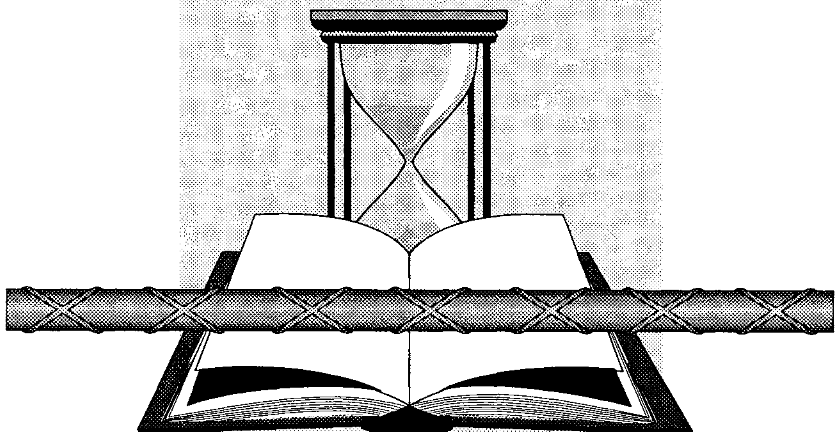
INTERIM REPORT

EPOXY-COATED REINFORCEMENT-
A HISTORICAL PERFORMANCE REVIEW



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Abstract

This report is a historical performance review of epoxy-coated reinforcement. The information in this report is presented in chronological order starting from the early 1970's, when the first bridge with epoxy-coated reinforcement was built, and ending with the presentation at the 1993 TRB sessions, where findings from the latest research investigations were revealed.

The report includes background information on fundamental applications of epoxy-coated reinforcing steel. Major laboratory studies, which addressed corrosion resistance and bond strength issues, are presented. Field investigations of meaningful structures with successful application of epoxy -coated reinforcing steel as well as structures with total failure of epoxy-coated reinforcement are also presented. The report also includes transcripted presentations, from the 1993 TRB sessions, which addressed quality control, adhesion characteristics, and other factors associated with long-term performance of epoxy-coated reinforcing steel.

Conclusions from the laboratory and field investigations, as well as from the presentations of 1993 TRB sessions are presented.

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(The opinions, findings, and conclusions expressed in this report are those of the authors and not necessarily those of the sponsoring agencies.)

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1. ABSTRACT

This report is a historical performance review of epoxy-coated reinforcement. The information in this report is presented in chronological order starting from the early 1970's, when the first bridge with epoxy-coated reinforcement was built, and ending with the presentations at the 1993 TRB sessions, where findings from the latest research investigations were revealed.

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Conclusions from the laboratory and field investigations, as well as from the presentations of 1993 TRB sessions are presented.

2. EXECUTIVE SUMMARY

This report is divided into four major parts: "Introduction", "Laboratory Performance ", "Field Performance", and "1993 TRB Presentations".

The "Introduction" presents background information on usage of the epoxy-coated reinforcement from the beginning of the 1970's to date.

The "Laboratory Performance" presents laboratory research findings. The results from up to three years of exposure tests indicate that epoxy-coated reinforcement is a good protection system against reinforcing steel corrosion. Based on one long-term study (seven years of exposure) it has been noted that degradation of coating's properties is associated with a time-chloride concentration relationship. Bond strength studies indicate that development length of epoxy-coated bars must be 15 % greater than the development length of uncoated reinforcement.

The "Field Performance" presents investigations of structures constructed with epoxy-coated reinforcement in chloride corrosive environments. Findings of these investigations are non-concurrent. Research findings of both good and poor performances of epoxy-coated reinforcement are presented. Good performances were reported from the investigations performed in Virginia, Maryland, Pennsylvania, New York, Wisconsin, Ohio, Minnesota, and Ontario, Canada (36, 47-50, 53-56, 58). In most of the examined structures there was no sign of corrosion of the (epoxy-coated) reinforcing steel, or corrosion was found to be negligible. When found, corrosion of the epoxy-coated reinforcing steel was attributed to coating damage or existence of cracks, which extended to the top mat of reinforcement. Age of the structures, at the time of the investigations, varied from five to 16 years.

A total failure of epoxy-coated reinforcing steel, as a protective method against corrosion, was found in the Florida Keys bridges (27, 51, 52, 57). A study of 18 structures selected from the Northern US and Canada and being in service for up to 18 years also showed poor performance of epoxy-coated reinforcing steel (59). These two investigations showed that epoxy-coated reinforcing steel will not provide long time protection against corrosion in severe environments: sea water in Florida and frequent deicing salt applications in Northern US and Canada. Florida investigations showed that application of epoxy-coated reinforcing steel will reduce the time to develop active corrosion by about five to six years in comparison with bare steel, while C-SHRP study showed that epoxy-coated reinforcing steel will extend the life of a structure for an average of five years (51, 52, 59).

Corrosion potential measurements of decks with epoxy coated bars were found, according to two study programs, to be inadequate and misleading, and no correlation was observed between half-cell readings and visual reinforcing bar ratings (54, 55, 56).

The " 1993 TRB Presentations" presents results of the latest investigations related to the application of epoxy-coated reinforcing steel as a protective method against corrosion of reinforcing steel. The main conclusion gleaned from these presentations is that the epoxy-coated reinforcing steel by itself cannot provide 50 years of protection against corrosion in severe environments. Failure of this protection system was attributed to degradation of the adhesion of the coating to the reinforcing steel and too high of a percentage defects in the coating film. Quality control, both in the plants and in the job sites, and application of multi-barrier systems was found to be necessary for the construction of more durable structures.

3. INTRODUCTION

During the 1960's most of the state highway agencies introduced a "bare road policy" which resulted in a significant increase in deicing salt usage (1). The amount of deicer salts applied on United States highways in 1970 was approximately four times more than in 1960. After 1970, roughly 10 million tons of salts were applied during every winter (2). Because of winter maintenance salting, a large number of concrete bridge decks were chloride contaminated and damage of reinforced concrete decks was greatly accelerated.

In the early 1970's, Peterson reported that most bridge decks in Pennsylvania, like many states, were exhibiting significant deterioration associated with extensive use of de-icing salts, after only seven to 10 years of service (3). The necessity of improving the corrosion resistance of reinforced concrete bridge decks granted rapid application of new concepts and materials for use on new and existing bridge decks.

The first large scale program to evaluate the applicability of different coatings for reinforcement protection was performed at the National Bureau of Standards in the early 1970's (4,5,6). A total of 47 different coatings, of which 21 were liquid and 15 were powder epoxies, was subjected to various performance tests. The tests included evaluation of chemical resistance, physical durabilities, and film integrity of cured coatings. Electrochemical measurements and bond tests of coated reinforcing bars were also performed. Findings of this study indicate that the best corrosion resistant coatings were epoxy and polyvinyl chloride. However, reinforcing bars with polyvinyl chloride coatings had unacceptable bond and creep characteristics when embedded in concrete. Powdered epoxies provided a more uniform and holiday free film than the liquid epoxies and the optimum film thickness ranged from 127 to 229 microns (5 to 9 mils [0.005 to 0.009 in]). Four powder epoxy coatings were recommended for future investigations.

In 1973, the first bridge with epoxy-coated reinforcement was built in West Conshohocken, Pennsylvania (7). Four spans of this fifteen-span-long bridge were constructed with epoxy-coated reinforcing steel, remaining spans with conventional black steel.

During the 1974 construction season, from a total deck area of 975,764 m^2 (1,167,000 square yards) in Pennsylvania, 21 % was constructed with epoxy-coated reinforcing steel, 40% galvanized reinforcement, and 39% latex modified concrete (3).

In 1976, use of epoxy-coated reinforcing steel was specified as one of "recent design changes" by the Michigan Department of State Highways and Transportation (8). By 1982, epoxy-coated reinforcing steel was specified by highway agencies in more than 40 states, and nationwide usage exceeded 90,720 metric tons (100,000 tons) (9).

The use of epoxy-coated reinforcing steel continued to grow. In 1987 at least 41 state transportation departments were using epoxy-coated bars for conventional structure concrete decks built without overlays (10). Bridge decks protected by epoxy coating in both top and bottom mats were found to cost only 4% more than decks with only top mat coats. There was still insufficient data on the corrosion protection performance of epoxy-coated reinforcing steel from field evaluations because most installations were less than 10 years of age. However, a comparison of different protective systems used to prevent the corrosion deterioration of concrete structures revealed that in 41 states epoxy coating was the most popular method for reinforcement protection (11). Lifetime costs, based on 50 years of service and with 1986 prices, showed that epoxy-coated reinforcement (top mat only or both mats) were cheaper than other protection systems: interlayer membrane, latex-modified concrete (LMC) overlay, or low slump dense concrete (LSDC) overlay (11). A cover thickness to 89 mm (3.5 in) was the only corrosion protection system with a lower life-cycle cost than epoxy-coated reinforcing steel.

In 1988, the first field corrosion protection failures of epoxy-coated reinforcing steel were reported (12). The importance of quality control and quality assurance for epoxy-coated reinforcement was addressed by Read (12). Two corrosion protection failures of epoxy-coated reinforcing steel applications were analyzed: one in the Florida Keys, the other in the Middle East. In the Florida Keys, it was pointed out that the ASTM A775 adhesion bend test was inadequate in comparison with current practice. In both cases poor surface (reinforcing steel) preparation as well as a lack in quality control and monitoring construction practices were pointed out.

By 1989, the use of epoxy-coated reinforcing steel had grown to 235,870 metric tons (260,000 tons) in the United States (13). There were 17 coating applicator firms with 34 plants in the United States and Canada. DeVekey also reported that organic resin-based (e.g. epoxy) coatings have been gaining acceptance for use with reinforcement in Great Britain, but a standard specification had not yet been developed (14).

This state-of-the-practice report addresses the laboratory and field performance of coated reinforcing steel. The findings presented in this report are in chronological order. A summary of the conclusions of both the laboratory and field performance investigations is also presented.

4. LABORATORY PERFORMANCE

Beyond the initial reinforcing steel coating development study, laboratory assessment of the long-term corrosion protection performance of coated reinforcement was almost nonexistent in the United States. No basic research was performed on the identification of corrosion protection mechanisms being provided by coated bar, as the nation wholeheartedly embraced the use of coated reinforcing steel during the late 1970's and early 1980's. Production line field installations by default became the research laboratory. Until the Florida Department of Transportation identified early corrosion protection failures of epoxy-coated reinforcement in bridge substructure elements (piers, columns, and cross-ties) in the Keys in 1988, the industry whole heartedly accepted epoxy-coated reinforcing steel as the primary single corrosion protection method.

Countries outside of North America were not as quick to accept epoxy-coated reinforcing steels as a corrosion protection system. Laboratory investigations were initiated to assess the corrosion protection effectiveness of epoxy-coated reinforcing steel. The following presents the results of laboratory investigations conducted both within and outside of the United States.

4.1 Corrosion Resistance

During 1976, Cork conducted a one-year outdoor exposure test, with coated and uncoated reinforcement in London, UK (15). Bars of 10 and 20 mm (0.4 and 0.8 in) diameter were brush painted with a protective coating and stored outdoors together with uncoated bars. After 12 months, the uncoated bars were covered with corrosion products whereas the coated bars were unaffected. In other tests, 20 mm (0.8 in) bars were embedded in 70 mm (2.75 in) concrete cubes and exposed to 1 % sodium chloride solution. Corrosion was accelerated with a 1.5 V direct current (DC) source. After a few weeks control cubes cracked, but none of the cubes with coated bars were damaged. It has also been found that bond strength between coated bars and concrete was unaffected by the coating.

In 1980 Virmani, Clear, and Pasko investigated the corrosion protection performance of two methods, epoxy-coated reinforcing steel and the corrosion inhibiting admixture calcium nitrite (16). Thirty-one slabs were fabricated with either non-specification epoxy-coated reinforcing steel or calcium nitrite admixture with black (uncoated) reinforcing steel (16). Epoxy-coated reinforcing steel performance was compared to uncoated steel without corrosion inhibiting admixtures. To accelerate the corrosion process, a moderately permeable concrete, w/c = 0.53, was used and some slabs, top mat only, were cast with chloride-contaminated concrete, 8.9 kg /M^3 (15 lbs/yd^3). Of the 31 slabs, 12 were fabricated with epoxy-coated bars, top mat only and top and bottom mats. Reinforcing steel used in this study was coated in 1977 and stored outdoors for over two years. The epoxy-coated bars had more than 82 holidays/meter (25 holidays/foot) (those which had lower number of holidays per foot were not used) and did not pass the bend test. They also did not meet the AASHTO and ASTM specifications. All slabs were stored outdoors in the Washington, DC area. The study also included an investigation of 17 bridge decks with epoxy-coated reinforcement in Kentucky and

Virginia. Field data showed that there was an electrical connection between top and bottom mats in some decks. Laboratory tests indicated that in case of electrical continuity between mats, decks with coated reinforcement top mat only, would require 12 times more time to consume the same amount of iron as concrete decks constructed with bare bars. In cases where both mats are constructed with coated reinforcing steel versus bare steel, the proportion is 46 to 1. The relationships indicate that epoxy-coated reinforcing steel could provide more than an order of magnitude reduction in the corrosion rate. It has also been found that corrosion rate was reduced due to a decreased oxygen reduction, small cathode areas on coated bar.

Kobayashi and Takewaka conducted a study on bare, epoxy-coated, and galvanized reinforcement from 1980-1983 (17). Variables included two types of epoxy coatings and three coating thicknesses: 100, 200, and 300 microns (0.004, 0.008, and 0.012 in). Small scale concrete specimens (10 x 10 x 110 cm [3.94 x 3.94 x 43.31 in]) were exposed to a marine splash zone environment. Findings included:

- thickness of the coating has a significant influence on mechanical and corrosion protection properties.
- macroscopic defects in epoxy coatings depend on quality of the surface preparation of the substrate material (reinforcing bar).
- bond strength of epoxy-coated bars is equal to about 80% of the bond strength of uncoated bars.
- damage to the coating films was not observed under fatigue loading.
- initially cracked beams with bare reinforcement developed large longitudinal cracks in the marine splash zone area.
- practically no reinforcement corrosion was observed in specimens with coating thickness of 200 microns (0.008 in).

In a two-year laboratory experiment conducted by Satake, Kamakura, Shirakawa, Mikami, and Swamy from 1981-1983, epoxy-coated reinforcing steel was compared with galvanized and plain uncoated bars (18). Variables included three coating thicknesses of 4, 8, and 12 mils (0.004, 0.008, and 0.012 in), and concrete cover depths of 2, 4, and 7 cm (0.79, 1.57, and 2.76 in). All specimens were precracked and subjected to a constant stress of 2000 kg/cm^2 (28.4 ksi) in the reinforcing steel. Specimens were then subjected to two corrosion tests: accelerated corrosion test (immersion in sea water at 60 ^{0}C (140^0 F) for six hours and then drying in the atmosphere for six hours), and an exposure test in the tidal zone in Kashima Harbor. After 24 months of accelerated corrosion testing, bars with epoxy coating of 200 to 300 microns (8 to 12 mils [0.008 to 0.012 in]) showed signs of corrosion and all coating properties were maintained even with the 20 mm (0.79 in) cover depth. Some blistering occurred on the 100 microns (4 mil [0.004 in]) coated specimens. Extensive corrosion products were present on all uncoated bars, even with a 70 mm (2.76 in) cover.

Pfeifer, Landgren, and Zoob evaluated 11 different corrosion protection systems during a three-year (1983-1986) laboratory study (19). A total of 124 small scale slabs were exposed to 48-weeks of wetting and drying cycles with salt water. Also 19 full scale slabs were exposed to salt water cycling for a period of one year. In addition to epoxy-coated reinforcing steel and prestressing strand, the study variables included cover depth and w/c ratio. Test results showed that no corrosion developed on any of the specimens constructed with epoxy-coated bars or prestressing strands, even when the chloride ion concentration in the vicinity of bars was up to 20 times greater than the corrosion threshold value. It was also reported that premarked holiday areas did not corrode.

In a five-year experimental program, carried out by Treadway and Davies from 1983-1988, black steel and two reinforcement protection systems, galvanized steel and epoxy-coated steel were compared (20). Specimens, concrete prisms, used in the study varied by type of reinforcement, chloride content, and cover depth. The corrosion of the specimens after five years of natural exposure was examined by visual, electrochemical, and destructive methods. Results of the study indicate that although total protection of reinforcing steel was not provided by epoxy coatings, a significant reduction in the rate of deterioration of specimens containing high levels of chloride was achieved. It was also observed that the corrosion process of epoxy-coated reinforcing steel is controlled cathodically.

Transport and Road Research Laboratory, UK, conducted a research study in the applicability of ASTM standards for British use (21). Two types of bars and two different coatings were included in the study. Some of the conclusions were as follows:

- a tolerance of only 10% in coating thickness should be allowed (versus ASTM range of 130 to 300 microns [0.005 to 0.012 in]).

- ASTM 775 M 8 + bend test (to 120^0 angle at $20\text{-}30^0$ C [$68\text{-}86^0$ F]) was found to be unrealistic (British Standards state that reinforcement should withstand bending to three diameters radii, 180° angle, and temperature range should be between five and 20°C [41 and 68^0 F]).

- test of chloride permeability on a detached film of 130 microns (0.005 in) thickness and abrasion test of 250 microns (0.010 in) thick coating on a steel plate were found to be of a doubtful value.

- coating continuity, chemical resistance, impact, and hardness tests were applicable.

- in general, use of ASTM 775 M standard in the UK was found to be impractical.

After the reported field failures of epoxy-coated reinforcing steel in the Florida Keys in 1988, Romano developed a method of quantifying disbondment of fusion bonded epoxy coating (22). Test specimens consisted of four epoxy-coated No. 10 bars, 30.5 cm (12 in) long. Epoxy patching compound was used to coat holidays and handling marks typically found on coated bars. Then artificial holidays were made on 6.5 % of the area of the bar. Specimens were tested in three exposure conditions which simulated marine, fresh water, and saline environments. All bars were immersed half way (15.2 cm [6 in]) in the solutions for a period of 30 days. Galvanic corrosion of the steel substrate was found to be the main cause of coating disbondment.

Perenchio, Fraczek, and Pfeifer conducted a one-year (1988) accelerated study on corrosion protection of prestressed systems for concrete bridges (23). Results showed that epoxies used for coating prestressing strands, steel ducts, anchorages, and associated hardware provided excellent corrosion protection. The authors recommended the use of epoxy-coated prestressing strand for maximum corrosion protection in both pretensioning and posttensioning methods.

A Finnish study of one powdered epoxy coating and three liquid epoxy paints in 1988 demonstrated that the powder-epoxy coating and coal tar epoxy paint had good corrosion protection properties in aggressive environments, after two years of exposure (24). Some of the specimens used in this study were constructed with an addition of 4% calcium chloride and some were initially cracked. It was also found that none of the coatings, after two-year exposure to tap and/or synthetic sea water, was totally impermeable.

Sagues (25, 26, 27) reported in 1989-1990 that corrosion damage to epoxy-coated reinforcing steel takes the form of extended metal loss with additional metal pitting taking place in metal-coating crevices in which low pH water accumulates. It was also observed that coating disbondment was not limited to the areas of high metal loss. In addition disbondment at the metal-coating interface before embedment in concrete can result from fabrication bending or from construction site exposure to sea water (water containing sodium chloride).

Sagues and Zayed (25, 26) reported in 1989 that the corrosion rate of bent epoxy-coated bars was an order of magnitude lower than that of black bars. The study analyzed the effects of fabrication and service conditions on the corrosion of epoxy-coated reinforcing steel in concrete. Thirty-seven specimens were partially exposed to a 5 % sodium chloride solution for 300 days to simulate exposure conditions of Florida's substructure bridge elements (piers, etc.). Test variables included epoxy-coated reinforcing steel suppliers, bending, surface distress, use of field patching compounds, presence of cracks in the concrete, and bending of the bar prior to coating application. Corrosion assessment methods included open circuitpotential, AC impedance measurements, and visual observations. Corrosion was observed in the areas where epoxy coating lost its adhesion due to fabrication bending.

A tentative model representing the first attempt to obtain quantitative corrosion rate information in a system consisting of partially debonded epoxy-coated reinforcing steel was proposed by Sagues and Zayed in 1989 (28).

Durability of concrete structures and the characteristics of epoxy-coated reinforcement was presented by Swamy in 1990 (29). The author found reinforcing steel coating as "decidedly the most effective method of ensuring corrosion-free life of steel reinforcement in concrete". He also pointed out that care should be taken in predicting real life behavior of epoxy-coated reinforcement based on laboratory test results with specimens having shallow cover depths.

Sohanghpurwala and Clear conducted an extensive laboratory testing program of the corrosion protection performance characteristics of epoxy-coated reinforcement (30, 3 1). Forty small scale slabs reinforced with epoxy-coated and/or black steel were exposed to 47 and/or 70 accelerated Southern Exposure Cycles. Study variables included seven different suppliers, bend diameter, coating thickness, bar fabrication, rate of bending, temperature of steel during bending, and patching of damaged areas before installation into concrete slabs. Test results indicated that specimens with epoxy-coated bars performed significantly better than specimens with black steel. Slabs with black steel were cracked and had rust stains. No cracks or rust stains were found on epoxy-coated reinforcing steel specimens. The macrocell currents on the bare bar slabs were found to be more than an order of magnitude higher than on slabs with epoxy-coated bars. However, about 20% of epoxy-coated specimens, most of which had visible damage prior to placement in slabs, showed corrosion rates equal to about one-half of the corrosion rates measured in the bare bar specimens. Electrical resistance data indicated that the coating was deteriorating with time. The effect of the various coating parameters on corrosion protection of the epoxy-coating was not distinguished during this study.

Scannell and Clear reported that "straight specification epoxy-coated reinforcing steels are many times more resistant to corrosion induced damage than uncoated bars when embedded in salt contaminated concrete and coupled to coated or uncoated bars in salt free concrete" after an outdoor exposure for a period of over 6.5 years in a northern United States environment (32, 33). After 3.1 years of exposure to salt ponding cycles of 3 % sodium chloride solution, chloride concentration at bar level exceeded 5.93 kg /m^3 (10 lb/yd^3), and salting cycles were discontinued. Only straight bars were evaluated in this study, and specimens varied by type of reinforcement: both mats coated, only top mat coated, and both mats uncoated. Best performance was achieved by specimens with both mats epoxy-coated.

Coating disbondment, which was found to be one of the main causes of deterioration of substructures of Florida segmental bridges, was evaluated in laboratory experiments conducted by Sagues and Powers (34). Test specimens were 30 cm (11.81 in) long, epoxy-coated bars, which were then immersed in three different solutions: calcium hydroxide, sodium chloride, and calcium hydroxide and sodium chloride. Part of the bars' coating was intentionally removed, approximately 0.25 %. Delamination of the coating was observed after exposure to a 3.5 % sodium chloride solution. The deterioration observed in the Florida Keys bridges was explained as a result of the combination of severe weathering environment prior to construction, damage due to handling and fabrication, and natural tendency for development of corrosion macrocells in concrete structures in subtropical marine environments.

Zayed and Sagues investigated the corrosion of manufactured epoxy-coated reinforcing steel with intentional surface damage in a naturally aerated 3.5 % sodium chloride solution for a period of

120 days (35). Straight and moderately bent bars were used in these tests. Corrosion potential measurements, electrochemical impedance spectroscope, and metallographical observations were used during the study. Findings indicated that straight and moderately bent epoxy-coated reinforcing bars, damaged with short scratches, behaved similarly. A mathematical model for estimation of a mass of oxidized metal was presented, and calculations were in a reasonable agreement with metallographic examinations at the end of the test.

Findings of Scannell and Clear (32) were confirmed after 8.5 years of testing. The only slabs which did not crack were the slabs with epoxy-coated reinforcing steel in top mat only and both mats. It was found that the maximum macrocell corrosion density during 8.5 years of exposure was only 0. 6 % for both mats coated, and 1.7 % for only top mat coated, of the corrosion density measured in specimens with bare bar mats.

Clear reported on 42 concrete slabs with straight and bent epoxy-coated reinforcing bars supplied by eight different bar coaters and subjected to Southern Exposure Cycling for a period of 1.35 years (36). It has been found that uncoated bars corroded and cracked the concrete, but no corrosion and concrete damage was found on specimens with epoxy-coated bars, though negligible or minor corrosion on some of the bars was detected. It was observed that straight bars performed slightly better than the bent bars. After Southern Exposure Cycles some of the slabs (from seven suppliers) were exposed to continuous tap water for a period of 10.5 months, and then to natural weathering for additional 9.5 months. It has been found, that during continuous ponding, bars from five suppliers performed poorly, while bars from two other suppliers performed well. Unfortunately, the corrosion protection characteristics of the superior corrosion resistant coated bars were not identified. Statistical analyses indicated that the following variables did not have significant effects on performance: coating thickness, bent bars coated before or after fabrication, fast versus slow bends, high temperature versus room temperature bends, and whether the coated bar was a straight or a bent one. Unfortunately nothing has been mentioned about bend tests in temperatures lower than 20^0 C ($68\ ^0$F). The only variable which has a significant effect on performance was the source of coated reinforcing steel. Test results were also compared to results from specimens with epoxy-coated reinforcement which were made concurrently with the above mentioned slabs. These additional specimens were subjected only to outdoor natural weathering for a period of three years, but no progressive deterioration was observed. It was concluded that the deterioration of epoxy-coated reinforcing steel probably resulted from a continuously wet environment.

Another study was performed by Sagues in 1991 to better explain the causes of reinforcing steel coating failure in Florida bridges (37). Laboratory tests showed that bars with mechanically damaged coating may develop corrosion in time on the order of one decade, if placed in conditions where macro-cell corrosion development is possible. Surface damage of the coating was found to be one of the main causes of disbondment. The surface damage was related to initial coating imperfections, shipping, fabrication, site exposure to salt water and UV light, handling, assembly procedures, positioning in concrete forms, concrete placement, and vibration. During this study, eight commercial epoxy-coated bars were examined. Under exposure to sodium chloride solution considerable product-to-product variability in the amount of disbondment was observed. Laboratory columns build with epoxy-coated reinforcing steel containing about 2% surface defects, were found to develop corrosion macrocell currents.

An Australian study showed that epoxy coating provided excellent protection to the steel as long as the coating was not damaged (38). Corrosion performance and pullout strength tests were performed with epoxy-coated and hot-dip galvanized reinforcing steel, and compared to results obtained for bare reinforcement. It has been found that ultimate strength of epoxy-coated reinforcing steel was about 17% lower than that of black steel. Another observation was that corrosion of the exposed ends of bars, regardless of the repair (touch-up), occurred freely and progressed along the bar under the coating.

An evaluation of the corrosion resistance characteristics was performed for bare mild steel, epoxy-coated steel, galvanized, and stainless clad reinforcing bars (39). Conclusions were based on a seven-year site exposure program, in which the chloride content of the fresh concrete varied. Chloride contamination levels used in the study were 2.37, 4.75 or 18.98 kg/m^3 (4, 8, or 32 lb/yd^3). It was found that specimens with epoxy-coated bars performed exceedingly well, in terms of corrosion protection, when chloride content was 2.37 or 4.75 kg /m^3 (4 or 8 lb/yd^3). However, significant corrosion was advancing under the epoxy-coating in specimens with a chloride content of 18.98 kg/m^3 (32 lb/yd^3). In the last case, coating breakdown and cracking of concrete occurred. Results of this extensive study demonstrated that epoxy-coatings have a finite tolerance limit of chloride concentration, after which deterioration initiates and progresses. Only specimens with stainless clad reinforcing steel showed no corrosion regardless of chloride contamination level.

4.2 Bond Strength

The flexural bond performance characteristics of epoxy-coated reinforcing steel wasevaluated and compared to mill scale and blast-cleaned steel surfaces (40). The influence of the epoxy coating on crack width and spacing was also evaluated. A total of 40 specimens, with No. 6 and No. 11 bars, were subjected to static and fatigue loadings. The results were:

- bond strength based on critical slip for the mill scale bars averaged 32% greater than for epoxy-coated bars.

- for specimens and loadings representative of concrete bridge deck slabs, no difference was found in terms of crack spacing for the short span specimens.

- the mill scale bars had 17 % greater flexural bond pullout strength than the epoxy-coated bars.

- under bond fatigue loading in the working stress range, the slip behavior of the mill scale, epoxy-coated and blast cleaned bars is essentially similar.

Based on these results, it was recommended that, to provide comparable performance with mill scale bars, a basic development length modification factor of 1. 15 should be used for epoxy-coated reinforcement.

Patil reported on pullout strength tests of coated reinforcement (41). Nine different coating materials were evaluated. Test results indicated that the bond between coated steel and concrete was not broken by the polymer system.

The fatigue bond strength of epoxy-coated reinforcing steel has also been evaluated (42). The investigation included both fatigue bond-slip performance and static bond strength of epoxy-coated reinforcement. Test results demonstrated that the epoxy coating had no influence on the fatigue bond strength. Test results also showed that the static bond strength of epoxy-coated bars without cyclic loading was slightly lower than that of mill scale bars and confirmed the previous findings of the need to increase the development length of epoxy coating by 15 % above mill scale bars.

Effects of reinforcement corrosion and protective coatings on the strength of the bond between concrete and steel has also been considered (43). The 6-month study evaluated bond strength and corrosion rate of epoxy-coated bars as compared to uncoated bars. Five specimens were prepared, each with different protection methods; epoxy-coated concrete surface; epoxy-coated reinforcing steel; epoxy-coated concrete surface and reinforcing steel; untreated specimen; control specimen. The results demonstrated that specimens with epoxy-coated reinforcing steel had lower initial bond strength than the control specimen, however, it has been found that this system was very effective in corrosion prevention.

A minimum of 15 % increase in development lengths of epoxy-coated reinforcement was again confirmed in another study on bond strength of epoxy-coated reinforcement (44). Twenty-one beams were cast, loaded, and measurements taken in the constant moment area. Test parameters were bar diameter, concrete strength, casting position, and coating thickness. Results demonstrated that for the case when the cover depth is less than three bar diameters or bar spacing is less than six bar diameters, development length for epoxy-coated bars should be increased by 50 % of uncoated bars. In all other cases a 15 % increase in development length should be sufficient. It was also observed that the epoxy coating did not significantly affect deflections and cracking load. However, the epoxy coating caused the width and spacing of cracks to increase.

A study on bond strength and deflections of concrete elements reinforced with epoxy-coated steel found that the bond strength ratios varied between 82 and 95 % of the values typical for uncoated steel (45). The study also verified previous findings that specimens cast with epoxy-coated reinforcing steel had wider cracks than cracks which appeared on specimens with uncoated reinforcement.

Experiments comparing bond strength of epoxy-coated reinforcing bars with different coating thicknesses, bar diameters and deformation patterns have been conducted (46). Reduction in bond strength of the epoxy-coated bars was determined by comparing the results of coated and uncoated bars. The results showed that the bond strength of deformed bars with five to 12 mils thick coating was significantly reduced. A reduction in bond strength was observed to be higher for bars with larger diameters. Deformation pattern was found to have an influence on magnitude of the reduction in bond strength.

4.3 Conclusions

4.3.1 Corrosion Resistance

The main conclusions which may be drawn from laboratory studies are as follows:

- A vast majority of the laboratory investigations has demonstrated that epoxy coated reinforcement is a very suitable method for corrosion protection. However, these investigations were generally short-term projects, length of the investigations were generally less than three years.

- One long-term research project (seven years of exposure) has shown that significant corrosion under the coating has occurred when the added chloride content was 32 lb/cy (39). This resulted in coating breakdown and concrete cracking. For chloride contents of 2.37 and 4.75 kg /m^3 (4 lb/yd^3 and 8 lb/yd^3) no corrosion has been observed. This indicates that there is a pessimistic chloride content for epoxy-coated reinforcing steel which is time-chloride concentration related.

Other conclusions are as follows:

- Corrosion process of epoxy-coated reinforcement is controlled cathodically (20)

- Corrosion products were observed mainly in the areas where coating lost adhesion (25,26)

- Main cause of coating disbondment is believed to be galvanic corrosion and surface damage (22)

- Corrosion rates of bars with initial damage to the coatings may be as high as 50% of the corrosion rates of bare bars (30, 31)

- One of the main factors influencing performance of the coating is the coating applicator (36)

- Deterioration of the epoxy coating bond to steel may result from a continuously wet environment (36)

- Corrosion may initiate at the exposed ends of bars and then progress freely along the bar under the coating (38)

4.3.2 Bond Strength

All research on bond strength of epoxy-coated bars, performed to date, are rather consistent in regard to bond development length. To compensate lower bond strength, researches agree that bond development length of epoxy-coated bars must be 15 % greater than uncoated reinforcement. Some researches observed wider cracks in test specimens with epoxy-coated reinforcement than those with bare steel. Deformation pattern was found by some to have significant influence on bond strength.

5. FIELD PERFORMANCE

As highway agencies began to embrace epoxy-coated reinforcing steel as the sole corrosion protection system for new bridge decks in the 1970's, some agencies initiated corrosion protection performance studies. However, the number of studies was small and most of them were short-term studies. Not until after Florida reported corrosion protection failures of the Key bridges did interest in the long-term protection performance of epoxy-coated reinforcement rekindle. The results of investigations on the performance of epoxy-coated reinforcing steel are reported herein on a state by state basis, except where the investigation includes states and Canadian Provinces.

5.1 Virginia

In 1977, McKeel reported on an investigation of four bridge decks in Carroll County, VA, two decks constructed with epoxy-coated steel and two decks with bare steel (47). The epoxy coating was one of the original four prequalified coatings. Initial testing included resistivity, corrosion potentials, and chloride contents. McKeel also reported on the performance of the four decks after 10 years of service (48). Field evaluations included visual inspection, delamination survey, corrosion potentials, and chloride contents. A spall had occurred at the expansion joint of one of the control (bare steel) decks which appeared to be corrosion related. Chloride contents measured in cracks on the epoxy-coated steel decks greatly exceed the corrosion threshold level of 0.71 kg/m^3 (1.2 lb/yd^3). Maximum chloride content at bar level in a crack was 1.96 kg/m^3 (3.3 lb/yd^3). However, there was no evidence of active corrosion.

5.2 Maryland

Munjal assessed the corrosion protection performance of 11 bridge decks with epoxy-coated steel and one control deck (bare steel) from 1975-1979 (49). Electrical resistance and corrosion potentials were measured every year and chloride contents the last two years. After five years, epoxy-coated reinforcement's corrosion protection characteristics were satisfactory. However, it was stated that "the effectiveness of the epoxy coating on the reinforcing steel to prevent corrosion cannot be judged at this time".

5.3 Minnesota

Hagen reported on the performance of three bridge decks constructed with epoxy-coated steel (50). Two of the decks had been in service for four years and one for seven years. The decks were visually inspected and corrosion potentials, cover depth, and chloride content measurements were taken. A delamination survey was also performed. No evidence of significant corrosion deterioration was noted due to the short service life. However, some corrosion activity was found on one deck. This was attributed to small openings in the epoxy coating.

5.4 *Pennsylvania*

Weyers and Cady reported on the corrosion performance of 22 bridge decks (11 epoxy-coated and 11 bare reinforcing steel) in Pennsylvania in 1984 (53). The concrete bridge decks constructed with epoxy-coated reinforcing steel ranged in age from six to 10 years as did the decks with bare reinforcing steel. All the decks were visually inspected and an in-depth inspection was conducted on two epoxy-coated and two bare steel decks. The in-depth inspection consisted of a delamination survey, steel cover depth, and level of chloride ion contamination with depth. Visual inspections revealed that 40% of the bare steel decks were in the initial stage of corrosion-induced deterioration. None of the decks with epoxy-coated steel showed any visual signs of reinforcement corrosion. The in-depth study confirmed corrosion of bare steel, a much higher area of deterioration, but no signs of deterioration had been found on the epoxy-coated steel decks. Weyers and Cady concluded that "epoxy-coated reinforcing steel does provide a level of corrosion protection in the field against the deterioration of the concrete caused by corroding reinforcing steel which is in its initial deterioration stage." They recommended that the decks be re-evaluated in five years.

Malesheske, Maurer, Mellott, and Arellano reported on a study of 148 bridge deck corrosion protection systems, epoxy-coated reinforcing steel, galvanized steel, waterproofing membranes, latex-modified concrete overlays, and low-slump-dense concrete overlays (54,55). All the decks were visually inspected. Corrosion potentials, concrete permeability, and chloride ion concentration measurements were taken on 21 selected bridge decks, including four with epoxy-coated reinforcing steel. The average age of bridge decks with epoxy-coated reinforcing steel was 7.67 years. Coating thickness ranged from 107 to 447 microns (4.2 to 17.6 mils), with the average of 234 microns (9.2 mils). Seven out of 12 inspected bars were out of specification. Coating specifications at the time the decks were constructed was 178 $\pm$ 51 microns (7 $\pm$ 2 mils). Observations showed that despite high chloride content, from 1.96 to 6.94 kg/m^3 (3.3 to 11.7 lb/yd^3), most of epoxy-coated reinforcing bars were almost in a perfect condition. Among all investigated protective systems, epoxy-coated reinforcing steel was found to be the most effective one. It was also shown that corrosion potential measurements of the decks with epoxy-coated reinforcing steel were inadequate and misleading, and no correlation was observed between half-cell readings and visual bar rating. Authors recommended that corrosion potential measurements not be used for performance determination of bridge decks with coated reinforcement. Observation related to applicability of corrosion potential measurements was later confirmed in Canadian study by Hededahl and Manning (56).

5.5 *Florida*

In 1988 Kessler and Powers reported on the inspection of segmental bridge substructures in the Florida Keys (51,52). Three of the four bridges showed significant signs of corrosion of epoxy-coated reinforcing steel. The structure without corrosion damage had 10.2 cm (4 in) of cover. However, cracks were present and samples taken from this structure showed evidence of coating disbandment. Florida's experience indicates that epoxy-coated reinforcing steel, when used in a substructure application and exposed to marine environment, is more susceptible to corrosion than bare steel. This conclusion was based on experience that bare steel will develop active corrosion in about 12-15 years in marine environments, and for epoxy-coated reinforcement this time reduces to 7-9 years.

Powers had earlier reported on field observations of one of the four Key bridges, the Long Key Bridge (57). Observations from substructure areas showed that:

- most areas had chloride concentrations well above 0.71 kg/m^3 (1.2lb/yd^3)

- higher concentrations of chlorides were found in cracked areas than in uncracked concrete

- corrosion related spalling has been observed at 2.5 cm (1 in) of concrete cover after four years of service, and at 5.1 cm (2 in) of cover after seven years

- coating failure and corrosion was evident in some fabricated (bend) areas of the epoxy-coated reinforcing steel

Laboratory tests showed that fracture and disbondment of the coating occurred under standard bend test (about 90% of all tests) and during fabrication bending. "Ten samples of fabricated epoxy-coated reinforcing bars cast in concrete and exposed to salt water developed corrosion within six months. Corrosion was verified by electropotential monitoring during the exposure periods and by visual examination upon opening the samples" (57). The coating manufacturer reported that no significant deficiencies existed in the quality of the epoxy-coated reinforcing bars taken from the structure (57).

Field observations of the three deteriorating Florida bridges, the Long Key Bridge, the Seven Mile Bridge, and the Niles Channel Bridge, were later reported by Sagues, Powers, and Zayed (27).

Spalling of the concrete cover had occurred before 10 years of service. It was reported that the deterioration of these structures could not be considered as an isolated example because they were built at different times and epoxy-coated reinforcement was provided by different manufacturers. The only information on the reinforcing steel used in the projects was that the material satisfied AASHTO specifications at the time of construction. Reinforcing steel samples taken from locations where severe corrosion was not present showed that the coating had holidays and imperfections, but not in excess of those allowed by the specifications. The general pattern of corrosion distribution was found to be the same in all affected structures. Most of the concrete spalls were detected between 0.61 and 1.83 m (2 and 6 feet) above high tide. Samples taken from these locations showed that the chloride concentrations at the depth of reinforcement were in the range of 1.07 to 3.92 kg /m^3 (1.8 to 6.6 lb/yd^3).

Observations taken about 4.88 m (16 feet) above high tide level showed no corrosion, but the coating adherence to the reinforcing steel was very weak. Authors suggested that mechanical coating damage, weathering exposure prior to concrete casting, and macrocell action during service in severely corrosive environment were the primary factors causing corrosion of epoxy-coated reinforcing steel.

Another suggestion was that chlorides or other species might have been trapped between the disbonded coating and the metal from the start, thus shortening the corrosion initiation period significantly. Lack of information made it impossible to present a reliable model explaining the failure of epoxy coatings in these three bridges.

5.6 New York

Corrosion of epoxy-coated steel reinforcement in 14 bridges, seven to 12 years old was found to be insignificant according to a survey performed in New York State by Perregaux and Brewster (58). In this survey, only the worst "cases" were selected, e.g. only bridges with known surface distresses. It was found that protection provided by epoxy-coating appeared to be satisfactory. The finding was based on visual observations that undercutting of the coating, pitting of the steel, or section loss of the bar had not occurred. Despite the above mentioned satisfactory performance of epoxy-coated bars, the authors could not quantify long-term corrosion protection at the time the study was performed.

5.7 CRSI

Data gathered from a survey of 13 bridges built between 1974 and 1981, nine to 16 years old, showed that 87% of the cores taken of the top mat of epoxy-coated reinforcing bars were essentially corrosion free. The 13 % exhibiting significant corrosion were from areas where cracks extended to the depth of the coated reinforcing steel (36). This finding was for bridge decks located in Virginia, Wisconsin, Pennsylvania, New York, and Ohio. The performance survey included visual observations, delamination survey, and drilled cores. During this study, "no signs of progressive deterioration were found, extrapolation of the results into the future" was not possible.

5.8 Ontario

The corrosion deterioration state of two barrier walls with epoxy-coated reinforcing steel and one wall with bare steel was assessed by Hededahl and Manning after nine years of service (56). The structures were purposely chosen because of their exposure to frequent application of de-icing salts. Walls were cracked and had very shallow cover depths. Corrosion performance data included the following measurements: cover thickness, percent chlorides, corrosion potentials, rate of corrosion, continuity tests, condition of steel and coating (visual inspection). Hededahl and Manning found epoxy coating to be an effective method for reinforcement protection in chloride contaminated concrete. After nine years of exposure, shallow corrosion of epoxy-coated bars was found, but only in the areas where the coating was damaged. The barrier wall with bare steel experienced extensive and severe corrosion of its reinforcement. Although results after nine years of exposure were satisfactory, Hededahl and Manning said that "the effectiveness of epoxy coating on reinforcing steel over the life of the structure is still uncertain".

5.9 C-SHRP

The Canadian Strategic Highway Research Program study on the corrosion protection performance of epoxy-coated reinforcing steel was started in 1990 (59). The project included the evaluation of structures with epoxy-coated reinforcing steel exposed to environments typical for Canada. The study included testing of epoxy-coated reinforcing bars from 12 Canadian and US coaters, seven Canadian and US jobsites, and 19 field structures constructed in Canada and the Northern US between 1974 and 1988. In addition, a 6-month environmental exposure test of epoxy-coated reinforcing bars in Toronto was performed to simulate jobsite conditions. Some of the conclusions were as follows:

- "The study showed that epoxy-coated reinforcing steel will not provide long-term protection to reinforcement in salt-contaminated concrete. An unexpected failure mechanism involving progressive loss of adhesion and underfilm corrosion in the highway concrete environment has been identified as active in northern and southern field structures. "
- "Present and proposed specifications, even if tightly enforced and modified..., will not provide assurance of long-term performance in salt-contaminated concrete."
- "The (sufficient) data indicate that the extended life will be in the range of only one to eight years, ..., and will probably average about five years."
- "Therefore, it is recommended that epoxy-coated reinforcing steel should NOT be used as the primary protective system on highway structures which are expected to experience chloride contamination (in excess of 0.77 kg/m^3 [1.3 lbs/yd^3] at reinforcing steel level) six or more years before the end of their desired low-maintenance life." For a typical deck with 5.1-6.4 cm (2 - 2.5 in) cover and w/c ratio equal to 0.4-0.45, total service life will be about 15 to 20 years.
- "Structural bond and creep characteristics of concrete containing epoxy-coated reinforcing steel should receive high priority since about half of the coated bars from field structures exhibited reduced coating adhesion."
- "Jobsite exposure of epoxy-coated reinforcing steel resulted in a large increase in the number of bare areas."

5.10 Conclusions

Field investigations of the corrosion protection performance of epoxy-coated reinforcing steel have not reached any consensus to date. Some researchers found epoxy-coated reinforcing steel performing much better than the bare steel while others had reported just the opposite.

Satisfactory performance of epoxy-coated reinforcing steel was reported by the following researchers:

- McKeel (47, 48). No evidence of active corrosion was found in the two Virginia bridge decks, constructed with epoxy coated reinforcement, after 10 years of service.

- Munjal (49). Performance of 11 bridge decks in Maryland was found to be satisfactory after five years of service.

- Weyers and Cady (53). The evaluation included 22 Pennsylvania bridge decks (11 epoxy-coated and 11 bare reinforcing steel) which were from six to 10 years in service. After visual inspection and an in-depth study of two decks with epoxy-coated reinforcing steel and two decks with bare steel, no signs of corrosion were found on decks constructed with epoxy coated reinforcement, while more than 40% of the decks with bare steel were found to be in the initial stage of corrosion-induced deterioration.

- Malesheske et al (54, 55). Epoxy coated reinforcing steel was found to be the best protective system for concrete bridge decks, after inspection of 148 bridges in Pennsylvania. An in-depth study of 21 structures included four bridges with epoxy-coated reinforcement and an average 7.67 years of service. Despite high chloride content, from 1.96 to 6.94 kg $/m^3$ (3.3 to 11.7 lb/yd^3), most of the coated bars were in almost perfect condition.

- Perregaux and Brewster (58). Corrosion of epoxy coated reinforcing steel was found to be insignificant according to a study of 14, seven to 12 years old bridges.

- Study performed by CRSI (36). A survey of 13 bridges, nine to 16 years old, located inVirginia, Wisconsin, Pennsylvania, New York, and Ohio revealed that epoxy coated reinforcement had no signs of progressive deterioration. About 13 % of the top mat epoxy-coated bars, all of which were in the areas where cracks extended to the depth of the reinforcement, were exhibiting significant corrosion. The remaining 87% were essentially corrosion free.

- Hededahl and Manning (56). Two barrier walls, one with epoxy coated and one with baresteel, were compared after nine years of service in a northern environment (Ontario, Canada). Shallow corrosion of epoxy coated reinforcing steel was found only in the areas where the coating was damaged. Severe and extensive corrosion was found on the bare steel.

- Hagen (50). No evidence of significant corrosion, in the epoxy coated reinforcement of three bridge decks, was found. Some corrosion activity, which was found on one of the bridge decks, was attributed to small openings in the epoxy coating.

The following researchers reported failure of epoxy coatings in protecting reinforcement from corrosion:

- Kessler and Powers (51, 52), Kessler (57), Sagues et al. (27). Significant corrosion of epoxy coated reinforcement was observed in the Florida Keys bridges in the areas between 0.61 and 1.83 m (2 and 6 feet) above high tide. Based on these observations it was suggested that the primary causes of the corrosion of epoxy coated reinforcing steel were as follows: mechanical coating damage, weathering exposure prior to concrete casting, and macrocell action during service (27).Another suggestion was that chlorides or other species might have been trapped between the disbonded coating and the metal from the start. Other findings of these studies were: epoxy-coated reinforcing steel used in substructure elements and exposed to marine environment is more susceptible to corrosion than the bare steel; bare steel will corrode in 12 to 15 years in a marine environment, while epoxy-coated reinforcing steel will corrode in about seven to nine years.

- C-SHRP (59). A survey of 19 field structures, built in Canada and Northern US between 1974 and 1988, showed that epoxy coatings cannot provide long-time protection to the reinforcing steel. It was also found that use of epoxy-coated reinforcing steel will extend the life of a structure for an average of only five years.

Data gathered during two independent study programs have shown that corrosion potential measurements of decks with epoxy coated reinforcing steel were inadequate and misleading (54, 55, 56). No correlation was observed between half-cell readings and visual bar rating.

6. 1993 TRANSPORTATION RESEARCH BOARD PRESENTATIONS, WASHINGTON, DC, JANUARY 1993 - CONCLUSIONS.

During the 1993 TRB presentations, devoted to the corrosion protection performance of the epoxy-coated steel in concrete, it was generally agreed that epoxy-coated reinforcing steel cannot provide long-term, 50 years or more, protection to the reinforcement in severe environments. The only way to build more durable structures is to use high(er) quality products (epoxy-coated bars) and to use multibarrier protection systems (Appendix A. 1, A. 3, A.4).

Some of the findings presented during sessions and associated with performance of epoxy-coated reinforcing steel were as follows:

- in most research programs and field evaluations, except the case of the Florida Keys bridges, epoxy-coated reinforcement performed better than the bare steel (A. 1, A. 2, A.5, A.10)

- typical field quality epoxy-coated reinforcing steel can prolong the life of a structure, in Northern US or Canadian environment, from three to six years, and eight to 11 years if one uses laboratory quality epoxy-coated bars (A. 1)

- microscopic cracks, found in the tensile areas of bent bars, could not be picked up by the standard 80,000 ohm holiday detector (A. 1)

- underfilm contamination may be in the range of 10-80 % on the bars from coaters, and 25-60 % on the bars from job sites (A. 1)

- variability of the 9 mil (film thickness) target was very high, range 3-20 mils with a standarddeviation almost equal to the mean, coefficient of variation approximately 100% (A.3)

- coating disbondment, in older structures, occurs regardless of chloride presence (A. 9)

- a reasonable correlation was found between the measured rate of corrosion on the bars, the chloride content of the concrete and the conductivity measured between exposed sections of the reinforcement (A.9)

- high electrical resistance properties of epoxy-coated reinforcing steel depends upon proper film thickness, good surface preparation, very low holiday counts, and proper repair to film defects (A.2)

- the only variable that had a significant effect on corrosion of epoxy-coated reinforcement was the source or coating applicator (A.2)

- damage of the coating caused by the concreting process may be up to 80% of the total surface damage which occurs after leaving the factory gates (A.10)

- performance of the coating was found to be greatly dependent on the oxygen permeability of the overlying concrete (Closing Panel Discussion, John Theopolis on work performed by Manchester University Corrosion Center, UK)

Two of the most important factors which were reported to have a dominant influence on the corrosion protection performance of epoxy-coated reinforcement are as follows (A. 1, A.9):

- the number of defects in the coating
- adhesion of the coating to the reinforcing bar

In summary, it was concluded that deterioration of the epoxy-coated reinforcing steel is mainly due to the following factors:

- thin film thickness (A.2)

- pinholes in the coating (A.2, A.10)

- underfilm contamination leading to adhesion failure (A. 1)

- damage of the coating caused during concrete placing operation (A. 10)

- unprotected bar ends (A. 10)

- patched areas (A.4, A.5)

During the presentation on quality control of epoxy-coated reinforcing steel, Mr. Theodore Neff stated that "a coating system... is only as good as its weakest point" (A.3). It is important then, that high quality epoxy-coated reinforcement will result in more durable structures. In order to achieve high quality epoxy-coated bars, the following procedures are recommended:

- test for level of backside contamination

- measure the profile or the texture of the surface (peak and valley tests)

- test for the presence of chlorides and soluble salts on the surface prior to the application of the coating

- measure film thickness

- ensure proper cure (temperature) and assure appropriate handling, storage, and placement of epoxy-coated bars

- repair in case of coating damage

There were two speakers who presented the European experiences and research programs related to the epoxy-coated steel (A.4, A. 10). One learned that the British standards and the German guidelines for epoxy-coated reinforcement are more demanding, from the quality point of view, than the ASTM. Bendability tests, for example, are performed for smaller radii. Also, amount of pinholes and maximum area of damages, allowed in the UK and in Germany, are about 50% of what is allowed by the ASTM standards. Also worth mentioning is the fact that patching of breakage areas in the coating, at building sites in Germany, was found to be "useless" (A.4).

The reflections from the 1993 TRB sessions are that new coatings with improved adhesion characteristics and a much smaller allowable pinhole specification needs to be developed. Another area for improvement is quality control, both in the plants and in the job sites. European and Japanese technologies need to be studied and implemented in the US if necessary. And from the researchers point of view, new rapid tests need to be developed to predict long term performance characteristics of epoxy-coated reinforcement in concrete.

7. REFERENCES

1. Bennett, J. 1986. "Corrosion of Reinforcing Steel in Concrete and its Prevention by Cathodic Protection." *Anti-Corrosion Methods and Materials,* Vol. 33, No. 11, p. 12-15, 17.

2. 1992. "Highway Deicing. Comparing Salt and Calcium Magnesium Acetate." *TRNews,* No. 163, pp. 17-19.

3. Peterson, P.C. 1977. "Concrete Bridge Deck Deterioration in Pennsylvania." *Chloride Corrosion of Steel in Concrete, ASTM STP 629.* Philadelphia, PA, pp. 61-88.

4. Clifton, J.R., H.F. Beeghly, F. Hugh, and R. G. Mathey. 1974. *Nonmetallic Coatings for Concrete Reinforcing Bars.* National Bureau of Standards, Report No. FHWA-RD-74-18, Washington, DC, Federal Highway Administration.

5. Clifton, J.R., H. F. Beeghly, F. Hugh, and R. G. Mathey. 1975. *Nonmetallic Coatings for Concrete Reinforcing Bars.* National Bureau of Standards, Building Science Series - 65, Washington, DC, Federal Highway Administration.

6. Clifton, J. R. 1976. "Protection of Reinforcing Bars with Organic Coatings." *Materials Performance, Vol.* 15, No. 5, pp. 14-17.

7. Kilareski, W. P. 1977. "Epoxy Coatings for Corrosion Protection of Reinforcement Steel." *Chloride Corrosion of Steel in Concrete,* ASTM STP 629, Philadelphia, PA, pp. 82-88.

8. Brown, M. G. 1976. "Control and Prevention of Deterioration of Concrete Bridge Decks." *Michigan Department of State Highways and Transportation Report No. FHWA-MI-1034,* Washington, DC, Federal Highway Administration.

9. 1983. "Research Pays Off. Rebar Tending: Quite an Art." *TRNews,* No. 108, p. 14.

10. Babei, K. and N. M. Hawkins. 1987. "Evaluation of Bridge Deck Protective Strategies." *National Cooperative Highway Research Program Report 297,* Washington, DC.

11. Babei, K. and N. M. Hawkins. 1988. "Evaluation of Bridge Deck Protective Strategies". *Concrete International,* p. 56-66.

12. Read, J. A. 1989. "FBECR. The Need For Correct Specification and Quality Control." *Concrete (London),* Vol. 23, No. 8, p. 23-27.

13. Gustafson, D. P. 1990. "Steel Reinforcement. "*ASTM Standardization News,* Vol. 18, No. 12, p. 38-42.

14. De Vekey, R. C. "The Durability of Steel in Masonry." *British Ceramic Transactions and*

15. Cork, H. A. 1977. "Coating Treatment for Reinforcing Steel." *Concrete (London),* Vol. 11, No. 1, p. 31-33.

16. Virmani, Y. P., K. C. Clear, and T. J. Pasko, Jr. 1983. "Time-to-Corrosion of Reinforcing Steel in Concrete Slabs, Volume 5: Calcium Nitrate Admixture or Epoxy-Coated Reinforcing Bars as Corrosion Protection Systems." *Report No. FHWA/RD83/012, Interim Report,* Washington, DC, Federal Highway Administration.

17. Kobayashi, K., and K. Takewaka. 1984. "Experimental Studies on Epoxy-Coated Reinforcing Steel for Corrosion Protection." *The Journal of Cement Composites and Lightweight Concrete,* Vol. 6, No. 2, p. 99-116.

18. Satake, J., M. Kamakura, K. Shirakawa, N. Mikami, and N. Swamy. 1983. "Long-Term Corrosion Resistance of Epoxy-Coated Reinforcing Bars." *Corrosion of Reinforcement in Concrete Construction,* Ellis Horwood Ltd., UK. Chapter 21.

19. Pfeifer, D. W., J. R. Langren, and A. Zoob. 1987. "Protective Systems for New Prestressed and Substructure Concrete." *Report No. FHWA/RD-86/193,* Washington, DC, Federal Highway Administration.

20. Treadway, K. W. J., and H. Davies. 1989. "Performance of Fusion-Bonded Epoxy-Coated Steel Reinforcement." *Structural Engineer,* Vol. 67, No. 6, p. 99-108.

21. Bishop, R. R. 1987. "The Specification of Epoxy-Coated Reinforcing Bars." *Report: Application Guide 6, Transport and Road Research Laboratory,* Department of Transport, Crowthorne, Berkshire, UK.

22. Romano, D. C. 1988. *Preliminary Investigation of Epoxy-Coated Reinforcing Steel Disbondment: Causes and Effects.* Florida Department of Transportation, Materials Office, Gainesville, FL.

23. Perenchio, W. F., J. Fraczek, and D. W. Pfeifer. 1989. "Corrosion Protection of Prestressing Systems in Concrete Bridges." *Report 313,* National Cooperative Highway Research Program, Washington, DC, 25 p.

24. Salparanta, L. 1988. "Corrosion Prevention of Concrete Reinforcement by Epoxy Coating." *Nordic Concrete Research,* No. 7, p. 250-258.

25. Sagues, A. A., and A. M. Zayed. 1989. "Corrosion of Epoxy-Coated Reinforcing Steel in Concrete - Phase 1. " *Report No. FL/DOT/SM089-419,* Florida Department of Transportation, Materials Office, Gainesville, FL.

26. Sagues, A. A., and A. M. Zayed. 1989. "Corrosion of Epoxy Reinforcing Steel in Concrete." Paper No. 379, *Corrosion 89,* New Orleans Convention Center, New Orleans, LA.

27. Sagues, A. A., R. Powers, and A. M. Zayed. 1990. "Marine Environment Corrosion of Epoxy-Coated Reinforcing Steel." *Corrosion of Reinforcement in Concrete, by* C. Page, K. Treadway, P. Bramforth, Elsevier Applied Science, London-New York, p. 539-549.

28. Sagues, A. A., and A. M. Zayed. 1989. "Evaluation of Corrosion Rate by Electrochemical Impedance in a System with Multiple Polarization Effects." Paper No. *25, Corrosion 89,* New Orleans Convention Center, New Orleans, LA.

29. Swamy, R. N. 1990. "Durability of Rebars in Concrete." *Durability of Concrete. G.*M. Idorn International Symposium, 1990 Annual ACI Convention, Toronto, Ontario, Canada, Report No. SP-131, pp. 67-98.

30. Sohanghpurwala, A. A., and K. C. Clear. 1990. "Effectiveness of Epoxy Coatings in Minimizing Corrosion of Reinforcing Steel in Concrete." Paper No. 89, *Transportation Research Board,* 69th Annual Meeting, Washington, DC.

31. Sohanghpurwala, A. A., and K. C. Clear. 1990. "Effectiveness of Epoxy Coatings in Minimizing Corrosion of Reinforcing Steel in Concrete." *Transportation Research Record,* No. 1268, pp. 193-204.

32. Scannell, W. T., and K. C. Clear. 1990. "Long-Term Outdoor Exposure Evaluation of Concrete Slabs Containing Epoxy-Coated Reinforcing Steel." *Transportation Research Record,* No. 1284, pp. 70-78.

33. Scannell, W. T., and K. C. Clear. 1990. "Long-Term Outdoor Exposure Evaluation of Concrete Slabs Containing Epoxy-Coated Reinforcing Steel." Preprint Paper No. 89-0431 . *Transportation Research Board,* 69th Annual Meeting, Washington, DC.

34. Sagues, A. A., and R. G. Powers. 1990. "Effect of Concrete Environment on the Corrosion Performance of Epoxy-Coated Reinforcing Steel." Paper No. 311, *Corrosion 90,* Las Vegas, NV.

35. Zayed, A. M., and A. A. Sagues. 1990. "Corrosion at Surface Damage on an Epoxy-Coated Reinforcing Steel." *Corrosion Science,* Vol. 30, No. 10, pp. 1025-1044.

36. Concrete Reinforcing Steel Institute. "CRSI Performance Research: Epoxy-Coated Reinforcing Steel." Interim Report, January, 1992.

37. Sagues, A. A. 1991. "Mechanism of corrosion of Epoxy-Coated Reinforcing Steel in Concrete." *Report No. FL/DOT/RMC/0543-3296.*

38. Yeomans, S. R. 1991. "Comparative Studies of Galvanized and Epoxy-Coated Steel Reinforcement in Concrete." *Durability of Concrete - Second International Conference.* Montreal, Canada, Report No. SP 126-19, Vol. 1, pp. 355-370.

39. Rasheeduzzafar, F. H. Dakhil, M.A. Bader, and M.M. Khan. 1992. "Performance of Corrosion Resisting Steels in Chloride-Bearing Concrete." *A CI Materials Journal.* Vol. 89, No. 5, pp. 439-448.

40. Johnston, D. P., and P. Zia. 1982. *Bond Characteristics of Epoxy-Coated Reinforcing Bars.* North Carolina State University, Report. No. FHWA/NC/82-002.

41. Patil, B. T., G. Ranganna, M. R. Gajendragad, and T. Ramchadran. 1983. "Pullout Strength of Coated Steel Reinforcement in Concrete, " *Indian Concrete Journal, Vol.* 57, No. 11, pp. 290-292.

42. Johnston, D. P., and P. Zia. 1984. "Bond Fatigue of Epoxy-Coated Reinforcing Bars." Materiaux at Constructions, *Materials and Structures,* Vol. 17, No. 97, pp. 30-34.

43. El-Sayed, H.A., M. M. Kamal, S. N. El-Ebiary, and H. Shahin. 1987. "Effects of Reinforcement Corrosion and Protective Coatings on the Strength of the Concrete/Steel Bond." *Corrosion Prevention & Control,* Vol. 34, No. 1, pp. 18-23.

44. Treece, R. A., and J. 0. Jirsa. 1989. "Bond Strength of Epoxy-Coated Reinforcing Bars." *ACI Materials Journal.* Vol. 86, No. 2, pp. 167-174.

45. Cleary D.B. and J. A. Ramirez. 1991. "Bond Strength of Epoxy-Coated Reinforcing Bars." *ACI Materials Journal.* Vol. 88, No. 2, pp. 146-149.

46. Choi, 0. C., H. Hadje-Ghaffari; D. Darwin, and S. L. McCabe. 1991. "Bond of Epoxy-Coated Reinforcement: Bar Parameters." *ACI Materials Journal,* Vol. 88, No. 2, pp. 207-216.

47. McKeel, W. T., Jr. 1977. *Evaluation of Epoxy-Coated Reinforcing Steel,* VHTRC 77R56, Virginia Highway Research Council, Charlottesville, VA.

48. McKeel, W. T., Jr. 1988. *Evaluation of Epoxy-Coated Reinforcing Steel,* Federal Highway Administration, Virginia Division.

49. Munjal, S. K. 1981. *Evaluation of Epoxy-Coated Reinforcing Steel in Bridge Decks,* Report No. FHWA-MD-82/03, Maryland State Highway Administration.

50. Hagen, M. G. 1982. *Bridge Deck Deterioration and Restoration - Final Report,* Report No. FHWA-MN-RD-83/01, Minnesota Department of Transportation.

51. Kessler, R. J. and R. G. Powers. 1988. *Corrosion of Coated Rebar: Keys Segmental Bridges, Monroe County,* Report No. 88-8A, Florida Department of Transportation, Materials Office, Gainesville, FL.

52. Kessler, R. J. and R. G. Powers. 1988. *Corrosion of Coated Rebar: Key Segmental Bridges, Monroe County,* Report Prepared for CRSI Meeting, Florida Department of Transportation, Materials Office, Gainesville, FL.

53. Weyers, R. E. and P. D. Cady. 1987. "Deterioration of Concrete Bridge Decks from Corrosion of Reinforcing Steel," *Concrete International,* Vol. 9, No. 1, pp. 15-20.

54. Malesheski, G., D. Maurer, D. Mellott, and J. Arellano. 1989. *Bridge Deck Protective Systems,* Report No. FHWA-PA-88-001 + 85-17, Pennsylvania Department of Transportation, Harrisburg, PA.

55. Malesheski, G. 1989. *Bridge Deck Protection with Epoxy and Galvanized Rebars Pennsylvania Experience,* NACE Northeast Regional Meeting, Baltimore, MD.

56. Hededahl, P., D. G. Manning. 1989. *Field Investigation of Epoxy-Coated Reinforcing Steel,* Technical Report MAT-89-02, Ministry of Transportation, Ontario, Canada.

57. Powers, R. G. 1987. *Corrosion of Substructure Long Key Bridge,* Florida Department of Transportation, Bureau of Materials and Research Corrosion Research Laboratory.

58. Perregaux, G. R., D. R. Brewster. 1992. *In-Service Performance of Epoxy-Coated Steel Reinforcement in Bridge Decks. Final Report,* Technical Report 92-3, New York State Department of Transportation.

59. Canadian Strategic Highway Research Program (CSHRP). 1992. *Effectiveness of Epoxy-Coated Reinforcing Steel. Final Report.*

APPENDIX A. 1993 TRANSPORTATION RESEARCH BOARD PRESENTATIONS, WASHINGTON, DC, JANUARY 1993.

EPOXY-COATED REBARS FOR REINFORCED CONCRETE STRUCTURES

TRANSPORTATION RESEARCH BOARD 72ND ANNUAL MEETING

JANUARY 1993

At the 1993 TRB Annual Meeting in Washington, D.C., a full day of technical session was devoted to the corrosion protection performance of epoxy-coated steel in concrete. The following presents a summary of each speaker's presentation and the forum held after the presentations. Transcripts of the tape of each speaker is presented in Appendix A. Note that the transcript of presenters is our best effort of what the presenters were saying. Speaker's wording or sentence structure was not changed in fear of changing the meaning of the presenter. Words were left as a blank (_____) when they were not understood.

A.1 THE LONG TERM PERFORMANCE OF EPOXY-COATED REBAR KENNETH C. CLEAR

The long-term effectiveness of epoxy-coated reinforcing steel on the basis of 18 years of my involvement with epoxy-coated reinforcing steel starting with the first bridge deck which was built in 1974 in Pennsylvania and extending through current work that is being done under the National Cooperative Highway Research Program. The primary source of information in the report that is relation to this presentation is a study that was done by my company for the Canadian Strategic Highway Research Program. The draft final report of that study was issued in March 1992 or subsequently 2 peer reviews done one by Wiss, Janney, Elstner Associates sponsored by the Concrete Reinforcing Steel Institute and the other by Professor Godfrey Peter Schiessl done for the Canadian SHRP Program. Subsequent to receipt of both of those peer reviews in the several sessions where the report was critiqued the report has been revised, additions have been added to take into account the suggestions of the peer reviewers and to point out certain areas where the peer reviewers were incorrect.

That report is now very close to the printers and will be available we expect within a month or so.

The work that was done under CSHRP was a culmination of work that had started with many other efforts. Including those of the Federal Highway Administration dating back to the early 70's, work that was sponsored at my company by the Concrete Reinforcing Steel Institute starting in 1985, work that was sponsored there by the Fusion Bonded Coating Association in 1982, cooperation by a number of states and provinces who aided us in taking cores and so really is a very combined effort that I could spend a lot of time thanking everyone who was involved, and whose effort could not have been accomplished in the time period that was involved without all those people's input.

The Florida Department of Transportation, of course, played an important role in that they did much of the leading edge work that we were able to follow on behind and the effort excellerated, of course, in 1988 when the Florida Keys failures occurred.

Our charge was primarily one and that was to determine whether epoxy-coated reinforcing steel in northern marine environments and deicing salt areas would provide long-term effectiveness against corrosion damage in a conventional concrete deck. The long-term was defined as 50 or more years. So it was obviously impossible to go out and look at existing decks, the oldest of which was approximately 18 years at the end of the study, 16 at the beginning and say yes, they are going to last 50 years. So you will find my talk does not say this is the condition of epoxy-coated bar decks today as a goal. The goal is to determine will we get 50 or more years of effectiveness in marine and deicing salt environments in North America with epoxy-coated reinforcing steel.

I thought that it might be valuable just to take a couple of minutes and talk about the process since I was the first speaker in this dual sessions and so I have a few slides of the process that is used to fusion-bond coat epoxy-coated reinforcing steel. The bars are typically cleaned, heated, here we see the cleaned bars moving through typical specifications called for a near white metal glass on the bar there is then a subsequent heating operation in most operations to get the bars to the proper temperature; the bars go into a chamber where powdered epoxy is sprayed and the bars themselves are grounded to allow the epoxy to attract electrostatically. Here is a water quenching that is done with most powders in use today immediately after or very shortly after it comes out of the coating chamber. Any type of specification testing is done which you saw previously with an inline holiday protection the bars are then stockpiled indoors for our team from transportation.

The first epoxy-coated reinforcing steel specification was prepared in 1975 and subesequently there were a number of changes through about 1978 with very little changes from 1978 until recently. As I indicated the problems that occurred in the Florida Keys after 6-10 years started a detailed review by a number of agencies and a number of organizations concerning epoxy-coated reinforcing steel. This is one of the piers after 6-10 years and that was epoxy-coated reinforcing steel.

The phenomenon that we see in these substructure areas is one that had not been documented previously and this slide is a little dark but basically the importance of it is its a bar removed from the Florida Keys substructures. The utility knife has been run down longitudinally and you literally can peel the epoxy off. Underneath is black and red rust corrosion products and most fearful it was low pH. So even though concrete is highly alkaline with a pH of 12 beneath the coating there were pH's in the range of 5, which is not alkaline. It was unexpected on the basis of the first 10 years of research on epoxy-coated reinforcing steel and there was no inkling that a failure mechanism of this type existed. Our charge soon became to determine whether or not that this failure mechanism was only applicable to the Florida Keys and what characteristics of the bar used in the Florida Keys caused it to occur or whether it was in a mechanism that would also be applicable in Northern North American environments.

The project approach that was taken was one to start off looking at the state of the art of in the rebar and other fields but this state of the art review was different from most in that we found that studies had been started but not continued or placed on hold even though the specimens still existed. So while we were able to go back into records especially those of the Federal Highway Administration and find unpublished data, it was also necessary to do extensive reviews of the specimens and studies of the specimens making measurements of after another 5-8 years had lapsed. From the state of the art review in other fields it became obvious that the existing specification tests were not adequate to properly address the issue and so development efforts were undertaken to develop new tests. These new tests took primarily two forms, one to define present condition of a bar taken from a core most usually, and secondly, to predict future performance. I've already said that it was necessary to extrapolate data to predict what would happen or would we get performance through 50 years and therefore it was necessary to simulate the environment that we expected the bars to be in and do so in a relatively short period of time. The evaluation needed to be relatively widespread so we looked at northern decks in both the United States and Canada choosing those decks that we thought had the most probability of chloride contamination and that placed us predominantly in those that were built in the 70's where possible. The long-term outdoor exposure slabs played a primary role in the process in that these slabs although chloride intrusion had been accelerated subsequent to chloride intrusion by rapid chloride intrusion by ponding or by mixing the chloride in the concrete, they simply had been exposed outdoors in northern environments. So it was quite obvious that performance of these specimens in northern environments would be indicative of what would happen in northern bridge decks. We looked at bars from coaters both bent and straight because the initial concerns in Florida related to bent bars and it was believed that perhaps it was the bending process that was causing the problem and there would be no problem with straight bars. In another year or two after 1988, of course, we knew that the problem also existed with straight bars.

We looked at bars from job sites in that they are quite different from bars from coaters. Bars that are obtained directly from coaters generally speaking do not have outdoor exposure for any length of time, do not have workmen handling them, whereas job site bars may have been stored on the job site for 6 weeks to 6 months typically, and have much more handling damage. The job site bars we looked at were straight primarily because by that time we knew that they were the best bars and if we couldn't get performance from the straight bars then we knew we weren't going to get it from bent bars.

And finally we looked at the effect of 6 month job site exposure on the bars. There was a general belief that job site exposure may be harmful to ECR and so we arranged to have bars that had been obtained directly from coaters shipped to Toronto where they were exposed to the benign environment for 6 months and then reevaluated. The data analysis involved updating state of the art, identifying the failure mechanisms that were involved in all the specimens that were looked at, and assessing the long term effectiveness as a result of that data. Overall more than 3,000 measurements on 317 epoxy-coated rebars, 173 cores from field structures, and 93 epoxy-coated concrete specimen that had been under exposure for a year to 16 years were done within the overall program. The field structures, there 6 structures in Canada and 13 in the Northern US, 17 of them were bridge decks and they varied from 3 - 16 years old, there were 2 barrier walls, one a noise barrier wall along a roadway and the second a bridge parrapet.

The coatings that were involved were Scotchcoat 202 and 213 which are your typical green epoxies the 202 being the original epoxy that was borrowed from the pipeline industry in 1974, 213 being that which has been used more recently primarily in the last 10 years. Fluflex 6080 was one of the original qualified epoxies and Armstrong R-349 and R-361 were epoxies that existed in the 70's and early 80's.

We took a minimum of 6 cores with epoxy-coated rebars in each structure and if we looked at the visual condition of the cores as taken they were quite good before the cores were taken apart and autopsied except that many were cracked, and we in fact purposely courted cracks because we knew that the chloride contents would typically be higher at cracks. So on a typical deck we would select cores such that we got low cover areas that were uncracked, we got cracked areas, and we got uncracked areas with the typical coverroom structure. The structures were not chosen randomly. They were chosen on the basis of what decks states had data on as to what coating was placed and which ones were the oldest with respect to chloride for obvious reasons.

Somewhat surpisingly, given the age of the structures, only 8 of the 19 structures had chloride levels at the reinforcing steel that were greater than 1.1 lbs/yd^3 which is typically accepted as the threshold for corrosion of uncoated rebar. And only 4 of the 19 structures had over 5 lbs/yd^3 at the level of steel, which is a level that is very corrosive to uncovered reinforcing steel. The average cover for all the bridge decks was 2.6 in which indicates that the work had been done by the various highway agencies in the last 15 years to improve cover, to get deeper cover was certainly achieved.

When we first broke the cores apart and began to examine the epoxy-coated rebar it became quite obvious that the ECR's from cracked cores that were greater than 8 years old typically showed corrosion. Those, of course, were the cores where the chloride was the highest. We found corrosion related to lamination and spalling on only 3 of the 18 structures. Quite interestingly, that's 3 of the 4 with chloride levels greater than 5 lbs/yd^3 or 75% of the structures that are in a severe chloride environment.

Concrete permeability was somewhat variable and we used the AASHTO T-277 concrete permeability test, but mostly they were low to moderate which again indicates that the concrete quality on bridge decks built in that time is far improved from that in the past. These are a few photographs that are contained in the report on a couple of the structures that did show the stress.

The two upper photographs are an overall and a close-up of the noise barrier wall from Ontario and the lower photographs are core from an area that was undoubtedly delaminated from a New York bridge deck. You can see the horizontal fracture on the core that is so typical of delamination as well as a vertical crack in the structure, you can also see the corrosion of the epoxy-coated reinforcing steel and the rust staining moving out into the concrete in the time period of 15 years.

If we look at the field structures and coating properties, I don't have time today to go through all these, but I think it is important to say that the coating thickness averaged about 9 mils which is very desirable with respect to specifications. The anchor pattern or the tooth that's created on the bar prior to applying the epoxy was right in line with that desired by specification. There are a couple items in this slide, however, that were somewhat surprising and that's in the area of holidays and bare areas and what we call dry knife adhesion. Holidays, as we use them in the definition, is a pinehole not visible to the naked eye as a size of a 10th of a millimeter or less, whereas a bare area is something that is visible to the eye.

If we look at the results the median we had 6 holidays per foot and 6 bare areas per foot which gives us a median of 12 holes in the coating per foot on the field bars as taken from cores. The variation is very large to greater than 64 and to 39 so we had bars, epoxy coated rebars coming in from field highways structures that had 100 breaks in the coating per foot. We also has some that had zero. So we have a very, very wide range. The average number of breaks were 20 per foot, and of course we have a material here that is performing on the basis of being an insulator and when you have 20 breaks per foot in the field it does very little good to have a specification that only allows 2 per foot in the plant. Most surprisingly, however, was the dry knife adhesion and this is an adhesion test to measure the adhesion of the epoxy to the sealed substraight. You basically take a sharp knife and make an X on the epoxy-coating and if anyone has ever tried this on a bar that has just been coated or has been in the lab for a number of years, it is literally impossible to then lift the epoxy off the surface. You can see the X in the center of this slide and in this particular bar, you can then literally lift the epoxy very easily from the surface. The adhesion between the epoxy coating and the reinforcing steel has been lost in this instance.

We found that the adhesion loss had occurred on roughly half and specifically 42% of all the bars that came in from the field. So this was a phenomenon that was similar to the phenomenon that had occurred in the Florida Keys. And yet it was occurring in Northern brige decks, and was primarily occurring on the older bridge decks, those 8 or more years old. So in roughly half the cases of all the bridge decks 3-16 years old we had this phenomenon occurring but in the older decks in a majority of them we had this adhesion loss phenomenon occurring. Why is that important? Well quite obviously with a barrier coating that has holes in it if you loose adhesion between the barrier and the seal substrate you then could move chloride and all other kinds of ions underneath the coating.

These are some shots of the cores or the bars that came out of other cores within the study. On the left hand side are the bars that came out of uncracked cores, so they generally have low chloride scars. On the right hand side are bars that came from cracked cores so the chlorides are significantly higher. You can see a variation in performance, with the cores in the uncracked concrete with low chloride the bars being in better condition. In the cracked situation, typical rusting showing up in the 2-16 year period.

To get beyond the 16 years it is necessary to predict how these bars would perform in the future. To do so we went back to the tests that have been in the specification annex for epoxy-coated rebars since 1975 and modified them as was necessary because were were dealing with steel bars and on the basis of the state of knowledge in that point in time. Modifications were first off, to shorten up the tests from 30 days to 7 days, and also to use a combined solution. One of the interesting things about this specification is it requires the testing of powders separately in lime water from chloride solutions, so you never have the situation where you are testing in a low pH solution with chloride.

On the basis of the work done by Florida DOT and the University of South Florida it was becoming evident that the alkaline environment in the presence of chloride was an important characteristic so we adopted the testing in chloride solution lime water and chloride solution. There is a lot of information in this slide, but I basically simply would point out that we had a desire that the knife adhesion not get any worse on those bars that still had good adhesion coming in from the field. We tested the bars with good adhesion for future performance because we knew what the performance of the bars with poor adhesion were. So we had a desire that the knife adhesion not deteriorate in the 7-day test when we were applying 2 volts. So we were forcing the corrosion, we have the epoxy coated bars as an anode and a cathode these are roughly 4 in long bars that came out of cores from the field.

We found that the performance was variable. Sometimes we saw no reduction in the adhesion, other times we saw complete loss of adhesion. With the median saying loss of adhesion occurred. So these are the 50% of the bars that had good adhesion to start with, our accelerated tests predicted that they would loose adhesion. The anodes and the cathodes debonded during this test and you also see the wide range of disbondment from 0-85% at the anode with a corroding side of the fourth corroding cell, and .6 to 80% at the cathode. The other data involves resistance measurements which indicate deterioration and relatively high currents, but if there is current flow of a significant amount in this test obviously the barrier qualities of the coating are broken down.

These slides show the results of some of these testing on the left hand side are the cathodes or the noncorroding side of the corrosion cell and on the righthand side are the anodes. We look at the center photograph what you see on the left hand side used to have epoxy coating on it. The epoxy coating totally disbonded and came off the cathode during the testing. It also came off the anode and rusting occurred beneath the anode. But you see on other bars that the performance was quite different. The one on the top we saw very little loss on the cathode, and we saw deep pitting into holes on the anode rather than the smoothe corrosion. The one in the lower photograph was intermediate performance. We saw a small amount of disbonding and some deep pitting. Again these are photographs of other specimens that came from the field. The lower one on the bottom shows excellent performance with very little disbondment or adhesion loss at all. The one on the top shows the deep pitting again with little loss and the center one showing intermediate performance.

I was quite surprised at the wide variation in performance that was seen on the bars that were the youngest coming from the field and were the best. We also performed another test that did not imply that it was similar to a test in the specification annex. This test ran for 45 days and simply was the immersion of the bars in calcium hydroxide solution containing sodium chloride, so there was no forced corrosion here.

The best bars from the field, the 18 best bars coming in from the field were the ones that were tested and they literally simply were placed in an alkaline chloride environment for 45 days taken out and allowed to dry in a dessicator for 7 days and then the knife adhesion was measured. In every instance, 100% of the cases, adhesion loss occurred on the epoxy-coated bars just as a result of being soaked in limewater and chloride for 45 days. Absolutely amazing considering the fact that that same test or similar version has been in the specification annex for 15 years for testing new powders, and every new powder that was ever prequalified was certified that you would not have such adhesion loss.

Whether its a problem with the previous testing or whether its the combined solution we don't know at this stage. But we do know that calcium hydroxide and salt are much more representative of real world concrete than just salt water alone or just limewater alone.

We also tested bars from coaters and job sites. These bars will range at looking at todays quality bars that were in better condition than the bars that were coming in from the field. I'll point out a couple things in this, and that's that the bare areas per foot, which are to be 0, of course, whenever the bars are coated and tested in the coating plant varied from 0-3 on the bars that we got directly from coaters. Now there was some shipment involved here but they generally were wrapped very carefully, sometimes, most often as individual bars. The bare areas on job site bars were 1-12 per foot as a range. Again, a very high number of holes in the coating per foot of bar, we are up to 15 in some of these instances. Also point out something called underfilm contamination. This is dirt and dust and remnants that remain on the surface of the steel when the powder is applied. In other fields, the pipeline industry specifically, they believe that to get good adhesion, you should never have underfilm contamination in excess of 25%. We found underfilm contamination varying from 10-80% on the bars that came directly from US and Canadian coaters and 25%-60% on bars obtained from job sites. We have a lot of contamination, and what that says is that on many of the bars half of the material is adhering to dirt and dust and contamination rather than to the steel substrates.

We ran accelerated corrosion tests on the parts from select coaters and primarily choose the best bars from the coaters that we possibly could choose of the realm of bars we had and this view graph simply compares the desired results which is a very high insulating characteristic throughout the test, a low current flow in this fourth corrosion test where we are applying 2 volts, a knife adhestion that stays strong which is a rating of 1, no anode disbonding and no cathode disbonding. Some might say that's pie in the sky wishful thinking, but this is a 7 day test and we are trying to predict to 50 years and obviously if we are going to be able as an engineering profession to rely on a corrosion protection system for upwards of 50 years or at least 40 because chloride contamination will begin to occur at that point on the severe structures, we are going to have to demand very significant insulating performance in short term accelerated tests. We found that these bars performed much better than the from the field or the bars from the job site. But we still saw wide ranges in performance. You see an end resistance varying from 180 to greater than 1.1 million ohms. 180 is quite low. Current density varied from 0-17 milliamps per square foot - 17 milliamps is an extremely high corrosion rate. The cathode knife adhesion at the end, they all started off with good adhesion, at the end again they varied.

Some of them had good adhesion, some ofthem had total loss of adhesion. It was only a small amount of anode disbonding but the cathode disbonding or the noncorroding side of the cell showed up to 16% disbonding.

In addition to the testing and looking at the properties that we looked at previously, one of the other requirements within the specification annex is that no additional holes in the coating develop during this 30 day test. In the modified 7-day test that we used on bars from coaters we documented the number of holes in the coating prior to starting the test and after the test. On the cathode side or the noncorroding side the number of film failures increased on 14% of the bars. Whereas on the anode side or the corroding side, the number of film failures increased on 74% of the bars. So during 7 days in this test 74% of the bars got more holes in them. Like the corroding side which indicated again failure in relationship to the protective properties of the coating and undoubtedly relates to the loss of adhesion that was occurring on many of the bars.

The chemical immersion test was also used. If you recall in this test we have 45 days. And again in every instance that we ran the test we lost adhesion in 45 days on bars directly from coaters, high quality bars, 100% of the cases. We also evaluated bent bars from 5 sources and those bars were similar in performance, but I'll call out to you the bare areas in the cracks in the bends. They were highly variable from 0 cracks at the bends to 32 with bare areas from 0-27. One interesting thing was that the cracks at the bend were microscopic in all instances and in many instances they could not be picked up by 80,000 ohm holiday detection which is what is used in the specks. Until they were stored for a significant length of time. One of the things that is brought out in the Wiss, Janney, Elstner report is they asked the question as to why there's a difference, a gross difference between the holiday results on bent bars by KCC, Inc. and those that WJE measured on 3-year old retaining bars. Well there is many reasons and these are documented in the CSHRP report including the fact that WJE made their tests incorrectly on the patched ends of the bars, but the most important, and on those ends we had made holes for holiday protection, but most importantly we found out that these cracks grow with time. And I would assume that it is the result of creep of the concrete. So when we take a bar and we get a bent bar in we test it using 80, 000 ohm holiday protection there are no holidays. We can sit in the lab for a year or two retest it, there's 32 holidays. Note here that the holidays are 0-5 with the cracks that advances to 32. So we are not picking them up in this initial test. We've suddenly retested these and we're picking up every one of them.

This is a shot of one of the bars where we've lost the coating totally on the bent area during the prediction portion or accelerated corrosion test. Six months after exposure had an influence on the bare areas on the bar. Before exposure we had a median of 2 bare areas per foot after exposure we had a median of 7 so bare areas tripled just due to 6 months of benign outdoor exposure, no chloride. Obviously, this outdoor exposure is causing burrs, and areas of thin coating to become exposed in the process.

I want to spend a few minutes talking about, if I could, the outdoor exposure study. We can't give them complete justice in this time period so I tried to summarize the results of a number of different studies. Three specifically the first is a 1974 FHWA slab study that was stated in 74 interim results were issued after a number of years, but the slabs were maintained at the Fairbanks Highway Research Station Outdoor Test Yard. If we collect all of that data these slabs were salted during 4 summers and then salting was stopped and they have just been exposed to natural weathering since that time. We find that on the poor quality bars and these were actual bars from the first bridge deck that was ever built with epoxy-coated bars in Pennsylvania we got bars from that bridge deck of both types of coating and then tested every bar and divided them up into poor quality and good quality. If we look at the poor quality bars the extended time to deterioration in comparison to uncoated bar was 3½ to 6 years.

The very high quality bars, those with no visible bare areas 0-2 holidays per foot under laboratory testing Scotchcoat 202 gave us 15 years extended time to deterioration. We now are at the point on those bars in that concrete that the coating has lost adhesion and underground corrosion is occurring. Another study was done by FHWA in 1980 in which 15 lbs of chloride was mixed directly in concrete. The bar quality was poor in all instances and these bars had been stored outside for 2-3 years before they were incorporated in concrete. They had a large number of coating breaks at the point they were incorporated in the concrete. On the other hand, they were certified to meet the specifications at the coating plant with the exception of the thick coating. So we have bars that had too thick a coating but met the specifications at the plant for less than 2 holidays per foot, no bare areas.

After 3 years outdoor exposure they had a lot of bare areas, they wouldn't pass the bend test at that point in time, and so they are noted as poor quality.

If you had only the top mat coated its 6.5 years, if both mats were coated 6.5 years of extended performance as well. The third is the 1982 study that was done at Kenneth C. Clear, Inc. about 3 years of ponding with 3% sodium chloride and these were high quality bars with variation in coating breaks estimated at .0005% to .01% and regardless of top mat only or both mats coated the extended time to deterioration was 7.6 years.

This is typically what slides you may have seen me show after 3-4 years of exposure of the FHWA slab. This is what they looked like later. The epoxy was good initially, subsequently significant corrosion. This is typically the second series of FHWA slabs the 1980 series that cracked in 88-89. This is an epoxy-coated bar slab 8 years old. These Scothcoat 213 epoxy-coated reinforcing steel looked like very high chloride, severe environment 7+ mil average coating thickness. In this instance the chloride was high because the cover was low. 8-10 years was the time of performance.

The failure mechanism of ECR on high quality bars is different from what we had assumed. There is a lot of adhesion failure mechanism followed by underfilm corrosion then iron-chloride complexing and pH reduction and then a hydrogen evolving cathode which allows the corrosion rates to go up many, many, many times. We get blistering, brittlement and cracking of the coating. Then we get significant macrocell action. So we were wrong in the past only looking for what we called conventional significant macrocell action. Poor quality is basically the same except you get the macrocell action at the same time once the chloride gets there. So for the last 15 years or the first 15 years of use of epoxy-coated bars the assumed primary failure mechanism has been in error. That which was seen in the Florida Keys is in fact the failure mechanism that is occuring in northern environments as well and its loss of adhesion of the coating undergoing corrosion blistering, cracking and then significant macrocell action. So it matters not what ones macrocell action is, failure will occur.

Conclusion to the work are the most important properties of ECR effective performance are the number of coating breaks and the coating adhesion long term in the concrete environment. the first of course we can measure. In recent efforts and specifications have been to reduce them the number of allowable coating breaks. I think I need to put those in perspective for you though.

At 2% allowable bare area, which was the typical specifications for 15 years you are allowed 3100 bare areas the size of a pinhead on a 10' length of #4 bar. If you go to .25% bare area that was recently recommended by the Federal Highway Administration, you are allowed about 350 bare areas the size of the head of a pin in a 10' bar. If you go to the .0006% bare area that is defined in the Wiss, Janney, Elstner report as that level that is associated with an 80-90% loss of protective qualities you allowed 1 bare area the size of a pinhead in a 10' bar. so what is happened we have changed from a product that is allowing 3100 bare areas to one allowing one. And I ask the question, "Can epoxy-coated rebar structures be built with one bare area the size of a pin every 10' on the bar?"

Coating adhesion, unfortunately, we cannot measure. Long term, we cannot simulate. Present work is going on to do that, to try to come up with quality control tests.

The conclusions are that I am going to skip over these, I think you have heard most of them. Five nothern field structures with concrete damage or field severe corrosion ECRs have been identified. The most important conclusions are that fusion bonded epoxy coatings will not be effective in providing long term 50-year or more corrosion protection to reinforcement in northern US and

Canadian deicing salt environments. It is expected that the increase in life for ECR structures over those with uncoated rebar will be in the range of only 3-6 years for the typical field quality ECR and 8-11 years for laboratory quality ECRs.

Where do we go from here? I think we need additional studies on the failure mechanisms of ECR and we need to concentrate looking on the good performing ECRs. In our bent bar study, bars from 2 sources performed quite well. And we don't have a full understanding as to why. It certainly is not related to the holidays or the thickness of the coating when the actual data are evaluated. Development of new coatings with greatly improved adhesions and fewer coating breaks I believe are needed. We need new rapid test techniques so we can predict the long term adhesion characteristics in concrete. We need to determine the bonding concrete properties of epoxy-coated bars with poor adhesion primarly because half our structures out there have poor adhesion and we have never tested the bonding concrete characteristics of such. We have some work in progress. I think we need to look at European and Japanese technology to determine whether it is superior or equal to that of North America. I believe that cathodic protection for the corroding structures that have already been built or some other electrochemical treatment. We've got to come up with a way to, I've been told there's a 100,000 structures out there with epoxy-coated bars. We need to come up with a way to extend their life before we have the delamination and spalling and we are into a patching mode.

Finally, I think the most important thing that I have learned from this overall effort is the need to continue long term outdoor exposure efforts on all protective systems for many, many years if in fact we are going to get valid predictions and as the Civil Engineering Community have good safety factors with respect to our corrosion protection systems when we are desiring lives with 50 or more years. Thank you very much.

Questions:

1. Medford, North Carolina Department of Transportation - Did you do any studies about light effect on the deterioration of epoxy? Did you look at that?

There were no control study of that type. the only effect was outdoor storage in Torono during that 6 month period. But, no in this study there were no controlled studies. There are studies going on to a limited extent in the ongoing NCHRP project on epoxy-coated bars. And that particular work is being done by Florida University.

2. Bob Sweeny, Canadian National Railways, Montreal: I'd like to have you comment on your failure criteria. It seems to me that epoxy-coated rebars are better than non-coated rebars, and we have quite a number of structures with not very good concrete that are about 1926 vintage that have not collapsed and have lots of corrosion in the rebars and all the rest of that sort of thing. Surely after you loose the epoxy-coating there then is a period of time afterwards during which the rebar corrodes to a level that you still live with and that might actually get your 50 year

service. I'm just very, very curious about your absolute failure criteria from a practical and civil engineers point of view as opposed to just looking at the rebar itself.

The question related to the failure criteria that was used and the point was made that bars can corrode a lot before the structures collapses. Most of the structures that I have been involved with in the last 23 years, maintenance was not performed because the structure was going to collapse. Maintenance was performed because of riding quality or the danger of falling concrete. Included in the projections that were made within the CSHRP report is the same amount of time from severe corrosion to severe concrete distress for uncoated bars as for epoxy-coated bars, and I believe that that is the maximum that one should consider by virtue of the fact that there is no physical sticking that occurs between epoxy-coated bars and concrete. Therefore, if you go to the Florida Keys and you go to an epoxy-coated bar structure that you have delamination and you hit it with a two-pound hand sledge, chuncks of concrete fall off that are large in size. If you go to an uncoated rebar structure with the same amount of damage, delamination, and corrosion, you have to beat on it tremendously to knock off chunks of concrete because of aggregate interlock and the movement of rustic. So I'm afeared that when we go from delamination to spalling with epoxy-coated bars it will actually be a shorter time period that it is with black steel. I assumed in the projections it could be the same, which is typically 2-5 years on the first crack to deterioration, but I believe that we should not assume more than that, and I guess only time will tell whether or not these predictions are correct. I can say this, that I concur with you that we saw nothing in our program with respect to corrosion that said epoxy-caoted bars were worse than uncoated bar. I have a worry concerning bond adhesion and once that is put aside, the point I think that I am trying to make is not that you should throw out epoxy-coated bars but rather that if you want assurance of a 50+ year life and you'r going to use epoxy-coated bars you're going to have to use it in concert with other protective systems if you are in a severe chloride environment.

3. Jerzy Zemajtis, Virginia Tech: I have a question about those 19 structures you were evaluating. Have the bars passed requirement tests, bend tests, etc. before embedding them in the concrete? Is there any information about them?

Question related to the information on the 19 structures. The answer to your question I believe is yes and maybe. Everyone of those structures was certified, the bars was certified by independent test labs to meet the AASHTO specifications. They were accepted by the State or Providence DOT as meeting the specifications. Now there are some questions, and we certainly know that it is highly variable with respect to specification enforcement and there were a number of years where there was a halo surrounding epoxy-coated bars and people weren't really enforcing the specifications. So on the books yes, in reality maybe.

A.2 THE PERFORMANCE OF EPOXY-COATED REBARS: REVIEW OF CONCRETE REINFORCING STEEL INSTITUTE RESEARCH STUDIES DON PFEIFER

This talk will summarize our review of CRSI Sponsored Corrosion Studies undertaken during the last 10 year period at Kenneth C. Clear, Inc. A 334 page report from Wiss, Janney is available from CRSI. This review was prompted by the following conclusions from the 3 year KCC study that utilized bent, and straight coated bars from 8 US factories.

Conclusion 1. During the continuous ponding the majority of the slabs containing epoxy-coated bent and straight rebar underwent a significant change. Mat to mat resistances were reduced many fold and microcell corrosion currents increased significantly to levels commonly seen on uncoated bars. Almost complete failure of the corrosion protection properties on many of the coated bars was indicated.

Conclusion 2. The only variable that had a significant effect on corrosion was the source or the factory.

Conclusion 3. The feature or features of these bars which yielded a superior performance under continuous ponding had not been defined.

To begin our study, WJE reviewed the 1974 FHWA study at the National Bureau of Standards concerning nonmetallic coatings for rebars. The main observations from NBS are shown here:

1. Two to 10 mil epoxy films are not impervious to water.

2. Epoxy films with differences of 3-10 mils can be essentially impervious to chloride ions.

3. Bars with no or few holidays provided acceptable protection.

4. Coating failures initiated on bar deformations.

5. Bars with high maintained electrical resistance following corrosion tests provided the best protection.

6. Electrical resistance decreases when holidays are present or when the coating deteriorates.

7. Five to 9 mil, essentially holiday free films should be used and extensively damaged areas should be repaired.

Our first effort concerning the KCC report were focused on reviewing the electrical resistance properties of the 144 test conditions from the 36 slabs containing straight and bent coated bars from the 8 factories. This plot shows the ratio of the mathematical resistance of the coated bar slabs the companion black bar slabs versus the measured corrosion current density during the end of these key test periods. The data general aligned along the 2-axes and this plot suggests either good performance with high maintained resistance or poor performance with low maintained resistance.

When plotted log log, the same data filled the more logical engineering trend. Still in agreement with the envious data indicating high maintained resistance correlating with good performance. This plot shows that some slabs maintain resistances up to 1500 times the black bar slabs and others lost essentially all their initial resistance or had low initial resistances to start with.

This slide shows the 72 KCC resistance data points after the 70 week second testing following up to 10.5 months of continuous tap water ponding as compared to the 1980 FHWA nonspecificated epoxy-coated bar study. This 1980 FHWA study reports straight bars that would not pass the bend test that had greater than 25 holidays per foot and that had intentional damage to the coatings ranging from .2 - .8%. These nonspec bars had extremely low initial and final resistance ratios of 4-10. The EE point is for a slab with coated bars in both top and bottom mats. The EB point is for coated bars in the top mat and black bars in the bottom mat. All _____ _____ slabs with EB type specimens. Well the 1983 ______ ______ FHWA timed corrosion report projected long serviceability for these EE and EB slabs. This projection has not come true. These EE and EB slabs are now badly crackly and should have been anticipated. These data demonstrate that the one year long test method used in 1980 was quite mild since these EE and EB slabs showed no signs of corrosion and loose cracking after the 1-year test period. It was not until 7 years later in 1987 that these nonspec bars started to crack. These EE and EB slabs had extremely low 28 day initial resistance ratios 4-10 due to the extensive number of holidays and manmade holes. Yet these horrible nonspec bars were able to survive for 7 years in the uncracked condition even when tested in a planned out-of-door worst case FHWA experiment.

Had these 1980 EE and EB bars been tested for 1 year in pH 7 salt water solutions instead of concrete slabs these nonspec bars would have corroded severely and the prolonged serviceability projections made in 1983 would never have been made.

Following this review of resistance properties the comprehensive investigation of corrosion tested slabs, noncorrosion tested slabs, retained epoxy-coated bars, and raw data was undertaken. The following tests were undertaken to determine the factors which created excellent performance as well as poor performance for these coated straight and bent bars from these 8 factories. Autopsies were performed on 11 corrosion tested slabs with 22 test conditions. Water corrosion tests were made on the concrete, chloride tests were made on the concrete, clear cover tests were made on the bars, alkaline silica reactivity effects were studied, electrical resistance tests were made on the epoxy-coated bars, the effects of soaking coated bars in sodium chloride solution on holiday count with 80,000 ohm holiday detectors were made. Differentials scanning calorimeter tests were made on the epoxy chips to determine the cure of the epoxy. Water absorption tests were made on the epoxy chips. Skin and electron microscopic studies were made on the epoxy chips and on the bars. Long term water ponding tests were made on the noncorrosion tested slabs. Full power microscopic and SEM examined the evaluations of the corrosion products under the epoxy chips were made. Backside contamination studies on the epoxy chips were made. Surface roughness characteristics tests were made on the retained blasted bare bars. Magnetic and optically epoxy ____ thickness were tested and holiday tests were made. In this talk a holiday is defined as any break in the coating which is detectable at 80,000 ohm detector and is of any size. The 11 evaluated corrosion tested slabs contained 22 test conditions since each slab contained a straight bar test condition and a bent bar test condition. These selected 22 test conditions represent the total range to corrosion performance from excellent to poor. Six conditions had essentially no corrosion current. Two had low current, four had moderate currents, and 10 conditions had high currents.

This a view of an autopsied slab. The bottom mat bars were black making the test method a worst case experiment when compared with using epoxy-coated bars in the bottom mat. The clear cover was one inch and the w/c ratio was .47 both further contributing to a worst case experimental condition. A total of 54 retained bare and epoxy-coated companion bars were also tested. Fifteen of the retained companion bars contained epoxy-coated bars with just electrical resistance, bare bars were also tested. This slide shows the electrical resistance of retained coated, and bare bars versus estimated percentage of exposed steel on the coated or bare bar. Typical holidays per foot are indicated. Coated bar resistance ranged from 8 ohms to 450,000 ohms. The 8 ohm coated bar has several huge inadquately coated areas. Other coated bars have invariable low resistances of 3-100 ohms that tend to have 32 holidays per foot. It is not surprising that these low resistance bars had companion slabs which exhibited poor corrosion performance during the testing. Even prior to the tap water ponding, these resistance tests on coated bars and our review of the initial 50-day mathematic slab resistances prior to corrosion testing of the slabs showed that coated bars with very low resistances were used.

Initial mathematical resistances from the 36 straight bar test conditions ranged from 1,400 ohms to 285,000 ohms, a difference of 200 times. Of the 36 straight bar conditions, 13 or nearly 40% had an extremely low mathematical resistance of less than 9,000 ohms and essentially all provided performance after tap water ponding. This slide the epoxy chips being carefully removed from dry ice chill and bent straight bars. These epoxy chips were used for our numerous film studies.

This slide tabulates the mathematical resistance ratio and corrosion current densities for a straight bar from source number 1, at times 0 after 70 SE cycles and after tap water ponding. This slide also shows the perfect condition of this source #1 straight bar after being bent to remove numerous epoxy film chips for SEM low power microscopic film thickness differential scanning calorimeter and backside contamination tests. This particular straight bar from source #1 had a test lab initial ACU resistance that was 2,044 times the companion black bar slab. This resistance ratio decreased to 1,404 after the 70 weeks of cycle testing and further decreased to 308 after the tap water ponding. All the current densities are essentially zero. The bar is perfect and the MBS conclusion concerning good performance associated with high maintained electrical resistance is verified.

This slide shows just the opposite. This straight bar from source #7 had an extremely low resistance ratio of 18. Such low initial ratios are a sure sign of excessive holidays, film defects, and poor performance. The resistance ratio dropped to 16 after the 70 week cyclic tests, and then to 6 after extended tap water ponding. Current densities were very low after 70 cycles and very high after tap water ponding. As should have been anticipated the corrosion was severe under the epoxy film after the tap water ponding. Film thickness, microscopic and SEM studies on film cross sections indentified the following factors, some of which need to be recognized in future specifications. Essentially none of the bars from the 8 factories achieved the investigations targeted film thicknesses of 6, 9, and 12 mils. The films are consistently thinner at the edge of deformations than in the areas between the deformations. Chloride ion did not appear to penetrate the epoxy film layer except at holidays and bare areas. Film form corrosion that which originates at holidays and defects was observed under the epoxy film. Bar marks and the edge of the deformations and ribs were often times found to be a point of corrosion weakness. Numerous corrosion spots were related to the thin films some as thin as 2 mils that contained or developed holidays. Thin films near edges that would not be detected by magnetic guage measurements taken in the flat areas between the deformations can be identified by microscopic measurements. Seven of the 22 slabs sent to Wiss, Janney contained bars having from 11-67% of the film thicknesses less than 5 mils. All 7 slabs developed corrosion except for 1 where 26 patches had been applied to the bent bar holidays. Thus, 32% of the slabs sent to Wiss, Janney had very thin films less than 5 mils.

Holidays as shown here be ink marks were measured on the 48 retained coated straight and bent bars that has never been in concrete or corrosion tested. This slide shows the average holidays per foot measured on these 48 retained bars from the 8 factories. The holiday counts vary widely from less than 1 per foot to 32 per foot. Numerous bars had greater than 2 holidays per foot, particularly the bent bars. The WJE holiday counts were in some cases much higher than the reviewed KCC holiday and bare area counts. These differences cannot be explained given the data available.

It was concluded that the following factors did not influence the test results. Differences of clear cover cover, water absorption of the concrete, water absorption of the epoxy, surface roughness of the blasted bare bar, backside contamination of the epoxy film, and the degree of cure of the epoxy film. The factor that repeatedly showed the dominant influence on corrosion performance was the interwoven electrical resistance holiday and bare area effects. Superimposed on the dominant factor was the very severe nature of the accelerated tests.

This slide showed corrosion current density on companion slabs after the 70 weekly cycles versus the WJE holiday counts per foot on retained bars. The data to the left of the .01 milliamps per square foot current density indicates that 74% of the 38 validated test conditions with bent and straight bars from the 8 factories had less than .01 milliamps per square foot current density. This

.01 milliamps per square foot current density is considered by some as satisfactory for long term performance of epoxy-coated bars.

The two specimens in the upper right hand corner with high current densities between .1 and 1.0 had 12 - 32 holidays per foot. This slide basically the same data, but after 70 cycle plus prolonged tap water ponding. The corrosion performance of a large number of the test conditions shifted to the right indicating higher current densities. However, as noted in the lower left hand corner the 8 specimens with less that 1 holiday per foot with straight or bent bars did not shift significantly or exceed the .01 milliamp square foot level. Some of the slabs with companion bars having 1-2 holidays per foot did not shift, some did. Essentially all slabs having companion bars with greater than 2 holidays per foot exceed the .01 milliamp per square foot level.

It is quite interesting how this plot suggests that the 2 holiday per foot AASHTO limit correlates with the same critical holiday count found in our investigation. This too confirms the essence of the 1974 MBS study. This plot shows the relationship between the initial, and that is at 50 days, mat to mat resistance of the 36 straight bar slabs and the percentage of these slabs that had current densities greater than .02 milliamps per square foot near the end of the 2 test periods. The black bar slabs had initial resistances of about 135 ohms. After 70 weeks of SE wet and dry cycles the chloride content at the rebar level averaged 21 lbs/yd^3 based upon KCC data. The black bar slabs were all cracked after 70 SE cycles with excessive corrosion staining. After prolonged tap water ponding, the black bar current increased by over 80% even with no added chloride. The corrosion current densities on the black bar slabs at the end of either test period was significantly higher than previous FHWA or CRSI's studies particularly after the tap water ponding. The initial 50 day resistances of the 36 straight bar slabs as shown in the left column rate from 1400 ohms to 280,000 ohms. As shown 13 of the 36 slabs had extremely low initial resistances between 1400 and 9000 ohms. The data tabulated in the 70 SE test colum show that not 1 of the 36 straight bar test conditions from 8 factories developed current densities greater than .02 milliamps per square foot. At the same time the black bar currents were 110 times the .02 level. KCC concluded that not a single rust strain or crack was found on any of the bent or straight epoxy coated bars at the end of this 70 week SE exposure.

This 70 week SE performance tabulation explained the good performance of bridge decks. The bridge decks constructed in the last 15 years were probably constructed with similar durability and initial resistance of coated bars. Yet these decks have good performance in line with the 70 SE performance where 85% of all straight and bent bar conditions had less that .02 milliamps per square foot current density during the 70 SE type testing. It can be speculate the 70 SE cycle testing is similar to the typical bridge deck which experiences cyclic wet and dry periods in that prolong testing. The data tabulated under the tap water column for the various epoxy coated rebar specimen shows that corrosion activity increased dramatically in the coated bar slabs having initial resistances lower than 9000 ohms. On the other hand, numerous slabs with the initial resistances higher than 40,000 ohms experience low current densities less than .02 after either test exposure. This tap water tabulation indicates that 16 of these 36 straight bar conditions or 44% had less than .02 millamps per square foot densities during tap water ponding. A review of the straight bar initial mat to mat resistance data from KCC shows that the corrosion performance of these 36 straight bar slabs after extended tap water ponding could have been estimated in most cases based solely upon the measured 50-day initial mat to mat resistance. It is also noteworthy that the 3 slabs with initial mat to mat resistance greater than 90,000 ohms showed flawless performance during both test periods. These 3 conditions were from factories 1 and 3 and not from factories 4 and 6 which had been previously indicated as providing best performance.

Another noteworthy WJE conclusion was the fact that 4 bars which had been previously indicated to provide superior performance had all holidays on the bent bars prior to tests. A signifcant program deviation which was not previously recorded. As a result of this observation, we were able to conclude that the 4 test conditions where the bent bar holidays had been patched all provided excellent performance. It must also be noted that while this data shows dramatic differences between the 2 test environments, these dramatic changes due to increased moisture effects around the coated bars were achieved primarily because the water could so easily penetrate the minimal 1 in cover through a high point 0.47 w/c ratio concrete. Had these same coated bars been embedded under 2½ in of a lower w/c ratio bridge deck concrete such dramatic differences would not have occurred so easily or quickly since the moisture would find it extremely difficult to reach the coated bars under 2½ in of cover, just as chloride ions finds it difficult to penetrate 2½ in.

As a side issue, this data certainly shows why cracks in bridge decks with coated bars should be repaired as soon as possible to minimize moisture exposure. This tabulation also suggests that serviceability projections based upon worst case experiments with 1 in cover are complex and difficult to make.

In conclusion, corrosion performance during the very severe 3 -year accelerated corrosion testing on bent and straight bars from 8 factories was related to the same vital ingredients first identified from FHWA by MBS in 1974. These listed observations from the 1974 research were reconfirmed by our study. Those coated straight and bent bars capable of maintaining high sustained electrical resistance properties after the severe 3-year testing provided excellent corrosion protection. High sustained electrical resistance properties depend upon proper film thickness, good surface preparation, very low holiday counts, and proper repairs to film defects. Of particluar significance is the observation that corrosion testing causes the initial mat to mat resistance to decrease significantly, and that a high maintained resistance following prolonged water exposure is best provided by having a very high initial resistance. This review also concluded that the previously specified 5 mil minimum film thickness requirement based upon between ribbed measurements is inadequate and is a dominant source of problems.

NO QUESTIONS

A.3 EPOXY-COATED REINFORCEMENT QUALITY CONTROL IN THE PLANT AND IN THE FIELD THEODORE NEFF

In Paul Pravati's words let me characterize this as one of the waivers, but waiver with a caveat. I believe in quality epoxy-coating and I would like to discuss today some of our thoughts regarding quality control techniques we believe would help to assure quality pride we need to get durable structures.

We will hear a lot of discussions today throughout the TRB conference about total quality management. I think this is kind of a new buzz word this year with good reason. I think our investment in the infrastructure, highway structures, is extremely important and quality is a major consideration, and one we cannot overlook. I think the need for quality was demonstrated quite clearly in the previous 2 presentations, I think as from the industry's perspective we are committed and convinced that quality control is an essential part of the future of corrosion protection.

Today in the 2 sessions this morning and this afternoon we will hear alot of discussion about failure mechanisms which will be quite important and help us but I think what I'll be discussing today is more focused on how can we avoid these failures in the first place or postpone them to an acceptable time frame.

Now if you look at various research studies on coated rebar over the years starting with the NBS study that thread has already been discussed many times. Better quality epoxy coatings, less damage, fewer coating breaks, generally performs better than the similar coating with more damage. Unfortunately that message has not been communicated from the research community to the specifiers to the industry through our specifications.

This is a slide that depicts what we have all operated under in terms of the damage provision for the last 15 years. The top 2 bars represent the approximate 2% surface damage limit that's permissable without repair in many standard specifications. Now the thought on this is changing quite dramatically in terms of specifications use, but this is the basic message we've sent to people is that this kind of thing is acceptable. I think that is something that has to be obviously reevaluated and changed. One of the major considerations at the applicator is in fact preparing the bar for coating. When we talk about surface preparation, we are talking both about cleaning the bar, removing contaminates, mill scale, rust, as well as roughening the bar in order to get the maximum adhesion of the coating. We are after what is referred to as a near white cleaning condition. The quality control technique typically used in the plants is to compare the blasted bar to avisual standard most often published by the Steel Structures Painting Council. Here you see a visual standard to give a check against this painting condition.

Cyrus Nye has implemented a voluntary plant certification program to encourage quality control procedures for coating applicators. In this program we've also required the use of a supplemental test to check this clean condition. This is what we refer to as the copper sulfate test. A copper sulfate solution is applied to the blasted bar as you can see here, and the solution reacts with the cleaned areas of steel and plates out to a common color. The uncleaned, mill scale, the dirt, any contaminants, do not react and remain wet. If you look at the surface under a 30X microscope you'll see something that ranges here and this is our visual standard from our certification manual. TRB would not let me show you the typed print that goes under there so there are some black strips there. It will give you an idea of the kind of visual standard that's used as an additional check we also require the visual standard check, but this gives a more refined look at this evaluating this near white cleaning condition.

The other aspect that has been mentioned already is to evaluate the level of backside contamination. Here we use a test we refer to as the backside contamination test. This is not a specification test, but one we've developed as part of this voluntary program to try to get a handle on this particular cleaning parameter. It's quite simple test, again, a bar that has been blast cleaned, you apply a piece of white adhesive tape, it's rubbed onto the surface with a burnishing tool or pencil or something and the tape is pulled off and the back of the tape which pulls off the contamination is examined under a 30X microscope or magnifying glass and then compared to a visual standard such as you see here. Now this test is proved to be very workable, the certification inspection to date of approximately 17-18 plants, the average of these plants has been running on 16-17% backside contamination with the low in the range of less than 10% to a high in the range of 40%. I think this is in comparison to some of the earlier data which is indicative of how quality control can lead to improvement. I think we've heard some statistics stated 40-50% in some of the previous studies, but the actual inspections of the plants to date are averaging 16-17% range. This is a very useful test, I personally would like to see it standardized and adopted into standard specification sometime in the future. It tends to be a very workable test.

Another aspect of surface preparation that we evaluate is the profile or the texture of the surface. A typical quality control test that has been used for many years is to measure the peak and valley depth that's created on the surface using what's referred to a replicate tape. This is a piece of tape that has a milar type material on one side, again it's applied to the blasted surface, rubbed with a burnishing tool, which reproduces the surface profile onto the tape and then micrometer is used to measure the depth of the peaks and valleys. This gives us an assessment of the maximum depth from the peak to the valley on the steel surface.

In recent years, again as part of our voluntary certification program, we've been encouraged to use some more sophisticated testing device what's referred to as a proflometer. In this particular device a stylus is moved across the steel surface and measures the peaks and valleys of the surface. You have the advantage of giving us a hard copy trace of the steel surface, it gives us a lot more information, not only the maximum peak to valley depth but the average and very importantly the number of peaks in a unit area. Our specifications, our quality control tests in the past have only been concerned how deep the peaks and valleys were. That's an important parameter, but we're also looking at how many of those peaks we have. So this particular test device has given us a lot of information. We are encouraging its use. Unfortunately its a very expensive piece of equipment but I think it does give us an enhanced assessment of the surface profile.

It also enables us to assess this roughness in a more consumptual way. This is a relationship somewhat arbitrary that we developed for this certification program where we not only look at the peak to valley depth but also try to assess the general roughness coefficient on the basis of number of peaks with the depth. This is not really rigorously developed from a scientific viewpoint but reflects a relative assessment of the relative surface area on the bar. If you look at it in the two-dimensional triangular type peak to valley this would be a very mathematically accurate way to measure that surface area. But it gives us a relative measure of that overall surface texture and roughness and again its only possible with this more sophisticated testing device.

The final aspect of surface preparation that we believe is critical relates to testing for chlorides and soluble salts on the surface prior to the application of the coating. Our studies have shown that in some cases and some plants there are chlorides present in the glass media, present on the steel surface prior to the coating being applied, and as a result we end up with soluble salts underneath the coating which is kind of defeating the purpose. Not only undesirable in that it may facilitate underfilm corrosion but due to the fact that epoxy coating is a semipermeable membrane, osmodic effects with those soluble salts may lead to disbondment and blistering of the coating. So its a very important parameter, one again that is not been addressed in standards specification.

In terms of a quality control technique we have a couple different tests involved. As part of a routine plant control, we recommend this use of a filter paper soaked in a potassium ferracyanide solution. This yellowish paper is applied to the blasted steel surface rinsed with deionized water, and any soft ferros salts present on the steel surface will turn the paper blue. That is again looked at under a 30X microscope and compared to a visual standard. I can't show you the visual standard because of the yellow color does not come out with enough contrast to pass the slide requirements of TRB, but very similar to the copper sulfate type of visual standard, but gives a somewhat ____ look at whether or not chlorides are present on the surface. Now we also recommend in this quality control procedures that when this kind of test gives an indication of chloride that a more accurate quantitative measure be used to determine the actual parts per million present on the steel surface and the techniques used there use some strips that are based on titration methods again are more accurate measured, but the point is we feel it is extremely important in the production process to test for the presence of chloride particularly in any plants located near the coast or plants where steel is shipped during winter months, may be subjected to roadway saltspray. So this is a factor that we're finding to be something that needs quality control consideration and is a test that we recommend.

The second aspect of quality control is very important at applicator is film thickness. We have heard a lot about this, a lot of discussion about the cracked beam test. A very important aspect of that study which has not been discussed a great deal is the fact that the film thicknesses recorded showed a high level of variability. As an example, the analysis of the 9 mil target specimen showed that the variability ranged from 3-20 mils with a standard deviation almost equal to the mean in some cases. That generated a lot of concern about what was the industry's capability with respect to controlling thickness.

In the last 6 months CRSI has conducted a quality control survey of coating applicators. In this plot represents responses from 18 plants with a minimum of 20 days of production, in some cases more, but a minimum of 20 days production. The green line that you see in the middle represents the survey average, and this plot is of the mean film thickness, and I think this is our traditional way of looking at thickness. In general there is not a great deal to be learned from this particular plot. I don't see any particular problem here. What's more meaningful in terms of quality control is to look at the variability assessed by the standard deviation of these values. And again, as far as the survey average, we are running pretty good there. The standard deviation being a little over .6 mils, and you can see in many cases plants are doing significantly better than the survey average. And this difference between the plants with high variability and low variability is really what we are dealing with when you talk about quality control at the applicator. Our objective would be to have everyone down to this low range down here. But overall the survey average of these 18 plants is very good.

Another major consideration at the applicator coinciding with the research observations is on holidays. We look at holidays, again these same 18 plants, the survey average tells us that the mean values are looking very good, we are averaging in this survey on the order of .25 holidays per foot. But you can also see that there is some variability among the industry. Likewise if you look at the variabilities of the function of the standard deviation again you see very good situation in general of these 18 plants represented by this survey.

This is, I think, important, we need to go beyond these averages when we're looking at research, when we're looking at quality control, I think we need to start taking a statistical look at some of these things, realizing that a coating system as a barrier system is only as good as its weakest point, and I think looking at specimen structures on an average type basis when we're talking about holidays, when we're talking thickness, will not fully explain some of the situations we see. And likewise when we look at quality I do think we have to look beyond these broad averages and look more to some of the detailed differences among some of these situations.

The other aspect of holidays that I want to emphasize is I think we do as a research community need to address the care and precision measurement. This is an example of a holiday tester used in a laboratory with a very small sponge, more time consuming but gives a very accurate reading. I think an example of a need for this care comes from the CRSI bent bar study. This is a specimen that was retested initially recorded to have no holidays, when in fact it had many holidays, particularly at the bar bend as you see here. The point here is not that we get different numbers but we need care and that's an important part of the quality control consideration. It's not only the point that you need measure it, but we need to do it carefully.

The final aspect, major concern at the applicator is just to ensure proper cure and primarily this involves control of the timed temperature relationship of the process. Ensuring that the bars are heated to the proper temperature. Traditionally this temperature is checked with temperature sensitive crayons that melt at certain temperature many applicators are looking at more sophisticated techniques to supplement this, but our basic concern here is to control the timed temperature relationship of the process.

Now another area that we've overlooked in terms of quality control a great deal is the fact that we also have to control quality at the fabricator, and certain parts of the country the fabricator's not necessarily the same company as the company that applies the coating. The fabricator should be held to the same care and handling requirements that the applicator is, that means proper common sense requirements. In bending, bending equipment must have the proper protection. You can see from this slide, rollers need to be protected, neoprene or sometimes plastic coverings for the rollers to prevent excessive damage to the bent bars. Also any damage incurred during the bending process needs to be repaired. Proposal in current ASTM specifications call for damage in the form of cracks, bare areas, to be repaired by the fabricator. But this is an area that we have not paid a great deal of attention to in the past. Likewise, bars are sheared to length or any other visible damage needs to be repaired just as it should be at the applicator.

Now the area that probably the biggest source of abuse and the biggest potential for improvement is at the job site in terms of the handling, storage, and placement of this material in the field. Handling for the most part, involves common sense type precautions. Principally, in terms of how the material's lifted and transported around the site. Bundles of coated reinforcement should be picked up in a manner to avoid excessive sagging in the bundles. This particular lifting technique could result in excessive damage and deformations. What's recommended is to use multiple pickup points, some cases strong back. Techniques to avoid this type of damage. Storage, again, general common sense precautions. This kind of practice, or something like this is not something we can live with. You know, it's been mentioned about the halo effect of epoxy coating and this is a direct result of our specifications. People didn't worry about this kind of thing because, hey, we're allowed 2% damage and it will still perform. But it created a sense of invulnerability at the job site that we should not have had and we don't need this kind of practice. Proper storage, blocking, above ground, are essential to get the kind of performance we need.

Simple things like how we strap the bundles together using nylon or protective coating wires just to prevent this unnecessary damage, not difficult but common sense. Placement nothing particularly unique here, put to use the use of coated tie wires or coated bar supports are necessary here.

Finally, repair is something I think, there's general agreement is absolutely necessary in the field. Proposals within ASTM call for repairing all visible damage; doing away with the 2%, doing away with the 1%, and basically, if you can see any damage, you are required to repair at all phases of the project. And at this point that appears to be on its way to approval within ASTM.

I put this slide in just to give you an example of what we can expect with good construction practices. This is data from an FHWA study, one of the 1980 studies, showing survey of damage on bridges in Kentucky and Iowa and what might actually be experienced with good construction practices. And you can see that with good practice, you should not be in this 2% area that we've been allowing.

To close I'd like to just point out that this quality control and the quality we are after in achieving our durable structure is a partnership. We need the commitment from industry, and I believe that it's there, but we also need the owner's awarness and attention. Specifiers must demand the quality necessary for the performance otherwise the competitive pressures of the marketplace will be a temptation to some to cut corners, and that's what we need to address. And finally, research support. We need better test techniques. We need to have a performance based criteria, so its a partnership and working together I do believe we can advance the state-of-the-art and get the performance we need out of these structures.

Finally, I close with a couple slides here. Its been stated by some that some of these quality control efforts will not make a difference. I would submit to you that they can and are extremely meaningful. To demonstrate that just a couple more statistics. This is just from our survey data, just a comparison of some of the improvement that can be made by adopting some of these more stringent quality control procedures. This is simply a comparison of non-certified plants and certified, its not intended to mean that the non-certified are not doing a good job, they are, in fact, if you look at the industry averages, this is a comparison of that survey data to the 1988 bent bar specimens provided, and again, the standard has been significantly reduced as has the industry survey average since compared to the 1988 bent bar study. My mind a significant indication of what kind of benefits we can achieve through quality control efforts. With that I'll close and just the statement that I believe quality control is achievable, meaningful, and cost-effective in our structures.

QUESTIONS

1. Ontario Ministry of Transportation - A great deal of trouble is taken at coating plant to make sure that the steel is clean and uncontaminated when the original coating is applied. But when steel is damaged or cut, by the fabricator or by the constructor, the very best that you get is a wire brushing of the damaged area before a patch is applied and the very worst is that they just patch it without any sort of brushing. Is there any evidence of that type of patching is as effective as the original coating, and if there is no evidence, why do we accept that type of patching at all?

Well, you have two types of patching. One that just covers over the rust and that I don't know that we have any evidence that that's effective. Where cleaning is done and the patching is done properly Don Pfieffer mentioned specimens from our bent bar study where patching had been done and those particular specimens even with prior damage performed extremely well so there is some, but I think that is one area of quality control that we need to work on. You know, cases where you go out in the field the bars have been corroding and rusting for weeks and then they just go paint over it, that clearly is not as good as doing a proper job of cleaning and preparing for patching so it is again a matter of enforcement, its doable, its not always abused, but you raise a good point about situations that do not achieve the objective.

2. Richard Weyers, Virginia Tech - During the process of looking at your plant and inspecting your plant, on cleanliness, on bar coating thickness, and so on, you saw some variations in there, some fairly high ones in there. Were you able to identify what the causes were there and were you able to correct those?

The question was I had showed some data showing in some cases some very high variability in terms of some of these different parameters in terms of thickness, holidays, contamination. Have we identified the cause of that at this point. The answer is no. I mean this data is only weeks old, but it's basically our first step is to begin to take a statistical look at this. Each plant is being given their own particular data and is in fact looking at it I know in almost all cases. I think that's our next step to try as an industry to determine how we can get everyone down to that superior level. And that is really our industry objective so we haven't completely answered that but in many cases we have identified factors that do cover and they incorporated those into our particular quality control programs.

A.4 EPOXY-COATED REBARS IN EUROPE: RESEARCH PROJECTS, REQUIREMENTS, AND EXPERIENCE IN USE PETER SCHIESSL

I am happy having been invited to come to Washington to give this presentation. The idea of this presentation is to give you some information about the requirements and the use of epoxy-coated reinforcement in Europe. The experience and consumption of epoxy-coated reinforcement in Europe is far more limited than the US. We started with some research projects 10 years ago and we started with the first use of epoxy-coated rebar about 5 years ago. We are well informed about the new findings here in your country, and of course our main questions are, we have two main questions.

First question, whether the technology we use in Europe defers from the technology and the outcoming quality of the epoxy-coated rebars you use here in the States, and secondly whether we can and whether there are sufficient possibilities to improve the quality of epoxy-coated rebar sufficiently to reach a sufficient long time durability. Similar to the limited use of epoxy-coated rebars in Europe the amount of money spent for real research grotex is solemly, I must say very limited. We cannot expect sensations with respect to new test results and I will give you just some ideas what happens in Europe.

Now looking at Europe we have a very interesting project in Denmark, a tunnel project, the great Belt Tunnel is a 12 kilometer long tunnel all together where the reinforcement of the tunnel segments is executed in epoxy-coated reinforcement and I will come back to this project again. In Britain we have first projects in '88, in the Netherlands first project a port building in '89, in Germany we started research in '82, and we had first project in '87, then we have starting from 1990 and '91 first certificates for coating plants for applicators, and in Switzerland we have the first roadcheck in '88. You see we started roughly 5 years ago using epoxy-coating reinforcement. There's another plant, coating plant in Norway that use epoxy-coated rebars in Norway as well as in the other countries limited. Then we have a new plant in Italy just recently were they can epoxy-coat reinforcement. I will just show you some pictures about this project here. It's interesting because they used fluidized backfitting coating a fitting box roughly 4 m long and the reinforcement cages welded before the coating that had been reblasted twice before welding together to a cage and then after having been welded to a cage so the surface cleanliness is very good in this case. The cage is preheated in an oven and then dipped in a very quick process only a few seconds, and after that going into an over again for curing. Other 2 pictures of the dipping process and the coated cage. The cages are stored in a plant, not outside and the storage time until concreting of the segments is less than 24 hours so no exposure between coating and concreting. The philiosophies there is to use a multibarrier protection system high quality concrete with the box of cement facia over 3-5 cm that's something more than 2" cover and in addition to that these coated reinforcement with enveloped cages where possibly if necessary in the later stage cathodic protection can be applied easily should the coating fail. Every single element is checked for coating thickness and holidays. I think this is an approach of coating technology for such a project that's very interesting and we think about that. By the way, this case their produced 64,000 elements, and this is such a case where you can produce 64,000 same or equal elements it makes sense to use this technology. For ordinary applications, or course, its much more difficult that the traditional technology. Just to give you a rough idea of what's the production of epoxy-coated reinforcement in Europe very limited compared to the situation in your country. United Kingdom about 3,000 tons a year, Switzerland about 2 plants installed there, about 1,500 tons a year, Norway less than 1,000 tons, Germany less than 500 tons, Italy and the Netherlands less than 100 tons a year, that's nearly nothing.

Now what the situation with respect to standards for epoxy-coated rebars in Europe, there is the British standard from 1990 and this is the only real standard in Europe for epoxy-coated rebars and there are guidelines in Switzerland, Germany, and the Netherlands that are more or less comparable to the standard. These three guidelines are very similar. The Swiss and the German guidelines are 100% the same, whereas the Dutch guidelines prefer differently from the other two guidelines but basically they are very similar. In the written publication you will find a comparison when the German and Swiss guidelines, the British standard and the ACM standard for all the requirements given there. Within this presentation I just picked out some single requirements, I will not go through the overall list just to show what is the difference. What we have is an adhesion test, the so-called hot-water test, I will come back to this test that's missing in your standard and we found that this test is very selective wirh respect to cleanliness of the steel bar and impermeability of the film. That's a combination of a film impermeability and surface preparation that meets quality of application of the coating. We found this test very selective. Then with respect to corrosion resistance, we applied salt spray test and the cathodic disbonding test, powder identification differential scanning calorimeter, chill temperature, introdotspectroscopy, tests to insure that every delivery of the powder to the plant is really unchanged with respect to the specifications.

Bendability of the bars again a difference when comparing, EFPT means this is our authority, this is the German requirement. We have in the testing, in the quality control testing for bar diameters smaller than 20 mm that's about 8 in, we have a mandrill diameter asked for 4 times the bar diameter compared to the ASTM, that's half the diameter we use in testing, and the requirement is no cracking and no pinholes to the naked eye. This is much more strict requirement and for bars bigger than 8 in we require 6 times the bar diameter and for the application later on we allow mandrill diameters that must be 2 diameters bigger than the mandrill diameter we use for testing. That means the testing requirement is more stringent than the requirement for the bending and use. What concerns the coating thickness the mean coating thickness similar to the requirements here, about 9 mils, but what we do in addition to the normal measurement we measure the thickness at cut slices along the bar. We cut the bar in two pieces and measure along and across the deformations to check the minimum thickness for example in the edges of the ribs it was mentioned this morning that in the edges of the ribs there is a tendancy of minimum thickness and this is checked regularly that depends very much on the type of powder and of course on the temperature during the coating process. I think this is an important addition that will test with respect to the allowed damages with respect to the pinholes not sketched here, with respect to the pinholes for the initial type testing we ask for no pinholes are allowed and for the continuous production the amount of pinholes is about half the allowed amount in your specification as we heard this morning from Ken Clear, that's still too much and I think basically that's correct. The other question is whether we can achieve during continuous production a very, very low pinhole percentage. With respect to the damages the area of damages that's in square millimeters, I apologize for that but that's the comparison. The maximum area of damages we allow is roughly half the area allowed in the ASTM standard and the total percentage we defer the damages at when leaving the coating plant and at the site and do not allow these patching future breakage areas at the building site because our test results shows when the steel has been exposed to natural environment even for very short periods if you then apply a patching its useless. The type of failure is a blistering of the coating and good coating quality withstands the hot water test a longer period of time and the criteria influencing or the properties influencing the time until blistering is the cleanliness of the steel surface, the curing of the coating and the profile of the steel surface and of course the impermeability as I mentioned before.

What happens is this type of blistering of the coating film you can see here these blisters are filled with water, water penetrates through the film and peels off a process to a certain extent similar to what we see when coatings come off. Now saw that when we investigated the properties of epoxy-coated reinforcement we started, as I mentioned, in 82 and all the material we've investigated then had time to blistering, that's what hot water resistance had to blistering less than 7 days. And then we said we need to have from the type of coating business we know that we need to have lower time to blistering to have good quality and we require higher quality and the powder producers all of them that appear to have powder producers we have no applications until now from American powder producers to get a German Certificate. And then they improved the quality by improving their powder towards the direction of lower times until recently. There are possibilities to improve the cover quality as well. We have heard a presentation on quality control at the applicator but I think there are possibilities to improve the powder quality as well. And in parallel to the improvement of the resistance of the hot water test during this development starting in the middle of the 80's until now we improved or the powders have been improved to withstand much higher strains during banding or in otherwords using banding mandrils with more diameters. This means another improvement of the coating quality and basically if we improve the flexibility of the coating normally the impermeability of the coating should decrease, but it must not necessarily be.

The development we have experienced shows that both is possible, increase in flexibility and decrease in impermeability. The powder producers must be asked or forced to do this development. Now some sure results similar to this to the accelerated corrosion tests Ken Clear has presented this morning the difference we have in the test set up that for the cathode electrode we use stainless steel. In the tests setup of Ken Clear both steels are coated. Coated bars we use a stainless steel just to have a well defined countelectrode and a well defined current flow if we apply a certain voltage. And we have artificial damages and what we are doing is anodic polarization than just exposing it to a solution and cathodic polarization in differenet electrolytes sodium chloride, that's a simulated portion usually with a pH value in the range of 13.1 and that's the same simulated chloride pore solution with sodium chloride. In fact, the anodic polarization does not say very much because what happens there this is a result you can see these very good here looking at this American bar that the disbondment or pitting in the area is just related to the rib pattern. That means the pitting happens in the area between the ribs that means disbonding or corrosion area doesn't take very much and do not overestimate the results of the anodic polarization. Much more interesting is the cathode polarization result. This is a result of the 30 days of exposure with 1,000 millivolt cathodic polarization and in the sodium chloride solution we just compare bonds from the US and Germany but we should not overestimate these results because these are single results. We have seen 100's of various bars tested in the US and we tested bars from 1 source, we have no comparison where to put them in on these bars or with the less good ones. In this case we had a better behavior of the German rebar than in the simulated pore solution cathodic polarization you see the amount of disbondment is much higher close to 100% in the case of the US bar and roughly 50% in the case of the German bar and if you go back or if you take the simulated pore solution plus sodium chloride the results are more or less, lets say, comparable. But we should not overestimate and overstress this results in this short piece. If we go to the test set up with 2 volts this is the accelerated corrosion tests according to this test setup Ken Clear presented. We have 2-3 serious, one with no pinholes, and then the German and the US bars with pinholes and a cathodic polarization is 2,000 millivolts which means 2 volts and the results very simply present is that indicates no pinholes, no corrosion, and no disbondment and in the case of pinholes we can destroy nearly everything and we need to be very careful with this accelerated test. It's similar to if we take a baby, and if we take and check its life expectancy we cannot and take and put it into boiling water. That's what we are very doing with accelerated testing because we are very often change the process so considerably compared to the natural processes, but we have no other choice, but we need to know what we are doing and we need to know what are the consequences.

I'm afraid I'm 2 minutes, maybe I have the possibility to go into the failure mechanisms that happened during the basic general discussions whenever they be asked for. A test to come to an end I think what we need to consider much more and there's a lot of discussions going on in Europe to come to a real decide what your ability not only if we can prolong the lifetime of our structures, not just take the one solution or the other or providing in some way. We need really to come to decide for durability and a very good example in deciding for durability is this story about the tunnel project where they used the multibarrier protection strategy. And this starts with the structure layout of the structure and ends with the use of material multibarrier protection system as I mentioned means what is a very high quality concrete, high cover, 2 inches or more, and use of epoxy-coated reinforcement and in this case because we have the very good cages provision for later CPV if necessary. One of the major concerns in Europe brought forward by the owners and by the authorities is when looking at epoxy-coated reinforcement, is it really wise to change the electrochemical barrier we have normally if we embed bare steel into concrete, we have that electrochemical barrier, the steel is protected against corrosion by passivation, and this is an electrochemical barrier. And if we use epoxy-coated reinforcement we change this electrochemical barrier to a physical barrier. And we need to follow quite different design principles if changing this protection strategy. This is really a basic question. If we change or replace the electrochemical barrier by a physical barrier we need to accept that some of the nondestructive testing methods and some of the electrochemical repair methods are very difficult later on because we have electrical isolation between the electrolytes and the concrete. This must be taken into account.

And another approach if we have very severe environmental conditions and we know normally we know looking at the structure, evaluating the structure, and that's another part of the decide for durability, we know that the areas, the sensitive areas where we can expect trouble, we know them. And why not to use replacable structure elements where possible, where we have very high environmental attack, for example, side barriers in the bridges, for example, it is very easy to construct them with replaceable from the beginning and then we need not to consider whether to use epoxy-coated reinforcement or not , or need not to discuss what is the prolongation of the service life its almost better to use epoxy-coated reinforcement because it will prolong the service life of this highly attacked structure elements and the situation is improved. Or we can try to separate environment and structure where possible, or we can even from the beginning consider local CP. Well thank you for your attention.

QUESTIONS

1. John Wayne Iowa DOT - With reference to your multibarrier strategies, where would you put to use of corrosion admixtures in the concrete in that senerio?

I don't know whether you know that the question was where I would or what is my opinion about corrosion inhibiting admixtures. We and at least in Germany, but in most of the countries of Europe we do not use corrosion inhibitors at least not the ones being used until now. These are anodic inhibitors and the problem of them is that they only can be active if they are soluble in the pore system, and being soluble they are liable to be washed out especially if you have cracks that this are the most sensitive areas, the most critical areas in the structure and and reinforced concrete is a system that has cracks by definition at especially in the region of cracks, we have results at this time that the inhibitors will be washed out and these concentration differences inhibitors are washed out and chlorides penetrate. And this concentration differences of inhibitors may cause very bad microcell conditions with very high corrosion rates in the region of cracks. So until now at least we do not use them and the discussion in the European standardization work at the moment is not to standardize corrosion inhibitors.

2. Ted Neff, CRSI - Were your comparisons of US and German powders, did you use US steel versus German steel because the chemistry could be different?

The bars we tested came from a American plant and were delivered to us. It was American steel and American powder and American application.

3. John Broomfield, Corrosion Development - I sure everyone was very interested in your story about bridge description. Obviously if you're not bending the rebar you don't have to

improve that requirement in either your specification nor in your performance control. So the world of powders is selected on a different basis if it won't need to be flexibilized and you can probably get farmore clearer in the coating by using a pipeline coating rather than the one that has been modified for bending in a normal rebar application.

The question was whether our requirement for the historical tunnel project where the rebars were bent before coating were different compared to the requirements for ordinary process. The answer is no the requirements were not. I have proposed a change to the requirements because I am of the same opinion that you that we don't need the requirement for bendability for flexibility in this way we would have been able to use powders less flexible and in this way basically less permeable. But this would have been a better specification, but they haven't done it.

A.5 THE FIELD AND LABORATORY TESTING OF EPOXY-COATED REINFORCING BARS IN CONCRETE THEODORE W. BREMNER

I would like to talk to you about some tests that I have done, we have done at the University of New Brunswick that involves laboratory accelerated testing and also exposure and marine exposure ______________ which I am sure many of you are familiar with. Essentially we are trying to find out just what temperature you should soak the baby to be in to be able to predict his lifetime performance.

I'd like to stress that like everything else in this world there is a slide show. We'll look at the first slide, and this was applied by a Canadian applicator. This type of leave is run of the mill production. This and similar ones were done and certified by a provincial highway authority as being typical of mill run stuff that goes through the plants. One of the problems in doing this in making thin slabs and accelerating the corrosion process is that you have to have a very uniform cover. When you make these timed typed of specimens you find out that the bar is not in one plane and after it was bent it then has to be bent again at the lab in a very careful way so that it does in fact give you the right cover.

The bar is 16 mm in diameter the cover is 20 mm both on the top, and on the bottom, and on the sides. This is within $\pm$ 2 mm. The size of the concrete that we will make is 15 mm thick 200 mm wide 300 mm long. Here is another view of it. And we believe in making lots of them. We have in this series here I am talking about 48 specimens. We have a great deal more. Some of these here were accelerated in a system that we have used for the past 17 years in protecting various types of concrete, supplementary cementing materials. Essentially we run similar kinds of tests on concrete to sort out what types of mixtures we should put in our marine exposure site so that you don't get either all your specimens dissintegrating or all of them remaining perfect for the lifetime of the people who might be interested in them. Here is a list of the specimens, and you will notice that we have a sea water simulated test which is the marine environmental simulation thing and I show you a little later on. This is series C. We have uncoated bars, we have bars with no damage, and we have bars with 1% and 2% damage. And we will as we go along show you just what the nature of the damage is.

This is the device that essentially the specimens are up in the air and this is a holding tank for ater down here and we put the water up here to give it a wet cycle. The temperatures that are involved in the hot cycle it is 2 hours in water and the water is at 32°C and then when it drains out it is 4 hours air drying at 69°C. So we have 2 hours and 4 hours so we have four cycles per day, and 32°C when its wet and 69°C when it is dry. Here's what the specimens look like, and as you can see these are ones that have corroded. This is another series we use in this work a water cement ratio of .6 type I portland cement, the maximimum aggregate size is 12 mm, and just some of them are put in a freeze/thaw condition. We have between 5% air. They're normally cured for 2 weeks in the standard fashion with curing.

After it has gone through the series of testing and as you can see here the top of the specimen here its stood up in the accelerated testing system and this is high water and this is low water. So low water circulates back and forth. Normally if we do this here it with concrete we get a 4-1 ratio of what happens at Big Island and what happens in our laboratory. In this systems it appears to be about 10-1. As you can see a person can split the specimens open and look at it and what you see is what you get with _____ bond. And looked at it here the slab of concrete has been sawed with a diamond saw both sides and then you break the piece off. And here is what the surface looks like after its been in the mats for something like 24 months, and here again is what you have, and here is a sample of epoxy-coated rebar and a piece of bar that is not coated. You can see that there is a significant substantial amount of corrosion occurring here and there is here the epoxy here there's essentially very little damage on this bar here. How do we make the damage to the bars? The bars have 7 for 1% damage, we have 7 pieces where the coating is taken off, and you can see it is cut off here in a square pattern. Here and there's others up here I believe and there one that would be down on the other side here.

If you look at the bar after 2 years in the mats, now you can see in some instances there is essentially this is the part that we file off, there is not damage; and then some other places there is a small amount of damage. And here again you can see how they are distributed. There is some with damage and some without. Here is the imprint where an uncoated has been placed. So obviously we want to look at what's happening to these specimens. We put electrical connections to them. We do half cell and linear polarization and a limited amount of ____ has worked on these specimens and we will just print off the slides, take a look at the transparencies.

This is our standard setup for testing the corrosion rate. The counter electrode is a 15 mm diameter stainless steel rod and I think you will perhaps see it in one of the other. Here is our polarization resistance readings for the uncoated bar and you will notice it is incredibly large for the epoxy-coated bars, and that as you go to more damage the resistance decreases, and you can see in this line it's 1. If you look at the number you can see how the polarization resistance decreases with time and there will be a graph showing this a little later on. Also the corrosion rate is shown here .32 in microamps per cm^2 we have after 7 months .32 and after 24 its about 10. Now we are looking here at uncoated bars this column here. You come over here and look at this column this is what we're getting and you can see the array of 0's. So we're not able to measure the corrosion rate after 24 months in this very aggresive environment. If you will look at 1% damage we're getting to the threshold level when it's a little better.

We think that it's starting to corrode a bit here. When you go to 2% damage fortunately you get a little bit more corrosion as one might expect. Let's look at the next transparency. This is the open circuit potential and if you'll look at what's going on here at 500 let's look at the one with the undamaged epoxy. And you can see its right up here, if you remember how we have some with 1% damage and 2% damage and you'll notice it starts moving down this way when we start looking at the next 2. Next one. And here you can see its now with the 1% damage coming down to where there might be some corrosion. And obviously there's potential for more corrosion with the 2% damage.

I'm sure you are interested in how stable is the system and you can see here that things with the uncoated bars is getting worse and worse and things are decidedly unhappy, and with the epoxy-coated bar the polarization resistance is staying essentially constant and its stabilizing in this aggressive media and this aggressive testing. And the 1% and 2% damage are obviously corroding. Now it's obviously unwise of me to say that if you have 1% damage for you to get 1% of the corrosion rate you would normally get with a plain bar. Obviously we need to do more testing. But right now it doesn't look as though there's a great deal of difference if you've got 2% damage you'll get a little bit more than 2% corrosion of what you would get with a plain bar.

Now we have obviously had to package this thing here so that the people at the University can get degree. More essentially it was staged where we like to do a lot more work on it. Everything has to be done in terms of students and that sort of thing.

Let's take a look at the next one. And these are some tests on the coated and uncoated bar. And this is the ACMP's work that we're doing and has been done by a man named Steven Clark at the Research and Productivity Council. It was after 7 weeks in the mats and you can see that one of the specimen's is corroding nicely and one is getting ready to. And the numbers going across here with the epoxy-coated bars relative to the small.

The US Corps of Engineers has an exposure site at Treat Island, Maine and they kindly let us put the specimens there. If you will look at it this is the site that most of you know and where the 10 mat specimens are. And here are the specimens at mid-tide level and here are the specimens at the high tide level, above and below high tide level.

Basically, what we're trying to do to get some system whereby you don't have to spend an absolute total fortune to do this kind of work. And really what you want to do is to be able to put the specimens here and leave them for 2, 3, or 4 years and then be able to come and very economically find out what's happening in comparison. And that's what we're going to do right now.

Let's look at the next slide the tide is coming in as the people are getting ready to go somewhere else right now. You can see our specimens are here and when the tide comes in the tide comes up to right about here in about another hour. Here are the specimens that had treated irons with the Corps of Engineers with people there are maintaining this it would be very, very difficult to do this kind of work as you probably realize those slabs of concrete would make a great barbeque and they're tied together in the hopes that people will not untie them and take them home. The island is relatively isolated you probably couldn't do this kind of work unless it was an island that was relatively difficult to get to and it is a great facility, it is also very easy way to have freezing and thawing. At this site, I mentioned the temperatures that were involved here it is very close to Canada, it is very obviously cold and when the tide comes in you normally in the wintertime you get a cycle of freezing and thawing or a cycle of freezing and normally we have about 100 cycles of freezing and thawing here. Here are the specimens inside diameter, pieces of reinforcing bars, epoxy-coated and not epoxy-coated. Here is what you get after 2 years. And again, I would like to point out I don't say that this what you would get with all the epoxy-coated reinforcing bars, but this is what you get in a real life situation if you'd call these slabs in that situation a real life situation, this is what you get. Here is the epoxy-coated bar with 2% damage, 0% damage obviously are the ones that are as received, and these are the plain bars. Let's look at them again. And some of them are mid-sized level. We did corrosion rate measurements on these and generally the corrosion rate at the high tide level is about double what it is a mid-tide level. Here's some more specimens. Now when you look at the specimens generally and compare them you're hard pressed to say that there's a lot the matter with this one. It's very difficult to find anything. This is after 2 years water cement ratio of .6 and this is what you get with the same kind of concrete cast the same day out of the same mix and there's a fair amount of corrosion occuring there as you can see. We have not actually tried to measure the depth of corrosion here, trying to relate it to a corrosion rate measurement, but that's something in the future. We also have not broken the rest of this concrete off we thought this made a nice picture at this stage. We have electrical connections here to do corrosion rate measurements on these things.

Here are other specimens. Here's a rather interesting one. To the best of our knowledge it is the only specimen that we have that we had trouble in the casting process. And you can see that somehow the cover here is almost 0. This after 2 years at Treat Island and this specimen did not perform significantly different than any of the others in terms of the corrosion measurements. And here you can see the corrosion being a little more uniform than it was on the other bars. The bars were all the same, all cut at the same length of bar, this was a bar from the Fredrickton, New Brunswick fabricating area. The epoxy comes from Ontario.

As you can see the fine print got me into a lot of trouble. And here we look at polarization resistance and would you believe these are the uncoated bars here, here, and here. Here is a, and this after 12 months, here is a bar with no flaws, supposedly, this is one with 1% damage, and this one is with 2% damage. We move across from here this is mid-tide level, this is below high tide and above high tide. Let's look at the next one and you can see what happens in 12 months. This is the end of 2 years. Let's just go back and you can look at this point here and you see it has dropped down to here. And these you can see has not changed much. Essentially and again no one really knows how quickly things are going to change and what the right temperature is for that baby, but I can tell you a lot of things don't appear to be happening to the specimens. I thought it appropriatly also that I should mention to you that there is a series of specimens at Treat Island with epoxy-coated bars that were put there in 1991 and these were a CAN MAT 3M program and we have here .45 and .6 w/c ratio. These things were placed in 1991, obviously nothing has happened to them yet that we can see. There are also at Treat Island there is these here, we have how many specimens here, there is 16 prisms similar to what we have here and with this cage that is completely independent of these other 2 the cover is 20 mm. We also have some with a cover of 70 mm. Obviously in 3, 4, or 5 years there needs to be some work done to see what kind of corrosion is taking place on these. We also have black steel, and also we have in terms of corrosion a regular steel in 1987 Can Mat put a series of 8 prisms with a w/c ratio of

.5 at Treat Island, Maine and the various mixes that they have two of them are 50% slag, 3 of them were at 25% fly ash and 4 are with sorry. There's 2 each of 50% slag, 2 of 25% flyash, and 2 with 10% silica fume. The cover again here is 20 and 40 mm. I can assure you that the cover and everything here is perfect when you get that other form there because the person who is doing that is now a brain surgeon and there's nobody that was fussier than him.

I'd like to tell you a little bit about another program that we have that I think is important. In 1967 a president of the University of the University of New Brunswick thought that it would be very nice to have a heated side walk. Underneath this sidewalk are the heating and cooling pipes and electrical services for the University of New Brunswick. It gets very cold there as you might imagine and the collapse of that slab a lot of bad things would happen. This is built in 1967 and by 1984 this had to be replaced. Now as you look at the concrete by and large there is very little freeze thaw damage, there is some structural cracks because of the pressure here from the roadway cracks and lack of top steel, but the concrete by and large is durable. You think about this room underneath if you look up at the ceiling I think this is what you will see, not quite. Here's what you see inside and that's what you would see in 1984 after 17 years the bars, many of them, were stuck in at that corner and stuck in at that corner and sagged down like this. To keep the University going they had forest of 4 X 4's in the tunnel propping the thing up. It was obviously a failure. I was not in a position to criticize the original design considering that the design was done by a firm owned by my boss who was the Head of the Engineering Department.

In the repairs they thought that maybe the best way to save face was to simply go to something different and that's where epoxy-coated rebar came in. Here's more of the problem, and there were, in fact, worse areas. Here they are cutting the slab out and you can see the bars there with completely bare. Now these slabs by and large work very well and the concrete was durable. The salt went through, corrosion was severe. Underneath, if you looked up you could see the white deposits. Fortunately I only need 2 more minutes.

Here is the epoxy-coated rebar and in spite of the fact that there man is going with a sledge hammer and looking as though he is going to make some repairs, work was done with care and consideration. This tunnel was built in 1984. The first month that it was built it developed horizontal cracks going across this way in the vertical plane about every six feet and the cracks were due to temperature and shrinkage cracks and you can see this bar right here, they intended to follow those bars, certainly in one instance. We have come back to epoxy-coated bars after 9 years and have cored slab here, cored it here, and have made measurements on the corrosion rate.

In the bars after 9 years, I believe that the concrete is not a great deal different than what was built in 1967. The only instance that we have of corrosion is in one of these bars that is laying right in the plane of the crack and in which if you look on the underside you can see a white deposit of salt and we tested and it is sodium chloride, and that bar, when we took the core out there was some corrosion and the corrosion was occurring where a piece of the bar had been patched. So it was under a patch where the corrosion had occurred. This is a little advertisement for our conference in 1996, the Third International Conference of Canada of ACI on the Durability of Concrete in the Marine Environment. It's here, held at St. Andrews, and Treat Island is right here. Hopefully we will have more information for this work at that time. If you look at our accelerated testing in the mats we expect to break open the 5-year specimen in 1994. Now if you look at this business generally what's happening in this accelerated testing thing occurs 10 times as fast as what's hapening in Treat Island, which we think is a realistic exposure site.

Now in 1994 we will have 5 years, and if you multiply that by 10 I will probably be able to give you a typical University answer as to whether the stuff will last for 50 years. It'll probably be yes, no, or maybe. Thank you very much.

QUESTIONS

1. John Broomfield, Corrosion Control - Ken Clear spoke about the loss of adhesion of the epoxy-coating to the rebar. In any of your post-mortem analysis of your specimens have you looked at that or do you propose to look at that property?

Thank you very much I think that is a very important question. We have not done anything on it. We simply go to a certain stage and our next stage is to make the little X and start peeling on it, photgraphing it, and looking at it. It is rather interesting in tearing the pieces apart. None of the epoxy-coated bars were damaged in the breaking of the pieces off on either side. I assume its reasonably well adhering for both the mats work, and for the stuff at Treat Island.

2. Could you make a comparison on Canadian specifications and American specifications on bars, are they the same or what?

I would say that they are exactly the same. We don't keep free trade to keep that going. We use ASTM standards for epoxy-coated rebar, and the people who supply the resin are a large North American companies that are based in the United States so it essentially, I think, identical. Also, these coaters belong to a North American Trade Association so I'm sure that it is identical. I think it just happens, maybe, then again I say I'm obviously, I think the system works, but I think it only works for the particular situation that we have here, maybe, and I think that there are producers in the United States that are equal to the producers in Canada.

3. Don Pfiffer, Wiss, Janney - Your Treat Island exposure is a very cold exposure, typically on an annual basis your bench tests is a very hot exposure. Your 60°C is like 160°F so that specimen is cycled from 90°F to like 160°F, or something like that, I don't know what it is, but it is a very hot environment, versus other things that have been done in the past. During the same period of time have you noticed any difference between the very cold specimen at Treat Island and these very hot specimens that are going through your cyclic tests?

I would add a third one, the tunnel. The tunnel does not freeze. Subject to a lot of salt from the sidewalk being splashed from the roadway as the cars go by its a steep hill, it is heavily salted and the water is frozen. So that goes from 0°C to perhaps 35°C. I think that the tests that we plan to do will reveal whethere there is, in fact, differences. I think that's yet to be determined. We have not really autopsied the bars other than to break them open and look at the corrosion rates.

REDIRECT: But you have been measuring currents potential during this period of time and they have been significantly different.

Well you see the rate of corrosion in the bench is 10 times what we have at Treat Island. So in 1994 we will be breaking them open and we will be looking at them and if the epoxy bars still not corroding, a person might be tempted to say that this will last for another 50 years. Maybe. Now that may be our best guess as to what's going to happen, now I don't know. I can give you an answer in 1994 perhaps, but it will probably be qualified.

GENERAL QUESTIONS:

1. Bill Hart, Florida Atlantic University - I will give a beautiful vocal description of my transparency. We also have placed a lot of emphasis on the hot water tests, as have Peter, although after his baby analogy, I'm not quite sure what he places his emphasis on. But, it is not clear to us that the blister versus no blister criteria or that the time to blistering is an appropriate parameter published to focus. And the reason for that is that many of the bars that we perhaps don't have a lot of confidence in for other reasons, have a lot time to blistering in the hot water tests. On the other hand what we do focus upon is adhesion of the coating after the hot water tests and more specifically the recovering of adhesion in the drying time after the hot water tests. It is generally known that epoxies loose adhesion when they're wet and the parameter is to what extent they recover that adhesion as they dry. If there's corrosion activity during the time of wetness there's less of a tendancy for adhesion to recover. We have some examples where adhesion recovers as the coating dries, others where it does not. But in a 2-month outdoor exposure period for a number of bars from different sources one of which was a marine exposure the other was several miles inland, it wasn't clear whether adhesion recovered or not. And this would be in the temperature not hot water. However, the adhesion values we measured on those atmospherically exposed bars were roughly about the same numbers as the adhesion values that we get after the hot water test. But the interesting point is that the adhesion values that we measured are relatively small fraction of the adhesion of the original bar prior to exposure, in fact we buy our test technique, which incidently is a mechanical quantitative mechanical testing technique, we are really unable to measure the adhesion of the original bars. We get failure at the adhesive, within the adhesive, or at the adhesive preventer base. But the adhesion after any of these tests is considerably less than the original adhesion, and that really brings us to the point of focusing upon adhesion and the loss of adhesion. During either the hot water test or relatively moderate at the sphere of exposure.

Basically, I very much agree to your statements and comments. As the other accelerated tests, the hot water test gives one result and we always need to try to get a set of various results. We have done similar testing of the adhesion before the the hot water test and after that and after drying and basically found the same relations as you reported about. What concerns the repeatability of the hot water test we found very good results. That means a good relation of underfilled continuation on the one hand and a kind of mystery, and a not too bad correlation between concrete blistering and the adhesion after the hot water test. If I understood your comment correctly you did not find this correlation.

RESPONSE: Well we really were not looking for that correlation but essentially in the 7-10 day test we typically don't see much blistering, but we see significant loss of adhesion.

ANSWER: That's correct, yes, but the requirements we set up because the adhesion test needs some records afterward. The requirement we have come to use if the adhesion goes completely we don't close the glycerine criteria.

RESPONSE: What concerns us is that the bars would pass the glycerine criteria but at the same time they've lost considerable adhesion.

ANSWER: Of course, yes. In any case I think we need to concentrate much more on the change of properties under certain type of exposure, not only on one certain fixed observation at a certain time, lets say a certain current criteria when we do microcell current tests, or a certain adhesion criteria after certain periods of time. Development of properties with time gives us much more information. If we are forced to do short term tests alot, accelerated tests in just one fixed area because its always the question where we started from, and how we have reached this minimum level.

2. I have a question for Peter here. I didn't understand the

CANNOT UNDERSTAND THE SPEAKER.

ANSWER: If we improve quality in general or normally we will increase cost. But if we compare the cost for later repair with the slightly increased initial costs then it is always a tremendous benefit to input a little bit more money in the beginning. This is the first comment on the last part. Multistage protection or multibarrier protection system I think seem to be wise as we have to realize that under very severe environmental conditions we infact do not have a protection system now that really works. For example, look at the Florida Keys problem. The concrete there, I would say is a good concrete. What cement they show if my information is correct in the range of 0.5 is not too bad. Cover in the range of 2-2 1/2 inches I think its good, but very extreme environmental conditions. If we have tidal sub, high temperatures all over the year, concrete structure elements partly immersed into the water I don't know any system providing us or with the long term durability of 50 years. It doesn't exist. So we should start when designing a structure, we should start from the beginning considering possibilities how to react if something goes wrong. This is what is meant with multibarrier protection system. And if we have this very severe environmental conditions and if we cannot separate environment from the structure and this is very often the case that we cannot very often we cannot, in bridge structures, for example, most of the cases can separate our structure from the environment from the chloride and its much cheaper than later repair. But in a column of a bridge structure in sea water, we cannot separate the structure from the chloride containing environment. Why not to forsee from the beginning the installation of local CP. For example possibily in combination with epoxy-coated reinforcement, but then if so we need to consider from the beginning the electrical coupling of the coated bars. This is the type of thinking I think we need to start with. Neither in Europe or in America this thinking has come through to the Engineers.

RESPONSE: I think it is very true to say that on the surface it appears as though this Florida situation is normal and that something happened to the epoxy. You have to start thinking a little bit about a thing called olythic lime rock. And its not the kind of aggregate that I have any experience with and it would be very unwise of me to say anything negative about it. But I think it is a bit different aggregate than a lot of us have had the experience with. Also, it has a lot of chloride in it whether these are bound or free I don't know but I'm sure somebody here in this audience does know and can offer an opinion. The other thing is I have used epoxy and have seen 10 shipments of epoxy in Canada and in Canadian situations, and I have other shipments of epoxy in other countries that was markedly and dramatically and significantly different in terms of visual appearance. I have also seen situations where epoxy was being put into concrete and there was a lot of epoxy missing. I have also seen as I travel around in situations, and I am not saying where and in what country and who's involved, but believe me the halo effect was in place for a long time and there were a lot of bad things happening. Now before you go through a belt and suspenders situations you ought to check that there is the person and there's nothing sexist in this, has a belt. You should make sure that at every instance that he had a failure that the belt was in fact actually buckled. We don't want to have an unbuckled belt and say the man had an accident.

SECOND RESPONSE: I'd like to make a comment about the obviously you have to have the philosphy that everything is going to corrode under some conditions and reflect back upon again the 1974 NBS study. They studied 44 materials at that time and not one of the 4 materials that were finally suggested as being the best, not one of those 4 epoxy materials went through those NBS tests without some corrosion. The only ones that went through that tests were the 25 and 26 mil vinyl materials or pvc materials at that time, but they were rejected for other reasons, but, you know, you have to realize that everything is going to corrode under certain conditions. None of those materials that were judged to be the best came through scot free. They did not.

THIRD RESPONSE: We've done everything we could do to extensively study Florida aggregate and find absolutely no effects of that aggregate on the corrosion situation in Florida. It hadn't been totally dissolved yet, but it was gone to the point where we recommended in the NCHRP panel agreed to not make the specimen be ___________ aggregate because its too costly. In my opinion that has absolutely nothing to do with the phenomena that has been seen. I'd like to talk about belts too because I've had to get a little bigger one recently. I disagree very much with the philosphy you brought out and agree with the philiospy that Peter brought out. As engineers we cannot hope we have a belt that works and say awe shucks, when we find out 10 years later that it's only going to last 10 years rather than 50. Our profession is based on public trust and therefore it's our responsibility to have a significant belief that a system that we place a 50 year life upon is going to have a life near 50 years. The mistake that was made in this profession with epoxy-coated bars in the 70's was wishful thinking. What you then say was wishful thinking. You think that your bars are better than my bars or Florida's bars, therefore your going to use them without other protective systems. I used to think that my bars were better than Florida bars, and therefore I recommended to the world that they use epoxy-coated bars alone. I was wrong when I got in and did detailed evaluation and testing. I must say that I wonder whether you will be wrong when you get in and do the necessary detailed evaluation and testing. If you are, and people have relied on you to build another 100,000 structures that are going to deteriorate prematurely, shame on all of us in my opinion. I think we have to be conservative with these systems, we have a public responsibility and there's nothing that I know of at this point in time, 50% of the bars we tested in the bent bar study come from coaters in the United States had 0 coating breaks, had the proper coating thickness as measured by the specifications between ribs and yet 68% of those were cracked in 3.5 years due to corrosion damage. All of them showed damage just in the southern exposure that I didn't find even when we did the studies until I got Peter Scheisel's report that said in my opinion there was damage going on even in southern exposures. So we went back the retained bars that we had kept for 2 years and checked them, and we found out that yes in fact there was damage going on. We're in a situation where the harder we look the worse it begins to appear in my opinion, and I think we have a responsibility to take the conservative path down to the process. I'd also like to say that yes, everything is going to corrode, but just because everything is going to corrode sometime doesn't mean that our engineering responsibilities can be dropped. We still have that engineering responsibility to the public.

RESPONSE 4: I in no way want to imply that anything will stop corrosion for ever in our situation with the epoxy-coated rebar. I think that we all live in a world where that system has got to be designed for a particular length of time and there is uncertainty in predicting the future and I recognize it is in this. I didn't realize that what I said was at variance with Don Pfiffer at all. I do however, believe that some of the systems seem to be working in a satisfactory fashion. Some of them don't. I believe I have been quite clear in my conversation and anybody who infers otherwise I think is not being fair that the Canadian bars are better than the American bars. I've been too long alongside the Americans to ever make that suggestion. I think that we are at a time when money is an issue. It's important to find out what works and what doesn't. It's important to screen out what doesn't work and what does work. And what's economical and what's not economical. And that, sir, is all I ever said on that subject.

EPOXY-COATED REBARS FOR REINFORCED CONCRETE STRUCTURES, PART 2

PETER SCHIESSL

A.6 LABORATORY AND FIELD EVALUATION OF EPOXY-COATED REINFORCEMENT
JULIO RAMIREZ

I'm going to talk to you about the performance of concrete bridge decks and slabs with epoxy-coated reinforcements from the structural standpoint, which is my background. For that reason also, this morning's session was quite a bit interesting to me a bit of a learning experience with regards to the issue of durability where epoxy-coated reinforcement is involved.

Today I will be reporting to you the results of a 3 year on-going research study at Purdue University sponsored by the Federal Highway Administration and the Indiana Department of Transportation. This study consists of 2 phases, a laboratory phase and a field phase. The laboratory phase is mainly addressing the structural performance, and the field phase we've walked 5 bridge decks in the State of Indiana and tried to assess the condition. So if I could have the lights and the first slide.

I'd like to acknowledge the graduate student working in this project. Mr. Hassan is a doctoral student, and as you know he is the one who is really doing the work. First the laboratory phase. Today we've tested 24 slab specimens. They've been divided into 12 sets of companion specimens. Of those 12 sets, 6 have been reinforced with #7 bars and the remaining 6 with #11. Each set consists of a specimen with coated reinforcement and a comparative specimen otherwise identical but without coated steel. They have been tested under repeated loading.

As it was mentioned this morning, whether the work is being conducted in the area of bond with epoxy-coated reinforcement at North Carolina State University, Texas, Kansas, and we've done some work at Purdue as well, most of the work has been ____________ in nature and the findings from that work are outlined here. First of all fewer and wider cracks have been observed in specimens with coated reinforcement. There is little difference in overall deflection between companion specimens, coated and uncoated reinforcement. The main difference has been observed with regards to the bond strength reduction. Bond strength reductions between 15 and 50% have been observed mainly with splitting as the critical mode of failure. This has been mainly we believe due to the loss of friction from the presence of coating and this has resulted, as you all know, in new ACI and AASHTO requirements for anchorage of epoxy-coated reinforcement which require longer development splice lengths for this report.

So our study was motivated the laboratory phase in trying to assess the effect of repeated loading, cyclic load on the structural performance of members reinforced with epoxy-coated steel. The structural performance both from the standpoint of service load behavior cracking inflections as well as from the ultimate strength standpoint. I hope that this information provided from the structural standpoint will be useful to those of you interested in the durability issue.

The specimens that we tested were lab specimens simply supported. You can see there the 2 supports and the specimen actually has a couple of overhangs. This goes between supports and overhangs centerways support and overhangs is 4'. The load is supplied at the end of the overhang, it is symmetric loading therefore it produces a constant loading region between the 2 supports. The specimens were cast in groups of 4. Each group of 4 consisted of 2 sets, each set was 1 was reinforcement #7 and the other with #11 bars there. As you can see there within each set you have the companion specimen with uncoated steel. The cross section of the #7 specimens were 8 X 24 in, the #11 were 12 X 28 in, and the length of the splice for the #7 was 12 in, for the #11 was 23 in. They were designed so as to fail in splitting mode failure to properly assess the bond strength.

The concrete strength target design strength was 4,000 psi. We used the clear cover 2½ in as that is the minimum cover allowed by the Indiana Department of Transportation in the bridge decks and slabs. This is a closeup of the splice region; this is what's located in the constant movement region of that slab, and as you can see there, each specimen had 3 bars, and in this case we are looking at one with a #11 and the splice length shown there in the constant movement region also shows transverse reinforcement. The transverse reinforcement consisted of #3 bars of 6 in also.

The test procedure, first we loaded the specimens with 2 or 3 initial monocrotic cycles up to the peak stress of the repeated loading portion of the test. The peak stress we tested 3 different levels, 24, 30, and 36 ksi and this is mainly to establish the effects of what would be about 6/10 of FY maximum service load stress. The stress range, we tested 2 different stress ranges, 8 and 16 ksi. After initial monocrotic cycles where we basically established the criteria, we took measurements in terms of strains and inflections, we proceeded to load the specimens up to 12 of 1 million cycles in blocks of 100,000 cycles. At the end of each block of 100,000 cycles we conducted a monitonic load cycle up to the peak stress and again we took the measurements in terms of strains, deflections, and cracks.

This shows the crack pattern for an epoxy-coated specimen. This one was #11 and you can see here the cracking in the splice regions, the splice is shown there. You can see there the splice region and then the cracking in the slab. What I want you to also notice is the splitting cracks that then led to failure in this specimen. When you compare this cracking with that of the companion specimen with uncoated steel again you can see that there are more cracks with the uncoated steel as was also observed in the monotone tests.

Tests results in terms of crack widths, the average flexural crack width after those 2 initial cycles was that #7 bars it was 26% greater for the epoxy-coated bars specimen where the #11 was 23%.

So again as previously observed in the monitonic load tests specimens with coated steel have wider cracks but also have fewer cracks. What is interesting is that after 1 million cycles there was no change between those 2 ratios. That doesn't mean that the crack width didn't increase but they increase at the same rate.

Well if this had been placed properly it would seem on what is now the horizontal axis the total deflection and on the vertical axis the number of cycles. In regards to deflection remember we mentioned that monitonic load testing showing essentially no difference in our tests this happens to be for a #7 specimen and at the initial portion of the test there we have 5% difference in deflection and at after the 1 million cycles the difference essentially remained the same. There was an increase in deflection but the difference between the coated and uncoated specimen was the same. For the #11 specimen we have total deflection here and number of cycles. Initially we also observe about the same difference, but then at failure the difference at the end actually seemed to decrease by between the coated and uncoated. In other words, the repeated loaded portion of the tests was more damaging for the uncoated steel there was more damage in terms of adhesion and friction which was nonexistent for the coated specimen after the initial cycles.

Test results in terms of bond strength failure values were pretty much along the range of those reported in the previous monotonic series of tests. We had a range between .73 and .95. What this is is basically the ratio of the bond stress at failure of the coated to the uncoated specimen. Okay, so that means that the coated specimen actually carried less bond stress at failure. The average was .83 for the #11, the average was .79. This reduction with the bar sizes is a bit misleading. Remember that the clear cover is the same, which is 2½ in. And the critical parameter in the failure of this type of specimen which is splitting mode is the cover to bar diameter ratio. So the #7 bars have a larger cover to diameter ratio than the #11 bars.

Also in the test series only 2 of those 24 specimens failed prior to that monitonic cycle to failure, in other words, I don't know if I explained this correctly. After the 1 million cycles if the specimen hadn't failed, then we would take it monitonically all the way to failure. All the specimens but two, both with epoxy-coated bars failed after the 1 million cycles. The 2 that I have here failed 1 the #7 bar at 600,000 cycles, the #11 bar at 300,000 cycles. The reason for that is that the deep stress that we took this particular specimen to was very close to the failure stress of that type of specimens so we were cycling very close to its failure stress level.

Conclusions, well, basically the effect of the repeated loading is one where within the stress ranges that we tested in the, I think that's important because we were trying to represent stress ranges in bridge decks not what you would for example expect in the case of an earthquake in a building, here we are talking about a large number of cycles in the stress range that there's really no difference in terms of bond strength as we showed there. If anything the repeated loading portion of the test effects more of the uncoated specimens than the coated specimens because you take care of their adhesion and friction that initially is there with the uncoated bar. You still have fewer cracks and wider cracks with epoxy-coated steel, but the difference is as we showed, tends to become less when you start applying repeated loading.

Here before I go into the laboratory phase I'd just like to point out that as you all know from what you've been listening to the durability of coating needs further research.

In the field stage we evaluated 5 bridge decks in the state of Indiana. The evaluation consisted of a delamination survey, crack mapping, core and chloride samples, concrete concrete cover both from cores and from using the R-meter and coating in regards to condition and thickness of bars striking from the back. Factors that we looked at environmental factors, we tried to when selecting, and this was selected in conjunction with the Indiana Department of Transportation, select a cross-section of environment in the State of Indiana. So you see we have bridges in the south of Indiana, in the north, and somewhere in the middle here. Traffic we also tried to address different traffic environments, heavy truck traffic in the northern part near Gary, Indiana, urban in Indianapolis and so on. The degree of salt application also in this sense Indiana has a reputation for sometimes being on the heavy side in the application of deicing salts. Storage methods - local practices and specifications and we also looked at coating process and type.

I'm just going to discuss today due to the time constraint one of the bridge decks we evaluated in Indiana. This one was the first one and it was located in the city of Indianapolis. It was a six-span, continuous composite steel-box girder bridge. The maximum span length was 206 feet, it was subjected to early heavy urban traffic its downtown Indianapolis, severe salt exposure. It has 6 lanes, and in all the evaluations we only looked at the outside lanes for obvious reasons.
This is the bridge in question. This actually represents the case of a concrete deck on a flexible structure so we almost expected to find the largest amount of cracking in this particular deck and that was the case. Most of the cracking is perpendicular to the longitudinal load carrying member to the box section. The maximum width of the average crack that we measured was .016 inches. We didn't find any signs of delamination or corrosion in the bars that we extract from the deck.

This table here summarizes the results of to date of our field evaluation. On the far column you have the bridge type, just to give you an idea of the age, Indianapolis one was built in 1985, South Bend, which is in the north part of the state 1983, this one is just outside of South Bend 1980, this one is in the southern part of the state 1985, and this one which is in Gary, Indiana, northern part of the state, 1980. Now those 1980's gives you 12 years which is about when they started using epoxy-coated reinforcement in Indiana in bridge decks for about 12 years. The average concrete strength that we obtained from our cores ranged between 5600 and 6000 psi. This was 6 inch cores, 1 to 1 high to diameter ratio. The average calculated cylinder strength again you see is between 4800 all the way to 6000. We measure an average cover ranging from between 2.4 as the lowest to a maximum of 3.82. The chloride content is a powder test. Again we looked at different levels. As you can see if you look at the northern bridges which is the South Bend, south of South Bend and Gary, Indiana, those would show you you would expect the largest amount of chloride content. But at the level of the steel which would be for most cases between levels b and c except for this one which is a little on the high side ranged between here at 2.1 and 1.5, 7 and 4, 12.15 and 3, and 3 and 1, finally we have this one here between 4.9 and 3.25.

Now I'd like to tell you a little bit about the Indo practice in regards to the use of epoxy-coated reinforcement in the state. Indiana has, this is the Materials and Test Division, has a practice of every 3,000 lbs of epoxy-coated steel that is used in Dansing, Indiana is evaluated. They take a sample of it and they run those tests listed there alternate strength ______ 180° bend the formation the coating thickness, and they don't do any checks for holidays mainly because when they start to do that they found so many that they decided it wasn't worth it.

Findings as far as their studies is concerned, coating thickness in general is within acceptable limits and towards the high side considering they were above the 90 mils. They found as I mentioned before a large number of defects. Construction practices, they know that this is what's reported in by inspectors that construction practices seemed to have relaxed over the years. In other words they started very well now they are sort of going down, and hopefully with this popularity of epoxy-coated reinforcement they will start again moving up in terms of quality control. They also know that bars for most jobs are stored over short periods of time.

A.7 MECHANISM OF CORROSION OF EPOXY-COATED REBARS ALBERTO SAGUES

This work 100% is being supported by the Florida Department of Transportation and also by the Federal Highway Administration. My co-author in this presentation is Rod Powers, Assistant State Corrosion Engineer for the Florida Department of Transportation.

This is a picture of the 7 mile bridge which is located in the Florida Keys. This is one out of 4 major structures that have experiences severe corrosion in their substructure as a result of deterioration of the epoxy-coated reinforcing steel. The morphology of the deterioration is seen in these kinds of pictures. There you see the external area of this bond detected by acoustic camera methods. Then if you remove the cover you see start of deterioration. Ken Clear showed some similar examples this morning. Most of the damage takes place in a region that is between 2 and 6 feet above the high tide zone. The rebar cover in these structures is typically in the area of 3 in, there is a little bit of variation, but it has seen damage at depths as deep as 5 inches under these conditions. The concrete at the rebar level has built a chloride contents that have been in some instances as high as 20-25 lbs/cy. We are talking about a very severe exposure condition.

This is a good example of the kind of deterioration that you see in regions where damage has taken place. You can indeed cut along the rebar, open it up and you see it damaged underneath. This is an example of another structure that has suffered severe corrosion. You can see the corrosion in the horizontal rebars, you can see the corrosion in the vertical areas. Now I wanted to point out something interesting. In some parts of the system, you can see the coating, you can go ahead and you can separate it quite easily from the underlying metal. In this particular case you see that the opening metal is clearly not experiencing any corrosion. But the bond between the epoxy and the metal is basically lost. Now we have observed this kind of damage not only next to corroded area but also way up in the substructure, way up several feet say 5-10 feet above the high tide area. We have also seen this kind of disbondment in structures where we haven't built hardly any chloride content yet at the depth of the rebar. Talking about fractions of the concrete may be more than 1.2 lb/cy and disbondment exists. In deep this has been observed in something like 24 of the 25 bridges that we have examined in an ongoing project which we will be talking about in the next presentation.

We have been able to show this kind of disbondment not only be means of a qualitative knife type tests but we have also developed a laboratory testing procedure whereby we can demark a small _____ tip ¼ in in diameter of the coating, attach the dowling to it, cap around the dowling and then put it up pretty much left in a _____ type of a measurement. Indeed we can quantitatively show the level of the field _____. I have a view graph for that that I can show if you have any questions.

Another important aspect of the morphology of the deliberation of serving the Florida Keys is this kind of thing. You can go ahead and you can cut the freshly exposed reinforcement steel bar and you see a liquid oozing out of it. You can go ahead with open up this and what you see underneath is a different liquid, you see corrosion, you test the pH of this material, the pH is fairly low anywhere between you say 4 and 5 in the field, and say between 3 and 5 in laboratory tests which would be a minimum to duplicate this. This corrosion morphology.

If you clean completely away the epoxy and rust and so on you can see that often times you have begun the severe pitting of the reinforcing steel bar.

Let me just very quickly tell you a couple of the observations that we have made in the field and the results of extensive series of laboratory experiments that we have conducted since 1986 in order to observe or determine the mechanism of this form of deterioration.

What you see here in this picture taken at the construction yard very recently this is about 2-3 years old and you see here a fairly large diameter bar, thicker than #10, and you can see that when you bend the material you indeed make them to lose mechanical adhesion between the coating and the metal. This is just simply the differential in the formation. This cannot, I think causes of course a partial failure of the corrosion protection concept. And what is originally thought is that a good deal of deterioration of serving the Keys was due to the fact that there was extensive fabrication in the rebar assemblies for the structures. We conducted a series of laboratory experiments studies in 1987 and we show that indeed you fabricate rebar you are very likely to promote corrosion. This is one of the specimens that was tested at that time. And here you can see that the difference being certainly coated delamination and then underneath corrosion cover accumulation of a low pH and liquids and be very much the kind of things we accelled in the field.

And I don't have the picture here for that but it if bends right here next to the ridge when you deform the material you see a serious of little tiny cracks in the epoxy where the corrosion begins to operate in that region. So in deed fabrication of the reinforcing steel bars was identified by our work we published this in Corrosion 1989 meeting was identified as an accelerated corrosion factor in that performance of epoxy-coated reinforcing steel.

Another serious of experiments that we conducted was caused by deterioration that the deterioration was seen not only as fabricated rebars but also was seen as straight bars but had never seen fabrication. We conducted theses experiments back in 1989 and we have been doing variations of this experiments ever since, which in covering steel bars and we introduce damage on the surface of the material, we expose these parts to a salted environment and then we determine the extent of corrosion in the bars and the extent of delamination of the coating around the regions where the damage has been originated.

The first series of experiments that we did were simply using sodium chloride solutions, nothing else because one of the concerns that we have was that perhaps a good deal of the deterioration of the bar took place right there at the construction yard. This is a sea side type of construction yard environment and you're are going to have seawater mist, you're going to have the surface of the bars in contact with sodium chloride solutions for extended periods of time. And sure enough if you go ahead and you have this type of material with that kind of surface damage you expose it to a sodium chloride solution just simply the unacceptable potential or under relatively mild cathodic polarization conditions you end up seeing damage of this kind. Around each one of the little marks that we created on the bar are of course in one of the defects or the holidays elsewhere in the bar you see these ____ sites regions disbonding. If you conduct this for a short time, the regions have very small, if you conduct this for a long time the regions are larger. We have seen these as I said at the opens of ____ look at these more diameter regions, and if you polarize them to say 750 millivolts negative or so then you see something that these evolve more severe.

We went ahead and we did experiments of this kind with calcium hydroxide solutions. We went ahead, we polarized the material anodically, we polarized the material cathodically, and we couldn't get any of these bond mends. We went ahead and we made pastes using sodium hydroxide solution. The reason for that is that the concrete pore solution, although it is rich in calcium ions, it is also normally quite rich in sodium and potassium ions. We saw that then under cathodic conditions the sodium hydroxide solution created delamination was almost identical to the one caused by sodium chloride. We also built single experiments with complete simulated pore solutions in which we put sodium, potassium, calcium, etc. and the count of pH that tends to create environment replicated pretty much observed in extracted pore solutions from concrete specimens. And in those cases sure enough there we also saw disbondment under cathodic conditions quite easily, and cathodic conditions meaning potentials in the area of say 300 millivolts versus columnar of 400 millivolts so.

We also went ahead and did anodic polarization measurements. Anodic polarization exposed to these specimens and we did that with simulated pore solutions containing chloride ions. And in those cases we also observed disbondment. In addition to observing disbondment, we saw this come out in morphology. So disbondment around the areas where you have your little nicks on the surface material you saw also accumulation of low pH liquid and corrosion products underneath. But we are going ahead in the laboratory with simulated pretty much of the conditions that were observed in the field. This led us to believe that what we were cutting over there was a situation where we have an environment consisted of a high pH liquid with chloride ions and exposed to a mild to severe anodic polarization.

Where can that anodic polarization come from? Well part of it can come directly due to low count selection in the system, but in a marine substructure environment you can have a combination of factors that makes for a particularly vicious environment. Down here very close to the tide area and below the water you're going to have water saturated complete, you're going to have a good amount of electrolyte absorbed in the concrete, this is the reinforcing steel bar if you will. But you're not going to have a lot of flow of oxygen in that area so this part of the system may not have a chloride activity high rate.

Way up there in the substructure you're going to have sure it enough it will amount to oxygen access, but you won't have much of an electrolyte in that area, and also you will have too many chlorides. Right here in the splash of operation zone you're going to have a vicious chloride concentration build up and also you'll have a reasonable amount of oxygen access. It's no wonder that corrosion is to begin in this area. Not only that up here we may a situation where we don't have a lot of chlorides but we have oxygen access so we may have cathodic reaction in this region. Electrotransfer from down here up to the cathodic reactions and in observations in this portion of the system.

Well we set out to do about 2-3 years ago to do these experiments to see how severe this kind of an effect would be in addition to whatever local seduction may exist. And experiments that we did consisted of building laboratory columns that looked like this one, they stood about 4 feet tall. These are epoxy-coated rebars and in here we have a small amount of damage you can see the little nicks as we go close to it. This has about 2% damage on their surface and its 2% damage throughout. The lower part of the system that displays in water that contains high chloride concentration at that part there is also an initial buildup of chloride so this concrete in here has something around 20 lbs/cy chloride, that's wrong. The complete one there up is chloride free. The chloride line is somewhere around here. What we could do is measure the actual current that was flowing from one part of the system to another or whatever was seen that what was where the corrosion was taking place and how much corrosion was taking place at least from a differential standpoint in various parts of the system.

These experiments we conducted using epoxy-coated bars currents and also later we did some experiments with black bar. Well anyway you can work in and make measurements of the microcell current in the system and find out how much current is in each element. And from there you can go ahead and you can actually determine the amount of made of this solution or you if you would perhaps the amount of extra made of this solution that's taking place in certain parts of the system as a result of interaction with the environment.

In Corrosion '91 we presented the results and also we presented the computer model that we used to extrapolate these results to actual field conditions. Namely we were able to obtain a handle to the amount of microcell current that we were observing in the field.

So what I'm going to do now is I'm going to go to the transparencies. So then, basically what you have is a situation where you can go ahead and you can predict the amount called deterioration that could be observed at different parts of the system. So for example, say at 6 m and if your concrete is 10 k and condition 8 happens to be for rebar because it vanishes in ___ corrosion in that case you would expect a microcell density _______ conditions if you complete this highly combative task you can see many of this kind.

This amount of activity by itself is enough to produce corrosion that can be observable maybe after 10 years of service or so if this material were black bar base and if these were available from black bar structures. So we were able to see that microcell activity in this type of system can be quite severe and can really account for a good amount of deterioration of service.

We have conducted very recently some experiments in which we impressed current in specimens made out of ______________ because specimens they come in actual reinforcing steel bar. Rebar can be an epoxy-coated bar with a certain number of defects, or it can be plain, or it can be a cut bar. With an external ________ _____________ ____________ we control a certain amount of current running through the system. We wish to run this experiment until the material cracks.

The results have been interesting and we want to share that with you today. Interesting the time for cracking is not very different when you go from epoxy-coated bar to black bars. They very like the 1 to 1 line and the funny thing that happens is the epoxy-coated rebar the bars tended to crack at the time that was maybe say 10-20% over the black bars. But give the same amount of current, the same amount of corrosion and sure enough cracks appear about the same time in both systems. So it doesn't seem to me that the crack observation of this to begin a crack observation is something that should appear in documenting epoxy-coated rebar.

Now what is interesting is something that was pointed out in a previous presentation. And that is that the average crack width tends to be significantly larger when you have epoxy-coated rebar with a different current used than when you black bar. So it could very easily be that this is at least a certain component or perhaps the division _______ in the case of epoxy-coated rebar and the case of black bar accompanied by the observations of that which may be somewhat of course ___________.

Let's go ahead and talk about mechanisms of deterioration. First of all let's talk about the kind of things that can happen that can be responsible for this disbondment between the epoxy and the metal when there is no cracks present. I'm just going to mention a couple of possible mechanisms of deterioration and these are both cathodic disbondment mechanisms. You're going to come across corrosion process somewhere in the system. You can find it in the system coating, the system metal, the system break in the coating, you may find it there, you might have oxygen flowing from the site into the exposure of the metal, that will consume electrons in the system and that will create oxy ions. The oxy ions can do a couple of things. They could actually attack the epoxy coating or they could attack the oxide that this is inevitable be present on the surface of the metal at the time of epoxy application. I cannot go very much into details but this model looks like it is one of the most likely mechanism for deterioration. At this time we have virtually no stopping materials to work on oxide present in both part just because they are coated with the epoxy coating.

Several sentences here were not understandable.

One thing that was brought up earlier before and I just want to throw this in for discussion, the concept that here we have for example set along microcell counter in a system and here we have a certain percentage of the member exposed. One of the percent is exposed is black bar. Zero percent is perfect is epoxy-coated rebar. One could say if from a very naive standpoint that the larger the percentage the larger the deterioration. I don't believe that that is the case. What I think I will see in many of the systems I think is greatly accelerated deterioration at the beginning and then maybe things will begin to look better.

The other concept that I wanted to mention this is food for thought. Here we have this piece of rebar or this structure in service type. Here is the amount of damage. This would be the critical amount of damage from external observation. The rust appearance, the cracks, you name it, it is all the like.

MORE TEXT WHICH WAS NOT ABLE TO BE TRANSCRIBED.

QUESTIONS

1. W. R. Grace - Up to this point I haven't really understood this, and I think I understand what's going on from what you said, but I just want to make sure I understand this. The difference here I think what you are proposing is that there's a different kind of corrosion going on with the epoxy-coated rebar. In other words you show the steel disappearing inside the epoxy coating and that I assume is an acid decomposition of the steel which is occurring inside the rebar and the cathodic reaction when you have epoxy coated rebar is hidden in production rather than the hydroxide mechanism.

Okay at this moment I don't think that we have a lot of evidence to say that the cathodic process is any different than the cathodic is on black bar. We will be simply having oxygen reduction taking place as the little holidays enter the system. Remainder of answer unable to be understood.

2. Richard Weyers - Virginia Tech - Alberto this idea of sodium and potassium aiding in the disbonding of coating is very intriguing. We talked about and we saw this morning about calcium hydroxides and sodium hydroxide. A hydroxide is a hydroxide ion, it doesn't matter what it is, but the idea that sodium or potassium is in fact disbonding that coating is very intriguing and I wonder if you had any supposition as to the mechanism that's involved in how that would happen.

Okay, the effect of sodium versus calcium being responsible for disbondment has been observed in other areas of the coating in corrosion. Just for example the 3 coatings for ______ corrosion and the like. The mechanism by where those ions are responsible for that at the moment escapes me a little bit. It could have to do with a pH with a rudiment pH come to reach the sides where the induction process is taking place.

3. Bob Lansing- National Penn - You showed slides and with a defect in it and said that with an open circuit there was very little disbondment from the area, but that when you put an applied voltage on it that the area was much larger and that the longer you had the larger the disbondment. Wouldn't that sort make you wonder about putting cathodic protection on it?

Sure, cathodic disbondment and cathodic protection systems is a very well known fact. So if your going to put on cathodic protection in the category of the materials similar to the kinds of materials that we have investigated which by the way were from _____ used by the Florida Department of Transportation, then yes I would expect one would apply cathodic protection to diminish the amount of the disbonding of the coating.

Is that an option for the kinds of structures that are out there now?

Well, I would say as long as you keep the cathodic protection going I think that it is probably alright. But if you remember that if you turn off that system you are going to accelerate whatever disbondment has already occurred. This is sort of like a compromise of the system on a coated system. That is why normally coating producers try to create materials that have practically no tendency for disbondment. Same materials that we have tested have shown that they have very poor performance. Even under certain conditions say 500-700 millivolts we have observed pitting and disbondment of 1-2 millimeters per month of exposure.

And I want to emphasis also that the purely cathodic portions of our laboratory specimens after about 1 year when we demolished one of the covers each one of the sections was completely disbonded, and this was in regions with fairly mild potentials, easily say 300, 350-400 millivolts negative. And 1 year of exposure was enough for complete disbondment.

4. Don Pfifer - Wiss, Janney - We observed in our autopsies of slabs that where invariably where we have pressure-induced bubbles through the autopsy process that bubble was opposite, always opposite to a hole in the concrete.

We have seen that in the laboratory all the time. Interestingly we have seen very few of those things happen in the field. If little bubbles in the backwall sometimes will be relatively dry, sometimes will be full of low pH liquid.

Our were always in the presence of _____ concrete in our experience.

The bubbles yes the disbonding not necessarily.

5. Could not hear the question.

At this moment I do not feel it is necessary to attack through the coating to explain what we are doing.

A.8 CORROSION OF EPOXY-COATED REBARS IN A MARINE ENVIRONMENT LARRY L. SMITH

Let me preface what I'm going to talk about with some statistics. We have 1200 miles of seacoast in our state. We are a southern environment and looking at the weather reports I think we had 80's in South Florida. The Department of Transportation maintains all the certain types of structures, we maintain about 5500 of similar types of structures which are maintained by counties and cities. Now 3,000 are in an environment that basically we are concerned about. Now what you must study in our discussion today will go basically with the Keys bridges and the position we took as an agency as a result of our investigation.

Now Dr. Alberto Sagues has been under contract with us for a number of years and based on the comments and discussions I think we made a wise choice in Alberto but that doesn't mean we're going to raise your fee.

Let's begin with the slides. Now the Department's first major usage of epoxy-coated reinforcing steel came about with the replacement of 48 bridges spanning the islands making up the Florida Keys. This particular photo is an aerial photo of the 7 Mile bridge, the new bridge is to the right. Now with this construction activity came along about $500 million from the federal government with a requirement that fusion bonded epoxy be applied to all reinforcing steel. Actually the program began in 1979 that structure to the right must be close to 60-70 years old. When most of the structures were completed somewhere around 1984, this particular bridge was completed in 1982 at a cost of $15 million its fondly referred to as the Long Key Bridge. Now this particular structure has some other concerns that we have and I'll try to slip those in as we go along. I'm not a structural engineer but primarily the way in which piers are made has given us some concern and actually there is an activity looking at some replacements.

Let me back up just a minute. The structures are generally going to be on a drill shaft beneath the water level. These are steel reinforced. Just above the water in the gaston place the member is also has epoxy steel, the big piers are steel reinforced with epoxy coating and I believe they were pre-cast and then assembled on the site.

In 1986, this is 4 years after completion, this is Long Key. You begin to see the crack. We're going to make a few taps. It was found that early cracks delineated massive delamination created by corrosion of epoxy-coated rebar. Now this is in the deep pier area and he can tell you about critical concern. The specified cover was 4 in and in this particular case, he actually noted the early corrosion between 2 and 2½ in which is a fairly, it's something that can happen in the field, but I have later data which will show that most of the corrosion is now at much greater depths.

Now by 1991 approximately 1/3 of the bents in this bridge had developed severe corrosion, most of which contain concrete at the proper cover specification. Incidently when you tap a structure this large with you got your maintenance engineer and the book with you and he sees that much fall away he shudders, your under the bridge and this is a primary bearing area for these big piers. We have found, and I think Dr. Sagues pointed that out it was 25 lb chloride at the 2 in depth. You've seen similar slides of it could easily in many cases peel back the corrosion we noted that a substance. In a previous slide if you recall when I first showed the crack there was a dark color in the concrete. We have not identified exactly what that is yet and that's before we tapped the cracked surface. We look a little closer I think this may be same slide here so we won't dwell on this one. We do have the pitting. We move to another structure. This is typical of what we call the 7 Mile Niles Channel in Keysville. Each consist of drill shaft steel reinforced concrete including precast steel reinforced concrete struts that's the one that goes completely to columns and those were cast in place. All of those structures contain epoxy-coated rebar.

Looking at the Niles Channel is one of 17 columns of the Niles Channel Bridge showing corrosion distress in 1988 6 years after completion. Cost to this structure is $9 million. In this particular instance thought it was less than the specified, we said that earlier, we thoroughly believe that what we were saying earlier were in the 2-3 in range as far as cover. By 1991 we had 34 summer corrosion induced falls. Now it's been talked about a little bit, what does the bond do. I've had people to tell me that if they detect corrosion in black bar and we choose to examine it by tapping away some of the concrete that we're going to get a smaller spall. More on that later.

Seven Mile first noticed in 1988. At right is one of the earlier spall areas that we, what you see there the sketching is what we now suspect as having some form of delamination. You can see the steel coming up from the shaft into the columns quite a concentration of steel in that particular area. Again early corrosion was a way it covered to 2-mile ______.

As we examined the bars as we look at all these bridges you can easily in many cases feel the way the corrosion, and thus the black substance that we are talking about again it was pointed out that the pH of that substance in the 4.5 - 5 range was the concrete would be a 9-12 range. At that time we were using bare tile wires, and if you will note that the bar wire has not corroded except where it comes in contact with the rebar itself.

Now in 1991 we begin to see some evidence of concern, I believe this is Niles Channel, not only Niles Channel Bridge but the 7-Mile Bridge. Now in all instances the cover met the minimum requirement. Now you may ask your yourself in 1986 we had 1 spalled area, and in 87, 3, 88 had 17 but we seen some progression. I think you have to remember that we're talking about a primary column that's holding up perhaps a 300 foot section so its not insignificant when you look at it in those terms. Why you did not see anything in 89 is because that's the year that we began to do some repair work, we got all our repair, I think we repaired all existing areas by 91.

Now when we tried to understand the source of corrosion problem one of the first things we looked was the chloride content of the concrete. Now I saw the earlier graph by one of the earlier speakers but these are the chloride contents at 2-3 in depth and this is the area that we first noticed corrosion. At the 2-4 ft elevation the majority of corrosion has been observed. We see a range of 12-23 pounds of chlorides per cubic yard. From my way of thinking this is significantly different from what we are seeing in some of the other research. Given the harsh environment condition in the Keys this doesn't surprise us, what does surprise us in the mid-80's was the epoxy-coating had failed to protect against these high chloride contents.

Now this was brought out by Mr. Neff earlier in what we were trying to do is to relate our experiences in evaluating and inspecting a fabricator that was doing the coating and I'm going to skip over these and spend some more time on other parts of it. What we're leading to, Mr. Neff, that quality control is a very serious issue and we think is as stated we probably have insisted on perhaps more quality control than many of the other states. One slide I'll show you here. We've talked about the chloride formality how the bars, the coating and so forth, you notice they are on wood, but we required that they have a quality control program where before we permit them to fabricate any of our work. That QC plan is the basis of which we allow them to do the coating, so you violate the QC plan and we will not accept them. So generally that 's some additional enforcing or an activity that goes on in the coater.

I believe that for the most part there is enough here to protect the steel prior to loading on the truck, and even on the truck. Now I'm not going to stand here and tell you that there hasn't been some changes, but what I'm going to tell you though is trying to resurrect what went on the _____ and what we do now we can find no significant differences, general specifications are the same as far as we can tell the nature in which all the processings is went about is pretty much the same. I think I will later on talk about 2% allowance in the specifications as far as loss of epoxy we would agree with what we've heard here today that that is significantly too high.

Let's talk about the real world now. I was intrigued by the moderator's comments earlier on being able to store those materials in a indoor facility in only a few days I think, if I heard right before they went into the concrete they were taken on. That's not the case in marine construction, at least not today it isn't. This is the beginning of a series of slides, and we will see some duplication here.

Because the cages and things which go into the structure are often fabricated near the site they are stored in some location we believe may take some care in protecting from scarring and so forth they can be in the site in excess of a year.

This just happens to be the other end of what you see the guy up on the little scaffolding there he's working on one end. Particular cage is 60 ft long, 5 ft high on the ends and 8-10 ft high in the middle so there's quite a bit of activity going on in the field even though these are inspected. Even after sometimes the cages are completed they are left to the environment and I think Dr. Sagues eloquently pointed out that it is a concern of ours. This may be the same slide but it got twisted just a little bit.

This bar hasn't been in the field that long. I was told that it may be out there perhaps 3 months. Closer examination you see there is no evidence of corrosion but there is a disbondment at this time. We talked about the uplift of the epoxy, of course with age your going to begin to just envy the vicinity of the facility. Your going to begin the rusting, you can even see that in the tower which is now coated. And perhaps a year later the steel is going to be moved to the actual construction site. Now that's the real world.

Now let's talk about the 2% damage to the bar. This is an artificial situation, but it is a #11 bar, 1 ft long, and you can count those little, I think there's approximately 30 3/16 in, about 3/16 an inch, areas that we've covered with epoxy. That is 2%. I don't believe no matter how lax you are on inspection that you would not get an inspector to allow steel that looked like that get very far on a construction site. Now we did a 30-month study where we rated the bend area A-F where A would be something that absolutely with the naked eye could not see any type of disbondment or breakage in the epoxy. This happens to be a grade A bar which a lot of the inspectors would have indicated it was in pretty good shape, but after 30 months, now I will confess that the solution and the super saturated solution, we put as much salt into the water as we could, but in a general inspection that bar would be classified as not having any loss of materials in that main section. Now that's a typical bar, it's already been _______________. If you look at that it's very difficult to find any defects. Now as we move a little closer this is a 6X microscope. Now I have to comment on the color change; that is the same bar, we have some difficulty in trying to photograph some of these things and retain the color. But if we go to 12X there's a significant change. Now let's back up just a little bit. I don't know exactly where that area is, I cannot see it, and it was mentioned earlier that when we began to see the problems with the Keys we moved and we actually had a specification prepared which required coating after fabrication. We abandoned that, because as the data began to come in we weren't sure that that would be a very significant and thus protecting our structures. Just peel the material away on the same bar.

Now I'm going to share with you an activity that we started in 1979 prior to the construction of the Keys bridges and these test files were comprised of structural quality concrete and thus far I haven't talked about concrete quality. I have 4 #4 with 1 in cover that began an accelerating effect. Each sample was a 6 X 6 X 10 foot long concrete specimen and was positioned in tidal waters which my corrosion people tell me is a rather severe environment. The lower 5 feet was buried in the soil and the normal high tide would reach within 2 feet of the top. We used bare bars, galvanized rebar, and epoxy-coated rebar.

We are going to obviously, and we've seen it all day, epoxy coated, let's talk about bond. Now we cleaned up ours just as good as anything bars in perfect condition we could find no imperfections. So we have taken the position, and in a word, that's a laboratory perfect piece of steel. Galvanized bar also came through looking rather good but it was difficult to break away the concrete. There was a tenacious bond in this case. I've been told by the moderator that I'm closing in on my time so we contracted with Dr. Sagues and I want to go again to this. Indicate that the research is not over, there's still activities going on continuously, we'll skip over that, that's especially redundant. We basically now use, we were starting as we were finishing the Keys, begin to use fly ash in all our structure concrete. We've gathered a great deal of data on those type of activities and we are also looking at calcium nitrite and silica fume. And now this is a slide I have I'll slip in. This is a very beautiful setting in South Florida, but to build in it, as you listen to the building industry and some of the others that have tried to work in it, it is a harsh environment in terms of the behavior of the materials. Bottom line is we no longer using epoxy-steel in anything.

QUESTIONS

1. Bob Lampton of International Paint - I have a question on the work you all are doing on the 7 Mile Bridge location. Are you doing any work on that bridge that's right next to it that's been there for so long to just compare them?

No we've rented out to the motor group. All the commercials that you see of automobiles running down a bridge that's the bridge so I don't think we are doing anything but leaving it there because it really advertises our state. But that is as far as I know that 's all we're doing. We did early on look at some of those materials. The steel is definitely different than some of those other structures.

Did you see anything else as far as the concrete or anything like that?

We did just to bring it back and recall I'm not sure. The early direction I think helped us zero in and try to use a better concrete. We investigated the question of aggregates, and contrary to a lot of ways those old bridges were built with materials right our of the Keys which have significantly we have a little _____ on any chlorides or any pre-stress structural concrete today. Some of those early structures were built would have violated our specifications, so its a very old era and there were a number of things that they did different, but the steel, in any case was significantly different.

3. What have we done to the quality of the concrete?

Dr. Sagues alluded to that. That is what we think is our main line defense. We now have specific concretes which can only be used in marine environments, we are moving and eventually we will evolve a specification which require certain permeabilities within those concrete. We have not finished our work, but additional cover, for instance the 7-Mile Bridge has epoxy-steel in the deck, and we have no corrosion because it is out of the environment that the substructure is in. The concrete in the Keys we would not use today. That may be why we have received it a littler earlier but I think it would certainly suggest that concrete is by far the best barrier in all of this. Not only the mixtures around, but how you place it and then quality control too. That we have moved ahead on that.

4. Rutgers of the University of New Brunswick - You have shown there a epoxy-coated rebar that you took out of a column, is that from the same __________?

It was a perfect bar going in and a perfect bar coming out. It had been exposed for 9 years.

A.9 EPOXY-COATED REINFORCEMENT: CANADIAN EXPERIENCE DAVID MANNING

The title here is rather bold. The Canadian Experience was presented largely by Ken Clear in his report on CSHRP Study this morning. But what I am going to talk about is largely the experience within the Province of Ontario. In Ontario we specify coated reinforcement in bridge superstructures since 1979 and in substructure components since 1982. I think we were also one of the first jurisdictions to Northern climates to report deficiencies in the performance of coated reinforcement.

The first example was this noise barrier wall which was also shown this morning which was built in 1981 and these next three series of photographs were taken in 1990. You will notice that the wall itself is located in very close proximity to the travel portion of the freeway. The concrete itself is very porous by design, that in places the cover is extremely shallow. All of which contribute to corrosive conditions. I'd also note that each of these panels contains but a single epoxy-coated bar which tends to reduce any possibility of macrocell action. Here we see a crack in one panel, spalling in the adjacent panel, and a close up of a spall in which you can see significant corrosion damage has occurred on the epoxy-coated bar.

The second example that we had of unsatisfactory performance, I'm getting ahead of myself. First of all, I need to explain the term end dam, which I'll be using later in the presentation. All our bridge decks are waterproofed except for the concrete block out adjacent to the expansion joint and this is what we term as the end dam. So the next example of unsatisfactory performance came during the replacement of an expansion joint as part of our normal maintenance operations in which it was discovered that the coated reinforcement in the end dam area corroded so that the next time that we were undertaking such a task, we took the opportunity to core the end dam prior to replacement of the joint. And despite the very large amount of cover that was involved, we're looking here at close to 4 inches, there was, in fact, significant corrosion on the reinforcing bars which were exposed. Here you see an indication of the amount of corrosion as shown by the imprint on the underside of the top of the core. At this point we were batting 2 out of 2 in terms of deficient performance of coated bars and end dams and became very concerned.

Because of these experiences we undertook a study to determine the condition of epoxy-coated rebar in structures in Ontario in 1992. The field activities in this study are complete, but the laboratory work is not. Therefore, what I will present today is a progress report with interim conclusions.

The sampling requirement which was adopted involved looking at 12 structures in 3 different age groups and examining the condition of 22 components within those 12 structures, together with bars that were sampled from 2 construction sites and from 1 coating plant. The field activities included a visual examination of the component in question, mapping of cracks, sounding for delamination, measurement of cover, measurement of the rate of corrosion using a linear polarization device, measurement of the electrical continuity of the bars, and the taking, of course, of the subsequent laboratory analysis.

Here we see a typical barrier wall, and barrier walls were in fact the focus of the study because of the practice of waterproofing decks. The first task is to map out the location of the reinforcing steel. Cores were normally taken at the intersection of the horizontal and vertical bars so that we had at least 2 bars in each core. 3LP measurements were taken at the middle point between the horizontal bars so that we were measuring corrosion only on 1 bar and the vertical steel was that steel which was nearest the surface of the concrete.

Sections were typically 6-7 feet long, and for any particular site. Here we see a site which included investigation in both the barrier wall and the side wall. There would typically be 3 sections on each barrier wall that were investigated. This is one of the sites that was sampled. One of our concerns is the time in which coated reinforcement is exposed on site, and our particular concern is in this form of construction where we place the barrier walls in on turn at sections. In some cases, bars can be exposed over the winter months and may be as long as 6 months before the concrete is placed.

The first series of laboratory tests included a visual examination of the bars, measurement of mashed and bare areas, determination of holidays, measurement of thickness, hardness, and adhesion using the knife test, measurement of the ACU resistance and also the determination of the chloride content of the concrete at the location of the reinforcement. When the slides were produced only 8 of the 12 structures were completed, and the additional laboratory tests listed here performed evaluation, underfilm contamination, hot water emersion, and anchor pattern had not begun. So the results of that I'll present today from those 8 structures which had been completed at the time that this presentation had been prepared.

As all of you that have been involved in this kind of an examination know well, it's very very difficult to summarize a very large amount of data in a forum in which it can be assimilated in 2-3 minutes in a presentation such as this, and this is my attempt. All of the data that is shown here is taken from barrier walls except where otherwise noted. We have represented 4 structures from the early age group, the 12-13 year old bridges, 2 bridges from that intermediate zone, 7-10, and 2 more recent structures. As far as cover was concerned we will note that we were having difficulty achieving cover back in that period of 1979-80 because of some design changes that were made at that time, cover has been generally satisfactory since that time.

The resistance test that I refer to is really not a very scientific technique at all. It simply involves measuring the AC resistance between bars that are exposed in the core holes in the deck such that the resistance path is simply not the same in all cases. And although it is not a precise test, all of our research we see here do indicate a lack of the insulated properties of the epoxy coating. The chloride values that are expressed here are expressed in terms of percent by mass of the concrete, they have been corrected for background concentrations and the threshold value for corrosion is about .03. You'll note that in the recent structures we've essentially zero chloride intrusion to the level of the reinforcement. In 2 of the older structures we are clearly above threshold value for corrosion, and for the other 3, we are sitting in that borderline case, but I'd ask you to remember that the measurements that are reported here are the average of at least 6 values, so if we are sitting on the threshold value, that means that in parts of that structure we can expect the chloride threshold value is exceeded.

If we then look at the rate of corrosion measurements and 1 microamp per square centimeter is very approximately the same as 1 milliamp per square foot, and the values of significance here are that anything less than .22 microamps per square centimeter is generally associated with a passive condition from .22 to about 1.1 would be considered slight corrosion, and anything in excess of 1.1 would be considered moderate corrosion. As you see that there's quite a good correlation between the chloride contents and the absence of corrosion, the high chloride contents and the presence of corrosion, and then in these borderline conditions in one case we are measuring no corrosion in the other situation we were measuring quite modest corrosion.

One of the structures of particular interest is the Englington Avenue Bridge and this particular barrier wall, and the reason for that is that we know a great deal about that particular component because it was included in a survey that we made in 1988 at which time we reported the wall to be in good condition. These slides were actually taken during that 1988 study in which we exposed a fairly substantial section of the wall to expose the rebar. The only deficiency that we found at that time was some damage at the top of one of these stirrups that appeared to have been caused by at the time of construction. Here you see a closeup of the condition of that reinforcement in 1988, and the fact that it is colored brown identifies it as being placed in a short period in 1979-1980 when we permitted the use of the rust colored epoxy-coating.

If we look at the condition of that wall today and this is another part of that same wall, we find numerous cracks in the star pattern, and you can see a crack there that's associated with the rebar. Under the other locations in the bar we have vertical cracks over stirrups and even spalls caused by corrosion of the reinforcement. This slide shows the range of the condition of the reinforcement that was removed from that bridge from that where the coating is in good condition but had been damaged on the ridge to very substantial corrosion.

I said it was difficult to summarize the field data, it is even more difficult to summarize all of the laboratory results. So let me try and take you through this slide. The visual test was reported here as nothing more than an average of visual rating based on a scale of 1-3 where 1 represented excellent, 2 represents good, and 3 represents poor. So the closer we are to 1 the closer we are to all the bars that were removed being rated in an excellent condition which is true of some of the more recent structures in the older structures, we are some tendency towards the, which inevitably means that some of the ratings were 3, being poor. The thickness measurements that are reported here in microns back in the period to the mid-80's the specified thickness was 180 microns $\pm$ 50 which is the same as 7 $\pm$ 2 mils. The specification was changed I think in about 1987 to require 90% of the readings between 100 - 300 microns. There are 2 to me surprising results from this data. What I have tried to show here is the range of measurements and the average for all the measurements that were taken. These are not individual values. These values are an average of 5 separate readings. What is surprising to me is the enormous range of thickness on these bars within any one structure and the very high average values and the fact that the averages are really quite consistent, but they are consistently high. The values reported here are only those readings that were taken between the indentations, taken only on the flat part of the bar between the indentations, were taken by or made with a microtest magnetic device, but have been verified by microscopic examination on a spot basis. The hardness value were remarkably consistent. They were consistent within each structure and between structures. Only in 1 case was the average slightly less than HB, it would be a minus if there were such thing in a range of tensiles, and the hardest was an average of 8. The scatter was extremely small and there is no evidence in terms of either hardness or thickness that the bars have changed over the 12 years that they reported here.

They did some testing that is perhaps the most interesting. The adhesion was measured on a scale of 1-3-5 where 1 corresponds to a well adhered coating, a ranging of 3 corresponds to a coating in which can be pried off in small pieces, but not pealed easily. A rating of 5 corresponds to a coating that can be pealed easily. Again I've tried to show the range and the average. You'll see that in the more recent samples the adhesion is generally excellent, not entirely so, isolated readings in the 1990 structures that caused the average to be higher than 1, but I think you will see a trend in the older structures to poorer adhesion, although the second worst edges is clearly not the only factor because the second worst average reading is from a structure that was built in 1985.

If we analyze that adhesion data in a slightly different fashion, and this slide now shows not only the adhesion that were reported in the previous slide which was nothing more than the 8 structures plus 1 side plus one plant, this includes all the adhesion data from all 12 structures, the 2 sites and the plants. I think the influence of age becomes quite compelling. For the recent structures we have only 6% which where the adhesion had been poor, for that 7-10 year period we're showing about 25% of the readings indicated poor adhesion and in the older structures, which were 10-12 years old, you're seeing almost half the readings exhibiting poor adhesion. We know for a fact that the poor adhesion that was measured on the Englington Avenue Bridge in 1992 was not poor adhesion when that structure was investigated in 1988. So we can say quite unequivocally that that change has occurred in the last 4 years.

I'll now show a few of the bars just simply to show the range of condition that we experienced. You may recall from the field data that the Creditville Bridge was a structure where we had fairly good adhesion but low resistance, and a high rate of corrosion. This was the Trafalga Bridge which showed a 1985 construction with the second worst adhesion, I think we show in here the ease with which the coating could be peeled from the surface. We contrast that with the other 1985 structure in the survey where we had excellent adhesion and no corrosion.

We move to the measurement of AC resistance and look at the relationship between logged AC resistance and the measured bar areas on the bar, we may even be so bold as to try and draw a line or a relationship of that type which would then be very similar to the data that was presented this morning. I think this confirms what other presenters have said that the very rapid loss in resistance that occurs within a small amount of bare area on the steel.

If we illustrate the relationship between AC resistance and the number of holidays in the bars, for those bars that did not have bare areas, we again see this very significant different in resistance which in those bars that contain zero holidays and those bars that contain 1 or more. The implications of the 2 holidays per foot requirement in the specifications have already been addressed by a number of the speakers this morning.

So by way of interim conclusion, what have we found to date? The first thing is that most of the structures appear to be in good condition. What I mean by that is if the typical bridge inspector went to any of those structures he would grade them in excellent condition except for the isolated spalls in the Englington Avenue Barrier Wall, there are no obvious physical defects in those structures. However, if one applied nondestructive testing techniques we did find that there was a reasonable correlation between the measured rate of corrosion on the bars, the chloride content of the concrete and the conductivity measured between exposed sections of the reinforcement. We have no reason to believe that there have been significant changes in the quality of the reinforcement that's been produced over the last 13 years. There's certainly been no apparent change in thickness or hardness.

And finally and perhaps the most significant finding to date, there is evidence of coating disbondment in older structures. This occurs in both the absence and the presence of chloride ions.

QUESTIONS

1. Paul Virmani, Federal Highway Administration - We would just want to know whether based on what you are doing based on CSHRP and based on your ongoing research and what you know about your rebar have you made any changes in updating in the way of requirements of using epoxy rebar? Or you thinking to do it?

I was intrigued by your opening comments this morning, Paul, when you said that the presentations today would be divided between the believers and the nonbelievers and I wondered where those of us that sit on the fence were fitted into that. What we have done, obviously we are concerned about these findings. We have changed requirements that don't allow coated reinforcement to be exposed on site for more than, I forget whether its 3 or 4 months, and it has to be covered if it is going to be left on site over the winter. Which set bar quality assurance procedures. We've had a whole series of meetings with the industry in which we are trying to improve the quality of the product and the quality assurance. We're very encouraged by the kind of program that the industry has introduced as presented this morning, but one day soon we're going to have to take a more definite position on whether we can afford to rely on epoxy-coated reinforcement to the same extent that we're relying on it today.

A.10 EFFECT OF DEFECTS ON THE DURABILITY OF EPOXY-COATED REINFORCEMENT DAVID THOMPSON

The paper was prepared by my colleague, Malcolm MacKensie, who is unfortunately away in Indonesia just now so he can't be with us. As Peter says I'm from the Transport Research Laboratory in the UK which some of you may know better by its former name as the Transport Road Research Laboratory. We've been renamed since April.

So again we're concentrated on defects in epoxy reinforcement. What I want to do briefly is to give you a little background on the situation in the UK both to give you some contacts to my presentation and to tell you a little of the research that's been going on there. I shall then tell you about our experiments about the results some of the results we have and try and make a conclusion.

Now turning to concerns in the UK about epoxy-coated reinforcement, I should say straight away that such reinforcement is not in wide-spread or general use in the UK. Its use isn't permitted in government owned highway structures in normal circumstances, it is not in the specifications, and I think that's probably due to conservatism amongst engineers in the UK. I think also arises from the assessed lives of our bridges, highway bridges, is 120 years, and that tends to make people think very hard about the durability of protective systems. I'm not here to defend that or to comment on it, but that does influence people's views.

Now it was realized and the point has been made many times over that when you use e-bar, your losing your high alkalinity, the natural protection that concrete gives to steel, and your replacing it with a barrier, and your placing a great deal of trust in that physical barrier, and there's been concerns then about what would go in the defects because there obviously must be defects, it was our view. What's going to happen over a very long time span? Furthermore, there isn't at present, I think, any very satisfactory type of nondestructive monitoring for in-situ bars which would tell you with confidence what's going on. And then finally, of course, if you do have problems, how shall you repair them? Well people are obviously beginning to think about that but again we haven't got any established procedures.

But despite these concerns, we do have a British Standard as Peter Scheissl pointed out this morning. It is more demanding, I think, than the ASTM standard and e-bars are used moderately as the figures given this morning tell you. Some of the more adventurous regional highway engineers use it as do some of our row engineers. Now let me just mention briefly research in the UK. I put at the top there bending tests; that work is really a laboratory assessment of the ASTM standard which some thought would be, could be adopted straight into British practice. Well we looked at that, at least my colleagues looked at this, and one of the things that came out of the bending schedule were not sufficiently honorous for UK practice and I'm not sure, incidently, Peter, in your table whether you got the right numbers in there. But the British standard right now is a good tighter on bending and in various other places than the ASTM standard.

Now we've also in the UK done some exposure tests and these were done by our colleagues in the building research establishment. They did long term tests on the natural exposure in prisms and slabs of the sort I'll be telling you about later on. These used imported materials from North America and they showed after 5 years much better performance than unprotected bars but there was the underfilm corrosion, there was some blistering, and hints that suggested to my colleagues that there might be problems in the longer term. They also did another interesting piece of work upon construction practice which I think deserves more attention. They actually monitored the numbers of defects and damage on bars from the factory gates through the fabrication process into a 2 meter high wall where into which concrete was poured using normal practice, I think, and then washed all the concrete out before it went off and looked at the defects again they found that 80% of the defects which they recorded from the factory gate to the end of the experiments had occurred during the concreted process and this had resulted in defects per meter over twice the permitted values at the factory gates. So I think if we're thinking about improving our standards and tests this area too needs some thought. We are at TRL looking beginning to look at monitoring methods for in-situ performance and we started then to think on how 2 years into a project which I'm now going to describe to you where we have used epoxy-coated reinforcement of UK manufacture in this case, although it was made before the British Standard was available.

Now we were trying to look at corrosion resistance so we had in mind things like good ends and what was going to happen if pinholes, deliberate damage, damaged repaired, and/or bends; all the sorts of things really that we've already heard about today. We in our experiments used 2 types of specimens, prisms and slabs. Now prisms are better called beams, I'll show you shortly. These prism results, I'm used to the term, contain straight bars with manufactured defects whereas the slabs contain both bent and straight bars as delivered. The materials, the concrete was the same in all specimens 42 mPa at 28 days, I'm told that's about 6,000 psi 3.2 chloride by weight on cement that in very crude terms is something like 10 times to threshold so that's a very aggressive environment. And the bars were, as I have said, UK manufacture. I put that to be at 7295, but there is some slight doubts as to whether that is strictly correct because on subsequent investigation that standard was not published at that time.
Now here are prisms. 400 mm X 100 mm X 100 mm, or 16 in X 4 in X 4 in, the bars you see are entirely embedded and their exposed in a natural rural environment in the south of England where the climate is really not at all severe. If you look at a cross section of those prisms you can see there's 4 bars in there. They're electrically isolated from one another, any one prism has either all plain steel or all coated bars, and that diagram isn't too good because the bars are in pairs, there's one in each corner, one pair with 10 mm cover to each surface the other with 20 mm cover to each surface. Now of those bars, of each pair of bars I should say, one is as received with its cut ends made good and the other have 1 mm diameter approximately it holds 4 of these along its length with one of the cut ends left unrepaired. So at a very simple experiment really we have prisms with either coated or uncoated bars, prisms were either cast in chloride or no chloride.

Now the Suffix is more difficult, it's a bit more complicated to explain. That's a plan of it. It's, I mentioned in inches, approximately 20 inches by 12 inches by 2½ inches thick. You'll see there a green bar which is a plain bar, so each specimen had one plain bar running its whole length. It also had one straight plain bar, the green one. We also had one straight coated bar the whole length, the blue one down at the bottom. And we also had 2 bent bars. Now the bent ones could either be both plain steel, they could either be both epoxy-coated or they could be one epoxy-coated and one plain. They were connected externally so we could look at the galvanic currents. You can see that diagram has a dividing line which is because we sometimes cast them in 2 portions as it were, so some of the specimens have no chloride at all, some have half with cast in chlorides, some have both with cast in chlorides. And there's a proportion of the ones that have no chloride which we've covered it. And you can see here a photograph of all the specimens, you can see the bars are protruding at either end, we've got a hip cast on there to catch the rain water or if we did pond them to hold the ponded salt solution. So the rain accumulated or not depending on the passage of the weather.

Again I'll attempt to summarize what was going on. Each specimen then had a plain bar right through it, a straight epoxy bar right through it, and then we have the combination of bent bars. Now by way of monitoring we did visual inspection as everyone does, we watched the galvanic currents between the bent bars, and I should say here that we don't use the galvanic currency in the way that's been traditionally done here. We simply record them to give us more insight into what's going or to help to give us an insight as to what's going on. And we looked at the potentials on the straight bars. Now you can see cracks and rust staining here. Now what we found as you might have expected its after 2 years, there was cracking and staining on all the prisms which had no epoxy-coated bars in them so all the plain unprotected bars we got cracking and staining.

By contrast only one of our prisms with epoxy-coated bars had any sign of cracking and we subsequently satisfied ourselves that was not due to any problem with the bar. Now on removal of the bars from these specimens after 1-2 years we found on the unprotected ones typically 10% of the area of the bar corroded the worst perhaps 30%. Whereas only epoxy-coated bars there was some black corrosion at the deliberate defects we made and at pinholes after 2 years and rather to our surprise the unrepaired cut ends looked in good shape but I'll return to that later on.

Now turning to the slabs specimens again as you might have expected that was the case after 2 years that we were only getting stain and cracking over the uncoated bars in the specimens. Now we came to try and quantify this. I'm talking again about the slabs, I'm talking about the straight bars comparing the unprotected and the coated bars in specimens in this example with chloride cast in throughout the specimen. You can see that typically we've got 15% of the area of the unprotected bars with corrosion that the green, and very low corrosion, perhaps 1% thereabout, on one of the epoxy-coated bars. And you see that's up to 1 year or 2 year on the epoxy bar on the we found no corrosion. And I think that illustrates the very ability of results that seem inevitably to come from this kind of work and it is why I think this work that has been done over here.

Now this illustrates the kind of result we got from the electrically connected bent bars. This is the case for casting chloride throughout the slab on both sides and you can see here after 2 years what we were finding the different combinations of bars connected. And it's much the same sort of story perhaps the results even more variable we had some of the unprotected bars with corrosion over 70% of the area and you can see that they, in fact after 1 year we had no detectable corrosion on the coated bars after 2 years this was the sort of thing we found, 1-2% of the area.

So to try and summarize that part of the work visually on bars as removed the epoxy-coated bars were doing better and you can sort of get the feel of this from this figure on galvanic currents between these 2 bent bars where you have in this case you have chloride casted in on 1 side and not on the other, and we've got a steel to steel, steel means bad steel, connected together and an epoxy bad steel couple. And you can see that the corrosion current on the, where you have no epoxy bar, in the connected path is very considerably greater and it does illustrate what we were seeing here at that time which was good protection given by the coated bar, but not perfect protection.

Now we went on then to look at more closely these bars and trying first mechanical removal and this is an illustration and this is with our materials manufactured in the UK so I think we are seeing here the sort of things you have been seeing over here. That we were getting underfilm corrosion which we assume has spread down from that unprotected end which you can't see on there I think but we did have some of those which were the uncut surface is like a cut surface that was unprotected was bright steel and yet we have corrosion spreading down what we assume spreading down from that end under the coating. And we found this sort of underfilm corrosion where we had also on the repaired ends and we have the sense that this material was whether corrosion got on was brittle as a candlewick way really quite easily.

Now not all the coatings could be readily removed mechanically by any means and so we resorted to some chemical stripping of the coating so we took the coating right off so we could really see what was going on under there and you could see that's the straight bar from one of the prisms where there's no electrical connections between the bars and you can see that corrosion that's spreading out from the deliberate defects and also from I presume must have been pinholes. So we are beginning to see more on the film corrosion than was at all evident from inspection of the bars as removed from the concrete. That's a bent bar from one of our slabs. Again after chemical cleaning and you can see there are quite extensive corrosion under the film that wasn't very evident before we stripped it. And we did find on bars from the slabs and also from the prisms that they were examples of the areas of this underfilm was on occasion as extensive as the areas that we had found on the unprotected bar.

Now I put this slide in because it illustrates the dilemma that we find ourselves in. The lower bar had no protection; we'd put it in what we call Clark solution to clean the corrosion product off and its not been done terribly well it seems to me but its been done well enough for you to see with the corrosion product removed that that bar's been pitted, its been quite severely attacked after a couple of years. Now the other bar has had its epoxy stripped off chemically and it also is put in Clark solution and the corrosion product, such as it was, removed and you can see it looks as good as new and does illustrate this tension that the epoxy bar that obviously performing better over 2 years.

And this really is a conclusion if you dare to call it a conclusion. After 2 years you've seen we've its quite clear that bars made in the UK have performed better than uncoated equivalents, but its also clear that we've got some older film corrosion developing that wasn't regulate evidence when you remove bars that you remove the bars from the concrete. I really thought that it is our concern to see what happens to this and what we should find when we break open more concrete in 5 years. And there I'll stop.

QUESTIONS

1. Don Pfifer - I'm curious on your epoxy-coated bar where you found the corrosion that was as broadly spread as on black bars, did it cause cracking?

Cracking in the concrete, no not at this stage. There was no cracking of concrete associated with that level of underfilm corrosion. We did have in our prism specimens with our straight bars all those that were unprotected had cracks. Okay. And all the on the slabs all the unprotected straight bars had cracking there. The bent plain bars I'm not sure about the cracking situation with those because obviously they stood or were cast in beside the straight ones and the situation isn't quite clear. We were surprised of the extent of that sir, that's on the film corrosion. The point I'm trying to make it wasn't severe in terms of _______, but it was there, and one's concerned about where its going to get to.

2. National Research Council Canada - Do you recall what chemical you used to remove your epoxy relative took if off?

I think it was methylene dychloride but I have to check with my notes. My colleagues tell me that the epoxy came off over night which surprised me.

CLOSING PANEL DISCUSSION

1. John Ryan, Trump Consultants, Toronto - my question is to Mr. Schmitt from Florida. One of the most interesting plotted things come out of Florida in recent times has been the epoxy-coated strand. Has Florida DOT had much experience with epoxy-coated strands and if so is it possible to learn anything that we may apply to epoxy-coating bars?

I wish I'd known that question was coming. I'm familiar with the fact that we are looking at it but unless Alberto's got any thoughts on it I think we are not about to adopt anything. We are still investigating it. I think you have to realize that we went through quite a shock in the mid-60's and I think any movement toward any coating could be ultimately adopted. We're going to move very slow. You know someone says Missouri is the "Show Me" state, we now say we're the "Prove Me" state.

Peter Sheissl - May I give you some information. These coated strands have been employed by a German contractor in various locations for cables for kings state bridges and the coating thickness is between 800 - 1000 microns that's about 1 millimeter thickness and now they have changed the process because the wire in the middle of the strand was not coated and protected in the beginning. And moisture can penetrate from the anchors into the interior of the strand and now they have changed the process to twist the off during the coating to be able to coat the middle wires as well and so they have a coating thickness at the middle wire of 100 microns as well and this improves the fatigue properties that were bad in the beginning considerably and this way they think they improved the quality of the coated strands considerable. Of course, the amount of oxygen, water weight, diffusing through this thick film is considerably lower and together with a certain monitoring procedure I think its a very workable process.

2. Richard Weyers - Virginia Tech - I guess I'm going to ask a question to the panel. What we've seen today is rebar exposed under various conditions in the South Florida environment to what I'll call the North harsh North climate, certainly where we come from. I guess the question is do we or do we believe that we are going to have satisfactory performance and satisfactory performance could be defined as 50 years in the United States or in the UK 120 years, do we or do we believe that we will have satisfactory performance from epoxy-coated rebars in any environment? Be that in the Northern environment or in the Southern environment and if so, where?

David Manning - The simple answer is no. We've learned today that our 2 factors which dominate or have the dominant influence on the performance of coated reinforcement, 1) the number of defects and, 2) appears to be adhesion. In our experience this lack of adhesion is of concern, recognizing that the specifications that were in effect at that time and the fact that there are strong indications that there have been no changes of significant changes in the bars over the years, one can expect that the bars in those structures contain a significant number of defects. Certainly the cores that have been removed and the bars have been extracted will support that viewpoint.

What is protecting us today is the fact that the quality of the concrete has been improved dramatically since the 1970's, but most of the data that I show shows relatively low chloride contents. But certainly there's no reason to expect that we are going to have satisfactory performance for 50 years.

- This is a point adding to your results, Larry, that worries me a little bit. If looking at a service life of 50 or even 120 years in the UK, if we produce an excellent concrete quality a sufficient thickness of the cover and if you then can expect that critical chloride levels at the steel are not allowed to reach it within the first 30 years, but if then after that period of time the coating as a protective barrier should work then what worries me is the loss of adhesion that you found and Alberto found even in areas where chloride levels are not high and this result is in accordance with what the results the pipe coating people found that for epoxy-coated pipes exposed to earth after 20 years of exposure in the earth there is in most of the cases a complete loss of adhesion. And what is there even if we would not have local defects, what is the protective ability now? I think this is one of the major questions that needs to be answered in future research.

David Thompson - I'd like to make an additional comment because we are talking about a loss of adhesion to these bars. Now we put these bars in concrete because we want the structure to work and to get the structure to work my understanding is that the concrete should be bonded to the steel. Now we get advice that disbondment of this steel in the concrete, I'm not talking about development, but what are we doing to this material as a structural material is a question I'd like to ask.

Answer: I would not be really afraid about that because if you had a good bond action the mechanical bond between steel and the concrete from the beginning the loss of adhesion between the coating and the steel at substrate I think would not harm too much. So I would not be too afraid in this respect.

Steve Chase - Federal Highway Administration - I work in our office of research in the structures division and for the last 4 months we've been looking at that specific issue. We cast some concrete slabs, with bars that had had disbondment between the epoxy coating and the steel substrate induced artificially to a modification of the accelerated corrosion tests that Ken Clear developed. After subjecting the bars to these tests for 7 days we got about 20-30% disbondment over the entire length of bar. We then cast these bars into slabs and tested them for positive moments and negative moments and also subjected these bars to pull out tests. The results that we found are that there was no significant difference in the flexural behavior in these slabs cast with these bars that had 20-30% disbondment in the lower mat of steel. On the pull out tests we found that there was about a 10% reduction in the bond strength for the disbonded bars versus the plain epoxy-coated bars. And we will be publishing a report on this work in about a month.

Ontario Department of Transportation - Mr. Thompson reported on additional damage which had occurred to the coated steel during the placement of the concrete. One can assume that this might have been caused by impact from the coarse aggregate as the concrete was dropped into the forms perhaps by the vibrators used when packing the concrete. Was there indication of what caused that damage so that one might minimize the damage that's caused during placement of concrete?

David Thompson - I'm reporting some work done by a man called Hugh Davis from the building research establishment and that paper is published and I can give you the reference. From memory, he does recall some damage by the poker vibrator, but I think a good deal of it was due to the aggregate in the placing procedure. But it is written up and you can read it up for yourself.

John Theopolis - We've been interested in this question of the different effect of adhesion of the coating and the holes in the coating in terms of the corrosion process. We know that we can affect the adhesion and the corrosion stress quite a lot by the manufacturing process, in fact the work being which David Thompson reported we'd improved the bond characteristics under corrosion stress for by an order of magnitude since we supplied the bars to the TRL by process effects essentially. What we were interested in is what is the difference between holes in the coating in terms of the coating performance versus its adhesion characteristics which is the more important factor. Manchester University Corrosion Center dates scant debriefs on an enormous amount of work on the performance of corrosion performance of high performance coatings and they got a very nice model to explain the thing which in fact I'll bet is similar to I think the model is very similar to it, they have actually got a mathematical set up. And so we asked them to do some work where they model the behavior of epoxy-coated reinforcement in concrete using their model and see what if you changed all the variables in it what will the most significant variables. A very interesting result came out. Which was it depends an awful lot on the oxygen permeability of the overlying concrete. In a situation where the overlying concrete has got a low resistance to oxygen ingress then holes become the dominant feature. In other words, the more holes you have the more corrosion you get, but if the alternative of a high impermeable concrete coating for example as you might get with a PFA blended concrete or very dense concrete the disbondment characteristics become very much more dominant and in fact you get a situation where an order of magnitude improvement in disbondment characteristics results in about a 2 order of magnitude improvement in overall corrosion performance. Now I have to say this is early results. It will be published when it is finalized and it does represent a much more model based upon the situation. Nonetheless I think it's important that people are aware that these situation exists and these relationships might exist and particularly that the resistance to oxygen supplied by the concrete coating may affect very much the performance of your samples. So when you are analyzing results could you please be bearing that in mind. I would be happy to talk to anybody after the meeting about the background to this if anybody wants to pick me up I'll be standing around and I can give you the information and I'd show you some of the slides that unfortunately you can't see right now.

William Clark - Did anybody see any evidence of chafing of the reinforcing steel here or abroad in specimens or in something you've dissected? My concern would be there's no bond to the concrete, and is that bar chafing inside the concrete damaging the coating?

ANSWER: After our tests we removed the bars the regions splice regions concrete and we didn't see any signs of damage to the coating like that.

Alberto Sagues - A couple of comments on the order of evidence that we have on the performance of epoxy-coated reinforcing steel as a corrosion protection method. And that is that all the positive evidence that we have on the material has existed to a great extent say 20 years ago it came from some impressive short term tests. Short term tests meaning a couple of years, 3 years, 5 years, something as long as 10 years or so in which it was clearly demonstrated that steel coated with a layer of inorganic coating behavior of bare steel. We have seen that in a couple of investigations today, that has been seen 10 years ago, it has been seen 20 years ago. And the problem is of course that those kinds of tests are accelerated tests. And in accelerated tests you inevitably take the chance that the mechanism of deterioration may have changed. So in doing tests the things that move very fast, things like for example oxygen can fall fast through the concrete we're operating under signal stresses or the opinion that the concrete may be reluctantly high under the slow chlorides arriving very fast to the surface of the steel and under those conditions I think that the overall value of the evidence is that epoxy-coated reinforcing steel works very, very well in deed.

Now when we are in natural applications in natural field application we are striving for a surface time maybe 2 times greater, maybe 3 times greater, maybe 10 times greater than what we are doing in the tests. In that case something may be happening to the coating for example some of the disbondment that we have seen today, maybe some other form of deterioration as seen, and by the time the chloride arrives it is a very different material than that it was encountered in the short term tests. This is seen in any table dealing with materials performance evaluation of this fashion. So the main question that we have from here is that if we are going to test things we want to be aware of what we are trying to test and we have to be very, very careful of any short term comparison between materials. Sometimes we can create a perceived improvement of the performance of the coating for example Germany they are developing a number of very stringent tests that they materials must pass. Sure enough the materials will pass the tests because they are designed so that they pass the tests. Now the question is are those tests relevant to the long term performance in concrete. We can only assume that they would be relevant to that. And just because the materials is passing a very severe bending tests, or very severe oil and water tests or some other efficient testing level, some cleverly conceived chemical type of experiment doesn't necessarily mean that's perform well in concrete in the long term and we have to always keep that in mind.

The other question that is appeared that is right now is that most everyone agreed that if we reduce the number of imperfections in the coating, the performance in short term tests is very good and well perhaps if it performs over the long term perhaps it is going to be very good. So now we are talking about a break levels or holiday levels or imperfection levels that are 1 order of magnitude may be 2 orders of magnitude late that we are talking as near as say 5 years ago and of course the question has appeared several times is that something that is practically feasible. For example we have some evidence from England as to the table they did here and as well and so on you pour the concrete, you vibrate, and so on and you create additional damage right there inside the forms. Maybe we will succeed by means of very careful procedures to go ahead and assemble these flawless cages that have been painted and patched and so on and we are going to some extremely involved inspection procedure to guarantee that that is the case it by itself will be quite a challenge. Now we go ahead and presumably we are going to such a level of quality of material we are going to have to examine the material after it is poured to see if the pour was done right, maybe we can envision some kind of an electrical service system that will go on the line. Then you have the question what if you find out that when they pour the concrete they damage it what do you do. Empty them all and start again? So we are getting now into an area where we may be asking a lot out of the product handling procedure and that is not impossible but I think we are talking about cost factors that are going to have to be examined. Thank you.