

CONCRETE DURABILITY: CORROSION PROTECTION



Compilation 25

American Concrete Institute

Concrete Durability: Corrosion Protection

ACI Compilation 25

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Preface

ACI Compilations combine material previously published in Institute periodicals to provide compact and ready reference on specific topics. The Material in a compilation does not necessarily represent the opinion of an ACI technical committee — only the opinions of the individual authors. However, the information presented here is considered to be a valuable resource for readers interested in the subject.

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Durability of Concrete

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Corrosion of Metals in Concrete

On The Cover: The largest privately funded public works project ever built in Australia has been completed and now ranks among the leading examples of tube technology in the world. In this photo, an immersed tube unit fitted with survey towers for location and winch pontoons for lowering is shown in Sydney Harbour. The city's landmark Opera House is in the background.



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A Review of Data and Economics on Microsilica and Calcium Nitrite

Protection Against Chloride-Induced Corrosion

by Neal S. Berke, Donald W. Pfeifer and Thomas G. Weil

Concrete structures are exposed to many severe environments: chemical acid attack in chemical plants, sewage attack in underground pipes, acid attack in food processing plants, abrasive action at dam spillways and freeze-thaw deterioration in northern climates. One of the most severe and widespread problems, however, is the internal damage caused by the corrosive action of external chlorides on embedded reinforcing steel and prestressed strands in concrete. These external chlorides are found in deicing salts and saltwater-marine environments.

Typical problem structures in the frost belt are parking garages and bridge decks, along with their supporting members. In an ocean environment, seawalls, piers and bridge piles, girders and decks are subjected to similar severe corrosion attack. Even concrete buildings constructed on land but near the ocean shore have shown signs of corrosion from chloride-laden air. Since corrosion can begin when very small amounts of chlorides (1.0 to 1.6 lb/yd³ [0.6 to 0.95 kg/m³] of concrete) are present at the level of the steel, it has become a serious and destructive nationwide problem.

Concrete in a nonaggressive environment is a strong, durable and long lasting building material. In aggressive environments, precautions are frequently needed to protect the embedded reinforcing. With

the use of deicing salts since the early 1960s to keep our roads clear of snow and ice, concrete bridges and parking garages are now deteriorating at alarming rates due to chloride-induced corrosion. Although deicing chemicals are only sporadically used in parking garages, cars carry salt-contaminated snow and water into the garage, and much of this salt remains behind after the cars leave.

Structures in a marine environment are also a special problem. Support piles in seawater encounter the most severe corrosion condition. The upper portion of the pile dries out due to exposure to air, heat and sun, thus pulling chloride-laden moisture upward from the concrete below the water surface. Bridge piles in warmer ocean water have been known to deteriorate in less than fifteen years. Concrete at the water line is subject to continual wetting and drying cycles as well as chloride concentration at the waterline from this wicking action. Even the bridge beams and girders are not immune; atmospheric chlorides have been known to contaminate concrete as high as 70 ft (21.3 m) above the waterline.

The resultant steel corrosion product can expand in size by roughly four times its original volume and create significant tensile stresses within the concrete. This causes the concrete to crack and spall, which allows additional chlorides to enter at an even faster rate.

Eventually, deterioration of the concrete and reinforcing will occur, which will require expensive rehabilitation and may even cause structural problems or failure.

Because of the magnitude of this problem, design engineers must rely on corrosion protection methods to protect these newly-designed structures against chloride-induced corrosion. While corrosion protection is a normal part of the design package in other industries, it is a fairly new design feature for concrete structures. Two products which are helping to win this battle are microsilica and calcium nitrite concrete admixtures.

Corrosion protection-type admixtures are unique in that their protection mechanism becomes an integral part of the concrete matrix. Added at the batch plant, they are uniformly mixed through the concrete and protect all embedded bars, strands and steel accessories. Microsilica reduces the permeability of the concrete, considerably slowing the ingress of water-borne chlorides. Calcium nitrite corrosion inhibitor promotes the stabilization of the steel's natural passivating layer, thereby controlling the corrosion rate. These admixtures may be used individually, or in combination for the most severe corrosive environments. Both products have a long history of application and success.

Keywords: calcium nitrite; chlorides; corrosion; economics; reinforced concrete; reinforcing steels; reviews; silica fume.

Illustration Of Rebar In Chloride-laden Concrete

Need for Quality Concrete Design

Quality concrete is the first line of defense against chloride-induced corrosion. ACI 318 "Building Code Requirements for Reinforced Concrete" and ACI 357 "Guide for the Design and Construction of Fixed Offshore Concrete Structures" describe in detail design parameters which will help extend the service life of a structure in a corrosive environment. The key design factors mentioned include: concrete with a water-cement ratio no higher than 0.40; adequate concrete cover over the steel of 1½ in. (38 mm) minimum, with 2 in. (51 mm) being preferred in ACI 318 and 2½ in. (64 mm) for concrete in a marine environment; good air entrainment for freeze-thaw durability; and correct consolidation, finishing, curing and jointing practices. Engineers should remember that ACI 318 section 7.5.2.1 allows a cover tolerance of ± ⅜ in. (10 mm) for reinforcing placement in a slab. If the design calls for 1½ in. (38 mm) concrete cover, actual placement may provide only 1⅛ in. (29 mm) cover and still meet code requirements. This is not adequate cover in a corrosive environment, so a specified 2 in. (51 mm) may be necessary.

These ACI recommendations are for conventional concrete, thus they may not be sufficient to protect concrete structures against chloride-induced corrosion for their entire design life. The addition of microsilica and/or calcium nitrite to the concrete has proven to be effective supplemental corrosion protection, based on extensive laboratory studies and actual structures which have been in service for a long time.

Microsilica

Condensed silica fume is a manufacturing by-product of silicon, ferrosilicon, or other silicon alloy production in a submerged arc electric furnace. Condensed silica fume

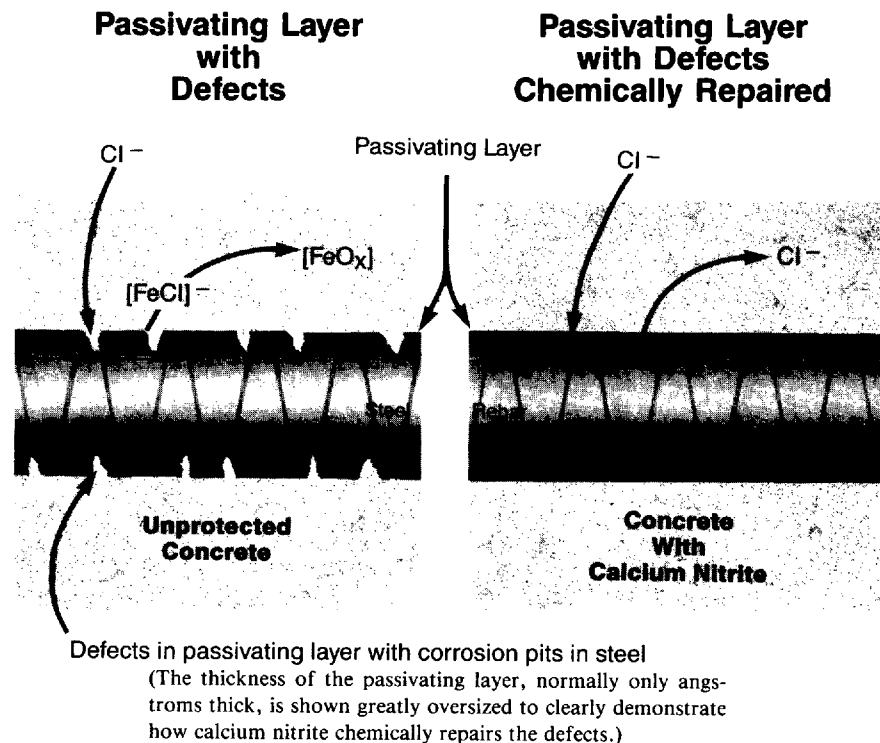


Fig. 1 — Illustration of reinforcing bar in chloride-laden concrete.

is classified as a microsilica which contains a silica (SiO_2) content of 85 percent or greater. It has an extremely fine particle size which has caused some researchers to call it a "super pozzolan". The ultra-fine particle size of roughly 0.1 micrometers (3.9×10^{-6} in.) allows the microsilica to fill the voids in the cement paste and between the cement paste and aggregate while the pozzolanic feature causes reaction with the excess calcium hydroxide ($\text{Ca}(\text{OH})_2$). This results in a far less permeable microstructure matrix that appears homogeneous with no gaps and no large crystals of $\text{Ca}(\text{OH})_2$.¹ Microsilica is available as a powder, a densified powder or a liquid slurry.

Concrete permeability to water and chloride ion is a design feature which unfortunately, cannot be measured quickly for upcoming projects. In response to this problem, the Federal Highway Administration developed a rapid test method for determining the apparent chloride permeability of various concretes. Entitled FHWA/RD-81/119 "Rapid Determination of the Chloride Permeability of Con-

crete",² it is commonly called the AASHTO T277 test method. Four-inch-diameter by two-inch-thick specimens are subjected to a 60 volt potential for six hours to measure the electrical charge passed in coulombs. There are at least a dozen parameters which can affect the final coulomb reading so an exact, reproducible test measurement is nearly impossible. Table 1 shows the five chloride permeability categories that were therefore created.

Concretes with coulomb readings that fall in the same category are considered to be equivalent as far as chloride permeability is concerned. Design engineers specifying microsilica have generally required coulomb readings in the 100 to 1,000 coulomb category which is classified as "very low". Some engineers believe inaccuracies exist in the FHWA rapid permeability test method and are specifying "percent microsilica by weight of cement" rather than coulomb levels. Usually a specified microsilica quantity is based on the severity of the service environment. A common microsilica amount is 7.5 percent by weight of cement for parking structures.

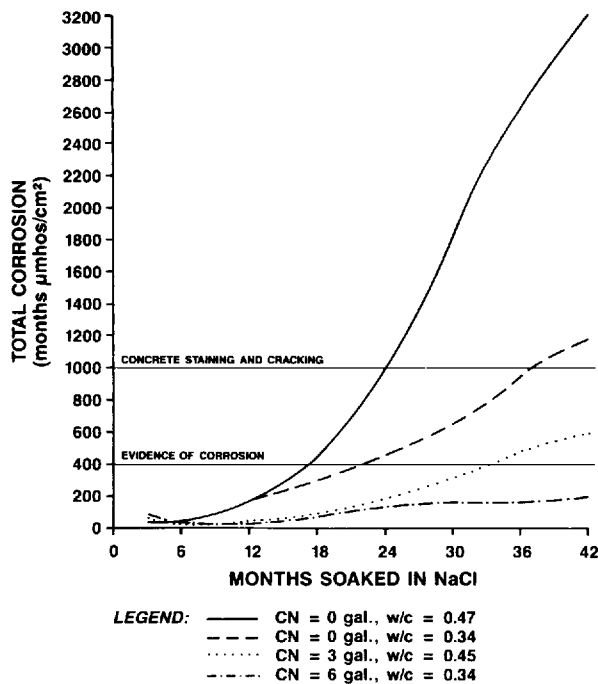


Fig. 2 — Calcium nitrate corrosion inhibitor — Total corrosion vs. time (in 3 percent NaCl).

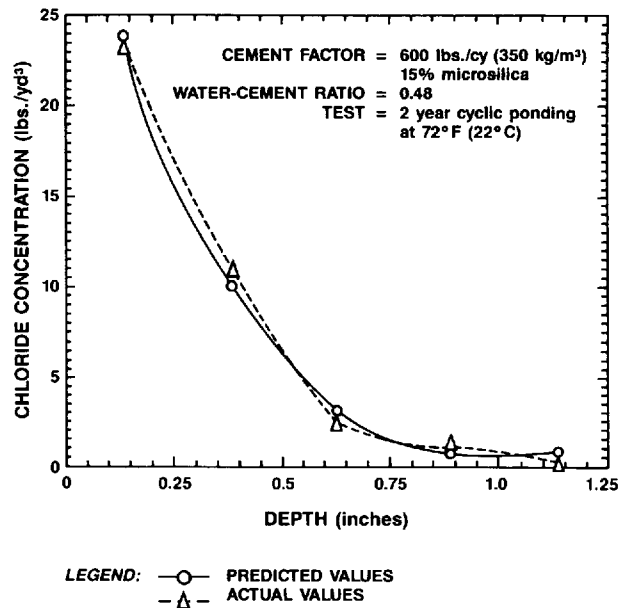


Fig. 3 — Chloride concentration vs. depth of concrete.

Table 1 — Chloride permeability based on charge passed.

Charge passed (coulombs)	Chloride permeability	Typical of
4,000	High	High water-cement ratio (>0.6). Conventional PCC.
2,000-4,000	Moderate	Moderate water-cement ratio (0.4-0.5) Conventional PCC.
1,000-2,000	Low	Low water-cement ratio (<0.4). Conventional PCC.
100-1,000	Very Low	Latex modified concrete internally sealed concrete.
100	Negligible	Polymer impregnated concrete. Polymer concrete.

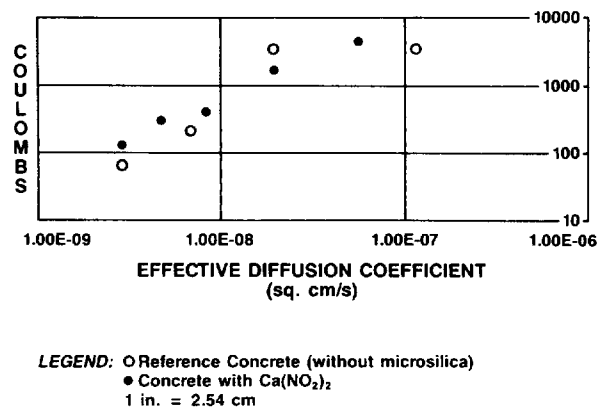


Fig. 4 — AASHTO T277 rapid chloride coulomb results vs. effective diffusion coefficients.

and 10 percent for piles in a marine environment.

The resistance of concrete to the passage of electricity has recently gained in popularity as an index of corrosion protection. Normal concrete measures between 4,000 to 9,000 ohm-cm. Adding microsilica to concrete can raise the concrete resistivity to 30,000 ohm-cm and above. It is generally believed that macrocell corrosion, the corrosion process between two separated mats of rebar, decreases with increasing resistivity.³ This should not be confused with stopping the corrosion process. Even though macrocell

corrosion may slow down at higher levels, microcell corrosion, which is corrosion at one location on the same bar, will continue unabated, but is considered less severe.

Several extensive research studies have been performed to examine the effects of microsilica on chloride ingress, concrete resistivity, and the AASHTO T277 coulomb test. Table 2⁴ shows mix designs with and without microsilica and/or fly ash. Table 3⁵ provides further data on the effect of mix design and microsilica on concrete properties, coulombs passed, and resistivity. These studies show that microsilica signif-

icantly increases concrete resistivity and decreases chloride permeability as measured by AASHTO T277. Furthermore, the efficiency of microsilica improves with decreased water-cement ratio. Effects of fly ash on rapid chloride coulombs and resistivity were found to be insignificant at 28 days and minor (compared to microsilica) at 180 days. Compressive strengths were also significantly improved with the use of microsilica.

Corrosion studies (ponding specimens with a 3 percent sodium chloride solution) are in progress on reinforced concrete specimens pro-

Table 2 — Concrete properties

Mix proportions (lbs/yd ³)								
Mix ID	Portland cement	Microsilica	Fly ash	Fine aggregate	Coarse aggregate	Water	HRWR (oz/cwt)	w/c
1	497	—	74	1148	1805	259	2	0.52
5	497	37	—	1183	1808	259	0	0.52
17	507	38	152	1055	1845	264	2	0.52
4	505	76	75	1087	1839	263	7	0.52
14	610	—	—	1173	1849	263	2	0.43
8	612	—	153	1021	1852	263	6	0.43
3	603	15	—	1142	1829	260	2	0.43
2	605	38	75	1044	1832	260	10	0.43
7	605	38	76	1044	1835	261	4	0.43
12	610	38	76	1051	1847	263	4	0.43
16	600	75	—	1156	1820	258	10	0.43
9	608	76	152	935	1847	262	10	0.43
13	708	—	76	1002	1840	263	10	0.37
11	708	15	151	911	1840	263	12	0.37
10	700	37	—	1024	1817	259	8	0.37
15	703	38	150	879	1825	260	10	0.37
6	706	76	76	918	1839	262	10	0.37

duced with the above concretes mixes. After 6 months of testing none of the specimens in the fly ash study were corroding.⁴ However, after 1.5 years of testing some control specimens with high water-cement ratios (0.48) are showing signs of corrosion while none of the microsilica specimens are yet corroding. The improvement in corrosion resistance with microsilica was attributed to a reduction in chloride ingress as well as the increase in resistivity. Microsilica-containing mixes showed substantial decreases in chloride ingress during this research.

A parallel study to that in Reference 4 was performed⁶ and the AASHTO T277 test results are shown in Table 4. The microsilica significantly reduced chloride permeability as measured by this method, and improvements were better at lower water-cement ratios (higher cement factors at constant water). The effects of fly ash at 28 days were found to be minor.

The NCHRP 244 Report⁷ has a test method to determine the effectiveness of sealers in preventing chloride ingress into concrete. The method involves the soaking of 4-inch concrete cubes in sodium chloride solutions and then determining absorbed chloride. Table 5 shows the results on the control and microsilica mixes. The microsilica mix absorbed about 50 percent less chloride than the control concrete even though these mixes used the same cement content and water-cement ratio. Note that, unlike the AASHTO T277 test, this method actually measures the chloride content of the concrete. The absorbed chloride levels for the microsilica mix were equivalent to those obtained from some of the best sealers tested⁷ in the NCHRP 244 study. An added advantage of the microsilica admixture over surface sealers is higher-strength concrete with little or no wear and tear.

Based upon the results of these various studies, the following con-

clusions can be made:

- Microsilica additions to concrete decrease the ingress of chloride during severe ponding tests.
- Microsilica additions to concrete significantly increase concrete resistivity and decrease AASHTO T277 rapid chloride permeability values (coulombs passed).
- Microsilica additions are more effective with lower water-cement ratios.
- Microsilica additions improve the compressive strength of concrete.

Calcium nitrite corrosion inhibitor

The highly-alkaline environment of concrete creates a natural protective, passivating layer of iron-oxide film on the steel in chloride-free concrete as shown in Figure 1. Chlorides which eventually migrate through the concrete can penetrate and break down the passivating layer to initiate the steel corrosion process. A corrosion inhibitor in-

Properties of fresh concrete			Properties of hardened concrete				
Slump (in)	Air content (percent)	Unit weight (lb/ft ³)	28-Day compressive strength (psi)	28-Day chloride permeability (coulombs)	28-Day resistivity (kohm-cm)	180-Day chloride permeability (coulombs)	180-Day resistivity (kohm-cm)
5.4	7.9	140	4060	8356	4.23	ND	5.93
3.7	7.8	140	5278	2201	18.61	ND	27.71
3.9	6.2	143	5829	1112	17.52	ND	44.72
4.5	6.6	142	6902	579	29.54	ND	72.33
5.0	6.0	144	5699	4192	5.53	3989	6.53
5.2	6.0	144	6221	3777	6.06	2154	12.09
4.5	6.8	143	5612	4880	7.79	ND	37.80
5.2	6.0	143	7439	456	33.4	319	71.46
4.5	6.6	143	7149	1035	21.35	ND	43.89
3.5	5.9	144	7279	610	21.68	ND	42.99
5.2	7.4	142	7265	301	51.75	342	72.53
3.9	5.9	144	8715	203	37.0	240	92.44
6.1	6.1	144	7265	2441	6.56	ND	12.02
5.6	6.5	144	8294	1081	11.59	ND	28.07
4.5	7.6	142	7120	816	25.47	ND	40.39
5.2	7.5	143	8149	416	26.57	ND	57.54
4.3	6.5	144	9077	283	38.36	ND	87.41

concrete delays the onset of corrosion initiation and controls the rate of corrosion by stabilizing the passivating layer of iron-oxide film.

Corrosion inhibitors have been used extensively and successfully in various industries for nearly 100 years. The use of an inhibitor in concrete is an extension of this concept. Although there are many inhibitors available, only calcium nitrite has proven corrosion inhibiting capabilities while enhancing the mechanical properties of concrete.

Corrosion testing on calcium nitrite concrete samples has been performed since the mid 1970's by private industry laboratories, the Federal Highway Administration and several state agencies. The conclusion of all these tests may be summarized by the following quote. . . "calcium nitrite can provide more than an order of magnitude reduction in the corrosion rate."⁸

The most recent privately funded, accelerated study contained 1,200

samples, 15 mix designs and three dosage rates of calcium nitrite, (0, 3 and 6 gallons per cubic yard).⁹ Samples were partially submerged in a 3 percent sodium chloride solution with a 1.3 in. (33 mm) concrete cover over the embedded reinforcing bars. Corrosion rates were determined using the polarization resistance techniques and also by periodically removing samples to determine the actual reinforcing steel appearance. By integrating these measurements over time, total corrosion is determined as shown in Figure 2. It has been found that corrosion rates above 20 $\mu\text{mho}/\text{cm}^2$ indicate corrosion.⁹ In Figure 2, samples with total corrosion over 400 month- $\mu\text{mho}/\text{cm}^2$ had corrosion rates above this threshold value of 20 $\mu\text{mho}/\text{cm}^2$. A total corrosion measurement of 1000 month- $\mu\text{mho}/\text{cm}^2$ is roughly equivalent to 1 mil average depth of corrosion on the steel, which is a sufficient amount to cause rust staining or cracking of the concrete at localized sites.

The points which can be drawn from Figure 2 are:

- The addition of calcium nitrite extends the time-to-corrosion initiation as opposed to an unprotected reference sample. This increases the service life of a structure.
- Total corrosion of samples with calcium nitrite is substantially less than in the unprotected corresponding reference mixes, especially at the 0.47 water-cement ratio. The 0.34 water-cement samples are beginning to show this difference.
- The corrosion rate in calcium nitrite protected concrete is less severe after corrosion initiation than in an unprotected concrete. This is due to the reservoir of calcium nitrite that is still available within the concrete. This feature also helps extend the service life of a structure.

Extensive research on the capabilities of corrosion inhibitors has also found that the protective benefits are increased as more is added. For calcium nitrite, it has been

Table 3 — Concrete properties

Mix proportions (lbs/yd ³)						
Mix ID	Percent silica fume by mass of cement	Portland cement	Microsilica	Fine aggregate	Coarse aggregate	w/c
1	0	587	0	1194	1819	0.48
2	3.75	588	22.0	1197	1822	0.48
3	7.5	585	44.6	1268	1813	0.48
4	15.0	591	89.0	1204	1834	0.48
5	0	556	0	1205	1723	0.43
6	3.75	593	22.2	1261	1838	0.43
7	7.5	573	43.0	1167	1779	0.43
8	7.5	575	43.1	1246	1782	0.43
9	15.0	598	91.2	1295	1853	0.43
10	0	571	0	1312	1770	0.38
11	3.75	585	21.9	1344	1814	0.38
12	7.5	591	44.3	1358	1832	0.38
13	15.0	599	90.0	1377	1858	0.38
Mean Values		584	39.3	1264	1811	0.43

NOTE: ROME is the relative dynamic modulus of elasticity. Scale factor ratings: 1 = Very slight scaling (no coarse aggregate visible); 2 = slight to moderate scaling; 3 = moderate scaling (some coarse agg. visible). ND = not determined.

*These chloride values are at 18 months.

shown that two gallons per cubic yard of concrete of a 30 percent solution will protect against the corrosive action of six pounds of chlorides per cubic yard. Table 6 shows that greater corrosion protection is afforded as more calcium nitrite is added.

Another major study, completed in 1987, investigated the macrocell corrosion rate of minislabs.¹⁰ The minislabs were subjected to chloride ponding and air drying for 48 weeks. Additional slabs with calcium nitrite were produced and tested along with the original slabs and the supplemental tests were extended from 48 to 116 weeks. Results concerning the control specimens and those with calcium nitrite are shown in Table 7, from which it is noted that calcium nitrite improves corrosion resistance. At all water-cement ratios and concrete cover, the time to corrosion was delayed in samples containing calcium nitrite, and its severity reduced once corrosion was initiated. As the water-cement ratio is decreased and the concrete cover is increased, the corrosion inhibition benefits of calcium nitrite significantly increase

even at relatively high chloride contents. These results compare favorably to published results previously mentioned.^{8,9}

Concrete permeability — chloride diffusion curves

Concrete is not a solid material, but rather one which allows the ingress of water and, unfortunately water-borne chlorides. The water-cement ratio has a dramatic affect on permeability, with lower-cement ratio concretes being much less permeable.¹¹ Concretes with the same water-cement ratio have the same general permeability, assuming that the other concrete parameters are equivalent or similar. If the water-cement ratio of a concrete mix is known, an estimate of the time to corrosion may be predicted. This assumes that quality materials are used, no extra water is added at the jobsite, proper consolidation, finish, curing and jointing procedures are adhered to and cracking does not occur. In essence, this means a quality project from a quality design.

Concrete permeability has been studied for close to fifteen years in

privately funded studies on concrete admixtures.¹² By analyzing data from long-term chloride-ponding studies and combining this with field data, concrete chloride diffusion coefficients have been determined. Chloride profiles were determined on laboratory minibeams that were subjected to alternate chloride ponding for two years.¹² The data was then analyzed using nonlinear regression analysis. The model used Fick's Law diffusion equation for a semi-infinite slab:

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

where C is the concentration at depth x and time t , D_{eff} is the effective diffusion coefficient and C_o is the pseudosurface concentration of chloride. The program solves for C_o and D_{eff} with no assumptions other than Fick's Law. The above model has been used by others^{13,14} and the model correlates well with the data¹² shown in Figure 3. Table 8 gives the effective diffusion coefficients for several concrete mixes. Note that microsilica additions significantly reduce the effective diffu-

Properties of fresh concrete			Properties of hardened concrete					
Slump (in)	Percent air	Unit weight (lb/ft ³)	28-Day compressive strength (psi)	28-Day chloride permeability (coulombs)	28-Day resistivity (kohm-cm)	300 cycles freeze thaw (ROME)	Scale factor	10 month total chloride 0.5-1.0 in. (lb/yd ³)
6.0	7.0	144	5160	3663	7.7	105	1	4.90*
5.4	7.0	144	5417	3175	16.3	—	—	ND
6.75	9.0	147	6346	348	45.4	100	0	ND
5.75	7.0	145	7357	198	94.7	102	1	0.43
9.75	10.5	138	5264	2585	9.3	—	—	ND
4.0	7.4	147	6547	2210	22.1	—	—	ND
8.25	8.0	140	7214	213	67.7	100	0	0.54
9.1	1.0	143	6751	—	67.0	—	—	ND
7.0	6.0	149	8582	98	118.0	—	—	ND
8.75	8.0	144	5782	3485	10.8	104	1	1.63*
3.5	8.0	147	9312	736	24.3	—	—	ND
8.25	7.0	149	9288	132	73.9	104	1	0.41
6.0	6.0	151	12119	75	161.0	102	1	0.53*
6.8	7.8	145	7318	1410	55.0	102	1	ND

fusion coefficient. This agrees qualitatively with AASHTO T277 rapid coulomb test results^{4,5,6} and with NCHRP cube permeability tests.¹⁵ Results for the unpublished studies^{6,15} are given in Tables 4 and 5.

By plotting the data from Table 7 on a log-log plot, a fairly good relationship between the rapid chloride permeability test and the effective diffusion coefficients (one curve for concrete with calcium nitrite and one for concrete without nitrite) can be shown (Fig. 4). This enables the designer to make a rough prediction as to the effective coefficient for a given coulomb level.

Upon determining effective diffusion coefficients, chloride profiles can be predicted as a function of time for a given general concrete quality if the chloride exposure and yearly average temperature are known. Unfortunately, except for the case of total immersion in sea water or a brackish environment, the chloride concentration at the surface is difficult to predict.

Data taken from several northern region parking structures in distress was analyzed, and chloride concen-

tration at the surface versus time was plotted. It appears for the worst case that the pseudoconcentration at the surface increased at the rate of about 2 lb (.9 kg) per year up to about 30 lb/yd³ (18 kg/m³) as shown in Figure 5. This agreed with severe alternate ponding of laboratory samples that also showed this concentration after 2 years. Note that a constant 3 percent NaCl exposure resulted in a surface concentration between 15 to 16 lb/yd³ (9 to 9.5 kg/m³).

Figure 6 shows the estimated chloride profile for a parking deck slab after 40 years, with an average temperature of 50 F (10 C). The model assumes that one dimensional diffusion is occurring with a surface concentration that increases by 2 lb (.9 kg) per year and becomes constant at 30 lb/yd³ (18 kg/m³). Figure 6 shows the decrease in chloride content at 40 years as the water-cement ratio is decreased. At some point it is more convenient to switch from a water-cement ratio measurement to a coulomb measurement. It was determined that conventional concrete with a 0.35 w/c ratio is approximately equivalent

to concrete at 1000 coulombs as measured by AASHTO T277.

For square concrete piles in the ocean, a 2-dimensional diffusion model was used to predict chloride profiles. Once again, the worst case splash-tidal zone position was studied, and a surface concentration of 30 lb/yd³ (18 kg/m³) of chloride was used (chloride concentrations at other heights would be expected to be lower). Figure 7 shows the predicted chloride profiles at the corner of a 14-in. (35 cm) square pile after 50 years at 65 F (18 C) water temperature.

Based on research and analysis of existing data, estimated projections can be made as to the amount of chloride ingress over time. A corrosion protection system can then be selected to meet this project life expectancy.

Corrosion protection life cycle costs

The design life of structures is an important design consideration. A structure which could potentially be a hazard at ten to twenty years due to corrosion deterioration would

Table 4 — Summary of chloride permeability test data

Mix no.	W/C	Silica Fume		Average compressive strength (psi)	Chloride permeability	
		(lbs/yd ³)	(percent by wt. of cement)		Coulombs	AASHTO T277 classification
1	0.52	0	0	4290	6353	High
5	0.52	37	7.5	4960	2110	Moderate
17	0.52	38	7.5	5290	1309	Low
4	0.52	76	15.1	5530	809	Very low
14	0.43	0	0	5190	5919	High
3	0.43	0	0	5980	3049	Moderate
3	0.43	15	2.5	5730	3436	Moderate
2	0.43	38	6.3	6530	587	Very low
7	0.43	38	6.3	6870	1079	Low
12	0.43	38	6.3	6770	733	Very low
16	0.43	75	12.5	6770	302	Very low
9	0.43	76	12.5	8140	433	Very low
13	0.37	0	0	6460	2868	Moderate
11	0.37	15	2.1	7510	1465	Low
10	0.37	37	5.3	6860	910	Very low
15	0.37	38	5.4	7770	676	Very low
6	0.37	76	10.8	9380	576	Very low

probably last fifty years with the proper corrosion protection system. Two examples were used to determine the estimated cost of several corrosion protection systems. The examples compared the estimated "hidden cost" of an unprotected concrete structure to the costs of protection offered by microsilica or calcium nitrite or microsilica with calcium nitrite. Both examples assumed a cement factor of between 600 to 700 lb (272 to 318 kg), the use of a superplasticizer for workability and properly entrained air for freeze-thaw durability.

The first example was a parking structure with a design life of forty years, a water-cement ratio of 0.40 and concrete cover over the reinforcing steel of 1½ in. (38 mm). An estimated cost comparison of no protection, calcium nitrite corrosion inhibitor and microsilica was made. From Figure 6, the amount of chlorides at the reinforcing bar at forty years was approximately 13 lb/yd³ (7.7 kg/m³). By using another variation of the curves, the time-to-corrosion was estimated at ten years, with staining and cracking in roughly 15 years. For 600 coulomb concrete (approximately 7½ percent microsilica), the chloride content was roughly 7 lb/yd³ (4 kg/m³).

For the unprotected structure, repairs would have to be made in 15 years to allow it to last a full forty year design life. Assuming a cost of \$10.00/ft² (\$108/m²) to \$15.00/ft² (\$161.00/m²) at that time to rehabilitate the concrete slabs and a 7 percent interest rate on funds, the present value of this future repair cost would be \$3.33/ft² (\$36.00/m²) to \$5.44/ft² (\$59.00/m²).

For calcium nitrite protection, 4 gal (0.015 m³) of a 30 percent solid solution should be used to reach the forty year design life (Table 6). The cost of the calcium nitrite protec-

tion system is roughly \$0.50 ft² (\$5.40/m²).

Even with the low permeability protection of 7½ percent microsilica by weight of cement, 7 lb (3.2 kg) of chloride ion will penetrate to the reinforcing bar level in forty years, and this is enough to cause staining and cracking. The cost of the microsilica is roughly \$0.65/ft² (\$7.00/m²). This system allows 40 years of service, although some deterioration may be present. The addition of 2 gal (0.0075 m³) of calcium nitrite along with the microsilica offers extra protection for the full forty years. This combination system cost is roughly \$0.90/ft² (\$9.70/m²).

The second example was a support pile in an ocean environment. The water-cement ratio was 0.35, water temperature was 65 F, (18 c) concrete cover was 2½ in. (64 mm) to main reinforcement (per ACI 357) and a design life of 50 years was required. Using Figure 7 and

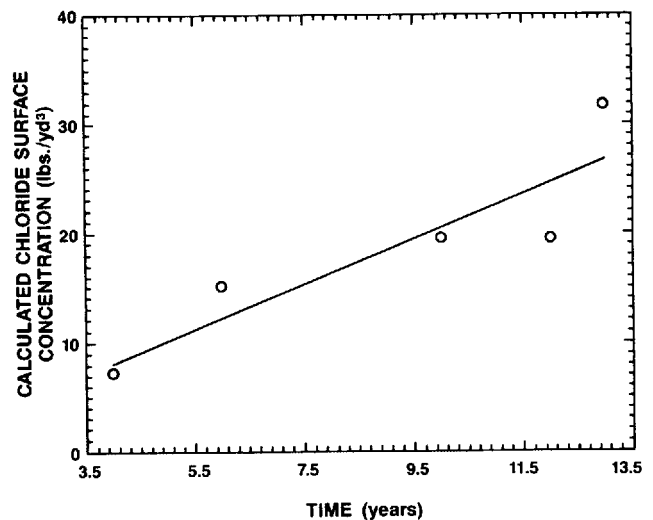
other calculations, it was determined that in the splash zone there may be 17 lb (7.7 kg) of chloride per yd³ (0.76 m³) of concrete at the reinforcement level. At this estimated chloride level:

- Unprotected concrete would have deteriorated long before the 50 year design life.
- Calcium nitrite required for fifty year protection is 5½ gal (0.02 m³) per yd³ (0.76 m³) (Table 6). Although 5½ gal (0.02 m³) protects to 15½ lb (7kg) of chlorides, the reservoir of calcium nitrite in the concrete should allow adequate protection to fifty years before deterioration begins.
- Microsilica required would be roughly 10 percent by weight of cement (approximately 300 coulombs) which would allow:

—Estimated chloride content at 50 years of 6 lb/yd³ (3.6 kg/m³). Here again, visible deterioration

Table 5 — NCHRP 244 Test Method results with and without microsilica.

Average weight gain after 21 days in salt solution, weight loss after 21 days of final air drying, chloride ion content of half cube, and reduction of weight gain, weight loss and chloride ion content when compared to the control concrete cubes.		
	Mix no. 1	Mix no. 2
Weight gain (percent of weight)	0.90	0.54
Reduction of weight gain (percent)	—	40.0
Weight loss (percent by weight)	0.66	0.37
Reduction of weight loss (percent of gained water)	73.0	69.0
Half-cube chloride ion content (percent by weight of concrete)	0.178	0.088
Reduction of chloride ion content ion concrete (percent)	—	51.0
Mix No. 1 - Control concrete containing a superplasticizer Mix No. 2 - Concrete containing 6 percent microsilica Mix Design: Cement = 515 lbs. W/C = 0.40 Air = 6.5 percent f'c = 6050 psi for control f'c = 8670 psi for microsilica mix		



LEGEND: ○ FIELD DATA POINTS

Fig. 5 — Calculated chloride surface concentration vs. time.

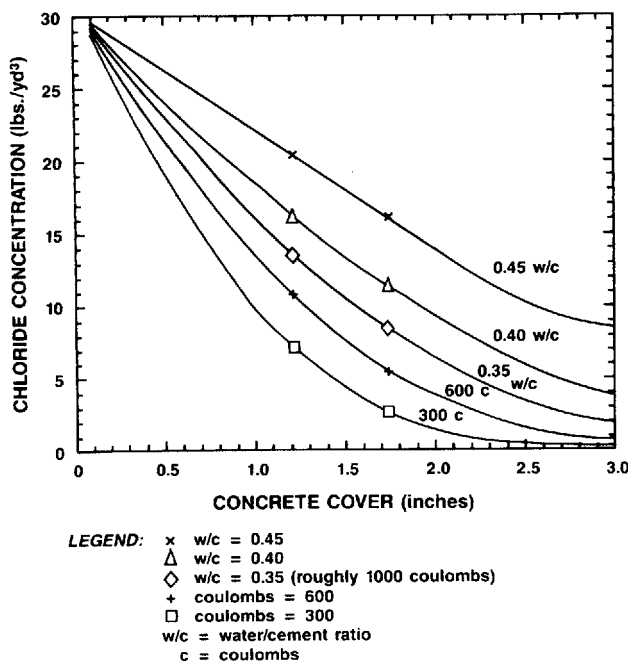


Fig. 6 — Estimated chloride concentration in a parking deck (ambient temperature = 50 F [10 C]; time = 40 years).

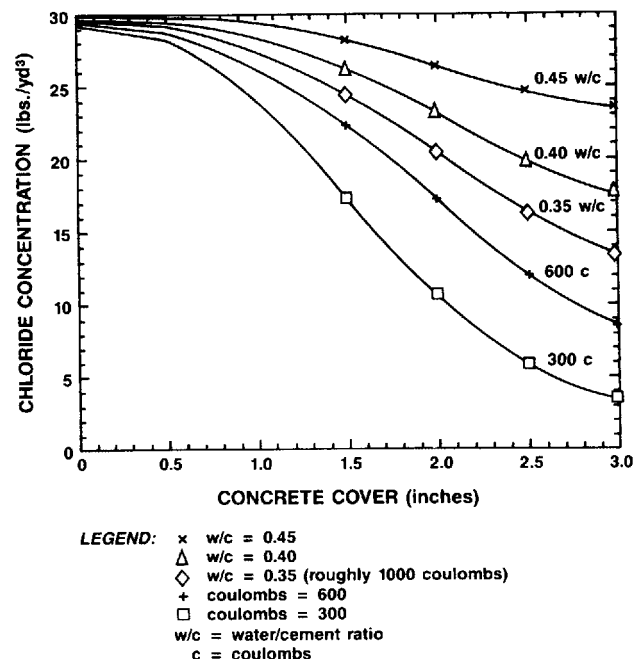


Fig. 7 — Estimated chloride concentration in concrete square piles in a marine environment (water temperature = 65 F [18 C]; time = 50 years).

Table 6 — Calcium nitrite dosage rates vs. chloride protection

Calcium Nitrite (gal/yd ³)	Chlorides (lb/yd ³)
2	6.0
3	9.9
4	13.0
5	15.0
6	16.0

would probably not begin until the 50 year design life is reached.

—Additional calcium nitrite required for the full 50 year protection is 2 gal/yd³.

Rather than look at cost of repair in this example, since rehabilitation costs in a marine environment are very high, a comparison of a different type will be made. By extending the 0.35 w/c chloride diffusion curve to even greater concrete cover over the reinforcement (not shown here), it is determined that an additional two inches of cover (for a total of 4½ inches) will allow the unprotected pile to reach fifty years of design life. Therefore, 5½ gallons per cubic yard of calcium nitrite or 10 percent microsilica by weight of cement is roughly equivalent to an additional 2 inches of concrete cover over the reinforcing or prestressed strand. In fact, the 1983 edition of AASHTO's "Standard Specifications for Highway Bridges", Section 4.17, "Concrete Exposed to Sea Water", calls for a clear cover of 4 in. (102 mm) to the nearest reinforcement.

Conclusion

Protection against chloride-induced corrosion is now a require-

Table 7 — Corrosion inhibitor test results

Time to corrosion and corrosion current							
Water-cement ratio	Sample type	Time to corrosion (weeks)	Concrete cover 1 in. (25 mm)		Concrete cover 2 in. (51 mm)		
			Corrosion current, μ a		Time to corrosion (weeks)	Corrosion current, μ a	
			48 weeks	116 weeks		48 weeks	116 weeks
0.50	Control	6	245	*	64	0	*
	4 gals CN	21	90	*	**	0	0
	6 gals CN	11	73	*	**	0	0
0.40	Control	5	128	*	***		
	4 gals CN	21	10	19	**	0	0
0.32	Control	2.5	110	*	***		
	4 gals CN	**	0	0	**	0	0
Chloride content (lbs/yd ³)							
Water-cement ratio		Concrete cover 1 in. (25 mm)		Concrete cover 2 in. (51 mm)			
		44 weeks	116 weeks	44 weeks	116 weeks		
0.50		16.8	20.0	0.8	3.0		
0.40		5.7	13.2	0.5	n/a		
0.32		2.0	10.4	0.3	n/a		

*Sample failed

**No corrosion activity after 116 weeks

***Sample not made

CN - Calcium nitrite sample

Information Source:

1. Protection Systems for New Prestressed and Substructure Concrete Report No. FHWA/RD-86/193, April 1987, US Department of Transportation
2. Corrosion Protection Tests on Reinforced Concrete September 1987 Wiss, Janney, Elstner Associates, Inc.

ment for concrete structures where deicing salts are used, or in marine environments. Fortunately, the engineering community now has the tools, in the form of corrosion protection systems, that will help these structures meet their design life.

Research has shown that quality concrete is the first line of defense against chloride-induced corrosion. Unfortunately, where deicing salts are used or in a marine environment, this is usually not enough to allow a structure to meet its design life. Microsilica and calcium nitrite corrosion inhibitor are two proven corrosion protection products which help delay the onset of corrosion. Microsilica reduces the permeability

of the concrete, which slows the ingress of chloride ions to the reinforcement, while calcium nitrite controls the rate of corrosion once the chlorides reach the reinforcement. Both products have been used separately or in combination to protect cast-in-place parking structures throughout the country. Microsilica is also rapidly gaining popularity for bridge decks and overlays, as well as where a combination of high strength and corrosion protection are required. Calcium nitrite has been used in approximately 250,000 yd³ (190,000 m³) of prestressed concrete girders and piles in bridges in all environments, and double tees for parking structures.

Table 8 — Comparison of effective chloride diffusion coefficients versus AASHTO T277 coulombs

Mix	Cement kg/m ³	Factor (lb/yd ³)	Percent MS	Calcium nitrite l/m ³ (gal/yd)	Water cement ratio	AASHTO T277 coulombs	10 ⁻¹⁰ eff cm ² /s
1	347	(587)	0	0	0.48	3663	11.0
2	343	(580)	0	20 (4)	0.48	4220	6.0
5	350	(591)	15	0	0.48	198	0.7
6	359	(607)	15	20 (4)	0.48	253	0.5
11	363	(614)	7.5	10 (2)	0.43	380	0.8
13	338	(571)	0	0	0.38	3485	2.0
14	352	(595)	0	20 (4)	0.38	1838	2.0
17	354	(599)	15	0	0.38	75	0.3
18	313	(580)	15	20 (4)	0.38	119	0.3

Note: D_{eff} at 22 C (72 F)**References**

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Corrosion Inhibitors in Concrete

by Neal S. Berke

Steel-reinforced concrete is one of the most widely used construction materials throughout the world, but corrosion of the reinforcing steel is causing the early failure of structures in environments where concrete is subjected to an ingress of chlorides.

The alkaline environment provided by the concrete protects the steel from corrosion. However, when the concrete is subjected to chlorides or carbonation, the normally passive steel can begin to corrode. The corrosion products have 2 to 4 times the volume of the original steel and the stresses produced cause the concrete to crack. Delamination and spalling eventually occur and the integrity of the concrete structure is lost.

Several methods of corrosion protection are currently being used to extend the life of reinforced concrete structures in a chloride environment. Commercially used methods include low permeability concrete produced with superplasticizers, microsilica, epoxy-coated steel, cathodic protection, latex modified concrete, sealers, and calcium nitrite corrosion inhibitor. In numerous cases, combinations such as microsilica and calcium nitrite or calcium nitrite and epoxy-coated steel have been successfully used.

The advantages of using inhibitors to provide corrosion protection are that the inhibitor is distributed throughout the concrete, protecting all the steel; and that concrete's low permeability prevents the inhibitor from being lost. The disadvantage is that the inhibitor must not have adverse effects on the concrete properties in the plastic and hardened states.

While numerous inhibitors have been suggested, only a small group have been seriously studied, and

only one (calcium nitrite) is used commercially on a wide scale in the United States, Japan, and Europe. This is partially based upon the fact that calcium nitrite in general improves the properties of hardened concrete and its accelerating effect is well within the range acceptable for ASTM C494 admixtures. This accelerating effect is easily offset using commercially available ASTM C 494 retarders.

Pre-1980 studies

The use of inhibitors in concrete has been reviewed by Griffin,¹ Craig and Wood,² Treadaway and Russell,³ Slater,⁴ and by Berke.⁵ With the exception of Berke's review in 1989, the reviews were written on work performed before 1980. Early studies looked at numerous inhibitors, with the most attention focused on sodium nitrite, potassium chromate, sodium benzoate, and stannous chloride.

Craig and Wood² examined the mechanical properties of mortars produced with sodium nitrite, potassium chromate, sodium benzoate, and calcium chloride. They found a marked decrease (as high as 20 to 40 percent) in compressive strengths when the inhibitors were added to the mortars. In contrast, calcium chloride increased compressive strength. Tensile strengths were adversely affected by sodium nitrite and sodium benzoate, but not by potassium chromate.

Corrosion testing showed that the inhibitors tested were not detrimental in the absence of admixed chloride. However, only sodium nitrite inhibited in the presence of chloride (Table 1).

Treadaway⁶ and Treadaway and Russell³ conducted tests of sodium nitrite and sodium benzoate. They also found a decrease in compressive strength when these chemicals were added to concrete. Corrosion

tests were conducted in alkaline solutions and in cement extracts. In the cement extract experiments they found that sodium nitrite inhibited corrosion in the presence of chlorides, whereas sodium benzoate did not. In their solution testing they emphasize that corrosion was uniform with sodium benzoate and pitting occurred with sodium nitrite. Initiation of corrosion was delayed with sodium nitrite, and the delay increased with the amount.

Solution testing involved the aeration of unbuffered samples over several weeks (the solution did not maintain its initial protective concrete-like properties) and the solutions were most likely carbonated. In such a case, uniform corrosion would be expected for benzoate and pitting would still be the mechanism of attack for nitrite. In good quality concrete, where carbonation is unlikely at the reinforcement level, the mechanism of corrosion in the presence of chloride will be pitting.^{7,8} Note that pitting is a highly localized form of corrosion. Thus, the sodium nitrite inhibitor did not change the corrosion process into one of pitting, and did protect the steel until carbonation took place.

The reviews^{1,2} mention the work of Arber and Vivian^{9,10} with stannous chloride, and indicate that it is a corrosion inhibitor. In Arber and Vivian's experiments stannous chloride appeared to be a noncorrosive set accelerator. Stannous chloride, in short term testing with and without admixed chlorides, and with no further chloride exposure from the environment, did appear to decrease corrosion as measured by tensile strength regression and steel appearance⁹ (mild corrosion and tensile loss still occurred). Stannous chloride on the other hand did not behave as an inhibitor.

The authors⁹ attribute the improved corrosion resistance to the

Table 1 — Visual observations of Craig and Wood (reference 2)

CaCl ₂ (percent)	Chemical		Percent of surface corroded					
	Compound	Percent	0	20	40	60	80	100
0			*†					
2				*†				
4					*†			
0	NaNO ₂	2	*†					
0	NaNO ₂	4	*†					
2	NaNO ₂	2	*†					
2	NaNO ₂	4	*†					
0	NaC ₂ H ₃ O ₂	4	*†					
	NaC ₂ H ₃ O ₂	6	*†					
2	NaC ₂ H ₃ O ₂	4			*			†
2	NaC ₂ H ₃ O ₂	6			†			*
0	K ₂ CrO ₄	2	*†					
0	K ₂ CrO ₄	4	*†					
2	K ₂ CrO ₄	2			*†			
4	K ₂ CrO ₄	4		†	*			

Note: * = specimen cured in air; † = specimen cured in water.

stannous ion. However, some of it is most likely due to a decrease in chloride content, since stannous chloride is 37 percent chloride, stannic chloride is 54 percent chloride, and calcium chloride is 64 percent chloride. Recent tests by Hope and Ip showed that stannous chloride was not a corrosion inhibitor.¹¹

Based upon the inhibitor experiments up to the mid 1970s, it is not surprising that inhibitors were not in large scale use in concrete. Sodium nitrite was identified as a good inhibitor, but it caused severe strength loss. Sodium or potassium salts of chromates and benzoates gave mixed results on inhibition and also reduced strength. Stannous chloride increased strengths, but still contained chloride and was relatively expensive. In short, the concrete technicians needed an inhibitor that was not detrimental to the concrete's properties, improved corrosion resistance conclusively, and was affordable.

Calcium nitrite

The commercial production of calcium nitrite as a stable solution of up to 40 percent dissolved solids encouraged the use of calcium nitrite as a corrosion inhibitor. Unlike sodium nitrite, in which the sodium cations are detrimental to strength and alkali-aggregate reactions, cal-

cium nitrite was found to be non-detrimental to mechanical properties. For example, calcium nitrite meets ASTM C 494 specifications as a concrete accelerator.¹²

Calcium nitrite is not detrimental to concrete's properties, unlike inhibitors based on sodium or potassium. Later studies showed that calcium nitrite is compatible with silica fume.^{13,14}

In the 1970s, numerous corrosion studies were performed documenting the corrosion inhibiting properties of calcium nitrite.^{12,15,16,17} It was determined that the mechanism of corrosion protection was that of anodic inhibition, and that steel in chloride containing concrete fails by pitting.

The first large scale results reported by a government agency were released in 1983 by the Federal Highway Administration (FHWA).¹⁸ Even though FHWA used a water-cement ratio that was high (>0.5), and added chlorides to the mix water, they concluded that "... calcium nitrite can provide more than an order of magnitude reduction in the corrosion rate." Their final conclusions were that calcium nitrite was effective at chloride to nitrite ratios below 1.0 based on admixed chlorides, and that ratios "... higher than 1 might be tolerable ..." for chlo-

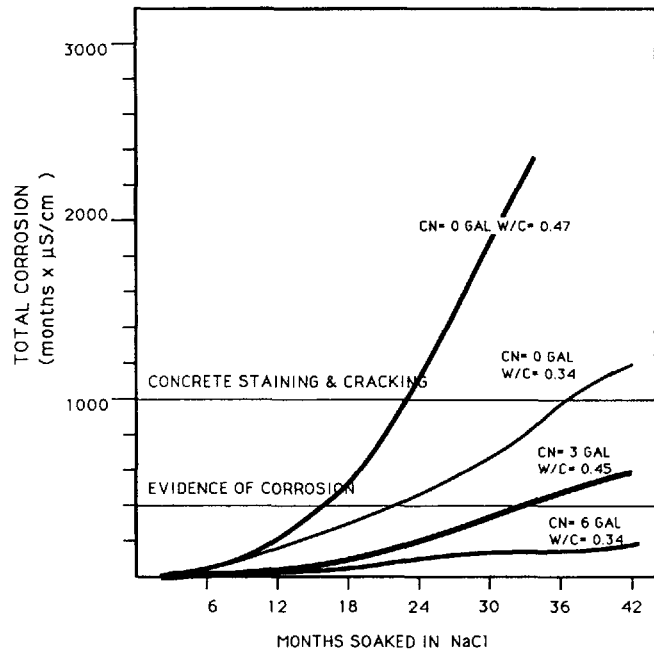


Fig. 1 — Calcium nitrite corrosion inhibitor total corrosion versus time in 3 percent NaCl.

ride ingress into initially chloride free concrete.¹⁹ A recent review on calcium nitrite showed that for non-admixed chlorides protection ratios are above 1.0 for chloride levels up to 9.5 kg/m³ (16 lb/yd³).²⁰ Solution experiments by Hope and Ip show that the ratios of chloride to nitrite can be higher than 11 to one.¹¹

The South Dakota Department of Transportation conducted a study in which reinforcing bars were embedded in concrete cylinders with and without admixed chlorides.²¹ Cylinders with admixed chlorides went into corrosion immediately (1 day) unless calcium nitrite was present. Samples with calcium nitrite remained passive for 700 days. Calcium nitrite also improved the corrosion resistance of samples without admixed chlorides. Chloride analyses at the reinforcing bar showed that calcium nitrite prevented corrosion at chloride-nitrite ratios of 1.6 to 2.2 (chloride contents exceeded 10.7 kg/m³ [18 lb/yd³]).²⁰

Figure 1 shows an updated curve of corrosion versus time first presented at the Corrosion-87 Symposium.²² After four years of accelerated corrosion with chloride levels at the steel exceeding 7 kg/m³ (12 lb/yd³) only specimens with calcium nitrite remained uncorroded.²³ It is clear that under long-term exposure,

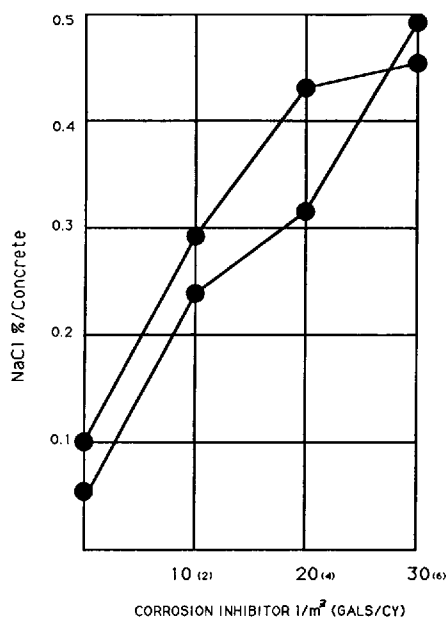


Fig. 2 — Effect of water-cement ratio, inhibitor content, and salt content on corrosion of steel in concrete.

ated testing, calcium nitrite is an effective inhibitor, and that the better the quality of the concrete the better the corrosion inhibition. This is further documented in another large scale study completed in 1987, in which the corrosion rates of mini-slabs were investigated.¹³

Additional data supporting the effectiveness of calcium nitrite corrosion inhibition has recently come from Japan.^{24,25} In accelerated tests with wetting and drying cycles at 80 C, calcium nitrite was found to be an effective inhibitor.²⁴ Table 2 summarizes the results showing cycles to corrosion as a function of chloride and inhibitor contents (33 percent solids by mass of solution).

In spite of the high water-cement ratios, calcium nitrite still proved effective (Table 2). Another wet/dry cycling test at 50 C demonstrated the advantages of using a lower water-cement ratio of 0.5 versus 0.6 as shown in Figure 2.²⁴

Nishibayashi et al²⁵ document long-term performance of "rust inhibitor," identified as calcium nitrite in their presentation, as a corrosion inhibitor in field marine exposures. They showed that, at the 20 l/m³ (4 gal/yd³) addition level of a 30 percent solution, calcium nitrite reduces corrosion.

Increasing the use of pozzolans to prevent corrosion by reducing permeability to chlorides is taking

place. Microsilica is one of the most effective pozzolans because its small particle size enables it to act as a filler and increases its chemical reactivity. Calcium nitrite is compatible with microsilica¹³ and should provide protection in the presence of chloride. Studies have shown that it also improves the corrosion performance of fly ash concrete²⁰ and concrete with slag.²⁵

In addition to protecting steel in concrete, recent testing shows that calcium nitrite also protects galvanized steel and aluminum.²⁶ This study alternately ponded mini-beams with 3 percent NaCl solutions, and measured corrosion by the macrocell and polarization resistance methods.

Calcium nitrite has been shown to provide benefits in cracked concrete.^{20,27} Figure 3 shows the long-term total corrosion (measured by polarization resistance and integrated) for reinforced notched mini-beams with cracks produced by third point loading. The cracks extended down to the reinforcing bar, half-way to the bar, and not at all. Cracked samples with calcium nitrite took longer to go into corrosion or were corroding at substantially reduced rates. The cracks were less than 0.01 in. wide.²⁰ More recently a study of unnotched beams cracked in flexure showed that calcium nitrite provided enhanced performance to chloride induced corrosion.²⁷

A recent German study²⁸ showed that sodium nitrite was also effective in reducing corrosion in cracked specimens, even at much higher than recommended water-cement ratios and low dosage rates of inhibitor. Other investigators have shown in general that thin cracking was not a significant factor in corrosion.^{29,30}

Extensive research shows that calcium nitrite will perform well in cracked concretes. This improved performance is due to several factors. One is that, in concrete produced to proper ACI guidelines and codes, crack sizes at the surface will be less than 0.2 mm (0.008 inch) and covers will be at least 38 mm (1.5 in.). Under these conditions the crack size at the reinforcing bar will be quite small and the solution will be buffered by the concrete. Furthermore, recent studies as part of the Strategic Highway Research

Program showed that nitrite modifies the oxide film on the reinforcing bar to be more protective than the film that naturally occurs in concrete.³¹ They also demonstrated that this improved oxide leads to enhanced corrosion resistance.³²

Studies have shown that calcium nitrite might decrease the resistivity of concrete.^{14,22} However, long-term corrosion data show that in spite of the decrease in resistivity, corrosion rates were significantly reduced.^{14,22,23} Likewise, increases in AASHTO T277 chloride permeability values with calcium nitrite were clearly shown to have no negative effects on actual chloride ingress into the concretes.^{13,14}

Proper design

It is important that the civil engineer do more than just add an inhibitor to the concrete. ACI 201 states that low water-cement ratios and good concrete cover should be employed when chloride exposures are moderate to severe. Indeed, several of the studies emphasized the need for low water-cement ratios and adequate cover.^{13,22,24} Furthermore, it is well documented that calcium nitrite becomes more effective as concrete quality improves.^{13,14,20,22,23} Since higher quality concretes outperform those of lesser quality, improvements with calcium nitrite are substantial.

If the concrete is to be subjected to freezing and thawing cycles then it must be properly air entrained according to ACI 201. Concretes with calcium nitrite and superplasticizers can be properly air entrained and pass freeze-thaw testing.¹⁴

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Table 2 — Number of wet-dry cycles until cracking occurs

NaCl Level (% concrete)	Corrosion inhibitor (1/m ³)			
	0	11.8	23.5	35.3
0	*			
0.1	*	*		
0.3	10	15	*	
0.5	10	10	15	*

w/c ratio = 0.6; *Did not crack. Corrosion inhibitor was 30% calcium nitrite solution.

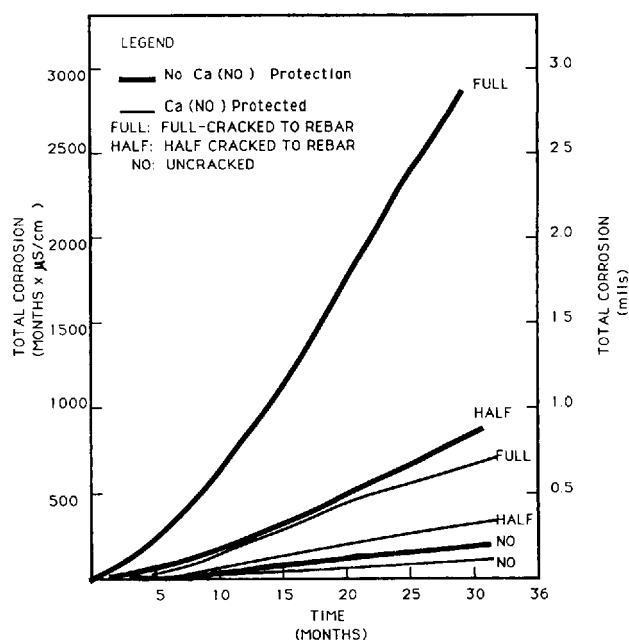


Fig. 3 — Total corrosion versus time.

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Selected for reader interest by the editors.

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Documents

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Evaluation of Bridge Deck Protective Strategies

by Khossrow Babaei and Neil M. Hawkins

This article documents existing knowledge on the effectiveness of different common strategies for protecting bridge decks against deterioration caused by de-icing salts. The strategies considered are increased cover to the top bar; low-slump, dense concrete overlay; latex-modified concrete overlay; interlayer membrane/asphalt concrete system; and epoxy-coated bars. Performance information was expressed in terms of the deck durability, effectiveness in preventing reinforcing bar corrosion, and concrete cracking. The information was analyzed for factors affecting the likely service life of a protected bridge, for costs to achieve a 50-year deck life, and for the relative effectiveness of a certain strategy.

Premature deterioration of concrete bridge decks is of major concern to the nation's highway agencies. Deterioration significantly worsens the ride quality of decks. Subsequent repairs and rehabilitation inconvenience the travelling public, and cause problems for the highway agency. Deck deterioration, depending on its magnitude and location, can also reduce a bridge's load carrying capacity.

Corrosion of embedded reinforcing steel, resulting from winter salt application and moisture, is the primary cause of bridge deck deterioration. The expansion that accompanies corrosion causes internal rupturing of the concrete and leads to spalls on the concrete's surface. Corrosion also creates losses in steel area and loss of bond between the steel and the concrete. Bridge deck deterioration has become a major concern because design practices of the 1960s and early 1970s seldom

considered protection of the deck against salt infiltration. Also, the use of salt to achieve bare road conditions has increased as the nation's highway network has increased.

Bridge decks are exposed to large changes in temperature and moisture, and are subjected to direct impact and repeated loading by traffic. These conditions, combined with salt applications, create a severe environment for the concrete. In addition to corrosion of the bars, the harsh environment can cause premature deterioration of the concrete surface.

Over the past two decades, many protective strategies have been developed to combat deterioration. Preventing bar corrosion has generally determined the protective strategies for new deck construction, such as epoxy coating and galvanizing bars; overlaying decks with low permeability concretes containing latex particles or waxbeads, or reducing water/cement ratios and increasing densities; overlaying or impregnating decks with polymers; increasing the bar cover thickness; waterproofing with interlayer membranes having asphalt concrete overlays; using sealers with or without overlays, and using cathodic protection.

Presently, commonly used systems for new deck construction are epoxy-coated bars in the deck, low-slump dense concrete overlays, interlayer waterproofing membranes with asphalt concrete overlays, and increased bar cover thickness (3 in. [76 mm] or more).

Research

This research was part of the National Cooperative Highway Research Program¹ to provide definitive information about the performance and likely service life of decks for which protection was part of the original construction. Generally these decks are performing better than unprotected decks, which in some cases have required rehabilitation after only a few years of service. Highway agencies want decks with at least 50 years of corrosion-free service life.

Overall performance information was collected from state highway departments in forty-six states for the five most commonly used protective strategies. Next, quantitative information on protected bridges with five to fifteen years of service was developed through a comprehensive literature review and analysis. In addition, personnel of several state transportation departments were interviewed and selected bridges in their jurisdictions were examined qualitatively.

The research defined performance as a function of:

- The durability of the bridge decks and their protective components, independent of evidence of bar corrosion.
- The bar corrosion prevention effectiveness of the given strategy.
- The degree of concrete cracking in the decks.

Keywords: bridge decks; cover; deicers; deterioration; epoxy resins; performance; protection; protective coatings; reinforced concrete.

Table 1 — Estimated years to maximum acceptable corrosion of reinforcing steel in bridge decks with design target bar depth of 3.5 in. (89mm).¹

W/C maximum design	W/C ² adjusted for construction	Number of annual salt applications							
		5	10	15	20	30	50	100	150
0.50	0.53	99	49	33	25	16	10	5	3
0.49	0.52	108	54	36	27	18	11	5	4
0.48	0.51	117	59	39	29	20	12	6	4
0.47	0.50	128	64	43	32	21	13	6	4
0.46	0.49	151	75	50	38	25	15	8	5
0.45	0.48	177	89	59	44	30	18	9	6
0.44	0.47	208	104	69	52	35	21	10	7
0.43	0.46	247	123	82	62	41	25	12	8
0.42	0.45	292	146	97	73	49	29	15	10

¹The table is prepared for 620 lbs. of salt per application per lane-mile

²Considering a tolerance of 0.03

□ Years to corrosion ≥ 47 years

Based on performance information, recommendations were made for tentative guidelines for the design and construction of bridge decks that should have service lives of 50 years or more.

The durability of bridge decks

The past performance of bridge decks indicates that they will require major maintenance, in the form of resurfacing or overlaying, before their 50-year service life is completed. Even when there is no deterioration caused by corrosion, maintenance is needed as a result of distress caused by traffic action, exposure to changes in moisture and temperature, and exposure to salts and freeze-thaw cycles. Typical forms of distress are extensive wear in wheeltracks, lack of skid resistance, scaling of concrete surfaces, and stripping of overlays.

Bridge decks without overlays

Decks in this category use conventional structural concrete only and are protected by epoxy coating on their bars, or by at least 3 in. (76 mm) of concrete cover over uncoated bars. Research revealed that only one highway department was dissatisfied with the scaling resistance of air-entrained concrete, but scaling in air-entrained concrete has been documented in installations and environments nationwide.^{2,3,4,5}

Also, bridge deck scaling was observed in localized deck areas representative of certain concrete batches or certain environments, such as gutter areas. Regardless of

the number of freeze-thaw cycles, scaling was observed in cold environments with high rates of snow-fall.

Traffic polishes the concrete's fine aggregate and causes a loss of skid resistance. It also wears away the mortar^{2,3,4} and eliminates transverse traction grooves built into the surface. Traffic wear can result in surface rutting. Rut depths of 0.5 in. (13 mm) or more can be expected in 15 to 20 years in urban areas where studded tires are permitted.

Bridge decks with concrete overlays

Research showed that two highway departments were dissatisfied with the scaling resistance of low-slump dense concrete, and two departments were dissatisfied with the scaling resistance of latex-modified concrete. However, the literature review showed that for latex-modified concrete, scaling has been considerably less than for low-slump dense concrete, where scaling of four percent of the deck area has been found after six years of service.⁶ In the same environment, scaling of latex-modified concrete has been limited to one percent after nine years of service.

Conflicting evidence exists on the skid resistance of these special concretes. Higher skid numbers have been obtained on latex-modified concrete than on low-slump dense concrete.⁷ An average skid number of 48, corresponding to two to six years of service, was obtained for latex-modified concrete, while an average number of 39, correspond-

ing to two to three years of service, was obtained for low-slump dense concrete. However, other research has shown the same skid resistance for both concretes.⁸ Wear for both concretes under traffic has been similar to that for conventional concrete. Traction grooves, initially $\frac{3}{16}$ in. (5-mm) deep, have totally disappeared in about seven years under a 10,000 average daily traffic volume and with the use of studded tires. One transportation department was dissatisfied with the wear of low-slump dense concrete and two were dissatisfied with the wear of latex-modified concrete.

Most concrete overlays are sufficiently integrated with the underlying decks. Debonded areas, if any, have been less than one percent of the decks' areas after up to seven years of service.^{6,9} In some cases, however, large areas of the overlay have debonded with little correlation to the age of the overlay. For example, one low-slump dense concrete overlay debonded over 35 percent of the deck after only about five years of service.⁹ Debonding has been attributed to inadequate texturing of the substrate and methods of applying bonding agents that result in no bond or insufficient bond when construction is complete.

For two-course new construction decks, one department was dissatisfied with the bonding of low-slump dense concretes, and four departments with the bonding of latex-modified concrete. For overlays applied on existing decks, six departments were dissatisfied with the bonding performance of low-slump dense concrete and eight were dissatisfied with latex-modified concrete. The higher numbers for the existing decks could be attributed to overlays being more common for existing decks than for new decks nationwide.

Bridge decks with asphalt concrete/membrane systems

Debonding and stripping of asphalt concrete overlays has been a major problem for some highway departments. Depending on the severity of the traffic and the environment, some systems have required removal and replacement in ten years or less. When water accumulates above the membrane, it weakens the bottom portion of the asphalt concrete^{6,10} and causes problems. Eight transportation departments reported stripping of asphalt concrete from two-course new construction decks.

For asphalt concrete subjected to average daily traffic volumes exceeding 10,000, skid numbers below 35 may be expected in about 15 years. Excessive wear of the surface in wheeltracks, especially in the presence of studded tires, combined with consolidation or lateral movement of the pavement, causes rutting in the asphalt concrete. Nine transportation departments claimed dissatisfaction with the wear of asphalt concrete overlays.

Corrosion prevention characteristics

The prime factors affecting the performance of concrete bridge decks are reinforcing bar corrosion and subsequent corrosion-induced concrete deterioration as a result of the application of de-icing salts. If bar corrosion begins, rehabilitation of the structure is very difficult.

In the following section, the ability of the five different protective strategies to prevent bar corrosion in bridge decks is reviewed with the goal of providing at least 50 years of corrosion-free bridge deck service. This review, however, does not consider the effects of cracks in the concrete on bar corrosion since those effects are virtually undocumented.

Bridge decks with 3.5 in. (89 mm) bar covers

The "effective service period" (i.e., the number of years until deterioration affects 5 percent of the

deck area) may be 50 years or more when salt exposure is less than 5 tons per lane-mile (2.8 Mg per lane-km) per year. For higher salt applications, the water/cement ratio for the concrete determines its chloride-proofing abilities and the service life. This study showed that construction procedures impose a tolerance of 0.03 on the water/cement ratio and thus, the limiting ratio for permeability considerations should be the specified value plus 0.03. For salt exposures greater than 10 tons per lane-mile (5.6 Mg per lane-km) per year, specified maximum water/cement ratios must be 0.42 or less if the effective service life is to

The prime factors affecting the performance of concrete bridge decks are reinforcing bar corrosion and subsequent corrosion-induced concrete deterioration as a result of the application of de-icing salts.

be 50 years. When salt applications reach 30 to 45 tons per lane-mile (16.9 to 25.4 Mg per lane-km) per year, the effective service period even for a specified water/cement ratio of 0.42 may be ten to 15 years only.

Bridge decks with concrete overlays

Three transportation departments were dissatisfied with the chloride proofing abilities of low-slump dense concrete overlays and six were dissatisfied with the chloride proofing abilities of latex-modified concrete overlays. That dissatisfaction is a result of the permeability of the overlays. For low-slump dense concrete overlays the

specified water/cement ratio is 0.32. However, the chloride permeability of field installations can be similar to that of a concrete with an actual water/cement ratio of 0.40 to 0.45. The permeability is higher because the stiffer, low-slump mix causes construction difficulties. The chloride permeability of field installations of latex-modified concrete is about the same as the permeability of latex-modified concrete made in the laboratory. A 1.5-inch (38 mm)-thick, latex modified concrete provides the same chloride permeability as a 2-inch (51-mm)-thick, low-slump dense concrete with an effective water/cement ratio of 0.40.

The "effective service period" of a protective system consisting of 1.5 in. (38 mm) of latex-modified concrete, or 2 in. (51 mm) of low-slump dense concrete representing a water/cement ratio of 0.40, placed on a deck with a specified water/cement ratio of 0.45 and a bar cover depth of 1.5 in. (38 mm), is about 50 years only if annual salt applications do not exceed 12 tons per lane-mile (6.8 Mg per lane-km).

Bridge decks with asphalt concrete/membrane systems

The field performance of preformed and thermoplastic applied-in-place membranes^{6,10,11,12} indicates that such membranes are effective in preventing chloride intrusion. A statistical analysis of the ten-year performance of membranes¹² has shown that the rate of increase in chloride contamination at a depth of 1 to 2 in. (25 to 51 mm) into the concrete, the typical location for the deck's reinforcing bars, is less than 0.004 lb/yd³ (0.002 kg/m³) per year. For that increase it takes more than 50 years before the concrete surrounding the bar is contaminated to the corrosion threshold level of 1.50 lb/yd³ (0.89 kg/m³). However, membrane deterioration can be expected after only 15 years of service due to the repeated stresses caused by traffic, and membranes can also suffer from age embrittlement. Three states were not satisfied with the chloride proofing abilities of interlayer membranes.

Table 2 — Estimated years to maximum acceptable corrosion of reinforcing steel in bridge decks overlaid with low-slump dense and latex-modified concrete.¹

Case	Number of annual salt applications					
	30	40	50	70	100	150
I: Overlay: 2 in. (51mm) LSDC or 1.5 in. (38mm) LMC Bare deck: 1.5 in. (38mm) Target Cover & W/C = 0.45	72	54	43	31	22	14
II: Overlay: 2.5 in. (64mm) LSDC or 1.75 in. (44mm) LMC Bare deck: 2.0 in. (51mm) Target Cover & W/C = 0.45	170	128	102	73	51	34

¹This table is prepared for 620 lbs. of salt per application per lane-mile

□ Years to corrosion ≥ 47 years

LSDC Low-Slump Dense Concrete

LMC Latex-Modified Concrete

Bridge decks with epoxy-coated bars

Field measurements of the electrical resistance in bridge decks between epoxy-coated top mats and uncoated bottom mats have shown that electrical contact can exist between the two mats.¹³ As a consequence, a coated top mat in chloride-contaminated concrete can corrode at local coating breaks or pinholes in the original coating at an accelerated rate when coupled with the uncoated bottom mat in chloride-free concrete. Such breaks can occur during transportation of the bars, construction, or service.

It has been estimated from laboratory research that at the same chloride exposure, corrosion would take 12 years to consume an amount of top mat epoxy-coated steel equal to the amount of top mat uncoated steel consumed in one year.¹³ The amount of metal consumed in the top mat can be reduced 46 times by epoxy-coating both the top and bottom steel mats.¹³ Epoxy-coating the top steel, combined with limits of 0.45 on the water/cement ratio and 2½ in. (64 mm) of cover over that steel, should provide 50 years of corrosion-free life even in severe chloride environments. This life can be further assured by epoxy-coating both the top and bottom steel.

Pull-out bond tests have indicated that the bond strength for certain epoxy coatings, with film thicknesses 10 mils (10.24 mm) or

less, is about 6 percent less than the bond strength of uncoated bars.¹⁴ That reduction is within the range of variability normal in bond tests. Flexural bond tests have shown that for bar development lengths greater than 12 in. (30.5 cm), the bond strength for an uncoated bar is 15 percent greater than the bond strength for an epoxy-coated bar.¹⁵ Flexural fatigue bond tests in the working stress range have shown that slip behaviors for uncoated and coated bars are similar.¹⁵

Cracking in concrete bridge decks

Concrete cracking affects both the durability and the corrosion prevention characteristics of bridge deck protective strategies by allowing moisture and salt to penetrate into the concrete. Cracking in both bare and concrete overlaid decks can be extensive enough to accelerate chloride contamination.

Transverse cracking is the most frequent cracking in bare decks.^{2,16,17,18,19} As much as 100 ft (30.5 m) of transverse cracking per 1,000 ft² (92.9 m²) of deck area has been observed in decks with bar covers deeper than 3 in. (76 mm).¹⁷ Transverse cracking in bare decks generally occurs along the uppermost bars and extends down. Flexing at cracks can wear away the epoxy coating on bars. The type of cracking found in two-course new construction bridge decks with low-slump dense or latex-modified con-

Table 3 — Comparison of anticipated life of bridge decks built with damaged epoxy coating with those built with undamaged epoxy coating (adapted from ref. 31)

	Type of coating				
	Top mat only			Both mats	
Degree of damage (percent of area)	0%	0.24%	0.86%	0%	0.86%
Ratio of anticipated life to life of undamaged coating	1.00	1.00	0.29	1.00	0.76

¹Based on years for corrosion to consume equal amount of steel

crete is generally a combination of random, transverse, and longitudinal cracking. The amount of cracking in concrete overlays can exceed 100 (30.5) lineal ft (m) per 1,000 ft² (92.9 m²) of deck area.^{6,9} Thus, sealing of the cracks can be a preventive maintenance.

Many factors can cause cracking during a deck's construction and service life. These important factors include:

Plastic shrinkage cracks — This type of cracking generally has a random pattern and appears more in low-slump dense and latex-modified concretes than in conventional concrete. Often the former mixes do not have sufficient bleed water for evaporation purposes and consequently shrinkage cracks form.

Flexural cracks in plastic concrete — In unshored construction, transverse cracks may form in a deck's plastic concrete over and near the supports of the continuous spans due to the deck's dead weight causing negative moments in the girder system.

Settlement cracks in plastic concrete — This type of cracking usually forms over and parallel to the uppermost bars after the concrete is finished and settled. These cracks are usually transverse, since the uppermost bar in most decks is transverse.

Drying shrinkage cracks — These cracks form in the concrete after it has been cured and as it loses moisture to the environment, and are usually random and transverse.

Table 4 — Probability of subsidence cracking predicted by regression analysis.³³

		Probability of Cracking ¹ (percent)								
Slump (in.)		2			3			4		
Bar Size		#4	#5	#6	#4	#5	#6	#4	#5	#6
Cover (in.)										
¾		80.4	87.8	92.5	91.9	98.7	100.0	100.0	100.0	100.0
1		60.0	71.0	78.1	73.0	83.4	89.9	85.2	94.7	100.0
1½		18.6	34.5	45.6	31.1	47.7	58.9	44.2	61.1	72.0
2		0.0	1.8	14.1	4.9	12.7	26.3	5.1	24.7	39.0

¹Computed probability values of less than 0 percent or greater than 100 percent are reported as 0 percent or 100 percent, respectively.

Flexural cracking under service conditions — Concrete bridge decks crack transversely under dead and/or repeated live loading, over and near the interior supports on continuous spans, due to the longitudinal flexibility of the superstructure. Concrete decks may also crack longitudinally due to differential girder deflections caused by transverse flexibility. Longitudinal cracking, however, does not occur frequently in bridge decks. Steel girder bridges have higher potential transverse crack regions than prestressed concrete girder bridges because they are more flexible. The effects of concrete overlays on decreasing the longitudinal flexibility of the superstructure is not significant.

Cracking caused by traffic-induced vibrations — The dynamic effects of traffic-induced vibrations can bring about cracking or can cause existing cracks in bridge decks to lengthen or deepen in the central portions of spans. The amplitude and frequency of the vibrations determine the risk of cracking due to dynamic effects.²⁰ More flexible structures may result not only in larger deflections but also in vibrations with larger amplitudes at essentially the same frequency. Thus, the net effect may be an increased risk of cracking from traffic-induced vibrations. Repeated vibrations over time can weaken concrete and cause cracks, especially at vertical planes of weakness such as those formed in plastic concrete and located directly above and parallel to the uppermost transverse reinforcing steel. Cracking due to vibrations over a long period of time will occur in older bridges, bridges with higher traffic volumes, higher traffic speeds, and longer span lengths.

Guidelines

In this section, procedures are suggested for providing more dura-

ble bridge decks through the minimization of concrete freeze-thaw scaling, surface skid and wear, and overlay debonding and stripping.

Bridge decks without overlays

Research⁵ has suggested that acceptable freeze-thaw resistance can be expected from bridge deck concretes if they meet certain requirements for air void spacing, water/cement ratio, and water absorption. The air void spacing factor of the hardened concrete should be less than 0.008 in. (0.20 mm). Fresh concrete with air contents less than 4.5 percent cannot ensure such air void spacing.⁵ The calculated value for the water/cement ratio during construction should be limited to 0.45 and water reducers should be used in the mix because they indirectly ensure a low water/cement ratio, regardless of the calculated value. Finally, the concrete's water absorption should be limited to 4.5 percent, as this reflects the combined effects of the degree of consolidation achieved during construction and the water/cement ratio.⁵

Elements that adversely influence the freeze-thaw durability of concrete include a surface that has been sprinkled or overworked during construction (such actions alter the surface's air void system and decrease its strength); an inadequate surface drainage system that causes ponding and ultimately saturation of the air-entrained concrete, especially for decks where de-icing salts are used; delays between concrete placement and curing;⁵ an insufficient period of air drying before the first application of de-icing salts;²¹

and treatment of the concrete surface with a sealant, whose effectiveness has not been demonstrated and thus may accelerate deterioration.⁵

For adequate skid resistance, a concrete surface needs a fine texture that provides close contact with a tire. A higher proportion of fine aggregate in the mix will provide better skid resistance.²² Some fine aggregates polish under traffic action to a degree that makes the surface slippery. One laboratory test that evaluates the polish susceptibility of aggregates is the acid insoluble residue test (ASTM D3042).

To provide coarse texture and prevent hydroplaning on wet surfaces, the concrete deck should be tined or broomed when plastic. Tining generally provides ⅜-in. (5-mm) deep transverse grooves in the concrete. Alternatively, a coarse texture can be built into the hardened concrete by saw cutting. When the grooves wear away under traffic, the coarse aggregate in the mix provides the pavement's skid resistance. Crushed aggregate provides better texture than uncrushed aggregate. Wear resistance can be increased by decreasing a concrete's water/cement ratio, increasing its cement factor, and employing proper finishing and curing procedures.²²

Bridge decks with concrete overlays

Low-slump dense concretes employ air contents of 6.5 ± 1.0 percent²¹ and water/cement ratios of about 0.32. However, up to ten times the normal dosage of air-entraining agent is sometimes needed to produce the desired air content.²¹

Thus, the effectiveness of the air-entraining agent for low-slump dense concrete mixes should be examined prior to its field use.²¹

Latex-modified concretes do not entrain air but do develop air voids with a typical upper limit of 6 percent, with a water/cement ratio of about 0.32. The air void spacing factor is generally greater than 0.008 in. (0.20 mm), but that factor is not significant for freeze-thaw durability^{23,24} since emulsion prevents water penetration and the void structure becomes unimportant.²³

The fine aggregate in the latex-modified mix provides the fine texture and the skid resistant qualities of the concrete overlay just as it does for conventional concrete. The skid resistance of latex-modified concrete is better than that of conventional concrete, since the former has a higher proportion of fine aggregate. The coarse texture of the overlay's surface is usually provided by traction grooves in the

The asphalt concrete must be well bonded to the membrane, but the amount of the bonding element, which may be the membrane itself, must be optimized to prevent horizontal slippage.

same manner as for conventional concrete. Low-slump dense concrete, due to its low water/cement ratio and high cement factor, has a greater potential to resist wear under traffic than the other types of concrete. However, the field performance of low-slump dense concrete does not necessarily support this potential. Overworking or sprinkling of the surface during the construction is likely because of the stiff nature of the mix, and that results in concrete with a lower surface strength.

Concrete overlays must have bond strength in excess of 100 psi (689,500 Pa) if debonding is to be

prevented. Repeated live loading, thermal cycling, and wetting and drying decrease the shear strength at the bonded interface. Overlays need a factor of safety on the initial shear bond strength of at least 5 to prevent debonding before the end of their service life. One laboratory method for determining shear strengths from core samples obtained in the field is Iowa Department of Transportation Test Method Number 406-June 1978.

The appropriate procedure for bonding concrete overlays for two-course new construction is generally to sand blast substrate that has been broomed to a rough texture during the first stage of construction, and to then apply a bonding agent on a dry substrate for low-slump dense concrete and on a wetted substrate for latex-modified concrete. If the deck is rained on after sand blasting but before overlaying, then blasting should be repeated when the deck is dry. It is also important that the bonding agent does not dry out when the overlay is applied.

If the second stage construction is delayed and the deck has been open to traffic even for a short time, scarifying to a depth of ¼ in. (6 mm) prior to sand blasting may be required. However, some agencies require scarification regardless of when the overlay is applied. To improve bond, some agencies scrub a mortar portion of the overlay mix onto the deck, while others require a grout specially made for that purpose. The latter procedure is a better construction practice since workers frequently put excess mortar material back into the overlay instead of wasting it.

Bridge decks with asphalt concrete/membrane systems

The primary cause of asphalt concrete overlays stripping off decks is the accumulation of water above the membrane in the bottom portion of the asphalt concrete. This phenomenon, combined with freezing and thawing and repeated hydraulic pressure from traffic, weakens the bottom layer of the asphalt concrete and the bond between the asphalt and the membrane. It can also strip the asphalt binder off the aggregate in that layer, resulting in no integrity among the mixture elements or

adhesion between them and the membrane. Another factor causing debonding and stripping is the formation of blisters under preformed sheet membranes.

The properties of asphalt concrete can be important for preventing stripping caused by water accumulation. High density and low air void content are desirable, but these properties must be balanced with the potential for loss of stability under traffic and high temperature. Special designs may be needed that

The primary cause of asphalt concrete overlays stripping off decks is the accumulation of water above the membrane in the bottom portion of the asphalt concrete.

include high quality aggregate, stiffer binders, and anti-stripping agents. The construction phase should employ effective compaction to satisfy design density. The asphalt concrete must be well bonded to the membrane, but the amount of the bonding element, which may be the membrane itself, must be optimized to prevent horizontal slippage. Weak seams should be avoided in asphalt concrete construction joints by using proper rolling patterns and maintaining hot longitudinal joints between paving lanes. Adequate surface drainage should be provided to prevent surface ponding. Drainage from the surface of membrane is possible by installing vertical pipes through the deck. Blistering under preformed membranes can be prevented by providing sufficient dead weight in the form of 2 to 3 in. (51 to 76 mm) of asphalt concrete overlay.²⁵ Blistering may also be prevented by applying a perforated sheet of bituminous felt or a 1-in. (25 mm) thick asphalt base course on the deck as a venting layer before applying the membrane.²⁵

The skid resistant qualities of the asphalt concrete depend on the na-

ture of the aggregate particles in the mix, which provides the fine texture, as well as the aggregate's gradation, which provides the coarse texture.²² A polish- and wear-resistant crushed aggregate and sufficient coarse aggregate for an adequate void structure ensure good skid resistance and allow surface water to drain, reducing the potential for hydroplaning. The wearing course needs a low void content, but that also results in a surface with too little skid resistance. Further, a dense mixture is more likely to deform and rut under traffic. Thus, the design of asphalt concrete for bridge decks requires a compromise among several factors.

Bridge decks with bars resistant to corrosion

Bridge decks with 3.5 in. (89 mm) of bar cover

Table 1 contains the likely num-

To achieve a 50-year or more effective service period, designs must consider the severity of salt application, the water/cement ratio and the cover thickness.

ber of years between construction and maximum acceptable corrosion (5 percent of the deck area) for decks designed with cover depths of 3.5 in. (89 mm). The table was developed in this research¹ and was based on FHWA laboratory work²⁶ and the evaluation of bridge deck performance. For the preparation of Table 1, it was assumed that 5 percent of the deck area would have a cover less than the design target depth (3.5 in. [89mm]) minus 0.65 in. (17 mm). The amount of time between corrosion and delamination can be assumed to be three years.²⁷

For Table 1, construction procedures were assumed to cause variations of 0.03 in the water/cement ratio.²⁸ To achieve a 50-year or more effective service period, designs must consider the severity of

salt application, the water/cement ratio and the cover thickness.

Bridge decks with concrete overlays

Table 2 contains the likely number of years between construction and maximum acceptable corrosion (5 percent of deck area) for decks designed with 2-in. (51-mm), low-slump dense or 1.5-in. (38-mm), latex-modified concrete overlays. The overlays are assumed to be placed on bare decks specified to have a 1.5-in. (36-mm) cover thickness and a water/cement ratio of 0.45. The table also gives the effective service life for 2.5-in. (64-mm), low-slump dense or 1.75-in. (45-mm), latex-modified concrete placed on decks specified to have a 2-in. (51-mm) cover thickness and a 0.45 water/cement ratio. The amount of time until delaminations appear can be determined by adding three years to the amount of time shown for corrosion.²⁷ Table 2 assumes that the chloride permeability of field installations of low-slump dense concrete is the same as the permeability of conventional concrete with an actual 0.40 water/cement ratio.²⁹ Some studies, however, suggest that insufficient consolidation and curing can cause that figure to be as high as 0.45.

To calculate the effective service period, it was assumed that the system comprised conventional concrete only with a water/cement ratio similar to that for the first stage construction, but with a cover thickness larger than that of the first stage construction, so that the same chloride permeability could be expected. Table 2 was then prepared employing the same procedure as that used for bare decks, so that similar tables can be prepared reflecting the effects of different variables such as overlay thickness, expected overlay field permeability, bar cover thickness, and the water/cement ratio for the first stage construction.

The effective service period of a concrete overlay is also a function of its durability, as discussed earlier, and is generally less than 50 years. At the end of that period the overlay must be replaced and/or resurfaced. Consideration should be given to the feasibility of removing the concrete overlay or the likelihood of accumulating dead loads, if

the initial overlay is not totally removed.

Bridge decks with asphalt concrete/membrane systems

Although the chloride-proofing abilities of some membrane systems seem to satisfy the 50-year service life criteria,¹² the actual life, which is generally ten to 15 years, depending on weathering and exposure to traffic, is governed by deterioration of the asphalt wearing course. Factors such as overlay stripping, wearing and rutting, or lack of skid resistance can require removal of the overlay and/or resurfacing. When the asphalt concrete is at least 2 in. (51 mm) thick and has deteriorated prematurely only on the surface, the upper portion of the asphalt concrete can be removed and replaced while keeping the membrane intact. However, in such cases consideration should be given to the membrane condition since

Factors such as overlay stripping, wearing and rutting, or lack of skid resistance can require removal and/or resurfacing of the asphalt concrete overlay.

membranes also age and deteriorate under traffic.

When the membrane is removed along with the asphalt concrete, conventional removal procedures, such as cold milling and scarifying, may result in irregularities in, and disturbances to, the concrete surface due to unexpected variations in the thickness of the asphalt concrete. New membranes cannot be applied to these irregularities. This problem may be solved by first applying a ½- to 1-in. (13 to 25 mm) thick leveling asphalt concrete course on the disturbed concrete deck followed by the membrane and the wearing course.

Bridge decks with epoxy-coated bars

FHWA's recommendations for maximizing the service life of an

Table 5 — Life-time cost of different bridge deck construction alternatives

	Alternative	Alternative description		Cost per sq. ft. of deck area (typical 1986 cost, \$)					Present value of 50 years life time cost ²	
				Lifetime cost						
				Initial cost	12 years	25 years	38 years	50 years		
Basic deck	Unprotected deck	I	7.5 in. (190mm) thick deck, ¹ 1.5 in. (38mm) cover	\$12.00	—	—	—	—	—	
Basic deck plus single protection	Cover thickness of 3.5 in. (89mm)	II	Alt. I plus 2 in. (51mm) additional cover	12.37		resurf. w/2 in. (51mm) conc.			\$13.93	
						5.28				
	Epoxy coated top mat	III	Alt. I plus 1 in. (25mm) additional cover plus epoxy-coated top mat	12.79		resurf. w/2 in. (51mm) conc.			14.35	
						5.28				
	Latex-modified or low-slump dense concrete overlay	IV	Alt. I plus 1.5 in. (38mm) latex or 2.0 in. (51mm) low-slump overlay	14.73		resurf. w/1.5 in. (38mm) latex or 2 in. (51mm) low-slump conc.			16.35	
						5.48				
Interlayer membrane	V	Alt. I plus 0.5 in. (13mm) additional cover plus 1.8 in. (46mm) AC/membrane	13.30	resurf. w/1.8 in. (46mm) AC/membrane	resurf. w/1.8 in. (46mm) AC/membrane	resurf. w/1.8 in. (46mm) AC/membrane			15.98	
				2.66	2.66	2.66				
Basic deck plus double protection	Epoxy-coated top and bottom mats	VI	Alt. III plus epoxy-coated bottom mat	13.39		Resurf. w/2 in. (51mm) concrete			14.95	
	Epoxy coated top mat & latex or low slump concrete overlay	VII	Alt. IV plus epoxy-coated top mat	15.33		resurf. w/1.5 in. (38mm) latex or 2 in. (51mm) low-slump conc.			16.95	
						5.48				
	Epoxy-coated top mat & interlayer membrane	VIII	Alt. V plus epoxy-coated top mat	13.90	resurf. w/1.8 in (46mm) AC/membrane	resurf. w/1.8 in. (46mm) AC/membrane	resurf. w/1.8 in. (46mm) AC/membrane			16.58
					2.66	2.66	2.66			

¹Alternative I (unprotected deck) is not considered as a possible strategy²Assuming 10% interest rate and 5% inflation rate.

epoxy-coated bar system are to leave no more than 0.25 percent of the damaged areas in the epoxy film unrepaired when the top reinforcing mat only is coated and no more than 2 percent unrepaired when both the top and bottom mats are coated.³⁰ These figures, which generally apply to each 1-ft (0.30m) length of bar, are based on the FHWA's rate of corrosion tests, in which comparisons were made, as shown in Table 3,³¹ of the anticipated life of bridge decks built with epoxy-coated bars with different degrees of damage.

In some bars the amount of damage existing before placing the concrete can exceed 0.25 percent of the surface area.¹³ Also, additional damage of about 0.50 percent in the form of wearing of the coating may

occur in transverse bars because of flexing at cracks.¹ Since damage in the coating may become a potential site for accelerated corrosion, bar pitting, and subsequent bar fatigue failure, the bottom mat should be epoxy-coated.

The long-term durability of epoxy coating in a chloride contaminated concrete remains a concern. Two major elements that may contribute to deterioration of the coating and corrosion of a bar are the presence of holidays (pinholes not detectable by the unaided eye) in the coating and weak adhesion of the coating to the bar. In this regard, the pinhole and adhesion requirements specified by AASHTO M284 or ASTM 3963 are especially important. These requirements limit the average number of pinholes in the coated

bar to 2 per linear foot (0.3m) and necessitate evaluating the adhesion of the coating by bending coated bars around a mandrel. The adhesion of touch-up epoxy coating placed over damaged areas is also a concern. There are indications that epoxy does not bond well to itself.

Based on the results of flexural and fatigue tests of bond between epoxy-coated bars and concrete, a basic development length modification factor of 1.15 should be used for epoxy-coated bars in order to provide comparable performance with uncoated bars.¹⁵ The pull-out bond strength for coated bars should be determined using concrete prisms and taken as equal to the smaller of the stresses corresponding to a free end slip of 0.002 in. (0.051 mm) or a loaded end slip