High Performance Concrete and Reinforcing Steel With a 100-Year Service Life

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This paper reviews chloride permeability issues for conventional and high performance concretes so as to provide realistic design and specification guidance for the construction of concrete structures in harsh environments. This review briefly discusses cracking issues of high strength, high performance concretes, a significant detrimental issue created by restraint to thermal and shrinkage contractions during the first 60 days of a structure's life and by the high modulus of elasticity and the low creep factors inherent with high performance concretes. The last portion of the paper reviews the five-year (1993 to 1998) Federal Highway Administration (FHWA) study at Wiss, Janney, Elstner Associates, Inc. (WJE) on high performance, corrosion-resistant steel reinforcing bars. This FHWA study was successful in identifying bar types that can provide 75 to 125 years of design life, even in the presence of cracks in the concrete when exposed in harsh saltwater environments.

To put the subject of this paper into proper perspective, a historical review of chloride permeability issues for both conventional and high performance concretes is presented.

This chloride permeability review begins with a summary of the American Association of State Highway and Transportation Officials (AASHTO) Specifications, since it was the AASHTO Specifications that guided the construction of all bridges and pavements in the United States.[1] Table 1 provides a listing of the AASHTO water-cement (w/c) ratio* and compressive strength requirements from 1941 to 1999.

The U. S. Interstate Highway System was, therefore, dominantly constructed from about 1958 and into the

Note: This paper is based on a presentation delivered by the author in Sydney, Australia, May 6, 1999.

*Today, the term "water-cementitious" ratio is preferred to "water-cement" ratio. However, in this paper, the two terms are used interchangeably.

early 1970s with concrete with no specific w/c ratio guidance from AASHTO; the concrete strength was very low at 20.6 MPa (3000 psi); and the deck clear cover was very low at 25 mm (1 in.). It is no wonder that the start of the use of deicing salts in the mid-1950s began to corrode the thousands of bridge structures constructed with such low performance concretes and minimal clear cover over low performance black bars.

By 1980, even the concrete substructures with 50 mm (2 in.) design clear cover were corroding. It is noteworthy that cast-in-place and precast seawater exposed concrete elements had specified 100 and 76 mm (4 and 3 in.) clear cover, respectively, from 1931 to 1986 without any change in their clear cover requirements over low-performance black bars.

It is also significant that the non-air-entrained Class A concrete was permitted by AASHTO on even higher 0.49 w/c ratio from 1974 to 1986, while Class A (AE) was permitted only a 0.44 w/c ratio. In 1995, AASHTO, for some unknown reason, increased the Class A (AE) maximum w/c ratio from 0.44 to 0.45, a high permeability concrete as shown in this review.

To better understand the chloride permeability of concrete, the FHWA undertook long-term saltwater ponding tests in Virginia in 1974[2] and again in Chicago in 1983.[3] Plots of chloride contents at specific depths for conventional concretes with w/c ratios of 0.60, 0.50, and 0.40 from the 1974 study are shown in Fig. 1.

It is abundantly clear from Fig. 1 that the 0.40 w/c ratio concrete was much less permeable to chloride than the 0.50 or 0.60 w/c ratio conventional concretes. The 1983 FHWA sponsored work at Wiss, Janney, Elstner Associates, Inc., is shown in Fig. 2. By 1983, high-range superplasticized concretes were used in precasting operations and in low w/c concrete bridge deck overlays at 0.28 to 0.32 w/c ratios.

These low w/c ratio concretes were, in fact, the first high performance, low permeability concretes with exceptionally low permeabilities as shown in Fig. 2. In fact, at the 25 mm (1 in.) depth level, the 0.28 w/c ratio concrete had 95 percent less chloride after 44 weeks of cycle saltwater ponding when compared to the 0.51 w/c ratio concrete, even though the concrete strengths were both high at 51.7 and 34.5 MPa (7500 and 5000 psi), respectively.

Table 1. AASHTO Concrete Specifications.

Year Class A (AE) (air entrained)	Maximum water-cement ratio w/c	Minimum concrete strength MPa	Minimum top of bridge deck slab cover mm
1941-1952	0.053	20.6	25
1953-1973	—	20.6	38 (1969-1973)
1974-1995	0.44	31.0	50
1995-1999	0.45	31.0	50

Note: 1 MPa = 145 psi; 1 mm = 0.0394 in.

Much of the above information is discussed in significant detail in an award winning paper in the July-August 1996 PCI JOURNAL.[4,5]

In contrast to AASHTO, the American Concrete Institute (ACI) mandated in 1989 a maximum 0.45 w/c ratio for all concrete exposed to freezing and thawing. For corrosion protection, ACI 318 further required a maximum w/c of 0.40 but allowed a 0.45 w/c ratio if the design clear cover was increased by 13 mm (0.5 in.), also in 1989. This 0.45 w/c alternative was eliminated by ACI in 1992, yet the AASHTO still permits 0.45 w/c for corrosion environments.

Such confusion in AASHTO and ACI specifications resulted in the need for a quick and less costly test for chloride permeability so that differences in chloride permeability between 0.30, 0.40, 0.45, and 0.50 w/c ratio concretes could be easily and quickly determined for all aggregate and admixture combinations, including silica fume and other pozzolans.

The FHWA sponsored a study to develop such a rapid, low-cost chloride permeability study and a report was issued in 1981.[6] The AASHTO Test Method T277, "Rapid Determination of the Chloride Permeability of Concrete,"[7] was adopted in 1983. Virtually the same test procedure was designated in 1991 by the American Society for Testing and Materials (ASTM) as ASTM C 1202, "Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration."[8]

The 1991 ASTM C 1202 test method did mandate a proper correlation study between long-term saltwater ponding and the six-hour coulomb test. This was a significant improvement since the 1983 AASHTO T277 test method did not require any correlation between actual diffusion and coulombs.

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 C 1202 coulomb values less than 1000, based on the Table 1 "Coulomb passed" ratings in AASHTO T277 and ASTM C 1202.

In fact, project specifications were used that required as low as 600 coulomb concretes using low w/c ratios and silica fume. Contractors, as well as precasters, experienced significant consolidation, finishing, and curing problems.

As a consequence, many lawsuits resulted due to the obvious lack of constructability with such low coulomb values of 600 to 1000. In addition, numerous projects resulted in cracked concrete from poor consolidation, questionable curing, lack of bleed water for finishing, and detrimental effects of high strength, high modulus of elasticity and low creep characteristics at early ages.

Numerous papers critical of these AASHTO T277 or ASTM C 1202 test methods were published on a worldwide basis.[5,9] Of particular concern was a general lack of correlation studies between long-term saltwater ponding tests (at least one to two years), and these six-hour rapid electrical resistance tests that determine the "coulomb" rating of each concrete.

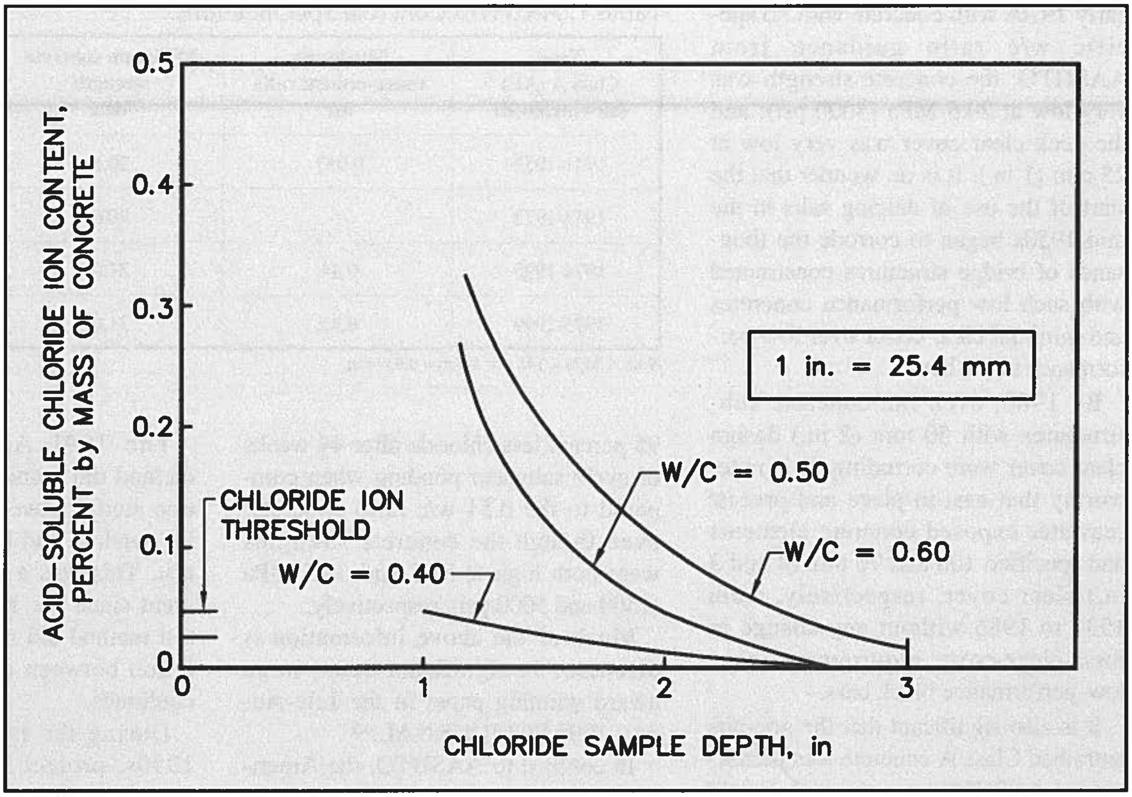

Fig. 1. Measured chloride profiles in moist-cured concrete from 1976 FHWA study in Virginia on 0.60, 0.50 and 0.40 w/c ratio conventional concrete after 2.3-year test period.

Most researchers were not undertaking the long-term saltwater ponding tests and few correlations were being published.

The few papers that contained meaningful long-term ponding studies illustrated questionable correlations for "coulomb" ratings in the 500 to 3000 coulomb range.[9] The 1996 paper in the PCI JOURNAL[5] clearly showed in Fig. 3 [Fig. 12 (Part 2)] that when actual long-term chloride diffusion coefficients are plotted versus "coulomb" values, there is a dramatic change in the relationship at a "coulomb" level of about 2500 to 3000, not at 1000 as implied by Table 1 in AASHTO T277 or ASTM C 1202.

In both studies at W. R. Grace in Cambridge, Massachusetts[10] and at WJE during the PCI study,[5] very low diffusion coefficients of 2.0 to 1.0 x 10^{-6} mm^2/second were achieved using practical and constructable concretes with coulomb values less than 3500, with the majority less than 2500 coulombs. These very low diffusion coefficients of 2 to 1 x 10^{-6} mm^2/second of the 0.31 to 0.38 w/c conventional concretes commonly used in the United States, are shown in Fig. 3.

These data from conventional concretes made in Boston and Chicago show, as T. C. Powers stated convincingly several decades ago, that concretes with w/c ratios less than 0.40 have discontinuous pore structures and, as a result, have low diffusion and permeability. These W. R. Grace and PCI data also confirm that low w/c ratio concretes used in the precast, prestressed concrete industry and for low w/c bridge deck overlays have been, in fact, high performance concretes. These data help to confirm why these precast concrete bridges and parking structures and cast-in-place bridge deck overlay concretes have been so durable in the United States, when compared to cast-in-place concrete that generally utilizes much higher w/c ratios of 0.45 to 0.50.

A recent 1998 paper by the Virginia Department of Transportation (VDOT)[11] shows other correlation studies between "coulomb" ratings and actual chloride diffusion coefficients after 2.5 year ponding tests on conventional concretes and pozzolan or slag-modified concretes at water to cementitious (w/cm) ratios of 0.35, 0.40, and 0.45. The VDOT data are shown in Fig. 4.

These data clearly show that the conventional 0.35 w/c ratio concrete has an extremely low diffusion coefficient of 0.2 x 10^{-6} mm^2/second even though the "coulomb" value is nearly 2000. The data also clearly show that essentially all of the 21 pozzolan or slag modified concretes with coulomb levels generally less than 1000 have higher diffusion coefficients.

The data indicate that essentially all the concretes with coulombs less than 2000 have very low diffusion coefficients of less than 1.0 x 10^{-6} mm^2/second and that the conventional control concretes with w/c ratios of 0.40 and 0.45 had diffusion coefficients 15 and 25 times greater than the 0.35 w/c ratio conventional concrete.

These VDOT data confirm the W.R. Grace and PCI studies that indicate that low permeability concretes that are constructable can have a "coulomb" rating as high as 2000.

It is interesting to note that the 1998 VDOT "Special Provision for Low Permeability Concretes" specifies the following "coulomb" levels based on "Laboratory Permeability Tests" at 28 days. The concrete is moist-cured at room temperature for seven days and then at 38°C (100°F) for 21 days prior to the coulomb test.

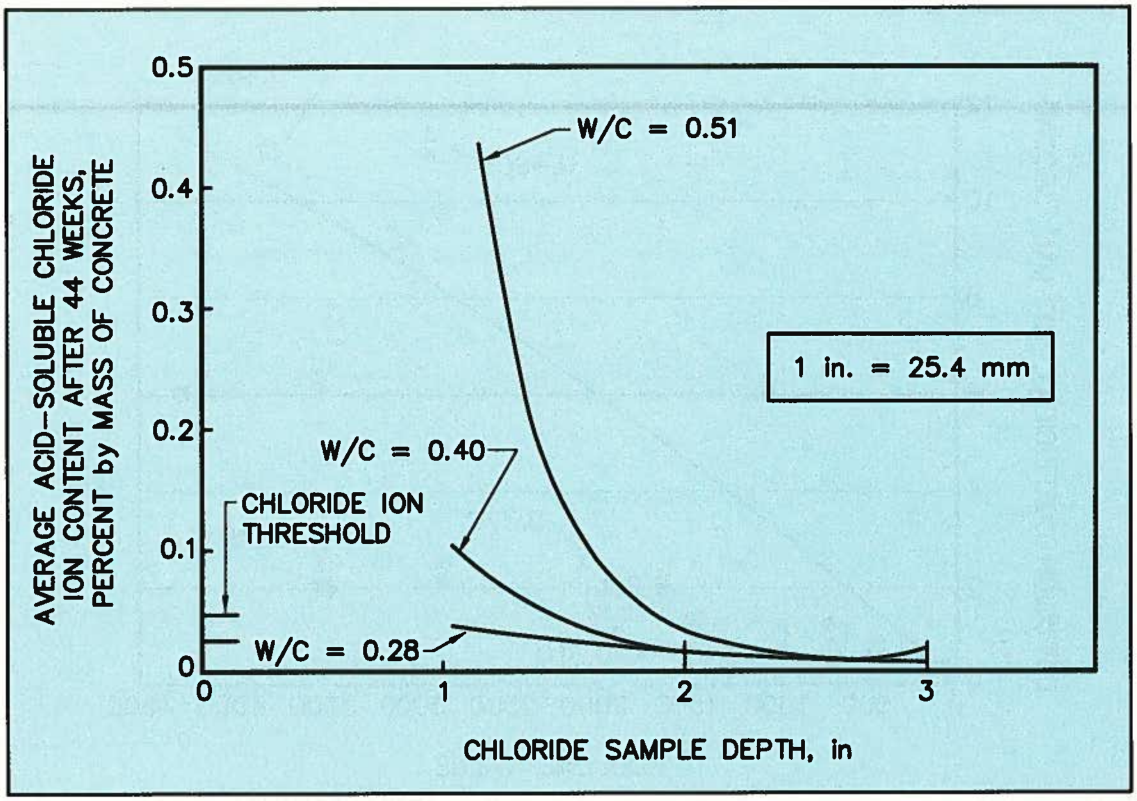

Fig. 2. Measured chloride profiles in moist-cured concrete from 1987 FHWA study in Illinois on 0.51, 0.40 and 0.28 w/c ratio conventional concrete after 44 weeks of severe cyclic ponding and air-drying testing.

- Prestressed concrete: 1500 or less coulombs
- Silica fume overlays: 1500 or less coulombs
- Latex-modified overlays: 1500 coulombs
- Bridge deck 2500 coulombs
- Substructures 3500 coulombs

VDOT requires the laboratory coulomb tests to provide coulomb values at least 500 below those listed above to account for variability in test results as shown in Fig. 4. These VDOT data support the concerns expressed in our 1996 PCI JOURNAL paper about the elimination of concrete suppliers from bidding, who could produce 2000 or 2500 coulomb concretes, just because the 1000 coulomb level could not be achieved, even in the face of data that showed that such 2000 coulomb concretes were equal in their diffusion coefficient to those with a 1000 coulomb rating. It is noteworthy that such 2000 coulomb concretes may be much more constructable and less prone to result in litigation than those with 1000 or less coulomb values, based on our experience across the United States.

The 1998 VDOT paper discusses jobsite problems with slump loss, placement, setting, consolidation, full-depth deck cracking for a particular 0.38 w/cm ratio concrete. The VDOT author stated, "This structure indicates the importance of proper use of admixtures, constructability, and good construction practices."

It is also noteworthy that the VDOT permits the field acceptance tests using the coulomb test to exceed the specification value by 1000 before being rejected. When the field test results exceed the specified coulomb value, the payment will be reduced by 0.005 percent for each coulomb above the specified value; however, the reduction shall not exceed 5 percent of the bid price of the concrete. Thus, prestressed concrete can be accepted with a coulomb value as high as 2500, with a payment reduction of 5 percent, and bridge deck concrete can be accepted with a coulomb value as high as 3500, also with a 5 percent reduction. These examples illustrate some of the variability issues associated with the coulomb test.

CRACKING OF HIGH PERFORMANCE CONCRETE

A compressive three-year study of transverse cracking in newly constructed bridge decks was undertaken by WJE for the U. S. National Cooperative Highway Research Program (NCHRP) between 1993 and 1996.[12] This study concluded that concretes with high early-age modulus of elasticity and low creep associated with high early-age compressive strength contribute significantly to this early-age cracking problem. Such cracking at 1 to 3 m (3.3 to 10 ft) apart along the length of the bridge, as discussed later, accelerates the corrosion of concrete reinforced with black, galvanized, and epoxy-coated bars.[13]

This project identified and ranked the factors or combinations of factors that contribute to transverse cracking of newly constructed bridge decks, an epidemic problem in the United States. These analyses on 18,000 bridge system scenarios determined that, for most bridges, concrete properties affect deck cracking more than any other factor.

As a result, a test procedure was developed to measure the cracking tendency of different concretes. In brief, the test involves casting a concrete ring against a steel inner ring that restrains the shrinking concrete and usually causes cracking. This test procedure was proposed for adoption by AASHTO so that transportation agen-

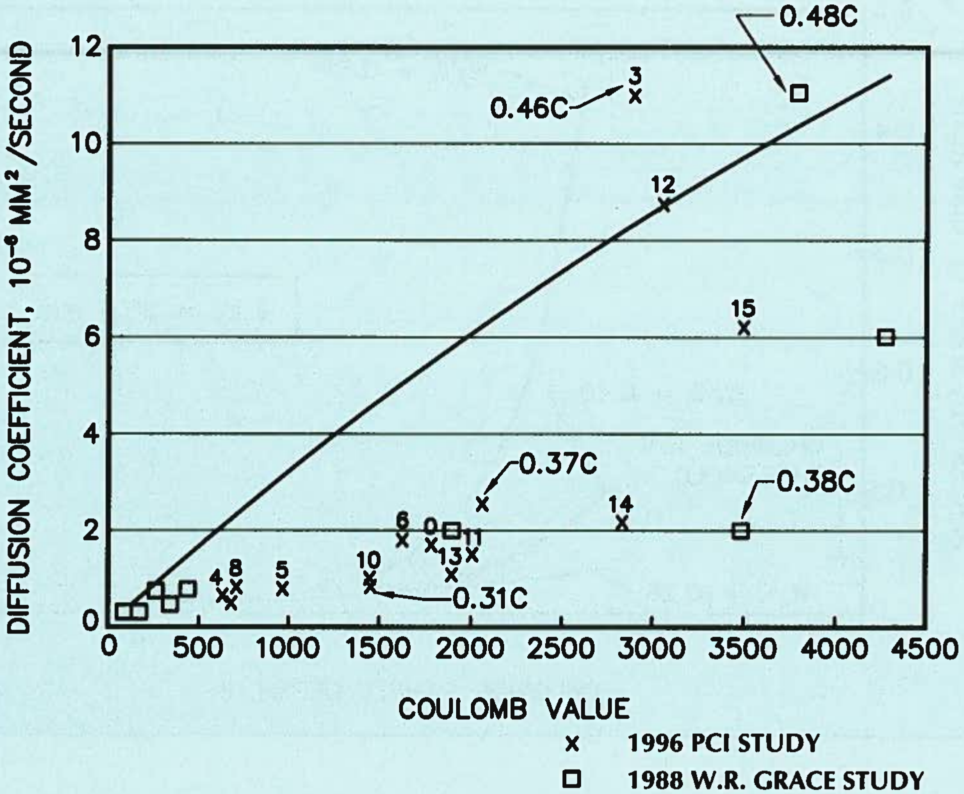

Fig. 3. Relationship between long-term diffusion coefficients and coulomb test values from 1988 W. R. Grace paper[10] and 1996 PCI JOURNAL paper by WJE.[5] C = conventional control concrete at 0.31, 0.37, 0.38, 0.46 and 0.48 w/c ratios.

cies across the country can test and develop concrete mixes that resist transverse cracking.

It is noteworthy that the measured behavior of the Portland-Columbia Bridge, during an application of a new bridge deck during this NCHRP project, showed an excellent correlation between the bridge deck cracking and the cracking of the restrained ring specimens that were stored at the bridge site during the reconstruction.

Numerous papers on high performance, low w/cm ratio concretes indicate that one-day compressive strengths and modulus of elasticity values are extremely high and can represent 90 to 95 percent of the 28-day strength, without any accelerated heat curing. Such early strength and modulus combined with natural low creep produce a concrete which, when restrained, appears to be brittle.

Another major cracking concern deals with excessive early-age heat of hydration evolution. The Texas Department of Transportation high performance concrete program developed a precast, prestressed bridge beam in which the more massive end block sections exceeded an internal concrete temperature of greater than 100°C (212°F), without any applied heat to the precast beam. Such excessive heat in combination with such high early-age strength and modulus of elasticity certainly create the conditions for cracking, and such cracking is occurring in the United States, not only in bridge decks but in other applications of high performance concretes. Many of these cracking problems were not seen in the past with conventional concrete.

It is critically important to realize that if the high performance concrete is cracked, the low diffusion properties are useless at the cracked locations. It is also noteworthy that published research papers are reporting significant autogenous shrinkage with moist-cured, high performance concretes with a low w/c ratio and high cementitious content. This shrinkage can be significant and has been attributed to self-desiccation of these high performance concrete. This autogenous shrinkage was apparently also not a problem with more conventional concretes in the past.

HIGH PERFORMANCE CORROSION-RESISTANT REINFORCING BARS

Controversy regarding the effectiveness of epoxy-coated reinforcing bars has prompted an increased interest in the development of new coatings for steel reinforcing bars and new types of solid corrosion-resistant steel and other metallic reinforcing bars. In 1993, the FHWA launched a five-year research project to evaluate corrosion-resistant reinforcing bars. The purpose of this research was to develop corrosion-resistant reinforcing bars that will result in a 75- to 100-year design life.

This comprehensive research program involved testing about 60 different organic, inorganic, ceramic, and metallic coatings or claddings on steel bars, as well as solid metallic bars. Bars being tested include epoxy-coated, other polymer-coated, ceramic-clad, galvanized-clad, epoxy-coated galvanized-clad, stainless-steel clad, nickel-clad, copper-clad, corrosion-resistant alloy-clad, inorganic silicate-clad, solid corrosion-resistant alloy steel, solid aluminum-bronze,

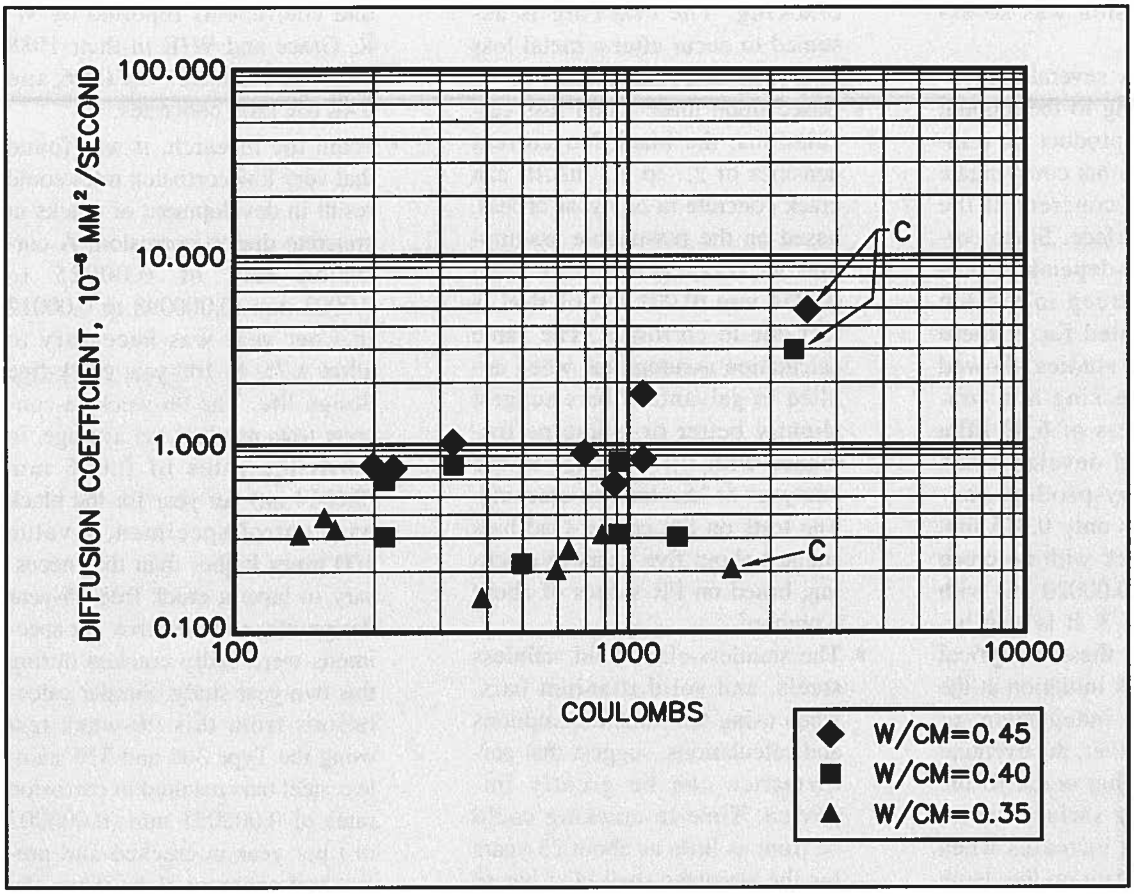

Fig. 4. Virginia DOT diffusion coefficients after 2.5-year ponding tests versus coulomb test values. C = conventional control concrete at 0.45, 0.40 and 0.35 w/c ratios.

solid titanium and solid stainless steel reinforcing.

All concrete slab specimens contained uncracked or precracked concretes, straight and 4D bent bars, and surface coating damage of 0.004 and 0.5 percent for the epoxy-coated bars or 0.50 percent for metallic-coated or solid metallic bars. The precracked specimens had a 0.30 mm (0.012 in.) wide crack directly over the embedded bar that had 25 mm (1 in.) cover. The pre-made crack penetrated directly down to the bar for a total length of 150 mm (6 in.).

Four major FHWA reports were published by FHWA in January 1995, May 1996, October 1996 and December 1998.[14-17] Other important reports by the authors were also published.[18-23] Significant conclusions discussed in these papers that are of interest to designers and owners of structures exposed to severe chloride environments are as follows:

1. The critical acid-soluble chloride concentration or chloride threshold for black reinforcing bars is about 0.2 percent of chloride ion by weight of cement for externally-applied or admixtured chlorides.[3,24] This is approximately 0.8 kg/m^3 (1.35 lb per cu yd) or 330 parts per million. This threshold is an extremely low number, which explains why corrosion of black reinforcing steel is such a major worldwide problem.

2. When epoxy-coated bars have the epoxy film damaged while exposing black steel, the chloride threshold to initiate corrosion is equal to that of black bars, that is, 0.8 kg/m^3 (1.35 lb per cu yd).

3. The chloride threshold for Type 316 stainless bars ranged from about 12 to 20 kg/m^3 (20.2 to 33.7 lb per cu yd) and averaged about 18 kg/m^3 (30.3 lb per cu yd) when Type 316 bars were used throughout the test specimens. When black bars were electrically connected to the Type 316 bars, the chloride threshold for Type 316 bars 316 averaged 18 kg/m^3 (30.3 lb per cu yd).

4. When Type 304 and 316 stainless bars were used throughout the specimens, with no black reinforcing bar electrical connection, the average corrosion thresholds, even in precracked concrete slabs, were very high, ranging from 11 to 18 kg/m^3 (18.5 to 30.3 lb per cu yd), values about 15 to 24 times as high as for black or damaged epoxy-coated bars.

5. A comprehensive review[16] of the world literature as well as analytical studies at WJE[6] from other corrosion studies show that for uncracked concrete, the amount of metal loss for black bars that will crack the concrete is as follows:

- The metal loss thickness calculated from four FHWA and Concrete Reinforcing Steel Institute (CRSI) studies range from 0.0013 to 0.038 mm (0.000051 to 0.0015 in.), and averaged 0.014 mm (0.00055 in.). These losses of steel surface are extremely small.
- Rodriguez et al.[25] concluded that the reduction of bar radius (metal loss due to corrosion) ranged from 0.015 to 0.038 mm (0.00059 to 0.0015 in.) at the time the cracks appeared. The authors conclude, "These data could explain the deterioration observed in some concrete structures although low corrosion rates are measured, since the amount of steel surface

lost from corrosion was so extremely low."

- Iding[16] undertook several analytical studies relating to the amount of corrosion by-product on reinforcing steel bars that could create cracking of the concrete at the corroding bar surface. Since corrosion is a time-dependent process, concrete creep in tension was also accounted for in these analyses. These studies showed that concrete cracking at a concrete tensile stress of 6.89 MPa (1000 psi) could develop when the corrosion by-product (expanded rust) was only 0.015 mm (0.00059 in.) thick with no creep and 0.005 mm (0.00020 in.) with a creep factor of 3. It is also believed, based on these analytical studies, that crack initiation at the bar perimeter is independent of clear cover; however, the eventual propagation of that crack to the exterior concrete surface due to internal pressure increases when the rust is dependent on the depth of the clear cover.
- This review shows that minimal corrosion current densities would result in steel reinforcing bar skin thickness losses of about 0.025 mm (0.00098 in.), which results in a volume of corrosion by-product which can crack concrete. This minimal steel loss is equivalent to a steel weight loss of only 0.64 percent for a 16 mm (0.63 in.) diameter bar. If a structure is to be protected for 75 to 100 years against corrosion damage, the average metal loss should, therefore, be 0.00025 to 0.0003 mm (0.0000098 to 0.000012 in.) per year.

6. The October 1996 FHWA report discussed the solution immersion corrosion tests on metallic-clad and solid metallic bars. Significant observations were as follows:

- Based upon well-known iron corrosion behavior, the listing in Table 2 shows the number of years of corrosion at different current densities, and the required polarization resistance (PR) associated with these current densities to cause concrete cracking. The cracking is assumed to occur after a metal loss of 0.0254 mm (0.001 in.).
- Based upon these metal loss calculations, the black bar current densities of 2.7 to 9.3 mA/ft^2 can crack concrete in one year or less, based on the reasonable assumption that concrete will crack when 0.0254 mm (0.001 in.) of steel is lost due to corrosion. The same calculation assumptions when applied to galvanized bars suggest slightly better or worse performance with little change in the one year or less time-to-cracking. The tests on the copper-clad bars suggest about five years to cracking, based on PR values of about 6 ohm.m^2.
- The stainless-clad, solid stainless steels, and solid titanium bars, when using the same assumptions and calculations, suggest that performance can be greatly improved. Time-to-cracking could be from as little as about 23 years for the stainless steel-clad bar to 50 years for the same bar during the initial 56-day testing at a lower chloride concentration. The solid stainless steel and titanium bars have consistently high PR values of about 100 to 700 ohm.m^2, which suggest time-to-cracking periods of about 100 years when tested in appropriate pH 13 solutions with added sodium chloride (NaCl).

7. The December 1998 Final FHWA Report which utilized an unusually severe corrosion testing of embedded reinforcing bars in uncracked and precracked concrete specimens, using straight and bent bars, all of which were damaged to simulate jobsite conditions, resulted in the following significant observations and conclusions:

- This severe two-year in-concrete test period produced an extremely high acid-soluble chloride content at 96 weeks at the bar level of about 18 kg/m^3 (30.3 lb per cu yd) or 25 times the corrosion threshold for black bars. The diffusion coefficient for this 0.47 w/c ratio concrete averaged about 26 x 10^{-6} mm^2/second, a value about twice as high as the 11 x 10^{-6} mm^2/second coefficients reported by W. R. Grace and WJE in their 1988 and 1996 studies on 0.46 and 0.48 w/c ratio concretes.
- From the research, it was found that very low corrosion rates could result in development of cracks in concrete due to corrosion. A corrosion rate of 0.00025 to 0.0003 mm (0.000098 to 0.00012 in.) per year was necessary to allow a 75- to 100-year crack-free design life. The 96-week in-concrete tests resulted, on average, in corrosion rates of 0.036 mm (0.0014 in.) per year for the black bar control specimen, a value 100 times higher than that necessary to have a crack-free 75-year design life, and the black bar specimens were badly cracked during this two-year study. Similar calculations from this 96-week test using the Type 304 and 316 stainless steel bars resulted in corrosion rates of 0.000051 mm (0.000002 in.) per year in cracked and precracked concrete slabs when the cathode was also stainless steel. This metallic loss rate with Type 304 and Type 316 stainless is six times lower than the 0.0003 mm (0.00012 in.) per year loss rate necessary to allow a 75- to 100-year crack-free design life, and the slabs were crack-free.
- All 12 slabs with black bars were badly cracked after 96 weeks of cyclic saltwater ponding and heating, and the extracted bars were severely corroded.
- The galvanized bars were tested in five configurations. The lowest corrosion rates were obtained when the galvanized bars were used in both mats in uncracked concrete. The average corrosion obtained in this configuration was 38 times less than that of the black bars. When a precracked specimen was used, the corrosion rates of slabs with a black cathode significantly increased. While galvanized bars are commonly bent, these data show that the corrosion increased when bent bars were tested with a black cathode. These data suggest that galvanizing should be done after fabrication.

At the end of 96 weeks, almost all slab specimens with galvanized bars exhibited cracks running parallel with the bars.

- The Type 304 stainless steel bars were tested in five configurations. The lowest corrosion rates were obtained when the Type 304 stainless steel bars were used in both mats. This configuration of bars was not influenced by the presence of the crack; both conditions had about 1500 times less corrosion than the black bar specimens. Of the 10 bars that were coupled with the black bar cathodes, five bars exhibited moderate-to-high corrosion currents. It was concluded that the Type 304 stainless steel was susceptible to chloride-induced corrosion when it was tested with a black bar cathode; whereas, when it was tested with a stainless steel cathode, it was not susceptible to any significant chloride-induced corrosion, even when in precracked concrete slabs.
- The Type 316 stainless steel bars were tested in five configurations. Low corrosion rates were obtained for all specimens containing the Type 316 stainless steel bars. The performance of the bars was not influenced by the presence of the crack, with all conditions having about 800 times less corrosion than the black bar specimens. During visual inspection of the slabs, only one of the bars exhibited any corrosion and, in that instance, the corrosion was regarded as minor. Results obtained for the Type 316 stainless steel bars indicate that these bars may be more suitable for in-concrete use than the Type 304 reinforcing bars and that these bars are less susceptible to galvanic effects if used in conjunction with black bars.
- The research supports continued use of epoxy-coated reinforcing bars, when tested in eight configurations, as a corrosion protection system, as in all cases, the corrosion rates of the epoxy-coated bars were less than that observed for the black bars. When epoxy-coated bars are to be used, it is appropriate to:
  - Use epoxy-coated reinforcing bars for both top and bottom mats of slabs.
  - Minimize damage to the reinforcing bars during shipment and placement.
  - Repair coating damage on-site.
  - Repair cracks in the concrete.

Table 2. Concrete cracking relationships.

Years required for 0.0254 mm metal loss and concrete cracking	Average current densities A/m²	Average polarization resistance ohm.m²
1	0.0218	1.19
10	0.0022	11.92
50	0.0004	60.47
100	0.00022	119.2

Note: 1 mm = 0.0394 in.; 1 m² = 10.76 sq ft.

Additionally, it was found for the epoxy-coated bars that:

- Use of the coated cathode significantly reduced the corrosion rates of all bar types, suggesting that the corrosion mechanism of epoxy-coated bars may be inhibition of the cathodic reaction that requires electrons, oxygen, and hydroxide to be present at the cathode bar surface.
- Few of the concrete specimens containing two layers of epoxy-coated bars cracked.
- Low corrosion rates were obtained for specimens containing epoxy-coated bars with high mat-to-mat resistance measurements.
- Solution immersion and cathodic debonding tests are poor predictors of long-term performance of the coated bars in concrete.
- Bars that used pretreatments did not perform significantly better than those without pretreatments.

CONCLUSIONS

Based on the FHWA investigation, it is concluded that Type 316 stainless-steel reinforcing bars should be considered at the design stage as a potential method for obtaining a 75- to 100-year design life. These bars had corrosion rates averaging 800 times lower than that of the black bars, even when tested in precracked concrete. Stainless steel bars are already in use in the United States and Canada, as a direct result of this FHWA research. For structures where repair to corrosion-induced damage is difficult, the additional costs associated with the stainless steel bars may be justified. Potential use includes marine substructures, tunnels, and bridges that carry significant traffic where closures for repairs would be problematic.

It is significant to note that all 12 tested bar types were more corrosion-resistant than black bars; however, this two-year severe test program shows that successful performance to achieve a 100-year service life requires about 99.8 percent reduction in corrosion current density, when compared to black bars. Numerous specimens with galvanized and epoxy-coated bars had 90 percent reductions of current, yet they were cracked at 96 weeks. This report clearly shows that corrosion current reductions of 500 times less than black bars is necessary and that reductions of only 10 times less corrosion current can and will eventually produce cracking and non-durable concrete structures.

A low water-cementitious ratio (less than 0.40) is essential in producing high quality, durable concrete. The precast, prestressed concrete industry has had a long tradition in fabricating precast concrete products with low water-cementitious ratios which have resulted in excellent long-term durabilities in aggressive environments.

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