SPECIAL REPORT

Durability Aspects of Precast Prestressed Concrete Part 1: Historical Review

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A review of past research on the effect of heat curing on strength, frost resistance, and AASHTO T 277 (also ASTM C 1202) "coulomb" values is presented, and the research experience compared to present-day codes, specifications, and test methods. Historically, properly heat-cured concretes produced at low water-cement ratios have been found to have strength and frost resistance properties equal to or better than conventionally-cured concretes. The AASHTO T 277 test, and the similar ASTM C 1202 test, were also reviewed as they relate to precast concrete, revealing that significant questions remain regarding their appropriateness for use in concrete project materials qualifications and specifications.

Since 1950, the engineering profession has observed that weather-exposed precast, reinforced concrete structures and precast, prestressed concrete structures with adequate air-void systems have exhibited excellent durability. The resistance of precast concrete to freezing and thawing and to corrosion of reinforcement has also been researched extensively since 1960. Some studies were made on properly air-entrained and properly steam-cured or heat-cured concretes, while other studies were performed on improperly air-entrained or non-air-entrained concretes and improperly steam-cured or heat-cured concretes.

Part 1 of this two-part report will review the specific conclusions of these previous studies that relate to the present state-of-the-art and specifications.

Part 2 presents comprehensive test results of a 1-year chloride ion permeability and coulomb study of heat-cured and moist-cured concretes with and without silica fume, subjected to various curing procedures. This investigation was funded by the Precast/Prestressed Concrete Institute (PCI) in 1994 and 1995. The comprehensive data from Part 2 further elucidate the excellent durability of heat-cured, low water-cement ratio (w/c) conventional concretes as used in the United States for the last 45 years.

Table 1. Typical freezing and thawing test data for air-entrained, 0.43 w/c Type IIIA cement concrete.

Cure type*	Age when ASTM C 290† freezing and thawing tests started	Durability factor§	Expansion§ (percent)	Weight change§ (percent)
28-day moist	31 days	95	0.019	-0.7
14-day moist and 14-day air dry	31 days	102	0.017	+0.4
16 hours at 160°F (71°C) and 14-day air dry	18 days	107	0.022	+0.3
16 hours at 160°F (71°C) and 7-day moist and 14-day air dry	25 days	103	0.025	+0.5

* All four cure types had 3 days of water soaking to saturate the concrete prism just prior to the freezing-thawing tests in water.

† Now part of ASTM C 666.

§ After 300 freezing and thawing cycles in water.

BACKGROUND

Numerous durability and concrete compressive strength studies on steam-cured or heat-cured concretes have been funded by the Federal Highway Administration (FHWA),[1] the Portland Cement Association (PCA),[2,3,4,5] PCI,[6,7,8,9] and many other agencies and companies. These studies have established that resistance to freezing and thawing relates primarily to proper air entrainment, as recommended by American Concrete Institute (ACI) Committee 201.

For $^3/_4$ in. (19 mm) nominal maximum size aggregate concretes with specified strengths of 5000 psi (34.5 MPa) and below, Table 4.2.1 of ACI 318-95 indicates $4^1/_2$ to $7^1/_2$ percent air for severe exposures and $3^1/_2$ to $6^1/_2$ percent air for moderate exposures. The 1995 ACI 318 Table 4.2.1 allows the use of 1 percent less total air content for concretes with specified compressive strengths greater than 5000 psi (34.5 MPa). Therefore, higher strength $^3/_4$ in. (19 mm) nominal aggregate precast concretes could have about $3^1/_2$ to $6^1/_2$ percent air for severe exposures, and $2^1/_2$ to $5^1/_2$ percent air for moderate exposures.

These air contents in air-entrained concretes have been shown in numerous studies to provide excellent resistance to freezing and thawing. However, significant PCA-funded studies in 1960 and 1978 on low water-cement ratio (w/c) (0.30 to 0.40) moist-cured concretes[10] and on low w/c (0.33) simulated steam-cured concrete and moist-cured concrete[2] showed that even non-air-entrained concretes were also very frost resistant when allowed an air-drying period before freezing and thawing tests in water.

The water absorption, chloride ion ingress, and frost resistant properties of concrete relate directly to the w/c. For decades, precast concretes have commonly had very low 0.30 to 0.40 w/c. These low w/c concretes were necessary to achieve the high early-age compressive and tensile strengths mandated by ACI 318, AASHTO, and PCI, which enabled early-age prestressing, stripping, and handling, often in periods less than 16 hours.

These low w/c contrast with the higher 0.45 to 0.60 w/c commonly used in cast-in-place concrete construction during the last 45 years. In 1989, ACI 318 mandated a maximum 0.45 w/c for all concrete exposed to freezing and thawing conditions. For corrosion protection from deicing salts, salt water, brine, and other harmful agents, ACI 318-89 required a 0.40 w/c but allowed a 0.45 w/c if their Section 7.7 minimum clear cover requirements were increased by 0.5 in. (13 mm). This 0.45 w/c alternative provision was eliminated in 1992. The 1995 AASHTO w/c maximum requirement for corrosive environments is 0.45.

The ACI 318-89 minimum clear covers were not revised in 1992 or 1995, nor were the ACI 318R-89 greater minimum clear covers suggested in the Corrosive Environment Commentary in ACI 318R. For cast-in-place concrete, ACI 318-95 (R.7.7.5) recommends a minimum cover of 2 in. (50 mm) for walls and slabs and $2^1/_2$ in. (64 mm) for other members. A $^1/_2$ in. (13 mm) reduction of cover is allowed by ACI 318R for precast concrete.

These current code practices illustrate that corrosion and frost resistance require properly air-entrained concretes with a maximum w/c of 0.40 or 0.45, depending on the exposure environment and which code is used. The following review of significant published research papers since 1960 was conducted to determine how the current 1995 ACI and AASHTO durability code requirements were developed, as related to precast, prestressed concrete.

HISTORICAL REVIEW

Presented here in chronological order is a review of resistance to freezing and thawing, compressive strength and heat curing investigations:

Klieger (1960)

In 1960, Klieger undertook a comprehensive study[2] at PCA of resistance of concrete to freezing and thawing to determine the effects of simulated steam curing of concrete at 160°F (71°C), continuous moist curing at 73°F (23°C), and a combination of continuous moist curing followed by air drying at 73°F (23°C). Non-air-entrained and air-entrained concretes with 0.33 and 0.43 w/c, respectively, were tested. A preset time of 3 to 4 hours was used prior to the heat curing. The average temperature rise was about 20°F (11°C) per hour, and the maximum heating period at 160°F (71°C) was 11 hours. Typical freezing and thawing test data from this study are shown in Table 1 for the properly air-entrained, 0.43 w/c Type IIIA cement concrete.

These data show that the air-entrained concrete, when properly heat cured at 160°F (71°C) and allowed a 14-day air-drying period after heat curing, exhibited the highest durability factor of 107 percent after 300 cycles of freezing and thawing in water. This exceeded both of the continuous moist-cured durability factors of 95 and 102 percent. It also exceeded the 103 percent durability factor of the heat-cured concrete that was moist cured for 7 days after steam curing.

These data show that subsequent moist curing of the steam-cured concrete decreased the durability factor. Similar conclusions regarding the lack of benefit from 7 days of supplemental moist curing were reached for the other Type IA and IIIA cement concretes tested in this PCA study on heat-cured air-entrained concretes. This study reached similar conclusions when 0.33 w/c no-slump, non-air-entrained concretes with air contents of 2.2 to 2.4 percent were tested. Therefore, this PCA study showed that 7 days of supplemental moist curing did not improve the frost resistance of steam-cured concretes given a reasonable 3- to 4-hour delay period and some air drying prior to freezing.

Klieger commented on the benefit of air drying, "This drying will normally occur prior to exposure and therefore from a practical standpoint this situation should be of little concern." Under the conditions of these severe 300 cycles of freezing and thawing conducted with the air-entrained concrete specimen always under water or ice, the companion continuously moist-cured concretes with Types IA and IIIA cements also needed this air-drying period to achieve durability factors greater than 100 percent.

Higginson (1961)

The 1961 paper by Higginson[11] titled "Effect of Steam Curing on the Important Properties of Concrete" suggested that supplemental fog curing after steam curing is necessary to improve the durability of steam-cured concrete. Unfortunately, this study was based on improper steam curing that included preset periods of only 1 and 3 hours. Therefore, the heat was applied prior to time of initial setting and a proper delay or preset, as used by Klieger in 1960, was not used. The report contains no data of initial setting time.

The 28-day strengths of Higginson's steam-cured concrete to 100, 130, and 160°F (38, 54 and 71°C) with a 1-hour delay averaged 68 percent of the moist-cured concrete with a coefficient of variation (CV) of 8.7 percent. The 28-day strengths of the steam-cured concrete with a 3-hour delay averaged 73 percent of the moist-cured concrete strengths, with a CV of 9.7 percent. Therefore, the 1- and 3-hour delays created on average 32 and 27 percent strength losses at 28 days, respectively, when compared to the continuously moist-cured concretes.

These strength reductions are now known to be related to the application of heat at ages before the ASTM C 403[12] time of initial setting had been achieved. As discussed later, the early-age application of heat creates large volume increases in fresh concrete, creating micro- and macro-cracks and permanent volume increases. Such cracked and expanded concretes would be expected to have poor frost resistance. These vital issues were apparently not widely recognized in 1961.

Higginson also used marginally air-entrained or possibly non-air-entrained concretes, with air contents stated to be 3 percent. No specific air contents were provided. The w/c of the concretes were also not provided nor discussed. The reported freezing and thawing data indicate that all of the moist-cured and steam-cured concretes were of highly questionable durability, because the failure criteria selected were based on a concrete weight loss of 25 percent. This extremely large weight loss contrasts with minor weight gains reported in 1960 by Klieger[2] for durable concretes.

Essentially, none of the 5-bag (279 kg/m^3) moist-cured or steam-cured concretes reached 300 cycles of freezing and thawing without suffering a 25 percent weight loss — unquestionably, non-durable concrete. The 7-bag (391 kg/m^3) concretes were shown to be more durable, yet even here the moist-cured concrete given 7 and 21 days of air drying did not reach the 300 cycles without a 25 percent weight loss, again indicating non-durable concrete.

The durability of steam-cured concrete should not be compared with moist-cured concrete, based on Higginson's paper, due to the use of only 1- and 3-hour delay or preset periods, which created severe strength losses at 28 days, the associated internal and surface cracks, and volume changes now known to be associated with these strength losses, as well as the questionable air contents. Therefore, Higginson's recommendation of supplemental 7 days of fog curing is inappropriate, based on Klieger's 1960 paper[2] and data developed by other researchers after 1961, as further discussed in this paper.

Hanson (1963)

The classic study at PCA by Hanson[3] in 1963 clearly showed in photographs that visible macro-cracking would occur in properly air-entrained 0.32 and 0.39 w/c concretes, when allowed only a 1-hour preset period prior to steam curing to air temperatures of 125 to 175°F (52 to 79°C). These concretes also suffered significant 28-day strength losses, ranging up to 50 percent. Macro-cracking was not detected in any concretes given the 3-, 5-, or 7-hour preset periods, even when cured at 175°F (79°C).

Hanson concluded that a delay period of about 5 hours, combined with a temperature rise of 40°F (22°C) per hour to about 150°F (66°C), would be optimum. These properly steam-cured Types I and III cement concretes achieved 28-day strengths of about 90 percent of the continuously moist-cured concrete and contained no visible cracks.

ACI Committee 517 (1963)

The ACI Committee 517 report "Low Pressure Steam Curing" was published in August 1963. A total of 30 published papers were reviewed, including the 1961 Higginson[11] and 1963 Hanson[3] papers. The report did not recommend the use of "supplemental moist curing" of any length after steam curing. The report also included the following observation:

> "As stated previously, the ultimate compressive strength of steam-cured concrete is not as great as that of concrete continuously moist cured at lower temperature; however, in actual practice concrete is often given very little moist curing so that the advantage of steam curing may be considerably greater than would be apparent from comparison with 28-day moist curing."

This statement is still true today. It is probably even more relevant today because less effective liquid curing compounds have all but replaced 7-day continuous moist curing at many jobsites.

Brown (1963)

In 1963, Brown[13] of the Virginia Highway Research Council used the penetration resistance method (ASTM C 403) to determine the time of initial setting.[12] His investigations concluded that the time at initial setting was a scientific method for determining a proper delay period, accounting for the differences in factors such as cement type and composition, w/c, seasonal temperatures, and the use of admixtures.

Hanson (1965)

In 1965, Hanson[4] extended his studies and concluded that a 3- to 5-hour delay period prior to steam curing was optimum, for structural lightweight concrete, to achieve the greatest 18-hour strength. For maximum compressive strength after 12 hours, he concluded that a 3-hour delay was better than a 5-hour delay. His work on lightweight concrete also concluded that the 1-hour delay period caused substantial early-age and 28-day strength losses.

Table 2. Typical concrete strength loss data for different delay periods.

	Strength loss at 28 days* (percent)		
	Curing temperature, °F (°C)		
Delay period (hours)	113† (45)	149† (65)	176† (80)
1.0	18	22	56
2.5	13	27	41
4.0	17	33	46
5.5	7	-4	2
7.0	9	-4	3

* As compared to continuous moist-cured concrete at 68°F (20°C), w/c = 0.50.
† Specimens put into bath immediately following delay period at 68°F (20°C).

Table 3. Typical restrained concrete strength loss data for 1/2-hour delay period.

	Strength loss at 28 days* (percent)							
	Curing temperature, °F (°C)							
Delay period (hour)	86† (30)	104† (40)	122† (50)	140† (60)	158† (70)	176† (80)	194† (90)	212† (100)
1/2	3	1	1	6	-5	1	5	3

* As compared to continuous moist-cured concrete at 68°F (20°C), w/c = 0.50.
† Sealed specimens put into water bath immediately following 1/2-hour delay period at 68°F (20°C).

Alexanderson (1972)

Alexanderson[14] reported on numerous heat curing tests using different delay periods, w/c, mixture proportions, cement types, air contents, and maximum temperatures of curing. Typical concrete strength loss data from his tests are shown in Table 2.

His experiments showed that a volumetric increase of the fresh concrete during heat curing is caused by pressure increases in the pores. By providing a proper delay period, the tensile strength of the fresh concrete increases so that during heating the pore pressures can be resisted by the higher tensile strength.

Cracking and strength loss could thus be minimized or totally prevented as shown in Table 2 for the 5.5- and 7.0-hour delays. His tests also demonstrated that the strength losses were greater with air-entrained concretes compared to non-air-entrained concretes.

Alexanderson also performed additional tests in which he prevented volumetric increases in the fresh concrete by using vertically restrained and sealed steel molds to show that the strength losses were caused by physical expansion and cracking, not by chemical effects. These tests used an extremely short delay period of 1/2 hour, a procedure that would normally cause severe volume increases, cracking, and strength loss. Typical data from this restrained concrete test series are shown in Table 3.

The data in Table 3 show that a 28-day strength loss did not occur when these restrained concrete specimens were tested at any temperature from 86 to 212°F (30 to 100°C). These data show that chemical causes for strength loss clearly play a secondary role during heat curing. They also establish the role of physical expansion in strength loss for heat-cured concrete, and illustrate the critical role that a proper delay period plays in the heat curing cycle.

AASHTO (1974)

In 1973, PCI, along with the AASHTO Subcommittee on Prestressed Concrete, prepared a proposed change to the Steam Curing Specification contained in the AASHTO Standard Specification for Highway Bridges, Division II — Construction, Section 4 "Concrete Structures," Article 2.4.33 "Prestressed

Table 4. Strength comparison of 28-day heat-cured specimens incorporating 40°F (22.2°C) per hour rate of rise after preset period with 28-day moist-cured specimens.

Mixture type	Cure temperature °F	Cure temperature (°C)	Heating period (hours)	Average 28-day heat-cured strength as percentage of moist-cured strength (percent)
I	110	(43)	3	99
	145	(63)	6	96
	180	(82)	14	101
III	110	(43)	3	103
	145	(63)	6	106
	180	(82)	15	106
I + HRWRA*	110	(43)	3	94
	145	(63)	6	94
	180	(82)	14	93

* High range water reducing agent.

Concrete," Subarticle E "Steam Curing." The revised Article 2.4.33 was retitled "Accelerated Curing with Low Pressure Steam or Radiant Heat."

This proposed change introduced for the first time the use of ASTM C 403 "Standard Test Method for Time of Setting of Concrete Mixtures by Penetration Resistance."[12] This test technique had not been previously recommended in AASHTO, ACI, PCI, or other highway department specifications, although most previous specifications did require some degree of delay period prior to applying significant heat to the concrete.

This proposed change was adopted and included in the 1974 Interim Specification, Bridges, as Interim No. 18. It was subsequently included in the 1977 AASHTO Standard Specifications for Highway Bridges, Twelfth Edition, in Section 2.4.33, Section E.[15] This change also removed the requirement that 6 days of additional water curing must be provided after the accelerated curing and that accelerated-cured concrete should not be exposed to temperatures below freezing for 6 days after accelerated curing.

Soroka et al. (1978)

Soroka et al.[16] showed in three different series of tests that concretes that were improperly heat cured to temperatures of 140 to 175°F (60 to 79°C), after delay periods of only ½ to 1 hour, suffered significant strength losses at 28 and 90 days, as would be expected. When companion concretes were cured in water at 68°F (20°C) for 7 days immediately after heat curing, the 28- and 90-day strength losses did not occur in most of their tests, indicating that the microcracks caused by the improper heat curing process were repaired by autogenous healing during the supplementary water curing period.

ACI Committee 517 (1980)

The ACI Committee 517 prepared a state-of-the-art report titled "Accelerated Curing of Concrete at Atmospheric Pressure." This document did not suggest "supplemental fog or moist curing" following accelerated curing, nor did it recommend the use of the ASTM C 403 time of initial setting test to determine the delay period.

Pfeifer et al. (1981)

A comprehensive state-of-the-art literature review report[8] on accelerated heat curing of precast concrete for the 1950 to 1980 period was published by PCI. Numerous relevant observations are discussed in this 182-page PCI Technical Report No. 1. In addition, a comprehensive laboratory study[7] was conducted using concretes with w/c from 0.30 to 0.43, 6.75 bags per cu yd (376 kg/m³) of Types I and III cements, 3 in. (75 mm) slump, proper preset or delay periods, and heat curing at 110 to 180°F (43 to 82°C).

The initial set and delay periods determined, using ASTM C 403, were 3 and 4 hours for the Types I and III cements, respectively. Twelve different curing cycles with different heating periods and maximum air temperatures were evaluated. The 28-day moist-cured strengths ranged from 5900 to 9100 psi (40.6 to 62.7 MPa). The average 28-day strength of the heat-cured specimens that incorporated the 40°F (22.2°C) per hour rate of rise after the preset period, as compared to the continuously moist-cured specimens at an age of 28 days, are given in Table 4.

These data show that properly heat-cured concrete suffered essentially no 28-day strength decrease compared to the continuously moist-cured concretes when stored as per AASHTO and ASTM procedures in saturated lime water after heat curing.

ACI Committee 517 (1987)

In 1987, ACI Committee 517 updated their state-of-the-art report, "Accelerated Curing of Concrete at Atmosphere Pressure." This document also did not suggest "supplemental fog or moist curing" following accelerated curing, but suggested the ASTM C 403 time of initial setting test for use in precasting plants.

AASHTO (1989)

The 1989 AASHTO Standard Specifications for Highway Bridges, Division II — Construction, Section 8 "Concrete Structures," Subsection 8.11 "Curing Concrete" discusses the curing of concrete.[17] The moist curing was specified as follows:

- Seven days of continuous curing for conventional concretes.
- Ten days of continuous curing for concretes when pozzolans in excess of 10 percent of the cement mass are used.
- The above curing periods may be reduced to the age when the concrete compressive strength reaches at least 70 percent of the specified strength for all structures, other than the top slabs of structures serving as finished pavements.

While the above 7- and 10-day moist-curing periods are appropriate, the provision of allowing the curing to end at the age when the jobsite concrete reaches 70 percent of the design strength is questionable for concrete walls, piers, abutments, columns, beams, barriers, and other components that will receive salt water splash, flow, and other exposures during their life.

With specified 28-day AASHTO compressive strengths at 4000 psi (27.6 MPa), the provision allows the curing to end when the strength is 2800 psi (19.3 MPa). This strength can easily be reached in 1 to 3 days with today's lower w/c required by the DOTs and AASHTO. While decks still require 7 to 10 days of moist curing, these other members such as columns, piers, walls, abutments, dividers, and barriers, which will also receive chlorides in their service life, can be put into service with minimal curing.

The steam or radiant heat curing was specified as follows:

- The steam-cured or heat-cured members shall be protected from freezing until 7 days after casting.
- The steam-cured or heat-cured members that will be exposed to salt water shall be kept wet for not less than 7 days including the heat-curing period. Otherwise, additional moist curing is not required.

The above two AASHTO requirements for steam-cured or heat-cured concretes were inconsistent with the published research data from the 1960 PCA freezing and thawing study[2] and the 1984 to 1987 FHWA study[1] on corrosion and chloride permeability of moist-cured and heat-cured AASHTO-grade 0.44 w/c concrete following severe 1-year salt water cycle tests. These previous studies indicate no need for 7 days of protection from freezing weather nor 6 days of additional wet curing following heat curing.

The supplemental wet or moist curing was in fact slightly detrimental in the 1960 PCA[2] study of air-entrained concretes because the concrete was somewhat wetter when subjected to the freezing and thawing tests. The 1987 FHWA study[1] indicated clearly that the chloride permeability of heat-cured concrete was about 50 percent less when compared to 3-day moist-cured concretes after a severe 1-year cyclic salt water exposure.

The Part 2 report of this study found similar improved permeability performance of heat-cured AASHTO-grade concretes over AASHTO-grade concretes cured in water or under wet burlap for 7 days during subsequent 1-year exposure to continuous salt water ponding.

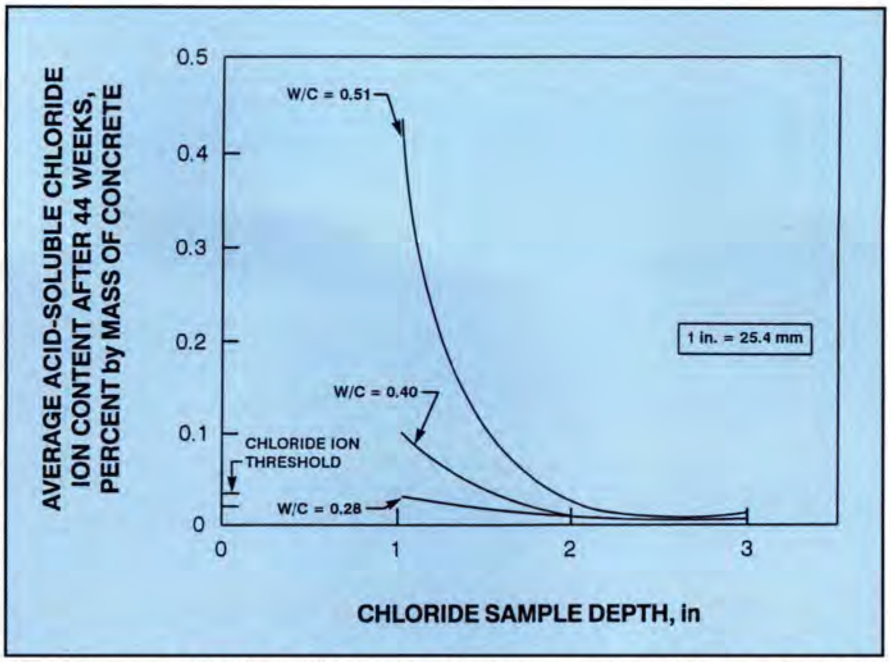

Fig. 1. Measured chloride profiles in moist-cured concretes from FHWA study (Ref. 1).

AASHTO (1992)

The 1992 AASHTO Standard Specification for Highway Bridges, Division II — Construction, Section 8.11 "Curing Concrete" has removed the requirements for 6 days of supplemental wet curing and no exposure to freezing conditions for 7 days in Subsection 8.11.3.5 "Steam or Radiant Heat Curing Method."[18] The 1989 provisions[17] for allowing jobsite curing to be discontinued when jobsite concrete reaches 70 percent of the specified strength are still present in 1992.

PERMEABILITY ASPECTS

The foregoing studies have dealt with effects of steam or heat curing on compressive strength and resistance to freezing and thawing. None studied the chloride ion permeability and corrosion protection offered by properly heat-cured concrete. The following discussions provide chloride ion permeability data from moist-cured and properly heat-cured concretes from 1984 to 1995.

While the 1992 AASHTO[19] and 1995 ACI 318[20] specification requirements call for maximum 0.45 and 0.40 w/c, respectively, for corrosion protection purposes, published data on moist-cured concrete demonstrate much better corrosion protection at lower w/c.

Low w/c precast concrete has been produced for decades, in most cases with 0.30 to 0.40 w/c and proper heat curing. These very low w/c concretes can be easily handled in precasting plants because the mixing and casting time periods are very short. The longer mixing, hauling and casting time requirements can make these same concretes more difficult to handle, finish and cure in cast-in-place concrete operations due to slump and air content losses, stickiness, and lack of bleeding.

Pfeifer et al. (1984 to 1987)

Between 1984 and 1987, an FHWA research project[1,21] on moist-cured and heat-cured conventional concretes was undertaken. Fig. 1 shows the average measured chloride ion content profiles after 44 weeks of testing from 90 conventional concrete slabs that were given 3 days of moist curing and had 0.51, 0.40, and 0.28 w/c. The corrosion threshold for reinforcing steel of 0.025 to 0.040 percent acid-soluble

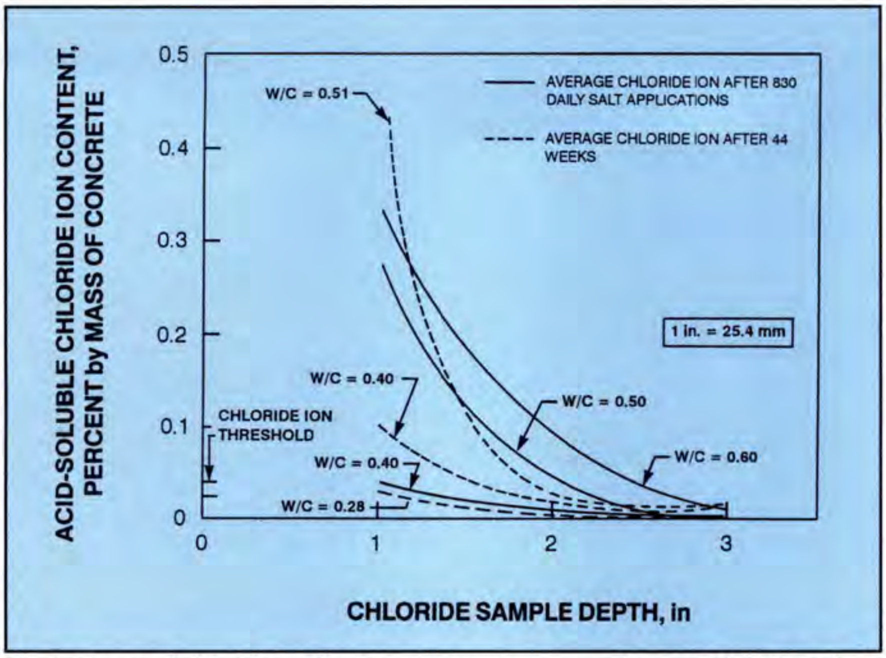

Fig. 2. Measured chloride profiles in moist-cured concretes from two FHWA studies (Refs. 1 and 22).

Table 5. 1-year chloride ion content of moist-cured and heat-cured specimens.

Member type	Cure type	Concrete type	Number of cores	1-year chloride ion content (percent by mass of concrete) ½ in. (13 mm)	1 in. (25 mm)	1¾ in. (44 mm)
Columns	Moist	Conventional	16	0.571	0.300	0.027
Beams	Moist	Conventional	12	0.533	0.263	0.009
Columns	Moist	Calcium nitrite	4	0.608	0.319	0.016
Bridge deck	Heat	Conventional	4	0.347	0.119	0.004
Bridge deck	Heat	Calcium nitrite	2	0.435	0.161	0.004

chloride ion content by mass of concrete determined in this study is also shown in Fig. 1. These data were generated during an indoor accelerated laboratory study using a 15 percent NaCl solution applied to the slabs for a 4-day period each week at about 60 to 80°F (16 to 27°C), followed by a 3-day air-drying period per week at 100°F (38°C).

Similar measured chloride ion content profiles after 2.3 years of outdoor FHWA corrosion studies in Virginia were reported in 1976.[22] These 7-day moist-cured conventional concrete slabs had w/c of 0.60, 0.50, and 0.40 and were ponded with 3 percent NaCl solutions on a daily basis for 830 days.

Fig. 2 depicts the measured chloride ion content profiles from the FHWA outdoor tests as compared to those of the indoor FHWA study shown in Fig. 1. A review of the data from these two studies reveals:

- The studies produced similar chloride ion content profiles at a given w/c at the conclusion of their long-term testing. However, higher chloride contents at the 1 in. (25 mm) depth were noted at the conclusion of the 1-year cyclic wet/dry test used during 1984 to 1987.
- With both studies, the 0.40 w/c concrete at the 1 in. (25 mm) depth absorbed only 14 to 20 percent of the chloride compared to the companion 0.50 w/c concrete.
- The 0.28 w/c concrete exhibited a chloride content at the 1 in. (25 mm) depth level that was only 5 percent of that in the 0.51 w/c ratio concrete tested in the 1984 to 1987 study. These two concretes had 28-day strengths of about 7500 and 5000 psi (51.7 and 34.5 MPa), respectively, yet a 95 percent reduction in chloride was measured between these two concretes.

Both of these long-term corrosion studies demonstrate that very low 0.30 to 0.40 w/c moist-cured concretes can dramatically reduce the chloride ion ingress and, consequently, significantly reduce the risk of steel corrosion when compared to conventional 0.45 to 0.50 w/c ratio moist-cured concretes.

None of the many corrosion studies reported before 1987 properly investigated the actual chloride permeability of heat-cured or steam-cured concrete. The 1984 to 1987 FHWA study[1] included a 1-year actual chloride permeability and corrosion study on properly heat-cured concrete vs. 3-day moist-cured concretes. The sponsors of this study specifically requested this comparison because there was a complete lack of measured chloride permeability data on heat-cured, low w/c concretes. This FHWA study on 19 relatively full-sized beams, columns, piles, and subdeck panels included a 1-year test series that compared 0.44 w/c heat-cured and moist-cured concretes. The 28-day strengths were approximately 6000 psi (41.4 MPa).

The full-sized columns and beams were moist cured for 3 days, while the full-sized precast, prestressed piles and bridge deck subpanels were only heat cured overnight at 130 to 140°F (54 to 60°C) for their total curing. The heat-cured members did not receive any supplemental moist curing after the overnight heat curing. These comparison tests were made with conventional concrete and with concrete containing a calcium nitrite corrosion inhibitor. The full-sized members were all cast with the 0.44 w/c required by AASHTO in 1984.

The concrete used in these members contained a nominal 6 bags per cu yd (334 kg/m³) Type I cement content, and a 6 ± ½ percent air content, and had a 3 to 5 in. (75 to 125 mm) slump. The calcium nitrite dose was 5.4 gal per cu yd (27 liters/m³). The ASTM C 403 time of initial setting for the conventional concrete was about 4 hours.

The specimens were subjected to a

1-year wetting and drying cycle consisting of 4 hours per day under a flowing 15 percent NaCl solution followed by normal laboratory air drying for 20 hours a day, all at 60 to 80°F (16 to 27°C). At the end of this 1-year test, the chloride ion contents were measured from cut slices centered on 0.50, 1.00, 1.75, 2.50 and 3.25 in. (13, 25, 44, 64 and 83 mm) depths from duplicate cores. The results are given in Table 5.

A plot that compares the 28 chloride ion contents from the moist-cured conventional concrete columns and beams vs. the four heat-cured conventional concrete bridge deck panel chloride ion contents is shown in Fig. 3. This plot and the other chloride data clearly show that the heat-cured conventional and heat-cured calcium nitrite concretes have substantially lower chloride ion permeability at the 1/2, 1, and 1 3/4 in. (13, 25, and 44 mm) depth levels compared to identical 3-day moist-cured concrete. Table 6 shows the percentage reductions in chloride achieved by the heat-cured concretes.

These data show that at the 1 in. (25 mm) depth level after a severe 1-year cyclic test, the heat-cured concrete had about 50 to 60 percent less chloride than the same moist-cured concrete, with either 0.44 w/c conventional or calcium nitrite concretes. At the 1/2 in. (13 mm) depth, the chloride reductions were about 30 to 40 percent.

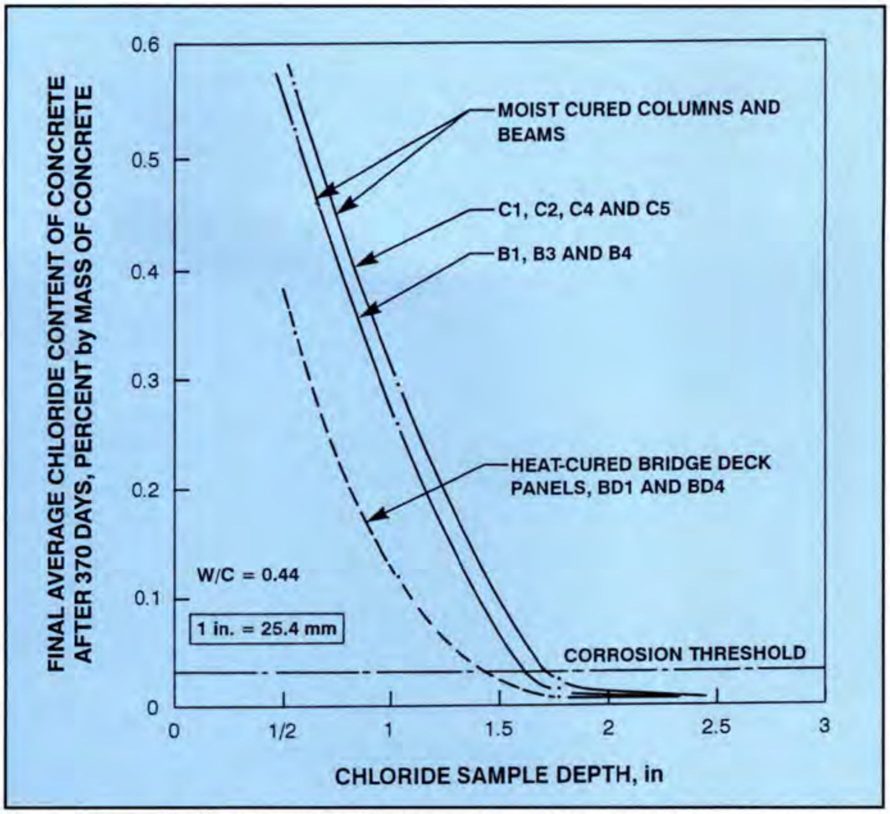

Fig. 3. Measured chloride profiles in moist-cured and heat-cured concretes from FHWA study (Ref. 1).

Table 6. Comparison of percent reduction of chloride levels in heat-cured concrete compared with moist-cured concrete.

Member type*	Concrete type*	Percent reduction in chloride in heat-cured concrete when compared to moist-cured concrete: 1/2 in. (13 mm)	1 in. (25 mm)
Columns	Conventional	39	60
Columns	Calcium nitrite	28	50
Beams	Conventional	35	55

* Moist-cured member type.

The Coulomb Test (1983 to 1995)

1983 — The AASHTO Test Method T277, "Rapid Determination of the Chloride Permeability of Concrete,"[23] was adopted in 1983. Virtually the same test procedure was designated in 1991 by ASTM as ASTM C1202, "Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration."[24]

1988 — During the late 1980s and early 1990s, project specifications were starting to limit concrete mixture proportions for corrosive environments to those with AASHTO T277 or ASTM C1202 coulomb values less than 1000, based on the Table 1 "Coulomb passed" ratings in AASHTO T277 and ASTM C1202.

At the same time, an ACI paper was published in 1988.[25] This paper contained estimated chloride gradients in a parking deck at age 40 years, and also for concrete piles in a marine environment at age 50 years for 0.45, 0.40, and 0.35 w/c concretes and 600 and 300-coulomb-rated concretes. The estimated chloride profiles in the garage are shown in Fig. 4. These estimated chloride gradients, based on Fick's law of diffusion, and the assumed constant 30 lb per cu yd (18 kg/m^3) concentration of chloride ion on the exterior surface, showed that conventional 0.35 w/c moist-cured concretes have reasonably similar estimated chloride gradients to a "600-coulomb" moist-cured concrete.

In November 1988, a document[26] based on the December 1988 ACI paper was distributed. This document contained the same estimated chloride gradients for the parking deck at 40 years and other estimated chloride gradients at 15, 40 and 75 years for 0.35 and 0.40 w/c ratio conventional moist-cured concretes and the "600-coulomb-rated" concrete. These estimated chloride gradients also showed that the 0.35 w/c ratio conventional concrete was reasonably similar to the "600-coulomb-rated" concrete at 40 years, and that both concretes contained large quantities of chloride at 40 years, as shown in Table 7.

All of these estimated chloride val-

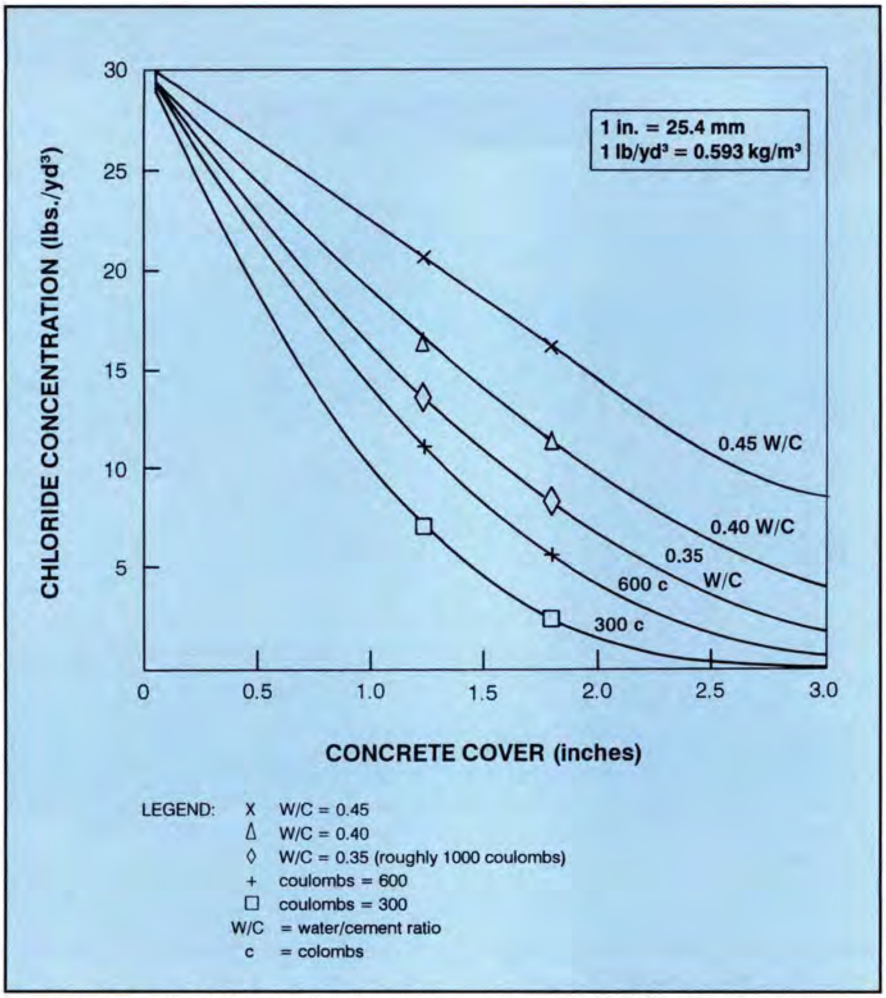

Fig. 4. Estimated chloride profiles in moist-cured concretes with various water-cement ratios and coulomb values, at age 40 years in a parking garage environment (Refs. 25 to 27).

Table 7. Estimated chloride content for moist-cured 0.35 w/c and 600-coulomb concretes.

Concrete type	Estimated chloride content at 40 years, lbs per cu yd (kg/m^3)		
	1 in. (25 mm)	$1^1/_2$ in. (38 mm)	2 in. (51 mm)
0.35 w/c	16.5 (9.8)	10.8 (6.4)	6.6 (3.9)
600-coulomb	13.7 (8.1)	7.6 (4.5)	4.0 (2.4)

Table 8. Estimated chloride content for moist-cured 0.35 w/c and 1000-coulomb concretes.

Concrete type	Estimated chloride content at 40 years, lbs per cu yd (kg/m^3)		
	1 in. (25 mm)	$1^1/_2$ in. (38 mm)	2 in. (51 mm)
0.35 w/c	16.5 (9.8)	10.8 (6.4)	6.6 (3.9)
1000-coulomb	16.2 (9.6)	10.5 (6.2)	6.3 (3.7)

ues far exceed the corrosion threshold for black reinforcing steel of about 1 to 2 lb per cu yd (0.6 to 1.2 kg/m^3).[1]

1990 — In late 1990, another document[27] was distributed that presented the same chloride gradients previously published[25,26] but included a "1000-coulomb" concrete estimated chloride gradient as shown in Fig. 5. This figure showed that the hypothetical 0.35 w/c ratio conventional concrete was essentially the same as the "1000-coulomb" concrete in the estimated chloride gradient at age 40 years. These 1000-coulomb concrete data show that the estimated chloride contents are essentially the same as 0.35 w/c conventional concrete and all are very high, as shown in Table 8.

These various estimated chloride content plots in Figs. 4 and 5 did not indicate the measured "coulomb ratings" for the hypothetical 0.45, 0.40, and 0.35 w/c ratio conventional concretes. A review of the December 1988 paper[25] shows that the tested 0.37 and 0.38 w/c conventional concretes had "coulomb values" of 2440, 2868, and 3485 — values much greater than 1000. These data suggest that a 0.35 w/c conventional concrete will not have a coulomb value of 1000, as suggested in these previous documents.[25,26,27]

The 1994 to 1995 tests performed during this present PCI-funded permeability study, as discussed in Part 2 of this report, substantiate the above observations.

1992 to 1995 — Routine testing in the early 1990s on properly heat-cured conventional concretes with w/c of 0.30 to 0.37 resulted in coulomb values in excess of 1000. Typical measured coulomb values were 1500 to 2500. These high quality conventional concretes would not meet project specifications requiring 1000 coulomb values. During the early 1990s, significant papers[28,29,30] from the United States, Spain, and Denmark were critical of the 6-hour "coulomb" test method. The author of the 1981 FHWA report[31] on the development of the coulomb test procedure was co-author of a follow-up report in 1992.[28] The following are quotations from this paper:

- Many users of the method believe that these values represent a large data base of concrete tests and are typical of what to expect in testing concretes of the types described. In fact, the table was constructed from results obtained on single cores of each concrete type, taken from the slabs originally supplied by the FHWA. As a further caution, in Appendix 1 of the FHWA report, the following advice is given: "The effect of such variables as aggregate type and size, cement content and composition, density, and other factors have not been evaluated. We recommend that persons using this procedure prepare a set of concretes from local materials and use these to establish their own correlation between charge passed and known chloride permeability for their own particular materials."

- A word of caution is advised, however, as the quantity measured by the RCPT is not permeability in the strictest sense, but an indication of permeability based on the ability of a given concrete specimen to conduct electric current. Any materials that cause concrete to be more (or less) conductive will increase (or decrease) the value obtained using the RCPT, irrespective of the effects which such materials or treatments have on actual permeability, diffusion, or other mass transport phenomena.
- In the authors' opinion, further work on definition of acceptable limits, on development of statistical acceptance schemes, and on improvement in the precision of the test must be done before this technique can be equitably applied to acceptance of silica fume and other types of concretes. Users must also recognize that chloride permeability depends not only on the mix design and the component materials, but also on aspects of construction such as degree of consolidation and type and extent of curing.

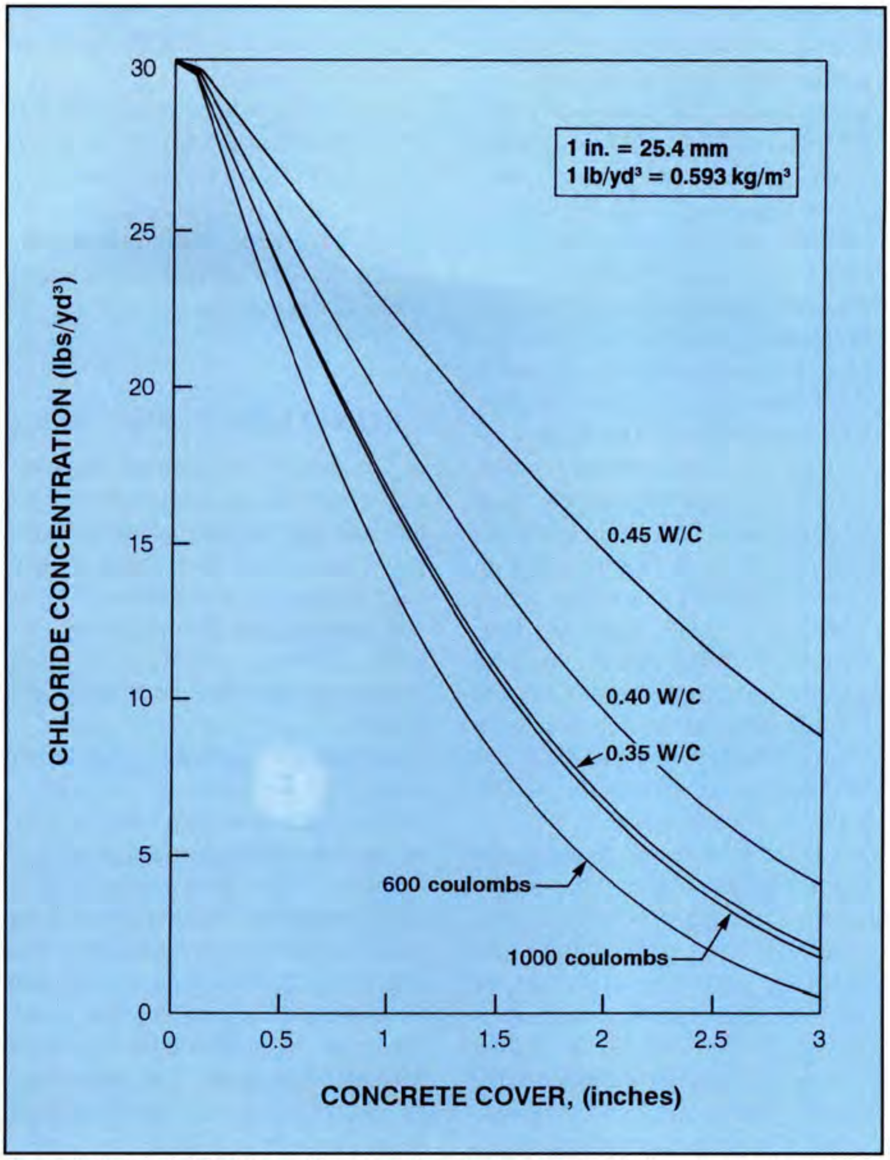

Fig. 5. Estimated chloride profiles in moist-cured concretes with various w/c and coulomb values, at age 40 years in a parking garage environment (Ref. 27).

Two of the authors of this paper were among the three authors of another critical paper[32] in 1994 that reviewed these other recently published papers and the original 1981 FHWA-funded study[31] that was used to develop the AASHTO T277 test method in 1983. As part of this review, the five papers[25,31,33,34,35] referenced in ASTM C1202,[24] which purportedly substantiated the use of the ASTM C1202 test method, and numerous other published papers that used the 6-hour coulomb test method to estimate broad chloride permeability classifications of concrete were examined.

The conclusions and recommendations from the present authors' 1994 paper[32] follow, because they attempt to explain the dilemma of this 6-hour rapid test method.

- Reliable and proper correlations do not exist between the 6-hour rapid chloride permeability test results and the 90-day ponding test results when different studies are compared. This lack of correlation is based upon numerous factors that are briefly discussed in this paper and more extensively discussed in other recent papers.
- The rapid test was never intended as a predictor of the quantitative amount of chloride that would penetrate into any given concrete. Those specifiers who are using the rapid test method for this purpose are at fault. As stated in ASTM C1202, the rapid test should not be used unless proper correlations are made with long-term ponding tests.
- Use of the rapid electrical test method to specify silica fume-modified and other pozzolan modified concrete, with their naturally high electrical resistivity, is premature. Adequate correlations, as required in ASTM C1202, between the rapid electrical tests method and the 90-day ponding tests do not exist for these concretes. Of great concern is the specification and use of higher w/c ratio concretes when based solely on the low "coulombs passed" values.
- Conventional concretes made with only portland cement may have coulomb values of 6 to 15 times higher than the same mixture with silica fume or slag cement. Much of

this difference is due to the inherent high electrical resistivity of these modified concretes. Typical conventional concrete may have a 5- to 10-fold decrease in coulombs passed when 7 percent silica fume is added, while the actual chloride ingress after 90-day ponding tests may decrease only one or two times.

- Chloride penetrability into concrete is dominated by the concrete w/c ratio, with additional benefits when silica fume, fly ash, latex and slag additions are used. The studies reviewed show that virtually impermeable conventional concretes can be produced with very low w/c ratios of 0.30 to 0.32, even though their coulomb values may range from 1000 to 5000. These data indicate that, during project bidding phases or during construction, the elimination of concretes with coulomb values of higher than 700 to 1000 based solely on ASTM C1202 is not appropriate.
- While further research regarding the general subject of chloride penetration of concrete is beneficial, it is essential in the case of the rapid chloride test. The concerns of ASTM C1202 regarding the correlation of the rapid chloride test and the 90-day ponding test for silica fume concrete have not been met adequately, making this application of the rapid chloride test highly questionable. Material selection for the design of low permeability concrete should be based on 90-day or longer ponding tests (AASHTO T259) and not ASTM C1202.
- Engineers continue to require rapid chloride tests of silica fume concrete, sometimes on a scale approaching that of routine jobsite quality control testing. Such indiscriminate use of the rapid chloride test — without development of initial correlation data on specific concretes — should be stopped.
- Table 1 in the ASTM C1202 specification should be removed because this "classification" system based upon coulombs passed values is incorrect and is not the intent originally proposed by the designers of the test procedure.

Since 1993, a number of other papers, articles, and letters[36-41] from the United States and other countries (South Africa, New Zealand, and Japan) have been distributed or published that are critical of the rapid coulomb test method.

CONCLUDING REMARKS

A review of the pertinent literature was performed to determine the history and past performance of concrete curing and composition effects as they have affected the performance of highway, parking, and other structural concrete systems exposed to large amounts of chloride, and freezing and thawing.

Historically, properly heat-cured concretes produced at low water-cement ratios have been found to have strength and frost resistance properties equal to or better than conventionally-cured concretes. When a proper heat curing procedure was followed, this improved durability was not found to be improved by supplemental moist curing of the precast concrete members after heat curing. The supplemental moist curing was only beneficial when improper heat curing was used.

A review of the effects of various parameters controlling the actual chloride permeability of concrete found that the most important aspect was the water-cement ratio. It was also found that the use of heat curing could reduce the permeability of AASHTO-grade 0.44 w/c concrete by 30 to 60 percent when compared to identical moist-cured concrete.

This decrease in chloride permeability was obtained with a concrete that received no supplemental moist curing after the heat-curing period, indicating that requirements for supplemental moist curing are unnecessary and probably undesirable. However, additional onsite curing of concrete beyond that required in the 1992 AASHTO curing specification should be required for moist-cured concrete to reflect the results of recent research.

This review of the AASHTO T277 test, and the similar ASTM C 1202 test, revealed that significant and serious questions remain regarding their appropriateness for use in concrete project materials qualifications and specifications. The correlation between long-term chloride permeability and results of the coulomb test appears to be highly variable and, as stated in ASTM C 1202, requires individual correlations between the tests for every concrete mixture.

The widely used 1000-coulomb limit for many specifications was found to be arbitrary for many concretes due to the widely different chloride permeabilities observed for concretes both meeting and failing such a limit-based specification. In fact, the coulomb test results often offered misleading and erroneous indications of chloride permeability. The test is known to be influenced by factors outside the concrete permeability. For example, the addition of other chemicals such as calcium nitrite is believed to increase the coulomb value, apparently without an increase in permeability.

These serious questions and their consequences resulted in the undertaking of a comprehensive long-term chloride permeability study of 7-day moist-cured concretes using two moist-curing techniques and overnight heat curing techniques with w/c of about 0.32, 0.37 and 0.46 that contain 0, 5 and 7.5 percent silica fume. The heat curing was limited to conventional concretes with no silica fume additions.

A total of 15 conditions were studied during 365-day constant salt water ponding according to AASHTO T259, water absorption and volume of permeable pore tests according to ASTM C642, and rapid chloride permeability tests according to AASHTO T277 and ASTM C1202. The results of this comprehensive laboratory study are presented in Part 2 of this report.

REFERENCES

1. Pfeifer, D., Landgren, J. R., and Zoob, A., "Protective Systems for New Prestressed and Substructure Concrete," FHWA Final Report No. FHWA/RD-86/193, Federal Highway Administration, April 1987, 126 pp.

2. Klieger, P., "Some Aspects of Durability and Volume Change of Concrete for Prestressing," *Journal*, PCA Research and Development Laboratories, V. 2, No. 3, September 1960, pp. 2-12. Also Research Department Bulletin 118, Portland Cement Association, Skokie, IL.

3. Hanson, J. A., "Optimum Steam-Curing Procedures in Precasting Plants," *ACI Journal*, V. 60, No. 1, January 1963. Also PCA Bulletin D62.

4. Hanson, J. A., "Optimum Steam-Curing Procedures for Structural Lightweight Concrete," *ACI Journal*, V. 62, No. 6, June 1965, p. 661. Also PCA Bulletin D92.

5. Hanson, J. A., "Prestress Loss as Affected by Type of Curing," PCI JOURNAL, V. 9, No. 2, April 1964, pp. 69-93. Also PCA Bulletin D75.

6. Pfeifer, D. W., "Maximum Strength of Heat-Cured Concrete in Relation to the Delay Period and Penetrometer Strength of Mortar Extract," Notes from PCI Energy Conservation in Curing, Precast/Prestressed Concrete Institute, Chicago, IL, March 1980.

7. Pfeifer, D. W., and Landgren, J. R., "Energy-Efficient Accelerated Curing of Concrete: A Laboratory Study for Plant Produced Prestressed Concrete," Technical Report No. 1, Precast/Prestressed Concrete Institute, Chicago, IL, December 1981.

8. Pfeifer, D. W., and Marusin, S. L., "Energy-Efficient Accelerated Curing of Concrete: A State-of-the-Art Review," Technical Report No. 1, Precast/Prestressed Concrete Institute, Chicago, IL, March 1981.

9. Pfeifer, D. W., and Landgren, R. J., "Energy-Efficient Accelerated Curing of Concrete for Plant-Produced Prestressed Concrete," PCI JOURNAL, V. 27, No. 2, March-April 1982, pp. 94-107.

10. Perenchio, W. F., and Klieger, P., "Some Physical Properties of High Strength Concrete," PCA Bulletin RDO56, Portland Cement Association, Skokie, IL, 1978.

11. Higginson, E. C., "Effect of Steam-Curing on the Important Properties of Concrete," *ACI Journal*, V. 58, No. 3, September 1961, pp. 281-298.

12. "Time of Setting of Concrete Mixtures by Penetration Resistance," ASTM Designation C 403, ASTM Annual Book of Standards, V. 4.02, American Society for Testing and Materials, Philadelphia, PA, 1995.

13. Brown, H. E., "An Investigation of the Delay Period and Related Factors in Steam-Cured Concrete," Virginia Council of Highway Investigation and Research and U.S. Department of Commerce, Washington, D.C., 1965.

14. Alexanderson, J., "Strength Losses in Heat-Cured Concrete," *Proceedings*, Swedish Cement and Concrete Research Institute, No. 43, Stockholm, Sweden, 1972.

15. AASHTO, "Accelerated Curing with Low Pressure Steam or Radiant Heat," *Standard Specifications for Highway Bridges*, Section 2.4.33, Prestressed Concrete, Subsection E, American Association of State Highway and Transportation Officials, Washington, D.C., 1977.

16. Soroka, I., Jaegermann, H., and Bentur, A., "Short-Term Steam Curing and Concrete Later-Age Strength," *Materials and Structures*, E & FN Spon, London, England, March-April 1978.

17. AASHTO, "Curing Concrete," *Standard Specifications for Highway Bridges*, Division II, Construction, Section 8.11, American Association of State Highway and Transportation Officials, Washington, D.C., 1989.

18. AASHTO, "Curing Concrete," *Standard Specifications for Highway Bridges*, Division II, Construction, Section 8.11, 15th Edition, American Association of State Highway and Transportation Officials, Washington, D.C., 1992.

19. AASHTO, *Standard Specifications for Highway Bridges*, 15th Edition, Washington D.C., 1992.

20. ACI Committee 318, "Building Code Requirements for Structural Concrete (ACI 318-95) and Commentary (ACI 318R-95)," American Concrete Institute, Farmington Hills, MI, 1995.

21. Pfeifer, D. W., Landgren, J. R., and Perenchio, W. F., "Concrete, Chlorides, Cover and Corrosion," PCI JOURNAL, V. 31, No. 4, July-August 1986, pp. 42-53.

22. Clear, K. C., "Time-to-Corrosion of Reinforcing Steel in Concrete Slabs: V. 3, Performance After 830 Daily Salt Applications," Report No. FHWA-RD-76/70, Federal Highway Administration, Washington, D.C., 1976.

23. "Rapid Determination of the Chloride Permeability of Concrete," AASHTO Designation T-277-93, American Association of State Highway and Transportation Officials, AASHTO Standard Specification — Part II Tests, Washington, D.C., 1993.

24. "Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration," ASTM Designation C1202-91, ASTM Standards Volume 04.02, American Society for Testing and Materials, Philadelphia, PA, 1992.

25. Berke, N. S., Pfeifer, D. W., and Weil, T. G., "Protection Against Chloride-Induced Corrosion," *Concrete International*, V. 10, No. 12, December 1988.

26. *Guide to Corrosion Protection Systems for Parking Structures*, Grace Engineering Services Group, Construction Products Division, W.R. Grace & Company, Cambridge, MA, November 1988.

27. *The Economics of Corrosion Protection Systems for Parking Structures*, Grace Engineering Services Group, Construction Products Division, W.R. Grace & Company, Cambridge, MA, November 1990.

28. Whiting, D., and Mitchell, T. M., "A History of the Rapid Chloride Permeability Test," Transportation Research Record 1335, Washington, D.C., 1992.

29. Andrade, C., "Calculation of Chloride Diffusion Coefficients in Concrete from Ionic Migration Measurements," *Cement and Concrete Research*, V. 23, 1993.

30. Arup, H., Sorensen, B., Frederiksen, J., and Thaulow, N., "The Rapid Chloride Permeation Test — An Assessment," presented at NACE Corrosion '93, New Orleans, LA, March 7-12, 1993.

31. Whiting, D., "Rapid Determination of the Chloride Permeability of Concrete," Final Report No. FHWA/RD-81/119, Federal Highway Administra-

tion, Washington, D.C., August 1981, NTIS No. PB 82130724.

32. Pfeifer, D. W., McDonald, D. B., and Krauss, P. D., "The Rapid Chloride Permeability Test and Its Correlation to the 90-Day Chloride Ponding Test," PCI JOURNAL, V. 39, No. 1, January-February 1994, pp. 38-47.

33. Whiting, D., "Permeability of Selected Concretes," *Permeability of Concrete*, SP-108, American Concrete Institute, Farmington Hills, MI, 1988.

34. Whiting, D., and Dziedzic, W., "Resistance to Chloride Infiltration of Superplasticized Concrete as Compared With Currently Used Concrete Overlay Systems," Final Report No. FHWA/OH-89/009, Construction Technology Laboratories, Skokie, IL, May 1989.

35. Ozyildirim, C., and Halstead, W. J., "Use of Admixtures to Attain Low Permeability Concretes," Final Report No. FHWA/VA-88-R11, Virginia Transportation Research Council, February 1988. Available as NTIS No. PB88201264 from National Technical Information Service, Springfield, VA.

36. Stretcher, P. E., and Alexander M. G., "A Chloride Conduction Test for Concrete," *Cement and Concrete Research*, V. 25, No. 6, 1995.

37. Torii, K., and Kawamura, M., "Pore Structure and Chloride Ion Permeability of Mortars Containing Silica Fume," *Cement and Concrete Composition*, V. 16, 1994.

38. Cement and Concrete Association of New Zealand, January 27, 1994 letter to ASTM Standards Coordination Committee regarding ASTM Designation C1202-91, negative ballot.

39. "Concrete Technology Today — Proper Use of RCPT Results," Portland Cement Association, Skokie, IL, November 1994.

40. Scanlon, J. M., and Sherman, M. R., "Use of Fly Ash to Improve Concrete Durability by Reducing Permeability," The Third National Concrete and Masonry Engineering Conference, San Francisco, CA, 1995.

41. Scanlon, J. M., and Sherman, M. R., "Use of Fly Ash to Improve Concrete Durability by Reducing Permeability," CANMET/ACI Fifth International Conference on Fly Ash, Silica Fume, Slag and Natural Pozzolans in Concrete, Milwaukee, WI, June 1995.

DISCUSSION NOTE

The Editors welcome discussion of reports and papers published in the PCI JOURNAL. The comments must be confined to the scope of the article being discussed. Please note that discussion of papers appearing in this issue must be received at PCI Headquarters by November 1, 1996.