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/m³) 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/m³) 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.

Delay period (hours)	Strength loss at 28 days* (percent) — Curing temperature, °F (°C): 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 ½-hour delay period.

Delay period (hour)	Strength loss at 28 days* (percent) — Curing temperature, °F (°C): 86† (30)	104† (40)	122† (50)	140† (60)	158† (70)	176† (80)	194† (90)	212† (100)
½	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 ½-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 ½ 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/m^3) 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.

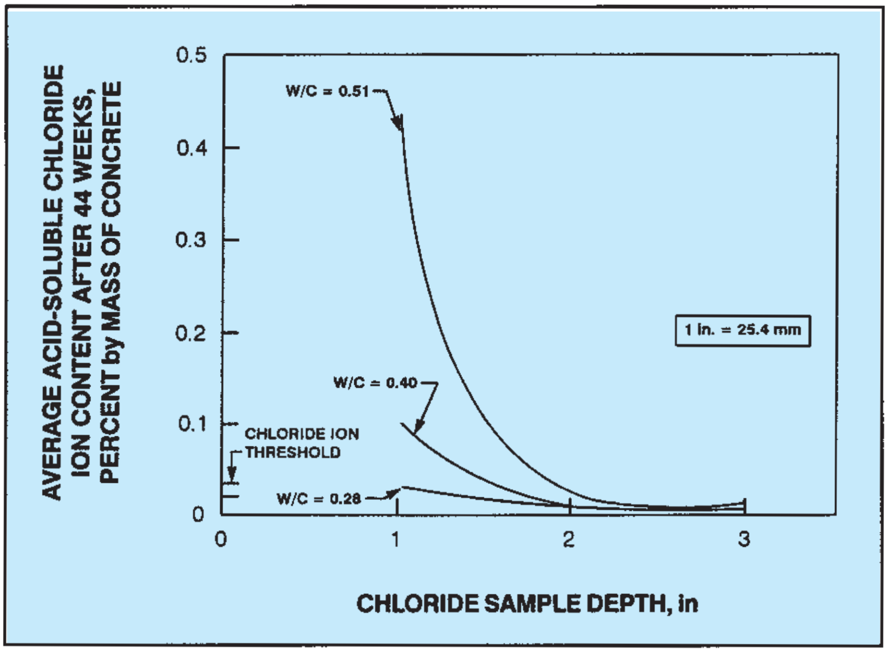

Fig. 1. Measured chloride profiles in moist-cured concretes from FHWA study (Ref. 1).

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.

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.

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.

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

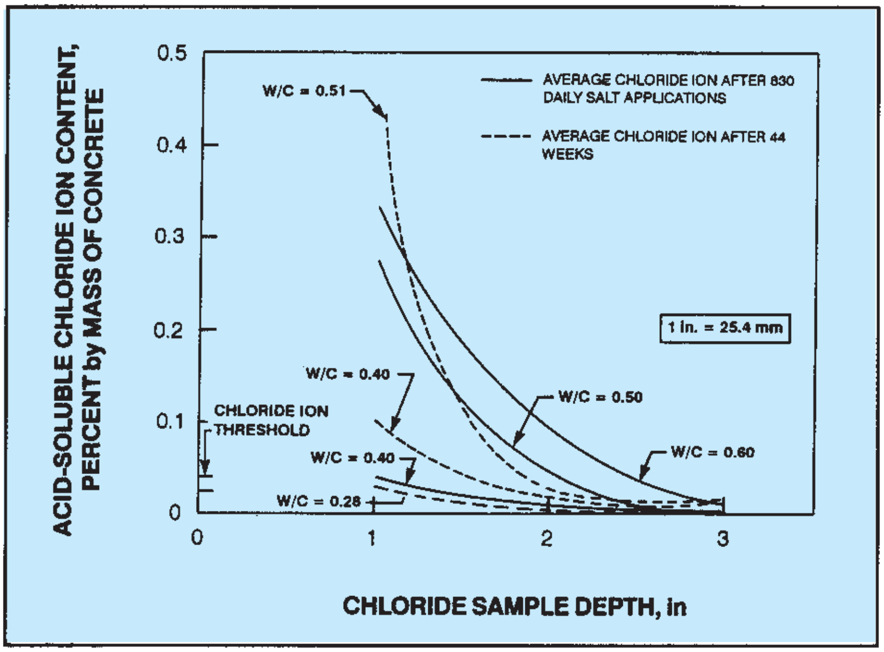

Fig. 2. Measured chloride profiles in moist-cured concretes from two FHWA studies (Refs. 1 and 22).

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/m^3) 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/m^3). 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 ½, 1, and 1¾ 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 ½ in. (13 mm) depth, the chloride reductions were about 30 to 40 percent.

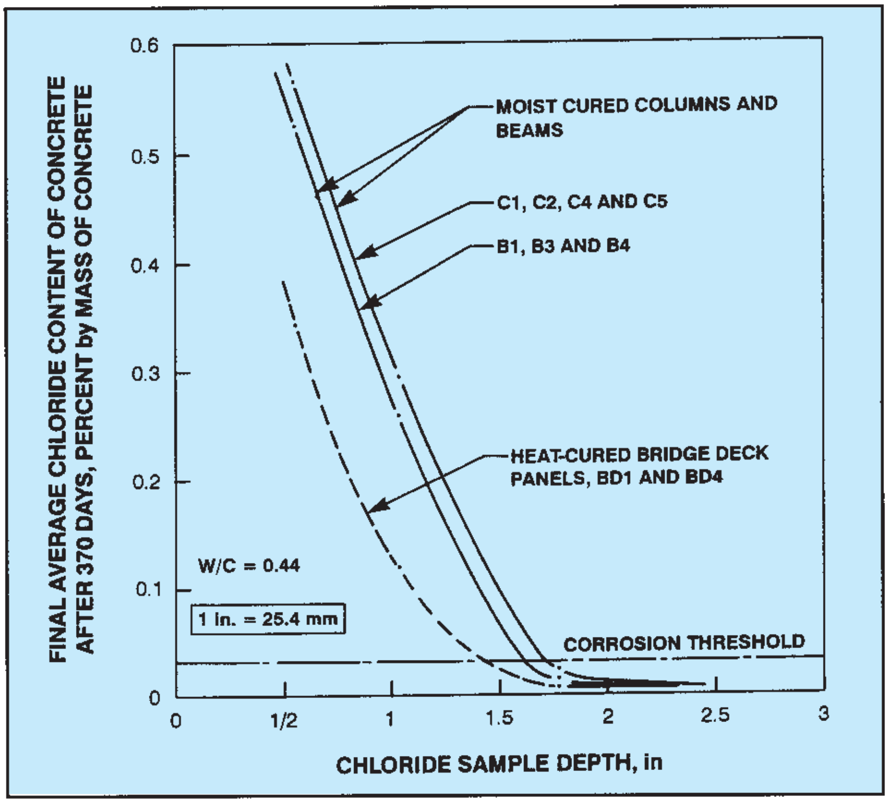

Fig. 3. Measured chloride profiles in moist-cured and heat-cured concretes from FHWA study (Ref. 1).

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: ½ 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/m³) 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-

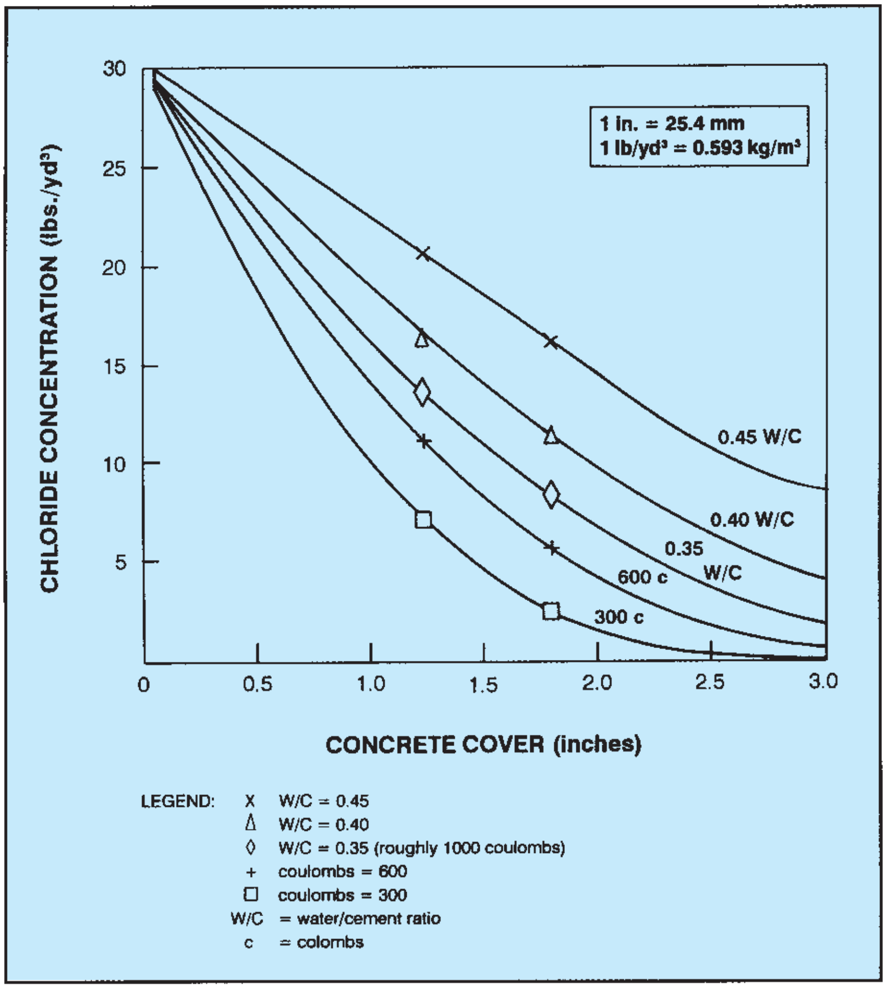

Fig. 4. Estimated chloride profiles in moist-cured concretes with various water-cement ratios and coulomb values, at age 40 years in a parking garage environment (Refs. 25 to 27).

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)	1½ 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)	1½ 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 coauthor 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.

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

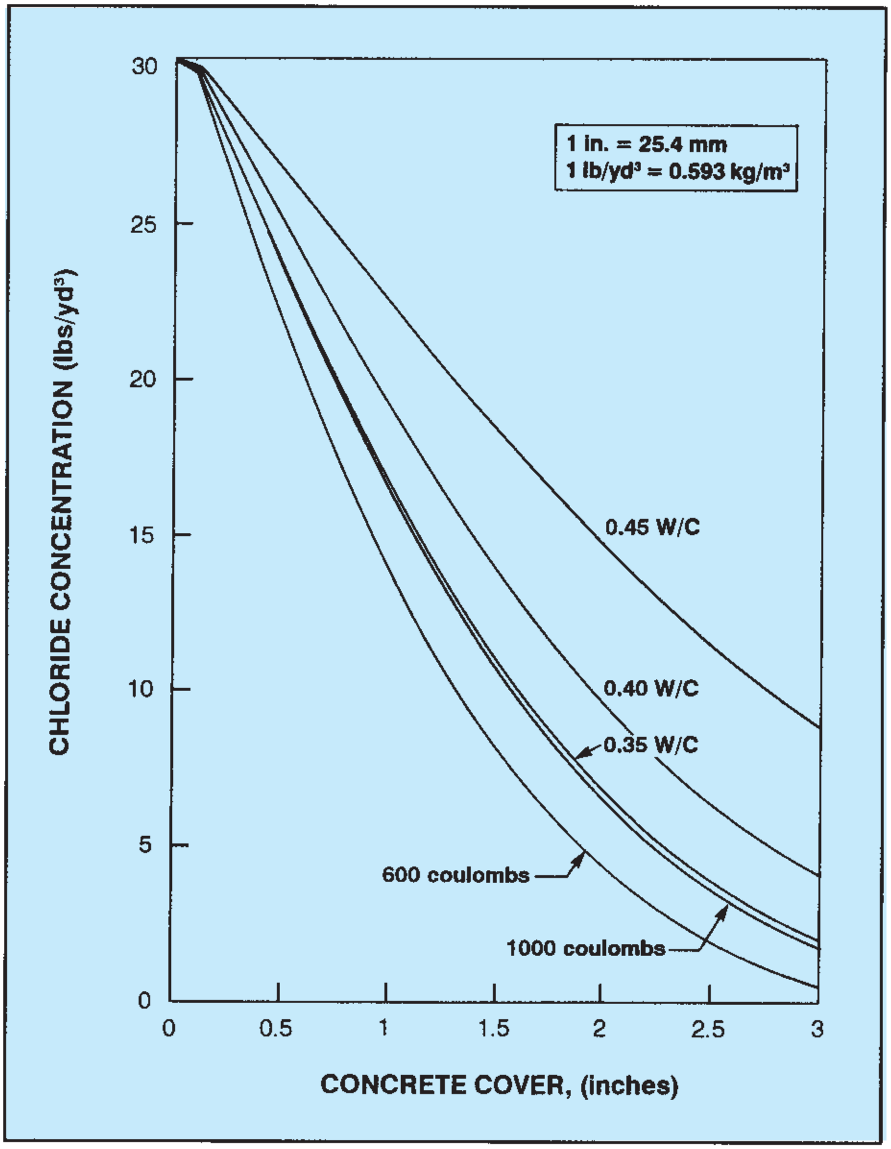

Fig. 5. Estimated chloride profiles in moist-cured concretes with various w/c and coulomb values, at age 40 years in a parking garage environment (Ref. 27).

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.

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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.

Durability Aspects of Precast Prestressed Concrete Part 2: Chloride Permeability Study

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A laboratory study was undertaken to investigate the good past performance of low water-cement ratio, heat-cured precast, prestressed concrete in highway bridges, parking garages, and other applications. The study included salt water ponding testing, AASHTO T 277 or ASTM C 1202 "coulomb" tests, compressive strength tests, and absorption and volume of permeable voids tests. Heat-cured, water-cured, and moist-cured concretes with water-cementitious ratio values of 0.46, 0.37 and 0.32 with and without silica fume were tested. Using the measured chloride contents, chloride diffusion coefficients were calculated and estimates of the time-to-corrosion were developed. The water-cement ratio was found to be the most important influence on the performance of the concrete, with low w/c, heat-cured conventional concretes having comparable performance to realistic silica fume concretes having 0.37 to 0.46 water-cementitious ratios. It was also found that the use of heat curing could reduce the permeability of AASHTO-grade, 0.46 w/c concrete by 40 to 50 percent. The addition of silica fume to concrete caused an increase in the absorption and volume of permeable voids in concrete, while heat curing was seen to decrease the absorption and volume of permeable voids in concrete.

The excellent durability of heat-cured, precast, prestressed concrete bridge and parking garage structures over the past 45 years, resulting from the use of water-cement ratios (w/c) between 0.30 and 0.40, is discussed in Part 1 of this report.[1]

In Part 1, a review of literature from 1960 to 1994 was performed to explain the history and past performance of precast, prestressed concrete highway, parking, and other structural concrete systems exposed to large amounts of chloride, and freezing and thawing. Essentially, all of these precast, prestressed concrete structural members were heat cured or steam cured without any in-plant supplemental moist curing following the overnight heat curing.

A 1987 Federal Highway Administration (FHWA) study[2] showed that the chloride permeabilities of heat-cured AASHTO-grade 0.44 w/c concretes with or without calcium nitrite were about 50 percent lower at the 1 in. (25 mm) depth, when compared to identical moist-cured conventional 0.44 w/c concrete after a severe 1-year cyclic salt water and air-drying test period on full-sized columns, beams, and bridge deck panels.

To verify this performance, the Precast/Prestressed Concrete Institute (PCI) funded this comprehensive 1-year laboratory study to answer questions relating to chloride permeability, water absorption, volume of permeable voids, compressive strength, coulomb values, diffusion coefficients, and times-to-corrosion for a wide range of heat-cured and moist-cured concretes.

The water-cement (w/c) values used for the conventional concretes were 0.46, 0.37, and 0.32, representative of typical AASHTO 0.45 w/c concrete, and of 0.37 to 0.32 w/c values commonly used in the precast concrete industry. Silica fume additions of 5.0 and 7.5 percent by mass of cement were also studied. The three conventional concretes were cured either in a water tank, under wetted burlap, or under wetted burlap in a heated chamber, while the silica fume concretes were cured under wetted burlap only.

While silica fume additions were used at all three water-cementitious materials ratios (w/cm), it is well recognized that silica fume concretes with 0.37 to 0.32 w/cm require more effort, experience, and knowledge to place and finish due to stickiness, slump loss, air loss, and a lack of bleed water, particularly when used for jobsite flatwork. In addition, if proper curing procedures are not followed, concretes containing silica fume with w/cm levels of less than 0.39 are more susceptible to cracking that has been ascribed to plastic shrinkage and self-desiccation.[3]

Field experience indicates that practical silica fume mixtures for cast-in-place concrete flatwork use w/cm values of about 0.40 to 0.45. Therefore, although this laboratory study used 0.32, 0.37, and 0.46 w/cm levels with both silica fume addition rates, the realistic corrosion performance comparisons should acknowledge that the lower 0.37 to 0.32 w/cm silica fume mixtures can be difficult to handle, consolidate and cure, especially with flatwork.

The main focus of the study was to determine chloride ingress of the various concretes subjected to salt water ponding. These tests were conducted using the AASHTO T 259 procedure,[4] except that the normal 90-day ponding period was increased to 365 days to provide more accurate chloride diffusion data. The 90-day period is too short to allow appreciable chloride ingress into these high quality concretes and to allow the calculation of diffusion coefficients. AASHTO T 277 or ASTM C 1202 "coulomb" tests,[5,6] ASTM C 32 compressive strength tests, and ASTM C 642 absorption and volume of permeable voids tests[7] were also performed.

MIXTURE PROPORTIONS AND SPECIMEN PREPARATION

Fifteen concrete conditions were tested to determine the influence of curing and silica fume additions on concrete permeability. When silica fume was used, the portland cement content was unchanged and silica fume solids were added to the constant cement contents at 5.0 and 7.5 percent by mass of cement. The testing matrix consisted of five groups of mixes, each tested at three different w/cm. These groups are:

- Conventional concrete — tank cure
- 5 percent silica fume concrete — burlap cure
- 7.5 percent silica fume concrete — burlap cure
- Conventional concrete — burlap cure
- Conventional concrete — heat cure

Materials

The same aggregate, sand, cement, and silica fume were used for all of the mixtures. The cement was LaFarge Type I. The high-range water-reducing admixture (HRWRA) was WRDA-19 and the air-entraining admixture (AEA) was Daravair. The coarse aggregate was a chloride-free river gravel from Eau Claire, Wisconsin, with a nominal maximum size of 3/4 in. (19 mm). The fine aggregate was a river sand, also from Eau Claire, Wisconsin. The silica fume was Force-10,000, supplied in a densified powder form.

During batching, the silica fume was premixed with an equal mass of water using a high speed electric mixer to form a slurry. The slurry was slowly added to the concrete during mixing. The water mixed with the silica fume was accounted for in the batch quantities. The mixes were cast in groups, with all three w/cm concretes of a given group being cast during the same morning.

Mixture Proportions and Plastic Concrete Characteristics

The concrete proportions are shown in Table 1, as well as the measured slump, air, and unit weight. The aggregate moisture contents were determined immediately prior to casting. All quantities are the saturated surface-dry (SSD) quantities. The quantities have been corrected to account for the water present in the AEA dilution, but not for the water in the original AEA or HRWRA. All mixtures were proportioned to contain the same amount of coarse aggregate, compensating for changes in cement content by changing the amount of fine aggregate.

Table 1. Mixture proportions (SSD quantities lb per cu yd) and unhardened concrete properties.

Curing type	Tank cure			5 percent silica fume burlap cure			7.5 percent silica fume burlap cure			Burlap cure			Heat cure		
Mixture number	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
Cement	808	679	527	796	689	540	805	691	540	793	694	535	802	686	543
Silica fume	0	0	0	40	34	27	60	52	40	0	0	0	0	0	0
Water	253	248	241	273	271	263	284	275	268	251	254	245	254	252	251
Sand	1258	1331	1445	1161	1278	1426	1136	1247	1393	1235	1361	1475	1249	1345	1498
3/4 stone	586	575	570	577	583	588	584	586	588	575	588	582	582	581	591
1/2 stone	659	646	641	649	656	661	657	659	661	647	661	655	654	653	665
3/8 stone	416	408	404	409	414	417	414	415	417	408	417	413	413	412	419
AEA (oz/cwt)*	2.2	2.6	1.9	4.9	4.5	2.6	5.1	3.6	2.3	2.8	2.8	1.5	2.8	3.1	2.2
HRWRA (oz/cwt)*	25.6	15.1	13.9	22.8	16.7	12.9	29.2	25.2	9.9	19.8	14.8	7.6	11.5	15.0	16.2
w/cm	0.31	0.37	0.46	0.33	0.38	0.46	0.33	0.37	0.46	0.32	0.37	0.46	0.32	0.37	0.46
Air (percent)	5.25	6.00	6.30	5.60	5.80	6.00	6.00	6.10	6.00	7.20	5.50	6.30	6.20	6.60	5.60
Slump (in.)	5.75	5.50	6.50	2.75	3.00	3.25	5.00	5.75	5.75	6.25	4.00	4.25	6.25	6.25	4.75
Unit weight (lbs/cu ft)	147.4	143.9	141.7	144.6	145.4	145.2	146.0	145.4	144.7	144.7	147.3	144.6	146.4	145.5	147.0

Note: 1 in. = 25.4 mm; 1 lb per cu ft = 16.018 kg/m^3; 1 lb per cu yd = 0.593 kg/m^3; 1 oz/cwt = 65.198 mL/100 kg.

* Computed based on the mass of cement.

Table 2. Cement content for concrete mixtures without silica fume.

Average w/cm	Coefficient of variation (CV) of w/cm (percent)	Portland cement content bags/cu yd	Portland cement content kg/m^3	CV of portland cement content (percent)
0.32	2.6	8.52	476	0.8
0.37	1.2	7.32	409	0.8
0.46	0.0	5.70	318	1.4

Table 3. Slump, air content and unit weight of concrete mixtures.

	Average	Range
Slump, in. (mm)	5.00 (127)	+1.25, -2.25 (+32, -57)
Air content, percent	6.03	+1.17, -0.78
Fresh unit weight, lb per cu ft (kg/m^3)	145.4 (2329)	+2.0, -3.7 (+32, -59)

The mixtures without silica fume were proportioned to have essentially the same water content while the cement contents varied according to the w/cm, as shown in Table 2.

The average water content of the mixtures and their coefficient of variation (CV) were as follows:

- Conventional = 30.0 gal per cu yd (149 liters/m^3), 1.8 percent CV
- 5 percent fume = 32.3 gal per cu yd (159 liters/m^3), 2.0 percent CV
- 7.5 percent fume = 33.1 gal per cu yd (164 liters/m^3), 2.9 percent CV

The air contents, slumps, and plastic unit weights were carefully controlled, as shown in Table 3.

Mixing

The concrete was mixed in a horizontal rotary-pan mixer using a mixing sequence of 3 minutes, followed by 3 minutes of rest, and an additional 2 minutes of mixing. After the second mixing period, the concrete was tested to determine the air content, unit weight, and slump. If necessary, the AEA and HRWRA dosages were adjusted and the concrete remixed and retested.

Three 12 x 12 x 5 in. (300 x 300 x 125 mm) slabs and four 4 x 8 in. (100 x 200 mm) cylinders were cast from each batch, and a sample of mortar was sieved and retained for the ASTM C 403 time of setting tests.[8] For the heat-cured mixtures, an additional four cylinders were cast to determine the effect of the heat curing on concrete strength. All test slabs and cylinders from this single batch were compacted on a table vibrator and finished with a wooden float.

Curing and Specimen Preparation

The lime-saturated water and wet burlap cure duration was selected as 7 days in accordance with the 1992 AASHTO requirements in Section 8.11[9] and in accordance with the 1995 ACI requirements given in Section 5.11 of ACI 318.[10]

The overnight heat cure, followed by no moist curing, was in accordance with the 1992 AASHTO Section 8.11.3.5 requirements, which do not require any moist curing after the overnight heat curing. This represents the typical heat curing that has taken place for decades in precast concrete plants. A supplemental 6-day moist curing was required in the 1989 AASHTO specification if the heat-cured concrete was to be eventually exposed to salt water; otherwise, no additional moist curing was required. The supplemental moist curing requirement in the 1989 specification was not previously required[11] and it has once again been eliminated in 1992.

For the first 24 hours after casting, the water tank-cured and burlap-cured specimens were left in forms and covered with wet burlap. When they were stripped, the burlap-cured slabs (including all of the silica fume mixtures) were wrapped in wet burlap and plastic and kept continuously wet until the concrete was 7 days old. The water tank-cured slabs were placed in lime saturated water for 3½ days after they were stripped. After 3½ days in the tank, they were removed and wrapped in wet burlap for another 2½ days.

After removing the burlap at 7 days, all of these slabs were transferred to a controlled climate room (CCR) held at 72°F (22°C) and 50 percent relative humidity until further testing. All of the cylinders for the water tank-cured and burlap-cured mixes were placed in the lime saturated water curing tank after they were stripped at an age of 1 day and remained in the tank until age 28 days, at which time they were tested or moved into the CCR until testing at 180 days.

In accordance with Section 3.4.2 of the PCI Manual for Quality Control,[12] the specimens to be heat cured were held under wet burlap and plastic sheeting at the laboratory temperature until the concrete had reached time-of-initial setting as defined in ASTM C 403. The waiting period ranged from 3.7 to 4.5 hours. The three slabs and four cylinders were then brought at a rate of 30°F per hour (17°C/hour) in a controlled-environment chamber to an air temperature of 145°F (63°C). The concrete was held at 145°F (63°C) for 7.5 hours, then allowed to cool to ambient temperature.

Table 4. Time of setting of concrete mixtures.

Cure and mixture type	Mixture number	As-mixed w/cm	Time of setting (hours)	
			Initial	Final
Tank cure	1	0.31	7.1	—
	2	0.37	7.7	—
	3	0.46	—	—
Burlap cure 5 percent silica fume	4	0.33	6.0	—
	5	0.38	4.7	—
	6	0.46	4.4	—
Burlap cure 7.5 percent silica fume	7	0.33	5.3	—
	8	0.37	4.7	—
	9	0.46	3.7	—
Burlap cure	10	0.32	7.3	—
	11	0.37	5.5	—
	12	0.46	4.7	—
Heat cure	13	0.32	4.5	5.7
	14	0.37	4.2	5.2
	15	0.46	3.7	4.6

Twenty-four hours after they were cast, all of the heat-cured slabs and cylinders were moved into the CCR without any further moist curing. For direct comparison to the strength cylinders cast from the other mixtures, four cylinders from the heat-cured mixtures were allowed to cure at room temperature in their molds for 24 hours, after which they were placed into lime-saturated water, where they remained until age 28 days.

At 28 days, the slabs were lightly sandblasted to remove surface laitance. Acrylic plastic dikes were attached to the surfaces of two of the three slabs and their sides were coated with a two-part epoxy. At an age of 37 days, two 4 in. (102 mm) diameter by 5 in. (127 mm) long cores were removed from the third undiked slab for later use in the rapid chloride permeability testing (AASHTO T 277 or ASTM C 1202).

TEST RESULTS

Tests performed on the unhardened and hardened concrete included time of setting, compressive strength, absorption and volume of permeable voids, AASHTO T 277 or ASTM C 1202 coulomb testing, and long-term chloride content at different depths.

Time of Setting

All concrete mixtures were tested according to ASTM C 403, "Standard Test Method for Time of Setting of Concrete Mixtures by Penetration Resistance." Only the times of initial setting were generally determined, as shown in Table 4. The times of initial setting of Mixtures 13, 14, and 15 were considerably shorter than the companion mixes without silica fume. This was due to the higher ambient temperature that day. The times of initial setting for all of the 0.32 w/cm mixtures were significantly longer than the 0.37 or 0.46 w/cm concretes due to the use of larger dosages of HRWRA required to achieve the lowest w/cm.

Compressive Strength

Cylinders from all 15 mixtures were tested to determine their 28-day, water-cured compressive strengths, as well as their 180-day strengths. These 60 test cylinders were all cured in the lime water tank until a concrete age of 28 days, except for four extra heat-cured cylinders cast from each of Mixtures 13, 14, and 15 that were placed into the CCR immediately after being removed from the heat-curing chamber. These 12 heat-cured cylinders were tested to determine if the preset period was sufficient to prevent 28-day strength damage to the concrete due to the heating process. To properly

Table 5. Compressive strength test results.

Cure and mixture type	As-mixed w/cm	Average 28-day compressive strength (psi)		Average 180-day compressive strength (psi)	
		Tank cure	Heat cure	Tank cure	Heat cure
Tank cure	0.31	6960	—	9330	—
	0.37	6520	—	7640	—
	0.46	5170	—	5910	—
Burlap cure 5 percent silica fume	0.33	7280	—	9050	—
	0.38	5370	—	7160	—
	0.46	5250	—	6110	—
Burlap cure 7.5 percent silica fume	0.33	8060	—	8650	—
	0.37	7200	—	8190	—
	0.46	5270	—	5770	—
Burlap cure	0.32	6880	—	9150	—
	0.37	6960	—	6880	—
	0.46	5390	—	6260	—
Heat cure	0.32	6560	6050*	7880	6600
	0.37	6560	5630*	6320	6440
	0.46	5590	5090*	6560	5170

Note: 1000 psi = 6.895 MPa.

* 10 percent less than measured strength.

account for the dry condition of the heat-cured cylinders at the time of testing, the 28-day measured strengths have been reduced by 10 percent.[13] The test results are listed in Table 5.

All of the 28-day strengths were over 5000 psi (34.5 MPa), with the lower w/cm concretes having the higher strengths, as expected. The effects of the silica fume addition on the 28-day strengths were varied. As compared to the burlap-cured conventional concrete control specimens, the addition of 5 percent silica fume resulted in 28-day strength changes of 5.8, -22.8, and -2.6 percent for the 0.33, 0.38, and 0.46 w/cm concretes, respectively. The poor performance of the 0.38 w/cm 5 percent silica fume is unexpected and unusual. This mixture did not exhibit any similarly unexpected results in other tests conducted in the program.

The addition of 7.5 percent silica fume resulted in strength changes of 17, 3.5, and -2.2 percent for the 0.33, 0.37, and 0.46 w/cm concretes, as compared to the burlap-cured conventional concrete specimens. The negative effect of the silica fume addition to the 0.38 and 0.46 w/cm mixtures is interesting, because the addition of silica fume material is typically expected to increase the concrete strength. Possibly, the additional water added to these silica fume mixtures to maintain the w/cm with the added silica fume material offset the normally expected strength increase. The 28-day strengths of all of the conventional water-cured mixtures were within 6 percent of the burlap-cured conventional concrete.

All of the 180-day strengths were over 5700 psi (39.3 MPa), with the lower w/cm concretes having the higher strengths, as expected. The effects of the silica fume addition on the 180-day strengths were again varied. As compared to the burlap-cured conventional concrete specimens, the addition of 5 percent silica fume resulted in 180-day strength changes of -1.1, 4.1, and -2.4 percent for the 0.33, 0.38, and 0.46 w/cm concretes, respectively. The addition of 7.5 percent silica fume resulted in strength changes of -5.5, 19.1, and -7.9 percent for the 0.33, 0.37, and 0.46 w/cm concretes, respectively, as compared to the burlap-cured conventional concrete specimens.

The overall negative effect of the silica fume addition to the 0.33 and 0.46 w/cm mixtures is again interesting, because the additional silica fume is typically expected to increase the concrete strength, especially at later concrete ages. The 180-day strengths of the tank-cured cylinders of the conventional concrete mixtures were within 11 percent of the nominally identical burlap-cured conventional concrete. This is a larger variation than was seen for the 28-day strengths.

The effect of heat curing was investigated by comparing the compressive strength test results from the heat-cured cylinders to the tank-cured cylinders from the same batch. Because normal weight concrete stored at 10 to 75 percent relative humidity after 7 days of initial moist curing can have 28-day compressive strengths about 10 percent higher than continuously moist-cured concrete,[13] the 28-day strengths of the heat-cured cylinder were reduced by 10 percent to account for the 27 days of air drying at 50 percent relative humidity following the overnight heat curing.

The 28-day strengths of the heat-cured cylinders (after 10 percent reduction) were 92, 86, and 91 percent of the companion tank-cured cylinders for the 0.32, 0.37, and 0.46 w/c concretes, respectively. The heat curing created no significant strength loss at 28 days and indicates that the preset period was appropriate. However, the 180-day strengths of the heat-cured specimens were 83, 102, and 79 percent of the tank-cured control concrete specimens for the 0.32, 0.37, and 0.46 w/c concretes, respectively.

Apparently, the tank-cured specimens continued to gain strength after being removed from the water curing tank at an age of 28 days, while the heat-cured specimens, when stored for 179 days in the controlled climate room drying environment at 73°F (22°C) and 50 percent relative humidity, remained at a nearly constant strength, with 180-day strengths 98, 103, and 92 percent of their 28-day strengths for the 0.32, 0.37, and 0.46 w/c concretes, respectively. Such long-term constant drying at 50 percent relative humidity would not occur outdoors and, therefore, these 180-day strengths are not typical as related to outdoor conditions.

Absorption and Volume of Permeable Voids

All 15 mixtures were tested according to ASTM C 642, "Standard Test Method for Specific Gravity, Absorption, and Voids in Hardened Concrete" to determine the water absorption after immersion, water absorption after immersion and boiling, and the volume of permeable voids in the concrete. This test was chosen to serve as an indicator of the short-term absorption characteristics of the concrete, as opposed to the long-term diffusion predominantly measured by the AASHTO T 259 long-term ponding tests. The test is performed by oven-drying the concrete specimen, immersing it in water for 48 hours, and finally testing it in water that is raised to boiling and held for 5 hours, weighing the specimen after each step. At the conclusion of the testing, the specimen is weighed while suspended in water.

The tests were conducted on two specimens for each mixture at a concrete age of 42 days. The test specimens consisted of the lower 3 in. (75 mm) portion of the 4 in. (100 mm) diameter cores taken from the unponded 5 in. (125 mm) test slab for conducting the rapid chloride permeability test. The results are listed in Table 6.

Very low absorptions and permeable void volumes of the heat-cured specimens were observed. The three heat-cured conventional concretes at all three w/c levels had lower absorptions and volumes of permeable voids than all the other 12 moist-cured mixtures, including all six of the moist-cured silica fume mixtures. The heat-cured specimens had absorptions and permeable void volumes 25 to 40 percent lower than the companion burlap-cured mixes, indicating that heat curing reduces the absorption and permeable void volume at all three w/c levels.

A 1994 paper by Gillott and Czarnecki[14] may help explain the absorption differences between these burlap-cured or tank-cured and heat-cured concretes. Their research determined that the microcracking of the 28-day fog-cured 0.35, 0.40, and 0.45 w/c conventional concretes was always greater than 28-day-old concrete that was fog-cured to 28 days after overnight heating to 185°F (85°C). The crack counts in their continuously fog-cured concretes during petrographic studies were about 135, 50, and 80 percent greater than the accelerated heat-cured concretes for the 0.35, 0.40, and 0.45 w/c concretes, respectively.

Table 6. Average concrete absorptions and permeable void volumes, sorted in order of decreasing boiling absorption.

Cure and mixture type	As-mixed w/cm	Absorption after immersion (percent)	Absorption after immersion and boiling (percent)	Volume of permeable voids (percent)	Slump (in.)
Burlap cure 5 percent silica fume	0.46	4.9	6.5	14.7	3 1/4
Burlap cure 5 percent silica fume	0.38	4.6	6.4	14.4	3
Burlap cure 5 percent silica fume	0.33	3.9	5.5	12.5	2 3/4
Burlap cure 7.5 percent silica fume	0.46	5.0	5.3	11.7	5 3/4
Burlap cure	0.46	4.5	4.7	10.7	4 1/4
Tank cure	0.46	4.1	4.4	9.9	6 1/2
Burlap cure 7.5 percent silica fume	0.37	4.2	4.3	9.9	5 3/4
Burlap cure	0.37	4.0	4.2	9.8	4
Burlap cure 7.5 percent silica fume	0.33	4.0	4.2	9.6	5
Burlap cure	0.32	3.8	4.0	9.3	6 1/4
Tank cure	0.37	3.6	3.9	8.9	5 1/2
Tank cure	0.31	3.4	3.7	8.5	5 3/4
Heat cure	0.46	3.2	3.5	8.0	6 1/4
Heat cure	0.37	2.7	2.9	6.8	6 1/4
Heat cure	0.32	2.3	2.5	5.8	4 3/4

Note: 1 in. = 25.4 mm.

The 5 and 7.5 percent silica fume mixtures had average volume of permeable voids that were about 100 and 50 percent greater, respectively, than the heat-cured mixtures, irrespective of their w/cm. The higher absorption and volume of permeable voids may be related to the higher susceptibility to cracking of low w/cm conventional and silica fume concretes.[3,14,15,16] Within each concrete mixture group, the absorptions and permeable void volumes were seen to be increasing with increasing w/cm, as would be expected. This observation serves to discount the effect of slump on the test results, because within each group the absorptions and permeable void volume increased in order of decreasing w/cm, regardless of slump.

AASHTO T 277 or ASTM C 1202 Testing

At a concrete age of 42 days, two specimens were tested according to AASHTO T 277 "Rapid Determination of the Chloride Permeability of Concrete" or ASTM C 1202 "Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration." To provide the best indication of the in-service condition of the concrete, the test specimens consisted of the top 2 in. (50 mm) portion of 4 in. (100 mm) diameter cores removed from the center portion of the unponded 5 in. (125 mm) test slab. The lower 3 in. (75 mm) portion of the core was tested to determine the absorption and permeable void volume, as previously described.

The average test results are summarized in Table 7 and ranged from 637 to 3410 coulombs, in the range expected. At a given w/cm, the heat-cured slabs exhibited the highest coulomb values, while the silica fume concretes exhibited the lowest. There were no substantial or consistent differences in coulombs between the 5 and 7.5 percent silica fume addition rates. The 5 percent silica fume concrete had the lower coulomb value for the 0.33 and 0.46 w/cm mixtures and

Table 7. Average AASHTO T 277 or ASTM C 1202 test results.

Cure and mixture type	As-mixed w/cm	42-day coulomb value
Tank cure	0.31	1431
	0.37	2004
	0.46	2909
Burlap cure 5 percent silica fume	0.33	637
	0.38	943
	0.46	1484
Burlap cure 7.5 percent silica fume	0.33	678
	0.37	726
	0.46	1696
Burlap cure	0.32	1411
	0.37	1965
	0.46	3041
Heat cure	0.32	1841
	0.37	2794
	0.46	3410

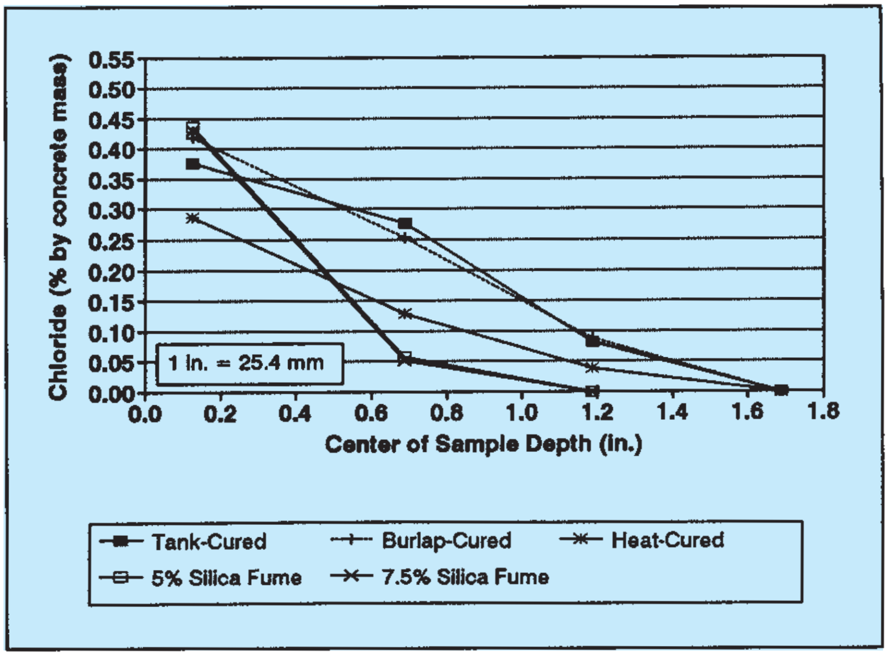

Fig. 2. Chloride profiles of 0.46 water-cementitious ratio concretes.

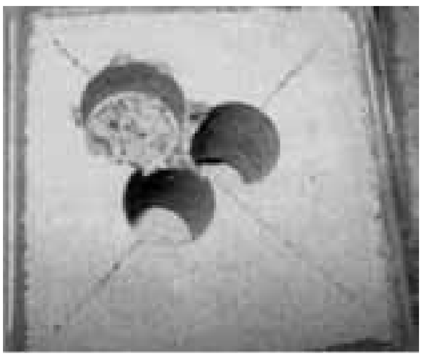

Fig. 1. Slab after coring to remove chloride samples.

the 7.5 percent silica fume concrete had the lower coulomb value for the 0.37 w/cm mixture.

The effect of curing can be observed by comparing the three conventional concrete mixtures. The tank-cured and burlap-cured specimens had essentially the same coulomb values, while the heat-cured specimens had somewhat higher coulomb values. Apparently, the early rapid curing of the heat-cured concrete, followed by a period of limited hydration while stored in air for 41 days in the CCR, resulted in a concrete with a lower electrical resistance and higher coulomb value than the identically proportioned 7-day water tank-cured and burlap-cured conventional concretes.

Note also that many of these low w/c concretes had higher coulomb values than would have been anticipated, based on Table 1 featured in the AASHTO T 277 or ASTM C 1202 specifications describing concretes as having high, moderate, low, very low,

Table 8. One-year concrete chloride content (percent by concrete mass).

Cure and mixture type	As-mixed w/cm	0 to 1/4 in. depth			1/2 to 7/8 in. depth			1 to 1 3/8 in. depth		
		Slab A	Slab B	Average	Slab A	Slab B	Average	Slab A	Slab B	Average
Tank cure	0.31	0.394	0.558	0.476	< 0.007	0.010	0.009	< 0.007	< 0.007	< 0.007
	0.37	0.417	0.522	0.469	0.106	0.091	0.099	< 0.007	< 0.007	< 0.007
	0.46	0.416	0.337	0.377	0.262	0.291	0.277	0.075	0.089	0.082
Burlap cure 5 percent silica fume	0.33	0.464	0.379	0.422	< 0.007	< 0.007	< 0.007	< 0.007	< 0.007	< 0.007
	0.38	0.393	0.563	0.478	0.010	< 0.007	0.009	< 0.007	< 0.007	< 0.007
	0.46	0.450	0.422	0.436	0.047	0.068	0.058	< 0.007	< 0.007	< 0.007
Burlap cure 7.5 percent silica fume	0.33	0.337	0.372	0.355	< 0.007	< 0.007	< 0.007	< 0.007	< 0.007	< 0.007
	0.37	0.398	0.300	0.349	0.010	0.010	0.010	< 0.007	< 0.007	< 0.007
	0.46	0.439	0.420	0.430	0.054	0.050	0.052	< 0.007	< 0.007	< 0.007
Burlap cure	0.32	—	—	0.506	0.013	0.025	0.019	< 0.007	< 0.007	< 0.007
	0.37	—	—	0.457	0.045	0.046	0.046	< 0.007	< 0.007	< 0.007
	0.46	0.445	0.392	0.419	0.244	0.262	0.253	0.074	0.102	0.088
Heat cure	0.32	0.421	0.392	0.407	0.008	0.030	0.019	< 0.007	< 0.007	< 0.007
	0.37	—	—	0.364	0.073	0.052	0.063	< 0.007	< 0.007	< 0.007
	0.46	—	—	0.287	0.146	0.113	0.130	0.039	0.038	0.039

Note: 1 in. = 25.4 mm.

or negligible chloride penetrability, as also observed in other research.[1,17,18]

Chloride Content Testing

One year after the continuous ponding started, the two slabs for each mixture were cored and tested to determine the chloride content in the following depth regions: $^1/_2$ to $^7/_8$ in., 1 to $1^3/_8$ in., and $1^1/_2$ to $1^7/_8$ in. (13 to 22 mm, 25 to 35 mm, and 38 to 48 mm). To reduce variability due to the sample size or aggregate concentrations, two 3 in. (75 mm) diameter cores were removed from the center of each slab and sliced into the depth increments described above.

After the two cores were sliced, the slices taken from the same depth region were pulverized, combined and analyzed using an acid-digestion potentiometric titration technique to determine acid-soluble chloride content essentially according to ASTM C 1152.[19] Following this testing, further work was performed to determine the chloride content in the uppermost $^1/_4$ in. (6 mm) of the concrete slabs. For this testing, a 4 in. (100 mm) diameter core was removed from each slab, sliced, ground, and analyzed. A slab after all the sampling is shown in Fig. 1.

For the initial portion of this additional work (Mixtures 10, 11, 14, and 15), one core was removed from each pair of slabs, and the two slices were interground to form a single sample. For the subsequent testing, the two slices were analyzed separately. Using this technique, the lower limit of detectable chloride was 0.007 percent by concrete mass. None of the 15 mixtures had measurable chloride in the $1^1/_2$ to $1^7/_8$ in. (38 to 48 mm) depth increment. The testing generally produced two measured chloride contents for each mixture, which were then averaged as listed in Table 8.

The results for the five different 0.46 w/cm concretes are shown in Fig. 2. The chloride profiles for the two silica fume concretes are clearly different than the three conventional concretes. Despite having chloride contents approximately 20 percent of the tank-cured or burlap-cured conventional concretes in the $^1/_2$ to $^7/_8$ in.

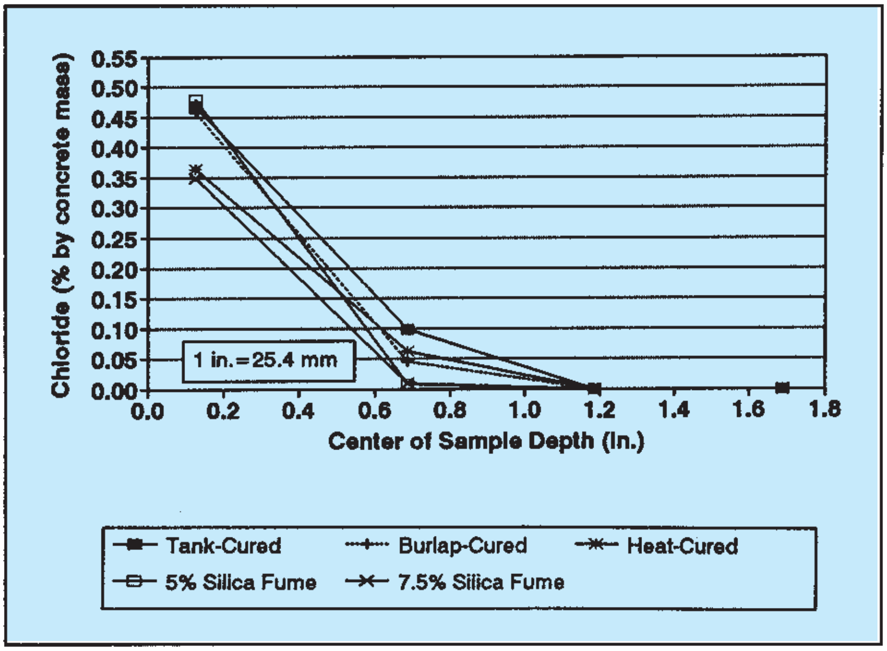

Fig. 3. Chloride profiles of 0.37 water-cementitious ratio concretes.

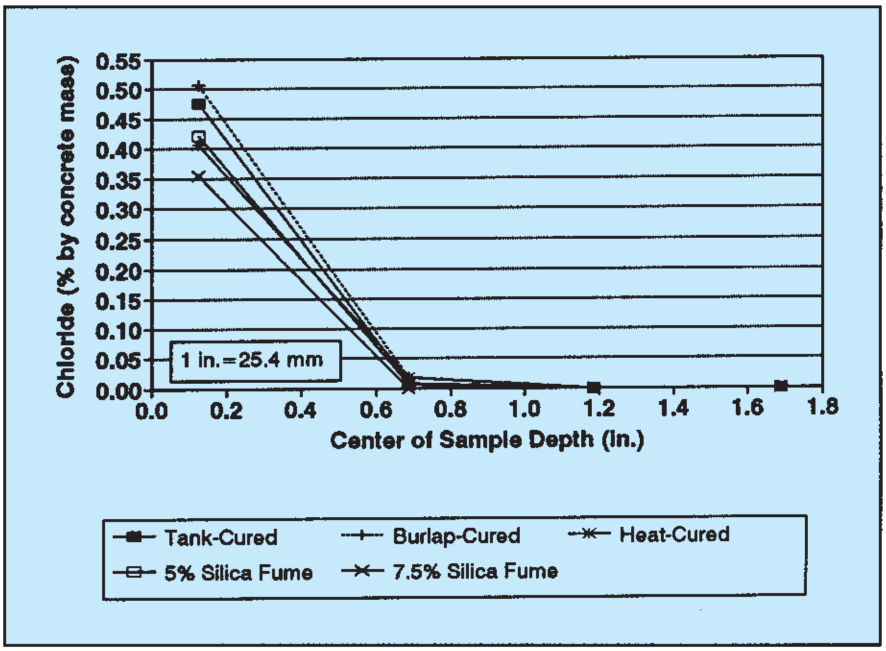

Fig. 4. Chloride profiles of 0.32 water-cementitious ratio concretes.

(13 to 22 mm) depth interval, the silica fume mixtures had similar or higher chloride contents in the 0 to $^1/_4$ in. (0 to 6 mm) depth interval. This may be explained by the significantly higher absorption and volume of permeable voids of the 0.46 w/cm ratio silica fume concretes, as previously discussed. Apparently, the improved chloride resistance associated with the addition of silica fume does not apply to the uppermost portion of the concrete, where absorption dominates over the slower diffusion process.

The chloride content in the 0 to $^1/_4$ in. (0 to 6 mm) depth interval for the heat-cured conventional concrete was the lowest of all five 0.46 w/cm mixtures, and about 35 percent lower than the two silica fume mixtures. The heat-cured conventional concrete had 50 percent reductions in chloride content, as compared to the water tank-cured and burlap-cured conventional concretes at the $^3/_4$ to $1^1/_2$ in. (19 to 38 mm) depth levels. This confirms the

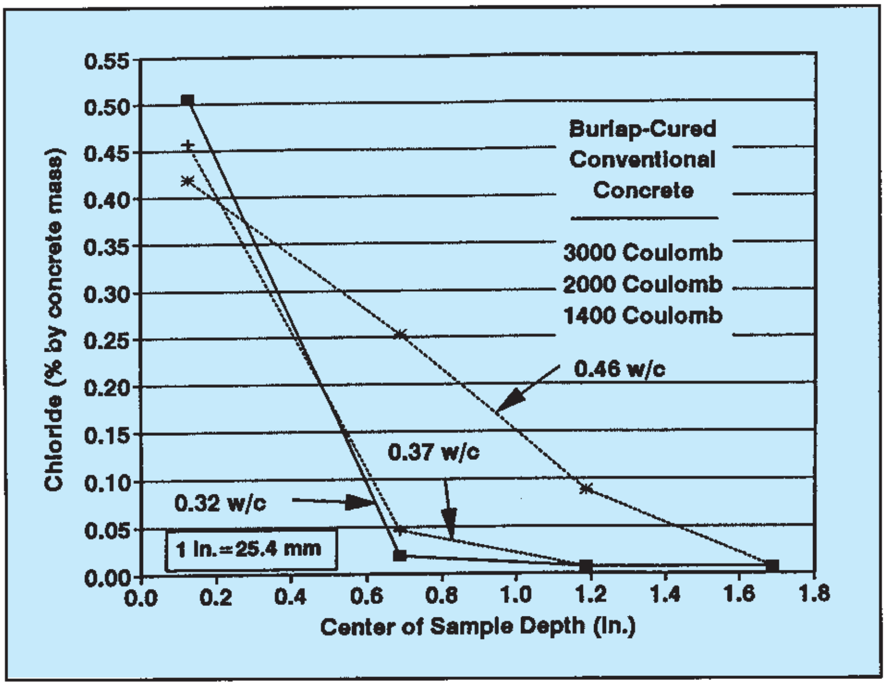

Fig. 5. Chloride profiles of burlap-cured conventional concretes.

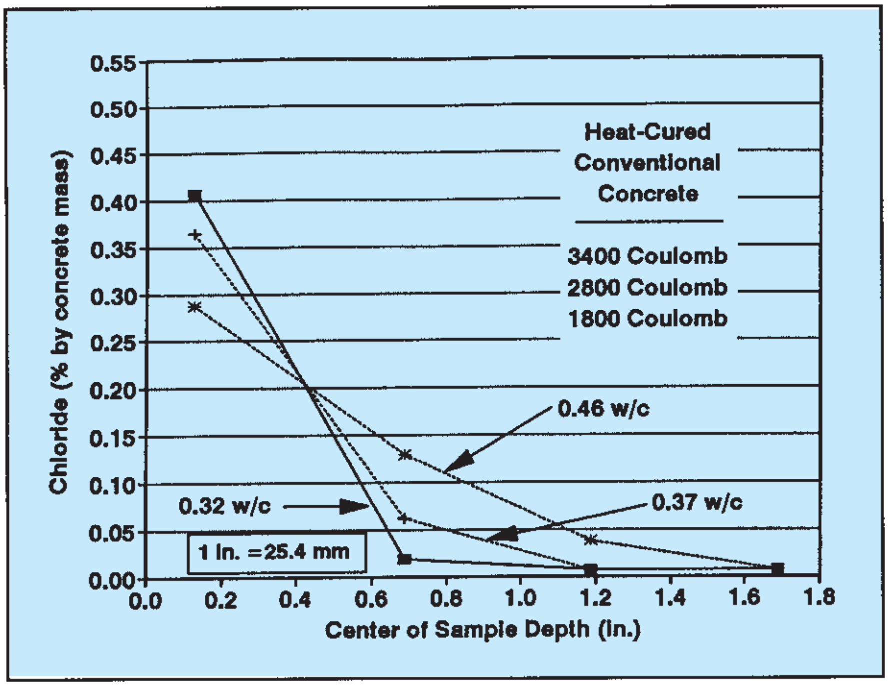

Fig. 6. Chloride profiles of heat-cured conventional concretes.

earlier work performed as part of the 1987 FHWA "Protective Systems for New Prestressed and Substructure Concrete" study[2] where similar 30 to 50 percent reductions in chloride intrusion into heat-cured AASHTO-grade concrete were observed, when compared to 3-day moist-cured, AASHTO-grade concrete.

At the 0.37 w/cm, the performances of the five different concretes were more similar, as shown by the similar shapes of the chloride profiles in Fig. 3. All concretes exhibited a significant drop in chloride content between the uppermost two-depth intervals, an observation only seen with the silica fume-modified concretes at the 0.46 w/cm.

As before, the two silica fume concretes had lower chloride contents than the other three conventional concretes, but the apparent improvement was dramatically less than with the 0.46 w/cm concretes. The heat-cured concrete had lower chloride contents than the burlap-cured concrete in the uppermost portion of the slab, but essentially identical chloride contents in the lower portion of the slabs.

For these five concretes, the 5 percent silica fume concrete had the same chloride content in the 0 to 1/4 in. depth interval as the tank-cured and burlap-cured control concretes, while the 7.5 percent silica fume concrete and the heat-cured concrete had significantly lower chloride content in the uppermost concrete level. The heat-cured concrete had a similar chloride level in the uppermost portion of the concrete to the 7.5 percent silica fume concrete.

At the 0.32 w/cm, all concretes had similar chloride profiles, as shown in Fig. 4. The only appreciable differences were the chloride concentrations in the near-surface region In this surface region, the 7.5 percent silica fume concrete had the lowest chloride content, followed by the heat-cured concrete and the 5 percent silica fume concrete. The burlap-cured and tank-cured conventional concretes had the highest near-surface chloride contents.

The dramatic change in the chloride profiles between the five different concretes from the 0.46 to the 0.37 and 0.32 w/cm ratio shows the overpowering effect of w/cm. At the lowest w/cm values, the beneficial effects of silica fume and heat curing of conventional concrete could not be seen due to the major reduction in chloride penetration caused solely by the decrease in the w/cm to 0.32.

The effect of the w/cm was pronounced, with the chloride contents of the 0.32 w/c and w/cm concretes at least 85 percent less than those of the comparable 0.46 w/c and w/cm concrete mixtures at the 1/2 to 7/8 in. (13 to 22 mm) depth. This can be clearly seen in Figs. 5, 6, and 7, which show the chloride profiles for the burlap-cured conventional concretes, heat-cured conventional concretes, and burlap-cured 7.5 percent silica fume concretes, respectively.

Note that in Figs. 5 and 6 for the burlap-cured and heat-cured conventional concretes, there is a reversal of relative chloride contents in the near-surface region of the slab, with the concretes with less chloride at depth having higher chloride contents near the surface. The reason for this is not known, but it probably relates to poorer surface characteristics of the lower w/cm concretes or more microcracks due to shrinkage of the higher cement content, lower w/cm concretes, as previously discussed. All 15 concretes had very high near-surface chlorides, ranging from about 11 to 20 lbs per cu yd (6.6 to 12 kg/m^3).

Chloride Diffusion Coefficient Calculations

A least-squares curve fitting technique was used to calculate the chloride diffusion coefficients and the salt water exposed surface chloride concentration. The calculation was performed assuming Fick's law of diffusion[20,21] according to the following equation:

$$C\left(x,t,C_o,D_{eff}\right)=C_o\left[1-\operatorname{erf}\left(\frac{x}{2\sqrt{tD_{eff}}}\right)\right]$$

where

x = sample depth

t = time

D_{eff} = effective diffusion coefficient

C_o = surface chloride concentration

erf = error function

During the curve fitting, the measured chloride concentrations at the four tested depths, x, were used to determine a least-squares fit for the effective diffusion coefficient, D_{eff}, and the surface chloride concentration, C_o, at a time, t, of 1 year. For Mixtures 4 and 7, the undetectable chloride in the $^1/_2$ to $^7/_8$ in. (13 to 22 mm) depth interval was assumed to be zero.

The surface chloride concentration reflects the chloride concentration C_o at the exterior surface of the concrete. The diffusion coefficient indicates the permeability of the concrete, with a smaller number indicating a less permeable concrete. The results of the calculations are shown in Table 9.

As shown in Table 9, the calcula-

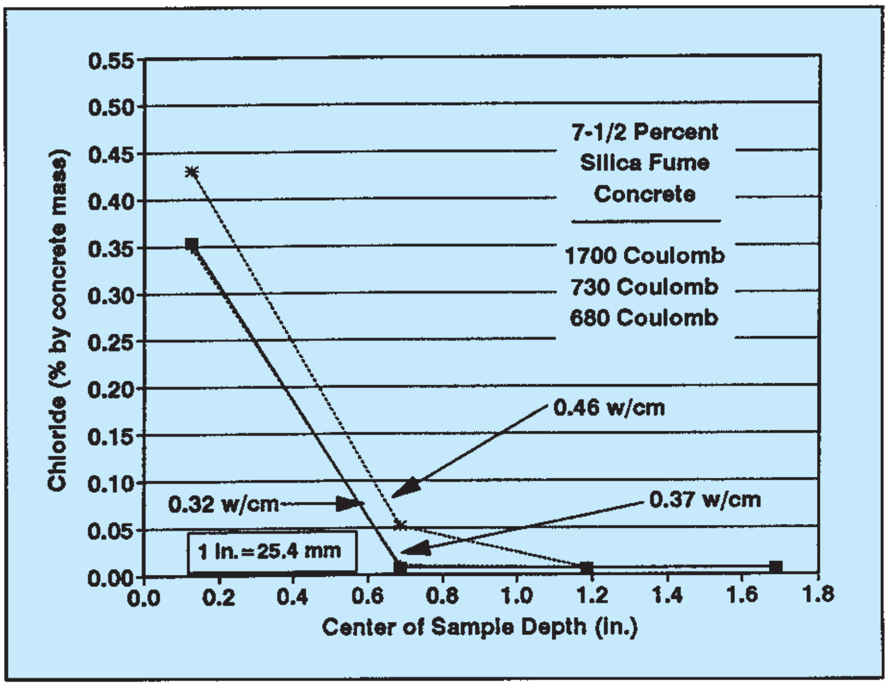

Fig. 7. Chloride profiles of 7.5 percent silica fume concretes.

Table 9. Calculated diffusion coefficients and surface chloride concentration.

Cure and mixture type	Mixture number	Diffusion coefficient (mm²/s × 10⁻⁶)	Surface chloride concentration (percent by concrete mass)
Tank cure	1	0.77	0.734
	2	2.51	0.586
	3	10.9	0.444
Burlap cure 5.5 percent silica fume	4	0.74	0.657
	5	0.77	0.737
	6	1.81	0.569
Burlap cure 7.5 percent silica fume	7	0.56	0.600*
	8	0.88	0.521
	9	1.69	0.567
Burlap cure	10	0.97	0.739
	11	1.52	0.613
	12	8.79	0.483
Heat cure	13	1.06	0.583
	14	2.15	0.464
	15	6.19	0.331

* A surface chloride of 0.600 was assumed for Mixture 7 to allow a reasonable curve-fit to be performed despite a lack of chloride data at and below the $^1/_2$ in. (13 mm) depth interval.

Table 10. Calculated average surface concentration for given w/cm.

w/cm	Cure type	Average chloride content at surface, C_o lb per cu yd	kg/m³	Coefficient of variation (percent)
0.32	Burlap and tank*	27.8	16.5	6.5
	Heat	22.8	13.5	—
0.37	Burlap and tank*	24.0	14.2	14.7
	Heat	18.2	10.8	—
0.46	Burlap and tank*	20.2	12.0	12.1
	Heat	13.0	7.7	—

* Conventional and silica fume mixtures, three or four mixtures.

Table 11. Estimated time-to-corrosion for continuously ponded specimens with 2 in. (50 mm) cover.

w/cm	Cure type, percent silica fume	Mixture number	Time-to-corrosion (year)
0.46	Tank — 0	3	1
0.46	Burlap — 0	12	1
0.46	Heat — 0	15	2
0.37	Tank — 0	2	4
0.46	Burlap — 5	6	5
0.37	Heat — 0	14	5
0.46	Burlap — 7.5	9	6
0.37	Burlap — 0	11	6
0.32	Heat — 0	13	9
0.32	Burlap — 0	10	9
0.32	Tank — 0	1	11
0.37	Burlap — 7.5	8	11
0.37	Burlap — 5	5	11
0.32	Burlap — 5	4	12
0.32	Burlap — 7.5	7	17

Table 12. Estimated time-to-corrosion for continuously ponded specimens with 3 in. (75 mm) cover.

w/cm	Cure type, percent silica fume	Mixture number	Time-to-corrosion (year)
0.46	Tank — 0	3	2
0.46	Burlap — 0	12	3
0.46	Heat — 0	15	4
0.37	Tank — 0	2	9
0.37	Heat — 0	14	11
0.46	Burlap — 5	6	12
0.46	Burlap — 7.5	9	13
0.37	Burlap — 0	11	14
0.32	Burlap — 0	10	20
0.32	Heat — 0	13	20
0.37	Burlap — 7.5	8	25
0.37	Burlap — 5	5	25
0.32	Tank — 0	1	26
0.32	Burlap — 5	4	28
0.32	Burlap — 7.5	7	38

tions indicate that the heat-cured concretes have between 18 and 36 percent less surface chloride than the average water tank-cured and burlap-cured conventional and silica fume concretes. Tables 9 and 10 further show that the low w/cm concretes generally have higher surface chloride concentrations than the higher w/cm mixtures, and that the coefficients of variation of the surface chlorides of burlap-cured and tank-cured conventional and silica fume concretes are low at 6 to 15 percent. All of these surface concentrations are less than the 30 lbs per cu yd (17.8 kg/m^3) discussed and used in other corrosion-related documents.[18,21,22,23]

Time-to-Corrosion Calculations

The Fick's law diffusion model was used to compare the relative permeability of the different concretes. This method is superior to earlier methods such as computing an "integral chloride,"[17] or comparing chloride contents at specific depth intervals,[17] as the diffusion equation serves to appropriately characterize the different chloride contents. Using Fick's law and the surface chloride concentrations and diffusion coefficients determined during the curve fitting, the chloride diffusion over periods longer than 1 year can be estimated.

The calculation assumes that the C_o surface concentration calculated after the 1-year ponding period will remain constant and that the diffusion coefficient does not change with time as the concrete matures. These assumptions are valid over short time periods, but very little research has investigated the time dependence of the diffusion parameters, so the long-term indications should be applied with care.

A 1^1/2 and 2 in. (38 and 50 mm) cover is recommended by the 1995 ACI 318-95R[10] for precast concrete for walls and slabs, and for other members, respectively, when exposed to corrosive environments. These ACI recommended values are 2 and 2^1/2 in. (50 and 62 mm) for cast-in-place concrete walls and slabs, and for other members, respectively, for the same environment.

The 1992 AASHTO[9] minimum cover for deck slabs exposed to deicing salts that have no positive corrosion protection is 2^1/2 in. (64 mm) for top reinforcement and 1 in. (25 mm) for bottom reinforcement. For prestressed concrete, the AASHTO minimum cover for prestressing steel and mild reinforcing steel is 2 in. (50 mm) for top of slab when deicers are used. AASHTO also states that when deicers are used, and where constant deicer contact cannot be avoided with the girders, or in locations where members are exposed to salt water, salt spray, or chemical vapor, additional cover should be provided.

AASHTO[9] also requires in Section 8.6.6 in Division II — Construction that cast-in-place concrete when exposed to salt water should have a 4 in. (100 mm) clear cover, unless indicated otherwise on the plans. AASHTO Sections 4.5.16.7 and

4.5.17.8 in Division I — Design call for 3 in. (76 mm) clear cover for precast piles and cast-in-place concrete piles when in a corrosive or marine environment or in alkali soils.

The time-to-corrosion for each of the 15 mixes was computed assuming a 2 and 3 in. (50 and 75 mm) cover over the reinforcing bars and a corrosion threshold of 0.022 percent acid-soluble chloride ion by concrete mass. The results are shown in Tables 11 and 12. Note that these times-to-corrosion are for continuously ponded specimens and are expected to be conservative estimates for parking garages, bridge decks, bridge substructures, and other structures that receive intermittent exposure to chloride.

The estimated time-to-corrosion calculations for 2 in. (50 mm) cover ranges from 1 to 17 years. The time-to-corrosion for all AASHTO-grade 0.46 w/cm concretes including the silica fume concretes are very low, ranging from 1 to 6 years. The benefits of using low w/cm concretes (0.37 to 0.32) with and without silica fume is easily seen with 7 of the 10 lower w/cm mixtures having time-to-corrosion of 9 to 17 years.

The time-to-corrosion estimates for the 0.46 and 0.37 w/cm silica fume concretes used in the construction of parking garages, bridge decks, and other members, were about 5 and 11 years, respectively. These values are much greater than the 1 year for the water tank-cured and burlap-cured AASHTO-grade 0.46 w/c conventional concretes.

The heat-cured conventional concretes with 0.37 and 0.32 w/c had estimated values of 5 and 9 years, respectively. These values are equivalent to the 5 to 11 years for the 0.46 to 0.37 w/cm silica fume concretes and essentially equal for 0.45 to 0.40 w/cm silica fume concretes typically used on current construction projects. Although the 12- to 17-year estimates for the 0.32 w/cm with the 5 and 7.5 percent silica fume mixtures were the longest computed time-to-corrosion estimates, these two concretes would be very difficult to use in typical construction projects where cast-in-place concrete operations and significant flatwork are required.

For more severe exposure, such as precast piles in salt water or alkali soils, or cast-in-place concrete in a corrosive or marine environment, AASHTO[9] requires the use of 3 to 4 in. (75 to 100 mm) of cover. The estimated time-to-corrosion for 3 in. (75 mm) cover ranges from 2 to 38 years. The conventional AASHTO-quality 0.46 w/c concretes with tank cure, burlap cure, or heat cure had time-to-corrosion estimates of 2 to 4 years. All of the 12 other concretes performed better, with estimates ranging from 9 to 38 years.

The 5 and 7.5 percent silica fume mixtures with w/cm of 0.46 to 0.37 have estimates of 12 to 25 years, essentially equal to the 11 to 20 years of the heat-cured 0.37 to 0.32 w/c conventional concretes. Among the other mixes, the 0.37 to 0.32 w/c burlap-cured conventional concretes had estimates of 14 to 20 years, and the 0.32 w/c tank-cured conventional concrete had an estimate of 26 years. These are about the same as the silica fume and heat-cured concretes already discussed. The 0.32 w/cm silica fume mixtures with 5 and 7.5 percent silica fume have the longest estimates of 28 to 38 years.

The benefits of using 2 in. (50 mm) cover vs. 1 in. (25 mm), or 3 in. (75 mm) cover vs. 2 in. (50 mm) cover in time-to-corrosion are shown in Table 13. The dramatic increase in the calculated times-to-corrosion illustrate that the time-to-corrosion of AASHTO-grade burlap-cured 0.46 w/c concrete is only marginally increased by 1 to 2 years when the cover is increased from 1 to 2 in. (25 to 50 mm) or from 2 to 3 in. (50 to 75 mm).

With commonly used 0.37 to 0.32 w/c heat-cured conventional concretes, the increases are about 5 to 10 times greater for the same cover increases. With 0.46 to 0.37 w/cm silica fume mixtures, the increases are about 6 to 12 times greater for the same cover increases. These data indicate that if increasing cover is to be used to increase the time-to-corrosion, it will be most effective when used with a low w/cm heat-cured conventional concrete or burlap-cured silica-fume concrete.

Table 13. Increase in time-to-corrosion.

w/cm	Cure type, percent silica fume	Mixture number	Increase in time-to-corrosion (years): 2 in. (50 mm) vs. 1 in. (25 mm)	Increase in time-to-corrosion (years): 3 in. (75 mm) vs. 2 in. (50 mm)
0.46	Tank — 0	3	1	1
0.46	Burlap — 0	12	1	2
0.46	Heat — 0	15	2	2
0.37	Tank — 0	2	3	5
0.37	Heat — 0	14	4	6
0.46	Burlap — 5	6	4	7
0.46	Burlap — 7.5	9	4	7
0.37	Burlap — 0	11	5	8
0.32	Burlap — 0	10	7	11
0.32	Heat — 0	13	7	11
0.37	Burlap — 7.5	8	9	14
0.38	Burlap — 5	5	9	14
0.31	Tank — 0	1	9	15
0.33	Burlap — 5	4	9	16
0.33	Burlap — 7.5	7	13	21

DISCUSSION OF TEST RESULTS

In this section, the effects of water-cementitious materials ratio, heat curing and silica fume together with comparable end-use concretes and validity of AASHTO and ASTM specifications are discussed.

Table 14. Comparison of chloride content reductions due to lowered water-cement ratios (FHWA[2] and present investigations).

Study	Change in w/c	Reduction in chloride at 1 in. (25 mm) depth (percent)
1987 FHWA	0.51 to 0.40	80
Present	0.46 to 0.37	80
1987 FHWA	0.51 to 0.28	95
Present	0.46 to 0.32	94

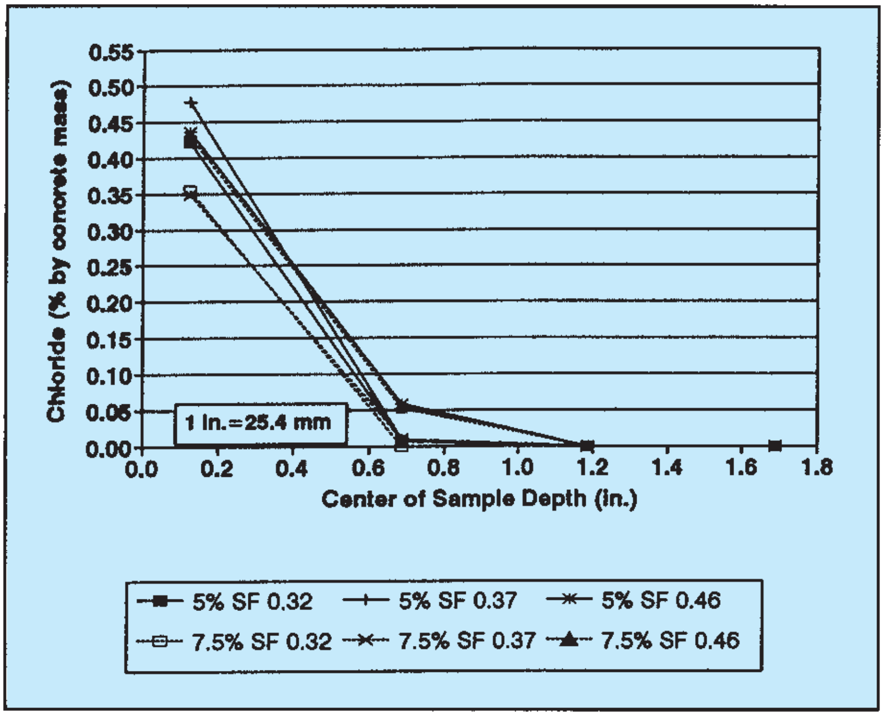

Fig. 8. Comparison of concretes with different silica fume dosages.

Effect of Water-Cementitious Materials Ratio

The most important conclusion gained from this work is the effectiveness of a low w/c or w/cm in decreasing the chloride permeability of concrete. As shown in Fig. 4, all five mixtures using a 0.32 w/cm had very low chloride contents of less than 0.020 percent by concrete mass in the $^1/_2$ to $^7/_8$ in. (13 to 22 mm) depth region.

In addition, all five mixtures produced using a w/cm of 0.37 had significantly lower chloride contents than their companion mixtures produced at a w/cm of 0.46, as shown in Figs. 2 and 3. The effect of low w/cm can also be seen in the very low chloride content in the 1 to $1^3/_8$ in. (25 to 35 mm) depth for all 10 mixtures with 0.32 and 0.37 w/cm, as shown in Table 8.

The overwhelming evidence of improved performance through lowering the w/c matches conclusions reached by Pfeifer et al., in a 1987 FHWA study.[2] A comparison of the chloride content reductions due to lowered w/c in that study and in the present study is shown in Table 14.

Effect of Heat Curing

The results show that the heat-cured conventional concretes at 0.32, 0.37, and 0.46 w/c produced water absorptions and volume of permeable voids that were much lower than the 12 moist- and tank-cured mixtures that contained 0, 5, and 7.5 percent silica fume and were moist-cured for 7 days. In fact, at all w/cm, the silica fume concretes had volume of permeable voids that were, on average, 100 and 50 percent greater than the heat-cured conventional concretes for the 5 and 7.5 percent silica fume addition rates, respectively.

The highest volume of permeable voids for heat-cured conventional concrete was only 8.0 percent, a value 17 percent lower than the 9.6 percent voids of the best performing silica fume mixture (0.33 w/cm with 7.5 percent silica fume), and 46 percent lower than the 14.70 percent voids of the worst performing silica fume concrete (0.46 w/cm with 5.0 percent silica fume). These high absorption values for all of the silica fume concretes may explain the high surface chloride concentrations for these silica fume concretes observed during the long-term ponding tests.

The long-term ponding tests showed that the AASHTO-grade 0.46 w/c heat-cured concrete had the lowest near-surface chloride concentration when compared to the 0.46 w/cm moist-cured concretes, including the 5.0 and 7.5 percent silica fume mixtures. These surface chlorides for the 0.46 w/c heat-cured concrete averaged about 36 percent less than the four other moist-cured 0.46 w/cm concretes with or without silica fume.

The long-term ponding tests also indicate that the heat-cured conventional concretes at 0.32, 0.37, and 0.46 w/c had lower or essentially equal chloride contents at all measured depths when compared to the burlap-cured or water tank-cured conventional concretes at the same w/c.

These current observations confirm similar conclusions about the beneficial effects of heat curing on chloride profiles in 0.44 w/c AASHTO-grade concrete studied in the 1987 FHWA study of heat-cured vs. moist-cured 0.44 w/c concrete.[2] This current study also showed that the 28-day compressive strength of properly heat-cured concrete was not significantly lowered by heat curing, even when the cylinders did not receive any supplemental wet or moist curing after the initial overnight heat curing.

This conclusion, in conjunction with the lower water and chloride absorption, lower volume of permeable voids, lower 1-year chloride content profiles, and comparable estimated time-to-corrosion values, indicates that heat-cured conventional concretes are far more impervious to water and chloride ingress than the same moist-cured

AASHTO-grade 0.46 w/c concretes. The 0.37 and 0.32 w/c heat-cured conventional concretes also had equal or greater resistance to water and chloride ingress than moist-cured conventional concretes with equal w/c levels.

The observations from these two major studies indicate that the removal of the AASHTO requirement for 6 days of supplemental moist curing of heat-cured concrete in 1992 was justifiable and appropriate.

Effect of Silica Fume

The addition of silica fume to the 0.46 w/cm specimens was seen to be highly beneficial in reducing chloride ingress. After the long-term ponding test, the chloride at 1/2 to 7/8 in. (13 to 22 mm) in these concretes were at least 78 percent lower than that in the conventional 0.46 w/c concrete. The addition of the silica fume prevented the measurable ingress of chloride to the 1 to 1 3/8 in. (25 to 35 mm) depth region.

The benefits of the silica fume were also apparent in the 0.37 w/cm mixes, where the 5 and 7.5 percent silica fume concrete had 75 to 80 percent less chloride than the companion burlap-cured conventional concrete in the 1/2 to 7/8 in. (13 to 22 mm) depth interval. The benefits of the silica fume were difficult to determine in the 0.32 w/cm mixes, because all of the 0.32 w/c and w/cm mixes had extremely low chlorides at all depths, except the near-surface region.

As shown in Fig. 8, chloride concentrations after ponding for the 5 and 7.5 percent silica fume mixes were similar for each w/cm. The 7.5 percent silica fume concretes also required higher admixture dosages, although they did have slightly higher slumps than the 5 percent silica fume mixtures. The compressive strength and coulombs values for the 7.5 percent silica fume mixes were not always better than the 5 percent silica fume mixes.

Comparable End-Use Concretes

The dramatic and overpowering effect of the w/c on the chloride permeability of concrete requires that the potential advantages of so-called "high performance" concretes, such as low w/c heat-cured concretes or those containing admixtures such as silica fume, be examined using realistic w/cm values for project application.

When comparing heat-cured precast concrete to moist-cured, cast-in-place concrete, it must be recognized that precast concrete typically must have a lower w/cm due to the need to strip and recycle the forms in short time periods. Two examples are given here; others could be constructed, keeping in mind the project requirements.

One such comparison would be of a conventional parking garage or bridge with only a minimal emphasis on corrosion protection for which no specialty contractors, extra effort, or materials would be required. Such structures are currently being built and require few changes in construction or production methods. The comparable concrete systems used in this hypothetical bridge or garage are: 0.46 w/c burlap-cured conventional concrete, 0.46 w/cm burlap-cured concrete with 5 or 7.5 percent silica fume, and a 0.37 w/c heat-cured conventional precast concrete. The measured and calculated properties of these four concretes are shown in Table 15.

Table 15. Measured and calculated concrete properties of four different mixtures.

Mixture number	w/cm	Coulombs	Diffusion coefficient $mm^2/s \times 10^{-6}$	Volume of permeable voids (percent)	Calculated surface chloride concentration (percent)	Time-to-corrosion (years) Cover 2 in.	Time-to-corrosion (years) Cover 3 in.
12	0.46	3040	8.8	10.7	0.48	1	3
6*	0.46	1484	1.8	14.7	0.57	5	12
9†	0.46	1690	1.7	9.3	0.57	6	13
14	0.37	2794	2.2	6.8	0.46	5	11

Note: 1 in. = 25.4 mm.
* 5 percent silica fume.
† 7.5 percent silica fume.
0.46 w/c conventional concrete (Mixture 12) with 7 days of burlap cure.
0.46 w/cm silica fume concretes (Mixtures 6 and 9) with 7 days of burlap cure.
0.37 w/c conventional concrete (Mixture 14) with overnight heat curing and no supplemental moist curing.

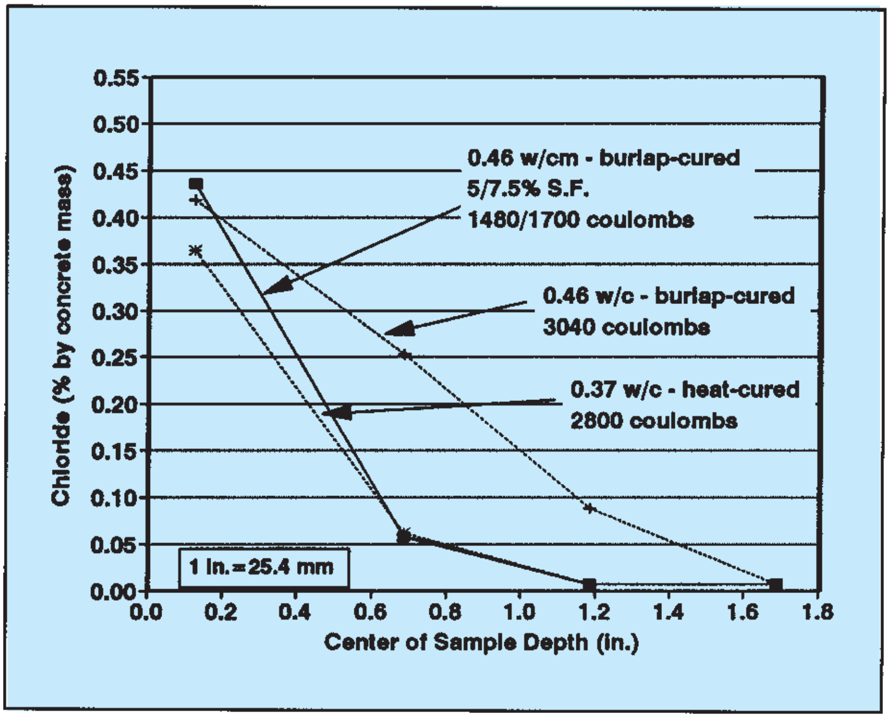

Fig. 9. Chloride profiles of comparable "conventional" systems.

The chloride profiles for these more easily constructed systems, along with

Table 16. Measured and calculated concrete properties of four different mixtures.

Mixture number	w/cm	Coulombs	Diffusion coefficient $mm^2/s \times 10^{-6}$	Volume of permeable voids (percent)	Calculated surface chloride concentration (percent)	Time-to-corrosion (years) Cover 2 in.	Time-to-corrosion (years) Cover 3 in.
11	0.37	1965	1.5	9.8	0.61	6	14
5*	0.37	943	0.8	14.4	0.74	11	25
8†	0.37	726	0.9	9.9	0.52	11	25
13	0.32	1841	1.1	5.8	0.58	9	20

Note: 1 in. = 25.4 mm.
* 5 percent silica fume.
† 7.5 percent silica fume.
0.37 w/c conventional concrete (Mixture 11) with 7 days of burlap cure.
0.37 w/cm silica fume concretes (Mixtures 5 and 8) with 7 days of burlap cure.
0.32 w/c conventional concrete (Mixture 13) with overnight heat curing and no supplemental moist curing.

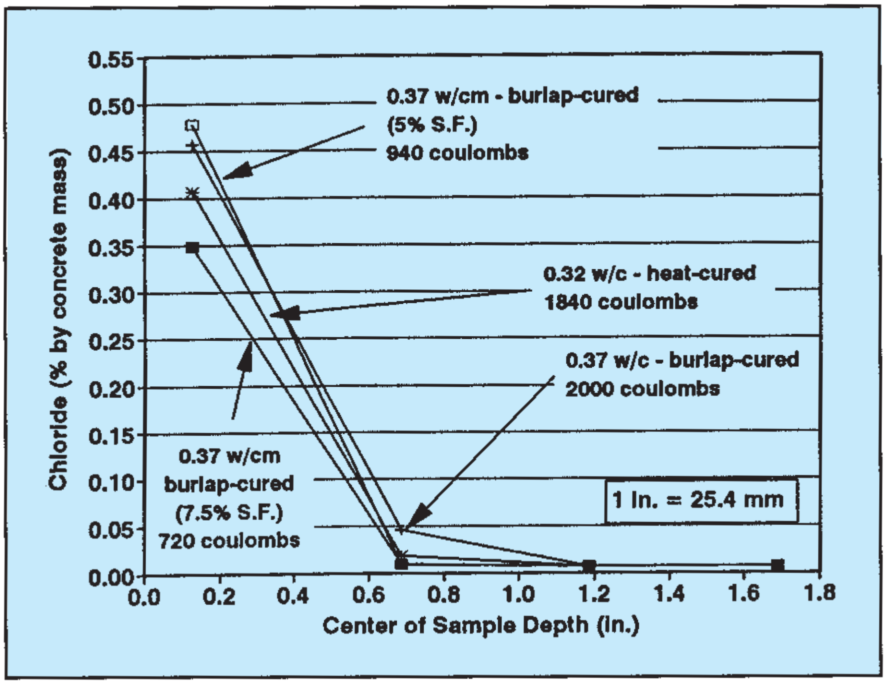

Fig. 10. Chloride profiles of comparable "higher quality" systems.

their coulomb values, are shown in Fig. 9. The silica fume and heat-cured precast concrete have essentially identical diffusion coefficients and times-to-corrosion, and both offer a substantial advantage over the conventional 0.46 w/c concrete.

Any of these three higher performance concretes with low diffusion coefficients of about 2.0×10^{-6} mm^2/s would be a good choice for prolonging the life of the concrete structure. This is true despite the 1200-coulomb difference between 2800-coulomb value for the heat-cured conventional concrete and the average 1600-coulomb value from the two silica fume concretes.

As another comparison, consider the concretes specified for significantly improved corrosion protection, shown in Table 16, that could be placed with more effort on the part of the concrete material suppliers and the contractor's workers. These concretes would include a 0.37 w/c burlap-cured conventional concrete, a 0.37 w/cm burlap-cured concrete containing 5 or 7.5 percent silica fume, or a heat-cured conventional concrete with a w/c of 0.32.

The resulting chloride profiles for these concretes are shown in Fig. 10. Either the addition of silica fume or the use of low 0.32 w/c heat-cured concrete results in substantially improved diffusion coefficients of about 1×10^{-6} mm^2/s and times-to-corrosion. This similar and substantial reduction in chloride penetration occurs despite a two-fold difference (830 vs. 1840) in the coulomb values of these concretes.

The increases in the time-to-corrosion were not reflected by these coulomb values. In fact, this low-permeability 0.32 w/c heat-cured concrete, with a coulomb value of 1840, would not have passed the 1000-coulomb cutoff used sometimes in project specifications.

Accuracy and Validity of AASHTO T 277 or ASTM C 1202 Testing

Comparison of the AASHTO T 277 or ASTM C 1202 coulomb values, the 1-year ponding test results, the diffusion coefficients, the surface chloride concentration C_o, and the calculated times-to-corrosion raises a number of serious questions regarding the accuracy of the T 277 or ASTM C 1202 tests and the permeability correlations presented in Table 1 of T 277 and C 1202, currently used in concrete specifications in an effort to ensure low-permeability concrete. The use of the T 277 or C 1202 test in an attempt to ensure long-term service life assumes incorrectly that the coulomb value is directly related to the long-term diffusion properties and the surface chloride concentration C_o, which in turn control Fick's law of diffusion of chloride through concrete.

The data in Tables 7 and 10 clearly indicate that there is no correlation between coulomb value and surface chloride concentration C_o.

The relationship between the calculated diffusion coefficient and the measured coulomb value for the 15 concretes tested during this study is shown in Fig. 11. Although there is an apparent relationship with lower coulomb values and lower diffusion coefficients, there is a large amount of scatter in the data and an apparent extreme sensitivity of the diffusion coefficient to the coulomb value. This is shown by Mixtures 1, 4, 5, 7, 8, 10, and 13, which have essentially the same very low diffusion coefficients ranging from 0.6 to 1.1 × 10^{-6} mm^2/s and coulomb values ranging from 600 to 1800 coulombs.

Similarly, Mixtures 6 and 9 (0.46 w/cm with silica fume contents of 5 and 7.5 percent, respectively) have essentially the same diffusion coefficient as Mixture 14 (0.37 w/c conventional heat-cured), of about 1.7 to 2.1 × 10^{-6} mm^2/s, yet there is a difference of 1200 coulombs.

A similar difference occurs between Mixture 4 (0.32 w/cm and 5 percent silica fume) and Mixture 1 (0.32 w/c conventional water tank-cured), with both mixes having the same very low diffusion coefficient of 0.75 × 10^{-6} mm^2/s and an 800 coulomb difference. These cumulative data show that there can be an 800 to 1200 coulomb decrease when silica fume is added and that essentially equal diffusion coefficients for 0.32 and 0.37 w/c conventional concretes can be obtained despite this 800 to 1200 coulomb difference. This same observation has been made in other research studies.[17]

To expand this lack of correlation, the data from these 15 concretes were combined with the data from a 1988 chloride diffusion study[18] of nine concretes with and without calcium nitrite with w/cm values ranging from 0.38 to 0.48 and silica fume contents of 0, 7.5, and 15 percent, as shown in Fig. 12. Four concretes with very high silica fume contents of 15 percent had coulomb values of only 75 to 253. The one concrete with 7.5 percent silica fume had a very low coulomb value of 380. The other four concretes contained no silica fume.

The data from these 24 concretes in Fig. 12 show that there is a dramatic change in the relationship at a coulomb level of about 2500 to 3000, not at 1000 coulomb as implied by Table 1 in the AASHTO T 277 and ASTM C 1202 specifications. In both studies, very low diffusion coefficients of 2.0 to 1.0 × 10^{-6} mm^2/s were achieved using practical concretes with coulomb values less than 3500, with the majority less than 2000 coulombs.

These very low diffusion coefficients of 2 to 1 × 10^{-6} mm^2/s are 80 to 90 percent lower than the 11 × 10^{-6} mm^2/s diffusion coefficients of the 0.46 to 0.48 w/c conventional moist-cured concretes used as controls in these two studies. Yet, two of these 14 practical concretes, one from each study, had coulomb values of 2800 to 3500 with very low diffusion coefficients of 2 × 10^{-6} mm^2/s.

The approximate relationship between the coulomb value and time-to-corrosion [assuming a 2 in. (50 mm) cover] results from the lack of correlation of the surface chloride concentration C_o to the coulomb value and the high sensitivity of the time-to-corrosion to the diffusion coefficient, shown in Fig. 13.

Because of these two factors, the small scatter in the data shown in Fig.

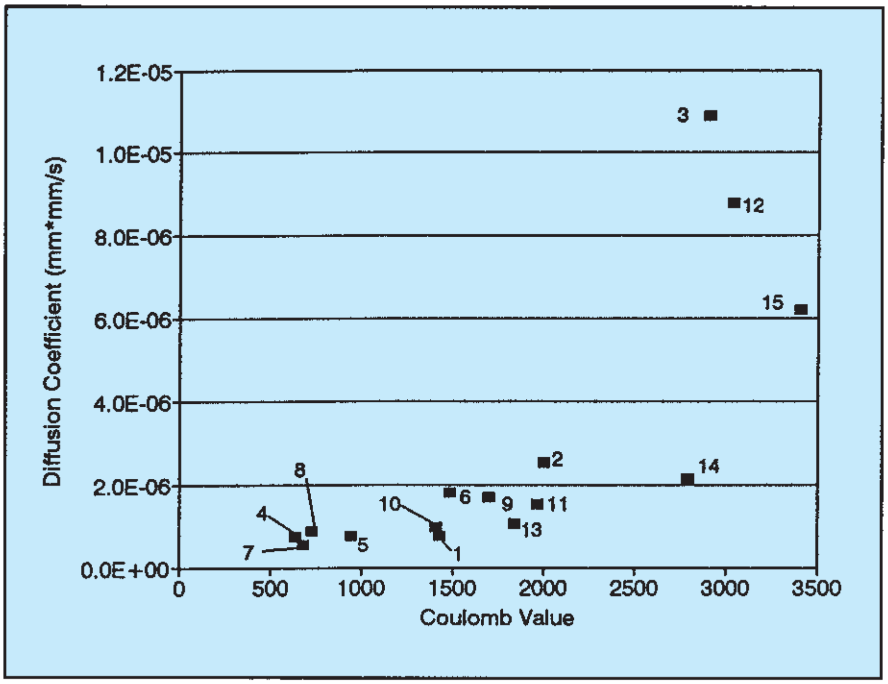

Fig. 11. Relationship between diffusion coefficient and coulomb value.

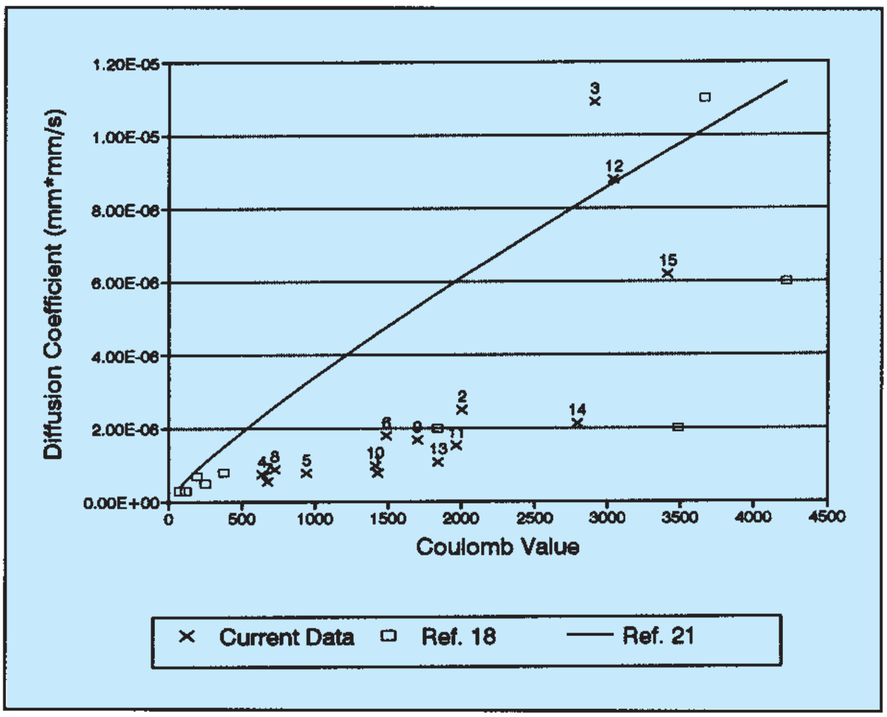

Fig. 12. Relationship between diffusion coefficient and coulomb value.

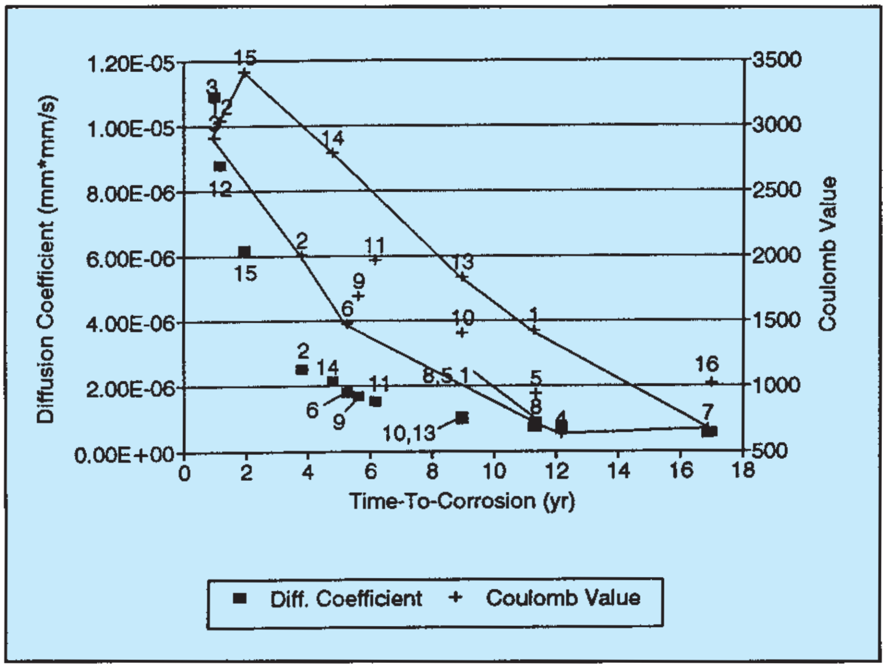

Fig. 13. Time-to-corrosion as estimated by diffusion coefficient and coulomb value.

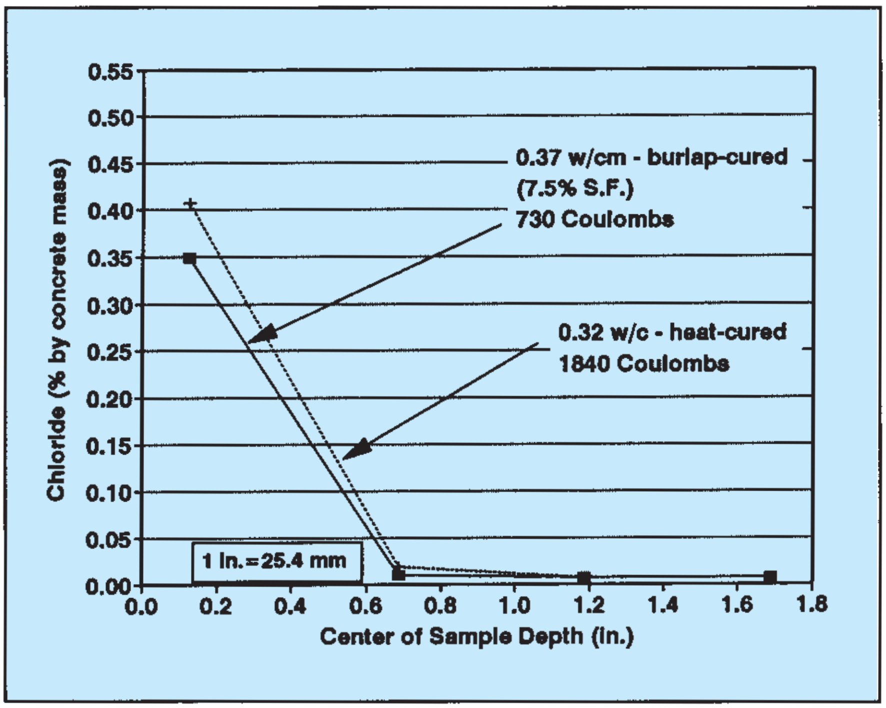

Fig. 14. Example of comparable concretes with different coulomb values.

11 is increased when the time-to-corrosion is calculated, resulting in the large scatter in the bounded area shown in Fig. 13. As a result, the calculated time-to-corrosion when using a coulomb-based estimate of the diffusion coefficient can cover a range of approximately 3 to 5 years for coulomb values of 500, 1000, and 2000. Conversely, given a constant time-to-corrosion, the estimated coulomb values can vary by 700 to 1200 coulombs.

As shown in Fig. 12, the combined data from these 24 concretes from these two studies do not appear to be correctly represented by the correlation equation developed from data from the 1988 paper[18] and presented in a 1994 paper.[21]

This sensitivity of calculated time-to-corrosion indicates that the diffusion coefficients and C_o factors should be determined using long-term ponding studies, rather than coulomb-based estimates, which determine neither of these two factors. An example of the error arising from using coulomb values to attempt to predict long-term performance is shown in Fig. 14. The 0.37 w/cm concrete with 7.5 percent silica fume and the 0.32 w/c heat-cured concrete had essentially identical chloride profiles, yet had more than a 1000 coulomb difference.

In addition to similarly performing concretes having very different coulomb values, some differently performing concretes can have very similar coulomb values. Another example of this error is shown in Fig. 15, where the 0.46 w/c burlap-cured concrete has a very similar coulomb value of 3000 to a 0.37 w/c heat-cured concrete with a coulomb value of 2800, yet their chloride profiles and their diffusion coefficients were vastly different, and their C_o values were essentially equal. Had the coulomb value been solely relied upon to indicate the relative permeability of the concrete and time-to-corrosion, grossly incorrect conclusions would have been reached, with the heat-cured 0.37 w/c conventional concrete being improperly classified.

These cumulative data from two studies illustrate that the 1000-coulomb cutoff level for discriminating between "real world" concrete diffusion coefficient, surface chloride concentrations and time-to-corrosion is unjustified and inappropriate. It is important to note that numerous concrete projects in the past with specified coulombs of 600 to 1000 have created jobsite construction problems and cracking, based on the authors' experience.

Based on this extensive study and the requirement for "constructable" projects with minimal concrete finishing, curing, cracking, and permeability problems, it is clear that concrete with coulomb levels of 1000 to 3000 can produce low permeability concretes. In fact, for concretes with coulombs below 3000, both studies discussed above show diffusion coefficients that are at least 80 percent lower than the 0.46 w/c AASHTO-quality conventional concrete Mixture 3 used in this study and the 0.48 w/c conventional concrete used in the other referenced study,[18,21] which both had coulomb values of 2900 to 3700 for these AASHTO-quality concretes.

CONCLUDING REMARKS

The major conclusions, observations, recommendations, and concerns from this time-to-corrosion and chloride permeability study are as follows:

1. The low 0.32 w/cm mixtures had longer times of initial setting, probably related to the higher HRWRA requirements to achieve this very low w/cm value. This increase could range from about 1 hour to 1.5 hours when compared to the 0.46 w/cm mixtures. This setting time increase influences jobsite construction and finishing operations and heat curing preset periods for precasting plants.

2. The four 0.46 and 0.37 w/cm silica fume mixtures produced 28- and 180-day compressive strengths that essentially equaled the 28- and 180-day strengths of equal w/c ratio, conventional concretes.

3. The 28-day strength of heat-cured conventional concretes with no supplemental moist curing after the overnight heat curing averaged 90 percent of their companion 28-day water tank-cured cylinders for the three w/c levels. Therefore, proper heat curing created no significant strength loss at 28 days at all three w/c levels.

4. The three heat-cured conventional concrete mixtures had the lowest water absorptions and volume of permeable voids when compared to the 12 water tank-cured or burlap-cured concretes with or without silica fume. The concretes with 5 and 7.5 percent silica fume additions had average volumes of permeable voids that were about 100 and 50 percent greater, respectively, than the heat-cured conventional concretes.

5. All 15 concrete mixtures produced very high near-surface chloride concentrations after the 365-day ponding. The lowest w/cm mixtures produced on average the highest calculated C_o surface chloride concentrations, and the highest w/cm mixtures produced the lowest calculated C_o surface chloride concentrations.

6. The heat-cured concretes had the lowest water absorptions and volume of permeable voids at age 42 days. The 365-day long-term ponding tests also indicate that the heat-cured slabs generally had 18 to 36 percent lower near-surface chloride concentration than the overall average of the burlap-cured and water tank-cured concretes with and without silica fume, at all w/cm levels.

7. The surface chloride concentrations of 13 to 23 lbs per cu yd (7.7 to 13.6 kg/m^3) for the heat-cured concretes are significantly lower than the 30 lbs per cu yd (17.8 kg/m^3) assumed in other published corrosion research studies pertaining to long-term life estimates.

8. This present study and a 1987 FHWA study indicate that the concrete w/c ratio is the dominant factor in reducing chloride permeability. When the w/c is reduced from AASHTO-quality 0.46 to 0.51 w/c levels to 0.37 to 0.40 levels, the chloride reduction at the 1 in. (25 mm) depth level after severe 1-year salt water exposure testing was about 80 percent. When the w/c is reduced further to 0.28 to 0.32 levels, the chloride reductions were about 95 percent of the AASHTO-quality 0.46 to 0.51 w/c concrete.

9. The 5 and 7.5 percent burlap-cured silica fume mixtures with 0.46 to 0.37 w/cm have estimated times-to-corrosion similar to those of heat-cured 0.37 to 0.32 w/c conventional concretes. These four silica fume mixtures produced very low diffusion coefficients of 0.8 to 1.8 × 10^{-6} mm^2/s, and the two heat-cured conventional concretes produced similar diffusion coefficients of 1.1 to 2.2 × 10^{-6} mm^2/s. Typical burlap-cured AASHTO-grade 0.46 w/c conventional concrete produced a high diffusion coefficient of 8.8 × 10^{-6} mm^2/s, a value 4 to 10 times greater than the six low-permeability concretes discussed above. The lowest average diffusion coefficient of 0.65 × 10^{-6} mm^2/s was from the 0.32 w/cm silica fume concretes with 5 and 7.5 percent fume. The typical AASHTO-grade 0.46 w/c conventional burlap-cured concrete diffusion coefficient is about 14 times this lowest coefficient measured on this difficult to use 0.32 w/cm concrete.

10. An increase in cover to improve corrosion performance is significantly more effective when low w/c concretes are used.

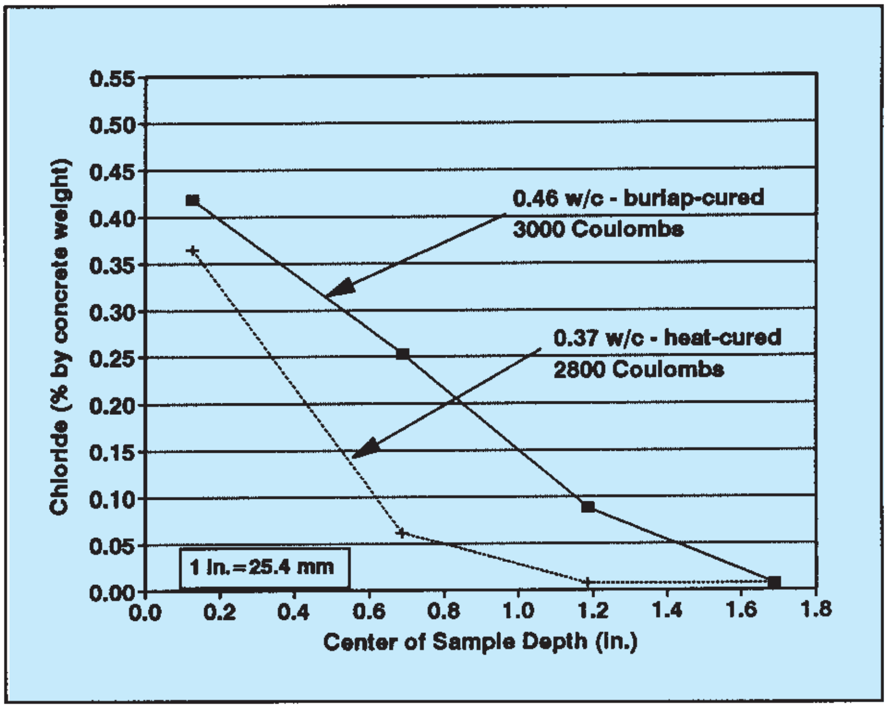

Fig. 15. Example of comparable concrete with similar coulomb values.

11. This investigation of the AASHTO T 277 and the ASTM C 1202 test methods revealed that significant and serious questions remain regarding their appropriateness for use in concrete qualification and project specifications. The correlation between long-term chloride diffusion, the surface chloride concentration C_o, the time-to-corrosion and the coulomb test recommended in the ASTM C 1202 test appears to be highly variable and requires individual correlations between these two types of tests for every concrete mixture. The widely used 1000 coulomb cut-off limit was found to be arbitrary and misleading for many concretes, due to the widely different chloride permeabilities and C_o factors observed for concretes both meeting and failing such a coulomb limit-based specification. The C_o chloride surface concentration did not correlate to the coulomb value. The use of heat curing was found in this study to increase the coulomb values of concrete without increasing its actual chloride permeability.

12. When similar end-use concretes are compared using estimated time-to-corrosion and long-term chloride ingress, low w/c heat-cured conventional concrete performs as well as practical w/cm concretes containing silica fume.

13. The best way to improve current concrete is to specify lower w/c concretes and enforce the specification requirements. Concrete should be specified to have a low chloride diffusion coefficient and a low C_o surface chloride concentration, as determined by long-term ponding tests. The 6-hour AASHTO T 277 and ASTM C 1202 tests for determining coulomb values only should not be used for specification purposes due to the variable effects of curing and other factors on the coulomb value. The test must only be used when proper correlations between coulomb values and long-term ponding test results have been established for the individual concretes under test, as already required in the ASTM C 1202.

14. To meet these new demands, materials suppliers, the precast concrete industry, and all other suppliers to the concrete industry should carry out long-term ponding tests (with at least 1 year of ponding) to demonstrate the performance of their specific materials and end product to the purchaser. By taking a long-term view of the concrete performance, the durability of all structures can be increased, lessening the need for repair and maintaining concrete as a state-of-the-art building material of choice for the transportation and construction industry.

During the testing described in this report, certain inconsistencies and weaknesses in the standardized tests were noted and reported by others.[24] The following recommendations and concerns to users of the test procedures have been provided in an effort to improve future specifications and testing of similar high performance concretes.

AASHTO Section 8.11 Curing Specifications

Except for top slabs of structures serving as finished pavements, the 1992 AASHTO specifications permit the moist curing to be terminated in less than 7 days, once the field-cured cylinders reach 70 percent of their design strength. With today's cement and lower w/c values, this permits the termination of moist curing from piers, columns, beams, parapets, walls, median barriers, and sound barriers, and other components that will be exposed to salt water during their life after as little as 1 to 3 days, based on the authors' experience. This procedure could result in improper moist curing for a number of highway structures that receive deicers.

AASHTO T 259 Ponding Test

Modifications to the AASHTO T 259 test procedures are required to enable the performance of high quality concretes to be differentiated. These modifications include:

1. Provision of a more realistic curing period. The T 259 recommended 14-day moist curing is unrealistic; a 7-day period is realistic because both ACI and AASHTO utilize this 7-day period.

2. Use of a higher chloride concentration for the ponding solution. The recommended 3 percent NaCl solution models sea water; however, it does not accurately model deicing salt solutions that frequently have significantly higher chloride concentrations. If a 15 percent salt solution is used, the measured chloride in the slabs will be greater, providing increased accuracy in the determination of the chloride contents and improved diffusion modeling.

3. Increase the ponding period. The 90-day test is only appropriate for distinguishing relatively porous and permeable concretes. A minimum test period of 365 days is recommended. As the test does not require extensive supervision or work during the ponding period, this extended period will have little impact on the cost of the tests.

4. Use of an increased number of chloride samples. The test procedure recommends only two chloride measurements to be made at depths $^1/_{16}$ to $^1/_2$ in. (1.6 to 13 mm) and at $^1/_2$ to 1 in. (13 to 25 mm). Two data points are totally insufficient to determine diffusion coefficients required for prediction of chloride concentrations during the life of the structure, and at least four depths should be used.

5. Use of core samples, rather than drilled samples. It is significantly better to use large samples cored from the slabs than to use small drilled powder samples that are usually subjected to errors from contamination and inaccurate drilling depths.

AASHTO T 277

This test method should be replaced with the newly revised ASTM C 1202 test method in order to specifically require correlation to long-term ponding data and to eliminate the misleading interpretation of Table 1.

ASTM C 1202

A number of changes should be made to reflect the changing state-of-the-knowledge reflected by this Part 2 paper and other papers on this controversial test method. These changes are:

1. The age of the concrete for conducting the coulomb tests should be specified when laboratory produced concrete mixtures are being evaluated and correlated to long-term ponding tests.

2. All laboratory produced specimens for coulomb testing and long-term ponding should have realistic moist curing of no longer than 7 days, as per ACI and AASHTO specifications for construction sites. Continuous moist curing to ages of 28 to 90 days prior to coulomb testing or ponding should be prohibited.

3. The long-term ponding used to correlate the material-specific performance of the concrete should be specified in ASTM C 1202 to be at least 365 days and should use a salt water solution of 15 percent, in order to obtain more accurate diffusion coefficients and surface chloride concentrations.

4. Table 1 showing correlation of coulombs to permeability should be removed.

General Concern

The authors of this paper are concerned about specifications that require the ASTM C 1202 or AASHTO T 277 "coulomb test" for the concrete acceptance for a project, yet which also prohibit the use of the "coulomb test" on drilled cores from the project to determine if the Owner actually obtained his specified "coulomb value." This reluctance to test the jobsite concrete is undoubtedly based on concerns about jobsite concrete consolidation and curing factors.

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