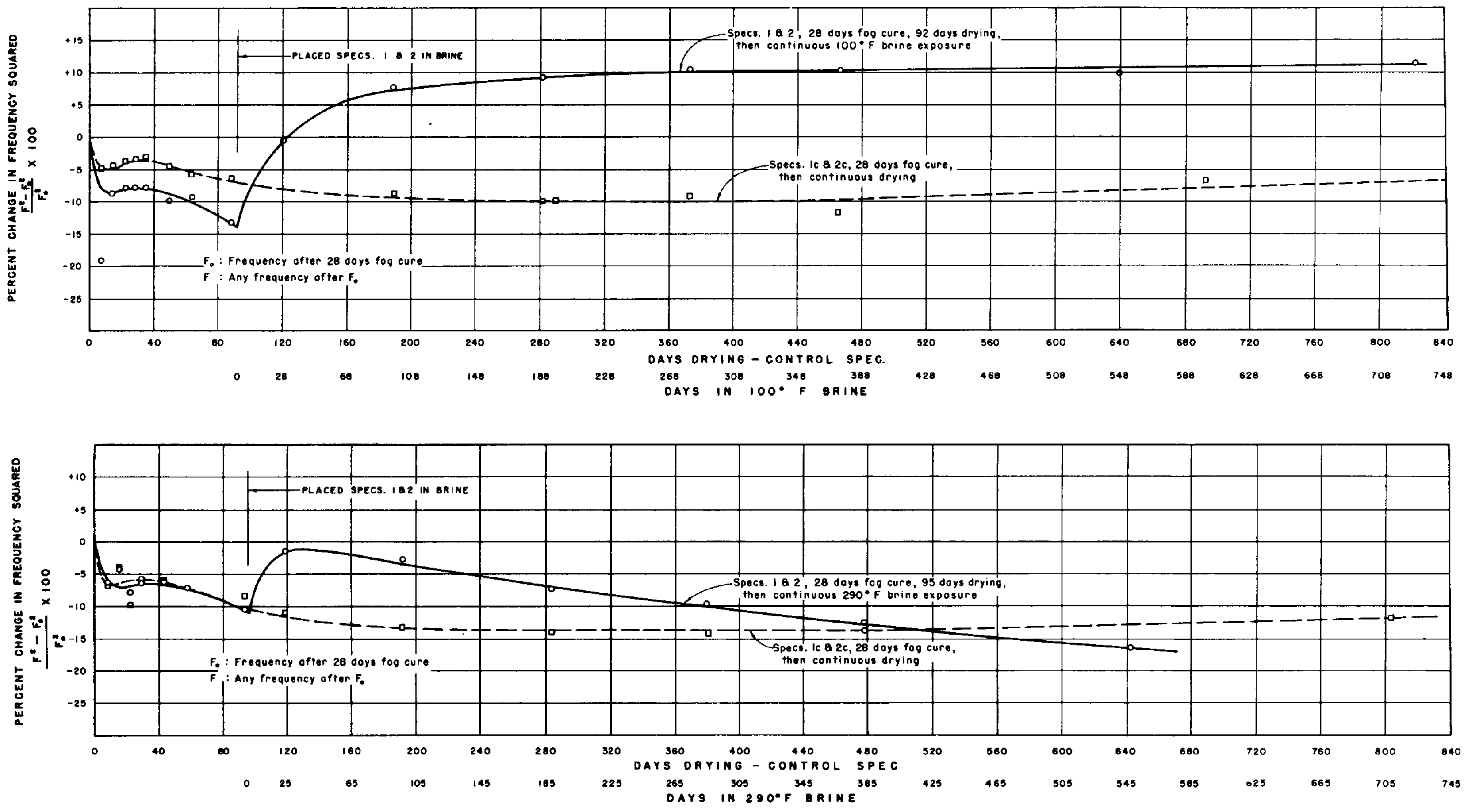

FIGURE III-21: THE CHANGE IN THE SQUARE OF THE FREQUENCY OF NATURAL AGGREGATE CONCRETE OVER A PERIOD OF TIME CAN BE INDICATIVE OF IT'S SOUNDNESS

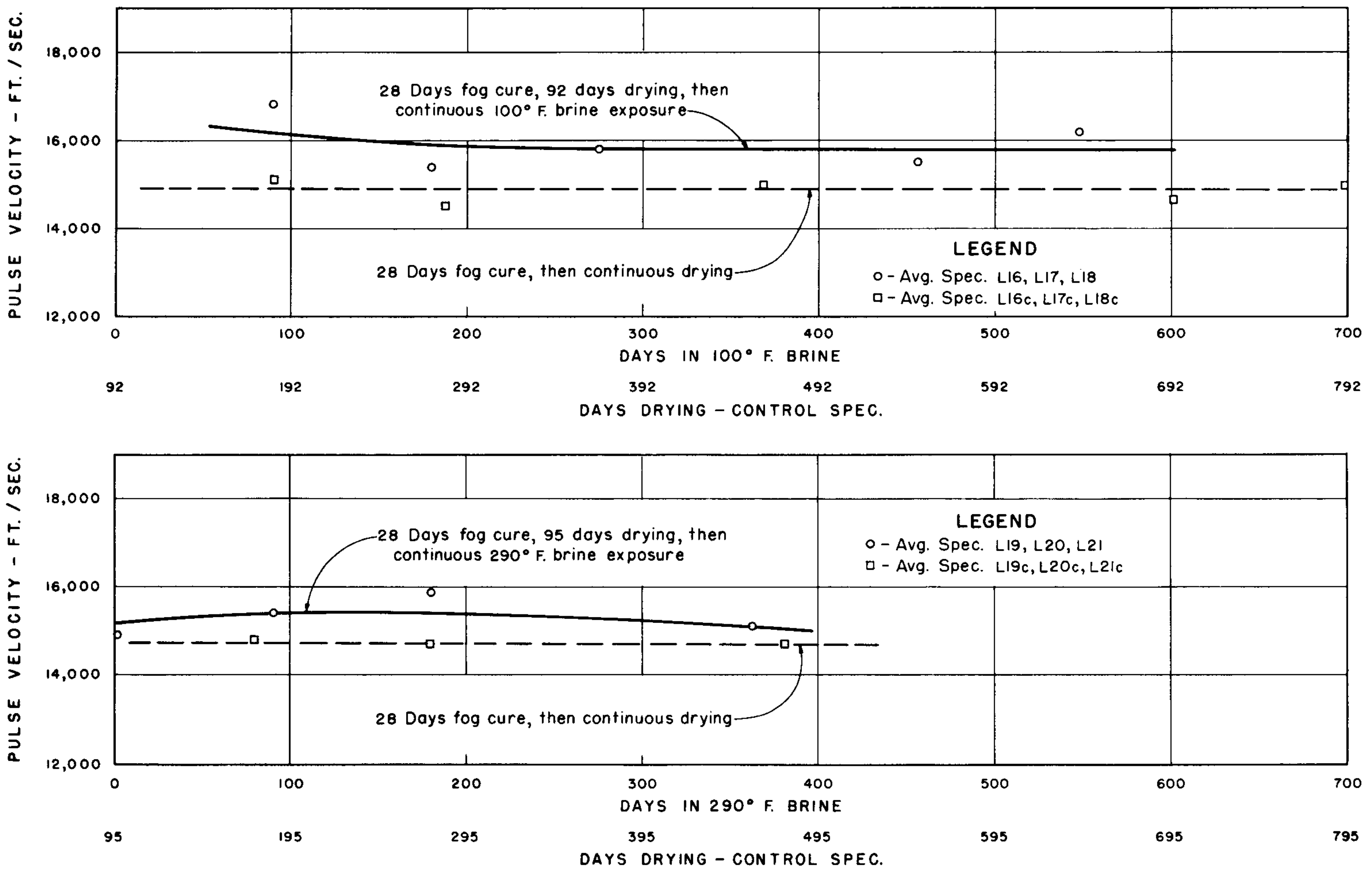

FIGURE III -22: PULSE VELOCITY OF NATURAL AGGREGATE CONCRETE SUBJECTED TO 100°F. AND 290°F. SYNTHETIC SEA WATER BRINE, AND CONTINUOUSLY DRIED CONTROL CONCRETE to the 100° F brine exhibited an initial rapid increase due, in part, to the absorbing of moisture into the concrete, and in part, by the continuation of hydration. This increase which continued slightly throughout the year of exposure is indicative of the continuing strength gain and accompanying internal and external soundness of the concrete. Resonant frequency of the continuously drying control concrete followed normal and expected trends throughout the testing period.

Determination of length and weight change of the concrete at periodic intervals provides another valuable tool to be used in evaluating the effects of the brine environment upon concrete. Data obtained from tests conducted upon the plain concrete bars indicated that the drying shrinkage of the concrete prior to brine exposure totaled between 500 and 550 millionths inch per inch which value is about average for high-quality concrete. After immersion in the 100° F brine for 28 days about half of this shrinkage, 271 millionths inch per inch, was regained, but subsequent length changes were negligible, Table III-26, Figure III-23.

Weight change of the concrete behaved very similarly to the length change in response to drying and immersion in 100° F brine, Table III-27 and Figure III-23. Both of these measurements increased when moisture was present and decreased when moisture was removed. This leads to the conclusion that apparently no significant expansion has occurred in the concrete as a result of chemical reaction between the cement matrix and the soluble sulfates in the synthetic sea water brine.

The relative ratio of length and weight change which occurred in the reinforced concrete was practically identical to those which occurred in the plain concrete except for magnitude, Tables III-26, III-27, and Figure III-24. Maximum drying shrinkage prior to brine exposure was about 390 millionths inch per inch of which half is regained after 1 year's exposure to the 100° F brine. Between 1 year and 18 months, a slight expansion was noted; however, at this point no abnormal length or weight changes have taken place as a result of sulfate attack or reinforcing steel corrosion.

As might be expected, the length change (shrinkage) of the pretensioned concrete during the drying period was greater than that of the plain concrete, and the expansion due to immersion in the brine environment was reduced as compared to that of the plain concrete. Stress in the concrete was determined through the use of SR-4 strain gages mounted on the 1/4-inch pretensioning wire within the concrete. Although the strain gages on these bars were lost during subsequent exposure to the hot brine, while usable, they indicated a stress in the concrete of about 750 and 960 psi at 90 days' drying and 90 days' drying plus 90 days in brine, respectively. The maximum shrinkage of 636 millionths inch per inch occurred in the pretensioned concrete after about 92 days' drying while the net shrinkage after 18 months' exposure to the 100° F brine was also about 594 millionths inch per inch, Table III-26 and Figure III-25. Continuously dried companion concrete underwent a total shrinkage of 827 millionths inch per inch after 693 days' drying.

Weight change which was very similar to that which occurred in the plain concrete is tabulated in Table III-27 and presented graphically in Figure III-25. One each of the 4- by 4- by 30-inch reinforced and pretensioned concrete bars were intentionally cracked in flexure prior to the brine exposure to simulate a potential crack in the prototype structure. The primary function of these particular specimens was to provide information regarding corrosion rates of reinforcing and pretensioning steel. Length change of these cracked specimens was of no significance and was not plotted in the figures.

Length and weight change measurements were obtained from concrete which had been coated with three different materials prior to the brine exposure, Tables III-26, III-27, and Figures III-26, III-27, and III-28. From these data a few important observations can be drawn. The silicone alkyd coating (Coating A) apparently did very little to prevent absorption of the brine by the coated specimen. This is evident from both the immediate expansion and weight increase experienced by the concrete within the first 30 days of brine exposure. Very little length or weight change occurred after the first 30 days of 100° F brine exposure.

Table III-2d

LENGTH CHANGE OF 4- BY 4- BY 30-INCH BARS IN MILLIONTHS INCH PER INCH
100° F Low-temperature Brine System
Corrosion of Concrete Investigation for OSW

Total age (days)		35	42	56	91	116	148	217	309	400	495	721	934	
Cure (days)	Fog	28	28	28	28	28	28	28	28	28	28	28	28	28
	Dry	7 (7)2/	14 (14)	28 (28)	63 (63)	88 (88)	92 (120)	99 (189)	101 (281)	102 (372)	102 (465)	146 (693)	176 (906)	
	Brine	0 (0)2/	0 (0)	0 (0)	0 (0)	0 (0)	28 (0)	90 (0)	180 (0)	270 (0)	365 (0)	547 (0)	730 (0)	
Specimen Number	**Type 1/**													
1 & 2	Pl	234	321	417	509	543	272	258	200	261	350	280	336	
1c & 2c	(Pl)2/	234	304	382	484	511	547	615	561	598	625	588	642	
3	Re	163	236	317	392	390	179	179	137	193	204	137	196	
4	Re	163	236	317	392	390	474/	22	+163/	4	28	+27	21	
3c & 4c	(Re)	138	218	314	388	375	404	447	400	430	440	402	439	
5	Prs	241	336	435	559	636	490	501	559	596	636	594	661	
6	Prs	241	336	435	559	636	5834/	637	605	586	736	687	750	
5c & 6c	(Prs)	224	311	414	563	608	687	736	753	790	857	827	878	
7 & 8	A	202	289	403	538	550	258	255	179	239	318	282	339	
19 & 20	(A)	199	268	357	506	547	558	615	578	605	643	602	653	
9 & 10	B	199	289	378	279	322	304	308	243	264	278	330	354	
21 & 22	(B)	202	286	374	297	368	408	461	464	480	518	488	521	
11 & 12	C	184	275	367	429	443	404	422	346	336	325	315	307	
23 & 24	(C)	184	293	385	522	540	-	-	575	608	640	595	646	

1/Pl = Plain Prs = Prestressed B = Coated with epoxy
Re = Reinforced A = Coated with silicone alkyd C = Coated with glass-filled epoxy
2/All ( ) refer to companion specimens which were not placed in brine, but left in laboratory air.
3/All specimen readings indicate shrinkage compared to the 28-day fog readings except those marked with a "+" which indicates expansion.
4/Broken in flexure before this reading. Bar readings not averaged.

Table III-27

WEIGHT CHANGE OF 4- BY 4- BY 30-INCH BARS IN POUNDS
100° F Low-temperature Brine System
Corrosion of Concrete Investigation for OSW

Total age (days)		35	42	56	91	116	148	217	309	400	495	721	934	
Cure (days)	Fog	28	28	28	28	28	28	28	28	28	28	28	28	28
	Dry	7 (7)2/	14 (14)	28 (28)	63 (63)	88 (88)	92 (120)	99 (189)	101 (281)	102 (372)	102 (465)	146 (693)	176 (906)	
	Brine	0 (0)2/	0 (0)	0 (0)	0 (0)	0 (0)	28 (0)	90 (0)	180 (0)	270 (0)	365 (0)	547 (0)	730 (0)	
Specimen Number	Type 1/													
1 & 2	Pl	0.49	0.60	0.73	0.97	1.07	0.35	0.22	0.13	0.13	0.11	+0.02	+0.06	
1c & 2c	(Pl)2/	0.41	0.56	0.67	0.92	1.01	1.08	1.22	1.24	1.30	1.39	1.34	1.39	
3	Re	0.37	0.49	0.63	0.93	1.01	0.30	0.19	0.12	0.07	0.03	+0.12	+0.16	
4	Re	0.37	0.49	0.63	0.93	1.01	0.254/	0.10	0.07	0.01	+0.01	+0.16	+0.19	
3c & 4c	(Re)	0.34	0.45	0.64	0.92	1.00	1.05	1.18	1.22	1.28	1.38	1.33	1.35	
5	Prs	0.26	0.31	0.39	0.59	0.67	0.09	+0.103/	+0.16	+0.17	+0.26	+0.37	+0.44	
6	Prs	0.26	0.31	0.39	0.59	0.67	+0.184/	+0.30	+0.35	+0.40	+0.41	+0.54	+0.55	
5c & 6c	(Prs)	0.12	0.17	0.26	0.40	0.49	0.57	0.68	0.73	0.77	0.81	0.75	0.83	
7 & 8	A	0.40	0.51	0.64	0.82	0.94	0.19	0.10	0.01	0.01	0.01	+0.12	+0.14	
19 & 20	(A)	0.37	0.44	0.55	0.71	0.84	0.90	1.02	1.05	1.11	1.19	1.19	1.25	
9 & 10	B	0.39	0.51	0.62	1.01	1.06	0.98	0.92	0.84	0.82	0.84	0.77	0.76	
21 & 22	(B)	0.39	0.51	0.61	1.01	1.06	1.10	1.19	1.24	1.29	1.35	1.38	1.47	
11 & 12	C	0.37	0.50	0.60	0.915/	+0.51	+0.55	+0.55	+0.60	+0.63	+0.63	+0.72	+0.89	
23 & 24	(C)	0.37	0.54	0.65	0.965/	1.22	1.226/	1.337/	1.39	1.42	1.49	1.49	1.53	

1/Pl = Plain   Prs = Prestressed   B = Coated with epoxy
Re = Reinforced   A = Coated with silicone alkyd   C = Coated with glass-filled epoxy

2/All ( ) refer to companion specimens which were not placed in brine, but left in laboratory air.

3/All specimen readings indicate weight loss compared to the 28-day fog readings except those marked with a "+" which indicates weight gain.

4/Broken in flexure before this reading. Bar readings not averaged.

5/77-day reading.

6/121-day reading.

7/171-day reading--Bar 23 reading only after this date.

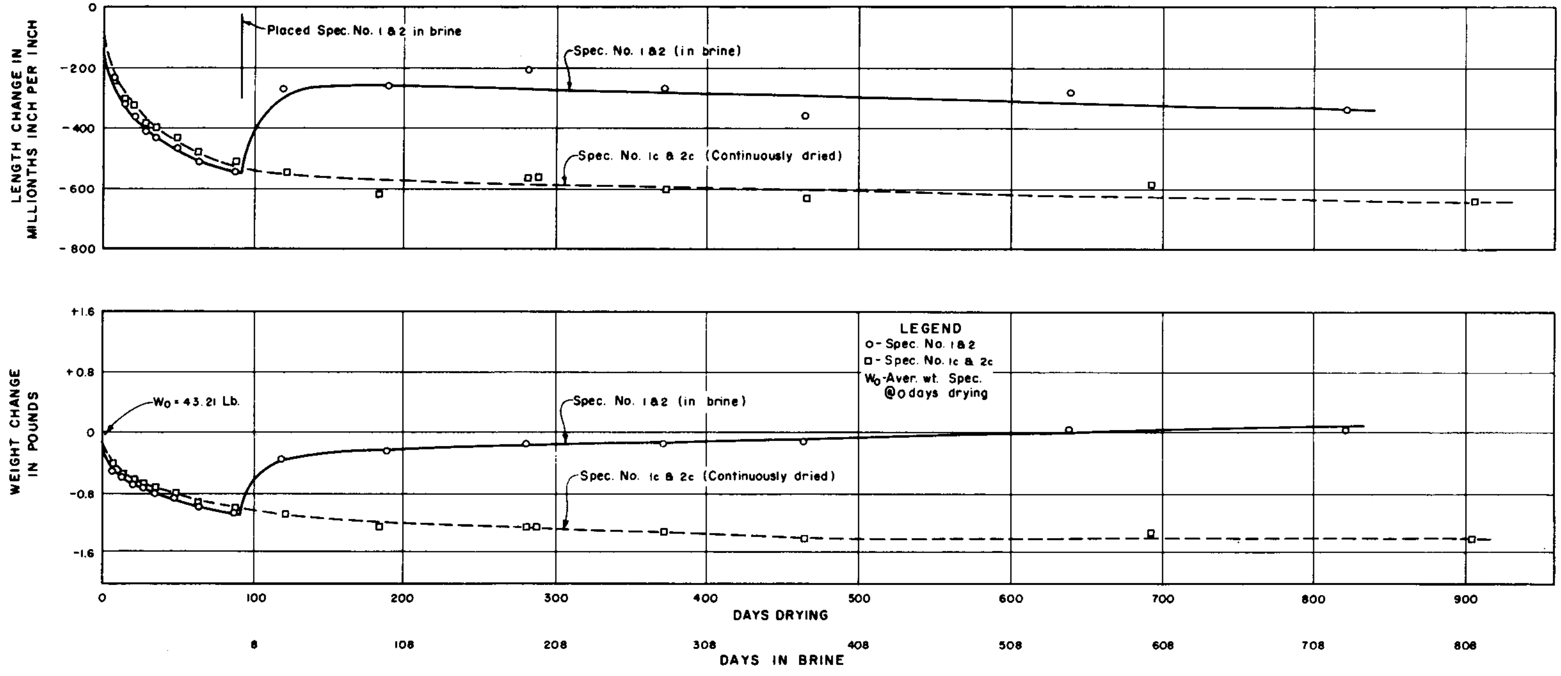

FIGURE III - 23: LENGTH AND WEIGHT CHANGE OF PLAIN NATURAL AGGREGATE CONCRETE SUBJECTED TO 100°F. BRINE AND CONTINUOUS DRYING

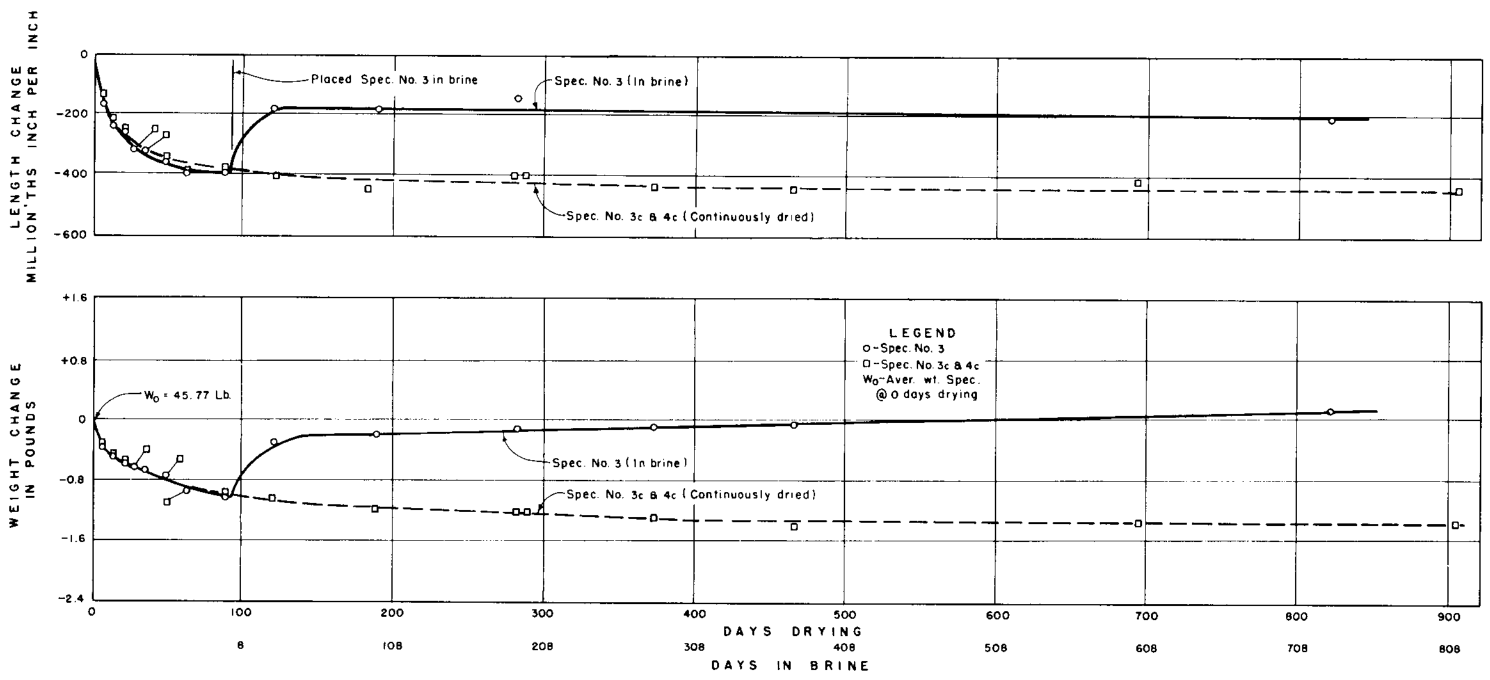

FIGURE III - 24 : LENGTH AND WEIGHT CHANGE OF REINFORCED NATURAL AGGREGATE CONCRETE SUBJECTED TO 100°F. BRINE AND CONTINUOUS DRYING

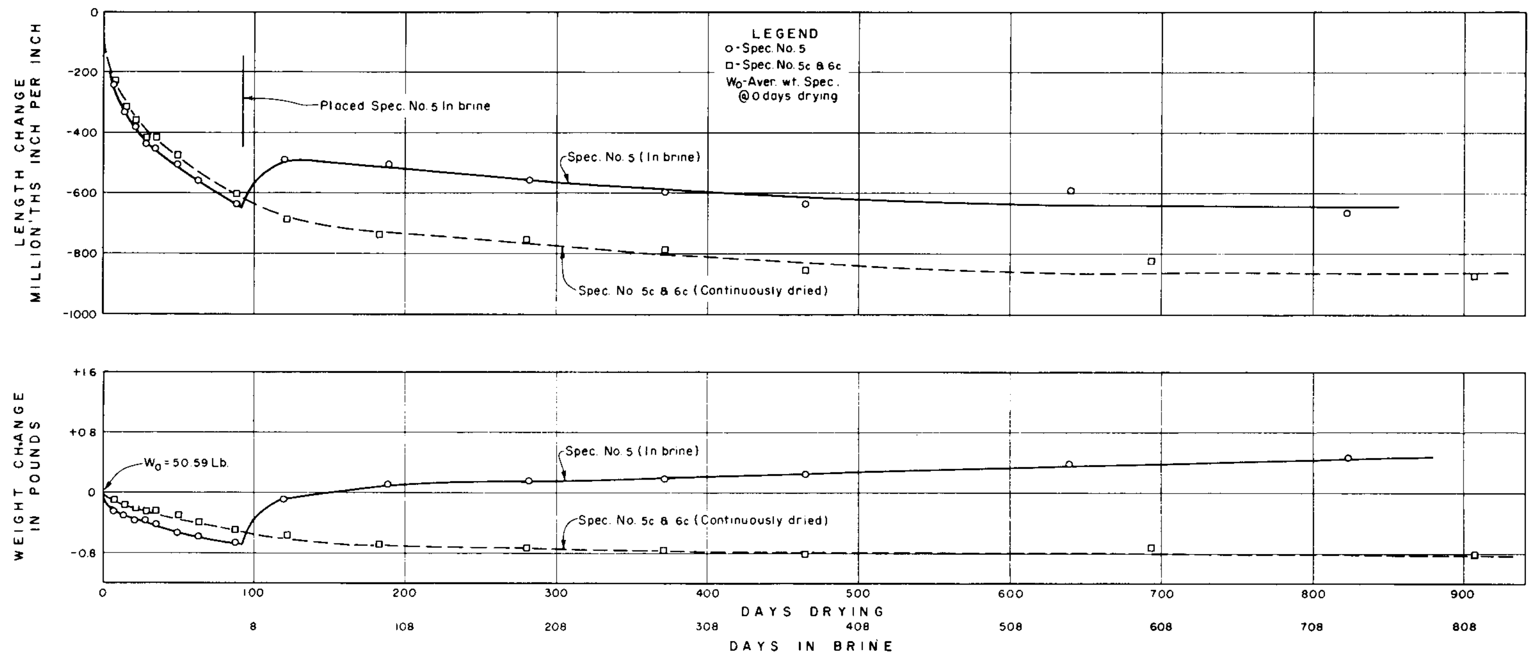

FIGURE III – 25 : LENGTH AND WEIGHT CHANGE OF PRESTRESSED NATURAL AGGREGATE CONCRETE SUBJECTED TO 100°F. BRINE AND CONTINUOUS DRYING

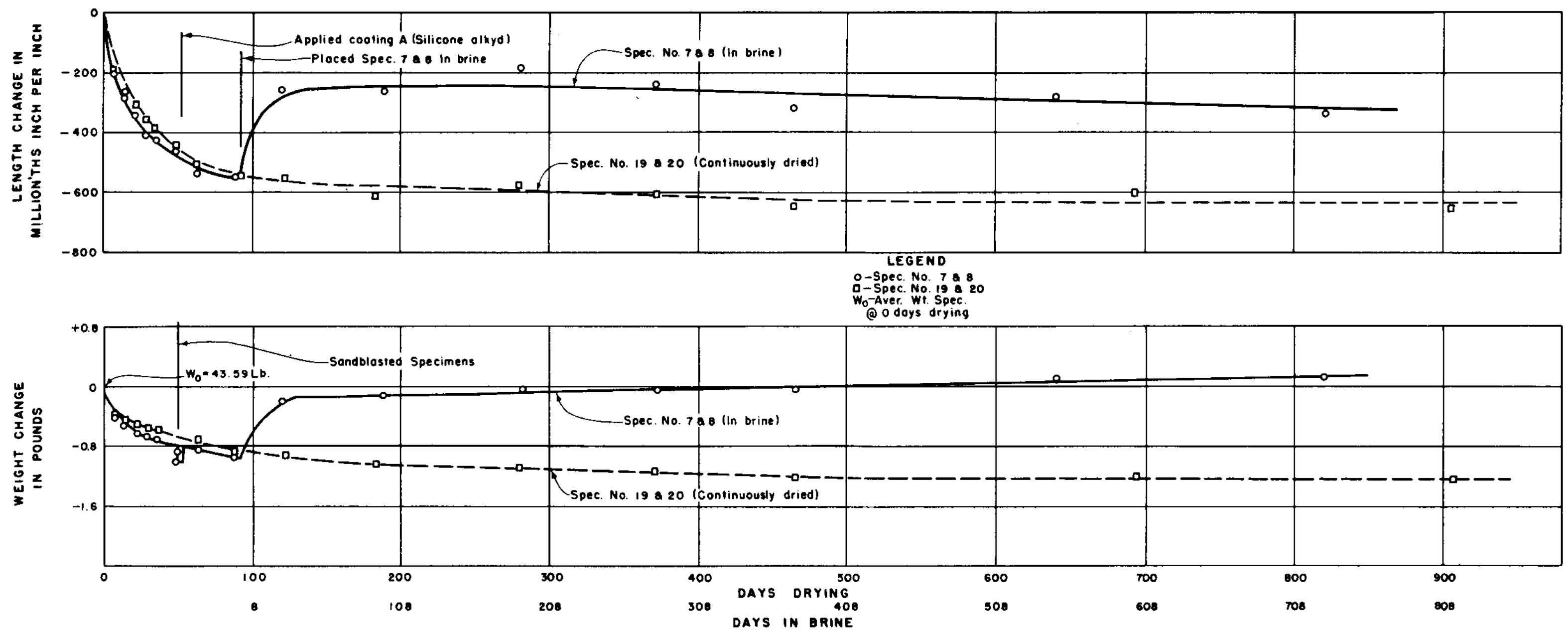

FIGURE III - 26: LENGTH AND WEIGHT CHANGE OF SILICONE ALKYD COATED NATURAL AGGREGATE CONCRETE SUBJECTED TO 100°F. BRINE AND CONTINUOUS DRYING

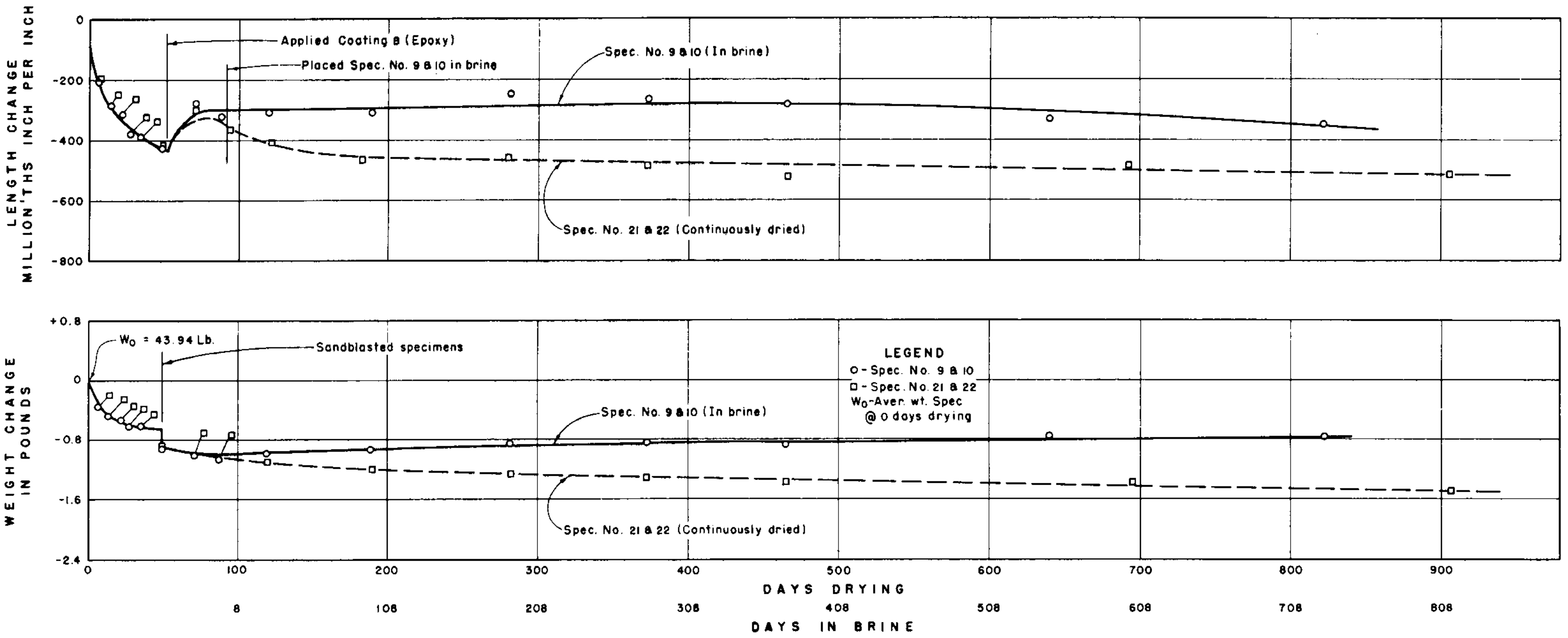

FIGURE III - 27: LENGTH AND WEIGHT CHANGE OF EPOXY COATED NATURAL AGGREGATE CONCRETE SUBJECTED TO 100°F. BRINE AND CONTINUOUS DRYING

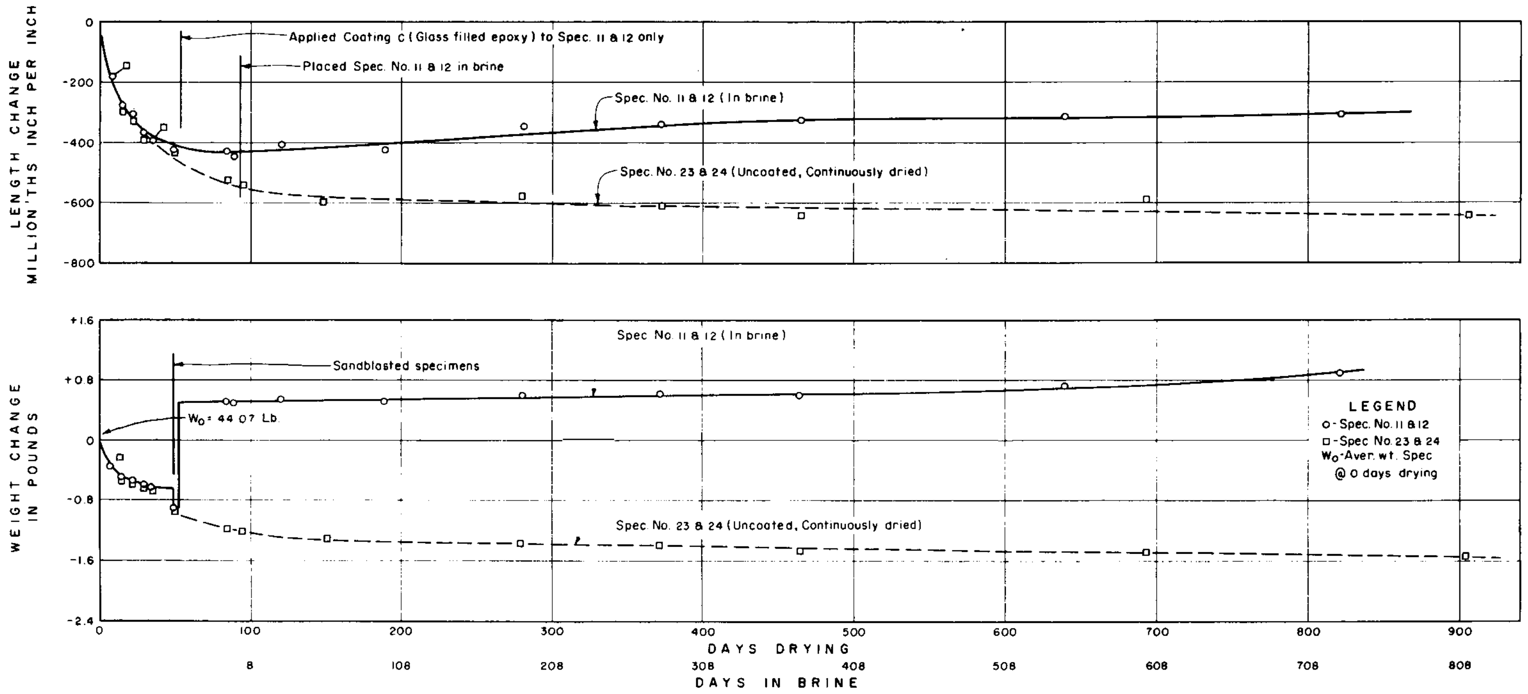

FIGURE III - 28: LENGTH AND WEIGHT CHANGE OF GLASS FIBER AND EPOXY COATED NATURAL AGGREGATE CONCRETE SUBJECTED TO 100°F. BRINE AND CONTINUOUS DRYING

Concrete prepared with the epoxy material (Coating B) also experienced an immediate expansion when placed in the 100° F brine, Figure III-27, although not nearly as great as that previously described. A phenomenon occurred here that is difficult to explain; a moderate expansion in the coated concrete was noted within the first 30 days of brine exposure but no similar weight increase occurred during this period of time. No significant length or weight change occurred in this coated concrete after the initial 30 days of brine exposure.

Concrete prepared with the glass fiber and epoxy material underwent no immediate length or weight change as did the other two coated concretes when placed in 100° F synthetic sea water brine, Tables III-26, III-27, and Figure III-28. The concrete did undergo a gradual expansion and increase in weight throughout the 2 years of brine exposure, and thus it is evident that neither this coating system nor the other two were effective in protecting the concrete from contact with the brine.

III. 6. 2. 2. 2. 290° F brine tests. --

The general condition of the concrete subjected for 18 months to the 290° F synthetic sea water brine can be considered both good and marginal. The interior concrete is still sound as evidenced by high compressive strengths, although somewhat less than at previous readings, and lack of microstructural cracks, expansion, or chemical reactions. The surface concrete however, is altered chemically and deteriorated to a depth of about 5 to 8 mm. This deterioration, which has thickened somewhat during the past 6 months' exposure, has softened the surface concrete, but, excepted for removal by jarring, this softened surface material is still intact. A rate of deterioration penetration equal to or greater than that which has occurred throughout this first year of testing would probably preclude the use of concrete in any area where 30 years' exposure to the 290° F brine was required. However, it is possible that the penetration of deterioration which was static for 6 months and which has only recently increased slightly is insignificant and as a result the use of concrete under these environmental conditions could be feasible.

Up to 180 days in brine the concrete had a thin coating of reddish colored material; the color being iron oxide. Subsequent to 180 days in brine, the same concrete had a thin coating of black material; the black accruing to the presence of magnetite, Figure III-29. There is some question as to the exact reason for this change of state phenomenon; however, it is likely due to a combination of increased iron oxide content and a very small amount of oxygen. It should be pointed out here that a complete change of brine was made each time that the system was shut down for specimen testing.

Maximum compressive strength of the concrete subjected to 290° F synthetic sea water brine was developed after about 90 days' exposure. Subsequently, it retrogressed from this maximum, 7,910 to 6,150 psi, at 547 days, Figure III-19. This retrogression has been somewhat proportional to the rate and depth of deterioration, and it is believed that the drop in compressive strength is due for the most part to the reduction in cross-sectional area of the sound concrete. No deterioration of the interior concrete has been noted.

Modulus of elasticity of the concrete subjected to the 290° F synthetic sea water brine as well as that of the companion concrete which had undergone the same initial 28 days' fog cure (100 percent RH, 73.4° F) followed by continuous drying, increased slightly with time through 270 days after which it leveled off, Table III-28 and Figure III-30. After 407 days' total age the modulus of the brine-exposed concrete was 14.6 percent greater than that of the continuously dried companions. However, after 693 days' total age this difference had dropped to 4.6 percent. Both the 4.50-million-psi modulus of the concrete after 18 months' brine exposure and the 4.75-million-psi modulus of the continuously dried companion concrete are equal to or above the 4.50-million-psi modulus considered average for good quality concrete; further, these values are indicative of the continued soundness of the brine-exposed concretes.

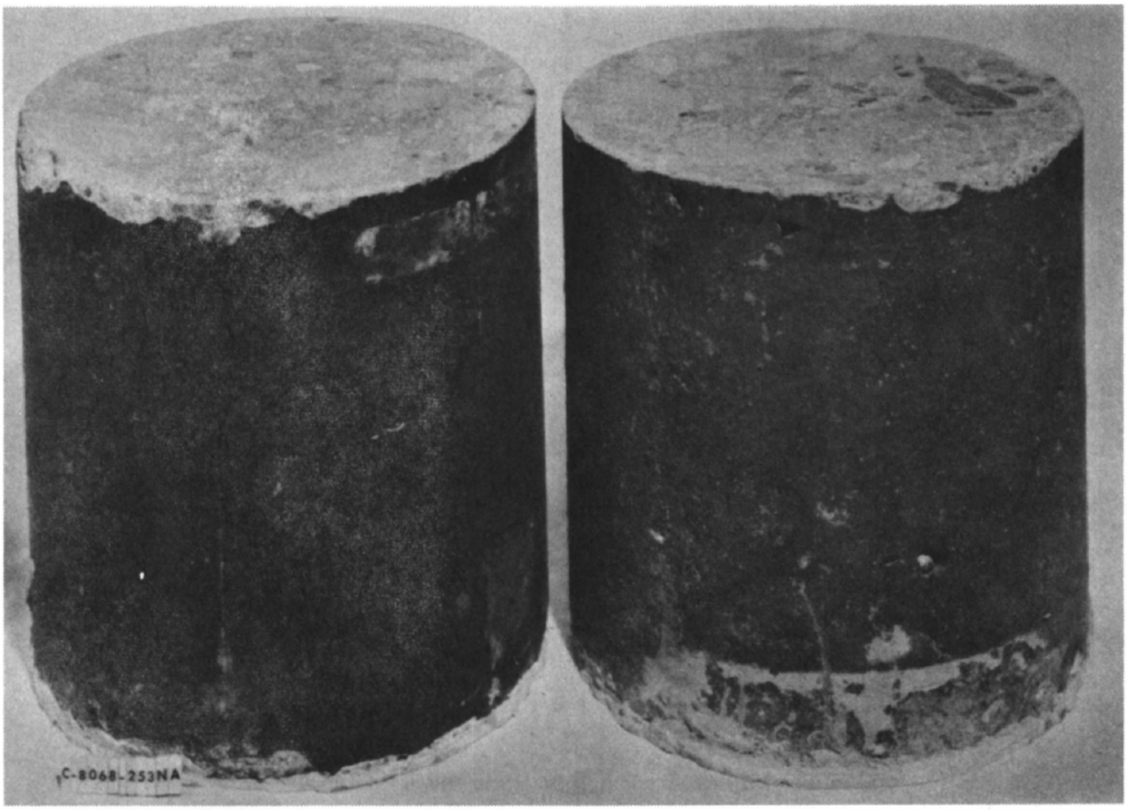

Figure III-29. Concrete exposed to the 290° F synthetic sea water brine for 180 days. The black colored specimen on left has a thin coating of material including magnetite; whereas, the reddish colored specimen on the right has a coating of material including iron oxide. Photo PX-D-55240

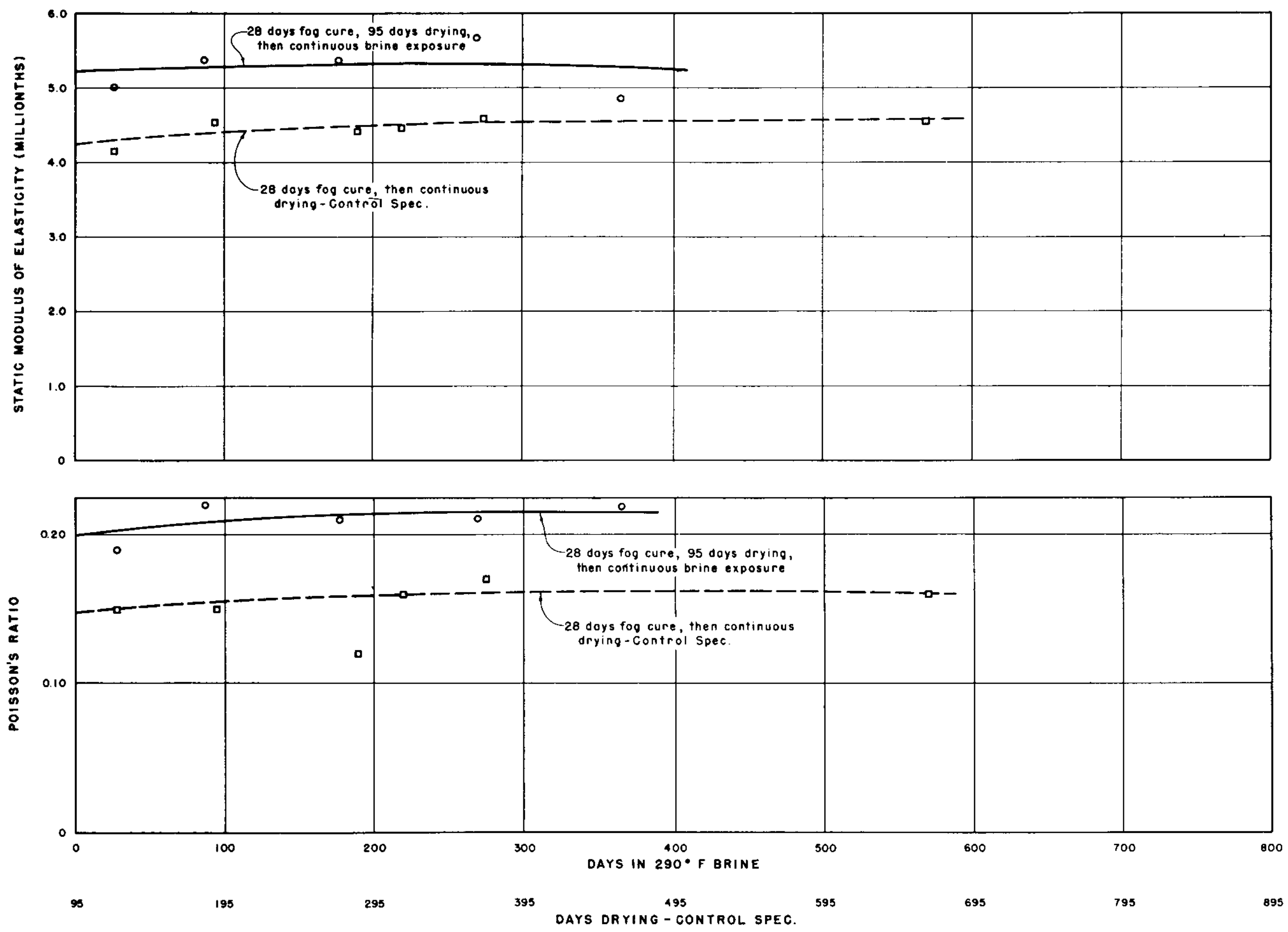

FIGURE III - 30: MODULUS OF ELASTICITY AND POISSON'S RATIO OF NATURAL AGGREGATE CONCRETE SUBJECTED TO 290° F SYNTHETIC SEA WATER BRINE, AND CONTINUOUSLY DRIED CONTROL CONCRETE. POINTS PLOTTED AT EACH AGE REPRESENTS THE AVERAGE OF THREE DIFFERENT SPECIMENS.

Table III-28

COMPRESSIVE STRENGTH AND ELASTIC PROPERTIES
6- by 12-inch Concrete Cylinders
290° F High-temperature System
Corrosion of Concrete Investigation
For OSW

Specimen No.	Fog cure (days)	Dry cure (days)	In brine (days)	Total age (days)	Compressive strength (psi)	E (million psi)	r	Unit weight pounds per cu foot	Pulse velocity foot per second
CCI-H1	28	95	31	154	8,470	5.32	0.18	159.5	16,600
CCI-H2	28	95	31	154	7,360	4.75	0.20	158.2	16,200
CCI-H3	28	95	31	154	6,930	5.01	0.20	160.7	16,100
Average	28	95	31	154	7,590	5.03	0.19	159.5	16,300
CCI-H1C	28	126	0	154	8,490	4.14	0.14	157.0	15,400
CCI-H2C	28	126	0	154	7,570	4.55	0.20	155.8	15,700
CCI-H3C	28	126	0	154	7,740	4.36	0.13	155.8	15,600
Average	28	126	0	154	7,930	4,35	0.15	156.2	15,600
CCI-H4	28	100	90	218	8,440	5.61	0.24	158.2	16,100
CCI-H5	28	100	90	218	7,760	5.48	0.24	155.1	15,800
CCI-H6	28	100	90	218	7,530	5.05	0.19	157.6	15,600
Average	28	100	90	218	7,910	5.38	0.22	157.0	15,800
CCI-H4C	28	190	0	218	8,450	4.61	0.16	155.1	15,400
CCI-H5C	28	190	0	218	7,940	4.63	0.14	153.9	15,400
CCI-H6C	28	190	0	218	7,820	4.38	0.15	154.5	15,000
Average	28	190	0	218	8,070	4.54	0.15	154.5	15,300
CCI-H7	28	105	180	310	7,820	5.57	0.23	159.5	16,500
CCI-H8	28	105	180	310	6,700	5.68	0.19	160.1	16,900
CCI-H9	28	105	180	310	7,000	5.45	0.20	160.1	16,900
Average	28	105	180	310	7,170	5.57	0.21	159.9	16,800
CCI-H7C	28	285	0	310	8,240	4.49	0.11	158.9	14,500
CCI-H8C	28	285	0	310	7,520	4.27	0.10	157.0	14,700
CCI H9C	28	285	0	310	7,620	4.51	0.14	156.4	14,300
Average	28	285	0	310	7,790	4.42	0.12	157.4	14,500

Table III-28--Continued

COMPRESSIVE STRENGTH AND ELASTIC PROPERTIES
6- by 12-inch Concrete Cylinders
290° F High-temperature System
Corrosion of Concrete Investigation for OSW

Specimen No	Fog cure (days)	Dry cure (days)	In brine (days)	Total age (days)	Compressive strength (psi)	E (million psi)	r	Unit weight pounds per cubic foot	Pulse velocity foot per second
CCI-H13	28	136	180	344	7,230	5.34	0.20	160.7	16,300
CCI-H14	28	136	180	344	6,740	4.96	0.20	160.7	16,300
CCI-H15	28	136	180	344	6,630	5.06	0.22	162.0	15,700
Average				344	6,870	5.12	0.21	161.1	16,100
CCI-H13C	28	316	0	344	8,100	4.88	0.15	158.2	15,100
CCI-H14C	28	316	0	344	7,390	4.27	0.18	153.2	14,900
CCI-H15C	28	316	0	344	7,520	4.33	0.16	154.5	14,200
Average				344	7,670	4.49	0.16	155.3	14,700
CCI-H10	28	109	270	407	7,570	5.69	0.22	160.1	15,900
CCI-H11	28	109	270	407	6,920	4.88	0.21	156.4	15,200
CCI-H12	28	109	270	407	6,540	5.24	0.20	158.9	15,200
Average				407	7,010	5.27	0.21	158.5	15,400
CCI-H10C	28	379	0	407	8,240	4.74	0.19	155.1	14,700
CCI-H11C	28	379	0	407	7,530	4.66	0.16	153.2	14,000
CCI-H12C	28	379	0	407	7,360	4.40	0.15	154.5	14,000
Average				407	7,710	4.60	0.17	154.3	14,200
CCI-H16	28	300	365	693	5,750	4.88	0.17	162.6	16,300
CCI-H17	28	300	365	693	5,340	4.77	0.23	162.0	15,800
CCI-H18	28	300	365	693	5,200	4.71	0.25	162.0	15,100
Average				693	5,430	4.79	0.22	162.2	15,700
CCI-H16C	28	665	0	693	7,940	4.70	0.14	155.1	14,500
CCI-H17C	28	665	0	693	7,320	4.55	0.17	156.4	14,600
CCI-H18C	28	665	0	693	7,340	4.48	0.16	153.2	14,900
Average				693	7,530	4.58	0.16	154.9	14,700

Poisson's ratio of the concrete exposed to the 290° F synthetic sea water brine did not change significantly throughout the test time, and its value of 0.22 after 365 days under test and 0.19 after 547 days is considered about average for good quality concrete. Companion continuously dried concrete experienced a slight gain in Poisson's ratio, but at 693 days' total age its value of 0.16 was still 27.3 percent less than that of the brine-exposed concrete, Table III-28 and Figure III-30.

Unit weight of the brine-exposed concretes continued to increase somewhat to 162.2 pounds per cubic foot while that of the companion concrete remained quite constant at 154.9 pounds per cubic foot, Table III-28. Apparently the slight increase in unit weight with exposure is due to the continued absorption.

As previously discussed under Section III.6.2.2.1. nondestructive pulse velocity measurements were made to determine the soundness of the concrete. The pulse velocity of concrete subjected to 290° F brine remained above 15,000 feet per second and essentially unchanged throughout 365 days, an indication of continued high quality, Table III-28 and Figure III-22. Conversely, companion continuously dried concrete experienced a constant decrease in pulse velocity through the same period of time. No deterioration of the interior concrete is apparent by these test results.

Resonant frequency of the concrete subjected to the 290° F synthetic sea water brine exhibited an initial rapid increase due, in part, to the absorbing of moisture into the concrete, and in part by the continuation of hydration, Table III-29. Subsequent to this initial increase, the resonant frequency has undergone a rather constant decrease to 18 months' brine exposure. The percent change (decrease) in the square of the frequency from that measured after 28 days' fog cure to that measured after 18 months' brine exposure amounted to about 16.4 percent. This is considerably less than the 25 percent reduction chosen as the point of failure of concrete.

As would be expected, the total drying shrinkage of the concrete prepared for the 290° F brine tests (496 millionths inch per inch) closely approximated that shrinkage which occurred in the low-temperature (100° F) brine concrete. However, the recovery of 359 millionths inch per inch, expansion, which occurred within the first 30 days in this brine exposure, Table III-30, Figures III-31, III-32, was 32.4 percent greater than that which occurred in the 100° F brine concrete during the same period of time. Further, the length of these concrete specimens continued to increase at least until 18 months' brine exposure, whereas, the concrete exposed to the 100° F brine underwent very little length change after the first 30 days of exposure. The net length change of the concrete after 18 months' brine exposure was a positive 330 millionths inch per inch, i.e., this concrete now occupies more volume than it did prior to drying at 28 days' total age. Continuously dried concrete experienced a length change (drying shrinkage) of about 606 millionths inch per inch at 802 days' drying, Figure III-31. This is substantially greater than the 500 millionths inch per inch value considered average for high-quality concrete.

Length change of the concrete subjected to the 290° F brine generally corresponds very closely with the weight change, i.e., curves plotted from length and weight change data nearly parallel each other, in each instance the measurement decreasing with drying and increasing with reimmersion, Tables III-30 and III-31, Figure III-31.

The pretensioned concrete bars, Figure III-33, continued to reduce in volume due to the pretensioning load while simultaneously increasing in weight due to absorption of the hot brine. Apparently this weight increase was due primarily to the absorption of moisture and secondarily to the thin film of extraneous material which adhered to the specimens. After 180 days in the high-temperature brine the net reduction in length from the first day of drying (28 days' total age) was about 832 millionths as compared to the 559 millionths inch per inch observed in the pretensioned concrete exposed to the 100° F brine. As in the case of the 100° F test specimens, SR-4 strain gages which were mounted on the pretensioning rods were lost after initial exposure to the 290° F brine. Stresses computed

Table III-29

FREQUENCY SQUARED OF NATURAL AGGREGATE CONCRETE
4- By 4- By 30-inch Bars
290° F System
Corrosion of Concrete Investigation for OSW

			[5]$F^2 \times 10^{-5}$		[5]$F^2 \times 10^{-5}$[9]		[5]$F^2 \times 10^{-5}$[10]	
Days[1] drying	Days brine	Total age	Spec 1 & 2	Spec 1c & 2c	Spec 3 & 4	Spec 3c & 4c	Spec 5 & 6	Spec 5c & 6c
0	0	28	[4]4.97	[4]4.90	5.04	4.86	3.54	3.48
7	0	35	4.66	4.58	4.80	4.66	3.54	3.40
14	0	42	4.76	4.72	4.49	4.76	3.54	3.56
21	0	49	4.58	4.42	4.53	4.45	3.60	3.52
28	0	56	4.66	4.62	4.46	4.76	3.64	3.50
42	0	70	4.66	4.62	4.46	4.72	3.58	3.48
56	0	84	4.62	4.53	4.60	4.60	3.56	3.64
92	0	120	4.45	4.49	4.26	4.36	3.52	3.42
[2]127	[3] 30	155	4.90	4.36	[7]4.69	4.49	[8]3.48	3.44
190	90	218	4.83	4.25	4.76	4.22	3.91	3.46
283	180	311	4.62	4.22	4.49	4.16	3.78	3.44
379	270	407	4.49	4.20	4.49	4.16	3.72	3.50
477	365	505	4.29	[6]4.29	4.36	4.22	3.66	3.58
802	547	830	4.16	4.33	-	4.29	3.60	3.60

[1] All specimens received an initial 28-day fog cure.

[2] Days drying from here on only applies to Specimens 1c and 2c.

[3] Days brine only applies to Specimens 1 and 2.

[4] These values are F_o as referred to in Figure III-

[5] Units for frequency (F) are cycles per second.

[6] Specimen 2c only.

[7] Specimen 3 only from here on.

[8] Specimen 5 only from here on.

[9] Reinforced concrete specimens.

[10] Pretensioned concrete specimens.

Table III-30

LENGTH CHANGE OF 4- BY 4- BY 30-INCH BARS IN MILLIONTHS INCH PER INCH
290° F High-temperature Brine System
Corrosion of Concrete Investigation for OSW

Total age (days)		35	42	56	84	120	155	218	311	407	505	830		
Cure (days)	Fog	28	28	28	28	28	28	28	28	28	28	28	28	28
	Dry	7 (7)2/	14 (14)	28 (28)	56 (56)	92 (92)	97 (127)	100 (190)	103 (283)	109 (379)	112 (477)	255 (802)		
	Brine	0 (0)2/	0 (0)	0 (0)	0 (0)	0 (0)	30 (0)	90 (0)	180 (0)	270 (0)	365 (0)	547 (0)		
Specimen														
Number	Type1/													
1 & 2	Pl	196	281	352	456	496	137	148	25	+53	+130	+330		
1c & 2c	(Pl)2/	214	281	366	467	504	522	547	543	604	609	606		
3	Re	182	252	320	381	421	+63/	+48	+111	+220	+302	-		
4	Re	182	252	320	381	421	+1274/	+166	+250	+349	+420	555		
3c & 4c	(Re)	179	217	288	370	425	404	429	404	454	477	446		
5	Prs	246	320	399	556	679	755	844	832	1,256	1,266	1,334		
6	Prs	246	320	399	556	679	1,4264/	1,494	1,496	1,535	1,573	1,645		
5c & 6c	(Prs)	243	317	416	556	643	658	790	771	857	898	910		
7 & 8	A	225	320	406	534	582	216	209	96	62	5	+137		
19 & 20	(A)	193	270	345	474	521	515	543	500	579	606	589		
9 & 10	B	211	263	341	297	382	219	187	78	44	23	+87		
21 & 22	(B)	211	259	334	286	375	393	436	432	486	509	492		
11 & 12	C	214	267	349	404	436	244	219	132	5	23	+66		
23 & 24	(C)	179	267	3596/	4505/	461	468	468	454	390	381	381		

1/Pl = Plain Prs = Prestressed B = Coated with epoxy
Re = Reinforced A = Coated with silicone alkyd C = Coated with glass-filled epoxy

2/All ( ) refer to companion specimens which were not placed in brine, but left in laboratory air.

3/All specimen readings indicate shrinkage compared to the 28-day fog readings except those marked with a "+" which indicates expansion.

4/Broken in flexure before this reading. Bar readings not averaged.

5/77 days drying.

6/Bar 24 not included after this date.

Table III-31

WEIGHT CHANGE OF 4- BY 4- BY 30-INCH BARS IN POUNDS
290° F High-temperature Brine System
Corrosion of Concrete Investigations for OSW

Total age (days)		35	42	56	84	120	155	218	311	407	505	830		
Cure (days)	Fog	28	28	28	28	28	28	28	28	28	28	28	28	28
	Dry	7	14	28	56	92	97	100	103	109	112	255		
		(7)2/	(14)	(28)	(56)	(92)	(127)	(190)	(283)	(379)	(477)	(802)		
	Brine	0	0	0	0	0	30	90	180	270	365	547		
		(0)2/	(0)	(0)	(0)	(0)	(0)	(0)	(0)	(0)	(0)	(0)		
Specimen Number	Type1/													
1 & 2	Pl	0.37	0.50	0.61	0.81	0.90	0.03	0.36	+0.23	+0.40	+1.04	+1.44		
1c & 2c	(Pl)2/	0.39	0.49	0.62	0.79	0.89	0.96	1.08	1.16	1.26	1.36	1.33		
3	Re	0.41	0.55	0.68	0.87	0.97	+0.053/	0.20	+0.20	+0.45	+0.93	-		
4	Re	0.41	0.55	0.68	0.87	0.97	+0.034/	0.40	+0.28	+0.50	+0.96	+1.52		
3c & 4c	(Re)	0.36	0.42	0.54	0.72	0.86	0.91	1.11	1.16	1.26	1.36	1.33		
5	Prs	0.13	0.19	0.23	0.39	0.52	+0.52	+0.39	+0.74	+0.92	+1.34	+1.75		
6	Prs	0.13	0.19	0.23	0.39	0.52	+0.504/	+0.36	+0.76	+1.03	+1.32	+1.62		
5c & 6c	(Prs)	0.12	0.17	0.26	0.37	0.48	0.53	0.73	0.75	0.80	0.86	0.81		
7 & 8	A	0.38	0.56	0.69	0.77	0.94	+0.06	0.31	+0.28	+0.44	+1.11	+1.51		
19 & 20	(A)	0.35	0.47	0.57	0.67	0.84	0.88	1.02	1.02	1.10	1.18	1.15		
9 & 10	B	0.35	0.43	0.52	0.95	1.02	0.45	0.34	+0.06	+0.18	+0.46	+0.69		
21 & 22	(B)	0.38	0.45	0.55	0.90	0.97	1.00	1.08	1.13	1.19	1.25	1.27		
11 & 12	C	0.40	0.48	0.74	+0.535/	+0.53	+0.96	+1.02	+1.27	+1.85	+2.12	+2.20		
23 & 24	(C)	0.36	0.52	0.84	+0.616/	+0.49	+0.48	+0.48	+0.49	+0.51	+0.49	+0.58		

1/Pl = Plain Prs = Prestressed B = Coated with epoxy
Re = Reinforced A = Coated with silicone alkyd C = Coated with glass-filled epoxy

2/All ( ) refer to companion specimens which were not placed in brine, but left in laboratory air.

3/All specimen readings indicated weight loss compared to the 28-day fog readings except those marked with a "+" which indicates weight gain .

4/Broken in flexure before this reading. Bar readings not averaged.

5/105-day reading.

6/105-day reading--Bar 23 reading only after this date.

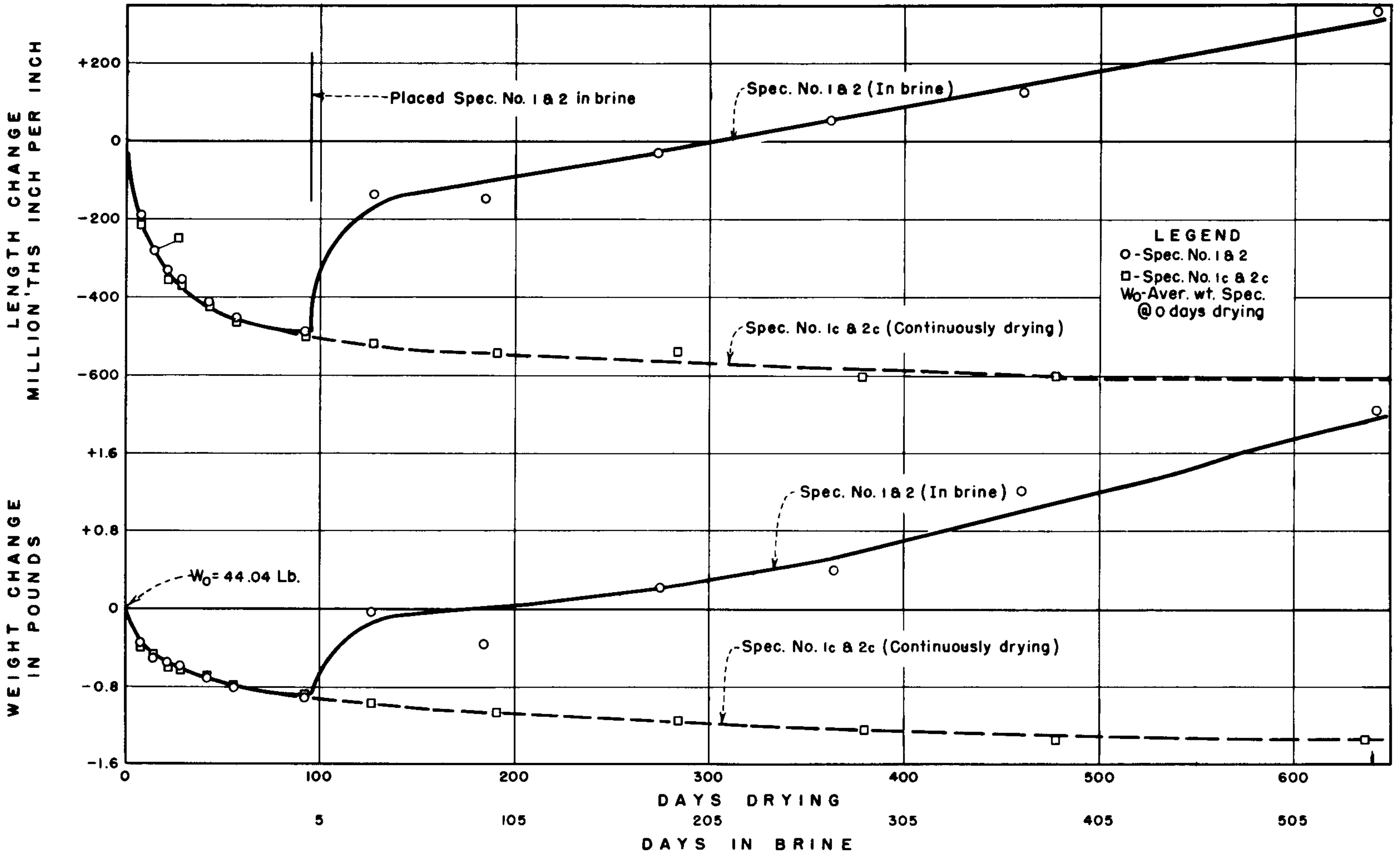

FIGURE III - 31: LENGTH AND WEIGHT CHANGE OF PLAIN NATURAL AGGREGATE CONCRETE SUBJECTED TO 290°F. BRINE AND CONTINUOUS DRYING

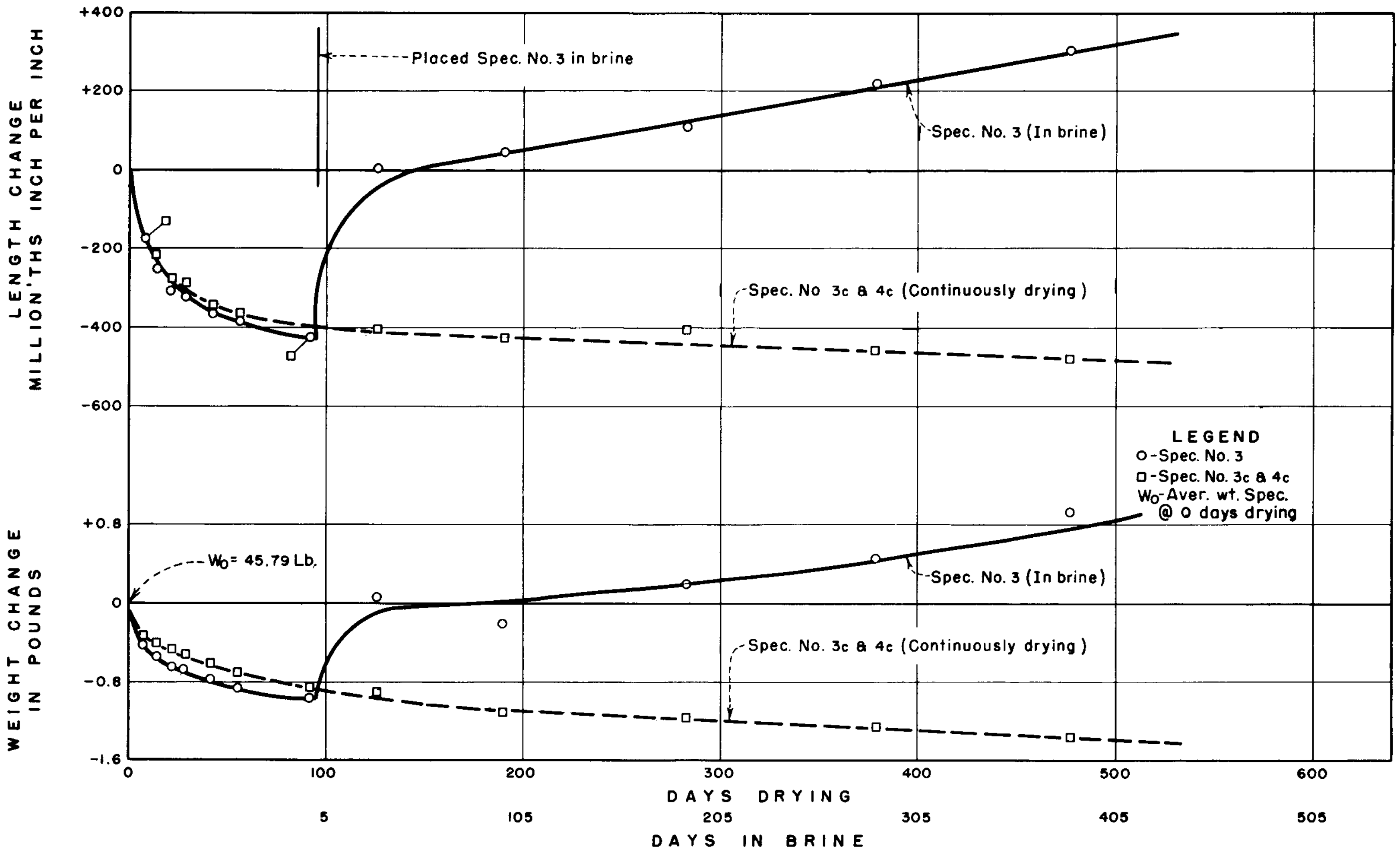

FIGURE III - 32: LENGTH AND WEIGHT CHANGE OF REINFORCED NATURAL AGGREGATE CONCRETE SUBJECTED TO 290°F. BRINE AND CONTINUOUS DRYING

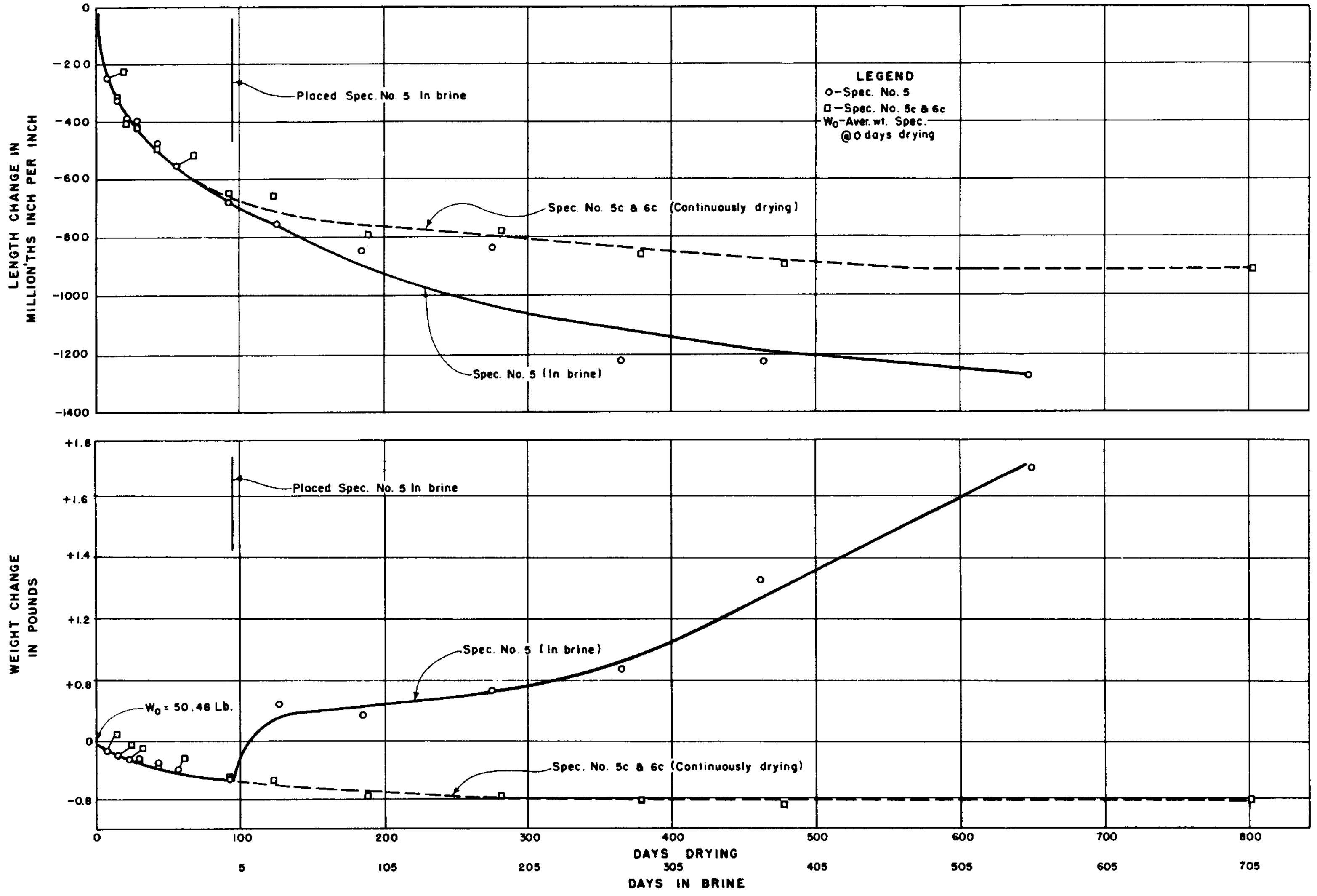

FIGURE III - 33: LENGTH AND WEIGHT CHANGE OF PRESTRESSED NATURAL AGGREGATE CONCRETE SUBJECTED TO 290°F. BRINE AND CONTINUOUS DRYING from strain readings taken prior to the brine exposure indicated a stress in the concrete after 28 days' fog cure and 93 days' drying of 1, 160 psi. This is about 54. 7 percent greater than the 750 psi load in the 100° F concrete after the same curing time. This 54. 7 percent greater load compares quite favorably with the 49 percent greater shrinkage as reported previously in this paragraph. For reason unknown at this time, length change readings obtained subsequent to those taken at 180 days' exposure were very erratic.

Length and weight changes which this concrete has undergone in drying and then expansion due to the brine are not considered unreasonable and do not indicate deterioration to the mass of the concrete.

Concrete coated with the silicone alkyd material and exposed to the 290° F synthetic sea water brine expanded slightly more than the plain concrete during the first 30 days of brine exposure, Table III-30, Figure III-34. Expansion of the concrete continued throughout the length of the brine exposure. These test results are indicative of the permeability to moisture of the silicone alkyd coating even though the alkyd coating did not show visible signs of deterioration after the first 30 days in the brine. (See discussion regarding this and the two coatings discussed further in this section in Section V.) Weight change of the high-temperature concrete varied similarly with the length change, and indicates that moisture was being absorbed into the concrete throughout its exposure to the brine.

The coated concrete subjected to the high-temperature brine for 30 days underwent an expansion which was 25. 3 percent greater than that which occurred in the concrete having undergone 30 days' exposure to the 100° F brine. Undoubtedly, this difference was due to the higher pressures of the high-temperature environment. Weight and length change of the continuously drying silicone alkyd coated concrete continued through 1 year to exhibit moderate weight loss due to loss of moisture and the accompanying shrinkage. Little or no change occurred between 1 year and 18 months.

The epoxy coated concrete (Coating B) also underwent expansion upon subjection to the 290° F brine primarily during the initial 30 days' exposure, Table III-30, Figure III-35. A slight increase in expansion or a decrease in net shrinkage occurred throughout the length of the test. An increase in weight accompanied the expansion, and is again indicative of the permeability of the epoxy coating material. The magnitude of the expansion and weight increase of the concrete during that initial 30-day brine exposure is 62. 9 and 43. 0 percent, respectively, less than that of the alkyd coated concrete. However, in either event they are not considered suitable as protective materials for concrete under these severe environments.

The glass fiber and epoxy coated concrete reacted similarly to the previously mentioned coated concrete, and, therefore, will not be discussed in detail, Tables III-30, III-31, Figure III-36. The important thing to note regarding the length and weight change of these coated specimens is that whether the coating itself failed completely, partially or only pinholes, the concrete volume wise reacted similarly to the plain concrete.

III. 6. 2. 2. 3. <u>250° F brine tests.</u> --

The general condition of the concrete subjected for 180 days to the 250° F synthetic sea water brine can be considered good. Although some minute surface alteration has been determined petrographically, no deterioration has taken place.

Compressive strength of the natural aggregate concrete subjected to the 250° F synthetic sea water brine increased from 7, 030 psi at 28 days' exposure to 7, 930 psi at 90 days' exposure which is a greater rate of increase than experienced in either the 100° or 290° F exposed concretes, Table III-32 and Figure III-19. However, no significant change in compressive strength occurred between 90 and 180 days' exposure. It is interesting to note that although the compressive strengths of the natural aggregate concretes exposed for 28 days

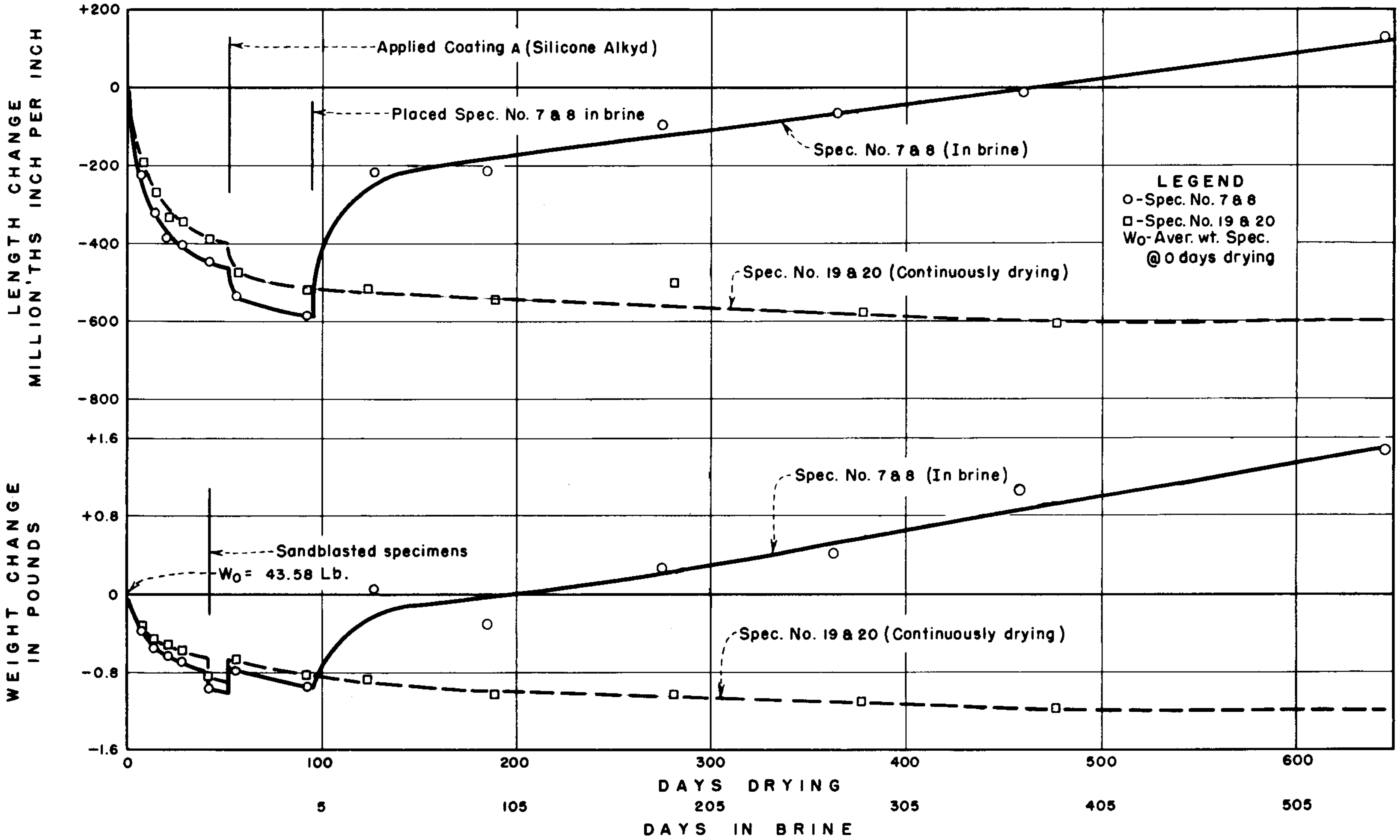

FIGURE III – 34: LENGTH AND WEIGHT CHANGE OF SILICONE ALKYD COATED NATURAL AGGREGATE CONCRETE SUBJECTED TO 290° F. BRINE AND CONTINUOUS DRYING

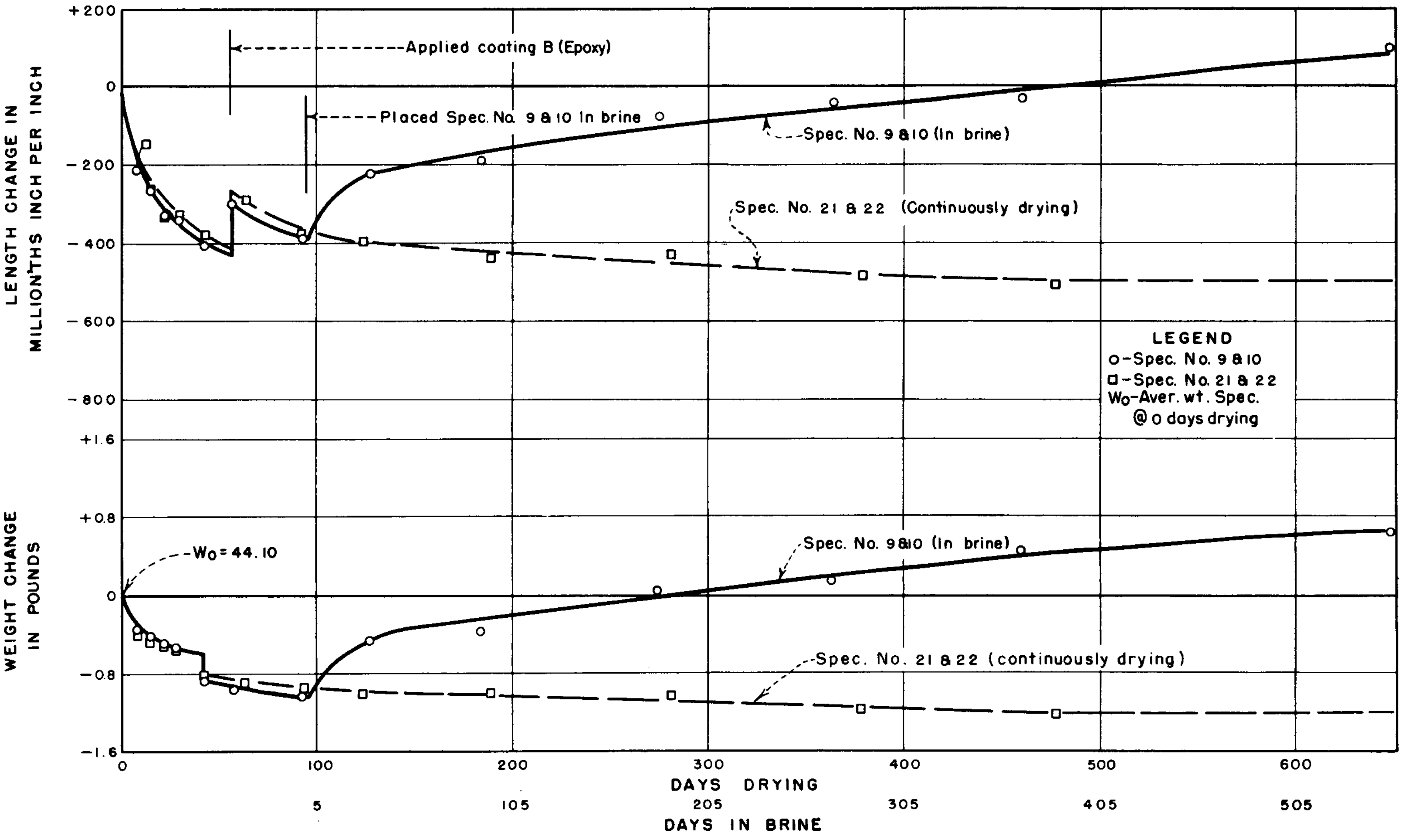

FIGURE III - 35: LENGTH AND WEIGHT CHANGE OF EPOXY COATED NATURAL AGGREGATE CONCRETE SUBJECTED TO 290°F. BRINE AND CONTINUOUS DRYING

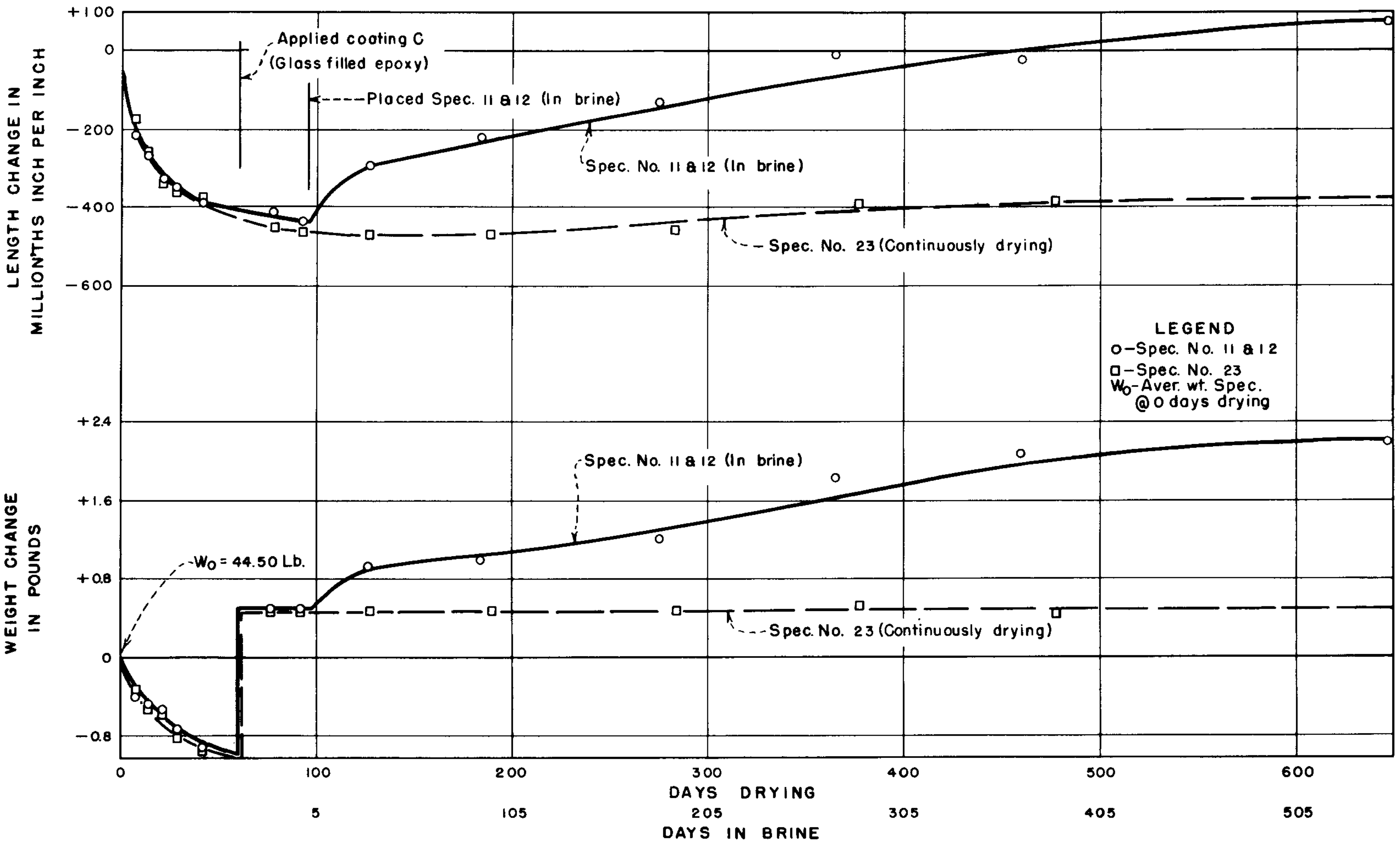

FIGURE III – 36 : LENGTH AND WEIGHT CHANGE OF GLASS FIBER AND EPOXY COATED NATURAL AGGREGATE CONCRETE SUBJECTED TO 290° F. BRINE AND CONTINUOUS DRYING

Table III-32

COMPRESSIVE STRENGTH AND ELASTIC PROPERTIES
6- by 12-inch Concrete Cylinders
250° F System
Natural Aggregate Concrete
Corrosion of Concrete Investigation for OSW

Specimen No.	Fog cure (days)	Dry cure (days)	In brine (days)	Total age (days)	Compressive strength (psi)	E (million psi)	r	Unit weight pounds per cu foot	Pulse velocity foot per second
CCI-T35	7	0	0	7	4,280	3.57	0.16	-	-
CCI-T36	7	0	0	7	4,120	3.57	0.19	-	-
Average					4,200	3.57	0.18		
CCI-T37				28	6,170	4.19	0.19	-	-
CCI-T38				28	5,730	3.96	0.16	-	-
Average					5,950	4.08	0.18		
CCI-T1	28	255	28	311	6,840	4.95	0.20	156.4	16,100
CCI-T2	28	255	28	311	7,500	5.31	0.20	157.0	16,100
CCI-T3	28	255	28	311	6,760	5.33	0.20	157.0	15,700
Average					7,030	5.20	0.20	156.8	16,000
CCI-T1C	28	283	0	311	7,820	4.95	0.19	154.5	15,100
CCI-T2C	28	283	0	311	8,400	4.70	0.16	155.1	15,100
CCI-T3C	28	283	0	311	7,870	5.18	0.19	155.8	15,100
Average					8,030	4.94	0.18	155.1	15,100
CCI-T4	28	310	90	428	7,920	5.37	0.17	157.0	15,800
CCI-T5	28	310	90	428	7,940	5.65	0.21	158.9	16,500
CCI-T6	28	310	90	428	7,940	5.77	0.20	158.2	15,900
Average					7,930	5.60	0.19	158.0	16,100
CCI-T4C	28	400	0	428	7,960	4.80	0.18	154.5	15,800
CCI-T5C	28	400	0	428	8,380	5.11	0.15	156.4	16,100
CCI-T6C	28	400	0	428	8,400	4.88	0.17	155.1	15,600
Average					8,250	4.93	0.17	155.3	15,800

Table III-32--Continued

COMPRESSIVE STRENGTH AND ELASTIC PROPERTIES
6- by 12-inch Concrete Cylinders
250^{o} F System
Natural Aggregate Concrete
Corrosion of Concrete Investigation for OSW

Specimen No.	Fog cure (days)	Dry cure (days)	In brine (days)	Total age (days)	Compressive strength (psi)	E (million psi)	r	Unit weight pounds per cu foot	Pulse velocity foot per second
CCI-T7	28	333	180	541	7,640	5.58	0.19	159.5	16,300
CCI-T8	28	333	180	541	8,240	5.85	0.21	159.5	15,900
CCI-T9	28	333	180	541	7,820	5.77	0.20	-	16,200
Average					7,900	5.73	0.20	159.5	16,100
CCI-T7c	28	513	0	541	8,310	5.01	0.19	154.5	15,100
CCI-T8c	28	513	0	541	8,150	5.15	0.21	155.1	15,400
CCI-T9c	28	513	0	541	8,080	5.32	0.20	157.0	14,900
Average					8,180	5.16	0.20	155.5	15,100

to the 100°, 225°, 250°, and 290° F brines varied from 7,590 to 7,030 psi, after 90 days' exposure to the same brines the compressive strengths only varied from 7,930 to 7,850 psi.

Compressive strength of the crushed limestone aggregate concrete subjected to the 250° F synthetic sea water brine averaged 5,840, 6,600, and 7,040 psi at 28, 90, and 180 days' exposure, respectively, Table III-33 and Figure III-37. This increase is comparable to both the natural and crushed limestone aggregate concrete subjected to other brine environments. These strengths give no indication of deterioration.

Modulus of elasticity of the natural and crushed limestone aggregate concrete subjected to the 250° F synthetic sea water brine average 5.20 and 5.15 million psi, respectively, at 28 days' exposure, 5.60 and 5.70 million psi, respectively, at 90 days' exposure, and 5.73 and 5.54 million psi, respectively, at 180 days' exposure, Tables III-32 and III-33, Figure III-38. These values are well above the 4.50 million psi considered average for good quality concrete, and no evidence of deterioration exists.

It should be pointed out that the modulus of elasticity of the natural aggregate concrete increased due to submergence in the brine, whereas, that the crushed limestone aggregate decreased when soaked in the brine. Since the cement and all other materials in these concretes except the coarse aggregates were of the same type and of nearly the same quantity it is apparent that these opposite effects due to soaking are caused by the type of coarse aggregate in the mix. In spite of the different changes having taken place in the modulus, one can see from the curves that whether the concrete undergoing brine exposure contains natural or crushed limestone aggregate the modulus is practically the same.

Poissons' ratio of the natural and crushed limestone aggregate concrete exposed to the 250° F synthetic sea water brine did not change significantly throughout the test period, and the last available values of 0.20 and 0.24, respectively, are considered about average for good quality concrete. Poissons' ratio of companion continuously dried concrete averaged 0.20 and 0.23 for the natural and crushed limestone aggregate concrete, respectively.

Unit weights of the natural and crushed limestone aggregate concretes subjected to the 250° F synthetic sea water brine averaged 159.5 and 151.4 pounds per cubic foot, respectively, at 180 days' exposure, which is an increase from the values obtained at 28 days' exposure, Tables III-32 and III-33. Apparently, the increase in unit weight with increase in exposure time is due to the continued moisture absorption. Very little change was noted in the unit weights of companion specimens.

Pulse velocity of natural aggregate concrete subjected to 250° F synthetic sea water brine remained essentially unchanged throughout the 180 days' of exposure while that of crushed limestone aggregate concrete increased substantially between 28 and 90 days with little change on to 180 days, Tables III-32, III-33, Figures III-39, and III-40. However, in all cases including the companion continuously dried concrete the pulse velocity was well above 15,000 feet per second indicating continued high-quality concrete.

Resonant frequency of the natural and crushed limestone aggregate concretes exhibited an initial rapid increase when subjected to the 250° F brine, as in the case of concretes exposed to the four other brine environments, Tables III-34 and III-35 and Figures III-41 and III-42. Apparently this is due in part to moisture absorption and continued hydration. It is significant to note that the magnitude of this initial abrupt increase in resonant frequency approximates that which occurred in the 290° F tests while the subsequent trends in the natural aggregate concrete approximates that which occurred in the 100° F tests. Resonant frequency of the crushed limestone aggregate concrete appears to be taking the same drop as experienced in the 290° F tests. In neither case do any signs of deterioration exist.

Data obtained from tests conducted upon the plain natural aggregate concrete bars indicated that the drying shrinkage of the concrete prior to brine exposure total

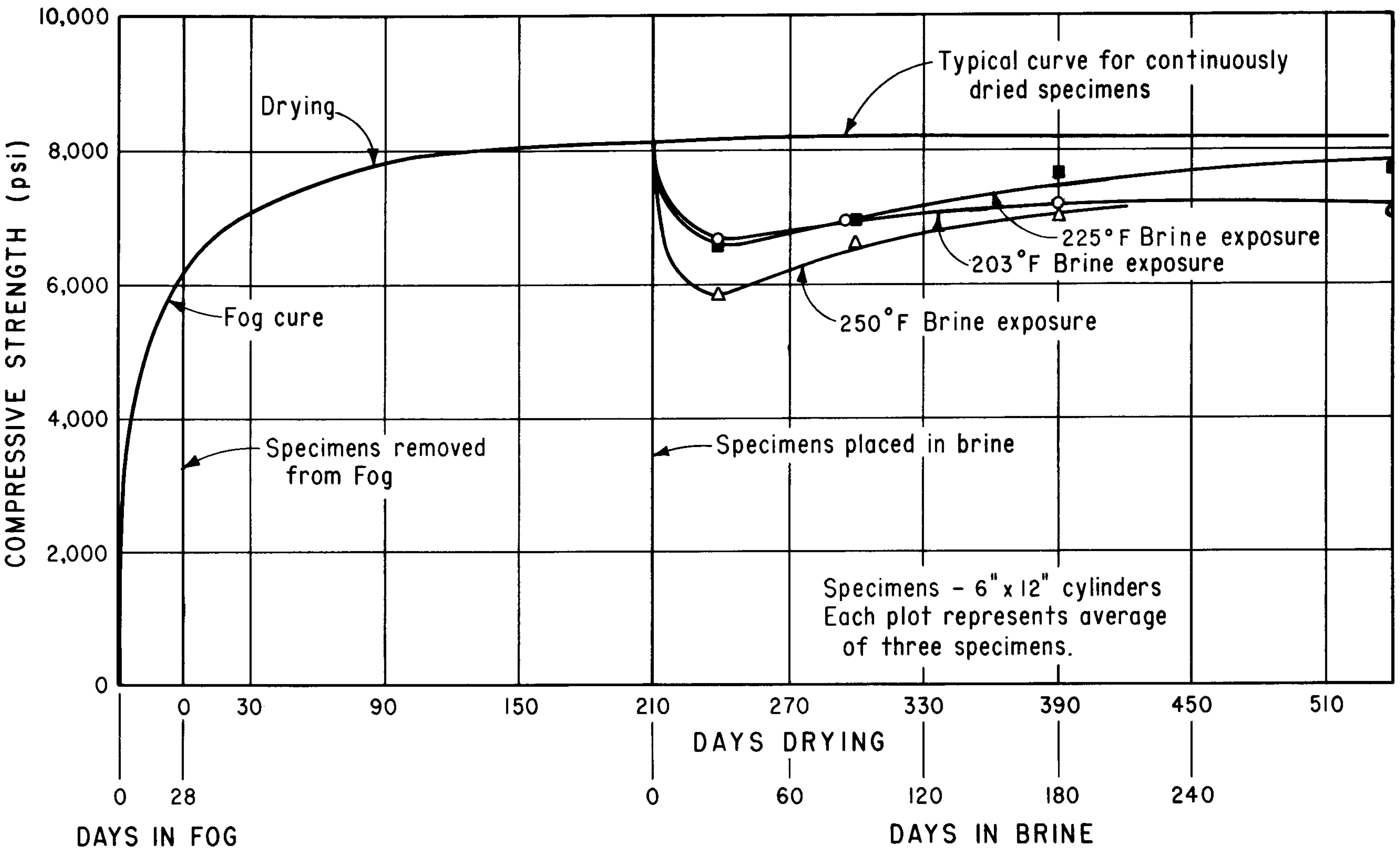

FIGURE III-37: COMPRESSIVE STRENGTHS OF CRUSHED LIMESTONE AGGREGATE CONCRETES SUBJECTED TO FOG, DRYING, AND VARIOUS BRINE ENVIRONMENTS

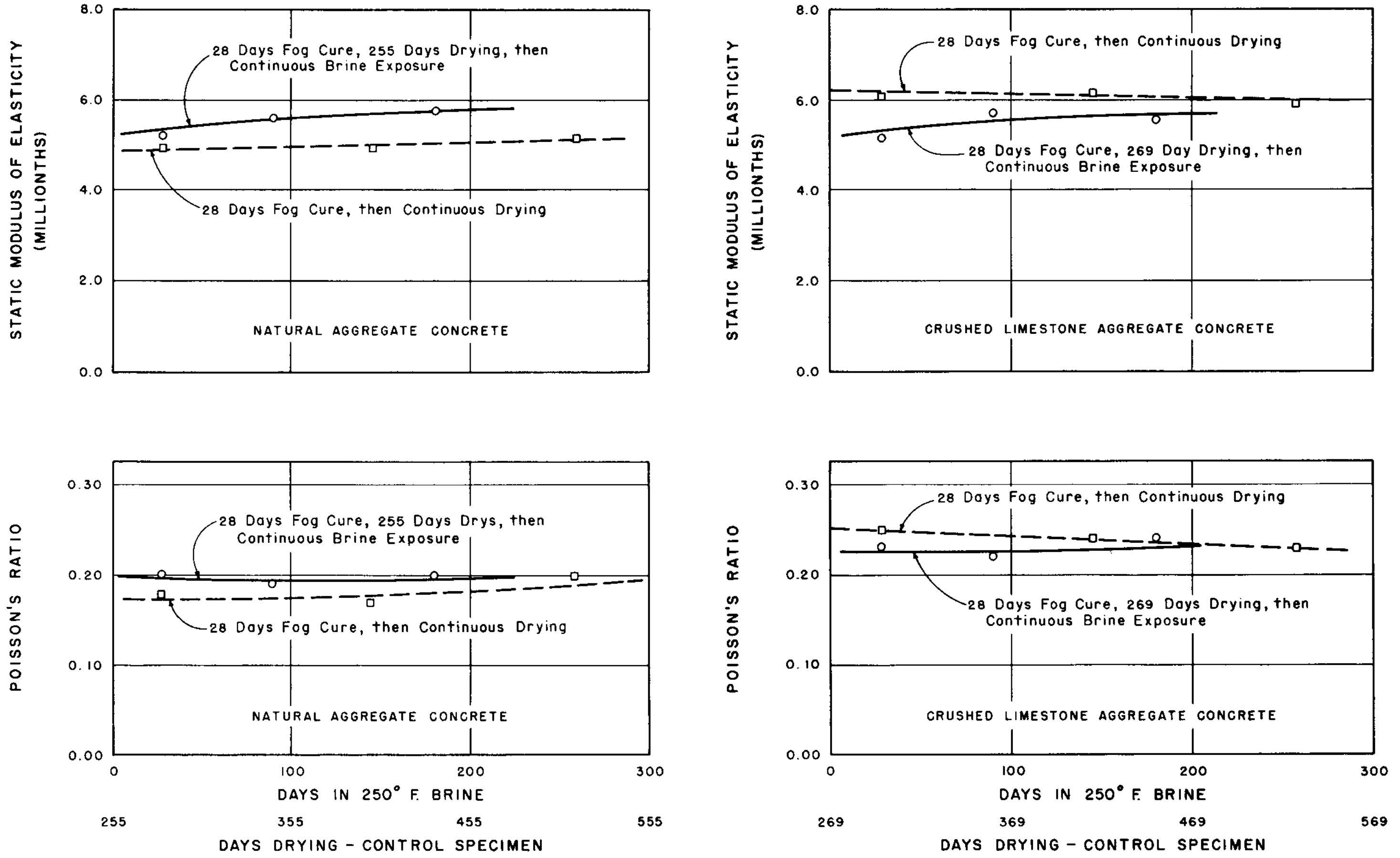

FIGURE III – 38: STATIC MODULI OF ELASTICITY AND POISSON'S RATIO OF NATURAL AGGREGATE CONCRETE AND CRUSHED LIMESTONE AGGREGATE CONCRETE SUBJECTED TO 250° F. SYNTHETIC SEA WATER BRINE, AND CONTINUOUSLY DRIED CONTROL CONCRETE. EACH PLOT REPRESENTS THE AVERAGE OF THREE DIFFERENT SPECIMENS

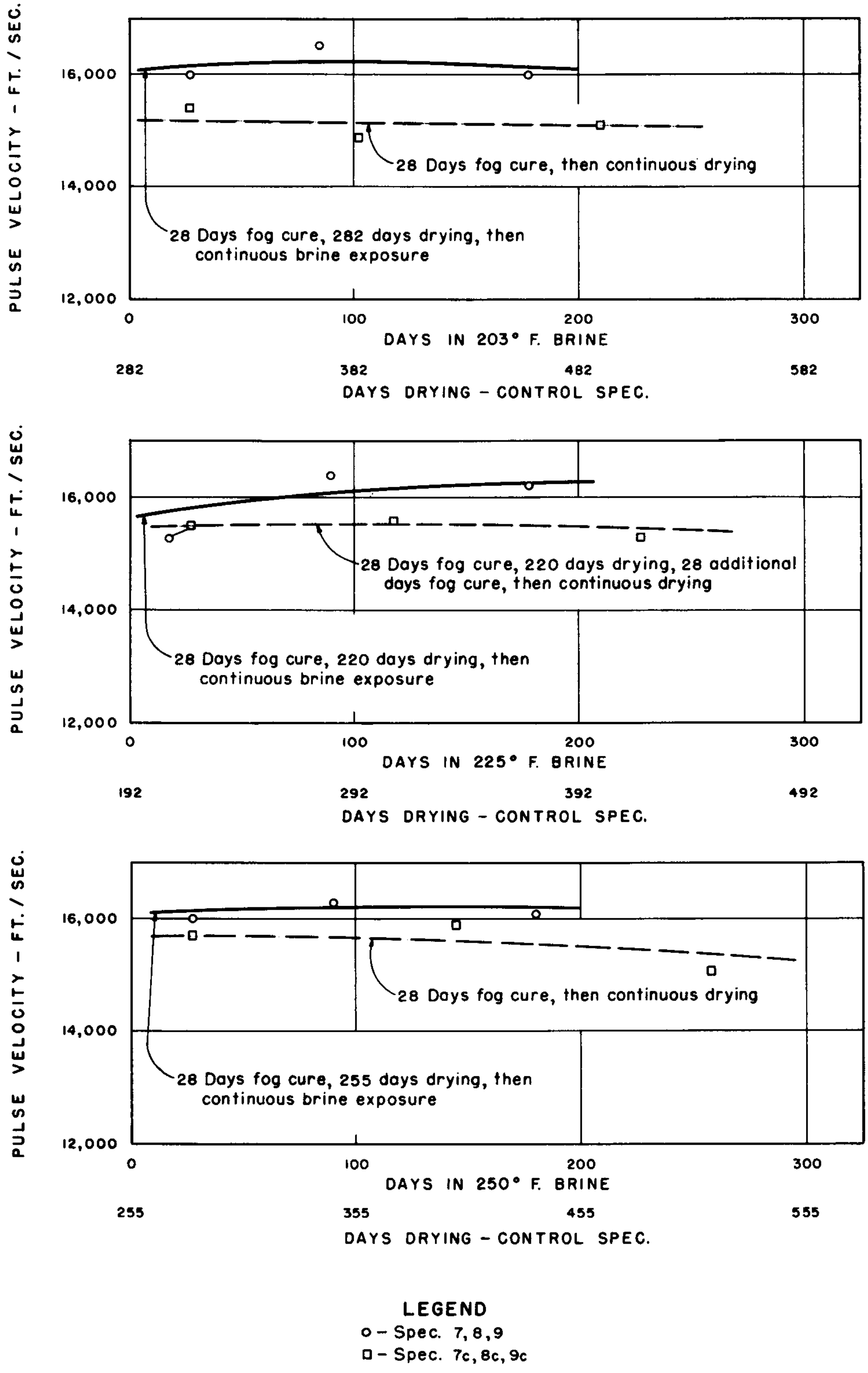

FIGURE III - 39 : PULSE VELOCITY OF NATURAL AGGREGATE CONCRETES SUBJECTED TO 203°, 225° & 250° F. SYNTHETIC SEA WATER BRINE

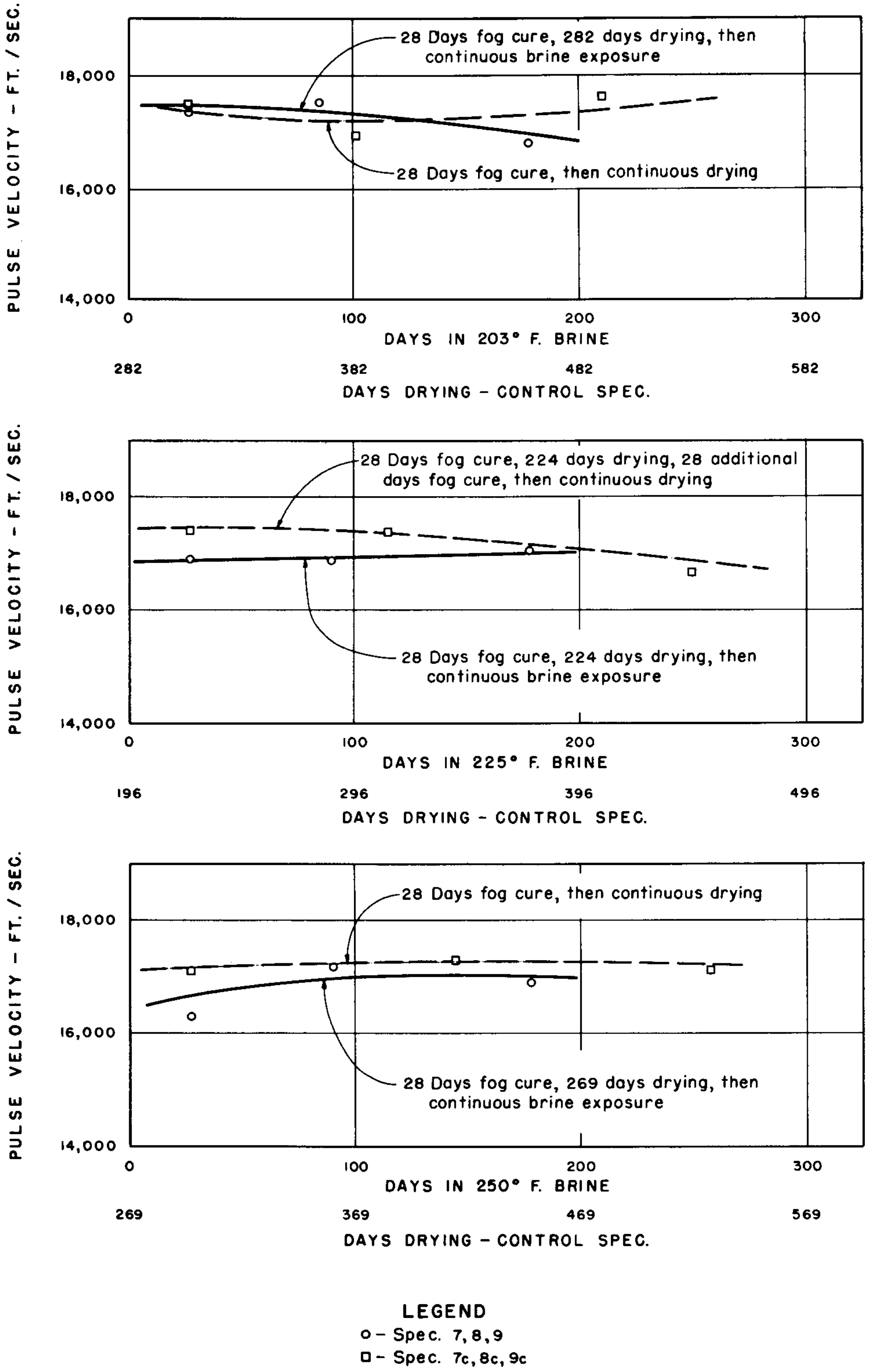

FIGURE III - 40: PULSE VELOCITY OF CRUSHED LIMESTONE AGGREGATE CONCRETE SUBJECTED TO 203°, 225° & 250° F. SYNTHETIC SEA WATER BRINE

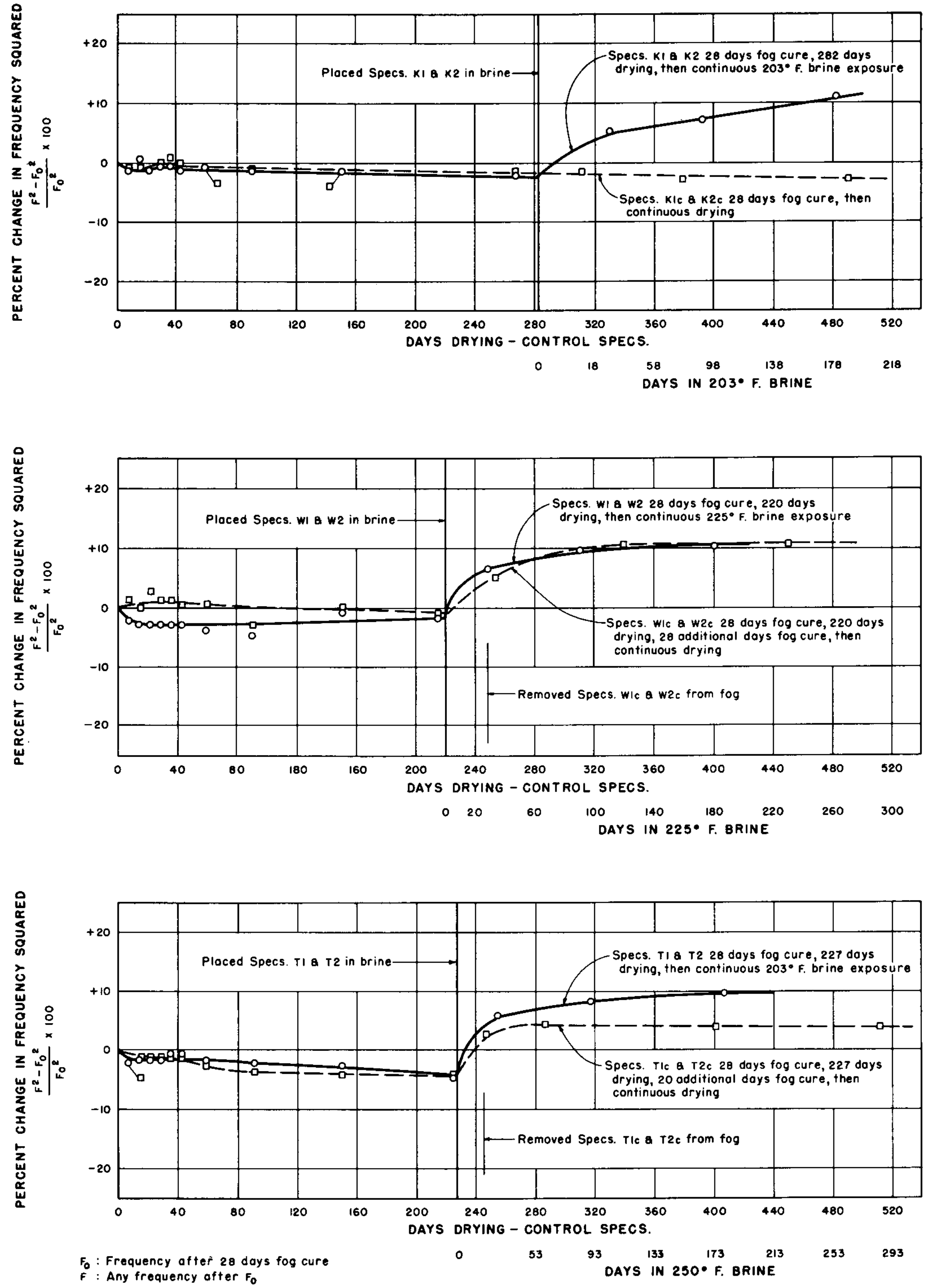

FIGURE III - 41 : CHANGES IN THE SQUARE OF THE RESONANT FREQUENCY OF NATURAL AGGREGATE CONCRETE SUBJECTED TO 203° F., 225° F. AND 250° F. BRINES

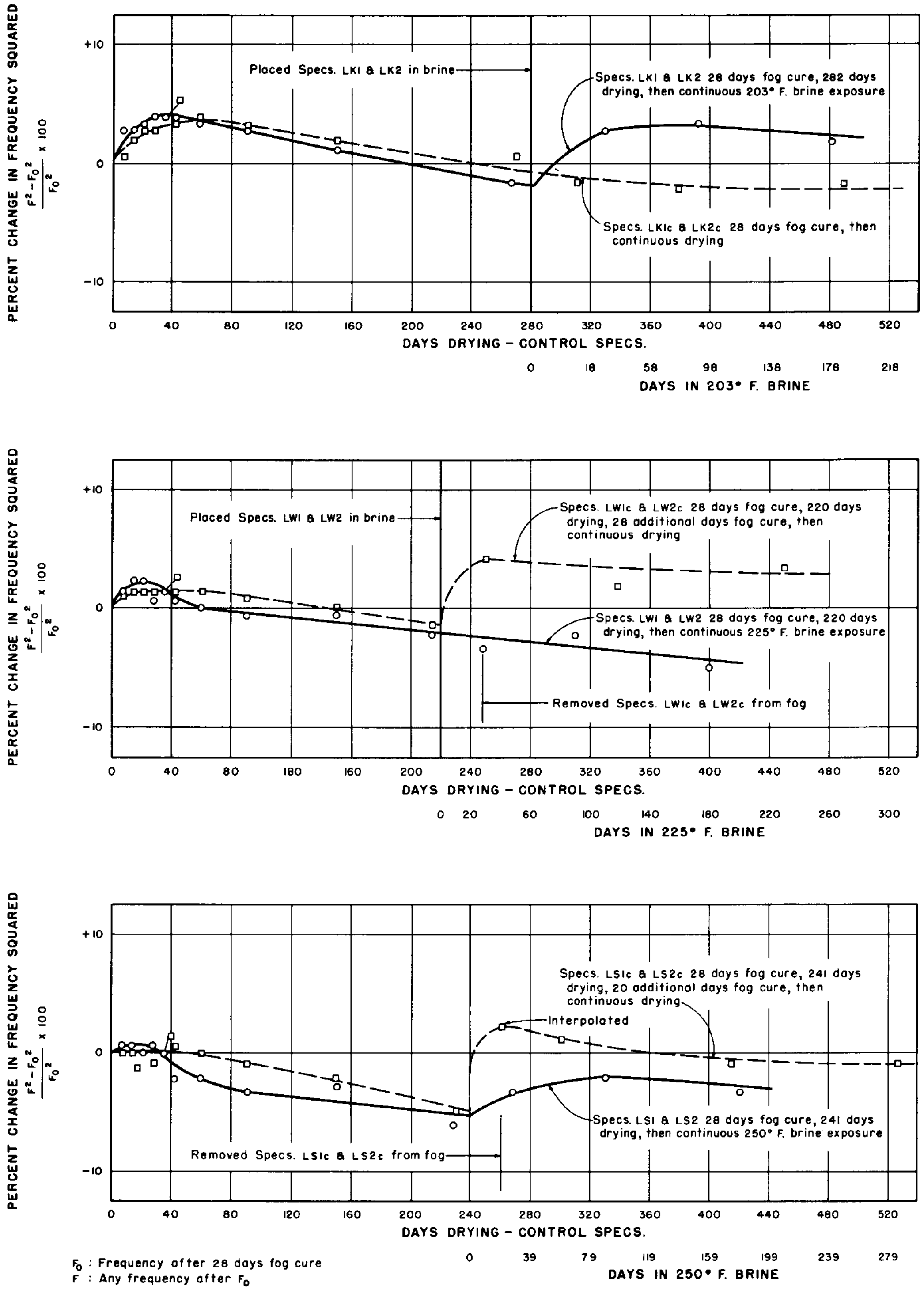

FIGURE III - 42: CHANGES IN THE SQUARE OF THE RESONANT FREQUENCY OF CRUSHED LIMESTONE AGGREGATE CONCRETE SUBJECTED TO 203° F., 225° F. AND 250° F. BRINES

Table III-33

COMPRESSIVE STRENGTH AND ELASTIC PROPERTIES
6- by 12-inch Concrete Cylinders
250° F System
Crushed Limestone Aggregate Concrete
Corrosion of Concrete Investigation for OSW

Specimen No.	Fog cure (days)	Dry cure (days)	In brine (days)	Total age (days)	Compressive strength (psi)	E (million psi)	r	Unit weight pounds per cu foot	Pulse velocity foot per second
CCI-LS35	7	0	0	7	4,440	4.60	0.22	-	-
CCI-LS36	7	0	0	7	4,280	4.82	0.20	-	-
Average					4,360	4.71	0.21	-	-
CCI-LS37	28	0	0	28	5,980	5.11	0.23	-	-
CCI-LS38	28	0	0	28	6,030	5.68	0.23	-	-
Average					6,000	5.40	0.23	-	-
CCI-LS1	28	269	28	325	5,980	4.98	0.23	149.5	15,900
CCI-LS2	28	269	28	325	5,760	5.32	0.24	149.5	15,900
CCI-LS3	28	269	28	325	5,780	5.15	0.21	149.5	15,900
Average					5,840	5.15	0.23	149.5	15,900
CCI-LS1C	28	297	0	325	8,170	6.21	0.25	149.5	16,600
CCI-LS2C	28	297	0	325	7,780	6.06	0.24	150.1	16,900
CCI-LS3C	28	297	0	325	7,850	6.01	0.26	148.3	16,900
Average					7,930	6.09	0.25	149.3	16,800
CCI-LS4	28	324	90	442	6,580	5.45	0.20	151.4	17,500
CCI-LS5	28	324	90	442	6,440	5.88	0.24	152.6	17,200
CCI-LS6	28	324	90	442	6,790	5.77	0.23	149.5	17,200
Average					6,600	5.70	0.22	151.2	17,300
CCI-LS4C	28	414	0	442	8,210	6.00	0.21	148.9	17,200
CCI-LS5C	28	414	0	442	8,100	6.40	0.25	150.1	17,000
CCI-LS6C	28	414	0	442	7,830	6.00	0.26	149.5	16,900
Average					8,050	6.13	0.24	149.5	17,000

Table III-33 Continued

COMPRESSIVE STRENGTH AND ELASTIC PROPERTIES
6- by 12-inch Concrete Cylinders
250° F System
Crushed Limestone Aggregate Concrete
Corrosion of Concrete Investigation for OSW

Specimen No.	Fog cure (days)	Dry cure (days)	In brine (days)	Total age (days)	Compressive strength (psi)	E (million psi)	r	Unit weight pounds per cu foot	Pulse velocity foot per second
CCI-LS7	28	347	180	555	7,460	5.65	0.25	150.1	17,000
CCI-LS8	28	347	180	555	6,990	5.30	0.22	151.4	16,800
CCI-LS9	28	347	180	555	6,670	5.66	0.25	152.6	17,000
Average					7,040	5.54	0.24	151.4	16,900
CCI-LS7c	28	527	0	555	8,600	5.90	0.23	148.9	16,900
CCI-LS8c	28	527	0	555	8,140	5.79	0.23	147.6	17,000
CCI-LS9c	28	527	0	555	8,520	6.00	0.24	149.5	17,400
Average					8,420	5.90	0.23	148.7	17,100

Table III-34

FREQUENCY SQUARED OF NATURAL AGGREGATE CONCRETE
4- by 4- by 30-inch Bars
250° F System
Corrosion of Concrete Investigation for OSW

Days1/ drying	Days brine	Total age	5/Frequency Squared Times 10^{-5} Specimens 1 & 2	Specimens 1c & 2c	Specimens 6/3 & 4	Specimens 6/3c & 4c	Specimens 7/7 & 8	Specimens 7/7c & 8c
0	0	28	4/4.60	4/4.69	4.69	4.65	3.62	3.72
7	0	35	4.49	4.60	4.52	4.49	3.70	3.72
14	0	42	4.52	4.62	4.49	4.46	3.72	3.74
21	0	49	4.52	4.62	4.56	4.56	3.72	3.82
28	0	56	4.52	4.62	4.62	4.38	3.70	3.74
35	0	63	4.56	4.62	4.62	4.49	3.74	3.84
42	0	70	4.56	4.62	4.62	4.49	3.78	3.84
59	0	87	4.52	4.56	4.52	4.38	3.72	3.82
91	0	119	4.49	4.52	4.49	4.46	3.72	3.78
150	0	178	4.46	4.49	4.42	4.38	3.66	3.70
225	0	253	4.38	4.49	4.33	4.33	3.74	3.78
2/287	3/28	315	4.87	4.90	4.83	4.87	3.91	4.10
401	90	429	4.97	4.87	5.01	4.83	4.07	4.12
513	180	541	5.04	4.87	5.04	4.83	4.10	4.12

1/All specimens received an initial 28-day fog cure.
2/Days drying from hereon only applies to Specimens 1c and 2c, 3c and 4c, and 7c and 8c.
3/Days brine only applies to Specimens 1 and 2, 3 and 4, and 7 and 8.
4/These values are F_o as referred to in Figure III-41.
5/Units for frequency (F) are cycles per second.
6/Reinforced concrete specimens.
7/Pretensioned concrete specimens.

Table III-35

FREQUENCY SQUARED OF CRUSHED LIMESTONE AGGREGATE CONCRETE
4- by 4- by 30-inch Bars
250° F System
Corrosion of Concrete Investigation for OSW

			5/Frequency Squared Times 10^{-5}					
Days1/ drying	Days brine	Total age	Specimens 1 & 2	Specimens 1c & 2c	Specimens 6/3 & 4	Specimens 6/3c & 4c	Specimens 7/7 & 8	Specimens 7/7c & 8c
0	0	28	5.18	5.15	5.11	5.25	3.94	3.78
7	0	35	5.21	5.15	5.11	5.21	4.03	3.91
14	0	42	5.21	5.15	5.07	5.21	4.07	3.91
21	0	49	5.18	5.15	5.07	5.21	4.07	3.91
28	0	56	5.21	5.11	5.11	5.21	4.10	3.91
35	0	63	5.18	5.15	5.07	5.04	4.10	3.91
42	0	70	5.07	5.18	4.97	5.07	4.07	3.91
60	0	88	5.07	5.15	5.01	5.21	4.07	3.91
91	0	119	5.01	5.11	5.07	5.14	4.07	3.91
151	0	179	5.04	5.04	4.97	5.14	4.10	3.91
239	0	267	4.87	4.90	4.90	4.97	3.99	3.87
2/301	3/28	279	5.01	5.21	4.90	5.30	4.10	4.03
		329				5.44		4.10
415	90	443	5.07	5.11	4.97	5.21	4.20	4.12
527	180	555	5.01	5.11	4.90	5.30	4.23	4.12

1/All specimens received an initial 28-day fog cure.
2/Days drying from hereon only applies to Specimens 1c and 2c, 3c and 4c, and 7c and 8c.
3/Days brine only applies to Specimens 1 and 2, 3 and 4, and 7 and 8.
4/These values are F_o as referred to in Figure III-42.
5/Units for frequency (F) are cycles per second.
6/Reinforced concrete specimens.
7/Pretensioned concrete specimens.

about 600 millionths inch per inch which is just slightly above average for high-quality concrete, Table III-36 and Figure III-43. After immersion in the 250° F synthetic sea water brine for 28 days about 80 percent of this shrinkage, 483 millionths per inch, was regained leaving a net shrinkage of 118 millionths inch per inch. After 180 days' exposure the net shrinkage dropped to 111 millionths inch per inch. Certainly no abnormal behavior of the concrete is apparent thus far.

Data obtained from tests conducted upon the plain crushed limestone aggregate concrete bars indicated that the drying shrinkage of the concrete prior to brine exposure totaled about 330 millionths inch per inch which is substantially less than that which occurred in the natural aggregate concrete and is considerably below that maximum considered average for good concrete, Table III-37 and Figure III-44. After immersion in the 250° F brine for 28 days, 551 millionths inch per inch expansion occurred, thus indicating that a greater volume now exists than originally, but certainly no signs of deterioration prevail.

Weight change of the two concretes behaved very similarly to the length change in response to drying and immersion in 250° F brine, Tables III-36 and III-37 and Figures III-43 and III-44. Both of these measurements increased when moisture was present and decreased when moisture was removed.

Companion concrete specimens were initially placed in the 100 percent RH atmosphere when the primary specimens were placed in the brine. However, a short time later they were placed back in the 50 percent RH atmosphere since it was concluded that as companion specimens the drying concrete would provide more suitable data. This change of exposure explains the shape of the companion curves in all of the figures dealing with the 250° and 225° F tests.

Results of tests conducted upon natural and crushed limestone aggregate reinforced concretes revealed that length and weight changes followed the same trends as those of the plain unreinforced concretes, Figures III-45 and III-46. Although these reinforced specimens are principally used to evaluate steel corrosion, the aforementioned test results affirm that nothing unusual is occurring in the specimens.

Contrary to what might be expected, the length change of pretensioned natural and crushed limestone course aggregate concretes underwent rather abrupt shrinkages when exposed to the 250° F brine, Figures III-47 and III-48. A careful analysis of this situation revealed that the creep of the concrete under load was probably greater than the expansion due to absorption thus resulting in greater total negative length change. Although no tests have been conducted in the Bureau laboratory to determine creep of concrete exposed to elevated temperature and moisture conditions, experience and related data indicate that concrete exposed to high temperatures and changing moisture conditions undergo greater creep than concrete at ambient conditions. Creep is discussed in detail in Section IV; however, a comparison between the curves of Figures III-43 and III-47 indicates that after 28 days' brine exposure the natural aggregate concrete must have undergone about 976 millionths inch per inch creep to have overcome the 483 millionths (+) inch per inch expansion to absorption. The fact that the apparent 1,061 millionths inch per inch creep of the crushed limestone aggregate concrete is 85 millionths inch per inch greater than that which occurred in the natural aggregate concrete can probably be attributed to the 720 psi greater pretensioning stress.

Notice that a normal weight increase occurred when the concrete was placed in the brine. Since a weight increase of this type is usually accompanied by expansion, it is apparent from the length change curve that the magnitude of the creep is still increasing slightly more than any expansion that might be occurring.

III. 6. 2. 2. 4. 225° F brine tests. --

The general condition of the concrete subjected for 180 days to the 225° F synthetic sea water brine can be considered good. No evidence of deterioration exists.

Table III-36

LENGTH AND WEIGHT CHANGE OF NATURAL AGGREGATE CONCRETE
250° F Temperature Brine System
4- by 4- by 30-inch Bars
Corrosion of Concrete Investigation for OSW

Total age (days)		35	42	56	87	119	178	253	315	429	541	
	Fog	28	28	28	28	28	28	28	28	28	28	28
Cure (days)	Dry	7 (7)	14 (14)	28 (28)	59 (59)	91 (91)	151 (151)	225 (225)	259 (287)	311 (401)	333 (513)	
	Brine	0 (0)	0 (0)	0 (0)	0 (0)	0 (0)	0 (0)	0 (0)	28 (0)	90 (0)	180 (0)	
Specimen Number	**Type[1]**											
						Length Change in Millionths Inch Per Inch						
1 & 2	Pl	211	283	354	465	523	567	601	118	130	111	
1c & 2c	(Pl)[2]	193	269	337	447	509	542	587	386[3]	517	561	
3 & 4	Re	157	208	268	337	377	410	416	+64[4]	+76	+82	
3c & 4c	(Re)	175	212	265	333	370	407	416	236[3]	328	368	
7 & 8	Prs	264	351	432	544	609	703	751	858	906	925	
7c & 8c	(Prs)	271	351	454	569	623	703	748	601	724	786	
						Weight Change in Pounds						
1 & 2	Pl	0.32	0.45	0.58	0.70	0.78	0.85	0.92	+0.03	+0.20	+0.28	
1c & 2c	(Pl)	0.31	0.44	0.55	0.66	0.76	0.86	0.91	0.33[3]	0.56	0.62	
3 & 4	Re	0.29	0.42	0.53	0.65	0.72	0.80	0.87	+0.01	+0.17	+0.27	
3c & 4c	(Re)	0.34	0.43	0.54	0.64	0.68	0.78	0.87	0.26[3]	0.46	0.51	
7 & 8	Prs	0.25	0.34	0.42	0.50	0.54	0.62	0.69	+0.04	+0.20	+0.30	
7c & 8c	(Prs)	0.28	0.36	0.47	0.55	0.59	0.67	0.72	0.24[3]	0.36	0.43	

[1] Pl = Plain; Re = Reinforced; Prs = Prestressed

[2] All ( ) refer to companion specimens which were not placed in brine, but left in laboratory air.

[3] Placed in fog June 7, 1966, but returned to 50 percent room on June 29, 1966, after 22 days in the fog.

[4] All specimen readings indicate shrinkage compared to the 28-day fog readings except those marked with a "+" which indicates expansion.

Table III-37

LENGTH AND WEIGHT CHANGE OF CRUSHED LIMESTONE AGGREGATE CONCRETE
250° F Temperature Brine System
4- by 4- by 30-inch Bars
Corrosion of Concrete Investigation for OSW

Total age (days):		35	42	56	88	120	179	267	329	443	555			
Cure (days)	Fog	28	28	28	28	28	28	28	28	28	28	28	28	28
	Dry	7	14	28	60	92	151	239	273	325	347			
		(7)	(14)	(28)	(60)	(92)	(151)	(239)	(301)	(415)	(527)			
	Brine	0	0	0	0	0	0	0	28	90	180			
		(0)	(0)	(0)	(0)	(0)	(0)	(0)	(0)	(0)	(0)			
Specimen Number	Type[1]	Length Change in Millionths Inch Per Inch												
1 & 2	Pl	108	135	178	242	270	308	329	+222[4]	+226	+268			
1c & 2c	(Pl)[2]	90	131	190	227	253	293	308	219[3]	278	304			
3 & 4	Re	92	127	172	203	220	254	261	+272	+398	+486			
3c & 4c	(Re)	97	120	162	192	217	243	258	159[3]	221	236			
7 & 8	Prs	159	221	313	375	421	477	519	839	910	961			
7c & 8c	(Prs)	148	221	295	368	414	487	533	442[3]	522	555			
		Weight Change in Pounds												
1 & 2	Pl	0.27	0.34	0.41	0.49	0.55	0.59	+0.66	+0.15	+0.17	+0.23			
1c & 2c	(Pl)	0.26	0.33	0.42	0.49	0.54	0.59	0.67	0.29[3]	0.40	0.47			
3 & 4	Re	0.28	0.26	0.33	0.40	0.46	0.49	0.57	+0.22	+0.40	+0.50			
3c & 4c	(Re)	0.33	0.36	0.42	0.51	0.56	0.60	0.68	0.22[3]	0.36	0.40			
7 & 8	Prs	0.22	0.26	0.30	0.34	0.38	0.46	0.52	+0.18	+0.26	+0.36			
7c & 8c	(Prs)	0.25	0.22	0.29	0.35	0.39	0.43	0.49	0.09[3]	0.19	0.25			

[1] Pl = Plain; Re = Reinforced; Prs = Prestressed
[2] All ( ) refer to companion specimens which were not placed in brine, but left in laboratory air.
[3] Placed in fog June 7, 1966, but returned to 50 percent room on June 29, 1966.
[4] All specimen readings indicate shrinkage compared to the 28-day fog readings except those marked with a "+" which indicates expansion.

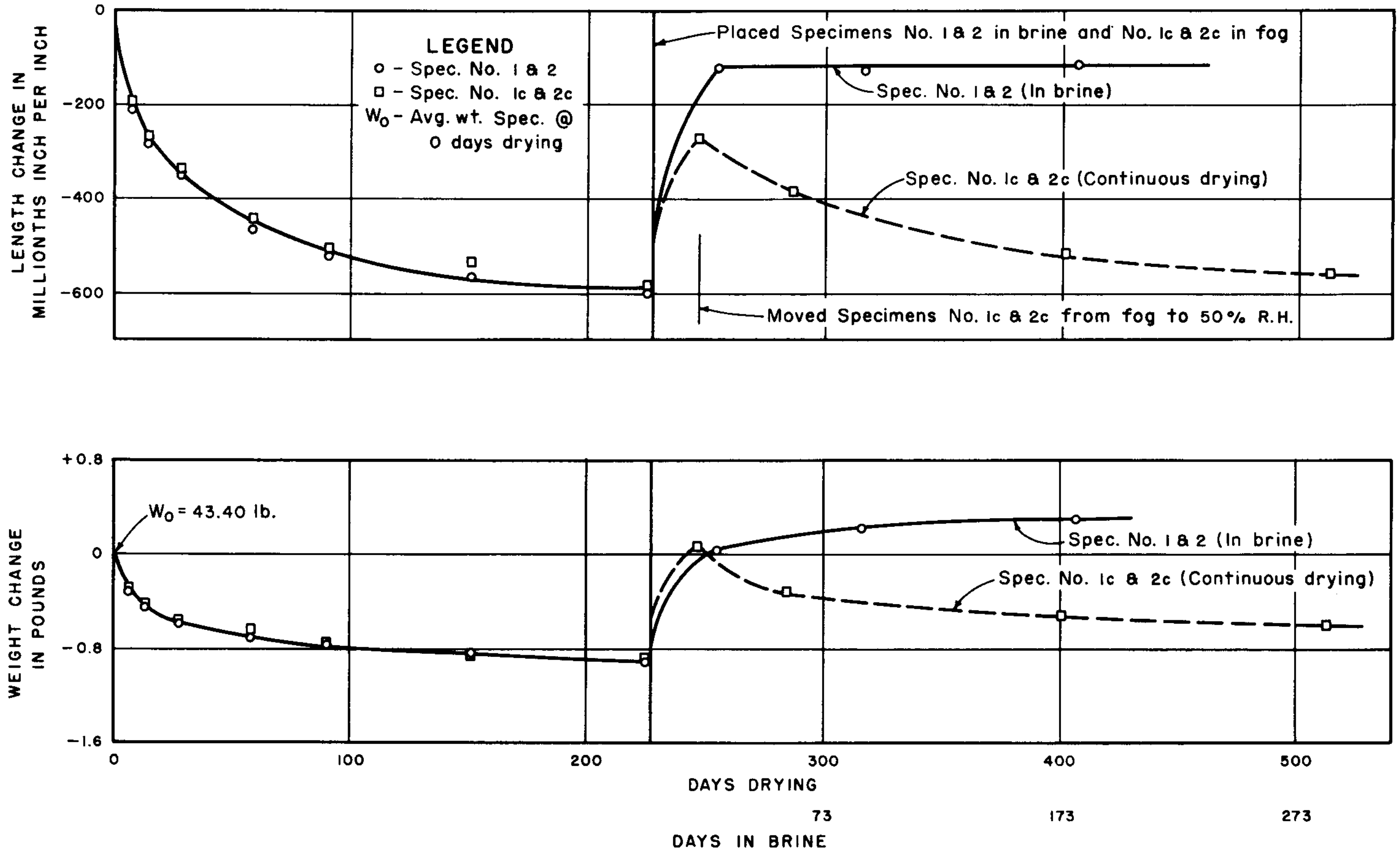

FIGURE III — 43: LENGTH AND WEIGHT CHANGE OF PLAIN NATURAL AGGREGATE CONCRETE SUBJECTED TO 250° F. BRINE AND CONTINUOUS DRYING

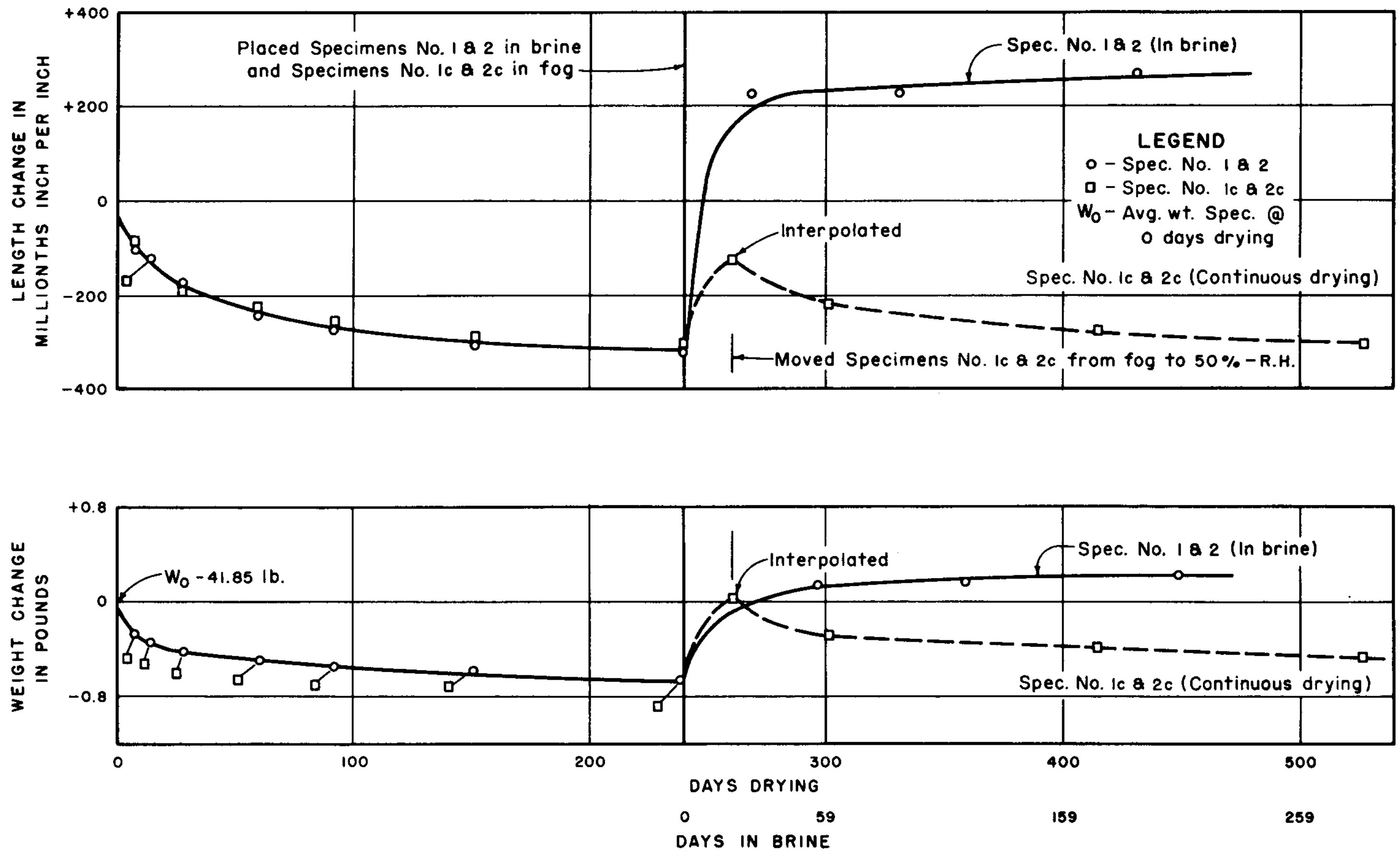

FIGURE III – 44: LENGTH AND WEIGHT CHANGE OF PLAIN CRUSHED LIMESTONE AGGREGATE CONCRETE SUBJECTED TO 250° BRINE AND CONTINUOUS DRYING

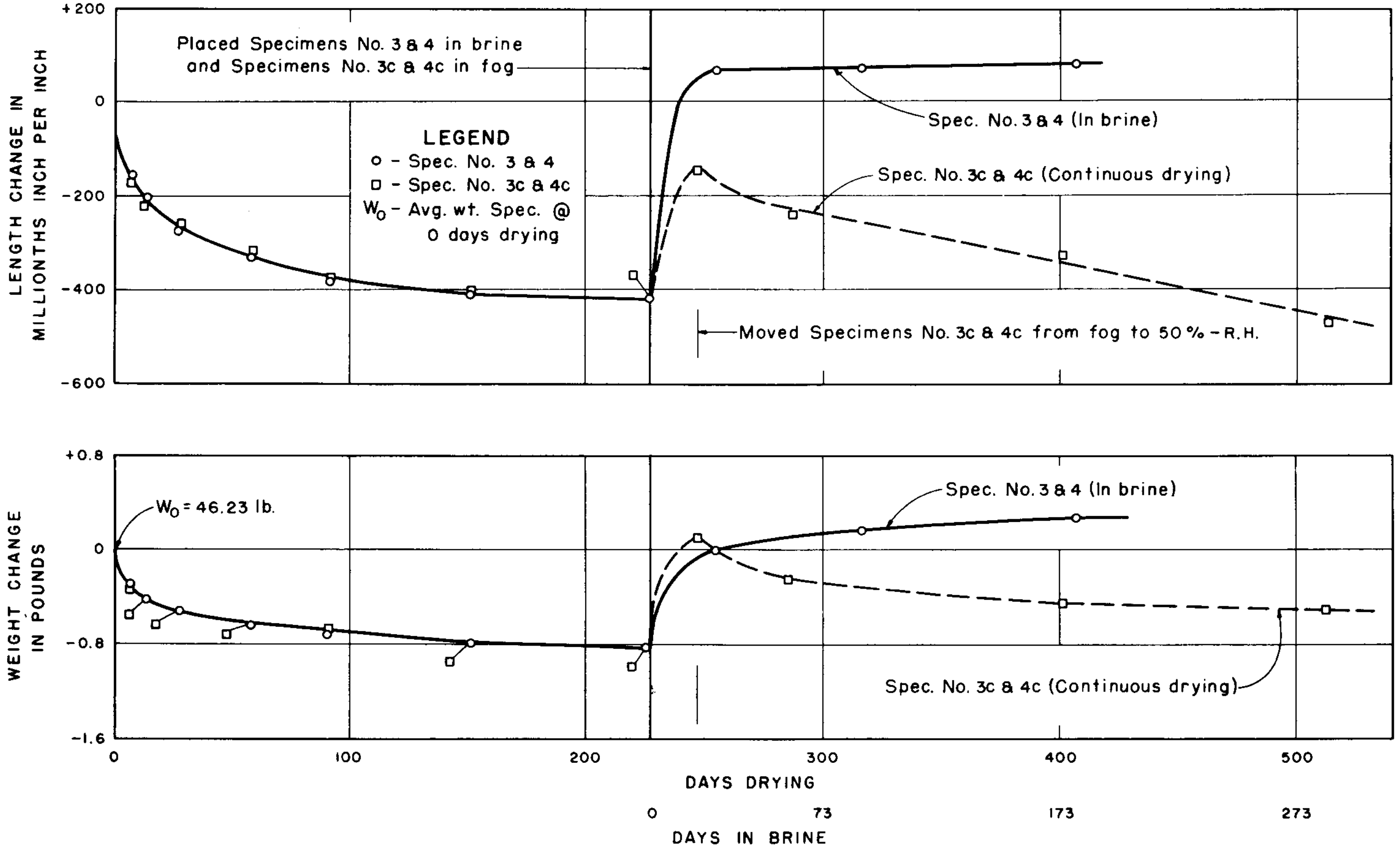

FIGURE III - 45: LENGTH AND WEIGHT CHANGE OF REINFORCED NATURAL AGGREGATE CONCRETE SUBJECTED TO 250° F. BRINE AND CONTINUOUS DRYING

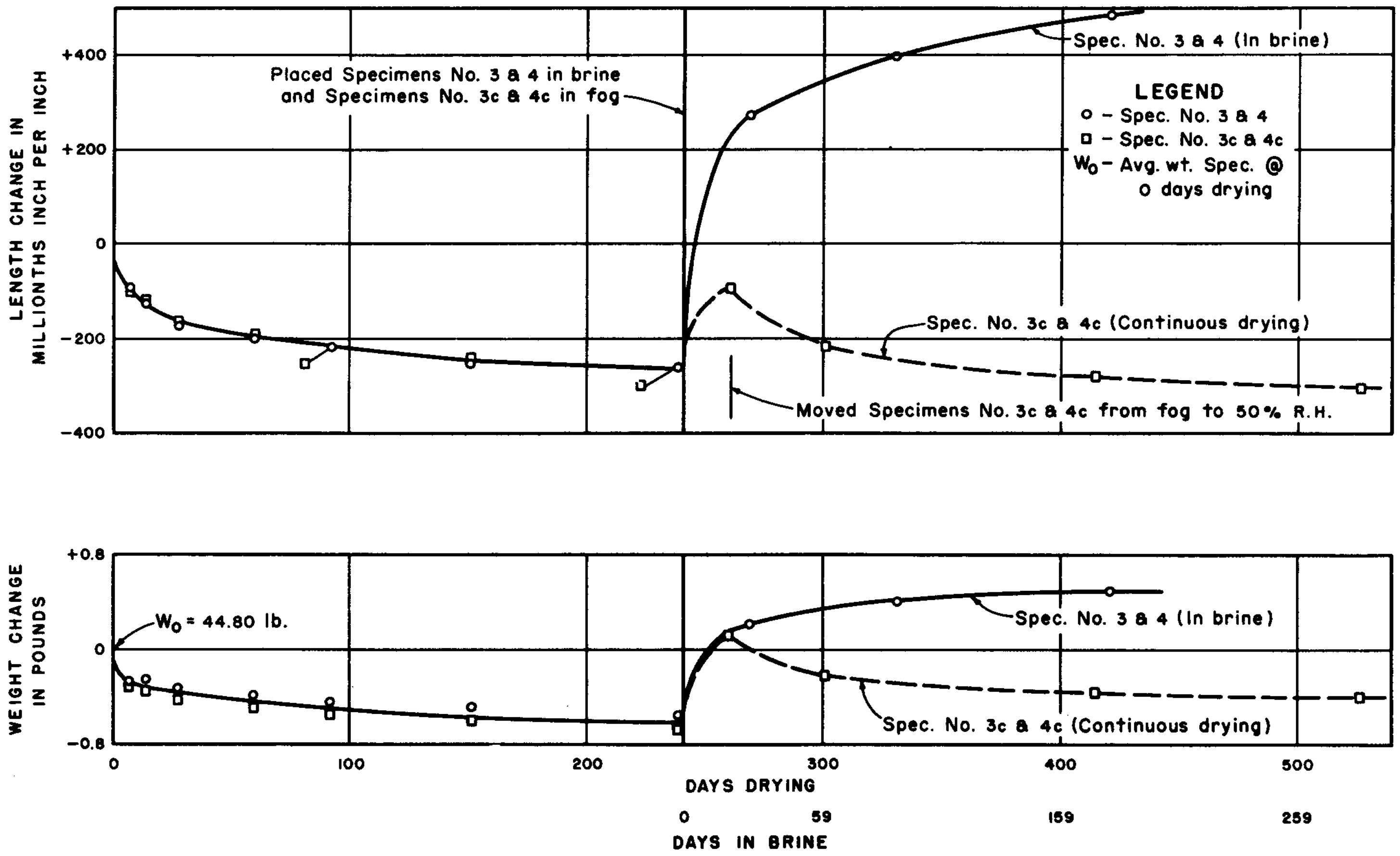

FIGURE III – 46: LENGTH AND WEIGHT CHANGE OF REINFORCED CRUSHED LIMESTONE AGGREGATE CONCRETE SUBJECTED TO 250° F. BRINE AND CONTINUOUS DRYING

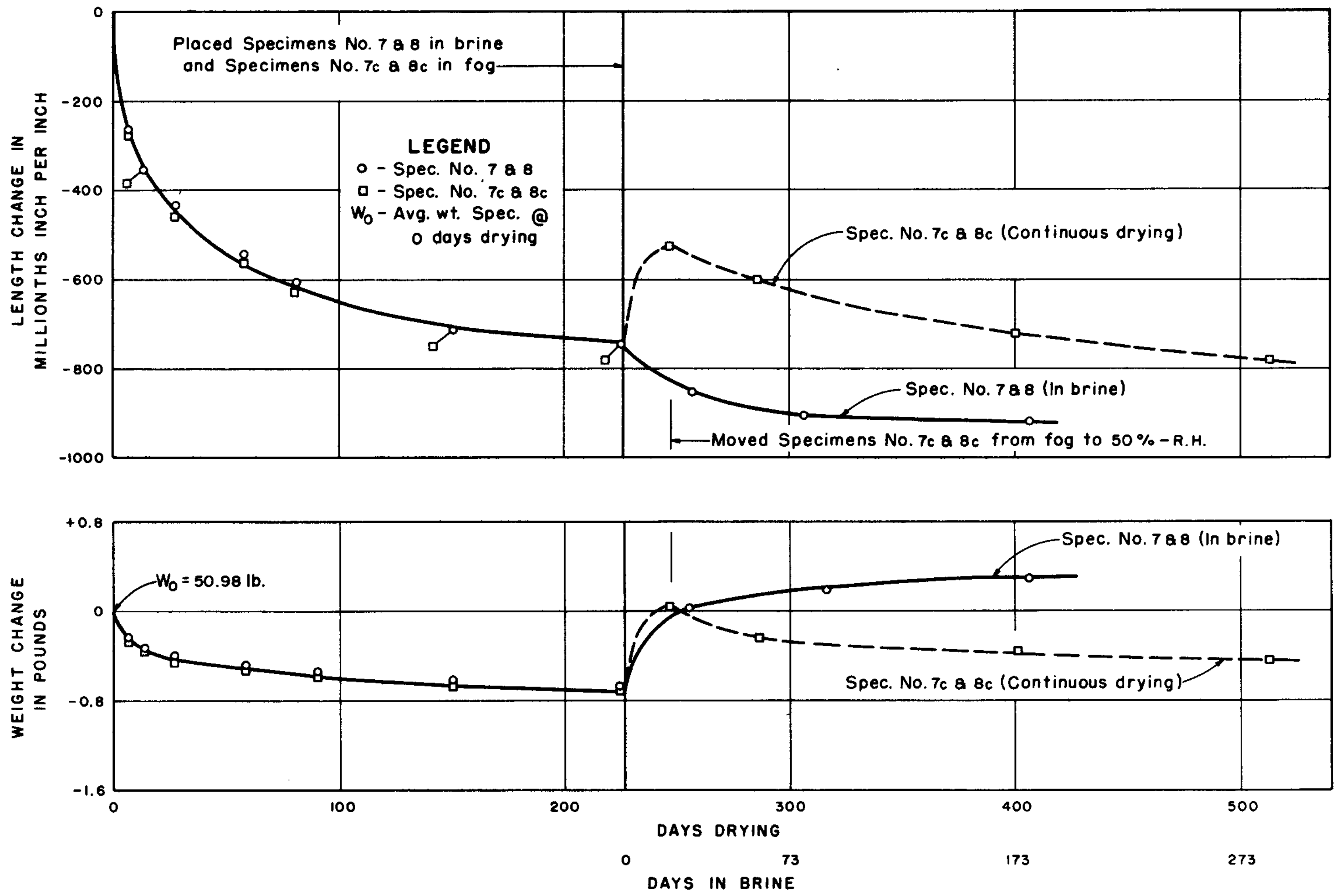

FIGURE III - 47: LENGTH AND WEIGHT CHANGE OF PRESTRESSED NATURAL AGGREGATE CONCRETE SUBJECTED TO 250° F. BRINE AND CONTINUOUS DRYING

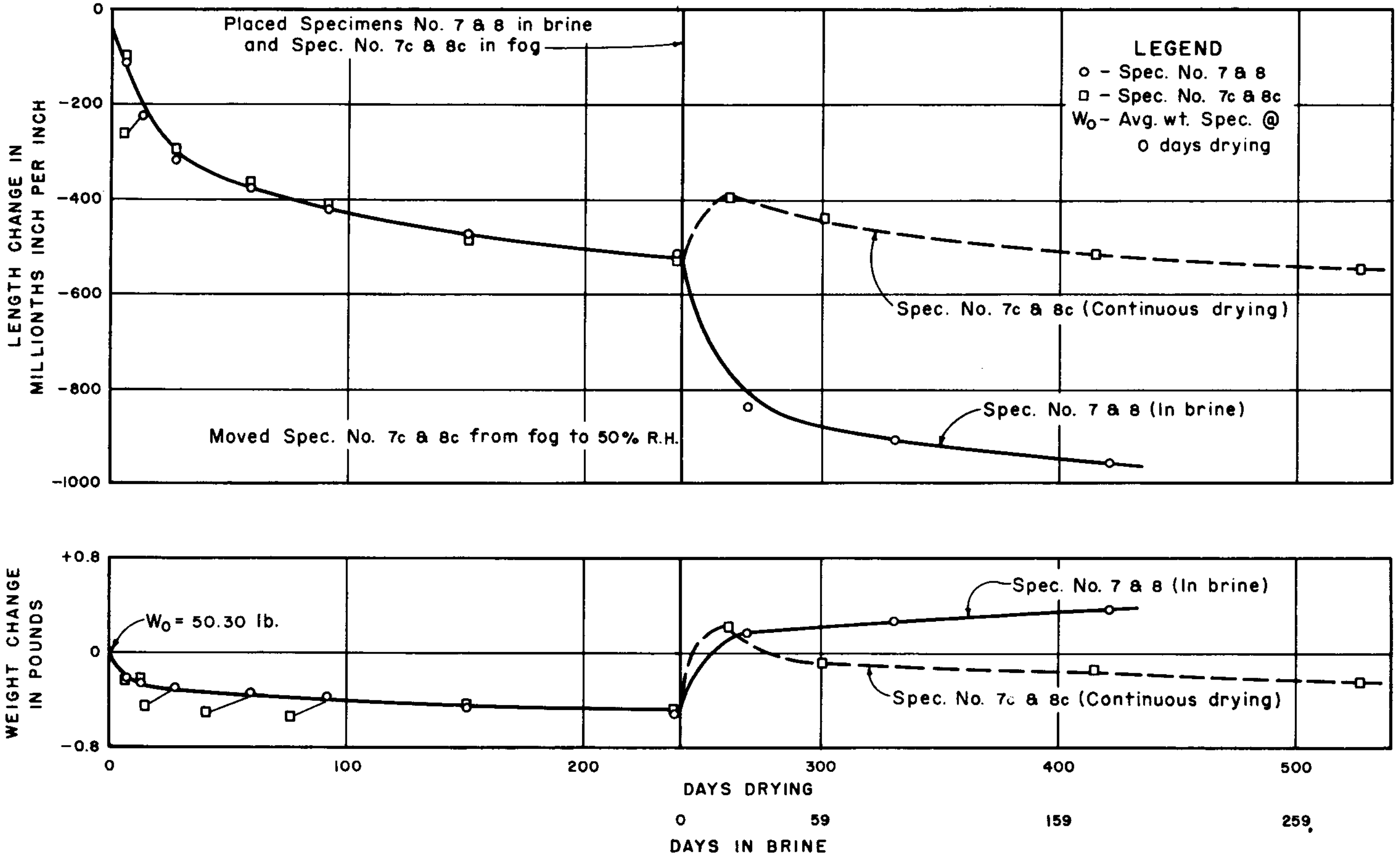

FIGURE III – 48: LENGTH AND WEIGHT CHANGE OF PRESTRESSED CRUSHED LIMESTONE AGGREGATE CONCRETE SUBJECTED TO 250° F. BRINE AND CONTINUOUS DRYING

Compressive strength of the natural aggregate concrete subjected to the 225° F synthetic sea water brine averaged 7, 130, 7, 910, and 8, 280 psi for exposures of 28, 90, and 180 days, respectively, Table III-38, Figure III-19.

This strength progression is comparable to that developed by similar concrete undergoing other hot brine exposures, and in no way indicates deteriorated concrete.

Compressive strength of crushed limestone aggregate concrete subjected to the 225° F synthetic sea water brine averaged 6, 540, 6, 970, and 7, 630 psi for exposures of 28, 90, and 180 days, respectively, Table III-39, Figure III-37. Here again, this strength progression is considered normal and no signs of deterioration exists.

Modulus of elasticity of the natural and crushed limestone aggregate concrete subjected to the 225° F synthetic sea water brine averaged 5. 67 and 5. 38 million psi, respectively, at 180 days' exposure, which in both cases is an increase from the values obtained after 28 days' exposure, Tables III-38 and III-39 and Figure III-49. In the case of the crushed limestone aggregate concrete the values obtained after 90 days' exposure were substantially greater than the aforementioned 180-day values. No explanation for this phenomenon is presently available.

Poissons' ratio of the natural and crushed limestone aggregate concrete exposed to the 225° F synthetic sea water brine did not change throughout the test period, and the values of 0. 20 and 0. 23, respectively, are about average for good concrete, Tables III-38 and III-39, and Figure III-49.

Unit weights of the natural and crushed aggregate concrete subjected to the 225° F synthetic sea water brine averaged 157. 6 and 150. 5 pounds per cubic foot at 180 days' brine exposure, Tables III-38 and III-39. The change in unit weight throughout the test period has been rather insignificant, apparently indicating that a stable moisture condition has been attained.

Pulse velocity of natural and crushed limestone aggregate concrete subjected to 225° F synthetic sea water brine increased only slightly between 28 and 180 days' exposure, Tables III-38 and III-39, and Figures III-39 and III-40. Pulse velocity of crushed limestone aggregate concrete averaged substantially more than that of the natural aggregate concrete which probably would not be expected since the natural aggregate concrete has a higher unit weight and higher static modulus of elasticity. This same phenomenon occurred in the concretes subjected to the 203° and 250° F systems, and at this point no valid explanation for this situation is available. However, in all cases pulse velocities are above 15, 000 feet per second and are indicative of the continued high quality of the concretes.

Resonant frequency of the natural and crushed limestone aggregate concretes underwent the same initial increase upon subjection to the 225° F synthetic sea water brine as occurred in the other brine environments, Tables III-40 and III-41 and Figures III-41 and III-42. At 180 days' exposure the frequency of the natural aggregate concrete continued to increase whereas the frequency of the crushed limestone aggregate concrete exhibited a slight decrease. In neither concrete are there any signs of deterioration based on the tests.

Data obtained from tests conducted on the plain natural aggregate concrete bars indicated that 68 percent or 406 millionths inch per inch of the 597 millionths inch per inch total prebrine drying shrinkage was regained after 28 days' exposure in the 225° F synthetic sea water brine, Table III-42 and Figure III-50. These figures are quite comparable to those obtained from tests conducted upon the 250° and 290° F exposed concrete.

Similar data obtained from tests conducted upon plain crushed limestone aggregate concrete indicated a total expansion of 463 millionths inch per inch resulting from 28 days' exposure to the 225° F brine, Table III-43 and Figure III-51. Since the maximum drying shrinkage prior to brine exposure totaled only 315 millionths inch per inch the volume of the concrete after 28 days' brine exposure is greater

Table III-38

COMPRESSIVE STRENGTH AND ELASTIC PROPERTIES
6- by 12-inch Concrete Cylinders
225° F System
Natural Aggregate Concrete
Corrosion of Concrete Investigation for OSW

Specimen No.	Fog cure (days)	Dry cure (days)	In brine (days)	Total age (days)	Compressive strength (psi)	E (million psi)	r	Unit weight pounds per cu foot	Pulse velocity foot per second
CCI-W35	7	0	0	7	3,750	3.58	0.17	-	-
CCI-W36	7	0	0	7	4,210	3.45	0.15	-	-
Average					3,980	3.52	0.16		
CCI-W37	28	0	0	28	5,620	4.30	0.22	-	-
CCI-W38	28	0	0	28	5,840	4.27	0.18	-	-
Average					5,730	4.28	0.20		
CCI-W1	28	220	28	276	7,000	5.37	0.21	157.0	16,100
CCI-W2	28	220	28	276	7,130	5.24	0.19	157.6	15,800
CCI-W3	28	220	28	276	7,270	5.40	0.21	158.2	16,100
Average					7,130	5.34	0.20	157.6	16,000
CCI-W1C	56[1]	220	0	276	6,760	4.56	0.20	155.8	15,600
CCI-W2C	56	220	0	276	6,920	4.33	0.14	158.2	15,400
CCI-W3C	56	220	0	276	6,720	4.59	0.17	158.2	15,300
Average					6,800	4.49	0.17	157.4	15,400
CCI-W4	28	248	90	366	7,710	5.58	0.23	157.0	16,300
CCI-W5	28	248	90	366	7,920	5.18	0.18	157.6	16,600
CCI-W6	28	248	90	366	8,100	5.78	0.20	160.7	16,900
Average					7,910	5.51	0.20	158.4	16,600
CCI-W4C	56[1]	310	0	366	7,450	4.67	0.11	154.5	16,200
CCI-W5C	56	310	0	366	8,140	4.91	0.15	154.5	15,800
CCI-W6C	56	310	0	366	8,280	5.10	0.16	155.8	15,700
Average					7,940	4.89	0.14	154.9	15,900

Table III-38--Continued

COMPRESSIVE STRENGTH AND ELASTIC PROPERTIES
6- by 12-inch Concrete Cylinders
225° F System
Natural Aggregate Concrete
Corrosion of Concrete Investigation for OSW

Specimen No.	Fog cure (days)	Dry cure (days)	In brine (days)	Total age (days)	Compressive strength (psi)	E (million psi)	r	Unit weight pounds per cu foot	Pulse velocity foot per second
CCI-W7	28	267	180	475	8,100	5.65	0.21	155.8	16,200
CCI-W8	28	267	180	475	8,280	5.42	0.20	157.6	16,100
CCI-W9	28	267	180	475	8,450	5.95	0.20	159.5	16,200
Average					8,280	5.67	0.20	157.6	16,200
CCI-W7C	56[1]	419	0	475	8,060	5.00	0.18	153.9	15,300
CCI-W8C	56	419	0	475	8,260	4.88	0.14	155.1	15,600
CCI-W9C	56	419	0	475	8,400	5.00	0.19	157.0	15,100
Average					8,240	4.96	0.17	155.3	15,300

1/During initial 28 days testing in brine, these companion cylinders were fog cured.

Table III-39

COMPRESSIVE STRENGTH AND ELASTIC PROPERTIES
6- by 12-inch Concrete Cylinders
225° F System
Crushed Limestone Aggregate Concrete
Corrosion of Concrete Investigation for OSW

Specimen No.	Fog cure (days)	Dry cure (days)	In brine (days)	Total age (days)	Compressive strength (psi)	E (million (psi)	r	Unit weight pounds per cu foot	Pulse velocity foot per second
CCI-LW35	7	0	0	7	4,160	4.87	0.24	-	-
CCI-LW36	7	0	0	7	4,320	4.78	0.21	-	-
Average					4,240	4.82	0.22		
CCI-LW37	28	0	0	28	6,440	5.74	0.27	-	-
CCI-LW38	28	0	0	28	6,580	5.75	0.23	-	-
Average					6,510	5.74	0.25		
CCI-LW1	28	224	28	280	6,540	5.34	0.22	148.9	17,000
CCI-LW2	28	224	28	280	6,530	5.26	0.22	151.4	16,800
CCI-LW3	28	224	28	280	6,540	5.41	0.23	152.6	16,900
Average					6,540	5.34	0.22	151.0	16,900
CCI-LW1C	56[1]	224	0	280	7,910	5.74	0.23	152.0	17,000
CCI-LW2C	56	224	0	280	7,850	6.02	0.22	152.6	17,400
CCI-LW3C	56	224	0	280	8,080	5.86	0.24	152.0	17,700
Average					7,950	5.87	0.23	152.2	17,400
CCI-LW4	28	248	90	366	6,760	5.85	0.24	149.5	17,700
CCI-LW5	28	248	90	366	6,860	5.46	0.22	151.4	17,200
CCI-LW6	28	248	90	366	7,300	5.60	0.22	149.5	17,400
Average					6,970	5.64	0.23	150.1	17,400
CCI-LW4C	56[1]	310	0	366	8,750	5.81	0.22	148.3	17,800
CCI-LW5C	56	310	0	366	8,820	6.17	0.25	149.5	18,200
CCI-LW6C	56	310	0	366	9,270	6.23	0.24	150.8	18,200
Average					8,950	6.07	0.24	149.5	18,100

Table III-39--Continued

COMPRESSIVE STRENGTH AND ELASTIC PROPERTIES
6- by 12-inch Concrete Cylinders
225° F System
Crushed Limestone Aggregate Concrete
Corrosion of Concrete Investigation for OSW

Specimen No.	Fog cure (days)	Dry cure (days)	In brine (days)	Total age (days)	Compressive strength (psi)	E (million psi)	r	Unit weight pounds per cu foot	Pulse velocity foot per second
CCI-LW7	28	267	180		7,360	5.51	0.20	149.5	17,000
CCI-LW8	28	267	180		7,500	5.18	0.23	152.0	17,200
CCI-LW9	28	267	180		8,030	5.44	0.25	150.1	17,200
Average					7,630	5.38	0.23	150.5	17,100
CCI-LW7C	56[1]	419	0		8,840	5.36	0.20	147.6	16,600
CCI-LW8C	56	419	0		9,140	6.10	0.25	150.1	16,600
CCI-LW9C	56	419	0		9,460	6.04	0.24	150.1	16,800
Average					9,150	5.83	0.23	149.2	16,700

[1] During initial 28 days testing in brine, these companion cylinders were fog cured.

Table III-40

FREQUENCY SQUARED OF NATURAL AGGREGATE CONCRETE
4- by 4- by 30-inch Bars
225° F System
Corrosion of Concrete Investigation for OSW

Days1/ drying	Days brine	Total age	5/Frequency Squared Times 10^{-5}					
			Specimens 1 & 2	Specimens 1c & 2c	Specimens 6/3 & 4	Specimens 6/3c & 4c	Specimens 7/7 & 8	Specimens 7/7c & 8c
0	0	28	4/4.60	4/4.55	4.56	4.73	3.72	3.78
7	0	35	4.49	4.62	4.38	4.62	3.74	3.82
14	0	42	4.46	4.56	4.38	4.60	3.72	3.78
21	0	49	4.46	4.70	4.42	4.60	3.81	3.84
28	0	56	4.46	4.62	4.42	4.62	3.75	3.91
34	0	63	4.46	4.62	4.42	4.65	3.75	3.94
42	0	70	4.46	4.60	4.42	4.65	3.84	3.91
59	0	87	4.42	4.60	4.33	4.60	3.84	3.87
90	0	118	4.38	4.42	4.30	4.62	3.78	3.87
150	0	178	4.55	4.55	4.16	4.53	3.84	3.84
214	0	242	4.51	4.52	4.16	4.38	3.74	3.87
2/252	3/28	280	4.90	4.79	4.76	4.90	4.10	4.03
339	90	367	5.04	5.04	5.01	5.11	4.22	4.16
450	180	478	5.07	5.04	5.01	5.11	4.25	4.10

1/All specimens received an initial 28-day fog cure.
2/Days drying from hereon applies to Specimens 1c and 2c; 3c and 4c; and 7c and 8c.
3/Days brine only applies to Specimens 1 and 2; 3 and 4; and 7 and 8.
4/These values are F_o as referred to in Figure III-41.
5/Units for frequency (F) are cycles per second.
6/Reinforced concrete specimens.
7/Pretensioned concrete specimens.

Table III-41

FREQUENCY SQUARED OF CRUSHED LIMESTONE AGGREGATE CONCRETE

4- by 4- by 30-inch Bars
225° F System
Corrosion of Concrete Investigation for OSW

			5/Frequency Squared Times 10-5					
Days1/ drying	Days brine	Total age	Specimens 1 & 2	Specimens 1c & 2c	Specimens 6/3 & 4	Specimens 6/3c & 4c	Specimens 7/7 & 8	Specimens 7/7c & 8c
0	0	28	4/5.04	4/5.11	4.87	5.11	3.94	3.97
7	0	34	5.11	5.18	5.07	5.16	3.97	3.94
14	0	42	5.15	5.18	5.07	5.18	3.97	3.99
21	0	49	5.15	5.18	5.04	5.11	3.99	3.99
28	0	56	5.07	5.18	5.07	5.16	3.97	3.99
35	0	63	5.11	5.18	5.07	5.18	4.03	3.99
42	0	70	5.07	5.18	4.90	5.18	4.03	3.99
59	0	87	5.04	5.18	5.07	5.16	3.97	3.99
90	0	118	5.01	5.15	5.07	5.07	3.97	3.94
150	0	178	5.01	5.11	5.04	5.07	3.97	3.94
214	0	242	4.93	5.04	4.97	4.97	3.90	3.96
2/252	3/28	280	4.87	5.33	4.65	5.30	3.97	4.16
339	90	367	4.93	5.21	4.97	5.26	4.10	4.16
450	180	478	7.79	5.30	4.76	5.35	4.07	4.12

1/All specimens received an initial 28-day fog cure.

2/Days drying from hereon applies to Specimens 1c and 2c; 3c and 4c; and 7c and 8c.

3/Days brine only applies to Specimens 1 and 2; 3 and 4; 7 and 8.

4/These values are F_o are referred to in Figure III-42.

5/Units for frequency (F) are cycles per second.

6/Reinforced concrete specimens.

7/Pretensioned concrete specimens.

Table III-42

LENGTH AND WEIGHT CHANGE OF NATURAL AGGREGATE CONCRETE
225° F Temperature Brine System
4- by 4- by 30-inch Bars
Corrosion of Concrete Investigation for OSW

Total age (days)		35	42	56	87	118	178	242	280	367	478			
Cure (days)	Fog	28	28	28	28	28	28	28	28	28	28	28	28	28
	Dry	7	14	28	59	90	150	214	224	249	270			
		(7)	(14)	(28)	(59)	(90)	(150)	(214)	(252)	(339)	(450)			
	Brine	0	0	0	0	0	0	0	28	90	180			
		(0)	(0)	(0)	(0)	(0)	(0)	(0)	(0)	(0)	(0)			
Specimen Number	Type[1]													
		Length Change in Millionths Inch Per Inch												
1 & 2	Pl	204	275	351	434	514	557	597	183.	182	155			
1c & 2c	(Pl)[2]	197	254	333	442	507	568	597	259[3]	457	517			
3 & 4	Re	175	225	269	334	371	407	426	5	+3	+41			
3c & 4c	(Re)	150	215	251	302	350	382	411	139[3]	275	317			
7 & 8	Prs	203	299	393	494	560	628	678	654	717	740			
7c & 8c	(Prs)	199	274	364	469	545	610	650	444[3]	592	683			
		Weight Change in Pounds												
1 & 2	Pl	0.32	0.42	0.54	0.62	0.72	0.76	0.82	0.01	+0.09[4]	+0.28			
1c & 2c	(Pl)	0.33	0.36	0.48	0.61	0.69	0.76	0.82	+0.12[3]	0.40	0.49			
3 & 4	Re	0.35	0.44	0.51	0.61	0.69	0.74	0.79	+0.07	+0.15	+0.33			
3c & 4c	(Re)	0.30	0.39	0.46	0.53	0.62	0.68	0.74	+0.14[3]	0.25	0.36			
7 & 8	Prs	0.23	0.30	0.37	0.47	0.52	0.52	0.64	+0.11	+0.22	+0.34			
7c & 8c	(Prs)	0.19	0.26	0.33	0.42	0.49	0.55	0.59	+0.12[3]	0.24	0.37			

[1] Pl = Plain; Re = Reinforced; Prs = Prestressed

[2] All ( ) refer to companion specimens which were not placed in brine, but left in laboratory air.

[3] Placed in fog May 21, 1966, but returned to 50 percent room on June 29, 1966.

[4] All specimen readings indicate shrinkage compared to the 28-day fog readings except those marked with a "+" which indicates expansion.

Table III-43

LENGTH AND WEIGHT CHANGE OF CRUSHED LIMESTONE AGGREGATE CONCRETE
225° F Temperature Brine System
4- by 4- by 30-inch Bars
Corrosion of Concrete Investigation for OSW

Total age (days)		35	42	56	87	118	178	242	280	367	478			
	Fog	28	28	28	28	28	28	28	28	28	28	28	28	28
Cure	Dry	7	14	28	59	90	150	214	224	249	270			
		(7)	(14)	(28)	(59)	(90)	(150)	(214)	(252)	(339)	(450)			
(days)	Brine	0	0	0	0	0	0	0	28	90	180			
		(0)	(0)	(0)	(0)	(0)	(0)	(0)	(0)	(0)	(0)			
Specimen Number	Type[1]													
		Length Change in Millionths Inch Per Inch												
1 & 2	Pl	107	142	190	229	266	297	315	+151[4]	+221	+343			
1c & 2c	(Pl)[2]	96	146	183	218	266	294	315	120[3]	262	279			
3 & 4	Re	92	135	169	198	234	265	282	+227	+297	+408			
3c & 4c	(Re)	103	139	169	204	234	251	272	99[3]	205	225			
7 & 8	Prs	130	202	257	331	375	421	460	575	638	691			
7c & 8c	(Prs)	130	202	254	324	365	429	460	298[3]	422	467			
		Weight Change in Pounds												
1 & 2	Pl	0.26	0.32	0.42	0.47	0.53	0.56	0.60	+0.10	+0.12	+0.24			
1c & 2c	(Pl)	0.26	0.32	0.38	0.46	0.52	0.56	0.59	+0.10	0.36	0.41			
3 & 4	Re	0.18	0.27	0.35	0.41	0.49	0.49	0.53	+0.10	+0.14	+0.33			
3c & 4c	(Re)	0.29	0.35	0.41	0.50	0.56	0.60	0.64	+0.06	0.32	0.38			
7 & 8	Prs	0.14	0.20	0.26	0.30	0.36	0.43	0.47	+0.20	+0.25	+0.34			
7c & 8c	(Prs)	0.22	0.30	0.36	0.40	0.46	0.51	0.54	+0.14[3]	0.24	0.38			

[1] Pl = Plain; Re = Reinforced; Prs = Prestressed
[2] All ( ) refer to companion specimens which were not placed in brine, but left in laboratory air.
[3] Placed in fog May 21, 1966, but returned to 50 percent room on June 29, 1966.
[4] All specimen readings indicate shrinkage compared to the 28-day fog readings except those marked with a plus which indicates expansion.

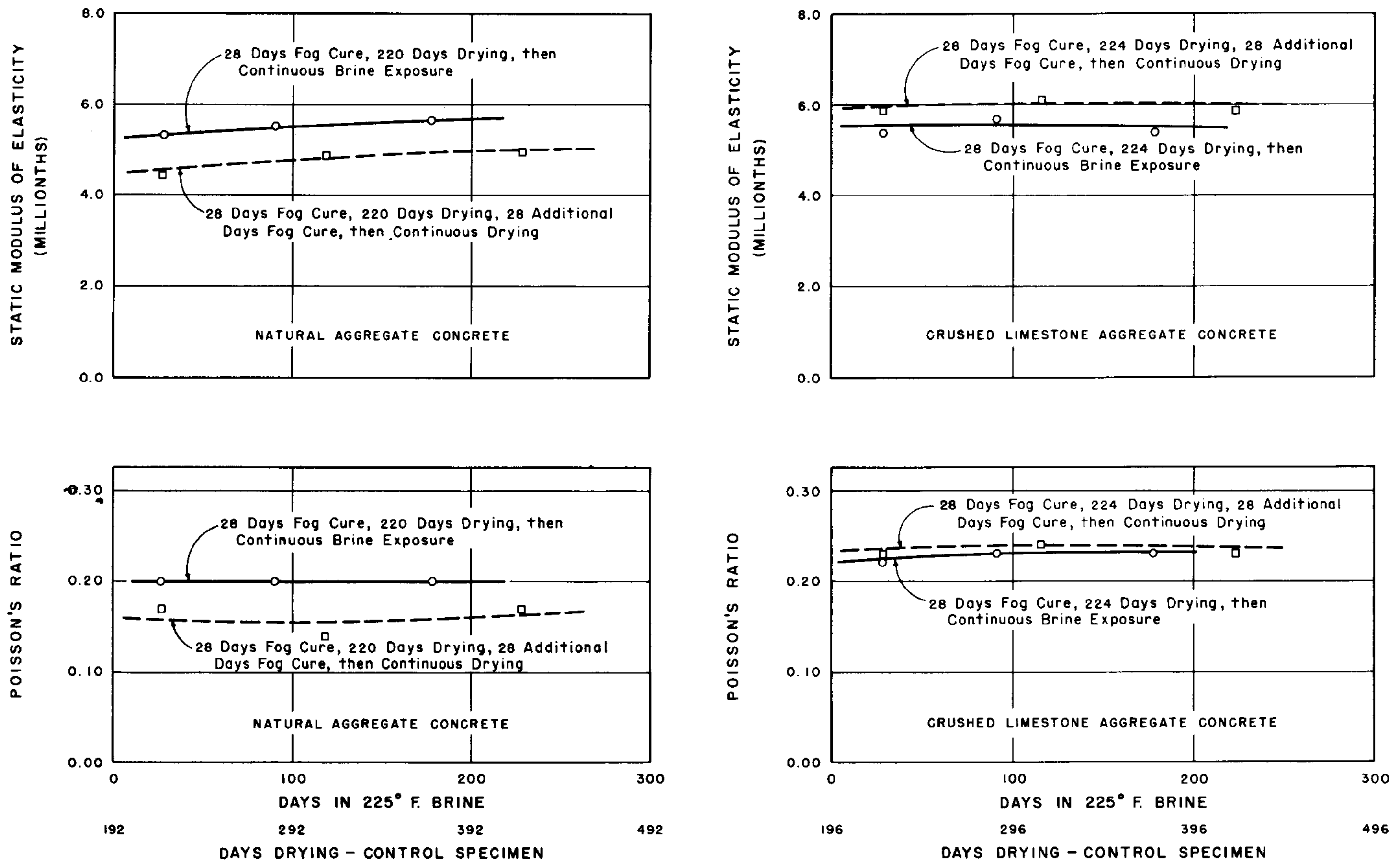

FIGURE III - 49: STATIC MODULI OF ELASTICITY AND POISSON'S RATIO OF NATURAL AGGREGATE CONCRETE AND CRUSHED LIMESTONE AGGREGATE CONCRETE SUBJECTED TO 225° F SYNTHETIC SEA WATER BRINE, AND CONTINUOUSLY DRIED CONTROL CONCRETE. EACH PLOT REPRESENTS THE AVERAGE OF THREE DIFFERENT SPECIMENS

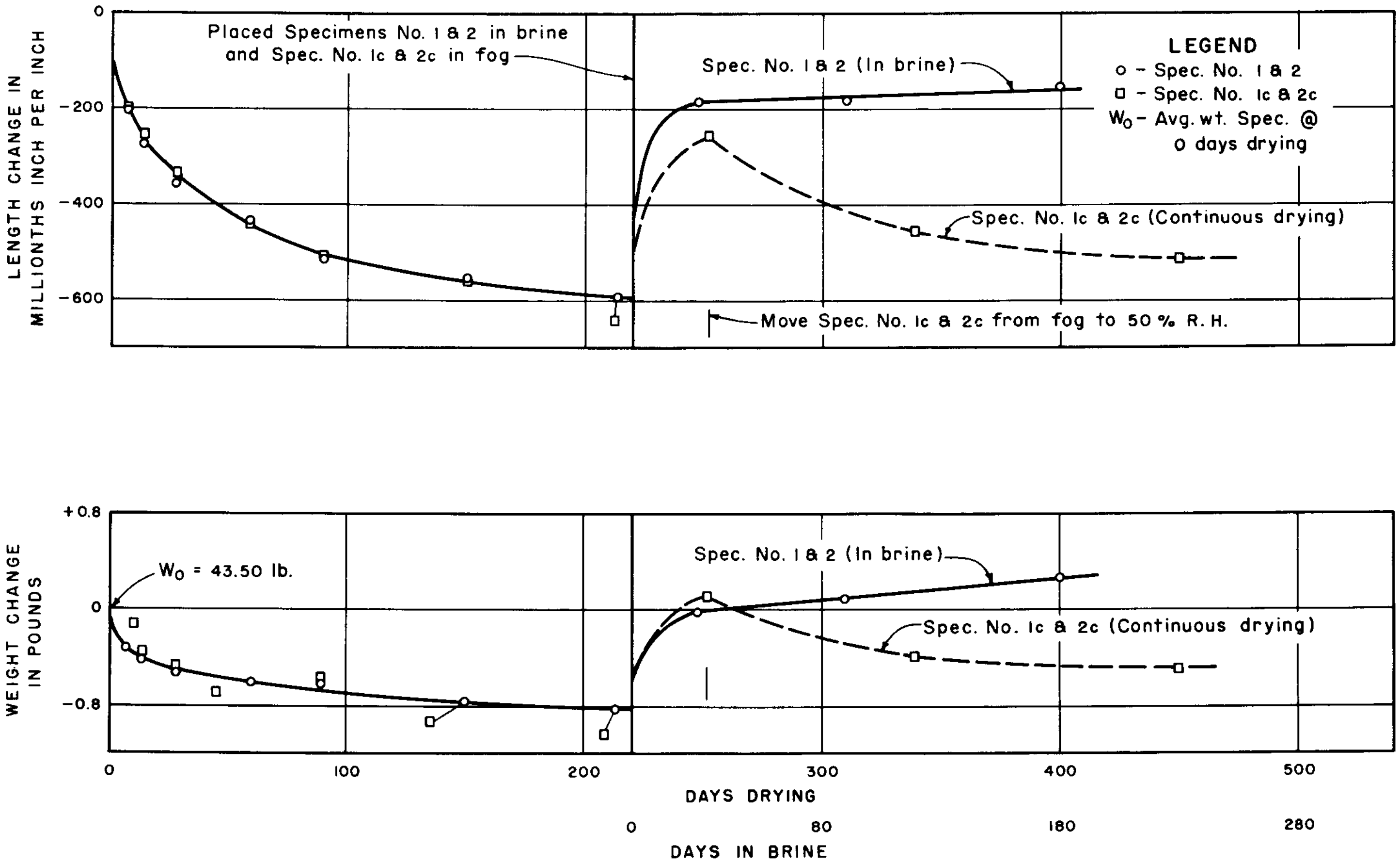

FIGURE III – 50 : LENGTH AND WEIGHT CHANGE OF PLAIN NATURAL AGGREGATE CONCRETE SUBJECTED TO 225° F. BRINE AND CONTINUOUS DRYING

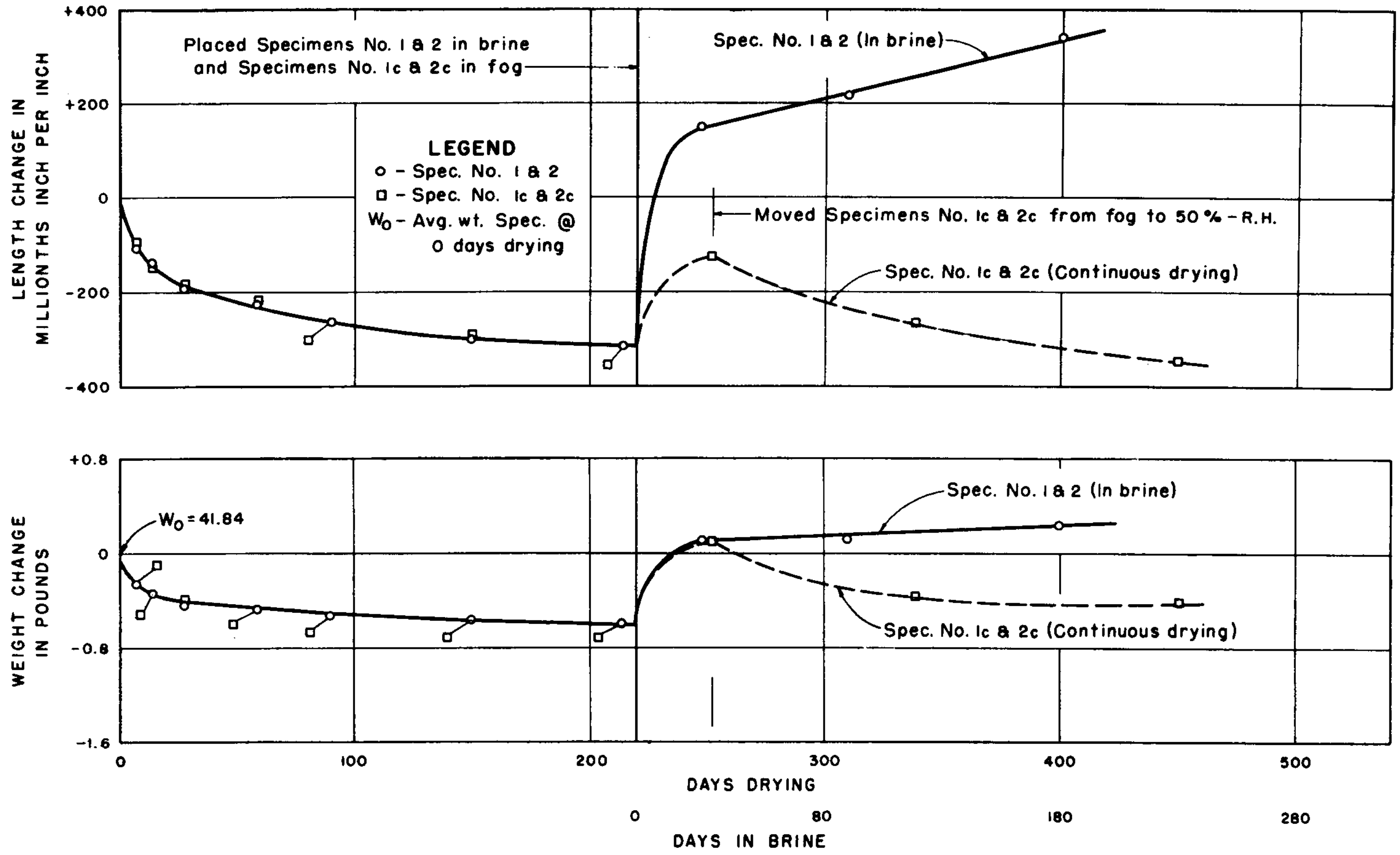

FIGURE III - 51 : LENGTH AND WEIGHT CHANGE OF PLAIN CRUSHED LIMESTONE AGGREGATE CONCRETE SUBJECTED TO 225° F. BRINE AND CONTINUOUS DRYING than it was originally. As in the case of the natural aggregate concrete, moderate expansion is progressing with continued brine exposure.

Weights of the two concretes under test are increasing normally and are consistent with the expansion.

Length and weight changes of reinforced natural and crushed limestone aggregate concretes responded very much as that of the plain concrete, Figures III-52 and III-53. The reinforced natural aggregate concrete underwent 171 millionths inch per inch less total shrinkage upon drying than comparable plain concrete and then upon exposure to the brine it expanded 7 millionths more than the plain concrete. Now after 180 days' exposure this reinforced concrete occupies slightly more volume than at any time before. The same relative length changes occurred in the reinforced crushed limestone aggregate concrete except that after 180 days' exposure it now occupies a substantial 400 millionths inch per inch greater volume than ever before.

Length and weight changes of pretensioned natural and crushed limestone aggregate concretes responded similarly to comparable concrete undergoing 250° F brine exposure although the magnitude was somewhat less, Figures III-54 and III-55.

Readings taken on strain gages mounted on the pretensioning rods were very erratic but did indicate that considerably more pretensioning stress was in the crushed limestone aggregate concrete the day the brine tests began than was in the natural aggregate concrete. This may account for the differences in net length change between the two concretes. In no case are there any signs that would indicate deleterious expansion causing deterioration.

III. 6. 2. 2. 5. 203° F brine tests. - -

The general condition of the concrete subjected for 90 days to the 203° F synthetic sea water brine can be considered excellent. No evidence of deterioration exists.

Compressive strength of the natural aggregate concrete subjected to the 203° F synthetic sea water brine averaged 6, 620, 7, 170, and 8, 270 psi for exposures of 28, 85, and 180 days, respectively, Table III-44 and Figure III-19. The strength development to 85 days is somewhat less than that of the other brine environments; however, at 180 days' exposure the strength is very nearly the same.

The compressive strength of the crushed limestone aggregate concrete subjected to the 203° F synthetic sea water brine averaged 6, 680, 6, 910, and 7, 190 psi for exposures of 28, 85, and 180 days, respectively, Table III-45 and Figure III-37. This 180-day strength is slightly less than that of the 225° F brine-exposed concrete and slightly greater than that of the 250° F brine-exposed concrete.

Modulus of elasticity of the natural and crushed limestone aggregate concretes subjected to the 203° F synthetic sea water brine for 180 days averaged 5. 74 and 5. 86 million psi, respectively, which in both cases is an increase over similar values obtained after 28 days of exposure, Tables III-44 and III-45 and Figure III-56. Here again, these values are well above the 4. 50 million psi considered average for good concrete, and certainly indicates that the concrete remains sound and of high quality.

Poisson's ratio of the natural and crushed limestone aggregate concrete exposed to the 203° F synthetic sea water brine did not change significantly throughout the test period, Tables III-44 and III-45 and Figure III-56. The values of 0. 20 and 0. 24, respectively, are considered about average for good quality concrete.

Unit weights of the natural and crushed limestone aggregate concretes subjected to the 203° F synthetic sea water brine averaged 158. 6 and 150. 3 pounds per cubic foot at 180 days' brine exposure, Tables III-44 and III-45. A slight increase was noted in the unit weight of the natural aggregate concrete between 28 and 90 days' exposure; whereas, for the same exposure the crushed limestone aggregate

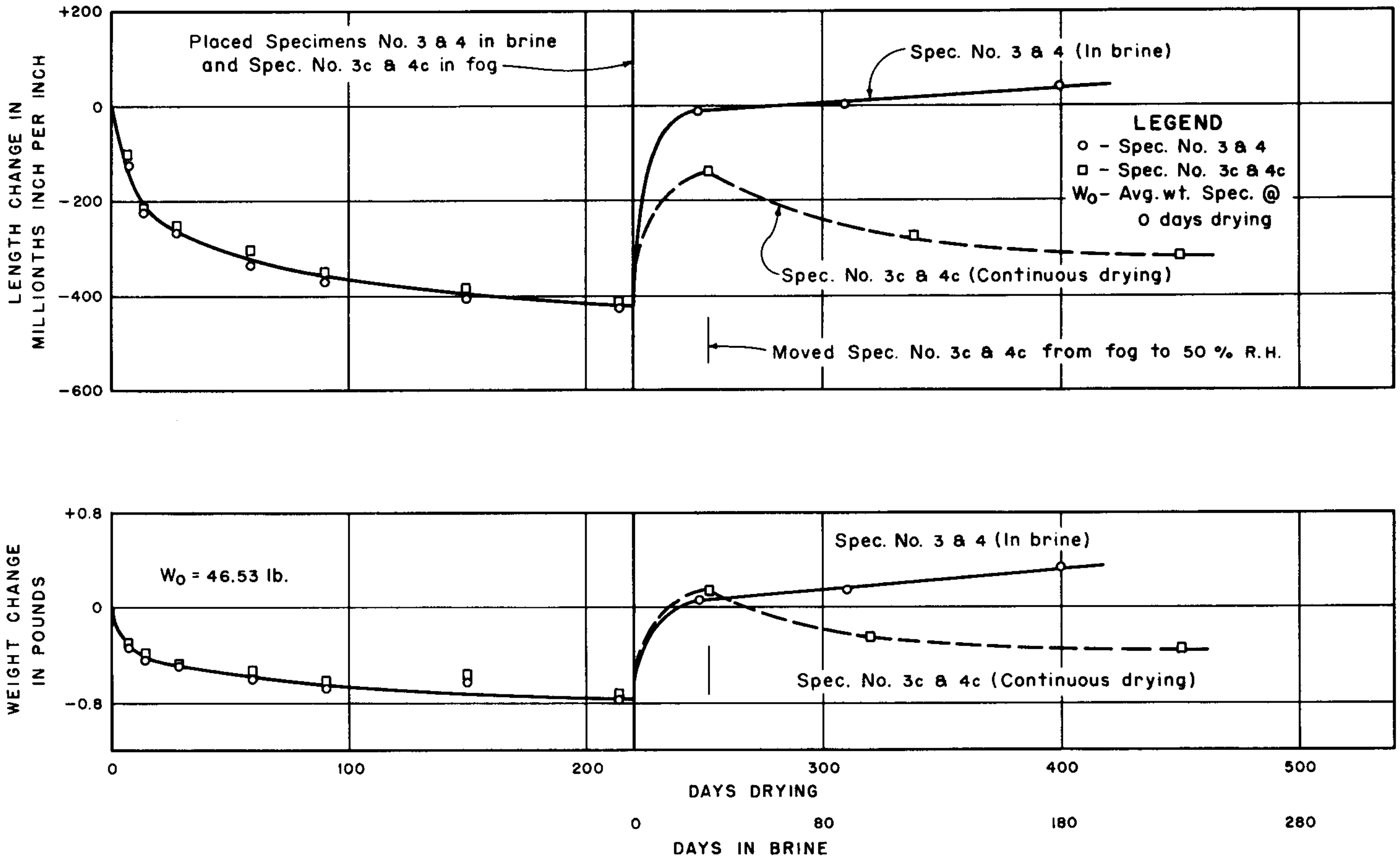

FIGURE III – 52 : LENGTH AND WEIGHT CHANGE OF REINFORCED NATURAL AGGREGATE CONCRETE SUBJECTED TO 225° F. BRINE AND CONTINUOUS DRYING

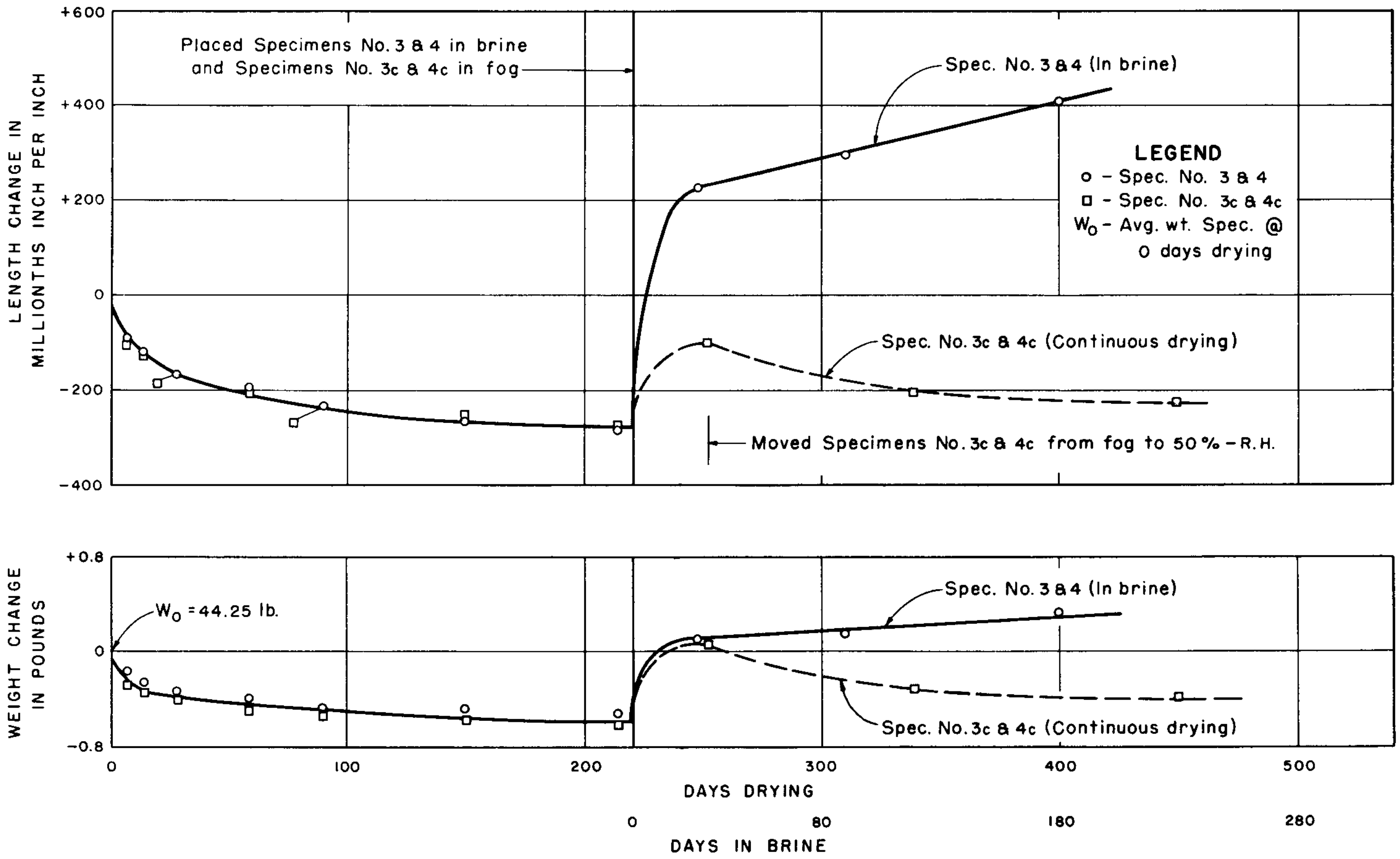

FIGURE III – 53: LENGTH AND WEIGHT CHANGE OF REINFORCED CRUSHED LIMESTONE AGGREGATE CONCRETE SUBJECTED TO 225° F. BRINE AND CONTINUOUS DRYING

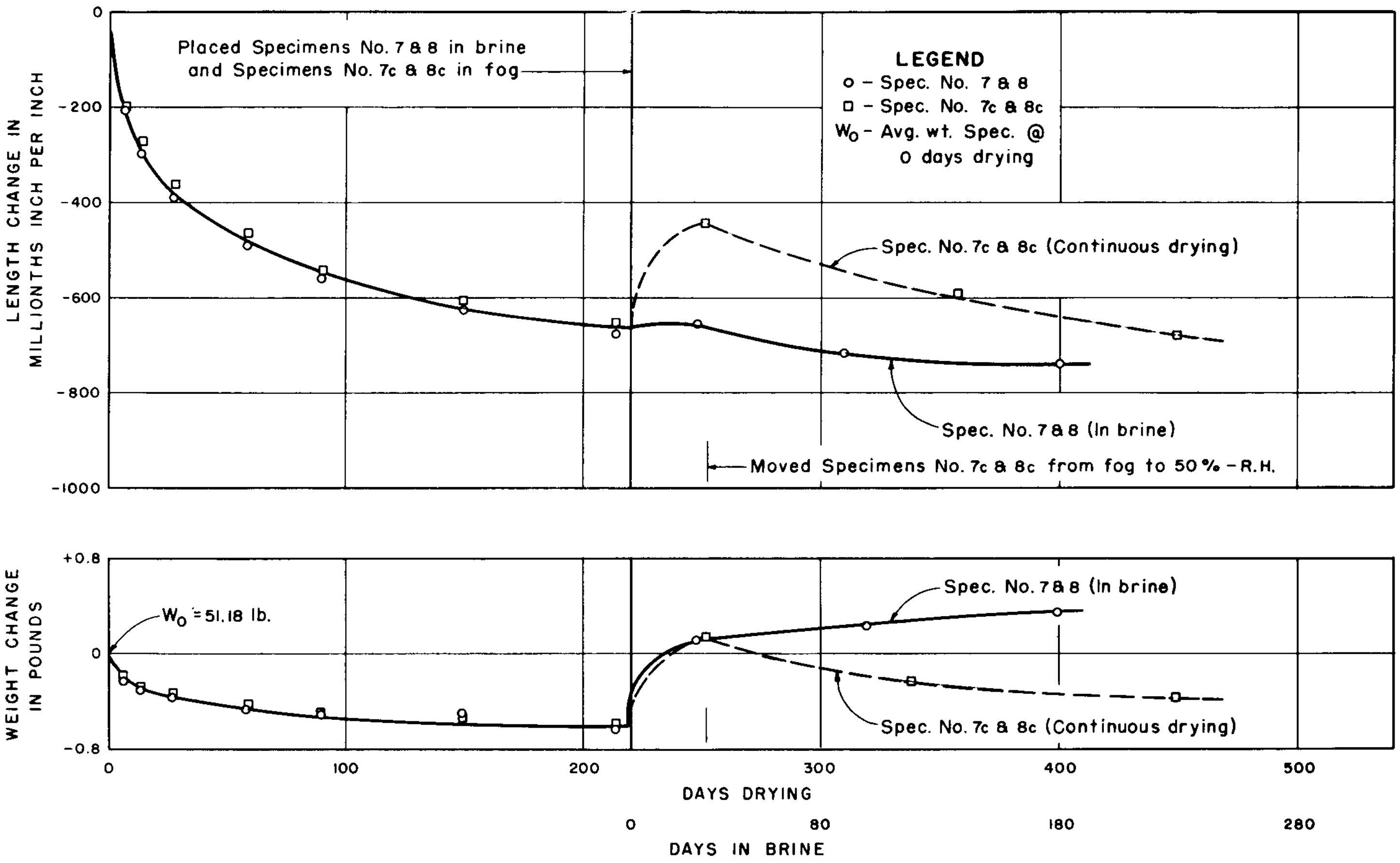

FIGURE III - 54: LENGTH AND WEIGHT CHANGE OF PRESTRESSED NATURAL AGGREGATE CONCRETE SUBJECTED TO 225° F. BRINE AND CONTINUOUS DRYING

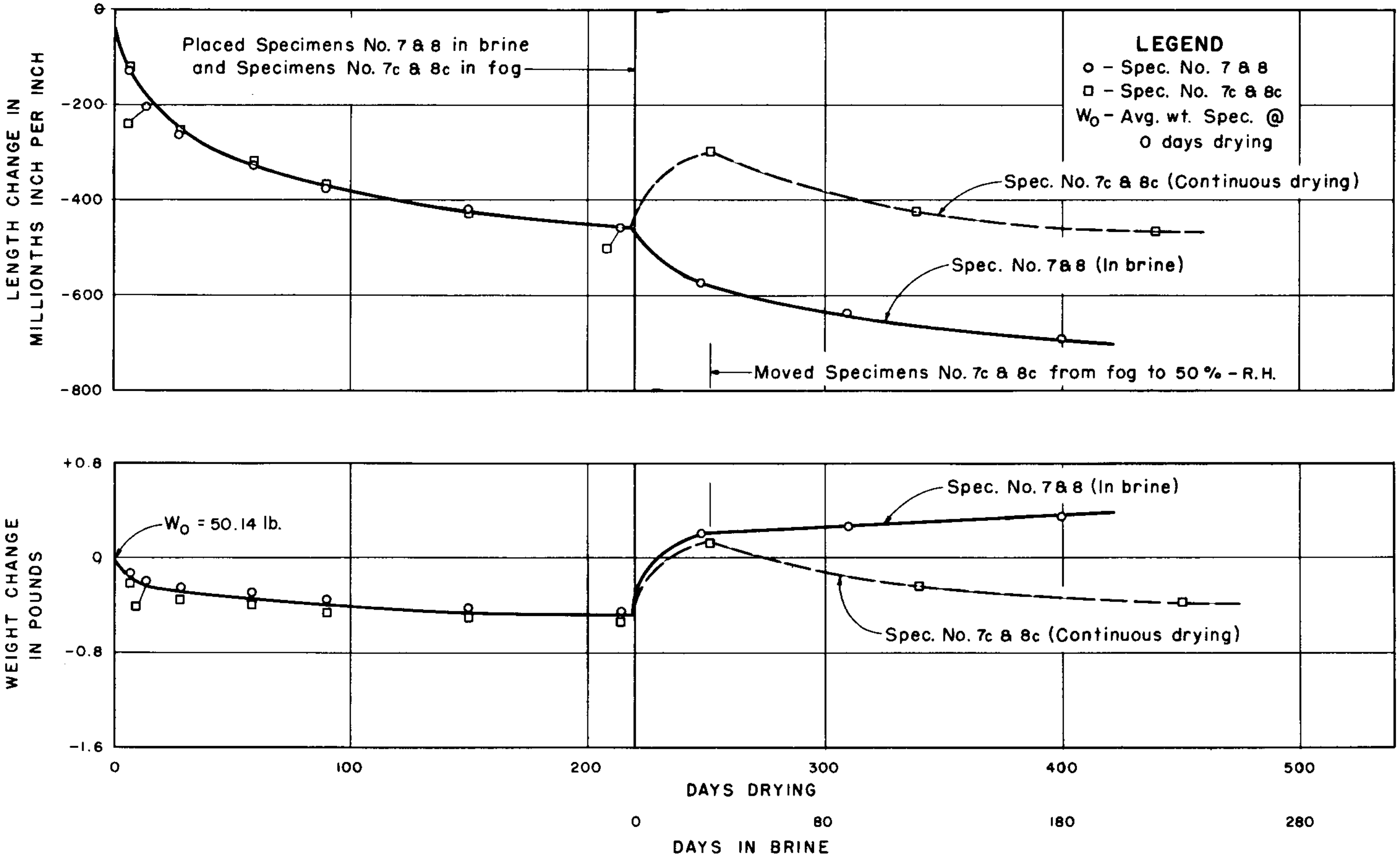

FIGURE III - 55 : LENGTH AND WEIGHT CHANGE OF PRESTRESSED CRUSHED LIMESTONE AGGREGATE CONCRETE SUBJECTED TO 225° F. BRINE AND CONTINUOUS DRYING

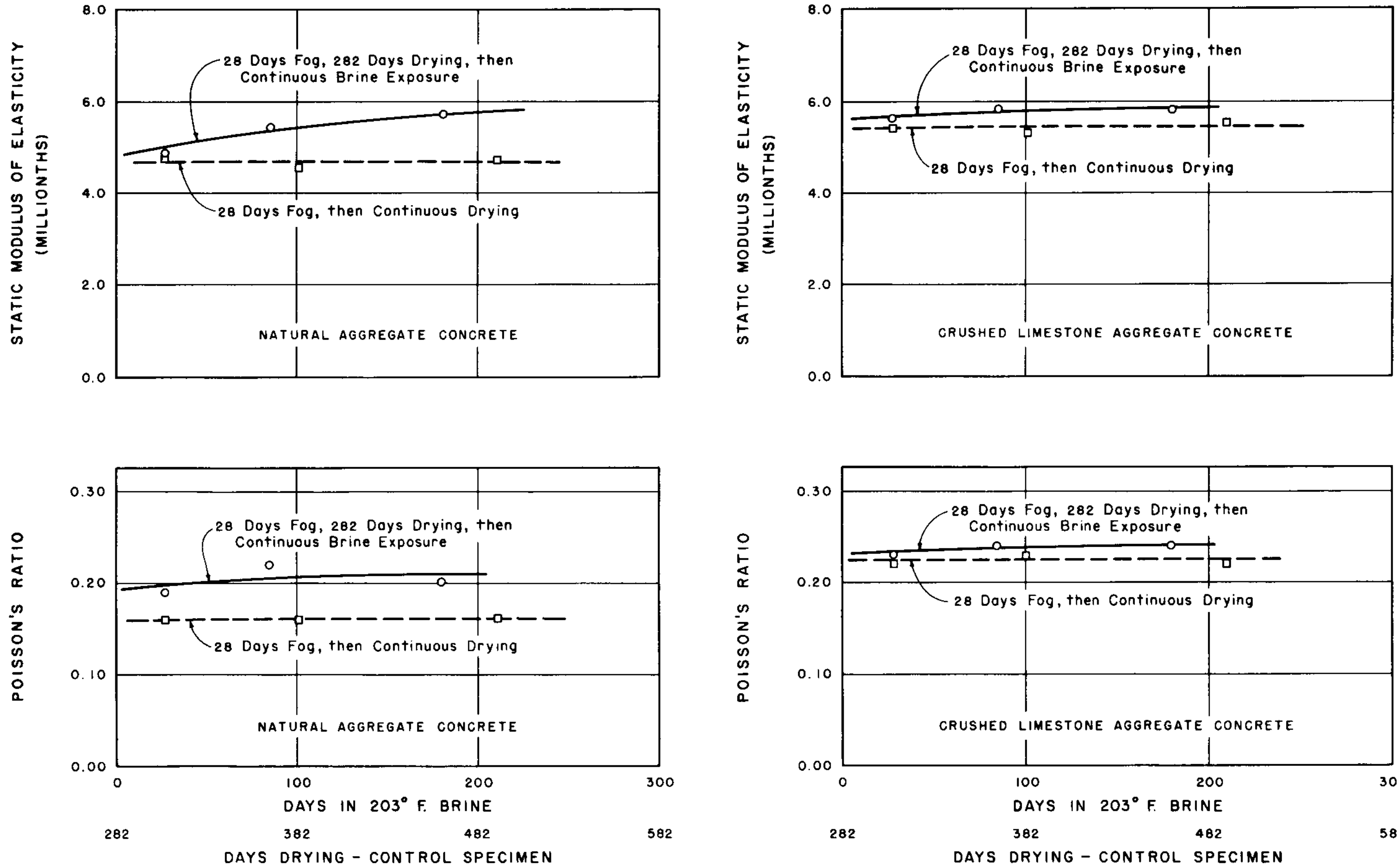

FIGURE III – 56: STATIC MODULI OF ELASTICITY AND POISSON'S RATIO OF NATURAL AGGREGATE CONCRETE AND CRUSHED LIMESTONE AGGREGATE CONCRETE SUBJECTED TO 203° F. SYNTHETIC SEA WATER BRINE, AND CONTINUOUSLY DRIED CONTROL CONCRETE. EACH PLOT REPRESENTS THE AVERAGE OF THREE DIFFERENT SPECIMENS

Table III-44

COMPRESSIVE STRENGTH AND ELASTIC PROPERTIES
6- by 12-inch Concrete Cylinders
203° F System
Natural Aggregate Concrete
Corrosion of Concrete Investigation for OSW

Specimen No.	Fog cure (days)	Dry cure (days)	In brine (days)	Total age (days)	Compressive strength (psi)	E (million psi)	r	Unit weight pounds per cu foot	Pulse velocity foot per second
CCI-K35	7	0	0	7	3,610	3.62	0.19	-	-
CCI-K36	7	0	0	7	3,930	3.35	0.18	-	-
Average					3,770	3.48	0.18		
CCI-K37	28	0	0	28	6,030	4.38	0.18	-	-
CCI-K38	28	0	0	28	6,230	4.43	0.21	-	-
Average					6,130	4.40	0.20		
CCI-K1	28	282	28	338	6,680	4.66	0.20	156.4	15,800
CCI-K2	28	282	28	338	6,580	4.83	0.18	157.0	15,900
CCI-K3	28	282	28	338	6,610	5.12	0.18	158.2	16,100
Average					6,620	4.87	0.19	157.2	15,900
CCI-K1C	28	310	0	338	7,220	4.60	0.16	153.2	14,900
CCI-K2C	28	310	0	338	7,140	4.80	0.15	156.4	15,300
CCI-K3C	28	310	0	338	7,850	4.86	0.17	155.8	15,600
Average					7,400	4.75	0.16	155.1	15,300
CCI-K4	28	298	85	411	7,130	5.16	0.21	156.4	16,900
CCI-K5	28	298	85	411	7,250	5.65	0.22	158.9	16,500
CCI-K6	28	298	85	411	7,140	5.42	0.22	158.9	16,900
Average					7,170	5.41	0.22	158.1	16,800
CCI-K4C	28	383	0	411	7,390	4.35	0.16	155.8	15,900
CCI-K5C	28	383	0	411	7,530	4.59	0.17	157.0	16,200
CCI-K6C	28	383	0	411	7,730	4.79	0.16	157.0	15,700
Average					7,550	4.58	0.16	156.6	15,900

Table III-44--Continued

COMPRESSIVE STRENGTH AND ELASTIC PROPERTIES
6- by 12-inch Concrete Cylinders
203° F Low-temperature System
Corrosion of Concrete Investigation for OSW

Specimen No.	Fog cure (days)	Dry cure (days)	In brine (days)	Total age (days)	Compressive strength (psi)	E (million psi)	r	Unit weight pounds per cu foot	Pulse velocity foot per second
CCI-K7	28	311	180	519	8,100	5.44	-	158.2	16,200
CCI-K8	28	311	180	519	8,030	5.85	0.21	158.9	16,200
CCI-K9	28	311	180	519	8,680	5.94	0.20	158.9	15,700
Average					8,270	5.74	0.20	158.6	16,000
CCI-K7C	28	491	0	519	7,180	4.56	0.18	154.5	14,900
CCI-K8C	28	491	0	519	7,320	4.73	0.15	155.8	15,100
CCI-K9C	28	491	0	519	7,840	4.84	0.16	155.8	15,200
Average					7,450	4.71	0.16	155.4	15,100

Table III-45

COMPRESSIVE STRENGTH AND ELASTIC PROPERTIES
6- by 12-inch Concrete Cylinders
203° F System
Crushed Limestone Aggregate Concrete
Corrosion of Concrete Investigation for OSW

Specimen No.	Fog cure (days)	Dry cure (days)	In brine (days)	Total age (days)	Compressive strength (psi)	E (million psi)	r	Unit weight pounds per cu foot	Pulse velocity foot per second
CCI-LK35	7	0	0	7	4,350	5.14	0.22	-	-
CCI-LK36	7	0	0	7	4,440	5.05	0.21	-	-
Average					4,400	5.10	0.22		
CCI-LK37	28	0	0	28	5,990	5.71	0.22	-	-
CCI-LK38	28	0	0	28	6,230	5.67	0.26	-	-
Average					6,110	5.69	0.24		
CCI-LK1	28	282	28	338	6,930	5.67	0.24	153.2	17,500
CCI-LK2	28	282	28	338	6,510	5.71	0.23	152.6	16,900
CCI-LK3	28	282	28	338	6,690	5.62	0.23	152.6	17,200
Average					6,680	5.67	0.23	152.8	17,200
CCI-LK1C	28	310	0	338	7,900	5.45	0.22	148.3	17,400
CCI-LK2C	28	310	0	338	7,520	4.85	0.21	148.3	17,400
CCI-LK3C	28	310	0	338	7,380	6.00	0.23	148.9	17,200
Average					7,600	5.43	0.22	148.5	17,300
CCI-LK4	28	298	85	411	6,900	6.34	0.24	150.8	17,800
CCI-LK5	28	298	85	411	6,760	5.77	0.24	149.5	18,300
CCI-LK6	28	298	85	411	7,070	5.56	0.25	150.1	18,200
Average					6,910	5.89	0.24	150.1	18,100
CCI-LK4C	28	383	0	411	7,840	5.46	0.23	148.9	18,200
CCI-LK5C	28	383	0	411	7,590	5.12	0.24	148.9	17,700
CCI-LK6C	28	383	0	411	8,080	5.37	0.23	148.3	17,700
Average					7,840	5.32	0.23	148.7	17,900

Table III-45--Continued

COMPRESSIVE STRENGTH AND ELASTIC PROPERTIES
6- by 12-inch Concrete Cylinders
230° F System
Crushed Limestone Aggregate Concrete
Corrosion of Concrete Investigation for OSW

Specimen No.	Fog cure (days)	Dry cure (days)	In brine (days)	Total age (days)	Compressive strength (psi)	E (million psi)	r	Unit weight pounds per cu foot	Pulse velocity foot per second
CCI-LK7	28	311	180	519	7,130	5.88	-	150.8	16,500
CCI-LK8	28	311	180	519	7,060	5.48	0.23	148.8	16,800
CCI-LK9	28	311	180	519	7,385	6.22	0.25	151.4	17,200
Average					7,190	5.86	0.24	150.3	16,800
CCI-LK7C	28	491	0	519	8,140	5.62	0.25	148.8	17,500
CCI-LK8C	28	491	0	519	7,500	5.54	0.18	150.1	17,700
CCI-LK9C	28	491	0	519	7,980	5.59	0.22	149.5	17,500
Average					7,870	5.58	0.22	149.4	17,600

concrete underwent a slight decrease in unit weight, although it showed a slighter increase at 180 days. In neither case is it indicative of deterioration.

Pulse velocity of the natural and crushed limestone aggregate concretes subjected to the 203° F synthetic sea water brine increased substantially between 28 and 90 days' exposure, but underwent a drop between 90 and 180 days' exposure, Tables III-44 and III-45 and Figures III-39 and III-40. In all cases the pulse velocity averages well above the 15,000 feet per second considered minimum for very good quality concrete.

Resonant frequency of the natural aggregate continued to increase between 28 and 180 days' exposure to the 203° F synthetic sea water brine, whereas that of the crushed limestone aggregate concrete increased to 90 days but then dropped slightly at 180 days' exposure; all values still indicate a very-high-quality concrete exists, Tables III-46 and III-47 and Figures III-41 and III-42.

Data obtained from tests conducted upon the plain natural aggregate concrete bars subjected to 203° F synthetic sea water brine showed that the resultant length changes are very similar to those that have occurred in 225° F brine-exposed concrete, both as to total drying shrinkage and as to expansion due to moisture, Table III-48 and Figure III-57. The 301 millionths inch per inch drying shrinkage of the crushed limestone aggregate concrete corresponds very closely with that of the 225° F system, but expansion resulting from 28 days' exposure to the 203° F brine totaled only 274 millionths inch per inch, whereas that of the 225° F system totaled 466 millionths inch per inch, Table III-49 and Figure III-58. A comparison of the expansion with the weight change indicates that the absorption of moisture is also substantially less than that of the 225° F concrete.

Weight increases of the two concretes under test in 203° F brine are comparable to the increase in expansion and appear normal.

Little need be said regarding the length and weight changes of reinforced natural and crushed limestone aggregate concretes since the drying shrinkage and the subsequent expansion due to brine exposure are both practically identical to that of the plain concrete discussed in the previous paragraph, Tables III-48 and III-49 and Figures III-59 and III-60.

Length and weight changes of prestressed natural and crushed limestone aggregate concretes exposed to the 203° F brine responded very much like the corresponding concretes subjected to the 225° F brine, Figures III-61 and III-62. Certainly, nothing appears unusual and no signs indicating deterioration exist.

III. 6. 3. Miscellaneous Tests

III. 6. 3. 1. Cavitation tests. --

Cavitation tests were conducted throughout the entire 2 years of 100° F brine exposure and 18 months of 290° F brine exposure, and visual observations of the orifices were made at periodic intervals. No cavitation damage was apparent in either of the two orifices, the 100° or the 290° F after 2 years and 1 year, respectively, of brine exposure, Figures III-63, III-64, III-65, and III-66. The only visible signs of deterioration are the spalling and some minor erosion of the softened invert surface in the high-temperature orifice. No such deterioration was noted in the orifice tested in the low-temperature loop.

III. 6. 3. 2. Reinforced concrete pipe tests. --

Another test conducted in the high-temperature (290° F) brine system was the exposure, under static conditions, of a 2-foot-long section of reinforced concrete pipe to the environment. This pipe section, which was 15 inches outside diameter, 7 inches inside diameter, and cast from the same concrete as the test specimens was fitted into the side of the brine test system, Figure III-67. This configuration allowed a differential temperature, pressure, humidity situation through the concrete, i.e., on the inside of the pipe the environment consisted of 290° F synthetic

Table III-46

FREQUENCY SQUARED OF NATURAL AGGREGATE CONCRETE
4- by 4- by 30-inch Bars
203° F System
Corrosion of Concrete Investigation for OSW

Days1/ drying	Days brine	Total age	5/Frequency Squared Times 10^{-5}					
			Specimens 1 & 2	Specimens 1c & 2c	Specimens 6/3 & 4	Specimens 6/3c & 4c	Specimens 7/7 & 8	Specimens 7/7c & 8c
0	0	28	4/4.76	4/4.83	4.76	4.73	3.78	3.82
7	0	35	4.69	4.79	4.76	4.76	3.78	3.91
14	0	42	4.79	4.79	4.76	4.69	3.84	3.94
21	0	49	4.69	4.79	4.76	4.73	3.84	3.97
28	0	56	4.73	4.83	4.76	4.69	3.84	3.97
35	0	63	4.73	4.87	4.76	4.73	3.84	3.97
42	0	70	4.69	4.83	4.76	4.69	3.84	3.94
58	0	86	4.73	4.79	4.83	4.73	3.84	3.97
90	0	118	4.69	4.79	4.79	4.73	3.82	3.97
150	0	178	4.69	4.76	4.65	4.56*	3.78	3.97
267	0	295	4.65	4.76	4.55	4.56	3.81	3.97
2/311	3/28	339	5.01	4.76	4.83	4.42	4.12	3.91
378	85	406	5.11	4.69	5.04	4.38	4.22	3.91
490	180	518	5.30	4.69	5.36	4.38	4.33	3.91

*New tubes scope.

1/All specimens received an initial 28-day fog cure.

2/Days drying from hereon applies to Specimens 1c and 2c; 3c and 4c; and 7c and 8c.

3/Days brine only applies to Specimens 1 and 2; 3 and 4; 7 and 8.

4/These values are F_o as referred to in Figure III-41.

5/Units for frequency (F) are cycles per second.

6/Reinforced concrete specimens.

7/Pretensioned concrete specimens.

Table III-47

FREQUENCY SQUARED OF CRUSHED LIMESTONE AGGREGATE CONCRETE
4- by 4- by 30-inch Bars
203° F System
Corrosion of Concrete Investigation for OSW

			5/Frequency Squared Times 10^{-5}					
Days1/ drying	Days brine	Total age	Specimens 1 & 2	Specimens 1c & 2c	Specimens 6/3 & 4	Specimens 6/3c & 4c	Specimens 7/7 & 8	Specimens 7/7c & 8c
0	0	28	4/5.16	4/5.16	5.04	5.04	4.03	3.99
7	0	35	5.30	5.18	5.07	5.21	3.99	3.99
14	0	42	5.30	5.26	5.16	5.11	3.99	4.07
21	0	49	5.31	5.30	5.11	5.18	4.07	4.07
28	0	56	5.36	5.30	5.21	5.21	4.07	4.07
35	0	63	5.36	5.36	5.21	5.18	4.07	4.10
42	0	70	5.36	5.33	5.21	5.18	4.07	4.10
59	0	86	5.33	5.36	5.26	5.18	4.07	4.10
90	0	118	5.30	5.33	5.21	5.16	4.03	4.07
150	0	178	5.21	5.26	5.07	5.04	4.07	4.07
267	0	295	5.07	5.18	5.04	4.97	3.99	3.94
2/311	3/28	339	5.30	5.07	5.07	4.97	4.12	3.94
378	85	406	5.33	5.04	5.18	4.83	4.10	3.82
490	180	518	5.26	5.07	5.04	4.83	4.22	3.94

1/All specimens received an initial 28-day fog cure.
2/Days drying from hereon applies to Specimens 1c & 2c; 3c and 4c; 7c and 8c.
3/Days brine only applies to Specimens 1 and 2; 3 and 4; 7 and 8.
4/These values are F_o as referred to in Figure III-42.
5/Units for frequency (F) are cycles per second.
6/Reinforced concrete specimens.
7/Pretensioned concrete specimens.

Table III-48

LENGTH AND WEIGHT CHANGE OF NATURAL AGGREGATE CONCRETE
203° F Temperature Brine System
4- by 4- by 30-inch Bars
Corrosion of Concrete Investigation for OSW

Total age (days):		35	42	56	86	118	178	295	339	406	518			
Cure (days)	Fog	28	28	28	28	28	28	28	28	28	28	28	28	28
	Dry	7	14	28	58	90	150	267	283	292	310			
		(7)	(14)	(28)	(58)	(90)	(150)	(267)	(311)	(382)	(490)			
	Brine	0	0	0	0	0	0	0	28	90	180			
		(0)	(0)	(0)	(0)	(0)	(0)	(0)	(0)	(0)	(0)			
Specimen Number	Type[1]													
		Length Change in Millionths Inch Per Inch												
1 & 2	Pl	171	235	327	422	500	573	597	195	161	125			
1c & 2c	(Pl)	164	224	306	415	475	551	579	582	582	590			
3 & 4	Re	99	156	206	283	336	391	397	13	+7[3]	+25			
3c & 4c	(Re)	96	163	223	286	325	391	393	393	390	390			
7 & 8	Prs	260	342	430	554	625	716	775	685	746	778			
7c & 8c	(Prs)	278	317	409	543	621	703	758	775	779	779			
		Weight Change in Pounds												
1 & 2	Pl	0.30	0.38	0.51	0.64	0.70	0.79	0.78	0.56	0.06	+0.08			
1c & 2c	(Pl)	0.31	0.38	0.50	0.62	0.67	0.76	0.75	0.79	0.82	0.84			
3 & 4	Re	0.22	0.30	0.40	0.50	0.55	0.67	0.64	0.04	0.03	+0.39			
3c & 4c	(Re)	0.21	0.33	0.43	0.52	0.56	0.69	0.65	0.69	0.73	0.75			
7 & 8	Prs	0.27	0.34	0.44	0.53	0.58	0.68	0.65	0.03	0.04	+0.10			
7c & 8c	(Prs)	0.23	0.29	0.38	0.50	0.55	0.62	0.59	0.61	0.65	0.67			

[1] Pl = Plain; Re = Reinforced; Prs = Prestressed.
[2] All ( ) refer to companion specimens which were not placed in brine, but left in laboratory air.
[3] All specimen readings indicate shrinkage compared to the 28-day fog readings except those marked with a "+" which indicates expansion.

Table III-49

LENGTH AND WEIGHT CHANGE OF CRUSHED LIMESTONE AGGREGATE CONCRETE
203° F Temperature Brine System
4- by 4- by 30-inch Bars
Corrosion of Concrete Investigation for OSW

Total age (days):		35	42	56	86	118	178	295	339	406	518			
Cure (days)	Fog	28	28	28	28	28	28	28	28	28	28	28	28	28
	Dry	7	14	28	58	90	150	267	283	292	310			
		(7)	(14)	(28)	(58)	(90)	(150)	(267)	(311)	(382)	(490)			
	Brine	0	0	0	0	0	0	0	28	90	180			
		(0)	(0)	(0)	(0)	(0)	(0)	(0)	(0)	(0)	(0)			
Specimen Number	Type[1]	Length Change in Millionths Inch Per Inch												
1 & 2	Pl	85	111	156	207	247	286	301	28	+38[3]	+95			
1c & 2c	(Pl)[2]	71	104	152	205	251	297	295	315	303	324			
3 & 4	Re	67	95	130	168	209	247	258	+18	+110	+149			
3c & 4c	(Re)	78	100	134	176	201	240	249	244	217	246			
7 & 8	Prs	153	205	266	340	393	459	508	578	629	694			
7c & 8c	(Prs)	114	175	248	319	376	443	488	505	499	531			
		Weight Change in Pounds												
1 & 2	Pl	0.24	0.28	0.39	0.41	0.46	0.53	0.49	0.02	0.01	+0.12			
1c & 2c	(Pl)	0.22	0.26	0.32	0.38	0.45	0.50	0.46	0.51	0.55	0.57			
3 & 4	Re	0.23	0.28	0.34	0.41	0.45	0.52	0.48	+0.01	0.00	+0.14			
3c & 4c	(Re)	0.26	0.31	0.37	0.47	0.46	0.53	0.51	0.56	0.58	0.63			
7 & 8	Prs	0.21	0.25	0.33	0.39	0.43	0.48	0.47	+0.04	+0.02	+0.13			
7c & 8c	(Prs)	0.20	0.24	0.30	0.36	0.40	0.46	0.44	0.49	0.52	0.54			

[1] Pl = Plain; Re = Reinforced; Prs = Prestressed
[2] All ( ) refer to companion specimens which were not placed in brine, but left in laboratory air.
[3] All specimen readings indicate shrinkage compared to the 28-day fog readings except those marked with a "+" which indicates expansion.

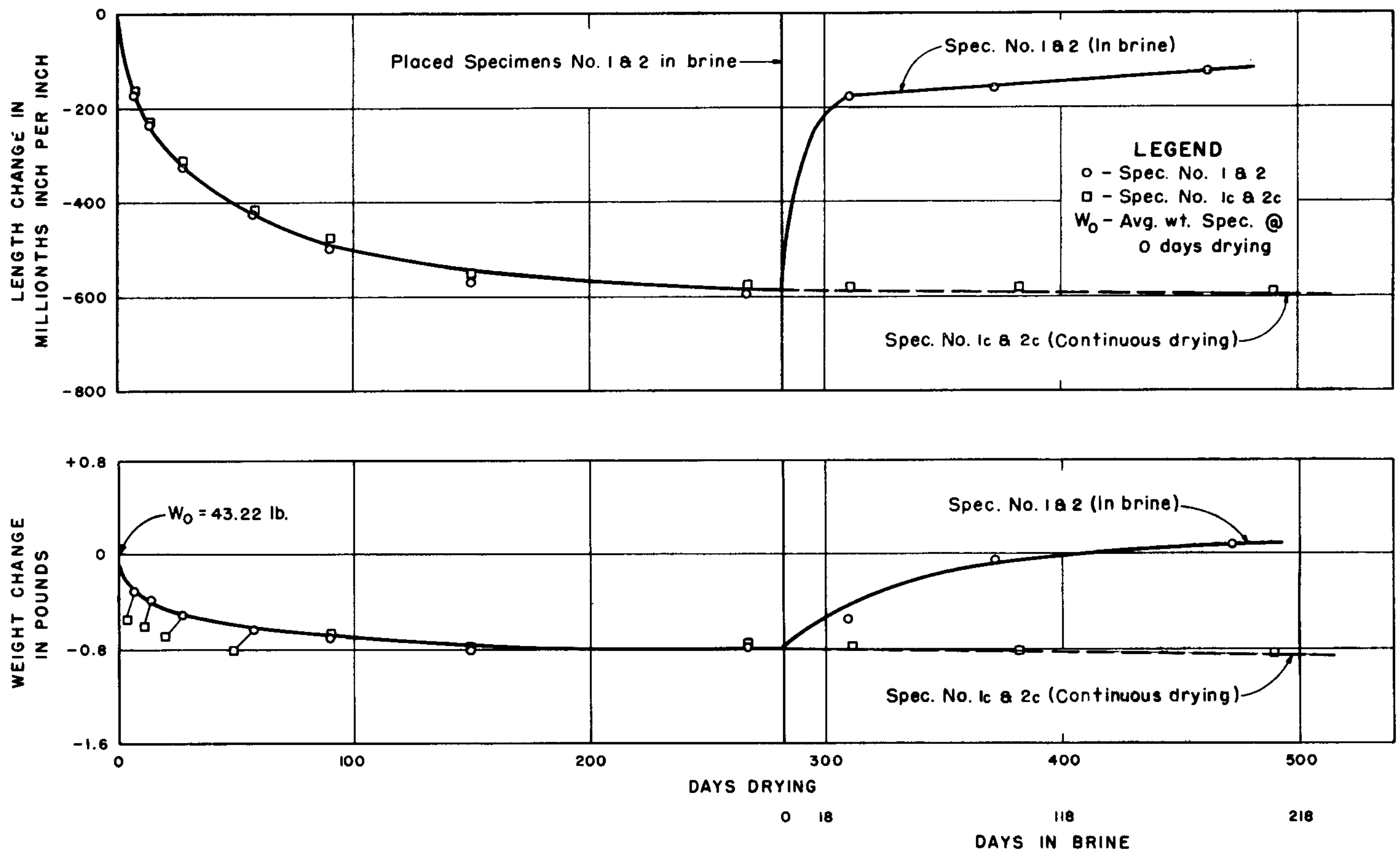

FIGURE III - 57: LENGTH AND WEIGHT CHANGE OF PLAIN NATURAL AGGREGATE CONCRETE SUBJECTED TO 203° F. BRINE AND CONTINUOUS DRYING

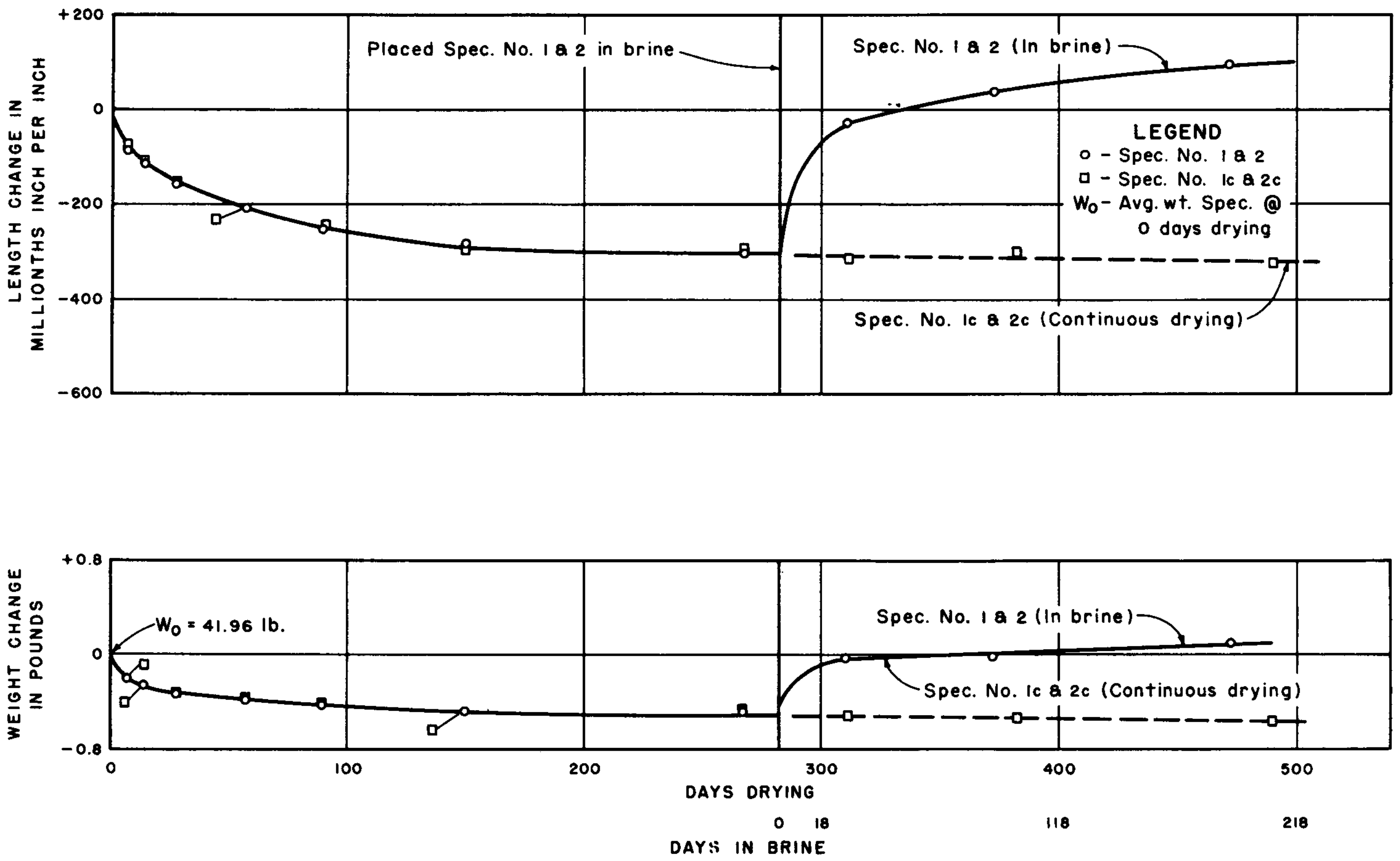

FIGURE III - 58 : LENGTH AND WEIGHT CHANGE OF PLAIN CRUSHED LIMESTONE AGGREGATE CONCRETE SUBJECTED TO 203° BRINE AND CONTINUOUS DRYING

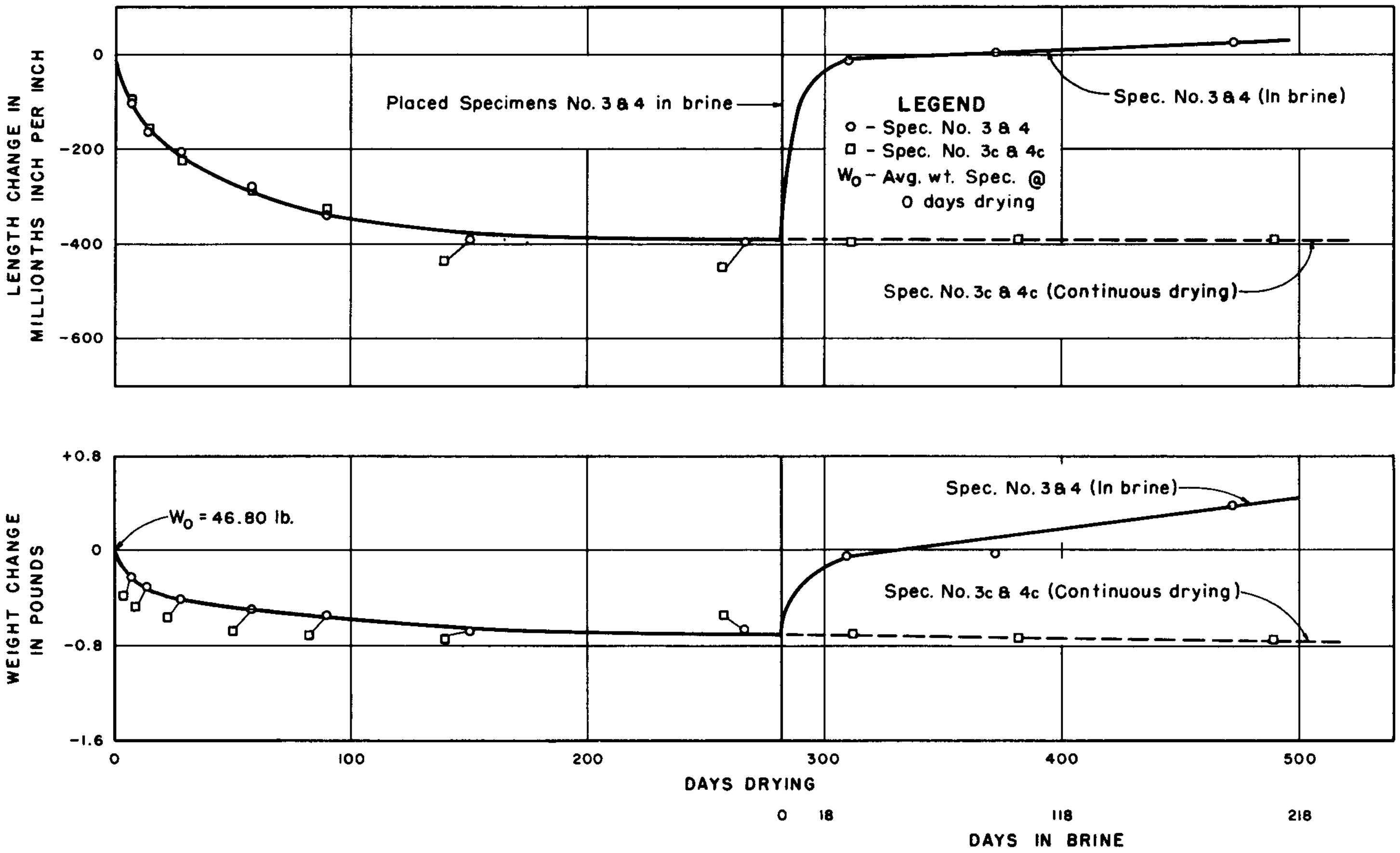

FIGURE III - 59: LENGTH AND WEIGHT CHANGE OF REINFORCED NATURAL AGGREGATE CONCRETE SUBJECTED TO 203° F. BRINE AND CONTINUOUS DRYING

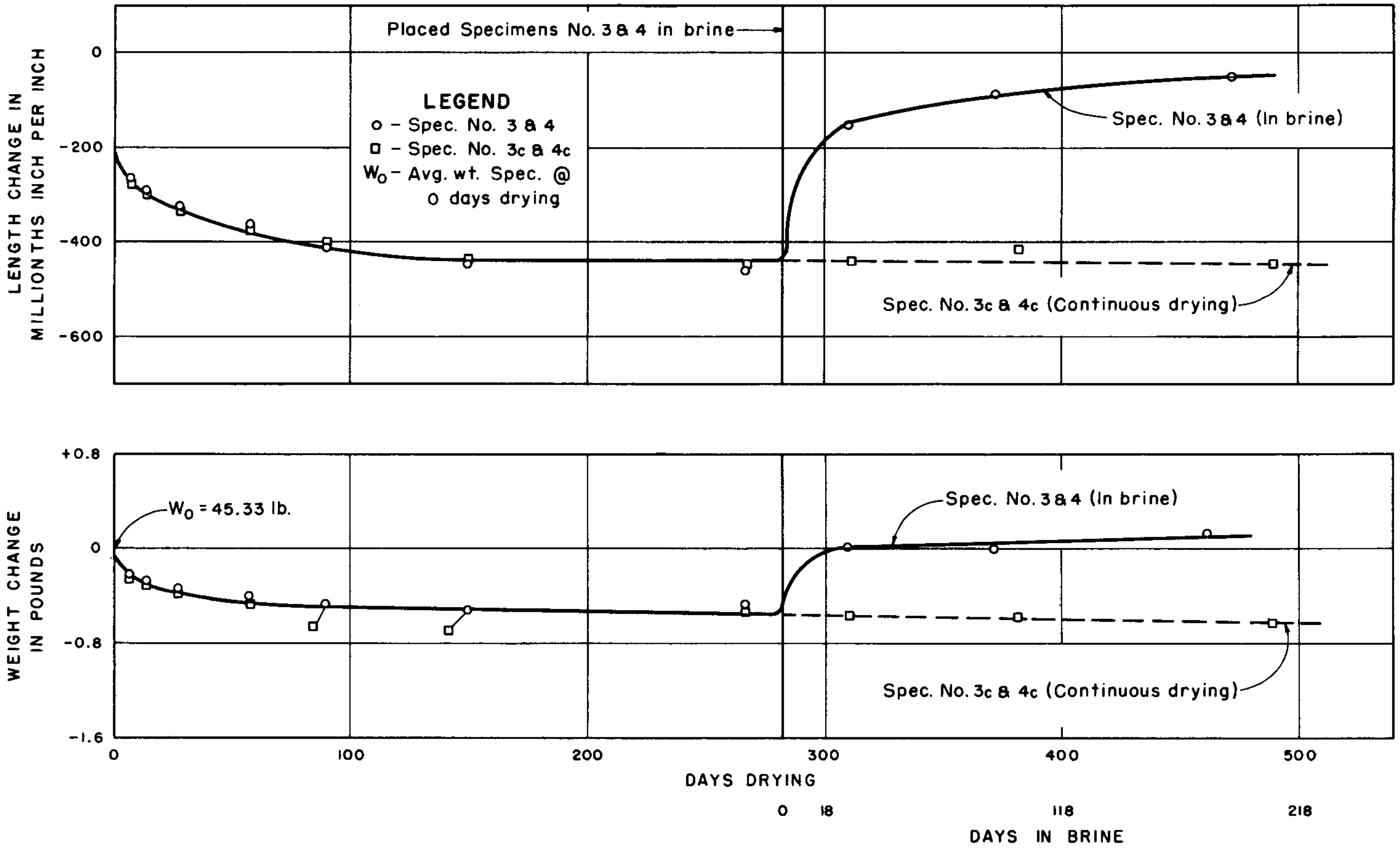

FIGURE III – 60: LENGTH AND WEIGHT CHANGE OF REINFORCED CRUSHED LIMESTONE AGGREGATE CONCRETE SUBJECTED TO 203° F. BRINE AND CONTINUOUS DRYING

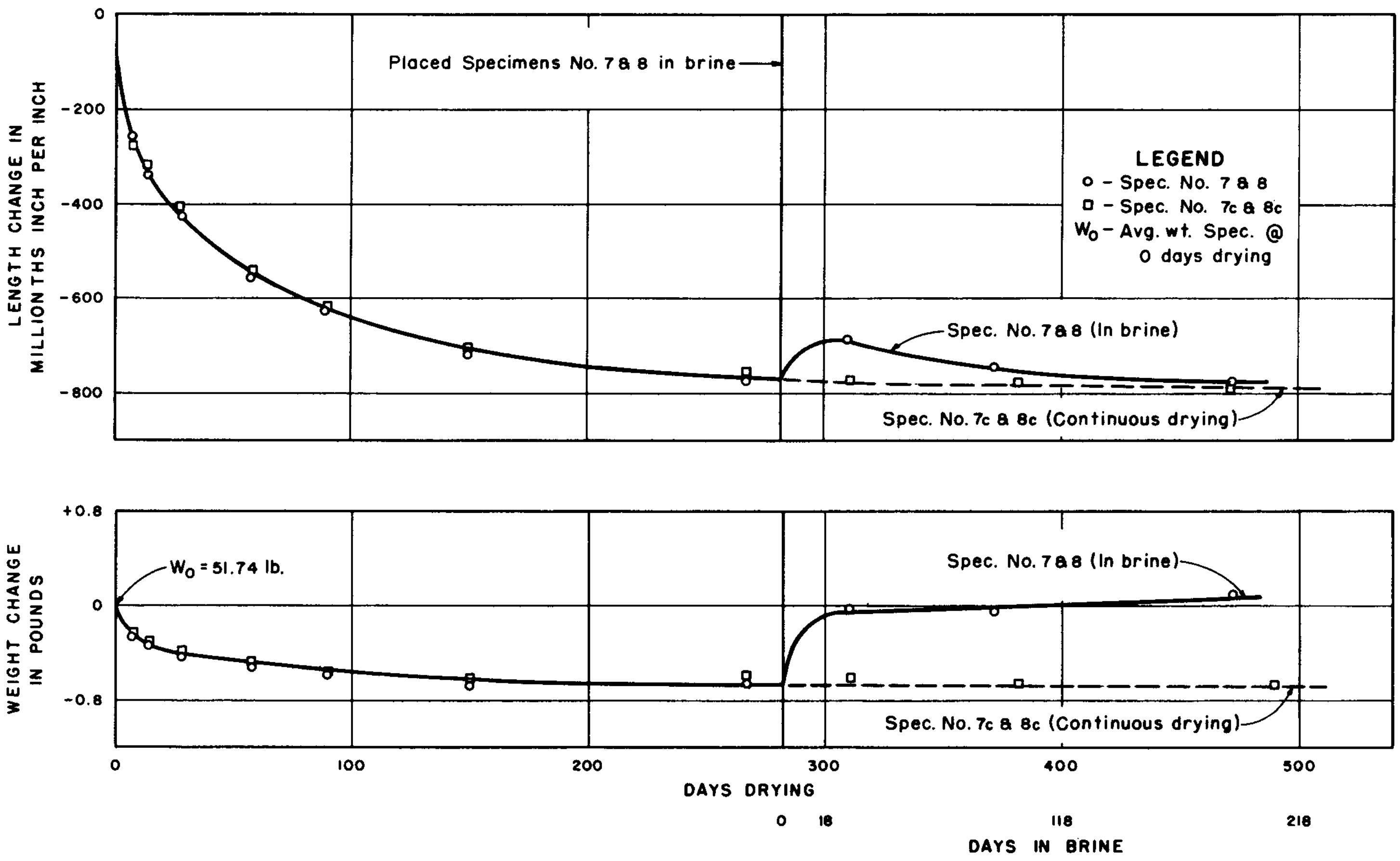

FIGURE III - 61 : LENGTH AND WEIGHT CHANGE OF PRESTRESSED NATURAL AGGREGATE CONCRETE SUBJECTED TO 203° F. BRINE AND CONTINUOUS DRYING

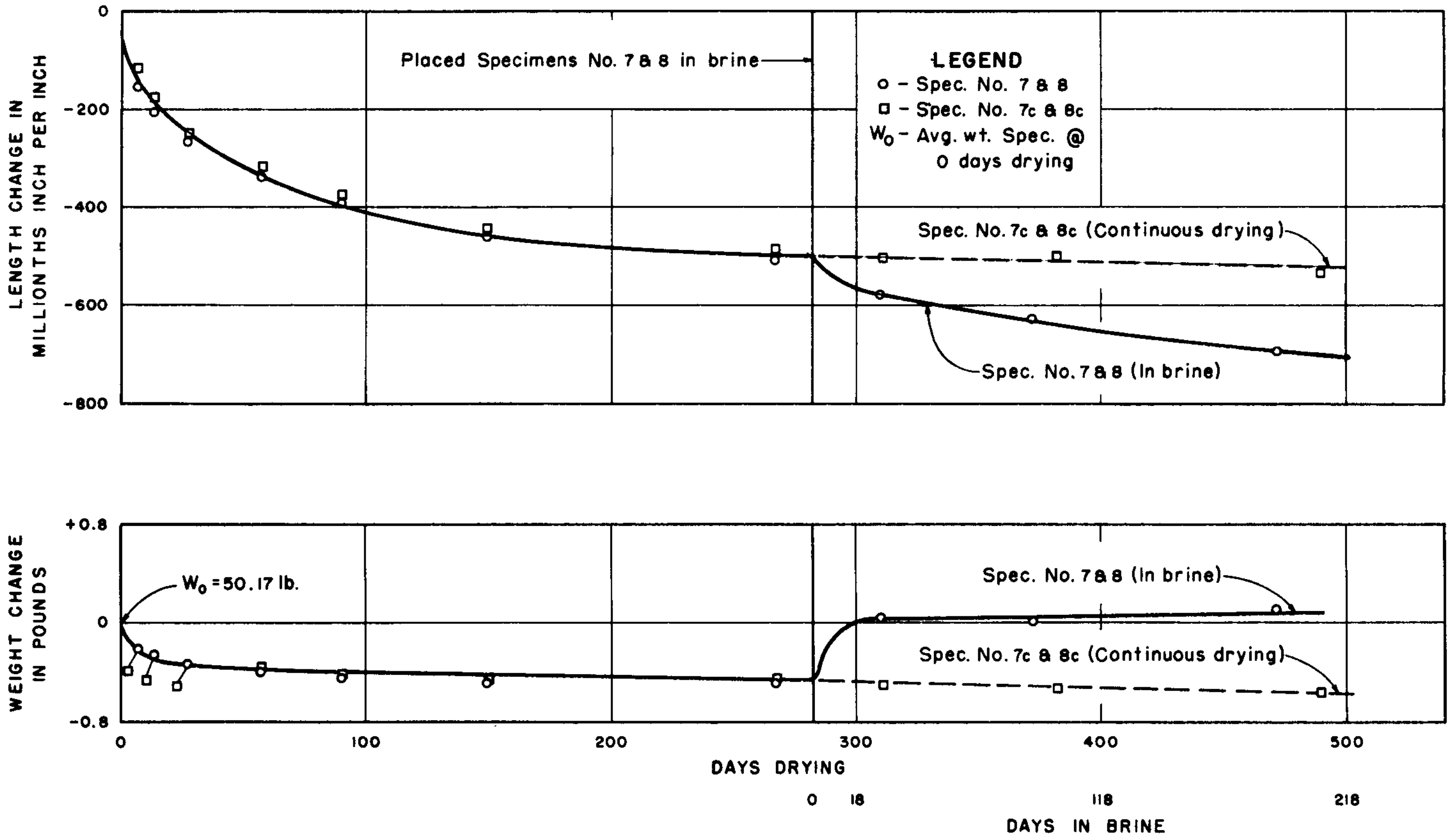

FIGURE III - 62 : LENGTH AND WEIGHT CHANGE OF PRESTRESSED CRUSHED LIMESTONE AGGREGATE CONCRETE SUBJECTED TO 203° F. BRINE AND CONTINUOUS DRYING

Figure III-63. Concrete orifice 4 by 9 by 8 inches prior to subjected continuous 10-feet-per-second flow of 100° F synthetic sea water. Photo PX-D-55286 NA

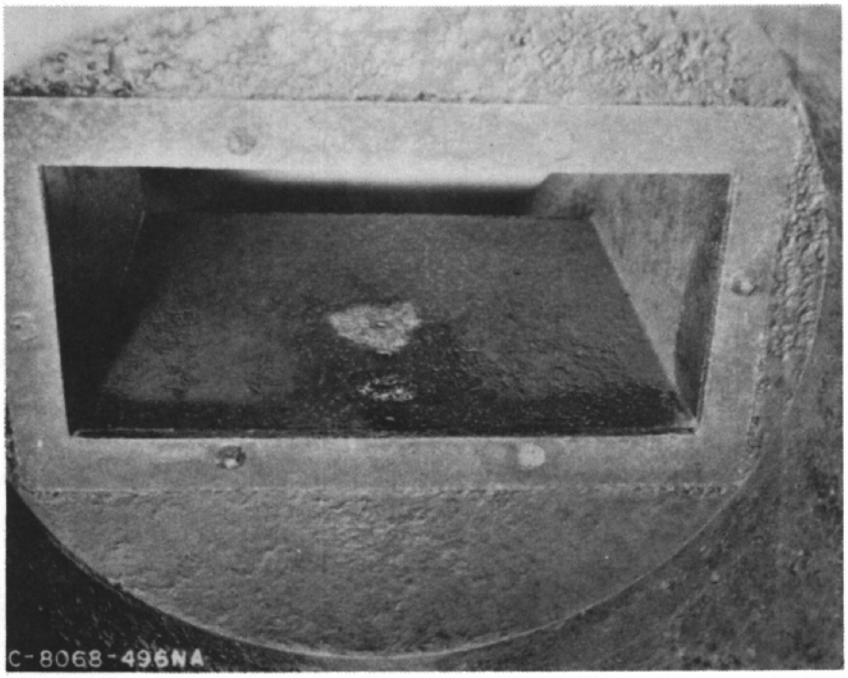

Figure III-64. Concrete orifice after 2 years' subjection to flowing 100° F synthetic sea water. Photo PX-D-60853

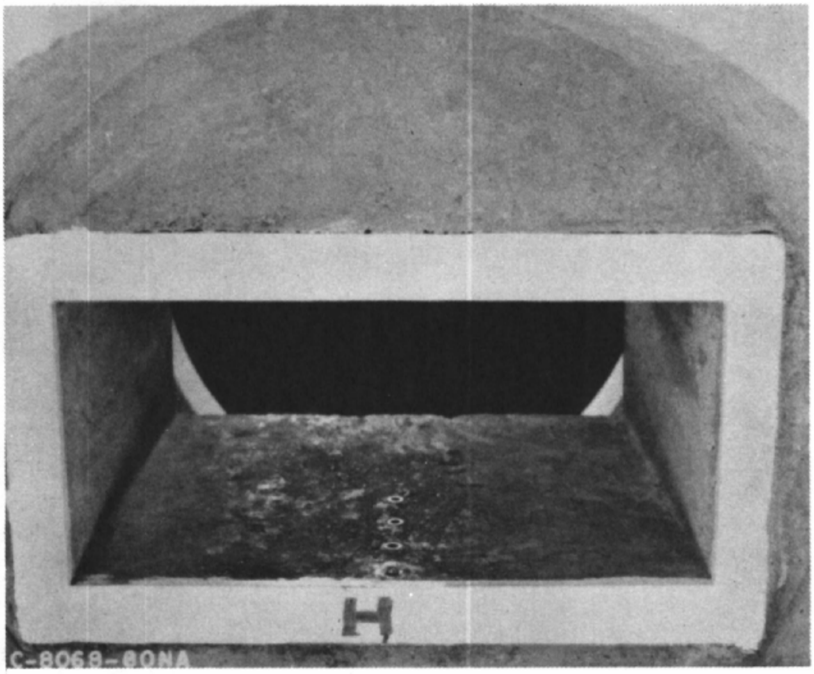

Figure III-65. Concrete orifice, 4 by 9 by 8 inches prior to subjection to a continuous 10-feet-per-second flow of 290° F synthetic sea water. Photo PX-D-53637

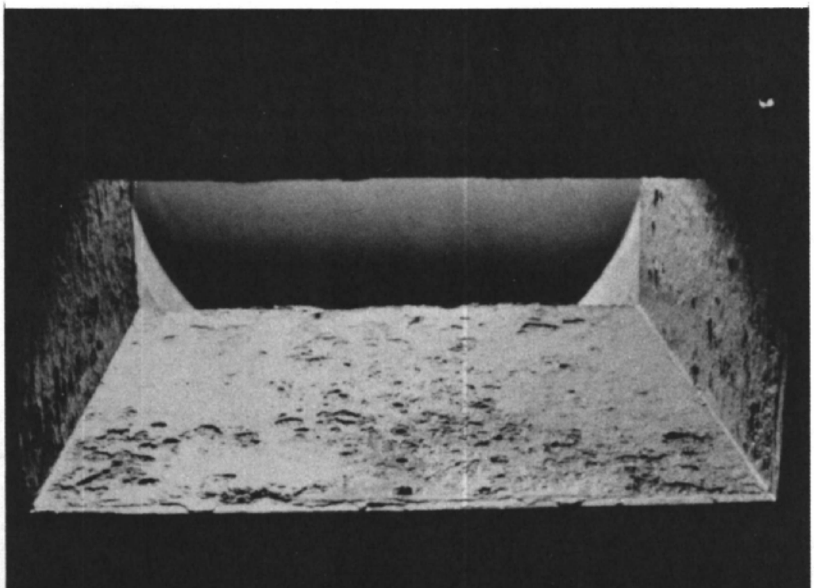

Figure III-66. Concrete orifice after 1 year's subjection to flowing 290° F synthetic sea water. Photo PX-D-53638

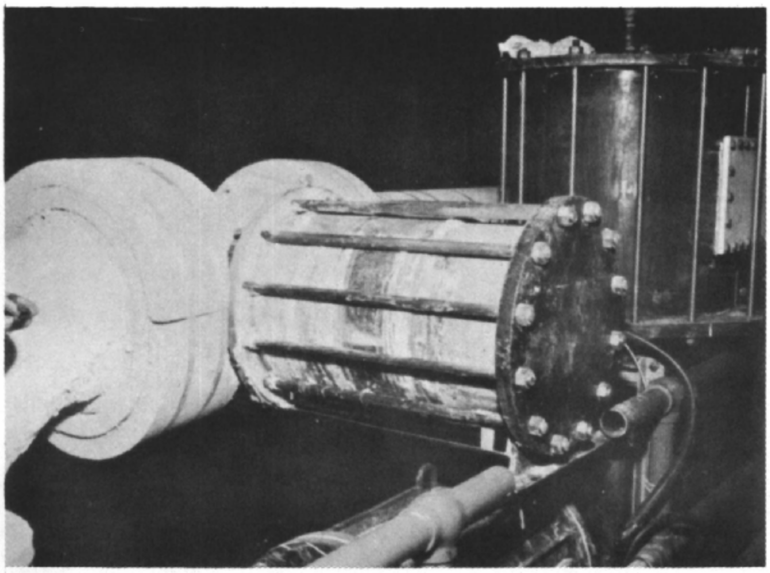

Figure III-67. Concrete pipe section undergoing severe test in 290° F synthetic sea water test environment. Photo PX-D-55279 NA

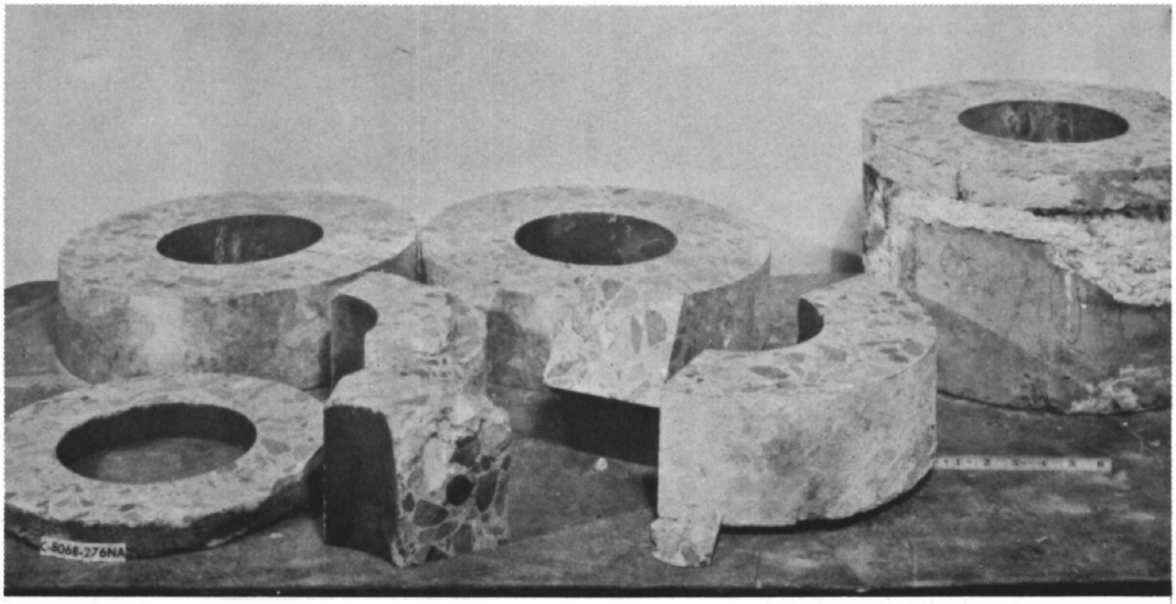

Figure III-68. View of sawed concrete pipe section after about 180 days' exposure to the hot brine environment. Photo PX-D-55280 NA sea water brine at 58 psi pressure; whereas, on the outside the environment was laboratory air at plus or minus 80° F and atmospheric pressure (12.2 psi). During the 180 days that this pipe section was under test it leaked brine in a number of places. Leakage appeared to be more severe at the beginning than at the end of the test although this was not determined by actual measurement of the leaked brine. Subsequent to brine exposure the pipe section was sawed into sections to permit a complete examination of the interior of the concrete and of the reinforcing steel, Figure III-68. A petrographic analysis was made and is presented in Section VII.

From visual observation of the interior of the pipe concrete, it appeared that:

1. Leakage occurred through some cracks;
2. A very thin softened layer of concrete covered the inside perimeter;
3. The interior concrete was sound; and
4. The reinforcing steel was slightly corroded.

III. 6. 3. 3. Asbestos-cement pipe tests. --

A few sections of 8-inch-diameter asbestos-cement (AC) pipe have been subjected to the 290° F brine through most of the systems operation, Figure III-69. These were random autoclave-cured specimens obtained from other investigations and were tested in the hot brine under static conditions. A petrographic analysis of the condition of these specimens after 593 days of exposure has been made and reported in Section VII. A visual inspection of the specimens indicated that brine had penetrated most of the way through the pipe.

III. 6. 3. 4. Thermal expansion tests. --

Tests have been conducted upon 2- by 2- by 4-inch bars sawed from larger sections of the natural aggregate concrete to determine the coefficient of thermal expansion. Results of these tests, which were conducted at three moisture conditions, oven dry, 75 percent saturated and 100 percent saturated, and between temperature limits of 30° and 90° F indicated coefficients of thermal expansion of 4.1, 4.8, and 4.0 millionths per degree F, respectively. Similar tests were also conducted upon specimens fabricated of crushed limestone coarse aggregate concrete at the same moisture conditions and temperature limits. Coefficient of thermal expansion values were 3.0, 3.5, and 2.7 millionths per degree F of specimens that were oven dry, 78 percent saturated and 100 percent saturated, respectively.

III. 6. 3. 5. Air and water permeability tests. --

Air and water permeability tests on the concretes utilized in this program have and are being made. Because the air permeability of concrete has not been determined in the past, special equipment for use in making such determinations was developed by modifying existing water permeability equipment.

Preliminary tests were conducted on both the natural and limestone aggregate concretes, referred to in Table III-15, Mixes No. 1 and 3. Water permeability tests utilized a constant 400-psi pressure, whereas the air permeability test pressures varied from 15 to 100 psi.

Water permeability of the natural and limestone aggregate high-quality concrete averaged 0.049-cubic-foot-per-square-foot per year per foot thickness at test pressure of 400 psi. Air permeability of the natural aggregate concrete averaged 0.5-, 1.9-, 2.5-, and 6.2-cubic-feet-per-square-foot per year per foot of thickness at test pressures of 15, 25, 50, and 100 psi, respectively. Air permeability of the limestone aggregate concrete averaged 0.0-, 0.0-, 0.2-, and 1.0-cubic-foot-per-square-foot per year per foot at test pressures of 15, 25, 50, and 100 psi, respectively.

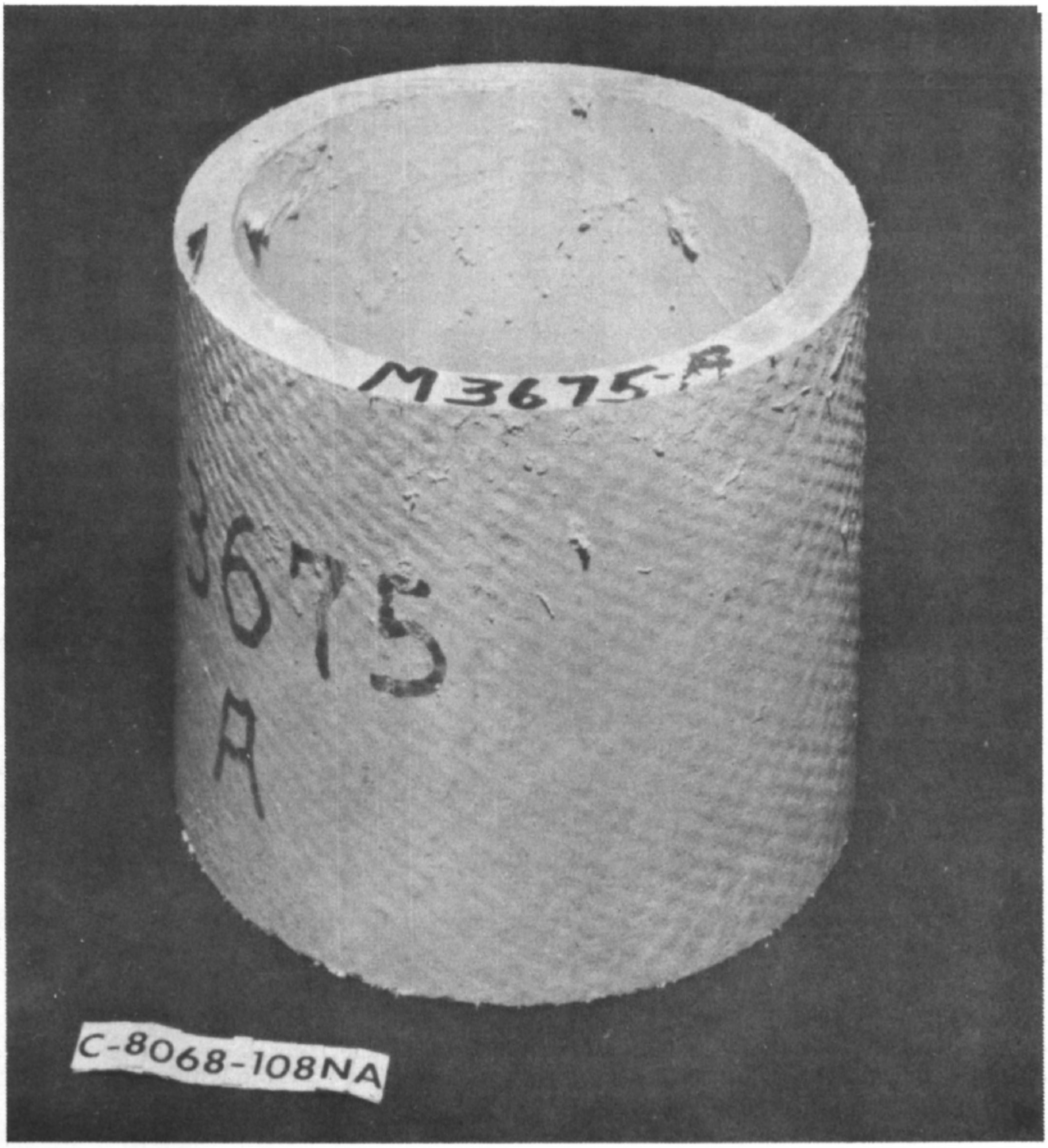

Figure III-69. Typical 8-inch-diameter section of asbestos-cement pipe as tested in the 290° F brine. Photo PX-D-55291 NA

Since the aforementioned preliminary tests, air permeability tests of concrete have been continued on an intermittent basis to evaluate equipment and test procedures. For these tests, equipment was modified to eliminate water coming in contact with the test specimens. The sensitivity of the measuring apparatus was increased.

Tests were performed under a constant pressure of 25 psi. Air pressure was supplied by in service lines, and air was filtered and regulated to desired pressure. Volume of air flowing through a specimen was measured by displacement of water as air collected in a calibrated chamber. This chamber was vented to permit measurements at atmospheric pressures, thus eliminating any effect of variable barometric pressures, Figure III-70.

Results of recent tests performed on concretes containing a good quality aggregate and two water-cement ratios confirm data reported by other investigators4/ that the moisture condition of concrete greatly influences its permeability to air. Tests performed in the Bureau of Reclamation laboratories, utilized fog-cured concrete and initially no air flowed through either 0.45 or 0.55 water-cement ratio concrete. Following oven drying for 3 days at 230° F and retesting, the 0.55 water-cement ratio concrete passed air freely, permitting a flow of 153.6 cc/sq ft/hr/ft/psi (71.1 $cc/cm^2/hr/cm/kg\ cm^2$) under 25-psi test pressure. However, the lower water-cement ratio concrete (0.45 W/C) remained impermeable and was further dried at 230° F for 2 weeks. Still, the concrete remained impermeable to air even under pressures to 60 psi. Further tests are contemplated to substantiate these data.

To observe the influence of water the 0.55 water-cement ratio concrete was uncapped while the constant 25-psi pressure was maintained on the bottom and the top of the specimen flooded with water. Initially, the water above the specimen showed vigorous bubbling over the entire surface. However, after 2 weeks of keeping the surface flooded the air flow steadily decreased from vigorous at first to essentially no flow. This reduction is related to bubbling pressure required to force air through a saturated porous media. This problem has been investigated in the Bureau of Reclamation laboratories (Soils Engineering Branch). Bubbling pressure in permeability tests of a water-saturated porous media is the pressure required to overcome surface tension between water and the pore walls through which the flow must pass. This pressure might be very high, depending upon the size of capillary tubes. Large pores require low pressures, while small pores require high pressures to overcome the surface tension.

Testing will be continued to further refine test equipment and test procedures. It is proposed to test concrete as a seal in a vacuum chamber. This will more closely simulate conditions under which concrete would be used in a distillation chamber. Since there will be water present, it is also proposed to test the concrete under different moisture conditions. Other studies will be made to determine the effect of water-cement ratio, temperature, aggregate, and slump on permeability of concrete to air.

III.7. References

1. "Effects of False Set in Portland Cement on Properties of Concrete," J. T. Dikeou, USBR Lab. Rept. C-1192, June 30, 1966.

2. "Seal Performance in Brine Circulating Pumps," J. R. Graham, USBR Lab. Rept. C-1253, November 1, 1967.

3. "Flyash Improves Concrete and Lowers Its Cost," A. S. Pearson and T. R. Galloway, Civil Engineering, September 1953.

4. "Paper on Permeability of Concrete to Air," Ontario Hydro-Research Quarterly, First Quarter, 1966.

5. "Evaluation of Concrete for Desalination Plants, "E. C. Higginson, J. E. Backstrom, (A paper presented at the ASCE Water Resources Engineering Conference, Denver, Colorado, May 16-20, 1966).

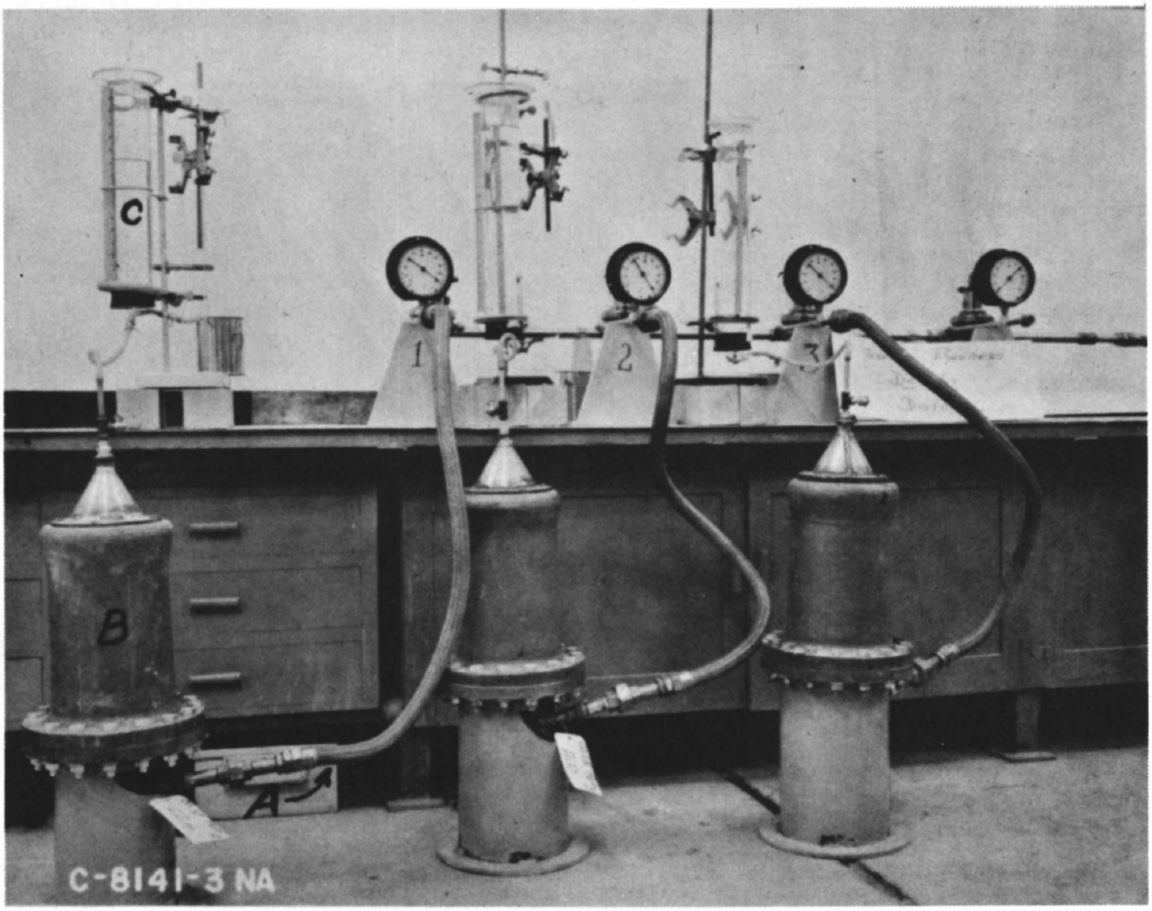

Figure III-70. Air permeability test equipment;

(A) Inflow air line
(B) Specimen chamber
(C) Apparatus used for measuring outflow.

Photo PX-D-60856

SECTION IV

PART II--PANEL AND MODULE STRUCTURAL STUDIES INCLUDING CREEP[1]

IV. 1. Purpose and Summary of Results

IV. 1. 1. Purpose

Design and operation of distillation plants for sea water conversion necessitates consideration of structural problems associated with high temperature, high humidity, and varied pressures. In studying the use of concrete as a shell material for evaporator vessels, high temperatures and temperature variations become extremely important. Limitations must be placed on temperature variations across the shell wall thickness to prevent overstress. "Startup" and "shutdown" temperature control rates may thus be required to prevent extreme slopes of the time dependent (unsteady state) temperature distribution curves.

The objectives of this investigation are to study the collective influence of high temperature, high humidity, and varying pressure on concrete structural elements and to compare theoretical numerical analysis methods for the heat transfer problem with test results obtained from model studies. The influence of each of these factors must be studied theoretically and experimentally. It is anticipated that reliable theoretical predictions of time dependent temperature distributions can be ascertained for the problem mentioned above.

IV. 1. 2. Summary of Results

The primary reason for conducting laboratory studies of a prototype wall panel was to determine how closely the actual unsteady-state-temperature distribution compared with the theoretical distribution obtained by a numerical procedure discussed in Sec. IV. 3. 3., General Report No. 37.

In general, the correlation of the actual with the theoretical temperature distribution was good. However, two problems had to be resolved before the correlation was acceptable. The first problem involved the use of the surface thermocouple readings as the boundary conditions for the theoretical solution. Since these temperatures are used to calculate the internal temperatures, an error here would produce poor correlation of results. It was found that the surface temperatures were too high. This was probably due to the radiant energy of the heat source. Therefore, the first subsurface thermocouple readings (one-half inch below the surface) were taken as the boundary temperatures. Using these temperatures, a satisfactory correlation was obtained. The second problem involved the treatment of the steel reinforcement gridwork in calculating the temperature distribution. Three methods were considered to account for the steel gridwork. (1) The steel grid with the concrete is considered as two parallel thermal resistors; (2) the steel plus the concrete is a composite material of weighted thermal properties; (3) the steel layer is to be ignored, considering the layer as concrete only. Methods 1 and 3 were rejected and Method 2 used in the final analysis. Method 1 was rejected because a large amount of hand calculations were necessary for computer use. Method 3 was rejected because the steel was not taken into account.

The time increment used was 1 minute. A complete time-temperature distribution was calculated through the wall for each 1 minute of theoretical heating or cooling time.

The temperatures of the surface layers of concrete were not used whenever such layers were exposed to the air.

Test procedures and part of the test results are given in Sections IV. 3. 1, IV. 5. 2, IV. 5. 3 and IV. 5. 4 of General Report No. 37.

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[1] Formerly entitled, "Wall Section Model Studies."

Figures IV-14 and IV-15 give a graphical comparison of actual and theoretical results of Test No. 1. Figure IV-14 gives the heating cycle; Figure IV-15 gives the cooling cycle. There is a greater variation between the comparable curves for the heating cycle of Test No. 1 (Figure IV-14) than for other tests. This variation can be attributed to two causes: (1) neglect of the variation in thermal properties of the highly influential insulating material and (2) apparent overweighing of the calculated conductive properties of the combined steel and concrete layers. This overweighing shows up in all the graphical results as a "flattening" of the theoretical slopes.

Figures IV-16 and IV-17 give the results in graphic form of Test No. 2. Figure IV-16 is the heating cycle; Figure IV-17 is the cooling cycle. The results are good as is shown by the close agreement of the curves at the 80-hour heating curve and the 36-hour cooling curve.

Figures IV-18 and IV-19 show the results graphically for the short duration test undergoing the rapid "hot face" temperature changes. Figures IV-12 and IV-13 give typical heating and cooling curves of boundary plane thermocouples. Figure IV-1 shows equipment used to determine the thermal properties of the wall concrete.

IV. 2. Definition of Project

IV. 2. 1. Wall Model Studies

A thorough study of the literature on heat transfer through walls revealed analytical and numerical methods for determing temperatures as a function of time. However, a true analytical solution is frequently impossible in light of boundary conditions, and even for the simplest of cases the method is quite cumbersome. On the other hand, the numerical approach is far simpler, much more flexible and adaptable to changing boundary conditions, and in addition gives a progressive solution to the problem so that the complete time-temperature history is at hand. The numerical procedure has been found to give reliable results when applied to heat transfer through walls of essentially homogeneous materials, either singly or in series with other materials. A shell wall of an evaporator consisting of reinforced concrete is, however, quite complex. Thermal properties of concrete vary due to the particular mix ingredients and the nonhomogeneous nature of concrete itself. In addition, the percentage of steel reinforcement will probably be quite high, and since its thermal characteristics are different than the surrounding concrete, its effect on the time-dependent temperature distribution may be appreciable. The problem is further complicated by the fact that the steel is not a solid layer in series with concrete but is present as a network layer of bars with concrete between. It was, thus, highly desirable that research be undertaken in the form of a laboratory thermal test on an instrumented reinforced concrete panel to confirm or disprove theoretical results obtained by a numerical method expanded to suit problems of this nature.

IV. 2. 2. Environmental Test Chamber Studies

The use of concrete as a shell material for evaporator vessels not only requires the investigation of its structural capabilities under thermal stresses but also its durability under conditions of high temperature, high humidity, and varied pressures.

The use of an environmental test chamber has been proposed to investigate the physical properties of concrete after subjection during varying lengths of time to different pressures, temperatures, and humidity conditions. A canvas of manufacturers showed that there is no available commercial chamber that will test the concrete under the conditions that are required; therefore the Bureau may design and instrumentate the chamber. The construction of the chamber will be done by a commercial firm.

IV. 3. Methods

IV. 3. 1. Test Requirements

Four tests were contemplated utilizing one reinforced concrete panel. A brief description of the proposed tests follows:

Test No. 1

Apply a gradual temperature rise and subsequent cooling at the rate of 5° per 15 minutes between 80° and 320° F to the hot surface of the concrete panel. The air adjacent to the cool side is to be maintained at 70° F. The limiting values being maintained in each case until a steady-state thermal gradient is attained. The test panel is to be thoroughly instrumented with thermocouples to enable recording of temperatures through the panel thickness at selected intervals of time. A 2-inch-thick fiberglas insulation layer is attached to the cool face of the panel. Edges of the panel should be supported but not restrained from horizontal and vertical movement. Adiabatic conditions should be maintained vertically and horizontally at the faces.

Test No. 2

Testing procedure for this test is identical to that of Test No. 1. The only exception is that the insulation layer is removed from the cool side.

Test No. 3

This test differs from Test No. 2 in that the rate of temperature rise and drop between 80° and 320° F on the hot surface of the panel is to be as rapid as practicable.

Test No. 4

Testing procedure is identical to Test No. 3 with exception that the insulation layer is again attached to the cool face.

IV. 3. 2. Material Properties

Supplementary tests for determination of thermal properties of the test panel materials were also required. These properties were utilized in the numerical solution for obtaining time-dependent temperature distributions. Required properties were determined by the Corps of Engineers method as described in their "Handbook of Concrete and Cement." Figure IV-1 shows the equipment required for obtaining the thermal properties.

IV. 3. 3. Numerical Method

The theoretical solution to the heat transfer problem is a numerical method which utilizes finite difference expressions to replace differential equations. Essentially, the method is an iteration process for determing temperatures on preselected reference planes as a function of time.

To begin with, thermal properties of the wall materials and boundary media must be known or assumed. Then, depending upon desired accuracy and certain interrelated restrictions, selection is made for finite space and time increments and appropriate dimensionless numbers and weighting factors are computed. Solution starts with a given temperature distribution through the wall and proceeds by steps, each step yielding the temperatures on the reference planes for an accumulated time increment. The repetitive calculations are carried out through as many steps as are necessary to arrive at a desired time. Since each step represents a given time, a complete time-temperature history is developed. Calculations of a "new" temperature at a given position and time utilizes the previously mentioned weighting factors and three "old" temperatures from the step before. The three influential "old" temperatures are those located on reference planes immediately to the left and right of the position in question as well as the temperature on the plane in question itself.

Solution to the heat transfer problem by the numerical method was programed for digital computer analysis. Considerable flexibility as well as accuracy can be achieved in the computer solution while dispensing with laborious hand calculations.

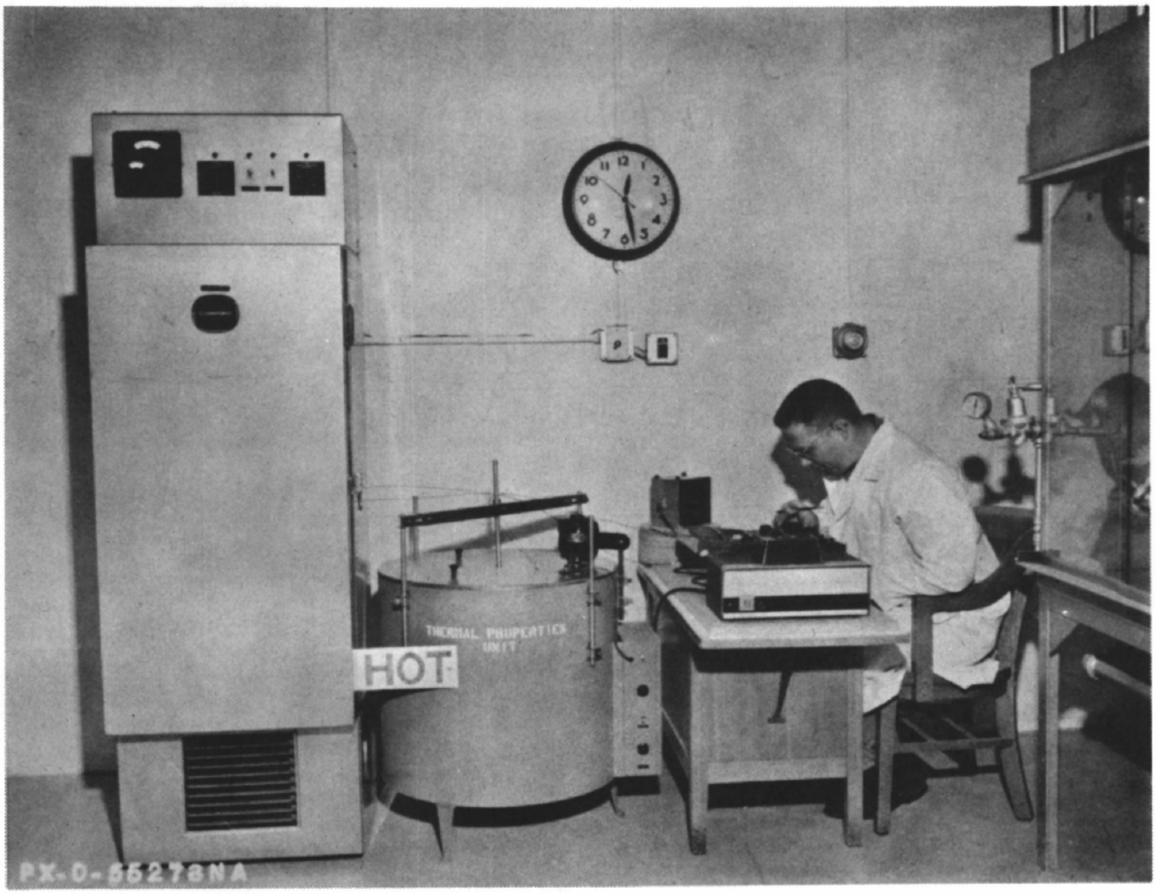


Figure IV-1. Equipment required to obtain thermal properties of concrete. Photo PX-D-55278 NA

IV. 3. 4. Environmental Testing of Concrete Specimens

The concrete specimens to be tested will be circular in shape with a minimum of 24-inch diameter of surface exposed to test environment. The thickness will vary from 6 to 18 inches. Both faces of the specimen will be exposed to environmental conditions simultaneously. The environmental conditions for one face will be a combination of the following ranges: temperature, 70° to 350° F; pressure, 1-100 psia; humidity, 90 to 100 percent. The opposite face will be exposed to a combination of the following: temperature, 32° to 150° F; pressure, 1-20 psia; humidity, 20 to 98 percent.

The concrete used in making the specimens will have many variations such as different types of cement, pozzolan, water-cement ratios, and aggregates.

Protective coatings for concrete will also be tested under various combinations of temperature, pressure, and humidity.

IV. 4. Apparatus

IV. 4. 1. Instrumented Test Panel

The test specimen was a reinforced concrete panel, 10 by 10 feet by 15 inches. The concrete mix was the same as Mix No. 1 described in detail in Table III-15. Reinforcing consisted of two A-15-type rebar mats of vertical No. 6 bars at 12 inches center to center (see Figures IV-2 and IV-5). Nine sets of thermocouples, with 11 units per set, were located through the wall, Figures IV-3 and IV-4. Each unit of a set was embedded at a different depth through the panel thickness with corresponding units of all the sets at the same depth. Figure IV-3 shows the means used to secure each thermocouple at 1 of the 9 thermocouple-set locations for concrete placement.

A 2-inch-thick layer of fiberglas rigid equipment insulation was attached to the cool wall face for 2 tests. A thermocouple was attached to the outer surface of this insulation layer at each of the 9 locations. Figures IV-4 and IV-5 show the arrangement and spacing of the thermocouples through the wall section. Including 2 thermocouples for air temperature measurements adjacent to each wall face, a maximum total of 110 thermocouples were used. Figures IV-6 and IV-7 show the forming and bracing for placement of the wall.

To obtain relative humidity changes in the wall during testing, five brass wells were embedded in the top of the wall spaced evenly across its thickness. Special relative humidity equipment is used to measure these changes.

After placement, the test panel was water cured for 28 days, with a resulting concrete compressive strength of 5, 960 psi. Figure IV-8 shows the panel during curing after the face forms had been removed. The wall section rested on its 1-inch-thick plywood floor form which was supported by 4 by 4's and concrete discs. The panel was braced by two A-frames at the ends and two logging chains at the top. Although this somewhat violates the nonrestraint requirement, it was considered necessary for safety.

A 200-channel digital scanner, shown in Figure IV-8, printed the reading of each thermocouple at any desired time. These values were then translated into temperatures from a thermocouple calibration table.

IV. 4. 2. Heating and Cooling System

The energy for controlling the temperature rise on the hot surface of the test panel was supplied by the large resistance heater shown in Figure IV-9. This heating system consisted of forty 500-watt strip heaters connected to form two 440-volt, 3-phase circuits. The heaters were spaced 1 foot from the test panel and arranged horizontally so as to provide uniform temperature on the hot wall face.

The heat chamber was well insulated with fiberglas insulation and lined with aluminum foil for heat reflection. A compressed air line was connected to 50 feet of 1/2-inch copper tubing which ran along the bottom, up one side, and across the top of the heat

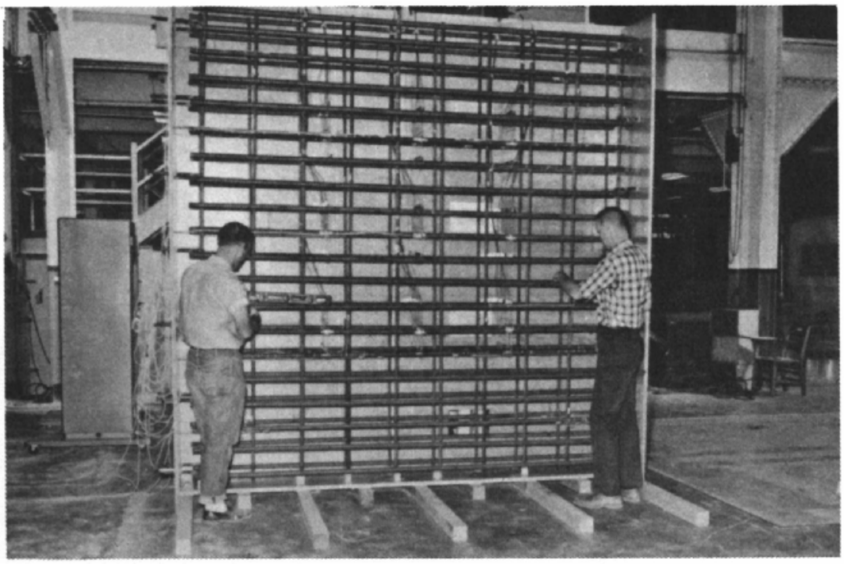


Figure IV-2. View of the two mats of reinforcing bars secured in place. Note that the thermocouples have been installed. Photo PX-D-55294 NA

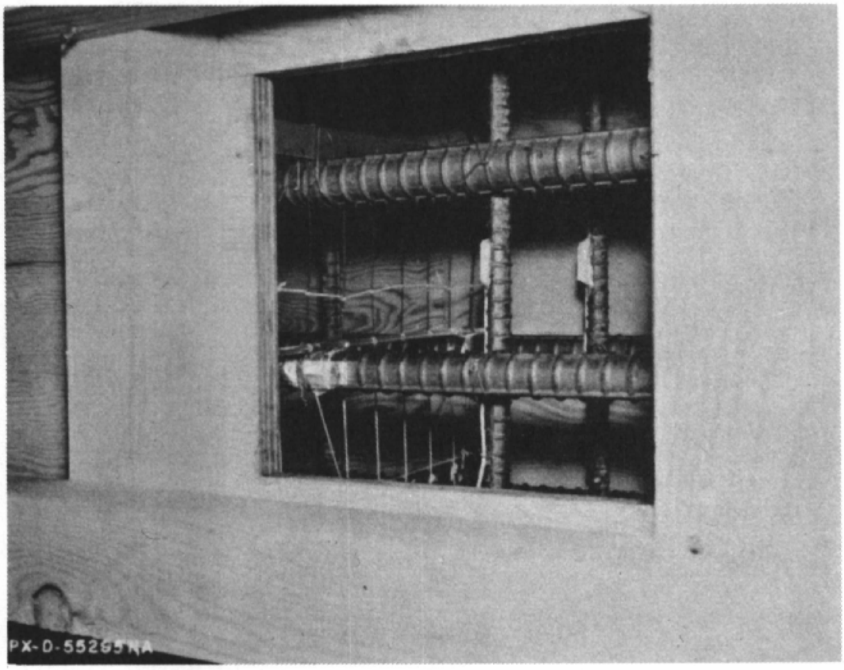
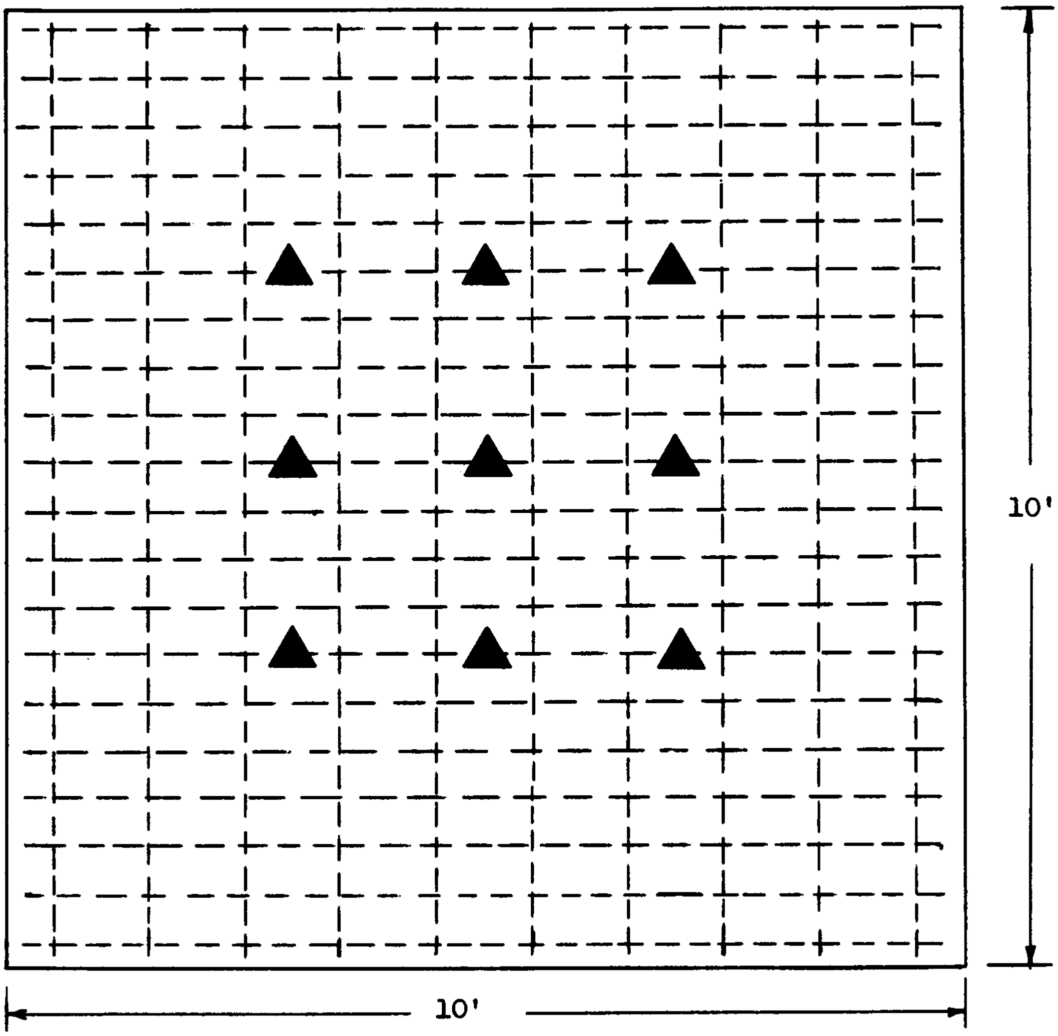


Figure IV-3. View through the forming prior to concrete placement, showing the means of securing a line of nine thermocouples for embedment at one of nine locations Photo PX-D-55295 NA

Instrumentation and Reinforcement in Concrete Test Panel

(front view)



Scale: 1 inch = 2 feet

▲ Thermocouple locations-- Gages to be mounted on wall surfaces, reinforcing bars, and embedded in the concrete.

Horizontal reinforcing bars are No. 10's spaced 6-inches apart.
Vertical reinforcing bars are No. 6's spaced 12-inches apart.
Minimum clearance between concrete wall surface and reinforcing bars is 2 inches.

Figure IV-4.

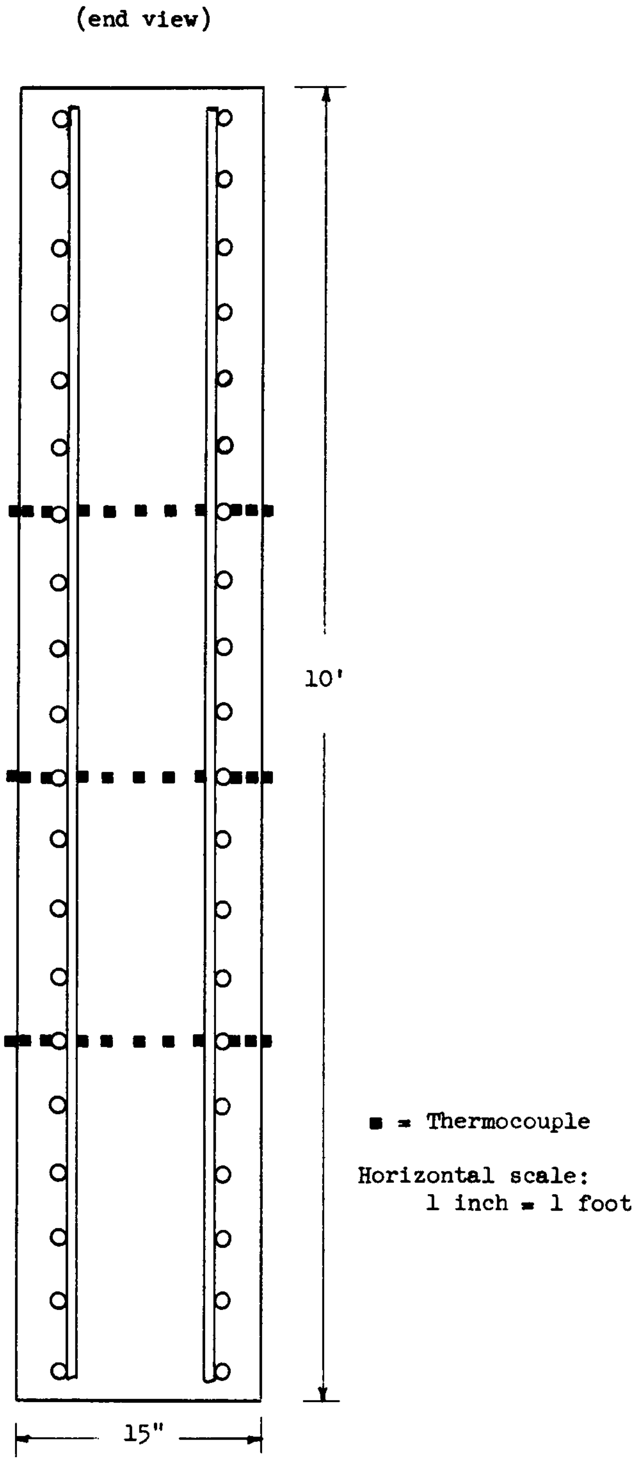


Figure IV. 5. Instrumentation and Reinforcement in Concrete Test Panel.

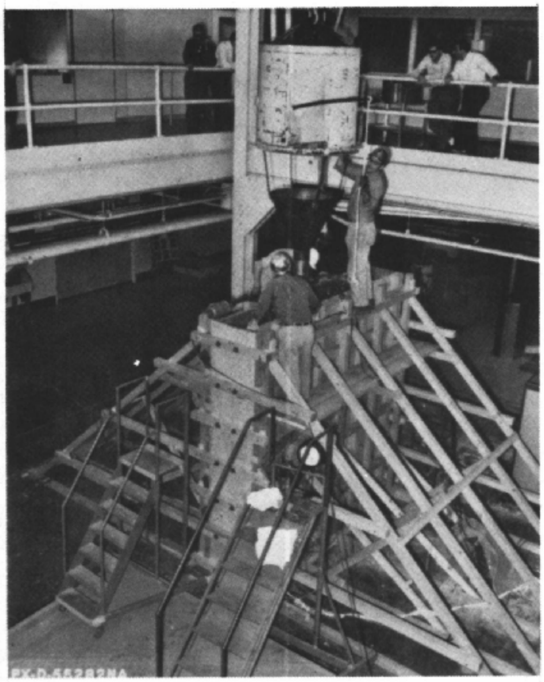


Figure IV-6. View of the forming and bracing during concrete placement.
Photo PX-D-55282 NA

Figure IV-7. View during the latter stages of the concrete placement operation.
Photo PX-D-55283 NA

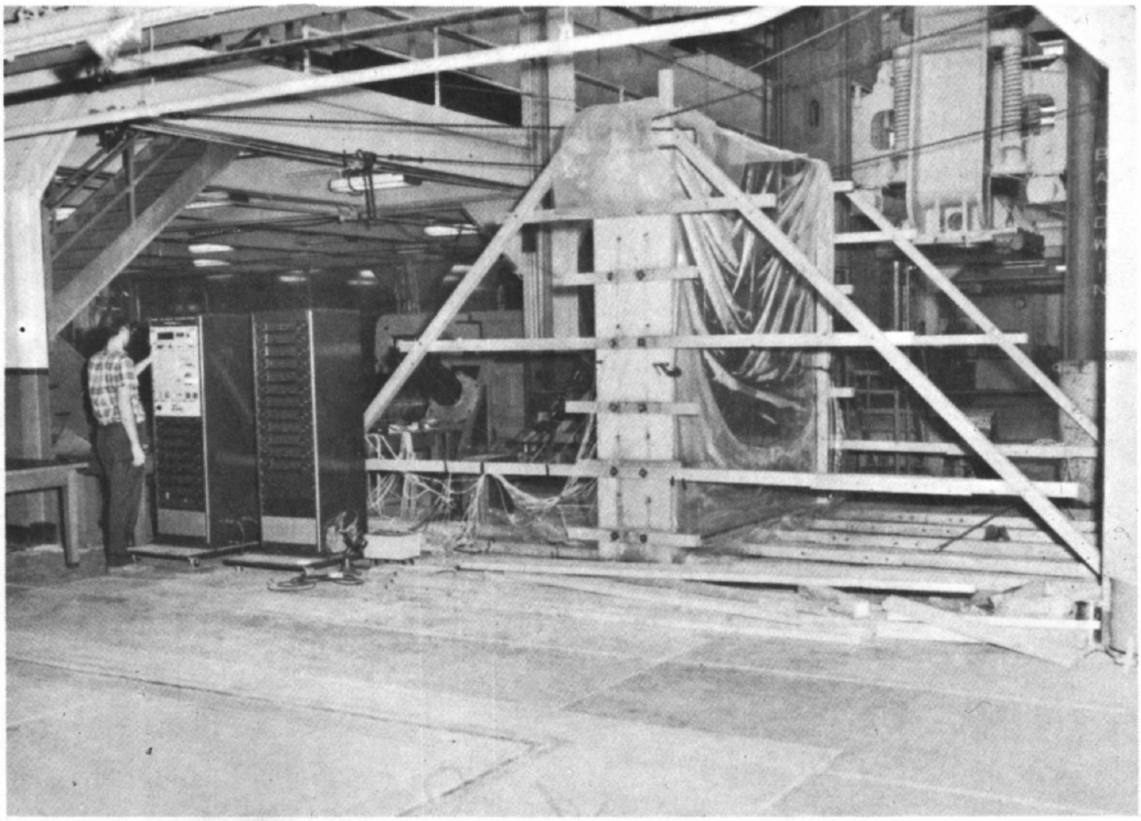


Figure IV-8. The test panel during the water curing period after face forms have been stripped. Note the digital scanner equipment which records the thermocouple readings. Photo PX-D-55277

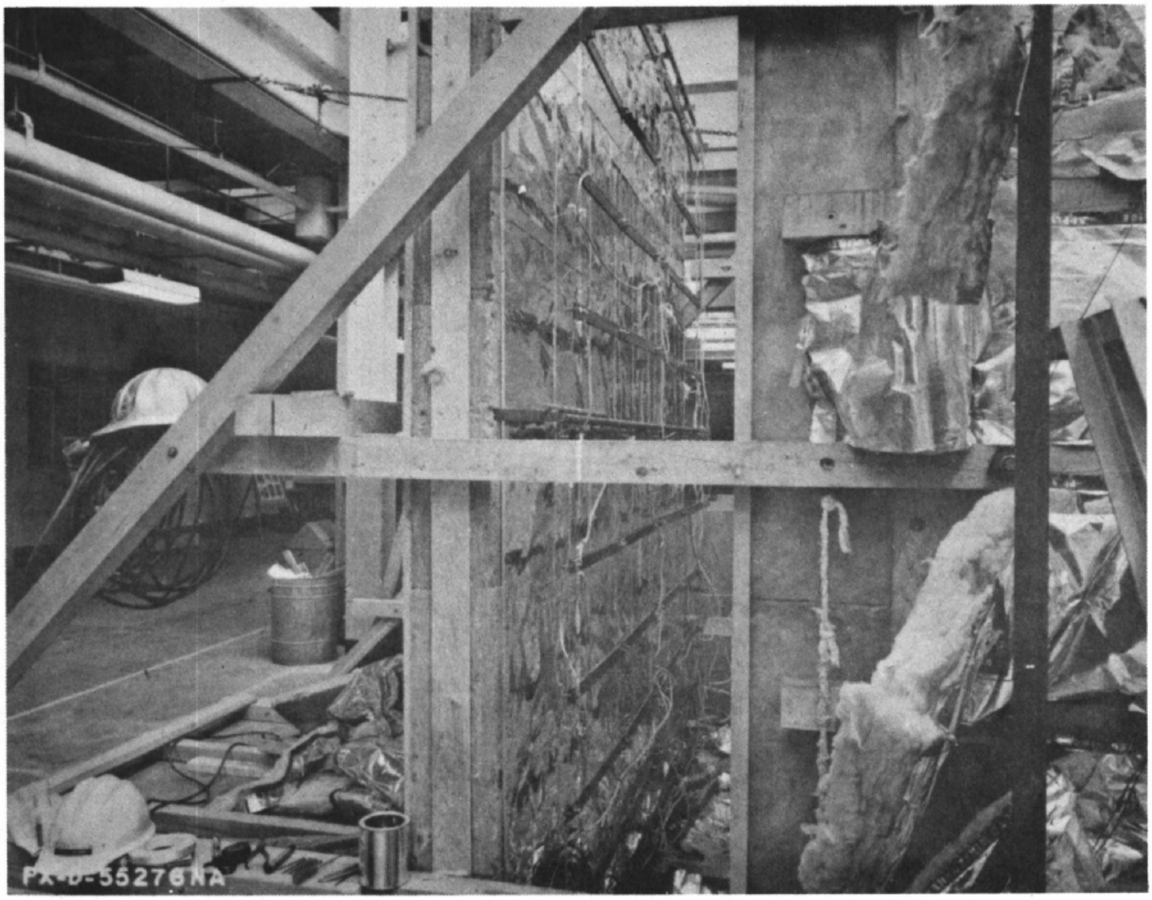


Figure IV-9. View of horizontal arrangement of strip heaters which supply the heat energy during testing. Photo PX-D-55276 NA chamber. Holes were drilled in the length of tubing across the top in such a manner that the released air helped to maintain a uniform temperature distribution over the hot face. During cooling, the insulation forming the ends and top of the heat chamber was removed in stages and fans and blowers were used to regulate the cooling temperatures on the hot surface.

An air-conditioned chamber (see Figures IV-10 and IV-11) maintained a constant air temperature of 70° F on the cool side of the wall. Two 1-1/2-ton-capacity air conditioners were used to accomplish this.

IV. 4. 3. Environmental Test Chamber Configuration

Basically, the chamber will be cylindrical in shape, approximately 6 to 7 feet long; and 36 inches in diameter, Figure IV-20. The concrete specimen will be located in the middle of the chamber so that each side can be exposed to test conditions. Observation portholes will be placed so that both sides of the specimen can be examined while the tests are in progress. Steam, air, and waterlines will be attached to both ends of the chamber to create the desired environments. Measuring and control appratus will be installed to control and maintain these environments within the prescribed limits.

IV. 5. Data and Results

IV. 5. 1. Trial Test

Prior to conducting the four main tests, a trial heating cycle was performed to lower the water content in the wall and to check the capability of the apparatus. Variations in temperature on the hot face met minimum requirements but a constant rate of temperature rise could not be maintained. Initially, the desired heating rate of 5° per 15 minutes was achieved, but when the temperature had risen 100° F the rate of increase gradually decreased. After 30 hours of heating, the hot face temperature was 310° F and holding. Since the heating requirements had not been met, the wall was gradually cooled. Useful information was obtained for subsequent testing. Minor modifications in the test apparatus to obtain more flexibility and heating capacity were made.

IV. 5. 2. Test No. 1

The peak temperature (320° F) was reached on the hot face after 12 hours of heating as per schedule. Temperature on the cool face at this time was 87° F after an initial temperature of 71° F. Required temperatures of 320° F on the hot face and 70° F in the cool chamber were then maintained until a nearly steady gradient through the wall was reached after 58 hours of heating. At this time the differential across the wall was 100° F. Cooling was begun on the hot side at the rate of 5° per 15 minutes but it could only be maintained for 8 hours with lesser rates resulting thereafter. Testing was terminated after 54 hours of cooling; the hot face was at 88° and the cool face at 125° F. The fiberglas insulation on the cool face was removed at this time. The wall was completely cooled to the original temperatures within 48 hours. Thermocouple readings were taken at least every half hour during testing. Observations of the wall after testing revealed some predominantly vertical random surface cracks at the center portion of both faces. It appears that this cracking is due mainly to drying and shrinkage but also may be related to partial restraint of the wall. The average relative humidity in the wall during this test dropped from 40 to 20 percent. The compressive strength of the concrete at the beginning of this test was 7, 080 psi.

IV. 5. 3. Test No. 2

No insulation layer was attached to the cool face for this test. Again, the hot face was heated at the rate of 5° per 15 minutes until 320° F was reached in 12 hours; the cool face had risen only 11° to 81° F. The peak temperature (320° F) was held longer for this test to make certain that a steady temperature gradient had been attained. After heating for a total of 80 hours, the gradient through the wall was for all practical purposes, steady with the cool face at 140° F.

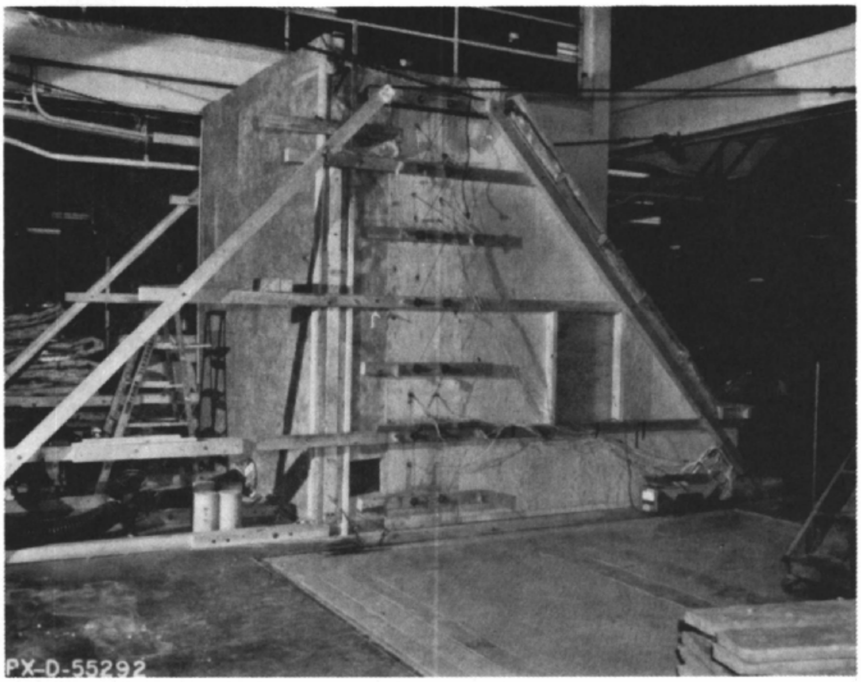


Figure IV-10. General view of the chambers surrounding the wall. Photo PX-D-55292

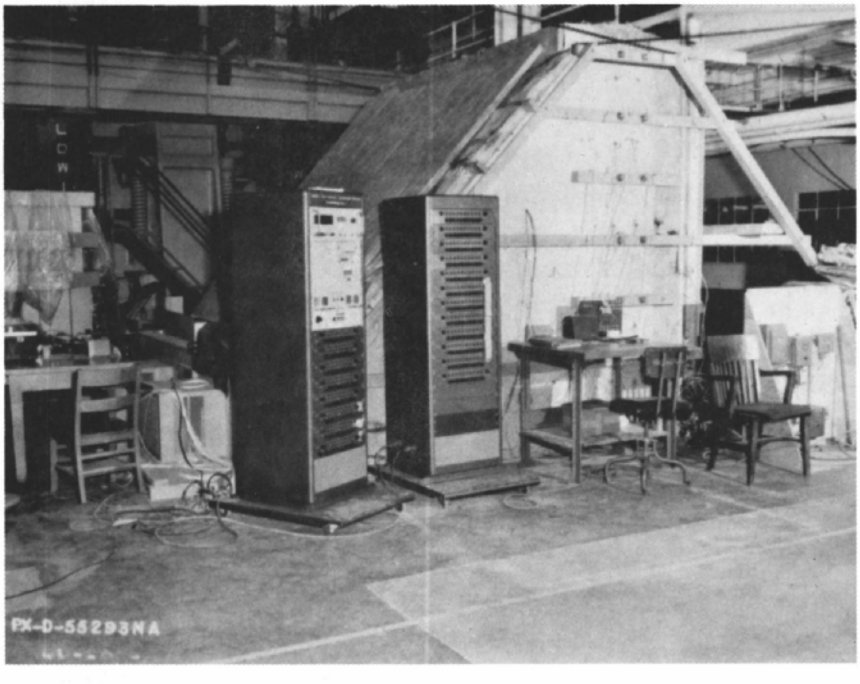
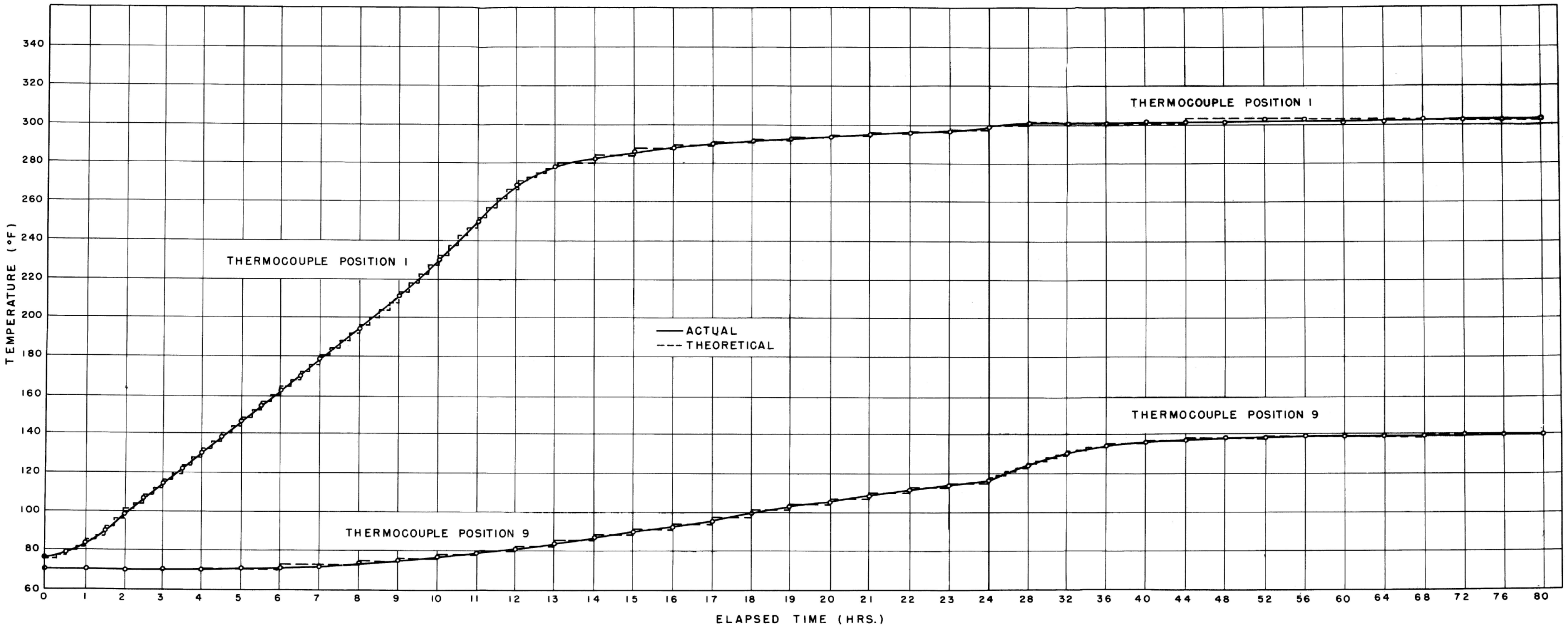
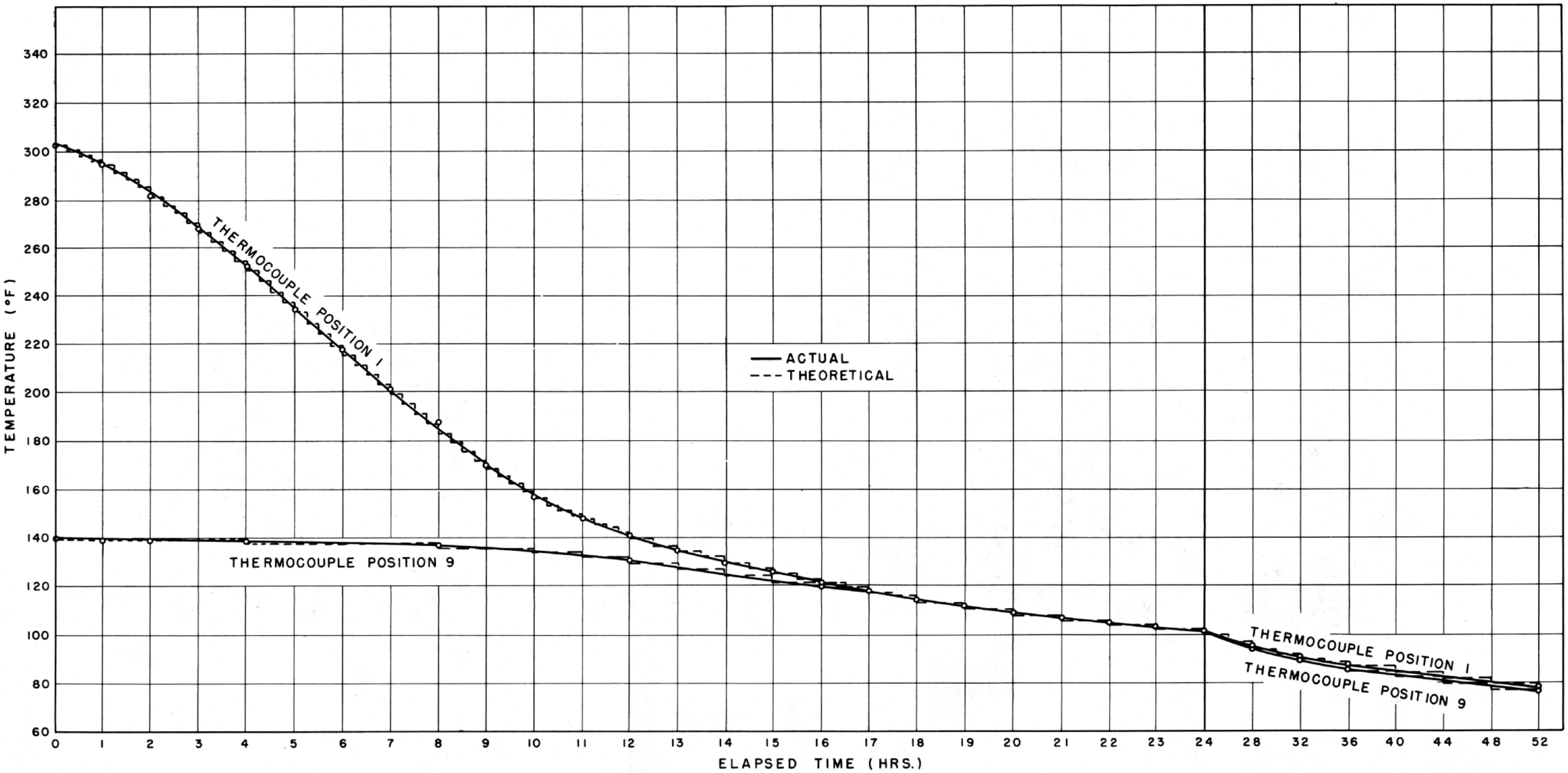


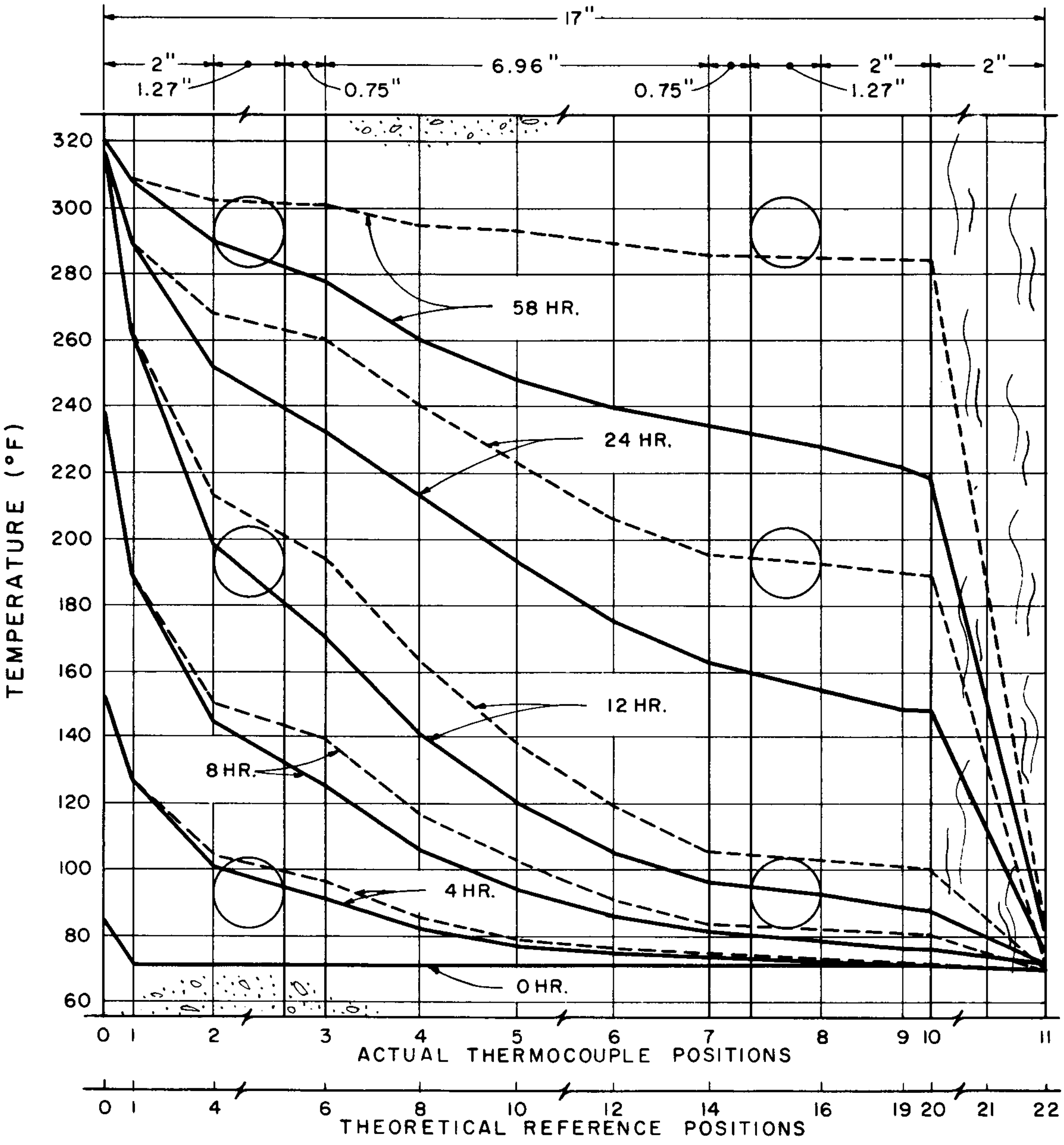
Figure IV-11. View showing the top and one end of the colling chamber. Photo PX-D-55293 NA



TEMPERATURE VERSUS TIME
REINFORCED CONCRETE WALL
TEST 2 -- HEATING PHASE
FIGURE IV -12



TEMPERATURE VERSUS TIME
REINFORCED CONCRETE WALL
TEST 2 -- COOLING PHASE
FIGURE IV -13



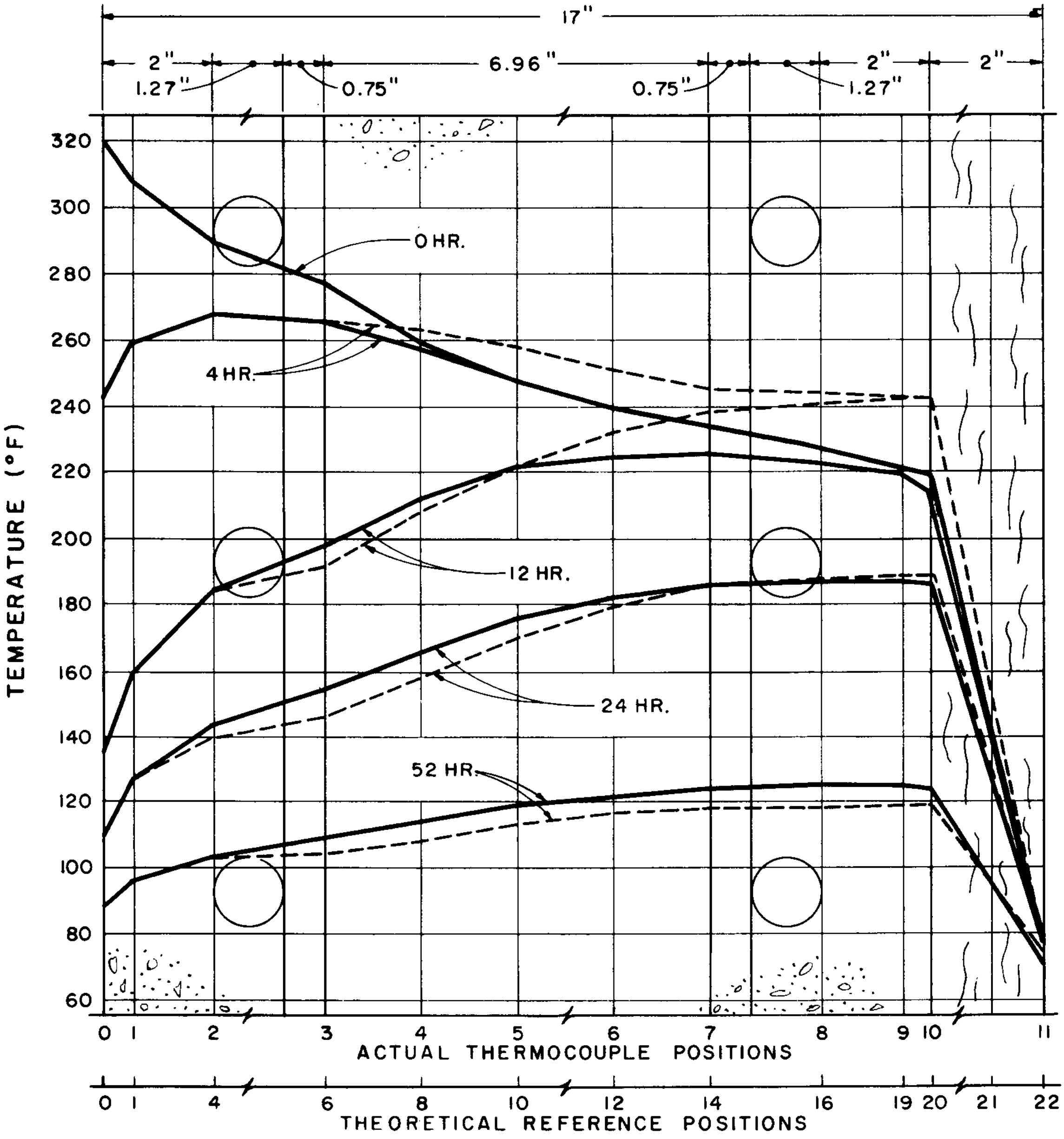
TEST NO. 1 (Actual and Theoretical)

HEATING

—— Actual ---Theoretical

TIME-DEPENDENT TEMPERATURE DISTRIBUTIONS
THRU REINFORCED CONCRETE WALL
W/2" FIBERGLASS INSUL.

FIGURE IV - 14



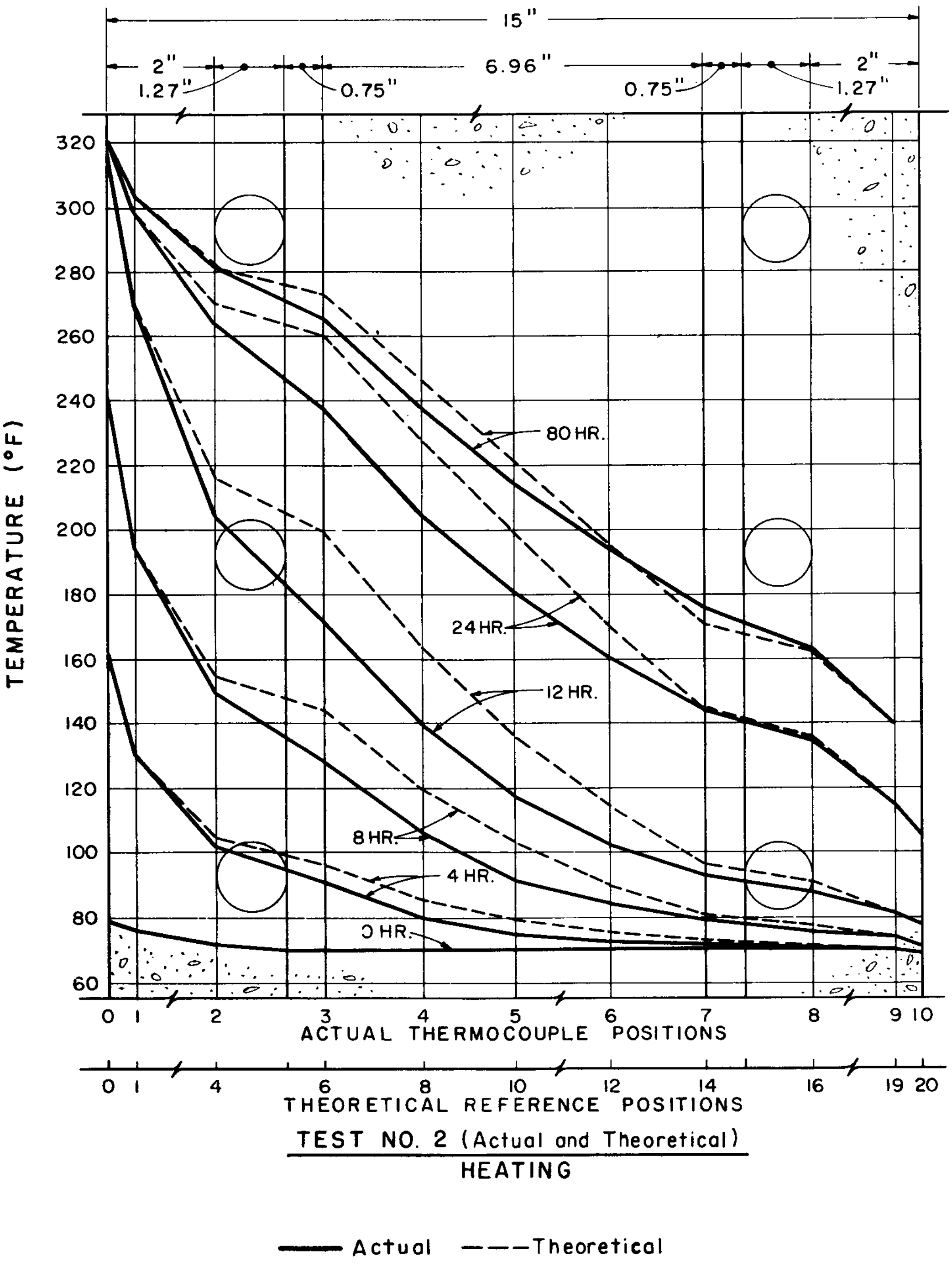
TEST NO. 1 (Actual and Theoretical)

COOLING

—— Actual ---Theoretical

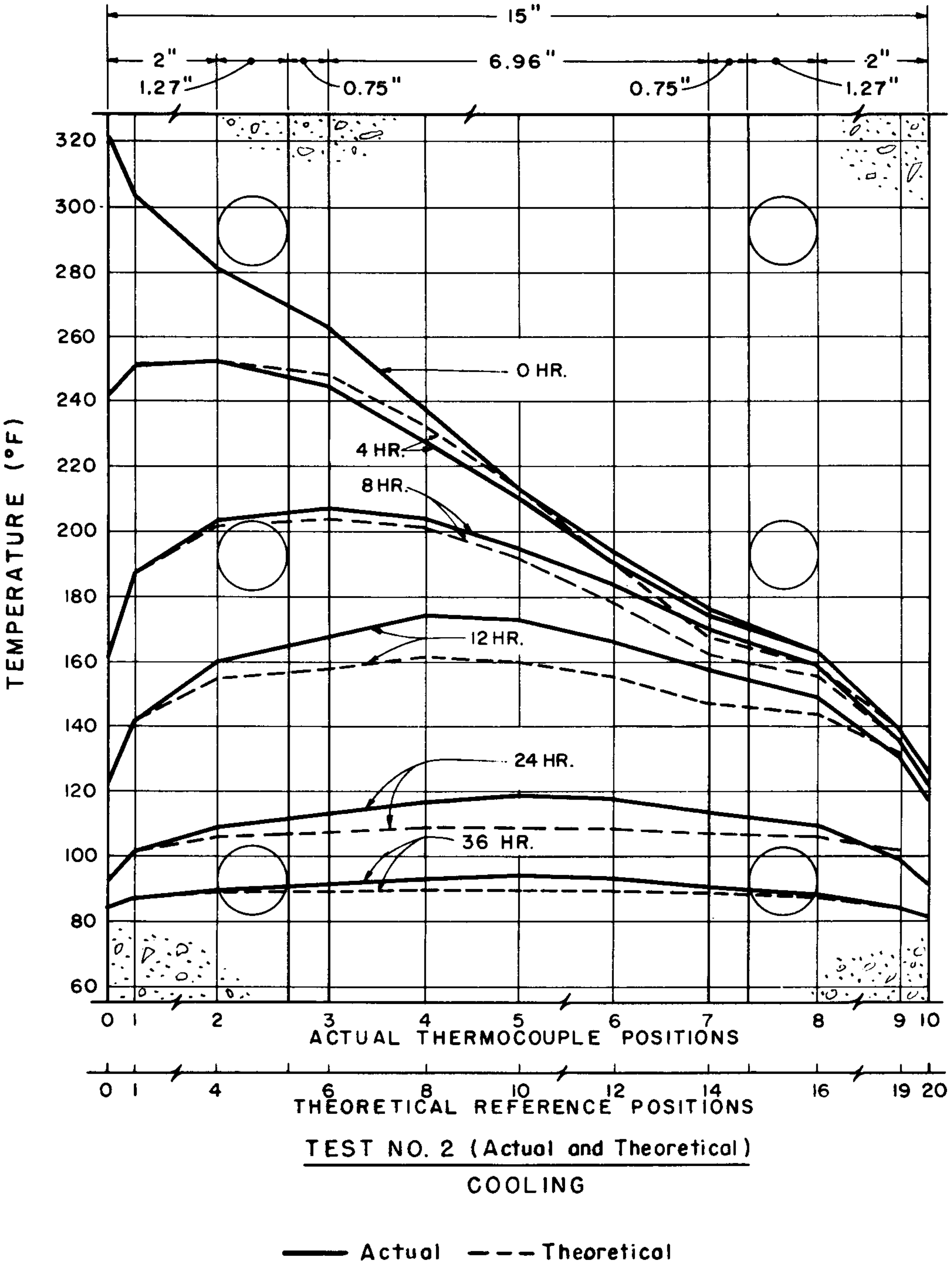
TIME-DEPENDENT TEMPERATURE DISTRIBUTIONS THRU REINFORCED CONCRETE WALL W/ 2" FIBERGLASS INSUL.

FIGURE IV - 15



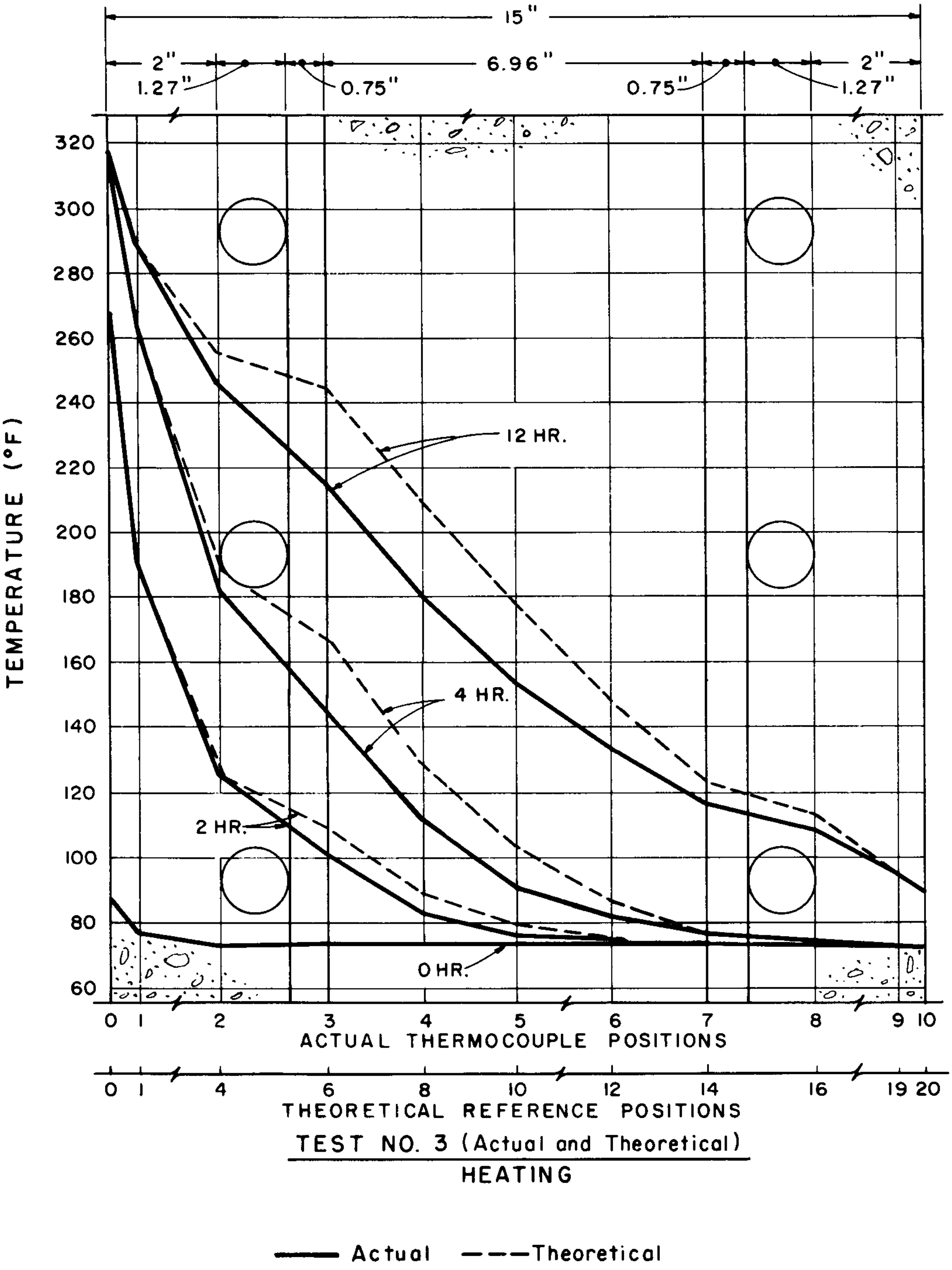
TIME-DEPENDENT TEMPERATURE DISTRIBUTIONS THRU REINFORCED CONCRETE WALL

FIGURE IV - 16



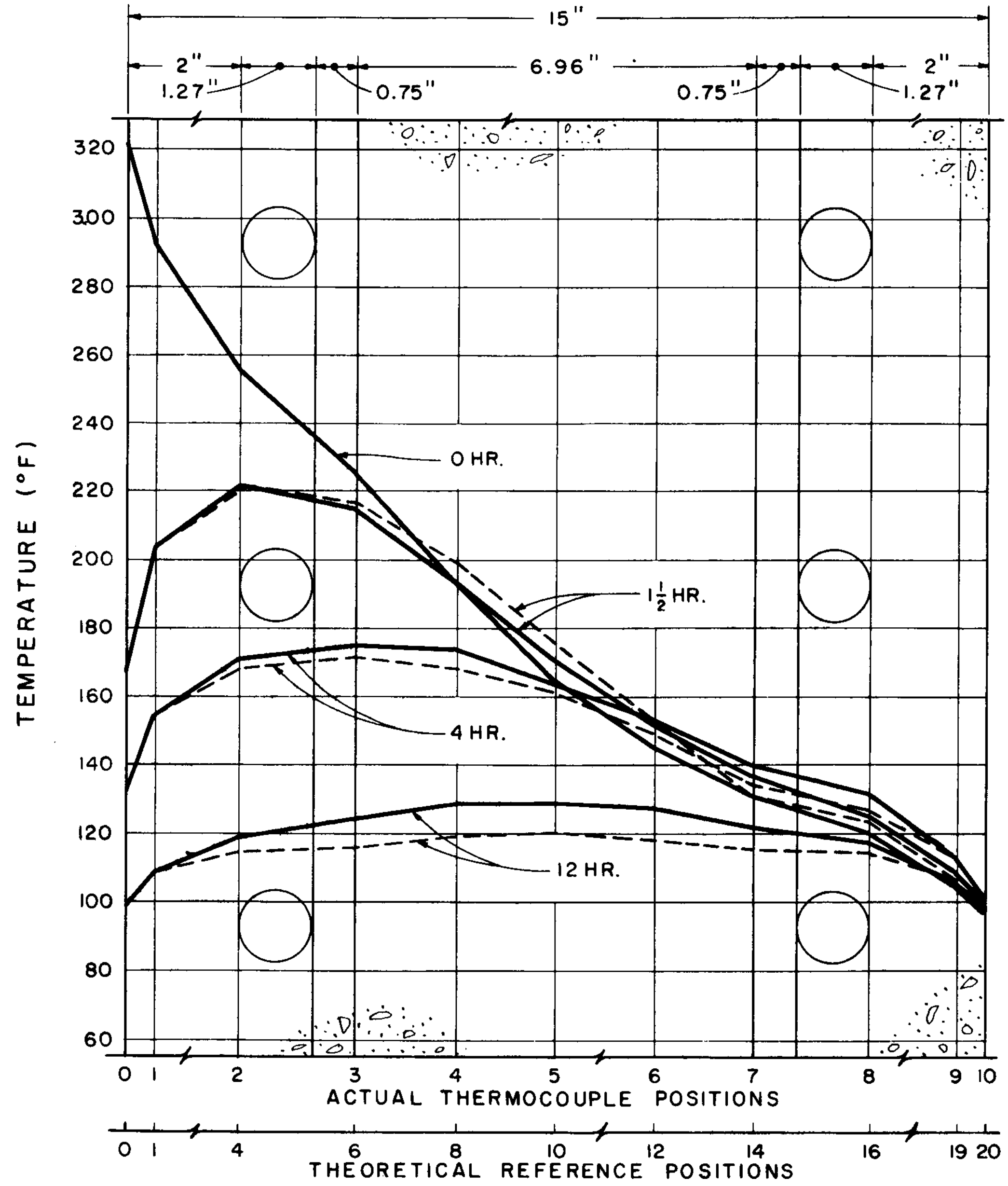
TIME-DEPENDENT TEMPERATURE DISTRIBUTIONS THRU REINFORCED CONCRETE WALL

FIGURE IV - 17



TIME-DEPENDENT TEMPERATURE DISTRIBUTIONS THRU REINFORCED CONCRETE WALL

FIGURE IV - 18



TEST NO. 3 (Actual and Theoretical)

COOLING

▬ Actual ---Theoretical

TIME-DEPENDENT TEMPERATURE DISTRIBUTIONS THRU REINFORCED CONCRETE WALL

FIGURE IV - 19

The cooling rate of 5° F per 15 minutes on the hot face was held for 9 hours with lesser rates resulting thereafter. After 38 hours of cooling, both faces were 80° F with a maximum of 92° F in the interior. All temperatures were 80° F or less after an additional 12 hours of cooling. Thermocouple readings were taken every half hour during testing. There was only a slight increase in surface cracking on the wall faces and the average relative humidity in the wall decreased from 20 to 16 percent during this test.

IV. 5. 4. Test No. 3

This test consisted of rapidly increasing the temperature on the hot face of the test panel with no insulation layer on the cool face. The required peak temperature of 320° F on the hot face was reached after 3-1/2 hours of heating. Twelve hours of heating produced a temperature gradient across the test panel which was above that of Test No. 2 for the corresponding time interval. Since this test provided sufficient data for the designers, heating was continued for only 3 more hours and then cooling was begun on the hot face. The cool face at this time had an average temperature of 98° F. Testing was terminated after 16 hours of cooling with an average temperature through the panel of 106° F. Temperatures were recorded every 7-1/2 minutes during the time intervals in which the temperature gradient was rapidly changing and every half hour otherwise. There was no important change in the surface cracks on the panel and no significant change in the relative humidity.

IV. 5. 5. Test No. 4

This test was identical to Test No. 3 except that the 2-inch insulation was to be placed on the cool face. Test No. 4 was not made because it was felt that the use of the insulation would not produce enough new data to justify the cost of running the test. It is theorized that the use of the insulation in the rapid heating and cooling cycle would only tend to flatten the gradient curves of the thermal distribution but would not speed up the heating cycle.

IV. 5. 6. Results

The amount of test data obtained was so voluminous that it would serve no purpose to include it in this report. A better interpretation of the test results can be obtained by careful study of the graphs included in the report.

IV. 6. Conclusions

Correlation between the actual test results and the theoretical values is good enough so that a time-dependent-temperature distribution of a reinforced concrete shell for use as an evaporator vessel can be calculated using the theoretical-numerical-analysis method.

IV. 7. Creep of Concrete at Elevated Temperatures

IV. 7. 1. Definitions

Because of the unusual conditions of exposure used in this investigation and their effects on the measured properties of concrete, it is believed advantageous to the reader to define the following terms as used in this report:

> Creep is the net time-dependent strain of loaded concrete under constant sustained stress; this strain taking place during a specific time interval after the initial strain due to load and following correction for movement of the unloaded control concrete subjected to the same ambient conditions during the same period of time. Creep is expressed hereafter in millionths inch per inch.
>
> Strain of loaded concrete is the gross actual measured deformation due to all causes (load, time, temperature change, drying, and possible chemical changes).

IV. 7. 2. Conclusions

1. Creep determined at ages up to 6 months varies directly with temperature and time, the relationship being approximately linear between 73° and 180° F. Beyond 180° F, the rate of increase in creep per degree F progressively diminishes, Figure IV-22.

2. Creep at the higher temperatures during the first 2 weeks is very rapid; but as the specimens become dry, the rate of creep is reduced rapidly, Figure IV-21.

3. Weight loss of the concrete varies directly with temperature and time, the rate of loss being greater during the first 10 days.

IV. 7. 3. Procedures

IV. 7. 3. 1. Specimens. --

Twenty-five cylinders 6 inches in diameter by 16 inches long were fabricated from one structural concrete mix for tests covered in this report. Mix data for the concrete are given in Table IV-1. After fabrication, all specimens remained in the molds for 24 hours at 73° F. They then were stripped, cured in a fog room at 73° F for 1 month, and dried at 73° F another month at 50 percent relative humidity.

During the third week of fog cure, all cylinders were removed from the fog room for about 1 hour while the ends were ground flat. They were then immediately replaced in the curing room. One day before elevated temperature tests, specimens were wrapped with fiberglas insulation. Purpose of this insulation was to minimize temperature changes in the concrete during readings when test chamber doors were open, Figure IV-24.

Test specimens were equipped with inserts on each of three gage lines, spaced at 120° intervals around the cylinders for reading strain measurements periodically with a Whittemore gage.

IV. 7. 3. 2. Tests. --

At 2 months' age, all creep specimens were put under 800-psi compression at 73° F in hydraulic loading frames. Immediately after the loading, temperature of the various chambers containing the cylinders was slowly raised to the test condition over a period of about 24 hours. Load on the creep specimens remained constant at 800 psi during the temperature change and during the subsequent 6-month test period. Tests were conducted at 73°, 130°, 180°, 230°, and 290° F. Stress was applied before raising temperatures so that conditions expected in an evaporator-type posttensioned desalting chamber would be met.

Strains were determined periodically on the loaded and unloaded concrete during 6 months of storage at the various elevated temperatures. Weight loss was determined only on the unloaded specimens through the 6 months' storage period. Weights of the loaded specimens were taken only before application of stress and again after unloading.

IV. 7. 4. Results and Discussion

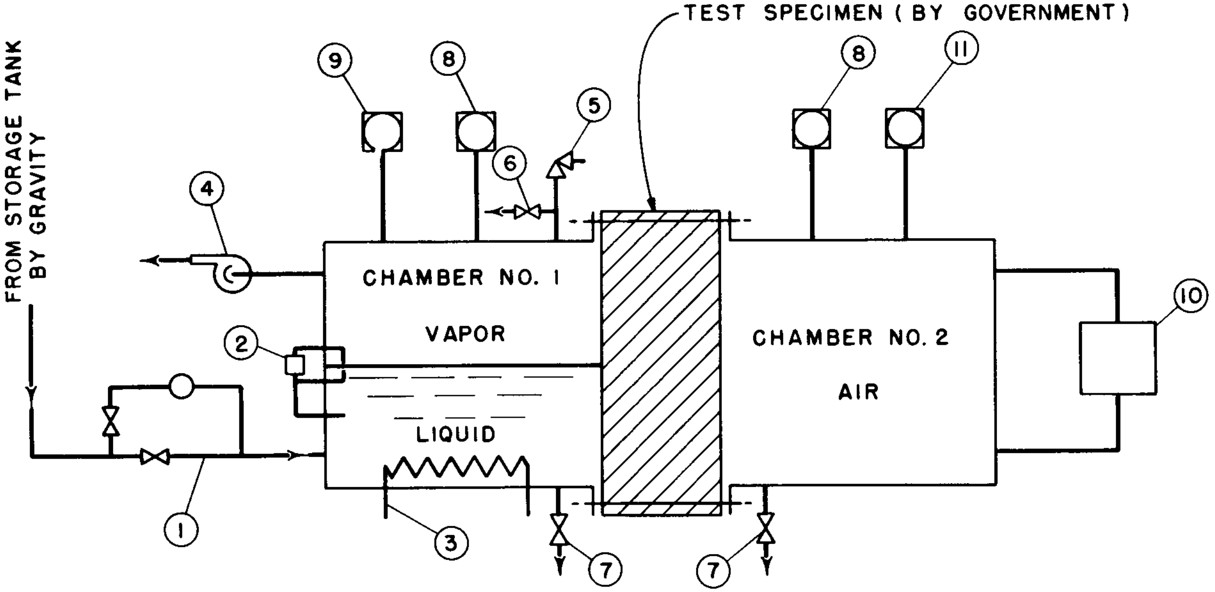
IV. 7. 4. 1. Creep. --

Creep strain is the time-dependent strain caused by the applied load. It takes place after the initial elastic movement during loading and is equal to the difference between the deformation of a loaded specimen and the deformation of an identical specimen that is not loaded, but is otherwise subjected to the same test environment.

Maximum creep of the structural concrete after 6 months' continuous 800-psi compressive stress at temperatures of 73°, 130°, 180°, 230°, and 290° F is 705 millionths and occurs at 290° F. This maximum at 6 months is five times the creep measured at 73° F on the same concrete. Six-month creep of the material at 130°, 180°, and 230° F is 2-2/3, 4, and 4-2/3 times, respectively, the creep at 73° F, Figure IV-21.

CHAMBER CONDITIONS

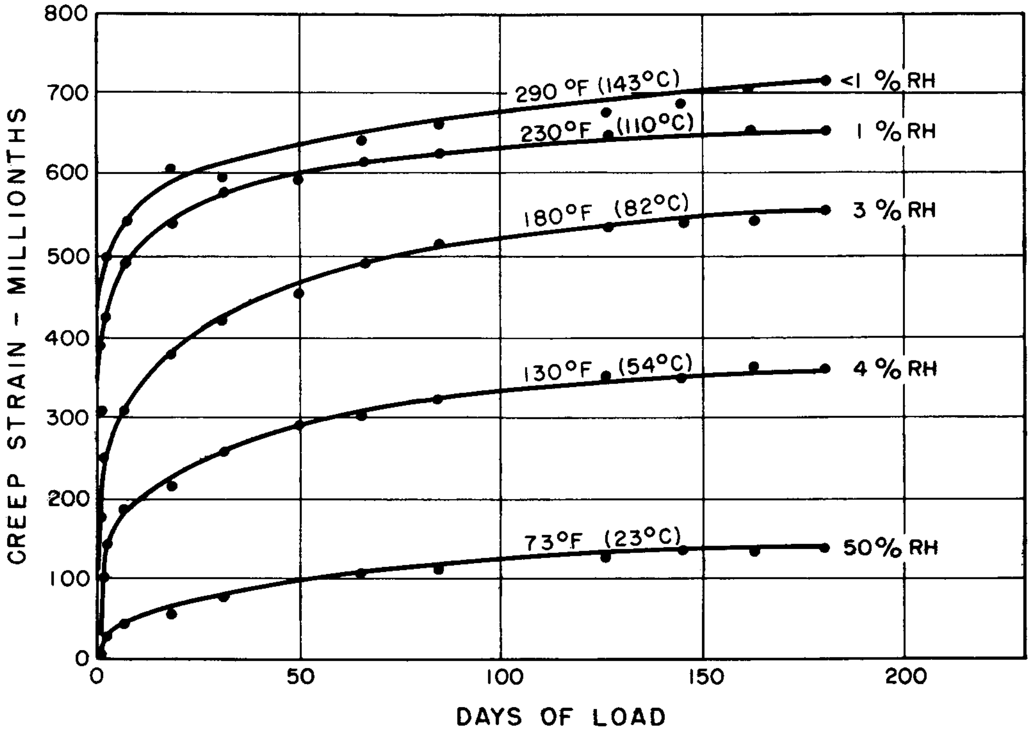
	CHAMBER NO. 1	CHAMBER NO. 2
ENVIRONMENT:	LIQUID AND SAT. VAPOR	AIR AT 20 % TO 95 % R.H
TEMPERATURE:	90° F. & 1 PSIA TO 350° F. & 135 PSIA	40° TO 150° F.
PRESSURE:		ATMOSPHERIC



FUNCTIONAL DIAGRAM OF COMPONENTS

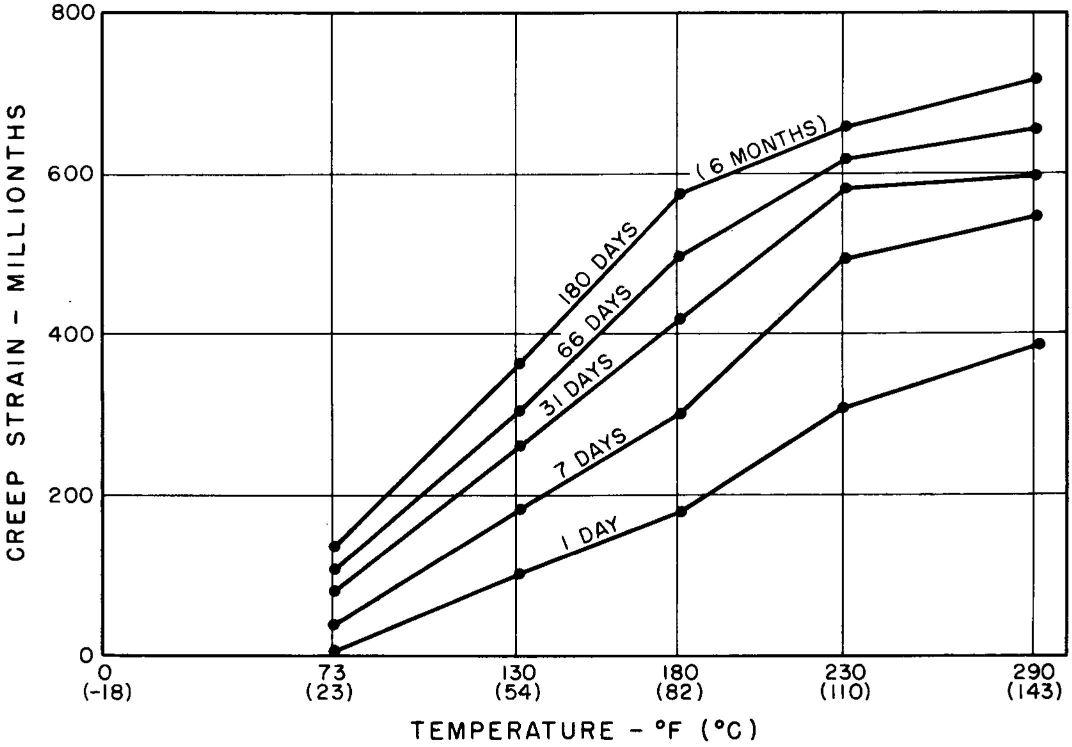
NO	FUNCTION	NO	FUNCTION
1	LIQUID FILLING AND MAKEUP	8	TEMPERATURE CONTROL, INDICATION AND RECORDING
2	LIQUID LEVEL CONTROL	9	PRESSURE CONTROL, INDICATION AND RECORDING
3	HEATING	10	AIR CONDITIONING SYSTEM
4	VACUUM	11	RELATIVE HUMIDITY CONTROL AND RECORDING
5	PRESSURE RELIEF		
6	BLOWDOWN		
7	DRAIN		

FIGURE IV-20 — ENVIRONMENTAL TEST UNIT



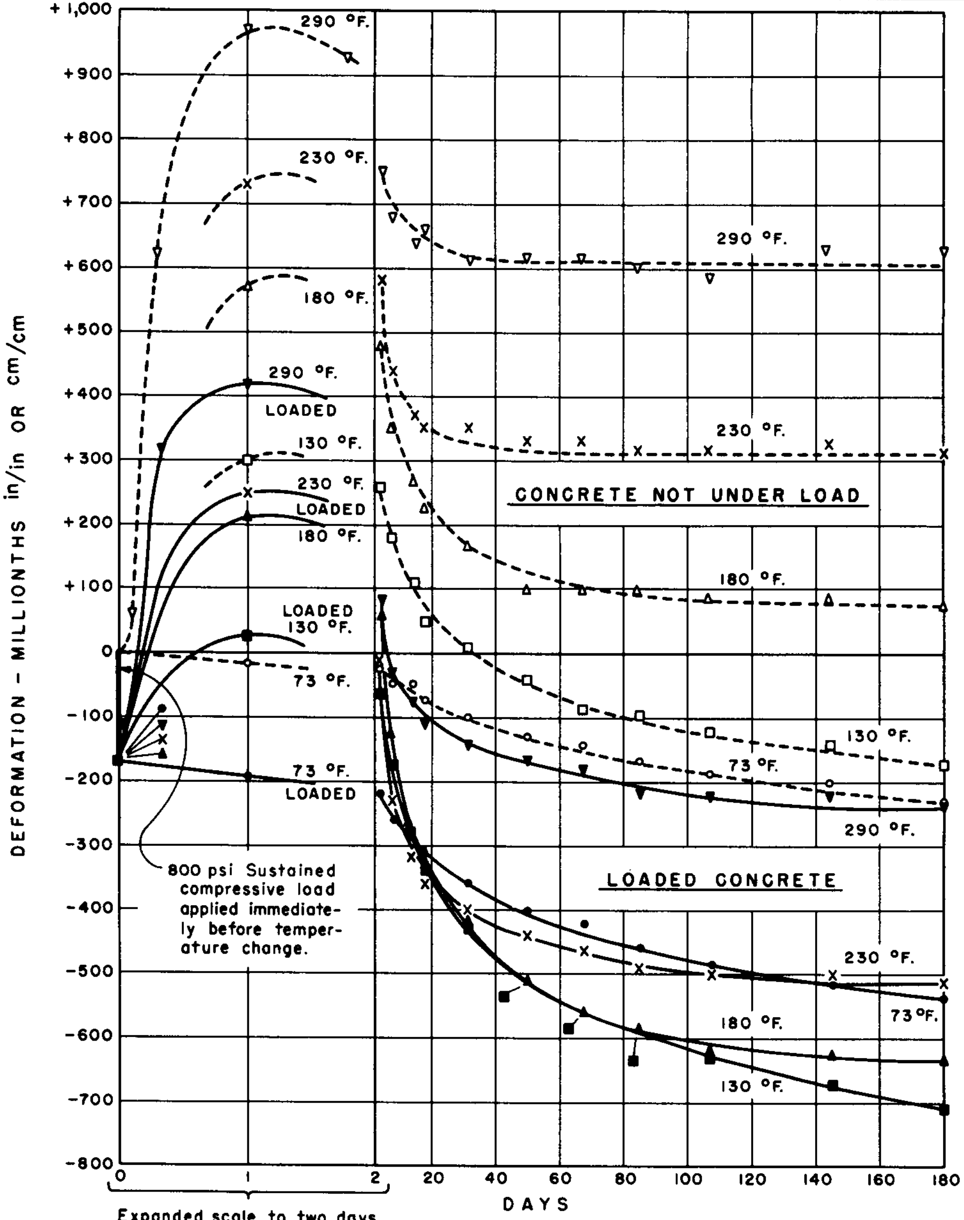
CREEP INCREASES WITH TEMPERATURES AND TIME

FIGURE IV -21



CREEP INCREASES WITH TEMPERATURES AND TIME

FIGURE IV -22



Expanded scale to two days.
Temperature increased from
73°F. to test temperature.

ACTUAL DEFORMATIONS OF SUSTAINED LOADED CONCRETE AND CONCRETE NOT LOADED

Maximum expansion occurs in the Unloaded Concrete during temperature rise to 290 °F.
Maximum six-month contraction occurs at 130 °F. in the Sustained Loaded Concrete.

FIGURE IV -23

NOTES

Specimens 6"x 16" unsealed cylinders with Whittemore strain gage inserts. All specimens cured one month 73 °F., 100 percent relative humidity and another month 73 °F. 50 percent relative humidity before sustained load test at 800 psi compression.

73 °F. IS 23 °C.
130 °F. IS 54 °C.
180 °F. IS 82 °C.
230 °F. IS 110 °C.
290 °F. IS 143 °C.

Table IV-1
MIX DATA
High Temperature Creep

Cement, lb per cu yd, Type V, M-5207	560
Pozzolan, lb per cu yd, fly ash, M-5152	140
*Coarse aggregate, lb per cu yd, 1-1/2 in. max, M-5036-1A	2,164
Sand, lb per cu yd, M-5036-1A	994
Water, lb per cu yd	262
Air, percent	4.9
Water reducing agent, cc per cu yd	606
Air entraining agent (5 percent sol NVX), cc per cu yd	179

*Good quality amphibole schist river aggregate.

As depicted in Figure IV-22, 6-month creep increased with increasing temperature from 73° to 290° F. The relationship between creep strain and temperature is linear from 73° to 180° F.

At the test temperatures of 230° and 290° F, creep rate during the first few days is very rapid compared with the lower temperatures, Figure IV-21. However, as time passes, the creep rate at these two highest temperatures reduces rapidly. This appears to be influenced by the rate of drying as well as the elevated temperatures. At 230° and 290° F, about 90 percent of the 6-month creep takes place during the first month, which is the period of most rapid drying. After drying is essentially complete (about 30 days), only a relatively small increase in creep results as time passes.

IV.7.4.2. Control cylinder deformation.--

Since load was applied to creep specimens before temperature increase to the various test conditions, the strain of the unloaded companion cylinders includes effects of drying-shrinkage, expansion during the temperature rise, and possible strains produced by any chemical changes in the material due to elevated temperatures.

Maximum strain of the unloaded concrete, which occurred during the first 1-1/2 days, was 970 millionths expansion and took place while raising the temperature from 73° to 290° F.

Temperature expansion apparently predominated during the early age movement, Figure IV-23, but the rate of drying-shrinkage was probably also maximum at this same time. The indicated strain is a combination of both of these phenomena and possibly others. After the initial expansion that takes place during the first 1-1/2 days, drying-shrinkage dominates movement of the unloaded concretes and at the two lower temperatures, the net length change after 6 months is shrinkage.

IV.7.4.3. Actual strain loaded concrete.--

Comparison of the total strains of the loaded concretes are shown in Figure IV-23. All specimens exhibit approximately the same initial strain due to application of 800-psi compressive stress at 73° F. Then during temperature rise, all loaded cylinders expand apparently at a greater rate than shrinkage or creep in all environments except ambient. Six-month storage of the specimens under sustained load at the elevated temperatures then causes a prolonged contraction of the concrete. Maximum total overall shortening of the cylinders from time of loading takes place at 130° F. Maximum shortening of the loaded material after temperature change, however, is 855 millionths at 180° F (from plus 220 millionths maximum expansion to minus 635 millionths 6-month contraction).

Increasing the temperature of loaded concrete at any particular constant moisture and stress condition will cause an increase in creep. At a constant temperature, drying of loaded concrete will cause shrinkage and also increase creep. In the case of creep at elevated temperature during drying, both temperature and rate of drying are affecting the amount of time-dependent strain to cause the 6-month movement.

At about 180° F, it appears that the temperature is sufficiently high to significantly increase creep but not high enough to rapidly dry the material so that the effects of these two phenomena combine to produce the maximum 6-month time-dependent deformation after temperature change at the 180° F temperature rather than at the higher 230° and 290° F temperatures.

IV.7.4.4. Weight loss.--

Weight loss during the 6-month period of elevated temperature storage is 3.5 percent at 290° F, 3.1 percent at 230° F, 2.7 percent at 180° F, 2.9 percent at 130° F, and 0.5 percent at 73° F, Table IV-2. At temperatures above the boiling point, nearly all of the weight loss occurs during the first few days of elevated temperature. The loss takes place more slowly at temperatures below boiling. Blowing air present in the 130° F test chamber (a walk-in calorimeter room with air circulation) is believed

Table IV-2

CREEP OF CONCRETE AT ELEVATED TEMPERATURES
Strains of Loaded and Nonloaded Concrete (Millionths in./in. or cm/cm)

Temperature, °F	Initial load at 73° F	Time under load (days)											
		1	3	7	14	18	31	50	66	85	107	144	180
		Length change of concrete under 800-psi sustained load											
290	-165	+415	+ 80	- 30	- 75	-105	-140	-170	-185	-220	-220	-215	-240
230	-170	+250*	- 20	-230	-320	-360	-400	-440	-460	-490	-500	-500	-510
180	-160	+220	+ 60	-125	-255	-305	-410	-510	-555	-580	-615	-620	-635
130	-175	+ 30	- 60	-175	-275	-340	-425	-505	-560	-590	-630	-670	-705
73	-170	-195	-220	-255	-280	-305	-355	-400	-420	-455	-480	-510	-540
		Length change of control cylinders not under load											
290	0	+970	+750	+680	+640	+660	+610	+620	+620	+600	+590	+630	+630
230	0	+730*	+580	+440	+370	+350	+350	+330	+330	+320	+320	+330	+320
180	0	+570	+480	+350	+270	+230	+170	+100	+100	+100	+ 90	+ 90	+ 80
130	0	+300	+260	+180	+110	+ 50	+ 10	- 40	- 80	- 90	-120	-140	-170
73	0	- 10	- 20	- 50	- 50	- 80	-100	-130	-140	-170	-190	-200	-230
	Creep strain plus initial deformation (difference between loaded and control):												
290	-165	-555	-670	-710	-715	-765	-750	-790	-805	-820	-810	-845	-870
230	-170	-480*	-600	-670	-690	-710	-750	-770	-790	-810	-820	-830	-830
180	-160	-350	-420	-475	-525	-535	-580	-610	-655	-680	-705	-710	-715
130	-175	-270	-320	-355	-385	-390	-435	-465	-480	-500	-510	-530	-535
73	-170	-185	-200	-205	-230	-225	-255	-270	-280	-285	-290	-310	-310
					Creep strain only								
290	-	-390	-505	-545	-550	-600	-585	-625	-640	-655	-645	-680	-705
230	-	-310	-430	-500	-520	-540	-580	-600	-620	-640	-650	-660	-660
180	-	-190	-260	-315	-365	-375	-420	-450	-495	-520	-545	-550	-555
130	-	- 95	-145	-180	-210	-215	-260	-290	-305	-325	-335	-355	-360
73	-	- 15	- 30	- 35	- 60	- 55	- 85	-100	-110	-115	-120	-140	-140
		Percent weight loss during elevated temperatures											
				6 days	10 days								
290	-	-	-	3.00	3.18	-	3.30	-	3.35	-	3.45	-	3.48
230	-	-	-	2.07	2.76	-	2.95	-	2.98	-	3.02	-	3.06
180	-	-	-	0.75	1.85	-	2.55	-	2.65	-	2.68	-	2.75
130	-	-	-	0.27	0.93	-	1.65	-	2.25	-	2.50	-	2.90
73	-	-	-	0.05	0.08	-	0.25	-	0.36	-	0.42	-	0.53

*Temperature of 230° F chamber only up to 175° F.

All specimens cured 1 month at 73.4° F (23° C) and 100-percent relative humidity, and another month at 73.4° F and 50-percent relative humidity before subjection to elevated temperatures.

Concrete at 73° F when load applied.
Temperature raised from 73° to test temperature in 24 hours after initial load.

to be the cause of the 6-month weight loss at 130° F, somewhat exceeding the loss at 180° F. Other test chambers were ovens with only little air movement. Loaded specimens lost nearly the same weight percentages as the unloaded cylinders during 6-month elevated temperature storage.

Average weight loss for all the specimens during the 1-month curing period at 50 percent relative humidity and 73° F (before elevated temperature storage) was 1.2 percent. Therefore, total weight loss for the concretes is the summation of the previously mentioned losses and 1.2 percent. Total losses are 4.7, 4.3, 3.9, 4.1, and 1.7 percent for the 290°, 230°, 180°, 130°, and 73° F concretes, respectively.

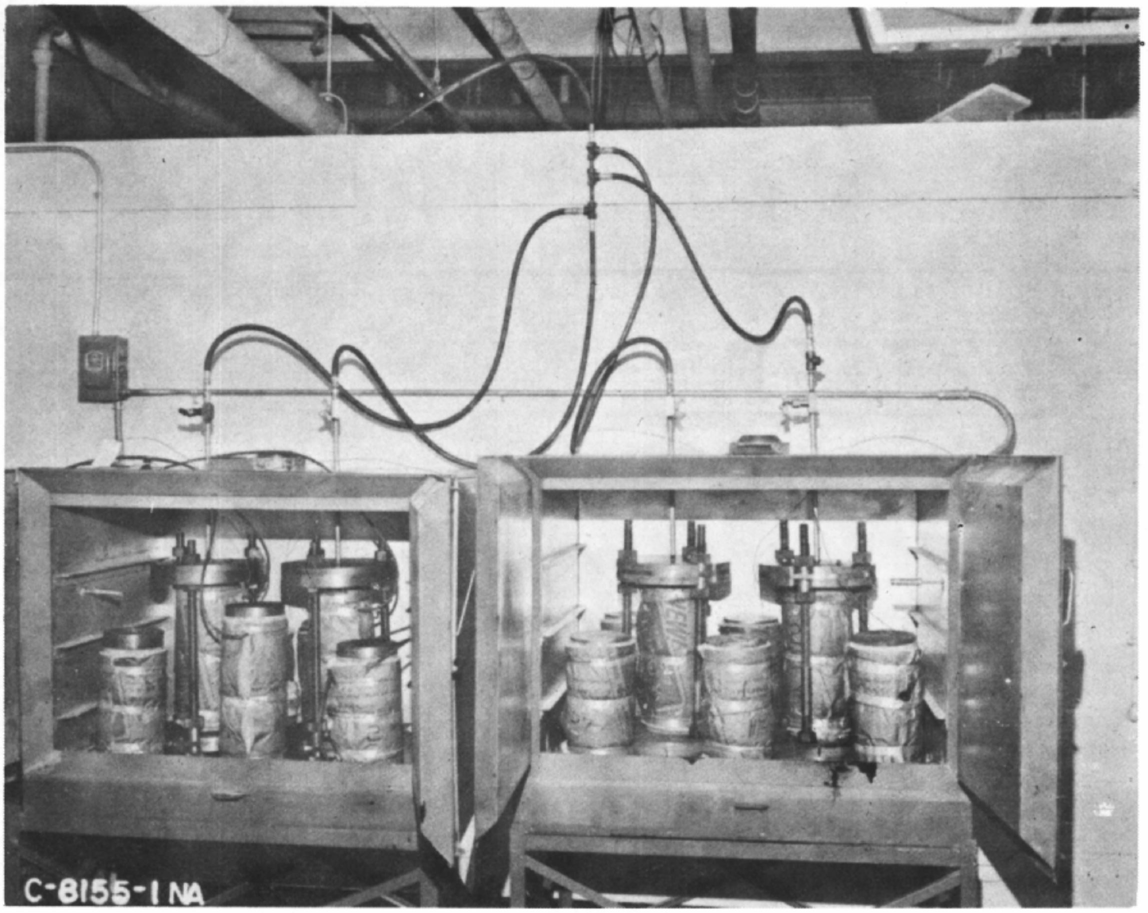


Figure IV-24. Typical insulated creep specimens in test chambers. Unloaded cylinders are controls for creep studies, and specimens for strength and elasticity determinations. (Insulation reduces temperature changes in the concrete during readings when chamber doors are open and during load investigations in testing machines.) PX-D-59940

IV.8. References

IV.8.1. Survey of Literature

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SECTION V

PART III--SELECTION AND EVALUATION OF ACCESSORY COATINGS, SEALANTS, AND POLYMERIC PRODUCTS

V.1. Introduction

Due to the fact that there are little published data and knowledge on the performance of polymeric materials at the proposed operating conditions and environments in desalination plants, an extensive research program to evaluate performance was established. Protective coatings, bonded liners, joint sealants, waterstop materials, composite joint sealing systems, and insulation materials were to be included in the program. The performance of these materials initially in relation to concrete and steel construction is of prime concern. Materials and/or methods of construction, such as plastic sandwich, honeycomb core panels, and others would be considered as materials became available and time permitted.

A variety of chemical-base materials may be needed throughout the various parts of the structure, such as protective linings and coatings, sealants, gaskets, or structural components. A material suitable for the lower temperature and pressure exposure may not be adequate for higher temperature exposure. On the other hand, the high temperature coatings may be more costly than those suitable for lesser temperatures. Economic feasibility as well as performance of these materials under the various conditions ultimately will be determined.

V.2. Technical Approach

Performances of these materials are determined by visual examinations and measurements after exposure to the test environments, which simulate actual exposure environments, to determine significant changes in appearance and in physical properties. When materials that will withstand the environment are found by these tests, performance evaluation under actual operating conditions should follow.

Important aspects include:

1. Condition and preparation of base specimens to eliminate variables related to differences in surfaces.
2. Correct preparation of the test materials.
3. Application as directed by the material manufacturer, or in certain instances by a technique selected by the laboratory.
4. Proper cure.
5. Precise initial measurements.
6. Frequent evaluation and measurement during the exposure period.
7. Final evaluation and extrapolation of data to predict long-term performance.

The test environments are as follows:

1. Preliminary tests.--
    a. A-1.--In brine in autoclave at 350° F.
    b. A-2.--In water in autoclave at 350° F.
    c. L-T.--In test tank on 100° F loop.
    d. H-T.--In test tank on 290° F loop.
2. Current brine tests.--
    a. 100° F, 5X concentration.

b. 203° F, 2.1X concentration.

c. 225° F, 2.1X concentration.

d. 250° F, 2.1X concentration.

e. 290° F, 1.15X concentration.

The brine exposures are conducted in special bypass loops in each main test loop as described elsewhere in this report.

3. Autoclave tests.--

a. 100° F, 0.95 psia.

b. 203° F, 12.25 psia.

c. 225° F, 18.91 psia.

d. 250° F, 29.8 psia.

e. 290° F, 57.5 psia.

The autoclave tests are complementary to the brine tests to provide performance data in steam and distilled water.

V.3. Preliminary Tests

Nine test coatings were applied to mortar and steel specimens for preliminary evaluation at 350° F. The base specimens were cleaned prior to application of the test materials by sandblasting. All test coatings were applied and cured according to the manufacturer's instructions. Seven samples of joint sealing materials were applied to small, clean mortar blocks and cured as required. Small pieces of rubber were cut from one type of rubber for preliminary testing also. Several other types of rubber were added to the preliminary tests at a later date and some were then carried over to the current tests. Selections of the test materials were based on manufacturer's recommendations and limited knowledge of performance at high temperatures in other environments.

Basic types of coatings were an epoxy primer with acrylic topcoats, an inorganic zinc silicate, an inorganic silicate sealer, an epoxy primer and enamel system, a silicone alkyd, an epoxy with heat post cure, and a heat-cured epoxy. Four types of RTV silicone joint sealers, one coal-tar polysulfide, and two types of polysulfide joint sealers, and vinylidene fluoride hexafluoropropylene rubber were the other test materials.

The test specimens were exposed in hot brine in an autoclave at 350° F for periods ranging from 1 week to 2 months. Evaluation was by visual examination at specific intervals.

The materials tested initially in the preliminary program are listed in Table V-1.

Most of the materials showed early deterioration; however, after 2 months' exposure, the epoxy with heat post cure and the heat-cured epoxy, and after 1 month's exposure the silicone alkyd showed little, if any, effects from the exposure. On the basis of these brief results, these three materials were selected for further tests on concrete bars in the test loops. Woven glass cloth reinforcement was added to the post-cured epoxy on the concrete bars. The bars coated with the reinforced, post-cured epoxy system are shown in Figure V-1; the bars coated with the heat-cured epoxy system are shown in Figure V-2; and the bars coated with the silicone alkyd are shown in Figure V-3.

Through 1-year exposure in the test loops at both 100° and 290° F, all coatings on the test bars and on the pipe were showing progressive damage, although all were essentially intact. Both epoxies were hard and brittle with cracking and disbonding. The silicone alkyd showed evidence of broken blisters and edge damage. Performance of the coated bars is discussed in Section III.6.2.2 and plotted in Figures III-26, III-27, III-28, III-34 III-35, and III-36. Evaluation of the coatings was discontinued after the first year.

Table V-1

PRELIMINARY TEST MATERIALS

No.	Material	Type	Exposure[1]				Remarks
			A-1	A-2	L-T	H-T	
I	Acrylic, with epoxy primer	Coating	X				Fail, 1 week, disintegration
II	Inorganic zinc silicate	Coating	X				Fail, 2 weeks, disintegration
	Epoxy	Coating	X				Fail, 1 week, disintegration
III	Inorganic silicate sealer	Coating	X				Fail, 6 weeks, disintegration
IV	Silicone alkyd	Coating	X				Selected as test coating on bars and as pipe lining--Fail
V	Epoxy	Coating	X				Selected as test coating on bars and as pipe lining--Fail
VI	Epoxy (heat cured)	Coating	X				Selected as test coating on bars and as pipe lining--Fail
VII	Neoprene	Coating		X		X	Coating thin--blisters (Material carried over to expanded tests)
VIII	Neoprene	Coating		X		X	Coating thin (Material carried over to expanded tests)
IX	Silicone	Joint sealer	X				Fail, 12 days, very soft and friable
X	Silicone	Joint sealer	X				Fail, 12 days, very soft and friable
XI	Silicone	Joint sealer	X				Fail, 1 month, disintegrated
XII	Silicone	Joint sealer	X				Fail, 1 month, very soft and friable
XIII	Polysulfide rubber with Buna N topcoat	Joint sealer	X				Fail, 1 month, blown, charred and friable
XIV	Hi-temp polysulfide rubber	Joint sealer	X				Fail, 1 month, blown, charred and friable
XV	Coal-tar polysulfide	Joint sealer		X			Fail, 6 weeks, charred
XVI	Polyvinyl chloride	Plastic waterstop		X			Fail, 1 week (complete disintegration)--2 types
XVII	Hexafluoropropylene-vinylidene fluoride	Rubber	X		X	X	Hardening after hi-temp exposures
XVIII	EPT (ethylene-propylene-terpolymer)	Rubber		X	X	X	OK after 10 months (carried over to expanded test)
XIX	Polyacrylate	Rubber		X		X	Hard and brittle after 3 months (carried over to expanded test)
XX	Ethylene-neoprene blend	Rubber		X	X	X	Removed from test after 9 months
XXI	Chlorobutyl	Rubber		X	X	X	Considerable swelling (carried over to expanded test)
XXII	Butyl	Rubber		X	X	X	Softened, 3 months--removed from test
XXIII	Butyl	Rubber		X	X	X	

[1] A-1: autoclave with brine at 350° F.
A-2: autoclave with water at 350° F.
L-T: 100° F test tank on loop.
H-T: 290° F test tank on loop.

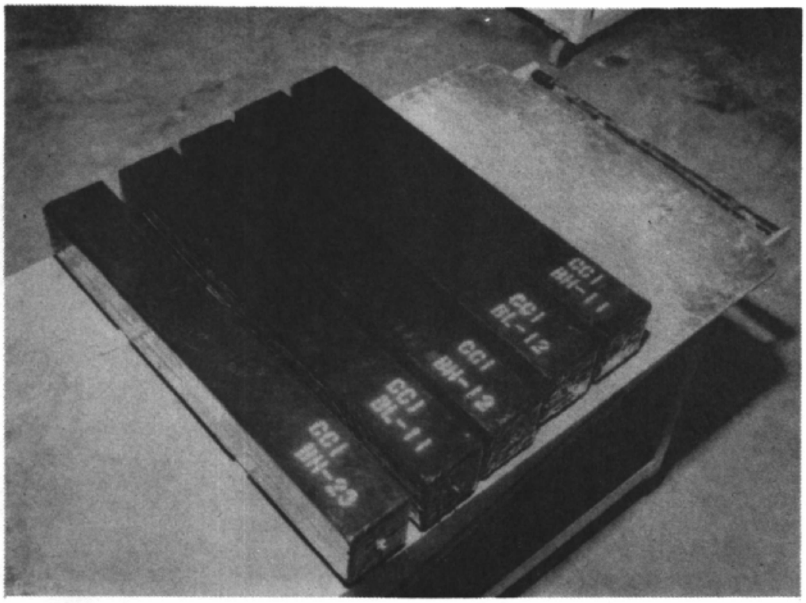


Figure V-1. Concrete test bars coated with a glass cloth reinforced epoxy system, Material No. V, are shown before exposure. Photo PX-D-55275 NA

Figure V-2. Concrete test bars coated with a heat-cured epoxy system, Material No. VI, are shown before exposure. Photo PX-D-55272 NA

Figure V-3. Concrete test bars coated with silicone alkyd coating, Material No. IV, are shown before exposure. Photo PX-D-55273 NA

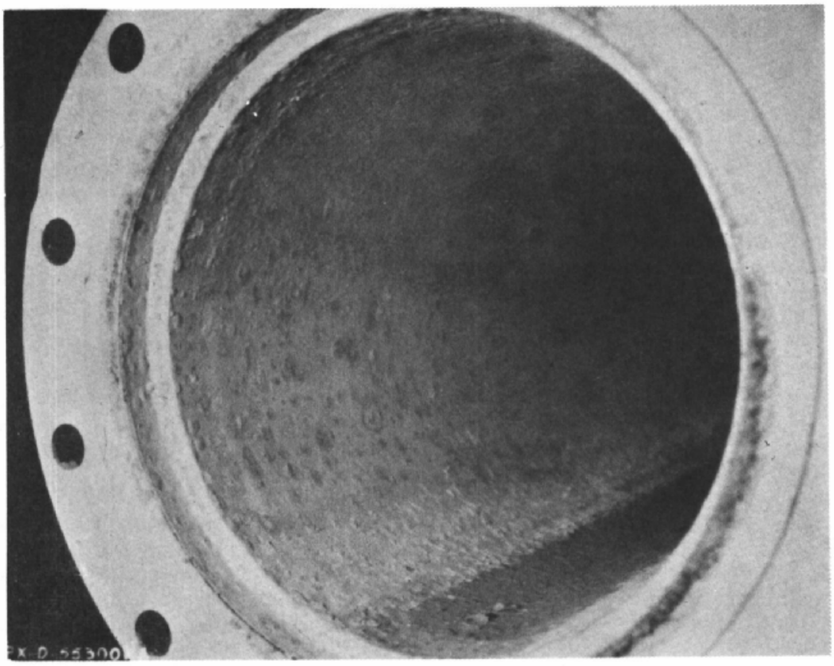


Figure V-4. Silicone alkyd pipe lining, Material No. IV, after 28 days' service at 100° F is shown. Blisters did not increase appreciably in size or number through 1-year exposure. Photo PX-D-55300 NA

The silicone alkyd pipe lining showed blisters at 28 days' exposure, Figure V-4, with slight progression through 1 year. After 1 year the pipe lining apparently was protecting the pipe from corrosion, although there was considerable evidence of deterioration. Evaluation of this test lining was likewise discontinued after the first year.

The joint sealer materials tested in the preliminary program are shown in Figures V-5, V-6, and V-7. Typical coating failures on steel and concrete specimens are shown in Figure V-8.

V. 3. 1. Conclusions from Preliminary Tests

Significant positive results are not available from the preliminary program, as all promising materials have been carried over to the current tests for further evaluation, as shown in Table V-1. However, some conclusions may be drawn on the brief tests conducted under the preliminary program. These are:

a. Coating materials listed in Table V-1 are not suitable for extended 350° F wet exposure.

b. RTV silicone and high-temperature polysulfides of the types tested are not suitable in 350° F wet exposure as joint sealants.

c. High-temperature butyl rubber and ethylene-propylene terpolymer rubber (EPT) are showing good promise in 350° F wet exposure.

d. The three test coatings applied to concrete bars did not offer adequate protection to the concrete because of the occurrence of discontinuities and other deficiencies in the test coatings themselves.

Preliminary tests have been discontinued, and further reporting under this title will not be made. All materials carried over to current tests are identified by new marks as indicated in the various tables.

V. 4. Current Tests

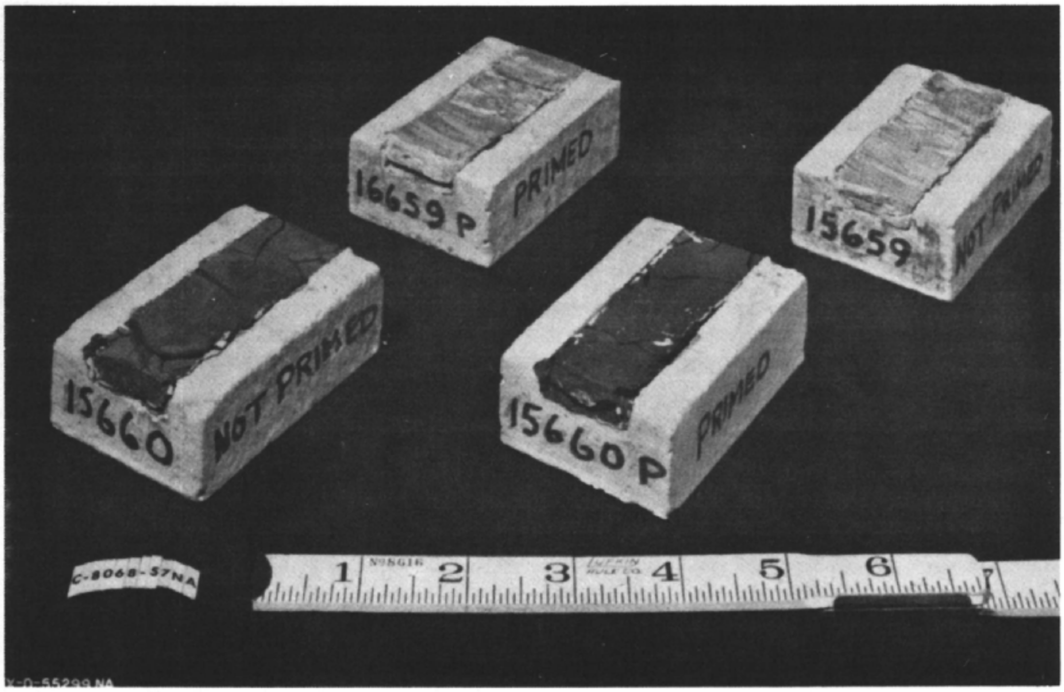
Originally, expanded tests on coatings, sealers, and polymeric products were to be conducted at temperatures ranging from 100° to 350° F. Hot brine and autoclave exposures (for steam and distilled water environments) were also anticipated. A variety of materials were obtained for testing under these conditions. However, after conferences with personnel from OSW's Distillation Division, the test conditions were modified to range from 100° to 290° F, as listed in Section V. 2. It was felt that more useful information would be obtained from tests conducted within the recommended range. After the lower maximum test temperature was established, test materials were requested for service only up to that point.

V. 4. 1. Materials

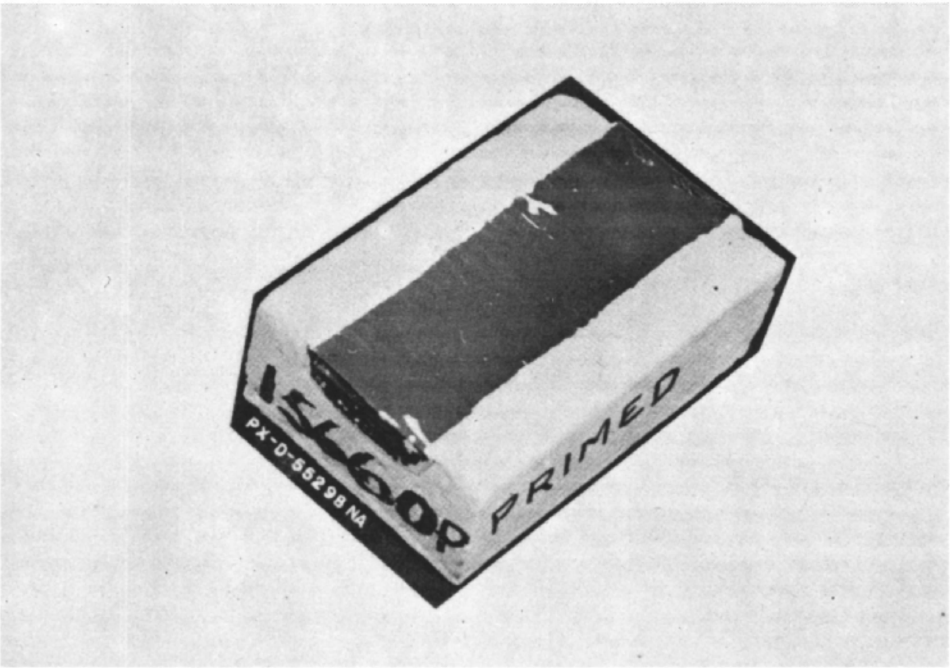
Protective coatings currently under evaluation are listed in Table V-2. Coatings numbered from C-39 have been added to the program since the first annual report, General Report No. 37, was issued. C-25 is no longer listed since it is the major constituent of C-26 and was not applied as a separate material.

Table V-3 lists joint sealer materials with J-6 the only addition since the previous report. Table V-4 lists plastic materials with P-5, P-6, and P-7 as new additions, and P-2 and P-4 deleted. P-2 actually is combined with the old P-1 as a new polyarylether formulation now designated P-1. P-4, chlorinated polyether, was dropped when additional test samples were not furnished when needed.

Rubber materials under study are listed in Table V-5. Rubber materials numbered from R-17 have been added to the program since the previous report. Rubber Samples No. R-1, R-2, R-3, R-4, R-12, R-13, and R-14 had preliminary tests only as the manufacturers declined to furnish additional samples for long-term tests. Preliminary tests only were run on Samples No. R-8 and R-10 also, but similar materials were furnished for long-term tests and were assigned new numbers.

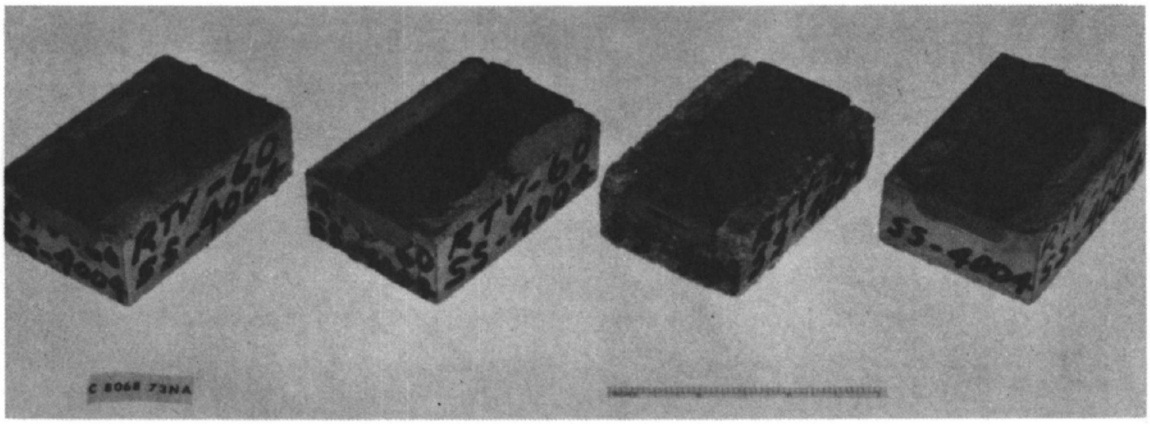


RTV silicone joint sealers, Material No. IX at top, Material No. X at bottom, after 12 days in 350° F brine are shown. Photo PX-D-55299 NA

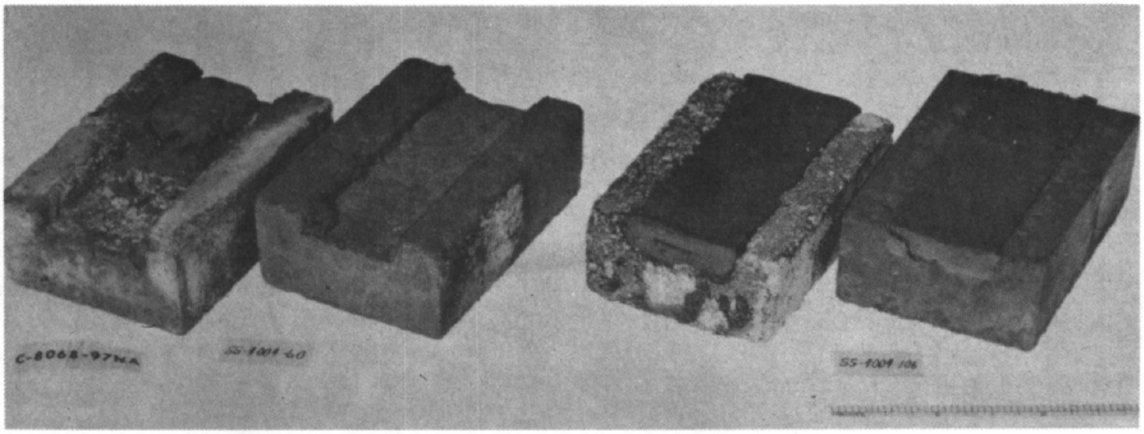


Typical specimen before exposure is shown in insert. Photo PX-D-55298 NA

Figure V-5.

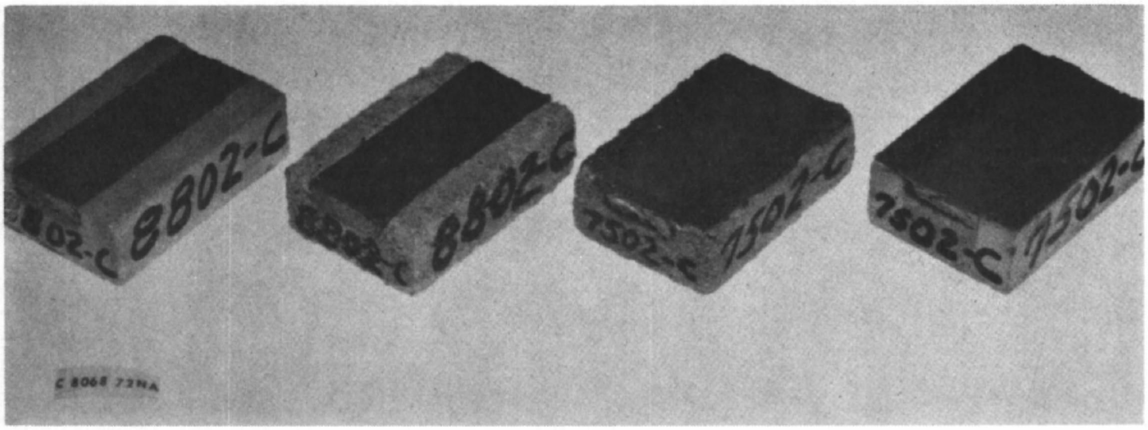


A. Before exposure.

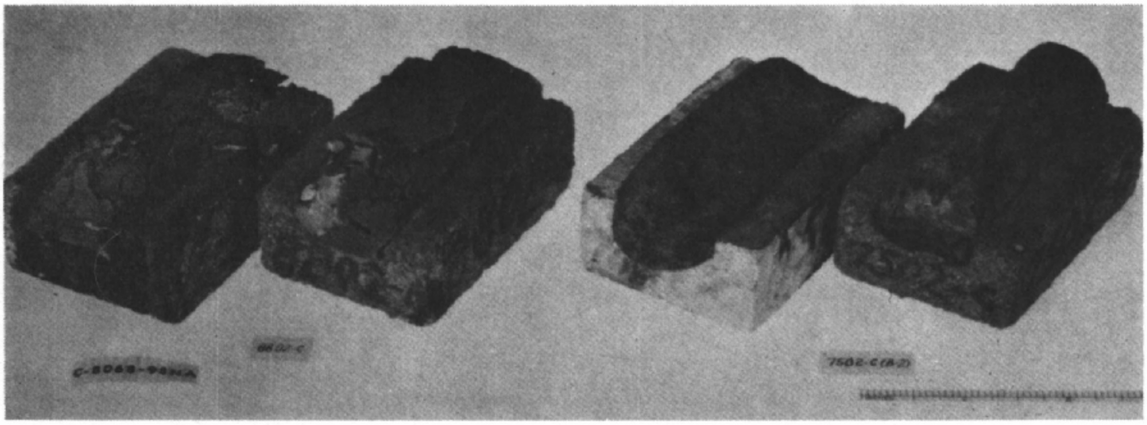


B. After 1 month in 350° F brine.

Figure V-6. RTV Silicone Joint Sealers Material No. XI at left, Material No. XII at right. Photos (top) PX-D-55270 NA and (bottom) PX-D-55271 NA



A. Before exposure.



B. After 1 month in 350° F brine.

Figure V-7. Hi-temp Polysulfide Joint Sealers Material No. XIII at right, Material No. XIV at left. Photos (top) PX-D-55268 NA and (bottom) PX-D-55269 NA

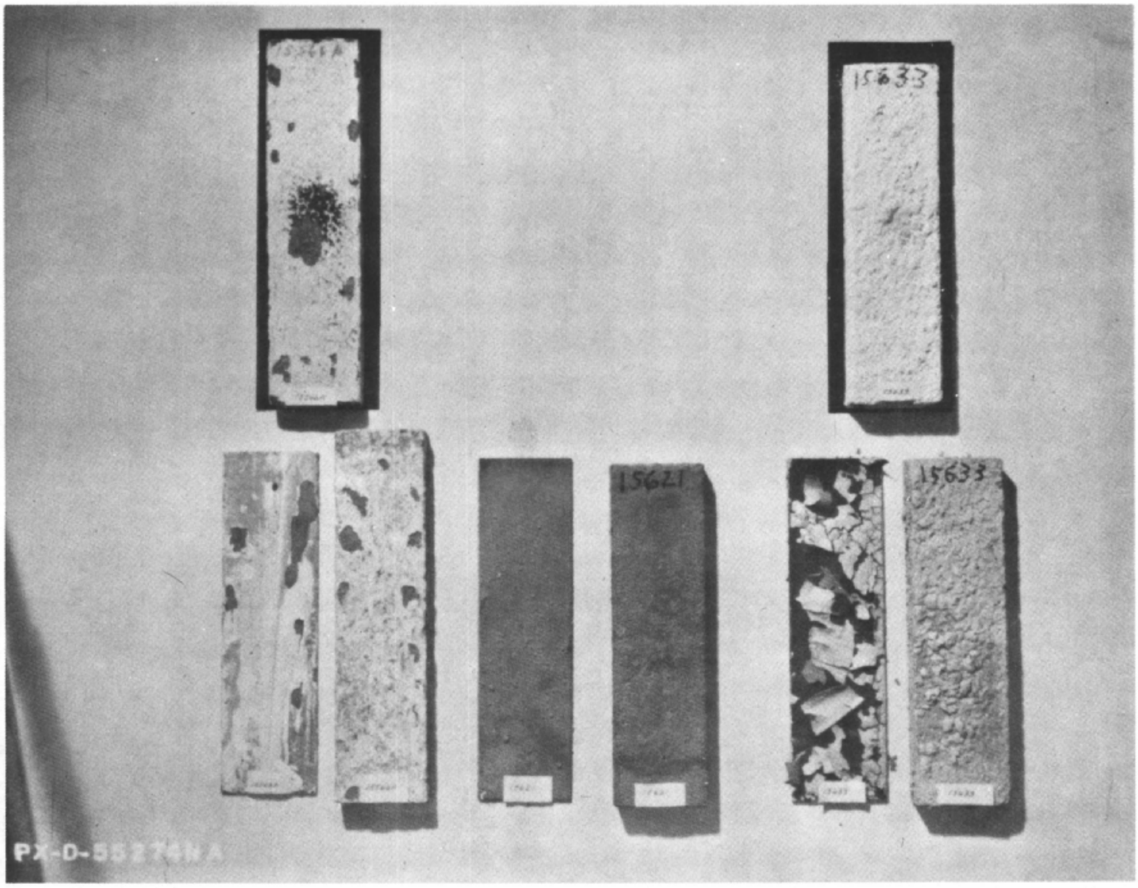


Figure V-8. Typical coating failures at termination of preliminary tests in 350° F autoclave are shown. Companion steel and mortar specimens are shown in the bottom row. Steel specimens are at the left in each set. The two top specimens were exposed to distillate drippings. Material No. I is at right, Material No. II is in the center, and Material No. IV is at the left. Photo PX-D-55274 NA

Table V-2

PROTECTIVE COATINGS UNDER TEST

No.	Basic type of material	Base specimens[1/]		Max test temp. °F	Remarks
		Concrete	Steel		
C-1	Neoprene	X	X	290	
C-2	Epoxy-phenolic	X	X	290	
C-3	Epoxy-phenolic	X	X	290	
C-4	Neoprene	X	X	290	Heat cured
C-5	Neoprene	X	X	290	
C-6	Chlorosulfonated polyethylene	X	X	290	
C-7	Chlorosulfonated polyethylene	X	X	290	
C-8	Resin varnish (generic type unknown)	X	X	290	Heat cured
C-9	Phenolic	X	X	290	
C-10	Vinyl	X	X	100	VR-3
C-11	Bituminous	X	X	290	
C-12	Phenolic		X	290	Experimental system
C-13	Phenolic		X	290	Experimental system
C-14	Coal-tar epoxy	X	X	250	MIL-P-23236, Class 2
C-15	Polyester	X	X	250	
C-16	EPT (ethylene-propylene terpolymer)	X	X	290	Experimental solution of EPT in solvent
C-17	Coal-tar epoxy	X	X	250	MIL-P-23236, Class 2
C-18	Coal-tar epoxy	X	X	250	MIL-P-23236, Class 2
C-19	Hexafluoropropylene-vinylidene fluoride	X	X	290	Experimental system with epoxy primer
C-20	Neoprene-asphalt	X	X	250	
C-21	Vinyl	X	X	100	VR-3
C-22	Vinyl	X	X	100	VR-3
C-23	Hexafluoropropylene-vinylidene fluoride	X	X	290	Experimental solution
C-24	Polyester	X	X	250	
C-26	Acrylic-epoxy, styrene modified	X	X	290	50-percent solution in styrene
C-32	Phenolic		X	290	Baking phenolic applied by manufacturer
C-33	Trifunctional silicone	X		290	Dip application by manufacturer
C-34	Trifunctional silicone		X	290	Spray application by manufacturer
C-36	Polypropylene	X	X	290	Flame sprayed
C-37	Chlorinated polyether		X	250	Applied by manufacturer
C-38	Polyarylether	X	X	290	Experimental resin solution
C-39	Polyethylene/polyviny acetate copolymer	X	X	290	Flame sprayed
C-40	Porcelain enamel		X	290*	Applied by manufacturer
C-41	Phenoxy	X	X	290	Difficulty in applying a satisfactory coating--No tests yet
C-43	Epoxy	X	X	290	
C-44	Epoxy (flexible)	X	X	290	
C-45	Polyimide		X	290	Baked enamel applied by manufacturer
C-47	Epoxy	X	X	290	
C-48	Inorganic zinc with stainless steel finish		X	290	
C-49	Silicone	X	X	290	
C-50	Polyurethane	X	X	225	
C-51	Modified epoxy	X		290	Applied by manufacturer
C-52	Sulfur	X		225	Applied by manufacturer
C-53	Epoxy		X	290**	Applied by manufacturer

1/

*250° F to 290° F exposure only.

**Pipe lining only at 290°.

Table V-3

JOINT SEALERS UNDER TEST

No.	Basic type of material	Max test temp. °F
J-1	Chlorosulfonated polyethylene	290
J-2	Neoprene	290
J-3	Neoprene	290
J-4	Butyl	290
J-5	Polysulfide	250
J-6	Epoxy	290

Table V-4

PLASTICS UNDER TEST

No.	Basic type of material	Max test temp. °F
P-1	Polyarylether	290
P-3	Polyphenylene oxide	290
P-5	Epoxy	290
P-6	Epoxy	290
P-7	Polyvinyl chloride	203

Table V-5

RUBBERS UNDER TEST

No.	Basic type of material	Max test temp. °F	Remarks
R-1*	Neoprene	290	
R-2*	Neoprene	290	
R-3*	EPT (ethylene-propylene terpolymer)	290	
R-4*	Chlorobutyl	290	
R-5	Neoprene	290	
R-6	Hi-temp butyl	350	350° F exposure in water in autoclave
R-7	Hi-temp butyl	350	350° F exposure in water in autoclave
R-8	EPT (ethylene-propylene terpolymer)	290	Sponge rubber--Preliminary tests only
R-9	Chlorobutyl	350	350° F exposure in water in autoclave
R-10**	Hi-temp butyl	290	
R-11	EPT (ethylene-propylene terpolymer)	290	
R-12*	EPT (ethylene-propylene terpolymer)	350	350° F exposure in water in autoclave
R-13*	Hexafluoropropylene-vinylidene fluoride	350	350° F exposure in water in autoclave
R-14*	Ethylene-neoprene blend	350	350° F exposure in water in autoclave
R-15	Polyacrylate	350	350° F exposure in water in autoclave
R-16	Polyacrylate	350	350° F exposure in water in autoclave
R-17	EPT (ethylene-propylene terpolymer)	290	
R-18	Hypalon (chlorosulfonated polyethylene)	290	
R-19	Flexible ebonite (natural)	290	
R-20	EPT (ethylene-propylene terpolymer)	290	
R-21	Butyl	290	
R-22	EPT (ethylene-propylene terpolymer)	290	
R-23	Butyl	290	See R-10
R-24	Butyl	290	See R-10
R-25	Epichlorohydrin	290	Partial tests at 290° only
R-26	Epichlorohydrin	290	Partial tests at 290° only
R-27	Polyacrylate	290	New samples in long-term tests
R-28	Polyacrylate	290	New samples in long-term tests
R-29	Butyl	290	
R-30	EPDM (ethylene-propylene diene)	290	
R-31	EPT/Butyl sponge	290	Replaces R-8

* Preliminary tests only--Manufacturer declined to furnish additional samples for long-term tests.
**Similar to R-23 or R-24.

V. 4. 2. Specimens

Several types of specimens are being utilized in this program. For the coatings tests, both portland cement mortar specimens and mild steel specimens are used. Plastic specimens for initial tests were of size and shape furnished by the manufacturer; however, with one exception a standard size of 6 by 6 by 1/8 inch is now used. Except for the very early exposure tests, when random size samples were tested, rubber samples are standard 6- by 6- by 0.074-inch tensile slabs. In a few cases, the 6- by 6-inch pieces were cut from standard sheet material.

Mild steel panels are 6 by 6 by 1/8 inch, AISI M1020 (SAE-1020) steel. The panels are solvent washed with xylene (Federal Specification TT-X-916b) and sandblast cleaned to base metal (white blast) just prior to use. The blasting sand is Del Monte white sand, EI-20, distributed nationally by Del Monte Properties Company, San Francisco, California, a standard blasting abrasive.

Mortar specimens are 6- by 6- by 1-inch blocks. The mix is as follows:

1 part by weight cement (laboratory blend, Type II)
0.38 part water
2 parts by weight graded sand

17 percent No. 8
17 percent No. 16
23 percent No. 30
23 percent No. 50
14 percent No. 100
6 percent pan
100 percent total

The blocks are cured for 28 days at 73° F and 100 percent relative humidity and then air dried for at least 7 days minimum. The blocks are then sandblasted and "sack rubbed" with a 2:1 sand-cement grout. The surface grout is cured for 3 days at 73° F and 100 percent relative humidity. Excessive surface protrusions are ground off by rubbing the grouted surfaces together, after which the blocks are rinsed clean and dried for 14 days minimum.

Initial joint sealer specimens are made up with two 3- by 2- by 1-inch portland cement mortar blocks with the sealer applied between the blocks. The mortar mix is identical to the mortar used in the base coatings specimens.

In addition, 20 large concrete bars have been fabricated for later tests on coatings and joint sealers. These are similar to the regular concrete test bars.

V. 4. 2. 1. Degassing. --

Early in the application schedule, during the application to mortar specimens of Coating No. C-8 which requires an oven cure, considerable foaming of the varnish was experienced as the temperature was raised to effect a cure. At first it was thought that this was the effect of retained solvent in the applied varnish; however, this theory was readily dispelled when a second mortar specimen with considerable longer drying time acted in an identical manner, and steel-coated specimens did not. Companion specimens of the varnish coating on mortar before and after cure are shown in Figure V-9.

It was concluded that the foaming was not caused by retained solvent, but was the result of contained gases and water seeking escape from the mortar as their volumes increased when the temperature was raised. The foaming was of greater magnitude as the specimen temperature rose above 200° F.

All other applied coating systems were then checked to see if similar deficiencies would result as the temperature was raised to the proposed operating level, 350° F. One specimen of mortar-coated blocks and one specimen of the coated steel panels from each set were heated in an oven to 350° F. All mortar block specimens showed

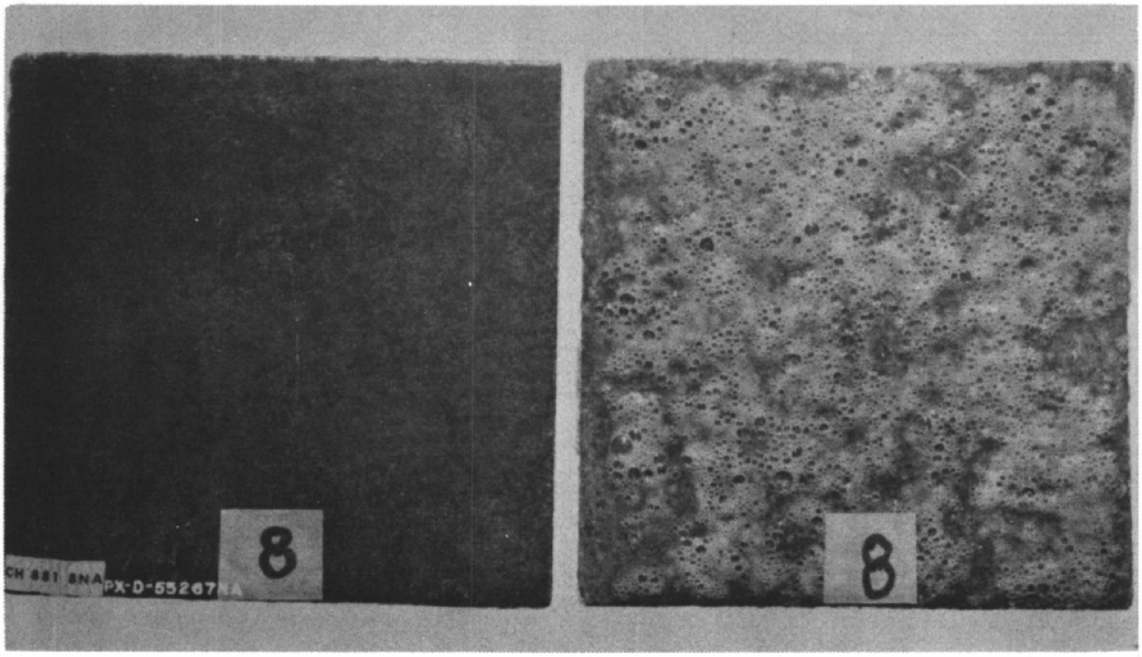


Figure V-9. Coating No. C-8 is shown on air-dried mortar specimens. At the left is a specimen with the full coating air dried, but before cure. At the right is a companion specimen after the oven cure. Considerable foaming occurred in the cured coating. Photo PX-D-55267 NA damage from expanding gases and water. These are shown in Figures V-10 through V-15. The elastomeric Coatings No. C-1, C-5, C-6, and C-7, displayed considerable blistering, whereas the rigid, hard Coatings No. C-2 and C-3, showed small curved cracks that appeared to be at the edges of slight swells resembling blisters, but without the characteristic bubble. The coated steel panels from each set did not have these deficiencies.

It is fortunate that this phenomenon was discovered before the test exposures started as the deficiencies may have been attributed incorrectly to coatings performance rather than to a condition of the base specimen.

Several techniques were tried to alleviate the situation. Air-dried mortar blocks were treated with inorganic silicate sealers, but these were ineffective. Degassing in a high vacuum was also tried, but again this was not effective. When the treated blocks were coated with Coating C-8, similar foaming occurred when heat curing was attempted.

However, an effective technique was developed. Mortar blocks preheated in an oven at 360° F for 12 hours minimum and then cooled in a desiccator did not display the foaming during subsequent oven curing of an applied film of Coating No. C-8. It was determined that the preheated blocks could be held in a desiccator for several days without surface effects occurring from expanding gases. Apparently ingress of atmospheric gases in the desiccator storage is of no consequence. This leads to the conclusion that the prime cause of the foaming and blistering that was experienced in applied coatings was free water and/or released water in the mortar specimens, which vaporized and expanded as the temperature was raised. All mortar specimens are now preheated at 360° F for 12 hours minimum (usually overnight or about 18 hours) and cooled in a desiccator prior to use. The results are illustrated in Figure V-16. Even though the maximum test temperature is now 290° F, preheating to 360° F is still used to keep this pretreatment constant for all specimens.

The effects of contained gases and water are thus minimized and will not influence the evaluation of performance of the test coatings. A secondary benefit has been realized also from the oven preheating. With air-dried blocks, considerable pinholing was occurring in the application of the test coatings. These required repair before the full coating system was applied to obtain a continuous film. This is characteristic of coating applications on mortar or concrete. With the oven preheated blocks virtually no pinholing has occurred during the coating applications.

This leads to the general conclusion that in an actual field application of coatings to concrete or mortar surfaces in high-temperature desalting plants, degassing and drying of concrete or mortar surfaces prior to coating application will be necessary.

V.4.3. Pipe Lining Tests

In addition to the evaluation tests on portland cement mortar blocks and steel panels, seven of the test coatings have been applied to the pipe and elbow sections of the coating test loops for additional evaluation. The coatings were applied by a commercial coatings applicator by purchase order. The applications were closely inspected by a Bureau inspector.

In addition, an extra piece of pipe was lined with an epoxy coating by a commercial pipe liner. The location of each test material is shown in Figure V-17.

V.4.4. Special Tests

Special tests are being conducted on occasion on materials that warrant investigation. These are conducted on materials that would have limited or singular use in a desalting plant.

V.4.4.1. Epoxy asphalt and asphaltic concrete. --

A mix of 9 percent epoxy-asphalt binder (based on dry weight of aggregate) combined with graded aggregate of 1/4-inch maximum size was studied as a possible flooring

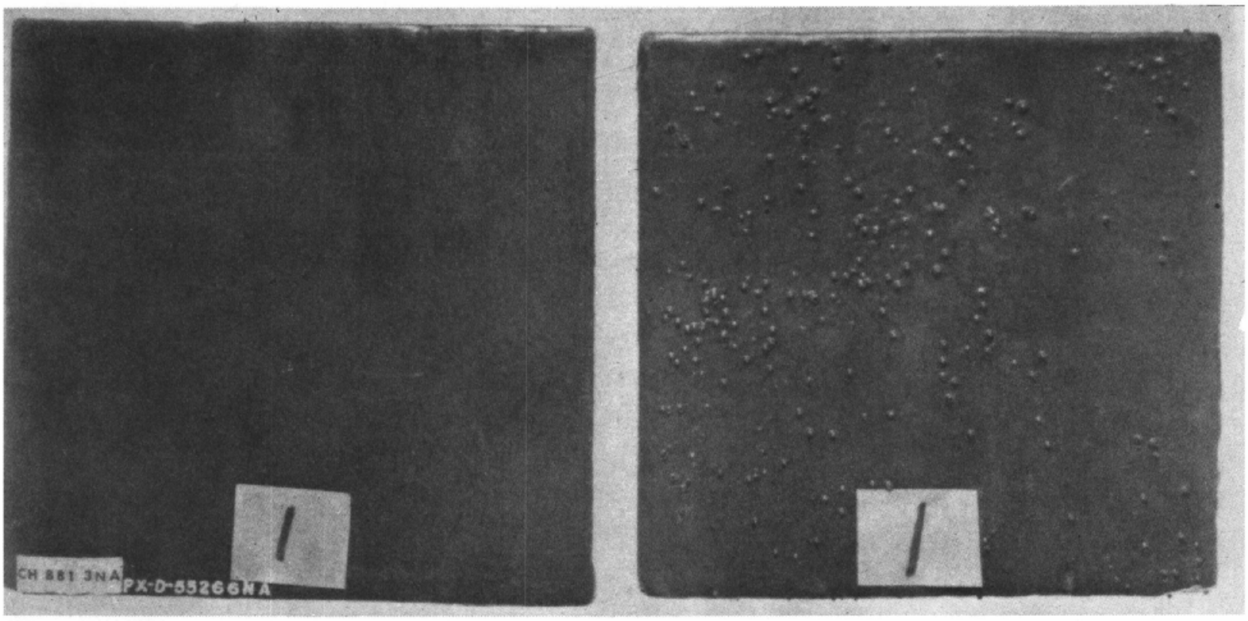


Figure V-10. Coating No. C-1 is shown on air-dried mortar specimens. At the left is a specimen with the full coating applied and air dried. At the right is a companion specimen that was heated in an oven to 350° F. Numerous small blisters are evident in the coating on the heated specimen. Photo PX-D-55266 NA

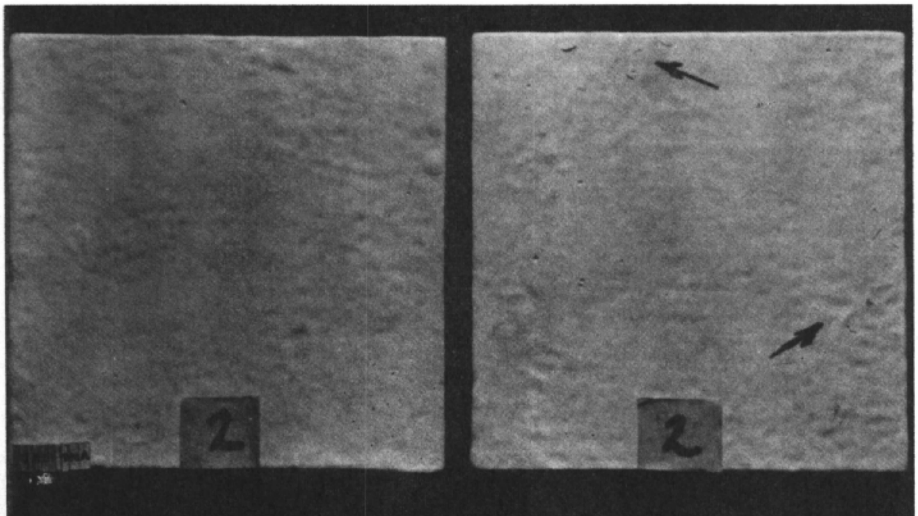


Figure V-11. Coating No. C-2 is shown on air-dried mortar specimens. At the left is a specimen with the full coating applied and air dried. At the right is a companion specimen that was heated in an oven to 350° F. Several small curved cracks, indicated by arrows are evident in the coating on the heated specimen. Photo PX-D-55265 NA

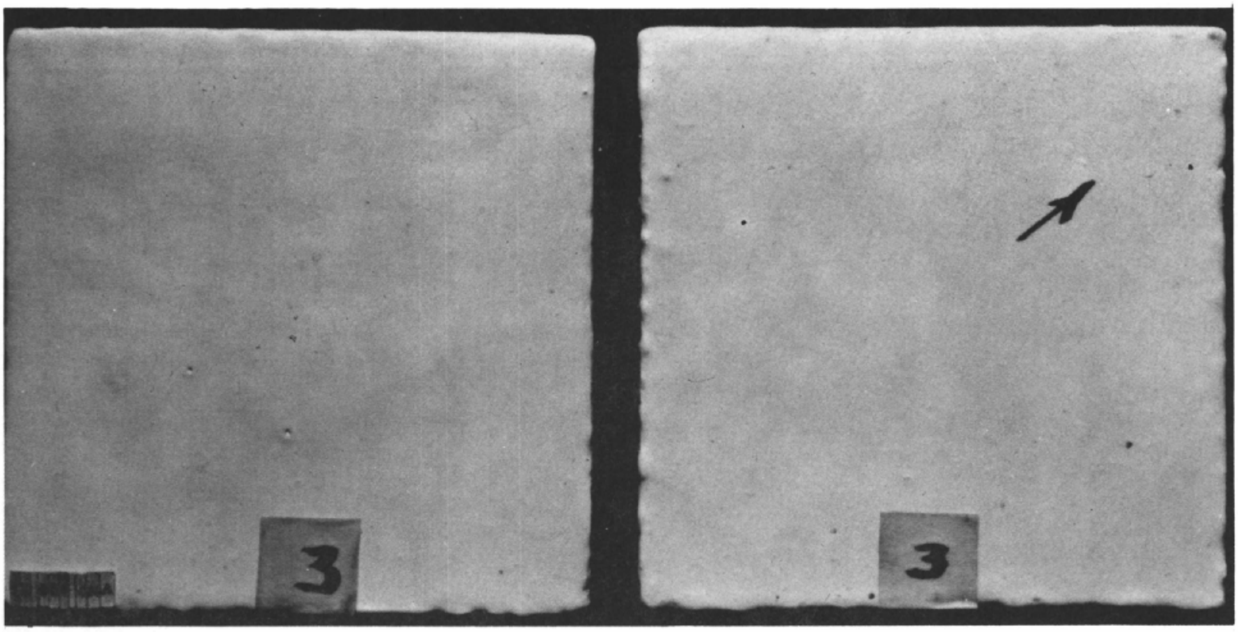


Figure V-12. Coating No. C-3 is shown on air-dried mortar specimens. At the left is a specimen with the full coating applied and air dried. At the right is a companion specimen that was heated in an oven to 350° F. Several small curved cracks, in the area marked by the arrow, are evident in the coating on the heated specimen. Photo PX-D-55250 NA

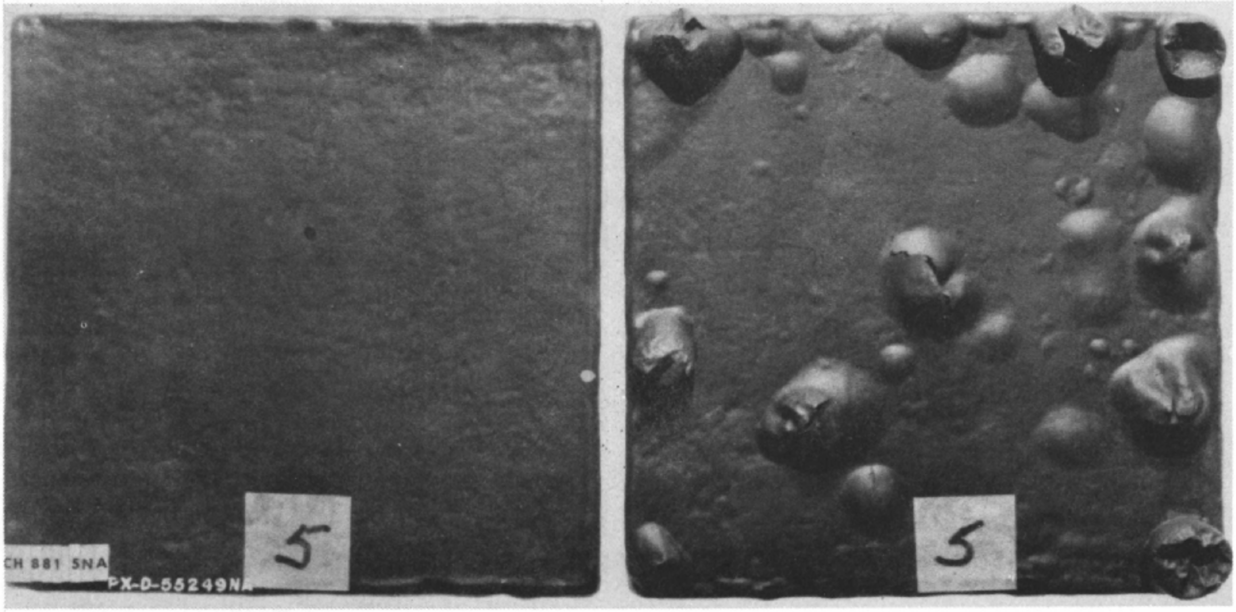


Figure V-13. Coating No. C-5 is shown on air-dried mortar specimens. At the left is a specimen with the full coating applied and air dried. At the right is a companion specimen that was heated in an oven to 350° F. Numerous large blisters appeared in the heated specimen. Characteristic solvent odor was not evident when the blisters were ruptured. Photo PX-D-55249 NA

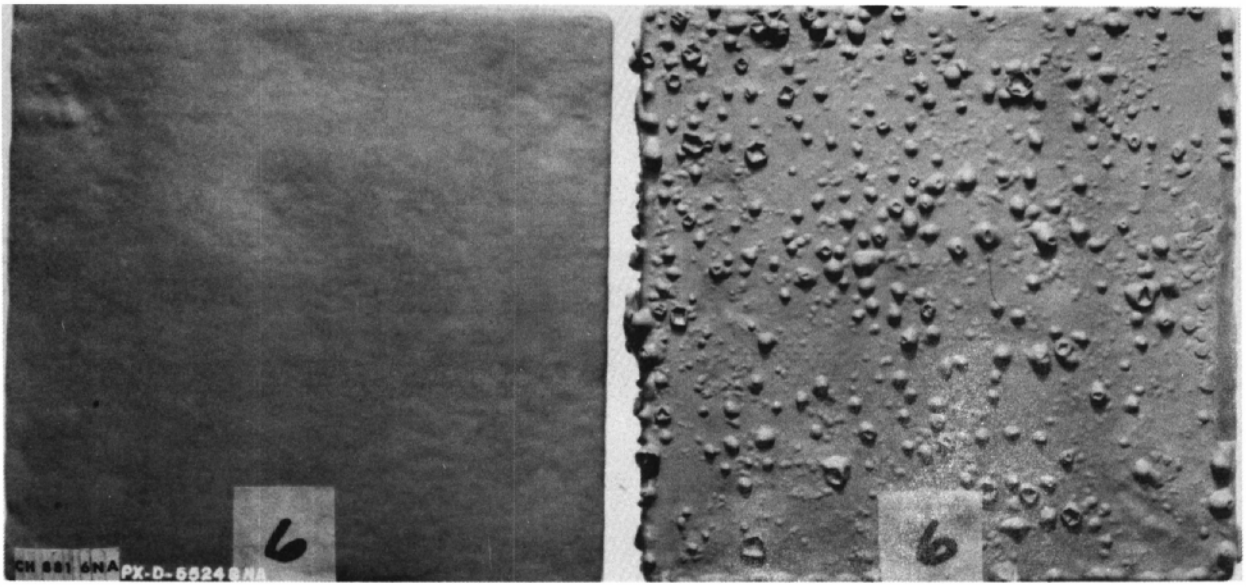


Figure V-14. Coating No. C-6 is shown on air-dried mortar specimens. At the left is a specimen with the full coating applied and air dried. At the right is a companion specimen that was heated in an oven to 350° F. Numerous blisters appeared in the heated specimen. Photo PX-D-55248 NA

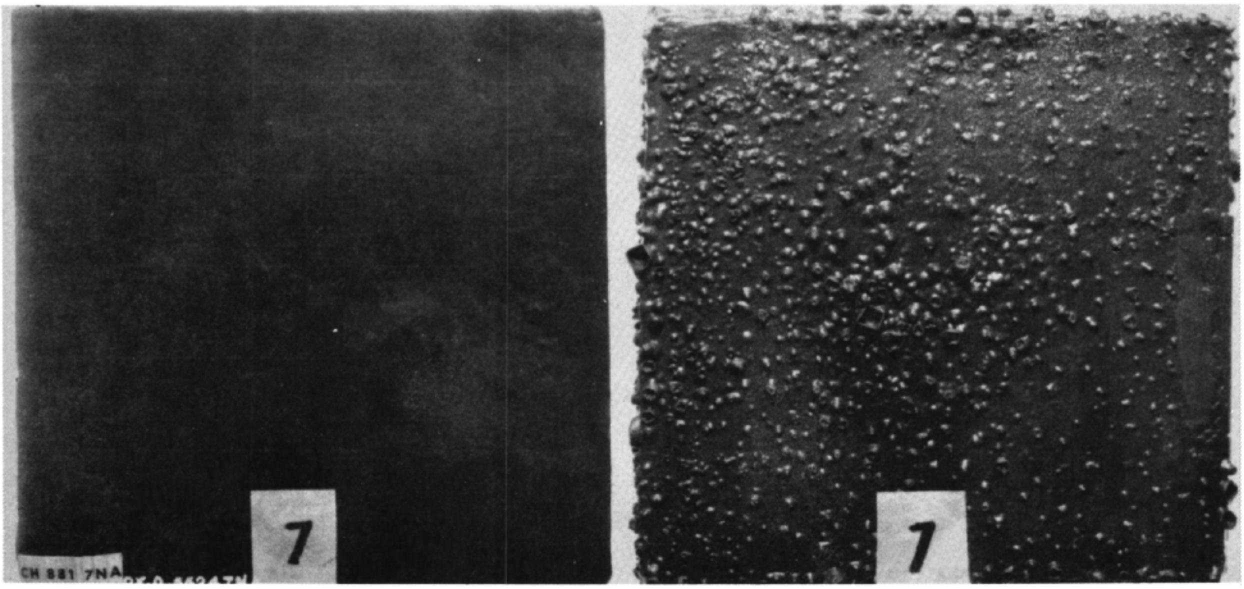


Figure V-15. Coating No. C-7 is shown on air-dried mortar specimens. At the left is a specimen with the full coating applied and air dried. At the right is a companion specimen that was heated in an oven to 350° F. Numerous blisters appeared in the heated specimen. Photo PX-D-55247 NA

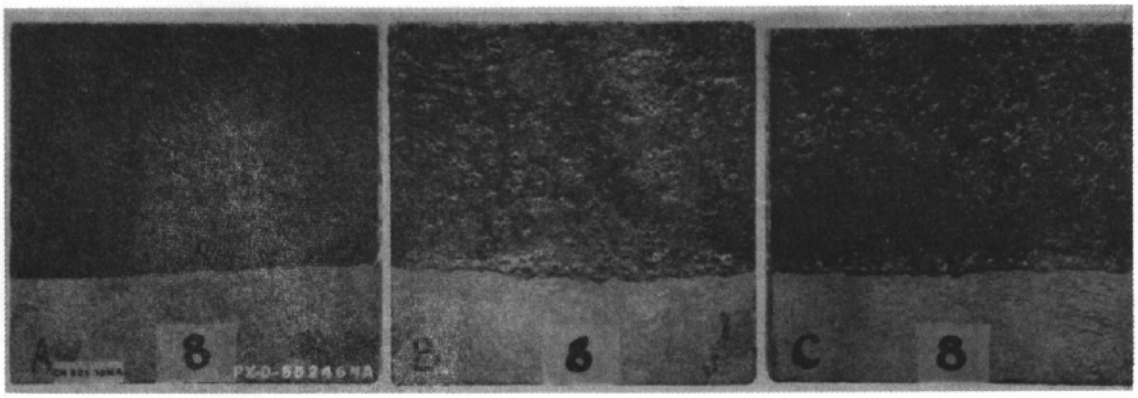
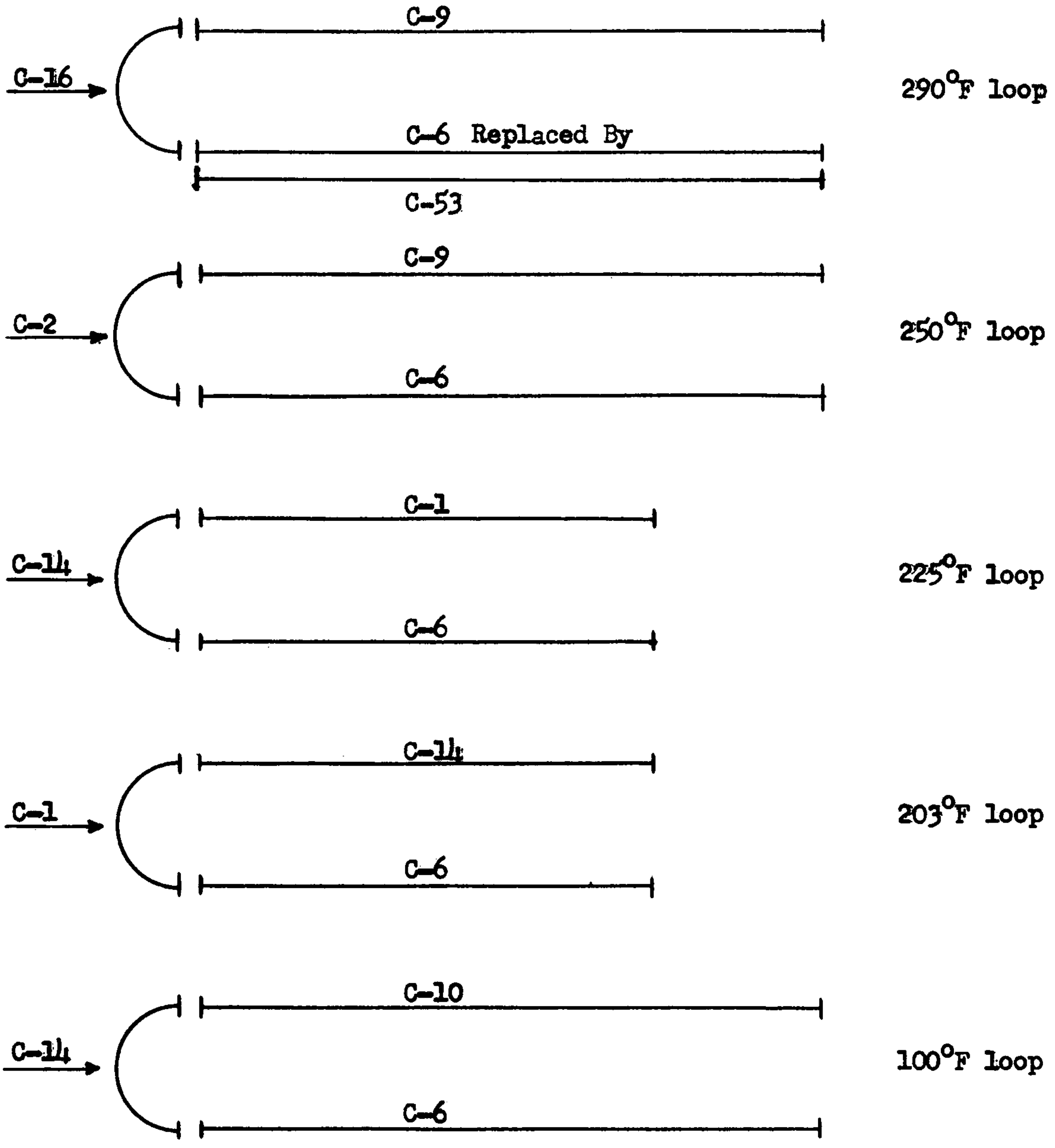


Figure V-16. The results of degassing mortar specimens are shown. Block A was preheated at 360° F and cooled in a desiccator; no foaming occurred in cure of Coating No. C-8. Block B was air dried with no subsequent treatment; considerable foaming occurred in cure of Coating No. C-8. Block C was vacuum degassed; considerable foaming occurred in cure of Coating No. C-8. Photo PX-D-55246 NA



Location of Test Linings
in Coatings Loops

FIGURE V-17

material for brine channels. The binder is a proprietary product consisting of a blend of 85 percent of 85 to 100 penetration paving grade asphalt and 15 percent epoxy resin.

A test specimen 4 inches in diameter by 4 inches high of the mix was molded using a hydraulic press while the mix was held at a temperature of 250° F. A compressive load of 3,000 psi was applied for 2 minutes to the specimen for compacting to laboratory standard density. The specimen represented a quality, compacted epoxy-asphalt concrete.

Physical properties of the epoxy-asphalt concrete were:

Density: 141.43 pounds per cubic foot
Air voids: 3.24 percent
28-day compressive strength,
  140° F oven cured: 3,795 psi

The molded specimen was tested in 350° F water in an autoclave. After 60 days' exposure, the binder had completely disintegrated. The remains consisted of some char and loose aggregate. From this it was concluded that epoxy-asphalt concrete is not suitable in 350° F wet environments. However, the mixture may have use at a lower temperature; this will be investigated as time permits.

A survey of industry was conducted to locate asphalt materials that might be suitable binders for 1/4-inch-maximum-size aggregate to serve as a flooring material for brine channels. Samples of an improved epoxy-asphalt binder and three newly developed proprietary asphalt binder products have been obtained and physical property tests have been conducted on the latter three materials. One of these has the unique combination of a high softening point and a fluid viscosity at 135° F.

Aggregates have been screened and prepared for fabrication of mix design test cylinders using the high softening point asphalt and the epoxy asphalt. Information from these tests will enable proper mix proportions to be established so that specimens may be prepared for exposure to selected environments of the saline water program.

V.4.4.2. Rubber and plastics materials.--

A variety of rubber materials are under evaluation, as listed in Table V-5. Rubber may be needed for waterstops, linings, seals, or gaskets. Changes in physical properties after exposure to the five brine and temperature environments in the test loops for the duration of the test period are being determined. Some preliminary exposures have been conducted in 350° F autoclave environment with water, and in test tanks on the original 100° and 290° F test loops. The physical property tests being conducted are:

1. Tensile strength and elongation:

   Rubber--ASTM: D-412; Grip velocity: 20"/min
   Hard rubber--ASTM: D-530; Grip velocity: 2"/min
   Rigid plastic--ASTM: D-638; Grip velocity: 0.2"/min

2. Tear strength: ASTM: D-642.

3. Indentation hardness: ASTM: D-2240 with a 5-second interval before reading using a single material thickness. Shore "D" units are used for P-1, P-3, and R-19. Shore "A" units are used for all other samples.

4. Property changes due to liquid immersion: ASTM: D-471.

5. Ozone resistance: ASTM: D-1149 (only initially).

6. Specific gravity: ASTM: D-792, Method A-1, except that since the entire sample was used, the weight and shape of the samples generally made it necessary to obtain weights accurate only to the nearest one-tenth gram. This

procedure was then established for all materials. The volume reported was obtained from the difference in the immersed and dry weight of the sample at 73.4° F.

The initial physical properties of the preliminary rubber samples are listed in Table V-6. Changes after 70 hours in boiling distilled water are also listed in Table V-6.

V.4.4.2.1. Specimens. --

Primary exposures commenced when the individual loops were placed in operation. Sufficient samples of elastomeric and plastic materials were introduced into these environments to allow for testing at various exposure time intervals. The samples were generally standard manufacturers' test slabs 6 inches square by 0.06 inch to 0.10 inch thick. One plastic material (P-3) was a disc measuring 4 inches in diameter. During the first 6 months of exposure, one sample of each material was removed for testing at 30-day exposure intervals. As the samples were removed they were washed to remove surface debris. They were then held for 1 hour in 23° C water, prior to obtaining the immersed weight, in order to achieve proper specimen temperature and to prevent air bubbles from clinging to the surface during the weighing. Samples were then dried and the length and width measured. Length was measured 1 inch from each edge and averaged while width was measured 1 inch from the bottom only. The dry weight of the samples was then obtained.

V.4.4.2.2. Plastics. --

The surface hardness of the plastic materials was measured using a Shore "D" durometer. The samples were then sent to the shops where tensile specimens were cut in a milling machine. These tensile specimens were given a final buffing and allowed to remain at standard conditions of 23° plus or minus 1° C temperature and 50 plus or minus 2 percent relative humidity for a minimum of 24 hours before testing. Tensile properties were obtained using a 600-pound load range for the load and an electronic extensometer (set at either 5 or 10 magnification as required) was used to obtain extension readings. Both the load and magnified extension signals were fed into an X-Y recorder and plotted as the test was in progress. Specimen thickness obtained in measuring the specimen cross section was used for the sample thickness.

V.4.4.2.3. Rubbers. --

The rubber samples were allowed to remain at standard conditions for a minimum of 24 hours before further testing. After this conditioning time the surface hardness was measured using the Shore "A" durometer in the lower portion of the sample and then tensile and tear specimens were cut from this same area and tested. A 100-pound range load cell and an extensometer designed to follow large specimen movements and send out an unmagnified signal were used. Both the load and extension signals were fed into an X-Y recorder and plotted as the test was in progress for both tensile tests and tear tests. Thickness measurements obtained from both tensile and tear specimens were averaged to obtain the sample thickness. After completion of physical properties testing, the unused portion of the specimen was marked for exposure time identification and retained in storage for any future visual observations that might be desired.

V.4.5. Brine Tests

All of the test materials are being exposed to the brine conditions in the test loops, as described in Section V-2 and elsewhere in this report. The exposures began when each of the test loops became operational. The coatings test loops are bypass sections in the main test loops and can be opened and closed independent of the rest of the loop system. Accordingly, more frequent readings are taken.

As of April 1967 the 100° F loop had 300 days, the 203° F loop had 210 days, the 225° F loop had 240 days, the 250° F loop had 180 days, and the 290° F loop had 240 days.

Table V-6

RUBBER MATERIALS

Material No.	R-1	R-2	R-3	R-4	R-5	R-6	R-7	R-9	R-10	R-11	R-12	R-13	R-14	R-15	R-16
Test / Type	Neo	Neo	EPT	Chl But	Neo	Hi-T But	Hi-T But	Chl But	Hi-T But	EPT	EPT	HFP-VF	Eth-Neo	Acrylic	Acrylic
INITIAL PHYSICAL PROPERTIES															
Specific gravity	1.51	1.52	1.16	1.12	1.43	1.11	1.12	1.09	1.09	1.13	1.13	1.85	1.19	1.26	1.26
Tensile strength, psi	1,921	1,685	2,114	2,794	2,385	2,088	1,827	2,125	1,536	1,568	2,594	2,856	2,305	2,413	2,090
Elongation, %	310	225	490	540	330	430	440	320	500	310	363	267	250	190	327
Tear strength, lbs/in	189	244	302	348	210	212	350	269	344	288	234	194	233	181	178
Shore "A" Hardness	52	62	55	48	61	45	56	57	56	68	63	67	65	54	53
Ozone resistance; hrs at 100° F, 50 pphm	160	300	1/ 650	160	1/ 650	240	1/ 1,030	70	40	1/ 1,030	1/ 1,200	1/ 1,200	1/ 1,200	1/ 1,200	1/ 1,200
AFTER EXPOSURE TO BOILING DISTILLED WATER FOR 70 HOURS															
% strength retained	67	97	102	102	84	99	100	95	117	108	99	100	82	75	95
% elongation retained	100	104	100	89	103	95	107	103	100	97	86	110	80	98	103
Point Shore Hardness change	-3.0	-8.0	-2.0	-3.0	-4.0	-1.5	+1.0	-3.0	-2.0	-1.5	-2.0	-1.0	-2.0	-3.0	-4.5
% swell, thickness	3.43	3.59	1.99	2.05	3.03	2.20	0.67	2.31	1.83	1.62	0.35	4.27	3.12	2.20	1.82
% swell, volume	7.03	8.29	4.90	5.93	7.20	5.29	1.03	6.14	3.32	1.42	0.90	7.18	4.31	3.54	4.44
% weight increase	4.90	6.18	4.88	5.85	5.22	6.53	1.62	7.03	3.29	1.48	0.77	4.24	4.37	4.16	4.17
AFTER EXPOSURE TO BOILING SEA SALT WATER FOR 70 HOURS															
% strength retained	95	92	98	98	83	107	102	99	115	104	104	103	75	96	97
% elongation retained	95	102	100	81	98	100	107	106	100	87	89	110	72	102	110
Point Shore Hardness change	-2.0	-9.0	-6.0	-3.0	-1.5	-1.0	-1.5	-3.0	-4.0	-0.5	-1.5	-1.0	-1.0	-4.0	-2.5
% swell, thickness	0.88	1.47	0.93	1.59	1.01	1.30	0.48	0.93	1.07	1.48	0.21	1.81	2.03	1.22	0.87
% swell, volume	2.15	4.39	1.72	3.99	2.30	3.14	0.79	2.76	1.44	0.03	0.27	2.36	2.02	1.23	2.02
% weight increase	1.79	3.49	2.27	4.43	1.85	4.07	1.66	4.54	2.19	0.48	0.39	2.38	2.28	2.32	2.22

1/Continuing.

For the periods indicated, the coating specimens on steel and concrete have been removed each 30 days for physical tests. As new specimens are prepared or received from the manufacturer, they are placed in test in each loop when the loop is out for coatings physical tests.

The specimens are washed and allowed to dry for 3 to 4 hours before physical tests are started. The coating thicknesses on steel specimens are measured with a standard thickness gage. A point index has been established so that measurements are taken in the same six places on each side of the specimens every 30 days. If the thickness is greater than 20 mils, the thickness is read to the nearest mil. If the thickness is between 1 and 20 mils, the thickness is measured to the nearest 0.5 mil. For less than 1 mil, the thickness is determined to the nearest 0.05 mil.

The specimens are also weighed to determine weight changes. Weight changes are recorded to the nearest 0.5 gram, and reflect moisture absorption if positive or erosion losses if negative. Along with the weight changes, the general performance and appearance of the coating is recorded. Special remarks are recorded about each defect as it relates to the performance of the coating.

V.4.6. Autoclave Tests

Complementary autoclave tests are planned for coating materials. These will be used to evaluate the performance in boiling distilled water and steam. Exposure temperatures are shown in Section V-2. Five autoclaves have been procured for these tests, but deficiencies in the design and construction requiring modifications have delayed the start of these tests. However, all specimens are prepared and the exposures should commence in the immediate future.

V.5. Discussion

Performances of the individual test materials are discussed in the paragraphs that follow.

V.5.1. Rubbers and Plastics

The tensile strengths (percent retained) of all rubber materials (except R-19, R-29, and R-30) at the five brine conditions are shown in Table V-6, and Figures V-18 through V-22. A survey of the test data indicated that at each temperature level the change in tensile strength due to exposure time was small. Therefore, average test values at each brine loop temperature were calculated and used for graphic comparison.

As shown in Figures V-18 through V-21 there has been little change in the tensile strength for a majority of the materials subjected to the 100°, 203°, 225°, and 250° F environments. In most cases over 90 percent of the original tensile strength was retained. Changes are more definite at the 290° F brine condition.

Plastics No. P-5, P-6, and P-7 are not being tested in all exposures because of lack of specimens. Insufficient data are available at report time for detailed discussions on these materials.

V.5.1.1. Plastic P-1. --

As shown in Figure V-23 this material has changed considerably in tensile strength and elongation. Although the change is more severe with increasing temperature and time, a significant change appears to have taken place within 30 days at each temperature. A plateau in the rate of change has been reached at the end of 60 days. There is insufficient length of testing to date to predict the stability of this plateau. Visual observations of the 290° F brine exposure samples showed some warping and a change from clear to milky appearance.

V.5.1.2. Plastic P-3. --

The data in Table V-8, Appendix V-7, indicates that there has been no significant change in the material during this period of exposure. The original tensile strength is very much below the range of tensile strengths encountered after the various exposures. The original tests will be reviewed and new specimens cut from samples retained in storage to investigate this occurrence.

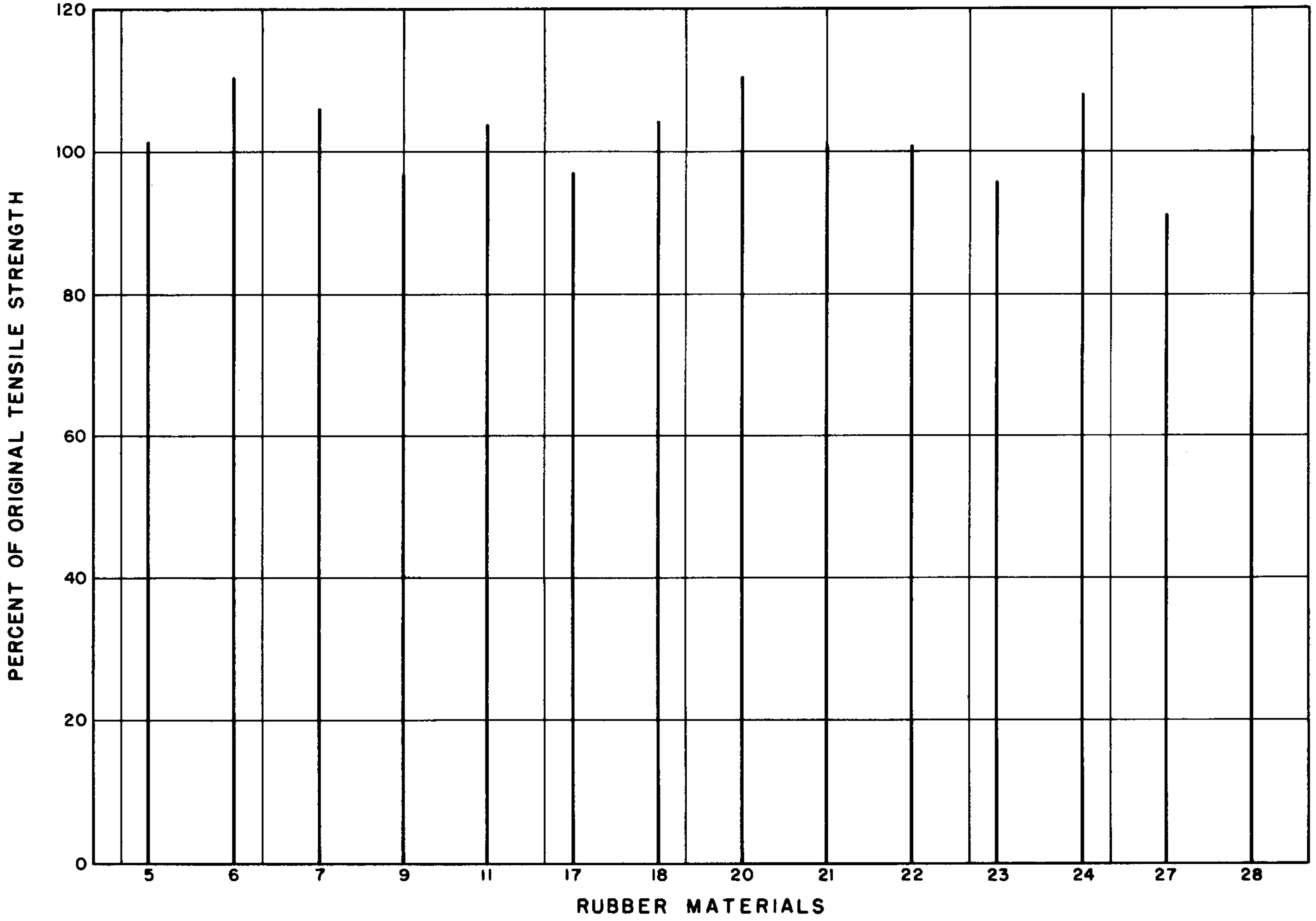


FIGURE V-18 – CHANGE IN TENSILE STRENGTH (PERCENT RETAINED) FOR VARIOUS RUBBER MATERIALS, AT 100°F, BRINE

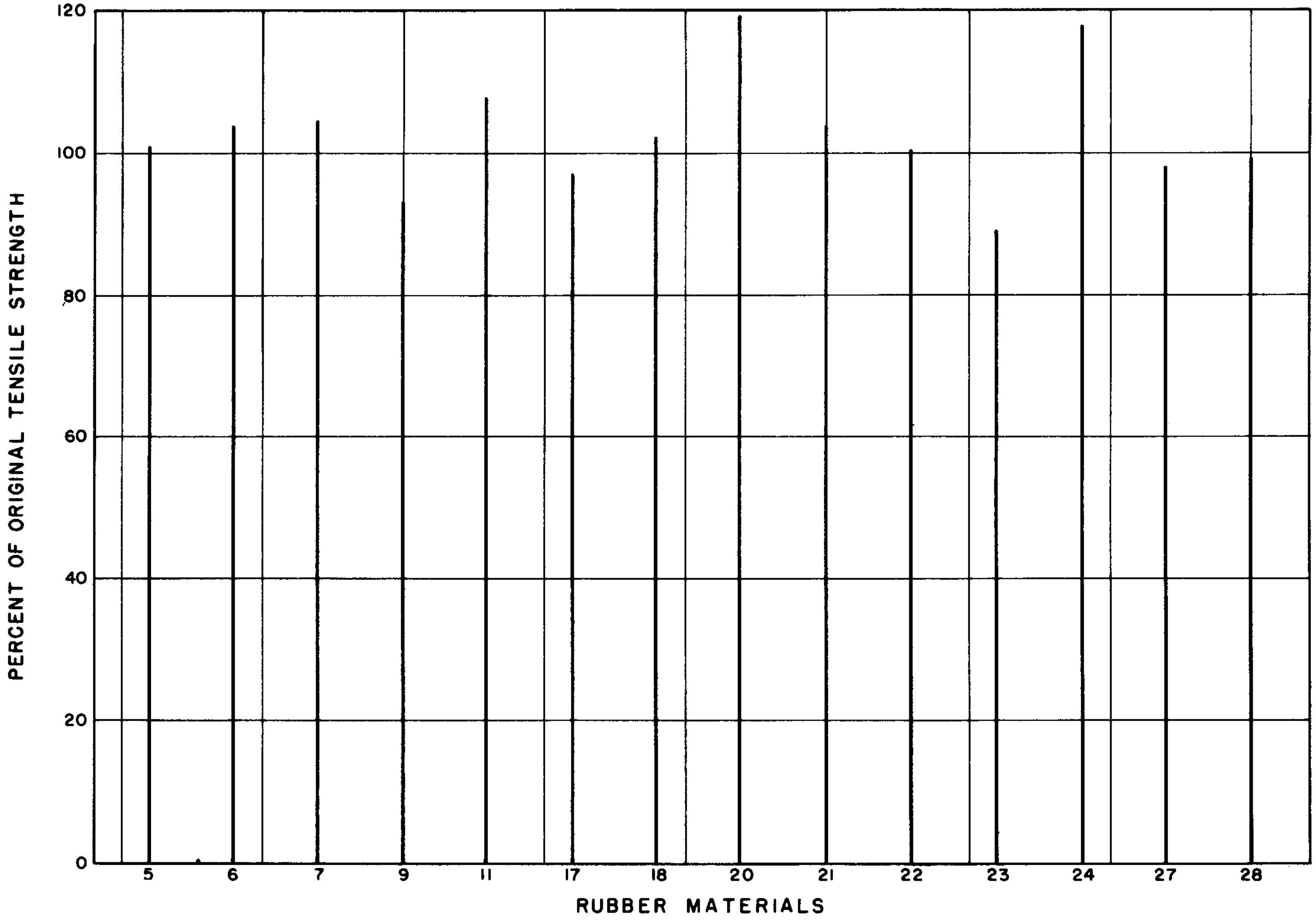


FIGURE V-19 - CHANGE IN TENSILE STRENGTH (PERCENT RETAINED) FOR VARIOUS RUBBER MATERIALS, AT 203 °F, BRINE

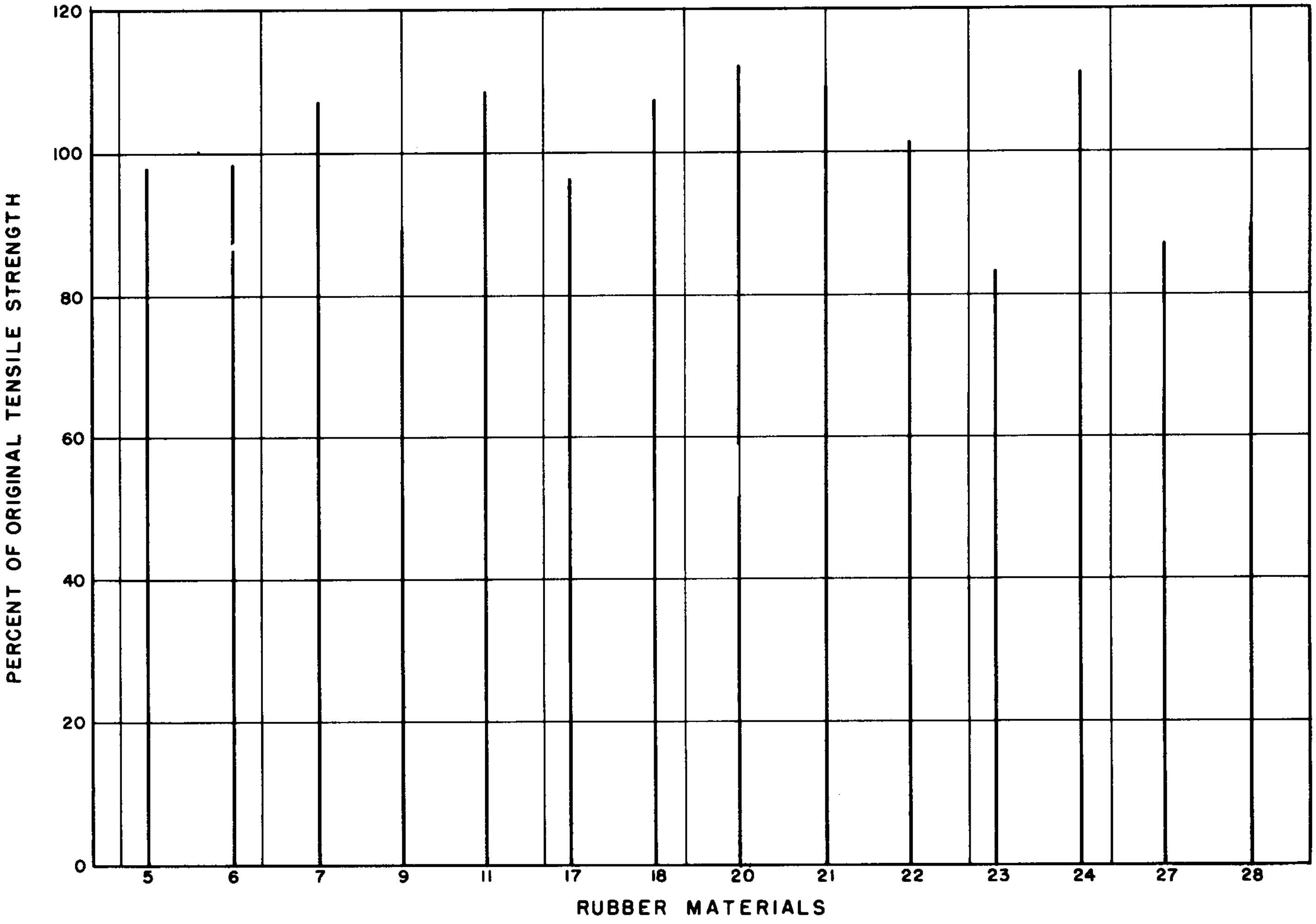


FIGURE V-20 – CHANGE IN TENSILE STRENGTH (PERCENT RETAINED) FOR VARIOUS RUBBER MATERIALS, AT 225 °F, BRINE

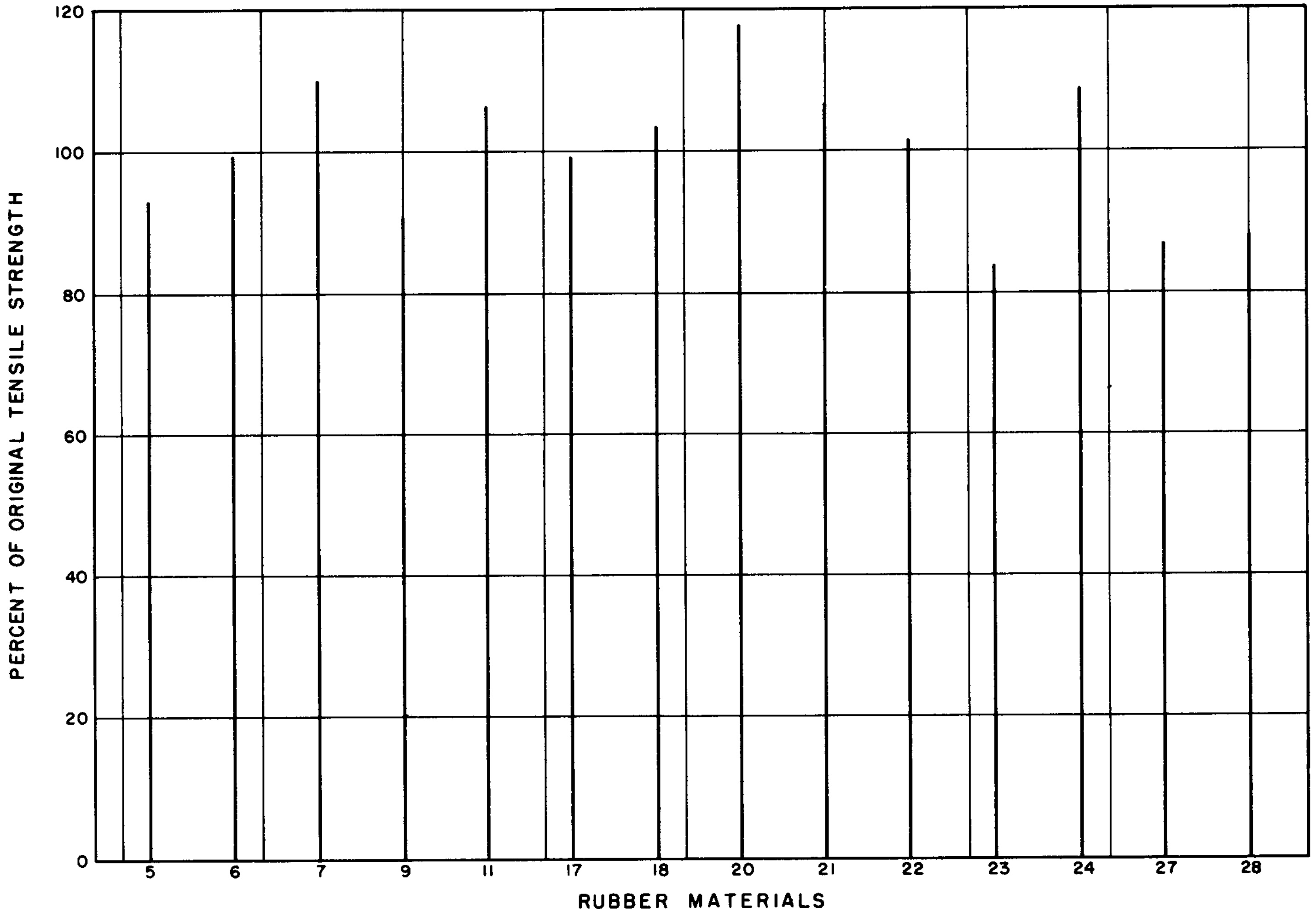


FIGURE V-21 – CHANGE IN TENSILE STRENGTH (PERCENT RETAINED) FOR VARIOUS RUBBER MATERIALS, AT 250 °F, BRINE

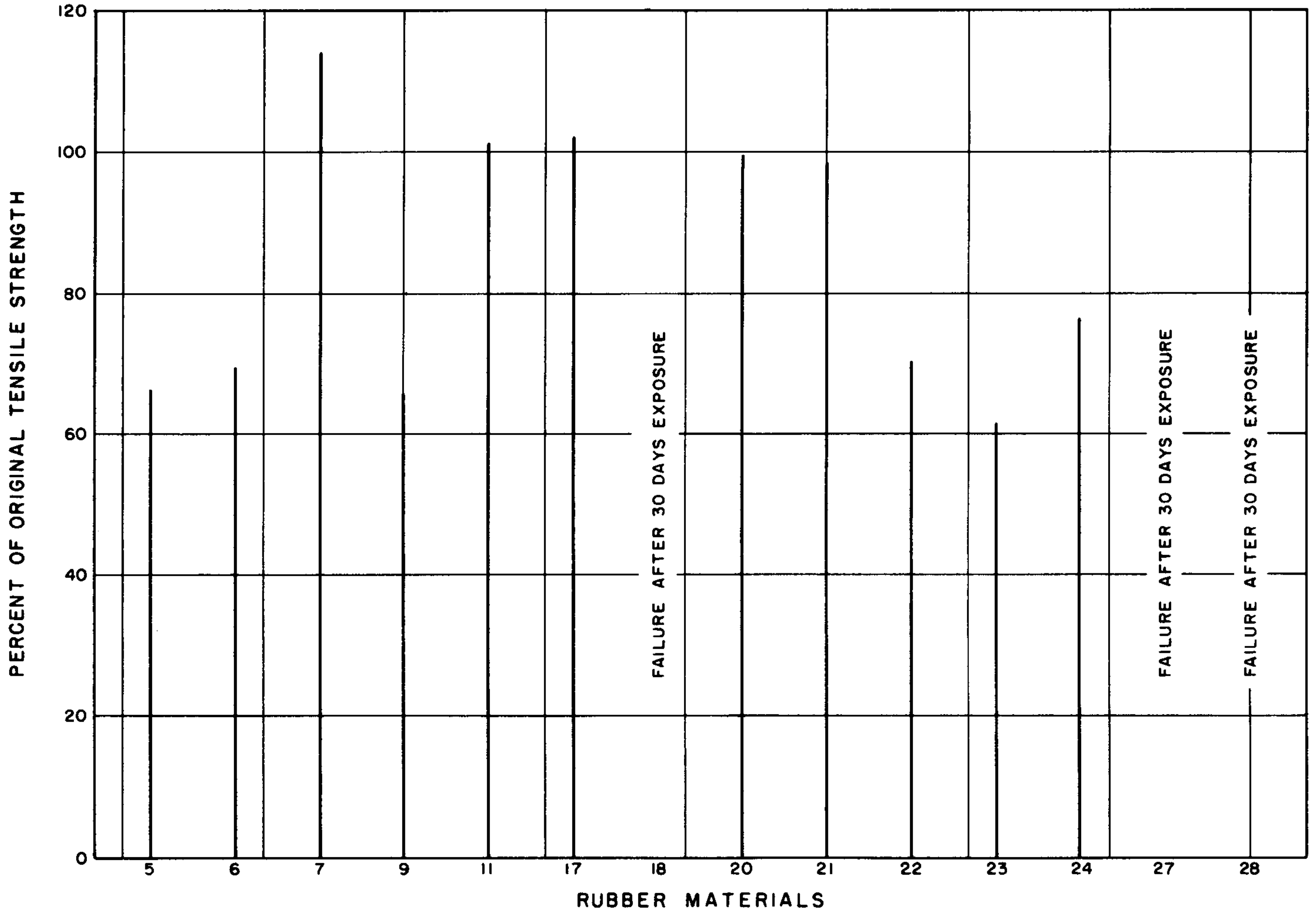


FIGURE V-22 - CHANGE IN TENSILE STRENGTH (PERCENT RETAINED) FOR VARIOUS RUBBER MATERIALS, AT 290 °F, BRINE

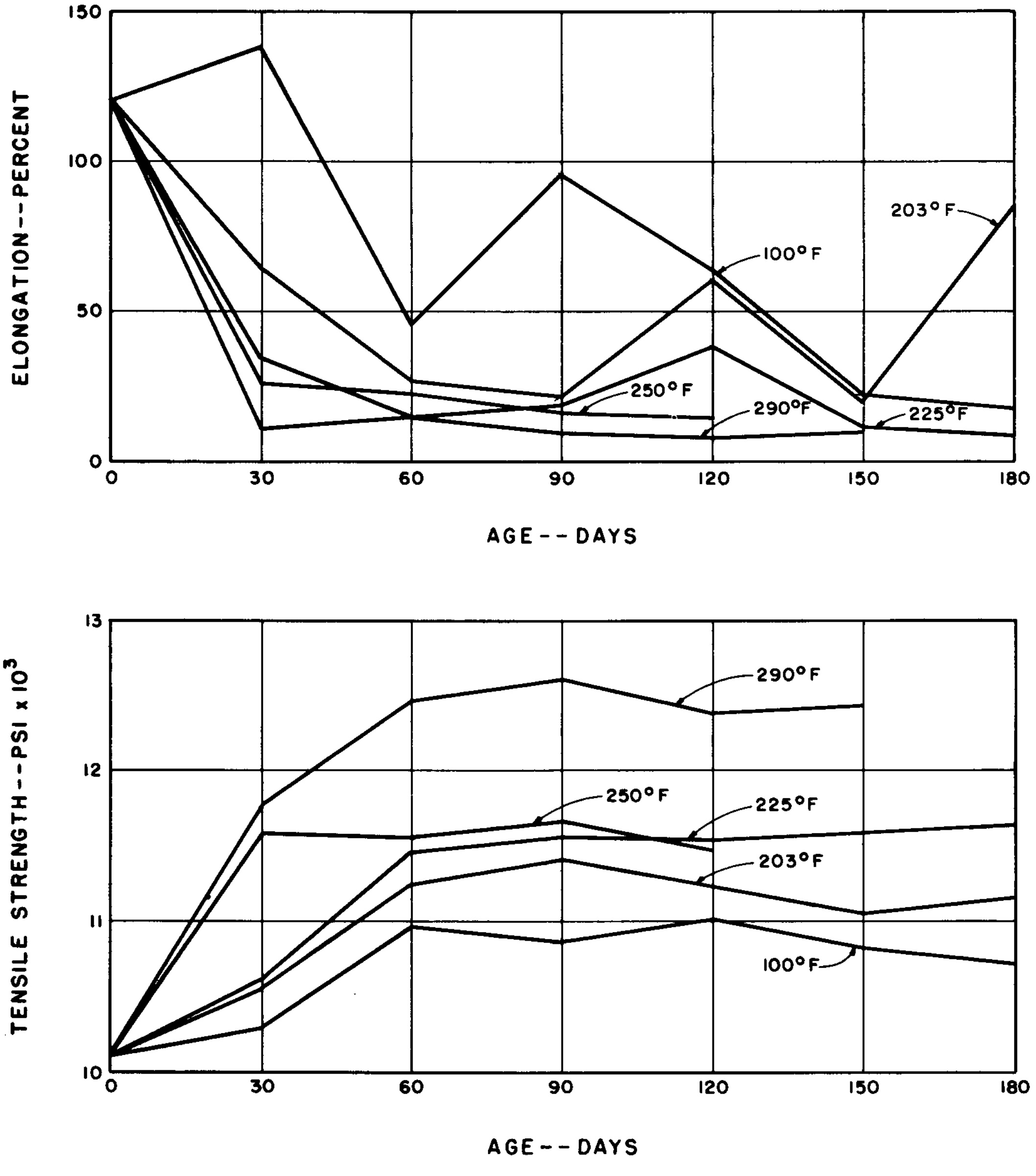


FIGURE V-23 - CHANGE IN ELONGATION AND TENSILE STRENGTH WITH EXPOSURE TIME AT VARIOUS BRINE CONDITIONS FOR PLASTIC NO. P-1.

V. 5. 1. 3. Rubber R-5. --

The test data and observations while handling the material indicate the rubber has undergone noticeable change during brine exposure. Although increased effort has been necessary to cut test specimens (using a die-type cutter), the tear strength and surface hardness has been decreasing, especially at higher temperatures. A much lower tensile strength and elongation were noted at 290° F although test values have fluctuated with exposure time.

V. 5. 1. 4. Rubber R-6. --

A greater reduction in tear strength is noted with the brine temperature at 225° F and above. In the 290° F brine loop the material increases in weight and volume (all dimensions) and there is a reduction in the tensile strength, elongation, and hardness.

V. 5. 1. 5. Rubber R-7. --

Very little change was measured at any exposure condition.

V. 5. 1. 6. Rubber R-9. --

Reduction in tear and tensile strength and elongation with time and temperature. Although the tensile properties at 290° F brine appear to have stabilized after a radical change, the high swell encountered may make the material unsuitable for use at this condition.

V. 5. 1. 7. Rubbers R-11 and R-17. --

Gradual decrease in elongation and hardness (softening) with increasing temperature occurs, but the rubbers appear stable at each temperature.

V. 5. 1. 8. Rubber R-18. --

This material was subject to high swelling at the 290° F brine condition. An increase in volume of over 300 percent occurred after 30 days' exposure. Physical properties were also radically changed as shown in Table V-15, Appendix V-7. Testing at 290° F brine was discontinued. No changes at other environments can be yet established with the exception of gradually reducing tear strength at 225° and 250° F.

V. 5. 1. 9. Rubber R-19. --

This material after 30 to 60 days' exposure has become hard and brittle at all temperatures except 100° F. The embrittlement and warping were severe enough in some cases that suitable specimens could not be obtained for testing. Under such circumstances the test values were recorded as zeros (Table V-16, Appendix V-7). At 100° F a softening of the material is indicated by increasing elongation and decreasing tensile strength. Visual observations of this material as well as the test data indicate that this material may not be serviceable for 203° F and higher brine conditions.

V. 5. 1. 10. Rubber R-20. --

A decrease in elongation, hardness (softening), and tear strength with an increase in temperature was noted. There is some indication of shrinkage at 290° F brine.

V. 5. 1. 11. Rubber R-21. --

Results are similar to R-17 and R-11. Gradual decrease in elongation and hardness (softening) occurs with increasing temperature, but the rubber appears stable at each temperature.

V.5.1.12. Rubber R-22.--

Very little change was detected below 290° F brine exposure. At the 290° F brine exposure, tensile strength of this material has decreased to less than 70 percent of original strength and the volume swell has exceeded 20 percent.

V.5.1.13. Rubber R-23.--

Little change below 290° F brine exposure. Softening indicated by decrease in hardness, strength and increase in elongation has occurred at 290° F brine.

V.5.1.14. Rubber R-24.--

Gradual decrease in tear strength, elongation, and hardness (softening) occurs with increasing temperature. Additional change in tensile strength and volume swell exceeding 20 percent indicate that this material may not be serviceable at 290° F brine condition.

V.5.1.15. Rubbers R-25 and R-26.--

Due to lack of the required number of specimens, the manufacturers' initial test values were used. The initial tensile strengths are 2,090 psi for R-25 and 1,940 psi for R-26; the initial elongation in percent, are 350 for R-25 and 370 for R-26; and the Shore hardnesses are 65 for R-25 and R-26. The tensile strength, elongation, and Shore hardness for R-25 after 90 days at 290° F are 958 psi, 170 percent, and 60. For the R-26, the values are 340 psi, 188 percent, and 46. These two specimens appear unsuitable for use at this condition.

V.5.1.16. Rubbers R-27 and R-28.--

After 30 days exposure to 290° F brine these materials were changed from a highly flexible state to a hard, brittle and cracked condition at standard temperature and humidity. When immersed in water they would swell over 100 percent and take on a gel-like structure. A similar condition occurred after 120 days at 250° F in brine except that the material was not cracked nor quite as brittle in the dry state. Testing has been discontinued on samples exposed to the 250° and 290° F brine environment.

V.5.1.17. Rubbers R-29 and R-30.--

No significant change has occurred during the maximum exposure of 60 days.

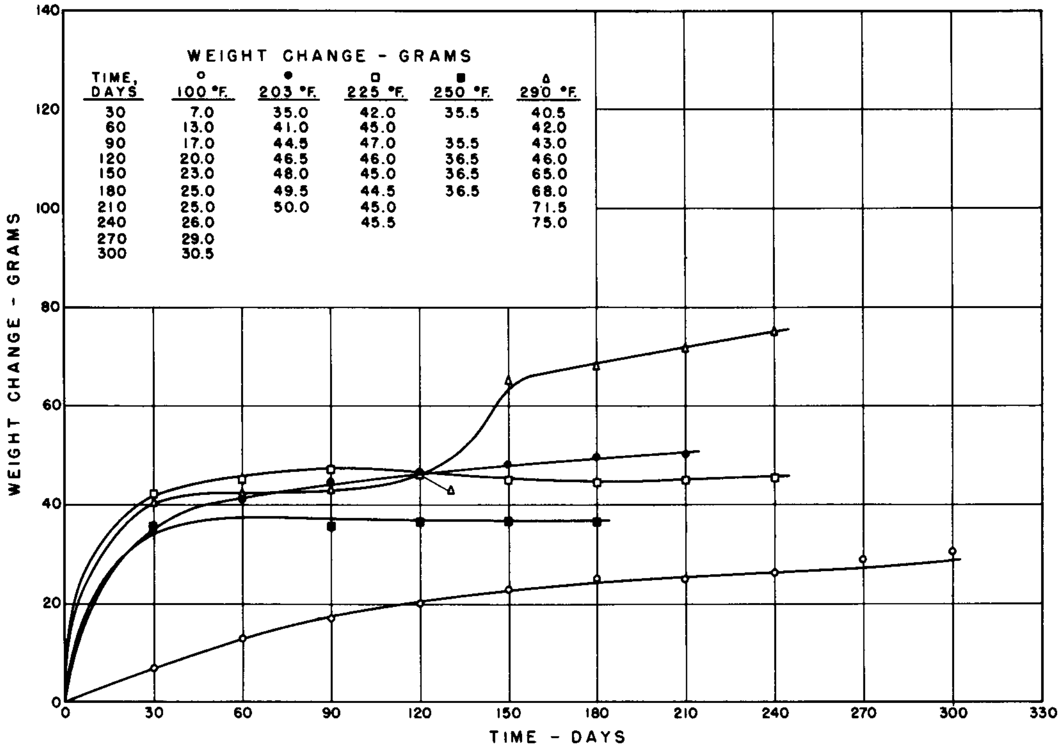
V.5.1.18. Rubber R-31.--

No results available at report time.

V.5.2. Coatings

V.5.2.1. Coatings C-1 and C-4.--

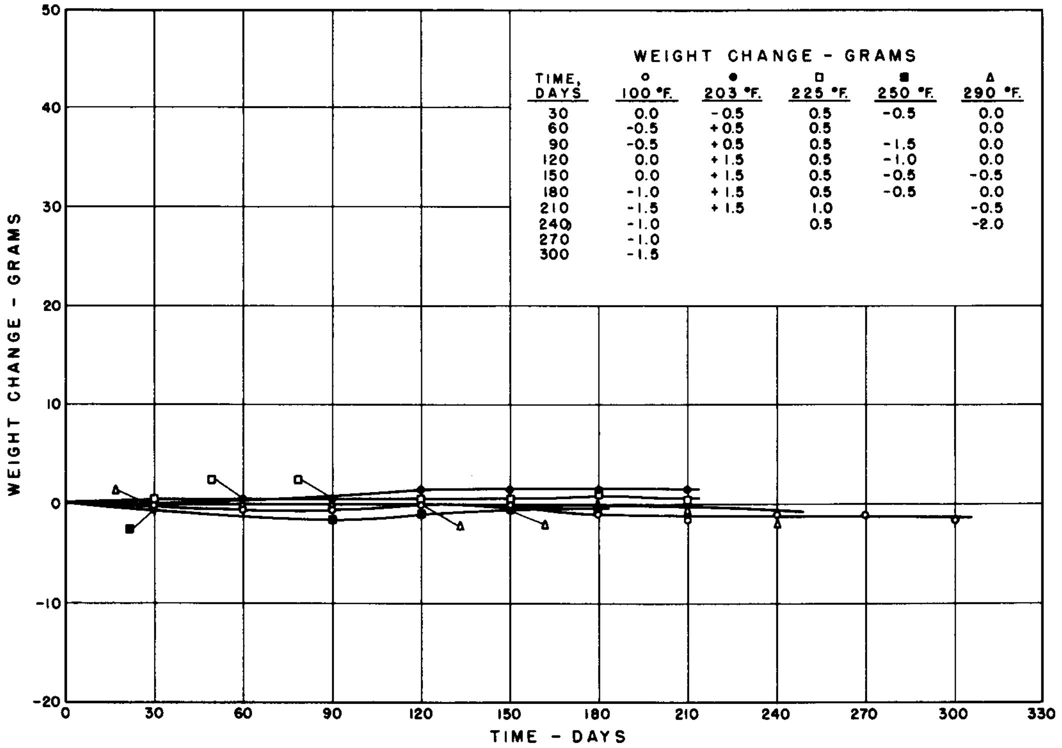
As shown in Figure V-24A on concrete, Coating C-1, neoprene, shows a considerable change in weight for the first 30 days in the 203°, 225°, 250°, and 290° F loops. A plateau in the rate of weight change was reached at about 60 days. On the 290° F specimen, a blister appeared accounting for the sharp rise in the weight change from the previous 90 days. The 100° F specimen weight continues to change as it approaches the plateau of the higher temperatures. On the steel specimens, the weight change is small as shown in Figure V-24B, with temperature and concentrations having little effect. To date the performance of the C-1 neoprene has been satisfactory on concrete up to 250° and to 290° F on steel. Coating C-4, similar to C-1 except for accelerator and heat curing, presented some application problems; however, the difficulties have been solved. Exposure on Coating C-4 tests will begin in June.



WEIGHT CHANGE - GRAMS

TIME, DAYS	○ 100 °F.	● 203 °F.	□ 225 °F.	■ 250 °F.	△ 290 °F.
30	7.0	35.0	42.0	35.5	40.5
60	13.0	41.0	45.0		42.0
90	17.0	44.5	47.0	35.5	43.0
120	20.0	46.5	46.0	36.5	46.0
150	23.0	48.0	45.0	36.5	65.0
180	25.0	49.5	44.5	36.5	68.0
210	25.0	50.0	45.0		71.5
240	26.0		45.5		75.0
270	29.0				
300	30.5				

A. ON CONCRETE



WEIGHT CHANGE - GRAMS

TIME, DAYS	○ 100 °F.	● 203 °F.	□ 225 °F.	■ 250 °F.	△ 290 °F.
30	0.0	- 0.5	0.5	-0.5	0.0
60	-0.5	+ 0.5	0.5		0.0
90	-0.5	+ 0.5	0.5	- 1.5	0.0
120	0.0	+ 1.5	0.5	- 1.0	0.0
150	0.0	+ 1.5	0.5	-0.5	-0.5
180	-1.0	+ 1.5	0.5	-0.5	0.0
210	- 1.5	+ 1.5	1.0		-0.5
240	- 1.0		0.5		-2.0
270	- 1.0				
300	- 1.5				

B. ON STEEL

COATING C - I - WEIGHT CHANGE VS TIME
FIGURE V-24

V. 5. 2. 2. Coatings C-2 and C-3. --

Coatings C-2 and C-3, epoxy-phenolics, shown in Figures V-25 and V-26, plateaued in 60 days for 203°, 225°, 250°, and 290° F at about 48 grams of weight change on concrete. At 210 days some cracking was noted with sharp changes in weight for the 290° F specimens. On steel the weight changes are small and may be caused by very slight delamination. The performance of the epoxy-phenolic coatings is satisfactory so far on concrete and on steel.

V. 5. 2. 3. Coating C-5. --

Figure V-27B shows for Coating C-5 a neoprene on steel that environments with atmospheric pressure or above result in positive weight changes. The environment with vacuum shows a negative weight change. On concrete, the 203° F specimen developed a large blister in the first 30 days and thus plateaued several grams higher than the 225°, 250°, and 290° F loop specimens. Coating C-5 appears to be performing well to date, discounting the early appearance of the blister.

V. 5. 2. 4. Coatings C-6 and C-7. --

Coatings C-6 and C-7, chlorosulfonated polyethylene, on concrete and steel have shown very poor performance at all temperatures from 203° F and above as indicated by Figures V-28 and V-29. The coatings appear to be deteriorating at 203° F after 210 days on concrete, but appear better on steel at that point. The coatings show little effect from exposure at 100° F for 300 days on concrete and steel.

V. 5. 2. 5. Coating C-8. --

Resin varnish Coating C-8 on concrete, as shown in Figure V-30A, has had intercoat adhesion failure with delamination between coats at 203°, 225°, 250°, and 290° F. Blisters and cracks were evident on the concrete specimens at 225°, 250°, and 290° F in 150 days. The concrete specimen at 100° F is performing satisfactorily. Coating C-8 on steel appears to be doing a satisfactory job of protecting the steel substrate at all temperatures.

V. 5. 2. 6. Coating C-9. --

Phenolic Coating C-9 on concrete, as shown in Figure V-31A, has performed satisfactorily. The 290° F specimen showed a crack at 150 days and was repaired at 180 days which accounts for the sharp increase in weight at 210 days. The 250° F specimen has a negative weight change which is presently not fully explainable but thought to be related to loss due to volatile. C-9 is performing satisfactorily on steel at all temperatures under test.

V. 5. 2. 7 Coatings C-10, C-21, and C-22. --

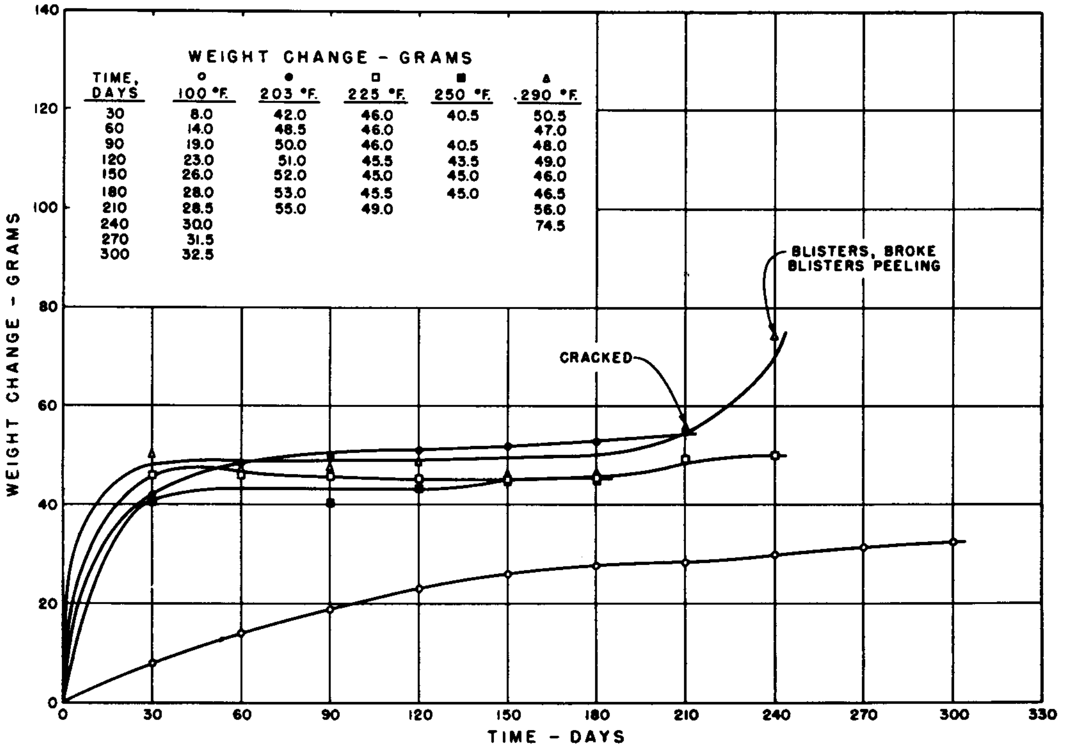
Vinyl coatings C-10, C-21, and C-22, shown in Figures V-32, V-43, and V-44, were only exposed at 100° F. On concrete the weight changes are nearly the same with satisfactory performance to date. There are nearly no weight changes on steel for these three vinyl coatings and they have shown excellent performance so far for the 100° F exposure.

V. 5. 2. 8. Coating C-11. --

Bituminous, Coating C-11, has performed poorly on concrete, Figure V-33. It has numerous pinhead blisters. At 290° F, the blisters have broken exposing the concrete and causing large weight changes. On steel the coating has blistered and peeled at 100° F in 150 days, but at 203°, 225°, 250°, and 290° F, this coating apparently is performing satisfactorily.

V. 5. 2. 9. Coatings C-12, C-13, and C-32. --

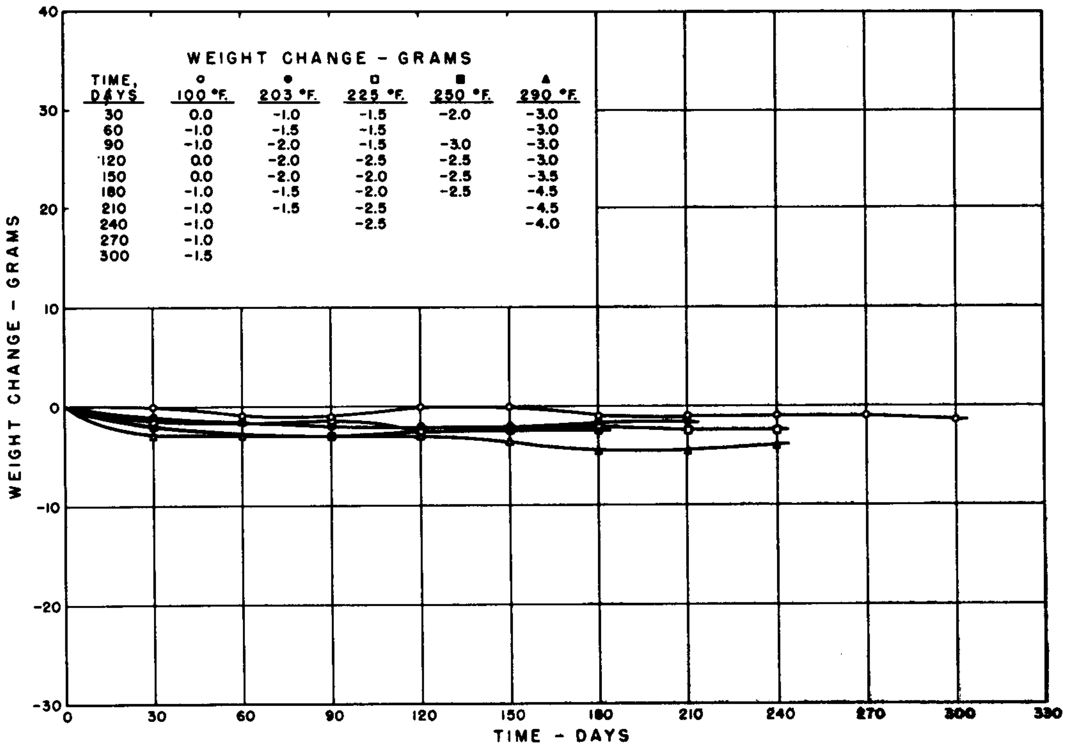
Coatings C-12, C-13, and C-32 are phenolics and applied only on steel. As shown in Figures V-34, V-35, and V-48, there are little weight changes. Coatings C-12



WEIGHT CHANGE - GRAMS

TIME, DAYS	○ 100 °F.	● 203 °F.	□ 225 °F.	■ 250 °F.	▲ 290 °F.
30	8.0	42.0	46.0	40.5	50.5
60	14.0	48.5	46.0		47.0
90	19.0	50.0	46.0	40.5	48.0
120	23.0	51.0	45.5	43.5	49.0
150	26.0	52.0	45.0	45.0	46.0
180	28.0	53.0	45.5	45.0	46.5
210	28.5	55.0	49.0		56.0
240	30.0				74.5
270	31.5				
300	32.5				

A. ON CONCRETE

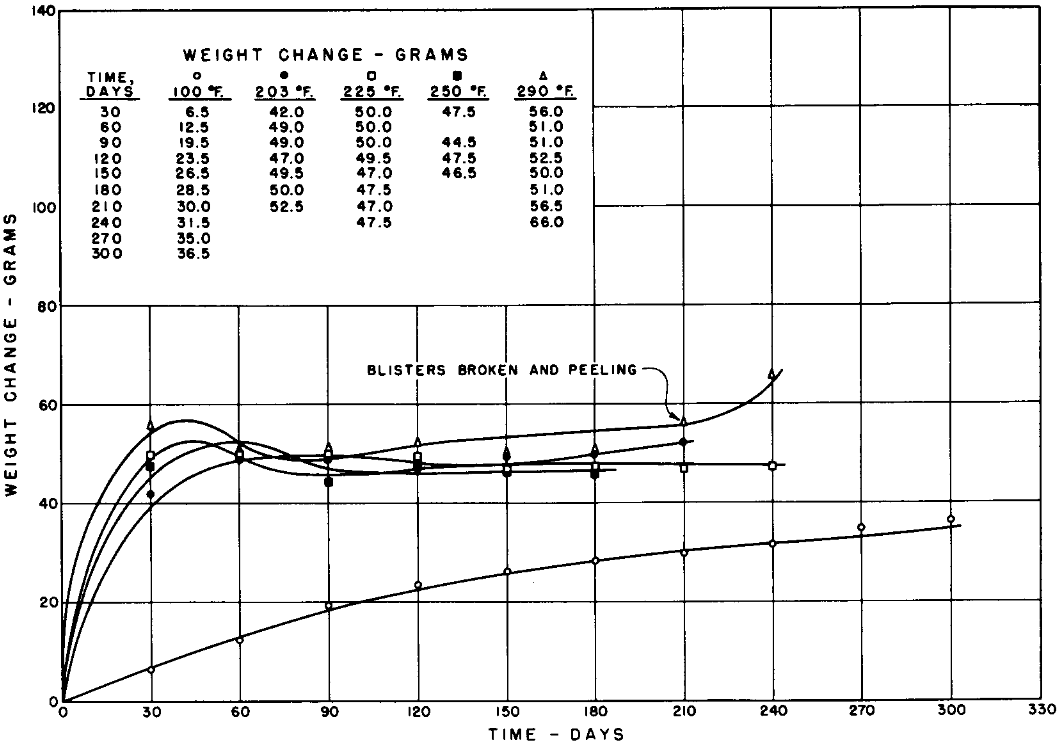


WEIGHT CHANGE - GRAMS

TIME, DAYS	○ 100 °F.	● 203 °F.	□ 225 °F.	■ 250 °F.	▲ 290 °F.
30	0.0	-1.0	-1.5	-2.0	-3.0
60	-1.0	-1.5	-1.5		-3.0
90	-1.0	-2.0	-1.5	-3.0	-3.0
120	0.0	-2.0	-2.5	-2.5	-3.0
150	0.0	-2.0	-2.0	-2.5	-3.5
180	-1.0	-1.5	-2.0	-2.5	-4.5
210	-1.0	-1.5	-2.5		-4.5
240	-1.0		-2.5		-4.0
270	-1.0				
300	-1.5				

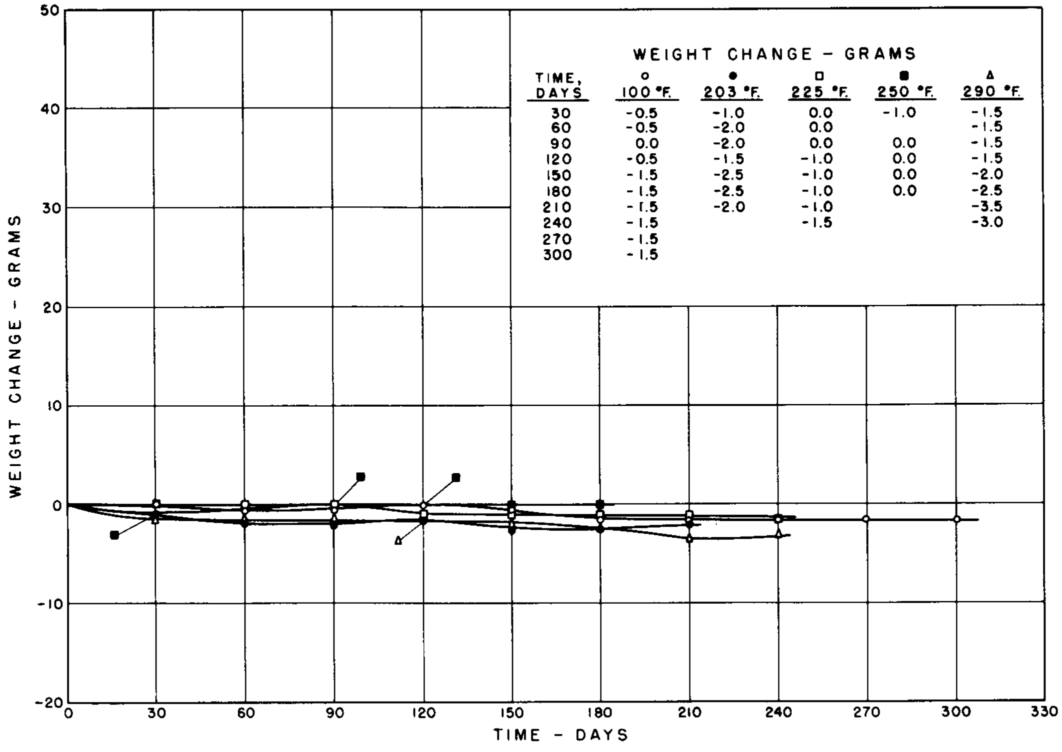
B. ON STEEL

COATING C-2 - WEIGHT CHANGE VS TIME
FIGURE V - 25



TIME, DAYS	WEIGHT CHANGE - GRAMS				
	○ 100 °F.	● 203 °F.	□ 225 °F.	■ 250 °F.	△ 290 °F.
30	6.5	42.0	50.0	47.5	56.0
60	12.5	49.0	50.0		51.0
90	19.5	49.0	50.0	44.5	51.0
120	23.5	47.0	49.5	47.5	52.5
150	26.5	49.5	47.0	46.5	50.0
180	28.5	50.0	47.5		51.0
210	30.0	52.5	47.0		56.5
240	31.5		47.5		66.0
270	35.0				
300	36.5				

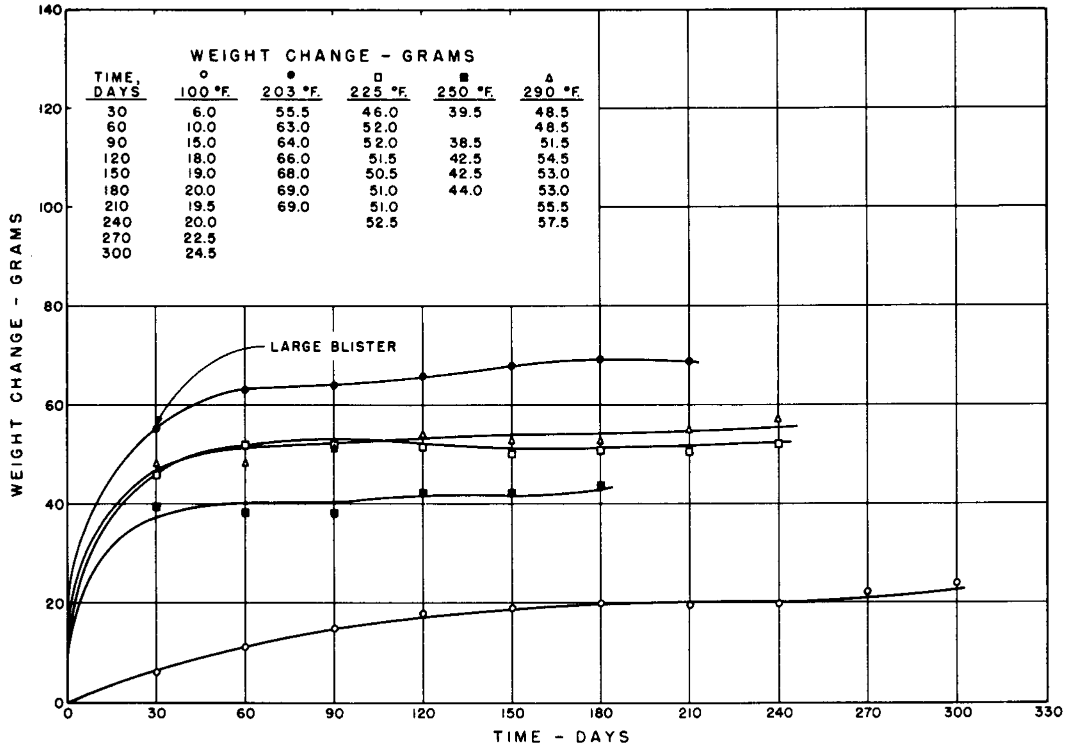
A. ON CONCRETE



TIME, DAYS	WEIGHT CHANGE - GRAMS				
	○ 100 °F.	● 203 °F.	□ 225 °F.	■ 250 °F.	△ 290 °F.
30	-0.5	-1.0	0.0	-1.0	-1.5
60	-0.5	-2.0	0.0		-1.5
90	0.0	-2.0	0.0	0.0	-1.5
120	-0.5	-1.5	-1.0	0.0	-1.5
150	-1.5	-2.5	-1.0	0.0	-2.0
180	-1.5	-2.5	-1.0	0.0	-2.5
210	-1.5	-2.0	-1.0		-3.5
240	-1.5		-1.5		-3.0
270	-1.5				
300	-1.5				

B. ON STEEL

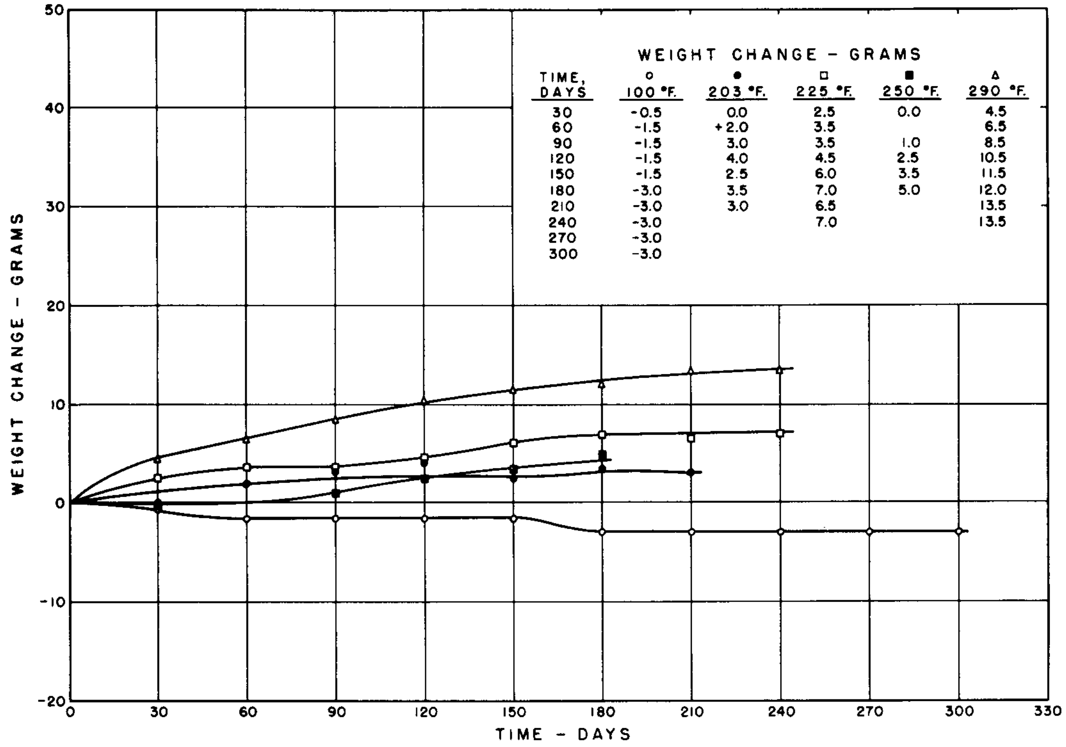
COATING C-3 - WEIGHT CHANGE VS TIME
FIGURE V-26



WEIGHT CHANGE - GRAMS

TIME, DAYS	○ 100 °F.	● 203 °F.	□ 225 °F.	■ 250 °F.	△ 290 °F.
30	6.0	55.5	46.0	39.5	48.5
60	10.0	63.0	52.0		48.5
90	15.0	64.0	52.0	38.5	51.5
120	18.0	66.0	51.5	42.5	54.5
150	19.0	68.0	50.5	42.5	53.0
180	20.0	69.0	51.0	44.0	53.0
210	19.5	69.0	51.0		55.5
240	20.0		52.5		57.5
270	22.5				
300	24.5				

A. ON CONCRETE

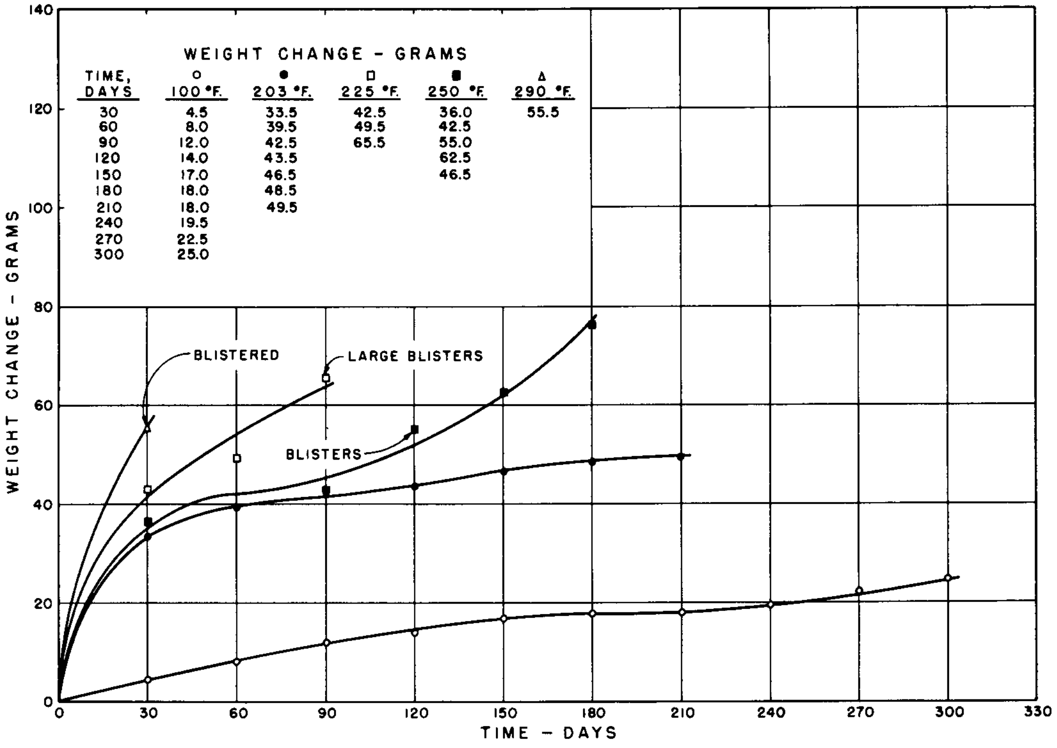


WEIGHT CHANGE - GRAMS

TIME, DAYS	○ 100 °F.	● 203 °F.	□ 225 °F.	■ 250 °F.	△ 290 °F.
30	-0.5	0.0	2.5	0.0	4.5
60	-1.5	+2.0	3.5		6.5
90	-1.5	3.0	3.5	1.0	8.5
120	-1.5	4.0	4.5	2.5	10.5
150	-1.5	2.5	6.0	3.5	11.5
180	-3.0	3.5	7.0	5.0	12.0
210	-3.0	3.0	6.5		13.5
240	-3.0		7.0		13.5
270	-3.0				
300	-3.0				

B. ON STEEL

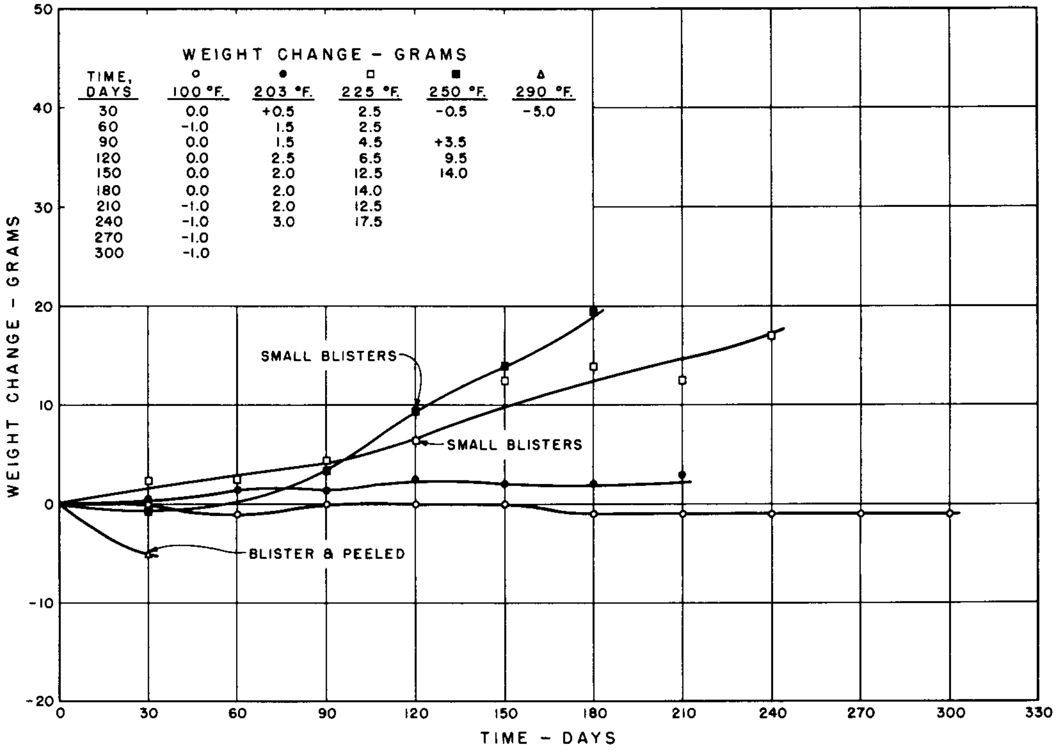
COATING C-5 - WEIGHT CHANGE VS TIME
FIGURE V - 27



WEIGHT CHANGE - GRAMS

TIME, DAYS	○ 100 °F.	● 203 °F.	□ 225 °F.	■ 250 °F.	△ 290 °F.
30	4.5	33.5	42.5	36.0	55.5
60	8.0	39.5	49.5	42.5	
90	12.0	42.5	65.5	55.0	
120	14.0	43.5		62.5	
150	17.0	46.5		46.5	
180	18.0	48.5			
210	18.0	49.5			
240	19.5				
270	22.5				
300	25.0				

A. ON CONCRETE



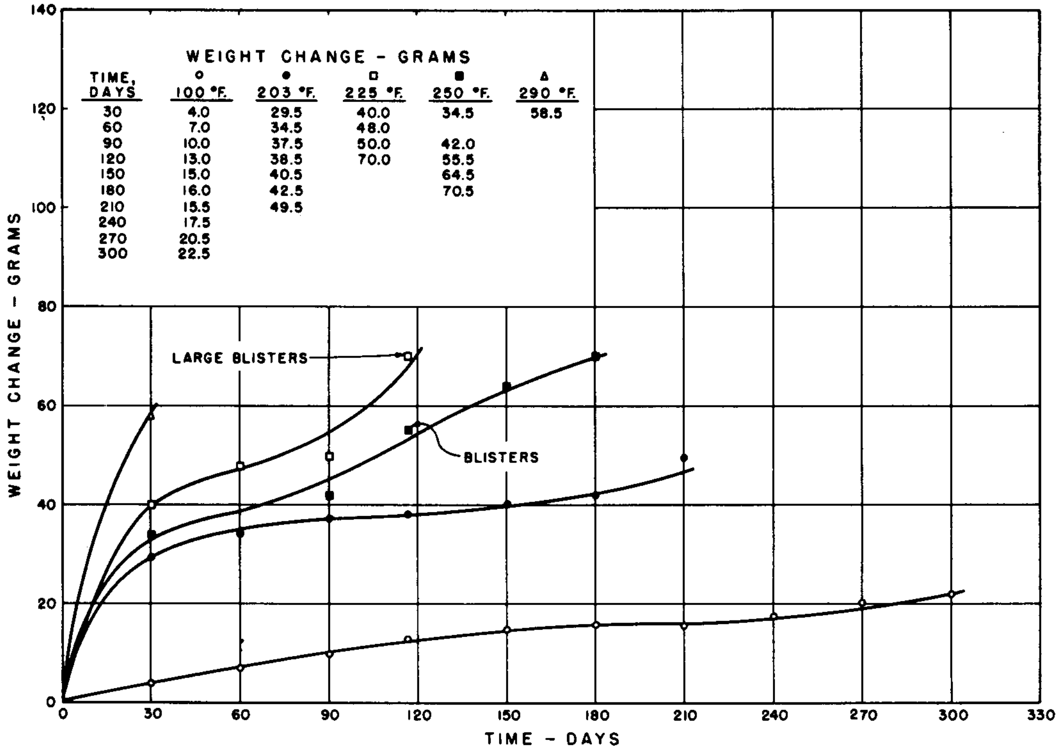
WEIGHT CHANGE - GRAMS

TIME, DAYS	○ 100 °F.	● 203 °F.	□ 225 °F.	■ 250 °F.	△ 290 °F.
30	0.0	+0.5	2.5	-0.5	-5.0
60	-1.0	1.5	2.5		
90	0.0	1.5	4.5	+3.5	
120	0.0	2.5	6.5	9.5	
150	0.0	2.0	12.5	14.0	
180	0.0	2.0	14.0		
210	-1.0	2.0	12.5		
240	-1.0	3.0	17.5		
270	-1.0				
300	-1.0				

B. ON STEEL

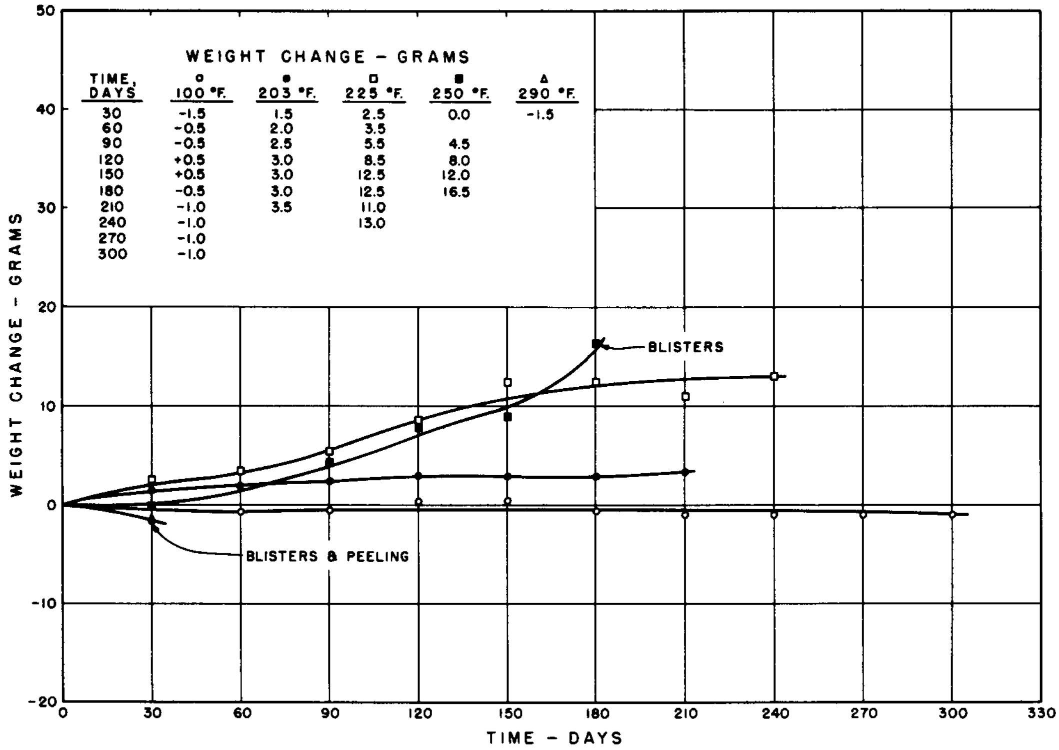
COATING C-6 - WEIGHT CHANGE VS TIME

FIGURE V - 28



TIME, DAYS	WEIGHT CHANGE - GRAMS				
	○ 100 °F.	● 203 °F.	□ 225 °F.	■ 250 °F.	△ 290 °F.
30	4.0	29.5	40.0	34.5	58.5
60	7.0	34.5	48.0		
90	10.0	37.5	50.0	42.0	
120	13.0	38.5	70.0	55.5	
150	15.0	40.5		64.5	
180	16.0	42.5		70.5	
210	15.5	49.5			
240	17.5				
270	20.5				
300	22.5				

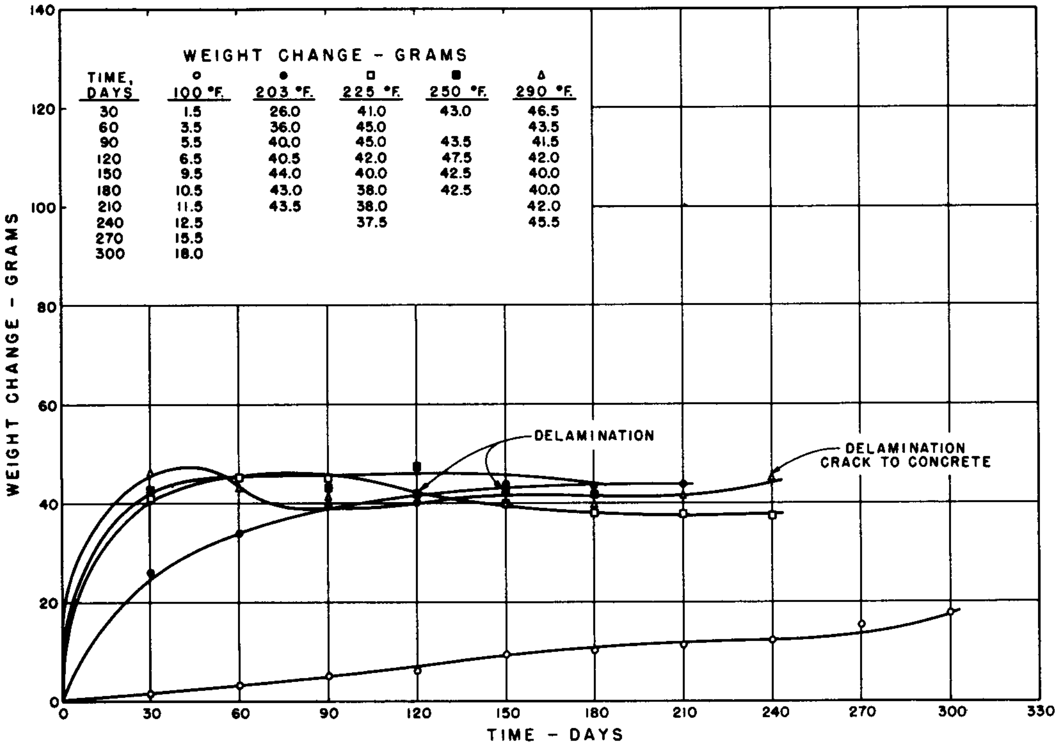
A. ON CONCRETE



TIME, DAYS	WEIGHT CHANGE - GRAMS				
	○ 100 °F.	● 203 °F.	□ 225 °F.	■ 250 °F.	△ 290 °F.
30	-1.5	1.5	2.5	0.0	-1.5
60	-0.5	2.0	3.5		
90	-0.5	2.5	5.5	4.5	
120	+0.5	3.0	8.5	8.0	
150	+0.5	3.0	12.5	12.0	
180	-0.5	3.0	12.5	16.5	
210	-1.0	3.5	11.0		
240	-1.0		13.0		
270	-1.0				
300	-1.0				

B. ON STEEL

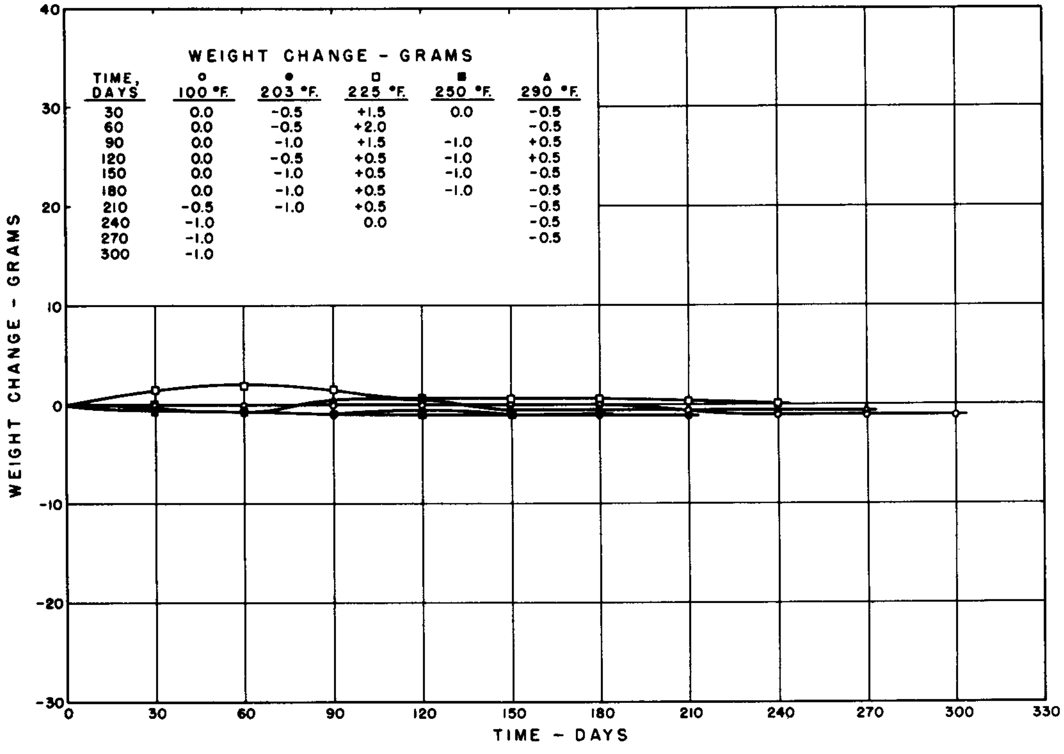
COATING C-7 - WEIGHT CHANGE VS TIME
FIGURE V - 29



WEIGHT CHANGE - GRAMS

TIME, DAYS	○ 100 °F.	● 203 °F.	□ 225 °F.	■ 250 °F.	△ 290 °F.
30	1.5	26.0	41.0	43.0	46.5
60	3.5	36.0	45.0		43.5
90	5.5	40.0	45.0	43.5	41.5
120	6.5	40.5	42.0	47.5	42.0
150	9.5	44.0	40.0	42.5	40.0
180	10.5	43.0	38.0	42.5	40.0
210	11.5	43.5	38.0		42.0
240	12.5		37.5		45.5
270	15.5				
300	18.0				

A. ON CONCRETE

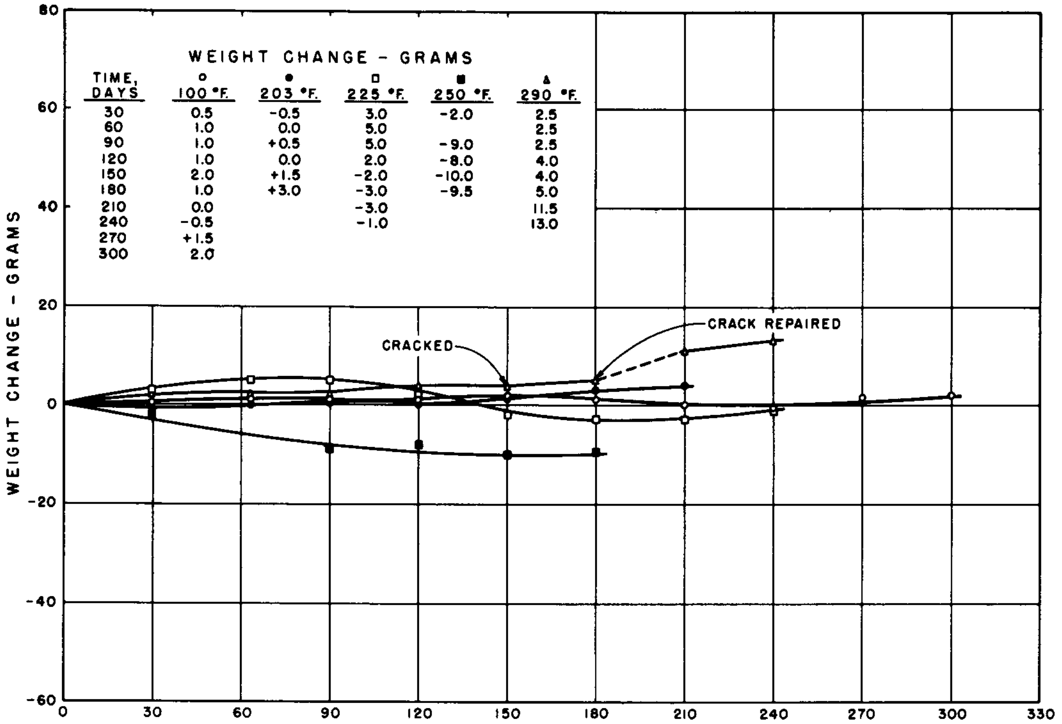


WEIGHT CHANGE - GRAMS

TIME, DAYS	○ 100 °F.	● 203 °F.	□ 225 °F.	■ 250 °F.	△ 290 °F.
30	0.0	-0.5	+1.5	0.0	-0.5
60	0.0	-0.5	+2.0		-0.5
90	0.0	-1.0	+1.5	-1.0	+0.5
120	0.0	-0.5	+0.5	-1.0	+0.5
150	0.0	-1.0	+0.5	-1.0	-0.5
180	0.0	-1.0	+0.5	-1.0	-0.5
210	-0.5	-1.0	+0.5		-0.5
240	-1.0		0.0		-0.5
270	-1.0				-0.5
300	-1.0				

B. ON STEEL

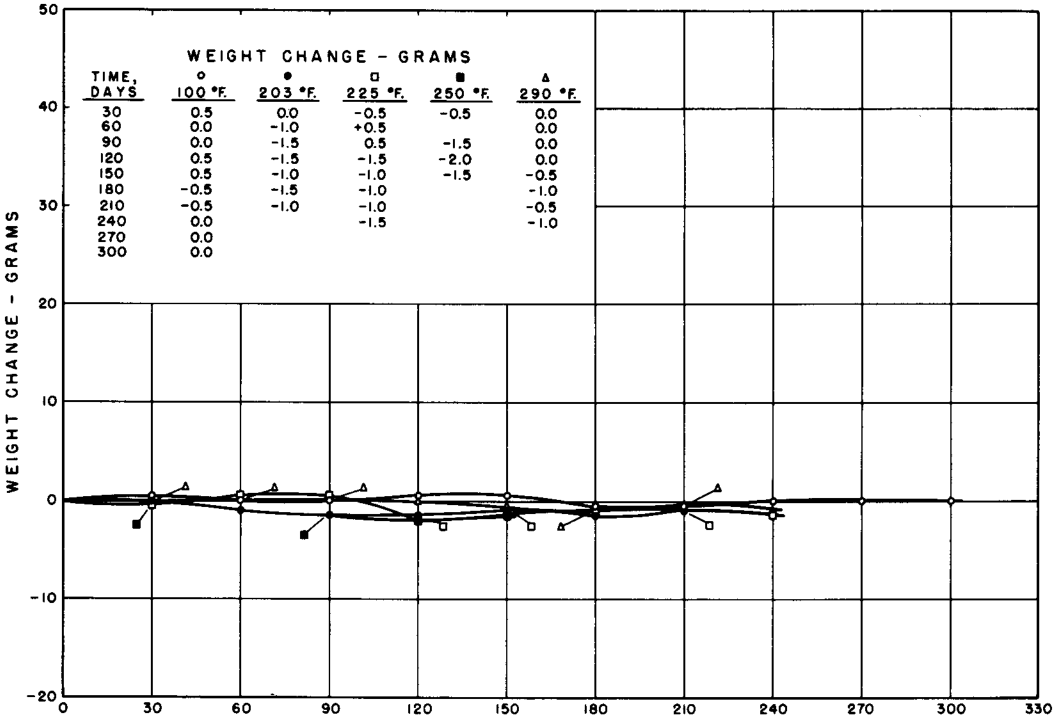
COATING C-8 - WEIGHT CHANGE VS TIME
FIGURE V - 30



WEIGHT CHANGE - GRAMS

TIME, DAYS	○ 100 °F.	● 203 °F.	□ 225 °F.	■ 250 °F.	▲ 290 °F.
30	0.5	-0.5	3.0	-2.0	2.5
60	1.0	0.0	5.0		2.5
90	1.0	+0.5	5.0	-9.0	2.5
120	1.0	0.0	2.0	-8.0	4.0
150	2.0	+1.5	-2.0	-10.0	4.0
180	1.0	+3.0	-3.0	-9.5	5.0
210	0.0		-3.0		11.5
240	-0.5		-1.0		13.0
270	+1.5				
300	2.0				

A. ON CONCRETE

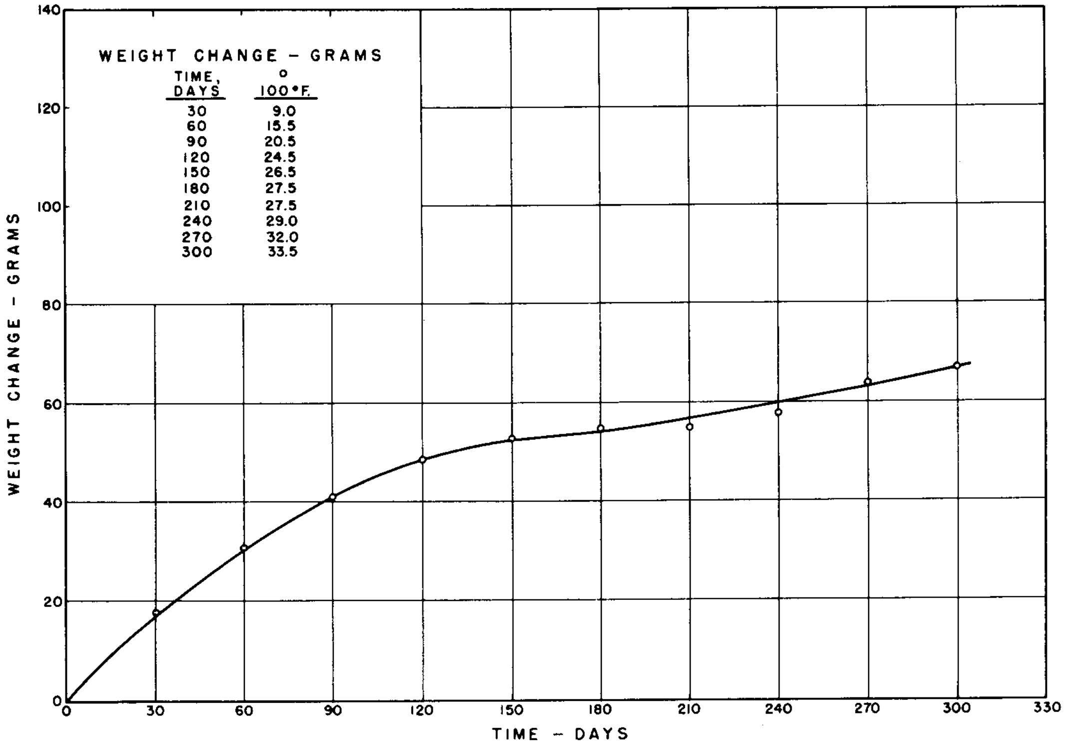


WEIGHT CHANGE - GRAMS

TIME, DAYS	○ 100 °F.	● 203 °F.	□ 225 °F.	■ 250 °F.	▲ 290 °F.
30	0.5	0.0	-0.5	-0.5	0.0
60	0.0	-1.0	+0.5		0.0
90	0.0	-1.5	0.5	-1.5	0.0
120	0.5	-1.5	-1.5	-2.0	0.0
150	0.5	-1.0	-1.0	-1.5	-0.5
180	-0.5	-1.5	-1.0		-1.0
210	-0.5	-1.0	-1.0		-0.5
240	0.0		-1.5		-1.0
270	0.0				
300	0.0				

B. ON STEEL

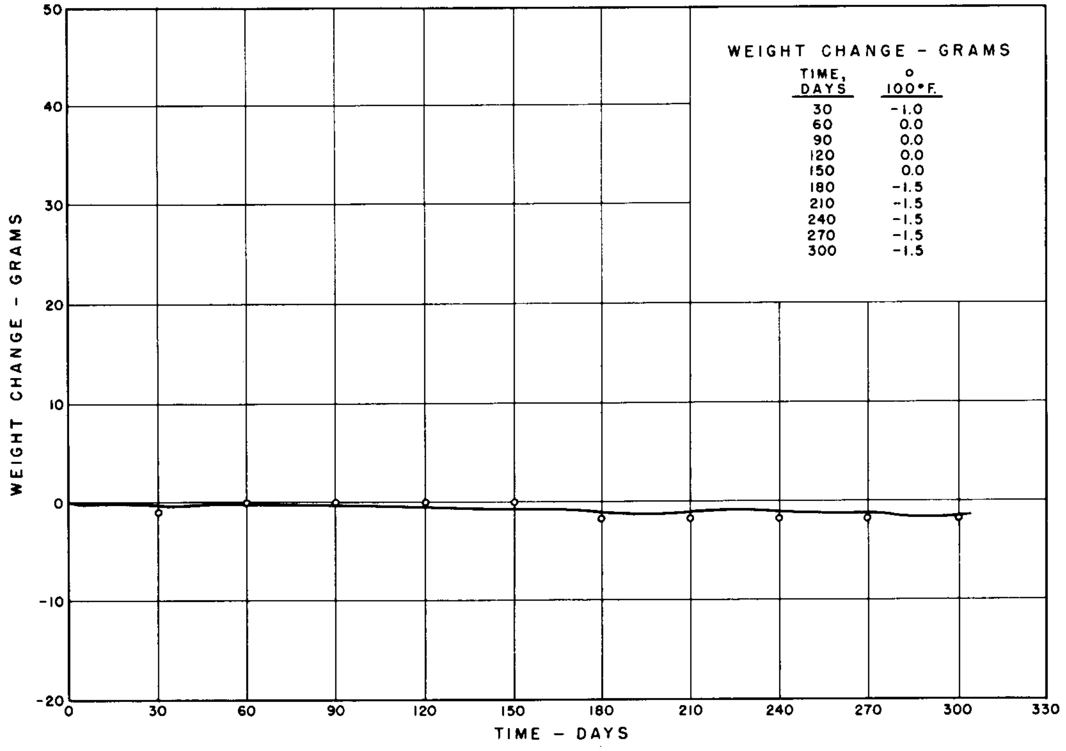
COATING C-9 - WEIGHT CHANGE VS TIME
FIGURE V - 31



WEIGHT CHANGE - GRAMS

TIME, DAYS	○ 100°F.
30	9.0
60	15.5
90	20.5
120	24.5
150	26.5
180	27.5
210	27.5
240	29.0
270	32.0
300	33.5

A. ON CONCRETE

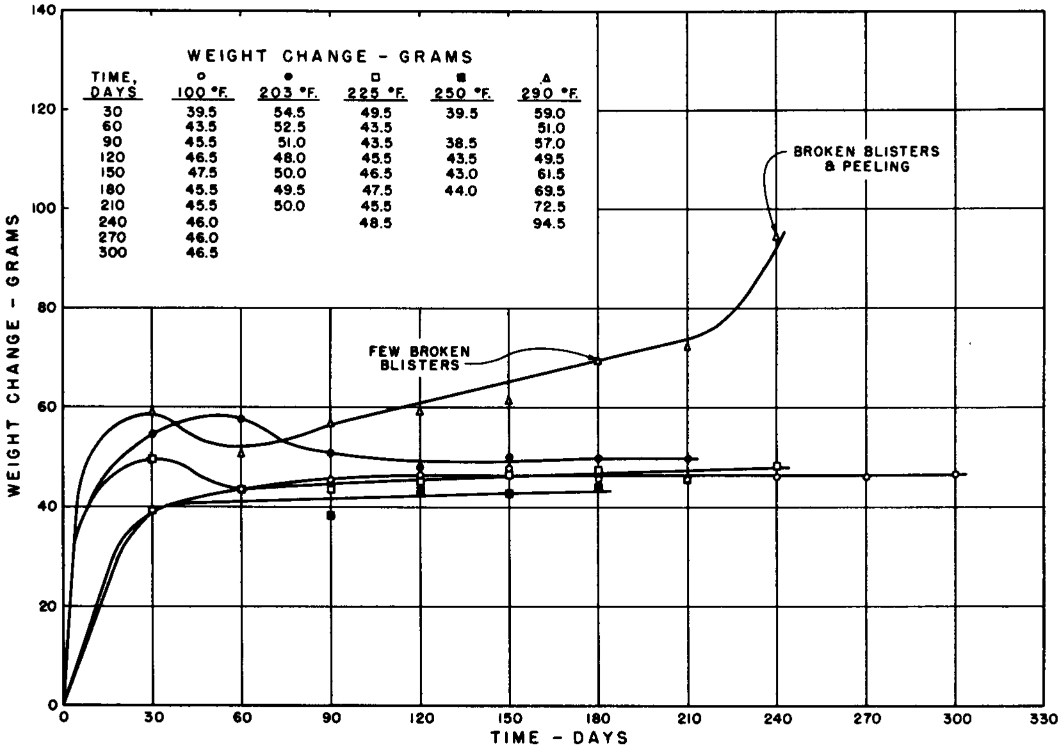


WEIGHT CHANGE - GRAMS

TIME, DAYS	○ 100°F.
30	-1.0
60	0.0
90	0.0
120	0.0
150	0.0
180	-1.5
210	-1.5
240	-1.5
270	-1.5
300	-1.5

B. ON STEEL

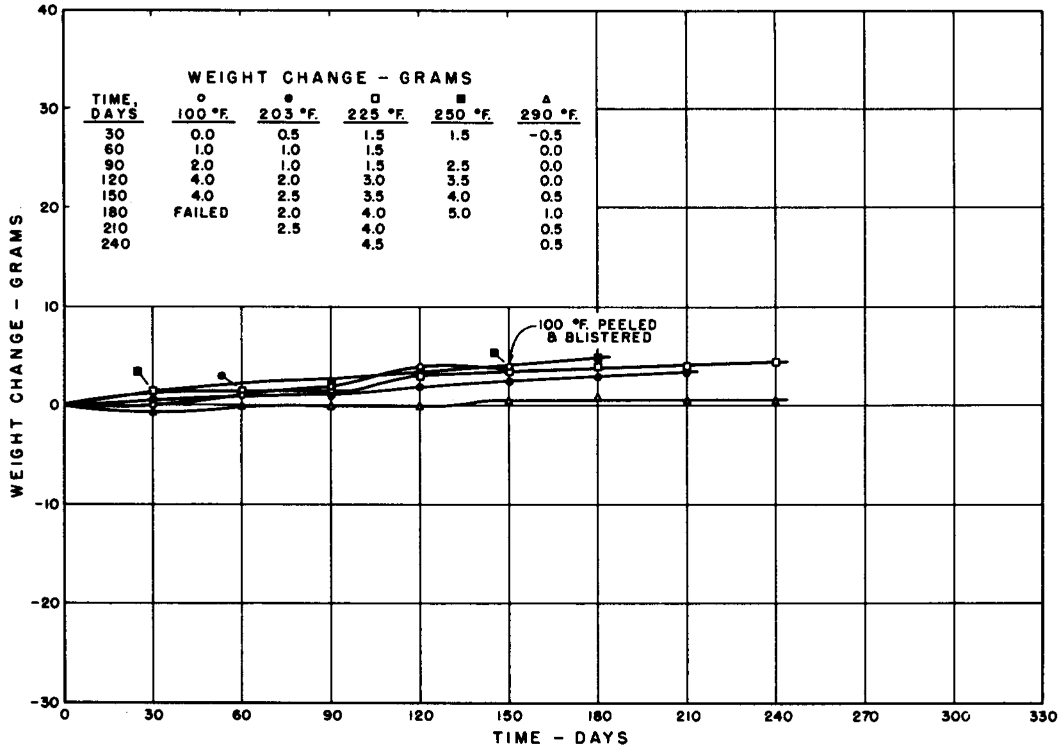
COATING C-10 - WEIGHT CHANGE VS TIME
FIGURE V-32



WEIGHT CHANGE - GRAMS

TIME, DAYS	○ 100 °F.	● 203 °F.	□ 225 °F.	■ 250 °F.	▲ 290 °F.
30	39.5	54.5	49.5	39.5	59.0
60	43.5	52.5	43.5		51.0
90	45.5	51.0	43.5	38.5	57.0
120	46.5	48.0	45.5	43.5	49.5
150	47.5	50.0	46.5	43.0	61.5
180	45.5	49.5	47.5	44.0	69.5
210	45.5	50.0	45.5		72.5
240	46.0		48.5		94.5
270	46.0				
300	46.5				

A. ON CONCRETE

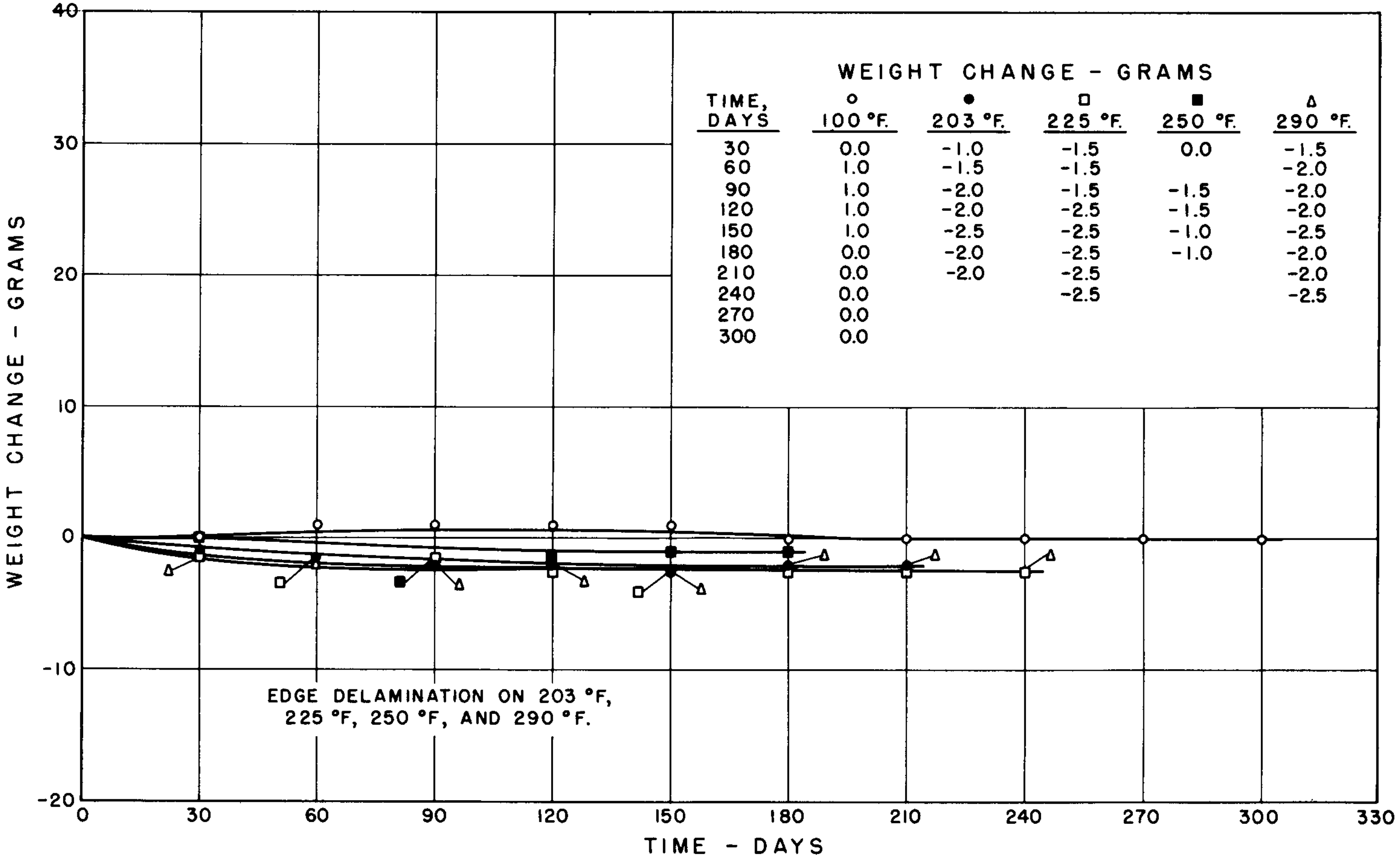


WEIGHT CHANGE - GRAMS

TIME, DAYS	○ 100 °F.	● 203 °F.	□ 225 °F.	■ 250 °F.	▲ 290 °F.
30	0.0	0.5	1.5	1.5	-0.5
60	1.0	1.0	1.5		0.0
90	2.0	1.0	1.5	2.5	0.0
120	4.0	2.0	3.0	3.5	0.0
150	4.0	2.5	3.5	4.0	0.5
180	FAILED	2.0	4.0	5.0	1.0
210		2.5	4.0		0.5
240			4.5		0.5

B. ON STEEL

COATING C-11 - WEIGHT CHANGE VS TIME
FIGURE V - 33

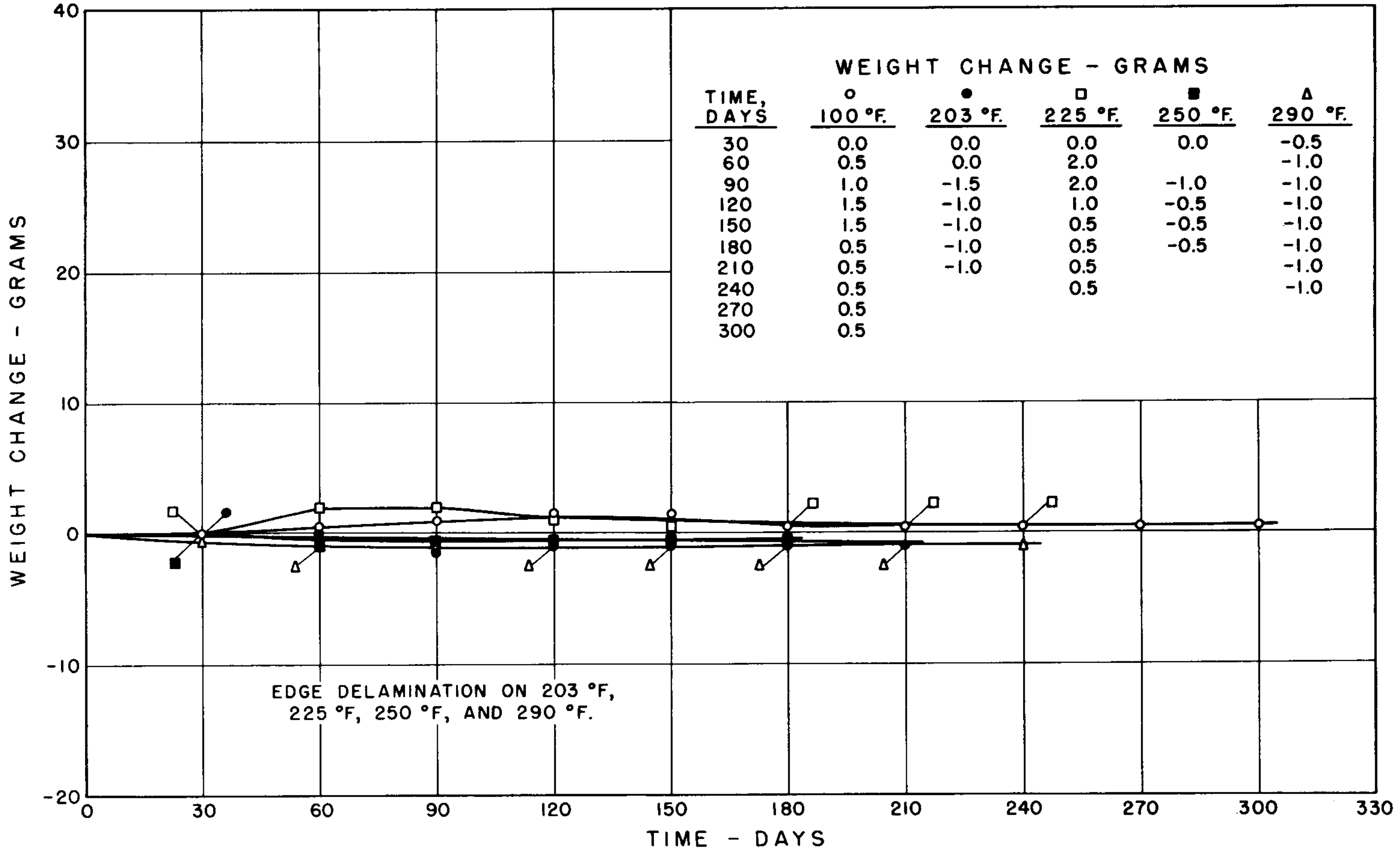


WEIGHT CHANGE - GRAMS

TIME, DAYS	○ 100 °F.	● 203 °F.	□ 225 °F.	■ 250 °F.	Δ 290 °F.
30	0.0	-1.0	-1.5	0.0	-1.5
60	1.0	-1.5	-1.5		-2.0
90	1.0	-2.0	-1.5	-1.5	-2.0
120	1.0	-2.0	-2.5	-1.5	-2.0
150	1.0	-2.5	-2.5	-1.0	-2.5
180	0.0	-2.0	-2.5	-1.0	-2.0
210	0.0	-2.0	-2.5		-2.0
240	0.0		-2.5		-2.5
270	0.0				
300	0.0				

ON STEEL

COATING C-12 - WEIGHT CHANGE VS TIME
FIGURE V-34



WEIGHT CHANGE - GRAMS

TIME, DAYS	○ 100 °F.	● 203 °F.	□ 225 °F.	■ 250 °F.	Δ 290 °F.
30	0.0	0.0	0.0	0.0	-0.5
60	0.5	0.0	2.0		-1.0
90	1.0	-1.5	2.0	-1.0	-1.0
120	1.5	-1.0	1.0	-0.5	-1.0
150	1.5	-1.0	0.5	-0.5	-1.0
180	0.5	-1.0	0.5	-0.5	-1.0
210	0.5	-1.0	0.5		-1.0
240	0.5		0.5		-1.0
270	0.5				
300	0.5				

ON STEEL

COATING C-13 - WEIGHT CHANGE VS TIME
FIGURE V-35

and C-13 have shown edge delamination of the top coat. All three coatings are showing good performance at all temperatures under test.

V.5.2.10. Coatings C-14, C-17, and C-18. --

Coal-tar epoxy coatings, C-14, C-17, and C-18, shown in Figures V-36, V-39, and V-40, are applied on both concrete and steel for temperatures of 100°, 203°, 225°, and 250° F. All failed in 30 days at 250° F on concrete. Coating C-14 has failed at 203° F in 30 days and 225° F in 60 days. Coating C-17 on concrete is still performing satisfactorily at 203° and 225° F. Coating C-18 failed in 60 days on concrete at 225° F, but is still performing satisfactorily at 203° F. Each coating on steel shows large negative weight changes for 203°, 225°, and 250° F. This is probably due to losses of oils from the coal tar. However, all three coatings apparently are still performing well on steel.

V.5.2.11. Coating C-15. --

Coating C-15, a polyester, performed unsatisfactorily at 203°, 225°, and 250° F on both steel and concrete as shown in Figure V-37. The polyester blistered and peeled from both the concrete and steel at these temperatures. It appears to be performing well at 100° F on the steel and concrete.

V.5.2.12. Coating C-16. --

EPT (ethylene-propylene terpolymer), Coating C-16, peeled off both concrete and steel at 100° F, Figure V-38. The coating was EPT in solvent which did not cure. A soft film resulted which had very poor performance.

V.5.2.13. Coating C-19. --

Coating C-19, hexafluoropropylene-vinylidene fluoride in solvent over an epoxy primer, shown in Figure V-41, blistered on concrete at 203° F and 60 days, but has shown no other defects at this temperature with continued exposure. At 290° F on concrete, this coating showed broken blisters (three tear spots) at 90 days. At 240 days at 290° F severe peeling from the broken blisters has caused failure of the coating. Also at 250° F at 120 days the coating has begun peeling. Performance is fair on concrete at 225° and 100° F. On steel C-19 looks good at all temperatures under test.

V.5.2.14. Coating C-20. --

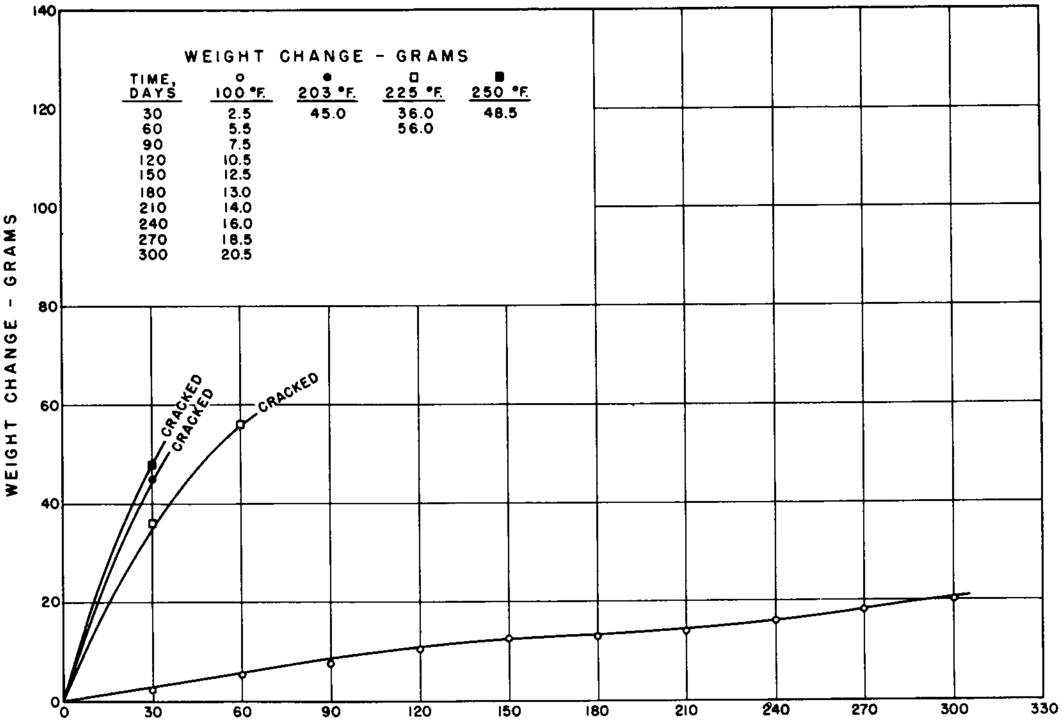
Neoprene-asphalt, Coating C-20, has reached plateaus on weight change in 60 days at 225°, 250°, and 290° F on concrete. After reaching these plateaus, the weight change has held nearly constant as shown by Figure V-42. At 203° and 100° F, the weight change has not plateaued, but still continues to rise. For temperatures of 203° F and above, each specimen showed surface sags after the first 30 days which did not increase with continued exposure. The sags appeared on the steel as well as the concrete. The neoprene-asphalt appears to be providing good protection for the substrates at all temperatures under test.

V.5.2.15. Coating C-23. --

C-23 is the same material as C-19 except without a primer. On concrete Coating C-23 started blistering and peeling in the first 60 to 90 days for the 203°, 225°, 250° and 290° F as shown by Figure V-45. As indicated in Figure V-45A on concrete the coating test was discontinued when blistering and peeling became severe. C-23 appears to be satisfactory on concrete at 100° F as well as on steel at all temperatures tested to date.

V.5.2.16. Coating C-24. --

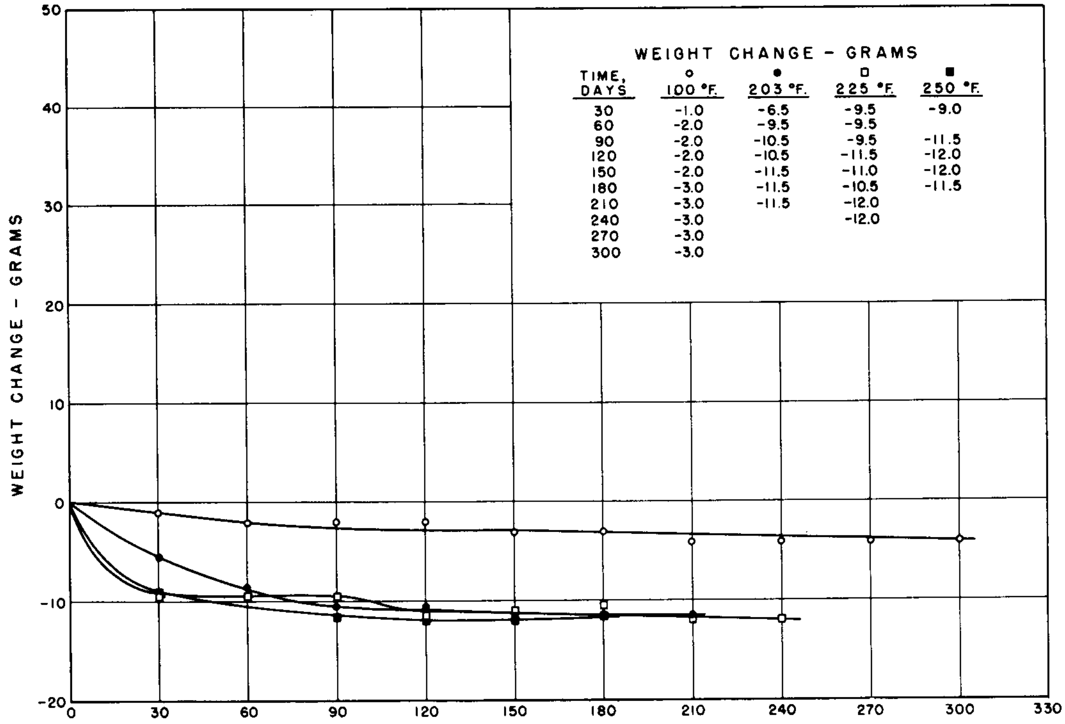
Coating C-24, a glass reinforced polyester, has performed unsatisfactorily as shown in Figure V-46. The coating showed cracking and peeling in the first 30 days on both steel and concrete.



WEIGHT CHANGE - GRAMS

TIME, DAYS	○ 100 °F.	● 203 °F.	□ 225 °F.	■ 250 °F.
30	2.5	45.0	36.0	48.5
60	5.5		56.0	
90	7.5			
120	10.5			
150	12.5			
180	13.0			
210	14.0			
240	16.0			
270	18.5			
300	20.5			

A. ON CONCRETE

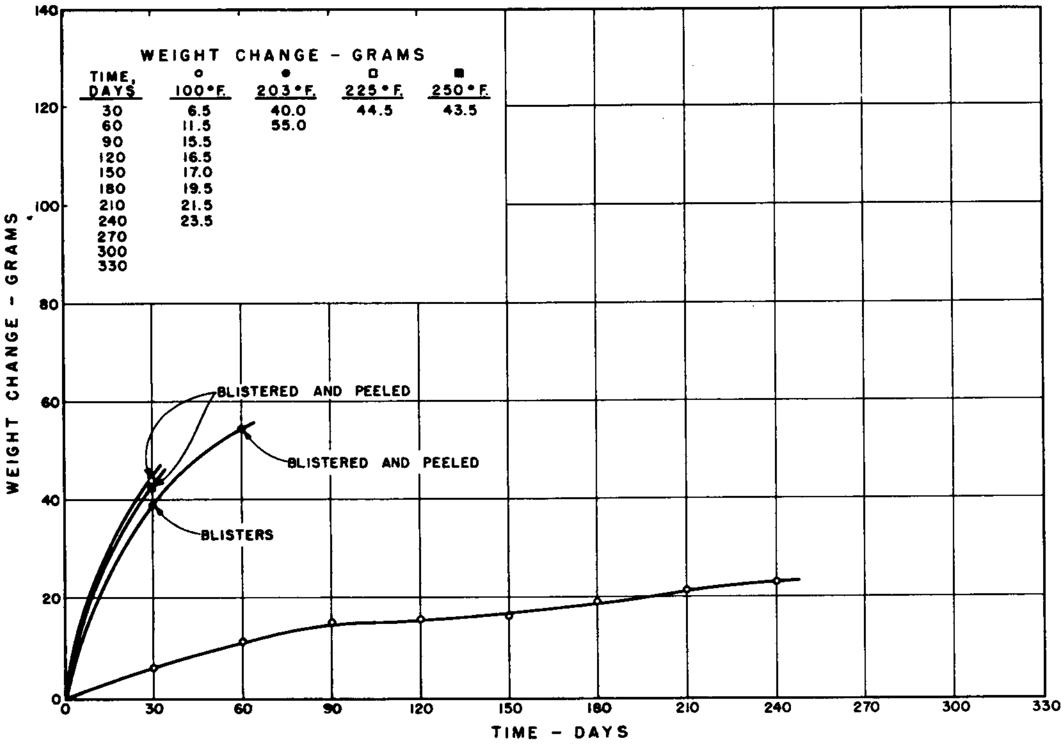


WEIGHT CHANGE - GRAMS

TIME, DAYS	○ 100 °F.	● 203 °F.	□ 225 °F.	■ 250 °F.
30	-1.0	-6.5	-9.5	-9.0
60	-2.0	-9.5	-9.5	
90	-2.0	-10.5	-9.5	-11.5
120	-2.0	-10.5	-11.5	-12.0
150	-2.0	-11.5	-11.0	-12.0
180	-3.0	-11.5	-10.5	-11.5
210	-3.0	-11.5	-12.0	
240	-3.0		-12.0	
270	-3.0			
300	-3.0			

B. ON STEEL

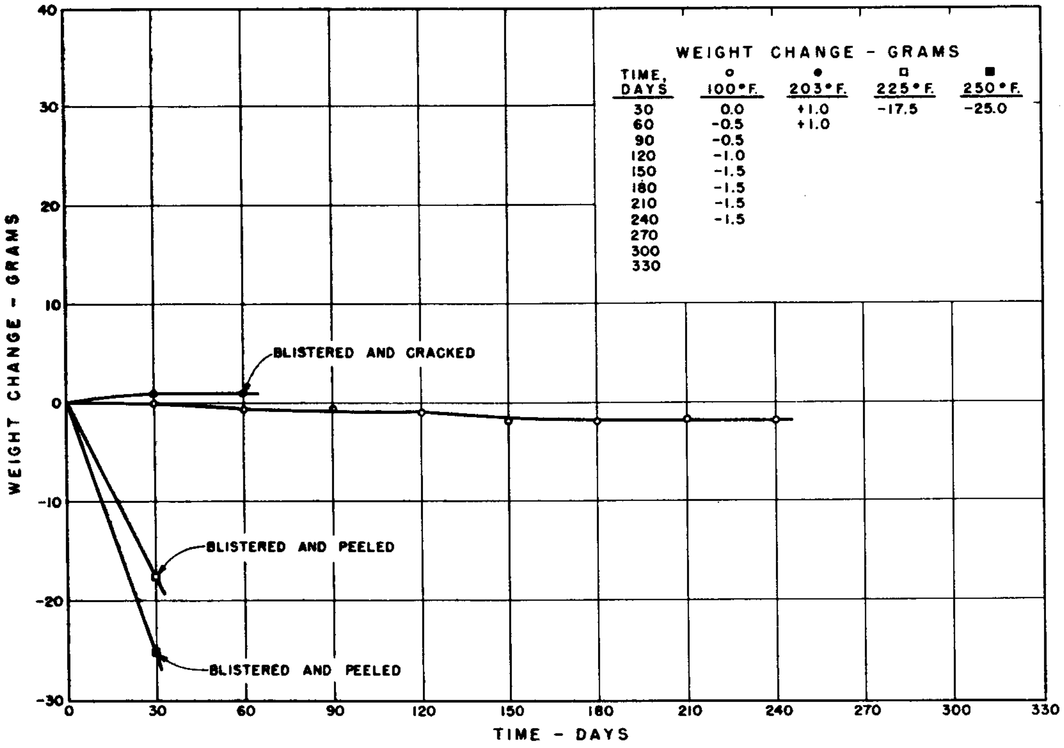
COATING C-14 - WEIGHT CHANGE VS TIME
FIGURE V-36



WEIGHT CHANGE - GRAMS

TIME, DAYS	○ 100°F.	● 203°F.	□ 225°F.	■ 250°F.
30	6.5	40.0	44.5	43.5
60	11.5	55.0		
90	15.5			
120	16.5			
150	17.0			
180	19.5			
210	21.5			
240	23.5			
270				
300				
330				

A. ON CONCRETE

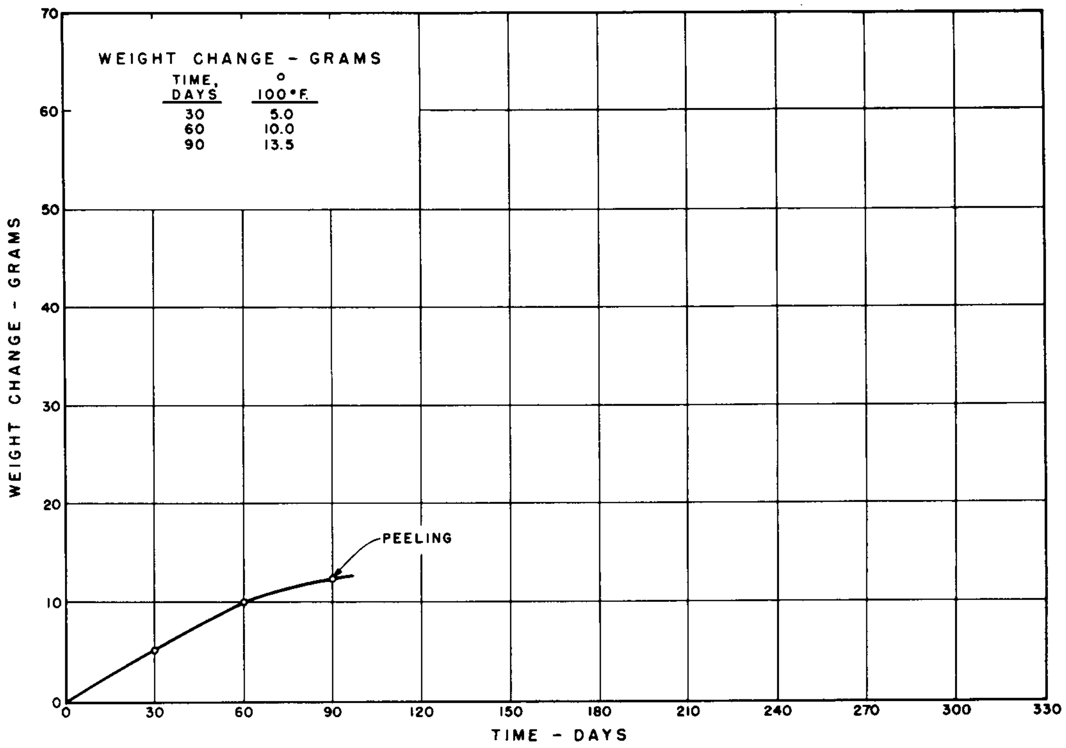


WEIGHT CHANGE - GRAMS

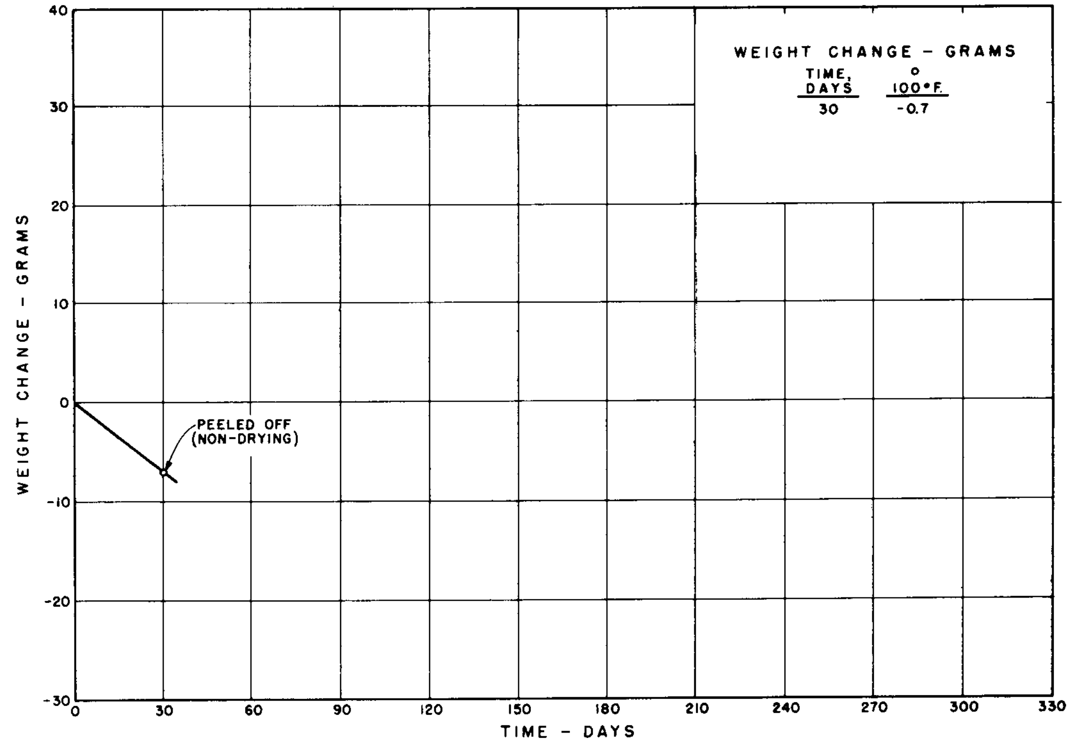
TIME, DAYS	○ 100°F.	● 203°F.	□ 225°F.	■ 250°F.
30	0.0	+1.0	-17.5	-25.0
60	-0.5	+1.0		
90	-0.5			
120	-1.0			
150	-1.5			
180	-1.5			
210	-1.5			
240	-1.5			
270				
300				
330				

B. ON STEEL

COATING C-15 - WEIGHT CHANGE VS TIME
FIGURE V-37

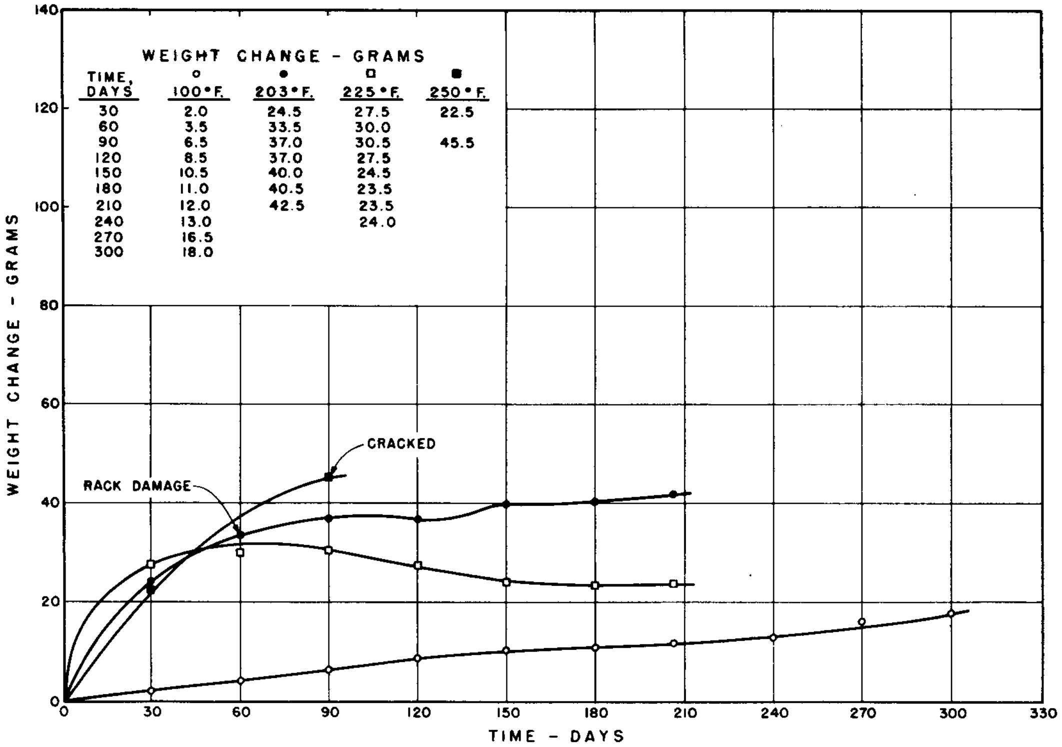


A. ON CONCRETE



B. ON STEEL

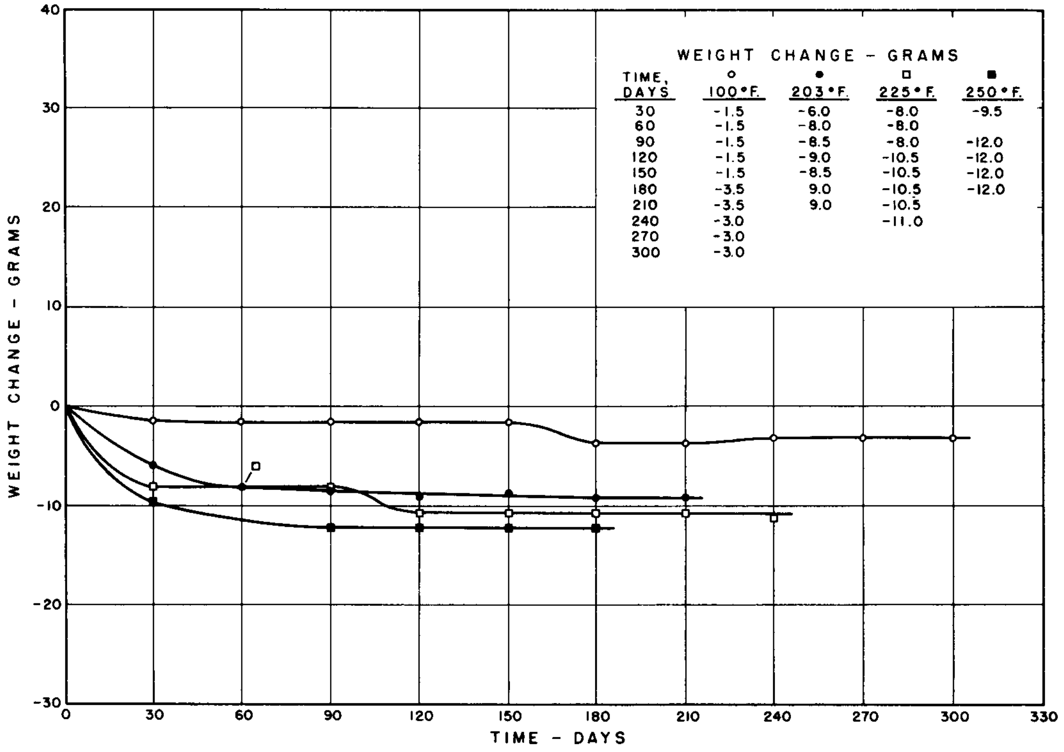
COATING C-16 - WEIGHT CHANGE VS TIME
FIGURE V-38



WEIGHT CHANGE - GRAMS

TIME, DAYS	○ 100°F.	● 203°F.	□ 225°F.	■ 250°F.
30	2.0	24.5	27.5	22.5
60	3.5	33.5	30.0	
90	6.5	37.0	30.5	45.5
120	8.5	37.0	27.5	
150	10.5	40.0	24.5	
180	11.0	40.5	23.5	
210	12.0	42.5	23.5	
240	13.0		24.0	
270	16.5			
300	18.0			

A. ON CONCRETE

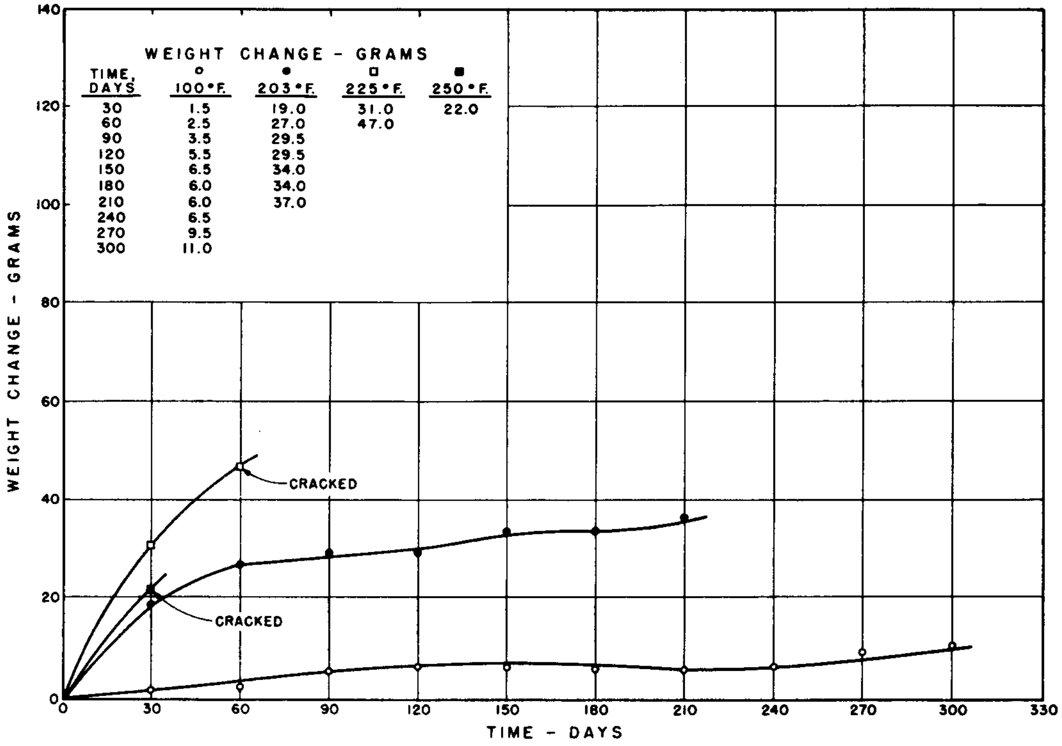


WEIGHT CHANGE - GRAMS

TIME, DAYS	○ 100°F.	● 203°F.	□ 225°F.	■ 250°F.
30	-1.5	-6.0	-8.0	-9.5
60	-1.5	-8.0	-8.0	
90	-1.5	-8.5	-8.0	-12.0
120	-1.5	-9.0	-10.5	-12.0
150	-1.5	-8.5	-10.5	-12.0
180	-3.5	9.0	-10.5	-12.0
210	-3.5	9.0	-10.5	
240	-3.0		-11.0	
270	-3.0			
300	-3.0			

B. ON STEEL

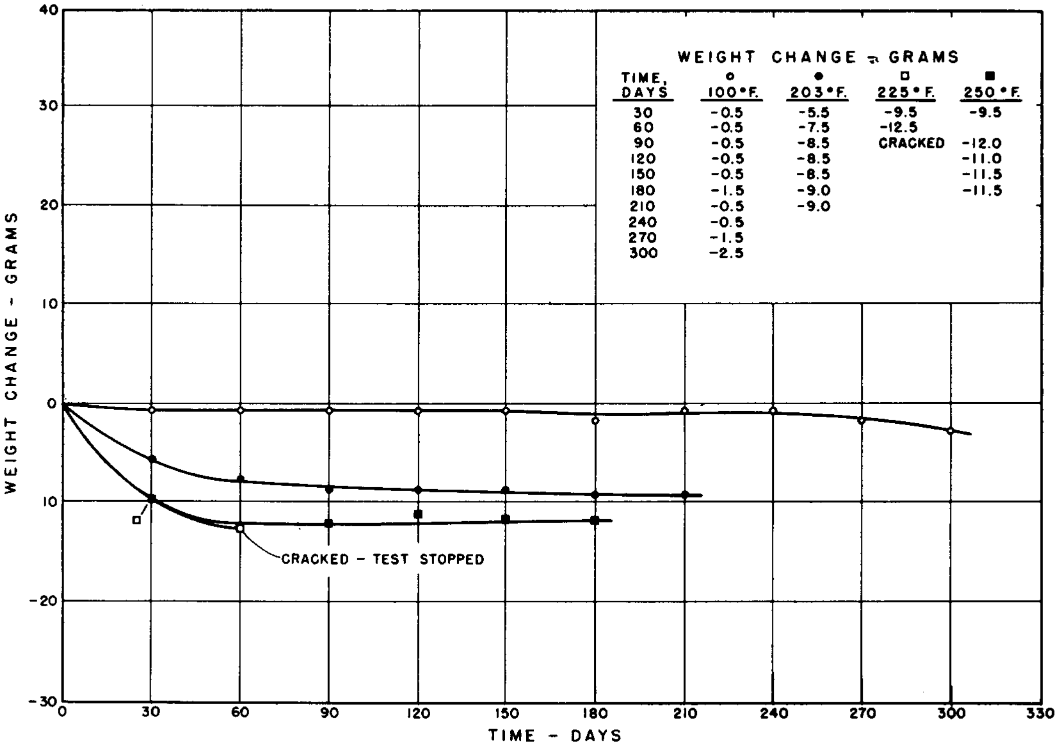
COATING C-17 - WEIGHT CHANGE VS TIME
FIGURE V-39



WEIGHT CHANGE - GRAMS

TIME, DAYS	○ 100°F.	● 203°F.	□ 225°F.	■ 250°F.
30	1.5	19.0	31.0	22.0
60	2.5	27.0	47.0	
90	3.5	29.5		
120	5.5	29.5		
150	6.5	34.0		
180	6.0	34.0		
210	6.0	37.0		
240	6.5			
270	9.5			
300	11.0			

A. ON CONCRETE

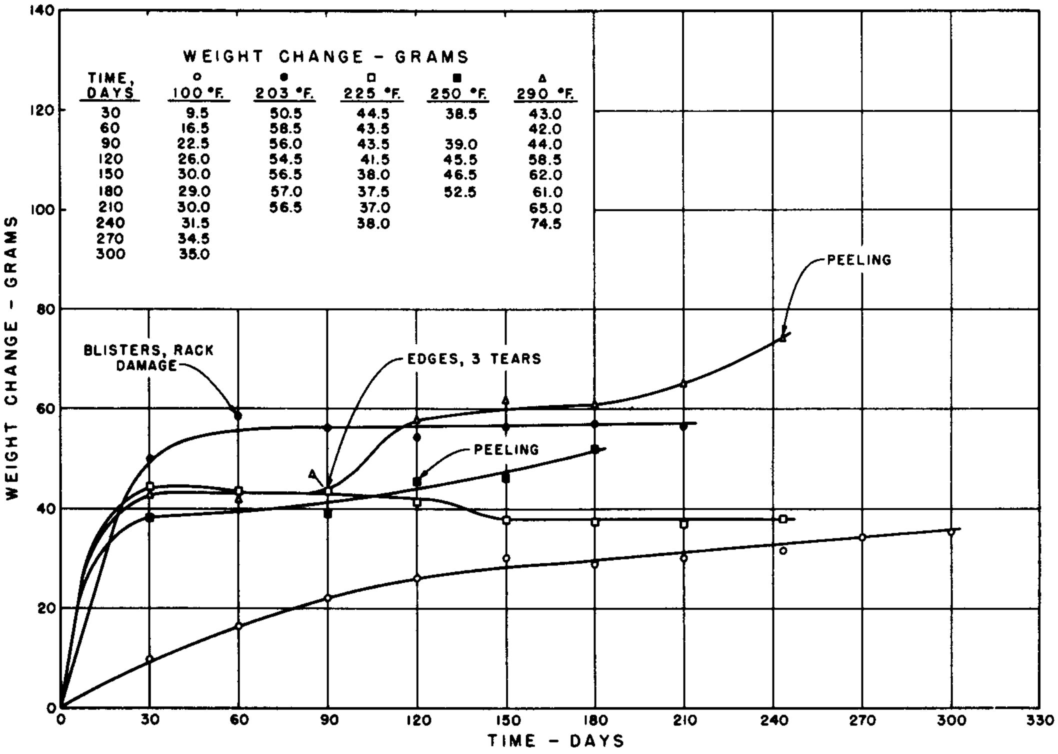


WEIGHT CHANGE - GRAMS

TIME, DAYS	○ 100°F.	● 203°F.	□ 225°F.	■ 250°F.
30	-0.5	-5.5	-9.5	-9.5
60	-0.5	-7.5	-12.5	
90	-0.5	-8.5	CRACKED	-12.0
120	-0.5	-8.5		-11.0
150	-0.5	-8.5		-11.5
180	-1.5	-9.0		-11.5
210	-0.5	-9.0		
240	-0.5			
270	-1.5			
300	-2.5			

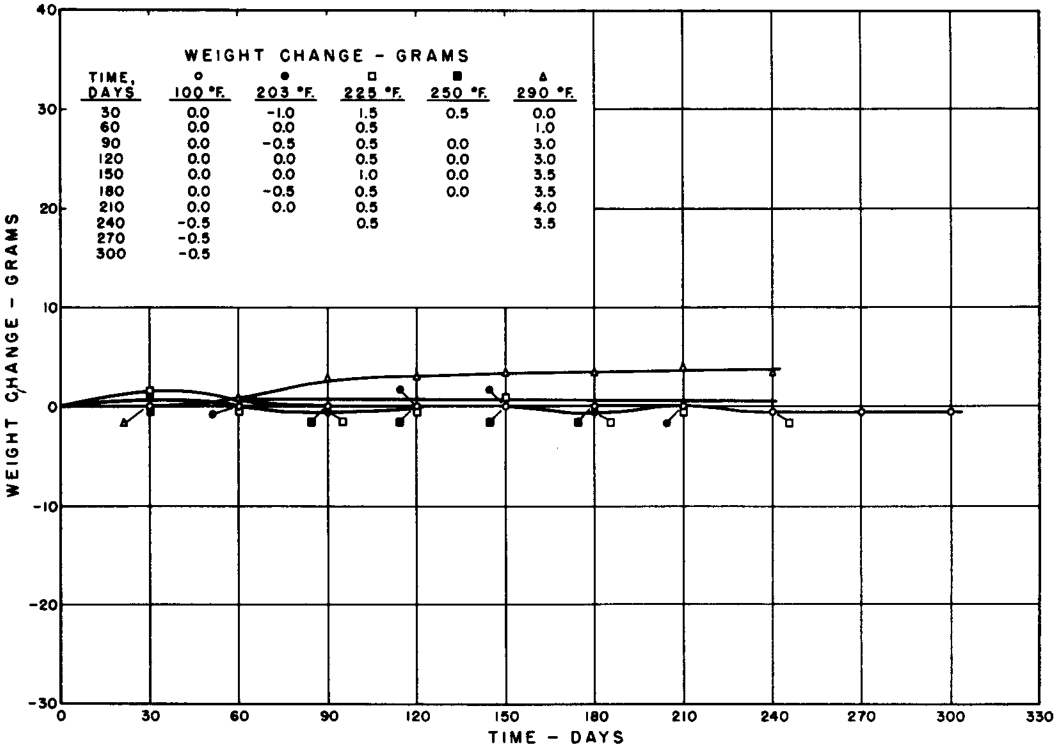
B. ON STEEL

COATING C-18 - WEIGHT CHANGE VS TIME
FIGURE V-40



TIME, DAYS	○ 100 °F.	● 203 °F.	□ 225 °F.	■ 250 °F.	△ 290 °F.
30	9.5	50.5	44.5	38.5	43.0
60	16.5	58.5	43.5		42.0
90	22.5	56.0	43.5	39.0	44.0
120	26.0	54.5	41.5	45.5	58.5
150	30.0	56.5	38.0	46.5	62.0
180	29.0	57.0	37.5	52.5	61.0
210	30.0	56.5	37.0		65.0
240	31.5		38.0		74.5
270	34.5				
300	35.0				

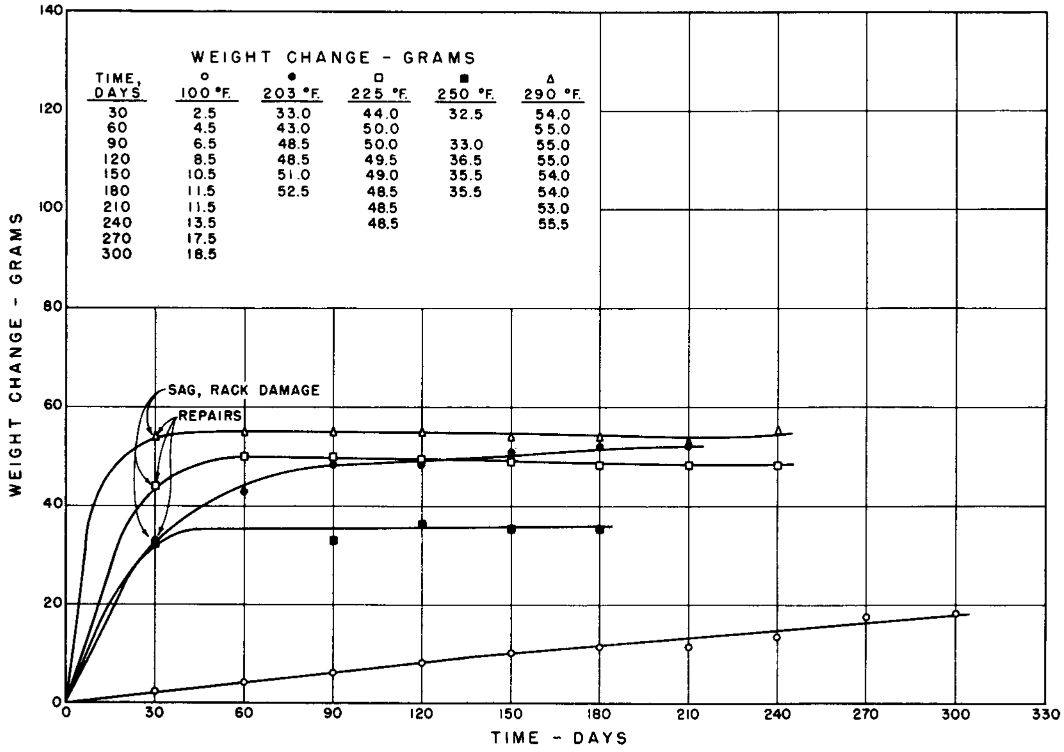
A. ON CONCRETE



TIME, DAYS	○ 100 °F.	● 203 °F.	□ 225 °F.	■ 250 °F.	△ 290 °F.
30	0.0	-1.0	1.5	0.5	0.0
60	0.0	0.0	0.5		1.0
90	0.0	-0.5	0.5	0.0	3.0
120	0.0	0.0	0.5	0.0	3.0
150	0.0	0.0	1.0	0.0	3.5
180	0.0	-0.5	0.5	0.0	3.5
210	0.0	0.0	0.5		4.0
240	-0.5		0.5		3.5
270	-0.5				
300	-0.5				

B. ON STEEL

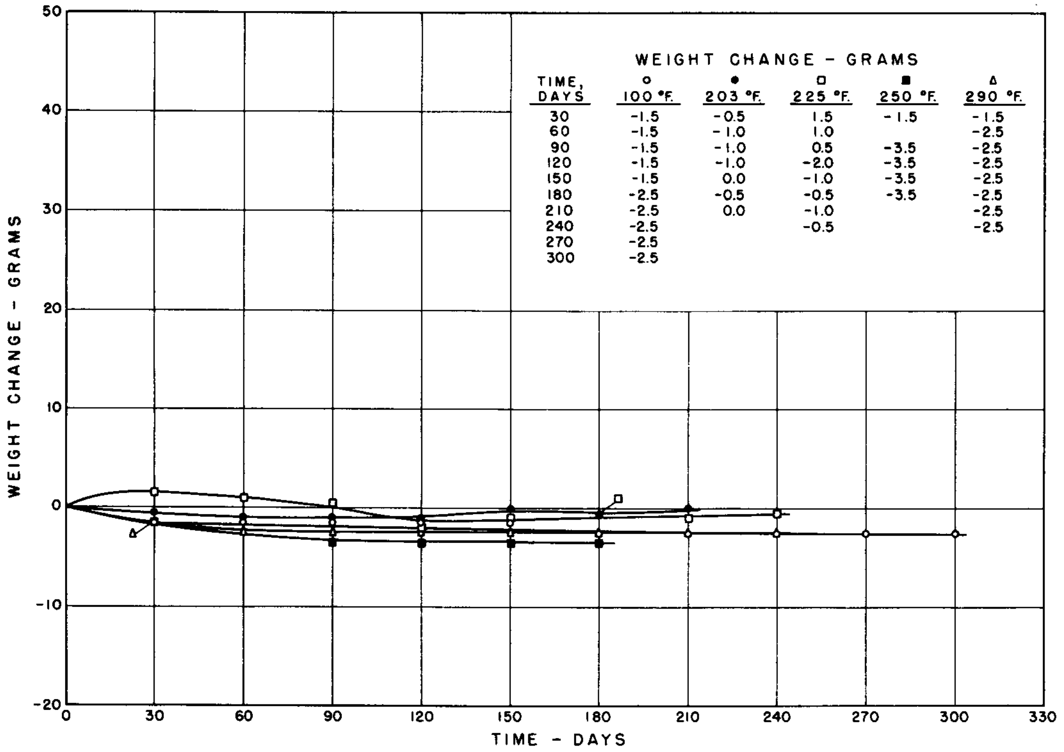
COATING C-19 – WEIGHT CHANGE VS TIME
FIGURE V - 41



WEIGHT CHANGE - GRAMS

TIME, DAYS	○ 100 °F.	● 203 °F.	□ 225 °F.	■ 250 °F.	△ 290 °F.
30	2.5	33.0	44.0	32.5	54.0
60	4.5	43.0	50.0		55.0
90	6.5	48.5	50.0	33.0	55.0
120	8.5	48.5	49.5	36.5	55.0
150	10.5	51.0	49.0	35.5	54.0
180	11.5	52.5	48.5	35.5	54.0
210	11.5		48.5		53.0
240	13.5		48.5		55.5
270	17.5				
300	18.5				

A. ON CONCRETE

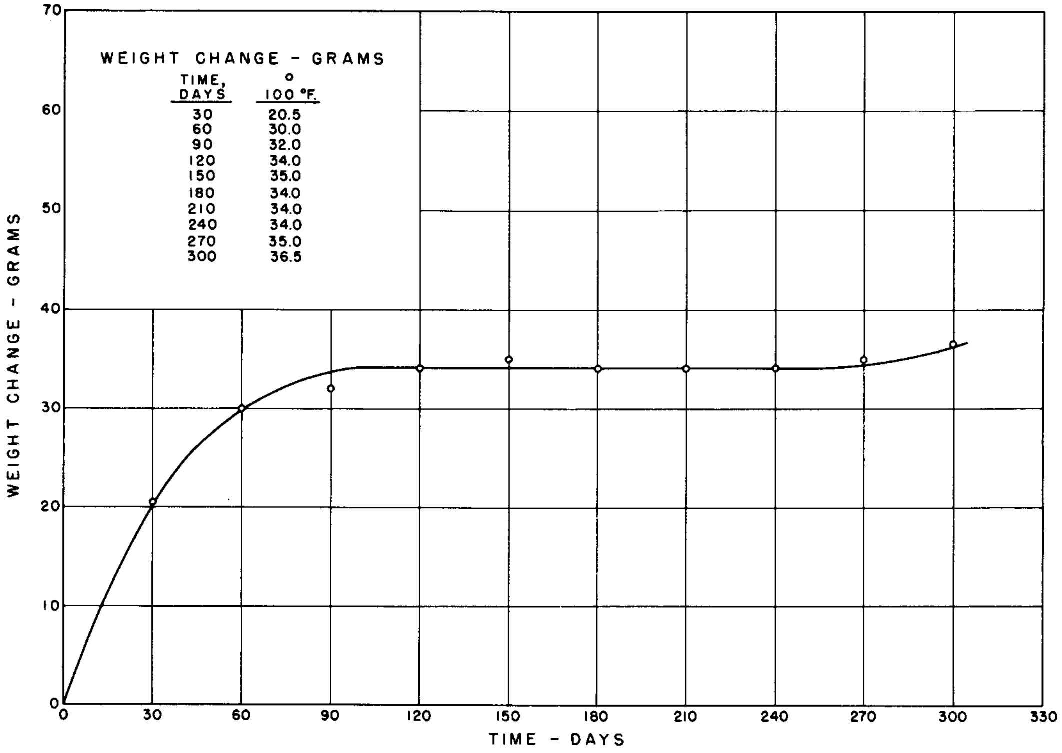


WEIGHT CHANGE - GRAMS

TIME, DAYS	○ 100 °F.	● 203 °F.	□ 225 °F.	■ 250 °F.	△ 290 °F.
30	-1.5	-0.5	1.5	-1.5	-1.5
60	-1.5	-1.0	1.0		-2.5
90	-1.5	-1.0	0.5	-3.5	-2.5
120	-1.5	-1.0	-2.0	-3.5	-2.5
150	-1.5	0.0	-1.0	-3.5	-2.5
180	-2.5	-0.5	-0.5	-3.5	-2.5
210	-2.5	0.0	-1.0		-2.5
240	-2.5		-0.5		-2.5
270	-2.5				
300	-2.5				

B. ON STEEL

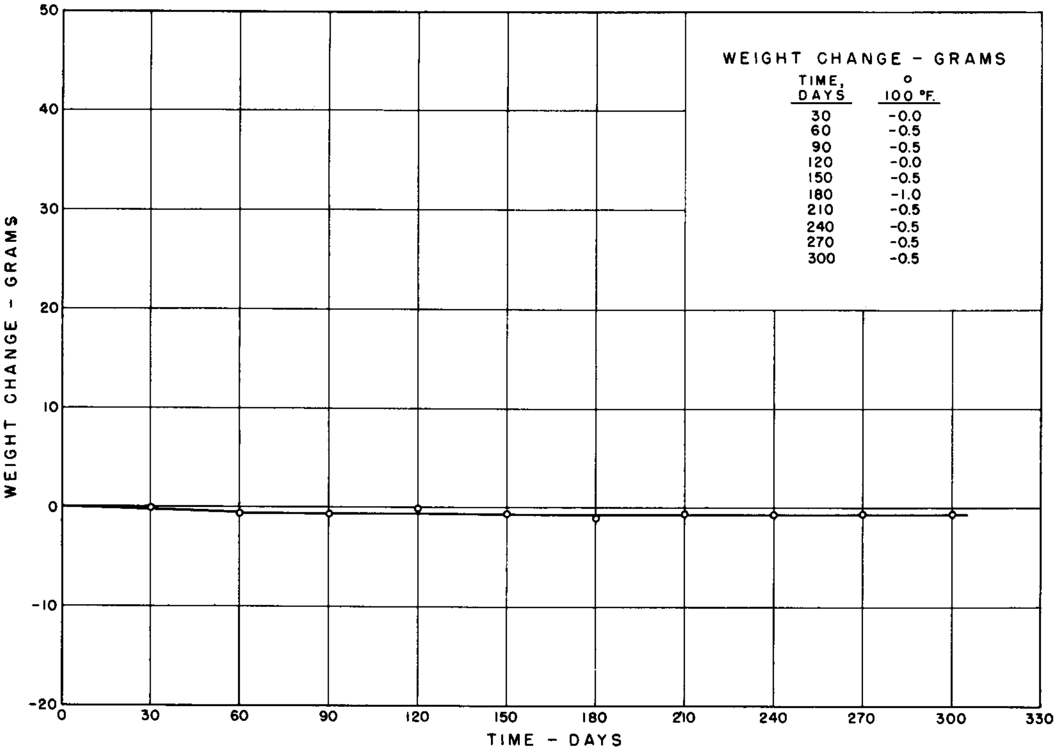
COATING C-20 – WEIGHT CHANGE VS TIME
FIGURE V-42



WEIGHT CHANGE - GRAMS

TIME, DAYS	100 °F.
30	20.5
60	30.0
90	32.0
120	34.0
150	35.0
180	34.0
210	34.0
240	34.0
270	35.0
300	36.5

A. ON CONCRETE

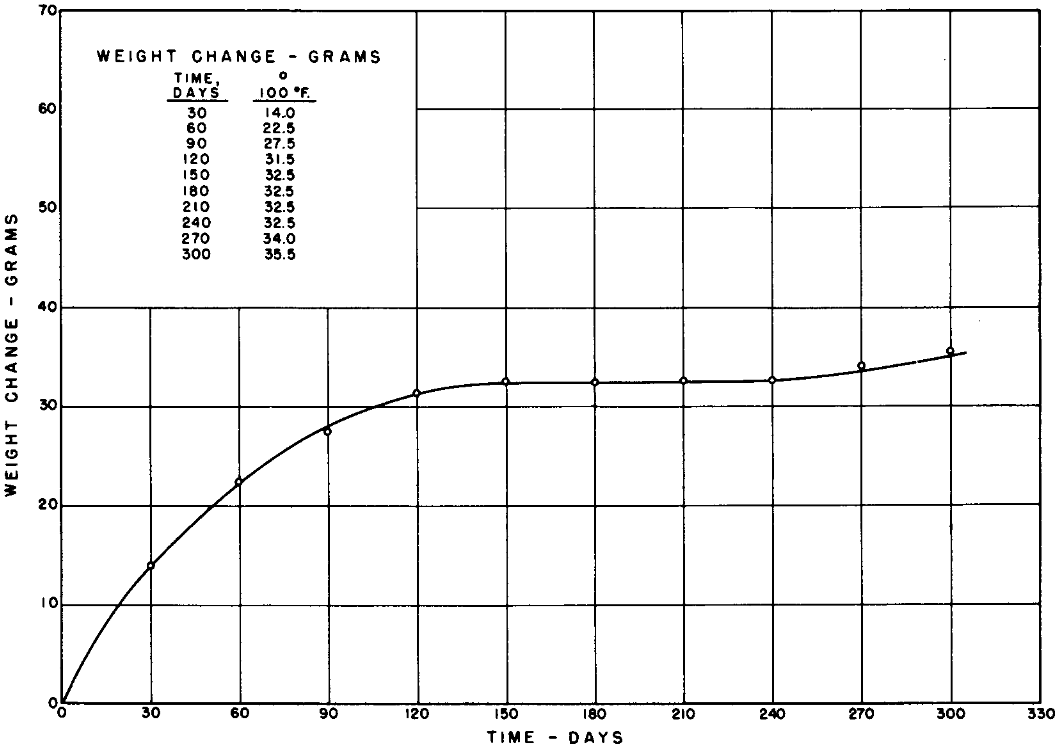


WEIGHT CHANGE - GRAMS

TIME, DAYS	100 °F.
30	-0.0
60	-0.5
90	-0.5
120	-0.0
150	-0.5
180	-1.0
210	-0.5
240	-0.5
270	-0.5
300	-0.5

B. ON STEEL

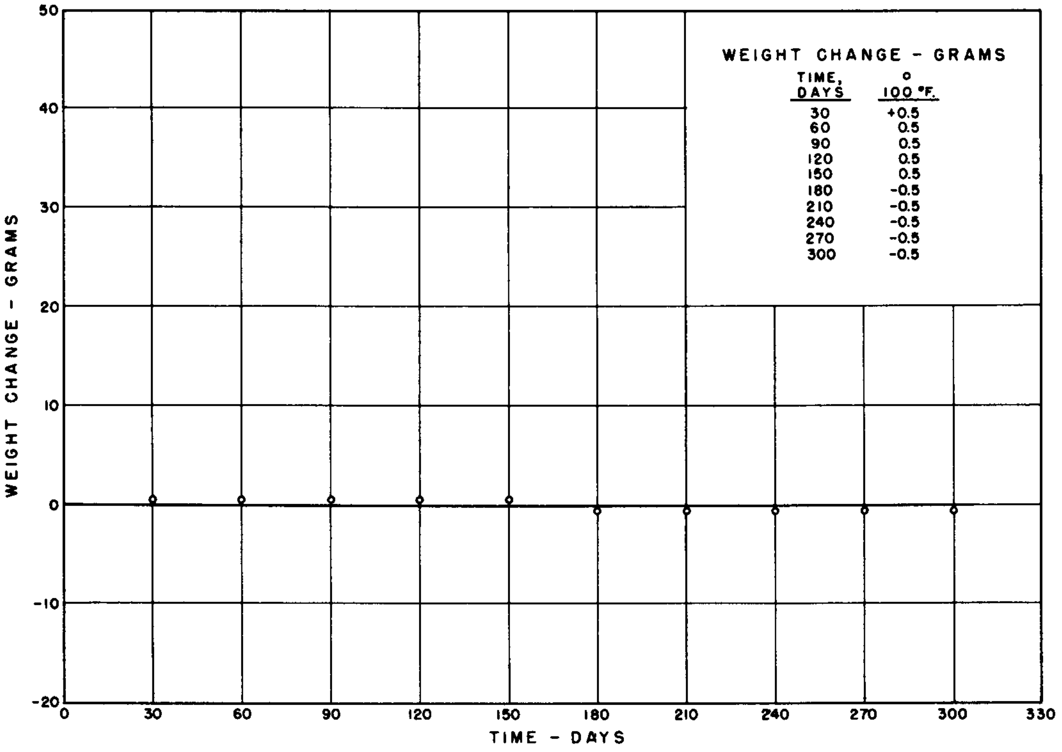
COATING C-21 - WEIGHT CHANGE VS TIME
FIGURE V-43



WEIGHT CHANGE - GRAMS

TIME, DAYS	100 °F.
30	14.0
60	22.5
90	27.5
120	31.5
150	32.5
180	32.5
210	32.5
240	32.5
270	34.0
300	35.5

A. ON CONCRETE

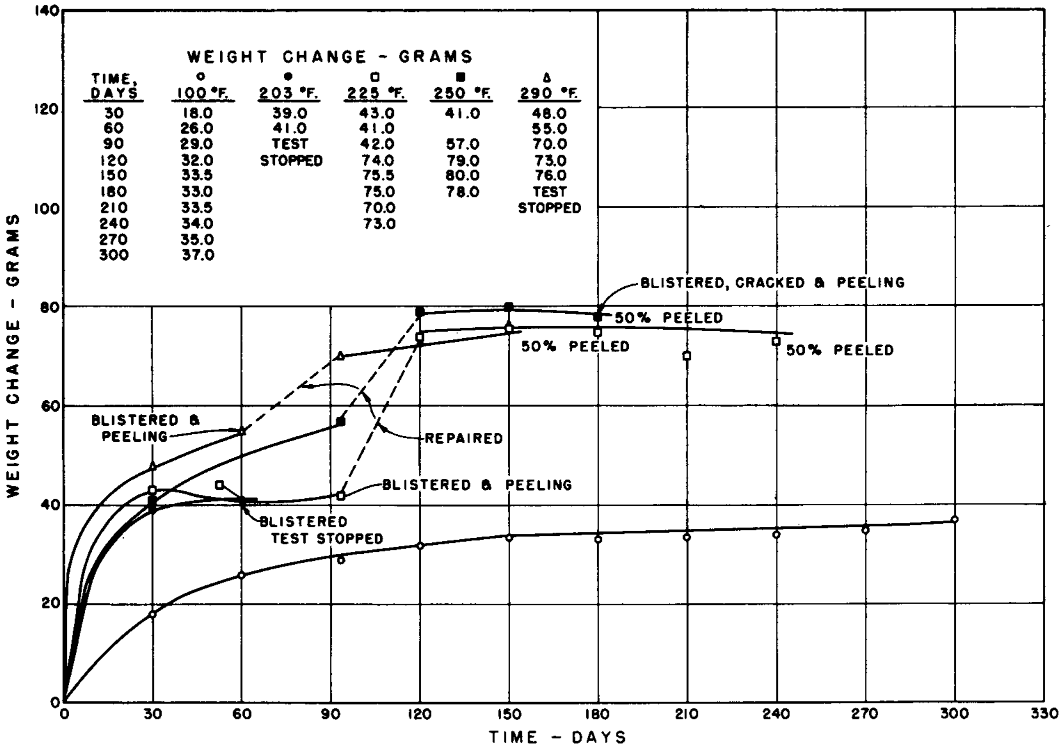


WEIGHT CHANGE - GRAMS

TIME, DAYS	100 °F.
30	+0.5
60	0.5
90	0.5
120	0.5
150	0.5
180	-0.5
210	-0.5
240	-0.5
270	-0.5
300	-0.5

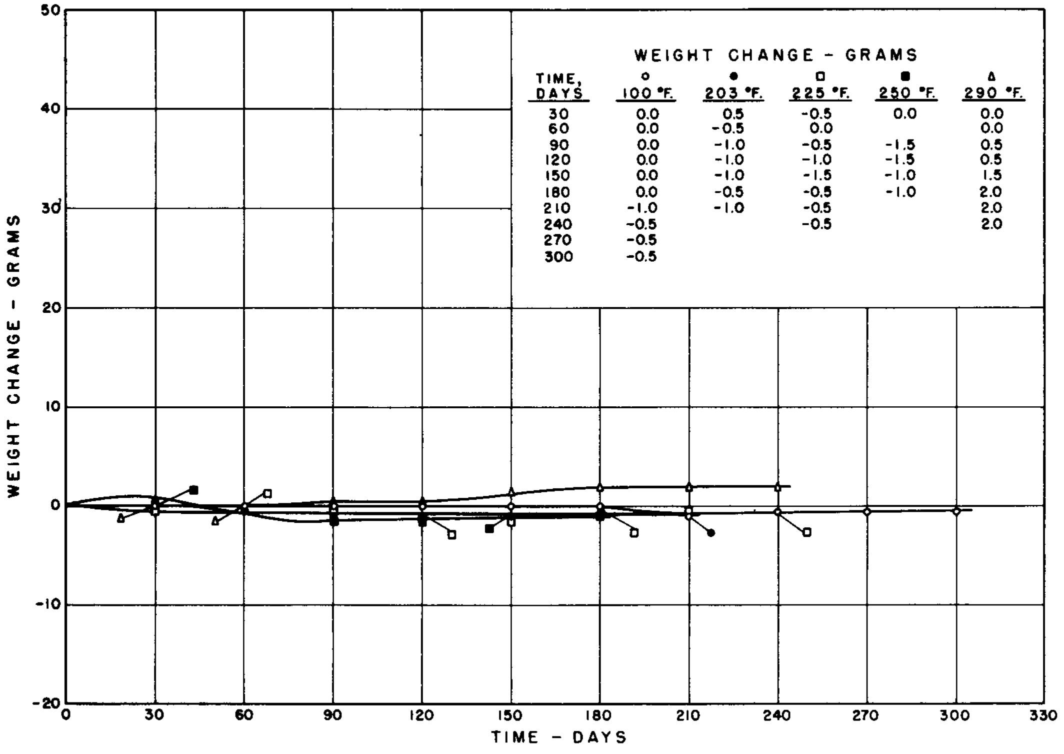
B. ON STEEL

COATING C-22 - WEIGHT CHANGE VS TIME
FIGURE V-44



	WEIGHT CHANGE - GRAMS				
TIME, DAYS	○ 100 °F.	● 203 °F.	□ 225 °F.	■ 250 °F.	△ 290 °F.
30	18.0	39.0	43.0	41.0	48.0
60	26.0	41.0	41.0		55.0
90	29.0	TEST	42.0	57.0	70.0
120	32.0	STOPPED	74.0	79.0	73.0
150	33.5		75.5	80.0	76.0
180	33.0		75.0	78.0	TEST
210	33.5		70.0		STOPPED
240	34.0		73.0		
270	35.0				
300	37.0				

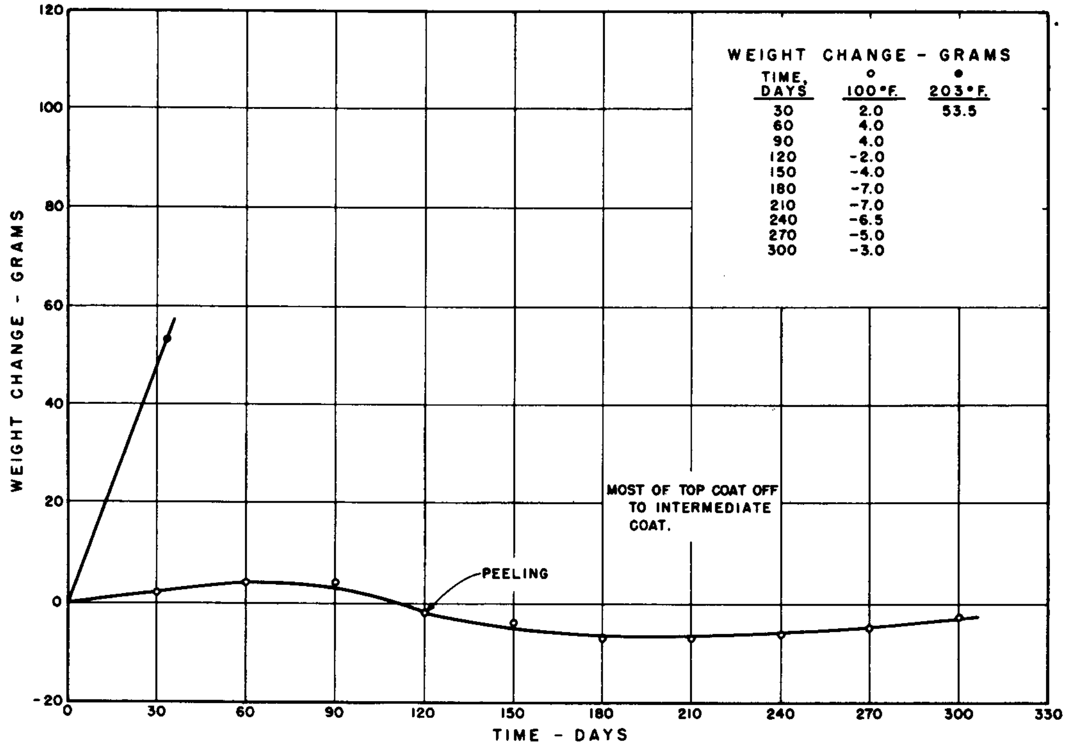
A. ON CONCRETE



	WEIGHT CHANGE - GRAMS				
TIME, DAYS	○ 100 °F.	● 203 °F.	□ 225 °F.	■ 250 °F.	△ 290 °F.
30	0.0	0.5	-0.5	0.0	0.0
60	0.0	-0.5	0.0		0.0
90	0.0	-1.0	-0.5	-1.5	0.5
120	0.0	-1.0	-1.0	-1.5	0.5
150	0.0	-1.0	-1.5	-1.0	1.5
180	0.0	-0.5	-0.5	-1.0	2.0
210	-1.0	-1.0	-0.5		2.0
240	-0.5		-0.5		2.0
270	-0.5				
300	-0.5				

B. ON STEEL

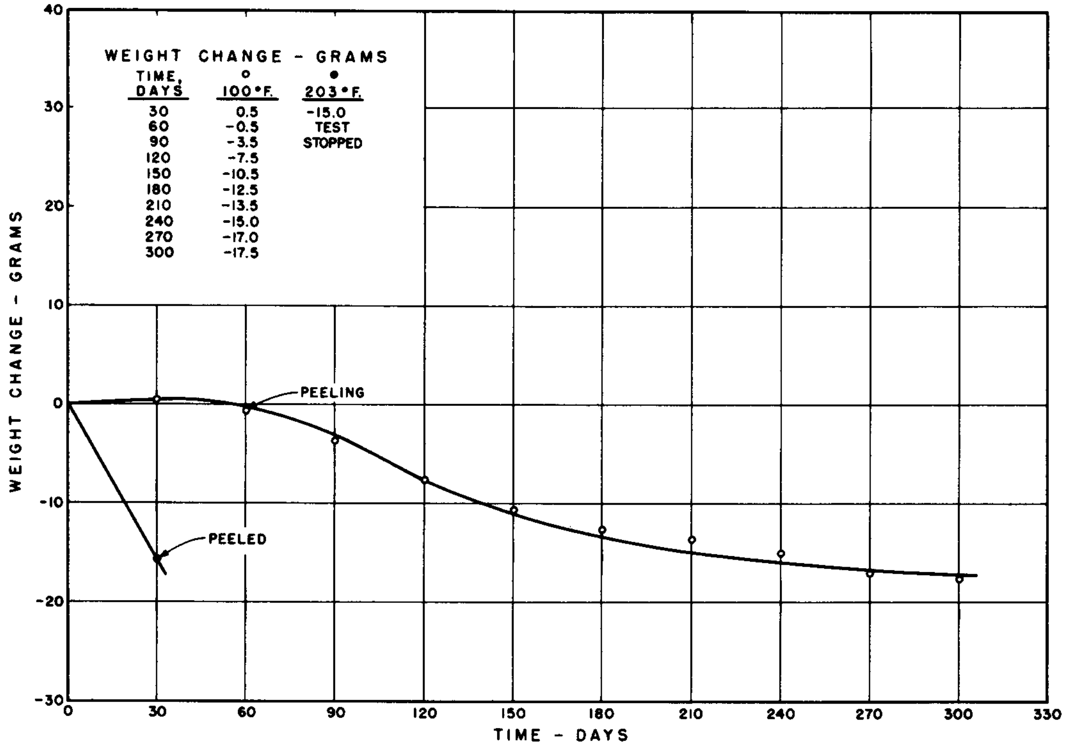
COATING C-23 - WEIGHT CHANGE VS TIME
FIGURE V-45



WEIGHT CHANGE - GRAMS

TIME, DAYS	○ 100°F.	● 203°F.
30	2.0	53.5
60	4.0	
90	4.0	
120	-2.0	
150	-4.0	
180	-7.0	
210	-7.0	
240	-6.5	
270	-5.0	
300	-3.0	

A. ON CONCRETE



WEIGHT CHANGE - GRAMS

TIME, DAYS	○ 100°F.	● 203°F.
30	0.5	-15.0
60	-0.5	TEST
90	-3.5	STOPPED
120	-7.5	
150	-10.5	
180	-12.5	
210	-13.5	
240	-15.0	
270	-17.0	
300	-17.5	

B. ON STEEL

COATING C-24 – WEIGHT CHANGE VS TIME
FIGURE V-46

V. 5. 2. 17. Coating C-26. --

Coating C-26, a styrene modified acrylic epoxy, showed cracking and peeling in the first 30 days on concrete for temperatures 203°, 225°, 250°, and 290° F as indicated by Figure V-47A. Cracking and peeling occurred in 30 days on steel at 250° and 290° F; at 90 days at 225° F; and 120 days at 203° F. Performance is satisfactory on concrete and steel at 100° F.

V. 5. 2. 18. Coatings C-33 and C-34. --

Performance of trifunctional silicone, Coating C-33 on concrete and Coating C-34 on steel are shown in Figure V-49A and B. The concrete specimens were coated by dipping and the steel specimens by spraying, all by the resin manufacturer. The coating peeled off the steel at all temperatures tested. After 210 days on concrete at 290° F, the coating appears gone and disintegration of the concrete has begun. At the other four test temperatures, it is still undetermined if any coating is left.

V. 5. 2. 19. Coating C-36. --

Coating C-36, flame sprayed polypropylene, showed disintegration and peeling on steel and concrete at 203°, 225°, 250°, and 290° F as indicated in Figure V-50. The coating has performed satisfactorily at 100° F for both concrete and steel.

V. 5. 2. 20. Coating C-37. --

Coating C-37, chlorinated polyether, on steel only has performed generally satisfactorily for the 90 days which it has been under test. Blisters developed in the first 30 days at 225°, 250°, and 290° F, but the blisters are not progressing and apparently are not effecting the protective performance, Figure V-51.

V. 5. 2. 21. Coating C-38. --

Coating C-38, polyarylether, on concrete, shown in Figure V-52, plateaued as expected. The curves of weight change versus time show very little difference between 225° and the 250° F performance. At 210 days at 290° F severe peeling occurred causing a large positive weight change. After 180 days at 290° F on steel, severe peeling and erosion developed causing failure of the coating. The polyarylether has performed poorly on concrete and steel at all temperatures tested.

V. 5. 2. 22. Coating C-39. --

Polyethylene-polyvinyl acetate copolymer, Coating C-39, showed disintegration and peeling on steel and concrete at 203°, 225°, 250°, and at 290° F on steel as indicated in Figure V-53. After failures below 290° F on concrete, the 290° F specimen was not exposed. Coating C-39 has performed fair at 100° F on both steel and concrete.

V. 5. 2. 23. Coating C-40. --

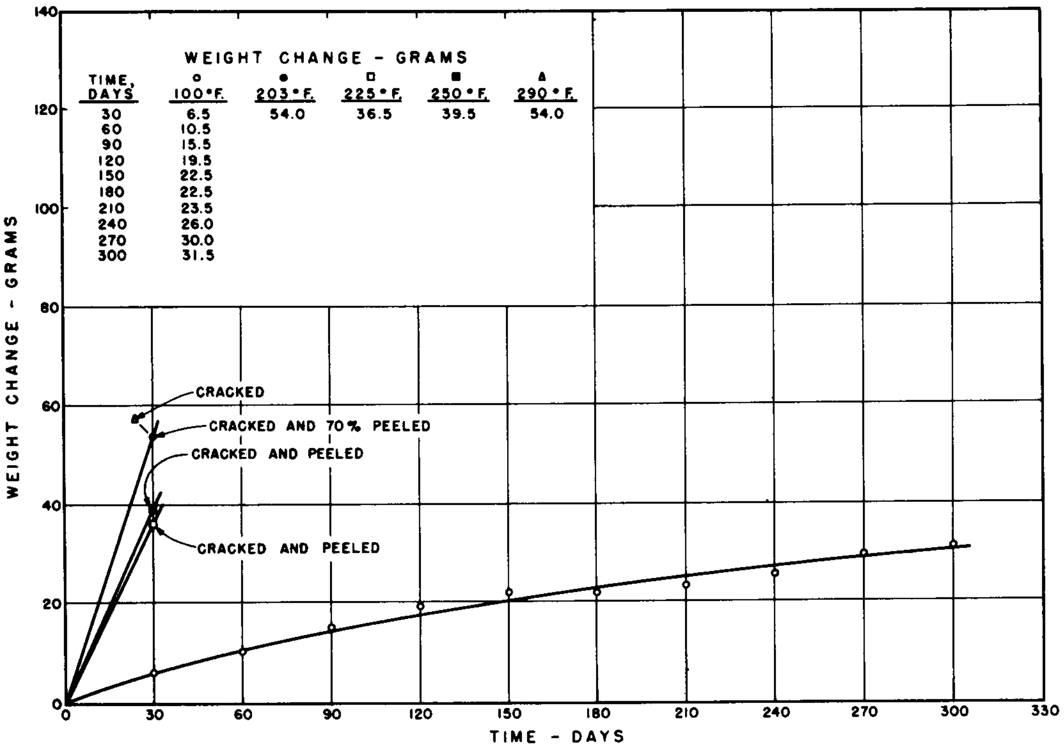
Coating C-40, porcelain enamel, is being tested only at 290° F on steel. As shown in Figure V-54, this coating has little weight change. It has shown excellent performance for the test to date.

V. 5. 2. 24. Coating C-41. --

Coating C-41, a cross-linked phenoxy, has not been tested because satisfactory applications could not be achieved. The supplier could not offer assistance. Work has been discontinued on C-41.

V. 5. 2. 25. Coating C-43. --

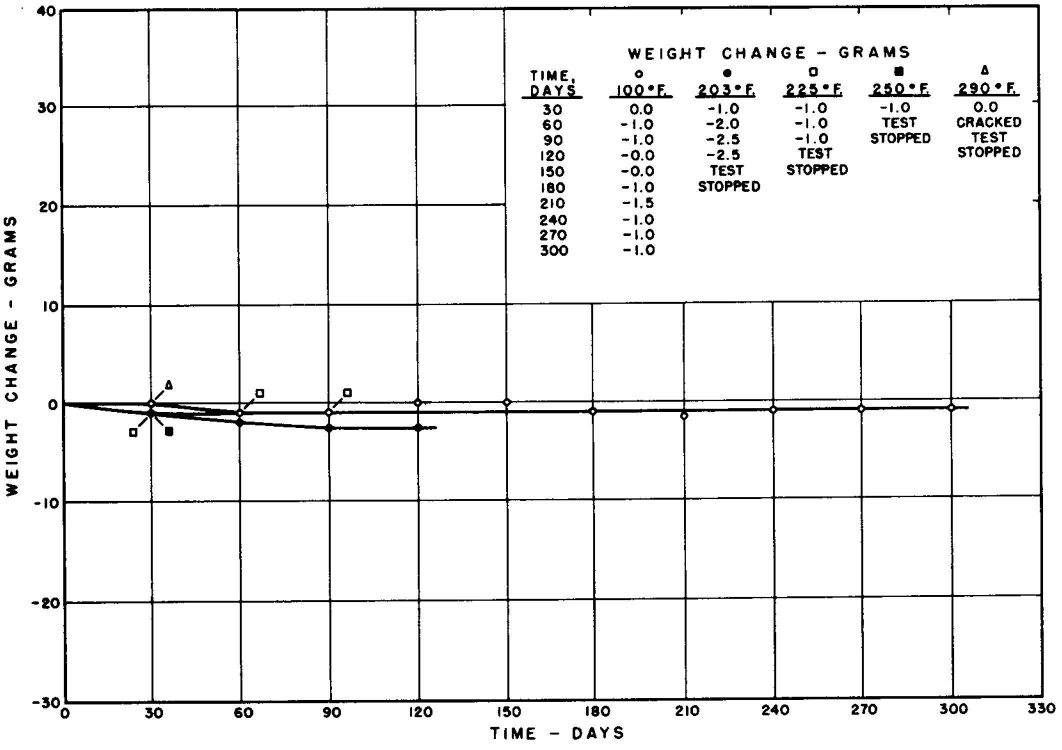
An epoxy, Coating C-43, has proved unsatisfactory after 30 days exposure at 225°, 250°, and 290° F on both steel and concrete. At 203° F at 60 days it also failed. Severe cracking, blistering, and peeling occurred on both steel and concrete at 203°, 225°, 250°, and 290° F causing failure. The coating appears to be satisfactory after 90 days at 100° F on steel and concrete, Figure V-55.



WEIGHT CHANGE - GRAMS

TIME, DAYS	○ 100°F.	● 203°F.	□ 225°F.	■ 250°F.	△ 290°F.
30	6.5	54.0	36.5	39.5	54.0
60	10.5				
90	15.5				
120	19.5				
150	22.5				
180	22.5				
210	23.5				
240	26.0				
270	30.0				
300	31.5				

A. ON CONCRETE

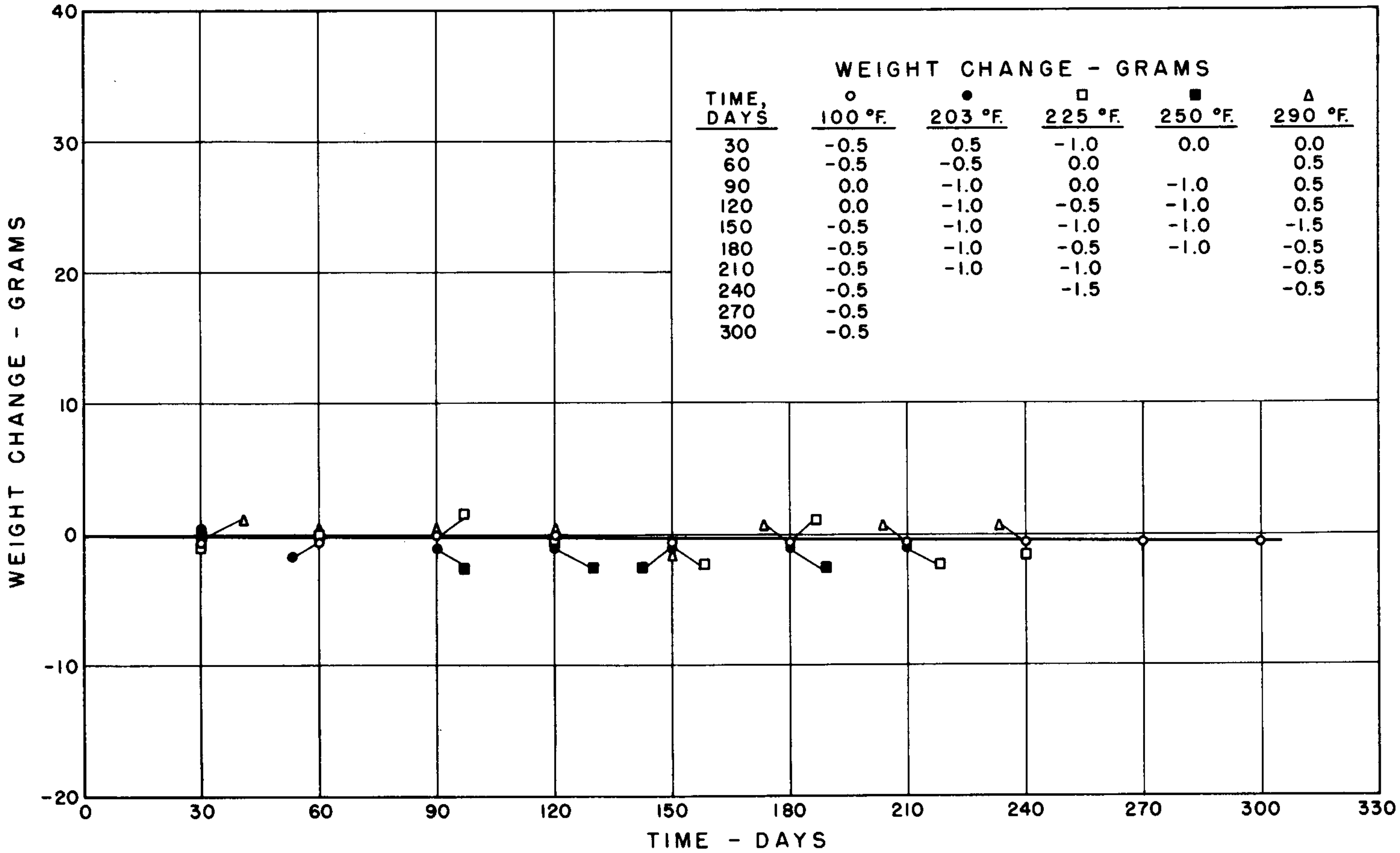


WEIGHT CHANGE - GRAMS

TIME, DAYS	○ 100°F.	● 203°F.	□ 225°F.	■ 250°F.	△ 290°F.
30	0.0	-1.0	-1.0	-1.0	0.0
60	-1.0	-2.0	-1.0	TEST STOPPED	CRACKED TEST STOPPED
90	-1.0	-2.5	-1.0		
120	-0.0	-2.5	TEST STOPPED		
150	-0.0	TEST STOPPED			
180	-1.0				
210	-1.5				
240	-1.0				
270	-1.0				
300	-1.0				

B. ON STEEL

COATING C-26 - WEIGHT CHANGE VS TIME
FIGURE V-47

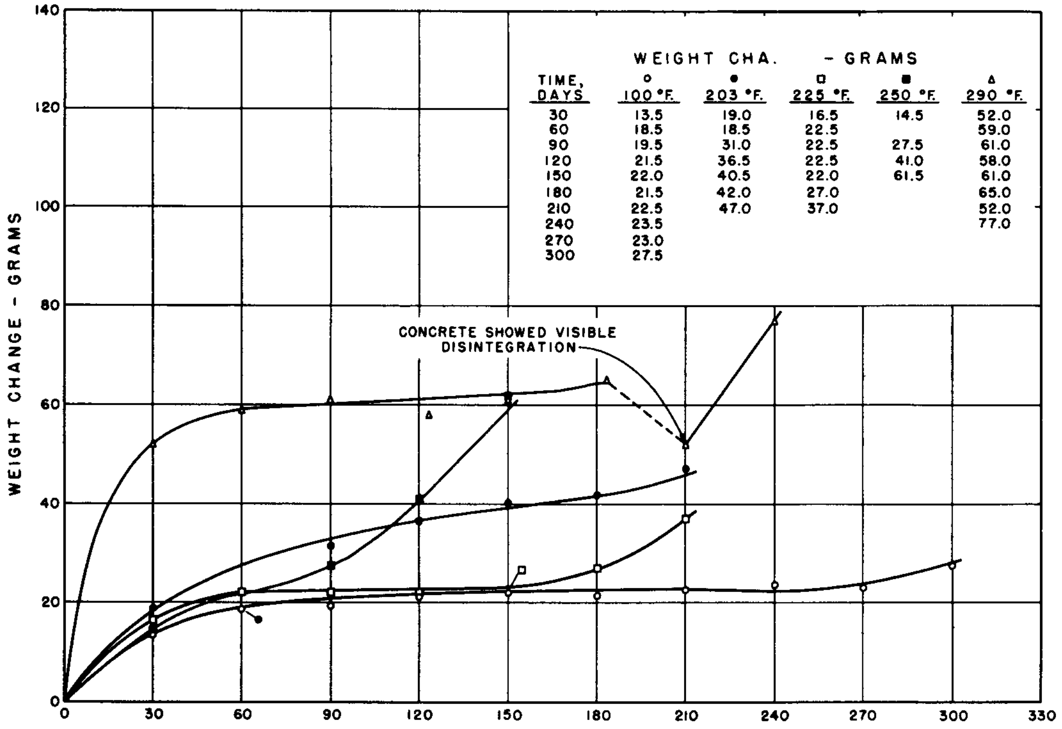


WEIGHT CHANGE - GRAMS

TIME, DAYS	○ 100 °F.	● 203 °F.	□ 225 °F.	■ 250 °F.	Δ 290 °F.
30	-0.5	0.5	-1.0	0.0	0.0
60	-0.5	-0.5	0.0		0.5
90	0.0	-1.0	0.0	-1.0	0.5
120	0.0	-1.0	-0.5	-1.0	0.5
150	-0.5	-1.0	-1.0	-1.0	-1.5
180	-0.5	-1.0	-0.5	-1.0	-0.5
210	-0.5	-1.0	-1.0		-0.5
240	-0.5		-1.5		-0.5
270	-0.5				
300	-0.5				

ON STEEL

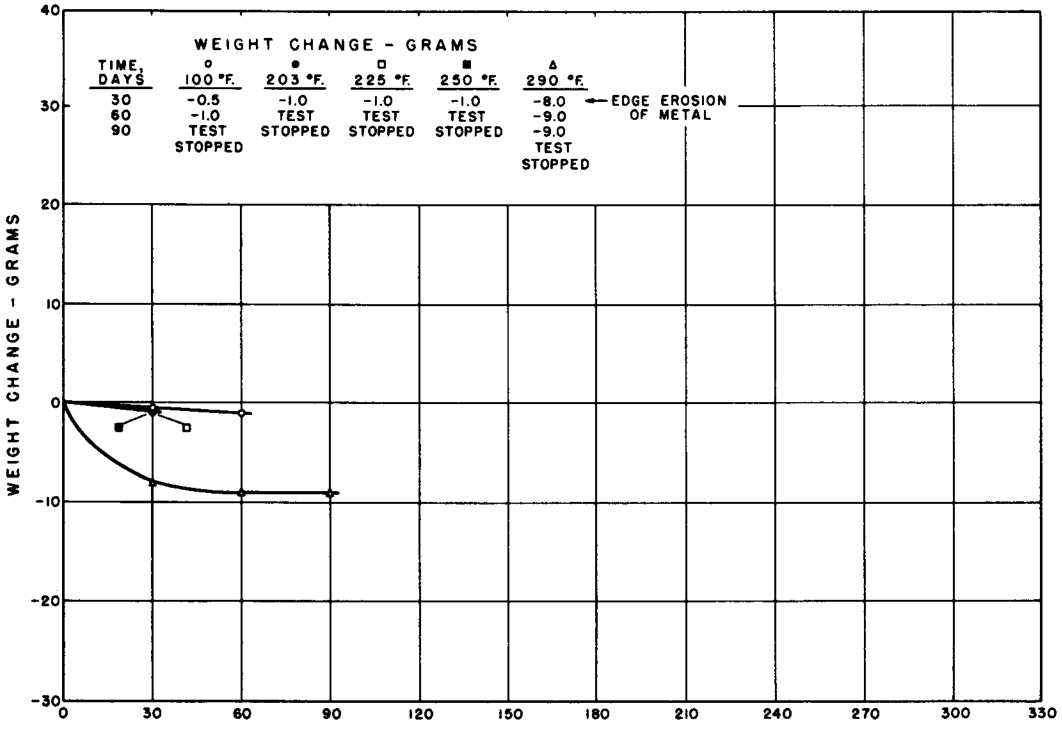
COATING C-32 - WEIGHT CHANGE VS TIME
FIGURE V-48



WEIGHT CHA. - GRAMS

TIME, DAYS	○ 100 °F.	● 203 °F.	□ 225 °F.	■ 250 °F.	△ 290 °F.
30	13.5	19.0	16.5	14.5	52.0
60	18.5	18.5	22.5		59.0
90	19.5	31.0	22.5	27.5	61.0
120	21.5	36.5	22.5	41.0	58.0
150	22.0	40.5	22.0	61.5	61.0
180	21.5	42.0	27.0		65.0
210	22.5	47.0	37.0		52.0
240	23.5				77.0
270	23.0				
300	27.5				

A. ON CONCRETE

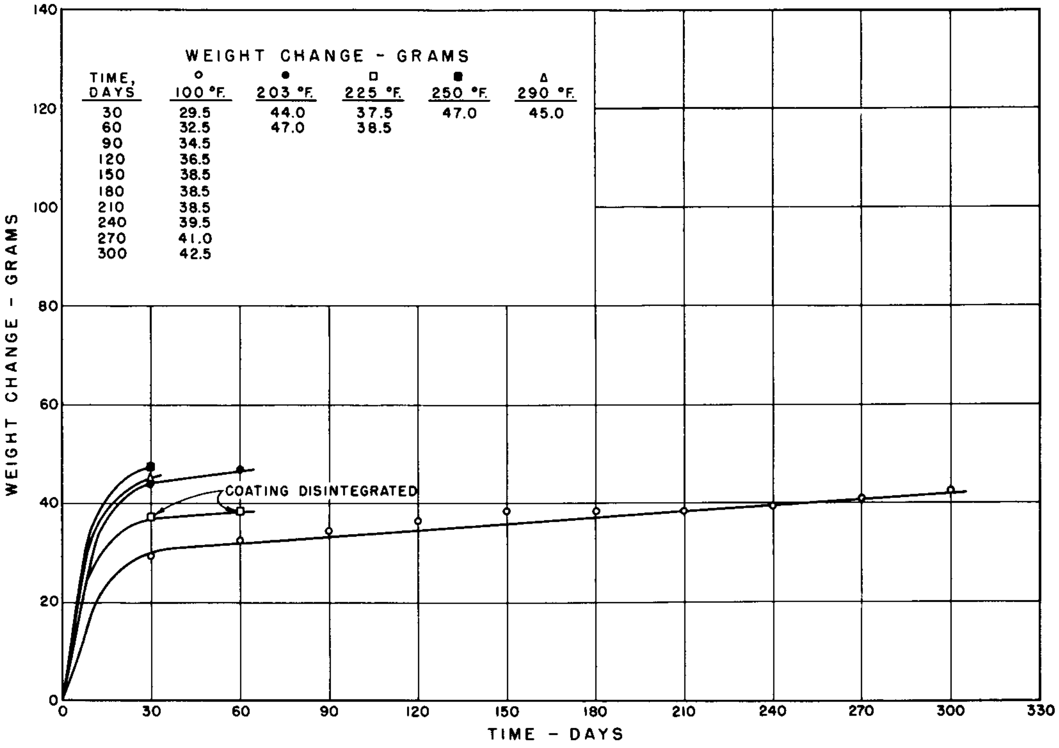


WEIGHT CHANGE - GRAMS

TIME, DAYS	○ 100 °F.	● 203 °F.	□ 225 °F.	■ 250 °F.	△ 290 °F.
30	-0.5	-1.0	-1.0	-1.0	-8.0 ← EDGE EROSION OF METAL
60	-1.0	TEST STOPPED	TEST STOPPED	TEST STOPPED	-9.0
90	TEST STOPPED				-9.0 TEST STOPPED

B. ON STEEL

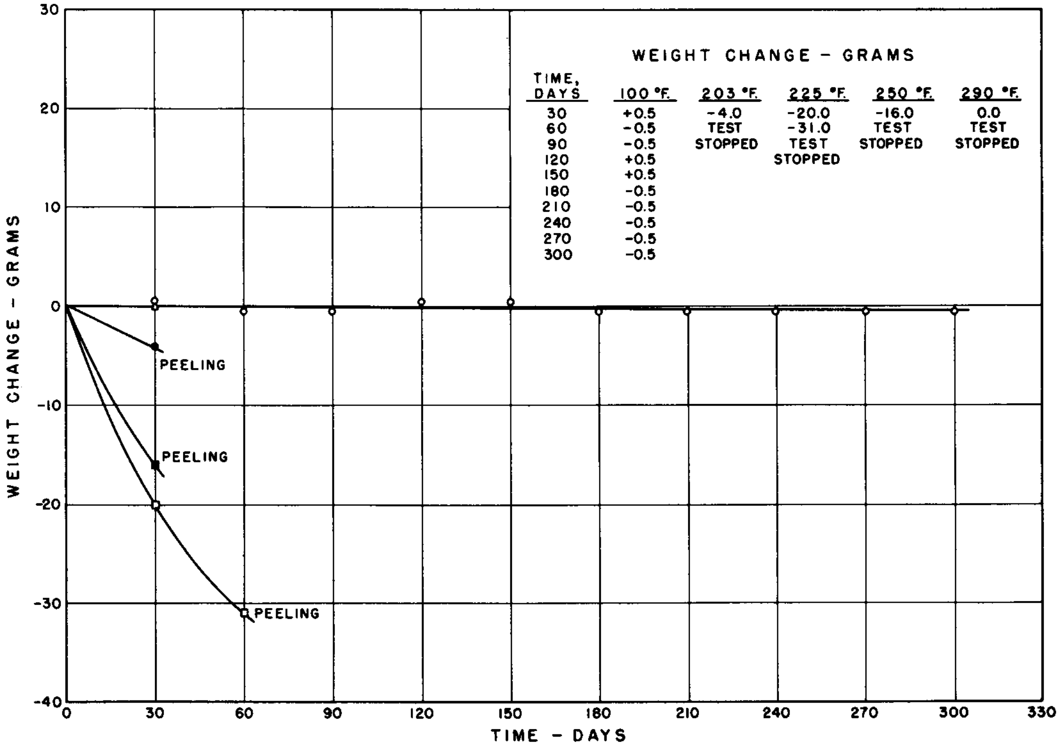
COATINGS C-33 & 34 - WEIGHT CHANGE VS TIME
FIGURE V - 49



WEIGHT CHANGE - GRAMS

TIME, DAYS	○ 100 °F.	● 203 °F.	□ 225 °F.	■ 250 °F.	Δ 290 °F.
30	29.5	44.0	37.5	47.0	45.0
60	32.5	47.0	38.5		
90	34.5				
120	36.5				
150	38.5				
180	38.5				
210	38.5				
240	39.5				
270	41.0				
300	42.5				

A. ON CONCRETE

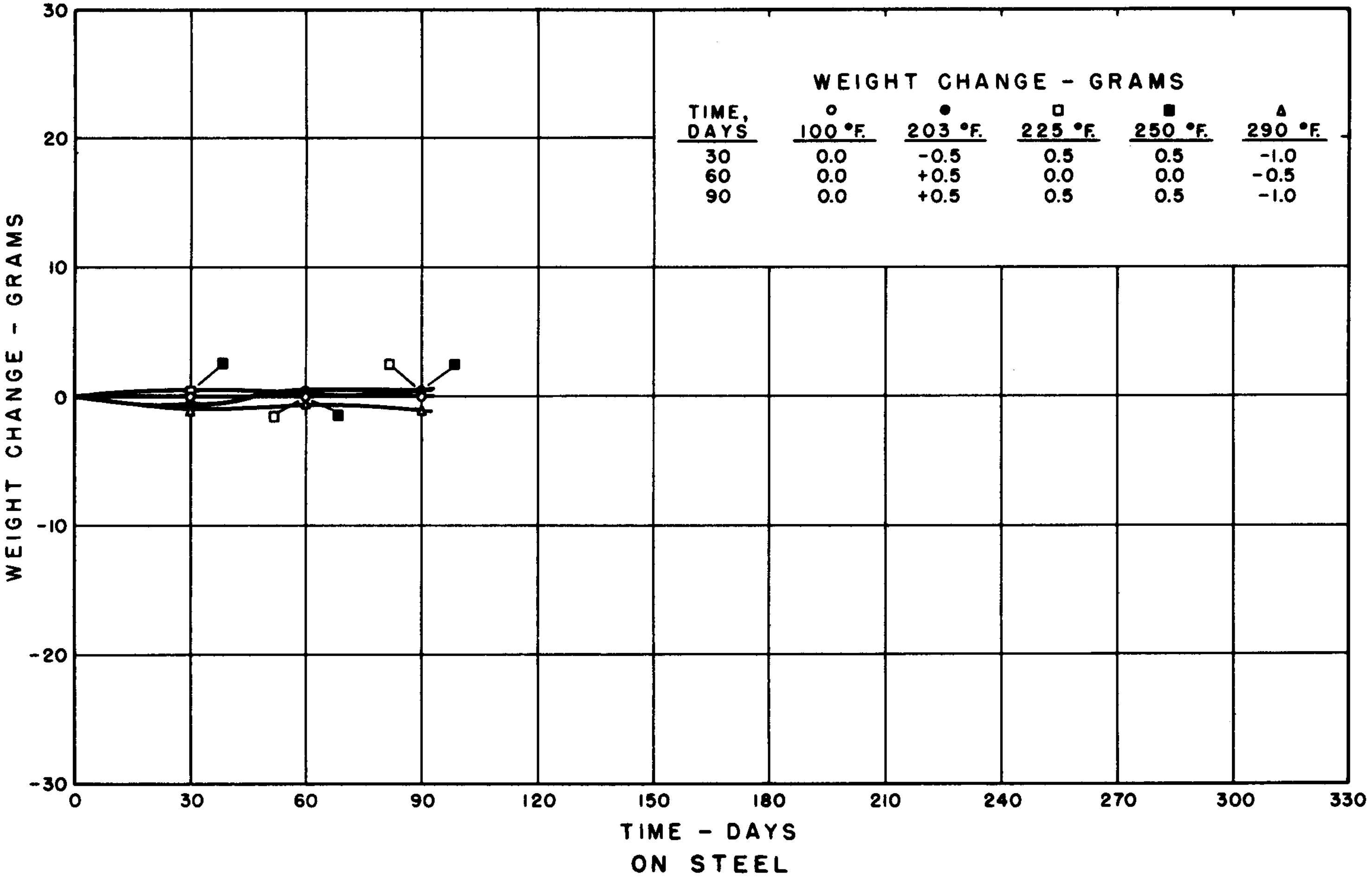


WEIGHT CHANGE - GRAMS

TIME, DAYS	100 °F.	203 °F.	225 °F.	250 °F.	290 °F.
30	+0.5	-4.0	-20.0	-16.0	0.0
60	-0.5	TEST	-31.0	TEST	TEST
90	-0.5	STOPPED	TEST	STOPPED	STOPPED
120	+0.5		STOPPED		
150	+0.5				
180	-0.5				
210	-0.5				
240	-0.5				
270	-0.5				
300	-0.5				

B. ON STEEL

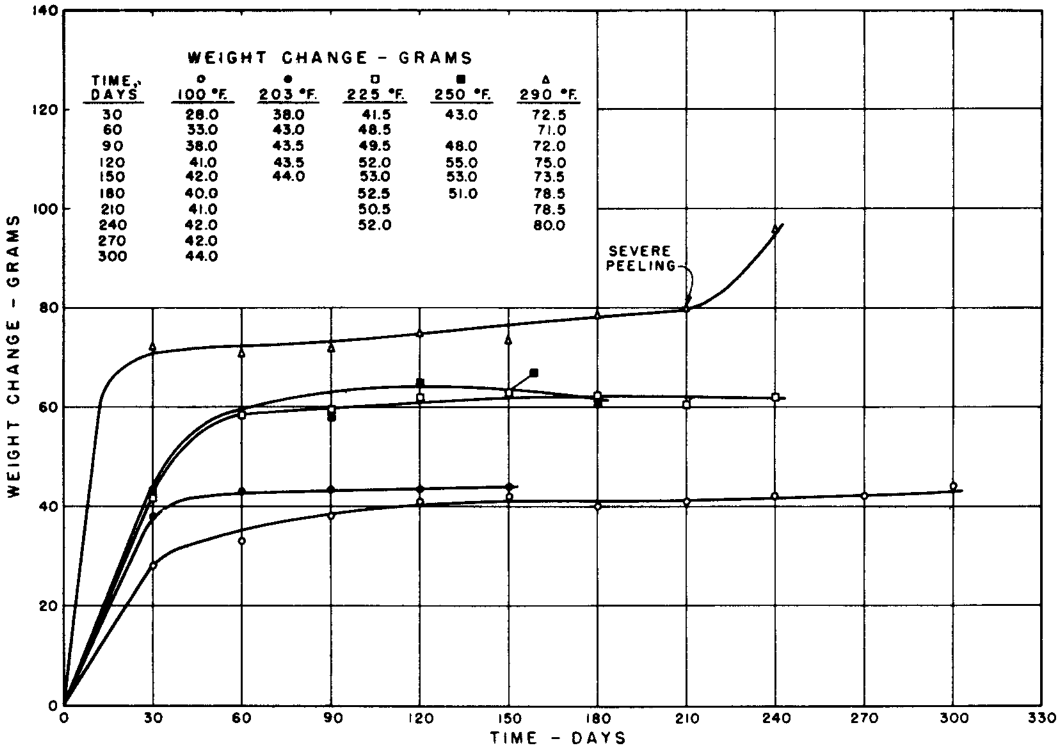
COATING C-36 - WEIGHT CHANGE VS TIME
FIGURE V-50



WEIGHT CHANGE - GRAMS

TIME, DAYS	○ 100 °F.	● 203 °F.	□ 225 °F.	■ 250 °F.	▵ 290 °F.
30	0.0	-0.5	0.5	0.5	-1.0
60	0.0	+0.5	0.0	0.0	-0.5
90	0.0	+0.5	0.5	0.5	-1.0

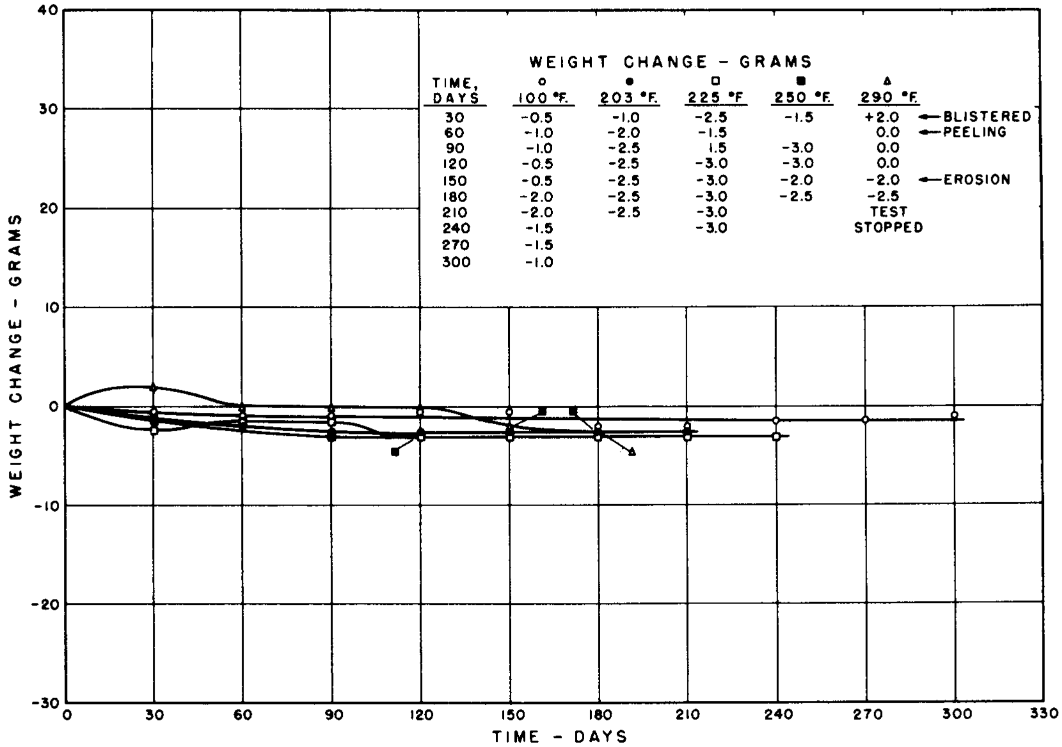
COATING C-37 - WEIGHT CHANGE VS TIME
FIGURE V-51



WEIGHT CHANGE - GRAMS

TIME, DAYS	○ 100 °F.	● 203 °F.	□ 225 °F.	■ 250 °F.	△ 290 °F.
30	28.0	38.0	41.5	43.0	72.5
60	33.0	43.0	48.5		71.0
90	38.0	43.5	49.5	48.0	72.0
120	41.0	43.5	52.0	55.0	75.0
150	42.0	44.0	53.0	53.0	73.5
180	40.0		52.5	51.0	78.5
210	41.0		50.5		78.5
240	42.0		52.0		80.0
270	42.0				
300	44.0				

A. ON CONCRETE

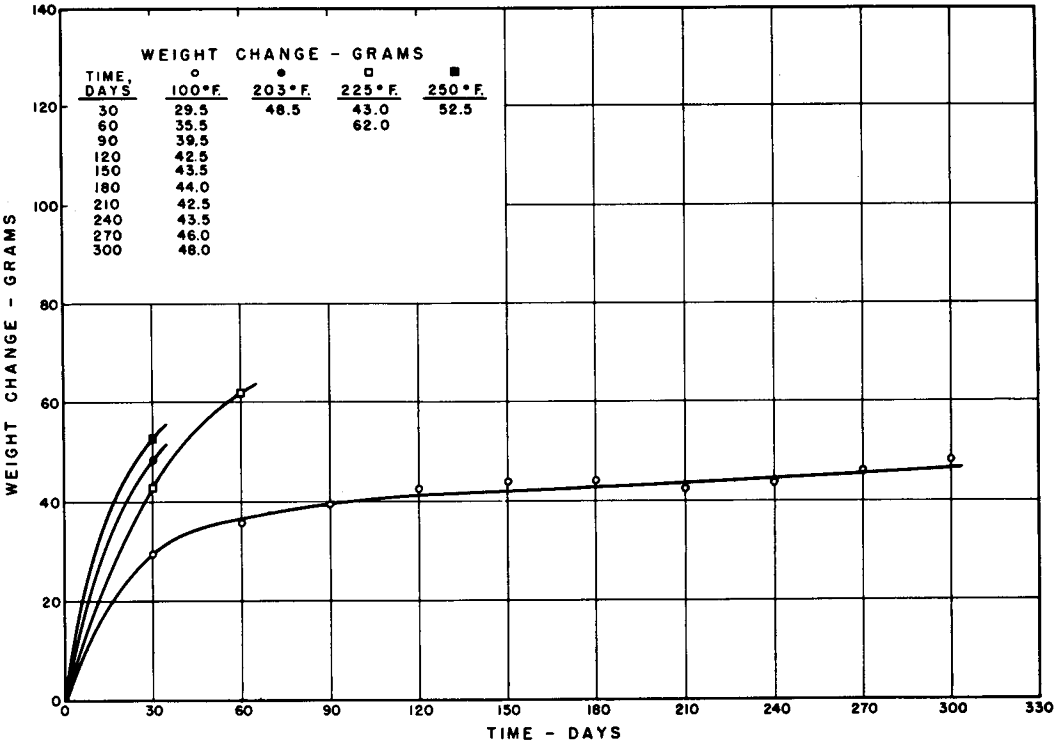


WEIGHT CHANGE - GRAMS

TIME, DAYS	○ 100 °F.	● 203 °F.	□ 225 °F	■ 250 °F.	△ 290 °F.
30	-0.5	-1.0	-2.5	-1.5	+2.0 ← BLISTERED
60	-1.0	-2.0	-1.5		0.0 ← PEELING
90	-1.0	-2.5	1.5	-3.0	0.0
120	-0.5	-2.5	-3.0	-3.0	0.0
150	-0.5	-2.5	-3.0	-2.0	-2.0 ← EROSION
180	-2.0	-2.5	-3.0	-2.5	-2.5
210	-2.0	-2.5	-3.0		TEST
240	-1.5		-3.0		STOPPED
270	-1.5				
300	-1.0				

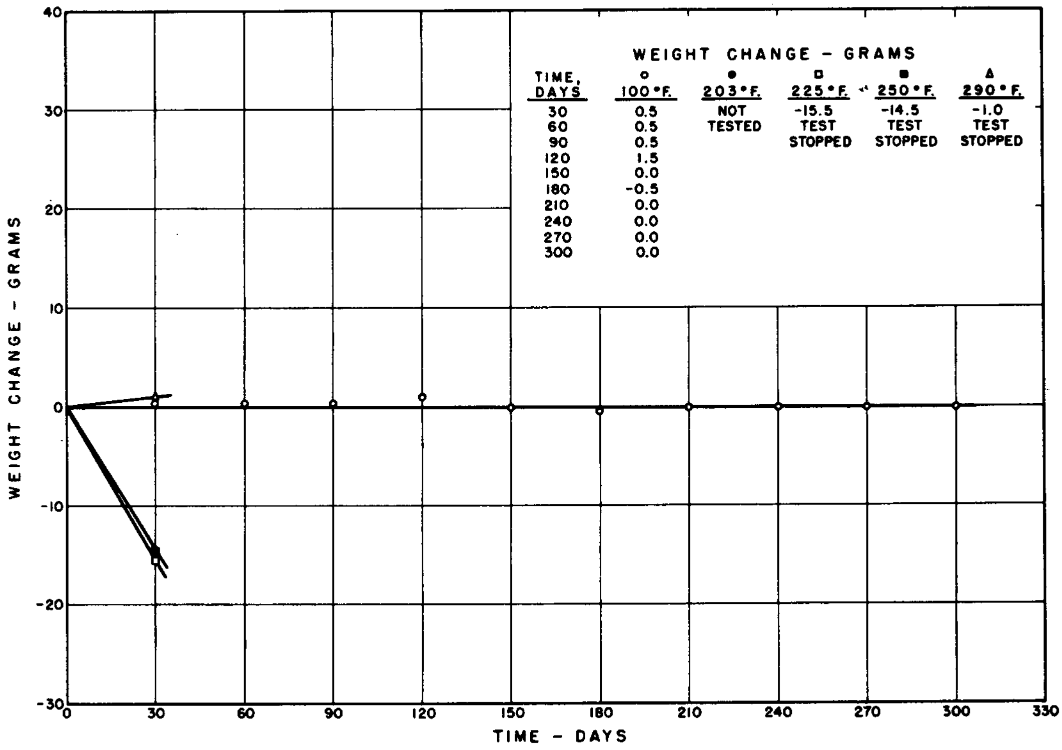
B. ON STEEL

COATING C-38 - WEIGHT CHANGE VS TIME
FIGURE V - 52



TIME, DAYS	WEIGHT CHANGE - GRAMS ○ 100°F.	 ● 203°F.	 □ 225°F.	 ■ 250°F.
30	29.5	48.5	43.0	52.5
60	35.5		62.0	
90	39.5			
120	42.5			
150	43.5			
180	44.0			
210	42.5			
240	43.5			
270	46.0			
300	48.0			

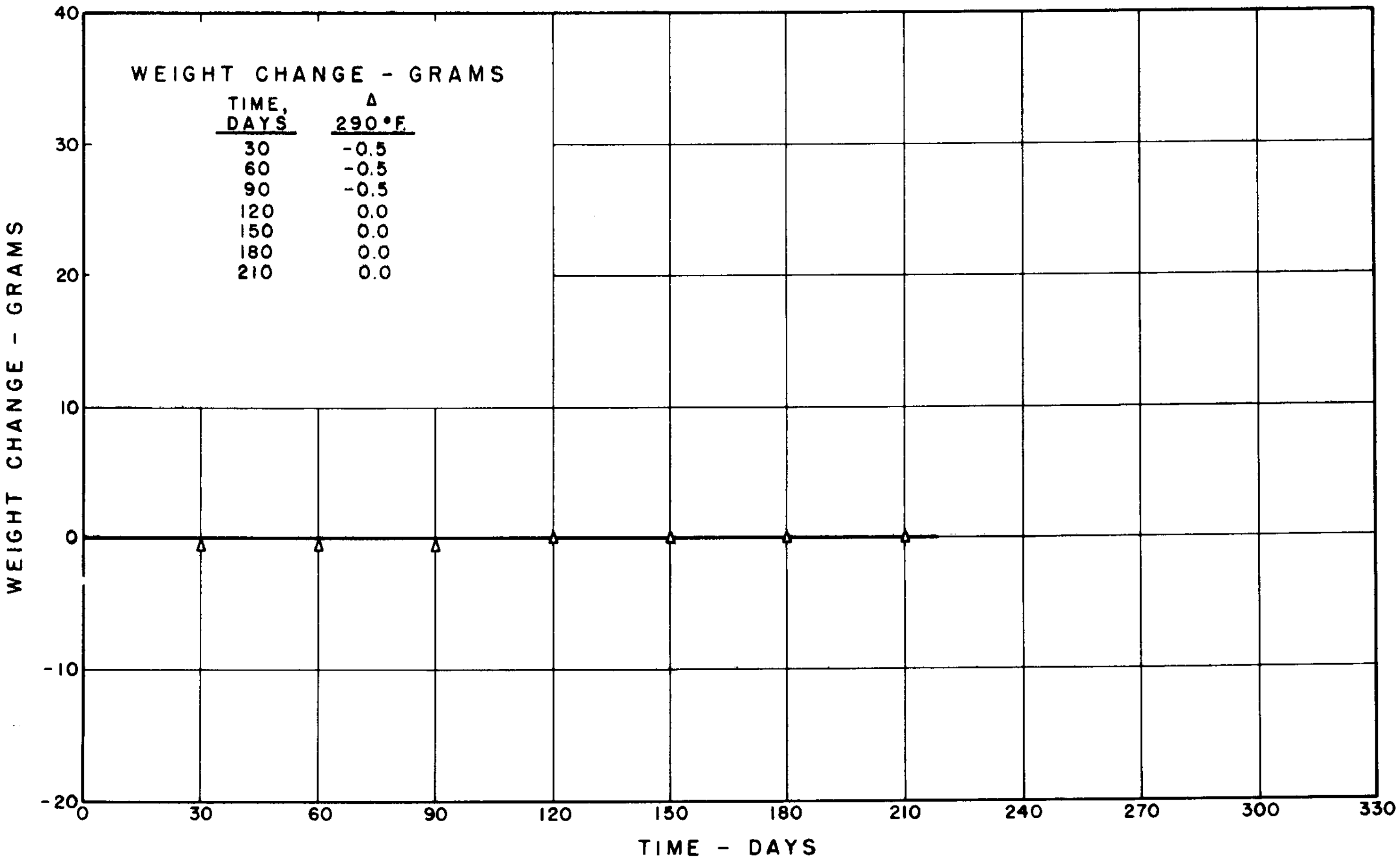
A. ON CONCRETE



TIME, DAYS	WEIGHT CHANGE - GRAMS ○ 100°F.	 ● 203°F.	 □ 225°F.	 ■ 250°F.	 △ 290°F.
30	0.5	NOT TESTED	-15.5 TEST STOPPED	-14.5 TEST STOPPED	-1.0 TEST STOPPED
60	0.5				
90	0.5				
120	1.5				
150	0.0				
180	-0.5				
210	0.0				
240	0.0				
270	0.0				
300	0.0				

B. ON STEEL

COATING C-39 - WEIGHT CHANGE VS TIME
FIGURE V-53

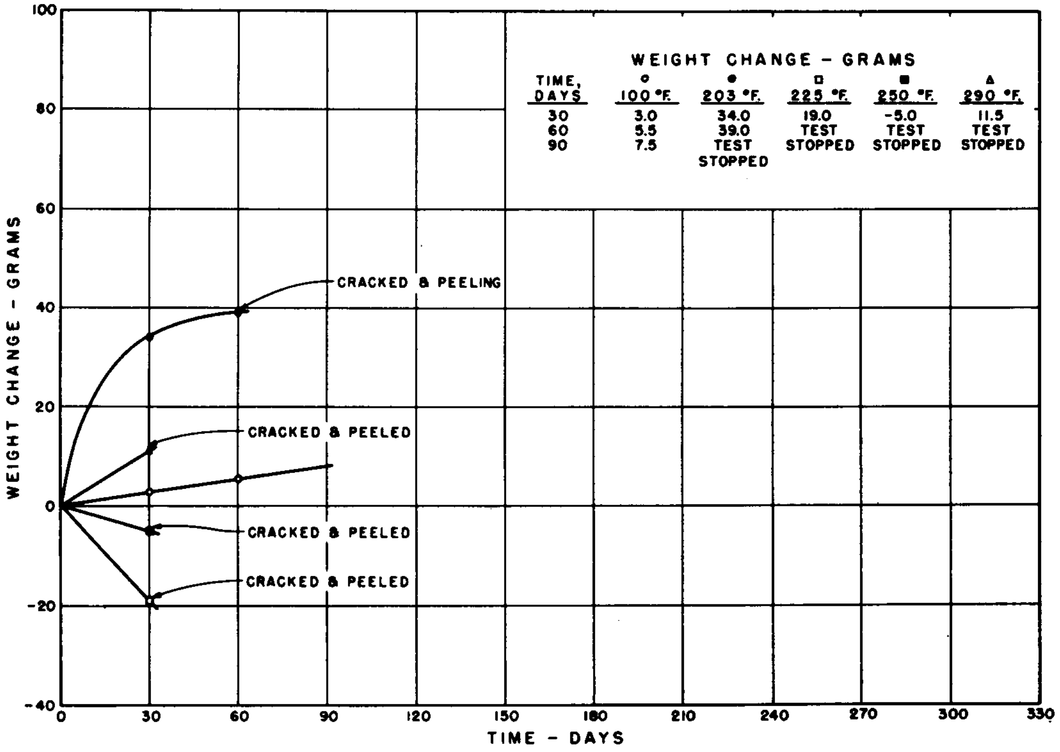


WEIGHT CHANGE - GRAMS

TIME, DAYS	Δ 290°F.
30	-0.5
60	-0.5
90	-0.5
120	0.0
150	0.0
180	0.0
210	0.0

ON STEEL

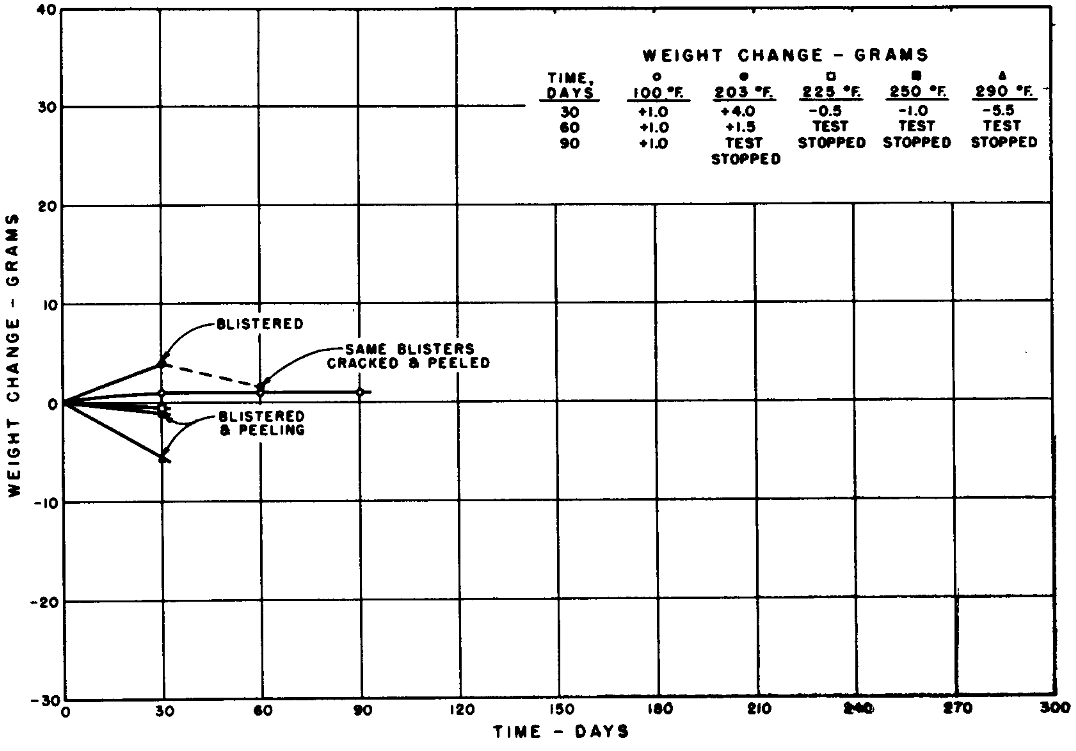
COATING C-40 - WEIGHT CHANGE VS TIME
FIGURE V-54



WEIGHT CHANGE - GRAMS

TIME, DAYS	○ 100 °F.	● 203 °F.	□ 225 °F.	■ 250 °F.	▲ 290 °F.
30	3.0	34.0	19.0	-5.0	11.5
60	5.5	39.0	TEST	TEST	TEST
90	7.5	TEST STOPPED	STOPPED	STOPPED	STOPPED

A. ON CONCRETE



WEIGHT CHANGE - GRAMS

TIME, DAYS	○ 100 °F.	● 203 °F.	□ 225 °F.	■ 250 °F.	▲ 290 °F.
30	+1.0	+4.0	-0.5	-1.0	-5.5
60	+1.0	+1.5	TEST	TEST	TEST
90	+1.0	TEST STOPPED	STOPPED	STOPPED	STOPPED

B. ON STEEL

COATING C-43 - WEIGHT CHANGE VS TIME
FIGURE V - 55

V.5.2.26. Coating C-44.--

Coating C-44, a flexible epoxy, developed cracking and peeling in 30 days at 203° F on concrete and steel. Since the cracking and peeling were severe, further exposure at this temperature was discontinued. After 150 days at 100° F, the flexible epoxy appears to be satisfactory as shown in Figure V-56.

V.5.2.27 Coating C-45.--

Coating C-45, a polyimide, applied only on steel, has performed well at 100° F as indicated in Figure V-57; however, the coating disintegrated at 203°, 225°, 250°, and 290° F.

V.5.2.28. Coating C-47.--

Coating C-47, an epoxy system, on steel and concrete has performed well for 90 days at 100°, 203°, 225°, 250°, and 290° F. As indicated in Figure V-58A, the 203°, 225°, 250°, and 290° F, the epoxy reached plateaus at 60 days for the weight change versus time curves. The 100° F specimen shows a straight line weight change versus time for the first 90 days. On steel small drops in weight are detected and are possibly caused by solvent losses.

V.5.2.29. Coating C-48.--

Coating C-48, an inorganic zinc with stainless steel finish, was applied only on steel. Surface erosion developed at 203°, 225°, 250° and 290° F causing failure of the coating at these temperatures. This coating appears to be satisfactory at 100° F after 90 days as indicated in Figure V-59.

V.5.2.30. Coating C-49.--

Coating C-49, a silicone, shown in Figure V-60, showed early bond failure and large blisters at 203°, 225°, 250°, and 290° F on concrete. Bond failures on steel at 250° and 290° F are also evident. The coating has kept its rubberiness at all temperatures; however, because of the bond failures it appears to be satisfactory only at 100° F on steel and concrete.

V.5.2.31. Coating C-50.--

Coating C-50, a polyurethane, shown in Figure V-61, had only 30 days exposure at report time. It looks poor at 203° F having formed numerous pinhead blisters on both steel and concrete. After 30 days at 100° F, it appears to be satisfactory on both steel and concrete.

V.5.2.32. Coatings C-51 and C-52.--

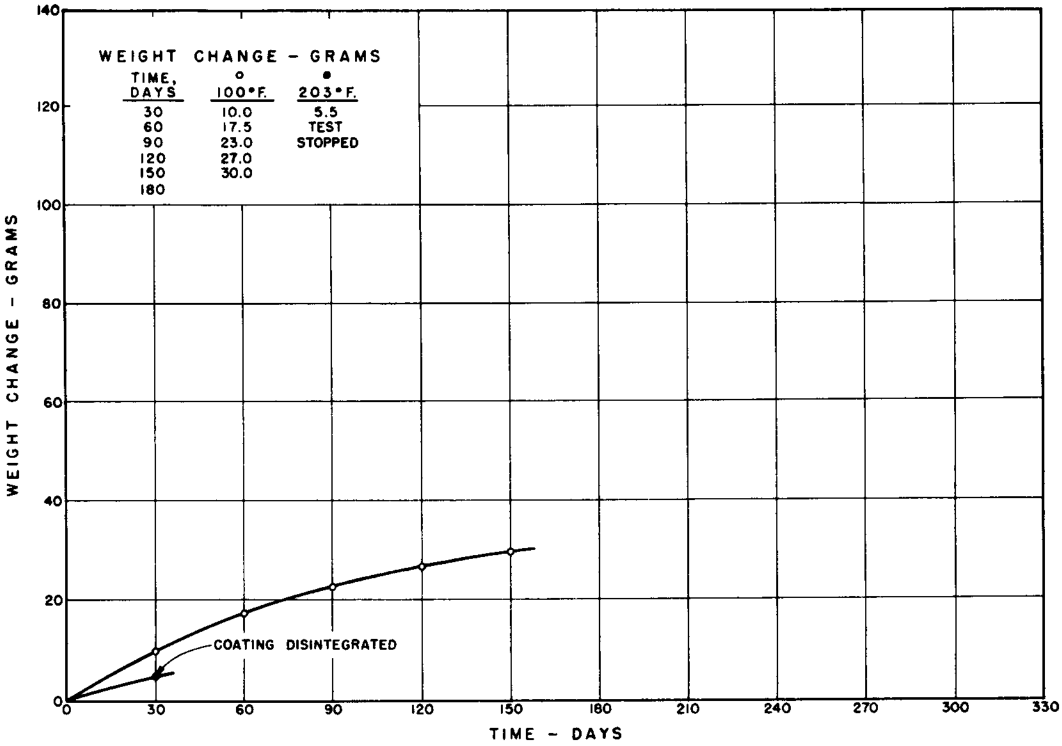
Coating C-51, a modified epoxy, and Coating C-52, sulfur, were started in April 1967. There are no results available for this report.

V.5.2.33. Coating C-53.--

An epoxy, Coating C-53, which is used only to line a pipe in the 290° F coating's loop has performed satisfactorily after 90 days' test.

V.5.2.34. Water absorption of concrete.--

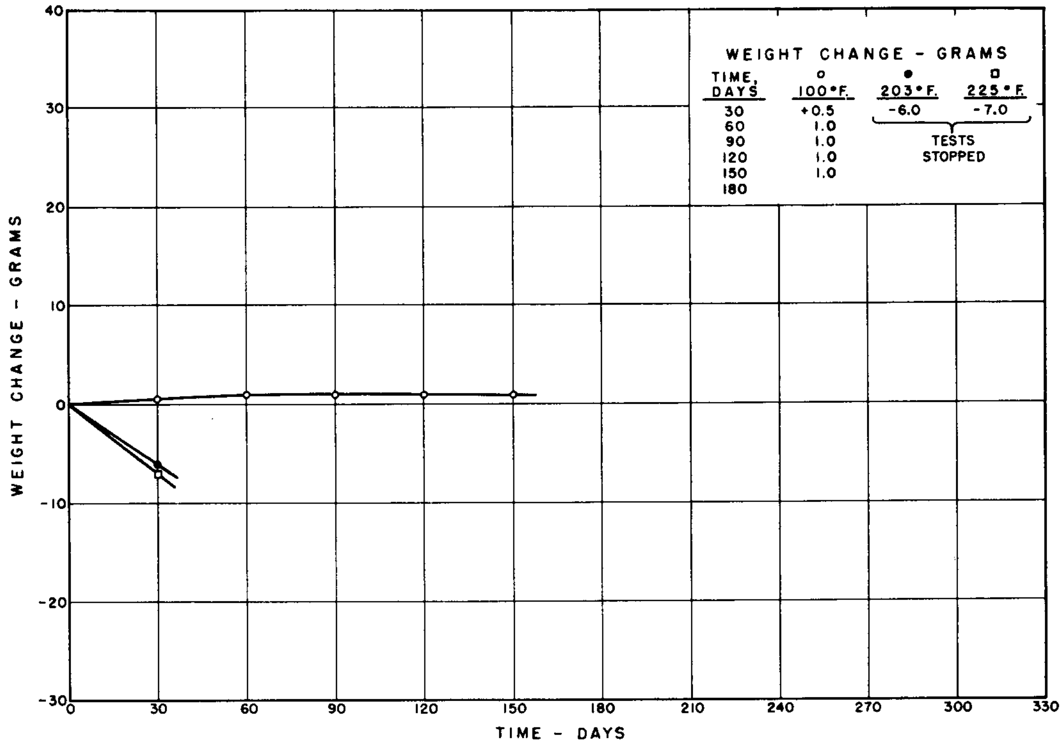
Figure V-62, water absorption of the concrete specimens used for coating studies shows most of the water was absorbed in the first 5 hours. After 70 hours at 77° F, the specimens absorbed 103.5 to 105.0 grams of water. These curves show that some coatings have allowed near the maximum possible water absorption.



WEIGHT CHANGE - GRAMS

TIME, DAYS	○ 100°F.	● 203°F.
30	10.0	5.5
60	17.5	TEST STOPPED
90	23.0	
120	27.0	
150	30.0	
180		

A. ON CONCRETE

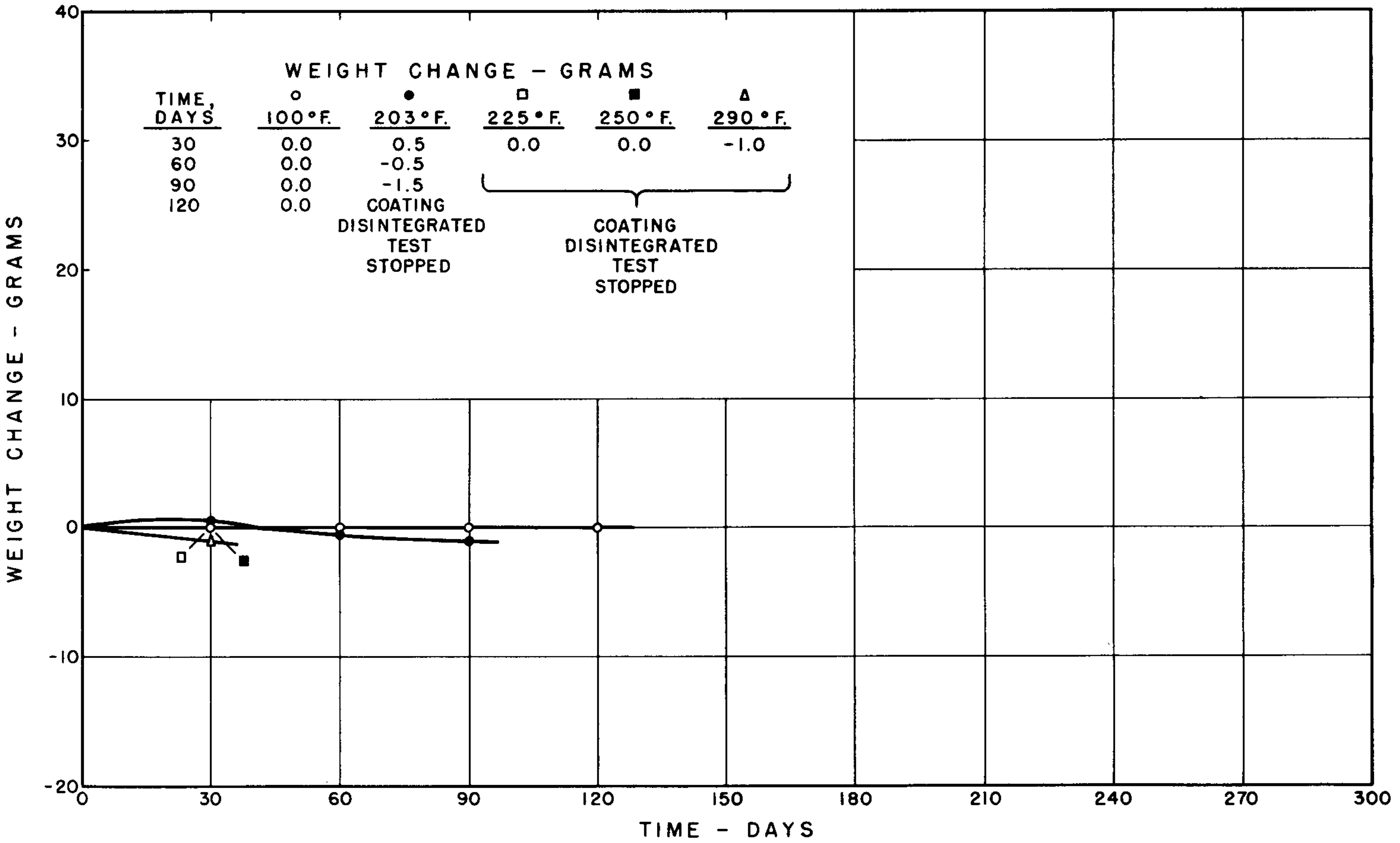


WEIGHT CHANGE - GRAMS

TIME, DAYS	○ 100°F.	● 203°F.	□ 225°F.
30	+0.5	-6.0	-7.0
60	1.0	TESTS STOPPED	
90	1.0		
120	1.0		
150	1.0		
180			

B. ON STEEL

COATING C-44 - WEIGHT CHANGE VS TIME
FIGURE V-56



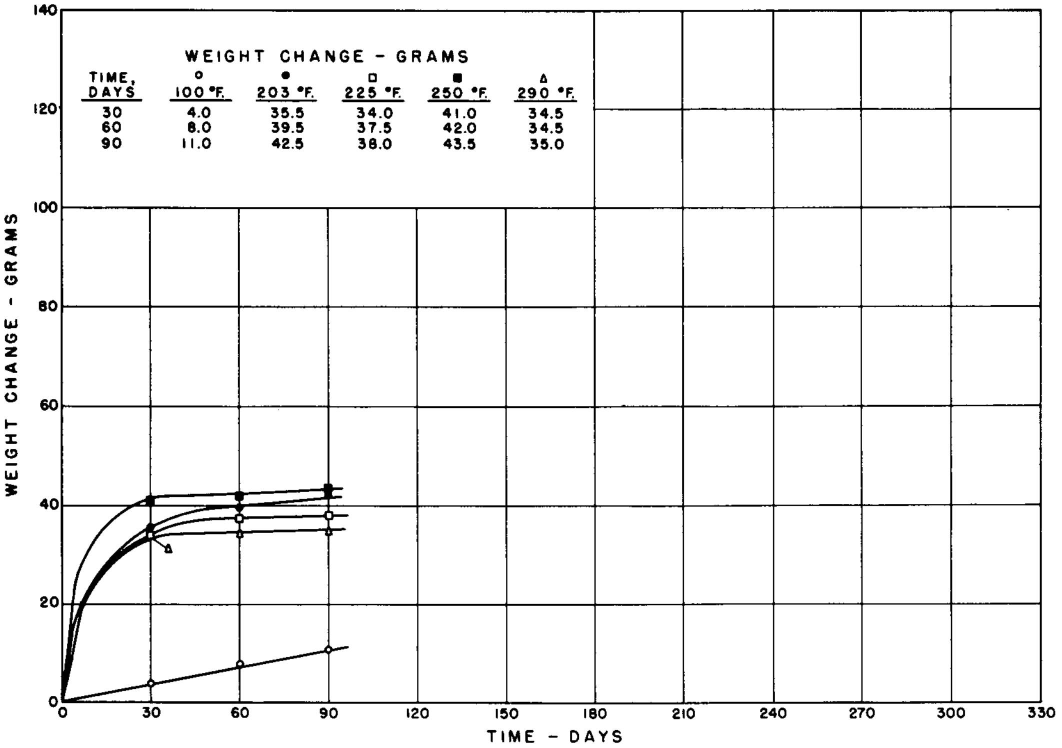
WEIGHT CHANGE - GRAMS

TIME, DAYS	○ 100°F.	● 203°F.	□ 225°F.	■ 250°F.	▲ 290°F.
30	0.0	0.5	0.0	0.0	-1.0
60	0.0	-0.5	COATING DISINTEGRATED TEST STOPPED		
90	0.0	-1.5			
120	0.0	COATING DISINTEGRATED TEST STOPPED			

ON STEEL

COATING C-45 - WEIGHT CHANGE VS TIME

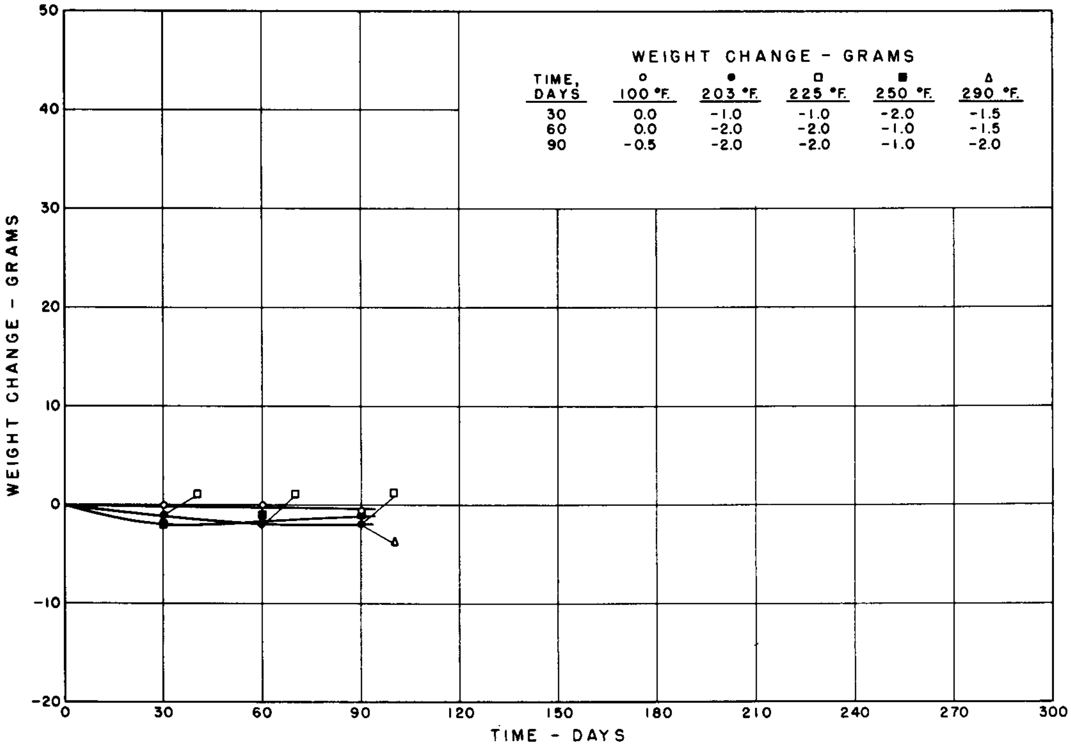
FIGURE V-57



WEIGHT CHANGE - GRAMS

TIME, DAYS	○ 100 °F.	● 203 °F.	□ 225 °F.	■ 250 °F.	Δ 290 °F.
30	4.0	35.5	34.0	41.0	34.5
60	8.0	39.5	37.5	42.0	34.5
90	11.0	42.5	38.0	43.5	35.0

A. ON CONCRETE

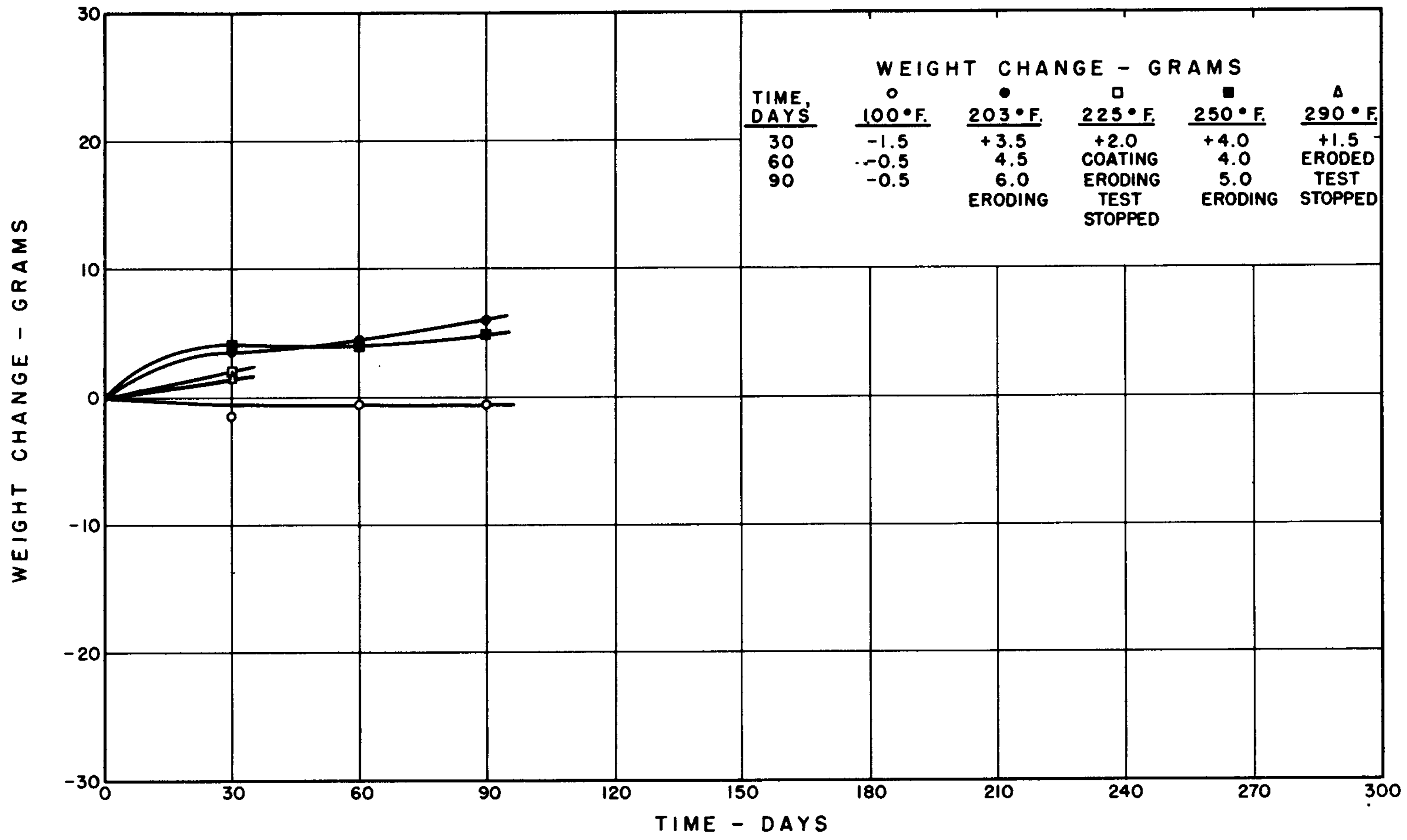


WEIGHT CHANGE - GRAMS

TIME, DAYS	○ 100 °F.	● 203 °F.	□ 225 °F.	■ 250 °F.	Δ 290 °F.
30	0.0	-1.0	-1.0	-2.0	-1.5
60	0.0	-2.0	-2.0	-1.0	-1.5
90	-0.5	-2.0	-2.0	-1.0	-2.0

B. ON STEEL

COATING C-47 - WEIGHT CHANGE VS TIME
FIGURE V-58



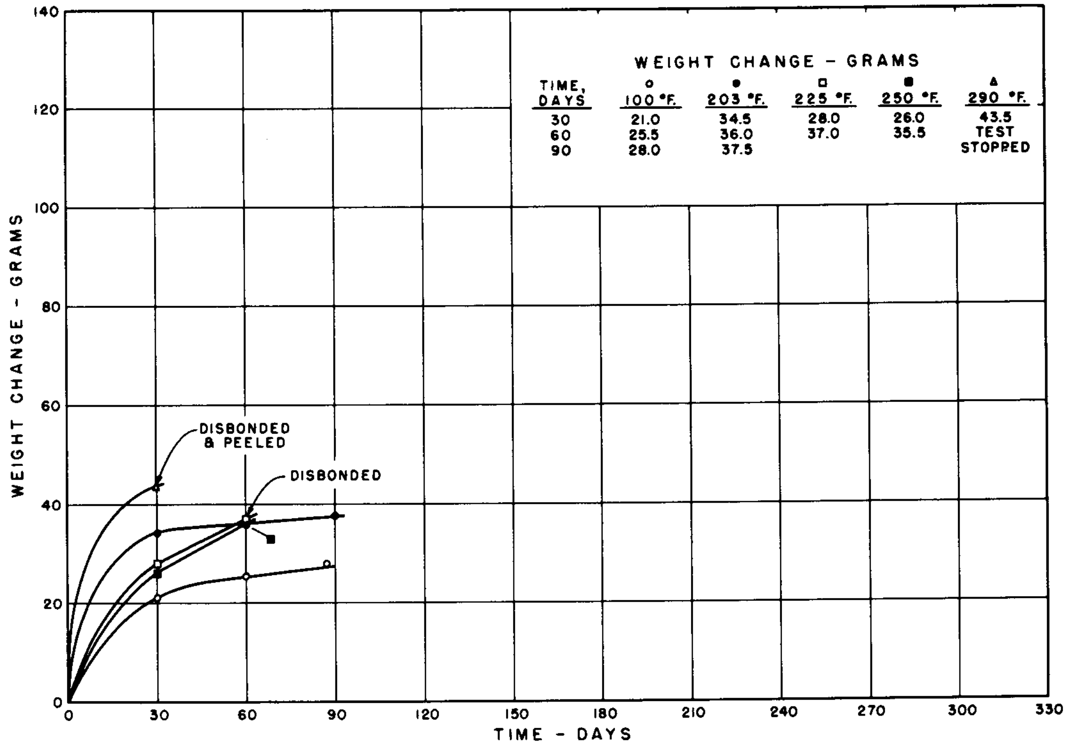
WEIGHT CHANGE - GRAMS

TIME, DAYS	○ 100°F.	● 203°F.	□ 225°F.	■ 250°F.	Δ 290°F.
30	-1.5	+3.5	+2.0	+4.0	+1.5
60	-0.5	4.5	COATING	4.0	ERODED
90	-0.5	6.0	ERODING	5.0	TEST
		ERODING	TEST	ERODING	STOPPED
			STOPPED		

ON STEEL

COATING C-48 - WEIGHT CHANGE VS TIME

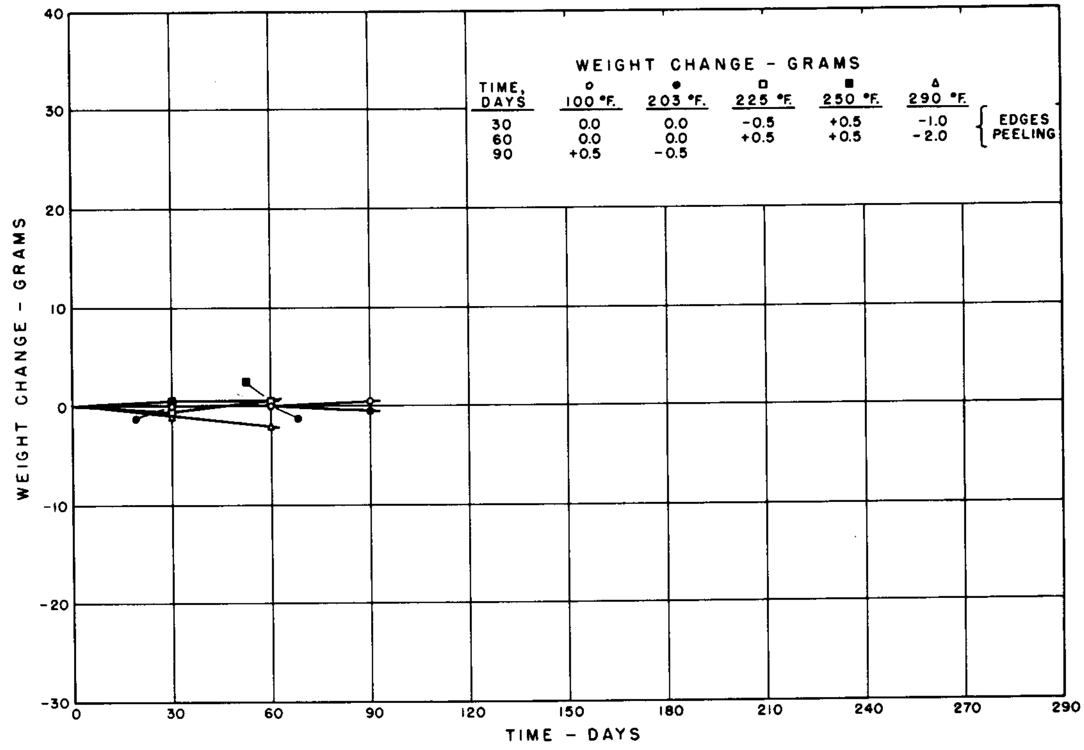
FIGURE V-59



WEIGHT CHANGE - GRAMS

TIME, DAYS	○ 100 °F.	● 203 °F.	□ 225 °F.	■ 250 °F.	▲ 290 °F.
30	21.0	34.5	28.0	26.0	43.5
60	25.5	36.0	37.0	35.5	TEST STOPPED
90	28.0	37.5			

A. ON CONCRETE

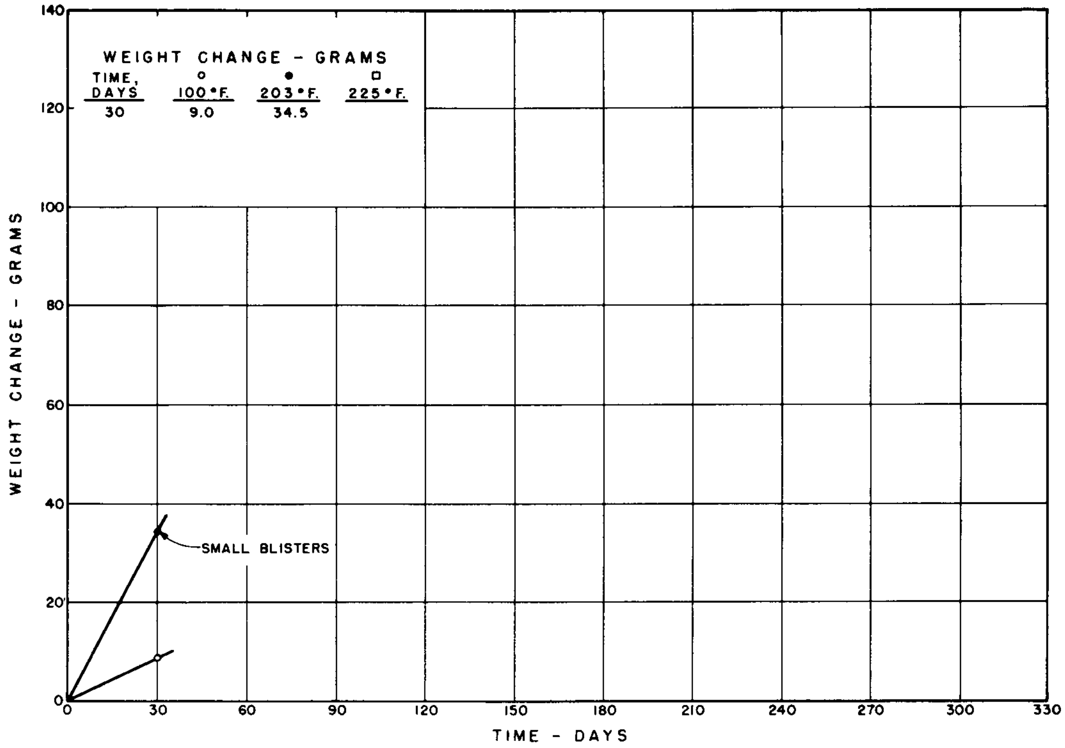


WEIGHT CHANGE - GRAMS

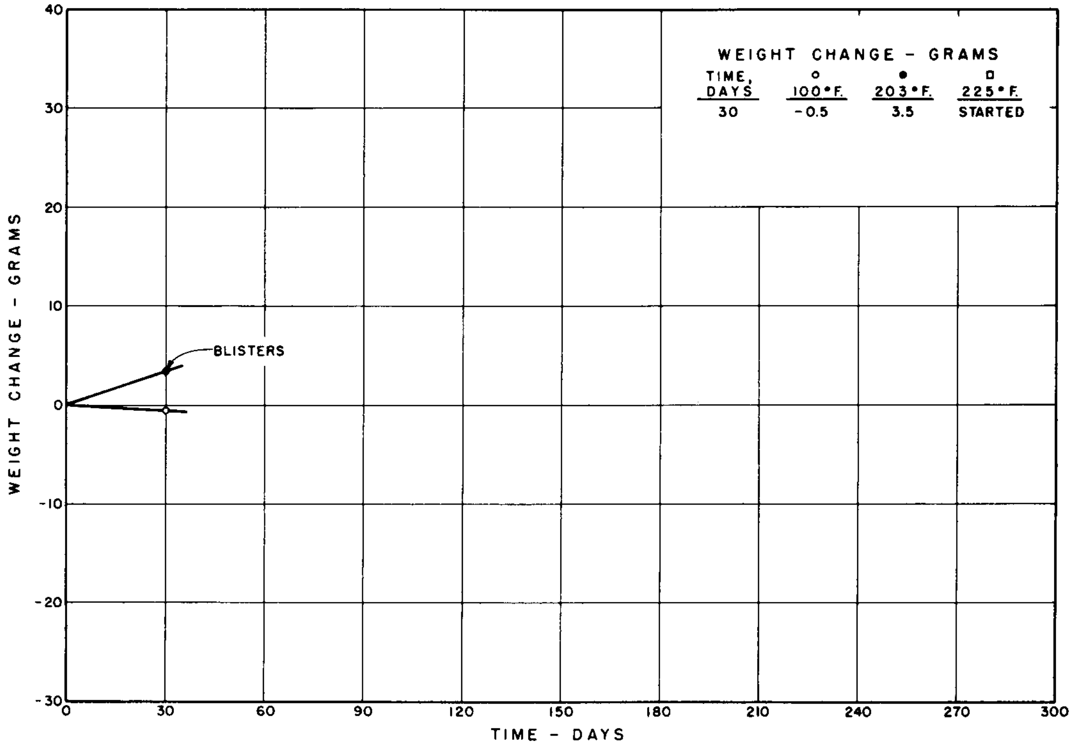
TIME, DAYS	○ 100 °F.	● 203 °F.	□ 225 °F.	■ 250 °F.	▲ 290 °F.	
30	0.0	0.0	-0.5	+0.5	-1.0	EDGES PEELING
60	0.0	0.0	+0.5	+0.5	-2.0	
90	+0.5	-0.5				

B. ON STEEL

COATING C-49 – WEIGHT CHANGE VS TIME
FIGURE V - 60



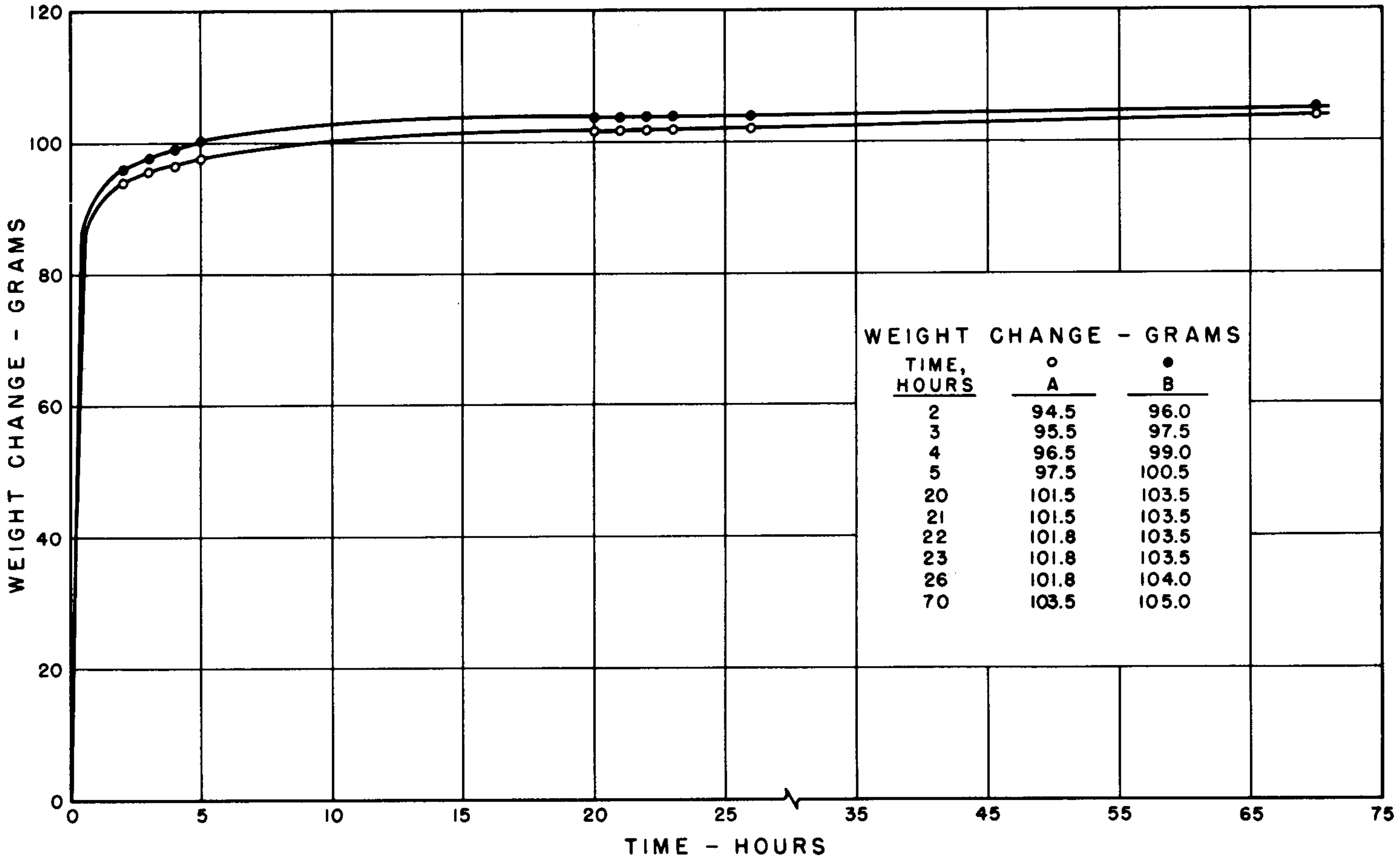
A. ON CONCRETE



B. ON STEEL

COATING C-50 - WEIGHT CHANGE VS TIME

FIGURE V-61



WEIGHT CHANGE - GRAMS

TIME, HOURS	○ A	● B
2	94.5	96.0
3	95.5	97.5
4	96.5	99.0
5	97.5	100.5
20	101.5	103.5
21	101.5	103.5
22	101.8	103.5
23	101.8	103.5
26	101.8	104.0
70	103.5	105.0

ON CONCRETE

WATER ABSORPTION - WEIGHT CHANGE VS TIME

FIGURE V - 62

V. 5. 3. Joint Sealers

Joint Sealer J-2 is the only joint material still under test. Specimens are being exposed in all brine environments. No performance data are yet available, but the appearance is good to date.

The other joint materials showed early disintegration at all temperatures except 100° F and are not suitable for the environment.

V. 6. Conclusions

Major conclusions reached from tests conducted to date are as follows:

*1. Coating materials listed in Table V-1 are not suitable for extended 350° F wet exposure. (February 1966)

*2. RTV silicones and high-temperature polysulfides of the types tested (Table V-1) are not suitable in 350° F wet exposure as joint sealants. (February 1966)

3. Epoxy-asphalt concrete is not suitable in 350° F wet exposure. (February 1966)

4. Degassing and drying of concrete or mortar surfaces prior to coating applications will be necessary to prevent coating disruptions from entrapped gases and water as the temperature of coated surfaces increases above the application temperature. (February 1966)

5. The test results and visual observations indicate that the following rubber materials would not provide satisfactory service at the brine temperatures indicated and testing has been discontinued at these conditions:

R-9	290° F
R-18	290° F
R-19	203°, 225°, 250°, 290° F
R-22	290° F
R-24	290° F
R-27	250°, 290° F
R-28	250°, 290° F

(April 1967)

6. EPT and butyl-rubber materials generally appear to date to be least affected by any exposure condition; the exceptions to this point out the importance of formulation within families of elastomers. (April 1967)

7. Neoprene rubber coatings as a class are showing good performance to date on both steel and concrete at all test temperatures through 290° F. Elastic properties are showing little change with increased exposure time. One type of neoprene joint sealant is showing good promise for extended service at 290° F. (April 1967)

8. The phenolic and epoxy modified phenolics as a class are showing excellent performance on steel at all test temperatures up through 290° F. These coatings are showing good performance on concrete with just occasional occurrence of small cracks amenable to repair through 290° F temperatures. (April 1967)

9. Many test materials such as the epoxies, vinyls, and others are performing well at 100° and/or 203° F, but are unsatisfactory at the higher temperatures. Better performance is generally found on steel substrates. (April 1967)

10. Moisture transfer rates through the test coatings on concrete increase as the temperature of the exposure increases as evidenced by greater weight increases at the higher temperatures. (April 1967)

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*Conclusions from preliminary tests; see Sections V. 3. and V. 3. 1.

11. It appears that the materials that are showing degradation and changes are being affected more by the temperature levels of the wet exposure than by the brine concentration. (April 1967)

12. Sufficient data are not yet available to predict useful lives or behavior trends of the various materials showing good service at the various test conditions. (April 1967)

APPENDIX V-7.

V.7.1. Electronic Data Processing

In order to eliminate the tedious and lengthy process of calculating specific gravity, average thickness, tear strength, tensile strength and sample volume, a small computer program was written. The program was then enlarged to allow for automatic sorting and tabulation of final data. A work form was prepared to allow measurements and instrument readings to be recorded directly on punch card data sheets. The computer program and a standardized documentation are included on the following pages.

```
      PROGRAM C2JSAL
     0DIMENSION SN(100), AGE (100.11), BLNG (100.11), WTH (100.11), NC (100),
     1TWTH (100.11), TNTH (100.11), TRTH (100.11), WT (100.11), SMWT (100.11),
     2TNLD (100.11), TRLD (100.11), ELONG (100.11), SHORE (100.11), TH (100.11),
     3SPG (100.11), TEAR (100.11), TENSIL (100.11), VOLUME (100.11), CK (13)
      READ (2.1) N, AST
    1 FORMAT (I4,4X,A4)
      READ (2; 2) (SN(J), J=1,N)
    2 FORMAT (10F8.0)
C     N IS THE NUMBER OF SAMPLES BEING SENT
C     SN(J) ARE THE SAMPLE NUMBERS
      DO 4 K=1, N
    4 NC(K) = 0
      DO 5 K=1, 1000
      READ (2.3) (CK(J), J=1,13), FINAL
    3 FORMAT (F8.0,F4.0,F6.0, 2F5.0, 8F6.0, A4)
      DO 6 L = 1, N
      IF (CK(1) .EQ. SN(L)) GO TO 7
    6 CONTINUE
      WRITE (3, 8) CK(1)
    8 FORMAT (1HO, 5X, 36H THERE IS DISAGREEMENT IN SAMPLE NO.,F8.1)
      GO TO 10
    7 NC(L) = NC(L)+1
      NS = NC(L)
      AGE   (L,NS) = CK(2)
      BLNG  (L,NS) = CK(3)
      WTH   (L,NS) = CK(4)
      TWTH  (L,NS) = CK(5)
      TNTH  (L,NS) = CK(6)
      TRTH  (L,NS) = CK(7)
      WT    (L,NS) = CK(8)
      SMWT  (L,NS) = CK(9)
      TNLD  (L,NS) = CK(10)
      TRLD  (L,NS) = CK(11)
      ELONG (L,NS) = CK(12)
      SHORE (L,NS) = CK(13)
   10 IF (FINAL .EQ. AST) GO TO 9
    5 CONTINUE
    9 DO 11, K = 1, N
      NS = NC(K)
      IF (NS .EQ. 0) GO TO 11
      DO 12, JR = 1, NS
      TH(K,JR) = (TNTH(K,JR) + TRTH(K,JR))/ 2.0
      SPG(K,JR) = WT(K,JR)/ (WT(K,JR)-SMWT(K,JR))
      TEAR(K,JR)= TRLD(K,JR)/ TRTH(K,JR)
      TENSIL(K,JR) = TNLD(K,JR)/ (TWTH(K,JR)* TNTH(K,JR))
      VOLUME(K,JR) = WT(K,JR) - SMWT(K,JR)
   12 CONTINUE
   11 CONTINUE
      DO 15 K = 1, N
      WRITE (3, 16) SN(K)
   16 FORMAT (1H1, 5X, 14H SAMPLE NUMBER, F8.1)
      WRITE (3,17)
   170FORMAT (1HO, 5X, 80H   AGE    WT    LNG    WTH    TH    SPG    SHO
     1RE   TEAR   TENSIL   ELONG   VOLUME   )
      NS = NC(K)
```

```
        IF (NS .EQ. 0) GO TO 15
        DO 15 L = 1, NS
      0 WRITE (3,18) AGE(K,L), WT(K,L), BLNG(K,L), WTH(K,L), TH(K,L),
      1 SPG(K,L), SHORE(K,L), TEAR(K,L), TENSIL(K,L), ELONG(K,L).
      2 VOLUME(K,L)
   18 0 FORMAT (10X,F5.0,F7.1,2F6.2, F6.3, F6.2, F6.0, F8.0, F9.0, F8.0,
      1 F7.1)
   15   CONTINUE
        CALL EXIT
        END
```

```
PROGRAM DOCUMENTATION


PURPOSE
---------
*  COMPUTATION AND DISPLAY OF ELESTOMER DATA AFTER EXPOSURE
   TO SALINE WATER.

NECESSARY INPUT VARIABLES
--------------------------------
*  N. LAST, SN(I). AGE(I,J), BLNG(I,J), WTH(I,J), TWTH(I,J), TNTH(I,J).
   TRTH(I,J), WT(I,J), SMWT(I,J), TNLD(I,J), TRLD(I,J), ELONG(I,J)
   SHORE(I,J)

     ARRAY LIMITS
     --------------
*       SEE ABOVE - FOR ALL ARRAYS I = 100
                                   J = 11

     DESCRIPTIONS
     --------------
*      N = NUMBER OF SAMPLES
       LAST = CHECK VARIABLE, MUST BE IN FIRST AND LAST CARD.
       SN(I) = SAMPLE IDENTIFICATION CODE
       AGE(I,J) = MONTHS OF EXPOSURE
       BLNG(I,J) = SPECIMEN LENGTH - INCHES
       WTH(I,J) = SPECIMEN WIDTH - INCHES
       TWTH(,J) = TENSILE SPECIMEN WIDTH - INCHES
       TNTH(I,J) = TENSILE SPECIMEN THICKNESS - INCHES
       TRTH(I,J) = TEAR SPECIMEN THICKNESS - INCHES
       WT(I,J) = SAMPLE DRY WEIGHT - GRAMS
       SMWT(I,J) = SAMPLE IMMERSED WEIGHT - GRAMS
       TNLD(I,J) = ULTIMATE TENSIL LOAD - POUNDS
       TRLD(I,J) = TEAR LOAD - POUNDS
       ELONG(I,J) = ULTIMATE ELONGATION - PERCENT
       SHORE(I,J) = SHORE HARDNESS - A OR D UNITS
```

```
          VARIABLE TYPES
          --------------
*           STANDARD REAL, INTEGRS AND ALPHANUMERIC.

        OUTPUT VARIABLES
        ----------------
*         SN(I), AGE(I,J), WT(I,J), BLNG(I,J), WTH(I,J), TH(I,J), SPG(I,J)
          SHORE(I,J), TEAR(I,J), TENSIL(I,J), ELONG(I,J), VOLUME(I,J)

          ARRAYS
          ------
*           SEE ABOVE - FOR ALL ARRAYS  I = 100

          DESCRIPTIONS
          ------------
*           TH(I,J) = AVERAGE SAMPLE THICKNESS - INCHES
            SPG(I,J) = SPECIFIC GRAVITY AT 73 DEGREES F.
            TEAR(I,J) = TEARING STRENGTH - POUNDS PER INCH THICKNESS.
            TENSIL(I,J) = TENSILE STRENGTH - POUNDS PER SQUARE INCH.
            VOLUME(I,J) = SAMPLE VOLUME - CUBIC CM. (CONVERTED TO
                          CUBIC INCHES)

          VARIABLE TYPES
          --------------
*           ALL VARIABLES ARE STANDARD REAL AND INTEGRS.

        STORAGE REQUIRED
        ----------------
*         STORAGE REQUIRED IS ABOUT

        TAPES USED
        ----------
*         TAPE 2 READ
          TAPE 3 WRITE

        WRITTEN BY                    DATE
        ----------                    ----
*         BERNIE JONES                 4/3/67
          PHILLIP F. ENGER

        ADDITIONAL INFORMATION
        ----------------------
*         NONE

*         END DOCUMENTATION
```

Table V-7

PLASTIC P-1

Exposure	Time (days)	Weight (grams)	Length (inches)	Width (inches)	Thickness (inches)	Specific gravity	Shore	Tear	Tensile strength (lbs/in^2)	Elongation (percent)	Volume (cu in.)
Initial	0	86.8	5.88	5.87	0.124	1.24	86	0	10,081	120	4.28
100° F	30	85.8	5.88	5.88	0.124	1.24	86	0	10,307	138	4.21
	60	89.0	5.88	5.88	0.127	1.24	86	0	10,974	46	4.38
	90	84.8	5.88	5.88	0.122	1.24	86	0	10,874	95	4.17
	120	87.2	5.88	5.88	0.126	1.24	85	0	11,014	64	4.28
	150	85.8	5.88	5.88	0.123	1.24	85	0	10,807	22	4.21
	180	88.4	5.88	5.87	0.126	1.24	85	0	10,709	18	4.35
203° F	30	84.6	5.88	5.88	0.121	1.24	85	0	10,565	64	4.15
	60	85.4	5.87	5.87	0.121	1.24	86	0	11,254	27	4.19
	90	85.8	5.90	5.90	0.121	1.24	86	0	11,406	22	4.22
	120	87.5	5.85	5.89	0.124	1.24	85	0	11,240	61	4.32
	150	88.4	5.86	5.90	0.126	1.24	87	0	11,049	21	4.34
	180	85.0	5.88	5.88	0.121	1.24	86	0	11,145	85	4.17
225° F	30	88.1	5.88	5.88	0.125	1.24	86	0	10,590	35	4.33
	60	87.3	5.90	5.90	0.124	1.24	87	0	11,452	15	4.28
	90	87.5	5.90	5.90	0.126	1.24	85	0	11,561	19	4.31
	120	87.2	5.90	5.89	0.128	1.24	85	0	11,534	38	4.28
	150	87.2	5.90	5.90	0.126	1.24	85	0	11,588	12	4.28
	180	87.4	5.88	5.88	0.125	1.25	86	0	11,627	8	4.28
250° F	30	87.9	5.88	5.88	0.125	1.24	86	0	11,585	26	4.32
	60	87.8	5.90	5.90	0.126	1.24	86	0	11,567	23	4.32
	90	98.1	5.88	5.88	0.127	1.24	85	0	11,674	16	4.33
	120	87.0	5.89	5.89	0.125	1.24	85	0	11,490	14	4.27
290° F	30	86.0	5.88	5.88	0.121	1.24	87	0	11,752	12	4.22
	60	87.9	5.88	5.88	0.126	1.24	88	0	12,474	15	4.32
	90	86.4	5.90	5.90	0.123	1.24	86	0	12,609	9	4.25
	120	87.0	5.85	5.90	0.126	1.24	85	0	12,409	7	4.27
	150	87.2	5.89	5.88	0.124	1.24	86	0	12,429	9	4.28

Table V-8

PLASTIC P-3

Exposure	Time (days)	Weight (grams)	Length (inches)	Width (inches)	Thickness (inches)	Specific gravity	Shore	Tear	Tensile strength (lbs/in.^{2})	Elongation (percent)	Volume (cu in.)
Initial	0	27.1	3.99	3.99	0.125	1.08	87	0	7,754	4	1.53
100° F	30	27.1	3.98	3.98	0.125	1.08	87	0	9,870	4	1.53
	60	27.0	3.98	3.98	0.123	1.08	86	0	8,517	3	1.53
	90	27.5	3.98	3.98	0.126	1.08	88	0	9,486	4	1.57
	120	27.1	3.97	3.97	0.124	1.08	86	0	10,126	5	1.54
	150	26.5	3.98	3.98	0.122	1.08	87	0	9,912	6	1.51
	180	27.3	4.00	4.00	0.126	1.08	89	0	9,428	4	1.55
203° F	30	27.3	3.98	3.98	0.126	1.08	87	0	10,110	4	1.54
	60	27.3	3.98	3.98	0.125	1.08	86	0	9,693	4	1.54
	90	27.0	4.00	4.00	0.124	1.08	86	0	9,953	4	1.53
	120	25.9	3.98	3.98	0.125	1.08	87	0	10,125	4	1.47
	150	27.2	3.98	3.98	0.126	1.08	88	0	10,190	5	1.54
	180	26.8	4.00	4.00	0.123	1.08	87	0	10,125	5	1.52
225° F	30	27.3	3.98	3.98	0.125	1.08	87	0	9,782	4	1.55
	60	27.7	3.98	3.98	0.126	1.08	88	0	9,174	3	1.57
	90	27.4	3.99	3.99	0.126	1.08	85	0	9,976	4	1.55
	120	26.9	3.98	3.98	0.125	1.08	88	0	10,500	4	1.53
	150	27.1	4.00	4.00	0.125	1.08	87	0	10,743	9	1.53
	180	27.2	4.00	4.00	0.125	1.08	89	0	10,560	5	1.53
250° F	30	27.7	3.98	3.98	0.126	1.08	88	0	10,173	4	1.56
	60	27.2	3.97	3.97	0.124	1.08	88	0	10,142	4	1.54
	90	27.0	4.00	4.00	0.124	1.08	86	0	9,569	3	1.53
	120	27.5	3.98	3.98	0.127	1.08	86	0	9,486	3	1.56
	150	27.6	4.00	4.00	0.127	1.08	87	0	10,439	4	1.56
290° F	30	26.8	3.98	3.98	0.123	1.08	86	0	10,981	4	1.52
	60	27.3	3.97	3.97	0.125	1.08	88	0	10,692	4	1.55
	90	26.9	3.99	3.99	0.123	1.08	87	0	9,984	3	1.52
	120	27.1	3.98	3.98	0.125	1.08	87	0	10,097	1	1.53
	150	27.1	4.00	4.00	0.126	1.08	86	0	11,188	4	1.53

Table V-9

RUBBER R-5

Exposure	Time (days)	Weight (grams)	Length (inches)	Width (inches)	Thickness (inches)	Specific gravity	Shore	Tear	Tensile strength (lbs/in.^{2})	Elongation (percent)	Volume (cu in.)
Initial	0	53.0	5.98	5.84	0.066	1.42	67	250	2,338	328	2.28
100° F	30	50.8	5.74	5.76	0.068	1.43	68	240	2,444	350	2.17
	60	56.1	6.02	5.98	0.068	1.42	68	246	2,233	310	2.42
	90	51.9	5.93	5.73	0.066	1.44	71	204	2,534	328	2.21
	120	50.4	5.85	5.80	0.065	1.45	71	220	2,396	320	2.11
	150	54.9	6.02	5.97	0.067	1.45	71	225	2,328	330	2.31
	180	53.1	5.98	5.86	0.067	1.45	71	179	2,024	280	2.24
203° F	30	56.9	5.91	6.08	0.069	1.42	67	222	2,305	300	2.44
	60	54.2	6.03	5.83	0.066	1.43	70	222	2,502	320	2.32
	90	52.4	5.88	5.82	0.066	1.44	72	178	2,422	285	2.22
	120	49.2	5.72	5.65	0.066	1.45	72	184	2,487	310	2.08
	150	51.8	5.77	5.85	0.066	1.46	72	144	1,940	248	2.17
	180	53.7	5.78	6.03	0.066	1.46	73	182	2,569	295	2.24
225° F	30	55.9	6.00	5.98	0.069	1.42	67	253	2,397	125	2.49
	60	53.6	5.73	5.95	0.068	1.43	68	190	2,188	290	2.28
	90	52.8	5.91	5.88	0.066	1.45	73	197	2,109	260	2.22
	120	52.1	5.70	6.14	0.065	1.45	73	147	2,298	290	2.19
	150	52.1	5.91	5.82	0.066	1.46	73	169	1,408	175	2.18
	180	54.6	5.93	6.03	0.066	1.44	72	181	2,474	278	2.31
250° F	30	53.1	5.90	5.89	0.067	1.42	69	213	2,311	290	2.28
	60	56.0	5.97	5.95	0.068	1.43	71	178	2,441	260	2.39
	90	51.8	5.88	5.82	0.065	1.45	73	173	2,427	250	2.18
	120	51.2	5.87	5.69	0.066	1.46	75	167	2,121	245	2.14
	150	53.0	5.98	5.76	0.067	1.47	74	156	1,535	165	2.20
290° F	30	63.9	6.03	6.41	0.074	1.34	68	102	1,365	168	2.92
	60	55.3	6.01	5.93	0.069	1.40	74	91	1,835	185	2.41
	90	53.8	6.00	6.08	0.066	1.42	75	137	1,500	145	2.31
	120	55.0	5.97	6.07	0.067	1.41	74	91	1,783	158	2.38
	150	62.7	6.31	6.08	0.069	1.35	74	99	1,252	125	2.83

Table V-10

RUBBER R-6

Exposure	Time (days)	Weight (grams)	Length (inches)	Width (inches)	Thickness (inches)	Specific gravity	Shore	Tear	Tensile strength (lbs/in.^{2})	Elongation (percent)	Volume (cu in.)
Initial	0	55.2	5.90	5.91	0.086	1.13	55	165	2,081	319	2.97
100° F	30	55.4	5.92	5.91	0.084	1.14	56	131	2,243	330	2.97
	60	57.9	5.94	5.94	0.087	1.14	56	153	2,344	330	3.09
	90	55.3	5.92	5.93	0.085	1.14	57	118	2,305	340	2.96
	120	60.6	5.94	5.95	0.089	1.15	55	135	2,388	350	3.23
	150	55.7	5.92	5.92	0.084	1.14	57	143	2,196	346	2.99
	180	57.3	5.97	5.92	0.085	1.14	57	128	2,278	349	3.07
203° F	30	59.0	5.98	6.00	0.084	1.14	57	178	2,247	340	3.16
	60	60.6	6.02	6.00	0.088	1.13	55	136	1,986	290	3.26
	90	56.9	6.02	6.02	0.082	1.14	58	104	2,233	310	3.05
	120	59.2	6.02	6.00	0.086	1.14	59	145	2,144	300	3.18
	150	58.9	6.02	6.01	0.087	1.14	58	129	2,207	280	3.16
	180	61.8	6.03	5.98	0.092	1.14	58	166	2,181	290	3.31
225° F	30	56.7	6.02	6.01	0.082	1.14	57	121	2,163	310	3.04
	60	59.1	6.04	6.05	0.081	1.14	58	90	1,733	270	3.17
	90	58.4	6.03	6.05	0.085	1.14	59	111	2,017	275	3.13
	120	61.3	6.03	6.01	0.089	1.14	59	112	2,061	280	3.29
	150	69.5	6.06	6.03	0.103	1.14	58	112	2,044	269	3.72
	180	58.1	6.09	6.04	0.083	1.14	61	94	2,069	278	3.11
250° F	30	58.2	6.00	5.99	0.087	1.14	57	160	2,230	300	3.12
	60	57.0	6.03	6.04	0.081	1.14	59	130	2,107	270	3.06
	90	59.0	6.04	6.04	0.086	1.14	58	105	1,991	265	3.17
	120	55.6	6.05	6.00	0.081	1.14	60	89	1,998	250	2.97
	150	59.1	6.06	6.01	0.087	1.14	60	80	2,024	270	3.16
290° F	30	74.8	6.25	6.20	0.106	1.12	57	71	1,585	214	4.06
	60	66.2	6.20	6.20	0.092	1.12	58	71	1,335	200	3.60
	90	66.0	6.18	6.17	0.096	1.12	59	92	1,319	180	3.59
	120	65.1	6.23	6.20	0.092	1.12	59	86	1,459	201	3.54
	150	62.2	6.23	6.22	0.085	1.12	62	72	1,515	205	3.39

Table V-11

RUBBER R-7

Exposure	Time (days)	Weight (grams)	Length (inches)	Width (inches)	Thickness (inches)	Specific gravity	Shore	Tear	Tensile strength (lbs/in.^{2})	Elongation (percent)	Volume (cu in.)
Initial	0	56.2	5.90	5.90	0.085	1.15	67	239	2,021	346	2.99
100° F	30	58.5	5.90	5.88	0.084	1.15	67	228	2,130	390	3.09
	60	57.6	5.90	5.90	0.084	1.15	68	257	2,175	400	3.06
	90	59.1	5.90	5.90	0.091	1.15	68	258	2,119	360	3.13
	120	58.2	5.93	5.90	0.085	1.14	66	212	2,221	410	3.11
	150	58.1	5.90	5.90	0.087	1.16	68	270	2,104	400	3.07
	180	59.0	5.90	5.88	0.090	1.15	68	194	2,103	418	3.12
203° F	30	56.2	5.90	5.92	0.086	1.15	67	236	2,088	325	2.97
	60	58.7	5.93	5.93	0.087	1.15	67	217	2,056	375	3.11
	90	65.4	5.95	5.93	0.100	1.16	63	190	2,135	370	3.45
	120	60.9	5.93	5.89	0.092	1.16	69	237	2,124	345	3.22
	150	55.5	5.95	5.90	0.084	1.16	68	219	2,192	383	2.92
	180	58.7	5.93	5.91	0.090	1.16	67	176	2,107	400	3.09
225° F	30	55.5	5.92	5.93	0.084	1.16	68	264	2,090	400	2.93
	60	56.2	5.93	5.95	0.083	1.16	68	232	2,192	365	2.96
	90	55.0	5.96	5.96	0.082	1.16	70	220	2,240	360	2.90
	120	55.3	5.92	5.94	0.084	1.16	70	153	2,226	330	2.91
	150	56.6	5.97	5.96	0.084	1.16	69	217	2,108	360	2.98
	180	60.6	5.93	5.92	0.085	1.16	69	214	2,162	333	3.19
250° F	30	58.1	5.94	5.94	0.089	1.16	68	249	2,158	335	2.06
	60	55.8	5.94	5.93	0.082	1.16	69	228	2,290	335	2.94
	90	56.0	5.93	5.96	0.085	1.16	70	169	2,247	320	2.96
	120	58.2	5.90	5.90	0.086	1.16	70	175	2,278	310	3.06
	150	57.5	5.94	5.94	0.089	1.16	70	204	2,197	348	3.02
290° F	30	60.1	6.04	6.01	0.086	1.15	68	173	2,248	315	3.19
	60	60.0	5.98	5.98	0.086	1.15	68	194	2,331	300	3.18
	90	58.4	5.98	5.96	0.086	1.15	69	157	2,338	290	3.09
	120	60.6	5.98	5.94	0.089	1.15	69	184	2,338	275	3.20
	150	60.8	5.97	5.97	0.089	1.15	69	196	2,229	308	3.22

Table V-12

RUBBER R-9

Exposure	Time (days)	Weight (grams)	Length (inches)	Width (inches)	Thickness (inches)	Specific gravity	Shore	Tear	Tensile strength (lbs/in.^{2})	Elongation (percent)	Volume (cu in.)
Initial	0	57.9	5.89	5.91	0.092	1.12	61	207	2,016	345	3.16
100° F	30	57.8	5.88	5.91	0.089	1.12	61	188	2,050	360	3.15
	60	56.4	5.90	5.92	0.086	1.12	61	205	2,051	360	3.08
	90	58.1	5.90	5.92	0.092	1.13	63	192	1,983	330	3.15
	120	56.0	5.90	5.92	0.084	1.12	62	196	2,010	340	3.04
	150	58.2	5.90	5.90	0.090	1.13	61	179	1,863	323	3.15
	180	58.2	5.87	5.94	0.087	1.13	61	158	1,778	314	3.14
203° F	30	59.6	5.99	5.99	0.090	1.13	58	236	1,960	325	3.23
	60	63.3	5.97	6.00	0.095	1.13	60	161	1,796	300	3.43
	90	65.0	6.02	6.01	0.095	1.12	59	180	1,884	300	3.53
	120	61.3	6.02	6.02	0.092	1.13	63	76	1,915	280	3.32
	150	61.8	6.03	6.03	0.093	1.13	62	166	1,829	280	3.35
	180	60.2	6.02	6.03	0.088	1.13	62	159	1,870	290	3.26
225° F	30	62.7	5.99	5.98	0.097	1.13	59	209	1,873	310	3.40
	60	61.1	6.03	6.06	0.090	1.13	62	158	1,846	290	3.32
	90	61.2	6.05	6.06	0.086	1.13	62	183	1,915	295	3.31
	120	67.0	6.06	6.04	0.098	1.13	61	135	1,892	285	3.63
	150	66.8	6.09	6.09	0.096	1.13	62	136	1,665	278	3.62
	180	64.3	6.08	6.08	0.092	1.13	63	151	1,568	245	3.49
250° F	30	61.6	6.00	6.00	0.091	1.13	61	243	1,830	286	3.34
	60	62.1	6.04	6.05	0.090	1.13	60	155	1,856	270	3.37
	90	62.2	6.05	6.07	0.090	1.13	63	138	1,853	275	3.37
	150	61.2	6.04	6.06	0.087	1.13	63	157	1,896	275	3.31
	180	57.8	6.06	6.06	0.084	1.13	65	156	1,744	270	3.12
290° F	30	78.8	6.59	6.50	0.107	1.10	58	113	1,018	180	4.38
	60	68.7	6.38	6.33	0.094	1.11	62	97	1,274	190	3.79
	90	70.1	6.30	6.23	0.096	1.11	62	72	1,629	210	3.85
	120	68.0	6.26	6.25	0.092	1.11	62	122	1,379	193	3.73
	150	71.7	6.38	6.29	0.096	1.10	60	84	1,262	195	3.97

Table V-13

RUBBER R-11

Exposure	Time (days)	Weight (grams)	Length (inches)	Width (inches)	Thickness (inches)	Specific gravity	Shore	Tear	Tensile strength (lbs/in.^{2})	Elongation (percent)	Volume (cu in.)
Initial	0	50.1	5.87	5.86	0.077	1.17	70	228	1,691	430	2.62
100 ° F	30	52.1	5.85	5.87	0.079	1.18	72	215	1,824	420	2.69
	60	49.9	5.85	5.86	0.077	1.18	73	238	1,865	380	2.58
	90	52.5	5.85	5.85	0.079	1.18	72	255	1,754	350	6.71
	120	52.0	5.85	5.85	0.083	1.18	71	227	1,716	340	2.68
	150	50.8	5.82	5.85	0.078	1.19	73	257	1,700	337	2.61
	180	51.9	5.93	5.85	0.082	1.18	72	204	1,751	380	2.67
203° F	30	51.2	5.88	5.88	0.077	1.19	72	211	1,893	340	2.63
	60	51.7	5.84	5.85	0.078	1.19	72	225	1,851	340	2.66
	90	50.3	5.88	5.85	0.077	1.19	74	196	1,812	310	2.59
	120	52.1	5.83	5.85	0.078	1.19	75	226	1,849	300	2.61
	150	53.2	5.85	5.85	0.081	1.20	73	244	1,774	283	2.71
	180	52.6	5.87	5.86	0.081	1.19	73	212	1,701	310	2.70
225° F	30	50.3	5.85	5.88	0.074	1.19	73	314	1,846	355	2.59
	60	51.3	5.88	5.90	0.077	1.19	72	199	1,859	290	2.63
	90	51.0	5.87	5.89	0.080	1.19	73	209	1,894	310	2.63
	120	51.4	5.84	5.86	0.079	1.19	75	186	1,761	250	2.64
	150	53.0	5.88	5.87	0.080	1.20	75	188	1,711	252	2.70
	180	53.8	5.88	5.90	0.079	1.20	73	228	1,856	298	2.75
250° F	30	51.2	5.85	5.85	0.077	1.19	72	260	1,830	325	2.62
	60	53.5	5.86	5.90	0.081	1.19	74	211	1,759	250	2.74
	90	51.9	5.85	5.90	0.079	1.19	75	191	1,787	240	2.66
	120	51.2	5.86	5.85	0.077	1.20	74	185	1,809	275	2.60
	150	53.8	5.91	5.89	0.079	1.20	74	180	1,847	283	2.74
290° F	30	58.9	5.96	5.96	0.085	1.18	70	205	1,752	270	3.05
	60	53.6	5.90	5.92	0.079	1.19	72	199	1,827	250	2.76
	90	50.5	5.88	5.86	0.077	1.19	74	171	1,562	220	2.60
	120	51.6	5.81	5.90	0.080	1.19	72	205	1,634	232	2.65
	150	54.1	5.89	5.86	0.085	1.18	73	151	1,758	263	2.79

Table V-14

RUBBER R-17

Exposure	Time (days)	Weight (grams)	Length (inches)	Width (inches)	Thickness (inches)	Specific gravity	Shore	Tear	Tensile strength (lbs/in.^{2})	Elongation (percent)	Volume (cu in.)
Initial	0	51.2	5.88	5.90	0.080	1.14	72	234	2,167	302	2.75
100° F	30	54.2	5.87	5.87	0.087	1.15	71	227	1,943	270	2.89
	60	54.2	5.87	5.87	0.080	1.15	70	193	2,146	310	2.88
	90	53.9	5.88	5.88	0.084	1.15	72	226	2,156	340	2.87
	120	50.8	5.88	5.88	0.077	1.15	72	86	2,147	300	2.70
	150	49.5	5.89	5.87	0.077	1.15	72	256	2,026	280	2.64
	180	51.2	5.87	5.85	0.077	1.15	73	207	2,139	324	2.71
203° F	30	55.1	5.88	5.90	0.083	1.15	70	273	2,046	295	2.74
	60	56.9	5.87	5.88	0.087	1.15	71	223	1,928	265	3.01
	90	50.9	5.88	5.89	0.079	1.15	74	203	2,184	290	2.20
	120	50.8	5.84	5.85	0.081	1.15	73	202	2,203	285	2.69
	150	51.7	5.85	5.88	0.080	1.16	72	241	2,128	320	2.23
	180	53.0	5.88	5.88	0.081	1.15	74	197	2,093	280	2.80
225° F	30	50.2	5.92	5.90	0.076	1.16	71	199	1,942	280	2.65
	60	54.8	5.92	5.93	0.081	1.15	72	197	2,091	295	2.84
	90	52.0	5.87	5.90	0.078	1.15	75	230	2,177	270	2.75
	120	49.8	5.85	5.86	0.079	1.15	75	190	2,206	280	2.64
	150	54.1	5.88	5.90	0.081	1.16	73	160	1,960	230	2.86
	180	51.3	5.88	5.87	0.078	1.16	74	223	2,037	248	2.71
250° F	30	55.9	5.90	5.86	0.086	1.16	73	287	2,095	275	2.94
	60	50.8	5.85	5.86	0.076	1.15	73	178	2,190	250	2.69
	90	50.4	5.88	5.86	0.077	1.16	75	160	2,130	240	2.66
	120	50.3	5.84	5.85	0.079	1.16	75	192	2,180	260	2.65
	150	49.3	5.85	5.87	0.077	1.16	76	146	2,199	270	2.59
290° F	30	51.8	5.82	5.84	0.081	1.16	75	198	2,161	210	2.73
	60	52.0	5.80	5.81	0.081	1.16	76	175	2,317	225	2.75
	90	52.1	5.72	5.78	0.083	1.16	78	185	2,372	215	2.74
	120	48.1	5.74	5.74	0.078	1.16	78	226	2,140	200	2.53
	150	51.1	5.73	5.77	0.079	1.15	75	162	2,123	223	2.70

Table V-15

RUBBER R-18

Exposure	Time (days)	Weight (grams)	Length (inches)	Width (inches)	Thickness (inches)	Specific gravity	Shore	Tear	Tensile strength (lbs/in.^{2})	Elongation (percent)	Volume (cu in.)
Initial	0	40.3	5.92	5.95	0.055	1.33	68	234	1,268	458	1.86
100° F	30	41.0	5.97	5.98	0.055	1.33	66	221	1,168	385	1.88
	60	40.6	5.96	5.88	0.056	1.32	67	224	1,446	445	1.88
	90	40.8	6.05	5.84	0.055	1.32	72	214	1,372	480	1.89
	120	37.8	5.80	5.78	0.053	1.34	71	202	2,853	480	1.73
	150	40.5	6.04	5.88	0.055	1.34	69	222	1,337	455	1.85
	180	40.9	6.03	5.88	0.055	1.34	74	225	1,293	420	1.86
203° F	30	41.8	6.16	5.88	0.056	1.32	66	232	1,518	460	1.94
	60	42.6	5.97	6.12	0.055	1.32	68	212	1,160	415	1.97
	90	39.0	5.97	5.81	0.054	1.34	73	212	1,238	415	1.78
	120	38.9	5.85	5.97	0.053	1.34	72	188	1,160	375	1.77
	150	38.3	5.81	5.95	0.054	1.34	73	235	1,235	405	1.74
	180	39.2	5.98	5.76	0.053	1.34	73	226	1,446	480	1.79
225° F	30	42.3	6.10	5.90	0.056	1.32	64	274	1,569	490	1.96
	60	43.9	5.98	6.14	0.057	1.32	66	228	1,447	480	2.03
	90	40.3	6.10	5.82	0.054	1.34	74	230	1,267	380	1.83
	120	40.5	6.03	5.79	0.055	1.34	75	209	1,358	390	1.85
	150	40.5	5.93	6.00	0.055	1.34	75	164	1,101	307	1.85
	180	45.0	6.33	5.92	0.057	1.30	71	158	1,417	408	2.12
250° F	30	44.0	6.08	5.93	0.058	1.31	65	247	1,578	475	2.05
	60	44.8	6.15	6.04	0.058	1.30	71	236	1,556	360	2.11
	90	40.8	5.90	6.03	0.055	1.32	73	189	1,123	315	1.88
	120	42.3	5.90	6.08	0.056	1.33	75	178	1,147	310	1.95
	150	42.5	6.03	6.20	0.055	1.33	74	154	1,129	308	1.95
290° F	30	141.2	9.71	9.72	0.080	1.07	47	42	172	98	8.03

Table V-16

RUBBER R-19

Exposure	Time (days)	Weight (grams)	Length (inches)	Width (inches)	Thickness (inches)	Specific gravity	Shore	Tear	Tensile strength (lbs/in.^{2})	Elongation (percent)	Volume (cu in.)
Initial	0	82.3	5.90	5.82	0.136	1.08	33	133	981	48	4.63
100° F	30	87.1	5.81	6.12	0.138	1.09	27	118	627	80	4.88
	60	88.0	6.36	5.90	0.131	1.09	36	166	745	105	4.92
	90	85.0	5.77	6.05	0.141	1.09	38	201	731	45	4.76
	120	89.1	5.83	6.13	0.145	1.09	36	156	599	57	4.98
	150	83.7	5.55	6.10	0.139	1.09	32	107	649	75	4.68
	180	86.4	5.98	5.78	0.139	1.11	43	160	616	56	4.76
203° F	30	96.3	5.76	6.64	0.142	1.10	47	251	1,135	33	5.32
	60	88.0	6.08	5.75	0.137	1.11	56	382	1,618	15	4.86
	90	86.8	6.22	5.76	0.137	1.10	56	325	1,796	4	4.70
	120	87.8	6.07	5.78	0.141	1.12	60	334	1,692	4	4.70
	150	88.5	5.74	6.27	0.139	1.11	56	214	1,329	6	4.86
	180	88.7	5.89	6.09	0.140	1.11	52	280	1,292	7	4.88
225° F	30	90.1	6.35	5.92	0.136	1.12	56	338	1,424	23	4.90
	60	86.8	5.95	5.95	0.136	1.13	58	354	1,579	14	4.69
	90	82.0	5.82	5.65	0.150	1.13	59	0	0	0	4.41
	120	89.0	5.70	6.09	0.145	1.14	63	269	1,758	4	4.77
	150	87.5	5.79	6.07	0.140	1.13	60	284	1,523	3	4.72
	180	88.2	5.94	5.93	0.138	1.13	58	258	1,475	4	4.76
250° F	30	86.4	5.43	6.06	0.142	1.11	55	302	1,287	2	4.76
	60	87.8	5.90	6.08	0.140	1.12	51	241	1,371	2	4.79
	90	86.1	5.90	5.94	0.136	1.12	60	292	1,735	4	4.71
	120	90.4	5.79	6.02	0.147	1.13	62	0	1,306	2	4.89
	150	85.1	5.80	5.83	0.140	1.13	63	328	1,872	4	4.59
290° F	30	93.3	5.85	6.13	0.138	1.13	54	0	1,449	1	5.03
	60	84.0	5.65	5.70	0.148	1.12	45	0	1,112	3	4.58
	90	93.5	5.98	6.00	0.139	1.12	59	0	1,318	2	5.11
	120	97.5	5.78	6.48	0.154	1.10	51	0	1,221	4	5.43
	150	92.1	5.92	6.07	0.152	1.11	50	0	989	5	5.06

Table V-17

RUBBER R-20

Exposure	Time (days)	Weight (grams)	Length (inches)	Width (inches)	Thickness (inches)	Specific gravity	Shore	Tear	Tensile strength (lbs/in.^{2})	Elongation (percent)	Volume (cu in.)
Initial	0	39.8	5.96	6.06	0.062	1.13	59	279	1,393	560	2.15
100° F	30	40.0	5.98	6.04	0.060	1.14	59	220	1,574	585	2.14
	60	40.0	5.98	6.00	0.062	1.14	58	258	1,627	450	2.14
	90	38.0	5.91	6.00	0.060	1.12	58	280	1,583	590	2.08
	120	39.5	6.04	6.05	0.060	1.13	58	279	1,478	590	2.14
	150	37.9	5.78	5.92	0.063	1.13	58	292	1,492	528	2.04
	180	40.9	6.03	5.98	0.062	1.16	64	238	1,549	412	2.16
203° F	30	40.2	5.95	5.90	0.063	1.15	62	259	1,524	440	2.13
	60	39.0	5.84	5.86	0.062	1.16	64	257	1,659	375	2.06
	90	39.8	5.88	6.09	0.062	1.15	66	256	1,650	455	2.12
	120	37.6	6.07	5.85	0.061	1.14	65	244	1,746	490	2.02
	150	39.8	5.80	6.10	0.062	1.17	66	205	1,774	390	2.08
	180	37.2	5.80	5.93	0.060	1.16	63	262	1,583	530	1.97
225° F	30	40.2	5.98	5.96	0.063	1.14	58	285	1,467	470	2.15
	60	38.1	5.90	5.88	0.063	1.14	56	273	1,482	530	2.04
	90	37.5	5.85	5.85	0.061	1.14	60	287	1,571	520	2.01
	120	39.5	5.85	6.05	0.061	1.15	66	168	1,706	420	2.09
	150	40.0	5.87	5.91	0.064	1.17	66	187	1,602	400	2.09
	180	39.1	5.87	5.88	0.062	1.16	63	225	1,489	388	2.05
250° F	30	40.4	5.95	5.97	0.062	1.15	61	276	1,569	500	2.14
	60	38.9	5.85	5.94	0.062	1.15	64	279	1,758	435	2.06
	90	39.0	5.89	5.90	0.062	1.15	66	180	1,491	345	2.07
	120	40.8	5.95	5.98	0.062	1.17	64	248	1,803	410	2.13
	150	39.3	5.99	5.99	0.060	1.17	68	197	1,571	359	2.04
290° F	30	39.6	5.93	5.90	0.063	1.18	70	115	1,403	318	2.06
	60	35.1	5.75	5.70	0.058	1.20	80	193	1,116	260	1.78
	90	33.6	5.55	5.65	0.056	1.20	80	187	1,722	230	1.71
	120	31.8	5.41	5.72	0.056	1.19	76	231	1,157	212	1.64
	150	34.7	5.63	5.76	0.059	1.18	75	139	1,506	290	1.79

Table V-18

RUBBER R-21

Exposure	Time (days)	Weight (grams)	Length (inches)	Width (inches)	Thickness (inches)	Specific gravity	Shore	Tear	Tensile strength (lbs/in.^{2})	Elongation (percent)	Volume (cu in.)
Initial	0	48.3	5.85	5.86	0.066	1.26	61	230	1,097	703	2.34
100° F	30	45.9	5.89	5.83	0.057	1.26	61	188	1,116	750	2.22
	60	50.0	5.90	5.85	0.067	1.26	63	208	1,089	760	2.42
	90	48.8	5.90	5.88	0.065	1.26	63	196	1,116	760	2.36
	120	53.5	5.90	5.85	0.071	1.26	64	194	1,135	730	2.58
	150	46.0	5.90	5.85	0.054	1.27	62	192	1,083	730	2.22
	180	50.5	5.89	5.89	0.068	1.27	65	201	1,085	745	2.43
203° F	30	51.3	5.91	5.86	0.068	1.27	69	205	1,085	690	2.47
	60	52.4	5.94	5.92	0.071	1.26	71	230	1,115	640	2.54
	90	51.6	5.93	5.93	0.069	1.25	73	252	1,256	600	2.51
	120	52.5	5.95	5.90	0.070	1.26	71	224	1,134	580	2.54
	150	50.1	5.97	5.90	0.066	1.26	71	191	1,121	612	2.43
	180	50.3	5.94	5.90	0.066	1.26	72	215	1,111	620	2.44
225° F	30	49.0	5.95	5.88	0.064	1.26	69	235	1,160	650	2.37
	60	52.2	5.94	5.92	0.069	1.26	70	218	1,177	585	2.53
	90	52.0	5.97	5.92	0.069	1.26	72	213	1,215	555	2.52
	120	52.5	5.93	5.90	0.069	1.26	69	188	1,177	530	2.55
	150	53.8	5.98	5.90	0.068	1.27	71	175	1,197	515	2.60
	180	49.0	5.95	5.87	0.063	1.27	74	207	1,250	539	2.36
250° F	30	51.5	5.91	5.88	0.069	1.27	69	200	1,131	560	2.47
	60	52.0	5.95	5.90	0.070	1.27	71	224	1,214	560	2.51
	90	47.0	5.92	5.93	0.063	1.27	72	224	1,259	540	2.26
	120	50.0	5.95	5.91	0.066	1.27	73	211	1,120	490	2.40
	150	52.5	5.98	5.96	0.069	1.27	73	197	1,159	540	2.53
290° F	30	55.6	6.13	6.13	0.066	1.22	67	180	1,003	510	2.78
	60	47.6	5.95	5.90	0.064	1.25	71	201	1,190	550	2.33
	90	48.9	5.95	5.86	0.065	1.26	75	203	1,080	515	2.38
	120	50.1	6.00	5.92	0.066	1.25	72	179	1,068	500	2.45
	150	53.9	6.03	5.93	0.070	1.24	75	189	1,072	519	2.58

Table V-19

RUBBER R-22

Exposure	Time (days)	Weight (grams)	Length (inches)	Width (inches)	Thickness (inches)	Specific gravity	Shore	Tear	Tensile strength (lbs/in.^{2})	Elongation (percent)	Volume (cu in.)
Initial	0	47.2	5.85	5.82	0.070	1.16	68	292	1,779	670	2.48
100° F	30	45.1	5.83	5.82	0.064	1.16	67	243	1,780	720	2.37
	60	45.5	5.85	5.85	0.070	1.16	69	230	1,839	700	2.39
	90	48.8	5.86	5.81	0.073	1.16	68	237	1,789	670	2.57
	120	45.3	5.84	5.80	0.070	1.16	70	261	1,886	685	2.39
	150	49.6	5.85	5.82	0.077	1.16	69	266	1,605	603	2.60
	180	51.0	5.86	5.83	0.078	1.17	69	231	1,892	745	2.67
203° F	30	51.0	5.88	5.85	0.075	1.16	66	250	1,728	640	2.69
	60	48.2	5.88	5.85	0.072	1.16	68	228	1,792	620	2.53
	90	50.0	5.85	5.86	0.072	1.16	70	237	1,828	565	2.62
	120	49.1	5.85	5.85	0.073	1.17	67	240	1,853	590	2.57
	150	47.5	5.86	5.80	0.067	1.18	70	219	1,631	590	2.47
	180	49.8	5.85	5.83	0.075	1.17	65	220	1,862	650	2.61
225° F	30	51.2	5.85	5.87	0.076	1.16	68	286	2,352	650	2.69
	60	48.3	5.90	5.86	0.071	1.17	68	235	1,882	640	2.53
	90	53.0	5.86	5.86	0.079	1.17	72	237	1,926	585	2.76
	120	51.9	5.85	5.85	0.080	1.17	69	227	1,738	530	2.72
	150	48.5	5.86	5.86	0.071	1.17	71	180	1,711	565	2.53
	180	51.9	5.83	5.87	0.077	1.18	71	241	1,746	603	2.69
250° F	30	45.4	5.86	5.86	0.070	1.16	68	316	1,835	630	2.39
	60	50.2	5.88	5.88	0.074	1.16	69	247	1,872	635	2.63
	90	46.2	5.87	5.87	0.066	1.17	70	230	1,750	550	2.42
	120	49.0	5.85	5.85	0.073	1.17	71	236	1,874	600	2.56
	150	49.1	5.88	5.88	0.073	1.17	72	235	1,692	550	2.55
290° F	30	69.2	6.54	6.56	0.079	1.11	55	295	933	550	3.81
	60	56.7	5.92	5.93	0.084	1.15	64	230	1,268	700	3.02
	90	51.2	5.94	5.93	0.076	1.16	66	210	1,365	620	3.70
	120	53.1	6.07	6.03	0.074	1.14	62	209	1,243	658	2.84
	150	62.2	6.20	6.07	0.080	1.13	64	160	1,081	678	3.37

Table V-20

RUBBER R-23

Exposure	Time (days)	Weight (grams)	Length (inches)	Width (inches)	Thickness (inches)	Specific gravity	Shore	Tear	Tensile strength (lbs/in.^{2})	Elongation (percent)	Volume (cu in.)
Initial	0	58.1	5.88	5.91	0.075	1.33	56	161	1,562	740	2.66
100° F	30	59.6	5.89	5.93	0.077	1.32	54	146	1,544	795	2.68
	60	56.5	5.90	5.90	0.072	1.33	58	139	1,515	718	2.59
	90	58.3	5.89	5.88	0.077	1.32	57	105	1,461	725	2.69
	120	56.8	5.89	5.89	0.073	1.32	60	143	1,464	690	2.62
	150	58.2	5.88	5.92	0.076	1.32	58	124	1,442	740	2.69
	180	58.8	5.88	5.93	0.078	1.33	58	125	1,551	740	2.70
203° F	30	61.6	5.90	5.91	0.083	1.32	55	165	1,486	720	2.84
	60	62.0	5.90	5.92	0.083	1.32	55	104	1,474	715	2.87
	90	61.0	5.92	5.92	0.080	1.32	59	123	1,132	635	2.82
	120	61.1	5.90	5.90	0.081	1.32	58	114	1,419	700	2.82
	150	58.5	5.90	5.95	0.077	1.33	57	124	1,332	720	2.69
	180	59.0	5.89	5.92	0.077	1.32	56	136	1,466	735	2.72
225° F	30	58.9	5.91	5.95	0.076	1.33	56	151	1,484	710	2.71
	60	57.8	5.92	5.92	0.074	1.32	54	154	1,423	710	2.67
	90	60.2	5.92	5.90	0.077	1.33	55	172	1,305	725	2.72
	120	62.4	5.90	5.90	0.084	1.32	54	121	1,340	730	2.89
	150	62.1	5.92	5.90	0.084	1.33	53	157	931	680	2.85
	180	59.2	5.92	5.91	0.078	1.33	53	164	1,299	788	2.76
250° F	30	58.8	5.90	5.90	0.075	1.33	54	146	1,283	720	2.70
	60	62.9	5.90	5.90	0.081	1.33	53	163	1,317	750	2.86
	90	58.8	5.92	5.95	0.076	1.33	56	154	1,326	785	2.70
	120	57.4	5.88	5.91	0.074	1.34	53	188	1,355	785	2.62
	150	58.4	5.94	5.91	0.076	1.35	53	160	1,252	810	2.64
290° F	30	61.2	6.04	5.98	0.077	1.30	43	143	959	890	2.86
	60	59.2	5.95	5.93	0.074	1.32	47	156	907	780	2.73
	90	58.0	5.91	5.91	0.074	1.34	49	137	989	900	2.65
	120	60.1	5.97	5.98	0.078	1.32	44	157	990	862	2.78
	150	64.9	6.02	5.97	0.085	1.31	48	129	918	890	3.02

Table V-21

RUBBER R-24

Exposure	Time (days)	Weight (grams)	Length (inches)	Width (inches)	Thickness (inches)	Specific gravity	Shore	Tear	Tensile strength (lbs/in.^{2})	Elongation (percent)	Volume (cu in.)
Initial	0	51.8	5.87	5.89	0.082	1.14	61	333	1,626	652	2.78
100° F	30	51.8	5.86	5.87	0.078	1.15	58	332	1,677	750	2.76
	60	52.0	5.90	5.92	0.078	1.14	61	318	1,723	664	2.78
	90	50.0	5.89	5.89	0.076	1.14	62	317	1,717	680	2.67
	120	51.0	5.85	5.88	0.077	1.15	61	307	1,815	675	2.72
	150	52.2	5.88	5.90	0.082	1.15	62	310	1,828	682	2.77
	180	50.2	5.84	5.86	0.075	1.15	63	303	1,822	698	2.66
203° F	30	53.6	5.92	5.92	0.081	1.15	62	376	1,961	595	2.85
	60	54.8	5.96	5.95	0.083	1.14	63	272	1,906	510	2.93
	90	53.1	5.95	5.98	0.080	1.15	67	251	1,930	470	2.83
	120	51.8	5.96	5.95	0.076	1.14	67	242	1,873	430	2.77
	150	53.3	5.96	5.95	0.081	1.15	66	236	1,901	420	2.83
	180	53.3	5.97	5.98	0.079	1.15	65	255	1,993	450	2.84
225° F	30	53.1	5.93	5.91	0.080	1.15	62	301	1,815	490	2.83
	60	53.1	5.98	6.00	0.078	1.14	63	378	1,809	460	2.83
	90	52.7	6.00	5.98	0.077	1.15	66	194	1,647	360	2.81
	120	57.3	5.98	6.00	0.084	1.14	65	203	1,949	420	3.06
	150	58.0	6.02	5.92	0.086	1.15	66	169	1,733	390	3.08
	180	56.7	6.08	6.04	0.080	1.14	66	168	1,865	395	3.03
250° F	30	55.5	5.97	5.99	0.084	1.14	62	274	1,883	458	2.96
	60	57.3	5.95	5.96	0.087	1.14	64	246	1,874	410	3.08
	90	55.0	5.97	6.01	0.083	1.15	66	189	1,699	370	2.93
	120	53.2	5.98	5.96	0.078	1.15	65	207	1,702	400	2.82
	150	53.2	6.01	5.98	0.078	1.15	67	186	1,733	375	2.82
290° F	30	75.6	6.71	6.65	0.092	1.10	59	153	898	255	4.18
	60	60.2	6.25	6.23	0.084	1.13	64	144	1,277	305	3.25
	90	58.1	6.20	6.26	0.081	1.13	65	136	1,224	320	3.13
	120	59.5	6.27	6.30	0.080	1.13	62	166	1,215	300	3.22
	150	67.9	6.55	6.33	0.082	1.11	63	127	1,213	312	3.83

Table V-22

RUBBER R-27

Exposure	Time (days)	Weight (grams)	Length (inches)	Width (inches)	Thickness (inches)	Specific gravity	Shore.	Tear	Tensile strength (lbs/in.^{2})	Elongation (percent)	Volume (cu in.)
Initial	0	68.0	5.78	5.83	0.102	1.31	73	123	2,141	146	3.16
100° F	30	64.8	5.82	5.80	0.096	1.32	73	127	1,990	145	3.00
	60	67.2	5.82	5.80	0.099	1.32	74	107	1,782	130	3.11
	90	66.4	5.85	5.83	0.100	1.32	73	137	2,082	120	3.07
	120	67.9	5.84	5.81	0.103	1.32	73	101	1,929	153	3.44
203° F	30	61.0	5.81	5.82	0.085	1.32	76	130	2,192	120	2.81
	60	63.1	5.80	5.80	0.087	1.32	75	120	2,292	120	2.92
	90	62.3	5.80	5.80	0.090	1.32	76	118	1,833	130	2.87
	120	64.2	5.77	5.83	0.093	1.32	75	136	2,057	135	2.96
225° F	30	64.7	5.85	5.80	0.084	1.33	75	122	2,060	130	2.98
	60	65.0	5.80	5.80	0.097	1.33	77	119	2,184	130	2.99
	90	64.0	5.82	5.80	0.095	1.34	85	132	1,777	128	2.91
	120	65.5	5.85	5.80	0.093	1.36	82	153	1,347	95	2.95
250° F	30	60.6	5.80	5.84	0.087	1.33	76	124	1,816	100	2.78
	60	64.7	5.80	5.80	0.095	1.33	81	121	2,142	125	2.96
	90	68.0	5.77	5.77	0.103	1.36	87	201	1,629	80	3.05
	Too brittle to test after 120 days exposure.										
290° F	Too brittle to test after 30 days exposure.										

Table V-23

RUBBER R-28

Exposure	Time (days)	Weight (grams)	Length (inches)	Width (inches)	Thickness (inches)	Specific gravity	Shore	Tear	Tensile strength (lbs/in.^{2})	Elongation (percent)	Volume (cu in.)
Initial	0	63.8	5.70	5.71	0.098	1.32	69	144	2,008	228	2.96
100° F	30	62.5	5.74	5.79	0.095	1.32	71	173	1,943	170	2.90
	60	65.0	5.75	5.80	0.082	1.32	70	174	2,299	220	3.02
	90	64.2	5.75	5.75	0.084	1.32	67	187	1,979	252	2.97
	120	64.2	5.70	5.77	0.086	1.32	68	109	1,964	283	2.97
203° F	30	67.0	5.76	5.79	0.100	1.32	74	188	2,024	205	3.11
	60	64.8	5.73	5.80	0.094	1.32	72	177	2,225	230	2.99
	90	64.2	5.80	5.75	0.089	1.33	70	159	1,787	220	2.96
	120	64.8	5.80	5.79	0.085	1.32	69	178	1,912	245	2.99
225° F	30	67.0	5.72	5.82	0.104	1.30	70	168	2,204	210	3.16
	60	65.8	5.69	5.75	0.103	1.33	74	211	1,917	160	3.03
	90	63.0	5.74	5.78	0.096	1.35	85	152	1,820	200	2.86
	120	68.8	5.90	5.77	0.098	1.35	79	173	1,219	95	3.12
250° F	30	64.9	5.79	5.75	0.092	1.32	73	135	1,944	230	2.99
	60	67.0	5.74	5.74	0.097	1.34	79	163	1,802	150	3.05
	90	70.4	5.74	5.74	0.102	1.36	85	174	1,535	130	3.15
	Too brittle to test after 120 days exposure.										
290° F	Too brittle to test after 30 days exposure.										

Table V-24

RUBBER R-29

Exposure	Time (days)	Weight (grams)	Length (inches)	Width (inches)	Thickness (inches)	Specific gravity	Shore	Tear	Tensile strength (lbs/in.^{2})	Elongation (percent)	Volume (cu in.)
Initial	0	48.3	5.82	5.83	0.076	1.13	67	72	2,560	613	2.61
100° F	30	48.7	5.85	5.85	0.075	1.14	65	277	2,046	618	2.62
	60	48.3	5.87	5.88	0.077	1.14	66	250	1,980	580	2.59
203° F	30	49.8	5.85	5.85	0.079	1.14	67	236	2,040	603	2.68
	60	48.5	5.88	5.88	0.076	1.14	66	283	1,997	570	2.61
250° F	30	49.2	5.90	5.88	0.078	1.14	70	281	2,021	586	2.64
290° F	30	50.0	5.91	5.90	0.078	1.13	69	284	2,037	582	2.70
	60	52.2	5.97	5.93	0.076	1.13	68	189	1,987	545	2.83

Table V-25

RUBBER R-30

Exposure	Time (days)	Weight (grams)	Length (inches)	Width (inches)	Thickness (inches)	Specific gravity	Shore	Tear	Tensile strength (lbs/in.^{2})	Elongation (percent)	Volume (cu in.)
Initial	0	48.9	5.82	5.82	0.075	1.16	64	170	1,278	629	2.57
100° F	30	48.2	5.83	5.85	0.073	1.16	63	161	1,290	641	2.53
	60	48.7	5.86	5.90	0.076	1.17	63	131	1,295	638	2.56
203° F	30	49.0	5.86	5.83	0.077	1.16	63	137	1,437	575	2.57
	60	48.8	5.82	5.83	0.073	1.16	63	140	1,420	565	2.56
250° F	30	47.8	5.85	5.83	0.073	1.17	65	150	1,519	590	2.51
290° F	30	49.0	5.80	5.80	0.078	1.16	62	228	1,595	640	2.57
	60	49.9	5.87	5.82	0.078	1.17	65	209	1,527	657	2.66

SECTION VI

PART IV--CORROSION STUDIES ON REINFORCING STEEL

VI. 1. Introduction

In order to determine the effect of the hot brines on the reinforcement in reinforced concrete, samples of steel reinforcing bars and steel pretensioning rods were embedded in concrete bars and cylinders.

Three types of specimens were used to monitor the corrosion (1) steel reinforcing rods, (2) steel pretensioning rods, and (3) electrical resistance corrosion meter probes with mild steel test elements. All steel specimens were embedded in concrete and the concrete was cured and tested in the same manner as the other specimens, Section III.

A steel reinforcing bar was embedded in a standard 4- by 4- by 30-inch concrete specimen (two specimens total). Electrical lead wires were brought from the rebar through the concrete to the outside to provide the necessary electrical connections for determination of the corrosion rate on the bar. Pretensioned 4- by 4- by 30-inch bar specimens were fabricated. No leads were required in this case as access to the pretensioning rods was available at the pretensioning anchorplates. Two specimens each were made using steel reinforcing and steel pretensioning rods for each separate loop. A single 6- by 12-inch cylinder specimen for the 290° F loop was fabricated and four electrical resistance corrosion measuring probes were embedded parallel to the axis of the cylinder at distances from the outer surface of 3/4, 1-1/2, 2-1/2, and 3 inches.

VI. 2. Corrosion Measurement

Laboratory investigation generally consists of determining the extent of corrosion by specimen weight loss or pit depth measurements. Application of either measurement to evaluate the effect of brine exposure on concrete embedded steel as a function of time requires the use of numerous similar specimens to be removed at different times and destructively tested to evaluate the progress of corrosion. It can be seen that the difficulties associated with weight loss and pit depth measurements preclude their use in a small test system where the number of specimens which may be tested is severely limited. In order to reduce the number of specimens required to monitor the corrosion, a nondestructive test was employed which permitted the test to be resumed without appreciable alteration of the test specimen.

To provide a nondestructive method of evaluating the corrosion rates of concrete embedded steel the electrochemical approach has been used to compute weight loss as a function of the corrosion current flowing in the steel. Determination of the corrosion current was made on the basis of the polarization break method devised by Schwerdtfegar2/3/8/ from the National Bureau of Standards. A list of general references on polarization curves is given at the end of this chapter. The basis of Schwerdtfegar's method is the determination of the minimum current necessary to reverse the anodic and cathodic corrosion currents respectively. From the current values the total corrosion current for the steel can be calculated and the corresponding corrosion rate determined by Faraday's electrochemical equivalent number.

The currents required to reverse the anodic and cathodic corrosion currents can be determined by plotting a polarization or V-log I curve for the steel involved.

Another method of corrosion measurement has been the use of corrosion meter test probes. This method uses the decreases in conducting cross section of a tube element in a corrosive environment to measure the progress of corrosion. The resistance of the corroding element is compared with a standard resistance and the resulting ratio of resistances is a measure of the corrosion of the exposed element. These elements are fabricated into probes which can be placed in the environment to be tested. For these tests, four probes were cast parallel to the axis in a concrete 6- by 12-inch cylinder at various distances from the outer surface of the cylinder. The probes were placed so that the electrical connection for making the required measurement was exposed.

VI. 3. Laboratory Procedure and Measurements

Corrosion determinations were made on all specimens at 28, 90, and 180 days. The 270-day and 2-year determinations were made on the 100° F specimens and 270-day readings were made on the 290° F specimens. Polarization curves were run on the 4- by 4- by 30-inch rebar specimens and pretensioned specimens. The test apparatus used is shown in the following schematic diagram.

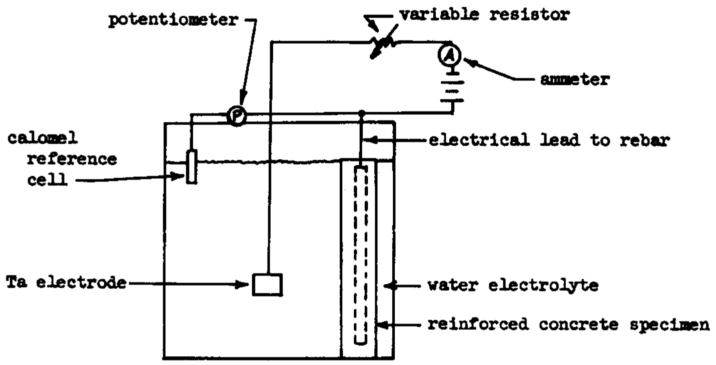

Figure VI-1; Polarization curve apparatus

As a current is applied the potential between the reference cell and the embedded rebar increases slowly, if at all, until the value of the impressed current is sufficient to override the anodic corrosion current, Ia. At this point the potential rises quite rapidly with increasing current. Thus the current to be measured is taken as the current at the break point of the potential-current curve. Similarly the cathodic current, Ic, is measured and the total cell current i_c is computed from the relationship

$$i_c = \frac{Ia \quad (Ic)}{Ia + Ic}$$

The corrosion rate W is then equal to the cell current times Faraday's number, F, or $W = Fi_c/t$ where t is time.

The corrosion meter probes were monitored during the tests using a commerical device for which the probes were made.

VI. 4. Test Results

VI. 4. 1. Conclusions

Based upon results obtained from the aforementioned corrosion tests the following conclusions seem justified:

1. No significant corrosion of the steel reinforcing bars or the steel pretensioning rods has occurred as a result of exposure to flowing synthetic sea water brine regardless of length of time or temperature.

2. Results of tests conducted on corrosion meter probes cast in 6- by 12-inch cylinders substantiate the results obtained by use of the polarization curves; i. e., no significant corrosion of embedded steel has occurred.

VI. 4. 2. Discussion

Typical corrosion data taken from the concrete-embedded steel reinforcing bars and pretensioning rods are listed in Table VI-1.

VI. 5. Test Programs

The present program consists of evaluating the different corrosion test specimens at the end of each programmed test period. This program has been expanded to include specimens from all test loops (100°, 203°, 225°, 250° and 290° F).

In addition to mild steel reinforcing materials, a stainless steel (302) and a nickel alloy will also be tested in the saline waters as reinforcing.

Table VI-1

CORROSION RATES OF EMBEDDED STEEL REINFORCING BARS AND PRETENSIONING RODS

Specimen	Accumulative test period (days)	Corrosion rate of steel at end of test period (gm/year for the bar)
CCI-BH3	28	0.2
	90	0.2
	180	0.3
	270	0.4
	1 year	0.2
	18 months	**NSC
CCI-BH4	28	0.1
	90	0.2
	180	0.2
	270	0.3
	1 year	0.3
	18 months	NSC
CCI-BH5	28	3.2
	90	2.8
	180	2.2
	270	0.6
	1 year	1.0
	18 months	NSC
CCI-BH6	28	1.1
	90	1.5
	180	2.4
	270	0.5
	1 year	2.2
	18 months	NSC
CCI-BL3	28	*L 0.1
	90	L 0.1
	180	L 0.1
	270	L 0.1
	1 year	--
	18 months	NSC
	2 years	NSC
CCI-BL4	28	0.1
	90	L 0.1
	180	L 0.1
	270	0.1
	1 year	--
	18 months	NSC
	2 years	NSC

*L = less than.
**NSC = no significant corrosion.

Table VI-1--Continued

CORROSION RATES OF EMBEDDED STEEL REINFORCING BARS AND PRETENSIONING RODS

Specimen	Accumulative test period (days)	Corrosion rate of steel at end of test period (gm/year for the bar)
CCI-BL5	28	15
	90	**NSC
	180	0.9
	270	0.6
	1 year	--
	18 months	NSC
	2 years	NSC
CCI-BL6	28	16
	90	NSC
	180	0.2
	270	0.4
	1 year	--
	18 months	NSC
	2 years	NSC

**NSC = no significant corrosion.

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SECTION VII

PART V--CONCRETE MICROSTRUCTURAL INVESTIGATIONS

VII. 1. Introduction

Concrete samples exposed for various times to synthetic sea water brines at elevated temperatures in laboratory test loops were examined petrographically and the progressive effect of this exposure on the concrete are described herein.

The samples consisted of 1- to 2-1/2-inch-thick slices from 4- by 4- by 30-inch concrete bars. Examination was made by stereoscopic microscope, polarizing microscope, X-ray diffraction, differential thermal analysis, and various other chemical and physical means as needed. The impact hammer (Type L) was used to give another index of durability (see Figure VII-1). The concrete was exposed to five environments of sea water and two environments of distilled water as shown in Table VII-1 and previously described in Part III of this report.

VII. 2. Conclusions

VII. 2. 1. Sea Water Environment

1. Concrete exposed to 100° F synthetic sea water brine for 18 months was sound and in good condition.

2. The interior of concrete specimens exposed to 290° F sea water brine for 18 months was sound; however, the exterior concrete was moderately deteriorated and shows chemical alteration, extensive microfractures, and some separation by large cracks. There was also a loss of surface hardness. The crushed quartz aggregate was beginning to show signs of poor bond, but the natural aggregate was affected to a lesser degree.

3. Concrete exposed to 203°, 225°, and 250° F sea water brine for 180 days was sound with only slight chemical reaction in the exterior surface. However, the limestone aggregate was being dissolved on the cut face of the samples. The natural aggregate was not so affected.

Table VII-1

CONCRETE SPECIMEN EXPOSURES

Exposure to 18 months	(100° F System)	Brine containing nominally 175,000 ppm salts. Quartz and natural aggregates.
Exposure to 180 days	(203° F System) (225° F System)	Synthetic sea water 73,000 ppm salts. Limestone and natural aggregate.
Exposure to 90 days	(250° F System)	Synthetic sea water containing 73,000 ppm salts. Epoxy-cement concrete and natural aggregate.
Exposure to 18 months	(290° F System)	Synthetic sea water containing 38,640 ppm salts. Quartz and natural aggregate.
Exposure to 126 days	(150° F System) (350° F System)	Distilled water. Quartz and natural aggregate.

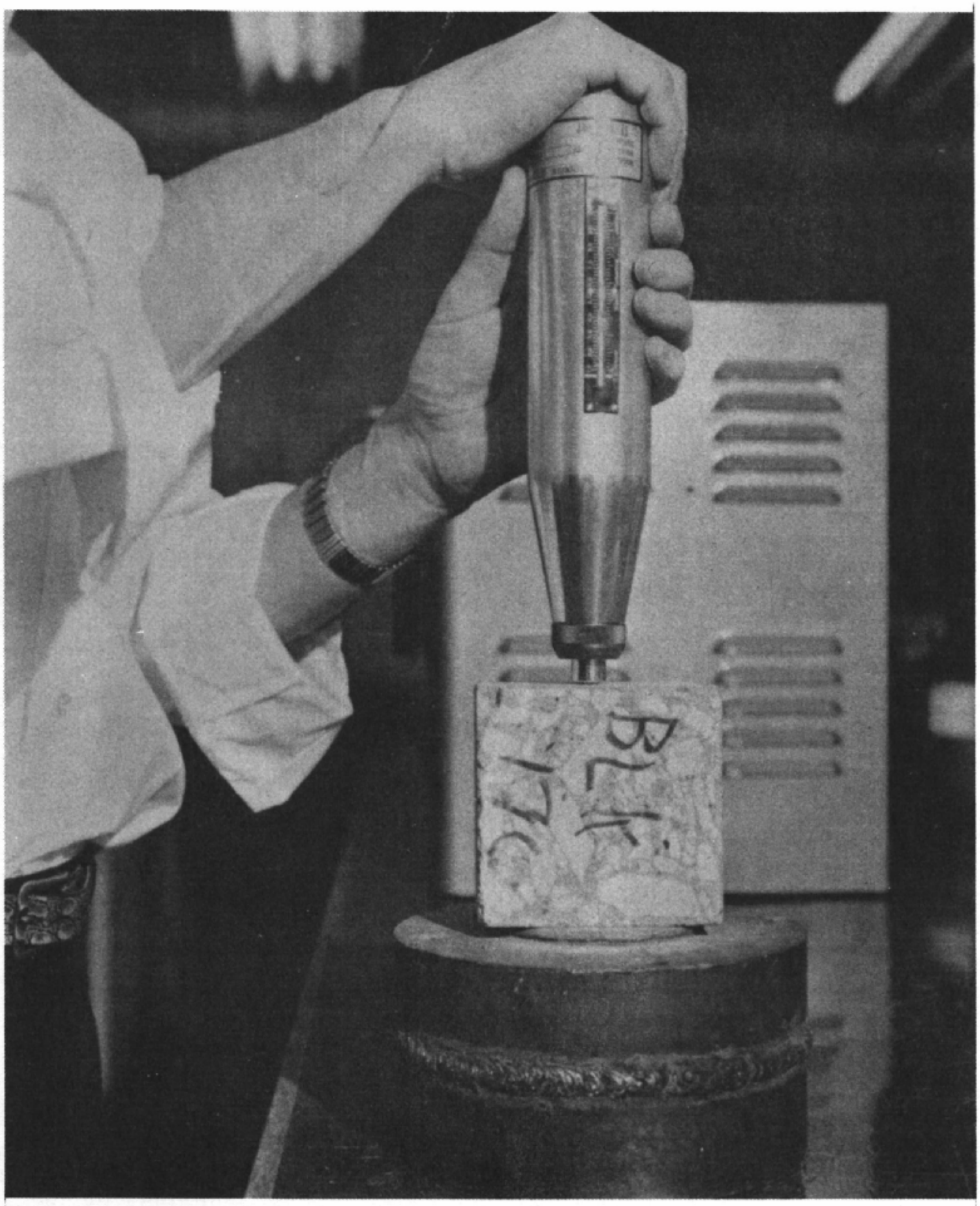

Figure-VII-1. Photograph of impact hammer (small concrete test hammer--Type "L"). Photo PX-D-60857

VII.2.2. Distilled Water Environment

1. The surface of concrete exposed to dripping distilled water was highly deteriorated. Loss of calcium hydroxide has occurred and some aggregate showed signs of deterioration.

VII.2.3. Miscellaneous Tests

1. The reinforced pipe section was deteriorated and fractured. The fractures were probably due to mechanical external stresses. Sulfate attack of the concrete and corrosion of the bars was due to penetration of the brine in the cracks.

2. The asbestos-cement pipe was slightly deteriorated along the outer surfaces to a depth of 3-6 mm. No sulfate crystals were visible. The epoxy lining was separated from half the surface.

VII.3. Unexposed Concrete

VII.3.1. Control Concrete

The concrete for these tests was made with Type V cement, fly ash, and a natural aggregate consisting of amphibolite schist, amphibolite, granite, diorite, quartz, quartzite, and metasandstone. There is an increasing amount of feldspar grains in the sand sizes and a small amount of opal. The concrete after curing and drying was hard, sound, and the aggregate bonding good. About 5 percent of entrained air voids were present and well distributed. The air voids were empty and smooth. There was the usual superficial coating of calcite on the exterior surfaces of the 4- by 4-inch bars caused by surface carbonation. About 3 percent of calcium hydroxide was present in the concrete. Other hydration products appear to be subcrystalline and were not detected by X-ray diffraction which is generally the case with concrete made with Type V cement. A moderate amount of unhydrated cement clinker mineral, mostly beta C_2S, was present which is normal for concrete of the age of these specimens. A second series contained limestone coarse aggregate.

VII.3.2. Epoxy Concrete

This concrete was a mixture of a natural aggregate, epoxy, pozzolan, and portland cement. The cement paste was well hydrated and homogenous. The voids were spherical and the size of the voids was slightly larger than ordinary mass concrete. The bond was good.

VII.3.3. Plastic Impregnated and Irradiated Concrete

The study of plastic impregnated and irradiated concrete is being conducted under other agreements and will be reported elsewhere. However, a few specimens of such treated concrete from a preliminary investigation are being subjected to the environments reported herein.

The petrographic analysis included two types of samples: methyl metacrylate and styrene impregnated and irradiated mortar bars. These bars were impregnated by the soak or vacuum method. The results indicated a thorough penetration by the styrene into bars while occasional patches of dry paste occurred in the methyl methacrylate impregnated bars. The strength of the impregnated bars increased from 2 to 2.4 times as compared with the unimpregnated bars. The styrene had the greatest increase in strength. The absorption had decreased by 86 to 98 percent with the vacuum impregnated samples having the greatest decrease. The comparative hardness increased from 1.4 to 1.9 times. The methyl methacrylate samples had the greatest increase in hardness.

VII.4. Interim Report on Concrete Deterioration

VII.4.1. General Outline

The progressive concrete deterioration for the successive test periods is outlined in Table VII.2. The deterioration which has occurred during the full test period covered by this report is outlined in greater detail in the following paragraphs.

Table VII-2

SUMMARY OF VISUAL EXAMINATION
OSW--CONCRETE
May 1967

Temperature--Time	Sample No.	Coating or alteration	Depth (mm)	Hammer	Description
100° F 28 days	BL-30	Aragonite coating	-	Rings	Good
	XBL-31				
90 days	BL-30	Aragonite coating	-	Rings	Good
	XBL-31				
180 days	BL-30	Aragonite coating	-	Rings	Good
	XBL-31				
270 days	BL-30	Aragonite coating	-	Rings	Good
	XBL-31				
360 days	BL-30	Aragonite coating	-	-	Good
	XBL-31				
18 months	BL-30	Aragonite coating	-	-	Good
	XBL-31				
203° F 28 days	BK	Oxide coating	0.75	*30.8	Good
	BLK			29.6	
90 days	BK	Oxide coating	1.0	30.2	Good
	BLK			29.5	
180 days	BK	Sulfate crystals	1.5	31.7	Slightly deteriorated**
	BLK		1.5	32.6	
225° F 28 days	BW	Oxide coating	0.5	-	Good
	BLW				
90 days	BW	Oxide coating	1.5	-	Good
	BLW		1.5	-	
180 days	BW	Sulfate crystals oxide coating	1.5	24.2	Slightly deteriorated
	BLW			31.1	
250° F 28 days	BT	Sulfate crystals	1.5	34.1	Good
	BLS	Oxide coating		34.0	
90 days	BT	Sulfate crystals,	1.0	34.8	Good
	BLS	Oxide coating	1.5	34.3	
180 days	BT	Sulfate crystals, oxide coating	2.0 1.5	32.0 31.5	Slightly deteriorated
	BLS	Sulfate crystals	1.5	31.5	
290° F 28 days	BH-30	Zinc coating	1.0	Rings	Good
	XBH-31				
90 days	BH-30	Sulfate formed	2.0	Rings	Good
	XBH-31		2.0		
180 days	BH-30	Sulfate increased	5.0	Slightly dull	Slightly deteriorated**
	XBH-31		5.0	Slightly dull	Slightly deteriorated**
290° F 208 days	(Reinf pipe)	Cracked and corroded rebar	2.5	-	Highly deteriorated
270 days	BH-30	Sulfate increased	5.0	Dull	Slightly deteriorated
	XBH-31		5.0	Dull	Slightly deteriorated
360 days	BH	Sulfate increased, oxide coating	5.0	-	Slightly deteriorated
18 months	BH	Abundant sulfate magnetite coating	8.0	11.0	Moderately deteriorated
	XBH	Abundant sulfate, magnetite coating	8.0	12.0	Moderately deteriorated

*Represents rebound values of impact hammer (Type L)
**Deterioration: High-fractured, poor bond, friable, chemical reaction; moderate (borderline case), moderate chemical attack, few fractures; slight, no fractures, occasional reaction

VII.4.2. 100° F Temperature System 18 Months' Exposure

A coating of black iron oxide was present, but no deterioration was observed in preliminary examination. The cement paste was good and the rebound test indicated no decrease in surface hardness.

In thin section, the interior paste was well hydrated and well bonded. A small amount of clinker was present. Slight crystalline areas were found near the exposed edge of the bar. No sulfate crystals were observed in the voids or the cement.

VII.4.3. 290° F Temperature System 18 Months' Exposure

The surface of the quartz aggregate samples disaggregated and broke off at the altered layer (5-8 mm) when the slab samples were sawed. The cement at the broken surface appeared dry and friable. There was a slight increase in thickness of the altered layer from the 1-year samples (4-5 mm). The outer surface of the samples was coated with black magnetite. The altered layer ranged from red to tan in color and was more porous and cracked. In thin section, the altered layer consisted of bands of calcium sulfate (anhydrite) with some iron oxide in the cement and crystals of anhydrite in the voids (Table VII-3, Figures VII-2 and VII-3). Near the border the bands of anhydrite weakened the bond and caused microcracking between aggregate and cement. Carbonation of the altered layer was also present (Table VII-4). In the interior concrete, a small quantity of tobermorite (calcium silicate) was still present. The tobermorite fills voids in most cases, but had not caused any cracking. The impact hardness test, as well as the rebound tests, showed a significant loss of hardness.

VII.4.4. 203° F Temperature System 180 Days' Exposure

The samples were sound, with the altered layer increasing to 1.5 mm. The samples were coated with a brown oxide mineral. In thin section, calcium sulfate crystals (0.10 mm) appear rarely and slight carbonation was seen at the rim of the samples (Table VII-4) and the surface hardness had increased (Tables VII-5 and VII-7).

VII.4.5. 225° F Temperature System 180 Days' Exposure

The samples were sound and alteration of the samples has increased from a depth of 0.25 to 1.5 mm. The samples have a greenish coating. In thin section, scarce calcium sulfate crystals (anhydrite) were seen in the natural aggregate samples. Slight carbonation was visible in the altered layer. The interior cement paste was good and there was no evidence of secondary minerals. A slight quantity of calcium hydroxide was present. No signs of calcium sulfate crystals or calcium hydroxide were observed in the limestone samples. Only slight carbonation was observed (Table VII-4). The limestone showed a slight decrease in hardness by the impact hardness and rebound tests (Table VII-5).

The permeability of the surface coating (magnetite) was tested by the dye-soak test overnight and showed the maximum penetration of the dye in the coated concrete was 0.5 cm, while the maximum penetration was about 1.5 cm in the uncoated surface. This was a difference of about 200 percent in penetration. More comprehensive investigation will be made to determine the permeability of the magnetite coating and its possible effect on the rate of alteration of the concrete. Radioisotope tracers as well as specimens with known permeabilities will be used in this investigation. It is believed the magnetite is a corrosion product from the loops and may have some effect on the rate of brine attack on the concrete.

VII.4.6. 250° F Temperature System 180 Days' Exposure

The samples were good and the altered layer had increased to 2.0 mm thickness. Numerous empty pebble sockets were observed on the cut face of the sample that was fabricated with limestone aggregate (see Figure VII-4). The loss of aggregate was apparently due to the aggregate having been dissolved by the brine. The samples now have a black coating. In thin section, crystals of calcium sulfate (0.33 mm) were more easily visible in the voids of the altered layer. The edges of the concrete were carbonated. The interior cement paste remains unchanged (Figure VII-4). Several of the

Table VII-3

SUMMARY OF MINERALOGICAL ANALYSIS

Temperature - Time	Sample No.	Calcium hydroxide	Anhydrite	Coating	Tobermorite**
100° F 0 days	BL-30	Present	-	-	-
	XBL-31	Present	-	-	-
28 days	BL-30	Less present	-	Aragonite**	-
	XBL-31			Aragonite	
90 days	BL-30	Half present	-	Aragonite	-
	XBL-31	-	-	Aragonite	-
180 days	BL-30	-	Ettringite**	Aragonite	-
	XBL-31	-	-	Aragonite	-
270 days	BL-30	-	-	Aragonite	-
	XBL-31	-	-	Aragonite	-
360 days	BL-30	-	-	Aragonite	-
	XBL-31	-	-	Aragonite	-
18 months	BL-30	-	-	Aragonite	-
	XBL-31	-	-	Aragonite**	-
203° F 28 days	BK	Slight	-	Yellow	-
	BLK	Slight	-		-
90 days	BK	-	-	Yellow	-
	BLK	-	-		
180 days	BK	-	Incipient crystals of anhydrite	Calcite Brown	-
	BLK	-	Incipient crystals of anhydrite		
225° F 28 days	BW	-	Incipient crystals of anhydrite	Yellow	-
	BLW	-	Incipient crystals of anhydrite		-
90 days	BW	-	Anhydrite	Yellow	-
	BLW	-			
180 days	BW	-	Anhydrite	Magnetic green	-
	BLW	-			
250° F 28 days	BT	-	Incipient crystals of anhydrite	Yellow	
	BLS	-	Incipient crystals		-
90 days	BT	-	Anhydrite	Yellow	-
	BLS	-			
80 days	BT	-	Anhydrite increased	Magnetite	-
	BLS	-		-	
290° F 0 days	BH-30	Present	-	-	-
	XBH-31				
28 days	BH-30	Trace*	-	Willemite**	Small amount of crystals
	XBH-31	Trace*	-	Willemite**	Small amount of crystals
90 days	BH-30	None	Anhydrite	Brown	Small amount of crystals
	XBH-31	-			Small amount of crystals
180 days	BH-30	-	Anhydrite layer increasing	Magnetite	Small amount of crystals
	XBH-31	-	Anhydrite layer increasing	Magnetite	Small amount of crystals
270 days	BH-30	-	Anhydrite layer increasing	Magnetite	Small amount of crystals
	XBH-31	-		-	
360 days	BH-30	-	Increased anhydrite layer	Magnetite, serpentine**	Small amount present
	XBH-31	-			
18 months	BH-30	-	Anhydrite, gypsum	Magnetite, clay, serpentine**	No data
	XBH-31	-	Anhydrite, gypsum		

*The fact that calcium hydroxide was not detected by X-ray diffraction could be due to interference by other minerals in the aggregate.
**X-ray diffraction analysis.

Table VII-4

PETROGRAPHIC CHARACTERISTICS OF BRINE-SOAKED CONCRETE
(May 1967)

Sample		Aggregate	Cement	Voids	Absorption, percent	Hammer Type "L"	Description
100° F	270 days	Natural and quartz aggregate. Coarse aggregate, poorly graded. Max size, 1-1/2 in.	Well hydrated, no calcium hydroxide, well bonded. Alter. - slight carbonation.	Well distributed, about 5%	-	Rings (Geology hammer)	Good, brown aragonite coating
	18 months	Natural and quartz aggregate. Coarse aggregate, poorly graded. Max size, 1-1/2 in.	Well hydrated, no calcium hydroxide, well bonded. Alter. - slight carbonation.	Well distributed, about 5%	-	-	Good, aragonite coating
290° F	270 days	Natural and quartz aggregate. Coarse aggregate, poorly graded. Max size, 1-1/2 in.	Well hydrated, no calcium hydroxide. Well bonded. Alter. - numerous calcium sulfate, abundant micro-cracks. Altered layer 5 mm. Magnetite coating.	Well distributed, about 5% Filled voids and cracks in altered layer.	-	Slightly dull ring (Geology hammer)	Slightly deteriorated*
	18 months	Natural and quartz aggregate. Coarse aggregate, poorly graded. Max size, 1-1/2 in.	Increased sulfate and cracking Altered layer 5-8 mm.	Increase in filled voids and cracks.	5.46	11.5	Moderately deteriorated
203° F	180 days	Natural and limestone aggregate. Coarse aggregate poorly graded. Max size, 1-1/2 in.	Well hydrated, no calcium hydroxide. Well bonded. Alter. - incipient calcium sulfate crystals at surface layer. Depth of layer, 1.5 mm. Iron-oxide coating.	Well distributed, about 5% Few filled voids.	-	32.1	Slightly deteriorated
225° F	180 days	Natural and limestone aggregate. Coarse aggregate poorly graded. Max size, 1-1/2 in.	Well hydrated, no calcium hydroxide. Well bonded. Alter. - small amount of calcium sulfate crystals in surface layer. Depth of layer, 1.5 mm.	Well distributed, about 5%	0.9	27.6	Slightly deteriorated
250° F	180 days	Natural and limestone aggregate. Coarse and poorly graded. Max size, 1-1/2 in.	Well hydrated, no calcium hydroxide. Well bonded. Alter. - small amount of calcium sulfate crystals in surface layer. Depth of layer, 2 mm. Magnetite coating.	Well distributed, about 5% Increased amount of filled voids in surface layer.	-	31.7	Slightly deteriorated
250° F	90 days	Natural, coarse aggregate, poorly graded. Max size, 1-1/2 in.	Well hydrated, no calcium by drains. Well bonded. No sulfate crystals.	Well distributed	-	27.4 (cut surface)	Good

*Deterioration: High - Fractured, poor bond, chemical reaction
Moderate - (Borderline case) moderate chemical attack, few fractures
Slight - No fractures, occasional reaction product visible

Table VII-5

IMPACT TEST RESULTS
OSW - Concrete

Temperature	Age	Sample	Ball	"L" hammer	"P" hammer	Absorption	Compressive strength*
203° F	28 days	BK	38.0	30.8	-	-	6,620
		BLK	47.0	29.6	-	-	6,680
	90 days	BK	40.0	30.2	-	3.9	7,180
		BLK	43.0	29.5	-	1.2	6,910
	180 days	BK	46.3	31.7	46.6	-	8,270
		BLK	52.0	32.6	43.6	-	7,190
225° F	28 days	BW	42	-	-	-	7,130
		BLW	42	-	-	-	6,540
	90 days	BW	42	-	-	7.2	7,910
		BLW	43	-	-	10.2	6,970
	180 days	BW	48	24.2	43.3	0.9	8,280
		BLW	49	31.1	-		7,630
250° F	28 days	BT	51	34.1	45	-	7,030
		BLS	51	34.0	45	-	5,840
	90 days	BT	51.8	34.8	44.5	2.0	7,930
		BLS	52.1	34.3	47.1	2.0	6,600
	180 days	BT	49.0	32.0	44.0	-	7,900
		BLS	50.3	31.5	47.0	-	7,040

*Performed by Concrete Laboratory on 6 by 12 cylinders. Average of three samples.

Table VII-6

DEPTH OF ALTERED LAYER VERSUS AGE
290° F System

Temperature	Time (days)	Alteration	Depth (mm)
290° F	0	-	0.0
	28		1.0
	90	Sulfate formed	1.5
	180	Sulfate increasing	5.0
	270	Sulfate increasing	5.0
	360	Sulfate increasing	5.0

Table VII-7

COMPARISON OF CONCRETE SURFACE LAYER ALTERATION

203° F	0	-	0.0
	28	-	0.75
	90	-	1.00
	180	Sulfate formed	1.5
225° F	0	-	0.0
	28	-	0.75
	90	-	1.50
	180	Sulfate formed	1.50
250° F	0	-	0.0
	28	Sulfate formed	1.0
	90	Sulfate increasing	1.75
	180	Sulfate increasing	2.00

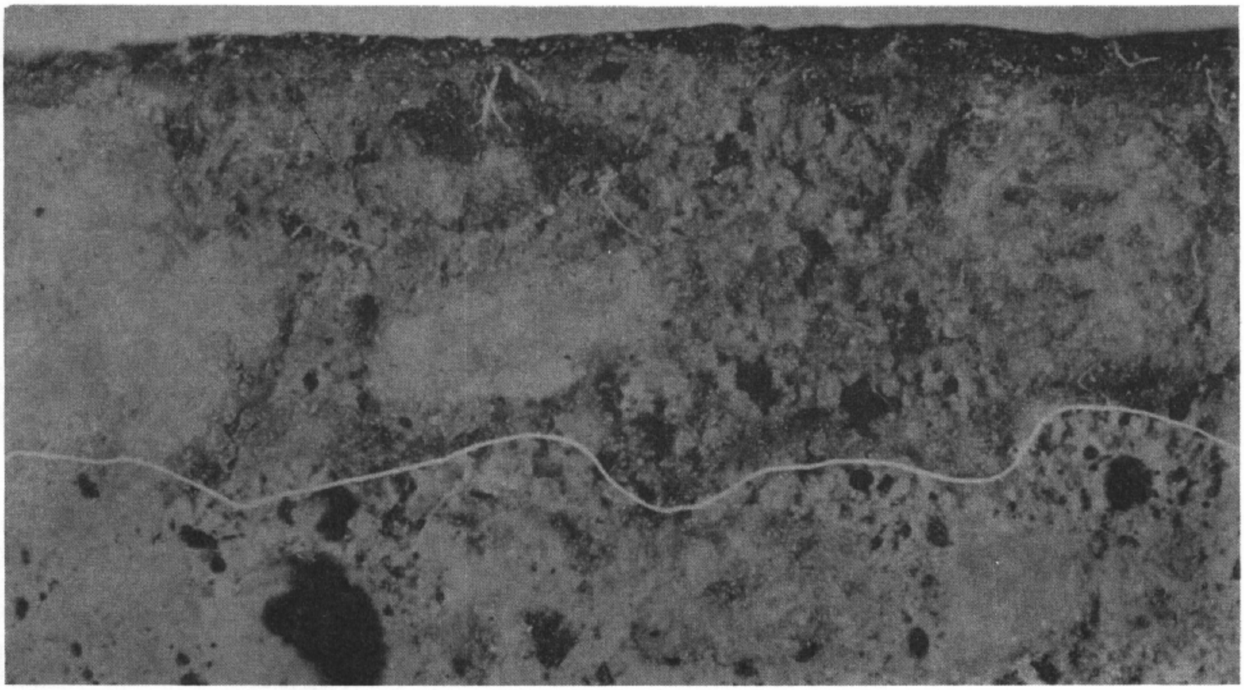

Figure VII-2. Cross-sectional view of concrete showing altered layer on surface after 1 year's exposure to 290° F brine. The horizontal pencil line across the middle of the photograph shows the line of demarcation between the altered layer at the top and unaltered concrete at bottom. Photo PX-D-53379 NA Magnification 12X

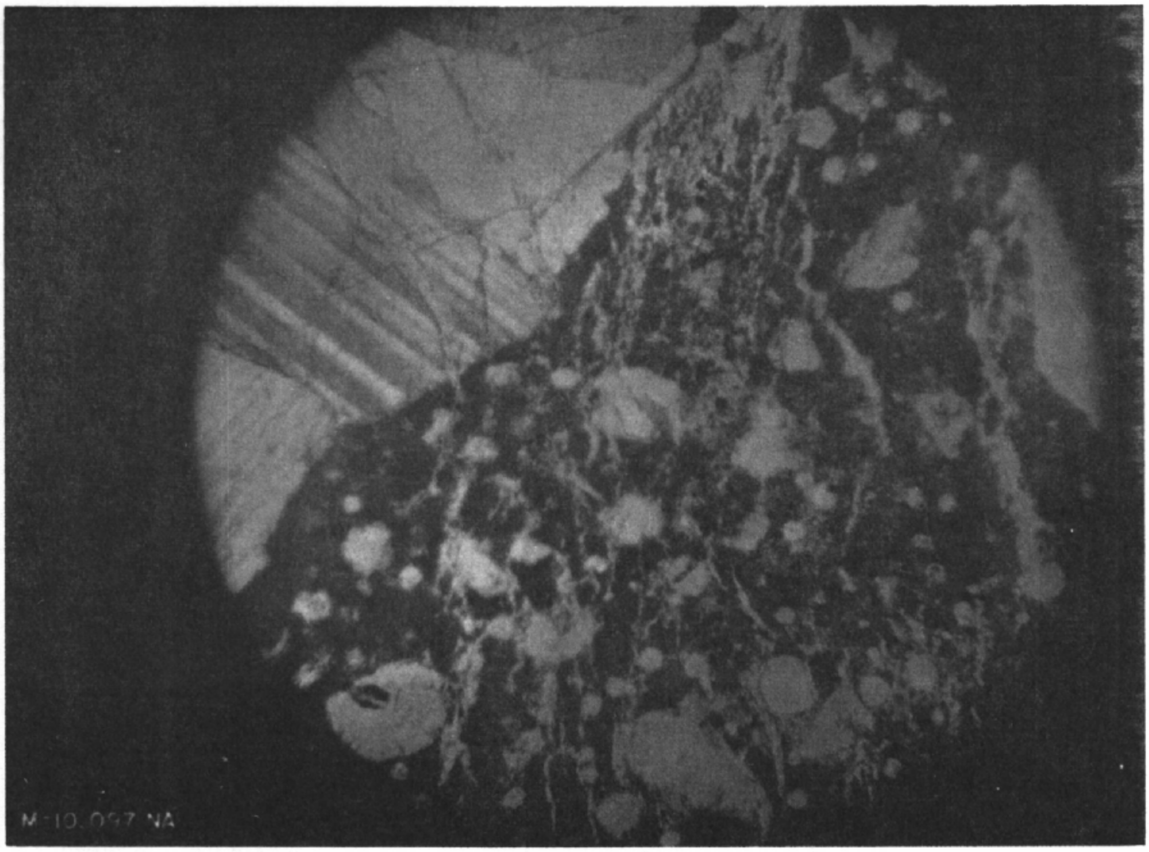

Figure VII-3. Photomicrograph of thin section of altered layer of concrete at 270 days and 290° F. Narrow, elongated white areas are anhydrite filled cracks. Circular white areas are anhydrite filled voids. Irregular and angular white areas are aggregate. Dark area is cement paste. Photo PX-D-60897

Crossed Nicols Magnification 25X

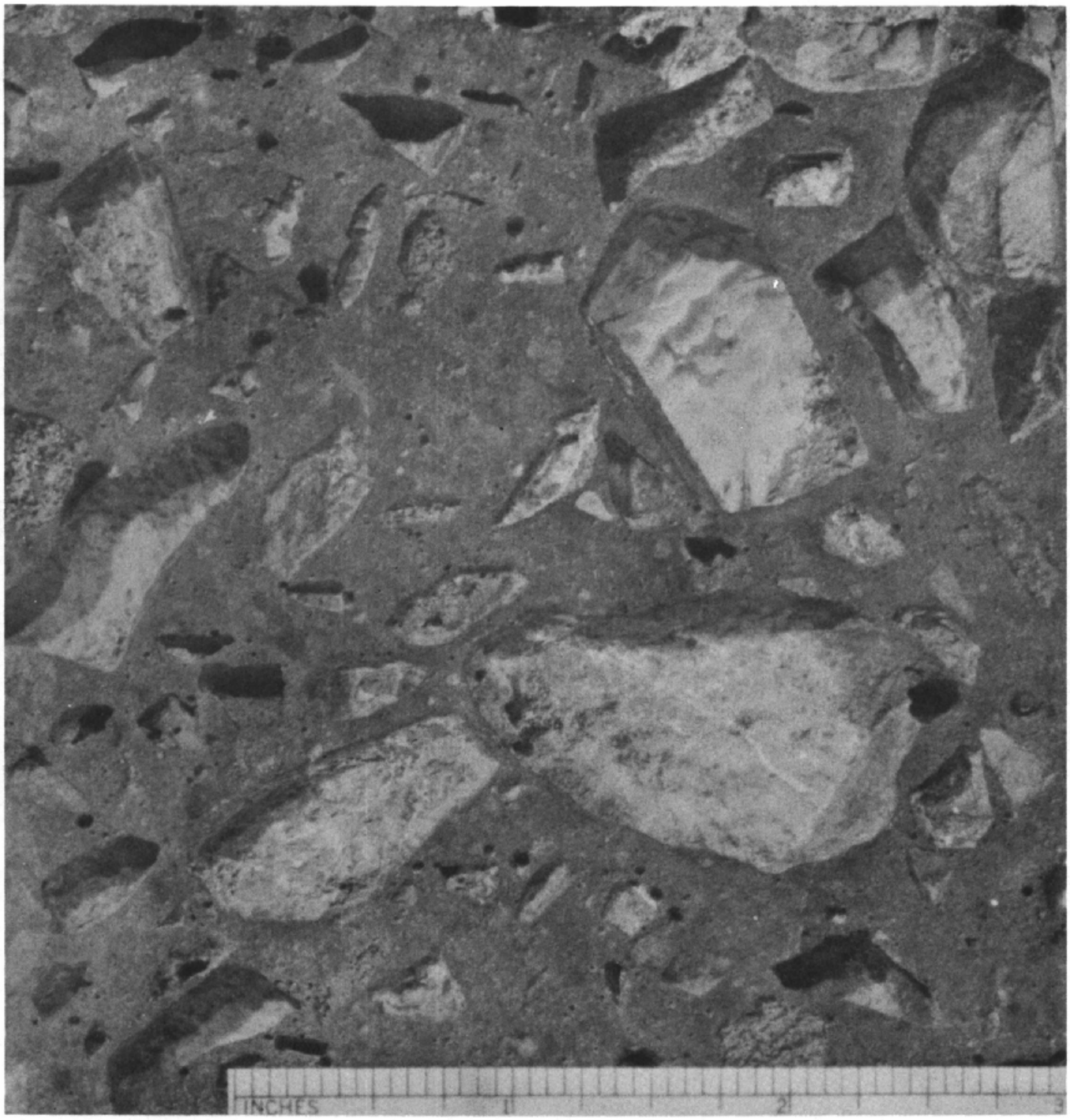

Figure VII-4. Photograph showing dissolution of limestone aggregate after 90 days in sea water brine at 250° F. Photo PX-D-60858 empty pebble sockets contained dolomite crystals (remnants of limestone). There has been a slight decrease in hardness by the impact-hammer tests (see Tables VII-5, and VII-7).

VII.4.7. Comparison of Test Results on Concrete

Graphic comparisons of test results for the different test systems are given in Figures VII-5, VII-6, and VII-7.

VII.4.8. Epoxy Concrete 90 Days' Exposure 250° F (Manufactured Bars and Cylinders)

The concrete was in good condition and a brown coating covered the surface of the specimens. The cement paste was well hydrated and homogenous. The voids were spherical and the size of the voids was slightly larger than ordinary mass concrete. The bond was good. There has been a slight increase in hardness over the 7-day samples.

VII.4.9. Concrete Subjected to Distilled Water Environment

Samples of concrete 7 by 3 by 1 inch were examined visually after being subjected to treatment by dripping 350° F and 150° F temperature distilled water for 126 days.

VII.4.10. Distilled Water 350° F Temperature System

The cement paste was gray, well hydrated and homogenous. There was a moderate amount of calcium hydroxide present and some carbonation has taken place. Dissolution and erosion of the surface layer of cement has washed away the fine aggregate and exposed the coarse aggregate to a depth of 6 mm. The remaining surface layer of cement was soft and chalky. A thin coating of iron oxide was on the surface (Table VII-8).

Some very slight erosion of the feldspar and mica in the schistose rocks has occurred but the quartz and more resistant minerals show no deterioration.

VII.4.11. Distilled Water 150° F Temperature System

The samples were similar in appearance to the samples above, but the deterioration was less severe.

VII.4.12. Reinforced Concrete Pipe in 290° F Brine System

The inner surface of the pipe was deteriorated from the brine after 208 days' exposure. The pipe wall was cracked part way through and the rebars were corroded. Several cracks about 3 inches in length along the outer concrete surface were seen. These cracks are open and iron stained and extend to a depth of 2 inches (the pipe wall is 4 inches thick). One of the fracters was around the circumference and another was along the length of the pipe forming a "T". These fractures intersect the reinforcing steel at numerous places (Figure VII-8).

The cement paste in general was in good condition, but the paste adjacent to the inner surfaces of the pipe was deteriorated to a depth of 2.4 mm (0.1 inch). This was about the same amount of deterioration as in the 4- by 4- by 36-inch bars of approximately the same exposure. A coating of iron oxide and anhydrite (calcium sulfate) was present which was soft and porous. There was also a thin zone of alteration under the brown coating which contains anhydrite in the voids and microcracks. Another area of deterioration was in the concrete which was in contact with the corroded rebars. The bond between the concrete and rebars has apparently been broken by corrosion of the rebars. The cement was carbonated at the surface of the cracks.

The reinforcing bars that intersect the cracks exhibit corrosion in the form of iron oxide scale on the surface of the bars. The bars that do not intersect the cracks show

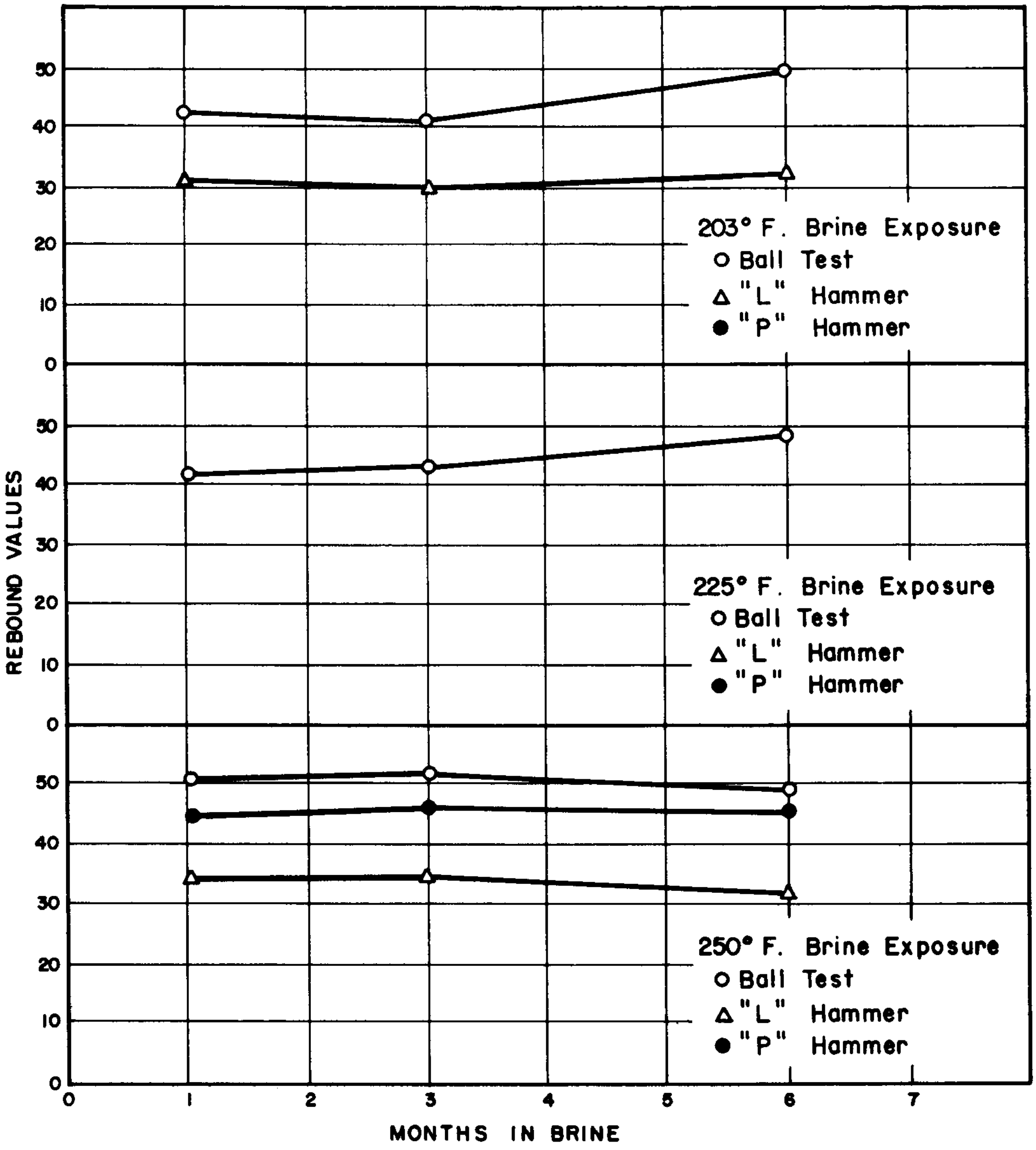

FIGURE VII-5: COMPARISON OF RESULTS OF IMPACT TESTS CONDUCTED ON CONCRETE EXPOSED TO VARIOUS BRINE ENVIRONMENTS

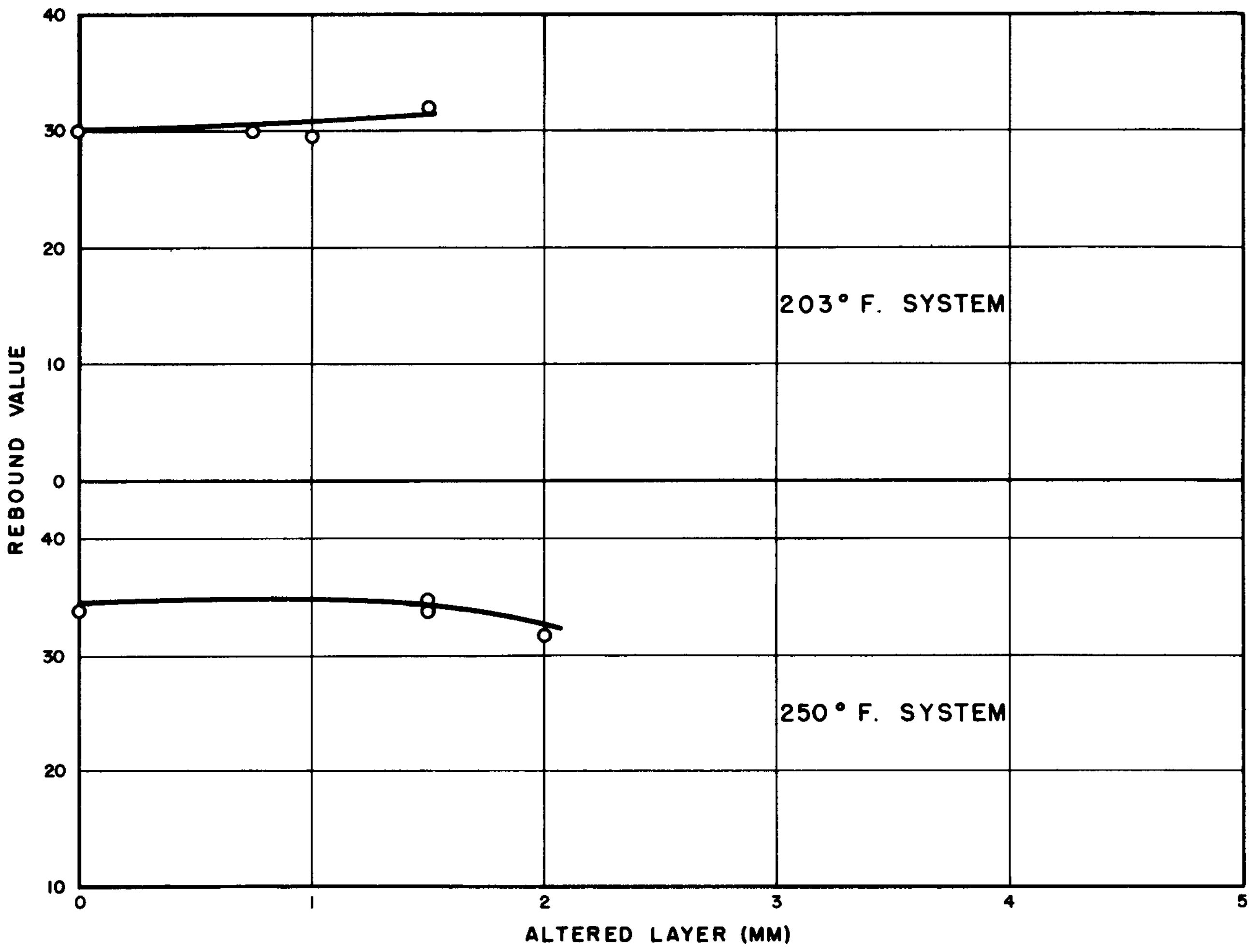

FIGURE VII- 6: COMPARISON OF REBOUND VALUE AND ALTERED LAYER

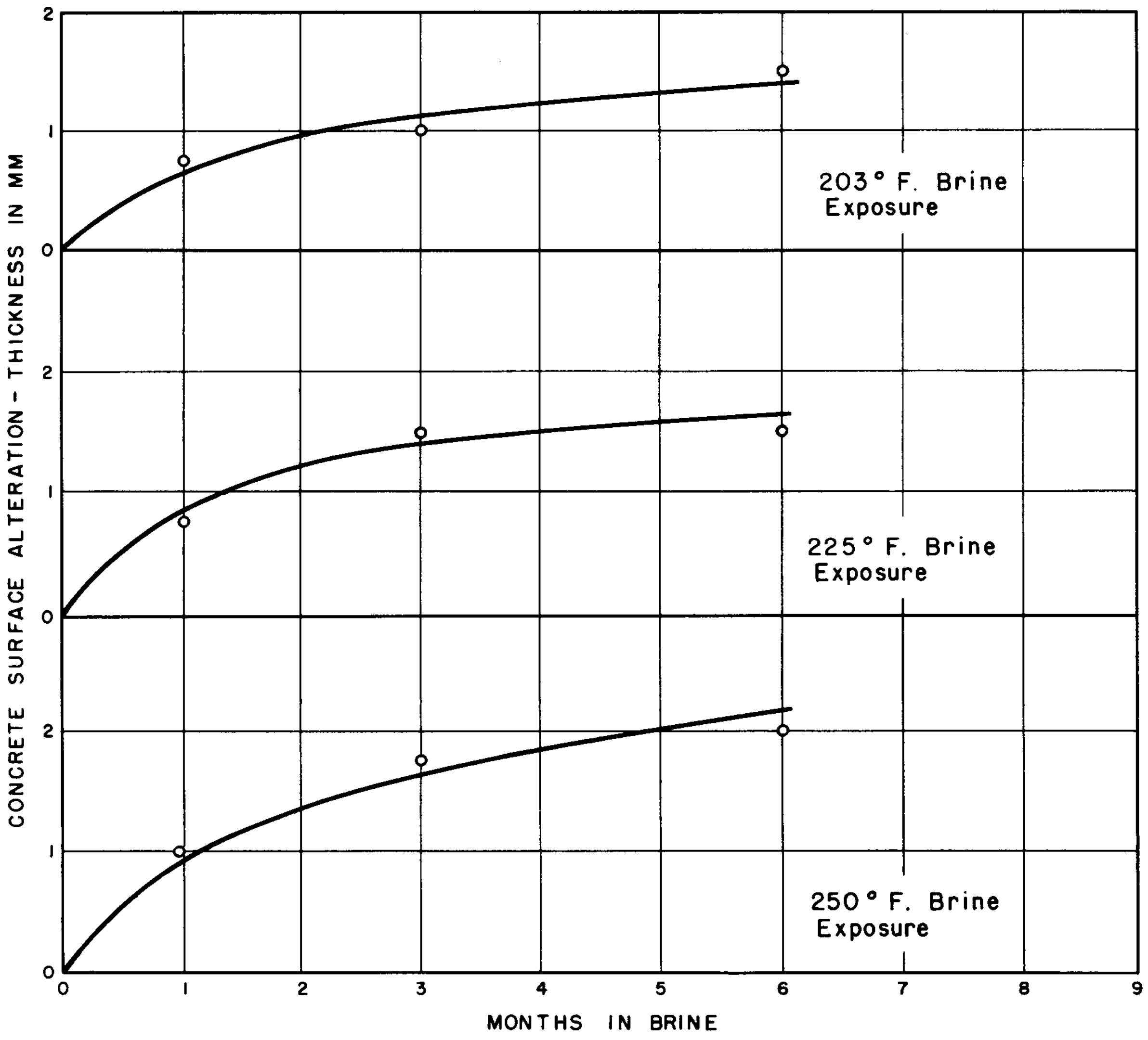

FIGURE VII - 7 : COMPARISON OF CONCRETE SURFACE LAYER ALTERATION

Table VII-8

CONCRETE IN CONTACT WITH DISTILLED WATER

Sample		Aggregate	Cement	Voids	Absorption, percent	Hardness	Description
Temperature	Time						
150° F	126 days	Natural and quartz aggregate. Coarse, poorly graded. Max size, 1-1/2 inches. Alteration, poor bond on surface	Well hydrated, well bonded. About 12% calcium hydroxide. Slight carbonation. Alteration, loss of cement. Depth of pitting, 0.5 to 1 mm: Oxide coating	Well distributed, about 5%	3.6-4.4	16-20	Slightly deteriorated,* surface leached and exposed
350° F	126 days	Natural and quartz aggregate. Coarse, poorly graded. Max size, 1-1/2 inches. Alteration, breakdown of amphibole pebbles	Well hydrated, well bonded, about 12% calcium hydroxide. Alteration, loss of cement and calcium hydroxide. Depth of pitting, 3 to 6 mm. Oxide coating	Well distributed, about 5%. Few filled voids	-	-	Moderately deteriorated, surface leached and exposed

*Deterioration:
High --Fractured, poor bond, chemical reaction
Moderate--(Borderline case) moderate chemical attack, few fractures
Slight --No fractures, occasional reaction product

only slight corrosion. The corrosion experienced by the embedded reinforcing bars was probably due to the lack of corrosion inhibiting free lime in the concrete. The free lime content of the test specimens had been purposefully reduced in order to reduce the content of soluble constituents in concrete. The extent of the cracking found in the reinforced concrete pipe permitted significant corrosion to occur whereas in the 6- by 6- by 30-inch bar specimen's the cracking was not extensive enough to destroy the protective environment afforded the steel by the concrete.

The fractures intersecting the reinforcing bars contain sodium chloride and iron oxide. These fractures appear to have been caused or started by external stresses upon the concrete, such as attachment of the pipe to the loop.

In the samples examined, there was no evidence of a continuous crack completely through the pipe wall. Instead, the brine entered the pipe wall in a crack in the angle of the flange about 1 inch in from the outer surface where the concrete pipe was fitted into the loop. In this area, corrosion of the rebar and penetration of the corrosion products into the crack were more extensive than in any other areas examined.

The fractures in the reinforced concrete pipe appear to have been caused by external stresses upon the concrete. The concrete was deteriorated but was comparable in quality to the other specimens of the same age and environment. The reinforcing steel was corroded where exposed to the brine. (See related photographs in Section III.)

VII.4.13. Asbestos-cement Pipe in 290° F Brine Solution

The 8-inch-diameter (7/8-inch-thick) and 6-inch-diameter (9/16-inch-thick) autoclaved asbestos pipe samples were slightly deteriorated and discolored along the outer surfaces to a depth of 3 to 6 mm. The surface was intact but soft and porous. The interior of the sample was sound (Figure VII-9).

This outer surface cement was moderately carbonated, cracked and slightly leached but no sulfate crystals were visible. There was no apparent difference in the samples except that the thicker pipe (7/8 inch) showed a deeper (6 mm) alteration layer. Also the epoxy lining protected the pipe where it was still bonded to the pipe, but it had separated from half the inner circumference of the pipe.

VII.5. Discussion

VII.5.1. 100° and 290° F Brine Systems

A comparison of the concrete exposed to high (290° F) and low (100° F) temperatures and saline conditions and containing natural and quartz aggregates, indicated that at 18 months in the high temperature loop (290° F), surface deterioration due to sulfate attack has progressed. The maximum depth of the altered layer was 8 mm. However, the tan layer is relatively porous and soft, and microcracks occur frequently. The depth of the thin reddish layer containing abundant anhydrite had slightly increased. The outer layer of magnetite had also increased. A comparison of the data (Figure VII-10) shows that the rate of deterioration was increasing. If the deteriorated portion of the concrete remains intact over a 30-year period (the curve becomes asymptotic), the amount of deterioration would be 0.2 inch. This must be regarded with some caution as petrographic analysis shows that sulfate activity was still present. However, the data indicated that the curve for a special case of the parabola $x = 0.2\quad t$ gives the best fit with a standard deviation of 1 mm. This curve representing the rate of deterioration projected for a period of 30 years would indicate a loss of 1.3 inches of concrete. It is believed the most likely effect is for the deterioration to follow a curve somewhere between the asymptotic and parabolic curves. In another case where the deteriorated layer might disintegrate and fall off, the curve would lie somewhere between a parabola (1.3 inches loss) and a straight line (7.8 inches loss) like a series of steps. The data give a general indication of the possible range of deterioration.

The apparent leveling off or lack of change in the depth of the altered layer for a period of 6 months was being examined. This appears to have occurred at about the same time the superficial oxide coating has reached a certain thickness and changed to magnetite.

Figure VII-8. Photograph of reinforced pipe section showing staining around rebar. Photo PX-D-60859

Magnification .8X

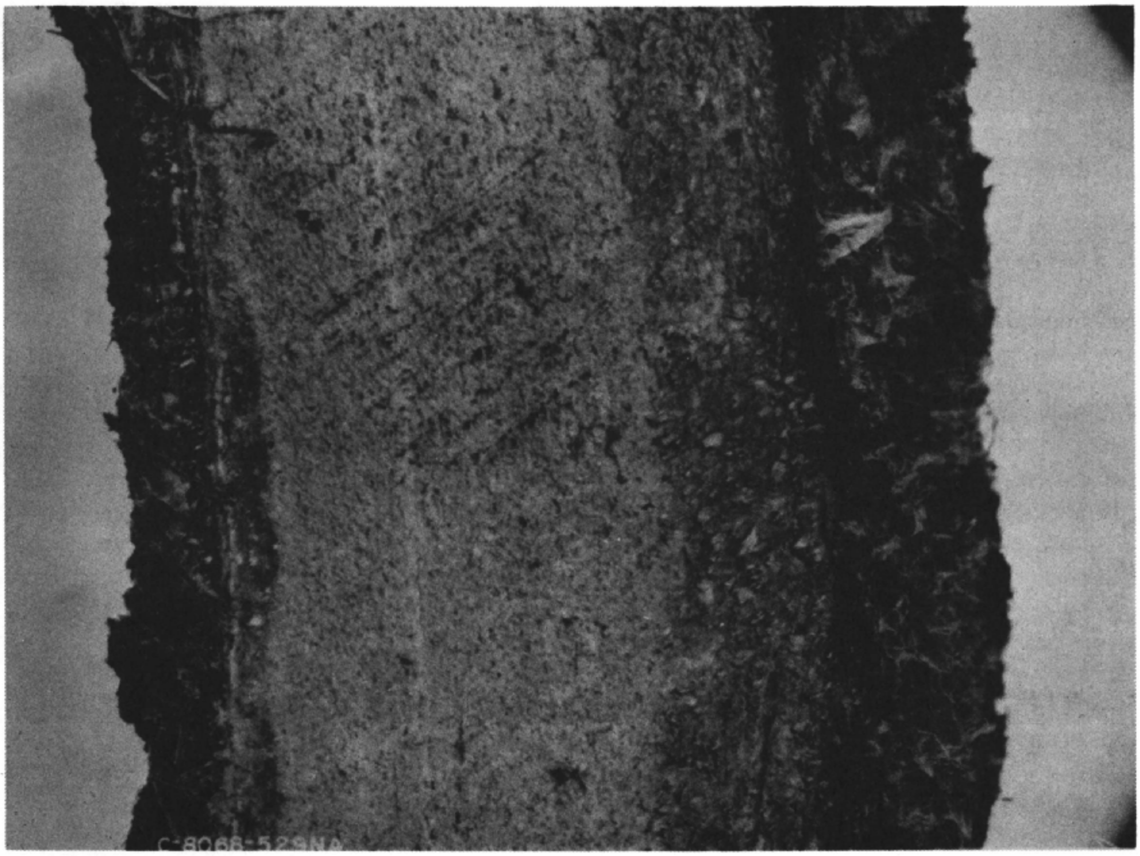

Figure VII-9. Photograph of asbestos-cement pipe showing alteration of outer surfaces (left and right sides). Photo PX-D-60860

Magnification 7X

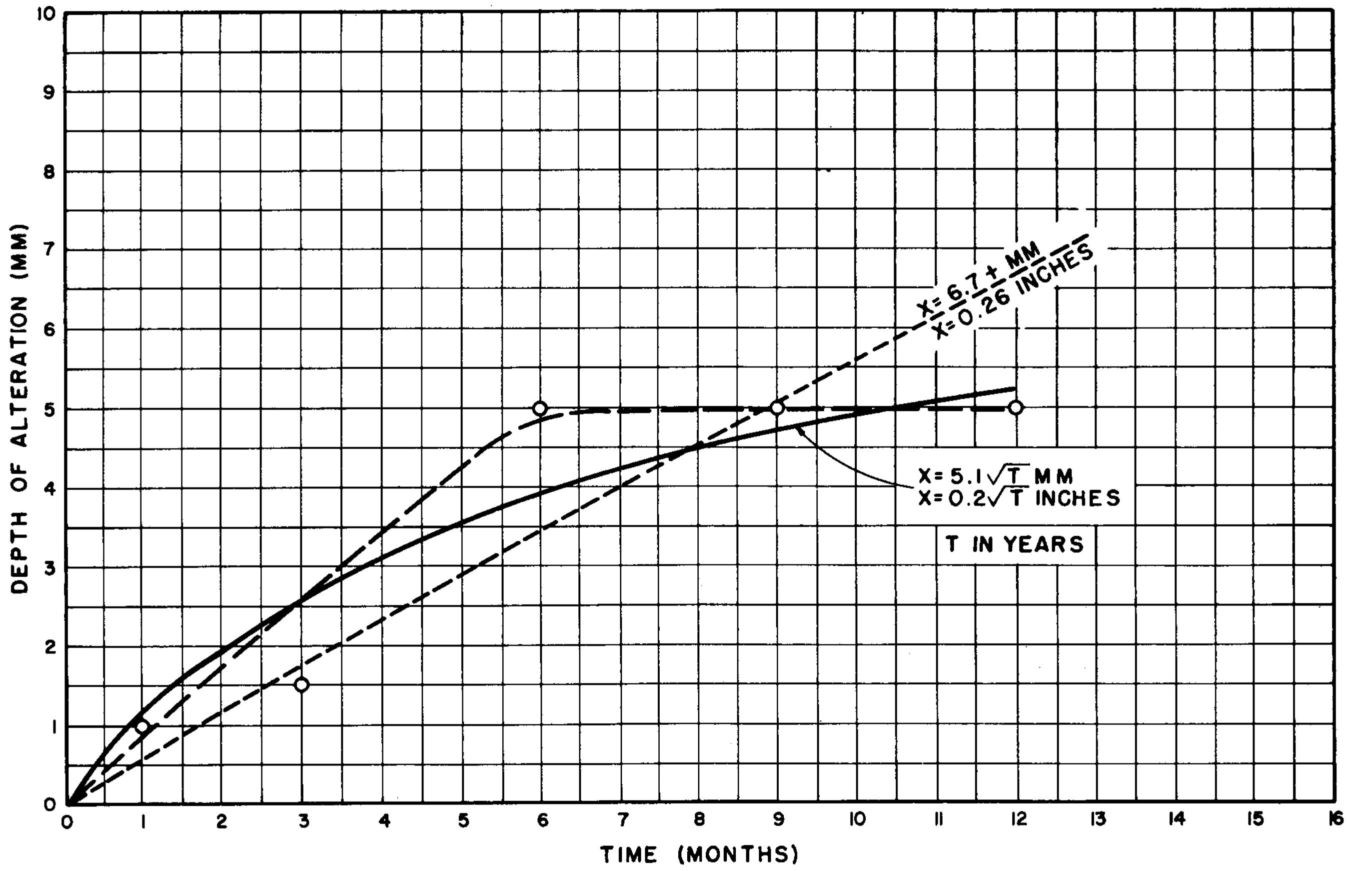

FIGURE VII -10: CONCRETE EXPOSED TO 290°F. BRINE WITH THREE DIFFERENT INTERPRETATIONS FOR THE SAME POINTS

At the low temperature (100° F) at 18 months' exposure no secondary products of sulfate attack were visible. The presence of easily altered minerals such as carbonate of calcium or iron in an unaltered state suggests that no alteration has taken place. The cement paste was well hydrated, homogenous, and well bonded. No loss of surface hardness has occurred.

VII.5.2. 203°, 225°, and 250° F Brine Systems

The concretes at the intermediate temperatures (203°, 225°, and 250° F) and containing natural and limestone aggretates at 180 days show surface deterioration but at a substantially lower rate than for the 290° F exposure depending on the temperature. The maximum depth of the altered layer was 2 mm compared with 5 mm for the 290° F samples at the same age. At 250° F the oxide coating has changed to magnetite as was the case with the higher temperature samples.

APPENDIX VII. 6.

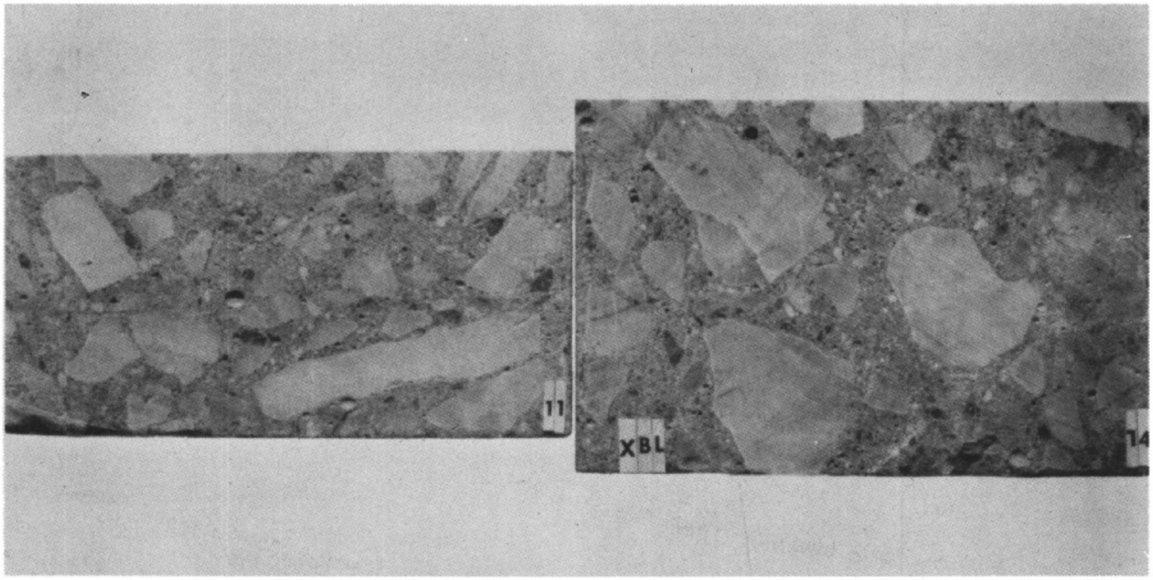

Figure VII-11. Photograph of quartz aggregate concrete after 12 and 18 months 100° F brine exposure. Photo PX-D-60861

Magnification .7X

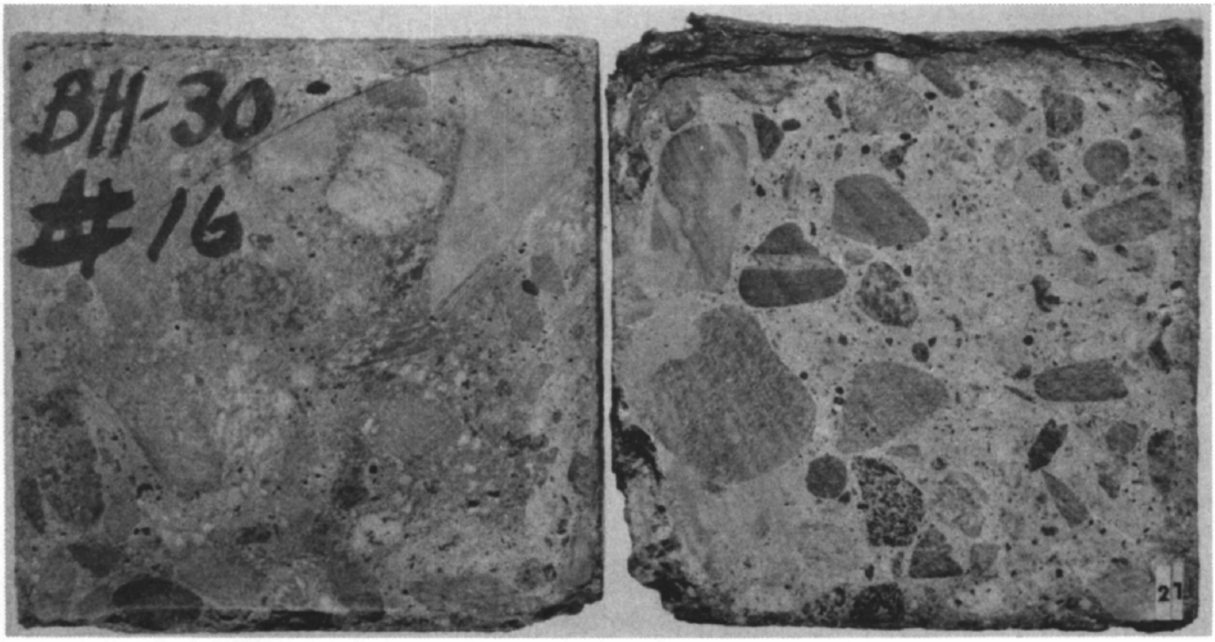

Figure VII-12. Photograph of natural aggregate concrete after 12 and 18 months 290° F brine exposure. Photo PX-D-60862

Magnification .7X

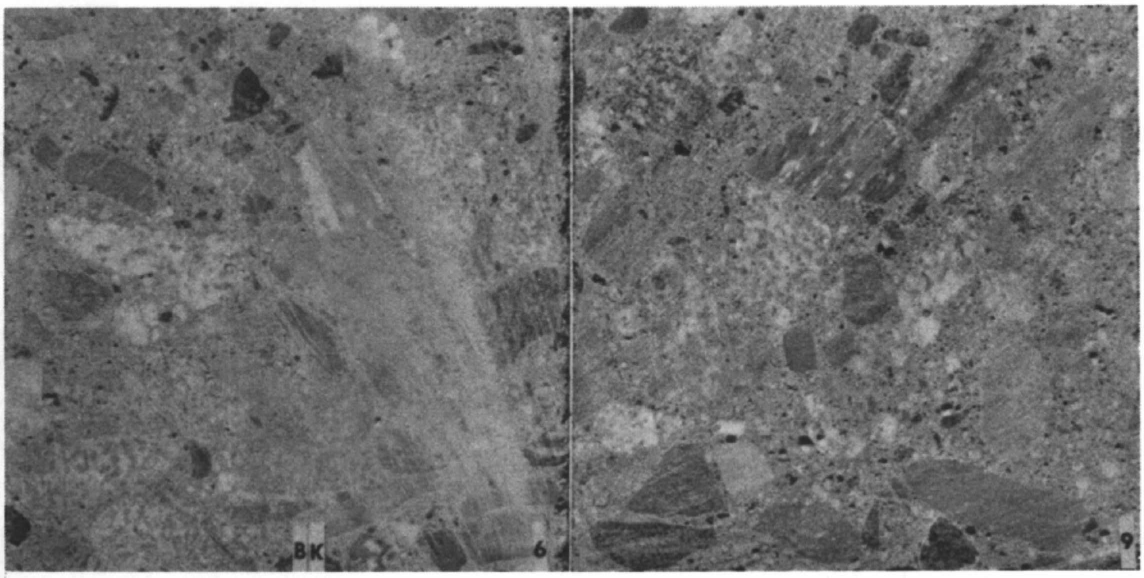

Figure VII-13. Photograph of natural aggregate concrete after 90 and 180 days exposure to 203° F brine. Photo PX-D-60863

Magnification . 7X

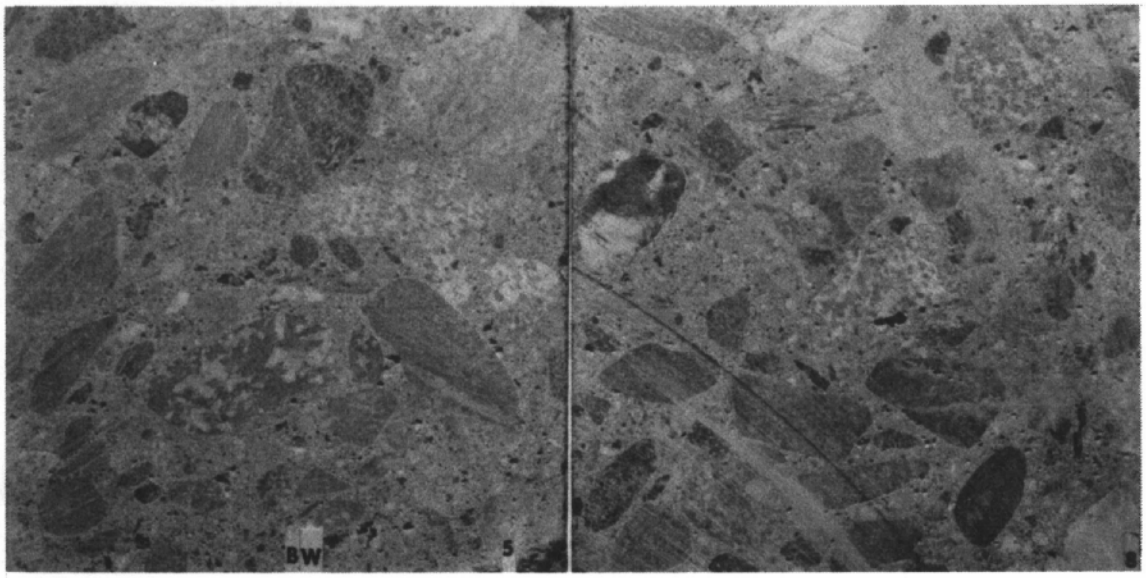

Figure VII-14. Photograph of natural aggregate concrete after 90 and 180 days' exposure to 225° F brine. Photo PX-D-60864

Magnification . 7X

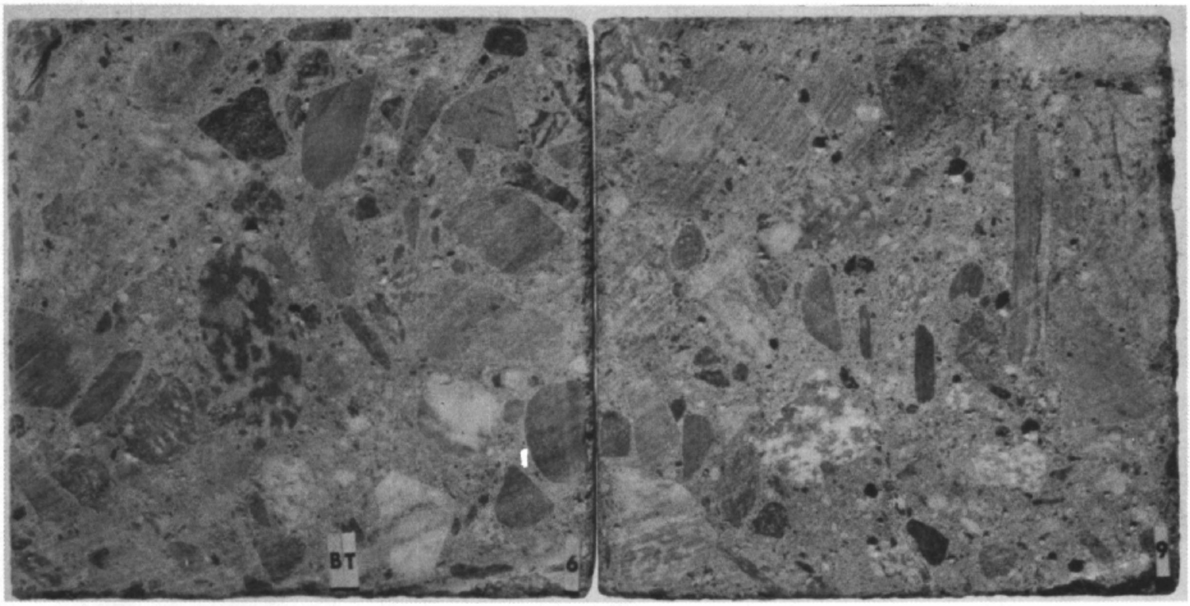

Figure VII-15. Photograph of natural aggregate concrete after 90 and 180 days' exposure to 250° F brine. Photo PX-D-60865

Magnification . 7X

SECTION VIII

STABLE BINDERS

Work Order No. 6

VIII. 1. Introduction

The paucity of information on behavior of concrete in saline and distilled water at elevated temperatures necessitated an investigation of potential binders of concrete at expected service temperatures. Much is known about sea water attack on concrete at normal temperatures and correctives for this problem have been developed. At elevated temperatures, it is expected that the reactions between sea water and concrete binders are different than at normal temperatures and that prevention of attack will require modifications in the chemical makeup of the binders.

Portland cement is the principal source of the binder for concrete. A calcium aluminate (CA) cement by the trade name of Lumnite has been used as a protective lining of steel pipe against corrosion. At the Freeport, Texas, sulfur plant, pipes so lined carry saline solutions at temperatures up to 320° F. The principal property of CA cements in structural concrete is high 24-hour strength. Many failures of such structures have been attributed to transformation of the hydrated cement to a denser phase known as hydrogarnet or isometric tricalcium aluminate hexahydrate (C_3AH_6). Moisture, temperatures in excess of about 80° F, and permeability of the paste are conditions that accelerate the formation of C_3AH_6. Information on behavior of CA cements at elevated temperatures is meager. The present program includes tests on these cements as possible lining materials for steel pipe, chambers, etc. in desalination plants.

VIII. 2. Conclusions

Based upon the tests conducted to date and described herein, the following conclusions seem justified:

1. Although the time of exposure of the specimens, made with portland cement and additions in sea water at 250° F is too short to permit extrapolations for predicting long time results, the overall results appear promising. The phases expected to cause difficulty by reducing strength have not formed or only in a small amount in the temperature range of 210° to 290° F.

2. The present study on CA cement showed that in the 90° to 110° F range, the strengths are highly variable among mixes made on different days. High strength was invariably associated with presence of calcium aluminate decahydrate (CAH_{10}) as major binder phase and low strengths with aluminum hydroxide/gibbsite (AH_3/G). The present results do not substantiate previous hypothesis that curing at temperatures above about 80° F causes conversion of the CAH_{10} to hydrogarnet (HG) and reduced strength. The formation of AH_3 is a factor at early ages. Only after 7 days' curing at 115° and 130° F was HG observed.

3. Apparently, the nucleation and formation of CAH_{10} as a major binder phase is the factor contributing most to early high and uniform strengths. Experiments are underway to secure preferential nucleation and, therefore, formation of CAH_{10}. "Seeding" with CAH_{10} and prehydration for 6 to 24 hours at 40° F had given the desired result. A search is underway for a low cost "seeding" material as addition to the cement. Success in these experiments would eliminate the unpredictable and eratic behavior during curing of calcium aluminate cements.

VIII. 3. Procedure of Tests

VIII. 3. 1. Raw Materials and Proportions

The raw materials were ground to desired finenesses. These results and composition data are recorded in Tables III-13, III-14, and VIII-1 and VIII-2 for the Type V cement, fly ash, Type I cement and electric furnace slag, respectively. Blends in following proportions were prepared in a laboratory mill containing a few pebbles.

1. Type V cement with replacements of 0.0, 7.5, 15.0, 22.5, 30.0, and 37.5 percent fly ash .

2. Type I cement-fly ash blends of above compositions were also prepared.

3. Type V cement with 30, 40, 50, and 60 percent replacement by the electric furnace slag.

4. Type V cement with 1.4 to 6.5 percent SO_3 as gypsum.

5. No percent Type V cement: 30 percent fly ash blend with 1.4 to 5.5 percent SO_3 as gypsum.

6. Blend of 50 percent Type V: 50 percent slag with 1 to 3 percent SO_3 as gypsum.

7. Other individual materials or blends in limited numbers.

VIII. 3. 2. Screening Tests on Portland Cement Binders

The tests on portland cements were designed to eliminate early in the program any binders not possessing desirable properties; the first among these being strength. It was possible to take a short cut in the preliminary tests through results of examination of cured pastes by X-ray diffraction (XRD) and differential thermal analysis (DTA). The phases in the pastes known to cause lowered strengths are readily identified. These are alpha dicalcium silicate hydrate (or αC_2SH) and the isometric solid solution known as hydrogarnets with a generalized formula of

$$C_3AH_6\text{-}C_3AS_3$$

$$C_3FH_6\text{-}C_3FS_3.$$

Presence of these phases in large amounts would immediately make the binder suspect as being of low strength. At the higher proposed temperatures of plant operations and using pozzolans, the most probable reaction product is tobermorite having a composition of $C_5S_6H_5$ and readily identified by a unique XRD pattern. This hydrate is a binder of excellent strength. Also, autoclaved cement-silica products have a sulfate resistance superior to that of normally cured cements. Well crystallized tobermorite, as shown by Soviet research, tends to be highly permeable. Low permeability is a desired property of the binders for purposes of reducing or stopping ingress of the corrosive solutions into the interior of the concrete.

The pastes were prepared generally with a water/cement ratio (W/C) of 0.25 because such pastes were neither too stiff for good placing nor too fluid to result in segregation. Additional W/C ratios (0.40 and 0.55) were used for development of specific information principally for determining any possible effect high water content may have on hydrated pastes. In the first preliminary tests, the pastes were contained in small nickel crucibles. These tests were conducted to determine if αC_2SH, the hydrogarnets, or well crystallized tobermorite formed in moderate to large amounts. As the program progressed, the pastes were cast as 1-inch cubes for compressive strength determination. After a strength test, a cube was crushed, washed with dry acetone, and then degassed for 18 hours under a final vacuum of 0.04 to 0.05 mm Hg (40 to 50 torr). The dried sample was examined by DTA and XRD.

VIII. 3. 3. Equipment

The required equipment was essentially that used in research in high-pressure steam (autoclave) curing. The small autoclaves or pressure pots were available in part in the laboratory. Others were purchased, still others in the laboratory were modified for this program. Some of the accessories: 1-inch cube molds, racks for the pressure pots, pressure controllers for the autoclaves, etc., were built or assembled in the Bureau shops.

As mentioned in the introduction, it was unknown if disruptive compounds would form in reaction between the cement pastes and sea water at the elevated temperatures.

Table VIII-1
CEMENT TEST DATA
Stable Binder Investigation for OSW

Project: Denver Office--Cement Research

Sample No. M-5097

Specifications No.: Federal No. SS-C-192d

Date Received: 1/27/65

Amount Received: Approx. 1000 lbs--2 bbls

Type of Cement: I, Low Alkali

Mill: Kaiser Cement and Gypsum Corp. Permanente, Calif. Plant

Reference Letter: Date

To

From

Subject

PHYSICAL PROPERTIES

Specific Gravity: 3.12

Autoclave Expansion: -0.005 Percent

Normal Consistency: 23.4 Percent

Initial Set: Hours 3 Minutes 15

Final Set: Hours 4 Minutes 30

Compressive Strength of 2- by 2-inch Cubes

1-day psi

3 days 2808 psi

7 days 4179 psi

28 days 6198 psi

Percent Air in Mortar: 7.9

False Set:

Initial, 1/4-min pen., mm 34

5-min pen., mm 8

Remix 1/4-min pen., mm 15

Remix 5-min pen., mm 8

Percent Passing No. 325 Sieve:

Blaine Surface, cm^2/g: 3,924

Heat of Hydration:

3 days cal/gram

7 days cal/gram

28 days cal/gram

Other Tests:

CHEMICAL ANALYSIS

Silicon Dioxide (SiO_2) 23.01 Percent

Aluminum Oxide (Al_2O_3) 5.05 Percent

Ferric Oxide (Fe_2O_3) 2.11 Percent

Calcium Oxide (CaO) 64.36 Percent

Magnesium Oxide (MgO) 0.72 Percent

Sulfur Trioxide (SO_3) 2.22 Percent

Sodium Oxide (Na_2O) 0.15 Percent

Potassium Oxide (K_2O) 0.25 Percent

Total Alkalies, as Na_2O 0.31 Percent

Loss on Ignition 1.69 Percent

Insoluble Residue 0.17 Percent

CHEMICAL COMPOUNDS

Tricalcium Silicate (C_3S) 43.73 Percent

Dicalcium Silicate (C_2S) 23.06 Percent

Tricalcium Aluminate (C_3A) 9.82 Percent

Tetracalcium Aluminoferrite (C_4AF) 6.41 Percent

Calcium Sulfate ($CaSO_4$) 3.81 Percent

Remarks:

Table VIII-2
POZZOLAN TEST DATA
Stable Binder Investigation for OSW

Project: Research
Sample No. M-3975 Specifications No. USBR, 3/1/61
Date Received: 1/20/61 Amount Received: 460
Identification of material: Electric furnace slag
Reference Letter: Date To
From Subject
Name of Deposit: Location: Monsanto, Idaho
Ownership: Monsanto Chemical Company Estimated Quantity: --

PETROGRAPHIC ANALYSIS

Description of Material: Sand-sized angular particles of crushed slag. The slag is quite glassy (about 60 percent) and the refractive index of the glass averages about 1.61.

PHYSICAL PROPERTIES

Grinding Time: Hours Minutes
Fineness:
Air Permeability 8,765 cm^2/cubic centimeter
Percent Retained on No. 325 Sieve 7.7
Specific Gravity: 2.87
Water Requirement: 99 Percent of Control
Difference in Percent Drying Shrinkage (Test bar minus control bar): 0.010

Reduction of Reactive Expansion at 14 Days: 59 Percent

Compressive Strength of 2-inch Cubes		PSI	Percent of Control
Control Cement:	Sealed cured, 7 days		
M-3650, Type II,	Sealed cured, 28 days		
low alkali	Sealed cured, 90 days		
With Pozzolan	Sealed cured, 7 days	3,578	86
Replacement: 35%	Sealed cured, 28 days	5,583	96
	Sealed cured, 90 days		

Compressive Strength of 2- by 4-inch Cylinders of Pozzolan-lime Mortar at 7 days, psi: 2275

CHEMICAL ANALYSIS

Silicon dioxide (SiO_2) plus Aluminum Oxide (Al_2O_3) plus Ferric Oxide (Fe_2O_3) 49.88 Percent
Magnesium oxide (MgO) 1.83 Percent
Sulfur trioxide (SO_3) 0.13 Percent
Loss on ignition 0.22 Percent
Moisture content 0.06 Percent
Exchangeable alkalies as Na_2O 0.55 Percent

Remarks:

CaO = 44.46 percent, P_2O_5 = 1.01 percent

Attack of sea water and sulfate solutions at room temperatures are easily and reliably determined by measuring expansion (increase in length of prisms) with a laboratory comparator. At elevated temperatures, deterioration might possibly occur without significant volume change. New test method or methods had to be developed for this purpose.

The products of reaction of sea water and cement pastes above about 200° F would be $Mg(OH)_2$, anhydrite ($CaSO_4$) and possibly other new phases (one to be described in this report). The formation of these phases can be detected by XRD/DTA and the amounts estimated. These reaction products may form only on or in the surfaces of the test specimen and cause clogging of the pores. Permeable paste specimens deliberately included as companion test specimens to ones of low permeability, would permit ingress of the solutions towards the interior of the specimen and early formation of the reaction products.

The progress of the reactions from the specimen surface inwardly can be followed by examination of "cuts" of the pastes at different and controlled depths from the surface. Facilities for collecting "cut" specimens at different and controlled depths have been designed and built by modifications of a metal turning lathe. The specimens for this purpose are 2- by 4-inch paste cylinders.

The suitability of dynamic modulus results for detecting deterioration is being examined using 1- by 1- by 11-1/4-inch mortar bars. These specimens are provided with reference points for measuring length change, if any, due to sea water attack.

The technique of collecting samples as lathe turnings from the surface of a cylinder is now being used and appears satisfactory. It still is too early for judging the potential of the dynamic modulus or length changes as measures of sea water attack at elevated temperatures.

VIII.4. Results (Portland Cement Binders)

VIII.4.1. XRD and DTA Tests

The series of pastes made of Type I--0.0 to 37.5 percent fly ash, Type V--0.0 to 37.5 percent fly ash, and cured for 7 days at 210° to 290° F were examined by DTA and XRD.

The results are summarized in Table VIII-3 with exception of a few mixes. αC_2SH was generally not detected and when present it was found in small amount. This phase is not expected to be a problem.

The hydrogarnets appeared in medium amount in mixes of highest fly ash content. Appearance in other samples was eratic and amounts were small to none. Formation of hydrogarnets is not expected to cause difficulty.

Tobermorite when detected was present in poorly crystallized form. High permeability due to presence of well-crystallized tobermorite is not expected, on basis of present results, to be a problem. However, because permeabilities have not yet been correlated with XRD data, the interpretation on XRD data is tentative at this point. Permeability tests will be conducted as required.

VIII.4.2. Strength Results

Strengths of Type I cement-fly ash and Type V cement-fly ash blends cured for 7 days at 250° F are reported in Figure VIII-1. The cements without fly ash gave highest strengths (by small amounts only). Increasing amount of fly ash caused steadily decreasing strengths, but none of the decreases were large. Both cements showed about the same strengths using a W/C ratio of 0.25. The Type I cement with W/C ratio of 0.40 gave higher strengths than corresponding mixes of the Type V cements. These and other results indicate that all fly ash compositions studied would be candidates for tests in saline water.

Table VIII-3

PHASES IN TYPE V-FLY ASH PASTES CURED AT 210° to 290° F FOR 7 DAYS

αC_2SH	Present in small amounts in 0.0 and 7.5 percent fly ash pastes, slightly more at higher temperatures. Amounts not considered harmful to strength.
Hydrogarnets	Most 37.5 percent fly ash pastes contained small to medium amounts apparently independent of temperature of curing. Presence was sporadic most generally only small amounts to none were observed.
Tobermorite	The 30.0 and 37.5 percent of fly ash mixes approach compositions of tobermorite. The diffraction lines of pastes of these mixes were generally weak and diffuse. Well-crystallized tobermorite did not form even in the 290° F cured for 7 days.

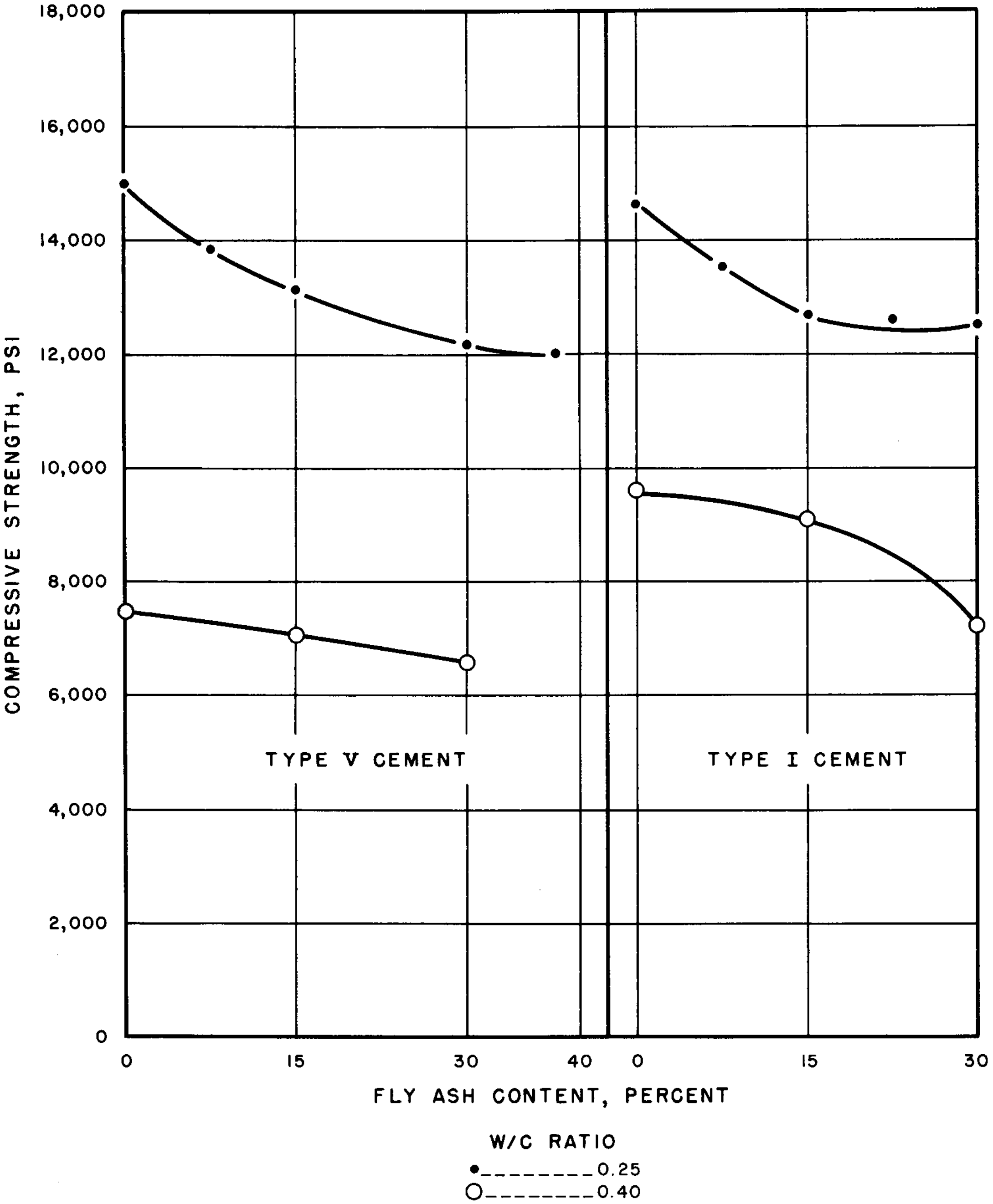

FIGURE VIII - 1: STRENGTH VS FLY ASH CONTENT
TYPES I AND V CEMENTS

Strength data for determination of optimum SO_3 content are given in Figure VIII-2 for Type V, 70 percent Type V: 30 percent fly ash, and 50 percent Type V: 50 percent slag cements. The Type V cement showed the highest optimum SO_3 content at 2.5 percent, the Type V fly ash blend at 1.4 percent and the Type V slag blend at 1.5 percent. The strength decreases are large and abrupt with SO_3 additions in excess of the optimum content.

More detailed information on smaller increments of SO_3 addition will be determined later if required.

VIII.4.3. Discussion of Strength Results

With the exception of mixes containing SO_3 in excess of the optimum, it appears that all compositions studied to date probably have satisfactory strength at 250° F curing. Because of the similarity in XRD and DTA data on all pastes cured at 210° and 290° F to those cured at 250° F, it is expected that results for temperatures of curing other than 250° F would probably also give satisfactory results.

The 50 percent Type V cement: 50 percent slag mix gave the highest strength at its optimum SO_3 content. The XRD results indicated a binder somewhat different from that found in the Type I and Type V cement-fly ash mixes. The significance of this is not yet apparent but will be watched especially in the specimens exposed to saline solutions.

VIII.4.4. Exposure of Selected Compositions in Sea Water

A total of 12 representative compositions each with W/C ratios of 0.30 and 0.50 were autoclaved at 250° F for 7 days then immersed in sea water of 2.1 times normal concentration at 250° F. The specimens are 2- by 4-inch paste cylinders for turning on the lathe as described, and 1- by 1- by 11-1/4-inch mortar bars for length measurements and dynamic modulus determination. A companion set of all specimens is in moist air storage possibly for exposure at 290° F to concentrated sea water.

Two examinations through 28 days of sea water exposures shows no indication of any physical change. Lathe turnings of 0.005-inch depth on two compositions show a pickup of significant amount of $Mg(OH)_2$ as expected. Lathe turnings with progressively increasing depths are expected to show if $Mg(OH)_2$ deposits only in the surface of the specimens with resulting clogging of the pores.

VIII.4.5. New Sulfate Bearing Phase

Examinations of specimens containing different amounts of SO_3 culminated in the discovery of an SO_3 bearing phase previously undetected. This compound is known as ellastadite with composition of $10\ CaO \cdot 3SiO_2 \cdot 3SO_3 \cdot (?)H_2O$. Its presence in the paste in this investigation has been associated with large strength decreases.

Buildup of SO_3 in concrete in excess of the optimum amounts of 1.5-2.5 percent in the cements from sea water would be expected to decrease strength. Ellastadite was synthesized and complete XRD data obtained on it. Using these reference XRD data in a re-examination of all the concrete specimens exposed at temperatures of 103° to 290° F did not reveal presence of ellastadite in any specimen. On basis of these observations, it has been tentatively concluded that some constituent in sea water inhibits formation of this phase in concrete.

VIII.5. Tests and Results on CA Cement

Lumnite prepared as a paste with a W/C ratio of 0.32, and a 50:50 mortar with graded Ottawa sand and W/C ratio of 0.50 is under test. Series of 1-inch cubes of both mixes have been cured at 90°, 110°, 140°, 170°, 200°, 230°, 260°, and 290° F for 7 days, tested for compressive strength and examined for phase content by XRD and DTA. In another series as a sequel to the above tests, both mixes are being cured at 73°, 90°, 100°, 115°, and 130° F for 1, 7, and 28 days and tested as above.

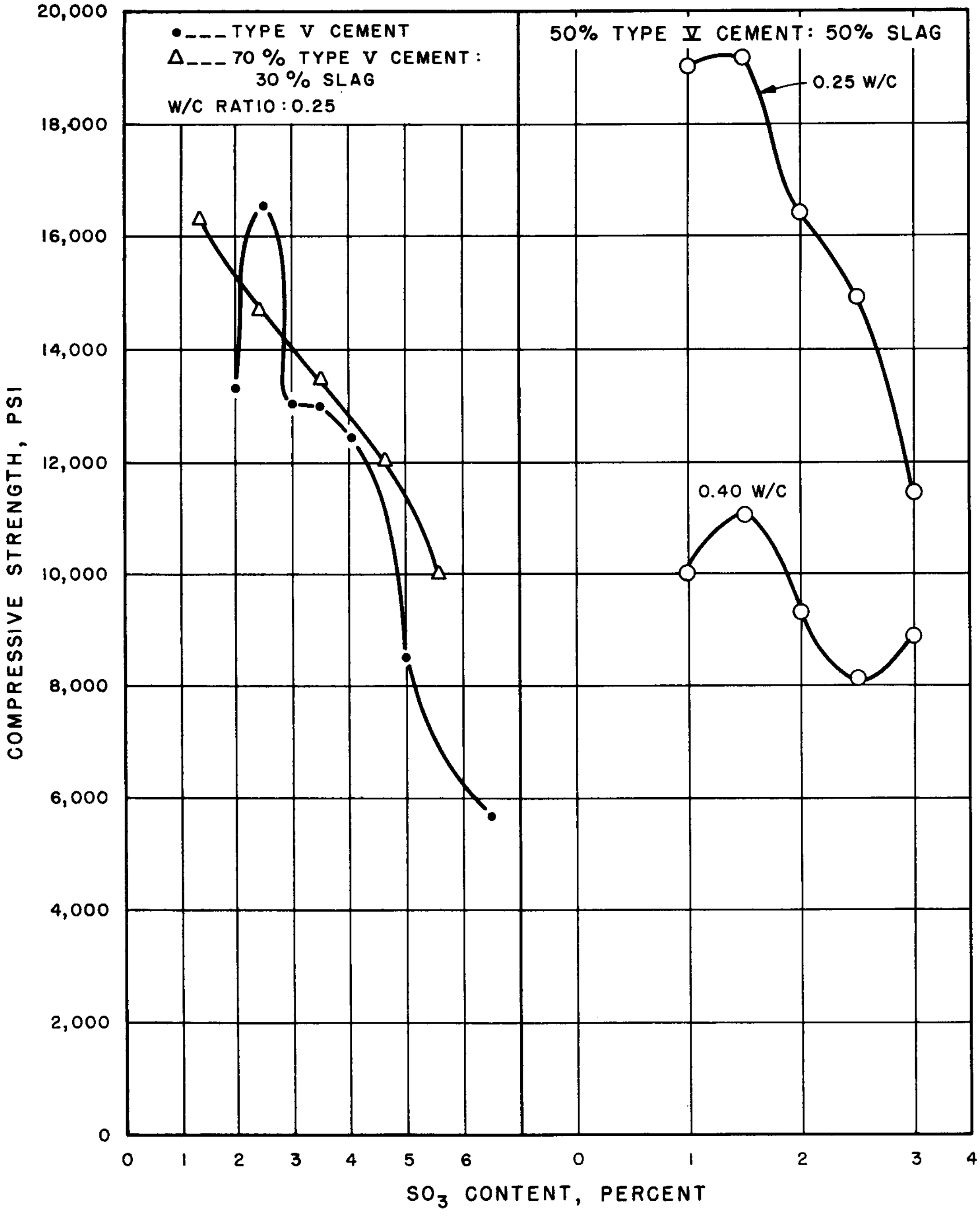

FIGURE VIII - 2: STRENGTH VS SO_3 CONTENT FOR DIFFERENT BLENDS

The 7-day strengths for the 90° to 290° F series are presented in Figure VIII-3. Before interpreting these results, the following description of potential hydration products is given. The hydrate, $CaO \cdot Al_2O_3 \cdot 10H_2O$ (CAH_{10}) is the principal binder. Under certain conditions as will become apparent, the phase $2CaO \cdot Al_2O_3 \cdot 8H_2O$ (C_2AH_8) also forms. Aluminum hydroxide (AH_3) is a normal reaction product and is not detectable by XRD. Its presence can be detected by DTA.

AH_3 recrystallizes to gibbsite (G) a monohydrate $Al_2O_3 \cdot H_2O$ (AH). The recrystallization is accelerated by increasing temperatures. A clear-cut identification of AH_3 and G as separate phases in some of the preparations has not yet been developed. For this reason AH_3/G in following discussions mean aluminum hydroxide and/or gibbsite. Gibbsite, in turn, with increasing temperature transformed to another monohydrate known as boehmite (B). The isometric phase C_3AH_6 especially at higher temperature may contain some Fe_2O_3 and SiO_2 substituted in its lattice. For this reason, the general name hydrogarnet (HG) will be used.

In general, the 7-day paste strength (Figure VIII-3) is about double that of the mortar at temperatures of 90° to 290° F. The highest strength, contrary to expectations occurred at 110° F. In the lower half of Figure VIII-3 are given the relative amounts of the hydration product as average of test result on pastes and mortars. These are qualitative results based on the comparative sizes of the DTA endotherms except for boehmite which was determined by XRD. At 90° F the binder is essentially CAH_{10}. At 110° F the binder is a mixture of CAH_{10} and AH_3/G. Between 90° and 170° F, the CAH_{10} was transformed to AH_3/G and the HG with a maximum of G at 170° F. Between 170° and 230° F, G transformed to B, and B and the HG comprised the binder at temperatures of 230° and 290° F.

With the exception of the high strength at 110° F, the variations in strength over the full temperature range were not particularly large.

In these 7-day strengths (90°-290° F), the presence of the graded Ottawa sand had no apparent effect on the kind or relative amount of phases nucleated. The lower strengths of the mortars are probably due to dilution of the paste by sand compared to the pastes. In the temperature range of 73° to 130° F curing, strengths were obtained at 1, 7, and 28 days. The 1-day strengths are given in Figure VIII-4. The presence of silica sand had a variable effect. At 73° F, the mortar was stronger than the paste and the binder was essentially all CAH_{10}. The binder of the paste consisted of a mixture of C_2AH_8 and CAH_{10}. Lower mortar strengths were obtained at 90° to 130° F compared to 73° F. The paste strength dropped to a low value at 90° F, but increased to a maximum at 100° F. The paste and mortar strengths were about the same at 115° and 130° F, the binder in all specimens was essentially AH_3/AH and C_2AH_8.

The large variations in paste strengths prompted additional tests as Runs No. 2 and 3 (Run No. 1 results are those plotted in Figure VIII-3). The strengths in psi and kinds of binders are as follows:

Run No.	73° F	Binder	90° F	Binder	100° F	Binder
1	10, 950	CAH_{10}	3, 300	C_2AH_8 AH_3/G	12, 500	C_2AH_8 CAH_{10}
2	12, 830	CAH_{10}	2, 330	C_2AH_8 AH_3/G	1, 250	AH_3/G
3	13, 700	CAH_{10}	12, 530	CAH_{10}	1, 000	AH_3/G

The 73° F strengths were consistently high and the binder was CAH_{10}. One test at 90° F and one at 100° F showed high strength, and CAH_{10} as binder. The other strengths at 90° and 100° F were drastically reduced and the binders contained much AH_3/AH. CAH_{10} gives high strength. Low strengths are due to AH_3/AH.

The 7-day strengths of the 73° to 130° F paste and mortar series are given in Figure VIII-5, the 110° F results are from the 90° to 290° F series. The 73° F mortar strength was slightly higher than the paste strength and both contained essentially CAH_{10}. At 7 days, as at 1 day, high strengths were associated with CAH_{10}. Only the 115° and 130° F samples of low strength, had binders consisting essentially of C_2AH_8 and AH_3/AH.

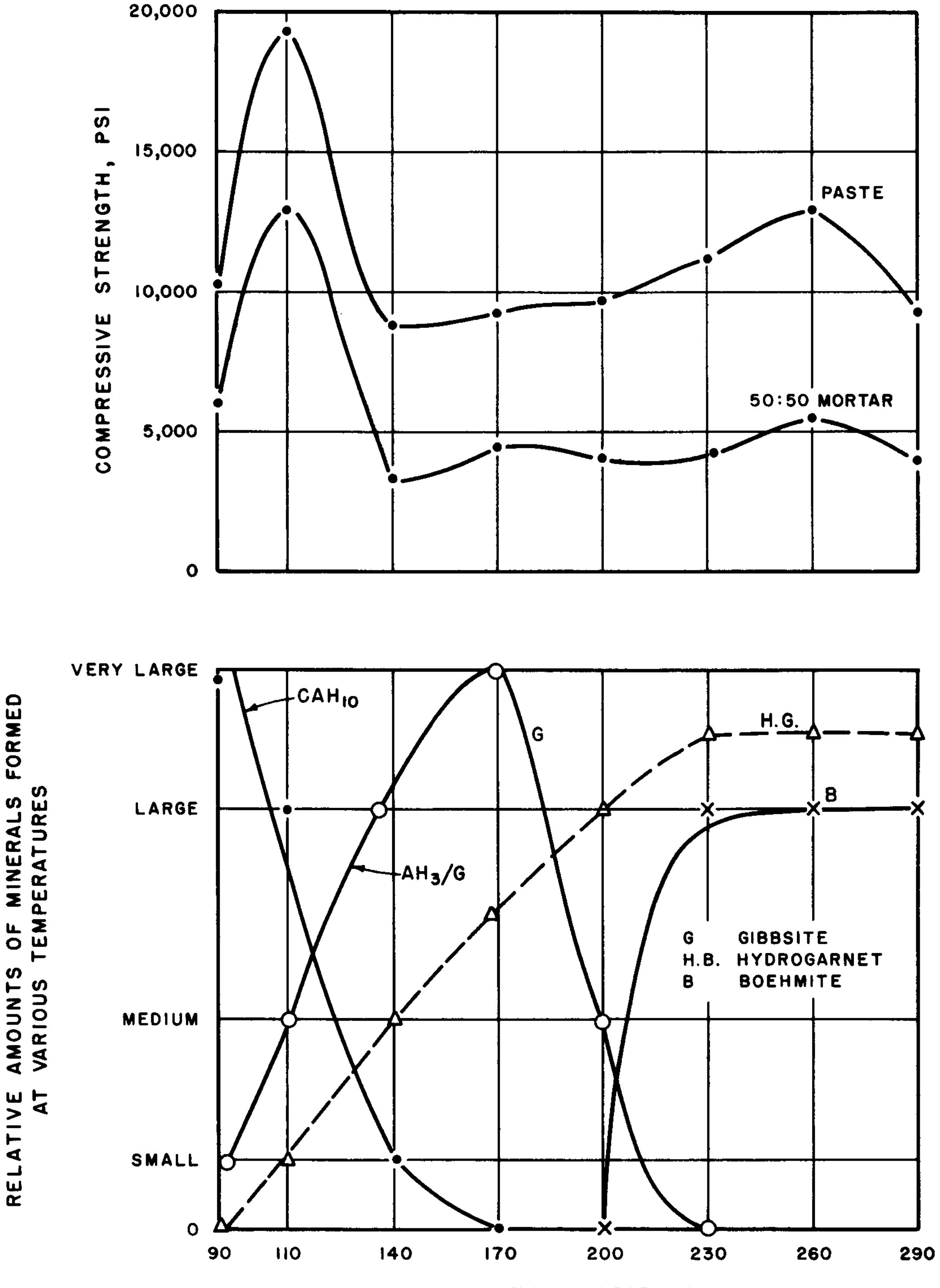

FIGURE VIII – 3: 7 DAY STRENGTH AND PHASE CONTENTS OF CALCIUM ALUMINATE CEMENT PASTE AND MORTAR VS CURING TEMPERATURE

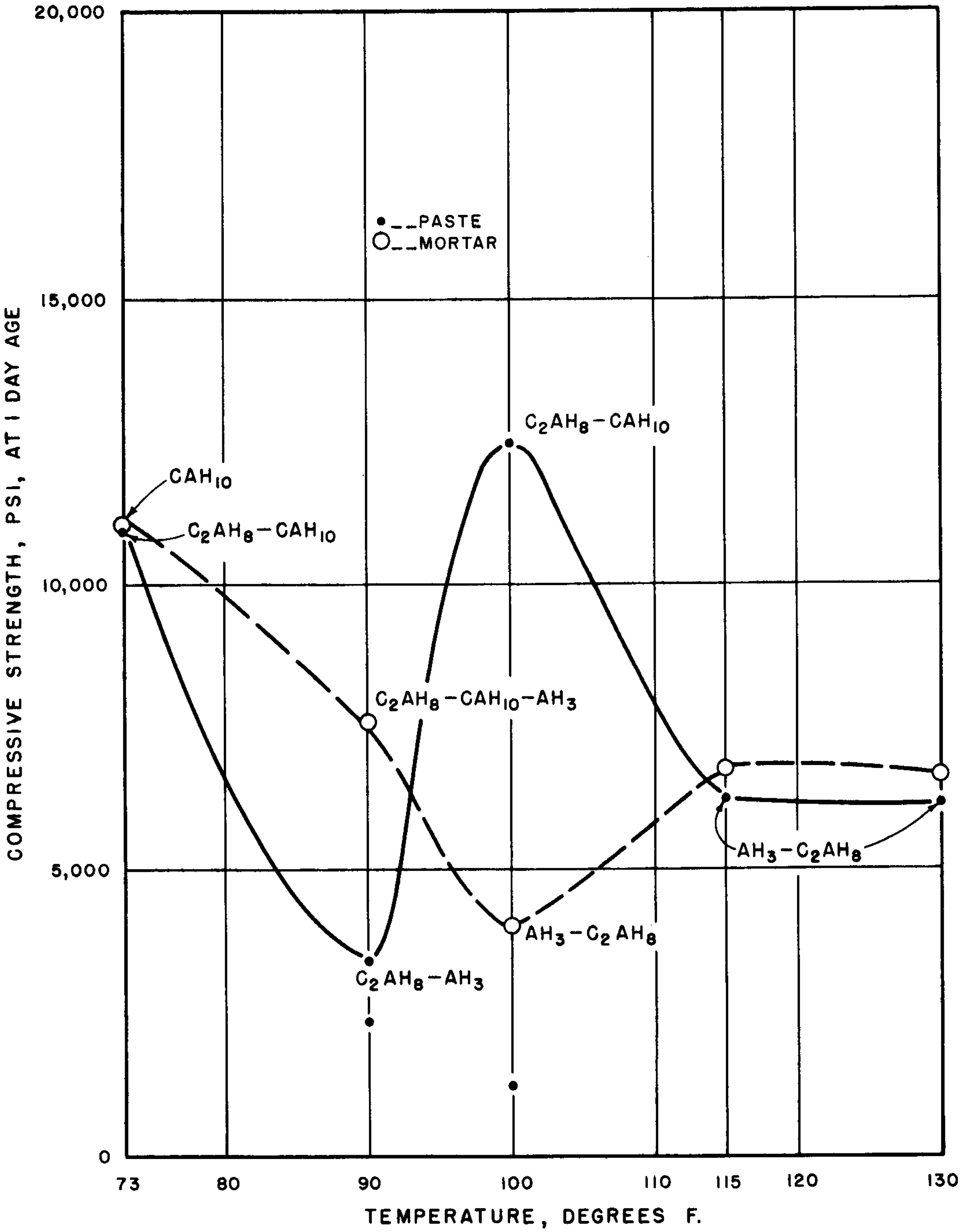

FIGURE VIII – 4: 1 DAY COMPRESSIVE STRENGTH OF CALCIUM ALUMINATE CEMENT PASTE AND MORTAR VS CURING TEMPERATURE

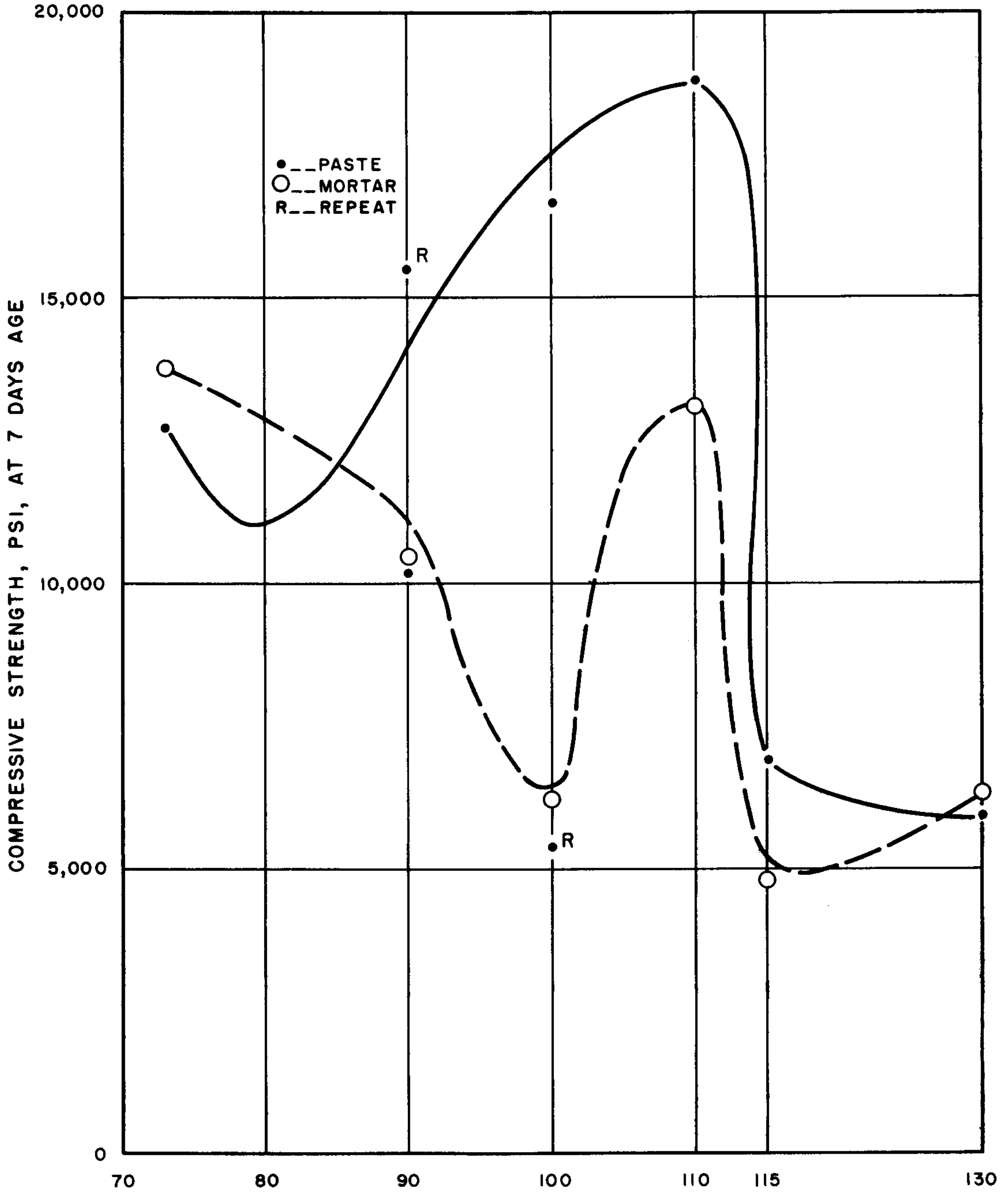

FIGURE VIII – 5: 7 DAY COMPRESSIVE STRENGTH OF CALCIUM ALUMINATE CEMENT PASTE AND MORTAR VS CURING TEMPERATURE

ACKNOWLEDGMENT

Many engineers and technicians in the Division of Research and the Division of Design participated in the work and in the preparation of this report. Major contributors were:

H. J. Cohan	General coordination
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D. L. Matchett	Environmental chamber design
C. E. Selander	Coatings, sealants, and polymeric materials studies
T. E. Backstrom	Reinforcement corrosion studies
E. J. Benton	Stable binder studies
W. M. Batts	Photographic work

CONVERSION FACTORS--BRITISH TO METRIC UNITS OF MEASUREMENT

The following conversion factors adopted by the Bureau of Reclamation are those published by the American Society for Testing and Materials (ASTM Metric Practice Guide, January 1964) except that additional factors (*) commonly used in the Bureau have been added. Further discussion of definitions of quantities and units is given on pages 10-11 of the ASTM Metric Practice Guide.

The metric units and conversion factors adopted by the ASTM are based on the "International System of Units" (designated SI for Systeme International d'Unites), fixed by the International Committee for Weights and Measures; this system is also known as the Giorgi or MKSA (meter-kilogram (mass)-second-ampere) system. This system has been adopted by the International Organization for Standardization in ISO Recommendation R-31.

The metric technical unit of force is the kilogram-force; this is the force which, when applied to a body having a mass of 1 kg, gives it an acceleration of 9.80665 m/sec/sec, the standard acceleration of free fall toward the earth's center for sea level at 45 deg latitude. The metric unit of force in SI units is the newton (N), which is defined as that force which, when applied to a body having a mass of 1 kg, gives it an acceleration of 1 m/sec/sec. These units must be distinguished from the (inconstant) local weight of a body having a mass of 1 kg; that is, the weight of a body is that force with which a body is attracted to the earth and is equal to the mass of a body multiplied by the acceleration due to gravity. However, because it is general practice to use "pound" rather than the technically correct term "pound-force," the term "kilogram" (or derived mass unit) has been used in this guide instead of "kilogram-force" in expressing the conversion factors for forces. The newton unit of force will find increasing use, and is essential in SI units.

Table I

QUANTITIES AND UNITS OF SPACE

Multiply	By	To obtain
	LENGTH	
Mil	25.4 (exactly)	Micron
Inches	25.4 (exactly)	Millimeters
	2.54 (exactly)*	Centimeters
Feet	30.48 (exactly)	Centimeters
	0.3048 (exactly)*	Meters
	0.0003048 (exactly)*	Kilometers
Yards	0.9144 (exactly)	Meters
Miles (statute)	1,609.344 (exactly)*	Meters
	1.609344 (exactly)	Kilometers
	AREA	
Square inches	6.4516 (exactly)	Square centimeters
Square feet	929.03*	Square centimeters
	0.092903	Square meters
Square yards	0.836127	Square meters
Acres	0.40469*	Hectares
	4,046.9*	Square meters
	0.0040469*	Square kilometers
Square miles	2.58999	Square kilometers
	VOLUME	
Cubic inches	16.3871	Cubic centimeters
Cubic feet	0.0283168	Cubic meters
Cubic yards	0.764555	Cubic meters
	CAPACITY	
Fluid ounces (U.S.)	29.5737	Cubic centimeters
	29.5729	Milliliters
Liquid pints (U.S.)	0.473179	Cubic decimeters
	0.473166	Liters
Quarts (U.S.)	946.358*	Cubic centimeters
	0.946331*	Liters
Gallons (U.S.)	3,785.43*	Cubic centimeters
	3.78543	Cubic decimeters
	3.78533	Liters
	0.00378543*	Cubic meters
Gallons (U.K.)	4.54609	Cubic decimeters
	4.54596	Liters
Cubic feet	28.3160	Liters
Cubic yards	764.55*	Liters
Acre-feet	1,233.5*	Cubic meters
	1,233,500*	Liters

Table II

QUANTITIES AND UNITS OF MECHANICS

Multiply	By	To obtain
MASS		
Grains (1/7,000 lb)	64.79891 (exactly)	Milligrams
Troy ounces (480 grains)	31.1035	Grams
Ounces (avdp)	28.3495	Grams
Pounds (avdp)	0.45359237 (exactly)	Kilograms
Short tons (2,000 lb)	907.185	Kilograms
	0.907185	Metric tons
Long tons (2,240 lb)	1,016.05	Kilograms
FORCE/AREA		
Pounds per square inch	0.070307	Kilograms per square centimeter
	0.689476	Newtons per square centimeter
Pounds per square foot	4.88243	Kilograms per square meter
	47.8803	Newtons per square meter
MASS/VOLUME (DENSITY)		
Ounces per cubic inch	1.72999	Grams per cubic centimeter
Pounds per cubic foot	16.0185	Kilograms per cubic meter
	0.0160185	Grams per cubic centimeter
Tons (long) per cubic yard	1.32894	Grams per cubic centimeter
MASS/CAPACITY		
Ounces per gallon (U.S.)	7.4893	Grams per liter
Ounces per gallon (U.K.)	6.2362	Grams per liter
Pounds per gallon (U.S.)	119.829	Grams per liter
Pounds per gallon (U.K.)	99.779	Grams per liter
BENDING MOMENT OR TORQUE		
Inch-pounds	0.011521	Meter-kilograms
	1.12985×10^6	Centimeter-dynes
Foot-pounds	0.138255	Meter-kilograms
	1.35582×10^7	Centimeter-dynes
Foot-pounds per inch	5.4431	Centimeter-kilograms per centimeter
Ounce-inches	72.008	Gram-centimeters
VELOCITY		
Feet per second	30.48 (exactly)	Centimeters per second
	0.3048 (exactly)*	Meters per second
Feet per year	0.965873×10^{-6}*	Centimeters per second
Miles per hour	1.609344 (exactly)	Kilometers per hour
	0.44704 (exactly)	Meters per second
ACCELERATION*		
Feet per second^2	0.3048*	Meters per second^2
FLOW		
Cubic feet per second (second-feet)	0.028317*	Cubic meters per second
Cubic feet per minute	0.4719	Liters per second
Gallons (U.S.) per minute	0.06309	Liters per second
FORCE*		
Pounds	0.453592*	Kilograms
	4.4482*	Newtons
	4.4482×10^{-5}*	Dynes

Multiply	By	To obtain
WORK AND ENERGY*		
British thermal units (Btu)	0.252*	Kilogram calories
	1,055.06	Joules
Btu per pound	2.326 (exactly)	Joules per gram
Foot-pounds	1.35582*	Joules
POWER		
Horsepower	745.700	Watts
Btu per hour	0.293071	Watts
Foot-pounds per second	1.35582	Watts
HEAT TRANSFER		
Btu in./hr ft^2 deg F (k, thermal conductivity)	1.442	Milliwatts/cm deg C
	0.1240	Kg cal/hr m deg C
Btu ft/hr ft^2 deg F	1.4880*	Kg cal m/hr m^2 deg C
Btu/hr ft^2 deg F (C, thermal conductance)	0.568	Milliwatts/cm^2 deg C
	4.882	Kg cal/hr m^2 deg C
Deg F hr ft^2/Btu (R, thermal resistance)	1.761	Deg C cm^2/milliwatt
Btu/lb deg F (c, heat capacity)	4.1868	J/g deg C
Btu/lb deg F	1.000*	Cal/gram deg C
Ft^2/hr (thermal diffusivity)	0.2581	Cm^2/sec
	0.09290*	M^2/hr
WATER VAPOR TRANSMISSION		
Grains/hr ft^2 (water vapor transmission)	16.7	Grams/24 hr m^2
Perms (permeance)	0.659	Metric perms
Perm-inches (permeability)	1.67	Metric perm-centimeters

Table III

OTHER QUANTITIES AND UNITS

Multiply	By	To obtain
Cubic feet per square foot per day (seepage)	304.8*	Liters per square meter per day
Pound-seconds per square foot (viscosity)	4.8824*	Kilogram second per square meter
Square feet per second (viscosity)	0.092903*	Square meters per second
Fahrenheit degrees (change)*	5/9 exactly	Celsius or Kelvin degrees (change)*
Volts per mil	0.03937	Kilovolts per millimeter
Lumens per square foot (foot-candles)	10.764	Lumens per square meter
Ohm-circular mils per foot	0.001662	Ohm-square millimeters per meter
Millicuries per cubic foot	35.3147*	Millicuries per cubic meter
Milliamps per square foot	10.7639*	Milliamps per square meter
Gallons per square yard	4.527219*	Liters per square meter
Pounds per inch	0.17858*	Kilograms per centimeter

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