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CRACK SEALING AND REPAIR OF OLDER SERVICEABLE BRIDGES USING POLYMER SEALERS

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16 Abstract

The objective of this study is to determine the feasibility of using High Molecular Weight Methacrylate (HMWM) and EPOXY Healer Sealers for crack sealing and repair of older serviceable bridges. The study included the field application of two HMWM materials and one EPOXY material to eight bridges throughout Kansas. The field application of the materials was performed in 1992 and the field-testing was completed in 1995. Each treated bridge deck also included an untreated control section.

Laboratory testing was performed and included the two HMWM materials and one EPOXY material applied in the field and an additional HMWM material that was not tested in the field. A control set of untreated beams was also included in the laboratory testing. Laboratory tests included wet/dry, freeze/thaw, and salt ponding and were completed in 1995.

The results of the field portion of the study were inconclusive. Chloride concentration levels of the sealed sections and the control sections were inconsistent. However, the results of the laboratory tests on the three sealers tested in the field, plus the additional sealer added for later evaluation, indicated a definite difference in the performance of the four sealers.

The results of the study indicated that the sealer must be properly applied shortly after the cracks are formed to maximize the penetration and protection of the structure. Attempts should be made to clean the cracks before application of the sealers. The optimum sealer would be one with a relatively low viscosity, 0.5 Pa·s or less, tensile elongation of ten (10) percent or more and a tensile strength of at least eight (8) MPa.

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ABSTRACT

The objective of this study is to determine the feasibility of using High Molecular Weight Methacrylate (HMWM) and EPOXY Healer Sealers for crack sealing and repair of older serviceable bridges. The study included the field application of two HMWM materials and one EPOXY material to eight bridges throughout Kansas. The field application of the materials was performed in 1992 and the field testing was completed in 1995. Each treated bridge deck also included an untreated control section.

The ability of the sealers to penetrate into the cracks and prevent chloride intrusion was evaluated. Due to variability of the penetration data, laboratory tests of concrete beams treated with the sealers were initiated in 1994.

The laboratory testing included the two HMWM materials and one EPOXY material applied in the field and an additional HMWM material that was not tested in the field. A control set of untreated beams was also included in the laboratory testing. Laboratory tests included wet/dry, freeze/thaw, and salt ponding and were completed in 1995.

The results of the field portion of the study were inconclusive. Chloride concentration levels of the sealed sections and the control sections were inconsistent. Percentage increase and decrease were nearly equal between the control sections and the sealed sections. In some cases, the treated portions of the bridge deck had higher chloride concentration increases than the control section. This indicates that the sealers could trap chlorides in the system and actually worsen the conditions.

Evaluation of the cores removed from the test sections on each bridge indicated that the penetration of the sealers also varied. None of the three materials consistently penetrated the full depth of the cracks.

The results of the laboratory tests on the three sealers tested in the field, plus the additional sealer added for later evaluation, indicated a definite difference in the performance of the four sealers. Sealing of cracks in a new bridge deck may be beneficial, as the sealed beams did not deteriorate as rapidly as the unsealed beams.

The difference in the performance of the sealers appears to be tied more to material properties such as viscosity, flexibility, and tensile strength, rather than to penetration.

The results of the study indicated that the sealer must be properly applied shortly after the cracks are formed to maximize the penetration and protection of the structure. Attempts should be made to clean the cracks before application of the sealers. The optimum sealer would be one with a relatively low viscosity, 0.5 Pa·s or less, tensile elongation of ten (10) percent or more and a tensile strength of at least eight (8) MPa.

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NOTICE

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DISCLAIMER

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

PROTECTION OF EXISTING CRACKED BRIDGE DECKS USING POLYMER SEALERS

INTRODUCTION

Statement Of Problem

Many older bridges have suffered excessive deterioration of the deck. Much of this deterioration is caused by the intrusion of water and chloride ion-bearing water from de-icing into the concrete. In the presence of moisture and oxygen, the chlorides attack the reinforcing steel within the bridge deck, causing corrosion of the reinforcing steel. The expansion of the corrosion products induces excess forces on the surrounding concrete that can be as high as 69 MPa (USDOT, FHWA, 1996). These forces cause delaminations and spalling, which along with the section loss of the reinforcing steel, significantly decrease the serviceability of the bridge. The presence of water in the cracks of the deck during freeze/thaw cycles also contributes to deterioration. As the water in the cracks freezes into ice, the larger volume of ice forces the concrete apart much as corrosion action does.

One of the first signs of bridge deck deterioration is cracking. General repair alternatives for cracks in bridge decks are to fill the cracks with sealers or to overlay the deck. Most overlay strategies add significant dead load to the bridge structure, which may cause the total load to approach or exceed the design limit of the structure. Bridge deterioration is critical because of the potential for catastrophic failure. Therefore, precautions should be taken to prevent excessive structural deterioration.

Over the past few decades, various concrete sealers have been studied and used throughout the country. The sealers were applied in an attempt to seal the surface and the cracks in the concrete to prevent or to slow water and chloride intrusion. Each sealer has specific properties to deal with a specific problem. For example, HMWM sealers may be more effective in penetrating into the existing cracks in bridge decks due to their extremely low viscosity and low surface tension. The sealers tested in this study were intended to seal the existing cracks but not the surface of the bridge decks.

Scope Of Study

The objective of this thesis is to study the feasibility of using High Molecular Weight Methacrylate (HMWM) and EPOXY Healer Sealers for crack sealing and repair of older serviceable bridges. The study included the field application of two HMWM materials and one EPOXY material to eight bridges throughout Kansas (see Appendix I for descriptions). The field application of these materials was performed in 1992 and the field testing was completed in 1995. Each treated bridge deck also included an untreated control section.

Each test section of the bridge decks was sampled for chloride content prior to the application of the sealers and again in 1995 at the completion of the field test. Cores were removed from each test section on each bridge immediately after application of the sealers, and over the subsequent three years, to evaluate the penetration and durability of the sealers. Due to variability of the penetration data, laboratory tests of concrete beams treated with the sealers were initiated in 1994.

The laboratory testing included the two HMWM materials and one EPOXY material applied in the field and an additional HMWM material that was not tested in the field. A control set of untreated beams was also included in the laboratory testing.

Laboratory tests included wet/dry, freeze/thaw, and salt ponding. The laboratory testing portion of the evaluation was completed in 1995 (the same time as the field testing was completed). Laboratory and field results of the effectiveness of the sealers were analyzed and evaluated.

LITERATURE REVIEW

Deterioration of Portland cement concrete structures usually is critically dependent on the ingress of substances through the concrete surface, usually water. Water is involved in nearly all forms of concrete deterioration: freeze/thaw, reinforcement corrosion, alkali-aggregate interaction reactions, dissolution, sulfate attack, and carbonation (Cady, 1994).

A major portion of this deterioration is caused by the intrusion of water and chloride ion-bearing water from de-icing into the concrete. Chlorides attack the reinforcing steel within the bridge deck causing corrosion of the reinforcing steel. The following reactions take place:

$$2Cl^- + Fe \leftrightarrows FeCl_2 + 2e^- \qquad (1)$$

$$FeCl_2 + 2OH^- \leftrightarrows Fe(OH)_2 + 2Cl^- \qquad (2)$$

The net reaction is:

$$Fe + 2OH^- \leftrightarrows Fe(OH)_2 + 2e^- \qquad (3)$$

In the presence of moisture and oxygen, the chloride reacts with the iron to form ferrous chloride. The ferrous chloride interacts with the hydroxyl ion (OH^-) available in the alkaline medium of concrete to form ferrous hydroxide ($Fe(OH)_2$) and liberate the chloride ions. The liberated chloride ions attack more iron. A chain reaction is developed and continues, leading to a progressive transformation of the reinforcing steel into spongy ferrous hydroxide (Bishara, 1997). The expansion of the corrosion products induces excess forces on the surrounding concrete that can be as high as 68,950 kPa (USDOT, FHWA, 1996). With this fact in mind, it is desirable to look for methods of sealing the concrete surface to prevent the intrusion of water and other substances.

Interest in the use of sealers dates as far back as the 1930s. As the awareness of the severity of corrosion problems has grown since the 1960s, the interest in the use of sealers has grown exponentially. To follow the interest in the use of sealers, the number of manufacturers and number of products available has also grown. Cady (Cady, 1994) identified 409 concrete sealers in approximately two dozen categories, produced by 169 manufacturers. This rapid growth in the industry was accompanied by lack of consensus of acceptable test methods to evaluate the performance of the sealers, and conflicting performance reports are common.

With the large number of sealers available, it is not practical for one organization, or individual, to test each material. Therefore, three materials, two HMWMs and one EPOXY, were initially chosen for testing in the field portion of this project. A fourth material, a HMWM, was added during the laboratory testing. The HMWM materials were similar chemically, but differed in material properties when cured. The varying physical properties are developed (by the manufacturers) by slightly altering the polymer chain and introducing proprietary radicals. The EPOXY differed in chemical makeup, as well as physical properties.

HMWMs are adhesives composed of methacrylate monomers. Curable methacrylate adhesives were first developed in West Germany in the late 1960s. The curing of the methacrylate monomers is accomplished by adding an initiator and a promoter to create an oxidation - reduction chain reaction. An intermediate free radical allows the monomer to build a high molecular weight polymer (Damico, 1990).

Epoxies are adhesives based on a reaction between biphenol A and epichlorohydrin. Epoxy adhesives are perhaps the most versatile of structural adhesives. Various formulations can create epoxies with a wide range of physical properties (Behm and Gannon, 1990).

While epoxies and HMWMs are polymers, the epoxies generally have higher tensile strengths and a wide range of strains to failure. HMWMs are generally a lower strength material and can also have a wide range of strains to failure. The epoxies will have tensile strengths four to five times that of HMWMs (Byrne, 1997).

The New Jersey Highway Authority (Goldberger, 1961) used epoxy as a bridge deck sealer on several structures as early as 1959 and 1960. The initial application in 1959 was completed mostly by hand and was relatively expensive. After evaluating the initial application procedures, two structures were treated in 1960 using more mechanized procedures to reduce application costs. Two-component spray equipment was used to apply the material at a thickness of 0.25 mm to 0.38 mm. This involved a coverage rate of approximately 2.45 m^2/L. After the application of the epoxy material, a layer of crushed emery was spread on the fresh epoxy to create a skid resistant surface. It should be noted that application rates and procedures developed in 1961 are very similar to those in use today. There is minimal information available as to the effectiveness of the material used to seal the decks or the longevity of the treatment. However, it is stated in the report that the Turnpike Authority intended to treat all other bridges on the Garden State Parkway. No information is readily available to determine if the applications were performed.

The California Department of Transportation (Caltrans) began the use of HMWM resin for sealing cracks in bridge decks in 1981 (Krauss, 1985). Initially, the cracks to be treated in the bridge decks were very large. Epoxy injection was assumed to be an appropriate remedy to prevent water and salt intrusion to the top mat of reinforcing steel. After an unsuccessful epoxy injection application, cracks just below the deck surface were examined and found to be about 0.05 mm or less in width. Subsequently, alternate crack sealing methods were attempted. In October 1981, Caltans treated the first bridge deck by a topical application through the use of squeeze bottles. HMWM resin was batch mixed and applied to each crack individually. Cobalt naphthenate promoter was added to the resin at a rate of 250 mL per 18.9 L of resin, previous to the beginning of the project. A peroxide initiator was added to the resin on the bridge deck at the time of application. The material was applied at a rate of approximately 2.45 m^2/L. High capillary action and low viscosity pulled the resin into cracks as small as, or smaller than, 0.013 mm. This method successfully treated cracks to a depth of 76 mm.

Caltrans' first topical treatment with HMWM was applied, in November 1981, to the Rio Vista Lift Span. The application method for the material on this bridge was performed in the more familiar method of flooding the bridge surface. After the material was poured on the deck, squeegees and brooms were used to distribute the material and seal the cracks. Sand was broadcast on the polymer for a skid resistant surface. Nine years after the material was applied on this structure, Caltrans removed cores to evaluate the penetration. The cores were found to be intact after removal. Microscopic and ultra-violet evaluation indicated that the average depth of penetration was also 76 mm (Jerzak, 1991).

After early successes in 1981 through 1983, Caltrans went on to treat a large number of bridge decks throughout the state of California. In 1988, Caltrans treated a bridge deck with HMWM, but applied the material by using small pumps and a spray bar. The two part resin system had one part pre-promoted, and the other part pre-initiated. The two portions were mixed previous to leaving the spray bar (Roberts, 1987). This application proved to be faster and equally effective.

Caltrans has developed a specification for the material, and the application of the material that has been adopted in part, or whole, by a number of Departments of Transportation, including the Kansas Department of Transportation.

The Mississippi River bridge at Keokuk, Iowa was completed and opened to traffic in November 1985 by the Iowa DOT. Full depth transverse cracks were noted before deck placement was completed. HMWM was applied to the bridge deck for the first time, in October 1986. The Iowa DOT adopted the Caltrans specification, with some changes to reflect the Iowa climate. This application of HMWM sealer to the bridge deck was performed by contract forces using a dual spray bar system. HMWM material was applied at a rate of approximately 2.45 m^2/L.

Core samples taken from the bridge deck were treated with a 50/50 mix of sulfuric acid and water, then heated to 60^oC in an oven for 2 hours. This procedure caused the organic, HMWM resin to turn black for ease in determining the extent of the resin penetration. All core sections revealed that the HMWM sealer penetrated to a depth of at least 51 mm. Cores also indicated that the HMWM had been effective in bonding the cracks together.

Leakage noted soon after the first application indicated that the cracks had not been sufficiently sealed, although the flow had been reduced. A second treatment of the material was applied in November 1987. HMWM resin was mixed by hand in five gallon buckets, poured on the deck, and spread with brooms. The application rate for the second treatment was slightly lighter than for the first. It was noted, in June 1988, that leakage was further reduced by the second application (Marks, 1988).

In 1988, the Virginia Transportation Research Council (VTRC), in cooperation with the Virginia Department of Transportation (VDOT), applied three HMWM resins on two bridge decks on I-81 in Virginia (Sprinkel, 1992). The two structures had tined (grooved) decks and were two and three years of age. Test applications indicated that a simple application of a HMWM resin was as effective in sealing some cracks as vacuum injection of methacrylate polymer, or pressure injection of epoxy. VDOT prepared the HMWM polymer in small batches and applied the material to the cracks individually, by using 7.5 L spray cans. After the cracks were sealed individually, the entire surface of the deck was flooded with polymer in the more traditional manner used by other DOTs.

When inspected shortly after application, VDOT found that the more flexible of the polymers revealed less cracking within the deck cracks. Within 15 months, most of the flexibility of the materials had been lost. Cores were removed from the decks and evaluated for permeability, material penetration, flexural strength, and tensile splitting strength.

Permeability tests indicated that the concrete that was sealed had 44 to 52 percent less permeability than the base concrete. However, the permeability increased after several years in service, although the more flexible material had lower permeability.

A petrographic analysis showed that, in general, the cracks were very narrow (less than 0.2 mm). Petrographic analysis also showed that the HMWM materials did not fill the cracks well at depths greater than 13 mm. Tests indicated that the treatment did not restore the flexural strength of the concrete. Inspection of the failed surfaces indicated that a large number of the surfaces were contaminated with dust, dirt, and carbonation. The VTRC ascertained that due to the foreign material in the cracks in structures bonding of the cracks is improbable. Tensile splitting tests also exposed contaminated surfaces. After three years, the brittle polymers had lost much of the protection provided. However, the more flexible polymer was still protecting the structure.

The VTRC evaluated the effectiveness of five gravity fill crack sealers, three two-component EPOXIES, one three-component HMWM, and one two-component polyurethane (Sprinkel and DeMars, 1995). The testing included flexural strength, freeze/thaw durability of repaired beams, and the gel times of the polymers. Beams were produced, broken, and repaired with the sealers and tested for flexural strength. Wires were used to maintain known crack widths at the repair location.

The cracks in the beams were 100 percent filled with the sealers. Under controlled laboratory conditions, all the sealers restored the beams to at least 100 percent of the original strength. As stated in the report, it is unlikely that this would be possible in the field considering the contamination found in most cracks. Freeze/thaw durability of the materials was determined by testing the flexural strength of repaired beams that had been subjected to 480 freeze/thaw cycles. Freeze/thaw cycling caused significant reductions in the flexural strengths of the repaired beams. Epoxy Number One (brand names were not reported) performed the best after the freeze/thaw cycling. Flexural strength of the repair decreased, both before and after freeze/thaw cycling, as the crack width increased. Penetration was tested by pouring the polymers through sand columns of varying gradations. The HMWM product performed the best in the penetration test, as it penetrated all sand columns 100 percent. When the performance of the materials was ranked the HMWM product was the leader, with two of the epoxy products second and third. The polyurethane product ranked last.

Construction of the Seven Mile Bridge was completed by the Florida DOT (FlaDOT) in 1981 and longitudinal cracks were noted in the deck in 1985 (Sika, 1990). The FlaDOT tested the effectiveness of five penetrating sealers and two methacrylate sealers. In December 1985, the penetrating sealers were applied to the bridge deck. In April 1986, two HMWM products were applied to the bridge deck (Kessler, Powers, Langley, 1986). Both HMWM products were applied with brooms and sand was broadcast on the surface for skid resistance. Both materials were applied at approximately 2.45 m^2/L. Cores were removed from both test sites to determine the penetration of the materials into the cracks. Penetration was determined by water beading and visual evaluation under a microscope. Bonding properties were determined by splitting cores with a chisel and hammer and the split tensile test. One of the HMWM materials was determined to have penetrated only 3 mm and showed little bonding ability. The second HMWM, however, penetrated 13 to 54 mm into the cracks, with significant bond strength in cracks ranging from 0.20 to 0.48 mm wide. The FlaDOT treated the entire deck of the Seven Mile Bridge with an HMWM product, based on the results of these tests. Application was performed using a truck to haul materials and pumps and a spray bar for uniform application. The material was applied at a rate of 3.01 m^2/L (Sika, 1990).

HMWM materials have also been used for cracked bridge deck repair in Texas and Idaho (Crawford). The material was batched in buckets and spread on the deck using brooms and/or squeegees at an approximate rate of 2.45 m^2/L on both projects. A minimal number of cores were removed from the projects to evaluate the penetration of the materials. Cores removed from the Texas project were 76 and 191 mm in length, with crack widths between 1 and 5 mm. The material penetrated to a depth of 25 and 76 mm, respectively. Cores removed from the Idaho project were 64 mm in length, with penetration between 6 and 10 mm. No crack width information was included in the report.

In the laboratory portion of this study, the sealers were subjected to freeze/thaw and wet/dry cycling to determine the durability of the materials. Durability was determined by evaluating the number of cycles the sealers could undergo before ponded chlorides could attack the reinforcing steel imbedded in the concrete test beams. To determine if the sealers had failed, corrosion rate measurements were made at various times during testing.

Corrosion rate is the measurement of the section loss of a sample of metal which is actively corroding. In the case of a bridge deck, the corroding metal is the portion of reinforcing steel under the corrosion rate equipment probe. Corrosion rate is quantified by mA/sq. ft., $\mu A/cm^2$, or mpy (mils per year). Each unit can be related to the other by the following relationships:

$$1\ \mu\text{A/cm}^2 = 0.4568\ \text{mpy} \tag{4}$$

$$1\ \text{mA/sq. ft.} = 1.1\ \mu\text{A/cm}^2 \tag{5}$$

$$1\ \text{mA/sq. ft} = 0.5025\ \text{mpy} \tag{6}$$

CC Technologies (USDOT, FHWA, 1991) and Concorr Inc. (USDOT, FHWA, 1996) referencing KCC Inc. (manufacturers of the 3LP corrosion rate device), stated the corrosion rate (Icor) and expected damage relationship shown in Table 1.

Icor less than 0.22 μA/cm^2	no corrosion damage expected.
Icor between 0.22 and 1.1 μA/cm^2	corrosion damage possible in the range of 10 to 15 years.
Icor between 1.1 and 11.0 μA/cm^2	corrosion damage possible in the range of 2 to 10 years.
Icor in excess of 11.0 μA/ cm^2	corrosion damage expected in 2 years or less.

TABLE 1 Corrosion Rate And Damage Relationship

Concorr Inc. (USDOT, FHWA, 1996) referencing GEOCISA, Spain, (manufacturer of the GECOR corrosion rate device), stated the corrosion rate and expected damage relationship shown in Table 2.

A comparison of Table 1 and Table 2 indicates that there is a differing of opinion as to what corrosion rate is damaging, and how damaging a given corrosion rate is.

Icor less than 0.10 μA/cm^2	passive condition.
Icor between 0.1 and 0.5 μA/cm^2	low corrosion.
Icor between 0.5 and 1.0 μA/cm^2	moderate corrosion.
Icor greater than 1.0 μA/cm^2	high corrosion rate.

TABLE 2 Corrosion Rate And Damage Relationship.

Documentation accompanying the Cortest instrument used for the beam testing offered no corrosion rate to damage relationships.

DESIGN OF EXPERIMENT

Field Portion

During the summer of 1991, eight bridges, throughout the six operating districts of the Kansas Department of Transportation, were selected for sealer testing. Bridges were chosen to cover a wide range of geographic and climatic regions (Figure 1).

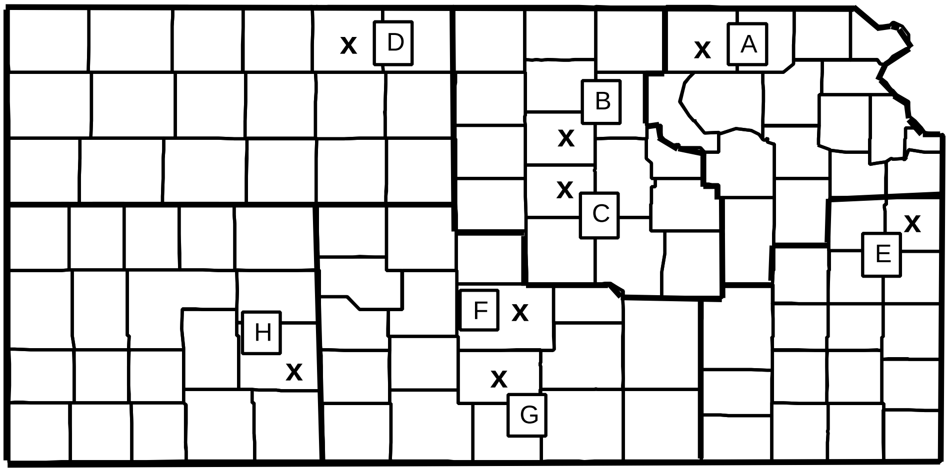

FIGURE 1 Location Of Sealed Bridge Decks

In addition to the location, the bridges chosen also had a significant amount of cracking of the deck. The structures chosen had a variety of substructures, and several had bridge deck wearing surfaces. At the time of sealer application, 1992, the structures varied in age from one to 29 years (Table 3).

Bridge	District	Substructure	Age
A	1	Steel Beam, Continuous	13
B	2	Steel Beam, Composite	2
C	2	Reinforced Concrete, Box Girder	13
D	3	Reinforced Concrete, Haunched Slab	29
E	4	Reinforced Concrete, Haunched Slab	14
F	5	Prestressed Beam, Composite	1
G	5	Steel Beam, Continuous	16
H	6	Prestressed Beam, Composite	12

TABLE 3 Bridge Location, Type, And Age.

Two types of HMWM sealer and one EPOXY sealer were chosen to be applied to the bridge decks. A third HMWM sealer was added to the laboratory portion of the study. The three HMWM materials had differing elongation, strength, and viscosity. The EPOXY sealer was a special low viscosity material developed for flood coat crack sealing. The physical properties that are significant to the performance of the sealers are listed in Table 4. Additional physical properties of the sealer materials are found in Appendix II.

Sealer	Viscosity Pa·s	Tensile Strength MPa	Tensile Elongation Percent
EPOXY	0.3 - 0.5	29.3	9.9
HMWM A	0.01 - 0.025	2.8	30
HMWM B	0.07 - 0.15	8.3	10
HMWM C	0.025	2.8	1.9

TABLE 4 Significant Material Properties

The next year, four sections of each bridge were selected for testing. Three sections received sealers and one section was identified as the control section. Two of the three selected sections of each bridge deck were treated with the HMWM A and HMWM B, one sealer for each section. The third selected section of each deck was treated with the EPOXY sealer. The test sections were each approximately 15.2 m in length and 3.7 to 9.1 m in width, depending upon bridge width and lane configuration.

Each section was sampled for chloride content prior to application of the sealers. Four sample locations were selected near the middle of each test section (Figure 2).

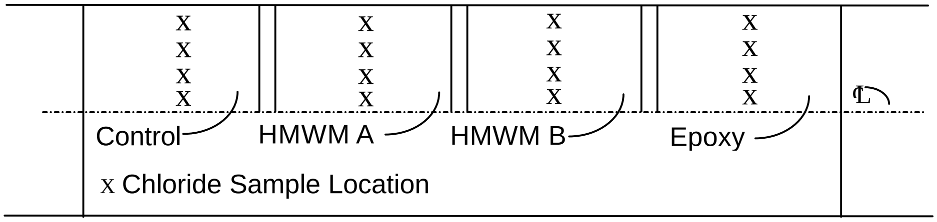

FIGURE 2 Typical Material Application And Chloride Sampling

One sample was taken at the curb line, one in each wheel path, and one between the wheel paths.

Samples were removed at three depths: between 0 and 19 mm, 19 mm and 38 mm, and 38 mm and 57 mm. Sample locations were not chosen randomly due to the high variability of chloride levels throughout a concrete bridge deck. In an effort to reduce the effect of location variability, non-random sampling allowed later samples to be taken at the same locations.

Samples were removed by vacuum drilling using a hollow bit placed in a vacuum swivel attached to an industrial vacuum and dust trap to capture the concrete dust. Collected chloride samples were returned to the Kansas Department of Transportation, Materials and Research Center. Water-soluble chloride levels were determined using the Kansas Department of Transportation Method 814, Water Soluble Chlorides (see Appendix III). Chloride levels as low as 0.6 kg/m$^{\mathbf{3}}$ of concrete (USDOT, FHWA, 1996) may result in the loss of passivity, and therefore allow corrosion of the reinforcing steel. The Kansas Department of Transportation historically considers a chloride level of 1.2 kg/m^3 as a critical level where corrosion of the reinforcing steel is almost assured.

Sealer applications were performed in June and July 1992. Previous to the application of the sealers, the test sections were cleaned using a high pressure washer capable of supplying pressures up to 28 MPa. Extra attention was given to large cracks during the cleaning procedure to ensure that as much debris as possible was removed. All asphalt-based crack sealers and deck patching materials were removed from the test sections. The decks were allowed to dry for 24 hours before the sealers were applied. If rain occurred during the 24-hour drying period, sealer application was delayed for an additional 24 hours. Due to the potential for a flash set of the materials at elevated temperatures, a maximum deck temperature of 32^{0}C was preferred. However the materials can be placed with deck temperatures as high as 38^{0}C. A flash set of the material would reduce or prevent penetration of the material into the cracks. Expansion of the decks at elevated temperatures will also tend to close the cracks, preventing penetration. Deck temperatures varied between 24^{0}C and 38^{0}C during application of the sealers. Applications were planned such that the deck temperatures would stay below 38^{0}C.

Each sealer was mixed according to the manufacturers' recommendations. The HMWM material was a three-part mixture, consisting of HMWM resin, cobalt promoter, and CHP (cumene hydroperoxide) initiator. Care was taken to prevent the peroxide and cobalt from coming in contact with each other, as the combination is flammable and potentially explosive. The general procedure was to prepare approximately 19 L of material at one time. Cobalt promoter was added to the resin first. After thorough mixing the CHP initiator was added, and the mixing was continued for several more minutes before application to the bridge deck. The two HMWM materials had different mixing ratios of resin, promoter, and initiator. Mixing ratios of each material are listed in Appendix II. The EPOXY system was a standard two-part system with a mixing ratio of 4 parts A to 1 part B by weight. To simplify field application, the material was measured and mixed by volume (a 3.5 to 1.0 volumetric mixing ratio). The general procedure was to prepare approximately 17.0 L of material: 3.8 L of part B was added to 13.2 L of part A and mix thoroughly. A complete mix was identified by a color change in both the HMWMs and the EPOXY. A complete mix usually required three to five minutes.

The sealers were applied approximately 17.0 L at a time by the flood coat method. Squeegees and brooms were used to ensure that sufficient material was applied to each crack. The equipment and supplies used to apply the sealers are listed in Appendix IV. Coverage rates varied between 1.74 m^2/L and 3.38 m^2/L depending on the material, and the bridge deck surface condition (Table 5).

Material	Minimum	Maximum	Average
EPOXY	1.79	3.26	2.28
HMWM A	2.03	3.38	2.63
HMWM B	1.74	3.26	2.29

TABLE 5 Application Rates Of Sealers, m^2/L.

Coverage rates were determined by dividing the area treated by the amount of material used to treat the area. Bridge decks with tining or with very large amounts of cracking required more material than decks with fewer or smaller cracks or no tining.

Tined bridge decks used more material due to the added surface area of the tining. The sealer materials also tended to pool in the bottom of the tining, filling them slightly and therefore, using more material.

Application rates suggested by suppliers and most commonly used by DOTs is approximately 2.45 m^2/L. As previously stated, coverage rates vary and will not effect the performance of the materials provided care is taken to fill the cracks as full as possible.

As the material was being spread, a slight head of material was maintained in front of the squeegee bar to ensure that enough material was available to fill the cracks. As each 17 L batch was being applied, another batch was prepared. This process was repeated until the area was completely covered and the cracks accepted no more material. Tined decks were swept with a stiff broom to remove excess sealer and to prevent filling of the tining.

All materials were allowed to cure until the surface became tacky or stringy when touched. Once the tack point was reached a coarse sand, an 8/20 or 10/20 blast sand, was applied at a rate of approximately 10 kg/m^2. The primary purpose of the application of the sealer was to seal the cracks in the bridge deck. Sealing the surface of the deck was of secondary importance, and minimally effective. Sealer remaining on the surface of the deck would become slick once completely cured. The sand was applied to give the bridge deck a non-skid surface until the excess material was worn off by traffic. The excess sealer remained on the deck for two to four years after application, depending on the material and the amount of traffic.

In late July 1992, after all materials had been applied, 50 mm cores were removed from the decks to determine the effective crack penetration of each sealer material. Cores were chosen such that each core contained a crack. Additional sets of 50-mm diameter cores were removed in the spring of 1993, 1994, and 1995 from each location. The cores were used for additional information on penetration, and to track the sealers' ability to withstand the environment experienced by bridge structures in Kansas. Very few cracks were not bonded together by the sealers when removed from the deck.

Cores were visually inspected for sealer penetration using a magnifying glass and a microscope, with and without ultraviolet light. The ultraviolet light caused the sealers to become slightly fluorescent, assisting in the visual evaluation of the cores. The width and depth of the cracks in the cores were recorded, as well as the apparent penetration. Each core was broken along the crack, if possible, and the exposed surface was visually inspected using the same procedure. All penetration depths, with and without ultraviolet light, were recorded (Appendix V). Excess dirt and laitance were also noted and recorded.

Samples for chloride testing were again removed from each sealed area and control area on each bridge in 1995. Samples were removed at the same depths and as close to the original (prior to sealer application) sample locations as possible, to attempt to offset the natural variability of chloride sampling (Appendix VI).

Penetration values varied widely in the 1992 and 1993 field data. Therefore, a set of laboratory experiments were developed and started in 1994. Field chloride levels were also found to be random when comparing 1995 data with 1992 data.

Laboratory Portion

Preparation Of Test Beams.

Standard Kansas Department of Transportation (Kansas, 1990) test beams, measuring 75 mm by 100 mm by 400 mm, were produced using the typical bridge deck high density overlay concrete mix noted in Table 6.

The concrete beams were produced in five batches due to the mixer limitations and to allow sufficient time for construction of the beams. Each batch included six test beams (a total of 30 beams) and three compression test cylinders. The five batches of concrete were numbered one through five, with the beams in each batch labeled A through F. Compression tests were performed on the cylinders according to ASTM C 39-86 (ASTM, 1993) to ensure consistency of the test mixes. The average compressive strength of the cylinders was 45.2 MPa, with a standard deviation of 2.6 Mpa.

Cement Content	371 kg/m^3
Water Content	145 kg/m^3
Coarse Aggregate	881 kg/m^3
Fine Aggregate	881 kg/m^3
Air Content	5 to 6.5%
Slump	25 mm
Density	84.4 kg/m^3

TABLE 6 Test Beam Concrete Mix

A 356 mm long, number 13 reinforcing bar was embedded in each beam at a depth of 50 mm. Reinforcing bars were supported in the molds during production of the beams by epoxy coated bar chairs. Epoxy-coated bar chairs were used to prevent corrosion of the bar chairs, and to prevent electrical continuity to the outside surface of the beam when testing for corrosion potential and corrosion rate. An insulated copper wire was attached to each end of each reinforcing bar with a stainless steel screw. The copper wires extended from each end of each beam and were used for the electrical connection to the corrosion potential and corrosion rate equipment during testing. The reinforcing bars were indicators for failure of the sealers. When the sealers failed, the chloride ions attacked the reinforcing bars causing corrosion to begin. The corrosion activity was detected by the corrosion rate equipment. The corrosion rate indicated the extent of the failure of the sealers. The reinforcing bar was a plain black bar with no epoxy coating. A black bar was used, so as not to introduce another unknown variable (the epoxy coating of the bar), and to expedite testing for corrosion at the time of sealer failure.

Beam Treatment.

After the standard 28-day curing period in the moist room, the beams were tested for corrosion potential and corrosion rate. After the corrosion tests were completed, the beams were subjected to an ASTM C 78-84 (ASTM, 1993) third-point loading sufficient to produce cracking to the depth of the reinforcing bar. In the field study, the cracks in each of the cores removed from the test bridges were measured and found to have an average crack width of approximately 0.3 mm. Small stainless steel shims were placed in the cracks of the laboratory prepared beams to maintain an average surface crack width of approximately 0.3 mm. Beams 1D, 2D, and 2F were damaged beyond use during the cracking process. The three damaged beams were tested to determine the initial chloride levels in the test beams and then discarded.

The beams to be treated were coated on the top surface with the sealers at application rates similar to those used in the field. The sides, bottom, and ends of each beam were coated with a water-proof, epoxy-based paint to prevent water penetration from surfaces other than the treated top surface. Repairs were made to the painted surfaces, as required, throughout the testing period.

Beam Testing.

The initial plan was to coat each of the six beams with one of the four sealer materials to be laboratory tested, and leave four beams unsealed as control beams. Due to the loss of three beams during preparation, one of the test groups only had five beams sealed. Five of the sealed beams from each group, and the four control beams, were subjected to wet/dry and freeze/thaw testing to determine the sealers' ability to withstand exposure to the elements.

An attempt was made to simulate the environmental conditions to which bridge decks are exposed in Kansas, using the standard test ASTM C 666-92 (ASTM, 1993) Procedure "B" for freezing conditions and cold-wet conditions. The Kansas Department of Transportation Wetting and Drying Test of Total Mixed Aggregate Concrete (KDOT, 1990) was used to simulate warm-wet and warm-dry conditions.

ASTM C 666-92 Procedure "B" has a temperature range between -18^{0}C and 4^{0}C. Beams were frozen in air and thawed in water. Each cycle is three hours in length, divided equally between freeze and thaw.

The Kansas Wetting and Drying Test involves placing the beams in a water bath for 16 hours at a temperature between 16^{0}C and 27^{0}C, and in an oven for 8 hours at a temperature between 53^{0}C and 55^{0}C. The cycle is performed daily, except on weekends and holidays when the beams are left in the water bath continually.

One of the five sealed beams from each group was to be used to track chloride intrusion by drill sampling. Concrete samples were removed from each of the four beams. The sample holes were sealed using a silicone sealer to prevent intrusion of the chlorides through the holes. This idea was later abandoned due to the rapid failure of the beams around the drill holes. The silicone sealer failed to seal the drill holes sufficiently to prevent moisture and chlorides from entering during the testing. However, the initial drillings were used to determine base-line chloride levels in the beams. The sixth sealed beam (fifth for one group) from each group was saw cut into sections to visually observe the sealer penetration. The beams included in this group were 1F, 2C, 2E, and 4B. Beam treatment is summarized in Appendix VII.

After cracking, and prior to sealer application, corrosion potential and corrosion rate levels were determined on each of the 27 remaining beams in both a dry state and a wet state. The cracked beams had much higher initial values than the uncracked beams. The sealers were then applied to 23 beams. The sealed beams were tested for corrosion potential and corrosion rate in a dry state before the freeze/thaw and wet/dry cycles were started.

By applying a voltage and measuring the current, the corrosion characteristics of a metal/solution electrochemical interface can be estimated (Cortest, 1992). The PR-Monitor measures the polarization resistance of the electrochemical interface. The polarization resistance is the resistance of the interface to direct-current flow. Polarization resistance is inversely proportional to the corrosion rate.

The polarization resistance technique was first defined by Stern and Geary (Stern and Geary, 1957). Mathematically, polarization resistance can be defined as the change in potential due to the change in current (dE/dI). The equation derived for dE/dI, (polarization resistance) is applicable at potentials very close (+/- 5 to +/- 20 mV) to the free corrosion potential. Solving the Stern and Geary equation for I_{cor}, the corrosion current, the equation becomes:

$$I_{cor} = \frac{B_a \times B_c}{2.3 \times (B_a + B_c) \times dE/dI} \qquad (7)$$

I_{cor} is the corrosion current and B_a and B_c are the anodic and cathodic Tafel constants respectively. B_a is the Tafel slope that characterizes the voltage versus current plot that defines the voltage necessary to start an anodic oxidation reaction in solution (moist concrete and noncorroding reinforcing steel interface). B_c is the Tafel slope that characterizes the voltage versus current plot that defines the voltage necessary to start a cathodic reduction reaction at the surface of the reinforcing steel. Tafel constants vary between 0.03 and 0.3 volts, depending upon the system. Stern and Geary showed that polarization resistance is equal to the slope of the linear plot of the potential versus current at the free corrosion potential. Polarization resistance measurement is accomplished by stepping the potential from (-5 to -20 mV) to (+5 to +20 mV) around the free corrosion potential, while measuring the applied current (Cortest, 1992). Once the polarization resistance is determined, I_{cor} can be calculated. The corrosion rate is determined by dividing I_{cor} by the area that is corroding and applying the proper conversion constant (see Page 9 and 10). In laboratory tests, the area corroding is the surface area of the sample. In bridge decks, the area corroding is the area of the reinforcing steel under the corrosion rate probe.

The corrosion potential and corrosion rate were determined using a Cortest Model PR-4500 Polarization Resistance (PR) Measurement System. The Cortest system is a micro-computer-based instrument capable of performing automated polarization measurements free of concrete (solution) resistance errors (US DOT, FHWA, 1991) using linear polarization to determine corrosion rate. The PR Monitor automatically steps the potential, plots the E versus I graph, reads the free corrosion potential, determines the PR, and calculates the corrosion rate (Cortest, 1992).

After the initial corrosion rates were obtained, the prepared beams were subjected to a series of freeze/thaw and wet/dry cycles to simulate the environmental conditions. Wet, dry, and freezing condition percentages were determined by contacting the National Weather Service. The average yearly number of snow days, rain days, days with temperatures below 0^0C, and the average number of days with temperatures above 32^0C were obtained (Table 7). The data was used to prevent excess exposure to freezing, heat or wet environments rather than to accurately simulate the actual Kansas weather.

Snow Days (greater than 25 mm)	7 days	1.9 %
Rain Days (greater than 2.5 mm)	96 days	26.3 %
Dry Days	262 days	71.8 %
Days Below 0^0 C	27 days	7.4 %
Days Between 0^0 and 32^0 C	297 days	81.4 %
Days Above 32^0 C	41 days	11.2 %

TABLE 7 Average Yearly Weather Conditions

As previously stated, an attempt was made to simulate the environmental conditions in Kansas using the standard test ASTM C 666-92 (ASTM, 1993) Procedure “B” for freezing conditions and cold-wet conditions. The Kansas Department of Transportation Wetting and Drying Test of Total Mixed Aggregate Concrete (KDOT, 1990) was used to simulate warm-wet and warm-dry conditions. The Kansas Department of Transportation has used the wet/dry test to identify alkali-silica reactive aggregates since the early 1940’s. The freeze/thaw test has been used in Kansas since the 1940’s to test aggregate soundness. The present procedure was implemented as a standard test for non-durable aggregates in 1981. Concrete and aggregate materials that successfully pass the freeze/thaw and wet/dry testing are able to withstand 25 to 30 years of service in Kansas. Tracking the number of test cycles to failure of the sealer allowed for an estimation of the life of the sealer by comparison to the standard test length of 300 cycles.

During the first four months of testing, the standard Kansas Wet/Dry and Freeze/Thaw testing procedures were followed (Appendix VIII). The beams were subjected to a continuous process of freeze/thaw, chloride ponding, wet/dry, chloride ponding, etc. The process consisted of approximately 39 freeze/thaw cycles, four days of chloride ponding, and 20 wet/dry cycles (Figure 3).

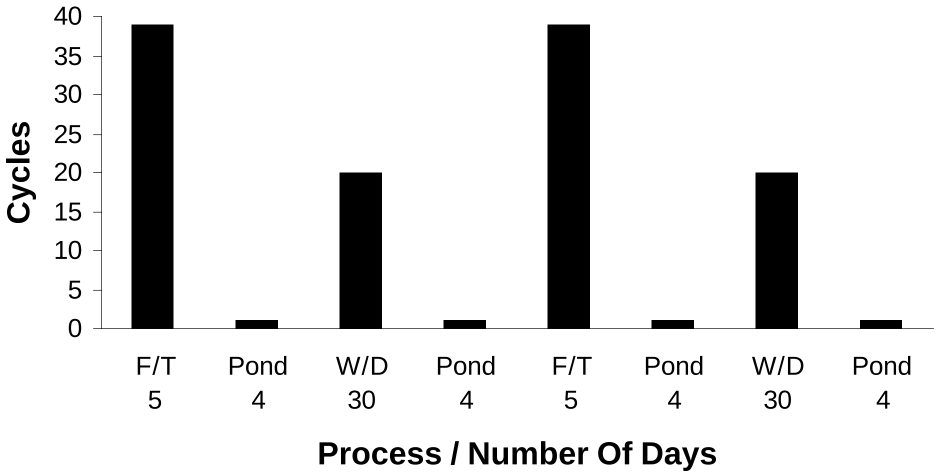

FIGURE 3 Testing Cycle

Evaluation of the amount of time in wet and dry conditions indicated that the beams were wet 65.1 percent of the time, dry 30.0 percent of the time and frozen 4.9 percent of the time. To reduce the amount of wet time, the testing schedule was adjusted to more closely represent actual Kansas weather conditions for the remaining five months of testing (Appendix VIII). The adjustment to the testing cycle included leaving the beams in the oven over the weekend rather than placing them in the water bath on Friday evening. The beams were not placed in the water bath until Monday night. This adjustment resulted in the beams being wet 44.1 percent of the time, dry 51.2 percent of the time, and frozen 4.9 percent of the time. Although this still was not perfect, it did more accurately represent the ratio of wet and dry weather in Kansas.

The adjustments to the treatment process affected the wet and dry time, but did not significantly affect the number of testing cycles the beams received. One week of standard wet/dry testing subjected the beams to five cycles. By leaving the beams in the oven from Friday morning until Monday evening, the beams received four wet/dry cycles each week.

During the ponding portion of the treatment process, a dam of packing tape and latex caulking was placed around the top edge of each beam. The tops of the beams were ponded with a seven percent (by weight) NaCl solution; e.g., a 4.2 percent chloride ion (Cl^-) solution. As noted by Uhling (Uhling, 1963), when the NaCl content exceeds approximately three percent, the oxygen solubility begins to decrease. A decrease in oxygen solubility decreases the amount of oxygen available for the corrosion process. Between two and five percent of NaCl, oxygen solubility varies only slightly. The beams were placed in the moist room at 100 percent humidity and 23^0C with plastic tented over the top surfaces to facilitate maintaining the NaCl solution level without overflowing the dam. The 100 percent humidity atmosphere of the moist room tended to dilute the ponding solution. A seven percent ponding solution was used in an effort to keep the NaCl concentration at, or above, three percent but not so high as to slow the corrosion reaction.

Corrosion potential and corrosion rate testing was performed after each ponding to determine if the sealer had failed and allowed the chloride to attack the reinforcing steel. Each beam testing was performed in a wet state in an attempt to obtain consistent readings (several low readings may have been due to the fact that the beams dried excessively between ponding and testing). The rate of corrosion measurement is a point in time indicator. The corrosion information obtained may change if temperature or moisture changes (US DOT, FHWA, 1991). Bridge deck readings should be taken continuously or intermittently, over a period of time, to obtain a more accurate assessment of corrosion conditions.

Visual inspection of the beams were made during corrosion rate measurements to evaluate the condition of the sealers and the beams. If excessive distress was noted, the beams were removed from testing. After the beams had either failed or the testing was concluded, chloride samples were removed from near the end of each beam and at the crack on each beam to determine the final chloride concentrations. Chloride samples were taken at depths of 25 and 45 mm.

Due to the inconsistency of the field chloride data, beams 3D, 4D, and 5D were placed under running tap water to determine if the chlorides could be leached out of the concrete. After several hours, the chloride levels dropped noticeably on seven of the 12 samples taken. The running water roughly imitated the excessively wet spring and summer that the state of Kansas experienced in 1993.

EXPERIMENTAL RESULTS OF FIELD DATA

Chloride Level Evaluation

As previously stated, the original plan was to evaluate the effectiveness of the sealers by comparing penetration of the sealers and chloride concentration increases in the concrete. Chloride levels in the structures were determined previous to the application of the sealers, and again after three years of service. Chloride level evaluations were performed by identifying four sample locations on each structure for each material and the control section. Samples were taken in 1992 and 1995 at three depths at each sample location: 0-19 mm, 19-37 mm, and 37-54 mm. To obtain an average chloride concentration for each sample depth, values were averaged.

The increase (or decrease) in chloride levels is the ratio of the 1995 to the 1992 values. A summary of the average ratios of the chloride concentration change between 1995 and 1992 is presented in Table 8.

	A	B	C	D	E	F	G	H
Control								
0-19 mm	0.98	1.05	1.07	1.13	1.00	1.79	1.31	2.46
19-38 mm	0.94	1.70	1.22	1.08	1.16	0.67	0.49	1.10
39-57 mm	1.10	0.64	0.76	0.94	1.09	1.13	0.58	1.48
EPOXY								
0-19 mm	1.51	0.88	0.77	1.11	0.98	1.19	0.95	3.80
19-38 mm	2.21	1.65	0.79	1.07	0.88	2.60	0.70	3.81
39-57 mm	3.14	0.56	0.80	1.08	1.17	1.45	0.87	1.35
HMWM A								
0-19 mm	1.10	0.99	1.26	1.05	0.67	1.46	1.27	0.36
19-38 mm	1.48	2.48	1.96	1.08	0.82	2.67	1.69	0.18
39-57 mm	1.42	1.18	2.27	1.33	0.84	2.19	1.44	0.44
HMWM B								
0-19 mm	1.66	0.85	0.96	0.97	1.32	1.35	1.15	3.90
19-38 mm	2.17	1.53	1.25	1.01	0.84	3.27	1.05	4.25
39-57 mm	3.55	0.87	1.08	1.12	0.70	1.25	0.97	3.65

TABLE 8 Average Ratio Of Chlorides (1995/1992), For Each Material, At Each Sample Depth, For Each Structure, A-H.

Over a period of three years, one would expect the chloride concentrations for an unsealed bridge deck to increase. A sealed bridge deck would be expected to remain at a constant contamination rate, or increase slightly, if the sealer was only partially effective. The average chloride concentration ratios of all the bridges combined is listed in Table 9 and shown in graphic form in Figure 4.

Sealer	Sample Depth, mm	Ratio 1995/1992
Control	0-19	1.35
	19-38	1.04
	39-57	0.96
EPOXY	0-19	1.40
	19-38	1.71
	39-57	1.30
HMWM A	0-19	1.02
	19-38	1.55
	39-57	1.39
HMWM B	0-19	1.52
	19-38	1.92
	39-57	1.65

TABLE 9 Average Ratio Of Chloride Levels For All Bridges.

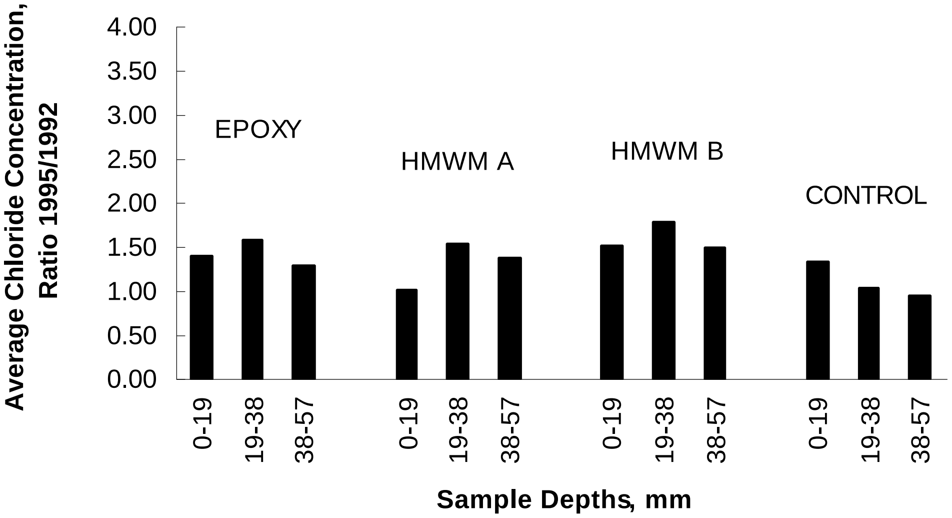

FIGURE 4 All Bridges, Average Ratio Of Chloride Levels

Again, when combining all of the structures, the average chloride concentration values are inconsistent and inconclusive and indicate very little difference as to the effectiveness of the sealers. The control sections, again, had a smaller increase in chloride content than the sections that were sealed.

Of the 370 samples, 93 samples were collected from the control sections and 277 samples were collected from the sealed sections. Fifty-nine percent of the samples taken from the control sections indicated an increase in the chloride concentration levels, 38 percent indicated a decrease, and three percent had no change, over three years.

The sealed sections had chloride concentration changes similar to the control sections. Fifty-seven percent of the samples removed from the sealed sections increased in chloride concentration, 42 percent decreased, and one percent had no change, after the three years. Percent increase, decrease and unchanged samples for each material are summarized in Table 10.

	Increase	Decrease	No Change
Control	59	38	3
EPOXY	58	41	1
HMWM A	54	44	2
HMWM B	58	41	1
All Sealers	57	42	1

TABLE 10 Percent Increase, Decrease, And No Change Of Chlorides.

The similarity between the sealed and control sections of the bridge decks indicated that the sealers were not effective. The increase of chlorides in the sealed sections may be explained by poor performance of the sealers. The decrease in chloride concentrations may be due to a leaching of chlorides out of the deck during heavy rains, such as those experienced throughout Kansas in the deluge of 1993. As previously stated, several of the test beams were subjected to running water for several hours to determine if the chloride concentration levels could be reduced by leaching. Seven of the 12 samples tested indicated a reduction in the chloride levels. This would indicate that leaching is a possibility. Any increase or decrease of chloride concentrations may also be due to the general inconsistency of chloride concentrations.

Reductions of the chloride levels were inconsistent due to the fact that, at some sample locations, the upper levels increased while lower levels decreased. Chloride concentration levels for the sealed portions of the decks were also inconsistent with increases at upper levels and decreases at lower levels. This type of inconsistency would indicate that the variation was influenced by the general inconsistency of chloride sampling. The location of the structure did not appear to have an effect on the inconsistency of the data.

The average ratio of the chloride concentrations for each sample depth, sealer, and control section for each structure is presented in graphical form in Appendix IX. Evaluating the sealer performance on each individual structure indicated that none of the three sealers out performed the others. The Epoxy sealer performed slightly better on four of the structures and the HMWM A sealer performed slightly better on three of the structures. The HMWM B sealer showed minimal performance on all structures. Evaluation of the increase of the chloride concentrations on the individual structures does not indicate a significant increase other than Bridges A, F and H which all had measurable increases.

The results would indicate that sealing an existing deck might trap chlorides in the system. The values may also have been effected by the general inconsistency of chloride sampling. Chloride samples removed from bridge decks tend to vary widely with location on the deck.

Various combinations of the data were looked at to determine if there was any type of trend that could explain the changes, or lack of changes, in the chloride concentration levels. No correlation was found. Appendix IX graphically indicates the high, low, and average chloride concentration levels for each sample depth, sealer, and control section for each structure for 1992 and 1995. Note that the chloride concentrations are plotted against the age of the structures in 1995. As noted in Table 3, the structures were of varying age and varying substructures. Table 11 lists the district the bridges are located in and the age of the structures in 1995.

Bridge	District	Age
A	1	16
B	2	5
C	2	16
D	3	32
E	4	17
F	5	4
G	5	19
H	6	15

TABLE 11 Age Of Structures, 1995

It is natural to assume that the age of the structure would influence initial chloride concentrations and the effectiveness of the sealers. This was not wholly supported by the field data as Bridge F, that is only four years old, had high chloride levels in the 0-19 mm sample depth range and Bridge G, which is relatively low in chloride levels, is 19 years old.

Sample depth 0 to 19 mm indicated that bridges B, D and F had high chloride levels on all test sections at the beginning of the testing period. This relationship remained relatively the same throughout the test period although the chloride concentration levels increased significantly for Bridge C in the HMWM A sections and for Bridge A in the HMWM B and the Epoxy test sections during the testing period. Bridge A was high in chlorides in the Control section throughout the test. Bridges B and F indicated higher chloride levels than one would expect considering the age of the structures, five and four years respectively. Both of these structures, however, are on highways with high traffic volumes.

Bridges A, C and D started with higher average chloride levels in the sealed sections and Bridges A, B and D had higher average chloride levels in the Control sections at the 19 to 38 mm sample depth. Bridge F also started with high chloride levels in the control section but not in the sealed sections. The level of chlorides in the Control section of Bridge F is an anomaly as the chloride levels in the sealed sections were quite low. The low chloride levels in the remainder of bridge F is what one would expect as Bridge F is only four years old. At the end of the testing period bridges A, B, C, and D had the highest chloride levels in the sealed sections. In the control sections, bridges A, B, D, and F continued to be high in chlorides. Bridge G had relatively low chloride levels considering the structure is 19 years old.

At sample depth 38 to 57 mm, bridges A, C, D and G generally had high chloride levels in the sealed sections throughout the test period. However, Bridge F was higher than Bridge G in the Epoxy section at the end of the test period. Bridges A, C, D, and H had high chloride levels in the Control section throughout the test period. Bridge F started the test period with low chloride contents, but increased noticeably by the end of the test period. Bridge B remained relatively low throughout the test period at this depth.

The four bridges (A, B, C, and D) in the northern half of Kansas generally had the higher chloride concentrations. Harsher winter weather requiring the use of more de-icing salt may be the explanation for this condition. However, bridges F and G were also found to have appreciable chloride concentration levels and were in the southern half of the state. Bridge F was four years old at the time of the test and bridge G was 19 years old at the time of the test. This is contrary to the expected trend. Usually, older structures would have higher chloride concentrations than newer structures. This inconsistency may be caused by salting patterns or by the public’s expectations of “dry” highways during winter weather and the Kansas Department of Transportation’s response to this desire by increasing the use of de-icing salt in recent years. Also bridges on highways with higher traffic volumes will tend to receive more treatments to increase traffic safety. Bridge G is on US Highway 54, a relatively high volume highway in southern Kansas.

The chloride concentration tests indicated that none of the sealer materials were effective in reducing the intrusion of chlorides into the bridge deck concrete. It is possible that the sealers may have trapped chlorides in the deck and, therefore, produced a more critical condition. However, the original purpose of the project was to evaluate the effectiveness of the sealers to seal existing cracks in the bridge deck. This point will be evaluated in the next section.

Material Penetration Evaluation

Penetration of the sealers was evaluated by inspecting 51 mm diameter cores. One core was removed from each test section on each bridge deck annually. The first set of cores was removed shortly after the application of the sealers in 1992. The last set of cores was removed in 1995.

The core locations were chosen such that each core included a crack. No effort was made to select locations with specific sized cracks. Cores removed from the bridge decks were of varying lengths. Cracks in the cores were found to extend completely through some cores and only partially through others (Appendix V). In some cases, the entire length of the crack in the core was not visible until the core was opened. Penetration was determined under natural and ultra-violet light by eye and microscope. Cracks in the cores were significantly contaminated with dirt from the surface. Many cores indicated contamination well below the penetration level of the sealers.

Core length, crack depth, crack width, depth of penetration of the sealers, and contamination were noted for each core. Penetration data varied widely with little or no correlation (Appendix V). Parameters investigated included crack width, crack depth, age of the structure, material type, penetration, and percent penetration. Percent penetration was found to be more useful in the analysis of the effectiveness of the sealers. Penetration means very little unless all of the cracks have the same depth. Penetration of 10 mm into a 12 mm crack would be significant. However, penetration of 10 mm into a 100 mm crack would be considered poor.

The relationship between crack width and crack depth was the first relationship to be evaluated (Figure 5). A general trend of the wider cracks being the deeper cracks can be determined. Therefore, one would expect that the wider cracks would have deeper penetration values. One would also expect that the wider cracks would have a higher percent penetration as the material should be able to penetrate a wider crack more easily.

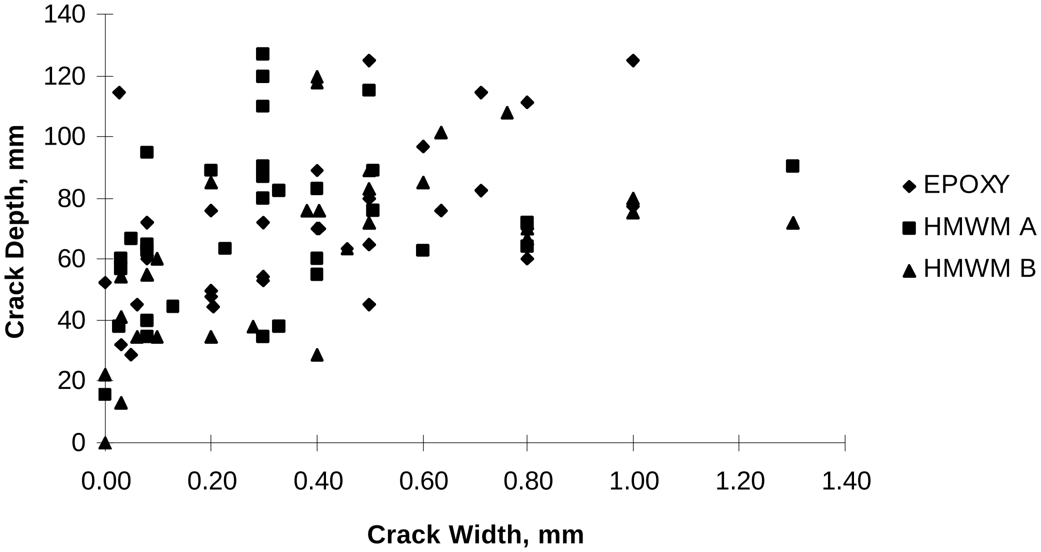

FIGURE 5 Crack Width Vs. Crack Depth

However, Figure 6 indicates that there is little correlation between crack depth and penetration. Many of the shallow cracks were only partially filled, while some of the deep cracks were completely filled. The one data point above the equality line was due to a crack that was difficult to see previous to breaking of the core. The data in Figures 7 and 8 indicates that comparing percent penetration to crack width also varies widely.

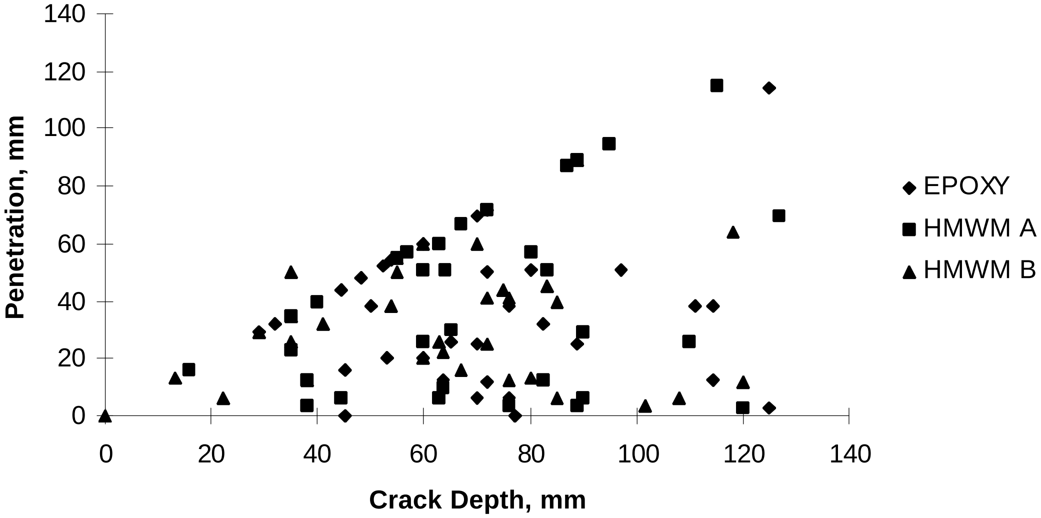

FIGURE 6 Penetration Vs. Crack Depth

Percent penetration compared to crack width (Figure 7) indicates a slight tendency to have less penetration as the cracks become wider. The data is split into material type, if the crack was completely penetrated or only partially penetrated, and by the material. No one material appeared to have performed better than the others. Only one of the cracks with 100 percent penetration had a crack width over 0.5 mm. This does not follow the logic that a wider crack would be easier to penetrate.

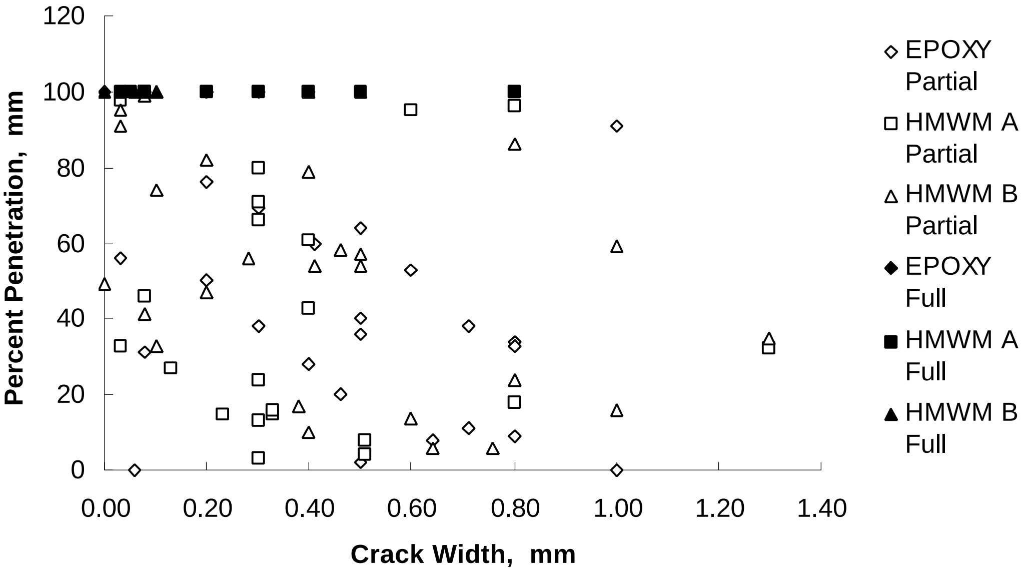

FIGURE 7 Percent Penetration (Area) Vs. Crack Width

By dividing the penetration data into partial and full crack penetration and plotting this against crack width, two groupings can be identified (Figure 8). The full penetration samples generally clustered above a penetration depth of 30 mm while the partial penetration samples generally clustered below 60 mm. This figure, again, shows the tendency for the full penetration cracks to be less than 0.5 mm in width.

Cracks with a depth greater than 30 mm, and a width less than 0.5 mm may fill more effectively due to a larger head of material in the crack and a cleaner crack. The head would help to drive the material deeper into the cracks and the narrower cracks would tend not to be as contaminated. The data indicates that the cracks can become too wide and hinder penetration. This may be due to the ability of contaminants to more easily penetrate the wider cracks. The wider cracks, although possibly deeper, have a greater tendency to be excessively contaminated reducing the penetration.

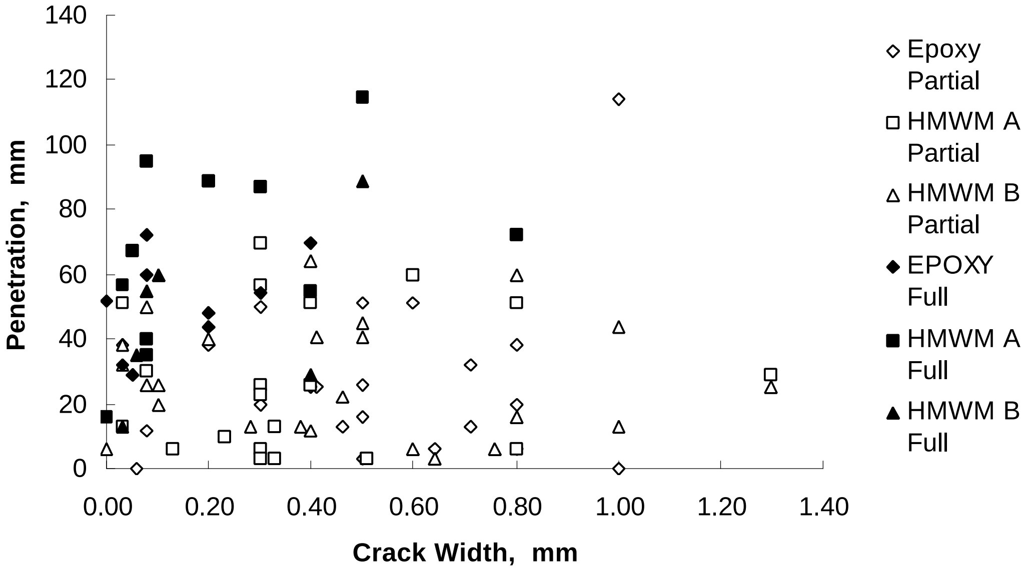

FIGURE 8 Partial And Full Penetration Vs Crack Width

The data indicated that the age of the structure had only a limited affect on the penetration of the sealers (Figure 9). Structures varied in age from one to 29 years when the material was applied.

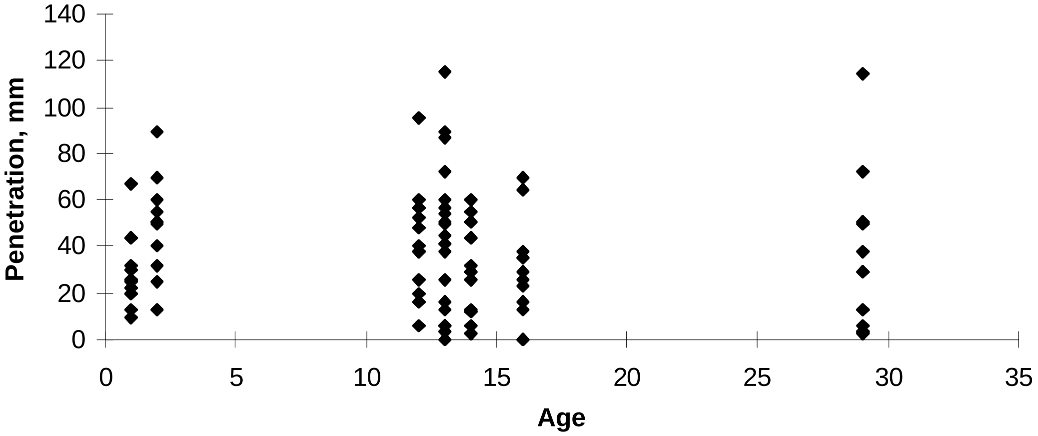

FIGURE 9 Percent Penetration (Area) Vs Age

Percent penetration varied greatly on all structures. However, the two newest structures (F and B), one and two years old at the time of application, had a minimum penetration of 15 percent. Structure H, 12 years old, had a minimum penetration of 38 percent. Cores removed from structure H had an average crack width of only 0.09 mm. This would indicate that on a newer structure the materials can more effectively penetrate the cracks. This is probably due to the cracks having been exposed for a shorter period of time to contaminants. The penetration of the cracks on the older structure supports the theory that the narrower cracks do not fill with contaminants as easily as wider cracks.

The Average Crack Width, Average Penetration, and Average Percent Penetration for each structure, separated by materials, are listed in Table 12. The crack widths varied widely for each structure and for each material. Average crack width varied widely from structure to structure. The average percent penetration also varied widely from structure to structure. HMWM A has an Average Percent Penetration of 62 percent with an Average Crack Width of 0.31 mm. Average Percent Penetration indicates that HMWM A penetrated the cracks better than HMWM B and the EPOXY. The HMWM A material penetration is more significant when the smaller Average Crack Width is considered.

Evaluation of the penetration data indicated that the penetration of the materials was inconsistent. However, the HMWM A material penetrated slightly better than the other materials. Crack width may be a critical factor in penetration effectiveness. Wider cracks, greater than 0.5 mm, may allow excess contamination thereby reducing the ability of the sealers to penetrate fully. Age of the structure may have some affect on the penetration. Newer structures, one to two years old, may tend to have cracks with lesser amounts of contamination. The lower contamination could increase the penetration of material.

		Average Crack Width (mm)	Average Penetration (mm)	Average Percent Penetration (crack area)
EPOXY				
Bridge A		0.58	33	53
Bridge B		0.48	51	68
Bridge C		0.60	32	38
Bridge D		0.58	40	38
Bridge E		0.10	37	83
Bridge F		0.54	21	30
Bridge G		0.16	21	48
Bridge H		0.18	40	78
	AVE	**0.40**	**34**	**55**
HMWM A				
Bridge A		0.40	66	70
Bridge B		0.43	45	68
Bridge C		0.27	26	34
Bridge D		0.81	39	61
Bridge E		0.19	22	38
Bridge F		0.19	33	51
Bridge G		0.18	35	70
Bridge H		0.05	52	100
	AVE	**0.32**	**40**	**62**
HMWM B				
Bridge A		0.37	44	66
Bridge B		0.60	42	50
Bridge C		0.74	22	32
Bridge D		0.47	20	51
Bridge E		0.29	27	66
Bridge F		0.40	30	61
Bridge G		0.14	31	84
Bridge H		0.05	33	70
	AVE	**0.39**	**32**	**60**

TABLE 12 Average Crack Width, Penetration, And Percent Penetration

EXPERIMENTAL RESULTS OF LABORATORY DATA

The results of the laboratory testing were much more consistent than the results of the field tests. Definite corrosion rate trends were measured as the concrete samples were subjected to continued salt ponding, freeze/thaw, and wet/dry testing cycles. As previously stated, several corrosion rate devices are available on the market. The Cortest Model PR4500 Polarization Resistance Measuring System, manufactured by Cortest Instrument Systems, was chosen to determine the corrosion potentials and corrosion rates for this study.

Bridges are generally designed for a life span of at least 50 years, with many requiring major deck repairs or deck replacement after 25 years. Therefore, preventing damaging corrosion for 10 to 15 years would increase the probability that the deck and entire structure would meet the design life. The corrosion rate and damage relationships stated in Tables 1 and 2, indicate a corrosion rate cap of 1.0 $\mu A/cm^2$ (approximately 0.46 mpy) is reasonable to minimize corrosion damage to sealed bridge decks.

Estimates of the length of time the sealers would be effective were developed by determining when the corrosion rate exceeded 1.0 $\mu A/cm^2$. The testing schedule previously outlined states that, after a period of 43 days, the material would have been subjected to a complete cycle of testing. One complete cycle of testing included approximately five days of freeze/thaw exposure, 30 days of wet/dry exposure, and four days of chloride ponding after both the freeze/thaw and wet/dry exposure. A complete testing cycle included 39 freeze/thaw cycles and 20 wet/dry cycles. Some variation to the duration of the test was necessary to allow for holidays and required field work.

By using the date that the test beam corrosion exceeded 1.0 $\mu A/cm^2$ and back calculating, the approximate number of test cycles to which the test beams were subjected, can be determined. Equation 8 is a proposed formula to determine the approximate length of time a sealer can protect a bridge deck. F/T and W/D are Freeze/Thaw and Wet/Dry cycles respectively.

$$\text{Estimated Life} = \left[\left[\frac{F/T}{300} + \frac{W/D}{300}\right] \Big/ 2\right] \text{X } 30 \qquad (8)$$

As previously stated, each of the standard test procedures used to evaluate the performance of the sealers required the test beams to survive 300 cycles. The Kansas Department of Transportation has an established history of using the Freeze/Thaw and Wet/Dry tests for aggregate and concrete durability. Concrete and aggregate materials that successfully pass the Freeze/Thaw and Wet/Dry testing withstand 25 to 30 years of service in Kansas. The proposed equation assumes that the Freeze/Thaw and Wet/Dry cycles have an equal effect on the deterioration of the sealers. The testing procedure was determined to prevent an unbalanced exposure of the materials to either Wet/Dry or Freeze/Thaw cycles. The number of cycles that the materials were subjected to were noted during the testing and are listed in the following evaluation of the sealer performance.

The unsealed control beams failed rapidly (Figure 10). This series of beams exceeded the damaging corrosion limit of 1.0 μA/cm^2 after only 50 days.

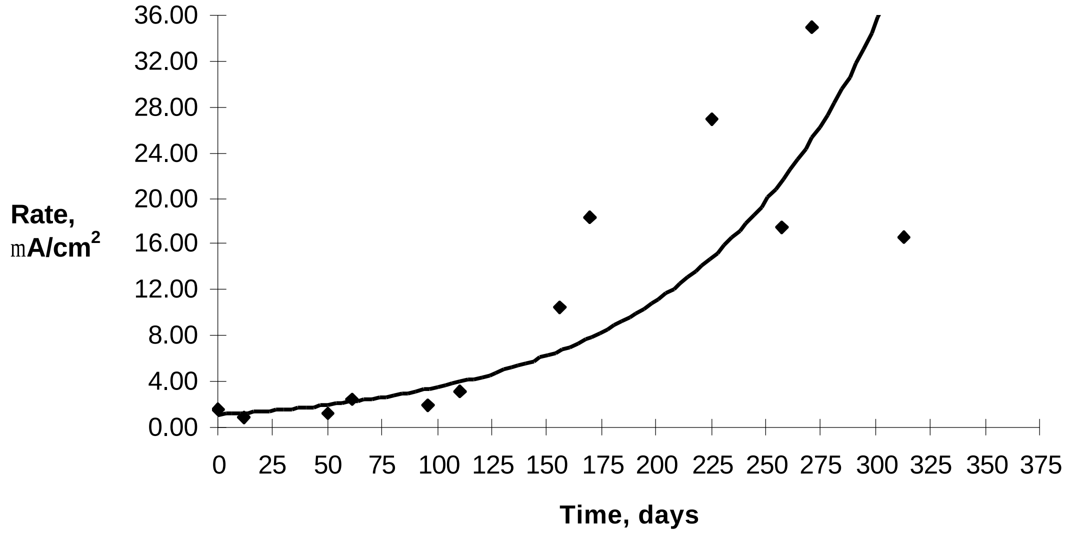

FIGURE 10 Corrosion Rate For The Control Beams.

The control beams were subjected to only 62 freeze/thaw cycles and 19 wet/dry cycles. The proposed damage equation indicates that a structure that has cracked, and has unprotected reinforcing steel, and is subjected to de-icing salt treatment, could develop damaging corrosion rates of 1.0 μA/cm^2 within four to five years.

The EPOXY sealed beams exceeded the 1.0 μA/cm^2 limit (Figure 11) after 271 days. This series of beams was subjected to 234 freeze /thaw cycles and 95 wet/dry cycles. The proposed equation indicates the EPOXY material could protect a bridge for 15 years or more.

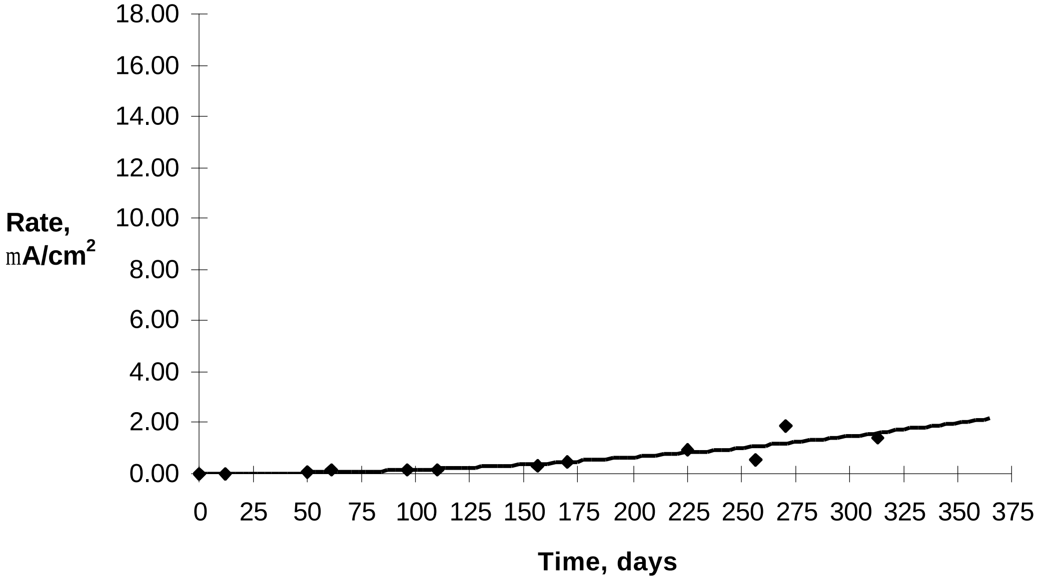

FIGURE 11 Corrosion Rate For The EPOXY Sealed Beams

The corrosion rate of the HMWM A sealed beams exceeded the 1.0 μA/cm^2 limit after approximately 156 days of testing (Figure 12). These beams were subjected to 117 freeze/thaw cycles and 61 wet/dry cycles. Again, the proposed equation indicates the HMWM A could protect a bridge for up to nine years.

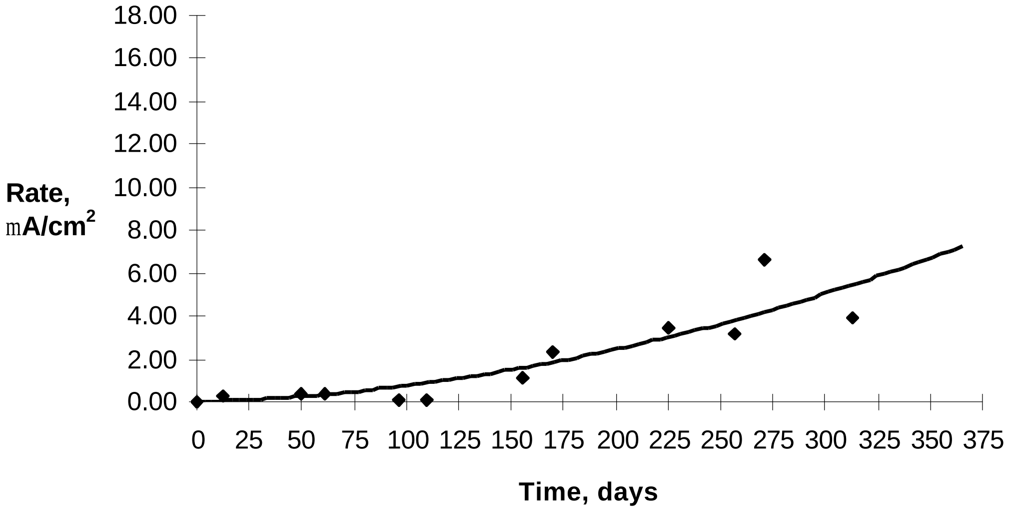

FIGURE 12 Corrosion Rate For The HMWM A Sealed Beams

The HMWM B sealed beams reached the 1.0 µA/cm^2 corrosion rate limit after approximately 170 days (Figure 13). The beams were subjected to 264 freeze/thaw cycles and 61 wet/dry cycles.

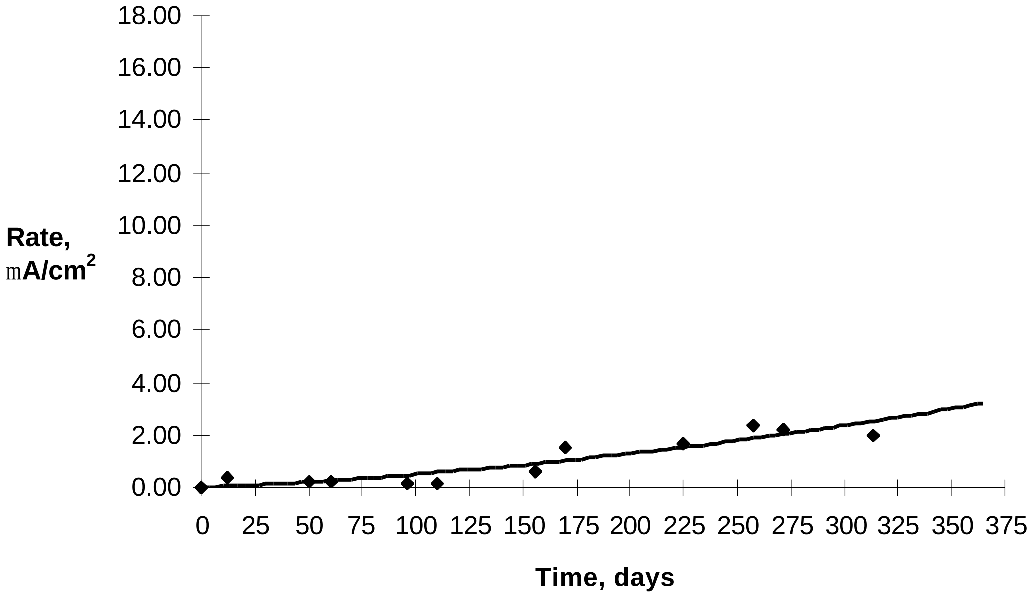

FIGURE 13 Corrosion Rate For The HMWM B Sealed Beams

The HMWM B material performed almost as well as the EPOXY material but the slope of the graph indicates that the performance rate would not have continued. The

HMWM B material, using the proposed equation, could protect a bridge for up to 11 years.

The HMWM C material failed first in the laboratory testing. The corrosion rate for the HMWM C sealed beams exceeded the 1.0 µA/cm^2 limit at approximately 110 days (Figure 14). The beams were subjected to 117 freeze/thaw cycles and 39 wet/dry cycles. Applying the proposed equation indicates that this material could protect a structure for a period of only eight years.

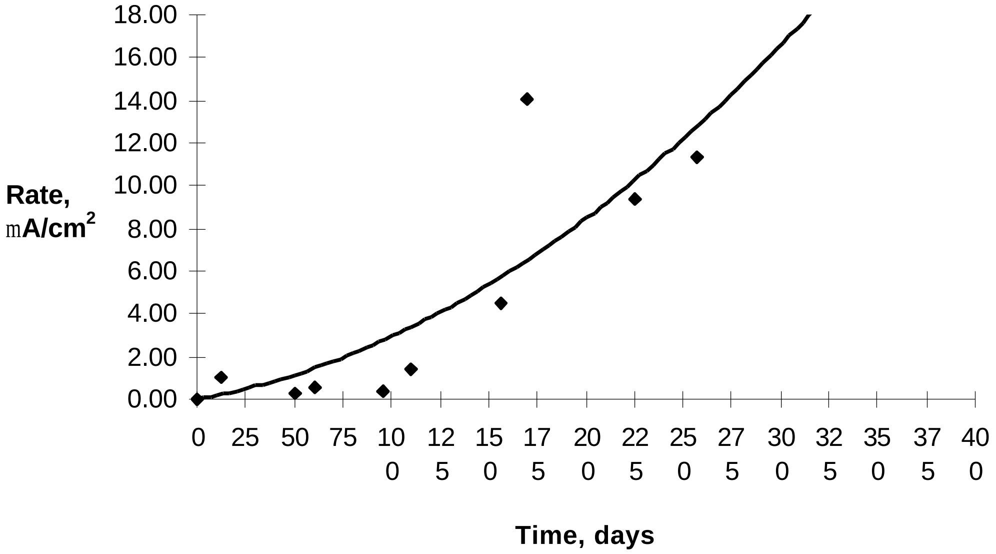

FIGURE 14 Corrosion Rate For The HMWM C Sealed Beams

The marked difference between the four sealers tested and the rapid corrosion attack on the unsealed beams is presented in Figure 15. The differences in the durability of the materials was due in part to the difference in the physical properties of the sealers.

The three material properties which are believed to be significant in the ability of the sealers to seal cracks for an extended period of time are viscosity, tensile strength, and tensile elongation. The viscosity is significant to the ability of the sealers to penetrate the cracks. The tensile strength and tensile elongation are significant to the material’s longevity as a sealer.

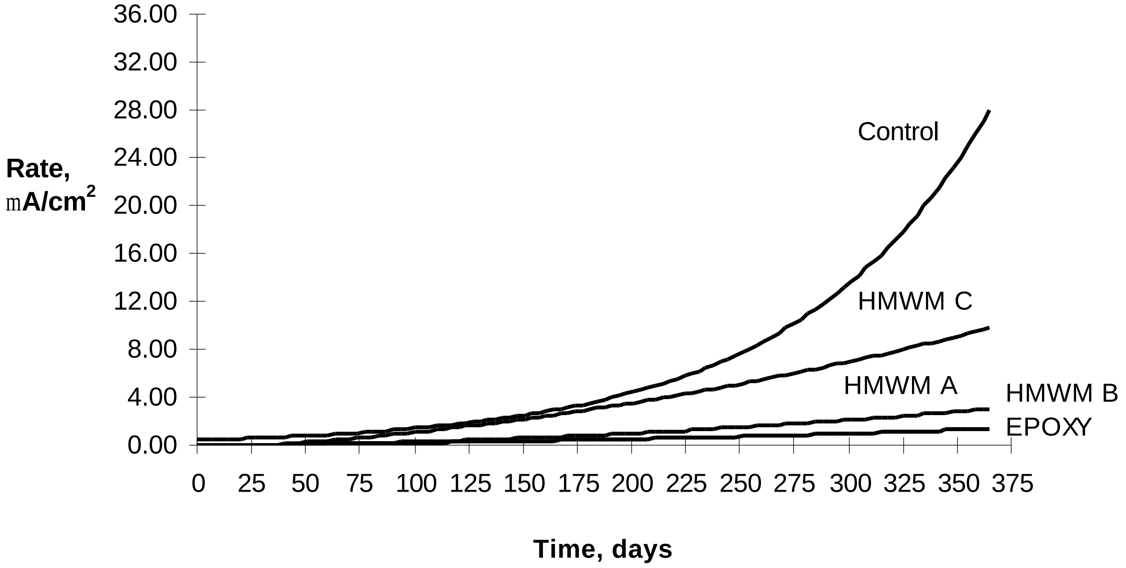

FIGURE 15 Corrosion Rate For Sealed And Control Beams

The laboratory testing did not include induced flexural stresses. All stresses induced were done so by the temperature differentials between the outside of the beams and the inside of the beams with the reinforcing bars, as the Wet/Dry and Freeze/Thaw procedures were carried out.

The widest range of temperature variation was caused when the beams were moved from room temperature (24^{o}C) to the low temperature of the Freeze/Thaw test (-18^{o}C). However, this 42^{o}C temperature differential was found only at the beginning and end of the Freeze/Thaw cycles.

The Wet/Dry test, however, subjected the beams to a temperature differential of 39^{o}C twice in each cycle. The oven warmed the beams slowly from a water temperature of 16^{o}C to the oven temperature of 55^{o}C. This temperature change would be slow enough that the outside and inside of the beams would expand at nearly the same rate. The steel reinforcing bar would also expand at nearly the same rate as the concrete surrounding it. A coefficient of thermal expansion of 0.000012 per degree centigrade can be used for both concrete and steel. However, when the beams were removed from the oven and placed in the water, the temperature drop on the surface of the beams would be rapid.

The worse case situation would be that the center of the beams and the reinforcing steel would still be at oven temperature. The concrete on the surface would attempt to contract. The reduction in length for a temperature change of 39°C, on 400 mm long beams, would be 0.1872 mm. A reduction in the length of the reinforcing bar of 0.1872 mm is a strain of 0.00053 mm/mm. A strain of this magnitude would require a stress of 105.7 MPa, well above the tensile strength of most concrete. Therefore, if the beams were not previously cracked, the deferential change in length would induce a crack. However, the stresses induced should open the existing crack, not create a second crack.

The forces exerted on the reinforcing bar by the concrete are analogous to a reinforced concrete tensile member. The maximum crack spacing for a reinforced concrete tensile member is equal to about four times the concrete cover thickness (ACI 1991). The concrete cover for the reinforcing steel in the beams is 51 mm, the maximum crack spacing is 200 mm. The cracks in the beams were placed at 200 mm.

Equation 9, developed by Broms and Lutz (ACI 1991), can be used to determine the maximum tensile crack width, W_{max}.

$$W_{max} = 4\, \varepsilon_s\, t_e \qquad (9)$$

In the previous equation, ε_s is the strain in the reinforcing steel and t_e is the effective concrete cover. The effective concrete cover for members with more than one reinforcing bar is a function of the reinforcing bar spacing. The test beams had only the single bar in the center of the beams, therefore, the effective concrete cover is the actual cover.

Using ε_s equal to 0.00053 mm/mm and t_e equal to 51 mm, W_{max} is 0.108 mm. The cracks produced in the beams previous to sealing were an average width of 0.30 mm. A crack width increase of 36 percent is determined by dividing 0.108 mm by 0.30 mm. This percent increase would be the worst case situation, the actual crack increase would be less, due to some cooling of the center of the beam and the reinforcing steel.

None of the materials tested had tensile elongation as high as 36 percent. However, evaluation of the materials' ability to withstand the testing procedure indicates that the tensile elongation was not the only material property that effected durability.

As noted previously in Table 4, the EPOXY had a higher viscosity (0.3-0.5 Pa·s), higher tensile strength (29.3 MPa), and 9.9 percent tensile elongation, when compared to the other materials. The higher viscosity may have prevented the material from being as efficient in penetrating the cracks. However, the higher strength (29.3 MPa), and tensile elongation (9.9%), made the material that did penetrate more durable.

HMWM B performed nearly as well as the EPOXY. This performance was due to the tensile strength (8.3 MPa) and tensile elongation of ten percent. The HMWM B material was able to move with the concrete in the rapidly changing environment created by the Wet/Dry testing. The higher tensile strength improved the HMWM B durability also.

HMWM A had similar viscosity (0.025 Pa·s) and tensile strength (2.8 MPa) as HMWM C. The difference between the two materials was that HMWM A had a tensile elongation of 30 percent. The high flexibility of the material made it better able to withstand the movement caused by the extreme changes in the environment. However, the lower tensile strength may have been the reason that the HMWM A material did not perform as well as the HMWM B, which had a lower viscosity.

HMWM C failed first in the laboratory testing. As previously stated, the material had a very low viscosity (0.025 Pa·s) and tensile strength (2.8 MPa). The material was very brittle with only a 1.9% tensile elongation. Due to the combination of low tensile strength and minimal elongation, HMWM C was not able to withstand the movement due to the rapid environmental changes.

As the testing proceeded, the beams were visually evaluated for distress when the corrosion rate readings were performed. The unsealed Control beams began to show surface distress as early as 120 days into the test. The sealers indicated some surface damage at 120 days. By 170 days into the testing, the unsealed Control beams were heavily scaled and the HMWM C sealer had long surface cracks and some heaving of the material. The other sealers showed little change. At 270 days into the testing, the unsealed Control beams had considerable surface damage and cracking. The HMWM C sealer was map cracked on the surface with longitudinal cracks. The beams sealed with the HMWM C material were in poor condition. The other sealed beams indicated little change.

At 280 days, the HMWM C beams were removed from testing due to extensive cracking of the beams. At this point, the beams with other sealers were beginning to show some surface distress, but the cracks in the beams were apparently still reasonably well sealed. At 285 days into the testing, the condition of the unsealed Control beams was such that consideration was given to stopping the testing. The remaining sealed beams, however, indicated little change. At 328 days, the Control beams were in such a condition that the testing was stopped. The remaining sealed beams were still in good enough condition to continue testing.

After completion of the testing, several of the sealed beams were cut at several locations to visually evaluate the extent of the corrosion on the reinforcing bar. There was minimal corrosion on the reinforcing bars. The unsealed beams had slightly more corrosion but the amount was still relatively small. This is an indication of how little corrosion is necessary to begin damaging a structure.

Visual penetration into the cracks of beams 1F, 2C, 2E and 4B was found to be between 13 and 25 mm. The EPOXY had the highest visual penetration with 25 mm. The ultra-violet penetration varied between 38 mm and 51 mm. All materials had an ultra-violet penetration of 51 mm. It should be noted that the reinforcing steel in the specimens was placed at a depth of approximately 50 mm.

The laboratory portion of the study indicated that if the material is able to penetrate consistently into the cracks, the sealer can afford protection to the bridge deck. The materials that performed the best in the laboratory portion had two specific physical properties that enhanced their performance. Those two properties were tensile strength and tensile elongation. Of these two physical properties, the tensile elongation appears to be the most significant. The Epoxy had a very high tensile strength (29.3 MPa) and HMWM B had a significant tensile strength (8.3 MPa). Both materials had a tensile elongation of ten percent. HMWM A did not perform as well as HMWM B and the Epoxy due to the fact that the tensile strength was low (2.8 MPa).

However, the high elongation, 30 percent, offset the effect of the low tensile strength and allowed the HMWM A to perform fairly well. These results indicate that the most effective sealer would be one with a relatively low viscosity, a tensile strength of at least eight (8) MPa, and a tensile elongation of at least ten (10) percent. An elongation value of 20 to 30 percent would be preferred if tensile strength is not reduced.

SUMMARY AND CONCLUSIONS

The results of the field portion of the study were inconclusive. Chloride concentration levels of the sealed sections were inconsistent with 57 percent of the samples increasing, 42 percent decreasing, and one percent remaining the same. In some cases, the upper levels increased while lower levels decreased. Chloride concentration levels of the control sections (unsealed portion) were also inconsistent. Forty-eight percent of the sample locations decreased in chloride concentration and 59 percent increased. Again, some locations increased at the upper levels and decreased at lower levels. In some cases, the sealed portions of the bridge deck had higher chloride concentration increases than the control section. This indicates that the sealers could trap chlorides in the system and actually worsen the conditions.

The structure type and age did not appear to have any effect on the performance of the sealers. The only item that appeared to have affected the increase of the chloride concentration was whether the structure was in the northern half or southern half of the state. This issue appeared to act upon all of the test sections equally, regardless of the sealer type or lack of sealer.

Evaluation of the cores removed from the test sections on each bridge indicated that the penetration of the sealers also varied. None of the three materials consistently penetrated the full depth of the cracks. The evaluation of the cores for material penetration indicated a large amount of dirt and laitance on the walls of the cracks. This contamination was also noted by the Virginia DOT (Sprinkle, 1992). VDOT indicated that the contamination has a direct effect on the penetration of the material and the ability of the material to bond the cracks. This also appears to have caused a reduction in penetration in this evaluation.

Various parameters were investigated with only a few correlations identified. In general, crack width increases with crack depth. One would expect deeper cracks to be wider and easier to penetrate. However, the data indicated very little correlation between crack depth and penetration. Crack Width versus Percent Penetration was also varied. However, all but one of the cracks which were 100 percent full were 0.5 mm in width or less. Crack Width versus Penetration indicated that the cracks which were 100 percent full were 30 mm or greater in depth. The results may be due to a combination of items. The slightly deeper crack would have a larger head of material to assist in forcing the material deeper into the crack. The smaller cracks, 0.5 mm or less, may not be as easily contaminated. Therefore, the combination of a cleaner crack with slightly more head may account for additional penetration.

Age of the structures had minimal effect on the penetration of the sealers. Structures F and B, one and two years old respectively when the sealers were applied, had minimum percent penetration values of 15 percent. Structure H, 12 years old, had a minimum penetration of 38 percent. Cores removed from structure H had an average crack width of only 0.09 mm. Indicating that narrower cracks may not become contaminated as easily.

The penetration data indicates that the extremely low viscosity HMWM A sealer may have been more effective in penetrating the cracks than the other two sealers. The HMWM A sealer did not show any greater ability to prevent an increase in chloride concentration.

The results of the laboratory tests on the three sealers tested in the field and the additional HMWM sealer added for the laboratory evaluation indicated a definite difference in the performance of the four sealers. Laboratory results indicated that the sealing of cracks in a new bridge deck may be beneficial, as the sealed beams did not deteriorate as rapidly as the unsealed beams.

The differences in the performance of the sealers appears to be tied more to material properties such as viscosity, flexibility, and tensile strength, rather than to penetration. Each of the materials penetrated the laboratory beams used for penetration evaluation to the depth of the reinforcing steel (50 mm).

The EPOXY and HMWM B sealers performed well in protecting the test beams throughout the laboratory evaluation. The HMWM A sealer did not protect the beams for as long a period, but was considerably better than the HMWM C sealer which failed early in the testing. The HMWM C material provided the least amount of protection. The protection afforded by the EPOXY was probably due to its high tensile strength and some flexibility. The HMWM B sealer performed nearly as well as the EPOXY.

This was probably due to the fact that it had the highest tensile strength of the HMWM sealers, and tensile elongation of ten (10) percent. The HMWM A material had a low tensile strength similar to that of the HMWM C sealer. However, the HMWM A sealer had a high elongation.

The effectiveness of the sealers is questionable when only the field results are considered. However, when the laboratory results are evaluated, it appears that a relatively low viscosity, high tensile strength and flexible material can protect a new cracked bridge deck. The sealer must be properly applied, shortly after the cracks are formed, to maximize the penetration and protection of the structure. By applying the material as early as possible, the cracks will have a minimum amount of contamination. Thus, one optimizes the sealers' ability to penetrate the cracks. Attempts should be made to clean the cracks before application of the sealers. The optimum sealer would be one with a relatively low viscosity, 0.5 Pa·s or less, tensile elongation of ten (10) percent or more and a tensile strength of at least eight (8) MPa.

Although a sealer may not fully penetrate, or completely seal a crack, it may still be beneficial. Any reduction in the amount of water and chloride intrusion into a bridge deck has the potential to slow corrosion and reduce freeze/thaw damage.

Recommendations For Further Work

There have been a number of studies performed on the crack sealer materials with a wide variety of results. Varying results are caused by the difficulty of simulating actual field conditions of the bridge decks in the laboratory for controlled testing. A method to simulate the pre-existence of chlorides in the concrete and the contamination in the cracks should be developed. Additional testing should be performed to determine the interaction between the freeze/thaw and the wet/dry portions of the testing to validate the proposed damage equation. Work should be performed to determine the affect flexural bending, produced in the bridge decks by traffic, has on the durability of the sealers.

Additional field work should be performed to further evaluate the affect that various physical properties of the sealers have on the ability to penetrate cracks, and on the longevity of the protection offered by the sealers. More effective methods to clean the cracks in the bridge decks should be identified to improve the penetration of the sealers and increase the effectiveness. Several new or near new structures (one to two years old) with cracked decks should be treated with the sealers to evaluate the penetration performance on cracks that have not been heavily contaminated by dirt and debris.

IMPLEMENTATION

The Kansas Department of Transportation has adopted the use of HMWMs and Epoxy healer sealers for use on newer structures which have developed excessive cracking of either the bridge deck or the bridge deck overlay. At the present time the Special Provision in Appendix 5 is used for HMWM applications and material approval. A Special Provision for Epoxy application and material approval has not been written. However material approval will be done following ASTM C 881 for epoxy materials and application instructions will be similar to those outlined in the HMWM Special Provision.

Several bridges have been treated with HMWM and portions of several structures have been treated with an Epoxy Sealer. The first structure treated with HMWM that was not an experimental application was Bridge No. 11 on state highway K-99 in Lyon County. This structure was constructed with a high-density concrete overlay. Due to equipment problems and hot windy weather the overlay had excessive cracking with some de-bonding from the subdeck. Holes were drilled and the de-bonded areas were low pressure injected with HMWM to bond the overlay to the subdeck. The entire surface of the overlay was then flood coated with a high elasticity HMWM. Cores were removed form this structure at several locations to determine the effectiveness of the injection and to determine the penetration of the HMWM into the cracks. The injection process was found to be successful in bonding the overlay to the subdeck and the HMWM was found to have penetrated the cracks to a depth of 25 to 50 mm.

The other two structures which have been treated with HMWM were Bridge No. 60 on US highway U-283 in Norton County and Bridge No. 44 on state highway K-18 in Graham County. Both structures were constructed of 90 mm prestressed panels with a 130 mm reinforced second coarse for a wearing surface. Due to dry and windy weather during placing, the second coarse of each structure cracked around the outside of each panel. Both structures were prepared and treated according to the Special Provision in Appendix 5. No cores were removed from either structure.

The first structure that was partially treated with the Epoxy material was Bridge No. 159 on US highway U-75 in Shawnee County. This structure was constructed with a Silica Fume overlay that had a section approximately 15 m in length that cracked due to an unexpected change in weather conditions. The surface was prepared as outlined in the Special Provision in Appendix 5 and material was chosen using ASTM C 881 as a guideline. Due to construction time constraints and a very high volume of traffic on this structure no cores were removed. Several other structures throughout Kansas have also been partially treated. The partial treatment has usually been required due to situations as described previously or has been required due to poor curing or fogging during construction. One structure in Pratt county was treated after cracks formed over the reinforcing steel due to extended set time caused by excess retarder in the concrete mix. Each crack was treated individually on this structure. However, on a structure in Overland Park, Kansas the cracking was extensive enough that the entire bridge subdeck was treated and then overlaid with a silica fume modified concrete overlay.

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APPENDIX I

BRIDGES TREATED WITH TEST MATERIALS

Struct.	Dist.	County	Serial Number	Hwy.	Bridge Type Deck Type	Const./ Mod.
A	1	Marshall	47	K 87	Steel Beam Continuous, Concrete Deck, No Tining, Epoxy Bars	1979
B	2	Ottawa	7	US 81	Steel Beam Composite Continuous, Concrete Deck, Medium Tining, Epoxy Bars	1990
C	2	Saline	39	IH 135	Reinforced Concrete Continuous Box Girder, Concrete Bridge Deck Wearing Surface, Light Tining, Black Bar	1963/ 1979
D	3	Phillips	15	US 36	Reinforced Concrete Continuous, Haunched Slab Deck, No Tining, Black Bars	1963

Struct.	Dist.	County	Serial Number	Hwy.	Bridge Type Deck Type	Const./ Mod.
E	4	Miami	63	US 69	Reinforced Concrete Continuous, Haunched Slab Deck, Concrete Bridge Deck Wearing Surface, Light Tining, Black Bar	1978
F	5	Reno	90	K 61	Prestressed Beam Continuous Composite, Concrete Deck, Heavy Tining, Epoxy Bars	1991
G	5	Kingman	65	US 54	Steel Beam Continuous, Concrete Deck, No Tining, Black Bars	1976
H	6	Ford	57	K 154	Prestressed Beam Continuous Composite, Concrete Bridge Deck Wearing Surface, Light Tining, Epoxy Bar	1980

APPENDIX II

PROPERTIES OF MATERIALS TESTED

EPOXY CONCRETE SEALER

Unmixed Properties

	Part A Epoxy Resin	Part B Curing Compound
Viscosity	0.65 Pa˙s	0.02 Pa˙s
Density	1108.4 kg/m^3	988.6 kg/m^3

Mixed Properties

Viscosity	0.3 - 0.5 Pa˙s
Density	1078.4 kg/m^3
Flash Point	93.3^{0}C
Gel Time	60 min
Tack Free Time	3 hr
Compressive Strength ASTM D 695	35.6 MPa
Tensile Strength ASTM D 638	29.3 MPa
Compressive Modulus	N/A
Flexural Modulus ASTM D 790	1.4 GPa
Elongation ASTM D 638	9.9%
Shear Bond	N/A
Pecent Solids	100

HIGH MOLECULAR WEIGHT METHACRYLATE A
LOW MODULUS 30% ELONGATION

Typical Mixing Proportions for Resin Promoter and Initiator.

Resin	Promoter	Initiator
18.93 liters	0.350 liters	1.050 liters

Mixed Properties

Viscosity	0.01 - 0.025 Pa˙s
Density	994.6 kg/m^3
Flash Point	82.2^{0}C
Gel Time	30 min
Tack Free Time	4 hr
Compressive Strength Cal Trans 551	23.4 MPa
Tensile Strength ASTM D 638	2.8 MPa
Compressive Modulus Cal Trans 551	2.3 GPa
Flexural Modulus	N/A
Elongation ASTM D 638	30%
Shear Bond Cal Trans 551	90 psi

HIGH MOLECULAR WEIGHT METHACRYLATE B
MEDIUM MODULUS 10% ELONGATION

Typical Mixing Proportions for Resin Promoter and Initiator.

Resin	Promoter	Initiator
18.93 liters	0.20 liters	0.60 liters

Mixed Properties

Viscosity	0.07 - 0.15 Pa·s
Density	994.6 kg/m^3
Flash Point	107.2^{0}C
Gel Time	80 min
Tack Free Time	4 hr
Compressive Strength ASTM C 109	44.8 MPa
Tensile Strength ASTM D 638	8.3 MPa
Tensile Modulus ASTM D 638	0.95 GPa
Flexural Modulus	N/A
Elongation ASTM D 638	10%
Shear Bond Cal Trans 551	N/A

HIGH MOLECULAR WEIGHT METHACRYLATE C
HIGH MODULUS 1.9% ELONGATION

Typical Mixing Proportions for Resin.

Part A	Part B
16.38 liters	0.655 liters

Mixed Properties

Viscosity	0.025 Pa·s
Density	1068.9 kg/m^3
Flash Point	104.4^{0}C
Gel Time	20 min
Tack Free Time	6 hr
Compressive Strength ASTM C 109	29.6 MPa
Tensile Strength ASTM D 638	2.8 MPa
Tensile Modulus ASTM D 638	0.28 GPa
Flexural Modulus ASTM D 790	17.2 MPa
Elongation ASTM D 638	1.9%
Shear Bond ASTM C 882	15.9 MPa

APPENDIX III

KANSAS DEPARTMENT OF TRANSPORTATION
LABORATORY METHOD 814
WATER SOLUBLE CHLORIDES

Apparatus

pH Meter - Orion model 810
Electrode - Orion Specific Ion Chloride model 94-17A
Electrode - Reference, double junction Orion model 90-02-00
Beaker - 400 ml Pyrex or Kimax
Flask - Filter 500 ml Pyrex or Kimax
Funnel - Buchner 9 cm Coors 2A
Filter Paper - Whatman No. 1, 9 cm
Pipette - Tippette 5 ml
Pipette - Transfer 20 ml class A
Burrette - 25 ml class A
Balance - Mettler Top Loader model P 1200 with air shield
Magnetic Stirrer - Motor driven stirrer and 50 mm Teflon covered magnet

Reagents

Silver Nitrate (0.1 N) - Dissolve 17.0 g of $AgNO_3$ in water and dilute to one liter. Standardize against 0.1000 N Sodium Chloride Solution.
Sodium Chloride (0.1000 N) - Dissolve 5.8450 g of NaCL in water and dilute to exactly one liter.
Reference Electrode Filling Solutions - Inner filling solution Orion No. 90-00-02; outer filling solution Orion No. 90-00-03(10% KNO_3)
Standardization of Silver Nitrate Solution (0.1 N) - Using a class A transfer pipette, transfer 20 ml of the standard Sodium Chloride solution to a 400 ml beaker. Dilute to 250 ml with water and add 5 ml of HNO_3 with the Tippette pipette. Titrate with Silver Chloride Solution as described under the Procedure. Calculate the normality as follows:

$$\text{Normality} = \frac{20 \times 0.1000}{T}$$

Where T = ml of the Silver Nitrate Solution

Sample

The sample will be collected boring dust with filter. Tare a 400 ml beaker on the balance. Transfer the bulk of the sample to the beaker. Using scissors, carefully cut the filter into small strips and add them to the beaker. Transfer the remainder of the sample to the beaker. Record the weight of the sample and filter to the nearest 0.02 g. Subtract 0.48 g for the filter and record the net weight of the sample.

Add 150 ml of water to the beaker and place the beaker on a hot plate. Allow the sample to boil for 20 minutes. Remove the beaker from the hot plate and allow it to cool enough to handle. Filter on Whatman No. 1 paper using a Buchner funnel, filter flask and vacuum and police the beaker. Wash the filter well with hot water. Quantitatively transfer the filtrate back into the original beaker and dilute to 250 ml. Add 5 ml of HNO_3 with the Tippette pipette and allow the sample to come to room temperature. Titrate with the Silver Nitrate Solution.

Titration

Insert the electrode in the holder and connect the chloride ion electrode to the input jack. Connect the reference electrode to the REF jack. Place a 1-inch Teflon covered magnet in the beaker and center the beaker on the magnetic stirrer. Lower the electrode assembly so that the tips of the electrodes extend about 12 mm below the surface of the solution. Start the magnetic stirrer and adjust the rate of stirring so that the solution is vigorously stirred without entrapping air in the solution. Titrate with the Silver Chloride Solution adding the Solution in small increments. Allow about 30 seconds after each addition before taking a millivolt reading. Record each volume and millivolt reading. Near the end point (about 200 millivolts) add the Silver Nitrate Solution in 0.1 ml increments. Obtain the end point by inspection of the data or by the maximum differential method.

Calculations

$$\text{Chloride, lbs./cu.yd.} = \frac{T \times N \times 137}{S}$$

Where: T = ml of Silver Nitrate Solution

N = Normality of the Silver Nitrate Solution

S = Sample weight

137 = 0.03546 x 0.0022046 x average weight /cu.yd. of concrete in grams (140 lbs./cu.ft. = 1,714,579 gm/cu.yd.).

APPENDIX IV

EQUIPMENT AND SUPPLIES USED FOR THE EXPERIMENTAL FIELD APPLICATION OF THE SEALERS.

High pressure (21 - 28 MPa) washer.

15,000 L water tank or truck.

2 - 19 L buckets for mixing and application.

2 - 3.8 L graduated buckets.

250 ml graduated plastic (polypropylene) beakers.

500 ml graduated plastic (polypropylene) beakers.

Stiff bristle brooms with removable handles.

Rubber squeegees with removable handles.

3/8 inch electric drill.

Mixing paddle.

Sand spreader.

Portable generator.

Rubber gloves.

Rubber boots.

19 L water container.

Dust masks.

Goggles.

Rubber gloves.

Glove guard.

Hand cleaner.

Methyl Ethyl Ketone.

APPENDIX V

MATERIAL PENETRATION SUMMARY

		Core Length (mm)	Crack Length (mm)	Crack Width (mm)	Penetration (mm)	Percent Penetration (crack area)
Bridge A						
EPOXY	1992	97	97	0.60	51	53
	1993	77	77	1.00	0	0
	1994	80	54	0.30	54	100
	1995	89	70	0.41	25	60
	AVE			**0.58**	**33**	**53**
HMWM A	1992	115	115	0.50	115	100
	1993	87	87	0.30	87	100
	1994	80	80	0.30	57	71
	1995	86	76	0.51	3	8
	AVE			**0.40**	**66**	**70**
HMWM B	1992	72	72	0.50	41	57
	1993	87	55	0.08	50	99
	1994	83	83	0.50	45	54
	1995	76	76	0.41	41	54
	AVE			**0.37**	**44**	**66**
Bridge B						
EPOXY	1992	70	70	0.40	70	100
	1993	72	72	0.30	50	69
	1994	80	80	0.50	51	64
	1995	83	83	0.71	32	38
	AVE			**0.48**	**51**	**68**
HMWM A	1992	63	63	0.60	60	95
	1993	55	55	0.40	55	100
	1994	83	83	0.40	51	61
	1995	83	83	0.33	13	15
	AVE			**0.43**	**45**	**68**
HMWM B	1992	72	72	1.30	25	35
	1993	85	85	0.20	40	47
	1994	89	89	0.50	89	100
	1995	76	76	0.38	13	17
	AVE			**0.60**	**42**	**50**

		Core Length (mm)	Crack Length (mm)	Crack Width (mm)	Penetration (mm)	Percent Penetration (crack area)
Bridge C						
EPOXY	1992	70	70	0.80	6	9
	1993	126	72	0.08	72	100
	1994	111	111	0.80	38	34
	1995	114	114	0.71	13	11
	AVE			**0.60**	**32**	**38**
HMWM A	1992	137	63	0.08	6	18
	1993	115	90	0.30	6	13
	1994	89	89	0.20	89	100
	1995	89	89	0.51	3	4
	AVE			**0.27**	**26**	**34**
HMWM B	1992	70	70	0.80	60	86
	1993	105	85	0.60	6	14
	1994	67	67	0.80	16	24
	1995	108	108	0.76	6	6
	AVE			**0.74**	**22**	**32**
Bridge D						
EPOXY	1992	125	125	1.00	114	91
	1993	125	125	0.50	3	2
	1994	76	76	0.20	38	50
	1995	76	76	0.64	6	8
	AVE			**0.58**	**40**	**38**
HMWM A	1992	90	90	1.30	29	32
	1993	72	72	0.80	72	100
	1994	162	64	0.80	51	96
	1995	89	38	0.33	3	16
	AVE			**0.81**	**39**	**61**
HMWM B	1992	80	80	1.00	13	16
	1993	77	35	0.20	50	82
	1994	73	13	0.03	13	100
	1995	114	102	0.64	3	6
	AVE			**0.47**	**20**	**51**

		Core Length (mm)	Crack Length (mm)	Crack Width (mm)	Penetration (mm)	Percent Penetration (crack area)
Bridge E						
EPOXY	1992	122	60	0.08	60	100
	1993	126	72	0.08	12	31
	1994	121	32	0.03	32	100
	1995	140	44	0.20	44	100
	AVE			**0.10**	**37**	**83**
HMWM A	1992	110	110	0.30	26	24
	1993	120	120	0.30	3	3
	1994	137	60	0.03	51	98
	1995	127	44	0.13	6	27
	AVE			**0.19**	**22**	**38**
HMWM B	1992	115	55	0.08	55	100
	1993	120	120	0.40	12	10
	1994	159	29	0.40	29	100
	1995	83	38	0.28	13	56
	AVE			**0.29**	**27**	**66**
Bridge F						
EPOXY	1992	65	65	0.50	26	40
	1993	60	60	0.80	20	33
	1994	89	89	0.40	25	28
	1995	64	64	0.46	13	20
	AVE			**0.54**	**21**	**30**
HMWM A	1992	65	65	0.08	30	46
	1993	60	60	0.40	26	43
	1994	67	67	0.05	67	100
	1995	64	64	0.23	10	15
	AVE			**0.19**	**33**	**51**
HMWM B	1992	75	75	1.00	44	59
	1993	60	60	0.10	20	33
	1994	143	41	0.03	32	95
	1995	67	64	0.46	22	58
	AVE			**0.40**	**30**	**61**

		Core Length (mm)	Crack Length (mm)	Crack Width (mm)	Penetration (mm)	Percent Penetration (crack area)
Bridge G						
EPOXY	1992	45	45	0.50	16	36
	1993	45	45	0.06	0	0
	1994	29	29	0.05	29	100
	1995	127	114	0.03	38	56
	AVE			**0.16**	**21**	**48**
HMWM A	1992	35	35	0.30	23	66
	1993	35	35	0.08	35	100
	1994	152	127	0.30	70	80
	1995	38	38	0.03	13	33
	AVE			**0.18**	**35**	**70**
HMWM B	1992	35	35	0.06	35	100
	1993	35	35	0.10	26	74
	1994	143	118	0.40	64	79
	1995	64	0	0.00	0	
	AVE			**0.14**	**31**	**84**
Bridge H						
EPOXY	1992	50	50	0.20	38	76
	1993	53	53	0.30	20	38
	1994	114	48	0.20	48	100
	1995	83	52	0.001	52	100
	AVE			**0.18**	**40**	**78**
HMWM A	1992	95	95	0.08	95	100
	1993	115	40	0.08	40	100
	1994	92	57	0.03	57	100
	1995	111	16	0.001	16	100
	AVE			**0.05**	**52**	**100**
HMWM B	1992	63	63	0.08	26	41
	1993	96	60	0.10	60	100
	1994	73	54	0.03	38	91
	1995	98	22	0.001	6	49
	AVE			**0.05**	**33**	**70**

APPENDIX VI

BRIDGE CHLORIDE SUMMARY

BRIDGE A

Sealer	Sample Depth, mm	Sample No.	Chloride Content, kg/m^3 1992	1995	Increase 1995/1992
CONTROL	0 - 19	13a	3.78	4.44	
		14a	lip	4.83	
		15a	7.01	6.96	
		16a	7.09	7.06	
		AVERAGE	5.96	5.82	0.98
	19 - 38	13b	2.91	3.46	
		14b	lip	2.61	
		15b	3.84	4.89	
		16b	4.87	3.53	
		AVERAGE	3.87	3.62	0.94
	38 - 57	13c	1.68	2.19	
		14c	lip	1.15	
		15c	2.26	2.80	
		16c	2.73	3.61	
		AVERAGE	2.22	2.44	1.10
EPOXY	0 - 19	1a	2.24	2.58	
		2a	1.54	3.57	
		3a	2.24	2.50	
		4a	lip	3.43	
		AVERAGE	2.00	3.02	1.51
	19 - 38	1b	1.36	2.10	
		2b	0.27	2.90	
		3b	1.18	1.73	
		4b	lip	1.59	
		AVERAGE	0.94	2.08	2.21
	38 - 57	1c	0.40	1.30	
		2c	0.28	1.85	
		3c	0.49	0.97	
		4c	lip	0.80	
		AVERAGE	0.39	1.23	3.14

BRIDGE A cont.

Sealer	Sample Depth, mm	Sample No.	Chloride Content, kg/m^3 1992	1995	Increase 1995/1992
HMWM A	0 - 19	5a	2.69	2.11	
		6a	5.33	6.27	
		7a	4.30	3.00	
		8a	lip	6.64	
		AVERAGE	4.11	4.51	1.10
	19 - 38	5b	1.71	2.04	
		6b	2.38	3.84	
		7b	2.27	1.78	
		8b	2.84	5.96	
		AVERAGE	2.30	3.41	1.48
	38 - 57	5c	0.54	1.20	
		6c	0.67	1.67	
		7c	1.32	1.06	
		8c	2.19	2.80	
		AVERAGE	1.18	1.68	1.42
HMWM B	0 - 19	9a	0.38	0.82	
		10a	2.07	3.56	
		11a	6.01	6.75	
		12a	lip	7.59	
		AVERAGE	2.82	4.68	1.66
	19 - 38	9b	0.24	0.33	
		10b	1.30	2.83	
		11b	2.73	4.61	
		12b	lip	4.61	
		AVERAGE	1.43	3.10	2.17
	38 - 57	9c	0.19	0.47	
		10c	0.90	1.69	
		11c	lip	3.07	
		12c	lip	2.49	
		AVERAGE	0.54	1.93	3.55

BRIDGE B

Sealer	Sample Depth, mm	Sample No.	Chloride Content, kg/m^3 1992	1995	Increase 1995/1992
CONTROL	0 - 19	13a	4.49	4.61	
		14a	4.22	3.44	
		15a	4.31	5.25	
		16a	5.56	6.17	
		AVERAGE	4.65	4.87	1.05
	19 - 38	13b	0.52	1.50	
		14b	2.19	1.05	
		15b	0.70	2.49	
		16b	0.87	2.21	
		AVERAGE	1.07	1.81	1.70
	38 - 57	13c	0.25	0.24	
		14c	1.83	0.18	
		15c	0.23	0.83	
		16c	0.31	0.41	
		AVERAGE	0.65	0.42	0.64
EPOXY	0 - 19	5a	5.14	4.21	
		6a	5.16	4.49	
		7a	4.18	4.40	
		8a	6.37	5.33	
		AVERAGE	5.21	4.61	0.88
	19 - 38	5b	0.68	1.34	
		6b	0.63	1.09	
		7b	1.17	1.46	
		8b	lip	1.55	
		AVERAGE	0.82	1.36	1.65
	38 - 57	5c	0.23	0.15	
		6c	0.24	0.00	
		7c	0.28	0.21	
		8c	0.26	0.21	
		AVERAGE	0.25	0.14	0.56

BRIDGE B cont.

Sealer	Sample Depth, mm	Sample No.	Chloride Content, kg/m^3 1992	1995	Increase 1995/1992
HMWM A	0 - 19	9a	4.39	4.77	
		10a	4.72	5.74	
		11a	5.74	5.30	
		12a	5.78	4.68	
		AVERAGE	5.16	5.12	0.99
	19 - 38	9b	0.74	2.35	
		10b	0.47	1.72	
		11b	0.97	2.14	
		12b	0.81	1.19	
		AVERAGE	0.75	1.85	2.48
	38 - 57	9c	0.24	0.33	
		10c	0.24	0.21	
		11c	0.22	0.29	
		12c	0.21	0.23	
		AVERAGE	0.23	0.27	1.18
HMWM B	0 - 19	1a	3.98	3.30	
		2a	6.74	5.17	
		3a	4.90	4.73	
		4a	6.69	5.81	
		AVERAGE	5.58	4.75	0.85
	19 - 38	1b	0.59	0.89	
		2b	1.73	1.45	
		3b	0.66	1.69	
		4b	1.07	2.18	
		AVERAGE	1.01	1.55	1.53
	38 - 57	1c	0.28	0.00	
		2c	0.24	0.27	
		3c	0.20	0.16	
		4c	0.26	0.21	
		AVERAGE	0.24	0.16	0.87

BRIDGE C

Sealer	Sample Depth, mm	Sample No.	Chloride Content, kg/m^3 1992	1995	Increase 1995/1992
CONTROL	0 - 19	13a	3.31	2.97	
		14a	2.19	1.83	
		15a	0.88	1.08	
		16a	0.83	1.83	
		AVERAGE	1.80	1.93	1.07
	19 - 38	13b	1.34	1.13	
		14b	0.39	0.33	
		15b	0.29	0.17	
		16b	0.27	1.18	
		AVERAGE	0.57	0.70	1.22
	38 - 57	13c	1.10	1.03	
		14c	0.62	0.47	
		15c	0.90	0.15	
		16c	0.25	0.55	
		AVERAGE	0.72	0.55	0.76
EPOXY	0 - 19	1a	0.98	0.76	
		2a	2.81	1.56	
		3a	3.23	2.10	
		4a	1.46	2.12	
		AVERAGE	2.12	1.64	0.77
	19 - 38	1b	0.23	0.67	
		2b	1.89	0.21	
		3b	2.11	2.73	
		4b	1.04	0.55	
		AVERAGE	1.32	1.04	0.79
	38 - 57	1c	0.22	1.27	
		2c	1.61	0.16	
		3c	2.66	2.89	
		4c	1.14	0.19	
		AVERAGE	1.41	1.13	0.80

BRIDGE C cont.

Sealer	Sample Depth, mm	Sample No.	Chloride Content, kg/m^3 1992	1995	Increase 1995/1992
HMWM A	0 - 19	5a	3.45	3.84	
		6a	3.78	5.94	
		7a	2.88	3.30	
		8a	1.29	1.27	
		AVERAGE	2.85	3.59	1.26
	19 - 38	5b	1.55	1.78	
		6b	1.32	4.50	
		7b	1.11	1.56	
		8b	0.37	0.68	
		AVERAGE	1.09	2.13	1.96
	38 - 57	5c	0.43	0.73	
		6c	1.18	3.57	
		7c	1.07	1.25	
		8c	0.20	0.98	
		AVERAGE	0.72	1.63	2.27
HMWM B	0 - 19	9a	3.01	2.49	
		10a	2.76	2.49	
		11a	5.13	3.81	
		12a	0.93	2.57	
		AVERAGE	2.96	2.84	0.96
	19 - 38	9b	2.16	2.39	
		10b	2.45	2.40	
		11b	3.03	2.84	
		12b	0.39	2.40	
		AVERAGE	2.01	2.51	1.25
	38 - 57	9c	2.62	2.36	
		10c	lip	2.53	
		11c	3.07	2.46	
		12c	1.09	2.38	
		AVERAGE	2.26	2.43	1.08

BRIDGE D

Sealer	Sample Depth, mm	Sample No.	Chloride Content, kg/m^3 1992	1995	Increase 1995/1992
CONTROL	0 - 19	1a	1.48	1.39	
		2a	2.69	4.58	
		3a	5.92	5.24	
		4a	5.04	5.86	
		AVERAGE	3.78	4.27	1.13
	19 - 38	1b	0.92	0.80	
		2b	1.30	2.29	
		3b	3.64	2.76	
		4b	3.27	3.99	
		AVERAGE	2.28	2.46	1.08
	38 - 57	1c	0.63	0.50	
		2c	0.68	0.97	
		3c	2.09	1.38	
		4c	2.12	2.31	
		AVERAGE	1.38	1.29	0.94
EPOXY	0 - 19	5a	5.34	4.53	
		6a	4.87	4.63	
		7a	4.52	6.98	
		8a	4.36	5.02	
		AVERAGE	4.78	5.29	1.11
	19 - 38	5b	3.50	3.01	
		6b	3.58	3.25	
		7b	2.74	4.46	
		8b	2.47	2.49	
		AVERAGE	3.07	3.30	1.07
	38 - 57	5c	2.56	2.40	
		6c	2.34	2.50	
		7c	1.83	2.63	
		8c	1.45	1.27	
		AVERAGE	2.04	2.20	1.08

BRIDGE D cont.

Sealer	Sample Depth, mm	Sample No.	Chloride Content, kg/m^3 1992	1995	Increase 1995/1992
HMWM A	0 - 19	13a	2.64	3.16	
		14a	5.15	4.86	
		15a	1.68	1.29	
		16a	3.01	3.82	
		AVERAGE	3.12	3.28	1.05
	19 - 38	13b	2.13	LIP	
		14b	2.86	3.56	
		15b	0.74	0.43	
		16b	1.17	1.60	
		AVERAGE	1.73	1.86	1.08
	38 - 57	13c	1.49	2.43	
		14c	2.26	3.01	
		15c	0.45	0.25	
		16c	0.67	0.77	
		AVERAGE	1.22	1.62	1.33
HMWM B	0 - 19	9a	1.86	1.23	
		10a	3.75	2.24	
		11a	4.99	5.62	
		12a	1.30	2.50	
		AVERAGE	2.98	2.90	0.97
	19 - 38	9b	1.09	0.76	
		10b	1.66	0.75	
		11b	2.21	3.18	
		12b	0.34	0.65	
		AVERAGE	1.32	1.34	1.01
	38 - 57	9c	0.53	0.40	
		10c	0.69	0.28	
		11c	0.97	1.80	
		12c	0.25	0.26	
		AVERAGE	0.61	0.69	1.12

BRIDGE E

Sealer	Sample Depth, mm	Sample No.	Chloride Content, kg/m^3 1992	1995	Increase 1995/1992
CONTROL	0 - 19	13a	0.46	0.46	
		14a	0.32	0.56	
		15a	1.38	0.49	
		16a	0.25	0.90	
		AVERAGE	0.60	0.60	1.00
	19 - 38	13b	0.23	0.23	
		14b	0.21	0.56	
		15b	0.48	0.20	
		16b	0.18	0.28	
		AVERAGE	0.27	0.32	1.16
	38 - 57	13c	0.23	0.23	
		14c	0.22	0.41	
		15c	0.33	0.17	
		16c	0.18	0.23	
		AVERAGE	0.24	0.26	1.09
EPOXY	0 - 19	9a	0.46	1.23	
		10a	1.19	0.34	
		11a	1.31	0.26	
		12a	0.54	1.60	
		AVERAGE	0.87	0.86	0.98
	19 - 38	9b	0.22	0.33	
		10b	0.75	0.20	
		11b	0.53	0.24	
		12b	0.22	0.75	
		AVERAGE	0.43	0.38	0.88
	38 - 57	9c	0.20	0.23	
		10c	0.30	0.20	
		11c	0.22	0.19	
		12c	0.21	0.47	
		AVERAGE	0.23	0.27	1.17

BRIDGE E cont.

Sealer	Sample Depth, mm	Sample No.	Chloride Content, kg/m^3 1992	1995	Increase 1995/1992
HMWM A	0 - 19	1a	1.49	0.60	
		2a	0.76	0.71	
		3a	0.46	0.32	
		4a	0.88	0.77	
		AVERAGE	0.90	0.60	0.67
	19 - 38	1b	0.49	0.23	
		2b	0.24	0.25	
		3b	0.22	0.22	
		4b	0.23	0.26	
		AVERAGE	0.29	0.24	0.82
	38 - 57	1c	0.26	0.18	
		2c	0.21	0.20	
		3c	0.20	0.19	
		4c	0.24	0.20	
		AVERAGE	0.23	0.19	0.84
HMWM B	0 - 19	5a	1.58	0.84	
		6a	0.71	0.57	
		7a	0.31	1.49	
		8a	0.58	1.28	
		AVERAGE	0.79	1.04	1.32
	19 - 38	5b	0.40	0.27	
		6b	0.75	0.23	
		7b	0.20	0.62	
		8b	0.37	0.32	
		AVERAGE	0.43	0.36	0.84
	38 - 57	5c	0.23	0.20	
		6c	0.61	0.23	
		7c	0.25	0.30	
		8c	0.30	0.24	
		AVERAGE	0.35	0.24	0.70

BRIDGE F

Sealer	Sample Depth, mm	Sample No.	Chloride Content, kg/m^3 1992	1995	Increase 1995/1992
CONTROL	0 - 19	1a	2.91	3.91	
		2a	2.30	4.42	
		3a	2.69	4.95	
		4a	2.19	4.82	
		AVERAGE	2.52	4.53	1.79
	19 - 38	1b	4.99	0.47	
		2b	0.38	0.65	
		3b	0.32	1.43	
		4b	0.72	1.76	
		AVERAGE	1.60	1.08	0.67
	38 - 57	1c	0.28	0.18	
		2c	0.22	0.18	
		3c	0.20	0.26	
		4c	0.55	0.80	
		AVERAGE	0.31	0.36	1.13
EPOXY	0 - 19	5a	2.22	2.29	
		6a	2.35	2.83	
		7a	2.54	3.59	
		8a	2.73	3.02	
		AVERAGE	2.46	2.93	1.19
	19 - 38	5b	0.25	0.82	
		6b	0.30	0.53	
		7b	0.31	0.96	
		8b	0.25	0.61	
		AVERAGE	0.28	0.73	2.60
	38 - 57	5c	0.20	0.24	
		6c	0.21	0.48	
		7c	0.20	0.26	
		8c	0.20	0.20	
		AVERAGE	0.20	0.30	1.45

BRIDGE F cont.

Sealer	Sample Depth, mm	Sample No.	Chloride Content, kg/m^3 1992	1995	Increase 1995/1992
HMWM A	0 - 19	9a	3.01	3.63	
		10a	2.97	4.17	
		11a	2.75	4.78	
		12a	3.41	5.09	
		AVERAGE	3.03	4.42	1.46
	19 - 38	9b	0.53	1.57	
		10b	0.52	1.31	
		11b	0.55	1.58	
		12b	0.77	1.88	
		AVERAGE	0.59	1.59	2.67
	38 - 57	9c	0.23	0.67	
		10c	0.22	0.32	
		11c	0.20	0.37	
		12c	0.21	0.52	
		AVERAGE	0.21	0.47	2.19
HMWM B	0 - 19	13a	2.19	3.18	
		14a	2.82	3.49	
		15a	3.53	5.15	
		16a	3.38	4.25	
		AVERAGE	2.98	4.02	1.35
	19 - 38	13b	0.28	0.97	
		14b	0.28	0.82	
		15b	0.36	1.29	
		16b	0.41	1.27	
		AVERAGE	0.33	1.09	3.27
	38 - 57	13c	0.21	0.21	
		14c	0.22	0.23	
		15c	0.20	0.30	
		16c	0.17	0.25	
		AVERAGE	0.20	0.25	1.25

BRIDGE G

Sealer	Sample Depth, mm	Sample No.	Chloride Content, kg/m^3 1992	1995	Increase 1995/1992
CONTROL	0 - 19	13a	0.30	1.26	
		14a	1.10	1.54	
		15a	2.62	4.89	
		16a	3.07	1.59	
		AVERAGE	1.77	2.32	1.31
	19 - 38	13b	0.29	0.36	
		14b	0.65	0.29	
		15b	0.46	0.52	
		16b	1.49	0.23	
		AVERAGE	0.72	0.35	0.49
	38 - 57	13c	0.23	0.17	
		14c	0.60	0.19	
		15c	0.22	0.21	
		16c	0.24	0.18	
		AVERAGE	0.32	0.19	0.58
EPOXY	0 - 19	9a	2.53	1.84	
		10a	1.26	2.59	
		11a	2.56	1.34	
		12a	2.29	2.48	
		AVERAGE	2.16	2.06	0.95
	19 - 38	9b	0.79	0.30	
		10b	0.25	0.26	
		11b	0.59	0.55	
		12b	0.25	0.21	
		AVERAGE	0.47	0.33	0.70
	38 - 57	9c	0.25	0.18	
		10c	0.23	0.19	
		11c	0.33	0.34	
		12c	0.21	0.19	
		AVERAGE	0.26	0.23	0.87

BRIDGE G cont.

Sealer	Sample Depth, mm	Sample No.	Chloride Content, kg/m^3 1992	1995	Increase 1995/1992
HMWM A	0 - 19	5a	0.88	3.18	
		6a	0.95	1.33	
		7a	3.29	1.79	
		8a	1.80	2.51	
		AVERAGE	1.73	2.20	1.27
	19 - 38	5b	1.50	3.67	
		6b	0.29	0.60	
		7b	1.37	1.04	
		8b	0.88	1.52	
		AVERAGE	1.01	1.71	1.69
	38 - 57	5c	1.45	1.99	
		6c	0.24	0.24	
		7c	0.34	0.66	
		8c	0.50	0.77	
		AVERAGE	0.63	0.92	1.44
HMWM B	0 - 19	1a	2.40	2.62	
		2a	1.99	2.20	
		3a	1.78	2.30	
		4a	1.96	2.23	
		AVERAGE	2.03	2.34	1.15
	19 - 38	1b	0.66	0.64	
		2b	1.15	1.07	
		3b	1.48	1.57	
		4b	0.91	1.15	
		AVERAGE	1.05	1.11	1.05
	38 - 57	1c	0.22	0.36	
		2c	1.03	0.74	
		3c	1.07	1.18	
		4c	0.60	0.55	
		AVERAGE	0.73	0.71	0.97

BRIDGE H

Sealer	Sample Depth, mm	Sample No.	Chloride Content, kg/m^3 1992	1995	Increase 1995/1992
CONTROL	0 - 19	13a	1.68	2.65	
		14a	0.36	1.22	
		15a	0.34	2.91	
		16a	0.59	0.54	
		AVERAGE	0.74	1.83	2.46
	19 - 38	13b	2.19	1.79	
		14b	0.22	0.79	
		15b	0.23	0.28	
		16b	0.21	0.27	
		AVERAGE	0.71	0.78	1.10
	38 - 57	13c	2.29	2.53	
		14c	0.25	0.34	
		15c	0.23	1.27	
		16c	0.18	0.22	
		AVERAGE	0.74	1.09	1.48
EPOXY	0 - 19	1a	0.28	0.22	
		2a	0.22	1.53	
		3a	0.26	0.32	
		4a	0.21	1.60	
		AVERAGE	0.24	0.92	3.80
	19 - 38	1b	0.23	0.00	
		2b	0.23	1.53	
		3b	0.23	0.38	
		4b	0.22	0.68	
		AVERAGE	0.23	0.65	3.81
	38 - 57	1c	0.21	0.18	
		2c	0.21	0.57	
		3c	0.22	0.18	
		4c	0.22	0.23	
		AVERAGE	0.21	0.29	1.35

BRIDGE H cont.

Sealer	Sample Depth, mm	Sample No.	Chloride Content, kg/m^3 1992	1995	Increase 1995/1992
HMWM A	0 - 19	9a	0.53	0.29	
		10a	2.02	0.25	
		11a	0.82	0.27	
		12a	0.28	0.50	
		AVERAGE	0.91	0.33	0.36
	19 - 38	9b	0.34	0.16	
		10b	1.76	0.16	
		11b	1.26	0.18	
		12b	0.21	0.15	
		AVERAGE	0.89	0.16	0.18
	38 - 57	9c	0.25	0.19	
		10c	1.33	0.38	
		11c	0.55	0.24	
		12c	0.19	0.22	
		AVERAGE	0.58	0.26	0.44
HMWM B	0 - 19	5a	0.34	0.39	
		6a	0.37	3.80	
		7a	0.38	0.47	
		8a	0.37	1.03	
		AVERAGE	0.36	1.42	3.90
	19 - 38	5b	0.22	0.00	
		6b	0.21	2.21	
		7b	0.22	0.17	
		8b	0.17	0.21	
		AVERAGE	0.20	0.65	4.25
	38 - 57	5c	0.21	0.00	
		6c	0.30	2.15	
		7c	0.18	0.22	
		8c	0.18	0.00	
		AVERAGE	0.22	0.59	3.65

APPENDIX VII

TEST BEAM SUMMARY

Beam No.	Material	Comments
1A	Control	Final Chlorides
1B	HMWM A	Final Chlorides
1C	Epoxy	Final Chlorides
1D	Damaged	Initial Chlorides
1E	HMWM C	Final Chlorides
1F	HMWM B	Penetration
2A	HMWM B	Final Chlorides
2B	HMWM A	Final Chlorides
2C	Epoxy	Penetration
2D	Damaged	Initial Chlorides
2E	HMWM C	Penetration
2F	Damaged	No Testing
3A	HMWM B	Initial and Final Chlorides
3B	HMWM A	Final Chlorides
3C	Epoxy	Initial and Final Chlorides
3D	Control	Initial and Final Chlorides
3E	HMWM C	Initial and Final Chlorides
3F	HMWM A	Initial and Final Chlorides
4A	HMWM B	Final Chlorides
4B	HMWM A	Penetration, Initial Chlorides
4C	Epoxy	Final Chlorides
4D	Control	Final Chlorides
4E	HMWM C	Final Chlorides
4F	Epoxy	Final Chlorides
5A	HMWM B	Final Chlorides
5B	HMWM A	Final Chlorides
5C	Epoxy	Final Chlorides
5D	Control	Initial and Final Chlorides
5E	HMWM C	Final Chlorides
5F	HMWM C	Final Chlorides

APPENDIX VIII

CONCRETE BEAM LABORATORY TESTING SCHEDULE

HMWM BEAMS
1994 Calendar for Testing

Month	Day	Test	HRS WET	HRS AIR	HRS FROZEN
August	1	F/T	7.5	9.0	7.5
August	2	F/T	12.0		12.0
August	3	F/T	12.0		12.0
August	4	F/T	12.0		12.0
August	5	F/T	6.0	12.0	6.0
August	6	NONE		24.0	
August	7	NONE		24.0	
August	8	PONDING	10.0	14.0	
August	9	PONDING	24.0		
August	10	PONDING	24.0		
August	11	PONDING	16.0	8.0	
August	12	P/R	8.0	16.0	
August	13	W/D	24.0		
August	14	W/D	24.0		
August	15	W/D	16.0	8.0	
August	16	W/D	16.0	8.0	
August	17	W/D	16.0	8.0	
August	18	W/D	16.0	8.0	
August	19	W/D	16.0	8.0	
August	20	W/D	24.0		
August	21	W/D	24.0		
August	22	W/D	16.0	8.0	
August	23	W/D	16.0	8.0	
August	24	W/D	16.0	8.0	
August	25	W/D	16.0	8.0	
August	26	W/D	16.0	8.0	
August	27	W/D	24.0		
August	28	W/D	24.0		
August	29	W/D	16.0	8.0	
August	30	W/D	16.0	8.0	
August	31	W/D	16.0	8.0	
September	1	W/D	16.0	8.0	
September	2	W/D	16.0	8.0	
September	3	W/D	24.0		
September	4	W/D	24.0		
September	5	W/D	24.0		

Month	Day	Test	HRS WET	HRS AIR	HRS FROZEN
September	6	W/D	16.0	8.0	
September	7	W/D	16.0	8.0	
September	8	W/D	16.0	8.0	
September	9	W/D	8.0	16.0	
September	10	NONE		24.0	
September	11	NONE		24.0	
September	12	PONDING	12.0	12.0	
September	13	PONDING	24.0		
September	14	PONDING	24.0		
September	15	PONDING	16.0	8.0	
September	16	PONDING	16.0	8.0	
September	17	NONE		24.0	
September	18	NONE		24.0	
September	19	P/R AND F/T	4.0	16.0	4.0
September	20	F/T	12.0		12.0
September	21	F/T	12.0		12.0
September	22	F/T	12.0		12.0
September	23	F/T	8.0	8.0	8.0
September	24	NONE		24.0	
September	25	NONE		24.0	
September	26	PONDING	13.0	11.0	
September	27	PONDING	24.0		
September	28	PONDING	24.0		
September	29	PONDING	24.0	0.0	
September	30	P/R	20.0	4.0	
October	1	W/D	24.0		
October	2	W/D	24.0		
October	3	W/D	16.0	8.0	
October	4	W/D	16.0	8.0	
October	5	W/D	16.0	8.0	
October	6	W/D	16.0	8.0	
October	7	W/D	16.0	8.0	
October	8	W/D	24.0		
October	9	W/D	24.0		
October	10	W/D	16.0	8.0	
October	11	W/D	16.0	8.0	
October	12	W/D	16.0	8.0	
October	13	W/D	16.0	8.0	

Month	Day	Test	HRS WET	HRS AIR	HRS FROZEN
October	14	W/D	16.0	8.0	
October	15	W/D	24.0		
October	16	W/D	24.0		
October	17	W/D	16.0	8.0	
October	18	W/D	16.0	8.0	
October	19	W/D	16.0	8.0	
October	20	W/D	16.0	8.0	
October	21	W/D	16.0	8.0	
October	22	W/D	24.0		
October	23	W/D	24.0		
October	24	W/D	16.0	8.0	
October	25	W/D	16.0	8.0	
October	26	W/D	16.0	8.0	
October	27	W/D	16.0	8.0	
October	28	W/D	8.0	16.0	
October	29	NONE		24.0	
October	30	NONE		24.0	
October	31	PONDING	12.0	12.0	
November	1	PONDING	24.0		
November	2	PONDING	24.0		
November	3	PONDING	16.0	8.0	
November	4	P/R		24.0	
November	5	NONE		24.0	
November	6	NONE		24.0	
November	7	F/T	5.5	13.0	5.5
November	8	F/T	12.0		12.0
November	9	F/T	12.0		12.0
November	10	F/T	12.0		12.0
November	11	F/T	12.0		12.0
November	12	F/T	12.0		12.0
November	13	F/T	12.0		12.0
November	14	PONDING	16.0	3.0	5.0
November	15	PONDING	24.0		
November	16	PONDING	24.0		
November	17	PONDING	24.0	0.0	
November	18	P/R	18.0	6.0	
November	19	W/D	24.0		
November	20	W/D	24.0		

Month	Day	Test	HRS WET	HRS AIR	HRS FROZEN
November	21	W/D	16.0	8.0	
November	22	W/D	16.0	8.0	
November	23	W/D	16.0	8.0	
November	24	W/D	24.0		
November	25	W/D	24.0		
November	26	W/D	24.0		
November	27	W/D	24.0		
November	28	W/D	16.0	8.0	
November	29	W/D	16.0	8.0	
November	30	W/D	16.0	8.0	
December	1	W/D	16.0	8.0	
December	2	W/D	16.0	8.0	
December	3	W/D	24.0		
December	4	W/D	24.0		
December	5	W/D	16.0	8.0	
December	6	W/D	16.0	8.0	
December	7	W/D	16.0	8.0	
December	8	W/D	16.0	8.0	
December	9	W/D	16.0	8.0	
December	10	W/D	24.0		
December	11	W/D	24.0		
December	12	W/D	16.0	8.0	
December	13	W/D	16.0	8.0	
December	14	W/D	16.0	8.0	
December	15	W/D	16.0	8.0	
December	16	W/D	16.0	8.0	
December	17	W/D	24.0		
December	18	W/D	24.0		
December	19	W/D	16.0	8.0	
December	20	W/D	16.0	8.0	
December	21	W/D	16.0	8.0	
December	22	W/D	8.0	16.0	
December	23	NONE		24.0	
December	24	NONE		24.0	
December	25	NONE		24.0	
December	26	NONE		24.0	
December	27	POND	12.0	12.0	
December	28	POND	24.0		

Month	Day	Test	HRS WET	HRS AIR	HRS FROZEN
December	29	POND	24.0		
December	30	POND	24.0		
December	31	POND	24.0		
		TOTAL HOURS	2392.0	1100.0	180.0

3672.0 TOTAL HOURS TEST TIME

117	F/T CYCLES COMPLETED	% WET	65.1
61	W/D CYCLES COMPLETED	% AIR	30.0
6	PONDINGS COMPLETED	% FROZEN	4.9
5	POTENTIAL/RATE TESTINGS COMPLETED		

F/T	Freeze/Thaw Testing
W/D	Wet/Dry Testing
P/R	Polarization Resistance Testing

1995 Calendar for Testing

Month	Day	Test	HRS WET	HRS AIR	HRS FROZEN
January	1	POND	24.0	0.0	0.0
January	2	POND	24.0	0.0	0.0
January	3	P/R	14.0	6.0	4.0
January	4	F/T	12.0		12.0
January	5	F/T	12.0		12.0
January	6	F/T	12.0		12.0
January	7	F/T	12.0		12.0
January	8	F/T	12.0		12.0
January	9	F/T	7.0	10.0	7.0
January	10	PONDING	13.0	11.0	
January	11	PONDING	24.0	0.0	
January	12	PONDING	24.0	0.0	
January	13	PONDING	12.0	12.0	
January	14	PONDING	0.0	24.0	
January	15	PONDING	0.0	24.0	
January	16	PONDING	0.0	24.0	
January	17	P/R	0.0	24.0	
January	18	W/D	8.0	16.0	
January	19	W/D	16.0	8.0	
January	20	W/D	8.0	16.0	
January	21	W/D	0.0	24.0	
January	22	W/D	0.0	24.0	
January	23	W/D	8.0	16.0	
January	24	W/D	16.0	8.0	
January	25	W/D	16.0	8.0	
January	26	W/D	16.0	8.0	
January	27	W/D	8.0	16.0	
January	28	W/D	0.0	24.0	
January	29	W/D	0.0	24.0	
January	30	W/D	8.0	16.0	
January	31	W/D	16.0	8.0	
February	1	W/D	16.0	8.0	
February	2	W/D	16.0	8.0	
February	3	W/D	8.0	16.0	
February	4	W/D	0.0	24.0	
February	5	W/D	0.0	24.0	
February	6	W/D	8.0	16.0	

Month	Day	Test	HRS WET	HRS AIR	HRS FROZEN
February	7	W/D	16.0	8.0	
February	8	W/D	16.0	8.0	
February	9	W/D	16.0	8.0	
February	10	W/D	8.0	16.0	
February	11	W/D	0.0	24.0	
February	12	W/D	0.0	24.0	
February	13	W/D	8.0	16.0	
February	14	W/D	16.0	8.0	
February	15	W/D	16.0	8.0	
February	16	W/D	16.0	8.0	
February	17	W/D	8.0	16.0	
February	18	NONE	0.0	24.0	
February	19	NONE	0.0	24.0	
February	20	PONDING	12.0	12.0	
February	21	PONDING	24.0	0.0	
February	22	PONDING	24.0	0.0	
February	23	PONDING	24.0	0.0	
February	24	PONDING	13.0	11.0	
February	25	NONE	0.0	24.0	
February	26	NONE	0.0	24.0	
February	27	P/R	0.0	24.0	
February	28	NONE	0.0	24.0	
March	1	F/T	6.0	11.0	7.0
March	2	F/T	12.0		12.0
March	3	F/T	12.0		12.0
March	4	F/T	12.0		12.0
March	5	F/T	12.0		12.0
March	6	F/T	17.0		7.0
March	7	PONDING	24.0	0.0	
March	8	PONDING	24.0	0.0	
March	9	PONDING	24.0	0.0	
March	10	PONDING	24.0	0.0	
March	11	PONDING	24.0	0.0	
March	12	PONDING	24.0	0.0	
March	13	P/R	8.0	16.0	
March	14	W/D	16.0	8.0	
March	15	W/D	16.0	8.0	
March	16	W/D	16.0	8.0	

Month	Day	Test	HRS WET	HRS AIR	HRS FROZEN
March	17	W/D	8.0	16.0	
March	18	W/D	0.0	24.0	
March	19	W/D	0.0	24.0	
March	20	W/D	8.0	16.0	
March	21	W/D	16.0	8.0	
March	22	W/D	16.0	8.0	
March	23	W/D	16.0	8.0	
March	24	W/D	8.0	16.0	
March	25	W/D	0.0	24.0	
March	26	W/D	0.0	24.0	
March	27	W/D	8.0	16.0	
March	28	W/D	16.0	8.0	
March	29	W/D	16.0	8.0	
March	30	W/D	16.0	8.0	
March	31	W/D	8.0	16.0	
April	1	W/D	0.0	24.0	
April	2	W/D	0.0	24.0	
April	3	W/D	8.0	16.0	
April	4	W/D	16.0	8.0	
April	5	W/D	16.0	8.0	
April	6	W/D	16.0	8.0	
April	7	W/D	8.0	16.0	
April	8	NONE	0.0	24.0	0.0
April	9	NONE	0.0	24.0	0.0
April	10	PONDING	12.0	12.0	0.0
April	11	PONDING	24.0	0.0	0.0
April	12	PONDING	24.0		0.0
April	13	PONDING	24.0		0.0
April	14	PONDING AND P/R	12.0	12.0	0.0
April	15	NONE	24.0		
April	16	NONE	24.0		
April	17	F/T	8.0	8.0	8.0
April	18	F/T	12.0	0.0	12.0
April	19	F/T	12.0		12.0
April	20	F/T	12.0		12.0
April	21	F/T	6.0	12.0	6.0
April	22	NONE	0.0	24.0	
April	23	NONE	0.0	24.0	

Month	Day	Test	HRS WET	HRS AIR	HRS FROZEN
April	24	PONDING	16.0	8.0	
April	25	PONDING	24.0		
April	26	PONDING	24.0		
April	27	PONDING	24.0		
April	28	PONDING AND P/R	16.0	8.0	
April	29	NONE	0.0	24.0	
April	30	NONE	0.0	24.0	
May	1	W/D	8.0	16.0	
May	2	W/D	16.0	8.0	
May	3	W/D	16.0	8.0	
May	4	W/D	16.0	8.0	
May	5	W/D	8.0	16.0	
May	6	W/D	0.0	24.0	
May	7	W/D	0.0	24.0	
May	8	W/D	8.0	16.0	
May	9	W/D	16.0	8.0	
May	10	W/D	16.0	8.0	
May	11	W/D	16.0	8.0	
May	12	W/D	8.0	16.0	
May	13	W/D	0.0	24.0	
May	14	W/D	0.0	24.0	
May	15	W/D	0.0	24.0	
May	16	W/D	0.0	24.0	
May	17	W/D	0.0	24.0	
May	18	W/D	0.0	24.0	
May	19	W/D	0.0	24.0	
May	20	W/D	0.0	24.0	
May	21	W/D	0.0	24.0	
May	22	W/D	8.0	16.0	
May	23	W/D	16.0	8.0	
May	24	W/D	16.0	8.0	
May	25	W/D	16.0	8.0	
May	26	W/D	8.0	16.0	
May	27	NONE	0.0	24.0	
May	28	NONE	0.0	24.0	
May	29	NONE	0.0	24.0	
May	30	W/D	8.0	16.0	
May	31	W/D	16.0	8.0	

Month	Day	Test	HRS WET	HRS AIR	HRS FROZEN
June	1	W/D	16.0	8.0	
June	2	W/D	8.0	16.0	
June	3	NONE	0.0	24.0	
June	4	NONE	0.0	24.0	
June	5	PONDING	16.0	8.0	0.0
June	6	PONDING	24.0	0.0	0.0
June	7	PONDING	24.0	0.0	0.0
June	8	PONDING	24.0	0.0	0.0
June	9	PONDING AND P/R	8.0	16.0	0.0
		TOTAL HOURS	1662.0	2004.0	150.0

3816.0 TOTAL HOURS TEST TIME

117	F/T CYCLES COMPLETED	% WET	43.6
58	W/D CYCLES COMPLETED	% AIR	52.5
7	PONDINGS COMPLETED	% FROZEN	3.9
8	POTENTIAL/RATE TESTINGS COMPLETED		

F/T	Freeze/Thaw Testing
W/D	Wet/Dry Testing
P/R	Polarization Resistance Testing

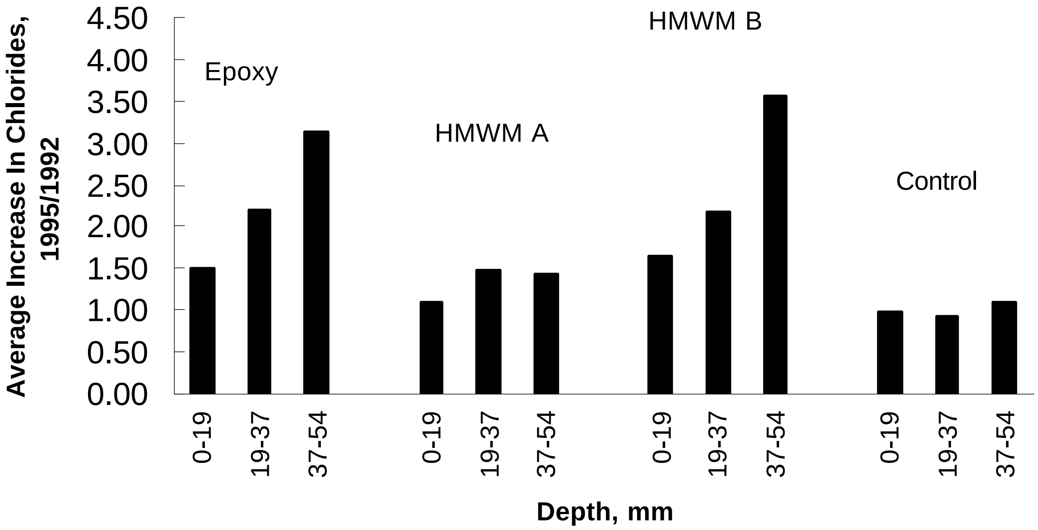
APPENDIX IX

CHLORIDE RATIO GRAPHS

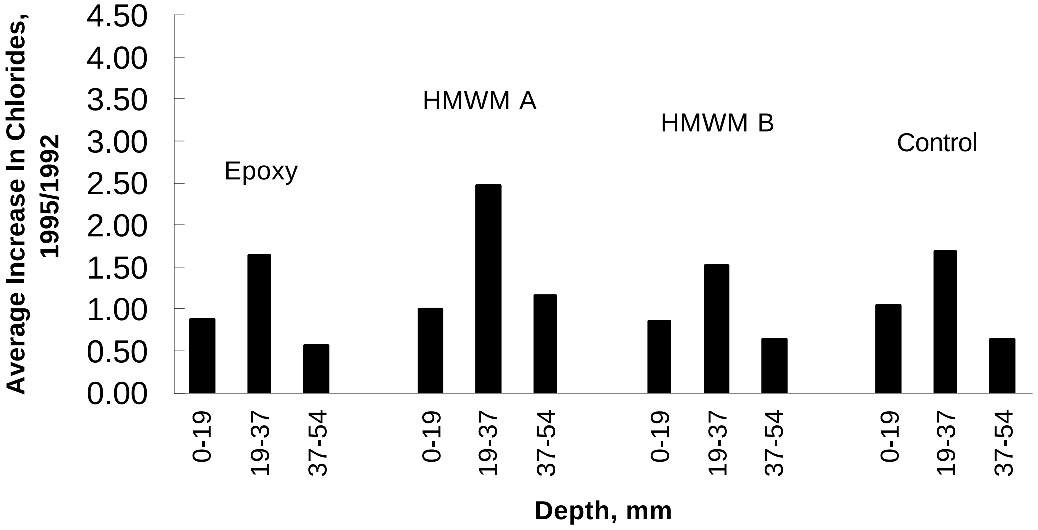
LOCATION AND AGE OF STRUCTURES, 1995.

Bridge	District	Age
A	1	16
B	2	5
C	2	16
D	3	32
E	4	17
F	5	4
G	5	19
H	6	15

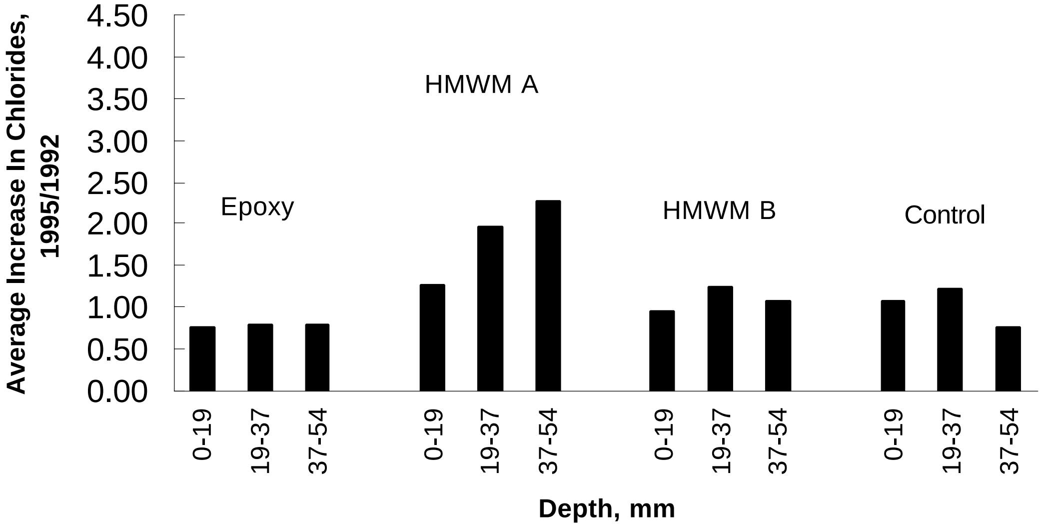
AVERAGE RATIO OF CHLORIDE LEVELS FOR EACH SAMPLE DEPTH, SEALER, AND CONTROL SECTION.



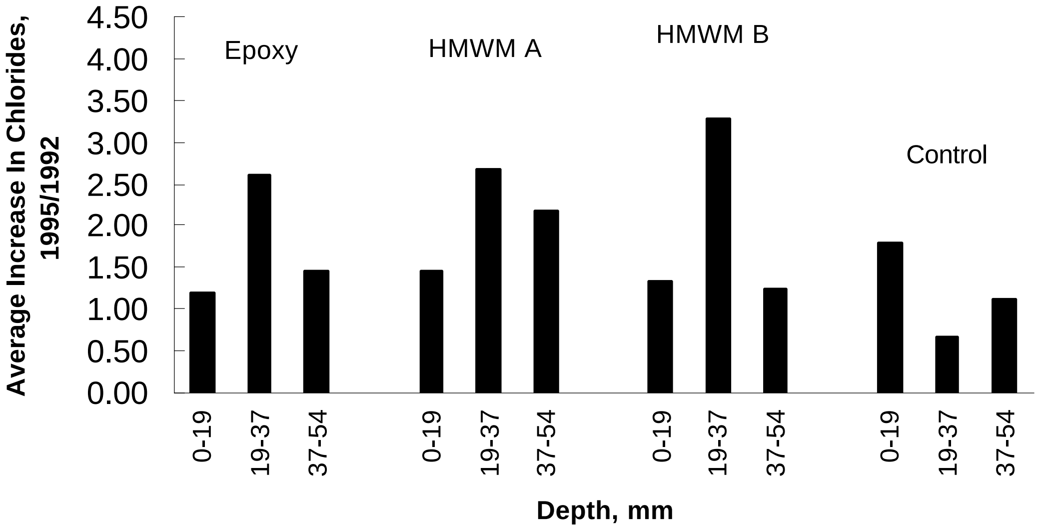
Bridge A, Average Ratio Of Chloride Levels



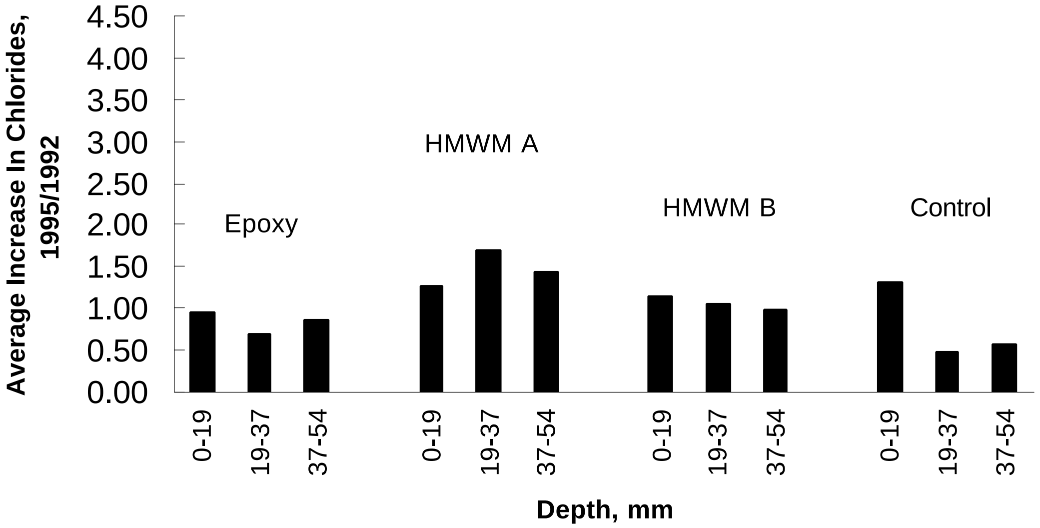
Bridge B, Average Ratio Of Chloride Levels



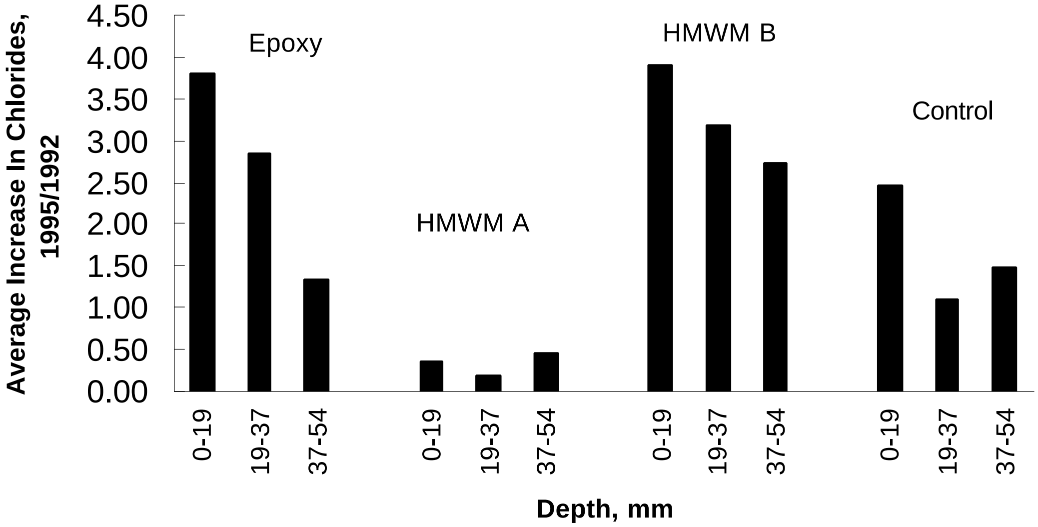
Bridge C, Average Ratio Of Chloride Levels



Bridge F, Average Ratio Of Chloride Levels

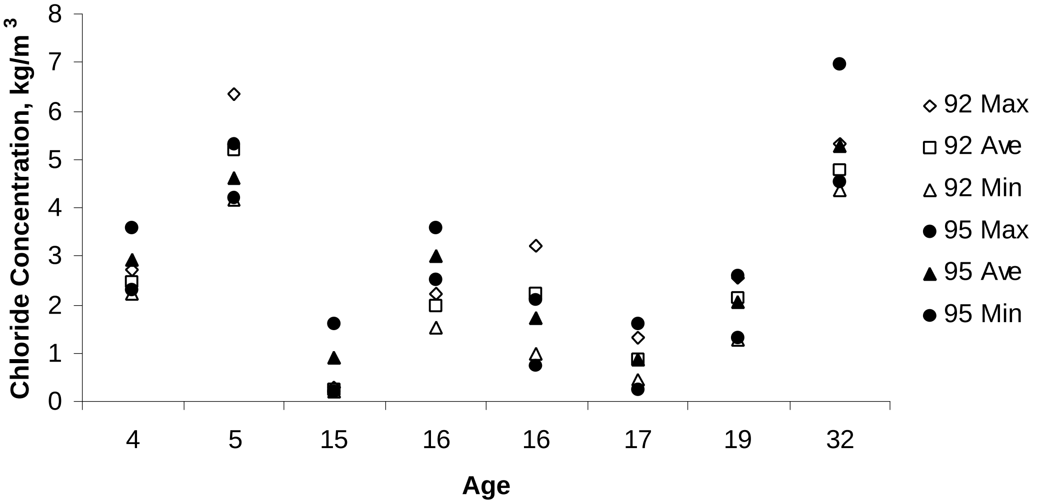


Bridge G, Average Ratio Of Chloride Levels

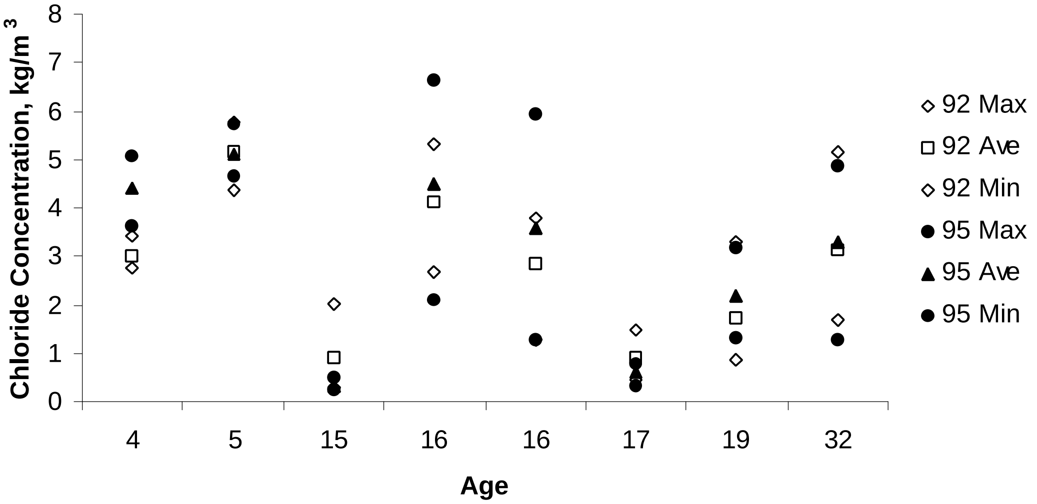


Bridge H, Average Ratio Of Chloride Levels

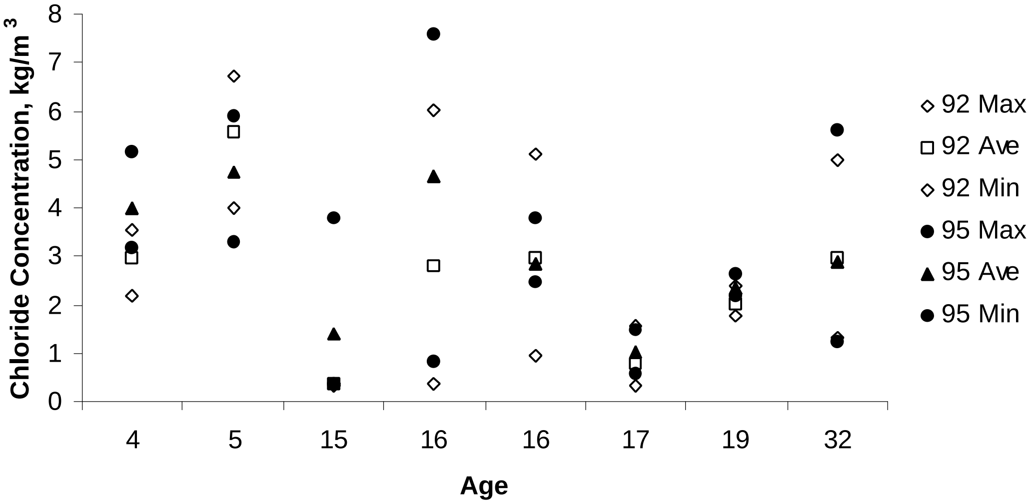
HIGH, LOW, AND AVERAGE CHLORIDE LEVELS FOR EACH SAMPLE DEPTH, SEALER, AND CONTROL SECTION FOR 1992 AND 1995.



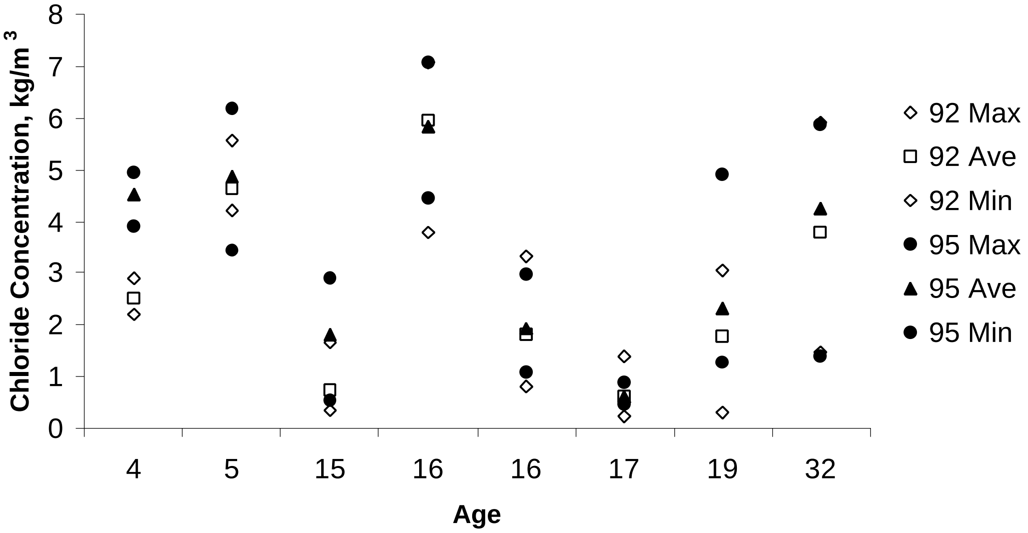
1992 - 1995 Chloride Concentration
0 - 19 mm Depth, EPOXY



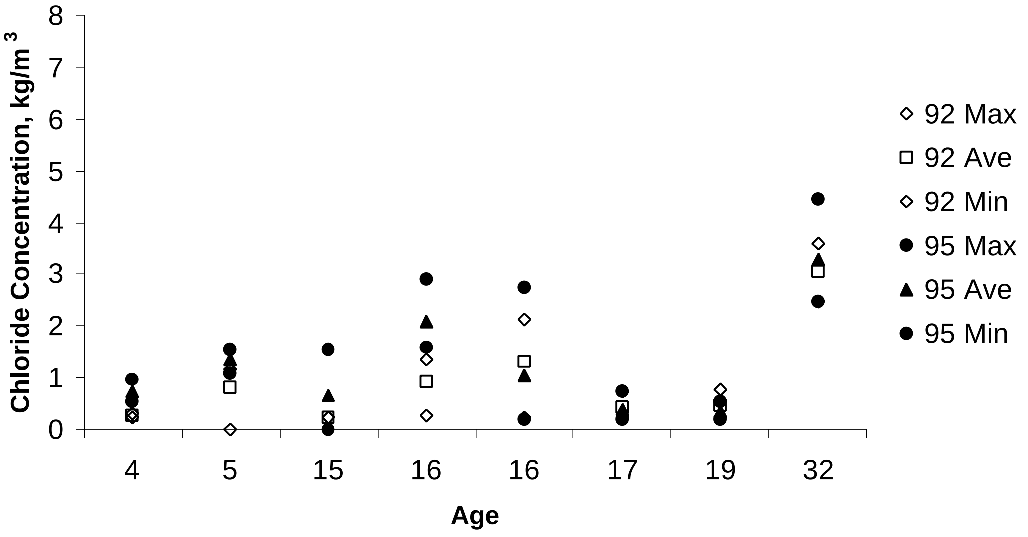
1992 - 1995 Chloride Concentration
0 - 19 mm Depth, HMWM A



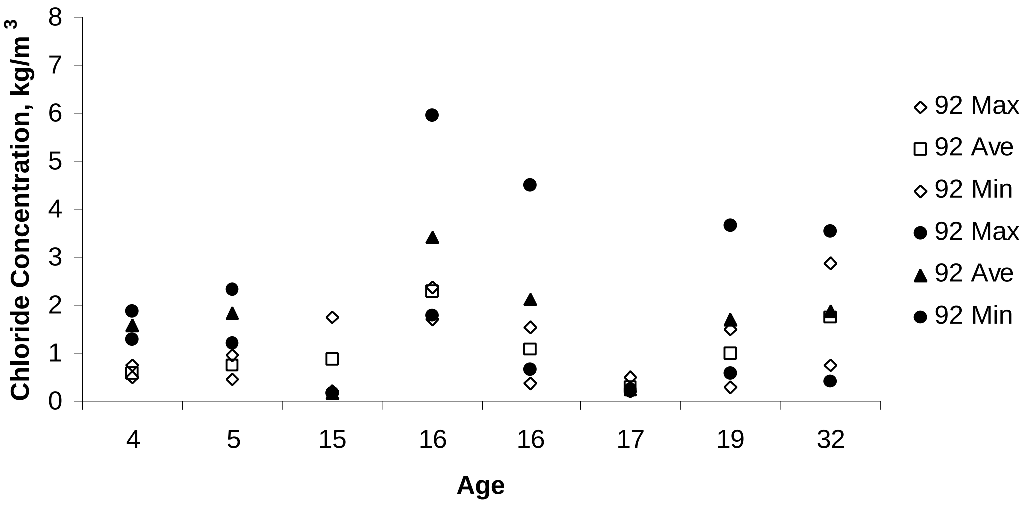
1992 - 1995 Chloride Concentration
0 - 19 mm Depth, HMWM B



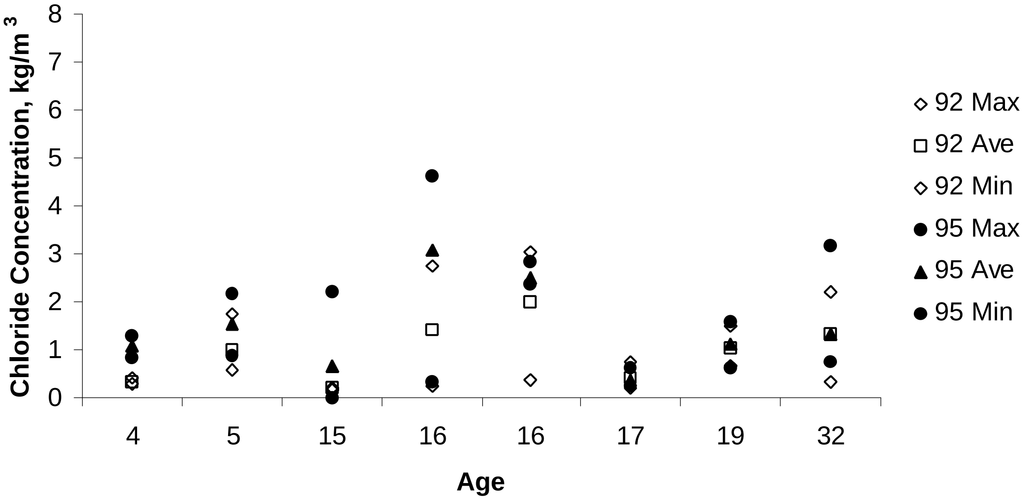
1992 - 1995 Chloride Concentration
0 - 19 mm Depth, Control



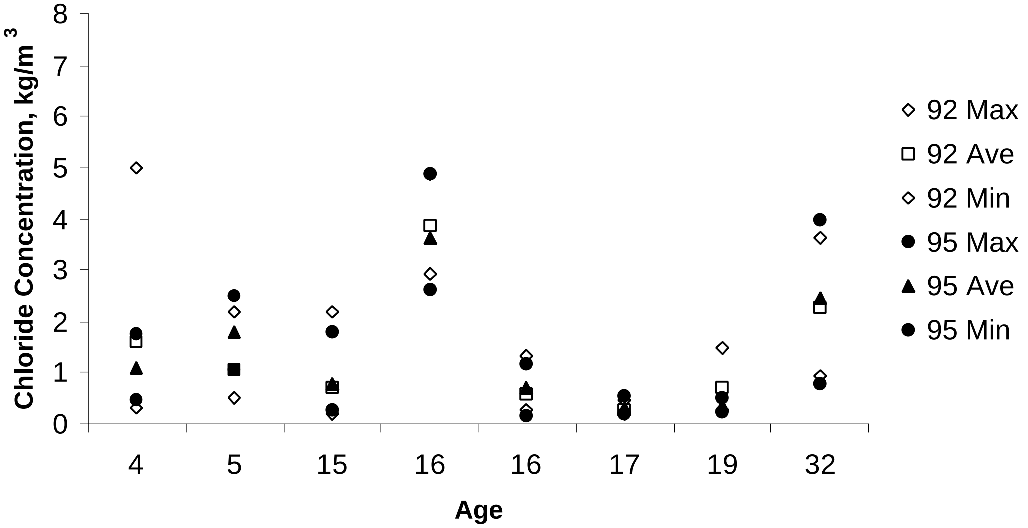
1992 - 1995 Chloride Concentration
19 - 38 mm Depth, EPOXY



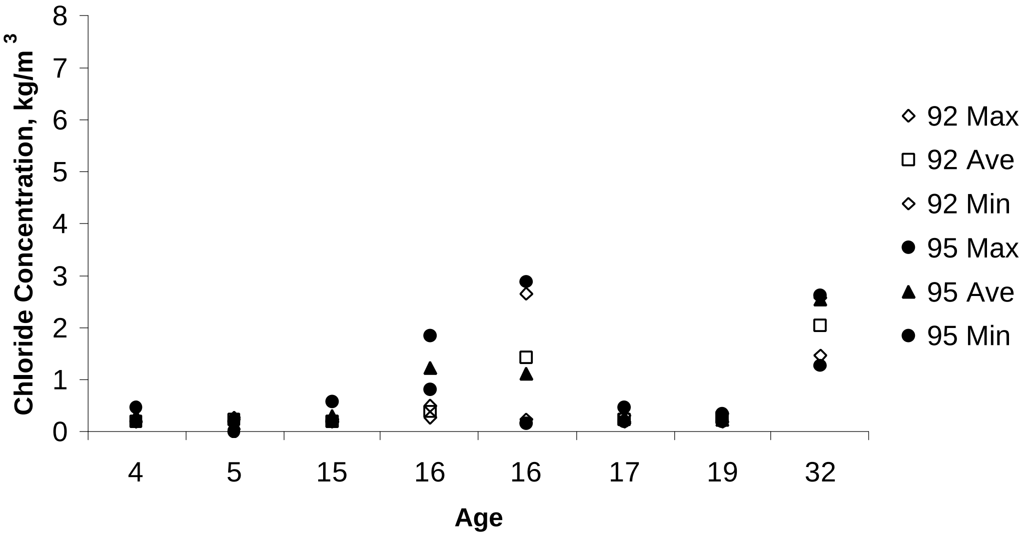
1992 - 1995 Chloride Concentration
19 - 38 mm Depth, HMWM A



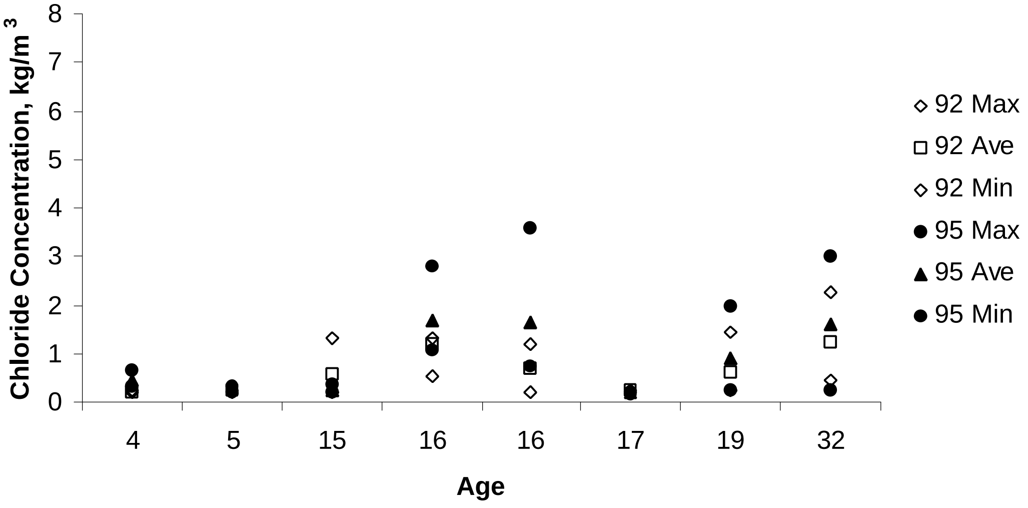
1992 - 1995 Chloride Concentration
19 - 38 mm Depth, HMWM B



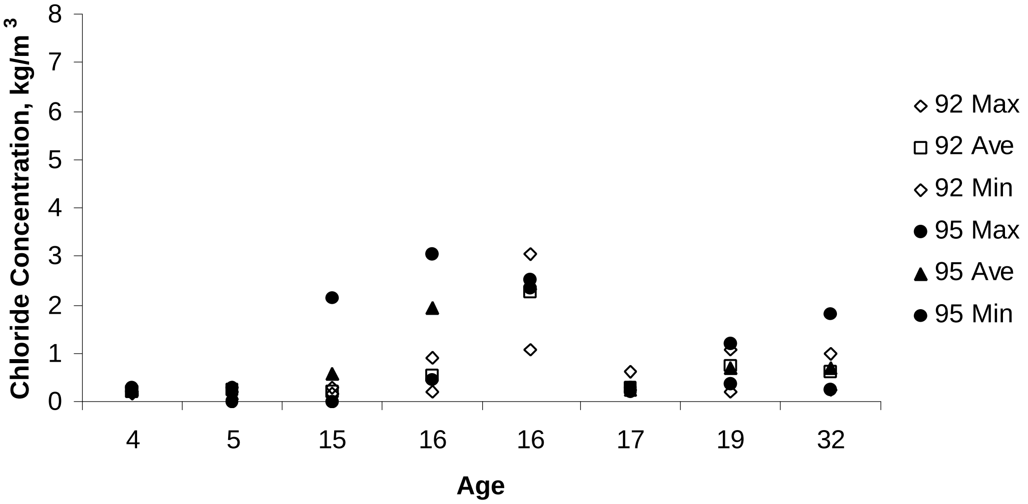
1992 - 1995 Chloride Concentration
19 - 38 mm Depth, Control



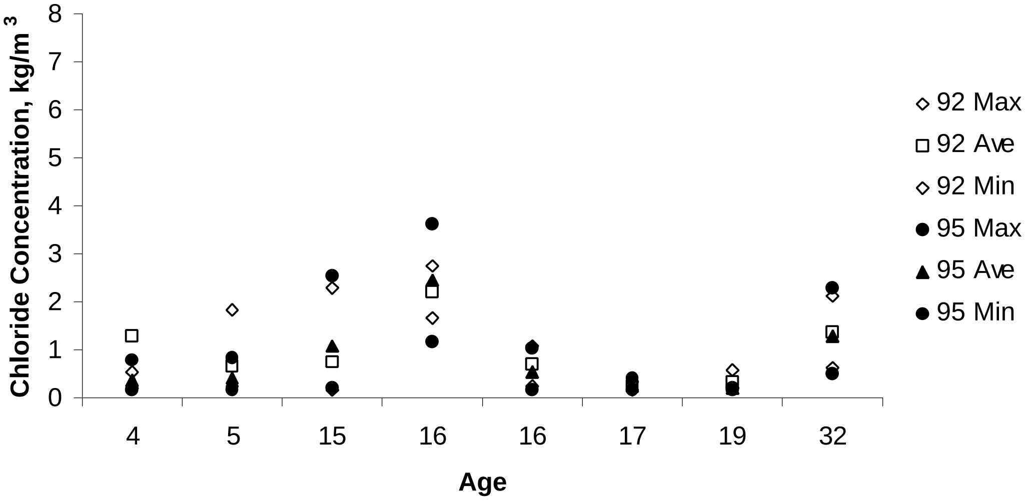
1992 - 1995 Chloride Concentration
38 - 57 mm Depth, EPOXY



1992 - 1995 Chloride Concentration
38 - 57 mm Depth, HMWM A



1992 - 1995 Chloride Concentration
38 - 57 mm Depth, HMWM B



1992 - 1995 Chloride Concentration
38 - 57 mm Depth, Control

APPENDIX X

KANSAS DEPARTMENT OF TRANSPORTATION
SPECIAL PROVISION
TO THE
STANDARD SPECIFICATIONS
EDITION OF 1990

KANSAS DEPARTMENT OF TRANSPORTATION
SPECIAL PROVISION
TO THE
STANDARD SPECIFICATIONS
EDITION OF 1990

NOTE: Whenever this Special Provision conflicts with plans or standard specifications, this Special Provision shall govern.

CONCRETE SEALER (HMWM)

1.0 DESCRIPTION

Furnish and apply a polymer material (monomer) which, when polymerized, can structurally rebond cracks and act as a film forming sealer against the ingress of chloride ions on concrete surfaces.

2.0 DEFINITION OF TERMS

A. Acrylic Resins are any of numerous thermoplastics or thermosetting polymers or copolymers of acrylic acid, methacrylic acid, esters of these acids, or acrylonitrile, used to produce synthetic rubbers and lightweight plastics.

B. Specific Gravity is the ratio of the density (the mass per unit volume) of the material to the density of water at $25^{o}C$.

C. Bulk cure is the time required before the methacrylate monomer has hardened to form a polymer.

D. Flash point is the lowest temperature at which a material will give off explosive or ignitable vapors.

E. High molecular weight methacrylate (HMWM) is a non-fuming monomer with low viscosity, low volatility, low initial shrinkage, and high flash point. Methacrylates are members of the acrylic family of monomers and polymers.

F. Initiators are chemicals that start the polymerization process.

G. Monomers are liquid molecules which are converted into solid polymers by reaction with a catalyst.

H. Promoters are chemicals used with initiators to accelerate the polymerization process.

I. Resin is a natural or synthetic, solid or semi-solid organic material of indefinite and often high molecular weight, having the tendency to flow under stress.

J. Shelf life is the maximum interval during which a material may be stored and remain in a usable and/or safe condition.

K. Surface cure is the amount of time required after placement before the HMWM treated concrete is free from surface liquid or tackiness.

L. Vapor pressure is a component of atmospheric pressure which is caused by the pressure of vapor; expressed in inches, centimeters, or millimeters of height of a column of mercury. Vapor pressure measures the volatility and depletion rate of monomers.

M. Viscosity is the resistance that a gaseous or liquid system offers to flow when it is subjected to shear stress.

N. Working time is the elapsed time after mixing methacrylate monomer with catalyst before solution viscosity begins to increase. Working time usually depends on temperature and the amount of initiators and promoters being used. Working time will also vary with the particular manufacturer's monomer system.

3.0 MATERIALS

A. Monomer Materials

1. High Molecular Weight Methacrylate (HMWM)

Supply a low viscosity, low odor, high molecular weight methacrylate monomer system that conforms to the following:

Viscosity 0.015-0.080 Pa·s	(Brookfield viscometer, Model RVT with a UL adaptor or Model LVF, #1 spindle and UL adaptor at 25°C.) (ASTM D 1824). Note: A higher viscosity material may be approved if the tensile elongation is increased.
Specific Gravity	0.9 - 1.1 (ASTM D 4669)
Flash Point	>93°C (Pensky Martens CC) (ASTM D 93)
Vapor Pressure	<133 pa at 25°C (ASTM D 323)
Shelf Life	Must be 1 year minimum at manufacturer's recommended environmental considerations.
Gel Time	>40 min - 100 g mass (ASTM D 2471)(thin film)
Tensile Elongation	30% minimum (ASTM D 638)
Cure Time	Bulk Cure >40 min but <2-1/2 hrs at deck temperature Surface Cure >40 min but <6 hrs at deck temperature
Bond Strength	>5.17 MPa (ASTM D 882)
Percent Volatiles	30% maximum (ASTM D 2369)

2. Initiators/Promoters

Supply a compatible promoter/initiator system capable of providing a Deck Surface Cure Time of no less than 40 min nor more than 6 hrs at deck temperature during application. The promoter/ initiator system must be formulated to allow for adjustment of the gel time to compensate for the changes in temperature during the treatment application. Provide to the Engineer a table showing correct proportions of initiator and promoter to be added to the monomer to achieve the cure time requirements based on the surface temperature.

3. General

Materials must be shipped within 6 months of manufacture date. Two component materials are acceptable provided all material requirements are met.

B. Aggregate Materials

Supply clean, dry sand, with a maximum moisture content of 1/2 of the percent absorption of the aggregate, that is free of dirt or other organic materials, and conforms to the following gradation:

Sieve Size	%Retained Max
4.75 mm	0-1
2.36 mm	0-10
600 mm	90-95
300 mm	90-100

U.S. Standard Sieve Size

This gradation is intended to allow the use of commercially available blast sands of No. 8/20 or 10/20. The Engineer may approve alternate gradations.

C. Safety Provisions

- Thoroughly train all personnel in the safe handling of all materials in accordance with the manufacturer's recommendations.
- Wear impervious protective gloves and splash-proof goggles during the application of the HMWM to prevent accidental eye or skin contact.
- Handle solvent for cleaning and flushing of equipment, tools, etc., in such a manner as to minimize personal and environmental hazards as approved by the Engineer. Provide an eye wash facility and soap and water wash station for the workers at the job site.
- Advise workmen that the HMWM monomer will soften gum rubber soled shoes and any clothing or leather that becomes saturated with the monomer will harden and become brittle.

D. Storage of Materials

- Store initiators and promoters separately to prevent accidental combination which can result in a violent reaction or explosion.
- Obtain a Material Safety Data Sheet (MSDS) for each material prior to shipment of material with information pertaining to the safe practices for storage, handling and disposal of all materials and to their explosive and combustion characteristics, health hazards and the recommended fire fighting techniques by the manufacturer.
- Post safety and handling information at all storage areas and at the job site.
- Provide a copy of such information to the Engineer.

1. Monomers

Store monomers in shaded areas away from the initiator storage area. Maintain sufficient ventilation in all storage areas to prevent the hazardous buildup of vapor concentration.

2. Initiators

Store initiators in a cool (<30^{o}C) place away from any monomer and promoter storage areas.

3. Promoters

Store promoters in a cool (<30^{o}C) place away from the initiator storage area.

E. Acceptance of Materials

1. High Molecular Weight Methacrylate

- Supply a Type A Certification with each shipment of monomer, initiator, and/or promoter material certifying that the material conforms to the requirements of these specifications.
- Supply a sample of each bulk component with an infrared spectrum in a Lotus compatible or Mattson 1000 FTIR Spectrometer format to the Bureau of Materials and Research prior to any application. All data will be maintained as confidential and used only for QC/QA purposes. The KDOT will perform an additional infrared spectrum on the material submitted for comparative purposes. A sample of each material will be

taken in the field and an infrared spectrum will be performed on the field sample prior to the application.

- No materials older than 6 months will be accepted.

2. Aggregates

Aggregate materials will be accepted on the basis of tests conducted on samples at a point or points designated by the Engineer.

4.0 CONSTRUCTION REQUIREMENTS

A. Surface Preparation

- Remove traces of asphaltic or petroleum products and any concrete curing seals by abrasive blasting prior to the power wash.
- High pressure wash the entire area to be treated.
- Pressure in excess of 13.8 Mpa. is required.
- Allow the deck to dry 48 hours after completion of the high pressure wash.
- Blow all loose materials from visible cracks and the surface of the deck using high pressure clean, filtered air free of water, oil, and other contaminants immediately prior to the application of the sealer.
- Tape off or otherwise protect expansion joints or other surfaces which are not to be coated with the sealer.
- Block all deck drains and/or dam openings in bridge rails to prevent the material from falling to water ways or traffic lanes below.
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B. Application of HMWM

- The manufacturer's representative must be on site during application of the HMWM sealer.
- Following a rain allow the deck to dry a minimum of 48 hours and blow any loose materials from the deck and all open cracks.
- Combine monomers, promoters, and initiators according to manufactures recommendations.
- Apply the treatment in a surface temperature range of 15^o C to 40^o C. If it is desired to work outside of this temperature range, the specific monomer manufacturer should be consulted for technical advice.
- Do not place the material if detrimental weather conditions are possible before complete cure of the sealer is obtained ie. rain or falling temperatures.

- Do not place materials if complete cure is not possible within the specified requirements of traffic control.
- Place the sealer in accordance with the manufacturer's recommendations as approved or directed by the Engineer or manufacturer's representative.
- Manually mix material by hand in suitably sized containers or by machine.
- For three part materials, use a two part monomer system utilizing a promoted monomer for one part and an initiated monomer for the other part for machine application.
- Spray nozzle pressure must be maintained so as not to atomize the sealer if a machine application is employed. Do not use compressed air to produce the spray.
- Mixing must be adequate to achieve a uniform blend of all components.
- For manual application limit the amount of mixed sealer to 20 L.
- Apply the sealer at a rate which will allow for complete sealing of cracks as recommended by the manufacturer's representative.
- Flood the concrete surfaces with the mixed sealer within 5 minutes after complete mixing (sooner if specified by the manufacturer) allowing penetration into the concrete and filling of all cracks.
- Begin redistribution of excess material within 5-10 minutes after application by use of squeegee or brooms. Pond excess material over cracks to facilitate filling of cracks and voids. Continue redistribution until the sealer begins to gel. Do not allow excess sealer to plug the tined surface of the bridge deck. After the cracks are full sweep excess sealer from tining before the material begins to gel.
- Material will be rejected if a significant increase in viscosity or change in gel time becomes evident.
- Fill imperfections or spalls with sealer and commercial quality concrete or sandblast sand, or small gravel and finish to a uniform surface.
- Broadcast sand either by hand or mechanical means on the entire treated area of the concrete surfaces to achieve a uniform coverage of 0.32±.03 kg per square meter. Place the sand as the monomer begins to gel. Placing of the sand previous to the gelling of the monomer may cause settlement excessive coating of the sand and loss of friction characteristics. When the sand cover adheres sufficiently to resist brushing by foot and the surface is tack free, as determined by the Engineer, traffic may be resumed. A slight oil may remain on the surface for several hours.

5.0 METHOD OF MEASUREMENT

Concrete Sealer (HMWM) will be measured by the square meter, to the nearest 0.1 sq m complete in place. Quantity for which payment to be made will be based on field dimensions.

6.0 BASIS OF PAYMENT

Payment for "Concrete Sealer (HMWM)" at the contract unit price will be full compensation for the specified work. The specified work includes furnishing all labor, materials (including treatment material), tools, equipment and incidentals, and performing all work involved in preparing the concrete surfaces and applying the treatment material. Application of the treatment material must be as shown on the plans, as specified in the Special Provision and as directed by the Engineer or the manufacturer's representative.

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