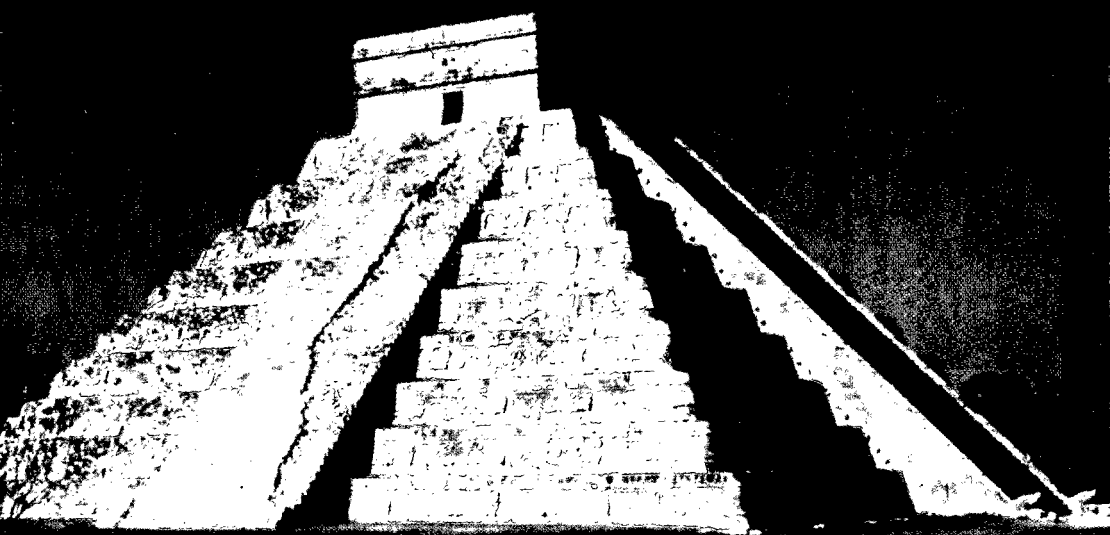


Innovations in Design with Emphasis on Seismic, Wind, and Environmental Loading; Quality Control and Innovations in Materials/ Hot-Weather Concreting



Proceedings, ACI Fifth International Conference
Cancun, Mexico
December 2002

Editor:
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Rising above the grassy plain in the center of Chichén Itzá is the giant Pyramid of Kukulcán constructed 1100-1300 A.D. Early Spanish visitors to the city referred to the pyramid as *El Castillo* (The Castle). Chichén Itzá, the ancient city whose name means “in the mouth at the Itzáe’s Well,” was, in its time of grandeur (between 800 and 1200 A.D.), the center of political, religious and military power in Yucatán.

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Editor

V. M. Malhotra



DISCUSSION of individual papers in this symposium may be submitted in accordance with general requirements of the ACI Publication Policy to ACI headquarters at the address given below. Closing date for submission of discussion is June 2003. All discussion approved by the Technical Activities Committee along with closing remarks by the authors will be published in the September/October 2003 issue of either ACI Structural Journal or ACI Materials Journal depending on the subject emphasis of the individual paper.

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Printed in the United States of America

Editorial production: Bonnie L. Schmick

Library of Congress catalog card number: 2002113749
ISBN: 0-87031-089-5

In December 1991, the American Concrete Institute sponsored an international conference on *Evaluation and Rehabilitation of Concrete Structures and Innovations in Design*. This was in keeping with the mission of the Institute, and the focus on emerging concrete technology in the Pacific Rim region, and to provide technical support for a growing number in ACI international chapters and members. The conference was held in Hong Kong, December 2-6, 1991. More than 80 papers were in the conference proceedings published by ACI as SP-128.

As a follow-up to the very successful Hong Kong conference, the ACI Board of Direction approved a second international conference to be held in 1994. The theme of the conference, held in Singapore, was *High Performance Concrete*. The proceedings of this conference were published as ACI SP-149.

The third international conference in this series was held in 1997 in Kuala Lumpur. The theme of the conference was *High-Performance Concrete, Design and Materials; and Recent Advances in Concrete Technology*. The proceedings of the conference were published as ACI SP-172.

In September 2000, the fourth international conference was held in Seoul, Korea. This conference was titled *Repair, Rehabilitation, and Maintenance of Concrete Structures, and Innovations in Design and Construction*. Conference proceedings were published as ACI SP-193.

The fifth ACI international conference was held in Cancun, Mexico in December 2002. The theme of the conference was *Innovation in Design with Emphasis on Seismic, Wind and Environmental Loading; Quality Control and Innovation in Materials/Hot Weather Concreting*. Sixty-nine papers were reviewed in accordance with ACI review procedures, and 48 accepted for publication as ACI SP-209. Twenty-one were selected for presentation and for publication as supplementary papers.

To all those authors whose submissions could not be included in the conference program, the Institute and the International Conference Executive Steering Committee extends their sincere appreciation for their interest and hard work. Special thanks are offered to members of the ACI Review Panel, who met in December 2001 to review the submissions. Without their prompt review and constructive comments, it would not have been possible to publish ACI SP-209 for distribution at the conference. The cooperation of the authors in accepting reviewers' suggestions and revising their manuscripts accordingly, is greatly appreciated.

The help and assistance of William R. Tolley, ACI Executive Vice President, and his staff in performing the administrative duties associated with this conference are gratefully acknowledged. Particular thanks are extended to Phyllis Erebor, Speaker/Manuscript Liaison Officer, for her dedication to the conference work.

Editor

V. M. Malhotra, Chairman
ACI 2002 Fifth International Conference on
*Innovation in Design with Emphasis on Seismic,
Wind and Environmental Loading; Quality Control
and Innovation in Materials/Hot Weather Concreting.*

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Effect of Curing in a High Temperature Environment on Compressive Strength of Concrete Incorporating Different Complementary Cementitious Materials

by R. Rivera-Villarreal and J. M. Rivera-Torres

Synopsis: This paper provides results of different types of curing in hot weather environment on the compressive strength of concrete made with portland cement and complementary cementitious materials (CCM) such as natural pozzolans, fly ash (FA), granulated blast furnace slag (GBFS), and silica fume (SF). In all concrete mixtures, a superplasticizing admixture (SPA) was used.

Nine series of concrete mixture were made. In seven of them (1, 2, 3, 4, 5, 6 and 9) the normal portland cement (NPC) content was 200 kg/m^3 and in two of them (7 and 8) the same amount of cement was used but it was a portland-natural pozzolan cement (PNPC). The CCM varies from 9.9 to 60.6% of the total cementitious material. The W/C in all series was 0.70 when using NPC or PNPC. The W/C+CM varied from 0.28 to 0.63. In all series the same amount of 1260 kg/m^3 of coarse aggregate was used.

Five different ways of curing were used. One was the initial and final ASTM curing at 23°C up till the age of testing as reference, and four different ways of curing in hot weather environment at 37°C for the first 24h were used. These final curings were: A) ASTM; F) three day and G) seven-day water spray for 15 minutes every 2h; and (E) covered by two layers of membrane.

Adequate compressive strength development (CSD) can be obtained using CCM but very good curing is necessary. Generally, by casting specimens at 37°C and put them under ASTM curing next day at $23\pm 2^\circ\text{C}$ (A), the strength at 28 days was lowered by about 8% and at six months by about 8% lower than these casting at 23°C .

Membrane curing was less effective at later ages mainly when fly ash was used. There exist an optimum amount of fly ash to obtain maximum compressive strength at later ages.

Keywords: compressive strength; final curing; fly ash; high temperature; membrane; portland cement; pozzolans; silica fume; slag; superplasticizer

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INTRODUCTION

In September 2000, the Strategic Development Council of the U. S. concrete industry launched an interesting project for the future of the concrete industry; a Unified Vision for concrete technology and its social impact 30 years from now; it was called Vision 2030. Predicting that by the year 2030 the industry will make processing improvements throughout the life cycle of concrete, and that as a logic consequence of this, concrete will become the most efficient and cost-effective material for construction.

This Vision expects that aggregates quarries, cement manufacturing, ready-mixed and concrete product plants will be viewed as assets to their neighboring communities. And one of the main strides is to use a variety of by-products from other industries as well as recycled concrete as a constituent materials in new concrete production [1].

Environmental issues will have a leading role in the sustainable development of the cement and concrete industry in the 21st century. Increased emission of greenhouse gases to the atmosphere is no longer environmentally and socially acceptable for overall sustainable development. The primary greenhouse gas emissions are carbon dioxide (CO₂) emissions. As the manufacture of portland cement contributes significantly to the CO₂ emissions, it is imperative either to develop new environmentally friendly cement manufacturing technologies, which should be economically acceptable; or to find substitute materials to replace a major part of the portland cement used in the concrete industry. The use of large volumes of fly ash and other CCM in the construction industry contribute in reducing these emissions [2]. These materials are cheaper than portland cement; therefore, the benefits of using these materials are environmental as well as economical.

Water permeability of concrete cover is related to durability and is usually associated with good concrete mixture design and workmanship. Microstructure of NPC is remarkably influenced when it is blended with a very fine ground CCM. The CCM densify the microstructure of hydrated cements, decreasing porosity that influences the overall properties of the resulting material. Good concrete practice should include adequate compaction and suitable curing. Early and long moist curing has been identified as an important requirement for durable concrete [3]. However, such curing requirement is often ignored in the actual construction process mainly when producing high strength concrete (HSC). Various references to the importance of concrete curing in hot weather environments can be found in [4,5,6, & 7].

The appropriate use of a suitable CCM and a HRWRA is expected to produce workable, strong and durable concrete. The pozzolanic CCM will react with calcium hydroxide that is the weakest compound formed and is liberated after the hydration of cement, which ends up in C-S-H hardened compounds and in a refined micro pore structure in concrete. The combined effect of pozzolanic reaction and the very fine particle size of CCM was expected to improve the strength as well as the water permeability of concrete.

Capillary voids and micro cracks, which are generally connected or interlinked, have a great influence on concrete permeability. The amount and size as well as connectivity of capillary voids depend on the W/C and its hydration [8].

The permeability of cement paste has also been related to concrete durability. A great deal of work has been done measuring and predicting permeability of cement paste. One aspect between permeability and pore structure of hardened cement paste was presented by Nyame, et al [9].

Many researches are going on using ternary cementitious materials to get sustainability of concrete materials with fly ash and slag as partial cement replacement materials, looking for good strength development in high strength and non-permeable concrete. One of these studies made with ash of Timber Industrial (TI) included concrete subjected to different curing conditions [10], they obtained good results for concrete grade 80 MPa in the pre-cast concrete industries. Autoclaved curing enhanced the potential of developing early strength.

Several techniques have been applied to study the pore structure of cement paste, such as the measurements of surface area and porosity by using mercury intrusion porosimetry (MIP), Brunaver-Emmett-Teller (BET) analysis, and surface electron microscopy (SEM).

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This study is an attempt to explore the possibility of producing durable and sustainable concrete in hot weather with minimum moist curing, and minimum amount of portland cement.

Materials

Two types of cement were used according the Mexican Standard NMX C414-1999; CPO 30R similar to ASTM C 150 type I portland cement and CPP 30R similar to ASTM C 595-98 blended hydraulic cement type P, but with a higher 28-day-strength-minimum of 30 MPa. The CCM used were: natural pozzolan, FA, GBFS, and condensed SF. The chemical analysis of all cementitious materials used is shown in Table 1 and physical properties in Table 2. The FA used meets ASTM class F requirements, except for the fineness, which is somewhat coarser, passing 45 μm 58% instead of 66% by ASTM.

The aggregates used were crushed limestone from Monterrey, N. L., Mexico area. The fine aggregate had a dry relative density of 2.68 and SSD absorption of 1.5%. The coarse aggregate had a dry relative density of 2.66 and SSD absorption of 0.7% and a 20.0 mm nominal maximum size.

Since in this study the concrete consistency is a factor, it is of great importance that the values measured through the consistency test should not be affected by aggregate grading variations. To avoid this problem and to maintain a uniform grading for each mixture, the fine and coarse aggregate were separated into different size fractions, which were then recombined in the required amount for each batch to provide the selected specific grading. The mean fine and coarse aggregate grading is shown in Table 3.

A sulfonated formaldehyde naphthalene concentrate superplasticizer admixture (SPA) with 1.12 density, was used in all nine series.

Mixture proportions

Nine concrete series (1 to 9) were made using different amounts of CCM as shown in Table 4. In seven of them (1 to 6, and 9) 200 kg/m^3 of NPC was used; and in Series 7 and 8 the same amount of cement was used, but it was NNPC. In all nine series the same amount of water was used to have $W/C = 0.70$. In Table 4 the amount and percentages of each CCM used are shown. In reference Series 1 only NPC was used; in Series 7 only NPPC (22 kg/m^3) was used. In all other seven mixtures using different amounts of CCM, the same amount of SF was used. This amount corresponds to 9.9% of NPC or NPPC.

One of the reasons why all the mixtures have the same amount of water (140 kg/m^3) and 200 kg/m^3 of NPC or NPPC with a $W/C = 0.70$, was to have the same amount of calcium hydroxide produced during cement hydration. The

W/C+CC varies from 0.70 to 0.28 and the yield of cementitious materials from 200 to 507 kg/m³.

In all series, the amount of coarse aggregate was 1260 kg/m³. To maintain the same amount of cement and water when using CCM, it was necessary to adjust the absolute volume of CCM deducting it from the corresponding amount of fine aggregate which in the reference series (without CCM or SPA) was 891 kg/m³.

SPA was used in all nine series, the total amount added was adjusted to give a final slump of 220 ± 20 mm. The mixture proportions are shown in Table 5.

Mixing and Casting

An Elrich concrete mixing machine 45L capacity was used, the drum rotating to 30 rpm and the paddles in countercurrent 450 rpm. The mixing procedure for reference concrete series 1 was as follows: To ensure the correct final proportions in the test batch, just prior to mixing the first batch, the mixer was buttered by mixing a batch proportioned to simulate closely the test batch. The mortar adhering to the mixer after discharging is intended to compensate for loss of mortar from the test batch. The dry aggregates and a part of the mixing water were put into the drum and the machine was started for a half of minute to saturate the aggregate; then, without stopping the machine the cementitious materials (200 kg/m³ of portland cement) were added, including the rest of the water needed to have 0.70 W/C. When all the materials were into the drum, the mixing continued for an additional minute. The machine was stopped for one minute, after that resting period, the mixing continued for one more minute. The machine was stopped and immediately the slump test was performed to measure the initial slump. After that, SPA was added to attain a slump of 220±20 mm, mixing for one more minutes.

The mixing procedure for all other series was similar except that when the remaining water was added, the fourth part of the SPA admixture was included to facilitate the mixing. After the resting period of one minute, the remaining SPA was added without stopping the machine and the mixing continued for one more minute.

Initial slump for the reference mixture was 40-50 mm, and the final slump was 220 ± 20 mm after adding SPA. Except for the reference mixture made at 23±2°C, all mixtures were made at 37°C ± 1. Specimens were casted in molds of 100 mm in diameter and 200 mm in height, three for each compressive strength test at the specified ages of 3, 14, 28 days, 3, 6 months and one year. However, only age 6 months results will be provided in this paper. Additional specimens were made to measure porosity and permeability up to 6 months, but the results are not included in this paper.

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Specimens were stored under five different curing conditions up to the age of testing. Daily information on ambient temperature, humidity, and evaporation was recorded; and the average high and minimum monthly values were determined as shown on Fig. 1.

Fresh Concrete Properties

All concrete series were casted at two different nominal concrete temperatures, 23°C and 37°C, as shown in Table 6, along with the determined air content, laboratory temperature, concrete temperature, and slump.

The dosage of SPA was adjusted in all the mixtures to obtain a final slump of 220 ± 20 mm, as shown in Tables 5 and 7.

Fresh Concrete

In Table 7 we can see that when the concrete temperature increases, so does the dosage of SPA increases as percentage of the Cementitious Material used. A very high increment is registered when no CCM were used, such mixtures are reference series 1 and 7. A higher increment is observed for reference Series 7 in which PNCP was used, this cement has natural pozzolans intergrinded. Portland Cement is the coarser particle size of cementitious materials, but to compensate the absolute volume when CCM is added, as already mentioned in Mixture Proportions, CCM added is finer than the finer aggregate that is substituting. As it can be seen in Table 5 where the % of SPA is about the same to have the same final slump. There is a possibility that the efficiency of a naphthalene based SPA is increased considerably when very fine cementitious materials are used instead of very fine aggregates. Albeit, more research is needed to confirm this possibility.

When the laboratory temperature was maintained at $23^{\circ} \text{C} \pm 2$, the concrete temperature rose up from 30° to 33.5°C ; an average increment of 7.4°C ; when the laboratory temperature was maintained at $37.5^{\circ}\text{C} \pm 0.5$, the concrete temperature rose up from 38° to 40.5°C ; an average increment of 2°C .

Curing

Five different ways of curing were used, it is important to define that we call initial curing those made the first 24 h and final curing after 24 h.

For the reference concrete (Series 1), ASTM C31M-96 curing (R) was used for initial and final curing.

For the rest of the concrete the initial curing was under a temperature of $37^{\circ}\text{C} \pm 1$ and humidity of $60\% \pm 10$, which were the same temperature and

humidity used when casting the concrete specimens, simulating a hot weather environment.

The final curing types (A, F, G, E) were as follows (see Table below):

TYPE OF CURING	INITIAL CURING		FINAL CURING	
R	Dry	23°C	Std ASTM curing room	23°C
A	Dry	37°C	Std ASTM curing 100% RH	23°C
F	Dry	37°C	3 day water spray every 2h	Ambient under shade
G	Dry	37°C	7 day water spray every 2h	Ambient under shade
E	Dry	37°C	Membrane	Ambient under shade

Curing A Standard ASTM (23 and 100% RH after 24 h)

Curing F 3 days spraying specimens with water 15 minutes every 2 h in ambient temperature under shade and no more water in same ambient till the time of test.

Curing G 7 days spraying specimens with water 15 min every 2 h in ambient temperature under shade till the time of test.

Curing E 2 layers of a membrane compound applied to specimens (200 mL/m² per layer) and then ambient temperature under shade till the time of testing. Carefulness was in applying the first layer of membrane to the surface when concrete be saturated but surface dry. To be sure not to lose any amount of water, prior to applying the membrane the specimens were soaked in water and surface dried.

Environmental Conditions

The environmental conditions of daily minimum, mean and highest temperatures, the daily minimum, mean and highest relative humidity (RH) and the daily minimum, mean and highest evaporation were recorded on site of curing. In Fig. 1 average monthly values are shown.

During the six months of environmental curing, May through November the highest temperature varies from 28.5°C to 37.5°C. The minimum relative humidity varies from 32.8% to 53.8% and the maximum evaporation varies from 0.24 L/m²/h to 0.46 L/m²/h.

Compressive Strength

Compressive strength was determined according to ASTM C-39-96 with the following modifications for those specimens under curing F, G and E. Prior to testing of specimens they were soaked in water for two hours trying to have saturated all the specimens at time of test. In the case where membrane was used, it was removed by scraping it prior to putting the specimens in water.

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In Table 7 compressive strength results for all series are shown, as well as type of curing.

RESULTS AND DISCUSSION

In all series, the minimum compressive strength was obtained when using the type F curing which was an intermittent spray with water 15 min. every two hours curing for three days, and then left under shadow up to the age of testing (Fig. 3 thru 11).

In all series, Fig. 3 to 11, the curing type G that was an intermittent spray with water 15 min. curing every two hours for seven days, the compressive strength increased very little compared to type F curing for higher W/C+CM; for lower W/C+CM the difference was about 10%.

In all series, Fig. 3 to 11, the compressive strength with Type E membrane curing was similar to type G curing in that little difference in strength was observed. The membrane curing type E prevented any loss of humidity after applying the first membrane layer.

In series 4, 5, and 6, fly ash was used and the compressive strength data are shown in Fig. 6, 7, and 8. There was a high difference in the way of curing in those having few water as curing F, G and E, the compressive strength is lower than those having more water available as type curing R and A at 28 days and strength increase of about 15% is observed and for later ages as 90 or 180 days, the strength increase is higher about 35%. This means that fly ash concrete needs good water curing for a long time to increase strength development.

The compressive strength development of the nine series of concrete mixtures for each five types of curing is shown in Figures 12, 13, 14, 15 and 16.

In all series, the curing type R, in which the standard ASTM curing were used all the time attained maximum compressive strength at ages 28 days and higher, except Series 6 in which was used the maximum CCM of 307 kg/m³ as shown in Fig. 8. In this case, casting was at 37°C and being for 24h at this temperature following with curing soaked in water curing at 23°C (Type A curing).

Series 3 having less than 50% of cementitious materials than Series 5 has higher early strength and about the same late strength (Fig. 17 to 21). The difference is that Series 3 has 27.7% GBFS and series 5 has 53.0% fly ash. Both series have same amount of portland cement SF and water. The effectiveness of GBFS was better than fly ash used.

Series 4 and 5 (see Fig. 6 and 7) have the same amount of portland cement, SF, water and very little amount of SPA. The main difference is that the amount of fly ash in Series 4 had 200 kg/m^2 and Series 5 had 250 kg/cm^3 , a difference of 50 kg/cm^3 of more fly ash, but the compressive strength of Series 4 has about the same strength. This means that there exists an optimum amount of fly ash for maximum strength.

In series 2 to 6 the same amount of PC (200 kg/m^3), the same amount of water (140 kg/m^3), the same amount of SF (22 kg/m^3) and same slump ($220 \pm 20 \text{ mm}$) were maintained. The superplasticizer dosage varied with ambient temperature from 1.22 to 1.35% of solids of total cementitious materials for 23°C and 1.30 to for 37°C . Series 2 and 6 have the same amount of PC and SF and same initial hot curing (37°C) but different W/C+CM, Series 2, 0.63 and Series 6, 0.28, the last one has quaternary cementitious materials in which we added furthermore FA and GBFS. For the same curing A, Series 2 (Fig. 4) the compressive strength at 6 months was 44.5 MPa and in Series 6 (Fig. 8) the compressive strength was 94.8 MPa, an increase of 213%.

The reference Series 1 (Fig. 3) with portland cement CPC 30R similar to ASTM type 1 and Series 7 (Fig. 9) with PNPC without any CCM but with SPA for reference R curing. The Series 7 with PNPC give more strength of 11.1% at 28 days and 22.1% at six months.

The Series 2 (Fig. 4) with portland cement and Series 8 (Fig. 10) with same amount of PNPC and same amount of SF and SPA have no difference for reference curing R in compressive strength at 28 days, but at six months Series 8 is higher 5.0%.

The Series 3 and Series 9 have same amount of portland cement and same amount of SF and using SPA. The only difference was that series 3 has 85 kg/m^3 of GBFS, and series 9 has 85 kg/m^3 of NP at 28 days series 9 with reference curing R had a 7.0% higher compressive strength and at six months 10.0%.

CONCLUSIONS

In hot weather environments usually the specimens are made and left at the site for the first 24 h. After that, they are cured under $>95\%$ humidity and $23 \pm 2^\circ\text{C}$ temperature, which is similar to curing type A. In this type of curing against type R curing, which is ASTM curing (casted at 23°C), under the conditions established in this study and for any amount of cementitious materials, the compressive strength at 28 days was lower for a mean value of 10.0% at later ages about the same 12.0%, except for Series 3 and 6, where GBFS was used, in Series 3 it was of only about 5.0% lower at 28 days and 13.0% at later ages, but in Series 6, besides using the same amount of GBFS,

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but a great amount of total cementitious material as about 60.0% by volume, which lowers W/C+CM to 0.28 at 28 days, practically is the same for both types of curing. For later ages as 180 days, the strength level was higher about 10.0% for curing A.

Two layers of good applied membrane were not as good as continuous water curing mainly when fly ash was used as in series 4, 5, and 6 when a 20% less compressive strength at 28 days and about 45.0% at six months of concrete casting at 37°C were obtained.

Intermittent water at 15.0 min for every 2h for 7 days is not enough to gain strength development as continued water curing. It will be similar to two layers of membrane.

Some GBFS, as used in this study, was more effective in developing compressive strength. Series 3 had an additional 85 kg/m³ of GBFS and Series 5 had an additional 250 kg/m³ of fly ash and other materials the same, even the higher amount of fly ash the compressive strength is around the same for any age and type of curing, with less amount of water as curing F, G, and E. For curing with water the Series 5 with fly ash have 12.0% higher at 28 days and about 27.0% at six months.

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ACKNOWLEDGEMENT

The authors would like to thank Cemex for supplying the portland cement and natural pozzolans used and Acon, S. A. de C.V. for supplying the admixtures.

The authors also want to thank students Robert Montalvo, Sergio Narváez, David Reyes and Iliana Garza for word processing and preparation of illustrations.

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Table 1. Chemical analysis of cementitious materials

Chemicals %	NMX		Nat Pozz	Fly Ash	GBFS
	CPC 30R	CPP 30R			
Silicon Dioxide (Si O ₂)	16.45	25.94	68.50	60.4	34.50
Aluminum Oxide (Al ₂ O ₃)	4.84	6.63	14.00	27.8	10.10
Ferrous Oxide (Fe ₂ O ₃)	1.67	2.97	2.50	3.35	1.00
Calcium Oxide (Ca O)	64.73	55.25	0.40	1.85	36.30
Magnesium Oxide (Mg O)	1.47	1.29	0.10	0.65	9.70
Sulfuric Oxide (SO ₃)	3.62	2.78	0.00	0.90	3.98
Sodium Oxide (Na ₂ O)	0.53	0.71	3.44	1.00	0.62
Potassium Oxide (K ₂ O)	0.97	0.56	5.50	0.80	0.80
Titanium Oxide (Ti O ₂)	0.00	0.32	0.16	0.95	1.31
Phosphorus Oxide (P ₂ O ₅)	0.09	0.00	0.25	0.25	0.04
Manganese Oxide (Mn ₂ O ₃)	0.00	0.00	0.00	0.00	1.29
Loss of ignition LOI	1.53	3.00	4.90	1.55	0.10
Total Alkalis (as Na ₂ O)	0.71	1.08	7.50	1.52	1.14

Table 2. Physical properties of cementitious materials

Properties	Type CPC 30R	Type CPP 30R	Natural Pozzolan	Fly Ash	GBFS	SF
Fineness Passing 45µm Blaine	89.60	94.58	94.3	58.0	98.9	100
Density	3.14	2.95	2.36	2.38	2.88	2.42
Time of setting						
Vicat test						
Initial set	94	95				
Final set	132	202				
Gillmore test						
Initial set	105	106				
Final set	143	179				
Compressive strength of 50 mm cubes MPa						
3 d	24.3	23.7				
7 d	27.9	31.0				
28 d	32.1	40.0				
Autoclave Expansion %	.0525	-0.018				
Strength activity index with CPC 30R %						
7 d			75.3	76.1	75.3	92.1
28 d			80.0	77.3	76.0	--

Table 3. Aggregate grading

Sieve size	Coarse, % passing Maximum size 19 mm (3/4")	Sieve size	Coarse, % passing
1 1/2"	---	#4	100
1"	100	#8	83
3/4"	92	#16	53
1/2"	---	#30	30
3/8"	39	#50	18
#4	3.4	#100	12
#8	0.7		

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Table 4. Amount of complementary cementitious materials in each concrete series

SERIES	FA		GBFS		NP		SF		TOTAL CCM		TOT *	W/C+CM
	kg/m ³	%	kg/m ³	%	kg/m ³	%	kg/m ³	%	kg/m ³	%	kg/m ³	
1	0	0	0	0	0	0	0	0	0	0	200	0.7
2	0	0	0	0	0	0	22	9.9	22	9.9	222	0.63
3	0	0	85	27.7	0	0	22	7.2	107	34.9	307	0.46
4	200	47.4	0	0	0	0	22	5.2	222	52.6	422	0.33
5	250	53.0	0	0	0	0	22	4.7	272	57.6	472	0.30
6	200	39.4	85	16.8	0	0	22	4.3	307	60.6	507	0.28
7	0	0	0	0	0	0	0	0	0	0	200	0.70
8	0	0	0	0	0	0	22	9.9	22	9.9	222	0.63
9	0	0	0	0	85	27.7	22	7.2	107	34.9	307	0.46

All Series had a W/C= 0.70

* Total of cementitious materials.

All mixtures were made using 200 kg/m³ of Portland cement except series 7 and 8, which were made using pozzolanic cement with aprox. 18% natural pozzolan.

Table 5. Mixture proportions for all 9 series

Series	Total Cementitious kg/m ³	W/C	W/ (C+CCM)	Aggregate kg/m ³		SPA solids 23°C		SPA solids 37°C	
				Coarse	Fine	kg/m ³	%*	kg/m ³	%*
1	200	0.7	0.70	1260	891	2.50	1.25	3.00	1.50
2	222	0.7	0.63	1260	861	3.00	1.35	3.47	1.57
3	307	0.7	0.46	1260	784	3.92	1.28	3.99	1.30
4	422	0.7	0.33	1260	638	5.60	1.33	5.68	1.35
5	472	0.7	0.30	1260	582	5.76	1.22	6.12	1.30
6	507	0.7	0.28	1260	558	6.44	1.27	7.00	1.38
7	200	0.7	0.70	1260	891	2.68	1.34	3.45	1.72
8	222	0.7	0.63	1260	861	3.48	1.57	3.68	1.76
9	307	0.7	0.46	1260	784	4.48	1.46	4.79	1.56

* % of total cementitious material.

Table 6. Fresh concrete properties at casting nominal temperature of 23°C and 37°C

Series	23°C				37°C			
	Air Content	Lab temp.	Concrete temp.	Slump	Air content	Lab Temp.	Concrete temp	Slump
	%	° C	° C	mm	%	° C	° C	mm
1	0.6	25.0	31.0	220	1.1	37.5	38.0	200
2	1.0	25.0	31.0	210	0.6	37.0	38.5	200
3	1.5	24.5	33.5	200	0.8	37.0	39.0	230
4	1.0	25.0	33.5	240	1.8	37.0	40.0	220
5	2.1	24.5	30.0	220	2.1	36.5	38.0	210
6	1.9	25.0	34.5	240	2.7	37.0	40.5	220
7	1.0	24.5	32.5	200	1.2	37.0	39.5	200
8	1.0	24.5	32.0	200	0.9	36.5	38.0	210
9	0.7	23.5	30.0	210	0.7	36.5	38.5	230

Table 7. Amount of SPA for all series at 23°C and 37°C in % of all cementitious materials

SERIES	23°C	37°C	DIFFERENCE	INCREMENT
1	1.25	1.50	0.25	20.0
2	1.35	1.57	0.22	16.3
3	1.28	1.30	0.02	1.6
4	1.33	1.35	0.02	1.5
5	1.22	1.30	0.08	6.6
6	1.27	1.38	0.11	8.7
7	1.34	1.72	0.38	28.4
8	1.57	1.76	0.19	12.1
9	1.46	1.56	0.10	6.8

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Table 8. Compressive strength for all series and type of curing

Series	Curing Type	CCM kg/m ³	CCM %	Lab Temp °C	Compressive Strength (MPa)				
					3	7	28	90	180
1	R	0	0	25.0	21.9	24.2	30.6	33.0	35.7
2		22	9.9	25.0	25.2	40.4	45.4	48.3	52.1
3		107	34.9	24.5	45.5	51.1	56.2	60.3	64.2
4		222	52.6	25.0	32.0	54.3	66.7	77.6	83.1
5		272	57.6	24.5	35.7	50.8	64.0	72.8	80.4
6		307	60.6	25.0	45.8	59.7	74.0	81.7	86.5
7		0	0	24.5	19.9	24.0	34.0	39.8	43.6
8		22	9.9	24.5	25.0	36.9	45.9	50.0	54.8
9		107	34.9	23.5	30.6	44.9	60.1	65.7	69.8
1	A	0	0	37.5	18.9	22.6	26.3	29.4	32.3
2		22	9.9	37	30.4	34.7	40.6	41.2	44.5
3		107	34.9	37	40.0	50.0	53.7	58.1	56.7
4		222	52.6	37	34.0	50.6	60.8	73.4	76.3
5		272	57.6	36.5	33.1	47.5	60.2	65.6	71.8
6		307	60.6	37	48.7	61.0	75.6	89.3	94.8
7		0	0	37	19.4	23.2	32.6	38.2	40.0
8		22	9.9	36	24.6	35.7	45.0	48.2	50.5
9		107	34.9	36.5	30.0	41.1	52.7	60.8	64.1
1	F	0	0	37.5	18.0	19.1	21.3	28.3	29.5
2		22	9.9	37	27.6	36.3	35.0	39.3	40.3
3		107	34.9	37	39.8	43.1	45.6	47.5	46.5
4		222	52.6	37	34.3	49.7	52.2	52.1	54.3
5		272	57.6	36.5	33.1	43.3	52.8	52.1	52.4
6		307	60.6	37	47.9	51.7	64.3	61.1	57.0
7		0	0	37	17.7	19.7	20.0	21.9	25.1
8		22	9.9	36	21.6	29.3	33.7	33.6	33.9
9		107	34.9	36.5	25.9	35.1	41.5	39.3	39.4
1	G	0	0	37.5	20.2	21.1	23.5	28.3	28.3
2		22	9.9	37	27.9	34.8	37.7	37.5	39.6
3		107	34.9	37	38.6	45.0	49.7	47.8	50.2
4		222	52.6	37	34.2	47.1	55.5	56.8	56.4
5		272	57.6	36.5	34.2	42.8	54.4	55.1	56.3
6		307	60.6	37	46.9	57.3	69.6	67.1	56.8
7		0	0	37	17.8	22.4	25.0	28.0	29.6
8		22	9.9	36	21.3	29.4	37.6	37.1	37.5
9		107	34.9	36.5	25.5	37.0	42.3	48.7	48.2

Table 8. Compressive strength for all series and type of curing (cont.)

Series	Curing Type	CCM kg/m ³	CCM %	Lab Temp °C	Compressive Strength (MPa)				
					3	7	28	90	180
1	E	0	0	37.5	17.7	20.1	21.8	28.3	28.1
2		22	9.9	37	21.7	38.4	39.5	38.8	41.8
3		107	34.9	37	44.7	48.5	51.6	51.8	52.4
4		222	52.6	37	41.5	50.9	54.7	57.8	56.9
5		272	57.6	36.5	39.9	44.4	49.8	53.5	53.8
6		307	60.6	37	56.4	64.7	69.6	71.5	67.9
7		0	0	37	18.4	27.0	28.6	27.4	27.1
8		22	9.9	36	29.7	34.9	40.3	44.9	43.6
9		107	34.9	36.5	31.7	40.0	51.6	51.7	53.3

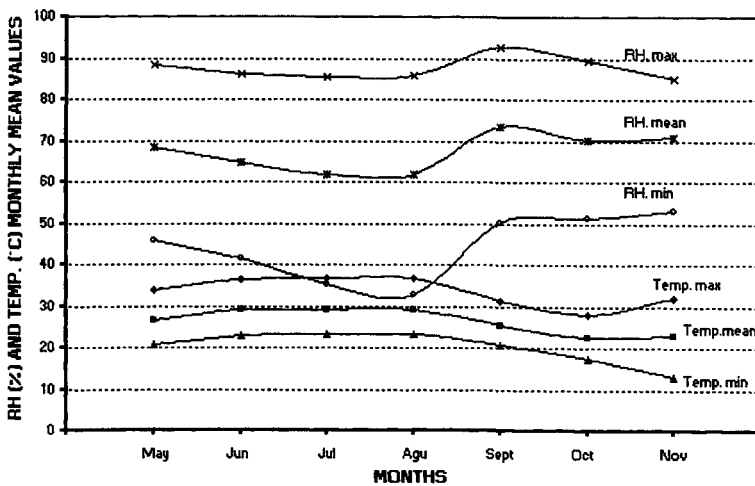


FIG. 1. MONTHLY AVERAGES OF DAILY OBSERVATIONS OF AMBIENT TEMPERATURE AND RELATIVE HUMIDITY AT UNIVERSIDAD AUTONOMA DE NUEVO LEON CAMPUS YEAR 2001.

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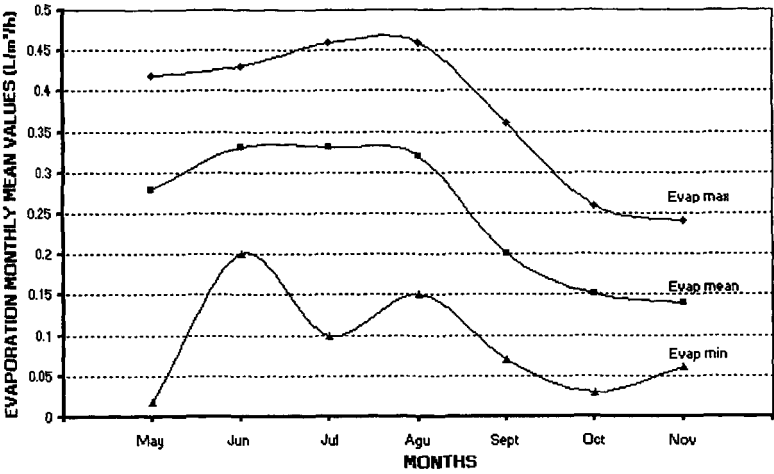


FIG. 2. MONTHLY AVERAGES OF DAILY OBSERVATIONS OF EVAPORATION AT UNIVERSIDAD AUTONOMA DE NUEVO LEON CAMPUS YEAR 2001

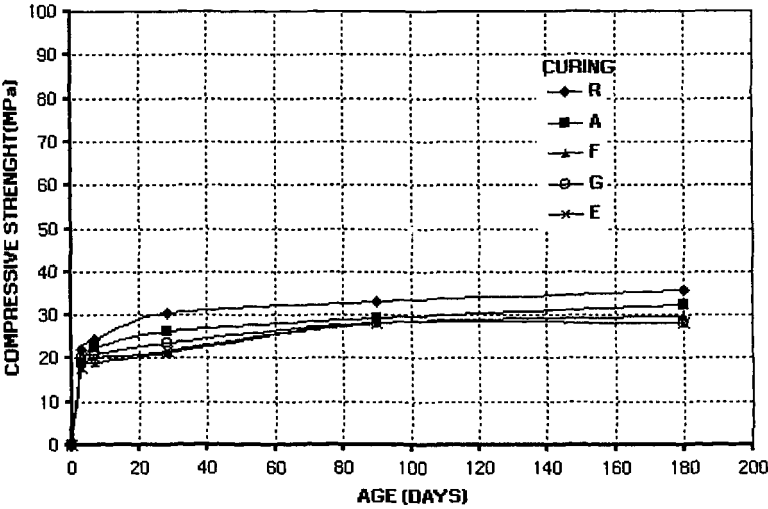


FIG. 3. COMPRESSIVE STRENGTH DEVELOPMENT FOR SERIES 1 WITH THE FIVE TYPES OF CURING. REFERENCE CONCRETE WITH PORTLAND CEMENT.

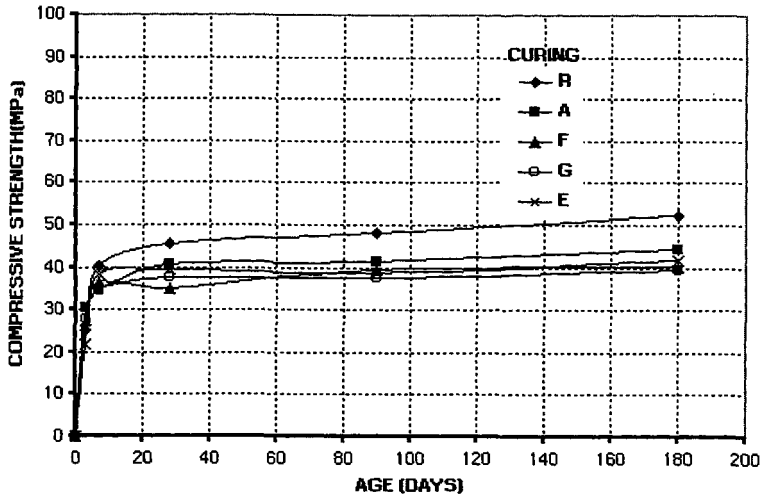


FIG. 4. STRENGTH DEVELOPMENT FOR SERIES 2 WITH THE FIVE TYPES OF CURING USING PORTLAND CEMENT AND SF.

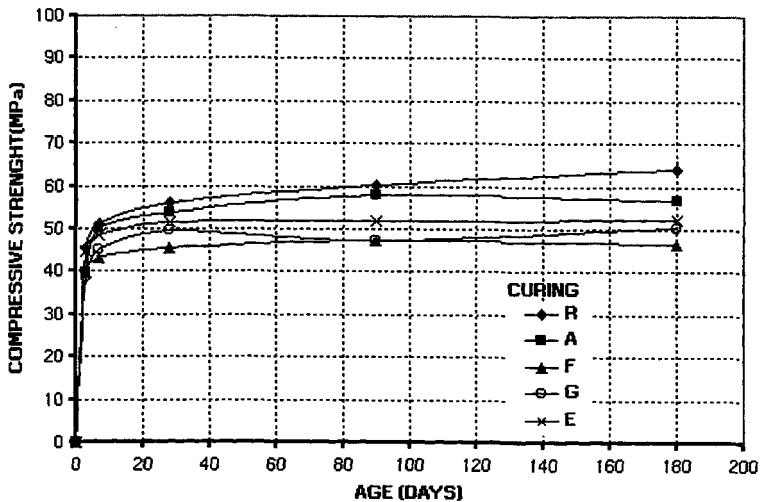


FIG. 5. STRENGTH DEVELOPMENT FOR SERIES 3 WITH THE FIVE TYPES OF CURING USING TERNARY CEMENTITIOUS MATERIALS: PORTLAND CEMENT 65.1%, GBFS 27.7% AND SF 7.2%.

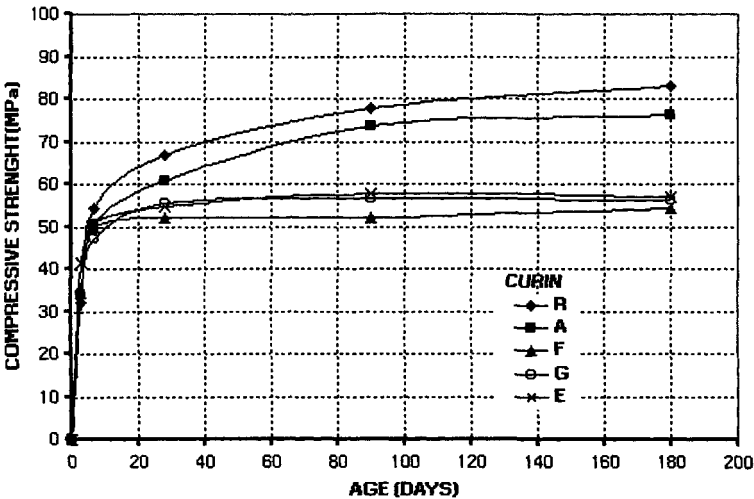


FIG. 6. STRENGTH DEVELOPMENT FOR SERIES 4 WITH THE FIVE TYPES OF CURING, USING TERNARY CEMENTITIOUS MATERIALS: PORTLAND CEMENT 47.4%, FLY ASH 47.4% AND SF 5.2%.

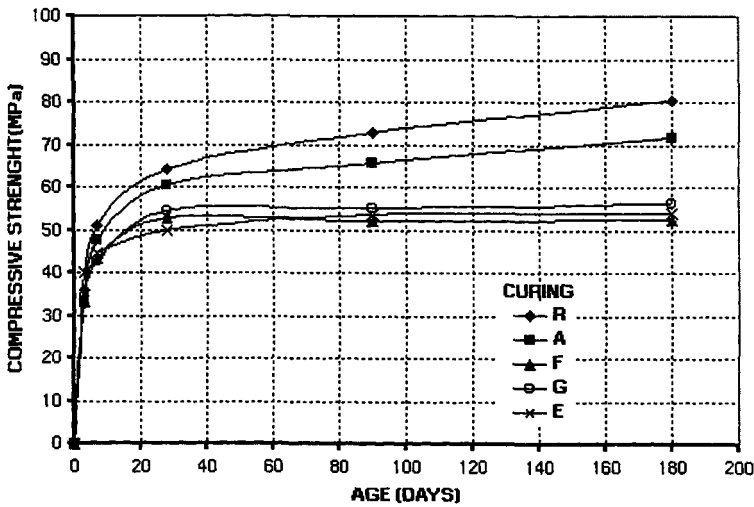


FIG. 7. STRENGTH DEVELOPMENT FOR SERIES 5 WITH THE FIVE TYPES OF CURING, USING TERNARY CEMENTITIOUS MATERIALS: PORTLAND CEMENT 57.7%, FLY ASH 53.0% AND SF 4.7%.

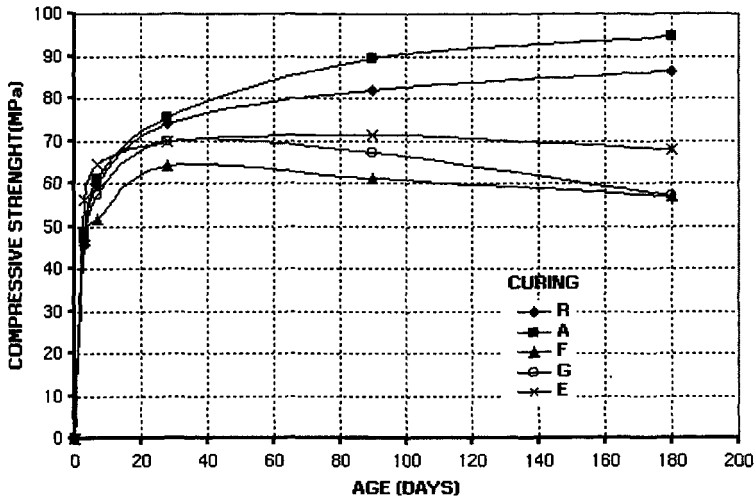


FIG. 8. STRENGTH DEVELOPMENT FOR SERIES 6 WITH THE FIVE TYPES OF CURING, USING QUATERNARY CEMENTITIOUS MATERIALS: PORTLAND CEMENT 39.5%, FLY ASH 39.4%, GBFS 16.8%, AND SF 4.3%.

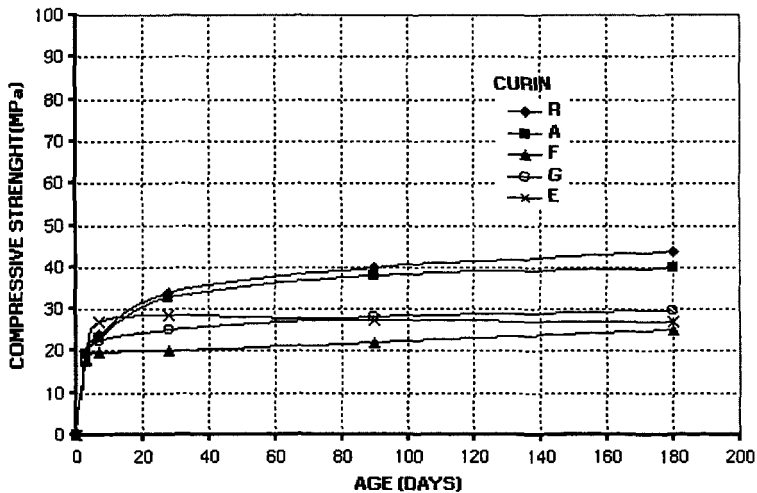


FIG. 9. STRENGTH DEVELOPMENT FOR SERIES 7 WITH THE FIVE TYPES OF CURING, REFERENCE CONCRETE WITH PNPC PRODUCED IN THE CEMENT FACTORY AND SPA, NO CCM WAS USED.

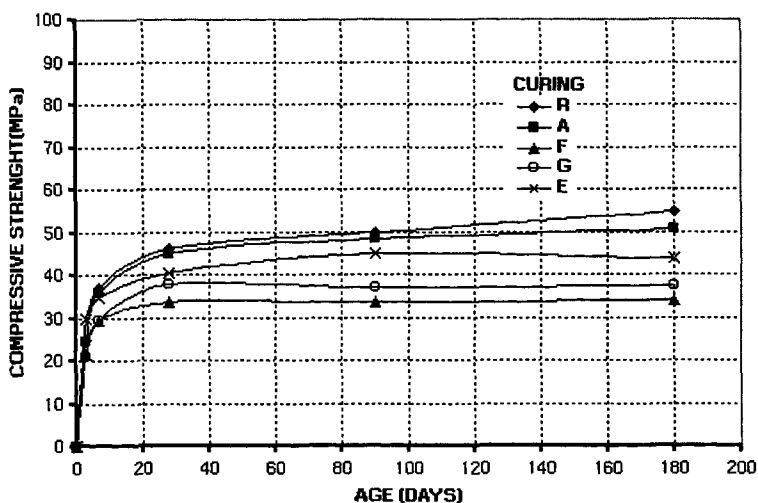


FIG. 10. STRENGTH DEVELOPMENT FOR SERIES 8 WITH THE FIVE TYPES OF CURING. SAME AS SERIES 7 (FIG. 9) BUT INCLUDING SF 9.9%.

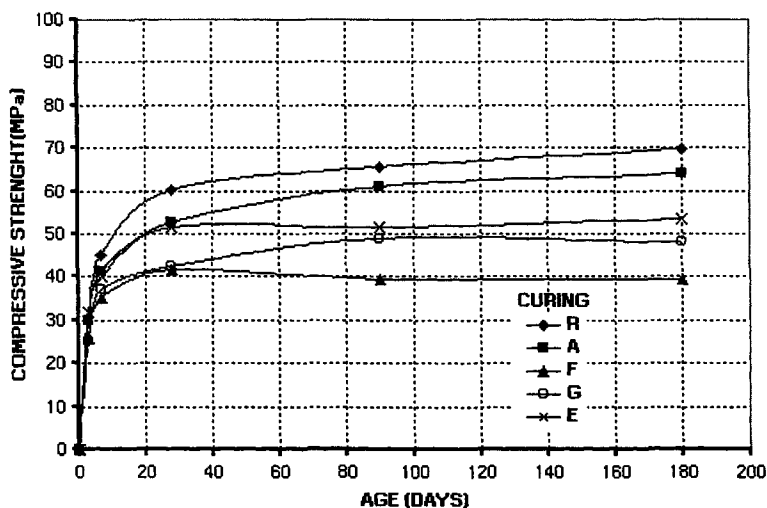


FIG. 11. STRENGTH DEVELOPMENT FOR SERIES 9 WITH THE FIVE TYPES OF CURING USING TERNARY CEMENTITIOUS MATERIALS: PORTLAND CEMENT 65.1%, NATURAL POZZOLAN 27.7%, AND SF 7.2%.

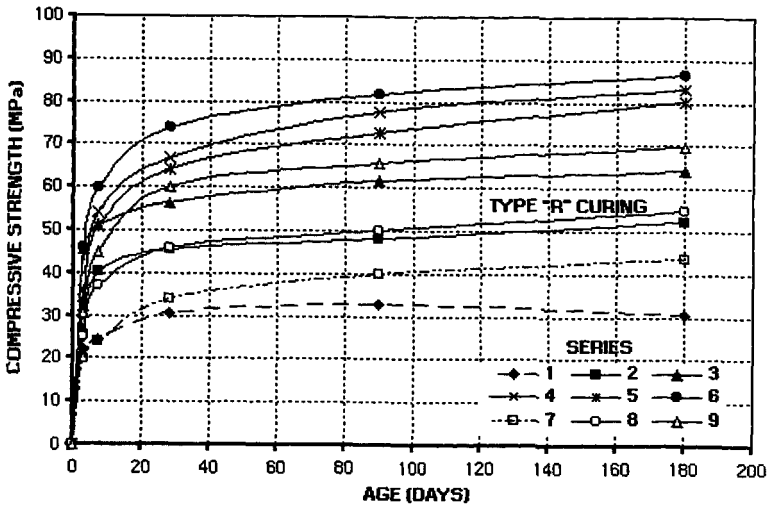


FIG. 12. COMPRESSIVE STRENGTH DEVELOPMENT FOR THE NINE SERIES USING ASTM STANDARD CURING "R". CONCRETE MADE AT 23°C.

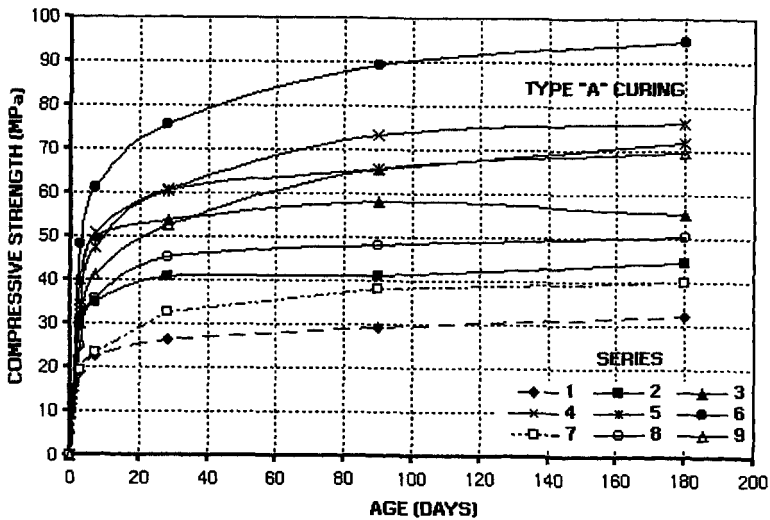


FIG. 13. COMPRESSIVE STRENGTH DEVELOPMENT FOR THE NINE SERIES FOR CURING TYPE "A". CONCRETE MADE AT 37°C AND MAINTAINED THIS TEMPERATURE FOR 24h.

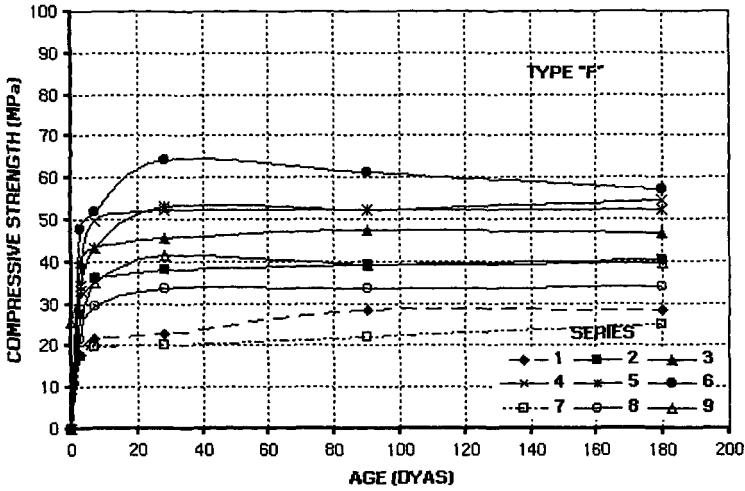


FIG. 14. COMPRESSIVE STRENGTH DEVELOPMENT FOR THE NINE SERIES FOR CURING TYPE "F". 24 h AT 37°C, NEXT 3 DAYS WATER SPRAY EVERY 2 h AND AFTER UNDER SHADE TO TIME OF TESTING.

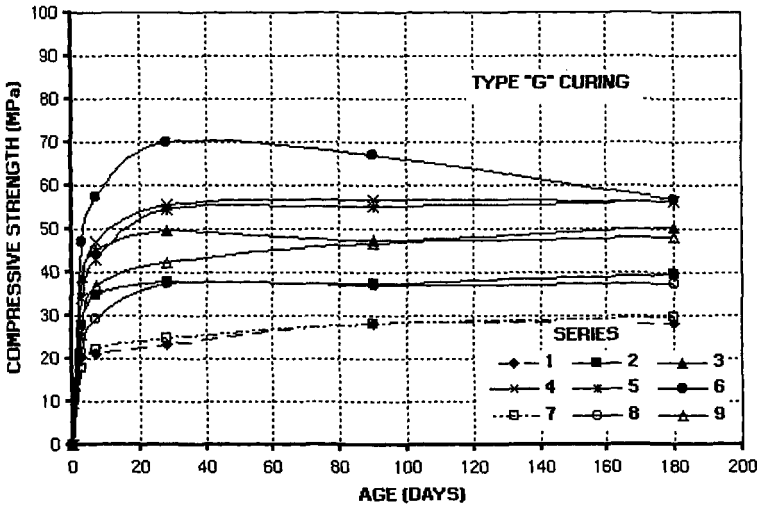


FIG. 15. COMPRESSIVE STRENGTH FOR THE NINE SERIES FOR CURING TYPE "G". 24 h AT 37°C, NEXT 7 DAYS WATER SPRAY EVERY 2 h AND AFTER UNDER SHADE TO TIME OF TESTING.

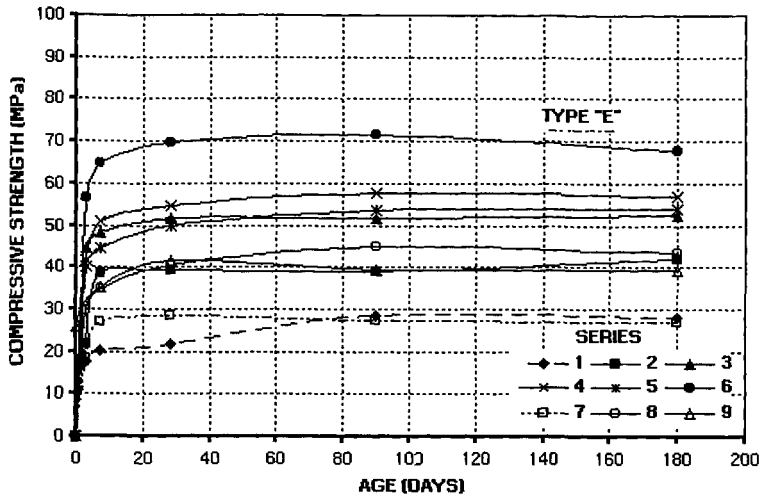


FIG. 16. COMPRESSIVE STRENGTH DEVELOPMENT FOR THE NINE SERIES FOR CURING TYPE E. 24 h AT 37°C, AFTER TWO MEMBRANE LAYERS WAS APPLICATED AND THEN UNDER SHADE TO TIME OF TESTING.

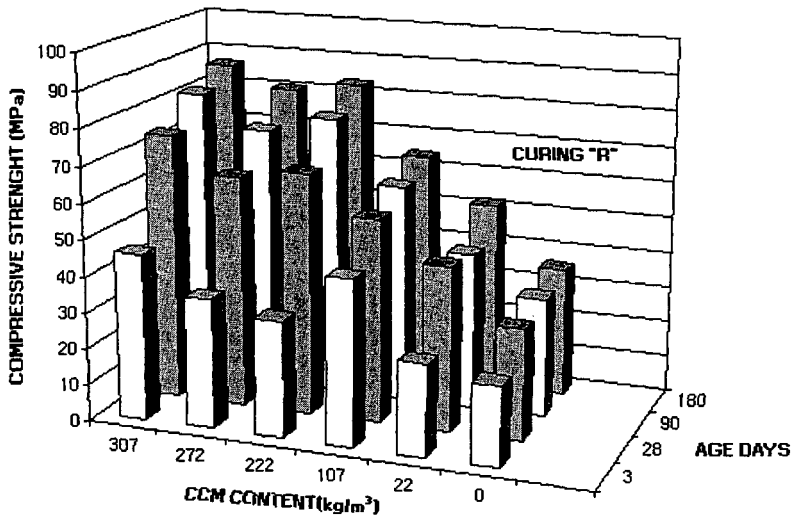


FIG.17. SERIES 1 TO 6 SHOWING THE CONTENT OF CCM EXCEPT PNPC , FOR CURING "R"

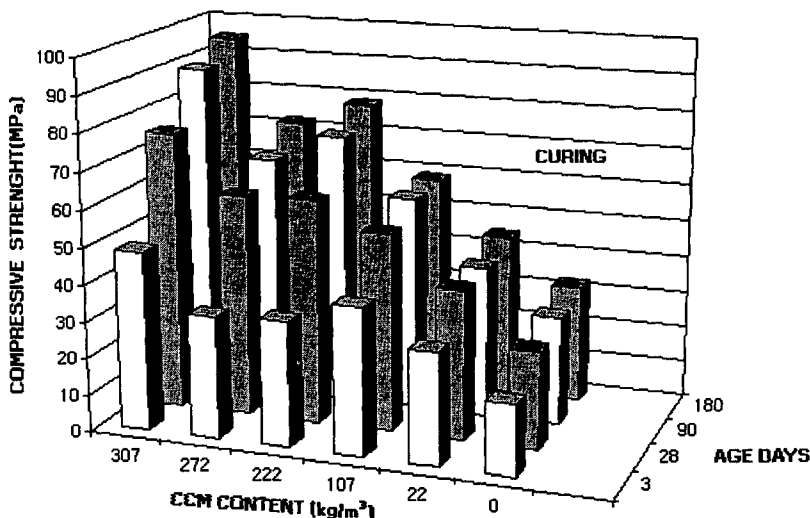


FIG.18. SERIES 1 TO 6 SHOWING THE CONTENT OF CCM EXCEPT PNPC, FOR CURING "A"

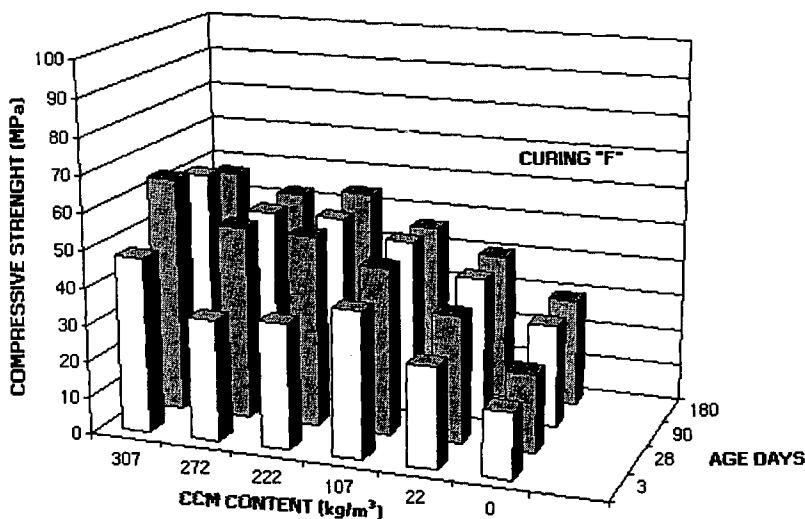


Fig.19. SERIES 1 TO 6 SHOWING THE CONTENT OF CCM EXCEPT PNPC, FOR CURING "F"

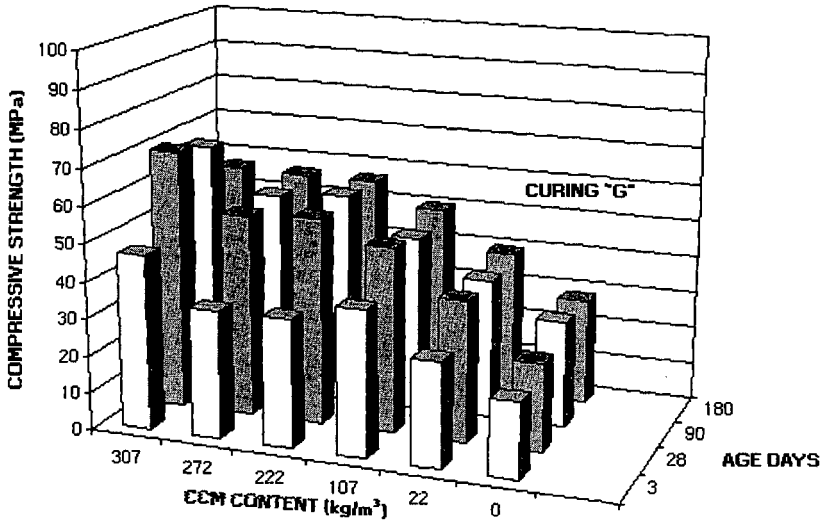


Fig. 20. SERIES 1 TO 6 SHOWING THE CONTENT OF CCM EXCEPT PNPC, FOR CURING "G"

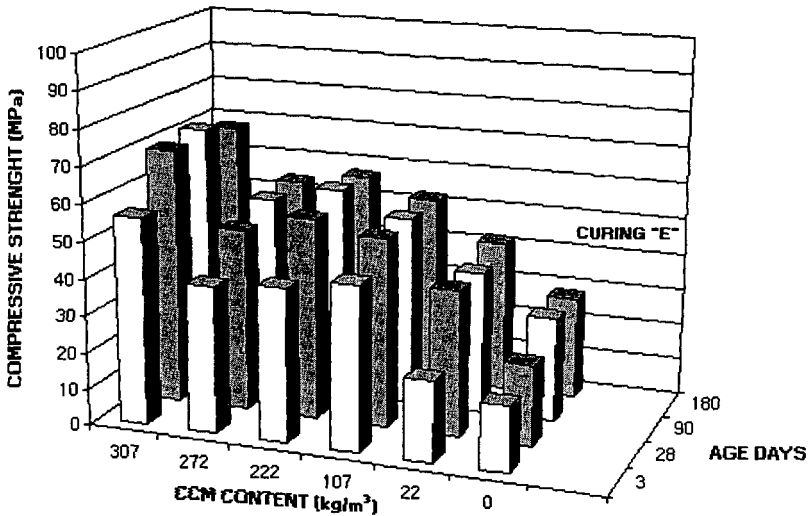


FIG.21. SERIES 1 TO 6 SHOWING THE CONTENT OF CCM EXCEPT PNPC, FOR CURING "R"

Marine Exposure of High-Strength Lightweight Concrete

by G. C. Hoff

Synopsis: An extensive laboratory program was initiated in the early 1980's to develop high-strength lightweight concrete for use in offshore oil and gas structures in severe marine environments. From the results of that development program, four mixtures were chosen to be evaluated under field conditions. Large prisms (305 by 305 by 914mm) of each mixture were placed in the tidal zone at the U.S. Army Corps of Engineers Severe Weather Exposure Station on the border between Canada and the United States. The mixtures used an expanded slate aggregate from the USA and a pelletized clay aggregate from Japan. All mixtures contained silica fume and had total binder content from 494 to 556 kg/cu m with water-binder ratios of 0.28 by mass. The concrete density varied from 1800 to 1990 kg/cu m, with 90-day strength from 60 to 73 MPa depending on the mixture. The prisms underwent annual visual and non-destructive evaluations. After 10-years exposure in the tidal zone, the prisms were removed to the laboratory where they were examined for strength, robustness and chloride ion penetration. This paper reports the results of the test program. In general, the overall performance looks very good.

Keywords: freezing and thawing; high-strength; lightweight aggregates; lightweight concrete; marine exposure

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INTRODUCTION

Between 1982 and 1986, eleven companies participated in a Joint Industry Project to study the use of high-strength lightweight concrete for Arctic applications. The results from that work are summarized in an earlier paper (1). Included in the evaluation program was the freezing and thawing behavior of high-strength lightweight concrete under a variety of laboratory test conditions. This information is specifically summarized in another paper (2). To complement the laboratory evaluations of freezing and thawing resistance, large specimens of selected mixtures from the initial program were produced and installed at the U. S. Army Corps of Engineers Severe Weather Exposure Station at Treat Island in Cobscook Bay near Eastport, Maine, USA. The island is on the border of the United States and Canada, and is at the entrance to the Bay of Fundy. At the exposure site, the specimens were on a wharf that becomes submerged in seawater with each tide. At the Northern latitude of Treat Island, the specimens also undergo approximately 100 cycles of freezing and thawing during the winter season when they alternate in and out of a submerged condition with the tide. The specimens were subjected to the marine exposure for 10 years. During that time period they underwent annual inspections. In 1995, after 10-years exposure, the specimens were retrieved from the exposure site and evaluated in the laboratory.

The concrete specimens were fabricated during a period from March 9 to March 26, 1985 at Construction Technology Laboratories, Skokie, Illinois and were shipped by truck to Eastport, Maine on May 30, 1985. Upon delivery, the truck driver, finding no one to receive them, just dropped them from the truck into a ditch on the side of the road and departed. They were subsequently retrieved and, in July 1985, were transported by boat to the exposure site and installed on the wharf. The initial installation of the specimens at Treat Island, their annual evaluations, and final retrieval with laboratory evaluations were conducted by researchers at the University of New Brunswick, Fredericton, New Brunswick, Canada.

This paper describes the effect of the severe marine exposure on the high-strength lightweight concrete that was evaluated, and relates the field exposure observations to the laboratory studies on the same types of concrete.

MATERIALS

Cement

Type II portland cement from a single source was used for all the concrete mixtures. The cement met the requirements of ASTM C 150, 'Standard Specification for Portland Cement,' for a Type I/II cement. It had a specific density of 3.15 and a low C_3A content of 5.5%. Chemical and physical analysis of the cement can be found in (1).

Supplementary Cementing Materials

Silica fume from a single supplier was used in all mixtures. The silica fume had a silica content of 92-93% and a specific density of 2.02. Chemical and physical analysis of the silica fume can be found in (1).

Aggregates

In the original test program (1), a screening of candidate lightweight aggregates (LWA) was conducted to select the most appropriate aggregates for study. Two aggregates were selected and were thoroughly studied (1). These same two aggregates were used in the test specimens of this program.

The first LWA was a crushed aggregate, 19 mm nominal size manufactured commercially from expanded slate in the USA. The oven-dry specific density was 1.35 while the saturated surface-dry (SSD) specific density was 1.46. The 24-hour absorption was 7.60%, and the dry-rodded unit weight was 775 kg/cu m.

The second LWA was a pelletized clay aggregate produced in Japan. It also came from a commercial production and had a nominal size of 16 mm. The oven-dry specific density was 1.33 while the saturated surface-dry (SSD) specific density was 1.45. The 24-hour absorption was 8.65%, and the dry rodded unit weight was 794 kg/cu m. For one of the mixtures evaluated (LWC-2 saturated)), the aggregate manufacturer provided pressure saturated aggregates that had an as-received moisture content of 21% and a specific density of 1.55 at that moisture content.

The fine aggregate used in all mixtures was a well-graded natural silica sand (Elgin Sand) with a specific density of 2.69.

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Chemical Admixtures

Each mixture contained both a high-range water reducing admixture (HRWRA) and an air-entraining admixture (AEA). The HRWRA was a sulphonated naphthalene formaldehyde condensate and met the requirements of ASTM C 494, "Chemical Admixtures for Concrete," for a Type F high-range water reducing admixture. It had a specific density of 1.20 and a solids content of 42%.

The AEA was a neutralized vinsol resin meeting the requirements of ASTM C 260, "Air-Entraining Admixtures for Concrete," and ASTM C 233, "Standard Test Method for Air-Entraining Admixtures for Concrete." It had a specific density of 1.05 and had a solids content of 6%. It was compatible with the HRWRA used in this study.

CONCRETE MIXTURE PROPORTIONS

Four mixtures from the initial program were evaluated and are described in Table 1. They are designated HSLWC, LWC1, LWC2 and LWC2 (saturated). LWC1 used a crushed lightweight aggregate while HSLWC and LWC2 used a pelletized lightweight aggregate. The fourth mixture, LWC2 (saturated) was similar to LWC2 but used pressure-saturated pelletized lightweight aggregate. In actual construction scenarios for these types of concretes, there is a high probability that the concrete will be pumped, and the pumping of lightweight concrete can be a problem if the aggregates are not properly conditioned prior to batching. The manufacturer of the pelletized aggregate of this study recommended that the moisture content of the aggregate should be between 20% and 25% by oven drying if the concrete was to be pumped. At those moisture levels, the amount of mixture water forced into the lightweight aggregate by the pump pressures would be minimized. The moisture content of the factory pressure-saturated aggregate used was 21%. The mixture proportions of LWC2 (saturated) were adjusted from LWC2 to account for the high moisture content of the aggregate.

TEST SPECIMENS

Two concrete prisms were made from each of the four basic mixtures of Table 1. Each prism had nominal dimensions of 305 mm by 305mm by 914 mm. A separate concrete batch was used for each prism. The identification of the mixtures and prisms is as follows:

Concrete <u>Mixture</u>	Prism <u>Number</u>	Concrete <u>Mixture</u>	Prism <u>Number</u>
HSLWC	11M	LWC2	31M
	12M		32M
LWC1	21M	LWC2	41M
	22M	(saturated)	42M

A summary of the fresh concrete properties for each batch is shown in Table 2.

After casting, the prisms were continuously moist cured for 7-days at 23°C and then cured in laboratory air until shipped. Four 150 mm by 300 mm cylinders were also made from each batch and were continuously moist cured at 23°C until tested. Two cylinders each were tested for compressive strength at 28- and 90-days age. The results are shown in Table 3.

FIELD SPECIMEN EVALUATIONS

Upon initial installation on the Treat Island exposure wharf, each specimen was photographed, and had its longitudinal pulse velocity and resonant frequency determined. The initial measurements taken in 1985 are shown in Table 5 and occurred when the specimens were between 4 and 5 months old.

Ultrasonic Pulse Velocity

The pulse velocity was determined in accordance with ASTM C 597, "Standard Test Method for Pulse Velocity Through Concrete." This method is used to assess the uniformity and relative quality of concrete, to indicate the presence of voids and cracks, and to indicate changes in the properties of the concrete. Fifty-millimeter diameter transducers were used with a coupling medium made of kaolinite and glycerol. The measuring meter was initially calibrated to a reference bar and checked periodically throughout the testing. When possible, five pulse transit times were determined along the longitude of each specimen. A measurement was taken at the center of each specimen and near each of the four corners. All other things remaining fairly constant, continuing decreases in the pulse velocity (increased pulse transit time) indicate internal deterioration of the concrete, principally due to development of cracks and microcracks.

Longitudinal Resonant Frequency

The longitudinal resonant frequency was determined in accordance with ASTM C 215, "Standard Test Method for Fundamental Transverse, Longitudinal, and Torsional Frequencies of Concrete Specimens." This test method is primarily used to detect significant changes in the dynamic modulus of elasticity of concrete that is undergoing exposure to weather or other types of deteriorating influences. The dynamic modulus of elasticity is a function of the mass of the specimen, the dimensions of the specimen, a correction factor that depends on the ratio of the radius of gyration to the length of the specimen and Poisson's Ratio, and the square of the resonant frequency. All other things remaining fairly constant, a progressive decrease in the resonant frequency indicates continuing deterioration of the concrete.

Annual Inspections

At approximately 1-year intervals, beginning in 1986 and continuing through 1995, the specimens were visually examined, photographed and had their pulse velocity and resonant frequency measured. During that time, the specimens were submerged and exposed to air with each tidal fluctuation and, during the winter season, underwent an average of 109 cycles of freezing and thawing per year. The actual observed number of freezing (internal concrete temperatures (-2.2°C) or below) and thawing (internal concrete temperatures $(+2.2^{\circ}\text{C})$ or above) cycles was as follows:

<u>Winter Season</u>	<u>No. of Cycles</u>	<u>Winter Season</u>	<u>No. of Cycles</u>
1985-86	144	1990-91	89
1986-87	109	1991-92	75
1987-88	110	1992-93	123
1988-89	106	1993-94	97
1989-90	135	1994-95	102

Specimen 21M

Sometime during the 1990-1991 exposure period, one end of specimen 21M broke off from the specimen. The crack that precipitated the break was evident during the 1990 annual inspection. The broken piece represented approximately 22% of the volume of the specimen. It was retrieved in December 1991 and taken to the University of New Brunswick where it was evaluated for density, tensile

and compressive strength, alkali-aggregate reactivity, hardened air-void content, and air void spacing. The results of the evaluation are described next.

The broken piece measured approximately 305 by 305 by 203 mm and weighed 38.6 kg. The break was a clean fracture with no other pieces broken off. When brought to the laboratory, it was weighed in both air and water and the density was calculated to be 1844 kg/cu m. Subsequent storage in a moist curing room did not increase the density.

Four 95 mm cores were drilled completely through the block. Two each were drilled from both the end of the block and the side of the block to check for cracks or other flaws in the concrete. No defects were visible. The cores were kept in a moist room until tested.

Tensile strength tests were performed on three of the cores using an indirect tensile strength test developed by the Building Research Station of Great Britain (3,4). This test basically involves injecting nitrogen gas into a sealed off section of core to build up an internal pore pressure. This pressure is increased until the core fractures. The tensile strength is read directly as the pressure needed to break the core. The results are shown in Table 6. Opposite ends of cores 1 and 2 were tested to get an indication of the tensile strength throughout the specimen. These two cores were taken from the side of the block and were parallel to each other.

Compressive strength tests were done according to ASTM C 39, "Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens," and ASTM C 42, "Obtaining and Testing Drilled Cores and Sawed Beams of Concrete," on the remaining pieces of cores 1, 2 and 3 after the tensile tests and on core 4 which was too short to do both a tension and compression test. The results are shown in both Table 4 and 6.

The pieces from the broken cores from the tensile strength test were used to check for the alkali-aggregate reaction (AAR) using the uranyl acetate fluorescent test. The fracture surfaces were coated with uranyl acetate and let stand for 3 to 5 minutes. The excess uranyl acetate was then rinsed off and the core put under an ultraviolet lamp. Any AAR on the surface then shows up as a bright green glow. Only traces of AAR were noted in fine aggregate (natural sand).

Optical and microchemical studies of the sawn and polished surfaces of concrete were carried out. Determinations of the air-void content and air-void spacing were done in accordance with ASTM C 457, "Standard Test Method for Microscopical Determination of Parameters of the Air-Void System in Hardened Concrete." The principal results were as follows:

Air Content, %	4.7
Specific Surface, mm ⁻¹	12.01
Spacing factor, mm	0.500

Post-exposure Evaluations

After the field measurements were made in 1995, the specimens were taken from the wharf and moved to the laboratory at the University of New Brunswick. Discussion of specimen breakage is presented in the following section. Specimens 11M and 31M experienced an additional break during retrieval that had not fully manifested itself on the wharf. All other breaks had occurred during the exposure period.

For strength testing, each of the specimens had three 95 mm cores drilled from it. The cores were drilled from the side of the specimen with one core being obtained, where possible, from each third of the specimen. One core from specimen 22M was inadvertently drilled across a crack and could not be tested. The cores were cut into 200 mm lengths, and the ends were ground for testing. As with the earlier evaluation of a piece of Specimen 21M, compressive strength tests of the cores were done according to ASTM C 39 and ASTM C 42. The results of the compressive strength testing are shown in Table 4.

An additional 95 mm core was drilled from near the center of each specimen to evaluate the extent of chloride ion penetration into the concrete over the ten-year exposure period. The specimens were prepared following Canadian Standard A23.2-4B, "Sampling and Determination of Water-Soluble Chloride Ion Content in Hardened Grout or Concrete," and the Mercuric Nitrate Test Method was used to determine the amount of soluble chloride. The chloride content was determined in five layers as shown in Table 7. The chloride content shown is the average from both specimens for that mixture. Any chloride content values below 100 mg/kg are shown as 100 mg/kg because this is the limit of precision of the test method.

SPECIMEN BREAKAGE

Five of the eight specimens had lateral breaks in them at the time they were removed from the wharf. All of the specimens had at least one of the top four corners broken. The lateral breaks occurred from 200 to 330 mm from the ends of the specimens. The orientation of the end of the specimen (towards the beach or towards the sea) made no difference. In general, the location of the breaks corresponded to the supporting timber that held the specimen off of the wharf surface. The first observance of a lateral crack in the specimen and the time when the specimen broke are as follows:

Specimen Number	Observed First Crack	Specimen Broke
11M	1991	1993
12M	1992	1994
21M	1990	1991
22M	1991	1993
31M	1993	1995

The researchers who carried out the annual inspections believe that the cracks were the result of rough handling of the specimens during their initial transportation to the exposure station, that was followed by a period of rough storms that displaced the specimens and caused a partial collapse of the supporting structure. No plausible explanation based on materials behavior could be developed. Once cracked, subsequent freezing and thawing cycles caused the material to separate. The failure surfaces were fairly straight and uniform. Once the specimens broke into smaller pieces, the resonant frequency measurements could not be related to the original specimen size and configuration, and were discontinued.

DISCUSSION OF FIELD EXPOSURE RESULTS

Chloride Ion Observations

The chloride ion contents observed in the first 11 mm indicate that there was little difference for the three mixtures (HSLWC, LWC2 and LWC2 (saturated)) made with the pelletized clay lightweight aggregate. The concrete made with the crushed slate lightweight aggregate contained approximately 30% more chlorides. At this close proximity to the surface of the concrete, it is probable that the open cells on the surface of the crushed aggregate had taken in some chlorides from sea water in addition to some that were present in the original concrete mixture.

For the second 11 mm, there was some ranking of the chloride levels with the HSLWC having the highest level, followed by the crushed slate aggregate mixture (LWC2), the pelletized clay mixture (LWC3) and the saturated pelletized clay aggregate mixture (LWC2 (saturated)). At subsequent depths (30 to 105 mm), the HSLWC had somewhat higher amounts of chloride than the other mixtures.

For the duration of this exposure period, the levels of chloride observed would not appear to be a problem for reinforcing bars that had the usual amounts of concrete cover (50 to 75 mm) typically specified for marine structures. This conclusion is supported by the observation in the initial study (1) that LWC1, LWC2 and HSLWC experienced density increases of less than 16 kg/cu m when submerged in simulated seawater for periods up to 1-year. This suggests that very little

moisture is entering the concrete at near-surface water depths experienced at Treat Island.

Pulse Velocity Measurements

Table 8 shows the ratio of the yearly pulse velocity measurement to the initial pulse velocity measurement (See Table 5). For the first eight years, five pulse velocity measurements were made (when possible) each year on each specimen. As noted earlier, a measurement was made near each of the four corners and at the center of the specimen. In years 9 and 10, only one measurement was made at the center of the surviving specimens because many of the corners had broken or had become irregular so that good contact between the transducer and the concrete could not be obtained. With the exception of Specimen 21M, which had an observed transverse crack that caused a reduction in the pulse velocity at the Year 5 inspection, the other specimens appeared to begin reductions in pulse velocity at Years 7 and 8. Because corner damage and some transverse cracking was occurring at that time, the measurements made at the corners (particularly the top two corners because the transverse cracking and corner cracking appeared to initiate at the top surface) tended to be low values that reduced the overall average value. Consideration of only the pulse velocities taken at the center of the specimen indicated virtually no change in pulse velocity until the specimens either broke or exceeded 9 years of exposure. The Year 10 pulse velocities of 32M, 41M and 42M were reduced approximately 10%. This suggests that the uncracked lightweight aggregate concrete was relatively unchanged in the marine environment up to 1000 cycles (approx.) of freezing and thawing.

Resonant Frequency

Table 9 shows the ratio of the yearly resonant frequency measurement to the initial resonant frequency measurement (See Table 5). As noted above, when all things remain fairly constant for a test specimen, continuing decreases in the resonant frequency indicate continuing deterioration of the concrete. Because the resonant frequency is related to the length of the specimen, when the specimen begins to appreciably shorten, the measured values of the shorter specimen can no longer be directly compared to the longer specimen. This is what happened as the some of the specimens began to have transverse cracks. The end points of the results shown in Table 9 reflect when the resonant frequency measurements were stopped for each specimen.

The results for LWC1 indicate that Specimens 21M and 22M began to experience some deterioration after a few years of exposure. This gradual deterioration continued at a fairly uniform rate until the specimens broke. After 3-years

exposure at Treat Island, the specimens had undergone 363 freezing and thawing cycles and were beginning to show a slight decrease in resonant frequency. For the laboratory test (1) with ASTM C 666, Procedure A, there was no change in the dynamic modulus at 300 cycles. After 6 or 7 years exposure at Treat Island, LWC1 had been subjected to 693 or 768 freezing and thawing cycles, respectively and had experienced approximately a 10-12% decrease in resonant frequency. The "rapid freeze thaw test" used for LWC1 in the initial program (1,2) was identical to the ASTM C 666, Procedure A except that it used simulated seawater. For the same number of cycles as Treat Island, the results in the laboratory also indicate a 10% reduction in the dynamic modulus.

Specimens 11M and 12M began to experience some deterioration after 5- to 6-years of exposure. This gradual deterioration also continued at a fairly uniform rate until the specimens broke. These results are similar to the laboratory results (1,2) with the field specimens performing slightly, but not significantly, better.

Specimens 31M, 32M, 41M and 42M all indicated exceptional performance after 1090 cycles of freezing and thawing.

Strength Considerations

From Tables 3 and 4, the following comparison of the compressive strength data can be made:

	28-day Cylinder Strength,	10-Year Average Core Strength,	Percent of Cylinder Strength,
<u>Mixture</u>	<u>MPa</u>	<u>MPa</u>	<u>%</u>
HSLWC	70.0	59.8	85
LWC1	59.3	47.3	80
LWC2	59.8	38.1	64
LWC2 (sat.)	58.3	53.3	91

For LWC1, the 8-year (Specimen 21M) and 10-year core strengths were lumped together as there was no significant difference in the average values.

The experience of this author (5) with cores from normal density high-strength concrete, taken at later ages (5-years) and compared to continuously moist cured cylinders, was that the core strength typically was only 70 to 80% of the cylinder strength. For normal density high-strength concrete, it is generally believed that due to the high binder content, the presence of silica fume and low water-binder ratio, a self-desiccation can occur that leads to internal microcracking, and associated strength regression with time. For lightweight concrete, which initially contains more moisture in the aggregate, less effect due to self-desiccation should be expected because the aggregates can supply additional water for the binder

hydration, particularly at the interfacial zone of paste and aggregate. This is evident to some degree for LWC1 (80%) and particularly for HSLWC (85%) and LWC2 (saturated) (91%). The core strengths of LWC2 are lower (64%), thus suggesting that the concrete may have begun to deteriorate due to the severe exposure.

The tensile strength (Table 6) measured from the cores of the broken piece of 21M (LWC1) averaged 4.8 MPa. Corresponding compressive strengths from cores of the same piece averaged 47.8 MPa. In the original work (1), for LWC1 the 28-day ratio of splitting tensile strength of cylinders (ASTM C 496, Splitting Tensile Strength of Cylindrical Concrete Specimens) to compressive strength of cylinders, both of which had been moist cured for 28-days, was only 6.6%. There is not enough data here to make any specific conclusions but the observed tensile strength appeared to be adequate so that there were no concerns about loss of tensile strength.

Surface Conditions

For higher numbers of cycles in the laboratory "spectral" testing (1,2), LWC1 and LWC2 showed extensive cracking, spalling and pitting, and HSLWC was experiencing extensive scaling. For the Treat Island exposure, HSLWC did not exhibit any pitting or scaling. LWC1, LWC2 and LWC2 (saturated) did have a small amount of pitting but it occurred only on the top surface of the specimens. That surface was the only unformed surface of the specimen, having been hand-finished with a trowel. All of the specimens did experience a small loss of the material at the corners. In a few instances, some of the corners were broken off. In general, the surfaces were considered satisfactory.

CONCLUSIONS

After exposing several mixtures of high-strength lightweight aggregate concrete in the tidal zone to a severe marine environment for 10-years, the following conclusions were reached:

1. Chloride contents in the concrete cover were quite low as would normally be expected from properly designed and built marine structures. Concrete made with a crushed slate aggregate had higher chloride contents near the surface, than comparable concrete made with a pelletized clay aggregate. This may be due to an open cell structure on the surface of the crushed aggregate that would adsorb some chloride when compared to the sealed surface of the pelletized aggregate. The higher strength concrete of this program (68.9 MPa)

generally had somewhat higher chloride content than the lower strength concrete (57.9 to 60.0 MPa).

2. Based on ultrasonic pulse velocity and resonant frequency measurements made annually over a 10-year period, the high-strength lightweight concrete prisms have performed quite satisfactorily on exposure to numerous wetting-drying and freezing-thawing cycles (an average of 109 freezing and thawing cycles per year).
3. With the exception of two prisms (31M and 32M) made with mixture LWC2, the core strengths of the concrete after 10-years were within expected values (80 to 90 percent) of the 28-day cylinder strength. Companion prisms (41M and 42M) to LWC2, made with the same lightweight aggregate but in a saturated condition, gave the highest core strength ratio (91 percent) thus suggesting that there may have been self-desiccation internally in the concrete mixture LWC2 made with unsaturated aggregate.
4. In general, the surface condition of the concrete was good after 10-years of severe marine exposure. A number of the unreinforced prisms experienced some through-thickness breakage and broken corners. This was attributed to rough handling prior to their installation and being tossed about on the wharf during severe storms.
5. While the concrete appears to give satisfactory performance up to 10-years age, these limited results are not sufficient to draw conclusions about long-term durability (50- to 100-years service life) in a severe marine environment.

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Table 1 -- Concrete Mixture Proportions

Mixture	kg/cu m							
	HSLWC		LWC1		LWC2		LWC2 (Sat.)	
Specimen	11M	12M	21M	22M	31M	32M	41M	42M
Cement, Type II	488	492	450	457	449	449	480	477
Fine Aggregate	908	492	565	574	542	542	614	610
Coarse Aggregate								
Crushed Type	----	----	570	579	----	----	----	----
Pelletized Type	353	356	----	----	574	574	618	613
Silica Fume	63	64	45	46	45	45	48	47
High-Range Water Reducer	5.82 liter	5.88 liter	3.87 liter	4.05 liter	4.08 liter	4.08 liter	5.23 liter	5.20 liter
Air-Entraining Admixture	0.70 liter	0.49 liter	0.90 liter	0.91 liter	0.90 liter	0.90 liter	0.84 liter	0.83 liter
Mix Water*	148	149	133	136	132	132	141	139
Water/Cement	0.315	0.314	0.307	0.307	0.303	0.303	0.304	0.304
Water/Cementitious	0.279	0.278	0.279	0.279	0.276	0.276	0.277	0.277

*Water added in addition to that contained in the chemical admixtures.

Table 2 -- Properties of Fresh Concrete

Concrete Mixture	Specimen Number	Slump, mm	Air Content, percent	Unit Weight, kg/cu m
HSLWC	11M	165	5.8	1983
	12M	200	5.0	1977
LWC1	21M	105	6.7	1799
	22M	165	5.0	1805
LWC2	31M	130	6.9	1796
	32M	150	7.4	1780
LWC2 (saturated)	41M	155	5.0	1805
	42M	130	5.8	1804