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16. Abstract

This report is a synthesis of the technical results of Texas Department of Transportation Study 4069 ("Mitigation Techniques for In-Service Structures with Premature Concrete Deterioration"). The Texas Department of Transportation is interested in developing techniques for mitigating or remediating premature concrete deterioration due to alkali-silica reaction (ASR), delayed ettringite formation (DEF), or both, in order to extend the life of potentially affected structures. Reports 4069-1 and 4069-2 consist, respectively, of a literature survey and an application of existing test methods to assess the comparative effectiveness of mitigation treatments for premature concrete deterioration. This report (Report 4069-3) describes the development and verification of a new test method, and the application of that test method to recommendations for specific treatments to mitigate premature concrete deterioration from ASR/DEF.

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Mitigation Techniques for In-Service Structures with Premature Concrete Deterioration: Synthesis Report

by

Amy E. Eskridge, Jeremy T. Klahorst, Richard E. Klingner, and Michael E. Kreger

Research Report 0-4069-3

Research Project 0-4069

MITIGATION TECHNIQUES FOR IN-SERVICE STRUCTURES
WITH PREMATURE CONCRETE DETERIORATION

conducted for the
Texas Department of Transportation

in cooperation with the
U.S. Department of Transportation
Federal Highway Administration

by the

CENTER FOR TRANSPORTATION RESEARCH
BUREAU OF ENGINEERING RESEARCH
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DISCLAIMER

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R. E. Klingner, Texas P.E. #42483
M. E. Kreger, Texas P.E. #65541
Research Supervisors

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SUMMARY

This report describes part of the work associated with Texas Department of Transportation Study 4069 ("Mitigation Techniques for In-Service Structures with Premature Concrete Deterioration"). The Texas Department of Transportation is interested in developing techniques for mitigating or remediating premature concrete deterioration due to alkali-silica reaction (ASR), delayed ettringite formation (DEF), or both, in order to extend the life of potentially affected structures. The parts of Study 4069 reported here consist of: a review of existing test methods for possible application to evaluate mitigation treatments; development of a new test method, if necessary; verification of the test method; application of the selected test method to evaluate mitigation treatments; and recommendations of specific treatments to mitigate premature concrete deterioration from ASR/DEF.

CHAPTER 1:
INTRODUCTION

1.1 BACKGROUND OF TXDOT STUDY 4069

Recent investigations of concrete bridge structures throughout Texas have shown an increasing number that are deteriorating prematurely. This Premature Concrete Deterioration has been attributed to two expansive distress mechanisms: Alkali-Silica Reaction (ASR) and Delayed Ettringite Formation (DEF). While chemically different, both mechanisms require moisture to drive the expansive processes.

The structural effect of this premature concrete deterioration was investigated in Project 1857: “Structural Assessment of In-Service Bridges with Premature Concrete Deterioration.” Results from that project enable engineers to determine whether or not a deteriorating member retains sufficient structural capacity to remain in service. Once it has been determined that a deteriorating member can remain in service, it is necessary to apply a mitigating technique to control further expansion from ASR or DEF, separately or in combination. Determination of the most effective mitigation techniques is the objective of this TxDOT Study 4069.

Work began on this task in Fall 2000, and resulted in an extensive literature study and a laboratory investigation of the effectiveness of a number of mitigation treatments (Eskridge 2002). Several mitigation treatments were selected, and the performance of specimens with those treatments was compared with that of control specimens, using ASTM tests for ASR and DEF resistance. Several mitigation treatments, including that currently used by TxDOT, appear to have desirable characteristics. Neither standard nor modified ASTM tests, however, could distinguish with sufficient clarity between effective and ineffective mitigation treatments. To address this issue, Eskridge proposed an accelerated test with severe wetting and drying exposure. The remaining objective of this study is to complete the development of such a test, and to use it for further evaluation of possible mitigation treatments.

The test method is discussed extensively in subsequent chapters of this report. Its underlying logic is explained briefly here. Premature concrete deterioration as a result of ASR and DEF requires a certain level of internal relative humidity in concrete. Rather than evaluating mitigation treatments by directly monitoring ASR and DEF damage, the effectiveness of mitigation treatments can be evaluated indirectly by monitoring the internal relative humidity in concrete. In the work described, a specific test method embodying that logic is developed; is evaluated under controlled conditions; is refined; and is used to evaluate possible mitigation treatments. The test method, placed in ASTM format, is also offered to TxDOT for standardization within ASTM. The principal work product of this study is a recommendation for TxDOT on specific mitigation treatments.

The relevant results of Study 1857, and of previous work on Study 4069, are summarized below. They show the need for the current work of Study 4069 reported in this report.

1.1.1 Results of TxDOT Study 1857

The results of TxDOT Study 1857 (“Structural Assessment of In-Service Bridges with Premature Concrete Deterioration”) show the effects of Premature Concrete Deterioration (PCD) on the capacity of damaged structural elements. Both of these mechanisms are accelerated by exposure to water. Relevant conclusions of Study 1857 include the following:

- PCD is the result of an internal expansion mechanism in the concrete. Its basic causes are Delayed Ettringite Formation (DEF) and Alkali-Silica Reaction (ASR), which usually occur together in some degree. Both mechanisms are accelerated by exposure to water (Boenig 2000).

- PCD causes reduced shear strength, compressive strength and modulus of elasticity of prestressed girders (Boenig 2000); and reduced bond strength between prestressing strands and concrete (Memberg 2002)
- Reduced shear capacity of a prestressed concrete bridge girder with PCD can be estimated, or its service life predicted, using an experimentally established S-N curve (Roche 2001).
- Decreased girder stiffness, and potential decreases in strength and service life, can be correlated to increases in vertical deflection, total web deformation index, crack ratio, damage index, and maximum shear-crack width (Roche 2001).
- Failure mode of prestressed concrete girders subjected to shear fatigue overloads may be fatigue of the shear reinforcement in tension and transverse bending near diagonal cracks, or both. Failure occurs after shear cracks have opened significantly due to progressive deterioration (Roche 2001).
- PCD can be predicted using a visual damage index (Boenig 2000).
- Methods should be studied to slow or arrest PCD in existing structures. That is the objective of TxDOT Study 4069.

1.1.2 Previously Reported Results of TxDOT Study 4069

Previously reported results of TxDOT Study 4069 include the following (Eskridge 2002):

- Since PCD requires moisture, mitigation treatments should involve coatings that are vapor-permeable and water-impermeable;
- Existing test methods could not distinguish between effective and ineffective mitigation treatments:
  - indoor testing (accelerated ASTM C 1293 procedure), indicates vapor permeability only, and did not distinguish among treatments;
  - outdoor testing (specimens placed outside and subjected to actual field conditions) obviously depends on ambient relative humidity, which was not sufficient during the test period to provoke ASR or DEF. Relatively humidity of the specimens correlated poorly with ambient relative humidity.
  - a series of Wet/Dry (specimens exposed to drastic changes in humidity) indicates vapor permeability only.
- Study 4069 should be extended to develop a test method suitable for distinguishing between effective and ineffective mitigation treatments.

1.2 SCOPE AND OBJECTIVES OF TXDOT STUDY 4069

TxDOT Study 4069 is divided into six tasks:

1. Conduct an extensive literature search to identify treatments being used or tested worldwide to mitigate or remediate deterioration from ASR, DEF, or both. Compile the results as a bibliography and report of various mitigation or remediation techniques. Evaluate the published results and select the mitigation or remediation treatments to be used in Tasks 2 and 3.
2. Fabricate a large number of concrete specimens with a combination of aggregates and cement with high susceptibility to ASR, DEF, or both. Induce premature deterioration by exposing the specimens to cycles of wetting and drying and to heat.

3. Use non-destructive evaluation (NDE) procedures and physical testing to evaluate the effectiveness of the mitigation or remediation treatments chosen in Task 1 on the concrete specimens with premature deterioration.
4. Prepare a report on the results of Tasks 2 and 3, with a recommendation for mitigation or remediation treatments to be evaluated in the field.
5. After acceptance by TxDOT of the proposal from Task 4, apply the recommended treatments to field structures with documented ASR/DEF deterioration. The effectiveness of the treatments will be evaluated with the NDE procedure developed in Study 1857 and used in Task 3.
6. Prepare a comprehensive report summarizing the results of the entire project.

Remaining elements of TxDOT Study 4069 can be into four tasks:

a) Conduct an extensive search for a test method that can distinguish between effective and ineffective mitigation treatments for ASR and DEF, separately or in combination.

b) If no suitable test method is found, develop and verify a new one.

c) Use the test method to evaluate the effectiveness of proposed mitigation treatments.

d) Prepare a report on the results of Tasks 1-3, with a recommendation for mitigation treatments to be evaluated in the field.

CHAPTER 2:
LITERATURE SEARCH

2.1 INTRODUCTION

2.1.1 Objectives

This literature search was conducted to identify treatments being used or tested worldwide to mitigate or remediate deterioration from alkali-silica reaction (ASR), delayed ettringite formation (DEF), or both. The proposed treatments are evaluated, according to published results, for use on Texas Department of Transportation structures displaying this damage. In addition, treatments used to mitigate other types of concrete deterioration are explored for their potential benefit against ASR/DEF deterioration.

2.1.2 How this Literature Search was Conducted

The initial focus of this literature search was treatments specific to the mitigation or remediation of ASR, DEF, or both. In this report, "mitigation" refers to actions that reduce the rate at which deterioration occurs, while "remediation" refers to actions that completely arrest deterioration and restore all or part of the structure's original strength, durability, and appearance. The ideal literature would describe the effects and efficacy of different treatments in mitigating or remediating ASR/DEF deterioration. Studies on treatment of ASR, though, are not extensive, and literature on treatment of DEF is very limited. The next step in this literature search, therefore, was to examine the known causes of ASR/DEF, and to investigate actions that have shown some success in addressing those causes.

The second focus of the search was treatments specific to the mitigation or remediation of other forms of concrete deterioration, such as sulfate attack, chloride ingress, and carbonation. The fundamental mechanisms of these forms of deterioration were studied, and literature was gathered on available methods of mitigation and remediation for them, including coatings, membranes, sealers, and electrochemical processes.

2.2 METHODS FOR MITIGATING OR REMEDIATING ASR

2.2.1 Causes of ASR[1]

ASR is a reaction between siliceous aggregate and high-alkali pore water in the surrounding cementitious matrix. A high alkali concentration in the pore water provides the hydroxyl ions that react with the silica to form a gel at the cementitious matrix and aggregate interface. This gel grows as it absorbs water from the environment, consequently generating expansive forces that can produce map cracking or surface pop-outs.

ASR deterioration requires the following conditions:

- high alkali concentration in the pore water;
- aggregate with reactive silica; and
- water.

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[1] Klingner, R.E., T.J. Fowler, and M.E. Kreger, "Mitigation Techniques for In-Service Structures with Premature Concrete Deterioration," proposal to Texas Department of Transportation, 2000.

2.2.2 The Goal of Mitigation or Remediation Methods for ASR

The goal for treating existing ASR-affected structures is to prevent water infiltration, one prerequisite for the reaction. At the same time, the treatment should permit the escape of water already in the structure, so that it does not continue to promote the reaction. Accordingly, the treatment, whether a penetrating coating or an encapsulation, must be impermeable to liquid water and permeable to water vapor.

2.2.3 Published Information on Mitigation or Remediation of ASR

2.2.3.1 ASR Mitigation Reference No. 1 (Abe et al. 1992)

This article addresses the comparative effectiveness of two coatings, one impermeable to water and the other permeable to water vapor, in reducing ASR-related expansion. The impermeable coating consisted of three layers of epoxy. The vapor-permeable coating consisted of silane followed by a flexible polymer-modified cement mortar (PCM). The control specimens were uncoated.

All specimens were placed outside for two years. Specimens with the vapor-permeable coating showed continuous negative expansion, whereas after six months the specimens with the impermeable coating had much greater expansion than the uncoated specimens. The investigators attribute this high expansion to the excess initial pore water that could not escape through the impermeable epoxy coating.

2.2.3.2 ASR Mitigation Reference No. 2 (Kamimoto et al. 1992)

The study described in this article measured the performance of several concentrations of a PCM using the criteria of water permeability, water-vapor permeability, elongation, adhesion, and expansion of a concrete specimen in the field. Water permeability and water-vapor permeability decreased with increasing polymer ratio, with the lowest permeability corresponding to the greatest tested polymer ratio, 0.75. Elongation of the PCM increased as the polymer ratio increased. Adhesion was greatest for a polymer ratio of 0.525.

For the field expansion tests, small, rectangular specimens were coated with either PCM or epoxy, while other specimens were left uncoated. Expansion was measured by change in length, and vapor permeability was measured by change in weight. The PCM-coated specimens had consistently low expansion, while the uncoated and epoxy-coated specimens had much higher overall expansion and greater rates of expansion. As the water-vapor permeability of the PCM increased, the specimens' expansion decreased.

2.2.3.3 ASR Mitigation Reference No. 3 (Fujii et al. 1989)

The specimens in this study were subject to outdoor conditions and cycles of wetting and drying, considered the most severe environmental conditions for Japan. The coatings were applied to newly constructed specimens when their moisture content had reduced to 10%. In the outdoor series, silane- and urethane-coated specimens had expansion equivalent to that of a non-reactive specimen, actually showing negative expansion. Epoxy-coated and methyl-methacrylate-coated specimens expanded severely and the coatings cracked. Sodium silicate-coated specimens showed expansion equivalent to that of the uncoated reactive specimens. All specimens had very high expansion under cycles of wetting and drying.

Expansion was found to be related to ratios of surface area to volume and treated surface area to total surface area. As those ratios increase, expansion decreases. It was concluded that structures with large ratios of surface area to volume would especially benefit from surface treatment.

The final series of tests was a comparison of the performance of silane, silane with a PCM cover, and silane with a methyl-methacrylate cover under cycles of wetting and drying. Silane/PCM-coated specimens had four times the expansion of specimens with the other two coatings after 32 weeks of exposure, but still less than all specimens from the first series of tests.

2.2.3.4 ASR Mitigation Reference No. 4 (Stokes 2000)

In this article the use of a lithium-based solution to treat ASR was described. Tests were conducted to compare the penetration ability of various lithium solutions, to assess the efficacy of the best solution, and to study how the timing of the treatment influenced this efficacy.

Penetration ability was assessed by placing various lithium salt solutions at several concentrations in cavities in cylinders, and then recording the volume of solution entering the cylinder. The greatest penetration was achieved with a 30% lithium nitrate solution with a blend of surfactants, surpassing the penetration of lithium hydroxide, formate, and acetate.

Reactive mortar bars and concrete prisms were then used to study efficacy and application timing. In reactive mortar bars, one-half the amount of lithium required as an admixture to control ASR reduced expansion to as little as 55% of that of uncoated control specimens. Also, lithium nitrate reduced expansion twice as much as lithium hydroxide. The lithium nitrate was used on concrete prisms, applied in one and five coats. The one-coat specimens exhibited 0.1% expansion and the 5-coat specimens exhibited 0.05% expansion. The investigators concluded from the timing tests on both mortar bars and concrete prisms that some prior expansion aided penetration, and thus effectiveness, by inducing cracking. Existing cracks provided a path for the coating to penetrate.

2.2.3.5 ASR Mitigation Reference No. 5 (Whitmore et al. 2000)

Electrochemical chloride extraction, used to drive chloride ions out of salt-contaminated structures, can easily be adapted to drive lithium ions into a structure. The potential benefits are shortened treatment time and an increase in the effective amount of lithium in the structure.

The anode for the process is a titanium-coated metallic mesh, the same as is often used for cathodic protection and chloride extraction. Reinforcement in the structure is the cathode. The impressed current comes from AC/DC rectifiers, which convert high-voltage AC to low-voltage DC. Lithium solutions supply the lithium ions and act as the electrolyte providing electrical continuity between the anode and cathode. An electric field is created between the mesh and reinforcement. Lithium, being a positive ion, is driven away from the mesh and toward the reinforcement, and is thus distributed in the concrete.

Field application to bridge decks in Virginia and Delaware, carried out by the investigating companies, showed rapid migration of the ion into the concrete in the first week of treatment. Each treatment period lasted eight weeks. No samples were taken to determine the total lithium content at the end of treatment.

2.2.3.6 ASR Mitigation Reference No. 6 (Baillemont et al. 2000)

This article describes the diagnosis, treatment and monitoring of an ASR-affected bridge in northeastern France. Considerable map cracking was found on all the piles and one deck. Measurements on several cracks from 1982 through 1995 indicated continuing opening.

A treatment of silane followed by a thin polymer-cement coating was applied to the piles and deck. This combination was chosen for its flexibility, good adhesion to concrete, and resistance to de-icing salts. Sensors were installed over six cracks to record opening. Temperature was also recorded. After one year of measurement, the width of the cracks was seen to vary only with temperature, indicating that the reaction had been slowed. Measurements will be continued for several years.

2.3 METHODS FOR MITIGATING OR REMEDIATING DEF

2.3.1 Causes of DEF[2]

Ettringite, a normal hydration product, is a reaction between sulfates, calcium aluminates and water. Primary ettringite, which forms before the concrete sets, is not deleterious. Damage is caused by DEF in the hardened concrete. Delayed ettringite forms from a reaction between decomposed primary ettringite and water, creating nests of ettringite in the paste. Research indicates that high amounts of sulfur, contributed by the clinker, may be another source of the reaction. In both cases, exposure to water over time causes the ettringite to reform, producing expansive forces. Hime (1996) claims that the reaction requires substantial water exposure over months or years, but other sources say ambient temperature and moisture are sufficient for the reaction (G.M. Idorn 2001). DEF, like ASR, shows as map cracking.

DEF deterioration requires the following conditions:

- decomposed ettringite or high amounts of sulfur; and
- water.

2.3.2 The Goal of Mitigation or Remediation Treatments for DEF

The goal of treating existing DEF-affected structures is the same as for ASR-affected structures -- prevent water infiltration and allow water vapor release.

2.3.3 Published Information on Mitigation or Remediation of DEF

No references were found for mitigation or remediation of existing DEF deterioration.

2.4 OTHER TYPES OF DETERIORATION IN REINFORCED CONCRETE

2.4.1 Damage to Concrete

In addition to ASR/DEF deterioration, concrete is susceptible to damage from sulfate attack, salt crystallization, and freeze/thaw cycling. These mechanisms involve direct damage to the cementitious matrix. They are discussed here because some techniques to mitigate or remediate them may be relevant to ASR/DEF.

2.4.1.1 Sulfate Attack

Sulfate attack is caused by water-soluble sulfates entering the concrete. The sulfates react either chemically or physically with calcium aluminate hydrates in the paste, causing the cementitious matrix to weaken and disintegrate (Mehta *et al* 1993).

2.4.1.2 Salt Crystallization

Salt crystallization is the physical action of salts crystallizing out of solution in the pores of concrete. The pressure of the salts in the pores can be great enough to cause cracking and scaling (local delamination) of the concrete surface (Mehta *et al* 1993).

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[2] Merrill, Brian D., *Premature Concrete Deterioration*, internal report, Texas Department of Transportation, 1997.

2.4.1.3 Freeze/Thaw Cycling

Freeze/thaw damage occurs by mechanisms similar to those of salt crystallization. Water freezing in the pores generates large hydraulic pressures, causing scaling and cracking. The use of de-icing salt magnifies freeze/thaw damage. Salt is hygroscopic, absorbing and retaining water in the pores, and thereby promoting a higher degree of saturation. Also, salt-concentration gradients in the concrete promote partial or differential freezing, generating osmotic pressures (Mehta *et al* 1993).

2.4.2 Corrosion of Reinforcement

Reinforced concrete elements can also deteriorate because of corrosion of their reinforcement. Corrosion of reinforcement deteriorates the concrete by filling it with a volume of corrosion products exceeding that of the original reinforcement, thereby inducing tensile stress in the surrounding concrete. Corrosion also reduces the cross-sectional area of the reinforcement and the bond between the concrete and reinforcement, thereby reducing the strength of the member.

Corrosion itself is an electrochemical process, requiring an anode, a cathode, an electrical conductor, and an electrolyte. The reinforcement acts as anode, cathode, and conductor, while any water in the concrete is the electrolyte. Normally, a passive layer of oxide film, maintained by the alkaline concrete, protects the steel. Corrosion of reinforcement is accelerated when the passivating layer is disrupted, which can occur by ingress of chlorides, carbonation, or electrolytic solutions (ACI 222 1996).

2.4.2.1 Ingress of Chlorides

The ingress of chlorides reduces the passive protective layer around the reinforcement. Pitting-type corrosion is produced on the steel surface where the passive layer is diminished by chlorides in the presence of oxygen and moisture. Chlorides diffuse into the concrete in solution, and can come from de-icing salts, fog, mist, or marine spray (Leeming 1990).

2.4.2.2 Carbonation

Concrete is generally very alkaline, providing an immune surrounding and maintaining the passivity. In the carbonation reaction, carbon dioxide from the air diffuses into the concrete and reacts with the cement to reduce this alkalinity. Below a pH of 9-10 the passive layer on the reinforcement breaks down and general corrosion begins. For carbonation to occur the concrete must be moist, but not saturated (Leeming 1990).

2.4.2.3 Lowered Resistivity

The corrosion process requires a conducting electrolyte. Dry concrete is highly resistive and prevents corrosion of the reinforcement. Conversely, pore water with dissolved salt is an electrolyte providing very low resistance for corrosion current. Reinforcement in wet concrete corrodes at a much higher rate than in dry concrete (Leeming 1990).

2.4.3 Relation of Other Types of Deterioration to ASR/DEF Deterioration

In all the types of concrete deterioration discussed above, water is the common factor.

- For freeze/thaw cycles and lowered resistivity, water is the root of the problem.
- Sulfate attack, salt scaling, and ingress of chloride all require water to transport the sulfate, salt, or chlorides that are the cause of the deterioration.
- Water is the agent that allows CO_2 to create carbonation damage.

Similarly, an external source of water is required for ASR/DEF deterioration. Many of the mitigating or remediating treatments for sulfate attack, salt scaling, freeze/thaw cycling, ingress of chlorides, carbonation, and lowered resistivity seek to prevent water infiltration, and therefore may be applicable as treatments for ASR/DEF deterioration. These treatments are discussed in Section 2.5.

2.5 SURFACE TREATMENTS FOR MITIGATING OR REMEDIATING OTHER TYPES OF DETERIORATION IN REINFORCED CONCRETE

A large body of literature has been accumulated over many years related to surface treatments, penetrating sealers, epoxies, and crack sealers for the purpose of keeping water out of concrete and thereby mitigating or remediating concrete deterioration. These surface treatments are discussed here for their potential benefit in mitigating or remediating ASR/DEF deterioration.

2.5.1 Coatings and Membranes

Coatings and membranes include epoxies, polymer cements, and urethanes. All of these provide a layer on the surface of the concrete. Membranes are impermeable to water, while coatings may or may not be impermeable.

2.5.2 Published Information on Coatings and Membranes

2.5.2.1 Coating Reference No. 1 (ACI 515-1985)

Chapter 4 of ACI 515 deals with waterproofing barrier systems, which are coatings intended to completely prevent the movement of moisture into concrete. Traditional systems use hot-applied, bituminous-saturated felt. Today many cold-applied systems are also used, including bituminous materials, elastomeric membranes, cementitious membranes, and metallic oxides.

Both cold and hot bituminous systems use fabrics for strength. They are not as adaptable to irregular shapes as cold liquid systems.

Chapter 4 of ACI 515 lists neoprene, neoprene-bituminous blends, polyurethane, polyurethane-bituminous blends, and epoxy-bituminous blends as elastomeric membranes. All have good elongation characteristics. Some need fiberglass cloth for reinforcement or to bridge cracks and joints larger than 1.5 mm (0.06 in.). All are liquid-applied systems, requiring multiple coats for complete coverage.

Cementitious membranes can be applied to damp, smooth, rough, or irregular surfaces, usually by trowel. Because cementitious membranes are rigid, they should have the same thermal coefficient as the substrate on which they are applied to minimize differential thermal expansion and resultant cracking. Also, cementitious membranes should not be used to span moving joints.

Metallic oxides are another type of rigid barrier. Fine metallic filings, sand, cement, water, and a catalyst oxidize to fill the pores of the concrete, decreasing permeability. Metallic oxides require 3 to 5 coats, constant moist curing for a proper finish, and good quality control.

2.5.2.2 Coating Reference No. 2 (O'Donoghue et al. 1998)

Moisture-cured urethanes react with ambient moisture to create polymers. For this reason they are inherently beneficial in applications where moisture is present. Their curing temperature is as low as 20°F (-7°C). Moisture-cured urethanes can cure very rapidly, however, preventing penetration into the concrete surface. In such cases, the urethane acts more like a coating than a penetrating sealer.

2.5.3 Penetrating Sealers

Penetrating sealers are solutions or suspensions that diffuse into the concrete near the surface. These include silane, siloxane, oils, high-molecular-weight methacrylate (HMWM), and penetrating epoxies. While not impermeable to liquid water, they create a hydrophobic layer, sometimes (as in the case of silane and siloxane) by chemical reaction with the concrete. Because they are clear, penetrating sealers offer the advantage of permitting continued observation of the concrete surface.

2.5.4 Published Information on Penetrating Sealers

2.5.4.1 Penetrating Sealer Reference No. 1 (ACI 515-1985)

Chapter 5 of ACI 515 deals with dampproofing barrier systems, which are coatings capable of reducing the rate of transmission of water into the concrete, but are not impermeable to liquid water. Dampproofing is suitable for areas not subjected to hydraulic pressure, while waterproofing is necessary if hydraulic pressure is present. Chapter 5 claims that dampproofing systems are not capable of bridging cracks. Advantages are low application cost and minimal surface preparation. Multiple coats are needed for complete coverage. Above-grade systems are listed as: water-based portland cement paste; portland cement with stearic acid, water repellants, or latex emulsions; latex paint; two-component epoxy paint; solvent-based, chlorinated rubber paint; two-component or moisture-cured polyurethane paints; and fish oil-based materials with mica and asbestos fillers.

Chapter 6 of ACI 515 deals with protective barrier systems, which provide resistance to degradation by chemicals, prevent staining, and prevent liquids from being contaminated by the concrete. These systems are more durable than waterproofing or dampproofing barriers. Chapter 6 categorizes protective barrier systems as providing protection against mild, intermediate, and severe exposures. The category most related to mitigation and remediation of ASR/DEF deterioration is “mild,” which addresses damage from de-icing salts, freeze/thaw cycling, and acidic solutions. Applicable systems are a mixture of moisture-permeable and impermeable sealers, including: polyvinyl butyral; polyurethane; methyl-methacrylate; alkyl-alkoxysilane; epoxy resins; acrylic resins; chlorinated rubber; styrene-acrylic copolymer; asphalt; coal tar; vinyl; and neoprene.

2.5.4.2 Penetrating Sealer Reference No. 2 (Wright et al. 1993)

In this investigation boiled linseed oil was compared to silane and siloxane for penetration ability, salt-water absorption, vapor permeability and other characteristics. Each sealer was applied to three concrete pavement sites for the field investigation. Cores were taken from each site to determine the penetration depth, salt-water absorption, and chloride intrusion. Laboratory specimens were cubes, prisms, and slabs, used for tests of sealer penetration, salt-water absorption, vapor permeability, abrasion, chloride intrusion, and freeze/thaw resistance.

Penetration of the linseed oil was comparable to that of the other sealers in the field tests, and two or more times greater in the laboratory tests. Linseed oil performed best at reducing salt-water absorption in the field, although absorption results in the laboratory were comparable to silane and worse than siloxane. The investigators also found that in the field, the volume of silane and siloxane in the pore structure of the concrete diminished over time, allowing more absorption. In contrast, the linseed oil maintained consistently low absorption. Chloride-ion content was lower for the linseed-oil cores both two and three years after application. Finally, linseed oil was less permeable to water vapor than the other sealers.

2.5.4.3 Penetrating Sealer Reference No. 3 (Marks 1988)

This article reports on the Iowa Department of Transportation’s use of HMWM to seal the entire deck of a bridge that had full-depth cracks that were observed to leak. The cracks were found to lie above the transverse reinforcement, and the Department decided that sealing of the deck was needed to protect the steel. According to Marks, HMWM was chosen because the California Department of Transportation had successfully used it as a sealer.

One coat of HMWM was applied and cores were taken one week later. The HMWM penetrated the full 2 in. (51 mm) of every core. Continued leakage, though at a lower rate, was observed along the entire deck during a steady rain and under standing water. The Department applied a second coat, but observations had not been made at the time of the article. The initial application was made when the temperature ranged between 45F and 55F (7C and 13C); the HMWM manufacturer suggested that the application temperature should be 50F (10C) or above. The second application was made at 60F (16C).

2.5.4.4 Penetrating Sealer Reference No. 4 (Basheer et al. 1998)

The article studied the effect of moisture in the concrete at the time of surface treatment application on the chloride intrusion and subsequent reinforcement corrosion. The surface treatments used were a 40% silane solution, a 100% silane solution, a silane-siloxane two-coat system, and a silane-acrylic two-coat system. Three moisture conditions were used: one very wet with little drying time; one very wet with considerably longer drying time; and one very dry. The specimens were repeatedly subjected to 7-day cycles of ponding with a sodium-chloride solution, followed by drying.

The depth of penetration of the treatments was greatest for the specimens that were driest. Very few specimens had corrosion of reinforcement, but those that did were from the group with the highest pre-application moisture condition. The investigators concluded that while all treatments reduced chloride infiltration, no one treatment was outstanding in this regard.

2.5.4.5 Penetrating Sealer Reference No. 5 (Rizzo et al. 1989)

This article investigated many penetrating sealers, including acrylic, polyurethane, gum resin, silicone, silane, and an acrylic topcoat for their permeability to water vapor and their resistance to absorption of water, ingress of chloride ions, and ingress of carbon dioxide. The silane sealers showed the lowest water absorption and chloride-ion ingress, although carbon-dioxide ingress was comparable to that of the other sealers.

A system of silane with an acrylic topcoat was then tested. It showed the positive benefits of the silane – reduced water absorption and chloride-ion ingress – while also reducing the ingress of carbon dioxide and remaining breathable. The silane-acrylic system was subjected to additional sulfate exposure, accelerated weathering, and freeze-thaw testing, and showed good results over the uncoated specimen. Cores from field applications on a tunnel in the United Kingdom exposed to frequent applications of de-icing salts confirmed that the silane-acrylic system substantially reduced the ingress of chloride ions.

2.5.4.6 Product Literature on Penetrating Sealers

In contrast to experimental findings reported here, some product literature does advertise epoxy-based penetrating sealers that are supposedly impermeable to liquid water and permeable to water vapor.[3]

2.5.5 Crack Sealers

Crack sealers are low-viscosity, flexible polymers applied specifically to cracks in reinforced concrete. Ideally, they penetrate the crack completely, thus eliminating an easy path for water entrance, and also restore structural strength to the member. Crack sealers include HMWM, epoxies, and urethanes.

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[3] Epoxy Systems™ Product #850

2.5.6 Published Information on Crack Sealers

2.5.6.1 Crack Sealer Reference No. 1 (Sprinkel et al. 1995)

In this article three epoxies, one HMWM, and one polyurethane were tested for penetration depth and flexural strength. Unreinforced beams measuring 3 x 4 x 11 in. (76 x 102 x 279 mm) were tested in flexure to failure and then repaired by applying one of the five crack sealers to the top surface of the beam while the sections were held together in a jig. Sealer was ponded on the top surface until the crack was entirely filled. After curing, the beams were again tested to failure in flexure and then split to measure penetration depth of the polymer. Additional penetration tests were conducted at varying temperatures, between 7°C and 35°C (45°F and 95°F), by pouring the polymers over sand. Under ideal temperatures, all five sealers filled cracks as narrow as 0.2 mm. Each of the five sealers reacted uniquely to high and low temperatures, with the HMWM performing consistently well over the entire range. All polymers restored 100% of original strength in laboratory flexural tests. Most test beams re-cracked in the concrete, rather than in the sealer. The investigators rated HMWM first for tested results, and third for overall results, ease of use, odor, safety, and cost.

The investigators found that while the polyurethane was easy to use and almost odor-free, it easily leaked and was not practical for overhead surfaces. The three epoxies ranged from low to very strong in odor, and from easy to difficult mixing ratios. The HMWM had very high odor, easily leaked because of low viscosity, was difficult to mix, and was potentially explosive if mixed incorrectly.

2.5.6.2 Crack Sealer Reference No. 2 (Fowler 1989)

The results of this investigation agree with those of Marks (1988). For this investigation, 4 x 6 x 12 in. (102 x 152 x 305 mm) slabs were cracked and then treated with three different HMWM. Slabs were treated under three conditions: dry, room temperature; dry, hot temperature; and wet, room temperature. HMWM restored 75-85% of the flexural strength under all conditions. HMWM filled 60-80% of the crack volume for the laboratory tests and in the cores taken from the treated field site. Cracks as small as 0.1 mm (0.004 in.) were filled. The investigators recommend 3 days drying time under normal conditions to achieve the best crack filling and strength gain.

2.6 ELECTROCHEMICAL TREATMENTS

The purpose of electrochemical treatment is to reverse corrosion damage of the reinforcement, with no direct benefit to the surrounding concrete and no attempt to prevent water infiltration. While these treatments do not remediate ASR/DEF deterioration, one finding in particular is worth noting.

Cathodic protection is the most common electrochemical method of arresting or reducing the corrosion of steel reinforcement. Cathodic protection produces hydroxyl ions, which can combine with reactive aggregates to cause ASR. The use of cathodic protection can therefore easily worsen an existing ASR problem (Thompson 1990).

2.7 SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS FROM LITERATURE SEARCH

2.7.1 Summary and Conclusions

While the articles referenced in this search do not comprise a complete list of mitigation and remediation treatments for ASR/DEF or other forms of concrete deterioration, they cover the range of methods currently researched and tested, and also contain important concepts. Many articles that corroborate the information given above or provide background are listed in Appendix G.

Polymer-modified cement mortar (PCM), silane, urethane, and lithium nitrate were found to be effective in reducing expansion from ASR. In some tests, the products were used as two-coat systems, such as silane with a PCM topcoat, with good results. Several references, however, report that epoxy promotes expansion. Methyl-methacrylate and sodium silicate are also not effective at reducing expansion.

Lithium can be used either in an applied solution or in an electrochemical process. Lithium nitrate is more effective and safer to use than lithium hydroxide. In the electrochemical process, lithium ions are driven into the concrete toward the reinforcement. The benefit of this process is an increase in the amount of useful lithium deposited in the concrete. Lithium is successful at reducing ASR expansion, but because it is not a hydrophobic sealer, it does not have the added benefit of protecting against other forms of deterioration.

Membranes and impermeable coatings promote expansion in existing concrete structures. They are therefore not appropriate for mitigation or remediation of ASR/DEF. The treatments described in Chapter 4 of ACI 515 are waterproofing membranes.

The treatments described in Chapters 5 and 6 of ACI 515 can be permeable or impermeable to liquid water. Many of the systems listed are discussed in other references in this report, including modified Portland cement, epoxy, polyurethane, methyl-methacrylate, silane, and acrylic resins.

Penetrating hydrophobic sealers have the greatest potential for controlling expansion from ASR/DEF. While not completely impermeable to water, they are permeable to water vapor. Silane, already mentioned as a specific ASR treatment, has been found to reduce chloride-ion content. Silane was especially effective at reducing chloride- and sulfate-ion ingress, carbon-dioxide intrusion, and weathering when applied with an acrylic topcoat. Silane systems remain breathable.

Boiled linseed oil performed as well or better than silane and siloxane in tests for salt-water and chloride intrusion. Linseed oil is inexpensive, but may need more frequent reapplication than other penetrating sealers.

Moisture-cured urethanes have promise for treating existing structures because of their need for moisture. Controlling the rate of cure so that moisture-cured urethanes can penetrate the concrete surface may improve their effectiveness at reducing expansion from ASR/DEF.

High-molecular-weight methacrylate (HMWM) has been reported as both a penetrating sealer and crack sealer. In Marks (1988), HMWM could not prevent leakage through the bridge deck. The poor results of the first application can possibly be attributed to the cool application temperature. As a crack sealer, the HMWM penetrated very small cracks and restored structural strength. Only sealing cracks in ASR/DEF-affected structures is not beneficial, however, because new cracks will inevitably form. Also, Stokes (2000) suggests that some cracking aids the penetration of the sealer.

2.7.2 Recommendations

To mitigate deterioration from ASR, DEF, or both, it is recommended to use the following treatments:

- Silane, PCM, urethane, and lithium nitrate solution, which reduce the expansion of ASR-affected specimens.
- Siloxane, which is similar in chemical makeup to silane, and performs like it in chloride-intrusion tests.
- Linseed oil, which performs as well as silane and siloxane in salt-water and chloride intrusion tests, and is much cheaper.
- HMWM, which has good penetration ability, but needs more research on its effectiveness as a penetrating sealer.

- Epoxy, which has shown negative results as a coating, but may be worth investigating as a penetrating sealer. It is safe to use and requires little or no re-application. It can also be used as a topcoat following treatment with lithium nitrate, silane, etc.

It is recommended to use these treatments, separately or in combination, on specimens made with aggregates and cement known to produce ASR/DEF deterioration, and to evaluate their effectiveness by measuring specimen expansion and internal moisture content, and by monitoring specimens with acoustic emission techniques.

2.7.3 Recommendations for Specific Structures

The mitigation options presented here apply to structures in general. Characteristics of specific studies may favor particular options or sets of options. One example of this, the Lake Ivie structure on FM 1929 is discussed in theses and reports for Study 1857. Such specific applications are not discussed further here.

CHAPTER 3:
SEARCH FOR SUITABLE TEST METHOD

3.1 OBJECTIVE OF SEARCH FOR A SUITABLE TEST METHOD

Previous research in Study 4069 used direct test methods involving the evaluation of damage under accelerated exposure. Results of that research show that those test methods are inadequate in determining the effectiveness of mitigation treatments (Eskridge 2002). The work of Study 4069 described here involves the development of indirect test methods. In this chapter, those indirect test methods are described in general.

Premature concrete deterioration associated with ASR and DEF progresses faster under conditions of high internal relative humidity of concrete (Boenig 2000). It should therefore be possible to evaluate the relative effectiveness of proposed mitigation treatments indirectly, by comparing the extent to which they limit the internal relative humidity of concrete subjected to cycles of wetting and drying. During the wetting stage of a cycle, an ideal mitigation treatment would keep the internal relative humidity of a concrete specimen lower than that of an untreated control specimen. During the drying stage, an ideal mitigation treatment would permit a treated specimen to lose internal moisture at the same rate as an untreated control specimen. Under repeated cycles of wetting and drying, a specimen with an ideal mitigation treatment would stay consistently drier inside, than an untreated control specimen.

An exhaustive search was conducted to find an appropriate existing indirect test method to determine the effectiveness of proposed mitigation treatments. An appropriate test method would include repeated cycles of wetting and drying. During the wetting stage of the cycle, the test would measure water impermeability; during the drying stage, the test would measure vapor permeability.

3.2 SEARCH FOR APPROPRIATE ASTM TEST METHODS

One developer of test methods is the American Society for Testing and Materials (ASTM). Table 3.1 lists potentially applicable ASTM test methods, each of which was investigated to see if it met any or all of the three prerequisites required for an adequate indirect test.

Table 3.1 Potentially applicable ASTM test methods

ASTM C 33	ASTM C 88	ASTM C 151	ASTM C157	ASTM C 227
ASTM C 231	ASTM C 289	ASTM C 295	ASTM C 341	ASTM C 441
ASTM C 490	ASTM C 511	ASTM C 586	ASTM C 642	ASTM C 666
ASTM C 827	ASTM C 878	ASTM C 940	ASTM C 948	ASTM C 1038
ASTM C 1084	ASTM C 1105	ASTM C 1218	ASTM C 1260	ASTM C 1293
ASTM D 545	ASTM D 1734	ASTM D 1985	ASTM E 96	ASTM E 154
ASTM E 337	ASTM G 109			

While most of the above test methods address water impermeability or vapor permeability, none involves investigation of those characteristics under cycles of wetting and drying. It was concluded that no existing ASTM test method met the prerequisites for this study.

3.3 SEARCH FOR APPROPRIATE AASHTO TEST METHODS

Another developer of test methods is the American Association of State Highway and Transportation Officials (AASHTO). Table 3.2 lists potentially applicable AASHTO test methods, each of which was examined to see if it met any or all of the three prerequisites for an adequate indirect test.

Table 3.2 Potentially applicable AASHTO test methods

AASHTO T42	AASHTO T50	AASHTO TP11	AASHTO TP60
AASHTO T259	AASHTO T260	AASHTO T299	AASHTO M201

While most of the above AASHTO test methods address water impermeability or vapor permeability, none involves investigation of those characteristics under cycles of wetting and drying. It was concluded that no existing ASTM test method was met the prerequisites for this study.

3.4 OTHER POSSIBLE SOURCES OF EXISTING TEST METHODS

A German test method, DIN 52-617, was also investigated. Its objective is to determine the water-absorption coefficient, which characterizes the amount of water absorbed per unit surface area as a result of capillary or absorptive action in a construction material whose surface is wetted under normal pressure conditions. Its protocol includes immersing specimens in a water bath. While this test method addresses water impermeability, it does not include a drying exposure that would address vapor permeability, nor does it subject specimens to cyclic exposure. Therefore, DIN 52-617 was judged inadequate for this study.

3.5 RESULTS AND CONCLUSIONS OF SEARCH FOR SUITABLE TEST METHOD

No ASTM, AASHTO or DIN test method was judged suitable for use as an indirect test in this study. For this reason, it was judged necessary to develop a new indirect test method that would address water impermeability and vapor permeability, under conditions of cyclic wetting and drying.

CHAPTER 4:
DEVELOPMENT AND VERIFICATION OF A SUITABLE INDIRECT TEST METHOD

4.1 DEVELOPMENT OF NEW TEST METHOD

4.1.1 Objectives

A new test method to compare the efficacy of proposed mitigation treatments for premature concrete deterioration must be able to determine the extent to which a possible mitigation treatment is water-impermeable, and the extent to which it is vapor-permeable. The initial development and verification of such a test method is the subject of this chapter.

4.1.2 Hypothetical Test Method

Previous work in Study 4069 (Eskridge 2002) had shown that direct test methods involving evaluation of damage under accelerated exposure were tedious and often inconclusive. This suggested that an indirect test method might be more suited for the current work of Study 4069. Since the destructive reactions associated with ASR and DEF require a certain level of internal relative humidity in the concrete, it should be possible to evaluate the relative effectiveness of proposed mitigation treatments indirectly, by comparing the extent to which they limit the internal relative humidity of concrete subjected to cycles of wetting and drying. During the wetting stage of a cycle, an ideal mitigation treatment would keep the internal relative humidity lower than that of an untreated control specimen. During the drying stage, an ideal mitigation treatment would permit a treated specimen to lose internal moisture at the same rate as an untreated control specimen. Under repeated cycles of wetting and drying, a specimen with an ideal mitigation treatment would stay consistently drier inside, than an untreated control specimen.

This logic is shown schematically in Figure 4.1 for a control specimen and two hypothetical mitigation treatments, one effective and the other ineffective. As ambient relative humidity is cycled from wet to dry and back again, the effectively treated specimen loses moisture at about the same rate as an untreated specimen, and gains moisture more slowly. The ineffectively treated specimen, in contrast, does not differ significantly from the untreated specimen as it is cycled.

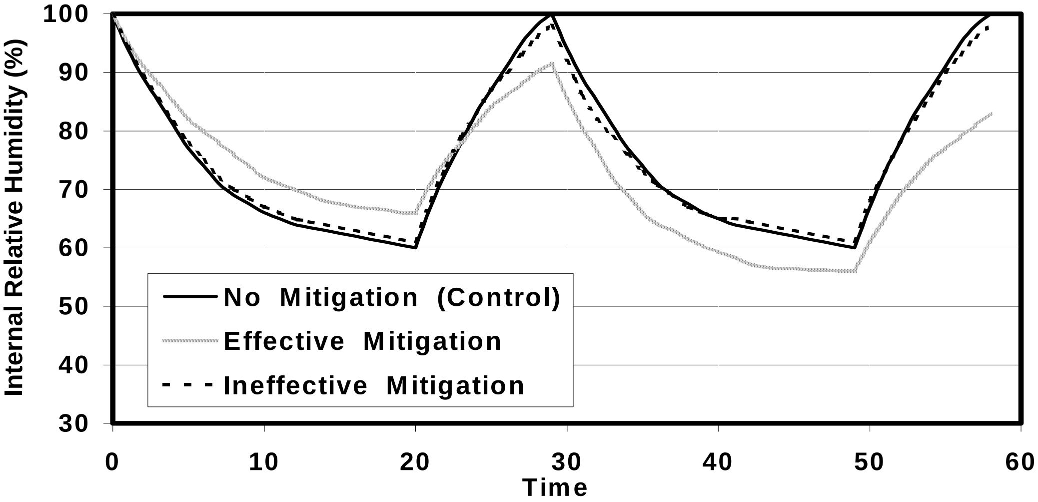

Figure 4.1 Hypothetical application of test method

4.1.3 Selection of Relative-Humidity Levels for Hypothetical Test Method

The hypothetical test method involves cycling between wetting and drying stages. In developing the test method, it is necessary to establish the relative humidity to which the specimens are exposed in each stage. That is the subject of this section.

The relative humidity for the wetting stage can logically be established as 100%, because the objective of the wetting stage is to subject specimens to as rapid wetting as possible. This is easily achieved by submersing the specimens in water, at a very shallow depth. During that submersion, the internal relative humidity of untreated specimens would approach 100%. The wetting stage would end when the internal relatively humidity was sufficiently stable and sufficiently close to 100%. The use of submersion rather than other conditions for the wetting stage was justified as follows. From field studies, it is apparent that field conditions are not replicated by exposure to ambient high humidity, because field structures show much higher deterioration in the presence of liquid water, either running over the structure (underneath the scuppers of the San Jacinto structure) or blown against the structure as driving rain (on the south faces of the exterior girders of the Beaumont structure). If specimens with a depth of 2 in. were barely submerged in water, the mid-plane of each specimen would be immersed to a depth of slightly greater than 1 in. This corresponds to a hydrostatic pressure of about 5 psf, or one-half the default hydrostatic pressure commonly used common ASTM standard tests (for example, ASTM E514) to measure water-penetration resistance to driving rain.

At that time, the drying stage would begin. To determine an appropriate ambient relative humidity for the drying stage is more difficult than for the wetting stage. Weather records of the National Oceanic and Atmospheric Administration (NOAA) indicate that in areas of Texas with observed premature concrete deterioration, the minimum average daily relative humidity is 60%. The ambient relative humidity for the drying stage was therefore set at that value.

In this way, cycling through the wetting and drying stages would reproduce the extremes of ambient relative humidity to which in-service structures are likely to be subjected in areas of Texas where premature concrete deterioration has been observed. The drying stage would end when the internal relatively humidity of an untreated specimen fell below 60%. At this time, if desired, a new wet stage would begin.

4.2 UNCONTROLLED PILOT TEST

To examine the general feasibility of this proposed test method, a series of tests was carried out in which untreated specimens were cast, wetted by submersion, and dried in ambient conditions in an air-conditioned space. In this report, that first test series is referred to as “uncontrolled pilot tests.”

4.2.1 Procedure Used for Uncontrolled Pilot Tests

The objective of the uncontrolled pilot tests was to determine the time required to reduce a specimen’s internal relative humidity (RH) from 100% to 60% (drying stage), and the time required to increase a specimen’s internal RH from 60% to 100% (wetting stage). Those same stage durations would be used in subsequent testing of treated specimens to evaluate proposed mitigation techniques.

The concrete specimens used for the pilot tests were fabricated like the test specimens previously used in Study 4069. They generally followed the requirements of ASTM C1293, with a square cross-section measuring 3 in. on each side, and a length of 12 in.

The same materials and mixing procedure were used as before during previous testing of Study 4069, which conformed to ASTM C 1293. Jobe sand was used as the fine aggregate due to its reactivity as confirmed during previous testing (Eskridge 2002). Portland cement with high alkali content (around 0.91%) and crushed river gravel as a coarse aggregate was used in the mix design. A single batch of fresh concrete, cast into steel molds, produced the specimens needed for the uncontrolled pilot test. Relative humidity measurement sleeves (plastic sleeves allowing insertion of a probe to measure internal relative humidity of concrete) were cast into the specimens with a nominal gage length between sleeves of 10 in. Each specimen has a measurement sleeve placed at a depth of 0.5 in. and another at a depth of 1.5 in. (at the longitudinal axis of the specimens). The specimens were allowed to cure for 7 days in a moist room. The drawing in Figure 4.2 shows a specimen with two measurement sleeves cast at depths of 0.5 in. and 1.5 in., representative of the specimens used in this study.

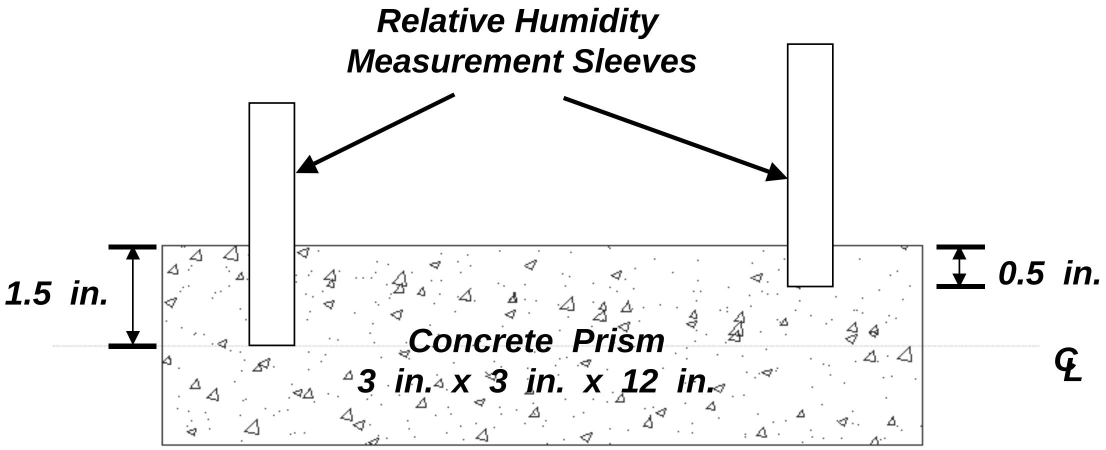

Figure 4.2 Fabricated specimen with measurement sleeves

Four replicate specimens were used, labeled A through D. During the 7-day curing period, the specimens were assumed to have 100% internal relative humidity. After curing, the specimens were demolded and placed in an air-conditioned space to start the first drying stage.

During each drying stage, the internal relative humidity and temperature of the specimens were measured daily as described in the next section. During each wetting stage, the internal relative humidity and temperature of the specimens were measured every day. Ambient relative humidity and temperature of the air-conditioned space were recorded every second day during each drying stage, and every day during each wetting stage.

4.2.1.1 Procedure for measuring internal relative humidity of specimens

The internal relative humidity of the specimens was measured with a Vaisala Model HM44 electronic humidity meter with solid-state probes. Humidity was measured by inserting the probe into a polyethylene measurement sleeve (Figure 4.3) that had been placed in the fresh concrete. The exposed end of the sleeve was sealed with a rubber plug having a small central hole that was also closed. The objective of the sleeve was to create a small, sealed volume of air with the same RH as the internal RH of the voids in the concrete.

The internal relative humidity of each specimen, and at two measurement locations (sleeves) was measured using the following steps, as directed by the manufacturer of the relative humidity-measurement equipment used in these tests (Vaisala Inc.):

1. Remove manufacturer’s rubber plug (designation 19267HM) from the opening at the exposed end of the measurement sleeve (designation 19266HM). That plug has a small central hole, which can be closed or opened.
2. Insert manufacturer’s probe (designation HMP44) into the exposed end of the measurement sleeve, and as far down the sleeve as possible.

3. Seal the measurement sleeve with the manufacturer’s rubber plug, leaving the small central hole open for the wire from the probe.
4. Allow 30 minutes for the probe to reach hygral equilibrium with the small surrounding air volume at the lower end of the sleeve, and presumably with the concrete at the bottom of the sleeve.
5. Plug the electronic connector from the rear of the probe into the digital indicator (designation HMI41).
6. Turn on indicator and record relative humidity and temperature.
7. Turn off indicator and unplug the electronic connector.
8. Remove the manufacturer’s rubber plug and probe from the open end of the sleeve.
9. Seal the measurement sleeve by re-inserting the manufacturer’s rubber plug into opening, this time with the small central hole closed.

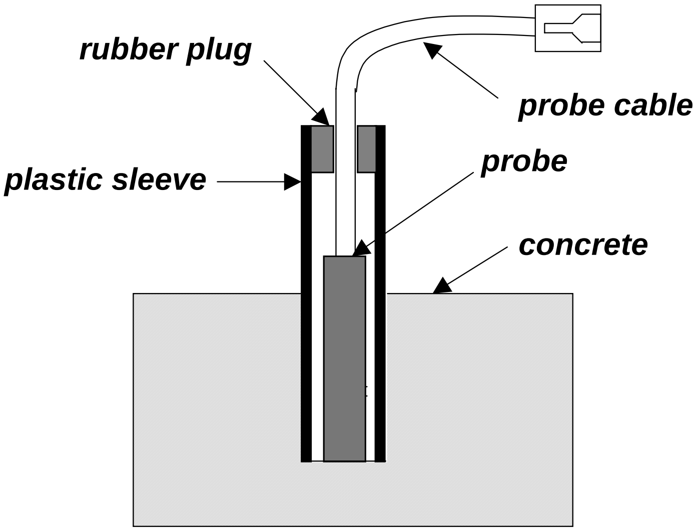

Figure 4.3 Cast-in-place sleeve with probe inside

4.2.2 Results from Uncontrolled Pilot Tests

The internal relative humidity of the specimens behaved erratically during the drying stage, fluctuating significantly instead of decreasing monotonically (Figure 4.4 and Figure 4.5).

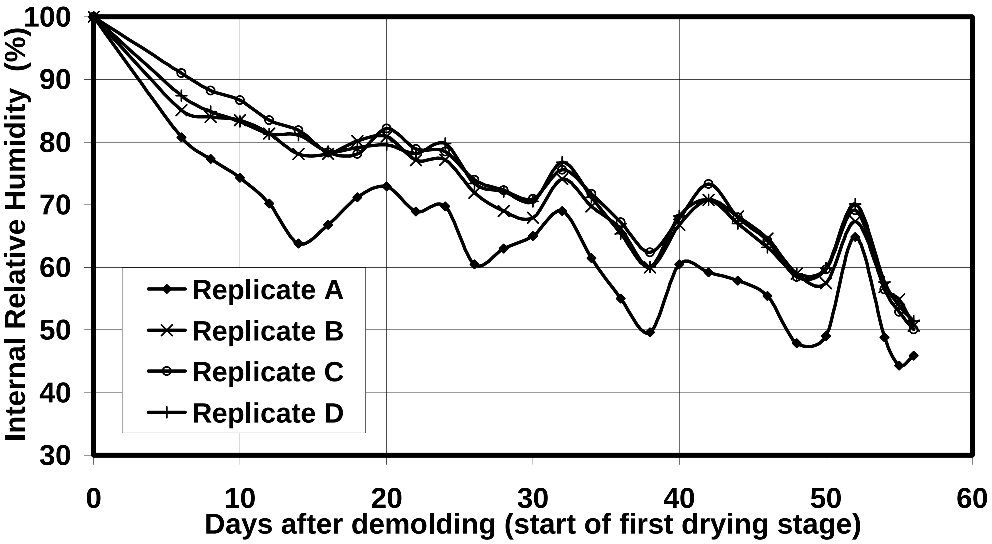

Figure 4.4 Internal RH at 0.5 in. depth during drying stage, uncontrolled pilot tests

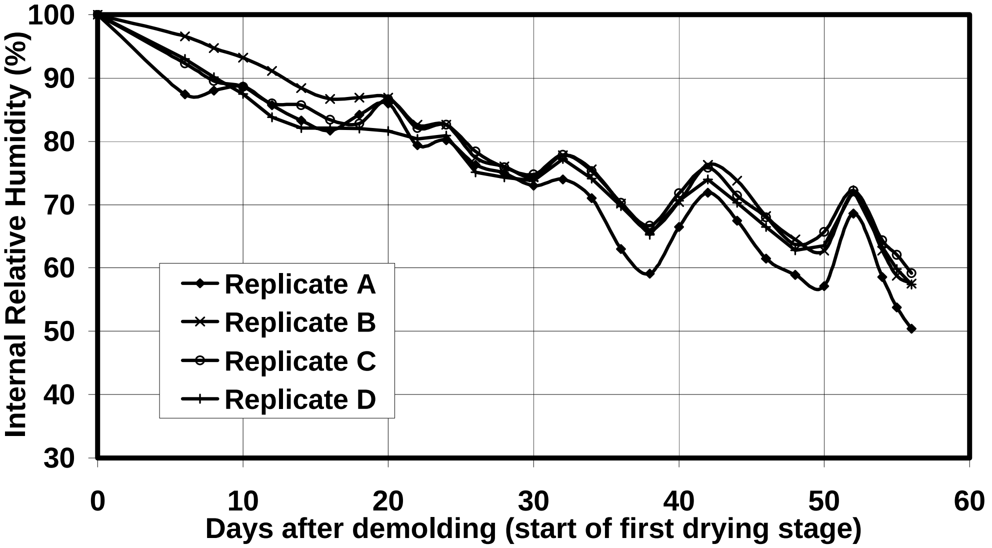

Figure 4.5 Internal RH at 1.5 in. depth during drying stage, uncontrolled pilot tests

The erratic behavior of the specimens during the drying stage might be attributed to the ambient relative humidity. If the ambient relative humidity fluctuates significantly, the internal relative humidity of the specimens might never reach a stable equilibrium value. Figure 4.6 shows the maximum and minimum recordings of ambient relative humidity during the drying stage.

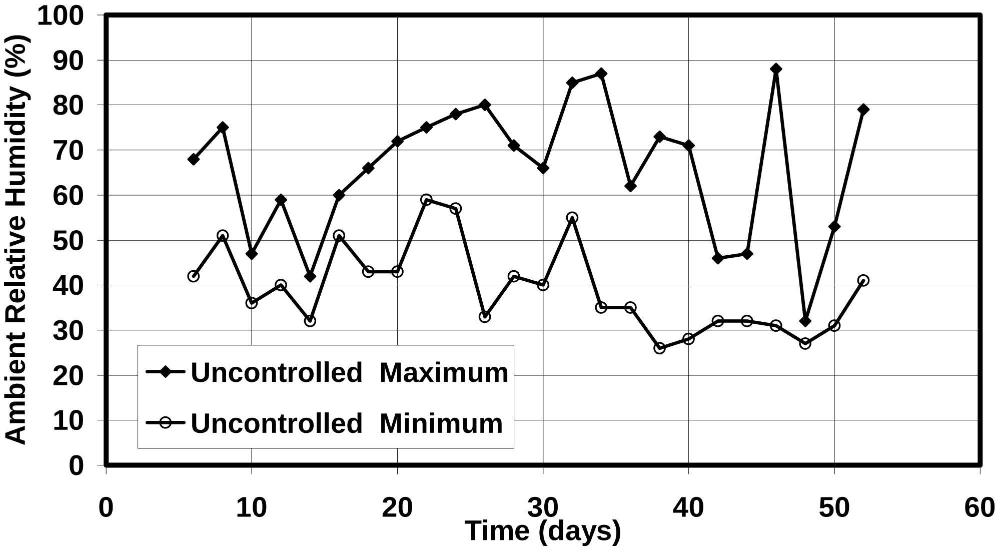

Figure 4.6 Ambient relative humidity during drying stage, uncontrolled pilot tests

Ambient relative humidity varied from as low as 26% to as high as 88% during the drying stage, and by as much as 55 percentage points over a two-day interval. The internal relative humidity of the specimens reacted to each large change in ambient relative humidity (daily maximum, daily minimum, or both.) The replicates' reactions to ambient relative humidity are seen in Figure 4.7, which plots the relative humidity of the replicates along with the ambient relative humidity. The former follows the latter. Significant increases in ambient RH were followed soon after by a slowing in the descent of the internal RH of the specimens, or even an increase in that internal RH. Correspondingly, when ambient RH decreased, the internal RH of the specimens decreased faster. Clearly, the internal relative humidity of the specimens during the drying stage was directly related to the ambient relative humidity of the air-conditioned space used for the drying stage.

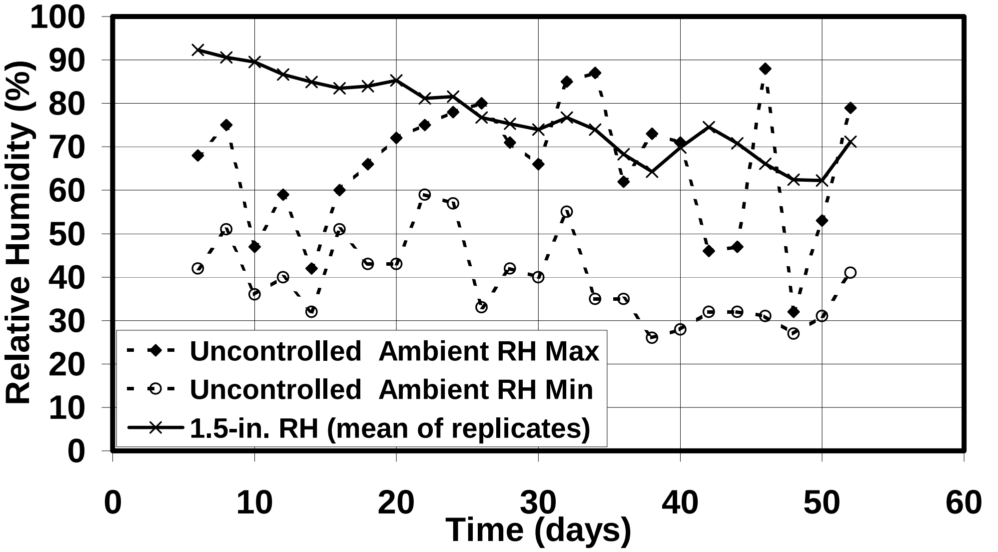

Figure 4.7 Average internal RH of replicates at 1.5 in. depth and ambient RH during drying stage

4.2.3 Conclusions from Uncontrolled Pilot Tests

Results from the uncontrolled pilot tests were inconsistent, because the internal relative humidity of the specimens followed changes in ambient relative humidity, which ranged from very low to very high, and also averaged higher than 60%. It would be unlikely for specimens to reach 60% internal relative humidity during the drying stage if the mean ambient relative humidity were higher than that value.

To achieve reliable and repeatable drying stages in the proposed test method, it was concluded that ambient relative humidity and temperature must be controlled. That is the subject of the next section.

4.3 CONTROLLED PILOT TESTS

The next series of tests, referred to as "controlled pilot tests," were identical to the uncontrolled pilot tests discussed above, except that the drying stage took place in an environmental chamber, under conditions of controlled temperature and relative humidity. In this section, the controlled pilot tests and their results are discussed.

4.3.1 Setup for Controlled Pilot Tests

The objective of the controlled pilot tests, like that of the uncontrolled pilot test, was to establish the required durations of the drying stage and wetting stage. The proposed test method was refined by using an environmental chamber to control ambient relative humidity and temperature during the drying stage. The relative humidity in the chamber was held constant by a dehumidifier. The temperature in the chamber was controlled by an air conditioner, which was necessary since heat from the dehumidifier would cause inconsistent ambient temperature. The environmental chamber used in the portion of this study is shown in Figure 4.8. No modifications were made to the wetting stage of the proposed test method.

During the drying stage, the specimens were placed in the environmental chamber, set to maintain a relative humidity of 30% and a temperature of 21°C. During the wetting stage, the specimens were submersed in water. The environmental chamber was not necessary during the wetting stage since the ambient relative humidity was a constant 100%, and the ambient temperature was almost constant at 21°C.

Figure 4.8 Environmental chamber used in controlled pilot tests

The controlled pilot tests began by placing the specimens that were cast for the uncontrolled pilot test in the environmental chamber for continued drying. The procedure of the proposed test method was not otherwise modified. A drying stage decreases the specimens' internal relative humidity to 60%, after which a wetting stage increases the specimens' internal relative humidity until it is sufficiently stable and close to 100%. A complete cycle consists of a drying stage followed by a wetting stage. If desired, any number of cycles could be imposed sequentially.

During the wetting stage, the internal relative humidity of the specimens and the ambient temperature were recorded daily. During the drying stage, the internal relative humidity and temperature of the specimens and the chamber's relative humidity and temperature were recorded every second day.

4.3.2 Results from Controlled Pilot Tests

During the controlled pilot tests, the ambient relative humidity and temperature in the environmental chamber was essentially constant, and the humidity was well below 60% (Figure 4.9). Because the ambient RH and ambient temperature were carefully controlled, the results of the drying stages and wetting stages were consistent from replicate to replicate within each cycle, and repeatable from cycle to cycle as well.

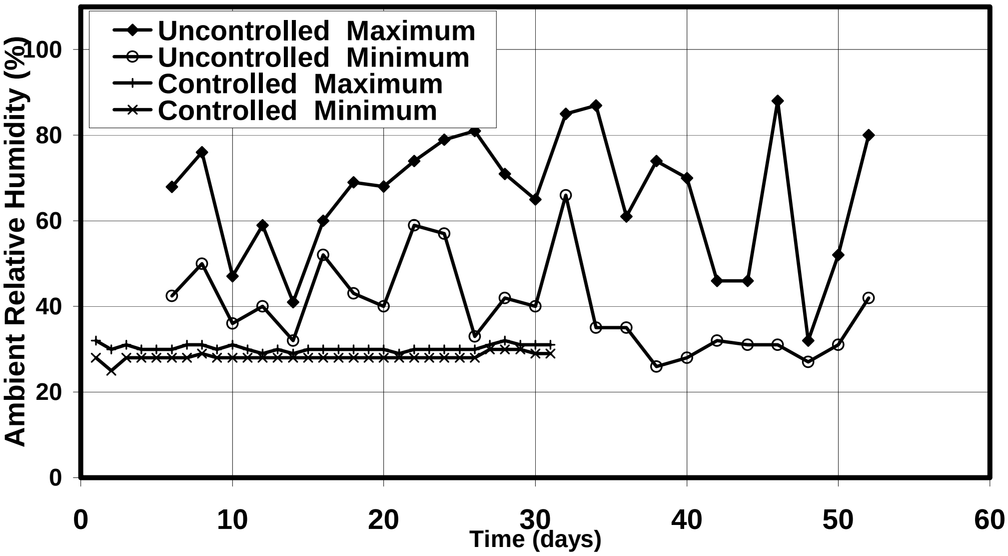

Figure 4.9 Ambient RH outside chamber (uncontrolled) and inside chamber (controlled)

A total of three complete cycles were imposed on the specimens. During each drying stage each replicate had relatively the same response as the other replicates (consistent behavior), and essentially the same response as in other drying stages (repeatable behavior). All specimens decreased in internal relative humidity monotonically throughout the drying stage. Figure 4.10 shows the internal relative humidity of the specimens at the 0.5-in. depth during the first drying stage. The specimens had similar behavior during the following drying stages (Appendix ***).

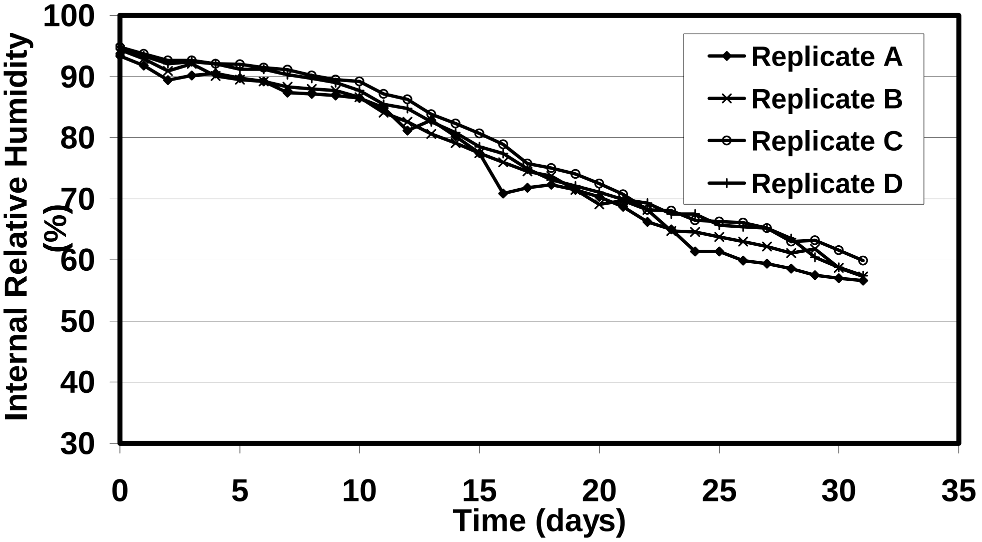

Figure 4.10 Internal RH at 1.5 in. depth during drying stage, controlled pilot tests

Each drying stage started with the concrete specimens with high internal relative humidity (around 95%) and continued until they reached an internal relative humidity below 60%. The drying stage of each cycle lasted around 31 days.

Similarly consistent and repeatable behavior was observed during the wetting stages. The internal relative humidity of the specimens increased monotonically during each wetting stage. Figure 4.11 shows the internal relative humidity of the specimens at the 1.5-in. depth during a wetting stage. The specimens had similar behavior during the other wetting stages, which are shown in Appendix ***B.

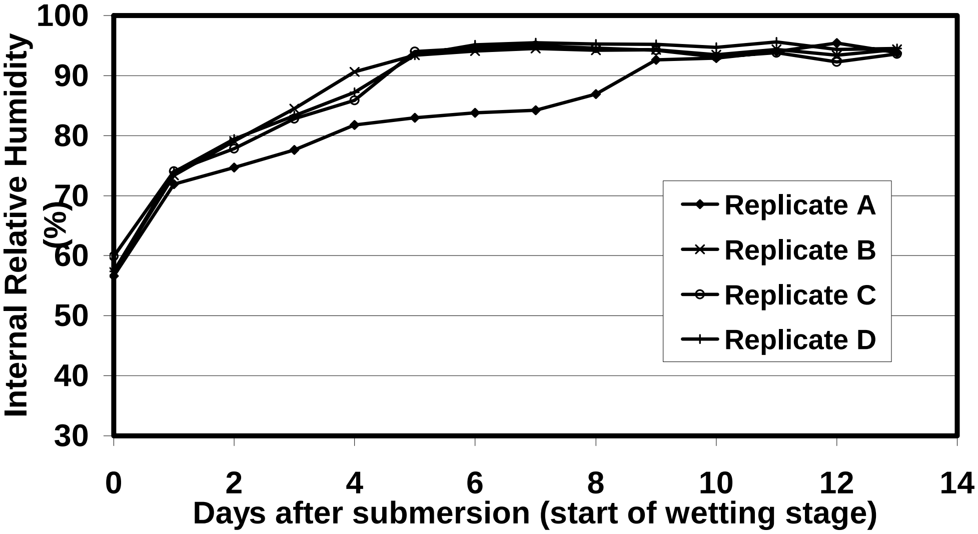

Figure 4.11 Internal RH at 1.5 in. depth during wetting stage, controlled pilot tests

Each wetting stage took each concrete specimen from an internal relative humidity below 60%, to a high and stable value of around 95%. The wetting stage of each cycle required around 10 days.

4.3.3 Conclusions from Controlled Pilot Tests

Based on the consistency and repeatability of internal relative humidity during the drying and the wetting stages, it was decided that the proposed test method, refined to control ambient conditions during the drying stage, would be suitable for evaluating the effectiveness of mitigation treatments. Based on the results of the controlled pilot tests, it was decided that the durations for the drying and wetting stages, as determined from the controlled pilot tests, would be used to test specimens treated with proposed mitigation techniques. The wetting and drying stages would start when untreated and otherwise identical control specimens reach 60% internal relative humidity and about 95% internal relative humidity, respectively. The proposed mitigation treatments would be evaluated by comparing the internal relative humidity of treated specimens with that of untreated specimens subjected to the proposed test method.

CHAPTER 5:
PROTOCOL FOR CONFIRMATORY TESTS

5.1 OVERVIEW OF CONFIRMATORY TESTS

After refining the test protocol using the controlled pilot tests, it was time for the next step -- confirming that the refined protocol would actually be useful in distinguishing between the performance of specimens with mitigation treatments, and the performance of untreated controlled specimens.

5.2 SPECIMENS FOR CONFIRMATORY TESTS

The specimens for the confirmatory tests were fabricated like those used for the pilot tests. Conforming to ASTM C 1293, the same materials and mixing procedure were used: Jobe sand as the fine aggregate; Type I Portland cement; and crushed river gravel as the coarse aggregate. The specimens were prisms with a square cross-section measuring 3 in. on each side, and a length of 12 in. Relative humidity measurement sleeves (plastic sleeves allowing insertion of a probe to measure internal relative humidity of concrete) were placed 10 in. apart in the top face of each specimen. One sleeve was placed at a depth of 0.5 in., and the other, at a depth of 1.5 in (halfway into the specimen). Unlike the pilot tests, the confirmatory tests required more than one batch (12 specimens) of fresh concrete. A total of three batches (28 specimens) of fresh concrete, cast into steel molds, produced the specimens needed for the confirmatory tests. Each batch of specimens was cured in a moist room for three days and then demolded to allow the next batch of specimens to be fabricated. After each batch of specimens was demolded, they were submersed in water to continue curing. In order for all specimens from all three batches to be in an equal state when testing begins, they were cured under water until the last batch had been submersed for 28 days. At the start of testing, all specimens had been cured for at least 28 days, the requirement of several treatment manufacturers. Since they were submersed for that length of time, the specimens were in hygral equilibrium at a high internal relative humidity.

The three batches produced 28 specimens, which were divided into 7 sets of 4 specimens each. Six sets of four concrete specimens were treated with different mitigation techniques. The seventh control set was left untreated.

5.3 MITIGATION TREATMENTS

The mitigation treatments, determined from previous work on Study 4069, are summarized in Table 5.1 and further described in the next subsection. TxDOT-specified products were obtained from approved suppliers; other products were obtained on the open market.

Before applying each mitigation treatment, the specimens were taken out of submersion, and their surfaces were cleaned by light pressure-washing. After the surfaces were allowed to dry, the mitigation treatments were applied by brush.

5.3.1 Mitigation Treatment #1 – Current TxDOT Treatment

The current TxDOT treatment for ASR/DEF consists of Type I Silane, followed by TxDOT Type 742h thinned Appearance Coat paint. The Silane used in this study, obtained from an approved list of suppliers, is manufactured by Chem-Trete (product designation BSM-40 VOC). All mitigation treatments requiring the use of silane use the same source (Chem-Trete). The gray appearance paint was supplied directly by TxDOT.

Table 5.1 Mitigation treatments used for confirmatory tests

Designation	Mitigation Treatment	Abbreviation
M1	TxDOT Penetrating Concrete Surface Treatment Type I – Silane, followed by TxDOT Type 742 Appearance Coat paint (thinned)	Sil + 742th
M2	TxDOT Penetrating Concrete Surface Treatment Type I –Silane	Silane
M3	TxDOT Penetrating Concrete Surface Treatment Type I – Silane, followed by Class B Type II Latex paint	Sil + latex
M4	TxDOT Penetrating Concrete Surface Treatment Type I – Silane, followed by Opaque Concrete Sealer	Sil + opq
M5	Lithium Nitrate, followed by TxDOT Penetrating Concrete Surface Treatment Type I – Silane	$LiNO_3$ + Sil
M6	Penetrating Epoxy	Epoxy
M7	Control; no mitigation treatment applied	Control

5.3.2 Mitigation Treatment #2 – Type I Silane

Type I Silane was used by itself to determine if TxDOT's Appearance Coat paint affects the water impermeability or vapor permeability, alone or in combination.

5.3.3 Mitigation Treatment #3 – Type I Silane & Acrylic Latex

Type I Silane was used in conjunction with TxDOT Class B Type II Coating for Concrete from Departmental Material Specification DMS-8110, also known as TxDOT acrylic latex. The acrylic latex was supplied directly by TxDOT.

5.3.4 Mitigation Treatment #4 – Type I Silane & Opaque Concrete Sealer

Type I Silane was used in conjunction with TxDOT Opaque Concrete Sealer, formally known as "silicone acrylic architectural stain." The opaque concrete sealer is manufactured and supplied by Sherwin Williams (product designation SWD D.O.T. Bridge and Highway Concrete Sealer B97-Series).

5.3.5 Mitigation Treatment #5 – Lithium Nitrate Solution & Type I Silane

Lithium Nitrate was used in conjunction with Type I Silane. The lithium nitrate solution used in this mitigation treatment is manufactured and supplied by Euclid Chemical (product designation Eucon ARC Treatment).

5.3.6 Mitigation Treatment #6– Penetrating Epoxy

A penetrating breathable epoxy, the last mitigation technique, was manufactured and supplied by EpoxySystems (product designation #850: Epoxy Penetrating Sealer). The epoxy was supplied as a two-component system. Component "A" was opened and the resin inside agitated until evenly blended. Next, Component "B" was opened and poured into Component "A." The combined mixture was agitated until uniform in color, and applied by brush to the dry surfaces of the specimens. As per the manufacturer's instructions, two coats were applied.

5.3.7 Mitigation Treatment #7 – Control

To determine the effectiveness of the above six mitigation techniques a control set of untreated specimens was used for comparisons. In theory, the control specimens should behave like the control specimens in the controlled pilot tests.

CHAPTER 6:
FURTHER REFINEMENT OF TEST METHOD

6.1 INITIAL RESULTS FROM TEST METHOD

After the refined pilot test was conducted and determined to be suitable for this study, the test method was applied to a second set of specimens to evaluate several proposed mitigation treatments. Here and elsewhere in this report, those tests are referred to as "confirmatory tests." Initial results from those specimens were quite different from those of the control specimens in the pilot test. Those differences were analyzed to determine their probable cause. Three supplemental experiments were conducted to further refine the test protocol so that results would be consistent with those of the refined pilot tests.

When the refined test method was applied to the second set of specimens, during the first drying stage, all specimens (untreated and treated) showed a decrease in measured internal relative humidity to a value of 60% in 14 days, roughly half the time originally required for the specimens in the pilot test. Figure 6.1 and Figure 6.2 show, for the confirmatory tests, the measured internal relative humidity of the four control specimens (labeled A through D) at depths of 0.5 and 1.5 in., respectively, during the first drying stage.

When all control specimens in the confirmatory tests reached a measured internal relative humidity of 60% at a depth of 1.5 in., the specimens were submersed in water, starting the first wetting stage. During this first wetting stage the measured internal relative humidity of the specimens did not increase monotonically, but rather fluctuated significantly over the next two months of observation (Figure 6.3 and Figure 6.4).

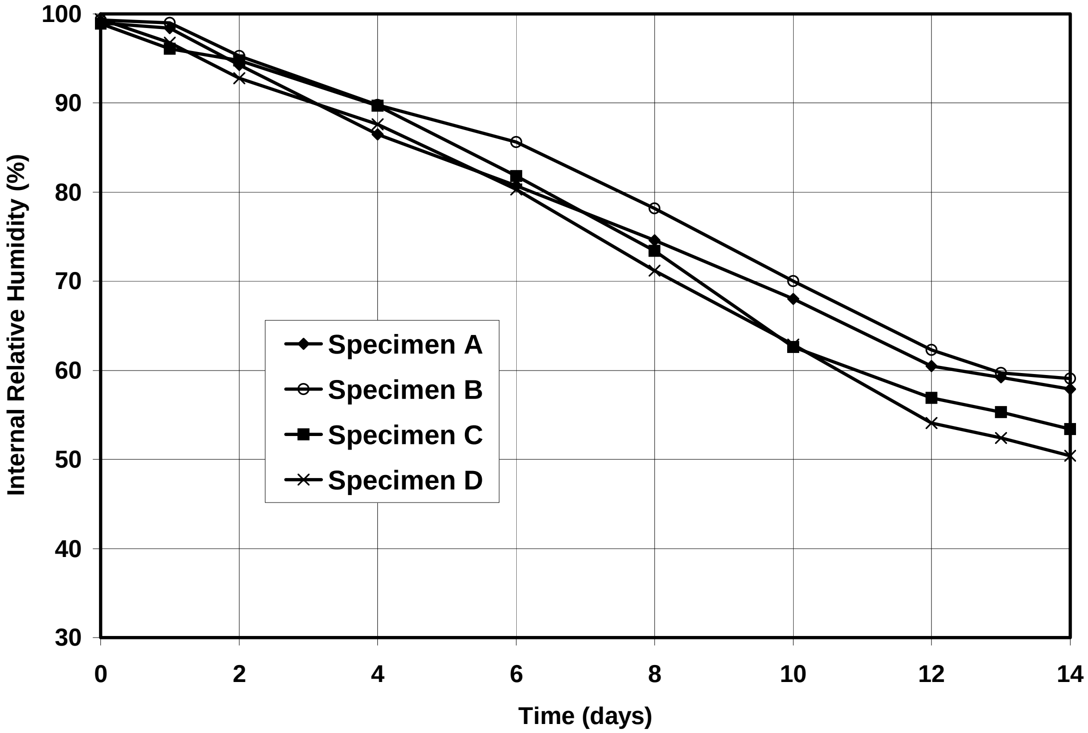

Figure 6.1 Measured internal relative humidity of control specimens at 0.5 in. depth during Drying Stage #1 (Confirmatory Test)

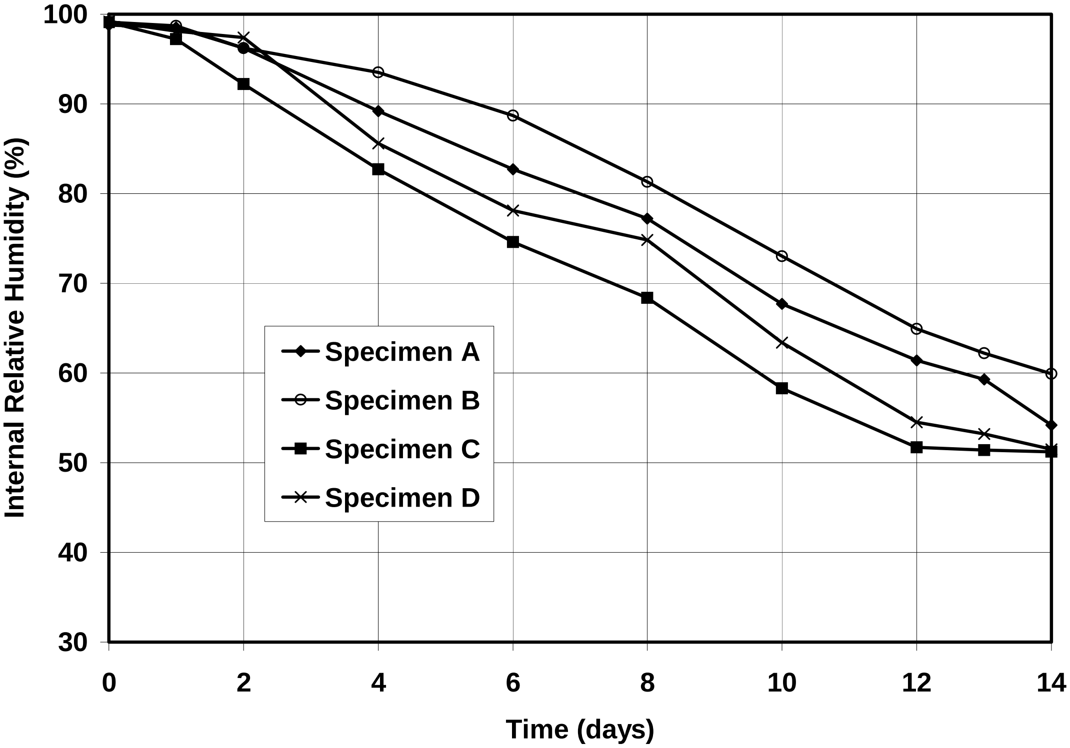

Figure 6.2 Measured internal relative humidity of control specimens at 1.5 in. depth during Drying Stage #1 (Confirmatory Test)

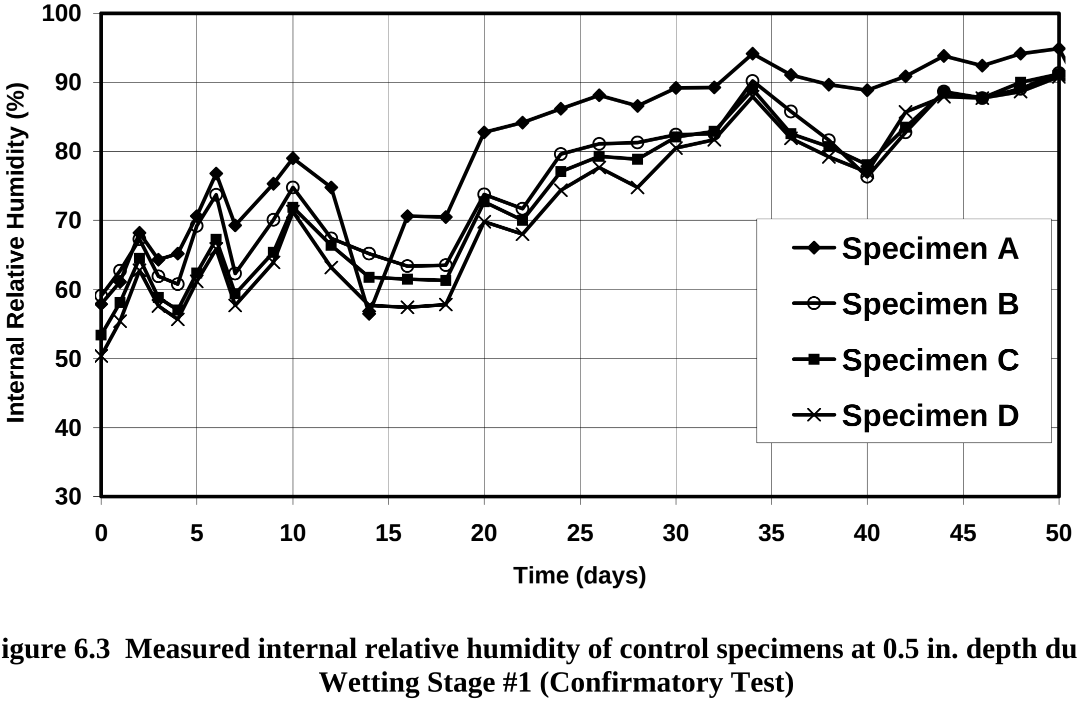

Figure 6.3 Measured internal relative humidity of control specimens at 0.5 in. depth during Wetting Stage #1 (Confirmatory Test)

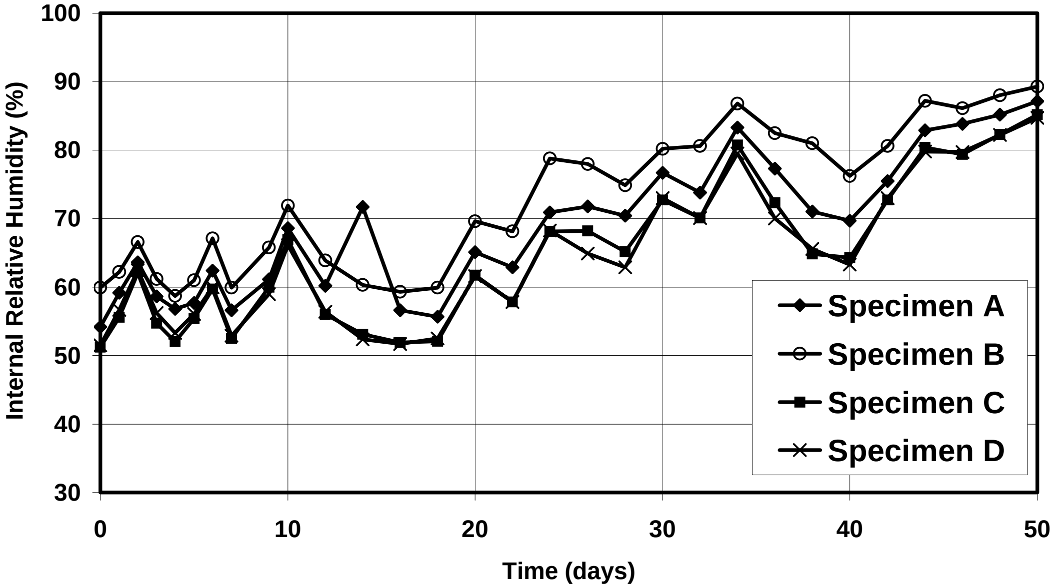

Figure 6.4 Measured internal relative humidity of control specimens at 1.5 in. depth during Wetting Stage #1 (Confirmatory Test)

This behavior in the wetting stage of the confirmatory tests contrasted with that previously observed in the refined pilot tests, in which the measured internal relative humidity of the control specimens increased monotonically to a high value (around 95%) within 10 days (Figure 6.5 and Figure 6.6).

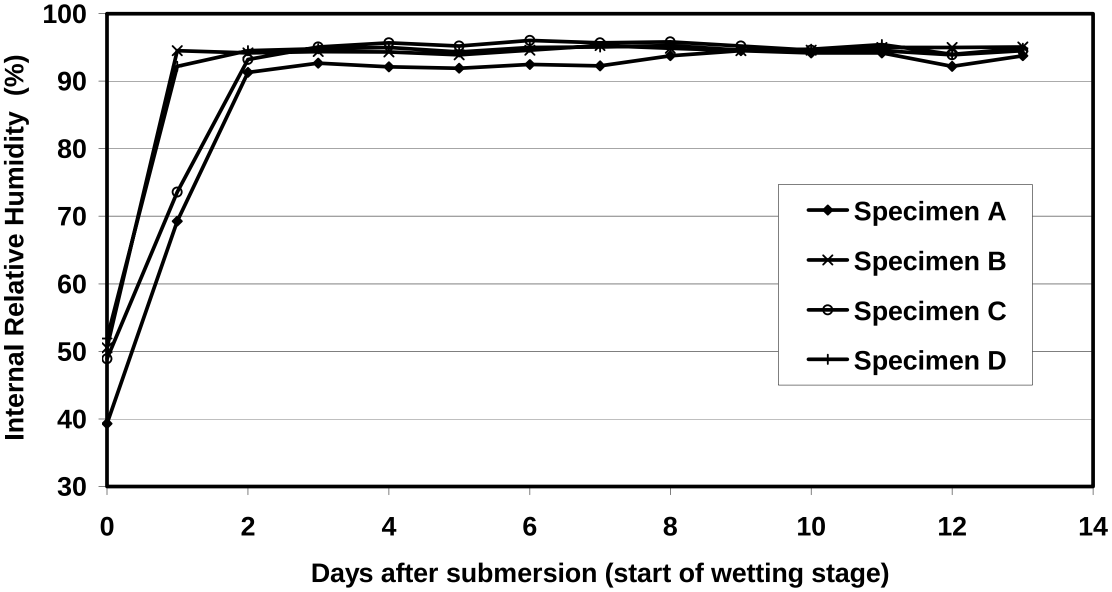

Figure 6.5 Measured internal relative humidity of control specimens at 0.5 in. depth during Wetting Stage #2 (Pilot Test)

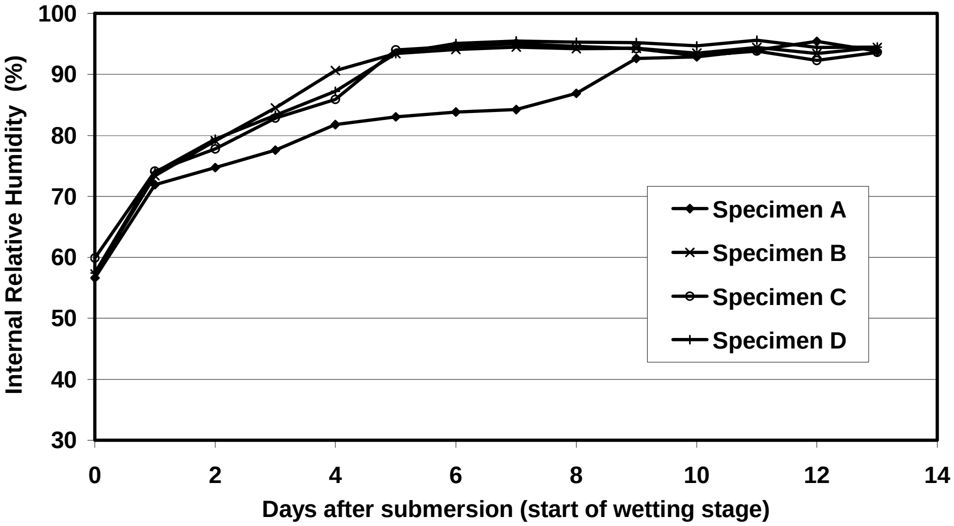

Figure 6.6 Measured internal relative humidity of control specimens at 1.5 in. depth during Wetting Stage #2 (Pilot Test)

During the wetting stage of the confirmatory test, another trend was evident in the data. The measured relative humidities of all specimens (treated and untreated alike) fluctuated together. When one specimen dropped in measured relative humidity, all other specimens would also drop. During this wetting stage, ambient relative humidity varied from as low as 30% to as high as 90%. When ambient relative humidity decreased, the measured relative humidity of the concrete specimens decreased as well. When ambient relative humidity increased, the specimens reacted analogously, increasing in measured relative humidity (Figure 6.7 and Figure 6.8). This was puzzling, because the specimens were submerged during the wetting stage, and their internal relative humidity should not have fluctuated with changes in ambient relative humidity.

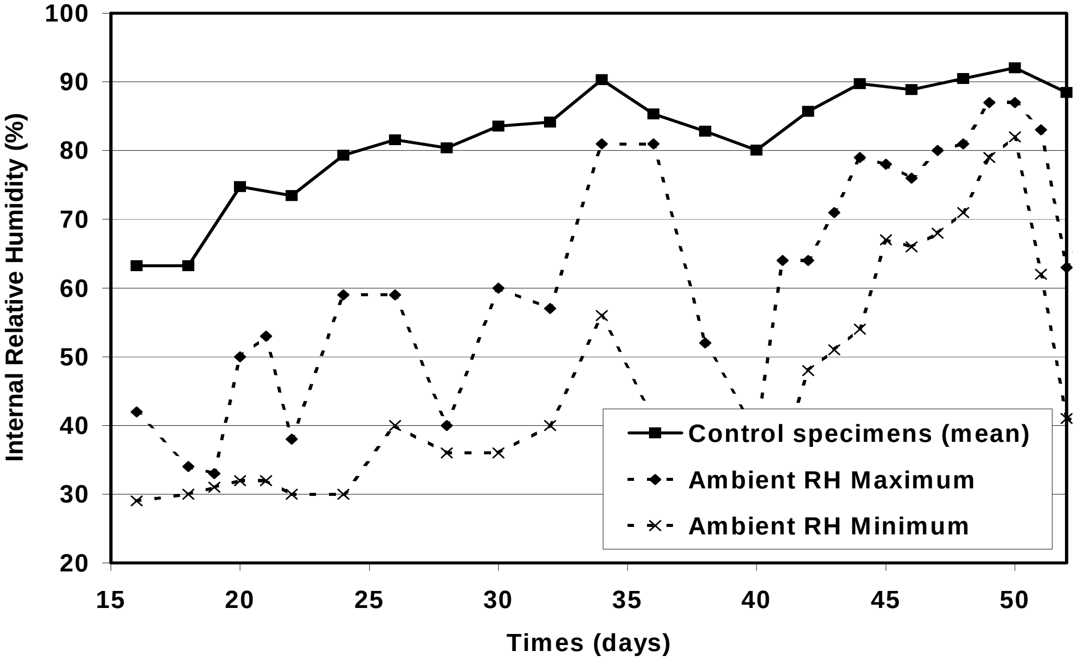

Figure 6.7 Average internal RH of control specimens at 0.5-in. depth and ambient RH during Wetting Stage #1 (Confirmatory Test)

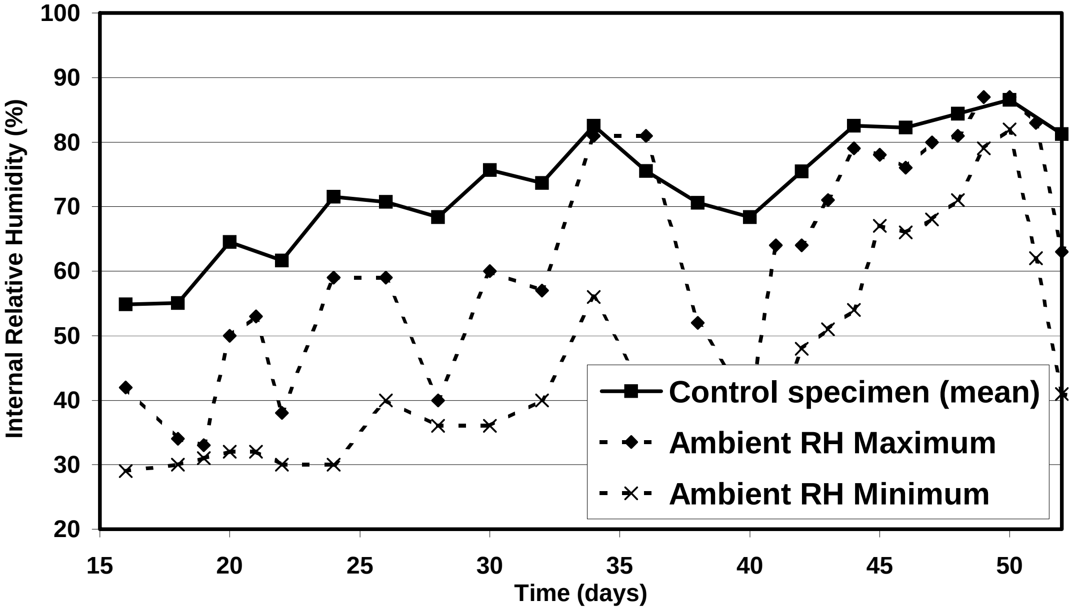

Figure 6.8 Average internal RH of control specimens at 1.5 in. depth and ambient RH during Wetting Stage #1 (Confirmatory Test)

These observations strongly suggested a correlation between the ambient relative humidity and measured relative humidity of concrete specimens. This correlation had not been observed during the pilot tests.

6.2 ANALYSIS OF DIFFERENCES BETWEEN PILOT TESTS AND CONFIRMATORY TESTS

In an effort to determine the causes of these differences between observed behavior in the pilot tests and the confirmatory tests, the differences were evaluated in more detail.

First, the possibility of differences due to the specimens themselves was investigated. The pilot test specimens and the confirmatory test specimens had been cast using the same materials, including Jobe sand for fine aggregate, crushed river gravel for coarse aggregate, the same water source, and the same high-alkali portland cement. The same mixing and casting procedure had been used for both sets of specimens as well. Based on this, it was concluded that differences in specimens could not be responsible.

Next, possible differences in measurement technique were explored. The same equipment was used to measure the specimens' internal humidity during the pilot tests and the confirmatory tests. During the wetting stage of the confirmatory test, the internal relative humidity of the pilot test specimens was measured. The pilot test specimens reacted in the same manner as before, indicating that the measuring equipment was functioning correctly and consistently. Evidently, differences in measurement technique were not responsible, either.

The only identifiable difference between the two sets of specimens was the orientation of the cast-in-place gage sleeves. The pilot test specimens had inadvertently been cast with the plastic sleeves oriented upside-down. This presented a problem, because the measuring probes did not fit down the sleeves due to the presence of an internal sealing ridge near the top inside of the sleeves. To solve this problem in the pilot tests, the sleeves were cut just under the ridge to allow the probes to slide down into the sleeves (Figure 6.9). The confirmatory test specimens, in contrast, were cast with the gage sleeves in correct orientation, and it was not necessary to cut them (Figure 6.10).

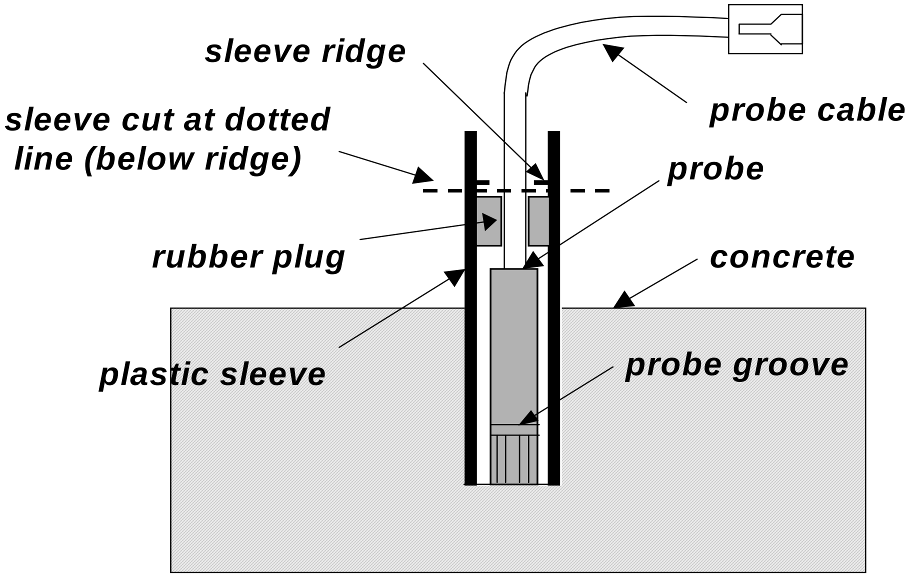

Figure 6.9 Orientation of sleeves in Pilot Tests

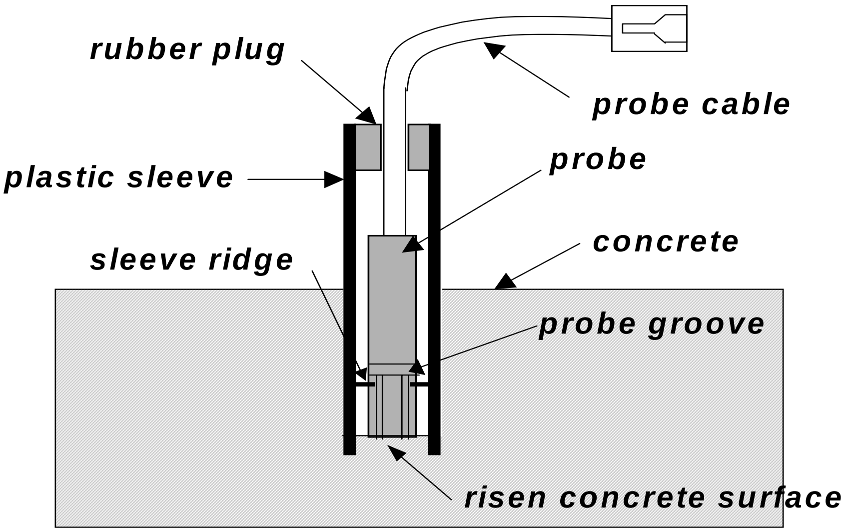

Figure 6.10 Orientation of sleeves in Confirmatory Tests

Precise measurements of the sleeves showed that they were not exactly cylindrical, but rather tapered, with a smaller diameter at the end with the internal ridge (0.469 in.) than at the other end (0.492 in.). In the incorrect orientation of the pilot tests, the rubber plugs used to seal the tops of the sleeves fit more tightly in the sleeves, than in the correct orientation of the confirmatory tests.

6.3 MANUFACTURER'S SUGGESTIONS

The manufacturer of the measuring equipment and sleeves (Vaisala Inc.) was contacted regarding this behavior. According to the manufacturer, the rubber plug was not intended to create an airtight seal with the plastic sleeve. Rather, the airtight seal was intended to be created when the measuring probe was inserted into the tube the internal ridge in the tube would mate with a corresponding groove in the probe (Figure 6.11).

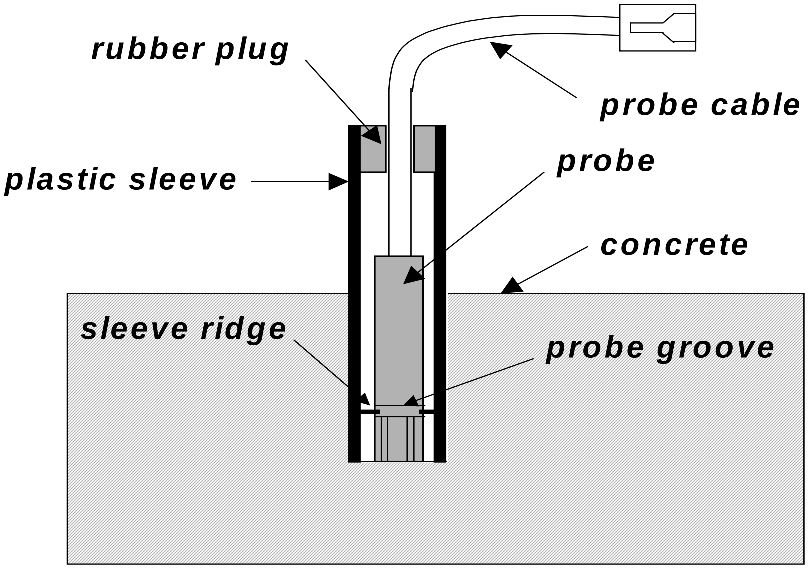

Figure 6.11 Manufacturer's suggested sleeve orientation

According to the manufacturer, the rubber plug was intended only to prevent dust and other harmful particles from entering the plastic sleeve. The "micro-environment" with constant RH was intended to be created between the base of the sleeve at the level of the concrete surface, and the sealing ridge.

In the specimens of this study, however, that micro-environment did not exist in the pilot tests or in the confirmatory tests. In the pilot tests, with the upside-down sleeves, the ridge had been removed (Figure 6.9). And in the confirmatory tests, with correctly oriented sleeves, the ridge usually did not function either, for the following reason. The original design of the measuring equipment and plastic sleeves did not contemplate casting the sleeves in fresh concrete. The plastic sleeves were originally intended to be inserted into drilled holes in hardened concrete, and then sealed with an appropriate sealing material. In this study, the sleeves were placed in fresh concrete. The fresh concrete usually rose slightly inside the sleeve, preventing the groove in the outer surface of the probe from mating with the internal ridge in the sleeve (Figure 6.10). The consequences of this discovery, and the ways in which it was successfully addressed, are discussed later in this chapter.

Another possible source for the observed behavior was the repetitive use of the sleeves. The manufacturer suggested that the probe created an effective seal when locked in place with the internal ridge. This meant, however, that when the probe was not in the sleeve, no effective seal would be present to prevent the ambient relative humidity from affecting the relative humidity inside the sleeve. From this design characteristic, it can be concluded that the manufacturer intended to have each sleeve used for only one measurement.

This study, however, called for the plastic sleeves to be cast in place in the concrete specimens, and then used for repeated measurements. Because of this, the manufacturer's intended sealing method was not viable, and it was necessary to develop a different but equally effective method of sealing the measurement sleeves. The method chosen was to seal the sleeves using some type of plug. Results from the pilot test clearly show that the sleeves were effectively sealed without using the manufacturer's method, and instead by plugging the smaller end of the cut plastic sleeve. The sleeves in the confirmatory test specimens were orientated correctly, however, and therefore their larger ends had to be plugged. The same sealing method used in the pilot tests could not be used in the confirmatory test.

6.4 SUPPLEMENTAL EXPERIMENTS

To ensure that the micro-environment inside the plugged sleeve is not directly affected by ambient relative humidity, it is necessary to use an effective seal between the plug and sleeve. Because of the measured variation in relative humidity within the sleeve during the wetting stage, the method used previously during the confirmatory test was determined to be ineffective, necessitating the development of a modified method. Three supplemental experiments were conducted to evaluate several sleeve-sealing methods. A sleeve-sealing method that proved to be effective in the supplemental experiments would be tested further by applying the method in confirmatory tests. Should those confirmatory tests be consistent with the controlled pilot tests, the test method, modified to use that sleeve-sealing method, would then be expected to be successful in comparing proposed mitigation treatments (Figure 6.12).

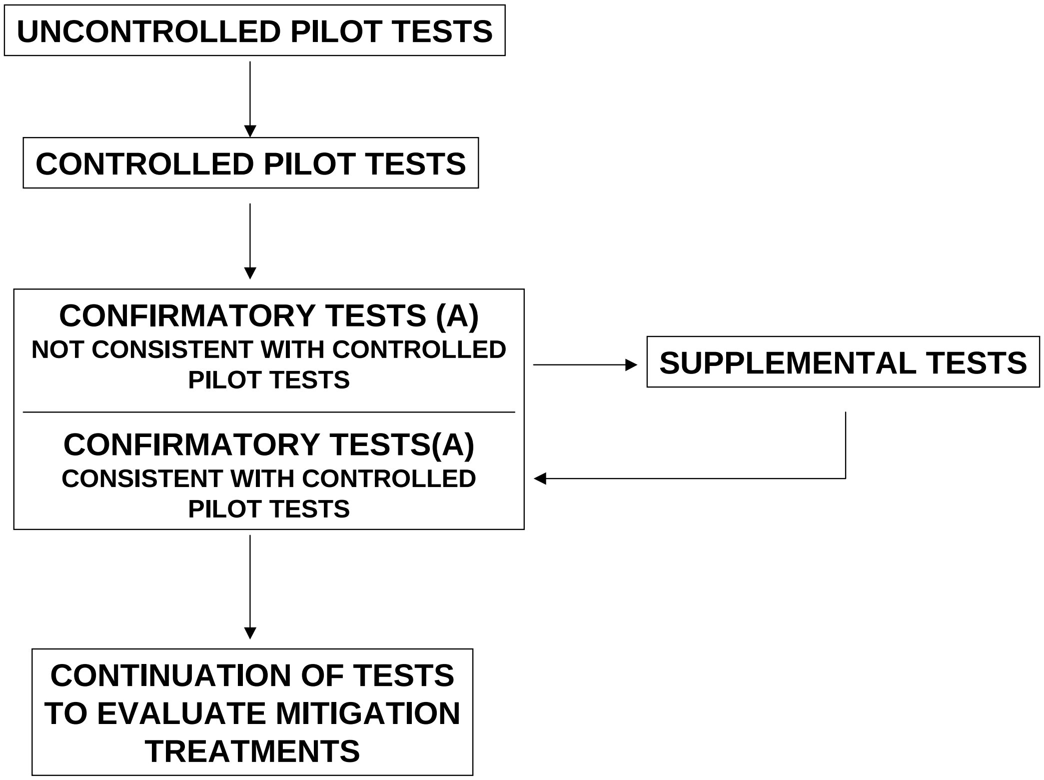

Figure 6.12 Flow chart of tests conducted in this phase of Study 4069

6.4.1 Supplemental "sleeve-with-concrete" experiment

A supplemental experiment was conducted to evaluate the effectiveness of three different sleeve-sealing methods. Each sealing method was used to seal the sleeves, cast at depths of 0.5 in. and 1.5 in., into two concrete specimens. The first sleeve-sealing method consisted of applying electrical tape to the circumference of the manufacturer's rubber plug (Figure 6.13). This was intended to increase slightly the diameter of the plug and therefore increase the tightness of the sleeve seal.

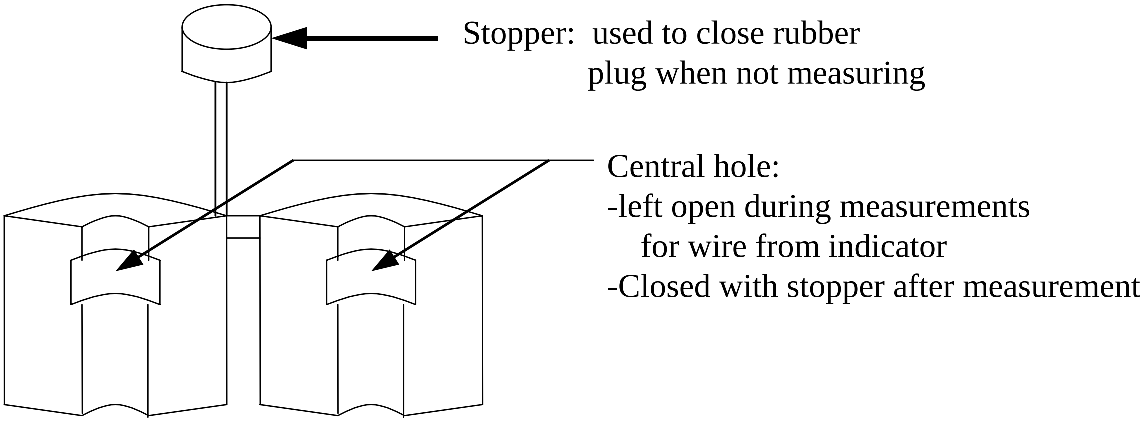

Figure 6.13 Manufacturer's rubber plug

The second sleeve-sealing method consisted of using tapered rubber plugs (Figure 6.14). One end of tapered plug has a diameter smaller than the sleeve opening, while the other end has a diameter larger than the opening. The tapered plug, in theory, creates an airtight seal when inserted into the sleeve. When the measuring probe is in the sleeve during the 30-minute equilibrium period, the tapered plug cannot be used to seal the sleeve. Instead, a taped rubber plug was used to seal the sleeve during the 30-minute measurement period. Further testing to evaluate the effectiveness of the taped rubber plug during the 30-minute measurement period is the subject of another supplemental experiment.

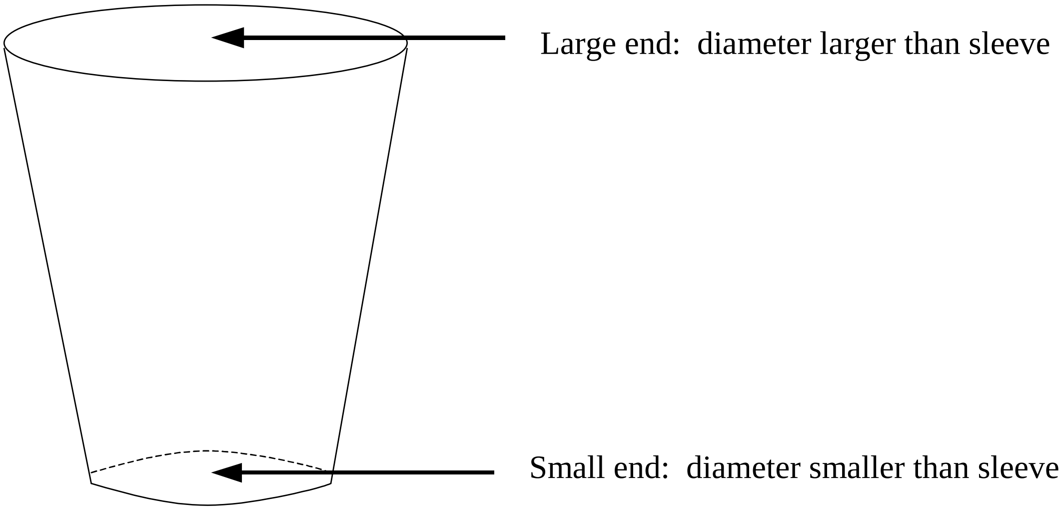

Figure 6.14 Tapered rubber plug

The third and last sleeve-sealing method consisted of cutting the sleeves and sealing them with the manufacturer's rubber plugs (Figure 6.13). Since the sleeves in the confirmatory test specimens are orientated correctly, the open ends of the sleeves have a larger inside diameter than the end in the concrete. Cutting the sleeves changes the height at which the plugs seal the sleeves. Since the inside diameter of the sleeve tapers along its height, the plug should create a tighter seal than the plug at the end of an uncut sleeve. The results of this test show that the properly oriented and cut sleeves, and the taped plugs, follow the ambient humidity to a greater extent than the tapered rubber plugs (Figure 6.15 and Figure 6.16), and are therefore less desirable than the tapered rubber plugs as a means of stabilizing the relative humidity within the measurement sleeves.

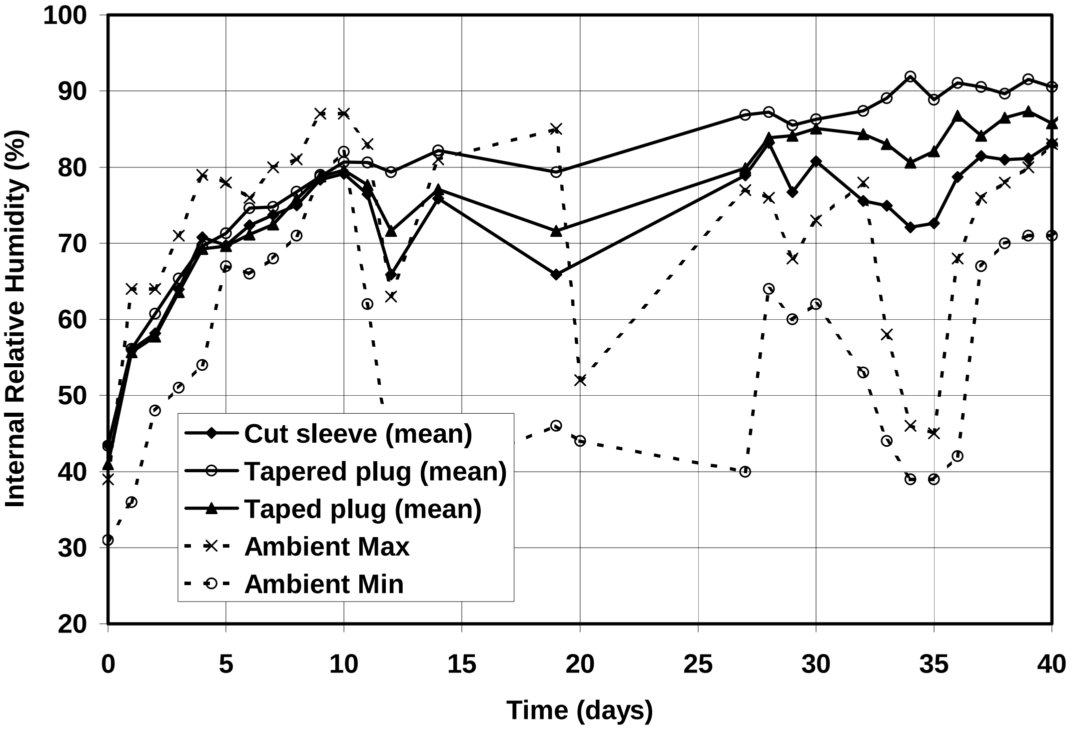

Figure 6.15 Average internal RH of control specimens with cut sleeves, taped sleeves, and tapered plugged sleeves at 0.5-in. depth (Supplemental Experiment #1)

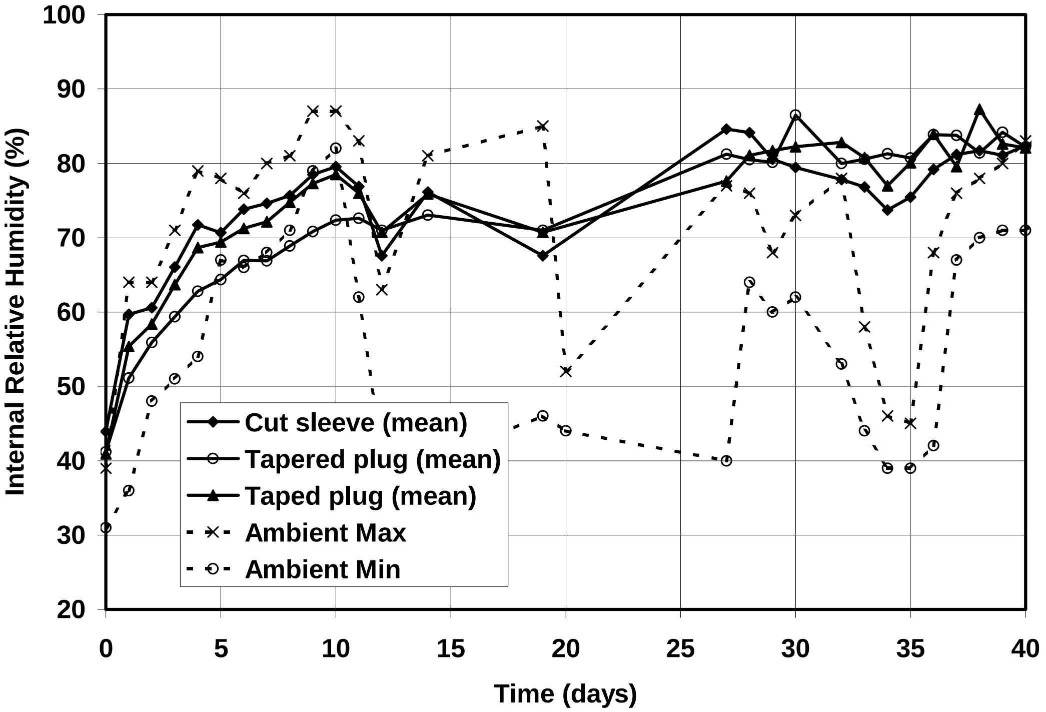

Figure 6.16 Average internal RH of control specimens with cut sleeves, taped sleeves, and tapered plugged sleeves at 1.5-in. depth (Supplemental Experiment #1)

6.4.2 *Supplemental "sleeve-without-concrete" experiment*

Another supplemental experiment was conducted to evaluate the effectiveness of three different sleeve-sealing methods without the presence of concrete, effectively removing the concrete from the list of variables. The supplemental experiment was set up by wetting the inside of the sleeve, sealing both ends of the sleeve, and placing the sleeve in the environmental chamber. Since the environmental chamber has a relatively low ambient relative humidity of 30%, the effectiveness of each sleeve-sealing method was determined by measuring the rate of decrease of the relative humidity within the sleeve. The faster a sealed sleeve decreased in relative humidity, the less effective the sleeve-sealing method was taken to be. During the first test, one end of each sleeve was sealed using electrical tape. During the second test, that same end was sealed using tapered plugs and electrical tape. The other end of the sleeves was sealed using the four different sealing methods described below. Two replicate sleeves were used for each combination of methods.

The first sleeve-sealing method consists of applying electrical tape to the circumference of the manufacturer's rubber plug (Figure 6.13) and inserting the plug in the larger end of the sleeve. The second sleeve-sealing method involved tapered plugs (Figure 6.14). As before, taped plugs were used during the measurement period when the tapered plugs couldn't be used. The tapered plugs are inserted in the larger end of the sleeve. The third sleeve-sealing method consists of plugging the larger end of the sleeve with the manufacturer's rubber plug (Figure 6.13), with no electrical tape applied to its circumference. The last sleeve-sealing method consists of cutting the smaller ends of the two sleeves in the same manner that the sleeves of the pilot test specimens had been cut. The manufacturer's rubber plug is then inserted into the cut (smaller) end of the sleeves. In this configuration the sleeves resemble the orientation and the seal of the sleeves of the pilot test specimens.

During the first test cycle, the sleeves whose larger end was sealed with the manufacturer's plug decreased the fastest in relative humidity, showing that this was the least effective sealing method for the first test cycle. The sleeves whose smaller end was cut and sealed with the manufacture's plug decreased the slowest in relative humidity, indicating that this was the most effective sealing method for the first test cycle (Figure 6.17).

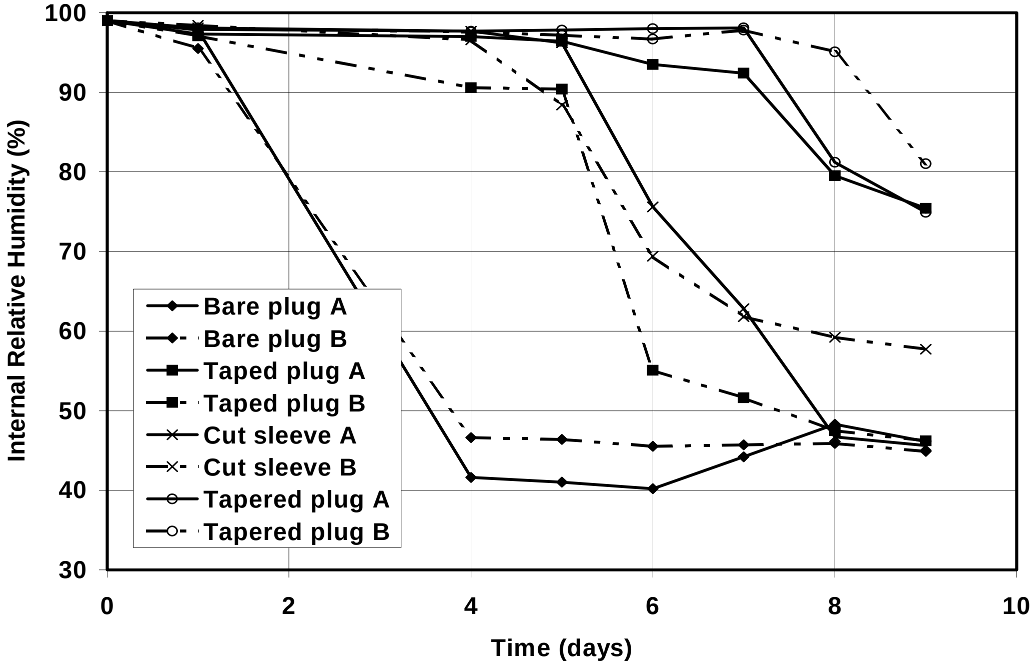

Figure 6.17 Relative humidity of sleeves without concrete (Supplemental Experiment #2, Test #1)

The sleeve-sealing method using the manufacturer's plug was the least effective during the first test cycle and also the second test cycle. The sleeve-sealing method involving the use of a tapered plug to seal the sleeve's larger end proved to be the most effective during the second test cycle (Figure 6.18).

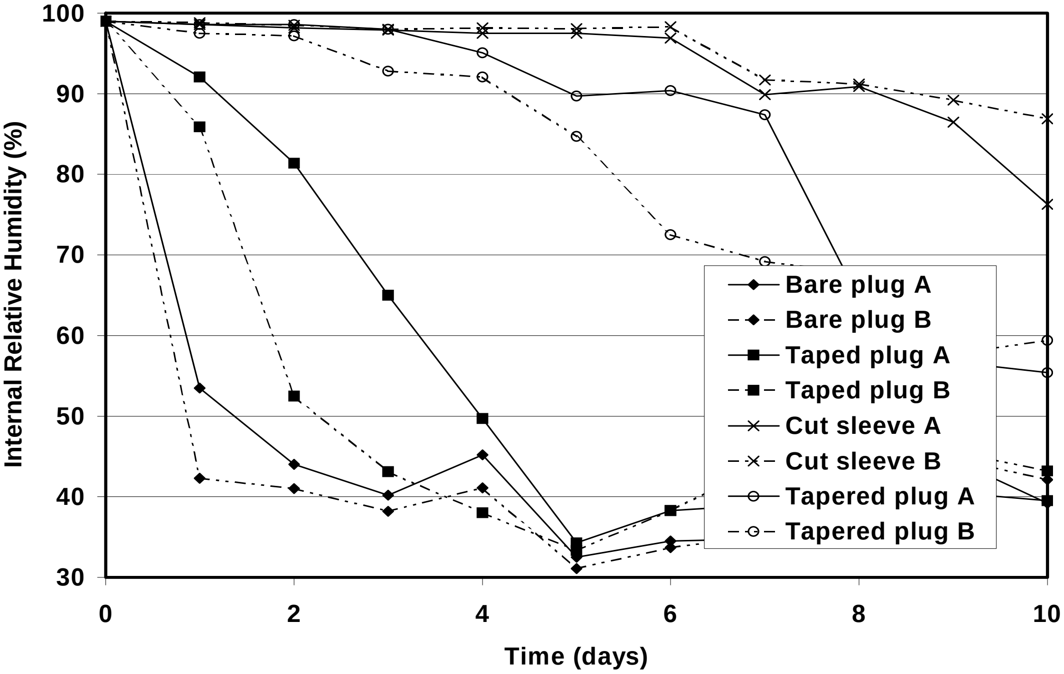

Figure 6.18 Relative humidity of sleeves without concrete (Supplemental Experiment #2, Test #2)

The results of these two test cycles show that the cut-sleeve sealing method used in the pilot tests and the tapered-plug sealing method developed in the supplemental tests are highly variable in effectiveness. Of the four sealing methods, however, these two performed much better than the taped-plug method and the manufacturer's-plug method.

6.4.3 Supplemental "measurement-period" experiment

A final supplemental experiment was set up to determine the effectiveness of the taped plug used during the 30-minute measurement period. The manufacturer's instructions indicate that when the probe is inserted into the sleeve, it should be left for 30 minutes to come into hygral equilibrium with the concrete, after which a measurement may be recorded. The taped plug was used during the 30-minute measurement period to seal sleeves that used tapered plugs at other times. Results from the previous supplemental experiments only reveal the ineffectiveness of using the manufacturer's taped plug to seal sleeves at all times; the objective of this last supplemental experiment is to determine the effectiveness of the chosen method of sealing the sleeves during only the 30-minute measurement period. Relative humidity measurements were taken every 5 minutes during the 30-minute measurement period. Several tests were conducted, some with high ambient relative humidity and others with low ambient relative humidity.

The results from tests with high ambient relative humidity were similar to those with low ambient relative humidity. In all cases the measured internal relative humidity of the concrete specimens reached equilibrium in roughly 20-25 minutes, and did not decrease thereafter (Figure 6.19 and Figure 6.20). These results clearly show the taped plug, while not effective for long durations, is an effective sealing method during the 30-minute measurement period. Taped plugs used during the 30-minute measurement period do not alter the effectiveness of using the tapered plug as a sleeve-sealing method. Effectiveness depends solely on the seal from using the tapered plugs.

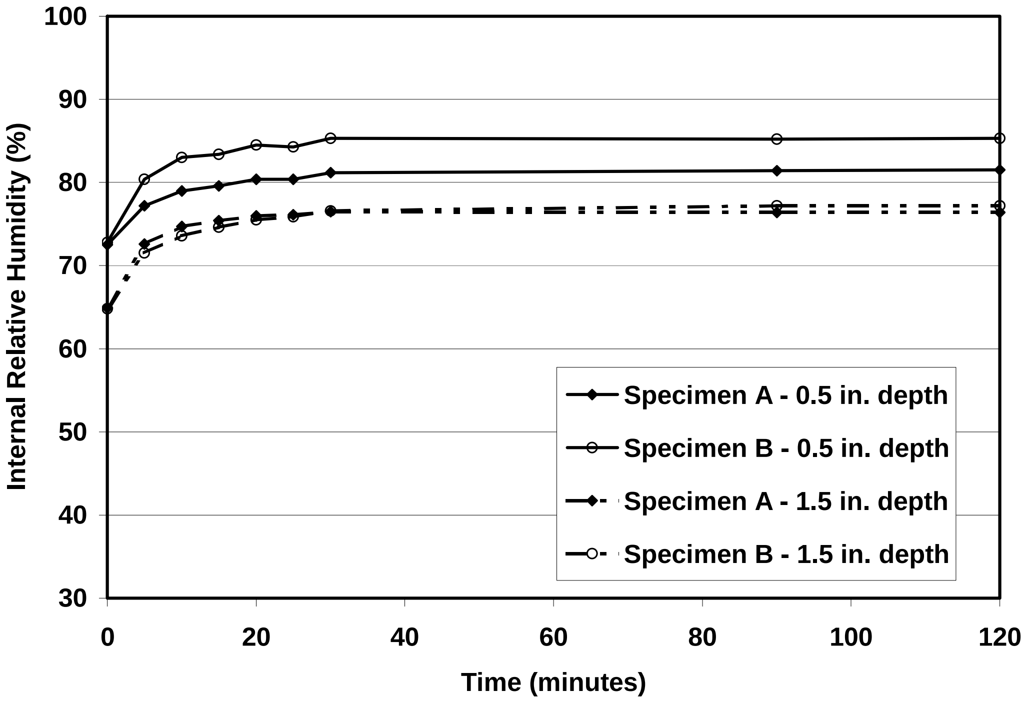

Figure 6.19 Internal RH of control specimens during measurement period with low (44%) ambient RH (Supplemental Experiment #3)

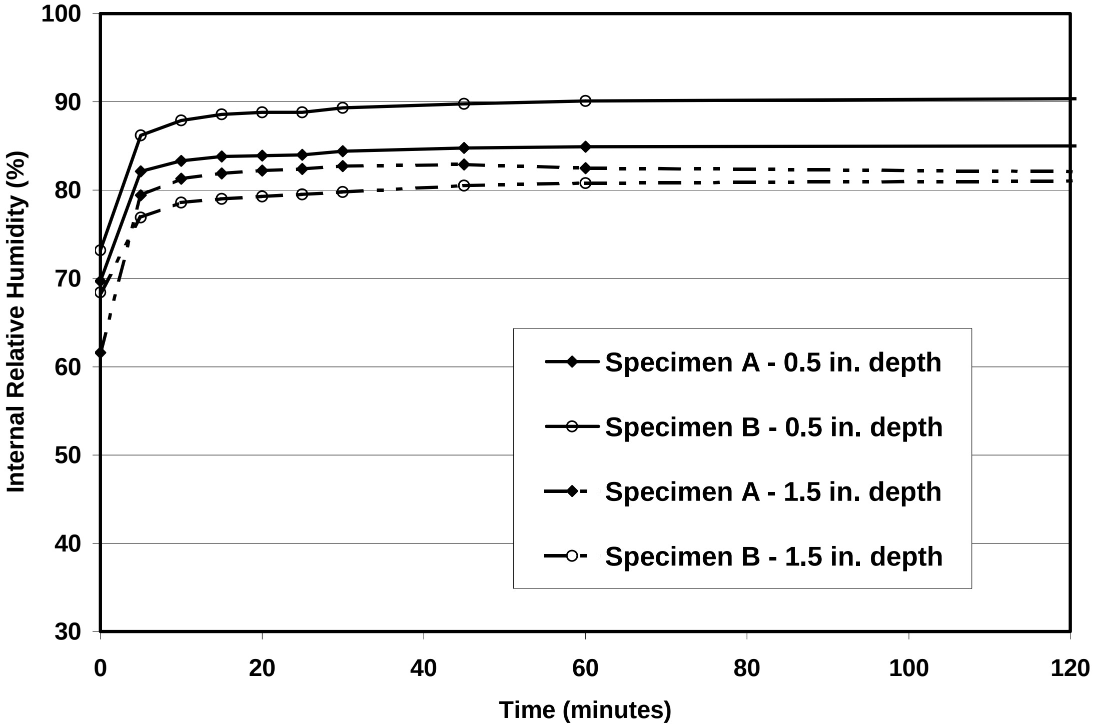

Figure 6.20 Internal RH of control specimens during measurement period with high (73%) ambient RH (Supplemental Experiment #3)

6.5 CONCLUSIONS FROM SUPPLEMENTAL EXPERIMENTS

The supplemental “sleeve-without-concrete” experiments evaluated three methods for sealing the larger end of the measurement sleeve during the wetting stage: a rubber plug; a tapered rubber plug; a taped rubber plug. They also evaluated the effectiveness of using a rubber plug to sealing the smaller end of a cut sleeve. Results from the supplemental experiments show that the cut sleeves and tapered rubber plugs are variable in effectiveness, but can be very effective at times. The rubber plug and the taped plug, in contrast, are ineffective.

The supplemental “measurement-period” experiment determined that taped rubber plugs, though ineffective for long periods, were an effective sleeve-sealing method for the 30-minute measurement period.

The supplemental “sleeve-with-concrete” experiment confirmed the results of the supplemental “sleeve-without-concrete” experiment and supplemental “measurement-period” experiment. The best way to maintain the micro-environment inside the measurement sleeve is to use a tapered rubber plug for long periods, and the manufacturer’s rubber plug, increased slightly in diameter by tape, for the 30-minute measuring period. Using this combination of sealing methods, the confirmatory test should give results similar to those of the pilot test, for control specimens. This is checked subsequently.

CHAPTER 7:
RESULTS FROM CONFIRMATORY TESTS B (FINAL TEST RESULTS) AND THEIR SIGNIFICANCE

7.1 INTRODUCTION TO RESULTS FROM CONFIRMATORY TESTS B

In this chapter are presented the results from Confirmatory Tests B (the final set of tests conducted in this study), including the modification described in Chapter 5. One entire cycle of testing was completed, consisting of a drying stage followed by a wetting stage. The results of those tests, presented in this chapter, are a crucial facet of the research described in this report. They serve two principal functions:

First, the results are compared with those of the Controlled Pilot Tests and the Confirmatory Tests A to test the validity of the hypothesis that the inconsistencies in results from Confirmatory Tests A were in fact due to problems with sealing the tubes containing the measurement probes. As discussed in Section 7.2 and 7.3, the results are found to be consistent, and the hypothesis is confirmed.

Second, the results from treated specimens are compared with the results from untreated control specimens to compare the effectiveness of different proposed mitigation treatments. As discussed in Sections 7.5 and 7.5, the results are found to be significant, and recommendations can be made based on them.

7.2 RESULTS FROM DRYING STAGE, CONFIRMATORY TESTS B

7.2.1 Results for Mitigation Treatment M1, Drying Stage

Figure 7.1 and Figure 7.2 show the average internal relative humidity of a typical set of treated specimens (M1) and the corresponding set of control specimens, at depths of 0.5 and 1.5 in. respectively, during the drying stage. In contrast to the results from Confirmatory Tests A, the internal relative humidity decreases monotonically, showing that the sealing methods developed in the Supplemental Tests were satisfactory.

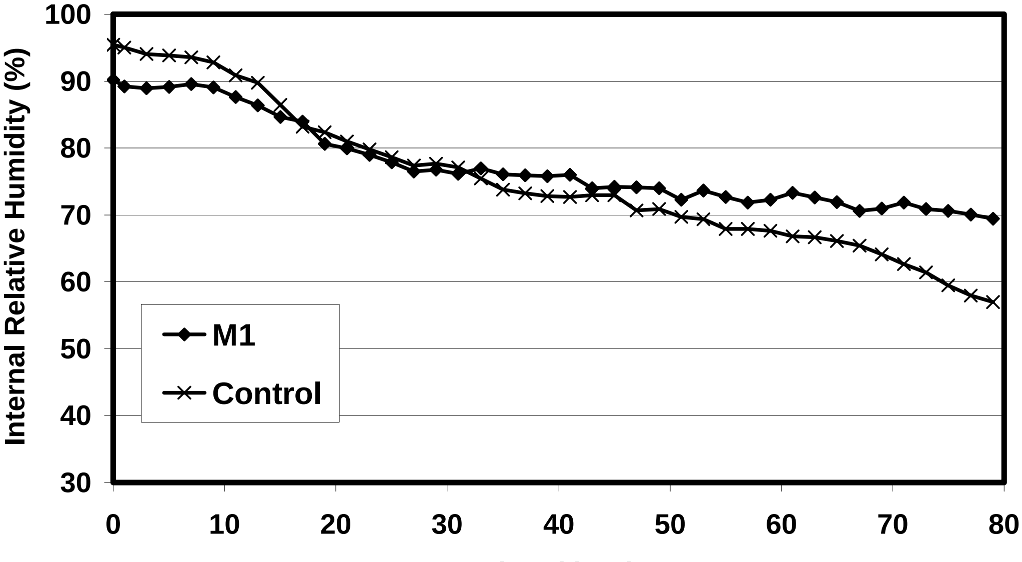

Figure 7.1 Average internal RH of M1 replicates and control replicates at 0.5-in. depth during the drying stage, Confirmatory Tests B

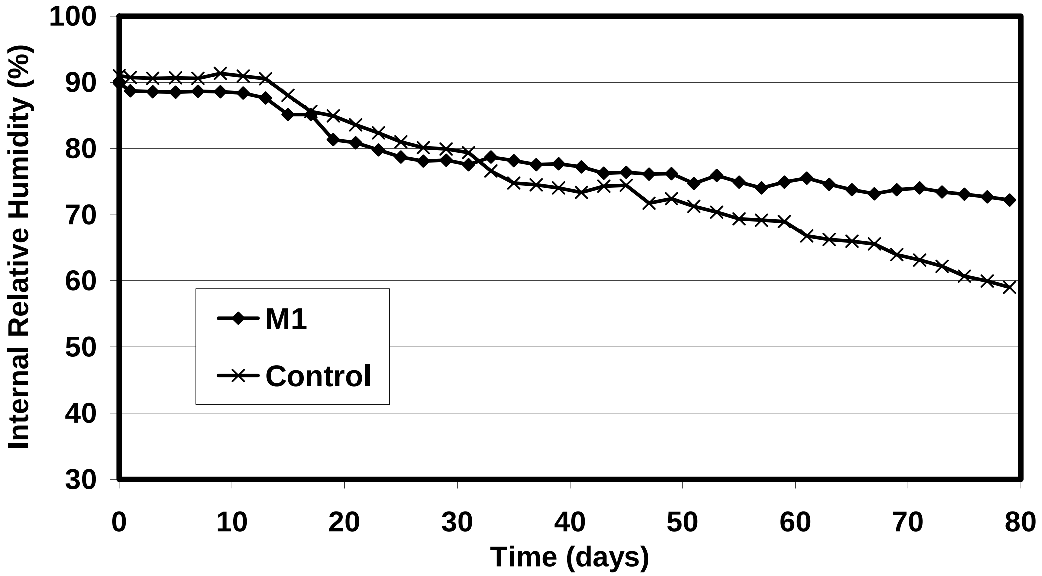

Figure 7.2 Average internal RH of M1 replicates and control replicates at 1.5-in. depth during drying stage, Confirmatory Tests B

7.2.1.1 Average Results for Each Mitigation Treatment, Drying Stage

Figure 7.3 and Figure 7.4 show the average internal relative humidity of each set of specimens (treated and control), at depths of 0.5 and 1.5 in. respectively, during the drying stage. Results are consistent among sets of specimens, showing that the sealing methods developed in the Supplemental Tests were satisfactory.

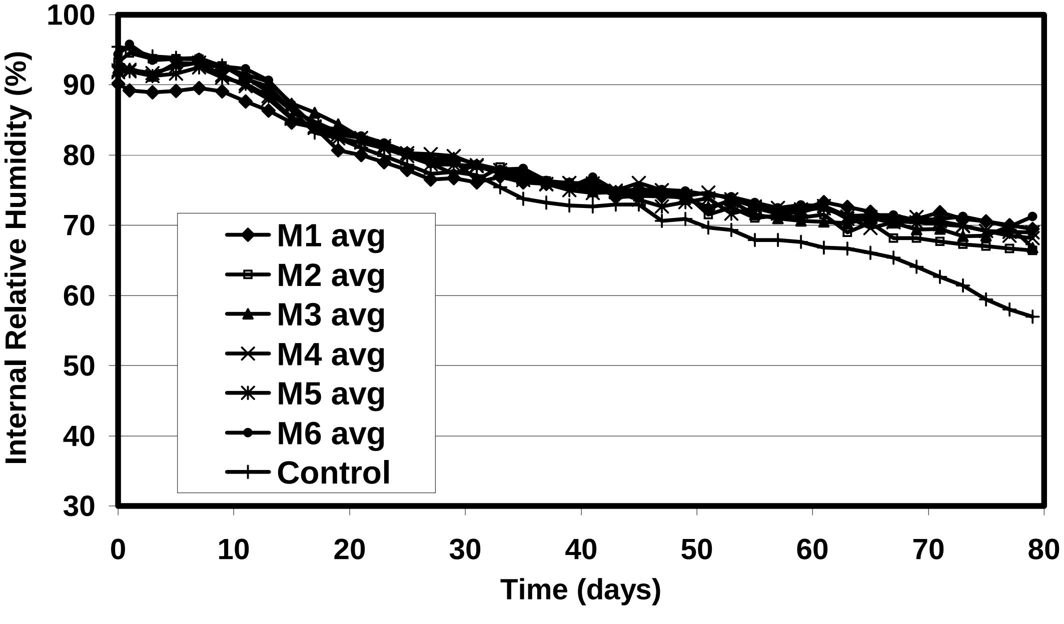

Figure 7.3 Average internal RH of each treated and untreated set of specimens at 0.5-in. depth during drying stage, Confirmatory Tests B

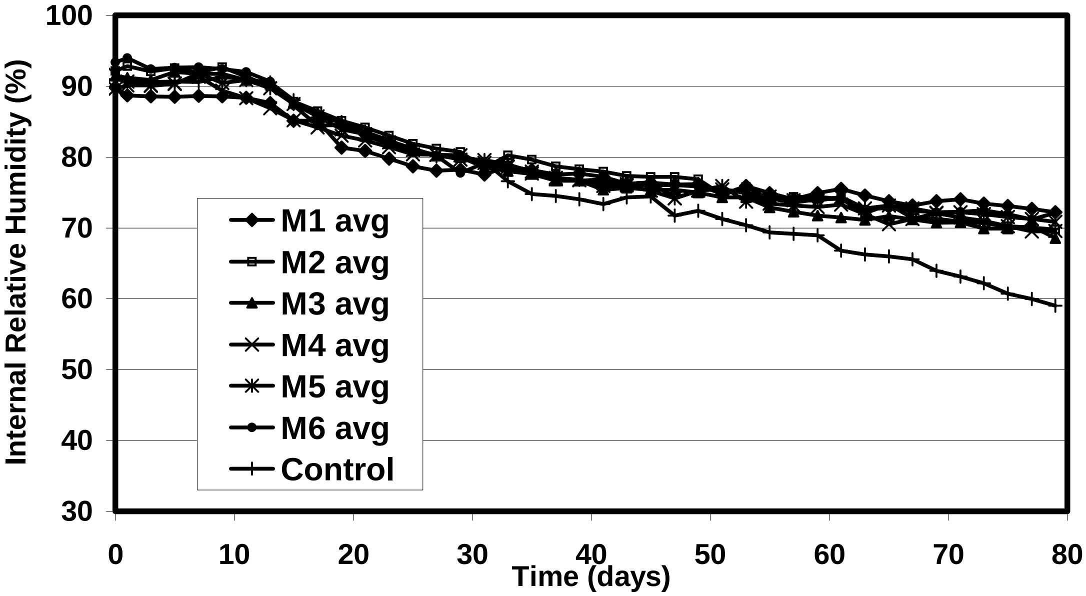

Figure 7.4 Average internal RH of each treated and untreated set of specimens at 1.5-in. depth during drying stage, Confirmatory Tests B

7.3 RESULTS FROM WETTING STAGE, CONFIRMATORY TESTS B

7.3.1 Results for Mitigation Treatment M1, Wetting Stage

Figure 7.5 and Figure 7.6 show the average internal relative humidity of a typical set of treated specimens and the control set of specimens, at depths of 0.5 and 1.5 in. respectively, during the wetting stage. Unlike the results from Confirmatory Tests A, the internal relative humidity increases monotonically, showing that the sealing methods developed in the Supplemental Tests were satisfactory.

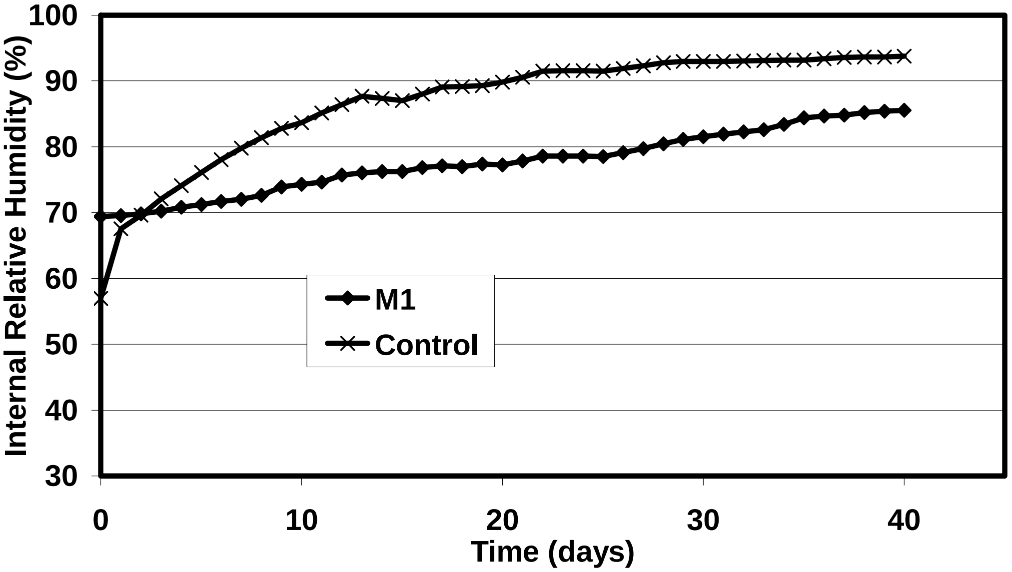

Figure 7.5 Average internal RH of M1 replicates and control replicates at 0.5-in. depth during wetting stage, Confirmatory Tests B

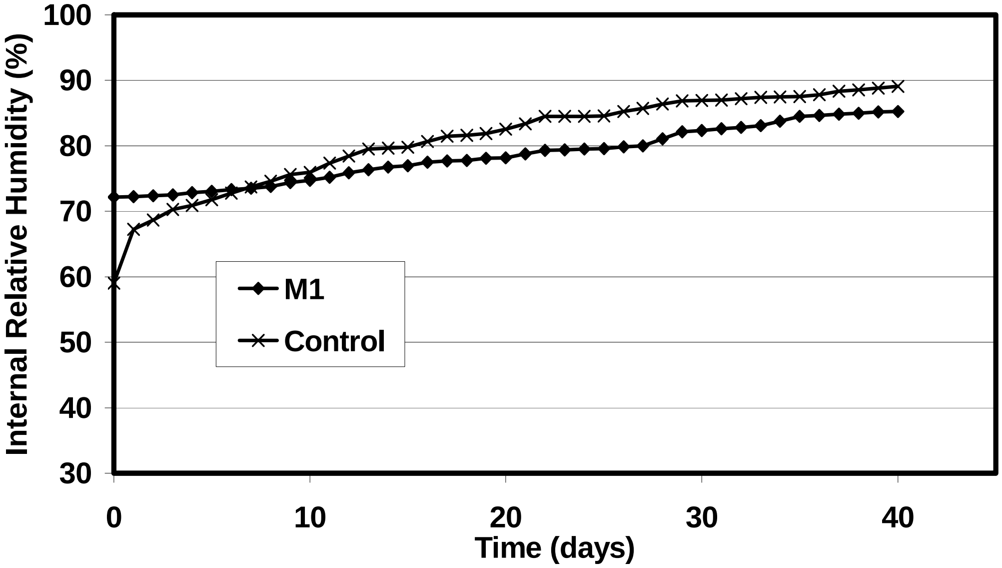

Figure 7.6 Average internal RH of M1 replicates and control replicates at 1.5-in. depth during wetting stage, Confirmatory Tests B

7.3.2 Average Results for Each Mitigation Treatment, Wetting Stage

Figure 7.7 and Figure 7.8 show the average internal relative humidity of each set of specimens (treated and control) during the wetting stage, at depths of 0.5 and 1.5 in. respectively. Results are consistent among sets of specimens, showing that the sealing methods developed in the Supplemental Tests were satisfactory.

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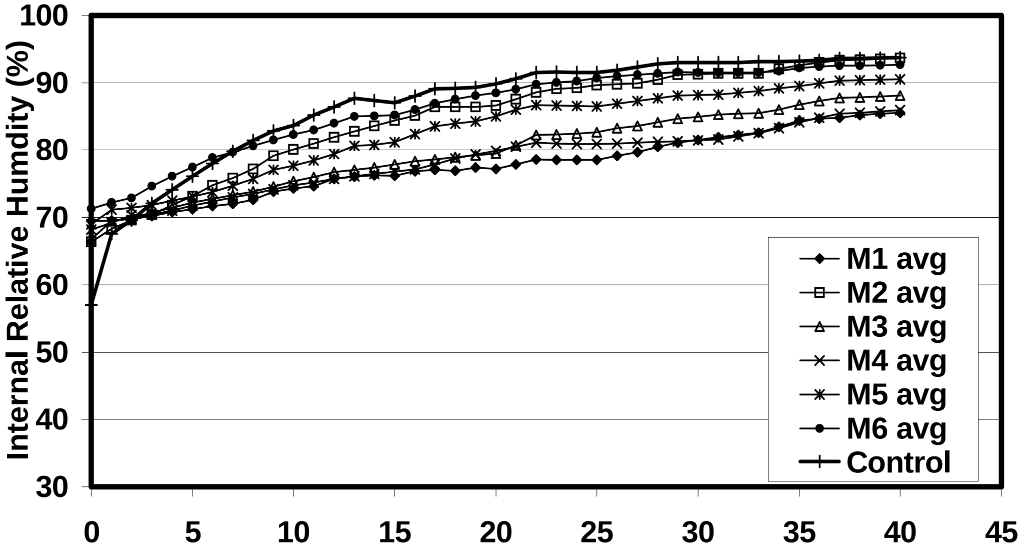

Figure 7.7 Average internal RH of each treated and untreated set of specimens at 0.5-in. depth during wetting stage, Confirmatory Tests B

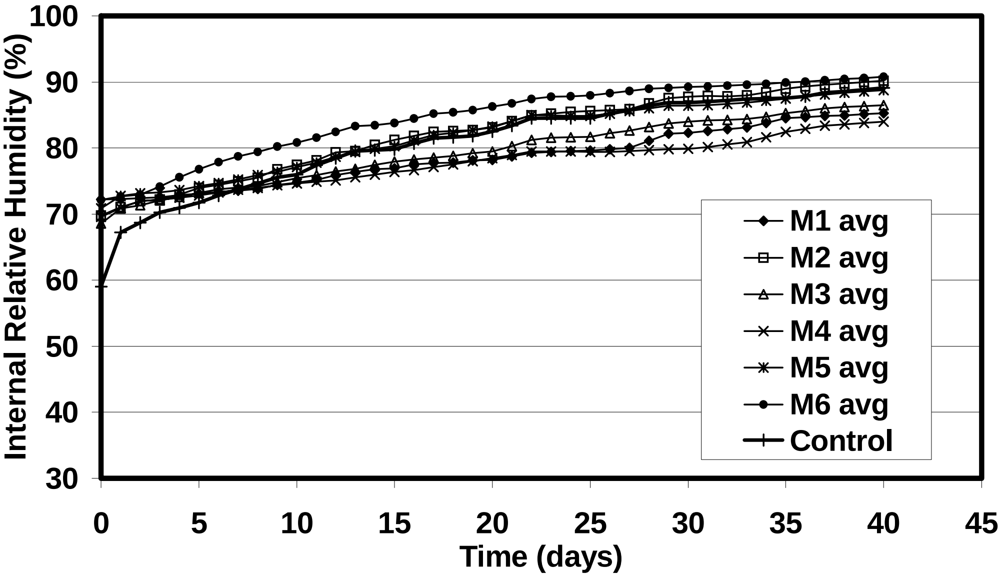

Figure 7.8 Average internal RH of each treated and untreated set of specimens at 1.5-in. depth during wetting stage, Confirmatory Tests B

7.4 POSSIBLE APPROACHES FOR COMPARISON OF RESULTS, CONFIRMATORY TESTS B

In Figure 7.9 and Figure 7.10 are shown, for depths of 0.5 and 1.5 in. respectively, the variation of average internal relative humidity, during a complete cycle of a drying stage followed by a wetting stage, for the M1 treated replicates and for the untreated (control) replicates. During the drying stage, the internal relative humidity of the M1 treated replicates decreases about the same as that of the untreated control specimens; during the wetting stage, it increases more slowly than that of the controls.

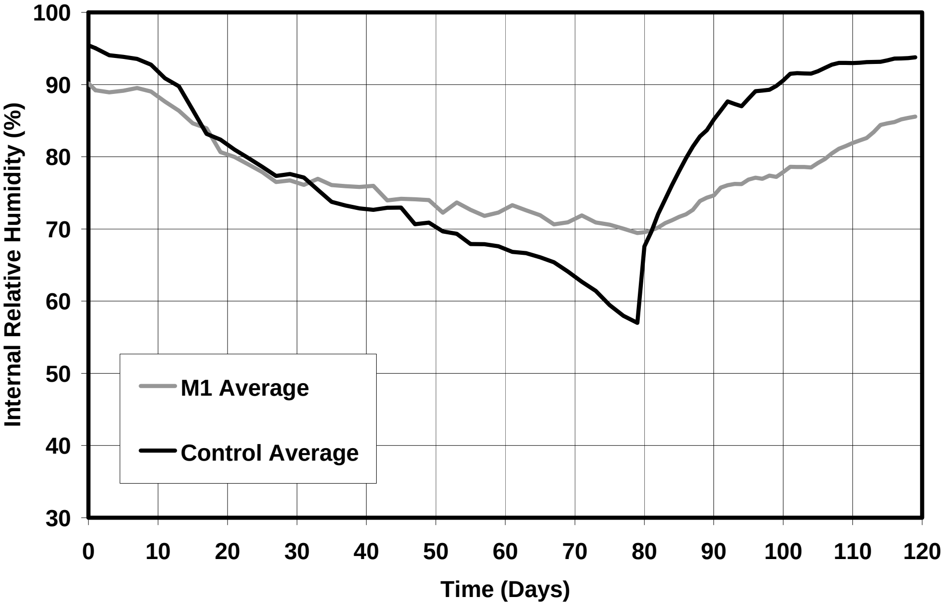

Figure 7.9 Average internal RH of M1 replicates and control replicates at 0.5-in. depth during entire cycle, Confirmatory Tests B

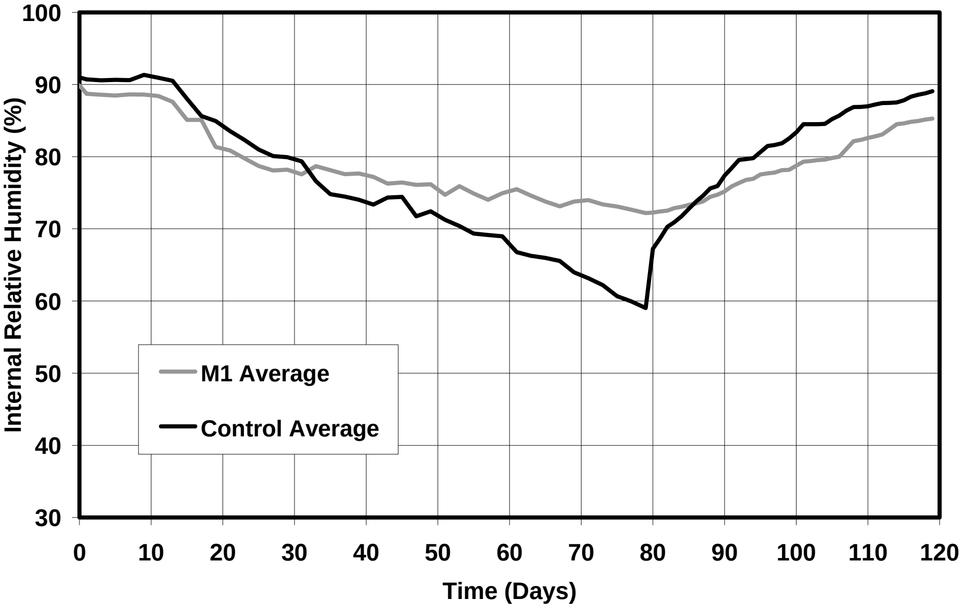

Figure 7.10 Average internal RH of M1 replicates and control replicates at 1.5-in. depth during entire cycle, Confirmatory Tests B

Corresponding information for Mitigation Treatments M2 through M6 is presented in Figure 7.11 through Figure 7.20.

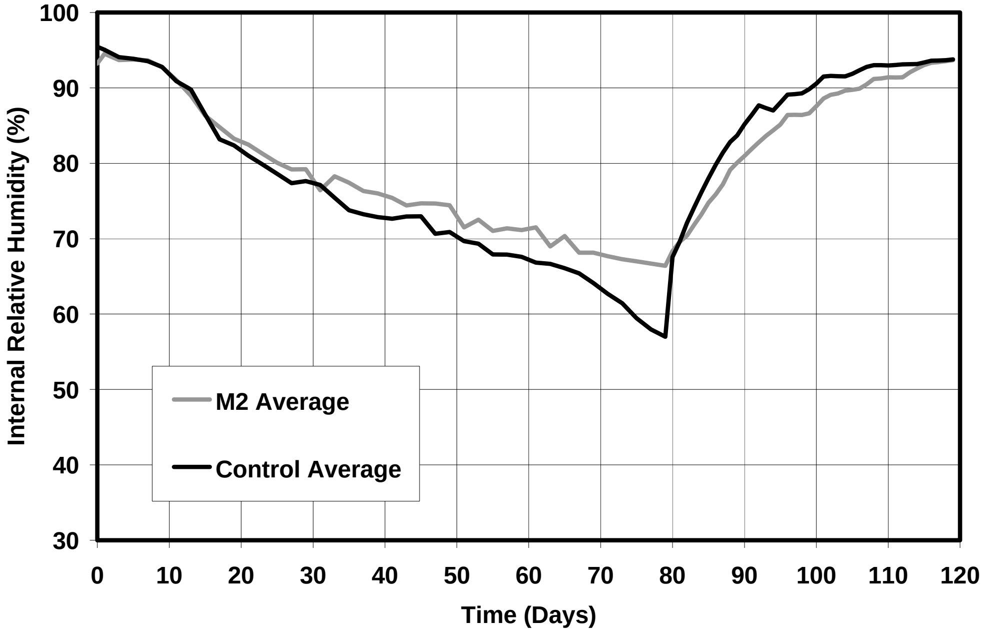

Figure 7.11 Average internal RH of M2 replicates and control replicates at 0.5-in. depth during entire cycle, Confirmatory Tests B

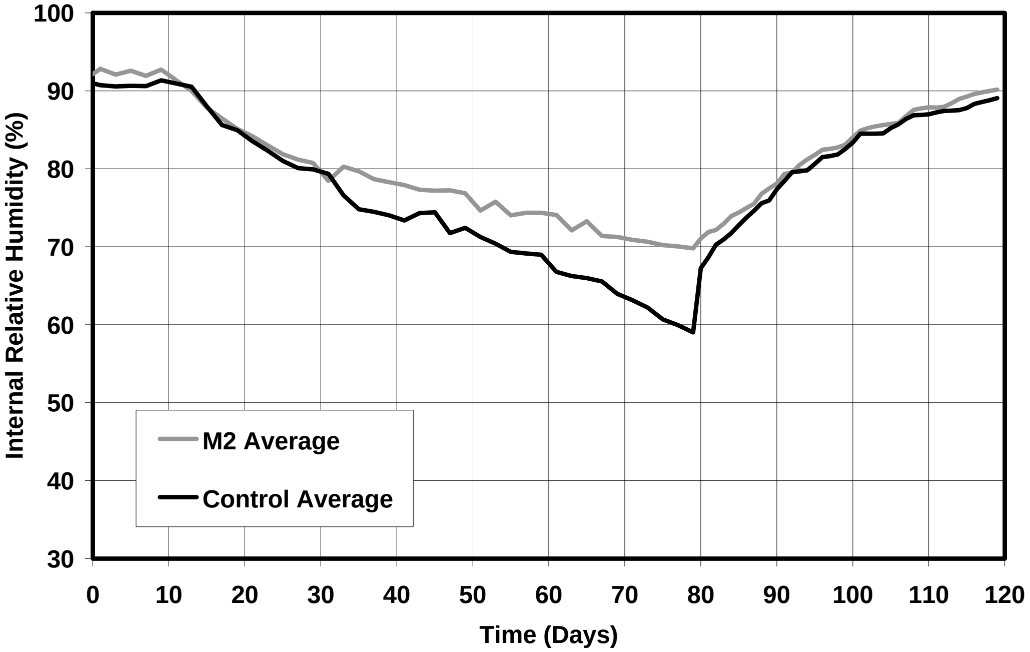

Figure 7.12 Average internal RH of M2 replicates and control replicates at 1.5-in. depth during entire cycle, Confirmatory Tests B

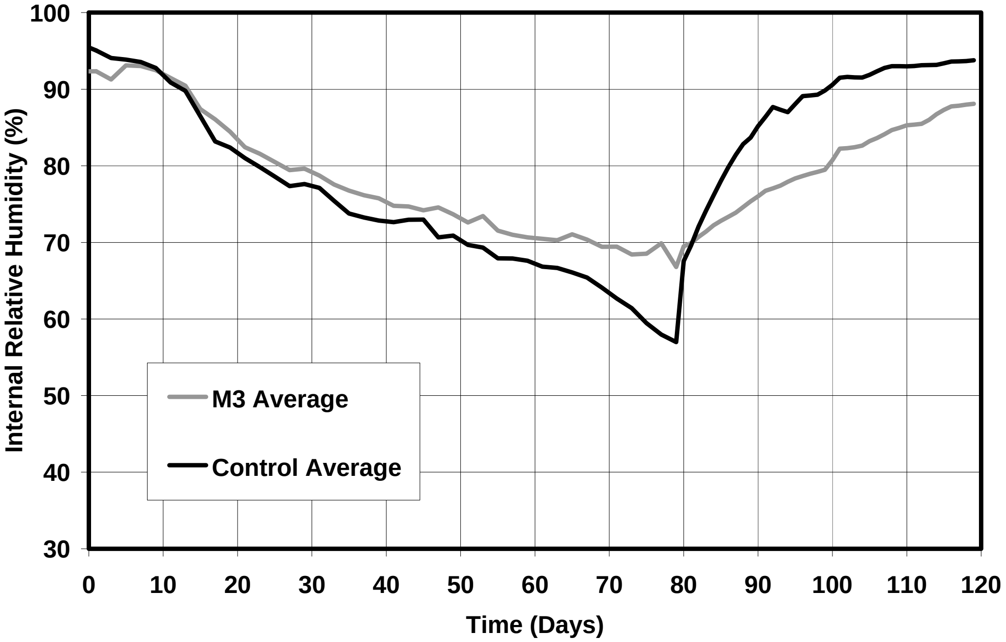

Figure 7.13 Average internal RH of M3 replicates and control replicates at 0.5-in. depth during entire cycle, Confirmatory Tests B

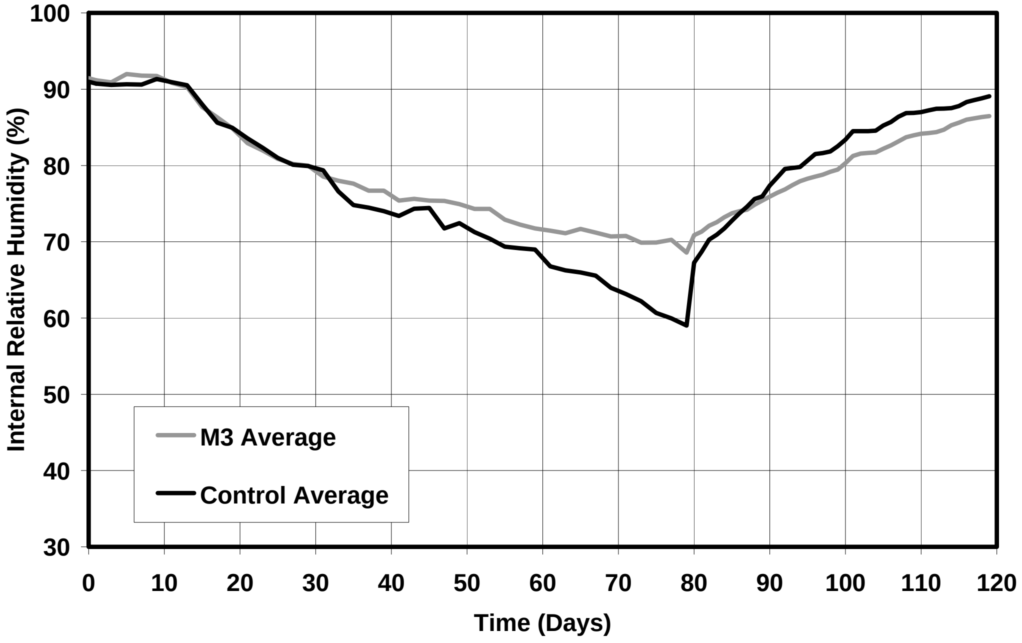

Figure 7.14 Average internal RH of M3 replicates and control replicates at 1.5-in. depth during entire cycle, Confirmatory Tests B

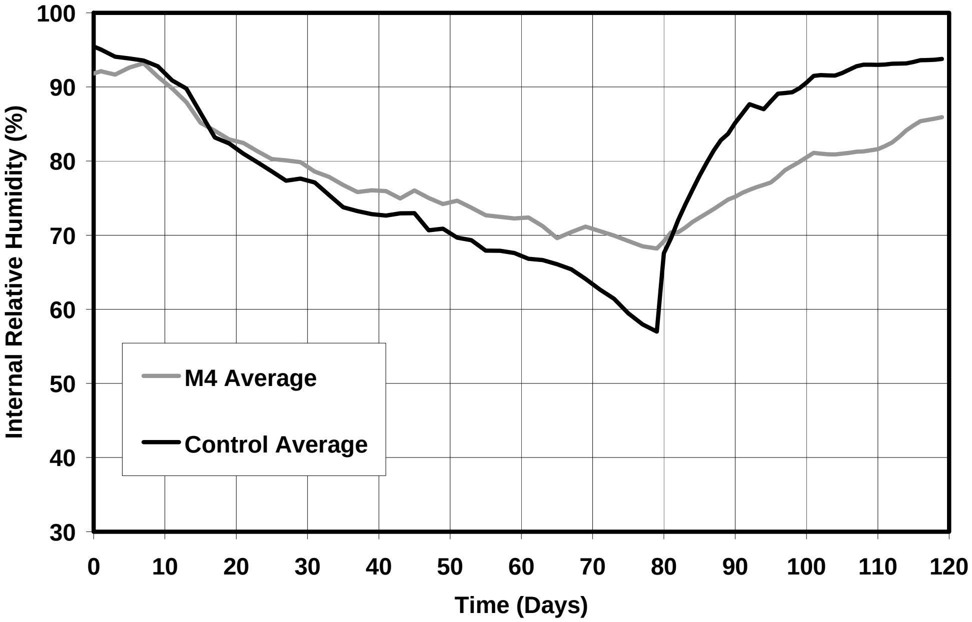

Figure 7.15 Average internal RH of M4 replicates and control replicates at 0.5-in. depth during entire cycle, Confirmatory Tests B

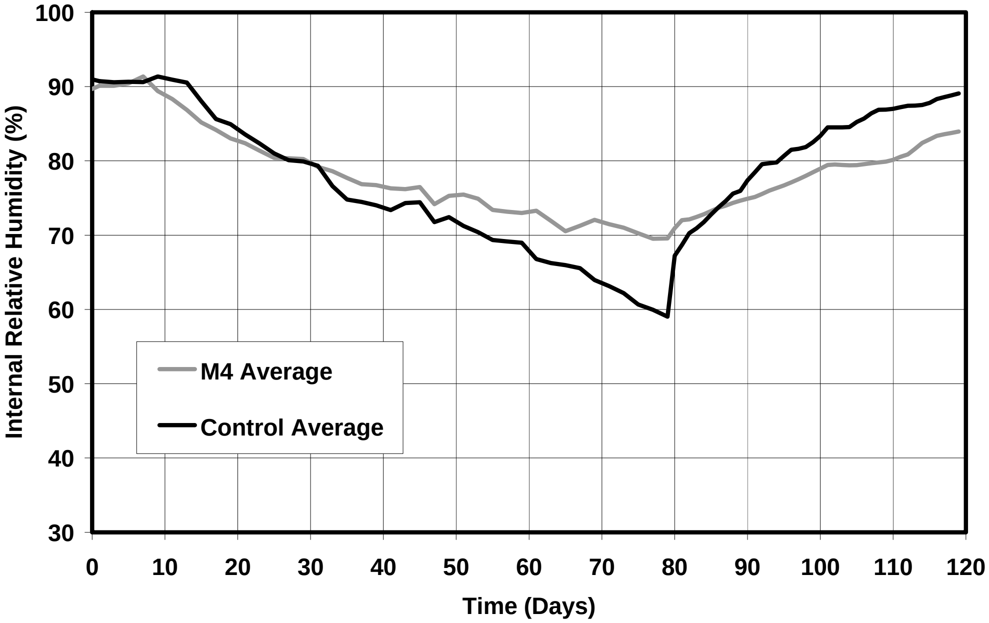

Figure 7.16 Average internal RH of M4 replicates and control replicates at 1.5-in. depth during entire cycle, Confirmatory Tests B

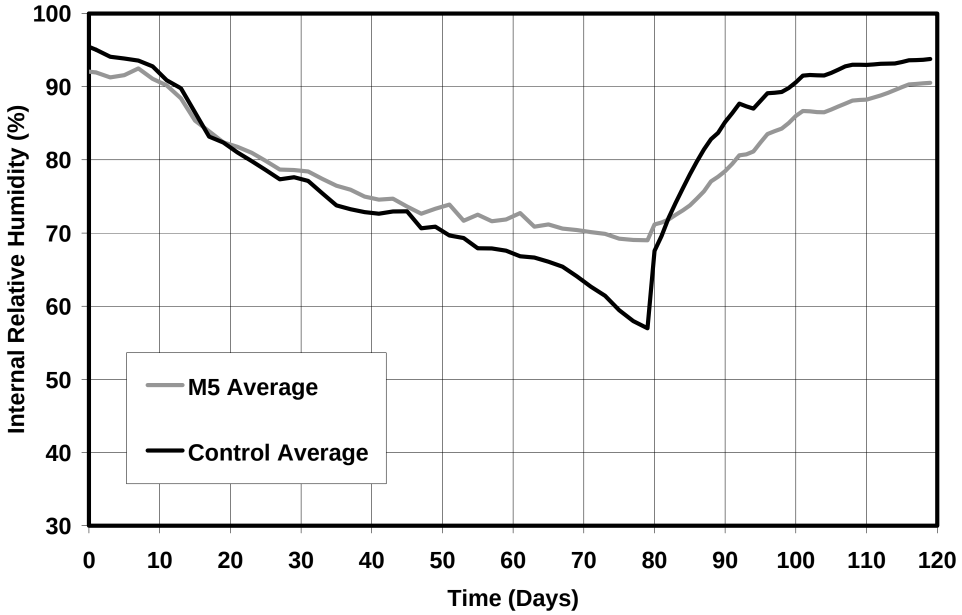

Figure 7.17 Average internal RH of M5 replicates and control replicates at 0.5-in. depth during entire cycle, Confirmatory Tests B

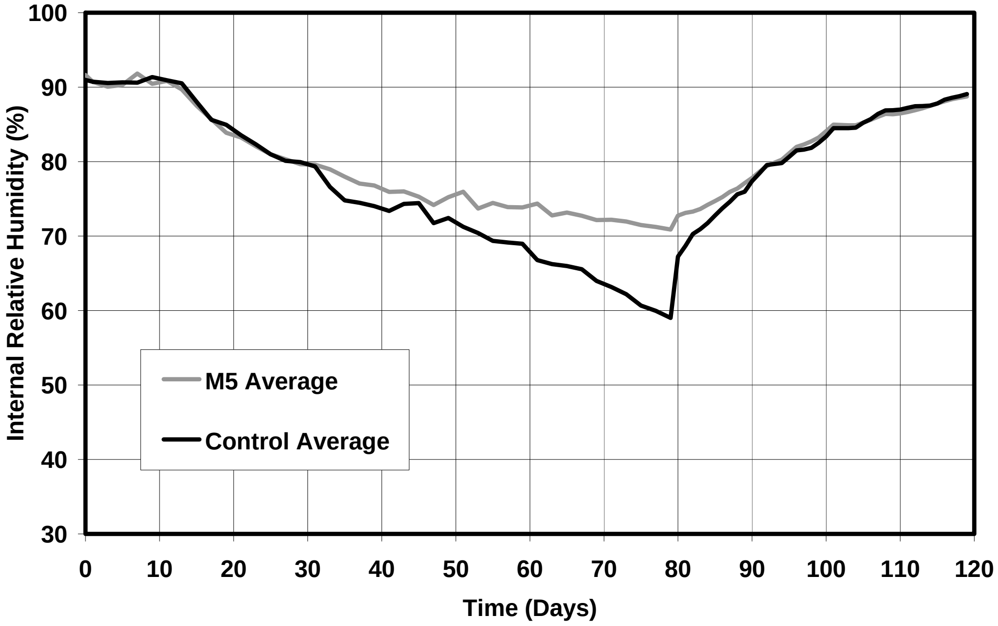

Figure 7.18 Average internal RH of M5 replicates and control replicates at 1.5-in. depth during entire cycle, Confirmatory Tests B

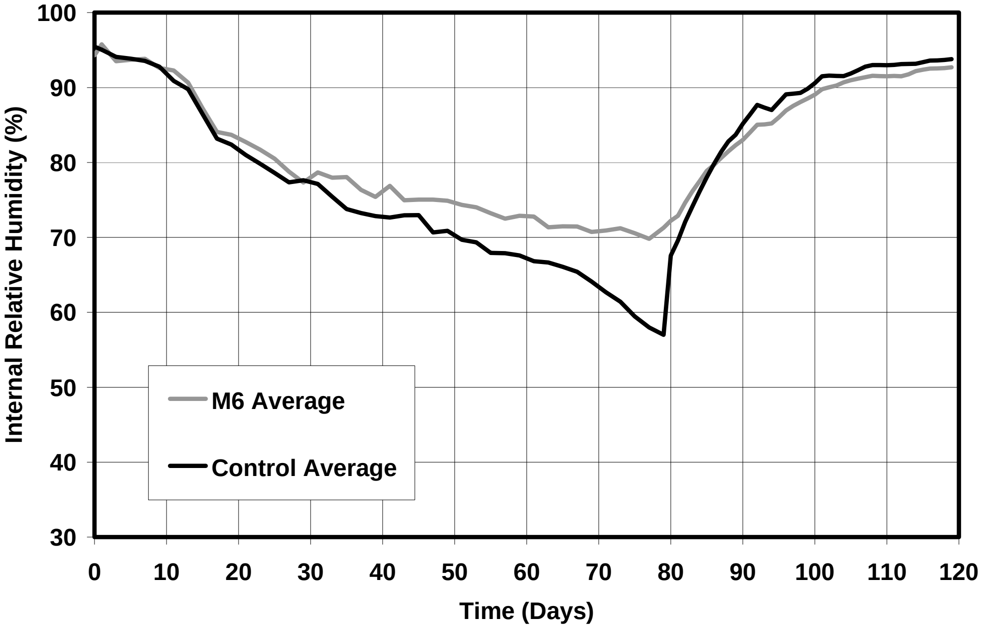

Figure 7.19 Average internal RH of M6 replicates and control replicates at 0.5-in. depth during entire cycle, Confirmatory Tests B

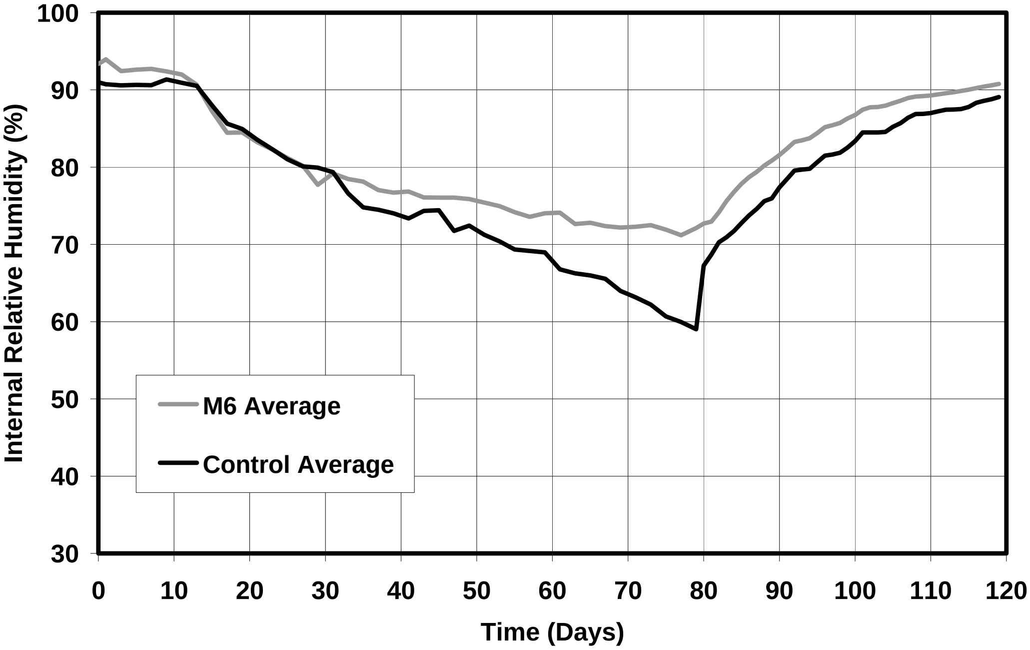

Figure 7.20 Average internal RH of M6 replicates and control replicates at 1.5-in. depth during entire cycle, Confirmatory Tests B

Results for Confirmatory Tests B, presented in Section 7.2 and 7.3, are examined further here, and their significance is discussed in Sections 7.5 through 7.7. Each mitigation treatment is evaluated by comparing the water impermeability and water-vapor permeability of treated specimens with that of control specimens. The water-vapor permeability of a specimen is evaluated during the drying stage of Confirmatory Test B, and the water impermeability is evaluated during the wetting stage of that same test.

As noted previously, ASR does not occur when a specimen's internal relative humidity is below 80%. To quantify the difference between the treated specimens and the control specimens, two evaluation methods are proposed.

- Evaluation Method 1: The effectiveness of a treatment is expressed in terms of the ratio of the average number of days that a treated set of specimens has an internal relative humidity above 80%, divided by the average number of days that the control set of specimens has an internal relative humidity above 80%. The lower the ratio, the more effective the treatment.
- Evaluation Method 2: The effectiveness of a treatment is expressed in terms of the ratio of the average difference in internal relative humidity of a treated set of specimens from the beginning to the end of a stage, divided by the average difference in internal relative humidity of the control specimens. The higher the ratio for the Drying Stage, the more effective the treatment. The lower the ratio for the Wetting Stage, the more effective the treatment.

Due to time constraints associated with the schedule of Study 4069, only the first evaluation method was used.

7.5 COMPARISON OF TREATMENTS BY AVERAGE EXPOSURE-TIME RATIOS

For each mitigation treatment, the ratio of the time above 80% RH for the treated specimens, divided by the time above 80% RH for the control specimens, was computed. In this report, this ratio is termed the "exposure-time ratio:"

$$Exposure-Time\ Ratio \equiv \frac{T_M}{T_C}$$

where T_M and T_C are as shown below, and are computed as discussed below, from subsidiary separate values for the drying and wetting stages.

7.5.1 Comparative Drying-Stage Exposure Times

Comparative average drying-stage exposure times for the specimens with Treatment M1 and for the control specimens are shown in Figure 7.21 and Figure 7.22 at depths of 0.5 in. and 1.5 in. respectively. The same times are given numerically in Table 7.1 and Table 7.2.

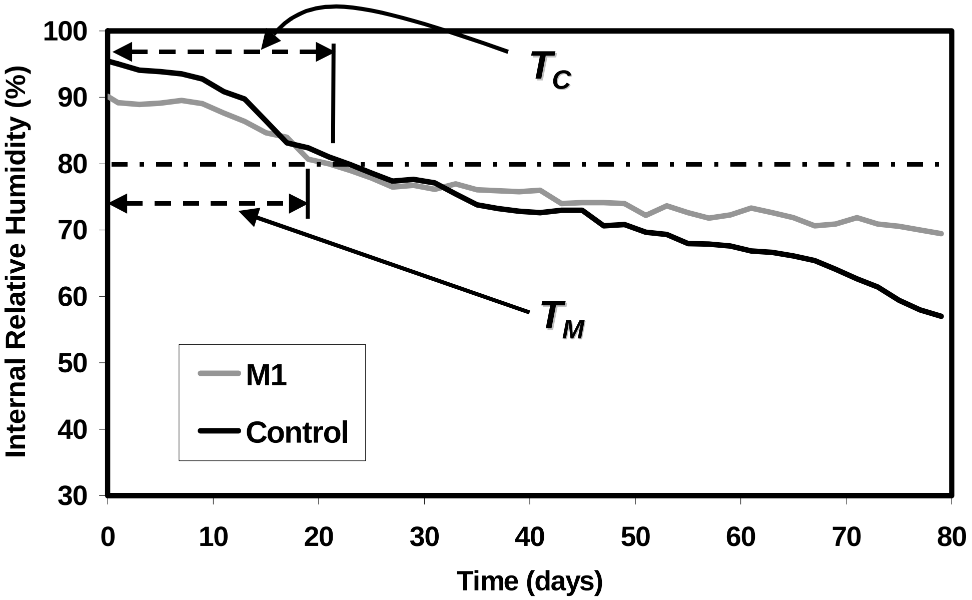

Figure 7.21 Average drying-stage exposure times for Treatment M1 at 0.5-in. depth, Confirmatory Tests B

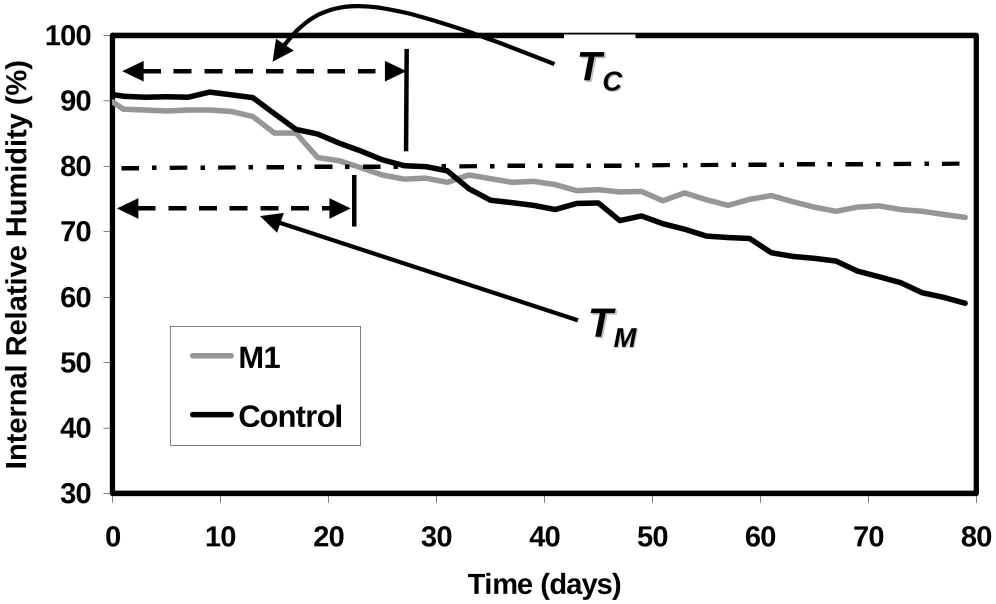

Figure 7.22 Average drying-stage exposure times for Treatment M1 at 1.5-in. depth, Confirmatory Tests B

Table 7.1 Average drying-stage exposure times for M1 treated and control specimens, 0.5-in. depth

Treatment	Average Drying-Stage Exposure Time, days
M1	20.75
M7 (control)	22

Table 7.2 Average drying-stage exposure times for M1 treated and control specimens, 1.5-in. depth

Treatment	Average Drying-Stage Exposure Time, days
M1	25
M7 (control)	28.5

In Table 6.3 and Table 6.4 are shown the average drying-stage exposure times for each set of treated and untreated specimens.

Table 7.3 Average drying-stage exposure times for each set of treated and untreated specimens, 0.5-in. depth

Treatment	Average Drying-Stage Exposure Time, days
M1	20.75
M2	24.75
M3	26.5
M4	28
M5	26
M6	26.25
M7 (Control)	22

Table 7.4 Average drying-stage exposure times for each set of treated and untreated specimens, 1.5-in. depth

Treatment	Average Drying-Stage Exposure Time, days
M1	25
M2	32
M3	29
M4	28.5
M5	28.5
M6	26.5
M7 (Control)	28.5

7.5.2 Comparative Wetting-Stage Exposure Times

Average wetting-stage exposure times for the specimens with Treatment M1 and for the control specimens are shown in Figure 6.23 and Figure 6.24 at depths of 0.5 in. and 1.5 in. respectively. The same times are given numerically in Table 6.5 and Table 6.6.

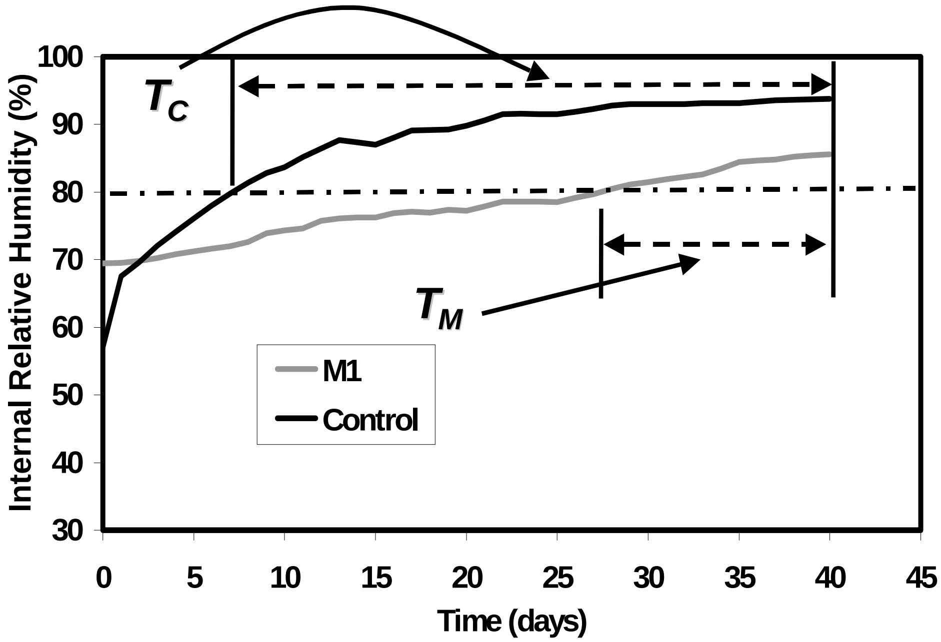

Figure 7.23 Average wetting-stage exposure times for Treatment M1 at 0.5-in. depth, Confirmatory Tests B

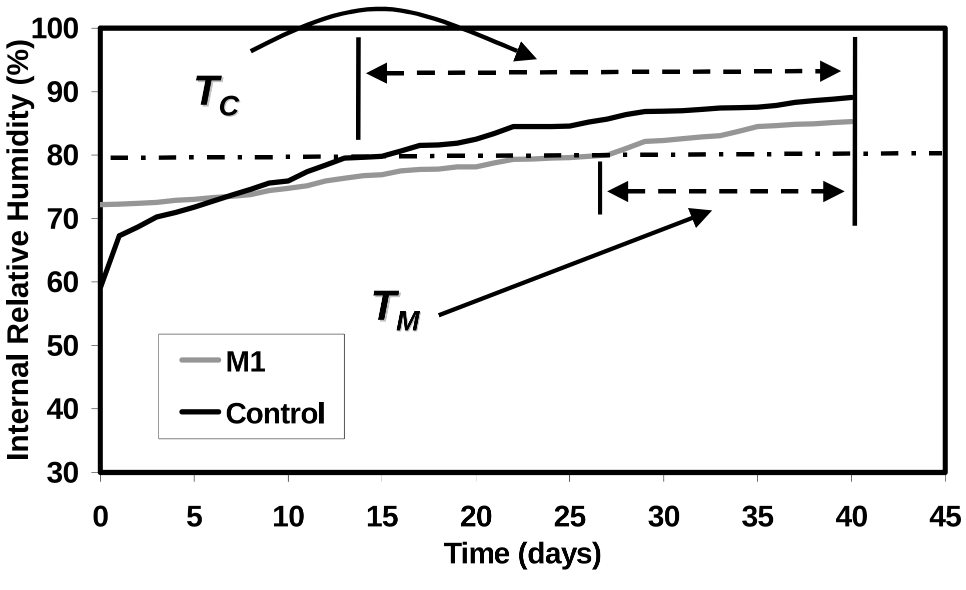

Figure 7.24 Average wetting-stage exposure times for Treatment M1 at 1.5-in. depth, Confirmatory Tests B

Table 7.5 Average wetting-stage exposure times for M1 treated and control specimens, 0.5-in. depth

Treatment	Average Wetting-Stage Exposure Time, days
M1	14.5
M7 (control)	32.75

Table 7.6 Average wetting-stage exposure times for M1 treated and control specimens, 1.5 in. depth

Treatment	Average Wetting-Stage Exposure Time, days
M1	15.5
M7 (control)	25.25

Corresponding results for all treatments are given in Table 6.7 and Table 6.8.

Table 7.7 Average wetting-stage exposure times for each set of treated and untreated specimens, 0.5-in. depth

Treatment	Average Wetting-Stage Exposure Time, days
M1	14.5
M2	30
M3	21
M4	16.5
M5	27.25
M6	33.25
M7 (Control)	32.75

Table 7.8 Average wetting-stage exposure times for each set of treated and untreated specimens, 1.5-in. depth

Treatment	Average Wetting-Stage Exposure Time, days
M1	15.5
M2	25.5
M3	20.25
M4	13
M5	26.25
M6	31
M7 (Control)	25.25

7.5.3 Average Total Exposure Times for Each Set of Specimens

Average total exposure times for the specimens with Treatment M1 and for the control specimens are shown in Figure 7.25 and Figure 7.26 at depths of 0.5 in. and 1.5 in. respectively. Corresponding results are given in tabular form in Table 7.9 and Table 7.10. In each case, the average total exposure time is obtained by summing the average drying-stage and the average wetting-stage times presented in the preceding sections.

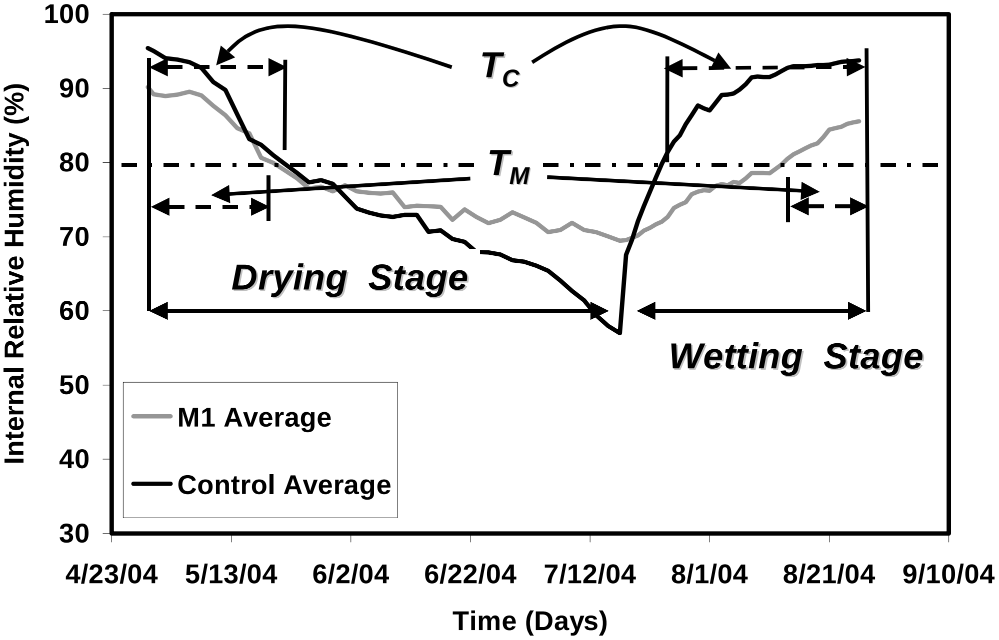

Figure 7.25 Average total exposure times for Treatment M1 at 0.5-in. depth, Confirmatory Tests B

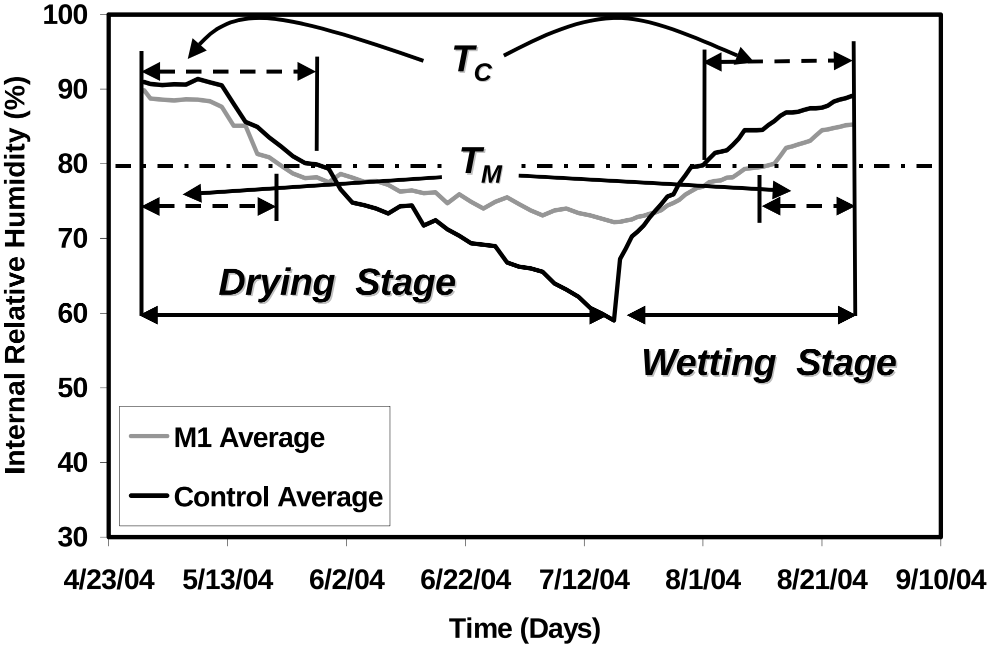

Figure 7.26 Average total exposure times for Treatment M1 at 1.5-in. depth, Confirmatory Tests B

Table 7.9 Average total exposure times for M1 treated and control specimens, 0.5-in. depth

Treatment	Average Total Exposure Time, days
M1	35.25
M7 (control)	54.75

Table 7.10 Average total exposure times for M1 treated and control specimens, 1.5-in. depth

Treatment	Average Total Exposure Time, days
M1	40.5
M7 (control)	53.75

Table 7.11 and Table 7.12 show the average total exposure times (drying-stage plus wetting-stage) for each set of treated and untreated specimens.

Table 7.11 Average total exposure times for each set of treated and untreated specimens, 0.5-in. depth

Treatment	Average Total Exposure Time, days
M1	35.25
M2	54.75
M3	47.5
M4	44.5
M5	53.25
M6	59.5
M7 (Control)	54.75

Table 7.12 Average total exposure times for each set of treated and untreated specimens, 1.5-in. depth

Treatment	Average Total Exposure Time, days
M1	40.5
M2	57.5
M3	49.25
M4	41.5
M5	54.75
M6	57.5
M7 (Control)	53.75

7.5.4 Comparative Average Exposure-Time Ratios

Exposure-time ratios, defined at the start of Section 6.5, are calculated as shown in Tables 6.13 and 6.14 for depths of 0.5 and 1.5 in. respectively.

Table 7.13 Calculation of Exposure-Time Ratios, 0.5-in. depth

Treatment	Drying-stage Exposure Time, days	Wetting-stage Exposure Time, days	Total Exposure Time, days	Exposure-Time Ratio
M1	20.75	14.5	35.25	0.64
M2	24.75	30	54.75	1.00
M3	26.5	21	47.5	0.87
M4	28	16.5	44.5	0.81
M5	26	27.25	53.25	0.97
M6	26.25	33.25	59.5	1.09
M7 (control)	22	32.75	54.75	1.00

Table 7.14 Calculation of Exposure-Time Ratios, 1.5-in. depth

Treatment	Drying-stage Exposure Time, days	Wetting-stage Exposure Time, days	Total Exposure Time, days	Exposure-Time Ratio
M1	25	15.5	40.5	0.75
M2	32	25.5	57.5	1.07
M3	29	20.25	49.25	0.92
M4	28.5	13	41.5	0.77
M5	28.5	26.25	54.75	1.02
M6	26.5	31	57.5	1.07
M7 (control)	28.5	25.25	53.75	1.00

At depths of 0.5 in. and 1.5 in. respectively, the exposure-time ratios alone are shown in Table 7.15 and Table 7.16.

Table 7.15 Exposure-Time Ratios, 0.5-in. depth

Treatment	Exposure-Time Ratio
M1	0.64
M2	1.00
M3	0.87
M4	0.81
M5	0.97
M6	1.09
M7 (control)	1.00

Table 7.16 Exposure-Time Ratios, 1.5-in. depth

Treatment	Exposure-Time Ratio
M1	0.75
M2	1.07
M3	0.92
M4	0.77
M5	1.02
M6	1.07
M7 (control)	1.00

Those tables lead to the initial observation that some mitigation treatments are apparently much more effective than others. In the following subsection, this observation is tested further.

7.5.5 t-Test Comparisons of Exposure-Time Ratios

To examine whether the apparent differences among average exposure-time ratios for different treatments are statistically significant, they can be evaluated using t-Tests. In this technique, differences among average values are compared with the coefficients of variation of the subsets from which those averages are calculated. For each treatment, the corresponding t-Test value is the probability, to a confidence level of 90%, that the exposure time of the treated specimens is statistically indistinguishable from that of the controls. For example, if a particular treatment has a t-Test value of 1% compared to the controls, the treated specimens have only a 1% probability of being statistically indistinguishable from the controls. Expressed in positive rather than negative terms, they have a 99% probability of being statistically distinguishable from the controls. The smaller the t-Test value, the more likely it is that the corresponding treatment works.

In Table 6.17 and Table 6.18, t-Test values for each set of treated and untreated specimens are given, for 0.5- and 1.5-in. depths respectively.

Table 7.17 t-Test values, 0.5-in. depth

Treatment	t-Test values (%)
M1	1.9
M2	100
M3	2.1
M4	8.8
M5	72.6
M6	7.8
M7 (Control)	100

Table 7.18 t-Test values, 1.5- in. depth

Treatment	t-Test values (%)
M1	8.1
M2	52.9
M3	10.6
M4	2.6
M5	77.2
M6	21.5
M7 (Control)	100

7.6 SIGNIFICANCE OF COMPARATIVE RESULTS FOR CONFIRMATORY TESTS B

In Sections 7.2 and 7.3, the consistency of results from Confirmatory Tests B, and their consistency with results obtained earlier in the Controlled Pilot Tests, shows that the sealing methods developed in the Supplemental Tests are effective. In Sections 7.5.4 and 7.5.5, Mitigation Treatments with significantly lower exposure-time ratios, and significantly low t-Test scores, are identified:

The t-Test values of Table 7.17 and Table 7.18 reiterate quantitatively what can be surmised from Table 7.15 and Table 7.16. Some treatments are indeed effective, and some treatments are indeed more effective than others. At both depths, Treatments M1 and M4 are effective, with exposure-time ratios of about 0.75 and t-Test scores of less than 10%.

7.7 LIFE-EXTENSION RATIOS AS A CONSEQUENCE OF EXPOSURE-TIME RATIOS

The exposure-time ratios introduced above, inverted, can be interpreted as “life-extension ratios.” If damage from premature concrete deterioration is assumed to increase linearly with exposure time, and specimens with a particular treatment have an average exposure-time ratio of 0.75 compared to control specimens, then the treated specimens would require (1/0.75) times as long, or 1.33 times as long, as the control specimens to experience the same amount of damage. In simple terms, their life would be extended by a factor of 1.33.

Using this concept, life-extension ratios are presented in Table 7.19 and Table 7.20 for each treatment. The numbers are in each case the reciprocal of the exposure-time ratios presented in Table 7.15 and Table 7.16 respectively.

Table 7.19 Life-Extension Ratios, 0.5-in. depth

Treatment	Life-Extension Ratios
M1	1.56
M2	1.00
M3	1.15
M4	1.23
M5	1.03
M6	0.92
M7 (Control)	1.00

Table 7.20 Life-Extension Ratios, 1.5 in. depth

Treatment	Life-Extension Ratios
M1	1.33
M2	0.93
M3	1.09
M4	1.30
M5	0.98
M6	0.93
M7 (Control)	1.00

At a depth of 0.5 in., the treatments identified as effective (M1 and M4) have life-extension ratios of about 1.3.

Finally, the exposure-time ratios (and corresponding life-extension ratios) calculated on the basis of the first complete cycle of drying and wetting, are probably conservative. The internal relative humidity of effectively treated specimens is likely to continue to decrease over time compared to that of control specimens, resulting in smaller exposure-time ratios, and larger life-extension ratios, for the second complete cycle than the first, and so on.

CHAPTER 8:
SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

8.1 SUMMARY

This report describes the work associated with TxDOT Study 4069 ("Mitigation Techniques for In-Service Structures with Premature Concrete Deterioration"). TxDOT is interested in developing techniques for mitigating or remediating premature concrete deterioration due to alkali silica reaction (ASR), delayed ettringite formation (DEF), or both, in order to extend the life of potentially affected structures. Study 4069, synthesized here, consists of: a literature search for mitigation or remediation techniques; the study and evaluation of existing test methods for evaluating the effectiveness of those methods; and the development and verification of a new test method for that purpose.

8.1.1 Literature Search

The literature search was conducted to identify treatments being used or tested worldwide to mitigate or remediate deterioration from ASR, DEF, or both. The proposed treatments were evaluated, according to published results, for use on Texas Department of Transportation structures displaying this damage. In addition, treatments used to mitigate other types of concrete deterioration were explored for their potential use to mitigate or remediate ASR/DEF deterioration.

The search identified treatments that were effective at reducing expansion of ASR-affected specimens or preventing other types of concrete deterioration. These treatments are: silane; siloxane; linseed oil; high-molecular-weight methacrylate (HMWM), epoxy, polymer-modified cement mortar (PCM); urethane; and lithium nitrate.

It was recommended to use these treatments, separately or in combination, on specimens made with aggregates and cement known to produce ASR/DEF deterioration, and to evaluate their effectiveness by measuring specimens' expansion and internal moisture content, and by monitoring them with acoustic emission techniques. Applications of these techniques to specific structures, such as FM 1929 at Lake Ivie, are discussed in theses and reports for Study 1857. It is not discussed further here.

8.1.2 Development of Test Method

The work reported here emphasized indirect test methods, involving comparison of the effects of different possible mitigation treatments, on the internal relative humidity of concrete specimens subjected to cycles of wetting and drying. After an unsuccessful search of current AASHTO, ASTM, and DIN test methods, a new test method was developed that satisfied the needs of this study. The new test method involved subjecting concrete specimens to cycles of wetting and drying under controlled temperature and relative humidity. It was reasoned that effective mitigation treatments would keep the internal relative humidity of the specimens below that of control specimens under such cycling, by permitting the escape of water vapor during drying stages, while stopping the penetration of liquid water during wetting stages. Because one form of premature concrete deterioration (ASR) has been shown to be inhibited when internal relative humidity drops below 80%, the effectiveness of mitigation treatments was compared quantitatively by using the time that treated specimens spent below that level of humidity during a complete cycle of drying and wetting, compared to untreated control specimens. Effectiveness of treatments was evaluated in terms of an "exposure-time ratio" (the ratio of the time that the internal relative humidity of a treated specimen remained above 80% during a complete exposure cycle, compared with that of an untreated specimen). Comparative life extension was evaluated in terms of a "life-extension ratio" (the reciprocal of the exposure-time ratio). Mean exposure-time ratios were compared for 6 different mitigation treatments, and the significance of the comparisons was evaluated using t-tests.

8.2 CONCLUSIONS

1) The following treatments are effective in mitigating premature concrete deterioration due to delayed ettringite formation or alkali-silica reaction, separately or in combination:
    - (M1) TxDOT Surface Treatment –Silane, plus TxDOT Appearance Coat Paint;
    - (M4) TxDOT Surface Treatment –Silane, plus Opaque Concrete Sealer.
2) For the first cycle of drying and wetting, those two treatments have exposure-time ratios of 0.64 and 0.81 respectively at a depth of 0.5 in., and 0.75 and 0.77 at a depth of 1.5 in. These ratios will probably decrease with further cycling.
3) Based on those exposure-time ratios, the two treatments have life-extension ratios of 1.56 and 1.23 respectively at a depth of 0.5 in., and 1.33 and 1.30 at a depth of 1.5 in.. Those ratios will probably increase with further cycling -- that is, actual life extensions are expected to be even greater over multiple cycles of exposure.

8.3 RECOMMENDATIONS FOR IMPLEMENTATION

- Current structures with Premature Concrete Deterioration should be treated with Mitigation Treatment M1 (Silane, plus TxDOT Appearance Coat Paint). This is TxDOT's current treatment for ASR/DEF. Mitigation Treatment M4 (Silane, plus Opaque Concrete Sealer) while equally effective in mitigating deterioration, may not be as effective in making caulked cracks less visible to users.
- Structures treated with Mitigation Treatment M1 should continue to be monitored to confirm the effectiveness of that treatment under field conditions.
- The new test method developed in this report should be formally submitted by TxDOT materials specialists to the appropriate ASTM committee for possible standardization by that organization.

8.4 RECOMMENDATIONS FOR FURTHER RESEARCH

Using the proposed test method, TxDOT materials specialists should conduct further cycling to compare the effectiveness of mitigation treatments during more than one complete exposure cycle.

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