

Concrete Bridges in Aggressive Environments

Philip D. Cady International Symposium



Richard E. Weyers
Editor

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PREFACE

The Philip D. Cady International Symposium was held in Minneapolis, Minnesota, on November 9 and 10, 1993. The symposium was held to honor a concrete educator/researcher. During his over 25 years at the Pennsylvania State University he has guided 76 graduate students through to completion of their educational and concrete research programs. He has introduced concrete technology to over 2500 undergraduate students. Philip Cady is best known for his research on the durability of concrete bridges in corrosive environments. His work has included the identification of the causes of early deterioration of concrete bridge decks, development of the polymer impregnation process for stopping the corrosion of steel in concrete bridge decks, service life prediction of concrete structures in chloride-laden environments, cost-effectiveness of the protection, repair, and rehabilitation of concrete structures in corrosive environments, and the development of condition assessment methods for concrete structures.

The symposium volume includes 15 papers on concrete bridges in aggressive environments. The papers address the performance, protection, assessment, and the repair and rehabilitation of concrete bridges. The performance papers include the corrosion protection afforded by concrete bridge deck overlays, corrosion in prestressed concrete bridges, and the use of calcium nitrite in field structures. Protection papers address the performance of silane sealers, coatings, and waterproofer. Condition assessment technologies include measuring the corrosion rate of steel in concrete and the diffusion of chloride ions in bridge decks with overlays. Experiences in the repair and rehabilitation of concrete bridges by practitioners is also presented.

All the papers presented in this publication were reviewed by recognized experts in accordance with the ACI review procedures. It is hoped that designers, constructors, and specifiers will be able to use the material presented to prolong the service of concrete structures in corrosive environments.

The Philip D. Cady Symposium was sponsored by ACI Committees 345, 222, 515, and 201. The symposium chairman wishes to thank those Committee Chairmen and the symposium Session Chairmen, David Whiting, David Manning, Kenneth C. Clear, Joseph Lamond, and Stella Marusin.

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**Field Performance of Latex-Modified
and Low-Slump Dense Concrete
Bridge Deck Overlays in
the United States**

by W.P. Chamberlin and R.E. Weyers

Synopsis: The service life of latex-modified concrete and low-slump dense concrete bridge deck overlays is estimated by extrapolating historical performance data obtained from the results of field research and investigations conducted in the United States and Canada. The data suggest that when concrete removal criteria are based on half-cell potential rather than actual damage, when removal of chloride contaminated concrete is extended to below the rebar, and when the substrate is sandblasted to remove microcracking prior to cleaning, a mean service life of 30 to 50 years is likely.

Keywords: Bridge decks; concretes; latex; microcracking; resurfacing; service life; slump

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Richard E. Weyers, P.E. is a Professor of Civil Engineering and Director of the Center for Infrastructure Assessment and Management at Virginia Tech, Blacksburg, VA, 24061-0105. He was awarded his Ph.D. degree from Penn State in 1983 and a Fellow of ACI. He has over 80 publications on the durability of concrete and is a member of ACI Committees 222, 345, and 548, and Secretary of Committee 365 Service Prediction.

INTRODUCTION

Among those treatments in the mainstream of current practice for rehabilitating concrete bridge decks subject to corrosion damage, only latex-modified concrete (LMC) and low-slump dense concrete (LSDC) overlays have been used frequently enough and long enough that reliable experienced-based estimates of service life can be made (1). Such estimates are important because they are the basis for life cycle cost analyses. Also, once a condition decay curve has been established, it can provide a standard against which to judge the potential service life of new alternative treatments during their early years of service.

The first LMC bridge deck overlay was placed in Michigan in 1957 (2); and the first LSDC overlays, in Iowa (3) and Kansas (4) in the early 1960s. Their intent was to restore the riding surface of the deck, to guarantee a specified minimum protective cover of chloride-free concrete, and to reduce the rate of future chloride ingress by virtue of their low permeability. It has been speculated that these overlays may also reduce corrosion rate by restricting the movement of oxygen and moisture to the surface of the reinforcement (5). Use of these systems was given impetus in the 1970s with publication of early results of the Federal Highway Administration's time-to-corrosion study (6,7), and by 1977 21 states reported one or both treatments to be a standard practice for deck rehabilitation (8). By 1989, that number had increased to 37 states (9).

In general, these high performance concrete overlays have been viewed as a cost effective way to repair and extend the life of a damaged deck and to retard the development of future corrosion damage, but not as a way to stop corrosion (8). Opinions regarding their service life vary widely, but average about 16 years (1). Work recently completed under Strategic Highway Research Project C-103, "Concrete Bridge Protection and Rehabilitation: Chemical and Physical Techniques", included estimating the service life of these treatments from historical data. Those estimates were made by extrapolating condition data reported by highway agencies in the United States and Canada, representing overlay evaluations made through 1991 (1, 10). The purpose of this paper is to describe those estimates and the basis upon which they were made.

THE DATA BASE

The data base was developed from a variety of sources, including: the open literature, questionnaires mailed to highway agencies, FHWA experimental features and demonstration project reports, and personal contacts. All primary leads were followed up by telephone to clarify and supplement the preliminary data and to develop secondary leads. For the purpose of the study, overlay performance was taken to be the change in condition of the overlay with time as measured by the cumulative percent of deck area damaged by delamination or spalling. No attempt was made to distinguish delaminations associated with bond failures from those associated with rebar corrosion and, unless otherwise indicated, patching was assumed to represent repaired spalls. Cracking, per se, was not included as a measure of condition.

Only data meeting the following criteria were included in the data base:

- 1) The condition evaluation must be quantitative, in the terms stated above;
- 2) The age of the evaluated deck in years (winters of service) at the time of the evaluation(s) must be known;
- 3) The deck surface area evaluated must be known or the damage must be expressed as a proportion of the area evaluated;
- 4) The geographic location of the deck must be known so that climatological data could be obtained; and
- 5) The AADT must be available for the portion of the

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deck evaluated.

Very little of the potentially useful data that was identified failed to meet the last four criteria.

Even though LMC and LSDC overlays have become widely accepted as bridge deck rehabilitation treatments and have been used extensively in the United States and elsewhere for 15-20 years, quantitative performance data were found to be scarce. Much of what was found existed in formal agency reports published to meet requirements of the HP&R or Experimental Features programs of FHWA, which are generally available; and in informal agency reports intended primarily for internal use, which are not generally available. Much of the data had not been published in any form and was provided raw from agency files. Only a small amount was in the open literature.

The data search identified 34 different field studies or investigations. Some of the studies (three) included as few as a single overlay application; one included as many as 50. Over one-half of them (twenty) included 5 applications or less. These studies were conducted by 16 different highway agencies and generated a total of 727 independent condition evaluations (286 LMC, 441 LSDC) of 305 individual overlay applications (152 LMC, 153 LSDC). The nature of the studies varied greatly. Many, apparently intended only to investigate first cost, constructability and early life performance, included overlays evaluated only once or twice (Figure 1), typically in the first few years after placement.

Others, clearly conceived to track longer-term performance as well, have been evaluated more frequently (Figure 1) and for longer periods of time (Figure 2). However, the available condition data for overlays diminishes after 10 years (Figure 3) and for overlays in service for more than 15 years is scant (Figure 3). The raw data from which the above summaries have been compiled, as well as the tables and figures that follow, is stored in the SHRP Project C-103 files.

SERVICE LIFE ESTIMATES

The historical data for LMC and LSDC overlays is summarized in Tables 1 and 2 in which overlay condition is represented by the mean percent of deck area damaged for all of the overlays evaluated at each of the

treatment ages indicated. Scattergrams of the mean damage values in Table 1, through the 15th year, are plotted for LSDC overlays in Figure 4a, for LMC overlays in Figure 4b, and for all of the overlays combined in Figure 4c. Because of the paucity of data for ages beyond 15 years, mean damage values for those years are not included in the scattergrams.

Even a casual comparison of Figures 4a and 4b suggests that differences in overlay type (LMC or LSDC) have not had a significant influence on the performance of those overlays that have been tracked, at least for the first 15 years of their life. However, examination of the differences in performance among overlays in individual study groups did reveal differences that are of considerable significance. These study-to-study differences were unrelated to the generic type of overlay (LMC or LSDC) or to any of the design, environmental or traffic factors that had been recorded for each of the treatment applications (overlay thickness, mean annual snowfall, snow storm frequencies, mean annual temperature and AADT). Typical of these differences is the example shown in Figure 5, which illustrates the performance of two groups of LSDC overlays constructed at different times by the same agency, but under different sets of conditions. Between the time that the first group was applied in the 1965-1972 period and the second group was applied in the 1973-1978 period, the agency had changed their deck preparation practices to remove not only damaged concrete but also concrete with half-cell potentials more negative than -0.35 v, and to assure a that newly exposed surfaces were freer of construction-induced microcracking than before by altering concrete removal techniques.

When the performance trend for each of the eleven individual studies for which enough data exists to define such a trend is calculated and the resulting trend lines are viewed together, as in Figure 6, two distinct performance groups with differences similar to those illustrated in Figure 5 become apparent:

Performance Group I - consisting of 6 of the study groups, 4 LSDC and 2 LMC; and

Performance Group II - consisting of 5 of the study groups, 4 LSDC and 1 LMC.

Clearly, the service life potential of overlays in these two groups has to be considered independently.

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The principal factors found to distinguish between them were the amount of chloride contaminated concrete allowed to remain in contact with rebar in the substrate, and the emphasis given to preparation of the substrate surface prior to overlay. Those treatments for which concrete removal criteria were based on half-cell potential rather than damage, for which concrete removal extended to below the rebar, and for which extraordinary care was taken to preclude the overlay being placed on microfractured substrate concrete, tended to be associated with Performance Group II. Sandblasting was used by at least one of the agencies as a final treatment before cleaning the surface prior to overlay.

For Performance Group I, mean damage values are given in Table 2 and are plotted for the first 15 years of service in Figure 7. These data suggest little difference between the performance of LMC and LSDC overlays in the first 15 years of service, and particularly in the first 10 years where the data is more abundant. The estimates of overlay service life are based on extrapolating the historical data to a condition of 40 percent total damage, as defined above. The 40 percent value has precedent in both the action policies of transportation agencies (11) and experience (12). A complete rationale for this criterion is given in Reference 1. Taking the two overlay types together:

- 1) A mean service life of 28.4 years is projected by straight-line least squares extrapolation; and
- 2) A mean service life of 18.2 years is projected by the best fit curvilinear function.

Both straight-line and curvilinear functions are given in Figure 7 because it is not yet clear from experience whether the decay curve for overlays should be represented by a straight line, as suggested by the performance curves of individual data sets (for example, those shown in Figures 5 and 6), or by a curved line of increasing slope, as has been the experience with unprotected decks (1). A modest argument in favor of the latter could be made on the basis of the slightly higher explanation provided by the curvilinear function, 0.92 vs 0.81, than by the straight line. Until this issue is clarified, a reasonable estimate for the mean service life of overlays in Group I, in the absence of local experience to the contrary, would be an average of the two extrapolated values, or about 23 years.

For Performance Group II, mean damage values are also given in Table 2, and are plotted in Figure 8 where

they have been superimposed on a 50-year, service-life trend line. Because the decay curve for these overlays is so flat, no distinction has been made between LMC and LSDC. While their performance to date does not lend itself to realistic extrapolation, it is safe to say that the Group II overlays have the potential for exceeding the service life of the Group I overlays very significantly. A 30-year minimum life is considered probable and one approaching 50 years possible.

SUMMARY

The use of thin, high performance concrete overlays to rehabilitate corrosion-damaged concrete bridge decks in the United States and Canada has been one of the highway industry's success stories of the last 20 years. Experience suggests that these treatments have the potential for extending the useful life of the riding surface of decks for considerably longer than had previously been thought. Variations in climate, traffic volume, and overlay type and thickness appear to be far less important determinants of their performance than the methods used to prepare the deck before the overlay is placed. When concrete removal criteria are based on half-cell potential rather than present damage, when removal of chloride contaminated concrete is extended to below the rebar, and when the substrate is sandblasted to remove microcracking prior to cleaning, service life potentials of 30 to 50 years are likely.

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TABLE 1 — AVERAGE CONDITION OF OVERLAYS AT DIFFERENT AGES BY OVERLAY TYPE

Age in Yrs.	LSDC		LMC		Combined	
	Number of Cond. Evals.	Mean Percent Damage	Number of Cond. Evals.	Mean Percent Damage	Number of Cond. Evals.	Mean Percent Damage
1	19	0.06	50	0.07	69	0.07
2	15	0.11	46	0.15	61	0.14
3	17	0.70	29	1.53	46	1.22
4	48	0.94	27	3.81	75	1.97
5	38	1.01	16	2.73	54	1.52
6	49	2.45	11	2.22	60	2.41
7	29	5.73	28	5.40	57	5.57
8	55	5.38	17	16.20	72	7.93
9	39	8.24	15	10.83	54	8.96
10	50	7.61	11	10.00	61	8.04
11	22	13.85	11	16.90	33	14.90
12	20	5.07	4	11.92	24	6.26
13	10	10.48	4	23.95	14	14.33
14	14	14.27	6	17.26	20	12.36
15	10	16.80	5	17.80	15	17.13
16	2	11.65	5	1.92	7	4.70
17	3	10.55	0	--	3	10.55
18	0	--	0	--	0	--
19	1	5.90	0	--	1	5.90
20	0	--	1	2.60	1	2.60
	441		286		727	

TABLE 2 — AVERAGE CONDITION OF OVERLAYS AT DIFFERENT AGES BY PERFORMANCE GROUP

Age	Group 1 LSDC		Group 1 LMC		Group 2	
	Number of Points	Mean Value	Number of Points	Mean Value	Number of Points	Mean Value
1	13	0.09	50	0.07	6	0
2	12	0.14	40	0.17	8	0
3	13	0.92	29	1.53	4	0
4	17	1.79	26	3.96	32	0.47
5	13	2.76	16	2.73	25	0.10
6	19	6.23	11	2.22	30	0.05
7	23	7.18	22	6.98	13	0.06
8	24	11.09	15	16.99	33	1.52
9	26	11.90	15	10.83	13	0.92
10	25	13.59	9	12.21	27	1.50
11	21	13.85	9	20.60	2	0.25
12	10	9.06	4	11.92	10	1.40
13	9	11.64	4	23.95	1	0
14	10	18.61	6	17.26	4	3.41
15	10	16.80	4	22.25	1	0
16	2	10.65	0	--	5	1.92
17	3	10.55	0	--	0	--
18	0	--	0	--	0	--
19	1	5.90	0	--	0	--
20	0	--	0	--	1	2.60
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	252		260		215	

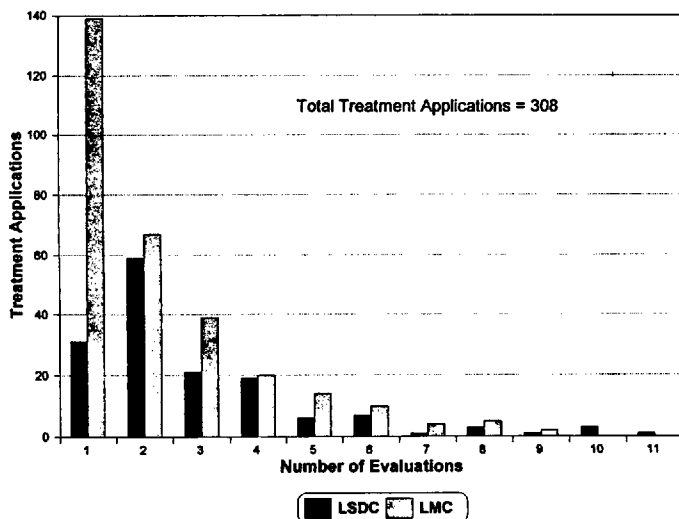


Fig. 1—Overlay evaluation frequencies

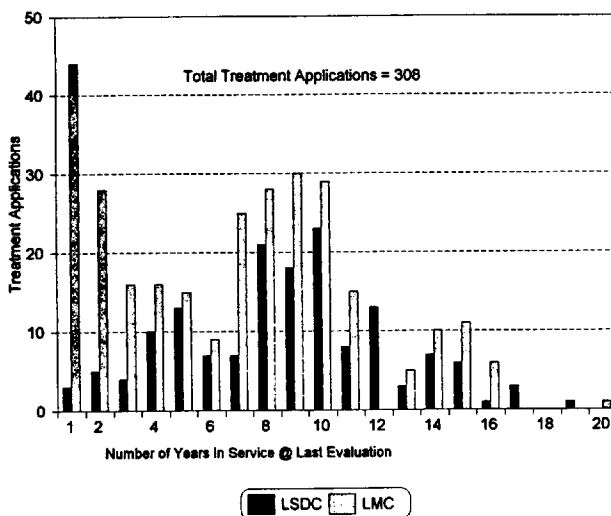


Fig. 2—Length of overlay evaluation terms

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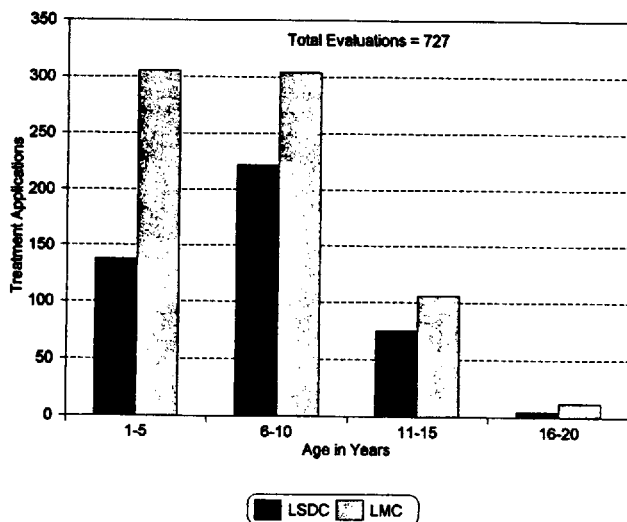


Fig. 3—Age distribution of overlay evaluations

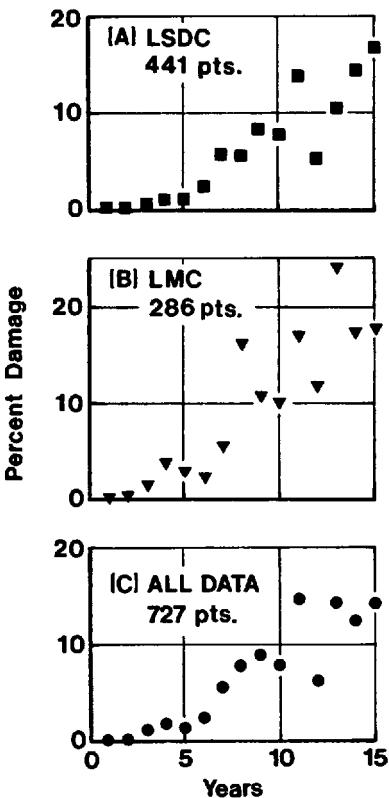


Fig. 4—Overlay performance by overlay type

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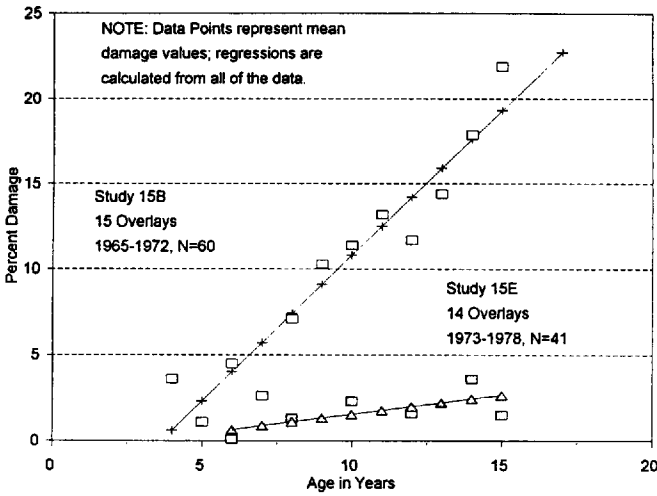


Fig. 5—Two LSDC overlay studies by the same agency

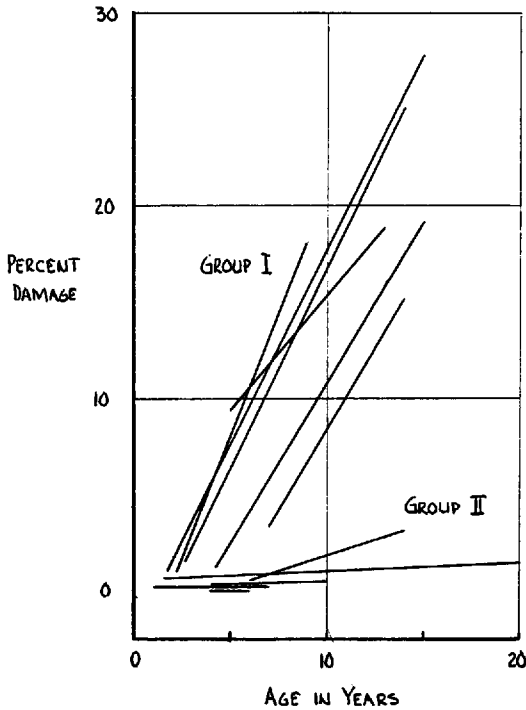


Fig. 6—Overlay performance groups

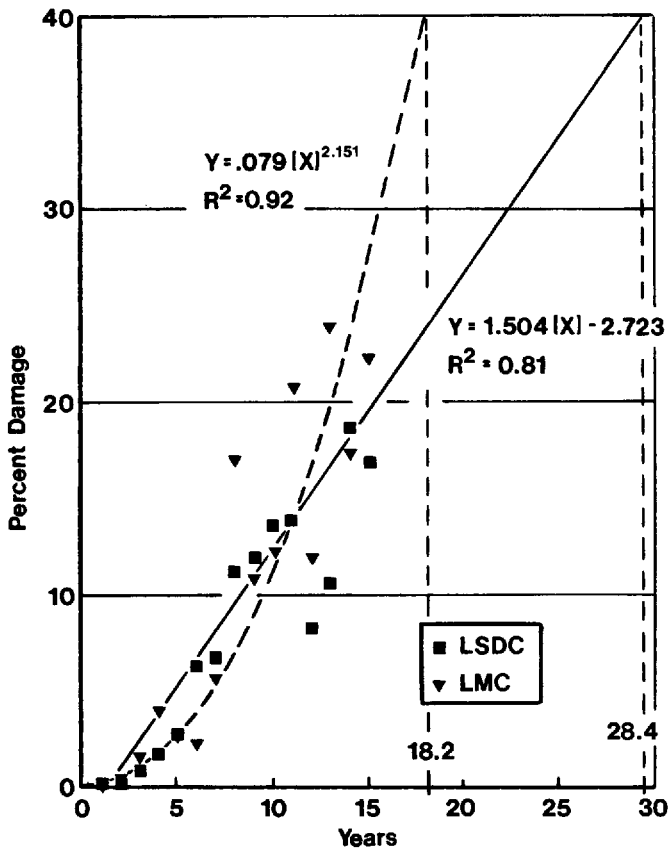


Fig. 7—Performance of Group 1 overlays

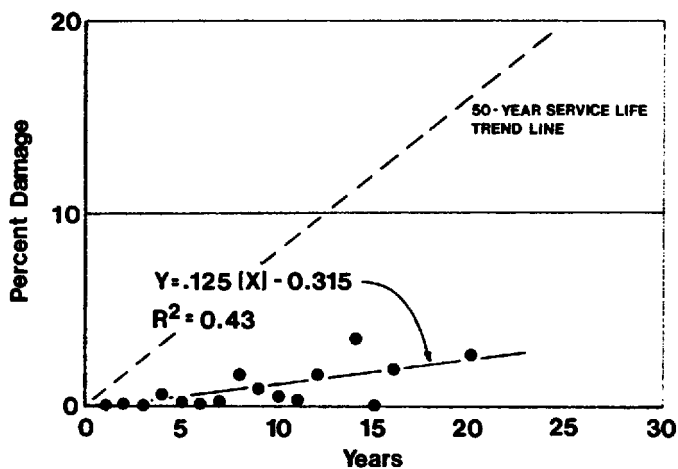


Fig. 8—Performance of Group 2 overlays

SP 151-2

Corrosion of Prestressed Reinforcing Steel in Concrete Bridges: State-of-the-Art

by M. Nagi and D. Whiting

Synopsis: The practice of prestressing steel has proven to be a very successful method of construction compared to conventional reinforced concrete in increasing load carrying capacity, improving crack control and slenderizing structural elements. However, corrosion in prestressed concrete has much more serious consequences than in normal reinforced concrete. Tendons are subjected to high mechanical stresses (often up to 70 to 80 percent of their tensile strength). Consequently, corrosive attack which results in weakening of the steel, if it oc

Under an FHWA contract dealing with rehabilitation of prestressed concrete bridge components by non-electrical methods, a comprehensive technology review focusing on corrosion of prestressing steel in highway structures was conducted and is summarized in this paper.

Types of corrosion and recent theories explaining stress corrosion and hydrogen embrittlement are presented. Susceptibility of prestressing steel to corrosion in prestressed and post-tensioned concrete structures are covered. Factors such as concrete materials, prestressing steel and environments which may influence such corrosion are categorized.

Laboratory and field studies dealing with a variety of corrosion issues in pretensioned and post-tensioned concrete are also presented. These issues include the development and improvement of grout materials for bonded tendons in post-tensioned concrete members, use of epoxy coated prestressing wires, and corrosion of unbonded tendons under severe exposure. Selected case histories and field evaluation of concrete bridges subjected to corrosion are also included.

This study gives an overview of corrosion problems in prestressed concrete members and should help engineers to diagnose causes of corrosion and select the right methods and materials to be used for rehabilitation as well as in new constructions.

Keywords: Bridges (structures); chlorides; concretes; corrosion; grout; hydrogen; prestressed concrete; prestressing steels; rehabilitation; reinforcing steels; stress corrosion resistance

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CORROSION

Corrosion, by definition, is the deterioration of a material, usually a metal, due to reaction with its environment. It is an electrochemical process that requires four elements in order to occur, these elements being a cathode, an anode, an electrolyte, and a conductor between the anode and cathode (1,2).

In the case of steel reinforcement embedded in concrete, the anode and cathode co-exist on the steel that also functions as a conductor, while the concrete acts as an electrolyte. In spite of the presence of this corrosion cell, corrosion of steel is unlikely to occur in concrete under normal conditions. In fact, steel reinforcement embedded in concrete is usually protected against corrosion by a passivating layer of iron oxide formed in the alkaline environment provided by concrete (1-4). When the protective oxide layer is destroyed locally or over the entire surface of the steel, corrosion will take place. The destruction of the passivated layer occurs when the alkalinity of concrete is reduced (e.g., by carbonation or when the chloride concentration is increased to a certain level) (5,6). Corrosion of the steel primarily causes the formation of voluminous iron corrosion products, which leads to delamination of the surrounding concrete. As the corrosion processes continue, the delamination eventually reaches the surface of concrete, resulting in a spall (7).

In prestressed concrete, corrosion potentially has much more serious consequences than in normal reinforced concrete. Tendons are subjected to high mechanical stresses, often up to 70 to 80 percent of their tensile strength. Consequently, corrosive attack (termed stress-corrosion) that results in significant weakening of the steel may lead to fracture (8,9). Two types of corrosion are recognized in prestressed concrete: localized or pitting corrosion and stress corrosion (9,10).

Pitting Corrosion

Pitting corrosion is the same type of corrosion as observed in normal reinforced concrete; it is electrochemical corrosion resulting from dissimilar or non-uniform metals, or dissimilar environments. Non-uniformity of prestressed steel is unlikely to cause formation of a corrosion cell except where parts of the wires have been subjected to sharp bends or kinks. The main factors influencing pitting corrosion are the presence of chloride and carbonation (7-15).

When chloride ions reach a sufficient concentration in the vicinity of the steel, localized corrosion may be initiated, provided moisture and oxygen are also present. Chloride ions are introduced into concrete in several ways. The most significant source of chlorides in prestressed concrete bridges is deicing salts used on bridge decks (5,6). Other sources of chlorides are chloride-contaminated aggregates, sea water spray, and marine atmospheres in offshore bridges. Chloride ions may rapidly penetrate concrete in pretensioned members where the prestressing wires are in direct contact with the concrete, as opposed to post-tensioned members where the wires are protected inside ducts. In post-tensioned members, chlorides could be introduced into grouts in bonded members when certain admixtures containing chloride have been used. Chlorides may also penetrate through anchorage zones if these are not completely encased in high quality mortars or concretes.

Carbon dioxide is another environmental factor that might lead to pitting corrosion in prestressed concrete. Alkalinity of concrete can be gradually reduced due to the carbonation process. This process involves the reaction of atmospheric carbon dioxide with the calcium hydroxide in the cement gel to form insoluble CaCO_3 and water, therefore reducing the alkalinity of cement paste and destabilizing the passivated layer of prestressing steel (14). As a result of pitting corrosion, the cross section of prestressed wire may be reduced locally, increasing tensile stress until there is a rupture of the wire.

In chloride environments, prestressing steels (high-strength steels) are less susceptible to pitting corrosion than ordinary mild reinforcing steel. A study conducted by Rengaswamy and Rajagopalan showed that the amount of chloride required to initiate pitting corrosion on prestressing steel is about two to three times that required to initiate pitting on ordinary steel (16). Although this would indicate that prestressing wires are less susceptible to corrosion, most building codes require that chloride concentration in prestressed concrete should be less than that of ordinary concrete; e.g., ACI Building Code 318-89 allows 0.06 percent chloride by weight of cement in prestressed concrete compared to 0.30 percent for normal reinforced concrete. This conservatism reflects the fact that prestressing wires are subjected to very high stress and any reduction in the wire cross section caused by pitting corrosion might lead to failure of that wire with serious structural implications.

Stress Corrosion

Stress corrosion, or stress corrosion cracking (SCC) refers to the transcrystalline (cracks passing through individual crystallites) and/or intercrystalline corrosion that occurs under certain circumstances in metals or alloys in which a state of static tensile stress exists (9). Generally, there is a threshold stress below which stress corrosion cracking does not occur.

Stress corrosion can be divided into two categories: anodic and cathodic corrosion. The anodic type takes place at the site of the anode in a corrosive situation where a specific active agent, different for each metal, is present. Intercrystalline stress corrosion of carbon steel such as prestress cables, in nitrate solutions is an example of anodic corrosion (9,10). It was suspected that chlorides could promote the occurrence of this type of corrosion, but researchers have found that this assumption is not correct, and stress corrosion cracking of chloride-contaminated concrete is unlikely to occur (7).

The second category of stress corrosion is cathodic corrosion. This type is mostly associated with hydrogen embrittlement as it occurs as a consequence of the cathodic reaction. Hydrogen embrittlement occurs when atomic hydrogen penetrates into metal lattice, where it recombines to hydrogen molecules producing high internal pressure. At one time, hydrogen embrittlement was not regarded as stress corrosion, and was treated separately (11,12,17-19). But, since embrittlement occurs in connection with cathodic reactions where hydrogen is generated at the cathode and occurs mainly under conditions of stress, this phenomenon has been considered as a type of stress corrosion. However, some researchers still treat hydrogen embrittlement separately. It is claimed that although the failure in both cases is sudden, brittle fracture, the processes leading to failure are fundamentally different (7). In addition, there are cases where hydrogen gas was adsorbed into the steel before it was stressed and failure occurred within a few hours of stressing (20).

Isecke developed a simplified description of the mechanism of hydrogen-induced stress corrosion cracking in a prestressed high-strength steel (21). Initially, a corrosion pit is formed at the steel surface due to anodic dissolution of iron. A stress corrosion crack is usually initiated at the base of the pit. Cathodic hydrogen is then evolved at the crack tip. The hydrogen is first adsorbed at the surface, then absorbed into the steel, and thereafter diffuses through the microstructure of the metal. Dislocations, grain boundaries, and inclusions are enriched with hydrogen, and when hydrogen concentrations exceed certain limits, brittle fracture occurs. The critical hydrogen concentration is dependant on the type of prestressing steel, grain structure, and stress level.

Another study regarding the influence of hydrogen on stress corrosion was conducted by Gavriljuk et al. in the early 1980's (22). They stated that in stress corrosion, hydrogen ions penetrate into the zone of high expansion at the crack tip, reducing the surface energy and easing crack growth. They studied the performance of prestressing steel in nitrate solutions, and concluded that when steel is "impregnated" with hydrogen under stress conditions, fracture occurs as a result of hydrogen embrittlement. Also, the factors which decrease hydrogen permeability of steel increase resistance to cracking, which confirms the significant role of hydrogen in stress corrosion cracking.

In post-tensioned members, corrosion cells caused by the presence of two dissimilar metals (galvanic corrosion) could lead to hydrogen embrittlement. If a galvanized duct is used and the grouting was inadequate, prestressing steel might touch the duct, and in the presence of moisture and air a galvanic cell may be formed. The formation of this cell could generate atomic hydrogen that could penetrate the steel and cause hydrogen embrittlement (7).

In any case, there is agreement in the literature regarding cathodic stress corrosion and hydrogen embrittlement in that the main source for hydrogen is the corrosion reaction, and the presence of hydrogen sulfide and sulfur dioxide can promote the entry of hydrogen into the steel. Also, the sensitivity of steel to hydrogen embrittlement depends on the nature and micro-structure of the steel, its surface condition, the level of tensile stress and the heat treatment to which the steel has been subjected. It has been found that cold drawn wire has a much lower tendency toward stress corrosion than oil-quenched and tempered wire of equal tensile strength (9).

SUSCEPTIBILITY TO CORROSION

Pretensioned Structures

In pretensioned concrete members, the prestressing tendons (wire, strands) are stressed to a predetermined tension and anchored to fixed bulkheads or molds. The concrete is then cast around the tendons and cured. Upon concrete hardening the tendons are released from their anchorages, thereby placing the cured concrete into a state of compression (23).

Pretensioning is the method most often used for the production of precast prestressed concrete elements in the United States because it offers great potential for mechanization. In bridge construction, the most popular pretensioned elements used in bridge superstructure are I and box beams. Six types of I beams developed and standardized by the American Association of State Highway and Transportation Officials (AASHTO) and the Prestressed Concrete Institute (PCI) are commonly used in bridge construction around the country. Modified bulb-T, double-Tee, and channel sections are also used in some States (24). Precast prestressed deck panels are also used on steel and prestressed concrete beams in new construction or for replacement of existing bridge decks (25).

While prestressed elements are routinely used in bridge superstructures, their use in substructures is relatively limited. This is because there are many differences between bridge sites with regards to soil-bearing characteristics, location of bedrock, and other local conditions, making it difficult to standardize these elements. (26) However, prestressed concrete is sometimes used in the construction of bridge substructure members such as piling, piers, and caps. A prefabricated abutment was used in a bridge in Virginia where prefabricated abutment elements were placed on the footing and then were tied together by post-tensioning (27). Another example of using precast prestressed concrete in a substructure is the pier construction in Linn Cove Viaduct in North Carolina where precast pier segments were placed on the top of each other and post-tensioned until the pier was completed (28).

Since the prestressing steel in pretensioned bridges is directly surrounded by concrete, its susceptibility to corrosion is similar to that of plain reinforcement in ordinary concrete. Factors affecting corrosion of prestressing steel in this type of structure are: concrete quality, prestressing steel type, and environment.

Concrete. Quality of concrete is an important aspect of corrosion protection of prestressing steel used in prestressed concrete. Porous concrete will allow chlorides, oxygen, and carbon dioxide to pass through the concrete; thereafter, steel will be depassivated and begin to corrode. Concrete should be produced with a water-to-cement (w/c) ratio below 0.45 where exposure to corrosive environments is anticipated. Concrete cover is an important factor affecting the corrosion of embedded steel. Inadequate cover thickness will enable aggressive substances such as chlorides to penetrate more quickly to the steel. AASHTO bridge specifications require that minimum concrete cover shall be 1.5 in (38 mm) and 2 in (51 mm) when deicers are used (29). When deicer chemicals are used, drainage details should ensure disposal of deicer solution without constant contact with the prestressed beams. Where such contact cannot be avoided, or in locations where members are exposed to salt water or salt spray, or when increased service life is desired, additional cover should be provided.

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The use of chemical admixtures that contain chlorides (such as calcium chloride) during concrete placement, in the presence of moisture, will cause future pitting corrosion of prestressing steel. Calcium chloride is not recommended for use in prestressed concrete, and its use is also decreasing in plain reinforced concrete. Chloride-free accelerators are now available and can be used as alternatives to calcium chloride (30,31).

It should be mentioned that the influence of concrete quality on susceptibility of prestressed concrete bridges to corrosion is the same for all bridge elements (beams, piers, caps, etc.). However, superstructure elements (beams and bridge decks) in cold areas where deicing agents are used are more susceptible to corrosion than substructure elements since they are directly subjected to deicing salts. In marine environments the opposite tends to be true, as support structure elements can be directly exposed to the action of marine spray and periodic tidal inundation.

Prestressing Steel. In prestressed concrete, high-strength steels are needed to achieve the required prestressing forces. Prestressing steel should remain elastic up to relatively high stresses, show sufficient ductility prior to failure, and have good bonding properties. The most popular sizes used in the United States are the 3/8-in (9-mm) and the 1/2-in (12-mm) diameter strands (32).

Small diameter tendons are generally used in pretensioned elements to allow for the bond between steel and concrete to develop over a shorter distance. These wires may be distributed through the entire structural section, or alternatively, in the form of cables or strands. Distribution of wires should provide proper spacing, good embedment of all steel, and adequate cover (9).

Based on the manufacturing procedures employed, prestressing steel can be divided into two categories: (1) cold-drawn and (2) quenched and tempered steel.

(1) Cold-drawn steel: In the cold-drawing process, wire is manufactured by drawing a hot-rolled rod through a series of dies, each of successively smaller diameter, until the wire is reshaped to the proper diameter. This process is performed following a heat treatment (patenting) applied to the rod. A uniform metallurgical structure is obtained that provides high tensile strength with high ductility. This imparts to the rod the ability to withstand the drawing required to produce the desired wire size and greatly enhanced tensile and yield strength (10,33-37).

Most specifications (ASTM C 416, AASHTO M 203) require cold-drawing wire to be subjected to stress-relieving continuous heat treatment. This relieves stresses built up during the drawing process. The most common prestressing steels manufactured in the United States are seven-wire stress-relieved strands (AASHTO M 203), stress-relieved wires (AASHTO M 204), and high-strength steel bars (AASHTO M 275). Two grades of stress-relieved strands are specified (grade 250 and 270). Compacted stress-relieved seven-wire strands are also available in which the strand has been compacted by drawing through a die or a similar compacting process and subsequently stress-relieved prior to winding into coils or reel-less packs (ASTM A 779).

The cold-drawing process leads to deformation of the crystalline structure of the steel, making the crystals longer in the direction of drawing. The elongated crystals tend to align in the direction of drawing, which is the direction of stressing. This form of deformation makes this type of steel less susceptible to stress corrosion for two reasons. First, it makes it harder for cracking to

penetrate the steel grain structure, since the crack has to travel along the boundary in the direction of elongated length before being able to continue on an inward course. Second, in an SCC situation where crack propagation can be directed parallel to the direction of stress rather than perpendicular to it, wire failure can be prolonged or in some cases prevented (10).

(2) Quenched and tempered (Q&T) steel: In this process, the rolled wire is heated to between 1472 and 1652 °F (800 and 900 °C) and quenched in oil, whereby the steel acquires a very strong, hard, and brittle structure. To avoid the problems associated with hardness, steel is treated immediately after quenching by tempering at between 752 and 932 °F (400 and 500 °C) (33,38,39).

The micro-structure of this steel consists of relatively fine spherical crystallites without orientation effects. This makes it more susceptible to stress corrosion than cold-drawn wire (9). This type of steel is not permitted in current AASHTO specifications.

While Q&T steel is not approved for use in the U.S., it has been used in Germany (35). In Japan, further development of Q&T steel has produced a silicon-chrome low-carbon heat-treated steel comparable in corrosion behavior to stress-relieved steel in accelerated tests (40).

Environment. In prestressed concrete bridges, environment plays a major role in corrosion of prestressing steel. Access of water (rain, snow) into concrete, in the presence of oxygen and chloride ions (considered the major corrosion initiator) leads to pitting corrosion of prestressing steel embedded in concrete. Chloride ions also penetrate concrete in marine environments. In marine bridges, migration of chloride into concrete can be from direct exposure to sea water (e.g. on a bridge pier), sea spray, or marine atmospheres (airborne chloride in marine fogs) (41). In the northern part of the United States, chloride intentionally applied to concrete as deicing salts can penetrate through deck joints and gutters and reach such prestressed elements as beams and columns (42).

Carbonation is another environmental effect where carbon dioxide enters concrete, reduces alkalinity, and eventually leads to corrosion of steel. Reaction of CO₂ with sulfides that may be present in high-alumina cement concrete may generate H₂S and lead to hydrogen embrittlement (9).

In industrial areas, the presence of polluting substances in the atmosphere, such as sulphur dioxide (SO₂), sulfur trioxide (SO₃), and hydrogen sulfide (H₂S), can lead to stress corrosion cracking in prestressed concrete bridges. These aggressive substances are more likely to attack prestressing wires during transportation and storage. Precautions have to be taken also during in-service exposure in such environment (9).

Post-Tensioning

In post-tensioned members, the prestressing tendons are stressed and anchored at the ends of the concrete member after the member has been cast and has attained sufficient strength (32). Two categories of post-tensioned construction are available: bonded and unbonded. In bonded post-tensioned construction, the tendon ducts are placed in the forms prior to casting then filled with a mortar grout after stressing. In unbonded construction, the tendons are greased and covered with plastic (43).

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Voided post-tensioned slabs and trapezoidal voided post-tensioned slabs are examples of post-tensioned systems currently used in bridge superstructure construction. Post-tensioned systems are also used in the construction of bridge substructures such as piers and abutments (28).

Segmental bridges are another example of post-tensioned systems where each span consists of several segmental units post-tensioned longitudinally.

Steel tendons in bonded post-tensioned concrete members are usually less susceptible to corrosion than those in pretensioned members under similar conditions. This is because the prestressing steel in the post-tensioning system is protected inside ducts and isolated from the surrounding concrete. Therefore, corrosive conditions such as the presence of chloride ions in concrete and the reduction of alkalinity of concrete that usually cause corrosion in prestressed concrete will have less effect on post-tensioned members. However, this protection depends on the quality of the ducts, grouts, and anchorage system. Susceptibility of post-tensioned concrete members to corrosion is influenced by several factors. These factors include: concrete, environment, prestressing and anchorage systems, sheaths and ducts, grouts, and joints.

Concrete. The influence of concrete on corrosion of post-tensioned members is less than in the case of pretensioned units. However, good quality concrete and optimum concrete cover are required to protect ducts, especially near anchorage zones.

Environment. Environmental factors influencing post-tensioning systems are similar to those described for pretensioned systems. However, the effect of the environment on corrosion of prestressing strands in post-tensioned members is less severe than that in pretensioned systems since the wires are protected inside the ducts. Nevertheless, anchorage zones in post-tensioned members are sensitive to corrosion and will be affected if they are exposed to leakage and/or chlorides.

Post-tensioning and anchorage systems. The most common systems used in post-tensioning are strand, bar, and wire systems.

Strand systems are based on the use of seven-wire strand with 270,000 lbf/in² (1,860 MPa) ultimate tensile strength in diameters of 3/8, 7/16, or 1/2 in (8, 11, or 13 mm). A conical wedge anchorage is generally used. The anchorage consists of a steel cast wedge plate in which the tendon is gripped by a two-piece wedge. After the concrete has gained the necessary strength, the strand is stressed and conical wedge grippers are inserted around the strand in a conical hole in the bearing plate to provide anchorage (43,44).

In bar systems, deformed high-strength bars are usually used with an ultimate strength of 150,000 lbf/in² (1,030 MPa), and with diameters ranging from 5/8 to 13/8 in (16 to 32 mm). A common anchorage for this system is a plate anchorage with a threaded coupling. Usually the deformations in the bars act as threads and make it possible to cut the bar at any point and screw on either an anchor or a coupler (43,44).

Wire post-tensioned systems use tendons made of high strength wires of 0.25 in (3 mm) in diameter and ultimate strengths of 240,000 lbf/in² (1,650

MPa). Button-head anchorages are used for the individual wires. Tendons for this system are usually cut to the exact length and button-headed in the shop. As previously mentioned, the manufacturing process of prestressing steel has an influence on its susceptibility to corrosion. Cold-drawn steel is used in post-tensioned systems as it is less susceptible to stress corrosion.

The anchorage zone is one of the regions most susceptible to corrosion in post-tensioned members. If the anchorage grouts are not in intimate contact with the anchorage components (due, for example, to shrinkage cracks developing between the mortar and surrounding concrete), moisture may penetrate to the prestressing wires and promote corrosion (45). In connecting the sheaths to anchorages in unbonded members, the sheaths are usually cut to a certain length to make a proper connection, leaving part of the steel exposed and subjected to moisture during the period between installing the tendons and placing the concrete. Waterproof tape is sometimes used to protect this part of the steel, but it is rather difficult to achieve a watertight seal in this way (45).

Schupack recommends that plastic sheaths be used to provide a positive waterproof connection to the anchorage and thereby provide improved protection against corrosion (46). He also recommends that the concrete cover over the end of the strand be at least 1 in (25 mm) in benign environments and 2 in (50 mm) in aggressive environments. Perenchio et al., in their study of corrosion protection of prestressing systems, confirmed the vulnerability of the anchorage region to corrosion damage (47). They indicated that epoxy coating on the anchorage and associated hardware generally provides very good to excellent corrosion protection. They also indicated that the end of the anchorage may require additional corrosion protection to that traditionally provided by dry-packed mortar.

Sheaths and Ducts. In post-tensioned concrete, because the tendon cannot be stressed until the concrete has been placed and has attained significant strength, it is necessary that the concrete be prevented from coming into contact with the tendons before it has hardened. This can be achieved by one of the following methods: In the first method, tendons are enclosed in factory-applied sheaths. This method is used for unbonded post-tensioned tendons. In the second method, preformed holes are located where post-tensioned wires are to be placed, and wires are inserted in these holes and tensioned and grouted. Use of metallic (normally galvanized steel) or plastic ducts is the third (and most popular) method. In this method, tendons are placed in ducts after the concrete is hardened. The post-tensioning wires are tensioned and the ducts are then grouted. The ducts are placed in the forms prior to casting the concrete, therefore, they should be sufficiently stiff to maintain their shape during placing and consolidation of concrete (48).

Corrosion of prestressing steel associated with unbonded tendons can occur in several ways: (1) grease, which is placed between the sheath and the tendons, can be scraped off during handling and storage, (2) grease could react with the sheath and the grease or sheath may cease to perform its function, grease may also oxidize, harden, dry, and crack as it ages (3) and might not be completely impermeable and ambient moisture will penetrate through the grease and to the steel, especially if the sheath is not sufficiently thick and watertight (45).

Unbonded tendons embedded in concrete are not widely used in highway bridges because of the difficulty of ensuring adequate corrosion protection

(7,49,50). However, they are used as external tendons, especially inside box beams, where the tendons can be inspected. The Long Key Bridge in Florida is a good example of using such a technique. External tendons are also used when bridges are being rehabilitated in order to restore structural integrity to the member.

Metallic ducts, which are widely used in post-tensioned concrete bridges, can lead to corrosion of prestressing wires if they are subjected to any kind of damage during shipping and storage or during placement of the concrete. If holes exist in the ducts, moisture may get to the wires prior to placement and cause corrosion of these wires (51). Also, plugs of concrete may be formed in these holes during casting. This will hinder the tensioning and grout injection and cause improper grouting that might lead to corrosion in these areas. Therefore, ducts have to be sufficiently strong to prevent crushing or puncture during installation or concrete placement, and stiff enough to prevent any cutting or crushing caused by prestressing steel. Also, they should have adequate chemical stability to avoid destructive reactions with cement, grout, or other substances (44,47).

Galvanized steel ducts have been found to achieve better corrosion resistance than bare steel. Polyethylene ducts and epoxy-coated steel ducts have shown comparable performance to galvanized when used under very severe conditions (47).

Although ducts and grouting can provide corrosion protection to tendons, precautions must be taken to protect tendons after they are placed in the ducts, but prior to grouting. These precautions include: (1) filling the ducts with water-soluble anti-corrosion oil that is then flushed out of the ducts with water a short time before grouting, or (2) using a vapor phase inhibitor (VPI) powder. This powder develops an increased vapor pressure even at low pressure, thus preventing the emission of harmful gases into the ducts, and forms a protective coating on the steel surface. It should be mentioned that ducts should be kept properly sealed after the VPI powder has been introduced (9,18,52).

Grouts. After stressing and anchoring, voids between tendons and ducts are filled with a mortar grout, which subsequently hardens. Grouting ensures bonding of the tendon to surrounding concrete, improves structural rigidity, and reduces the risks of corrosion for the steel tendons (32).

Most grouts consist of portland cement and water, either with or without special additives. Portland cement mortar, if properly used, provides ideal corrosion protection. High alkalinity of cement passivates the steel wires and dense cement paste prevents the penetration of corrosive ions. Corrosion of prestressing steel caused by grouting in many cases can be attributed to poor grouting practices as well as grouting materials. Corrosion can occur when voids are formed inside the ducts leaving tendons exposed to air and moisture.

The major problems encountered in grouting are line plugs, formation of water and air voids, and shrinkage cracks (53). Line plugs (when one-way flow of grout cannot be maintained) can be caused by damage to the duct, foreign material within the duct, and rapid stiffening of the grout. If these plugs are formed, the grout should be immediately flushed out of the duct.

The formation of water lenses and voids caused by bleeding of the grout mixture or by improper grouting practices can be reduced by maintaining a water/cement ratio of less than 0.45. Water left in the ducts from the flushing

operation (which may cause water lenses) can be removed by continuing the flow of grout until the excess water is removed (53).

Perenchio et al. in their NCHRP study, compared two specialty grout formulations (calcium nitrite-modified and silica fume-modified) with conventional grout. Their investigation showed that the two grout formulations did not provide significantly improved corrosion protection during very severe corrosion tests compared to the conventional grout. They indicated that grouting procedures are as important as the grout materials (47). While Thompson et al. in their recent FHWA-sponsored study on grout development, indicated that the use of modifiers and additives such as silica fume in grouts provides improved resistance to pressure-induced grout bleeding and improved corrosion performance (54).

Finally, it should be mentioned that in bonded post-tensioned bridges, if the tendons are properly grouted, corrosion and even failure of end anchorages is not immediately dangerous to the structural integrity of the bridge, although with certain designs some loss of shear capacity may occur. However, the continued integrity of anchorages would provide added insurance against loss of prestress in cases where the grouting was not 100 percent effective (18).

Joints in segmental bridges. In some segmental bridges constructed in the early 1950's, mortar was used to fill segmental joints. The reasons for using mortar at this time was that segments were cast individually in molds (not match cast) and it was therefore necessary to use this material to assure uniform bearing between the segments. The need for mortar is now avoided by using the match casting technique. In this technique, several segments are sequentially cast against each other using a light coating in the joint between the members to prevent their becoming bonded to each other.

Normally, the ends of individual segments are coated with epoxy resin at the time of assembly. The epoxy is used to ensure a good fit and to provide protection against the intrusion of air and moisture (48). For joints between segments in compression members (e.g., prefabricated bridge piers), the European Concrete Committee (CEB-FIP) recommends that bedded joints with mortar, concrete, or polymers as padding material may be used. Dry joints (with no intermediate padding material) may be used where great accuracy in manufacturing and installation is applied (55).

Improper protection at segmental joints at the Ynys-y-Gwas Bridge allowed moisture and chlorides to reach tendons and led to corrosion and collapse of the bridge (56).

CORROSION STUDIES

Laboratory Studies

A great deal of the research in the area of corrosion of prestressing steel relates to laboratory testing methodology of prestressing steel in conditions where cables were not actually embedded in concrete. Corrosion performance in solutions of various compositions may differ significantly from that in actual internal concrete environment.

In the early 1980's, FIP (Federation Internationale De La Precontrainte) published a report summarizing a joint experimental study conducted by several countries aimed at developing a stress corrosion test for steel prestressing

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wires (17). Six types of tests were considered in this comprehensive study. These tests were:

- Tests in boiling nitrate solution (without electrical polarization).
- Tests in hydrogen sulfide (H_2S) with or without cathodic polarization.
- Tests in 20 percent solution of ammonium thiocyanate (NH_4CNS) with and without cathodic polarization.
- Tests in decinormal solution of sulphuric acid (H_2SO_4) with and without cathodic polarization.
- Tests in distilled water with electrical polarization.
- Tests in a saturated solution of calcium hydroxide with anodic and cathodic polarization.

Several factors such as steel type and diameter, temperature, oxygen concentration in solution, and load applied to specimens were considered in this research program.

Following several years of investigation, it was concluded that the ammonium thiocyanate test is the optimum stress corrosion test. This test consists of determining the failure time of a sample maintained under a constant tensile load in a solution of ammonium thiocyanate at constant temperature. This test is mainly used to evaluate tendons based on their resistance to hydrogen embrittlement.

Stress corrosion of high carbon steel wires was evaluated by Schwier (57). Steel wires were subjected to hot solutions of calcium ammonium nitrate under different loads. This study showed that oil-tempered wires are considerably less resistant to stress corrosion than the drawn and stress-relieved steel wires.

The susceptibility to stress corrosion of prestressing steel processed using different final thermomechanical treatments was studied by Cvitanovicova and Mikulec (58). Steel wires were tested in a 20 percent solution of ammonium thiocyanate at 176 °F (80 °C). They found that optimizing mechanical and relaxation properties improves the resistance to stress corrosion. They also concluded that steels with a modified chemical composition are more susceptible to SCC than wire made in a similar manner from standard carbon steel.

A recent study of corrosion of cold drawn and stress-relieved prestressing steel was conducted at the Central Electrochemical Research Center in India (59). In this study, steel specimens were tested in a phosphate medium at room temperature and exposed to constant cathodic, anodic, or unpolarized conditions and stressed to 90 percent of ultimate stress. Results indicated that shortest failure times were experienced under the unpolarized conditions. This indicates that overpotentials are not necessary for hydrogen embrittlement to occur in highly prestressed wires. In further experiments, stress thresholds as low as 10 percent of proof stress were found to induce susceptibility to SCC. They concluded that susceptibility of cold-drawn wires to stress corrosion in a phosphate medium is high under unpolarized conditions and decreases under anodic and cathodic polarization.

As Elice reported, "the accelerated tests developed by FIP and others do not represent conditions in portland cement environments" (40). Therefore, in order to completely understand the corrosion performance of prestressing steel in concrete, steel should preferentially be tested in concrete environments under conditions similar to field exposure.

Several major laboratory studies dealing with corrosion of prestressing steel in a concrete environment have been reported. One of the most comprehensive efforts was the NCHRP study conducted by Moore et al. in the late 1960's. The main purpose of this investigation was to evaluate the corrosion of steel in prestressed concrete bridges and to develop methods for minimizing corrosion damage. Both prestressed and post-tensioned concrete bridges were evaluated (18). In addition to the testing of steel wires in simulated conditions (e.g., by oxygen-concentration cells), concrete specimens were made to optimize cover thickness and evaluate the effectiveness of steel coatings. Flow behavior of grouts was also studied and information on such variables as water-to-cement ratio, type, and amount of additive were also provided. Full-scale grouting tests were made based on the laboratory findings. Results of this research program established a basis for understanding and improving the performance of prestressing steel in concrete bridges. The main conclusion was that corrosion of prestressing steel is not a serious problem in concrete bridges and should not be a problem in the future if certain precautions are considered (18). These include the use of sound crack-free concrete with cover thickness recommended in AASHTO specifications, protection of prestressing steel during shipment and storage to prevent corrosion, and minimizing or eliminating voids between the tendons and grout by maintaining the pumping pressure on the grout for several hours after the grouting has been completed.

Corrosion of unbonded tendons in concrete has also been studied in the laboratory. Two major studies dealing with corrosion performance of unbonded tendons were performed outside the United States. In Japan, performance of post-tensioned concrete specimens subjected to long-term marine exposure was monitored and evaluated. Concrete prisms were fabricated and post-tensioned with unbonded tendons in order to investigate the performance of grease-and plastic-covered prestressing strand and anchor fittings for unbonded tendons embedded in concrete. Some of these prisms were placed on the shore so that they were immersed in sea water at high tide and were dried in the air at ebb tide. One set of these prisms were checked after 1 year of marine exposure. No sign of damage was observed and tendons and anchorages performed well. After 10 years of exposure, many cracks were observed on the concrete surface due to corrosion of non-prestressing steel. Damage was also found at the anchorages on all specimens. Although the exposure conditions were very severe, the strands of the unbonded tendons were found to be corrosion-free. It was concluded that the treatments applied for the corrosion protection of the anchorages were not satisfactory. However, tensile testing of anchored tendons showed that the efficiency of anchorage fittings remained around 100 percent. The authors stated that careful detailing is essential for better corrosion protection of the anchorage fittings (60).

In the Netherlands, long-term exposure tests were conducted on post-tensioned concrete specimens. Two unbonded post-tensioned slabs were fabricated and subjected to outdoor conditions for more than 3 years. Anchorage recesses were given various protective treatments and were then filled with different types of mortar. The main conclusion of this study was that the best results with respect

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to corrosion protection of the anchorage zone could be achieved by applying an epoxy bonding agent to all parts of the anchorage recess before the mortar filling is introduced. Their study also showed that non-shrink mortar for filling the recess did not offer any advantages over conventional mortars when exposed to the same conditions (45).

Another comprehensive laboratory research program dealing with performance of prestressing steel in concrete was conducted by Perenchio et al. (47). An accelerated corrosion test program was undertaken on bare and epoxy-coated prestressing strands using pretensioning and post-tensioning methods with traditional and new materials and hardware. Plastic and epoxy-coated steel ducts were used and compared with bare and galvanized steel ducts. Epoxy-coated anchorage hardware was also evaluated. Epoxy-coated prestressing strands demonstrated excellent corrosion performance in both pretensioned and post-tensioned specimens under severe exposure conditions. These conditions include subjecting specimens to alternate air drying and ponding with salt water, concrete cover of 1 in (25 mm) or less, and initially inducing flexural cracks in the specimens to facilitate penetration of salt water through concrete. This study also showed that epoxy coating of the anchorage and associated hardware provides very good protection against corrosion. Also in post-tensioned members, plastic and epoxy ducts have better performance with regard to protection of prestressing steel under severe exposure conditions.

The most recent laboratory study in the areas related to corrosion of prestressing steel was sponsored by the FHWA for improving grouts for bonded tendons in post-tensioned bridge structures.(54). The objectives of this study were to develop new mixture designs for grout used in ducts to provide long-term corrosion protection for prestressing steel and to develop and apply new accelerated corrosion test methods to the grouts. Specimens were prepared using different types of grout, which included standard grout, grout incorporating silica fume, and latex-modified grouts. An accelerated corrosion test (ACTM) and modified chloride permeability test (AASHTO 277) were performed on these specimens. The main conclusion of this research program was that additives such as silica fume and latex modifiers that significantly reduce chloride permeability also significantly improve corrosion performance of grouts.

Field Studies

Reports of field investigations and research regarding the corrosion performance of prestressing steel in concrete are very limited. The earliest field evaluation was conducted by the U.S. Army Engineer Waterways Experiment Station in the early 1960's (611,62). In this project, 20 air-entrained, post-tensioned beams were fabricated. Four different types of post-tensioning systems were used. Beams had either external or flush type end anchorages and were protected by either air-entrained or epoxy end caps or end plugs. The beams were installed at the Treat Island exposure station in Maine where they underwent field exposure of twice daily tidal inundations and an average of 131 cycles of freezing and thawing each winter. Beams were inspected yearly to determine the condition of the end anchorages.

Several reports summarizing the progress of this research program were published. In report no. 4, O'Neil summarized the laboratory investigation on eight of these beams after 13 years of field exposure (61). This investigation included: visual observation and photographic inspection, structural testing, and

physical and chemical analyses. The main conclusions obtained from this study were:

- 3/4-in (19-mm) concrete cover is determined to be inadequate to protect steel from exposure to seawater and freezing and thawing.
- Steel strands were not structurally damaged when they were surrounded by standard grout inside ducts.
- Paper conduit filled with grease provided the strands with the least amount of protection during long-term field exposure.
- The best protection to the end anchorages was provided by epoxy end caps.
- Flush anchorage plugs showed excellent weathering resistance. However, since they were surrounded by concrete, protection afforded to anchorages depended on the performance of concrete, and the plugs only protected the anchorage as well as the concrete itself.

Moore et al., in their NCHRP study, located out-of-service bridges for evaluation of the field performance of prestressed members (18). Two bridges were selected for this study. A fragment of a pretensioned beam from an out-of-service bridge in Colorado had been placed on the bank of a river for 1.5 years. The bridge had been destroyed by a flood after 2 years of service. Visual inspection showed that interstices between the center and outer wires of the 0.5-in (13 mm) diameter strand were filled with hardened cement paste which retarded moisture penetration and corrosion of the steel wires. Laboratory examination showed that no corrosion had occurred on any outer surfaces of the strand that had been in contact with concrete.

Prestressing steel was removed from an out-of-service bridge in California after 11 years of marine exposure. It was found that the steel had maintained its initial physical properties. Corrosion of the steel only occurred at the ends of the tendons, and even at the ends penetration inward along the wire was minimal because of the presence of hardened paste in the interstices of the strand. Based on the examination of these out-of-service bridges, the authors concluded that concrete is capable of providing excellent protection for pretensioned steel.

A field evaluation of inservice prestressed concrete bridges was conducted by CTL in the mid- 1980's (33). In a research program sponsored by the FHWA, a total of seven bridges were subjected to detailed surveys. The surveys included: visual examination, delamination of concrete cover, petrographic analysis of cores, chloride concentrations of the concrete, measurement of chloride permeability, potential surveys, and metallurgical analyses on prestressing steel obtained from the structures. Bridges selected for the study were located both in northern climates subjected to application of deicing salts and in southern areas subjected to marine environments. This investigation showed that the deck slab joints were largely responsible for corrosion-related deterioration of concrete in members of prestressed bridges in northern areas. The most common joint defects were: deterioration of joint sealer, lack of sealer, and cracking of concrete around joints. The highest concentration of chlorides in concrete was found in beams where most corrosion-related deteriorations took place, i.e., in areas adjacent to leaking joints. A sufficient drainage system (including proper sizing of intakes, removable intake grids, and a pipe slope not less than 60 degrees) was

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found to be important in providing effective removal of storm water and deicing salts from a bridge deck without the draining of water onto adjacent support elements. While low cover can lead to premature corrosion failure, insufficient concrete cover over prestressing steel was found in only two bridges in this study. This was attributed to faulty design and/or poor construction practices. In summary, inadequate concrete cover, malfunctioning drains, leaking expansion joints, and poor protection of the anchorage systems were among the most important causes of corrosion of the bridges evaluated in this study.

In 1984, the Michigan Department of Transportation initiated a research program to evaluate the overall condition of their prestressed concrete bridges, by determining the chloride content of several representative bridges and inspecting a sampling of the entire statewide prestressed concrete beam bridge population. Among 4,500 bridges in Michigan, 626 use prestressed concrete support beams. Most of these structures are over 20 years old. I-beams and box beams are the most common types of beams used in these bridges. In the Materials and Technology Engineering and Science issue published by the Michigan Department of Transportation in December 1990, a summary of this project was presented. Salvaged box and I-beams from several bridges were subjected to destructive testing (63). The beams were photographed and visually evaluated and all cracking and staining were categorized. Concrete samples were taken for chloride analysis at the level of steel strands at various locations. Steel strands were removed from the beams and visually inspected to determine their overall condition.

Five I-beams and seven box beams were selected for this phase of the investigation. Beams were selected from county roads over rivers and from the interstate system. It was concluded that the prestressed concrete beams were in good condition. Any deterioration observed on these beams was of a relatively minor magnitude. The following conclusions with relation to corrosion were obtained:

- All steel strands removed were from structures carrying local traffic. The data indicate that steel strands removed from 23-year-old I-beams were in better condition than those removed from the 14-year-old box beams.
- The ends of the box beams on all structures exhibited much greater deterioration than the midspan areas. This is the result of the salt-laden water leaking from the joints over the supports, leading to higher chloride contents and consequently more deterioration of box beams near supports.
- The ends of I-beams also showed more deterioration. However, on the majority of the I-beam structures, limited concrete delamination and spalling were observed at the beam ends.
- Water staining between adjoining box beams was more visible on Interstate bridges. Almost no water stains were found on the county structures.
- It appears from the beams evaluated in this study that the performance of I-beams was better than that of box beams. Although the I-beams were older by 9 years, they were in much better conditions than the box beams. However, the chloride content was higher in the I-beams

since most of them were on interstate highways, while the majority of the box beams were on county bridges.

- For beams over traffic lanes, chloride analysis showed that the traffic flow has an effect on the chloride content of the prestressed beams. The traffic-sprayed beams had higher chloride contents.

CASE HISTORIES

Three recent case histories related to concrete highway bridges were selected for this review. These cases are: investigation and rehabilitation of prestressed box beams on an elevated expressway in Canada, collapse of a segmental post-tensioned concrete bridge in South Wales, and the long-term performance of a demolished prestressed concrete bridge in Germa.

Case No. 1

This case was presented by Tork in two papers. (64,65). The first, published in 1985, dealt with the investigation of prestressed concrete box beams on an elevated roadway in Canada, and the second, published in 1989, dealt with rehabilitation procedures used on the beams. The roadway consists of simply supported, precast, prestressed concrete box beams supported on reinforced concrete bents, and covered by a cast-in-place concrete slab and asphalt overlay. The concrete cover over the beams is 2 to 4 in (50 to 100 mm). The wearing surface is approximately 3-in (75-mm) thick, consisting of two layers of asphalt. There are 46 spans on the main roadway and 59 spans on the approach ramps. The total structure contains 1,511 beams (most of them pretensioned). They are laterally keyed by continuous mortar keys, and are lightly transversely prestressed by two strands at mid-span and at quarter points. Prestressing steel in all beams is a stress-relieved 0.5-in (13-mm) diameter seven-wire strand stressed to 270,000 lbf/in² (1,860 MPa).

Methods of investigation included visual observation, coring, and interior inspection of the box beams. Two major types of deterioration were observed. First, spalling, cracking, and chemical breakdown of concrete and rusting of reinforcing steel at transverse joints was noted. Secondly, rusting of prestressing and reinforcing steel with consequential delamination, cracking, and spalling of concrete was observed on beam soffits. Both types of deterioration were attributed to chloride penetration. Initial observations on transverse joints identified spalling on exposed side faces at the ends of the exterior beams. Closer observations showed that spalling occurred at many joints. Concrete below the gutters on both sides was soft and friable. At these locations the concrete had obviously been exposed to extensive leakage of salt water from the defective expansion joints. Visual observations from the ground on beam soffits showed leakage at nearly all transverse joints, and at a considerable number of longitudinal joints between the beams. The worst spalling was observed at drain pipes that were located at gutters and just outside the faces of the concrete bents. Analyses of samples taken from different locations indicated that the sources of chloride were salty water leaking downward from the roadway above and/or salt spray from the roadway below.

The most serious aspect of deterioration in this structure was the corrosion of prestressing strands that was discovered at various locations at the beam soffits. While the prestressing strands were completely rusted along the soffit, no spalling

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(only longitudinal cracks) was observed. It was found that water had leaked into the interior of the box, penetrated through the concrete and reached the prestressing strands. In one case 450 gallons (2,000 L) of water was found inside a beam. Loss of strand due to rusting in these beams required immediate repair. Concrete near the rusted strands was removed, galvanized steel mesh was affixed to the chipped-out area and the repair completed with latex-modified shotcrete.

Modern joints with replaceable seals were installed that are expected to remain substantially watertight with regular maintenance. A new drainage system with open hoppers was also installed. This system is expected to remove the surface water away from the deck quickly and reliably, and thereby reduce the danger of salt-water leakage through the joints and the deck.

Case No. 2

An investigation of the collapse of a single-span, segmental post-tensioned concrete bridge was presented by Woodward (66). Ynys-y-Gwas bridge was built in 1953 over the River Afan in South Wales and collapsed in 1985.

The bridge consisted of a single span of 60 ft (18.3 m) composed of nine longitudinal I-beams, two box section edge beams, and two slabs 5-ft (1.52-m) wide that span the gaps between the I-beams and box edge beams. The beams were made up of eight segments, post-tensioned longitudinally with Freyssinet tendons consisting of twelve 0.2-in (5-mm) diameter wires. The transverse joints between the beams were packed with mortar. Transverse distribution of load was provided by diaphragms 5-in (127-mm) wide at one end of each segment and transversely post-tensioned using Freyssinet tendons.

Initial inspection showed that prestressing wires were installed in accordance with the original plans. It had been observed that the mortar cover over the transverse anchorages did not always fill the recess in the side of the beam, and where wires protruded, they were severely corroded. Following the initial inspection, some of the segments were taken to the laboratory. Tendons, grout, ducts, concrete, mortar, and longitudinal packing material were examined. Samples were taken for measurement of total chloride content in the concrete.

Longitudinal and transverse ducts were examined, most of these ducts were either fully grouted or contained only small air voids. Some of them, however, contained larger voids. Chloride analyses showed that only trace amounts of chloride were present in grout taken from ducts running within the segments, whereas much larger concentrations were found near the joints. There was no serious corrosion on prestressing steel within the segments; most wires were corroded where they crossed the joints. The conditions of wires at the sections that had failed varied from surface corrosion with no loss of sectional area to severe loss of section accompanied by heavy scaling and pitting. Concrete cores taken from the structure indicated that the concrete was in excellent condition, consisting of dense, high quality paste.

Chloride analyses of more than 100 samples taken from both I-beam and edge beam joints showed that chloride concentrations ranged from 0.06 to 2.04 percent by weight of cement. Chlorides were found over the whole concrete surface and in the grout.

The investigators concluded that the collapse of the bridge was due to corrosion of tendons where they crossed the joints. Failure occurred when the cross sectional area of the steel had been reduced to the point where it could no longer carry the imposed load. The primary cause of corrosion was the presence of chloride that had been introduced to the bridge through the use of deicing salts. Woodward stated that the findings of this investigation showed how vulnerable joints in segmental construction can be to salt attack, and this must have implications for modern-day segmental construction.

Following the failure of Ynys-y-Gwas Bridge, concern increased over voided areas left in ducts of grouted post-tensioned bridges which could promote corrosion of the tendons at these voided areas. On September 25, 1992 the U.K. Department of Transport issued a press release expressing these concerns as they impact the ability of current designs of post-tensioned bridges to resist deterioration and failure caused by corrosion (67). The release further states that the Department will not commission any new bridges of the grouted post-tensioned type until new standards are available. Work will continue on such bridges which are in advanced stages of design or currently under construction, however, additional quality control will be adopted during construction. It should be noted that the concern of the U.K. authorities most likely stems, in the main, from the above mentioned case. Although failure analysis indicated that corrosion had occurred in this case mainly where tendons crossed through mortar-packed joints (and not within the ducts themselves), this one failure has apparently caused considerable concern in the U.K. with the performance of all elements of post-tensioned structures.

Case No. 3

Long-term performance of a prestressed concrete bridge was presented by Isecke (21). This investigation was based on the evaluation of a demolished bridge. The demolition process of Schmargendorf bridge in Germany was carried out successively to allow rebuilding of a new bridge in parallel with the old one.

The post-tensioned prestressed concrete bridge was constructed in 1960 (68). The bridge utilized post-tensioned segmental construction with longitudinal post-tensioning of the segments and transverse post-tensioning of the deck. Numerous longitudinal and diagonal cracks were observed on the bridge 10 years after construction. Another major inspection conducted in 1978 showed more newly developed failures. It was determined that it would be more economical to demolish the bridge and build a new one, since repair procedures would be very costly.

Long-term performance of bridge materials as a function of environmental conditions was studied. Quality of concrete, prestressing steel, ducts, and grouting mortar were evaluated. Quality of concrete with regard to strength was good compared to the original concrete. Chloride contents in the superstructure elements varied as a function of position and concrete depth. The highest values were found near bridge deck areas and expansion joints.

The anchorages of the prestressing systems near expansion joints were not protected. They were destroyed by highly aggressive deicer solution flowing through expansion joints from the deck surface. Anchorage parts that were not subjected to chlorides were in good condition.

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Chloride contents in the grout were relatively low, except in the transverse ducts in the deck sections of the bridge. The high chloride concentration in the bridge deck concrete led to deterioration of the metal ducts allowing the chlorides to reach the grout. Fracture of prestressing wires in these areas was observed.

Only prestressing steel totally embedded in chloride-free grout showed no sign of corrosion. Poor grouting resulted in various amounts of corrosion, including local pitting as well as large corroded surface areas. The greatest amounts of corrosion attack occurred in partially grouted ducts in the interstices between fully grouted and exposed areas due to formation of macroscopic galvanic cells.

In ungrouted ducts, the reduction of area due to corrosion was very small. However, cracks with depths of 0.015 in (400 μm) were observed on these wires. Cracking was attributed to fatigue failure of the prestressing steel due to live traffic loads.

SUMMARY

The consequences of corrosion of prestressing steel in prestressed concrete are much more critical than that of ordinary steel used in plain reinforced concrete. Since tendons in prestressed concrete are subjected to high tensile strength, any small reduction in their cross section caused by corrosion may lead to break of these tendons.

Two types of corrosion are recognized: pitting and stress corrosion. While pitting corrosion is the same type of corrosion observed in ordinary steel, stress corrosion is associated only with prestressing steel. Stress corrosion cracking and stress corrosion testing of prestressing wires are widely covered in the literature. Most of these studies show that cold-drawn stress-relieved prestressing wire is less susceptible to stress corrosion and hydrogen embrittlement than quenched and tempered steel. Therefore, quenched and tempered steel is not permitted to be used in bridge construction. In concrete bridge structures, the main environmental condition that leads to corrosion of prestressing steel in both superstructure and substructure members of bridges is the presence of chlorides.

Laboratory and field investigations regarding the corrosion performance of prestressing steel in pretensioned concrete members demonstrate that good quality concrete and sufficient concrete cover will provide corrosion protection to embedded prestressing steel. However, this must be accompanied by adequate drainage systems and proper expansion joints.

Studies have led to the development of new grouts and improvement of grouting practices to provide better corrosion protection of tendons in post-tensioned concrete bridge members. Other protective measures such as the use of coated ducts can also afford improved protection to the tendons.

The anchorage zone in post-tensioned members is especially vulnerable to corrosion damage. Field performance, as well as laboratory studies, indicate that if anchorage recesses are not sealed and completely protected, moisture and chloride ions will penetrate to the prestressing wires and cause corrosion.

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Condition Assessment of Field Structures with Calcium Nitrite

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Synopsis: Calcium nitrite corrosion inhibitor has been commercially available in the United States since 1978. In that period of time over 200 parking structures, 100 marine structures, and over 230,000 cubic meters (300,000 cubic yards) of precast/prestressed bridge girders have been constructed with concrete containing calcium nitrite.

In this paper several of the oldest structures, along with several test sites were evaluated to determine the corrosion performance. The condition assessment included a visual evaluation of the structure, determination of chloride and nitrite contents in the concretes, and determination of the corrosion activity. The corrosion tests consisted of corrosion potential mapping and polarization resistance testing to determine the corrosion rates at the time of the evaluation. These assessment techniques are applicable to all steel reinforced concrete structures with or without some modifications.

The assessment showed that all of the structures with calcium nitrite are performing well. In two cases there is evidence that corrosion is in progress on adjacent structures that were not protected with calcium nitrite. The nitrite analyses document that calcium nitrite is stable in concrete and remains at the reinforcing bars. Diffusion of chloride is not increased in the concretes with calcium nitrite, and there is evidence of a reduction in chloride penetration in some cases.

Keywords: Bridge girders; calcium nitrite; chlorides; corrosion; harbor structures; parking facilities; structures; tests