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Condition Evaluation of Concrete Bridges Relative to Reinforcement Corrosion

Volume 4: Deck Membrane Effectiveness and a Method for Evaluating Membrane Integrity

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Abstract

No techniques exist for nondestructively evaluating the integrity of preformed membranes in place. Further, little is known regarding the continued effectiveness of membranes that are damaged during installation or in service. Therefore, an intensive investigation was conducted to evaluate membrane performance and effectiveness. The main objectives of this study were to: develop a nondestructive test method for evaluating the integrity of membranes in place; investigate the effectiveness of membranes, relative to chloride barrier properties, as a function of membrane integrity; and evaluate the factors that may affect membrane performance, including membrane type and age, climate, and deicer application rate.

A nondestructive methodology using ultrasonic pulse velocity was developed to predict the membrane status. The laboratory study and field validation concluded that preformed membrane systems reduce chloride intrusion, when properly installed and overlaid with at least 2.5-in. (6.4-cm) hot-mix asphalt, which should be kept in a good service condition. An average life of 40 years is expected for preformed membrane systems, and an increase of 25 years in bridge deck life is expected.

Executive Summary

Membrane systems have been used extensively in the United States and Europe as a preventive technique to reduce reinforcing steel corrosion in bridge decks. In the United States, 22 states use membranes as a standard bridge deck protective system, and 19 states have used membrane systems in the past. However, the effectiveness and integrity of membrane systems have not been sufficiently investigated. Therefore, an extensive laboratory investigation and field validation were performed to detect defects in membranes nondestructively and evaluate membrane effectiveness as a chloride barrier.

To study the performance of membranes under controlled conditions, 48 simulated bridge deck slabs were built in the laboratory: 36 were 5 ft x 5 ft x 7.63 in. (1.52 m x 1.52 m x 19.4 cm) and 12 were 5 ft x 4 ft x 9.63 in. (1.52 m x 1.22 m x 24.5 cm) with a 2-in. (5.1-cm) deep galvanized steel stay-in-place form (SIP). Four slabs were for control (no overlays), and another four were overlaid with 2.5-in. (6.4-cm) thick hot-mix asphalt overlay (without membranes). The remaining 40 were covered with one of three types of preformed membranes (Bituthene 5000, Royston 10-A, and Protecto Wrap M400) then overlaid with hot-mix asphalt. The membranes were perforated with different sizes and frequencies of round holes to investigate the membrane effectiveness. The slabs were then exposed to 72 applications of 2.3-percent salt ponding over 9 months, simulating deicing salt applications equivalent to the average of two winters in New England states.

Nondestructive testing methods were evaluated, and a methodology was developed using ultrasonic pulse velocity to detect defects in membranes. A statistical model was developed to predict the membrane status from the ultrasonic pulse velocity measurements.

A total of 15 bridge decks, which had preformed membranes installed when new, were evaluated in the states of New Hampshire, Maine, and Vermont. Approximately 45 nondestructive tests were carried out, and an average of 14 ground truth cores were obtained per bridge deck. The ultrasonic nondestructive evaluation method correlated strongly with

the ground truth data. Powdered concrete samples at three depths were taken from the core locations on the bridges and the laboratory slabs to measure the in situ chloride contents.

The performance of membrane is affected by snowfall, salt application, and age. The membrane effectiveness is affected by perforations of 1/4 in. (6 mm) or larger. The protected slabs showed low chloride contents compared to the unprotected ones. Therefore, the membranes reduce the intrusion of chlorides. Considering all the factors, no major difference in the performance between the three types of membranes was observed. Membranes are expected to increase the life of bridge decks by at least 25 yr when installed on newly constructed bridge decks.

1

Introduction

Needs

In the early 1960s, the severity of bridge corrosion problems fostered the development of experimental techniques for both the construction of new bridges and the rehabilitation of existing structures. Among the techniques used for newly constructed bridge decks are: increased cover depth, sealants, coated reinforcing steel, cathodic protection, and membranes. Currently, concrete overlays (latex-modified and low-slump dense concrete) and membranes are the most widely used protection systems. Membranes have been used both on existing bridge decks and on newly constructed bridge decks.

Membranes have been used extensively for many years in parts of North America, especially the New England states, and in Europe, to prevent bridge deck deterioration. Use of membranes increased dramatically in the United States with the advent of a bridge deck protection policy in 1972 (*1*) requiring that a deck protection system be applied to all federal-aid structures. The installation of a membrane system was one of the most convenient methods of complying with Federal Highway Administration (FHWA) policy.

The rapid expansion of the membranes market led to the introduction of numerous products, and this in turn caused difficulties for highway agencies who had to deal with unidentified performance characteristics. Therefore, highly variable performances were experienced in the field due to the number of different membrane materials and their diverse origins.

Membranes may be categorized as preformed, thermoplastic, polyurethane, epoxy, and tar emulsion. The standard preformed, thermoplastic, and polyurethane systems have demonstrated adequate field chloride protection performance (*2*). The standard preformed systems are the most widely used and presently have a field performance record of 16 yr (*3*). Thus, the effective service life of these membranes may be significantly greater than the service life of the bituminous pavement on membrane-treated bridge decks. If the membrane is still providing a desired level of protection at the time the bituminous surface is to be replaced, the most cost-effective maintenance procedure may be to restore the riding quality by removing and replacing only the upper 75 percent of the bituminous surface. Thus, a cost-effective technique to evaluate the present condition and predict the future integrity and effectiveness of membranes is needed.

Objectives

The objectives of this research are to develop a method to evaluate the integrity of membranes, to determine the chloride barrier effectiveness of membranes as a function of membrane integrity, and to identify the factors that affect membrane performance. Integrity and effectiveness are interrelated membrane properties. If the membrane remains unbreached, there is a high probability that it will be an effective chloride barrier. The degree of bonding to the bridge deck and to the hot-mix asphalt layer is also an important factor. Even if the integrity of the membrane is breached, the membrane may still effectively extend the corrosion initiation time. The effectiveness of a membrane is believed to be dependent on the size and frequency of the perforations per unit area.

Research Scope

The approach used in this project to accomplish the research objectives was initiated by an extensive literature review of the nondestructive testing techniques used to detect defects in pavement structure systems and of the performance of different types of membranes. To evaluate the integrity and effectiveness of preformed membrane systems, the three preformed membrane types mostly used in the United States were chosen to study their performance under controlled conditions. A total of 48 slabs were built; 40 of them were installed with membranes and then overlaid with hot-mix asphalt. The membranes were installed with different size and frequency of perforations per unit area, except for four membranes that were installed intact. The other eight were control slabs--four that were overlaid with only hot-mix asphalt and four that were bare concrete. The slabs were exposed to a total of 72 salt applications (2.3-percent Cl^-) to simulate average salt application of two winters in the

New England states. To monitor the corrosion progress during that period, the potential drops through a 10-ohm resistor connected between the upper and lower rebar mats were recorded monthly, as were the rebar mats' temperatures.

An evaluation of different nondestructive methods was conducted. The feasibility of using infrared technology, pulsed radar, and ultrasonic pulse velocity was investigated. The ultrasonic pulse velocity system showed repeatable and consistent results on small-scale specimens and thus was evaluated further and field-validated on different bridge deck slabs.

The membrane status was investigated after 72 salt applications, using the ultrasonic pulse velocity system, half-cell potential measurements, and chloride content measurements. The chloride content measurements were taken at three different depths from the surface of the concrete: 0 to ½, ½ to 1, and 1 to 1½ in. (0 to 13, 13 to 25, and 25 to 38 mm). The chloride contents were measured according to the method developed in Volume 6 of this report series.

To assist in the evaluation of the field performance of preformed membranes, a questionnaire was sent to 22 states requesting the identification of the most critical factors related to performance of membranes relative to the environmental, construction, and service areas. A statistically based sampling plan was developed from the questionnaire responses. The field validation was performed in three states: New Hampshire, Maine, and Vermont. Five different bridges were evaluated in each of the above three states. An average of 45 nondestructive tests using the ultrasonic pulse velocity system and an average of 13 ground truth measurements were performed on each field validation bridge deck. The ground truth evaluations included cores and chloride content measurements at three depths from the concrete surface identical to the laboratory chloride measurements.

Statistical analyses were performed on the laboratory and field data in order to develop regression models to interpret the membrane status from the ultrasonic pulse velocity and the ground truth data. The membrane effectiveness was also verified by the chloride content measurements using a statistical approach.

2

Background

Membranes

The rapid expansion of the use of membranes as a preventive technique to stop chlorides from reaching the reinforcing steel in concrete bridge decks has led to the introduction of numerous membrane products. Since performance criteria are not identified, membrane selection is a difficult task. Because of the different materials and quality that have been used in membranes, the field experience has been highly variable. A literature review was performed on the properties and performances of different membrane types.

Membrane Systems and Their Installation

The different types of membranes can be classified into two main categories: sheet systems and liquid systems (*4*). Sheet systems include the various preformed factory-manufactured rolls that are bonded to bridge decks to form a continuous membrane. These systems can be categorized into bituminized fabrics and mineral-dressed bitumen protective sheets. The bituminised fabric sheets consist of a central core of absorbent material impregnated and coated with bitumen. Core materials are either polyester fleece, glass cloth, or woven polypropylene. Polymeric sheets are extruded blends of various base polymers in which other polymers, binders, plasticisers, and inert fillers are included. Elastomer-based sheets are either vulcanized butyl or polyisoprene rubber. The butyl type is laminated with a bitumen-saturated felt on the underside. Bituminous laminated boards are a preformed lamination of bitumen-saturated felt sandwiching finely crushed aggregates. Some of the

sheet membranes have a bonded protective sheet, which is similar to bituminized fabric membranes except that it has minimal waterproofing properties.

Liquid systems consist of one or two components of latex or chemical curing solutions that are applied to the concrete surface. Bitumen-based liquids are various bitumen solutions blended in hydrocarbon solvents, two-part polymer-modified compositions, or refined natural or elastomer-modified mastic asphalts. The resin-based membranes are one-or two-part moisture curing or two-part chemical curing based on urethane, epoxy, or acrylic resins. Urethane-based systems are all elastomer with either carborundum or coal tar (pitch urethanes). Other polyurethane systems, known as pitch epoxies, are modified with coal tar. The acrylic systems are based on polymethylmethacrylate resin. Some of the liquid systems are mineral filled.

Few problem areas were identified in membrane construction. Certain precautions need to be taken at the design stage to prevent membrane leakage or over-saturation at the membrane level. Properly designed sealing details at changes in sections, near the curbs, and at expansion joints are essential. The following three main precautions are necessary prior to construction.

Surface Texture

A stationary waterproofing system can be achieved if the concrete surface has an acceptable texture. Plain-textured and ridged surfaces are usually suitable for interlayer membranes. The texture for mastic asphalt application needs to be tamped so that the ridges do not exceed 5-mm in height in order to provide the needed mechanical interlock. However, a smooth finish is essential when epoxy and polyurethane resins are used to develop a uniform thickness of membrane. Also, the membrane should be applied to dry and clean concrete. Therefore, the concrete should be free from any resinous compound that might have been used for concrete curing.

Blistering

One of the major problems affecting the membrane systems is blistering. When the deck surface is heated by solar radiation, the water in the concrete is evaporated beneath the waterproofing membrane, which forms blisters in the membrane. This may also be caused by the evaporation of solvents left in the bituminous priming coat, which is usually applied to the concrete before the application of a bituminous adhesive. To overcome this problem, it

is advised to keep the surface completely dry and to have the primers cured completely before carrying out any subsequent operation. Also, the prefabricated sheets need to be secured sufficiently to resist the vapor pressure. Sometimes, horizontal ventilation is provided by the inclusion of a perforated dry layer between the primed concrete deck and bituminous adhesive. However, it has the disadvantage that moisture may have access to cracks, porous areas in the concrete, or through punctures in the membrane system. Also, the partial debonding of the membrane, which is sometimes used to provide venting, may result in a higher risk of movement in the bituminous surface used due to the dynamic surface loading. However, use of ventilating pipes is usually recommended. The application of a protective layer or adequate overlaid hot-mix asphalt thickness in the shortest possible time after the membrane is placed helps to prevent blistering.

Protective Layers

Some of the waterproofing membranes, especially those less than 1/4 in. (6 mm) thick, are often liable to be damaged by construction activities. To prevent damage to the membrane sheets during the laying out of hot-mix asphalt, a protective layer is required. Usually, a protective layer of 3/4 in. (20 mm) of hot sand asphalt is laid, using a rubber-tired bituminous paver to keep the braking and skewing to a minimum. However, the recent development of the heavy-duty membranes, which do not require this extra protective layer, provides savings in labor and better continuity in the construction procedure.

Factors Affecting Membrane Performance

Many factors adversely affect membrane performance as a chloride barrier. Some of these important factors are discussed below.

Ambient Temperature During Application

At low temperatures, usually below 41°F (5°C), some of the materials used in manufacturing membranes may become stiff and brittle. This stiffness is affected by the composition, type of laminated construction, and the membrane thickness. The materials that usually experience an increase of stiffness associated with lower temperatures are unrefined bitumen, oxidized and aged asphalt, mastic asphalt, epoxy resin, and ethylene-based polymers. The stiffness of these materials increases with increasing molecular weight and/or asphaltic content. Elastomeric polyurethane-based or elastomer-modified membranes and thin polymer

sheets tend to remain flexible. Also, at high temperatures, above 77°F (25°C), bitumen in unreinforced membranes flows and results in unstable systems.

Membrane Installation

Membranes placed on bridge decks prior to placing the hot-mix asphalt overlay are exposed to damage from the presence of loose aggregates or from the accidental dropping of construction tools (*5*). A chisel test was performed by Price (*5*) on many types of membranes using a drop height of 2 to 40 in. (50 to 1000 mm) and a 3/4-in. (20-mm) wide chisel head with a 90° tip angle that weighed 2.2 lb (1.0 kg). It was shown that chisels penetrated all membranes less than 0.06 in. (1.5 mm) thick. Membranes are also damaged by fuel spills and other solvent materials used on construction sites.

Wearing Surface Application

The laying of a hot-mix asphalt overlay over membranes subjects the membrane sheet to a range of high temperatures that is dependent on the asphalt laydown and compaction temperatures. The softening of asphalt cement normally takes place between 167 and 257°F (75 and 125°C) and flow occurs between 257 and 302°F (125 and 150°C). Therefore, bitumen-based membranes begin to deteriorate when the asphalt starts to soften and flow; membranes start to move, and aggregates are pressed into them. This action is more obvious in laminated membranes. Liquid membranes, such as mastic asphalt, normally soften at 122°F (50°C). As the material flows, thickness of membrane varies, but a satisfactorily bonded membrane increases the resistance of aggregate penetration. It was believed that flowing bitumen might fill the ruptures that could occur in membranes. However, if these holes are caused by aggregates (which usually stay in the same spot), the integrity of the membranes is already reduced. The tensile strength and elongation of many membranes cannot be assessed because the hot-mix asphalt damages them (*5*). The tensile strength and elongation are different for different membranes at subzero temperatures. Ethylene propylene, chlorosulphonated polyethylene, and ethylene vinyl acetate polymer sheets have a small elongation at fracture.

Water Absorption

Because hot-mix asphalt is more permeable than the membrane, there is a risk that water will accumulate at the overlay/membrane interface. In addition to the high probability of the

membrane debonding from the wearing surface under this condition, a potential danger is membrane damage during freezing periods. Although mastic asphalt and air-blown bitumens have a low water absorption, these materials are liable to incur pin holes and to embrittle in the long term, which reduces their effectiveness as waterproofing systems. Glass fibers or felt, used as outer layers on membranes, some thin coal tar-modified urethanes, and liquid asphalt coatings have a high water absorption. Water enters these systems from unsealed cut ends or from punctured surfaces. Usually, the bitumen-based membranes have less absorption, although they tend to have an initially high absorption potential during the flow and cooling stages.

Water Transmission

Water transmission is generally related to poor bonding, especially at the joints of membranes and/or pin or blow holes in the systems. Poorer bonding is usually found in latex and air-blown bitumen systems. Although the practice of treating bridge decks with silane prior to membrane installation is increasing, the advantage of silane in this case is minimal since the membrane should act as the water barrier. If water-bearing deicing salts reach the concrete bridge deck, the potential for the corrosion of the embedded steel reinforcement increases. The potential for chloride attack is the greatest when the membrane debonds from the concrete at the lap joints. In general, the factors that affect the chloride penetration of the membrane and the permeability of the membrane are: the degree of bonding of the membrane to the concrete; the quality of lap joints; the extent of membrane damage during construction; water absorption of the membrane; and the thickness of the membrane.

Field Performance of Bridge Deck Membrane Systems

For the past 2 decades, waterproofing membrane systems have been extensively used on concrete bridge decks as a chloride barrier. In a recent study by Chamberlin (*6*), 29 of 52 highway agencies in the United States and Canada who responded to a questionnaire indicated that they use membrane systems as a standard bridge deck protection system. Two states indicated that they used membranes only as an experimental protection system. Twenty others have used membranes in the past but no longer consider them as standard or experimental. Only one state highway agency indicated that they have never used membrane systems.

An extensive evaluation of bridge deck membrane systems was conducted in the state of Vermont (*7,8*). The study evaluated 33 membrane systems on 59 bridge decks over a period of 11 yr (1971-1982). The study included 15 preformed systems, 7 epoxies, 5 thermoplastic materials, 4 polyurethanes, and 2 tar emulsion systems. The annual average daily traffic (AADT) on the bridges ranged from 370 to 1990 vehicles. The average equivalent axle load (EAL) was 3 to 18 kips. The average number of freeze-thaw cycles was between 80 to 115 cycles per year. The salt applications during the investigation period averaged 3 lb/ft^2 of deck surface, which is typical of 11 winters of deicing chemicals (*3*). Both electrical resistivity tests and Cl^- content determination of concrete samples were conducted. Concrete samples were obtained at critical points: at the curb, wheel path area, and a spot where a satisfactory performance of the membrane is expected. The concrete samples were obtained at a depth of 0 to 1 in. (0 to 25 mm) and 1 to 2 in. (25 to 51 mm). Membrane performance was evaluated on the basis of the degree of chloride contamination. The concrete was considered contaminated if the chloride content was 50 ppm more than the chloride level at construction. The study concluded that preformed membrane systems performed the best. Only 7 percent of the samples were chloride contaminated at the 1- to 2-in. (25- to 51-mm) depth. The study showed that membranes performed well up to the last reported evaluation year, 1989 (*3*) and expressed an opinion that preformed membrane systems would provide protection from chloride contamination for 50 yr or longer. Study results also suggested placing protection boards during construction, preventing the opening of bridges to construction traffic traveling on the membrane and the first course pavement, and using a minimum 2-in. (51-mm) thick overlay as a wearing surface.

One of the earliest experimental users of membrane systems to combat rebar corrosion was the state of Kansas. Membranes were used as a rehabilitation method on chloride-contaminated bridge decks. From 1967 through 1974, almost 12,000 yd^2 (10,000 m^2) of membrane were placed on bridge decks from 6 to 50 yr old.

A long-term investigation on membrane performance in Kansas was conducted over a period of 16 yr (*9*). The bridges were located in areas that experience about 60 freeze-thaw cycles annually, and the ambient temperature varies from -40°F to 120°F (-40 to 49°C). The bridges were exposed to 20 salt applications per year at a rate of 1300 lb (590 kg) per two-lane mile. Nine bridges were investigated with five different types of membranes. They ranged in age from 12 to 15 yr old at the time the membranes were installed. The membrane systems included in the investigation were one preformed, two liquid/preformed, and two liquid. The preformed membrane was polypropylene with coal tar placed over a primer. The liquid/preformed membrane was a nonwoven polypropylene fabric with cationic emulsified asphalt. The first liquid/preformed system had chert aggregate rolled into RS-2K emulsion for the wearing surface, and, for the other, the fabric was placed over an AC-5 thin

coat and covered with hot-mix asphalt. Both liquid systems were cold-applied, modified coal-tar and elastomeric polyurethane, one with a 55-lb (25-kg) grade asphalt-impregnated roofing sheet over it, and the other with catalyst (curing agent) added before application and covered with a Number 40 asphalt roofing sheet. Both liquid systems were overlaid with hot-mix asphalt.

Most of the membranes installed in Kansas during the period of investigation have performed well, including a membrane installed on a 56-yr-old deck. As previously noted, all the membranes included in the investigation were installed on existing chloride-contaminated bridge decks. The study indicated that, since in Kansas the evaporation rate is higher than the precipitation rate, the membranes seem to perform well on salt-contaminated decks. Apparently, the membranes retarded the downward movement of water and chlorides, which, in addition to the high evaporation, reduced the amount of imbibed water and kept chloride and water near the surface. The study also concluded that care in installation and proper timing of all procedures proved to be necessary during construction.

Another case study was reported by LaCroix (*10*) in which he studied the performance of membrane systems in 20 interstate bridge decks over a period of 7 yr. The study concluded that membrane systems would have to perform well for at least 4.5 yr in order to be an economical treatment. Also, the study suggested that repairing chloride-contaminated bridge decks by partial patching and installing membrane systems can be a cost-effective alternative to total deck replacement.

In summary, the above three cases supported the use of membrane systems as chloride barriers and recommended their use if properly installed.

Potential Nondestructive Test Methods for Membrane Evaluation

Nondestructive testing of pavement systems can be divided into four categories: surface methods, vibration methods, radioactive methods, and electrical methods. These four categories can be further divided into the methods shown in Figure 2-1 (*11*). However, not all of the techniques presented in Figure 2-1 can be applied in the field, and there are some other technologies that may be applied to pavement systems like laser technology, infrared thermography, and image processing (*12*). The technologies that can be used to detect defects in pavement systems are discussed below, along with the feasibility of using these techniques for detecting defects in membranes.

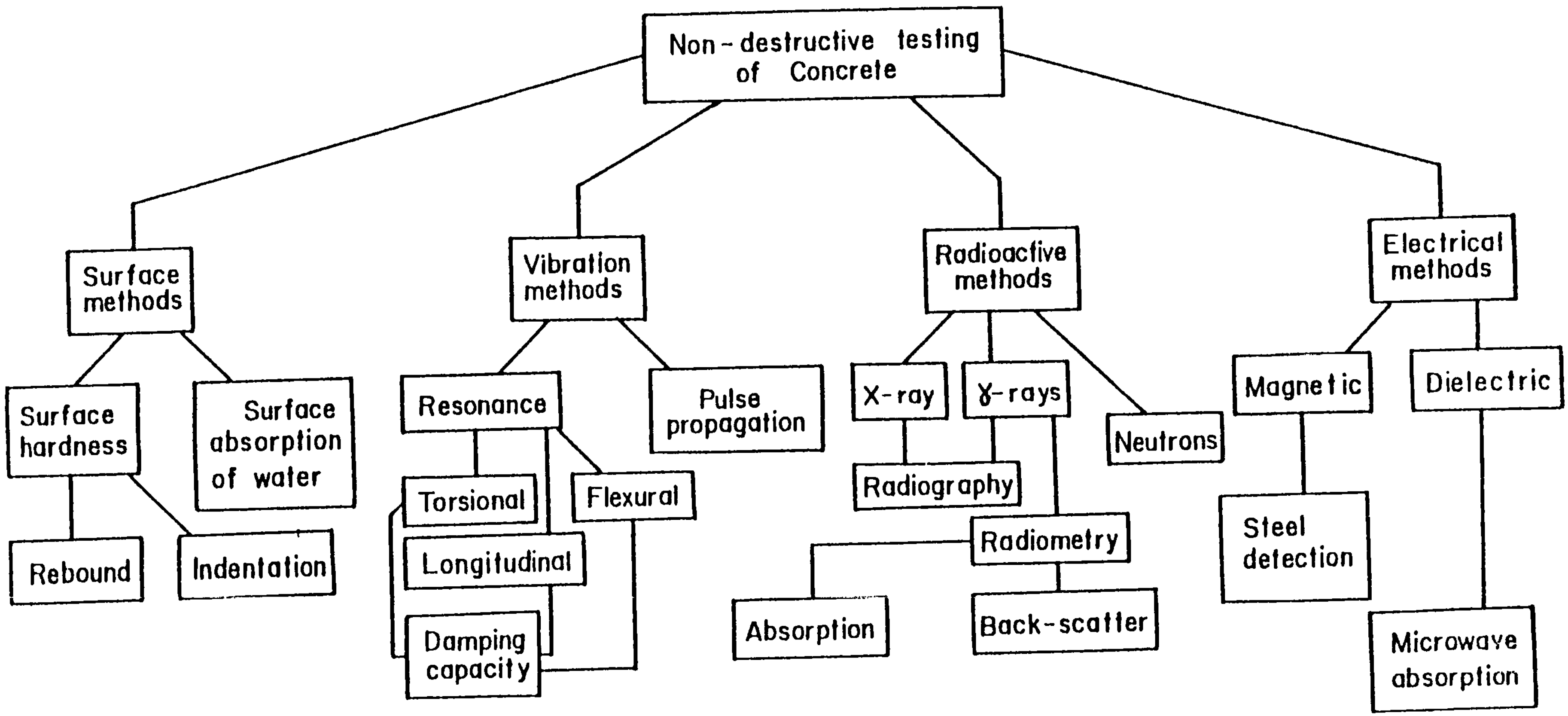


Figure 2-1. Range of techniques that have been investigated or developed for nondestructive testing of pavement systems (*11*).

Acoustic Emission Technique

Acoustic emissions are small-amplitude elastic waves created by localized deformations in concrete at points strained beyond their elastic limit. Kinetic energy is released in the propagation of elastic waves throughout a material during deformation processes. Sensors placed on the surface of a test specimen detect the small displacements. The source of the deformation is located by the variations in the time of the stress waves' arrival at each sensor. The acoustic emissions are then amplified, filtered, processed, and recorded on a magnetic tape or sent directly to a computer for analysis (*13*). This technique has been used to study the rate of cracking and the presence and growth of fatigue cracks in metals for more than 30 yr (*11,13-17*). However, during the 1950s, L'Hermite (*18*) was able to record sound when concrete was stressed. During the early experiments, a microphone, connected to an amplifier and a device to record the emitted sound characteristics, was mounted on stressed concrete specimens (*18,19*). It was found that different sounds were obtained at different loading stages. An attempt was made by Green (*20*) to correlate the acoustic emissions and the static Young's modulus and Poisson's ratio. This technique was also investigated by Malhotra (*21*) to determine the initiation of failures.

This technique can only be used when there is a change in deformation and stress and cannot be used for individual or comparative measurements of concrete in a static condition of loading. Due to the fact that this method requires loading the target structure, no further evaluation was conducted.

Dynamic Method

The dynamic (vibration) test methods have been used in concrete testing (in the laboratory and in the field) for the past five decades. This method has been used to measure the fundamental properties of concrete in the laboratory, and check the quality of concrete in bridge piers, pavements, and hydraulic structures (*2*). This method can be categorized as either a resonant frequency method or a pulse velocity method, as shown in Figure 2-2. The resonant frequency method is based on the determination of the fundamental resonant frequency of the vibration of a specimen, which is generated electromechanically. The application of this method, which has been standardized by the American Society for Testing and Materials (ASTM), has been limited to laboratory testing. The pulse velocity method can be either a mechanical sonic pulse velocity or ultrasonic pulse velocity method. The mechanical sonic pulse velocity method involves the measurement of the travel time of longitudinal or compressional waves generated by a single-impact hammer blow or repetitive blows. The ultrasonic pulse velocity method involves measurement of the travel time of

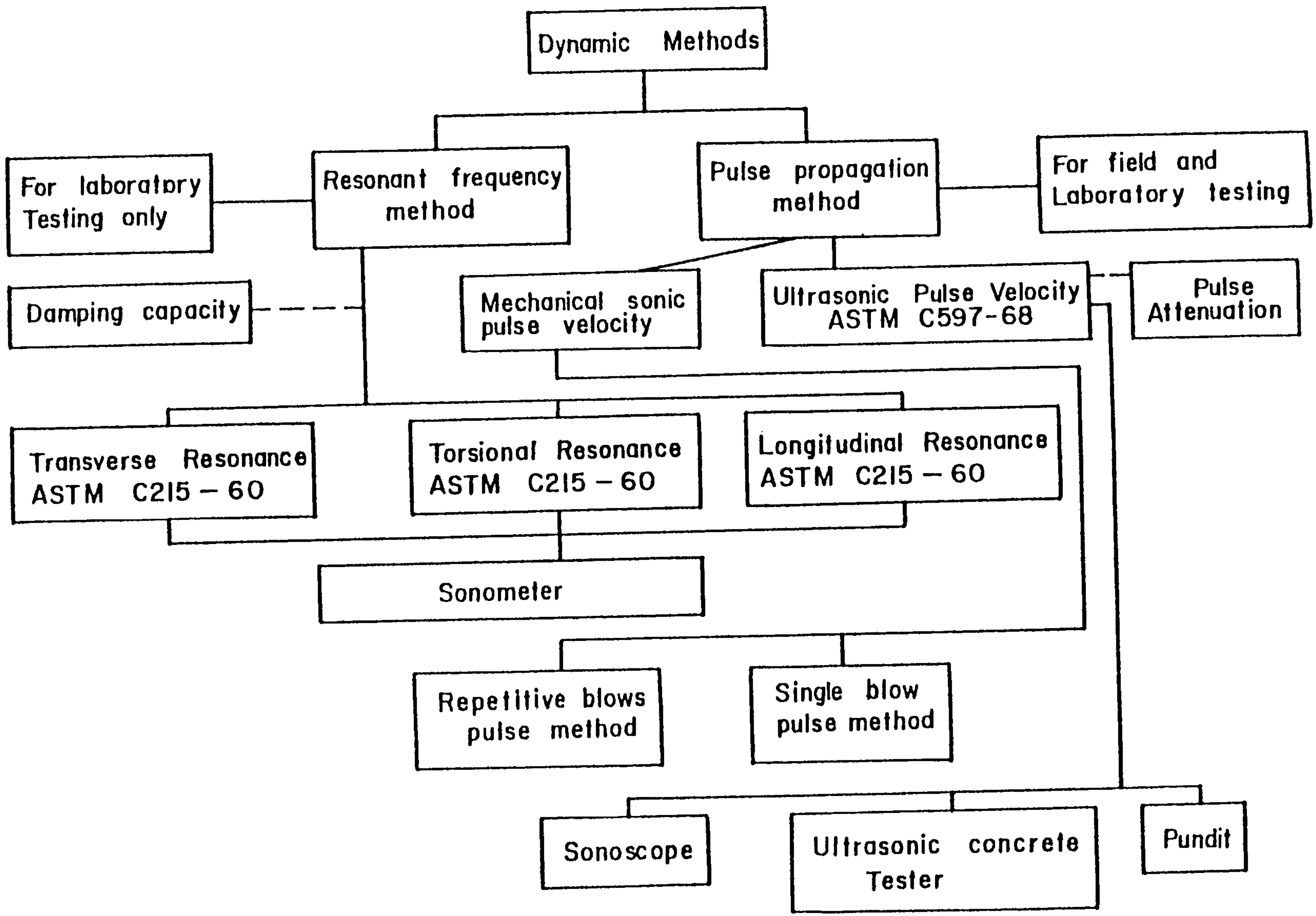


Figure 2-2. Dynamic methods (*22*).

electronically generated mechanical pulses. The transit time is measured by a digital meter and the electronic pulse is detected by an oscilloscope. This method has been standardized by ASTM as C597-83.

Ultrasonic testing has become a well-established method for the inspection of many kinds of products. The position and size of a flaw can be determined by measuring the time needed by a pulse of ultrasonic vibrations to travel from a transducer and to be retraced as an echo from the flaw. The V-meter is a low-frequency ultrasonic test system that directly measures the velocity of a pulse signal transmitted through a material. Although it is designed for concrete testing, it is also used to test other materials.

The instrument uses two piezoelectric transducers. One transducer transmits a pulsed ultrasonic signal and transfers ultrasonic energy to the tested material. The second transducer, which is located on an opposite or parallel surface, receives and detects the signal. Accurate measurements of the signal velocity can be used effectively to evaluate material properties and to detect flaws and changes in composition, as in layered systems.

When vibration is generated, three types of waves are propagated in the material: longitudinal waves, shear waves, and Rayleigh waves. Since the longitudinal wave travels faster than the other two, it is usually considered the only concern. The velocity of the longitudinal ultrasonic wave in concrete is very closely related to the elastic modulus, since any changes in the density or Poisson's ratio gives a proportionate change in the elastic modulus.

The frequencies between 20 and 250 kHz are considered suitable for construction materials, and 50 kHz is considered appropriate for testing concrete under field conditions. The higher the frequency, the narrower the beam of the propagated pulse and the greater the attenuation. In addition to the appropriate frequency, accuracy of transit time can only be ensured if a good coupling can be achieved between the transducers and the material surface.

As an ultrasonic pulse travels through a material and meets a material-flaw interface, there is a change in the transmission of energy across the interface. This, in turn, affects the transit time, which will be different for the same material without a flaw. Therefore, it is possible to make use of this effect for locating flaws.

Ultrasonic measurements have been used in many studies, including the structural analysis of bridge decks. Muenow (*23*) used the method to detect structural defects in the upper 2 in. (5 cm) of concrete bridge decks. The purpose of the investigation was to take ultrasonic measurements at random in order to detect the affected areas. The low velocity

measurements were able to detect structural damages. Suspect areas were subsequently cored and proven to be weak areas. Because this technique has a good record of detecting defects in concrete structures, it was chosen for further evaluation in the laboratory.

Nuclear Technology

Nuclear technology is currently being used in various applications of nondestructive testing of materials. In the highway industry, backscattered neutrons have been used to measure the density and moisture content of pavement materials. This application has been used as an acceptable test and quality control procedure. Nuclear magnetic resonance (NMR) has been used to monitor the variation of the in situ moisture content of various types of soils (*24*). The feasibility of using nuclear technology in the evaluation of in situ pavement condition has been investigated by Al-Qadi et al. (*12*). That study indicated that the technique has considerable potential and is reasonably safe because it does not use high energy particle bombardment. However, the technique uses strong magnets, and care has to be taken when working near steel objects. Figure 2-3 (*12*) shows the results of using nuclear technology to penetrate asphaltic concrete. The figure indicates that higher X-ray energy or a suitable gamma source is required. However, gamma sources usually have the advantage of being compact, discrete-energetic, and less expensive.

Gamma radiography is believed to be a beneficial tool for the inspection of concrete, and it provides a permanent record of defects (*25*). The principle of gamma radiography is the observation of the difference in attenuation of incident radiation resulting from the difference in the thickness and density of the material, as well as the energy of the radiation used. The observation is made by the interaction of the emergent radiation with a suitable photographic film. The developed film shows light and dark areas corresponding to the presence of steel and voids in the concrete. The steel produces more attenuation, and consequently less interaction, while the voids produce more interaction. It was used successfully, during the late 1950s, for locating steel and voids in concrete, and voids in the grouting of post-tensioned and prestressed concrete (*26-28*). However, the main use, so far, has been to detect the extent of internal faults and failures in structures. This technique has an advantage over X-rays, since it has the possibility of using high energy radiation without extensive danger of high voltage equipment and at low cost for the radiation production. However, the main disadvantages are the shielding requirements and the difficulty of obtaining a small high-intensity source in order to produce a good resolution in the radiograph.

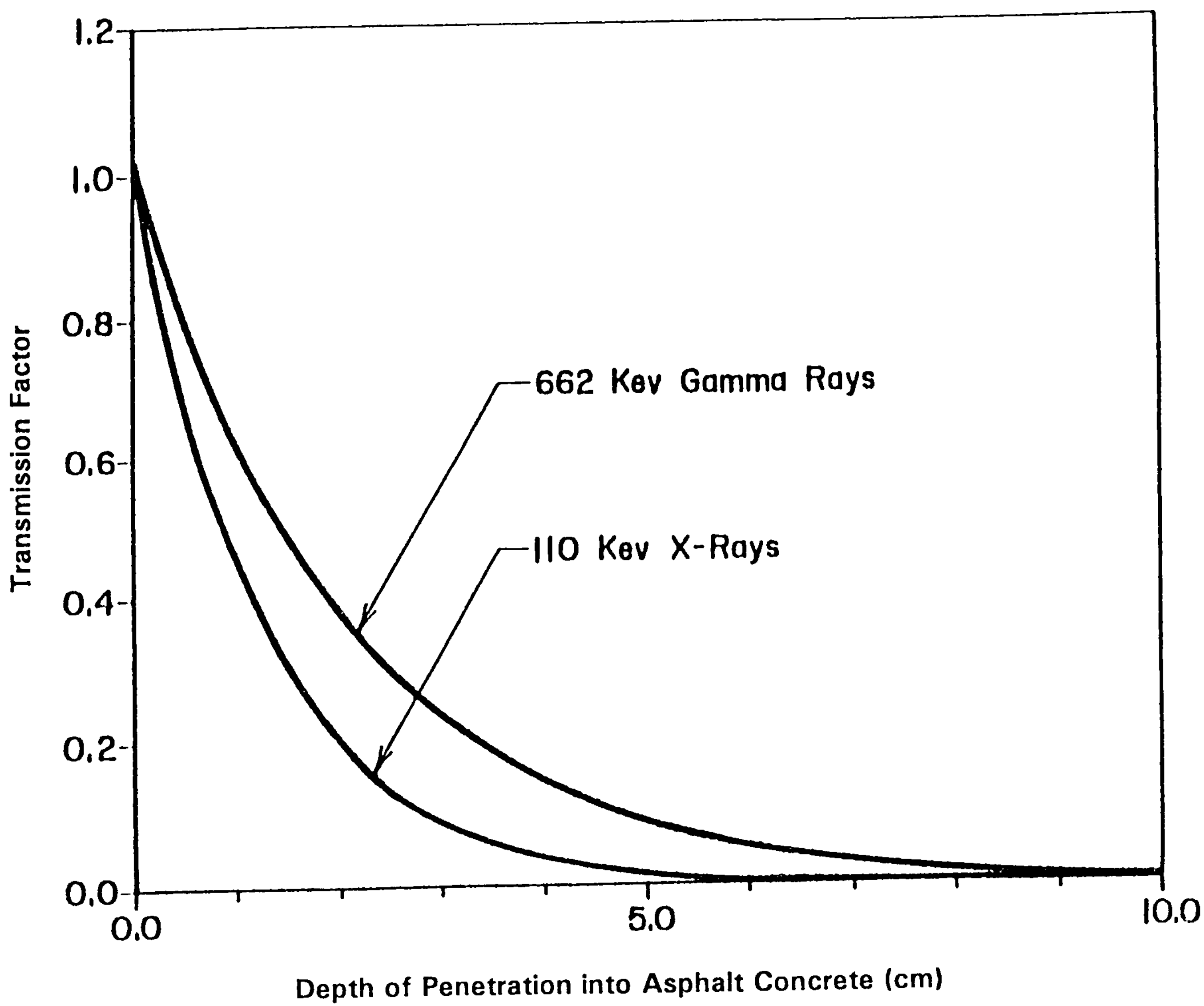


Figure 2-3. Relationship between depth of penetration and transmission factors for gamma rays and X-rays (*12*).

The availability of portable containers for high-activity sources has made it possible to carry out radiography in the field with shorter exposure times. However, the most feasible field application of gamma radiography is as a quality control tool in the checking of construction techniques at a time when remedial measures can be taken. Therefore, it was decided not to include this technique in the study because of its limiting field use characteristics.

Electromagnetic Waves

Electromagnetic technology can be used to evaluate defects in pavement systems, including detecting voids underneath rigid pavements, locating rebars, and detecting moisture content in asphaltic concrete (*29*). The principle of the ground-penetrating radar involves an induced single pulse from a transmitter followed by cessation of transmitted energy over a short time interval during which the reflected signals return to a receiver. When the electromagnetic waves are directed into a pavement surface, a portion is reflected back to the transducer from the air-surface boundary. The remaining waves propagate through the pavement until they strike another boundary, at which point another portion is reflected back. The portion not reflected penetrates through the second layer and other subsequent layers of materials and repeats the penetration and reflection until all the waves have been dissipated. The differing dielectric properties of the various layers will result in different reflection amplitude, reflection coefficient, and phase. The difference in dielectric constants between layers makes it possible to distinguish among the different layers.

During the past decade, many investigations were performed on bridge decks to evaluate the feasibility of using radar to detect defects. Ulriksen (*30*) investigated the deterioration of concrete bridges overlaid with hot-mix asphalt. He found that an increase in the reflection amplitude at the hot-mix asphalt and concrete boundary resulted when the chloride content increased. Manning and Holt (*31*) were able to verify the ability of radar to detect the asphalt/concrete interface. A signal processing technique to interpret the waveform data and an algorithm for identifying asphalt/concrete debonding was developed by Chung et al. (*32*). Clemena (*33*) used the graphic black/white "B-scan" radar output to analyze bridge deck condition. He was able to determine delaminated areas. Maser (*34*) reported a high correlation between total deterioration in hot-mix asphalt overlaid bridge decks and predicted radar data based on waveform calculations.

The commercial radar systems operate at frequencies around 1 GHz and yield a maximum spatial resolution of 0.2 to 0.4 in. (5 to 10 mm) in portland cement concrete and 0.4 to 0.6 in. (10 to 15 mm) in asphaltic concrete (*35*). This fundamental limitation in resolution

significantly reduces the ability of radar to directly detect the fine dielectric discontinuities that result from damaged or debonded membrane on a bridge deck.

The researchers felt that pulsed radar and continuous electromagnetic wave equipment can be used to detect defects in membranes if a higher frequency is used as well as a focused antenna, which will increase the resolution of the collected data. The higher frequency will reduce the electromagnetic penetration into the bridge deck. However, a frequency up to 18 GHz was proven to be capable of penetrating the asphaltic layer up to more than 3 in. (76 cm) (*36*). However, portable equipment operating at this frequency is not available at this time, and thus could not be included in this study.

3

Laboratory Experimental Program

The laboratory investigation focused on the evaluation of two specimen groups: 1- x 1-ft (0.3- x 0.3-m) specimens, referred to as the "indoor" specimens, and 5- x 5-ft (1.5- x 1.5-m) specimens, referred to as the "outdoor" slabs. The first group was prepared for evaluating nondestructive testing methods. The second group was for investigating the integrity and effectiveness of membranes.

Specimen Preparation

Indoor Specimens

Twelve small-scale specimens, 1 ft x 1 ft x 8 in. (0.3 m x 0.3 m x 20.5 cm), were cast using a Virginia Department of Transportation bridge deck A4AE concrete mixture (see Table 3-1). Each specimen was reinforced with three no. 4 (13-mm) rebars. Bituthene 5000 membrane, with slits 1/8, 1/4, and 3/8 in. wide by 1 in. long (3, 6, and 9 mm wide by 2.5 cm long) cut in the center of the membrane, was placed on nine specimens. The specimens were then overlaid with 2.5-in. (6.4-cm) of hot-mix asphalt (Virginia S-5). The mix design for the hot-mix asphalt is presented in Table 3-2. The three control specimens were not overlaid with hot-mix asphalt, but an intact membrane was installed on one of them. The other two were bare concrete.

Table 3-1. Portland cement concrete mixture.

Ingredient	Dry Weight	
Cement	635 lb	
Water	204 lb	
CA†	1882 lb	
FA*	1208 lb	
Air Entraining Agent	8.0 oz	
Retarder	19.2 oz	
Mixture Parameters	**Mean**	**Standard Deviation**
Slump (in.)	3.4	0.7
Air (%)	6.6	1.0
w/c	0.32	0.01
Compressive Strength, 7 days (psi)	4120	88
Compressive Strength, 28 days (psi)	5300	138

†AASHTO No. 57 size
*Natural Sand
Note: 1 in. = 25.4 mm; 1 oz = 28.35 g; 1 lb = 0.454 kg.

Outdoor Slabs

A total of 48 slabs were cast. Thirty-six slabs, 5 ft x 5 ft x 7.63 in. (1.52 m x 1.52 m x 19.4 cm), were cast with removable wooden forms. Twelve slabs, 5 ft x 4 ft x 9.63 in. (1.52 m x 1.22 m x 24.5 cm), were cast with galvanized steel stay-in-place (SIP) bridge deck forms. The total thickness, 9.63 in. (24.5 cm), includes the 2-in. (5.1-cm) thick SIP form. The SIP forms were 20-gage (0.9-mm) galvanized steel with a 6-in. (15.2-cm) pitch, type BF2/6. The slabs were reinforced with a top and bottom rebar mat with no. 5 (16 mm) bars at 8 in. (20.3 cm) as reinforcement and no. 4 (13 mm) bars at 12-in. (30.5-cm) temperature steel. The top and bottom rebar mats were electrically isolated. A 2.5-ft (0.76-m) length of

Table 3-2. Bituminous concrete mixture.

Gradation		
Sieve Size	Percent Passing	Marshall Results Using 50 Blows Compaction
1/2 in.	100.0	VTM = 5.4 percent
3/8 in.	97.4	VMA = 17.9 percent
No. 4	71.8	VFA = 69.5 percent
No. 8	45.5	Stability = 1950 lb
No. 16	31.2	Flow = 1950 lb
No. 30	23.0	Flow = 14.7
No. 50	15.9	Density = 147.1 pcf
No. 100	10.7	Asphalt content = 5.4 percent
No. 200	8.2	

Note: 1 in. = 25.4 mm; 1 lb = 0.454 kg; 1 pcf = 16.02 kg/m^3.

epoxy-coated tie wire was connected to the end of the center reinforcing bar of the top and bottom mat. The electrical connection of the epoxy-coated wire to the reinforcing bar was established with a no. 10/24 cadmium-plated screw. The connection was wrapped with electroplating tape. The epoxy-coated wire was brought through a side of the wooden form in order to provide the electrical connection between the top and bottom mats.

A type T-thermocouple was cast into each slab at its center point and at the depth of the center no. 5 bar in the bottom and top mat and brought through the side of the wooden form with the epoxy-coated tie wire.

The cover depth of the top mat was 2 in. (5.1 cm) and the bottom mat had a 1-in. (2.5-cm) cover depth with a 3.375-in. (8.6-cm) clearance between the two rebar mats. The position of the rebar mats was held in place with epoxy-coated chairs during the concrete placing. For the slabs cast with SIP forms, the bottom cover depth was 1 in. (2.5 cm) plus the 2-in. (5.1-cm) valleys at the valley locations in the corrugated forms. The concrete was supplied by a

local ready-mix concrete supplier. The concrete mixture design was the same as for the "Indoor Specimens" (see Table 3-1).

After 7 days of curing, the top surface was air dried, and commercially available standard preformed membranes were placed on 40 slabs. Three different types of preformed membranes were used: Series A (Bituthene 5000), Series B (Royston 10-A), and Series C (Protecto Wrap M-400). Series D is the same membrane as Series A, except that Series D is the 5- x 4-ft (1.52 - x 1.2-m) slabs with SIP forms. The membrane systems were placed according to the manufacturers' recommendations. Membranes were installed on 10 slabs in each series. Nine slabs, with membranes, from each series were punched with different hole sizes and frequencies. The tenth membrane in each series was placed as received, intact. The nine perforated membranes in each series had three hole sizes (1/8, 1/4, and 3/8 in. [3, 6, 9 mm, approximately]) and three hole frequencies (0.5 percent, 1.0 percent, and 2.0 percent). All the holes are located in a 1-ft^2 (30.5-cm^2) area in the center of the slab. The hole sizes and frequencies are presented in Table 3-3, and a schematic diagram is shown in Figure 3-1. The eight remaining slabs were control slabs. Four were bare reinforced concrete slabs and the other four slabs were overlaid with a 2.5-in. (6.4-cm) thick layer of hot-mix asphalt.

The sides of the concrete slabs and the edges of the asphalt-concrete interfaces were coated with epoxy. A watertight electrical box was mounted on each slab. The electrical boxes housed the top and bottom rebar mat thermocouple and a 10-ohm resistor, which was used to complete the electrical circuit between the bottom and top rebar mats. A 1-in. (2.5-cm) high dike was placed on each slab surface, 1 in. (2.5 cm) from the edges. The sides and the areas outside the dikes were sealed with a mixture of "Thoroseal," cement, lime, latex, and "Quick Plug" mix. All slabs were covered with roof-tops built from wood and covered with roofing paper and plastic sheets to prevent rainwater from ponding on the surface.

Temperature Effects on Membranes

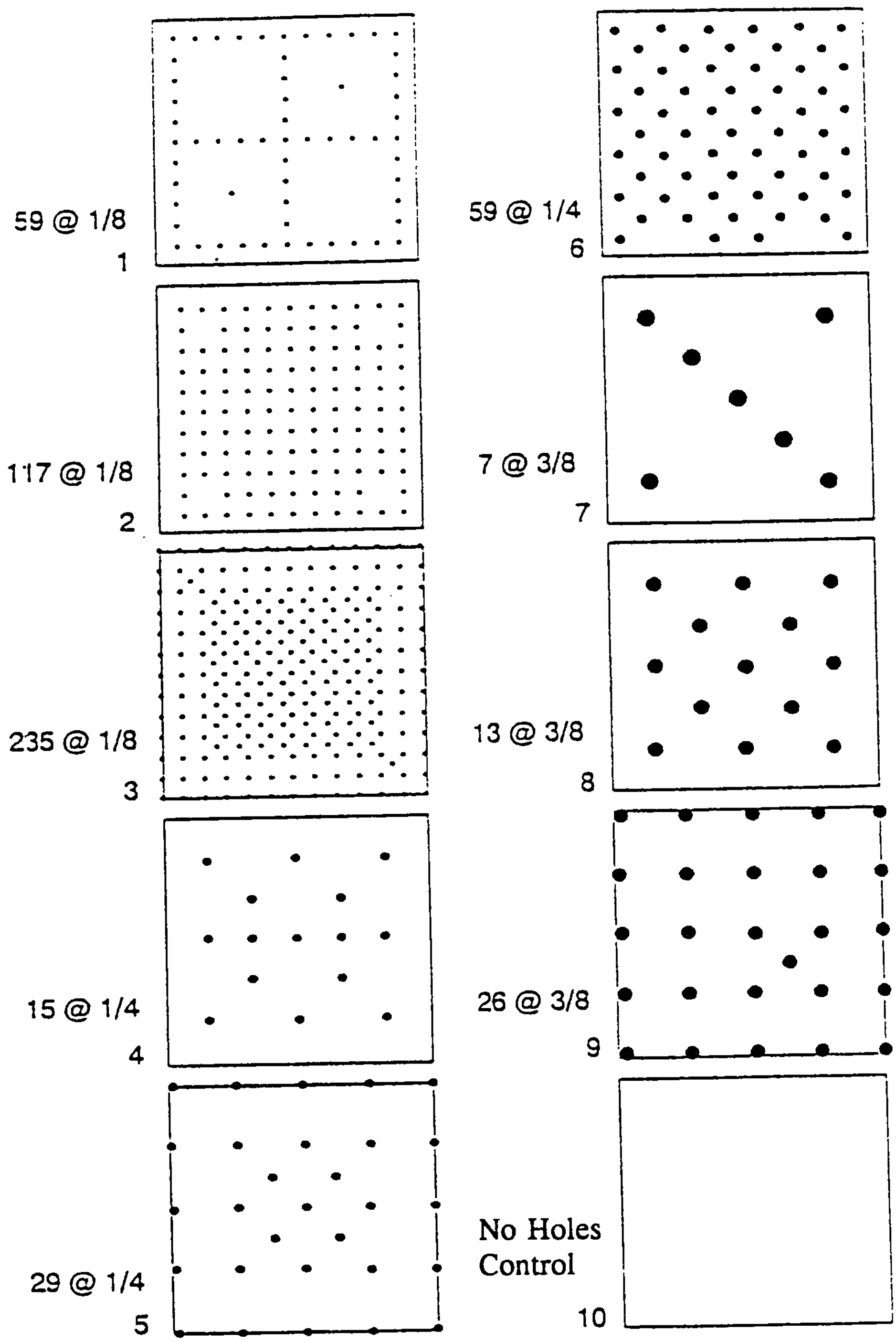
The temperature effect on the membrane was studied in two phases. In the first phase, 4-in. (10.2-cm) diameter disks of each of the three membrane types were exposed to elevated temperatures. The temperature range used was 220 to 270°F (104 to 132°C) to simulate the laying down and compaction temperature of hot-mix asphalt in the field. Bituthene 5000 showed a flow within the membrane where thin and thick spot areas were formed that lead to the development of tiny holes in the membrane. The Protecto Wrap membrane experienced some shrinkage, especially at the edges, which leads to an uneven, blistered-like surface membrane. The Royston 10-A showed minimal shrinkage and flow.

Table 3-3. Membrane types and perforation size and frequency.

Series	Template	Slab Designation: WR Grace (Bithuthene 5000)	Royston (10A)	Protecto Wrap (M-400)	WR Grace (Bithuthene 5000)
1	59 at 1/8 in.	A1	B1	C1	D1
2	117 at 1/8 in.	A2	B2	C2	D2
3	235 at 1/8 in.	A3	B3	C3	D3
4	15 at 1/4 in.	A4	B4	C4	D4
5	29 at 1/4 in.	A5	B5	C5	D5
6	59 at 1/4 in.	A6	B6	C6	D6
7	7 at 3/8 in.	A7	B7	C7	D7
8	13 at 3/8 in.	A8	B8	C8	D8
9	26 at 3/8 in.	A9	B9	C9	D9
10	None	A10	B10	C10	D10
11	None (Asp + Con)	A11	B11	C11	D11
12	None (Con)	A12	B12	C12	D12

Note: 1 in. = 25.4 mm.

The second phase involved the preparation of 12 Marshall specimens overlaying membranes with different hole sizes and patterns. A 1/4-in.-thick plexiglass sheet, which is temperature resistant up to 350°F, was placed on the mold base. The membrane was placed on the plexiglass piece. The following hole patterns were used in the membrane: 1 x 1/8 in. (25 x 3 mm), 5 at 1/8 in. (ϕ) (3 mm [ϕ]), 1 x 3/8 in. (25 x 9 mm), and 3 at 3/8 in. (ϕ) (9 mm [ϕ]). The hot-mix asphalt, with the same mixture as the slab overlays, was placed and compacted for 50 blows using an automatic Marshall compactor.



Note: each square represents a square foot (0.093 m^2) of material.

Figure 3-1. Schematic diagram of the perforation frequencies and sizes.

The holes, 5 at 1/8 in. (ϕ) (3 mm [ϕ]) and 1 x 1/8 in. (25 x 3 mm), in Bituthene membrane remained almost the same size after compaction. However, the 3 at 3/8 in. (ϕ) (9 mm [ϕ]) and 1 x 3/8 in. (25 x 9 mm) holes were increased dramatically in size, and the performance of the membrane with large holes was unsatisfactory. The bonding to hot-mix asphalt was satisfactory. The 5 at 1/8 in. (3 mm) and 1 x 1/8 in. (25 x 3 mm) holes in the Royston 10-A membrane remained almost the same after compaction, except that some pin holes were developed in the second case. The 3 at 3/8 in. (ϕ) (9 mm [ϕ]) holes were almost the same, while an increase in size was noted in the 1 x 3/8 in. (25 x 9 mm) holes, and tiny holes were developed. The bonding of the membrane to hot-mix asphalt was satisfactory. In general, the Royston 10-A membrane performed better than the Bituthene membrane.

All the holes in Protecto Wrap membranes were almost the same after compaction, and no new holes were developed. The bonding to hot-mix asphalt was the same as for the other two membranes. In general, the Protecto Wrap membrane system performed the best. However, there is an important factor that is not considered during this stage of the experiment--the bonding to the portland cement concrete. Therefore, a firm conclusion of membrane performance could not be reached at this time and was deferred until results from the slabs and field investigation were analyzed.

Deicing Salt Application

The 48 outdoor slabs were ponded with salt solution twice a week for 9 months for a total of 72 applications. Each application consisted of 0.25 lb (113.5 g) of salt (NaCl) diluted in 10.83 lb (4.92 kg) of water (2.3 percent). The slabs were thus salted at a rate equivalent to 11.4 tons/lane-mi/yr. This rate simulated the average salt application rate of the New England states: Connecticut, 7.63 tons/lane-mi/yr; Maine, 6.40 tons/lane-mi/yr; Massachusetts, 18.34 tons/lane-mi/yr; New Hampshire, 13.49 tons/lane-mi/yr; and Vermont, 11.31 tons/lane-mi/yr. (Note: 1 ton/lane-mi/yr = 565 kg/lane-km/yr, approximately.)

During the deicing application period, the temperature of the top and bottom rebar mats was measured on a monthly basis. The potential drop across the 10-ohm resistor was also measured on a monthly basis to monitor corrosion activity.

Development of Nondestructive Testing Methodology

To date, no nondestructive testing method has been completely successful in detecting defects or holes in membranes. However, as indicated in Chapter 2, many nondestructive testing

methods have been used to detect defects in reinforced concrete pavements and bridge decks. Therefore, three methods were examined in the feasibility study of this project: pulsed radar, an infrared system, and ultrasonic pulse velocity (V-meter).

A pulsed radar using 1-GHz frequency was used in an effort to detect holes in the membranes installed on the outdoor slabs. Due to the thin (1/8-in. [3.2 mm]) membrane sheets used, the pulsed radar was unable to detect any of the defects in the membranes, and the interpretation of the data was very complex. The pulsed radar was able to distinguish between the hot-mix asphalt layer and the concrete layer, but could not detect even the existence or the nonexistence of the membrane. This is due to the similarity in dielectric properties of asphalt and membrane sheets. Also, some of the electromagnetic waves were strongly reflected by the rebars due to the high penetration of the electromagnetic waves at this frequency.

As a result of these findings, it was concluded that pulsed radar is not feasible as a nondestructive testing method in this application. However, it is believed that by using a focused antenna and increasing the frequency, the probability of detecting the defects in the membrane sheets might be increased dramatically. Unfortunately, such a system is not presently available.

An attempt was made to use infrared technology as a possible method. This attempt also was unsuccessful. The researchers observed that any object placed on the slab and removed will have an effect on the infrared image for sometime later. In summary, it is the researchers' opinion that this technology is so sensitive to variations in surface properties and activities as to mask any changes in the subsurface.

The ultrasonic pulse velocity (V-meter) was considered as a possible candidate for a nondestructive testing method in this study. The V-meter used has a range of 20 to 500 KHz and the transmittal and receiver transducers operate at 54 KHz. This method was selected to be evaluated as a relative measuring device to distinguish between damaged and intact membranes as well as the nonexistence of a membrane in bridge decks overlaid with bituminous concrete.

The V-meter measures the elapsed time for the earliest part of a pulse transmitted and received by 54-KHz piezoelectric transducers. Through the use of timing circuits and displays, the elapsed time is determined and displayed in microseconds. The V-meter transducers may be arranged on the surfaces of the material being tested with the transmission being direct, indirect, or semi-direct. The direct method is the most satisfactory one since the longitudinal pulses leaving the transmitter are propagated mainly in a direction

normal to the transducer face. The indirect method, where both transducers are located on the same surface, is normally considered the least satisfactory arrangement because it is affected by the different layers in the materials. However, for this research, this was considered as being a useful factor when detecting holes in the membrane or debonding between membrane and concrete bridge deck and/or asphaltic layer, because the wave front has to pass through these defects twice. The pulse velocity method, therefore, was selected for preliminary feasibility testing.

4

Preliminary Investigation

The main objective of the preliminary evaluation was to identify the effectiveness of the method in detecting membrane defects. Therefore, the method was first used on the 1- x 1-ft (30.5- x 30.5-cm) specimens. The transducers were used in the direct mode, with one of the transducers placed on the bituminous surface and the other on the opposite concrete surface of the specimen. This measurement technique was used first because it was thought that if defects in membranes could not be detected with this measurement setup (the most accurate one), there was little hope of success in using the V-meter in the indirect mode. The wave path length in the direct method is the thickness of the test specimen.

The V-meter generates pulses that traverse the tested specimen in a specific time called the "transit time." The velocity of the pulse is calculated by dividing the path length by the transit time. This requires an accurate measurement of the specimen thickness. The velocity measurements are presented in Table 4-1. The velocity of the pulse was used to distinguish between the nine overlayed concrete specimens with holes in the membrane (*a* through *i*), the bare concrete specimen (*j*), and the concrete specimen with a membrane but no overlay (*k*). The direct measurements were taken at three positions. The preliminary results indicated that the V-meter is able to distinguish between the different specimens successfully. The ultrasonic pulse velocity for concrete is 16,174 ft/sec (4,930 m/sec), while it is 14,969 ft/sec (4,563 m/sec) for concrete and membrane and 13,311 ft/sec (4,057 m/sec) for concrete overlaid with membrane and hot-mix asphalt.

Table 4-1. Ultrasonic pulse velocity results for the indoor specimens using direct method.

Specimen	Specimen Thickness (in.)	Transit Time (Average) (μs)	Velocity (Average) (ft/sec)
a	9.3	55.9	13,781
b	9.0	55.5	13,514
c	9.3	56.6	13,620
d	9.3	54.3	14,188
e	9.1	58.3	13,051
f	9.1	58.8	12,710
g	9.3	57.9	13,321
h	9.1	58.4	13,021
i	9.0	59.6	12,595
j	7.0	36.1	16,174
k	7.1	39.7	14,969

Note: 1 in. = 25.4 mm; 1 ft/sec = 0.3 m/sec (approximately).

The next step of the preliminary testing phase was to investigate the feasibility of using the V-meter in the indirect measurement mode. The indirect measurement mode would have to be used in the field to be a practical tool in assessing the condition of membranes. The transducers were placed on the bituminous surface in three different positions relative to the orientation of the rebars in the top mat, namely: parallel to and between, perpendicular to, and parallel to and over. Four different distances between transmitter and receiver were also evaluated: 3.5, 5.0, 6.0, and 7.0 in. (8.9, 12.7, 15.2, and 17.8 cm). In addition to the transmittal time, the waveform was captured with an oscilloscope and plotter. The measurements at the 3.5-in. (8.9-cm) distance were the most consistent and repeatable. However, due to the fact that the measurements might be affected by edge diffraction, the four transducer measurement separation distances were repeated on the outdoor slabs.

On the outdoor slabs, the 3.5-in. (8.9-cm) distance was also found to be the most repeatable and consistent. Table 4-2 presents the measurements taken at different locations on Slab A10 using the 7-in. (17.8-cm) and 3.5-in. (8.9-cm) distances. The same comparison was performed on other slabs, and the same conclusion was reached. Therefore, a distance of 3.5 in. (8.9 cm) between transducers was adopted to be used throughout the remaining phases of the study. This distance is short enough to prevent the effect of reflected pulses from the bridge deck bottom and large enough to prevent interference from surface waves. According to Galan (*37*), the distance between transducers should be at least 0.9 times the wavelength in the material. The average apparent velocity of the slabs is 12,767 ft/sec (3,891 m/sec) at a frequency of 54 KHz using 3.5-in. (8.9-cm) distance between transducers. Therefore, the apparent wavelength is 3.0 in. (7.6 cm) Thus, the minimum distance between transducers should be 2.7 in. (6.9 cm). This condition is satisfied by using 3.5-in. (8.9-cm) distance.

Table 4-2. Apparent pulse velocity of Slab A10 at different locations using 7-in. (17.8 cm) and 3.5-in. (8.9 cm) transducer spacings.

	Apparent Pulse Velocity at 7 in. (ft/sec)	Apparent Pulse Velocity at 3.5 in. (ft/sec)
	9972	12518
	9921	12465
	11006	13629
	9616	12307
	10008	13823
	11154	12518
	11090	12411
	10645	12465
Mean	10414	12767
Standard Deviation	594	559
Coefficient of Variation	5.7%	4.4%

Note: 1 in. = 25.4 mm; 1 ft/sec = 0.3 m/sec (approximately).

The waveform for the 1- x 1-ft (30.5- x 30.5-cm) specimens and the outdoor slabs was evaluated. A good correlation was found between the largest absolute amplitude and defects of membrane for the specimens using the direct method. However, a lower correlation was found for the indirect method. The absolute amplitude was noticed to be affected by the surface roughness. Also, the correlation between the absolute wave amplitude and membrane defects during the initial field validation work (see Chapter 5) was not encouraging. Considering the reduced accuracy and impracticality of using an oscilloscope and plotter in the field, further analysis of the waveform was not warranted.

Corrosion Testing after Deicer Applications

After 72 deicing salt applications, the half-cell potentials were measured for all the outdoor slabs. A grid was drawn on each slab to obtain an average of 20 to 25 half-cell potential measurements. The results are summarized in Table 4-3. In Slabs 1 through 11 for all series, potentials were measured through the overlay. Active corrosion potentials, more negative than 350 mV, were detected in only two slabs: C7 and D12. Thus, the effectiveness of membranes as chloride barriers could not be assessed from CSE potential measurements. The potential drop across the 10-ohm resistor between the top and bottom rebar mats, which were measured over the 9-month salting period, also proved inconclusive in assessing the effectiveness of the membranes.

Pulse Velocity Versus Membrane Condition

The V-meter measurements were taken on all slabs. The measurements were taken at the center of each slab and at two other locations: one on the slab centerline at a location where adjacent segments of the membrane overlap, and the other at the same distance from the center, but on the orthogonal slab centerline. The measurement at the center was labeled "X," the second "Y," and the third "Z." After all the measurements were taken, ground truth cores were obtained. A hand-held coring machine with a water-cooled 3-in. (7.6-cm) diamond bit was used to drill through the bituminous concrete in the 40 slabs, A1 through D10. Two cores were extracted from each slab, one at the center and the other at one of the other two locations where the V-meter measurements were taken. Cores were obtained alternately: for example, at Slab A1, the second core was made at Point Y; at Slab A2, the second core was extracted at Point Z.

Table 4-3. Average potentials for slabs before chloride content measurements.

Slab I.D.	Average Potential (-mV)	Min. Potential (-mV)	Max. Potential (-mV)	Temp. Top Mat (°F)	Temp. Bottom Mat (°F)
A1	145.3	143.0	146.0	---*	87.9
A2	229.9	225.0	233.0	93.7	88.7
A3	141.2	110.7	192.9	73.0	72.1
A4	187.4	183.0	192.0	71.6	71.7
A5	---*	---*	---*	---*	---*
A6	120.2	110.2	137.3	---*	---*
A7	204.0	166.2	241.0	76.2	74.1
A8	163.1	154.7	170.3	92.3	90.7
A9	132.4	117.7	162.9	91.5	84.9
A10	183.3	146.5	205.0	93.5	82.4
A11	58.8	34.0	95.0	88.7	85.2
A12	191.6	116.5	300.0	---*	---*
B1	146.8	120.1	172.1	---*	88.8
B2	107.7	86.7	128.9	95.3	93.3
B3	129.5	97.3	278.3	72.5	72.5
B4	104.9	34.5	126.2	81.8	---*
B5	102.4	70.3	127.3	98.2	91.5
B6	127.2	90.0	144.8	90.6	84.3
B7	---*	---*	---*	95.0	---*
B8	158.9	132.3	194.4	97.5	90.5
B9	---*	---*	---*	---*	---*
B10	121.7	97.5	136.0	---*	97.7
B11	81.1	37.9	118.9	86.0	79.8
B12	116.3	100.8	129.0	98.9	89.9
C1	146.3	114.0	172.4	99.3	93.3
C2	158.1	127.6	190.6	88.1	---*
C3	162.5	158.6	167.5	72.5	72.1
C4	91.4	87.6	94.9	72.8	---*
C5	119.9	108.2	135.9	74.4	73.5
C6	159.3	147.1	186.9	88.2	82.7
C7	284.3	243.0	349.0	87.2	81.1
C8	138.7	107.8	158.1	94.8	---*
C9	---*	---*	---*	---*	---*
C10	102.2	61.5	121.0	91.2	---*
C11	100.5	49.9	121.5	---*	86.1
C12	220.0	185.5	253.0	107.2	96.2

Table 4-3. *(continued)*

Slab I.D.	Average Potential (-mV)	Min. Potential (-mV)	Max. Potential (-mV)	Temp. Top Mat (°F)	Temp. Bottom Mat (°F)
D1	147.9	103.6	182.6	---*	---*
D2	114.4	57.6	142.6	105.4	97.1
D3	81.5	65.3	116.4	103.6	95.5
D4	126.5	85.9	154.6	104.0	95.2
D5	117.0	70.1	140.1	103.4	95.9
D6	115.2	98.8	138.4	102.0	95.1
D7	128.9	110.4	173.8	103.4	92.6
D8	70.0	11.8	174.4	---*	98.9
D9	---*	---*	---*	---*	---*
D10	158.3	140.7	179.2	98.2	89.2
D11	100.9	20.6	198.2	---*	92.6
D12	345.1	266.0	511.0	---*	88.3

*Bad connection.
Note: °F = 1.8 x °C + 32.

After the bituminous concrete cores were extracted from all slabs with membranes, the membranes were carefully evaluated in place and then removed and evaluated again. Membranes were rated from 0 to 10. Zero indicates an extremely deteriorated membrane with no bonding to the asphalt layer nor to the concrete surface. Ten indicates a membrane in excellent condition with very strong bonding to the asphalt layer and the concrete surface. A rating from 7 to 10 indicates that the membrane is in good condition. A rating from 3 to 7 indicates that the membrane is in a moderate condition and that tiny holes and/or debonding exists, which may affect the membrane performance, but the membrane is generally in a satisfactory condition. A rating below 3 is considered dangerous, and the membrane is probably ineffective as a chloride barrier. The rating was divided between membrane status and bonding to both layers. This approach was used because the V-meter could not detect small holes in the membranes, but was found to be sensitive to debonding. Of course, any holes that exist in a membrane will eventually lead to debonding due to moisture penetration. The thickness of the hot-mix asphalt layer was found to have an insignificant effect on the V-meter measurements.

The V-meter measurements and membrane ratings are presented in Table 4-4. They indicate that the performance of membranes with holes is poorer than the performance of intact

Table 4-4. V-meter measurements and membrane evaluation.

Slab I.D.	Position X V-Meter Reading (μs)	Membrane Rating	Position Y V-Meter Reading (μs)	Membrane Rating	Position Z V-Meter Reading (μs)	Membrane Rating
A1	19.2	5.0	19.1	5.5	22.3	---*
A2	19.6	5.5	20.1	---*	21.5	7.0
A3	19.8	5.0	21.1	7.5	20.6	---*
A4	19.3	5.0	22.5	8.0	20.7	6.0
A5	20.5	5.5	23.0	8.5	19.7	---*
A6	19.9	5.5	20.5	---*	23.1	8.5
A7	18.4	4.5	21.3	7.0	20.8	---*
A8	19.5	5.0	17.7	---*	18.5	5.0
A9	17.3	3.5	19.1	5.0	17.0	---*
A10	21.1	6.5	17.4	---*	20.7	6.0
B1	18.5	4.5	20.6	7.0	22.5	---*
B2	20.4	5.5	21.2	---*	22.2	7.0
B3	17.5	3.5	20.0	8.0	22.0	---*
B4	19.2	5.0	22.4	---*	20.6	6.0
B5	19.4	5.0	20.8	5.5	19.8	---*
B6	20.1	5.5	20.0	---*	20.9	6.0
B7	17.3	3.5	19.1	6.0	19.4	---*
B8	19.0	5.0	20.7	---*	22.6	8.0
B9	17.5	3.5	23.4	8.5	18.1	---*
B10	20.7	6.0	18.0	---*	18.6	4.0
C1	19.0	5.0	20.6	6.0	20.3	---*
C2	19.2	5.0	20.9	---*	21.1	6.5
C3	18.6	4.5	20.9	6.5	20.3	---*
C4	19.6	5.0	20.1	---*	21.3	7.0
C5	18.6	4.5	20.2	6.0	19.6	---*
C6	17.5	3.5	18.9	---*	19.6	5.5
C7	18.1	4.0	20.8	6.5	21.2	---*
C8	19.4	5.5	20.5	---*	21.4	7.0
C9	18.1	4.5	21.5	7.0	22.1	---*
C10	20.4	6.0	19.3	---*	20.8	6.5
D1	18.5	4.5	20.7	6.0	20.5	---*
D2	19.6	5.0	23.1	---*	20.3	6.0
D3	18.9	5.0	20.0	6.0	21.2	---*
D4	19.9	5.5	21.5	---*	20.2	6.0
D5	19.2	5.0	21.9	6.5	20.1	---*

Table 4-4. *(continued)*

Slab I.D.	Position X V-Meter Reading (μs)	Membrane Rating	Position Y V-Meter Reading (μs)	Membrane Rating	Position Z V-Meter Reading (μs)	Membrane Rating
D6	18.3	4.0	19.9	---*	19.3	6.0
D7	18.0	4.0	21.5	7.0	21.5	---*
D8	18.0	4.0	19.4	---*	19.3	5.5
D9	18.6	4.5	19.8	5.5	19.0	---*
D10	20.1	6.0	20.6	---*	20.2	6.0

*No cores obtained.

membranes. The holes made in membrane Series A and C remained almost the same size; however, an enlargement in the hole size in Series B membrane was occasionally noticed. Series C membranes showed the least amount of change in hole size. This observation was in agreement with earlier temperature effects on the three different membrane types.

In general, the performance of the three membrane types was satisfactory with respect to their overall as-placed condition. However, bonding to the asphalt and/or concrete varied. It was observed that the Royston membrane bonded best to the concrete, while the Bituthene membrane bonded best to the asphaltic concrete. The bonding of Series C (Protecto Wrap) membrane was the poorest. This explains the unchangeable hole sizes in this type of membrane in the slabs and in the laboratory Marshall specimens after compacting the hot-mix asphalt.

The V-meter measurements were correlated with the membrane rating. Regression analysis was performed to indicate the membrane status from the V-meter readings, resulting in the following statistical model:

$$\text{MemRate} = -10.6 + 0.816 \text{ V-meter} \quad (4\text{-}1)$$

where

MemRate = the membrane rating
V-meter = the V-meter measurement in μs

A coefficient of determination (R^2) value of 85.9 percent indicates a good correlation between the membrane rating and the V-meter measurement. Also, a root mean square error (s) of 0.447 indicates a strong relationship between the dependent and independent variables. A plot of the membrane rating versus the V-meter measurement is presented in Figure 4-1. A 95-percent level of significance for the two-tail t-test concluded that both the slope and the constant are significant.

Figure 4-1 also presents the 95-percent confidence interval of the regression model. The model indicates a strong correlation between the V-meter reading and membrane rating. Therefore, the laboratory investigation was conclusive in demonstrating the feasibility of using pulse velocity (V-meter) to determine the present status of the membrane.

Chloride Content and Corrosion Potentials Versus Membrane Condition

After the membranes were removed from each slab at the core hole locations, half-cell potential measurements (CSE, ASTM C876) were taken directly on the concrete surface. The half-cell potentials are presented in Table 4-5 for all the slabs with overlays. The difference in potentials for a particular slab directly over the concrete or through the hot-mix asphalt overlay is insignificant, as shown in Tables 4-3 and 4-5. This indicates that the membranes were sufficiently electrically continuous to permit half-cell potentials. However, they were sound enough to reduce the penetration of water and chloride, as noticed when the cores were removed. The salt water accumulated between the membrane and the asphaltic layer, which may also explain the poor bonding between the asphalt layer and the membrane.

In order to evaluate the effectiveness of membranes under laboratory conditions, chloride contents were measured for each slab. Powdered concrete samples were obtained using a 1 1/8-in. (28.6-mm) diameter vacuum bit and collection unit. The powdered concrete samples were obtained at three depths: 0 to 1/2, 1/2 to 1, and 1 to 1 1/2 in. (0 to 13, 13 to 25, and 25 to 38 mm). The three depths are referred to as 1, 2, and 3, respectively. The chloride content of the powdered concrete samples was determined by the procedure described in Volume 6 of this report. The chloride contents, in lb/yd^3, and membrane ratings are presented in Table 4-6. The concrete samples for Slabs A1, A2, and A4 were taken 5 days after the membranes were removed. During that period, salt water had covered the core holes, which again demonstrated the ability of membranes to act as a chloride barrier. The concrete samples obtained from these locations showed very high chloride content. This was most likely due to the surface chlorides contaminating the samples during drilling. It also appears that chloride samples at Depths 1 and 3 may have been in advertently switched at location A1-X. Slab A11 has a low chloride content, which may be

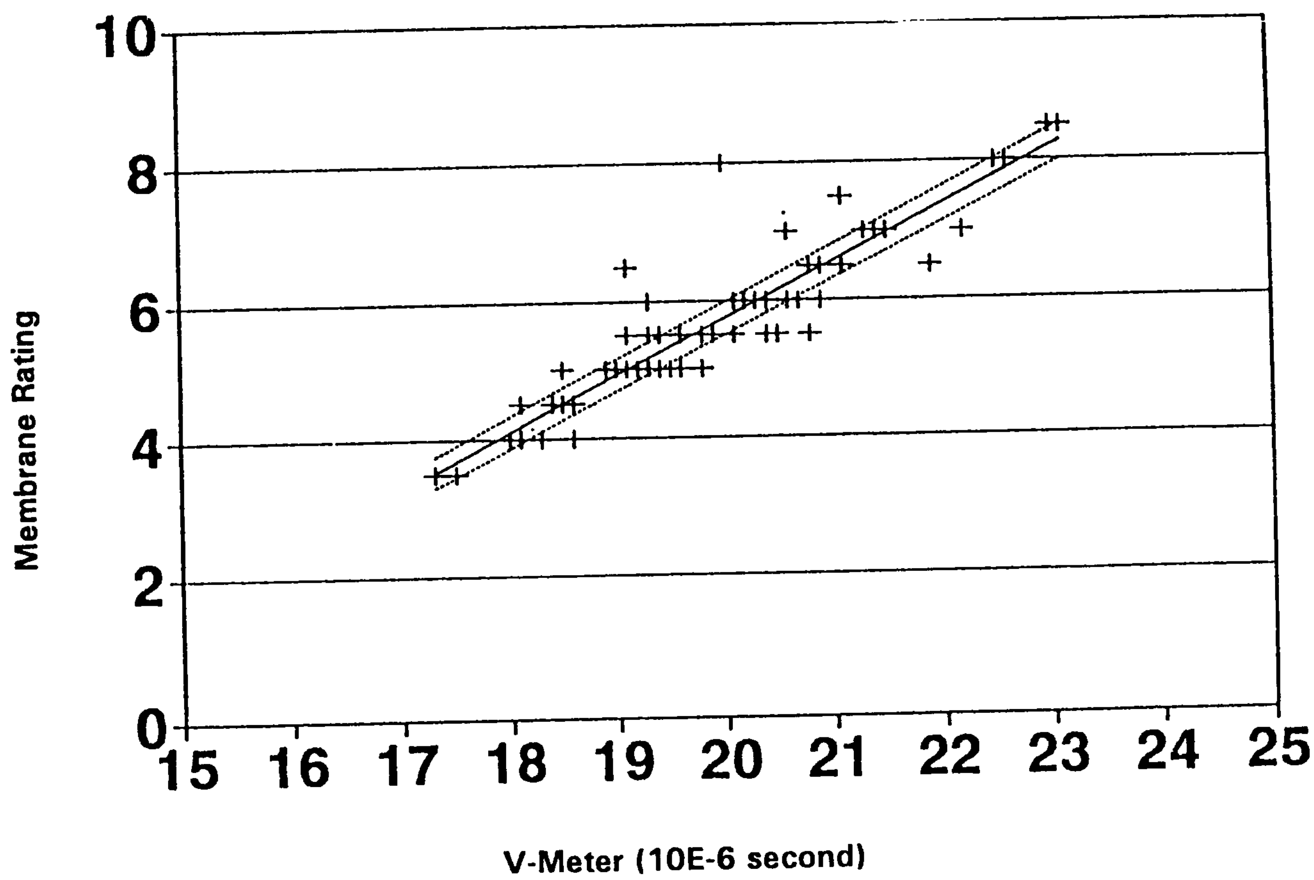


Figure 4-1. The membrane rating model with 95-percent confidence interval (laboratory slabs).

Table 4-5. Potential measurements for slabs in mV.

Slab I.D.	Position X	Position Y	Position Z
A1	-180.0	-157.0	---*
A2	-225.0	---*	-227.0
A3	-161.0	-173.0	---*
A4	-197.0	-196.0	-196.0
A5	---*	---*	---*
A6	-143.0	---*	-146.0
A7	-208.0	-205.0	---*
A8	-179.0	---*	-189.0
A9	-177.0	-176.0	---*
A10	-189.0	---*	-178.0
B1	-162.0	-165.0	---*
B2	-107.0	---*	-105.0
B3	-168.0	-164.0	---*
B4	-135.0	---*	-128.0
B5	-78.0	-90.0	---*
B6	-137.0	---*	-147.0
B7	-168.0	-142.0	---*
B8	-164.0	---*	-107.0
B9	-95.0	-99.0	---*
B10	-124.0	---*	-115.0
C1	-153.0	-154.0	---*
C2	-161.0	---*	-160.0
C3	-158.0	-169.0	---*
C4	-121.0	---*	-117.0
C5	-125.0	-120.0	---*
C6	-161.0	---*	-151.0
C7	-229.0	-229.0	---*
C8	-149.0	---*	-146.0
C9	---*	---*	---*
C10	-115.0	---*	-110.0
D1	-165.0	-146.0	---*
D2	-111.0	---*	-84.0
D3	-54.0	-140.0	---*
D4	-115.0	---*	-140.0
D5	-96.0	-147.0	---*

Table 4-5. (*continued*)

Slab I.D.	Position X	Position Y	Position Z
D6	-82.0	---*	-52.0
D7	-130.0	-143.0	---*
D8	-74.0	---*	-49.0
D9	-78.0	-94.0	---*
D10	-186.0	---*	-205.0
D11	-89.0	-87.0	---*

*Bad connection.

due to some leakage at the membrane-bituminous concrete layer interface, while A10 indicated a chloride content higher than expected. In general, the chloride measurements of Depth 2 (3/4 in. or 19 mm) indicated that a hole size of 3/8 in. (10 mm) resulted in a higher degree of chloride contamination of the concrete. Also, the higher the percent of the perforated area, the greater the chloride content.

In the Series B slabs, the chloride contents at Position B were the lowest. Position B is the location of the overlapping in membrane sheets. This indicates that a thicker membrane is a better chloride barrier. However, the opposite was observed in Series C, in that the overlap locations were more contaminated. This observation is in agreement with an earlier finding that Protecto Wrap membrane demonstrated the least bonding. However, the membrane in Series C performed satisfactorily as a chloride barrier and indicated less chloride contamination compared to Series A and B. It must be noted here that a membrane was installed on Slab C11 by mistake. The membrane existence was first detected by the V-meter and then verified during the core sampling. The chloride contents in Series D slabs were relatively low, although the same membrane as the one on Series A was installed on them. This series showed lowest chloride contamination among all the series. The slab overlaid with hot-mix asphalt without membranes showed two to three times the chloride contamination compared to protected slabs, and, bare (control) slabs, as expected, showed a very high chloride contamination. The identification X, Y, Z, Z1, and Z2 was used in the control slabs to indicate that concrete samples were obtained at random locations where the highest potentials were recorded.

Table 4-6. Chloride content and membrane rating.

Slab	Cl (1) (lb/yd^3)	Cl (2) (lb/yd^3)	Cl (3) (lb/yd^3)	Membrane Rating
A1-X†	12.988	25.903	31.871	5.0
A1-Y	18.329	11.493	6.433	5.5
A2-X	12.088	1.955	0.772	5.5
A2-Z	9.668	2.537	0.791	7.0
A3-X	0.731	<0.04	<0.04	5.0
A3-Y	0.397	<0.04	<0.04	7.5
A4-X	17.193	7.572	3.267	5.0
A4-Z	7.766	0.753	0.849	8.0
A5-X	1.303	<0.04	<0.04	5.5
A5-Y	2.094	<0.04	<0.04	8.5
A6-X	6.238	<0.04	<0.04	5.5
A6-Z	3.010	0.415	<0.04	8.5
A7-X	1.281	<0.04	<0.04	4.5
A7-Y	2.069	<0.04	<0.04	7.0
A8-X	3.194	0.528	0.694	5.0
A8-Z	3.493	0.528	0.583	5.0
A9-X	2.354	0.113	0.063	3.5
A9-Y	3.034	0.334	<0.04	5.0
A10-X	6.520	0.180	0.163	6.5
A10-Z	3.179	0.457	0.096	6.0
A11-X	0.548	0.096	0.096	---*
A11-Y	1.907	0.282	0.047	---*
A12-X	10.612	1.362	0.299	---*
A12-Y	15.188	2.484	0.475	---*
A12-Z	8.680	3.854	0.404	---*
A12-Z	13.348	5.488	1.427	---*
B1-X	3.813	0.724	1,053	4.5
B1-Y	0.705	0.024	0.009	7.0
B2-X	0.595	0.934	0.487	5.5
B2-Z	1.033	0.595	0.934	7.0
B3-X	3.504	1.558	1.558	3.5
B3-Y	1.278	0.993	0.993	8.0
B4-X	2.190	0.212	<0.04	5.0
B4-Z	4.953	1.536	1.195	6.0
B5-X	5.607	1.134	1.033	5.0
B5-Y	2.494	0.895	0.800	5.5
B6-X	1.603	---*	1.093	5.5
B6-Z	4.575	0.954	1.154	6.0

Table 4-6. *(continued)*

Slab	Cl (1) (lb/yd^3)	Cl (2) (lb/yd^3)	Cl (3) (lb/yd^3)	Membrane Rating
B7-X	2.442	1.013	0.974	3.5
B7-Y	3.065	0.954	1.033	6.0
B8-X	3.194	0.528	0.694	5.0
B8-Z	3.493	0.528	0.583	8.0
B9-X	2.606	0.230	0.732	3.5
B9-Y	1.667	0.675	0.510	8.5
B10-X	2.796	0.351	0.456	6.0
B10-Z	1.024	0.601	0.564	4.0
B11-X	2.660	0.403	0.299	---*
B11-Y	0.984	0.403	0.492	---*
B12-X	8.682	0.546	---*	---*
B12-Y	12.693	0.656	0.163	---*
B12-Z	12.763	2.660	0.113	---*
B12-Z1	9.667	0.675	0.163	---*
C1-X	0.351	0.421	0.299	5.0
C1-Y	0.230	0.247	0.385	6.0
C2-X	0.213	0.299	0.113	5.0
C2-Z	0.213	0.333	0.196	6.5
C3-X	2.005	0.528	0.474	4.5
C3-Y	0.846	0.247	0.063	6.5
C4-X	0.403	0.196	0.213	5.0
C4-Z	2.553	0.316	0.247	7.0
C5-X	0.368	0.528	0.368	4.5
C5-Y	1.488	0.456	0.583	6.0
C6-X	1.554	0.385	0.456	3.5
C6-Z	0.656	0.299	0.130	5.5
C7-X	0.350	0.524	0.000	4.0
C7-Y	0.687	0.687	0.761	6.5
C8-X	1.070	0.780	0.780	5.5
C8-Z	0.837	0.687	0.669	7.0
C9-X	0.687	0.614	0.507	4.5
C9-Y	1.464	0.560	0.524	7.0
C10-X	0.578	0.650	0.542	6.0
C10-Z	0.507	0.507	0.650	6.5
C11-X	1.336	0.780	0.560	---*
C11-Y	1.336	0.669	0.385	---*
C12-X	15.097	13.304	7.182	---*
C12-Y	20.455	7.884	1.729	---*

Table 4-6. *(continued)*

Slab	Cl (1) (lb/yd³)	Cl (2) (lb/yd³)	Cl (3) (lb/yd³)	Membrane Rating
C12-Z	22.019	12.139	5.589	---*
C12-Z1	22.563	8.632	2.740	---*
C12-Z2	18.704	7.645	2.152	---*
D1-X	0.488	0.104	<0.04	4.5
D1-Y	0.506	0.087	<0.04	6.0
D2-X	0.561	0.380	0.275	5.0
D2-Z	0.711	0.206	0.155	6.0
D3-X	0.654	0.362	0.292	5.0
D3-Y	0.432	0.398	1.027	6.0
D4-X	1.278	0.309	0.223	5.5
D4-Z	0.206	0.155	0.257	6.0
D5-X	0.673	0.327	0.104	5.0
D5-Y	0.808	0.327	0.309	6.5
D6-X	0.292	0.189	0.071	4.0
D6-Z	0.172	0.172	<0.04	6.0
D7-X	0.172	0.223	0.488	4.0
D7-Y	0.344	0.240	0.104	7.0
D8-X	0.155	0.121	0.257	4.0
D8-Z	0.172	<0.04	0.104	5.5
D9-X	0.344	0.292	<0.04	4.5
D9-Y	0.309	0.275	0.189	5.5
D10-X	0.257	0.071	0.087	6.0
D10-Z	0.415	0.275	0.155	6.0
D11-X	0.598	<0.04	0.054	---*
D11-Y	0.926	0.087	0.054	---*
D12-X	9.012	1.153	---*	---*
D12-Y	11.944	2.094	0.170	---*
D12-Z	10.524	2.810	0.291	---*
D12-Z1	10.962	3.970	1.303	---*
D12-Z2	11.944	1.217	0.052	---*

†Chloride content progression in reverse order--samples may have been switched inadvertently.
*Sample not obtained.
Note: 1 lb/yd³ = 0.59 kg/m³ (approximately).

To further examine the effect of hole size on the performance of all membrane types, the chloride contents were averaged for the membrane types for specific locations (see Table 4-7). The average results showed that a 1/4-in. (6-mm) hole is a critical size when considering the average chloride content at Depth 1 (1/4 in. [6 mm]). The chloride contents of the slabs with membranes having a hole size of 1/4 in. (6 mm) and larger are two to three times the chloride contents of the slabs with membranes of hole sizes of 1/8 in. (3 mm) (Slabs 1 through 3). However, the chloride contents are approximately the same for the other two depths, 3/4 in. (19 mm) and 1 1/4 in. (32 mm).

In order to determine the effect of perforation percentage, the chloride content results were analyzed considering the perforation percent, regardless of the hole size and frequency (see Table 4-8). The effect of perforation was very pronounced for Series C slabs, which is the only series for which the hole size did not change after the hot-mix asphalt overlay was applied and the series that showed the least bonding to asphaltic overlay and concrete surface. This observation is true for the chloride measurements at all three depths for the C series. The chloride contents at Depth 2 for the other series generally increased with increasing perforation percentage. However, no correlation was observed when considering the chloride at Depth 3 (1 1/4 in. [32 mm]). The reason for these variations is the fact that some of the slabs were not accurately leveled and that the hot-mix asphalt layer thickness varied. In general, the chloride contents in Series A and B slabs were high compared to Series C and D slabs. However, when all of the series are considered, the chloride contents at all depths are proportional to the perforation percentage, as shown in Table 4-8. To investigate the effect of membrane type on the chloride content, a regression analysis was performed. The R^2 value for chloride content and the membrane type was very low. Considering Chloride Content 1, R^2 was 10 percent (r = 32 percent); for Chloride Content 2, R^2 was 38 percent (r = 62 percent); and for Chloride 3, R^2 was 50 percent (r = 71 percent). These values indicate that no strong correlation exists between the membrane type and the chloride content.

Table 4-7. Average chloride content at a specific location for the four series slabs.

Position On Slab	Avg. Cl (1) (lb/yd^3)	Avg. Cl (2) (lb/yd^3)	Avg. Cl (3) (lb/yd^3)
1X	1.533	0.417	0.472
1Y	0.472	0.421	0.275
2X	0.444	0.550	0.275
2Z	0.641	0.354	0.118
3X	1.730	0.629	0.590
3Y	0.747	0.432	0.550
4X	1.297	0.236	0.157
4Z	2.555	0.668	0.550
5X	1.985	0.511	0.393
5Y	1.730	0.432	0.432
6X	2.437	2.083	0.432
6Z	2.123	0.472	0.354
7X	1.061	0.472	0.393
7Y	1.533	0.472	0.472
8X	1.887	0.472	0.629
8Z	2.005	0.432	0.511
9X	1.494	0.314	0.354
9Y	1.612	0.472	0.314
10X	2.555	0.314	0.314
10Z	1.336	0.472	0.354

Note: 1 lb/yd^3 = 0.59 kg/m^3 (approximately).

Table 4-8. Chloride content average for a specific location.

Percent Holes (%)	Avg. Cl (1) (lb/yd³)	Avg. Cl (2) (lb/yd³)	Avg. Cl (3) (lb/yd³)
0.5 (Series A)	---*	---*	---*
1.0 (Series A)	2.240	0.275	0.373
2.0 (Series A)	3.105	0.079	0.039
0.5 (Series B)	2.791	0.629	0.708
1.0 (Series B)	3.145	0.865	0.747
2.0 (Series B	2.555	0.904	1.140
0.5 (Series C)	0.354	0.393	0.197
1.0 (Series C)	0.550	0.550	0.432
2.0 (Series C)	1.415	0.511	0.472
0.5 (Series D)	0.708	0.236	0.236
1.0 (Series D)	0.472	0.275	0.236
2.0 (Series D)	0.432	0.393	0.118
0.5 (All Series)	1.297	0.377	0.342
1.0 (All Series)	1.533	0.507	0.444
2.0 (All Series)	1.887	0.432	0.472

Note: 1 lb/yd³ = 0.59 kg/m³ (approximately).
*Sample not obtained.

5

Field Trials

Questionnaire

In order to investigate membrane performance under field conditions and validate the V-meter as a nondestructive testing method to predict membrane deterioration, a field study was performed. The field investigation was initiated by choosing the 22 states that reported *(6)* using membranes as a bridge deck protection system. A questionnaire was sent to the 22 state Departments of Transportation along with a letter explaining the project and requesting that up to six engineers from each state, who have adequate experience in membrane performance, fill out the questionnaire. A total of 54 individual responses from 18 states were received.

The questionnaire was presented in three sections covering those areas that are believed to influence membrane performance. The first section considered service effects, the second section considered environmental effects, and the third section considered construction effects. The participants were asked to rate the proposed influence factors from 1 to 10 with 1 indicating no effect and 10 indicating a strong effect. The participants were also asked to rank the importance of the three performance-determining effects (service, environment, and construction). The questionnaire's results are presented in Table 5-1. The researchers felt that the New England states as a group have the most experience in membrane installation and performance. Therefore, another analysis was performed based on the New England states' questionnaire responses (see Table 5-2).

Table 5-1. Analysis of questionnaire responses.

Part I: Service Effects on Membrane Performance (1-10)

Factor	No. of Responses	Average Rate	Standard Deviation	Coefficient of Variation (%)
Acceleration Areas	54	5.63	2.98	52.9
Braking Areas	54	6.98	2.91	41.7
Departure Slabs	54	2.89	1.67	57.8
Approaching Slabs	54	3.74	2.24	59.9
Turning Areas	54	5.35	2.67	49.9
Wheel Path Areas	53	6.72	2.59	38.5
Areas between Wheel Paths	53	3.43	2.04	59.5
Passing Lane	53	3.55	1.93	54.4
Slow Lane	50	3.76	2.44	64.9
Shoulder Area	52	3.10	2.18	70.3
Curb and Gutters	52	6.19	3.06	49.4
Drainage	51	6.73	2.69	40.0
Bridge Slope	51	5.69	2.60	45.7
Bridge Super Elevation	51	5.82	2.62	45.0
Bridge Horizontal Curvature	51	5.16	2.68	51.9
Bridge Vertical Curvature	51	4.71	2.44	51.8
Bridge Vibration	51	4.24	2.71	63.9
AADT	53	7.23	2.32	32.1
Part II: Environmental Effects on Membrane Performance (1-10)				
Snowfall	54	4.70	2.52	53.6
Rainfall	54	4.74	2.37	50.0
Humidity	53	3.94	2.22	56.3

Table 5-1. *(continued)*

Part II: Environmental Effects on Membrane Performance (1-10)

Factor	No. of Responses	Average Rate	Standard Deviation	Coefficient of Variation (%)
Temperature	54	6.04	2.54	42.1
Deicing Application	54	6.50	2.87	44.2
Part III: Construction Effects On Membrane Performance (1-10)				
Type of Preformed Membrane	54	7.77	2.19	28.2
Thickness of Preformed Membrane	54	6.89	2.33	33.8
Method of Membrane Application	51	7.86	2.11	26.8
Bonding between Membrane and portland cement concrete	55	8.93	1.41	15.8
Type of Superstructure	54	4.58	2.45	53.5
Moisture Content of portland cement concrete	52	6.39	2.35	36.8
Roughness of Concrete Deck	52	7.59	1.91	25.2
Maximum Aggregate Size in Asphaltic Mixture	51	6.86	2.18	31.8
Type of Asphalt Cement	51	5.31	2.36	44.4
Hot-Mix Asphalt (HMA) Temperature at Lay-down	49	7.52	2.04	27.1
Aggregate Angularity in HMA	49	5.96	2.43	40.8
Thickness of HMA Layer	49	7.31	2.25	30.8

Table 5-1. *(continued)*

Part III: Construction Effects on Membrane Performance (1-10)

Factor	No. of Responses	Average Rate	Standard Deviation	Coefficient of Variation (%)
Compaction Effort	47	7.02	2.19	31.2
Compaction Method	47	6.44	2.22	34.5
Fuel Spillage (During Construction)	51	7.92	2.23	28.2
Environment (During Construction)	48	7.47	2.48	33.2
Major Effects: (1-3)				
Service Effects	51	2.19	0.56	25.6
Construction Effects	51	2.34	0.88	37.6
Environmental Effects	51	1.71	0.82	48.0

As shown in Tables 5-1 and 5-2, there is a general agreement between the New England states and all the responding Departments of Transportation. In the service effects, the participants believed that the areas where membranes are most seriously exposed to deterioration are acceleration and braking areas, turning areas, wheel paths, curb and gutter zones, drainage areas, bridge curvatures and slopes, and areas with high AADT. Table 5-1 indicates that AADT is the most important factor; however, the coefficient of variation is 32 percent. In Table 5-2 (the New England states), curb and gutters are considered to be the most important factor with a coefficient of variation of 21 percent. Temperature and deicing chemical application rate are considered important environmental factors in both tables. The construction factors seem to have a major effect on membrane performance. As shown, bonding of the membrane to the bridge deck surface and to the hot-mix asphalt overlay is considered to be the most important factor. In Table 5-2, the effect of the bonding of the membrane to the concrete surface was rated the highest of all effects. Therefore, this factor is considered a very important factor in rating membrane performance in this study. The type of asphalt cement is considered the least important. Among the three effects, service,

Table 5-2. Analysis of responses of New England States.

Part I: Service Effects on Membrane Performance (1-10)

Factor	No. of Responses	Average Rate	Standard Deviation	Coefficient of Variation (%)
Acceleration Areas	21	6.05	3.06	50.6
Braking Areas	21	7.71	2.99	38.8
Departure Slabs	20	2.45	1.66	67.8
Approaching Slabs	20	3.55	2.31	65.1
Turning Areas	21	5.81	2.57	44.2
Wheel Path Areas	21	6.05	2.72	45.0
Areas between Wheel Paths	20	3.05	1.94	63.6
Passing Lane	21	3.33	1.75	52.6
Slow Lane	21	3.33	2.23	67.0
Shoulder Area	20	2.60	2.01	77.3
Curb and Gutters	21	8.19	1.71	20.9
Drainage	20	7.90	2.35	29.7
Bridge Slope	20	6.15	2.83	46.0
Bridge Super Elevation	20	5.85	2.78	47.5
Bridge Horizontal Curvature	20	4.70	2.88	61.3
Bridge Vertical Curvature	20	4.95	2.65	53.5
Bridge Vibration	20	4.00	3.10	77.5
AADT	21	6.81	2.13	31.3

Part II: Environmental Effects on Membrane Performance (1-10)

Factor	No. of Responses	Average Rate	Standard Deviation	Coefficient of Variation (%)
Snowfall	20	4.10	2.47	60.2
Rainfall	20	4.05	2.27	56.0
Humidity	19	2.89	1.83	63.3

Table 5-2. *(continued)*

Part II: Environmental Effects on Membrane Performance (1-10)

	No. of Responses	Average Rate	Standard Deviation	Coefficient of Variation (%)
Temperature	20	5.95	2.71	45.5
Deicing Application	20	6.65	2.54	38.2

Part III: Construction Effects On Membrane Performance (1-10)

	No. of Responses	Average Rate	Standard Deviation	Coefficient of Variation (%)
Type of Preformed Membrane	20	8.30	1.82	21.9
Thickness of Preformed Membrane	20	6.85	2.43	35.5
Method of Membrane Application	21	8.42	1.31	15.6
Bonding between Membrane and portland cement concrete	20	9.48	0.66	7.0
Type of Superstructure	19	3.65	1.98	54.2
Moisture Content of portland cement concrete	19	6.21	2.76	44.4
Roughness of Concrete Deck	21	8.15	1.65	20.2
Maximum Aggregate Size in Asphaltic Mixture	20	5.90	1.92	32.5
Type of Asphalt Cement	21	4.00	1.83	45.8
Hot-Mix Asphalt (HMA) Temperature at Lay-down	21	7.85	1.9	24.2
Aggregate Angularity in HMA	21	6.00	2.43	40.5
Thickness of HMA Layer	20	7.05	2.33	33.0
Compaction Effort	20	6.83	2.41	35.3

Table 5-2. *(continued)*

Part III: Construction Effects on Membrane Performance (1-10)

	No. of Responses	Average Rate	Standard Deviation	Coefficient of Variation (%)
Compaction Method	21	6.17	2.24	36.3
Fuel Spillage (During Construction)	20	7.37	2.58	35.0
Environment (During Construction)	19	7.58	2.16	28.5
Major Effects: (1-3)				
Service Effects	20	2.25	0.43	19.1
Construction Effects	20	2.00	0.97	48.5
Environmental Effects	20	1.90	0.97	51.1

environment, and construction, the environmental effects are considered the least important, while the other two are considered almost as equally important.

Among the states who were sent the questionnaire, seven states who have extensive experience with membranes were chosen for the field validation testing. The states chosen were Vermont, New Jersey, New Hampshire, Connecticut, Rhode Island, Massachusetts, and Maine. A letter was sent to each state requesting information on a total of nine bridges encompassing three types of membranes and three age categories--1 to 8, 9 to 13, and over 13 yr old. The membranes were to have been installed on newly constructed bridge decks.

Based on the responses to the requests for information, three candidate states, Vermont, New Hampshire, and Maine, were contacted and field validation dates were scheduled. Five bridges were chosen from each state. Information regarding the chosen bridges is presented in Table 5-3. New Hampshire indicated that the membranes used in their bridge decks are the same as the three membrane types used in the laboratory investigation. However, no record of the membrane types on each bridge is available. Among the five bridges evaluated in New Hampshire, four have a Royston 10-A membrane. Different terms were used for

Table 5-3. Information on evaluated bridges.

State, Bridge No.	Bridge Location	Highway/Route	Date Constructed	Membrane Brand	Span Length (ft)	No. of Lanes	Avg. Daily Temp. (°F)	Avg. Annual Snow-fall (in.)	Avg. Annual Deicing Applica-tion (ton/lane-mile/yr)	AADT	% Truck
NH, 107/074	Stratford	Spur Road	1984	Royston 10-A	52.0	2	N/A	45.7	10.2	100	4
NH, 117/157	Tilton (Northfield)	I-93 (SB)	1979	Royston 10-A	342.0	2	N/A	66.5	18.3	6380	9
NH, 118/158	Tilton (Northfield)	I-93 (NB)	1979	Royston 10-A	333.0	2	N/A	66.5	18.3	6380	9
NH, 117/035	Hillsborough	US 202	1988	Bituthene	112.0	2	N/A	57.0	14.9	3080	7
NH, 165/177	Concord	Rt. 9/I-393	1988	Royston 10-A	303.0	2	N/A	65.4	16.9	4000	6
ME, 6255	Caribou	US 1	1971	Royston 10-A	437.0	2	38.9	103.0	4.0	8000	10
ME, 1376	South Portland	US 1 (SB)/I-295	1974	Protecto Wrap M 400	508.0	2	45.9	80.0	2.0	15600	6
ME, 5807	Augusta	Ramp F	1959	---*	202.5	2	44.0	78.0	3.0	5561	10
ME, 0822	Biddeford	Biddeford Connector	1989	Bituthene	390.0	2	45.0	80.0	2.0	6830	5

Table 5-3. (*continued*)

State, Bridge No.	Bridge Location	Highway/Route	Date Constructed	Membrane Brand	Span Length (ft)	No. of Lanes	Avg. Daily Temp. (°F)	Avg. Annual Snow-fall (in.)	Avg. Annual Deicing Applica-tion (ton/lane-mile/yr)	AADT	% Truck
ME, 1562	Brewer	Ramp/I-395	1986	Royston 10-A	240.0	2	43.8	77.0	3.0	6700	7
VT, 64N	Newbury	I-91 (NB)	1973	Bituthene	98.0	2	45.0	36.0	7.3	2030	N/A
VT, 64S	Newbury	I-91 (SB)	1973	Protecto Wrap M 400	128.0	2	45.0	36.0	7.3	2030	N/A
VT, 71S	Barnet	I-91 (SB)	1977	Royston 10	113.0	2	44.0	36.0	8.9	1745	11
VT, 66	Hardwick	VT-15	1984	Royston 10-A	92.0	2	42.0	36.0	9.5	4730	5
VT, 71	Hartford	US 4	1986	Royston 10-A	114.0	2	46.0	38.0	9.0	4300	N/A

Note: °F = 1.8 x °C + 32; 1 in. = 25.4 mm; 1 ton/lane-mi/yr = 565 kg/lane-km/yr (approximately).
*Same structure as Royston 10.

membrane identification in Maine (mostly not brand names). Therefore, identification of the membranes was obtained during the testing. In general, the three types of membranes evaluated in the laboratory were evaluated in the field.

Test Procedure and Sampling Plan

The V-meter measurements were taken at locations where the questionnaire analysis indicated that the membrane would most likely be damaged and deteriorated. Also, other locations at each bridge were chosen where the membrane was expected to perform well or areas where cracks were present, to study the effects of cracked surfaces on membrane.

A statistically based sampling plan for pulse velocity testing, based on the questionnaire analysis, was developed. The plan was divided into two categories: longitudinal bridge deck plan and transverse bridge deck plan. The V-meter measurements in the longitudinal bridge deck plan were taken as follows: 18 percent of the measurements were on the shoulder area and 82 percent on the lanes. If the bridge had two lanes in the same direction, then 65 percent of the lane measurements were on the slow lane and 35 percent were on the fast lane. Twenty-five percent of the total measurements in the longitudinal bridge deck plan were taken on the entrance and exiting span areas. The sampling plan for the transverse direction was as follows: 30 percent of the measurements were on the wheel paths, 10 percent on the lane center, 20 percent between the lane center and wheel paths, 10 percent between outer wheel path and lane edge, 5 percent at the center line, if available, and 25 percent were decided in the field, for each individual bridge deck, considering locations of interest where wearing surface problems were manifested. Ground truth cores were extracted to verify the V-meter measurements. Twenty percent of the cores were extracted where the V-meter measurements indicated good membrane status, i.e., high V-meter readings (if available). The extracted cores were distributed between longitudinal and transverse plan locations as possible. Cores also had to be extracted at locations where the wearing surface of the bridge deck was deteriorated, although, in some cases, the V-meter measurements were impossible.

An average of 45 ultrasonic pulse velocity (V-meter) measurements were obtained from each bridge deck. An average of 14 cores were obtained from each bridge (ground truth) to verify the V-meter readings and to study membrane performance. Membranes were removed from coring locations and evaluated. The same rating procedure used in the laboratory investigation was applied in this part of the investigation. Figures 5-1 and 5-2 show the pulse velocity instrumentation setup at one of the test sites.

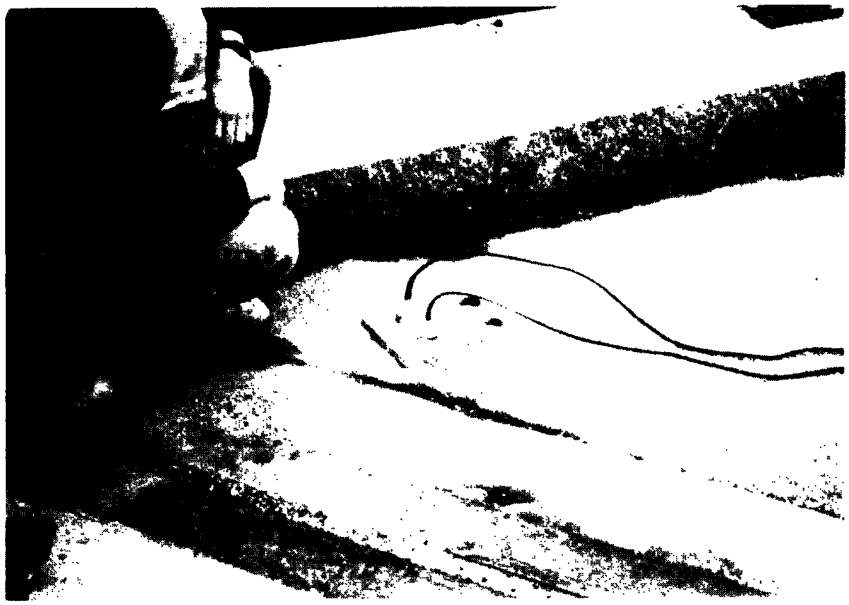


Figure 5-1. Transducer array in place for measuring pulse velocity on a bridge deck.

Figure 5-2. Pulse velocity instrumentation being operated from the back of a van at a test site.

Field Validation in New Hampshire

Five bridges were chosen in New Hampshire for field validation. They are located in Stratford, Tilton, Concord, and Hillsborough. The bridge in Stratford is a two-way, two-lane bridge with a very low AADT. It was constructed in 1984. A Royston 10-A membrane was installed with an average hot-mix asphalt overlay thickness of 1.6 in. (4.1 cm). Two bridges on Interstate 93 were evaluated in Northfield-Tilton. One carries the southbound lanes and the other, the northbound. Both are one-way, two-lane bridges constructed in 1978. The membrane installed on both is Royston 10-A. The hot-mix asphalt overlay is 2.3 in. (5.8 cm) thick (average). The bridges are on a superelevation of 8 percent. The Concord Bridge is on Route 9 over I-393 (ramp) constructed in 1988. It is a two-way, two-lane bridge with a Royston 10-A membrane. The average hot-mix asphalt overlay thickness is 2.3 in. (5.8 cm). The Hillsborough Bridge is on US 202 leading to an intersection. It is a two-way, three-lane with a Bituthene membrane, and it was constructed in 1988. The average overlay depth is 1.75 in. (4.4 cm). The overlays on all the bridge decks, except the Hillsborough Bridge, were in good condition. Although the Hillsborough Bridge was constructed recently, it showed major rutting and shoving problems at the exiting end of the bridge deck. During the tests, it was noticed that the deteriorated and distressed part of the overlay is at the braking area where the affected lane is used by heavily loaded trucks.

In general, the V-meter measurements indicated that the membrane systems had performed satisfactorily, except for the exiting span area in Hillsborough Bridge. Although the bridge was constructed recently, the V-meter measurement indicated more defects in the membrane system than with the other bridges. This was proven later during coring. This unsatisfactory performance was highly affected by the deteriorated and distressed overlay; in some areas, the overlay thickness is found to be only 0.75 in. thick.

Cores were obtained from several locations on each bridge, and the membranes were evaluated. A statistical analysis was performed to investigate the relationship between the V-meter measurements and the membrane rating. The following model was obtained:

$$\text{MemRate} = -5.88 + 0.602 \text{ V-meter} \qquad (5\text{-}1)$$

where

MemRate = the membrane rating
V-meter = the V-meter measurement in μs

A coefficient of determination (R^2) value of 87.5 percent obtained here indicates a strong correlation between the V-meter measurements and the membrane rating. The root mean square error value of 0.539 also indicates a good correlation between the variables. A two-tail t-test indicates that the two coefficients of the model are significant at the 95-percent level or better. The 95-percent confidence interval is presented in Figure 5-3.

Powdered concrete samples were obtained at different locations at three depths: 0 to 1/2, 1/2 to 1, and 1 to 1 1/2 in. (0 to 13, 13 to 25, and 25 to 38 mm). The chloride content was determined for each sample. The chloride content was high in all bridges at the entrance and exiting span areas, inner wheel path, outer wheel path, and shoulder. A regression model was used to identify the factors that influence chloride contamination. The variables include the sample location, the membrane type, the membrane rating, the AADT, the annual snowfall, and total salt application. The correlation coefficients between chloride content at Depths 1, 2, and 3 and the independent variables are 79.9 percent, 88.1 percent, and 81.1 percent, respectively. The membrane rating and location were the most important factors with lowest standard error for each coefficient of the regression model. The ADDT was found to be the least important.

The chloride contents in the Tilton bridges were considered high. The membrane was found to have very low bonding to the concrete in these two bridge decks compared to the other evaluated bridge decks. The membrane in the Hillsborough Bridge generally performed well, and the bonding to both layers was satisfactory except in the area where rutting had occurred. There, membrane performance was unsatisfactory. Some of the rutted areas were cracked, although the cracks did not propagate through the entire asphalt thickness. However, cracks were observed in the membrane systems oriented in the same direction as the cracks on the surface. The membrane in the Stratford Bridge performed well, and the bonding to both layers was satisfactory. However, in some locations, bonding to the concrete surface was poor due to installation of the membrane on dirty surfaces. The membrane in the Concord Bridge generally performed well. The poorest membrane performance on the bridge deck was noticed at the shoulder area where many of the membrane samples were deteriorated or had small holes in them.

Tables A-1 through A-5 in Appendix A, pp. 92-107, present the pulse velocity, membrane rating, and chloride content data for each of the New Hampshire bridges.

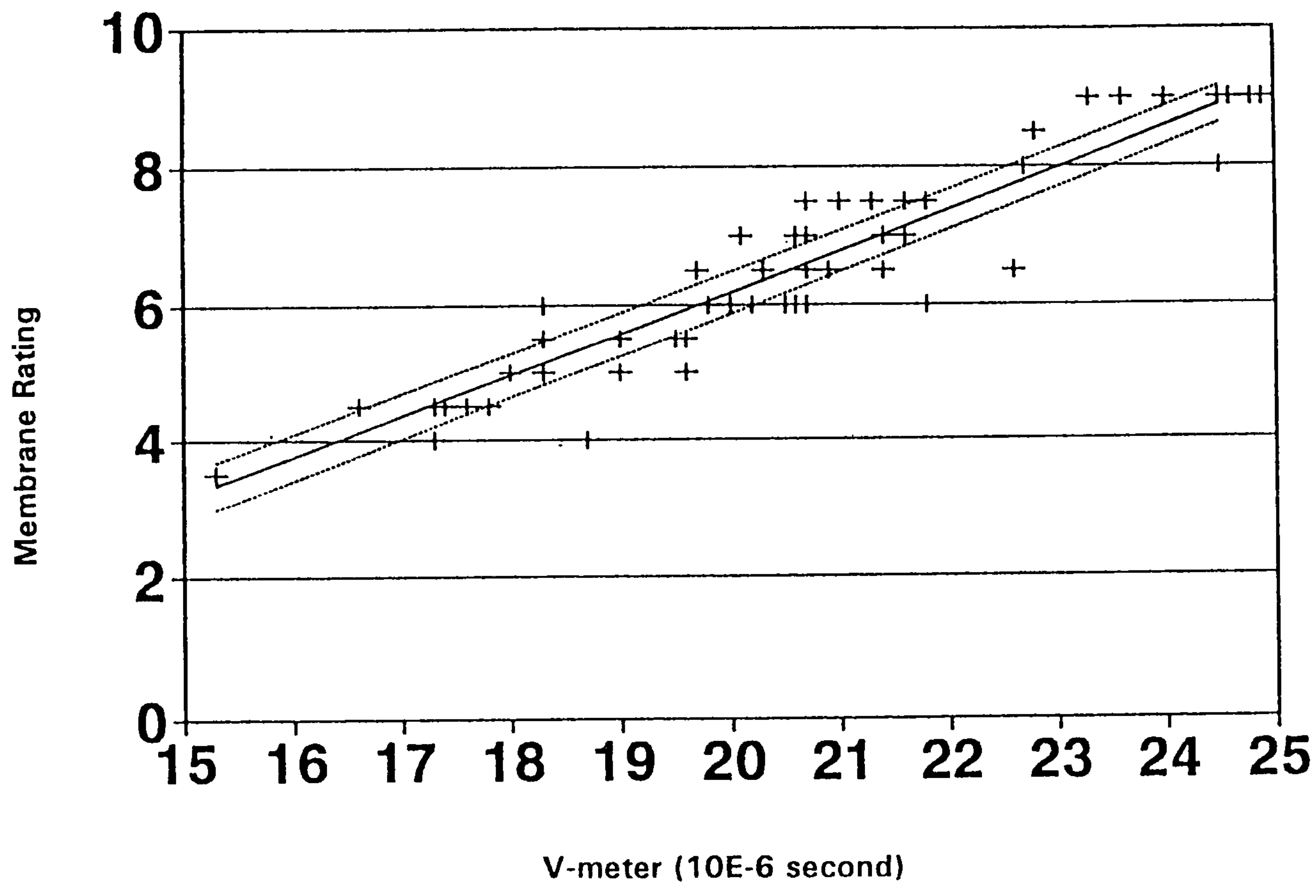


Figure 5-3. The membrane rating model with 95-percent confidence interval for New Hampshire bridges.

Field Validation in Maine

The five selected bridges are distributed geographically through Maine. They are located in Caribou, Beddeford, Augusta, Brewer, and South Portland. The bridges were constructed between 1959 and 1989. The AADT varies from 5,000 to more than 15,000 vehicles. The three types of membranes investigated in the laboratory also were evaluated in the five bridges in Maine. The bridge in Caribou was constructed in 1971, and a Royston 10-type membrane was installed on the deck. The average hot-mix asphalt overlay thickness was found to be 1.2 in. (3.0 cm), but at more than one location it was less than 0.75 in. (1.9 cm). The bridge is a two-lane, two-way bridge. The Brewer Bridge was constructed in 1986 and has a Royston 10 membrane installed on its deck. The average hot-mix asphalt overlay thickness is 3.1 in. (7.9 cm). The bridge is two-lane, two-way. The Augusta Bridge is ramp F, which was constructed in 1959 and has two lanes in a two-way configuration. The membrane installed was undefined by the Maine Department of Transportation. However, during the evaluation stage, it was determined to be the same type as a Royston 10 membrane. The average hot-mix asphalt overlay thickness is 2.6 in. (6.6 cm). The Biddeford Bridge is a new bridge constructed in 1989 with two lanes in two-way directions. A Bituthene membrane was installed. The average hot-mix asphalt overlay thickness was found to be 3.2 in. (8.1 cm). The South Portland Bridge is a connection of two ramps forming two lanes in the same direction. The bridge has a high AADT of 15,600 vehicles. The bridge was constructed in 1974 and has a Protecto Wrap M400 membrane. The average hot-mix asphalt overlay thickness is 2.2 in. (5.6 cm).

The Caribou Bridge showed evidence of rutting in the wheel paths throughout the bridge. Cracks had developed in or near the distressed areas. The Brewer Bridge has experienced rutting in some locations. However it was less pronounced than the Caribou Bridge. The Augusta Bridge showed many longitudinal and transverse cracks in the asphaltic overlay. The cracks were deep in some locations and propagated through the membrane. At the beginning, it was thought that some of the cracks were reflection cracks, but after recovering the membrane, it was noticed that the cracks propagated from the surface. In general, the asphaltic overlay on this bridge was in unsatisfactory condition. The Biddeford Bridge, which is a new bridge, was in very good condition. When the cores were removed from that bridge it was noticed that water accumulated in the core holes, which indicated that the membrane was performing well as a chloride barrier. The South Portland Bridge had some cracks and distressed locations. However, its condition was satisfactory. In general, the asphaltic overlays in the bridges evaluated in Maine were in relatively satisfactory condition, except for the overlay in the Augusta Bridge.

The V-meter measurements indicated that membrane systems were, in general, in a satisfactory condition except for the Augusta Bridge. The severely deteriorated overlay in the Augusta Bridge led to the deterioration of the membrane. In two tested locations in the Augusta Bridge, the V-meter measurement indicated a possible nonexistence of membrane. When the cores were obtained, no membrane was found in these locations. This may have occurred during the placing of the bituminous concrete overlay. In the Caribou Bridge, the membrane was more brittle than in the other bridges. However, it was, in most cases, intact. The membrane was found to be in unsatisfactory condition in some locations at the entrance span area of the Biddeford Bridge. The asphaltic overlay at these locations had a very low asphalt content and, therefore, no bonding existed between the bituminous concrete and the membrane.

The V-meter measurements were paired with the membrane ratings. The following statistical model was obtained:

$$\text{MemRate} = -6.31 + 0.624 \text{ V-meter} \tag{5-2}$$

where

MemRate = the membrane rating
V-meter = the V-meter measurement in μs

A coefficient of determination (R^2) value of 91.6 percent was obtained, indicating a good correlation between the V-meter measurements and the membrane rating. Also, a root mean square error of 0.447 supports this conclusion. A two-tail 95-percent significance t-test indicated that the two coefficients of the model are significant. The 95-percent confidence interval is presented in Figure 5-4.

Powdered concrete samples were also obtained from test locations in the five Maine Bridge decks at depths of: 0 to 1/2, 1/2 to 1, and 1 to 1 1/2 in. (0 to 13, 13 to 25, and 25 to 38 mm). The chloride content was low in all bridges, except for the Augusta Bridge which is 32 years old. By extrapolation, the Augusta Bridge is the only bridge deck where active corrosion is expected. The inner and outer wheel path and entrance and exiting span area locations have higher chloride contents compared to other locations. The least chloride contaminated locations are the lane centers. The chloride contents of the samples obtained from the shoulders were relatively lower than for other locations, which is in contradiction to what was found in New Hampshire.

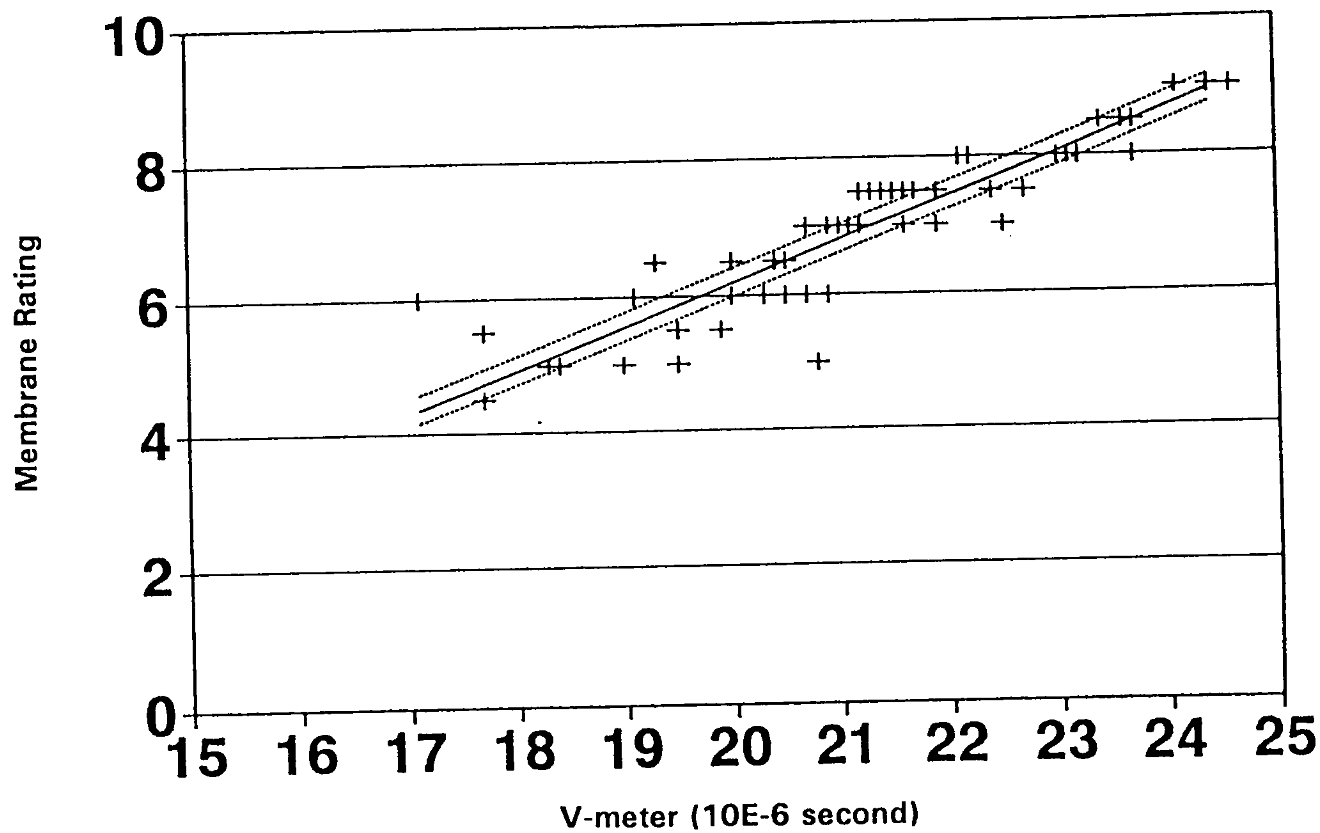


Figure 5-4. The membrane rating model with 95-percent confidence interval for Maine bridges.

A regression model was developed to identify the factors that influence the degree of chloride contamination. The variables include the sample location, the membrane type, the membrane rating, the AADT, the annual snowfall, and the total salt application. The correlations between the chloride contents at Depths 1, 2, and 3 and the independent variables are 84.5 percent, 78.7 percent, and 78.3 percent, respectively. The most important factor was the membrane type followed by the amount of annual snowfall and salt application rate.

The deteriorated condition of the overlay on the Augusta Bridge is considered a major factor in the unsatisfactory performance of the membrane. Considering the amount of salt application (see Table 5-3), the membrane systems are performing well in reducing the chloride content.

Tables A-6 through A-10, pp. 106-121 in Appendix A, present the pulse velocity, membrane rating, and chloride content data for each of the Maine bridges.

Field Validation in Vermont

The bridges chosen in Vermont are located in Newbury, Barnet, Hardwick, and Hartford. The bridges were constructed between 1973 and 1986. The AADT varies between 1,750 and 4,750. The two Newbury bridges were constructed in 1973. The northbound bridge has a Protecto Wrap M400 membrane, while the southbound bridge has a Bituthene membrane. The average overlay thickness in the southbound deck is 2.3 in. (5.8 cm), while the northbound is 2.4 in. (6.1 cm). Both bridges are two-lane, one-way direction. The Barnet Bridge was constructed in 1977 and has a Royston 10 membrane. The average overlay thickness is 2.5 in. (6.4 cm). The bridge is a two-lane, one-way direction. The Hardwick Bridge was constructed in 1984 and has a Royston 10-A membrane. It has an overlay thickness of 3.1 in. (7.9 cm). It is a two-lane, two-way bridge. The Hartford Bridge was constructed in 1986, and it has a Royston 10-A membrane. It has an overlay thickness of 2.6 in. (6.6 cm). It is a two-lane, two-way bridge.

The two Newbury bridges had some localized cracks. Other than the cracks, the overlay was in satisfactory condition. The Barnet Bridge was rutted throughout. The overlay of the Hardwick Bridge was in a good condition. However, the most recently constructed bridge among the bridges evaluated in Vermont, the Hartford Bridge, had severe shoving problems at one of the exiting span areas. The deteriorated area had been patched unsuccessfully. Some cracks were noticed in the same span also. The overlay condition on the evaluated bridges in Vermont was better than that in New Hampshire and Maine.

The bonding of the membrane in the Newbury Bridge (southbound) to the concrete surface was found to be satisfactory; however, the bonding to the asphaltic layer was found to be low. The bonding of the membrane to the concrete surface of the Newbury Bridge deck (northbound) was found to be less than that of the southbound, and the bonding to the asphaltic layer was also low. In the Barnet Bridge, the membrane was bonded to the asphaltic layer better than to the concrete surface. The bonding of the membrane on the Hardwick Bridge varied from one location to another. In many locations, the membrane was found to have some holes in it. Some of the overlay cores showed stripped asphaltic material. The membrane bonding, in the Hartford Bridge, was better to the asphaltic layer than the concrete surface. The deteriorated overlay locations showed no bonding between the membrane and the concrete surface.

The V-meter measurements indicated satisfactory membrane condition, except in cracked locations where the membrane was also found to be cracked even though the cracks did not appear, in many cases, to penetrate through the asphaltic concrete layer. The V-meter measurements were correlated with the evaluated membranes. The following model was obtained:

$$\text{MemRate} = -9.99 + 0.781 \text{ V-meter} \tag{5-3}$$

where

MemRate = the membrane rating
V-meter = the V-meter measurement in μs

A coefficient of determination (R^2) value of 96.1 percent indicates a good correlation between the V-meter measurements and the membrane rating. The root mean square error was found to be 0.316, which supports this conclusion. The two-tail 95-percent significant t-test indicates that the model's two coefficients are significant. The 95-percent confidence interval is presented in Figure 5-5. In general, the model developed from Vermont's data gave the strongest correlation among all the evaluated bridge decks.

Powdered concrete samples were obtained from the five evaluated bridges decks in Vermont at three depths: 0 to 1/2, 1/2 to 1, and 1 to 1 1/2 in. (0 to 13, 13 to 25, and 25 to 38 mm). The most highly chloride-contaminated samples were found, in general, in the inner and outer wheel paths, shoulders, and entrance and exit span areas.

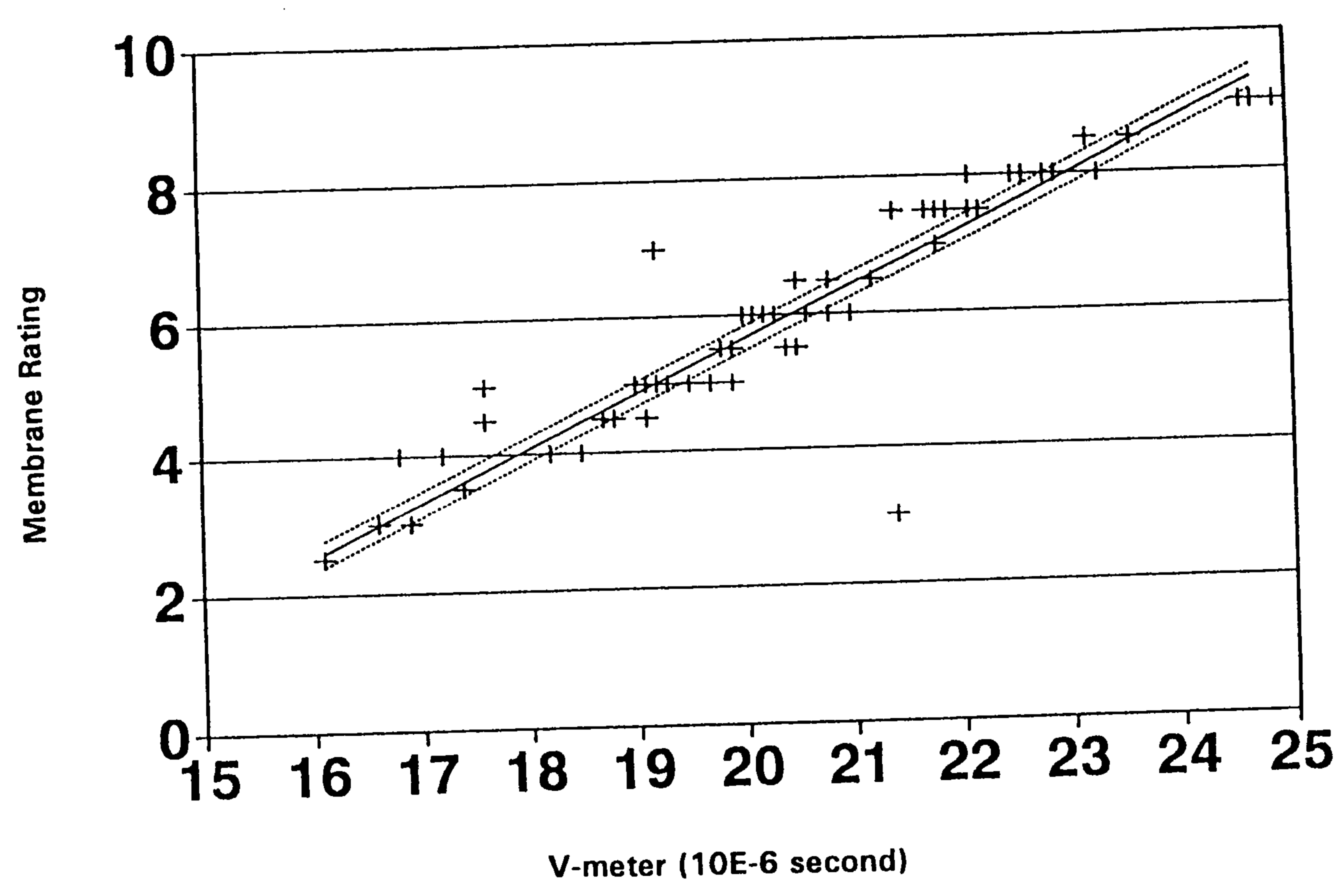


Figure 5-5. The membrane rating model with 95-percent confidence interval for Vermont bridges.

The chloride content in the Barnet Bridge was higher than anticipated. This bridge is exposed to more salt applications per year than the Newbury bridges. In general, except for the Barnet Bridge, the membrane performance in Vermont was satisfactory, and none of the other four bridges has yet reached the chloride corrosion threshold point. The Newbury bridges, which are 18 yr old, clearly indicate that a membrane system is a good corrosion protective technique.

Tables A-11 through A-15, pp. 122-137 in Appendix A, present the pulse velocity, membrane rating, and chloride content data for each of the Vermont bridges.

General Discussion

Field Model

In order to evaluate the overall effectiveness of the V-meter measurements in correlating with the membrane rating, a statistical analysis was conducted considering all the V-meter measurements and the membrane evaluation results (ratings) for the 15 bridges in the 3 states. The following statistical model was developed:

$$\text{MemRate} = -7.39 + 0.668 \text{ V-meter} \qquad (5\text{-}4)$$

where

MemRate = the membrane rating
V-meter = the V-meter measurement in μs

The coefficient of determination (R^2) value of 89.3 percent obtained in this analysis indicates a strong correlations between the V-meter measurements and the membrane rating for different states and different membrane types. The root mean square error is found to be 0.520, which supports this conclusion. The two-tail 95-percent significance t-test indicates that the two coefficients of the developed model are significant. The 95-percent confidence interval is presented in Figure 5-6. A statistical analysis was performed to identify the factors that influence the degree of chloride contamination of the concrete. The independent variables include sample location, membrane type, membrane rating, AADT, annual snowfall, and total salt application. The correlations between the chloride content at Depths

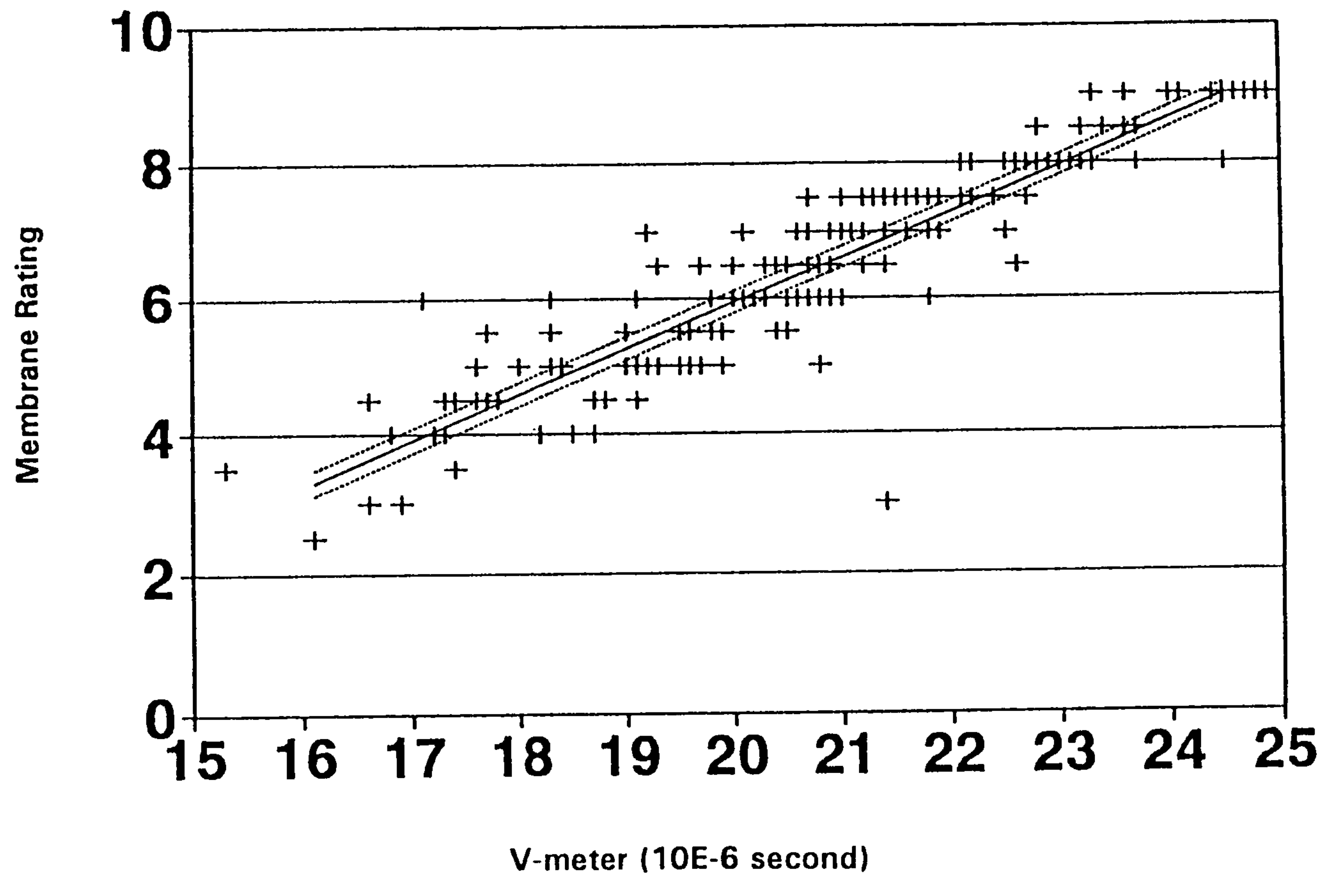


Figure 5-6. The membrane rating model with 95-percent confidence interval for all of the New England states.

1, 2, and 3 and the independent variables are 71.1 percent, 72.6 percent, and 65.9 percent, respectively. The amount of annual snow was found to be the most important factor. The AADT is the least important.

Field/Laboratory Model

Another model was considered for investigation in which all of the results from the laboratory outdoor slabs and the bridges are included. The model is as follows:

$$\text{MemRate} = -8.05 + 0.696\ \text{V-meter} \tag{5-5}$$

where

MemRate = the membrane rating
V-meter = the V-meter measurement in μs

The coefficient of determination (R^2) value of 88.2 percent obtained in this analysis indicates a strong correlation between the V-meter measurement and the membrane rating for all tested locations. The root mean square error was found to be 0.520, which supports the above conclusion. The two-tail 95-percent significance t-test indicates that the two coefficients of the developed model are significant. The 95-percent confidence interval is presented in Figure 5-7.

The latter two models (equations 5-4 and 5-5) give comparable results at similar confidence levels. However, due to the fact that the data in the first model, equation 5-4, was obtained under service conditions, while some of the data in the second model was obtained under controlled laboratory conditions, it is suggested that the former be used to evaluate membrane status from V-meter measurements in the field.

General Recommendations on Membrane Performance

The membrane performance, in general, in the evaluated bridge decks is considered satisfactory. The major problem noticed in the field is the bonding of the membrane to both layers (see Figures 5-8 and 5-9). Losing some of the bonding property of the membrane will lead to increased water and chloride intrusion. In order to preserve good bonding with the

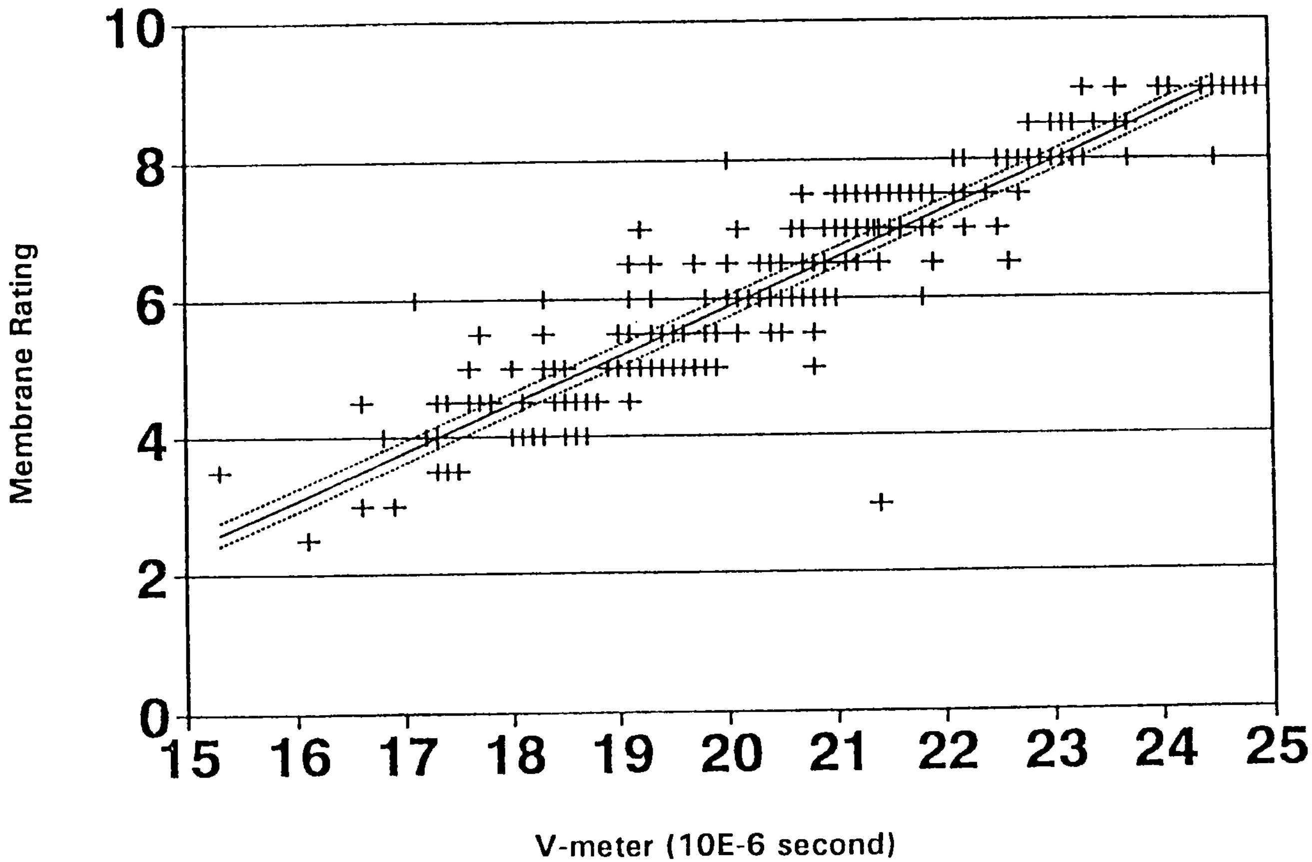


Figure 5-7. The membrane rating model with 95-percent confidence interval slab and bridge data.

Figure 5-8. Lack of bond between the membrane and the concrete bridge deck.

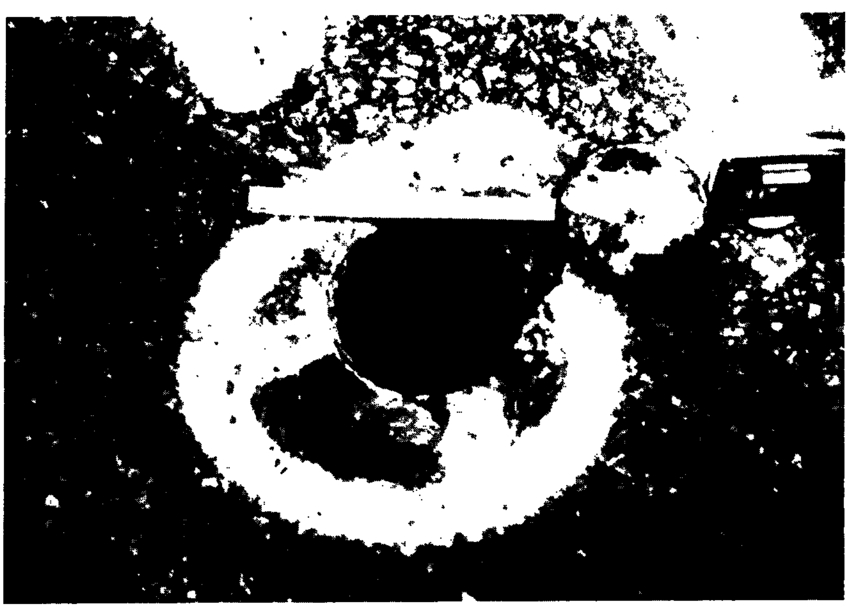


Figure 5-9. Lack of bond between the membrane and the asphaltic overlay.

concrete surface, it should be free from any dirt and possess acceptable roughness. The use of the correct method of installing membranes is a major factor in developing a proper bond. During the membrane installation, the overlapped and the edge areas should be sealed well to prevent any water and chloride intrusion. No construction equipment should be allowed to operate over an unprotected membrane.

The hot-mix overlay should be placed over the membrane as soon as possible. The membrane used should be able to withstand the laydown temperature of the hot-mix asphalt without flowing. The bridge should not be open to traffic unless the asphaltic material is completely set. The hot-mix asphalt overlay should be kept in satisfactory condition. Any deterioration of the overlay affects the membrane performance. It has been observed during the study that a cracked overlay surface results in a cracked membrane in most cases (see Figure 5-10). In some locations, where rutting was observed and the overlay depth was less than 1 in. (2.5 cm), the membrane was severely deteriorated (see Figure 5-11).

The overlay depth during service should not be allowed to reach less than 2.0 in. (5.1 cm). Therefore, the hot-mix asphalt overlay should have a depth of at least 2.5 in. (6.4 cm). This will allow for preserving the membrane if rutting should occur or if milling of the asphaltic wearing course is required for rehabilitation purposes, since membrane life is usually much greater than that of the overlay. The overlay should allow the water to flow to the drainage system and not accumulate above the membrane; this will lead to stripping problems in the overlay and reduce the bonding between the membrane and the overlay. The locations where the membrane is expected to be deteriorated first, in addition to the distressed overlay surface, are in the wheel paths and shoulders, especially at the entrance and exit areas.

Period Needed to Initiate Corrosion

The average chloride content for each bridge at the three evaluated depths is presented in Table 5-4. Although it is believed that corrosion may initiate in the steel rebars when the chloride content in the concrete reaches a level of 1.2 lb/yd^3 (0.7 kg/m^3), this may not be the case in protected bridges. To initiate corrosion, the availability of oxygen and water is essential. The membrane system is believed to reduce these two factors, since it was observed in many bridges where asphaltic cores were removed, that water rapidly filled the core holes. Therefore, it is suggested that corrosion will initiate at a higher chloride content for the protected membrane. This suggestion is supported by the chloride analysis of the powdered concrete sample obtained from position 17 of the South Portland Bridge in Maine (see Table A-7, p. 109, in the Appendix). The reinforcing bar was exposed at that position

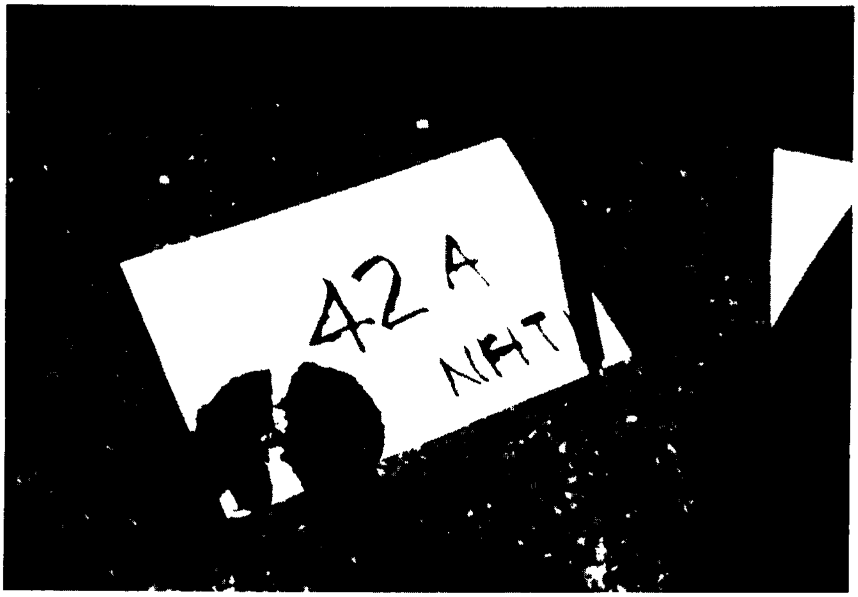


Figure 5-10. Crack through asphaltic overlay reflected through membrane.

Figure 5-11. Deteriorated membrane from rutted area.

Table 5-4. Average chloride content in evaluated bridges.

State, Bridge No.	Avg. Cl (1) (lb/yd^3)	Avg. Cl (2) (lb/yd^3)	Avg. Cl (3) (lb/yd^3)
NH, 107/074	0.637	0.478	0.218
NH, 117/157	4.513	3.933	3.163
NH, 118/158	5.271	5.336	4.870
NH, 117/035	0.030	0.030	0.030
NH, 165/177	0.069	0.044	0.030
ME, 6255	0.237	0.092	0.056
ME, 1376	0.778	0.643	0.592
ME, 5807	4.270	3.655	2.955
ME, 822	0.030	0.030	0.030
ME, 1562	0.497	0.259	0.275
VT, 64N	0.122	0.083	0.052
VT, 64S	0.098	0.122	0.105
VT, 71S	1.799	1.309	0.890
VT, 66	0.128	0.030	0.036
VT, 71	0.171	0.155	0.158

Note: 1 lb/yd^3 = 0.59 kg/m^3.

and was found to be in good condition even though the chloride ion content at the rebar level was 2.3 lb/yd^3 (1.36 kg/m^3). Of the bridges evaluated, the only ones that are expected to be actively corroding are the two Tilton bridges in New Hampshire and the Augusta bridge in Maine. The high chloride contents in Tilton bridges is unexpected, especially since many bridges older than these showed much lower chloride contents. The Augusta Bridge is 32 yr old. To study the effect of selected independent variables on the amount of chloride content in the bridge, a statistical analysis was performed. The chloride contents at the three depths were considered the dependent variables, one variable at a time. The bridge deck age (same as membrane age), membrane type, AADT, total amount of snow, and total amount of salt applied were considered the independent variables. Other variables were found insignificant

and do not affect any statistical model. The correlation coefficients obtained for the chloride contents at Depths 1, 2, and 3 and the independent variables are 95.9 percent, 96.2 percent, and 95.5 percent, respectively (R^2 = 92.0 percent, 92.5 percent, and 91.2 percent, respectively). The cumulative amount of salt used is the most important factor. The statistical model for chloride content at Depth 2 (average depth of .75 in. [1.9 cm]) is presented below:

$$\begin{aligned} \text{Cl}(2) \quad = \quad & -1.28 - 0.164\,(\text{Age}) + 0.119\,(\text{Membrane Type}) \\ & + 0.000163\,(\text{AADT}) + 0.0278\,(\text{Total Salt}) \\ & + 0.000561\,(\text{Total Snow}) \end{aligned} \tag{5-6}$$

where Cl (2) is the chloride content in lb/yd^3 (1 lb/yd^3 = 0.59 kg/m^3); age is in years; membrane type is 1 for Bituthene, 2 for Royston, and 3 for Protecto Wrap; total snow is in inches (1 in. = 2.54 cm); and total salt is in tons/lane-mi (note: 1 ton/lane-mi = 565 kg/lane-km). Notice that the snowfall and salt use are cumulative values over the age of the deck. The coefficient of determination (R^2) for the above model is 92.5 percent and the root mean square error is 0.547. This indicates a good correlation between the dependent and independent variables. This model is based on the average chloride contents in each bridge.

If it is assumed that the chloride content needed to initiate corrosion at a rebar in a bridge deck with a 2-in. (5.1-cm) concrete cover is 5 lb/yd^3 (2.95 kg/m^3) at 0.75 in. (1.9 cm) from the surface, and that the average annual snowfall is 60 in., the AADT is 5,000, and the salt application is 9 tons/lane-mi/yr (5.1 mt/lane-km/yr), then the corrosion will start after 43 to 45 yr, depending on which membrane is used. This would appear to provide, as a conservative estimate, about 25 yr of additional service life in contrast to unprotected bridge decks.

The effect of salt application, snowfall, membrane type, and AADT on the chloride content-age relationship was investigated, using the model represented by equation 5-6. The results are presented in Figures 5-12 through 5-15, respectively. The following information was used as a reference to variables that are constant for a specific variable effect: snowfall is 60 in./yr (1.52 m/yr), membrane type is Bituthene, salt application is 9 tons/lane-mi/yr (5.1 mt/lane-km/yr), and AADT is 5,000. Figure 5-12 indicates that salt application is the most pronounced variable while the effect of membrane type (Figure 5-14) is minimal, confirming the results of the laboratory tests. Using equation 5-6 or Figures 5-12 through 5-15, one can estimate the time when corrosion may initiate for a specific bridge deck, based on corrosion threshold chloride concentrations.

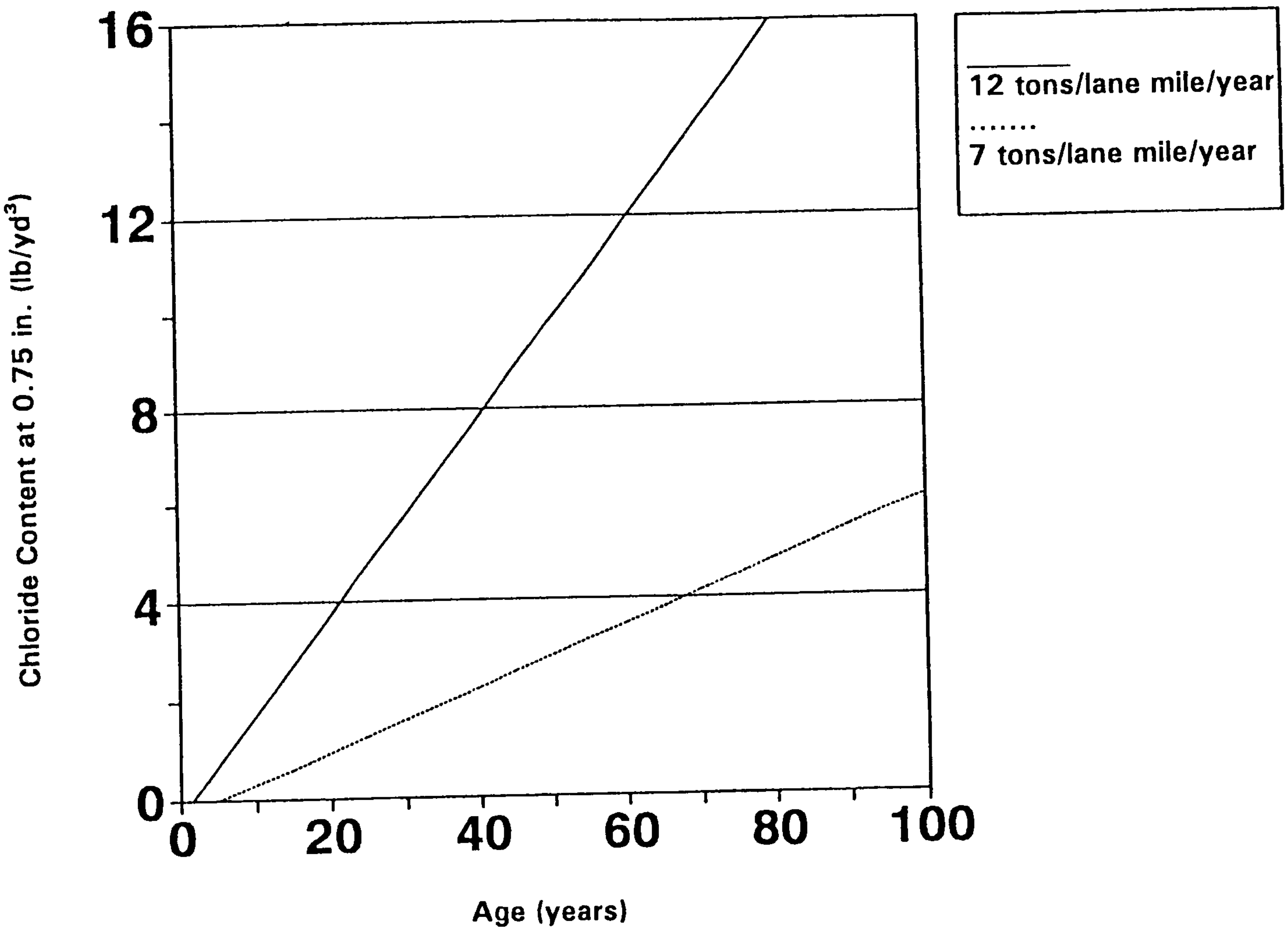


Figure 5-12. Effect of salt application on chloride content-age relationship.

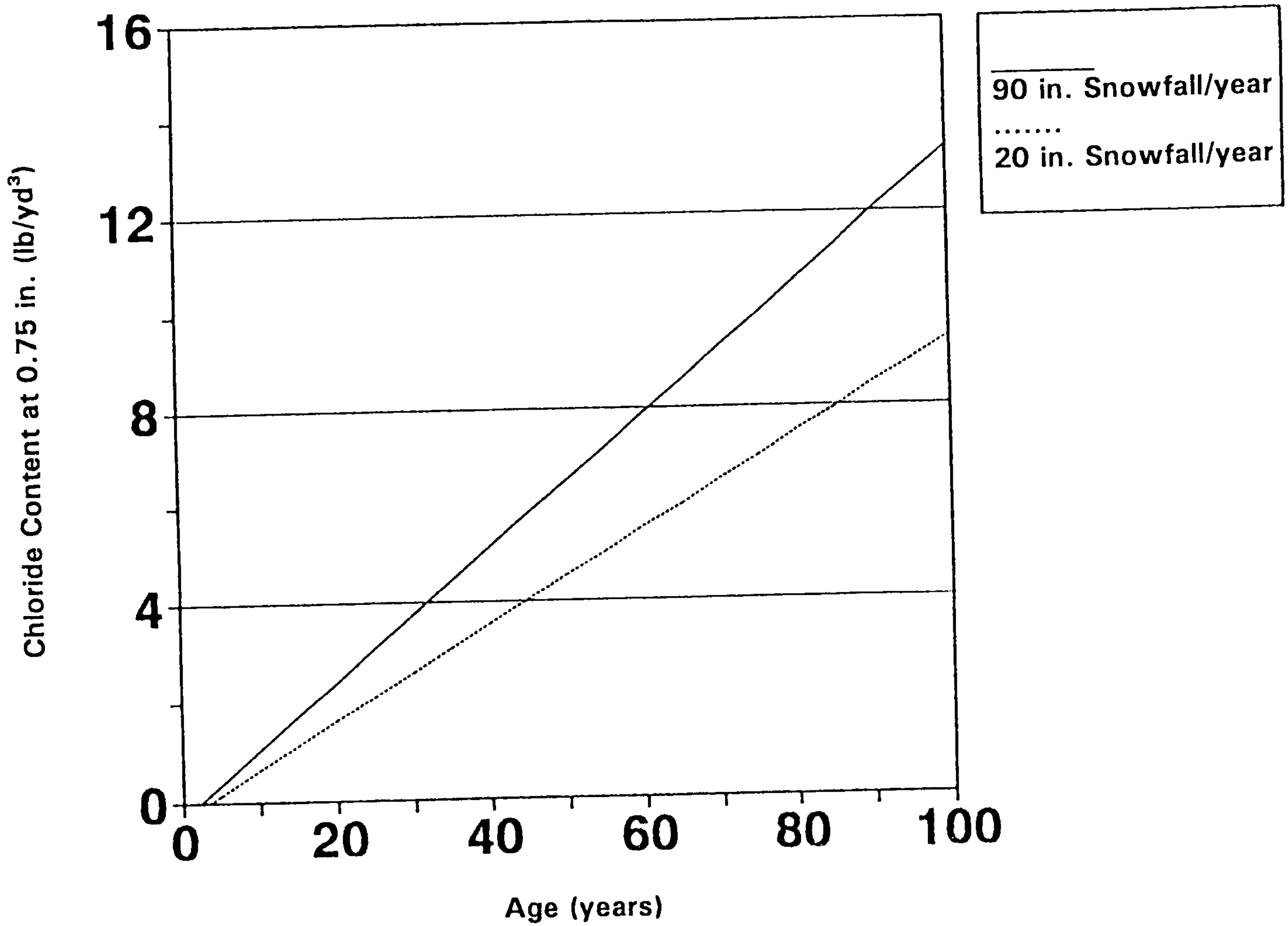


Figure 5-13. Effect of snowfall on chloride content-age relationship.

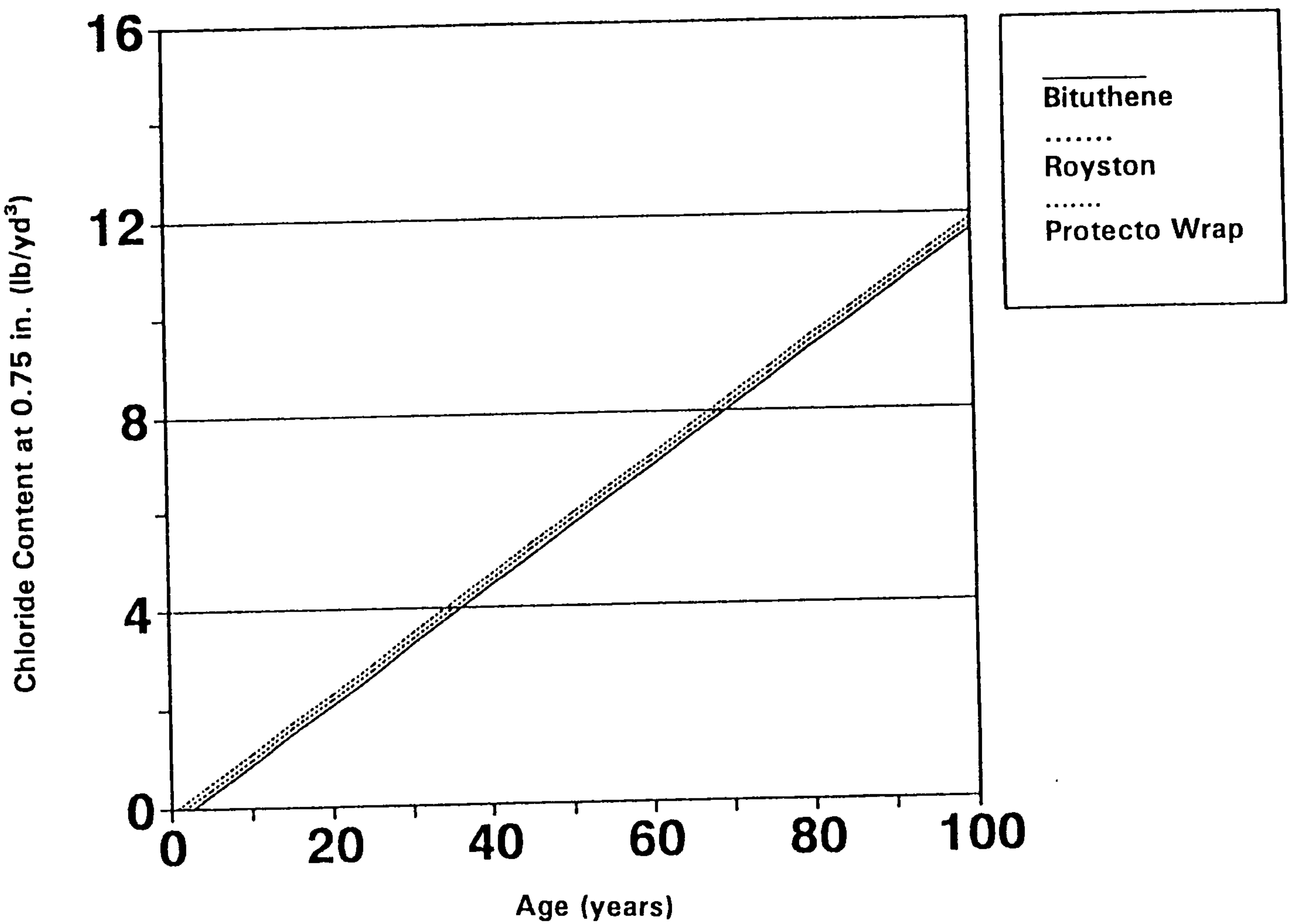


Figure 5-14. Effects of membrane type on chloride content-age relationship.

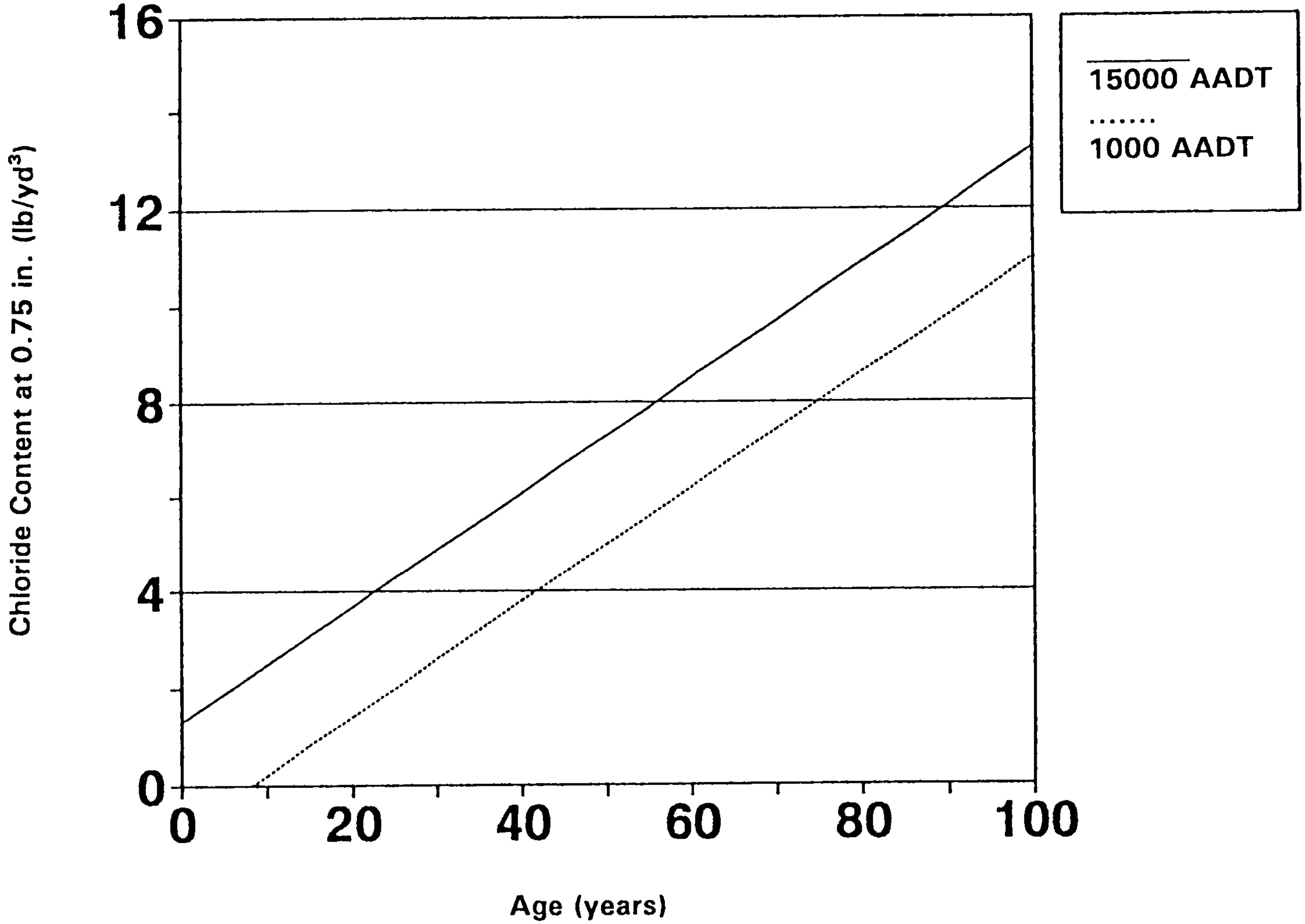


Figure 5-15. Effect of AADT on chloride content-age relationship.

6

Findings and Conclusions

To investigate the integrity and effectiveness of membrane systems, a comprehensive laboratory investigation was conducted. Three types of preformed membranes were studied by installing them on slabs simulating a typical bridge deck. The membranes were perforated using different hole sizes and frequencies or percent of perforations per unit area. The slabs were overlaid with hot-mix asphalt and exposed to deicing salt equal to the average used in the New England states over two winters. Electrochemical potential drops between the top and bottom rebar mats and temperatures in the slabs were monitored throughout the study. The effect of temperature on membrane systems was also investigated.

An evaluation of nondestructive testing methods was performed. The ultrasonic pulse velocity system (V-meter) was selected and gave repeatable and consistent measurements in detecting defects in the membrane system. V-meter measurements were strongly correlated with membrane status, and a statistical model was developed to predict the membrane status from the V-meter measurements. Concrete samples were obtained from two locations in the slabs. The chloride contents of the concrete samples were evaluated at three depths: 0 to 1/2, 1/2 to 1, and 1 to 1 1/2 in. (0 to 13, 13 to 25, and 25 to 38 mm).

A survey determined that 22 states use membranes as protective systems. A questionnaire was sent to these 22 states to investigate the major factors affecting the membrane performance and the location of the areas where the membrane is at high risk of deterioration. Relative to service factors, the survey results indicate that the areas where membranes are most seriously exposed to deterioration are acceleration and braking areas, turning areas, wheel paths, curb and gutter zones, drainage areas, bridge curvatures and

slopes, and areas with high ADT. Temperature and deicing chemical application rate are considered to be the most important environmental factors. Construction factors that are reported to be most critical are the degrees of bonding of the membrane to the bridge deck surface and to the asphalt overlay. Fifteen bridges were evaluated in New Hampshire, Maine, and Vermont. An average of 45 ultrasonic pulse velocity measurements was taken at each bridge and an average of 14 cores was obtained for each deck. A strong correlation was found between the V-meter measurements and the membrane status. A statistical model was developed to predict the membrane status from the V-meter measurements. Concrete samples also were obtained from each bridge at three depths: 0 to 1/2, 1/2 to 1, and 1 to 1 1/2 in. (0 to 13, 13 to 25, and 25 to 38 mm). The chloride contents for these samples were determined. A statistical analysis was performed to study the major factors affecting the amount of chloride contamination.

Findings

Temperature was found to have an effect on the membrane sheets. At 220 to 275°F, the Bituthene 5000 membrane became semi-solid and started to flow. The Protecto Wrap M400 membrane experienced some shrinkage at the edges. Royston 10-A performed the best. However, since the temperature is usually applied to only one surface of the membrane, Marshall specimens were compacted on one side over membrane sheets with various hole sizes, frequencies, and shapes. Bituthene 5000 membrane showed an increase in the hole size for an initial 3/8-in. (10-mm) opening. Royston 10-A showed an increase in the hole size at a 3/8-in. (10-mm) wide strip cut. However, the holes in the Protecto Wrap M400 membrane did not change in size or shape.

Ultrasonic pulse velocity was able to detect defects in the membrane systems, while pulsed radar and infrared thermography failed to identify the presence of the membrane in the overlaid slabs. A statistical model was developed correlating the ultrasonic pulse velocity with the membrane status for the indoor slabs with a correlation coefficient of 92.7 percent (R^2 = 85.9 percent).

A major difference was noticed in the chloride contents between the protected and unprotected slabs. The unprotected slabs experienced higher chloride contents compared to the protected slabs. The chloride content results indicate that chlorides were higher at the overlapping location when Protecto Wrap membrane was used, which reflects its lower bonding characteristics; while the Royston 10-A membrane showed a high bonding property and resulted in less chlorides at the overlapping locations. The hole size of 1/4 in. (6 mm) was observed to be the critical size. Hole sizes of 1/4 in. (6 mm) or larger allowed two to

three times the chloride to penetrate the concrete, compared to a 1/8-in. (3-mm) hole and intact membrane. The percent of perforation was found to correlate with the chloride content--the higher the percent of perforated area, the higher the chloride contamination of the concrete. However, the effect of membrane type was found to be minimal in both the laboratory and the field studies.

The field validation work included a total of 15 bridges in Maine, New Hampshire, and Vermont. The ultrasonic pulse velocity measurements were found to be strongly correlated with the membrane status at a correlation coefficient of 95.5 percent (R^2 = 89.3 percent). Most of the membrane deterioration was found in the inner and outer wheel paths, entrance and exit span areas and at the shoulders. Membrane performance was found to be affected by the overlay condition and the extent of bonding to both the underlying concrete deck and the asphalt concrete overlay.

The chloride contents of the evaluated bridges indicates that membrane systems are impeding chloride contamination. The average chloride content in each bridge is influenced by the amount of snowfall, total salt application and AADT.

Conclusions

Based on the results of the research described in this report, the following conclusions appear to be warranted:

- Properly installed preformed membrane systems are an effective means of protecting bridge decks from chloride intrusion.
- Bridge deck life may be extended by 25 yr by a properly installed membrane.
- The effectiveness of membranes is affected by perforations of 1/4 in. (6 mm) or larger.
- The membrane integrity is affected by its bonding to the concrete surface and to the hot-mix asphalt overlay. Membranes should be installed on clean surfaces.
- Membrane performance is highly affected by the asphaltic overlay condition. An overlay of at least 2.5 in. (6.4 cm) is recommended to be used.

- Although more data are needed to confirm this test procedure, the research from this project indicates that ultrasonic pulse velocity can be used to evaluate a membrane on an existing concrete deck with an asphalt overlay.

Test Procedure

A recommended test procedure, in ASTM format, has been prepared for use of pulse velocity measurements for assessing the condition of preformed membrane systems on bridge decks. It is presented in Volume 8, "Procedure Manual," of this report series.

Appendix A

Test Data

Table A-1. Test data for Stratford, New Hampshire, Bridge No. 107/074.

Position I.D.	V-Meter Reading (μs)	Position Location*	Chloride Content† Cl(1) (lb/yd^3)	Cl(2) (lb/yd^3)	Cl(3) (lb/yd^3)	Membrane Rating
1	---	Shoulder				
2	22.6	O.W.T.				
3	22.6	I.W.T.				
4	20.4	O.W.T.				
5	18.4	B.W.T.				
6	21.3	I.W.T.				
10	23.5	O.W.T. (A/S)	1.712	0.778	<0.04	6.0
11	17.8	B.W.T. (A/S)	2.073	1.854	0.740	4.5
12	---	I.W.T. (A/S)	1.418	0.683	0.097	3.5
13	24.5	O.W.T.	0.301	<0.04	<0.04	8.0
14	21.4	B.W.T.	0.114	0.114	<0.04	7.0
15	22.6	I.W.T.	<0.04	<0.04	<0.04	---
16	24.6	Shoulder	1.806	0.817	<0.04	9.0
17	20.9	Shoulder	0.896	0.976	0.936	6.5
18	20.7	Shoulder (D/S)	0.976	1.712	0.896	6.5
19	23.3	Shoulder (D/S)	<0.04	<0.04	0.301	9.0
20	20.5	O.W.T. (D/S)	<0.04	<0.04	<0.04	6.0

Table A-1. *(continued)*

Position I.D.	V-Meter Reading (μs)	Position Location*	Chloride Content† Cl(1) (lb/yd³)	Cl(2) (lb/yd³)	Cl(3) (lb/yd³)	Membrane Rating
21	---	B.W.T. (D/S)	<0.04	<0.04	<0.04	5.0
22	21.4	I.W.T. (D/S)	<0.04	<0.04	<0.04	7.0
28	---	C.L.	<0.04	<0.04	<0.04	2.0
29	---	I.W.T. (A/S)	0.080	<0.04	<0.04	5.5

*Note: A/S: Entrance Span Area
B.W.T.: Between Wheel Tracks
C.L.: Center Line
D/S: Exiting Span Area
I.W.T.: Inside Wheel Track
O.W.T.: Outer Wheel Track
See "Pulse Velocity Versus Membrane Condition" in Chapter 4 for a detailed description of the rating system.

†1 lb/yd³ = 0.59 kg/m³ (approximately)

Table A-2. Test data for Tilton, New Hampshire, Bridge No. 117/157.

Position I.D.	V-Meter Reading (μs)	Position Location*	Chloride Content† Cl(1) (lb/yd^3)	Cl(2) (lb/yd^3)	Cl(3) (lb/yd^3)	Membrane Rating
1	22.5	Shoulder				
2	---	Shoulder				
3	---	O.W.T.				
4	24.0	B.W.T.				
5	---	I.W.T.				
6	24.9	Shoulder	3.768	2.782	2.352	9.0
7	23.6	Shoulder	3.930	4.062	3.124	9.0
8	21.8	O.W.T. (A/S)				6.0
9	16.6	B.W.T. (A/S)	4.542	4.904	4.298	4.5
10	18.3	I.W.T. (A/S)	4.867	4.941	4.757	5.0
11	20.6	Shoulder (Near Drain)	9.018	8.173	8.483	6.0
12	18.7	Shoulder (Near Drain)	4.196	3.424	3.037	4.0
13	21.0	Shoulder	3.424	3.183	2.672	7.5
14	18.6	O.W.T.				
15	20.2	B.W.T.				
16	15.3	I.W.T.	5.359	6.213	4.794	3.5
17	21.1	Shoulder (Near Drain)				
18	17.8	Shoulder				

Table A-2. (*continued*)

Position I.D.	V-Meter Reading (μs)	Position Location*	Chloride Content† Cl(1) (lb/yd³)	Cl(2) (lb/yd³)	Cl(3) (lb/yd³)	Membrane Rating
19	14.3	Shoulder				
20	13.3	Shoulder				
21	16.7	O.W.T. (D/S)				
22	14.7	B.W.T. (D/S)				4.0
23	21.8	I.W.T. (D/S)	4.062	3.095	1.736	7.5
24	19.9	I.W.T.				
25	18.3	I.W.T.				
26	18.5	O.W.T.				
30	20.6	I.W.T. (A/S)				
31	---	B.W.T. (A/S)				
32	21.8	O.W.T. (A/S)	6.342	5.320	3.672	---
33	22.3	Shoulder (A/S)				
34	21.5	Shoulder				
35	20.1	I.W.T.	1.309	0.552	0.497	7.0
36	22.0	B.W.T.				
37	--	O.W.T.				
38	24.5	Shoulder				
39	23.5	Shoulder				
40	20.7	I.W.T. (D/S)	2.536	1.057	0.319	7.0

Table A-2. (*continued*)

Position I.D.	V-Meter Reading (μs)	Position Location*	Chloride Content† Cl(1) (lb/yd^3)	Cl(2) (lb/yd^3)	Cl(3) (lb/yd^3)	Membrane Rating
41	23.9	B.W.T. (D/S)				
42	---	O.W.T. (D/S)	5.320	3.424	1.374	4.0

*Note: A/S: Entrance Span Area
B.W.T.: Between Wheel Tracks
C.L.: Center Line
D/S: Exiting Span Area
I.W.T.: Inside Wheel Track
O.W.T.: Outer Wheel Track
See "Pulse Velocity Versus Membrane Condition" in Chapter 4 for a detailed description of the rating system.

†1 lb/yd^3 = 0.59 kg/m^3 (approximately)

Table A-3. Test data for Tilton, New Hampshire, Bridge No. 118/158.

Position I.D.	V-Meter Reading (μs)	Position Location*	Chloride Content† Cl(1) (lb/yd^3)	Cl(2) (lb/yd^3)	Cl(3) (lb/yd^3)	Membrane Rating
1	---	Shoulder (A/S)				
2	21.5	Between Shoulder and O.W.T. (A/S)				
3	19.9	O.W.T. (A/S)				
4	21.1	B.W.T. (A/S)				
5	21.3	I.W.T.(A/S)				
6	---	Between B.W.T. and I.W.T. (A/S)	3.771	3.455	2.216	3.5
7A	21.2	C.L. (A/S)				
7B	22.5	C.L.				
8	24.0	Between C.L. and I.W.T.	0.427	0.391	0.355	9.0
9	24.6	I.W.T.				
10	20.7	O.W.T.				
11	---	B.W.T.	6.905	6.239	6.368	1.5
12	21.7	Shoulder				
13	21.6	I.W.T.				
14	24.8	B.W.T.	9.354	7.864	8.015	9.0
15	20.8	O.W.T.				
16	21.6	I.W.T.				
17	23.0	B.W.T.				
18	20.6	O.W.T.	7.864	10.170	8.693	7.0

Table A-3. (*continued*)

Position I.D.	V-Meter Reading (μs)	Position Location*	Chloride Content† Cl(1) (lb/yd^3)	Cl(2) (lb/yd^3)	Cl(3) (lb/yd^3)	Membrane Rating
19	---	C.L.	7.278	7.964	7.374	6.0
20	22.1	Between C.L. and I.W.T.				
21	24.0	I.W.T.				
22	22.8	B.W.T.				
23	21.3	O.W.T.				
24	24.4	Shoulder				
25	21.0	I.W.T. (D/S)				
26	22.4	B.W.T. (D/S)				
27	21.8	O.W.T. (D/S)				
28	21.3	I.W.T.				
29	24.6	B.W.T.				
30	24.4	O.W.T.				
31	---	Shoulder				
32	22.7	O.W.T.	6.589	6.282	4.135	8.0
33	---	B.W.T.	6.633	6.544	6.412	3.5
34	---	I.W.T. (A/S)				
35	---	B.W.T. (A/S)				
36	---	O.W.T. (A/S)				
37	22.5	Shoulder (A/S)				
38	24.5	Shoulder (A/S)	4.034	4.203	2.948	9.0
39	---	Shoulder	5.298	6.196	8.375	2.0
40	---	I.W.T.				

Table A-3. (*continued*)

Position I.D.	V-Meter Reading (μs)	Position Location*	Chloride Content† Cl(1) (lb/yd³)	Cl(2) (lb/yd³)	Cl(3) (lb/yd³)	Membrane Rating
41	---	B.W.T.				
42	---	O.W.T.				
43	---	I.W.T.				
44	---	B.W.T.				
45	22.5	O.W.T.				
46	23.2	Shoulder	0.744	<0.04	<0.04	---
47	23.2	I.W.T.				
48	23.5	Between I.W.T. and B.W.T.				
49	23.3	Between O.W.T. and B.W.T.				
50	24.8	O.W.T.	5.067	5.695	4.806	9.0
51	23.6	Between O.W.T. and Shoulder				
52	23.5	Shoulder	4.552	4.341	3.580	1.5
53	---	Between C.L. and I.W.T. (D/S)				
54	---	I.W.T. (D/S)				
55	23.7	B.W.T. (D/S)				

Table A-3. (*continued*)

Position I.D.	V-Meter Reading (μs)	Position Location*	Chloride Content† Cl(1) (lb/yd^3)	Cl(2) (lb/yd^3)	Cl(3) (lb/yd^3)	Membrane Rating
56	21.8	Between B.W.T. and O.W.T. (D/S)				
57	23.6	O.W.T. (D/S)				
58	21.6	Shoulder (D/S)				
59	---	Shoulder (D/S)				

*Note: A/S: Entrance Span Area
B.W.T.: Between Wheel Tracks
C.L.: Center Line
D/S: Exiting Span Area
I.W.T.: Inside Wheel Track
O.W.T.: Outer Wheel Track
See "Pulse Velocity Versus Membrane Condition" in Chapter 4 for a detailed description of the rating system.

†1 lb/yd^3 = 0.59 kg/m^3 (approximately)

Table A-4. Test data for Hillsborough, New Hampshire, Bridge No. 117/035.

Position I.D.	V-Meter Reading (μs)	Position Location*	Chloride Content† Cl(1) (lb/yd^3)	Cl(2) (lb/yd^3)	Cl(3) (lb/yd^3)	Membrane Rating
3	18.3	B.W.T.				
4	18.8	I.W.T.				
5	19.3	I.W.T.				
6	19.3	B.W.T.				
7	18.2	O.W.T.				
8	18.6	Shoulder (A/S)				
9A	18.4	O.W.T. (A/S)				
9B	20.7	O.W.T. (A/S)	<0.04	<0.04	<0.04	7.5
10	19.0	B.W.T. (A/S)	<0.04	<0.04	<0.04	5.0
11	19.6	I.W.T. (A/S)	<0.04	<0.04	<0.04	5.5
12	17.7	I.W.T. (A/S)				
13	18.8	B.W.T. (A/S)				
14	19.8	O.W.T. (A/S)				
15	18.3	O.W.T.	<0.04	<0.04	<0.04	6.0
16	---	B.W.T.				
17	18.9	I.W.T.				

Table A-4. (*continued*)

Position I.D.	V-Meter Reading (μs)	Position Location*	Chloride Content† Cl(1) (lb/yd³)	Cl(2) (lb/yd³)	Cl(3) (lb/yd³)	Membrane Rating
18	21.9	Between O.W.T. and Shoulder				
19	19.6	O.W.T.				
20	19.3	B.W.T.				
21	19.2	Between B.W.T. and I.W.T.				
22	18.4	I.W.T.				
23	18.7	C.L.				
24	18.7	I.W.T.				
25	21.0	B.W.T.				
26	20.3	O.W.T.				
27	19.6	O.W.T.	<0.04	<0.04	<0.04	5.0
28	17.4	B.W.T.	<0.04	<0.04	<0.04	4.5
29	21.0	I.W.T.	<0.04	<0.04	<0.04	7.5
30	21.4	Between Shoulder and O.W.T.				
31	18.7	O.W.T.				
32	19.3	B.W.T.				
33	24.5	I.W.T.	<0.04	<0.04	<0.04	9.0
34	18.0	Between Shoulder and O.W.T.				

Table A-4. (*continued*)

Position I.D.	V-Meter Reading (μs)	Position Location*	Chloride Content† Cl(1) (lb/yd^3)	Cl(2) (lb/yd^3)	Cl(3) (lb/yd^3)	Membrane Rating
35	21.3	O.W.T.	<0.04	<0.04	<0.04	7.5
36	17.5	Between B.W.T. and I.W.T.				
37	18.5	B.W.T.				
38	17.4	Between B.W.T. and I.W.T.				
39	18.4	I.W.T.				
40	18.2	I.W.T.				
41	18.7	B.W.T.				
42	20.0	O.W.T.	<0.04	---	<0.04	6.0
43	--	I.W.T.	<0.04	<0.04	<0.04	---
44	18.2	Between O.W.T. and Shoulder (D/S)				
45	18.1	O.W.T. (D/S)	<0.04	<0.04	<0.04	5.0
46	20.0	B.W.T.(D/S)	<0.04	<0.04	<0.04	4.5
47	18.4	I.W.T. (D/S)				
48	18.9	I.W.T. (D/S)				
49	18.9	B.W.T.(D/S)				

Table A-4. (*continued*)

Position I.D.	V-Meter Reading (μs)	Position Location*	Chloride Content† Cl(1) (lb/yd³)	Cl(2) (lb/yd³)	Cl(3) (lb/yd³)	Membrane Rating
50	19.2	O.W.T. (D/S)				
51	22.4	O.W.T.				
52	21.5	B.W.T.				
53	19.8	I.W.T.				
54	21.3	I.W.T.	<0.04	<0.04	<0.04	7.5
55	22.8	B.W.T. (A/S)	<0.04	<0.04	<0.04	8.5
56	20.1	O.W.T. (A/S)				
57	20.3	I.W.T.				
58	20.2	B.W.T.				
59	21.8	O.W.T.				
61	20.2	I.W.T. (D/S)	<0.04	<0.04	<0.04	6.0
62	19.7	B.W.T. (D/S)	<0.04	<0.04	<0.04	6.5
63	19.1	O.W.T. (D/S)				

*Note: A/S: Entrance Span Area
B.W.T.: Between Wheel Tracks
C.L.: Center Line
D/S: Exiting Span Area
I.W.T.: Inside Wheel Track
O.W.T.: Outer Wheel Track
See "Pulse Velocity Versus Membrane Condition" in Chapter 4 for a detailed description of the rating system.

†1 lb/yd³ = 0.59 kg/m³ (approximately)

Table A-5. Test data for Concord, New Hampshire, Bridge No. 165/177.

Position Number	V-Meter Reading (μs)	Position Location*	Chloride Content† Cl(1) (lb/yd^3)	Cl(2) (lb/yd^3)	Cl(3) (lb/yd^3)	Membrane Rating
1	19.8	I.W.T. (A/S)	<0.04	<0.04	<0.04	6.0
2	21.6	B.W.T. (A/S)	<0.04	<0.04	<0.04	7.5
3	18.3	O.W.T.(A/S)	<0.04	---	<0.04	5.5
4	20.6	Shoulder (A/S)	<0.04	<0.04	<0.04	6.0
5	21.4	Shoulder (A/S)	<0.04	<0.04	<0.04	6.5
6	25.8	Shoulder	<0.04	<0.04	<0.04	6.0
7	20.7	I.W.T.	---	<0.04	<0.04	6.0
8	22.6	B.W.T.	<0.04	<0.04	<0.04	6.5
9	17.3	O.W.T.	<0.04	<0.04	<0.04	4.5
10	26.0	Shoulder				
11	21.6	Shoulder	<0.04	<0.04	<0.04	7.0
12	23.8	Shoulder				
13	19.0	Shoulder	<0.04	<0.04	<0.04	5.5
14	21.5	I.W.T.				
15	20.0	Between B.W.T. and I.W.T.				
16	20.9	B.W.T.				
17	21.7	Between B.W.T. and O.W.T.				
18	19.0	O.W.T.				

Table A-5. (*continued*)

Position Number	V-Meter Reading (μs)	Position Location*	Chloride Content† Cl(1) (lb/yd^3)	Cl(2) (lb/yd^3)	Cl(3) (lb/yd^3)	Membrane Rating
19	20.0	Between O.W.T. and Shoulder				
20	19.3	Between O.W.T. and Shoulder				
21	17.4	Shoulder				
22	19.5	Shoulder				
23	21.2	Shoulder				
24	18.6	Shoulder				
25	19.8	Shoulder				
26	20.2	Between I.W.T. and C.L. (D/S)	<0.04	<0.04	<0.04	6.0
27	20.3	I.W.T. (D/S)				
28	21.8	Between I.W.T. and B.W.T.(D/S)				
29	20.3	B.W.T.(D/S)	<0.04	<0.04	<0.04	6.5
30	17.3	Between B.W.T. and O.W.T. (D/S)	0.611	0.233	<0.04	4.0
31	17.6	O.W.T. (D/S)	<0.04	<0.04	<0.04	4.5
32	16.2	Between O.W.T. and Shoulder (D/S)				

Table A-5. (*continued*)

Position Number	V-Meter Reading (μs)	Position Location*	Chloride Content† Cl(1) (lb/yd^3)	Cl(2) (lb/yd^3)	Cl(3) (lb/yd^3)	Membrane Rating
33	19.8	Shoulder (D/S)				
34	19.5	Shoulder (D/S)	<0.04	<0.04	<0.04	5.5
35	19.2	Shoulder (D/S)				

*Note: A/S: Entrance Span Area
B.W.T.: Between Wheel Tracks
C.L.: Center Line
D/S: Exiting Span Area
I.W.T.: Inside Wheel Track
O.W.T.: Outer Wheel Track
See "Pulse Velocity Versus Membrane Condition" in Chapter 4 for a detailed description of the rating system.

†1 lb/yd^3 = 0.59 kg/m^3 (approximately)

Table A-6. Test data for Caribou, Maine, Bridge No. 6255.

Position I.D.	V-Meter Reading (μs)	Position Location*	Chloride Content† Cl(1) (lb/yd³)	Cl(2) (lb/yd³)	Cl(3) (lb/yd³)	Membrane Rating
1	20.7	Shoulder (A/S)	<0.04	<0.04	<0.04	7.0
2	15.3	O.W.T. (A/S)				
3	14.3	B.W.T. (A/S)	<0.04	<0.04	<0.04	3.0
4	20.6	I.W.T.(A/S)				
5	14.5	Shoulder				
6	14.4	O.W.T.	0.079	<0.04	<0.04	3.0
7	13.3	B.W.T.				
8	13.6	I.W.T.	<0.04	<0.04	<0.04	2.5
9	14.3	Shoulder (Near Drain)				
10	14.3	Between O.W.T. and Shoulder				
11	14.5	Between B.W.T. and O.W.T.				
12	15.6	B.W.T.				
13	15.1	I.W.T.				
14	14.2	Between C.L. and I.W.T.				
15	14.3	Shoulder				
16	15.6	Shoulder				
17	17.3	Between O.W.T. and Shoulder				

Table A-6. (*continued*)

Position I.D.	V-Meter Reading (μs)	Position Location*	Chloride Content† Cl(1) (lb/yd³)	Cl(2) (lb/yd³)	Cl(3) (lb/yd³)	Membrane Rating
18	17.2	Between B.W.T. and O.W.T.				
19	16.6	B.W.T.				
20	17.7	I.W.T.				
21	17.5	Between C.L. and I.W.T.				
22	19.3	Shoulder				
23	19.6	Between O.W.T. and Shoulder				
24	17.7	O.W.T.	<0.04	<0.04	<0.04	3.0
25	18.4	B.W.T.				
26	17.1	I.W.T.	<0.04	<0.04	<0.04	5.5
27	18.3	Between C.L. and I.W.T.				
28	19.3	C.L.	<0.04	<0.04	<0.04	6.0
29	---	I.W.T.	0.725	0.463	0.373	1.0
30	19.6	O.W.T.				
31	19.6	B.W.T.				
32	20.4	I.W.T.	<0.04	<0.04	<0.04	6.5
33	20.0	O.W.T. (D/S)	<0.04	<0.04	<0.04	6.0

Table A-6. (*continued*)

Position I.D.	V-Meter Reading (μs)	Position Location*	Chloride Content† Cl(1) (lb/yd³)	Cl(2) (lb/yd³)	Cl(3) (lb/yd³)	Membrane Rating
34	17.7	B.W.T. (D/S)	<0.04	<0.04	<0.04	4.5
35	22.5	I.W.T. (D/S)	0.822	<0.04	<0.04	7.0
CTR	---		1.190	0.409	<0.04	--

*Note: A/S: Entrance Span Area
B.W.T.: Between Wheel Tracks
C.L.: Center Line
D/S: Exiting Span Area
I.W.T.: Inside Wheel Track
O.W.T.: Outer Wheel Track
See "Pulse Velocity Versus Membrane Condition" in Chapter 4 for a detailed description of the rating system.

†1 lb/yd³ = 0.59 kg/m³ (approximately)

Table A-7. Test data for South Portland, Maine, Bridge No. 1376.

Position I.D.	V-Meter Reading (μs)	Position Location*	Chloride Content† Cl(1) (lb/yd^3)	Cl(2) (lb/yd^3)	Cl(3) (lb/yd^3)	Membrane Rating
1	20.3	B.W.T. (A/S)	0.249	0.181	<0.04	6.0
2	23.6	O.W.T. (A/S)	0.336	0.336	0.479	8.5
3	24.7	Shoulder				
4	23.1	Shoulder (Near Drain)	0.443	0.443	0.407	8.0
5	19.4	O.W.T.				
6	21.1	B.W.T. (A/S)				
7	20.9	O.W.T. (A/S)				
8	20.4	Between C.L. and O.W.T.				
9	22.4	O.W.T.	0.479	0.267	0.198	7.5
10	19.2	B.W.T.				
11	20.8	I.W.T.	0.249	0.354	0.461	6.5
12	19.6	Between Shoulder and I.W.T.				
13	20.7	B.W.T.	---	0.336	0.372	7.5
14	21.4	I.W.T.				
15	18.4	B.W.T.	0.626	0.608	0.664	5.0
16	19.4	O.W.T.				
17	20.5	I.W.T. (D/S)	3.154	2.536	2.326	6.0
18	18.4	Between I.W.T. and B.W.T. (D/S)				

Table A-7. (*continued*)

Position I.D.	V-Meter Reading (μs)	Position Location*	Chloride Content† Cl(1) (lb/yd^3)	Cl(2) (lb/yd^3)	Cl(3) (lb/yd^3)	Membrane Rating
19	18.4	B.W.T. (D/S)	1.529	1.266	0.936	5.0
20	19.3	O.W.T. (D/S)				
21	19.3	B.W.T.				
22	19.7	Between I.W.T. and C.L. (D/S)				
23	19.5	I.W.T. (D/S)				
24	18.4	C.L.	0.589	0.443	0.372	---
25	19.7	I.W.T.				
26	21.1	I.W.T. (A/S)	0.534	0.425	0.626	7.0
27	19.3	C.L.				
28	21.0	I.W.T.	0.461	0.589	0.534	7.0
29	21.7	C.L.	1.057	0.837	0.552	7.5
30	19.1	I.W.T.				
31	19.0	I.W.T. (A/S)	0.740	0.389	0.407	5.0
32	20.5	I.W.T.	0.443	0.534	0.515	6.5
33	23.6	Between C.L. and I.W.T.				
34	20.7	B.W.T.				

Table A-7. (*continued*)

Position I.D.	V-Meter Reading (μs)	Position Location*	Chloride Content† Cl(1) (lb/yd^3)	Cl(2) (lb/yd^3)	Cl(3) (lb/yd^3)	Membrane Rating
35	19.3	B.W.T. (A/S)				
36	21.55	I.W.T. (A/S)				

*Note: A/S: Entrance Span Area
B.W.T.: Between Wheel Tracks
C.L.: Center Line
D/S: Exiting Span Area
I.W.T.: Inside Wheel Track
O.W.T.: Outer Wheel Track
See "Pulse Velocity Versus Membrane Condition" in Chapter 4 for a detailed description of the rating system.

†1 lb/yd^3 = 0.59 kg/m^3 (approximately)

Table A-8. Test data for Augusta, Maine, Bridge No. 5807.

Position I.D.	V-Meter Reading (μs)	Position Location*	Chloride Content† Cl(1) (lb/yr^3)	Cl(2) (lb/yr^3)	Cl(3) (lb/yr^3)	Membrane Rating
1	20.9	L.W.P. (A/S)				
2	21.7	B.W.T. (A/S)	<0.04	<0.04	<0.04	7.5
3	22.2	O.W.T. (A/S)	5.415	4.517	3.580	8.0
4	21.5	O.W.T.				
5	18.4	B.W.T.				
6	20.5	L.W.P.				
7	---	B.W.T.	5.574	1.844	3.363	3.0
8	21.5	O.W.T.	4.624	4.769	3.968	7.5
9	20.6	B.W.T.				
10	19.9	L.W.P.				
11	20.0	Shoulder	2.190	1.844	1.892	6.5
12	19.5	O.W.T.				
13	19.0	B.W.T.				
14	21.6	I.W.T.				
15	19.8	Between C.L. and I.W.T.	5.901	3.771	3.643	5.5
16	21.3	Between O.W.T. and Shoulder				
17	19.8	O.W.T.				
18	22.2	B.W.T.				
19	21.8	I.W.T.				
20	19.5	Between C.L. and I.W.T.				

Table A-8. (*continued*)

Position I.D.	V-Meter Reading (μs)	Position Location*	Chloride Content† Cl(1) (lb/yr^3)	Cl(2) (lb/yr^3)	Cl(3) (lb/yr^3)	Membrane Rating
21	---	I.W.T.	---	5.029	3.035	---
22	18.3	O.W.T. (D/S)	4.843	4.101	3.333	5.0
23	19.1	B.W.T. (D/S)	2.398	3.363	1.965	6.0
24	19.4	I.W.T. (D/S)				
25	19.6	Between C.L. and I.W.T. (D/S)				
26	17.9	O.W.T.				
27	18.2	B.W.T.				
28	19.6	C.L.				
29	22.7	I.W.T. (A/S)				
30	19.2	B.W.T. (A/S)				
31	21.6	O.W.T. (A/S)	4.733	3.153	1.796	7.5
32	---	Between B.W.T. and I.W.T.	6.997	6.544	4.169	2.0
33	21.4	Between C.L. and I.W.T.				
34	20.3	I.W.T.				
35	18.1	B.W.T.				
36	20.4	O.W.T.	2.039	1.990	1.518	6.5
37	17.8	Between O.W.T. and Shoulder				
38	23.5	Shoulder				
39	19.7	I.W.T.				

Table A-8. (*continued*)

Position I.D.	V-Meter Reading (μs)	Position Location*	Chloride Content† Cl(1) (lb/yr³)	Cl(2) (lb/yr³)	Cl(3) (lb/yr³)	Membrane Rating
40	18.3	B.W.T.				
41	20.3	O.W.T.				
42	---	B.W.T.	9.019	8.586	7.374	2.0
43	24.3	Between C.L. and I.W.T. (D/S)				
44	---	I.W.T. (D/S)	1.749	1.632	1.702	2.5
45	20.0	B.W.T. (D/S)				
46	20.8	O.W.T. (D/S)				

*Note: A/S: Entrance Span Area
B.W.T.: Between Wheel Tracks
C.L.: Center Line
D/S: Exiting Span Area
I.W.T.: Inside Wheel Track
O.W.T.: Outer Wheel Track
See "Pulse Velocity Versus Membrane Condition" in Chapter 4 for a detailed description of the rating system.

†1 lb/yd^3 = 0.59 kg/m^3 (approximately)

Table A-9. Test data for Beddeford, Maine, Bridge No. 082.

Position I.D.	V-Meter Reading (μs)	Position Location*	Chloride Content† Cl(1) (lb/yd^3)	Cl(2) (lb/yd^3)	Cl(3) (lb/yd^3)	Membrane Rating
1	22.1	I.W.T. (A/S)				
2	21.7	B.W.T. (A/S)	<0.04	<0.04	<0.04	8.0
3	24.6	O.W.T. (A/S)	<0.04	<0.04	<0.04	9.0
4	21.2	Shoulder (A/S)				
5	20.9	Shoulder (A/S)	<0.04	<0.04	<0.04	7.0
6	22.0	Sealed Shoulder				
7	20.3	I.W.T.				
8	22.0	B.W.T.				
9	21.2	O.W.T.				
10	21.5	I.W.T.				
11	22.7	B.W.T.	<0.04	<0.04	<0.04	7.5
12	23.7	O.W.T.	<0.04	<0.04	<0.04	8.5
13	25.7	Shoulder				
14	24.6	Sealed Shoulder				
15	---	B.W.T.	<0.04	<0.04	<0.04	2.5
16	23.8	I.W.T.				
17	26.1	B.W.T.				
18	---	O.W.T.				
19	23.0	Sealed Shoulder	<0.04	<0.04	<0.04	8.0

Table A-9. (*continued*)

Position I.D.	V-Meter Reading (μs)	Position Location*	Chloride Content† Cl(1) (lb/yd3)	Cl(2) (lb/yd^3)	Cl(3) (lb/yd^3)	Membrane Rating
20	24.9	I.W.T.				
21	---	B.W.T.	<0.04	<0.04	<0.04	2.0
22	---	O.W.T.				
23	24.1	Shoulder	<0.04	<0.04	<0.04	9.0
24	22.8	Sealed Shoulder				
25	22.4	I.W.T.				
26	20.1	O.W.T.	<0.04	<0.04	<0.04	6.0
27	19.8	I.W.T. (D/S)				
28	19.5	B.W.T. (D/S)				
29	20.0	O.W.T. (D/S)	<0.04	<0.04	<0.04	6.0
30	19.7	Shoulder (D/S)				
31	18.8	L.W.P.				
32	19.4	B.W.T.				
33	21.2	O.W.T. (A/S)				
34	21.6	B.W.T. (A/S)				
35	21.2	I.W.T. (A/S)	<0.04	<0.04	<0.04	7.5
36	20.9	Sealed Shoulder (Near Drain)	<0.04	<0.04	<0.04	6.0
37	20.6	Between C.L. and I.W.T.				
38	20.3	I.W.T.				

Table A-9. (*continued*)

Position I.D.	V-Meter Reading (μs)	Position Location*	Chloride Content† Cl(1) (lb/yd3)	Cl(2) (lb/yd^3)	Cl(3) (lb/yd^3)	Membrane Rating
39	20.3	B.W.T.				
40	20.6	O.W.T.				

*Note: A/S: Entrance Span Area
B.W.T.: Between Wheel Tracks
C.L.: Center Line
D/S: Exiting Span Area
I.W.T.: Inside Wheel Track
O.W.T.: Outer Wheel Track
See "Pulse Velocity Versus Membrane Condition" in Chapter 4 for a detailed description of the rating system.

†$1\ lb/yd^3 = 0.59\ kg/m^3$ (approximately)

Table A-10. Test data for Brewer, Maine, Bridge No. 1562.

Position I.D.	V-Meter Reading (μs)	Position Location*	Chloride Content† Cl(1) (lb/yd3)	Cl(2) (lb/yd^3)	Cl(3) (lb/yd^3)	Membrane Rating
1	24.6	Between C.L. and I.W.T. (A/S)	0.856	0.215	<0.04	9.0
1A	---	I.W.T. (A/S)	0.164	0.181	0.284	---
2	23.1	B.W.T. (A/S)	0.443	<0.04	<0.04	8.0
3	23.3	O.W.T. (A/S)	0.626	0.389	0.372	8.0
4	21.4	Between O.W.T. and Shoulder	0.645	<0.04	<0.04	7.5
5	19.9	Shoulder	<0.04	<0.04	<0.04	5.5
5A	24.4	I.W.T.				
6	19.5	B.W.T.				
7	21.9	O.W.T.				
8	21.6	Shoulder				
8A	21.9	Sealed Shoulder				
9	22.5	Between C.L. and I.W.T.				
10	25.3	I.W.T.				
11	22.2	Between I.W.T. and B.W.T.				
12	18.9	B.W.T.				
13	21.3	Between B.W.T. and O.W.T.	<0.04	<0.04	<0.04	7.5
14	23.2	O.W.T.				

Table A-10. (*continued*)

Position I.D.	V-Meter Reading (μs)	Position Location*	Chloride Content† Cl(1) (lb/yd3)	Cl(2) (lb/yd^3)	Cl(3) (lb/yd^3)	Membrane Rating
15	23.8	Between O.W.T. and Shoulder				
16	21.0	Shoulder				
17	21.7	Shoulder				
18	21.9	Sealed Shoulder (Near Drain)	<0.04	<0.04	<0.04	7.0
19	21.1	Between C.L. and I.W.T.				
20	22.5	I.W.T.				
21	21.9	B.W.T.	<0.04	<0.04	<0.04	7.5
22	21.6	O.W.T.	<0.04	<0.04	<0.04	7.0
23	22.5	Between O.W.T. and Shoulder				
23A	18.5	Sealed Shoulder				
24	20.6	Between I.W.T. and C.L. (D/S)				
25	19.5	I.W.T. (D/S)	0.336	0.080	0.407	5.0
26	23.4	B.W.T. (D/S)	0.497	0.443	0.232	8.5
27	23.4	O.W.T. (D/S)	0.570	0.534	0.479	8.5

Table A-10. (*continued*)

Position I.D.	V-Meter Reading (μs)	Position Location*	Chloride Content† Cl(1) (lb/yd3)	Cl(2) (lb/yd^3)	Cl(3) (lb/yd^3)	Membrane Rating
28	20.3	Between O.W.T. and Shoulder (D/S)				
29	20.7	Shoulder (D/S)				
30	20.6	Shoulder (D/S)				
31	20.6	I.W.T.				
32	20.4	B.W.T.				
33	20.1	O.W.T.				
34	23.2	B.W.T.				
35	20.6	I.W.T. (A/S)				
36	---	B.W.T. (A/S)	1.037	0.817	0.664	9.0
37	23.7	O.W.T. (A/S)	1.462	1.182	1.830	8.0
38	23.2	Shoulder (A/S)				
39	23.4	B.W.T.				
40	22.6	Between B.W.T. and O.W.T.	0.215	<0.04	<0.04	---
41	21.3	O.W.T.	1.440	0.319	0.130	7.0
42	21.6	B.W.T. (D/S				

Table A-10. (*continued*)

Position I.D.	V-Meter Reading (μs)	Position Location*	Chloride Content† Cl(1) (lb/yd3)	Cl(2) (lb/yd^3)	Cl(3) (lb/yd^3)	Membrane Rating
43	22.2	B.W.T. (D/S)				
44	21.3	O.W.T. (D/S)				
45	21.6	Shoulder (D/S)				

*Note: A/S: Entrance Span Area
B.W.T.: Between Wheel Tracks
C.L.: Center Line
D/S: Exiting Span Area
I.W.T.: Inside Wheel Track
O.W.T.: Outer Wheel Track
See "Pulse Velocity Versus Membrane Condition" in Chapter 4 for a detailed description of the rating system.

†1 lb/yd^3 = 0.59 kg/m^3 (approximately)

Table A-11. Test data for Newbury, Vermont, Bridge No. 645.

Position I.D.	V-Meter Reading (μs)	Position Location*	Chloride Content† Cl(1) (lb/yd3)	Cl(2) (lb/yd³)	Cl(3) (lb/yd³)	Membrane Rating
1	22.2	I.W.T. (A/S)	<0.04	<0.04	<0.04	7.5
2	19.9	B.W.T. (A/S)				6.0
3	---	O.W.T. (A/S)				
4	23.2	Shoulder (A/S)	<0.04	<0.04	<0.04	8.5
5	24.6	I.W.T.	<0.04	<0.04	<0.04	9.0
6	20.5	B.W.T.				
7	23.3	O.W.T.	<0.04	<0.04	<0.04	8.0
8	20.8	Shoulder				
9	22.4	Shoulder				
10	24.7	Sealed Shoulder	<0.04	<0.04	<0.04	9.0
11	---	O.W.T.	<0.04	<0.04	<0.04	--
12	23.8	Between I.W.T. and B.W.T.				
13	22.4	Between B.W.T. and O.W.T.				
14	22.4	I.W.T.				
15	---	I.W.T. (D/S)	<0.04	<0.04	<0.04	--
16	---	Between O.W.T. and B.W.T. (D/S)				
17	22.9	B.W.T. (D/S)	<0.04	<0.04	<0.04	8.0

Table A-11. (*continued*)

Position I.D.	V-Meter Reading (μs)	Position Location*	Chloride Content† Cl(1) (lb/yd3)	Cl(2) (lb/yd^3)	Cl(3) (lb/yd^3)	Membrane Rating
18	---	Between O.W.T. and B.W.T. (D/S)				
19	---	O.W.T. (D/S)				
20	24.3	Shoulder (D/S)				
21	22.6	Shoulder (D/S)				
22	23.1	I.W.T. (A/S)				
23	22.8	B.W.T. (A/S)	<0.04	<0.04	<0.04	7.5
24	---	O.W.T. (A/S)				
25	24.9	Shoulder (A/S)	1.023	1.496	1.232	9.0
26	26.4	I.W.T.				
27	21.9	B.W.T.	<0.04	<0.04	<0.04	7.0
28	21.8	O.W.T.				
29	22.1	C.L.	<0.04	<0.04	<0.04	7.5
30	22.6	Between C.L. and I.W.T.	<0.04	<0.04	<0.04	8.0
31	21.3	I.W.T.				
32	21.5	Between B.W.T. and I.W.T.				
33	21.2	B.W.T.				
34	21.8	O.W.T.	<0.04	<0.04	<0.04	7.5

Table A-11. (*continued*)

Position I.D.	V-Meter Reading (μs)	Position Location*	Chloride Content† Cl(1) (lb/yd3)	Cl(2) (lb/yd^3)	Cl(3) (lb/yd^3)	Membrane Rating
35	21.4	Between Shoulder and O.W.T.				
36	20.0	Shoulder				
37	21.3	Shoulder				
38	22.8	I.W.T.	<0.04	<0.04	<0.04	8.0
39	21.2	B.W.T.				
40	21.4	O.W.T.				
41	20.1	Between O.W.T. and Shoulder				6.0

*Note: A/S: Entrance Span Area
B.W.T.: Between Wheel Tracks
C.L.: Center Line
D/S: Exiting Span Area
I.W.T.: Inside Wheel Track
O.W.T.: Outer Wheel Track
See "Pulse Velocity Versus Membrane Condition" in Chapter 4 for a detailed description of the rating system.

†1 lb/yd^3 = 0.59 kg/m^3 (approximately)

Table A-12. Test data for Newbury, Vermont, Bridge No. 64N.

Position I.D.	V-Meter Reading (μs)	Position Location*	Chloride Content† Cl(1) (lb/yd3)	Cl(2) (lb/yd^3)	Cl(3) (lb/yd^3)	Membrane Rating
1	21.7	Sealed Shoulder (A/S)	0.706	0.409	0.164	7.5
2	21.8	Shoulder (A/S)	<0.04	<0.04	<0.04	7.5
3	20.8	Shoulder (A/S)				
4	21.0	O.W.T. (A/S)	<0.04	<0.04	<0.04	6.0
5	17.9	B.W.T. (A/S)				
6	19.3	I.W.T. (A/S)	<0.04	<0.04	<0.04	5.0
7	20.8	C.L.	<0.04	<0.04	<0.04	6.0
8	17.6	I.W.T.				
9	18.2	C.L.	<0.04	<0.04	<0.04	4.0
10	18.3	O.W.T.				
11	19.0	Shoulder				
12	20.0	Sealed Shoulder				
13	16.9	Between C.L. and I.W.T.	<0.04	<0.04	<0.04	3.0
14	17.4	Between I.W.T. and B.W.T.				
15	17.8	O.W.T.				

Table A-12. (*continued*)

Position I.D.	V-Meter Reading (μs)	Position Location*	Chloride Content† Cl(1) (lb/yd3)	Cl(2) (lb/yd^3)	Cl(3) (lb/yd^3)	Membrane Rating
16	19.0	Between O.W.T. and Shoulder				
17	20.6	Shoulder	<0.04	<0.04	<0.04	6.0
18A	18.1	Between O.W.T. and Shoulder				
18B	17.6	Between O.W.T. and Shoulder	<0.04	<0.04	<0.04	4.5
19	19.8	Between C.L. and I.W.T.				
20	18.6	Between B.W.T. and I.W.T.				
21	18.2	Between B.W.T. and O.W.T.				
22	16.6	O.W.T.	<0.04	<0.04	<0.04	3.0
23	18.2	Shoulder				
24	16.8	Shoulder				
25	19.1	I.W.T. (A/S)	0.063	<0.04	<0.04	5.0
26	17.6	B.W.T.				
27	19.0	O.W.T.				
28	24.2	Shoulder	0.744	0.500	0.233	---
29	18.2	Between C.L. and I.W.T.				
30	19.2	B.W.T.				

Table A-12. *(continued)*

Position I.D.	V-Meter Reading (μs)	Position Location*	Chloride Content† Cl(1) (lb/yd3)	Cl(2) (lb/yd^3)	Cl(3) (lb/yd^3)	Membrane Rating
31	20.1	O.W.T.				
32	22.5	I.W.T.	<0.04	<0.04	<0.04	8.0
33	21.3	B.W.T.				
34	20.5	Between B.W.T. and O.W.T.				
35	18.9	O.W.T.				
36	19.8	Shoulder	0.079	<0.04	<0.04	5.5
37	18.8	I.W.T.				
38	22.1	B.W.T.	<0.04	<0.04	<0.04	8.0
39	20.3	O.W.T.				
40	20.3	Shoulder				
41	20.5	Between C.L. and I.W.T.				
42	20.8	I.W.T.				
43	18.7	B.W.T.				
44	20.5	Between B.W.T. and O.W.T.				

Table A-12. (*continued*)

Position I.D.	V-Meter Reading (μs)	Position Location*	Chloride Content† Cl(1) (lb/yd3)	Cl(2) (lb/yd^3)	Cl(3) (lb/yd^3)	Membrane Rating
45	21.4	O.W.T.	<0.04	<0.04	---	---
46	20.8	Between O.W.T. and Shoulder				
47	21.5	Shoulder				

*Note: A/S: Entrance Span Area
B.W.T.: Between Wheel Tracks
C.L.: Center Line
D/S: Exiting Span Area
I.W.T.: Inside Wheel Track
O.W.T.: Outer Wheel Track
See "Pulse Velocity Versus Membrane Condition" in Chapter 4 for a detailed description of the rating system.

†1 lb/yd^3 = 0.59 kg/m^3 (approximately)

Table A-13. Test data for Barnet, Vermont, Bridge No. 71S.

			Chloride Content†			
Position I.D.	V-Meter Reading (μs)	Position Location*	Cl(1) (lb/yd3)	Cl(2) (lb/yd^3)	Cl(3) (lb/yd^3)	Membrane Rating
1	16.1	Between C.L. and I.W.T. (A/S)	5.837	5.837	4.264	2.5
2	21.4	I.W.T. (A/S)				
3	23.8	B.W.T. (A/S)	0.936	0.515	0.130	8.5
4	20.6	O.W.T. (A/S)				
5	---	Shoulder (A/S)	0.645	0.215	0.407	---
6	21.8	Shoulder (A/S)				
7	22.7	Sealed Shoulder (A/S)	0.608	0.425	0.232	8.0
8	21.4	I.W.T.				
9	19.3	B.W.T.				
10	19.1	O.W.T.	0.759	0.534	0.319	4.5
11	19.7	I.W.T.	2.563	1.203	0.626	5.0
12	19.5	I.W.T.				
13	18.9	Between I.W.T. and B.W.T.				
14	19.1	B.W.T.				
15	20.5	O.W.T.				

Table A-13. *(continued)*

Position I.D.	V-Meter Reading (μs)	Position Location*	Chloride Content† Cl(1) (lb/yd3)	Cl(2) (lb/yd^3)	Cl(3) (lb/yd^3)	Membrane Rating
16	20.4	Between O.W.T. and Shoulder	0.515	<0.04	<0.04	6.0
17	22.0	Between O.W.T. and Shoulder				
18	20.6	Shoulder				
19	22.6	Shoulder				
20	19.9	Sealed Shoulder				
21	20.6	I.W.T.				
22	19.2	B.W.T.				
23	21.6	O.W.T.				
24	17.2	Between C.L. and I.W.T. (D/S)	0.130	<0.04	<0.04	4.0
25	17.7	I.W.T. (D/S)				
26	19.4	Between I.W.T. and B.W.T. (D/S)				
27	18.4	B.W.T. (D/S)				
28	17.6	O.W.T. (D/S)	1.507	1.309	1.119	4.0
29	22.5	Shoulder (D/S)				
30	21.4	Shoulder (D/S)				

Table A-13. (*continued*)

Position I.D.	V-Meter Reading (μs)	Position Location*	Chloride Content†			Membrane Rating
			Cl(1) (lb/yd3)	Cl(2) (lb/yd^3)	Cl(3) (lb/yd^3)	
31	23.7	Shoulder (A/S)				
32	19.2	O.W.T. (A/S)	1.288	1.078	0.608	5.0
33	20.3	B.W.T. (A/S)				
34	22.5	I.W.T. (A/S)				
35	20.5	O.W.T.				
36	20.7	B.W.T.				
37	20.6	I.W.T.				
38	19.8	Shoulder				
38A	21.4	Shoulder	1.643	0.856	0.589	7.0
39	16.8	O.W.T.	5.166	3.672	2.326	3.0
41	19.5	B.W.T.				
42	20.4	I.W.T.				

*Note: A/S: Entrance Span Area
B.W.T.: Between Wheel Tracks
C.L.: Center Line
D/S: Exiting Span Area
I.W.T.: Inside Wheel Track
O.W.T.: Outer Wheel Track
See "Pulse Velocity Versus Membrane Condition" in Chapter 4 for a detailed description of the rating system.

†1 lb/yd^3 = 0.59 kg/m^3 (approximately)

Table A-14. Test data for Hardwick, Vermont, Bridge No. 66.

Position I.D.	V-Meter Reading (μs)	Position Location*	Chloride Content† Cl(1) (lb/yd3)	Cl(2) (lb/yd^3)	Cl(3) (lb/yd^3)	Membrane Rating
1	19.0	Shoulder (A/S)	0.130	<0.04	0.047	5.0
2	20.8	Shoulder (A/S)				
3	18.5	Between Shoulder and O.W.T. (A/S)	<0.04	<0.04	<0.04	4.0
4	19.0	O.W.T. (A/S)				
5	19.9	B.W.T. (A/S)	<0.04	<0.04	<0.04	5.5
6	20.3	I.W.T. (A/S)				
7	19.8	Between C.L. and I.W.T. (A/S)				
8	20.5	Shoulder				
9	20.2	Shoulder				
10	20.3	Shoulder	0.215	<0.04	<0.04	6.0
11	17.8	O.W.T.				
12	19.0	B.W.T.				
13	20.8	I.W.T.	0.164	<0.04	<0.04	6.5
14	21.4	Shoulder				
15	20.5	O.W.T.	<0.04	<0.04	<0.04	6.5
16	19.7	B.W.T.				
17	17.4	I.W.T.				

Table A-14. (*continued*)

Position I.D.	V-Meter Reading (μs)	Position Location*	Chloride Content† Cl(1) (lb/yd3)	Cl(2) (lb/yd^3)	Cl(3) (lb/yd^3)	Membrane Rating
18	17.4	Shoulder (D/S)	0.063	<0.04	<0.04	3.5
19	18.5	Shoulder (D/S)				
20	18.7	O.W.T. (D/S)	0.319	<0.04	<0.04	4.5
21	19.3	B.W.T. (D/S)				
22	19.5	I.W.T. (D/S)				
23	23.3	I.W.T. (A/S)				8.0
24	21.1	B.W.T. (A/S)				
25	19.6	O.W.T. (A/S)				
26	19.5	Shoulder (A/S)	0.164	<0.04	<0.04	5.0
27	19.3	I.W.T.	<0.04	<0.04	0.097	---
28	19.4	B.W.T.				
29	19.9	O.W.T.				
30	19.9	I.W.T.				
31	19.2	B.W.T.				
32	20.0	O.W.T.				
33	20.2	Shoulder	0.249	<0.04	<0.04	6.0
34	19.3	I.W.T.				
35	18.8	B.W.T.				
37	18.8	O.W.T.	<0.04	<0.04	<0.04	4.5

Table A-14. (*continued*)

Position I.D.	V-Meter Reading (μs)	Position Location*	Chloride Content† Cl(1) (lb/yd3)	Cl(2) (lb/yd^3)	Cl(3) (lb/yd^3)	Membrane Rating
38	20.2	Shoulder				
39	20.2	I.W.T. (D/S)				
40	21.0	B.W.T. (D/S)				
41	20.8	O.W.T. (D/S)	0.215	<0.04	<0.04	6.5
42	20.7	Shoulder (D/S)				

*Note: A/S: Entrance Span Area
B.W.T.: Between Wheel Tracks
C.L.: Center Line
D/S: Exiting Span Area
I.W.T.: Inside Wheel Track
O.W.T.: Outer Wheel Track
See "Pulse Velocity Versus Membrane Condition" in Chapter 4 for a detailed description of the rating system.

†1 lb/yd^3 = 0.59 kg/m^3 (approximately)

Table A-15. Test data for Hardwick, Vermont, Bridge No. 71.

Position I.D.	V-Meter Reading (μs)	Position Location*	Chloride Content† Cl(1) (lb/yd3)	Cl(2) (lb/yd^3)	Cl(3) (lb/yd^3)	Membrane Rating
1	20.8	Shoulder	0.324	0.342	0.395	6.0
2	21.3	Shoulder (A/S)				
3	21.5	Between Shoulder and O.W.T. (A/S)				
4	19.0	Between O.W.T. and B.W.T. (A/S)				
5	18.8	B.W.T. (A/S)	0.289	0.168	0.220	4.5
6	19.5	I.W.T. (A/S)				
7	19.6	I.W.T.				
8	19.5	B.W.T.				
9	20.4	Between B.W.T. and O.W.T.	0.289	0.220	0.254	5.5
10	20.5	O.W.T.				
11	21.8	Shoulder	0.272	0.272	0.307	7.0
12	20.7	I.W.T.	0.272	0.254	0.202	6.0
13	20.3	B.W.T.				
14	19.9	O.W.T.				
15	20.5	Between Shoulder and O.W.T.	0.254	0.168	0.220	5.5
16	20.7	O.W.T.				

Table A-15. (*continued*)

Position I.D.	V-Meter Reading (μs)	Position Location*	Chloride Content† Cl(1) (lb/yd3)	Cl(2) (lb/yd^3)	Cl(3) (lb/yd^3)	Membrane Rating
17	---	O.W.T.				
18	22.3	O.W.T.	0.151	0.202	0.151	---
19	19.5	Between C.L. and I.W.T. (D/S)				
20	19.2	I.W.T. (D/S)	0.067	0.083	0.067	5.0
21	18.8	B.W.T. (D/S)				
22	18.8	O.W.T. (D/S)	<0.04	<0.04	0.083	4.5
23	18.9	Between O.W.T. and Shoulder (D/S)				
24	20.5	Shoulder (D/S)				
25	20.2	Shoulder (D/S)				
26	18.9	Shoulder (A/S)				
27	18.7	O.W.T. (A/S)	<0.04	<0.04	0.083	4.5
28	18.7	B.W.T. (A/S)				
29	19.9	I.W.T. (A/S)	0.050	<0.04	<0.04	5.0
35	19.3	I.W.T.	0.151	0.100	<0.04	5.0

Table A-15. (*continued*)

Position I.D.	V-Meter Reading (μs)	Position Location*	Chloride Content† Cl(1) (lb/yd3)	Cl(2) (lb/yd^3)	Cl(3) (lb/yd^3)	Membrane Rating
38	19.2	O.W.T.	0.185	0.117	0.134	5.0
39	18.4	Between O.W.T. and Shoulder				
40	19.2	Shoulder				
41	19.3	Shoulder				
42	19.5	Between C.L. and I.W.T. (D/S)				
43	19.6	I.W.T. (D/S)				
44	19.9	B.W.T. (D/S)				
45	20.2	O.W.T. (D/S)				
46	21.2	Shoulder (D/S)	<0.04	<0.04	<0.04	6.5

*Note: A/S: Entrance Span Area
B.W.T.: Between Wheel Tracks
C.L.: Center Line
D/S: Exiting Span Area
I.W.T.: Inside Wheel Track
O.W.T.: Outer Wheel Track
See "Pulse Velocity Versus Membrane Condition" in Chapter 4 for a detailed description of the rating system.

†1 lb/yd^3 = 0.59 kg/m^3 (approximately)

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